Hydraulic Engineering Circular Number 18

Evaluating Scour at Bridges

May 2001 - 4th edition
Publication No. FHWA-NHI-01-001

Errata Sheet

October 6, 2001 – version 1.0

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6	6.6	Last sentence in bold should read: **The minimum value of K_4 is 0.4**. Delete: "and it should only be used when $V_1 < V_{ic\ D50}$."
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12	E.3	Appendix E, title should be: **Sturm Abutment Scour Equations**
13	E.4 - E.7	Throughout the Appendix: The subscript "o" should be "0" (zero), i.e. "Y_{fo}" should be "Y_{f0}" The subscripts "l" and "i" should be "1" (one) i.e., "V_{ml}" should be "V_{m1}" and "V_{mic}" should be "V_{m1c}"
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15	E.6	Item 6c: First line should be: "Compute the critical velocity..."
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Hydraulic Engineering Circular No. 18

Evaluating Scour At Bridges

Fourth Edition

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16. This document is the fourth edition of HEC-18. It presents the state of knowledge and practice for the design, evaluation and inspection of bridges for scour. There are two companion documents, HEC-20 entitled "Stream Stability at Highway Structures," and HEC-23 entitled "Bridge Scour and Stream Instability Countermeasures." These three documents contain updated material from previous editions and the publication, "Interim Procedures for Evaluating Scour at Bridges," issued in September 1988 as part of the FHWA Technical Advisory T 5140.20, "Scour at Bridges." T5140.20 has since been superseded by T 5140.23, "Evaluating Scour at Bridges" dated October 28, 1991. This fourth edition of HEC-18 contains revisions obtained from further scour-related developments and the use of the 1995 edition by the highway community.

The major changes in this fourth edition of HEC-18 are: change in nomenclature to using General Scour to include both contraction scour and other general scour components, changing the order of Chapters 2 and 3 so that the policy chapter entitled "Designing Bridges to Resist Scour" comes before the chapter entitled "Basic Concepts and Definitions of Scour;" and separating Chapter 4 into separate chapters dealing with each of the major scour components. In addition, a new K_4, to account for coarse bed material in the pier scour equation, improved methods to compute scour for complex pier configurations, example problems, and additional information on computer programs for modeling tidal hydraulics are given. There is no change in the recommendations regarding abutment scour.

In addition to minor editorial revisions, the following substantive changes have been made in this revised edition of HEC-18: revised definition of grain roughness k_s, (p. xii); changed guidance on determining the magnitude of the 500-year flood (p. 2.9). Note 3 definition of K_3 factor corrected (p. 6.5); revised guidance on minimum value of K_4 factor (p. 6.6); revised definition of grain roughness k_s (p. 6.13); and corrected multiplier for kinematic viscosity in Appendix A (Table A.7).

17. Key Words

scour design, contraction scour, local scour, pier scour, abutment scour, scour susceptible, scour critical, clear-water scour, live-bed scour, superflood, bridge inspection, countermeasures, tidal scour

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LIST OF SYMBOLS

a	=	Pier width, m (ft)
A	=	Maximum amplitude of elevation of the tide or storm surge, m (ft)
A_e	=	Flow area of the approach cross section obstructed by the embankment, m² (ft^2)
A_c	=	Cross-sectional area of the waterway at mean tide elevation--half between high and low tide, m² (ft^2)
	=	Net cross-sectional area in the inlet at the crossing, at mean water surface elevation, m^2 (ft^2)
C_d	=	Coefficient of discharge
D	=	Diameter of the bed material, m (ft)
	=	Diameter of smallest nontransportable particle in the bed material, m (ft)
D_m	=	Effective mean diameter of the bed material in the bridge, mm or m
	=	1.25 D_{50}
D_{50}	=	Median diameter of the bed material, diameter which 50% of the sizes are smaller, mm or m
D_{84}	=	Diameter of the bed material of which 84% are smaller, mm or m
D_{90}	=	Diameter of the bed material of which 90% are smaller, mm or m
Fr	=	Froude Number [V/(gy)½]
	=	Froude Number of approach flow upstream of the abutment
	=	Froude Number based on the velocity and depth adjacent to and upstream of the abutment
Fr_1	=	Froude Number directly upstream of a pier
g	=	Acceleration of gravity, m/s^2 (ft/s²)
h_{1-2}	=	Head loss between sections 1 and 2, m (ft)
h_c	=	Average depth of flow in the waterway at mean water elevation, m (ft)
H	=	Height (i.e., height of a dune), m (ft)
H_b	=	Distance from the low chord of the bridge to the average elevation of the stream bed before scour, m (ft)
K	=	Various coefficients in equations as described below
	=	Conveyance in Manning's equation $\frac{(AR^{2/3})}{n}$, m^3/s (ft^3/s)
	=	Bottom width of the scour hole as a fraction of scour depth, m (ft)
K_o	=	Velocity head loss coefficient on the ocean side or downstream side of the waterway
K_b	=	Velocity head loss coefficient on the bay or upstream side of the waterway
K_s	=	Shields coefficient
K_1	=	Correction factor for pier nose shape
	=	Coefficient for abutment shape

K_2	=	Correction factor for angle of attack of flow
	=	Coefficient for angle of embankment to flow
K_3	=	Correction factor for increase in equilibrium pier scour depth for bed condition
K_4	=	Correction factor for armoring in pier scour equation
k_1 & k_2	=	Exponents determined in Laursen live-bed contraction equation, depends on the mode of bed material transport
k_s	=	Grain roughness of the bed, m (ft)
L	=	Length of pier, m (ft)
L_c	=	Length of the waterway, m (ft)
L′ or L	=	Length of abutment (embankment) projected normal to flow, m (ft)
n	=	Manning's n
n_1	=	Manning's n for upstream main channel
n_2	=	Manning's n for contracted section
Q	=	Discharge through the bridge or on the overbank at the bridge, m^3/s (ft^3/s)
Q_e	=	Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
Q_{max}	=	Maximum discharge in the tidal cycle, m^3/s (ft^3/s)
	=	Maximum discharge in the inlet, m^3/s (ft^3/s)
Q_t	=	Discharge at any time, t, in the tidal cycle, m^3/s (ft^3/s)
Q_1	=	Flow in the upstream main channel transporting sediment, m^3/s (ft^3/s)
Q_2	=	Flow in the contracted channel, m^3/s (ft^3/s). Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges or water overtopping the approach roadway
Q_{100}	=	Storm-event having a probability of occurrence of one every 100 years, m^3/s (ft^3/s)
Q_{500}	=	Storm-event having a probability of occurrence of one every 500 years, m^3/s (ft^3/s)
q	=	Discharge per unit width, $m^3/s/m$ ($ft^3/s/ft$)
	=	Discharge in conveyance tube, m^3/s (ft^3/s)
R	=	Hydraulic radius
	=	Coefficient of resistance
SBR	=	Set-back ratio of each abutment
S_1	=	Slope of energy grade line of main channel, m/m (ft/ft)
S_f	=	Slope of the energy grade line, m/m (ft/ft)
S_o	=	Average bed slope, m/m (ft/ft)
S_s	=	Specific gravity of bed material. For most bed material this is equal to 2.65
t	=	Time from the beginning of total cycle, min
T	=	Total time for one complete tidal cycle, min
	=	Tidal period between successive high or low tides, s

V	=	Average velocity, m/s (ft/s)
	=	Characteristic average velocity in the contracted section for estimating a median stone diameter, D_{50}, m/s (ft/s)
V_{max}	=	Q_{max}/A', or maximum velocity in the inlet, m/s (ft/s)
V_1	=	Average velocity at upstream main channel, m/s (ft/s)
	=	Mean velocity of flow directly upstream of the pier, m/s (ft/s)
V_2	=	Average velocity in the contracted section, m/s (ft/s)
V_c	=	Critical velocity, m/s (ft/s), above which the bed material of size D, D_{50}, etc. and smaller will be transported
V_{c50}	=	Critical velocity for D_{50} bed material size, m/s (ft/s)
V_{c90}	=	Critical velocity for D_{90} bed material size, m/s (ft/s)
V_e	=	Q_e/A_e, m/s (ft/s)
V_f	=	Average velocity of flow zone below the top of the footing, m/s (ft/s)
V_i	=	Approach velocity when particles at a pier begin to move, m/s (ft/s)
V_{max}	=	Maximum average velocity in the cross section at Q_{max}, m/s (ft/s)
V_R	=	Velocity ratio
V_*	=	Shear velocity in the upstream section, m/s (ft/s)
	=	$(\tau_o/\rho) = (gy_1S_1)^{½}$
VOL	=	Volume of water in the tidal prism between high and low tide levels, m^3 (ft^3)
W	=	Bottom width of the bridge less pier widths, or overbank width (set back distance less pier widths, m (ft)
	=	Topwidth of the scour hole from each side of the pier of footing, m (ft)
W_1	=	Bottom width of the upstream main channel, m (ft)
W_2	=	Bottom width of the main channel in the contracted section less pier widths, m (ft)
ω	=	Fall velocity of the bed material of a given size, m/s (ft/s)
y	=	Depth of flow, m (ft). This depth is used in the Neill's and Larson's equation as the upstream channel depth to determine V_c.
	=	Depth of flow in the contracted bridge opening for estimating a median stone diameter, D_{50}, m (ft)
	=	Amplitude or elevation of the tide above mean water level, m (ft), at time t
y_a	=	Average depth of flow on the floodplain, m (ft)
y_f	=	Distance from the bed to the top of the footing, m (ft)
y_o	=	Existing depth of flow, m (ft)
y_{ps}	=	Depth of pier scour, m (ft)
y_s	=	Average contraction scour depth, m (ft)
y_s	=	Local scour depth, m (ft)
y_s	=	Depth of vertical contraction scour relative to mean bed elevation, m (ft)
y_{sc}	=	Depth of contraction scour, m (ft)
y_1	=	Average depth in the upstream main channel or on the floodplain prior to contraction scour, m (ft)

	=	Depth of flow directly upstream of the pier, m (ft)
	=	Depth of flow at the abutment, on the overbank or in the main channel for abutment scour, m (ft)
y_2	=	Average depth in the contracted section (bridge opening) or on the overbank at the bridge, m (ft)
	=	Average depth under lower cord, m (ft)
Z	=	Vertical offset to datum, m (ft)
τ_2, τ_o	=	Average bed shear stress at the contracted section, Pa or N/m^2 (lbs/ft^2)
τ_c	=	Critical bed shear stress at incipient motion, N/m^2 (lbs/ft^2)
γ	=	Specific weight of water, N/m^3 (lbs/ft^3)
ρ	=	Density of water, kg/m^3 ($slugs/ft^3$)
ρ_s	=	Density of sediment, kg/m^3 ($slugs/ft^3$)
θ	=	Angle of repose of the bed material (ranges from about 30° to 44°)
	=	Skew angle of flow with respect to pier
	=	Skew angle of abutment (embankment) with respect to flow
	=	Angle, in degrees, subdividing the tidal cycle
ΔH	=	Maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, m (ft)

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This manual is a major revision of the third edition of HEC-18 which was published in 1995. The writers wish to acknowledge the technical assistance of C. Dunn and L.J. Harrison, formerly FHWA; GKY and Associates; D. Halverson, formerly Minnesota DOT; J.R. Richardson, formerly RCE, Inc.; and L. Butler, formerly U.S. Army Corps of Engineers Waterways Experiment Station, all of whom contributed to the previous edition(s) of HEC-18.

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GLOSSARY

abrasion: Removal of streambank material due to entrained sediment, ice, or debris rubbing against the bank.

aggradation: General and progressive buildup of the longitudinal profile of a channel bed due to sediment deposition.

alluvial channel: Channel wholly in alluvium; no bedrock is exposed in channel at low flow or likely to be exposed by erosion.

alluvial fan: A fan-shaped deposit of material at the place where a stream issues from a narrow valley of high slope onto a plain or broad valley of low slope. An alluvial cone is made up of the finer materials suspended in flow while a debris cone is a mixture of all sizes and kinds of materials.

alluvial stream: A stream which has formed its channel in cohesive or noncohesive materials that have been and can be transported by the stream.

alluvium: Unconsolidated material deposited by a stream in a channel, floodplain, alluvial fan, or delta.

alternating bars: Elongated deposits found alternately near the right and left banks of a channel.

anabranch: Individual channel of an anabranched stream.

anabranched stream: A stream whose flow is divided at normal and lower stages by large islands or, more rarely, by large bars; individual islands or bars are wider than about three times water width; channels are more widely and distinctly separated than in a braided stream.

anastomosing stream: An anabranched stream.

angle of repose: The maximum angle (as measured from the horizontal) at which gravel or sand particles can stand.

annual flood: The maximum flow in one year (may be daily or instantaneous).

apron: Protective material placed on a streambed to resist scour.

apron, launching: An apron designed to settle and protect the side slopes of a scour hole after settlement.

armor (armoring): Surfacing of channel bed, banks, or embankment slope to resist erosion and scour. (a) Natural process whereby an erosion- resistant layer of relatively large particles is formed on a streambed due to the removal of finer particles by streamflow; (b) placement of a covering to resist erosion.

articulated concrete mattress: Rigid concrete slabs which can move without separating as scour occurs; usually hinged together with corrosion-resistant cable fasteners; primarily placed for lower bank protection.

average velocity: Velocity at a given cross section determined by dividing discharge by cross sectional area.

avulsion: A sudden change in the channel course that usually occurs when a stream breaks through its banks; usually associated with a flood or a catastrophic event.

backfill: The material used to refill a ditch or other excavation, or the process of doing so.

backwater: The increase in water surface elevation relative to the elevation occurring under natural channel and floodplain conditions. It is induced by a bridge or other structure that obstructs or constricts the free flow of water in a channel.

backwater area: The low-lying lands adjacent to a stream that may become flooded due to backwater.

bank: The sides of a channel between which the flow is normally confined.

bank, left (right): The side of a channel as viewed in a downstream direction.

bankfull discharge: Discharge that, on the average, fills a channel to the point of overflowing.

bank protection: Engineering works for the purpose of protecting streambanks from erosion.

bank revetment: Erosion-resistant materials placed directly on a streambank to protect the bank from erosion.

bar: An elongated deposit of alluvium within a channel, not permanently vegetated.

base floodplain: The floodplain associated with the flood with a 100-year recurrence interval.

bay: A body of water connected to the ocean with an inlet.

bed: The bottom of a channel bounded by banks.

bed form: A recognizable relief feature on the bed of a channel, such as a ripple, dune, plane bed, antidune, or bar. Bed forms are a consequence of the interaction between hydraulic forces (boundary shear stress) and the bed sediment.

bed layer: A flow layer, several grain diameters thick (usually two) immediately above the bed.

bed load: Sediment that is transported in a stream by rolling, sliding, or skipping along the bed or very close to it; considered to be within the bed layer (contact load).

bed load discharge (or bed load): The quantity of bed load passing a cross section of a stream in a unit of time.

bed material: Material found in and on the bed of a stream (May be transported as bed load or in suspension).

bedrock: The solid rock exposed at the surface of the earth or overlain by soils and unconsolidated material.

bed sediment discharge: The part of the total sediment discharge that is composed of grain sizes found in the bed and is equal to the transport capability of the flow.

bed shear (tractive force): The force per unit area exerted by a fluid flowing past a stationary boundary.

bed slope: The inclination of the channel bottom.

blanket: Material covering all or a portion of a streambank to prevent erosion.

boulder: A rock fragment whose diameter is greater than 250 mm.

braid: A subordinate channel of a braided stream.

braided stream: A stream whose flow is divided at normal stage by small mid-channel bars or small islands; the individual width of bars and islands is less than about three times water width; a braided stream has the aspect of a single large channel within which are subordinate channels.

bridge opening: The cross-sectional area beneath a bridge that is available for conveyance of water.

bridge waterway: The area of a bridge opening available for flow, as measured below a specified stage and normal to the principal direction of flow.

bulk density: Density of the water sediment mixture (mass per unit volume), including both water and sediment.

bulkhead: A vertical, or near vertical, wall that supports a bank or an embankment; also may serve to protect against erosion.

bulking: Increasing the water discharge to account for high concentrations of sediment in the flow.

catchment: See drainage basin.

causeway: Rock or earth embankment carrying a roadway across water.

caving: The collapse of a bank caused by undermining due to the action of flowing water.

cellular-block mattress: Interconnected concrete blocks with regular cavities placed directly on a streambank or filter to resist erosion. The cavities can permit bank drainage and the growth of vegetation where synthetic filter fabric is not used between the bank and mattress.

channel: The bed and banks that confine the surface flow of a stream.

channelization: Straightening or deepening of a natural channel by artificial cutoffs, grading, flow-control measures, or diversion of flow into an engineered channel.

channel diversion: The removal of flows by natural or artificial means from a natural length of channel.

channel pattern: The aspect of a stream channel in plan view, with particular reference to the degree of sinuosity, braiding, and anabranching.

channel process: Behavior of a channel with respect to shifting, erosion and sedimentation.

check dam: A low dam or weir across a channel used to control stage or degradation.

choking (of flow): Excessive constriction of flow which may cause severe backwater effect.

clay (mineral): A particle whose diameter is in the range of 0.00024 to 0.004 mm.

clay plug: A cutoff meander bend filled with fine grained cohesive sediments.

clear-water scour: Scour at a pier or abutment (or contraction scour) when there is no movement of the bed material upstream of the bridge crossing at the flow causing bridge scour.

cobble: A fragment of rock whose diameter is in the range of 64 to 250 mm.

concrete revetment: Unreinforced or reinforced concrete slabs placed on the channel bed or banks to protect it from erosion.

confluence: The junction of two or more streams.

constriction: A natural or artificial control section, such as a bridge crossing, channel reach or dam, with limited flow capacity in which the upstream water surface elevation is related to discharge.

contact load: Sediment particles that roll or slide along in almost continuous contact with the streambed (bed load).

contraction: The effect of channel or bridge constriction on flow streamlines.

contraction scour: Contraction scour, in a natural channel or at a bridge crossing, involves the removal of material from the bed and banks across all or most of the channel width. This component of scour results from a contraction of the flow area at the bridge which causes an increase in velocity and shear stress on the bed at the bridge. The contraction can be caused by the bridge or from a natural narrowing of the stream channel.

Coriolis force: The inertial force caused by the Earth's rotation that deflects a moving body to the right in the Northern Hemisphere.

countermeasure: A measure intended to prevent, delay or reduce the severity of hydraulic problems.

crib: A frame structure filled with earth or stone ballast, designed to reduce energy and to deflect streamflow away from a bank or embankment.

critical shear stress: The minimum amount of shear stress required to initiate soil particle motion.

crossing: The relatively short and shallow reach of a stream between bends; also crossover or riffle.

cross section: A section normal to the trend of a channel or flow.

current: Water flowing through a channel.

current meter: An instrument used to measure flow velocity.

cut bank: The concave wall of a meandering stream.

cutoff: (a) A direct channel, either natural or artificial, connecting two points on a stream, thereby shortening the original length of the channel and increasing its slope; (b) A natural or artificial channel which develops across the neck of a meander loop (neck cutoff) or across a point bar (chute cutoff).

cutoff wall: A wall, usually of sheet piling or concrete, that extends down to scour-resistant material or below the expected scour depth.

daily discharge:	Discharge averaged over one day (24 hours).
debris:	Floating or submerged material, such as logs, vegetation, or trash, transported by a stream.
degradation (bed):	A general and progressive (long-term) lowering of the channel bed due to erosion, over a relatively long channel length.
deep water (for waves):	Water of such a depth that surface waves are little affected by bottom conditions; customarily, water deeper than half the wavelength.
depth of scour:	The vertical distance a streambed is lowered by scour below a reference elevation.
design flow (design flood):	The discharge that is selected as the basis for the design or evaluation of a hydraulic structure.
dike:	An impermeable linear structure for the control or containment of overbank flow. A dike-trending parallel with a streambank differs from a levee in that it extends for a much shorter distance along the bank, and it may be surrounded by water during floods.
dike (groin, spur, jetty):	A structure extending from a bank into a channel that is designed to: (a) reduce the stream velocity as the current passes through the dike, thus encouraging sediment deposition along the bank (permeable dike); or (b) deflect erosive current away from the streambank (impermeable dike).
diurnal tide	Tides with an approximate tidal period of 24 hours.
discharge:	Volume of water passing through a channel during a given time.
dominant discharge:	(a) The discharge of water which is of sufficient magnitude and frequency to have a dominating effect in determining the characteristics and size of the stream course, channel, and bed; (b) That discharge which determines the principal dimensions and characteristics of a natural channel. The dominant formative discharge depends on the maximum and mean discharge, duration of flow, and flood frequency. For hydraulic geometry relationships, it is taken to be the bankfull discharge which has a return period of approximately 1.5 years in many natural channels.
drainage basin:	An area confined by drainage divides, often having only one outlet for discharge (catchment, watershed).
drift:	Alternative term for vegetative "debris."

ebb tide: Flow of water from the bay or estuary to the ocean.

eddy current: A vortex-type motion of a fluid flowing contrary to the main current, such as the circular water movement that occurs when the main flow becomes separated from the bank.

entrenched stream: Stream cut into bedrock or consolidated deposits.

ephemeral stream: A stream or reach of stream that does not flow for parts of the year. As used here, the term includes intermittent streams with flow less than perennial.

equilibrium scour: Scour depth in sand-bed stream with dune bed about which live bed pier scour level fluctuates due to variability in bed material transport in the approach flow.

erosion: Displacement of soil particles due to water or wind action.

erosion control matting: Fibrous matting (e.g., jute, paper, etc.) placed or sprayed on a stream- bank for the purpose of resisting erosion or providing temporary stabilization until vegetation is established.

estuary: Tidal reach at the mouth of a river.

fabric mattress: Grout-filled mattress used for streambank protection.

fall velocity: The velocity at which a sediment particle falls through a column of still water.

fascine: A matrix of willow or other natural material woven in bundles and used as a filter. Also, a streambank protection technique consisting of wire mesh or timber attached to a series of posts, sometimes in double rows; the space between the rows may be filled with rock, brush, or other materials.

fetch: The area in which waves are generated by wind having a rather constant direction and speed; sometimes used synonymously with fetch length.

fetch length: The horizontal distance (in the direction of the wind) over which wind generates waves and wind setup.

fill slope: Side or end slope of an earth-fill embankment. Where a fill-slope forms the streamward face of a spill-through abutment, it is regarded as part of the abutment.

filter: Layer of fabric (geotextile) or granular material (sand, gravel, or graded rock) placed between bank revetment (or bed protection) and soil for the following purposes: (1) to prevent the soil from moving through the revetment by piping, extrusion, or erosion; (2) to prevent the revetment from sinking into the soil; and (3) to permit natural seepage from the streambank, thus preventing the buildup of excessive hydrostatic pressure.

filter blanket: A layer of graded sand and gravel laid between fine-grained material and riprap to serve as a filter.

filter fabric (cloth): Geosynthetic fabric that serves the same purpose as a granular filter blanket.

fine sediment load: That part of the total sediment load that is composed of particle sizes finer than those represented in the bed (wash load). Normally, the fine-sediment load is finer than 0.062 mm for sand-bed channels. Silts, clays and sand could be considered wash load in coarse gravel and cobble-bed channels.

flanking: Erosion around the landward end of a stream stabilization countermeasure.

flashy stream: Stream characterized by rapidly rising and falling stages, as indicated by a sharply peaked hydrograph. Typically associated with mountain streams or highly disturbed urbanized catchments. Most flashy streams are ephemeral, but some are perennial.

flood tide: Flow of water from the ocean to the bay or estuary.

flood-frequency curve: A graph indicating the probability that the annual flood discharge will exceed a given magnitude, or the recurrence interval corresponding to a given magnitude.

floodplain: A nearly flat, alluvial lowland bordering a stream, that is subject to frequent inundation by floods.

flow-control structure: A structure either within or outside a channel that acts as a countermeasure by controlling the direction, depth, or velocity of flowing water.

flow hazard: Flow characteristics (discharge, stage, velocity, or duration) that are associated with a hydraulic problem or that can reasonably be considered of sufficient magnitude to cause a hydraulic problem or to test the effectiveness of a countermeasure.

flow slide: Saturated soil materials which behave more like a liquid than a solid. A flow slide on a channel bank can result in a bank failure.

fluvial geomorphology: The science dealing with the morphology (form) and dynamics of streams and rivers.

fluvial system: The natural river system consisting of (1) the drainage basin, watershed, or sediment source area, (2) tributary and mainstem river channels or sediment transfer zone, and (3) alluvial fans, valley fills and deltas, or the sediment deposition zone.

freeboard: The vertical distance above a design stage that is allowed for waves, surges, drift, and other contingencies.

fresh water:	Water that is not salty as compared to sea water which generally has a salinity of 35 000 parts per million.
Froude Number:	A dimensionless number that represents the ratio of inertial to gravitational forces in open channel flow.
gabion:	A basket or compartmented rectangular container made of wire mesh. When filled with cobbles or other rock of suitable size, the gabion becomes a flexible and permeable unit with which flow- and erosion-control structures can be built.
general scour:	General scour is a lowering of the streambed across the stream or waterway at the bridge. This lowering may be uniform across the bed or non-uniform. That is, the depth of scour may be deeper in some parts of the cross section. General scour may result from contraction of the flow or other general scour conditions such as flow around a bend.
geomorphology/morphology:	That science that deals with the form of the Earth, the general configuration of its surface, and the changes that take place due to erosion and deposition.
grade-control structure (sill, check dam):	Structure placed bank to bank across a stream channel (usually with its central axis perpendicular to flow) for the purpose of controlling bed slope and preventing scour or headcutting.
graded stream:	A geomorphic term used for streams that have apparently achieved a state of equilibrium between the rate of sediment transport and the rate of sediment supply throughout long reaches.
gravel:	A rock fragment whose diameter ranges from 2 to 64 mm.
groin:	A structure built from the bank of a stream in a direction transverse to the current to redirect the flow or reduce flow velocity. Many names are given to this structure, the most common being "spur," "spur dike," "transverse dike," "jetty," etc. Groins may be permeable, semi-permeable, or impermeable.
grout:	A fluid mixture of cement and water or of cement, sand, and water used to fill joints and voids.
guide bank:	A dike extending upstream from the approach embankment at either or both sides of the bridge opening to direct the flow through the opening. Some guidebanks extend downstream from the bridge (also spur dike).
hardpoint:	A streambank protection structure whereby "soft" or erodible materials are removed from a bank and replaced by stone or compacted clay. Some hard points protrude a short distance into the channel to direct erosive currents away from the bank. Hard points also occur naturally along streambanks as passing currents remove erodible materials leaving nonerodible materials exposed.

headcutting: Channel degradation associated with abrupt changes in the bed elevation (headcut) that generally migrates in an upstream direction.

helical flow: Three-dimensional movement of water particles along a spiral path in the general direction of flow. These secondary-type currents are of most significance as flow passes through a bend; their net effect is to remove soil particles from the cut bank and deposit this material on a point bar.

hydraulics: The applied science concerned with the behavior and flow of liquids, especially in pipes, channels, structures, and the ground.

hydraulic model: A small-scale physical or mathematical representation of a flow situation.

hydraulic problem: An effect of streamflow, tidal flow, or wave action such that the integrity of the highway facility is destroyed, damaged, or endangered.

hydraulic radius: The cross-sectional area of a stream divided by its wetted perimeter.

hydraulic structures: The facilities used to impound, accommodate, convey or control the flow of water, such as dams, weirs, intakes, culverts, channels, and bridges.

hydrograph: The graph of stage or discharge against time.

hydrology: The science concerned with the occurrence, distribution, and circulation of water on the earth.

imbricated: In reference to stream bed sediment particles, having an overlapping or shingled pattern.

icing: Masses or sheets of ice formed on the frozen surface of a river or floodplain. When shoals in the river are frozen to the bottom or otherwise dammed, water under hydrostatic pressure is forced to the surface where it freezes.

incised reach: A stretch of stream with an incised channel that only rarely overflows its banks.

incised stream: A stream which has deepened its channel through the bed of the valley floor, so that the floodplain is a terrace.

invert: The lowest point in the channel cross section or at flow control devices such as weirs, culverts, or dams.

island: A permanently vegetated area, emergent at normal stage, that divides the flow of a stream. Islands originate by establishment of vegetation on a bar, by channel avulsion, or at the junction of minor tributary with a larger stream.

jack: A device for flow control and protection of banks against lateral erosion consisting of three mutually perpendicular arms rigidly fixed at the center. Kellner jacks are made of steel struts strung with wire, and concrete jacks are made of reinforced concrete beams.

jack field: Rows of jacks tied together with cables, some rows generally parallel with the banks and some perpendicular thereto or at an angle. Jack fields may be placed outside or within a channel.

jetty: (a) An obstruction built of piles, rock, or other material extending from a bank into a stream, so placed as to induce bank building, or to protect against erosion; (b) A similar obstruction to influence stream, lake, or tidal currents, or to protect a harbor (also spur).

lateral erosion: Erosion in which the removal of material is extended horizontally as contrasted with degradation and scour in a vertical direction.

launching: Release of undercut material (stone riprap, rubble, slag, etc.) downslope or into a scoured area.

levee: An embankment, generally landward of top bank, that confines flow during high-water periods, thus preventing overflow into lowlands.

littoral transport or drift: Transport of beach material along a shoreline by wave action. Also, longshore sediment transport.

live-bed scour: Scour at a pier or abutment (or contraction scour) when the bed material in the channel upstream of the bridge is moving at the flow causing bridge scour.

load (or sediment load): Amount of sediment being moved by a stream.

local scour: Removal of material from around piers, abutments, spurs, and embankments caused by an acceleration of flow and resulting vortices induced by obstructions to the flow.

longitudinal profile: The profile of a stream or channel drawn along the length of its centerline. In drawing the profile, elevations of the water surface or the thalweg are plotted against distance as measured from the mouth or from an arbitrary initial point.

lower bank: That portion of a streambank having an elevation less than the mean water level of the stream.

mathematical model: A numerical representation of a flow situation using mathematical equations (also computer model).

mattress: A blanket or revetment of materials interwoven or otherwise lashed together and placed to cover an area subject to scour.

meander or full meander: A meander in a river consists of two consecutive loops, one flowing clockwise and the other counter-clockwise.

meander amplitude: The distance between points of maximum curvature of successive meanders of opposite phase in a direction normal to the general course of the meander belt, measured between center lines of channels.

meander belt: The distance between lines drawn tangent to the extreme limits of successive fully developed meanders.

meander length: The distance along a stream between corresponding points of successive meanders.

meander loop: An individual loop of a meandering or sinuous stream lying between inflection points with adjoining loops.

meander ratio: The ratio of meander width to meander length.

meander radius of curvature: The radius of a circle inscribed on the centerline of a meander loop.

meander scrolls: Low, concentric ridges and swales on a floodplain, marking the successive positions of former meander loops.

meander width: The amplitude of a fully developed meander measured from midstream to midstream.

meandering stream: A stream having a sinuosity greater than some arbitrary value. The term also implies a moderate degree of pattern symmetry, imparted by regularity of size and repetition of meander loops. The channel generally exhibits a characteristic process of bank erosion and point bar deposition associated with systematically shifting meanders.

median diameter: The particle diameter of the 50th percentile point on a size distribution curve such that half of the particles (by weight, number, or volume) are larger and half are smaller ($D_{50.}$)

mid-channel bar: A bar lacking permanent vegetal cover that divides the flow in a channel at normal stage.

middle bank:	The portion of a streambank having an elevation approximately the same as that of the mean water level of the stream.
migration:	Change in position of a channel by lateral erosion of one bank and simultaneous accretion of the opposite bank.
mud:	A soft, saturated mixture mainly of silt and clay.
natural levee:	A low ridge that slopes gently away from the channel banks that is formed along streambanks during floods by deposition.
nominal diameter:	Equivalent spherical diameter of a hypothetical sphere of the same volume as a given sediment particle.
nonalluvial channel:	A channel whose boundary is in bedrock or non-erodible material.
normal stage:	The water stage prevailing during the greater part of the year.
overbank flow:	Water movement that overtops the bank either due to stream stage or to overland surface water runoff.
oxbow:	The abandoned former meander loop that remains after a stream cuts a new, shorter channel across the narrow neck of a meander. Often bow-shaped or horseshoe-shaped.
pavement:	Streambank surface covering, usually impermeable, designed to serve as protection against erosion. Common pavements used on streambanks are concrete, compacted asphalt, and soil-cement.
paving:	Covering of stones on a channel bed or bank (used with reference to natural covering).
peaked stone dike:	Riprap placed parallel to the toe of a streambank (at the natural angle of repose of the stone) to prevent erosion of the toe and induce sediment deposition behind the dike.
perennial stream:	A stream or reach of a stream that flows continuously for all or most of the year.
phreatic line:	The upper boundary of the seepage water surface landward of a streambank.
pile:	An elongated member, usually made of timber, concrete, or steel, that serves as a structural component of a river-training structure.
pile dike:	A type of permeable structure for the protection of banks against caving; consists of a cluster of piles driven into the stream, braced and lashed together.

piping: Removal of soil material through subsurface flow of seepage water that develops channels or "pipes" within the soil bank.

point bar: An alluvial deposit of sand or gravel lacking permanent vegetal cover occurring in a channel at the inside of a meander loop, usually somewhat downstream from the apex of the loop.

poised stream: A stream which, as a whole, maintains its slope, depths, and channel dimensions without any noticeable raising or lowering of its bed (stable stream). Such condition may be temporary from a geological point of view, but for practical engineering purposes, the stream may be considered stable.

probable maximum flood: A very rare flood discharge value computed by hydro-meteorological methods, usually in connection with major hydraulic structures.

quarry-run stone: Stone as received from a quarry without regard to gradation requirements.

railbank protection: A type of countermeasure composed of rock-filled wire fabric supported by steel rails or posts driven into streambed.

rapid drawdown: Lowering the water against a bank more quickly than the bank can drain without becoming unstable.

reach: A segment of stream length that is arbitrarily bounded for purposes of study.

recurrence interval: The reciprocal of the annual probability of exceedance of a hydrologic event (also return period, exceedance interval).

regime: The condition of a stream or its channel with regard to stability. A stream is in regime if its channel has reached an equilibrium form as a result of its flow characteristics. Also, the general pattern of variation around a mean condition, as in flow regime, tidal regime, channel regime, sediment regime, etc. (used also to mean a set of physical characteristics of a river).

regime change: A change in channel characteristics resulting from such things as changes in imposed flows, sediment loads, or slope.

regime channel: Alluvial channel that has attained, more or less, a state of equilibrium with respect to erosion and deposition.

regime formula: A formula relating stable alluvial channel dimensions or slope to discharge and sediment characteristics.

reinforced-earth bulkhead: A retaining structure consisting of vertical panels and attached to reinforcing elements embedded in compacted backfill for supporting a streambank.

reinforced revetment:	A streambank protection method consisting of a continuous stone toe-fill along the base of a bank slope with intermittent fillets of stone placed perpendicular to the toe and extending back into the natural bank.
relief bridge:	An opening in an embankment on a floodplain to permit passage of overbank flow.
retard (retarder structure):	A permeable or impermeable linear structure in a channel parallel with the bank and usually at the toe of the bank, intended to reduce flow velocity, induce deposition, or deflect flow from the bank.
revetment:	Rigid or flexible armor placed to inhibit scour and lateral erosion. (See bank revetment).
riffle:	A natural, shallow flow area extending across a streambed in which the surface of flowing water is broken by waves or ripples. Typically, riffles alternate with pools along the length of a stream channel.
riparian:	Pertaining to anything connected with or adjacent to the banks of a stream (corridor, vegetation, zone, etc.).
riprap:	Layer or facing of rock or broken concrete dumped or placed to protect a structure or embankment from erosion; also the rock or broken concrete suitable for such use. Riprap has also been applied to almost all kinds of armor, including wire-enclosed riprap, grouted riprap, sacked concrete, and concrete slabs.
river training:	Engineering works with or without the construction of embankment, built along a stream or reach of stream to direct or to lead the flow into a prescribed channel. Also, any structure configuration constructed in a stream or placed on, adjacent to, or in the vicinity of a streambank that is intended to deflect currents, induce sediment deposition, induce scour, or in some other way alter the flow and sediment regimes of the stream.
rock-and-wire mattress:	A flat wire cage or basket filled with stone or other suitable material and placed as protection against erosion.
roughness coefficient:	Numerical measure of the frictional resistance to flow in a channel, as in the Manning's or Chezy's formulas.
rubble:	Rough, irregular fragments of materials of random size used to retard erosion. The fragments may consist of broken concrete slabs, masonry, or other suitable refuse.
runoff:	That part of precipitation which appears in surface streams of either perennial or intermittent form.
run-up, wave:	Height to which water rises above still-water elevation when waves meet a beach, wall, etc.

sack revetment: Sacks (e.g., burlap, paper, or nylon) filled with mortar, concrete, sand, stone or other available material used as protection against erosion.

saltation load: Sediment bounced along the streambed by energy and turbulence of flow, and by other moving particles.

sand: A rock fragment whose diameter is in the range of 0.062 to 2.0 mm.

scour: Erosion of streambed or bank material due to flowing water; often considered as being localized (see local scour, contraction scour, total scour).

sediment or fluvial sediment: Fragmental material transported, suspended, or deposited by water.

sediment concentration: Weight or volume of sediment relative to the quantity of transporting (or suspending) fluid.

sediment discharge: The quantity of sediment that is carried past any cross section of a stream in a unit of time. Discharge may be limited to certain sizes of sediment or to a specific part of the cross section.

sediment load: Amount of sediment being moved by a stream.

sediment yield: The total sediment outflow from a watershed or a drainage area at a point of reference and in a specified time period. This outflow is equal to the sediment discharge from the drainage area.

seepage: The slow movement of water through small cracks and pores of the bank material.

seiche: Long-period oscillation of a lake or similar body of water.

semi-diurnal tide Tides with an approximate tidal period of 12 hours.

set-up: Raising of water level due to wind action.

set-up, wave: Height to which water rises above still-water elevation as a result of storm wind effects.

shallow water (for waves): Water of such a depth that waves are noticeably affected by bottom conditions; customarily, water shallower than half the wavelength.

shear stress: See unit shear force.

shoal: A relatively shallow submerged bank or bar in a body of water.

sill:	(a) A structure built under water, across the deep pools of a stream with the aim of changing the depth of the stream; (b) A low structure built across an effluent stream, diversion channel or outlet to reduce flow or prevent flow until the main stream stage reaches the crest of the structure.
silt:	A particle whose diameter is in the range of 0.004 to 0.062 mm.
sinuosity:	The ratio between the thalweg length and the valley length of a stream.
slope (of channel or stream):	Fall per unit length along the channel centerline or thalweg.
slope protection:	Any measure such as riprap, paving, vegetation, revetment, brush or other material intended to protect a slope from erosion, slipping or caving, or to withstand external hydraulic pressure.
sloughing:	Sliding or collapse of overlying material; same ultimate effect as caving, but usually occurs when a bank or an underlying stratum is saturated.
slope-area method:	A method of estimating unmeasured flood discharges in a uniform channel reach using observed high-water levels.
slump:	A sudden slip or collapse of a bank, generally in the vertical direction and confined to a short distance, probably due to the substratum being washed out or having become unable to bear the weight above it.
soil-cement:	A designed mixture of soil and Portland cement compacted at a proper water content to form a blanket or structure that can resist erosion.
sorting:	Progressive reduction of size (or weight) of particles of the sediment load carried down a stream.
spill-through abutment:	A bridge abutment having a fill slope on the streamward side. The term originally referred to the "spill-through" of fill at an open abutment but is now applied to any abutment having such a slope.
spread footing:	A pier or abutment footing that transfers load directly to the earth.
spur:	A permeable or impermeable linear structure that projects into a channel from the bank to alter flow direction, induce deposition, or reduce flow velocity along the bank.
spur dike:	See guide bank.

stability:	A condition of a channel when, though it may change slightly at different times of the year as the result of varying conditions of flow and sediment charge, there is no appreciable change from year to year; that is, accretion balances erosion over the years.
stable channel:	A condition that exists when a stream has a bed slope and cross section which allows its channel to transport the water and sediment delivered from the upstream watershed without aggradation, degradation, or bank erosion (a graded stream).
stage:	Water-surface elevation of a stream with respect to a reference elevation.
still-water elevation:	Flood height to which water rises as a result of barometric pressure changes occurring during a storm event.
stone riprap:	Natural cobbles, boulders, or rock dumped or placed as protection against erosion.
stream:	A body of water that may range in size from a large river to a small rill flowing in a channel. By extension, the term is sometimes applied to a natural channel or drainage course formed by flowing water whether it is occupied by water or not.
streambank erosion:	Removal of soil particles or a mass of particles from a bank surface due primarily to water action. Other factors such as weathering, ice and debris abrasion, chemical reactions, and land use changes may also directly or indirectly lead to bank erosion.
streambank failure:	Sudden collapse of a bank due to an unstable condition such as removal of material at the toe of the bank by scour.
streambank protection:	Any technique used to prevent erosion or failure of a streambank.
storm surge:	Coastal flooding phenomenon resulting from wind and barometric changes. The storm surge is measured by subtracting the astronomical tide elevation from the total flood elevation (Hurricane surge).
storm tide:	Coastal flooding resulting from combination of storm surge and astronomical tide (often referred to as storm surge)
suspended sediment discharge:	The quantity of sediment passing through a stream cross section above the bed layer in a unit of time suspended by the turbulence of flow (suspended load).
sub-bed material:	Material underlying that portion of the streambed which is subject to direct action of the flow. Also, substrate.

subcritical, supercritical flow: Open channel flow conditions with Froude Number less than and greater than unity, respectively.

tetrahedron: Component of river-training works made of six steel or concrete struts fabricated in the shape of a pyramid.

tetrapod: Bank protection component of precast concrete consisting of four legs joined at a central joint, with each leg making an angle of 109.5° with the other three.

thalweg: The line extending down a channel that follows the lowest elevation of the bed.

tidal amplitude: Generally, half of tidal range.

tidal cycle: One complete rise and fall of the tide.

tidal day: Time of rotation of the earth with respect to the moon. Assumed to equal approximately 24.84 solar hours in length.

tidal inlet: A channel connecting a bay or estuary to the ocean.

tidal passage: A tidal channel connected with the ocean at both ends.

tidal period: Duration of one complete tidal cycle. When the tidal period equals the tidal day (24.84 hours), the tide exhibits diurnal behavior. Should two complete tidal periods occur during the tidal day, the tide exhibits semi-diurnal behavior.

tidal prism: Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.

tidal range: Vertical distance between specified low and high tide levels.

tidal scour: Scour at bridges over tidal waterways, i.e., in the coastal zone.

tidal waterways: A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands, inlets to bays, crossings between bays, tidally affected streams, etc.

tides, astronomical: Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating earth. Also, daily tides.

tieback: Structure placed between revetment and bank to prevent flanking.

timber or brush mattress: A revetment made of brush, poles, logs, or lumber interwoven or otherwise lashed together. The completed mattress is then placed on the bank of a stream and weighted with ballast.

toe of bank:	That portion of a stream cross section where the lower bank terminates and the channel bottom or the opposite lower bank begins.
toe protection:	Loose stones laid or dumped at the toe of an embankment, groin, etc., or masonry or concrete wall built at the junction of the bank and the bed in channels or at extremities of hydraulic structures to counteract erosion.
total scour:	The sum of long-term degradation, general (contraction) scour, and local scour.
total sediment load:	The sum of suspended load and bed load or the sum of bed material load and wash load of a stream (total load).
tractive force:	The drag or shear on a streambed or bank caused by passing water which tends to move soil particles along with the streamflow.
trench-fill revetment:	Stone, concrete, or masonry material placed in a trench dug behind and parallel to an eroding streambank. When the erosive action of the stream reaches the trench, the material placed in the trench armors the bank and thus retards further erosion.
tsunami:	Long-period ocean wave resulting from earthquake, other seismic disturbances or submarine landslides.
turbulence:	Motion of fluids in which local velocities and pressures fluctuate irregularly in a random manner as opposed to laminar flow where all particles of the fluid move in distinct and separate lines.
ultimate scour:	The maximum depth of scour attained for a given flow condition. May require multiple flow events and in cemented or cohesive soils may be achieved over a long time period.
uniform flow:	Flow of constant cross section and velocity through a reach of channel at a given time. Both the energy slope and the water slope are equal to the bed slope under conditions of uniform flow.
unit discharge:	Discharge per unit width (may be average over a cross section, or local at a point).
unit shear force (shear stress):	The force or drag developed at the channel bed by flowing water. For uniform flow, this force is equal to a component of the gravity force acting in a direction parallel to the channel bed on a unit wetted area. Usually in units of stress, Pa (N/m^2) or (lb/ft^2).
unsteady flow:	Flow of variable discharge and velocity through a cross section with respect to time.

upper bank:	The portion of a streambank having an elevation greater than the average water level of the stream.
velocity:	The time rate of flow usually expressed in m/s (ft/sec). The average velocity is the velocity at a given cross section determined by dividing discharge by cross-sectional area.
vertical abutment:	An abutment, usually with wingwalls, that has no fill slope on its streamward side.
vortex:	Turbulent eddy in the flow generally caused by an obstruction such as a bridge pier or abutment (e.g., horseshoe vortex).
wandering channel:	A channel exhibiting a more or less non-systematic process of channel shifting, erosion and deposition, with no definite meanders or braided pattern.
wandering thalweg:	A thalweg whose position in the channel shifts during floods and typically serves as an inset channel that conveys all or most of the stream flow at normal or lower stages.
wash load:	Suspended material of very small size (generally clays and colloids) originating primarily from erosion on the land slopes of the drainage area and present to a negligible degree in the bed itself.
watershed:	See drainage basin.
waterway opening width (area):	Width (area) of bridge opening at (below) a specified stage, measured normal to the principal direction of flow.
wave period:	Time interval between arrivals of successive wave crests at a point.
weephole:	A hole in an impermeable wall or revetment to relieve the neutral stress or pore pressure in the soil.
windrow revetment:	A row of stone placed landward of the top of an eroding streambank. As the windrow is undercut, the stone is launched downslope, thus armoring the bank.
wire mesh:	Wire woven to form a mesh; where used as an integral part of a countermeasure, openings are of suitable size and shape to enclose rock or broken concrete or to function on fence-like spurs and retards.

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CHAPTER 1

INTRODUCTION

1.1 PURPOSE

The purpose of this document is to provide guidelines for the following:

1. Designing new and replacement bridges to resist scour
2. Evaluating existing bridges for vulnerability to scour
3. Inspecting bridges for scour
4. Improving the state-of-practice of estimating scour at bridges

1.2 BACKGROUND

The most common cause of bridge failures is from floods scouring bed material from around bridge foundations. Scour is the engineering term for the erosion caused by water of the soil surrounding a bridge foundation (piers and abutments). During the spring floods of 1987, 17 bridges in New York and New England were damaged or destroyed by scour. In 1985, 73 bridges were destroyed by floods in Pennsylvania, Virginia, and West Virginia. A 1973 national study for the Federal Highway Administration (FHWA) of 383 bridge failures caused by catastrophic floods showed that 25 percent involved pier damage and 75 percent involved abutment damage.(1) A second more extensive study in 1978 indicated local scour at bridge piers to be a problem about equal to abutment scour problems.(2) A number of case histories on the causes and consequences of scour at major bridges are presented in Transportation Research Record 950.(3)

From available information, the 1993 flood in the upper Mississippi basin, caused 23 bridge failures for an estimated damage of $15 million. The modes of bridge failures were 14 from abutment scour, two from pier scour, three from pier and abutment scour, two from lateral bank migration, one from debris load, and one from unknown cause.(4)

In the 1994 flooding from storm Alberto in Georgia, there were over 500 state and locally owned bridges with damage attributed to scour. Thirty-one of state-owned bridges experienced from 15 to 20 feet of contraction scour and/or long-term degradation in addition to local scour. These bridges had to be replaced. Of more than 150 bridges identified as scour damaged, the Georgia Department of Transportation (GADOT) also recommended that 73 non-federal aid bridges be repaired or replaced. Total damage to the GADOT highway system was approximately $130 million.(4)

The American Association of State Highway and Transportation Officials (AASHTO) standard specifications for highway bridges has the following requirements to address the problem of stream stability and scour.(5)

- Hydraulic studies are a necessary part of the preliminary design of a bridge and should include. . .estimated scour depths at piers and abutments of proposed structures.

- The probable depth of scour shall be determined by subsurface exploration and hydraulic studies. Refer to Article 1.3.2 and FHWA Hydraulic Engineering Circular (HEC) 18 for general guidance regarding hydraulic studies and design.

- . . .in all cases, the pile length shall be determined such that the design structural load may be safely supported entirely below the probable scour depth.

1.3 COMPREHENSIVE ANALYSIS

This manual is part of a set of HECs issued by FHWA to provide guidance for bridge scour and stream stability analyses. The three manuals in this set are:

HEC-18	Evaluating Scour at Bridges
HEC-20	Stream Stability at Highway Structures[6]
HEC-23	Bridge Scour and Stream Instability Countermeasures[7]

The Flow Chart of Figure 1.1 illustrates graphically the interrelationship between these three documents and emphasizes that they should be used as a set. A comprehensive scour analysis or stability evaluation should be based on information presented in all three documents.

While the flow chart does not attempt to present every detail of a complete stream stability and scour evaluation, it has sufficient detail to show the major elements in a complete analysis, the logical flow of a typical analysis or evaluation, and the most common decision points and feedback loops. It clearly shows how the three documents tie together, and recognizes the differences between design of a new bridge and evaluation of an existing bridge.

The HEC-20 block of the flow chart outlines initial data collection and site reconnaissance activities leading to an understanding of the problem, evaluation of river system stability and potential future response. The HEC-20 procedures include both qualitative and quantitative geomorphic and engineering analysis techniques which help establish the level of analysis necessary to solve the stream instability and scour problem for design of a new bridge, or for the evaluation of an existing bridge that may require rehabilitation or countermeasures. The "Classify Stream," "Evaluate Stability," and "Assess Response" portions of the HEC-20 block are expanded in HEC-20 into a six-step Level 1 and an eight-step Level 2 analysis procedure. In some cases, the HEC-20 analysis may be sufficient to determine that stream instability or scour problems do not exist, i.e., the bridge has a "low risk" of failure regarding scour susceptibility.

In most cases, the analysis or evaluation will progress to the HEC-18 block of the flow chart. Here more detailed hydrologic and hydraulic data are developed, with the specific approach determined by the level of complexity of the problem and waterway characteristics (e.g., tidal or riverine). The "Scour Analysis" portion of the HEC-18 block encompasses a seven-step specific design approach which includes evaluation of the components of total scour (see Chapter 3).

Since bridge scour evaluation requires multidisciplinary inputs, it is often advisable for the hydraulic engineer to involve structural and geotechnical engineers at this stage of the analysis. **Once the total scour prism is plotted, then all three disciplines must be involved in a determination of structural stability**.

For a new bridge design, if the structure is stable the design process can proceed to consideration of environmental impacts, cost, constructability, and maintainability. If the structure is unstable, revise the design and repeat the analysis. For an existing bridge, a finding of structural stability at this stage will result in a "low risk" evaluation, with no further action required. However, a Plan of Action should be developed for an unstable existing bridge (scour critical) to correct the problem as discussed in Chapter 12 and HEC-23.[7]

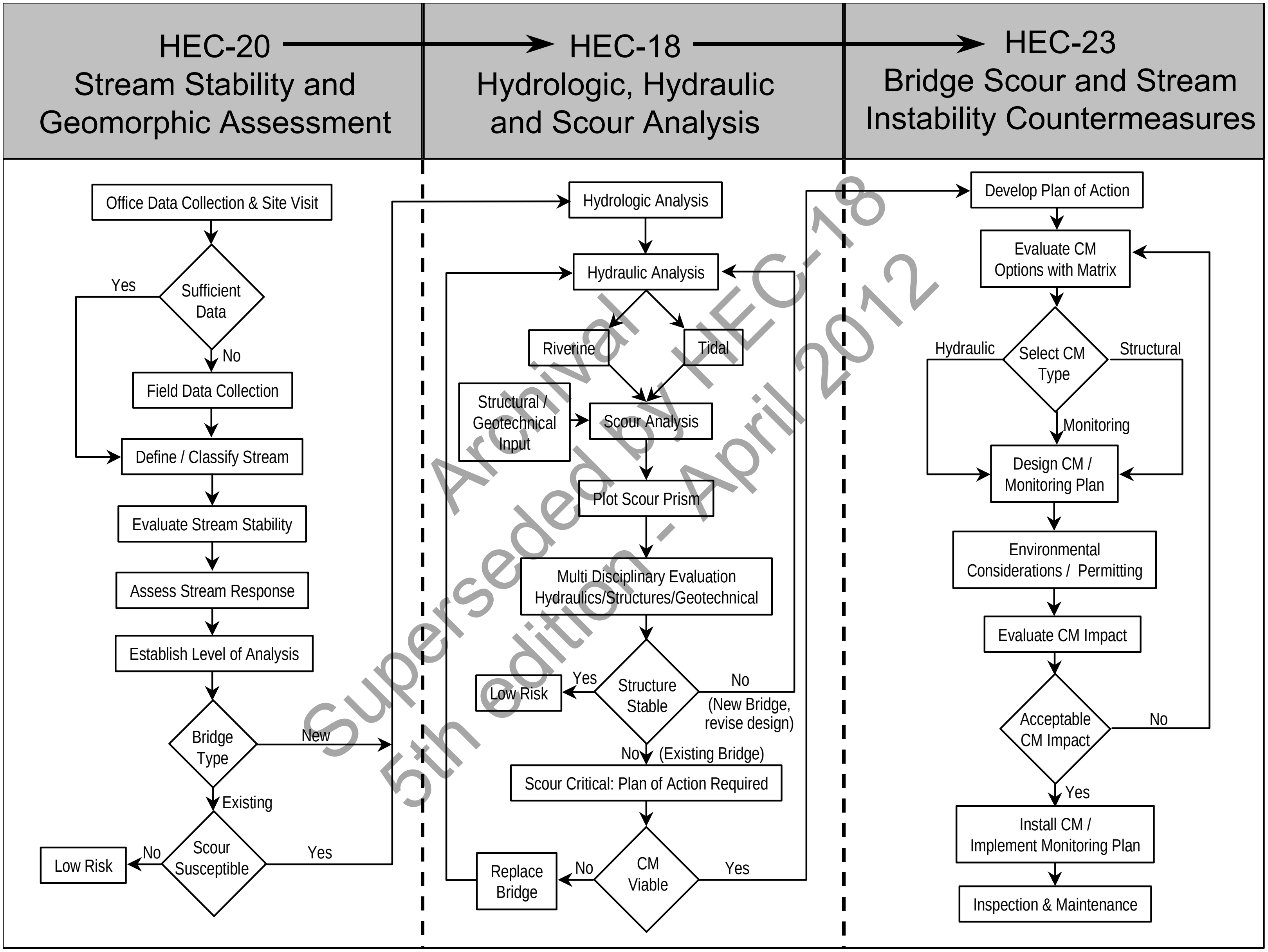

Figure 1.1. Flow chart for scour and stream stability analysis and evaluation.

The scour problem may be so serious that installing countermeasures would not provide a viable solution and a replacement or substantial bridge rehabilitation would be required. If countermeasures would correct the stream instability or scour problem at a reasonable cost and with acceptable environmental impacts, the analysis would progress to the HEC-23 block of the flow chart.

HEC-23 provides a range of resources to support bridge scour or stream instability countermeasure selection and design. A countermeasure matrix in HEC-23 presents a variety of countermeasures that have been used by State departments of transportation (DOTs) to control scour and stream instability at bridges. The matrix is organized to highlight the various groups of countermeasures and identifies distinctive characteristics of each countermeasure. The matrix identifies most countermeasures used and lists information on their functional applicability to a particular problem, their suitability to specific river environments, the general level of maintenance resources required, and which DOTs have experience with specific countermeasures. Finally, a reference source for design guidelines is noted.

HEC-23 includes specific design guidelines for the most common (and some uncommon) countermeasures used by DOTs, or references to sources of design guidance. Inherent in the design of any countermeasure is an evaluation of potential environmental impacts, permitting for countermeasure installation, and redesign, if necessary, to meet environmental requirements. As shown in the flow chart, to be effective most countermeasures will require a monitoring plan, inspection, and maintenance.

1.4 MANUAL ORGANIZATION

The procedures presented in this document contain the state-of-knowledge and practice for dealing with scour at highway bridges.

- Chapter 1 gives the background of the scour problem, a flowchart for a comprehensive analysis using HEC-18, HEC-20, and HEC-23, organization of this manual and improvements needed in the state-of-knowledge of scour.
- Chapter 2 gives recommendations for designing bridges to resist scour.
- Basic concepts and definitions are presented in Chapter 3.
- Methods for estimating long-term aggradation and degradation are given in Chapter 4.
- Chapter 5 provides procedures and equations for determining contraction scour and discusses other general scour conditions.
- Chapter 6 provides equations for calculating and evaluating local scour depths at piers.
- Chapter 7 discusses local scour at abutments and the equations for predicting scour depths at abutments.
- Chapter 8 provides a comprehensive example of scour analysis for a river crossing.
- Chapter 9 provides an introduction to tidal processes and scour analysis methods for bridges over tidal waterways.
- Chapter 10 explains how the National Bridge Scour Evaluation program determines the vulnerability of existing bridges to scour and gives the status of the program.

- Chapter 11 explains how the National Scour Evaluation program relates to the National Bridge Inspection Standards (NBIS). It also presents guidelines for inspecting bridges for scour.

- Chapter 12 explains the need for and details of a Plan of Action to protect a bridge that has been determined to be scour critical.

1.5 OBJECTIVES OF A BRIDGE SCOUR EVALUATION PROGRAM

The need to minimize future flood damage to the nation's bridges requires that additional attention be devoted to developing and implementing improved procedures for designing and inspecting bridges for scour.(8) Approximately 83 percent of the 583,000 bridges in the National Bridge Inventory are built over waterways. Statistically, we can expect hundreds of these bridges to experience floods in the magnitude of a 100-year flood or greater each year. Because it is not economically feasible to construct all bridges to resist all conceivable floods, or to install scour countermeasures at all existing bridges to ensure absolute invulnerability from scour damage, some risks of failure from future floods may have to be accepted. **However, every bridge over water, whether existing or under design, should be assessed as to its vulnerability to floods in order to determine the prudent measures to be taken.** The added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure which can easily be two to ten times the cost of the bridge itself. Moreover, the need to ensure public safety and minimize the adverse effects resulting from bridge closures requires our best efforts to improve the state-of-practice for designing and maintaining bridge foundations to resist the effects of scour. **The hydraulic design of bridge waterways is typically based on flood frequencies somewhat less than those recommended for scour analysis in this publication.**

The procedures presented in this manual serve as guidance for implementing the recommendations contained in the FHWA Technical Advisory T5140.23 entitled, "Evaluating Scour at Bridges."(9) The recommendations have been developed to summarize the essential elements which should be addressed in developing a comprehensive scour evaluation program. A key element of the program is the identification of scour-critical bridges which will be entered into the National Bridge Inventory using the FHWA document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges."(10)

1.6 DUAL SYSTEM OF UNITS

This edition of HEC-18 uses dual units (SI metric and English). The "English" system of units as used throughout this manual refers to U.S. customary units. **In Appendix A, the metric (SI) unit of measurement is explained. The conversion factors, physical properties of water in the SI and English systems of units, sediment particle size grade scale, and some common equivalent hydraulic units are also given**. This edition uses for the unit of length the meter (m) or foot (ft); of mass the kilogram (kg) or slug; of weight/force the newton (N) or pound (lb); of pressure the Pascal (Pa, N/m^2) or (lb/ft^2); and of temperature the degree centigrade (°C) or Fahrenheit (°F). The unit of time is the same in SI as in English system (seconds, s). Sediment particle size is given in millimeters (mm), but in calculations the decimal equivalent of millimeters in meters is used (1 mm = 0.001 m) or for the English system feet (ft). The value of some hydraulic engineering terms used in the text in SI units and their equivalent English units are given in Table 1.1.

Table 1.1. Commonly Used Engineering Terms in SI and English Units.		
Term	SI Units	English Units
Length	1 m	3.28 ft
Volume	1 m^3	35.31 ft^3
Discharge	1 m^3/s	35.31 ft^3/s
Acceleration of Gravity	9.81 m/s^2	32.2 ft/s^2
Unit Weight of Water	9800 N/m^3	62.4 lb/ft^3
Density of Water	1000 kg/m^3	1.94 $slugs/ft^3$
Density of Quartz	2647 kg/m^3	5.14 $slugs/ft^3$
Specific Gravity of Quartz	2.65	2.65
Specific Gravity of Water	1	1
Temperature	°C = 5/9 (°F - 32)	°F

1.7 STATE-OF-KNOWLEDGE AND PRACTICE FOR ESTIMATING SCOUR AT BRIDGES

Some of the problems associated with estimating scour and providing cost-effective and safe designs are being addressed in research and development programs of the FHWA and individual DOTs. The following sections detail the most pressing research needs.

1. **Field Measurements of Scour**. The current equations and methods for estimating scour at bridges are based primarily on laboratory research. Very little field data have been collected to verify the applicability and accuracy of the various design procedures for the range of soil conditions, stream flow conditions, and bridge designs encountered throughout the United States. In particular, DOTs are encouraged to initiate studies for the purpose of obtaining field measurements of scour and related hydraulic conditions at bridges for evaluating, verifying, and improving existing scour prediction methods. In excess of 20 states have initiated cooperative studies with the Water Resources Division of the U.S. Geological Survey (USGS) to collect scour data at existing bridges. A model cooperative agreement with the USGS for purposes of conducting a scour study was included in the FHWA guidance "Interim Procedures for Evaluating Scour at Bridges," which accompanied the September 1988 FHWA Technical Advisory.[11, 9]

2. **Scour Monitoring and Measurement Equipment**. Many bridges in the United States were constructed prior to the development of scour estimation procedures. Some of these bridges have foundations which are vulnerable to scour; however, it is not economically feasible to repair or replace all of these bridges. Therefore, these bridges need to be monitored during floods and closed before they fail. The FHWA, in cooperation with DOTs and the Transportation Research Board, has conducted research to develop scour monitoring and measuring instruments.[12] This research has developed several instruments for scour monitoring and measurement (see Chapter 7, HEC-23).[7] However, there is a need for additional research to develop additional instrumentation and equipment to measure scour for research and to indicate when a bridge is in danger of collapsing due to scour.

3. **Equipment and Methods to Determine Unknown Foundations**. Many of the 575,000 bridges have Unknown foundations. Research sponsored by FHWA, in cooperation with DOTs and the Transportation Research Board has investigated techniques and instruments to identify the type and depth of unknown foundations for most existing

bridges. Additional research is needed to perfect the methods and instruments and to develop alternative methods and equipment (Appendix L).

4. **Hydraulic Variables for Scour Computations**. Advances have been made in developing computational software to establish hydraulic variables for scour computations, including 1- and 2-dimensional, steady and unsteady models. Recent research has provided guidance for applying these models to estimating scour for coastal (tidal) bridges.(13) Most, if not all, of the commonly used scour prediction equations have been incorporated into these models. However, applications methodologies are required to facilitate the use of more appropriate hydraulic variables that can be obtained from more sophisticated computer models. World wide web sites providing hydraulic models applicable to scour computations include:

   - www.fhwa.dot.gov/bridge/hydsoft.htm
   - www.hec.usace.army.mil/software/index.html

5. **Pressure Flow**. Research sponsored by FHWA has developed equations and methods to determine pier and abutment local scour depths when a bridge is submerged (pressure flow).(14) A regression equation for vertical contraction scour is available, but combinations of vertical and lateral contraction scour need to be investigated.

6. **Field and Laboratory Studies of Scour**. Laboratory studies are needed to better understand certain elements of the scour processes and develop alternate and improved scour countermeasures. Only through controlled experiments can the effect of the variables and parameters associated with scour be determined. Through these efforts, scour prediction equations can be improved and additional design methods for countermeasures developed. Results from these laboratory experiments must be verified by ongoing field measurements of scour.

   Laboratory and field research is needed to:

   a. Improve methods to predict scour depths associated with pressure flow,

   b. Improve equations for abutment scour,

   c. Improve methods for estimating scour when abutments are set back from the channel with overbank flow,

   d. Conduct fundamental research on the mechanics of riverine and tidal scour,

   e. Determine methods to predict scour depths when there is ice or debris buildup at a pier or abutment,

   f. Improve our knowledge of the influence of graded, armored, or cohesive bed material on maximum local scour at piers and abutments,

   g. Improve methods for determining the size and placement (elevation, width, and location) of riprap in the scour hole to protect piers and abutments,

   h. Determine the width of scour hole as a function of scour depth and bed material size,

i. Improve our knowledge of the effects of flow depth and velocity on scour depths,

j. Improve our understanding of the bridge scour failure mechanism which would combine the various scour components (pier, abutment, contraction, lateral migration, degradation) into an estimate of the scoured cross section under the bridge,

k. Improve methods to predict the effect of flow angle of attack against a pier or abutment on scour depth,

l. Determine the effect of wide piers and variable pier widths on scour depths,

m. Determine the impact of overlapping scour holes, and

n. Determine scour depths in structures designed as bottomless culverts, that is culverts founded on spread footings and placed on erodible soil.

CHAPTER 2

DESIGNING BRIDGES TO RESIST SCOUR

2.1 DESIGN PHILOSOPHY AND CONCEPTS

Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to the 100-year flood, or a smaller flood if it will cause scour depths deeper than the 100-year flood. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the occurrence of a superflood in order of magnitude of a 500-year flood. This requires careful evaluation of the hydraulic, structural, and geotechnical aspects of bridge foundation design.

Guidance in this chapter is based on the following concepts:

1. The foundation should be designed by an interdisciplinary team of engineers with expertise in hydraulic, geotechnical, and structural design.

2. Hydraulic studies of bridge sites are a necessary part of a bridge design. These studies should address both the sizing of the bridge waterway opening and the design of the foundations to be safe from scour. The scope of the analysis should be commensurate with the importance of the highway and consequences of failure.

3. Consideration must be given to the limitations and gaps in existing knowledge when using currently available formulas for estimating scour. **The designer needs to apply engineering judgment in comparing results obtained from scour computations with available hydrologic and hydraulic data to achieve a reasonable and prudent design.** Such data should include:

   a. Performance of existing structures during past floods

   b. Effects of regulation and control of flood discharges

   c. Hydrologic characteristics and flood history of the stream and similar streams

   d. Whether the bridge is structurally continuous

4. The principles of economic analysis and experience with actual flood damage indicate that it is almost always cost-effective to provide a foundation that will not fail, even from a very large flood event or superflood. Generally, occasional damage to highway approaches from rare floods can be repaired quickly to restore traffic service. On the other hand, a bridge which collapses or suffers major structural damage from scour can create safety hazards to motorists as well as significant social impacts and economic losses over a long period of time. Aside from the costs to the DOTs of replacing or repairing the bridge and constructing and maintaining detours, there can be significant costs to communities or entire regions due to additional detour travel time, inconvenience, and lost business opportunities. Therefore, a higher hydraulic standard is warranted for the design of bridge foundations to resist scour than is usually required for sizing of the bridge waterway. This concept is reflected in the following design procedure.

2.2 GENERAL DESIGN PROCEDURE

The general design procedure for scour outlined in the following steps is recommended for determining bridge type, size, and location (TS&L) of substructure units:

Step 1. Select the flood event(s) that are expected to produce the most severe scour conditions. Experience indicates that this is likely to be the 100-year flood or the overtopping flood when it is less than the 100-year flood. Check the 100-year flood or the overtopping flood (if less than the 100-year flood) and other flood events if there is evidence that such events would create deeper scour than the 100-year or overtopping floods. Overtopping refers to flow over the approach embankment(s), the bridge itself, or both. See Appendix B for a discussion of extreme event combinations.

Step 2. Develop water surface profiles for the flood flows in Step 1, taking care to evaluate the range of potential tailwater conditions downstream of the bridge which could occur during these floods. The FHWA microcomputer software WSPRO, is recommended for this task.(15) The U.S. Army Corps of Engineers (USACE) Hydrologic Engineering Center's River Analysis System (HEC-RAS) can also be used.(16, 17)

Step 3. Using the seven-step Specific Design Approach in Section 2.4, estimate total scour for the worst condition from Steps 1 and 2 above. The resulting scour from the selected flood event should be considered in the design of a foundation. For this condition, minimum geotechnical safety factors commonly accepted by DOTs should be applied. For example, for pile design in friction, a commonly applied factor of safety ranges from two to three, for the 100-year or overtopping flood.

Step 4. Plot the total scour depths obtained in Step 3 on a cross section of the stream channel and floodplain at the bridge site.

Step 5. Evaluate the results obtained in Steps 3 and 4. Are they reasonable, considering the limitations in current scour estimating procedures? The scour depth(s) adopted may differ from the equation value(s) based on engineering judgment.

Step 6. Evaluate the bridge TS&L on the basis of the scour analysis performed in Steps 3 through 5. Modify the TS&L as necessary.

a. Visualize the overall flood flow pattern at the bridge site for the design conditions. Use this mental picture to identify those bridge elements most vulnerable to flood flows and resulting scour.

b. The extent of protection to be provided should be determined by:

- Degree of uncertainty in the scour prediction method

- Potential for and consequences of failure

Step 7. Perform the bridge foundation analysis on the basis that all streambed material in the scour prism above the total scour line (Step 4) has been removed and is not available for bearing or lateral support. All foundations should be designed in accordance with the AASHTO Standard Specifications for Highway Bridges.[5] In the case of a pile foundation, the piling should be designed for additional lateral restraint and column action because of the increase in unsupported pile length after scour. In areas where the local scour is confined to the proximity of the footing, the lateral ground stresses on the pile length which remains embedded may not be significantly reduced from the pre-local scour conditions.

a. Spread Footings On Soil

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration

- Place the bottom of the footing below the total scour line from Step 4

- The top of the footing can act as a local scour arrester

b. Spread Footings On Rock Highly Resistant To Scour

Place the bottom of the footing directly on the cleaned rock surface for massive rock formations (such as granite) that are highly resistant to scour. Small embedments (keying) should be avoided since blasting to achieve keying frequently damages the sub-footing rock structure and makes it more susceptible to scour. If footings on smooth massive rock surfaces require lateral constraint, steel dowels should be drilled and grouted into the rock below the footing level.

c. Spread Footings On Erodible Rock

Weathered or other potentially erodible rock formations need to be carefully assessed for scour. An engineering geologist familiar with the area geology should be consulted to determine if rock or soil or other criteria should be used to calculate the support for the spread footing foundation. The decision should be based on an analysis of intact rock cores, including rock quality designations and local geology, as well as hydraulic data and anticipated structure life. An important consideration may be the existence of a high quality rock formation below a thin weathered zone. For deep deposits of weathered rock, the potential scour depth should be estimated (Steps 4 and 5) and the footing base placed below that depth. Excavation into weathered rock should be made with care. If blasting is required, light, closely spaced charges should be used to minimize overbreak beneath the footing level. Loose rock pieces should be removed and the zone filled with clean concrete. In any event, the final footing should be poured in contact with the sides of the excavation for the full designed footing thickness to minimize water intrusion below footing level. Guidance on scourability of rock formations is given in FHWA memorandum "Scourability of Rock Formations" dated July 19, 1991[18] (see Appendix L).

d. Spread Footings Placed On Tremie Seals And Supported On Soil

- Insure that the top of the footing is below the sum of the long-term degradation, contraction scour, and lateral migration

- Place the bottom of the footing below the total scour line from Step 4

e. <u>For Deep Foundations (Drilled Shaft And Driven Piling) With Footings Or Caps</u>

Placing the top of the footing or pile cap below the streambed a depth equal to the estimated long-term degradation and contraction scour depth will minimize obstruction to flood flows and resulting local scour. Even lower footing elevations may be desirable for pile supported footings when the piles could be damaged by erosion and corrosion from exposure to river or tidal currents. For more discussion on pile and drilled shaft foundations, see the manuals on Design and Construction of Driven Pile Foundations and Drilled Shafts.[19, 20]

f. <u>Stub Abutments on Piling</u>

Stub abutments positioned in the embankment should be founded on piling driven below the elevation of the thalweg including long term degradation and contraction scour in the bridge waterway to assure structural integrity in the event the thalweg shifts and the bed material around the piling scours to the thalweg elevation.

Step 8. Repeat the procedure in Steps 2 through 6 above and calculate the scour for a superflood. It is recommended that this superflood (or check flood) be on the order of a 500-year event. However, flows greater or less than these suggested floods may be appropriate depending upon hydrologic considerations and the consequences associated with damage to the bridge. An overtopping flood less than the 500-year flood may produce the worst-case situation for checking the foundation design. The foundation design determined under Step 7 should be reevaluated for the superflood condition and design modifications made where required.

a. Check to make sure that the bottom of spread footings on soil or weathered rock is below the total scour depth for the superflood.

b. **All foundations should have a minimum factor of safety of 1.0 (ultimate load) under the superflood conditions.** Note that in actual practice, the calculations for step 8 would be performed concurrently with steps 1 through 7 for efficiency of operation.

2.3 DESIGN CONSIDERATIONS

2.3.1 General

1. Raise the bridge superstructure elevation above the general elevation of the approach roadways wherever practicable. This provides for overtopping of approach embankments and relief from the hydraulic forces acting at the bridge. This is particularly important for streams carrying large amounts of debris which could clog the waterway at the bridge.

It is recommended that the elevation of the lower cord of the bridge be increased a minimum of 0.9 m (3 ft) above the normal freeboard for the 100-year flood for streams that carry a large amount of debris.

2. Superstructures should be securely anchored to the substructure if buoyant, or if debris and ice forces are probable. Further, the superstructure should be shallow and open to minimize resistance to the flow where overtopping is likely.

3. Continuous span bridges withstand forces due to scour and resultant foundation movement better than simple span bridges. Continuous spans provide alternate load paths (redundancy) for unbalanced forces caused by settlement and/or rotation of the foundations. This type of structural design is recommended for bridges where there is a significant scour potential.

4. Local scour holes at piers and abutments may overlap one another in some instances. If local scour holes do overlap, the scour is indeterminate and may be deeper. The topwidth of a local scour hole on each side of the pier ranges from 1.0 to 2.8 times the depth of local scour. A topwidth value of 2.0 times the depth of local scour on each side of a pier is suggested for practical applications.

5. For pile and drilled shaft supported substructures subjected to scour, a reevaluation of the foundation design may require a change in the pile or shaft length, number, cross-sectional dimension and type based on the loading and performance requirements and site-specific conditions.

6. At some bridge sites, hydraulics and traffic conditions may necessitate consideration of a bridge that will be partially or even totally inundated during high flows. This consideration results in pressure flow through the bridge waterway. Chapter 6 has a discussion on pressure flow scour for these cases.

2.3.2 Piers

1. Pier foundations on floodplains should be designed to the same elevation as pier foundations in the stream channel if there is a likelihood that the channel will shift its location over the life of the bridge.

2. Align piers with the direction of flood flows. Assess the hydraulic advantages of round piers, particularly where there are complex flow patterns during flood events.

3. Streamline piers to decrease scour and minimize potential for buildup of ice and debris. Use ice and debris deflectors where appropriate.

4. Evaluate the hazards of ice and debris buildup when considering use of multiple pile bents in stream channels. Where ice and debris buildup is a problem, consider the bent a solid pier for purposes of estimating scour. Consider use of other pier types where clogging of the waterway area could be a major problem.

5. Scour analyses of piers near abutments need to consider the potential of larger velocities and skew angles from the flow coming around the abutment.

2.3.3 Abutments

1. The equations used to estimate the magnitude of abutment scour were developed in a laboratory under ideal conditions and for the most part lack field verification. Because conditions in the field are different from those in the laboratory, these equations tend to over predict the magnitude of scour that may be expected to develop. Recognizing this, it is recommended that the abutment scour equations be used to develop insight as to the scour potential at an abutment. Engineering judgment must be used to determine if the abutment foundation should be designed to resist the computed local scour. As an alternate, abutment foundations should be designed for the estimated long-term degradation and contraction scour. Riprap and/or guide banks should be used to protect the abutment for this alternative. In summary, riprap or some other protection should always be used to protect the abutment from erosion. Proper design techniques and placement procedures for rock riprap and guide banks are discussed in HEC-23.(7)

2. Relief bridges, guide banks, and river training works should be used, where needed, to minimize the effects of adverse flow conditions at abutments.

3. Where ice build-up is likely to be a problem, set the toe of spill-through slopes or vertical abutments back from the edge of the channel bank to facilitate passage of the ice.

4. Wherever possible, use spill-through (sloping) abutments. Scour at spill-through abutments is about 50 percent of that of vertical wall abutments.

5. Riprap or a guide bank 15 m (50 ft) or longer, or other bank protection methods should be used on the downstream side of an abutment and approach embankment to protect them from erosion by the wake vortex.

2.3.4 Superstructures

The design of the superstructure has a significant impact on the scour of the foundations. Hydraulic forces that should be considered in the design of a bridge superstructure include buoyancy, drag, and impact from ice and floating debris. The configuration of the superstructure should be influenced by the highway profile, the probability of submergence, expected problems with ice and debris, and flow velocities, as well as the usual economic, structural and geometric considerations. Superstructures over waterways should provide structural redundancy, such as continuous spans (rather than simple spans).

<u>Buoyancy</u>. The weight of a submerged or partially submerged bridge superstructure is the weight of the superstructure less the weight of the volume of water displaced. The volume of water displaced may be much greater than the volume of the superstructure components if air is trapped between girders. Also, solid parapet rails and curbs on the bridge deck can increase the volume of water displaced and increase buoyant forces. The volume of air trapped under the superstructure can be reduced by providing holes (vents) through the deck between structural members. Superstructures should be anchored to piers to counter buoyant forces and to resist drag forces. Continuous span designs are also less susceptible to failure from buoyancy than simple span designs.

<u>Drag Forces</u>. Drag forces on a submerged or partially submerged superstructure can be calculated by Equation 2.1:

$$F_d = C_d \rho H \frac{V^2}{2} \tag{2.1}$$

where:

F_d = Drag force per unit of length of bridge, N/m (lb/ft)
C_d = Coefficient of drag (2.0 to 2.2)
ρ = Density of water, 1000 kg/m^3 (1.94 slugs/ft^3)
H = Depth of submergence, m (ft)
V = Velocity of flow, m/s (ft/s)

Floating Debris and Ice. Where bridges are destroyed by debris and ice, it usually is due to accumulations against bridge components. Waterways may be partially or totally blocked by ice and debris, creating hydraulic conditions that cause or increase scour at pier foundations and bridge abutments, structural damage from impact and uplift, and overtopping of roadways and bridges. Floating debris is a common hydraulic problem at highway stream crossings nation-wide. Debris hazards occur more frequently in unstable streams where bank erosion is active and in streams with mild to moderate slopes, as contrasted with headwater streams. Debris hazards are often associated with large floods, and most debris is derived locally along the streambanks upstream from the bridge. After being mobilized, debris typically moves as individual logs which tend to concentrate in the thalweg of the stream. It is possible to evaluate the abundance of debris upstream of a bridge crossing and then to implement mitigation measures, such as removal and or containment, to minimize potential problems during a major flood (see additional discussion in HEC-20, Chapter 4).[6]

Ice Forces. Superstructures may be subjected to impact forces from floating ice, static pressure from thermal movements or ice jams, or uplift from adhering ice in water of fluctuating levels. The latter is usually associated with relatively large bodies of water. Superstructures in these locations should normally be high enough to be unaffected. Research is needed to define the static and dynamic loads that can be expected from ice under various conditions of ice strength and streamflow.

In addition to forces imposed on bridge superstructures by ice loads, ice jams at bridges can cause exaggerated backwater and a sluicing action under the ice. There are numerous examples of foundation scour from this orifice flow under ice as well as superstructure damage and failure from ice forces. Accumulations of ice or drift may substantially increase local pier and abutment scour especially if they are allowed to extend down to near the channel bed. Ice also has serious effects on bank stability. For example, ice may form in bank stabilization materials, and large quantities of rock and other material embedded in the ice may be floated downstream and dumped randomly when the ice breaks up. Banks are subjected to piping forces during the drawdown of water surface elevation after the breakup.

Debris Forces. Information regarding methods for computing forces imposed on bridge superstructures by floating debris is also lacking despite the fact that debris causes or contributes to many failures. Floating debris may consist of logs, trees, house trailers, automobiles, storage tanks, lumber, houses, and many other items representative of floodplain usage. This complicates the task of computing impact forces since the mass and the resistance to crushing of the debris contribute to the impact force.

A general equation for computing impact forces is:

$$F = M dv / dt = \frac{MV^2}{2S} \tag{2.2}$$

where:

F	=	Impact imparted by the debris, N (lb)
M	=	Mass of the debris, kg (slugs)
S	=	Stopping distance, m (ft)
V	=	Velocity of the floating debris prior to impact, m/s (ft/s)

In addition to impact forces, a buildup of debris increases the effective depth of the superstructure and the drag coefficient may also be increased. Perhaps the most hazardous result of debris buildup is partial or total clogging of the waterway. This can result in a sluicing action of flow under the debris which can result in scour and foundation failure or a shift in the channel location from under the bridge.

2.4 SPECIFIC DESIGN APPROACH

The seven specific steps recommended for estimating scour at bridges are:

Step 1: Determine scour analysis variables

Step 2: Analyze long-term bed elevation change

Step 3: Compute the magnitude of contraction scour

Step 4: Compute other general scour depths.

Step 5: Compute the magnitude of local scour at piers

Step 6: Determine abutment foundation type, protection and elevation. Computation of local scour depths may be used to aid in this determination.

Step 7: Plot and evaluate the total scour depths as outlined in Steps 4 through 6 of the General Design Procedure in Section 2.2.

The engineer should evaluate how reasonable the individual estimates of general scour (contraction and other) and local scour depths are in Steps 3, 4, and 5 and evaluate the reasonableness of the total scour in Step 7. The results from this Specific Design Approach complete Steps 1 through 6 of Section 2.2. The design must now proceed to Steps 7 and 8 of the General Design Procedure in Section 2.2.

The procedures for each of the steps are discussed in the following sections with reference to specific chapters where detailed procedures and equations are given.

2.5 DETAILED PROCEDURES

2.5.1 Step 1: Determine Scour Analysis Variables

1. Determine the magnitude of the discharges for the floods in Steps 1 and 8 of the General Design Procedure in Section 2.2, including the overtopping flood when applicable. For guidance for a particular state in determining the magnitude of the 500-year flood, contact with the U.S. Geological Survey Water Resources District office is suggested. Experience has shown that the incipient overtopping discharge often puts the most stress on a bridge. However, special conditions (angle of attack, pressure flow, decrease in velocity or discharge resulting from high flows overtopping approaches or going through relief bridges, ice jams, etc.) may cause a more severe condition for scour with a flow smaller than the overtopping or 100-year flood.

2. Determine if there are existing or potential future factors that will produce a combination of high discharge and low tailwater control. Are there bedrock or other controls (old diversion structures, erosion control checks, other bridges, etc.) that might be lowered or removed? Are there dams or locks downstream that would control the tailwater elevation seasonally? Are there dams upstream or downstream that could control the elevation of the water surface at the bridge? Select the lowest reasonable downstream water-surface elevation and the largest discharge to estimate the greatest scour potential. Assess the distribution of the velocity and discharge per foot of width for the design flow and other flows through the bridge opening. Also, consider the contraction and expansion of the flow in the bridge waterway, as well as present conditions and anticipated future changes in the river.

3. Determine the water-surface profiles for the discharges judged to produce the most scour from step 1, using WSPRO, or HEC River Analysis System (HEC-RAS).[15, 16, 17] In some instances, the designer may wish to use BRI-STARS.[21] Hydraulic studies by the USACE, USGS, the Federal Emergency Management Agency (FEMA), etc. are potentially useful sources of hydraulic data to calibrate, verify, and evaluate results from WSPRO or HEC-RAS. The engineer should anticipate future conditions at the bridge, in the upstream watershed, and at downstream water-surface elevation controls as outlined in HEC-20.[6] From computer analysis and from other hydraulic studies, determine input variables such as the discharge, velocity and depth needed for the scour calculations.

4. Collect and summarize the following information as appropriate (see HEC-20 for a step-wise analysis procedure[6]).

   a. Boring logs to define geologic substrata at the bridge site

   b. Bed material size, gradation, and distribution in the bridge reach

   c. Existing stream and floodplain cross section through the reach

   d. Stream planform

   e. Watershed characteristics

   f. Scour data on other bridges in the area

g. Slope of energy grade line upstream and downstream of the bridge

h. History of flooding

i. Location of bridge site with respect to other bridges in the area, confluence with tributaries close to the site, bed rock controls, man-made controls (dams, old check structures, river training works, etc.), and confluence with another stream downstream

j. Character of the stream (perennial, flashy, intermittent, gradual peaks, etc.)

k. Geomorphology of the site (floodplain stream; crossing of a delta, youthful, mature or old age stream; crossing of an alluvial fan; meandering, straight or braided stream; etc.) (see HEC-20 and HDS 6)[6, 22]

l. Erosion history of the stream

m. Development history (consider present and future conditions) of the stream and watershed, collect maps, ground photographs, aerial photographs; interview local residents; check for water resource projects planned or contemplated

n. Sand and gravel mining from the streambed or floodplain up- and downstream from site

o. Other factors that could affect the bridge

p. Make a qualitative evaluation of the site with an estimate of the potential for stream movement and its effect on the bridge

2.5.2 Step 2: Analysis of Long-Term Bed Elevation Change

Using the information collected in Step 1, above, and procedures in HEC-20[6] and Chapter 4, determine the long-term trend in the streambed elevation.

2.5.3 Step 3: Compute the Magnitude of Contraction Scour

Using the information collected in Step 1, above, compute the magnitude of the contraction scour using the equations and procedures in Chapter 5.

2.5.4 Step 4: Determine the Magnitude of Other General Scour Components

Using the information collected in Step 1, above, determine the magnitude of other general scour components, if any, using the procedures discussed in Chapter 5.

2.5.5 Step 5: Compute the Magnitude of Local Scour at Piers

Using the information collected in Step 1, above, compute the magnitude of local pier scour using the equations and procedures in Chapter 6.

2.5.6 Step 6: Determine the Foundation Elevation for Abutments

Using the information collected in Step 1, above, compute the magnitude of abutment scour using the information and procedures in Chapter 7.

2.5.7 Step 7: Plot the Total Scour Depths and Evaluate the Design

Plot the Total Scour Depths. On the cross section of the stream channel or other general floodplain at the bridge crossing, plot the estimate of long-term bed elevation change, contraction scour, and local scour at the piers and abutments. Use a distorted scale so that the scour determinations will be easy to evaluate. Make a sketch of any planform changes (lateral stream channel movement due to meander migration, etc.) that might be reasonably expected to occur.

1. Long-term elevation changes may be either aggradation or degradation. However, only degradation is considered in scour computations.

2. Contraction or other general scour is then plotted from and below the long-term degradation line.

3. Local scour is then plotted from and below the contraction scour line.

4. Plot not only the depth of scour at each pier and abutment, but also the scour hole width. Use 2.0 times the depth of local scour, y_s, to estimate scour hole width on each side of the pier.

Evaluate the Total Scour Depths.

1. Evaluate whether the computed scour depths are reasonable and consistent with the design engineer's previous experience, and engineering judgment. If not, carefully review the calculations and design assumption in order to modify the depths. These modifications must reflect sound engineering judgment.

2. Evaluate whether the local scour holes from the piers or abutments overlap between spans. If so, local scour depths can be larger though indeterminate. For new or replacement bridges, the length of the bridge opening should be reevaluated and the opening increased or the number of piers decreased as necessary to avoid overlapping scour holes.

3. Evaluate other factors such as lateral movement of the stream, stream flow hydrograph, velocity and discharge distribution, movement of the thalweg, shifting of the flow direction, channel changes, type of stream, or other factors.

4. Evaluate whether the calculated scour depths appear too deep for the conditions in the field, relative to the laboratory conditions. **Abutment scour equations are for the worst-case conditions.** Rock riprap and/or a guide bank could be a more cost-effective solution than designing the abutment to resist the computed abutment scour depths.

   If the calculated scour depths appear too deep, consider recalculating the hydraulic variables after long-term degradation and/or contraction scour are accounted for. This may decrease the total scour depth.

5. Evaluate cost, safety, etc. Also, account for ice and/or debris effects.

6. In the design of bridge foundations, the bottom foundation elevation(s) should be at or below the total scour elevation(s) as discussed in Section 2.2.

<u>Reevaluate the Bridge Design</u>. Reevaluate the bridge design on the basis of the foregoing scour computations and evaluation. Revise the design as necessary. This evaluation should consider the following questions:

1. Is the waterway area large enough (e.g., is contraction scour too large)?

2. Are the piers too close to each other or to the abutments (i.e., do the scour holes overlap)? Estimate the topwidth of a scour hole on each side of a pier at 2.0 times the depth of scour. If scour holes overlap, local scour can be deeper.

3. Is there a need for relief bridges? Should they or the main bridge be larger?

4. Are bridge abutments properly aligned with the flow and located properly in regard to the stream channel and floodplain?

5. Is the bridge crossing of the stream and floodplain in a desirable location? If the location presents problems:

   a. Can it be changed?

   b. Can river training works, guide banks, abutment setback from the channel, or relief bridges serve to provide for an acceptable flow pattern at the bridge?

6. Is the hydraulic study adequate to provide the necessary information for foundation design?

   a. Are flow patterns complex?
   b. Should a 2-dimensional, water-surface profile model be used for analysis?
   c. Is the foundation design safe and cost-effective?
   d. Is a physical model study needed/warranted?

CHAPTER 3

BASIC CONCEPTS AND DEFINITIONS OF SCOUR

3.1 GENERAL

Scour is the result of the erosive action of flowing water, excavating and carrying away material from the bed and banks of streams and from around the piers and abutments of bridges. Different materials scour at different rates. Loose granular soils are rapidly eroded by flowing water, while cohesive or cemented soils are more scour-resistant. **However, ultimate scour in cohesive or cemented soils can be as deep as scour in sand-bed streams.** Under constant flow conditions, scour will reach maximum depth in sand- and gravel-bed material in hours; cohesive bed material in days; glacial till, sandstones, and shale in months; limestone in years, and dense granite in centuries. Under flow conditions typical of actual bridge crossings, several floods may be needed to attain maximum scour.

