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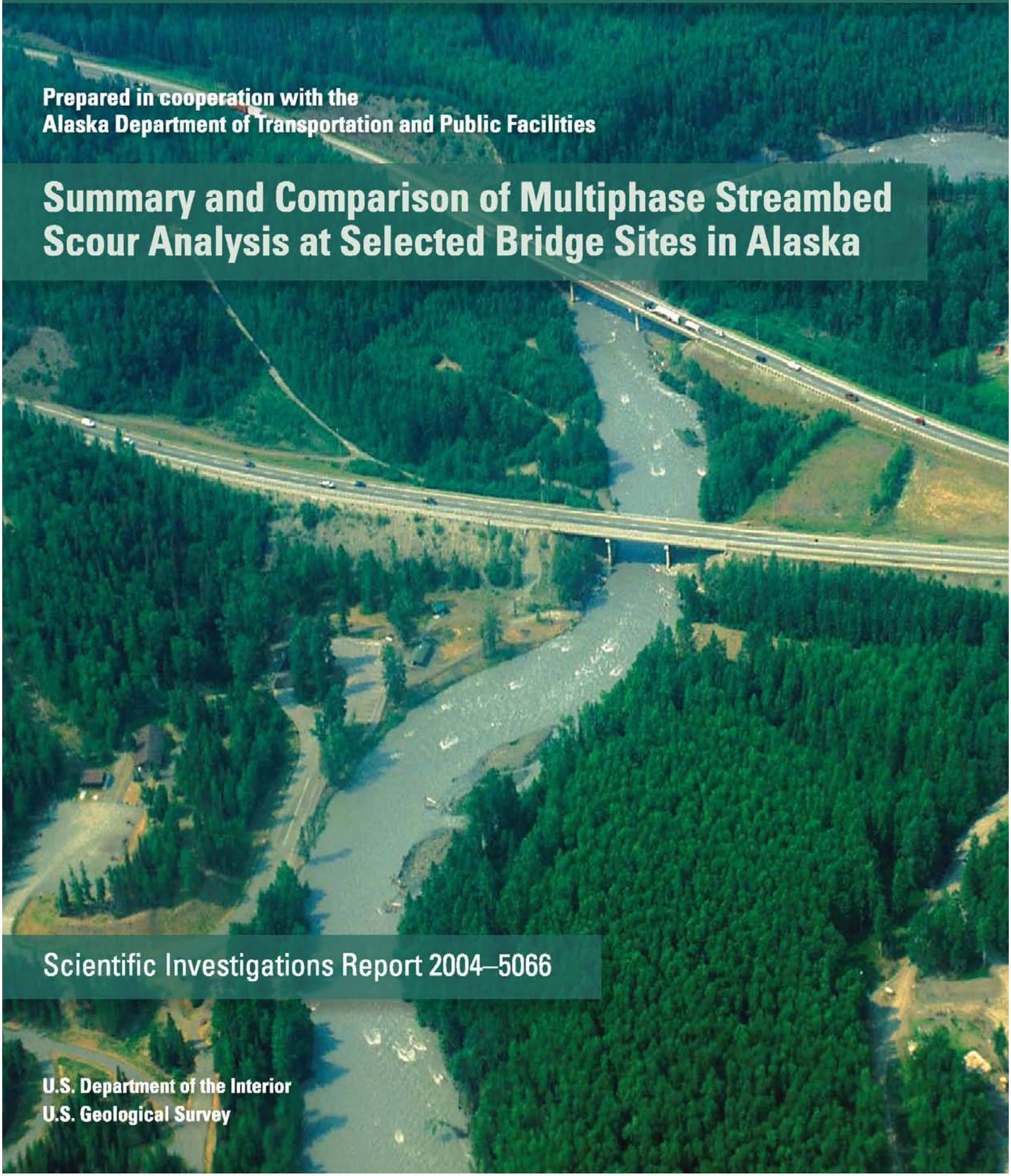
Summary and Comparison of Multiphase Streambed Scour Analysis at Selected Bridge Sites in Alaska

Scientific Investigations Report 2004–5066



U.S. Department of the Interior
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Cover: Oblique aerial photograph of Glenn Highway bridge crossings of Eagle River, Alaska.
Photograph by Jeffrey S. Conaway

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U.S. Geological Survey, Reston, Virginia: 2004
For sale by U.S. Geological Survey, Information Services
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Suggested citation:
Conaway, J.S., 2004, Summary and comparison of multiphase streambed scour analysis at selected bridge sites in Alaska: U.S. Geological Survey Scientific Investigations Report 2004–5066, 34 p.

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CONVERSION FACTORS AND DATUM

CONVERSION FACTORS

Multiply	By	To obtain
cubic foot per second (ft^3/s)	0.02832	cubic meter per second (m^3/s)
cubic foot per second per square mile [($ft^3/s/mi^2$]	0.01093	cubic meter per second per square kilometer [($m^3/s/km^2$]
foot (ft)	0.3048	meter (m)
mile (mi)	1.609	kilometer (km)
square mile (mi^2)	259.0	hectare
	2.590	square kilometer (km^2)

DATUM

Datum: Local datums used in this report were arbitrary set at the time of the fieldwork or by the Alaska Department of Transportation and Public Facilities and are not necessarily referenced to any National Geodetic Vertical Datum.

Summary and Comparison of Multiphase Streambed Scour Analysis at Selected Bridge Sites in Alaska

By Jeffrey S. Conaway

Abstract

The U.S. Geological Survey and the Alaska Department of Transportation and Public Facilities undertook a cooperative multiphase study of streambed scour at selected bridges in Alaska beginning in 1994. Of the 325 bridges analyzed for susceptibility to scour in the preliminary phase, 54 bridges were selected for a more intensive analysis that included site investigations. Cross-section geometry and hydraulic properties for each site in this study were determined from field surveys and bridge plans. Water-surface profiles were calculated for the 100- and 500-year floods using the Hydrologic Engineering Center's River Analysis System and scour depths were calculated using methods recommended by the Federal Highway Administration.

Computed contraction-scour depths for the 100- and 500-year recurrence-interval discharges exceeded 5 feet at six bridges, and pier-scour depths exceeded 10 feet at 24 bridges. Complex pier-scour computations were made at 10 locations where the computed contraction-scour depths would expose pier footings. Pressure scour was evaluated at three bridges where the modeled flood water-surface elevations intersected the bridge structure.

Site investigation at the 54 scour-critical bridges was used to evaluate the effectiveness of the preliminary scour analysis. Values for channel-flow angle of attack and approach-channel width were estimated from bridge survey plans for the preliminary study and were measured during a site investigation for this study. These two variables account for changes in scour depths between the preliminary analysis and subsequent reanalysis for most sites. Site investigation is needed for best estimates of scour at bridges with survey plans that indicate a channel-flow angle of attack and for locations where survey plans did not include sufficient channel geometry upstream of the bridge.

Introduction

The U.S. Geological Survey (USGS), in cooperation with the Alaska Department of Transportation and Public Facilities (ADOT&PF), began studying the susceptibility of Alaskan bridges to streambed scour in 1994. A multiphase approach was applied to bridges selected by ADOT&PF as potentially susceptible to scour. Heinrichs and others (2001) documented results from the initial phase of this project, referred to hereafter as Phase 1 analysis. Phase 1.5 and Phase 2 analyses were completed in 2002.

Phase 1 evaluated streambed scour for the 100- and 500-year recurrence-interval floods at 325 of Alaska's approximately 800 bridges (Heinrichs and others, 2001). In this initial analysis, the step-backwater water-surface profile (WSPRO) model (Shearman, 1990) was used to construct hydraulic models of 100- and 500-year discharges using channel geometry determined from existing bridge plans and either assumed or, when available, measured hydraulic properties. Hydraulic variables computed by these models were used to calculate contraction and pier scour using federally recommended techniques and equations. From these initial results, ADOT&PF selected 54 sites (fig. 1, table 1) for further evaluation consisting of detailed field surveys and new hydraulic models and scour analyses. The addition of field data allowed for an updated evaluation of scour and documented changes in channel geometry since the bridge plans were surveyed.

Sites from the Phase 1 analysis were selected for either Phase 1.5- or Phase 2-level analysis. These two levels of investigation are distinguished by the quantity of field data collected; the Phase 1.5 analysis was the less intensive. Division into these groups was based on the magnitude of existing scour, scour depths calculated for the Phase 1 analysis, and data that were available for the Phase 1 analysis. No sites were evaluated at all three levels of analysis.

Figure 1. Location of bridges in Alaska selected for Phase 1.5 or Phase 2 analysis of susceptibility to streambed scour.

Table 1. Bridges in Alaska from the Phase 1 streambed scour analysis selected for further analysis, and the 100- and 500-year recurrence-interval flood discharges and depths of scour estimated in the Phase 1 analysis

[Data from Heinrichs and others (2001). **ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. ft^3/s, cubic feet per second; ft, feet. –, no data]

ADOT & PF No.	Bridge name	Route name	Road mile point	Year built	Discharge (ft^3/s)		Depth (ft) Contraction scour		Depth (ft) Pier scour	
					100-year	500-year	100-year	500-year	100-year	500-year
				Phase 1.5						
205	Copper River near Chitina	McCarthy Road	34.6	1971	126,000	151,000	1.0	1.1	13.7	14.0
240	Little Susitna River	Parks Highway	57.1	1964	3,800	4,570	.0	.0	10.3	10.8
277	Taylor Creek	Taylor Highway	50.3	1977	1,350	2,360	4.4	.9	no pier	no pier
308	Skagway River	Klondike Highway	1.7	1974	18,500	27,500	–	–	–	–
396	Deception Creek	Fishhook/Willow Road	48.4	1972	1,600	2,060	.3	.3	3.6	3.8
401	Moose Creek	Petersville Road	7.1	1974	4,360	7,010	2.0	9.6	no pier	no pier
505	Tanana River Near Tok	Alaska Highway	1,303.3	1944	51,900	59,700	.0	.0	28.1	28.7
509	Robertson River	Alaska Highway	1,345.3	1944	9,520	11,700	.0	.0	21.6	22.4
518	Johnson River	Alaska Highway	1,380.4	1944	7,520	9,250	.0	.0	26.6	28.1
520	Gerstle River	Alaska Highway	1,392.7	1944	5,100	6,340	.1	.2	18.6	19.2
530	Little Salcha River	Richardson Highway	328.4	1967	3,130	3,920	1.5	2.2	12.2	13.5
543	Granite Creek	Glenn Highway	62.4	1958	1,920	2,390	.4	.5	17.0	18.1
544	Kings River	Glenn Highway	66.4	1961	7,600	9,310	2.0	2.4	21.6	23.1
547	Hicks Creek	Glenn Highway	96.5	1956	1,370	1,700	.0	.0	12.0	12.5
548	Caribou Creek	Glenn Highway	106.9	1953	9,550	11,800	4.5	5.0	34.5	36.6
556	Valdez Glacier Stream	Richardson Highway	0.9	1965	24,400	30,600	5.8	7.3	9.7	11.7
572	Klutina River	Old Richardson	0.4	1957	10,700	11,800	.6	.5	16.1	16.3
573	Tazlina River	Richardson Highway	110.7	1973	79,400	109,000	2.9	2.8	34.6	37.2
603	Snow River West Channel	Seward Highway	17.1	1965	2,100	2,600	.6	.7	9.3	9.8
658	Little Tok River	Old Tok Highway	7.5	1951	9,430	11,200	.0	.0	4.5	4.7
678	Little Goldstream Creek	Parks Highway	314.8	1958	2,260	2,870	2.5	1.8	3.5	3.7
686	Clearwater Creek	Denali Highway	55.9	1957	4,420	5,400	1.1	1.6	12.4	13.4
687	Susitna River	Denali Highway	79.2	1956	37,600	48,200	.3	.5	18.7	19.5
690	Seattle Creek	Denali Highway	110.9	1954	3,090	5,730	5.9	.0	no pier	no pier
742	Chilkat River	Haines Highway	23.8	1958	32,600	42,900	.3	.3	4.5	4.8
832	Boulder Creek	Steese Highway	125.3	1971	2,130	3,840	.7	.3	no pier	no pier
833	Albert Creek	Steese Highway	131.2	1978	3,890	4,860	2.2	2.8	no pier	no pier
999	Glacier Creek	Alyeska Road	2.3	1967	7,020	10,200	3.3	4.4	12.5	14.3
1220	Cowee Creek	Glacier Highway	1.9	1971	9,870	12,400	3.2	.7	4.9	5.0
1261	Middle Fork Koyukuk River No. 1	Dalton Highway	188.5	1972	25,100	28,500	.9	1.0	8.2	8.4
1282	Middle Fork Koyukuk River No. 2	Dalton Highway	190.8	1972	20,300	23,400	.3	.4	13.1	13.7
1283	Middle Fork Koyukuk River No. 3	Dalton Highway	204.3	1972	10,300	12,000	1.5	3.1	3.4	3.4
1284	Middle Fork Koyukuk River No. 4	Dalton Highway	204.5	1972	7,880	9,180	1.5	1.6	4.6	4.8
1329	Little Chena River	Nordale Road	3	1975	7,910	11,800	.0	.0	5.1	5.8
1389	No Name creek near Seward	Exit Glacier Road	4.9	1994	980	1,230	.1	.1	3.6	3.7

Table 1. Bridges in Alaska from the Phase 1 streambed scour analysis selected for further analysis, and the 100- and 500-year recurrence-interval flood discharges and depths of scour estimated in the Phase 1 analysis–*Continued*

[Data from Heinrichs and others (2001). **ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. ft^3/s, cubic feet per second; ft, feet. –, no data]

