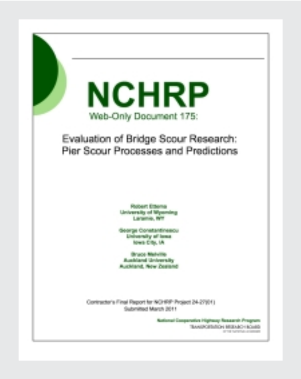

Evaluation of Bridge Scour Research: Pier Scour Processes and Predictions

DETAILS

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Suggestions, technical and editorial, were provided by the Panel appointed for this NCHRP Project. The Panel members are listed in the table below. The research team gratefully acknowledges the suggestions provided by the Panel.

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a = pier width (without debris)
a_e = width of an equivalent uniform pier
a^* = effective pier width
a_F = pier foundation width (e.g., caisson width)
a_p = projected width of the pier; $a_p = b\sin\theta + a\cos\theta$ for rectangular piers
b = pier length
B_d = width of floating debris raft
c = parameter describing soil cohesion effects
D = median diameter of bed particle
D_x = diameter of bed material of which x% are smaller
Eu = Euler number; $Eu = V^2/ga$
f = Darcy-Weisbach resistance coefficient
Fr = Froude number; $Fr=V/(gy)^{0.5}$
Fr_c = critical Froude number $Fr_c=V_c/(gy)^{0.5}$
Fr_a = critical Froude number defined with pier width $Fr_a=V/(ga)^{0.5}$
Fr_d = densimetric Froude number; $Fr_d = \dfrac{V}{\left(\dfrac{g(\rho_s-\rho)d_{50}}{\rho}\right)^{1/2}}$
g = acceleration of gravity
H = depth of region within the scour hole covered by coarse sediment
H_d = height of debris
L_d = length of debris upstream from the pier face
L_p = pier distance from the abutment toe
K_s = pier aspect ratio parameter
K_θ = pier alignment parameter
K_I = flow intensity parameter
n=shedding frequency of large scale vortices in the wake of the pier
q = average discharge intensity upstream from the bridge (m^2/s)
q_2 = local discharge intensity in contracted channel (m^2/s)
Re = Reynolds number; $Re=\rho Va/\mu$
St = Strouhal number; $St=na/V$
t = time;
t^* = equilibrium time scale; $t^*=Vt_e/a$
t_e = time to develop the equilibrium depth of scour
T_d = thickness (vertical dimension) of floating debris or ice raft
V_* = shear velocity
V_{*c} = critical shear velocity for bed sediment entrainment
V = mean approach flow velocity; $V = V_*\sqrt{8/f}$
V_a = threshold velocity for the transition from clear-water to live-bed conditions for non-uniform sediments in Melville (1997) method
V_c = critical approach flow velocity for entrainment of bed sediment
V_c'= incipient velocity for local scour at a pier

V_{lp} = live-bed peak velocity in the Sheppard method, similar to V_a in the Melville (1997) method
V_{cd_x} = critical velocity for incipient motion for grain size D_x
V_{icd_x} = approach velocity required to initiate scour at the pier for grain size D_x
W_d= width of debris in the direction normal to the flow
y = undisturbed approach flow depth
y_{cs} = uniform flow depth over a flat bed of particle sizes equivalent to the upstream coarse surface particles
y_{fs} = uniform flow depth over a flat bed of particle sizes equivalent to the underlying surface fine particles
y_f = flow depth at the abutment toe or on the floodplain for abutments on floodplains
y_r = regime depth in Blench (1967) pier scour formula
y_S = local maximum total scour depth
y_{Sf} = local maximum scour depth in the layer of fine sediment
$y_{s\text{-}p}$ = scour-depth component for the pier column in the flow
$y_{s\text{-}cap}$ = scour-depth component for the pile cap or footing in the flow
$y_{s\text{-}pile\ group}$ = scour-depth component for the piles exposed to the flow
y_{Spier} = scour depth at the pier with the abutment present
y_{S0pier} = scour depth at the pier without the abutment
$y_{Sabutment}$ = abutment scour depth
y_t = local scour depth at current time t
Y = level of the top of the footing, cap or caisson measured from general bed level

Greek Symbols

γ = specific weight of water
μ = molecular viscosity of water
ν = kinematic viscosity of water
Ω = parameter describing the shape of the pier face (upstream side)
ρ = water density
ρ_s = sediment density
σ_g = geometric standard deviation of the boundary particle size distribution
τ_1 = grain roughness component of bed shear
τ_c = critical shear stress at threshold of motion
θ = angle of approach flow relative to pier alignment

EXECUTIVE SUMMARY

Introduction

This report evaluates the current state of knowledge regarding bridge-pier scour, assesses leading methods for reliable design estimates of scour depth, proposes a structured methodology for scour-depth estimation for design purposes, and indicates pier-scour aspects in need of further research. It focuses particularly on research information obtained since 1990, showing that this information provides considerable new insights that compel the need to change the design method currently recommended by the principal authoritative design guides (notably FHWA's HEC-18[1], and AASHTO[2]) and used widely by bridge-engineering practitioners. Additionally, it indicates that several important aspects of pier scour processes remain inadequately understood and not yet incorporated into design methods.

Problem Statement

Pier scour is among the more complex and challenging water flow and boundary erosion phenomenon to understand, let alone formulate. This is true for all types of piers, though pier shape complexities and bridge-site complications, such as woody debris accumulation, readily complicate scour depth estimation. The erosive flow field is a class of junction flow (i.e., flow at the junction of a structural form and a base plane), a notably three-dimensional, unsteady flow field marked by interacting turbulence structures. Additionally, the foundation material's capacity to resist erosion typically is uncertain.

The numerous complexities associated with pier scour have caused it to persist as an active topic of research since 1990. Though the addition of new data may be helpful in defining the influences of parameters affecting pier scour, there is a sense that a form of stock-taking is needed to assess the current state of knowledge regarding pier scour, and determine whether the present design method in HEC-18 adequately reflects current knowledge. Accordingly, there is a need for an incisive evaluation of the overall results of bridge-scour research published since 1990, especially in the context of determining how present design methods should be updated, revised, replaced, or confirmed as being suitably accurate.

Objectives

The evaluation's principal objectives are as follow:

1. Complete a detailed literature review of pier-scour processes and prediction, concentrating especially on research conducted or published after 1990. The review cites some publications prior to 1990, to give a sense of the temporal development of knowledge about pier scour, and because many of the design methods use data and concepts developed prior to 1990;

[1] Federal Highway Administration, Hydrologic Engineering Circular 18, "Evaluating Scour at Bridges," by Richardson and Davis (2001)
[2] AASHTO ~ Association of American State Highway and Transportation Officials

2. Summarize the state of knowledge on bridge-pier scour processes, doing so in a way that explains how variations in the flow, sediment, and geometrical variables (thereby the main design parameters) influence scour. Also discuss how scour is affected by channel geomorphology, boundary material (sediment, soil, rock), the proximity of bridge components, and the accumulation of woody debris or ice;
3. Delineate proven relationships between scour depth and the various parameters influencing scour at bridge piers;
4. Evaluate the technical adequacy, strengths and limitations of the leading existing methods to predict scour. An important consideration is whether the commonly used methods for scour estimation adequately reflect current understanding of scour processes. Discuss the uncertainties associated with the leading methods for scour-depth prediction, and address any unresolved issues associated with the methods;
5. Propose specific research findings for adoption in AASHTO policies and procedures. Also, document the ranges of applicability and limitation of research findings proposed for adoption into AASHTO policies and procedures; and,
6. Recommend research needed (field studies, laboratory studies, and numerical simulations) to fill gaps where research results are not yet conclusive enough for adoption by AASHTO and wide-scale use by practitioners.

Evaluation Considerations

Several key considerations guide the evaluation:

1. The flow field and the potential maximum scour depth, at a pier scale, change in accordance with three variables – effective pier width, undisturbed approach flow depth, y, and erodibility of the boundary material in which the pier is sited. Of these variables, effective pier width and flow depth are of prime importance, because they determine the overall structure of the flow field. Effective pier width, a^*, embodies pier form and alignment relative to approach flow. For non-cohesive boundary material (silts, sand, gravel), material erodibility is expressible in terms of a representative particle diameter.

2. To understand pier-scour processes and develop reliable relationships for design estimation of scour depth, it is necessary to comprehend the main flow-field features driving scour, and how the features may adjust in importance with varying pier sizes and shapes, and flow conditions. The flow field differs substantially for the narrow-pier and wide-pier categories of pier scour, with the transition-pier category being intermediate between them. The report defines these categories approximately as

    - Narrow piers ($y/a^* > 1.4$), scour typically deepest at the pier face
    - Transition piers ($0.2 < y/a^* < 1.4$), scour depth deepest at pier face, and is influenced by y/a^*
    - Wide piers ($y/a^* < 0.2$), for which scour typically is deepest at the pier flank. It is relatively rare for bridge piers to be in this category

3. Because considerable uncertainty attends flow conditions and boundary material at bridge waterways, design prudence requires estimation of a potential maximum scour depth, rather than scour-depth prediction. Potential maximum scour depth is the greatest scour depth attainable for a given pier flow field, and can be determined using the primary variables named in item 2. Lesser scour depths result as additional variables are considered, but the uncertainties associated with the variables diminish the estimation reliability. Besides the uncertainties related to foundation material erodibility, the temporal development of scour entails significant uncertainty. Prediction (not design estimation) of scour depth for most pier sites usually involves a high level of uncertainty.

Several factors alter pier flow fields and complicate design estimation of pier scour. Factors affecting pier flow field include flow influences exerted by increased complexity of pier geometry, adjoining bridge components (abutment or submerged bridge deck), debris or ice accumulation, and channel morphology. Additionally, flow conditions commonly vary with time, as the dominant scour events occur during the passage of flood flows. Factors affecting boundary erosion include uncertain erosion characteristics of material (clay, rock), layering of boundary material, and protective vegetation.

Approach to Scour-Depth Estimation

The large number of parameters that may influence pier scour at a bridge site makes it infeasible for a single method to provide reliable design estimates of pier scour depth. A structured, or graduated, approach is needed, whereby the varying levels of pier shape or site complexity are matched with existing practical methods for scour-depth estimation. This approach, though mentioned in HEC-18 and a few other publications on scour-depth estimation, requires stronger expression in design guides.

The present report approaches scour depth estimation in terms of a hierarchy of pier shape and site complexity levels:

1. Simple, single-column pier forms;
2. Common pier forms comprising a more complex geometry;
3. Common pier forms in complex situations (e.g., debris accumulation); and,
4. Uncommon or Complex pier forms and situations.

This approach is not entirely new. It is usual for difficult or complex pier circumstances to receive additional design attention; for instance, large piers of complex configuration, and piers in complicated waterway sites.

Conclusions

The evaluation's main conclusions are:

1. The literature review conducted for the evaluation shows that, since 1990, substantial advances have been made in understanding pier-scour processes, and predicting scour depth. In particular, the following aspects of pier scour have advanced:

i. The roles of variables and parameters defining pier scour processes;
ii. The leading predictive methods for scour-depth prediction better reflect parameter influences;
iii. Knowledge regarding pier scour in clay and weak rock;
iv. Insight into pier-site complications caused by debris and ice, and by interaction with bridge components (abutment and bridge deck); and,
v. The capacity of numerical modeling to reveal the three-dimensional and unsteady flow field at piers in ways that laboratory work heretofore has been unable to provide.

These advances address research problems identified in NCHRP Project 24-8, *Scour at Bridge Foundations: Research Needs*, published in 1996. They also address aspects of pier scour not envisioned for NCHRP 24-8, such as the roles of turbulence structures within the pier flow field, three-dimensional numerical modeling, and scaling issues in the conduct of hydraulic models of pier scour.

2. The current state of knowledge on pier-scour processes is such that the ensuing aspects of pier scour remain insufficiently understood:

i. The pier flow field, especially how it systematically changes with variations of the two primary length scales (effective pier width and flow depth). In other words, more work is needed to define the systematic changes in the flow fields associated with the narrow- , transition-, and wide-pier categories of pier scour;
ii. Scour of boundary materials whose erosion characteristics are inadequately understood (some soils, rock). However, existing reliable data indicate that scour depths in cohesive soils and weak rock do not exceed those in non-cohesive material;
iii. Quantification of factors further complicating pier flow field (notably debris or ice accumulation, proximity of bridge components, channel morphology) and erodibility of foundation material (especially layered material where the surface layer protects lower layers); and,
iv. Temporal development of pier-scour depth.

3. The evaluation outlines the well-understood relationships between scour depth and significant parameters. A group of primary parameters are identified that define the structure and geometric scale of the pier flow field, and therefore determine the potential maximum scour depth. The secondary parameters characterize scour-depth sensitivities within the geometric scale limit, and normally lead to scour depths less than the potential maximum scour depth. The values of the secondary parameters are subject to considerable uncertainty at pier sites.

The primary parameters are –

y/a, which indicates the geometric scale of the pier flow field in terms of approach-flow depth, *y*, and pier width, *a*

a/D, which relates the length scales of pier width and median diameter of bed particle, *D*

Ω, *a/b*, and θ , which define pier face shape, aspect ratio of pier cross-section (face width/ pier length), and approach flow alignment to pier, respectively. These parameters may be merged with pier width, *a*, to form the compound variable *a** = effective pier width. It can be useful to express the two length-scale parameters as *y/a** and *a*/D*

The evaluation also explains the limiting extents to which parameter influences can be isolated from each other. Some variables exert multiple influences. Consequently, a limit is soon reached with the approach of formulating a predictive method based on a semi-empirical assembly of parameter influences.

4. An important conclusion drawn from the evaluation is the need to transition from the present semi-empirical method for design estimation of scour depth used in HEC-18[3] (Richardson and Davis 2001) to a new method better reflecting the relationships between the primary variables and the potential maximum scour depth at a pier. A new method is recommended, the semi-empirical, Sheppard-Melville method as advanced during NCHRP Project 24-32 (Sheppard et al. 2011).

   In terms of estimating a potential maximum scour depth, related to the scale of the pier flow field, the Sheppard-Melville method simplifies to

$$\frac{y_s}{a^*} = 2.5\tanh\left(\left(\frac{y}{a^*}\right)^{0.4}\right) \qquad (1)$$

   This equation can be used for piers founded in sediment or cohesive soil. The designer can use the full Sheppard-Melville method if wishing to account for the influences of a^*/D and V/V_c, where V is the mean approach flow velocity and V_c is the critical approach flow velocity for entrainment of bed sediment. The designer, however, must recognize the uncertainties introduced with consideration of these parameters, and with attempting to account for the temporal development of scour.

   The Sheppard-Melville method better reflects flow-field changes and thereby scour processes, as expressed in terms of the parameters of primary importance, and therefore is more readily extended to the transition- and wide-pier categories; i.e., it more expressly includes the length scales, *y/a** and *a*/D*, and includes the flow intensity parameter, V/V_c. Additionally, it is the method developed (under NCHRP 24-32) in an effort to account for flow-field adaptation to variations of

[3] And adopted in AASHTO policies and procedures

y/a and *a/D*. A useful aspect of the Sheppard-Melville method is that it can be simplified to reflect potential maximum scour depth associated with the three pier flow-field categories (narrow, transition, and wide). A weakness of the Richardson and Davis (2001) method is that it does not.

Full use of the Sheppard-Melville method for predicting scour depth presently requires a modicum of further research regarding prediction of scour depth during live-bed scour at piers in the transition-pier category, and to a certain extent in the wide-pier category.

The Richardson and Davis (2001) method has been in use for the past several decades, and has been refined over time. It is well calibrated for estimating scour depth for piers in the narrow-pier category of scour, and into the transition-pier category. Its scour depth estimates for these applications concur reasonably well with those obtained using the Sheppard-Melville method. Nonetheless, the method is shown not to reflect scour processes as well as does the Sheppard-Melville method.

5. Due to the limits inherent in semi-empirical formulations of pier-scour depth, the evaluation proposes the use of a structured methodology for design estimation of pier-scour depth. The methodology, summarized in Table 1, addresses four levels of pier-site complexity. As pier-site complexity increases a graduated shift occurs from design reliance on a semi-empirical method (Sheppard-Melville) to hydraulic modeling possibly aided by numerical modeling.

Table 1 Summary of proposed structured design methodology

Pier Design Complexity (Pier Form and/or Pier Site)	**Design Method**
i. Simple single-column pier forms (narrow- and transition-pier categories)	**Semi-empirical method:** Transition from current HEC-18 method (Richardson and Davis 2001) to Eq. (7.1) as simplified from Sheppard et al. (2011), NCHRP Project 24-32)
ii. Common pier forms (piers with multiple components; e.g., column, pile cap, pile group)	**Semi-empirical method with effective pier-shape and alignment factors** Consider hydraulic model to validate scour-depth estimate
iii. Common pier forms in complicating situations (debris or ice accumulation, abutment proximity, bridge-deck	**Empirical method combined with procedure to address scour contribution of site complication**

submergence)	Consider hydraulic model to validate scour-depth estimate
iv. Complex or unusual pier situations (where reliable method or information on parameter influences does not exist; e.g., scour for wide-pier category)	**Hydraulic modeling, possibly aided by numerical modeling** An approximate scour-depth estimate may be obtained using an empirical method suitably developed for the wide-pier category

The methodology enables the designer to account for the scour processes, yet also recognize the limits of existing semi-empirical methods for scour-depth estimation. The leading semi-empirical methods (Sheppard-Melville, Richardson and Davis) for scour-depth prediction were developed using data for simple pier forms, and adapted for common pier forms. The accuracy of the methods reduces as pier form complexity increases.

6. The following specific proposals are made with respect to the updating of HEC-18 and the AASHTO's manuals:

   i. Adopt the structured design methodology described in Chapter 7;
   ii. After the completion of further research, replace the Richardson and Davis (2001) method with Eq. (1) above, and have the option to use the Sheppard-Melville method. The former method inadequately reflects scour processes, though is well-adapted empirically to scour data. The latter method better reflects scour processes. A modicum of further research is needed to complete the development of the Sheppard-Melville method. The present evaluation has occurred at a transitional period when it is recognized that the present design method for simple and common pier forms should be replaced, but a satisfactory replacement method is not fully completed.

      The present version of HEC-18 should not be updated until after completion of the small amount of additional research associated with the full development of the Sheppard-Melville method.

7. Prioritized research needs are given for design methodology and scour processes, respectively. In a similar manner as used for NCHRP 24-8, the priorities are assigned as critical, high, medium, and low.

   For design methodology development, two research topics are of critical priority:

   i. Estimation of potential maximum scour depth for piers in the changeover range from transition- to wide-pier categories, especially for live-bed conditions; and,
   ii. A reliable method for estimating potential maximum scour at piers in the wide-pier category.

For understanding scour processes, no research topics are identified as critical priority, though several are of high priority, all of which concern improved understanding of how site complications affect pier flow field:

i. Debris accumulations;
ii. Flow field at common pier shapes with multiple components (notably, column, pile cap, piles);
iii. Flow field interaction with bridge components, such as a bridge deck or abutment; and,
iv. Flow field interaction with channel features.

In terms of longer-range research development, a transition underway is recent advances in experimental and numerical techniques used to investigate bridge scour processes that can capture the dynamics of the main turbulent coherent structures (e.g., horseshoe vortex system at the base of the pier, eddies shed in the separated shear layers, large-scale rollers in the wake behind the pier) affecting pier scour. The recent advances include Particle Image Velocimetry and Laser Doppler Velocimetry techniques for experimental studies; and, Large Eddy Simulation, and Detached Eddy Simulation, computer-simulation investigations using fully three-dimensional non-hydrostatic methods. These experimental and computational approaches promise new insight into understanding the fundamental physics of the flow and sediment transport processes at bridge piers and can lead to the development of more accurate relationships to predict local scour depth.

CHAPTER 1
INTRODUCTION

1.1 Introduction

Scour of foundation material at bridge piers is a long-standing design concern for bridge crossings of waterways. This report evaluates the current state of knowledge regarding bridge-pier scour, assesses the leading current methods to provide reliable design estimates of scour depth, and proceeds to recommend a structured approach to scour-depth estimation for design purposes. It focuses particularly on research information obtained since 1990, showing that this information provides considerable new insights that compel the need to change the approach currently recommended by the principal authoritative design guides (notably FHWA's HEC-18[4], and AASHTO[5]) and used widely by bridge-engineering practitioners. Additionally, it indicates that several important aspects of pier scour processes remain inadequately understood and not yet incorporated into design methods.

A prominent theme running through the report is the importance of understanding the pier flow field and its erosion capacity for the range of pier sizes, and erodibility of various pier foundation materials. The flow field is highly three-dimensional and unsteady, and differs substantially in accordance with varying combinations of pier width and form, flow depth, and boundary-material diameter (or erosion resistance of channel boundaries). Its capacity to erode the foundation material at a pier often is difficult to predict accurately.

Flow field complexities and diversity of foundation materials prevent design method elegance for scour-depth estimation. Instead, design is reliant on empirical and semi-empirical relationships, and on hydraulic models, to relate scour depths to the erosion capacity of pier flow fields, and erosion resistance of boundary materials. The large variation in factors potentially influencing pier flow field and boundary erosion resistance requires design methodology sufficiently comprehensive to account for the more important individual parameter pier scour influences to be considered. Yet the methodology must also treat pier scour from a holistic, or systems analysis, perspective when the number of parameters is too numerous, or the parameters are insufficiently independent, to be described practically in terms of a series of individual parameter influences. The report presents such a methodology.

The evaluation and proposals presented here were prepared for eventual use in updating the two AASHTO manuals *Policy for Design of Highway Drainage Facilities* and *Recommended Procedures for Design of Highway Drainage Facilities*, so that these manuals present the best available guidelines for pier scour estimation and countermeasure design, and strong directions as to further research. The report's proposals are intended for eventual use by AASHTO in developing policies and

[4] Federal Highway Administration, Hydrologic Engineering Circular 18, "Evaluating Scour at Bridges," by Richardson et al. (2001)

[5] AASHTO ~ Association of American State Highway and Transportation Officials

procedures in the bridge scour areas. The evaluation also identifies research and education needs regarding pier scour.

1.2 Motivation

The need to evaluate present knowledge about pier scour, and determine the extents to which existing scour-estimation methods reflect this knowledge, has been expressed in several publications prepared since 1990 by national agencies and societies in the US: e.g., reports stemming from NCHRP Project 24-08, "Scour at Bridge Foundations: Research Needs" (Parola et al. 1996, Lagasse et al. 2004), also NCHRP Report 417 (Parola et al. 1998), Kattell and Eriksson (1998), U.S. Geological Survey (2002), and Mueller and Wagner (2005). It is generally considered that widely used scour estimation methods inadequately reflect present knowledge about scour processes, in particular how the primary length scales -- pier width, flow depth, and sediment size -- are of primary importance for defining the structure and geometric scale of the pier flow field, and thereby scour depth.

The present evaluation shows that, while the individual scour influences of the many bridge waterway variables are now well understood for simple cylindrical, or common pier designs, and that recently developed scour estimation methods attempt to encompass these influences, the principal difficulty confronting reliable scour depth estimation at the moment is development of a method that comprehensively accounts for the diverse site factors complicating scour development and estimation (e.g., debris accumulation at piers, piers of large and unusual design). The evaluation suggests that accurate scour-depth estimation is not always possible, and consequently that greater use might be made of contemporary methods for real-time monitoring of pier foundation conditions (so-called smart monitoring). Of particular concern is the inadequate capacity of the established methods to estimate reliable scour depths at wide piers (i.e., piers of width greater than nominally ten feet).

Information used for the evaluation was drawn from a broad range of sources, including agency reports, books, and technical papers. Close attention was given to recent or current NCHRP projects on pier scour: notably, scour at wide piers and long, skewed piers (NCHRP Project 24-32, documented in Sheppard et al. 2011); effects of debris on bridge-pier scour (NCHRP Project 24-26); pier scour in cohesive soils (NCHRP Project 24-15(1)); and, pier scour in rock (NCHRP Project 24-29). There is considerable overlap between NCHRP 24-32 and the present evaluation, insofar that both studies examine the variables influencing scour, and compare the veracity of methods for estimating scour depth. Also of major use is the book Bridge Scour by Melville and Coleman (2000). Dr Bruce Melville is a member of the research teams for NCHRP Projects 24-27(01) and 24-32.

Companion studies to the present evaluation are NCHRP 24-27(02) "Evaluation of Abutment Scour Research: Processes and Prediction"; and NCHRP 24-27(03) "Evaluation of Bridge-Scour Research: Geomorphic Processes and Predictions." Bruce Melville and Robert Ettema serve on the research teams for NCHRP projects 24-27(01) and 24-27(02).

1.3 Objectives

The evaluation's principal objectives are as follow:

1. Complete a detailed literature review of pier-scour processes and prediction, concentrating especially on research conducted or published after 1990. The review cites publications prior to 1990, to give a sense of the temporal development of knowledge about pier scour, and because many of the design methods use data and concepts developed prior to 1990;

2. Summarize the state of knowledge on bridge-pier scour processes, doing so in a way that explains how variations in the flow, sediment, and geometrical variables (thereby the main design parameters) influence scour. Also discuss how scour is affected by channel geomorphology, boundary material (sediment, soil, rock), the proximity of bridge components, and the accumulation of woody debris or ice;

3. Delineate proven relationships between scour depth and the various parameters influencing scour at bridge piers;

4. Evaluate the technical adequacy, strengths and limitations of the leading existing methods to predict scour. An important consideration is whether the commonly used methods for scour estimation adequately reflect current understanding of scour processes. Discuss the uncertainties associated with the leading methods for scour-depth prediction, and address any unresolved issues associated with the methods;

5. Propose specific research findings for adoption in AASHTO policies and procedures. Also, document the ranges of applicability and limitation of research findings proposed for adoption into AASHTO policies and procedures; and,

6. Recommend research needed (field studies, lab studies, and numerical investigations) to fill gaps where research results are not yet conclusive enough for adoption by AASHTO and wide-scale use by practitioners.

The present report is written in a form so that its information can be readily incorporated in the bridge-scour sections of the AASHTO manuals *Policy for Design of Highway Drainage Facilities* and *Procedures for Design of Highway Drainage Facilities*.

1.4 Key Considerations

The set of objectives listed in Section 1.3 required that the evaluation assess several key considerations:

1. To understand pier-scour processes and develop reliable relationships for design estimation of scour depth, it is necessary to ***understand the main flow-field features driving scour for varying pier situations;***

2. The flow field, and the potential maximum scour depth, at a pier scale change in accordance with ***three variables – effective pier width, flow depth, and erodibility of the foundation material*** in which the pier is sited. Of these variables, effective pier width and flow depth are of prime importance, because they determine the overall structure of the flow field. Effective pier width embodies pier form and orientation to approach flow. For non-cohesive foundation material (silts, sand, and gravel) erodibility is expressible in terms of a representative particle diameter.

3. Because considerable uncertainty attends flow and foundation material at bridge waterways, design prudence requires ***estimation of a potential maximum scour depth***, rather than scour-depth prediction. Potential maximum scour depths are the greatest scour depth attainable for a given pier flow field, and can be determined using the primary variables named in item 2. Lesser scour depths result as additional variables are considered, but the uncertainties associated with the variables diminish the estimation reliability. Prediction (not design estimation) of scour for most pier sites involves a significant level of uncertainty.

4. It is important to consider how pier flow field is affected by conditions in an entire bridge waterway**.** ***Several waterway factors alter pier flow field and complicate design estimation of pier scour***. They include flow influences exerted by increased complexity of pier geometry, adjoining bridge components (abutment or submerged bridge deck), debris or ice accumulation, and channel morphology. Factors affecting boundary erosion include uncertain erosion characteristics of material (clay, rock), layering of boundary material, and protective vegetation.

The report identifies relatively simple pier forms and situations, as well as common pier forms, and shows how these may be complicated by fairly common processes at bridge waterways. Such processes include abutment proximity, debris accumulation, channel morphology issues, and variable strata of boundary material. It also shows how the current scour-estimation methods may have limited applicability in certain situations where piers are of unusually large size or uncommon form.

The three variables mentioned in item 2, and depicted in Figure 1-1, control the scale of maximum scour-hole depth, doing so principally by controlling the flow field and its erosive strength at a pier. Arguably, effective pier width and flow depth set maximum scour depth at a pier. Approach flow velocity is an important variable, but secondarily so in terms of defining the maximum <u>design</u> scour depth at a pier site; in this context, approach velocity is important in defining the effective width and form of a pier.

The key considerations point to the need for a structured methodology for design estimation of pier scour depth. Figures 1-2 and 1-3, which show that pier location often cannot be considered in isolation from other components of a bridge waterway, illustrate the need for the methodology. The former figure shows the pier supports for a long multi-span bridge over a wide channel, while the latter figure shows the pier supports for a shorter, three-span bridge over a comparatively small channel. The central pier in

Figure 1-2 could be considered in isolation from the complicating considerations of abutment proximity and channel alignment, but the local flow fields at other piers are to varying extents affected by flow around the abutments and over the channel's floodplain. For the shorter bridge shown in the latter figure, the two piers cannot be considered in isolation, and are markedly affected by flow around the abutment and over the floodplain, and likely by variable erodibility of foundation material.

1.5. Complexities

Using Figures 1-1 through 1-3, it is useful to point out the main sources of substantial complexity that complicate the development of reliable comprehensive (accounting for all important variable influences) design relationships for estimating scour depth at piers:

1. Complexity of flow field (evolving; large-scale turbulence; highly three-dimensional); overall difficulty in comprehending all the phenomena involved and their interactions;

2. The fundamental problem of simultaneously scaling three lengths (flow depth, bed material size, structure size);

3. Variations in channel boundary material;

4. Differences in pier structure;

5. The complicating interaction of pier scour and other processes at bridge waterways, such as accumulation of woody debris, ice, bridge over-topping, abutment proximity, channel morphology, and fluvial bedforms;

6. The potentially large number of parameters involved; and,

7. Addressing the foregoing issues so as to arrive at a practicable method for design estimation of scour depth. Though much is known about scour processes, heretofore no comprehensive design methodology exists linking all the main factors that influence scour at a pier site.

The highly unsteady, three-dimensional flows around piers marked by macro-turbulence structures are hard to visualize, let alone measure, in their entirety. It presently is practically impossible, by means of existing laboratory instrumentation and facilities, to visualize the entire instantaneous flow field around a bridge pier.

For a given geometry of the pier, three length scales are involved (Figure 1-1) – structure width (assuming structure length and height are proportional to width), flow depth, and bed sediment diameter. Whereas it usually is straightforward (for an undistorted model) to have model structure dimensions proportionate with flow depth, there is a lower limit in linking particle diameter relative to structure width and flow depth. The physical behaviour of particle beds changes when particle diameter decreases. As particle diameter drops below about 0.7mm, a bed may form ripples. As diameter drops below about 0.1mm, inter-particle cohesion becomes increasingly pronounced. These changes affect scour at a pier. Ripple bedforms scale with particle diameter, whereas dunes scale with flow depth (e.g., ASCE, 1975). For cohesive soil and weak rock as foundation material, particle size no longer is a meaningful index for erodibility. For laboratory modeling purposes, however, the erodibility or shear strength of such material scales with the hydraulic model's length scale (ASCE 2000).

These difficulties can be especially challenging when investigating scour at piers of more complex form.

1.6. Report Organization

The report is organized in accordance with the evaluation's objectives:

1. Description of the essential processes associated with pier scour for comparatively simple cylindrical piers in situations uncomplicated by additional considerations such as debris accumulation;

2. Development of a framework of essential parameters needed when discussing the essential pier scour processes, and assessing whether leading design methods adequately reflect scour processes at simple or typical pier forms;

3. Extension of the parameter framework to include processes further complicating scour at piers (e.g. accumulation of woody debris);

4. Assessment of design methods for estimating scour-depth at simple, single-column piers or piers of common form. The assessment examines the extents to which the leading existing methods reflect the important parameter influences;

5. Assessment of design methods for scour-depth estimation when further processes complicate scour (e.g., debris accumulation);

6. Recommendation of a structured set of methods for estimating pier scour at bridges. The set of methods must be suitable for likely adaptation by AASHTO;

7. Use contemporary instrumentation methods for monitoring scour at piers (smart monitoring); and,

8. Identification of the necessary research and education needs to improve scour depth estimation. A hierarchy of design methods obliges designers to adequately understand scour processes, and the limits to which scour depth can be accurately estimated.

The principal outcome of the tasks is a structured (or graduated) design methodology that takes into account, to the extent practicable, individual parameter influences on scour depth, yet also recognizes the need for a "systems" or holistic approach to the set of connected flow and erosion processes active during pier scour.

The nomenclature used in HEC-18 (Richardson and Davis 2001) is used herein, as a substantial audience for the report will be engineers already familiar with this nomenclature.

1.7. Relationship to Other NCHRP Projects

This project is one of three projects conducted under NCHRP Project 24-27 *Evaluation of Bridge Scour*. The projects similarly aim at assessing existing knowledge and estimation methods regarding scour at bridge waterways:

- NCHRP 24-27(01) *Evaluation of Bridge Pier-Scour Research*
- NCHRP 24-27(02) *Evaluation of Bridge Abutment-Scour Research*
- NCHRP 24-27(03) *Evaluation of Bridge-Scour Research: Geomorphic Processes and Prediction*

There is a close relationship between the present and NCHRP Project 24-32 *Scour at Wide Piers and Long Skewed Piers* (Sheppard et al. 2011). The two principal investigators for the latter project, which had started about one year before the present project, were involved on the Research Team and Expert Team for the present project. The extensive evaluation of pier scour processes and predictive methods conducted for NCHRP 24-32 were of immediate use for the present project, which accordingly utilizes the results from NCHRP 24-32.

The proposals given by the present project, however, differ in several aspects from those given by NCHRP 24-32. In particular, the proposed design methodology presented herein places a lower importance, for the purpose of design scour estimation, on the exact mapping of scour depth to many parameter influences. Instead, the methodology focuses on the leading parameters and taking into account the uncertainties associated with them. Consequently, the present methodology acknowledges the influences of time-rate of scour and stage of live-bed scour, but considers them of secondary importance for most pier design situations. The typically high levels of uncertainty associated with the time development of scour, and stage of live-bed scour (as well as periods of live-bed scour), potentially introduce considerable uncertainty in design estimation of scour.

Additionally, the design methodology presented here is broader in options than the single method NCHRP 24-32 proposes.

The present project also builds on the prior work sponsored by NCHRP and other groups. In this respect, it will link to reports prepared for NCHRP Projects 24-08 *Scour at Bridge Foundations: Research Needs*, and NCHRP Project 20-07(Task 178) - *Evaluation and Update of NCHRP Project 24-08, Scour at Bridges Foundations: Research Needs* (Parola et al. 1996). In particular, the project reflects back to NCHRP Project 24-08 (Parola et al. 1996) to assess progress in pier-scour knowledge and estimation methods since then. It also uses the findings available from the following NCHRP studies recently completed or currently underway:

1. NCHRP Project 24-14, *Scour at Contracted Bridge Sites*;
2. NCHRP Project 24-15, *Pier and Abutment in Cohesive Soils*;
3. NCHRP Project 24-20, *Prediction of Scour at Bridge Abutments*;
4. NCHRP Project 24-26, *Effects of Debris on Bridge-Pier Scour*;
5. NCHRP Project 24-29, *Scour at Bridge Foundations on Rock*; and,
6. NCHRP Project 24-34, *Risk-Based Approach for Bridge Scour Prediction.*

Additionally, the project evaluates information syntheses available in significant reports completed for federal agencies: notably, HEC-18, *Evaluating Scour at Bridges* (Richardson and Davis 2001), and *Channel Scour at Bridges in the United States*, (Landers and Mueller, 1996). Leading books or compendia on scour also were examined: notably Hoffmans and Verheij 1997, Richardson and Lagasse 1999, and Melville and Coleman 2000.

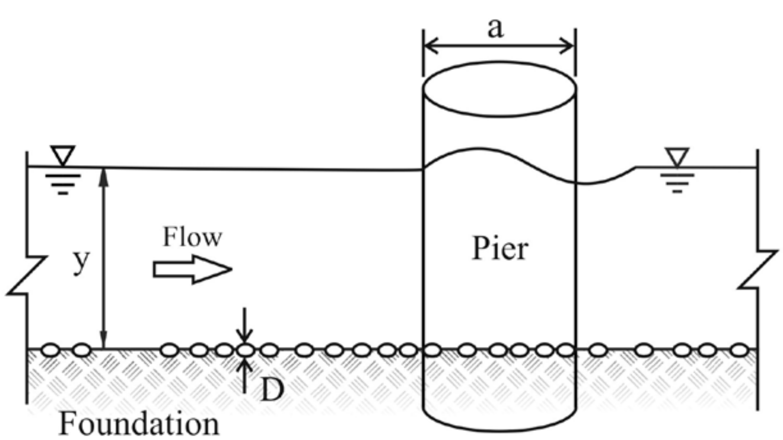
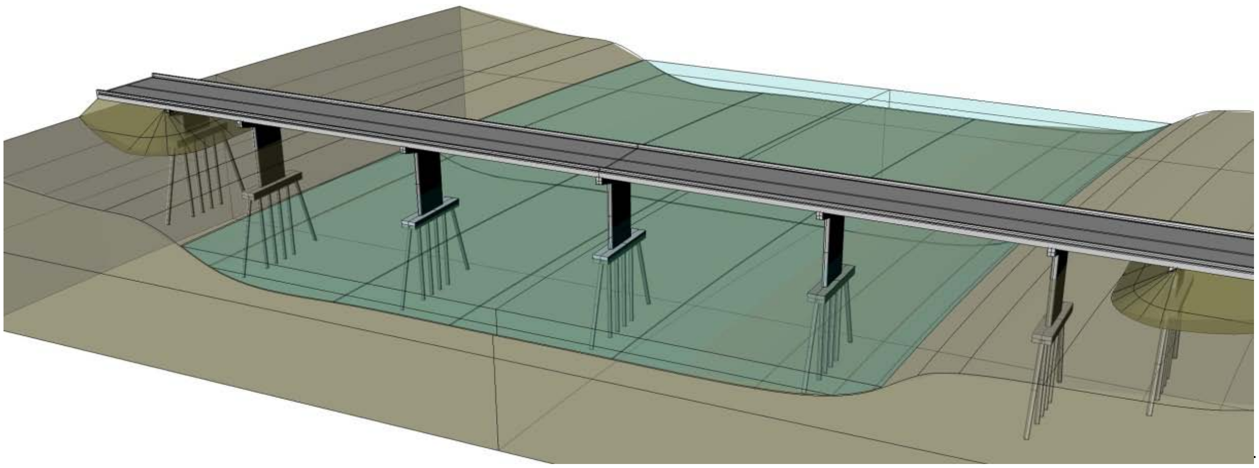
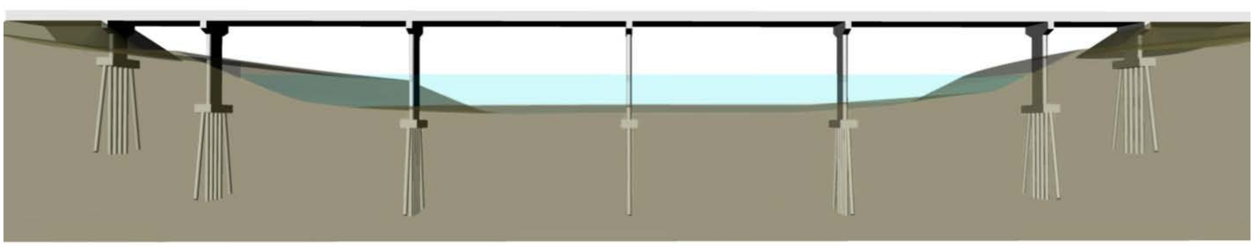


Figure 1-1 Three length scales (structure, flow depth, and sediment size (or shear strength when considering laboratory hydraulic models)) prescribe the flow field at a pier. The inherent difficulty of equally scaling the three lengths makes hydraulic modeling intrinsically approximate



(a)



(b)

Figure 1-2 Sketches showing a "long" multi-span bridge with multiple piers and abutments; (a) oblique perspective, and (b) cross-sectional view. In some cases pier-pier and/or pier-abutment interactions may be significant

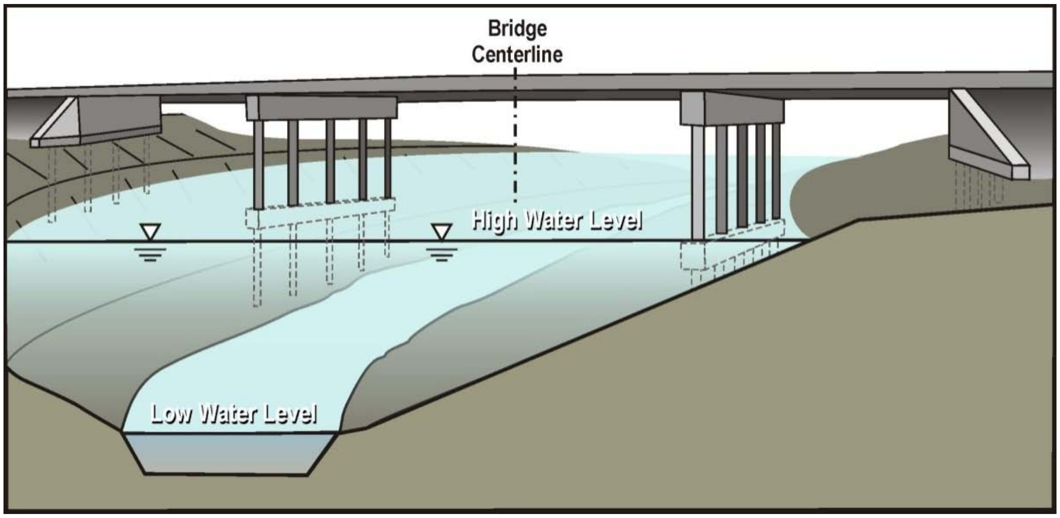


Figure 1-3 A sketch showing the foundations of a "short," three-span bridge. Depending on the water level and scour around the foundation, the flow fields in the vicinity of the pier may be significantly different

CHAPTER 2
SCOUR AS A DESIGN CONCERN

2.1 Introduction

When considering pier scour, it is necessary to consider pier structure and its influence on the scouring flow field, the foundation material supporting the pier, and the processes whereby flow erodes foundation material (sediment, soil, clay, rock) around the pier.

2.2 Pier Function and Structure

For efficient structural performance, and minimal environmental impact on the channel spanned, bridges over rivers commonly have a comparatively short first span, with a pier placed near the toe of, or within, the spill-slope of a bridge abutment, as illustrated in Figures 1-2 and 1-3. The piers of a multi-span bridge typically are not positioned in the same local flow conditions or boundary material. Also, variations in pier location at a bridge commonly require variations in pier structure; i.e., differences in pile cap or footing elevation, and in pile length.

Bridge piers support superstructure spans, doing so by transferring design loads to the channel boundary. Design loads include the deck weight and live-load, and the hydrodynamic load exerted by water flowing around the pier. Load transfer occurs typically via a simple slab footing, a set of end-bearing piles, or a set of friction-bearing piles. A consideration not well recognized is that pier structure affects the manner of pier scour failure, which in turn affects the form and dimensions of the scour hole. In other word, piers fail as scour develops (rather than a scour hole forming to equilibrium depth, then the pier collapses into it). This point is elaborated in Section 2.2.

The simplest pier form is the single-column pier, especially cylinder of circular cross-section. However, simple circular piers are not usual. Circular cylinder elements do commonly exist as a pile extending to a pile cap, with the pile cap elevated above the water level, as shown in Figure 2-1. Most U.S. Departments of Transportation and other bridge-designing agencies use common designs involving piers of more complicated form. Figure 2-2 for example shows a common pier structure, comprising a pier column, pile cap, and cluster of piles (friction- or end-bearing). The layout and dimensions of common designs conform to the design loads anticipated for piers. A deficiency in the leading methods for scour-depth estimation is their basis largely on laboratory data, the great majority of which have involved simple cylindrical pier forms (circular or rectangular cylinders), usually not coinciding with common pier designs. Most field data are for piers of diverse forms. A research need to be mentioned early in this report is for scour estimation methods to link more expressly to common pier structures, such as shown in Figure 2-2.

The effective form of pier structures may change during flow events, owing to the accumulation of woody debris or ice against the pier (Figure 2-3), or other factors such as flow direction and exposure of piles beneath a pile cap. Such changes influence the pier flow field, pier loading, and scour.

More complicated and unique pier forms often are required for less common bridge sites, such as for large bridges, and channel circumstances where piers must withstand additional loads. Bridges in large tidal flows, flows subject to dynamic ice conditions, and bridges possibly subject to vessel impact, for example, require more complicated piers. Figure 2-4 illustrates such an example.

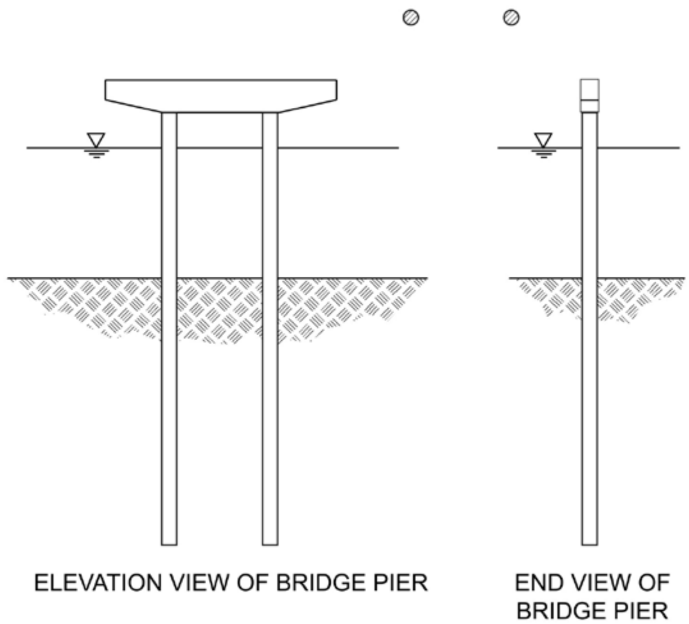


Figure 2-1 A simple pier form comprising two cylindrical cylinders

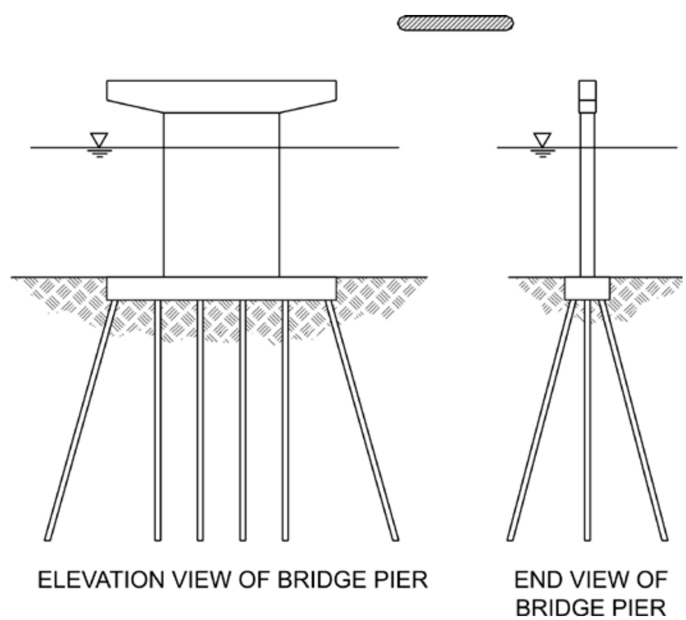


Figure 2-2 A common pier structure used for two-lane bridges. The pier comprises a column supported by a pile group with a pile cap

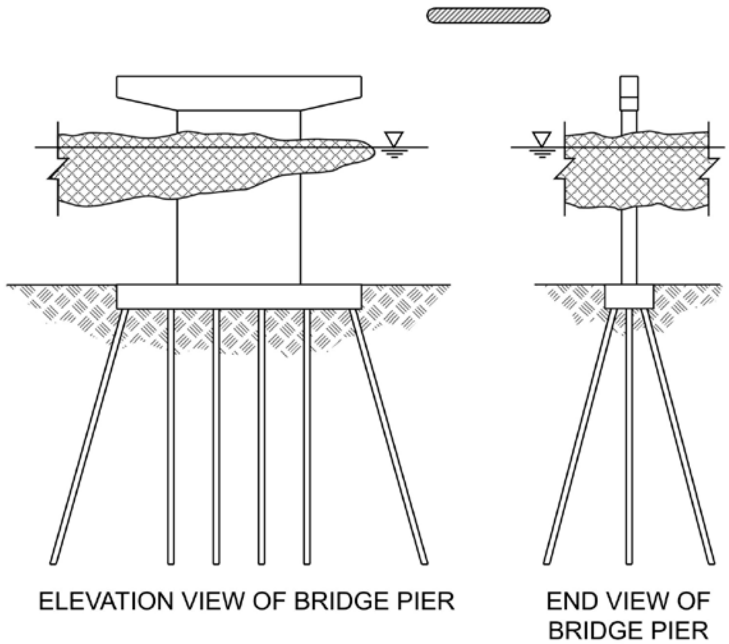


Figure 2-3 A common pier form in a flow situation complicated by debris or ice accumulation

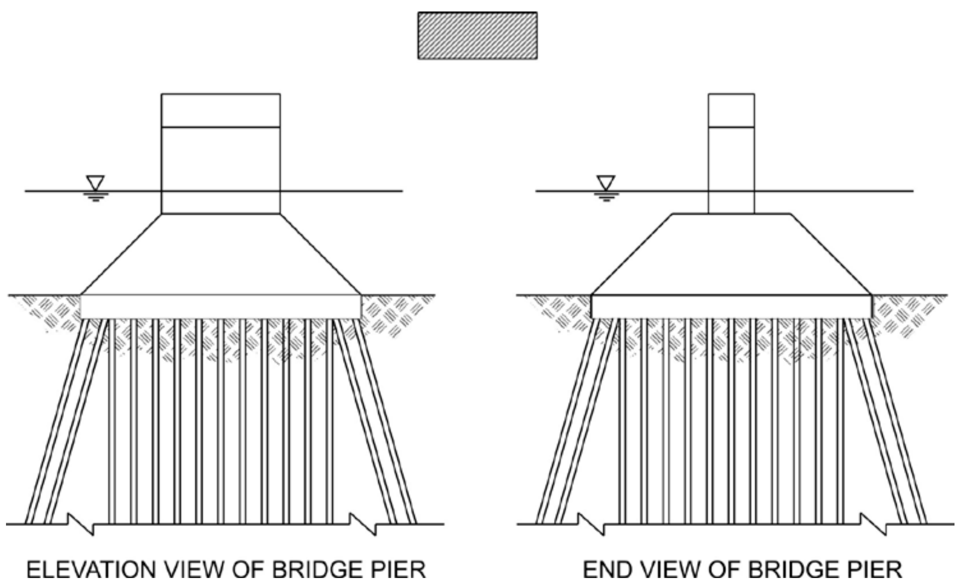


Figure 2-4 Some bridges, such unusually large bridges, or bridges in unusual circumstances, require large piers of uncommon design

2.3 Design Depth for Pier Foundation

The design scour depth must be taken into account when sizing and positioning the foundation base. For piers on spread footings, the top of the footing must be below the estimated design scour depth, so that the footing is not undermined. For piers on friction-bearing piles, pile length must include design scour depth so as to ensure sufficient friction length of pile (Figure 2-5). Scour reduces the friction length of piles and increases the risk of pile buckling. To be kept in mind is that reduction of support will cause a pier to settle or tilt in various ways, as indicated in Figure 2-6. Pier settling and tilting may alter effective pier form, increase flow-field capacity to scour, and deepen scour. Field examples of pier settlement and tipping are shown in Figures 2-7 and 2-8, respectively.