Determining the magnitude of scour is complicated by the cyclic nature of the scour process. Scour can be deepest near the peak of a flood, but hardly visible as floodwaters recede and scour holes refill with sediment.

Designers and inspectors need to carefully study site-specific subsurface information in evaluating scour potential at bridges, giving particular attention to foundations on rock. Massive rock formations with few discontinuities are highly resistant to scour during the lifetime of a typical bridge.

All of the equations for estimating contraction and local scour are based on laboratory experiments with limited field verification. However, contraction and local scour depths at piers as deep as computed by these equations have been observed in the field. The equations recommended in this document are considered to be the most applicable for estimating scour depths.

A factor in scour at highway crossings and encroachments is whether it is **clear-water** or **live-bed** scour. Clear-water scour occurs where there is no transport of bed material upstream of the crossing or encroachment or the material being transported from the upstream reach is transported through the downstream reach at less than the capacity of the flow. Live-bed scour occurs where there is transport of bed material from the upstream reach into the crossing or encroachment. This subject is discussed further in Section 3.4.

This document presents procedures, equations, and methods to analyze scour in both riverine and coastal areas. In riverine environments, scour results from flow in one direction (downstream). In coastal areas, highways that cross waterways and/or encroach longitudinally on them are subject to tidal fluctuation and scour may result from flow in two directions. In waterways influenced by tidal fluctuations, flow velocities do not necessarily decrease as scour occurs and the waterway area increases. In tidal waterways as waterway area increases, the discharge may increase. This is in sharp contrast to riverine waterways where the principle of flow continuity and a constant discharge requires that velocity be inversely proportional to the waterway area. **However, the methods and equations for determining stream instability, scour and associated countermeasures can be applied to both riverine and coastal streams**.[23,24] The difficulty in tidal streams is in determining the hydraulic parameters (such as discharge, velocity, and depth) that are to be used in the scour equations. Tidal scour is discussed in Chapter 9.

3.2 TOTAL SCOUR

Total scour at a highway crossing is comprised of three components:

1. Long-term aggradation and degradation of the river bed

2. General scour at the bridge

   a. Contraction scour
   b. Other general scour

3. Local scour at the piers or abutments

These three scour components are added to obtain the total scour at a pier or abutment. This assumes that each component occurs independent of the other. Considering the components additive adds some conservatism to the design. In addition, **lateral migration** of the stream must be assessed when evaluating total scour at bridge piers and abutments.

3.2.1 Aggradation and Degradation

Aggradation and degradation are long-term streambed elevation changes due to natural or man-induced causes which can affect the reach of the river on which the bridge is located. 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.

3.2.2 General Scour

General scour is a lowering of the streambed across the stream or waterway bed at the bridge. This lowering may be uniform across the bed or non-uniform, that is, the depth of scour may be deeper in some parts of the cross section. General scour may result from contraction of the flow, which results in removal of material from the bed across all or most of the channel width, or from other general scour conditions such as flow around a bend where the scour may be concentrated near the outside of the bend. General scour is different from long-term degradation in that general scour may be cyclic and/or related to the passing of a flood.

3.2.3 Local Scour

Local scour involves removal of material from around piers, abutments, spurs, and embankments. It is caused by an acceleration of flow and resulting vortices induced by obstructions to the flow. Local scour can be either clear-water or live-bed scour.

3.2.4 Lateral Stream Migration

In addition to the types of scour mentioned above, naturally occurring lateral migration of the main channel of a stream within a floodplain may affect the stability of piers in a floodplain, erode abutments or the approach roadway, or change the total scour by changing the flow angle of attack at piers and abutments. Factors that affect lateral stream movement also affect the stability of a bridge foundation. These factors are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, and the characteristics of the bed and bank materials (see HEC-20, and HDS 6).[6, 22]

The following sections provide a more detailed discussion of the various components of total scour.

3.3 LONG-TERM STREAMBED ELEVATION CHANGES (AGGRADATION AND DEGRADATION)

Long-term bed elevation changes may be the natural trend of the stream or the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity of the bridge crossing. Long-term aggradation and degradation do not include the cutting and filling of the streambed in the vicinity of the bridge that might occur during a runoff event (general and local scour). A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the long-term streambed changes must be estimated. Methods to estimate long-term streambed elevation changes are discussed in Chapter 4.

3.4 CLEAR-WATER AND LIVE-BED SCOUR

There are two conditions for contraction and local scour: **clear-water** and **live-bed** scour. Clear-water scour occurs when there is no movement of the bed material in the flow upstream of the crossing or the bed material being transported in the upstream reach is transported in suspension through the scour hole at the pier or abutment at less than the capacity of the flow. At the pier or abutment the acceleration of the flow and vortices created by these obstructions cause the bed material around them to move. Live-bed scour occurs when there is transport of bed material from the upstream reach into the crossing. Live-bed local scour is cyclic in nature; that is, the scour hole that develops during the rising stage of a flood refills during the falling stage.

Typical clear-water scour situations include (1) coarse-bed material streams, (2) flat gradient streams during low flow, (3) local deposits of larger bed materials that are larger than the biggest fraction being transported by the flow (rock riprap is a special case of this situation), (4) armored streambeds where the only locations that tractive forces are adequate to penetrate the armor layer are at piers and/or abutments, and (5) vegetated channels or overbank areas.

During a flood event, bridges over streams with coarse-bed material are often subjected to clear-water scour at low discharges, live-bed scour at the higher discharges and then clear-water scour at the lower discharges on the falling stages. Clear-water scour reaches its maximum over a longer period of time than live-bed scour (Figure 3.1). This is because clear-water scour occurs mainly in coarse-bed material streams. In fact, local clear-water scour may not reach a maximum until after several floods. Maximum local clear-water pier scour is about 10 percent greater than the equilibrium local live-bed pier scour.

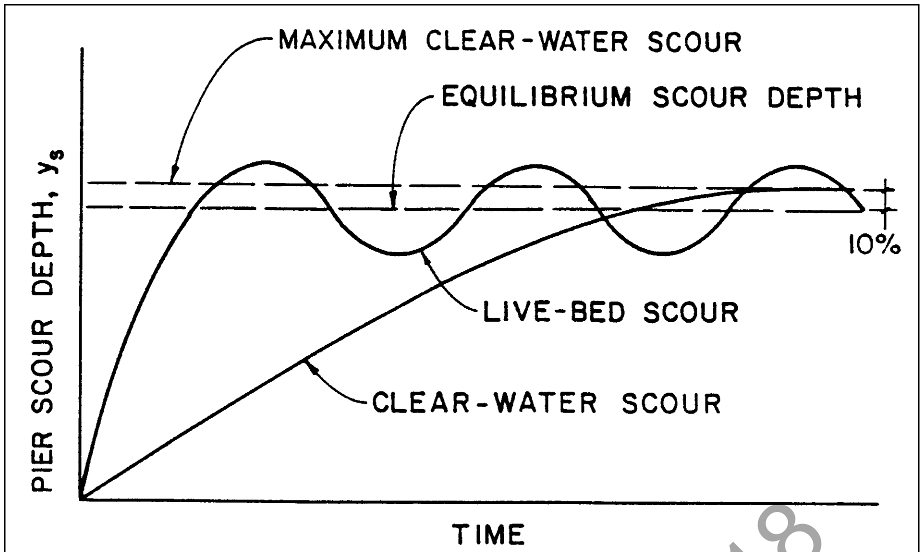

Figure 3.1. Pier scour depth in a sand-bed stream as a function of time.

Critical velocity equations with the reference particle size (D) equal to D_{50} are used to determine the velocity associated with the initiation of motion. They are used as an indicator for clear-water or live-bed scour conditions. If the mean velocity (V) in the upstream reach is equal to or less than the critical velocity (V_c) of the median diameter (D_{50}) of the bed material, then contraction and local scour will be clear-water scour. Also, if the ratio of the shear velocity of the flow to the fall velocity of the D_{50} of the bed material (V_*/ω) is greater than 2, contraction and local scour may be clear-water. If the mean velocity is greater than the critical velocity of the median bed material size, live-bed scour will occur. An equation to determine the critical velocity for a given flow depth and size of bed material is derived in Appendix C and given in Chapter 5.

This technique can be applied to any unvegetated channel or overbank area to determine whether scour is clear-water or live-bed. This procedure should be used with caution for assessing whether or not scour in the overbank will be clear-water or live-bed. For most cases, the presence of vegetation on the overbank will effectively bind and protect the overbank from erosive velocities. Also, in the overbank, generally the velocities are small and the bed material so fine that most overbank areas will experience clear-water scour.

Live-bed pier scour in sand-bed streams with a dune bed configuration fluctuates about the equilibrium scour depth (Figure 3.1). This is due to the variability of the bed material sediment transport in the approach flow when the bed configuration of the stream is dunes. In this case (dune bed configuration in the channel upstream and through the bridge), maximum depth of pier scour is about 30 percent larger than equilibrium depth of scour. However, with the exception of crossings over large rivers (i.e., the Mississippi, Columbia, etc.), the bed configuration in sand-bed streams will plane out during flood flows due to the increase in velocity and shear stress. For general practice, the maximum depth of pier scour is approximately 10 percent greater than equilibrium scour.

For a discussion of bedforms in alluvial channel flow, see Chapter 3 of HDS 6.[22] Equations for estimating local scour at piers or abutments are given in Chapters 6 and 7 of this document. These equations were developed from laboratory experiments and limited field data for both clear-water and live-bed scour.

3.5 GENERAL SCOUR

3.5.1 Contraction Scour

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction of the stream channel or by a bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach. Contraction scour, in a natural channel or at a bridge crossing, involves removal of material from the bed across all or most of the channel width. Methods to estimate live-bed and clear-water contraction scour are presented in Chapter 5.

In coastal waterways which are affected by tides, as the cross-sectional area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets contraction scour may result in a continual lowering of the bed (long-term degradation).

Live-bed contraction scour is typically cyclic; for example, the bed scours during the rising stage of a runoff event and fills on the falling stage. The cyclic nature of contraction scour causes difficulties in determining contraction scour depths after a flood. The contraction of flow at a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clear-water picks up additional sediment from the bed upon reaching the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment.

Other factors that can cause contraction scour are (1) natural stream constrictions, (2) long highway approaches to the bridge over the floodplain, (3) ice formations or jams, (4) natural berms along the banks due to sediment deposits, (5) debris, (6) vegetative growth in the channel or floodplain, and (7) pressure flow.

3.5.2 Other General Scour

Other general scour conditions can result from erosion related to the planform characteristics of the stream (meandering, braided or straight), variable downstream control, flow around a bend, or other changes that decrease the bed elevation. General scour conditions can occur at bridges located upstream or downstream of a confluence. These scour conditions are discussed in Section 5.8 and HDS 6.[22]

3.6 LOCAL SCOUR

The basic mechanism causing local scour at piers or abutments is the formation of vortices (known as the horseshoe vortex) at their base (Figure 3.2). The horseshoe vortex results from the pileup of water on the upstream surface of the obstruction and subsequent acceleration of the flow around the nose of the pier or abutment. The action of the vortex removes bed material from around the base of the obstruction. The transport rate of sediment away from the base region is greater than the transport rate into the region, and, consequently, a scour hole develops. As the depth of scour increases, the strength of the horseshoe vortex is reduced, thereby reducing the transport rate from the base region. Eventually, for live-bed local scour, equilibrium is reestablished between bed material inflow and outflow and scouring ceases. For clear-water scour, scouring ceases when the shear stress caused by the horseshoe vortex equals the critical shear stress of the sediment particles at the bottom of the scour hole.

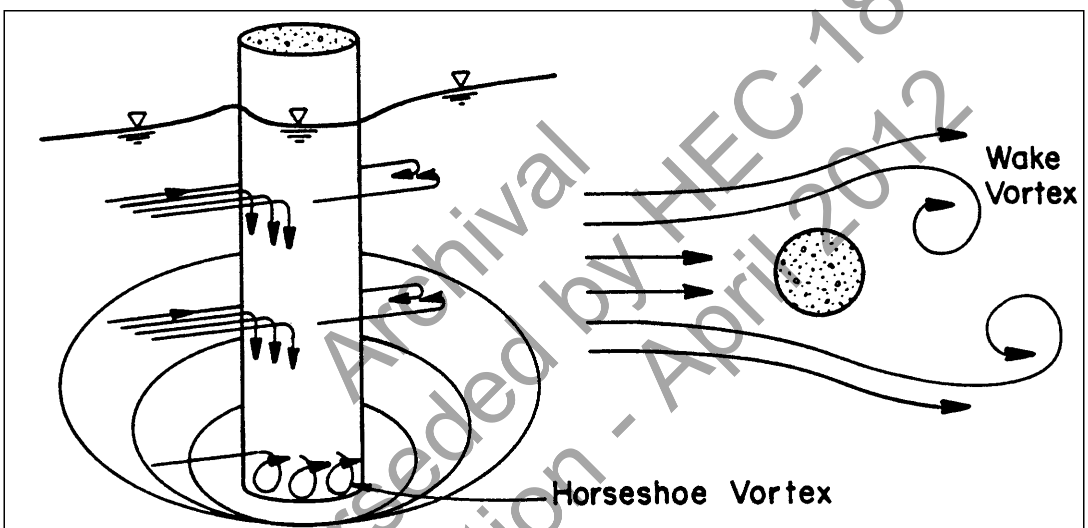

Figure 3.2. Schematic representation of scour at a cylindrical pier.

In addition to the horseshoe vortex around the base of a pier, there are vertical vortices downstream of the pier called the wake vortex (Figure 3.2). Both the horseshoe and wake vortices remove material from the pier base region. However, the intensity of wake vortices diminishes rapidly as the distance downstream of the pier increases. Therefore, immediately downstream of a long pier there is often deposition of material.

Factors which affect the magnitude of local scour depth at piers and abutments are (1) velocity of the approach flow, (2) depth of flow, (3) width of the pier, (4) discharge intercepted by the abutment and returned to the main channel at the abutment (in laboratory flumes this discharge is a function of projected length of an abutment into the flow), (5) length of the pier if skewed to flow, (6) size and gradation of bed material, (7) angle of attack of the approach flow to a pier or abutment, (8) shape of a pier or abutment, (9) bed configuration, and (10) ice formation or jams and debris.

1. Flow velocity affects local scour depth. The greater the velocity, the deeper the scour. There is a high probability that scour is affected by whether the flow is subcritical or supercritical. However, most research and data are for subcritical flow (i.e., flow with a Froude Number less than 1.0, Fr < 1).

2. Flow depth also has an influence on the depth of local scour. An increase in flow depth can increase scour depth by a factor of 2 or greater for piers. With abutments, the increase is approximately 1.1 to 2.15 depending on the shape of the abutment.

3. Pier width has a direct influence on depth of local scour. As pier width increases, there is an increase in scour depth. There is a limit to the increase in scour depth as width increases. Very wide piers (see Section 6.3) do not have scour depths as deep as predicted by existing equations.

4. In laboratory flume studies, an increase in the projected length of an abutment (or embankment) into the flow increased scour; whereas, this is not the case in the field. Due to the relatively small scale of a laboratory flume, floodplain flow intercepted by the embankment and returned to the main channel is directly related to the length of the obstruction. However, in the field case the embankment length is not a good measure of the discharge returned to the main channel. This results in "ineffective flow" on the floodplain which can be even more pronounced on wide heavily vegetated floodplains. In order to properly apply laboratory derived abutment scour equations to the field case, an assessment must be made of the location of the boundary between "live flow" and "ineffective flow." The location of this boundary should then be used to establish the length of the abutment or embankment for abutment scour computations (see Section 7.2).

5. Pier length has no appreciable effect on local scour depth as long as the pier is aligned with the flow. When the pier is skewed to the flow, the pier length has a significant influence on scour depth. For example, doubling the length of the pier increases scour depth from 30 to 60 percent (depending on the angle of attack).

6. Bed material characteristics such as size, gradation, and cohesion can affect local scour. Bed material in the sand-size range has little effect on local scour depth. Likewise, larger size bed material that can be moved by the flow or by the vortices and turbulence created by the pier or abutment will not affect the maximum scour, but only the time it takes to attain it. Very large particles in the bed material, such as coarse gravels, cobbles or boulders, may armor the scour hole. Research at the University of Aukland, New Zealand, by the Washington State DOT, and by other researchers developed equations that take into account the decrease in scour due to the armoring of the scour hole.[25, 26, 27, 28] Richardson and Richardson combined these equations into a simplified equation, which accounted for bed material size.[29] However, field data are inadequate to support these equations at this time.

   Molinas in flume experiments sponsored by FHWA, showed for Froude Numbers less than 1.0 (Fr < 1.0), and a range of bed material sizes, that when the approach velocity (V_1) of the flow is less than the critical velocity (V_c) of the D_{90} size of the bed material, the D_{90} size will decrease the scour depth.[30]

   The size of the bed material also determines whether the scour at a pier or abutment is clear-water or live-bed scour. This topic is discussed in Section 3.4.

Fine bed material (silts and clays) will have scour depths as deep as sand-bed streams. This is true even if bonded together by cohesion. The effect of cohesion is to influence the time it takes to reach maximum scour. With sand-bed material the time to reach maximum depth of scour is measured in hours and can result from a single flood event. With cohesive bed materials it may take much longer to reach the maximum scour depth, the result of many flood events. Scour in cohesive bed material is discussed in Section 12.9 and Appendix L

7. Angle of attack of the flow to the pier or abutment has a significant effect on local scour, as was pointed out in the discussion of pier length. Abutment scour is reduced when embankments are angled downstream and increased when embankments are angled upstream. According to the work of Ahmad, the maximum depth of scour at an embankment inclined 45 degrees downstream is reduced by 20 percent; whereas, the maximum scour at an embankment inclined 45 degrees upstream is increased about 10 percent.[31]

8. Shape of the nose of a pier or an abutment can have up to a 20 percent influence on scour depth. Streamlining the front end of a pier reduces the strength of the horseshoe vortex, thereby reducing scour depth. Streamlining the downstream end of piers reduces the strength of the wake vortices. A square-nose pier will have maximum scour depths about 20 percent greater than a sharp-nose pier and 10 percent greater than either a cylindrical or round-nose pier. The shape effect is negligible for flow angles in excess of five degrees. Full retaining abutments with vertical walls on the stream side (parallel to the flow) and vertical walls parallel to the roadway will produce scour depths about double that of spill-through (sloping) abutments.

9. Bed configuration of sand-bed channels affects the magnitude of local scour. In streams with sand-bed material, the shape of the bed (bed configuration) as described by Richardson et al. may be ripples, dunes, plane bed, or antidunes.[32] The bed configuration depends on the size distribution of the sand-bed material, hydraulic characteristics, and fluid viscosity. The bed configuration may change from dunes to plane bed or antidunes during an increase in flow for a single flood event. It may change back with a decrease in flow. The bed configuration may also change with a change in water temperature or suspended sediment concentration of silts and clays. The type of bed configuration and change in bed configuration will affect flow velocity, sediment transport, and scour. HDS 6 discusses bed configuration in detail.[22]

10. Potentially, ice and debris can increase the width of the piers, change the shape of piers and abutments, increase the projected length of an abutment, and cause the flow to plunge downward against the bed. This can increase both local and contraction scour. The magnitude of the increase is still largely undetermined. Debris can be taken into account in the scour equations by estimating how much the debris will increase the width of a pier or length of an abutment. Debris and ice effects on contraction scour can also be accounted for by estimating the amount of flow blockage (decrease in width of the bridge opening) in the equations for contraction scour. Limited field measurements of scour at ice jams indicate the scour can be as much as 3 to 10 m (10 to 30 ft).

3.7 LATERAL SHIFTING OF A STREAM

Streams are dynamic. Areas of flow concentration continually shift banklines, and in meandering streams having an "S-shaped" planform, the channel moves both laterally and downstream. A braided stream has numerous channels which are continually changing. In a braided stream, the deepest natural scour occurs when two channels come together or when the flow comes together downstream of an island or bar. This scour depth has been observed to be 1 to 2 times the average flow depth.

A bridge is static. It fixes the stream at one place in time and space. A meandering stream whose channel moves laterally and downstream into the bridge reach can erode the approach embankment and can affect contraction and local scour because of changes in flow direction. A braided stream can shift under a bridge and have two channels come together at a pier or abutment, increasing scour. Descriptions of stream morphology are given in HDS 6 and HEC-20.[22, 6]

Factors that affect lateral shifting of a stream and the stability of a bridge are the geomorphology of the stream, location of the crossing on the stream, flood characteristics, the characteristics of the bed and bank material, and wash load. It is difficult to anticipate when a change in planform may occur. It may be gradual or the result of a single major flood event. Also, the direction and magnitude of the movement of the stream are not easily predicted. While it is difficult to evaluate the vulnerability of a bridge due to changes in planform, it is important to incorporate potential planform changes into the design of new bridges and design of countermeasures for existing bridges. These factors are discussed and analysis techniques are presented in HEC-20.[6]

Countermeasures for lateral shifting and instability of the stream may include changes in the bridge design, construction of river control works, protection of abutments with riprap, or careful monitoring of the river in a bridge inspection program. **Serious consideration should be given to placing footings/foundations located on floodplains at elevations the same as those located in the main channel**. Control of lateral shifting requires river training works, bank stabilizing by riprap, and/or guide banks. The design of these works is beyond the scope of this circular. Design methods are given by FHWA in HEC-23,[7] HDS 6,[22] HEC-11,[33] and similar publications.[34,35] The USACE and AASHTO provide additional guidance.[36,37,38,39]

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CHAPTER 4

LONG-TERM AGGRADATION AND DEGRADATION

4.1 INTRODUCTION

This chapter discusses the factors affecting long-term bed elevation changes, methods available for estimating these changes, and the role of sediment transport computer models that are available to compliment HEC-20 procedures. This chapter links long-term degradation to the other components of scour at a bridge site. In following chapters methods and equations are given for determining the other components of total scour. Procedures for estimating long-term aggradation and degradation at a bridge are presented in HEC-20.[6]

4.2 LONG-TERM BED ELEVATION CHANGES

Long-term bed elevation changes may be the natural trend of the stream or may be the result of some modification to the stream or watershed. The streambed may be aggrading, degrading, or in relative equilibrium in the vicinity of the bridge crossing. In this section, long-term trends are considered. Long-term aggradation and degradation do not include the cutting and filling of the streambed at a bridge that might occur during a runoff event (general and local scour). A stream may cut and fill at specific locations during a runoff event and also have a long-term trend of an increase or decrease in bed elevation over a longer reach of a stream. The problem for the engineer is to estimate the long-term bed elevation changes that will occur during the life of the structure.

A long-term trend may change during the life of the bridge. These long-term changes are the result of modifications to the stream or watershed. Such changes may be the result of natural processes or human activities. The engineer must assess the present state of the stream and watershed and then evaluate potential future changes in the river system. From this assessment, the long-term streambed changes must be estimated.

Factors that affect long-term bed elevation changes are dams and reservoirs (up- or downstream of the bridge), changes in watershed land use (urbanization, deforestation, etc.), channelization, cutoffs of meander bends (natural or man-made), changes in the downstream channel base level (control), gravel mining from the streambed, diversion of water into or out of the stream, natural lowering of the fluvial system, movement of a bend and 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 result in aggradation. The elevation of the bed under bridges which cross streams tributary to a larger stream will follow the trend of the larger stream unless there are controls. Controls could be bed rock, dams, culverts or other structures. The changes in bed elevation decrease the further upstream the bridge is from the confluence with another stream or from other bed elevation controls.

The USACE, USGS, and other Federal and State agencies should be contacted concerning documented long-term streambed variations. If no data exist or if such data require further evaluation, an assessment of long-term streambed elevation changes for riverine streams should be made using the principles of river mechanics. Such an assessment requires the consideration of all influences upon the bridge crossing, i.e., runoff from the watershed to a stream (hydrology), sediment delivery to the channel (watershed erosion), sediment transport capacity of a stream (hydraulics), and response of a stream to these factors (geomorphology and river mechanics).

With coastal streams, the principles of both river and coastal engineering mechanics are needed. In coastal streams, estuaries or inlets, in addition to the above, consideration must be given to tidal conditions, i.e., the magnitude and period of the storm surge, sediment delivery to the channel by the ebb and flow of the tide, littoral drift, sediment transport capacity of the tidal flows, and response of the stream, estuary, or inlet to these tidal and coastal engineering factors.

Significant morphologic impacts can result from human activities. The assessment of the impact of human activities requires a study of the history of the river, estuary, or tidal inlet, as well as a study of present water and land use and stream control activities. All agencies involved with the river or coastal area should be contacted to determine possible future changes.

4.3 ESTIMATING LONG-TERM AGGRADATION AND DEGRADATION

To organize an assessment of long-term aggradation and degradation, a three-level fluvial system approach can be used. The three level approach consists of (1) a qualitative determination based on general geomorphic and river mechanics relationships, (2) an engineering geomorphic analysis using established qualitative and quantitative relationships to estimate the probable behavior of the stream system to various scenarios or future conditions, and (3) physical models or physical process computer modeling using mathematical models such as BRI-STARS[21] and the USACE HEC-6[40] to make predictions of quantitative changes in streambed elevation due to changes in the stream and watershed. Methods to be used in Levels (1) and (2) are presented in HEC-20 and HDS 6.[6, 22]

For coastal areas, where highway crossings (bridges) and/or longitudinal stream encroachments are subject to tidal influences, the three-level approach used in fluvial systems is also appropriate (Chapter 9). The following sections outline procedures that can assist in identifying long-term trends in vertical stability.

4.3.1 Bridge Inspection Records

The biannual bridge inspection reports for bridges on the stream where a new or replacement bridge is being designed are an excellent source of data on long-term aggradation or degradation trends. Also, inspection reports for bridges crossing streams in the same area or region should be studied. In most states the biannual inspection includes taking the elevation and/or cross section of the streambed under the bridge. These elevations are usually referenced to the bridge, but these relative bed elevations will show trends and can be referenced to sea level elevations. Successive cross sections from a series of bridges in a stream reach can be used to construct longitudinal streambed profiles through the reach.

4.3.2 Gaging Station Records

The USGS and many State Water Resource and Environmental agencies maintain gaging stations to measure stream flow. In the process they maintain records from which the aggradation or degradation of the streambed can be determined. Gaging station records at the bridge site, on the stream to be bridged and in the area or region can be used.

Where an extended historical record is available, one approach to using gaging station records to determine long-term bed elevation change is to plot the change in stage through time for a selected discharge. This approach is often referred to as establishing a "specific gage" record.

Figure 4.1 shows a plot of specific gage data for a discharge of 14 m^3/sec (500 cfs) from about 1910 to 1980 for Cache Creek in California. Cache Creek has experienced significant gravel mining with records of gravel extraction quantities available since about 1940. When the historical record of cumulative gravel mining is compared to the specific gage plot, the potential impacts are apparent. The specific gage record shows more than 3 m (10 ft) of long-term degradation in a 70-year period.

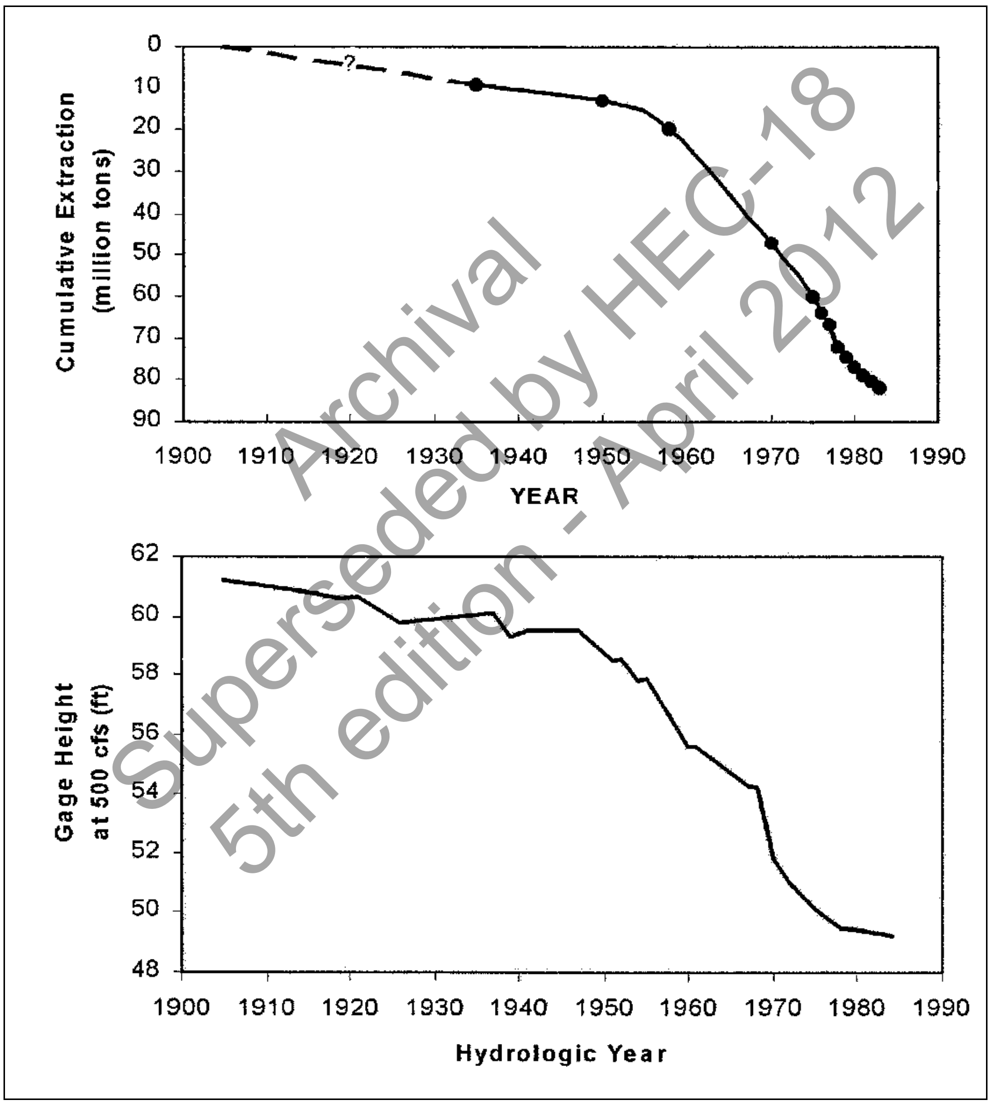

Figure 4.1. Specific gage data for Cache Creek, California.

4.3.3 Geology and Stream Geomorphology

The geology and geomorphology of the site needs to be studied to determine the potential for long-term bed elevation changes at the bridge site. Quantitative techniques for streambed aggradation and degradation analyses are covered in detail in HEC-20.[6] These techniques include:

- Incipient motion analysis
- Analysis of armoring potential
- Equilibrium slope analysis
- Sediment continuity analysis

Sediment transport concepts and equations are discussed in detail in HDS 6.[22]

4.3.4 Computer Models

Sediment transport computer models can be used to determine long-term aggradation or degradation trends. These computer models route sediment down a channel and adjust the channel geometry to reflect imbalances in sediment supply and transport capacity. The BRI-STARS[21] and HEC-6[40] models are examples of sediment transport models that can be used for single event or long-term estimates of changes in bed elevation. The information needed to run these models includes:

- Channel and floodplain geometry
- Structure geometry
- Roughness
- Geologic or structural vertical controls
- Downstream water surface relationship
- Event or long-term inflow hydrographs
- Tributary inflow hydrographs
- Bed material gradations
- Upstream sediment supply
- Tributary sediment supply
- Selection of appropriate sediment transport relationship
- Depth of alluvium

These models perform hydraulic and sediment transport computations on a cross section basis and adjust the channel geometry prior to proceeding with the next time step. The actual flow hydrograph can be used as input. BRI-STARS[21] also has an option where width adjustment can be predicted.

4.3.5 Aggradation, Degradation, and Total Scour

Using all the information available estimate the long-term bed elevation change at the bridge site for the design life of the bridge. Usually, the design life is 100 years. **If the estimate indicates that the stream will degrade, use the elevation after degradation as the base elevation for general and local scour. That is, total scour must include the estimated long-term degradation**. If the estimate indicates that the stream will aggrade, then (1) make note of this fact to inspection and maintenance personnel, and (2) use existing ground elevation as the base for general and local scour.

4.3.6 Inspection, Maintenance, and Countermeasures

The estimate of long-term aggradation or degradation in the final design should be communicated to inspection and maintenance personnel. This information will aid them in tracking long-term trends and provide feedback for future design and evaluation. HEC-23[7] outlines techniques for controlling long-term bed elevation changes and provides design guidance for countermeasures commonly used for vertical stability problems.

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CHAPTER 5

GENERAL SCOUR

5.1 INTRODUCTION

General scour is the general decrease in the elevation of the bed across the bridge opening. It does not include localized scour at the foundations (local scour) or the long-term changes in the stream bed elevation (aggradation or degradation). General scour may not have a uniform depth across the bridge opening. General scour can be cyclic, that is, there can be an increase and decrease of the stream bed elevation (cutting and filling) during the passage of a flood.

The most common general scour is contraction scour. There are several cases and flow conditions for contraction scour. Typically, contraction scour occurs where the bridge opening is smaller than the flow area of the upstream channel and/or floodplain. Other general scour conditions can result from erosion related to planform characteristics of the stream, flow around a bend, variable downstream control, or other changes that decrease the bed elevation at the bridge. In this chapter, methods and equations will be presented to estimate general scour.

5.2 CONTRACTION SCOUR

5.2.1 Contraction Scour Conditions

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). In the case of **live-bed scour**, the fully developed scour in the bridge cross section reaches equilibrium when sediment transported into the contracted section equals sediment transported out. As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For **live-bed** scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For **clear-water** scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the section. Normally, for both live-bed and clear-water scour the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in.

Clear-water contraction scour occurs when (1) there is no bed material transport from the upstream reach into the downstream reach, or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With clear-water contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material.

There are four conditions (cases) of contraction scour at bridge sites depending on the type of contraction, and whether there is overbank flow or relief bridges. Regardless of the case, contraction scour can be evaluated using two basic equations: (1) **live-bed** scour, and (2) **clear-water** scour. For any case or condition, it is only necessary to determine if the flow in the main channel or overbank area upstream of the bridge, or approaching a relief bridge, is transporting bed material (live-bed) or is not (clear-water), and then apply the appropriate equation with the variables defined according to the location of contraction scour (channel or overbank).

To determine if the flow upstream of the bridge is transporting bed material, calculate the critical velocity for beginning of motion V_c of the D_{50} size of the bed material being considered for movement and compare it with the mean velocity V of the flow in the main channel or overbank area upstream of the bridge opening. If the critical velocity of the bed material is larger than the mean velocity ($V_c > V$), then clear-water contraction scour will exist. If the critical velocity is less than the mean velocity ($V_c < V$), then live-bed contraction scour will exist. To calculate the critical velocity use the equation derived in the Appendix C. This equation is:

$$V_c = K_u \, y^{1/6} \, D^{1/3} \tag{5.1}$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, m/s (ft/s)
y	=	Average depth of flow upstream of the bridge, m (ft)
D	=	Particle size for V_c, m (ft)
D_{50}	=	Particle size in a mixture of which 50 percent are smaller, m (ft)
K_u	=	6.19 SI units
K_u	=	11.17 English units

The D_{50} is taken as an average of the bed material size in the reach of the stream upstream of the bridge. It is a characteristic size of the material that will be transported by the stream. Normally this would be the bed material size in the upper 0.3 m (1 ft) of the stream bed.

Live-bed contraction scour depths may be limited by armoring of the bed by large sediment particles in the bed material or by sediment transport of the bed material into the bridge cross-section. Under these conditions, live-bed contraction scour at a bridge can be determined by calculating the scour depths using both the clear-water and live-bed contraction scour equations and using the smaller of the two depths.

5.2.2 Contraction Scour Cases

Four conditions (cases) of contraction scour are commonly encountered:

Case 1. Involves overbank flow on a floodplain being forced back to the main channel by the approaches to the bridge. Case 1 conditions include:

a. The river channel width becomes narrower either due to the bridge abutments projecting into the channel or the bridge being located at a narrowing reach of the river (Figure 5.1);

b. No contraction of the main channel, but the overbank flow area is completely obstructed by an embankment (Figure 5.2); or

c. Abutments are set back from the stream channel (Figure 5.3).

Case 2. Flow is confined to the main channel (i.e., there is no overbank flow). The normal river channel width becomes narrower due to the bridge itself or the bridge site is located at a narrower reach of the river (Figures 5.4 and 5.5).

Case 3. A relief bridge in the overbank area with little or no bed material transport in the overbank area (i.e., clear-water scour) (Figure 5.6).

Case 4. A relief bridge over a secondary stream in the overbank area with bed material transport (similar to Case 1) (Figure 5.7).

Notes:

1. **Cases 1, 2, and 4** may either be live-bed or clear-water scour depending on whether there is bed material transport from the upstream reach into the bridge reach during flood flows. To determine if there is bed material transport compute the critical velocity at the approach section for the D_{50} of the bed material using the equation given above and compare to the mean velocity at the approach section. To determine if the bed material will be washed through the contraction determine the ratio of the shear velocity (V_*) in the contracted section to the fall velocity (ω) of the D_{50} of the bed material being transported from the upstream reach (see the definition of V_* in the live-bed contraction scour equation). If the ratio is much larger than 2, then the bed material from the upstream reach will be mostly suspended bed material discharge and may wash through the contracted reach (clear-water scour).

2. **Case 1c is very complex.** The depth of contraction scour depends on factors such as (1) how far back from the bank line the abutment is set, (2) the condition of the overbank (is it easily eroded, are there trees on the bank, is it a high bank, etc.), (3) whether the stream is narrower or wider at the bridge than at the upstream section, (4) the magnitude of the overbank flow that is returned to the bridge opening, and (5) the distribution of the flow in the bridge section, and (6) other factors.

   The main channel under the bridge may be live-bed scour; whereas, the set-back overbank area may be clear-water scour.

   WSPRO[15] or HEC-RAS[16,17] can be used to determine the distribution of flow between the main channel and the set-back overbank areas in the contracted bridge opening. However, the distribution of flow needs to be done with care. Studies by Chang[41] and Sturm [42] have shown that conveyance calculations do not properly account for the flow distribution under the bridge.

   If the abutment is set back only a small distance from the bank (less than 3 to 5 times the average depth of flow through the bridge), there is the possibility that the combination of contraction scour and abutment scour may destroy the bank. Also, the two scour mechanisms are not independent. Consideration should be given to using a guide bank and/or protecting the bank and bed under the bridge in the overflow area with rock riprap. See HEC-23[7] for guidance on designing rock riprap.

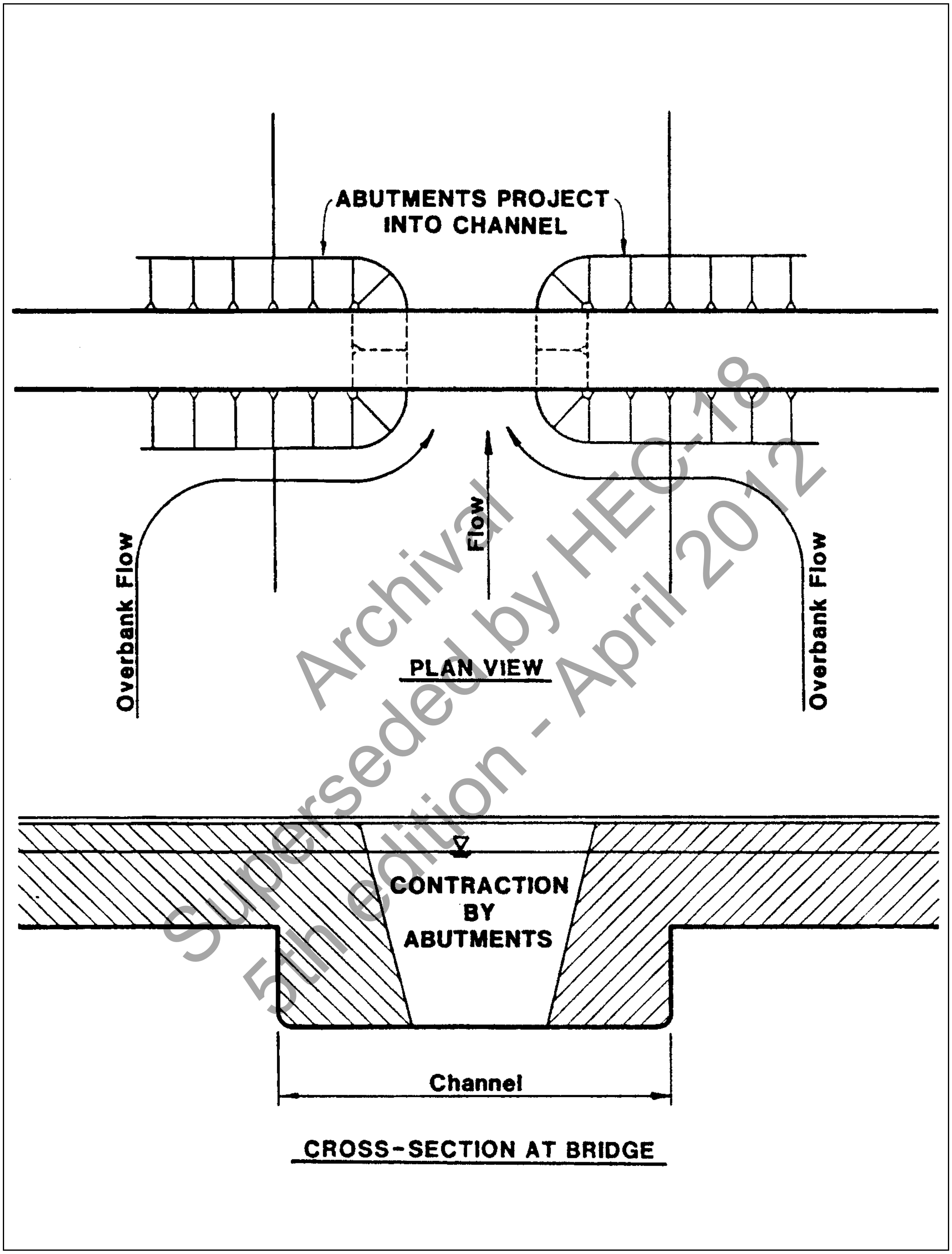


Figure 5.1. Case 1A: Abutments project into channel.

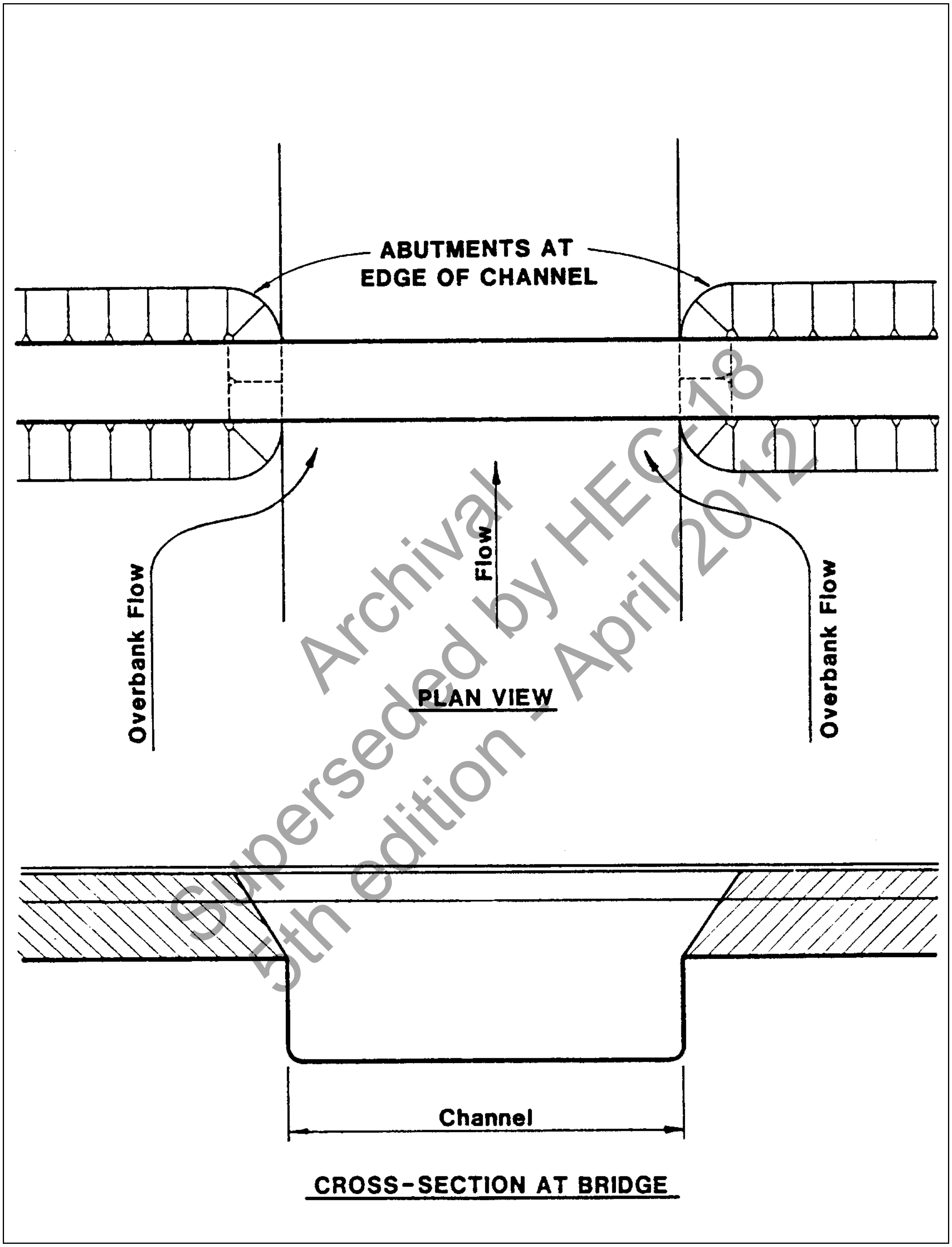


Figure 5.2. Case 1B: Abutments at edge of channel.

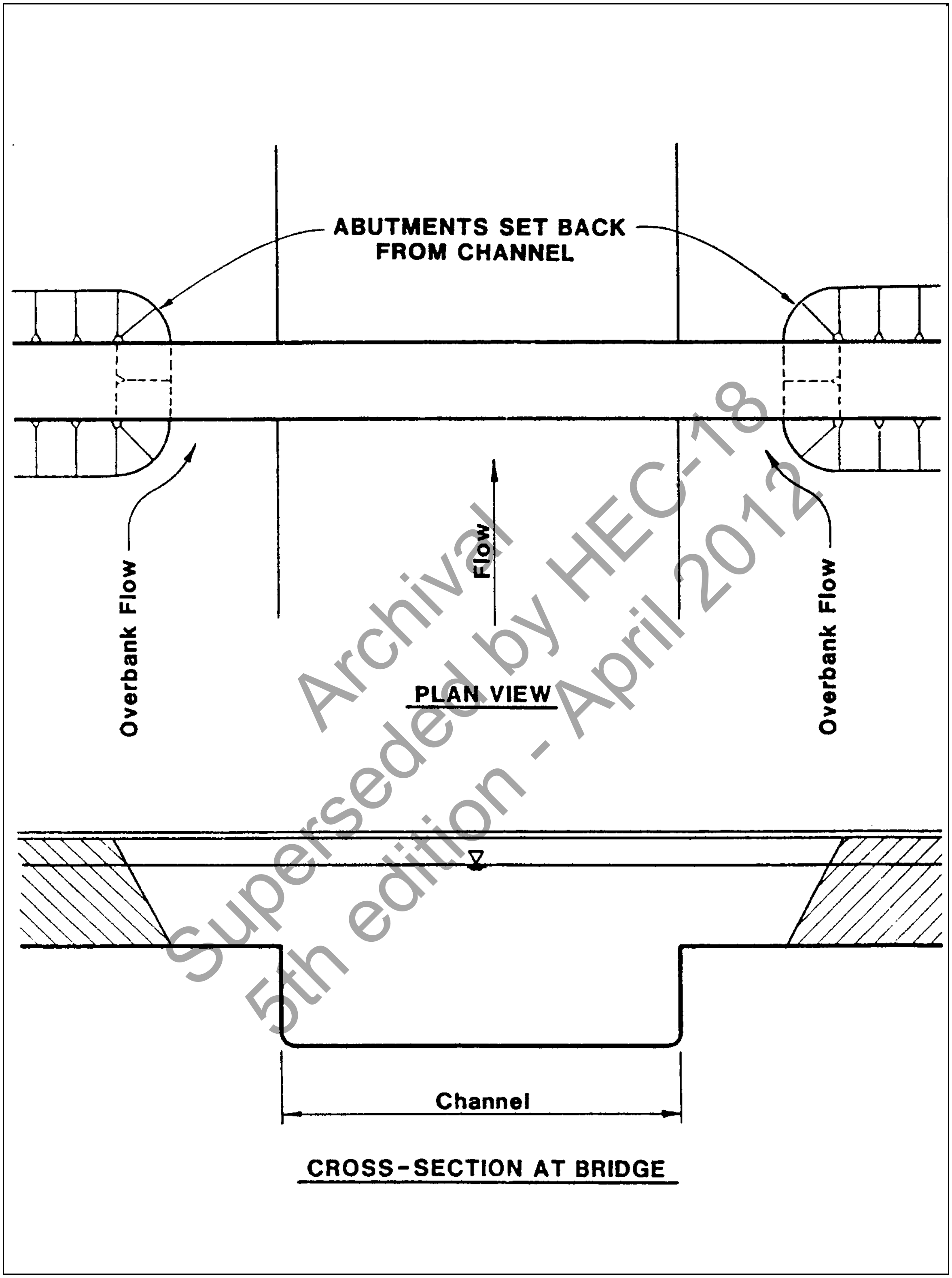


Figure 5.3. Case 1C: Abutments set back from channel.

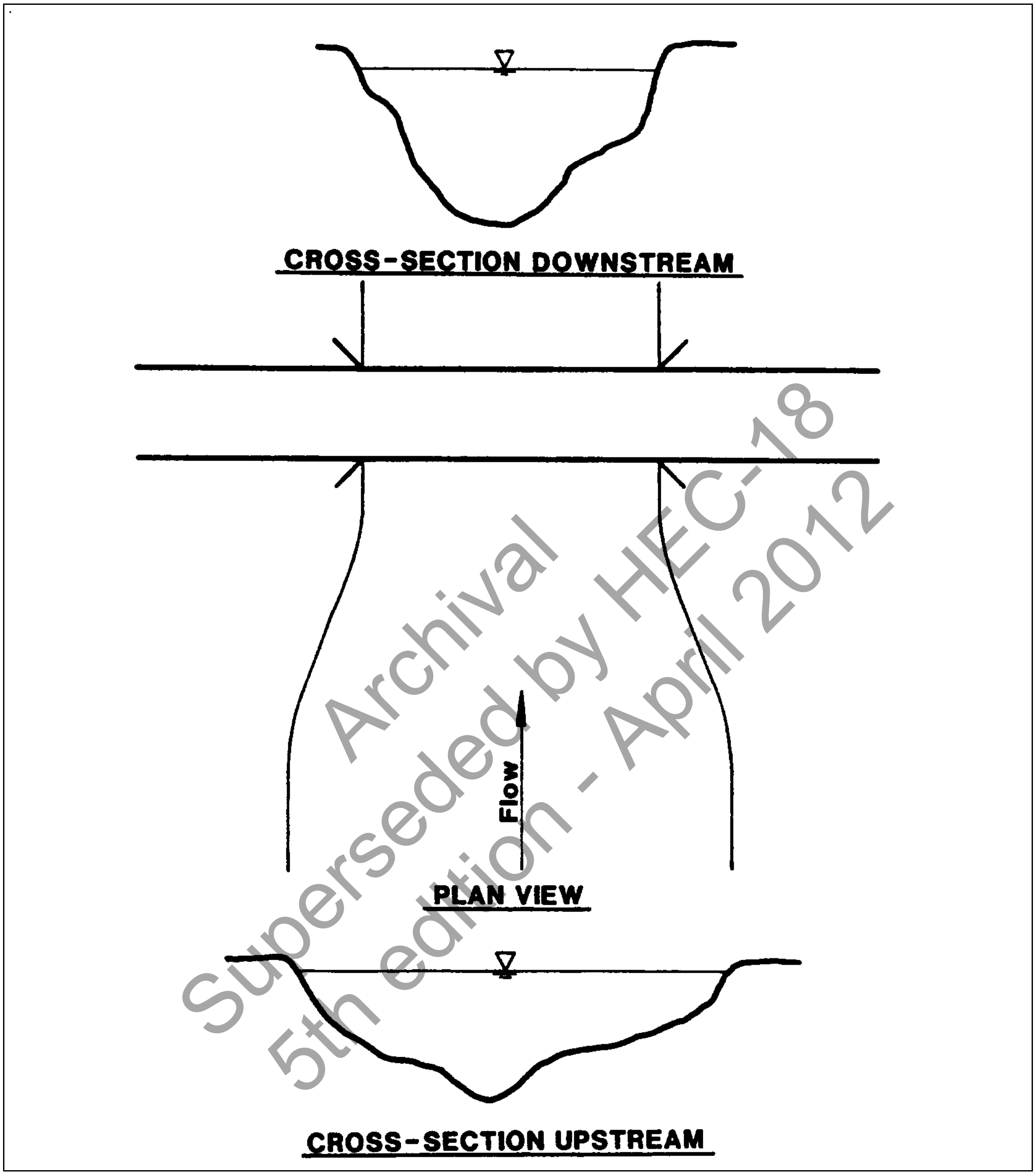


Figure 5.4. Case 2A: River narrows.

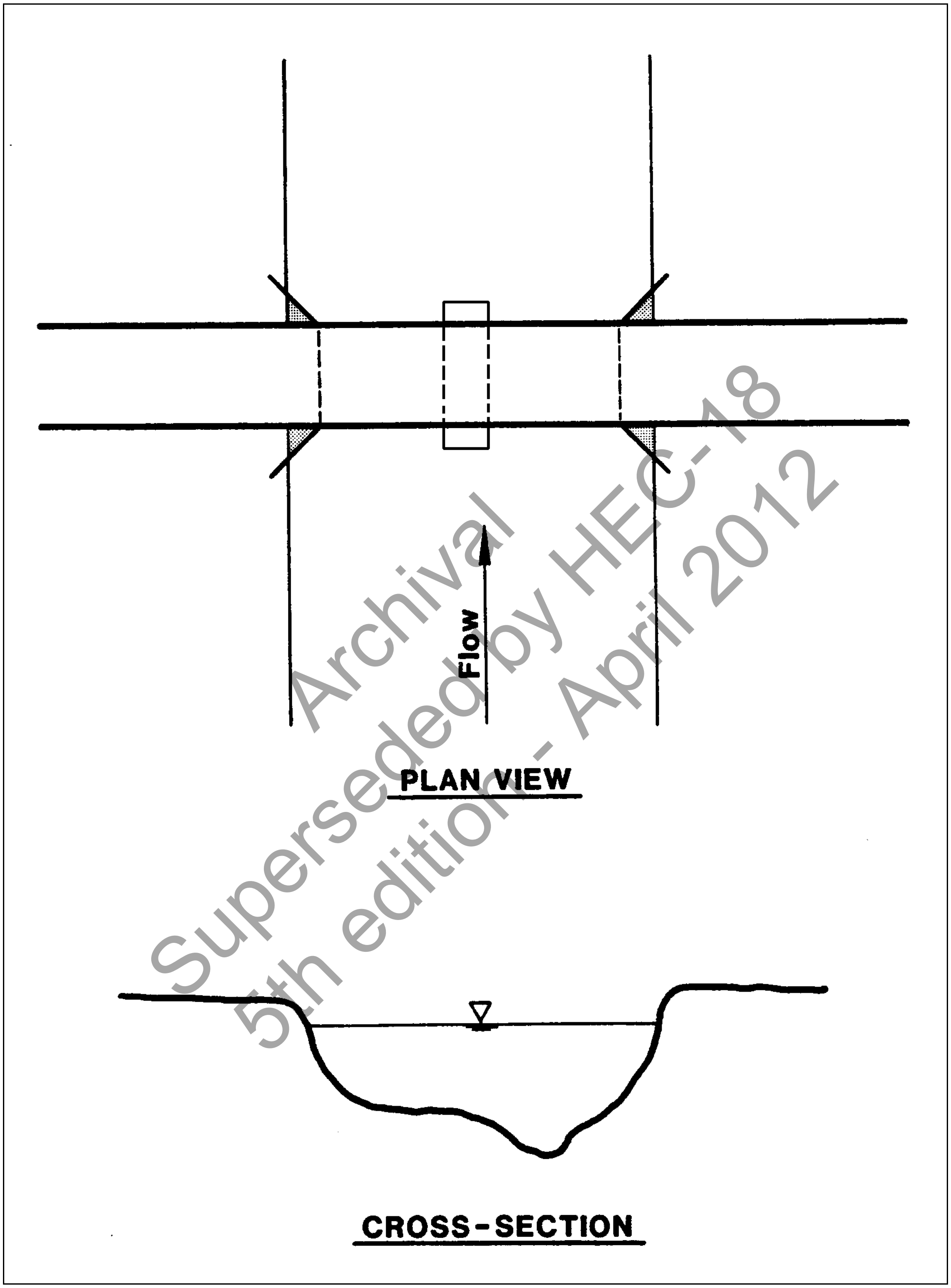


Figure 5.5. Case 2B: Bridge abutments and/or piers constrict flow.

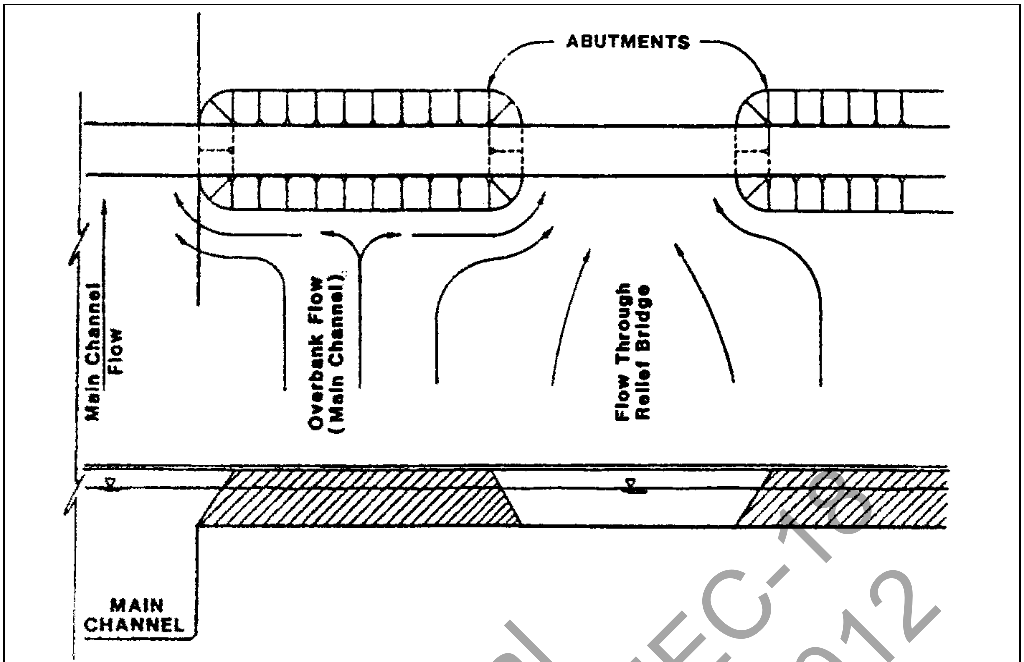


Figure 5.6. Case 3: Relief bridge over floodplain.

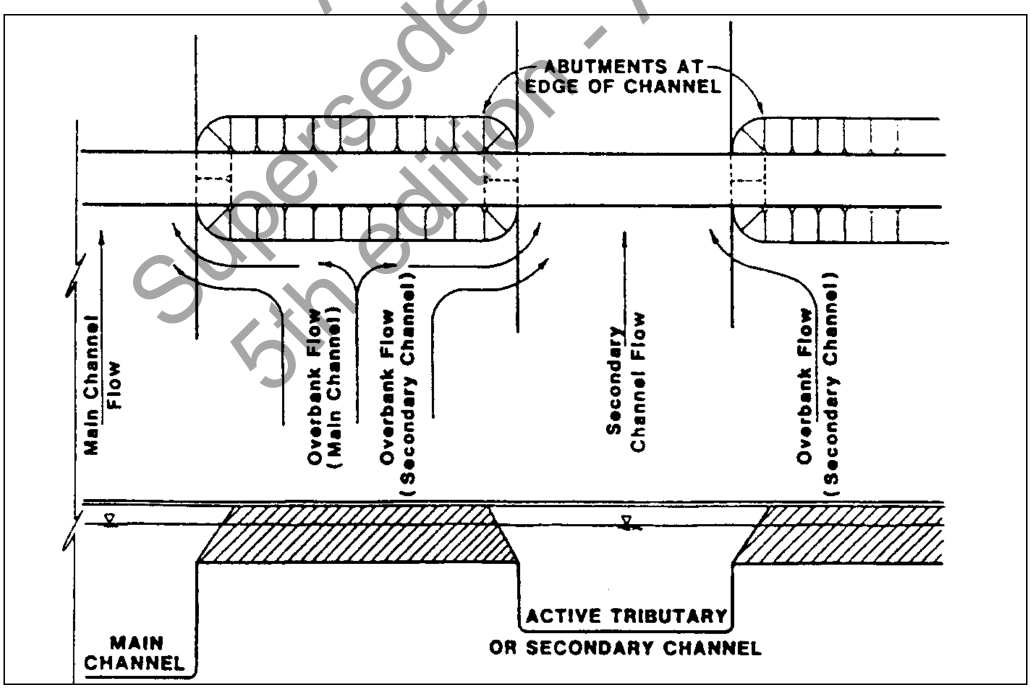


Figure 5.7. Case 4: Relief bridge over secondary stream.

3. **Case 3** may be clear-water scour even though the floodplain bed material is composed of sediments with a critical velocity that is less than the flow velocity in the overbank area. The reasons for this are (1) there may be vegetation growing part of the year, and (2) if the bed material is fine sediments, the bed material discharge may go into suspension (wash load) at the bridge and not influence contraction scour.

4. **Case 4** is similar to Case 3, but there is sediment transport into the relief bridge opening (live-bed scour). This case can occur when a relief bridge is over a secondary channel on the floodplain. Hydraulically this is no different from case 1, but analysis is required to determine the floodplain discharge associated with the relief opening and the flow distribution going to and through the relief bridge. This information could be obtained from WSPRO[15] or HEC-RAS.[16, 17]

5.3 LIVE-BED CONTRACTION SCOUR

A modified version of Laursen's 1960 equation for live-bed scour at a long contraction is recommended to predict the depth of scour in a contracted section.[43] The original equation is given in Appendix C. The modification is to eliminate the ratio of Manning's n (see the following Note #3). The equation assumes that bed material is being transported from the upstream section.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \tag{5.2}$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \tag{5.3}$$

where:

y_1 = Average depth in the upstream main channel, m (ft)
y_2 = Average depth in the contracted section, m (ft)
y_o = Existing depth in the contracted section before scour, m (ft) (see Note 7)
Q_1 = Flow in the upstream channel transporting sediment, m^3/s (ft^3/s)
Q_2 = Flow in the contracted channel, m^3/s (ft^3/s)

W_1 = Bottom width of the upstream main channel that is transporting bed material, m (ft)
W_2 = Bottom width of the main channel in the contracted section less pier width(s), m (ft)
k_1 = Exponent determined below

V_*/ω	k_1	Mode of Bed Material Transport
<0.50	0.59	Mostly contact bed material discharge
0.50 to 2.0	0.64	Some suspended bed material discharge
>2.0	0.69	Mostly suspended bed material discharge

V_* = $(\tau_o/\rho)^{1/2} = (g y_1 S_1)^{1/2}$, shear velocity in the upstream section, m/s (ft/s)
ω = Fall velocity of bed material based on the D_{50}, m/s (Figure 5.8)
For fall velocity in English units (ft/s) multiply ω in m/s by 3.28
g = Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)
S_1 = Slope of energy grade line of main channel, m/m (ft/ft)

τ_o = Shear stress on the bed, Pa (N/m^2) (lb/ft^2)
ρ = Density of water (1000 kg/m^3) (1.94 slugs/ft^3)

Notes:

1. Q_2 may be the total flow going through the bridge opening as in cases 1a and 1b. **It is not the total flow for Case 1c**. For Case 1c contraction scour must be computed separately for the main channel and the left and/or right overbank areas.

2. Q_1 is the flow in the main channel upstream of the bridge, not including overbank flows.

3. The Manning's n ratio is eliminated in Laursen live-bed equation to obtain Equation 5.2 (Appendix C).This was done for the following reasons. The ratio can be significant for a condition of dune bed in the upstream channel and a corresponding plane bed, washed out dunes or antidunes in the contracted channel. However, Laursen's equation does not correctly account for the increase in transport that will occur as the result of the bed planning out (which decreases resistance to flow, increases the velocity and the transport of bed material at the bridge). That is, Laursen's equation indicates a decrease in scour for this case, whereas in reality, there would be an increase in scour depth. In addition, at flood flows, a plane bedform will usually exist upstream and through the bridge waterway, and the values of Manning's n will be equal. Consequently, the n value ratio is not recommended or presented in Equation 5.2.

4. W_1 and W_2 are not always easily defined. In some cases, it is acceptable to use the topwidth of the main channel to define these widths. Whether topwidth or bottom width is used, it is important to be consistent so that W_1 and W_2 refer to either bottom widths or top widths.

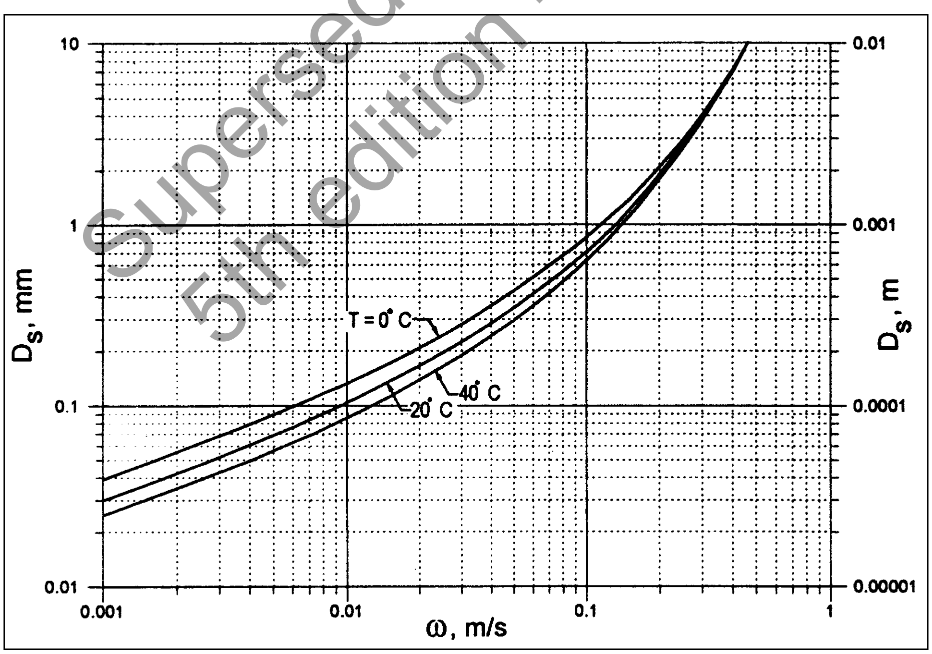


Figure 5.8. Fall velocity of sand-sized particles with specific gravity of 2.65 in metric units.

5. The average width of the bridge opening (W_2) is normally taken as the bottom width, with the width of the piers subtracted.

6. Laursen's equation will overestimate the depth of scour at the bridge if the bridge is located at the upstream end of a natural contraction or if the contraction is the result of the bridge abutments and piers. At this time, however, it is the best equation available.

7. In sand channel streams where the contraction scour hole is filled in on the falling stage, the y_0 depth may be approximated by y_1. Sketches or surveys through the bridge can help in determining the existing bed elevation.

8. **Scour depths with live-bed contraction scour may be limited by coarse sediments in the bed material armoring the bed. Where coarse sediments are present, it is recommended that scour depths be calculated for live-bed scour conditions using the clear-water scour equation (given in the next section) in addition to the live-bed equation, and that the smaller calculated scour depth be used.**

5.4 CLEAR-WATER CONTRACTION SCOUR

The recommended clear-water contraction scour equation is based on a development suggested by Laursen[44] (presented in the Appendix C). The equation is:

$$y_2 = \left[\frac{K_u Q^2}{D_m^{2/3} W^2}\right]^{3/7} \qquad (5.4)$$

$$y_s = y_2 - y_o = \text{(average contraction scour depth)} \qquad (5.5)$$

where:

y_2	=	Average equilibrium depth in the contracted section after contraction scour, m (ft)
Q	=	Discharge through the bridge or on the set-back overbank area at the bridge associated with the width W, m^3/s (ft^3/s)
D_m	=	Diameter of the smallest nontransportable particle in the bed material (1.25 D_{50}) in the contracted section, m (ft)
D_{50}	=	Median diameter of bed material, m (ft)
W	=	Bottom width of the contracted section less pier widths, m (ft)
y_o	=	Average existing depth in the contracted section, m (ft)
K_u	=	0.025 SI units
K_u	=	0.0077 English units

Equation 5.4 is a rearranged version of 5.1.

Because D_{50} is not the largest particle in the bed material, the scoured section can be slightly armored. Therefore, the D_m is assumed to be 1.25 D_{50}. For stratified bed material the depth of scour can be determined by using the clear-water scour equation sequentially with successive D_m of the bed material layers.

5.5 CONTRACTION SCOUR WITH BACKWATER

The **live-bed** contraction scour equation is derived assuming a uniform reach upstream and a long contraction into a uniform reach downstream of the bridge. With live-bed scour the equation computes a depth after the long contraction where the sediment transport into the downstream reach is equal to the sediment transport out. The **clear-water** contraction scour equations are derived assuming that the depth at the bridge increases until the shear-stress and velocity are decreased so that there is no longer any sediment transport. With the clear-water equations it is assumed that flow goes from one uniform flow condition to another. Both equations calculate contraction scour depth assuming a level water surface ($y_s = y_2 - y_o$). A more consistent computation would be to write an energy balance before and after the scour. For live-bed the energy balance would be between the approach section (1) and the contracted section (2). Whereas, for clear-water scour it would be the energy at the same section before (1) and after (2) the contraction scour.

Backwater, in extreme cases, can decrease the velocity, shear stress and the sediment transport in the upstream section. This will increase the scour at the contracted section. The backwater can, by storing sediment in the upstream section, change live-bed scour to clear-water scour.

5.6 CONTRACTION SCOUR EXAMPLE PROBLEMS (SI)

5.6.1 Example Problem 1 - Live-Bed Contraction Scour (SI)

Given:

The upstream channel width = 98.2 m; depth = 2.62 m
The discharge is 773 m^3/s and is all contained within the channel. Channel slope = 0.004 m/m
The bridge abutments consist of vertical walls with wing walls. Bridge width = 37.2 m; with 3 sets of piers consisting of 3 columns, 0.38 m in diameter.
The bed material size: from 0 to 0.9 m, the D_{50} is 0.31 mm and below 0.9 m the D_{50} is 0.70 mm with a fall velocity of 0.10 m/s
Original depth at bridge is estimated as 2.16 m

Determine:

The magnitude of the contraction scour depth.

Solution:

1. Determine if it is live-bed or clear-water scour.

Average velocity in the upstream reach

$V = 773/ (2.62 \text{ X } 98.2) = 3.0$ m/s

For velocities this large and bed material this fine **live-bed** scour will occur. Check by calculating V_c for 0.7 mm bed material size. If live-bed scour occurs for 0.7mm it would also be live-bed for D_{50} = 0.3 mm.

$V_c = 6.19\ (2.62)^{1/6}\ (0.0007)^{1/3} = 0.65$ m/s

Live-bed contraction scour is verified

2. Calculate contraction scour

a. Determine k_1 for mode of bed material transport

$V_* = (9.81 \text{ X } 2.62 \text{ X } 0.004)^{0.5} = 0.32$ m/s

$\omega = 0.10$; $V_*/\omega = 3.2$; $k_1 = 0.69$

b. Live-bed contraction scour. Equation 5.2

$$\frac{y_2}{2.62} = \left[\frac{98.2}{36.06}\right]^{0.69} = 2.00$$

$Q_1 = Q_2$

y_2 = 2.62 X 2.00 = 5.24 m from water surface.
y_s = 5.24 - 2.16 = 3.08 m from original bed surface

5.6.2 Example Problem 2 - Alternate Method (SI)

An alternative approach to calculating y_s in Problem 1 is to calculate the scour depth using both the clear-water and the live-bed equation and take the smaller scour depth.

a. Live bed-bed scour depth is 3.08 m from Problem 1.

b. Clear-water scour depth (Equation 5.4)

$$D_m = 1.25\ D_{50} = 1.25\ (0.0007) = 0.0009\ m$$

$$y_2 = \left[\frac{0.025\,(773)^2}{0.0009^{2/3}\,(36.06)^2}\right]^{3/7} = 21.12\,m$$

y_s = 21.12 - 2.16 = 18.96 m from original bed surface

c. Live-bed scour (3.08 m < 18.96 m). The sediment transport limits the contraction scour depth rather than the size of the bed material.

5.6.3 Example Problem 3 - Relief Bridge Contraction Scour (SI)

The 1952 flood on the Missouri River destroyed several relief bridges on Highway 2 in Iowa near Nebraska City, Nebraska. The USGS made continuous measurements during the period April 2 through April 29, 1952. This data set is from the April 21, 1952 measurement (measurement # 1013). The discharge in the relief bridge was 368 m^3/s. The measurement was made on the upstream side of Cooper Creek ditch using a boat and tag line.

Q = 368 m^3/s; Bridge width (minus piers) = 91.4 m; Area = 706.43 m^2
$V_{average}$ = 0.52 m/s; y_0 = 1.28 to 1.62 m

D_{50} = 0.24 mm (D_m = 1.25 x 0.24 = 0.3 mm)
Clear- water scour because of low velocity flow on the floodplain (Equation 5.4)
Calculate y_2:

$$y_2 = \left[\frac{0.025\,(368)^2}{(0.0003)^{2/3}\,(91.4)^2} \right]^{3/7} = 6.89\,\text{m}$$

y_2 = 6.89 m from the water surface, this compares to 7.71 m measured at the site.

5.7 CONTRACTION SCOUR EXAMPLE PROBLEMS (ENGLISH)

5.7.1 Example Problem 1 - Live-Bed Contraction Scour (English)

Given:

The upstream channel width = 322 ft; depth = 8.6 ft
The discharge is 27,300 cfs and is all contained within the channel. Channel slope = 0.004 (ft/ft)
The bridge abutments consist of vertical walls with wing walls, width = 122 ft; with 3 sets of piers consisting of 3 columns 15 inches in diameter.
The bed material size: from 0 to 3 ft the D_{50} is 0.31 mm (0.0010 ft) and below 3 ft the D_{50} is 0.70 mm (0.0023 ft) with a fall velocity of 0.33 ft/sec
Original depth at bridge is estimated as 7.1 ft

Determine:

The magnitude of the contraction scour depth.

Solution:

1. Determine if it is live-bed or clear-water scour.

Average velocity in the upstream reach

V = 27,300/(8.6 x 322) = 9.86 ft/s

For velocities this large and bed material this fine **live-bed** scour will occur. Check by calculating V_c for 0.7 mm bed material size. If live-bed scour occurs for 0.7mm it would also be live-bed for 0.3mm.

$V_c = 11.17\,(8.6)^{1/6}\;(0.0023)^{1/3} = 2.11$ ft/s

Live-bed contraction scour is verified

2. Calculate contraction scour

a. Determine K_1 for mode of bed material transport

$V_* = (32.2 \times 8.6 \times 0.004)^{0.5} = 1.05$ ft/s

$\omega = 0.33$; $V_*/\omega = 3.2$; $K_1 = 0.69$

b. Live-bed contraction scour. Equation 5.2

$Q_1 = Q_2$

$$\frac{y_2}{8.6} = \left[\frac{322}{118.25}\right]^{0.69} = 2.00$$

y_2 = 8.6 x 2.00 = 17.2 ft from water surface.
y_s = 17.2 - 7.1 = 10.1 ft from original bed surface

5.7.2 Example Problem 2 - Alternate Method (English)

An alternative approach is demonstrated to calculating y_s in Problem 1 to determine if scour is clear-water or live-bed. In this method calculate the scour depth using both the clear-water and the live-bed equation and take the smaller scour depth.

a. Live-bed scour depth is 10.1 ft from Problem 1.

b. Clear-water scour depth (Equation 5.4)

$$D_m = 1.25\ D_{50} = 1.25\ (0.0023) = 0.0030\ \text{ft}$$

$$y_2 = \left[\frac{0.0077\ (27{,}300)^2}{0.0030^{2/3}\ (118.25)^2}\right]^{3/7} = 69.31\text{ft}$$

y_s = 69.31 - 7.1 = 62.2 ft from original bed surface

c. Live-bed scour (10.1 ft < 62.2 ft). The sediment transport limits the contraction scour depth rather than the size of the bed material.

5.7.3 Example Problem 3 - Relief Bridge Contraction Scour (English)

The 1952 flood on the Missouri River destroyed several relief bridges on Highway 2 in Iowa near Nebraska City, Nebraska. The USGS made continuous measurements during the period April 2 through April 29, 1952. This data set is from the April 21, 1952 measurement (measurement #1013). The discharge in the relief bridge was 13,012 cfs. The measurement was made on the upstream side of Cooper Creek ditch using a boat and tag line.

Q = 13,012 cfs; Bridge width (minus piers) = 300 ft; Area = 7,604 ft^2
$V_{average}$ = 1.71 ft/s; y_0 = 4.2 to 5.3 ft
D_{50} = 0.24 mm (D_m = 1.25 x 0.24 = 0.3 mm)
Clear- water scour because of low velocity flow on the floodplain (Equation 5.4)

$$y_2 = \left[\frac{0.0077\ (13{,}012)^2}{(0.0010)^{2/3}\ (300)^2}\right]^{3/7} = 22.6\ \text{ft}$$

y_2 = 22.6 ft from the water surface, this compares to 25.3 ft measured at the site.

5.8 OTHER GENERAL SCOUR CONDITIONS

5.8.1 Discussion

In a natural channel, the depth of flow is usually greater on the outside of a bend. In fact, there may well be deposition on the inner portion of the bend at a point bar. If a bridge is located on or close to a bend, the general scour will be concentrated on the outer portion of the bend. Also, in bends, the thalweg (the part of the stream where the flow is deepest and, typically, the velocity is the greatest) may shift toward the inside of the bend as the flow increases. This can increase scour and nonuniform distribution of scour in the bridge opening. In some cases during high flow the point bar may have a channel (chute channel) eroded across it. This can further skew the distribution of scour in the bridge reach. Consequently, other general scour conditions such as these are differentiated from contraction scour which involves removal of material from the bed across all or most of the channel width.

The relatively shallow straight reaches between bendway pools are called crossings. With changes in discharge and stage the patterns of scour and fill can also change in the crossing and pool sequence. These geomorphic processes are discussed in more detail in HEC-20 and HDS 6.[6,22] These processes are considered part of general scour. They are cyclic and may be in equilibrium around some general bed elevation. There are no equations for predicting these changes in elevation. Generally, a study of the stream using aerial photographs and/or successive cross section surveys can determine trends. In this case, the long-term safety of the bridge depends, primarily, on inspection.

Some general scour conditions are associated with a particular channel morphology. Braided channels will have deep scour holes when two channels come together downstream from a bar or island (confluence scour). At other times a bar or island will move into the bridge opening concentrating the flow onto a pier or abutment or changing the angle of attack. In anabranching flow, where flow is in two or more channels around semi-permanent islands, there is a problem of determining the distribution of flow between the channels, and over time the distribution may change. The bridge could be designed for the anticipated worst case flow distribution or designed using the present distribution. In either case, inspection and maintenance personnel should be informed of the potential for the flow distribution and scour conditions to change.

Other general scour can be caused by short-term (daily, weekly, yearly, or seasonal) changes in the downstream water surface elevation that control backwater and hence, the velocity through the bridge opening. Similarly, a bridge located upstream or downstream of a confluence can experience general scour caused by variable flow conditions on the main river and tributary. Because this scour is reversible, it is considered other general scour rather than long-term aggradation or degradation. These channel changes and other general scour conditions are also discussed in HEC-20 and HDS 6.[6,22]

5.8.2 Determining Other General Scour

Scour at a bridge cross-section resulting from variable water surface elevation downstream of the bridge (e.g., tributary or downstream control) is analyzed by determining the lowest potential water-surface elevation downstream of the bridge insofar as scour processes are concerned. Then determine contraction and local scour depths using these worst-case conditions.

General scour in a channel bendway resulting from the flow through the bridge being concentrated toward the outside of the bend is analyzed by determining the superelevation of the water surface on the outside of the bend and estimating the resulting velocities and depths through the bridge. The maximum velocity in the outer part of the bend can be 1.5 to 2 times the mean velocity. A physical model study can also be used to determine the velocity and scour depth distribution through the bridge for this case.

Estimating general scour across the bridge cross-section for unusual situations involves particular skills in the application of principles of river mechanics to the site-specific conditions. To determine the scour across the bridge opening in many bridge crossings will require 2-dimensional (2-D) computer programs (for example, FESWMS[45] - see discussion Chapter 9, Section 9.5) or a physical model (HEC-23).[7] Such studies should be undertaken by engineers experienced in the fields of hydraulics and river mechanics.

CHAPTER 6

DETERMINATION OF LOCAL SCOUR AT PIERS

6.1 GENERAL

Local scour at piers is a function of bed material characteristics, bed configuration, flow characteristics, fluid properties, and the geometry of the pier and footing. The bed material characteristics are granular or non granular, cohesive or noncohesive, erodible or non erodible rock. Granular bed material ranges in size from silt to large boulders and is characterized by the D_{50} and a coarse size such as the D_{84} or D_{90} size. Cohesive bed material is composed of silt and clay, possibly with some sand which is bonded chemically (see discussion in Chapter 3). Rock may be solid, massive, or fractured. It may be sedimentary or igneous and erodible or non erodible.

Flow characteristics of interest for local pier scour are the velocity and depth just upstream of the pier, the angle the velocity vector makes to the pier (angle of attack), and free surface or pressure flow. Fluid properties are viscosity, and surface tension which for the field case can be ignored.

Pier geometry characteristics are its type, dimensions, and shape. Types of piers include single column, multiple columns, or rectangular; with or without friction or tip bearing piles; with or without a footing or pile cap; footing or pile cap in the bed, on the surface of the bed, in the flow or under the deck out of the flow. Important dimensions are the diameter for circular piers or columns, spacing for multiple columns, and width and length for solid piers. Shapes include round, square or sharp nose, circular cylinder, group of cylinders, or rectangular. In addition, piers may be simple or complex. A simple pier is a single shaft, column or multiple columns exposed to the flow. Whereas, a complex pier may have the pier, footing or pile cap, and piles exposed to the flow.

Local scour at piers has been studied extensively in the laboratory; however, there is limited field data. The laboratory studies have been mostly of simple piers, but there have been some laboratory studies of complex piers. Often the studies of complex piers are model studies of actual or proposed pier configurations. As a result of the many laboratory studies, there are numerous pier scour equations. In general, the equations are for live-bed scour in cohesionless sand-bed streams.

A graphical comparison by Jones of the more common equations is given in Figure 6.1.(46) An equation given by Melville and Sutherland to calculate scour depths for live-bed scour in sand-bed streams has been added to the original figure.(28) Some of the equations have velocity as a variable, normally in the form of a Froude Number. However, some equations, such as Laursen's do not include velocity.(43) A Froude Number of 0.3 was used in Figure 6.1 for purposes of comparing commonly used scour equations. Jones also compared the equations with the available field data. His study showed that the CSU equation enveloped all the data, but gave lower values of scour than the Jain and Fischer, Laursen, Melville and Sutherland, and Neill equations.(22,47,48,28,46) The CSU equation includes the velocity of the flow just upstream of the pier by including the Froude Number in the equation. On the basis of Jones' studies(46) the Colorado State University (CSU) equation was recommended in the Interim Procedures that accompanied FHWA's Technical Advisory T5140.20.(11,9) With modifications, the CSU equation was recommended in previous editions of HEC-18. The modifications were the addition of coefficients for the effect of bed form and size of bed material.

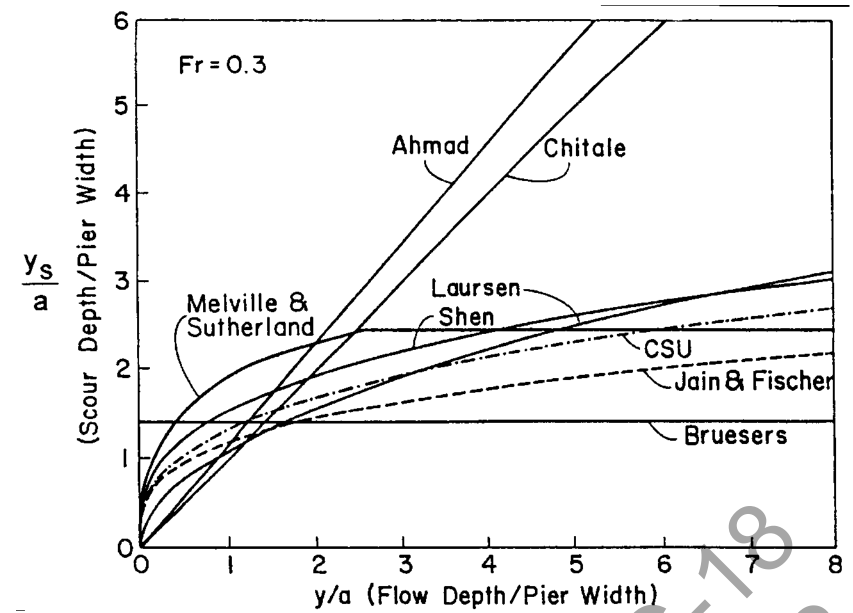


Figure 6.1. Comparison of scour equations for variable depth ratios (y/a) (after Jones).[46]

Mueller[49] compared 22 scour equations using field data collected by the USGS[50]. He concluded that the HEC-18 equation was good for design because it rarely under predicted measured scour depth. However, it frequently over-predicted the observed scour. The data contained 384 field measurements of scour at 56 bridges (Figure 6.2).

From laboratory data, Melville and Sutherland reported 2.4 as an upper limit for the depth of scour to pier width ratio (y_s/a) for cylindrical piers.[28] In these studies, the Froude Number was less than 1.0. Chang[51] also, noted that in all the data he studied, there were no values of the ratio of scour depth to pier width (y_s/a) larger than 2.3. However, values of y_s/a around 3.0 were obtained by Jain and Fischer for chute-and-pool flows with Froude Numbers as high as 1.5.[47] The largest value of y_s/a for antidune flow was 2.5 with a Froude Number of 1.2. These upper limits were derived for circular piers and were uncorrected for pier shape or for skew. Also, pressure flow, ice or debris can increase the ratio.

From the above discussion, the ratio of y_s/a can be as large as 3 at large Froude Numbers. Therefore, it is recommended that the maximum value of the ratio be taken as 2.4 for Froude Numbers less than or equal to 0.8 and 3.0 for larger Froude Numbers. These limiting ratio values apply only to round nose piers which are aligned with the flow.