ADOT & PF No.	Bridge name	Route name	Road mile point	Year built	Discharge (ft^3/s)		Depth (ft) Contraction scour		Depth (ft) Pier scour	
					100-year	500-year	100-year	500-year	100-year	500-year
Phase 2										
233	Chena River at 40 mile	Chena Hot Springs Road	39.5	1967	22,100	27,400	8.3	15.9	8.0	8.8
242	North Fork Chena River	Chena Hot Springs Road	55.3	1967	4,220	5,220	.8	.8	4.5	4.7
254	Susitna River at Sunshine	Parks Highway	104.2	1965	186,000	206,000	6.2	6.5	16.7	17.3
255	Chulitna River	Parks Highway	97.5	1970	83,200	103,000	.4	.4	23.8	25.0
355	Birch Creek	Steese Highway	147.1	1957	36,000	42,000	.6	.8	9.1	9.5
527	Salcha River	Richardson Highway	323.3	1967	50,600	64,900	3.3	3.9	20.7	21.7
535	Eagle River	Glenn Highway	140.1	1981	6,920	8,710	2.1	3.0	9.9	10.6
539	Knik River Old Glenn	Old Glenn Highway	8.6	1975	79,400	104,000	11.7	13.4	10.8	11.7
557	Lowe River	Richardson Highway	14.8	1974	22,100	28,500	1.1	1.5	7.2	7.7
558	Lowe River	Richardson Highway	16.5	1985	19,400	25,100	.3	.3	17.2	18.3
574	Gulkana River	Richardson Highway	126.9	1974	18,400	22,200	5.3	6.3	6.8	6.9
608	Ptarmigan Creek	Seward Highway	23.1	1952	1,610	2,070	.1	.1	6.7	7.0
609	Falls Creek	Seward Highway	25	1951	1,220	1,750	.5	.6	3.4	3.7
694	Nenana River at Park Bend Creek	Parks Highway	231.2	1973	25,400	30,300	.2	.2	12.5	12.9
1025	Resurrection Creek	Hope Road	16.9	1969	4,590	6,520	1.8	2.4	7.5	8.1
1147	Nenana River at Park Station	Parks Highway	237.9	1970	43,200	53,600	.8	1.7	9.9	10.3
1255	Fish Creek	Dalton Highway	114	1972	3,800	4,710	1.4	1.9	9.5	10.1
1341	Eagle River	Glenn Highway	139.9	1974	6,920	8,710	1.8	1.7	12.9	13.6
1513	Ft. Hamlin Hills	Dalton Highway	72.6	1982	1,830	2,330	1.8	2.6	no pier	no pier

Background

Streambed scour is the leading cause of bridge failure in the United States (Murillo, 1987). The costs associated with restoring damaged structures are substantial, but are less than five times the indirect costs associated with the disruption of traffic (Rhodes and Trent, 1999). These costs and the societal repercussions are even greater in Alaska, where alternate ground transportation routes between many cities do not exist. Bridge and culvert damage associated with the effects of flow hydraulics occurs several times in Alaska every year. Quantifying the susceptibility of these structures to streambed scour helps to prioritize mitigation, monitoring, and redesign efforts.

Streambed scour at bridges results from the complex hydraulic conditions created either by the contraction of flow through a bridge or by the interaction of flow with bridge piers and abutments that results in the hydraulic erosion of the channel bed or banks. Streambed scour at bridges is separated into general scour, contraction scour, and local scour. General scour is the natural channel degradation and lateral erosion that occur regardless of the bridge structure. Contraction scour results from the decrease in channel width through a bridge reach and the attendant increase in flow velocity and sediment transport capability. Local scour at piers results from horseshoe vortices that form at the upstream, downstream, and sides of piers. Literature on streambed scour is copious; Richardson and Lagasse (1999) have edited a compendium on the subject that provides a thorough overview of historical and current research.

Purpose and Scope

The USGS developed a multiphase approach for the analysis of streambed scour at Alaska's bridges. ADOT&PF used the data from Phase 1 to identify scour-susceptible structures for further analysis. This report presents Phase 1.5 and Phase 2 estimations of contraction and pier scour derived from hydraulic modeling of 100- and 500-year floods at bridges identified as scour critical. The hydraulic models were constructed from existing data and field data collected for this study. The methodologies for data collection, model generation, and scour estimates are described and evaluated.

Methodology and results from this study are compared with those of the preliminary Phase 1 analysis of Heinrichs and others (2001), which did not include the collection of field data and used less intensive hydraulic modeling software.

Methodology

The Phase 1.5 and Phase 2 analyses built upon existing information by providing current field data. These data were used to construct and calibrate a one-dimensional surface-water model capable of computing water-surface elevations and hydraulic variables necessary to estimate scour at bridges. The models were used to estimate scour for 100- and 500-year floods using techniques outlined by the Federal Highway Administration's Hydraulic Engineering Circular No. 18, hereafter referred to as HEC-18 (Richardson and Davis, 2001). Hydraulic variables needed to calculate scour were computed with the Hydrologic Engineering Center River Analysis System (HEC-RAS; Brunner, 2001), which is a one-dimensional surface-water modeling program.

Existing data for the sites in this study consisted of the Phase 1 analysis and the bridge as-built plans that are provided by ADOT&PF. The as-built plans generally include a detailed topographic map of the channel near the bridge at the time of construction. Other possible sources of existing data include discharge measurements, streamflow-gaging station records, indirect computations of discharge, or ADOT&PF bridge inspections. The discharges for the 100- and 500-year floods for each river crossing in this study (table 1) were estimated by Heinrichs and others (2001) using standard flood-frequency analyses (Interagency Advisory Committee on Water Data, 1982) and regional regression equations as outlined by Jones and Fahl (1994).

Site Selection

The results from the Phase 1 analysis were used to select sites for more intensive study. A site was selected for further study if it met any of the following criteria.

- Phase 1 estimates of contraction scour were greater than 5 ft.
- Phase 1 estimates of local scour at piers were greater than 10 ft.
- The water-surface profile calculated during Phase 1 intersected the bridge at the 100- and(or) 500-year discharge (pressure flow).
- Bridge as-built plans or discharge measurements indicate high channel-flow angles of attack at the bridge piers.
- Piers are set on shallow or exposed footings.
- Bridge inspections reported scour.
- Channel data available from bridge as-built plans were insufficient for accurate Phase 1 scour analysis.

Sites were selected for study at the Phase 1.5 level if it met any of the following criteria.

- They lacked detailed bridge plans,
- Slope could not be accurately estimated,
- Channel-flow angle of attack determined from bridge plans was significant and needed verification.
- Phase 1 hydraulic modeling indicated pressure-flow conditions.

Thirty-five bridges were selected for Phase 1.5 analysis (table 1). Bridges with large calculated scour depths from the Phase 1 study or those with known potential for significant scour during extreme floods were selected for Phase 2 analysis. Nineteen bridges were selected for Phase 2 analysis (table 1).

Phase 1.5 Data Collection and Model Parameters

Standard field data collected for a Phase 1.5 analysis were a discharge measurement, bridge sounding, water-surface slope, site sketch, and photographs. Data collection was abbreviated at sites that had recent bridge as-built plans, bridge inspections, or discharge measurements. Discharge was not measured at some locations because the discharge at the time of the site visit was insufficient to calibrate a hydraulic model. At a minimum, the channel-flow angle of attack was field verified and the channel slope was measured.

Hydraulic models were constructed using a combination of field data and the bridge as-built plans. Bridge soundings or discharge-measurement data, in combination with geometry from the bridge as-built plans, were used to construct cross sections immediately upstream and downstream of the bridge. Channel-geometry data from the cross section at the bridge also were integrated with overbank data from the bridge as-built plans or the field survey to construct a full-valley cross section. This section was copied two times upstream and two times downstream of the bridge using a surveyed water-surface slope to adjust cross-section elevations used to construct the model. Cross sections typically were separated by a distance of one bridge opening. This spacing usually was sufficient to span the difference over which contraction and expansion of flow occurred in response to the bridge structure. At locations where the channel constricts through the bridge, the sounding or discharge-measurement data were expanded laterally to integrate with the approach and exit overbank data.

The Phase 1 study assumed Manning's roughness values of 0.035 and 0.10 for the channel and overbanks, respectively, unless other data sources were available. These default values are a good approximation for Alaskan streams and were retained for Phase 1.5 locations unless a discharge measurement and surface-water slope were available. Slope and discharge-measurement data and the Manning equation can be used to calculate a roughness coefficient for the channel. When photographs were available, overbank roughness values were estimated using techniques outlined by Hicks and Mason (1991).

Phase 2 Data Collection and Model Parameters

The comprehensive field survey for Phase 2 sites included surveying channel cross sections at the bridge, in the approach to the bridge, and the exit from the bridge. Channel data obtained by boat or wading were integrated with bank and overbank data to complete each cross section. The number of surveyed cross sections varied, but a minimum of three was obtained at each bridge. Ideally, cross sections were separated by a distance equivalent to the bridge opening width, but this distance can vary depending on field conditions. When surveyed cross sections were insufficient to construct the hydraulic model, the field data were copied using the surveyed water-surface slope in the approach and exit sections as needed. All surveys were tied to the vertical datum from the bridge as-built plans.

In addition to surveying cross sections, data collection for Phase 2 included measuring discharge and surveying the corresponding water-surface slope, observing site geomorphology, channel-flow angle of attack at the piers, current scour conditions, and initial estimation of roughness coefficients for the channel and overbanks, and describing streambed material.

Roughness coefficients for the Phase 2 sites were calculated using the same procedures that were used for the Phase 1.5 sites. Roughness values were further calibrated with HEC-RAS.

Boundary Conditions and Computation of Water-Surface Profiles

HEC-RAS was used to calculate water-surface profiles for the Phase 1.5 and Phase 2 analyses. HEC-RAS is based on the solution of the one-dimensional energy equation and is capable of modeling flow regimes that are subcritical, supercritical, and a combination of the two (mixed flow regime). Energy losses are evaluated with roughness coefficients and contraction and expansion coefficients. The momentum equation is used in place of the energy equation where the flow varies greatly, including hydraulic jumps, bridge constrictions, pressure flow, and locations where flow transitions between subcritical and supercritical. Hydraulic variables used to construct the models are summarized in table 4 (at back of report).

Water-surface profiles were calculated for the 100- and 500-year floods and for calibration discharges. The calibration discharge was either a discharge measurement made during the field visit for the Phase 1.5 or Phase 2 analysis or an existing high-flow direct or indirect discharge measurement. The step-backwater models for the Phase 1.5 and Phase 2 analyses were identical with the exception of the calibration process.

The Phase 1.5 and Phase 2 models were constructed and calibrated using the following procedure.

1. Entered cross-section geometry and defined overbanks on the basis of field observations or slope breaks. Model geometry also includes deck, abutment, and pier elevations and dimensions that were obtained from existing bridge as-built plans.
2. Assigned computed or estimated roughness coefficients to each cross section as a composite value or separately for the channel and overbanks.
3. Defined boundary conditions. Boundary conditions were set at the downstream cross section for subcritical flow regime and at the downstream and upstream cross sections for the mixed flow regime. Water-surface slope was used for the normal-depth boundary condition for the Phase 1.5 models. If the slope was not available, either a friction slope was calculated from a discharge measurement or a slope from a topographic map was used. For Phase 2 models, the surveyed water-surface elevation at the downstream cross section was used as a boundary condition for the calibration discharge.
4. Calculated initial water-surface profiles and calibrated the model to measured values. For Phase 1.5 models, the average velocity, flow area, and flow width of the calibration discharge were compared with the measured values. Channel-roughness coefficients were adjusted until agreement was reached between modeled and measured values. Phase 2 models were calibrated by adjusting roughness coefficients at each cross section until the modeled water-surface elevation for the calibration discharge matched the surveyed water-surface elevation at each cross section. After roughness coefficients for the channel were calibrated and the model was rerun, the energy-gradient slope at the downstream cross section was calculated and used as the normal-depth boundary condition for models of the 100- and 500-year floods.
5. Calculated water-surface profiles for the 100- and 500-year floods with the calibrated model. If Froude numbers exceeded 1.0 for any cross section, an upstream boundary condition (the slope of the energy gradient at this section, determined from the calibration discharge) was established and the model was rerun using the mixed flow regime. Interpolated cross sections were inserted between existing sections if the water surface varied greatly, the channel expands or contracts rapidly, or the gradient changed abruptly.
6. Computed final water-surface profiles and hydraulic variables necessary for the scour computations.

Estimation Methods for Contraction Scour, Pier Scour, Complex Pier Scour, and Pressure Scour

Estimates of scour were computed using the equations and methodology outlined in HEC-18 (Richardson and Davis, 2001). Contraction scour was computed at all sites, pier scour was computed at crossings supported by piers, complex pier scour was computed if the foundation of the pier was wider than the pier and was exposed to the flow, and pressure scour was computed at locations where a modeled water-surface elevation intersected the bridge structure. Abutment scour was not evaluated because of the large computational uncertainties and because most abutments on Alaskan bridges are armored with riprap to inhibit scour (Heinrichs and others, 2001). Flow widths, depths, and velocities were calculated with HEC-RAS and were used to compute the scour estimates for the 100- and 500-year floods. The reference surface for all calculations was the streambed elevation at the bridge determined from the as-built survey plans.