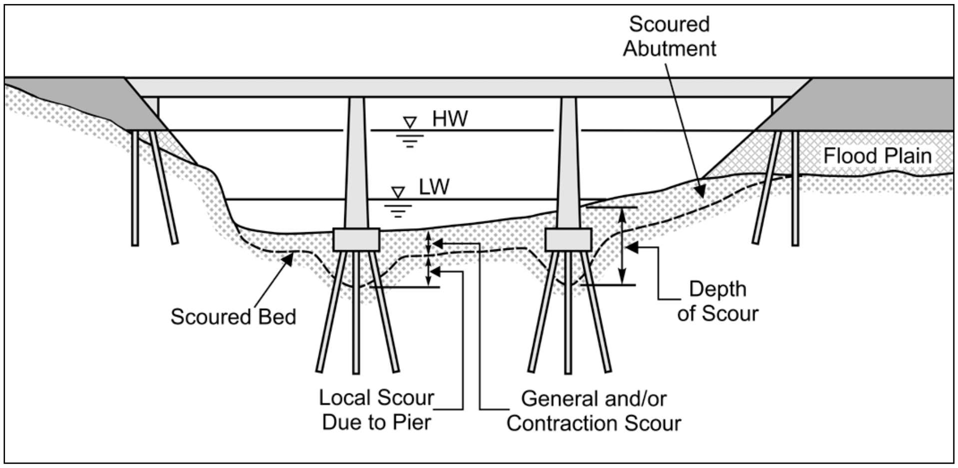
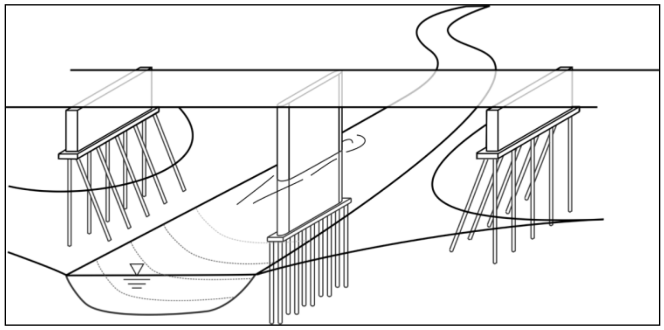
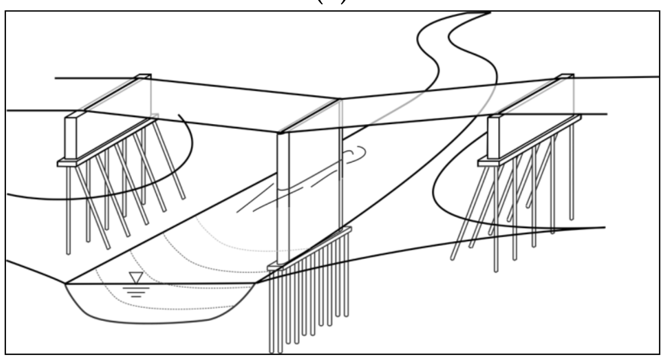


Figure 2-5 Scour reduces the effective length of friction-bearing piles



(a)



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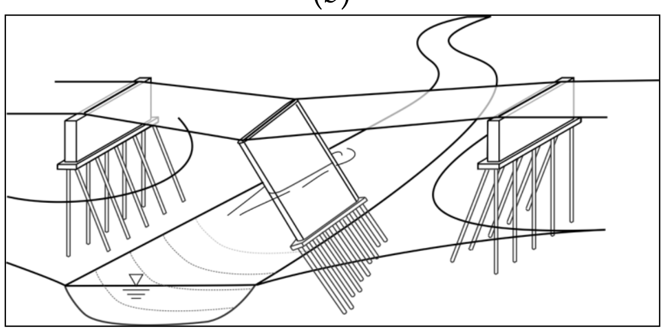

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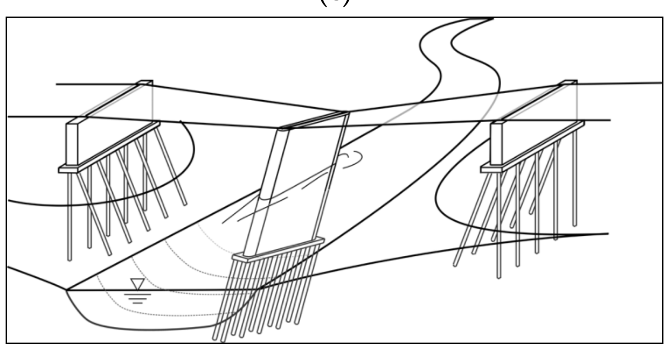


(d)

Figure 2-6 Scour reduces pier support, causing pier settlement (a) → (b), bottom rotation of pier (a) → (c), or top rotation of pier (a) → (d)

Figure 2-7 Bridge pier settled vertically owing to scour reduction of pier support

(a)

(b)

Figure 2-8 Pier tipped owing to scour: (a) forward tipping; and, (b), backward tipping. These photos raise interesting questions: How does scour develop when a pier rotates as it loses support or gets pushed back by flow pressure? Does pier tipping deepen scour? Also, (a) illustrates pier propensity to collect woody debris

2.4 Current U.S. Design Methods for Pier Scour

For U.S. bridge design, the leading method for design estimation of pier scour is the method herein termed the Richardson and Davis (2001) method. It often is colloquially called the CSU method, because it stems from extensive research conducted at Colorado State University. This method currently is in FHWA's design guide HEC-18, and is recommended by AASHTO.

Other methods may be used regionally, though in conjunction with the HEC-18 method; e.g., the method developed by Wilson (1995) who uses extensive field data from bridges in Mississippi. A method proposed by Sheppard and Miller (2006) is being used increasingly, notably in Florida (FDOT 2010).

The method developed by Melville 1997, (also in Melville and Coleman 2000) is used in conjunction with the Richardson et al. method, though is not formally recommended by HEC-18 or AASHTO. It is used quite extensively in other countries besides the U.S. Melville's method merged with that by Sheppard and Miller (2006) has evolved into the Sheppard-Melville method, consequent to NCHRP Project 24-32 (Sheppard et al. 2011).

As mentioned in Section 1.2, a perceived significant deficiency of the existing methods is their predisposition to yield unacceptably conservative estimates of scour depth at wide piers. Additionally, concerns exist that the methods do not apply well to commonly used pier forms or piers of unusual form. An aspect inadequately reflected by the existing methods is the substantial differences pier flow field associated with variations in pier size, flow depth, and foundation material.

2.5 Need for a Structured Design Approach

The potentially extensive set of parameters influencing pier depth, and the variable complexity of pier flow field, require a structured approach to design estimation of pier scour depth. The approach should not rest on a single, universal pier-scour relationship for estimating scour depth, but identify several levels of pier site complexity, and the current best methods for scour-depth estimation for each level. Such an approach is inadequately articulated in current design methods, such as HEC-18.

Figure 2-9 outlines the main elements of the approach this report recommends. It relates pier site complexity (also pier size) essentially to the varying practicalities of two methods – formulation in a semi-empirical or rational equation, and simulation in model of the pier site. The design methodology should comprise the following levels of pier form and site complexity:

1. Simple or single-column pier forms;
2. Common pier forms comprising a more complex geometry;
3. Common pier forms in difficult situations; and,
4. Uncommon or Complex pier forms and situations.

The methodology reflects the pier situations in the general views shown as Figures 1-2 and 1-3, which respectively illustrate the pier supports for a long multi-span bridge, and a short, three-span bridge. The central pier in Figure 1-2 could be considered a fairly simple pier in isolation, but the local flow fields at other piers are to varying extents affected by flow around the abutments and over the channel's floodplain. The two piers in the shorter bridge (Figure 1-3) cannot be considered in isolation, and may be affected by flow around the abutment and over the floodplain. Further, the methodology entails using the methods having best present prospect for addressing pier-site complexities.

As the present report explains, the approach is not entirely new. Chapter 5 shows several methods that attempt to account for the influences of major parameters. Additionally, as explained in Chapter 5, it is reasonably common for difficult or complex pier circumstances to receive additional design attention; for instance, large piers of complex geometry, and piers in complicated channel bathymetries.

The complexities listed in Section 1.4 make a single design relationship or method infeasible for estimating scour depth at all pier situations in bridge waterways. For situations involving a pier of relatively simple geometry in an uncomplicated channel, the methodology indicates that scour depth can be estimated using essentially a rational (parameter influence) equation comprising a linear combination of factors expressing parameter influences. Estimation accuracy diminishes as parameter number increases, in accordance with increasing pier or site complexity. For piers of increasing geometric complexity, pier geometries complicated by factors altering pier form (e.g., debris or ice), and site difficulties (e.g., in channel morphology), the design approach necessarily entails modeling the pier situation and the scour it generates.

The methodology outlined in Figure 2-9 prompts several questions:

1. Which rational equation(s) should be used?
2. What are the limits for using a rational equation, and, consequently, when is it necessary to model a pier site?
3. How reliable are models (hydraulic and numerical) for simulating scour?
4. Can soil and rock properties be incorporated into equations or methods (including hydraulic and numerical models) for estimating scour depth?

If substantial uncertainty attends the use of a method (equation and/or model), or site conditions, the methodology emphasizes the importance of site monitoring and management. Uncertainties frequently occur with approach-flow conditions, erosion characteristics of bed or floodplain material, and prospective changes in channel morphology.

The ensuing three chapters review the scour processes and parameters to be considered for scour-depth estimation, indicate limits of present scour knowledge and formulation, and identify the leading rational equations for scour-depth estimation. Chapter 6 then elaborates the design methodology.

2.6 Synopsis of Post-1990s Research

The numerous complexities associated with design estimation pier-scour depth cause scour to remain an active topic of civil engineering research. Though publication of research findings on scour extends back at least one hundred years (notably, Engels in 1894, at Dresden University, Germany (Freeman 1929)), papers dealing with investigation of scour processes and scour depth prediction are still regularly published in journals (e.g., ASCE Journal of Hydraulic Engineering [JHE], IAHR Journal of

Hydraulic Research [JHR]) and conference proceedings (e.g., Scour and Erosion, 2006, 2008).

The literature on bridge scour is replete with papers presenting new flume data and observations on various aspects of scour, especially scour at vertical cylinders placed in sand beds. A survey of recent issues of Journal of Hydraulic Engineering and Journal of Hydraulic Research, the two leading hydraulic engineering journals, shows that, during 2008 and 2009, JHE and JHR published 17 and 19 journal papers on bridge scour. The papers, reflecting much of the research conducted since 1990, address the following aspects of scour:

1. Continued strong interest in basic scour processes;
2. Increased interest in pier flow field;
3. Influence of boundary complexities (non-uniform sediments, clay, rock); and,
4. Design for complex pier forms (large piers, overtopped bridges, debris and ice effects).

Several comprehensive summaries of scour research and design relationships to estimate local scour at bridge foundations have been published since 1990. A selection of notable publications include Breusers and Raudkivi (1991), Lagasse et al. (1995), Richardson and Davis (1995), Parola et al. (1996), Hoffmans and Verheij 1997, Dey (1997), Raudkivi (1998), Whitehouse (1998), Hamill (1998), Richardson and Lagasse (1999), Melville and Coleman (2000), Sumer and Fredsoe (1997, 2002), FHWA (1996, 2001), Briaud et al. (2004a), Sheppard and Miller (2006), Lagasse et al (2009), and Sheppard et al. (NCHRP 24-32). Also, website information provides useful information about pier scour (e.g., the following website provided by the Federal Highway Administration, http://www.fhwa.dot.gov/engineering/hydraulics/bridgehyd/index.cfm).

A useful development since 1990 is the availability of internet information regarding pier scour. An increasing amount of information can be accessed via the internet, thereby helping to improve understanding of scour processes and how to design for them.

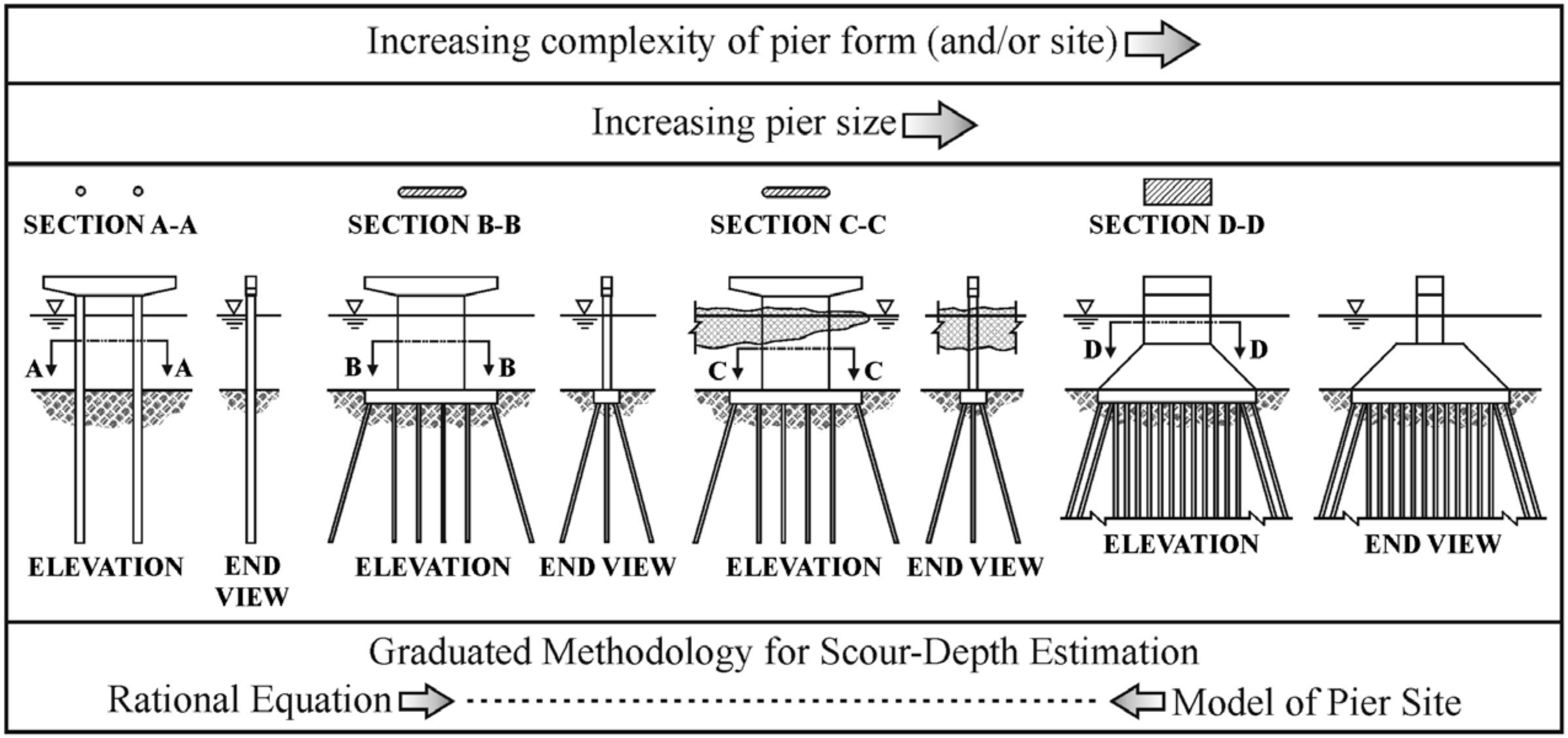


Figure 2-9 Overview of structured design approach

CHAPTER 3
PIER-SCOUR PROCESSES

3.1 Introduction

Pier scour processes are intricate and challenging to formulate (even empirically or approximately), let alone fully comprehend. This statement holds for scour at all types of piers, especially those whose geometry consists of several components (column, pile cap, piles). The processes are made more complicated by the highly three-dimensional and unsteady characteristics of the flow field at piers, and by the nature of boundary material erosion. Furthermore, the complexities can be readily amplified by variations in bridge-waterway geometry, boundary material in which a pier is founded, and other considerations like woody-debris or ice accumulation at a pier. Figure 3-1 illustrates some of the complexities for a representative bridge waterway, e.g., different boundary materials, proximity of abutment.

The ensuing sections begin by examining pier scour of boundary material at a cylindrical pier founded in a rectangular channel bed or floodplain; most methods for estimating pier scour were developed for this situation. Considered then are factors adding complexity to scour at actual bridge sites. Such factors include pier-form complexity, pier location relative to bridge waterway morphology, and debris accumulation. These factors primarily affect pier flow field.

The scour processes are discussed in terms of variables associated with the component sets of variables influencing pier scour.

1. Pier foundation material;
2. Pier flow field; and,
3. Erosion of foundation material at a pier.

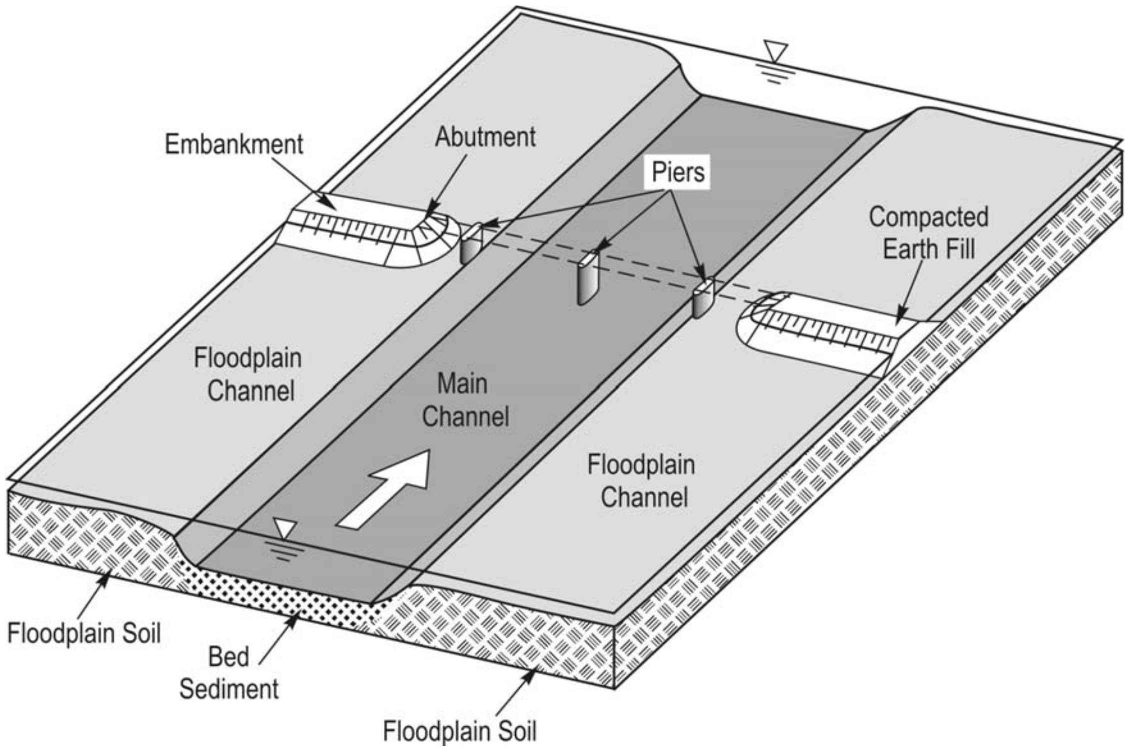


Figure 3-1 Sketch showing flow through a bridge site involving complex interactions between the floodplain, the main channel and the piers situated close to the floodplain and main channel, especially during high flow conditions

3.2 Pier Foundation Material

Rivers channels and floodplains form in widely varying combinations of rock, sediment, and soils (or clays). The great majority of scour-prone bridge waterways, however, comprise main channel beds formed of alluvial sediments (non-cohesive material), at least through the upper strata of the beds. Therefore the majority of pier-scour studies have focused on scour at piers in alluvial beds. The floodplains of such channels typically are a mix of sediments and soils (cohesive sediment, possibly with organic content, and clay). It is not uncommon for piers to be founded in clay or on rock beds, sometimes underlying upper strata of alluvial sediments. Comparatively few studies focus on scour at piers in clay or rock, though notable pier failures have involved piers on clay or rock.

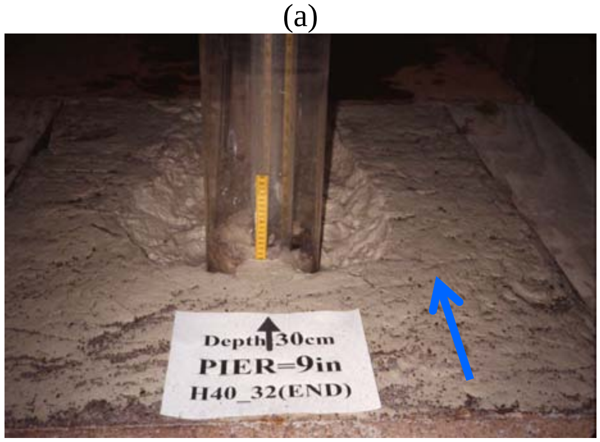
Scour-hole formation at piers in sediment has been observed frequently, especially in laboratory flumes. Accordingly, much of the review in subsequent sections of this report summarizes scour in a single layer of sediment. In this regard, the main variables characterizing single layers of sediment are relatively easily identified and expressed in terms of non-dimensional parameters (Chapter 5).

Much less frequently observed, and fraught with potentially numerous additional variables, is scour in layered sediments, and in clay and rock. Chapter 5 outlines the essential aspects of these parameters. They include variables characterizing layer dimensions (for layered sediments and soils), strength for soils and rocks, and structure of joints and fractures (rocks).

Somewhat different scour forms evolve in accordance with whether scour occurs in sediment, clay, or rock, as depicted in Figure 3-2. In terms of scour form, it also is useful to include a view of scour by air flow at a cylinder in snow (Figure 3-3), a light cohesive boundary material. An important observation for the scour forms in Figures 3-2 and 3-3 is that the overall scale of scour depth does not seem to vary markedly with the different foundation materials, though scour geometry does vary to some extent.

The well-known inverted-frustum form of scour hole develops at piers in single layers of sediment, the upstream side-slope of the scour hole being related closely to the static angle of repose of the sediment (Figure 3-2a). For strong cohesive material (clay), the scour hole is less regular, with deepest scour occurring at the pier flanks. For weaker cohesive material, like snow, the scour hole is more cylindrical, as cohesion enables the material to have a vertical face. Scour in rock is influenced by jointing and fracturing in the rock, and consequently can produce a notably irregular scour form. Scour in sand and gravel produce a deposition mound or bar behind the pier. Scour in other materials does not.

(a)



(b)

(c)

Figure 3-2 Differences in scour form at a cylinder; (a) sand bed, (b) clay bed (Briaud et al. 2004), and (c) rock bed (Hopkins and Beckham, 1999). The maximum depth of scour is approximately similar for each material, but the location of deepest scour differs

Figure 3-3 Scour in a weak cohesive material (snow)

3.3 Pier Flow Field

To understand pier scour, it is necessary understand the flow field at a pier, and how the flow field varies with pier size and form, as well as flow depth and foundation material. A difficulty in this respect, however, is that the flow field is a class of junction flow (i.e., flow at the junction of a structural form and a base plane), a notably three-dimensional, unsteady flow field marked by interacting turbulence structures. The eroding forces exerted on the foundation material supporting the pier are generated by flow contraction around the pier, by a pronounced down-flow at the pier's leading edge, and by turbulence structures of a wide range of turbulence scales. Variations of pier width and form, and flow depth, alter the flow field, enhancing or weakening these flow features.

In terms of prevailing ranges of pier width, *a*, and flow depth, *y*, it is convenient to identify and discuss three categories of pier flow field, which produce significantly different pier scour morphologies:

1. Narrow piers ($y/a > 1.4$), for which scour typically is deepest at the pier face;
2. Transitional piers ($0.2 < y/a < 1.4$); and,
3. Wide piers ($y/a < 0.2$), for which scour typically is deepest at the pier flank. Under design flow conditions, i

The values of y/a indicated for the flow-field categories are based on data trends delineating differences in the relationship between scour depth and y/a (e.g., Melville and Coleman 2000). Figure 4-1, subsequently in Chapter 4, defines pier width, *a*, in terms of as-constructed pier form. The foregoing categories are defined better in terms of effective pier width, a_*, which takes into account approach flow angle and pier form.

The ensuing sub-sections briefly describe the main flow features and illustrate how they differ for these three categories of flow field. For substantially more detailed descriptions of the flow field, refer for example to Kirkil et al. (2006, 2008). The closing sub-section takes a moment to show examples of the detailed insights now obtainable using numerical models of pier flow fields, insights presently not accessible from laboratory or field investigation.

The pier flow field may become more complicated if the pier has a complex shape, such as a column supported on a pile cap underpinned by a pile cluster, as in Figures 1-2 or 1-3. Additionally, the close presence of an abutment and/or a channel bank further complicates the flow field.

3.3.1 Narrow Piers

The main features of the flow field at narrow piers can be explained by viewing the flow field commensurate with scour at an isolated circular cylindrical pier in a relatively deep, wide channel. Figure 3-4 illustrates the main features of the flow field for a pier founded in sediment, and conveys a sense of the flow field intricacies to be considered when attempting to understand scour at a simple, single-column pier. For a sediment foundation, scour is deepest at the pier face.

An interacting and unsteady set of flow features entrains and transports sediment from the pier foundation. They include: flow impact against the pier face, producing a down-flow and an up-flow with roller; flow converging, contracting, then diverging; the generation, transport and dissipation of large-scale turbulence structures (macro-turbulence) at the base of the pier-foundation junction (commonly termed the horseshoe vortex); detaching shear layer at each pier flank; and, wake vortices convected through the pier's wake. The features evolve as scour develops.

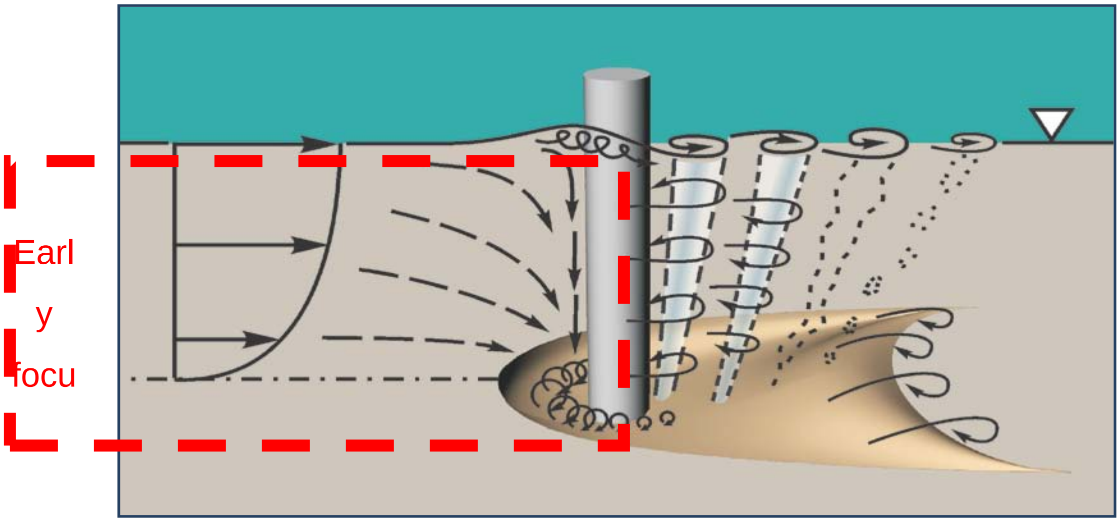


Figure 3-4 The main flow features forming the flow field at a narrow pier of circular cylindrical form. Early research focused on flow immediately upstream of the pier (dashed area)

Flow approaching the pier decelerates, impinges against the pier's centreline, and then strongly deflects both down and up the pier's face. These two vertical flows act almost as wall-attached jet-like flows along the pier's centreline, one directed up toward the free surface, and the other down toward the bed. The up-flow attains a height approximating a stagnation head, interacts with the free surface, and forms a surface roller or vortex. The stagnation pressure on the upstream face of the pier attains a maximum near the level where these two jet-like flows form. Also, at the stagnation line the deceleration is greatest. The deceleration decreases as the bed and, respectively, the free surface are approached. The down-flow is driven by the resulting downward gradient (below the still water level) of stagnation pressure along the pier's leading face. This downward gradient results largely because the velocity distribution of the approach flow is commensurate with a fully turbulent shear flow; i.e., velocity generally decreases toward the bed. As the scour hole develops, the down-flow is augmented by the approach flow diverging into the scour hole.

In addition to the vertical component of flow at the pier's leading face, flow contracts as it passes around the pier's sides. Local values of flow velocity and bed shear stress thereby increase around the pier's sides. For many piers, the increases are such that scour begins at the sides of a pier. Once the scour region develops as a hole fully around the pier, the down-flow and the necklace (or horseshoe) vortices strengthen. Scour-hole formation draws flow into the hole.

The influences of turbulence structures have become better realized during the past decade, though they are not yet adequately understood and taken into account by scour-depth relationships. Research prior to about 1990 focused essentially on flow approaching a pier, and the horseshoe vortex system at the scour-hole base, with little attention given to the turbulence structures around the entire pier. The turbulence structures, together with local flow convergence/contractions, around the broad fronts and flanks of piers, or between piles of complex pier configurations, are erosive flow mechanisms of primary importance. The turbulence structures are not isolated from each other. They intrinsically connect within the flow field. Also, it is not enough to focus on one category of turbulence structure, notably the well-known (but still inadequately understood) horseshoe vortex system; or on one flow convergence; e.g., the down-flow at the leading edge of a pier. As significant as these individual flow features are, they alone do not account for sediment erosion from a scour hole.

The flow field, during all stages of scour development, is marked by the presence of organized, coherent turbulence structures, notably:

1. A horseshoe vortex system formed of several necklace vortices (the standard term for junction flows) commonly termed the horseshoe vortex. It forms around the pier's leading perimeter. These vortices wrap around the pier's base such that the legs are oriented approximately parallel to the approaching flow. The legs break up and are shed intermittently;
2. Small but very energetic elongated eddies (vortex tubes whose main axis is approximately vertical relative to the bed) in the detached shear layers;
3. Large-scale rollers or wake vortices, which form behind the two flanks of the pier, and are shed into its wake. As they advect away from the pier, the wake vortices expand in diameter, then dissipate and break up;
4. A horizontal vortex formed by flow passing over the stationary, depositional mound formed at the exit slope from the scour hole. The location and amplitude of the mound depend on the power of the wake vortices shed from the pier (the weaker the vortices, the closer the mound to the pier); and,
5. A surface roller situated close to the junction between the free surface and the upstream face of the pier. The roller is akin to a bow wave of a boat moving through water.

In summary, the down-flow impingement on the bed, along with the wide range of turbulence structures present in the flow field, entrain and transport material from the scour hole. The details and interaction of the flow field vary with pier shape, angle of attack, and the stage of scour development between initiation and equilibrium, but the essential consideration is that these flow features are responsible for scour. Therefore, to understand how scour develops, to model scour, and to estimate scour depth it is necessary to understand the general structure of the flow field, and determine how flow entrains and transports foundation material from the scour hole. Also, it is important to recognize that the flow field evolves during different stages of scour.

The flow field becomes even more complicated if the pier has a complex shape, such as a column supported on a pile cap underpinned by a pile cluster, as in Figure 1-2. Additionally, the flow field can be complicated by debris or ice accumulation, the proximity of an abutment, and aspects of channel morphology.

3.3.2 Transition Piers

The main flow-field features described for narrow piers exist in the flow field of piers within the transition range of *y/a*, but the features now begin to alter in response to reductions of *y* and or increases in *a*. The closer proximity of the water surface to the foundation boundary (for constant pier width), or the increased width of a pier (for constant flow depth), partially disrupt the formation of the features, and thereby reduce their capacity to erode foundation material. Though further research is needed to systematically describe and document the flow field changes, ample data show that reductions in *y/a* result in shallower scour depths for this transition category of flow field (see Section 4.3).

Figure 3-5 depicts a sequence of flow field adjustments commensurate with three values of *y/a*, indicating how the scour capacity of flow field reduces. The down-flow at the pier face becomes less well developed because it has a shortened length over which to develop, whereas the up-flow associated with the (flow stagnation) bow wave remains essentially unchanged. The vorticity (circulation) of the large-scale turbulence structures (horseshoe vortex) aligned more-or-less horizontally in the pier flow field weakens as the down-flow weakens, and the vertically aligned turbulence structures (wake vortices) also weaken due to the increased importance of bed friction in a shallower flow.

3.3.2 Wide Piers

For wide piers, the flow approaching the pier decelerates, turns, and flows laterally along the pier face before contracting and passing around the sides of the pier. The down-flow at the pier face is weakly developed, and only slightly erodes the foundation at the pier centerplane. The circulation of the necklace vortices peaks at vertical sections situated around the flanks of the pier. Flow velocities near the pier are greatest where flow contracts around the pier's sides. Erosive turbulence structures now principally comprise wake vortices and the part of the horseshoe vortex system located in the scour region close to each flank of the pier. Deepest scour occurs at the pier flanks. Figure 3-6 schematically illustrates the flow field around a wide pier.

For a given flow depth, greater pier width increases flow blockage and therefore causes more of the approach flow to be swept laterally along the pier face than around the pier's flanks. Increased blockage modifies the lateral distribution of approach flow over a longer distance upstream of a pier.

The flow field around each side of a wide pier is essentially the same as that at an abutment built with a solid foundation extending with depth into the foundation material (also that at a long spur dike or coffer dam).

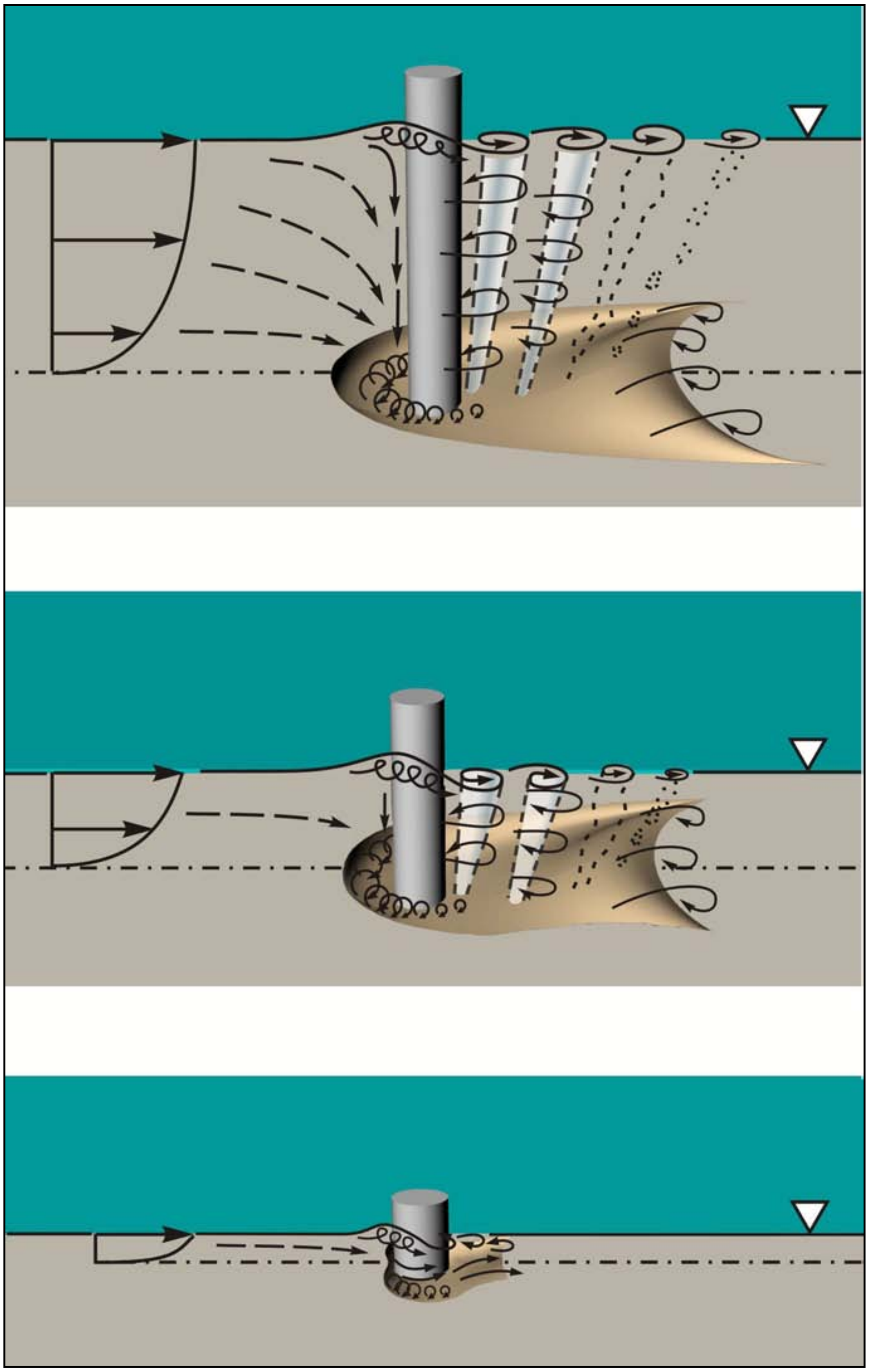


Figure 3-5 Variation of flow field with reducing approach flow depth; narrow to transitional pier of constant pier width. The sketches contain the horseshoe vortex, the bow vortex, and the lee-wake vortices. The downflow is represented by the vertical arrow close to the upstream face of the pier

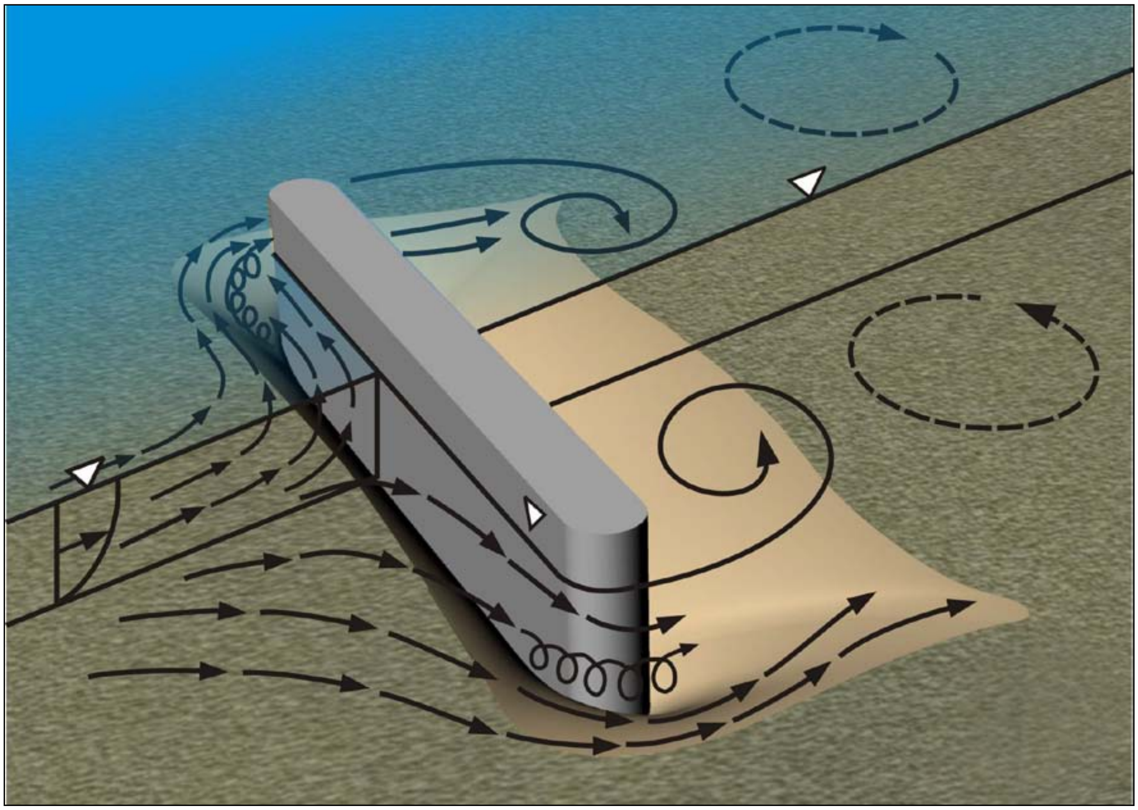


Figure 3-6 Main features of the flow field at a wide pier (y/a < 0.2)

3.2.4 New Insights from Numerical Modeling

Since 1990, and especially in the recent ten years, major progress has been made with numerical modeling of flow at piers. The models available today can resolve all the main flow features and their unsteady interactions. It is useful to include here short examples of the highly detailed information available from such models.

Initial Flow Field. Before scour when the bed is flat, adverse pressure gradients slowing the approach flow cause flow separation near the bed. The incoming boundary layer on the bottom surface around the pier separates. The resulting necklace vortices (forming the horseshoe vortex system) develop within the separated region around the upstream part of the pier's base, and are a consequence of the reorganization of the boundary layer vorticity downstream of the flow separation line. The necklace vortices have the same sense of rotation as the vorticity in the upstream boundary layer. For most flow conditions, the location, size and intensity (circulation) of the necklace vortices vary in time. Phenomena such as vortex pairing between a secondary necklace vortex and the primary necklace vortex, or between two secondary necklace vortices and vortex bursting phenomena occur. As the dominant upstream boundary layer vorticity is in the transverse direction, due to the adverse pressure gradients close to the upstream face of the pier, the necklace vortices originating in the separation region stretch around the pier. The sides of the vortex lines become oriented in the streamwise direction, with the vorticity being of opposite sense in the two sides (legs). The main necklace vortices entrain fluid from the down-flow at the upstream face of the pier, drawing it out toward the bed. Another interesting phenomenon is that the capacity of the wake roller vortices to entrain sediment is strongly dependent on the shape of the pier. Compared to circular cylinders and cylinders with smooth edges, cylinders with sharp edges (e.g., piers of rectangular shape) tend to form strong roller vortices that maintain their coherence until the bed surface. As a result, strong bed friction velocity values occur beneath the rollers that are convected in the near wake region (e.g., see Figure. 3-10). This flow field aspect explains the higher rates of scour observed behind piers of rectangular section compared to circular piers during the initial stages of the scour process.

The strength, evolution, and fluctuating formation of the vortices upstream of the pier are affected by flow Reynolds number (yV/ν), the characteristics of the bed roughness, the shape and size of the evolving scour hole, the level of free-stream turbulence, and the shape of the pier.

Developing Scour Hole. Once a scour hole forms around the pier, the initial system of vortices changes substantially, their size and behaviour dictated by the shape of the scour hole. In other words, the horseshoe vortex system now forming in the scour hole is similar to a recirculation flow region forming due to the drop in the bed elevation at the upstream side of the scour hole. The necklace vortices forming the horseshoe vortex system are visualized in Figure 3-7 for a circular pier with a large scour hole close to equilibrium scour depth.

An important flow feature is the shedding of energetic vortices inside the detaching separation layer forming on each pier flank. In most cases, these vortices resemble vortex tubes. Away from the pier, the axis of these vortex tubes is close to vertical. For circular piers that do not contain sharp edges (as for a rectangular pier) the vortices are tilted sideways in the near bed region, because the angle at which flow separates from the pier varies with distance from the bed. For the usual range of pier diameters and approach-flow velocities, the separation angle is about 170^o near the bed and decreases to about 85^0 away from the bed (the 0^o location is at the pier's leading edge) if the wake is subcritical. Such insights are available from Large Eddy Simulation numerical models (Kirkil et al. 2006, 2008), and to some extent from Particle-Image Velocimetry experiments (e.g., Unger and Hager 2007). These energetic vortices can entrain sediment, conveying it through the wake vortex system.

The portion of the flow field in the near wake is dominated by the shedding of large-scale roller or wake vortices that can induce large, unsteady forces on the bed as they are convected away from the pier. The forces are sufficient to transport sediment away from the scour hole as it develops. The regularity of shedding of the roller vortices can be significantly affected by the upwelling of flow close to the symmetry plane behind the pier. The suppression of the vortex shedding has been observed both experimentally and numerically for circular and rectangular piers in shallow flows with a large scour hole (Kirkil et al. 2008).

Figure 3-8 is a useful illustration of the highly three-dimensional and contorted path of flow and suspended sediment particles through a scour hole. The paths involve extensive rotations and vertical movements. The form of the pier and the stage of scour development affect the entrainment and transport of foundation material from the scour hole.

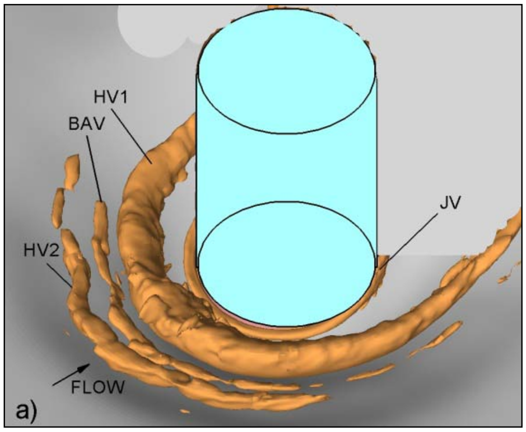


Figure 3-7 Visualization of the main vortices forming the horseshoe vortex system,(HV) system in the mean flow field around a circular pier in a scoured bed. HV1 is the main necklace vortex; HV2 and BAV are secondary necklace vortices; JV is a junction corner vortex (Kirkil et al., 2008)

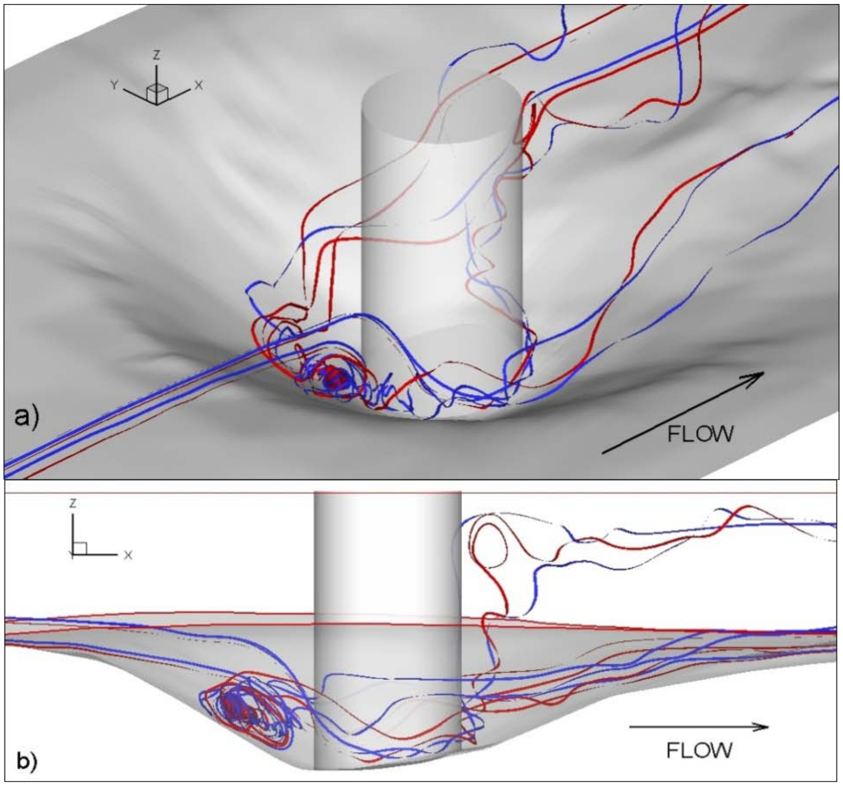


Figure 3-8 Numerical simulation showing example flow paths (and fine-sediment paths) around a pier during scour; (a) top view, (b) side view (Kirkil et al., 2008)

3.4 Erosion of Foundation Material

The main flow features work together in entraining and transporting foundation material from around a pier, and thereby determining scour morphology and the potential maximum scour depth. The following description outlines how the flow field erodes sediment to form a scour hole at a narrow pier in sediment, the most commonly studied pier scour situation. The illustrations shown are from numerical modeling, including from flow around a simulated transitional pier.

Flume studies over the years (e.g., Melville and Raudkivi 1977, Dargahi 1990, Oliveto and Hager 2002) show that scour initiates in the contracted flow region along the pier flanks, and beneath the detaching shear layer and wake vortices shed behind the pier. The initial scour zones grow and extend back around the pier face.

The down-flow at the pier face impinging on the erodible bed quickly erodes a groove immediately adjacent to the front of the pier. As the groove deepens, it triggers the formation of a frustum-shaped scour hole around the pier's upstream perimeter. Further deepening of the groove undermines the scour-hole slope, causing local avalanches of sediment, which then the flow sweeps from the scour hole, thus maintaining the slope at the local repose angle of the sediment as scour deepens. The down-flow together with the horseshoe vortex system comprise the two main flow features responsible for the removal of sediment particles and the growth of the upstream side of the scour hole formed in sediment.

Because sediment erodes from all around the pier base, other flow features facilitate scour. At the pier's sides, flow contracts and accelerates, which results into a local increase of the bed shear stress. The convection of highly energetic vortex tubes within the detached shear layers usually results in a strong amplification of the bed shear stress at the pier flanks. Additionally, the main necklace vortices in the horseshoe vortex system remove sediment not only from the upstream part of the scour hole, but also from the pier flanks as patches of highly vortical fluid detach from the legs of these necklace vortices. This is one of the main mechanisms that explains the growth of the scour hole both laterally and behind the pier. Figure 3-9, from a numerical simulation (Kirkil et al., 2009), shows the momentary formation of streaks of high bed shear stress behind the pier. These streaks occur when one side of a necklace vortex is stretched toward the back of the pier and a streak (or eddy) of vorticity detaches from the necklace vortex. As this streak moves away from the pier, its vorticity is high enough to significantly amplify the bed shear stress beneath it.

The wake vortices play an important role transporting sediment away from the pier. Depending on the flow conditions and the shape of the pier, they also can induce locally high bed-shear-stress values as they are convected away from the pier. This role becomes especially pronounced for transitional and wide piers. For example, the contours of instantaneous bed friction velocity shown in Figure 3-10 illustrate this occurrence for flow past a large-aspect ratio rectangular pier on at flat bed at the start of the scour. The strength of the wake vortices and the local amplification of the bed friction velocity beneath them are much smaller for circular piers of same width.

A deposition-dune or mound forms downstream of the pier due to the deceleration of the sediment particles entrained in the regions of high bed shear stress and pressure fluctuations. The deceleration occurs once the particles entrained by the various eddies move downstream of the pier and these eddies weaken and eventually dissipate. Some of the sediment particles are entrained into the recirculation region behind the pier. These particles can move away from the bed as a result of their entrainment by upwelling motions.

As the scoured region at the upstream base of the pier grows, the overall size of the horseshoe vortex system increases but the down-flow velocity near the scour-hole base reduces as do the bed shear-stress values. In the case of clear-water scour, when these stresses decay close to the local value (adjusted for gravitational force effects) corresponding to the threshold for sediment entrainment, scour deepening ceases and the flow and bathymetry are at an equilibrium scour condition. In reality, even at equilibrium conditions some local intermittent erosion and deposition of bed particles occurs (e.g., Roulund et al., 2005), but the overall scour form does not alter.