6.2 LOCAL PIER SCOUR EQUATION

To determine pier scour, an equation based on the CSU equation is recommended for both live-bed and clear-water pier scour.[22] The equation predicts maximum pier scour depths. The equation is:

$$\frac{y_s}{y_1} = 2.0\, K_1\, K_2\, K_3\, K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43} \quad (6.1)$$

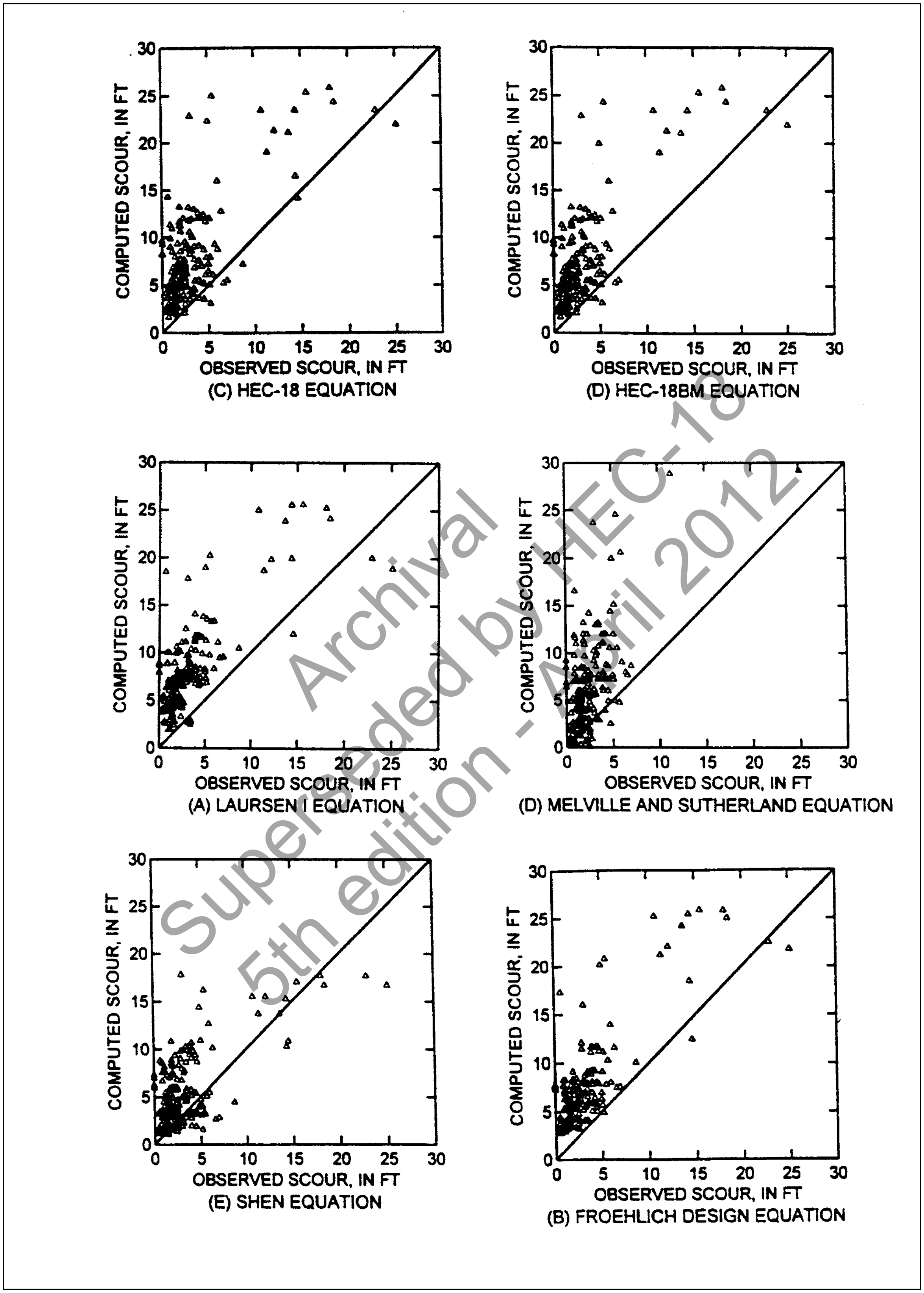


Figure 6.2. Comparison of scour equations with field scour measurements (after Mueller).[49]

As a Rule of Thumb, the maximum scour depth for round nose piers aligned with the flow is:

$y_s \leq$ 2.4 times the pier width (a) for Fr $\leq$ 0.8 (6.2)
$y_s \leq$ 3.0 times the pier width (a) for Fr > 0.8

In terms of y_s/a, Equation 6.1 is:

$$\frac{y_s}{a} = 2.0\, K_1\, K_2\, K_3\, K_4 \left(\frac{y_1}{a}\right)^{0.35} Fr_1^{0.43} \tag{6.3}$$

where:

y_s	=	Scour depth, m (ft)
y_1	=	Flow depth directly upstream of the pier, m (ft)
K_1	=	Correction factor for pier nose shape from Figure 6.3 and Table 6.1
K_2	=	Correction factor for angle of attack of flow from Table 6.2 or Equation 6.4
K_3	=	Correction factor for bed condition from Table 6.3
K_4	=	Correction factor for armoring by bed material size from Equation 6.5
a	=	Pier width, m (ft)
L	=	Length of pier, m (ft)
Fr_1	=	Froude Number directly upstream of the pier = $V_1/(gy_1)^{1/2}$
V_1	=	Mean velocity of flow directly upstream of the pier, m/s (ft/s)
g	=	Acceleration of gravity (9.81 m/s^2) (32.2 ft/s^2)

The correction factor, K_2, for angle of attack of the flow, θ, is calculated using the following equation:

$$K_2 = (\cos\theta + L/a \sin\theta)^{0.65} \tag{6.4}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 6.4 and Table 6.2. Table 6.2 illustrates the magnitude of the effect of the angle of attack on local pier scour.

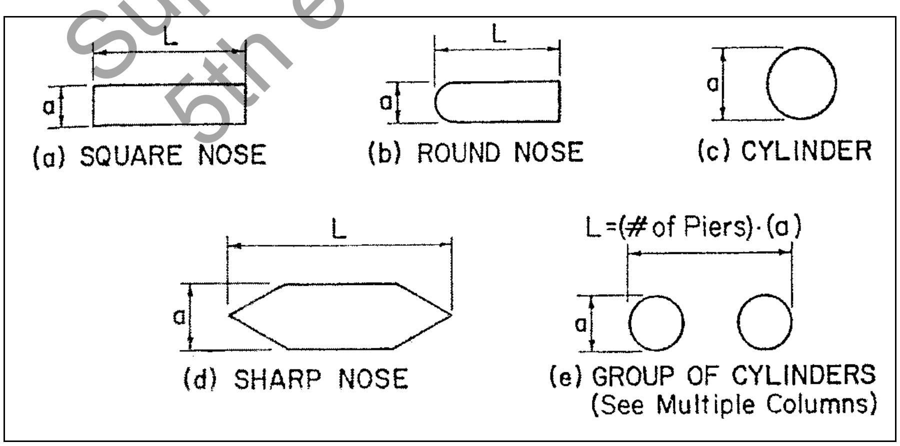


Figure 6.3. Common pier shapes.

Table 6.1. Correction Factor, K_1, for Pier Nose Shape.

Shape of Pier Nose	K_1
(a) Square nose	1.1
(b) Round nose	1.0
(c) Circular cylinder	1.0
(d) Group of cylinders	1.0
(e) Sharp nose	0.9

Table 6.2. Correction Factor, K_2, for Angle of Attack, θ, of the Flow.

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	5.0

Angle = skew angle of flow
L = length of pier, m

Table 6.3. Increase in Equilibrium Pier Scour Depths, K_3, for Bed Condition.

Bed Condition	Dune Height m	K_3
Clear-Water Scour	N/A	1.1
Plane bed and Antidune flow	N/A	1.1
Small Dunes	$3 > H \geq 0.6$	1.1
Medium Dunes	$9 > H \geq 3$	1.2 to 1.1
Large Dunes	$H \geq 9$	1.3

Notes:

1. The correction factor K_1 for pier nose shape should be determined using Table 6.1 for angles of attack up to 5 degrees. **For greater angles, K_2 dominates and K_1 should be considered as 1.0**. If L/a is larger than 12, use the values for L/a = 12 as a maximum in Table 6.2 and Equation 6.4.

2. The values of the correction factor K_2 should be applied only when the field conditions are such that the entire length of the pier is subjected to the angle of attack of the flow. Use of this factor will result in a significant over-prediction of scour if (1) a portion of the pier is shielded from the direct impingement of the flow by an abutment or another pier; or (2) an abutment or another pier redirects the flow in a direction parallel to the pier. For such cases, judgment must be exercised to reduce the value of the K_2 factor by selecting the effective length of the pier actually subjected to the angle of attack of the flow. **Equation 6.4 should be used for evaluation and design**. Table 6.2 is intended to illustrate the importance of angle of attack in pier scour computations and to establish a cutoff point for K_2 (i.e., a maximum value of 5.0).

3. The correction factor K_3 results from the fact that for plane-bed conditions, which is typical of most bridge sites for the flood frequencies employed in scour design, the maximum scour may be 10 percent greater than computed with Equation 6.1. In the **unusual** situation where a dune bed configuration **with large dunes** exists at a site during flood flow, the maximum pier scour may be 30 percent greater than the predicted equation value. This may occur on very large rivers, such as the Mississippi. For smaller streams that have a dune bed configuration at flood flow, the dunes will be smaller and the maximum scour may be only 10 to 20 percent larger than equilibrium scour. For antidune bed configuration the maximum scour depth may be 10 percent greater than the computed equilibrium pier scour depth.

4. Piers set close to abutments (for example at the toe of a spill through abutment) must be carefully evaluated for the angle of attack and velocity of the flow coming around the abutment.

The correction factor K_4 decreases scour depths for armoring of the scour hole for bed materials that have a D_{50} equal to or larger than 2.0 mm and D_{95} equal to or larger than 20 mm. The correction factor results from recent research by Molinas and Mueller. Molinas's research for FHWA showed that when the approach velocity (V_1) is less than the critical velocity (V_{c90}) of the D_{90} size of the bed material and there is a gradation in sizes in the bed material, the D_{90} will limit the scour depth.[30, 52] Mueller and Jones[53] developed a K_4 correction coefficient from a study of 384 field measurements of scour at 56 bridges. The equation developed by Jones[54] given in HEC-18 Third Edition should be replaced with the following:

- If $D_{50} < 2$ mm or $D_{95} < 20$ mm, then $K_4 = 1$
- If $D_{50} \geq 2$ mm and $D_{95} \geq 20$ mm

then:

$$K_4 = 0.4\,(V_R)^{0.15} \tag{6.5}$$

where:

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0 \tag{6.6}$$

and:

V_{icDx} = approach velocity (m/s or ft/sec) required to initiate scour at the pier for the grain size D_x (m or ft)

$$V_{icD_x} = 0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x} \tag{6.7}$$

V_{cDx} = critical velocity (m/s or ft/s) for incipient motion for the grain size D_x (m or ft)

$$V_{cD_x} = K_u\, y_1^{1/6}\, D_x^{1/3} \tag{6.8}$$

where:

y_1 = Depth of flow just upstream of the pier, excluding local scour, m (ft)
V_1 = Velocity of the approach flow just upstream of the pier, m/s (ft/s)
D_x = Grain size for which x percent of the bed material is finer, m (ft)
K_u = 6.19 SI Units
K_u = 11.17 English Units

While K_4 provides a good fit with the field data the velocity ratio terms are so formed that if D_{50} is held constant and D_{95} increases, the value of K_4 increases rather than decreases.[53] For field data an increase in D_{95} was always accompanied with an increase in D_{50}. **The minimum value of K_4 is 0.4**.

6.3 PIER SCOUR CORRECTION FACTOR FOR VERY WIDE PIERS

Flume studies on scour depths at wide piers in shallow flows and field observations of scour depths at bascule piers in shallow flows indicate that existing equations, including the CSU equation, overestimate scour depths. Johnson and Torrico[55] suggest the following equations for a K_w factor to be used to correct Equation 6.1 or 6.3 for wide piers in shallow flow. **The correction factor should be applied when the ratio of depth of flow (y) to pier width (a) is less than 0.8 (y/a < 0.8); the ratio of pier width (a) to the median diameter of the bed material (D_{50}) is greater than 50 (a/D_{50} > 50); and the Froude Number of the flow is subcritical**.

$$K_w = 2.58\left(\frac{y}{a}\right)^{0.34} Fr_1^{0.65} \quad \text{for } V / V_c < 1 \tag{6.9}$$

$$K_w = 1.0\left(\frac{y}{a}\right)^{0.13} Fr_1^{0.25} \quad \text{for } V / V_c \geq 1 \tag{6.10}$$

where:

K_w = Correction factor to Equation 6.1 or 6.3 for wide piers in shallow flow.
The other variables as previously defined.

Engineering judgment should be used in applying K_w because it is based on limited data from flume experiments. Engineering judgment should take into consideration the volume of traffic, the importance of the highway, cost of a failure (potential loss of lives and dollars) and the change in cost that would occur if the K_w factor is used.

6.4 SCOUR FOR COMPLEX PIER FOUNDATIONS

6.4.1 Introduction

As Salim and Jones[56,57,58] point out most pier scour research has focused on **solid piers** with limited attention to the determining scour depths for (1) pile groups, (2) pile groups and pile caps, or (3) pile groups, pile caps and solid piers exposed to the flow. The three types of exposure to the flow may be by design or by scour (long-term degradation, general (contraction) scour, and local scour, in addition to stream migration). In the general case, the flow could be obstructed by three substructural elements, herein referred to as the scour-producing components, which include the pier stem, the pile cap or footing, and the pile group. Nevertheless, ongoing research has determined methods and equations to determine scour depths for complex pier foundations. The results of this research are recommended for use and are given in the following sections. Physical Model studies are still recommended for complex piers with unusual features such as staggered or unevenly spaced piles or for major bridges where conservative scour estimates are not economically acceptable. However, the methods presented in this section provide a good estimate of scour for a variety of complex pier situations.

The steps listed below are recommended for determining the depth of scour for any combination of the three substructural elements exposed to the flow,[59] but engineering judgment is an essential element in applying the design graphs and equations presented in this section as well as in deciding when a more rigorous level of evaluation is warranted. Engineering judgment should take into consideration the volume of traffic, type of traffic (school bus, ambulance, fire trucks, local road, interstate, etc.), the importance of the highway, cost of a failure (potential loss of life and dollars) and the increase in cost that would occur if the most conservative scour depth is used. The stability of the foundation should be checked for:

- The scour depths should be determined for the 100-year flood or smaller discharge if it causes deeper scour and the superflood, i.e., the 500-year flood, as recommended in this manual.

- If needed use computer programs (HEC-RAS,[16, 17] WSPRO,[15] FESWMS,[45] etc.) to compute the hydraulic variables.

- Total scour depth is determined by separating the scour producing components, determining the scour depth for each component and adding the results. The method is called **"Superposition of the Scour Components."**

- Analyze the complex pile configuration to determine the components of the pier that are exposed to the flow or will be exposed to the flow which will cause scour.

- Determine the scour depths for each component exposed to the flow using the equations and methods presented in the following sections.

- Add the components to determine the total scour depths.

- Plot the scour depths and analyze the results using an interdisciplinary team to determine their reliability and adequacy for the bridge, flow and site conditions, safety and costs.

- Conduct a physical model study (Section 6.9) if engineering judgment determines it will reduce uncertainly, increase the safety of the design and/or reduce cost.

6.4.2 Superposition of Scour Components Method of Analysis

The components of a complex pier are illustrated in Figure 6.4.[59] This is followed by a definition of the variables. Note that the pile cap can be above the water surface, at the water surface, in the water or on the bed. The location of the pile cap may result from design or from long-term degradation and/or contraction scour. The pile group, as illustrated, is in uniform (lined up) rows and columns. This may not always be the case. The support for the bridge in many flow fields and designs may require a more complex arrangement of the pile group. In more complex pile group arrangements, the methods of analysis given in this manual may give smaller or larger scour depths.

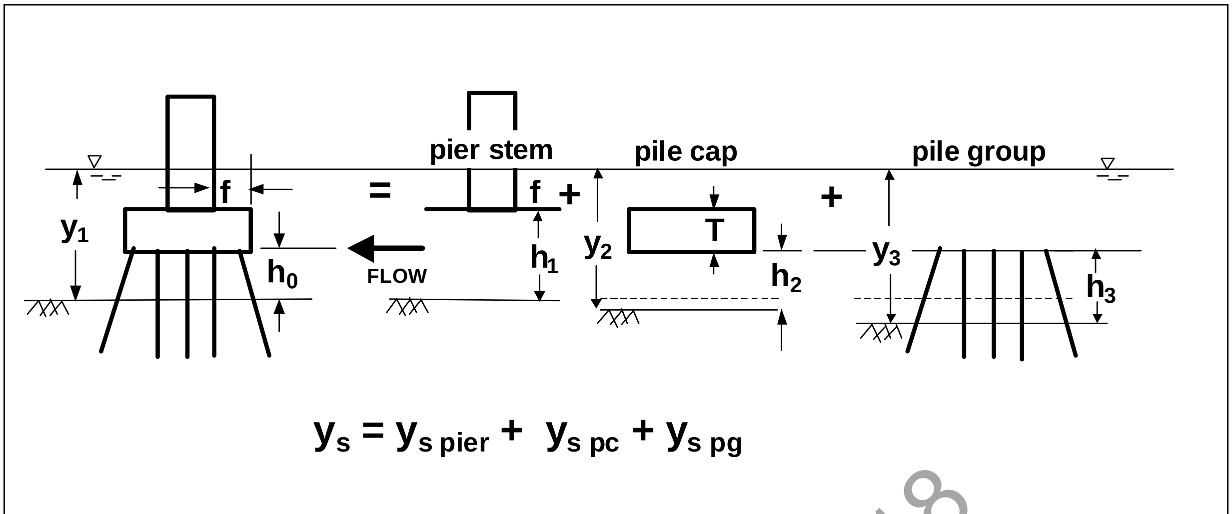


Figure 6.4. Definition sketch for scour components for a complex pier.(59)

The variables illustrated in Figure 6.4 and others used in computations are as follows:

f	=	Distance between front edge of pile cap or footing and pier, m (ft)
h_o	=	Height of the pile cap above bed at beginning of computation, m (ft)
h_1	=	$h_o + T$ = height of the pier stem above the bed before scour, m (ft)
h_2	=	$h_o + y_{s\ pier}/2$ = height of pile cap after pier stem scour component has been computed, m (ft)
h_3	=	$h_o + y_{s\ pier}/2 + y_{s\ pc}/2$ = height of pile group after the pier stem and pile cap scour components have been computed, m (ft)
S	=	Spacing between columns of piles, pile center to pile center, m (ft)
T	=	Thickness of pile cap or footing, m (ft)
y_1	=	Approach flow depth at the beginning of computations, m (ft)
y_2	=	$y_1 + y_{s\ pier}/2$ = adjusted flow depth for pile cap computations m (ft)
y_3	=	$y_1 + y_{s\ pier}/2 + y_{s\ pc}/2$ = adjusted flow depth for pile group computations, m (ft)
V_1	=	Approach velocity used at the beginning of computations, m/sec (ft/sec)
V_2	=	$V_1(y_1/y_2)$ =adjusted velocity for pile cap computations, m/sec (ft/sec)
V_3	=	$V_1(y_1/y_3)$ =adjusted velocity for pile group computations, m/sec (ft/sec)

Total scour from superposition of components is given by:

$$y_s = y_{s\ pier} + y_{s\ pc} + y_{s\ pg} \tag{6.11}$$

where:

y_s	=	Total scour depth, m (ft)
$y_{s\ pier}$	=	Scour component for the pier stem in the flow, m (ft)
$y_{s\ pc}$	=	Scour component for the pier cap or footing in the flow, m (ft)
$y_{s\ pg}$	=	Scour component for the piles exposed to the flow, m (ft)

Each of the scour components is computed from the basic pier scour Equation 6.1 using an equivalent sized pier to represent the irregular pier components, adjusted flow depths and velocities as described in the list of variables for Figure 6.4, and height adjustments for the pier stem and pile group. The height adjustment is included in the equivalent pier size for the pile cap. In the following sections guidance for calculating each of the components is given.

6.4.3 Determination of the Pier Stem Scour Depth Component

The need to compute the pier stem scour depth component occurs when the pier cap or the footing is in the flow and the pier stem is subjected to sufficient flow depth and velocity as to cause scour. The first computation is the scour estimate, $y_{s\,pier}$, for a full depth pier that has the width and length of the pier stem using the basic pier equation (Equation 6.1). In Equation 6.1, a_{pier} is the pier width and other variables in the equation are as defined previously. This base scour estimate is multiplied by $K_{h\,pier}$, given in Figure 6.5 as a function of h_1/a_{pier} and f/a_{pier}, to yield the pier stem scour component as follows:

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0 K_1 K_2 K_3 K_4 \left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{g y_1}}\right)^{0.43}\right] \tag{6.12}$$

where:

$K_{h\,pier}$ = Coefficient to account for the height of the pier stem above the bed and the shielding effect by the pile cap overhang distance "f" in front of the pier stem (from Figure 6.5)

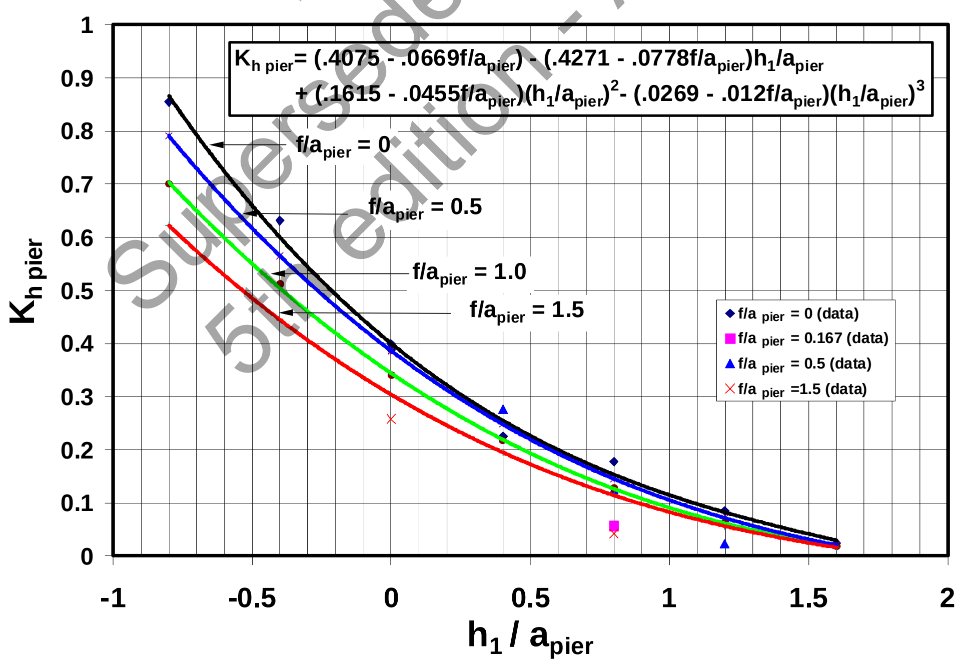


Figure 6.5. Suspended pier scour ratio.[59]

The quantity in the square brackets in Equation 6.12 is the basic pier scour ratio as if the pier stem were full depth and extended below the scour.

6.4.4 Determination of the Pile Cap (Footing) Scour Depth Component

The need to compute the pile cap or footing scour depth component occurs when the pile cap is in the flow by design, or as the result of long-term degradation, contraction scour, and/or by local scour attributed to the pier stem above it. As described below, there are two cases to consider in estimating the scour caused by the pile cap (or footing). Equation 6.1 is used to estimate the scour component in both cases, but the conceptual strategy for determining the variables to be used in the equation is different (partly due to limitations in the research that has been done to date). In both cases the wide pier factor, K_w, in Section 6.3 may be applicable for this computation.

<u>Case 1</u>: The bottom of the pile cap is above the bed and in the flow either by design or after the bed has been lowered by scour caused by the pier stem component. The strategy is to reduce the pile cap width, a_{pc}, to an equivalent full depth solid pier width, a^*_{pc}, using Figure 6.6. The equivalent pier width, an adjusted flow depth, y_2, and an adjusted flow velocity, V_2, are then used in Equation 6.1 to estimate the scour component.

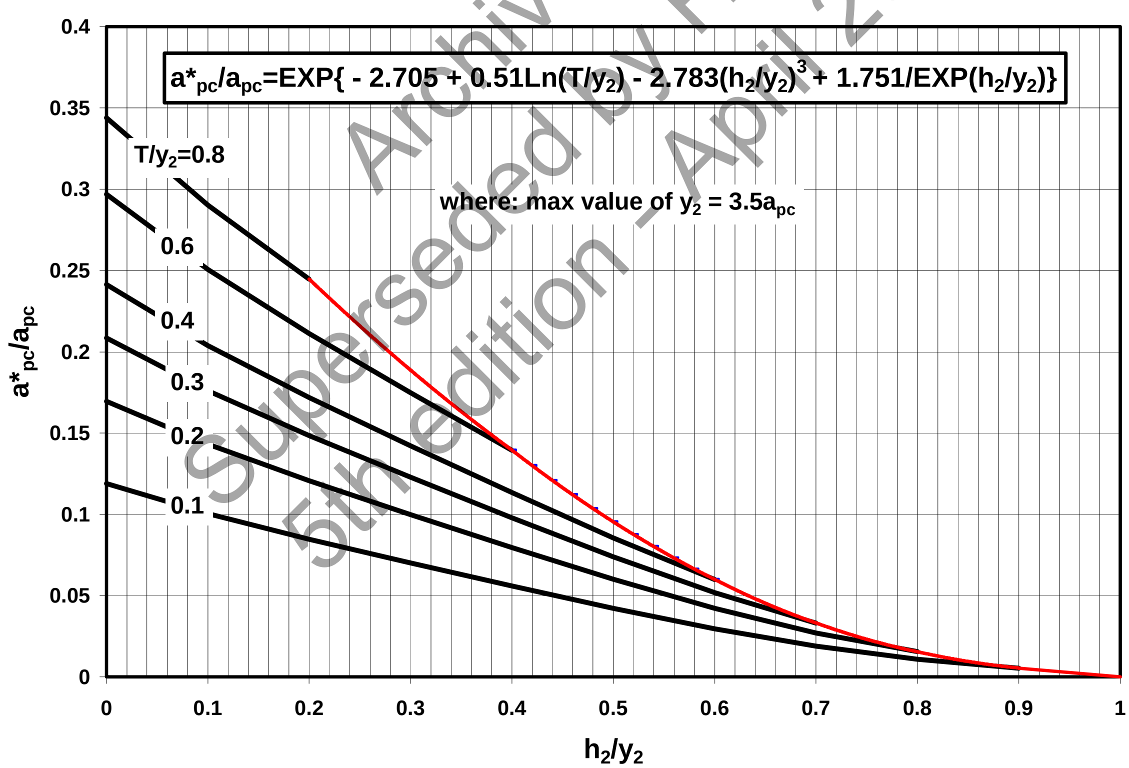


Figure 6.6. Pile cap (footing) equivalent width.(59)

Case 2: The bottom of the pile cap or footing is on or below the bed. The strategy is to treat the pile cap or exposed footing like a short pier in a shallow stream of depth equal to the height to the top of the footing above bed. The portion of the flow that goes over the top of the pile cap or footing is ignored. Then, the full pile cap width, a_{pc}, is used in the computations, but the exposed footing height, y_f, (in lieu of the flow depth), and the average velocity, V_f, in the portion of the profile approaching the footing are used in Equation 6.1 to estimate the scour component.

An inherent assumption in this second case is that the footing is deeper than the scour depth so it is not necessary to add the pile group scour as a third component in this case. If the bottom of the pile cap happens to be right on the bed, either the case 1 or case 2 method could be applied, but they won't necessarily give the same answers. If both methods are tried, then engineering judgment should dictate which one to accept.

Details for determining the pile cap or footing scour component for these two cases are described in the following paragraphs.

Case 1. Bottom of the Pile Cap (Footing) in the Flow above the Bed

T = Thickness of the pile cap exposed to the flow, m (ft)
h_2 = $h_o + y_{s\,pier}/2$, m (ft)
y_2 = $y_1 + y_{s\,pier}/2$, = adjusted flow depth, m (ft)
V_2 = $V_1(y_1/y_2)$ = adjusted flow velocity, m/s (ft/s)

where:

h_o = Original height of the pile cap above the bed, m (ft)
y_1 = Original flow depth at the beginning of the computations before scour, m (ft)
$y_{s\,pier}$ = Pier stem scour depth component, m (ft)
V_1 = Original approach velocity at the beginning of the computations, m/s (ft/s)

Determine a^*_{pc}/a_{pc} from Figure 6.6 as a function of h_2/y_2 and T/y_2 (note that the maximum value of $y_2 = 3.5\ a_{pc}$).

Compute $a^*_{pc} = (a^*_{pc}/a_{pc})\ a_{pc}$; where a^*_{pc} is the width of the equivalent pier to be used in Equation 6.1 and a_{pc} is the width of the original pile cap. Compute the pile cap scour component, $y_{s\,pc}$ from Equation 6.1 using a^*_{pc}, y_2, and V_2 as the pier width, flow depth, and velocity parameters, respectively. The rationale for using the adjusted velocity for this computation is that the near bottom velocities are the primary currents that produce scour and they tend to be reduced in the local scour hole from the overlying component. **For skewed flow use the L/a for the original pile cap as the L/a for the equivalent pier to determine K_2.** Apply the wide pier correction factor, K_w, if (1) the total depth, $y_2 < 0.8\ a^*_{pc}$, (2) the Froude Number $V_2/(g\ y_2)^{1/2} < 1$, and (3) $a^*_{pc} > 50\ D_{50}$. The scour component equation for the case 1 pile cap can then be written:

$$\frac{y_{s\,pc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43} \quad (6.13)$$

Next, the pile group scour component should be computed. This is discussed in Section 6.4.5.

Case 2. Bottom of the Pile Cap (Footing) Located On or Below the Bed.

One limitation of the procedure described above is that the design chart in Figure 6.6 has not been developed for the case of the bottom of the pile cap or footing being below the bed (i.e., negative values of h_2). In this case, use a modification of the exposed footing procedure that has been described in previous editions of HEC-18. The previous procedure was developed from experiments in which the footing was never undermined by scour and tended to be an over predictor if the footing is undermined.

As for case 1:

$$y_2 = y_1 + y_{s\,pier}/2, \text{ m (ft)}$$
$$V_2 = V_1(y_1/y_2), \text{ m/s (ft/s)}$$

The average velocity of flow at the exposed footing (V_f) is determined using the following equation:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93\dfrac{y_f}{k_s}+1\right)}{\ln\left(10.93\dfrac{y_2}{k_s}+1\right)} \tag{6.14}$$

where:

- V_f = Average velocity in the flow zone below the top of the footing, m/s (ft/s)
- V_2 = Average adjusted velocity in the vertical of flow approaching the pier, m/s (ft/s)
- ln = Natural log to the base e
- y_f = $h_1 + y_{s\,pier}/2$ = distance from the bed (after degradation, contraction scour, and pier stem scour) to the top of the footing, m (ft)
- k_s = Grain roughness of the bed (normally taken as the D_{84} for sand size bed material and 3.5 D_{84} for gravel and coarser bed material), m (ft)
- y_2 = Adjusted depth of flow upstream of the pier, including degradation, contraction scour and half the pier stem scour, m (ft)

See Figure 6.7 for an illustration of variables.

Compute the pile cap scour depth component, $y_{s\,pc}$ from Equation 6.1 using the full pile cap width, a_{pc}, y_f, V_f as the width, flow depth, and velocity parameters, respectively. The wide pier factor K_w in Section 6.3 should be used in this computation if (1) the total depth $y_2 < 0.8\,a_{pc}$, (2) the Froude Number $V_2/(gy_2)^{1/2} < 1$, and (3) $a_{pc} > 50\,D_{50}$. Use y_2/a_{pc} to compute the K_w factor if it is applicable. The scour component equation for the case 2 pile cap or footing can then be written:

$$\frac{y_{s\,pc}}{y_f} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a_{pc}}{y_f}\right)^{0.65}\left(\frac{V_f}{\sqrt{g\,y_f}}\right)^{0.43} \tag{6.15}$$

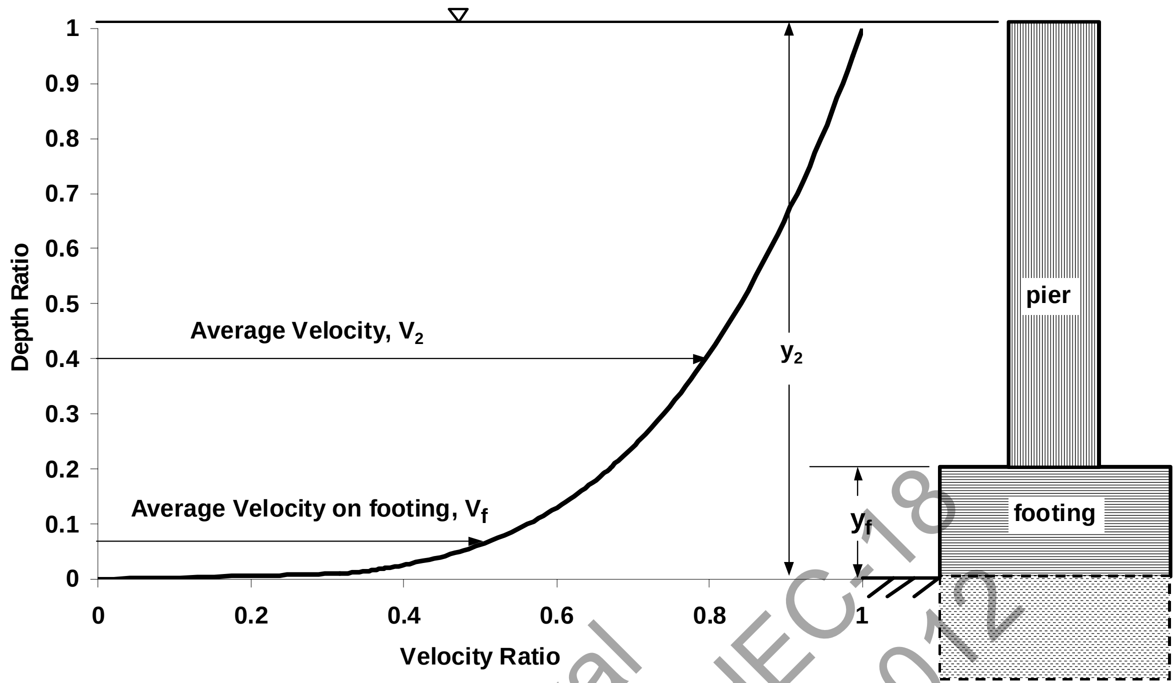


Figure 6.7. Definition sketch for velocity and depth on exposed footing.

In this case assume the pile cap scour component includes the pile group scour and compute the total scour depth as:

$$y_s = y_{s\,pier} + y_{s\;pc} \qquad \text{(For case 2 only)} \qquad (6.16)$$

In earlier editions of HEC-18, the recommendation was to use the larger of the exposed footing scour estimate or the pier stem scour estimate, treating the pier stem portion as a full depth pier that extended below the scour depth. **Now the recommendation is to add the components using a more realistic estimate of the pier stem component and using an adjusted approach velocity, V_2, to calculate V_f and the wide pier correction in the computations for the exposed footing component.**

6.4.5 Determination of the Pile Group Scour Depth Component

Research by Salim and Jones(56,57,58,60) and by Smith(61) has provided a basis for determining pile group scour depth by taking into consideration the spacing between piles, the number of pile rows and a height factor to account for the pile length exposed to the flow. Guidelines are given for analyzing the following typical cases:

- Special case of piles aligned with each other and with the flow. No angle of attack.

- General case of the pile group skewed to the flow, with an angle of attack, or pile groups with staggered rows of piles.

The strategy for estimating the pile group scour component is the same for both cases, but the technique for determining the projected width of piles is simpler for the special case of aligned piles. The strategy is as follows:

- Project the width of the piles onto a plane normal to the flow.
- Determine the effective width of an equivalent pier that would produce the same scour if the pile group penetrated the water surface.
- Adjust the flow depth, velocity and exposed height of the pile group to account for the pier stem and pile cap scour components previously calculated.
- Determine the pile group height factor based on the exposed height of the pile group above the bed.
- Compute the pile group scour component using a modified version of Equation 6.1.

<u>Projected width of piles</u>

For the special case of aligned piles, the projected width, a_{proj}, onto a plane normal to the flow is simply the width of the collapsed pile group as illustrated in Figure 6.8.

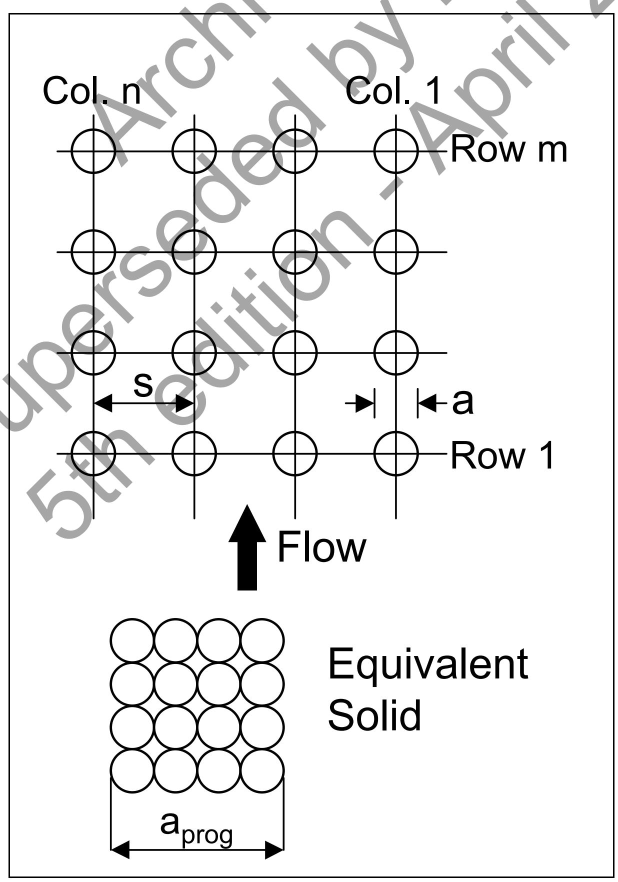


Figure 6.8. Projected width of piles for the special case of aligned flow.

For the general case, Smith[61] determined that a pile group could be represented by an equivalent solid pier that has an effective width, a^*_{pg}, equal to a spacing factor multiplied by the sum of the non-overlapping projected widths of the piles onto a plane normal to the flow direction. The aligned pile group is a special case in which the sum of the non-overlapping projected widths happens to be the same as the width of the collapsed pile group. The procedure for the general case is the same as the procedure for the aligned pile groups except for the determination of the width of the equivalent solid which is a more tedious process for the general case. The sum of the projected widths can be determined by sketching the pile group to scale and projecting the outside edges of each pile onto the projection plane as illustrated in Figure 6.9 or by systematically calculating coordinates of the edges of each pile along the projection plane. The coordinates are sorted in ascending order to facilitate inspection to eliminate double counting of overlapping areas. Additional experiments are being conducted at the FHWA hydraulics laboratory to test simpler techniques for estimating the effective width, but currently Smith's summation technique is a logical choice.

Smith attempted to derive weighting factors to adjust the impact of piles according to their distance from the projection plane, but concluded that there was not enough data and the procedure would become very cumbersome with weighting factors. **A reasonable alternative to using weighting factors is to exclude piles other than the two rows and one column closest to the plane of projection as illustrated by the bold outlines in Figure 6.9**.

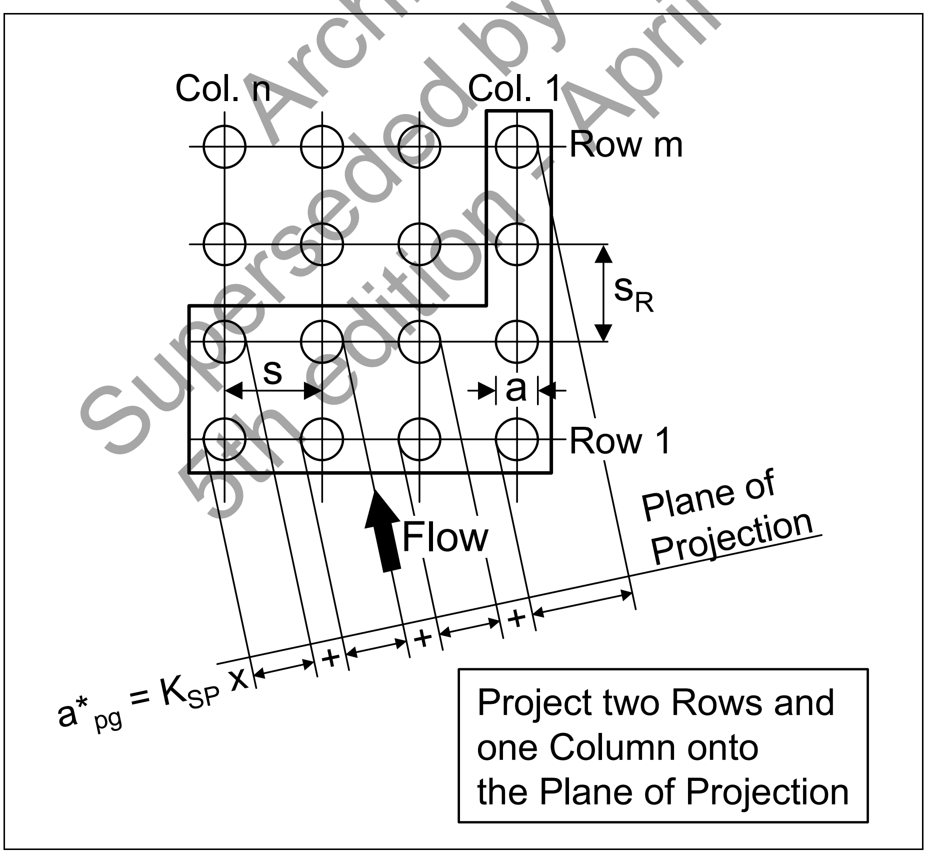


Figure 6.9. Projected width of piles for the general case of skewed flow.

Effective width of an equivalent full depth pier

The effective width of an equivalent full depth pier is the product of the projected width of piles multiplied by a spacing factor and a number of aligned rows factor (used for the special case of aligned piles only).

$$a^*_{pg} = a_{proj} K_{sp} K_m \tag{6.17}$$

where:

a_{proj}	=	Sum of non-overlapping projected widths of piles (see Figures 6.8 and 6.9)
K_{sp}	=	Coefficient for pile spacing (Figure 6.10)
K_m	=	Coefficient for number of aligned rows, m, (Figure 6.11 - note that K_m is constant for all S/a values when there are more than 6 rows of piles)
K_m	=	1.0 for skewed or staggered pile groups

The number of rows factor, K_m, is 1.0 for the general case of skewed or staggered rows of piles because the projection technique for skewed flow accounts for the number of rows and is already conservative for staggered rows.

Adjusted flow depth and velocity

The adjusted flow depth and velocity to be used in the pier scour equation are as follows:

$$y_3 = y_1 + y_{s\ pier}/2 + y_{s\ pc}/2, \text{ m (ft)} \tag{6.18}$$

$$V_3 = V_1 (y_1/y_3), \text{ m/s (ft/s)} \tag{6.19}$$

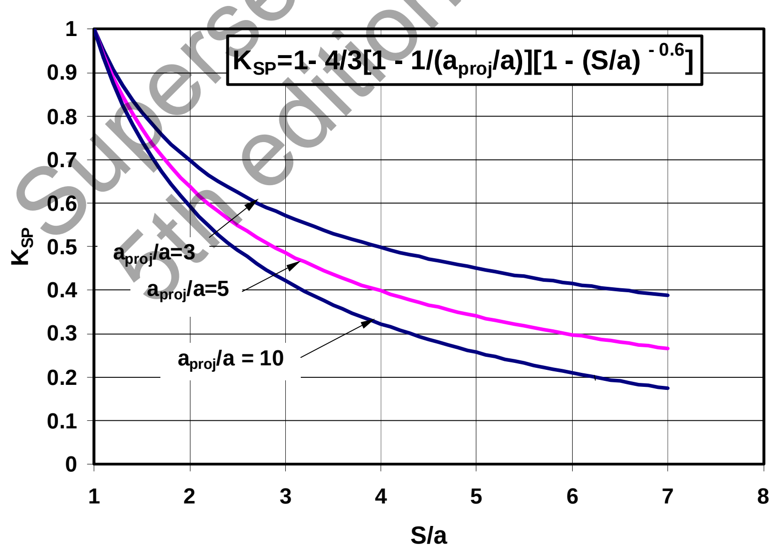


Figure 6.10. Pile spacing factor (refer to Sheppard).[62]

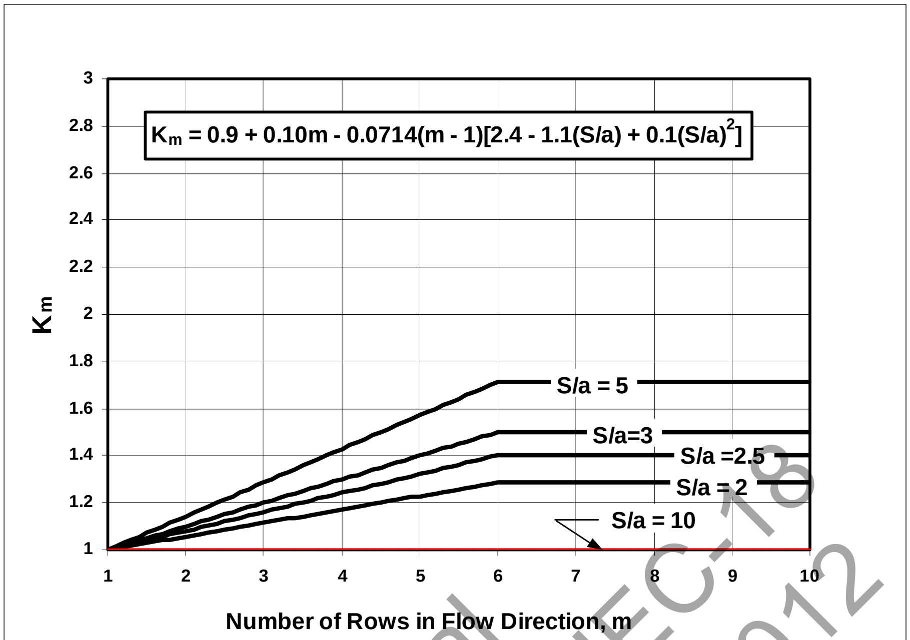


Figure 6.11. Adjustment factor for number of aligned rows of piles (refer to Sheppard).[62]

The scour equation for a pile group can then be written as follows:

$$\frac{Y_{s\,pg}}{y_3} = K_{h\,pg}\left[2.0K_1K_3K_4\left(\frac{a^*_{pg}}{y_3}\right)^{0.65}\left(\frac{V_3}{\sqrt{g y_3}}\right)^{0.43}\right] \tag{6.20}$$

where:

$K_{h\,pg}$ = Pile group height factor given in Figure 6.12 as a function of h_3/y_3 (note that the maximum value of $y_3 = 3.5\ a^*_{pg}$)

h_3 = $h_0 + y_{s\,pier}/2 + y_{s\ pc}/2$ = height of pile group above the lowered stream bed after pier and pile cap scour components have been computed, m, (ft)

K_2 from Equation 6.1 has been omitted because pile widths are projected onto a plane that is normal to the flow. The quantity in the square brackets is the scour ratio for a solid pier of width, a^*_{pg}, if it extended to the water surface. This is the scour ratio for a full depth pile group.

6.4.6 Determination of Total Scour Depth for the Complex Pier

The total scour for the complex pier from Equation (6.11) is:

$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$

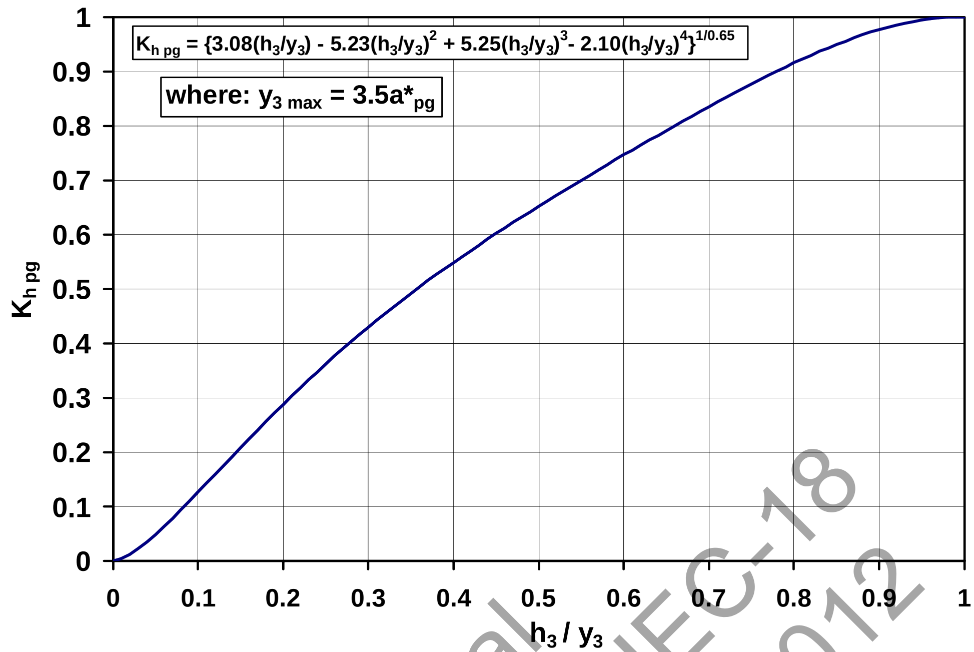


Figure 6.12. Pile group height adjustment factor (refer to Sheppard).[62]

The guidelines described in this section can be used to compute scour for a simple full depth pile group in which case the first two components will be zero and the pile group height factor will be 1.0. Engineering judgment must be used if debris is considered a factor in which case it would be logical to treat the pile group and debris as a vertical extension of the pile cap and to compute scour using the case 2 pile cap procedure described previously.

In cases of complex pile configurations where costs are a major concern, where significant savings are anticipated, and/or for major bridge crossings, physical model studies are still the best guide. Nevertheless, the guidelines described in this section provide a first estimate and a good indication of what can be anticipated from a physical model study.

In many complex piers, the pile groups have a different number of piles in a row or column, the spacing between piles is not uniform, and the widths of the piles may not all be the same. An estimate of the scour depth can be obtained using the methods and equations in this section. However, again it is recommended that a physical model study be conducted to arrive at the final design and to determine the scour depths.

6.5 MULTIPLE COLUMNS SKEWED TO THE FLOW

For multiple columns (illustrated as a group of cylinders in Figure 6.13) skewed to the flow, the scour depth depends on the spacing between the columns. The correction factor for angle of attack would be smaller than for a solid pier. Raudkivi in discussing effects of alignment states "...the use of cylindrical columns would produce a shallower scour; for example, with five-diameter spacing the local scour can be limited to about 1.2 times the local scour at a single cylinder."[26]

In application of Equation 6.1 with multiple columns spaced less than 5 pier diameters apart, the pier width 'a' is the total projected width of all the columns in a single bent, normal to the flow angle of attack (Figure 6.13). For example, three 2.0 m (6.6 ft) cylindrical columns spaced at 10.0 m (33 ft) would have an 'a' value ranging between 2.0 and 6.0 m (6.6 and 33 ft), depending upon the flow angle of attack. **This composite pier width would be used in Equation 6.1 to determine depth of pier scour.** The correction factor K_1 in Equation 6.1 for the multiple column would be 1.0 regardless of column shape. The coefficient K_2 would also be equal to 1.0 since the effect of skew would be accounted for by the projected area of the piers normal to the flow.

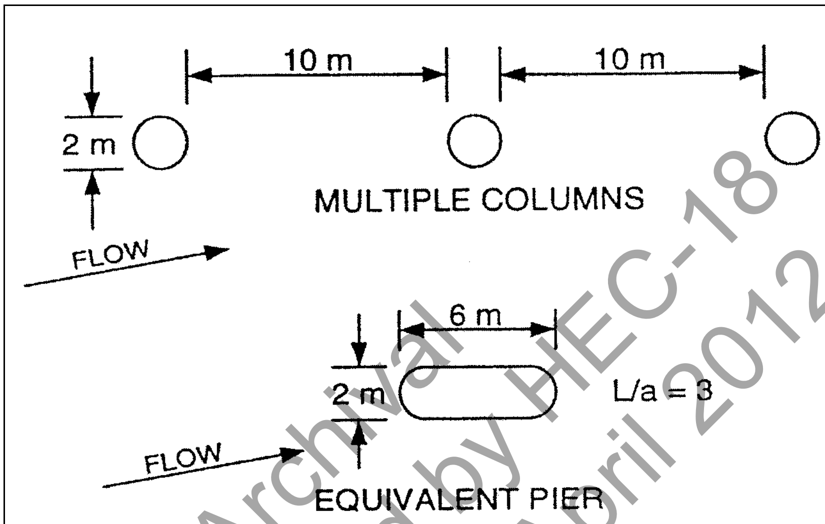


Figure 6.13. Multiple columns skewed to the flow.

The scour depth for multiple columns skewed to the flow can also be determined by determining the K_2 factor using Equation 6.4 and using it in Equation 6.1. The width "a' in Equation 6.1 would be the width of a single column. An example problem illustrates all three methods of obtaining the scour depth for multiple columns.

If the multiple columns are spaced 5 diameter or greater apart; and debris is not a problem, limit the scour depths to a maximum of 1.2 times the local scour of a single column.

The depth of scour for a multiple column bent will be analyzed in this manner except when addressing the effect of debris lodged between columns. If debris is evaluated, it would be logical to consider the multiple columns and debris as a solid elongated pier. The appropriate L/a value and flow angle of attack would then be used to determine K_2 in Equation 6.4.

Additional laboratory studies are necessary to provide guidance on the limiting flow angles of attack for given distance between multiple columns beyond which multiple columns can be expected to function as solitary members with minimal influence from adjacent columns.

6.6 PRESSURE FLOW SCOUR

Pressure flow, which is also denoted as orifice flow, occurs when the water surface elevation at the upstream face of the bridge is greater than or equal to the low chord of the bridge superstructure (Figure 6.14). Pressure flow under the bridge results from a pile up of water on the upstream bridge face, and a plunging of the flow downward and under the bridge. At higher approach flow depths, the bridge can be entirely submerged with the resulting flow being a complex combination of the plunging flow under the bridge (orifice flow) and flow over the bridge (weir flow).

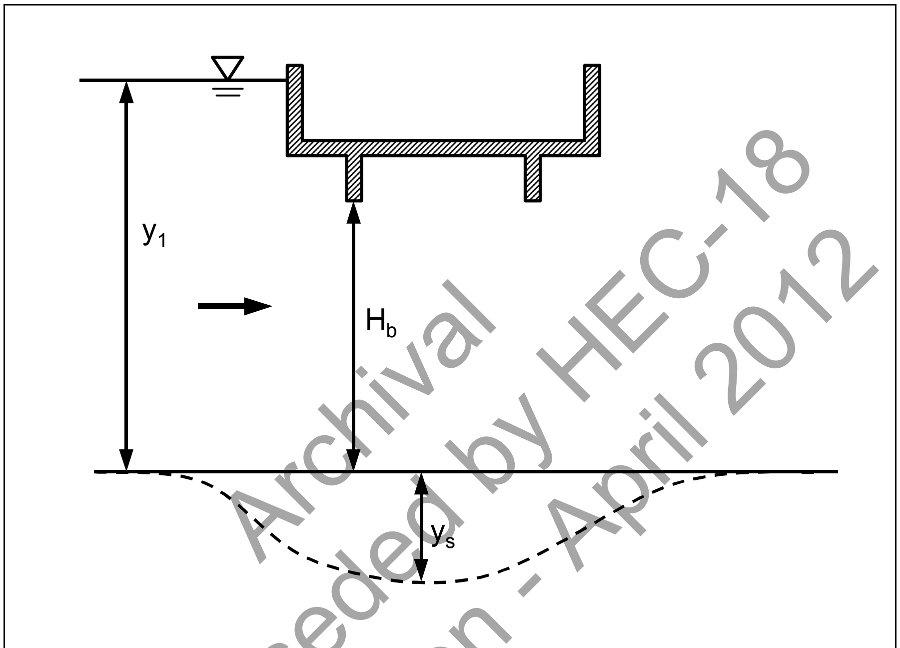


Figure 6.14. Definition sketch of vertical contraction scour resulting from pressure flow.

In many cases, when a bridge is submerged, flow will also overtop adjacent approach embankments. This highway approach overtopping is also weir flow. Hence, for any overtopping situation the total weir flow can be subdivided into weir flow over the bridge and weir flow over the approach. Weir flow over approach embankments serves to reduce the discharge which must pass either under or over the bridge. In some cases, when the approach embankments are lower than the low chord of the bridge, the relief obtained from overtopping of the approach embankments will be sufficient to prevent the bridge from being submerged.

The hydraulic bridge computer models WSPRO or HEC-RAS are suitable for determination of the amount of flow which will flow over the roadway embankment, over the bridge as weir flow, and through the bridge opening as orifice flow, provided that the top of the highway is properly included in the input data.[15, 16, 17] These models can be used to determine average flow depths and velocities over the road and bridge, as well as average velocities under the bridge. **It is recommended that one of these models be used to analyze the scour problem when the bridge is overtopped with or without overtopping of the approach roadway.**

With pressure flow, the local scour depths at a pier or abutment can be much larger than for free surface flow with similar depths and approach velocities. The increase in local scour at a pier subjected to pressure flow results from the flow being directed downward towards the bed by the superstructure (vertical contraction of the flow) and by increasing the intensity of the horseshoe vortex. The vertical contraction of the flow can be a more significant cause of the increased scour depth. However, in many cases, when a bridge becomes submerged, the average velocity under the bridge is reduced due to a combination of additional backwater caused by the bridge superstructure impeding the flow, and a reduction of the discharge which must pass under the bridge due to weir flow over the bridge and/or approach embankments. **As a consequence of this, increases in local scour attributed to pressure flow scour at a particular site, may be offset to a degree by lower velocities through the bridge opening due to increased backwater and a reduction in discharge under the bridge due to overtopping of the bridge and approach embankments.**

Limited studies of pressure flow scour have been made in flumes at Colorado State University and FHWA's Turner Fairbank Highway Research Center which indicate that pier scour can be increased 200 to 300 percent by pressure flow.[63, 64, 65] Both studies were for clear-water scour (no transport of bed material upstream of the bridge). Arneson[66] conducted a more extensive study of pressure flow scour under live bed conditions. FHWA's Turner Fairbank Laboratory and Arneson's study concluded that (1) pressure flow scour is a combination of vertical contraction scour and local pier scour, (2) the local pier scour component was approximately the same as the free-surface local pier scour measurements for the same approach flow condition, and 3) the two components were additive. Arneson's equation, derived from multiple linear regression of his data, for bed vertical contraction scour is:

$$\frac{y_s}{y_1} = -5.08 + 1.27\left(\frac{y_1}{H_b}\right) + 4.44\left(\frac{H_b}{y_1}\right) + 0.19\left(\frac{V_a}{V_c}\right) \qquad (6.21)$$

where:

- y_s = Depth of vertical contraction scour relative to mean bed elevation, m (ft)
- y_1 = Depth of flow immediately upstream of the bridge, m (ft)
- H_b = Distance from the low chord of the bridge to the average elevation of the stream bed before scour, m (ft)
- V_a = Average velocity of the flow through the bridge opening before scour occurs, m/s (ft/s)
- V_c = Critical velocity of the D_{50} of the bed material in the bridge opening, m/s (ft/s)

The procedure for calculating pier scour for pressure flow is as follows:

a. Determine the flow variables using a 1-dimensional or 2-dimensional computer model such as WSPRO, HEC-RAS, FESWMS, or RMA-2.

b. Calculate the critical velocity V_c of the D_{50} of the bed material in the bridge opening.

c. Use the flow variables and critical velocity to compute the vertical contraction scour (Equation 6.21).

d. Use the flow variables to compute the local pier scour using Equations 6.1 or 6.3 and the other procedures presented in previous sections.

e. Add the scour components obtained in c and d to obtain the local pier scour for pressure flow.

f. Use engineering judgment to evaluate the local pressure flow pier scour .

6.7 SCOUR FROM DEBRIS ON PIERS

Debris lodged on a pier can increase local scour at a pier. The debris may increase pier width and deflect a component of flow downward. This increases the transport of sediment out of the scour hole. When floating debris is lodged on the pier, the scour depth can be estimated by assuming that the pier width is larger than the actual width. The problem is in determining the increase in pier width to use in the pier scour equation. Furthermore, at large depths, the effect of the debris on scour depth should diminish (for additional discussion, see HEC-20[6]).

As with estimating local scour depths with pressure flow, only limited research has been done on local scour with debris. Melville and Dongol have conducted a limited quantitative study of the effect of debris on local pier scour and have made some recommendations which support the approach suggested above.[67] However, additional laboratory studies will be necessary to better define the influence of debris on local scour.

An interim procedure for estimating the effect of debris on local scour at piers is presented in Appendix D.

6.8 TOPWIDTH OF SCOUR HOLES

The topwidth of a scour hole in cohesionless bed material from one side of a pier or footing can be estimated from the following equation:(68)

$$W = y_s\ (K + \text{Cot}\ \theta) \tag{6.22}$$

where:

W = Topwidth of the scour hole from each side of the pier or footing, m
y_s = Scour depth, m (ft)
K = Bottom width of the scour hole related to the of scour depth
θ = Angle of repose of the bed material ranging from about 30° to 44°

The angle of repose of cohesionless material in air ranges from about 30° to 44°. Therefore, if the bottom width of the scour hole is equal to the depth of scour y_s (K = 1), the topwidth in cohesionless sand would vary from 2.07 to 2.80 y_s. At the other extreme, if K = 0, the topwidth would vary from 1.07 to 1.8 y_s. Thus, the topwidth could range from 1.0 to 2.8 y_s and depends on the bottom width of the scour hole and composition of the bed material. In general, the deeper the scour hole, the smaller the bottom width. In water, the angle of repose of cohesionless material is less than the values given for air; therefore, a topwidth of 2.0 y_s is suggested for practical applications (Figure 6.15).

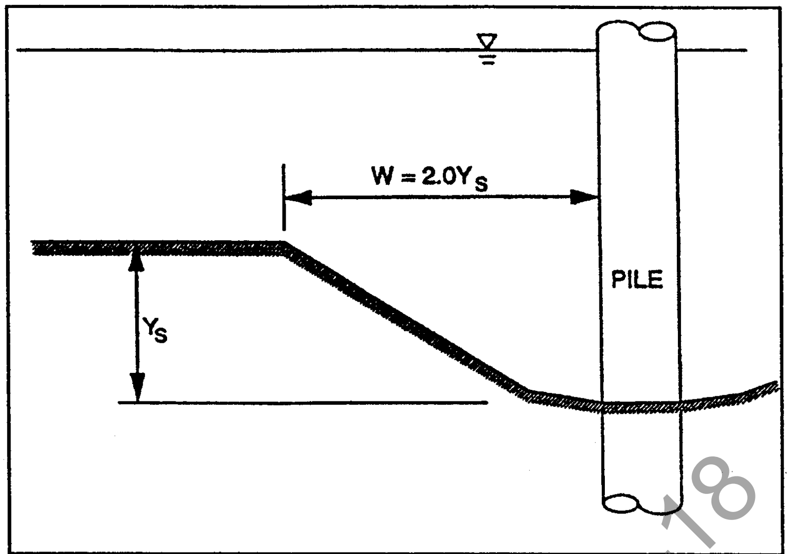


Figure 6.15. Topwidth of scour hole.

6.9 PHYSICAL MODEL STUDIES

For unusual or complex pier foundation configurations a physical model study should be made. The scale between model and prototype is based on the Froude criteria, that is, the Froude number for the model should be the same as for the prototype. In general it is not possible to scale the bed material size. Also, at flood flows in sand bed streams the sediment transport conditions will be live-bed and the bed configuration will be plane bed. Whereas, in the model live-bed transport conditions will be ripples or dunes. These are incomparable pier scour conditions. Therefore, it is recommended that a bed material be used that has a critical velocity just below the model velocity (i.e., clear-water scour conditions). This will usually give the maximum scour depth; but a careful study of the results needs to be made by persons with field and model scour experience. For additional discussion of the use of physical modeling in hydraulic design, see HEC-23.[7]

6.10 PIER SCOUR EXAMPLE PROBLEMS (SI)

6.10.1 Example Problem 1 - Scour at a Simple Solid Pier (SI)

Given:

Pier geometry: a = 1.22 m , L = 18 m, round nose
Flow variables: y_1 = 3.12 m, V_1 =3.36 m/s
Angle of attack = 0 degrees, g = 9.81 m/s^2
Froude No. = $3.36/(9.81 \times 3.12)^{0.5} = 0.61$
Bed material: D_{50} = 0.32 mm, D_{95} = 7.3 mm
Bed Configuration: Plane bed.

Determine:

The magnitude of pier scour depth.

Solution:

Use Equation 6.1.

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$y_s\ /\ 3.12 = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \times (1.22\ /\ 3.12)^{0.65} \times 0.61^{0.43} = 0.97$$

$$y_s = 0.97 \times 3.12 = 3.03\,m$$

6.10.2 Example Problem 2 - Angle of Attack (SI)

Given:

Same as Problem 1 but angle of attack is 20 degrees

Solution:

Use Equation 6.4 to compute K_2

$$K_2 = (Cos\theta + L\ /\ a\ Sin\theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 6.4 (see Table 6.2).

L/a = 18 /1.22 = 14.8 > 12 use 12

K_2 = (Cos 20 + 12 Sin 20)$^{0.65}$ = 2.86

y_s = 3.03 X 2.86 = 8.7 m

6.10.3 Example Problem 3 - Coarse Bed Material (SI)

Given:

Same as Problem 1 but the bed material is coarser
Bed material: D_{50} = 17.8 mm, D_{95} = 96.3 mm
Bed configuration: Plane Bed

Determine:

If the coarse bed material would decrease local scour depth. Determine K_4 and y_s.

Solution:

Use Equations 6.5, 6.6, 6.7, and 6.8

K_4 = 1 if D_{50} <2 mm or D_{95} < 20 mm

If $D_{50} \geq$ 2 mm and $D_{95} \geq$ 20 mm

then:

$$K_4 = 0.4\,(V_R)^{0.15}$$

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0$$

where:

V_{icDx} = Approach velocity required to initiate scour at the pier for the grain size D_x, m/s

$$V_{icDx} = 0.645 \left(\frac{D_x}{a}\right)^{0.053} V_{cD_x}$$

V_{cDX} = Critical velocity for incipient motion for the grain size D_x ,m/s

$$V_{cDX} = 6.19\ y_1^{1/6}\ D_x^{1/3}$$

$$V_{cD50} = 6.19\,(3.12)^{1/6}\,(0.0178)^{1/3} = 1.95 \text{ m/s}$$

$$V_{cD95} = 6.19\,(3.12)^{1/6}\,(0.0963)^{1/3} = 3.43 \text{ m/s}$$

$$V_{icD50} = 0.645\,(0.0178 / 1.22)^{0.053}\,(1.95) = 1.01 \text{ m/s}$$

$$V_{icD95} = 0.645\,(0.0963 / 1.22)^{0.053}\,(3.43) = 1.93 \text{ m/s}$$

$$V_R = \frac{(3.36 - 1.01)}{(1.95 - 1.93)} = 117.5$$

$$K_4 = 0.4\,(117.5)^{0.15} = 0.82$$

$$y_s = 0.82 \times 3.03 = 2.48 \text{ m}$$

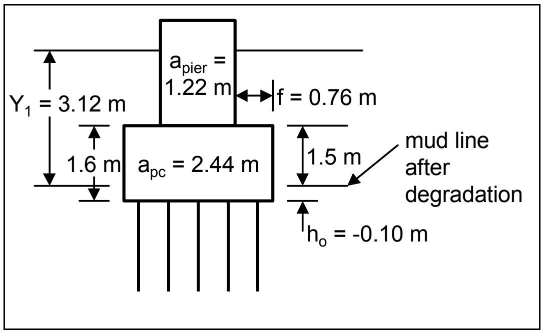
6.10.4 Example Problem 4 - Scour at Complex Piers (Solid Pier on an Exposed Footing)(SI)

Given:

The pier in Problem 1 (Section 6.10.1) is on a 2.44 m wide by 1.60 m high by 19.81 m long rectangular footing. Footing extends 0.76 m upstream from the pier stem. The footing is on an unspecified pile foundation. The footing is exposed 1.50 m by long-term degradation. Determine the local scour.

Pier geometry: a_{pier} = 1.22 m, L= 18 m, round nose
Pile cap or footing geometry: a_{pc} (or a_f) = 2.44 m, L = 19.81 m, T = 1.60 m, f = 0.76 m
Approach flow: y_1 = 3.12 m, V_1 = 3.36 m/s
Angle of attack: 0 degrees

Froude No. = $3.36/(9.81x3.12)^{0.5} = 0.61$
Bed material: $D_{50} = 0.32$ mm, $D_{84} = 7.3$ mm, plane bed
See sketch below:



Local Scour from Pier Stem

f = 0.76 m
$h_1 = h_0 + T = -0.10 + 1.60 = 1.50$ m
$K_{h\ pier}$ = function (h_1/a_{pier}, f/a_{pier}) (from Figure 6.5)
$h_1/a_{pier} = 1.5/1.22 = 1.23$
$f/a_{pier} = 0.76/1.22 = 0.62$
$K_{h\ pier} = 0.06$

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{g\,y_1}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pier}}{y_1} = 0.06\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{1.22}{3.12}\right)^{0.65}\left(\frac{3.36}{\sqrt{9.81\times 3.12}}\right)^{0.43}\right]$$

$y_{s\ pier} = 0.06x[0.97]x3.12 = 0.18$ m

Note: the quantity in the square brackets is the scour ratio for a full depth pier.

Local Scour from the Pile Cap or Footing

Assume the average bed elevation in the vicinity of the pier lowers by ½ the pier stem scour.

$y_2 = y_1 + y_{s\ pier}/2 = 3.12 + 0.18/2 = 3.21$ m

$V_2 = V_1(y_1/y_2) = 3.36\ (3.12/3.21) = 3.26$ m/s

$h_2 = h_0 + y_{s\ pier}/2 = -0.10 + 0.09 = -0.01$

The bottom of the pile cap is below the adjusted mud line; use Case 2 computations for an exposed footing.

$$y_f = h_1 + y_{s\ pier}/2 = 1.50 + 0.09 = 1.59 \text{ m}$$

The velocity on the footing is:

$$\frac{V_f}{V_2} = \frac{\ln\left(10.93\dfrac{y_f}{k_s}+1\right)}{\ln\left(10.93\dfrac{y_2}{k_s}+1\right)} = \frac{\ln\left(10.93\dfrac{1.59}{0.0073}+1\right)}{\ln\left(10.93\dfrac{3.21}{0.0073}+1\right)} = 0.92$$

Note: Assume $K_s = D_{84} = 7.3$ mm

$$V_f = 0.92 x V_2 = 0.92 \text{ x } 3.26 = 2.99 \text{ m/s}$$

$$\frac{y_{s\,footing}}{y_f} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a_f}{y_f}\right)^{0.65}\left(\frac{V_f}{\sqrt{g y_f}}\right)^{0.43}$$

$$\frac{y_{s\,footing}}{y_f} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{2.44}{1.59}\right)^{0.65}\left(\frac{2.99}{\sqrt{9.81 \text{x} 1.59}}\right)^{0.43} = 2.83$$

Note that $y_2/a_f = 1.31$ (>0.8); use $K_W = 1.0$

$$y_{s\ footing} = 2.83 y_f = 2.83 \text{ x } 1.59 = 4.50 \text{ m}$$

Total Local Pier Scour Depth

$$y_s = y_{s\ pier} + y_{s\ footing} = 0.18 + 4.50 = 4.68 \text{ m}$$

6.10.5 Example Problem 5 - Scour at a Complex Pier with Pile Cap in the Flow (SI)

During the design of the new Woodrow Wilson Bridge over the Potomac River several complex pier configurations were tested in physical model studies. The purpose of this problem is to analyze local scour for the possible condition that the main channel migrated to the pier configured as shown in Figure 6.16. It was determined that the water surface elevations would be +2.23 m and +2.96 m for the Q_{100} and the Q_{500} events respectively and the velocities in the main channel would be 3.41 m/sec and 4.27 m/sec for the Q_{100} and the Q_{500} events respectively. The following computations are for the Q_{100} event:

Initial parameters

y_1 = 15.79 m
V_1 = 3.41 m/sec
a_{pier} = 9.754 m
a_{pc} = 16.23 m

h_0 = 7.77 m
$h_1 = h_0 + T$ = 12.65 m (resolution of the pile cap thickness below)
S = 4.19 m (center to center spacing of piles)
T = 4.88 m (assign half of the tapered portion of the cap to the pile cap and half to the pier)
f = 2.627 m (Figure 6.16)
zero angle of attack

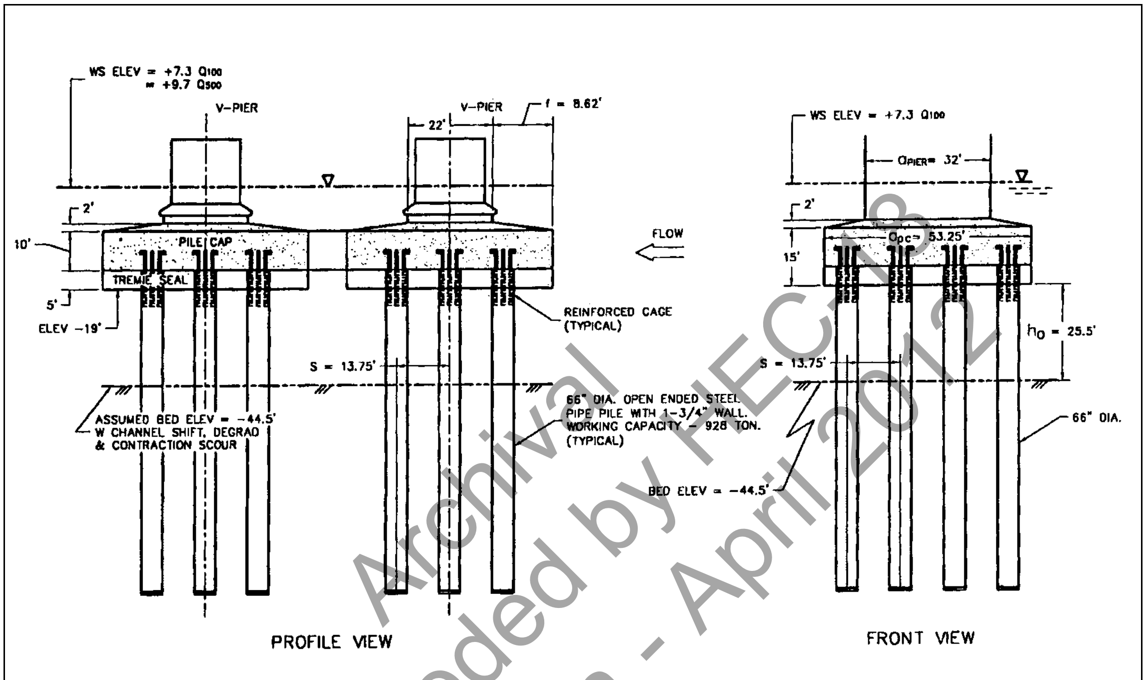


Figure 6.16. Model of complex pier geometry for the Woodrow Wilson Bridge.

Pier Stem Component

f/a_{pier} = 2.627/9.754 = 0.27

h_1/a_{pier} = 12.65/9.754 = 1.30

$K_{h\,pier}$ = 0.062 (from Figure 6.5)

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0 K_1 K_2 K_3 K_4 \left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{g y_1}}\right)\right]^{0.43}$$

$$\frac{y_{spier}}{15.79} = 0.062\left[2.0(1.1)(1.0)(1.1)(1.0)\left(\frac{9.754}{15.79}\right)^{0.65}\left(\frac{3.41}{\sqrt{(9.81)15.79}}\right)\right]^{0.43} = 0.0627$$

The quantity in the brackets is the scour ratio for a full depth pier that extends below the scour hole.

$y_{s\ pier} = 0.0627 \times 15.79\ m = 0.99\ m$

Pile Cap Component

$h_2 = h_0 + y_{s\ pier}/2 = 7.77 + 0.495 = 8.27\ m$

$y_2 = y_1 + y_{s\ pier}/2 = 15.79 + 0.495 = 16.28\ m$

$V_2 = V_1 \times (y_1/y_2) = 3.41 \times (15.79/16.28) = 3.31\ m/s$

Note: For Figure 6.6, $y_2 = 3.5a_{pc} = 56.81 > 16.28$; use $y_2 = 16.28\ m$

$h_2/y_2 = 0.51$

$T/y_2 = 4.88/16.28 = 0.30$

$\frac{a^*_{pc}}{a_{pc}} = 0.07$ (from Figure 6.6)

$a^*_{pc} = 0.07 \times 16.23 = 1.10\ m$

This is the width of a full depth pier that would produce the same scour depth as the isolated pile cap will produce.

$$\frac{y_{s\,pc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43}$$

$$\frac{y_{s\,pc}}{16.28} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{1.10}{16.28}\right)^{0.65}\left(\frac{3.31}{\sqrt{(9.81)(16.28)}}\right)^{0.43} = 0.236$$

Note that $y_2/a^*_{pc} = 14.8$ (>0.8); use $K_w = 1.0$

$y_{s\ pc} = 0.236 \times 16.28 = 3.84\ m$

Pile Group Component

$h_3 = h_0 + (y_{s\ pier} + y_{s\ pc})/2 = 7.77 + (0.99 + 3.84)/2 = 10.19\ m$

$y_3 = y_1 + (y_{s\ pier} + y_{s\ pc})/2 = 15.79 + (0.99 + 3.84)/2 = 18.20\ m$

$V_3 = V_1 \times (y_1/y_3) = 3.41 \times (15.79/18.20) = 2.95\ m/sec$

$a_{proj} = 4 \times 1.676 = 6.71\ m$ (from Figure 6.8)

$a_{proj}/a = 6.71 / 1.676 = 4.0$

S/a = 4.19/1.676 = 2.5 (relative center to center spacing of piles)

K_{sp} = 0.58 (from Figure 6.10)

K_m = 1.16 (From Figure 6.11 for three rows per foundation; foundations separated)

$a^*_{pg} = K_{sp} \times K_m \times a_{proj} = 0.58 \times 1.16 \times 6.71 = 4.51$ m

Note: for Figure 6.12, $y_{3\,max} = 3.5 \times a^*_{pg} = 15.79 < 18.20$; use y_3 = 15.79 m

h_3/y_3 = 10.19 / 15.79 = 0.65

$K_{h\,pg}$ = 0.79 (from Figure 6.12)

$$\frac{y_{s\,pg}}{y_3} = K_{hpg}\left[2.0K_1K_2K_3K_4\left(\frac{a^*_{pg}}{y_3}\right)^{0.65}\left(\frac{V_3}{\sqrt{gy_3}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pg}}{15.79} = 0.79\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{4.51}{15.79}\right)^{0.65}\left(\frac{2.95}{\sqrt{(9.81)(15.79)}}\right)^{0.43}\right] = 0.41$$

$y_{s\,pg}$ = 0.41 x 15.79 = 6.47 m

Total Estimated Scour

$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg}$ = 0.99 + 3.84 + 6.47 = 11.3 m

6.10.6 Example Problem 6 - Scour at Multiple Columns (SI)

Calculate the scour depth for a pier that consists of six 0.406 m columns spaced at 2.29 m with a flow angle of attack of 26 degrees. Debris is not a problem and there is no armoring at this site.

Data:

Columns: 6 columns 0.406 m, spaced 2.29 m
Velocity: V_1 = 3.4 m/s; Depth: y_1 = 6.1 m
Angle of attack: 26 degrees
Spacing coefficient = S/a = 2.29/0.406 = 5.6; S/a > 5.0
Assume K_3 = 1.1 for plane bed condition

Determine the depth of local scour:

Three methods of calculating the scour depth will be illustrated:

a. Scour depth according to Raudkivi[26] is 1.2 times the local scour of a single column.

$$\frac{y_s}{6.1} = 2.0 \times 1.0 \times 1.0 \times 1.1 \times 1.0 \left(\frac{0.406}{6.1}\right)^{0.65} \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.266$$

y_s = 6.1 X 0.266 X 1.2 = 1.95 m

b. Compare this value with that computed by collapsing the columns.

Collapsed pier width = 6 X 0.406 = 2.44 m

Projected pier width = L Sin 26^0 + a Cos 26^0 = 2.44 Sin 26^0 + .406 Cos 26^0 = 1.44 m

$$\frac{y_s}{6.1} = 2.0(1.0)(1.0)(1.1)1.0 \left(\frac{1.44}{6.1}\right)^{0.65} \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.604$$

y_s = 3.68 m

c. The scour depth can be calculated for multiple columns by calculating the depth for a single column and multiplying it by the K_2 factor given in Equation 6.4. For example:

K_2 = (Cos 26^0 + 2.44/0.406 Sin 26^0 $)^{0.65}$ = 2.27

$$\frac{y_s}{6.1} = 2.0(1.0)(2.27)(1.1)(1.0) \left(\frac{0.406}{6.1}\right)^{0.65} \left(\frac{3.4}{(9.81 \times 6.1)^{0.5}}\right)^{0.43} = 0.603$$

y_s = 6.1 X 0.603 =3.68 m

Spacing between columns for this pier is greater than 5 times column diameter so method (a) applies. Also, a model study of the pier gave a scour depth of 1.95 m. Therefore:

y_s = 6.1 X 0.266 X 1.2 = 1.95 m

6.10.7 Example Problem 7 - Pier Scour with Pressure Flow (SI)

An existing bridge is subjected to pressure flow to the top of a solid guard rail at the 100-year return period flow. There is only a small increase in flow depth at the bridge for the 500-year return period flow due to the large overbank area. A HEC-RAS model of the flow gives the following data:

Data:

y_1 = 9.75 m, V_1 = 2.93 m/s, q_1 = 28.56 cms/m
Pier width a = 0.914 m, is round nose, solid, aligned with the flow
Sand bed with D_{50} = 0.4 mm and D_{84} = 0.9 mm
Distance from stream bed to lower chord (H_b) is 7.93 m before scour

Calculate the local pier scour:

Vertical Contraction Scour Depth

$y_s/y_1 = -5.08 + 1.27\ y_1/H_b + 4.44\ H_b/y_1 + 0.19\ V_a/V_c$

$V_c = 6.19\ (y_1)^{1/6}\ (D_{50})^{1/3} = 6.19\ (9.75)^{1/6}\ (0.0004)^{1/3} = 0.669$ m/s

$V_a = q_1/H_b = 28.56/7.93 = 3.60$ m/s

$y_s/9.75 = -5.08 + 1.27\ (9.75/ 7.93) + 4.44\ (7.93/9.75) + 0.19\ (3.60/0.669)$

$y_s/9.75 = 1.12$ and $y_s = 10.9$ m

Local Pier Scour

$y_2 = H_b + y_s = 7.93 + 10.92 = 18.85$ m

$V_2 = V_a\ (H_b\ /y_2\ ) = 3.60\ (7.93/18.85) = 1.51$ m/s

$y_s/y_1 = 2.0\ K_1\ K_2\ K_3\ K_4\ (a/y_1)^{0.65}\ (Fr)^{0.43}$

$K_1 = K_2 = K_4 = 1.0$;$K_3 = 1.1$; $Fr = 1.52 / (9.81 X 18.85)^{0.5} = 0.11$

$y_s/18.85 = 2.0 X 1.1 X (0.914/18.85)^{0.65} (0.11)^{0.43} = 0.12$

$y_s = 18.85 X 0.12 = 2.26$ m

Total Scour

$y_s = 10.92 + 2.26 = 13.2$ m

6.11 PIER SCOUR EXAMPLE PROBLEMS (ENGLISH)

6.11.1 Example Problem 1 - Scour at a Simple Solid Pier (English)

Given:

Pier geometry: a = 4.0 ft, L = 59 ft, round nose
Flow variables: $y_1 = 10.2$ ft, $V_1 = 11.02$ ft/s
Angle of attack = 0 degrees, g = 32.2 ft/s^2
Froude No. = $11.02/(32.2 \times 10.2)^{0.5} = 0.61$
Bed material: $D_{50} = 0.32$ mm (0.0011 ft), $D_{95} = 7.3$ mm (0.024 ft)
Bed Configuration: Plane bed

Determine:

The magnitude of pier scour depth.

Solution:

Use Equation 6.1.

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$y_s / 10.2 = 2.0 x 1.0 x 1.0 x 1.1 x 1.0 x (4.0 / 10.2)^{0.65} x 0.61^{0.43} = 0.97$$

$$y_s = 0.97 \, x 10.22 = 9.9 \, ft$$

6.11.2 Example Problem 2 - Angle of Attack (English)

Given:

Same as Problem 1 but angle of attack is 20 degrees

Solution:

Use Equation 6.4 to compute K_2

$$K_2 = (Cos\ \theta + L / a\ Sin\ \theta)^{0.65}$$

If L/a is larger than 12, use L/a = 12 as a maximum in Equation 6.4 (see Table 6.2).

L/a = 18 /1.22 = 14.8 > 12 use 12

$K_2 = (Cos\ 20 + 12\ Sin\ 20)^{0.65} = 2.86$

$y_s = 9.9 X 2.86 = 28.4$ ft

6.11.3 Example Problem 3 - Coarse Bed Material (English)

Given:

Same as Problem 1 but the bed material is coarser
Bed material: D_{50} = 17.8 mm, (0.058 ft); D_{95} = 96.3 mm, (0.316 ft)
Bed configuration: Plane Bed

Determine:

If the coarse bed material would decrease local scour depth. Determine K_4 and y_s.

Solution:

Use Equations 6.5, 6.6, 6.7, and 6.8
K_4 = 1 if D_{50} <2 mm or D_{95} < 20 mm
if $D_{50} \geq 2$ mm and $D_{95} \geq 20$ mm

then:

$$K_4 = 0.4\ (V_R)^{0.15}$$

$$V_R = \frac{V_1 - V_{icD_{50}}}{V_{cD_{50}} - V_{icD_{95}}} > 0$$

where:

V_{icDx} = Approach velocity required to initiate scour at the pier for the grain size Dx , ft/s

V_{icDx} = $0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cD_x}$

V_{cDx} = Critical velocity for incipient motion for the grain size D_x ,ft/s

V_{cDx} = $11.2\ y_1^{1/6}\ D_x^{1/3}$

V_{cD50} = $11.2\ (10.2)^{1/6}\ (0.058)^{1/3}$ = 6.38 ft/s

V_{cD95} = $11.2\ (10.2)^{1/6}\ (0.316)^{1/3}$ = 11.23 ft/s

V_{icD50} = $0.645\ (0.058 / 4.0)^{0.053}\ (6.38)$ = 3.29 ft/s

V_{icD95} = $0.645\ (0.316 / 4.0)^{0.053}\ (11.23)$ = 6.33 ft/s)

V_R = $\frac{(11.02 - 3.29)}{(6.38 - 6.3)} = 154.6$

K_4 = $0.4\ (154.6)^{0.15} = 0.85$

y_s = 0.85 X 9.9 = 8.4 ft

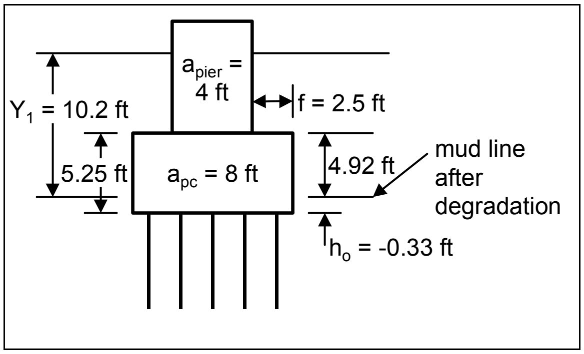
6.11.4 Example Problem 4 - Scour at Complex Piers (Solid Pier on an Exposed Footing) (English)

Given:

The pier in Problem 1 (Section 6.11.1) is on a 8.0 ft wide by 5.25 ft high by 65 ft long rectangular footing. Footing extends 2.5 ft upstream from the pier. The footing is on an unspecified pile foundation. The footing is exposed 4.92 ft by long-term degradation. Determine local pier scour.

Data:

Pier geometry; a_{pier} = 4.0 ft, L = 59 ft, round nose
Pile cap or footing geometry, a_{pc} (or a_f) = 8 ft, L = 65 ft, T = 5.12 ft, f = 2.5 ft
Approach flow: y_1 = 10.2 ft, V_1 = 11.02 ft/s
Angle of attack = 0 degrees
Froude No. = $11.02/(32.2 \times 10.2)^{0.5}$ = 0.61
Bed material: D_{50} = 0.32 mm, D_{84} = 7.3 mm, Plane bed
h_0 = 4.92 - 5.25 = -0.33 ft
See sketch below:



Local Scour from Pier Stem

$f = 2.5$ ft
$h_1 = h_0 + T = -0.33 + 5.25 = 4.92$ ft
$K_{h\,pier}$ = function (h_1/a_{pier}, f/a_{pier}) (from Figure 6.5)
$h_1/a_{pier} = 4.92/4.0 = 1.23$
$f/a_{pier} = 2.5/4 = 0.62$
$K_{h\,pier} = 0.06$

$$\frac{y_{s\,pier}}{y_1} = K_{h\,pier}\left[2.0K_1K_2K_3K_4\left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pier}}{y_1} = 0.06\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{4.0}{10.2}\right)^{0.65}\left(\frac{11.02}{\sqrt{32.2 \times 10.2}}\right)^{0.43}\right]$$

$y_{s\,pier} = 0.06 \text{x}[0.97] \text{x} 10.2 = 0.6$ ft

Note: the quantity in the square brackets is the scour ratio for a full depth pier.

Local Scour from the Pile Cap or Footing

Assume the average bed elevation in the vicinity of the pier lowers by ½ the pier stem scour.

$y_2 = y_1 + y_{s\,pier}/2 = 10.2 + 0.3 = 10.5$ ft

$V_2 = V_1(y_1/y_2) = 11.02\ (10.2/10.5) = 10.7$ ft/s

$h_2 = h_0 + y_{s\,pier}/2 = -0.33 + 0.3 = -0.03$ ft

The bottom of the pile cap is below the adjusted mud line; use Case 2 computations for an exposed footing.