Contraction Scour

Contraction scour at the bridges in this study was evaluated with the live-bed contraction-scour equation. Under live-bed conditions, contraction scour in the bridge section reaches a maximum when sediment transport into the contracted section equals sediment transport out or when the mean velocity equals the critical velocity of the mean-diameter bed material. This equilibrium is reached when the transport capacity in the contracted section decreases because of increasing channel area and the attendant decrease in flow velocity. At constrictions that result from bridge crossings, cross-section width generally cannot increase, so channel area is increased by degradation of the channel. The equation recommended in HEC-18 (Richardson and Davis, 2001) for estimating live-bed contraction scour is a modified version of an equation developed by Laursen (1960) that simplifies sediment transport functions to balance sediment transport through to contracted section with sediment transport through the approach section.

$$y_s = y_1\left[\left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}}\left(\frac{W_1}{W_2}\right)^{k_1}\right] - y_0 , \qquad (1)$$

where

- y_s is the contraction scour depth, in feet;
- y_1 is the average depth in the upstream main channel, in feet;
- y_0 is the existing depth in contracted section before scour, in feet;
- Q_1 is the discharge in the main channel of the approach section that is transporting sediment, in cubic feet per second;
- Q_2 is the discharge in the contracted section, in cubic feet per second;
- W_1 is the width of the main channel of the approach section that is transporting sediment, in feet;
- W_2 is the width of the main channel in the contracted section that is transporting sediment, in feet; and
- k_1 is a coefficient that accounts for the method of sediment transport. For this study, transport at all sites is assumed to be mostly in contact with bed material and the coefficient for this condition is 0.59.

The estimated contraction scour depth in equation 1 (y_s) is the difference between the maximum flow depth in the contracted section after scour has occurred and the flow depth that existed prior to any scour (y_0). Estimation of y_0 is difficult because the channel geometry in the contracted section at the time of the study likely had already been modified by contraction scour. The channel depth in the approach section (y_1) can be used as an approximation of y_0, based on the assumption that channel depth in the uncontracted approach section is a good representation of channel depth in the bridge section before any contraction scour occurred. The equation used to estimate contraction scour for this study and for the Phase 1 analysis is

$$y_s = y_1\left[\left(\frac{Q_2}{Q_1}\right)^{\frac{6}{7}}\left(\frac{W_1}{W_2}\right)^{k_1}\right] - y_1 , \qquad (2)$$

where all terms have been previously defined.

The approach section used in the contraction-scour equation was located at least one bridge-opening width upstream of the bridge. If the channel contraction begins upstream of this cross section, the next upstream section that is above the constriction was used in the computation.

Pier Scour

Estimates of pier scour depend on the flow characteristics directly upstream of the bridge pier, the characteristics of the bed material, and the geometry of the pier and its footing. The equation recommended in HEC-18 for pier scour includes correction factors for pier shape, channel-flow angle of attack, bed configuration, and the armoring effect of large-size bed material. The pier-scour equation is described in detail in HEC-18 (Richardson and Davis, 2001) and is summarized here. The equation for local scour at a pier is

$$y_s = 2.0K_1K_2K_3K_4a^{0.65}y_1^{0.35}Fr_1^{0.43} , \qquad (3)$$

where

y_s is the pier scour depth, in feet;

K_1 is the correction factor for pier nose shape;

K_2 is the correction factor for channel-flow angle of attack;

K_3 is the correction factor for channel bed form;

K_4 is the correction factor for armoring of bed material;

a is the pier width, in feet;

y_1 is the flow depth directly upstream of the pier in feet; and

Fr_1 is the Froude number just upstream from the pier.

For round-nosed piers that are aligned with the flow, scour is limited to 2.4 times the pier width for Froude numbers less than 0.80 and 3.0 times the pier width for Froude numbers equal to or in excess of 0.80 (Richardson and Davis, 2001). The correction factor for channel-flow angle of attack, K_2, can be computed with

$$K_2 = (\mathrm{Cos}\theta + La^{-1}\mathrm{Sin}\theta)^{0.65}, \quad (4)$$

where

L is the length of the pier along the direction of flow, in feet;

q is the channel-flow angle of attack, in degrees; and

all other terms are previously defined.

The correction factors for channel bedform and armoring of the bed material were constant for the sites used in this study. The correction factor for channel bedform, K_3, is 1.1 for bedforms with a magnitude less than 10 ft. This condition was assumed to exist at all locations in this study. The correction factor for armoring of the bed, K_4, is applied when the median diameter of the bed material is greater than 0.006 ft. This correction decreases the estimation of pier scour to account for armoring of the scour hole by large bed material. This condition could occur at some locations in this study, but grain-size estimations for this study were not accurate enough to apply this correction factor. Not applying this factor to locations where bed material armors scour holes will result in a more conservative estimate (overestimate) of scour. For this study, equation 4 was reduced to

$$y_s = 2.2K_1K_2a^{0.65}y_1^{0.35}Fr_1^{0.43}, \quad (5)$$

where all terms have been previously defined.

Complex Pier Scour

Local scour at a pier is affected by the pier's foundation if the foundation is wider than the pier and is exposed to the flow. For situations where the footing is located at or below the streambed, the footing can limit local scour at the pier by disrupting the flow vortices induced by the pier (Parola and others, 1996; Melville and Coleman, 2000). When the footing is exposed to the flow, it can induce vortices that will cause scour in front of and along side of the foundation (Parola and others, 1996; Melville and Coleman, 2000).

HEC-18 recommends independent calculation of scour for the pier stem, the pier foundation, and any piles supporting the foundation. These scour components are then summed to determine complex pier scour. Estimates of complex pier scour were made at sites with observed exposed footings and for sites where the streambed elevation was lower than the top of the footing after estimated contraction scour for the 100- and 500-year floods was subtracted from the lowest point in the as-built cross section. For this study, scour was estimated for the pier and foundation components, but not for the pile-group component. Computing the pile-group component was unnecessary for the sites in this study because either the pile group was not exposed by the estimated contraction scour or the foundation is surrounded by sheet piling. The equation recommended by HEC-18 for the pier-stem scour component of complex scour is similar to equation 5 and is expressed as follows.

$$y_{s\ pier} = k_{h\ pier}2.2K_1K_2a^{0.65}y_1^{0.35}Fr_1^{0.43}, \quad (6)$$

where

$y_{s\ pier}$ is the scour component of the pier stem, in feet;

$K_{h\ pier}$ is a coefficient to account for the height of the pier base above the bed and the shielding affect of the footing; and

all other terms are previously defined.

The equation to determine $K_{h\ pier}$ is

$$\begin{aligned} K_{h\ pier} = {} & (0.4075 - 0.0669\ f\ a^{-1}) \\ & - (0.4271 - 0.0778\ f\ a^{-1})\ h_1\ a^{-1} \\ & + (0.1615 - 0.0455\ f\ a^{-1})(h_1\ a^{-1})^2 \\ & - (0.269 - 0.012\ f\ a^{-1})(h_1\ a^{-1})^3, \end{aligned} \quad (7)$$

where

f is the distance between front edge of the pier foundation and pier stem, in feet;

h_1 is the height of the pier base above the bed, in feet; and

all other terms are previously defined.

The scour estimation for the pier foundation treats the foundation like a short, wide pier and uses equation 5, but the exposed footing height is used for flow depth and the approach velocity is the average velocity in the segment of flow intersecting the foundation. The average velocity in the water column intersecting the foundation is estimated by

$$V_f = V_2\left[\frac{\ln\left(10.93\frac{y_f}{K_s}+1\right)}{\ln\left(10.93\frac{y_2}{K_s}+1\right)}\right], \qquad (8)$$

where

V_f is the average velocity in the segment of flow intersecting the foundation, in feet per second;

V_2 is $V_1(y_1 y_2^{-1})$, the average adjusted velocity upstream of the pier, in feet per second; V_1 is the approach velocity, in feet per second, and y_1 is the approach flow depth, in feet;

y_f is $h_1 + 0.5\ (y_{s\ pier})$, the distance from the bed to the top of the footing after contraction scour and pier scour, in feet;

K_s is the D_{84} (the particle size for which 84 percent are finer by weight) of the bed material, in feet (default of 0.16 ft for this study); and

y_2 is $y_1 + 0.5\ (y_{s\ pier})$, the adjusted flow depth upstream of the pier including contraction scour and one-half of the pier stem scour, in feet;

The equation to estimate scour for the foundation is

$$y_{s\ found} = 2.2K_1K_2a^{0.65}y_f^{0.35}\left(\frac{V_f}{\sqrt{gy_f}}\right)^{0.43}, \qquad (9)$$

where

$y_{s\ found}$ is the scour component for the pier foundation exposed to the flow, in feet;

g is the constant for acceleration due to gravity, in feet per second squared; and

all other terms are previously defined.

Complex pier scour is then the sum of the pier-stem component and the pier-foundation component

$$y_{s\ found} = y_{s\ pier} + y_{s\ found}, \qquad (10)$$

where

y_{cps} is the total scour depth of the pier stem and its foundation, in feet; and

all other terms have been previously defined.

Pressure Scour

Free-surface flow changes to pressure-flow conditions when rising floodwaters intersect or submerge the bridge deck, resulting in an additional component of scour. This term, pressure scour, is evaluated using an equation developed by Arneson and Abt (1999) for live-bed conditions. The equation is

$$y_{ps} = y_1\left[-5.08 + 1.27\left(\frac{y_1}{H_b}\right) + 4.44\frac{H_b}{y_1} + 0.19\frac{V_a}{V_c}\right], \qquad (11)$$

where

y_{ps} is the depth of scour resulting from the pressure flow condition, in feet;

H_b is the distance from the bridge low steel to the average streambed elevation before scour, in feet;

V_a is the average velocity inside the bridge before scour, in feet per second; and

V_c is the incipient motion velocity of the D_{50} of the bed material, in feet per second.

V_c is defined by

$$V_c = 11.17y_1^{0.17}D_{50}^{0.33}, \qquad (12)$$

where

D_{50} is the particle size for which 50 percent are finer by weight; and

all other terms are previously defined.

Under pressure-flow conditions, the magnitude of pier scour is approximately the same as it is under free-surface flow conditions (Jones and others, 1999). Pressure-flow scour is computed after lateral contraction scour is estimated (Arneson and Abt, 1999). For this study, contraction scour was evaluated independent of the pressure flow conditions by modeling the flood flows without a bridge deck for the flows to intersect. The contraction scour that occurred under this condition was then subtracted from the cross sections at the bridge. The bridge deck then was included in another hydraulic simulation with the modified cross sections at the bridge to evaluate pressure scour after the contraction scour had occurred. Total scour at pressure-flow sites is then the sum of the local pier scour, the contraction scour, and the pressure scour calculated using equation 11 (Arneson and Abt, 1999).

Hydraulic Modeling Results and Evaluation

The hydraulic models were constructed and calibrated with existing information and with data collected at the time of the field visit. The hydraulic parameters used to generate the surface-water models and the hydraulic data needed for the computation of scour are presented in tables 4-8 (at back of report). The calibrated models then were extrapolated to accommodate the 100- and 500-year flood flows. Simulations of large flood flows with models calibrated to smaller discharges can result in hydraulic inaccuracies.

The magnitude of the 100- and 500-year discharges and the geometry of the bridge openings can create scenarios that are difficult to model with HEC-RAS or other programs. Supercritical flow, velocities in excess of 10 ft/s, hydraulic jumps, and pressure flow are conditions found at many sites in this study that can vary three-dimensionally in both time and length scales. For highly varied flow, the one-dimensional modeling solution averages the momentum equation over the channel instead of computing the velocity at each point. The velocity at a pier could be greater or less than this averaged velocity. One-dimensional models capture a small portion of the active processes in the channel, but are efficient at making predictions over long length and time scales (Nelson and others, 2003). The hydraulic variables generated by HEC-RAS for adverse flow conditions are considered the best available for this level of study, and multi-dimensional modeling should be considered for additional studies at locations with large estimates of scour and complex flow regimes.

Insufficient channel-geometry data limits the accuracy of hydraulic data. Most of the Phase 1.5 sites lack channel-geometry data upstream and downstream of the bridge and rely on templating, or copying, and expanding the surveyed bridge section on the basis of field observations and channel slope or information from the ADOT&PF bridge as-built plans. Data available from the bridge as-built plans vary from site to site and were limited in upstream and downstream extent from the bridge. Copying cross sections from the bridge data is particularly inadequate for sites where the channel is constricted significantly by the roadway approaches to the bridge. The depth of the channel at the bridge section will be greater than that in the approach and exit sections, and copying this section through the reach results in an over-estimation of channel area and overall lower modeled water surfaces and flow velocities. For many locations, cross-section end points were extended vertically because the as-built plans did not include overbank topography. This procedure accommodated the recurrence-interval discharges within the channel, but resulted in a greater amount of effective flow in the channel. For sites with significant channel contraction or overbank flow, the Phase 2 analysis is more appropriate than the Phase 1.5 analysis because it includes approach, exit, and overbank channel geometries.

The extent of geometric data available was a limiting factor for models of pressure flow. The bridge section surveyed for the Phase 1.5 sites rarely extended laterally beyond the bridge itself. Without these data, flow over the roadway approaches to the bridge could not be modeled and all discharge was routed through the bridge section. If the roadway approaches were lower than the bridge deck, enough discharge could pass over this area to limit or eliminate pressure flow. Phase 1.5 pressure-flow sites would be better evaluated with the addition of detailed hydraulic cross sections and roadway geometry.