The foregoing description indicates the importance of understanding the interactions of the turbulence structures in elucidating pier scour. This consideration is even more critical for piers of complex geometry. Then, the complexity of the flow and of the dynamic interactions among the main coherent structures is even greater; e.g., the effects of the underflow beneath a pile cap, the vortex shedding taking place in the wake of the exposed foundation elements supporting the main pile. The literature lacks a detailed study of these processes.

The foregoing description is for erosion of sediment. The description must be modified for erosion of clay or rock. Turbulence structures play an even larger role in eroding these latter two materials.

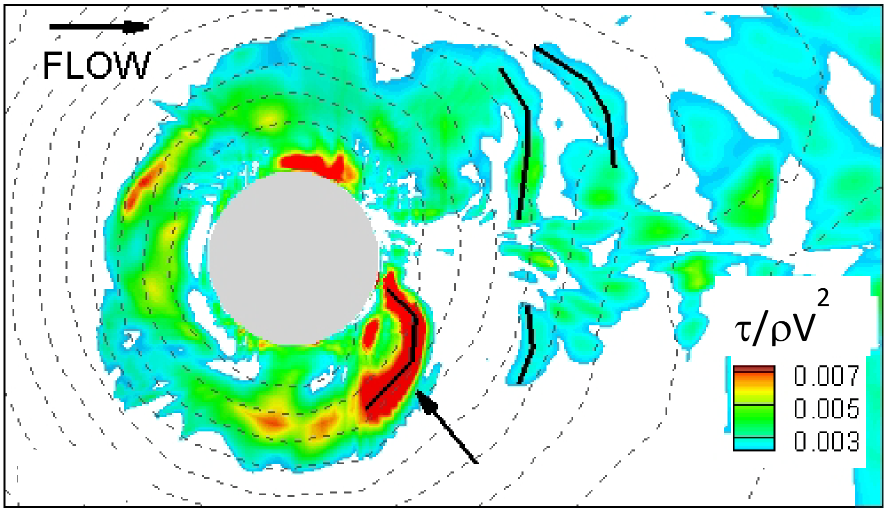


Figure 3-9 Distribution of instantaneous bed shear stress around a narrow circular pier with scour hole. Note the high value beneath the leg of the main necklace vortex on the right side of the pier. This streak of vorticity is detaching from the horseshoe vortex and is convected behind the pier parallel to the deformed bed (Kirkil et al., 2008)

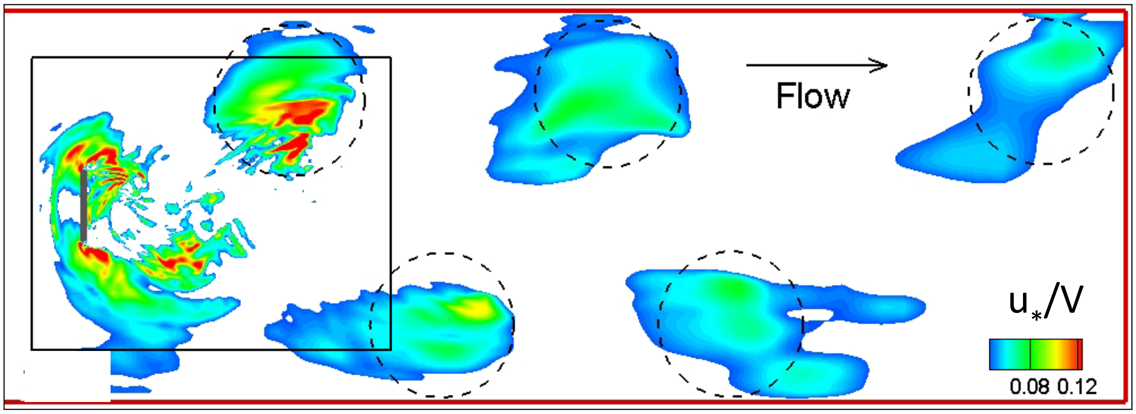


Figure 3-10 Numerical simulation showing distribution of instantaneous bed friction velocity in the flow past a high aspect ratio rectangular cylinder at the start of the scour process (flat bed)

CHAPTER 4
PARAMETER FRAMEWORK

4.1 Introduction

Quantitative explanation of how pier flow field, erodibility of foundation material, and erosion processes influence pier scour depth requires a framework of parameters linking variables influencing pier scour. The framework's central parameters can be determined by considering the variables associated with scour at a cylindrical pier in a single stratum of non-cohesive sediment. Additional parameters quickly arise when considering practical aspects of pier design and actual site conditions at bridge waterways. This chapter focuses on scour at cylindrical piers in non-cohesive sediment. Chapter 5 subsequently introduces pier site complications to be considered.

The set of variables involved with the comparatively simple situation of a uniform cylindrical pier in a single stratum of non-cohesive sediment entails a surprising number of complexities, some of which have only become recognized during the past two decades.

A few considerations must be kept in mind:

1. The pier flow field and scour depth vary substantially with pier form and dimensions, and approach flow variables;

2. Identifying the necessary full set of parameters at play, even for a cylindrical pier, is not as straightforward as prior publications on pier scour suggest; and,

3. As the number of variables considered increases, the parameter framework soon becomes intricate. Cross connections exist between parameters including common variables (e.g., nominal pier width), which appear in several parameters. The variation of one parameter along an axis of influence often incurs the variation of another parameter. Consequently, rational-type equations relating scour depth to parameter influences become more cumbersome and arguably less accurate (if indeed appropriate).

Though most variables of importance had been identified before 1990, developments in instrumentation and computer-simulation techniques since 1990 (especially during the recent decade) have revealed additional parameters, or parameter influences, of importance to pier scour:

1. New fundamental parameters relating to the pier flow field (especially the turbulence structures in it) or boundary erosion characteristics (especially clay and rock); and,

2. Parameters describing the additional processes complicating scour (notably, soil cohesiveness, debris or ice accumulation, abutment proximity, and bridge-deck submergence).

The parameter framework must be consistent and comprise relevant parameters. As several variables exert multiple influences often interlinked, parameter sets can be formed of less relevant parameters. For example, bed material "size" affects flow field geometry and dynamics, relative roughness of bed, time evolution of scour, and the hydraulic deformation of the bed. The multiple influences of some variables cause them to be combined as alternative parameters, usually traditional hydraulic engineering parameters (Froude number or Reynolds number) some of which do not accurately express scour physics. Additionally, when the framework of parameters becomes overly intricate, because too many parameter influences cannot be treated independently, the framework loses its utility for formulating a general relationship for scour depth. Then, scour at a pier must be viewed in terms of a system of interconnecting influences, best treated by means of simulation (hydraulic modeling and numerical modeling).

Simple relationships between scour depth and dominant parameters or variables (e.g., effective pier width) lead to practical, envelop curves useful for design estimation of the potential maximum scour depth, and for providing a tangible sense of scour depth magnitude.

Few publications identify and discuss the full, interconnected framework of parameter influences. Melville and Coleman (2000) give a particularly comprehensive coverage of parameter influences known up to about the late 1990s. More recently, Sheppard and Millar (2006) provide a useful broad review.

4.2. Variables at a Cylindrical Pier in a Single Foundation Stratum

The processes contributing to pier scour at a cylindrical pier in a single stratum of non-cohesive foundation material involve the basic variables shown in Figure 4-1. A contextual framework of non-dimensional parameters relating the magnitudes of length, time, and force associated with the processes, is discussed next. This framework brings together the findings reported in the extensive literature on pier scour, and reveals gaps where influences are inadequately explained or quantified.

The functional relation between the depth of local scour, y_s, and the pertinent variables can be stated as

$$y_s = function \begin{bmatrix} flow\ (\rho, \mu, V, y, g), bed\ material\ \left(D, \sigma_g, \rho_s, V_c,\right), pier\ (a, b, \Omega, \theta), \\ time\ (t), \end{bmatrix} \tag{4.1}$$

In Eq. (4.1),

ρ and μ = fluid density and molecular viscosity, respectively; V = depth-averaged velocity of approach flow; y = approach flow depth; and, g = gravity acceleration

V_c = critical shear velocity for bed sediment entrainment; D and σ_g = median size and geometric standard deviation of the foundation material particle size distribution; ρ_s = sediment density; and c = a parameter describing cohesiveness of the material. The ensuing discussion focuses on coheshionless foundation material, leaving consideration of material cohesiveness to Chapter 5, which addresses complications at pier sites.

a = pier width; b = pier length; Ω = parameter describing the shape of the pier face (upstream side); θ = angle of the flow relative to pier alignment

t = time

The following set of non-dimensional parameters can be developed using a, V, and ρ as the normalizing (or independent) variables:

$$\frac{y_s}{a} = function\ (\Omega, \theta, \frac{a}{b}, \frac{y}{a}, \frac{D}{a}, \frac{V}{V_c}, \frac{V^2}{ga}, \frac{\rho V_* a}{\mu}, \sigma_g, \frac{\rho_s}{\rho}, \frac{tV}{a}) \tag{4.2}$$

At times it is useful to use shear velocity instead of depth-averaged velocity, V. Shear velocity V_* relates to the V, as $V = V_* \sqrt{8/f}$, where $f = function\,(D/y)$ is the Darcy-Weisbach resistance coefficient for fully turbulent flow. This relationship also expresses the critical shear velocity for bed sediment entrainment V_{*c} in terms of the critical mean approach flow velocity for entrainment of bed sediment, V_c. Thus, V, *and* V_c could be replaced with V_* and V_{*c} in Eq. (4.2). However, use of V instead of V_* affords greater clarity in describing the relationship between pier flow field and scour depth.

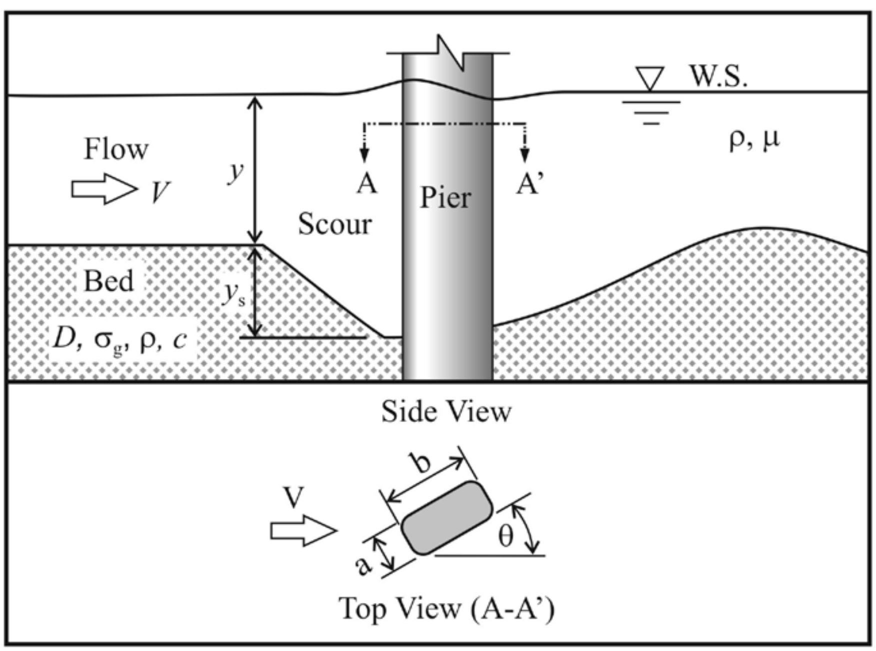


Figure 4-1 Variables influencing pier scour at a cylindrical pier

Some of the parameters in Eq. (4.2) can be combined to aid explanation of processes influencing scour; e.g., combination of y/a and D/a gives the length ratio y/D, of use when talking about relative roughness of the approach flow, and bedform presence in the approach flow. Because the variables can form alternative parameters, it is possible to assemble less meaningful parameters than those in Eq. (4.2). For example, flow Froude number ($V/(gy)^{0.5}$) could arise, but it is not of immediate use other than distinguishing whether the approach flow is sub-critical or super-critical.

4.3. Primary and Secondary Parameters

The parameter framework of the parameters in Eq. (4-2) comprises sets of primary and secondary parameters. The primary parameters define the structure and geometric scale of the pier flow field, and therefore determine the potential maximum scour depth. The secondary parameters characterize scour-depth sensitivities within the geometric scale limit, and normally lead to scour depths less than the potential maximum scour depth. The values of the secondary parameters are subject to considerable uncertainty at pier sites.

The primary parameters relate directly to the pier flow field:

y/a indicates the geometric scale of the pier flow field (in a vertical cross-sectional plain transverse to the pier, and streamwise to the pier)

a/D relates the length scales of pier width and median diameter of bed particle

Ω, a/b, and θ, define pier face shape, aspect ratio of pier cross-section, and approach flow alignment to pier, respectively. These parameters may be merged with pier width, a, to form the compound variable a^* = effective pier width. It can be useful to express the two length-scale parameters as y/a^* and a^*/D

The secondary parameters have magnitudes prescribed by the primary parameters, with regard to potential maximum scour depth:

V/V_c expresses the extent or stage of sediment transport on the approach flow bed, often termed the flow intensity. When sediment diameter, D, is set, V_c is prescribed. This parameter distinguishes whether clear-water or live-bed scour (bedload movement in the approach flow) conditions prevail in the approach flow to the pier

V^2/ga is an Euler number relating vorticity induced inertial forces in the pier flow field relative to gravity acceleration

$\rho Va/\mu$ is the pier Reynolds number. Though viscous effects are unlikely to have an effect on the scour depth at the pier because fully turbulent flow occurs around bridge piers, the inclusion of a Reynolds number accounts for the dependence of wake-vortex shedding on pier Reynolds number

σ_s is geometric standard deviation of bed particles, and characterizes sediment uniformity

tV/a characterizes (in conjunction with other parameters) the temporal development of scour associated with pier flow field and nature of foundation material

Before proceeding to discuss parameter influences, it is useful to make several comments:

1. Some equations for estimating scour depth (Appendix A) identify a group of primary parameters defining a potential maximum geometric scale of scour (*y/a*, *a/D*, Ω, *a/b*, and θ). Many others do not;

2. The coupled role of parameters V^2/ga and $\rho Va/\mu$ has been identified quite recently. They express similitude in the flow power associated with large-scale turbulence structures generated in the pier flow field;

3. Only the methods developed by Melville 1997 (see also Melville and Coleman 2000) and Sheppard and Miller (2006) include the primary parameters;

4. Some variables appear in several parameters, and complicate the parameter framework. In particular, pier width, *a*, appears in several of the parameters influencing the pier flow field:

    i. Relative "shallowness" *a/y* of the flow field;
    ii. Relative "coarseness" *a/D* of the scour hole base, and the possibility of bedforms developing in a sufficiently large scour hole;
    iii. Vorticity (or intensity of circulation) and frequency of coherent turbulence structures in the pier flow field ($\frac{\rho Va}{\mu}$ and $\frac{V^2}{ga}$);
    iv. Variation of effective pier shape (with Ω and θ), and thereby all the above influences; and,
    v. The time rate of scour development, *tV/a*.

5. As noted later in Section 4.5, certain regions of the parameter framework lack data to confirm parameter influences, notably regions where parameter values are difficult to attain, such as live-bed scour at piers in the wide pier category.

6. An approach using semi-empirical equations based on selected key parameters is useful for approximate estimation of scour depth. The accuracy of such equations increases up to a point, as more parameters are considered. However, the utility and accuracy of such an approach then may diminish when attempting to account for parameter influences involving variables exerting several influences. Also, the large number of parameters to be considered may render such equations unwieldy. This situation quickly arises for piers formed of multiple components (e.g., column on a pile-cap with piles), and in pier sites complicated by additional

considerations. Moreover, acquiring the data to establish the quantitative relationships of parameter influences quickly entails very extensive programs of laboratory experiments and field observations.

4.4. Parameter Influences

This section evaluates the known influences that individual parameters in Eq. (4.2) exert on scour, and points out knowledge gaps. Despite numerous studies aimed at describing the parameter influences of parameters, there remain significant gaps in the overall understanding of scour. The gaps are attributable to difficulties in conducting laboratory flume experiments at large geometric scales, with isolating some of the parameter influences, and the large number of parameter influences to be examined. Also, there is a paucity of reliable field data in certain parameter ranges.

The evaluation extensively uses the substantial documentation provided by Melville and Coleman (2000), presently the most comprehensive publication explaining parameter influences on pier scour. Because certain parameter influences have come to light since 2000, the present evaluation extends beyond Melville and Coleman (2000). However, the evaluation is not intended to be highly detailed, but rather indicate parameter influences.

An important consideration insufficiently recognized heretofore is that the parameter influences are not fully independent from each other. Further, the more parameters influencing scour at a pier, the more cross-connected become the parameter influences. In the following descriptions of parameter influences, when the cross-section of the pier is not specified, the cross-section is taken to be circular.

4.4.1 Flow-field Scale, *y/a*

The parameter y/a defines the geometric scale of the pier flow field and, therefore, potential maximum scour depth. It is central to discussion of pier flow field and its variations.

Melville and Coleman (2000) categorize pier scour flow field and processes in terms of three classes of y/a, and suggest the three categories in Table 4-1. The inequalities defining each were derived from plots of laboratory data as evident in Figure 4-2. They used the same laboratory data to define different functional relationships, $y_s = f(y/a)$, describing the influence of flow shallowness on local scour depth (Table A-1 and Appendix A). For flow depths large compared to pier width (i.e., for narrow piers), scour depth increases proportionately with pier width, and is independent of y. Figure 3-4 illustrates the flow field commensurate with this category. Conversely, for the wide pier category, scour depth increases proportionately with y and is independent of a; Figure 3-6 illustrates the flow field. For intermediate depth flows, scour depth depends on both y and a (Figure 3-5).

The solid line in Figure 4-2 envelops the data and applies, from left to right respectively, to wide-, intermediate-, and narrow-width piers at threshold conditions. The dashed lines in Figure 4-2 indicate scour depths for different values of V/V_c, and show that, for clear-water scour at reduced flow velocities, lesser scour depths are developed. The lines have been plotted assuming a linear relation between clear-water scour depth and flow velocity. If other scour depth influences (especially sediment gradation, abutment shape and channel geometry) were present, actual scour depths would be further reduced from the maxima defined by the lines in Figure 4-2.

Table 4-1 Classification of local scour processes at bridge piers in terms of y/a (Melville and Coleman 2000); the limits are approximate values beyond which different trends occur

Pier Class	*y/a*	Pier Scour Dependence
Narrow	$y/a > 1.4$	$y_s \propto a$
Intermediate width	$0.2 \le y/a \le 1.4$	$y_s \propto (ay)^{0.5}$
Wide	$y/a < 0.2$	$y_s \propto y$

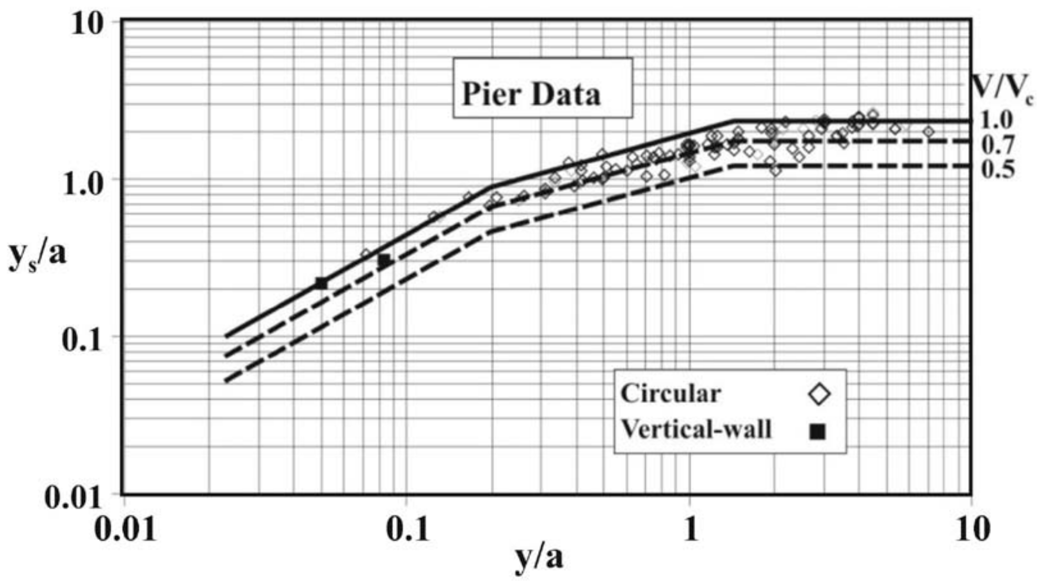


Figure 4-2 Influence of y/a on local scour depth expressed as y_s/a (Melville and Coleman 2000)

Figure 4-2 shows the trends only for clear-water scour. A similar figure has yet to be developed for live-bed scour, especially for piers in the wide-pier category. This deficiency is a gap in data, insight, and ultimately in scour-depth formulation for a common condition of pier scour.

For a pier in a broad, sand-bed channel of given intensity of sediment mobility V/V_c, particle size, and flow depth, increasing pier width modifies the parameters discussed immediately above. In most situations, pier widening may not lead to increased scour depth relative to effective width of pier. As a pier widens in a given flow, the flow field becomes shallower (relative to pier width), and the scour alters in depth and eventually in geometry. Figure 3-6, in Chapter 3, illustrates a typical scour formation at a wide pier. The following factors come into play:

1. The proportion of the approach flow to be diverted into the evolving scour hole diminishes;

2. Some features of the pier flow field, notably the counter-clockwise surface roller, interfere with the down-flow into the scour hole and the clockwise horseshoe vortex;

3. The energy of the macro-turbulence structures (e.g., wake eddies) in the pier flow field decreases, because the diameter of the structures increases, but the rotational velocity remains approximately constant;

4. For wide, model-scale piers in sand beds, bedforms begin to develop at the base and exit slope of a scour hole. These bedforms increase the scour resistance of the scour boundary;

5. During live-bed conditions, bedform size diminishes relative to scour hole size, with as yet unclear net consequences for the time-averaged, and the extreme, values of scour depth; and,

6. More time is needed to develop the scour hole at wider piers. Under clear-water scour conditions, the asymptotic temporal approach to equilibrium scour depth can be a matter of hours (small cylinders in laboratory flumes) and of the order of weeks for large piers (even in laboratory flumes). Under live-bed scour, a median equilibrium scour condition is attained quite quickly, within a day or so for large piers, although the passage of bedforms may cause major fluctuations in scour depth. The duration of flow conditions is an increasingly important factor for wider piers, especially under clear-water scour conditions, as briefly discussed subsequently for the time-development of scour; and,

7. When y/a is less than about 1, the formation of the sediment deposition bar behind the pier affects scour hole development. The height of the bar height can extend over a large portion of the flow depth. The bar therefore alters the flow field at the pier's rear by causing flow to be diverted to the sides of the bar and, thereby, reducing the erosive power of flow over the bar. The net effect is a widening of the bar, while the exit slope from the scour hole remains relatively steep. Over time, turbulence generated by flow around the pier gradually erodes the bar, and the scour hole may deepen somewhat further.

A recognized weakness of the existing methods (Appendix A) for scour-depth estimation is their inadequate inclusion or articulation of the foregoing influences, even for the methods proposed since 1990; notably, Hoffmans and Verheij (1997), Johnson (1999), Melville and Coleman (2000), Richardson et al. (2001), Kohli and Hager (2001), Sheppard and Miller (2006, also Sheppard and Renna 2005). To varying extents, these methods yield scour depth estimates that exceed observed depths at wide piers. Piers tens of feet wide, though, are reported as creating scour holes considerably shallower (relative to pier width) than the maximum depths found from laboratory flume tests with small circular cylinders. The methods proposed by Melville and Coleman (2000), Richardson et al. (2001), Sheppard and Miller (2006) indicate an increasing influence of flow depth on local scour depth for shallower flows. Melville and Coleman (2000) go furthest to differentiate the influence of *y/a* in terms of categories of scour depth sensitivity. Some methods show that the effect of flow depth on scour depth vanishes asymptotically with increasing flow depth (e.g., Breusers and Raudkivi 1991, Melville and Coleman 2000).

4.4.2 Relative Coarseness, *a/D*

In accordance with the relative length scales it embodies, and considerations of similitude in hydraulic modeling of sediment transport and flow resistance, the parameter *a/D* expresses a relative coarseness of the flow boundary, but also links to other influences. In hydraulic modeling, exact replication of flow-depth, pier-width, and particle-diameter length scales is unusual. Consequently, scale effects in dynamic similitude are associated with *a/D*, because most hydraulic modeling (or laboratory flume experiments) maintain the length ratio *y/a* more-or-less the same as in field situations, but not so for the length ratio *a/D*.

Data from small-scale laboratory experiments show that, for uniform sediment, local scour depths are affected by sediment coarseness when the sediment is either relatively large or relatively small. Several studies explain that, for smaller values of the sediment coarseness ratio, individual particles are large relative to the groove excavated by the down-flow and erosion is impeded because the rough and porous bed dissipates some of the energy of the down-flow. When *a/D* is less than about 8, individual particles are so large relative to the pier that scour is mainly due to erosion at the sides of the pier and scour is further reduced. Figure 4-3 shows the trend for clear-water scour depth at piers subject to varying *a/D*, indicating too that ripple formation for medium and fine sands ($D \leq 0.7$mm) limits scour depths under clear-water conditions; for such sands, ripples form at sub-threshold conditions on the approach bed, and cause sediment transport into the scour hole. Figure 4-4 shows a comparable trend for live-bed scour depth; no separate cluster of data occurs for ripple-forming sands.

The trends in Figures 4-3 and 4-4, obtained for piers of circular cross-section, show that scour depth is influenced by sediment size for *a/D* less than about 50. For *a/D* exceeding 50, the influence of the sediment size on scour depth is negligible. These trends used by several scour prediction methods to account for the effect of sediment coarseness are shown in the sketch in Figure 4-5.

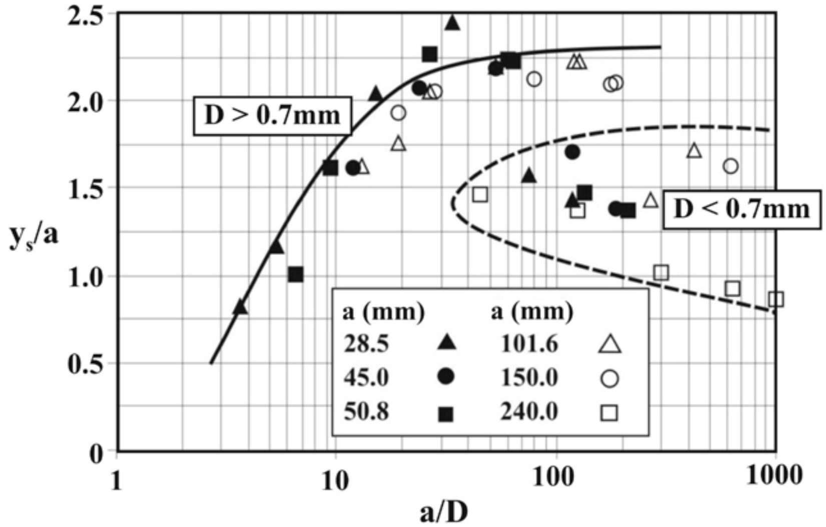


Figure 4-3 Influence of sediment coarseness on local scour depth at piers for clear-water scour conditions (Melville and Coleman 2000)

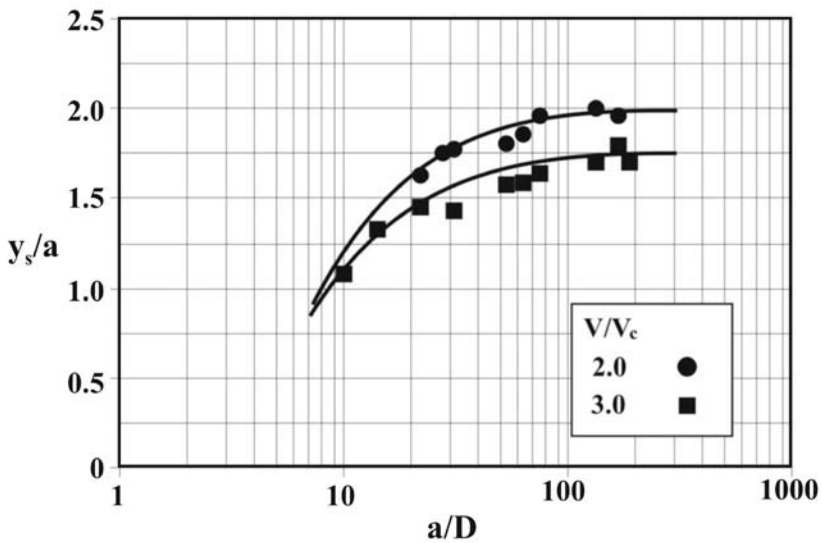


Figure 4-4 Influence of sediment coarseness on local scour depth at piers at different flow intensities for live-bed scour conditions (Melville and Coleman 2000)

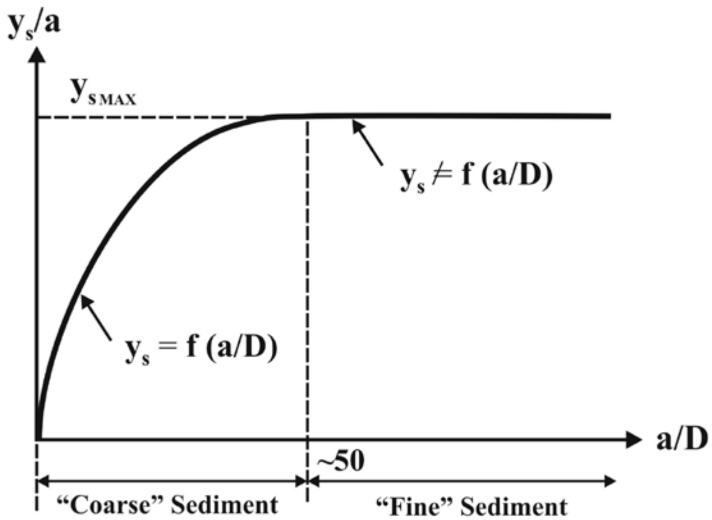


Figure 4-5 Local scour depth variation with sediment coarseness (Melville and Coleman 2000)

However, for much larger values of *a/D*, representative of prototype sized piers founded in sandy materials, recent data by Sheppard et al. (2004) and Lee and Sturm (2008) demonstrate significant scour depth reductions for increasing *a/D*. The reductions for *a/D* > 50 are shown schematically in Figure 4-6, which contains a larger data set and presents a more comprehensive trend than does Figure 4-4.

Sheppard et al. (2004) used three different diameter circular piers (0.114, 0.305 and 0.914m), three different uniform non-cohesive sediment diameters (0.22, 0.80 and 2.90mm) and a range of water depths and flow velocities. The tests extended the range of ratios of *a/D* to 4,155. Lee and Sturm (2008) used the Sheppard et al. (2004) laboratory data, together with field measurements from three field sites monitored by the United States Geological Survey (Sturm et al., 2004) and field measurements of Landers and Mueller (1996) and Mueller and Wagner (2005) to extend the range of *a/D* to about 10,000 (see Figure 4-6).

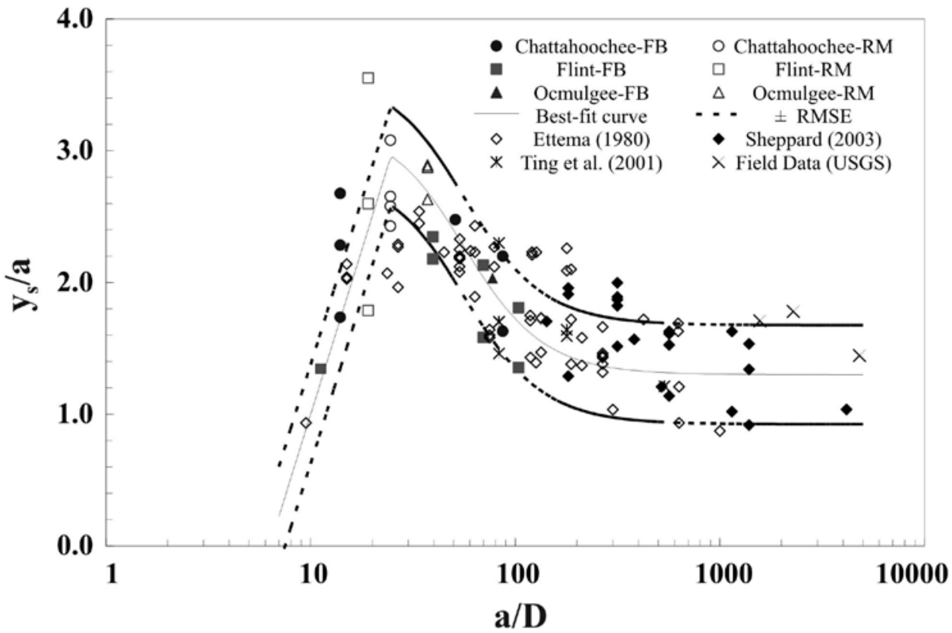


Figure 4-6 Influence of sediment size a/D_{50} on local scour depth y_s/a (Lee and Sturm 2008)

4.4.3 Pier Face Shape, Ω

As scour depth is consequent to the flow field a pier develops, the shape of a pier's face affects scour depth. Piers are constructed in a variety of basic face shapes, as Figure 4-7 illustrates. Numerous data show that blunter shapes induce slightly deeper scour. This effect occurs because blunter shapes increase flow contraction at the pier, and they increase flow-field vorticity. The more complicated overall shapes (pier column on piles or footing) are considered further in Chapter 5.

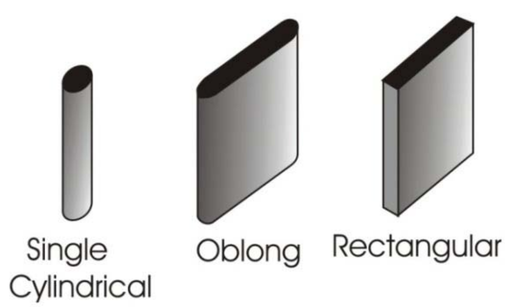


Figure 4-7 Basic pier shapes

Shape effects, for pier face, usually are given as a multiplying factor K_s that accounts for the difference in local scour between a particular pier shape and a simple circular cylindrical pier subject to the narrow pier category indicated in Table 4-1. Such factors have been proposed by several early studies (e.g., Tison 1940, Laursen and Toch 1956, Chabert and Engeldinger 1956), and are well accepted (Melville and Coleman 2000, Richardson et al. 2001, Sheppard 2006).

The recommended shape factors for cylindrical piers given in Table 4-2 are normally used (e.g., Melville and Coleman 2000, Richardson et al. 2001, Sheppard et al. 2006). The factors show that shape of the pier face is relatively insignificant for simple cylindrical piers. The form factors should only be used where the pier is aligned with the flow. A small change in pier alignment will eliminate any benefit from a streamlined shape. Also, the factors may become less accurate for transition piers, and may not apply to wide piers (as defined in Table 4-1).

Table 4-2 Shape factors for uniform piers (Richardson et al. 2001)

Pier Shape	Factor, K_s
Circular	1.0
Round Nosed	1.0
Square Nosed	1.1
Sharp Nosed	0.9

4.4.4 Pier Aspect Ratio, *a/b*

The factor K_s, for shape of pier face does not account for the influence of pier aspect ratio *a/b* (width to length ratio). The influence of this ratio for these shapes is illustrated by laboratory data in Table 4-3 for overall pier shapes and aspect ratios indicated in Figure 4-8. For the same projected width of pier (140mm), scour depth varies substantially. The variations are due to differences in the pier flow field generated by each cylindrical form. The flow field (Figure 3-4) adjusts in response to bluffness of pier face, sharpness of corners, and the overall structure and spacing of turbulence structures generated. Further research is needed to document such flow-field changes and relate them to scour depth.

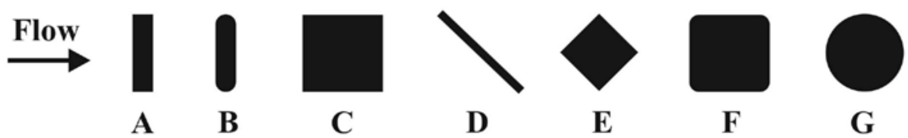


Figure 4-8 Cylinders differing in cross-sectional shape, but having the same projected width to the flow (Mostafa 1994)

Table 4-3 Comparison of local scour depths for the pier shapes shown in Figure 4-8 (Mostafa 1994)

Shape (Figure 4-17)	width/length ratio, *a/b*	Projected width of pier (mm)	$\frac{y_{s(noncircular)}}{y_{s(circular)}}$
A	4	140	1.50
B	4		1.33
C	1		1.29
D	200		1.28
E	1		1.28
F	1		1.07
G	1		1.00

4.4.5 Pier Alignment, θ

The depth of local scour for all shapes of pier, except circular, is strongly dependent on pier alignment θ to the approach flow. As θ increases, scour depth increases because the effective frontal width of the pier is increased.

A figure or chart of multiplying factors, K_θ, is recommended for use to account for the influence of flow alignment on scour depth at non-circular piers. Figure 4-9 shows the importance of alignment. For example, the local scour depth at a rectangular pier b/a = 8 is nearly tripled at an angle of attack of 30^o. The angle of attack at bridge crossings may change significantly during floods for braided channels, and it may change progressively over a period of time for meandering channels. The use of circular piers, a row of piles or other shapes of low (length-to-width) aspect ratios is beneficial, where such changes in flow alignment are possible.

The K_θ values in Figure 4-9 were obtained (Laursen and Toch 1956) for rectangular cylindrical piers by normalising the measured scour depths with the value at θ = 0°. Richardson et al. (2001) provides a table of values drawn from the curves in Figure 4-9. Ettema et al. (1998) show that the curves in Figure 4-9 are reasonably consistent with new laboratory data (Mostafa, 1994), but note that the maximum scour depth at skewed piers of low aspect ratio (small b/a) occurs at skew angles slightly less than 90°. This latter phenomenon arises because the projected width a_p ($a_p = b \sin\theta + a \cos\theta$ for rectangular piers) of such piers is larger than for θ = 90°. For example, the maximum projected width ($da_p/d\theta = 0$) for a rectangular pier, with b/a = 6, occurs at $\theta = \tan^{-1}(b/a)$ = 80.5°.

The combined influences of pier alignment and shape are sometimes combined with pier width, so as to be expressed as an effective pier width, a_*. As shown in Chapter 6, the leading methods for predicting scour depth commonly use a_*.

The scour depth related to effective pier width, though, may not increase, because other parameters may exert influences; e.g., *y/a* and *a/D*, as discussed in subsequent sub-sections. Research since 1990 indicates that the use of the curves is not without complication in this regard (e.g., Ettema et al. 1998). The influences of *y/a* and *a/D* on scour depth may vary with skew angle, which affects the upstream-projected width of the pier obstructing the flow. Although the influences of alignment, water depth and sediment size are connected, existing methods of estimating scour depth treat them as mutually independent.

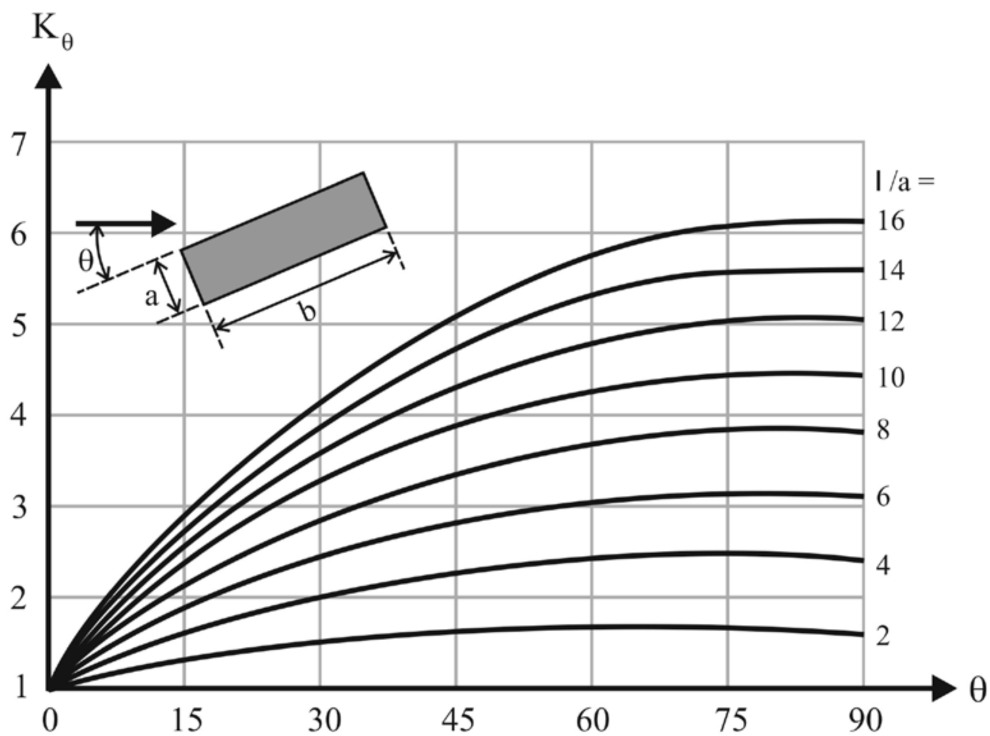


Figure 4-9 Local scour depth variation with pier alignment (Laursen and Toch, 1956)

4.4.6 Flow Intensity, V/V_c

Local scour at piers can occur under live-bed or clear-water conditions. Clear-water scour occurs for velocities up to the threshold velocity for general bed movement, for which there is no supply of sediment to the scour hole from upstream. Clear-water conditions are typically encountered on the flood channel of a compound river channel. Live-bed scour occurs when sediment is continuously supplied to the scour hole and the equilibrium depth is attained when there is a balance between the sediment supply and that transported out of the hole. The differences between clear-water and live-bed scour are highlighted in this section.

The variation of local scour depth at piers with flow intensity (and approach flow velocity), as evident from laboratory data, is shown in Figure 4-10.

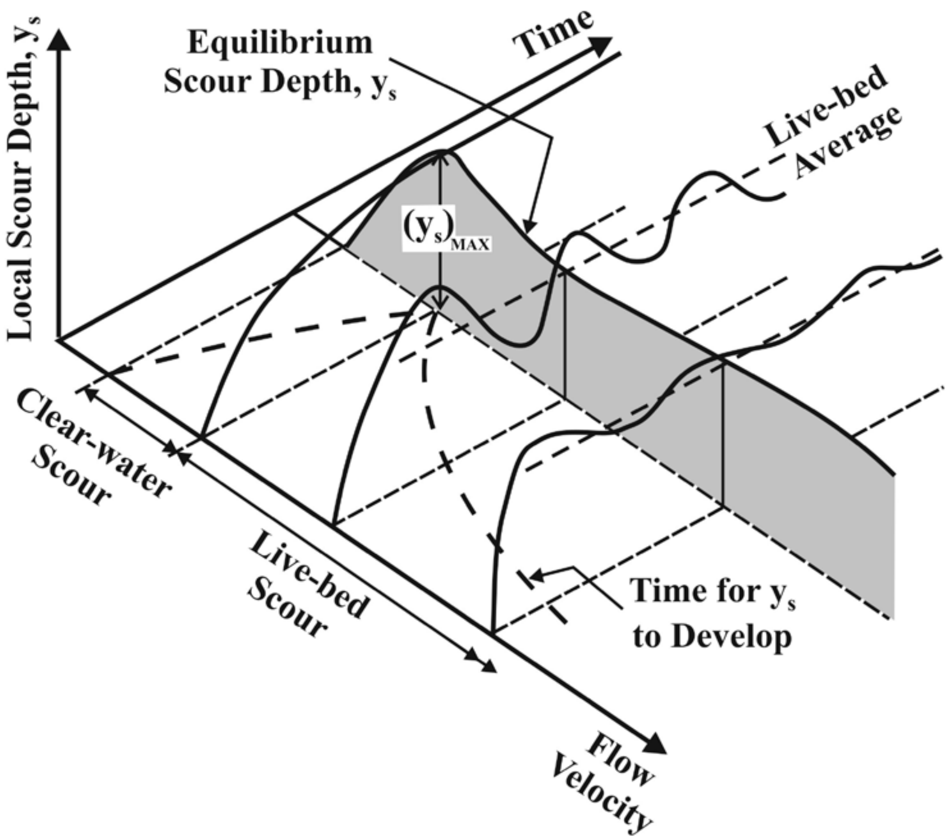


Figure 4-10 Local scour depth variation with flow intensity, V/V_c (Melville and Coleman 2000)

Under clear-water conditions, the local scour depth in uniform sediment increases almost linearly with velocity to a maximum at the threshold velocity. The maximum scour depth is called the threshold peak. As the velocity exceeds the threshold velocity, the local scour depth in uniform sediment first decreases and then increases again to a second peak, these changes being relatively small, but the threshold peak is not exceeded providing the sediment is uniform. The second peak occurs at about the transition flat bed stage of sediment transport on the channel bed and is termed the live-bed peak. These trends have been observed by others, including studies conducted over fifty years ago: Chabert and Engeldinger (1956), Garde et al. (1961), Shen et al. (1966), Maza Alvarez (1968), Gill (1972), Raudkivi and Ettema (1983), Chiew (1984), Baker (1986) and Dongol (1994). Consequently, in the laboratory, the maximum local scour depth in uniform sediments occurs at the threshold condition and the live-bed scour depth is largely independent of flow velocity. This fact is acknowledged in early studies, including Laursen and Toch (1956), Shen et al. (1969), Shen (1971), Hancu (1971) and Breusers et al. (1977). The weak dependency of scour depth on flow velocity is an important consideration for establishing a method for design estimation of scour depth.

The scour depth variations under live-bed conditions are a consequence of the size and steepness of the bed features occurring at particular flow velocities (Chee, 1982; Chiew, 1984; Melville 1984; Raudkivi, 1986; Melville and Sutherland, 1988; and Dongol, 1994). The steeper and higher the bedforms, the lesser the observed scour depth because the sediment supplied with the passage of a given bedform is not fully removed from the scour hole prior to the arrival of the next bedform. The live-bed peak occurs at about the transition flat bed condition when the bedforms are very long and of negligible height. Anti-dunes dissipate some energy at higher velocities and the local scour depth appears to decrease again. The magnitude of the scour depth fluctuations due to bedform migration is approximately equal to the half-amplitude of the bedforms (see Melville and Coleman, 2000, Section 4.7), indicating that the scour depth due to bedforms is about one-half the bedform height (Shen et al., 1966; Chee, 1982; Chiew, 1984; and Dongol, 1994).

For non-uniform sediments, the scour depth maxima are termed the armour peak and the live-bed peak. Armouring occurs for $V<V_a$ and the scour depth is limited accordingly. Beyond the threshold velocity for the transition from clear-water to live-bed conditions for non-uniform sediments, V_a, the armouring diminishes and live-bed conditions pertain. The live-bed peak, which typically exceeds the armour peak, occurs at the transition flat bed condition when all particle sizes in the non-uniform sediment are in motion. At the live-bed peak, the scour depth is about the same for uniform and non-uniform sediments of the same median size.

Equilibrium is reached much more rapidly under live-bed conditions than under clear-water conditions (Figure 4-10). Thus, the live-bed peak may be the critical condition for design because clear-water conditions may not last long enough for the scour depth, associated with the threshold peak, to be attained.

Figure 4-11 (uniform sediments) and Figure 4-12 (non-uniform sediments) are plots of laboratory data from many sources for local scour at piers in terms of the flow intensity parameter K_I. The flow intensity parameter is defined, for each set of data, as the scour depth at a particular flow intensity, y_s, divided by the maximum scour depth for the data set, y_{smax} (see also Figure 4-10), where V is systematically varied for each data set and all other dependent parameters are held constant. The scour maxima used occur at the threshold peak for uniform sediments and the live-bed peak for non-uniform sediments. The non-uniform sediment data are plotted in terms of a transformed velocity parameter, $[V\text{-}(V_a\text{-}V_c)]/V_c$. The transformed velocity parameter aligns the armour peaks (that is $V = V_a$) for non-uniform sediments with varying σ_g with the threshold peak for uniform sediments. For uniform sediments, $V_a \equiv V_c$, and $[V\text{-}(V_a\text{-}V_c)]/V_c \equiv V/V_c$. The transformed velocity parameter incorporating V_a largely accounts for the effects of sediment non-uniformity as well as those of flow velocity, although the smaller values of scour depth at $[V\text{-}(V_a\text{-}V_c)]/V_c \approx 1$, as σ_g increases, remain. Therefore, the effects of sediment nonuniformity are mostly accounted for in the flow intensity factor.

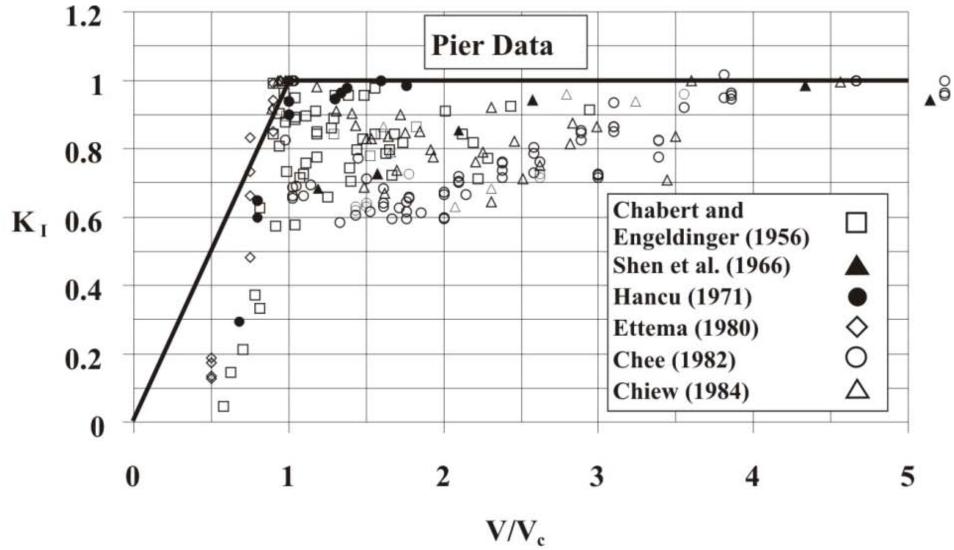


Figure 4-11 Influence of flow intensity on local scour depth in uniform sediment (Melville and Coleman 2000)

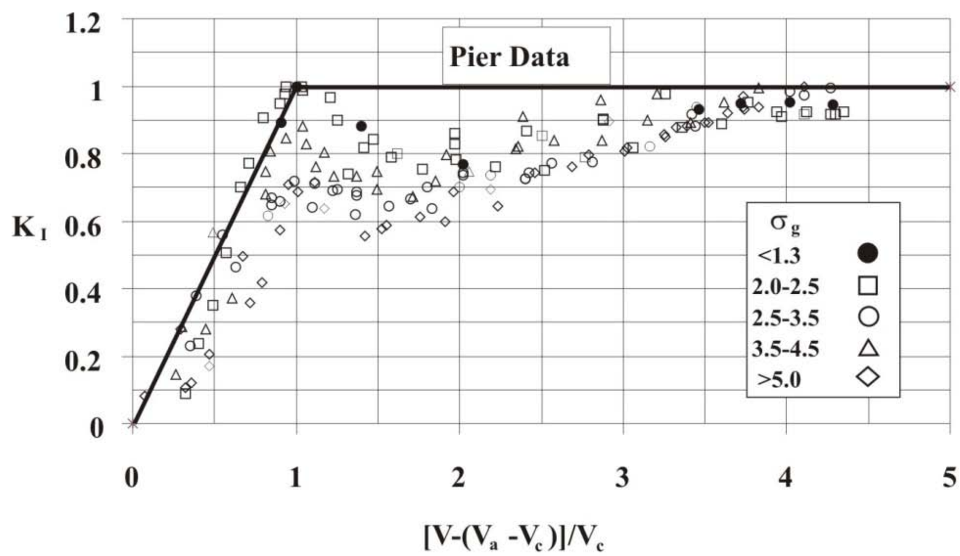


Figure 4-12 Influence of flow intensity on local scour depth in non-uniform sediment (Melville and Coleman 2000)

4.4.7 Sediment Non-uniformity, σ_g

Research since 1990 has not significantly advanced the insights obtained from early laboratory studies (e.g., Ettema 1976, 1980, Chiew 1984 and Baker 1986) regarding the effects of sediment-diameter non-uniformity on local scour depth under clear-water conditions at piers. Some of the data from these studies are given in Figure 4-13 for clear-water scour.