$y_f = h_1 + y_{s\,pier}/2 = 4.92 + 0.3 = 5.22$ ft

The velocity on the footing is:

$$\frac{V_f}{V_2}=\frac{\ln\left(10.93\,\frac{y_f}{k_s}+1\right)}{\ln\left(10.93\,\frac{y_2}{k_s}+1\right)} \quad = \quad \frac{\ln\left(10.93\,\frac{5.22}{.024}+1\right)}{\ln\left(10.93\,\frac{10.5}{0.024}+1\right)} = 0.92$$

Note: assume $k_s = D_{84} = 7.3$ mm = 0.024 ft

$V_f = 0.92 \mathrm{x} V_2 = 0.92 \times 10.7 = 9.84$ ft/s

$$\frac{y_{s\,footing}}{y_f}=2.0K_1K_2K_3K_4K_w\left(\frac{a_f}{y_f}\right)^{0.65}\left(\frac{V_f}{\sqrt{gy_f}}\right)^{0.43}$$

$$\frac{y_{s\,footing}}{y_f}=2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{8.0}{5.22}\right)^{0.65}\left(\frac{9.84}{\sqrt{32.2\times5.22}}\right)^{0.43}=2.83$$

Note that $y_2/a_f = 1.31$ (>0.8); use $K_W = 1.0$

$y_{s\,footing} = 2.83y_f = 2.83 \times 5.22 = 14.8$ ft

Total Local Pier Scour Depth

$y_s = y_{s\,pier} + y_{s\,footing} = 14.8 + 0.6 = 15.4$ ft

6.11.5 Example Problem 5 - Scour at a Complex Pier with Pile Cap in the Flow (English)

During the design of the new Woodrow Wilson Bridge over the Potomac River, several complex pier configurations were tested in physical model studies. The purpose of this problem is to analyze local scour for the possible condition that the main channel migrated to the pier configured as shown in Figure 6.16. It was determined that the water surface elevations would be +7.3 ft and + 9.7 ft for the Q_{100} and the Q_{500} events respectively and the velocities in the main channel would be 11.2 ft/sec and 14 ft/sec for the Q_{100} and the Q_{500} events respectively. The following computations are for the Q_{100} event:

Initial parameters:

$y_1 = 51.8$ ft
$V_1 = 11.2$ ft/sec
$a_{pier} = 32$ ft
$a_{pc} = 53.25$ ft
$h_0 = 25.5$ ft
$h_1 = h_0 + T = 41.5$ ft (resolution of the pile cap thickness below)
S = 13.75 ft (center to center spacing of piles)

T= 16 ft (assign half of the tapered portion of the cap to the pile cap and half to the pier)
f = 8.62 ft (Figure 6.16)
zero angle of attack

Pier Stem Component

f/a_{pier} = 8.62/32 = 0.27

h_1/a_{pier} = 41.5/32 = 1.30

$K_{h\ pier}$ = 0.062 (from Figure 6.5)

$$\frac{y_{s\,pier}}{y_1} = K_{hpier}\left[2.0\,K_1 K_2 K_3 K_4 \left(\frac{a_{pier}}{y_1}\right)^{0.65}\left(\frac{V_1}{\sqrt{gy_1}}\right)\right]^{0.43}$$

$$\frac{y_{spier}}{51.8} = 0.062\left[2.0(1.1)(1.0)(1.1)(1.0)\left(\frac{32}{51.8}\right)^{0.65}\left(\frac{11.2}{\sqrt{(32.2)51.8}}\right)^{0.43}\right] = 0.0629$$

The quantity in the brackets is the scour ratio for a full depth pier that extends below the scour hole.

$y_{s\ pier}$ = 0.0629 x 51.8 ft = 3.2 ft

Pile Cap Component

$h_2 = h_0 + y_{s\ pier}/2$ = 25.5 + 1.6 = 27.1 ft

$y_2 = y_1 + y_{s\ pier}/2$ = 51.8 + 1.6 = 53.4 ft

$V_2 = V_1 \times (y_1/y_2)$ = 11.2 x (51.8/53.4) = 10.9 ft/s

Note: For Figure 6.6, $y_{2max} = 3.5\ a_{pc}$ = 186.38 > 53.4; use y_2 = 53.4 ft

h_2/y_2 = 0.51

T/y_2 = 16/53.4 = 0.30

$$\frac{a^*_{pc}}{a_{pc}} = 0.07$$ (from Figure 6.6)

a^*_{pc} = 0.07 x 53.25 = 3.7 ft

This is the width of a full depth pier that would produce the same scour depth as the isolated pile cap will produce.

$$\frac{y_{s\,pc}}{y_2} = 2.0 K_1 K_2 K_3 K_4 K_w \left(\frac{a^*_{pc}}{y_2}\right)^{0.65} \left(\frac{V_2}{\sqrt{g y_2}}\right)^{0.43}$$

$$\frac{y_{s\,pc}}{53.4} = 2.0(1.1)(1.0)(1.1)(1.0)(1.0)\left(\frac{3.7}{53.4}\right)^{0.65} \left(\frac{10.9}{\sqrt{(32.2)(53.4)}}\right)^{0.43} = 0.24$$

Note that y_2/a^*_{pc} = 14.4 (>0.8); use K_w = 1.0

$y_{s\ pc}$ = 0.24 x 53.4 = 12.8 ft

Pile Group Component

$h_3 = h_0 + (y_{s\ pier} + y_{s\ pc})/2$ = 25.5 +(3.2 + 12.8)/2= 33.5 ft

$y_3 = y_1 + (y_{s\ pier} + y_{s\ pc})/2$= 51.8 +(3.2 + 12.8)/2 = 59.8 ft

$V_3 = V_1 \times (y_1/y_3)$ =11.2 x (51.8/59.8) = 9.7 ft/s

a_{proj} = 4 x 5.5 = 22.0 ft (from Figure 6.8)

a_{proj}/a = 22.0 / 5.5 = 4.0

S/a = 13.75/5.5 = 2.5 (relative center to center spacing of piles)

K_{sp} = 0.58 (from Figure 6.10)

K_m = 1.16 (From Figure 6.11 for three rows per foundation; foundations separated)

$a^*_{pg} = K_{sp} \times K_m \times a_{proj}$ = 0.58 x 1.16 x 22.0 = 14.8 ft

Note: in Figure 6.12, $y_{3\,max} = 3.5 \times a^*_{pg}$ = 51.8 < 59.8; use y_3 = 51.8 ft

h_3/y_3 = 33.5/51.8 = 0.65

$K_{h\ pg}$ = 0.79 (from Figure 6.12)

$$\frac{y_{s\,pg}}{y_3} = K_{hpg}\left[2.0 K_1 K_2 K_3 K_4 \left(\frac{a^*_{pc}}{y_3}\right)^{0.65} \left(\frac{V_3}{\sqrt{g y_3}}\right)^{0.43}\right]$$

$$\frac{y_{s\,pg}}{51.8} = 0.79\left[2.0(1.0)(1.0)(1.1)(1.0)\left(\frac{14.8}{51.8}\right)^{0.65} \left(\frac{9.7}{\sqrt{(32.2)(51.8)}}\right)^{0.43}\right] = 0.41$$

$y_{s\ pg}$ =0.41 x 51.8 = 21.24 ft

Total Estimated Scour

$y_s = y_{s\,pier} + y_{s\,pc} + y_{s\,pg} = 3.7 + 12.8 + 21.24 = 37.74$ ft

6.11.6 Example Problem 6 - Scour at Multiple Columns (English)

Calculate the scour depth for a pier that consists of six 16-inch columns spaced at 7.5 ft with an flow angle of attack of 26 degrees. Debris is not a problem and there is no armoring at this site.

Data:

Columns: 6 columns 1.33 ft, spaced 7.5 ft
Velocity: $V_1 = 11.16$ ft/s; Depth: $y_1 = 20.0$ ft
Angle of attack: 26 degrees
Spacing coefficient = S/a = 7.5/1.33 = 5.6; S/a > 5.0
Assume $K_3 = 1.1$ for plane bed condition

Determine the depth of local scour:

Three methods of calculating the scour depth will be illustrated.

a. Scour depth according to Raudkivi[26] is 1.2 times the local scour of a single column.

$$\frac{y_s}{20} = 2.0x1.0x1.0x1.1x1.0\left(\frac{1.33}{20}\right)^{0.65}\left(\frac{11.16}{(32.2x20)^{0.5}}\right)^{0.43} = 0.266$$

$y_s = 20 \text{ X } 0.266 \text{ X } 1.2 = 6.4$ ft

b. Compare this value with that computed by collapsing the columns.

Collapsed pier width = 6 X 1.33 = 8.0 ft

Projected pier width = L Sin 26^0 + a Cos 26^0 = 8.0 Sin 26^0 + 1.33 Cos 26^0 = 4.70 ft

$$\frac{y_s}{20} = 2.0\,(1.0)\,(1.0)\,(1.1)\,(1.0)\left(\frac{4.7}{20}\right)^{0.65}\left(\frac{11.16}{(32.2\,x\,20)^{0.5}}\right)^{0.43} = 0.603$$

$y_s = 12.1$ ft

c. The scour depth can be calculated for multiple columns by calculating the depth for a single column and multiplying it by the K_2 factor given in Equation 6.4. For example:

$K_2 = (\text{Cos } 26^0 + 8.0/1.33 \text{ Sin } 26^0)^{0.65} = 2.27$

$$\frac{y_s}{20} = 2.0\,(1.0)\,(2.27)\,(1.1)\,(1.0)\left(\frac{1.33}{20}\right)^{0.65}\left(\frac{11.16}{(32.2\,x\,20)^{0.5}}\right)^{0.43} = 0.603$$

$y_s = 20 \text{ X } 0.603 = 12.1$ ft

Spacing between columns for this pier is greater than 5 times column diameter so method (a) applies. Also, a model study of the pier gave a scour depth of 6.4 ft. Therefore:

$y_s = 20 \times 0.266 \times 1.2 = 6.4$ ft

6.11.7 Example Problem 7 - Pier Scour with Pressure Flow (English)

An existing bridge is subjected to pressure flow to the top of a solid guard rail at the 100-year return period flow. There is only a small increase in flow depth at the bridge for the 500-year return period flow due to the large overbank area. A HEC-RAS model of the flow gives the following data:

Data:

$y_1 = 32$ ft, $V_1 = 9.61$ ft/s, $q_1 = 307.5$ cfs/ft
Pier width a = 3.0 ft, is round nose, solid, aligned with the flow
Sand bed with $D_{50} = 0.4$ mm and $D_{84} = 0.9$ mm
Distance from stream bed to lower chord (H_b) is 26 ft before scour

Calculate the local pier scour:

Vertical Contraction Scour Depth

$y_s/y_1 = -5.08 + 1.27\ y_1/H_b + 4.44\ H_b/y_1 + 0.19\ V_a/V_c$

$V_c = 11.2\ (y_1)^{1/6}\ (D_{50})^{1/3} = 11.2\ (32)^{1/6}\ (0.0013)^{1/3} = 2.18$ ft/s

$V_a = q_1/H_b = 307.5/26 = 11.82$ ft/s

$y_s/32 = -5.08 + 1.27\ (32/26) + 4.44\ (26/32) + 0.19\ (11.82/2.18)$

$y_s/32 = 1.12$ and $y_s = 35.9$ ft

Local Pier Scour

$y_2 = H_b + y_s = 26 + 35.9 = 61.9$ ft

$V_2 = V_a\ (H_b/y_2) = 11.82\ (26/61.9) = 4.96$ ft/s

$y_s/y_1 = 2.0\ K_1\ K_2\ K_3\ K_4\ (a/y_1)^{0.65}\ (Fr)^{0.43}$

$K_1 = K_2 = K_4 = 1.0$; $K_3 = 1.1$; $Fr = 4.96 / (32.2 \times 61.9)^{0.5} = 0.11$

$y_s/61.9 = 2.0 \times 1.1 \times (3.0/61.9)^{0.65}\ (0.11)^{0.43} = 0.12$

$y_s = 7.4$ ft

Total Scour

$y_s = 35.9 + 7.4 = 43.3$ ft

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CHAPTER 7

EVALUATING LOCAL SCOUR AT ABUTMENTS

7.1 GENERAL

Scour occurs at abutments when the abutment and embankment obstruct the flow. Several causes of abutment failures during post-flood field inspections of bridge sites have been documented:[69]

- Overtopping of abutments or approach embankments
- Lateral channel migration or stream widening processes
- Contraction scour
- Local scour at one or both abutments

Abutment damage is often caused by a combination of these factors. Where abutments are set back from the channel banks, especially on wide floodplains, large local scour holes have been observed with scour depths of as much as four times the approach flow depth on the floodplain. As a general rule, the abutments most vulnerable to damage are those located at or near the channel banks.

The flow obstructed by the abutment and approach highway embankment forms a horizontal vortex starting at the upstream end of the abutment and running along the toe of the abutment, and a vertical wake vortex at the downstream end of the abutment (Figure 7.1).

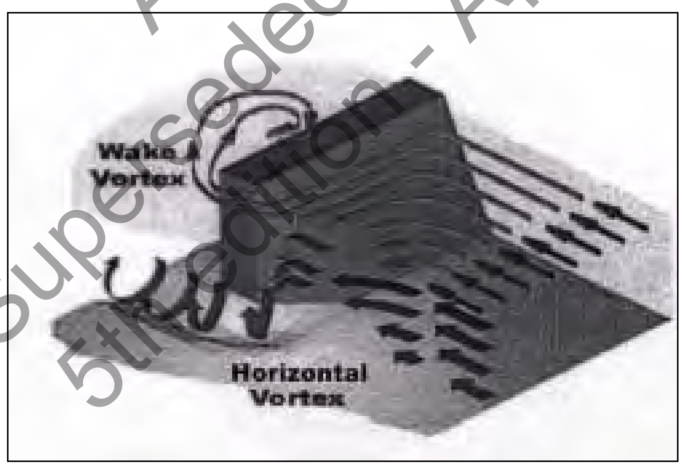

Figure 7.1. Schematic representation of abutment scour.

The vortex at the toe of the abutment is very similar to the horseshoe vortex that forms at piers, and the vortex that forms at the downstream end is similar to the wake vortex that forms downstream of a pier. Research has been conducted to determine the depth and location of the scour hole that develops for the horizontal (so called horseshoe) vortex that occurs at the upstream end of the abutment, and numerous abutment scour equations have been developed to predict this scour depth.

Abutment failures and erosion of the fill also occur from the action of the downstream wake vortex. However, research and the development of methods to determine the erosion from the wake vortex has not been conducted. An example of abutment and approach erosion of a bridge due to the action of the horizontal and wake vortex is shown in Figure 7.2.

Figure 7.2. Scour of bridge abutment and approach embankment.

The types of failures described above are initiated as a result of the obstruction to the flow caused by the abutment and highway embankment and subsequent contraction and turbulence of the flow at the abutments. There are other conditions that develop during major floods, particularly on wide floodplains, that are more difficult to foresee but that need to be considered in the hydraulic analysis and design of the substructure:[69]

- Gravel pits on the floodplain upstream of a structure can capture the flow and divert the main channel flow out of its normal banks into the gravel pit. This can result in an adverse angle of attack of the flow on the downstream highway with subsequent breaching of the embankment and/ or failure of the abutment.

- Levees can become weakened and fail with resultant adverse flow conditions at the bridge abutment.

- Debris can become lodged at piers and abutments and on the bridge superstructure, modifying flow conditions and creating adverse angles of attack of the flow on bridge piers and abutments.

7.2 ABUTMENT SCOUR EQUATIONS

7.2.1 Overview

Equations for predicting abutment scour depths such as Liu et al., Laursen, Froehlich, and Melville are based entirely on laboratory data.[70,48,71,72] The problem is that little field data on abutment scour exist. Liu et al.'s equations were developed by dimensional analysis of the variables with a best-fit line drawn through the laboratory data.[70] Laursen's equations are based on inductive reasoning of the change in transport relations due to the acceleration of the flow caused by the abutment.[48] Froehlich's equations were derived from dimensional analysis and regression analysis of the available laboratory data.[71] Melville's equations were derived from dimensional analysis and development of relations between dimensionless parameters using best-fit lines through laboratory data.[72]

Until recently, the equations in the literature were developed using the abutment and roadway approach length as one of the variables. This approach results in excessively conservative estimates of scour depth. Richardson and Richardson pointed this out in a discussion of Melville's (1992) paper:[73,72]

> "The reason the equations in the literature predict excessively conservative abutment scour depths for the field situation is that, in the laboratory flume, the discharge intercepted by the abutment is directly related to the abutment length; whereas, in the field, this is rarely the case."

Figure 7.3. illustrates the difference. Thus, equations for predicting abutment scour would be more applicable to field conditions if they included the discharge intercepted by the embankment rather than embankment length. Sturm[42,74] concluded that a discharge distribution factor is the appropriate variable to use on local scour depth rather than abutment length.

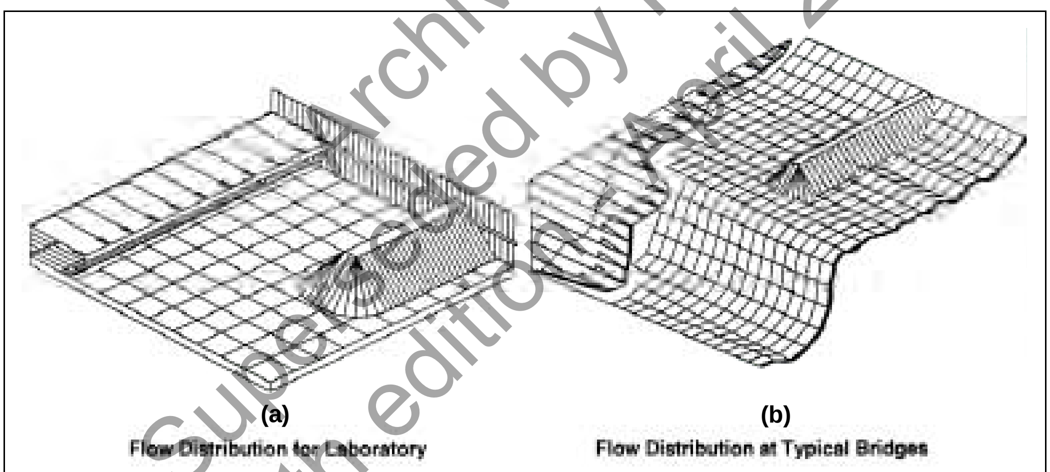

Figure 7.3. Comparison of (a) laboratory flow characteristics to (b) field flow conditions.

Abutment scour depends on the interaction of the flow obstructed by the abutment and roadway approach and the flow in the main channel at the abutment. The discharge returned to the main channel at the abutment is not simply a function of the abutment and roadway length in the field case. Richardson and Richardson noted that abutment scour depth depends on abutment shape, discharge in the main channel at the abutment, discharge intercepted by the abutment and returned to the main channel at the abutment, sediment characteristics, cross-sectional shape of the main channel at the abutment (especially the depth of flow in the main channel and depth of the overbank flow at the abutment), and alignment.[73] In addition, field conditions may have tree-lined or vegetated banks, low velocities, and shallow depths upstream of the abutment. Most of the early laboratory research failed to replicate these field conditions.

Recent research sponsored by the National Cooperative Highway Research Program of the Transportation Research Board has developed an equation to determine abutment scour that includes the discharge intercepted by an abutment and its approach rather than abutment and approach length.[75] The equation and method are presented in Appendix E. In addition, Maryland State Highway Administration has developed a method to determine scour depths at abutments, which is presented in Appendix F.[41,76] Both methods are under development and show promise of improving abutment scour calculations. They should be used with caution, and use of engineering judgment is needed for application at this time.

Abutment foundations should be designed to be safe from long-term degradation, lateral migration, and contraction scour; and protected from local horizontal and wake vortex scour with riprap and/or guidebanks, dikes, or revetments protected with riprap. The two equations provided in this chapter should be used as guides in the design.

7.2.2 Abutment Scour Parameter Determination

Many of the abutment scour prediction equations presented in the literature use the length of an abutment (embankment) projected normal to flow as an independent variable. In practice, the length of embankment projected normal to flow that is used in these relationships is determined from the results of 1-dimensional hydraulic models such as WSPRO[15] or HEC-RAS.[16,17] These models assume an average velocity over the entire cross section (Figure 7.3a). In reality, conveyance and associated velocity and flow depth at the outer extremes of a floodplain are much less, particularly in wide and shallow heavily vegetated floodplains (Figure 7.3b). This flow is typically referred to as "ineffective" flow. When applying abutment scour equations that use the length of embankment projected normal to flow, it is imperative that the length used be the length of embankment blocking "live" flow.

The length of embankment blocking "live" flow can be determined from a graph of conveyance versus distance across a representative cross-section upstream of the bridge (Figure 7.4). If a relatively large portion of a cross-section is required to convey a known amount of discharge in the floodplain, then the length of embankment blocking this flow should probably not be included when determining the length of embankment for use in the abutment scour prediction relationship. Alternately, if the flow in a significant portion of the cross-section has low velocity and/or is shallow, then the length of embankment blocking this flow should probably not be used either. Both WSPRO[15] and HEC-RAS[16,17] can easily compute conveyance versus distance across a cross section.

For example, Figure 7.4 shows the plan view of an embankment blocking three equal conveyance tubes on the right floodplain at a bridge. Since the right conveyance tube occupies the majority of floodplain but conveys only one-third of the floodplain flow, it should not be included in the "live" flow area for determining L'. In this case the length of embankment, L', blocking the "live" flow is approximately the length of the two inner conveyance tubes. In the event that the conveyance versus distance graph does not show a conclusive break point between "live" flow and ineffective flow, an alternative procedure is to estimate L' as the width of the conveyance tube directly upstream of the abutment times the total number of conveyance tubes (including fractional portions) obstructed by the embankment. This length is more representative of the uniform flow conditions in the laboratory experiments used to develop abutment scour equations.

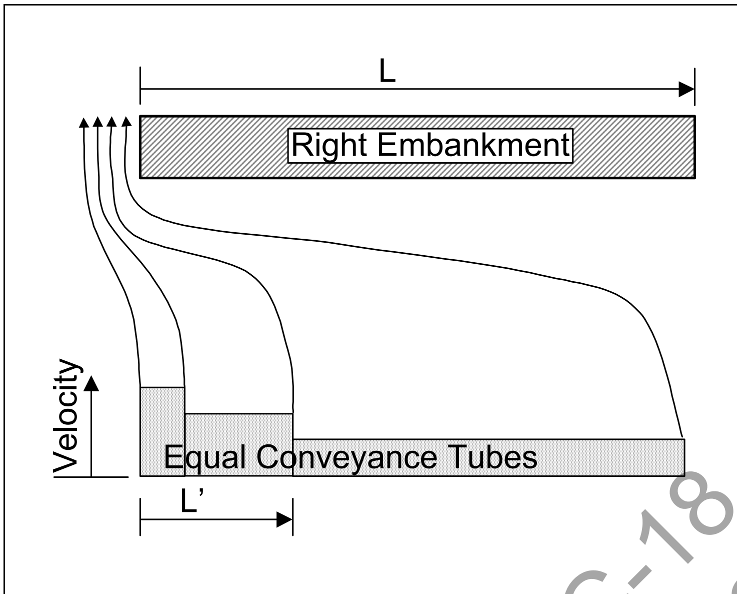

Figure 7.4. Determination of length of embankment blocking live flow for abutment scour estimation.

7.3 ABUTMENT SITE CONDITIONS

Abutments can be set back from the natural stream bank, placed at the bankline or, in some cases, actually set into the channel itself. Common designs include stub abutments placed on spill-through slopes, and vertical wall abutments, with or without wingwalls. Scour at abutments can be live-bed or clear-water scour. The bridge and approach road can cross the stream and floodplain at a skew angle and this will have an effect on flow conditions at the abutment. Finally, there can be varying amounts of overbank flow intercepted by the approaches to the bridge and returned to the stream at the abutment. More severe abutment scour will occur when the majority of overbank flow returns to the bridge opening directly upstream of the bridge crossing. Less severe abutment scour will occur when overbank flows gradually return to the main channel upstream of the bridge crossing.

7.4 ABUTMENT SKEW

The skew angle for an abutment (embankment) is depicted in Figure 7.5. For an abutment angled downstream, the scour depth is decreased, whereas the scour depth is increased for an abutment angled upstream. An equation and guidance for adjusting abutment scour depth for embankment skew are given in Section 7.7.1.

7.5 ABUTMENT SHAPE

There are three general shapes of abutments: (1) spill-through abutments, (2) vertical walls without wing walls, and (3) vertical-wall abutments with wing walls (Figure 7.6). These shapes have varying angles to the flow. As shown in Table 7.1, depth of scour is approximately double for vertical-wall abutments as compared with spill-through abutments. Similarly, scour at vertical wall abutments with wingwalls is reduced to 82 percent of the scour of vertical wall abutments without wingwalls.

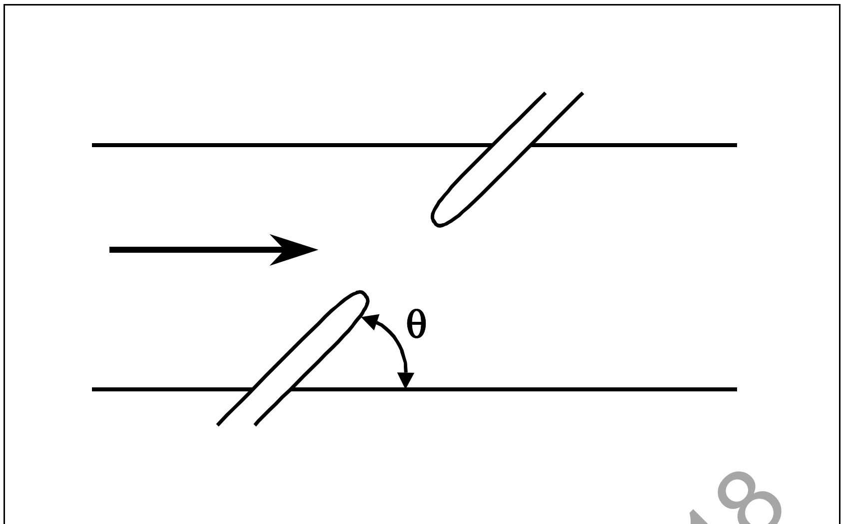

Figure 7.5. Orientation of embankment angle, θ, to the flow.

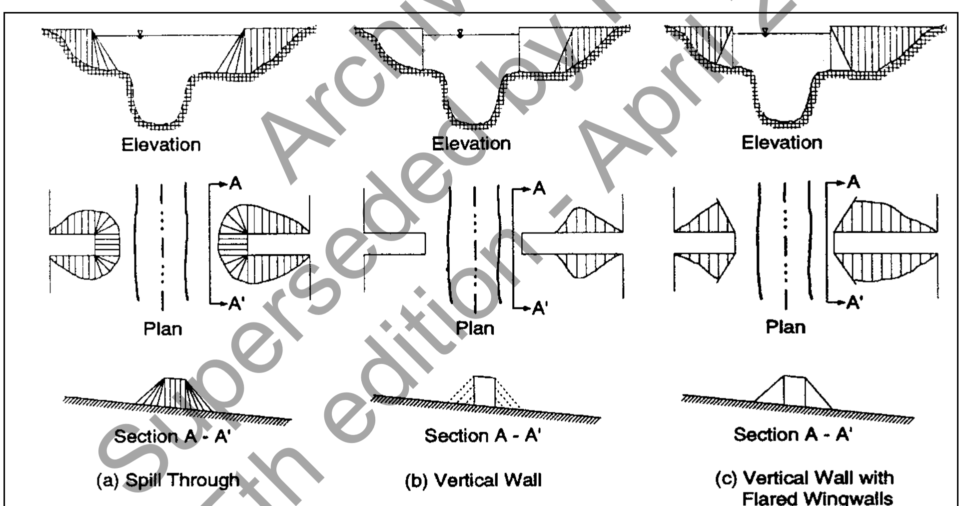

Figure 7.6. Abutment shape.

Table 7.1. Abutment Shape Coefficients.	
Description	K_1
Vertical-wall abutment	1.00
Vertical-wall abutment with wing walls	0.82
Spill-through abutment	0.55

7.6 DESIGNING FOR SCOUR AT ABUTMENTS

The preferred design approach is to place the abutment foundation on scour resistant rock or on deep foundations. Available technology has not developed sufficiently to provide reliable abutment scour estimates for all hydraulic flow conditions that might be reasonably expected to occur at an abutment. **Therefore, engineering judgment is required in designing foundations for abutments. In many cases, foundations can be designed with shallower depths than predicted by the equations when they are protected with rock riprap and/or with a guide bank placed upstream of the abutment designed in accordance with guidelines in HEC-23.[7] Cost will be the deciding factor**.

Based on lessons learned from field evaluations of damaged abutments, consideration should be given to designing deep foundations (piles and shafts) to support both vertical wall abutments and stub abutments on spill-through slopes for the condition where the approach embankment is breached and all supporting soil around the abutment (including the spill through slope) has been removed (see Figure 7.2). Piling for abutments should be driven below the elevation of the long-term degradation and contraction scour. The potential for lateral channel instability should also be considered when designing abutment foundation depths. Some State DOTs evaluate the abutment for scour in a manner similar to that of a pier.

On wide floodplains or on floodplains with complex conditions which could affect future flood flows (confluences, adverse meander patterns and bends, gravel mining pits, ponding of the flow, levee systems, etc.) additional scour countermeasures such as guidebanks, dikes or revetments should be evaluated for inclusion with the initial bridge construction. The intent here is to establish a control to maintain a favorable approach flow condition at the abutment even though upstream conditions may change.

The potential for lateral channel migration, long-term degradation and contraction scour should be considered in setting abutment foundation depths near the main channel. It is recommended that the abutment scour equations presented in this chapter be used to develop insight as to the scour potential at an abutment.

Where spread footings are placed on erodible soil, the preferred approach is to place the footing below the elevation of total scour. If this is not practicable, a second approach is to place the top of footings below the depth of the sum of contraction scour and long-term degradation and to provide scour countermeasures. For spread footings on erodible soil, it becomes especially important to protect adjacent embankment slopes with riprap or other appropriate scour countermeasures. The toe or apron of the riprap serves as the base for the slope protection and must be carefully designed to resist scour while maintaining the support for the slope protection.

In summary, as a minimum, abutment foundations should be designed assuming no ground support (lateral or vertical) as a result of soil loss from long-term degradation, stream instability, and contraction scour. The abutment should be protected from local scour using riprap and/or guide banks. Guidelines for the design of riprap and guide banks are given in HEC-23.[7] To protect the abutment and approach roadway from scour by the wake vortex several DOTs use a 15-meter (50-ft) guide bank extending from the downstream corner of the abutment. Otherwise, the downstream abutment and approach should be protected with riprap or other countermeasures.

In the following sections, two equations are presented for use in estimating scour depths as a guide in designing abutment foundations. The methods can be used for either **clear-water or live-bed** scour.

7.7 LIVE-BED SCOUR AT ABUTMENTS

As a check on the potential depth of scour to aid in the design of the foundation and placement of rock riprap and/or guide banks, Froehlich's[70] live-bed scour equation or the HIRE equation in HDS 6[22] can be used.

7.7.1 Froehlich's Live-Bed Abutment Scour Equation

Froehlich[71] analyzed 170 live-bed scour measurements in laboratory flumes by regression analysis to obtain the following equation:

$$\frac{y_s}{y_a} = 2.27\, K_1\, K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1 \tag{7.1}$$

where:

K_1	=	Coefficient for abutment shape (Table 7.1)
K_2	=	Coefficient for angle of embankment to flow
K_2	=	$(\theta/90)^{0.13}$ (see Figure 7.4 for definition of θ) $\theta<90°$ if embankment points downstream $\theta>90°$ if embankment points upstream
L'	=	Length of active flow obstructed by the embankment, m (ft)
A_e	=	Flow area of the approach cross section obstructed by the embankment, m^2 (ft^2)
Fr	=	Froude Number of approach flow upstream of the abutment = $V_e/(gy_a)^{1/2}$
V_e	=	Q_e/A_e, m/s (ft/s)
Q_e	=	Flow obstructed by the abutment and approach embankment, m^3/s (ft^3/s)
y_a	=	Average depth of flow on the floodplain (A_e/L), m (ft)
L	=	Length of embankment projected normal to the flow, m (ft)
y_s	=	Scour depth, m (ft)

It should be noted that Equation 7.1 is not consistent with the fact that as L' tends to 0, y_s also tends to 0. The 1 was added to the equation so as to envelope 98 percent of the data. See Section 7.2.2 and Figure 7.4 for guidance on estimating L'.

7.7.2 HIRE Live-Bed Abutment Scour Equation

An equation based on field data of scour at the end of spurs in the Mississippi River (obtained by the USACE) can also be used for estimating abutment scour.[22] This field situation closely resembles the laboratory experiments for abutment scour in that the discharge intercepted by the spurs was a function of the spur length. The modified equation, referred to herein as the HIRE equation, is applicable when the ratio of projected abutment length (L) to the flow depth (y_1) is greater than 25. This equation can be used to estimate scour depth (y_s) at an abutment where conditions are similar to the field conditions from which the equation was derived:

$$\frac{y_s}{y_1} = 4\ Fr^{0.33}\ \frac{K_1}{0.55}\ K_2 \tag{7.2}$$

where:

y_s	=	Scour depth, m (ft)
y_1	=	Depth of flow at the abutment on the overbank or in the main channel, m (ft)
Fr	=	Froude Number based on the velocity and depth adjacent to and upstream of the abutment
K_1	=	Abutment shape coefficient (from Table 7.1)
K_2	=	Coefficient for skew angle of abutment to flow calculated as for Froehlich's equation (Section 7.7.1)

7.8 CLEAR-WATER SCOUR AT AN ABUTMENT

Equations 7.1 and 7.2 are recommended for both live-bed and clear-water abutment scour conditions. If a method other than Froehlich's equation is used, it is suggested that scour for both the clear water and live bed condition be computed (see Appendix E and Appendix F). Engineering judgment should then be used to select the most appropriate scour depth.

7.9 ABUTMENT SCOUR EXAMPLE PROBLEMS (SI)

7.9.1 Example Problem 1 (SI)

Determine abutment scour depth for the following conditions to aid in scour evaluation and design of countermeasures. The right abutment is at the bankline with 3.00 m of overbank flow width. The left abutment projects into the channel 61.96 m. Each of these lengths represents the full length of obstruction of active flow. The projection on the left side is the result of stream erosion and widening. The right channel bank is 0.61 m high and the embankment extends back 3.00 m to a 3 m high bank. The bridge and approach are oriented at a 10° angle upstream to the flow from the right side.

Given:

Upstream channel depth = 2.62 m
Discharge = 773.05 m^3/s
Bridge is vertical wall with wingwalls

Original (unscoured) depth of flow at bridge is estimated as 2.16 m

Right Abutment

$L = L' = 3\,\mathrm{Cos}\,10^{\circ} = 2.95\,\mathrm{m}$

$y_a = 2.62 - 0.61 = 2.01\,\mathrm{m}$

$$\frac{L}{y_a} = \frac{2.95}{2.01} = 1.47 < 25 \text{ (Use Froehlich Equation)}$$

$$\frac{y_s}{y_a} = 2.27\,K_1\,K_2\left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

$K_1 = 0.82$

$$K_2 = \left(\frac{\theta}{90}\right)^{.13} = \left(\frac{100}{90}\right)^{0.13} = 1.01 \text{ (Abutment angles } 10^{\circ} \text{ upstream)}$$

$A_e = 2.01 x 2.95 = 5.93\,\mathrm{m}^2$

$Q_e = 17.8\,\mathrm{m}^3/\mathrm{s};\ V_e = 3.00\,\mathrm{m/s}$ (Q_e and V_e are obtained from HEC-RAS)

$$Fr = \frac{V_e}{\sqrt{g y_a}} = \frac{3.00}{(9.81 x 2.01)^{1/2}} = 0.68$$

$$\frac{y_s}{2.01} = 2.27\ (0.82)\ (1.01)\left(\frac{2.95}{2.01}\right)^{.43}(0.68)^{0.61} + 1 = 2.75$$

$y_s = 2.75 \, x \, 2.01 = 5.53\,\mathrm{m}$

Left Abutment

$L = 61.96\ \mathrm{Cos}\ 10^{\circ} = 61.02\,\mathrm{m}$

$y_1 = 2.16\,\mathrm{m}$

$$\frac{L}{y_1} = \frac{61.02}{2.16} = 28.25 > 25 \text{ (Use HIRE Equation)}$$

$$\frac{y_s}{y_1} = 4\,Fr^{0.33}\,\frac{K_1}{0.55}\,K_2$$

$v_1 = 3.72\,\mathrm{m/s}$ (From HEC-RAS stream tube next to abutment)

$$Fr = \frac{v_1}{\sqrt{gy_1}} = \frac{3.72}{(9.81 x 2.16)^{1/2}} = 0.81$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{80}{90}\right)^{.13} = 0.98$$

$$\frac{y_s}{y_1} = 4(0.81)^{0.33}\frac{0.82}{0.55}(0.98) = 5.45$$

$$y_s = 5.45 \, x \, 2.16 = 11.8 \, m$$

7.10 ABUTMENT SCOUR EXAMPLE PROBLEMS (English)

7.10.1 Example Problem 1 (English)

Determine abutment scour depth for the following conditions to aid in scour evaluation and design of countermeasures. The right abutment is at the bankline with 9.8 ft of overbank flow width. The left abutment projects into the channel 200 ft. Each of these lengths represents the full length of obstruction of active flow. The projection on the left side is the result of stream erosion and widening. The right channel bank is 2 ft high and the embankment extends back 9.8 ft to a 9.8 ft high bank. The bridge and approach are oriented at a 10° angle upstream to the flow from the right side.

Given:

Upstream channel depth = 8.6 ft
Discharge is 27,300 cfs
Bridge is vertical wall with wingwalls

Original (unscoured) depth of flow at bridge is estimated as 7.1 ft

Right Abutment

$$L = L' = 9.8 \, Cos \, 10° = 9.7 \, ft$$

$$y_a = 8.6 - 2.0 = 6.6 \, ft$$

$$\frac{L}{y_a} = \frac{9.7}{6.6} = 1.47 < 25 \text{ (Use Froehlich Equation)}$$

$$\frac{y_s}{y_a} = 2.27 K_1 K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{\theta}{90}\right)^{.13} = \left(\frac{100}{90}\right)^{0.13} = 1.01 \text{ (Abutment angles 10° upstream)}$$

$$A_e = 6.6 \text{ x } 9.7 = 64.0 \text{ ft}^2$$

$Q_e = 629$ cfs; $V_e = 9.8$ ft / s (Q_e and V_e are obtained from HEC-RAS)

$$Fr = \frac{V_e}{\sqrt{g y_a}} = \frac{9.8}{(32.2 \text{ x } 6.6)^{1/2}} = 0.67$$

$$\frac{y_s}{6.6} = 2.27 (0.82)(1.01)\left(\frac{9.7}{6.6}\right)^{.43} (0.67)^{0.61} + 1 = 2.74$$

$$y_s = 2.74 \text{ x } 6.6 = 18.1 \text{ ft}$$

Left Abutment

$$L = 200 \text{ Cos } 10° = 197.0 \text{ ft}$$

$$y_1 = 7.1 \text{ ft}$$

$$\frac{L}{y_1} = \frac{197.0}{7.1} = 27.7 > 25 \text{ (Use HIRE Equation)}$$

$$\frac{y_s}{y_1} = 4 Fr^{0.33} \frac{K_1}{0.55} K_2$$

$v_1 = 12.2$ ft / s (From HEC-RAS stream tube next to abutment)

$$Fr = \frac{v_1}{\sqrt{g y_1}} = \frac{12.2}{(32.2 \text{ x } 7.1)^{1/2}} = 0.81$$

$$K_1 = 0.82$$

$$K_2 = \left(\frac{80}{90}\right)^{.13} = 0.98$$

$$\frac{y_s}{y_1} = 4(0.81)^{0.33}\,\frac{0.82}{0.55}\left(0.98\right) = 5.45$$

$$y_s = 5.45 \, x \, 7.1 = 38.7 \text{ ft}$$

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CHAPTER 8

COMPREHENSIVE EXAMPLE SCOUR PROBLEM

8.1 GENERAL DESCRIPTION OF PROBLEM

This example problem is taken from a paper by Arneson et al.[77] FHWA's WSPRO computer program was used to obtain the hydraulic variables. The program uses 20 stream tubes to give a quasi 2-dimensional analysis. Each stream tube has the same discharge (1/20 of the total discharge). The stream tubes provide the velocity distribution across the flow and the program has excellent bridge routines. The problem presented here is worked in SI (metric) units, however, the same problem worked in English units is presented in Appendix H. The solution follows Steps 1-7 of the specific design approach of Chapter 2 (Section 2.4).

A 198.12-m long bridge (Figure 8.1) is to be constructed over a channel with spill-through abutments (slope of 1V:2H). The left abutment is set approximately 60.5 m back from the channel bank. The right abutment is set at the channel bank. The bridge deck is set at elevation 6.71 m and has a girder depth of 1.22 m. Six round-nose piers are evenly spaced in the bridge opening. The piers are 1.52 m thick, 12.19 m long, and are aligned with the flow. The 100-year design discharge is 849.51 m^3/s. The 500-year flow of 1444.16 m^3/s was estimated by multiplying the Q_{100} by 1.7 since no hydrologic records were available to predict the 500-year flow.

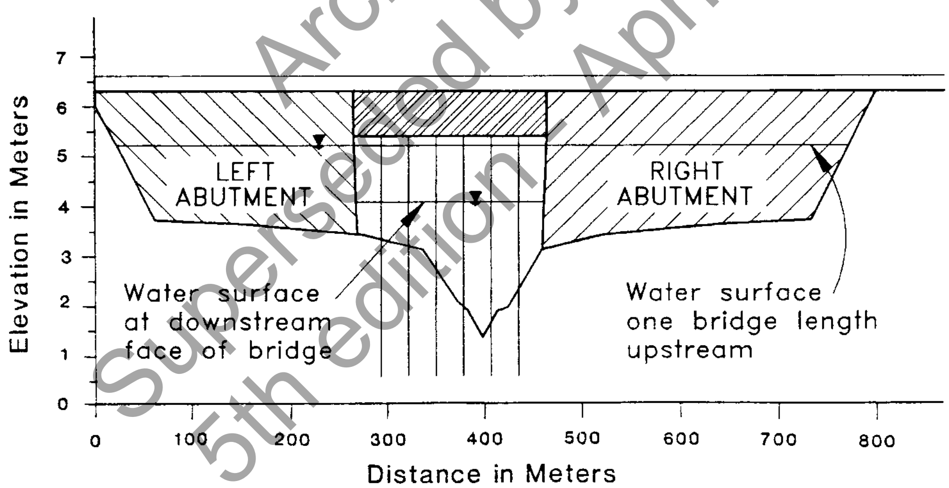

Figure 8.1. Cross section of proposed bridge.

8.2 STEP 1: DETERMINE SCOUR ANALYSIS VARIABLES

From Level 1 and Level 2 analysis: a site investigation of the crossing was conducted to identify potential stream stability problems at this crossing. Evaluation of the site indicates that the river has a relatively wide floodplain. The floodplain is well vegetated with grass and trees; however, the presence of remnant channels indicates that there is a potential for lateral shifting of the channel.

The bridge crossing is located on a relatively straight reach of channel. The channel geometry is relatively the same for approximately 300 m up- and downstream of the bridge crossing. The D_{50} of the bed material and overbank material is approximately 0.002 m (2 mm). The maximum grain size of the bed material is approximately 0.008 m (8 mm). The specific gravity of the bed material was determined to be equal to 2.65.

The river and crossing are located in a rural area with the primary land use consisting of agriculture and forest.

Review of bridge inspection reports for bridges located upstream and downstream of the proposed crossing indicates no long-term aggradation or degradation in this reach. At the bridge site, bedrock is approximately 46 m below the channel bed.

Since this is a sand-bed channel, no armoring potential is expected. Furthermore, the bed for this channel at low flow consists of dunes which are approximately 0.3 to 0.5 m high. At higher flows, above the Q_5, the bed will be either plane bed or antidunes.

The left and right banks are relatively well vegetated and stable; however, there are isolated portions of the bank which appear to have been undercut and are eroding. Brush and trees grow to the edge of the banks. Banks will require riprap protection if disturbed. Riprap will be required upstream of the bridge and extend downstream of the bridge.

8.2.1 Hydraulic Characteristics

Hydraulic characteristics at the bridge were determined using WSPRO.[15] Three cross sections were used for this analysis and are denoted as "EXIT" for the section downstream of the bridge, "FULLV" for the full-valley section at the bridge, and "APPR" for the approach section located one bridge length upstream of the bridge. The bridge geometry was superimposed on the full-valley section and is denoted "BRDG." Values used for this example problem are based on the output from the WSPRO model which is presented in Appendix G (SI). Specific values for scour analysis variables are given for each computation separately and cross referenced to the line numbers of the WSPRO output.

The HP2 option was used to provide hydraulic characteristics at both the bridge and approach sections. This WSPRO option subdivides the cross section into 20 equal conveyance tubes. Figures 8.2 and 8.3 illustrate the location of these conveyance tubes for the approach and bridge cross section, respectively. Figure 8.4 illustrates the average velocities in each conveyance tube and the contraction of the flow from the approach section through the bridge. Figure 8.4 also identifies the equal conveyance tubes of the approach section which are cut off by the abutments.

Hydraulic variables for performing the various scour computations were determined from the WSPRO output (Appendix G) and from Figures 8.2, 8.3, and 8.4. These variables which will be used to compute contraction scour and local scour are presented in Tables 8.1 through 8.6.

Contraction scour will occur both in the main channel and on the left overbank of the bridge opening. For the main channel, contraction scour could be either clear-water or live-bed depending on the magnitude of the channel velocity and the critical velocity for sediment movement. A computation will be performed to determine the sediment transport characteristics of the main channel and the appropriate contraction scour equation.

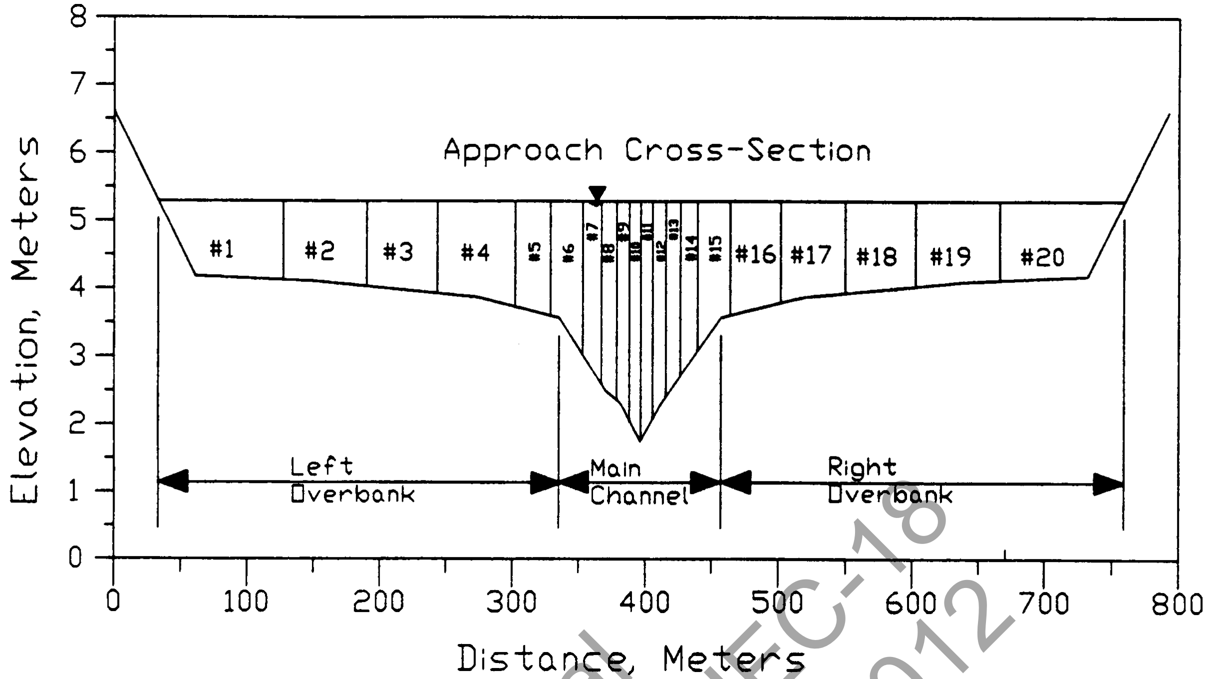

Figure 8.2. Equal conveyance tubes of approach section.

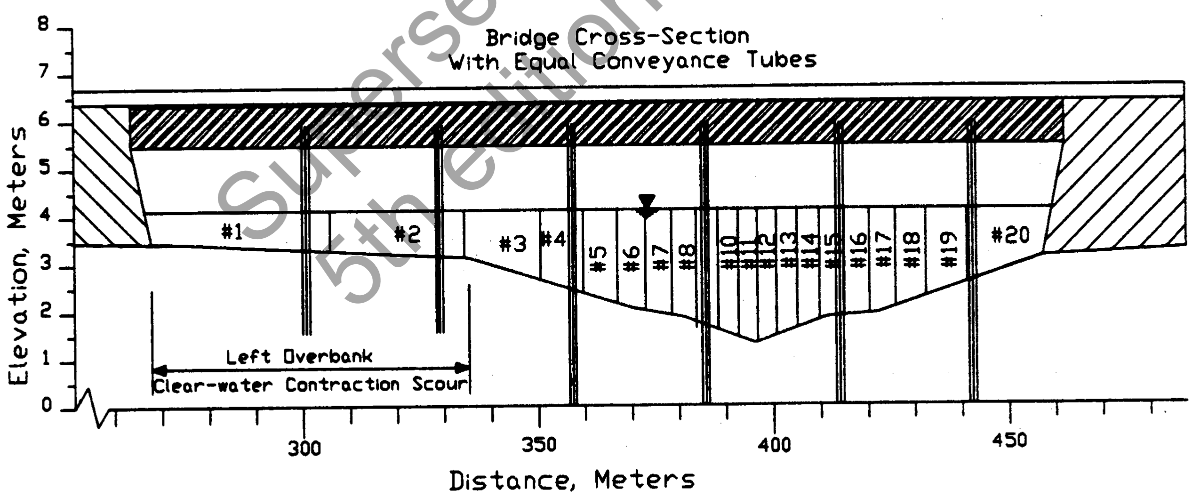

Figure 8.3. Equal conveyance tubes of bridge section.

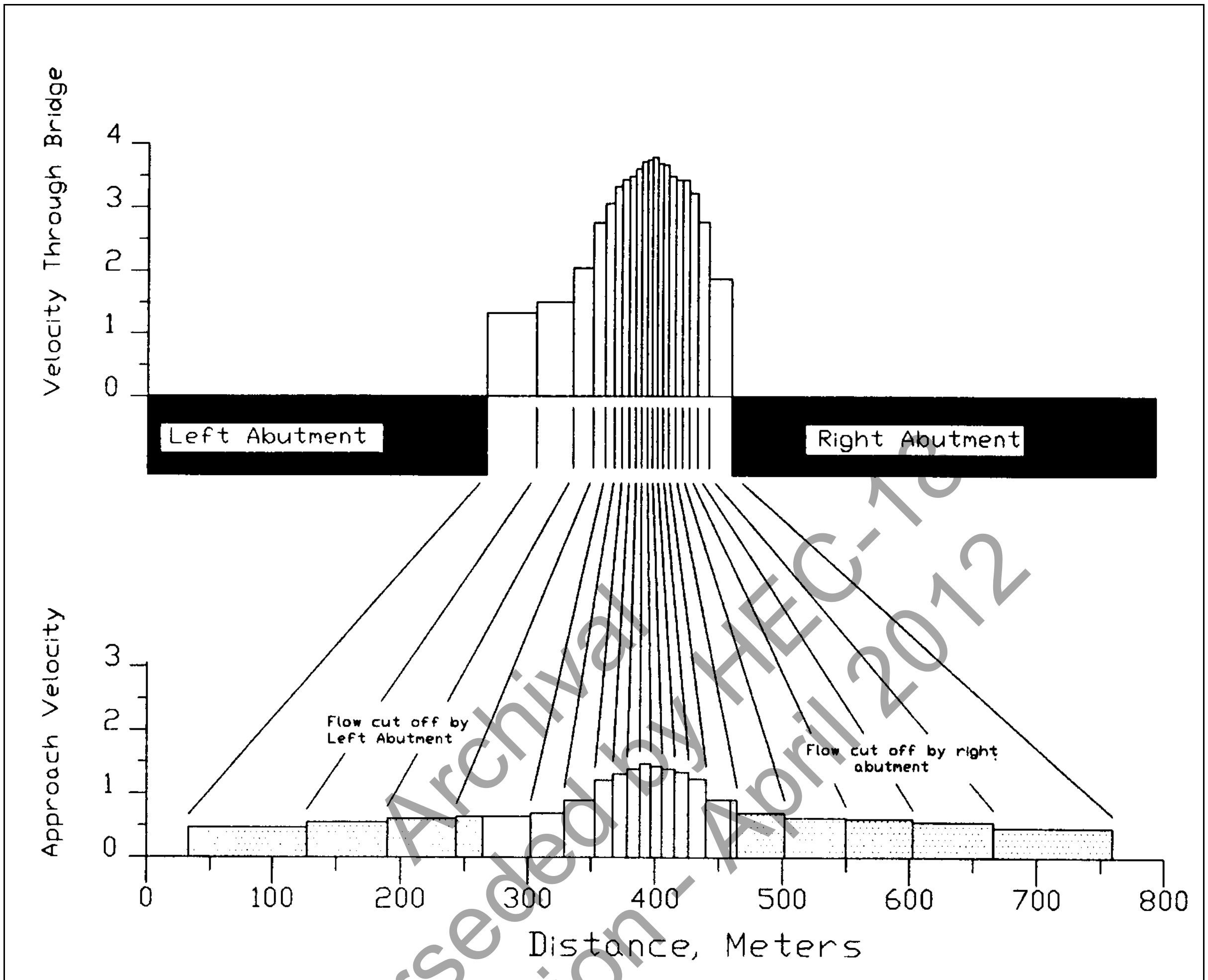

Figure 8.4. Plan view of equal conveyance tubes showing velocity distribution at approach and bridge sections.

Table 8.1. Hydraulic Variables from WSPRO for Estimation of Live-bed Contraction Scour.

		Remarks
Q (m^3/s)	849.51	Total discharge, line 8 of WSPRO input or Line 26 of WSPRO output.
K_1 (Approach)	19 000	Conveyance of main channel of approach. Line 378 of WSPRO output, SA#2.
K_{total} (Approach)	39 150	Total conveyance of approach section. Line 380 of WSPRO output.
W_1 or TOPW (Approach) (m)	121.9	Topwidth of flow (TOPW). Assumed to represent active live bed width of approach. Line 378 of WSPRO output, SA#2.
A_c (Approach) (m^2)	320	Area of main channel approach section. Line 378, SA#2.
WETP (Approach) (m)	122.0	Wetted perimeter of main channel approach section. Line 378 of WSPRO output, SA#2.
K_c (Bridge)	11 330	Conveyance of main channel through bridge. Line 333 of WSPRO output, SA#2.
K_{total} (Bridge)	12 540	Total conveyance through bridge. Line 334 of WSPRO output.
A_c (Bridge) (m^2)	236	Area of the main channel, bridge section. Line 333 of WSPRO output, SA #2.
W_c (Bridge) (m)	122	Channel width at the bridge. Difference between subarea break-points defining banks at bridge, line 109 of WSPRO output.
W_2 (Bridge) (m)	115.9	Channel width at bridge, less 4 channel pier widths (6.08 m).
S_f (m/m)	0.002	Average unconstricted energy slope (SF). Line 260, or 266 of WSPRO output.

Table 8.2. Hydraulic Variables from WSPRO for Estimation of Clear-water Contraction Scour on Left Overbank.

		Remarks
Q (m^3/s)	849.51	Total discharge, (see Table 8.1).
Q_{chan} (Bridge) (m^3/s)	767.54	Flow in main channel at bridge. Determined in live-bed computation of step 3A.
Q_2 (Bridge) (m^3/s)	81.97	Flow in left overbank through bridge. Determined by subtracting Q_{chan} (listed above) from total discharge through bridge.
D_m (Bridge Overbank) (m)	0.0025	Grain size of left overbank area. $D_m = 1.25\ D_{50}$.
$W_{setback}$ (Bridge)(m)	68.8	Topwidth of left overbank area (SA #1) at bridge. Line 332, of WSPRO output.
$W_{contracted}$ (Bridge) (m)	65.8	Set back width less two pier widths (3.04 m)
A_{left} (Bridge) (m^2)	57	Area of left overbank at the bridge. Line 332 of WSPRO output, SA #1.

Table 8.3. Hydraulic Variables from WSPRO for Estimation of Pier Scour (Conveyance Tube Number 12).

		Remarks
V_1 (m/s)	3.73	Velocity in conveyance tube #12. Line 314 of WSPRO output.
Y_1 (m)	2.84	Mean depth of tube #12. Line 315 of WSPRO output.

Table 8.4. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using Froehlich's Equation for Left Abutment.		
		Remarks
Q (m^3/s)	849.51	Total discharge (Table 8.1)
q_{tube} (m^3/s)	42.48	Discharge per equal conveyance tube, defined as total discharge divided by 20.
#Tubes	3.5	Number of approach section conveyance tubes which are obstructed by left abutment. Determined by super-imposing abutment geometry onto the approach section (Figure 8.4)
Q_e (m^3/s)	148.68	Flow in left overbank obstructed by left abutment and approach embankment. Determined by multiplying # Tubes and q_{tube}.
A_e (left abut.) (M^2)	264.65	Area of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 347 of WSPRO output.
L (m)	232.80	Length of abutment projected into flow, determined by adding top widths of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 346 of WSPRO output.
L′ (m)	169.4	Length of active flow obstructed by embankment. Width or approach section conveyance tube directly upstream of abutment times the number of conveyance tubes blocked by embankment. (290.5 - 242.1) x 3.5 = 169.4 Note: Conveyance tube widths from line 346 of WSPRO output.

Table 8.5. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Left Abutment.		
		Remarks
V_{tube} (m/s) (Bridge x-Section)	1.29	Mean velocity of conveyance tube #1, adjacent to left abutment. Line 304 of WSPRO output.
y_1 (m) (Bridge x-Section)	0.83	Average depth of conveyance tube #1. Line 305 of WSPRO output.

Table 8.6. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Right Abutment.		
		Remarks
V_{tube} (m/s)	2.19	Mean velocity of conveyance tube 20, adjacent to right abutment. Line 319 of WSPRO output.
y_1 (m)	1.22	Average depth of conveyance tube 20. Line 320 of WSPRO output.

In the overbank area adjacent to the left abutment, clear-water scour will occur. This is because the overbank areas upstream of the bridge are vegetated, and because the velocities in these areas will be low. Thus, returning overbank flow which will pass under the bridge adjacent to the left abutment will not be transporting significant amounts of material to replenish the scour on the left overbank adjacent to the left abutment.

Because of this, two computations for contraction scour will be required. The first computation, which will be illustrated in Step 3A will determine the magnitude of the contraction scour in the main channel. The second computation, which is illustrated in Step 3B will utilize the clear-water equation for the left overbank area. Hydraulic data for these two computations are presented in Tables 8.1 and 8.2 for the channel and left overbank contraction scour computations, respectively.

Table 8.3 lists the hydraulic variables which will be used to estimate the local scour at the piers (Step 5). These hydraulic variables were determined from a plot of the velocity distribution derived from the WSPRO output (Figure 8.5). For this example the highest velocities and flow depths in the bridge cross section will be used (at conveyance tube number 12). Only one pier scour computation will be completed because the possibility of thalweg shifting and lateral migration will require that all of the piers be set assuming that any pier could be subjected to the maximum scour producing variables.

Local scour at the left abutment and right abutment will be illustrated in steps 6A and B using the HIRE equation. Scour variables derived from the WSPRO output for these computations are presented in Tables 8.4 and 8.5.

8.3 STEP 2: ANALYZE LONG-TERM BED ELEVATION CHANGE

Evaluation of stage discharge relationships and cross sectional data obtained from other agencies do not indicate progressive aggradation or degradation. Also, long-term aggradation or degradation are not evident at neighboring bridges. Based on these observations, the channel is relatively stable vertically, at present. Furthermore, there are no plans to change the local land use in the watershed. The forested areas of the watershed are government-owned and regulated to prevent wide spread fire damage, and instream gravel mining is prohibited. These observations indicate that future aggradation or degradation of the channel, due to changes in sediment delivery from the watershed, are minimal.

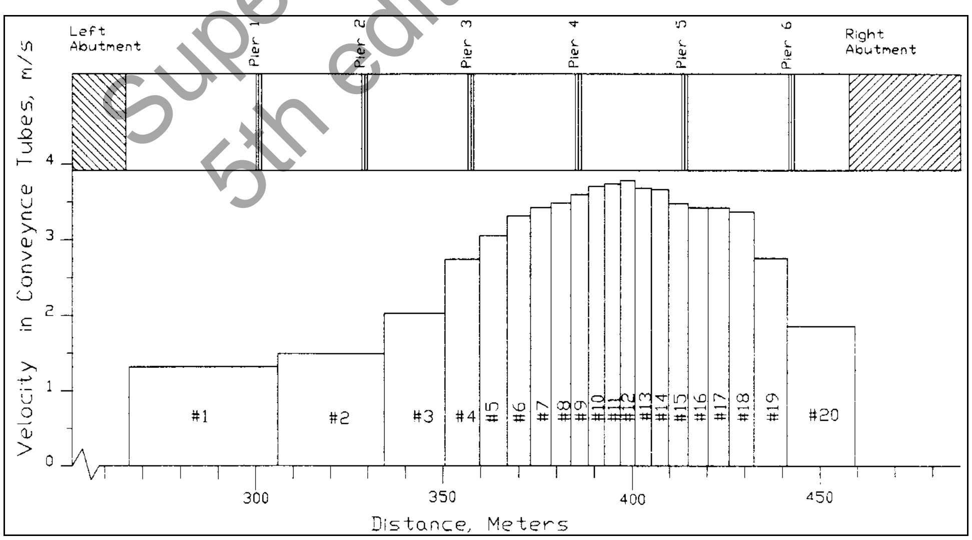

Figure 8.5. Velocity distribution at bridge crossing.

Based on these observations, and due to the lack of other possible impacts to the river reach, it is determined that the channel will be relatively stable vertically at the bridge crossing and long-term aggradation or degradation potential is considered to be minimal. However, there is evidence that the channel is unstable laterally. This will need to be considered when assessing the total scour at the bridge.

8.4 STEP 3A: COMPUTE THE MAGNITUDE OF THE GENERAL (CONTRACTION) SCOUR IN MAIN CHANNEL

As a precursor to the computation of contraction scour in the main channel under the bridge, it is first necessary to determine whether the flow condition in the main channel is either live-bed or clear-water. This is determined by comparing the critical velocity for sediment movement at the approach section to the average channel velocity of the flow at the approach section as computed using the WSPRO output. This comparison is conducted using the average velocity in the main channel of the approach section to the bridge. If the average computed channel velocity is greater than the critical velocity, the live-bed equation should be used. Conversely, if the average channel velocity is less than the critical velocity, the clear-water equation is applicable. The following computations are based on the quantities tabulated in Table 8.1.

The discharge in the main channel of the approach section is determined from the ratio of the conveyance in the main channel to the total conveyance of the approach section. By multiplying this ratio by the total discharge, the discharge in the main channel at the approach section (Q_1) is computed.

$$Q_1 = Q\ (K_1 / K_{total}) = 849.51\ m^3/s \left(\frac{19\ 000}{35\ 150}\right)$$

$$Q_1 = 412.28\ m^3/s$$

The average velocity in the main channel of the approach section is determined by dividing the discharge computed in Equation 8.1 by the cross-sectional area of the main channel.

$$V_1 = (Q_1 / A_c) = \left(\frac{412.28}{320}\right) = 1.29\ m/s$$

The average flow depth in the approach section is determined by dividing the flow area by the topwidth of the channel.

$$y_1 = \left(A_1 / TOPW\right) = \left(\frac{320}{121.9}\right) = 2.63\ m$$

The channel velocity is compared to the critical velocity of the D_{50} size for sediment movement (V_c) to determine whether the flow condition is either clear-water or live-bed.

$$V_c = 6.19\ y_1^{1/6}\ D_{50}^{1/3}$$

$$V_c = 6.19\ (2.63\ \text{m})^{1/6}\ (0.002\ \text{m})^{1/3}$$

$$V_c = 0.92\ \text{m/s}$$

Since the average velocity in the main channel is greater than the critical velocity ($V_1 > V_c$), the flow condition will be live-bed. The following computations illustrate the computation of the contraction scour using the live-bed equation.

The following computation determines the mode of bed material transport and the factor k_1. All hydraulic parameters which are needed for this computation are listed in Table 8.1.

The hydraulic radius of the approach channel is:

$$R = \frac{A_c}{WETP} = \frac{320\ \text{m}^2}{122\ \text{m}} = 2.62\ \text{m}$$

Notice that the hydraulic radius of the approach is nearly equal to the average flow depth computed earlier (Equation 8.3). This condition indicates that the channel is wide with its width greater than 10 times the flow depth. **If the width was less than 10 times the average flow depth, the channel could not be assumed to be wide and the hydraulic radius would deviate from the average flow depth**.

The average shear stress on the channel bed is:

$$\tau_o = \gamma\ R\ S$$

$$\tau_o = (9810\ \text{N/m}^3)\ (2.62\ \text{m})\ (0.002\ \text{m/m}) = 51.4\ \text{N/m}^2 = 51.4\ \text{Pa}$$

The shear velocity in the approach channel is:

$$V_* = (\tau_o / \rho)^{0.5} = (54.1 / 1000)^{0.5} = 0.227\ \text{m/s}$$

Bed material is sand with D_{50} = 0.002 m (2mm).
Fall velocity (ω) = 0.21 m/s from Figure 5.8 at 20°C and D_s = 2 mm

Therefore

$$\frac{V_*}{\omega} = \frac{0.227}{0.21} = 1.08$$

From the above, the coefficient k_1 is determined (from the discussion for Equation 5.2) to be equal to 0.64 which indicates that the mode of bed material transport is a mixture of suspended and contact bed material discharge.

The discharge in the main channel at the bridge (Q_2) is determined from the ratio of conveyances for the bridge section. This procedure for obtaining the discharge is similar to the procedure used to obtain the discharge in the main channel of the approach which was previously illustrated in Equation 8.1.

$$Q_2 = Q\,(K_2 / K_{total}) = 849.51\ m^3/s \left(\frac{11\,330}{12\,540}\right)$$

$$Q_2 = 767.54\ m^3/s$$

The channel widths at the approach and bridge section are given in Table 8.1. Therefore all parameters to determine live-bed contraction scour have been determined and Equation 5.2 can be employed.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1}$$

$$\frac{y_2}{2.63} = \left(\frac{767.54}{412.28}\right)^{6/7} \left(\frac{121.9}{115.9}\right)^{0.64} = 1.76$$

$$y_2 = (2.63)(1.76) = 4.63\text{m}$$

Live-bed contraction scour is calculated by subtracting the flow depth in the bridge (y_0) from y_2. The bridge channel flow depth (y_0) is the area divided by the topwidth, y_0 = 236 m^2/122 m = 1.93 m. Therefore, the depth of contraction scour in the main channel is:

$$y_s = y_2 - y_0 = 4.63\ m - 1.93\ m = 2.7\ m$$

This amount of contraction scour is large and could be minimized by increasing the bridge opening, providing for relief bridges in the overbank, or in some cases, providing for highway approach overtopping.

If this were the design of a new bridge, the excessive backwater (0.61 m) would require a change in the design to meet FEMA backwater requirements. The increase in backwater is obtained by subtracting the elevation given in line 264 from the elevation given in line 281 in Appendix G. However, in the evaluation of an existing bridge for safety from scour, this amount of contraction scour could occur and the scour analysis should proceed.

8.5 STEP 3B: COMPUTE GENERAL (CONTRACTION) SCOUR FOR LEFT OVERBANK

Clear-water contraction scour will occur in the overbank area between the left abutment and the left bank of bridge opening. Although the bed material in the overbank area is soil, it is protected by vegetation. Therefore, there would be no bed-material transport into the set-back bridge opening (clear-water conditions). The subsequent computations are based on the discharge and depth of flow passing under the bridge in the left overbank. These hydraulic variables were determined from the WSPRO output and are tabulated in Table 8.2.

Computation of clear-water contraction scour (Equation 5.4)

$$y_2 = \left[\frac{0.025\ Q^2}{(D_m^{2/3}\ W_{contracted}{}^2)}\right]^{3/7}$$

Computation of contraction scour flow depth in left overbank area under the bridge, y_2:

$$y_2 = \left[\frac{0.025\ (81.97\ m^3 / s)^2}{(0.0025\ m)^{2/3}\ (65.8\ m)^2} \right]^{3/7} = 1.38\ m$$

Computation of average flow depth in left overbank bridge section, y_0:

$$y_0 = \frac{A}{TOPW} = \frac{(57.0\ m^2)}{(68.8\ m)} = 0.83\ m$$

Therefore, the clear-water contraction scour in the left overbank of the bridge opening is:

$$y_s = y_2 - y_0 = 1.38\ m - 0.83\ m = 0.55\ m$$

8.6 STEP 4: COMPUTE THE MAGNITUDE OF OTHER GENERAL SCOUR COMPONENTS

The crossing is on a relatively straight reach with no channel braiding, and there are no downstream controls of water surface elevations. Thus, the other general scour components (bend scour, confluence scour, etc) will not be a factor.

8.7 STEP 5: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT PIERS

It is anticipated that any pier under the bridge could potentially be subject to the maximum flow depths and velocities derived from the WSPRO hydraulic model (Table 8.3). Therefore, only one computation for pier scour is conducted and assumed to apply to each of the six piers for the bridge. This assumption is appropriate based on the fact that the thalweg is prone to shifting and because there is a possibility of lateral channel migration.

8.7.1 Computation of Pier Scour

The Froude Number for the pier scour computation is based on the hydraulic characteristics of conveyance tube number 12. Therefore:

$$Fr_1 = \frac{V}{(g\ y_1)^{0.5}} = \frac{3.73\ m / s}{[(9.81\ m / s^2)\ (2.84\ m)]^{0.5}}$$

$$Fr_1 = 0.71$$

For a round-nose pier, aligned with the flow and sand-bed material:

$$K_1 = K_2 = K_4 = 1.0$$

For plane-bed condition:

$K_3 = 1.1$

Using Equation 6.3:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4 \left(\frac{a}{y_1}\right)^{0.65} Fr_1^{0.43}$$

$$\frac{y_s}{2.84} = 2\ (1)\ (1)\ (1.1)\ (1) \left(\frac{1.52\ m}{2.84\ m}\right)^{0.65} (0.71)^{0.43}$$

$$\frac{y_s}{2.84} = 1.26$$

$$y_s = 3.6\ m$$

From the above computation the maximum local pier scour depth will be 3.6 m.

8.7.2 Correction for Angle of Attack

The above computation assumes that the piers are aligned with the flow (skew angles are less than 5°). However, if the piers were skewed to the flow by more than 5°, the value of y_s/y_1, as computed above, would need to be adjusted by K_2. The following computations illustrate the adjustment for piers skewed 10°.

$$\frac{L}{a} = \frac{12.2\ m}{1.52\ m} = 8$$

K_2 can then be obtained by using Equation 6.4 for an L/a of 8 and a 10° angle of attack. For this example, K_2=1.67. Applying this correction:

$$\frac{y_s}{2.84} = 1.67\ (1.26) = 2.1$$

$$y_s = 6.0\ m$$

Therefore, the maximum local pier scour depth for a pier angled 10° to the flow is 6.0 m.

8.7.3 Discussion of Pier Scour Computation

Although the estimated local pier scour would probably not occur at each pier, the possibility of thalweg shifting, which was identified in the Level 1 analysis, precludes setting the piers at different depths even if there were a substantial savings in cost. This is because any of the piers could be subjected to the worst-case scour conditions.

It is also important to assess the possibility of lateral migration of the channel. This possibility can lead to directing the flow at an angle to the piers, thus increasing local scour.

Countermeasures to minimize this problem could include riprap for the channel banks both up- and downstream of the bridge, and installation of guide banks to align flow through the bridge opening.

The possibility of lateral migration precludes setting the foundations for the overbank piers at a higher elevation. Therefore, in this example the foundations for the overbank piers should be set at the same elevation as the main channel piers.

8.8 STEP 6A: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT LEFT ABUTMENT

8.8.1 Computation of Abutment Scour Depth Using Froehlich's Equation

For spill-through abutments, K_1 =0.55. For this example, the abutments are set perpendicular to the flow; therefore, K_2=1.0. Abutment scour can be estimated using Froehlich's equation with data derived from the WSPRO output (Table 8.4).

The y_a value at the abutment is assumed to be the average flow depth in the overbank area. It is computed as the cross-sectional area of the left overbank cut off by the left abutment divided by the distance the left abutment protrudes into the overbank flow.

$$y_a = \frac{A_e}{L} = \frac{264.65\,m^2}{232.80\,m} = 1.14\,m$$

The average velocity of the flow in the left overbank (Figure 8.4) which is cut off by the left abutment is computed as the discharge cutoff by the abutment divided by the area of the left overbank cut off by the left abutment.

$$V_e = \frac{Q_e}{A_e} = \frac{148.68\,m^3/s}{264.65\,m^2} = 0.56\,m/s$$

Using these parameters, the Froude Number of the overbank flow is:

$$Fr = \frac{V_e}{(g\ y_a)^{1/2}} = \frac{0.56\,m/s}{[(9.81\,m/s^2)\,(1.14\ m)]^{0.5}} \qquad (8.25)$$

Fr = 0.17

Using Froehlich's equation (Equation 7.1):

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2\ \left(\frac{L'}{y_a}\right)^{0.43}\ Fr^{0.61} + 1$$

$$\frac{y_s}{1.14} = 2.27\,(0.55)\ (1.0)\ \left(\frac{169.4}{1.56}\right)^{0.43}\ (0.17)^{0.61} + 1$$

$$\frac{y_s}{1.14\,m} = 4.64$$

$$y_s = 5.3\,m$$

Using Froehlich's equation, the abutment scour at the left abutment is computed to be 5.9 m.

8.8.2 Computation of Abutment Scour Depth Using the HIRE Equation

The HIRE equation for abutment scour is applicable for this situation because L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the abutment end which is listed in Table 8.5. Therefore, the Froude Number of this flow is:

$$Fr_1 = \frac{1.29\,m/s}{[(9.81\,m/s^2)\,(0.83m)]^{0.5}} = 0.45$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0 (Equation 7.2):

$$\frac{y_s}{0.83\ m} = 4\ Fr_1^{0.33} = 4\ (0.45)^{0.33} = 3.07$$

$$y_s = 2.6\ m$$

From the above computation, the depth of scour at the left abutment as computed using the HIRE equation, is 2.6 m.

8.9 STEP 6B: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT RIGHT ABUTMENT

The HIRE equation for abutment scour is also applicable for the right abutment since L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the end of the right abutment and listed in Table 8.6. The Froude Number of this flow is:

$$Fr_1 = \frac{2.19\,m/s}{[(9.81\,m/s^2)\,(1.22\ m)]^{0.5}} = 0.63$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0:

$$\frac{y_s}{1.22 \text{ m}} = 4 \; Fr_1^{0.33} = 4 \; (0.63)^{0.33} = 3.43$$

$$y_s = 4.2 \text{ m}$$

From the above computation, the depth of scour at the right abutment, as computed using the HIRE equation is 4.2 m.

8.10 DISCUSSION OF ABUTMENT SCOUR COMPUTATIONS

Abutment scour as computed using the Froehlich equation(70) will generally result in deeper scour predictions than will be experienced in the field. These scour depths could occur if the abutments protruded into the main channel flow, or when a uniform velocity field is cut off by the abutment in a manner that most of the returning overbank flow is forced to return to the main channel at the abutment end. For most cases, however, when the overbank area, channel banks and area adjacent to the abutment are well vegetated, scour depths as predicted with the Froehlich equation will probably not occur.

All of the abutment scour computations (left and right abutments) assumed that the abutments were set perpendicular to the flow. If the abutments were angled to the flow, a correction utilizing K_2 would be applied to Froehlich's equation and to the equation from HDS 6.(22) However the adjustment for skewed abutments is minor when compared to the magnitude of the computed scour depths. For example, if the abutments for this example problem were angled 30° upstream ($\theta = 90° + 30° = 120°$), the correction for skew would increase the computed depth of abutment scour by no more than 3 to 4 percent for the Froehlich and HIRE equation, respectively.

8.11 STEP 7: PLOT TOTAL SCOUR DEPTH AND EVALUATE DESIGN

As a final step, the results of the scour computations are plotted on the bridge cross section and carefully evaluated (Figure 8.6). For this example, only the computations for pier scour with piers aligned with the flow were plotted and the abutment scour computations reflect the results from the HIRE equation. The topwidth of the local scour holes is suggested as 2.0 times y_s.

It is important to evaluate carefully the results of the scour computations. For example, although the total scour plot indicates that the total scour at the overbank piers is less than for the channel piers, this does not indicate that the foundations for the overbank piers can be set at a higher elevation. Due to the possibility of channel and thalweg shifting, all of the piers should be set to account for the maximum total scour. Also, the computed contraction scour is distributed uniformly across the channel in Figure 8.6. However, in reality this may not be what would happen. With the flow from the overbank area returning to the channel, the contraction scour could be deeper at both abutments. The use of guide banks would distribute the contraction scour more uniformly across the channel. This would make a strong case for guide banks in addition to the protection they would provide to the abutments. The stream tube velocities could be used to distribute the scour depths across this section.

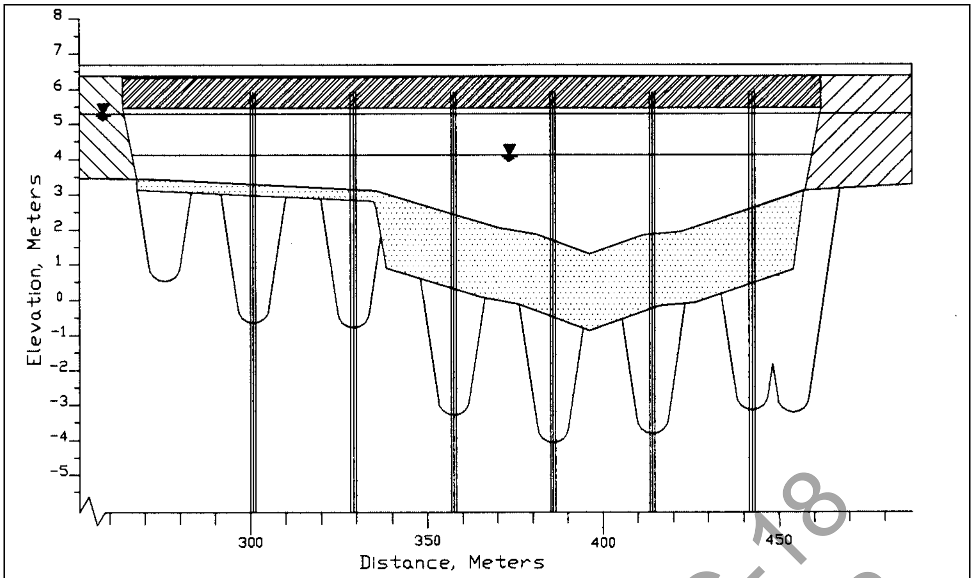

Figure 8.6. Plot of total scour for example problem.

The plot of the total scour also indicates that there is a possibility of overlapping scour holes between the sixth pier and right abutment, and it is not clear from where the right abutment scour should be measured, since the abutment is located at the channel bank. Both of these uncertainties should be avoided for replacement and new bridges whenever possible. Consequently, it would be advisable to set the right abutment back from the main channel. This would also tend to reduce the magnitude of contraction scour in the main channel.

The possibility of lateral migration of the channel will have an adverse effect on the magnitude of the pier scour. This is because lateral migration will most likely skew the flow to the piers. This problem can be minimized by using circular piers. An alternative approach would be to install guide banks to align the flow through the bridge opening.

A final concern relates to the location and depth of contraction scour in the main channel near the second pier and toe of the right abutment. At these locations, contraction scour in the main channel could increase the bank height to a point where bank failure and sloughing would occur. It is recommended that the existing bank lines be protected with revetment (i.e., riprap, gabions, etc.). Since the river has a history of channel migration, the bridge inspection and maintenance crews should be briefed on the nature of this problem so that any lateral migration can be identified.

The plot of the scour prism in Figure 8.6 should be reploted to show the potential for the scour to occur at any location in the bridge opening. This is shown in Figure 8.7

8.12 COMPLETE THE GENERAL DESIGN PROCEDURE

This design problem uses Steps 1 through 7 of the specific design approach (Chapter 2) and completes Steps 1 through 6 of the general design procedure in Chapter 2. The design must now proceed to Steps 7 and 8, which include bridge foundation analysis and consideration of the check for superflood. This is not done for this example problem.

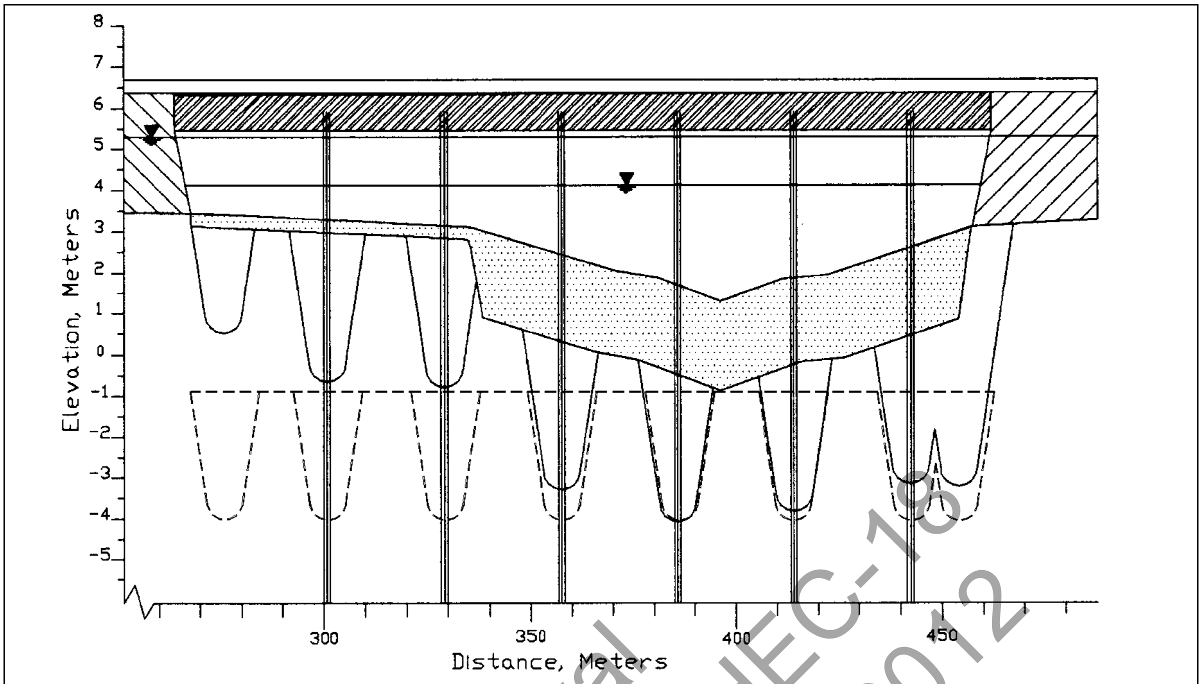

Figure 8.7. Revised plot of total scour for example problem.

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CHAPTER 9

SCOUR ANALYSIS FOR TIDAL WATERWAYS

9.1 INTRODUCTION

In the coastal region, scour at bridges over tidal waterways that are subjected to the effects of astronomical tides and storm surges is a combination of long-term degradation, contraction scour, local scour, and waterway instability. These are the same scour mechanisms that affect non-tidal (riverine) streams. Although many of the flow conditions are different in tidal waterways, the equations used to determine riverine scour are applicable if the hydraulic conditions (depth, discharge, velocity, etc.) are carefully evaluated.[23, 24]

This chapter presents methods and equations for determining stream stability and scour at tidal inlets, tidal estuaries, bridge crossings to islands and streams affected by tides (tidal waterways). Analysis of tidal waterways is very complex. The hydraulic analysis must consider the magnitude of the 100- and 500-year storm surge (storm tide - see Section 9.2 Glossary), the characteristics (geometry) of the tidal inlet, estuary, bay or tidal stream and the effect of any constriction of the flow due to the bridge. In addition, the analysis must consider the long-term effects of the normal tidal cycles on long-term aggradation or degradation, contraction scour, local scour, and stream instability. Coastal analyses require a synthesis of complex meteorological, bathymetric, geographical, statistical, and hydraulic disciplines and knowledge. The methods and equations presented in this chapter provide an overview of application of these elements in the context of tidal scour analyses.

A storm tide or storm surge in coastal waters results from astronomical tides, wind action, and rapid barometric pressure changes. In addition, the change in elevation resulting from the storm surge may be increased by resonance in harbors and inlets, whereby, the tidal range in an estuary, bay, or inlet is larger than on the adjacent coast.

The astronomical tidal cycle with reversal in flow direction can increase long-term degradation, contraction scour, and local scour. If sediment is being moved on the flood and ebb tide, there may be no net loss of sediment in a bridge reach because sediments are being moved back and forth. Consequently, no net long-term degradation may occur. However, local scour at piers and abutments can occur at both the inland and ocean side of the piers and abutments and will alternate with the reversal in flow direction. If, however, there is a loss of sediment in one or both flow directions, there will then be long-term degradation in addition to local scour. Also, the tidal cycles may increase bank erosion, migration of the channel, and thus, increase stream instability.

The complexity of the hydraulic analysis increases if the tidal inlet or the bridge constrict the flow and affect the amplitude of the storm surge (storm tide) in the bay or estuary so that there is a large change in elevation between the ocean and the estuary or bay. A constriction in the tidal inlet can increase the velocities in the constricted waterway opening, decrease interior wave heights and tidal range, and increase the phase difference (time lag) between exterior and interior water levels. Analysis of a constricted inlet or waterway may require the use of an orifice equation rather than tidal relationships.

For the analysis of bridge crossings of tidal waterways, a three-level analysis approach similar to the approach outlined in HEC-20 is suggested.[6] **Level 1** includes a qualitative evaluation of the stability of the inlet or estuary, estimating the magnitude of the tides, storm surges, and flow in the tidal waterway, and attempting to determine whether the hydraulic analysis depends on tidal or river conditions, or both. **Level 2** represents the engineering analysis necessary to obtain the velocity, depths, and discharge for tidal waterways to be used in determining long-term aggradation, degradation, contraction scour, and local scour. The hydraulic variables obtained from the Level 2 analysis are used in the riverine equations presented in previous chapters to obtain total scour. Using these riverine scour equations, which are for steady-state equilibrium conditions for unsteady, dynamic tidal flow may result in estimating deeper scour depths than will actually occur (conservative estimate), but this represents the state of knowledge at this time for this level of analysis.

For complex tidal situations, **Level 3** analysis using physical and 2-dimensional computer models may be required. This section will be limited to a discussion of Levels 1 and 2 analyses. In Level 2 analyses, unsteady 1-dimensional or quasi 2-dimensional computer models may be used to obtain the hydraulic variables needed for the scour equations. **The Level 1, 2, and 3 approaches are described in more detail in later sections.**

The steady-state equilibrium scour equations given in previous sections of this manual are suitable for use to determine scour depths in tidal flows. As mentioned earlier, tidal flows resulting from storm surges are unsteady but no more so than most unsteady riverine flows. For both cases, scour depths are conservative.

9.2 OVERVIEW OF TIDAL PROCESS

9.2.1 Glossary

Bay A body of water connected to the ocean with an inlet.

Diurnal tide Tides with an approximate tidal period of 24 hours.

Ebb or ebb tide Flow of water from the bay or estuary to the ocean.

Estuary Tidal reach at the mouth of a river.

Flood or flood tide Flow of water from the ocean to the bay or estuary.

Littoral transport or drift Transport of beach material along a shoreline by wave action. Also, longshore sediment transport.

Run-up, wave Height to which water rises above still-water elevation when waves meet a beach, wall, etc.

Semi-diurnal tide Tides with an approximate tidal period of 12 hours.

Set-up, wave Height to which water rises above still-water elevation as a result of storm wind effects.

Still-water elevation Flood height to which water rises as a result of barometric pressure changes occurring during a storm event.

Storm surge Coastal flooding phenomenon resulting from wind and barometric changes. The storm surge is measured by subtracting the astronomical tide elevation from the total flood elevation (Hurricane surge).

Storm tide Coastal flooding resulting from combination of storm surge and astronomical tide (often referred to as storm surge)

Tidal amplitude Generally, half of tidal range.

Tidal cycle One complete rise and fall of the tide.

Tidal day Time of rotation of the earth with respect to the moon. Assumed to equal approximately 24.84 solar hours in length.

Tidal inlet A channel connecting a bay or estuary to the ocean.

Tidal passage A tidal channel connected with the ocean at both ends.

Tidal period Duration of one complete tidal cycle. When the tidal period equals the tidal day (24.84 hours), the tide exhibits diurnal behavior. Should two complete tidal periods occur during the tidal day, the tide exhibits semi-diurnal behavior.

Tidal prism Volume of water contained in a tidal bay, inlet or estuary between low and high tide levels.

Tidal range Vertical distance between specified low and high tide levels.

Tidal waterways A generic term which includes tidal inlets, estuaries, bridge crossings to islands or between islands, inlets to bays, crossings between bays, tidally affected streams, etc.

Tides, astronomical Rhythmic diurnal or semi-diurnal variations in sea level that result from gravitational attraction of the moon and sun and other astronomical bodies acting on the rotating Earth.

Tsunami Long-period ocean wave resulting from earthquake, other seismic disturbances or submarine land slides.

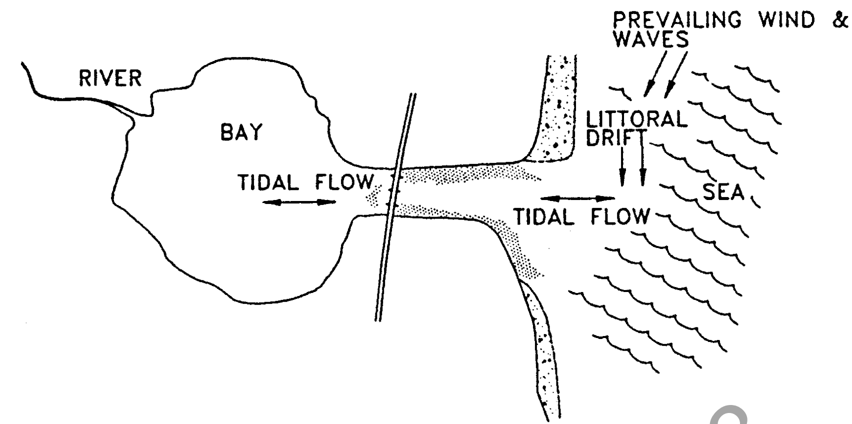
Waterway opening Width or area of bridge opening at a specific elevation, measured normal to principal direction of flow.

Wave period Time interval between arrivals of successive wave crests at a point.