Hydraulic models are sensitive to the value selected for channel roughness. An overestimate of roughness will produce models with lower flow velocities, higher water-surface elevations, and greater flow widths than may occur naturally. Roughness values used in the hydraulic models for this study were calibrated to measured data when possible, rather than relying on estimation procedures. Roughness values were calibrated for the discharge at the time of the survey and used for the simulations of the 100- and 500-year flood flows. Calibration discharge at 25 of the Phase 1.5 sites and 15 of the Phase 2 sites was less than 25 percent of the 100-year discharge. Calibration of channel roughness to these low discharges for the flood models was likely an overestimation because channel roughness normally decreases with increasing depth. In many cases, flow in the overbanks, where the roughness is generally higher, compensates for the decreased roughness in the channel that results from the greater flow depths. For braided channels, the flow tends to expand rather than deepen and roughness values do not decrease, except through constricting bridge sections. To determine the sensitivity of pier and contraction scour computations to the value computed for channel roughness, six sites were selected for additional hydraulic modeling. These sites had Manning’s roughness values ranging from 0.045 to 0.080 and were calibrated to discharges less than 20 percent of the 100-year discharge. The hydraulic models for these six sites were rerun with a Manning’s roughness of 0.035, which was the default value used in the Phase 1 analysis (Heinrichs and others, 2001). The hydraulic variables from these simulations were then used to recalculate pier and contraction scour. The hydraulic variables and resulting computed values for pier and contraction scour for the original simulations using the calibrated roughness values and those from the second simulations using the default roughness of 0.035 are presented in table 2.

Table 2. Sensitivity analysis of channel-roughness values for six bridge sites in Alaska in the Phase 1.5 scour analysis where the calibration discharge is less than 20 percent of the 100-year recurrence-interval discharge

[**ADOT & PF** No.: Alaska Department of Transportation and Public Facilities. ft^3/s, cubic feet per second; ft, feet; ft/s, feet per second]

ADOT & PF No.	Manning's roughness coefficient *n*	Average approach depth (ft)	Approach velocity (ft/s)	Approach channel width (ft)	Bridge average depth (ft)	Bridge section width (ft)	Depth of contraction scour (ft)	Depth upstream of pier (ft)	Velocity upstream of pier (ft/s)	Froude number at bridge	Depth of pier scour (ft)
Phase 1.5 calibrated roughness											
543	0.055	1.9	7.7	144	1.8	136	0.2	4.7	8.6	0.70	9.1
556	.064	12.0	6.9	292	10.7	215	3.7	17.3	13.5	.57	3.7
557	.049	7.3	9.1	333	7.5	334	.0	14.1	11.7	.55	7.1
694	.050	11.4	8.0	281	11.6	253	.5	15.1	9.0	.41	11.0
742	.080	13.8	5.5	[1]415	12.7	463	.6	16.6	6.3	.27	4.3
999	.045	5.4	5.5	236	5.2	157	1.6	8.7	10.5	.63	11.9
Phase 1 default roughness values from Phase 1											
543	0.035	1.6	9.0	130	1.7	109	0.1	3.7	14.4	1.33	10.9
556	.035	10.1	8.5	284	8.9	204	3.4	15.2	17.3	.78	4.0
557	.035	5.8	12.0	319	5.9	319	.0	12.3	15.4	.77	7.7
694	.035	9.5	10.2	262	9.4	238	.7	12.4	11.6	.58	12.0
742	.035	8.2	9.6	415	7.5	441	.5	11.0	11.4	.61	5.3
999	.035	4.8	6.3	232	4.5	151	1.7	7.9	12.3	.77	12.6

[1]Approach channel width determined from as-built survey plans.

Decreasing the channel roughness from the calibrated value to the default estimate had an equal influence on all the hydraulic variables used in the computation of live-bed contraction scour. For example, a decrease in flow depth was accompanied by a decrease in approach channel width. Sensitivity analysis of equation 2 shows that all variables have a moderate to significant influence on the contraction scour depth (Glenn, 1994). Therefore, the accuracy of the channel roughness was not critical to the computation of contraction scour at most sites. However, the geometry of the approach channel at some locations may have limited changes in the approach width. Under these conditions, the approach depth, rather than the approach width was more responsive to variance in channel roughness, and the computation of contraction scour was affected.

The computation of pier scour was more responsive than the computation of contraction scour to variances in channel roughness. Flow depth upstream of the pier and the Froude number at the bridge were sensitive to changes in roughness, whereas most of the other variables in equation 5 remained constant. The Froude number was used in equation 5 to represent the approach flow velocity, which increased with decreased channel roughness. Sensitivity analysis of equation 5 indicated that the approach velocity's influence on pier scour was second only to pier width, which was a constant for most scenarios (Glenn, 1994). Decreasing channel roughness increased pier scour by as much as 20 and 23 percent for bridges 543 and 742, respectively, and by less than 10 percent for the other four sites (table 2).

Phase 1.5 and Phase 2 Analyses and Evaluation

Scour depth was computed for contraction scour, pier scour, pressure scour, and complex pier scour for 100- and 500-year floods at 35 Phase 1.5 and 19 Phase 2 bridge sites. Computed scour depths are summarized in table 3 and complete calculations are included in tables 5-8 (at back of report). Computed contraction and pier-scour depths greater than 5 and 10 ft, respectively, were used as criteria to identify Phase 1 bridges for further analysis. Computed contraction scour depths for Phase 1.5 and Phase 2 sites were greater than 5.0 ft at six sites for the 100-year discharge. Computed pier-scour depths were greater than 10.0 ft for the 100-year discharge at 23 bridges. Complex pier-scour depth was computed for six sites where the pier-footing elevation was above the lowest streambed elevation after the contraction-scour calculations. Pressure scour was evaluated at 10 bridges, but pressure-flow conditions persisted at only three sites after contraction-scour depths were accounted for. Total maximum scour at a bridge in this study was the summation of the contraction and pier-scour depths. For pressure-flow sites, the pressure-scour depth was added to the contraction and pier-scour depths. When the pier footing is exposed to the flow, the complex pier-scour depth should be used in place of the pier-scour depth.

Potential scour at bridges is estimated using equations derived mainly from simplified studies in laboratory flumes. The applicability of these equations to actual conditions is limited. The scour estimates tend to be conservative when compared with the limited field measurements of scour that have been made. Mueller (1996) compared predictions from the pier-scour equations recommended in HEC-18 with field data and concluded that the scour estimates rarely under-predicted measured scour depths. In a study of measured versus predicted pier scour in New Hampshire, Boehmler and Olimpio (2000) also found that the HEC-18 equations consistently over-predicted scour. In a study of streambed scour at bridges in Alaska, Norman (1975) found that equation 2 is a good estimate of contraction scour at streams with gravel or cobble beds. Results from a comparison of measured and calculated scour depths for two bridges on the Copper River in Alaska by Brabets (1994) varied. Equation 3 provided the closest value to measured pier scour at one bridge and under-predicted scour by 25 percent at the other bridge (Brabets, 1994). Results from a comparison of measured and predicted contraction scour for the Copper River study also varied. Equation 2 over-predicted scour at the first bridge by 3.4 ft and at the second bridge by only 0.5 ft (Brabets, 1994).

The effectiveness of the contraction-scour predictions for the Phase 1.5 analysis was limited by the lack of channel and overbank data at the approach to the bridges. Channel widths typically were estimated in the field or from the bridge as-built plans rather than being surveyed. Estimating approach-channel widths from bridge as-built plans can be insufficient to predict contraction scour accurately. Even with a recent and accurate bridge plan it is difficult to accurately delimit the portion of the channel that is actively transporting sediment. Limited or no overbank data in the approach to the bridge resulted in modeled discharges and depths larger than what would be expected for actual conditions where some portion of the flow would extend beyond the main channel. Channel and overbank data in the approach to the bridge that are required for an accurate estimate of contraction scour using the methodology recommended by HEC-18 were collected for the Phase 2 analysis.

Uncertainty in roughness values and channel-flow angle of attack on piers at high flows influenced the computation of local scour at piers or the Phase 1.5 and Phase 2 analyses. Based on the sensitivity analysis of pier scour to changes in channel roughness (table 2) and the comparisons of measured to calculated values discussed previously, uncertainty in channel-roughness estimates for higher discharges will produce a result that is within the resolution of equation 5. Channel-flow angle of attack was a significant factor in the calculation of pier scour and varied with changes in stage and flow conditions. Changes in channel-flow angle of attack could not be determined without field verification at higher stages or multi-dimensional flow modeling. Estimates of channel-flow angle of attack for each site were based on the measured value and the channel morphology upstream of the bridge that would control the angle at higher discharges.

Estimates of complex pier scour were made for sites where the footing elevation was above the streambed after the computed contraction scour for the 100- and 500-year floods was subtracted from the lowest point on the as-built cross section at the bridge. Identifying complex scour sites by this method was a conservative approach for locations where the thalweg was not located near a pier and channel migration was not of concern. The complex pier-scour estimate, rather than the normal pier-scour estimate, should be used at the sites in table 3 unless the thalweg is not in proximity to bridge piers and long-term observation indicates channel stability.

Table 3. Summary of computed scour depths for 100- and 500-year recurrence-interval discharges for the Phase 1.5 and Phase 2 streambed scour analyses at selected bridge sites in Alaska

[**ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. ft, feet. –, no data]

ADOT & PF No.	Depth (ft)									
	Contraction scour		Pier scour		Pressure scour		Complex pier scour		Total scour	
	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Phase 1.5										
205	0.7	0.9	15.1	15.6	–	–	26.9	28.1	27.6	29.0
240	.0	.0	12.1	12.7	–	–	–	–	12.1	12.7
277	6.9	13.3	–	–	–	–	–	–	6.9	13.3
308	.7	.9	16.2	17.4	–	–	–	–	16.9	18.3
396	.9	1.3	12.2	13.1	–	–	–	–	13.1	14.4
401	1.9	6.4	–	–	–	8.1	–	–	1.9	14.5
505	6.0	6.5	8.9	9.1	–	–	14.3	14.6	20.3	21.1
509	.0	.0	24.9	26.9	–	–	–	–	24.9	26.9
518	.0	.0	20.8	21.5	–	–	–	–	20.8	21.5
520	.0	.0	9.1	9.6	–	–	–	–	9.1	9.6
530	.0	.0	10.2	10.7	–	–	–	–	10.2	10.7
543	.0	.0	9.1	9.4	–	–	–	–	9.1	9.4
544	.0	.0	7.2	7.2	–	–	13.9	14.4	13.9	14.4
547	.0	.0	12.8	13.4	–	–	–	–	12.8	13.4
548	3.6	4.3	32.6	33.9	–	–	–	–	36.1	38.2
556	2.3	2.5	4.0	4.2	–	–	–	–	6.2	6.7
572	.0	.0	14.1	14.5	–	–	14.8	15.0	14.8	15.0
573	8.3	10.6	12.2	12.8	–	–	–	–	20.4	23.4
603	.6	.6	10.1	10.6	–	–	–	–	10.7	11.2
658	1.7	2.3	–	–	0	.2	–	–	1.7	2.5
678	2.2	3.0	3.5	3.8	2.1	.2	–	–	7.8	7.0
686	.8	1.0	3.3	3.4	–	–	–	–	4.1	4.4
687	1.4	1.6	13.0	13.8	–	–	–	–	14.4	15.4
690	12.0	11.6	–	–	–	–	–	–	12.0	11.6
742	.0	.0	4.3	4.6	–	–	–	–	4.3	4.6
832	.6	.8	–	–	–	–	–	–	.6	.8
833	1.9	1.9	–	–	–	–	–	–	1.9	1.9
999	1.5	1.8	11.9	12.7	–	–	–	–	13.4	14.5
1220	2.4	2.3	3.1	3.1	–	–	–	–	5.5	5.4
1261	1.1	1.2	9.4	9.7	–	–	–	–	10.6	11.0
1282	.0	.0	19.9	20.5	–	–	–	–	19.9	20.5
1283	1.2	1.9	3.3	3.6	–	–	–	–	4.5	5.5
1284	2.2	2.4	3.4	3.5	–	–	–	–	5.6	5.9
1329	.4	.6	10.3	11.0	–	–	–	–	10.6	11.6
1389	.0	.0	–	–	–	–	–	–	.0	.0

Table 3. Summary of computed scour depths for 100- and 500-year recurrence-interval discharges for the Phase 1.5 and Phase 2 streambed scour analyses at selected bridge sites in Alaska–*Continued*

[**ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. ft, feet. –, no data]

ADOT & PF No.	Depth (ft)									
	Contraction scour		Pier scour		Pressure scour		Complex pier scour		Total scour	
	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Phase 2										
233	6.5	6.4	8.1	9.2	–	–	–	–	14.6	15.6
242	.8	.9	4.4	4.5	–	–	–	–	5.1	5.4
254	5.0	5.3	17.6	18.1	–	–	–	–	22.6	23.4
255	.0	.0	20.1	20.9	–	–	–	–	20.1	20.9
355	1.6	1.9	8.5	8.9	–	–	–	–	10.1	10.8
527	2.7	3.1	18.0	19.0	–	–	–	–	20.7	22.1
535	2.6	3.2	15.0	15.0	–	–	–	–	17.6	18.2
539	12.0	14.4	11.4	12.7	–	–	20.3	31.2	32.3	45.6
557	.0	.0	7.1	7.3	–	–	–	–	7.1	7.3
558	.0	.0	11.7	12.2	–	–	–	–	11.7	12.2
574	2.7	3.0	6.8	7.1	–	–	–	–	9.5	10.1
608	.0	.0	6.5	6.8	–	–	–	–	6.5	6.8
609	.4	.5	3.7	3.9	–	–	–	–	4.1	4.4
694	.9	1.1	11.0	11.5	–	–	19.8	20.5	20.7	21.6
1025	.0	.0	6.6	6.9	–	–	–	–	6.6	6.9
1147	1.4	1.8	7.2	7.2	–	–	–	–	8.6	9.0
1255	.5	.5	9.4	9.6	–	–	–	–	9.9	10.1
1341	1.5	1.7	8.8	9.3	–	–	–	–	10.3	11.0
1513	2.2	3.3	–	–	–	–	–	–	2.2	3.3

Limited research has been done on the vertical contraction scour that results from full or partial submergence of a bridge. Arneson and Abt (1999) have conducted the most extensive study to date of pressure-flow scour under live-bed conditions, and their equation is recommended in the current edition of HEC-18. Contraction scour and pressure scour were evaluated independently because submergence of the bridge structure impedes the flow and can create additional backwater at the bridge constriction. This backwater can reduce the approach velocities below the critical velocity of the bed material required for contraction scour. Contraction scour will occur up to a point, until the flow changes to pressure flow and backwater conditions develop. Evaluating contraction scour independently, and then modifying the channel to reflect the scour, eliminated modeled pressure-flow conditions at several bridges. Calculations of pressure scour are summarized in table 7 for the three sites where pressure-flow conditions persisted after the channel area was increased to reflect the contraction scour.