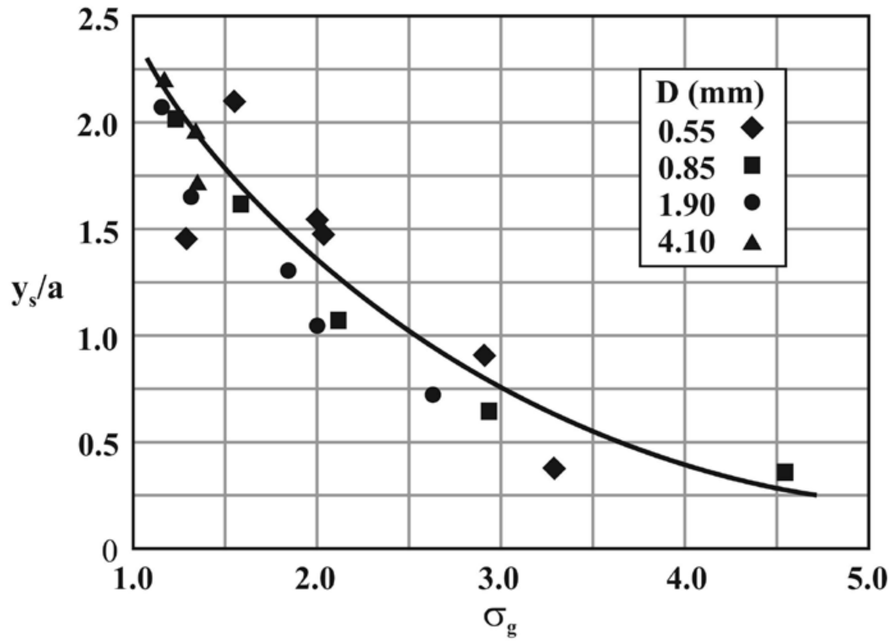


Figure 4-13 Influence of sediment non-uniformity on local scour depth at piers subject to clear water scour (Melville and Coleman 2000)

The broader context of the data is given in Figure 4-14, which schematically summarises the trends associated with varying intensity of bed material motions, V/V_c. Around the threshold condition, $V/V_c \approx 1$, armouring occurs on the approach flow bed and at the base of the scour hole. The armoured bed in the erosion zone at the base of the scour hole significantly reduces the local scour depth. Conversely, at high values of V/V_c, when the flow is capable of entraining most grain sizes within the non-uniform sediment, sediment non-uniformity has only a minor effect on the scour depth. At intermediate values of V/V_c, the effect of σ_g reduces progressively with increasing flow velocity between these two limits, as more and more of the grains are transported by the flow.

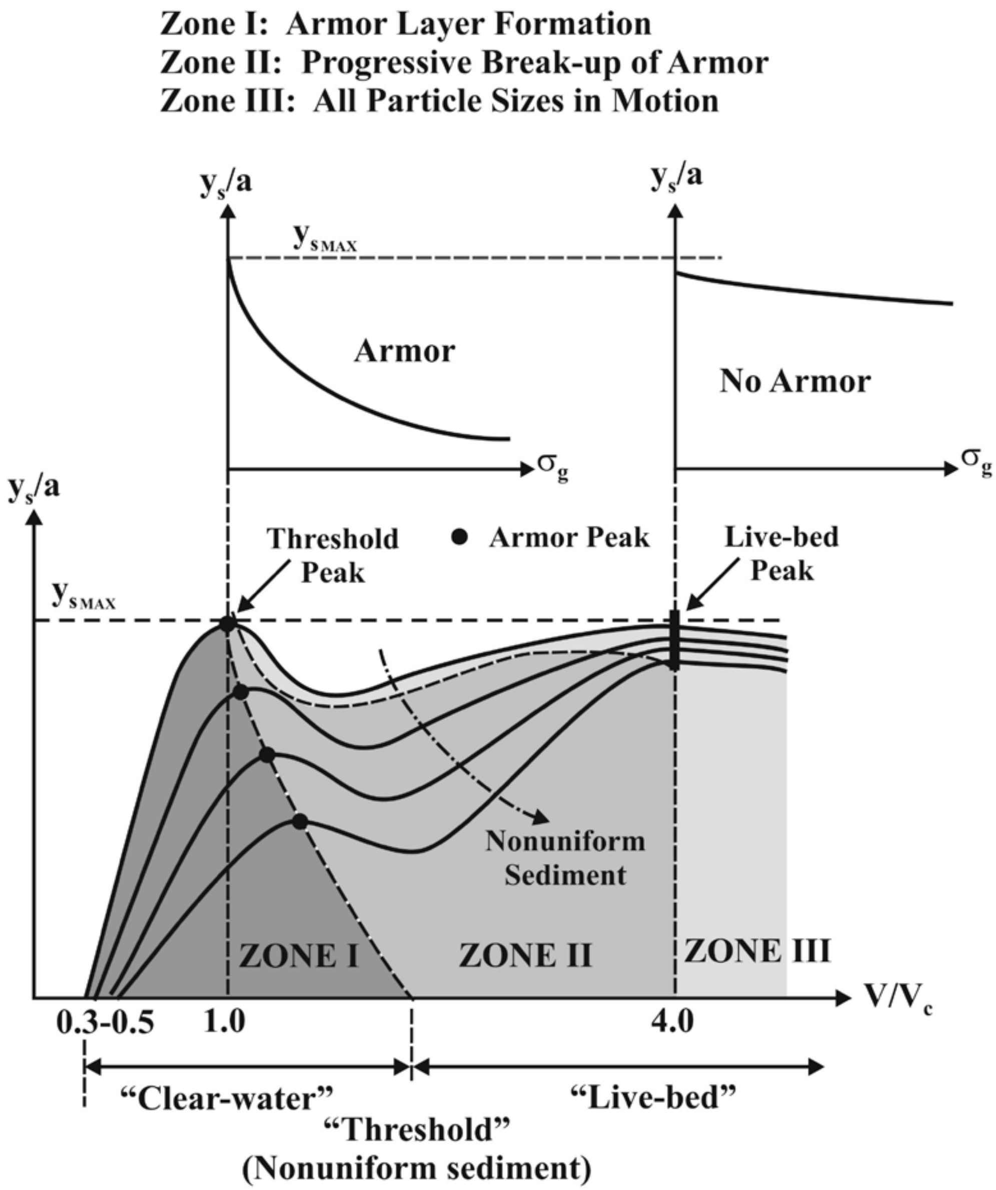


Figure 4-14. Local scour depth variation with sediment non-uniformity (Melville and Coleman 2000)

4.4.8 Power of Turbulence Structures, *Eu* and *Re*

The energy associated with turbulence structures in the pier flow field can be characterized in terms of a pier Euler number, $Eu = V^2/ga$, and the frequency of vortex formation and break-up/shedding in terms of pier Reynolds number, $Re = \rho Va/\mu$.; Ettema et al. (1998) and Ettema et al. (2006).

A substantial scale effect occurs in loose-bed models used to investigate local scour at various hydraulic structures. It arises because of inconsistencies in flow field similitude incurred when intensity of bed sediment movement is used as the primary criterion for similitude, as elaborated briefly below. This effect can be seen in the study reported by

Ettema et al. (2006) for piers, and by Sturm and Chrisochoides (1997), who investigated scale effects in two hydraulic models of bridge abutments. It arises because of the difficulty in simultaneously satisfying three length scales (y, a, and D).

The parameters V^2/ga and $\rho Va/\mu$. can be interpreted as, expressing similitude in the energy and frequency (hence power) of eddies shed from a pier. This is because, for flow as an eddy, V^2/ga is a normalized expression of eddy energy. Given the range of length scales commonly used in scour experiments, Reynolds number in terms of viscous effect is unlikely to have direct bearing on pier-scour depth. However, Reynolds number also influences the frequency of shedding, n, which can be estimated using the relationship between Strouhal number (St) and Reynolds number (Re) for flow around circular cylinders;

$$\frac{na}{V} = function\left(\frac{\rho Va}{\mu}\right) \tag{4.3}$$

For the typical values of cylinder widths and flow velocities used in flume experiments, as well as for cylindrical piers in rivers, Re lies between 10^3 to 10^5, for which $St \approx 0.2$. Thus for cylinders in the same approach flow (V = constant), the frequency of vortex shedding, n, is inversely proportional to cylinder width (diameter). In other words, smaller cylinders in the same flow generate eddies at a greater rate.

The capacity of vortices to erode sediment has been shown by several studies. For instance, Dargahi (1989, 1990) began by studying the vortex systems formed by a cylinder on a flat bed. He looked at the interaction of the horseshoe vortex and wake vortices, observing that the main oscillatory frequency of the horseshoe vortices is close to the shedding frequency associated with the cylinder's Strouhal number, and concluded that the two mechanisms are connected. Also, he observed that wake vortices cause bursts to occur downstream of the cylinder. Dargahi (1990) studied vortex shedding for a cylinder placed in a deformable bed, and found essentially the same vortex shedding behaviour as for the fixed flat bed. His observations, together with those reported from scour development around circular cylinders (e.g., Hjorth 1975, Melville 1975, Ettema 1980) and bed-sediment transport generally (e.g., Müller and Gyr 1986, Yalin 1992), indicate that the passage of vortices increases sediment entrainment and movement.

Ettema et al. (1998) present data suggesting that scour depth at piers does not scale linearly with pier width unless there is more-or-less complete geometric similitude of pier, flow and bed sediment particles. The non-linearity can result in laboratory flume studies of local scour (at scale-reduced model piers) leading to deeper scour holes relative to pier width than any likely to occur in the field.

Many laboratory experiments have been undertaken using sands to model sand bed rivers. Consequently, the model bed material relative to the pier size is larger than its scaled counterpart in the field. To ensure similitude of the state of bed mobility requires that the value of V/V_c be maintained the same in the laboratory and the field, implying that the flow velocity used in the laboratory may need to be larger than that derived from Froude scaling of the flow velocity in the field. Hence, the Froude number used in laboratory experiments may be larger than that for the corresponding field conditions.

Ettema et al.'s (1998) data show that scour depth, relative to pier width, may increase with pier Euler number. The parameter V^2/ga is useful for describing energy gradients for flow around a pier. It can be considered to express the ratio of stagnation head $V^2/2g$ to pier width. Flow-field similitude requires preservation of flow patterns such that pressure heads along flow paths scale directly with the geometric scale relating a model pier in the laboratory to a pier in the field. For the same stagnation head $V^2/2g$, steeper gradients occur at narrower piers. A narrower pier will induce a larger value of V^2/ga , and thereby larger y_s/a, than will a wider pier in the same flow field.

Further data, in addition to those presented by Ettema et al. (1998), are needed to quantify the influence of the Euler number on local scour depth at piers. Nevertheless, the data do indicate that using a value for the maximum scour depth at circular piers of 2.5*a* is conservative for all pier sizes larger than about 0.1m. A recent study by Ettema et al. (2006) suggests the need for the addition of a factor for *Eu* effects in scour prediction methods (see variation of y_s/a with the Euler number in Figure 4-15). Based on their laboratory data, Ettema et al. (2006) defined a correction factor that could be used to account for Euler number effects in laboratory-scale tests.

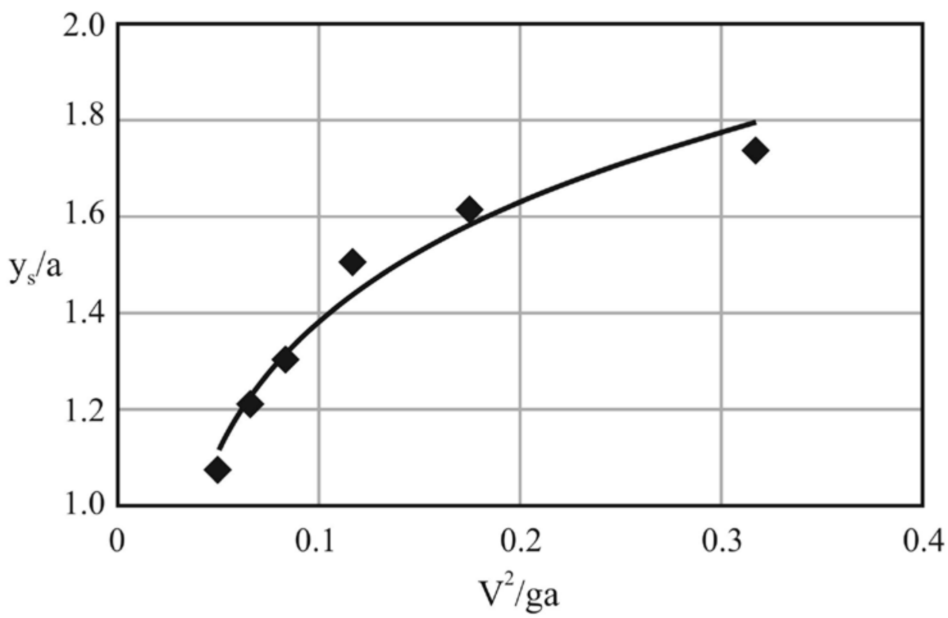


Figure 4-15 Variation of y_s/a with Euler number (Ettema et al., 2006)

4.4.9 Time Rate of Scour, *tV/a*

Though the objectives set for the present evaluation do not expressly include evaluation of the time-rate of scour development, it is useful to mention that differences in pier flow fields and in foundation material affect the rate of scour development. The parameter *tV/a* relates scour duration to pier width *a*, and approach velocity *V*. Many studies focus on the time-development of scour at piers of simple cylindrical form. However, relationships expressing the time rate of scour development for the narrow-, transition-, and wide-pier categories (Table 4-1) likely differ, as do relationships expressing scour in foundation material comprising non-cohesive, cohesive, or weak rock.

It is well known that local scour under clear-water or live-bed conditions develops at quite different rates, and are subject to different equilibrium balances (erosion of sediment from scour hole, and sediment inflow and outflow rates, respectively for the two conditions). Figure 4-16, a simplification of Figure 4-10, conceptually illustrates the time-development trends. Under clear-water scour conditions, the scour depth develops asymptotically towards an equilibrium depth of scour. Under live-bed conditions, the equilibrium depth is reached more quickly and thereafter the scour depth oscillates due to the passage of bed features past the pier or abutment. Therefore, in most cases when scour occurs under live-bed conditions, the time–development of scour usually is not of major significance. Still, there are bridge sites where limits in the duration of scouring flows constrain scour depths. This is true especially for scour developing under clear-water conditions in which no significant bedload occurs, but sometimes even for scour under live-bed conditions. For instance, time can be a constraint for scour associated with storm-surge flows at tidal or estuary bridge sites, and for sites a short distance downstream from large dams.

There exist much laboratory data on the time-development of scour at piers, most being for clearwater scour (e.g., Chabert and Engeldinger 1956, Shen et al 1969, Hjorth 1975, Nakagawa and Suzuki 1975, Ettema 1980, Yanmaz and Altinbilek 1991, Bertoldi and Jones 1998, Melville and Chiew 1999, Miller 2003, Dey and Raikur 2005, Yanmaz 2006, Oliveto and Hager, 2005, Kothyari et al., 2007). Also, quite a few formulations have been proposed for scour development at piers (e.g., Hjorth 1977, Ettema 1980, Johnson and McCuen 1991, Miller and Sheppard 2002 (also Miller 2003), Faruque and Nago 2003, Dey and Raikur 2005, Yanmaz 2006, Oliveto and Hager, 2005, Kothyari et al., 2007, Lai et al. 2009), or for indicating a temporal scale for scour development (e.g., Melville and Chiew 1999). However, the vast majority of the data, and most of the formulations, are for scour at circular cylindrical piers in the narrow-pier category. The methods proposed by Oliveto and Hager (2005) and Kothyari et al. (2007) can be applied for piers of various shape and predict an unbounded logarithmic increase in the local scour depth with time.

Predicting the time-development of scour at piers in situations of limited flow duration remains a challenge. Such situations include piers on floodplains that inundate only during relatively rare flood events; piers in tidal environments subject to storm-surge effects; and piers subject to tsunami effects. During these conditions, the scour condition arguably is predominantly a clearwater-scour condition, though site variations may occur.

Several papers outline the difficulties of scour estimation for these conditions; e.g., Hughes (1999), Normets et al. (2003), Tonkin et al. (2003), and Kothyari et al. (2007).

It is useful to point out there is little information on the time development of clear-water local scour at piers considered wide or of complex geometry. The data and the formulations upon which the time-development relationships are based do not take into account Ω, θ influences, nor do they adequately account for values of *a/y* and *a/D* in ranges found for wide piers, and therefore are of questionable validity when scaled, or applied, to wide piers and long skewed piers.

The expressions are largely semi-empirical, or essentially empirical, in nature. Some of them begin with rational expressions of sediment transport rate out of a conical scour hole. These expressions eventually rely on empirical calibrations connecting the formulation to laboratory data. Other expressions simply normalize scour depths, as scour progresses, with an equilibrium scour depth. However, the expressions demonstrate the difficulty of arriving at a general predictive formulation for scour development that will be valid over several orders of magnitude of cylindrical pier diameter, not to mention a method sufficiently general to handle piers of complex or unusual geometry. They also suggest that, when estimating time-dependent scour at piers of complex geometry, the viable approach is to develop relationships for common (or comparatively similar) pier geometries.

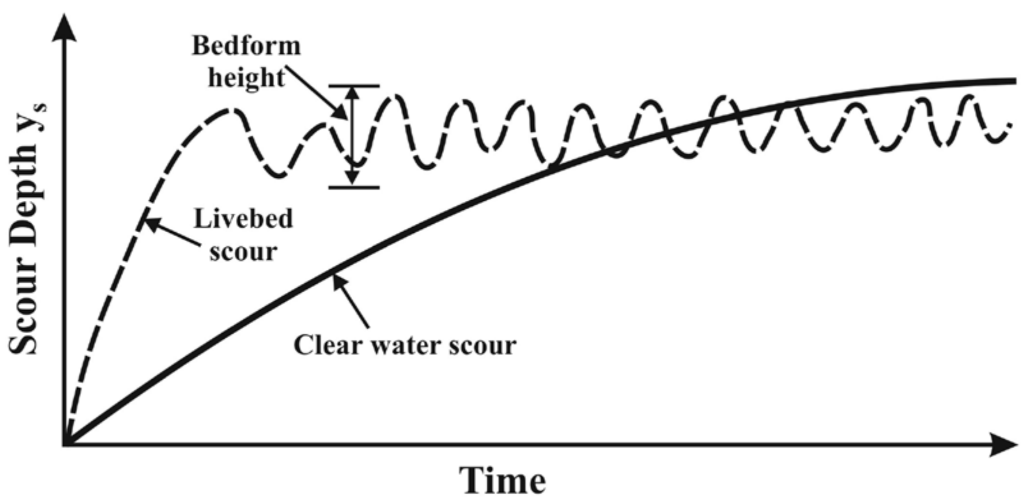


Figure 4-16 Time-development of scour at a cylindrical pier subject to clear-water or live-bed conditions

Though field data on the temporal development of pier scour are scarce, measurements during routine bridge surveys (such as conducted by state departments of transportation) indicate that multiple flood events may be required before scour reaches a maximum depth at a pier site, especially for piers founded in material than does not erode in a particle-by-particle manner as does non-cohesive sediment; notably, foundations comprising cobble-armoured beds, or beds comprising cohesive soils and/or weak rock.

For example, Hopkins and Beckham's (1999) comprehensive study of rock scour at bridge piers and abutments in Kentucky shows that for scour of weak rock may occur over several years before becoming noticeably severe. They also found no clear relationship between rock scour and bridge age. The equilibrium scour depth in live-bed conditions fluctuates due to the effects of bed-form migration. The dashed lines in Figure 4-16 represent the temporal average scour depth under live-bed conditions. As Figure 4-10 infers, variations in the duration of approach velocity magnitude, or variation in the resistance and nature of foundation material erosion, may complicate the time development of scour at a pier site.

Most equations for depth of local scour give the equilibrium depth and are, therefore, conservative regarding temporal effects. For the live-bed conditions that typically occur in floods, equilibrium scour depths are appropriate. However, where clear-water scour conditions exist, the equilibrium depth of scour may be overly conservative. The actual scour may be only a small fraction of the equilibrium scour depth, which could take weeks to fully develop. To achieve equilibrium conditions in small-scale laboratory experiments of clear-water scour depth development at bridge foundations, it is necessary to run the experiments for several days. Data obtained after lesser times, say 10 to 12 hours, can exhibit scour depths less than 50% of the equilibrium depth.

The shape of the flood flow hydrograph is important as well as the flood duration. Typically, the flood duration determines if the equilibrium live-bed scour will develop. After the flood peak passes, the flow recedes. The duration of the recession period is also important. With flow recession, clear-water conditions may prevail, which could induce additional scour, especially if near-threshold conditions are maintained over a considerable period.

Chiew and Melville (1996), also Melville and Chiew (1997), presents extensive laboratory data that describe the temporal development of local scour at circular bridge piers (of diameter a) under clear-water conditions. Their data show that local clear-water scour depth increases asymptotically towards the equilibrium local scour depth, y_s, Figure 4-17 shows that local scour depths at the same stage of development (t/t_e, where t_e is the time to develop the equilibrium depth of scour) are reduced at lower values of V/V_c.

Melville and Chiew (1997) show that both t_e and y_s are subject to similar influences of flow and sediment properties, as might be expected because they are inherently interdependent. The main trends from their plots, which show the dependence of a (dimensionless) equilibrium time scale t^* ($=Vt_e/a$) on flow shallowness (y/a), flow intensity (V/V_c) and sediment coarseness (a/D), are illustrated in the sketches in Figure 4-18. Flow shallowness effects are depicted in the upper diagram of Figure 4-18.

The equilibrium time scale increases with y/a for shallow flows and becomes independent of y/a for deep flows. The apparent limit to the influence of flow shallowness on t^* occurs at $y/a \sim 6$. The maximum value of t^* is about $2.5\text{x}10^6$. The middle diagram shows that the equilibrium time scale increases rapidly with flow intensity for clear-water scour conditions, attaining a maximum value at the threshold condition. At higher live-bed flows, t^* is expected to rapidly decrease again, as shown by the dashed line (refer also to Figure 4-10). The lower diagram indicates that t^* increases asymptotically with *a/D*. The limit to sediment coarseness influence on t^* occurs at $a/D \approx 100$.

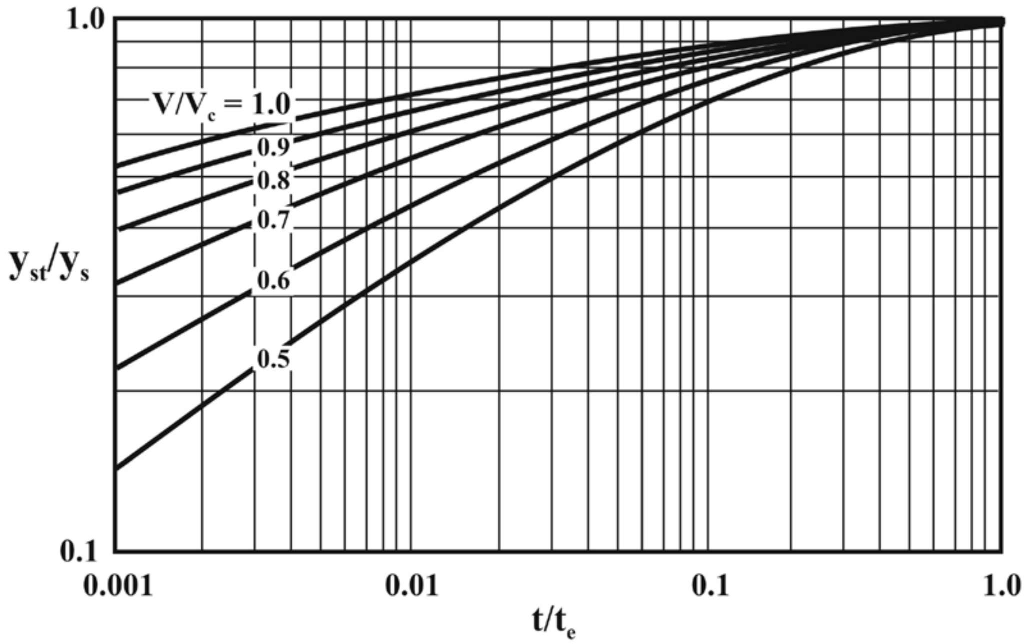


Figure 4-17 Temporal development of local scour at piers, clear-water conditions (Melville and Chiew 1997)

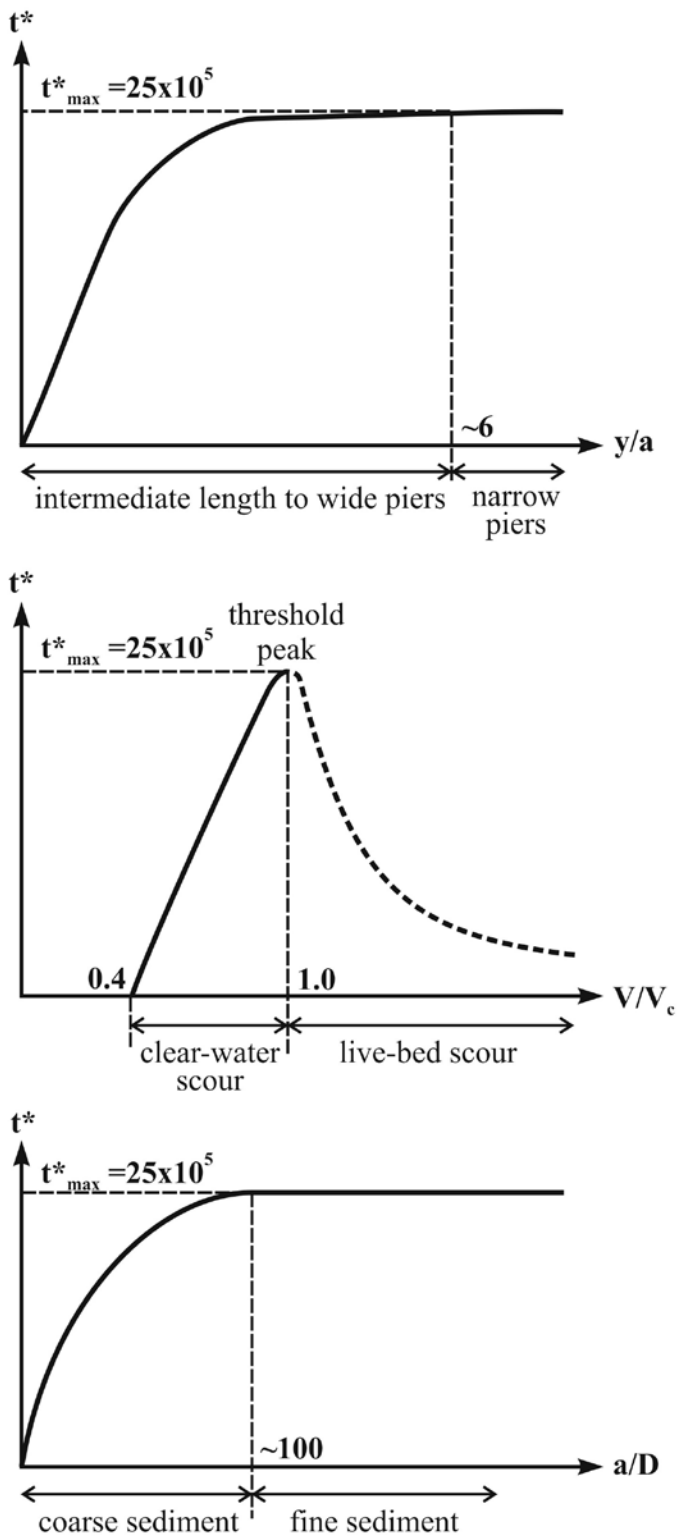


Figure 4-18 Equilibrium time-scale variation with flow shallowness, flow intensity and sediment coarseness

4.5 Data Quality and Gaps

The parameter trends are based largely on data obtained from laboratory flume experiments, and to a far lesser extent on field measurements at actual pier sites. The flume experiments normally use similitude considerations to interpret the data obtained. At times the quality of data is uncertain (or not documented), and data gaps exist for uncommon or hard-to-model pier situations. Moreover, laboratory experiments have limitations, which arise for several reasons:

1. The vast majority of laboratory studies have used small-scale, simple cylindrical forms to replicate piers. Consequently, the difficulty of satisfying the similitude requirements associated with three length scales (flow depth, pier width, and boundary material) typically have not been satisfied, and thereby requires the interpretation of laboratory data so as to account for scale effects;

2. A large number of variables influence scour depth, even for cylindrical piers. Therefore, a great number of tests are needed to define the influences. Most studies have involved series of tests examining the influence of a variable cast in terms of one independent parameter at a time. Though useful for some variables (e.g. pier form), this approach may yield incomplete insights when a variable exerts multiple influences, or when different variables in a parameter are adjusted. For instance, varying flow depth y in the parameter a/y may produce somewhat different results than varying pier width a, because other parameters containing these variables are hard to keep constant;

3. Laboratory techniques for flow velocity measurement and visualization have not been able to reveal the full temporal variations of the complex flow field at piers. A new approach, numerical modeling is needed for this purpose;

4. The manner whereby the pier flow field entrains and transports material from the scour hole still is not adequately understood and reflected in the semi-empirical equations for scour-depth estimation;

5. Limitations on flume size and flow capacity have meant that there are few data from tests done with large diameter piers (say a > 0.75m) in live-bed scour conditions. Remarkably few flumes have the capacity to accommodate such tests; and,

6. The difficulties of replicating cohesive or rocky foundation material, along with the considerable period needed to run scour tests with these materials, has resulted in only a modest amount of data for scour in these foundation materials.

For design estimation of scour depth, however, the potential maximum scour depths for the narrow- and transition-pier categories can be determined reliably from laboratory data. The same cannot yet be said for wide-pier scour.

CHAPTER 5
PIER SITE COMPLICATIONS

5.1 Introduction

Several factors complicate pier sites at bridge waterways, and thereby introduce additional processes and parameters to be considered when estimating pier scour depth. The additional processes of practical importance can be grouped in two categories:

1. Factors affecting pier flow field:

    i. Bridge structure (pier form, abutment proximity, deck submergence);
    ii. Debris or ice accumulation at a pier or across the bridge waterway;
    iii. Channel geometry; and,
    iv. Tide-affected flows.

    These factors may substantially change the overall flow field at a pier, relative to the flow field at an isolated pier in a steady approach flow. Some factors (e.g. pier-shape complexity, or abutment proximity) change the flow field so much that the scour-depth estimation methods discussed subsequently in Chapter 6 do not readily apply. Other factors (notably, debris at an individual pier), considered with certain limitations, may be treated through the proposed use of an effective or equivalent pier width.

2. Factors complicating assessment of the erosion resistance of foundation material:
    i. Multiple strata of alluvium;
    ii. Cohesive soil (clay); and,
    iii. Weak rock.

    The considerations associated with these factors cause foundation material to erode in a less predictable manner (at least given the present state of knowledge) compared to erosion of a continuous homogeneous boundary of non-cohesive foundation material.

A third factor of less practical importance, though needing to be addressed here, concerns the possible influence on scour depth of washload sediment. This factor may affect the approach flow and the erosion resistance of the approach bed.

When considering the forgoing site complications, Eq. (4.2) must be broadened to include combinations of parameters from the framework indicated in Eq. (5.1);

$$y_s = function \begin{bmatrix} flow\ (\rho, \mu, V, y, g), bed\ material\ \left(D, \sigma_g, \rho_s, c, V_c,\right), pier\ (a, \Omega, \theta), \\ time\ (t), \\ bridge\ structure, \\ debris\ or\ ice\ accumulation, \\ channel\ geometry, \\ foundation\ material\ complexities, \\ washload\ in\ approach\ flow \end{bmatrix} \tag{5.1}$$

Design estimation to account for these possible complications can be difficult when the pier is of unusually large size, especially relative to flow depth, or when the pier is of an unusual form. The design estimation of scour depth at wide cylindrical piers (as defined in Table 4-1) is then compounded by the additional number of variables to be considered.

This chapter reviews knowledge regarding pier site complications, but does not attempt to identify all the main variables and parameters associated with each complication. Instead, references for further information are provided. Also, this chapter does not elaborate scour in tide-affected flows, except to note that the pier flow field is subject to tidal ebb and flow, which may cyclically reverse the approach flow to a pier.

5.2 Pier Structure

Chapter 4 discusses scour at simple cylindrical piers extending to depth. However, pier structures typically comprise multiple components as illustrated in Figure 5-1. The additional components add uncertainty to design estimation of scour depth. Design estimation of scour depth must account for pier structure.

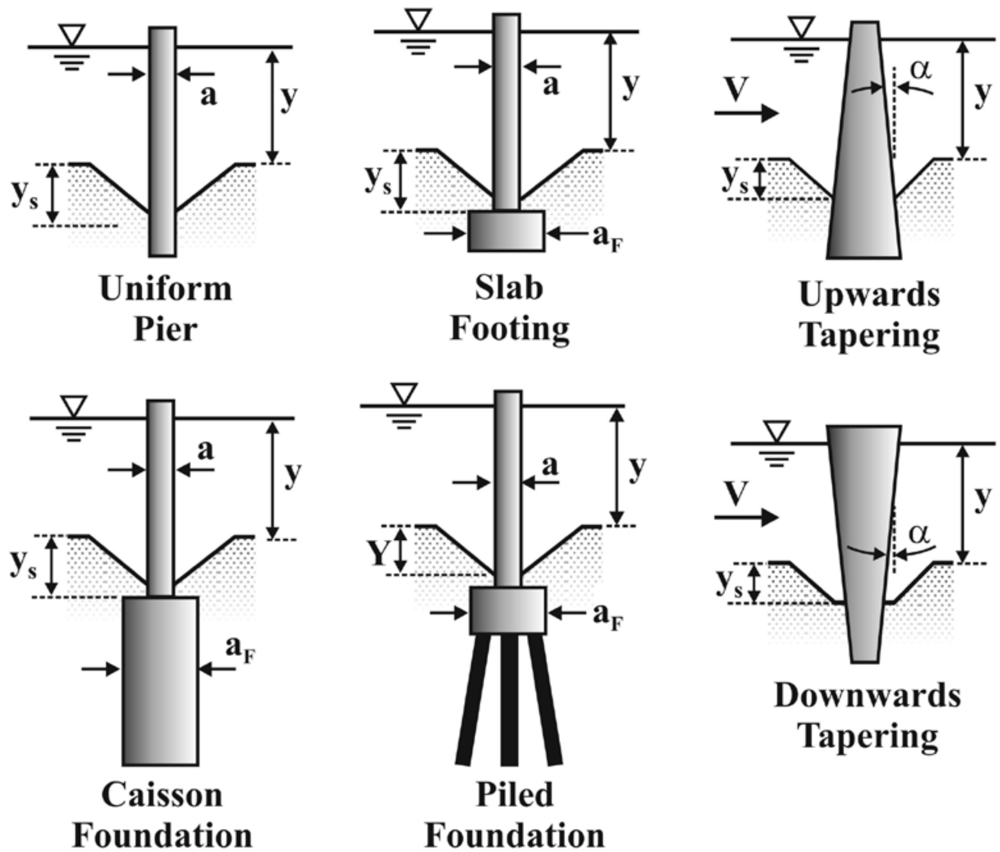


Figure 5-1 Common pier structures comprise a column on a slab footing, or column on pile cap with underpinning piles (Melville and Coleman 2000)

For piers tapered on the upstream and downstream faces, the slope, in elevation, of the leading edge of the pier affects the local scour depth. Downwards-tapering piers induce deeper scour than a circular pier of the same top width, and vice-versa. Shape factors for local scour at tapered piers have been proposed by Neill (1973), Chiew (1984), and Breusers and Raudkivi (1991).

For piers founded on a (wider than the pier) slab footing, caisson or pile cap, placing the footing, cap or caisson with its top below the boundary level can be effective in reducing the local scour depth through interception of the down-flow. However, if the top of the wider foundation element comes to the bed level (in the undisturbed flow region away from the pier), or even above, the scour depth is increased. Unless definite predictions are possible, it is risky to rely on limitation of the scour depth through flow interception by the wider base element.

The results of laboratory investigations of local scour at piers founded on slab footings and caissons have been quite extensively reported in studies before 1990 (notably, Chabert and Engeldinger 1956, Tsujimoto et al. 1987, Imamoto and Ohtoshi 1987, Jones 1989), and in publications since 1990 (Jones et al. 1992, Fotherby and Jones 1993, Parola et al. 1996b, and Melville and Raudkivi 1996). Local scour at piers founded on piles similarly has been extensively studied (early studies by Hannah 1978, Raudkivi and Sutherland 1981, Jones 1989), with more recent publications by Richardson and Davis 1995, Sheppard et al. 1995, and Salim and Jones 1996). A consideration for all of these studies is that pier structure can vary quite extensively.

Melville and Raudkivi (1996, also in Melville and Coleman 2000) identify five cases of pier scour for piers comprising a column supported by a slab footing, pile-cap on piles, or caisson (Figure 5-2):

- Case I, where the top of the footing, cap or caisson remains buried below the base of the scour hole;
- Case II, where the top of the footing, pile-cap, or caisson is exposed within the scour hole below the general bed level;
- Case III, where the top of the footing, pile-cap, or caisson is above the general bed level;
- Case IV, where the top of the footing, pile-cap, or caisson is at or above the water surface level; and,
- Case V, where the pile-cap is clear of water surface. The submerged portion of the pier comprises a set of piles; e.g., like a pile bent.

For Case I, the local scour is unaffected by the presence of the footing, pile-cap, or caisson, while for Case II, the local scour typically is reduced from that at a uniform pier, due to interception of the down-flow. For Case III, the local scour depth can be increased or decreased compared to that at a uniform pier. The Case III scour depth at a pier founded on a larger size pile-cap or caisson is increased, with the maximum local scour occurring for Case IV, where the top of the pile-cap or caisson is at or above the water surface level. Cases III and IV scour depths at piled foundations may be increased or decreased. Figure 5-2 schematically illustrates the trends, in which the level of the top of the footing, pile-cap, or caisson, Y, is measured from the general bed level and is positive downwards.

Melville and Raudkivi (1996) made a detailed laboratory study of local scour at a non-uniform cylindrical pier, comprising a circular pier (diameter a) founded on a larger diameter circular caisson (diameter a^*). The diameter ratio (a/a^*) was varied systematically between 0.12 and 1.0, while the level of the top of the caisson (Y) spanned Cases I, II, III and IV. Their data are plotted in Figure 5-3.

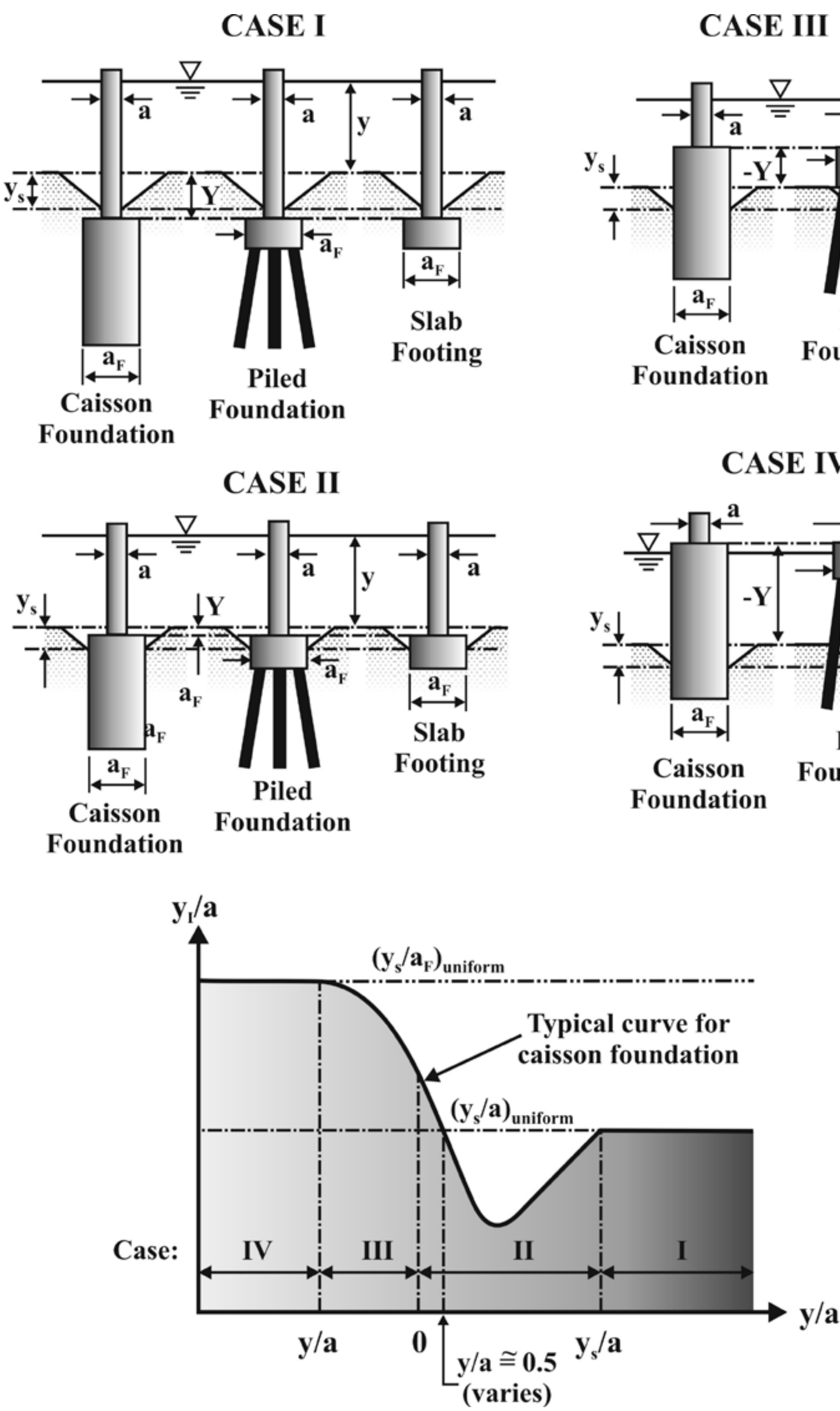
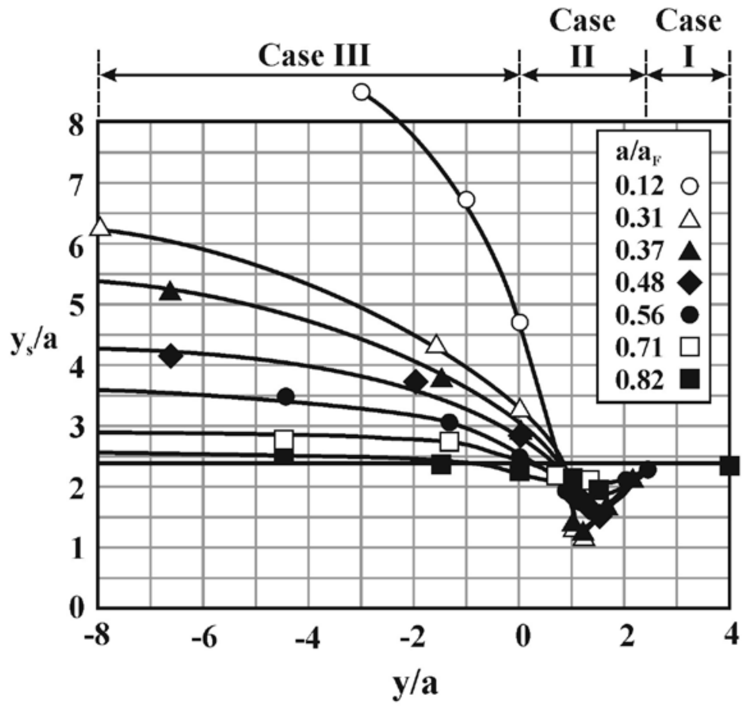


Figure 5-2 Scour depth variation for four cases of non-uniform pier shape (Melville and Coleman, 2000). Not shown is the case when the pile cap is fully above the water surface



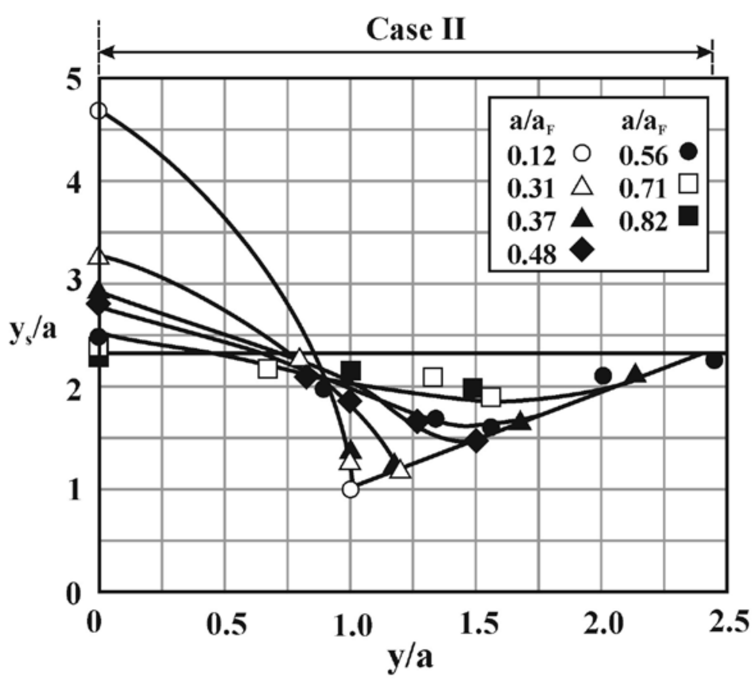


Figure 5-3 Influence of non-uniform shape on local scour depth at piers (Melville and Raudkivi 1996)

For scour at piled foundations where the pile cap is clear of the water surface (Case V), the comprehensive study by Hannah (1978) remains useful. He found that the maximum scour depth is closely related to the dimension of the pile group as a whole, as seen from upstream. Accordingly, he recommended that a single line of piles should be used in preference to piers for angles of attack greater than 8°. Similar results were obtained subsequently by Nazariha and Townsend (1997). Based on experiments by Jones (1989), Richardson and Davis (1995) recommend for Case III that the larger of two scour estimates be used: one based on the width of the pier column, and the other on the width of the pile cap, the latter using the flow depth and flow velocity in an assumed flow zone obstructed by the pile cap. This recommendation facilitates a design estimate, but does not reflect the flow field causing the scour.

Jones and Sheppard (2000) proposed a fairly involved approach to estimating scour depth at a pile-supported pier. The method is presented also by Richardson and Davis (2001) and FDOT (2010). Essentially, the method entails dismantling the pier's elements then summing the scour depths attributable to the pier column, pile cap, and pile group, as indicated in Figure 5-2 and Eq. (5-2).

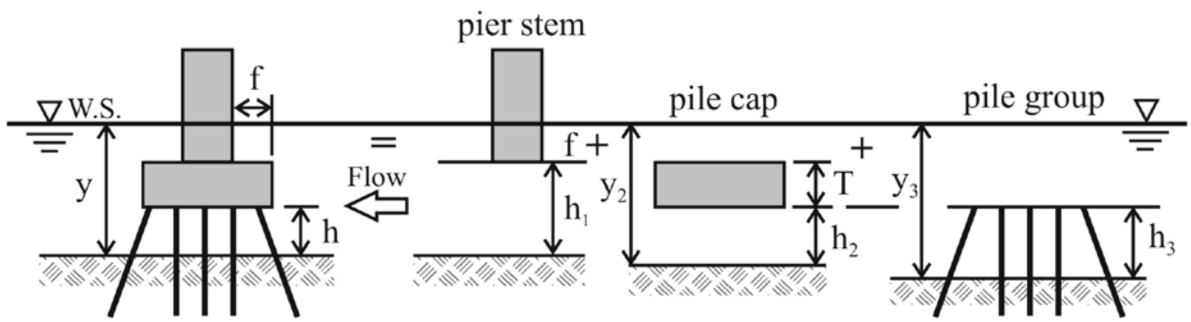


Figure 5-4. The disassembly approach proposed by Jones and Sheppard (2000) for estimating scour depth at a pile-supported pier

$$y_s = y_{s\text{-}pile\ stem} + y_{s\text{-}pile\ cap} + y_{s\text{-}pile\ group} \qquad (5\text{-}2)$$

in which total depth of pier scour is the sum of scour-depth component for the pier column in the flow, $y_{s\text{-}pile\ stem}$; the scour component for the pile cap or footing in the flow, $y_{s\ pile\ cap}$, and the scour component for the piles exposed to the flow, $y_{s\text{-}pile\ group}$. The variables in Figure 5-4 are defined by Richardson and Davis (2001). The procedure entails calculating a scour depth for each element treated as an equivalent cylinder subject to flow depths and velocities, and with height adjustment for the column and pile group. Richardson and Davis (2001) outline the calculation steps. Though each of these structural elements affects scour depth, the summation of scour attributable to individual parts lacks a physical basis, because it does not relate to the flow field producing scour.

The foregoing review indicates how increased complication of pier shape introduces uncertainty in design estimation of scour depth. Scour at some piers of typical form can be estimated using pier-shape factors developed for them, but such factors do not exist for many typical pier forms, and can be unreliable when pier alignment to flow varies.

5.3 Abutment Proximity

Many piers are sited near the toe of an abutment, as illustrated in Figure 5-5. Consequently, the pier is within the flow field generated by the abutment, and pier scour occurs within the region of abutment scour. Because an abutment may develop a much larger scour than usually occurs at a pier, scour depth at the pier is dominated by abutment scour. Design estimation of scour depth should check whether abutment proximity will influence scour depth.