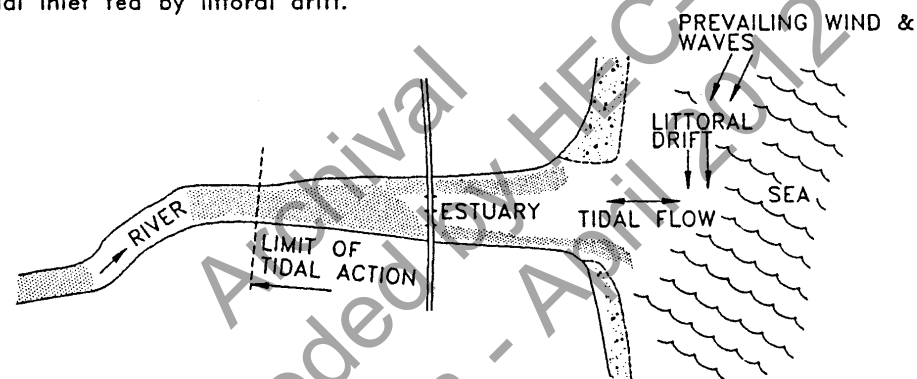
9.2.2 Definition of Tidal and Coastal Processes

Typical bridge crossings of tidal waterways are sketched in Figure 9.1. From this figure, tidal flows can be defined as being between the ocean and a bay (or lagoon), from the ocean into an estuary, or through passages between islands.

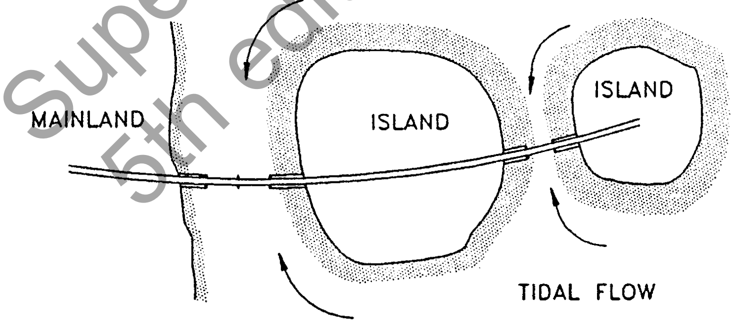
Flow into (flood tide) and out of (ebb tide) a bay or estuary is driven by tides and by the discharge into the bay or estuary from upland areas. Assuming that the flow from upland areas is negligible, the ebb and flood in the bay or estuary will be driven solely by tidal fluctuations and storm surges as illustrated in Figure 9.2. With no inflow of water from rivers and streams, the net flow of water into and out of the bay or estuary will be nearly zero. Increasing the discharge from rivers and streams will lead to a net outflow of water to the ocean.



1. Inlets between the open sea and an enclosed lagoon or bay, where most of the discharge results from tidal flows. Tidal inlet fed by littoral drift.



2. River estuaries where the net discharge comprises river flow as well as tidal flow components



3. Passages between islands, or between an island and the mainland, where a route to the open sea exists in both directions.

Figure 9.1. Types of tidal waterway crossings (after Neill).(78)

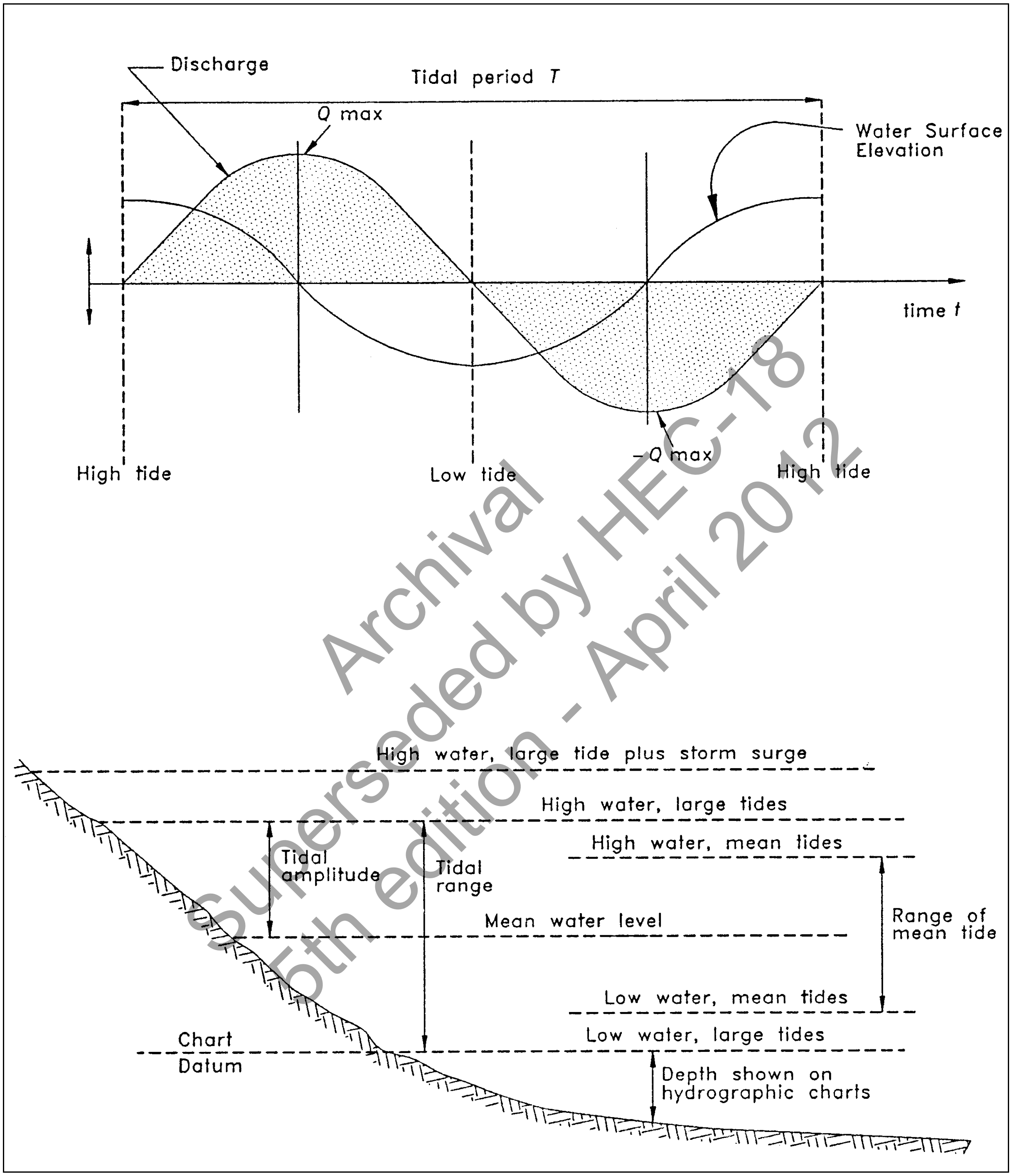


Figure 9.2. Principal tidal terms (after Neill).[78]

Figure 9.2 illustrates the elevation and time variable nature of astronomical tides. For astronomical tides, maximum flood and ebb (or the time of maximum current and discharge) can be assumed to occur at the inflection point of (or halfway between) high tide and low tide, but actually can occur before or after the midtide level depending on the location. The addition of a storm surge to a high astronomical tide can lead to additional water surface elevations (High water, large tide plus storm surge in Figure 9.2), additional current, and associated flooding.

In the most conservative scenario, the greatest potential flood elevation would occur at the time where the high astronomical tide and maximum storm surge height coincide in time. In this circumstance, the maximum discharge would occur when the astronomical tidal period and the period associated with the storm surge event are the same value. The presence of any inland flood discharge would influence this discharge, particularly during the period when the flood levels recede (ebb).

Hydraulically, the above discussion presents two limiting cases for evaluation of the flow velocities in the bridge reach. With negligible flow from the upland areas, the flow through the bridge opening is based solely on the ebb and flood resulting from tidal fluctuations or storm surges. Alternatively, when the flow from the streams and rivers draining into the bay or estuary (inland flood) is large in relationship to the tidal flows (ebb and flood tide), the effects of tidal fluctuations are negligible. For this latter case, the evaluation of the hydraulic characteristics and scour can be accomplished using the methods described in previous chapters for inland rivers.

Bridge scour in the coastal region results from the unsteady diurnal and semi-diurnal flows resulting from astronomical tides, large flows that can result from storm surges (hurricanes, nor'easters), and the combination of riverine and tidal flows. The forces which drive tidal fluctuations are, primarily, the result of the gravitational attraction of the sun and moon on the rotating earth (astronomical tides), wind and storm setup, and geologic disturbances (tsunamis). These different forces which drive tides produce varying tidal periods and amplitudes. In general semi-diurnal astronomical tides having tidal periods of approximately 12 hours occur in the lower latitudes while diurnal tides having tidal periods of approximately 24 hours occur in the higher latitudes. Typically, the storm surge period correlates with the associated storm type. Hurricane surges generally last from 12 to 15 hours. Nor'easters may produce a storm surge lasting several days. In general, storm surge periods may be assumed to be longer than astronomical tidal periods.

The continuous rise and fall of astronomical tides will usually influence long-term trends of aggradation or degradation, contraction and local scour. Worst-case hydraulic conditions for contraction and local scour are usually the result of infrequent tidal events such as storm surges and tsunamis. Storm surges and tsunamis are a single event phenomenon which, due to their magnitude, can present a significant threat to a bridge crossing in terms of scour. The hydraulic variables (discharge, velocity, and depths) and bridge scour in the coastal region can be determined with as much precision as riverine flows. These determinations are conservative and research is needed for both cases to improve scour determinations. Determining the magnitude of the combined flows can be accomplished by simply adding riverine flood flow to the maximum tidal flow, if the drainage basin is small, or routing the design riverine flows to the crossing and adding them to the storm surge flows.

The small size of the bed material (normally fine sand) as well as silts and clays with cohesion and littoral drift (transport of beach sand along the coast resulting from wave action) affect the magnitude of bridge scour. Mass density stratification of the water typically has a minor influence on bridge scour. Peak flows from storm surges may not have durations long enough to reach the ultimate scour depths determined from existing scour equations. Sediment transport equations can be used to compute the rate of contraction scour (see Section 9.6), but the time dependent characteristics of local scour require further research. Diurnal and semi-diurnal astronomical tides can cause long-term degradation if there is no source of sediment except at the crossing. At some locations, this has resulted in long-term degradation of 0.3 to 1.0 m (1.0 to 3.3 ft) per year with no indication of stopping.[79, 80] Existing scour equations can predict the magnitude of this scour, but not the time history.[23, 24]

Mass density stratification (saltwater wedges), which can result when the denser more saline ocean water enters an estuary or tidal inlet with significant freshwater inflow, can result in larger velocities near the bottom than the average velocity in the vertical velocity profile. With careful evaluation, the correct velocity can be determined for use in the scour equations. With storm surges, mass density stratification will not normally occur. The density difference between salt and freshwater, except as it causes saltwater wedges, is not significant enough to affect scour equations. Density and viscosity differences between fresh and sediment-laden water can be much larger in riverine flows than the density and viscosity differences between salt and freshwater.

Salinity can affect the transport of silts and clays by causing them to flocculate and possibly deposit, which may affect stream stability and must be evaluated. Salinity may affect the erodibility of cohesive sediments, but this will only affect the rate of scour, not ultimate scour. Littoral drift is a source of sediment to a tidal waterway.[81, 82] An aggrading or stable waterway may exist if the supply of sediment to the bridge from littoral drift is large. This will have the effect of minimizing contraction scour, and possibly local scour. Conversely, long-term degradation, contraction scour and local scour can be exacerbated if the sediment from littoral drift is reduced or cut off. Evaluating the effect of littoral drift is a sediment transport problem involving historical information, future plans (dredging, jetties, etc.) for the waterway and/or the coast, sources of sediment, and other factors.

Evaluation of total scour at bridges crossing tidal waterways requires the assessment of long-term aggradation or degradation, local scour and contraction scour. Long-term aggradation or degradation estimates can be derived from a geomorphic evaluation coupled with computations of live-bed contraction scour if sediment transport is changed.

Although the hydraulics of flow for tidal waterways is complicated by the presence of two directional flow, the basic concept of sediment continuity is valid. Consequently, a clear understanding of the principle of sediment continuity is essential for evaluating scour at bridges spanning waterways influenced by tidal fluctuations. Technically, the sediment continuity concept states that the sediment inflow minus the sediment outflow equals the time rate of change of sediment volume in a given reach. More simply stated, during a given time period the amount of sediment coming into the reach minus the amount leaving the downstream end of the reach equals the change in the amount of sediment stored in that reach.

As with riverine scour, tidal scour can be characterized by either live-bed or clear-water conditions. In the case of live-bed conditions, sediment transported into the bridge reach will tend to reduce the magnitude of scour. Whereas, if no sediment is in transport to re-supply the bridge reach (clear-water), scour depths can be larger.

In addition to sediments being transported from inland areas, sediments are transported parallel to the coast by ocean currents and wave action. This littoral transport of sediment serves as a source of sediment supply to the inlet, bay or estuary, or tidal passage. During the flood tide, these sediments can be transported into the bay or estuary and deposited.

During the ebb tide, these sediments can be re-mobilized and transported out of the inlet or estuary and either be deposited on shoals or moved further down the coast as littoral transport (Figure 9.3).

Sediment transported to the bay or estuary from the inland river system can also be deposited in the bay or estuary during the flood tide, and re-mobilized and transported through the inlet or estuary during the ebb tide. However, if the bay or estuary is large, sediments derived from the inland river system can deposit in the bay or estuary in areas where the velocities are low and may not contribute to the supply of sediment to the bridge crossing. The result is clear-water scour unless sediment transported on the flood tide (ocean shoals, littoral transport) is available on the ebb. Sediments transported from inland rivers into an estuary may be stored there on the flood and transported out during ebb tide. This would produce live-bed scour conditions unless the sediment source in the estuary was disrupted. Dredging, jetties or other coastal engineering activities can limit sediment supply to the reach and influence live-bed and clear-water conditions.

Application of sediment continuity involves understanding the hydraulics of flow and availability of sediment for transport. For example, a net loss of sediment in the inlet, bay or tidal estuary could be the result of cutting off littoral transport by means of a jetty projecting into the ocean (Figure 9.3). For this scenario, the flood tide would tend to erode sediment from the inlet and deposit sediment in the bay or estuary while the ensuing ebb tide would transport sediment out of the bay or estuary. Because the availability of sediment for transport into the bay is reduced, degradation of the inlet could result. As discussed later, as the cross sectional area of the inlet increases, the flow velocities during the flood tide increase, resulting in further degradation of the inlet. This can result in an unstable inlet which continues to enlarge as a result of sediment supply depletion.

From the above discussion, it is clear that the concept of sediment continuity provides a valuable tool for evaluation of aggradation or degradation trends of a tidal waterway. Although this principle is not easy to quantify without direct measurement or hydraulic and sediment continuity modeling, the principle can be applied in a qualitative sense to assess long-term trends in aggradation or degradation.

9.3 LEVEL 1 ANALYSIS

The objectives of a Level 1 qualitative analysis are to determine the magnitude of the tidal effects on the crossing, the overall long-term stability of the crossing (vertical and lateral stability) and the potential for waterway response to change.

The first step in evaluation of highway crossings is to determine whether the bridge crosses a river which is influenced by tidal fluctuations (tidally affected river crossing) or whether the bridge crosses a tidal inlet, bay or estuary (tidally controlled). The flow in tidal inlets, bays and estuaries is predominantly driven by tidal fluctuations (with flow reversal), whereas, the flow in tidally affected river crossings is driven by a combination of river flow and tidal fluctuations. Therefore, tidally affected river crossings are not subject to flow reversal but the downstream tidal fluctuation acts as a cyclic downstream control. Tidally controlled river crossings will exhibit flow reversal.

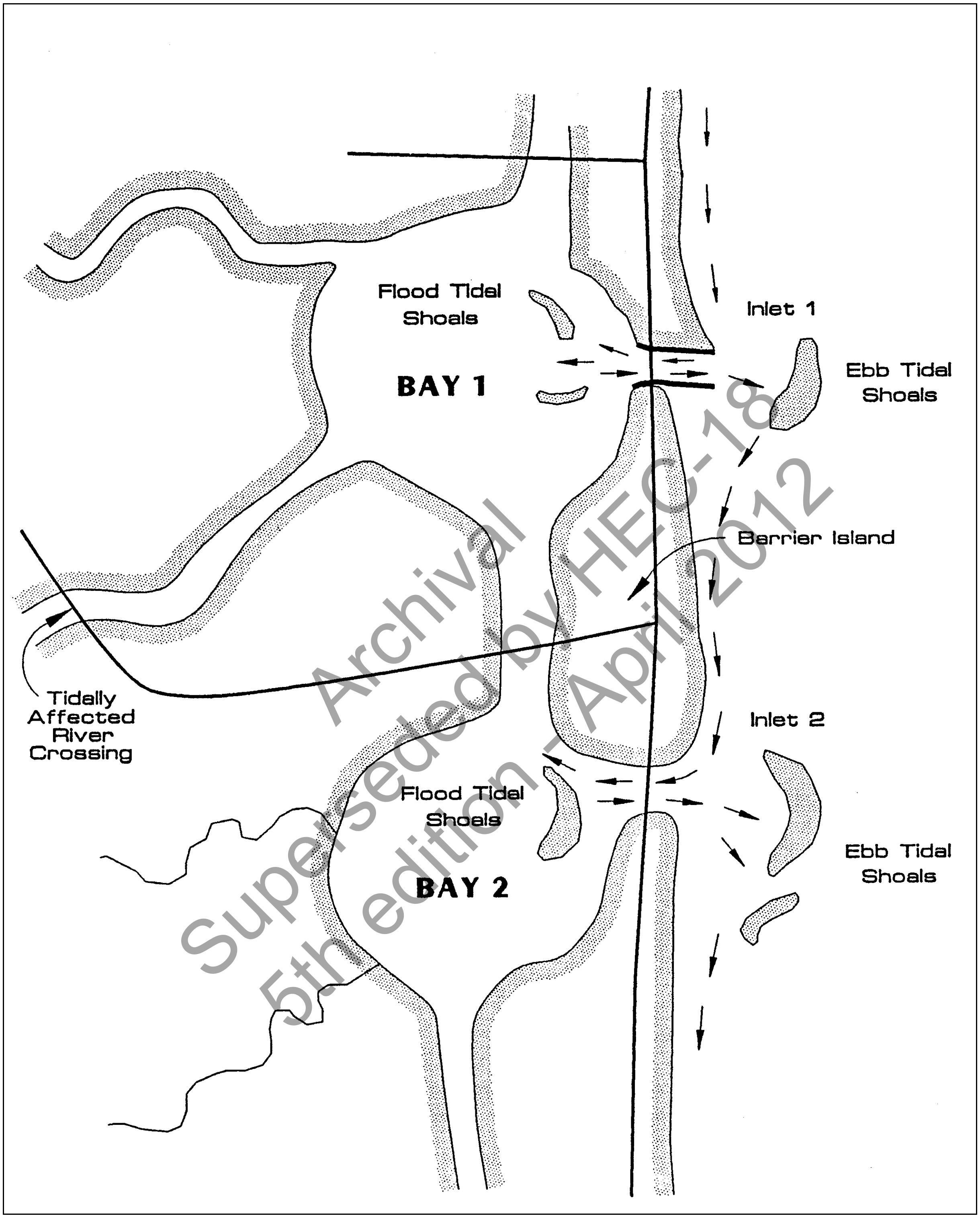


Figure 9.3. Sediment transport in tidal inlets (after Sheppard).[81]

9.3.1 Tidally Affected River Crossings

Tidally affected river crossings are characterized by both river flow and tidal fluctuations. From a hydraulic standpoint, the flow in the river is influenced by tidal fluctuations which result in a cyclic variation in the downstream control of the tail water in the river estuary. The degree to which tidal fluctuations influence the discharge at the river crossing depends on such factors as the relative distance from the ocean to the crossing, riverbed slope, cross-sectional area, storage volume, and hydraulic resistance. Although other factors are involved, relative distance of the river crossing from the ocean can be used as a qualitative indicator of tidal influence. At one extreme, where the crossing is located far upstream, the flow in the river may only be affected to a minor degree by changes in tailwater control due to tidal fluctuations. As such, the tidal fluctuation downstream will result in only minor fluctuations in the depth, velocity, and discharge through the bridge crossing.

As the distance from the crossing to the ocean is reduced, again assuming all other factors as equal, the influence of the tidal fluctuations increases. Consequently, the degree of tail water influence on flow hydraulics at the crossing increases. A limiting case occurs when the magnitude of the tidal fluctuations is large enough to reduce the discharge through the bridge crossing to zero at high tide. River crossings located closer to the ocean than this limiting case have two directional flows at the bridge crossing, and because of the storage of the river flow at high tide, the ebb tide will have a larger discharge and velocities than the flood tide.

For the Level 1 analysis, it is important to evaluate whether the tidal fluctuations will significantly affect the hydraulics at the bridge crossing. If the influence of tidal fluctuations is considered to be negligible, then the bridge crossing can be evaluated based on the procedures outlined for inland river crossings presented previously in this document. If not, then the hydraulic flow variables must be determined using dynamic tidal flow relationships. This evaluation should include extreme events such as the influence of storm surges and inland floods.

From historical records of the stream at the highway crossing, determine whether the worst-case conditions of discharge, depths and velocity at the bridge are the 100- and 500-year return period tide and storm surge, or the 100- and 500-year inland flood or a combination of the two. Historical records could consist of tidal and stream flow data from Federal Emergency Management Agency (FEMA), National Oceanic and Atmospheric Administration (NOAA), USACE, and USGS records; aerial photographs of the area; maintenance records for the bridge or bridges in the area; newspaper accounts of previous high tides and/or flood flows; and interviews in the local area.

If the primary hazard to the bridge crossing is from inland flood events, then scour can be evaluated using the methods given previously in this circular and in HEC-20.[(6)] If the primary hazard to the bridge is from tide and storm surge or tide, storm surge and inland flood runoff, then use the analyses presented in the following sections on tidal waterways. If it is unclear whether the worst hazard to the bridge will result from a storm surge, maximum tide, or from an inland flood, it may be necessary to evaluate scour considering each of these scenarios and compare the results.

9.3.2 Tidal Inlets, Bays, and Estuaries

For tidal inlets, bays and estuaries, the goal of the Level 1 analysis is to determine the stability of the inlet and identify and evaluate long-term trends at the location of the highway crossing. This can be accomplished by careful evaluation of present and historical conditions of the tidal waterway and anticipating future conditions or trends.

Existing cross-sectional and sounding data can be used to evaluate the stability of the tidal waterway at the highway crossing and to determine whether the inlet, bay or estuary is increasing or decreasing in size, or is relatively stable. For this analysis it is important to evaluate these data based on past and current trends. The data for this analysis could consist of aerial photographs, cross section soundings, location of bars and shoals on both the ocean and bay sides of an inlet, magnitude and direction of littoral drift, and longitudinal elevations through the waterway. It is also important to consider the possible impacts (either past or future) of the construction of jetties, breakwaters, or dredging of navigation channels.

Sources of data would be USACE, FEMA, USGS, U.S. Coast Guard (USCG), NOAA, local Universities, oceanographic institutions and publications in local libraries. For example, a publication by Bruun, "Tidal Inlets and Littoral Drift" contains information on many tidal inlets on the east coast for the United States.[82]

A site visit is recommended to gather such data as the conditions of the beaches (ocean and bay side); location and size of any shoals or bars; direction of ocean waves; magnitude of the currents in the bridge reach at mean water level (midway between high and low tides); and size of the sediments. Sounding the channel both longitudinally and in cross section using a conventional "fish finder" sonic fathometer is usually sufficiently accurate for this purpose.

Observation of the tidal inlet to identify whether the inlet restricts the flow of either the incoming or outgoing tide is also recommended. If the inlet or bridge restricts the flow, there will be a noticeable drop in head (change in water surface elevation) in the channel during either the ebb or flood tide. If the tidal inlet or bridge restricts the flow, an orifice equation may need to be used to determine the maximum discharge, velocities and depths (see the Level 2 analysis of this section).

Velocity measurements in the tidal inlet channel along several cross sections, several positions in the cross section and several locations in the vertical can also provide useful information for verifying computed velocities. Velocity measurements should be made at maximum discharge (Q_{max}). Maximum discharge usually occurs around the midpoint in the tidal cycle between high and low tide (Figure 9.2), although constricted inlets usually cause peak discharge to occur closer to high and low tides.

The velocity measurements can be made from a boat or from a bridge located near the site of a new or replacement bridge. If a bridge exists over the channel, a recording velocity meter could be installed to obtain measurements over several tidal cycles. Currently, there are instruments available that make velocity data collection easier. For example, broad-band acoustic Doppler current profiles and other emerging technologies will greatly improve the ability to obtain and use velocity data.

In order to develop adequate hydraulic data for the evaluation of scour, it is recommended that recording water level gages located at the inlet, at the proposed bridge site and in the bay or estuary upstream of the bridge be installed to record tide elevations at 15-minute intervals for several full tidal cycles. This measurement should be conducted during one of the spring tides where the amplitude of the tidal cycle will be largest. The gages should be referenced to the same datum and synchronized. The data from these recording gages are necessary for calibration of tidal hydraulic models such as ACES-INLET[83], or other unsteady 1 or 2-dimensional hydraulic flow models such as UNET, FESWMS-2D, and RMA-2V.[84,45,85,86] These data are also useful for calibration of WSPRO or HEC River Analysis System (RAS) when the bridge crosses tidally affected channels.[15,16,17] A more complete description of the unsteady flow models and data requirements for model application are given in Section 9.4.7.

The data and evaluations suggested above can be used to estimate whether present conditions are likely to continue into the foreseeable future and as a basis for evaluating the hydraulics and total scour for the Level 2 analysis. A stable inlet could change to one which is degrading if the channel is dredged or jetties are constructed on the ocean side to improve the entrance, since dredging or jetties could modify the supply of sediment to the inlet. In addition, plans or projects which might interrupt existing conditions of littoral drift should be evaluated.

It should be noted that in contrast to an inland river crossing, the discharge at a tidal inlet is not fixed. In inland rivers, the design discharge is fixed by the runoff and is virtually unaffected by the waterway opening. In contrast, the discharge at a tidal inlet can increase as the area of the tidal inlet increases, thus increasing long-term aggradation or degradation and local scour. Also, as Neill points out, constriction of the natural waterway opening may modify the tidal regime and associated tidal discharge.[78]

9.4 LEVEL 2 ANALYSIS

9.4.1 Introduction

Level 2 analysis involves the basic engineering assessment of scour problems at highway crossings. Scour equations developed for inland rivers are recommended for use estimating and evaluating scour for tidal flows. However, in contrast to the evaluation of scour at inland river crossings, the evaluation of the hydraulic conditions at the bridge crossing using either WSPRO or HEC-RAS is only suitable for tidally affected crossings where tidal fluctuations result in a variable tailwater control without flow reversal.[15, 16, 17] Other methods, described in this chapter, are recommended for tidally affected and tidally controlled crossings where the tidal fluctuation has a significant influence on the tidal hydraulics.

Several methods to obtain hydraulic characteristics of tidal flows at the bridge crossing are available. These range from simple procedures to more complex 2-dimensional and quasi 2-dimensional unsteady flow models. The use of the simpler hydraulic procedures is discussed and illustrated with example problems in Sections 9.8 and 9.9. An overview of the unsteady flow models which are suitable for modeling tidal hydraulics at bridge crossings is presented in Section 9.5. The use of the simpler hydraulic procedures given in this section can give large values if their underlying assumptions are violated. In these cases, 1- and 2-dimensional computer models can give more realistic values.

9.4.2 Evaluation of Hydraulic Characteristics

The velocity, depth and discharge at the bridge waterway are the most significant variables for evaluating bridge scour in tidal waterways. Direct measurements of the value of these variables for the design storm are seldom available. Therefore, it is usually necessary to develop the hydraulic and hydrographic characteristics of the tidal waterway, estuary or bay, and calculate the discharge, velocities, and depths in the crossing using coastal engineering equations. These values can then be used in the scour equations given in previous sections to calculate long-term aggradation or degradation, contraction scour, and local scour.

Unsteady flow computer models were evaluated under a pooled fund research project administered by the South Carolina Department of Transportation (SCDOT).[87] The purpose of this study was to identify the most promising unsteady tidal hydraulic models for use in scour analyses. The study identified UNET, FESWMS-2D, and RMA-2V as being the most applicable for scour analysis.[84,45,85,86] The research funded by the South Carolina pooled fund project is being continued to enhance and adapt the selected models so that they are better suited to the assessment of scour at bridges.

The models recommended by the pooled fund study differ in terms of their capabilities, degree of complexity, applicability and method of numerical modeling. UNET is supported by the USACE.[84] UNET is a 1-dimensional unsteady flow model and is applicable to channel networks. FESWMS-2D is an unsteady 2-dimensional finite element model developed by the USGS with support from the FHWA.[45] FESWMS-2D can be used for steady and unsteady flow analyses and incorporates structure hydraulics. RMA-2V is a 2-dimensional finite element hydrodynamic model that can be used for steady or unsteady flow analyses.[85, 86] FESWMS-2D and RMA-2V can also incorporate surface stress due to wind.

Although these unsteady flow models are suitable for determining the hydraulic conditions, their use requires careful application and calibration. The effort required to utilize these models may be more than is warranted for many tidal situations. As such, the use of these models may be more applicable under a Level 3 analysis. However, these models could be used in the context of a Level 2 analysis, if deemed necessary, to better define the hydraulic conditions at the bridge crossing.

Alternatively, either a procedure by Neill for unconstricted waterways, or an orifice equation for constricted tidal inlets can be used to evaluate the hydraulic conditions at bridges influenced by tidal flows.[78] A step-wise procedure for using these two methods to determine hydraulic conditions and scour is presented in the following sections. The selection of which procedure to use depends on whether or not the inlet is constricted. In general, narrow inlets to large bays as illustrated in Figure 9.1 can usually be classified as constricted; whereas, estuaries, which are also depicted on Figure 9.1 can be classified as unconstricted. However, these guidelines should not be construed as absolute.

The procedure developed by Neill can be used for unconstricted tidal inlets.[78] This method, which assumes that the water surface in the tidal prism is level, and the basin has vertical sides, can be used for locations where the boundaries of the tidal prism can be well defined and where heavily vegetated overbank areas or large mud flats represent only a small portion of the inundated area. Thick vegetation tends to attenuate tide levels due to friction loss, thereby violating the basic assumption of a level tidal prism. The discharges and velocities may be over estimated using this procedure if vegetation will attenuate tidal levels. In some complex cases, a simple tidal routing technique or 2-dimensional flow models may need to be used instead of this procedure (see Section 9.5).

Observation of an abrupt difference in water surface elevation during the normal ebb and flow (astronomical tide) at the inlet (during a Level 1 analysis) is a clear indication that the inlet is constricted. However, the observation of no abrupt change in water surface during astronomical tidal fluctuations does not necessarily indicate that the inlet will be unconstricted when extreme events such as a storm surge occur. In some cases, it may be necessary to compute the tidal hydraulics using both tidal prism and orifice procedures.

Then, judgment should be used to select the worst appropriate hydraulic parameters for the computation of scour.

Velocity measurements made at the bridge site (see Level 1) can be useful in determining whether or not the inlet is constricted as well as for calibration or verification of the tidal computation procedure. Using tidal data at the time that velocity measurements were collected, computed flow depths, velocities and discharge can be compared and verified to measured values. This procedure can form a basis for determining the most appropriate hydraulic computation procedure and for adjusting the parameters in these procedures to better model the tidal flows.

9.4.3 Design Storm and Storm Tide

Normally, long-term aggradation or degradation at a tidal inlet or estuary are influenced primarily by the periodic tidal fluctuations associated with astronomical tides. Therefore, flow hydraulics at the bridge should be determined considering the tidal range as depicted in Figure 9.2 for evaluation of long-term aggradation or degradation.

Extreme events associated with inland floods and storm tides should be used to determine the hydraulics at the bridge to evaluate local and contraction scour. Typically, events with a return period corresponding to the 100- and 500-year storm tide and inland flood need to be considered. Difficulty arises in determining whether the storm tide, inland flood or the combination of storm tide and inland flood should be considered controlling. The effect of the inland flood discharges (if any), would be most significant during the period when storm tide floodwaters recede (ebb), as those discharges would likely add to, and increase the storm tide associated discharges.

When inland flood discharges are small in relationship to the magnitude of the storm tide and are the result of the same storm event, then the flood discharge can be added to the discharge associated with the design tidal flow, or the volume of the runoff hydrograph can be added to the volume of the tidal prism. If the inland flood and the storm tide may result from different storm events, then, a joint probability approach may be warranted to determine the magnitude of the 100- and 500-year flows.

In some cases there may be a time lag between the storm tide discharge and the stream flow discharge at the bridge crossing. For this case, stream flow-routing methods such as the USACE HEC-1 model can be used to estimate the timing of the flood hydrograph derived from runoff of the watersheds draining into the bay or estuary.[88]

For cases where the magnitude of the inland flood is much larger than the magnitude of the storm tide, evaluation of the hydraulics reduces to using the equations and procedures recommended for inland rivers. The selection of the method to use to combine inland flood and storm tide flows is a matter of judgment and must consider the characteristics of the site and the storm events.

9.4.4 Scour Evaluation Concepts

The total scour at a bridge crossing can be evaluated using the scour equations recommended for inland rivers and the hydraulic characteristics determined using the procedures outlined in the previous sections. However, it should be emphasized that the scour equations and subsequent results need to be carefully evaluated considering other (Level 1) information from the existing site, other bridge crossings, or comparable tidal waterways or tidally affected streams in the area.

Evaluation of long-term aggradation or degradation at tidal highway crossings, as with inland river crossings, relies on a careful evaluation of the past, existing and possible future condition of the site. This evaluation is outlined under Level 1 and should consider the principles of sediment continuity. A longitudinal sonic sounder survey of a tide inlet is useful to determine if bed material sediments can be supplied to the tidal waterway from the bay, estuary or ocean. When available, historical sounding data should also be used in this evaluation. Factors which could limit the availability of sediment should also be considered.

Over the long-term in a stable tidal waterway, the quantity of sediment being supplied to the waterway by ocean currents, littoral transport and inland flows and being transported out of the tidal waterway are nearly the same. If the supply of sediment is reduced either from the ocean or from the bay or estuary, a stable waterway can be transformed into a degrading waterway. In some cases, the rate of long-term degradation has been observed to be large and deep. An estimate of the maximum depth that this long-term degradation can achieve can be made by employing the clear-water contraction scour equations to the inlet. For this computation the flow hydraulics should be developed based on the range of mean tide as described in Figure 9.2. It should be noted that the use of this equation would provide an estimate of the worst case long-term degradation which could be expected assuming no sediments were available to be transported to the tidal waterway from the ocean or inland bay or estuary. As the waterway degrades, the flow conditions and storage of sediments in shoals will change, ultimately developing a new equilibrium. The presence of scour resistant rock would also limit the maximum long-term degradation.

Potential contraction scour for tidal waterways also needs to be carefully evaluated using hydraulic characteristics associated with the 100- and 500-year storm surge or inland flood as described in the previous section. For highway crossings of estuaries or inlets to bays, where either the channel narrows naturally or where the channel is narrowed by the encroachment of the highway embankments, the live-bed or clear water contraction scour equations can be utilized to estimate contraction scour.

Soil boring or sediment data are needed in the waterway upstream, downstream, and at the bridge crossing in order to determine if the scour is clear-water or live-bed and to support scour calculations if clear-water contraction scour equations are used. Equation 5.1 and the ratio of V_*/ω can be used to assess whether scour would be clear-water or live-bed.

A mitigating factor which could limit contraction scour concerns sediment delivery to the inlet or estuary from the ocean due to the storm surge and inland flood. A surge may transport large quantities of sediment into the inlet or estuary during the flood tide. Likewise, inland floods can also transport sediment to an estuary during extreme floods. Thus, contraction scour during extreme events may be classified as live-bed because of the sediment being delivered to the inlet or estuary from the combined effects of the storm surge and inland flood. The magnitude of contraction scour must be carefully evaluated using engineering judgment which considers the geometry of the crossing, estuary or bay, the magnitude and duration of the discharge associated with the storm surge or inland flood, the basic assumptions for which the contraction scour equations were developed, and mitigating factors which would tend to limit contraction scour.

Evaluation of local scour at piers can be made by using Equation 6.1 as recommended for inland river crossings. This equation can be applied to piers in tidal flows in the same manner as given for inland bridge crossings. However, the flow velocity and depth will need to be determined considering the design flow event and hydraulic characteristics for tidal flows.

9.4.5 Scour Evaluation Procedure for an Unconstricted Waterway

This method applies only when the tidal waterway or the bridge opening does not significantly constrict the flow and uses the tidal prism method as discussed by Neill.[77]

STEP 1. Determine the net waterway area at the crossing as a function of elevation. Net area is the gross waterway area between abutments minus area of the piers. It is often useful to develop a plot of the area versus elevation.

STEP 2. Determine tidal prism volume as a function of elevation. The volume of the tidal prism at successive elevations is obtained by planimetering successive sounding and contour lines and calculating volume by the average end area method. The tidal prism is the volume of water between low and high tide levels.

STEP 3. Determine the elevation versus time relation for the 100- and 500-year storm tides. The ebb and flood tide elevations can be approximated by either a sine or cosine curve. A sine curve starts at mean water level and a cosine curve starts at the maximum tide level. The equation for storm ebb tide that starts at the maximum elevation is:

$$y = A \text{ Cos } \theta + Z \tag{9.1}$$

where:

Y = Amplitude or elevation of the tide above mean water level, m (ft) at time t
A = Maximum amplitude of elevation of the tide or storm surge, m (ft). Defined as half the tidal range or half the height of the storm surge
θ = Angle subdividing the tidal cycle, one tidal cycle is equal to 360°

$$\theta = 360\left(\frac{t}{T}\right)$$

t = Time from beginning of total cycle, minutes
T = Total time for one complete tidal cycle, minutes
Z = Vertical offset to datum, m (ft)

The tidal range (difference in elevation between high and low tide levels) is equal to twice the amplitude. One-half the tidal period is equal to the time between high and low tide. These relations are shown in Figure 9.2. A figure similar to Figure 9.2 can be developed to illustrate quantitatively the tidal fluctuations and resultant discharges.

To determine the elevation versus time relation for the 100- and 500-year storm tides, two values must be known:

- storm tidal range
- storm tidal period

As stated earlier, FEMA, USACE, NOAA, and other federal or state agencies compile records which can be used to estimate the 100- and 500-year storm tide elevation, mean sea level elevation, and low tide elevation. These agencies also are the source of data to determine the 100- and 500-year storm tide period.

Storm tides, may have different periods than the astronomical semi-diurnal and diurnal tides which have periods of approximately 12 and 24 hours, respectively. This is because storm tides are influenced by factors other than the gravitational forces of the sun, moon and other celestial bodies. Factors such as the wind, path of the hurricane or storm creating the storm tide, fresh water inflow, shape of the bay or estuary, etc. influence both the storm tide amplitude and period.

STEP 4. Determine the discharge, velocities and depth. Neill has stated the maximum discharge in an ideal tidal estuary may be approximated by the following equation:[(78)]

$$Q_{max} = \frac{3.14\ VOL}{T} \tag{9.2}$$

where:

Q_{max}	=	Maximum discharge in the tidal cycle, m^3/s (ft^3/s)
VOL	=	Volume of water in the tidal prism between high and low tide levels, m^3 (ft^3)
T	=	Tidal period between two successive high tides or two successive low tides, s

A simplification of Equation 9.2, suggested by Chang, is to assume the tidal prism has vertical sides.[(51)] With this assumption, which eliminates the need to compute the volume in the tidal prism by adding the volume of successive elevations, Equation 9.2 becomes:

$$Q_{max} = \frac{3.14\ A_s\ H}{T} \tag{9.2a}$$

where:

A_s	=	Surface area of the tidal prism at mean tide elevation, m^2 (ft^2)
H	=	Elevation difference (tidal range) between high and low tide levels, m (ft)

In the idealized case, Q_{max} occurs in the estuary or bay at mean water elevation and at a time midway between high and low tides when the slope of the tidal energy gradient is steepest (Figure 9.2).

The corresponding maximum average velocity in the waterway is:

$$V_{max} = \frac{Q_{max}}{A_c} \tag{9.3}$$

where:

V_{max} = Maximum average velocity in the cross section (where the bridge will be located) at Q_{max}, m/s (ft/s)

H = Cross-sectional area of the waterway at mean tide elevation, halfway between high and low tide, m^2 (ft^2)

It should be noted that the velocity as determined in the above equations represents the average velocity in the cross section. This velocity will need to be adjusted to estimate velocities at individual piers to account for nonuniformity of velocity in the cross section. As for inland rivers, local velocities can range from 0.9 to approximately 1.7 times the average velocity depending on whether the location in the cross section was near the banks or near the thalweg of the flow.

Neill's studies indicate that the maximum velocity in estuaries is approximately 30 percent greater than the average velocity computed using Equation 9.3. If a detailed analysis of the horizontal velocity distribution is needed, the design discharge could be prorated based on the conveyance in subareas across the channel cross section.

Another useful equation from Neill is:[78]

$$Q_t = Q_{max} \ \text{Sin}\left(360 \ \frac{t}{T}\right) \tag{9.4}$$

where:

Q_t = Discharge at any time t in the tidal cycle, m^3/s (ft^3/s)

The velocities calculated with this procedure can be plotted and compared with any measured velocities that are available for the bridge site or adjacent tidal waterways to evaluate the reasonableness of the results.

STEP 5. Evaluate the effect of flows derived from inland riverine flow on the values of discharge, depth and velocities obtained in step 4. This evaluation may range from simply neglecting the inland flow into a bay (which may be so small that it is insignificant in comparison to the tidal flows), to routing the inland flow into the bay or estuary. If an estuary is a continuation of the stream channel and the storage of water in it is small, the inland flow can simply be added to the Q_{max} obtained from the tidal analysis and the velocities then calculated from Equation 9.3. However, if the inland flow is large and the bay or estuary sufficiently small that the inland flow will increase the tidal prism, the inland flood hydrograph should be routed through the bay or estuary and added to the tidal prism. The USACE HEC-1 could be used to route the flows.[87] In some instances, trial calculations will be needed to determine if and how the inland flow will be included in the discharge through the bridge opening.

STEP 6. Evaluate the discharge, velocities and depths that were determined in steps 4 and 5 above (or the following section for constricted waterways). Use engineering judgment to evaluate the reasonableness of these hydraulic characteristics. Compare these values with values for other bridges over tidal waterways in the area with similar conditions. Compare the calculated values with any measured values for the site or similar sites. Even if the measured discharge values for astronomical tides are much lower than the design storm tide discharge, they will give an appreciation of the magnitude of discharge to be expected.

STEP 7. Evaluate the scour for the bridge using the values of the discharge, velocity and depths determined from the above analysis using the scour equations recommended for inland bridge crossings presented previously. Care should be used in the application of these scour equations, using the guidance given previously for application of the scour equations to tidal situations.

9.4.6 Scour Evaluation Procedure for a Constricted Waterway

The procedures given above <u>except</u> for Steps 2 and 4 (the determination of the tidal prism, discharge, velocity and depth for unconstricted waterways) are followed. To determine these hydraulic variables when the constriction is caused by the channel and not the bridge, the following equation for tidal inlets taken from van de Kreeke[89] or Bruun[90] can be used.

$$V_{max} = C_d \sqrt{2g\Delta H} \qquad (9.5)$$

$$Q_{max} = A_c \; V_{max} \qquad (9.6)$$

where:

V_{max}	=	Maximum velocity in the inlet, m/s (ft/s)
Q_{max}	=	Maximum discharge in the inlet, m^3/s (ft^3/s)
C_d	=	Coefficient of discharge ($C_d < 1.0$)
g	=	Acceleration due to gravity, 9.81 m/s^2 (32.2 ft/s^2)
ΔH	=	Maximum difference in water surface elevation between the bay and ocean side of the inlet or channel, m (ft)
A_c	=	Net cross-sectional area in the inlet at the crossing, at mean water surface elevation, m^2 (ft^2)

The difference in water surface elevation, ΔH, should be for the normal astronomical tide, the 100-year storm tide and the 500-year storm tide. The difference in height for the normal astronomical tide is used to determine potential long-term degradation at the crossing if the crossing has a deficient or interrupted sediment supply (e.g., by construction of a jetty which cuts off littoral drift). This condition can lead to the inlet becoming unstable and degrading (i.e., enlarging) indefinitely.

The coefficient of discharge (C_d) for most practical applications can be assumed to be equal to approximately 0.8. Alternatively, the coefficient of discharge can be computed using the equations given by van de Kreeke[89] or Bruun:[90]

$$C_d = (1/R)^{1/2} \qquad (9.7)$$

where

$$R = K_o + K_b + \frac{2g\, n^2\, L_c}{k_u^2\, h_c^{4/3}} \qquad (9.8)$$

and

R	=	Coefficient of resistance
K_o	=	Velocity head loss coefficient on the ocean side or downstream side of the waterway taken as 1.0 if the velocity goes to 0
K_b	=	Velocity head loss coefficient on the bay or upstream side of the waterway. Taken as 1.0 if the velocity goes to 0
n	=	Manning's roughness coefficient
L_c	=	Length of the waterway (inlet), m (ft)
h_c	=	Average depth of flow in the waterway at mean water elevation, m (ft)
K_u	=	1.0 SI
K_u	=	1.486 English

The values of K_o and K_b depend on local hydrodynamic conditions, but are generally greater than 0.5. For a flood tide exiting an inlet to a large bay the coefficient K_b can be taken as 1.0.

If ΔH is not known or cannot be determined easily, a hydrologic routing method developed by Chang et al., which combines the above orifice equations (Equation 9.5 - 9.8) with the continuity equation, can be used.[91] The total flow approaching the bridge crossing at any time (t) is the sum of the riverine flow (Q) and tidal flow. The tidal flow is calculated by multiplying the surface area of the upstream tidal basin (A_s) by the drop in elevation (H_s) over the specified time ($Q_{tide} = A_s\, dH_s/dt$). This total flow approaching the bridge is set equal to the flow calculated from the orifice equation.

$$Q + A_s \frac{dH_s}{dt} = C_d\ A_c\ \sqrt{2g\ \Delta H} \qquad (9.9)$$

where:

A_c	=	Bridge waterway cross-sectional area, m^2 (ft^2)
H_s	=	Water surface elevation in the tidal basin upstream of the bridge, m (ft)
Q	=	Riverine discharge m^3/s (ft^3/s)

All other variables are as previously defined.

Equation 9.9 may discretized with respect to time as denoted in Equation 9.10 for the time interval, $\Delta t = t_2 - t_1$. Subscripts 2 and 1 represent the end and beginning of the time interval, respectively.

$$\frac{Q_1+Q_2}{2}+\frac{A_{S1}+A_{S2}}{2}\frac{H_{S1}-H_{S2}}{\Delta T}=C_d\left(\frac{A_{C1}+A_{C2}}{2}\right)\sqrt{2g\left(\frac{H_{s1}+H_{S2}}{2}-\frac{H_{t1}}{2}\right)} \qquad (9.10)$$

For a given initial condition, t_1, all terms with subscript 1 are known. For $t=t_2$, the downstream tidal elevation (H_{t2}), riverine discharge (Q_2), and waterway cross-sectional area (A_{c2}) are also known or can be calculated from the tidal elevation. Only the water-surface elevation (H_{s2}) and the surface area (A_{s2}) of the upstream tidal basin remain to be determined. Because surface area of the tidal basin is a function of the water-surface elevation, the elevation of the tidal basin at time t_2 (H_{s2}) is the only unknown term in Equation 9.10, and this term can be determined by trial-and-error to balance the values on the right and left sides.

Chang et al. suggest the following steps for computing the flow:[91]

Step 1. Determine the period and amplitude of the design tide(s) to establish the time rate of change of the water-surface on the downstream side of the bridge.

Step 2. Determine the surface area of the tidal basin upstream of the bridge as function of elevation by planimetering successive contour intervals and plotting the surface area vs. elevation.

Step 3. Plot bridge waterway area vs. elevation.

Step 4. Determine the quantity of riverine flow that is expected to occur during passage of the storm tide through the bridge.

Step 5. Route the flows through the contracted waterway using Equation 9.10, and determine the maximum velocity of flow.

In most cases, development of a UNET or other 1-dimensional unsteady flow model will be as easy as performing the routing described above.

Using the tidal hydraulics determined as described above for constricted inlets, the scour computations can proceeded according to steps 5, 6, and 7 presented previously for the unconstricted waterway.

9.5 TIDAL CALCULATIONS USING UNSTEADY FLOW MODELS

9.5.1 Tidal Hydraulic Models

Alternatively, the tidal hydraulics at the bridge can be determined using one of several unsteady flow models in lieu of either Neill's procedure, the orifice equation or Chang's procedure. A brief overview of these models is presented below. This information was derived from a pooled fund study (HPR552) administered by the SCDOT.[13,87] All quotes presented in this section are from the final report documenting the first phase of this study.

ACES is an acronym for the Automated Coastal Engineering System and was developed by the USACE in an effort to incorporate many of the various computational procedures typically needed for coastal engineering analysis into an integrated, menu-driven user environment.[83] There are seven separate computation modules for wave prediction, wave theory, littoral processes and other useful modules. One such module denoted as ACES-INLET is a spatially integrated numerical model for inlet hydraulics. This module can be used to determine discharges, depths and velocities in tidal inlets with up to two inlets connecting a bay to the ocean. This module can be used in place of, or in addition to, the procedures given in steps 3 and 4, above, for tidal inlets. **ACES-INLET is applicable only where the project site is at or very near the inlet throat (i.e., for bridges crossing inlets) (Figure 9.1).**

The pooled fund study states:[13]

> "*ACES-Inlet is simple and easy to use. A minimum of data are required and the menu-driven environment makes user input straightforward. The primary limitation of the model is its reliance on numerous empirical coefficients. In addition to requiring keen judgment on the part of the user, the empirical relations greatly oversimplify the inlet dynamics. Model results can be regarded as rough approximations, useful for reconnaissance-level investigations.*"

Other modules incorporated into ACES may be useful in evaluating tidal highway crossings. These modules can be used to estimate wave and tidal parameters, littoral drift, wave run-up and other aspects of tidal flow which could influence the design or evaluation of bridge crossings over tidal inlets connecting bays to the ocean.

UNET is a 1-dimensional unsteady flow model.(84) Although simpler to use than more complex 2-dimensional models, UNET can model networks of open channels, and bifurcations and flow around islands. According to the pooled fund study:

> *"UNET is extremely flexible in modeling of channel networks, storage areas, bifurcations, and junctions. Both external boundaries (hydrographs, stage hydrographs) and internal boundary conditions (gated and uncontrolled spillways, bridges, culverts, and levee systems) can be included. UNET uses a modified HEC-2 file format to facilitate data entry and UNET can use the HEC-DSS database for input and output."*

According to the pooled fund study, the advantages and limitations of UNET are:

> *"UNET uses an efficient implicit numerical formulation solution techniques. Of the reviewed unsteady 1-dimensional flow models, UNET is the only model which intrinsically evaluated bridges, culverts, and embankment overtopping.... Although UNET does not simulate flow separation (2-D), off-channel storage (ineffective flow areas) can be used to represent these areas. The primary limitation of this model is the exclusion of wind effects."*

FESWMS-2DH is a 2-dimensional unsteady flow model developed by the USGS and FHWA.(45) This model uses a finite element numerical simulation and has options for simulation of steady or unsteady flow over highway embankments and through culverts. The model has been incorporated into the SMS(91) user interface. The critique of FESWMS-2DH in the pooled fund study states:

> *"The options for weir flow and culvert flow are particularly well suited to highway application. The variable friction formulation permits realistic modeling of floodplains. FESWMS-2DH has limitations similar to those of other 2- models, e.g. inability to simulate stratified flows or complex near-field phenomena where vertical velocities are not negligible. The relative complexity of the model (as compared to 1-D models) requires some expertise for model setup and use."*

RMA-2V is a widely used 2-dimensional unsteady flow model which uses a finite element numerical procedure.(85,86) The model is incorporated into the SMS user interface which provides additional applications including SED2D which, when linked with RMA-2V, modifies the geometry of the waterway using computations of sediment erosion, sedimentation and transport during each time step of the hydrodynamic model. The critique of RMA-2V in the pooled fund study states:

> *"RMA-2V and the TABS/FastTABS system (now in SMS) offer a rigorous 2-D solution to the shallow water equations coupled with sediment transport capabilities and advanced pre/post processors. The finite element spatial discretization is accurate and can easily represent complex physical systems. Other capabilities include simulation of wetting and drying elements and flow control structures..."*

Of the four unsteady models, ACES and UNET are significantly simpler than either FESWMS or RMA-2V. Because of this, ACES and UNET can be considered to be more adaptable to Level 2 type analysis due to their relative simplicity. Although FESWMS and RMA-2V can be used as part of an advanced Level 2 analysis, their use is more consistent with a Level 3 analysis. As indicated earlier, efforts to enhance and improve these models so that they better support highway applications are ongoing. Future enhancements and versions of these models will likely provide for simpler application and better estimates of the hydraulic conditions which influence scour.

Another advancement in scour analysis of bridges over tidal waterways is the production of a manual on tidal hydraulic modeling for bridges.[87] This manual was developed as part of the second phase of a pooled fund study.[13] The manual includes methods for developing realistic tidal and storm surge boundary conditions, discussions on the applicability of various hydraulic modeling approaches (tidal prism, orifice, routing, hydrodynamic modeling), and examples on the use of 1- and 2-dimensional modeling. Guidance is also being developed on when to include inland runoff with storm surge simulations, effects of wind, time dependency of scour, and wave height determination. Figure 9.4 shows an example of a synthetic storm surge hydrograph added to a daily tide. This is a realistic representation of the surge that could be used as an ocean boundary condition for hydrodynamic modeling. Hydrodynamic modeling has been used on numerous projects to evaluate the scour potential of new and existing bridges.

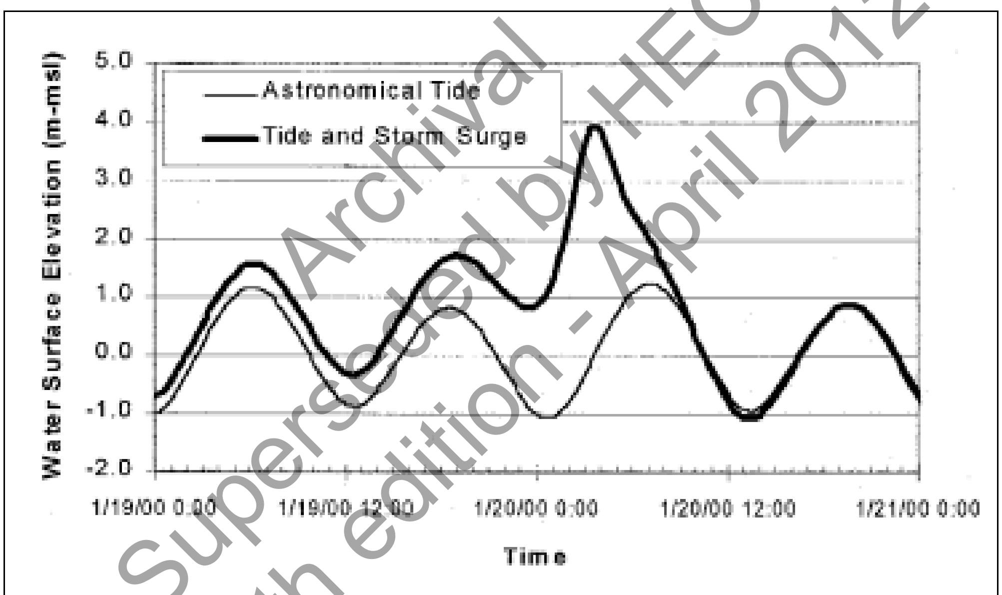


Figure 9.4. Synthetic storm surge hydrograph combined with daily tide.[87]

9.5.2 Data Requirements for Hydraulic Model Verification

Whenever a hydraulic model is employed, it is necessary to calibrate the model to insure that the results will adequately represent the flow conditions which are likely to occur during an extreme event. Because of this, any model, including WSPRO and HEC-RAS should be verified against actual data.[15,16,17] For inland rivers systems model verification is reasonably straightforward. Known discharges and water surface elevations are used to adjust the downstream boundary conditions and resistance parameters until a close agreement between measured data and model output is obtained. Although similar, model verification using unsteady flow models is more difficult due to the unsteady nature of the flow. The following paragraphs discuss data needs for model verification of unsteady flow models.

Ideally, synoptic measurements of the following data are required to validate hydraulic modeling using any of the above mentioned unsteady flow models:

- Tidal elevations in the ocean and back-bay locations
- Velocity measurements are needed in the inlet throat as well as at proposed project sites
- Boundary condition data for any back-bay, open-water boundaries; these data may be elevation, velocity, discharge, or any combination of these parameters
- Wind speed and direction if wind energy influences in the tidal system

The above data may be available from previous studies of the tidal system (for example, USACE or NOAA studies) or may be collected for a specific project.

9.6 TIME DEPENDENT CHARACTERISTICS OF TIDAL SCOUR

In tidal areas, hurricane storm surges often produce extreme hydraulic conditions. Computing **ultimate contraction scour** amounts for these conditions may not be reasonable based on the short duration (often less than 3 hours) of the flow produced by the surge. Based on equations in a Scour Manual published in the Netherlands,[93] (see also Transportation Research Board Research Results Digest[94]), the time development of scour holes can be estimated. To provide confirmation of these results, the Yang[95] sediment transport equation was used to compute contraction scour hole development based on the erosion of the scour hole equal to the transport capacity in the contracted bridge opening. The scour rates for this situation are shown on Figures 9.5 and 9.6. Figure 9.5 shows the complete development of scour with time plotted on a logarithmic axis and Figure 9.6 shows the first 100 hours of development with time plotted on an arithmetic axis. The scour rates predicted by the two methods are extremely similar and indicate that the scour that could be generated in a few hours during a storm surge is significantly less than the ultimate contraction scour condition.

Also shown in Figures 9.5 and 9.6 is the development of a **pier scour** hole for the same hydraulic conditions. The pier scour hole reaches 90 percent of ultimate scour in the first 20 hours while the clear-water **contraction scour** reaches only about 30 percent of ultimate scour.

The Dutch equations are based on clear-water scour and the conditions used to test the Yang equation were close to clear-water. The Dutch Scour Manual[93] indicates that under live-bed conditions scour reaches ultimate conditions more rapidly and that the ultimate scour is less than the equivalent clear-water case which is consistent with current U.S. guidance. Figure 9.7 shows the development of contraction scour (using the Yang equation) under varying amounts of upstream sediment supply relative to the transport capacity in the bridge opening. This approach involves a basic sediment continuity analysis as outlined in HEC-20.[6] For the case shown, if the upstream channel is supplying 50 percent of the contracted section transport capacity, the scour hole reaches the ultimate depth in approximately one hour. Based on this review, it appears that under storm surge conditions contraction scour should be analyzed on a case-by-case basis to assess the level of contraction scour that could occur over a short time. It also suggests that local scour occurs more rapidly and time dependence is a less significant factor.

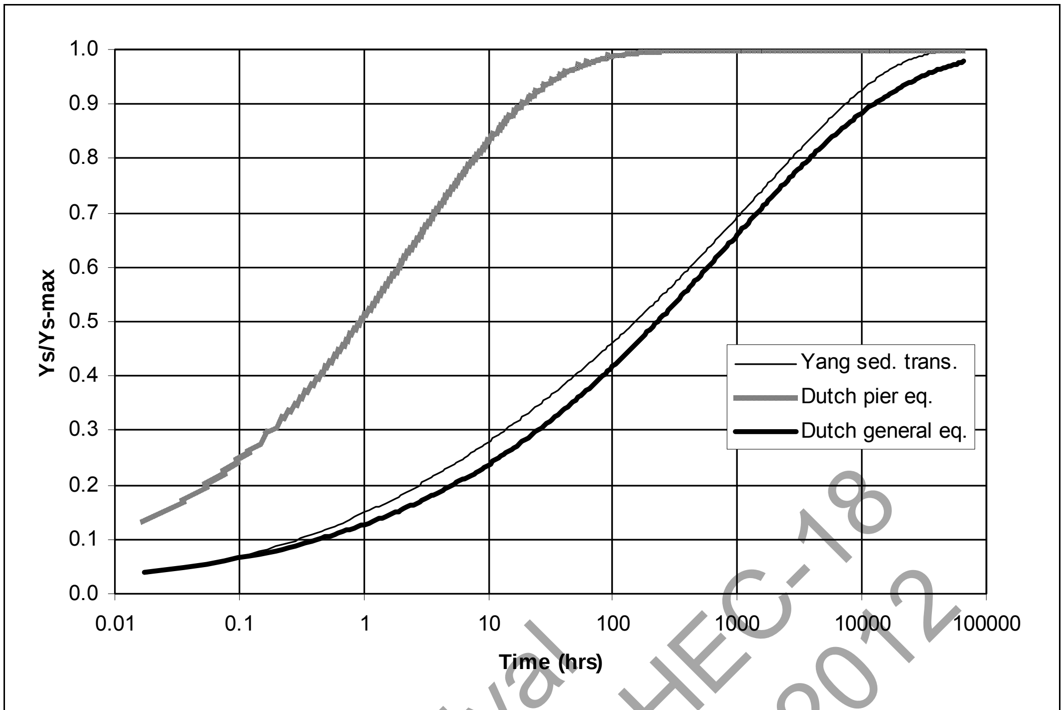


Figure 9.5. Time development of scour.

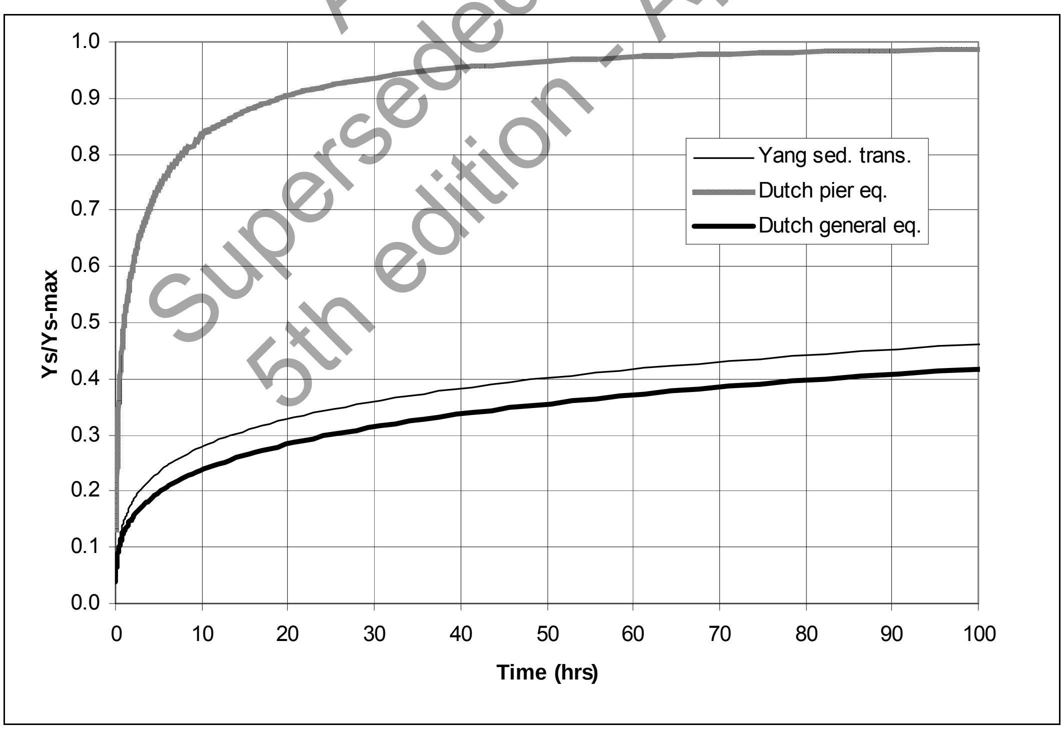


Figure 9.6. Initial scour development.

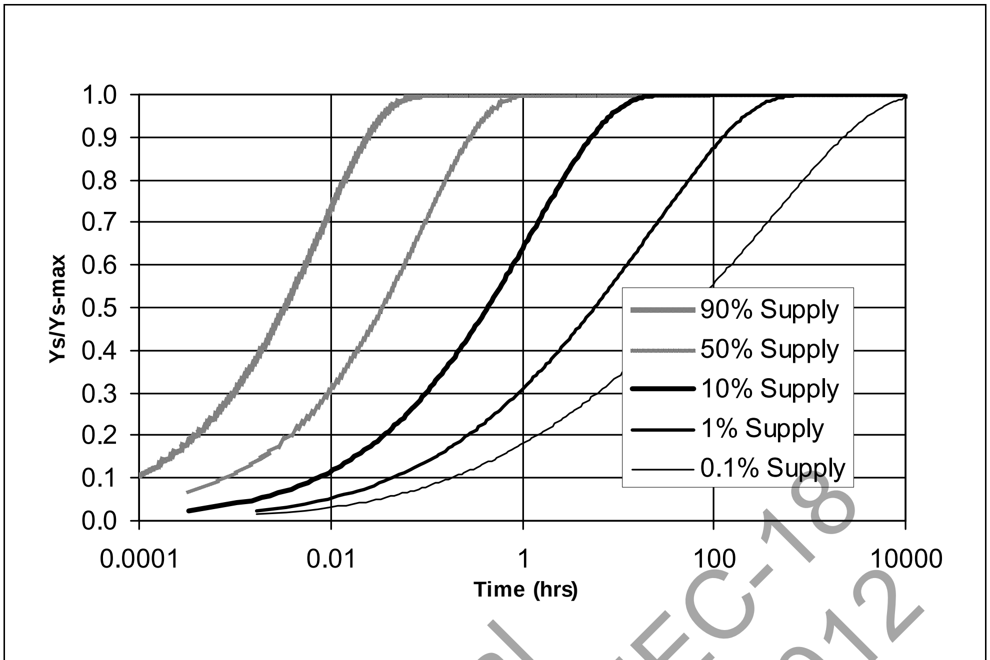


Figure 9.7. Contraction scour development with sediment supply.

9.7 LEVEL 3 ANALYSIS

As discussed in HEC-20, Level 3 analysis involves the use of physical models or more sophisticated computer models for complex situations where Level 2 analysis techniques have proven inadequate.[6] In general, crossings that require Level 3 analysis will also require the use of qualified hydraulic engineers. Level 3 analysis by its very nature is specialized and beyond the scope of this manual.

9.8 TIDAL SCOUR EXAMPLE PROBLEMS (SI)

9.8.1 Example Problem 1 - Tidal Prism Approach (Unconstricted Waterway) (SI)

In this example problem, the discharge, velocity, depths, and scour are to be determined for an existing bridge across a tidal estuary as part of an ongoing scour evaluation. The bridge is 818.39 m long, has vertical wall abutments and 16 bents each consisting of two 3.66 m diameter circular piers supported on piles. Neither the bridge nor the tidal waterway constricts the flow.

For this evaluation, the bridge maintenance engineer has expressed concern about observed scour at one of the piers. This pier is located where the velocities at the pier are approximately 30 percent greater than the average velocities. The water depth at the pier referenced to mean sea level, is 3.75 m. The actual depth of flow at the pier will need to be increased to account for additional water depth caused by the storm surge for the computation of pier scour.

Level 1 Analysis

a. Level 1 analysis has determined that the storm surge for the 100- and 500-year return period produces discharge, velocity and depths that are much larger than those from inland runoff. There is minimal littoral drift and historical tides are low. From FEMA, the storm surge tidal range for the 100-year return period is 2.19 m and for the 500-year return period is 2.87 m. Measured maximum velocity in the waterway at mean sea level for a tide of 0.67 m was only 0.21 m/s.

   Sonic soundings in the waterway indicate that there is storage of sediment in the estuary directly inland from the bridge crossing. This was determined by observing that the elevation of the bed of the waterway at the bridge site was lower than the elevation of the bottom of the estuary further inland. Although no littoral drift is evident, there is storage of sediment at the mouth of the estuary between the ocean and the bridge crossing.

b. Stability of the estuary and crossing was evaluated by examination of the periodic bridge inspection reports which included underwater inspections by divers, evaluation of historical aerial photography, and depth soundings in the estuary using sonic fathometers. From this evaluation it was determined that the planform of the estuary has not changed significantly in the past 30 years. These observations indicate that the estuary and bridge crossing has been laterally stable.

   Evaluation of sounding data at the bridge indicates that there has been approximately 1.52 m of degradation at the bridge over the past 30 years; however, the rate of degradation in the past five years has been negligible. Underwater inspections indicted that local scour around the piers is evident.

c. A search of FEMA, USACE, and other public agencies for inland flood and storm surge data was conducted. These data will be discussed under the Level 2 analysis.

d. Grain size analysis of the bed material indicates that the bed of the estuary is composed of fine sand with a D_{50} of approximately 0.27 mm (0.00027 m).

e. Velocities measured at Q_{max} during a large astronomical tide indicated that the maximum velocity in the bridge section was approximately 30 percent greater than the average velocity.

Level 2 Analysis

STEP 1. A plot of net waterway area as a function of elevation is given in Figure 9.8. Net waterway area is the average area at the bridge crossing less the area of the piers.

STEP 2. A plot of volume of the tidal prism as a function of elevation is also presented in Figure 9.8. The plot was developed by planimetering the area of successive sounding and contour lines and multiplying the average area by the vertical distance between them.

STEP 3. A synthesized storm surge for the 100- and 500-year return period was developed and is presented in Figure 9.8. It was obtained as follows:

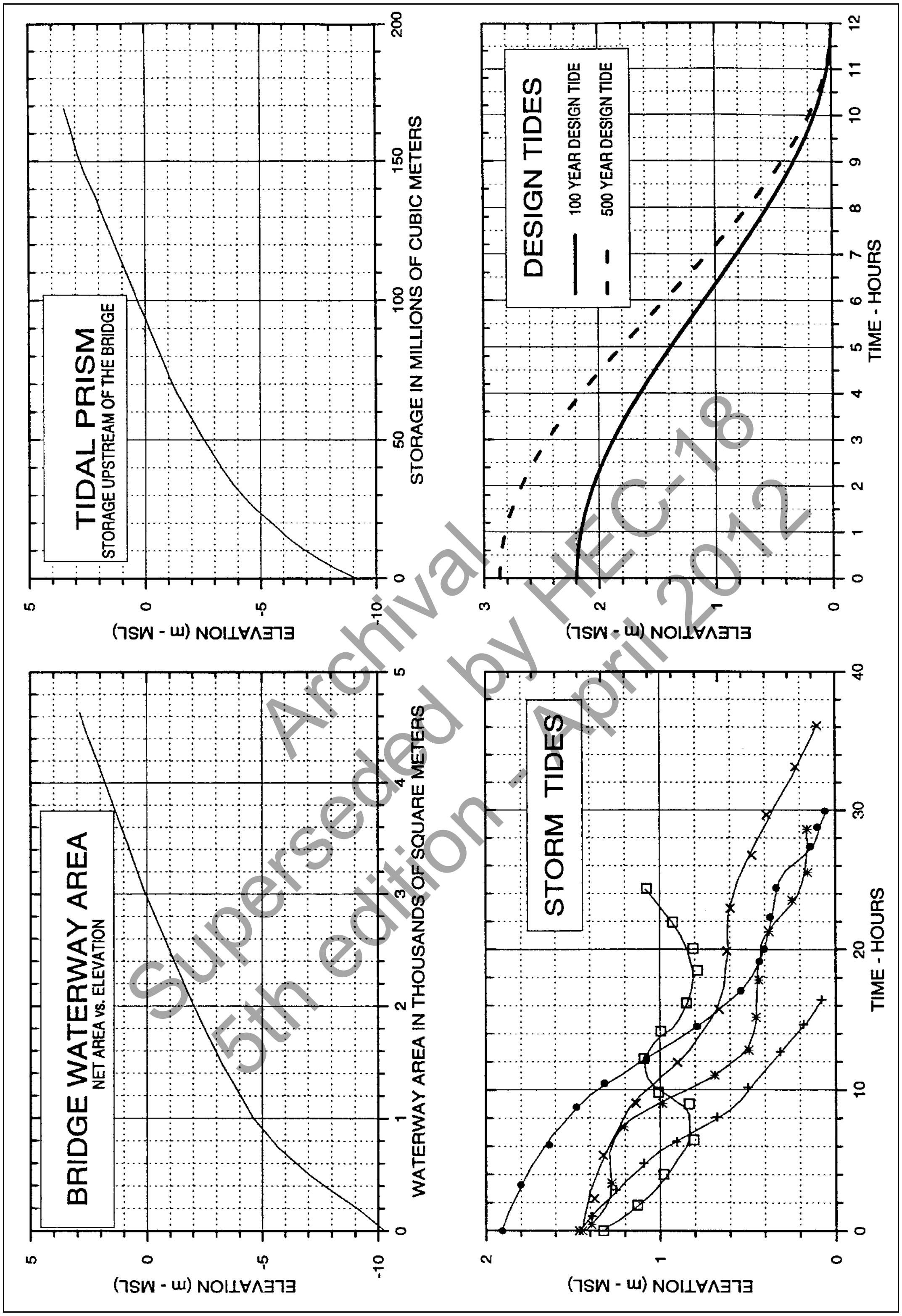


Figure 9.8. Tidal parameters for Example Problem 1 (SI).

An idealized graph for one half the tidal period, beginning at high tide was developed using the cosine equation (Equation 9.1). This plot can be used to develop an idealized tidal cycle for any waterway. Tidal range and period are needed to use the idealized tide cycle to develop a synthesized tidal cycle for this waterway.

The tidal ranges were obtained from a FEMA coastal flood insurance study during the Level 1 analysis (Table 9.1).

Table 9.1. Tidal Ranges Derived from FEMA Flood Study.

Return Period (yr)	High Tide (m)	Low Tide (m)
100	2.19	0
500	2.87	0

The tidal period is more difficult to determine because it is affected by more than the gravitational attraction of the moon and sun. At this waterway location, the direction of the storm and the characteristics of the estuary affected the tidal period. To determine the tidal period, major storm tides were plotted in Figure 9.8. Review of these historical storm tides reveals that (as expected) most events occur over a duration longer than an astronomical tidal period. Only a single event exhibits a seemingly semi-diurnal response. Given these characteristics and behavior, analyses yield a conservative estimate that approximately 12 hours pass between the highest and lowest elevations. This assumption would therefore indicate that the associated storm tide period (T) is 24 hours.

STEP 4. Using the data developed in Steps 1 to 3 and the equations given previously the maximum tidal discharge (Q_{max}) and maximum average tidal velocity (V_{max}) are calculated. The values used in the calculations are given in Table 9.2.

STEP 5. The 100- and 500-year return period peak inland flow into the estuary was obtained from a USGS flood frequency study. These values are also given in Table 9.2.

Table 9.2. Design Discharge and Velocities.

	100-Year Storm Tide	500-Year Storm Tide
Maximum storm tide elevation, m	2.19	2.87
Mean storm tide elevation, m	1.10	1.44
Low storm tide elevation, m	0.0	0.0
Tidal prism volume (millions of cubic meters) Figure 9.8	46.40	60.80
Net waterway area at mean storm tide elevation (A_c), m^2	3620	3809
Tidal period, h	24.0	24.0
Q_{max} (Tidal), m^3/s (Equation 9.2)	1686.3	2209.6
V_{max} (Tidal), m/s (Equation 9.3)	0.47	0.58
Inland peak runoff (discharge), m^3/s	141.03	224.29
Q_{max} (Tidal plus runoff), m^3/s	1827.33	2433.83
V_{max} (Tidal plus runoff), m/s ($V_{max} = Q_{max}/A_c$)	0.50	0.64
Average flow depth - A_c/width, m	4.42	4.65

Average flow depths can be determined by dividing the flow area as listed in Table 9.2 by the channel width (818.4 m). Therefore, the average flow depths for the 100- and 500-year event are 4.42 and 4.65 m, respectively.

The peak discharge from the 100- and 500-year inland flow hydrograph is very small in comparison to the storage volume in the estuary. In this case, adding the inland peak discharge to the maximum tidal discharge will be a conservative estimate of the maximum discharge and maximum average velocity in the waterway. If the inland inflow into the estuary had been large, the flood could be routed through the estuary using standard hydrologic modeling techniques.

STEP 6. A comparison of the calculated velocities with the measured velocities indicate that they are reasonable. The discharge and velocities given in Table 9.2 are acceptable for determining the scour depths. However, the average velocity will have to be adjusted for the nonuniformity of flow velocity in the vicinity of the bridge to obtain the velocities for determining local scour at the piers.

STEP 7. Calculate the components of total scour using the information collected in the Level 1 and Level 2 analyses.

Long-Term Aggradation/Degradation

The Level 1 analysis indicates that the channel is relatively stable at this time. However, there is an indication that over the past 30 years the channel has degraded approximately 1.52 m. Since the degradation rate has been negligible in the last five years, no additional degradation will be anticipated.

Contraction Scour

Contraction scour depends on whether the flow will be clear-water or live-bed. Equation 5.1 is used to determine the critical velocity for the 100-year hydraulics.

$$V_c = 6.19\,(4.42)^{1/6}\,(0.00027)^{1/3} = 0.5\,m/s$$

This indicates that the 100-year storm surge combined with the inland flow may result in velocities greater than or equal to the critical velocity; therefore, contraction scour will most likely be live-bed. This conclusion is made considering that velocities in excess of the average velocity will be expected due to the nonuniformity of the velocity in the bridge opening, as determined during the Level 1 analysis.

Applying the live-bed contraction scour equation, it is noted that the ratio of discharges is equal to unity (i.e., there is no overbank flow). Therefore, the contraction scour will be influenced by the contraction resulting from the bridge piers reducing the flow width at the bridge crossing. Using Equation 5.2, and assuming that the mode of sediment transport is mostly suspended load (k_1=0.69), the estimate of live-bed contraction scour for the 100-year event is:

$$\frac{y_2}{4.42} = \left[\frac{818.39}{759.84}\right]^{0.69} = 1.05$$

$y_2 = 4.64\,m$

$y_s = 4.64 - 4.42 = 0.22\,m$

Therefore, the contraction scour for the 100-year event is approximately 0.22 m. Recomputation for the 500-year event with an average flow depth of 4.65 m results in an estimate of contraction scour of approximately 0.24 m.

Local Scour at Piers

The hydraulic analysis estimates average velocities in the bridge cross section only. Because of this, an estimate of the maximum velocity at the bridge pier is made to account for non-uniform velocity in the bridge cross section. The average velocity will be increased by 30 percent since velocities for normal flows (Level 1) indicated that the maximum velocity was observed to be approximately 30 percent greater than the average. Therefore the maximum velocity for the 100- and 500-year event are 0.65 and 0.83 m/s, respectively.

K_1, K_2, and K_4 equal 1.0. K_3 will be equal to 1.1 since the bed condition at the bridge is plane-bed. The depth of flow at the pier for the 100- and 500-year storm surge is determined by adding the mean storm tide elevation from Table 9.2 to the flow depth at the pier referenced to mean sea level (3.75 m). From this, y_1 will be equal to 4.85 and 5.19 m for the 100- and 500-year storm surge, respectively.

Applying Equation 6.1 for the 100-year event:

$$\frac{y_s}{4.85} = 2.0\,(1.0)(1.0)(1.1)(1.0)\left[\frac{3.66}{4.85}\right]^{0.65}(0.094)^{0.43} = 0.66$$

From the above equation, the local scour at the piers is 3.2 m. Considering the 500-year event, local pier scour is 3.6 m.

9.8.2 Example Problem 2 - Constricted Waterway

This problem presents a Level 2 analysis of a bridge over a tidal inlet where the waterway constricts the flow. In addition, it illustrates how depletion of sediment supplied to the tidal inlet can result in a continual and severe long-term degradation. The length of the inlet is 457.2 m, the width of the bridge opening and inlet is 124.97 m, Manning's n is 0.03, depth of flow at mean water level is 6.1 m and area A_c is 761.81 m^2. The D_{50} of the bed material is 0.30 mm and the D_m (1.25 D_{50}) is 0.375 mm (0.000375 m).

From tidal records, the long-term average difference in elevation from the ocean to the bay, through the waterway, averaged for both the flood and ebb tide is 0.183 m. The difference in elevation for the 100-year storm surge is 0.549 m and for the 500-year storm surge is 0.884 m.

a. Determine the long-term potential degradation that may occur because construction of jetties has cut off the delivery of bed sediments from littoral drift to the inlet.

For this situation, long-term degradation can be approximated by assuming clear-water contraction scour and using the average difference in water surface between the ocean and bay for astronomical tides. The hydraulic computation uses the orifice equations (Equations 9.5 through 9.10).

Using Equation 9.8, determine R (assume K_o = 0.7 and K_b = 1.0 for this location)

$$R = 0.7 + 1.0 + \frac{2(9.81)(0.03)^2\,457.2}{(6.10)^{4/3}}$$

$$R = 2.42$$

From Equation 9.7 determine C_d

$$C_d = \left(\frac{1}{2.42}\right)^{1/2}$$

$$C_d = 0.643$$

Using Equation 9.5, determine V_{max}

$$V_{max} = 0.643\sqrt{(2)(9.81)(0.183)}$$

$$V_{max} = 1.22\,m/s$$

Using Equation 9.6 determine Q_{max}

$$Q_{max} = V_{max}\,A_c = 1.22\,(761.81)$$

$$Q_{max} = 929.41\,m^3/s$$

Potential long-term degradation for fine bed material is determined using the clear-water contraction scour equation (Equation 5.4):

$$y = \left[\frac{0.025\,(929.41)^2}{(0.000375)^{2/3}\,(124.97)^2}\right]^{3/7} = 10.94\,m$$

$$y_s = 10.94 - 6.10 = 4.84\,m$$

Discussion of Potential Long-Term Degradation

This amount of scour would occur in some time period that would depend on the amount of sediment that was available from the bay and ocean side of the waterway to satisfy the transport capacity of the back and forth movement of the water from the flood and ebb tide.

Even if there was no sediment inflow into the waterway, the time it would take to reach this depth of scour is not known.

To determine the length of time would require the use of an unsteady tidal model, and conducting a sediment continuity analysis (see Section 9.6). Using a tidal model and sediment continuity analysis, calculate the amount of sediment eroded from the waterway during a tidal cycle and determine how much degradation this will cause. Then using this new average depth, recalculate the variables and repeat the process. Knowing the time period of the tidal cycle, then the time to reach a scour depth of 4.84 m could be estimated for the case of no sediment inflow into the waterway. Estimates of sediment inflow in a tidal cycle could be used to determine the time to reach the above estimated contraction scour depth when there is sediment inflow.

When the long-term degradation reaches 4.84 m, the scouring may not stop. The reason for this is that the discharge in the waterway is not limited, as in the case of inland rivers, but depends on the amount of flow that can enter the bay in a half tidal cycle. As the area of the waterway increases the flood tide discharge increases because, as an examination of Equations 9.5 and 9.6 show the velocity does not decrease. There may be a slight decrease in velocity because the difference in elevation from the ocean and the bay might decrease as the area increases. However, R in Equation 9.8 decreases with an increase in depth.

Although the above discussion would indicate that long-term degradation would increase indefinitely, this is not the case. As the scour depth increases there would be changes in the relationship between the incoming tide and the tide in the bay or estuary, and also between the tide in the bay and the ocean on the ebb tide. This could change the difference in elevation between the bay and ocean. At some level of degradation the incoming or out-going tides could pick up sediment from either the bay or ocean which would then satisfy the transport capacity of the flow. Also, there could be other changes as scour progressed, such as accumulation of larger bed material on the surface (armor) or exposure of scour resistance rock which would decrease or stop the scour.

In spite of these limiting factors, the above problem illustrates the fact that with tidal flow, in contrast to river flow, as the area of the cross section increases from degradation there may be no decrease in velocity and discharge.

b. Determine V_{max}, Q_{max} for the 100-year storm surge and a depth of 6.1 m.

The values of R and C_d do not change.

$$V_{max} = 0.643\sqrt{(2)(9.81)(0.549)}$$

$$V_{max} = 2.11 m/s$$

$$Q_{max} = 2.11(761.81) = 1607.42 m^3/s$$

These values or similar ones depending on the long-term scour depth, would be used to determine the local scour at piers and abutments using equations given previously. These values could also be used to calculate contraction scour resulting from the storm surge.

9.9 TIDAL SCOUR EXAMPLE PROBLEMS (English)

9.9.1 Example Problem 1 - Tidal Prism Approach (Unconstricted Waterway) (English)

In this example problem, the discharge, velocity, depths, and scour are to be determined for an existing bridge across a tidal estuary as part of an ongoing scour evaluation. The bridge is 2,685 ft long, has vertical wall abutments and sixteen 12 ft diameter circular piers supported on piles. Neither the bridge nor the tidal waterway constricts the flow.

For this evaluation, the bridge maintenance engineer has expressed concern about observed scour at one of the piers. This pier is located where the velocities at the pier are approximately 30 percent greater than the average velocities. The water depth at the pier referenced to mean sea level is 12.30 ft. The actual depth of flow at the pier will need to be increased to account for additional water depth caused by the storm surge for the computation of pier scour.

Level 1 Analysis

a. Level 1 analysis has determined that the storm surge for the 100- and 500-year return period produces discharge, velocity, and depths that are much larger than those from inland runoff. There is minimal littoral drift and historical tides are low. From FEMA, the storm surge tidal range for the 100-year return period is 7.18 ft and for the 500-year return period is 9.42 ft. Measured maximum velocity in the waterway at mean sea level for a tide of 2.20 ft was only 0.70 ft/s.

   Sonic soundings in the waterway indicate thta there is storage of sediment in the estuary directly inland from the bridge crossing. This was determined by observing that the elevation of the bed of the waterway at the bridge site was lower than the elevation of the bottom of the estuary further inland. Although no littoral drift is evident, there is storage of sediment at the mouth of the estuary between the ocean and the bridge crossing.

b. Stability of the estuary and crossing was evaluated by examination of the periodic bridge inspection reports which included underwater inspections by divers, evaluation of historical aerial photography, and depth soundings in the estuary using sonic fathometers. From this evaluation it was determined that the planform of the estuary has not changed significantly in the past 30 years. These observations indicate that the estuary and bridge crossing has been laterally stable.

   Evaluation of sounding data at the bridge indicates that there has been approximately 5.0 ft of degradation at the bridge over the past 30 years; however, the rate of degradation in the past five years has been negligible. Underwater inspections indicted that local scour around the piers is evident.

c. A search of FEMA, USACE, and other public agencies for inland flood and storm surge data was conducted. These data will be discussed under the Level 2 analysis.

d. Grain size analysis of the bed material indicates that the bed of the estuary is composed of fine sand with a D_{50} of approximately 0.27 mm (0.00089 ft).

e. Velocities measured at Q_{max} during a large astronomical tide indicated that the maximum velocity in the bridge section was approximately 30 percent greater than the average velocity.

Level 2 Analysis

STEP 1. A plot of net waterway area as a function of elevation is given in Figure 9.9. Net waterway area is the average area at the bridge crossing less the area of the piers.

STEP 2. A plot of volume of the tidal prism as a function of elevation is also presented in Figure 9.9. The plot was developed by planimetering the area of successive sounding and contour lines and multiplying the average area by the vertical distance between them.

STEP 3. A synthesized storm surge for the 100- and 500-year return period was developed and is presented in Figure 9.9. It was obtained as follows:

An idealized graph for one half the tidal period, beginning at high tide was developed using the cosine equation (Equation 9.1). This plot can be used to develop an idealized tidal cycle for any waterway. Tidal range and period are needed to use the idealized tide cycle to develop a synthesized tidal cycle for this waterway.

The tidal ranges were obtained from a FEMA coastal flood insurance study during the Level 1 analysis (Table 9.3).

Table 9.3. Tidal Ranges Derived from FEMA Flood Study.

Return Period (yr)	High Tide (ft)	Low Tide (ft)
100	7.20	0
500	9.42	0

The tidal period is more difficult to determine because it is affected by more than the gravitational attraction of the moon and sun. At this waterway location, the direction of the storm and the characteristics of the estuary affected the tidal period. To determine the tidal period, major storm tides were plotted in Figure 9.9. Review of these historical storm tides reveals that (as expected) most events occur over a duration longer than an astronomical tidal period. Only a single event exhibits a seemingly semi-diurnal response. Given these characteristics and behavior, analyses yield a conservative estimate that approximately 12 hours pass between the highest and lowest elevations. This assumption would therefore indicate that the associated storm tide period (T) is 24 hours.

STEP 4. Using the data developed in Steps 1 to 3 and the equations given previously the maximum tidal discharge (Q_{max}) and maximum average tidal velocity (V_{max}) are calculated. The values used in the calculations are given in Table 9.4.

STEP 5. The 100- and 500-year return period peak inland flow into the estuary was obtained from a USGS flood frequency study. These values are also given in Table 9.4.

Average flow depths can be determined by dividing the flow area as listed in Table 9.4 by the channel width (2,685 ft). Therefore the average flow depth for the 100- and 500-year event are 14.5 and 15.3 ft, respectively.

The peak discharge from the 100- and 500-year inland flow hydrograph is very small in comparison to the storage volume in the estuary. In this case, adding the inland peak discharge to the maximum tidal discharge will be a conservative estimate of the maximum discharge and maximum average velocity in the waterway. If the inland inflow into the estuary had been large, the flood could be routed through the estuary using standard hydrologic modeling techniques.