Comparison of Phase 1.5 and 2 Analyses with Phase 1 Analysis

The Phase 1 analysis was a rapid, cost-effective assessment of scour susceptibility using existing data. The quality of this assessment was based on the quantity and accuracy of data from the bridge as-built plans and other sources. Most bridges in this study were built across dynamic channels that could be aggrading, degrading, and migrating laterally. These changes in the channel can be a response to the change in hydraulics caused by the bridge structure or to more long-term adjustments that could be related to tectonics or changes in flow regime or sediment supply. The lowest elevation from the as-built bridge survey for the streambed at the bridge was compared with the elevation measured during the site investigation for the Phase 1.5 and Phase 2 analyses, and the change in elevation is shown in figure 2.

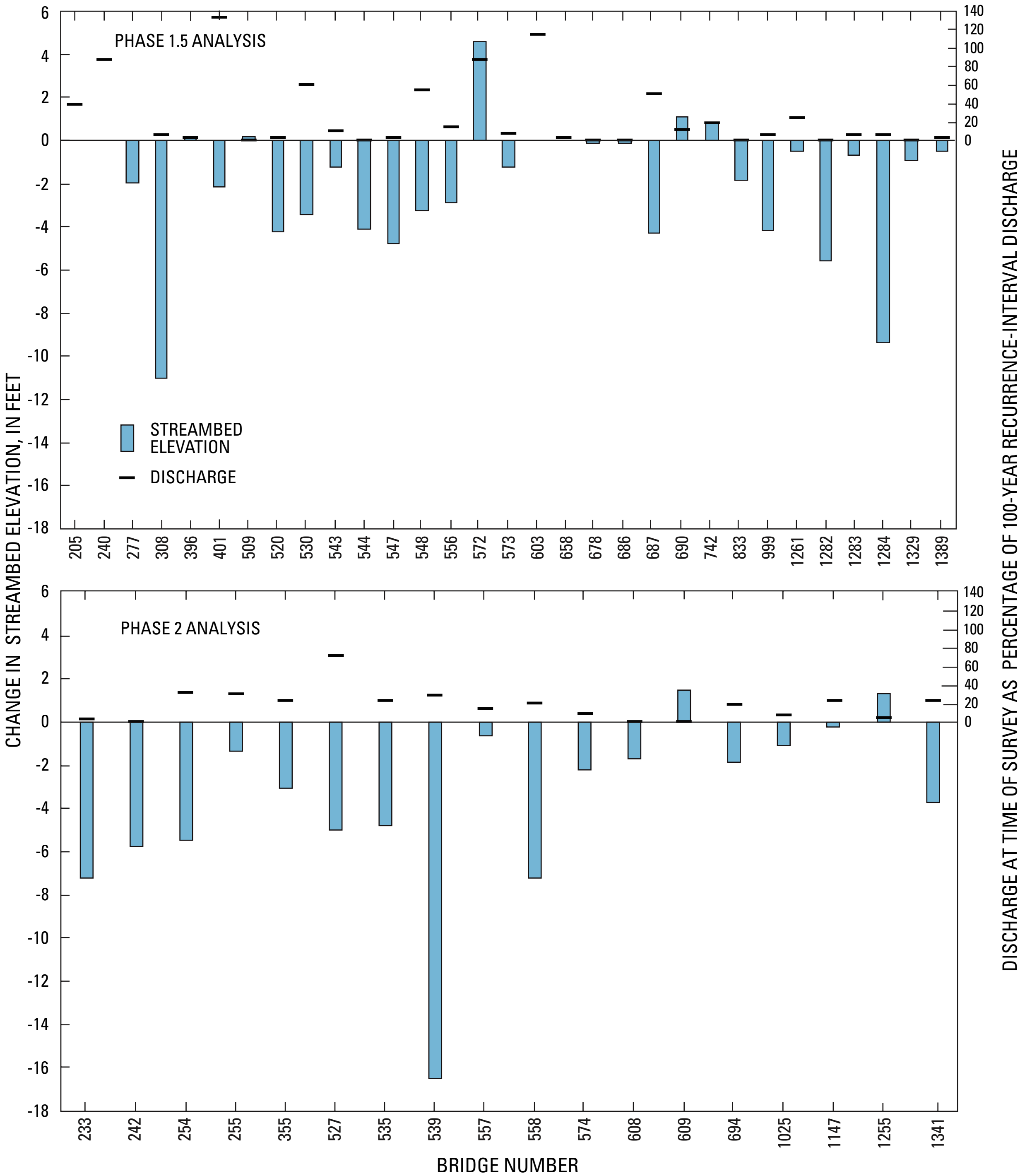

Figure 2. Change in streambed elevation at bridge between the lowest elevation streambed from the as-built bridge survey and the elevation measured for the Phase 1.5 or Phase 2 for selected bridge sites in Alaska.

Discharge at the time of measurement is shown as percentage of the 100-year recurrence-interval discharge. Five locations that were not resurveyed for the Phase 1.5 analysis are not shown.

The discharge measured during the Phase 1.5 or Phase 2 analysis is included on figure 2 as a percentage of the 100-year recurrence-interval flood discharge to illustrate that the measured elevation used in this study may not have necessarily been obtained during a high flow event. A general degradation in channel depth can be seen for most sites. Determining the process behind the degradation was beyond the scope of this study, but these data confirm the dynamic nature of these channels. Scour values presented in this study were in reference to the streambed elevation from the as-built plans and did not take into consideration the channel changes measured during the Phase 1.5 or Phase 2 analyses.

Comparison of Hydraulic Models

Hydraulic variables used in the scour computations for the Phase 1 analysis were calculated using the step-backwater water-surface profile (WSPRO) model (Shearman, 1990). WSPRO is a water-surface profile computational model for one-dimensional, gradually varied, steady flow in open channels. WSPRO and HEC-RAS are both sufficient for determining hydraulic variables for subcritical flow and flow that does not intersect the bridge structure. In situations where the flow is supercritical, changes to supercritical through the reach, or intersects the bridge structure, HEC-RAS provides a more rigorous hydraulic analysis with multiple methods for determining water-surface profiles.

Comparison of HEC-RAS and WSPRO outputs for a site is difficult because the data used to construct the models are not at the same level of detail. The hydraulic parameters used to construct the models can be compared. Comparison of the estimated channel-roughness values for the Phase 1.5 analysis and calibrated channel-roughness values for the Phase 2 analysis with values from Phase 1 (either an assumed channel roughness of 0.035 or a value estimated from existing information) indicated an average increase from the Phase 1 values (fig. 3). The surveyed channel slopes used for the Phase 1.5 and Phase 2 analyses were compared with the slope estimated from 1:63,000 topographic maps and used to copy cross sections for Phase 1, and the changes varied greatly (fig. 3). The variation in surveyed slope values when compared to those measured from topographic maps illustrated the importance of this field measurement for accurate templating of channel geometry and determination of roughness coefficients using the Manning's equation.

Model output and detail of data used to generate it can be compared generally by evaluating the modeled Froude number at the bridge for the different levels of analysis. The magnitude of the Froude number was affected by channel slope, roughness values, and method used to compute the water-surface profile. The Froude values for Phase 1 and Phase 1.5 analyses were in good agreement for values less than 0.5, but the variance increased for higher Froude values (fig. 4). (Modeled Froude values for the Phase 1.5 and Phase 2 analyses did not exceed 1.0.) Several of the outliers on figure 4 were locations with modeled pressure flow that were evaluated under a different procedure for the Phase 1.5 analysis. The Froude values for Phase 2 showed less agreement with those from the Phase 1 models, but generally tended to be lower numbers that were attributed to the increased channel roughness values selected for the Phase 2 analyses (fig. 5).

Contraction Scour

The validity of the contraction-scour depths determined in the Phase 1 analysis was limited by the extent, availability, and age of channel and overbank data for the approach section of the river. This computation was improved for the Phase 1.5 analysis by including field estimation of approach-channel widths and for the Phase 2 analysis by including surveys of the approach channel. Comparison of the approach widths for the Phase 1 analysis and the Phase 1.5 and 2 analyses showed a general increase in the widths in the later analyses (fig. 6).

At several sites, contraction-scour depths for the 100- and 500-year flood discharges computed for the Phase 1 and Phase 1.5 analyses differed greatly (fig. 7). The following three factors, determined through a sensitivity analysis of input variables, accounted for the difference in computed scour between the analyses.

1. Channel geometry in the approach: Increases or decreases in the Phase 1.5 modeled flow width in the approach channel or bridge section explained the differences in computed scour for most of the sites (fig. 7).
2. Channel geometry at the bridge: Changes in the modeled bridge-section width reflect measured channel aggradation or degradation in the bridge section since the time of the survey for the bridge as-built plans.
3. Pressure flow: Procedures by which the sites with pressure-flow conditions were evaluated explain differences in computed scour at some sites. Contraction scour was not evaluated independent of the pressure-flow conditions for the Phase 1 analysis, and the values were based on the backwater conditions that develop when the flow intersects the bridge.

The difference in scour values at one site was attributed to a three-fold increase in the measured slope of the channel and resultant change in hydraulic variables. Bridges that have estimated scour-depth values in excess of 5 ft for the Phase 1 or 1.5 level analyses would be better evaluated with the inclusion of detailed hydraulic cross sections. Detailed hydraulic cross sections would also improve simulations of pressure flow for Phase 1.5 sites.

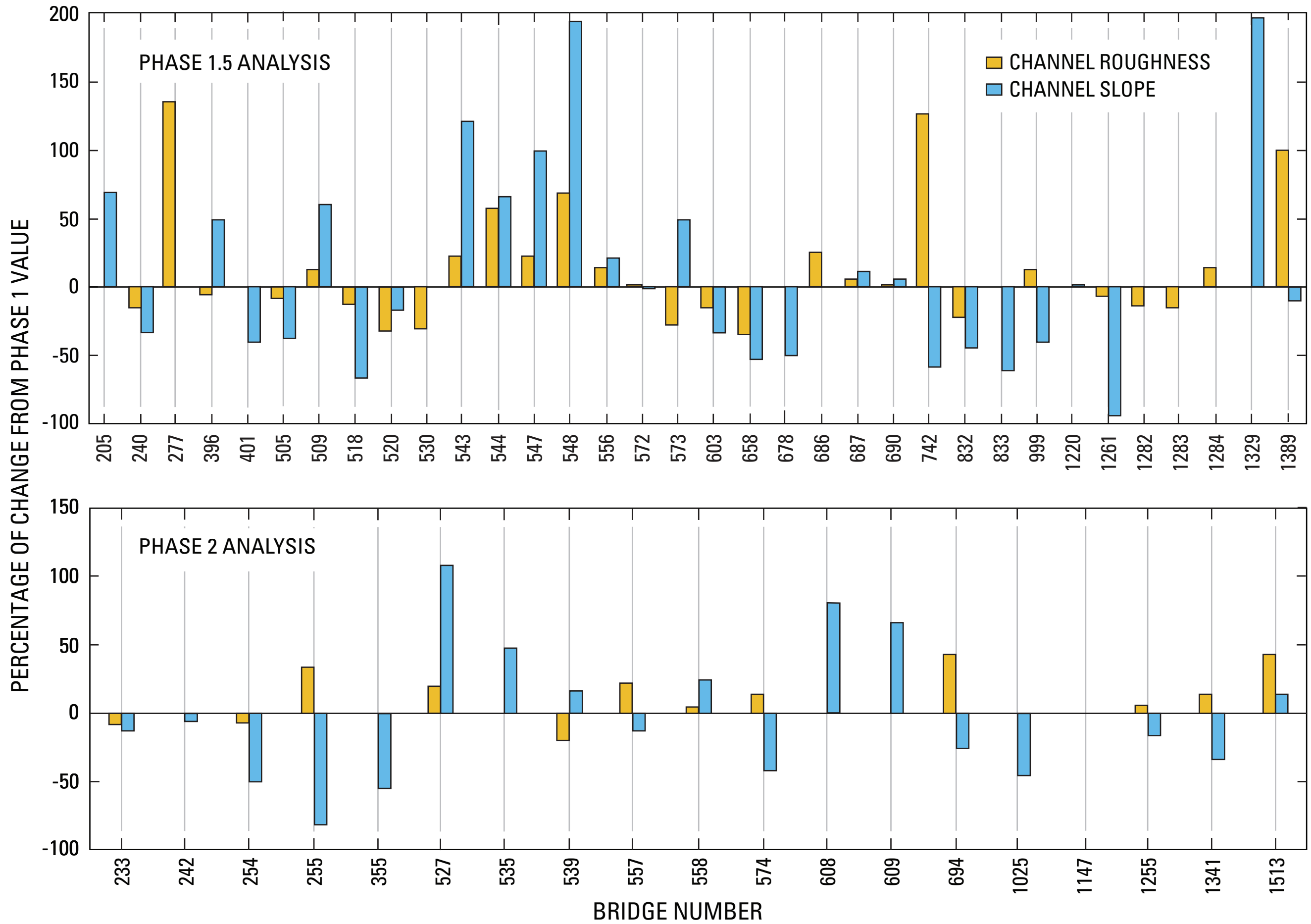

Figure 3. Percentage of change between Phase 1 values and Phase 1.5 and Phase 2 values for channel slope and roughness at bridge for selected bridge sites in Alaska.
Note that slope for the Phase 1.5 Bridge 1329 increases 600 percent.