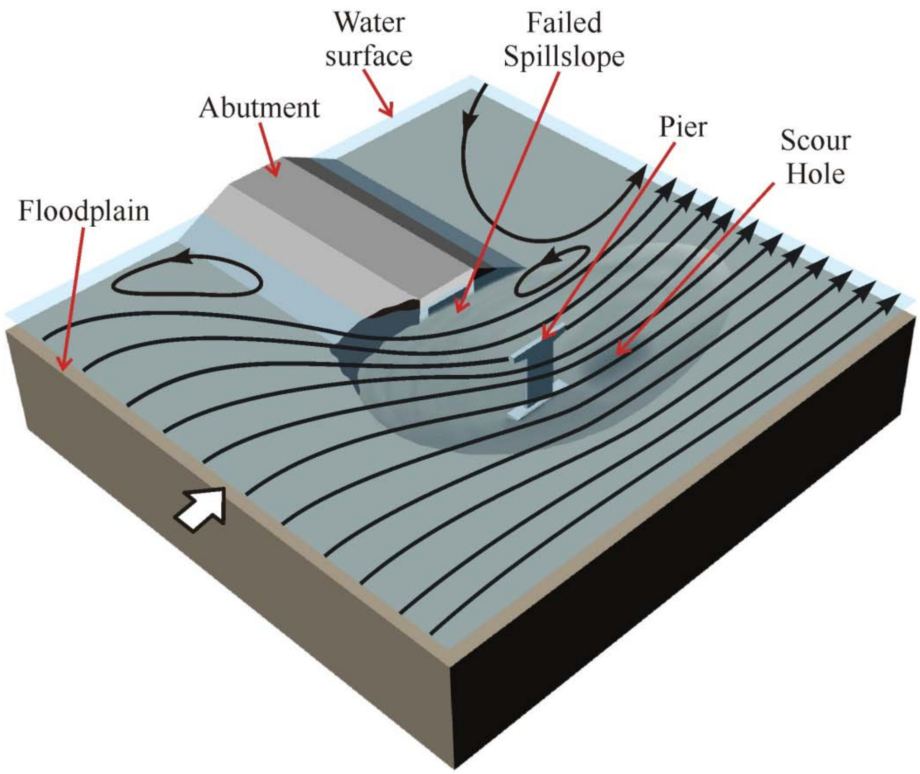


Figure 5-5 Abutment proximity close to a pier may substantially alter the flow field and scour at a pier

Despite the potential severity of scour at piers near abutments, this aspect of pier scour has received little attention. To quite varying extents, three studies investigated how pier and abutment proximity affect scour at an abutment and pier in simple rectangular channels.

Hong (2005) examined the influence of pier presence on bridge contraction scour and pier scour, and concluded that pier presence affects the location of deepest contraction scour in the vicinity of an abutment, but gave no relationship for estimating scour depth at a pier near an abutment. An earlier study by Croad (1989) investigated scour at the specific situation of a pier close to the spill-slope of a spill-through abutment. He found that the depth of scour at the pier essentially was determined by abutment scour, and proposed that pier scour be estimated as 0.9 times abutment scour depth.

Ettema et al. (NCHRP 24-20, 2009) conducted a set of extensive laboratory experiments on scour depth at piers near abutments. Figure 5-6 shows the abutment proximity layout examined. A significant parameter is L_p/y_f, where L_p = pier distance from the abutment toe, and y_f = flow depth at the abutment toe. For abutments on floodplains, y_f = flow depth on the floodplain. Figure 5-7 presents the findings as y_{Spier}/y_{S0pier} versus L_p/y_f, where y_{Spier} is the scour depth at the pier with the abutment present, $y_{Sabutment}$ is the scour at the abutment and y_{S0pier} is the scour depth at the pier without the abutment.

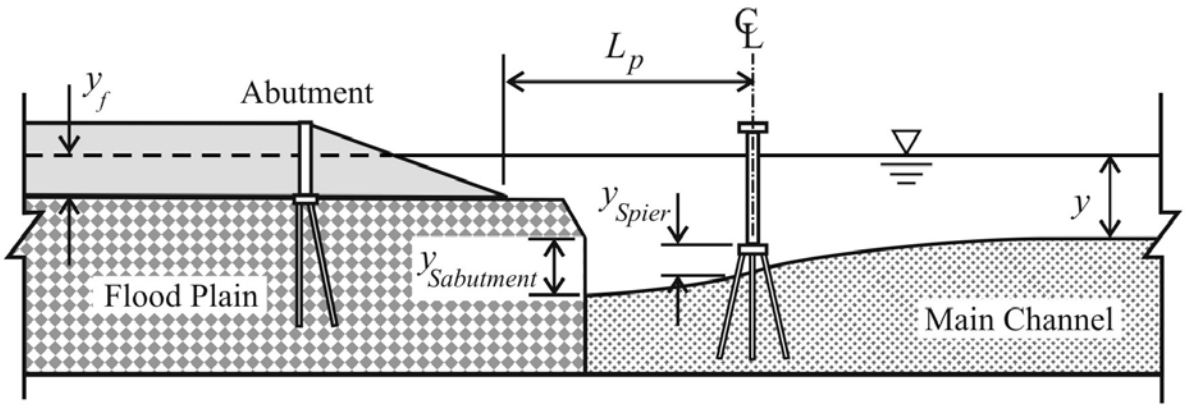


Figure 5-6 A pier close to the abutment is within the flow field generated by the abutment

The main findings ensue:

1. When a pier is located within the immediate vicinity of an abutment (notably, near the toe of an abutment), abutment scour dominates scour. For this situation, the existing equations for estimating pier scour-depth do not apply (the equations discussed in Chapter 6 and Appendix A). Scour depth at the pier is essentially equivalent to scour depth generated by flow around the abutment.

2. Figure 5-7 shows that initially y_{Spier}/y_{S0pier} does not vary with L_p/y_f, because the pier was enveloped by the abutment flow field. Pier presence did not influence abutment scour depth substantially; y_{Spier} coincided with the abutment scour depth, $y_{Sabutment}$. In other words, abutment proximity fully dominated pier scour development and its depth. As L_p/y_f further increased, the abutment's influence decreased and so did pier scour depth. Eventually, when L_p/y_f exceeded about 11 to 12, pier scour depth became equivalent to the local, pier-scour depth, as if no abutment were present; i.e., $y_{Spier}/y_{S0pier} = 1$.

3. A curve developed from the study's data could be used in estimating scour depth at a pier close to an abutment (Figure 5-7). When the pier is at the abutment toe, or near it, one curve limit indicates scour depth equivalent to abutment scour depth. When the pier is distant from an abutment, the other curve limit indicates scour depth is equivalent to scour at the pier in isolation.

4. The study's data show that pier presence does not substantially increase scour depth at an abutment. An increase in abutment scour depth of about 10% occurred when a pier was close to a wing-wall abutment. However, when a pier

was at the toe of an erodible spill-through abutment on a floodplain, pier presence reduced scour depth by about 10 to 20%.

5. Pier presence can influence abutment scour location, though not depth. When a pier is a short distance out from an abutment, a wider scour hole may develop than would have occurred at the abutment alone. This effect occurs because the additional scouring influence of the flow field at the pier acts to widen the abutment scour region. Pier presence, if close to the abutment, moves the location of maximum scour depth closer to the abutment's centerline axis.

6. Bathymetry measurements as scour progresses reveal that the location of maximum scour depth moves from an initial location at the upstream corner of an abutment to a location near the abutment's downstream corner. Pier presence, if close to the abutment, moved the location of maximum scour depth closer to the abutment's centerline axis.

7. For abutments on erodible floodplains pier presence gives a different trend than when the floodplain was fixed. When the pier was at the abutment toe, it was protected by embankment spill-slope soil and riprap, which failed and collected around the pier, and thereby preventing scour at the pier. However, when the pier was sufficiently distant from the abutment toe, so that failed spill-slope soil and riprap did not reach the pier, scour occurred at the pier. The maximum scour depth exceeded scour at an isolated pier (y_{S0pier}), but was considerably less than for the same pier position but with a fixed floodplain. Because the embankment eroded, and thus the depth and width of abutment scour was reduced, the reach of abutment influence on pier scour was less than when the floodplain was fixed.

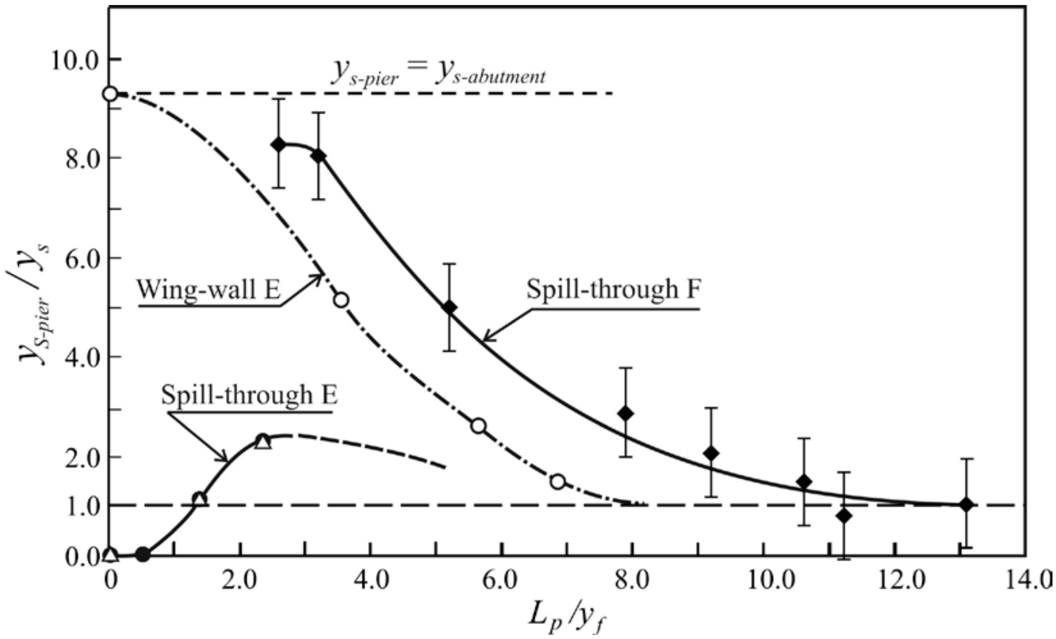


Figure 5-7 Normalized scour depth at pier relative to scour depth at a spill-through abutment, and wing-wall abutment, on an erosion resistant or fixed flood plain (F) or an erodible flood plain (E). The smallest value of L_p/y_f coincides with the toe of the spill-through abutment, at the edge of the fixed floodplain. The error bars indicate relative dune height.

5.4 Bridge-Deck Submergence

During major floods through some bridge waterways, the bridge deck may become submerged, as illustrated in Figure 5-8. This situation introduces an additional scour process that can erode the boundary at a pier site such that the net depth of scour can greatly exceed the depth associated to pier scour alone. Design estimation of scour depth should check whether deck submergence is likely to influence scour depth.

Figure 5-8 Bridge beams submerged by flood water

Scour associated with a submerged bridge superstructure is attributable to flow contraction under the bridge. The misnomer "pressure scour" is often used for this scour process. The water surface elevation upstream of a bridge is above the low chord of the bridge superstructure, causing a vertical contraction of the flow, thereby possibly eroding the boundary across the bridge waterway. Several studies have examined scour at submerged bridges, notably Richardson and Davis (1995), Arneson (1977), Arneson and Abt (1998), Umbrell et al. (1998), Lyn (2008), and Guo et al. (2009). The last reference provides the most comprehensive assessment and formulation of scour.

The study by Guo et al. (2009) articulates the current state of knowledge regarding this form of scour, which can be summarized as follows:

1. No study has yet closely examined pier scour for conditions when the bridge deck is submerged. Essentially, scour at a pier develops as if the pier were in a narrow region of contraction scour;
2. Three general cases of submerged-deck scour occur: Upstream beam submerged, no water over deck; all beams submerged, no water over deck; and, deck fully submerged. Figure 5-9 illustrates one of these cases;
3. The equilibrium scour profiles are approximately consistent across the main channel of the bridge waterway;
4. The maximum scour depth is located about 15% of the deck width downstream of the downstream edge of the deck;
5. Scour extends about one deck-width upstream of the deck, and a deposition mound forms about 2.5 deck widths downstream of the bridge;

6. As expected, the scour depth increases with greater difference in water levels either side of the bridge deck, and decreases with reduced erodibility of the boundary material;
7. Recent research (e.g., that by Guo et al. 2009) is leading to improved formulations of scour depth due to deck submergence, and likely will replace the current method in HEC-.

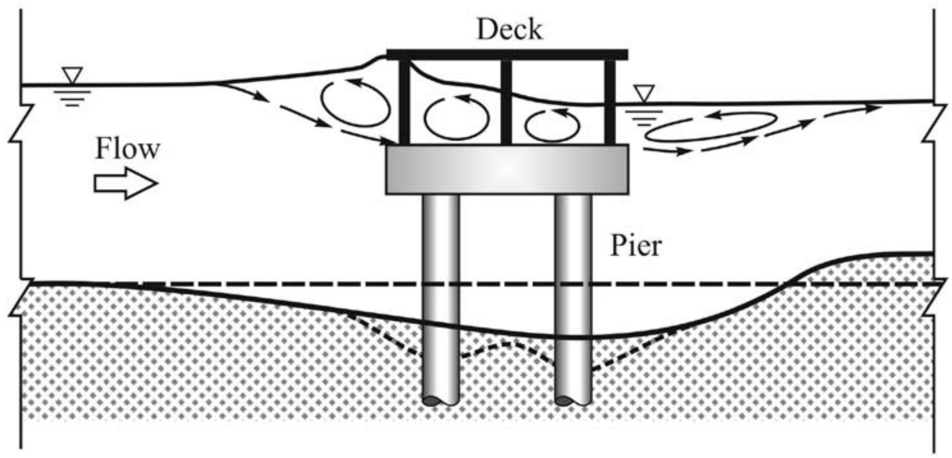


Figure 5-9 Scour at piers beneath a partially submerged bridge deck (adapted from Guo et al. 2009)

5.5. Woody Debris, or Ice, Accumulation

During flood and high-flow conditions, many rivers carry appreciable quantities of floating debris, especially tree branches and trunks. Rivers subject to frigid winter conditions also may convey large amounts of ice. Consequently, bridge piers in such rivers are prone to accumulate debris and ice, as illustrated in Figures 5-10a, b. The presence of large accumulations of debris or ice has been a significant factor in the failure of several bridges. An accumulation may amplify scour depth, and it may increase the streamwise load water exerts against a pier. Design estimation of scour depth at a pier site should check whether debris or ice considerations need to be taken into account.

Given the numerous variables likely introduced with debris or ice accumulation, and uncertainties associated with accumulation extents, it can be difficult to develop a single method with which to account for the effects of accumulations on scour depth at a pier.

An immediate question, for instance, is whether the accumulation will be limited to individual piers, or will it extend across several piers, possibly the entire bridge waterway. It is quite usual for debris to accumulate at individual piers (in the case of longer bridges, Figure 1.1) or across several piers (shorter bridges, Figure 1.2). For ice, however, it is more usual for ice to accumulate across the entire waterway (as in Figure 5-10b). In accordance with these accumulation extents, three design scenarios arise:

1. Accumulation occurs at a single pier, modifying the pier's effective width and form; and,

2. Accumulation occurs across several piers;

3. Accumulation extends across the bridge waterway, modifying the waterway form, backing-up flow, and thus possibly exacerbating contraction scour, as well as pier scour and abutment scour. This scenario, more typical for ice accumulation, is akin to scour at a submerged bridge (Section 5.3), because the channel flow is deflected under the debris in a manner similar to flow deflection under a bridge deck.

None of these design scenarios readily lend themselves to design estimation of scour depth. The first scenario has received some study. The second and third scenarios entail numerous uncertainties, and have not yet been treated.

(a)

(b)

Figure 5-10 Accumulation of woody debris (a), and ice rubble (b) at bridge waterways, affects flow locally at a pier and through the entire bridge waterway

When debris accumulates at a bridge pier, it may do so in masses normally referred to as debris rafts. The accumulated debris causes a larger obstruction to the flow than the pier without debris (Figure 5-11). Depending on the accumulation thickness and elevation at a pier, the additional flow obstruction may cause pier scour depths in excess of depths under conditions without debris accumulation.

Figure 5-11 Woody debris accumulation at a single pier (Lagasse et al. 2010)

The likelihood for debris accumulation at bridge foundations depends on a number of factors, including the availability of debris material, the potential for such material to be washed into streams and rivers, and the shape of the bridge foundations. In a study of woody debris transport in a Tennessee River, Diehl and Bryan (1993) found that the predominant large debris type comprised tree trunks with attached root masses. Such trees usually fall into a river because of bank erosion. Hence bank instability is an important catchment characteristic in identifying basins with a high potential for abundant production of debris. McClellan (1994) found, using small-scale laboratory models, that debris accumulations could be formed such that they extend from the water surface to the streambed in all flow conditions. Under low Froude number conditions, the debris rafts tended to be shallow and extensive in plan area, while under high Froude number conditions, the debris rafts tended to be deep and narrow. Substantial further advances were made by Diehl (1997), who developed guidelines for assessing debris yield from watersheds, and the extent of debris accumulation at piers. This effort was extended yet further by NCHRP Project 24-26 (Lagasse et al. 2010), with the intent of providing bridge designers improved guidelines for assessing debris accumulation extents.

The two leading studies on debris effects on pier scour are those by Lagasse et al. (2010) and Melville and Dongol (1992). The studies essentially agree in their findings as to how debris can affect pier scour.

1. Debris rafts can lead to deeper scour holes, with the extent of deepening varying with the shape, thickness, and elevation of the raft.

2. The greatest increases in scour depth were found when the debris formed a thick rectangular mass extending about one flow depth upstream of the pier (Lagasse et al. 2010). Triangular accumulations, thickest at the pier centreline, deflected flow

laterally as well as downwards, producing a shallower but wider scour hole than if no debris were present.

3. The main process whereby debris deepens scour is the deflection of flow downwards to the pier base. This process is akin to flow deflection at a submerged, or partially submerged, bridge deck (Section 5.3).

The approach used by Lagasse et al. (2010) and Melville and Dongol (1992) to quantify the influence of debris on scour depth has been to convert the accumulation into an equivalent pier width. The conversion uses the ratio of scour at a pier with debris to that a pier without debris. This approximation is useful when the debris accumulation is at a single pier. When debris extends across several piers, the conversion becomes more approximate. If the extent of debris accumulation, even at a series of single piers, causes a substantial backwater effect, the conversion may become overly approximate.

Lagasse et al. (2010) give an equation for estimating an equivalent pier width for use with the Richardson and Davis (2001) equation for estimating pier scour depth;

$$
\begin{aligned}
a_e &= \frac{K_{d1}\left(H_d W_d\right)\left(L_d / y\right)^{K_{d2}} + \left(y - K_{d1} H_d\right)a}{y} && \text{for } \mathrm{L_d/y} > 1.0 \\
a_e &= \frac{K_{d1}\left(H_d W_d\right) + \left(y - K_{d1} H_d\right)a}{y} && \text{for } \mathrm{L_d/y} \le 1.0
\end{aligned}
\qquad (5\text{-}3)
$$

where a = pier width (without debris) normal to the approach flow, H_d = thickness of debris, W_d = width of debris normal to the flow, y = depth of approach flow, L_d = length of debris upstream from the pier face, and K_{d1} and K_{d2} = coefficients depending on the shape of the debris accumulation. Values are given for triangular and rectangular debris masses. Figure 5-12 illustrates a rectangular debris accumulation, and indicates the variables involved in Eq. (5-3). In developing the equation, nine geometrically unique rectangular (in planform and profile) debris shapes and seven geometrically unique conical debris shapes (triangular in planform) were tested.

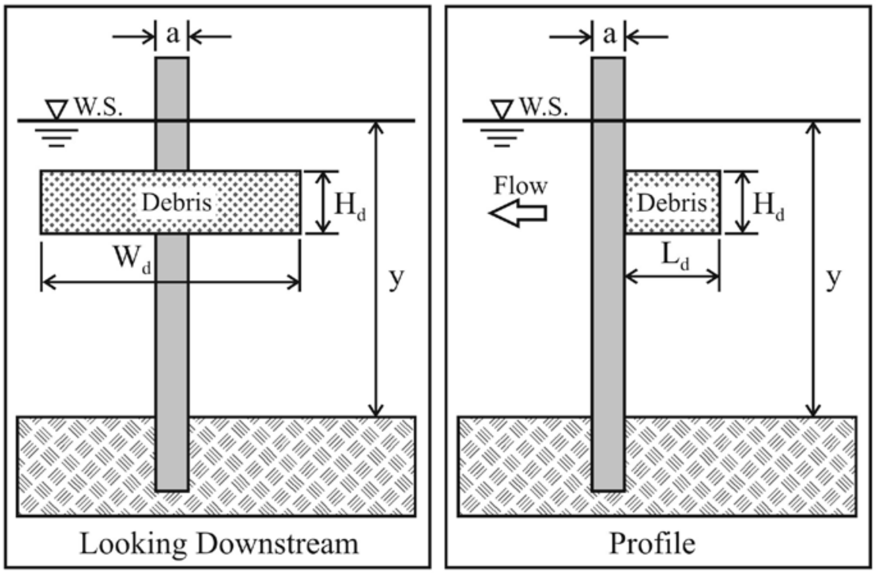


Figure 5-12 Rectangular accumulation of debris at a pier (Lagasse et al. 2010)

The equation was verified, using laboratory data, for use with the equation proposed by Richardson and Davis (2001). It has not been verified for use with any other scour depth equation. Such verification is the topic of further research (Chapter 9).

Melville and Dongol (1992) investigated local scour depths at circular bridge piers with debris rafts. The debris was modelled as an impervious circular cylinder, concentric to the pier and having its upper surface at the water surface level. They proposed the following expression for the equivalent size, a_e, of the uniform circular pier that induces about the same scour depth as the actual pier with accumulated debris:

$$a_e = \frac{0.52 T_d W_d + (y - 0.52 H_d) a}{y} \tag{5-4}$$

where H_d and W_d = thickness (vertical dimension) and width of the floating debris raft; and a = pier width, as shown in Figure 5-13. The equivalent width can be used to estimate local scour depth where debris is present, if the dimensions of the likely debris accumulation can be estimated. Figure 5-13 also shows trends in the data. For the study, T_d/b varied from 0.52 to 1.64, while W_d/a varied from 2.1 to 6.9. The maximum local scour depth recorded was 3.6a, representing a 50% increase over that at a uniform circular pier (y_s = 2.4a). The maximum scour depths occurred when the debris raft extended to about the undisturbed bed level, that is, $T_d \approx y$, as also reported by Lagasse et al. (2008). Derived from data for circular piers and debris masses, it is unclear how Eqs (5-3) and (5-4) apply to other pier shapes.

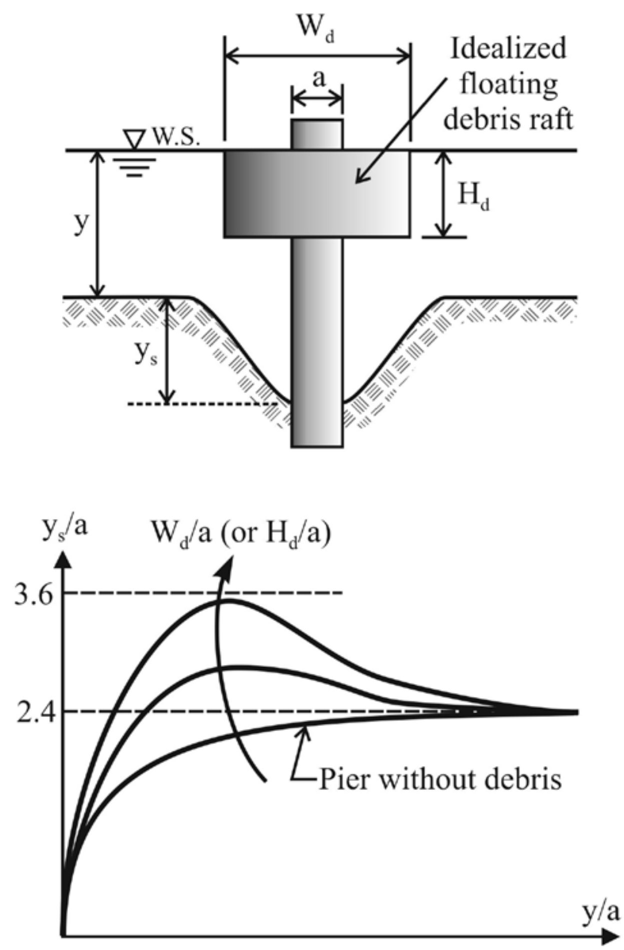


Figure 5-13 Local scour-depth variation with quantity of floating debris (Melville and Coleman 2000)

The literature on pier scour at bridge piers subject to ice accumulation is quite sparse. Only a handful of studies have been completed (e.g., Zabilansky 1996, Hains 2004, Hains et al. 2004, Zabilansky and White 2005). Commonly ice accumulates across the entire bridge water waterway, such that the approach flow depth rises, and pier scour is not significantly affected. If a thick accumulation of ice jam occurs at a bridge waterway, the flow and scour condition becomes similar to scour at a submerged bridge deck, as elaborated in sub-section 5.4.

5-6 Channel Morphology

Channel morphology may influence pier scour by locally affecting flow behaviour, sediment movement, and bathymetry at a pier site. NCHRP 24-27(03), *Evaluation of Bridge-Scour Research: Geomorphic Processes and Predictions*, a companion to the present project, reviews existing knowledge regarding morphologic processes related to channel stability at bridge sites. The evaluation outlines that, though the processes themselves are well understood, there can be considerable uncertainty as to how they impact pier scour. Some influences can be difficult to take into account, especially when they trigger unplanned changes in channel alignment and bathymetry.

The following aspects of channel morphology complicate reliable estimation of pier scour depth:

1. Long-term degradation and aggradation of the channel;
2. Lateral migration
3. Non-uniform cross-sectional shape of the approach channel;
4. Pier site proximity to a channel bend, a large bar, confluences of channels, or a local irregularity in channel bed or bank alignment;
5. Pier closeness to a channel bank;
6. The lateral distribution of flow velocity to a pier;
7. The effect of the cross-sectional shape of a compound channel on the flow intensity parameter V/V_c, at a pier; and,
8. The interaction of pier scour and contraction scour at a pier site.

Some aspects often are foreseeable, such as the lateral migration of the approach channel (or its thalweg), or the bathymetric features associated with a channel confluence. Other aspects may not be foreseeable at the time of pier design, especially inadvertent consequences of a subsequent engineering activity (e.g., a dam) that alters the stage-discharge relationship at a bridge site.

Common major concerns are channel bed degradation or aggradation, and bed lateral migration. These concerns can be addressed during pier design as long as the velocity and depth of approach flow used to estimate pier scour depth represent foreseen flow conditions during the design life of the pier. In this regard, hydraulic laboratory modeling or numerical modeling may be needed to assess flow conditions at a bridge site. Then, the following methods may be used to address the channel-morphology concerns:

1. Guide the approach flow so as to conform to required design conditions. This is achieved by means of channel-training works (e.g., hard points, spur dikes, guide-banks); and,
2. Monitor bridge waterway conditions to check for potentially adverse channel morphology effects. A wise old saying (Neil 1980) holds that "person who overlooks water under bridge will find bridge under water." The need for monitoring increases when erosion processes within bridge waterways (at abutments, piers, channel banks, shift in thalweg) are complicated by increasing interactions with each other, and may cause scour depth estimations to be of uncertain accuracy. Design relationships for scour at piers and abutments typically are derived from laboratory tests of piers and abutments simulated in simplified conditions that do not replicate the complexities of actual bridge sites. Therefore, as indeed recognized by most agencies responsible for bridges, there is on-going need to monitor bridges to ensure that their foundations and approach embankments are not imperilled by the various erosion that may occur.

One set of questions still to be addressed adequately concerns the combination of pier scour and some other scour forms, notably contraction scour, or scour at some other channel feature such as a confluence of approach flows (at a confluence of two channels, or behind an island or large bar). Figure 5-14 illustrates a case where pier scour occurred within a narrow region of contraction scour on a grassy flood plain. In some respects, this case is akin to Case III of pier scour in layered sediment (sub-section 5.7). There seem to be very few studies examining this aspect of pier scour. There are, however, several field-case photos of pier scour in situations where both pier scour and contraction scour evidently occurred (e.g., Figure 5-14).

Figure 5-14 Pier scour and abutment/contraction scour on flood plain. Channel geometry and vegetation substantially affect the approach flow to the pier

5.7. Layered Sediments

Many sedimentary deposits are heterogeneous and often distinct layers are present. If a more resistant layer underlies a readily erodible layer, scouring is inhibited and lesser scour depths may develop than predicted using the surface material characteristics. In some instances, coarse sediments cover deposits of finer material. Ettema (1980) (also, Breusers and Raudkivi, 1991, Melville and Coleman 2000) identified the following four cases of coarser material overlying finer sediment, as shown in Figure 5-15:

- Case 1, where the covering layer is thicker than the local scour depth, is the usual scour problem (in the coarser sediment).
- Case 2, the local scour penetrates the covering layer and induces a disintegration of the layer in both the upstream and the downstream directions for a considerable distance. The end conditions are those of local scour in the underlying sediment due to the new hydraulic conditions. The total (local) scour depth is $y_s + (y_f - y_c)$, where y_c and y_f = uniform flow depth over a flat bed of particle sizes equivalent

to the upstream coarse surface particles and the underlying surface fine particles, respectively. Ettema (1980) determined that the total local scour depth for Case 2 was always less than that for Cases 3 and 4.

- Case 3, the covering layer disintegrates in the downstream direction only, leaving a step just upstream of the pier. The local pier scour develops at the bottom of this step. This case was found to give the deepest scour (Ettema 1980). Breusers and Raudkivi (1991) give the following expression for *H* (Figure 5-15):

  $$H = 2.6\,(y_f - y_c) \tag{5-5}$$

  An expression for the relation between y_c and y_f is given also (Breusers and Raudkivi 1991). The total local scour is the sum of *H* and the local scour depth, y_s, in the finer sediment, as for Case 2.
- Case 4, the covering layer disintegrates only over a small area downstream of the pier but remains intact at both sides of the pier and upstream of it. For this case, the scour depth was about 3*a*, the increase from that at a pier in uniform sediment being mainly due to reduced downstream support of the scour hole.

Bed-material layers vary locally in extent and details, such that scour-depth estimation should take into account site extents of sediment layering, and consider the merits of placing a pier in a location least at risk from the consequences of layering. This complication remains a topic of active research (Raikar and Dey 2009), and relates closely to the design of riprap protection of piers (e.g., Mashahir et al. 2010, Lagasse et al. 2007).

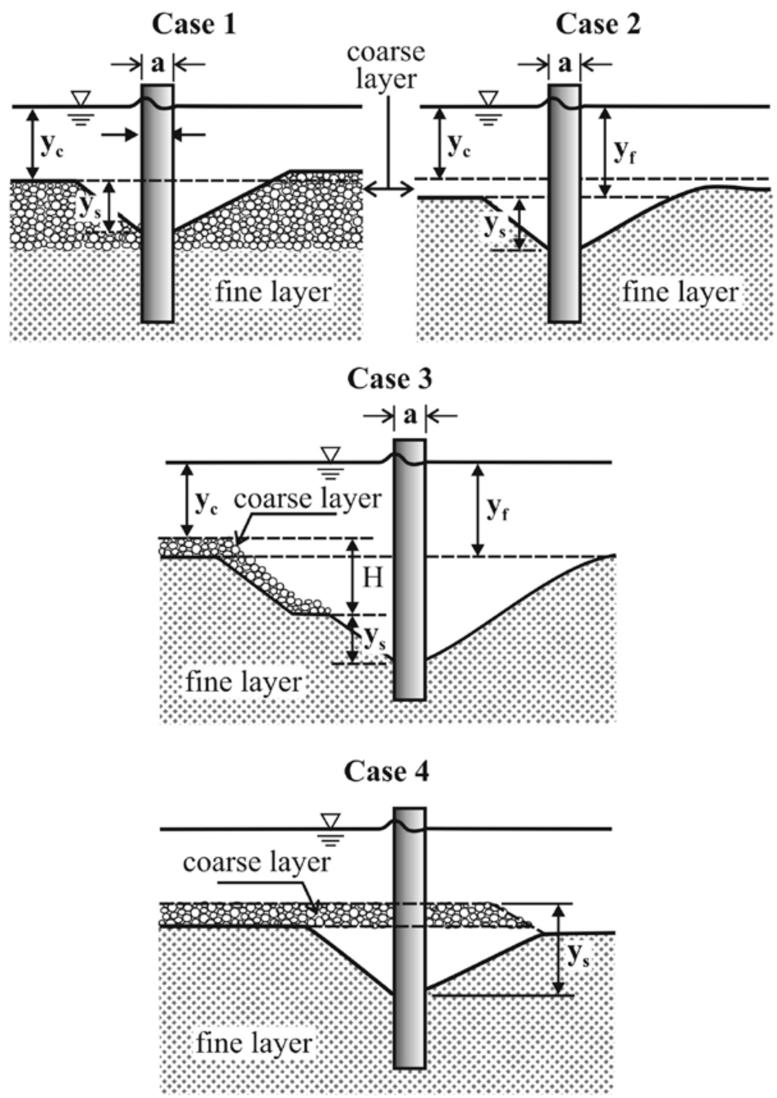


Figure 5-15 Local scour in layered sediments (Breusers and Raudkivi, 1991); y_c = depth to top of coarse layer, y_s = depth to top of fine layer

5.8 Scour of Cohesive Soil

Since 1990, insight regarding pier scour in cohesive soils or clay has advanced substantially. NCHRP 24-15 (Briaud et al. 2004a; also, Briaud et al. 2004b), in particular, provides comprehensive insights. Other notable studies are reported by Hosni (1995), Molinas and Abdeldayem (1998), Ansari et al. (2003), Debnath and Chauhuri (2010). The principal findings of these studies can be summarized as follows:

1. Increased clay content in a predominantly sand bed reduces scour depth, for constant approach flow conditions, because the addition of cohesive strength increases sediment resistance to erosion. The extents of scour reduction vary with clay mineralogy (Molinas and Abdeldayem (1998), Ansari et al. (2003), Debnath

and Chaudhuri (2010). As the clay content increased, the location of deepest scour shifted from the pier face to the pier flanks (Anasari et al. 2003);

2. Observations (Ansari et al. 2003, Briaud et al. 2004a) indicate that clay mostly erodes in clusters or lumps of material. The size of the lumps varies with clay properties;
3. The geometry, location and extent of scour at piers differ substantially in clay from those in cohesionless material. The scour holes reported by Ansari et al. (2003) show deepest scour at the pier flanks, and are similar in appearance as the scour hole depicted in Figure 3-2b (from Briaud et al 2004). Most laboratory scour holes had minimal scour at the pier face;
4. The scour holes observed during laboratory tests were no deeper than developed by scour in a sand bed;
5. As clay erodes from the scour hole it does not deposit as a scour mound immediately downstream of the scour hole; and,
6. Scour typically develops much slower in clay materials than in non-cohesive materials. Briaud et al. (2004) indicate that scour rates can be of the order of 1,000 times slower in clay. The relatively slow rate of scour development may mean that flood durations may be too short to generate significant local scour depths.

Briaud et al. (2004a) and Ansari et al. (2003) usefully discuss the clay property variables affecting clay erodibility. They include soil unit weight, mineralogy, moisture content, and a set of physio-chemical factors affecting the levels of electromagnetic and electrostatic inter-particle forces within clay. For differing combinations of these properties, clay may erode at different rates, and in somewhat different manners. Additionally, non-uniformities and cracks in clay can affect clay erodibility. Turbulence structures in the pier flow field evidently exert oscillatory stresses on the clay material, and through a process of fatigue dislodge clay lumps from the scour region. The details whereby the pier flow field interacts with the clay boundary, and scours it, have not been studied.

A difficulty with estimating scour depth in clay, and mixtures of clay and non-cohesive material, is determining the erosion resistance or critical shear stress (or velocity) for clay. To address this difficulty, Briaud et al. (2004a, b), and others, use a device termed an Erosion Function Apparatus to ascertain the erodibility of a 75mm-diameter surface of clay and other material. They use this device to estimate rates of scour at model piers.

5.9 Scour of Weak Rock

It has been assumed for design estimation of pier scour (e.g., Richardson and Davis 2001) that a rock foundation beneath a pier is not readily subject to scour. This assumption was exposed to question by the failure in 1996 of a pier at Schoharie Creek Bridge (Lagasse et al. 1988, Richardson and Davis 1988). The rock beneath a pier footing was sufficiently weak that flow eroded rock fragments, leading to the pier's collapse. NCHRP 24-29 *Scour at Bridge Foundations on Rock* presently is underway to assess scour of weak rock foundations (Keaton et al. 2009). A few individual studies of pier scour in rock have been conducted. Hopkins and Beckham (1999) for example, investigated scour of rock beneath piers at several hundred bridges in Kentucky.

In general, observations regarding pier scour of rock can be summarized as follow:

1. Scour of weak rock is a possible concern for piers on slab footings;
2. The scour depths are considerably less than observed at pier scour of a sand foundation material. Hopkins and Beckham (1999) found that, when pier scour occurred, it was less than about 0.25m deep;
3. Rock material commonly fails by virtue of fatigue associated with oscillatory loads imposed by turbulence structures in the pier flow field. Physical and chemical and weathering of joints within rock helps facilitate scour of rock;
4. The concern for scour of weak rock may increase if the rock weakens when exposed to freeze-thaw cycles, such as can occur for footings exposed to frigid air during low flow periods during winter conditions; and,
5. Scour holes in rock are irregular in form, in accordance with patterns of failure of the rock.

5.10 Suspended Sediment (Silt and Clay) in Flow

The presence of suspended clay and silt in the flow toward a pier in a planar bed is reported to influence scour depth (Sheppard et al. 2002, Clunie 2002). However, the mechanism for this influence is unclear. Two possible mechanisms need to be examined:

1. The presence of suspended sediment might alter the flow field; and/or,
2. Deposition of suspended sediment might increase bed resistance to erosion.

To date, insufficient study has confirmed the veracity of these mechanisms. Studies imply that both mechanisms could be at play under certain conditions of flow and suspended-sediment concentration.

Most studies examining these mechanisms involve flow and bed sediment movement on planar beds. Fortier and Scobey (1926) conducted an early empirical study, and found that the velocities for bed erosion inception were significantly higher in turbid water compared to clear water, which points toward a reduction of the bed shear stress with increased turbidity. They suggested that drag reduction should occur because a certain part of the available flow energy will be spent to transport the suspended sediment particles. As suspended sediment dissipates energy from the flow turbulence, more energy is needed to erode bed sediment.

To be noted is that the influence reduces scour depth (Sheppard et al. 2002, Clunie 2002). Therefore, this influence is of little practical interest for the purpose of scour depth estimation, though may aid interpretation of field data on scour depth.

5.10.1 Flow Field Modification

Suspended sediment may alter the flow field by stratifying flow density within the approach flow that modifies the vertical profile of approach-flow velocity and flow-field turbulence. Larger concentrations of sediment near the bed may cause flow density stratification near the bed, if the concentration of sediment is sufficiently great.

Modification of approach-flow distribution and turbulence structure by stratification would affect scour, though suitably controlled studies have yet to be done to yield a quantitative relationship of general use. The experiments reported by Sheppard et al. (2002) using a large-scale, field flume suggest that the presence of suspended sediment in the near bed region reduces scour depth. Other experiments, in a different context, conducted by Best and Leeder (2006) with a seawater/clay suspension over a mud bed show reduced near bed velocities with the increase in the clay concentration. The authors attributed this to the modification of the turbulence structure (e.g., a reduction in the rate of turbulence burst events which reduces the momentum exchange within the boundary layer) in the near bed region due to the increased concentration of sediment. The presence of suspended sediment particles in the near bed region and in the scour hole may globally damp the turbulence and decrease shear velocity at the bed. This mechanism was suggested by Li and Gust (2000).

Reduced vertical gradient of approach velocity, and thus magnitude of shear velocity, could reduce scour at a pier. The magnitude of the downflow velocity at the pier face (Figure 3-4) varies in accordance with approach-flow gradient as well as overall magnitude.

The complexity of the experimental setup and procedures needed to investigate suspended sediment influence on pier flow field, and much more simply for a straight channel flow, has resulted in the lack of research and data to investigate this influence. Some studies do exist, though. For these few studies, the range of suspended sediment concentration was quite limited, and more data are needed to confirm the trends obtained, as well as identify the range over which these effects can be significant and could affect scour depth.

The recent Direct Numerical Simulation study by Cantero et al. (2008) indicates that the presence of self-stratified suspended sediment in a channel flow can substantially decrease the bed shear stress, and thereby diminish the erosion capacity of the flow. Though their study was performed with a no-slip boundary condition at the top surface (closed channel) their findings are directly applicable to the open channel flow. Their simulations show that the mean velocity profile loses its symmetry for sediment having dimensionless settling velocities larger than $1.5\text{x}10^{-2}$, and the position of the maximum moves toward the bottom wall. Meanwhile the velocity profiles near the bottom wall were found to deviate from the sharper turbulence profile to a rounder laminar-like profile. This result was attributed to thickening of the bottom wall layer including the viscous sub-layer. The finding is consistent with that of Li and Gust (2000), who related decrease in the bed shear velocity with increase in suspended sediment concentration.

The study by Cantero et al. (2008) also showed that, for dimensionless settling velocities larger than $2\text{x}10^{-2}$ (the ratio between the concentrations at the bottom and top boundaries was close to 10), the bed shear stress decreased more than 50% compared to when no suspended sediment was present. The mixing induced by wall turbulence is inhibited by the flow stratification in the lower part of the channel where the concentration levels are relatively large. A Rouse concentration profile develops. When the settling velocity increases, the gradient of concentration at the bed also increases. Also relevant for dimensionless settling velocities larger than $2x10^{-2}$, the logarithmic region disappeared at the bottom wall while it was still present at the top wall. One should mention the channel Reynolds number was small so for larger channel Reynolds numbers it is expected that a modified log-law region will still be present for a relatively large stratification and associated large sediment concentration gradients in the lower part of the channel. For smaller settling velocities and stratification, a log-law region can still be identified at the bottom wall but the value of the von Karman constant has to be decreased. Thus, a kinematic effect due to the presence of suspended sediment in the flow is present. This effect modifies the mean velocity profile over the depth and decreases the bed shear stress.

For a given parameter value, the ratio between the bed friction velocity and its critical value decreases, which means that the local scour depth should decrease. This effect should be relatively easy to account for in classical local scour methods if the quantitative relationship between the mean concentration of suspended sediment and the decrease in the non-dimensional bed friction velocity in the incoming channel is known.

The clear-water scour experiments conducted by Sheppard et al. (2004) in a flume in the Conte USGS-BRD laboratory in Turner Falls, Massachusetts resulted in significant differences in the local scour depths as the concentration of the suspended sediment varied in the water supply. Pier diameter varied between 0.11m and 0.91m, and the mean bed sediment size varied between 0.22mm and 2.9mm. As the suspended sediment concentration of the water supply (reservoir of a power plant off the Connecticut River) could not be controlled, scour-depth reduction as a function of the suspended sediment concentration was not quantified. When the water turbidity was higher, lower equilibrium scour depths resulted. The differences were significant; high turbidity diminished the equilibrium scour depth by more than 50% compared to low turbidity conditions. Video recordings of these two experiments conducted with low and high turbidity showed no apparent major difference in flow conditions. The only exception was a reduction of the sediment movement in the bed in the high turbidity experiment. The tests in which the effect of the suspended sediment was the largest were those with a mean sediment size of 0.22mm. Only one test with a bed sediment diameter of 0.8mm was significantly affected by the presence of suspended fine sediment. These tests were conducted over a large range of flow and pier sizes. As the measured concentration of fine sediment in the water column were less than 0.1g/l, the experiments show that relatively small concentrations of sediment are needed to significantly affect the equilibrium scour depth.

5.10.2 Bed Erosion Resistance

Deposition of suspended sediment amidst the bed particles can increase bed resistance to erosion (Garcia, 2008). This effect would occur in the scour hole, as well as on the approach bed.

As the increase in water turbidity was gradual, Sheppard et al. (2002) and Smyre (2002) concluded the suspended sediment must have a gradual influence on the scour hole development. The influence should be larger in the later stages of the scour process, when bed particles in the scour hole are closer to an incipient motion condition.

Also, the presence of deposited suspended sediment amidst bed particles reduces the angle of repose of the bed particles in the scour hole. This effect causes bed particles to slide down the slope of the scour hole more readily than normal, and results in a wider scour hole. In the initial stages of the scour, the development of the scour hole is expected to be similar to the case when suspended sediment is not present, because the erosive forces greatly exceed the erosion resistance of the bed at the pier. As scour progresses, the bed particles become more resistant to entrainment, because the interstices between them infill with fine sediment. The net effect is a shallower, wider scour hole. The observations by Sheppard et al. (2002) confirm that these mechanisms occurred for their experiments.

Experiments conducted with live-bed conditions at the University of Auckland offer mixed results regarding the influence of the suspended sediment concentration on the equilibrium scour depth. Two live-bed tests indicate an increase in the scour depth as a result of the presence of suspended sediment in the incoming flow. Meanwhile, clear-water tests confirm that the equilibrium scour depth decreases with the increase in the suspended sediment concentration.

5.10.3 Conclusion

The flow and bed-resistance mechanisms stated at the beginning of this sub-section indeed can be at play and affect scour depth. The encouraging conclusion, though, is that washload does not deepen scour compared to scour development in clear-water conditions.

CHAPTER 6
LEADING METHODS FOR SCOUR-DEPTH PREDICTION

6.1 Introduction

This chapter is a review of the existing leading methods for design estimation of pier-scour depth. The method best reflecting current understanding of scour processes is identified. An important consideration in reviewing the methods is their possible inclusion of the primary parameters defining the potential maximum depth resulting from scour processes.

Based on the evaluation presented in Appendix A, the leading scour-depth prediction methods are Melville (1997, also in Melville and Coleman 2000), Richardson and Davis (2001), Sheppard and Miller (2006), and Sheppard-Melville (NCHRP 24-32). These methods most comprehensively reflect scour processes. The Melville (1997) method differs from the other two insofar that it is directly intended for design estimation of maximum scour depth, doing so by means of envelope curves encompassing data on parameter influences. The other two methods aim to replicate data trends for specific parameters. Each method is based mainly on scour data for cylindrical piers, and to a lesser extent on data from common pier forms consisting of multiple components (as illustrated in Figures 2-1 and 2-2).

The Richardson and Davis (2001) method presently is used in FHWA's HEC-18. It comprises a series of adaptations of what is colloquially known as the CSU equation (e.g., an earlier version of the method is given by Richardson and Davis 1995), which HEC-18 has used for the past several decades. The method provides scour-depth estimates that, considered in terms of the low rate of pier failure attributable to scour, has served bridge designers quite well for common pier sizes. However, concerns exist that it provides unreasonably large values of scour depth for piers in the wide-pier category, and its upper-bound estimate for live-bed scour is too high. Also, research since 1990 shows the method inadequately reflects certain aspects of pier-scour processes.

The Melville (1997) method is used quite extensively in several countries, and provides the most comprehensive coverage of parameters influencing scour (as demonstrated by Chapter 4). The following two aspect of the Melville method relate directly to the key considerations stated in Section 1.4:

1. The method is organized in accordance with changes in the pier flow field (as indicated in Sections 3.1 and 4.2 for the parameter y/a). Design relationships differ in accordance as to whether the pier is narrow, transitional, or wide; and,

2. The method provides envelope curves for design use, rather than curves fitted through data.

However, the Melville (1997) method is in the process of being merged with the method more recently developed by Sheppard and Miller (2006). NCHRP Project 24-32 has moved forward with developing the Sheppard-Melville method (Sheppard et al. 2011).

Therefore, the Melville method per se is not considered further in this section. The method proposed by Sheppard and Miller (2006) was developed in response to short-comings perceived in the Richardson et al. (2001) method recommended in HEC-18. The Sheppard-Melville method builds on the insights contained in Melville (1997) and Sheppard (2006). It uses the two primary parameters *y/a* and *a/D*, includes the parameter expressing influences of pier shape relative to flow alignment, and includes the flow intensity parameter, V/V_c. As indicated in the draft final report for NCHRP Project 24-32, and explained subsequently in Section 6.4, further research is needed to complete the method's development with regard to scour in the wide-pier category ($y/a^* < 0.2$). For the purpose of design estimation of potential maximum scour depths, however, a simplified version of the method can be established, as explained subsequently in Section 6.4.

The present evaluation indicates the need to transition from the method currently used in HEC-18 (Richardson and Davis 2001), to the Sheppard-Melville method elaborated in NCHRP 24-32 (Sheppard et al. 2011). The latter (and more recently developed) method better reflects current understanding of pier-scour processes. However, the Sheppard-Melville method requires some additional adjustment to reflect parameter influences. Field verification of the Sheppard-Melville method is a priority for further research.

6. 2 Richardson and Davis (2001) Method

The Richardson et al. (2001) for estimating the depth of local scour at piers, colloquially called the CSU equation, extends back about 35 years, and has been updated several times to account for additional parameter influences (Richardson and Davis 1975, 1993, 2001). It is based on the equation

$$\frac{y_s}{a} = 2.0K_1K_2K_3K_4K_W\left(\frac{y}{a}\right)^{0.35}Fr^{0.43} \tag{6.1}$$

where $Fr = V/(gy)^{0.5}$; and K_1, K_2 and K_3 = adjustment factors accounting for pier nose shape, angle of attack of flow, and state of bed-sediment motion, respectively. K_1 varies between 0.9 and 1.1, K_2 varies between 1.0 and 5.0, and K_3 (essentially the Laursen and Toch 1960 curves), varies between 1.1 and 1.3.

The factor, K_4, is

$$K_4 = 0.4V_R^{0.15} \tag{6.2}$$

where

$$V_R = \frac{V - V_{icd_{50}}}{V_{cd_{50}} - V_{icd_{90}}} > 0 \tag{6.3}$$

V_{icd_x} is approach velocity required to initiate scour at the pier for grain size D_x, given by:

$$V_{icd_x} = 0.645\left(\frac{D_x}{a}\right)^{0.053} V_{cd_x} \tag{6.4}$$

and V_{cd_x} is critical velocity for incipient motion for grain size D_x, given by:

$$V_{cd_x} = 6.19 y^{1/6} D_x^{1/3} \tag{6.5}$$

K_4 decreases scour depths for armouring of the scour hole for bed sediments for which $D \geq 2$mm and $D_{90} \geq 20$mm; for $D < 2$mm or $D_{90} < 20$mm, $K_4 = 1$; D_{90} = the particle diameter for which 90% of particles are finer. The minimum value of K_4 is 0.4.

The method's use in HEC-18 includes a factor, K_w, for very wide piers. This correction factor is to be applied when $y/a < 0.8$, $a/D_{50} > 50$ and $Fr < 1$. K_w is given as

$$K_w = 2.58\left(\frac{y}{a}\right)^{0.34} Fr^{0.65} \qquad \text{for } V/V_c < 1 \tag{6.6a}$$

$$K_w = 1.0\left(\frac{y}{a}\right)^{0.13} Fr^{0.25} \qquad \text{for } V/V_c \geq 1 \tag{6.6b}$$

The shape factor K_1 applies to simple pier shapes. A procedure is included to account for the shape of complex piers. The local scour at a complex pier is estimated as the sum of the scour due to the proportion of the pier stem in the flow, the scour due to the pile cap or footing in the flow, and the scour due the piles exposed to the flow. A rather complicated set of equations and graphical relationships is given for estimation of each scour component. The rationale for this assumption is questionable, because it does not adequately relate pier structure to pier flow field and erosion processes.

For piers comprising multiple columns skewed to the flow, Richardson and Davis (2001) recommend using the composite pier projected width in Eq. (6.1), with $K_1 = K_2 = 1$. For multiple columns spaced more than five diameters apart, it is further recommended that the maximum scour depth be limited to 1.2 times that of a single column.

In accordance with Froude Number value, the Richardson and Davis (2001) equation has the following upper bounds for round-nose pier aligned with the flow:

$$\begin{aligned} \frac{y_s}{a} &= 2.4 \qquad Fr \leq 0.8 \\ \frac{y_s}{a} &= 3.0 \qquad Fr > 0.8 \end{aligned} \tag{6.7}$$

The higher value of y_s/a for live-bed scour is open to question, as it has not been corroborated by data other than the few laboratory data on which it is based.