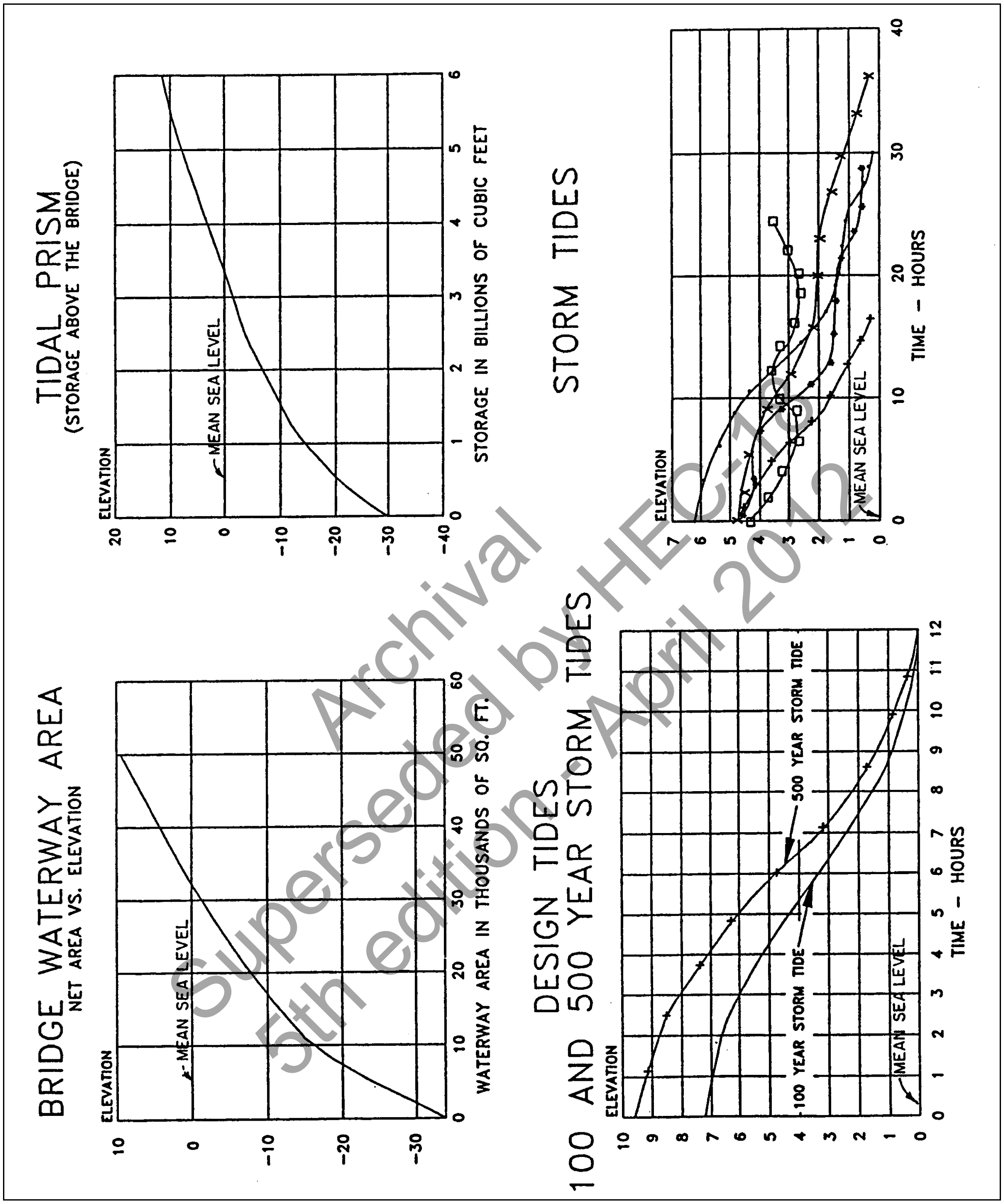


Figure 9.9. Tidal parameters for Example Problem 1 (English).

Table 9.4. Design Discharge and Velocities.		
	100-Year Storm Tide	500-Year Storm Tide
Maximum storm tide elevation, ft	7.19	9.42
Mean storm tide elevation, ft	3.61	4.72
Low storm tide elevation, ft	0.0	0.0
Tidal prism volume, ft^3, Figure 9.9	1,639	2,147
Net waterway area at mean storm tide elevation (A_c), ft^2	39,000	41,000
Tidal period, h	24.0	24.0
Q_{max} (Tidal), ft^3/s (Equation 9.2)	59,550	78,030
V_{max} (Tidal), ft/s (Equation 9.3)	1.54	1.90
Inland peak runoff (discharge), ft^3/s	4,980	7,920
Q_{max} (Tidal plus runoff), ft^3/s	64,530	85,950
V_{max} (Tidal plus runoff), ft/s ($V_{max} = Q_{max}/A_c$)	1.64	2.10
Average flow depth (A_c/width), ft	14.5	15.26

STEP 6. A comparison of the calculated velocities with the measured velocities indicate that they are reasonable. The discharge and velocities given in Table 9.4 are acceptable for determining the scour depths. However, the average velocity will have to be adjusted for the nonuniformity of flow velocity in the vicinity of the bridge to obtain the velocities for determining local scour at the piers.

STEP 7. Calculate the components of total scour using the information collected in the Level 1 and Level 2 analyses.

Long-Term Aggradation/Degradation

The Level 1 analysis indicates that the channel is relatively stable at this time. However, there is an indication that over the past 30 years the channel has degraded approximately 5.0 ft. Therefore, for this evaluation, an estimate of long-term degradation of approximately 5.0 ft for the future will be assumed.

Contraction Scour

Contraction scour depends on whether the flow will be clear-water or live-bed. Equation 5.1 is used to determine the critical velocity for the 100-year hydraulics.

$$V_c = 11.17\,(14.50)^{1/6}\,(0.00089)^{1/3} = 1.68\ \text{ft / s}$$

This indicates that the 100-year storm surge combined with the inland flow may result in velocities greater than or equal to the critical velocity; therefore, contraction scour will most likely be live-bed. This conclusion is made considering that velocities in excess of the average velocity will be expected due to the nonuniformity of the velocity in the bridge opening, as determined during the Level 1 analysis.

Applying the live-bed contraction scour equation, it is noted that the ratio of discharges is equal to unity (i.e., there is no overbank flow). Therefore, the contraction scour will be influenced by the contraction resulting from the bridge piers reducing the flow width at the bridge crossing. Using Equation 5.2, and assuming that the mode of sediment transport is mostly suspended load (k_1=0.69), the estimate of live-bed contraction scour for the 100-year event is:

$$\frac{y_2}{14.50} = \left[\frac{2685}{2493}\right]^{0.69} = 1.05$$

$$y_2 = 15.26\ \text{ft}$$

$$y_s = 15.26 - 14.50 = 0.76\ \text{ft}$$

Therefore, the contraction scour for the 100-year event is approximately 0.76 ft. Recomputation for the 500-year event with an average flow depth of 15.26 ft results in an estimate of contraction scour of approximately 0.80 ft.

Local Scour at Piers

The hydraulic analysis estimates average velocities in the bridge cross section only. Because of this, an estimate of the maximum velocity at the bridge pier is made to account for non-uniform velocity in the bridge cross section. The average velocity will be increased by 30 percent since velocities for normal flows (Level 1) indicated that the maximum velocity was observed to be approximately 30 percent greater than the average. Therefore the maximum velocity for the 100- and 500-year event are 2.13 and 2.72 ft/s, respectively.

K_1, K_2, and K_4 equal 1.0. K_3 will be equal to 1.1 since the bed condition at the bridge is plane-bed. The depth of flow at the pier for the 100- and 500-year storm surge is determined by adding the mean storm tide elevation from Table 9.4 to the flow depth at the pier referenced to mean sea level (12.3 ft). From this, y_1 will be equal to 15.9 and 17.0 ft for the 100- and 500-year storm surge, respectively.

Applying Equation 6.1 for the 100-year event:

$$\frac{y_s}{15.9} = 2.0\,(1.0)(1.0)(1.1)(1.0)\left[\frac{12.0}{15.9}\right]^{0.65}(0.094)^{0.43} = 0.66$$

From the above equation, the local scour at the piers is 10.5 ft. Considering the 500-year event, local pier scour is 11.8 ft.

9.9.2 Example Problem 2 - Constricted Waterway (English)

This problem presents a Level 2 analysis of a bridge over a tidal inlet where the waterway constricts the flow. In addition, it illustrates how depletion of sediment supplied to the tidal inlet can result in a continual and severe long-term degradation. The length of the inlet is 1,500 ft, the width of the bridge opening and inlet is 410 ft, Manning's n is 0.03, depth of flow at mean water level is 20.0 ft and area A_c is 8,200 ft^2. The D_{50} of the bed material is 0.30 mm and the D_m (1.25 D_{50}) is 0.375 mm (0.0012 ft).

From tidal records, the long-term average difference in elevation from the ocean to the bay, through the waterway, averaged for both the flood and ebb tide is 0.6 ft. The difference in elevation for the 100-year storm surge is 1.8 ft and for the 500-year storm surge is 2.9 ft.

a. Determine the long-term potential degradation that may occur because construction of jetties has cut off the delivery of bed sediments from littoral drift to the inlet.

For this situation, long-term degradation can be approximated by assuming clear-water contraction scour and using the average difference in water surface between the ocean and bay for astronomical tides. The hydraulic computation uses the orifice equations (Equations 9.5 through 9.10).

Using Equation 9.8, determine R (assume K_o = 0.7 and K_b = 1.0 for this location).

$$R = 0.7 + 1.0 + \frac{2(32.2)(0.03)^2\,1500}{(1.49)^2\,(20.0)^{4/3}}$$

$$R = 2.42$$

From Equation 9.7 determine C_d

$$C_d = \left(\frac{1}{2.42}\right)^{1/2}$$

$$C_d = 0.643$$

Using Equation 9.5, determine V_{max}

$$V_{max} = 0.643\sqrt{(2)(32.2)(0.6)}$$

$$V_{max} = 4.0\ \text{ft / s}$$

Using Equation 9.6 determine Q_{max}

$$Q_{max} = V_{max}\,A_c = 4.0\,(8{,}200)$$

$$Q_{max} = 32{,}800\ \text{cfs}$$

Potential long-term degradation for fine bed material is determined using the clear-water contraction scour equation (Equation 5.4):

$$y = \left[\frac{0.0077\,(32{,}800)^2}{(0.0012)^{2/3}\;(410)^2}\right]^{3/7} = 36.3\ \text{ft}$$

$$y_s = 36.3 - 20.0 = 16.3\ \text{ft}$$

Discussion of Potential Long-Term Degradation

This amount of scour would occur in some time period that would depend on the amount of sediment that was available from the bay and ocean side of the waterway to satisfy the transport capacity of the back and forth movement of the water from the flood and ebb tide.

Even if there was no sediment inflow into the waterway, the time it would take to reach this depth of scour is not known.

To determine the length of time would require the use of an unsteady tidal model, and conducting a sediment continuity analysis (see Section 9.6). Using a tidal model and sediment continuity analysis, calculate the amount of sediment eroded from the waterway during a tidal cycle and determine how much degradation this will cause. Then using this new average depth, recalculate the variables and repeat the process. Knowing the time period of the tidal cycle, then the time to reach a scour depth of 16.3 ft could be estimated for the case of no sediment inflow into the waterway. Estimates of sediment inflow in a tidal cycle could be used to determine the time to reach the above estimated contraction scour depth when there is sediment inflow.

When the long-term degradation reaches 16.3 ft, the scouring may not stop. The reason for this is that the discharge in the waterway is not limited, as in the case of inland rivers, but depends on the amount of flow that can enter the bay in a half tidal cycle. As the area of the waterway increases the flood tide discharge increases because, as an examination of Equations 9.5 and 9.6 show the velocity does not decrease. There may be a slight decrease in velocity because the difference in elevation from the ocean and the bay might decrease as the area increases. However, R in Equation 9.8 decreases with an increase in depth.

Although the above discussion would indicate that long-term degradation would increase indefinitely, this is not the case. As the scour depth increases there would be changes in the relationship between the incoming tide and the tide in the bay or estuary, and also between the tide in the bay and the ocean on the ebb tide. This could change the difference in elevation between the bay and ocean. At some level of degradation the incoming or out-going tides could pick up sediment from either the bay or ocean which would then satisfy the transport capacity of the flow. Also, there could be other changes as scour progressed, such as accumulation of larger bed material on the surface (armor) or exposure of scour resistance rock which would decrease or stop the scour.

In spite of these limiting factors, the above problem illustrates the fact that with tidal flow, in contrast to river flow, as the area of the cross section increases from degradation there may be no decrease in velocity and discharge.

b. Determine V_{max}, Q_{max} for the 100-year storm surge and a depth of 20.0 ft.

The values of R and C_d do not change.

$$V_{max} = 0.643\sqrt{(2)(32.2)1.8}$$

$$V_{max} = 6.92\,ft/s$$

$$Q_{max} = 6.92(8200) = 56{,}744\,ft^3/s$$

These values or similar ones depending on the long-term scour depth, would be used to determine the local scour at piers and abutments using equations given previously. These values could also be used to calculate contraction scour resulting from the storm surge.

CHAPTER 10

NATIONAL SCOUR EVALUATION PROGRAM

10.1 INTRODUCTION

The State departments of transportation (DOTs) have been conducting scour evaluations of their bridges over water in accordance with the 1991 FHWA Technical Advisory T 5140.23.[9] A scour screening started in 1988 as the result of Technical Advisory T 5140.20 which was superseded by T 5140.23[9] (see Appendix I). The evaluation is to be conducted by an interdisciplinary team of hydraulic, geotechnical and structural engineers who can make the necessary engineering judgments to determine the vulnerability of a bridge to scour. In general, the program consisted of screening all bridges over water to determine their scour vulnerability, and setting priorities for their evaluation. Each DOT structured its own evaluation program using guidelines furnished by FHWA. The screening and evaluation has helped bridge owners in rating each bridge in the National Bridge Inventory (NBI) using rating codes for item 113, Scour Critical Bridges.[10] A description of Item 113 rating codes is given in Appendix J along with the other codes for rating bridge foundations, i.e., Item 60 - Substructures, Item 61 - Channel and Channel Protection, Item 71 - Waterway Adequacy, Item 92 - Critical Feature Inspection, Item 93 - Critical Feature Inspection Date.

As of November 2000, virtually all bridges (99.9 percent) had received an initial screening and more than 90 percent of all bridges had been evaluated for scour. More than half of the DOTs have reported a 90 percent or better completion percentage for the evaluation of all their bridges over waterways.

10.1.1 The Scour Evaluation Program

The scour evaluation program consisted of:

1. Screening all bridges over water to determine:

    a. Whether or not a bridge is vulnerable to scour damage; i.e., whether the bridge is a low risk, scour susceptible, or scour critical bridge; and

    b. Priorities for making bridge scour evaluations.

    c. Scour screening to involve an office review and, if needed, a field inspection.

2. Evaluations consisted of:

    a. Review of bridge plans (when available) to determine foundation types, the elevation of footings and pile tips and the subsurface soils or rock on which the bridge is founded. If plans are not available, other sources of information, such as bridge inspection reports, were reviewed for available information. In some cases, the bridge foundations were unknown (see Appendix K). State DOTs have reported over 89,000 bridges with unknown foundations, meaning that the foundation type, material and/or tip elevations are unknown.

b. Development of hydrologic and hydraulic information for use in estimating scour at the bridge foundations.

c. Review of office files, inspection reports and other available information regarding previous actions taken to maintain and protect the bridge over its service life.

d. Conducting a field inspection to evaluate present conditions and to assess potential problems, which may occur during a future flood event.

e. Evaluation by the interdisciplinary team of the ability of the bridge to resist the anticipated scour based on the above findings, and the rating of the bridge under Item 113, Scour Critical Bridges.

f. An interdisciplinary team consisting of a DOT's structural engineer, geotechnical engineer, hydraulic engineer, and bridge engineer.

3. Developing a plan of action for bridges identified by the interdisciplinary team as scour critical.

Scour evaluation required a broader scope of study and effort than those considered in a bridge inspection. The major purpose of the bridge inspection is to identify changed conditions which may reflect an existing or potential problem. The scour evaluation program has served as the mechanism to design new bridge foundations for scour and to evaluate the condition of existing bridge foundations through an engineering process.

In the following sections the results, to date, of the DOTs screening and evaluation of their bridges is given followed by a general description of the screening and evaluation process.

10.2 SCOUR EVALUATION RESULTS (1988 to 2000)

Bridges screened by the bridge owner as scour susceptible or scour critical needed to be evaluated for scour vulnerability. The evaluation was conducted by either (1) an assessment based on an office review of inspection reports and judgment and/or (2) an analysis using guidelines presented in this manual and HEC-20,[(6)] "Stream Stability at Highway Structures." Generally, the evaluation was accomplished by an interdisciplinary team comprised of hydraulic, structural, geotechnical engineers. Figure 10.1 shows a summary of the status of scour evaluations as of November 2000. Bridges with unknown foundations and over tidal waters are currently being evaluated by many State DOTs.

10.3 SCOUR SCREENING AND EVALUATION PROCESSES

Each DOT developed its own program for conducting its scour evaluations. In general the following approach was used by the DOTs to assess the vulnerability of existing bridges to scour:

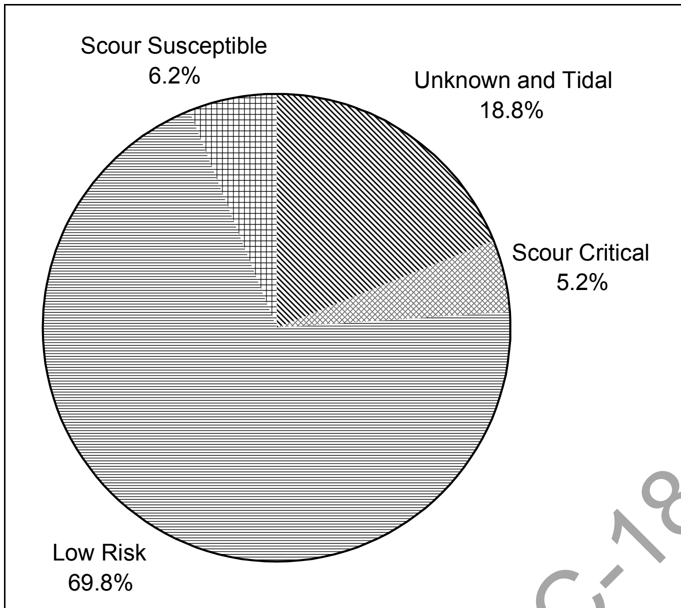


Figure 10.1. Scour evaluation status (as of November 15, 2000).

STEP 1. All bridges over waterways were screened into five categories: (1) low risk, (2) scour susceptible, (3) scour critical, (4) unknown foundations, or (5) tidal. Bridges which were particularly vulnerable to scour failure were identified immediately and the associated scour problem addressed. These particularly vulnerable "scour susceptible" bridges were:

a. Bridges currently experiencing scour or that have a history of scour problems during past floods as identified from maintenance records and experience, bridge inspection records, etc.

b. Bridges over streams with erodible streambeds with design features that make them vulnerable to scour, including:

- Piers and abutments designed with spread footings or short pile foundations;

- Superstructures with simple spans or nonredundant support systems that render them vulnerable to collapse in the event of foundation movement; and

- Bridges with inadequate waterway openings or with designs that collect ice and debris. Particular attention was given to structures where there are no relief bridges or embankments for overtopping, and where all water must pass through or over the structure.

c. Bridges on aggressive streams and waterways, including those with:

- Active degradation or aggradation of the streambed;

- Significant lateral movement or erosion of streambanks;

- Steep slopes or high velocities;

- Instream sand and gravel and other materials mining operations in the vicinity of the bridge; and

- Histories of flood damaged highways and bridges.

d. Bridges located on stream reaches with adverse flow characteristics, including:

   - Crossings near stream confluences, especially bridge crossings of tributary streams near their confluence with larger streams;

   - Crossings on sharp bends in a stream; and

   - Locations on alluvial fans.

STEP 2. Scour susceptible bridges and bridges with unknown foundations (See Appendix K) were prioritized by conducting a preliminary office and field examination of the list of bridges compiled in Step 1, using the following factors as a guide:

a. The potential for bridge collapse or for damage to the bridge in the event of a major flood; and

b. The functional classification of the highway on which the bridge is located, and 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. Field and office scour evaluations were conducted on the bridges prioritized in Step 2 using an Interdisciplinary Team of hydraulic, geotechnical, and structural engineers:

a. The evaluation procedure estimated scour for a superflood, a flood exceeding the 100-year flood, and then analyzed the foundations for vertical and lateral stability for this condition of scour. This evaluation approach was the same as the check procedure set forth in Section 2.2, Step 8. An overtopping flood was used where applicable. The difference between designing a new bridge and assessing an old bridge is simply that the location and geometry of a new bridge and its foundation are not fixed as they are for an existing bridge. Thus, the same steps for predicting scour at the piers and abutments were carried out for an existing bridge as for a new bridge. As with the design of a new bridge, engineering judgment was exercised in establishing the total scour depth for an existing bridge. The maximum scour depths that the existing foundation can withstand was compared with the total scour depth. An engineering assessment was made as to whether the bridge should be classified as a scour critical bridge; that is, whether the bridge foundations will be unstable if the estimated scour were to occur.

b. The results of the scour evaluation study was entered into the bridge inventory in accordance with the instructions in the FHWA "Recording and Coding Guide" (see Appendix J).[10] The following codes were used:

   - Bridges assessed as "low risk" for Item 113 (Scour Critical Bridges) were coded as an "9, 8, 7, 5, or 4."

   - Bridges with unknown foundations (except for interstate bridges) were coded as a "U" in Item 113, indicating that a scour evaluation/calculation has not been made.

- Bridges over tidal waterways were coded "T" and monitored with the regular inspection cycle and with appropriate underwater inspections. These bridges in the most part have been evaluated.

- Bridges assessed to be "scour susceptible" are coded as "6" for Item 113 until such time that further scour evaluations determine foundation conditions.

- Interstate bridges with unknown foundations or over tidal waterways are coded as 6.

- Bridges considered scour critical based on an assessment or calculation are coded as a 3 for Item 113. Bridges coded as scour critical, based on an observed condition are coded as 2, 1, or 0.

STEP 4. Bridges identified as scour critical from the office and field review or during a bridge inspection in Step 2 should have a plan of action developed for correcting the scour problem (see Chapter 12). This plan of action should include:

a. Specific instructions regarding the type and frequency of inspections to be made at the bridge, particularly in regard to monitoring the performance and closing of the bridge, if necessary, during and after flood events.

b. A schedule for the timely design and construction of scour countermeasures determined to be needed for the protection of the bridge.

STEP 5. After completing the scour evaluations for the list of potential problems compiled in Step 1, the remaining waterway bridges included in the State's bridge inventory should be evaluated. In order to provide a logical sequence for accomplishing the remaining bridge scour evaluations, another bridge list should be established, giving priority status to the following:

a. The functional classification of the highway on which the bridge is located with highest priorities assigned to arterial highways and lowest priorities to local roads and streets.

b. Bridges that serve as vital links in the transportation network and whose failure could adversely affect area or regional traffic operations.

The ultimate objectives of the scour evaluation program are to (1) evaluate all bridges over streams in the National Bridge Inventory, (2) determine those foundations which are stable for estimated scour conditions and those which are not, and (3) provide scour protection for scour critical bridges until the bridge can be made safe from scour. This may include scour protection to reduce the risk such as riprap, closing the bridge during high water, monitoring of scour critical bridges during, and inspection after flood events. The final objective (4) would be to replace the bridge or install properly designed scour countermeasures in a timely manner, depending upon the perceived risk involved.

STEP 6. Bridge owners have come to recognize that the rating of bridges for Item 113, Scour Critical Bridges, and the prioritization of bridges for installation of scour countermeasures are not a one-time effort. There is a continuing need to review the Item 113 rating of all bridges during routine inspections and especially after flood events.

A rating of "low risk" for a structure may be changed to "scour critical" after the occurrence of a single flood for a number of reasons including (1) lateral migration of the channel, (2) head cutting and channel degradation with resultant exposure of pile foundations, (3) shifting of the channel thalweg so that a severe angle of attack develops for a pier or abutment which increases local scour. Similarly, a scour critical bridge protected with riprap may require immediate attention after a flood if the riprap is displaced and scour undermines pier or abutment foundations. The bridge inspector should be trained to recognize changes to the river and the effect of such changes on the bridge foundation. The inspector can code Item 113 for the observed scour condition if scour calculations are available to compare the observed with the existing condition. The inspector is charged with notifying his (her) supervisors when significant changes are noticed. The interdisciplinary team should promptly inspect the changed conditions so that appropriate action, commensurate with the perceived risk, can be initiated. The bridge should then be immediately recoded for Item 113 and the related items pertaining to scour and bridge and channel stability set forth in Appendix J.

10.4 UNKNOWN FOUNDATIONS

Bridges are classified as having unknown foundations when the type (spread footing, piles, columns), material (steel, concrete, or timber), dimensions (length, width, or thickness), reinforcing, and/or elevation are unknown. They are classified as "U" in Item 113 of the Coding Guide (Appendix J). The screening program in the national evaluation program has identified about 89,000 bridges with unknown foundations. Research under the National Cooperative Highway Research Program (NCHRP) has investigated nondestructive testing methods which in many cases can determine pile length. Appendix K provides a status report and guidance for a plan of action for protecting bridges with unknown foundations from scour.

CHAPTER 11

INSPECTION OF BRIDGES FOR SCOUR

11.1 INTRODUCTION

There are two main objectives to be accomplished in inspecting bridges for scour:

1. Accurately record the present condition of the bridge and the stream, and

2. Identify conditions that are indicative of potential problems with scour and stream stability for further review and evaluation by others.

In order to accomplish these objectives, the inspector needs to recognize and understand the interrelationship between the bridge, the stream, and the floodplain. Typically, a bridge spans the main channel of a stream and perhaps a portion of the floodplain. The road approaches to the bridge are typically on embankments which obstruct flow on the floodplain. This overbank or floodplain flow must, therefore, return to the stream at the bridge and/or overtop the approach roadways. Where overbank flow is forced to return to the main channel at the bridge, zones of turbulence are established and scour is likely to occur at the bridge abutments. Further, piers and abutments may present obstacles to flood flows in the main channel, creating conditions for local scour because of the turbulence around the foundations. After flowing through the bridge, the flood water will expand back to the floodplain, creating additional zones of turbulence and scour.

The following sections present guidance for the bridge inspector's use in developing an understanding of the overall flood flow patterns at each bridge inspected. Guidance on the use of this information for rating the present condition of the bridge and evaluating the potential for damage from scour is also presented. When an actual or potential scour problem is identified by a bridge inspector, the bridge should be further evaluated by an Interdisciplinary Team using the approach discussed in Chapter 10. The results of this evaluation should be recorded under Item 113 of the "Recording and Coding Guide" (Appendix J).[8, 9, 10]

If the bridge is determined to be scour critical, a Plan of Action (Chapter 12) should be developed for installing scour countermeasures. Also, the rating of the bridge substructure (Item 60 of the Recording and Coding Guide) should be consistent with the rating of Item 113 for the observed scour on the substructure.[10]

11.2 OFFICE REVIEW

It is desirable to make an office review of bridge plans and previous inspection reports prior to making the bridge inspection. Information obtained from the office review provides a better basis for inspecting the bridge and the stream. Items for consideration in the office review include:

1. Has an engineering scour evaluation study been made? If so, is the bridge scour-critical?

2. If the bridge is scour-critical, has a Plan of Action been developed?

3. What do comparisons of streambed cross sections taken during successive inspections reveal about the streambed? Is it stable? Degrading? Aggrading? Moving laterally? Are there scour holes around piers and abutments?

4. What equipment is needed (rods, poles, sounding lines, sonar, etc.) to obtain streambed cross sections?

5. Are there sketches and aerial photographs to indicate the planform location of the stream and whether the main channel is changing direction at the bridge?

6. What type of bridge foundation was constructed? (Spread footings, piles, drilled shafts, etc.) Are footing and pile tip elevations known? Do the foundations appear to be vulnerable to scour? What are the sub-surface soil conditions? (sand, gravel, silt, clay rock?)

7. Do special conditions exist requiring particular methods and equipment (divers, boats, electronic gear for measuring stream bottom, etc.) for underwater inspections?

8. Are there special items that should be looked at? (Examples might include damaged riprap, stream channel at adverse angle of flow, problems with debris, etc.)

11.3 BRIDGE INSPECTION

11.3.1 Safety Considerations

The bridge inspection team should understand and practice prudent safety precautions during the conduct of the bridge inspection. Warning signs should be set up at the approaches to the bridge to alert motorists of the activity on the bridge. This is particularly important if streambed measurements are to be taken from the bridge, since most bridges have minimal clearances between the parapet and the edge of the travel lane. Inspectors should wear brightly colored vests so that they are conspicuous to motorists.

When measurements are made in the stream, the inspector should be secured by a safety line whenever there is deep or fast flowing water and a boat should be available in case of emergency. If waders become overtopped, they will fill and may drag the inspector downstream and under water in a matter of a few seconds.

The inspection team should leave word with their office regarding their schedule of work for the day. The team should also carry a cell phone with them so that they can get immediate help in the event of an emergency.

11.3.2 FHWA Recording and Coding Guide

During the bridge inspection, the condition of the bridge waterway opening, substructure, channel protection, and scour countermeasures should be evaluated, along with the condition of the stream.

The FHWA Recording and Coding Guide (Appendix J) contains guidance for the following items:[10]

1. Item 60: Substructure
2. Item 61: Channel and Channel Protection
3. Item 71: Waterway Adequacy
4. Item 113: Scour Critical Bridges

The guidance in the Recording and Coding Guide for rating the present condition of Items 61, 71, and 113 is set forth in detail. Guidance for rating the present condition of Item 60, Substructure, is general and does not include specific details for scour; however, the rating given to Item 60 should be consistent with the one given for Item 113 whenever a rating of 2 or below is determined for Item 113.

The following sections present approaches to evaluating the present condition of the bridge foundation for scour and the overall scour potential at the bridge.

11.3.3 General Site Considerations

In order to appreciate the relationship between the bridge and the river it is crossing, observation should be made of the conditions of the river up- and downstream of the bridge:

- Is there evidence of general degradation or aggradation of the river channel resulting in unstable bed and banks?
- Is there evidence of on-going development in the watershed and particularly in the adjacent floodplain that could be contributing to channel instability?
- Are there active gravel or sand mining operations in the channel near the bridge?
- Are there confluences with other streams? How will the confluence affect flood flow and sediment transport conditions?
- Is there evidence at the bridge or in the up- and downstream reaches that the stream carries large amounts of debris? Is the bridge superstructure and substructure streamlined to pass debris, or is it likely that debris will hang up on the bridge and create adverse flow patterns with resulting scour?
- The best way of evaluating flow conditions through the bridge is to look at and photograph the bridge from the up- and downstream channel. Is there a significant angle of attack of the flow on a pier or abutment?

11.3.4 Assessing the Substructure Condition

Item 60, Substructure, is the key item for rating the bridge foundations for vulnerability to scour damage. When a bridge inspector finds that a scour problem has already occurred, it should be considered in the rating of Item 60. Both existing and potential problems with scour should be reported so that a scour evaluation can be made by an interdisciplinary team. The scour evaluation is reported on Item 113 in the Recording and Coding Guide.[10] If the bridge is determined to be scour critical, the rating of Item 60 should be consistent to that of Item 113 to ensure that existing scour problems have been considered. The following items are recommended for consideration in inspecting the present condition of bridge foundations:

1. Evidence of movement of piers and abutments;
    - Rotational movement (check with plumb line)
    - Settlement (check lines of substructure and superstructure, bridge rail, etc., for discontinuities; check for structural cracking or spalling)
    - Check bridge seats for excessive movement

2. Damage to scour countermeasures protecting the foundations (riprap, guide banks, sheet piling, sills, etc.). Examples of damage could include riprap placed around piers and/or abutments that has been removed or replaced with river run bed material. A common cause of damage to abutment riprap protection is runoff from the ends of the bridge which flows down to the riprap and undermines it. This condition can be corrected by installing bridge-end drains.

3. Changes in streambed elevation at foundations (undermining of footings, exposure of piles), and

4. Changes in streambed cross section at the bridge, including location and depth of scour holes.

   - Note and measure any depressions around piers and abutments

   - Note the approach flow conditions. Is there an angle of attack of flood flow on piers or abutments?

In order to evaluate the conditions of the foundations, the inspector should measure the elevation of the streambed to a common bench mark at the bridge cross section during each inspection. These cross-section elevations should be plotted to a common datum and successive cross sections compared. Careful measurements should be made of scour holes at piers and abutments, probing soft material in scour holes to determine the location of a firm bottom. If equipment or conditions do not permit measurement of the stream bottom, this condition should be noted for further action.

11.3.5 Assessing Scour Potential at Bridges

The items listed in Table 11.1 are provided for bridge inspectors' consideration in assessing the adequacy of the bridge to resist scour. In making this assessment, inspectors need to understand and recognize the interrelationships between Item 60 (Substructure), Item 61 (Channel and Channel Protection), Item 71 (Waterway Adequacy), and 113 (Scour-Critical Bridges). As noted earlier, additional follow-up by an interdisciplinary team should be made utilizing Item 113 (Scour Critical Bridges) when the bridge inspection reveals a potential problem with scour (Appendix J).

11.3.6 Underwater Inspections

Perhaps the single most important aspect of inspecting the bridge for actual or potential damage from scour is taking and plotting of measurements of stream bottom elevations in relation to the bridge foundations. Where conditions are such that the stream bottom cannot be accurately measured by rods, poles, sounding lines or other means, other arrangements, such as underwater inspections, need to be made to determine the stream bottom elevation around the foundations and to determine the condition of the foundations. Other approaches to determining the cross section of the streambed at the bridge include:

1. Use of divers

2. Use of electronic scour detection equipment (HEC-23[7])

Table 11.1. Assessing the Scour Potential at Bridges.

1. UPSTREAM CONDITIONS

a. Banks

STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions; channel stabilization measures such as dikes and jetties.

UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures etc.

b. Main Channel

- Clear and open with good approach flow conditions, or meandering or braided with main channel at an angle to the orientation of the bridge.
- Existence of islands, bars, debris, cattle guards, fences that may affect flow.
- Aggrading or degrading streambed.
- Evidence of movement of channel with respect to bridge (make sketches, take pictures).
- Evidence of ponding of flow.

c. Floodplain

- Evidence of significant flow on floodplain.
- Floodplain flow patterns - does flow overtop road and/or return to main channel?
- Existence and hydraulic adequacy of relief bridges (if relief bridges are obstructed, they will affect flow patterns at the main channel bridge).
- Extent of floodplain development and any obstruction to flows approaching the bridge and its approaches.
- Evidence of overtopping approach roads (debris, erosion of embankment slopes, damage to riprap or pavement, etc.).
- Evidence of ponding of flow.

d. Debris

- Extent of debris in upstream channel.

e. Other Features

- Existence of upstream tributaries, bridges, dams, or other features, that may affect flow conditions at bridges.

Table continues

Table 11.1. Assessing the Scour Potential at Bridges (continued).

2. CONDITIONS AT BRIDGE

a. Substructure

- Is there evidence of scour at piers?
- Is there evidence of scour at abutments (upstream or downstream sections)?
- Is there evidence of scour at the approach roadway (upstream or downstream)?
- Are piles, pile caps or footings exposed?
- Is there debris on the piers or abutments?
- If riprap has been placed around piers or abutments, is it still in place?

b. Superstructure

- Evidence of overtopping by flood water (Is superstructure tied down to substructure to prevent displacement during floods?)
- Obstruction to flood flows (Does superstructure collect debris or present a large surface to the flow?)
- Design (Is superstructure vulnerable to collapse in the event of foundation movement, e.g., simple spans and nonredundant design for load transfer?)

c. Channel Protection and Scour Countermeasures

- Riprap (Is riprap adequately toed into the streambed or is it being undermined and washed away? Is riprap pier protection intact, or has riprap been removed and replaced by bed-load material? Can displaced riprap be seen in streambed below bridge?)
- Guide banks (Spur dikes) (Are guide banks in place? Have they been damaged by scour and erosion?)
- Stream and streambed (Is main current impinging upon piers and abutments at an angle? Is there evidence of scour and erosion of streambed and banks, especially adjacent to piers and abutments? Has stream cross section changed since last measurement? In what way?)

d. Waterway Area Does waterway area appear small in relation to the stream and floodplain? Is there evidence of scour across a large portion of the streambed at the bridge? Do bars, islands, vegetation, and debris constrict the flow and concentrate it in one section of the bridge or cause it to attack piers and abutments? Do the superstructure, piers, abutments, and fences, etc., collect debris and constrict flow? Are approach roads regularly overtopped? If waterway opening is inadequate, does this increase the scour potential at bridge foundations?

Table continues

Table 11.1 Assessing the Scour Potential at Bridges (continued).
3. DOWNSTREAM CONDITIONS a. Banks STABLE: Natural vegetation, trees, bank stabilization measures such as riprap, paving, gabions, channel stabilization measures such as dikes and jetties. UNSTABLE: Bank sloughing, undermining, evidence of lateral movement, damage to stream stabilization measures, etc. b. Main Channel • Clear and open with good "getaway" conditions, or meandering or braided with bends, islands, bars, cattle guards, debris, and fences that retard and obstruct flow. • Aggrading or degrading streambed. • Evidence of movement of channel with respect to the bridge (make sketches and take pictures). • Evidence of extensive bed erosion. c. Floodplain • Clear and open so that contracted flow at bridge will return smoothly to floodplain, or restricted and blocked by dikes, development, trees, debris, or other obstructions. • Evidence of scour and erosion due to downstream turbulence. d. Other Features • Downstream dams or confluence with larger stream which may cause variable tailwater depths. (This may create conditions for high velocity flow through bridge.)

For the purpose of evaluating resistance to scour of the substructure under Item 60 of the Recording and Coding Guide, the questions remain essentially the same for foundations in deep water as for foundations in shallow water:[10]

1. What is the configuration of the stream cross section at the bridge?

2. Have there been any changes as compared to previous cross section measurements? If so, does this indicate that (1) the stream is aggrading or degrading; or (2) local or contraction scour is occurring around piers and abutments?

3. What are the shapes and depths of scour holes?

4. 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?

5. Has riprap around a pier been moved or removed?

Technical Advisory T5140.21[96] contains additional guidance for underwater inspections by divers.

11.3.7 Notification Procedures

A positive means of promptly communicating inspection findings to proper agency personnel must be established. **Any condition that a bridge inspector considers to be of an emergency or potentially hazardous nature should be reported immediately**. That information as well as other conditions which do not pose an immediate hazard, but still warrant further action, should be conveyed to the interdisciplinary team for review.

A report form is, therefore, needed to communicate pertinent problem information to the hydraulic, structural, and geotechnical engineers. An existing report form may currently be used by bridge inspectors within a DOT to advise maintenance personnel of specific needs. Regardless of whether an existing report is used or a new one is developed, a bridge inspector should be provided the means of advising the interdisciplinary team of problems in a timely manner.

11.3.8 Post-Inspection Documentation

Following completion of the bridge inspection, the new channel cross section should be compared with the cross sections taken during previous inspections. The results of the comparison should be evaluated and documented. Many bridge inspectors now utilize lap top computers to facilitate the documentation of the inspection findings. Computers will also facilitate plotting of successive channel cross-sections to enable rapid evaluation of the changes. A bridge scour expert system, CAESAR,[97] is available to assist in this process.

11.4 CASE HISTORIES OF BRIDGE INSPECTION PROBLEMS

11.4.1 Introduction

Since 1987 there have been three bridge failures with loss of life that illustrate the importance of bridge inspections. In two of the failures inspectors failed to observe changed conditions that if corrected may have saved the bridge. In one case, the inspectors documented the changes, but there was no follow-up action to evaluate the changes and to protect the bridge. In the following sections, the inspection problems associated with these bridge failures are described and issues related to inspection are highlighted.

11.4.2 Schoharie Creek Bridge Failure

On April 5, 1987 the New York State Thruway Authority Bridge (I-90) over Schoharie Creek collapsed killing 10 persons[98,99] (see also HEC-23,[7] Design Guideline 8). The National Transportation Safety Board investigated the collapse and gave as the probable cause as:

"............the failure of the New York State Thruway Authority to maintain adequate rip rap around the bridge piers, which led to severe erosion in the soil beneath the spread footings. Contributing to the accident were ambivalent plans and specifications used for construction of the bridge, an inadequate NYSTA bridge inspection program, and inadequate oversight by the New York State Department of Transportation and the Federal Highway Administration. Contributing to the severity of the accident was the lack of structural redundancy in the bridge."

The bridge was built in 1953 on piers with spread footings and no piles. The footings were 1.5 m (5 ft) deep, 5.5 m (18 ft) wide and 25 m (82 ft) long. The tops of the footings were at the streambed and incised into a substrate consisting of ice contact stratified draft (glacial till). The footings were protected by riprap. In 1955 the bridge survived a larger flood (2084 m^3/s (73,600 cfs)) than the 1987 flood (1759 m^3/s (62,100 cfs)). However, from 1953 to 1987 the bridge was subjected to many floods which progressively removed riprap from the piers, enabling the spread footings to be undermined during the April 1987 flood (Figures 11.1 and 11.2).

The NYSTA inspected the bridge annually or biennially with the last inspection on April 1, 1986. A 1979 inspection by a consultant hired by NYSDOT indicated that most of the riprap around the piers was missing (Figures 11.1 and 11.2); however, the 1986 inspection failed to detect any problems with the condition of the riprap at the piers. Based on the Safety Board findings, the conclusions from this failure are that inspectors and their supervisors must recognize that riprap does not necessarily make a bridge safe from scour, and inspectors must be trained to recognize when riprap is missing and the significance of this condition.

11.4.3 Hatchie River Bridge Failure

On April 1, 1989 the northbound U.S. Route 51 bridge over the Hatchie River in Tennessee collapsed killing eight persons[100,101] (see also HEC-23,[7] Design Guideline 1). The National Transportation Safety Board investigated the collapse and gave as the probable cause:

".........the northward migration of the main river channel which the Tennessee Department of Transportation failed to evaluate and correct. Contributing to the severity of the accident was the lack of redundancy in the design of the bridge spans."

A 2-lane bridge on Route 51 was opened to traffic in 1936. It was (1,219 m (4,000 ft)) long and spanned the main channel (approximately 91 m (300 ft)) and the majority of the floodplain. In 1974 a second 2-lane (southbound) bridge was added. Its length was 305 m (1,000 ft) and centered approximately on the main channel downstream from the northbound bridge. The earthfull approaches to the new southbound bridge blocked the floodplain flow that had formerly moved through the open bents of the 1936 (northbound) bridge. This concentrated the flow in both bridges and caused the main channel to move northward and into the floodplain bents of the northbound bridge.

Each of the floodplain bents of the 1936 (northbound) bridge was on a pile cap (bottom elevation 237.9 ft) supported by five untreated wooden piles 6 m (20 ft) long. The main channel bridge was on piers with a pile cap (bottom elevation 223.67 ft) supported on 6 m (20 ft) long precast concrete piles. The northward movement of the channel exposed the piles of the bent next to the channel to local pier scour and it collapsed dropping three spans. The channel migration was documented by Tennessee DOT and U.S. Army Corps of Engineers (USACE) data.[101] At the time of the collapse the flow was not large 244 m^3/s (8,620 cfs) but the flow was overbank and of long duration. The maximum flood peak for the 1989 flood season was (813 m^3/s (28,700 cfs)) with a 3-year recurrence interval.

Figure 11.1. Photograph of riprap at pier 2, October 1956.[98,99]

Figure 11.2. Photograph of riprap at pier 2, August 1977 (flow is from right to left).[98,99]

Since 1975, the bridge had been inspected on 24 to 26 month intervals and the last inspection was in September 1987. The NTSB report stated "the 1979, 1985, and 1987 inspection reports accurately identified the channel migration around column bent 70," (the floodplain bent that failed). The report further stated "....on-site inspections of the northbound U.S. 51 Bridge adequately identified the exposure of the column bent footings and piles due to the northward migration of the Hatchie River channel." The report also noted that the inspectors did not have design or as-built plans with then during the inspection. Because of this, the inspectors were mistaken in the thickness of the pile cap and calculated that 0.3 m (1 ft) of the bent piles was exposed. Whereas, the piles were actually exposed .9 m (3 ft) in 1987. The Safety Board noted other (unrelated) bridge collapses where inspectors did not have design or as-built plans, and as a result, deficiencies were overlooked that contributed to bridge failures. Therefore, the Safety Board believes that "it is essential for inspectors to have available bridge design or as-built plans during the on-site bridge inspection."

The NTSB noted that although TDOT inspectors measured the streambed depth at each substructural element and the USACE maintained historical channel profile data at the bridge "a channel profile of the river was not being maintained by TDOT." As a result the TDOT evaluator of the inspection report used only the 1985 and 1987 measurements and would not have been able to determine the extent of channel migration. In other words, if the profiles had been plotted, the evaluator should have easily detected the lateral migration.

The Safety Board also noted that an underwater inspection did not occur in 1987 because the bridge foundation was submerged less than 3 m (10 ft), TDOT criteria at that time. In 1990, TDOT changed the criteria to 1 m (3.5 ft). The Safety Board stated "a diver inspection of the bridge should have been conducted following the 1987 inspection because of the exposure of the untreated timber piles noted in the inspection report."

In conclusion, inspectors should have design or as-built plans on site during an inspection and should measure and plot a profile of the river cross section at the bridge. Submerged bridge elements that can not be examined visually or by feel should have an underwater inspection. Good communication must be established between inspectors, evaluators and decision makers. Changes in the river need to be evaluated through comparisons of successive channel cross sections to determine whether the changes are (1) random and insignificant or (2) represent a significant pattern of change to the channel which may endanger the stability of the bridge.

11.4.4 Arroyo Pasajero Bridge Failure

On March 10, 1995 the two I-5 bridges over Los Gatos Creek (Arroyo Pasajero) in the California Central Valley near Coalinga collapsed killing seven persons and injuring one. CALTRANS retained a team of engineers from FHWA, USGS, and private consultants to investigate the accident. No report was prepared by CALTRANS but three of the investigators, in the interest of bridge engineering, prepared a paper which was published by ASCE.[102] The probable cause of the failure was:

> The minimum scour depth from long-term degradation 3 m (10 ft) from inspection records, contraction scour 2.6 m (8.5 ft) calculated using Laursen's live bed equation, and local pier scour 2 m (6.7 ft) determined from a model study, exposed 2.7 m (8.9 ft) of the cast in place columns below the point where there was steel reinforcement. The force of the flood waters (at an angle of attack of 15 to 26 degrees) on the unreinforced columns, with their area increase by a web wall and debris, caused the bridge to fail.

The bridges, built in 1967, were 37 m (122 ft) long, with vertical wall abutments (with wing walls) and three piers. Each pier consisted of six 406 mm (16 inch) cast in place concrete columns. The columns were spaced 2.3 m (7.5 ft) on centers. They were embedded 12.5 m (41 ft) below original ground surface but only had steel reinforcing for 5.2 m (17 ft) below the original ground surface. The abutments were on pile-supported footings and the piles were 11.3 m (36.7 ft) long. A flood in 1969 lowered the bed 1.83 m (6 ft) and damaged one column. In repairing the damage CALTRANS maintenance constructed a web wall 2.4 or 3.6 m (8 or 12 ft) high, 11.6 m (38 ft) long and 0.6 m (2 ft) wide around the columns to reinforce them. The elevation of the bottom of the web wall was unknown.

Los Gatos Creek is an ephemeral stream (dry most of the time) which drains from the eastern side of the coastal range onto an alluvial fan whose head is approximately 3.2 km (2 mi) upstream of the two bridges. About 548 m (1,800 ft) upstream of the bridges Chino creek (also ephemeral) joins Los Gatos Creek. At the time of construction Chino Creek spread over and infiltrated into its alluvial fan. Some time after construction a channel was constructed connecting the two streams and increasing the drainage area of Los Gatos Creek by about 33 percent.

The Los Gatos Creek channel upstream of the bridge is from 91 to 122 m (300 to 400 ft) wide, but only 46 to 76 m (150 to 250 ft) wide downstream. The 37 m (122 ft) wide bridge severely constricts the channel and the March 10, 1995 flood ponded upstream of the bridge. From 1955 to 1995, differential land subsidence between bench marks approximately 2.4 km (1.5 miles) upstream and 8.5 km (5.3 mi) downstream was measured as 3.5 m (11.5 ft). The bed of the stream is sand and the bedform is plane bed. Discharges are hard to quantify for this stream. For the 1995 flood, the USGS using slope area methods determined that the discharge ranged from 462 to 1141 m^3/s (16,300 to 40,300 cfs) and the most probable discharge was 773 m^3/s (27,300 cfs) with a recurrence interval of 75 years based on historical data.

The factors involved in the I-5 bridge failure were:

- Increase in channel slope by subsidence
- Change in the original design by maintenance adding a web wall between columns to repair damage from an earlier flood. With an angle of attack from 15 to 26 degrees this action potentially increased local pier scour depth by a factor of 3.6 to 4.4
- Increase in drainage area of 33 percent above the bridge by land use change and the construction of a channel to link two streams (Chino Creek to Los Gatos Creek)
- Long-term degradation of 3 m (10 ft) since the bridge was built
- Significant contraction of the flow, i.e., channel width of 91 to 122 m (300 to 400 ft) wide to a bridge width of 37 m (122 ft)

In conclusion, the various factors that contributed to this failure illustrate the complexities of inspection and the need for all elements of a DOT (inspection, maintenance, design and management) to be involved in the process. Inspectors must continually observe the conditions at the bridge, and the stream channel above and below the bridge, and communicate actions, conditions, and changes in the bridge and stream to the different sections of the organization.

11.4.5 Conclusions

These three cases illustrate the difficulty and necessity for inspection of bridges. They also illustrate the need for good communication between DOT inspection, maintenance, design and management. Inspectors must have design or as-built plans on site; must take, plot, and compare cross sections of the channel at the bridge, and they must observe and carefully document the conditions of the bridge and the channel upstream and downstream. Maintenance must inform inspection, design and others when they make changes to a bridge or channel. A "can do" attitude is great but sometimes the consequences can be bad. Communication is very important. Design needs to inform inspection and maintenance of design assumptions and what to look for. Maintenance, because they are the "eyes" of the DOT team, must look for changes and inform others.

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CHAPTER 12

SPECIAL CONSIDERATIONS FOR SCOUR AND STREAM INSTABILITY

12.1 INTRODUCTION

Most bridge owners have now implemented comprehensive programs, inspections and operational procedures to make their bridges less vulnerable to damage or failure from scour. New bridges are designed to resist damage from scour, while existing bridges are inspected regularly and evaluated to determine if a present or potential condition exists that may render the bridge vulnerable to damage during a future flood. When such a condition is found to exist, the bridge is coded as a scour critical bridge, and a plan of action should be developed to address the best way of mitigating the scour problem. Features that make a bridge less vulnerable to damage or failure from scour or stream instability are generally referred to as countermeasures. Countermeasures can be (1) incorporated in the initial design or (2) added after the initial construction.

This chapter outlines special considerations for reducing the risk or making a bridge safe from scour and stream instability. General guidance regarding the use of scour and stream instability countermeasures is provided. Guidance regarding the selection, design and implementation of specific stream instability and scour countermeasures is given in HEC-23.[7] In addition, considerations for evaluating scour in unusual situations, such as scour in cohesive soils or rock, are introduced (with details provided in separate appendices). Cohesive soil and rock can reduce the magnitude of both local scour and general scour at bridge foundations.

12.2 PLAN OF ACTION

A **plan of action** should be developed for each existing bridge found to be scour critical. The two primary components of the plan of action are instructions regarding the type and frequency of inspections to be made at the bridge, and a schedule for the timely design and construction of countermeasures to make a bridge safe from scour and stream stability problems. Depending on the risk, the plan might include development and implementation of a monitoring and/or inspection program, or immediate installation of countermeasures to reduce the risk of failure from scour or stream instability. The plan could include instructions for closure of a bridge, if needed.

HEC-23[7] (Chapter 2) outlines management and inspection strategies that should be considered when developing a plan of action for a scour critical bridge. Issues related to closing and re-opening a bridge are also discussed.

Developing a schedule for the timely design and construction of countermeasures requires defining the preferred countermeasure alternative. It is typical that several different alternatives might be appropriate for a given scour or stream stability problem at a bridge. These alternatives could include hydraulic countermeasures, structural countermeasures or monitoring, either individually or in some combination. To evaluate the engineering feasibility of possible alternatives, conceptual designs and preliminary cost estimates should be prepared. The various alternatives developed should be presented in the plan of action, and a narrative provided describing why the preferred alternative was chosen.

To facilitate selection of alternatives to be considered in the plan of action, a matrix describing the various countermeasures and their attributes has been developed and is presented in HEC-23.[7] HEC-23 also includes general guidance for design of countermeasures, and specific design guidelines for a variety of stream instability and scour countermeasures.

12.3 NEW BRIDGES

For new bridges, the best solutions for minimizing scour damage include:

1. Locating bridges to avoid adverse flood flow patterns

2. Streamlining bridge elements to minimize obstructions to the flow

3. Designing foundations to resist scour, using the guidance in Chapters 2 through 10

4. Designing bridge pier foundations to resist scour without relying on the use of riprap or other countermeasures

5. Designing abutment foundations on piles or on rock, where practicable; for spread footings on soil, placing the footing deep enough to minimize the scour hazard; or protecting the abutment by well designed riprap and/or other suitable countermeasures

6. Incorporating measures to control stream instability (guidebanks, spurs, check dams, etc.) as a part of the initial construction when the potential exists for significant lateral movement or degradation of the channel (see HEC-23)[7]

7. Providing as-built plans (depicting bridge layout, foundations, pile tip elevations, etc.), bridge soils and scour reports and other documented hydrologic and hydraulic design information in a permanent file for the use of bridge maintenance and inspection units. Most DOTS include this information as a part of the permanent bridge plans. The information on design assumptions and site conditions can serve as base line data to evaluate future changes in a river channel and to determine if the changes could affect the safety of the bridge (See examples given in Section 11.4).

12.4 EXISTING BRIDGES

For existing bridges, some of the countermeasures available for protecting the bridge from scour and stream instability are listed below in a rough order of cost (see HEC-23[7] for selection and design guidance):

1. Bridge inspection and scour monitoring programs; closing bridges when necessary

2. Providing riprap at piers and monitoring

3. Providing riprap at abutments and monitoring

4. Constructing guide banks (spur dikes)

5. Constructing river training countermeasures and channel improvements

6. Strengthening the bridge foundations

7. Constructing sills or drop structures (check dams)

8. Constructing relief bridges or lengthening existing bridges

12.5 INSPECTING AND MONITORING BRIDGES FOR SCOUR

Periodic inspections of all bridges serve as the foundation for the bridge owner's management plan to assure the public safety. This includes underwater inspection of foundations located in deep water. Underwater inspection is required when the bridge foundations cannot be visibly inspected by wading.[96] A river and its floodplain are constantly changing, whereas the bridge and its foundation are fixed. A measuring system is necessary to track the lateral and vertical movement of the channel bed over time. The measurements will serve to help in the determination of whether changes are random and within acceptable tolerances, or whether definite trends are occurring which may threaten the stability of the bridge (see Chapter 11).

Gradual river changes are common. As a consequence, the engineer may wait too long to take action. As the degree of encroachment and scour hazard increases, the number of alternative countermeasures available decreases, and costs of correction are correspondingly increased. Threshold values for vertical and horizontal river bed changes should be provided to the inspector. The bridge inspector should report immediately in a special report, as well as the routine inspection report, when changes exceed the threshold values.

Special attention should be given to the condition of scour critical bridges during these periodic inspections. Further, special scour monitoring efforts should be put into effect as necessary to assure that these bridges remain stable. There is a wide range of monitoring procedures which can be used, depending on the condition of the scour critical bridge. The plan of action prepared for each scour critical bridge will serve as the basis for (1) selecting the appropriate monitoring procedures and (2) providing special instructions to the bridge inspector regarding the procedures. Monitoring may include:

- Increasing the frequency and intensity of bridge inspections, using portable scour measuring devices where necessary to check scour critical bridge elements

- Stationing inspectors at the bridge during and immediately after flood events, and providing them with portable equipment to measure scour depths

- Installing permanent scour monitoring equipment at bridge piers and abutments (see HEC-23,[7] Chapter 7)

- Preparing geotechnical stability analyses of bridge piers or abutments to determine the scour depth at which the bridge becomes unstable and should be closed

- Closing the bridge to traffic when conditions become unsafe

The plan of action for a bridge should include special instructions to the bridge inspector, as to when a bridge should be closed to traffic. Guidance should also be given to DOT and other State officials on bridge closures. Contingency plans should be prepared in advance of any bridge closure so that rerouting of traffic can be handled in an orderly fashion.

12.6 COUNTERMEASURES TO REDUCE THE RISK

There are a number of scour critical bridges for which the installation of countermeasures to reduce the risk from scour represents the most practical and cost effective solution. Typical examples of these measures which could reduce, but not eliminate, the scour threat include:

- Placement of riprap around exposed foundations (see Appendix J for guidance)
- Use of grout bags and grout to underpin footings that have been undermined (see HEC-23(7) design guidelines)
- Installation of bendway weirs or spurs at a bend that is migrating towards a bridge abutment so as to redirect the flow away from the abutment (see HEC-23(7) design guidelines)
- Placement of guide banks to move scour away from the abutment foundation

Such countermeasures, if properly installed, may serve successfully for many years in protecting the bridge. While they reduce the risk from scour, they may be subject to failure over an extended period of time or even during a single flood event. They need to be carefully checked during routine inspections and after flood events, especially when used at scour critical bridges.

Installing a scour countermeasure to reduce the risk can serve effectively at bridges where it is not practical or economically justified to undertake repairs to make the bridge safe from scour or to replace the bridge. Examples include:

- Bridge that has only a few years of service life remaining before it is scheduled for replacement
- Small bridges with limited under clearances where it is difficult to install measures to make the bridge safe
- Structures on low volume roads where the risks to the public from a bridge failure are minimal

12.7 COUNTERMEASURES TO MAKE A BRIDGE SAFE FROM SCOUR

Countermeasures to make a bridge safe from scour are distinguished from countermeasures to reduce the risk primarily by the scope of the work involved in their design, installation, and cost. Typically, such countermeasures will be designed on the basis of a hydrologic and hydraulic study of the river to withstand scour associated with a design flood (for scour) and a check flood (for scour). Measures to make a bridge safe from scour include structural changes to the foundations of the bridge. They may also include riprap revetments when designed in accordance with appropriate hydrologic and hydraulic criteria as set forth in HEC-23.(7)

12.8 SCHEDULING CONSTRUCTION OF SCOUR COUNTERMEASURES

It is important for the bridge owner to develop realistic schedules for the installation of scour countermeasures. Lead-time must be provided for the design of the countermeasure and for obtaining necessary permits. Regulatory agencies will usually appreciate the need for emergency work to keep a bridge from failing, and will cooperate in expediting approval of the work (see HEC-23,[7] Chapter 4). However, they are understandably reluctant to consider every scour countermeasure project as emergency work. Coordination with the regulatory agency personnel on a regular basis is needed to assure that the designs for scour countermeasures are prepared in accordance with regulatory requirements. If the installation of a scour countermeasure will require special design procedures that are not in keeping with the normal permit requirements, then this issue needs to be discussed early on in one of the coordination meetings.

The scheduling of scour countermeasure projects should be based on the relative priorities of competing projects. In turn, these priorities should be based, primarily, on the perceived risk to the safety of the persons who travel on the affected highways.

12.9 SCOUR IN COHESIVE SOILS

The maximum depth of local scour at piers in cohesive soils is the same as in non-cohesive soils.[103,104,105] Time is the difference. Maximum scour depth is reached in hours or one runoff event in non-cohesive sand, but may take days and many runoff events in cohesive clays. Local pier scour in cohesive clays may be 1,000 times slower than non-cohesive sand.[103] In addition, by inference, contraction scour and local scour at abutments in cohesive soils do not reach maximum depth as rapidly, but the ultimate scour depth will be the same as for non-cohesive soil.

The equations and methodologies presented in previous chapters, which predict the maximum scour depth in non-cohesive soil, may, in some circumstance be too conservative. The pier scour equation represents an envelope curve of the deepest scour observed during the various laboratory studies and field data. There is much merit in using a conservative approach, taking into consideration the wide range of soil characteristics, the intricate interactions between soil and water, and the uncertainties inherent in predicting flood flows and their flow patterns through the bridge over its service life. When applied with engineering judgment, this conservative approach is usually reasonable and cost effective.

On the other hand, there are site conditions and bridges where an alternative method for scour evaluation would be appropriate. Examples include bridge foundations on highly scour-resistant cohesive soils where the useful life of the bridge is short in relation to the expected number of scouring floods and rate of scour in cohesive soils, bridges scheduled to be replaced in a couple of years, or bridges on low traffic volume roads which are monitored. Significant savings can be achieved for bridges under these conditions, when the characteristics of the cohesive soils to resist scour are taken into account in the design of the foundation. Consequently, guidelines and a technique for evaluating scour in cohesive soils, based on recent research,[103,104] are presented in Appendix L.

12.10 SCOUR IN ROCK

As noted, the equations and methods given in previous chapters are for determining scour depths for the design of bridge foundations in non-cohesive soils. In Chapter 2, recommendations are given for bridge foundations on rock highly resistant to scour. **The problem is determining if rock is resistant to scour**. The determination if the bridge foundations are founded in scour resistance rock and the design of foundations in rock require the expertise of geologist and geotechnical engineers. In addition to standard geologic and geotechnical tests, core or block samples can be taken and subjected to flume studies. The Erosion Function Apparatus (EFA), described in the Appendix L, or a simply constructed or available flume can be used to determine the scourability of the rock material. In Appendix M, four recommendations are given for determining if rock formations are scour resistant; however, additional research is needed in this area.

12.11 OTHER LITERATURE ON SCOUR

Additional information and guidance on stream stability and scour at bridges can be found in several recent publications on these topics. These include a scour manual on European practice from the Netherlands,[93] a book on bridge scour which summarizes the present state of knowledge and practice in New Zealand,[106] and a compendium of papers collected from American Society of Engineers (ASCE) water resources conferences which summarizes research and practice, primarily in the United States, from 1991 to 1998.[107] Highlights of the contents of these publications are indicated in the following paragraphs.

The purpose of the Dutch scour manual[93] is to provide the civil engineer with practical methods to calculate the dimensions of scour holes and to furnish an introduction to the most relevant literature. The manual contains guidelines which can be used to solve problems related to scour in engineering practice and also reflects the results of research projects on the phenomena of scour which have been conducted in the Netherlands during the last several decades.

The manual summarizes and extends the theoretical work of Breusers and Raudkivi, and suggests that the Breusers equilibrium method can be applied directly in engineering practice for all situations where local scour is expected and for nearly all types of structures. Highlights of the manual include:

- Basic concepts
- Sills and jets
- Abutments and spur dikes
- Bridge piers
- Coastal and offshore structures
- Case studies

The New Zealand book on bridge scour covers the description and analysis of scour at bridge foundations. The central focus is the combination of old and new design methods into a comprehensive methodology for bridge-scour design. The book is based upon an extensive summary of existing research results and design experience and it is intended to serve as both a handy reference text and a manual for the practicing bridge designer. A unique aspect of the book is its presentation of thirty-one detailed case studies of scour-induced bridge failure to provide designers with an understanding of processes involved and cases against which design methodologies can be tested. Highlights of the book include:

- New Zealand case histories of bridge scour damage
- Data requirements and basic engineering analyses
- General scour including bend scour and confluence scour
- Contraction and local scour
- Design method for total scour
- Applications and scour countermeasures

The ASCE Compendium contains all the abstracts of the stream stability and scour papers from the proceedings of the Hydraulics Division of the American Society of Civil Engineers annual conferences from 1991 to 1998. Most of the abstracts are from sessions sponsored by the Hydraulic Division's Sedimentation Committee Task Committee on "Bridge Scour Evaluation." In addition, selected authors were invited to write an extended or updated paper on the subject of their original paper. These 75 new papers are included in the Compendium. The abstracts and papers are assembled into the following topics:

- U.S. national bridge scour evaluation program
- Stream stability and geomorphology
- Local scour at bridge piers and abutments
- Contraction scour
- Instrumentation for measuring and monitoring scour
- Field measurements of bridge scour
- Computer and physical modeling of bridge scour
- Bridge scour in tidal waterways
- Countermeasures for stream instability and bridge scour
- Economics and risk analysis of bridge scour
- Research needs

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CHAPTER 13

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APPENDIX A

METRIC SYSTEM, CONVERSION FACTORS, AND WATER PROPERTIES

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APPENDIX A

Metric System, Conversion Factors, and Water Properties

The following information is summarized from the Federal Highway Administration, National Highway Institute (NHI) Course No. 12301, "Metric (SI) Training for Highway Agencies." For additional information, refer to the Participant Notebook for NHI Course No. 12301.

In SI there are seven base units, many derived units and two supplemental units (Table A.1). Base units uniquely describe a property requiring measurement. One of the most common units in civil engineering is length, with a base unit of meters in SI. Decimal multiples of meter include the kilometer (1000m), the centimeter (1m/100) and the millimeter (1 m/1000). The second base unit relevant to highway applications is the kilogram, a measure of mass which is the inertial of an object. There is a subtle difference between mass and weight. In SI, mass is a base unit, while weight is a derived quantity related to mass and the acceleration of gravity, sometimes referred to as the force of gravity. In SI the unit of mass is the kilogram and the unit of weight/force is the newton. Table A.2 illustrates the relationship of mass and weight. The unit of time is the same in SI as in the English system (seconds). The measurement of temperature is Centigrade. The following equation converts Fahrenheit temperatures to Centigrade, $^{\circ}C = 5/9\ (^{\circ}F - 32)$.

Derived units are formed by combining base units to express other characteristics. Common derived units in highway drainage engineering include area, volume, velocity, and density. Some derived units have special names (Table A.3).

Table A.4 provides useful conversion factors from English to SI units. The symbols used in this table for metric units, including the use of upper and lower case (e.g., kilometer is "km" and a newton is "N") are the standards that should be followed. Table A.5 provides the standard SI prefixes and their definitions.

Table A.6 provides physical properties of water at atmospheric pressure in SI system of units. Table A.7 gives the sediment grade scale and Table A.8 gives some common equivalent hydraulic units.

Table A.1. Overview of SI Units.		
	Units	Symbol
Base units		
length	meter	m
mass	kilogram	kg
time	second	s
temperature*	kelvin	K
electrical current	ampere	A
luminous intensity	candela	cd
amount of material	mole	mol
Derived units		
Supplementary units		
angles in the plane	radian	rad
solid angles	steradian	sr
*Use degrees Celsius (°C), which has a more common usage than kelvin.		

Table A.2. Relationship of Mass and Weight.			
	Mass	Weight or Force of Gravity	Force
English	slug pound-mass	pound pound-force	pound pound-force
metric	kilogram	newton	newton

Table A.3. Derived Units With Special Names.			
Quantity	Name	Symbol	Expression
Frequency	hertz	Hz	s^{-1}
Force	newton	N	$kg \cdot m/s^2$
Pressure, stress	pascal	Pa	N/m^2
Energy, work, quantity of heat	joule	J	$N \cdot m$
Power, radiant flux	watt	W	J/s
Electric charge, quantity	coulomb	C	$A \cdot s$
Electric potential	volt	V	W/A
Capacitance	farad	F	C/V
Electric resistance	ohm	Ω	V/A
Electric conductance	siemens	S	A/V
Magnetic flux	weber	Wb	$V \cdot s$
Magnetic flux density	tesla	T	Wb/m^2
Inductance	henry	H	Wb/A
Luminous flux	lumen	lm	$cd \cdot sr$
Illuminance	lux	lx	lm/m^2

Table A.4. Useful Conversion Factors.			
Quantity	From English Units	To Metric Units	Multiplied By*
Length	mile yard foot inch	km m m mm	1.609 0.9144 0.3048 25.40
Area	square mile acre acre square yard square foot square inch	km^2 m^2 hectare m^2 m^2 mm^2	2.590 4047 0.4047 0.8361 0.09290 645.2
Volume	acre foot cubic yard cubic foot cubic foot 100 board feet gallon cubic inch	m^3 m^3 m^3 L (1000 cm^3) m^3 L (1000 cm^3) cm^3	1233 0.7646 0.02832 28.32 0.2360 3.785 16.39
Mass	lb kip (1000 lb)	kg metric ton (1000 kg)	0.4536 0.4536
Mass/unit length	plf	kg/m	1.488
Mass/unit area	psf	kg/m^2	4.882
Mass density	pcf	kg/m^3	16.02
Force	lb kip	N kN	4.448 4.448
Force/unit length	plf klf	N/m kN/m	14.59 14.59
Pressure, stress, modulus of elasticity	psf ksf psi ksi	Pa kPa kPa MPa	47.88 47.88 6.895 6.895
Bending moment, torque, moment of force	ft-lb ft-kip	N · m kN · m	1.356 1.356
Moment of mass	lb · ft	m	0.1383
Moment of inertia	lb · ft^2	kg · m^2	0.04214
Second moment of area	in^4	mm^4	416200
Section modulus	in^3	mm^3	16390
Power	ton (refrig) Btu/s hp (electric) Btu/h	kW kW W W	3.517 1.054 745.7 0.2931
*4 significant figures; underline denotes exact conversion			

Table A.4. Useful Conversion Factors (continued).			
Quantity	From English Units	To Metric Units	Multiplied by*
Volume rate of flow	ft^3/s cfm cfm mgd	m^3/s m^3/s L/s m^3/s	0.02832 0.0004719 0.4719 0.0438
Velocity, speed	ft/s	m/s	<u>0.3048</u>
Acceleration	f/s^2	m/s^2	<u>0.3048</u>
Momentum	lb · ft/sec	kg · m/s	0.1383
Angular momentum	lb · ft^2/s	kg · m^2/s	0.04214
Plane angle	degree	rad mrad	0.01745 17.45
*4 significant figures; underline denotes exact conversion			

Table A.5. Prefixes.					
Submultiples			Multiples		
deci	10^{-1}	d	deka	10^{1}	da
centi	10^{-2}	c	hecto	10^{2}	h
milli	10^{-3}	m	kilo	10^{3}	k
micro	10^{-6}	μ	mega	10^{6}	M
nano	10^{-9}	n	giga	10^{9}	G
pica	10^{-12}	p	tera	10^{12}	T
femto	10^{-15}	f	peta	10^{15}	P
atto	10^{-18}	a	exa	10^{18}	E
zepto	10^{-21}	z	zetta	10^{21}	Z
yocto	10^{-24}	y	yotto	10^{24}	Y

Table A.6. Physical Properties of Water at Atmospheric Pressure in SI Units.

Temperature		Density	Specific Weight	Dynamic Viscosity	Kinematic Viscosity	Vapor Pressure	Surface Tension[1]	Bulk Modulus
Centigrade	Fahrenheit	kg/m^3	N/m^3	$N \cdot s/m^2$	m^2/s	N/m^2 abs.	N/m	GN/m^2
0°	32°	1,000	9,810	1.79×10^{-3}	1.79×10^{-6}	611	0.0756	1.99
5°	41°	1,000	9,810	1.51×10^{-3}	1.51×10^{-6}	872	0.0749	2.05
10°	50°	1,000	9,810	1.31×10^{-3}	1.31×10^{-6}	1,230	0.0742	2.11
15°	59°	999	9,800	1.14×10^{-3}	1.14×10^{-6}	1,700	0.0735	2.16
20°	68°	998	9,790	1.00×10^{-3}	1.00×10^{-6}	2,340	0.0728	2.20
25°	77°	997	9,781	8.91×10^{-4}	8.94×10^{-7}	3,170	0.0720	2.23
30°	86°	996	9,771	7.97×10^{-4}	8.00×10^{-7}	4,250	0.0712	2.25
35°	95°	994	9,751	7.20×10^{-4}	7.24×10^{-7}	5,630	0.0704	2.27
40°	104°	992	9,732	6.53×10^{-4}	6.58×10^{-7}	7,380	0.0696	2.28
50°	122°	988	9,693	5.47×10^{-4}	5.53×10^{-7}	12,300	0.0679	
60°	140°	983	9,643	4.66×10^{-4}	4.74×10^{-7}	20,000	0.0662	
70°	158°	978	9,594	4.04×10^{-4}	4.13×10^{-7}	31,200	0.0644	
80°	176°	972	9,535	3.54×10^{-4}	3.64×10^{-7}	47,400	0.0626	
90°	194°	965	9,467	3.15×10^{-4}	3.26×10^{-7}	70,100	0.0607	
100°	212°	958	9,398	2.82×10^{-4}	2.94×10^{-7}	101,300	0.0589	

[1]Surface tension of water in contact with air

Table A.7. Physical Properties of Water at Atmospheric Pressure in English Units.

Temperature		Density	Specific Weight	Dynamic Viscosity	Kinematic Viscosity	Vapor Pressure	Surface Tension[1]	Bulk Modulus
Fahrenheit	Centigrade	Slugs/ft^3	Weight lb/ft^3	lb-sec/ft^2	ft^2/sec	lb/in^2	lb/ft	lb/in^2
32	0	1.940	62.416	0.374×10^{-4}	1.93×10^{-5}	0.09	0.00518	287,000
39.2	4.0	1.940	62.424					
40	4.4	1.940	62.423	0.323	1.67	0.12	.00514	296,000
50	10.0	1.940	62.408	0.273	1.41	0.18	.00508	305,000
60	15.6	1.939	62.366	0.235	1.21	0.26	.00504	313,000
70	21.1	1.936	62.300	0.205	1.06	0.36	.00497	319,000
80	26.7	1.934	62.217	0.180	0.929	0.51	.00492	325,000
90	32.2	1.931	62.118	0.160	0.828	0.70	.00486	329,000
100	37.8	1.927	61.998	0.143	0.741	0.95	.00479	331,000
120	48.9	1.918	61.719	0.117	0.610	1.69	.00466	332,000
140	60.0	1.908	61.386	0.0979	0.513	2.89		
160	71.1	1.896	61.006	0.0835	0.440	4.74		
180	82,2	1.883	60.586	0.0726	0.385	7.51		
200	93.3	1.869	60.135	0.0637	0.341	11.52		
212	100	1.847	59.843	0.0593	0.319	14.70		

[1]Surface tension of water in contact with air

Table A.8. Sediment Particles Grade Scale.

Size				Approximate Sieve Mesh Openings Per Inch		Class
Millimeters		Microns	Inches	Tyler	U.S. Standard	
4000-2000	-----	-----	160-80	-----	-----	Very large boulders
2000-1000	-----	-----	80-40	-----	-----	Large boulders
1000-500	-----	-----	40-20	-----	-----	Medium boulders
500-250	-----	-----	20-10	-----	-----	Small boulders
250-130	-----	-----	10-5	-----	-----	Large cobbles
130-64	-----	-----	5-2.5	-----	-----	Small cobbles
64-32	-----	-----	2.5-1.3	-----	-----	Very coarse gravel
32-16	-----	-----	1.3-0.6	-----	-----	Coarse gravel
16-8	-----	-----	0.6-0.3	2 1/2	-----	Medium gravel
8-4	-----	-----	0.3-0.16	5	5	Fine gravel
4-2	-----	-----	0.16-0.08	9	10	Very fine gravel
2-1	2.00-1.00	2000-1000	-----	16	18	Very coarse sand
1-1/2	1.00-0.50	1000-500	-----	32	35	Coarse sand
1/2-1/4	0.50-0.25	500-250	-----	60	60	Medium sand
1/4-1/8	0.25-0.125	250-125	-----	115	120	Fine sand
1/8-1/16	0.125-0.062	125-62	-----	250	230	Very fine sand
1/16-1/32	0.062-0.031	62-31	-----	-----	-----	Coarse silt
1/32-1/64	0.031-0.016	31-16	-----	-----	-----	Medium silt
1/64-1/128	0.016-0.008	16-8	-----	-----	-----	Fine silt
1/128-1/256	0.008-0.004	8-4	-----	-----	-----	Very fine silt
1/256-1/512	0.004-0.0020	4-2	-----	-----	-----	Coarse clay
1/512-1/1024	0.0020-0.0010	2-1	-----	-----	-----	Medium clay
1/1024-1/2048	0.0010-0.0005	1-0.5	-----	-----	-----	Fine clay
1/2048-1/4096	0.0005-0.0002	0.5-0.24	-----	-----	-----	Very fine clay

Table A.9. Common Equivalent Hydraulic Units.

Volume

Unit	Equivalent							
	cubic inch	liter	u.s. gallon	cubic foot	cubic yard	cubic meter	acre-foot	sec-foot-day
liter	61.02	1	0.264 2	0.035 31	0.001 308	0.001	810.6 E - 9	408.7 E - 9
U.S. gallon	231.0	3.785	1	0.133 7	0.004 951	0.003 785	3.068 E - 6	1.547 E - 6
cubic foot	1728	28.32	7.481	1	0.037 04	0.028 32	22.96 E - 6	11.57 E - 6
cubic yard	46 660	764.6	202.0	27	1	0.746 6	619.8 E - 6	312.5 E - 6
$meter^3$	61 020	1000	264.2	35.31	1.308	1	810.6 E - 6	408.7 E - 6
acre-foot	75.27 E + 6	1 233 000	325 900	43 560	1 613	1 233	1	0.504 2
sec-foot-day	149.3 E + 6	2 447 000	646 400	86 400	3 200	2 447	1.983	1

Discharge (Flow Rate, Volume/Time)

Unit	Equivalent					
	gallon/min	liter/sec	acre-foot/day	$foot^3$/sec	million gal/day	$meter^3$/sec
gallon/minute	1	0.063 09	0.004 419	0.002 228	0.001 440	63.09 E - 6
liter/second	15.85	1	0.070 05	0.035 31	0.022 82	0.001
acre-foot/day	226.3	14.28	1	0.504 2	0.325 9	0.014 28
$feet^3$/second	448.8	28.32	1.983	1	0.646 3	0.028 32
million gal/day	694.4	43.81	3.068	1.547	1	0.043 82
$meter^3$/second	15 850	1000	70.04	35.31	22.82	1

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APPENDIX B

EXTREME EVENTS

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APPENDIX B

EXTREME EVENTS

B.1 INTRODUCTION

In 1994, AASHTO introduced an entirely new set of specifications based on the concept of load and resistance factor design (LRFD) methodology. The factors were developed from the theory of reliability based upon current statistical knowledge of loads and structural performance. In the evaluation of scour at bridge structures, there are two conditions, or limit states, that are of primary interest in design:

1. Service Limit States, or limit states relating to stress, deformation and cracking

2. Strength Limit States, or limit states relating to strength and stability

The design flood for scour is used in the evaluation of these limit states.

The Extreme-Event Limit States relate to events with return periods in excess of the design life of the bridge. There are generally three such limit states that may involve consideration of the effect of scour at bridges:

1. A flood event exceeding a 100-year flood (The check flood for scour or superflood is used to evaluate scour for this event as described in Chapter 2, a 500-year flood is recommended for the check flood for scour)

2. An earthquake

3. A vessel collision with the bridge

In addition to the above, there are other conditions possibly relating to scour that the designer may determine are significant for a specific watershed, such as ice loads or debris from logging operations, etc.

Events 2 and 3, above, are related to scour with regard to the possibility that they could occur at the same time that a flood event is occurring. The loss of foundation support due to scour could then impact on the stability of the foundation in resisting the earthquake or vessel collision forces. Recommendations for the consideration of the joint-probability of one of these events with a flood event are discussed below.

B.2 CHANGES IN FOUNDATIONS DUE TO LIMIT STATE FOR SCOUR

The consequences of changes in foundation conditions resulting from the design flood for scour shall be considered at strength and service limit states in accordance with the standards set forth in the AASHTO LRFD Specifications.[1]

The consequences of changes in foundation conditions due to scour resulting from the check flood for bridge scour and from hurricanes shall be considered at the extreme event limit state.

Scour is not a force effect, but by changing the conditions of the substructure it may have a significant effect in altering the force effects acting on structures. The AASHTO LRFD Specifications, Section 3, sets forth detailed requirements for applying loads and load factors to bridge foundations. The extreme event limit states and the loads to be applied for these limit states are explained in this section.

The strength and service limit states are used in the design of a bridge foundation. Structures designed to resist damage from scour will be designed under this provision using normal design considerations and factors of safety selected by the foundation engineer. The assumption is made that all material in the scour prism has been removed and is unavailable for foundation support.

Scour shall be considered in extreme event load combinations as outlined below:

<u>Extreme Event I - Load combination including earthquake</u>

This extreme event limit state includes water loads and earthquakes. The probability of a major flood and an earthquake occurring at the same time is very small. Therefore, consideration of basing water loads and scour depths on mean discharges may be warranted (when considering the joint probability of an earthquake and scour). Mean discharges are considered to be normal (non-flood) flows representing the typical or daily flows in the river.

<u>Extreme Event II - Load combination related to ice load, collision by vessels and vehicles, and certain hydraulic loads with a reduced live load other than that which is a part of the vehicular collision load</u>

This extreme event limit state is a load combination for extreme events such as ice loads, collision by vessels and vehicles, and the check flood for scour. Its application for the check flood for scour involves a reduced live load on the structure of 50 percent. The assumption is made that all material in the scour prism has been removed and is unavailable for foundation support. The structure is to remain stable for this condition, but is not required to have any reserve capacity to resist loads.

The recurrence interval of these extreme events is expected to exceed the design life of the bridge. The joint probability of these events is extremely low, and, therefore, the events are specified to be applied separately.

The Engineer is cautioned to consider the following when applying the above noted AASHTO specifications to the evaluation of the joint probability of a flood and another extreme event. These considerations incorporate recommendations from some of the papers presented at a conference on "The Design of Bridges for Extreme Events" sponsored by the Federal Highway Administration in December 1996.(2)

- There are several current studies underway to evaluate the joint probability of extreme events. Until further and more definitive conclusions are drawn from these studies, judgment is necessary in evaluating site-specific factors on a case by case basis that could affect the safety of the traveling public.

- A differentiation must be made between long-term scour (degradation) and short-term scour (local scour and general (contraction) scour). It is reasonable to consider expected long-term degradation in evaluating the joint probability of occurrence of scour with an earthquake or vessel collision event since it is associated with a period of many years.

On the other hand, live-bed local scour and contraction scour may occur only for a period of hours or days before the scour hole refills; consequently, the joint probability of this type of scour with an earthquake or vessel collision is very low. In some cases, clear-water scour holes may occur and not refill or refill very slowly. While the joint probability of the occurrence of a 100-year flood/clear-water scour hole and another extreme event is very low, the engineer may wish to consider a clear-water scour hole associated with a lesser flood event.

- The probability of the simultaneous occurrence of an extreme vessel collision load (by a ship or barge transiting the navigable channel at normal operating speeds) and short-term scour resulting from a 100-year flood is very low and can be neglected as a load combination. The probability of the simultaneous occurrence of a vessel collision load from a single (empty) hopper barge floating in the waterway at the speed of the current and both long- and short-term scour is valid and should be considered in the design where applicable.

B.3 REFERENCES

1. American Association of State Highway and Transportation Officials, 1994, "LRFD Bridge Design Specifications and Commentary," First Edition, Washington, D.C.

2. Federal Highway Administration, 1996, "The Design of Bridges for Extreme Events," Conference Proceedings," Washington, D.C.

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APPENDIX C

CONTRACTION SCOUR AND CRITICAL VELOCITY EQUATIONS

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APPENDIX C

Contraction Scour and Critical Velocity Equations

C.1 CONTRACTION SCOUR

Contraction scour occurs when the flow area of a stream at flood stage is reduced, either by a natural contraction or bridge. It also occurs when overbank flow is forced back to the channel by roadway embankments at the approaches to a bridge. From continuity, a decrease in flow area results in an increase in average velocity and bed shear stress through the contraction. Hence, there is an increase in erosive forces in the contraction and more bed material is removed from the contracted reach than is transported into the reach. This increase in transport of bed material from the reach lowers the natural bed elevation. As the bed elevation is lowered, the flow area increases and, in the riverine situation, the velocity and shear stress decrease until relative equilibrium is reached; i.e., the quantity of bed material that is transported into the reach is equal to that removed from the reach, or the bed shear stress is decreased to a value such that no sediment is transported out of the reach.

In coastal waterways which are affected by tides, as the cross-sectional area increases the discharge from the ocean may increase and thus the velocity and shear stress may not decrease. Consequently, relative equilibrium may not be reached. Thus, at tidal inlets contraction scour may result in a continual lowering of the bed (long-term degradation).

Live-bed contraction scour is typically cyclic; for example, the bed scours during the rising stage of a runoff event and fills on the falling stage. The contraction of flow due to a bridge can be caused by either a natural decrease in flow area of the stream channel or by abutments projecting into the channel and/or piers blocking a portion of the flow area. Contraction can also be caused by the approaches to a bridge cutting off floodplain flow. This can cause clear-water scour on a setback portion of a bridge section or a relief bridge because the floodplain flow does not normally transport significant concentrations of bed material sediments. This clear-water picks up additional sediment from the bed in the bridge opening. In addition, local scour at abutments may well be greater due to the clear-water floodplain flow returning to the main channel at the end of the abutment.

Other factors that can cause contraction scour are (1) natural stream constrictions, (2) long highway approaches to the bridge over the floodplain, (3) ice formations or jams, (4) natural berms along the banks due to sediment deposits, (5) debris, (6) vegetative growth in the channel or floodplain, and (7) pressure flow.

<u>Contraction Scour Equations</u>. There are two forms of contraction scour depending upon the competence of the uncontracted approach flow to transport bed material into the contraction.

Live-bed scour occurs when there is streambed sediment being transported into the contracted section from upstream. In this case, the scour hole reaches equilibrium when the transport of 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 material sediment transport in the uncontracted approach flow is negligible or the material being transported in the upstream reach is transported through the downstream reach at less than the capacity of the flow. In this case, the scour hole reaches equilibrium when the average bed shear stress is less than that required for incipient motion of the bed material.

Contraction scour equations are based on the principle of conservation of sediment transport (continuity). As scour develops, the shear stress in the contracted section decreases as a result of a larger flow area and decreasing average velocity. For **live-bed** scour, maximum scour occurs when the shear stress reduces to the point that sediment transported in equals the bed sediment transported out and the conditions for sediment continuity are in balance. For **clear-water** scour, the transport into the contracted section is essentially zero and maximum scour occurs when the shear stress reduces to the critical shear stress of the bed material in the bridge cross-section.

C.2 LIVE-BED CONTRACTION SCOUR EQUATION

Live-bed contraction scour occurs at a bridge when there is transport of bed material in the upstream reach into the bridge cross section. With live-bed contraction scour the area of the contracted section increases until, in the limit, the transport of sediment out of the contracted section equals the sediment transported in. Normally, the width of the contracted section is constrained and depth increases until the limiting conditions are reached.

Laursen derived the following live-bed contraction scour equation based on a simplified transport function, transport of sediment in uniform flow upstream and downstream of a long contraction, and other simplifying assumptions.[1]

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \left(\frac{n_2}{n_1}\right)^{k_2} \qquad \text{(C.1)}$$

$$y_s = y_2 - y_o = \text{(Average scour depth, m)} \qquad \text{(C.2)}$$

where:

y_1	=	Average depth in the upstream main channel, m
y_2	=	Average depth in the contracted section, m
y_o	=	Existing depth in the contracted section before scour, m
Q_1	=	Flow in the upstream channel transporting sediment, m^3/s
Q_2	=	Flow in the contracted channel, m^3/s. Often this is equal to the total discharge unless the total flood flow is reduced by relief bridges, water overtopping the approach roadway, or in the setback area
W_1	=	Bottom width of the upstream main channel, m
W_2	=	Bottom width of main channel in the contracted section, m
n_1	=	Manning's n for upstream main channel
n_2	=	Manning's n for contracted section
k_1 & k_2	=	Exponents determined below depending on the mode of bed material transport

V_*/ω	k_1	k_2	Mode of Bed Material Transport
<0.50	0.59	0.066	Mostly contact bed material discharge
0.50 to 2.0	0.64	0.21	Some suspended bed material discharge
>2.0	0.69	0.37	Mostly suspended bed material discharge

V_* = $(gyS_1)^{1/2}$ shear velocity in the upstream section, m/s
ω = Median fall velocity of the bed material based on the D_{50}, m/s (see Figure 3 in Chapter 4)
g = Acceleration of gravity (9.81 m/s^2)
S_1 = Slope of energy grade line of main channel, m/m
D_{50} = Median diameter of the bed material, m

The location of the upstream section for y_1, Q_1, W_1, and n_1 needs to be located with engineering judgment. If WSPRO is used to obtain the values of the quantities, then the upstream channel section is located a distance equal to one bridge opening from the upstream face of the bridge.

C.3 CLEAR-WATER CONTRACTION SCOUR EQUATIONS

Clear-water contraction scour occurs in a bridge opening when (1) there is no bed material transport from the upstream reach into the downstream reach or (2) the material being transported in the upstream reach is transported through the downstream reach mostly in suspension and at less than capacity of the flow. With **clear-water** contraction scour the area of the contracted section increases until, in the limit, the velocity of the flow (V) or the shear stress (τ_o) on the bed is equal to the critical velocity (V_c) or the critical shear stress (τ_c) of a certain particle size (D) in the bed material. Normally, the width (W) of the contracted section is constrained and the depth (y) increases until the limiting conditions are reached.

Following a development given by Laursen[2] equations for determining the clear-water contraction scour in a long contraction were developed in metric units. For equilibrium in the contracted reach:

$$\tau_o = \tau_c \tag{C.3}$$

where:

τ_o = Average bed shear stress, contracted section, Pa (N/m^2)
τ_c = Critical bed shear stress at incipient motion, Pa (N/m^2)

The average bed shear stress using y for the hydraulic radius (R) and Manning's equation to determine the slope (S_f) can be expressed as follows:

$$\tau_o = \gamma\, y\, S_f = \frac{\rho\, g\, n^2\, V^2}{y^{1/3}} \tag{C.4}$$

For noncohesive bed materials and fully developed clear-water contraction scour, the critical shear stress can be determined using Shields relation[2, 3]

$$\tau_c = K_s\,(\rho_s - \rho)\, g\, D \tag{C.5}$$

The bed in a long contraction scours until $\tau_o = \tau_c$ resulting in

$$\frac{\rho\ g\ n^2\ V^2}{y^{1/3}} = K_s\ (\rho_s - \rho)\ g\ D \qquad \text{(C.6)}$$

Solving for the depth (y) in the contracted section gives

$$y = \left[\frac{n^2\ V^2}{K_s\ (S_s - 1)\ D}\right]^3 \qquad \text{(C.7)}$$

In terms of discharge (Q) the depth (y) is

$$y = \left[\frac{n^2\ Q^2}{K_s\ (S_s - 1)\ D\ W^2}\right]^{3/7} \qquad \text{(C.8)}$$

where:

- y = Average equilibrium depth in the contracted section after contraction scour, m
- S_f = Slope of the energy grade line, m/m
- V = Average velocity in the contracted section, m/s
- D = Diameter of smallest nontransportable particle in the bed material, m
- Q = Discharge, m^3/s
- W = Bottom width of contracted section, m
- g = Acceleration of gravity (9.81 m/s^2)
- n = Manning's roughness coefficient
- K_s = Shield's coefficient
- S_s = Specific gravity (2.65 for quartz)
- γ = Unit weight of water (9800 N/m^3)
- ρ = Density of water (1000 kg/m^3)
- ρ_s = Density of sediment (quartz, 2647 kg/m^3)

Equations C.7 and C.8 are the basic equations for the **clear-water** scour depth (y) in a long contraction. Laursen, in English units used a value of 4 for K_s $(\rho_s-\rho)g$ in Equation C.5; D_{50} for the size (D) of the smallest nonmoving particle in the bed material and Strickler's approximation for Manning's n ($n = 0.034\ D_{50}^{1/6}$).[2] Laursen's assumption that $\tau_c = 4\ D_{50}$ with $S_s = 2.65$ is equivalent to assuming a Shields parameter $K_s = 0.039$.