The field measurement of the approach-channel geometry in the Phase 2 models resulted in the most significant differences in contraction-scour depth relative to Phase 1 models. Phase 1 contraction scour was overestimated when the Phase 2 data indicated the approach channel was narrower than the geometry determined from the as-built bridge plans. Phase 1 contraction scour was overestimated at one site when the Phase 2 data indicated the channel roughness was lower than the Phase 1 value (fig. 8). Results from the Phase 2 contraction scour computations generally were lower at sites where the Phase 1 estimate of scour was greater than 4.0 ft.

Pier Scour

The estimation of pier scour by equation 5 was computed using hydraulic variables from HEC-RAS and pier dimensions, pier shape, and the channel-flow angle of attack. The pier dimensions and shape are consistent for a site for the different levels of analysis, the exception being sites where further investigation indicated a need to model a group of piers as one solid pier or vice versa. The hydraulic variables flow depth and Froude number can be used to show a difference between the levels of analysis based upon the hydraulic parameters used to generate the model. For the Phase 1 analysis, the channel-flow angle of attack that was estimated from the as-built plans was measured in the field for the Phase 1.5 and 2 analyses.

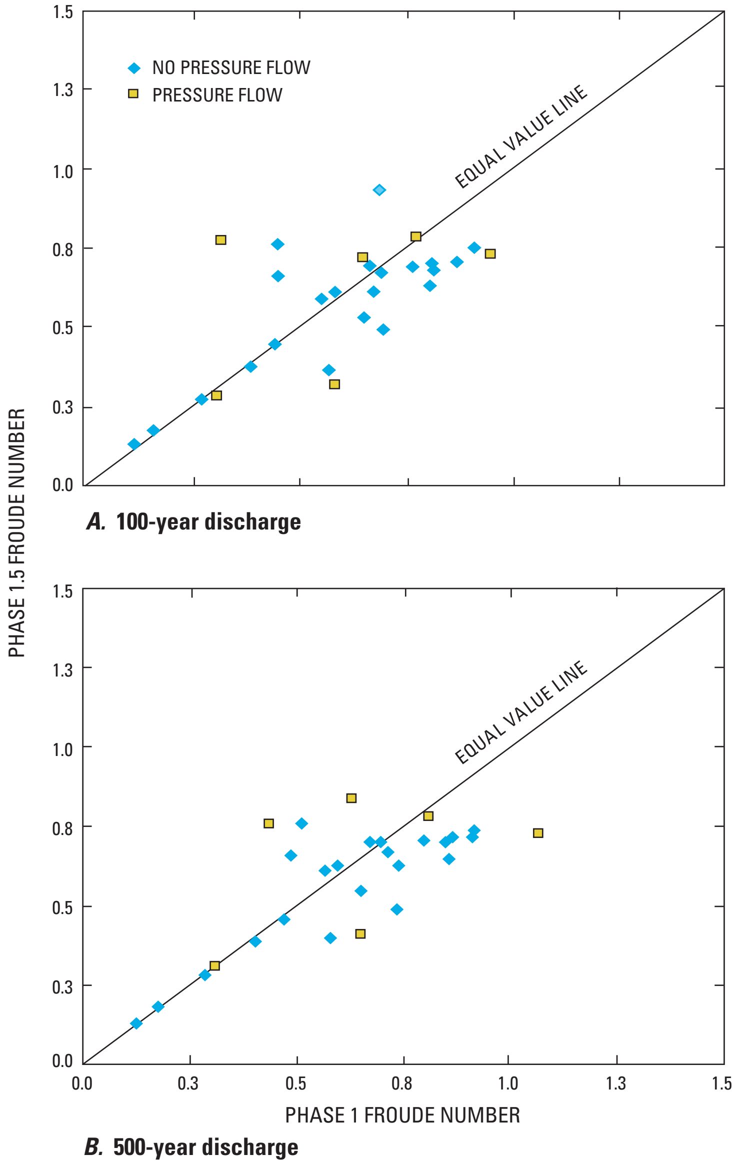

Figure 4. Comparison of modeled Froude numbers at the bridge, determined by Phase 1 and Phase 1.5 analyses for the 100-year and 500-year recurrence-interval discharge for selected bridge sites in Alaska.

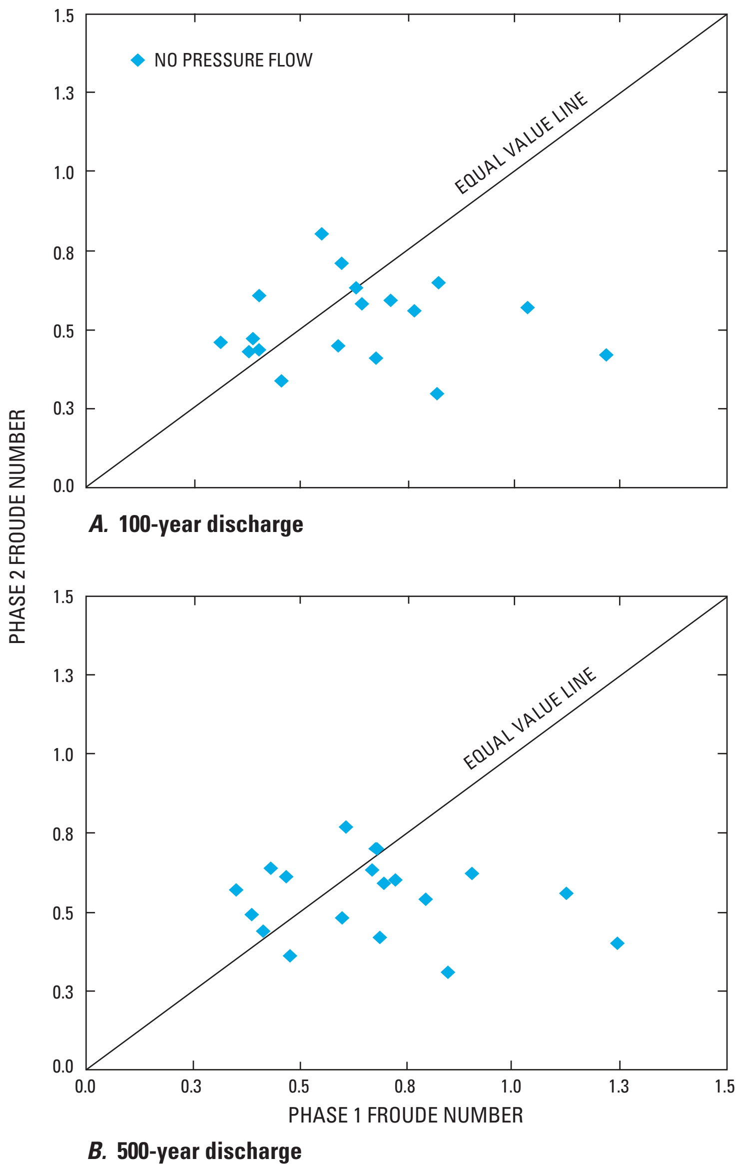

Figure 5. Comparison of modeled Froude numbers at the bridge, determined by Phase 1 and Phase 2 analyses for the 100-year and 500-year recurrence-interval discharge for selected bridge sites in Alaska.

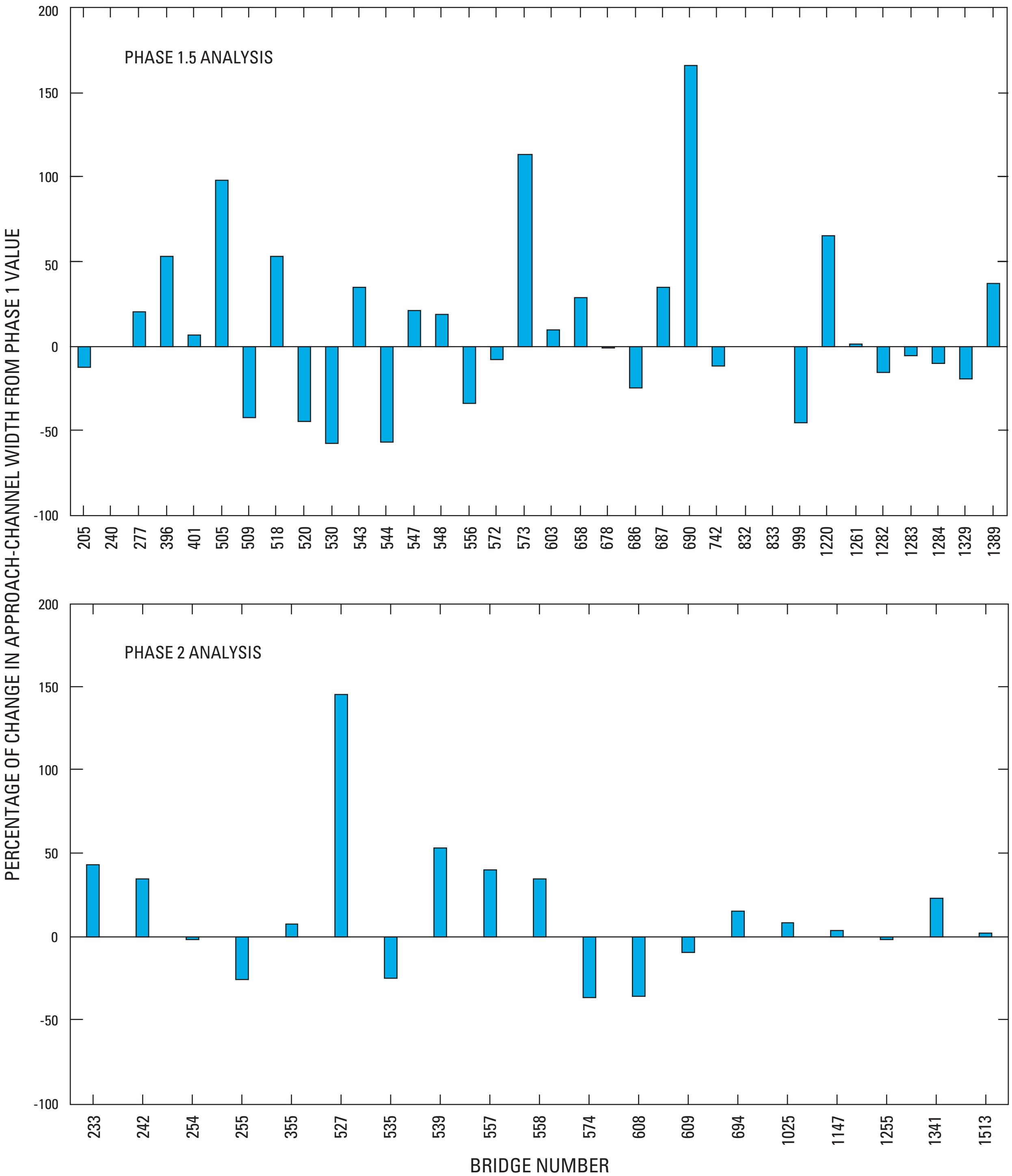

Figure 6. Percentage of change in approach-channel width between the Phase 1 analysis and the Phase 1.5 and Phase 2 analyses for selected bridge sites in Alaska.

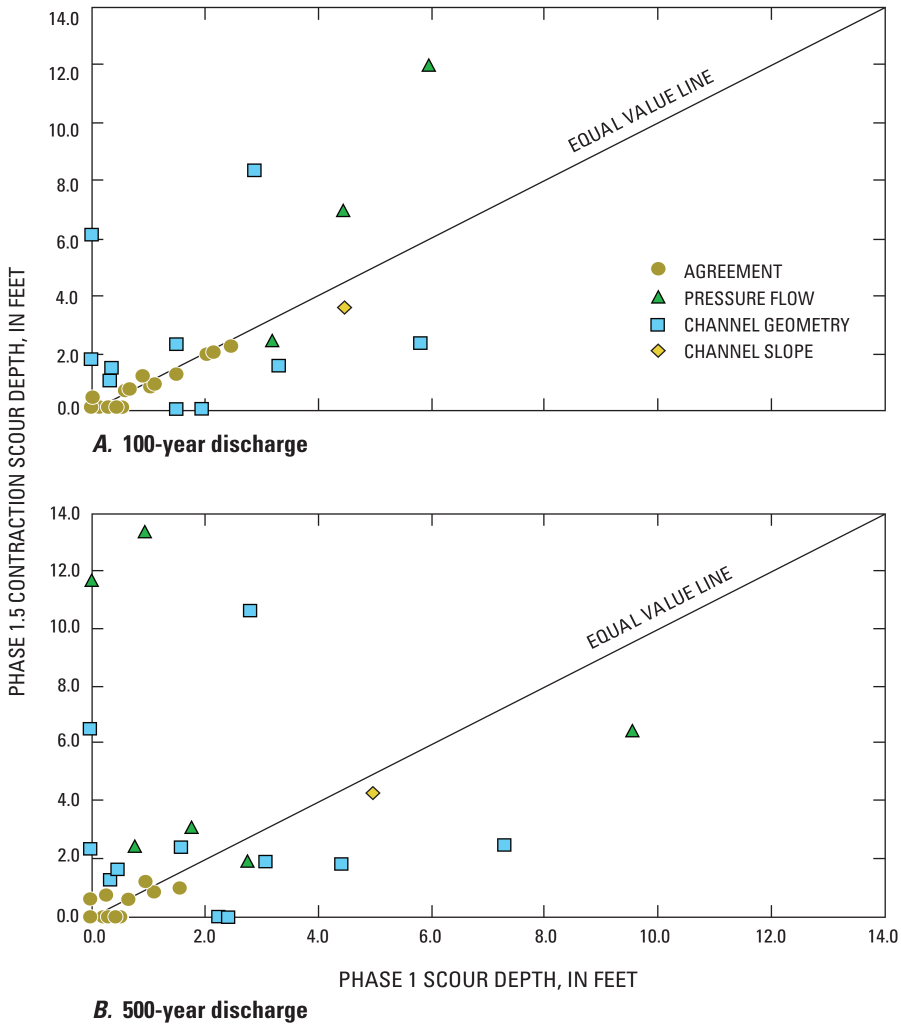

Figure 7. Comparison of contraction-scour depths computed by Phase 1 and Phase 1.5 analyses for the 100-year and 500-year recurrence-interval discharge and factor responsible for the difference in scour depths for selected bridge sites in Alaska.