6.3 The Sheppard-Melville Method (NCHRP Project 24-32)

The Sheppard-Melville method builds on the method proposed by Sheppard and Miller (2006), following more-or-less the same parameter approach inherent in the Melville (1997) method. The method, described in Sheppard et al. (2011), uses an effective pier diameter, a^*, the diameter of a circular pile that will experience the same equilibrium scour depth as the subject structure under the same flow and sediment conditions. In other words, pier shape and alignment factors are used to determine a^*, which then is used in the method's equation set.

The Sheppard-Melville method comprises Eqs. (6.8) to (6.12), applied to ranges of bed material mobility. For clear-water scour ($0.4 < V/V_c < 1$),

$$\frac{y_s}{a^*} = 2.5 f_1\left(\frac{y}{a^*}\right) f_2\left(\frac{V}{V_c}\right) f_3\left(\frac{a^*}{D_{50}}\right) \tag{6.8}$$

In the live-bed scour range up to the live-bed peak ($1 < V/V_c < V_{lp}/V_c$)

$$\frac{y_s}{a^*} = f_1\left(\frac{y}{a^*}\right)\left[2.2\left(\frac{\frac{V}{V_c}-1}{\frac{V_{lp}}{V_c}-1}\right) + 2.5\left(\frac{\frac{V_{lp}}{V_c}-\frac{V}{V_c}}{\frac{V_{lp}}{V_c}-1}\right) f_3\left(\frac{a^*}{D_{50}}\right)\right] \tag{6.9}$$

and in the live-bed scour range above the live-bed peak ($V/V_c > V_{lp}/V_c$)

$$\frac{y_s}{a^*} = 2.2 f_1\left(\frac{y}{a^*}\right) \tag{6.10}$$

where:

$$f_1 = \tanh\left[\left(\frac{y}{a^*}\right)^{0.4}\right]$$

$$f_2 = \left\{1 - 1.2\left[\ln\left(\frac{V}{V_c}\right)\right]^2\right\} \tag{6.11}$$

$$f_3 = \left[\frac{\left(\frac{a^*}{D}\right)}{0.4\left(\frac{a^*}{D}\right)^{1.2} + 10.6\left(\frac{a^*}{D}\right)^{-0.13}}\right]$$

and V_{lp} is the live-bed peak velocity, much the same as V_a in the Melville (1997) method. The sediment critical velocity, V_c, is calculated using Shield's curve. The live-bed peak velocity, V_{lp}, is computed using a modification of van Rijn's (1993) prediction of the conditions under which the bed planes out. Hence V_{lp} is computed as the larger of:

$$V_{lp} = 0.6\sqrt{gy} \tag{6.12a}$$

or

$$V_{lp} = 5V_c \tag{6.12b}$$

The method reflects well-known data trends (Figures 4-14 and 6-1) indicating that there are two local maximums in the scour depth versus V/V_c plots. The first local maximum occurs at transition from clear-water to live-bed scour conditions, i.e. at $V/V_c = 1$. The second maximum, referred to here as the "live-bed peak" is thought to occur at the conditions where the bed planes out. The velocity that produces the live-bed scour peak is referred to as the live-bed peak velocity and is denoted by, V_{lp}.

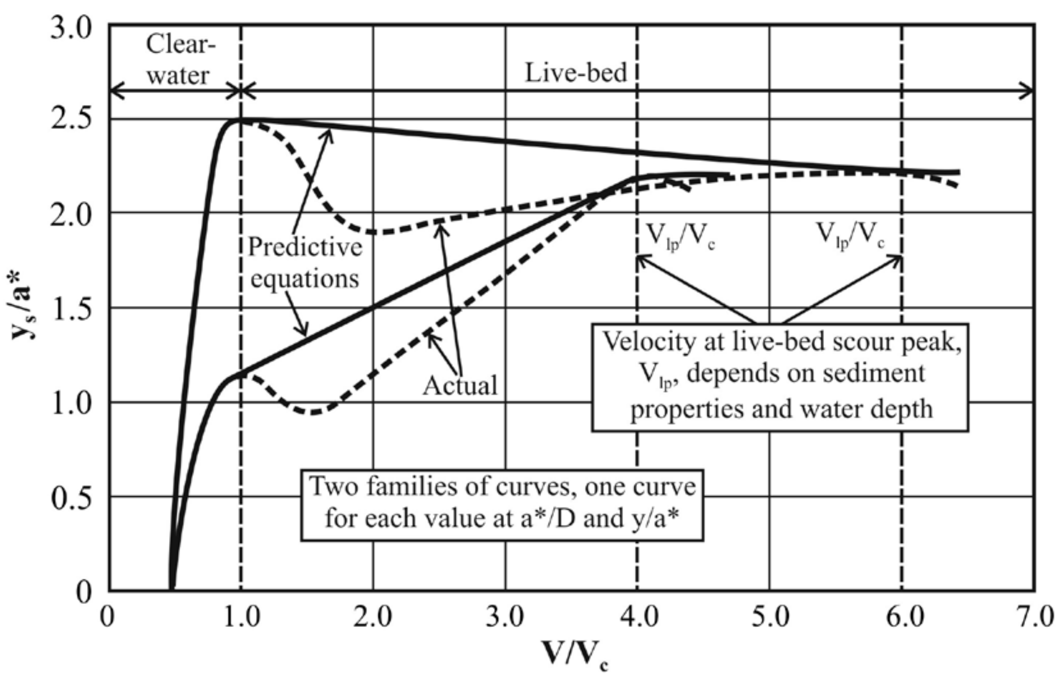


Figure 6-1 Normalized equilibrium scour depth, y_s/a^*, versus flow intensity ratio, V/V_c, for constant values of y/a^* and a^*/D (Sheppard et al, 2011)

6.4 Discussion

For the ranges of laboratory-based, parameter values on which they are based, the Richardson and Davis (2001) and Sheppard-Melville (Sheppard et al. 2011) methods are acceptably accurate, and give reasonably comparable estimates (Appendix A). They were developed for essentially cylindrical piers, and empirically relate measured scour depths to selected parameters. However, each method is not entirely reconciled with aspects of the flow field and erosion processes at piers; the Sheppard-Melville method is better reconciled than the Richardson and Davis method.

In accordance with the third objective set for this project (Section 1.4), three criteria must be considered when further evaluating the two methods:

1. Reflection of proven relationships between parameters and scour processes;

2. Capacity to encompass recently identified parameter influences; and,

3. Reasonable expectation that the methods, semi-empirical equations generally, give acceptably accurate estimates of scour depth.

The ensuing sub-sections address these questions with respect to the methods developed by Richardson and Davis (2001) and Sheppard et al. (2011). An important consideration is that an effective design method need not reflect all parameter influences, but rather ensure that it has the capacity to the estimate potential maximum scour depth for a pier site. This consideration entails that the method use the primary parameters identified in Chapter 4.

6.4.1 Reflection of Proven Parameter Relationships

The parameters of prime importance for estimating potential maximum depth of pier scour are y/a and a/D, because these parameters delineate the maximum potential dimensions and shape of the scour hole for given a pier site. Pier shape and alignment factors qualify pier width a, which then could be expressed as an effective width, a^*. The parameter V/V_c is important for ascertaining whether the approach flow has the capacity to attain the potential maximum scour depth. However, as Section, 5.3 states, it is not necessary to know V/V_c in order to estimate that scour depth.

Neither the Richardson and Davis (2001) method nor the Sheppard-Melville method is configured in accordance with the three categories of pier width indicated in Table 4-1 in terms of y/a. The narrow- and wide-pier categories entail two substantially different flow fields and dominant scour processes, with the transition-pier category evolving between the two flow fields. Though both methods recognize that a difference exists between narrow- and wide-pier flow fields, the methods do not expressly relate to the different flow fields. This deficiency is a weakness in the scour-estimation approach taken by both methods.

The Sheppard-Melville method is configured in terms of two ranges of V/V_c, clear-water and live-bed. It expressly includes the three parameters, though presently is unclear as to how best to include them, for live-bed scour. The Richardson and Davis method expressly includes y/a, but not a/D and V/V_c. It instead empirically links state of bed motion through the combined influences of K_4 and Fr. As explained in sub-section 6.4.2, this linkage is not readily extended. Though both methods include y/a, they do not deal satisfactorily with very wide piers, particularly in live-bed scour; there is a paucity of data for this wide-pier condition.

The Richardson and Davis method, though well-correlated empirically to the available data upon which it is based, has the following limitations:

1. Scour is estimated as proportional to $Fr^{0.43}$. Since Froude numbers (Fr) are typically higher in streams with coarser beds, estimated local scour depths tend to be larger in coarser materials; for example, an increase in Fr from 0.3 to 0.8 leads to about a 50% increase in scour depth. This is inconsistent with laboratory data. The Froude number dependence expressed in Eq. (6.1) may be valid when comparing results for the same bed materials, but not when comparing different bed materials. As argued in the parameter framework, and pointed out by others (e.g. Neill 1993), Fr would seem not to have prominent physical significance in scour processes, other than indicating sub- or super-critical flow condition through the bridge waterway.

2. The parameter a/D is missing from Eq. (6.1). Consequently, this equation does not expressly reflect several significant affects of bed particle size (or boundary material erodibility), including the parameter's influence on flow-field vorticity, an important aspect driving scour.

3. It is unclear why the maximum value of y_s/a would increase from 2.4 when $Fr \leq 0.8$, to 3.0 when $Fr > 0.8$. There is no sustained body of laboratory or field data indicating that y_s/a reaches 3.0 for circular cylindrical piers, at least not without the influence of a scale effect such as attributable to inaccurate scaling of turbulence structures (as discussed in sub-section 4.4.8).

4. The veracities of factors K_4 and K_W in Eq. (6.1) are uncertain. They pertain to bed sediment non-uniformity and wide pier effects, respectively.

The Sheppard-Melville method (Sheppard et al. 2011) reflects some of the recent understandings of the influences exerted by the parameter a/D. It reflects several aspects of pier scour, as explained in Section 4.4.2: i.e., scaling of vorticity generated by flow around a pier (and around components of a pier (column, pile cap, pile), relative roughness of particle size relative to pier diameter. Lee and Sturm's (2008) data show that the peak of y_s/a occurs at a/D~25 and is followed by a sharp decay of y_s/a until a/D~100. The decay rate decreases for $a/D > 200$, such that y_s/a becomes approximately constant for $a/D > 500$. This trend implies that the sediment coarseness effect should decay with increase in the ratio a/D for fine sediments. Data from Sheppard (2007) for wide piers also show that y_s/a reduces with increasing a/D for fine sediments. However, Sheppard proposes a monotonic decay of y_s/a with a/D after the maximum scour depth is reached. To be kept in mind is the fact that these trends are indicated for a simple pier form, and include a mix of ripple-forming sands and coarser particle sizes. The trends become more complicated for pier designs involving a pier column, pile cap, and piles.

The Richardson and Davis method is based on (what is known as) the CSU equation, progressively refined over the years to incorporate new research findings, but the basis of the method has remained essentially the same. According to Richardson et al (1990), the CSU[6] equation was determined from a plot of laboratory data for circular cylindrical piers. The data used were derived from Chabert and Engeldinger (1956) and CSU (Shen et al, 1966), the majority of the data being from the former source. Sediment sizes ranged from 0.24 mm to 0.52 mm, so that ripples would have formed on the bed of the laboratory flume. As is now well known (Chapter 4), ripple formation at flow velocities approaching the threshold condition for bed material entrainment significantly reduces scour depths measured in the laboratory. The outcome of the influence of ripple formation is an apparent dependence on flow velocity for live-bed scour depth, not demonstrated in data for uniform non-ripple-forming sediments.

The discussion leads to the conclusion that the Sheppard-Melville method better expresses the physical relationships between more of the principal parameters (y/a, a/D, and V/V_c,) and processes than does the Richardson and Davis method, and therefore is the more robust method for predicting scour depth over a broader range of parameter values. Yet the Sheppard-Melville method is unclear regarding the variation of y_s/a for --

1. Live-bed conditions. It is unclear why the parameter a^*/D is not significant for flow conditions producing high bed mobility; i.e., $V/V_c > V_{lp}/V_c$, but is significant for clear-water scour.

2. Wide-pier scour. The method does not indicate when the scouring flow field is in the narrow-pier, transitional-pier, or wide-pier category. The original Melville (1997) method better defines these categories.

6.4.2 Capacity to Include Recently Identified Parameter Influences

An important point made in Chapter 4 is that some variables exert multiple influences that do not lend themselves to evaluation in isolation of each other. For example, particle diameter is such a variable. Therefore, it is increasingly difficult for a method based on a semi-empirical, or rational, equation to include more parameter influences, especially if the equation does not expressly include the primary parameters forming the parameter framework.

By virtue of its use of the primary parameters y/a and a/D, along with V/V_c, expressing stage of bed mobility, the Sheppard-Melville method is better configured to include additional recently identified parameter influences than is the Richardson and Davis method. The Sheppard-Melville method, though, requires more research to resolve the points mentioned above. In particular, the method's present arrangement in terms of three ranges of value for V/V_c, and use of a single term ($f_1(y/a)$) may hamper its capacity to be extended for estimating scour depth for narrow-, transitional-, and wide-pier categories.

The Richardson and Davis method has been extended several times to reflect additional parameter influences, notably non-uniformity of bed particle diameter, bed particle mobility, and wide-pier scour. Experience with these extensions, though, has been

[6] Colorado State University, where extensive research on pier scour has been conducted

mixed[7]. It has not proven straightforward to tune-up the basic equation (based on Froude number) so as to embody additional parameter or process influences.

6.4.3 Limits of Methods

For the data ranges of the parameters on which they are based, the Richardson and Davis, and Sheppard-Melville methods yield acceptable scour-depth predictions. As shown in Figure A-1 through A-12 of Appendix A, the two methods give on the whole comparable scour-depth predictions for cylindrical piers in the narrow-pier category. As expected with semi-empirical equations generally, the two methods become limited in their capacity for accurate estimation pier scour depth when extended beyond the parameters and data range upon which they are based. However, the Sheppard-Melville method is less constrained in this regard because it better reflects scour processes framed in terms of the primary parameters y/a and a/D. It is more readily used to identify the potential maximum scour depth at a pier site.

The utility of an empirical or "rational-method" approach, whereby scour depth is related to a combination of individual parameter influences, diminishes when the number of parameter influences increases. Additionally, the linked influences of parameters, makes it infeasible for a single equation, such as Eqs (6.1) and (6.8), to embody more than the main parameter influences associated with either the narrow-, transitional-, or wide-pier categories of scour at cylindrical piers and the simpler common pier forms. For design estimation of potential maximum scour depth, though, it is not necessary to account for more than the primary parameters mentioned in chapter 4. Chapter 7 suggests how the Sheppard-Melville method should be simplified for design estimation of potential maximum scour depth.

Additionally, a common difficulty for design estimation of a potential maximum scour depth at piers, and limit in the use of the leading predictive equations, is determining the value of effective pier width a^* to used in the estimation. For these piers, the flow field usually is not adequately defined, and therefore accurate selection of a^* and estimation of scour depth cannot be expected from a predictive equation based on other pier forms.

To overcome the limits of the two leading predictive methods (or any single method), it is practicable to identify a structured design approach to be applied in accordance with pier size and form complexity, and site complications.

The approach would use the best information and techniques currently available for estimating scour depth:

1. For piers in the narrow-, transition-, and wide-pier categories, develop simplified forms of the leading equations for design estimation of potential maximum scour depth at cylindrical piers;

[7] Anecdotal discussion with designers

2. Tailor the simplified forms of the equations for application to common pier forms. This effort may require conducting hydraulic model tests with a selection of such pier forms;

3. Use modeling (hydraulic and possibly numerical) to determine design scour depth at uncommon and very wide piers. If scour at a very wide pier were essentially scour at an abutment with a deep foundation, it may be possible to adapt an abutment scour equation so as to estimate wide-pier scour.

The methodology can be applied to obtain estimates of potential maximum scour depth, whose uncertainty can be offset by the use of a margin of safety (e.g., from an upper-bound design curve, or a safety factor).

CHAPTER 7
PROPOSED DESIGN METHODOLOGY

7.1 Introduction

This chapter proposes a structured methodology for design estimation of pier-scour depth. The methodology, introduced in Chapter 2, is structured in terms of pier size and shape complexity, as well as site complications. It takes into account the primary parameters determining the scale of scour depth (Chapter 4), recognizes the difficulties introduced by site complications (Chapter 5), and works within the limitations of the leading predictive methods to estimate scour depth (Chapter 6).

The design methodology seeks to determine the potential maximum scour depth at a pier site. It does not aim at accurate prediction of scour depth for prescribed combinations of parameters, as would a scientifically oriented relationship aimed at close expression of parameter trends. The potential maximum scour depth is the deepest possible scour associated with the flow field generated at a pier. Two length scales (effective pier width, and approach flow depth) determine the structure of the pier flow field in turbulent open-channel flow and, thereby, the potential maximum depth of scour. The actual scour depth attained depends on the erosion characteristics of the foundation material at a pier.

The methodology does not rely on a single, basic equation for scour-depth estimation, but instead comprises several approaches graduated in accordance with pier shape and size, as well as site complications. It does, though, include such an equation for design estimation of scour depth for simpler pier shapes; Eq. (6.8). As pier size increases for a given flow depth and boundary material, pier flow field and scour capacity alter. Larger piers more commonly are less cylindrical in pier-column form and have more complicated foundations (e.g. more supporting piles). Moreover, such piers typically are in the transition- or wide-pier categories for scour (Chapter 4).

It soon becomes infeasible to use a single basic equation for accurate estimation of scour depth for all pier sites. Figures 1-2 and 1-3, illustrations of example long- and short-bridge sites, respectively, give a sense of the variability of pier sites. The potentially large number of variables involved, and the multiple influences that some parameters exert on pier flow field, can reduce estimation accuracy quickly.

7.2 Structured Design Methodology

The proposed design methodology assigns four levels of complexity, as outlined in Table 7-1. The levels relate to effective pier form, and thereby the complexity of the scouring flow field:

1. **Simple, single-column pier forms in the narrow- and transition-pier categories** usually cylindrical, for which the influences of the primary parameters identified in Chapter 5 are well defined and can be addressed reasonably well by means of an empirical equation framing the influences. The term “reasonably

well," here, implies that a clear quantitative relationship for scour-depth exists. Chapter 6 identifies and discusses the two leading equations for this purpose.

2. **Common pier forms** as commonly prescribed for ranges of bridge span and load. It is normal (e.g., most state departments of transportation in the U.S.), for pier forms and dimensions to be reasonably common. Such pier forms typically consist of a column on a pile cap supported by piles, or on a footing; in some designs, the bridge deck may be supported directly on a pile cap. More shape parameters need to be considered (e.g., spacing of piles), and the parameter influences become less independent as the pier flow field becomes more complex. To a certain extent, scour at these pier forms can be addressed using an empirical equation, when reliable pier-shape factors exist. If reliable shape factors do not exist, empirical equations may yield approximate scour-depth estimates that then should be verified with results from a hydraulic model (i.e. system simulation). In the future, scour depth estimates may be feasible by means of proven numerical models.

3. **Common pier forms in complicated situations** notably caused by the presence of debris or ice accumulation, the close proximity of an abutment, or possibly by channel morphology or geology issues. These pier situations can be addressed using the results from system simulation (hydraulic models, later numerical models), with some rationalization of parameter influences for the pier form under consideration. An approximate estimate of scour depth could be obtained by applying an adjustment factor to a basic, empirical equation. Considerable uncertainty attends such an estimate, however.

4. **Complex or uncommon pier forms and situations** where the influences of major parameters are insufficiently clear. These situations require a system simulation (holistic) approach to scour-depth estimation (analyzing the whole flow and erosion system at a pier, rather than analyzing individual parameter influences successively). Simulation is needed, by means of hydraulic-models and/or numerical models. The prospects for substantial developments in hydraulic modeling and numerical modeling must be considered for this component of the graduated approach.

The proposed methodology reflects the pier situations shown in Figures 1-2 and 1-3, which respectively illustrate the pier supports for a long multi-span bridge and a shorter, three-span bridge. The central pier in Figure 1-2 could be considered a fairly simple pier in isolation, but the local flow fields at other piers are to varying extents affected by flow around the abutments and over the channel's floodplain. The two piers in the shorter bridge (Figure 1-3) cannot be considered in isolation, and are markedly affected by flow around the abutment and over the floodplain. Similar illustrations can be prepared illustrating piers of more complex construction. No single existing method of scour estimation applies accurately to all pier situations.

A structured design methodology is not new. It is fairly normal for difficult or complex pier circumstances to receive additional design attention by means of hydraulic modeling, and possibly numerical modeling. However, the structured design approach proposed here formally outlines a more graduated set of considerations to be followed than is given in existing design guides for pier-scour estimation, such as the guides given by HEC-18 or AASHTO.

At bridge sites where large uncertainties attend pier scour, considerable reliance must be placed upon effective monitoring of pier site conditions. Design uncertainties may arise with uncertain changes in channel morphology, the possible construction of other engineered works (e.g., another bridge nearby), the erosion characteristics of bed or floodplain material, unclear combination of scour processes, and the development of scour over time. In addition, as described in Section 7.3, measured conservatism should be used in the design estimation of scour depth.

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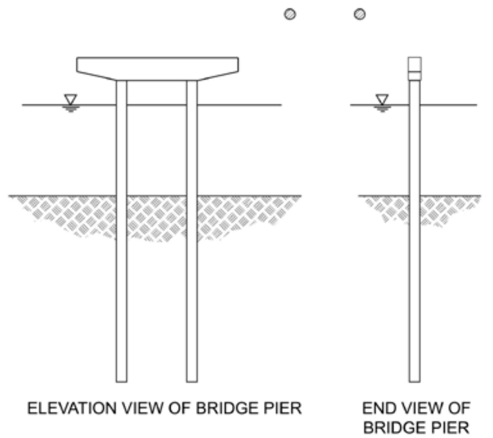
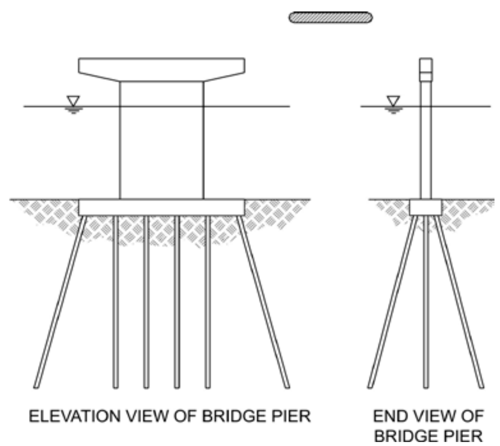
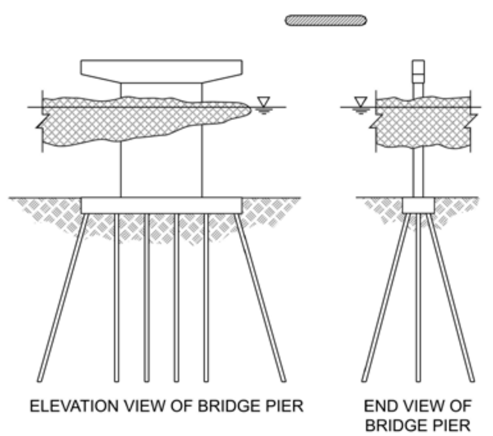
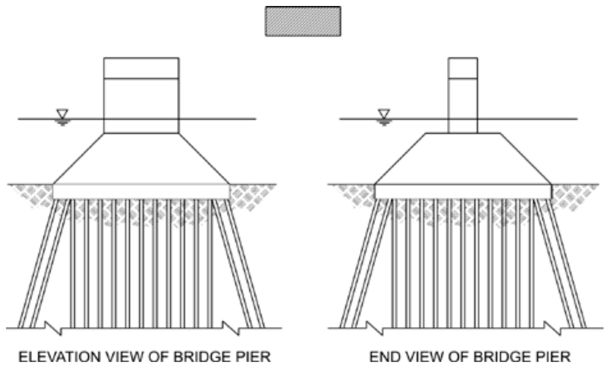
Table 7-1 Structured design approach

Pier Form and Situation	**Estimation Method**
1. Simple pier forms in flow-field categories— • Narrow pier • Transition pier 	Empirical equation: Transition from Richardson and Davis (2001) to Sheppard et al. (2011, NCHRP Project 24-32)
2. Common pier forms in flow-field categories— • Narrow pier • Transition pier 	Empirical equation; and, possible resort to a hydraulic model to reduce design uncertainty
3. Common pier forms at complicating sites subject to in flow-field categories— • Narrow pier • Transition pier 	Empirical equation; and, <u>increased</u> resort to a hydraulic model to reduce design uncertainty

4. Complex or unusual pier forms, including • wide piers		Resort to a hydraulic model, with possible use of a numerical model (system simulation) For wide piers, use empirical equation developed for wide-pier flow field

7.3 Uncertainty and Conservatism in Design Estimation

Design estimation of scour depth at a pier site often entails significant uncertainty regarding the accurate accounting of parameter influences possibly determining scour depth. It reduces when design estimation focuses on the primary parameters determining the potential maximum scour depth at a pier site, and parameters leading to lesser scour depths need not be addressed. This focus introduces a measured, reasonable level of conservatism into design estimation of scour depth, because it indicates the scour depth magnitude that could be attained at a particular pier site. However, while reasonable conservative estimates presently can be made for piers in the narrow- and transition-pier category (y/a exceeding about 1.0 and 0.5, respectively), such estimates become increasingly conservative and unreliable for piers in the wide-pier category.

Uncertainty in scour-depth estimation arises from several sources:

1. Changes in flow pier flow field, as characterized in terms of the primary parameters involving pier diameter y/a^* and a^*/D;

2. Pier site complications, such as inadequate definition and quantification of foundation material erodibility, and as a consequence of debris or ice accumulation; and,

3. The time development of scour introduces significant uncertainties associated with duration of flow conditions and variations in rate of foundation material erosion.

For live-bed conditions, an additional source of uncertainty relates to estimation of bedform (dune) amplitude at a pier site, and how to account for bedform amplitude when estimating scour depth. Bedform amplitude depends on approach flow depth and velocity, and varies as they vary. This source of uncertainty is addressable in terms of reliable bedform estimation, and linearly combining bedform amplitude with estimated pier scour depth.

For piers in the narrow-pier category, pier flow field scales with pier width, irrespective of waterway boundary material. Therefore, an approximate upper-bound estimate of scour depth can be obtained by relating maximum scour depth to pier width. The extensive body of laboratory data from model cylindrical piers points to a maximum scour depth of approximately 2.5 times the effective diameter of a cylindrical pier. Therefore, a reasonable, conservative estimate of maximum scour depth is

$$y_{sMAX}/a^* = 2.5 \qquad (7.1)$$

in which the effective pier width, a^*, includes factors accounting for pier shape and orientation. Eq. (7.1) is useful as setting an upper-bound magnitude for envisioning scour at simple cylindrical piers. At some pier sites, changeable flow orientation introduces uncertainty into this estimate. It should be noted that Eq. (7.1) is based on data obtained for small, laboratory-scale piers subject to a scale effect attributable to inadequate scaling of turbulence structures in the pier flow field (Section 4.4.5). It also envelops all known reliable field data. For Eq. (7.1), the condition $y/a^* > 0.2$ should prevail; i.e., design scour conditions place piers in the narrow- and transition-pier categories.

As y/a^* decreases below the narrow-pier limit (in practical terms, as pier size increases), Eq. (7.1), becomes increasingly conservative. The estimate given by Eq. (7.1) may still be acceptable for piers with the transition pier category ($0.2 \leq y/a^* \leq 1.4$), though it increasingly becomes unrelated to the flow field associated with Eq. (7.1) and, thereby, conservative. It completely loses physical meaning for piers in the wide-pier category, because the pier flow field is substantially different than the narrow-pier flow field. For a wide pier, scour predominantly occurs by virtue of flow contraction around the pier flanks, and the erosive influence of turbulence structures at the pier flanks and wake. Scour depth varies with the extent to which flow must contract as it passes around the pier; with scour at each flank becoming practically equivalent to scour at an abutment (or caisson) with a solid deep foundation, such as sheet-piling. At present, there is no upper-bound limit for wide piers comparable to Eq. (7.1). Melville (1997) proposes a design relationship based on curve fitting of laboratory data, but it is insufficiently corroborated for use in estimating an upper-bound scour depth; the relationship, elaborated in Table A-1, is $y_s = 4.5y$.

The structured design methodology in this chapter aims to provide reasonable conservatism in scour-depth estimation for piers in the narrow- to transition-pier categories (or common pier forms in straightforward situations), but also places increased reliance on holistic simulation, and on bridge monitoring for more complex situations. Formal risk-assessment analysis of pier scour is described by Johnson (1994, 1995), and is the subject on NCHRP Project 24-34, *Risk-Based Approach for Bridge Scour Prediction*, currently underway.

7.4 Single-Column Piers in the Narrow- and Transition-Pier Categories

Cylindrical pier forms are piers of a single cylindrical body whose flow field comprises the fully developed features illustrated in Figure 3-4 for a single circular cylinder, or a pair of such cylinders as depicted in Table 7-1. This category includes slender elongated piers reasonably well aligned with the flow. The main features of the flow field are well understood, though exactly how the flow field changes with changes in the parameters y/a^* and a^*/D remains inadequately understood, and quantified, for the purpose of quantifying the erosive forces that the flow exerts on foundation material.

The designer estimating scour depth at a pier site should determine the potential maximum depth of scour associated with a given flow field as scaled in terms of effective pier width (a^*), flow depth (y), and flow intensity at the site (V/V_c). A lesser depth can be selected for design use at the designer's discretion. Doing so entails considering additional parameter influences, and increases design uncertainty. The ensuing thoughts attend this approach:

1. Design estimation must account for the influences of the primary parameters, especially the geometric scale parameter y/a^*, and parameters defining an effective pier width, a^*;

2. Design estimation recognizes that substantial uncertainty and variability exists for parameters at pier sites. The length-scale parameter y/a^* typically is relatively fixed, and has minimal uncertainty for most bridge waterways;

3. Substantial uncertainty occurs in determining, and in actual variations of, V/V_c;

4. Design estimation of a potential maximum scour depth averts the uncertainties associated with the temporal development of scour; and,

5. There is a mild difference between the potential maximum scour depth for clear-water and live-bed scour conditions, though the latter requires accounting for the additional foundation exposure caused by bedforms on the approach bed.

In accordance with these design considerations, design estimation of scour for narrow- and transition-pier categories would proceed as indicated in Figure 7-1. An envelope resides around the clear-water and live-bed scour conditions. The envelope position varies with y/a^* and a^*/D. If the influence of a^*/D is not considered, the envelope lowers with decreasing y/a^* for the transition- and wide-pier categories.

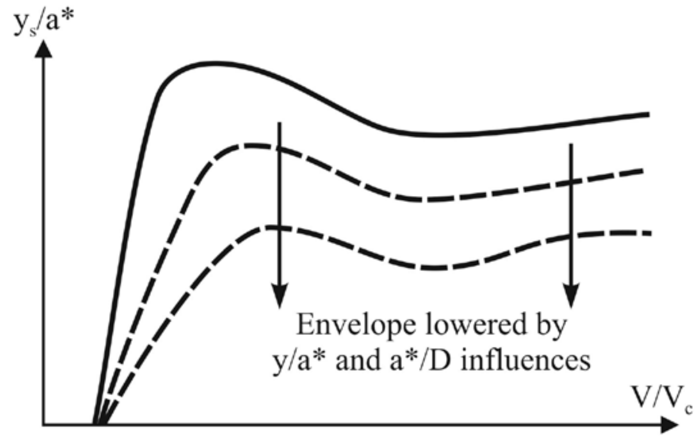


Figure 7-1 Envelope of potential maximum scour depth for clear-water and live-bed scour conditions at piers

The evaluation presented in Chapter 6 indicates that two existing rational methods empirically best reflect, at this point in time, the scour-depth variations for the narrow- and transition-pier categories: Richardson and Davis (2001), the method currently used in HEC-18; and, the Sheppard et al. method (2011) as developed in NCHRP Project 24-32. Either of these methods yields credible scour depth estimates for pier sites in the narrow-pier category and subject to clear-water conditions. The methods are well-correlated to the extensive data available for such sites.

Of the two methods, however, the Sheppard-Melville method better reflects flow-field changes and thereby scour processes, as expressed in terms of the parameters of primary importance, and therefore is more readily extended to the transition- and wide-pier categories; i.e., it more expressly includes the length scales, y/a^* and a^*/D, and includes the flow intensity parameter, V/V_C. Additionally, it is the method developed (with funding for NCHRP 24-32) in an effort to account for flow-field adaptation to variations of y/a and a/D. A useful aspect of the Sheppard-Melville method is that it can be simplified to reflect potential maximum scour depth associated with the three pier flow-field categories (narrow, transition and wide). A weakness in the Richardson and Davis (2001) method is that it does not. A further weakness is that it uses y/a rather than y/a^*.

For design estimation of a potential maximum scour depth at a pier site, the Sheppard-Melville method can be simplified so as to avert the uncertainties associated with the parameters V/V_c and a^*/D. The simplification relates potential maximum scour depth to the parameter of prime importance, y/a^* for clear-water and live-bed scour.

In terms of estimating a potential maximum scour depth, related to the scale of the pier flow field, the Sheppard-Melville method, Eq. (6.8), simplifies to

$$\frac{y_s}{a^*} = 2.5 f_1\left(\frac{y}{a^*}\right) = 2.5 \tanh\left[\left(\frac{y}{a^*}\right)^{0.4}\right] \qquad (7.2)$$

This equation can be used for piers founded in sediment or cohesive soil, and subject to the narrow- and transition-pier flow fields. The designer can use Eq. (6.8) if wishing to account for the influences of a^*/D and V/V_c. The designer, however, must recognize the uncertainties introduced with consideration of these parameters.

The Richardson and Davis (2001) method (Eq. 6.1) does not lend itself to this simplification in terms of y/a^*. Therefore, it is less suited for design estimation of the potential maximum scour depth at piers in the transition-pier through the wide-pier categories; i.e., it does not adequately reflect the influence of parameter y/a^*.

7.5 Common Pier Forms

Common pier forms typically comprise several structural or form components, as indicated in Table 7-1. Simpler common piers may comprise two cylindrical piles with a pile cap above the design flow elevation. Another basic pier form comprises a pier column supported by a cluster of cylindrical piles. The additional elements add complexity to the flow field, increase the possible number of parameters to consider, and thereby complicate reliance on an empirical equation for scour depth estimation. As pier form complexity increases, estimation uncertainty also increases.

The ideal of having a set of scour estimation equations each tailored for common pier form is appealing. However, such a set does not exist, and certainly no one equation applies reliably to all common pier forms. Therefore, it is prudent to use a twofold design approach:

1. Use the Sheppard-Melville method, Eq. (7.1) or Eq. (6.8) adapted with a pier-shape factor so as to estimate scour at complex piers. Because the adaptations for complex piers are approximate and do not relate directly to the pier flow field, it is important to take into account the uncertainty associated with applying the adapted equation; and,

2. Consider the value of conducting hydraulic model tests to address the uncertainty associated with scour depth estimation. If the pier is large, such tests may be a worthwhile design task. If the pier is small, the uncertainty could be addressed by taking a more conservative estimate with a simple-pier equation.

There is merit in conducting a series of hydraulic model tests to determine the scour performance of common pier forms. Such a series of tests would seek to obtain the maximum scour depths at each pier for variations of y/a^*. The Sheppard-Melville (NCHRP 24-32) and Richardson and Davis (2001) equations offer procedures to account for non-cylindrical pier forms. However, each procedure is approximate and involves substantial uncertainty.

7.6 Common Pier Forms in Complex Situations

Two categories of situation complicate, and contribute uncertainty to, design estimation of scour depth at a pier of simple or common form:

1. Alteration of the pier flow field by several processes, notably abutment proximity, debris or ice accumulation, bridge over-topping, channel morphology issues; and,

2. Unknown foundation erodibility (erosion resistance and manner of erosion) of the bed material at the pier location. Pier sites may have complex boundary materials.

These sets of factors require additional design steps to be applied in design estimation of scour depth. The ensuing sections recommend how these complicating situations could be addressed.

It is not a straightforward matter to forecast alteration of pier flow field. However, NCHRP Project 24-20 shows that, when a pier is within a certain distance from an abutment, scour at the pier is dominated by the abutment flow field and scour.

7.6.1 Abutment Proximity

When a pier is located within the immediate vicinity of an abutment (notably, near the toe of an abutment), abutment scour dominates pier scour. For this situation, the design equations for simple or common pier forms do not apply. Scour depth at the pier is essentially equivalent to scour depth generated by flow around the abutment. A curve developed from NCHRP Project 24-20, "Estimation of Scour at Abutments," could be used in estimating scour depth at a pier close to an abutment. When the pier is at the abutment toe, or near it, one curve limit indicates scour depth equivalent to abutment scour depth. When the pier is distant from an abutment, the other curve limit indicates scour depth is equivalent to scour at the pier in isolation. In some cases, it may be necessary to superimpose an estimate of pier scour on the estimated abutment scour depth.

7.6.2 Woody Debris or Ice

During floods, many rivers carry appreciable quantities of floating debris such as branches and roots of trees. If the debris becomes caught at bridge piers and abutments, it can accumulate into large masses of material. A foundation with accumulated material causes a larger obstruction to the flow than without debris; the additional flow obstruction generally causes local scour depths in excess of depths under conditions without debris accumulation. The designer is recommended to follow the recommendations in NCHRP Project 24-26, *Effects of Debris on Bridge Pier Scour*, (Lagasse et al., 2010), which in turn is based on earlier studies of debris production and accumulation by Diehl (1997), and scour depth estimation at piers with debris accumulation by Melville and Dongol (1992).

Lagasse et al. (2008) present guidelines for predicting the size, location, and geometry of debris accumulations at bridge piers. The guidelines are presented as a series of flowcharts, included as an appendix in their report. Additionally, Lagasse et al. (2010) give equations for estimating an equivalent pier width for use with the Richardson and Davis (2001) pier scour equation; see Section 5.5. The equations are adaptable for use with the method by Sheppard-Melville (NCHRP 24-32). In situations anticipated to be more complicated, or different, than those studied by Lagasse et al. (2010), the designer is encouraged to consider conducting hydraulic modeling to get scour-depth estimates.

7.6.3 Bridge Deck Over-topping

The design methods suggested by Guo et al. (2009) provide the most current means of estimating scour at submerged bridge decks. This scour is a form of contraction scour. Their method, however, does not expressly include scour at a pier at a submerged bridge deck. At this time, the designer should linearly combine the scour depths associated with the pier (assuming the deck is not submerged) and the contraction scour attributed to deck submergence. Further research is needed to ascertain how the scour depths actually combine.

7.6.4 Channel Morphology

The designer is referred to the findings from NCHRP 24-27(03), *Evaluation of Bridge-Scour Research: Geomorphic Processes and Predictions*. It is rather usual for the pier to be designed so that its foundation extends lower than the lowest possible elevation of the thalweg passing through the bridge waterway. Considerable designer judgment is required when channel morphology issues come into consideration.

7.6.5 Armoured Boundary Surface, Layered Bed Sediment

The designer is referred to Section 5.7 for guidance regarding scour-depth estimation for a pier in layered foundation material.

7.5.6 Weak Rock

The designer is referred to the findings from of NCHRP 24-29, *Scour at Bridge Foundations on Rock*, for insight. To be kept in mind is the fact that scour depths in rock have not been shown to exceed those in non-cohesive sediment or cohesive soils.

7.7 Wide, Complex, or Uncommon Pier Forms

For the wide-pier category of pier scour, and generally when pier geometry and site conditions are sufficiently complex as to be beyond the range of parameter values covered by those used in formulating the Sheppard-Melville method (or the Richardson and Davis (2001) method), the designer must take a system simulation approach. System simulation entails the use of hydraulic-model and/or numerical-simulation testing to estimate a design depth of scour. Such testing provides a holistic insight into scour development, serving not only to obtain a design estimate for scour depth, but also revealing how and where scour develops, and adding assurance that the scour depth is within a certain magnitude estimated approximately using the method for a simple pier (such as by the Sheppard-Melville method).

For wide piers of cylindrical shape, and extending as a solid cylindrical shape into the foundation material, a scour depth estimate could be obtained using an abutment-scour equation. The deepest scour occurs at the pier flanks, where the pier flow field is similar to flow around this form of abutment (or caisson). Further research is needed to confirm this method for scour-depth estimation at wide piers.

The ensuing sub-sections briefly evaluate the prospects for improved scour-depth estimation using hydraulic modeling and numerical modeling, or hybrid modeling involving both approaches.

7.7.1 Hydraulic-Modeling

For some pier situations there is little alternative other than hydraulic modeling to estimate a design pier scour depth. Such situations arise when complex flow patterns or intricate transport processes are involved, and reliable answers cannot be obtained by means of existing methods for estimating scour depth. Indeed, the existing design methods for pier scour were themselves developed by means of hydraulic modeling using laboratory flumes. As with all loose-bed hydraulic modeling, care is needed to address potential scale effects associated with the three length scales involved (pier width, flow depth, bed material). At this point in time, the complexity of the flow and sediment movement around a pier is still beyond the capability of numerical simulation models based on computational fluid dynamic (CFD) codes. However, numerical simulation holds substantial promise for greater design use.

Over the past decade, hydraulic modeling is increasingly used in combination with computational models to investigate difficult flow situations that each modeling method alone would be inadequate to address. These combined modeling techniques are discussed in Section 1.6, which outlines modeling strategies.

Besides their direct use to produce information that cannot be reliably obtained by some other means for design or operational purposes, hydraulic models have additional benefits. They can be a form of relatively inexpensive insurance, reducing the uncertainty associated with a design or an operational procedure. A comparatively small investment in a hydraulic model study, especially in the case of expensive constructed works, may help allay concerns regarding the viability of a design or a procedure. The cost of a model study typically is insignificant compared to the cost of the actual installation, herein (and traditionally) called the prototype. A hydraulic model also can be useful for public relations purposes, demonstrating to the lay person as well as the sceptical engineer how a design or a procedure will function. It usually is a convenient device for communicating complex hydraulic ideas.

The similitude principles that form the basis for hydraulic modeling are fairly straightforward. However, a difficulty incurred with preparation of a manual on hydraulic modeling is determining the extent of background information to be covered in order to adequately present the similitude criteria. A diverse range of flow or dynamic situations are treatable using hydraulic modeling. Though an attractive feature of hydraulic modeling is that similitude principles and criteria are readily understood, their implementation requires a sound understanding of the underlying physical processes and recognition of a model's capacity to replicate those processes.

Few models exactly replicate all the processes involved with a particular flow situation. Shortcomings in models usually are termed scale effects or laboratory effects. The former term describes the incomplete satisfaction of a full set of similitude criteria associated with a particular situation. Scale effects increase in severity as the ratio of prototype to model size increases or the number of physical processes to be replicated simultaneously increases. Laboratory effects arise because limitations in space, model constructability, or instrumentation impede precise replication or measurement. They also arise from incorrect replication of boundary conditions.

Ever since the establishment of hydraulics laboratories, there has been a trend for more accurate quantitative information from hydraulic models. This trend has required refinement of similitude criteria for improved definition of processes, and finding means to overcome practical constraints, such as being limited largely to one model liquid (water). It also has required innovative efforts to overcome some of the restrictions imposed by laboratory facilities, such as limits in space and instrumentation capabilities. ASCE Manual 97 (ASCE 2000) provides useful guidance concerning the effective employment of hydraulic models to address a broad range of hydraulic engineering issues.

Significant improvements have been made in instrumentation and laboratory equipment and modeling methodology. Many of the improvements have been facilitated by the computer and ancillary electronic instrumentation. Though much had been accomplished with point gauge and Pitot tube, newer hydraulics problems required more sophisticated instrumentation. Computer-aided data-acquisition systems increased the sophistication of modeling by facilitating measurement and collection of large amounts of data hitherto considered impractical or impossible to acquire. Models could now be run with greater flexibility in terms of location, scheduling, variables measured, as well as processes observed. A major advance has been in the capacity to observe and measure features of large-scale turbulence structures, such as exist in pier flow fields.

Hydraulic modeling is reliant on instrument technology, and has evolved in scope as instrumentation developments have enabled aspects of flow to be illuminated and measured. In this sense, hydraulic modeling is as sophisticated as the instrumentation used to conduct the modeling. As instrumentation and computer technologies continue to progress, so too will hydraulic modeling and its utility for pier scour investigations.

7.7.2 Numerical Modeling

A growing number of numerical methods can be usefully applied to study flow at bridge piers and pier scour. An important limitation, at present, is that numerical simulations with a loose bed are not yet sufficiently reliable to predict bed evolution or equilibrium scour bathymetry. However, fully three-dimensional numerical simulations can provide critical information on the complex flow fields and bed shear stress distributions at different stages of the pier scour process. This advantage holds for complex pier forms. Used together with hydraulic models, numerical investigations can provide a better understanding of scour processes and mechanisms at bridge piers of complex geometries. Information on the flow patterns and turbulence structure at relevant flow conditions (e.g., from normal to flood conditions) obtained from simulations of flow past bridges containing piers and abutments can be used in the design process before a bridge is built. The potential for improved design and the minimization of over-design is substantial.

Most current numerical investigations of bridge pier flows used steady RANS models with wall functions (e.g., see Olsen and Melaaen, 1993, Richardson and Panchang, 1998, Olsen and Kjellesvig, 1998, Wang and Jia, 2000, Ali and Karim, 2002, Salaheldin et al., 2004, Roulund et al., 2000). This method can easily be applicable for bridge piers and bridge abutments of complex geometry at field Reynolds numbers. For the case of a highly unsteady flow characterized by large-scale unsteady vortex shedding and severe adverse pressure gradients, the use of the wall-function approach which assumes the validity of the law-of-the-wall near solid boundaries is questionable. The use of a steady model excludes capturing not only of the unsteady vortex shedding behind the pier but also the unsteady dynamics of the horseshoe vortex system. Thus, the use of this approach to understand the flow physics is very limited. More importantly, the accuracy of the mean flow and turbulence predictions obtained using steady RANS simulations is relatively poor due to the flow complexity.

Unsteady RANS methods are somewhat more accurate (e.g., Wei et al., 1997, Paik et al., 2004, Ge et al., 2005). For example, Paik et al. (2004) performed RANS simulations of a complex bridge pier configuration at a Reynolds number of 100,000. The geometry corresponded to a rectangular open channel with four bottom-mounted rectangular piers located one behind the other along the flow direction. The simulation successfully captured the unsteady large-scale vortex shedding and wake-boundary layer interactions present in such flows.

One should mention several attempts (e.g., Wei et al., 1997, Nurtjahyo et al., 2002, Chen, 2002, Nagata et al., 2005) to use RANS methods coupled with a movable bed module to predict the flow field and the bed evolution and equilibrium bathymetry. For example, Chen (2002) calculated the scour evolution around a complex system of multiple piers (e.g., one of the cases contained four rectangular piers and two abutments) using grids containing approximately 500,000 cells at a Reynolds number of 24,000. The contraction and local scour patterns around the piers were found to be in fair agreement with general experimental observations of the scouring process.

Though certainly an important direction for future research, 3D steady RANS simulations with movable bed are not yet sufficiently reliable to predict scour depth for bridge pier design in natural waterways. For simple circular-cross-section, narrow piers (y/a > 1.4), some such simulations (e.g., Rouland et al. 2005) predict reasonable estimates of scour depth, especially at the upstream side. The main disadvantages of the RANS approach is that it cannot accurately predict massively separated flows and flows in which large adverse pressure gradient are present. It cannot give any information on the temporal dynamics of the main eddies which drive the scour process. Past experience shows that both steady and unsteady RANS models fail, to a variable extent, to predict important aspects of massively separated flows dominated by unsteady vortex shedding and large-scale vortex interactions (e.g., see Rodi, 1997, Rodi et al., 1997, Tokyay and Constantinescu, 2006, McCoy et al., 2008). For example, RANS simulations cannot correctly predict the structure of the horseshoe system vortex during the initial stages of the scour process when the scour hole is very small. Additionally, such simulations fail to predict the bimodal nature of the large scale oscillations of the main horseshoe vortex in front of the pier. Thus, RANS simulations have marginal use for understanding pier flow fields and consequent erosion and deposition processes.

Research over the last ten years has proven that eddy-resolving three-dimensional numerical simulations of flow past bridge piers can provide a detailed description not only of the mean flow field and turbulence structure at a level that is very hard to obtain from experiments but also of the coherent structures controlling the bed erosion phenomena. Information from fully three-dimensional numerical investigations that can characterize in a qualitative and quantitative way mean flow, turbulence and scour processes and the strength of the scour processes function of the main flow and pier geometry parameters are essential to be able to improve existing methods or propose new methods to estimate scour depths at bridge piers. For example, scour prediction equations neglect the scaling of the large-scale coherent structures present in the region surrounding the pier. This is a major deficiency of present methods to estimate scour. To take this into account in scour design, a detailed qualitative and quantitative knowledge of the dynamics of the coherent structures at different stages of the scour process is needed. Of course, a major direction for future scour research is how the additional information and insight into the fundamental flow and sediment transport processes available from state-of-the-art experimental and numerical techniques can be used to develop more accurate relationships to predict local scour depth around bridge piers.

Among the eddy resolving techniques the most popular is Large Eddy Simulation (LES). In LES, the unsteady dynamics of the energetically important scales in the flow is directly computed, and only the effect of the filtered (small) scales on the resolved scales is modelled. Eddy resolving simulations can provide three-dimensional visualizations of the entire (averaged and instantaneous) flow field, thereby bringing into view the main coherent structures in the flow, their dynamics, and the nature of their interactions with other large-scale eddies or with the bed. Thus, they can be used as an effective tool to better understand the physics of bridge pier flows. In particular, such simulation can elucidate the role of the large-scale coherent structures in the scouring process. Eddy resolving numerical simulations are much less expensive than comprehensive PIV experimental investigations of the flow around bridge piers. Moreover, eddy resolving simulations can provide information on quantities that are very difficult to estimate accurately based on experimental measurements. For example, it is feasible to visualize the distribution of the bed shear stress on the whole bed surface, get information on the distribution of the pressure fluctuations in the near bed region, visualize the whole horseshoe vortex system at a certain moment in time, and capture the interactions between the necklace vortices and the eddies that populate the detached shear layers at the sides of the pier.

Earlier LES simulations of flow past circular and rectangular bridge piers with flat and deformed bed were reported by Tseng et al. (2000) and by Choi and Chang (2002) at relatively low Reynolds numbers. Though these LES simulations performed on relatively coarse meshes appear to better capture the mean flow compared to RANS simulations, the simulations were less successful in capturing the structure of the horseshoe vortex system in the mean and instantaneous flow fields.

More recent LES simulations using state of the art subgrid-scale models and fine meshes (e.g., Kirkil et al, 2008, Koken and Constantinescu, 2008a-b) have allowed the investigation of the role of coherent structures in the scouring process at bridge piers and abutments of simplified geometry. These simulations were shown to accurately predict the flow patterns and turbulence structure compared to other numerical methods. These studies provided a detailed investigation of the structure of the horseshoe vortex system as it wraps around the upstream base of the pier. For example, the simulation of Kirkil et al. (2008) captured the presence of bimodal aperiodic oscillations inside the HV system region, as well as the associated increase in the turbulence intensity. The simulation results showed the interactions among these large-scale structures (necklace vortices, wake vortices, vortices present in the separated shear layers) are controlling, to a large extent, the scouring mechanisms around the pier. Several mechanisms that can explain the growth of the scour hole laterally and behind the cylinder were identified. LES simulations allowed for the first time a detailed study of the distributions of the pressure root-mean-square fluctuations at the bed and of the distributions of the instantaneous and time-averaged bed shear stress that are very difficult to estimate accurately from experiments.