From experiments in flumes and studies in natural rivers with bed material of sand, gravel cobbles, and boulders, Shield's coefficient (K_s) to initiate motion ranges from 0.01 to 0.25 and is a function of particle size, Froude Number, and size distribution.[4, 5, 6, 7, 8, 9] Some typical values for K_s for Fr. < 0.8 and as a function of bed material size are (1) $K_s = 0.047$ for sand (D_{50} from 0.065 to 2.0 mm); (2) $K_s = 0.03$ for median coarse-bed material (2 mm > D_{50} < 40 mm) and (3) $K_s = 0.02$ for coarse-bed material (D_{50} > 40 mm).

In metric units, Strickler's equation for n as given by Laursen is $0.041\ D_{50}^{1/6}$, where D_{50} is in meters. Research discussed in HDS 6[3] recommends the use of the effective mean bed material size (D_m) in place of the D_{50} size for the beginning of motion ($D_m = 1.25\ D_{50}$). Changing D_{50} to D_m in the Strickler's equation gives $n = 0.040\ D_m^{1/6}$. Substituting K_s = 0.039 into Equations C.7 and C.8 gives the following equations for y:

$$y = \left[\frac{V^2}{40\ D_m^{2/3}} \right]^3 \qquad (C.9)$$

$$y = \left[\frac{Q^2}{40\ D_m^{2/3}\ W^2} \right]^{3/7} \qquad (C.10)$$

$$y_s = y - y_o = (\text{average scour depth}) \qquad (C.11)$$

where:

- A = Discharge through contraction, m^3/s
- D_m = Diameter of the bed material (1.25 D_{50}) in the contracted section, m
- W = Bottom width in contraction, m
- y_o = Average existing depth in the contracted section, m

The **clear-water** contraction scour equations assume homogeneous bed materials. However, with clear-water scour in stratified materials, using the layer with the finest D_{50} would result in the most conservative estimate of contraction scour. Alternatively, the clear-water contraction scour equations could be used sequentially for stratified bed materials.

Equations C.8 and C.10 do not give the distribution of the contraction scour in the cross section. In many cases, assuming a uniform contraction scour depth across the opening would not be in error (e.g., short bridges, relief bridges and bridges, with simple cross sections and on straight reaches). However, for wide bridges, bridges on bends, bridges with large overbank flow, or crossings with a large variation in bed material size distribution, the contraction scour depths will not be uniformly distributed across the bridge opening. In these cases, Equations C.7 or C.9 can be used if the distribution of the velocity and/or the bed material is known. The computer program WSPRO uses stream tubes to give the discharge and velocity distribution in the cross section.(10) Using this distribution, Equations C.7 or C.9 can be used to estimate the distribution of the contraction scour depths. Equations C.8 or C.10 are used to determine the average contraction scour depth in the section.

Both the **live-bed** and **clear-water** contraction scour equations are the best that are available and should be regarded as a first level of analysis. If a more detailed analysis is warranted, a sediment transport model like BRI-STARS could be used.(11)

C.4 CRITICAL VELOCITY OF THE BED MATERIAL

The velocity and depth given in Equation C.7 are associated with initiation of motion of the indicated particle size (D). Rearranging Equation C.7 to give the critical velocity (V_c) for beginning of motion of bed material of size D results in

$$V_c = \left[\frac{K_s^{1/2} (S_s - 1)^{1/2}\ D^{1/2}\ y^{1/6}}{n} \right] \qquad (C.11)$$

Using K_s = 0.039, S_s = 2.65, and n = 0.041 $D^{1/6}$

$$V_c = 6.19\ y^{1/6}\ D^{1/3} \tag{C.12}$$

where:

V_c	=	Critical velocity above which bed material of size D and smaller will be transported, m/s
K_s	=	Shields parameter
S_s	=	Specific gravity of the bed material
D	=	Size of bed material, m
y	=	Depth of flow, m
n	=	Manning's roughness coefficient

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APPENDIX D

INTERIM PROCEDURE FOR ESTIMATING PIER SCOUR WITH DEBRIS

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APPENDIX D

Interim Procedure for Estimating Pier Scour with Debris

D.1 ASSUMPTIONS

1. Debris aligns with the flow direction and attaches to the upstream nose of a pier. The width of the accumulation, W, on each side of the pier is normal to the flow direction.

2. The trailing end of a long slender pier does not add significantly to pier scour for that portion of the length beyond 12 pier widths. This is consistent with the current guideline in HEC-18 to cut K_2 at L/a = 12.

3. The effect of the debris in increasing scour depths is taken into account by adding a width, W, to the sides and front of the pier. Engineering judgment and experience is used to determine the width, W.

D.2 SUGGESTED PROCEDURE

1. Use K_1 and K_2 = 1.0

2. Project the debris pile and up to twelve pier widths of the pier length normal to the flow direction as follows:

L' = L or 12(a) (whichever is less)

a_{proj} = 2W+a Cosθ or W+a Cosθ + L' Sin θ (whichever is greater)

3. Use K_1, K_2, K_3, K_4, and a_{proj} in the HEC-18 pier scour equation as follows:

$$\frac{y_s}{y_1} = 2.0(1.0)(1.0)K_3 K_4 \left(\frac{a_{proj}}{y_1}\right)^{0.65} Fr_1^{0.43}$$

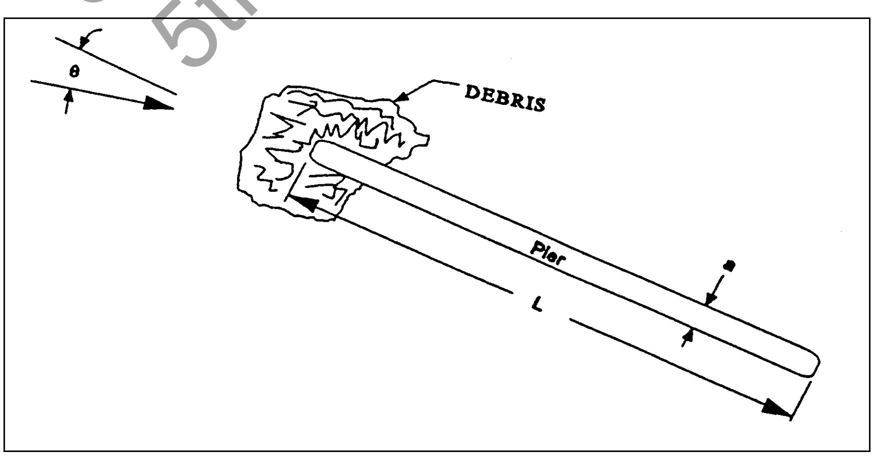

Figure D.1. Schematic for debris procedure.

D3. EXAMPLE PROBLEM (SI)

NVFAS 228 Bridge over the Humbolt River South Fork
Flow: depth, y_1 = 2.42 m; V_1 = 3.60 m/s; Fr_1 = 0.74
Pier: a = 0.46 m; L = 12.62 m; Skew to flow direction = 15 degrees
Debris: Local assumption for accumulation W = 0.61 m extended in front and on each side of pier

Computations:

L/a = 12.62/0.46=27.6>12: use L' = 12 (0.46) = 5.52 m

a_{proj} = 1.22 + 0.46 (Cos 15°) = 1.66 m or
0.61 + 0.46 (Cos 15°) + 5.52 Sin 15° = 2.48 m

$$\frac{y_s}{2.42} = 2.0\ (1.0)\ (1.0)\ (1.1)\ (1.0) \left(\frac{2.48}{2.42}\right)^{0.65} (0.74)^{0.43}$$

$$y_s = 1.98\,(2.42) = 4.79\,\text{m}$$

D.4 EXAMPLE PROBLEM (English)

NVFAS 228 Bridge over the Humboldt River South Fork
Flow: depth, y_1 = 7.9 ft; V_1 = 11.81 ft/s; Fr_1= 0.74
Pier: a = 1.5 ft; L = 41.4 ft; Skew to flow direction = 15 degrees
Debris: Local assumption for accumulation W = 2.0 ft extended in front and on each side of pier

Computations:

L/a = 41.4/1.5 = 27.6>12: use L' = 12 (1.5) = 18 ft

a_{proj} = 4.0 + 1.5 (Cos 15°) = 5.4 ft or
2.0 + 1.5 (Cos 15°) + 18 (Sin 15°) = 8.1 ft

use 8.1 ft

$$\frac{y_s}{7.9} = 2.0\ (1.0)\ (1.0)\ (1.1)\ (1.0) \left(\frac{8.1}{7.9}\right)^{0.65} (0.74)^{0.43}$$

$$y_s = 1.96\ (7.9) = 15.5\ \text{ft}$$

APPENDIX E

STURM ABUTMENT SCOUR EQUATIONS

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APPENDIX E

Sturm Abutment Scour Equations

E.1 INTRODUCTION

Sturm[1,2] utilized a flume with a compound channel to evaluate abutment scour. His research was funded by the National Transportation Board's National Cooperative Highway Research Program (NCHRP). He recognized that scour at abutments setback from the bankline or at the bankline depends on the interaction between main channel flow and the flow obstructed by the abutment. At the interface between the two flows is where vortices and momentum exchange occur which cause scour. Sturm determined that the use of a discharge distribution factor (M) is a better measure of the effect of flow redistribution, vortices and momentum exchange on scour at a bridge abutment than abutment length. From his flume experiments he developed equations and a method for determining scour in compound channels. The prediction method shows a strong correlation between predicted scour and measured scour (Figure E.1). The dashed lines of uncertainty represent a difference of +/- 30 percent from the measured value. No factor of safety was applied to the computed values in Figure E.1.

In the following sections the results of his research are given.

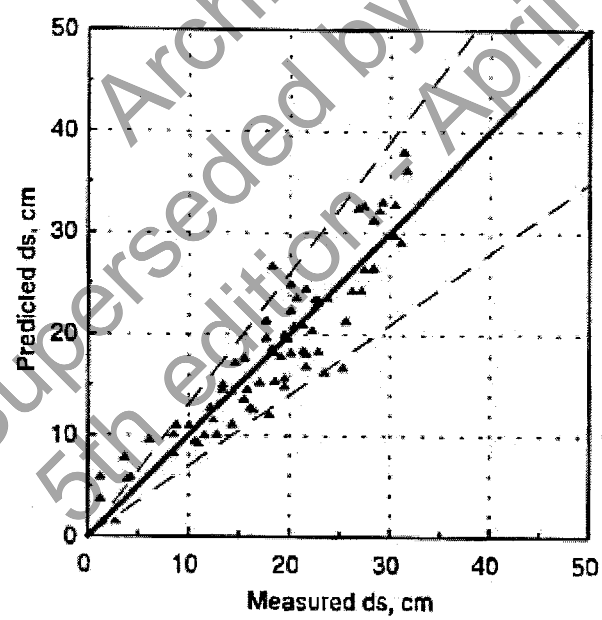

Figure E.1. Comparison of measured and predicted scour depths Sturm Method.[1,2]

E.2 STURM'S EQUATION FOR CLEAR-WATER ABUTMENT SCOUR

Sturm's scour prediction equation for clear-water scour around setback and bankline abutments is:

$$y_s / y_{f0} = 8.14\ K_{st}\ (q_{f1} / MV_{xc}\ y_{f0} - 0.4) + FS \quad (E.1)$$

where:

y_s = Depth of scour at the abutment, m (ft)
y_{f0} = Average depth of flow on the floodplain at the approach section for existing conditions based on normal flow conditions in the river without backwater from the proposed bridge, m (ft)
K_{st} = Sturm's abutment shape factor
q_{f1} = Unit flow rate on the approach floodplain section that will be blocked by the embankment at Section 2. The conditions are based on the proposed structure in place and creating backwater effects at the approach section, m^3/s /m (cfs /ft)
M = Discharge distribution factor as defined below
V_{xc} = Critical velocity at the approach floodplain section for existing conditions based on normal flow conditions in the river without backwater from the proposed bridge, m/s (ft/sec)
FS = Factor of Safety with a recommended value of 1.0

E.3 STURM'S EQUATION FOR LIVE-BED SCOUR AT BANKLINE ABUTMENTS

$$y_s / y_{f0} = 2.0\ K_{st}\ [q_{m1} / (MV_{m0c}\ y_{f0}) - 0.47] + FS \quad (E.2)$$

where:

y_s = Depth of scour at the abutment, m (ft)
y_{f0} = Average depth of flow on the floodplain (see E.4, Step 5), m (ft)
K_{st} = 1.0
q_{m1} = Unit flow rate in the main channel at the approach Section 1 for the approach critical velocity, i.e., (V_{m1c} x y_{m1}), $m^3/s/m$ (cfs/ft)
M = Discharge distribution factor (see E.4, Step 1)
V_{m0c} = Critical velocity in the main channel for unconstricted flow at depth y_{m0} (see E.4, Step 8), m/sec (ft/sec)
FS = Factor of Safety with a recommended value of 1.0

Note: Equation E.2 is based on experimental results for clear water scour around bankline abutments. Its extension to the live-bed case by assuming threshold live-bed scour is tentative at this time.

E.4 SOLVING STURM'S EQUATIONS

Sturm's equations are solved for through the application of the following steps:

1. Run WSPRO(3) or HEC-RAS(4) for the condition of the proposed bridge in place, creating a backwater at the approach Section 1 to the bridge. Compute the following for the left and right floodplains <u>in the approach Section 1</u> (Figure E.2) using the output from the water surface profile model to determine the overtopping flow and the flow distribution in the channel and on the floodplain:

M = discharge distribution factor

= $(Q_{1/2\ channel} + Q_{floodplain} - Q_{blocked\ flow})/(Q_{1/2\ channel} + Q_{floodplain})$

in which $Q_{1/2\ channel}$ is the discharge from the centerline to the bank of the main channel in the approach section; $Q_{floodplain}$ is the floodplain discharge in the approach section; and $Q_{blocked\ flow}$ is the floodplain discharge blocked by the embankment in the approach section.

The value of M needs to be determined separately for the right and left floodplains. For this purpose, it is assumed that the flow is divided down the centerline of the channel. The left half of the channel is used to calculate M for the left abutment, and the right half of the channel is used to calculate M for the right abutment. If there is overtopping flow, the denominator in the above equation should include only the flow going under the bridge. The overtopping flow will need to be distributed proportionally (according to the site conditions) between the flows for the left and right abutments.

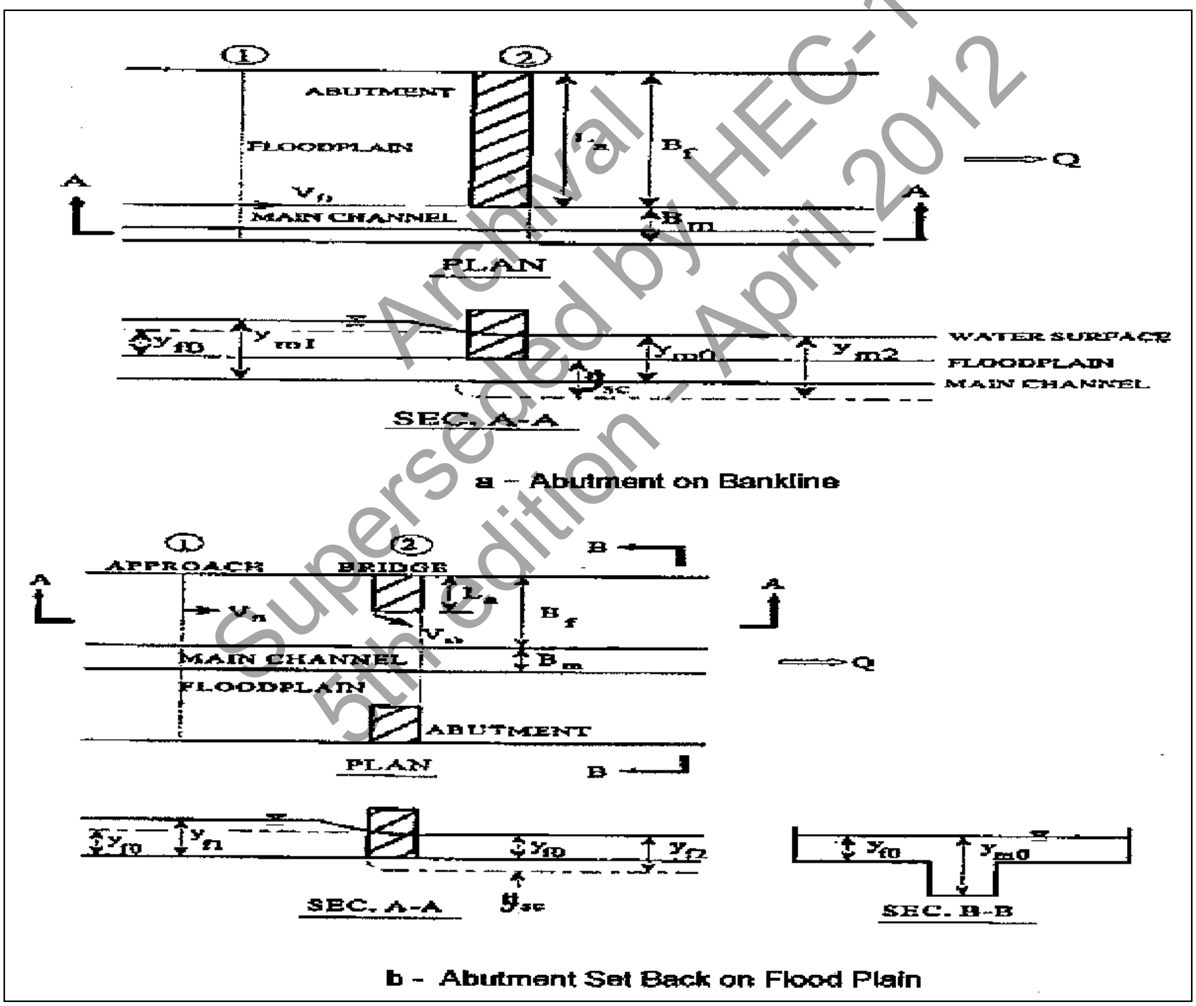

Figure E.2. Definition sketches for application of the Sturm method.

2. y_{f1} = average flow depth in the blocked section of flow in the approach section with a length approximately equal to the distance L_a, m (ft) as determined from the water surface profile model (Figure E.2). It is calculated as the blocked flow area divided by L_a.

3. V_{f1} = average flow velocity in the blocked section = $Q_{blocked\ flow}$ /(L_a x y_{f1}), m/s (ft/sec)

4. $q_{f1} = V_{f1} \times y_{f1}$ m^3/s /m (cfs/ft)

   Next, run WSPRO or HEC-RAS for the existing normal depth condition without the proposed bridge in place and determine the following parameters for the left and right floodplains in the approach Section 1:

5. Compute y_{f0} = average depth of flow on the floodplain, m (ft)

6. Compute the critical velocity of flow, V_{xc}, m/s (ft/sec)

   a. For abutments set back from the channel banks, $V_{xc} = V_{f0c}$). Compute the critical velocity of flow (V_{f0c}) corresponding to the depth of flow, y_{f0} on the floodplain for unconstricted flow and the D_{50} grain size of the floodplain soils using Equation 5.1, Chapter 5.

   b. For abutments at or near the channel banks, $V_{xc} = V_{m0c}$ Compute the critical velocity of the flow (V_{m0c}) from the hydraulic radius of flow of the main channel for unconstricted flow and the D_{50} grain size of the channel bed material using Equation 5.1, Chapter 5.

   c. Compute the critical velocity in the approach Section 1, V_{f1c} or V_{m1c} , for the constricted flow in the same way as for the unconstricted flow except use the approach depth for the constricted flow and determine if the abutment scour will be clear water or live bed by comparing with V_{f1} or V_{m1}.

7. Select the appropriate scour equation:

   a. **Clear-water Scour**

   For clear-water scour, go to Step 8.

   b. **Live-bed Scour for Set Back Abutment**

   If the scour is live-bed scour and the abutment is set back, make the following adjustments: Set $V_{f1} = V_{f1c}$; recompute Step 4 as $q_{fl} = V_{f1c}$ (y_{f1}) and continue to Step 8. (Take into account the effect of floodplain vegetation in estimating V_{f1c}).

   c. **Live-bed Scour for Bankline Abutment**

   If the scour is live bed scour and the abutment is on or near the bankline, use the scour prediction equation for live bed scour at bankline abutments given in Section E.3.

8. Compute the abutment shape factor for the left and right abutments:

   a. Compute the abutment shape factor K_{st} for spill through slopes:

      Compute X_a : $X_a = q_{f1} / (M \; V_{xc} \; y_{f0})$
      q_{f1} from Step 4
      M from Step 1
      V_{xc} from Step 6a or 6b
      y_{f0} from Step 5

      Compute K_{st} :
      $K_{st} = 1.52 (X_a - 0.67)/ (X_a - 0.40)$

      where:

      $0.67 \leq X_a \leq .1.2$

      $K_{st} = 1.0$ where $X_a \geq 1.2$
      $K_{st} = 0.0$ where $X_a \leq 0.67$

   b. For vertical wall abutments, with or without wingwalls, abutment shape factor $K_{st} = 1.0$

9. Compute the value of y_s / y_{f0} and the abutment scour depth, y_s, from Equation E.1.

10. Evaluate the value of $y_s / y_{f0:}$

    Use a maximum value of 10 for y_s / y_{f0}, based on experimental data.

    If V_{f1} (Step 3) equals or exceeds the critical velocity V_{f1c} for setback abutments, then live bed scour occurs and V_{f1} is set equal to V_{f1c} .

    The datum for measuring y_s is the channel bottom. The bottom of the scour hole is a vertical distance of ($y_s + y_{f0}$) below the water surface for existing conditions.

    For bankline abutments, regardless of whether the scour is clear water or live bed, the calculated scour depth includes both abutment scour and contraction scour.

    For bankline abutments, check for the possibility of live bed scour by determining if $V_{m1} \geq V_{m1c.}$ V_{m1} = average velocity in the main channel at the approach section and V_{m1c} = critical velocity in the main channel at the approach section. Compute V_{m1c} by Equation 5.1 using the hydraulic radius of the main channel for constricted flow and the D_{50} particle size of the channel bed material. If $V_{m1} \geq V_{m1c}$, set $V_{m1} = V_{m1c}$ and use the live bed scour procedure equation presented in Section E.3.

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4. U.S. Army Corps of Engineers, 1998, "River Analysis System," HEC-RAS, User's Manual Version 2.2, Hydrologic Engineering Center, Davis, CA.

APPENDIX F

MARYLAND ABUTMENT SCOUR EVALUATION METHOD ABSCOUR

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APPENDIX F

Maryland Abutment Scour Evaluation Method
ABSCOUR

F.1 MARYLAND SHA ABUTMENT SCOUR PROGRAM (ABSCOUR)

Maryland SHA developed a procedure for determining abutment scour based on coefficients applied to contraction scour. The equations and method are presented in this appendix for those states that might want to use the method to compare with the equations and advice given in Chapter 7.

The Maryland SHA abutment scour equations and methods are based on the research and development of Chang.[1, 2] Chang applied Laursen's long contraction theory to both clear-water and live-bed scour. He developed a "velocity adjustment factor" k_v to account for the non-uniform velocity distribution in the contracted section, and a "spiral-flow adjustment factor" k_f at the abutment toe that depends on the approach Froude number. The value of k_v was based on potential flow theory, and k_f was determined by Chang from the analysis of a collection of abutment scour experiments in laboratory flumes.[3]

F.1.2 Live-bed Abutment Scour

For live-bed abutment scour the equation is:

$$\frac{y_{2a}}{y_1} = K_f \left[\frac{k_v q_2}{q_1} \right]^{K_2} \qquad \text{(F.1)}$$

where:

- y_{2a} = Total flow depth in the abutment scour hole after scour has occurred, measured from the water surface to the bottom of the scour hole, m (ft)
- y_1 = Approach flow depth, m (ft)
- q_1 = Flow rate per unit width in the approach section, $m^3/s/m$ ($ft^3/s/ft$)
- q_2 = Flow rate per unit width in contracted section, $m^3/s/m$ ($ft^3/s/ft$) (Determination of q_1 and q_2 is explained in a section below)
- k_v = $0.8 (q_1/q_2)^{1.5} + 1$
- k_f = $0.35 + 3.2 F_1$ for live-bed scour

Equation F.1 applies to live-bed scour. It should be used for clear-water scour only for the condition where the shear stress in the approach section (Section 1) is at the critical value.

Values of k_v should range from 1.0 to 1.8. If the calculated value is smaller or larger than this range, use the limiting value.

Values of k_f should range from 1.0 to 3.3. If the calculated value is smaller or larger than this range, use the limiting value.

The Froude number in the approach Section 1 (F_1) = $V_1/(gy_1)^{0.5}$. where V_1 = average flow velocity in the approach floodplain or channel section (m/s or ft/s) and y_1 = average flow depth in the approach floodplain or channel section (m or ft).

K_2 = Laursen's sediment transport function = $0.11\,(\tau_c / \tau_1 + 0.4)^{2.2} + 0.623$ (F.2)

where:

- τ_c = Critical shear stress of soil, N/m^2 (lb/ft^2)
- τ_1 = Shear stress at approach section, N/m^2 (lb/ft^2), $\tau_1 \geq \tau_c$

The value of K_2 varies from 0.637 to 0.857. If $\tau_c \geq \tau_1$, select a value of K_2 equal to 0.857.

Unpublished studies by Chang have shown that, while K_2 is based on a concept that is similar to the K_1 coefficient in the table accompanying the live-bed contraction scour equation (Equation 5.2), the values of these coefficients are derived in different ways and cannot be mathematically correlated.

Figure F.1 illustrates the variables used in Equations F.1 and F.2. Both equations are non-dimensional and can be used either for English or SI units. The same symbols are used for flow depth in the main channel and floodplain, but the subscript is changed to denote the approach section and the bridge section.

F.1.3 Clear-Water Abutment Scour

Clear-water scour occurs If the shear stress in the approach Section 1 is less than critical, or if the approach section is armored. The clear-water abutment scour equation is as follows:

$$y_{2a} = k_f\,(k_v)^{0.857}\,y_{2c} \qquad \text{(F.3)}$$

where:

- y_{2a} = Total depth of flow at the abutment, measured from the water surface down to the bottom of the abutment scour hole, m (ft)
- y_{2c} = Clear water contraction scour depth in the channel or on the floodplain (beyond the abutment scour hole) at critical velocity $y_{2c} = q_2/V_c$, m (ft). Equation 5.1 or other similar equations can be used to compute V_c. Another approach would be to compute y_{2c} directly from Laursen's clear-water contraction scour Equation 5.4.
- K_v = Dimensionless coefficients as defined above in live-bed scour
- K_f = $0.1 + 4.5\,F_1$ for clear-water scour

Equation F.3 can be used either for English units or SI units.

When using Equations F.1 and F.3, the Engineer needs to take into account that the actual field conditions will most likely vary from the simple geometry depicted in Chapter 7 (Figure 7.6). Judgment is necessary in adjusting the theoretical scour to reflect actual field conditions.

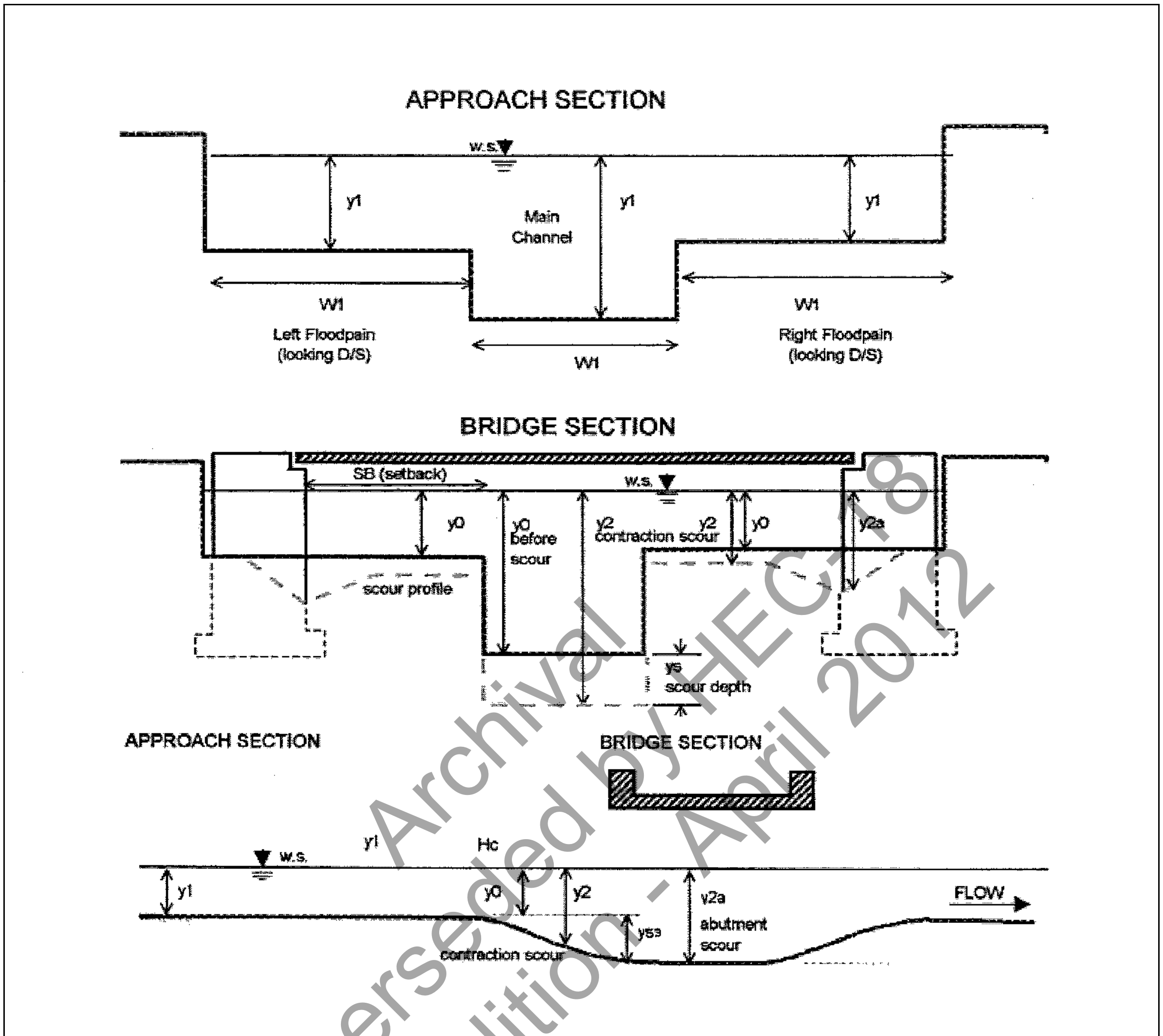

Figure F.1. Definition sketches for scour computations.

F.2 COMPUTATION OF UNIT DISCHARGE

Equations F.1 and F.3 were developed based on simple rectangular geometry for the channel and floodplains (Figure F.1). The method for computing unit discharges at Section 2 in the main channel and on the floodplain under the bridge (for setback abutments) is based on information obtained from the laboratory studies conducted by Sturm and others. The first step in this process is to determine in which category the abutment setback from the channel bank should be placed: **short setback, intermediate setback, or long setback**. The description below is based on the assumption that the left or right floodplain width is essentially the same at Section 1 as it is at Section 2 (Figure F.1). Where there is a significant difference in the floodplain width at Section 1 and Section 2, the Engineer will need to use judgment in selecting the most appropriate method for selecting the unit flow discharge.

F.2.1 Short Setback

If the setback from the main channel bank to the toe of the abutment is equal to or less than five times the depth of flow in the main channel at the bridge, the flow in the main channel and on the floodplain under the bridge is assumed to be mixed flow, having the same velocity. Note that this computation must be made separately for the left and the right floodplains. The average flow velocity through the bridge is computed as $V_{short} = Q_{bridge}/A_{bridge}$. Q_{bridge} is equal to Q_{total} - $Q_{overtopping}$. A_{bridge} is equal to the total bridge waterway area below the water surface. The unit discharge at any point under the bridge, in the channel or the overbank area, is computed as:

$$q = V_{short}\,(y) \tag{F.4}$$

where:

q	=	Unit flow rate, $m^3/s\ /m$ (cfs/ft)
V_{short}	=	Computed average velocity through the bridge determined by the above noted equation $V_{short} = Q_{bridge}/A_{bridge}$, m/s (ft /sec)
y	=	Depth of flow at the point of interest, m (ft)

F.2.2 Long Setback

If the abutment setback is greater than 75 percent of the total floodplain width at the approach section, the assumption is made that the channel flow, Q, at Section 2 under the bridge is the same as the channel flow, Q, at the approach Section 1. Similarly, the flow in the left or right floodplain in the approach Section 1 remains the same in the floodplain section under the bridge. (This is considered to be a conservative assumption.) The unit discharge on the left or right floodplain at Section 1 is computed as $q_1 = Q/W_1$ where Q is the floodplain flow and W_1 is the width of the floodplain. At the bridge Section 2, $q_2 = Q/W_2$ where W_2 is the setback distance to the abutment. It follows that:

$$q_2 = q_1\,(W_1 / W_2) \tag{F.5}$$

where:

q_z	=	Unit flow rate at setback abutment on floodplain, $m^3/s\ /m$ (cfs/ft)
q_1	=	Unit flow rate at approach Section 1 on the floodplain, $m^3/s\ /m$ (cfs/ft)
W_1	=	Width of floodplain at approach Section 1, m (ft)
W	=	Width of floodplain under bridge (abutment setback) at Section 2, m (ft)
V_{long}	=	q_2 / y_2 where y = the depth of flow at the point of interest, m (ft)

F.2.3 Intermediate Setback

In some cases, the abutment setback from the channel bank will be located at a point between the short setback and the long setback described in the forgoing sections. This location is defined as an intermediate setback. An interpolation scheme is used to compute the velocity ($V_{intermediate}$) and corresponding unit discharge ($q_{intermediate}$). This scheme provides for a smooth transition from the velocity associated with the short setback to the velocity associated with the long setback. $V_{intermediate}$ is determined by using the following three steps:

1. Calculate V_{short} at a setback distance equal to five times the channel depth at the bridge (Setback = 5 y_o = SB_{short})

2. Calculate V_{long} at a setback distance equal to 75 percent of the total floodplain width at the approach Section 1 (Setback = 0.75 W1 = SB_{long})

3. Calculate $V_{intermediate} = V_{short} - ((V_{short} - V_{long})/ (SB_{long} - SB_{short}))\ (SB - SB_{short})$ where SB = setback distance to abutment

The unit discharge, q, is then determined as $V_{intermediate}$ (y), where y is the depth of flow at the abutment.

Equations F.1 and F.3 compute the combined contraction scour and local abutment scour; therefore, contraction scour depths should not be added to the values obtained for scour at the abutment. Measurements of y_{2a} or y_{2c} are made from the water surface to the bottom of the abutment scour hole or to the contracted channel bed elevation, respectively.

The actual depth of abutment scour, y_{sa}, m (ft) is determined from Equation F.1 or Equation F.3 by subtracting the initial flow depth before scour, y_0 , from the flow depth to the bottom of the scour hole, y_{2a} :

$$y_{sa} = y_{2a} - y_0 \qquad \text{(F.6)}$$

F.3 ABUTMENT SHAPE FACTOR (K_t)

The scour depth, y_{sa}, determined in Equation F.6 must be modified by multiplying it by the abutment shape factor. The abutment shape factors given in Chapter 7, Table 7.1 apply only to short abutments in Maryland's abutment scour equations. As the length of the abutment and approach road in the floodplain increase, the effect of a spill through slope in reducing scour is decreased. For long approach road sections on the floodplain, this coefficient will approach a value of 1.0. Similarly, scour for vertical wall abutments with wingwalls on short abutment sections is reduced to 82 percent of the scour of vertical wall abutments without wingwalls. As the length of the abutment and approach road in the floodplain increase, the effect of the wingwall in reducing scour is decreased. For long approach road sections in the floodplain, this coefficient will approach a value of 1.0.

F.3.1 Maryland's Coefficient for Spill-Through Abutments

$$K_t = -.55 + 0.05((L / dL) - 1) \qquad \text{(F.7)}$$

where:

L	=	Total embankment encroachment length from the water's edge on the floodplain to the toe of the spill through slope, m (ft)
dL	=	Distance from the spill through toe to the point where the water surface intersects the spill through slope, m (ft)

If L/dL > 10, K_t = 1.0

F.3.2 Maryland's Coefficient for Vertical Wall with Wingwalls Abutments

$$K_t = 0.82 + 0.02\,((L / dL) - 1) \qquad \text{(F.8)}$$

where:

L	=	Total embankment encroachment length from the water's edge on the floodplain to the face of the abutment, m (ft)
dL	=	Distance measured parallel to the embankment from the end of the wingwall to the face of the abutment, m (ft)

If L/dL > 10, K_t = 1.0

F.3.3 Maryland's Coefficient for Vertical Wall without Wingwalls Abutments

For vertical wall abutments without wingwalls, K_t = 1.0

F.4 SKEW ANGLE FACTOR

The scour depth, y_{sa}, determined in Equation F.6 must be modified by multiplying it by the skew angle factor determined in Chapter 7, Section 7.2.

F.5 FACTOR OF SAFETY

Comparisons of computed vs. measured scour depths have been made using data from Sturm's tests[(4)] and other sources (Figure F.2). The lines of uncertainty represent a difference of +/-20 percent from the measured value. The Engineer may wish to apply a Factor of Safety of 20 to 40 percent of the computed scour value to account for this variation. (No Factor of Safety was applied to the computed values).

F.6 ABSCOUR PROGRAM

As noted in Chapter 5, the estimation of contraction scour at bridges involves consideration of a number of variables and becomes a complex process, particularly for Case 1c where the abutments are set back from the channel edge. For this reason, the Maryland SHA procedure for estimating abutment scour has been incorporated in a Windows-type software program entitled ABSCOUR to calculate contraction scour and abutment scour. The program facilitates rapid evaluation of the various factors affecting abutment scour and enables the Engineer to select the conditions and the scour analysis most appropriate for the site under evaluation. Various refinements have been incorporated in the program that would not be practical for use in a manual method. The ABSCOUR program is available from the Maryland SHA.

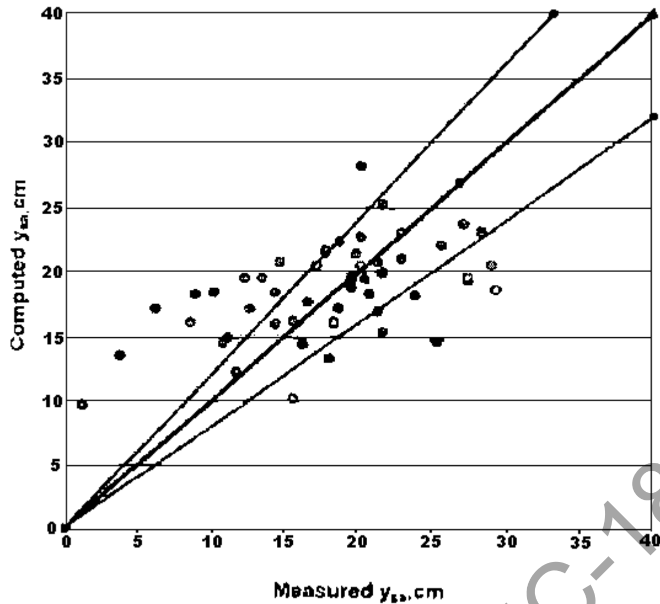

Figure F.2. Comparison of measured and predicted scour depths, Maryland SHA Equations.

F.7 REFERENCES

1. Chang, F. and S.R. Davis, 1999, "Maryland SHA Procedure for Estimating Scour at Bridge Abutments, Part I - Live Bed Scour," ASCE Compendium, Stream Stability and Scour at Highway Bridges, Richardson and Lagasse (eds.), Reston, VA.

2. Chang, F. and S.R. Davis, 1999, "The Maryland State Highway Administration ABSCOUR Program," Maryland SHA.

3. Palaviccini, M., 1993, "Scour Prediction Model at Bridge Abutments," Dissertation in partial fulfillment of the requirements of Doctor of Engineering Dissertation, The Catholic University of America, Washington, D.C.

4. Sturm, T.W., 1999, "Abutment Scour Studies for Compound Channels," U.S. Department of Transportation, Federal Highway Administration, September.

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APPENDIX G

WSPRO INPUT AND OUTPUT FOR EXAMPLE PROBLEMS IN CHAPTER 8 AND APPENDIX H

- **G1 (SI**
- **G2 (English)**

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APPENDIX G1

WSPRO Input and Output for Chapter 8 Example Problem (SI)

Line #	Input parameters

```
*f
T1        WORKSHOP PROBLEM - SCOUR CREEK - METRIC CONVERSION
T2        ESTIMATING SCOUR AT BRIDGES - COMPUTER SIMULATION
T3        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
*
SI        1
*
Q         849.51
SK        0.002
*
XS   EXIT  228.6 * * * .002
GR        0,5.79  30.48,4.57  60.96,3.35  152.4,3.28  274.32,3.05  335.28,2.74
GR        370.33,1.68  381.00,1.49  396.24,0.93  411.48,1.48  422.15,1.55
GR        457.2,2.74  518.16,3.05  640.08,3.28  731.52,3.35  762.00,4.57
GR        792.48,5.79
N           0.042     0.032     0.042
SA               335.28    457.2
*
XS   FULLV 426.72
*
BR   BRDG  426.72
BL 1       198.12   335.28   457.2
BC         5.49
CD         3  15.24  2  6.71
AB         2
PD 0       1.72  9.14   6
N          0.042   0.032
SA            335.28
*
XS   APPR  640.08
*
HP 2 BRDG  4.23  1  4.23  849.51
HP 1 BRDG  4.15  1  4.15
HP 2 APPR  5.27  1  5.27  849.51
HP 1 APPR  5.27  1  5.27
*
EX
ER
```

OUTPUT DATA FOR CHAPTER EXAMPLE PROBLEM

Line #	Input parameters

```
   ***********************  W S P R O  ***************************
     Federal Highway Administration  -  U. S. Geological Survey
            Model for Water-Surface Profile Computations.
       Run Date & Time: 10/26/94  1:55 pm     Version V081594
        Input File: scourcrm.dat   Output File: scourcrm.lst
  *----------------------------------------------------------------*
*F
 ***                   Input Data In Free Format                   ***

T1         WORKSHOP PROBLEM - SCOUR CREEK - METRIC CONVERSION
T2         ESTIMATING SCOUR AT BRIDGES - COMPUTER SIMULATION
T3         CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
SI         1
                    Metric (SI) Units Used in WSPRO
               Quantity    SI Unit              Precision
             ------------ -------------------- -----------
              Length       meters                  0.001
              Depth        meters                  0.001
              Elevation    meters                  0.001
              Widths       meters                  0.001
              Velocity     meters/second           0.001
              Discharge    cubic meters/second     0.001
              Slope        meter/meter             0.001
              Angles       degrees                 0.01
             ------------ -------------------- -----------
Q          849.51
 ***   Processing Flow Data; Placing Information into Sequence  1   ***
SK         0.002
  ************************  W S P R O  ***************************


         *---------------------------------------------------*
         *      Starting To Process Header Record EXIT       *
         *---------------------------------------------------*
XS   EXIT  228.6 * * * .002
GR         0,5.79  30.48,4.57  60.96,3.35  152.4,3.28  274.32,3.05  335.28,2.74
GR         370.33,1.68  381.00,1.49  396.24,0.93  411.48,1.48  422.15,1.55
GR         457.2,2.74  518.16,3.05  640.08,3.28  731.52,3.35  762.00,4.57
GR         792.48,5.79
N            0.042    0.032    0.042
SA                335.28    457.2

 ***   Completed Reading Data Associated With Header Record EXIT    ***
 ***   Storing Header Data In Temporary File As Record Number  1    ***

 ***                Data Summary For Header Record EXIT             ***
 SRD Location:      229.   Cross-Section Skew:    .0   Error Code   0
 Valley Slope:    .00200   Averaging Conveyance By Geometric Mean.
 Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                   X,Y-coordinates (17 pairs)
     X          Y           X          Y           X          Y
----------  ----------   ----------  ----------   ----------  ----------
      .000       5.790       30.480       4.570       60.960       3.350
   152.400       3.280      274.320       3.050      335.280       2.740
   370.330       1.680      381.000       1.490      396.240        .930
   411.480       1.480      422.150       1.550      457.200       2.740
   518.160       3.050      640.080       3.280      731.520       3.350
   762.000       4.570      792.480       5.790
----------  ----------   ----------  ----------   ----------  ----------
                 Minimum and Maximum X,Y-coordinates
  Minimum X-Station:       .000  ( associated Y-Elevation:    5.790 )
  Maximum X-Station:    792.480  ( associated Y-Elevation:    5.790 )
  Minimum Y-Elevation:     .930  ( associated X-Station:    396.240 )
  Maximum Y-Elevation:    5.790  ( associated X-Station:    792.480 )

Subarea Breakpoints (NSA =   3):
        335.    457.
 Roughness Coefficients (NSA =  3):
```

```
          .042    .032    .042
             *---------------------------------------------------*
            *      Finished Processing Header Record EXIT       *
            *---------------------------------------------------*
      *************************  W S P R O  ***************************

            *---------------------------------------------------*
            *      Starting To Process Header Record FULLV      *
            *---------------------------------------------------*

   XS   FULLV 426.72

    ***   Completed Reading Data Associated With Header Record FULLV   ***
    ***   No Roughness Data Input, Propagating From Previous Section   ***
    ***   Storing Header Data In Temporary File As Record Number  2    ***

    ***              Data Summary For Header Record FULLV              ***

    SRD Location:      427.   Cross-Section Skew:    .0   Error Code   0
    Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
    Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                         X,Y-coordinates (17 pairs)
        X          Y            X          Y            X          Y
   ----------  ----------   ----------  ----------   ----------  ----------
         .000       6.186       30.480       4.966       60.960       3.746
      152.400       3.676      274.320       3.446      335.280       3.136
      370.330       2.076      381.000       1.886      396.240       1.326
      411.480       1.876      422.150       1.946      457.200       3.136
      518.160       3.446      640.080       3.676      731.520       3.746
      762.000       4.966      792.480       6.186
   ----------  ----------   ----------  ----------   ----------  ----------

                      Minimum and Maximum X,Y-coordinates
     Minimum X-Station:        .000  ( associated Y-Elevation:    6.186 )
     Maximum X-Station:     792.480  ( associated Y-Elevation:    6.186 )
     Minimum Y-Elevation:     1.326  ( associated X-Station:    396.240 )
     Maximum Y-Elevation:     6.186  ( associated X-Station:    792.480 )

 Subarea Breakpoints (NSA =   3):
         335.    457.

 Roughness Coefficients (NSA =  3):
          .042    .032    .042

            *---------------------------------------------------*
            *     Finished Processing Header Record FULLV       *
            *---------------------------------------------------*
      *************************  W S P R O  ***************************

            *---------------------------------------------------*
            *      Starting To Process Header Record BRDG       *
            *---------------------------------------------------*
   BR   BRDG  426.72
   BL  1       198.12   335.28   457.2
   BC          5.49
   CD          3  15.24  2  6.71
   AB          2
   PD  0       1.72  9.14   6
   N           0.042   0.032
   SA             335.28

    ***   Completed Reading Data Associated With Header Record BRDG    ***
    ***   Storing Header Data In Temporary File As Record Number  3    ***

    ***              Data Summary For Header Record BRDG               ***

    SRD Location:      427.   Cross-Section Skew:    .0   Error Code   0
    Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
    Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00
                          X,Y-coordinates (13 pairs)
        X          Y            X          Y            X          Y
   ----------  ----------   ----------  ----------   ----------  ----------
      263.788       5.490      267.852       3.458      274.319       3.446
```

```
       335.279       3.136      370.329       2.076      380.999       1.886
       396.239       1.326      411.479       1.875      422.149       1.946
       457.199       3.136      457.200       3.136      461.908       5.490
       263.788       5.490
    ----------  ----------   ----------  ----------   ----------  ----------
                    Minimum and Maximum X,Y-coordinates
      Minimum X-Station:     263.788  ( associated Y-Elevation:    5.490 )
      Maximum X-Station:     461.908  ( associated Y-Elevation:    5.490 )
      Minimum Y-Elevation:     1.326  ( associated X-Station:    396.239 )
      Maximum Y-Elevation:     5.490  ( associated X-Station:    263.788 )

  Subarea Breakpoints (NSA =   2):
 335
  Roughness Coefficients (NSA =  2):
          .042    .032

  Discharge coefficient parameters:
   BRTYPE   BRWDTH   EMBSS    EMBELV   USERCD
      3       15.2    2.00      6.71   ******

  Pressure flow elevations:  AVBCEL =     5.49   PFELEV =     5.49

  Abutment parameters:
    ABSLPL   ABSLPR   XTOELT   YTOELT   XTOERT   YTOERT
       2.0   ******    267.9      3.5    457.2      3.1

  Bridge Length and Bottom Chord component input data:
    BRLEN   LOCOPT   XCONLT   XCONRT   BCELEV    BCSLP   BCXSTA
    198.1       1.     335.     457.     5.49   ******  *******

  Pier Data:  Number  1 Pier/Pile Code: 0.
        ELEV  WDTH  #P/P       ELEV  WDTH  #P/P       ELEV  WDTH  #P/P
        1.72    9.1   6.00

            *----------------------------------------------------*
            *      Finished Processing Header Record BRDG      *
            *----------------------------------------------------*
      *************************  W S P R O  ***************************

            *----------------------------------------------------*
            *      Starting To Process Header Record APPR      *
            *----------------------------------------------------*

    XS   APPR  640.08

     ***   Completed Reading Data Associated With Header Record APPR    ***
     ***   No Roughness Data Input, Propagating From Previous Section   ***
     ***   Storing Header Data In Temporary File As Record Number  4    ***

     ***             Data Summary For Header Record APPR                ***

     SRD Location:      640.   Cross-Section Skew:    .0   Error Code   0
     Valley Slope:   .00200    Averaging Conveyance By Geometric Mean.
     Energy Loss Coefficients ->   Expansion:   .50   Contraction:   .00

                       X,Y-coordinates (17 pairs)
         X          Y            X          Y            X          Y
    ----------  ----------   ----------  ----------   ----------  ----------
          .000       6.613       30.479       5.393       60.959       4.173
       152.399       4.103      274.319       3.873      335.279       3.563
       370.329       2.503      380.999       2.313      396.239       1.753
       411.479       2.302      422.149       2.373      457.199       3.563
       518.159       3.873      640.079       4.103      731.519       4.173
       761.999       5.393      792.479       6.613
    ----------  ----------   ----------  ----------   ----------  ----------
                     Minimum and Maximum X,Y-coordinates
      Minimum X-Station:        .000  ( associated Y-Elevation:    6.613 )
      Maximum X-Station:     792.479  ( associated Y-Elevation:    6.613 )
      Minimum Y-Elevation:     1.753  ( associated X-Station:    396.239 )
      Maximum Y-Elevation:     6.613  ( associated X-Station:    792.479 )

  Subarea Breakpoints (NSA =   3):
         335.    457.

```

```
Roughness Coefficients (NSA =  3):
         .042    .032    .042

Bridge datum projection(s):  XREFLT  XREFRT  FDSTLT  FDSTRT
                            ******* ******* ******* *******

           *----------------------------------------------------*
           *      Finished Processing Header Record APPR        *
           *----------------------------------------------------*
     *************************  W S P R O  ***************************

  HP  2 BRDG  4.23  1  4.23  849.51
  HP  1 BRDG  4.15  1  4.15
  HP  2 APPR  5.27  1  5.27  849.51
  HP  1 APPR  5.27  1  5.27
  EX

          *====================================================*
          *     Summary of Boundary Condition Information     *
          *====================================================*

             Reach      Water Surface   Friction
     #    Discharge      Elevation      Slope        Flow Regime
     --   ---------    -------------   --------   -------------------
     1      849.51      ********        .0020       Sub-Critical
     --   ---------    -------------   --------   -------------------

          *====================================================*
          *        Beginning  1 Profile Calculation(s)        *
          *====================================================*

     *************************  W S P R O  ***************************

                       WSEL     VHD       Q        AREA      SRDL      LEW
                       EGEL      HF       V         K        FLEN      REW
                       CRWS      HO      FR #       SF       ALPHA     ERR
                    --------- ------ --------- --------- --------- ---------
 Section: EXIT          3.832   .173   849.509   622.871      .000    48.894
 Header Type: XS        4.006   .000     1.364  18992.99      .000   743.584
 SRD:    228.600        3.615   .000      .622     .0000     1.830      .000

 Section: FULLV         4.231   .172   849.509   624.430   198.119    48.837
 Header Type: FV        4.404   .395     1.360  19053.13   198.119   743.642
 SRD:    426.719        4.011   .000      .620     .0020     1.828      .002

        <<< The Preceding Data Reflect The "Unconstricted" Profile >>>

 Section: APPR          4.658   .172   849.509   624.574   213.360    48.829
 Header Type: AS        4.830   .424     1.360  19059.62   213.360   743.648
 SRD:    640.080        4.438   .000      .620     .0020     1.828      .002

        <<< The Preceding Data Reflect The "Unconstricted" Profile >>>

         <<< The Following Data Reflect The "Constricted" Profile >>>
               <<< Beginning Bridge Hydraulics Computations >>>

 Section: BRDG          4.151   .769   849.509   294.018   198.119   266.464
 Header Type: BR        4.921   .620     2.889  12559.79   198.119   459.231
 SRD:    426.719        3.990   .293     1.004     .0020     1.806      .000

 Specific Bridge Information   C     P/A   PFELEV    BLEN     XLAB     XRAB
 Bridge Type 3   Flow Type 1 ------ ----- -------- -------- -------- --------
 Pier/Pile Code  0           .7441  .034    5.489  198.120  267.851  457.197
 -------------------------- ------ ----- -------- -------- -------- --------

 Section: APPR          5.268   .050   849.509  1058.158   198.120    33.581
 Header Type: AS        5.318   .323      .802  39088.53   213.359   758.896
 SRD:    640.080        4.438   .074      .263     .0020     1.534     -.003
               Approach Section APPR  Flow Contraction Information
             M( G )   M( K )     KQ       XLKQ     XRKQ     OTEL
            -------- -------- --------- -------- -------- --------
                .722     .426   22535.5  271.518  463.594    5.175
            -------- -------- --------- -------- -------- --------
```

```
                   <<< End of Bridge Hydraulics Computations >>>

     *************************  W S P R O  ***************************

   ***    Beginning Velocity Distribution For Header Record BRDG     ***
          SRD Location:    426.720       Header Record Number  3

       Water Surface Elevation:      4.230             Element # 1
       Flow:     849.510   Velocity:  2.75  Hydraulic Depth:   1.600
       Cross-Section Area:    309.17         Conveyance:    13531.24
          Bank Stations ->  Left:    266.307   Right:    459.388

X STA.       266.3      305.8      332.9      348.6      358.2      366.0
  A( I )            32.8       27.5       19.8       15.8       14.9
  V( I )            1.29       1.54       2.15       2.68       2.86
  D( I )             .83       1.01       1.26       1.64       1.91

X STA.       366.0      372.2      378.1      383.5      388.3      392.6
  A( I )            13.2       13.2       12.5       12.1       11.8
  V( I )            3.22       3.21       3.39       3.50       3.61
  D( I )            2.11       2.24       2.35       2.52       2.69

X STA.       392.6      396.8      400.8      405.2      410.1      415.4
  A( I )            11.6       11.4       11.7       12.2       12.6
  V( I )            3.65       3.73       3.62       3.47       3.38
  D( I )            2.82       2.84       2.66       2.49       2.35

X STA.       415.4      421.0      427.2      434.3      443.5      459.4
  A( I )            12.8       13.8       14.3       15.7       19.4
  V( I )            3.32       3.09       2.97       2.71       2.19
  D( I )            2.31       2.22       1.99       1.71       1.22


     *************************  W S P R O  ***************************

   ***    Compute Cross-Section Properties For Header Record BRDG    ***
           SRD Location:     426.720       Header Record Number  3

Water     S  Cross      Cross                  Bank Station
Surface   A  Section    Section  Top    Wetted  ------------  Hydrlic  Critical
Elevation #  Conveyance Area(s) Width   Pmtr    Left  Right   Depth     Flow
--------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
           1    1208.24     57.   68.8  68.98                   .834    164.12
           2   11333.03    236.  123.9 124.25                  1.906   1021.92
    4.150      12541.26    294.  192.8 193.22  266.5  459.2   1.523   1052.71
--------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
     *************************  W S P R O  ***************************

   ***    Beginning Velocity Distribution For Header Record APPR     ***
          SRD Location:    640.080       Header Record Number  4

       Water Surface Elevation:      5.270             Element # 1
       Flow:     849.510   Velocity:   .80  Hydraulic Depth:   1.460
       Cross-Section Area:   1059.34         Conveyance:    39151.16
          Bank Stations ->  Left:     33.541   Right:    758.937

X STA.        33.5      124.4      186.1      242.1      290.5      330.4
  A( I )            86.2       72.9       71.9       67.3       63.1
  V( I )             .49        .58        .59        .63        .67
  D( I )             .95       1.18       1.28       1.39       1.58

X STA.       330.4      352.8      366.9      378.4      388.4      396.9
  A( I )            42.7       34.7       32.2       30.5       28.9
   V( I )             .99       1.22       1.32       1.39       1.47
  D( I )            1.91       2.45       2.80       3.05       3.38

X STA.       396.9      405.5      415.4      426.5      440.0      462.5
  A( I )            28.5       30.2       32.0       33.9       43.5
  V( I )            1.49       1.41       1.33       1.25        .98
  D( I )            3.34       3.03       2.88       2.52       1.93

X STA.       462.5      501.9      549.6      604.8      668.4      758.9
  A( I )            62.2       66.4       71.0       75.2       85.8
  V( I )             .68        .64        .60        .57        .49
```

```
  D( I )            1.58       1.39       1.29       1.18        .95


     *************************  W S P R O  ***************************

   ***    Compute Cross-Section Properties For Header Record APPR    ***
           SRD Location:    640.080       Header Record Number  4

Water     S  Cross      Cross                      Bank Station
Surface   A  Section    Section  Top   Wetted  ------------  Hydrlic  Critical
Elevation #  Conveyance Area(s) Width   Pmtr    Left  Right   Depth     Flow
--------- -- ---------- ------- ------ ------- ------ ------ ------- ---------
           1   10075.62    370.  301.7  301.76                  1.225   1281.66
           2   18999.92    320.  121.9  121.98                  2.622   1622.42
           3   10075.62    370.  301.7  301.76                  1.225   1281.66
    5.270      39151.16   1059.  725.4  725.50   33.5  758.9   1.460   3237.29
--------- -- ---------- ------- ------ ------- ------ ------ ------- ---------

  ER

   ******************  Normal end of WSPRO execution.  *****************
   ***************  Elasped Time:   0 Minutes  0 Seconds  **************
```

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APPENDIX G2

WSPRO Input and Output for Appendix H Example Problem (English)

INPUT DATA FOR CHAPTER 4 EXAMPLE PROBLEM

```
T1       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
T2       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
T3       HEC-18 - EVALUATING SCOUR AT BRIDGES
*
Q        30000
SK        0.002
*
XS   EXIT  750 * * * .002
GR       0,19   100,15   200,11   500,10.75   900,10   1100,9.0    1215,5.5
GR       1250,4.9   1300,3.05   1350,4.85   1385,5.1   1500,9.0    1700,10
GR       2100,10.75   2400,11   2500,15   2600,19
N         0.042    0.032    0.042
SA             1100      1500
*
XS   FULLV 1400
*
BR   BRDG  1400
BL 1      650   1100   1500
BD        4  22
CD        3  50  2  22
AB        2
PW        5.65  30
N        0.042   0.032
SA           1100
*
AS   APPR  2100
*
HP 2 BRDG  13.82 * * 30000
*
HP 1 BRDG  13.54 1 13.54
*
HP 2 APPR  17.36 * * 30000
*
HP 1 APPR  17.36 1 17.36
*
EX
ER
```