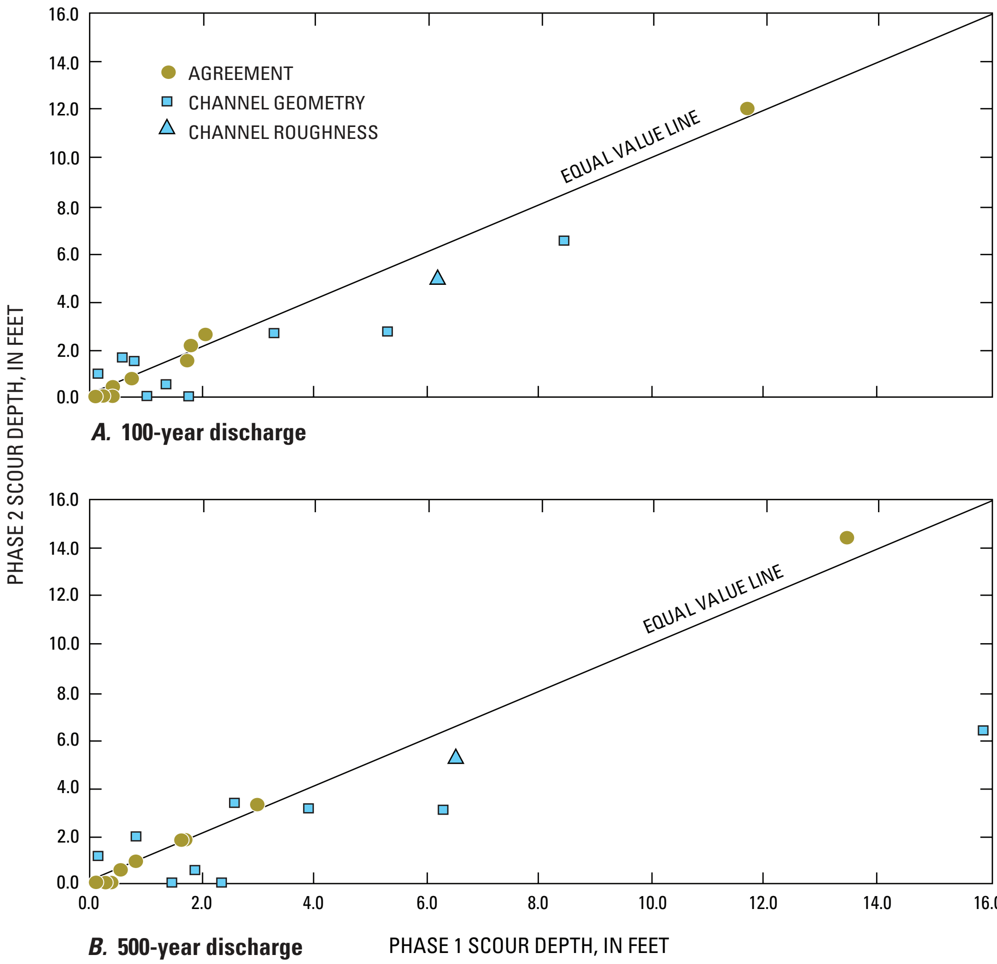

Figure 8. Comparison of contraction-scour depths computed by Phase 1 and Phase 2 analyses for the 100-year and 500-year recurrence-interval discharges and factor responsible for the difference in scour depths for selected bridge sites in Alaska.

Comparisons of pier-scour depths for the 100- and 500-year discharges for the Phase 1 and Phase 1.5 analyses are shown in figure 9 and for the Phase 1 and Phase 2 analyses in figure 10. There was a positive association between the levels of analysis, with a few outliers. Sensitivity analysis on these outliers determined that angle of attack, Froude number, and pier shape were responsible for the variance in the scour-depth values between the levels of analysis.

The channel-flow angle of attack at a bridge accounted for most of the difference between scour values for the different levels of analysis in this study (figs. 9 and 10). Increases or decreases from the angle used for the Phase 1 study can be attributed to a better measure of the value from field observations and also to migration of the channel since the as-built survey of the bridge. Field verification, preferably at high flow, was critical for an accurate estimate of angle of attack. The Phase 1 analysis used dated survey plans that do not reflect the current flow conditions at sites with dynamic channels.

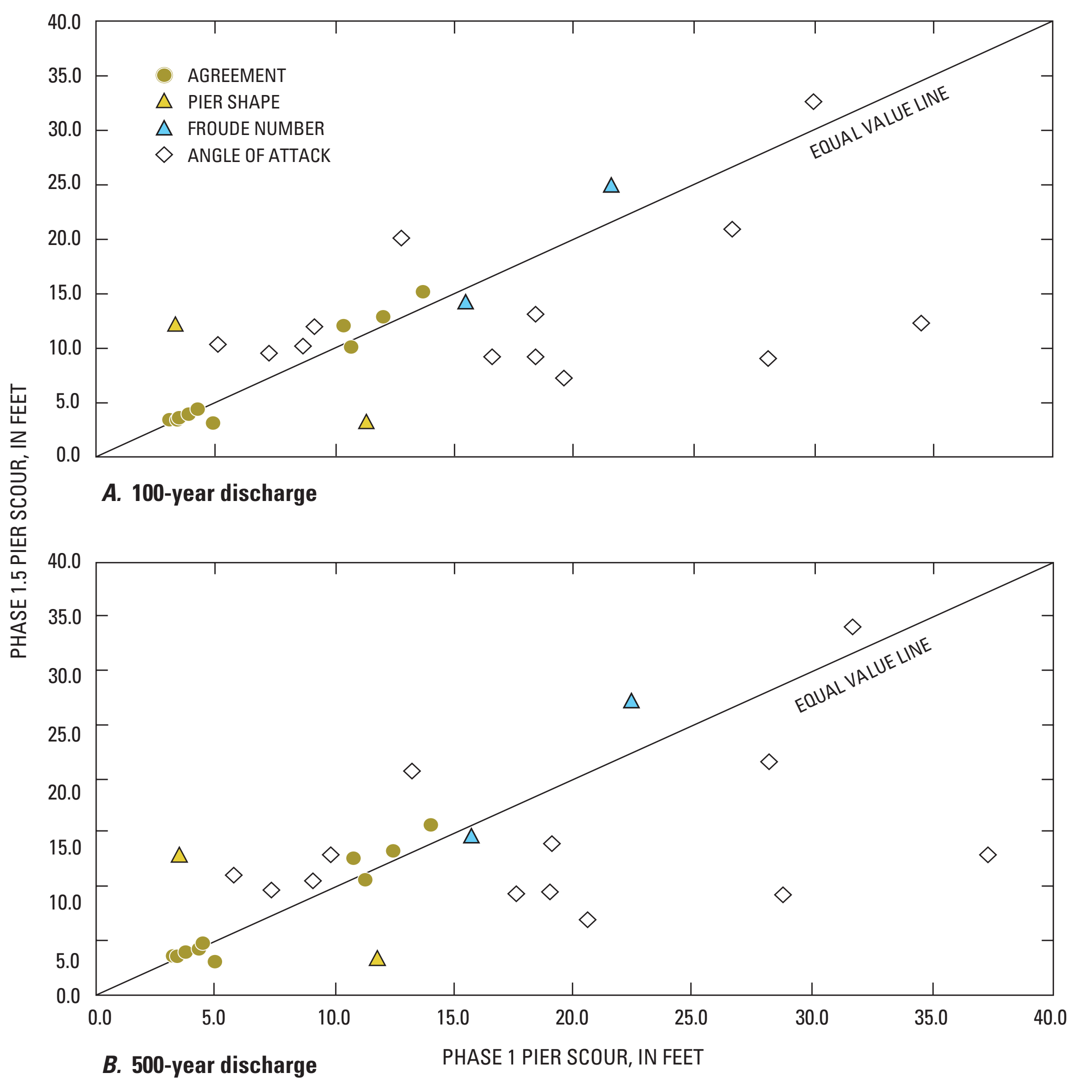

Figure 9. Comparison of pier-scour depths computed by Phase 1 and Phase 1.5 analyses for the 100-year and 500-year recurrence-interval discharges and factor responsible for the difference in scour depths for selected bridge sites in Alaska.

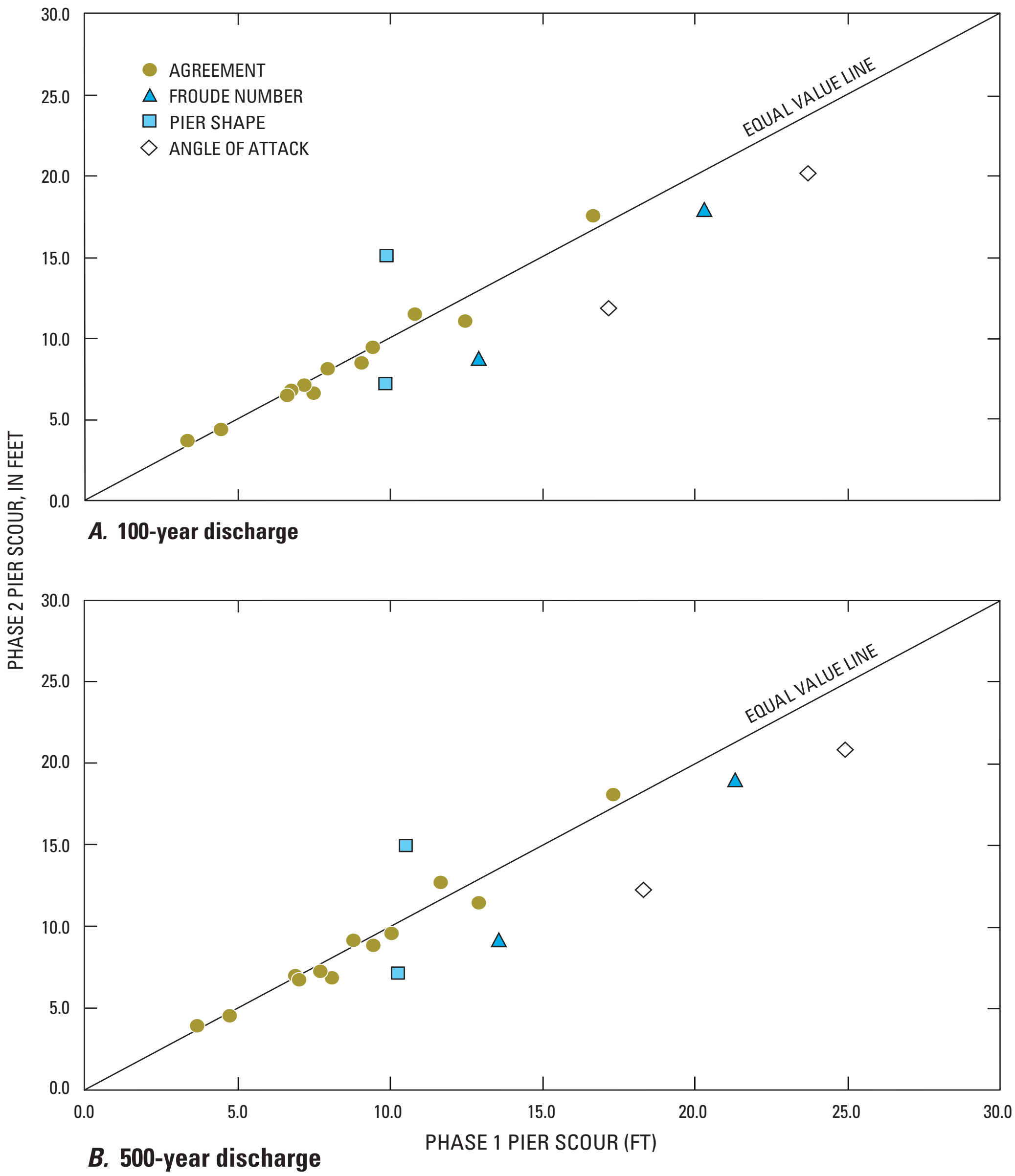

Figure 10. Comparison of pier-scour computed by Phase 1 and Phase 2 analyses for the 100-year and 500-year recurrence-interval discharges and factor responsible for the difference in scour depths for selected bridge sites in Alaska.