Hybrid RANS-LES methods like Detached Eddy Simulation (DES) are a good alternative to LES (e.g., Paik et al., 2007, Kirkil et al., 2009, Koken and Constantinescu, 2009). Their big advantage is that they facilitate simulation of flow past bridge piers at field Reynolds numbers without using wall functions. One of the disadvantages of DES, which is the most popular non-zonal RANS-LES method, is that the subgrid-scale model employed in regions where DES is in LES mode has much less physics and is significantly more dissipative compared to the dynamic Smagorinsky model or to other state-of-the-art LES models. Both LES and DES allow investigation of flow past bridges in river reaches of complex bathymetry.

Based on the proven success of LES and DES for simpler pier geometries, similar simulations can be conducted with bathymetries corresponding to the different stages of the scouring process, between initiation and equilibrium, to understand the flow and scour mechanisms at complex piers. This is important because the role and relative importance of the different coherent structures may change during the scour process. Also, the nature of their interactions may change. Several specific applications addressing important gaps in knowledge of scour processes at bridge piers in which numerical, eddy resolving techniques may help understand the flow structure and scour processes are mentioned in Chapter 8.

CHAPTER 8
RESEARCH NEEDS

8.1 Introduction

The longevity of pier scour as a research topic reflects the potential complexities of the flow field and erosion processes involved. The flow field is highly three-dimensional, marked by a set of interacting turbulence structures, unsteady, and varies with pier form and size, flow depth and the erodibility of pier foundation material. Additionally, erosion processes evolve as scour progresses. Moreover, a number of bridge waterway features may modify the flow field, and foundation material may be of variable and unknown erodibility.

This chapter outlines the research needs required to improve the reliability of design estimates of scour depth, as obtained using the design methodology described in Chapter 6. The research needs are structured in two broad terms:

1. Design issues; and,
2. Better understanding of scour processes in order to address the design issues.

There also are research needs related to longer-term improvements in design methodology. These needs, not elaborated here, concern overall developments in computer-based modeling and bridge-site monitoring. In several decades, the advances in numerical modeling conceivably could facilitate the practical estimation of pier-scour depth estimation. They already enable remarkably comprehensive complete insights into pier flow fields.

The priority range used for NCHRP 24-8, "Scour at Bridge Foundations: Research Needs" (Parola et al. 1996a), is used herein. The priorities are assigned as critical, high, medium, and low. Table 8-1 defines the thoughts associated with each priority. Low priority research needs are not included here.

The research needs identified here relate to those identified in NCHRP Projects 24-32 and 24-08, as recapped in Appendix B. However, distinguishing themes of the present recommendations are an emphasis on the development of a practicable approach to design estimation for potential maximum scour depth at a pier site, and how potential maximum scour depth varies with pier flow field. The ensuing sections briefly elaborate the research needs listed in Tables 8-2 and 8-3 for single-column piers, and more complex pier forms and complicating site factors.

8.2 Research Needs for Single-Column Piers

Table 8-2 lists research needs whose resolution would advance the design methodology recommended in Chapter 7 for single-column piers, the simplest pier form. The needs are presented in terms of design issues and scour processes.

8.2.1 Design Issues

The following research topics aim at design method improvements:

1. ***Piers in the transition- through wide-pier categories***. Though numerous laboratory studies have been conducted of piers in the narrow- to transition-pier categories ($0.2 < y/a*$), additional research is needed for piers in the transition- though wide-pier categories ($y/a* < 1.4$), especially regarding three considerations:

   i. The Sheppard-Melville method simplified as Eq. (7.2) is immediately useful for design estimation of a potential maximum scour depth, but the method's full form (Eq. (6.8)) requires further development for piers in the transition-pier category and subject to live-bed scour. Data and observations (laboratory and field) for pier scour subject to these conditions are fairly scarce, owing to the lack of laboratory flumes of sufficient cross-sectional area and flow capacity, and to the difficulty of obtaining field data. In particular, the influences of the parameter $a*/D$ need to be determined for live-bed scour over the full range of pier-scour categories;

   ii. Pier scour for the changeover from the transition- to wide-pier scour categories requires more investigation to extend the Sheppard-Melville method more reliably into the wide-pier category, or ascertain the method's limit of applicability in this regard;

   iii. Further to items i and ii, the role of pier alignment (and the commensurate alignment factor used to assess an equivalent pier diameter, $a*$) should be investigated for piers in the transition- to wide-pier categories. For such piers, the alignment factors may not be valid, because the flow field and scour condition changes from one category to the next;

2. ***The combination of pier scour and contraction scour***, especially for scour of cohesive soil. Though Briaud et al. (2005) in NCHRP 24-15 investigated pier scour in cohesive soil, and subject to constricted flow, in laboratory flume experiments, additional work should examine the overall scour bathymetry of contraction and pier scours; and,

3. ***Likely occurrence of maximum scour***. Eq. (7.2) as recommended provides an estimate for the potential maximum scour depth at a pier site. There is merit in determining a procedure for assessing the uncertainty associated with the occurrence of the potential maximum scour depth at a pier.

Topics 1 and 2 align with the objectives of NCHRP Project 24-32, "Scour at Wide Piers and Long Skewed Piers" (Sheppard et al. 2011). Topic 3 aligns with the objectives of NCHRP 24-34, "Risk-Based Approach for Bridge Scour Prediction."

8.2.2 Scour Processes

The research topics listed below aim at better understanding of scour processes in order to improve design methods:

1. ***Flow Field.*** To understand scour and its depth variations with the principal parameters (i.e., those characterizing pier size and form, flow depth, and the erosion resistance of foundation material), it is necessary to understand how pier flow field varies with the parameters. The ensuing topics require further research:

   i. ***Systematic changes in the main flow field features with the primary parameters determining potential maximum scour depth***. This research need reflects the inadequate understanding of the pier flow field and how it changes with variations of pier size and (cylindrical[8]) form, flow depth, and bed particle size (or erosion resistance of foundation material). Though many studies illuminate aspects of pier flow field, at present no single study or published document systematically describes these changes. In particular, the flow field variations associated with the three categories of pier flow field need improved definition:

      a. Pier flow field leading to deepest scour at the leading face of the pier (narrow-pier category of pier scour);
      b. Pier flow field causing deepest scour at the sides of the pier (wide-pier category of pier scour); and,
      c. The transitional condition between these categories (as occurs when flow orientation varies at a long skewed pier in a relatively shallow flow).

      All existing scour equations use curves indicating the effects of pier shape and alignment associated with the narrow-pier scour category. Further research is needed to provide curves for the transition- to wide-pier categories of scour. Achieving this entails knowledge regarding the main flow field features for these categories.

   ii. ***Influence of bedforms on pier flow field***. Live-bed scour is marked by the presence of bedforms, whose dimensions depend on length scales[9] other than pier width. However, pier width determines the length scale of scour, and thereby the presence of bedforms within the scour hole. Two topics need further research:

      a. Influence of approach-bed bedforms on pier flow field for categories a, b, and c in Item i above; and,
      b. Influence of scour-zone bedforms on pier flow field.

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[8] Circular, rectangular, other

[9] flow depth (dunes) and bed particle size (ripples)

2. **Erosion of Foundation Material.** Erosion of foundation material at a pier occurs in several ways, depending on the nature of the material (non-cohesive sediment, cohesive soil, rock). Remaining research needs address aspects of flow field entrainment and transport of foundation material:

   i. ***Erosion of non-cohesive foundation material by turbulence structures and unsteady flow features in the pier flow field***. To understand pier scour, it is necessary to know how boundary material is entrained, transported and deposited by turbulence structures in the pier flow field, for flow field categories i, ii, and iii. In ways still to be defined, the turbulence structures vary in importance in accordance with pier size and (cylindrical) form, flow depth, and bed particle size. The literature on pier scour inadequately addresses the role of the set of influential turbulence structures (including horseshoe vortex system, wake vortices).

   ii. ***Erosion of cohesive foundation material***. NCHRP Project 24-15, "Pier and Abutment in Cohesive Soils," yielded considerable insights into scour of cohesive foundation material. In addition to the substantially longer period needed to scour clay, the study showed that clay eroded as fragments or chunks subject to steady and oscillatory flow forces acting in the scour hole. Further research is needed to:

      a. Determine and document how clay erodes, especially for several types of clay subject to the narrow-, transition-, and wide-pier categories of scour; and,
      b. Confirm whether or not pier scour in clays does not attain greater potential maximum depths than scour in cohesionless sediments.

   iii. ***Erosion of rock foundation material***. NCHRP Project 24-29, "Scour at Bridge Foundations on Rock," now underway with a review of pier scour of rock foundation material, will suggest research needs regarding pier scour in rock foundations. Though the project's findings have yet to be disseminated, several prominent facets of scour in rock should be noted here:

      a. Because the unsteady hydrodynamic forces associated with the turbulence structures in the pier flow field play a major part in scour of rock around bridge piers, more information is needed as to how pier scour develops in rock of different erosion characteristics, and how the resulting scour forms relate to pier flow field; and,
      b. Confirm whether or not pier scour in various rock types does not attain greater potential maximum depths than scour in cohesionless sediments.

8.3 Research Needs for Complex Pier Forms and Site Factors

Table 8-3 lists research needs whose resolution would advance the design methodology recommended in Chapter 7 for piers whose form is more complex than a single column or which face more complicating site factors; i.e.,

1. Piers of common form (column, pile cap, piles, or spread footing);
2. Piers subject to complicating site factors; and,
3. Piers of unusual form or complexity.

The research needs are presented in terms of design issues and scour processes.

8.3.1 Design Issues

The following research topics aim at design method improvements:

1. ***Common pier forms***. The frequent use of common pier forms suggests there is merit in conducting hydraulic model tests to obtain scour data for such piers. The purposes of the tests would be as follow:

    i. Develop shape and alignment factors for common pier forms by relating the scour data to scour depth at a cylindrical pier. Adapt the relationship for cylindrical piers (Eq. (7.2)) so that it accurately reflects scour at common pier forms not of simple cylindrical form; and,

    ii. Obtain a body of data on potential maximum scour depth at common pier forms. The data should be valid for the pier scour and wide-pier scour situations, and enable use of the fully developed Sheppard-Melville method.

    iii. As for common pier forms, there is merit in determining a procedure for assessing the uncertainty associated with the occurrence of the potential maximum scour depth at a pier.

2. ***Complicating site factors***. The design methodology must accommodate site factors complicating, or contributing uncertainty to, the flow field at a pier, or erodibility of the boundary material. An effort is needed to adapt Eq. (7-2), or empirical relationships using scour data obtained for common pier forms, to account for the complicating factors:

    i. Determine how scour-depth estimation should account for bridge-deck submergence;
    ii. Confirm a design guideline regarding pier scour estimation for piers in layered sediment, particularly when the top layer acts as a protective armoring layer; and,
    iii. Though major progress has been made in understanding and formulating debris-accumulation effects on scour depth (Lagasse et al. 2010), scope exists for extending this progress to encompass debris accumulation at

selected common pier forms especially prone to debris accumulation (e.g., pier bents).

Topics i and iii align with recommendations stemming from NCHRP Project 24-26, "Effects of Debris on Pier Scour." A large debris raft, or ice accumulation, extending across several piers or an entire bridge waterway may modify the pier flow field similarly as would a submerged bridge deck. Item ii relates to channel geomorphology issues as considered in NCHRP Project 24-27(03), "Evaluation of Bridge-Scour Research: Geomorphic Processes and Predictions."

3. ***Wide piers and uncommon pier forms***. As indicated in Section 7.7, a reliable method has yet to be developed for scour-depth estimation at piers in the wide-pier category. The Sheppard-Melville (Sheppard et al. 2011) and Melville (1997) methods purport to be of use for wide-pier scour, but are based on data from comparatively few laboratory experiments. It is likely that an existing method for abutment scour could be adapted for this purpose. Moreover, as wide-piers are built it will be useful to obtain field data from them. Such data should be compared with data from abutments, sheet-pile coffer dams, and spur dikes built with solid foundations that penetrate to depth within a channel bed.

Time is a further complicating factor for some piers. Equilibrium scour depth may not be attained during a single, scouring flow event. Moreover, with time, channel conditions in the vicinity of the bridge waterway may alter.

8.3.2 Scour Processes

1. **Flow Field Factors.** The following practical aspects of bridge-waterway hydraulics affect the flow field at pier and require further research:

   i. ***Flow field at piers of multiple components***. Many common pier forms comprise multiple components (e.g., pier column, pile cap, piles or spread footing) that may complicate the pier flow field, and thereby increase the uncertainty of scour depth estimation, in comparison with scour at cylindrical piers. In support of the use of pier-shape factors with the estimation equation, and further laboratory tests on scour at such piers, it is useful to understand how the main flow features at a cylindrical pier alter for common pier forms.

   ii. ***Pier flow field interactions with bridge features.*** The pier flow field can be altered substantially by flow around other features of a bridge waterway. Flow around other features, notably an abutment, may substantially alter the structure of the pier flow field, or may alter approach-flow orientation and magnitude. Two interactions are common for piers, and require additional research:

a. Abutment proximity. A pier within the influence region of an abutment flow field is affected by the abutment flow field, to the extent that scour at the pier can be primarily attributable to the abutment flow field. Though some work on abutment proximity has been done, further research is needed to determine how the distance of abutment influence extends for varying conditions of abutment length and channel geometry, and then to use this information for use in design estimation of scour depth at piers near abutments; and,
b. Bridge-deck submergence. The useful work done on the influence of deck submergence on pier scour under clear-water scour conditions should be extended for live-bed conditions.

iii. ***Influence of debris rafts, and ice accumulations***, on pier flow field, and thereby on scour depth. The final report for NCHRP Project 24-26 (Lagasse et al. 2010) states the following topics require further research:

a. Adaptation of the design method for use with the design methodology recommended in the present project;
b. Undertaking laboratory tests extending findings from debris accumulation at simple cylindrical piers to common pier forms comprising multiple components, and with varying approach-flow angle; and,
c. Undertaking further tests with a broader variation of debris raft geometries, including rafts that extend upstream of the pier (as with an ice cover), and debris rafts extending across more than one pier. This factor is directly similar to scour at a submerged bridge deck.

2. **Erosion of foundation material**. The essential concern is the pier-site presence of an armouring layer of material overlaying strata of more erodible material, which if exposed to the pier flow field would produce a deeper scour than if the pier foundation were in a single layer of material. The following factors complicate scour-depth estimation, and require further investigation:

i. ***Scour development in layered boundary material***. A fairly frequent pier site complication concerns the presence of a gravel stratum overlaying strata of sands or finer gravels. Though this situation has been researched, further research is needed especially regarding the following aspects:

a. Investigation and documentation of field situations; and,
b. Development of design scenarios indicating a possible potential maximum scour depth associated with scour in these conditions.

ii. ***Scour in layered weak rock***. This site complication relates to item 2 in Section 7.2.

a. Investigation and documentation of field situations; and,

b. Confirming whether or not pier scour in various rock types attains greater potential maximum depths than scour in cohesionless sediments.

iii. ***Scour at piers on a vegetated or grassy floodplain***. A vegetated or grassy surface may extend the clear-water condition scour beyond the critical entrainment condition for the foundation soil or sediment beneath. However, once flow strips the vegetation from the foundation surface, scour occurs more widely around the pier as well as at the pier. Several issues require resolution:

a. Investigation and documentation of field situations; and,
b. Confirming whether or not pier scour in grassy floodplains attains greater potential maximum depths than scour in cohesionless sediments.

Table 8-1 Priority range for research needs (Adapted from NCHRP Project 24-8 (Parola et al. 1996))

Critical Priority • Research that is necessary for improved solutions to widespread and costly problems • Research that is necessary to ensure public safety • Research that is essential to maintain an effective research agenda • Development of guidelines for the application of methodologies
High Priority • Research that is judged to have a high benefit-to-cost ratio • Research that is applicable to a large number of bridges • Research that will lead to substantial long-term improvement in scour prediction methodology
Medium Priority • Research on problems that relate to large numbers of bridges, but where benefits of a solution cannot be estimated • Research that will increase the accuracy of predictive procedures in the long-term, but without immediate impact
Low Priority • Research on problems specific to a small number of bridges • Research on problems that infrequently cause bridge damage

Table 8-2 Research topics and priorities for single-column piers

Topics	Priority
DESIGN ISSUES	
1. Estimation of potential maximum scour depth for piers in the changeover range from transition- to wide-pier categories, especially for live-bed conditions	Critical
2. Combination of pier scour and contraction scour, especially for cohesive soils	High
3. Procedure for ascertaining probability that potential maximum scour will occur	High
PHYSICAL PROCESSES	
1. Flow field	
i. Systematic changes in flow field for piers in the transition- to wide-pier categories, determining potential maximum scour depth	High
ii. Influence of bedforms on flow field during live-bed scour	Medium
2. Erosion of Foundation Material	
i. Changes in erosion processes and scour form with changes from transition to wide-pier categories. Especially important are alignment and shape factors for use in scour-depth estimation	High
ii. Erosion of non-cohesive material by turbulence structures	Medium
iii. Erosion of cohesive material by turbulence structures	Medium
iv. Temporal development of scour for the transition-pier category of scour, and for non-cohesive and cohesive foundation materials	Medium

Table 8-3 Research topics and priorities for considerations complicating scour-depth estimation

Topics	Priority
DESIGN ISSUES – COMMON PIER FORMS	
1. Reliable shape and alignment factors for common pier forms. Adapt the Sheppard-Melville method, notably as expressed by Eq. (7.2), for use with common pier forms	Critical
2. A body of scour data for common piers. The data can be used with the full Sheppard-Melville method	Medium
3. A procedure for ascertaining probability that potential maximum scour will occur	Medium
DESIGN ISSUES – COMPLEXITIES	
1. Accounting for bridge-deck submergence when estimating pier scour depth	High
2. Pier scour in layered sediment, particularly when the top layer acts as an armoring layer; and, in vegetated floodplains	High
3. Scour-depth estimation for selected common pier forms should account for debris accumulation	Medium
DESIGN ISSUES – COMPLEX PIER FORMS	
1. Scour depth estimation for piers of unusual or intricate geometry (development of hybrid approach)	Medium
PHYSICAL PROCESSES	
1. Flow field	
i. Main features of flow field at common pier shapes with multiple components (notably – column, pile-cap, piles)	High
ii. Pier flow field interaction with bridge components, especially a bridge deck	High
iii. Influence of debris rafts or ice accumulations	Medium

iv. Flow field interaction with channel features	Medium
2. Erosion of Foundation Material	
The erosion processes associated with the flow-field complexities mentioned above	Medium

CHAPTER 9
CONCLUSIONS

9.1 Introduction
This chapter presents the evaluation's main conclusions, which link to the following considerations outlined in Section 1.4:

1. The flow field and the potential maximum scour depth, at a pier scale, change in accordance with three variables – effective pier width, flow depth, and erodibility of the foundation material in which the pier is sited. Of these variables, effective pier width and flow depth are of prime importance, because they determine the overall structure of the flow field. Effective pier width, a_*, embodies pier form and alignment relative to approach flow velocity. For non-cohesive foundation material (silts, sand, and gravel) erodibility is expressible in terms of a representative particle diameter.

2. To understand pier-scour processes and develop reliable relationships for design estimation of scour depth, it is necessary to comprehend the main flow-field features driving scour, and how the features may adjust in importance with varying pier sizes and shapes, and flow conditions. The flow field differs substantially for the narrow-pier and wide-pier categories of pier scour, with the transition-pier category being intermediate between them. These categories can be defined approximately as

   - Narrow piers ($y/a_* > 1.4$), scour typically deepest at the pier face
   - Transition piers ($0.2 < y/a_* < 1.4$), scour depth deepest at pier face, and is influenced by y/a_*
   - Wide piers ($y/a_* < 0.2$), for which scour typically is deepest at the pier flank. It is relatively rare, for design scour estimation, that piers are in this category

   The foregoing categories can be expressed in terms of *y/a*. Pier-scour literature normally expresses data trends in terms of a dependent variable (notably, flow depth or bed particle diameter) relative to a pier's constructed width, *a*.

3. Because considerable uncertainty attends flow and boundary material at bridge waterways, design prudence requires estimation of a potential maximum scour depth, rather than scour-depth prediction. Potential maximum scour depth is the greatest scour depth attainable for a given pier flow field, and can be determined using the primary variables named in item 2. Lesser scour depths result as additional variables are considered, but the uncertainties associated with the variables diminish the estimation reliability. Besides the uncertainties associated with foundation material erodibility, the temporal development of scour entails significant uncertainty. Prediction (not design estimation) of scour depth for most pier sites usually involves a high level of uncertainty.

Several factors alter pier flow fields and complicate design estimation of pier scour. Factors affecting pier flow field include flow influences exerted by increased complexity of pier geometry, adjoining bridge components (abutment or submerged bridge deck), debris or ice accumulation, and channel morphology. Factors affecting boundary erosion include uncertain erosion characteristics of material (clay, rock), layering of boundary material, and protective vegetation. Figures 1-2 and 1-3, shown at the report's outset, give a sense of the variability of pier sites, and infer the potentially large number of variables involved in pier scour.

9.2 Conclusions

The evaluation leads to the following main conclusions regarding the six objectives given in Section 1.2:

1. The literature review conducted for the evaluation shows that, since 1990, substantial advances have been made in understanding pier-scour processes, and predicting scour depth. In particular, the following aspects of pier scour have advanced:

   i. The roles of variables and parameters defining pier scour processes;
   ii. The leading predictive methods for scour-depth prediction better reflect parameter influences;
   iii. Knowledge regarding pier scour in clay and weak rock;
   iv. Insight into pier-site complications caused by debris and ice, and by interaction with bridge components (abutment and bridge deck); and,
   v. The capacity of numerical modeling to reveal the three-dimensional and unsteady flow field at piers in ways that laboratory work heretofore has been unable to provide.

   These advances address research problems identified in NCHRP Project 24-8, *Scour at Bridge Foundations: Research Needs* (Parola et al. 1996), such as scour at wide and skewed long piers, and the scour effects of debris accumulation. They also address aspects of pier scour not envisioned for NCHRP 24-8, such as the roles of turbulence structures within the pier flow field, three-dimensional numerical modeling, and scaling issues in the conduct of hydraulic models of pier scour.

   However, further significant research has yet to be done in each of these areas.

2. The current state of knowledge on pier-scour processes is such that the knowledge points indicated in Objective 2 are addressed in Chapter 3, with elaboration in the references cited. The ensuing aspects of pier scour remain insufficiently understood:

   i. The pier flow field, especially how it systematically changes with variations of the two primary length scales (effective pier width and flow depth). In other words, more work is needed to define the systematic changes in the flow fields associated with the narrow- , transition-, and wide-pier categories of pier scour. Figures 3-4, 3-5, and 3-6 illustrate the flow fields associated with the three categories, respectively;

ii. Scour of boundary materials whose erosion characteristics are not adequately understood (some soils, rock). However, existing reliable data indicate that scour depths in cohesive soils and weak rock do not exceed those in cohesionless material;

iii. Quantification of factors further complicating pier flow field (notably debris or ice accumulation, proximity of bridge components, channel morphology) and erodibility of foundation material (especially layered material where the surface layer protects lower layers); and,

iv. Temporal development of pier-scour depth.

3. The evaluation (Chapter 4) outlines the well-understood relationships between scour depth and significant parameters. Extensive use is made of Melville and Coleman (2000) in delineating the relationships, with more recent information cited from other sources. Notable examples of recent information are with regard to similitude in hydraulic modeling of turbulence structures; scour at large piers; erosion of clay and, to a lesser extent, erosion of rock; and, the purported influences of suspended sediment.

   A group of primary parameters are identified in Section 4.3. They define the structure and geometric scale of the pier flow field, and therefore determine the potential maximum scour depth. The secondary parameters characterize scour-depth sensitivities within the geometric scale limit, and normally lead to scour depths less than the potential maximum scour depth. The values of the secondary parameters are subject to considerable uncertainty at pier sites.

   The primary parameters are –

   y/a, which indicates the geometric scale of the pier flow field in terms of approach-flow depth and pier width (in a vertical cross-sectional plain transverse to the pier, and a plain streamwise to the pier)

   a/D, which relates the length scales of pier width and median diameter of bed particle

   Ω, a/b, and θ, which define pier face shape, aspect ratio of pier cross-section (face width/length), and approach flow alignment to pier, respectively. These parameters may be merged with pier width, a, to form the compound variable a_* = effective pier width. These parameters may be merged with pier width, a, to form the compound variable a^* = effective pier width. It can be useful to express the two length-scale parameters as y/a^* and a^*/D

The evaluation also explains the limiting extents to which parameter influences can be isolated from each other. Some variables exert multiple influences. Consequently, a limit is soon reached with the approach of formulating a predictive method based on a semi-empirical assembly of parameter influences.

4. An important conclusion drawn from the evaluation (Chapter 5) is the need to transition from the present semi-empirical method for design estimation of scour depth used in HEC-18[10] (Richardson and Davis 2001) to a new method better reflecting the relationships between the primary variables and the potential maximum scour depth at a pier. A new method is proposed, the semi-empirical, Sheppard-Melville method as advanced during NCHRP Project 24-32 (Sheppard et al. 2011).

   The Sheppard-Melville method better reflects flow-field changes and thereby scour processes, as expressed in terms of the parameters of primary importance, and therefore is more readily extended to the transition- and wide-pier categories; i.e., it more expressly includes the length scales, y/a_* and a_*/D, and includes the flow intensity parameter, V/V_C. Additionally, it is the method developed (with funding for NCHRP 24-32) in an effort to account for flow-field adaptation to variations of y/a and a/D. A useful aspect of the Sheppard-Melville method is that it can be simplified to reflect potential maximum scour depth associated with the three pier flow-field categories (narrow, transition and wide). A weakness in the Richardson and Davis (2001) method is that it does not.

   Full use of the Sheppard-Melville method for predicting scour depth presently requires a modicum of further research (outlined in Chapter 8) regarding prediction of scour depth during live-bed scour at piers in the transition-pier category, and to a certain extent in the wide-pier category.

   The Richardson and Davis (2001) method has been in use for the past several decades, and has been refined over time. It is well calibrated for estimating scour depth for piers in the narrow-pier category of scour, and into the transition-pier category. Its scour depth estimates for these applications concur reasonably well with those obtained using the Sheppard-Melville (NCHRP 24-32) method, as indicated in Appendix A. Nonetheless, the method is shown not to reflect scour processes as well as does the Sheppard-Melville method.

5. Due to the limits inherent in semi-empirical formulations of pier-scour depth, the evaluation proposes the use of a structured methodology for design estimation of pier-scour depth. The methodology, outlined in Chapter 6, addresses four levels of pier-site complexity (Table 6-1). As pier-site complexity increases a graduated shift occurs from design reliance on a semi-empirical method (Sheppard-Melville) to hydraulic modeling possibly aided by numerical modeling, as indicated in Table 9-1.

[10] And adopted in AASHTO policies and procedures

The methodology enables the designer to account for the scour processes, yet also recognize the limits of existing semi-empirical methods for scour-depth estimation. The leading semi-empirical methods (Sheppard-Melville, Richardson and Davis) for scour-depth prediction were developed using data for simple pier forms, and adapted for common pier forms. The accuracy of the methods reduces as pier form complexity increases.

Table 9-1 Summary of proposed structured design methodology

Pier Design Complexity (Pier Form and/or Pier Site)	Design Method
i. Simple single-column pier forms (narrow- and transition-pier categories)	**Semi-empirical method:** Transition from current HEC-18 method (Richardson and Davis 2001) to Eq. (7.1) as simplified from Sheppard et al. (2011), NCHRP Project 24-32
ii. Common pier forms (piers with multiple components; e.g., column, pile cap, pile group)	**Semi-empirical method with effective pier-shape and alignment factors** Consider hydraulic model to validate scour-depth estimate
iii. Common pier forms in complicating situations (debris or ice accumulation, abutment proximity, bridge-deck submergence)	**Empirical method combined with procedure to address scour contribution of site complication** Consider hydraulic model to validate scour-depth estimate
iv. Complex or unusual pier situations (where reliable method or information on parameter influences does not exist; e.g., scour for wide-pier category)	**Hydraulic modeling, possibly aided by numerical modeling** An approximate scour-depth estimate may be obtained using an empirical method suitably developed for the wide-pier category

6. The following specific recommendations are made with respect to the updating of AASHTO's manuals:

   i. Adopt the structured design methodology described in Chapter 7; and,
   ii. Transition from the Richardson and Davis (2001) method to the Sheppard-Melville method (Sheppard et al. 2011), given in simplified form as Eq. (1) above. The former method inadequately reflects scour processes, though is well-adapted empirically to scour data. The latter method better reflects scour processes. A modicum of further research is needed to complete the development of the Sheppard-Melville method. The present evaluation has occurred at a transitional period when it is recognized that the present design method for simple and common pier forms should be replaced, but a satisfactory replacement method is not fully completed.

The present version of HEC-18 should recommend the use of the current method (Richardson and Davis, 2001) and the Sheppard-Melvile method. In due course, the latter method should replace the former one.

7. Tables 8-2 and 8-3 list prioritized research needs for design methodology and scour processes, respectively. In a similar manner as used for NCHRP 24-8 (Parola et al. 1996), the priorities are assigned as critical, high, medium, and low (definitions in Table 8-1).

   For design methodology development, two research topics are of critical priority:

   i. Estimation of potential maximum scour depth for piers in the changeover range from transition- to wide-pier categories, especially for live-bed conditions; and,
   ii. A reliable method for estimating potential maximum scour at piers in the wide-pier category.

   For understanding scour processes, no research topics are identified as critical priority, though several are of high priority, all of which concern improved understanding of how site complications affect pier flow field:

   i. Debris accumulations;
   ii. Flow field at common pier shapes with multiple components (notably, column, pile cap, piles);
   iii. Flow field interaction with bridge components, such as a bridge deck or abutment; and,
   iv. Flow field interaction with channel features.

   In terms of longer-range research development, a transition underway is recent advances in experimental and numerical techniques used to investigate bridge scour processes that can capture the dynamics of the main turbulent coherent structures (e.g., horseshoe vortex system at the base of the pier, eddies shed in the separated shear layers, large-scale rollers in the wake behind the pier) affecting pier scour. The recent advances include Particle Image Velocimetry (PIV) and Laser Doppler Velocimetry (LDV) based experimental studies and Large Eddy Simulation (LES) and Detached Eddy Simulation (DES) numerical investigations using fully three-dimensional non-hydrostatic methods. These experimental and computational approaches promise new insight into understanding the fundamental physics of the flow and sediment transport processes at bridge piers and can lead to the development of more accurate relationships to predict local scour depth.

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APPENDIX
SCOUR-DEPTH ESTIMATION METHODS

A.1 INTRODUCTION

This appendix briefly evaluates existing methods for estimating scour depth. It includes methods proposed prior to 1990 though focussing especially on methods developed since 1990. The evaluation considers each method's capacity to express the scour influences of the primary parameters indentified in Chapter 4, and includes a comparison of scour depths predicted for a range of hypothetical pier conditions. The outcome of the evaluation is to identify the two methods considered further in Chapter 6: the Richardson and Davis (2001) method as presently used in HEC-18, and the Sheppard-Melville method as developed recently in NCHRP Project 24-32.

A.2 EVALUATION CRITERIA

The evaluation of the scour-estimation methods considers several criteria:

1. Adequacy in addressing scour processes as represented by the primary dimensionless parameters identified in Chapter 4;
2. Limitations of design relationships (ranges of results in hypothetical applications);
3. Categorization and acceptability of laboratory experiments and research techniques (experimental duration, variety of particle sizes and types of sediments, realistic geometries and scales, characterization of flow field, degree of idealization);
4. Verification of design relationships with field and other lab data; and,
5. Applicability and ease of use for design practice (e.g., in AASHTO manual).

A.3 EXPRESSION OF PARAMETER INFLUENCES

Numerous equations have been proposed for estimation of the depth of local scour at bridge piers. A selection of some of the better known equations is given in Table A-1. The equations are listed chronologically. Table A-2 indicates the parameter influences represented in each of the methods.

Table A-1 A chronological listing of pier scour equations

Reference	Equation	Standard format (for comparison)	Notes
Inglis (1949)	$\frac{y_s + y}{a} = 2.32\left[\frac{q^{2/3}}{a}\right]^{0.78}$	$\frac{y_s}{a} = 2.32\left[\frac{q^{2/3}}{a}\right]^{0.78} - \frac{y}{a}$	q = average discharge intensity upstream from the bridge (m^2/s)
Ahmad (1951)	$y_s + y = K_s q_2^{2/3}$	$\frac{y_s}{a} = K_s \frac{q_2^{2/3}}{a} - \frac{y}{a}$	q_2 = local discharge intensity in contracted channel (m^2/s)
Laursen (1958)	$\frac{a}{y} = 5.5\frac{y_s}{y}\left[\left(\frac{y_s}{11.5y}+1\right)^{1.7} - 1\right]$	$\frac{y_s}{a} \approx 1.11\left(\frac{y}{a}\right)^{0.5}$	applies to live-bed scour

Chitale (1962)	$\frac{y_s}{y} = 6.65Fr - 0.51 - 5.49Fr^2$	$\frac{y_s}{a} = \left[6.65Fr - 0.51 - 5.49Fr^2\right]\left(\frac{y}{a}\right)$	applies to live-bed scour
Laursen (1963)	$\frac{a}{y} = 5.5\frac{y_s}{y}\left[\frac{\left(\frac{y_s}{11.5y}+1\right)^{7/6}}{\left(\frac{\tau_1}{\tau_c}\right)^{0.5}} - 1\right]$	At the threshold condition, $\frac{y_s}{a} \approx 1.34\left(\frac{y}{a}\right)^{0.5}$	applies to clear-water scour τ_1 = grain roughness component of bed shear τ_c = critical shear stress at threshold of motion
Larras (1963)	$y_s = 1.05K_sK_\theta a^{0.75}$	$\frac{y_s}{a} = 1.05K_sK_\theta a^{-0.25}$	
Neill (1964)	$y_s = 1.5a^{0.7}y^{0.3}$	$\frac{y_s}{a} = 1.5\left(\frac{y}{a}\right)^{0.3}$	the factor 1.5 applies for circular piers
Breusers (1965)	$y_s = 1.4a$	$\frac{y_s}{a} = 1.4$	derived from data for tidal flows
Blench (1969)	$\frac{y_s + y}{y_r} = 1.8\left(\frac{a}{y_r}\right)^{0.25}$	$\frac{y_s}{a} = 1.8\left(\frac{y_r}{a}\right)^{0.75} - \frac{y}{a}$	y_r=regime depth $=1.48(q^2/F_B)^{1/3}$ where $F_B=1.9(D)^{0.5}$, d in mm and q in m^2/s
Shen et al. (1969)	$y_s = 0.000223\left(\frac{Va}{\nu}\right)^{0.619}$	$\frac{y_s}{a} = 2.34\left(\frac{y}{a}\right)^{0.381} Fr^{0.619} y^{-0.06}$	standard format equation is given for kinematic viscosity of water, $\nu = 1 \times 10^6$ m^2/s
Coleman (1971)	$\frac{V}{\sqrt{2gy_s}} = 0.6\left(\frac{V}{a}\right)^{0.9}$	$\frac{y_s}{a} = 0.54\left(\frac{y}{a}\right)^{0.19} Fr^{1.19} y^{0.41}$	
Hancu (1971)	$\frac{y_s}{a} = 2.42\left(\frac{2V}{V_c} - 1\right)\left(\frac{V_c^2}{ga}\right)^{1/3}$	$\frac{y_s}{a} = 2.42\left(\frac{y}{a}\right)^{1/3} Fr^{2/3}$	$(2V/V_c-1) = 1$ for live-bed scour Standard format equation is given for threshold condition
Neill (1973)	$y_s = K_s a$	$\frac{y_s}{a} = K_s$	K_s=1.5 for round-nosed and circular piers; K_s=2.0 for rectangular piers
Breusers et al. (1977)	$\frac{y_s}{a} = f\left(\frac{V}{V_c}\right)\left[2.0\tanh\left(\frac{y}{a}\right)\right]K_sK_\theta$	$\frac{y_s}{a} = 2.0\tanh\left(\frac{y}{a}\right)K_sK_\theta$	$f(V/V_c)=0$ $\quad V/V_c \le 0.5$ $=(2V/V_c-1)$ $\quad 0.5 < V/V_c < 1$ $=1$ $\quad V/V_c > 1$ Equation (*) given at the threshold condition
Jain and Fischer (1980)	$\frac{y_s}{a} = 1.86\left(\frac{y}{a}\right)^{0.5}(Fr - Fr_c)^{0.25}$	$\frac{y_s}{a} = 1.86\left(\frac{y}{a}\right)^{0.5}$	$Fr=V/(gy)^{0.5}$ $Fr_c=V_c/(gy)^{0.5}$ Standard format equation is given at the threshold condition
Jain (1981)	$\frac{y_s}{a} = 1.84\left(\frac{y}{a}\right)^{0.3} Fr_c^{0.25}$	$\frac{y_s}{a} = 1.84\left(\frac{y}{a}\right)^{0.3}$	Standard format equation is given at the threshold condition
Chitale (1988)	$y_s = 2.5a$	$\frac{y_s}{a} = 2.5$	
Melville and Sutherland (1988)	$\frac{y_s}{a} = K_I K_y K_d K_s K_\theta$	$\frac{y_s}{a} = 2.4 K_y K_d K_s K_\theta$	For an aligned pier, $y_{s)max} = 2.4K_sK_da$ Standard format equation is given at threshold condition
Froehlich (1988)	$\frac{y_s}{a} = 0.32K_sFr^{0.2}\left(\frac{a_p}{a}\right)^{0.62}\left(\frac{y}{a}\right)^{0.46}\left(\frac{a}{D_{50}}\right)^{0.08} + 1$	$\frac{y_s}{a} = 0.32K_sFr^{0.2}\left(\frac{a_p}{a}\right)^{0.62}\left(\frac{y}{a}\right)^{0.46}\left(\frac{a}{D_{50}}\right)^{0.08} + 1$	a_p = projected width of pier $Fr = V/\sqrt{gy}$

May and Willoughby (1990)	$y_s = 2.4 f_s\left(\frac{y_s}{y_{sc}}\right)\left(\frac{y_{sc}}{y_{sm}}\right)$		For circular cylinder: $f_s = 1.0$ $\frac{y_s}{y_{sc}} = 1 - 3.66\left(1 - \frac{V}{V_c}\right)^{1.76}$ $\quad 0.52 \le \frac{V}{V_c} \le 1.0$ $= 1.0$ $\quad \frac{V}{V_c} > 1.0$ $\frac{y_{sc}}{y_{sm}} = 0.55\left(\frac{y}{a}\right)^{0.6}$ $\quad \frac{y}{a} \le 2.7$ $= 1.0$ $\quad \frac{y}{a} > 1.0$
Breusers and Raudkivi (1991)	$\frac{y_s}{a} = 2.3 K_y K_s K_d K_\sigma K_\theta$	$\frac{y_s}{a} = 2.3 K_y K_s K_d K_\sigma K_\theta$	For an aligned pier, $y_{smax} = 2.3 K_s K_d K_\sigma a$
Richardson and Davis (1995)	$\frac{y_s}{a} = 2 K_s K_\theta K_3 K_4 \left(\frac{y}{a}\right)^{0.35} Fr^{0.43}$	$\frac{y_s}{a} = 2 K_s K_\theta K_3 K_4 \left(\frac{y}{a}\right)^{0.35} Fr^{0.43}$	K_3 = factor for mode of sediment transport K_4 = factor for armouring by bed material $y_{smax} = 2.4b$ $\quad Fr \le 0.8$ $y_{smax} = 3b$ $\quad Fr > 0.8$
Gao et al.(1993)	$y_s = 0.46 K_\varsigma a^{0.60} y^{0.15} D_{50}^{-0.07}\left[\frac{V - V_c}{V_c - V_c}\right]^\eta$ $V_c = \left(\frac{y}{a}\right)^{0.14}\left[17.6\left(\frac{\rho_s - \rho}{\rho}\right) D_{50} + 6.05x10^{-7}\left(\frac{10 + y}{D_{50}^{0.72}}\right)\right.$ $V_c = 0.645\left(\frac{D_{50}}{a}\right)^{0.053} V_c$ where y_s, a, y, D_{50}, V, V_c, V_c' are in S.I. units.	$\frac{y_s}{a} = 0.46 K_\zeta \left(\frac{y}{a}\right)^{0.4}\left(\frac{y}{D_{50}}\right)^{0.07} y^{-0.32}$	V_c'=incipient velocity for local scour at a pier K_ζ=shape and alignment factor η=1 for clear-water scour <1 for live-bed scour i.e., $\eta = \left(\frac{V_c}{V}\right)^{9.35 + 2.23 \log D_{50}}$ where D_{50} is in S.I. units Standard format equation is given for threshold condition
Ansari and Qadar (1994)	$y_s = 0.86 a_p^{3.0}$ $\quad a_p < 2.2\,m$ $y_s = 3.60 a_p^{0.4}$ $\quad a_p > 2.2\,m$	$\frac{y_s}{a_p} = 0.86 a_p{}^2$ $\quad a_p < 2.2\,m$ $\frac{y_s}{a_p} = 3.60 a_p{}^{-0.6}$ $\quad a_p > 2.2\,m$	a_p = projected width of pier
Wilson (1995)	$\frac{y_s}{a^*} = 0.9\left(\frac{y}{a^*}\right)^{0.4}$	$\frac{y_s}{a^*} = 0.9\left(\frac{y}{a^*}\right)^{0.4}$	a^* = effective width of pier

Melville (1997)	$y_s = K_{yb} K_I K_d K_s K_\theta$	$y_s = K_{yb} K_d K_s K_\theta$	$K_{yb} = 2.4a$ $y/a > 1.4$ $K_{yb} = 2(ya)^{0.5}$ $0.2 \le y/a \le 1.4$ $K_{yb} = 4.5y$ $y/a < 0.2$ Standard format of equation is given at the threshold condition $K_I = \frac{V-(V_{lp}-V_{c)}}{V_c}$ $if\ \frac{V-(V_{lp}-V_{c)}}{V_c} < 1.0$ $K_I = 1$ $if\ \frac{V-(V_{lp}-V_{c)}}{V_c} \ge 1.0$ $K_D = 0.57 \log_{10}\left(2.24\frac{a}{D_{50}}\right)$ $if\ \frac{a}{D_{50}} \le 25$ $K_D = 1$ $if\ \frac{a}{D_{50}} > 25$
Richardson and Davis (2001)	$\frac{y_s}{a} = 2K_s K_\theta K_3 K_4 K_w \cdot \left(\frac{y}{a}\right)^{0.35} Fr^{0.43}$	$\frac{y_s}{a} = 2K_s K_\theta K_3 K_4 K_w \cdot \left(\frac{y}{a}\right)^{0.35} Fr^{0.43}$	K_3 = factor for mode of sediment transport K_4 = factor for armouring by bed material K_w = factor for very wide piers after Johnson and Torrico (1994) y_{smax}=2.4a Fr≤0.8 y_{smax}=3a Fr>0.8
Briaud et al. (2004)	$y_{s\max} = 0.18 K_w K_{sp} K_\theta \left(\frac{aV}{\nu}\right)^{0.6}$	$K_w = 0.85\left(\frac{y}{a}\right)^{0.34}$ if $y/a < 1.62$ $K_w = 1$, if $y/a > 1.62$	Developed for scour of clay boundaries, but claimed suitable for alluvium also K_w = a flow depth factor K_{sp} = a pier shape factor (limited to cylindrical piers) K_θ = an approach-flow factor

Miller and Sheppard (2002)	$\frac{y_s}{a^*} = 2.5 f_1\left(\frac{y}{a^*}\right) f_2\left(\frac{V}{V_c}\right) f_3\left(\frac{a^*}{D_{50}}\right)$ $if\ 0.47 < \frac{V}{V_c} < 1.0$ $\frac{y_s}{a^*} = f_1\left(\frac{y}{a^*}\right)\left[2.2\left(\frac{\frac{V}{V_c}-1}{\frac{V_{lp}}{V_c}-1}\right) + 2.5\left(\frac{\frac{V_{lp}}{V_c}-\frac{V}{V_c}}{\frac{V_{lp}}{V_c}-1}\right) f_3\left(\frac{a^*}{D_{50}}\right)\right]$ $if\ 1 < \frac{V}{V_c} < \frac{V_{lp}}{V_c}$ $\frac{y_s}{a^*} = 2.2 f_1\left(\frac{y}{a^*}\right)$ $if\ \frac{V}{V_c} > \frac{V_{lp}}{V_c}$ $f_1 = \tanh\left[\left(\frac{y}{a^*}\right)^{0.4}\right]$ $f_2 = \left\{1 - 1.75\left[\ln\left(\frac{V}{V_c}\right)\right]^2\right\}$ $f_3 = \left[\frac{\left(\frac{a^*}{D_{50}}\right)}{0.4\left(\frac{a^*}{D_{50}}\right)^{1.2} + 10.6\left(\frac{a^*}{D_{50}}\right)^{-0.13}}\right]$	
Kothyari et al. (2007)	$\frac{y_s}{a} = 0.0068\left(\frac{y}{a}\right)^{1/3}\left(\frac{D_{86}}{D_{16}}\right)^{0.5} Fr_d^{1/5} \log(t/t_R)$ where $t_R = (ya^2)^{1/3} / \{(D_{86}/D_{16})^{1/6}[(g(\rho_s - \rho)/\rho))D_{50}]^{1/2}\}$ and $Fr_d = \frac{V}{\left(\frac{g(\rho_s - \rho)D_{50}}{\rho}\right)^{1/2}}$	

Table A-2 Dimensionless parameters included in the selected pier scour equations (The table is divided to indicate methods proposed since 1990)

Reference	*y/a*	*a/D*	*Pier Shape*	*Pier Alignment*	V/V_c
Inglis (1949)					
Ahmad (1951)					
Laursen (1958)	✓				
Chitale (1962)					
Laursen (1963)	✓				
Larras (1963)			✓	✓	
Neill (1964)	✓		✓		
Breusers (1965)					
Blench (1969)	✓				
Shen et al. (1969)	✓				
Coleman (1971)	✓				
Hancu (1971)	✓				✓
Neill (1973)			✓		
Breusers et al. (1977)	✓		✓	✓	✓
Jain and Fischer (1980)	✓				✓
Jain (1981)	✓				✓
Froehlich (1988)	✓	✓	✓		
Chitale (1988)					
Melville and Sutherland (1988)	✓	✓	✓	✓	✓
Methods Proposed since 1990					
May and Willoughby (1990)	✓		✓		✓
Breusers and Raudkivi (1991)	✓	✓	✓	✓	
Gao et al.(1993)	✓	✓	✓	✓	✓
Ansari and Qadar (1994)					
Richardson and Davis (1995)	✓		✓	✓	
Wilson (1995)	✓				
Melville (1997)	✓	✓	✓	✓	✓
Richardson and Davis (2001)	✓		✓	✓	
Briaud et al. (2004)	✓		✓	✓	
Miller and Sheppard (2002)	✓	✓	✓	✓	✓
Kothyari et al. (2007)	✓				✓

Because design estimation focuses on the maximum scour depth at a pier of given structure, it is useful to recall which parameters are important for scour-depth estimation:

1. The parameters of primary importance are those pertaining to pier size (*y/a*, and *a/D*), pier shape, and pier alignment. In particular, as explained in Chapter 3. the

parameter y/a^* defines the scale of the pier flow field (a^* is effective pier width), and thereby the potential maximum scour depth;

2. The parameter V/V_c indicates the capacity of flow to erode and convey material on the approach boundary to a pier site, and affects scour depth. In terms of estimating a potential maximum scour depth it can be argued that the flow intensity parameter, V/V_c, is not a parameter of primary importance for pier sites where V/V_c, is likely to exceed unity, and the time development of scour is not an important concern. Scour at piers in most rivers develops sufficiently rapidly, and in high-flow conditions, that V/V_c is at a value producing maximum scour depth. Flow intensity can be important when flow duration is an issue; and,

3. It could be argued that when time development of scour is of concern (e.g., clear-water scour at wide piers), considerable uncertainty attends duration of V/V_c values and scour development. However over sufficient time (e.g., the design life of a bridge), scour may attain its maximum depth; and,

4. Other parameters are of lesser significance, because they do not influence potential maximum scour depth For example, sediment uniformity, σ_g, is not critical because at design flows the bed material is at, or close to, the point of being fully mobilized.

Several of the earlier equations do not include any of the important dimensionless parameters; e.g. Inglis (1949), Ahmad (1951) and Chitale (1962). Further, several of the more recent methods do not either: notably Wilson (1995), Briaud et al. (2004), Richardson and Davis (2001), and Kothyari et al. (2007). The methods by Goa et al. (1993), Melville (1997), and Miller and Sheppard (2002) include all of the dimensionless parameters listed in Table A-2. The flow-scale parameter, y/a, appears in most methods developed since 1990. Several methods do not include the influence of a/D. Some do not include V/V_c.

The methods by Breusers et al. (1977), Breusers and Raudkivi (1991), Melville and Sutherland (1988) and Melville (1997) have similar format. Melville (1997), builds upon, and is a refined version, of the earlier methods. Recently, under NCHRP Project 24-32, the Melville (1997) method is merged with the Sheppard and Miller (2006) method to form the Sheppard-Melville method. Similarly, the methods by Richardson and Davis (2001) represent development of the same basic approach (e.g., Richardson and Davis 1995). The elements of the method developed by Sheppard are presented in several publications (e.g., Miller and Sheppard 2002, FDOT 2010). Herein, Sheppard and Miller (2006) is cited as the representative publication.

A.4 COMPARISON OF SCOUR-DEPTH PREDICTIONS

This section compares the scour depth equations. It does so using the analysis already completed for NCHRP Project 24-32, whose objectives coincide in part with those of the present evaluation. The comparison applies the equations in Table A-1 to a number of hypothetical pier conditions, ranging from laboratory to prototype scales. For completeness of evaluation, all the equations discussed in the previous section are included in the plots, either individually or in combination for cases where the original equation by a certain author has been replaced or augmented by a newer equation; e.g. the Melville (1997) equation is plotted in preference to that by Melville and Sutherland (1988). To be kept in mind is that some equations were developed expressly to give upper-bound estimates of scour depth (e.g., Melville 1997), while others comprise an equation developed as a close, empirical fit of data trends (e.g., Richardson and Davis 2001). This difference is useful to keep in mind when comparing scour depths obtained from the various methods.

The results of these analyses are plotted in Figures A-1 through A-12 for all combinations of the parameter values shown in Table A-3. The equations are plotted chronologically in the figures.

Table A-3 Range of parameter values for Figures A-1 through A-12

V/V_c	*y/a*	*D (mm)*	*a(m)*
1, 3	0.33, 1, 3	0.2, 3	0.05, 1, 10

In the bar charts, negative scour depths are indicated by missing bars (i.e. they are plotted as zero scour). When viewing the plots it is useful to note that green (a = 0.05m) represents usual laboratory scale, red represents nominally typical pile sizes (a = 1.0m), while blue represents piers considered very wide (a = 10m). In the figures, V_1 is equivalent to V, and y_1 is equivalent to y.