OUTPUT

```
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

         *** RUN DATE & TIME: 09-10-92  10:08

 T1      SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
 T2      CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
 T3      HEC-18 - EVALUATING SCOUR AT BRIDGES
 *
 Q       30000
*** Q-DATA FOR SEC-ID, ISEQ =          1
 SK      0.002
 *
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "EXIT "
 XS   EXIT  750 * * * .002
 GR       0,19   100,15   200,11   500,10.75   900,10   1100,9.0    1215,5.5
 GR       1250,4.9   1300,3.05   1350,4.85   1385,5.1   1500,9.0   1700,10
 GR       2100,10.75   2400,11   2500,15   2600,19
 N          0.042    0.032    0.042
 SA             1100      1500
 *

*** FINISH PROCESSING CROSS SECTION - "EXIT "
*** CROSS SECTION "EXIT " WRITTEN TO DISK, RECORD NO. =  1

--- DATA SUMMARY FOR SECID "EXIT " AT SRD =    750.  ERR-CODE =     0

    SKEW    IHFNO    VSLOPE       EK       CK
      .0       0.    .0020      .50      .00

X-Y COORDINATE PAIRS  (NGP =  17):
      X      Y          X      Y          X      Y          X      Y
      .0   19.00     100.0   15.00     200.0   11.00     500.0   10.75
   900.0   10.00    1100.0    9.00    1215.0    5.50    1250.0    4.90
  1300.0    3.05    1350.0    4.85    1385.0    5.10    1500.0    9.00
  1700.0   10.00    2100.0   10.75    2400.0   11.00    2500.0   15.00
  2600.0   19.00

 X-Y MAX-MIN POINTS:
    XMIN      Y        X    YMIN      XMAX      Y        X    YMAX
      .0   19.00   1300.0    3.05    2600.0   19.00       .0   19.00

SUBAREA BREAKPOINTS (NSA =  3):
      1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
       .042    .032    .042
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "FULLV"
 XS   FULLV 1400
```

```
  *

*** FINISH PROCESSING CROSS SECTION - "FULLV"
*** NO ROUGHNESS DATA INPUT, WILL PROPAGATE FROM PREVIOUS CROSS SECTION.
*** CROSS SECTION "FULLV" WRITTEN TO DISK, RECORD NO. =  2

--- DATA SUMMARY FOR SECID "FULLV" AT SRD =   1400.  ERR-CODE =      0

    SKEW     IHFNO    VSLOPE        EK        CK
      .0        0.     .0020       .50       .00

X-Y COORDINATE PAIRS  (NGP =  17):
      X      Y          X      Y          X      Y          X      Y
      .0   20.30     100.0   16.30     200.0   12.30     500.0   12.05
   900.0   11.30    1100.0   10.30    1215.0    6.80    1250.0    6.20
  1300.0    4.35    1350.0    6.15    1385.0    6.40    1500.0   10.30
  1700.0   11.30    2100.0   12.05    2400.0   12.30    2500.0   16.30
  2600.0   20.30

 X-Y MAX-MIN POINTS:
    XMIN       Y         X    YMIN      XMAX       Y         X    YMAX
      .0   20.30    1300.0    4.35    2600.0   20.30        .0   20.30

SUBAREA BREAKPOINTS (NSA =  3):
      1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
        .042    .032    .042
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "BRDG "
  BR   BRDG  1400
  BL 1       650   1100   1500
  BD         4  22
  CD         3  50  2  22
  AB         2
  PW         5.65  30
  N         0.042   0.032
  SA             1100
  *

*** FINISH PROCESSING CROSS SECTION - "BRDG "
*** CROSS SECTION "BRDG " WRITTEN TO DISK, RECORD NO. =  3

--- DATA SUMMARY FOR SECID "BRDG " AT SRD =   1400.  ERR-CODE =      0

    SKEW     IHFNO    VSLOPE        EK        CK
      .0        0.     .0020       .50       .00

X-Y COORDINATE PAIRS  (NGP =  13):
      X      Y          X      Y          X      Y          X      Y
   865.4   18.00     878.7   11.34     900.0   11.30    1100.0   10.30
  1215.0    6.80    1250.0    6.20    1300.0    4.35    1350.0    6.15
  1385.0    6.40    1500.0   10.30    1500.0   10.30    1515.4   18.00
```

```
   865.4   18.00

 X-Y MAX-MIN POINTS:
    XMIN      Y        X    YMIN     XMAX      Y        X    YMAX
   865.4   18.00    1300.0   4.35    1515.4   18.00    865.4   18.00

SUBAREA BREAKPOINTS (NSA =  2):
      1100.

ROUGHNESS COEFFICIENTS (NSA =  2):
       .042    .032

BRIDGE PARAMETERS:
 BRTYPE  BRWDTH    LSEL  USERCD  EMBSS   EMBELV  ABSLPL  ABSLPR
    3      50.0    18.00 *******   2.00    22.00    2.00 *******

DESIGN DATA:    BRLEN  LOCOPT  XCONLT  XCONRT
              650.0      1.   1100.   1500.

             GIRDEP  BDELEV   BDSLP   BDSTA
               4.00   22.00 ******* *******

PIER DATA:  NPW =  1    PPCD = 0.
    PELV  PWDTH     PELV  PWDTH     PELV  PWDTH     PELV  PWDTH
     5.65  30.0
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

        SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
        CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
        HEC-18 - EVALUATING SCOUR AT BRIDGES
           *** RUN DATE & TIME: 09-10-92  10:08

*** START PROCESSING CROSS SECTION - "APPR "
  AS   APPR  2100
  *
  HP 2 BRDG  13.82 * * 30000

*** FINISH PROCESSING CROSS SECTION - "APPR "
*** NO ROUGHNESS DATA INPUT, WILL PROPAGATE FROM PREVIOUS CROSS SECTION.
*** CROSS SECTION "APPR " WRITTEN TO DISK, RECORD NO. =  4

--- DATA SUMMARY FOR SECID "APPR " AT SRD =   2100.  ERR-CODE =      0

    SKEW     IHFNO    VSLOPE       EK       CK
      .0       0.     .0020       .50      .00

X-Y COORDINATE PAIRS  (NGP =  17):
      X      Y          X      Y          X      Y          X      Y
      .0   21.70     100.0   17.70     200.0   13.70     500.0   13.45
   900.0   12.70    1100.0   11.70    1215.0    8.20    1250.0    7.60
  1300.0    5.75    1350.0    7.55    1385.0    7.80    1500.0   11.70
  1700.0   12.70    2100.0   13.45    2400.0   13.70    2500.0   17.70
  2600.0   21.70

 X-Y MAX-MIN POINTS:
    XMIN      Y        X    YMIN     XMAX      Y        X    YMAX
      .0   21.70    1300.0   5.75    2600.0   21.70       .0   21.70

SUBAREA BREAKPOINTS (NSA =  3):
```

```
     1100.   1500.

ROUGHNESS COEFFICIENTS (NSA =  3):
      .042    .032    .042

BRIDGE PROJECTION DATA:  XREFLT  XREFRT  FDSTLT  FDSTRT
                        ******* ******* ******* *******
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

     SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
     CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
     HEC-18 - EVALUATING SCOUR AT BRIDGES
       *** RUN DATE & TIME: 09-10-92  10:08



   VELOCITY DISTRIBUTION:  ISEQ =  3;  SECID = BRDG ;  SRD =    1400.

      WSEL     LEW     REW    AREA       K       Q    VEL
     13.82   873.8  1507.0  3286.9  470494.   30000.   9.13

X STA.      873.8     1003.3     1096.9     1150.0     1180.3     1203.9
  A(I)         346.5      305.9      225.0      166.6      149.6
  V(I)          4.33       4.90       6.67       9.00      10.03

X STA.     1203.9     1223.7     1241.9     1259.0     1274.4     1288.4
  A(I)         137.8      133.3      131.0      126.9      123.1
  V(I)         10.89      11.26      11.45      11.82      12.18

X STA.     1288.4     1301.6     1314.7     1329.0     1344.3     1361.3
  A(I)         122.0      120.7      123.8      124.5      131.2
  V(I)         12.29      12.43      12.11      12.05      11.43

X STA.     1361.3     1379.0     1397.3     1418.7     1447.3     1507.0
  A(I)         133.2      133.3      141.9      165.3      245.2
  V(I)         11.26      11.25      10.57       9.07       6.12
1
  *
  HP 1 BRDG  13.54 1 13.54
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

     SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
     CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
     HEC-18 - EVALUATING SCOUR AT BRIDGES
       *** RUN DATE & TIME: 09-10-92  10:08
   CROSS-SECTION PROPERTIES:  ISEQ =  3;  SECID = BRDG ;  SRD =    1400.

    WSEL  SA#     AREA        K   TOPW   WETP  ALPH    LEW     REW     QCR
            1     600.   40797.   226.   226.                  5553.
            2    2510.  392654.   406.   407.                 35385.
   13.54         3110.  433451.   632.   634.  1.16   874.  1506.  36279.
1
  *
  HP 2 APPR  17.36 * * 30000
1
WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS
```

```

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08



    VELOCITY DISTRIBUTION:  ISEQ =  4;  SECID = APPR ;  SRD =    2100.

        WSEL     LEW     REW    AREA       K       Q    VEL
       17.36   108.5  2491.5 11565.0 1414915.   30000.   2.59

 X STA.       108.5      416.1      623.7      798.5      951.8     1077.6
   A(I)          978.0      823.0      752.7      711.6      658.1
   V(I)           1.53       1.82       1.99       2.11       2.28

 X STA.      1077.6     1158.1     1204.1     1241.5     1274.0     1301.7
   A(I)          506.1      373.9      346.5      327.0      309.8
   V(I)           2.96       4.01       4.33       4.59       4.84

 X STA.      1301.7     1330.6     1363.3     1399.1     1443.3     1522.7
   A(I)          318.4      327.1      340.0      368.6      502.7
   V(I)           4.71       4.59       4.41       4.07       2.98

 X STA.      1522.7     1646.7     1803.5     1977.8     2184.8     2491.5
   A(I)          649.2      727.8      749.9      820.2      974.5
   V(I)           2.31       2.06       2.00       1.83       1.54
1
   *
   HP 1 APPR  17.36 1 17.36
1
 WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
 P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08
    CROSS-SECTION PROPERTIES:  ISEQ =  4;  SECID = APPR ;  SRD =    2100.

     WSEL  SA#    AREA        K   TOPW   WETP  ALPH    LEW    REW     QCR
           1    4049.  366963.  992.  992.                   46430.
           2    3467.  680989.  400.   400.                  57923.
           3    4049.  366963.  992.  992.                   46430.
    17.36        11565. 1414915.  2383.  2383.  1.53   108.  2492. 117067.
1
   *
   EX

 +++ BEGINNING PROFILE CALCULATIONS --   1
1
 WSPRO        FEDERAL HIGHWAY ADMINISTRATION - U. S. GEOLOGICAL SURVEY
 P060188          MODEL  FOR  WATER-SURFACE  PROFILE  COMPUTATIONS

       SCOUR EXAMPLE #2 - HYPOTHETICAL EXAMPLE
       CONTRACTION, PIER, AND ABUTMENT SCOUR CALCULATIONS
       HEC-18 - EVALUATING SCOUR AT BRIDGES
         *** RUN DATE & TIME: 09-10-92  10:08

  XSID:CODE   SRDL    LEW     AREA   VHD    HF     EGL    CRWS      Q    WSEL
```

```
     SRD   FLEN    REW       K  ALPH    HO     ERR     FR#     VEL

EXIT :XS   ******   161.    6692.   .57 *****   13.14   11.86  30000.   12.57
    750. ******  2439.  670723.  1.83 ***** *******     .62    4.48

FULLV:FV    650.   161.    6706.   .57  1.30   14.44 *******  30000.   13.88
   1400.   650.  2439.  672489.  1.83   .00     .01     .62    4.47
       <<<<<THE ABOVE RESULTS REFLECT "NORMAL" (UNCONSTRICTED) FLOW>>>>>

APPR :AS    700.   161.    6700.   .57  1.39   15.84 *******  30000.   15.27
   2100.   700.  2439.  671817.  1.83   .00     .00     .62    4.48
       <<<<<THE ABOVE RESULTS REFLECT "NORMAL" (UNCONSTRICTED) FLOW>>>>>

         <<<<<RESULTS REFLECTING THE CONSTRICTED FLOW FOLLOW>>>>>

 XSID:CODE   SRDL    LEW     AREA   VHD    HF     EGL    CRWS      Q    WSEL
     SRD   FLEN    REW       K  ALPH    HO     ERR     FR#     VEL

BRDG :BR    650.   874.    3107.  2.69  2.01   16.23   13.27  30000.   13.54
   1400.   650.  1506.  432822.  1.86  1.07     .00    1.05    9.66

   TYPE PPCD FLOW     C    P/A    LSEL   BLEN   XLAB   XRAB
     3.   0.   1.   .734   .076   18.00   650.   879.  1500.

 XSID:CODE   SRDL    LEW     AREA   VHD    HF     EGL    CRWS      Q    WSEL
     SRD   FLEN    REW       K  ALPH    HO     ERR     FR#     VEL

APPR :AS    650.   108.   11574.   .16  1.02   17.52   14.56  30000.   17.36
   2100.   697.  2492. 1416461.  1.52   .28    -.02     .26    2.59

     M(G)   M(K)      KQ   XLKQ   XRKQ    OTEL
     .722   .430  811434.   891.  1521.    17.08

                <<<<<END OF BRIDGE COMPUTATIONS>>>>>
  ER

1  NORMAL  END  OF  WSPRO  EXECUTION.
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APPENDIX H

COMPREHENSIVE EXAMPLE SCOUR PROBLEM (ENGLISH UNITS)

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APPENDIX H

Comprehensive Example Scour Problem (English Units)

H.1 GENERAL DESCRIPTION OF PROBLEM

This example problem is taken from a paper by Arneson et al.[77] FHWA's WSPRO computer program was used to obtain the hydraulic variables. The program uses 20 stream tubes to give a quasi 2-dimensional analysis. Each stream tube has the same discharge (1/20 of the total discharge). The stream tubes provide the velocity distribution across the flow and the program has excellent bridge routines. The problem presented here is an English version of the comprehensive scour problem in Chapter 8, which is worked in metric (SI) units. The solution follows Steps 1-7 of the specific design approach of Chapter 2 (Section 2.4).

A 650-foot long bridge (Figure H.1) is to be constructed over a channel with spill-through abutments (slope of 1V:2H). The left abutment is set approximately 200 ft back from the channel bank. The right abutment is set at the channel bank. The bridge deck is set at elevation 22 ft and has a girder depth of 4 ft. Six round-nose piers are evenly spaced in the bridge opening. The piers are 5 ft thick, 40 ft long, and are aligned with the flow. The 100-year design discharge is 30,000 cfs. The 500-year flow of 51,000 cfs was estimated by multiplying the Q_{100} by 1.7 since no hydrologic records were available to predict the 500-year flow.

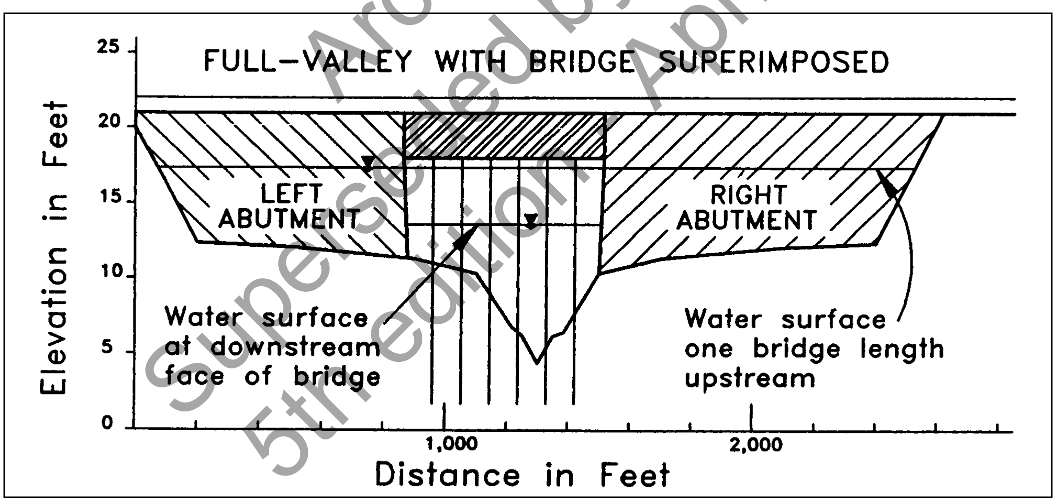

Figure H.1. Cross section of proposed bridge.

H.2 STEP 1: DETERMINE SCOUR ANALYSIS VARIABLES

From Level 1 and Level 2 analysis: a site investigation of the crossing was conducted to identify potential stream stability problems at this crossing. Evaluation of the site indicates that the river has a relatively wide floodplain. The floodplain is well vegetated with grass and trees; however, the presence of remnant channels indicates that there is a potential for lateral shifting of the channel.

The bridge crossing is located on a relatively straight reach of channel. The channel geometry is relatively the same for approximately 1,000 ft up- and downstream of the bridge crossing. The D_{50} of the bed material and overbank material is approximately 2 mm. The maximum grain size of the bed material is approximately 8 mm. The specific gravity of the bed material was determined to be equal to 2.65.

The river and crossing are located in a rural area with the primary land use consisting of agriculture and forest.

Review of bridge inspection reports for bridges located upstream and downstream of the proposed crossing indicates no long-term aggradation or degradation in this reach. At the bridge site, bedrock is approximately 150 ft below the channel bed.

Since this is a sand-bed channel, no armoring potential is expected. Furthermore, the bed for this channel at low flow consists of dunes which are approximately 1 to 1.5 ft high. At higher flows, above the Q_5, the bed will be either plane bed or antidunes.

The left and right banks are relatively well vegetated and stable; however, there are isolated portions of the bank which appear to have been undercut and are eroding. Brush and trees grow to the edge of the banks. Banks will require riprap protection if disturbed. Riprap will be required upstream of the bridge and extend downstream of the bridge.

H.2.1 Hydraulic Characteristics

Hydraulic characteristics at the bridge were determined using WSPRO.[15] Three cross sections were used for this analysis and are denoted as "EXIT" for the section downstream of the bridge, "FULLV" for the full-valley section at the bridge, and "APPR" for the approach section located one bridge length upstream of the bridge. The bridge geometry was superimposed on the full-valley section and is denoted "BRDG." Values used for this example problem are based on the output from the WSPRO model which is presented in Appendix G. Specific values for scour analysis variables are given for each computation separately and cross referenced to the line numbers of the WSPRO output.

The HP2 option was used to provide hydraulic characteristics at both the bridge and approach sections. This WSPRO option subdivides the cross section into 20 equal conveyance tubes. Figures H.2 and H.3 illustrate the location of these conveyance tubes for the approach and bridge cross section, respectively. Figure H.4 illustrates the average velocities in each conveyance tube and the contraction of the flow from the approach section through the bridge. Figure H.4 also identifies the equal conveyance tubes of the approach section which are cut off by the abutments.

Hydraulic variables for performing the various scour computations were determined from the WSPRO output (Appendix G) and from Figures H.2, H.3, and H.4. These variables, which will be used to compute contraction scour and local scour, are presented in Tables H.1 through H.6.

Contraction scour will occur both in the main channel and on the left overbank of the bridge opening. For the main channel, contraction scour could be either clear-water or live-bed depending on the magnitude of the channel velocity and the critical velocity for sediment movement. A computation will be performed to determine the sediment transport characteristics of the main channel and the appropriate contraction scour equation.

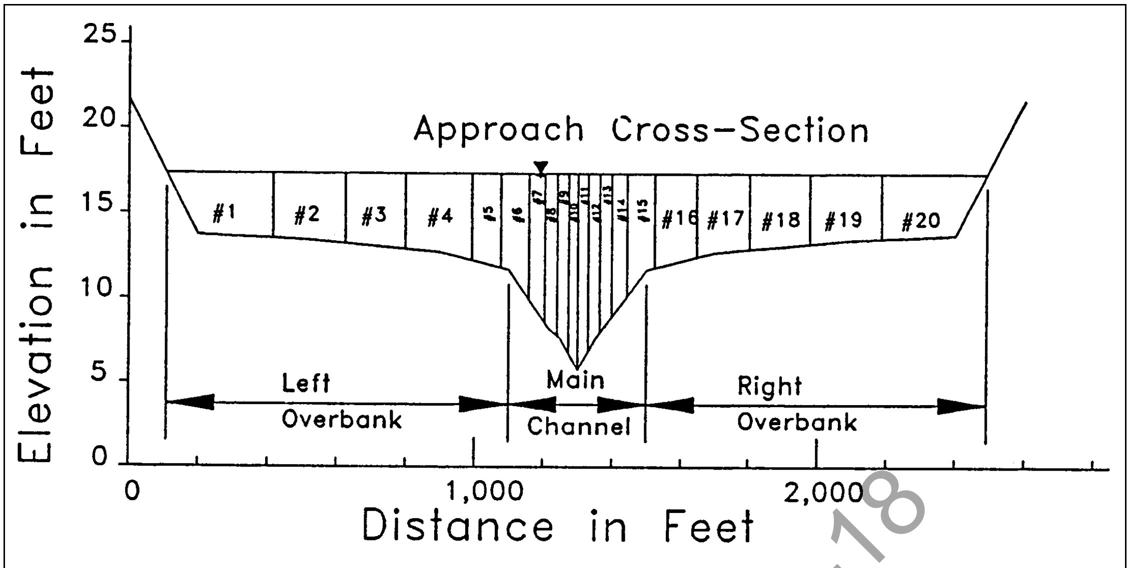

Figure H.2. Equal conveyance tubes of approach section.

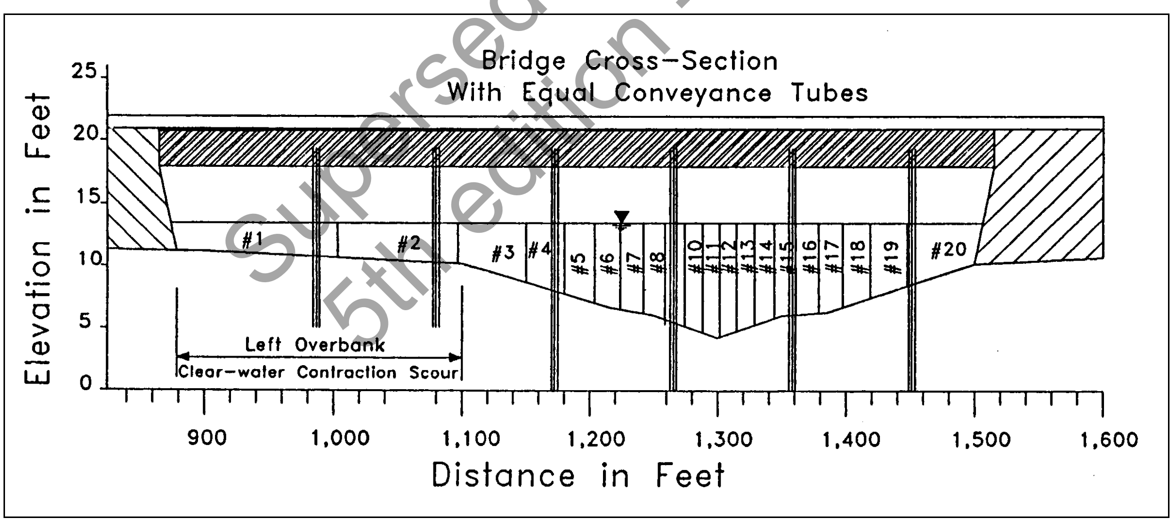

Figure H.3. Equal conveyance tubes of bridge section.

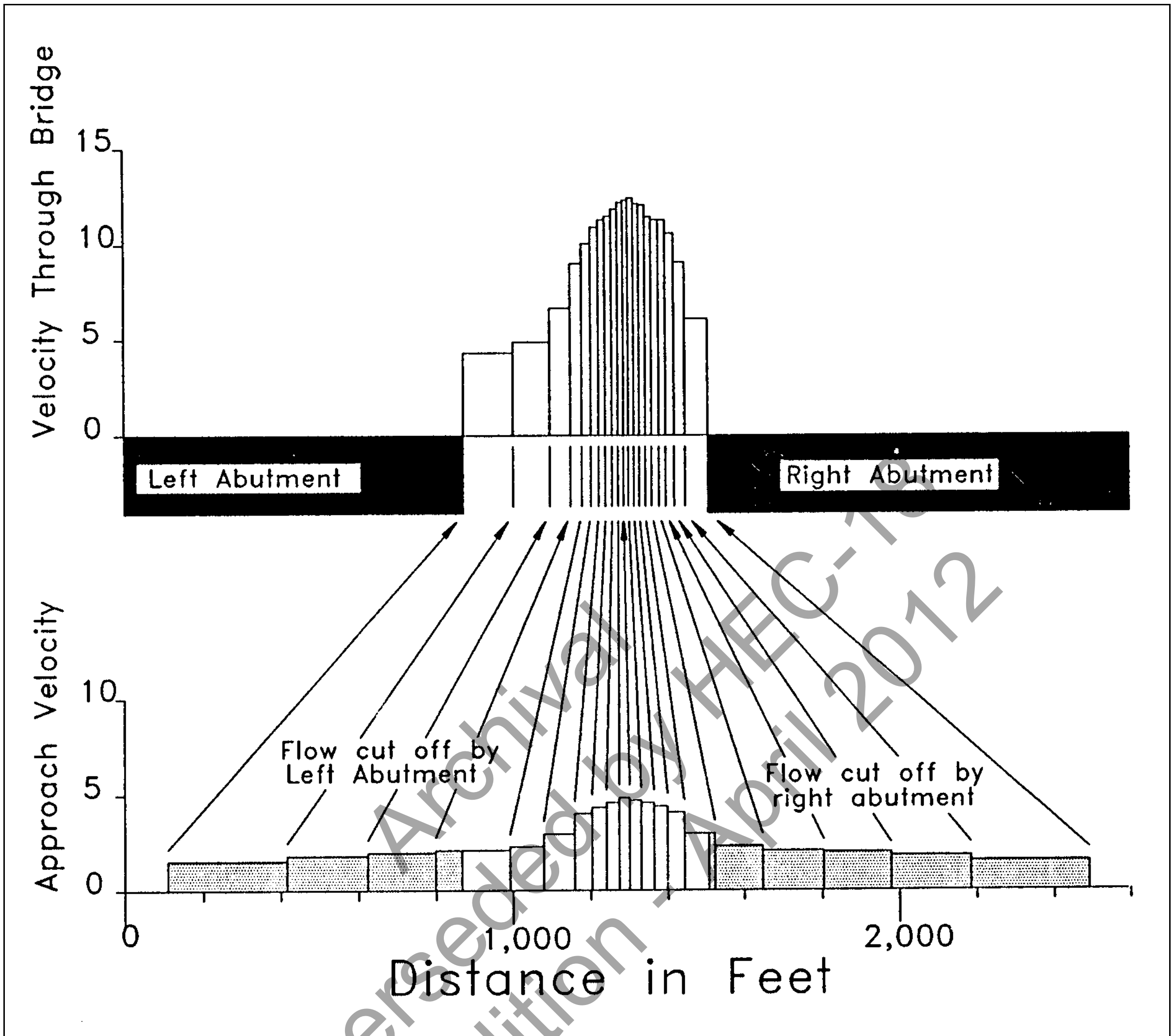

Figure H.4. Plan view of equal conveyance tubes showing velocity distribution at approach and bridge sections.

Table H.1. Hydraulic Variables from WSPRO for Estimation of Live-bed Contraction Scour.		
		Remarks
Q (cfs)	30,000	Total discharge, line 5 of WSPRO input or Line 11 of WSPRO output.
K_1 (Approach)	680,989	Conveyance of main channel of approach. Line 295 of WSPRO output, SA#2.
K_{total} (Approach)	1,414,915	Total conveyance of approach section. Line 297 of WSPRO output.
W_1 or TOPW (Approach) (ft)	400	Topwidth of flow (TOPW). Assumed to represent active live bed width of approach. Line 295 of WSPRO output, SA#2.
A_c (Approach) (ft^2)	3,467	Area of main channel approach section. Line 295, SA#2.
WETP (Approach) (ft)	400	Wetted perimeter of main channel approach section. Line 295 of WSPRO output, SA#2.
K_c (Bridge)	392,654	Conveyance of main channel through bridge. Line 244 of WSPRO output, SA#2.
K_{total} (Bridge)	433,451	Total conveyance through bridge. Line 245 of WSPRO output.
A_c (Bridge) (ft^2)	2,510	Area of the main channel, bridge section. Line 244 of WSPRO output, SA #2.
W_c (Bridge) (ft^2)	400	Channel width at the bridge. Difference between subarea break-points defining banks at bridge, line 93 of WSPRO output.
W_2 (Bridge) (ft)	380	Channel width at bridge, less 4 channel pier widths (6.08 m).
S_f (ft/ft)	0.002	Average unconstricted energy slope. Defined as the headloss listed on line 318 or 322 of the WSPRO output divided by the distance between cross sections listed on lines 316, 319, and 323.

Table H.2. Hydraulic Variables from WSPRO for Estimation of Clear-water Contraction Scour on Left Overbank.		
		Remarks
Q (cfs)	30,000	Total discharge, (see Table H.1).
Q_{chan} (Bridge) (cfs)	27,176.4	Flow in main channel at bridge. Determined in live-bed computation of Step 3A.
Q_2 (Bridge) (cfs)	2,823.6	Flow in left overbank through bridge. Determined by subtracting Q_{chan} (listed above) from total discharge through bridge.
D_m (Bridge Overbank) (ft)	0.00825	Grain size of left overbank area. $D_m = 1.25\ D_{50}$.
$W_{setback}$ (Bridge)(ft)	226	Topwidth of left overbank area (SA #1) at bridge. Line 243, of WSPRO output.
$W_{contracted}$ (Bridge) (ft)	216	Set back width less two pier widths (10 ft)
A_{left} (Bridge) (ft^2)	600	Area of left overbank at the bridge. Line 243 of WSPRO output, SA #1.

Table H.3. Hydraulic Variables from WSPRO for Estimation of Pier Scour (Conveyance Tube Number 12).		
		Remarks
V_1 (ft/s)	12.43	Velocity in conveyance tube #12. Line 224 of WSPRO output.
Y_1 (ft)	9.21	Mean depth of tube #12. Computed as area divided by topwidth of conveyance tube.

Table H.4. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using Froehlich's Equation for Left Abutment.		
		Remarks
Q (cfs)	30,000	Total discharge (Table H.1)
q_{tube} (cfs)	1,500	Discharge per equal conveyance tube, defined as total discharge divided by 20.
#Tubes	3.5	Number of approach section conveyance tubes which are obstructed by left abutment. Determined by super-imposing abutment geometry onto the approach section (Figure H.4)
Q_e (cfs)	5,250	Flow in left overbank obstructed by left abutment and approach embankment. Determined by multiplying # Tubes and q_{tube}.
A_e (left abut.) (ft^2)	2,910	Area of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 266 of WSPRO output.
L (ft)	766.65	Length of abutment projected into flow, determined by adding top widths of approach section conveyance tubes number 1, 2, 3, and half of tube 4. Line 265 of WSPRO output.
L′ (ft)	536.6	Length of active flow obstructed by embankment. Width of approach section conveyance tube directly upstream of abutment times the number of conveyance tubes blocked by the embankment (951.8-798.5) x 3.5 = 536.6. Note: Conveyance tube widths from line 265 of WSPRO output.

Table H.5. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Left Abutment.		
		Remarks
V_{tube} (ft/s) (Bridge x-Section)	4.33	Mean velocity of conveyance tube #1, adjacent to left abutment. Line 216 of WSPRO output.
y_1 (ft) (Bridge x-Section)	2.68	Average depth of conveyance tube #1. Computed as area divided by topwidth of conveyance tube

Table H.6. Hydraulic Variables from WSPRO for Estimation of Abutment Scour Using HIRE Equation for Right Abutment.		
		Remarks
V_{tube} (ft/s)	6.12	Mean velocity of conveyance tube 20, adjacent to right abutment. Line 228 of WSPRO output.
y_1 (ft)	4.11	Average depth of conveyance tube 20. Computed as area divided by topwidth of conveyance tube.

In the overbank area adjacent to the left abutment, clear-water scour will occur. This is because the overbank areas upstream of the bridge are vegetated, and because the velocities in these areas will be low. Thus, returning overbank flow which will pass under the bridge adjacent to the left abutment will not be transporting significant amounts of material to replenish the scour on the left overbank adjacent to the left abutment.

Because of this, two computations for contraction scour will be required. The first computation, which will be illustrated in Step 3A will determine the magnitude of the contraction scour in the main channel. The second computation, which is illustrated in Step 3B will utilize the clear-water equation for the left overbank area. Hydraulic data for these two computations are presented in Tables H.1 and H.2 for the channel and left overbank contraction scour computations, respectively.

Table H.3 lists the hydraulic variables which will be used to estimate the local scour at the piers (Step 5). These hydraulic variables were determined from a plot of the velocity distribution derived from the WSPRO output (Figure H.5). For this example the highest velocities and flow depths in the bridge cross section will be used (at conveyance tube number 12). Only one pier scour computation will be completed because the possibility of thalweg shifting and lateral migration will require that all of the piers be set assuming that any pier could be subjected to the maximum scour producing variables.

Local scour at the left abutment and right abutment will be illustrated in Steps 6A and B using the HIRE equation. Scour variables derived from the WSPRO output for these computations are presented in Tables H.4 and H.5.

H.3 STEP 2: ANALYZE LONG-TERM BED ELEVATION CHANGE

Evaluation of stage discharge relationships and cross sectional data obtained from other agencies do not indicate progressive aggradation or degradation. Also, long-term aggradation or degradation are not evident at neighboring bridges. Based on these observations, the channel is relatively stable vertically, at present. Furthermore, there are no plans to change the local land use in the watershed. The forested areas of the watershed are government-owned and regulated to prevent wide spread fire damage, and instream gravel mining is prohibited. These observations indicate that future aggradation or degradation of the channel, due to changes in sediment delivery from the watershed, are minimal.

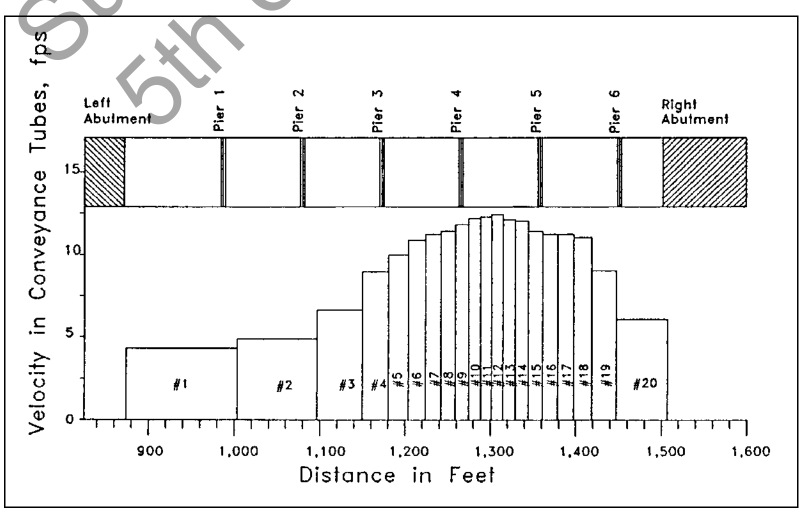

Figure H.5. Velocity distribution at bridge crossing.

Based on these observations, and due to the lack of other possible impacts to the river reach, it is determined that the channel will be relatively stable vertically at the bridge crossing and long-term aggradation or degradation potential is considered to be minimal. However, there is evidence that the channel is unstable laterally. This will need to be considered when assessing the total scour at the bridge.

H.4 STEP 3A: COMPUTE THE MAGNITUDE OF THE GENERAL (CONTRACTION) SCOUR IN MAIN CHANNEL

As a precursor to the computation of contraction scour in the main channel under the bridge, it is first necessary to determine whether the flow condition in the main channel is either live-bed or clear-water. This is determined by comparing the critical velocity for sediment movement at the approach section to the average channel velocity of the flow at the approach section as computed using the WSPRO output. This comparison is conducted using the average velocity in the main channel of the approach section to the bridge. If the average computed channel velocity is greater than the critical velocity, the live-bed equation should be used. Conversely, if the average channel velocity is less than the critical velocity, the clear-water equation is applicable. The following computations are based on the quantities tabulated in Table H.1.

The discharge in the main channel of the approach section is determined from the ratio of the conveyance in the main channel to the total conveyance of the approach section. By multiplying this ratio by the total discharge, the discharge in the main channel at the approach section (Q_1) is computed.

$$Q_1 = Q\ (K_1 / K_{total}) = 30{,}000\ \text{cfs} \left(\frac{680{,}989}{1{,}414{,}915} \right)$$

$$Q_1 = 14{,}439\ \text{cfs}$$

The average velocity in the main channel of the approach section is determined by dividing the discharge computed in Equation H.1 by the cross-sectional area of the main channel.

$$V_1 = (Q_1 / A_c) = \left(\frac{14{,}439}{3{,}467} \right) = 4.16\ \text{ft/s}$$

The average flow depth in the approach section is determined by dividing the flow area by the topwidth of the channel.

$$y_1 = \left(A_1 / \text{TOPW} \right) = \left(\frac{3{,}467}{400} \right) = 8.7\ \text{ft}$$

The channel velocity is compared to the critical velocity of the D_{50} size for sediment movement (V_c) to determine whether the flow condition is either clear-water or live-bed.

$$V_c = 11.2\ y_1^{1/6}\ D_{50}^{1/3}$$

$$V_c = 11.2\ (8.7\ \text{ft})^{1/6}\ (0.0066\ \text{ft})^{1/3}$$

$$V_c = 3.0 \text{ ft / s}$$

Since the average velocity in the main channel is greater than the critical velocity ($V_1 > V_c$), the flow condition will be live-bed. The following computations illustrate the computation of the contraction scour using the live-bed equation.

The following computation determines the mode of bed material transport and the factor k_1. All hydraulic parameters which are needed for this computation are listed in Table H.1.

The hydraulic radius of the approach channel is:

$$R = \frac{A_c}{WETP} = \frac{3{,}467 \text{ ft}^2}{400 \text{ ft}} = 8.7 \text{ ft}$$

Notice that the hydraulic radius of the approach is equal to the average flow depth computed earlier (Equation H.3). This condition indicates that the channel is wide with its width greater than 10 times the flow depth. **If the width was less than 10 times the average flow depth, the channel could not be assumed to be wide and the hydraulic radius would deviate from the average flow depth**.

The average shear stress on the channel bed is:

$$\tau_o = \gamma \, R \, S$$

$$\tau_o = (62.4 \text{ lb/ft}^3)\,(8.7 \text{ ft})\,(0.002 \text{ ft/ft}) = 1.08 \text{ lb/ft}^2$$

The shear velocity in the approach channel is:

$$V_* = (\tau_o / \rho)^{0.5} = (1.08 / 1.94)^{0.5} = 0.75 \text{ ft / s}$$

Bed material is sand with $D_{50} = 0.0066$ ft

Fall velocity (ω) = 0.9 ft/s from Figure 5.8 at 20°C and D_s = 2 mm

Therefore

$$\frac{V_*}{\omega} = \frac{0.75}{0.9} = 0.83$$

From the above, the coefficient k_1 is determined (from the discussion for Equation 5.2) to be equal to 0.64 which indicates that the mode of bed material transport is a mixture of suspended and contact bed material discharge.

The discharge in the main channel at the bridge (Q_2) is determined from the ratio of conveyances for the bridge section. This procedure for obtaining the discharge is similar to the procedure used to obtain the discharge in the main channel of the approach which was previously illustrated in Equation H.1.

$$Q_2 = Q(K_2 / K_{total}) = 30{,}000 \text{ cfs}\left(\frac{392{,}654}{433{,}451}\right)$$

$$Q_2 = 27{,}176 \text{ cfs}$$

The channel widths at the approach and bridge section are given in Table H.1. Therefore all parameters to determine live-bed contraction scour have been determined and Equation 5.2 can be employed.

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7}\left(\frac{W_1}{W_2}\right)^{k_1}$$

$$\frac{y_2}{8.7} = \left(\frac{27{,}176}{14{,}439}\right)^{6/7}\left(\frac{400}{380}\right)^{0.64} = 1.78$$

$$y_2 = (8.7)(1.78) = 15.5 \text{ ft}$$

Live-bed contraction scour is calculated by subtracting the flow depth in the bridge (y_0) from y_2. The bridge channel flow depth (y_0) is the area divided by the topwidth, y_0 = 2510 ft^2/400 ft = 6.3 ft. Therefore, the depth of contraction scour in the main channel is:

$$y_s = y_2 - y_0 = 15.5 \text{ ft} - 6.3 \text{ ft} = 9.2 \text{ ft}$$

This amount of contraction scour is large and could be minimized by increasing the bridge opening, providing for relief bridges in the overbank, or in some cases, providing for highway approach overtopping.

If this were the design of a new bridge, the excessive backwater (2 ft) would require a change in the design to meet FEMA backwater requirements. The increase in backwater is obtained by subtracting the elevation given in line 322 from the elevation given in line 340 in Appendix G. However, in the evaluation of an existing bridge for safety from scour, this amount of contraction scour could occur and the scour analysis should proceed.

H.5 STEP 3B: COMPUTE GENERAL (CONTRACTION) SCOUR FOR LEFT OVERBANK

Clear-water contraction scour will occur in the overbank area between the left abutment and the left bank of bridge opening. Although the bed material in the overbank area is soil, it is protected by vegetation. Therefore, there would be no bed-material transport into the set-back bridge opening (clear-water conditions). The subsequent computations are based on the discharge and depth of flow passing under the bridge in the left overbank. These hydraulic variables were determined from the WSPRO output and are tabulated in Table H.2.

Computation of clear-water contraction scour (Equation 5.4)

$$y_2 = \left[\frac{0.0077\,Q^2}{(D_m^{2/3}\ W_{contracted}{}^2)}\right]^{3/7}$$

Computation of contraction scour flow depth in left overbank area under the bridge, y_2:

$$Y_2 = \left[\frac{0.0077\,(2{,}823.6\,\text{CFS})^2}{(0.0083\,\text{FT})^{2/3}\,(216\,\text{FT})^2}\right]^{3/7} = 4.4\,\text{FT}$$

Computation of average flow depth in left overbank bridge section, y_0:

$$y_0 = \frac{A}{TOPW} = \frac{(600\ \text{ft}^2)}{(226\ \text{m})} = 2.7\,\text{ft}$$

Therefore, the clear-water contraction scour in the left overbank of the bridge opening is:

$$y_s = y_2 - y_0 = 4.4\,\text{ft} - 2.7\,\text{ft} = 1.7\,\text{ft}$$

H.6 STEP 4: COMPUTE THE MAGNITUDE OF OTHER GENERAL SCOUR COMPONENTS

The crossing is on a relatively straight reach with no channel braiding, and there are no downstream controls of water surface elevations. Thus, the other general scour components (bend scour, confluence scour, etc) will not be a factor.

H.7 STEP 5: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT PIERS

It is anticipated that any pier under the bridge could potentially be subject to the maximum flow depths and velocities derived from the WSPRO hydraulic model (Table H.3). Therefore, only one computation for pier scour is conducted and assumed to apply to each of the six piers for the bridge. This assumption is appropriate based on the fact that the thalweg is prone to shifting and because there is a possibility of lateral channel migration.

H.7.1 Computation of Pier Scour

The Froude Number for the pier scour computation is based on the hydraulic characteristics of conveyance tube number 12. Therefore:

$$Fr_1 = \frac{V}{(g\ y_1)^{0.5}} = \frac{12.43\ \text{ft/s}}{[(32.2\ \text{ft/s}^2)\ (9.21\text{ft})]^{0.5}}$$

$$Fr_1 = 0.72$$

For a round-nose pier, aligned with the flow and sand-bed material:

$$K_1 = K_2 = K_4 = 1.0$$

For plane-bed condition:

$$K_3 = 1.1$$

Using Equation 6.3:

$$\frac{y_s}{y_1} = 2.0\ K_1\ K_2\ K_3\ K_4\ \left(\frac{a}{y_1}\right)^{0.65}\ Fr_1^{0.43}$$

$$\frac{y_s}{9.21\text{ft}} = 2\ (1)\ (1)\ (1.1)\ (1)\ \left(\frac{5.0\text{ ft}}{9.21\text{ft}}\right)^{0.65}\ (0.72)^{0.43}$$

$$\frac{y_s}{9.21} = 1.28$$

$$y_s = 11.8\ \text{ft}$$

From the above computation the maximum local pier scour depth will be 11.8 ft.

H.7.2 Correction for Angle of Attack

The above computation assumes that the piers are aligned with the flow (skew angles are less than 5°). However, if the piers were skewed to the flow by more than 5°, the value of y_s/y_1, as computed above, would need to be adjusted by K_2. The following computations illustrate the adjustment for piers skewed 10°.

$$\frac{L}{a} = \frac{40\text{ ft}}{5\text{ ft}} = 8$$

K_2 can then be obtained by using Equation 6.4 for an L/a of 8 and a 10° angle of attack. For this example, K_2=1.67. Applying this correction:

$$\frac{y_s}{9.21} = 1.67\ (1.28) = 2.1$$

$$y_s = 19.3\text{ ft}$$

Therefore, the maximum local pier scour depth for a pier angled 10° to the flow is 19.3 ft.

H.7.3 Discussion of Pier Scour Computation

Although the estimated local pier scour would probably not occur at each pier, the possibility of thalweg shifting, which was identified in the Level 1 analysis, precludes setting the piers at different depths even if there were a substantial savings in cost. This is because any of the piers could be subjected to the worst-case scour conditions.

It is also important to assess the possibility of lateral migration of the channel. This possibility can lead to directing the flow at an angle to the piers, thus increasing local scour. Countermeasures to minimize this problem could include riprap for the channel banks both up- and downstream of the bridge, and installation of guide banks to align flow through the bridge opening.

The possibility of lateral migration precludes setting the foundations for the overbank piers at a higher elevation. Therefore, in this example the foundations for the overbank piers should be set at the same elevation as the main channel piers.

H.8 STEP 6A: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT LEFT ABUTMENT

H.8.1 Computation of Abutment Scour Depth Using Froehlich's Equation

For spill-through abutments, K_1 = 0.55. For this example, the abutments are set perpendicular to the flow; therefore, K_2 = 1.0. Abutment scour can be estimated using Froehlich's equation with data derived from the WSPRO output (Table H.4).

$$y_a = \frac{A_e}{L} = \frac{2{,}910\ \text{ft}^2}{766.6\ \text{ft}} = 3.80\ \text{ft}$$

The y_a value at the abutment is assumed to be the average flow depth in the overbank area. It is computed as the cross-sectional area of the left overbank cut off by the left abutment divided by the distance the left abutment protrudes into the overbank flow.

The average velocity of the flow in the left overbank (Figure H.4) which is cut off by the left abutment is computed as the discharge cutoff by the abutment divided by the area of the left overbank cut off by the left abutment.

$$V_e = \frac{Q_e}{A_e} = \frac{5{,}250\ \text{cfs}}{2{,}910\ \text{ft}^2} = 1.8\ \text{ft/s}$$

Using these parameters, the Froude Number of the overbank flow is:

$$Fr = \frac{V_e}{(g\ y_a)^{1/2}} = \frac{1.8\ \text{ft/s}}{[(32.2\ \text{ft/s}^2)\ (3.8\ \text{ft})]^{0.5}}$$

Fr = 0.16

Using Froehlich's equation (Equation 7.1):

$$\frac{y_s}{y_a} = 2.27\ K_1\ K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1$$

$$\frac{y_s}{3.8} = 2.27\,(0.55)\ (1.0) \left(\frac{536.6}{3.8}\right)^{0.43} (0.17)^{0.61} + 1$$

$$\frac{y_s}{3.8\,\text{ft}} = 4.56$$

$$y_s = 17.3\,\text{ft}$$

Using Froehlich's equation, the abutment scour at the left abutment is computed to be 17.3 ft.

H.8.2 Computation of Abutment Scour Depth Using the HIRE Equation

The HIRE equation for abutment is applicable for this situation because L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the abutment end which is listed in Table H.5. Therefore, the Froude Number of this flow is:

$$Fr_1 = \frac{4.33\,\text{ft/s}}{[(32.2\,\text{ft/s}^2)\,(2.68\,\text{ft})]^{0.5}} = 0.47$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0 (Equation 7.2):

$$\frac{y_s}{2.68\,\text{ft}} = 4\ Fr_1^{0.33} = 4\ (0.47)^{0.33} = 3.12$$

$$y_s = 8.4\ \text{ft}$$

From the above computation, the depth of scour at the left abutment as computed using the HIRE equation, is 8.4 ft.

H.9 STEP 6B: COMPUTE THE MAGNITUDE OF LOCAL SCOUR AT RIGHT ABUTMENT

The HIRE equation for abutment is also applicable for the right abutment since L/y_1 is greater than 25.

The HIRE equation is based on the velocity and depth of the flow passing through the bridge opening adjacent to the end of the right abutment and listed in Table H.6. The Froude Number of this flow is:

$$Fr_1 = \frac{6.12\,\text{ft/s}}{[(32.2\,\text{ft/s}^2)\,(4.11\text{ft})]^{0.5}} = 0.53$$

Using the HIRE equation with K_1 = 0.55 and K_2 = 1.0:

$$\frac{y_s}{4.11\text{ft}} = 4\,Fr_1^{0.33} = 4\,(0.53)^{0.33} = 3.24$$

$y_s = 13.3\,\text{ft}$

From the above computation, the depth of scour at the right abutment, as computed using the HIRE equation is 13.3 ft.

H.10 DISCUSSION OF ABUTMENT SCOUR COMPUTATIONS

Abutment scour as computed using the Froehlich equation[70] will generally result in deeper scour predictions than will be experienced in the field. These scour depths could occur if the abutments protruded into the main channel flow, or when a uniform velocity field is cut off by the abutment in a manner that most of the returning overbank flow is forced to return to the main channel at the abutment end. For most cases, however, when the overbank area, channel banks and area adjacent to the abutment are well vegetated, scour depths as predicted with the Froehlich equation will probably not occur.

All of the abutment scour computations (left and right abutments) assumed that the abutments were set perpendicular to the flow. If the abutments were angled to the flow, a correction utilizing K_2 would be applied to Froehlich's equation and to the equation from HDS 6.[22] However the adjustment for skewed abutments is minor when compared to the magnitude of the computed scour depths. For example, if the abutments for this example problem were angled 30° upstream ($\theta = 90° + 30° = 120°$), the correction for skew would increase the computed depth of abutment scour by no more than 3 to 4 percent for the Froehlich and HIRE equation, respectively.

H.11 STEP 7: PLOT TOTAL SCOUR DEPTH AND EVALUATE DESIGN

As a final step, the results of the scour computations are plotted on the bridge cross section and carefully evaluated (Figure H.6). For this example, only the computations for pier scour with piers aligned with the flow were plotted and the abutment scour computations reflect the results from the HIRE equation. The topwidth of the local scour holes is suggested as 2.0 y_s.

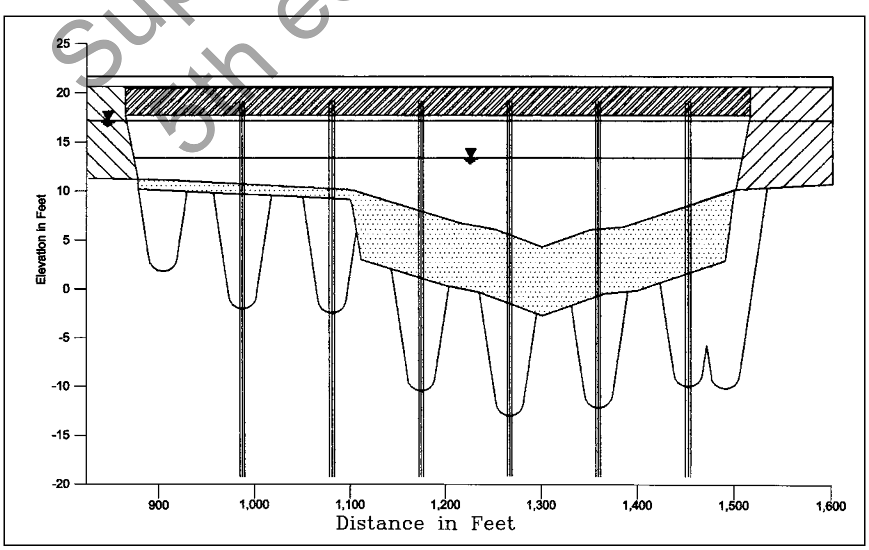

Figure H.6. Plot of total scour for example problem.

It is important to evaluate carefully the results of the scour computations. For example, although the total scour plot indicates that the total scour at the overbank piers is less than for the channel piers, this does not indicate that the foundations for the overbank piers can be set at a higher elevation. Due to the possibility of channel and thalweg shifting, all of the piers should be set to account for the maximum total scour. Also, the computed contraction scour is distributed uniformly across the channel in Figure H.6. However, in reality this may not be what would happen. With the flow from the overbank area returning to the channel, the contraction scour could be deeper at both abutments. The use of guide banks would distribute the contraction scour more uniformly across the channel. This would make a strong case for guide banks in addition to the protection they would provide to the abutments. The stream tube velocities could be used to distribute the scour depths across this section.

The plot of the total scour also indicates that there is a possibility of overlapping scour holes between the sixth pier and right abutment, and it is not clear from where the right abutment scour should be measured, since the abutment is located at the channel bank. Both of these uncertainties should be avoided for replacement and new bridges whenever possible. Consequently, it would be advisable to set the right abutment back from the main channel. This would also tend to reduce the magnitude of contraction scour in the main channel.

The possibility of lateral migration of the channel will have an adverse effect on the magnitude of the pier scour. This is because lateral migration will most likely skew the flow to the piers. This problem can be minimized by using circular piers. An alternative approach would be to install guide banks to align the flow through the bridge opening.

A final concern relates to the location and depth of contraction scour in the main channel near the second pier and toe of the right abutment. At these locations, contraction scour in the main channel could increase the bank height to a point where bank failure and sloughing would occur. It is recommended that the existing bank lines be protected with revetment (i.e., riprap, gabions, etc.). Since the river has a history of channel migration, the bridge inspection and maintenance crews should be briefed on the nature of this problem so that any lateral migration can be identified.

The plot of the scour prism in Figure H.6 should be reploted to show the potential for the scour to occur at any location in the bridge opening. This is shown in Figure H.7

H.12 COMPLETE THE GENERAL DESIGN PROCEDURE

This design problem uses Steps 1 through 7 of the specific design approach (Chapter 2) and completes Steps 1 through 6 of the general design procedure in Chapter 2. The design must now proceed to Steps 7 and 8, which include bridge foundation analysis and consideration of the check for superflood. This is not done for this example problem.

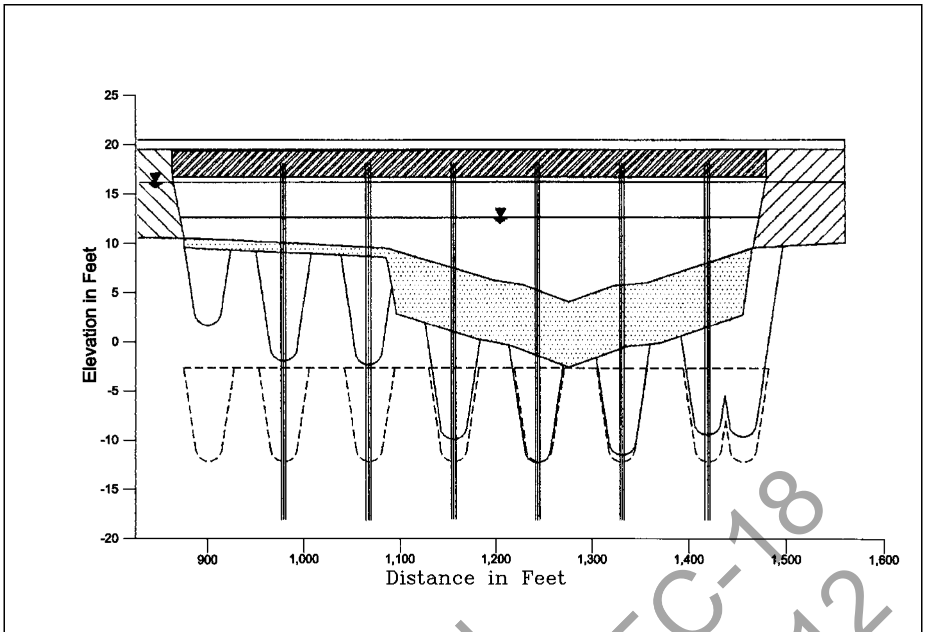

Figure H.7. Revised plot of total scour for example problem.

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APPENDIX I

FHWA TECHNICAL ADVISORY T 5140.23

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EVALUATING SCOUR AT BRIDGES

T 5140.23 October 28, 1991

1. PURPOSE. To provide guidance on developing and implementing a scour evaluation program for:

   a. designing new bridges to resist damage resulting from scour;

   b. evaluating existing bridges for vulnerability to scour;

   c. using scour countermeasures; and

   d. improving the state-of-practice of estimating scour at bridges.

2. CANCELLATION. Technical Advisory T 5140.20, Scour at Bridges, dated September 16, 1988, is cancelled.

3. BACKGROUND.

   a. The need to minimize future flood damage to the Nation's bridges requires that additional attention be devoted to developing and implementing improved procedures for designing, protecting and inspecting bridges for scour. (See National Bridge Inspection Standards, 23 CFR 650 Subpart C.) Current information on this subject has been assembled in the Federal Highway Administration (FHWA) design publication Hydraulic Engineering Circular (HEC) 18, "Evaluating Scour at Bridges," FHWA-IP-90-017 (*FHWA NHI 01-001, fourth edition*).

   b. Paragraph 4 contains the FHWA recommendations for developing and implementing a scour evaluation program. The recommendations have been developed based on the review and evaluation of the existing policies and guidance pertaining to bridge scour set forth in paragraph 5. The procedures in HEC 18 provide approaches for implementing these recommendations.

4. RECOMMENDATIONS FOR DEVELOPING AND IMPLEMENTING A SCOUR EVALUATION PROGRAM. Every bridge over a waterway, whether existing or under design, should be evaluated as to its vulnerability to scour in order to determine the prudent measures to be taken for its protection. Most waterways can be expected to experience scour over a bridge's service life (which could approach 100 years). Exceptions might include waterways in massive, competent rock formations where scour and erosion occur on a scale that is measured in centuries. [See HEC 18, Chapter 2 (*Chapter 3 in the fourth edition*)]. The added cost of making a bridge less vulnerable to scour is small when compared to the total cost of a failure which can easily be two or three times the original cost of the bridge. Moreover, the need to ensure public safety and to minimize the adverse effects stemming from bridge closures requires the best effort to improve the state-of-practice of designing and maintaining bridge foundations to resist the effects of scour. The recommendations listed below summarize the essential elements which should be addressed in developing a program for evaluating bridges and providing countermeasures for scour. Detailed guidance regarding approaches for implementing the recommendations is included in HEC 18.

a. Interdisciplinary Team. Scour evaluations of new and existing bridges should be conducted by an interdisciplinary team comprised of hydraulic, geotechnical and structural engineers. [See HEC 18, Chapters 3 and 5 (*Chapters 2 and 10 in the fourth edition*)].

b. New Bridges. Bridges over tidal and non-tidal waterways with scourable beds should withstand the effects of scour from a superflood (a flood exceeding the 100-year flood) without failing; i.e., experiencing foundation movement of a magnitude that requires corrective action.

(1) Hydraulic studies should be prepared for bridges over waterways in accordance with Article 1.3.2 of the Standard Specifications for Highway Bridges of the American Association of State Highway and Transportation Officials (AASHTO) and the floodplain regulation of the FHWA as set forth in 23 CFR 650, Subpart A.

(2) Hydraulic studies should include estimates of scour at bridge piers and evaluation of abutment stability. Bridge foundations should be designed to withstand the effects of scour without failing for the worst conditions resulting from floods equal to or less than the 100-year flood. [See HEC 18, Chapters 3 and 4 (*Chapter 2 in the fourth edition*)]. Bridge foundations should be checked to ensure that they will not fail due to scour resulting from the

occurrence of a superflood on the order of magnitude of a 500-year flood. [See HEC 18,Chapter 3, (*Chapter 2 in the fourth edition*)].

(3) The geotechnical analysis of bridge foundations should be performed on the basis that all stream bed material in the scour prism above the total scour line for the design flood (for scour) has been removed and is not available for bearing or lateral support. In addition, the ratio of ultimate to applied loads should be greater than 1.0 for conditions of scour for the superflood. [See HEC 18, Chapter 3 (*Chapter 2 in the fourth edition*)].

(4) Data on scour at bridge piers and abutments should be collected and analyzed in order to improve existing procedures for estimating scour. (See HEC 18, Chapter 1.)

c. Existing Bridges. All existing bridges over tidal and non-tidal waterways should be evaluated for the risk of failure from scour during the occurrence of a superflood on the order of magnitude of a 500-year flood. [See HEC 18, Chapter 5 (*Chapter 10 in the fourth edition*)].

(1) An initial screening process should identify bridges susceptible to scour and establish a priority list for evaluation. [See HEC 18, Chapter 5 (*Chapter 10 in the fourth edition*)].

(2) Bridge scour evaluations should be conducted for each bridge to determine whether it is scour critical. A scour critical bridge is one with abutment or pier foundations which are rated as unstable due to:

(a) observed scour at the bridge site or

(b) a scour potential as determined from a scour evaluation study. [See HEC 18, Chapter 5 (*Chapter 10 in the fourth edition*)] .

(3) The procedures in Chapter 5 of HEC 18 (*Chapter 10 of the fourth edition*) should be followed in conducting and documenting the results of scour evaluation studies

d. Scour Critical Existing Bridges. A plan of action should be developed for each existing bridge determined to be scour critical. [See HEC 18, Chapter 5 (*Chapters 2 and 10 of the fourth edition*)].

(1) The plan of action should include instructions regarding the type and frequency of inspections to be

made at the bridge, particularly in regard to monitoring the performance and closing of the bridge, if necessary, during and after flood events. [See HEC 18, Chapter 7 (*Chapter 12 in the fourth edition*)].

(2) The plan of action should include a schedule for the timely design and construction of scour countermeasures determined to be needed for the protection of the bridge. [See HEC 18, Chapter 7 (*Chapter 12 in the fourth edition*)].

e. Bridge Inspectors. Bridge inspectors should receive appropriate training and instruction in inspecting bridges for scour. [See HEC 18, Chapter 6 (*Chapters 11 and 12 in the fourth edition*)].

(1) The bridge inspector should accurately record the present condition of the bridge and the stream. At least one cross section at each bridge should be documented and compared with previously recorded cross section(s) at the site. Pier locations and footing elevations should be included.

(2) The bridge inspector should identify conditions that are indicative of potential problems with scour and stream stability.

(3) Effective notification procedures should be available to permit the inspector to promptly communicate findings of actual or potential scour problems to others for further review and evaluation.

(4) Special attention should be focused on the routine inspection of scour critical bridges and on the monitoring and closing as necessary of scour critical and other bridges during and after floods.

5. EXISTING POLICY AND GUIDANCE. The following existing policy and guidance serve as the basis for the recommendations set forth in paragraph 4.

a. AASHTO Standard Specifications for Highway Bridges. The FHWA has accepted these specifications for the design of highway bridges. The 1991 Interim Specifications contain requirements for designing bridges to resist scour. Particular attention is directed to Article 1.3.2, Hydraulic Studies, which advises that, "Hydraulic studies . . . should include applicable parts of the following outline:" Included in this outline is item 1.3.2.3 (b), Estimated scour depth at piers and abutments of proposed structures.

b. AASHTO Manual for Bridge Maintenance. The FHWA endorses the guidance contained in this 1987 Manual for Bridge Maintenance. Particular attention is directed to the following two statements which support the recommendations contained in this Technical Advisory:

(1) "The primary function of the bridge maintenance program is to maintain the bridges in a condition that will provide for safe and uninterrupted traffic flows. The protection of the investment in the structure facility through well programmed repairs is second only to the safety of traffic and to the structure itself." (p. 25.)

(2) "Determining an effective solution to a stream bed or river problem is difficult. Settlement of foundations, local scour, bank erosion, and channel degradation are complex problems and cannot be solved by one or two prescribed methods. Hydraulic, geotechnical, and structural engineers are all needed for consultation prior to undertaking the solution of a serious maintenance problem. In some cases, certain remedial work could actually be detrimental to the structure." (p. 155.)

c. AASHTO Manual for Maintenance Inspection of Bridges. The FHWA endorses the guidance provided in the current version of this manual which serves as a standard and provides uniformity in the procedures and policies in determining the physical condition and maintenance needs of bridges. The manual emphasizes the importance of documenting and comparing cross sections taken upstream of bridges over time to discern potential scour problems.

d. Code of Federal Regulations, 23 CFR 650, Subpart C. The 1989 revision of this FHWA regulation on the National Bridge Inspection Standards requires that bridge owners maintain a bridge inspection program that includes procedures for underwater inspection. This Technical Advisory and HEC 18 provide guidance on the development and implementation of procedures for evaluating bridge scour to meet the requirements of the regulation.

e. Memorandum From the Director, Office of Engineering, to Regional Federal Highway Administrators and Direct Federal Program Administrator Dated April 17, 1987. This memorandum stated in part, "Each State should evaluate the risk of its bridges being subjected to scour damage during floods on the order of a 100 to 500 year return period or more."

f. FY 1991 High Priority Research Program of the FHWA. The FHWA recognizes the subject of scour at bridges as a long range high priority national program area for research and recommends that appropriate studies be carried out to improve the state-of-practice of designing new bridges and evaluating existing bridges for scour.

Thomas O. Willett, Director
Office of Engineering

APPENDIX J

FHWA 1995 RECORDING AND CODING GUIDE FOR THE STRUCTURE INVENTORY AND APPRAISAL OF THE NATION'S BRIDGES

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APPENDIX J

FHWA 1995 Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges

J.1 CODING GUIDE

This appendix contains relevant material for recording and coding the results of the evaluation of scour at bridges (Items 60, 61, 71, 92, 93, 113). The material is excerpted from the Federal Highway Administration document "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges," dated 1995.[1] Recently implemented revisions are included on Items 60 and 113 as shown in the enclosed extracts from the Coding Guide (see Attachment 1, Appendix J).

J.2 COUNTERMEASURES

If a bridge is scour critical (Item 113 code of 3 or less), a countermeasure should be considered to decrease the risk of failure of the foundation. If a countermeasure is installed using the criteria listed below, the bridge owner has the following Item 113 coding options: (A) use a code of 8 if the bridge foundation can be determined to be stable by assessment or by installation of properly designed countermeasures, or (B) use a code of 7 to indicate a countermeasure has been installed to mitigate an existing problem with scour and to reduce the risk of failure during a flood event.

In general, the riprap must be designed to withstand the appropriate bridge structure design frequency. The criteria apply to existing bridges. All new bridge designs must have stable foundations designed for the estimated hydraulics and scour. The criteria that must be met are:

1. The countermeasure must be designed to provide the same level of stability as the bridge structure. For example, if the bridge structure was designed using a 100-year event then the countermeasure must be stable and withstand a 100-year event.

2. The design must be supported by appropriate hydraulics and scour computations. These may include the incipient roadway overtopping event, design event, 100-year flood and the 500-year flood. If the bridge design was not supported by appropriate hydraulics and scour computations, then these computations should be made to determine the actual level of service the bridge provides.

3. A geotextile filter, geotextile bags, or fascine mat must be used (see HEC-23,[2] the FHWA publication, "Geosynthetic Design and Construction Guidelines,"[3] or HEC-11.[4]

4. For example, if riprap is used as a pier scour countermeasure, it should be sized according to the HEC-23[2] pier riprap sizing equation or other appropriate approach. If a class of riprap is used, then the median size of the riprap class must equal or exceed the design median size (D_{50}). Figures J.1 and J.2 show preliminary recommendations for pier riprap design.

- The top of the riprap should be located at the channel bed elevation or, if a complete channel riprap armor is installed, flush with the riprap armor at the pier or abutment. Riprap mounded around the pier is not acceptable.

- The required thickness of riprap is dependent on the amount of contraction scour expected during the design event. The thickness will be a minimum of three times the median riprap size ($3xD_{50}$) unless the computed contraction scour amount is greater. If the contraction scour exceeds $3xD_{50}$ then the bottom of the riprap must extend down to the contraction scour elevation and the top of the riprap remains at the channel bed.

- The riprap will extend at least twice the pier width or 1.2 times the computed pier scour depth, which ever is greater, but may also be controlled by contraction scour. The riprap will launch away from the pier due to contraction scour. The post-event riprap configuration must be estimated using a 1V:1.5H slope to ensure that the riprap surface extends at least the pier width after the design event. Figures J.1 and J.2 show two methods for constructing pier riprap. In Figure J.1, the vertical riprap edge is achieved by using temporary sheet pile. Figure J.2 shows riprap placement using excavation only.

- The riprap must be inspected at a minimum interval of two years and, as a minimum, after any flood equaling or exceeding the 25-year recurrence interval.

J.3 REFERENCES

1. Federal Highway Administration, 1995, "Recording and Coding Guide for the Structure Inventory and Appraisal of the Nation's Bridges, Report No. FHWA-PD-96-001, U.S. Department of Transportation, Washington, D.C.

2. Lagasse, P.F., L.W. Zevenbergen, J.D. Schall, and P.E. Clopper, 2001, "Bridge Scour and Stream Instability - Countermeasures - Experience, Selection, and Design Guidelines, Hydraulic Engineering Circular No. 23, Second Edition, FHWA NHI 01-003, Federal Highway Administration, Washington, D.C.

3. Holz, D.H., B.R. Christopher, and R.R. Berg, 1995, "Geosynthetic Design and Construction Guidelines," National Highway Institute, Publication No. FHWA HI-95-038, Federal Highway Administration, Washington, D.C., May.

4. Brown, S.A. and E.S. Clyde, 1989, "Design of Riprap Revetment," Hydraulic Engineering Circular No. 11, FHWA-IP-016, prepared for FHWA, Washington, D.C.

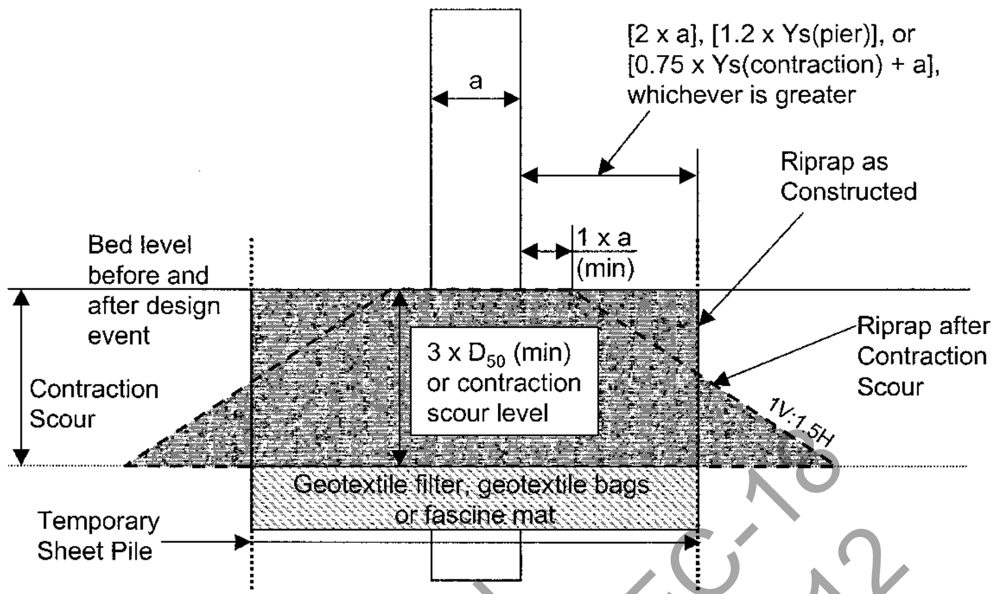

Figure J.1. Riprap design using temporary sheet pile.

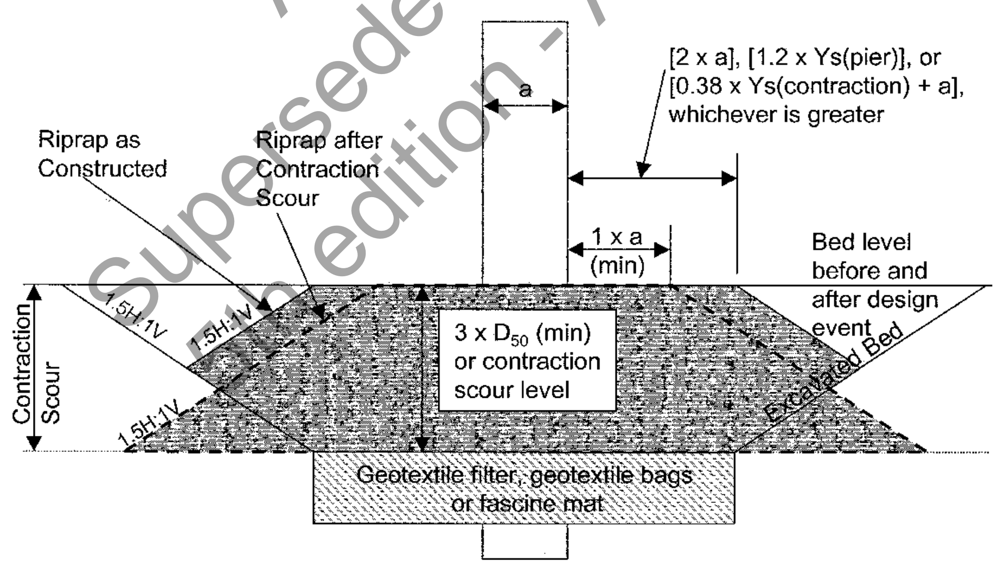

Figure J.2. Riprap design using excavation only.

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ATTACHMENT B

EXTRACTS FROM THE CODING GUIDE

Items 58 through 62 - Indicate the Condition Ratings

In order to promote uniformity between bridge inspectors, these guidelines will be used to rate and code Items 58, 59, 60, 61, and 62. The use of the AASHTO Guide for Commonly Recognized (CoRe) Structural Elements is an acceptable alternative to using these rating guidelines for Items 58, 59, 60, and 62, provided the FHWA translator computer program is used to convert the inspection data to NBI condition ratings for NBI data submittal.

Condition ratings are used to describe the existing, in-place bridge as compared to the as-built condition. Evaluation is for the materials related, physical condition of the deck, superstructure, and substructure components of a bridge. The condition evaluation of channels and channel protection and culverts is also included. Condition codes are properly used when they provide an overall characterization of the general condition of the entire component being rated. Conversely, they are improperly used if they attempt to describe localized or nominally occurring instances of deterioration or disrepair. Correct assignment of a condition code must, therefore, consider both the severity of the deterioration or disrepair and the extent to which it is widespread throughout the component being rated.

The load-carrying capacity will not be used in evaluating condition items. The fact that a bridge was designed for less than current legal loads and may be posted shall have no influence upon condition ratings.

Portions of bridges that are being supported or strengthened by temporary members will be rated based on their actual condition; that is, the temporary members are not considered in the rating of the item. (See Item 103 - Temporary Structure Designation for the definition of a temporary bridge.)

Completed bridges not yet opened to traffic, if rated, shall be coded as if open to traffic

Item 60 - Substructure 1 digit

This item describes the physical condition of piers, abutments, piles, fenders, footings, or other components. Rate and code the condition in accordance with the previously described general condition ratings. Code N for all culverts.

All substructure elements should be inspected for visible signs of distress including evidence of cracking, section loss, settlement, misalignment, scour, collision damage, and corrosion. The rating factor given to Item 60 should be consistent with the one given to Item 113 whenever a rating factor of 2 or below is determined for Item 113 - Scour Critical Bridges.

The substructure condition rating shall be made independent of the deck and superstructure.

Integral-abutment wingwalls to the first construction or expansion joint shall be included in the evaluation. For non-integral superstructure and substructure units, the substructure shall be considered as the portion below the bearings. For structures where the substructure and superstructure are integral, the substructure shall be considered as the portion below the superstructure.

The following general condition ratings shall be used as a guide in evaluating Items 58, 59, and 60:

Code	Description
N	NOT APPLICABLE
9	EXCELLENT CONDITION
8	VERY GOOD CONDITION - no problems noted.
7	GOOD CONDITION - some minor problems.
6	SATISFACTORY CONDITION - structural elements show some minor deterioration.
5	FAIR CONDITION - all primary structural elements are sound but may have minor section loss, cracking, spalling or scour.
4	POOR CONDITION - advanced section loss, deterioration, spalling or scour.
3	SERIOUS CONDITION - loss of section, deterioration, spalling or scour have seriously affected primary structural components. Local failures are possible. Fatigue cracks in steel or shear cracks in concrete may be present.
2	CRITICAL CONDITION - advanced deterioration of primary structural elements. Fatigue cracks in steel or shear cracks in concrete may be present or scour may have removed substructure support. Unless closely monitored it may be necessary to close the bridge until corrective action is taken.
1	"IMMINENT" FAILURE CONDITION - major deterioration or section loss present in critical structural components or obvious vertical or horizontal movement affecting structure stability. Bridge is closed to traffic but corrective action may put back in light service.
0	FAILED CONDITION - out of service - beyond corrective action.

Item 61 - Channel and Channel Protection 1 digit

This item describes the physical conditions associated with the flow of water through the bridge such as stream stability and the condition of the channel, riprap, slope protection, or stream control devices including spur dikes. The inspector should be particularly concerned with visible signs of excessive water velocity which may affect undermining of slope protection, erosion of banks, and realignment of the stream which may result in immediate or potential problems. Accumulation of drift and debris on the superstructure and substructure should be noted on the inspection form but not included in the condition rating.

Rate and code the condition in accordance with the previously described general condition ratings and the following descriptive codes:

Code	Description
N	Not applicable. Use when bridge is not over a waterway channel).
9	There are no noticeable or noteworthy deficiencies which affect the condition of the channel.
8	Banks are protected or well vegetated. River control devices such as spur dikes and embankment protection are not required or are in a stable condition.
7	Bank protection is in need of minor repairs. River control devices and embankment protection have a little minor damage. Banks and/or channel have minor amounts of drift.
6	Bank is beginning to slump. River control devices and embankment protection have widespread minor damage. There is minor stream bed movement evident. Debris is restricting the channel slightly.
5	Bank protection is being eroded. River control devices and/or embankment have major damage. Trees and brush restrict the channel.
4	Bank and embankment protection is severely undermined. River control devices have severe damage. Large deposits of debris are in the channel.
3	Bank protection has failed. River control devices have been destroyed. Stream bed aggradation, degradation or lateral movement has changed the channel to now threaten the bridge and/or approach roadway.
2	The channel has changed to the extent the bridge is near a state of collapse.
1	Bridge closed because of channel failure. Corrective action may put back in light service.
0	Bridge closed because of channel failure. Replacement necessary.

Item 71 - Waterway Adequacy 1 digit

This item appraises the waterway opening with respect to passage of flow through the bridge. The following codes shall be used in evaluating waterway adequacy (interpolate where appropriate). Site conditions may warrant somewhat higher or lower ratings than indicated by the table (e.g., flooding of an urban area due to a restricted bridge opening).

Where overtopping frequency information is available, the descriptions given in the table for chance of overtopping mean the following:

Remote	-	greater than 100 years
Slight	-	11 to 100 years
Occasional	-	3 to 10 years
Frequent	-	less than 3 years

Adjectives describing traffic delays mean the following:

Insignificant	-	Minor inconvenience. Highway passable in a matter of hours.
Significant	-	Traffic delays of up to several days.
Severe	-	Long term delays to traffic with resulting hardship.

Functional Classification			
Principal Arterials – Interstates, Freeways, or Expressways	Other Principal and Minor Arterials and Major Collectors	Minor Collectors, Locals	Description
Code			
N	N	N	Bridge not over a waterway.
9	9	9	Bridge deck and roadway approaches above flood water elevations (high water). Chance of overtopping is remote.
8	8	8	Bridge deck above roadway approaches. Slight chance of overtopping roadway approaches.
6	6	7	Slight chance of overtopping bridge deck and roadway approaches.
4	5	6	Bridge deck above roadway approaches. Occasional overtopping of roadway approaches with insignificant traffic delays.

Functional Classification			Description
Principal Arterials – Interstates, Freeways, or Expressways	Other Principal and Minor Arterials and Major Collectors	Minor Collectors, Locals	
Code			
3	4	5	Bridge deck above roadway ap proaches. Occasional overtopping of roadway approaches with significant traffic delays.
2	3	4	Occasional overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	3	Frequent overtopping of bridge deck and roadway approaches with significant traffic delays.
2	2	2	Occasional or frequent overtopping of bridge deck and roadway approaches with severe traffic delays.
0	0	0	Bridge closed.

Item 92 - Critical Feature Inspection 9 digits

Using a series of 3-digit code segments, denote critical features that need special inspections or special emphasis during inspections and the designated inspection interval in months as determined by the individual in charge of the inspection program. The designated inspection interval could vary from inspection to inspection depending on the condition of the bridge at the time of inspection.

Segment	Description	Length
92A	Fracture Critical Details	3 digits
92B	Underwater Inspection	3 digits
92C	Other Special Inspection	3 digits

For each segment of Item 92A, B, and C, code the first digit Y for special inspection or emphasis needed and code N for not needed. The first digit of Item 92A, B, and C must be coded for all structures to designate either a yes or no answer. Those bridges coded with a Y in Item 92A or B should be the same bridges contained in the Master Lists of fracture critical and special underwater inspection bridges. In the second and third digits of each segment, code a 2-digit number to indicate the number of months between inspections only if the first digit is coded Y. If the first digit is coded N, the second and third digits are left blank.

Current guidelines for the maximum allowable interval between inspections can be summarized as follows:

Fracture Critical Details	24 months
Underwater Inspection	60 months
Other Special Inspections	60 months

EXAMPLES:	Item	Code
A 2-girder system structure which is being inspected yearly and no other special inspections are required.	92A 92B 92C	Y12 N__ N__
A structure where both fracture critical and underwater inspection are being performed on a 1-year interval. Other special inspections are not required.	92A 92B 92C	Y12 Y12 N__
A structure has been temporarily shored and is being inspected on a 6-month interval. Other special inspections are not required.	92A 92B 92C	N__ N__ Y06

Item 93 - Critical Feature Inspection Date 12 digits

Code only if the first digit of Item 92A, B, or C is coded Y for yes. Record as a series of 4-digit code segments, the month and year that the last inspection of the denoted critical feature was performed.

Segment	Description	Length
93A	Fracture Critical Details	4 digits
93B	Underwater Inspection	4 digits
93C	Other Special Inspection	4 digits

For each segment of this item, when applicable, code a 4-digit number to represent the month and year. The number of the month should be coded in the first 2 digits with a leading zero as required and the last 2 digits of the year coded as the third and fourth digits of the field. If the first digit of any part of Item 92 is coded N, then the corresponding part of this item shall be blank.

EXAMPLES:	Item	Code
A structure has fracture critical members which were last inspected in March 1986. It does not require underwater or other special feature inspections.	93A 93B 93C	0386 (blank) (blank)
A structure has no fracture critical details, but requires underwater inspection and has other special features (for example, a temporary support) for whichthe State requires special inspection. The last underwater inspection was done in April 1986 and the last special feature inspection was done in November 1985.	93A 93B 93C	(blank) 0486 1185

Item 94 - Bridge Improvement Cost 6 digits

Code a 6-digit number to represent the estimated cost of the proposed bridge or major structure improvements in thousands of dollars. This cost shall include only bridge construction costs, excluding roadway, right of way, detour, demolition, preliminary engineering, etc. Code the base year for the cost in Item 97 - Year of Improvement Cost Estimate. Do not use this item for estimating maintenance costs.

This item must be coded for bridges eligible for the Highway Bridge Replacement and Rehabilitation Program. It may be coded for other bridges at the option of the highway agency.

EXAMPLES:

		Code
Bridge Improvement Cost	$ 55,850	000056
	250,000	000250
	7,451,233	007451

Item 113 - Scour Critical Bridges 1 digit

Use a single-digit code as indicated below to identify the current status of the bridge regarding its vulnerability to scour. Evaluations shall be made by hydraulic/geotechnical/structural engineers. Guidance on conducting a scour evaluation is included in the FHWA Technical Advisory T 5140.23 titled, "Evaluating Scour at Bridges."[1] Detailed engineering guidance is provided in the Hydraulic Engineering Circular 18 titled "Evaluating Scour at Bridges."[2] Whenever a rating factor of 2 or below is determined for this item, the rating factor for Item 60 -- Substructure and other affected items (i.e., load ratings, superstructure rating) should be revised to be consistent with the severity of observed scour and resultant damage to the bridge. A plan of action should be developed for each scour critical bridge (see FHWA Technical Advisory T 5140.23, HEC 18 and HEC 23[3]). A scour critical bridge is one with abutment or pier foundation rated as unstable due to (1) observed scour at the bridge site (rating factor of 2, 1, or 0) or (2) a scour potential as determined from a scour evaluation study (rating factor of 3). It is assumed that the coding of this item has been based on an engineering evaluation, which includes consultation of the NBIS field inspection findings.

Code Description

N Bridge not over waterway.

U Bridge with "unknown" foundation that has not been evaluated for scour. Until risk can be determined, a plan of action should be developed and implemented to reduce the risk to users from a bridge failure during and immediately after a flood event (see HEC 23).

T Bridge over "tidal" waters that has not been evaluated for scour, but considered low risk. Bridge will be monitored with regular inspection cycle and with appropriate underwater inspections until an evaluation is performed ("Unknown" foundations in "tidal" waters should be coded U.)

9 Bridge foundations (including piles) on dry land well above flood water elevations.

8 Bridge foundations determined to be stable for the assessed or calculated scour condition. Scour is determined to be above top of footing (Example A) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge[4]), by calculation or by installation of properly designed countermeasures (see HEC 23).

7 Countermeasures have been installed to mitigate an existing problem with scour and to reduce the risk of bridge failure during a flood event. Instructions contained in a plan of action

have been implemented to reduce the risk to users from a bridge failure during or immediately after a flood event.

6 Scour calculation/evaluation has not been made. (Use only to describe case where bridge has not yet been evaluated for scour potential.)

5 Bridge foundations determined to be stable for assessed or calculated scour condition. Scour is determined to be within the limits of footing or piles (Example B) by assessment (i.e., bridge foundations are on rock formations that have been determined to resist scour within the service life of the bridge), by calculations or by installation of properly designed countermeasures (see HEC 23).

4 Bridge foundations determined to be stable for assessed or calculated scour conditions; field review indicates action is required to protect exposed foundations (see HEC 23).

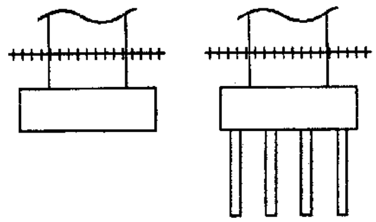
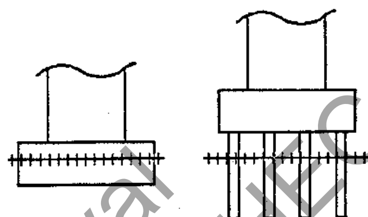
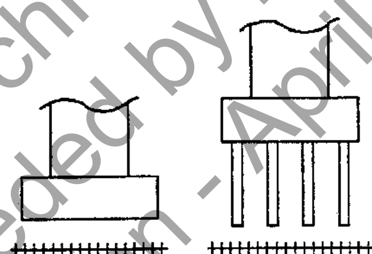
3 Bridge is scour critical; bridge foundations determined to be unstable for assessed or calculated scour conditions:
- Scour within limits of footing or piles. (Example B)
- Scour below spread-footing base or pile tips. (Example C)

2 Bridge is scour critical; field review indicates that extensive scour has occurred at bridge foundations, which are determined to be unstable by:
- a comparison of calculated scour and observed scour during the bridge inspection, or
- an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.

1 Bridge is scour critical; field review indicates that failure of piers/abutments is imminent. Bridge is closed to traffic. Failure is imminent based on:
- a comparison of calculated and observed scour during the bridge inspection, or
- an engineering evaluation of the observed scour condition reported by the bridge inspector in Item 60.

0 Bridge is scour critical. Bridge has failed and is closed to traffic.

[1]FHWA Technical Advisory T 5140.23, Evaluating Scour at Bridges, dated October 28, 1991.
[2]HEC 18, Evaluating Scour at Bridges, Fourth Edition.
[3]HEC 23, Bridge Scour and Stream Instability Countermeasures, Second Edition.
[4]FHWA Memorandum "Scourability of Rock Formations," dated July 19, 1991.

EXAMPLES:	CALCULATED SCOUR DEPTH	ACTION NEEDED
A. Above top of footing		None - indicate rating of 8 for this item
B. Within limits of footing or piles		Conduct foundation structural analysis
C. Below pile tips or spread-footing base		Provide for monitoring and scour countermeasures as necessary

SPREAD FOOTING (NOT FOUNDED IN ROCK) | PILE FOOTING

= Calculated scour depth

APPENDIX K

UNKNOWN FOUNDATIONS

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APPENDIX K

Unknown Foundations

K.1 INTRODUCTION

Bridges are classified as having unknown foundations when the type (spread footing, piles, columns), dimensions (length, width, or thickness), reinforcing, and/or elevation are unknown. They are classified as U in Item 113 of the coding guide (Appendix I). The screening program in the National Evaluation program has identified 90,000 bridges with unknown foundations. Research under the National Cooperative Highway Research Program (NCHRP) has investigated nondestructive testing methods which in many cases can determine pile length. This appendix provides a status report and guidance for protecting bridges with unknown foundations from scour.

K.2 PLAN OF ACTION

For bridges with unknown foundations a Plan of Action should be developed (see Chapter 12). The plan of action to take into consideration the service life of the bridge, the volume and type of traffic, and important of the highway (interstate, primary or rural farm to market). The Plan of Action includes:

- Describing the foundation and scour condition
- Timely installation of countermeasures to reduce the risk from scour (e.g., riprap.)
- Development and implementation of a scour monitoring and/or inspection program
- Development of a plan for closure of the bridge, if needed
- Determining if nondestructive test is economical and feasible to determine foundation characteristics
- Schedule timely design and construction of a new bridge or countermeasures to make the bridge safe from scour and stream instability

K.3 NONDESTRUCTIVE TESTING (NTD) RESEARCH

NCHRP Project 21-5 initiated in 1996, identified and tested the following NTD methods:[1,2]

- Sonic echo/impulse response
- Bending wave method
- Ultraseismic test method
- SASW method
- Dynamic foundation response method
- Borehole parallel seismic test method
- Borehole sonic method
- Borehole radar method
- Induction field method

As a result of the above research, a second phase of this project (NCHRP 21-5 (2)) was initiated to research and develop equipment, field techniques, and analysis methods for the most promising methods. The methods selected were:

- Ultraseismic (including sonic echo/impulse response and bending wave methods)
- Borehole of parallel seismic and induction field

In general the results of testing NTD methods have not been as satisfactory as the initial research indicated. The results of NCHRP Project 21-5 indicate that of all the surface and borehold methods, the Parallel Seismic test was found to have the broadest applications for determining the bottom depth of substructures. Of the surface tests (no boring required), the Ultraseismic test has the broadest application to the determination of the depths of unknown bridge foundations but will provide no information on piles below larger substructure (pilecaps). The Sonic Echo/Impulse Response, Bending Wave, Spectral Analysis of Surface Wave, and Borehold Radar methods all had more specific applications.[3] It is recommended that at this time a Plan of Action and appropriate countermeasures continue to be used as the primary measures to protect bridges with unknown foundations from failure from scour.

K.4 OTHER TEST PROCEDURES

K.4.1 Core Drilling

A simple method used by one State Highway Agency (SHA) to explore unknown foundations is to use a drilling rig to core the bridge deck and to continue down through the pier or abutment footing into the supporting soil or rock under the foundation. This procedure has been used successfully to determine the foundations of some 40 structures and to reclassify the structures as known foundations for purposes of rating them for Item 113, Scour Critical Bridges.

K.4.2 Forensic Engineering

There may be a considerable amount of information in the files of the bridge owner that can be reviewed for information pertaining to the bridge foundations even though as-built plans are no longer available:

- Inspection records may indicate channel bed elevations taken over a period of time. In one state, a concerted effort was made to record channel bed elevations at many bridges immediately after a major flood occurred in 1973. This information now serves as a benchmark for assessing current conditions. If the channel bed is now four or five feet higher than it was in 1973, and the bridge was not damaged in the 1973 flood, this information becomes very useful in assessing the risk posed to the structure by the river.

- Inspectors may have documented exposed foundations in the aftermath of previous floods. While the foundation may no longer be visible, this knowledge of the elevation of the top or bottom of a footing will help the engineer to determine necessary information about the bridge foundation.

- Channel bed under bridges is subject to scour and subsequent infilling of material back into the scour hole. The infill material is likely to be soft fine material that can be easily probed with a reinforcing rod. Careful probing will reveal the elevation of the tops of footings located several feet below the channel bed. Inspections records will often contain basic information about the bridge foundation and whether it is a spread footing or on piles. This information can be used to estimate the footing dimensions within a reasonable degree of accuracy so that an assessment can be made as to whether worst-case scour conditions are likely to exceed the bottom of the footing.

K.5 REFERENCES

1. Transportation Research Board, 1996, "Nondestructive Destructive of Unknown Subsurface bridge Foundations- Results of NCHRP Project 21-5." Research Results Digest No. 213, Transportation Research Board, Washington, D.C.

2. Olson, L.D., F. Jalinoos, and M.F. Aouad, 1995. "Determination of Unknown Subsurface bridge Foundations-Final Report," NCHRP Project 21-5, Transportation Research Board, National Research Council, Washington, D.C.

3. Olson, L.D. and M.F. Aouad, 1998, "NCHRP 21-5 Research Results on Determination of Unknown Bridge Foundation Depths," ASCE Proceedings of International Water Resources Engineering Conference, August 3-7, 1998, Memphis, TN.

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APPENDIX L

SCOUR IN COHESIVE SOILS

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APPENDIX L

Scour In Cohesive Soils

L.1 INTRODUCTION

The maximum depth of local scour at piers in cohesive soils is the same as in non-cohesive soils.[1,2,3] Time is the difference. Maximum scour depth is reached in hours or one runoff event in non-cohesive sand, but may take days and many runoff events in cohesive clays. Local pier scour in cohesive clays may be 1,000 times slower than non-cohesive sand.[1] In addition, by inference, contraction scour and local scour at abutments in cohesive soils do not reach maximum depth as rapidly; but the ultimate scour depth will be the same as for non-cohesive soil.

The equations and methodologies presented in this manual, which predict the maximum scour depth in non-cohesive soil, may, in some circumstance be too conservative. The pier scour equation represents an envelope curve of the deepest scour observed during the various laboratory studies and field data. There is much merit in using a conservative approach, taking into consideration the wide range of soil characteristics, the intricate interactions between soil and water, and the uncertainties inherent in predicting flood flows and their flow patterns through the bridge over its service life. When applied with engineering judgment, this conservative approach is usually reasonable and cost-effective.

On the other hand, there are site conditions and bridges where an alternative method for scour evaluation would be appropriate. Examples include bridges founded on highly scour-resistant cohesive soils where the useful life of the bridge is short in relation to the expected number of scouring floods and rate of scour in cohesive soils, bridges scheduled to be replaced in a couple of years, or bridges on low traffic volume roads which are monitored. Significant savings can be achieved for bridges under these conditions, when the characteristics of the cohesive soils to resist scour are taken into account in the design of the foundation. It is not good engineering judgment to design foundations for scour less than the maximum for bridges in cohesive soils that have a long or undetermined design life, have a very large traffic volume, are not monitored, or serve hospitals or schools. However, it is always good engineering practice to use several methods to determine scour depths and use engineering judgment in the design of bridge foundations.

Cohesive soils include silts and clays. According to the unified soil classification system, silts and clays are soils which have more than 50% by weight of particles passing the 0.075mm sieve opening. Silt size particles are between 0.075mm and 0.002mm and clay size particles are smaller than 0.002mm. Cohesive soils are not classified by grain size, but instead by their degree of plasticity which is measured by the Atterberg limits.

Because cohesive soils can scour much slower than non-cohesive soils, it is reasonable to include the scour rate in the calculations. Indeed, while one flood may be sufficient to create the maximum scour depth (z_{max}) in cohesionless soils, the scour depth after many years of flood history at a bridge in an erosion resistant cohesive soil may only be a fraction of z_{max}. The scour rate effect in cohesive soils can be measured by an erosion rate versus shear stress relation. This relation can be used to calculate the scour depth in the case of cohesive soils. This calculated scour depth along with the calculated maximum scour depth, bridge site conditions, type of highway, life cycle of the bridge, traffic volume and comfort level of the DOT can be used in the design of the foundations.

Briaud et al.[1,2] developed a device to measure the scour rate in cohesive soils and equations and methods to use this rate to determine the scour depth at bridges in cohesive soils. The method is called SRICOS for scour rate in cohesive soils. The SRICOS method was developed on the basis of flume tests, numerical testing, and erosion testing of the soil. The device to measure the erosion rate is called EFA (Erosion Function Apparatus). In the following sections the SRICOS method will be described.

L.2 SRICOS METHOD

The first step in the SRICOS method is to develop a plan for testing of the subsurface soils at the bridge site. Representative soils samples are obtained with Shelby tubes and shipped to the laboratory for testing. At the laboratory the EFA is used to determine the erosion rate versus shear stress curve, Figure L.1. The erosion rate dz/dt is defined as the vertical distance scoured per unit of time and is reported in mm/hr. The shear stress, τ, is the shear stress imposed at the water soil interface and is given in N/m^2.

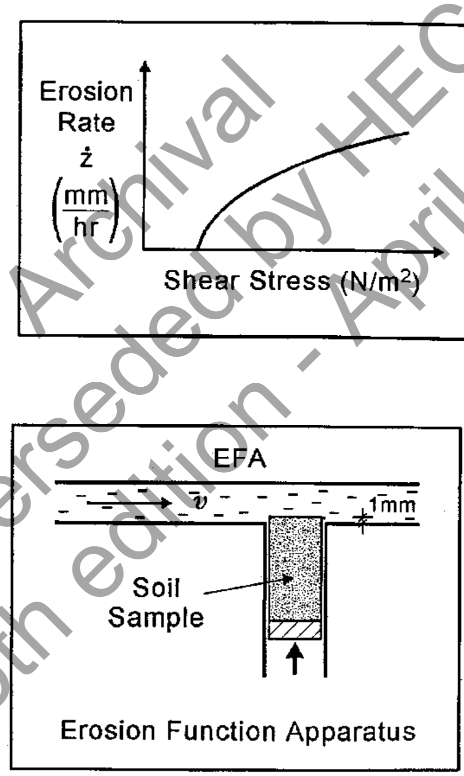

Figure L.1. Erosion rate vs. shear stress and the Erosion Function Apparatus.

The dz/dt versus τ curve is a measure of the erodibility of the soil. Typically the erosion rate dz/dt is zero until the critical shear stress, τ_c, is reached and then dz/dt increases as τ increases. The dz/dt versus τ curve can be measured with the EFA (Erosion Function Apparatus).[1] Once the dz/dt versus τ curve is obtained the method to predict the pier scour depth as a function of time proceeds as follows. First, the maximum shear stress τ_{max} around the bridge pier is calculated:[1]

$$\tau_{max} = 0.0094\ \rho\ V^2 \left[\frac{1}{\log Re} - \frac{1}{10}\right] \tag{L.1}$$

where:

ρ	=	Density of water
V	=	Mean approach velocity
Re	=	Pier Reynolds number

Second, the initial scour rate dz/dt_i corresponding to τ_{max} is read on the dz/dt vs. τ curve. Third, the maximum depth of scour z_{max} is calculated using the pier scour equations and methods given in Chapter 6.

Note that Briaud(1) determined that z_{max} in cohesive soils is very close to that for cohesionless soils. It was found that the maximum depths of scour in clays and in sands were approximately the same in flume experiments. In those same experiments, however, it was found that the scour hole in clay developed to the side and in the back of the pier and not in the front of the pier. This indicates that for scour in clay the front of the pier may not be the best place to install monitoring equipment.

It is then possible to make scour predictions by applying a detailed velocity (shear stress) history over the design life of the bridge and summing the erosion rates for the cumulative amount of time that the shear stress exceeds the critical shear stress. This requires the use of a computer program which can also consider the case of a layered soil system.(2) The limitation of this method is that it is for circular bridge piers and for water depth over pier diameter ratios larger than 2. Existing correction factors are recommended for other cases.

To apply this approach to contraction scour, the computed hydraulic shear stress would be used directly rather than Equation L.1, which is specific to circular piers. For abutment scour, a relationship would need to be developed to determine local shear stress, or a detailed 2-dimensional model would need to be used to compute shear stresses in the vicinity of the abutment toe.

L.3 REFERENCES

1. Briaud, J.-L., F.C.K. Ting, H.C. Chen, R. Gudavaiii, S. Perugu, G. Wei, 1999a, "SRICOS: Prediction of Scour Rate in Cohesive Soils at Bridge Piers," ASCE Journal of Geotechnical and Geoenvironmental Engineering, Vol. 125, No. 4, pp 237-246, Reston, VA.

2. Briaud, J.-L., F.C.K. Ting, H.C. Chen, R. Gudavaiii, K. Kwak, B. Philogene, S.W. Han., S. Perugu, G. Wei, P. Nurtjahyo, Y. Cao, Y. Li, 1999b, "SRICOS Prediction of Scour Rate at Bridge Piers," Report 2937-F, Texas Depart. of Transportation, Texas A&M University, Civil Engineering, College Station, TX 77843-3136.

3. Richardson, E.V., L.J. Harrison, and S.R. Davis, 1991, “Evaluating Scour at Bridges,” HEC-18, Report FHWA-IP-90-017, Washington, D.C.

APPENDIX M

SCOUR COMPETENCE OF ROCK

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APPENDIX M

Scour Competence of Rock

M.1 INTRODUCTION

The equations and methods given in this manual are for determining scour depths for the design of bridge foundations in granular soils (silts, sands, gravels, cobbles, and boulders). In Chapter 2 recommendations are given for the design of bridge foundations on rock highly resistant to scour. **The problem is determining if rock is resistance to scour.** There are examples in the literature of bridge foundation failure form scour in what was supposed to be rock**.** An example is the failure of the I-90 bridge over Schoharie Creek in upstate New York (see Chapter 11, Section 11.4). The rock foundation material was massive, extensive, and very hard with a blow count on the order of 80 to 100. However, when subjected to water flow in a flume test it started to erode at a velocity of 1.22 m/s (4 ft/s) and would erode rapidly at a velocity of 2.44 m/s (8 ft/s).[1,2]

The determination if the bridge foundations are founded in scour resistance rock and the design of foundations in rock require the expertise of geologist and geotechnical engineers. In addition to standard geologic and geotechnical tests, core or block samples can be taken and subjected to flume studies. The Erosion Function Apparatus (EFA) described in Appendix L or a simply constructed or available flume can be used to determine the scourability of the rock material. In the following sections four recommendations are given for determining if rock foundations are scour resistance. However, additional research is needed in this area. The four recommendations are:

- Geologic, geomorphologic, and geotechnical analyses
- July, 1991 memorandum from the FHWA titled "Scourability of Rock Formations, (see Attachment 1, Appendix M)
- Flume tests to determine the resistance of rock to scour
- Erodibility Index procedure

M.2 GEOLOGIC, GEOMORPHOLOGY, STREAM STABILITY AND GEOTECHNICAL ANALYSIS

The geology, geomorphology and stream stability of the bridge crossing, and geotechnical analysis of the foundation material are extremely important. Coring of the site must be extensive to determine the location and extent (depth and area) of the rock. The cores to be subjected to the standard field classification and soil mechanics tests. In addition, where the scourability of the rock type is unknown, erosion tests as described later should be made. The geologic formation on which the bridge foundations are to be constructed needs to be determined and mapped (depth, areal extent and massiveness). The scour resistance of the geologic formation needs to be known or determined. The geomorphology of the site needs to be determined and related to the erodibility of the foundation material (alluvial fan, karst topography, desert, mountain or plain stream, etc). The long-term stability of the stream should be estimated.

Some questions to be answered are:

- Is the competent rock only a relatively thin layer 0.6 to 1 m (2 to 3 ft) that can be undermined?
- Is there the potential for a headcut or nickpint from downstream to undermine the rock?
- What is the geologic formation for the foundation (granite, sandstone, glacial till, etc)?
- What is the scour experience of bridges in the area or in similar geologic formations?
- Is the foundation material subjected to freezing and thawing?
- Is the foundation material susceptible to leaching by flowing water (limestone)?
- What is the planform of the stream at the bridge crossing (meandering, braided or straight)?
- Is the stream aggaiding or degrading?
- Are the foundation material subject to abrasion by the sediment discharge of the stream? If so, how resistance to abrasion is the rock material?

M.3 FLUME TESTS

Samples (standard core or other square or round samples) of the foundation material that is thought to be resistant to the erosion action of water can be tested in flumes. Any flume that is used for hydraulic research can be used if it has a sufficiently large range of velocities at a depth of 0.15 m (0.5 ft) or more. At modest cost a flume can be built to determine the resistance of a rock sample to erosion. The Erosion Function Apparatus (EFA) used to determine the erodibility of cohesive soils can and has been used to determine the erodibility of rock samples.[3,4] The EFA determines the scour rate in mm/hr vs. shear stress in N/m^2 or velocity in m/s. The apparatus and method are described in Appendix L. The samples should be subjected to velocities as large as are to be expected at the bridge crossing and placed in the flume flush with the floor or only slightly projecting into the flow. Projections of 1mm (.03 in) to 3 or 4 mm (0.16 in) are acceptable. If standard cores are not taken a square approximately 0.3 m (1 ft) should be sufficient to test.

Flume tests can determine if the rock material will not erode for the expected velocity or shear stress, or if the material will erode. In some cases a time rate of erosion (mm/hr, inches/hr) can be obtained. In the latter case, methods proposed in Appendix L can be used to determine if the maximum calculated scour can be reduced.

In obtaining samples of the foundation material care must be exercised to not destroy the integrity of the foundation material at the bridge site. Often, with the help of a geologists, samples can be taken and tested of similar material from another location.

M.3.1 Examples of Flume Erosion Tests

Ice Compacted Glacial Till Erodibility tests of the ice compacted glacial till that was the foundation material for the I-90 bridge over Schoharie Bridge were made in flume tests at Cornell University.[2,5,6] Although the foundation material was extremely dense, difficult to penetrate with piles or to excavate, erosion would start at a velocity of 1.22 m/s (4 ft/s) and would be large at 2.4 m/s (8.0 ft/s).

Caliche Soil Layers Erodibility test were made of caliche layers that are found in the bed of dry arroyos in the desert soils of Arizona. Caliche soil layers are soils composed of silt, sand, gravel or cobbles cemented by secondary calcium carbonate precipitate. The layers may be a few centimeter (inches) to several meters (ft) thick and erodibility may range form easily to very hard. Tests were made using the EFA on a 3 inch (0.76 m) core [7] and using a specially constructed flume on three 1 ft. (0.3 m) roughly cubic samples.[8] In the EFA tests the core was subjected to velocities ranging from O.21 m/s to 4.7 m/s (0.7 to 15.4 ft/s). Both the bottom and top of the 3 inch core was tested. Erosion rates for the top of the core ranged from 0.15 mm/h (0.006 inch/hr) at a velocity of 0.21 m/s (0.70 ft/s) to 219.8 mm/hr (8.7 inch/hr) at 1.46 m/s (4.79 ft/s). The erosion rates for the bottom layer ranged from 0 mm/hr at 0.53 m/s (1.73 ft/s) to 22.05 mm/hr (0.87 inch/hr) at 2.43 m/s (7.97 ft/s). The core as tested was approximately 70 mm (2.76 inch) in length. Similar results were obtained using a specially constructed flume on the three 1 ft. (0.3 m) roughly cubic samples. For the sample that had the smallest erosion rate, the rate ranged from 0.60 mm/hr (0.24 inch/hr) at 0.75 m/s (2.46 ft/s) to 2.10 mm/hr (0.83 inch/hr) at 3.14 m/s (10.30 ft/s). The sample with the largest erosion rate the rate ranged from 4.12 mm/hr (0.16 inch/hr) at a velocity of 0.64 m/s (2.10 ft/s) to 177.6 mm/hr (6.99 inch/hr) at a velocity of 2.96 m/s (9.71 ft/s) . The results of the tests showed the variability in the erodibility of the caliche layers, and the comparability and usefulness of the two testing methods.

M.4 ERODIBILITY INDEX METHOD

Annandale[9,10] developed an Erodibility Index, which is identical to Kirsten's Excavatability Index,[11] to quantify the relative ability of non-uniform earth material to resist erosion. He proposed a relation between the Erodibility Index and stream power for use in determining pier scour. Measurements of pier scour collected at FHWA's Turner-Fairbank Highway Research Center Hydraulics Laboratory were used to study a relationship between scour depth and stream power in order to develop a practical application of the Erodibility Index to scour prediction.[12] The Erodibility Index to quantify the ability of rock material to resist erosion and the development of a relation between the index and stream power for contraction and local scour at piers and abutments appears feasible, but needs further research.

The Index is defined as:

$$K = M_s \,.\, K_b \,.\, K_d \,.\, J_s$$

where:

K	=	Erodibility Index
M_s	=	Intact mass strength number
K_b	=	Block size number
K_d	=	Discontinuity or inter-particle bond shear strength number
J_s	=	Orientation and shape number

The values of these parameters are determined by making use of field and/or laboratory observations, and tables published in Annandale,[9] Kirsten,[11] and the National Engineering Handbook.[13] The mass strength number M_s represents the strength on an intact representative sample of the earth material without regard to geologic heterogeneity within the mass. K_b is a function of the Rock Quality Designation (RQD) in the case of rock and is a function of an effective particle diameter in the case of granular material. K_d represents the shear strength at the interface of failure planes, such as fissures or slickensides in clay, or joints and fractures in rock. This value can be estimated from the properties of joint and fracture planes in the case of rock, or from tri-axial tests in the case of granular materials. The orientation and shape number is a function of the dip and strike of rock, and of the relative shape of individual rock blocks. J_s accounts for the structure of the ground with respect to stream flow. It is a complex function that considers orientation and shape of individual blocks with respect to stream flow. Additional description of the variables is given in Annandale,[9] and Annandale and Kirsten.[14]

For this application stream power is defines as:

$$P = \gamma q \, S$$

where:

P = Stream power, kg/s, (lb/s)
γ = Unit weight of water, kg/m^3, (lb/ft^3)
q = Unite discharge of water, m^3/s, (ft^3/s)
S = Slope of the energy grade line, m/m (ft/ft)

M.5 REFERENCES

1. NTSB,1988, “Collapse of the New York Thruway (I-90) Bridge over the Schoharie, Creek, Near Amsterdam, New York, April 5, 1987,” NTSB/HAR-88/02, NTSB, Washington, D.C.

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ATTACHMENT 1

FHWA 1991 MEMORANDUM "SCOURABILITY OF ROCK FORMATIONS"

Federal Highway Administration
Date: July 19, 1991
Subject: Scourability of Rock Formations

From: Chief, Bridge Division
Office of Engineering

To: Regional Federal Highway Administrators
Federal Lands Highway Program Administrator

Usually rock is regarded as the best bearing material for structural foundations, however, there are conditions, such as sinkholes in limestone, weathering and scourability which can present problems. Bridge foundation failures have occurred due to scour of rock or rock-like materials. This memorandum presents interim guidance on empirical methods and testing procedures to assess rock scourability until results of ongoing research permit more accurate evaluation procedures. These empirical methods are commonly used by geotechnical engineers and geologists to determine rock mass. engineering properties such as, allowable bearing pressures for shallow and deep foundations. Footing elevations on rock should be conservatively selected based on experience and the indirect qualitative interpretation of the methods discussed below. While safety of the traveling public is the primary design consideration, bridge designers should recognize that scour assumptions have a significant impact on the cost and constructibility of foundations and overly conservative assumptions should be avoided.

Academic geologic studies have shown that even the hardest of rocks can scour when exposed to moving water. However, the time for a finite depth of scour, is not possible to predict at this time. Empirical methods can be used to approximate rock scourability within the lifetime of a structure. Several properties contribute to the quality, bearing capacity and soundness of rock. Hence, no single -index property will correctly assess the potential for scour. Designers are encouraged to utilize a combination of the following methods to assess rock scourability until a more quantitative procedure becomes available.

1. <u>Subsurface Investigation</u>

The objective of a subsurface investigation for shallow foundations on rock should permit an identification of rock type, determination of discontinuity frequency and recovery of high quality rock core samples for testing and evaluation. The number of drill holes par substructure unit should be based on the footing size, structure criticality and variability of subsurface conditions. A minimum of one boring per substructure unit and a 3.3 meter (10 foot) minimum core length below the bottom of footing are recommended.

Rock core sample quality is greatly influence by drilling equipment and technique. Poor drilling techniques will penalize rock quality) assessments by lowering core recovery and rock quality designation (RQD). Rock cores should be obtained with NX diameter size core barrels 5.4 centimeters, (2 1/8 inch) or larger. Double or triple tube core barrels should be used for all structural foundation projects.

2. Geologic Formation/Discontinuities

Rock type and frequency of discontinuities have a significant impact on engineering properties. The three classes of rock based on geologic origin are igneous, sedimentary and metamorphic. Igneous rocks are formed by solidification of molten material from deep beneath the earth's surface. They are generally uniform in structure and lack stratification and cleavage planes. Examples of igneous rock are granite, diorite, gabbro, basalt and diabase.

Sedimentary rocks are products of disintegration and decomposition by weathering of preexisting rock. These rocks are formed by mechanical cementation, chemical precipitant and pressure. Examples of sedimentary rock are sandstone, limestone, dolomite, shale and chert. Some common - features of sedimentary rock are rounded grains, stratifications, inclination of bedding planes and abrupt color changes between layers.

Metamorphic rock is formed from igneous or sedimentary rocks which have been altered physically or chemically by intense heat and pressure. Examples are quartzite, marble. slate and schist. Some features include the ease with which parallel layers break into slabs. In general, harder and more sound rock is less susceptible to scour.

If rocks were free of defects, then the allowable bearing pressure could be taken conservatively as the average compression strength of unconfined rock core samples. However, rock masses are seldom free of imperfections and fractures which have a significant influence on rock behavior. The spacing of discontinuities is an indication of overall rock quality. Spacing is measured-d as the perpendicular distance between parallel discontinuities. Measurement is easily accomplished for rock outcrops, but is difficult from vertical drill holes. Drill cores with one fracture or less per foot would indicate a good quality rock mass. High fracture frequency (five or six fractures per foot) would indicate a poorer quality rock which would be considerably weaker and more scourable.

3. Rock Quality Designation (RQD)

The RQD value is a modified computation of percent rock core recovery that reflects the relative frequency of discontinuities, the compressibility of the rock mass & may say indirectly be utilized as a measure of scourability. The RQD is determined by measuring and summing all the pieces of sound rock 10.2 centimeters (4 inches) and longer in length in a core run, and dividing this by the total core run length. The RQD should be computed using NX diameter cores or larger and on samples from double tube core barrels. Figure I provides an example of RQD computation and a relationship between RQD and rock quality. Table I provides a relationship between RQD, rock type and allowable bearing pressures. Scourability potential will increase as the quality of the rock becomes poorer. Rock with an RQD value less than 50 percent should be assumed to be soil-like with regard to scour potential.

4. Unconfined Compressive Strength (qu, ASTM D29361)

The primary intact rock property of interest for foundation design is unconfined compressive strength. Although it is known that strength of jointed rocks is generally less than individual units of the rock mass, the unconfined compressive strength provides an upper limit of the rock mass bearing capacity and an index value for rock classification. In general, samples with unconfined strengths below 1724 Kpa (250 psi) are not considered to behave as rock. As unconfined compressive strength increases, bearing capacity generally increases and scourability decreases. There is only a generalized correlation between unconfined compression strength and scourability.

5. Slake Durability Index (SDI, International Society of Rock Mechanics)

The SDI is a test used on metamorphic and sedimentary rocks such as slate and shale. An SDI value of less than 90 indicates a poor rock quality. The lower value of SDI, the more scourable and less durable the rock.

6. Soundness (AASHTO T104)

The laboratory test for soundness of rock uses a soaking procedure in magnesium or sodium sulfate solution. Generally, the less sound the rock, the more scourable it will be. Threshold loss rates of 12 (sodium) and 18 (magnesium) percent can be used as an indirect measure of scour potential.

7. Abrasion (AASHTO T96)

The Los Angeles Abrasion Test is an empirical test to assess abrasion of aggregates. In general, the less a material abrades during this test, the less it will scour. Materials with loss percentages greater than 40 should be considered scourable.

The above procedures can be effectively utilized to produce a rational screening process to assess rock scourability until more quantitative, methods become available.

Stanley Gordon

ROCK QUALITY DESIGNATION (RQD) EXAMPLE

An example is given below from a core run of 152.4 cm (60 inches). For this particular case the total core recovery is 127 cm (50 inches) yielding a core recovery of 83 percent. On the-modified basis, only 99 cm (34 inches) are counted and the RQD is 65 percent.

CORE RECOVERY, in	MODIFIED CORE RECOVERY, in
10	10
2	
2	
3	
4	4
5	5
3	
4	4
6	6
4	4
2	
5	5
50	39

% Core Recovery = 50/60 - 83%; RQD= 39/60 = 65%

A general description of the rock quality can be made from the RQD value.

RQD (ROCK QUALITY DESIGNATION)	DESCRIPTION OF ROCK QUALITY
0- 25	very poor
25-50	poor
50-75	fair
75- 90	good
90 -100	excellent

FIGURE 1

TABLE I

RECOMMENDED ALLOWABLE BEARING PRESSURE FOR FOOTINGS ON ROCK

MATERIAL	ALLOWABLE CONTACT PRESSURE (Kpa)
Such igneous and sedimentary rock as crystalline bedrock, including granite, diorite, gneiss, traprock; and hard limestone, and dolomite, in sound condition:	
RQD = 75 to 100 percent	11491 (120 tsf)
RQD = 50 to 75 percent	6224 (65 tsf)
RQD = 25 to 50 percent	2873 (30 tfs)
RQD - 0 to 25 percent	958 (10 tfs)
Such metamorphic rock as foliated rocks, such as schist or slate; and bedded limestone, in sound condition:	
RQD > 50 percent	3830 (40 tfs)
RQD < 50 percent	958 (10 tfs)
Sedimentary rocks, including hard shales and sandstones, in sound condition:	
RQD > 50 percent	2394 (25 tfs)
RQD < 50 percent	958 (10 tfs)
Soft or broken bedrock (excluding shale), and soft limestone:	
ROD > 50 percent	1149 (12 tfs)
ROD < 50 percent	766 (8 tfs)
Soft shale	383 (4 tfs)