Figures A-1 through A-3 show scour depth predictions for scenarios comprising clear-water to live-bed transition (threshold) flows (V/V_c = 1), fine sand (D = 0.2mm), and three different flow depth-to-pier width ratios (y/a = 3, 1, 0.33). Figures A-4 through A-6 are a parallel set of results for live bed conditions (V/V_c = 3). Similarly, Figures A-7 through A-12 apply to scenarios involving coarser sediment (D = 3mm).

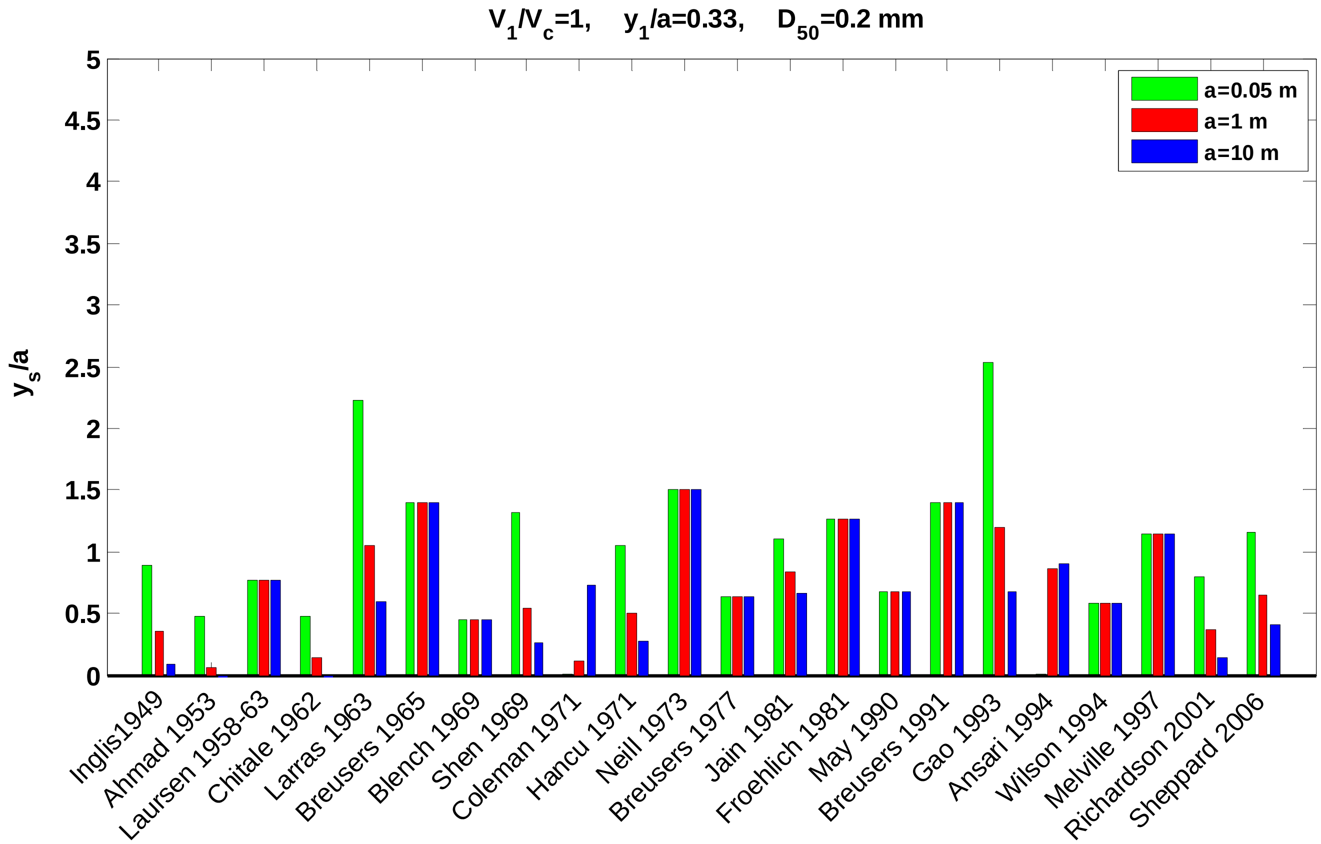

Figure A-1 Comparison of normalized local scour depth predictions using 22 different equations/methods for transition from clear-water to live-bed scour conditions. Pier width large compared to the water depth, fine sand.

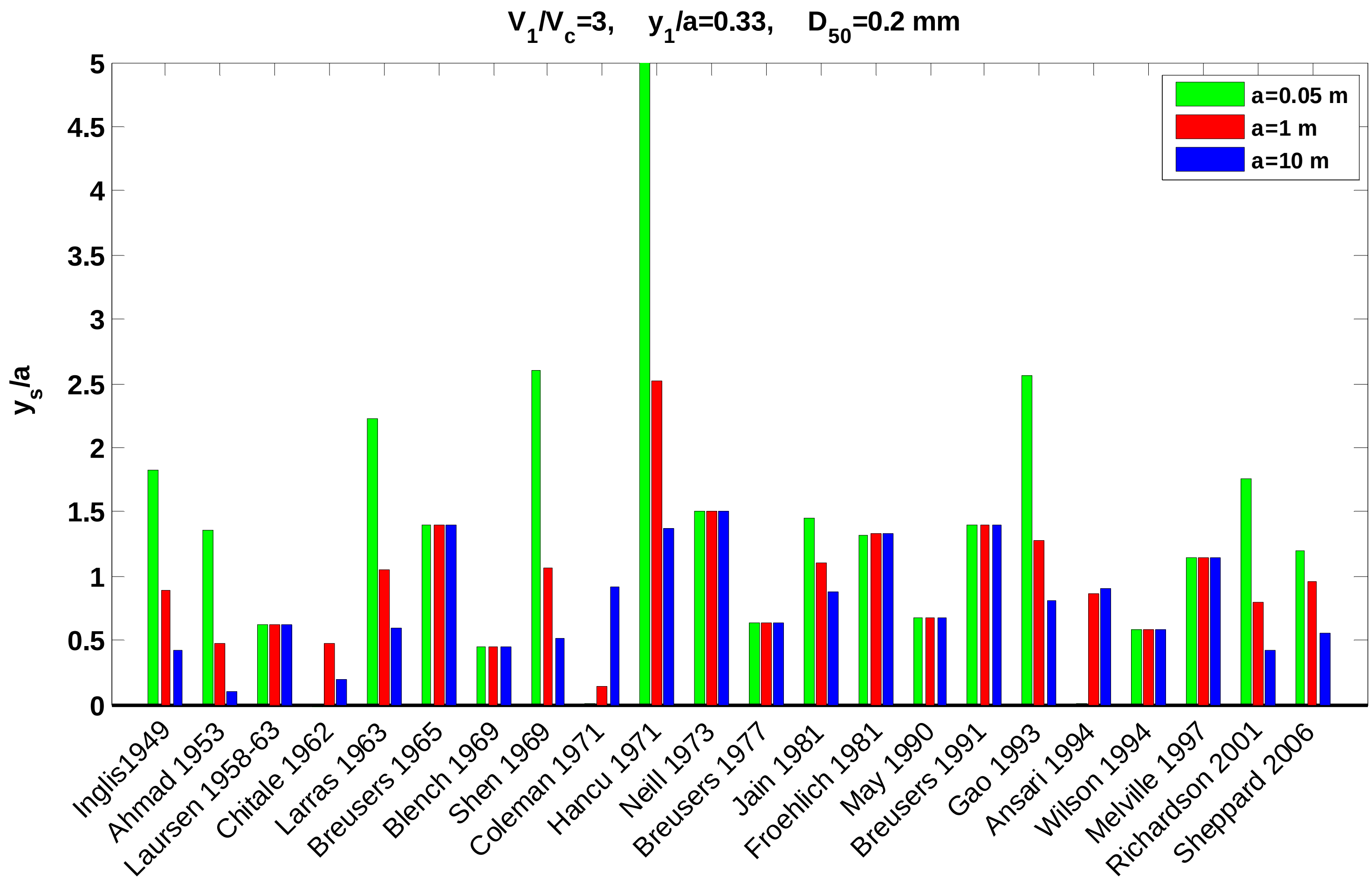

Figure A-2 Comparison of normalized local scour depth predictions using 22 different equations/methods for a particular live-bed scour condition. Pier width large compared to the water depth, fine sand.

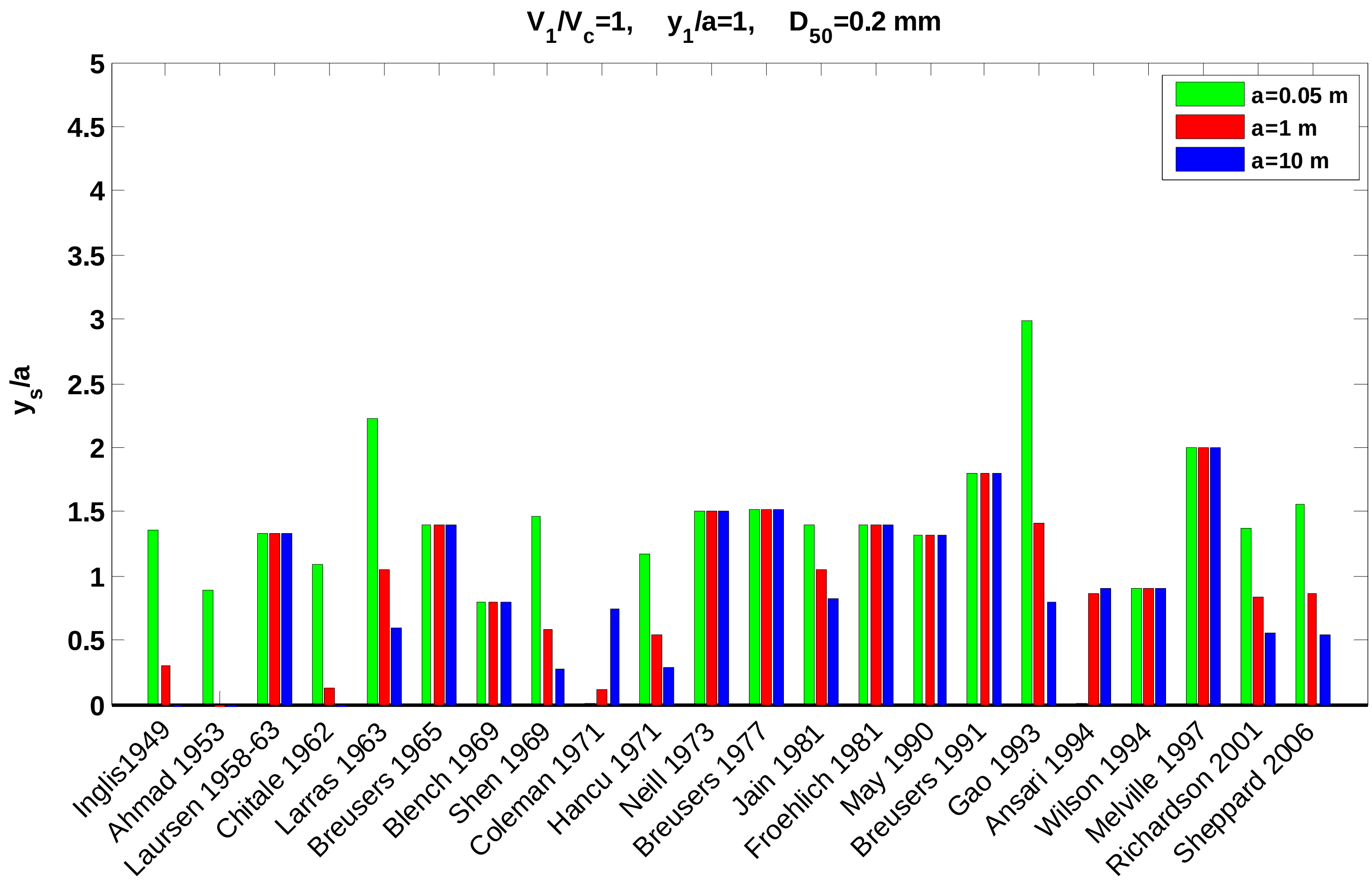

Figure A-3 Comparison of normalized local scour depth predictions using 22 different equations/methods for transition from clear-water to live-bed scour conditions. Pier width equal to water depth, fine sand

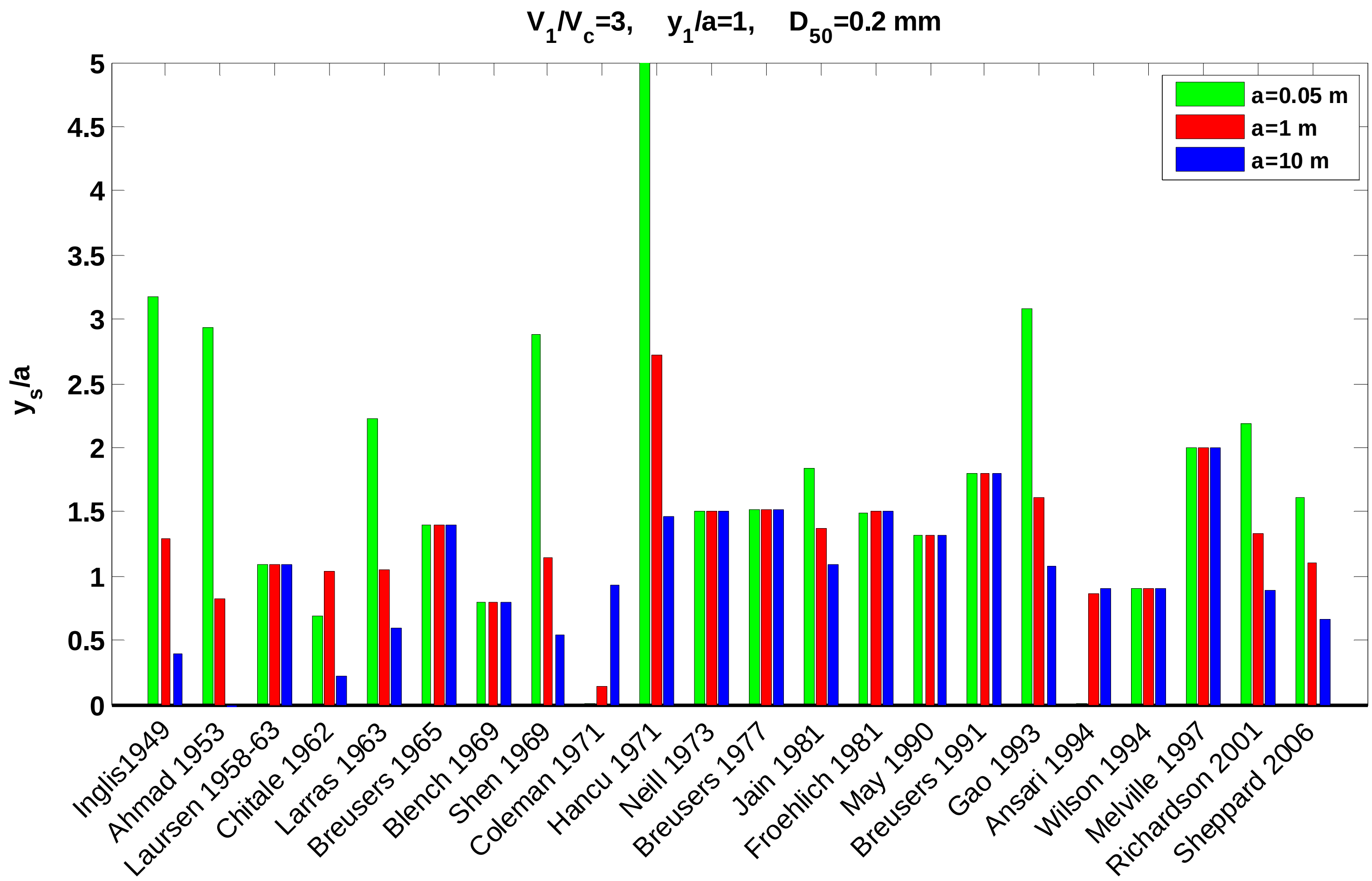

Figure A-4 Comparison of normalized local scour depth predictions using 22 different equations/methods for a particular live-bed scour condition. Pier width equal to water depth, fine sand

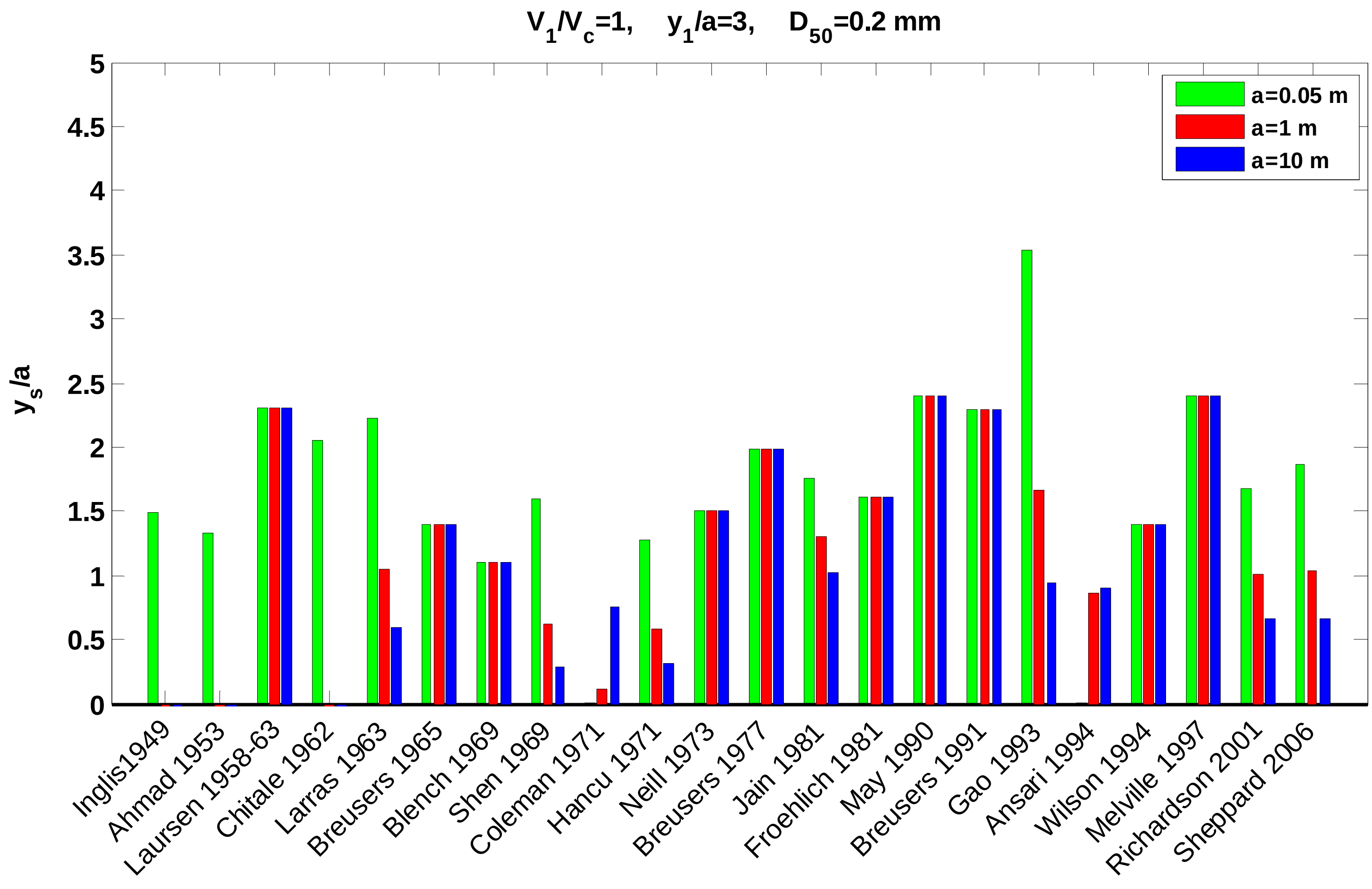

Figure A-5 Comparison of normalized local scour depth predictions using 22 different equations/methods for transition from clear-water to live-bed scour conditions. Deep water relative to pier width, fine sand

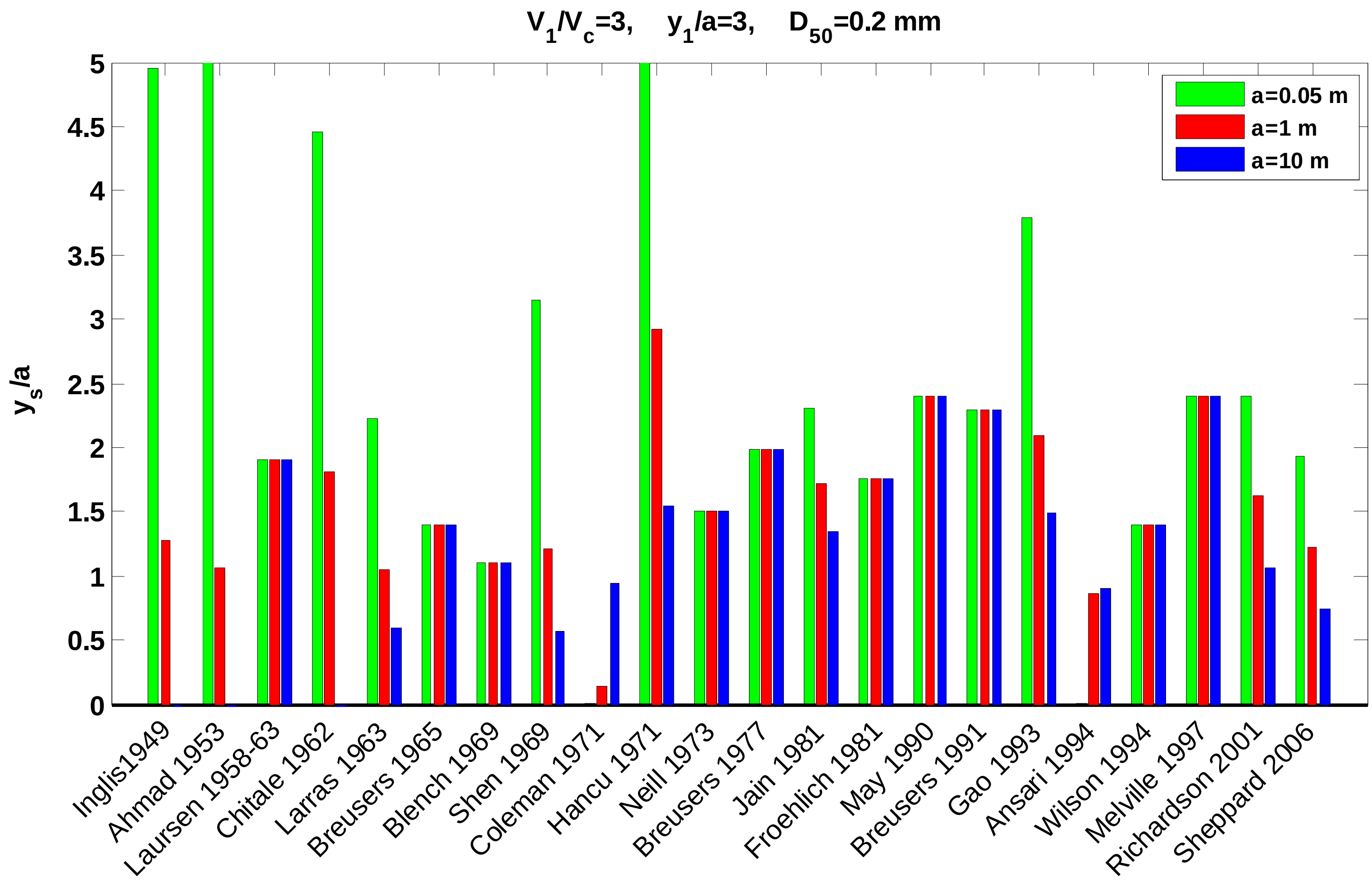

Figure A-6 Comparison of normalized local scour depth predictions using 22 different equations/methods for a particular live-bed scour condition. Deep water relative to pier width, fine sand

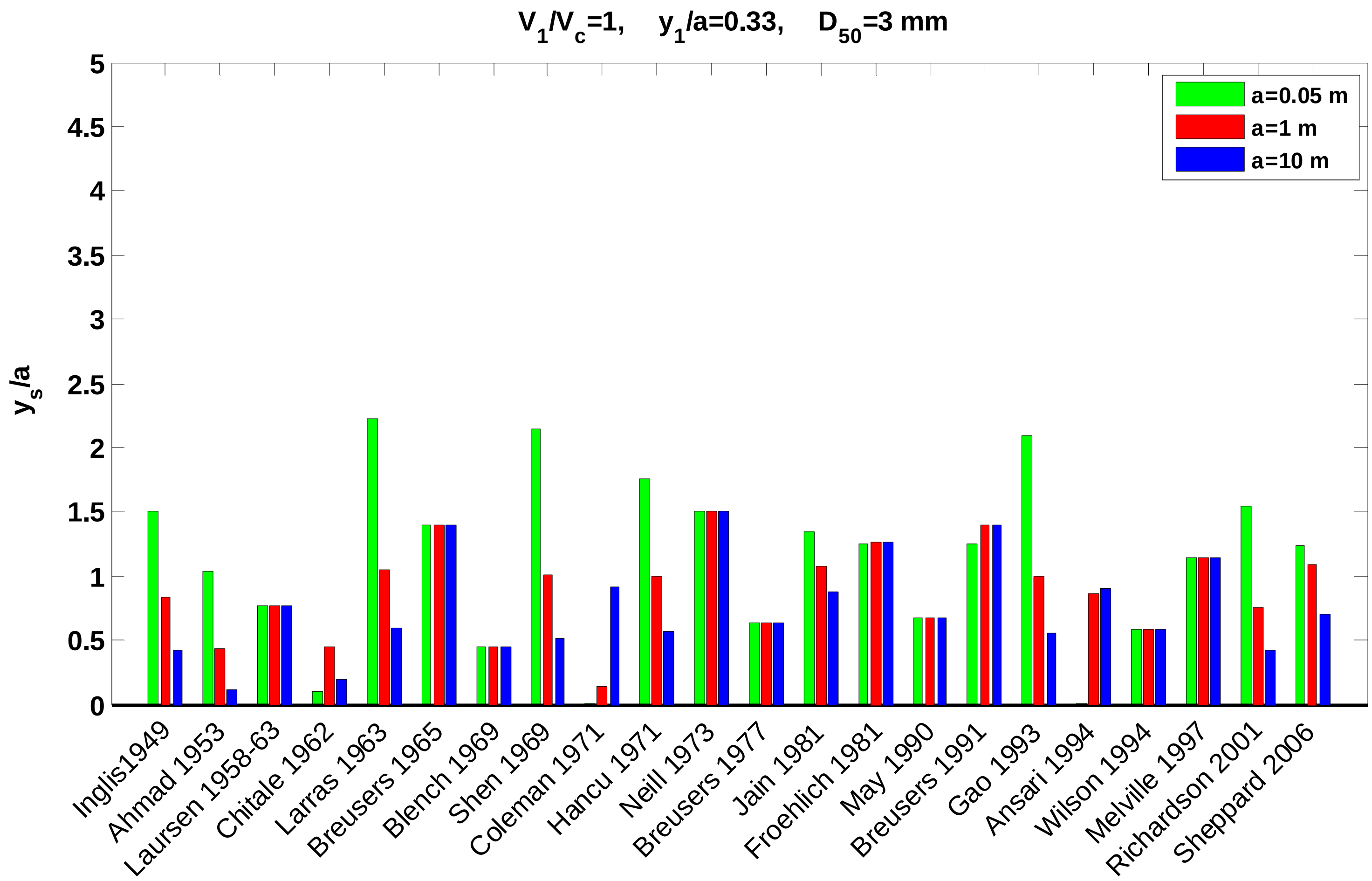

Figure A-7 Comparison of normalized local scour depth predictions using 22 different equations/methods for transition from clear-water to live-bed scour conditions. Pier width large compared to water depth, very coarse sand

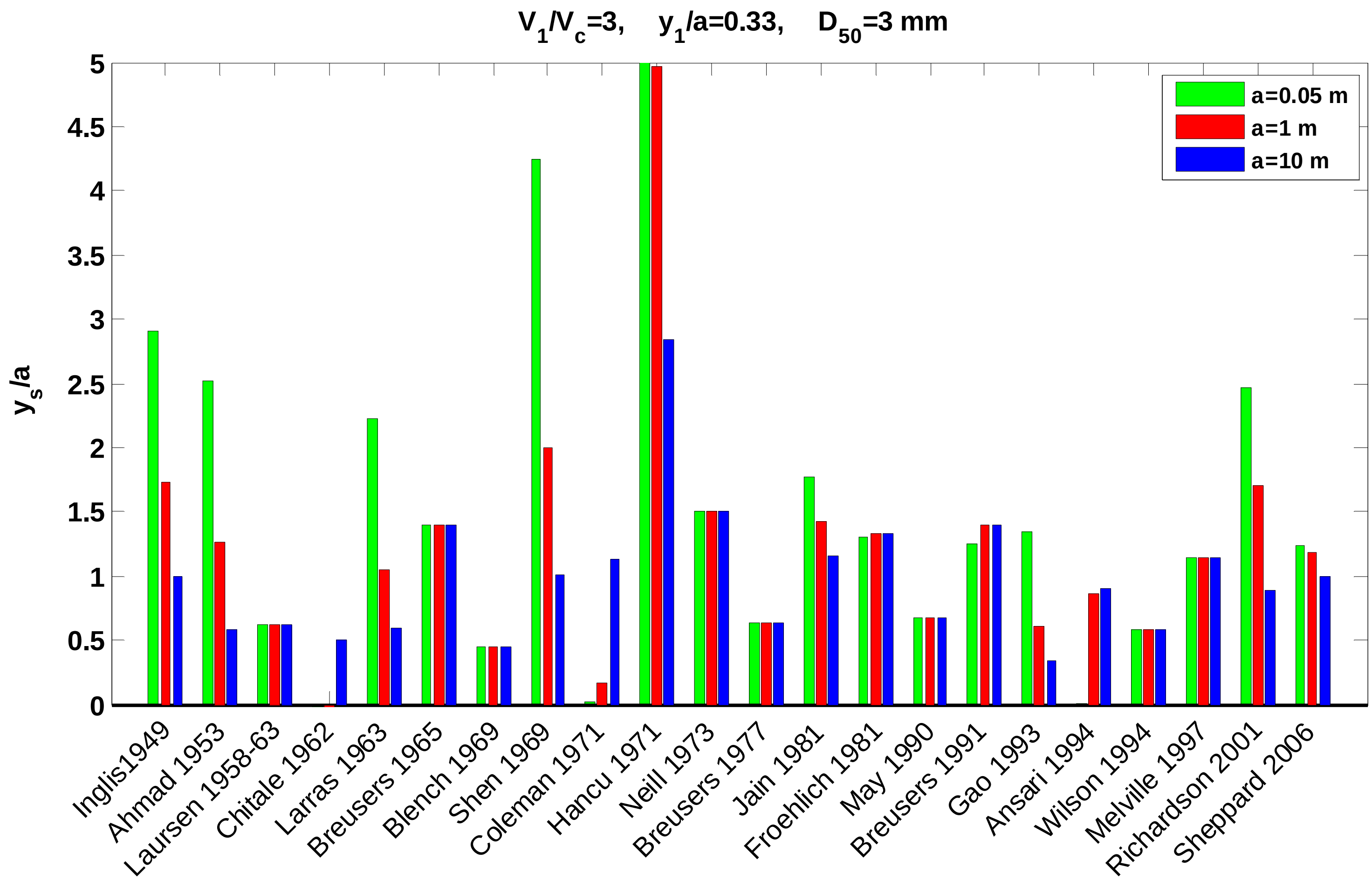

Figure A-8 Comparison of normalized local scour depth predictions using 22 different equations/methods for a particular live-bed scour condition. Pier width large relative to water depth, fine sand, very coarse sand

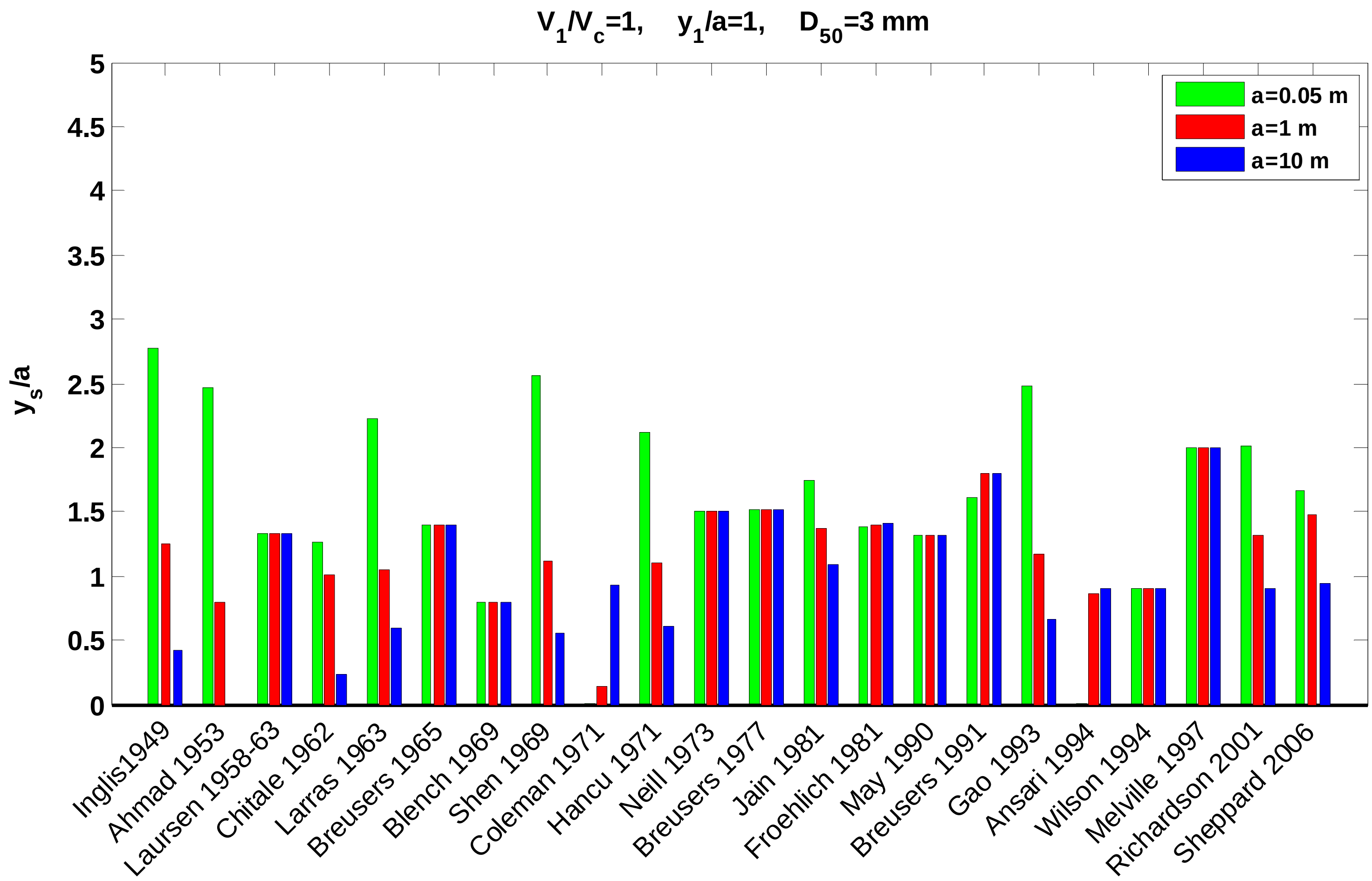

Figure A-9 Comparison of normalized local scour depth predictions using 22 different equations/methods for transition from clear-water to live-bed scour conditions. Pier width equal to water depth, very coarse sand

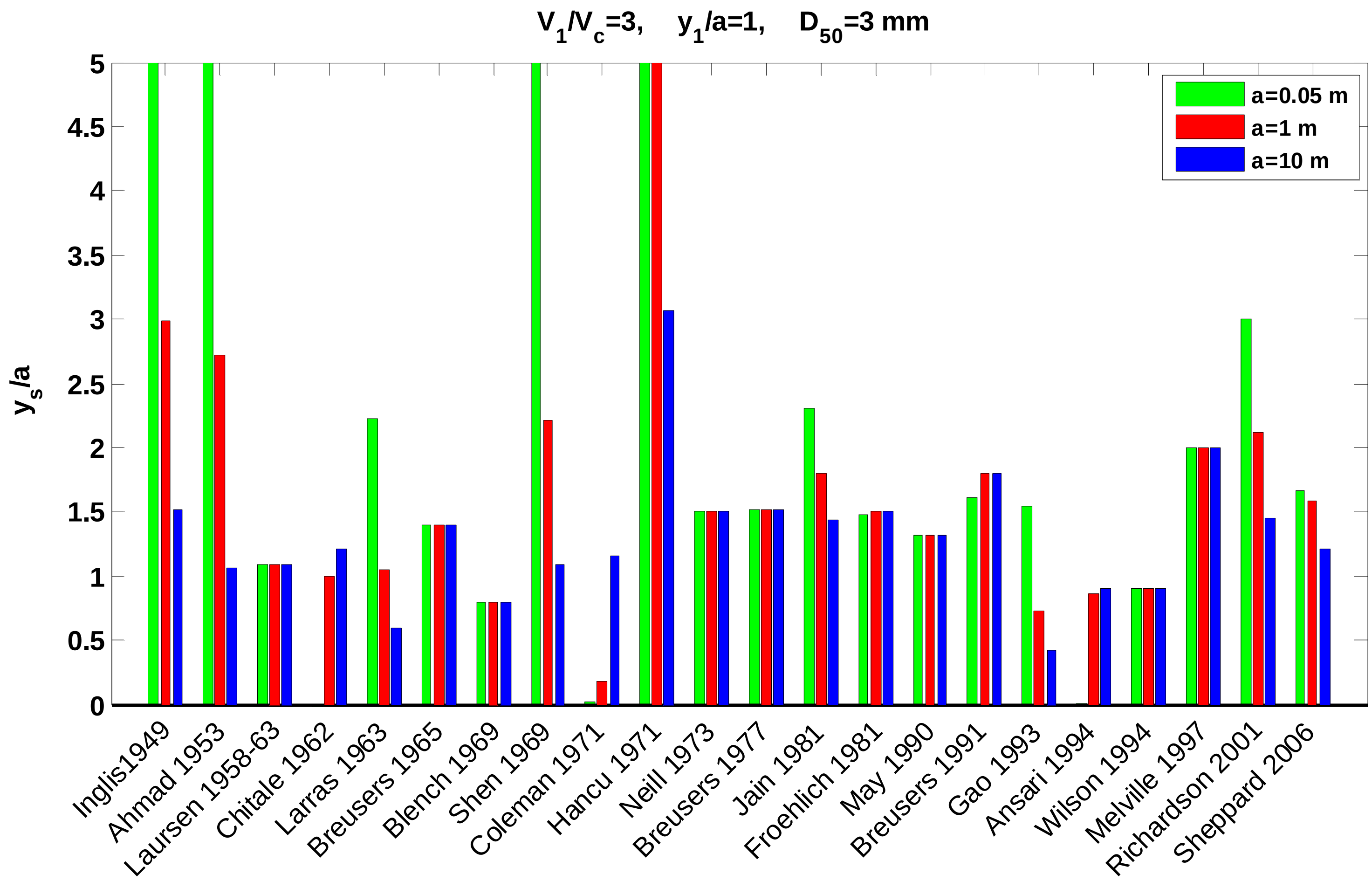

Figure A-10 Comparison of normalized local scour depth predictions using 22 different equations/methods for a particular live-bed scour condition. Pier width equal to water depth, very coarse sand

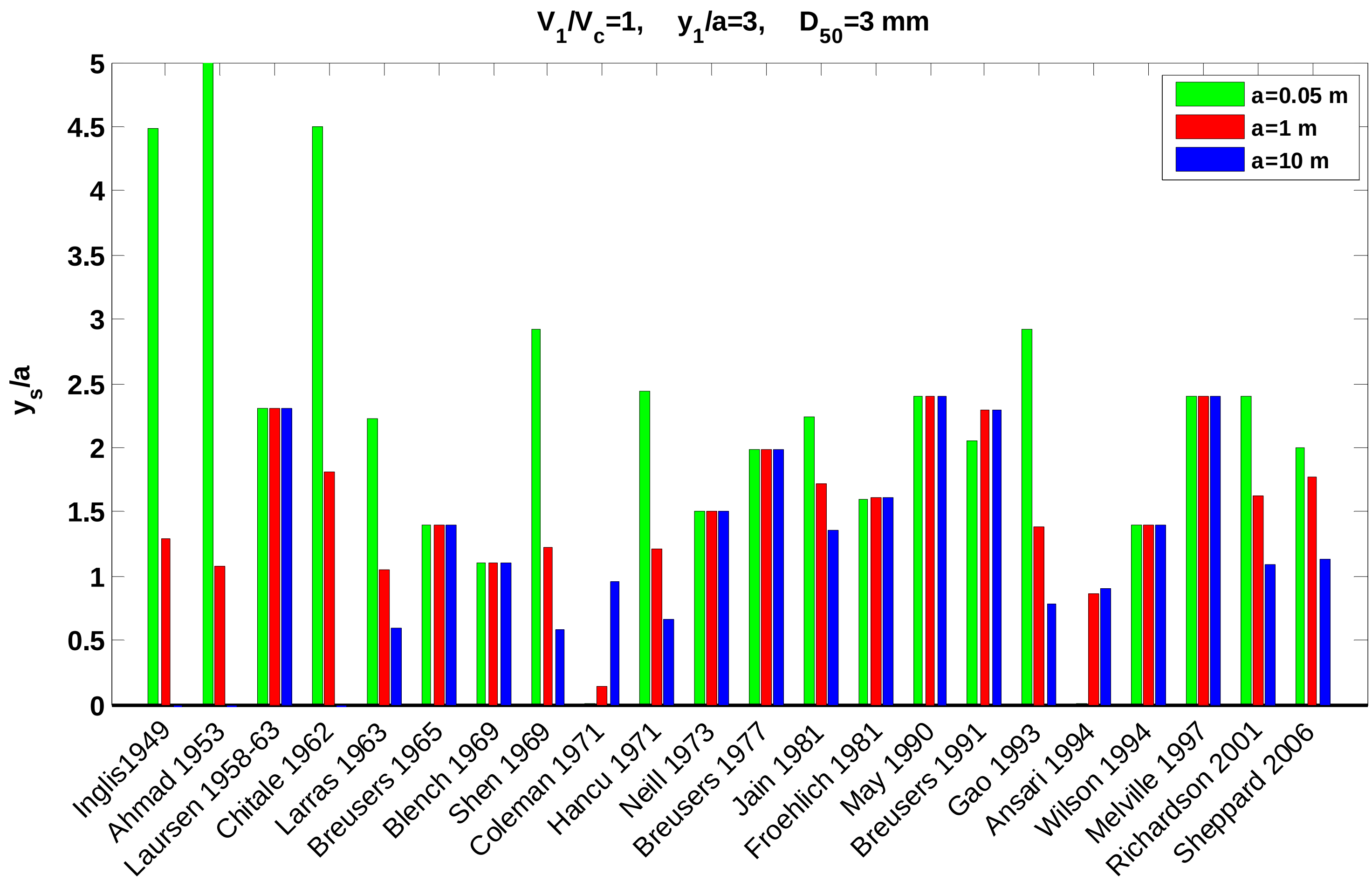

Figure A-11 Comparison of normalized local scour depth predictions using 22 different equations/methods for transition from clear-water to live-bed scour conditions. Deep water relative to pier width, very coarse sand.

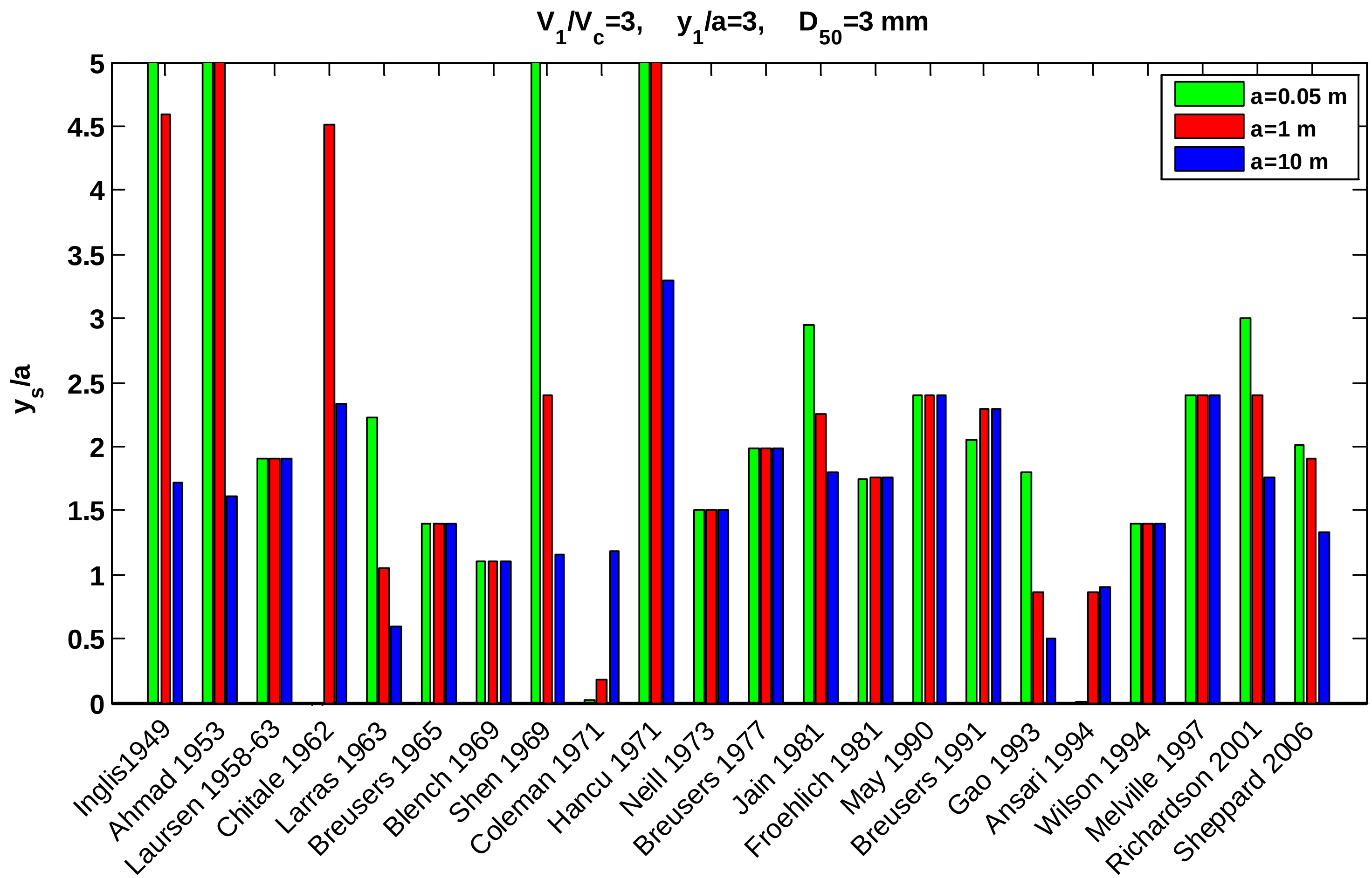

Figure A-12 Comparison of normalized local scour depth predictions using 22 different equations/methods for a particular live-bed scour condition. Deep water relative to pier width, very coarse sand

The methods used in Figures A-1 through A-12 were developed during the period 1949 to 2006. Improvements in the understanding of pier scour processes inevitably have resulted in improvements to scour-depth predictive methods. For example, several of the earlier equations give negative scour depths. Also, the variability amongst the equations tends to reduce with time.

Considering the variation in the predictions of local scour for different sized piers ("laboratory" to "typical field" to "very large field"), it is evident that some methods have scour depth ratios decreasing with increasing pier size, while other equations show constant values of scour depth ratio from laboratory to field. Conversely, the Coleman (1971) equation shows larger scour depth ratios in the field than in the laboratory, which is unlikely to be correct.

The main objective of these plots is to identify the methods producing unrealistic results for prototype scale piers and thus should be eliminated from further consideration. The regime equations proposed by Inglis (1949), Ahmad (1951) and Chitale (1962) yield negative scour depths in some cases. As noted above the, Coleman (1971) equation has an unrealistic trend with increasing pier size and therefore is eliminated. Several other equations predict unacceptably high scour depth ratios; i.e., Inglis (1949), Ahmad (1951), Chitale(1962), Hancu (1971) and Shen et al. (1969), and thus are eliminated as well. This leaves 23 methods/equations for further consideration.

Referring to Table A-2, several of the remaining equations include few or none of the important dimensionless parameters. In this category are the methods by Laursen (1958, 1963), Larras (1963), Neill (1964, 1973), Breusers (1965), Blench (1969), Jain and Fischer (1980), Jain (1981), Chitale (1988), Ansari and Qadar (1994), Briaud et al. (2004), and Kothyari(2007). These methods are eliminated from further consideration on this basis. It is noted also, that the method by Ansari and Qadar (1994) can lead to very low scour depths compared to predictions by most of the other more recent equations.

The remaining 11 methods include those by Breusers et al. (1977), Breusers and Raudkivi (1991), Melville and Sutherland (1988) and Richardson and Davis (1995). As discussed earlier, these equations have been superseded by more recent equations, notably Melville (1997) and Richardson and Davis (2001). The method by Gao et al. (1993) can lead to excessively high scour depths in fine sediments, as shown in the comparison plots for D = 0.2mm. In addition, it is noted that the equation by May and Willoughby (1990) was developed from data pertinent to large offshore structures. Eliminating these 6 methods leaves the following 5 methods for further consideration: Froehlich (1988), Wilson (1995), Melville (1997), Richardson and Davis (2001), and Sheppard and Miller (2006, or Sheppard-Melville).

A.5 DISCUSSION

By virtue of its extensive use in the U.S., and its current use in HEC-18, the Richardson and Davis (2001) method requires further consideration. Moreover, the Sheppard and Miller (2006, now Sheppard-Melville) method, recommended based on the extensive appraisal in NCHRP Project 24-32, also requires further consideration. The two methods yield overall comparable estimates of scour depth in the assessment shown in Section A.4. However, this finding does not mean that each method automatically yields an estimate of the potential maximum scour depth at a pier. These two methods can be identified as the two leading methods, and accordingly are examined further in Chapter 6.

Of the five remaining methods mentioned above, two can be dropped because they include fewer parameters, as indicated in Table A-4, which shows the number of parameters each method includes, and because they are unlikely to be adopted for use in HEC-18; i.e., Froehlich (1988) and Wilson (1995).

The Melville (1997) method has been merged with the Miller and Sheppard (2002) method, under NCHRP Project 24-32, and therefore Melville (1997) is not considered further herein. However, it is worth noting the method's strengths as a design method, and in delineating differences in scour-depth trends with varying values of y/a. As a design method, Melville (1997) provides an upper-bound estimate of scour depth estimated in accordance with a wide set of parameter influences. The method's characterization of parameter influences is more physically based than prior methods.

Table A-4 Some characteristics of the remaining 5 equations

Method	**Main data sources**	**No. of important dimensionless parameters**
Froehlich (1988)	field and laboratory	3
Wilson (1995)	field	1
Melville (1997)	laboratory	6
Richardson and Davis (2001)	laboratory	4
Sheppard and Miller (2006, Sheppard-Melville)	laboratory	6