Summary and Conclusions

The U.S. Geological Survey began studying the scour susceptibility of bridges in Alaska in 1994. The initial phase of this project used existing data to perform a preliminary Phase 1 scour assessment at 325 bridges. Based on these assessments, the Alaska Department of Transportation and Public Facilities (ADOT&PF) selected 54 bridges for further analysis. These analyses included on-site inspection and collection of detailed hydraulic cross sections. The bridges were evaluated at either the Phase 1.5 or Phase 2 level analysis. Division into these two groups was based on the magnitude of measured scour and the calculated scour depths from Phase 1. The Hydrologic Engineering Center's River Analysis System (HEC-RAS) was used to calculate the hydraulic variables needed to compute estimates of scour for 100- and 500-year recurrence-interval flood discharges. Contraction scour, pier scour, complex pier scour, and pressure scour were calculated using federally recommended techniques and equations outlined in the Federal Highway Administration's Hydraulic Engineering Circular No. 18 (HEC-18).

The less-intensive Phase 1.5 analysis was completed at 35 of the 54 bridges. Data collection included a discharge measurement, water-surface slope survey, verification of the channel-flow angle of attack, and estimation of the approach-channel width. Computed contraction scour depths for the 100-year flood discharge were greater than 5 feet for four bridges and pier-scour depths were greater than 10 feet for 16 bridges.

The Phase 2 analysis was performed at 19 bridges. Data collection for the Phase 2 analysis was analogous to the Phase 1.5, with the addition of detailed hydraulic cross sections surveyed upstream and downstream of the bridge. Computed contraction-scour depths from the 100-year flood discharge were greater than 5 feet at two bridges and pier-scour depths were greater than 10 feet at seven bridges.

Comparing the analysis methodologies and results of the Phase 1.5 and Phase 2 analyses highlighted the effectiveness and weakness of each phase of scour analysis. Estimation, rather than field surveys, of the approach channel width limited the validity of the Phase 1.5 contraction-scour computation. Phase 1.5 analyses at bridges with modeled pressure flow lack sufficient hydraulic cross-section data for an accurate representation of flow hydraulics. The Phase 2 analysis would be more appropriate at locations with contracted openings and modeled pressure flow. Most channel-roughness values for both levels of analysis were calibrated to discharges less than the 100- and 500-year floods. Contraction and pier scour were evaluated for sensitivity to this variable. At selected bridges, there was little resultant change in the contraction scour depths and the change in the pier-scour depths was within the resolution of the pier-scour equation. Locations where an increase or decrease to the channel-flow angle of attack will result in significant change in pier scour should be visited at high flow or considered for multi-dimensional flow analysis. Supercritical flow, velocities in excess of 10 feet per second, hydraulic jumps, and pressure flow are conditions found at many sites in this study and are subject to multi-dimensional variation in both time and length scales. A more accurate simulation of these adverse flow conditions would be provided by a multi-dimensional flow model.

Because field measurements were made at the sites for the Phase 1.5 and 2 analyses, the results of these analyses were useful in evaluating the effectiveness and limitations of the Phase 1 analysis. Estimated channel-roughness values for the Phase 1.5 analyses and calibrated channel-roughness values for the Phase 2 analyses were on the average larger than those used for the Phase 1 study. The surveyed channel slopes both increased and decreased when compared with the slopes measured from topographic maps for the Phase 1 analysis. The differences in slope and roughness values were reflected in the general lack of agreement between modeled Froude numbers. Sensitivity analysis of the contraction-scour equation determined that changes in the approach-channel geometry were responsible for most major differences in scour depths between the levels of analysis. Contraction scour for locations that had modeled pressure flow was computed for Phase 1 using a procedure different from the subsequent analyses, and the results cannot be compared. The Phase 2 contraction-scour depths, which include more data than the Phase 1.5, generally were less than the Phase 1 depths. Pier-scour depths for Phase 1.5 were either larger or smaller than those for the Phase 1 analysis. Most of the inconsistencies were explained by an increase or decrease in the channel-flow angle of attack used in the computation. Most of the sites in the Phase 1.5 analysis were selected because the existing estimates of channel-flow angle of attack were either high or were unsupported. Pier-scour depths at locations selected for Phase 2 analysis generally were less than the values computed for the Phase 1 analysis.

The Phase 1 analysis was a cost-effective preliminary assessment of both pier and contraction scour that was used to select bridges for site investigation. The Phase 1.5 analysis was sufficient for most estimates of pier scour, but should not have been considered adequate at sites with significant channel contractions through the bridge. The Phase 2 analysis provided the best estimate of one-dimensional scour analysis using HEC-18 equations. Scour depths summarized in this report are being used by ADOT&PF to focus monitoring efforts and emplacement of countermeasures at river crossings in Alaska that are susceptible to scour.

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Table 4. Selected hydraulic variables used to construct hydraulic models for analyses of streambed scour at selected bridge sites in Alaska

[**ADOT & PF No.**: Alaska Department of Transportation and Public Facilities number. **Discharge**: 100- and 500-year discharges were estimated using methodology of Jones and Fahl (1994). **Cross-section source**: a, discharge measurement; b, survey; c, bridge sounding; d, as-built survey. ft^3/s, cubic feet per second; ft, feet; ft/ft, feet per foot. –, no data]

ADOT & PF No.	Discharge (ft^3/s)			Water-surface slope (ft/ft)	Manning's roughness coefficient		Minimum bed elevation (ft)		Cross-section source	Date surveyed
	100-year	500-year	Measured		Channel	Overbank	Measured	As-built		
Phase 1.5 analysis										
205	126,000	151,000	48,300	0.0017	0.030	0.05	448.0	448.0	a	08/02/99
240	3,800	4,570	3,320	.0002	.030	.050	375.0	375.0	a	10/12/86
277	1,350	2,360	426	.0027	.083	–	1,884.0	1,886.0	c	08/03/97
308	18,500	27,500	1,160	.0074	.040	–	54.1	65.2	a	08/31/01
396	1,600	2,060	36.3	.0060	.033	.070	164.2	1,64.0	c	07/16/99
401	4,360	7,010	5,790	.0060	.038	.050–.100	86.1	88.2	c	10/11/86
505	51,900	59,700	21,700	.0001	.032	–	1,513.5	–	a	08/23/01
509	9,520	11,700	–	.0080	.040–.045	–	1,445.2	1,445.0	b	05/30/91
518	7,520	9,250	–	.0035	.035	–	60.4	–	c	09/24/99
520	5,100	6,340	133	.0063	.041	–	1,326.8	1,331.0	b	09/15/98
530	3,130	3,920	1,900	.0050	.035	.090	609.1	612.5	c	07/31/99
543	1,920	2,390	184	.0335	.055	–	478.8	480.0	c	07/15/99
544	7,600	9,310	–	.0100	.055	–	79.8	83.9	b	07/19/99
547	1,370	1,700	35.9	.0200	.055	–	85.0	89.8	c	07/19/99
548	9,550	11,800	5,210	.0178	.059	–	1,766.7	1,770.0	a	06/16/73
556	24,400	30,600	3,640	.0073	.040	–	20.1	23.0	a	08/05/98
572	10,700	11,800	9,320	.0069	.041	.100	1,006.0	1,001.4	c	09/02/98
573	79,400	109,000	5,230	.0030	.026	.034–.092	1,105.7	1,106.9	c	07/22/97
603	2,100	2,600	2,400	.0010	.030	.100	440.0	440.0	a	12/02/85
658	9,430	11,200	240	.0019	.023	.065	93.1	93.1	c	08/01/99
678	2,260	2,870	–	.0025	.040	.045	373.5	373.6	c	08/20/67
686	4,420	5,400	–	.0100	.044	–	2,880.9	2,881.0	c	09/12/96
687	37,600	48,200	18,800	.0010	.032	–	2,417.7	2,422.0	a	10/17/67
690	3,090	5,730	358	.0160	.046	.100	2,305.9	2,304.8	c	08/03/99
742	32,600	42,900	6,200	.0025	.080	.100	113.7	112.9	a	08/30/01
832	2,130	3,840	–	.0080	.035	.100	1,053.7	1,053.7	d	05/14/70
833	3,890	4,860	–	.0023	.035	.100	844.2	846.0	c	08/09/99
999	7,020	10,200	440	.0090	.045	–	99.8	104.0	a	07/18/99
1220	9,870	12,400	–	.0041	.035	.100	39.8	39.8	d	04/01/70
1261	25,100	28,500	5,950	.0010	.026	.030–.070	1,187.5	1,188.0	a	06/11/98
1282	20,300	23,400	–	.0030	.026	–	1,210.7	1,216.3	c	07/26/99
1283	10,300	12,000	611	.0013	.030	.075	1,376.3	1,377.0	c	07/26/99
1284	7,880	9,180	467	.0020	.040	–	1,370.5	1,379.8	c	07/26/99
1329	7,910	11,800	–	.0070	.035	.100	445.1	446.0	c	08/06/99
1389	980	1,230	31.0	.0450	.070	–	207.5	208.0	c	07/17/99

Table 4. Selected hydraulic variables used to construct hydraulic models for analyses of streambed scour at selected bridge sites in Alaska—*Continued*

[**ADOT & PF No.**: Alaska Department of Transportation and Public Facilities number. **Discharge**: 100- and 500-year discharges were estimated using methodology of Jones and Fahl (1994). **Cross-section source**: a, discharge measurement; b, survey; c, bridge sounding; d, as-built survey. ft^3/s, cubic feet per second; ft, feet; ft/ft, feet per foot. –, no data]

ADOT & PF No.	Discharge (ft^3/s)			Water-surface slope (ft/ft)	Manning's roughness coefficient		Minimum bed elevation (ft)		Cross-section source	Date surveyed
	100-year	500-year	Measured		Channel	Overbank	Measured	As-built		
Phase 2 analysis										
233	22,100	27,400	733	0.0013	0.032	0.058–0.145	726.8	734.0	b	06/06/98
242	4,220	5,220	–	.0066	.045	.06	1,092.3	1,098.0	b	09/22/94
254	186,000	206,000	59,900	.0010	.02–.028	.06–.07	242.6	248.0	b	08/26/98
255	83,200	103,000	24,800	.00145	.028–.04	.05	480.7	482.0	b	08/25/98
355	36,000	42,000	8,520	.0005	.03	.08	631.0	634.0	a	08/09/99
527	50,600	64,900	36,400	.0025	.036	.09	629.0	634.0	a	09/06/95
535	6,920	8,710	1,600	.0080	.04	.12–.15	258.9	263.7	b	09/01/98
539	79,400	104,000	23,000	.0007	.027–.037	.08	19.5	36.0	b	07/13/99
557	22,100	28,500	3,270	.0070	.049	.049	365.4	366.0	a	08/05/98
558	19,400	25,100	3,940	.0050	.042	.13	429.7	440.0	a	08/04/98
574	18,400	22,200	1,580	.0035	.03–.04	.06–.1	1,362.8	1,365.0	a	08/31/98
608	1,610	2,070	–	.0090	.035	.075	451.8	453.5	b	09/01/95
609	1,220	1,750	–	.0200	.04	.1	473.5	472.0	b	08/31/95
694	25,400	30,300	4,850	.0030	.05	.1	1,784.2	1,786.0	a	08/12/98
1025	4,590	6,520	339	.0055	.03–.035	.08	18.9	20.0	b	07/18/99
1147	43,200	53,600	1,030	.0050	.04	.1	1,504.8	1,505.0	b	08/13/98
1255	3,800	4,710	181	.0050	.03–.037	.05	836.2	834.9	b	06/27/99
1341	6,920	8,710	1,600	.0080	.04	.12–.16	198.8	202.5	b	09/01/98
1513	1,830	2,330	–	.0057	.05	.01	80.0	–	b	06/24/99

Table 5. Computed contraction-scour depths, and hydraulic variables used in computation for the 100-year and 500-year recurrence-interval flood discharges at selected bridge sites in Alaska

[**ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. **Discharge at bridge:** 100- and 500-year discharges were calculated using methodology of Jones and Fahl (1994). ft^3/s, cubic feet per second; ft, feet]

ADOT & PF No.	Discharge at bridge (ft^3/s)		Width of approach channel (ft)		Discharge at approach (ft^3/s)		Flow depth in approach (ft)		Width of channel at bridge (ft)		Depth of flow at bridge (ft)		Depth of contraction scour (ft)	
	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Phase 1.5 analysis														
205	126,000	151,000	843	844	123,415	146,419	12.4	13.9	791.1	793.8	12.3	13.6	0.7	0.9
240	3,800	4,570	181	186	3,800	4,570	7.8	8.6	179.4	186.7	7.7	8.4	.0	.0
277	1,260	2,210	30.0	30.0	859	1,224	9.0	12.1	20.1	20.1	9.4	10.9	6.9	13.3
308	18,500	27,500	491	500	18,500	27,500	5.3	6.9	402.5	408.9	4.9	6.3	.7	.9
396	1,600	2,060	90	99	1,600	2,059	3.4	3.8	59.5	61.1	5.3	5.9	.9	1.3
401	4,360	7,010	79	79	3,575	4,276	7.1	10.1	70.7	71.0	9.1	10.8	1.9	6.4
505	51,900	59,700	1,482	1,485	51,900	59,700	18.2	19.8	914.4	915.8	18.1	19.7	6.0	6.5
509	9,520	11,700	257	321	9,520	11,700	4.3	4.3	248.6	304.5	4.2	4.2	.0	.0
518	7,520	9,250	394	471	7,520	9,250	3.9	3.4	379.5	453.2	3.3	3.4	.0	.0
520	5,100	6,340	223	223	5,100	6,340	3.6	4.1	216.4	216.6	3.4	3.9	.0	.0
530	3,130	3,920	32	32	2,587	2,931	9.8	11.3	46.3	49.5	6.6	7.2	.0	.0
543	1,920	2,390	144	147	1,920	2,390	1.8	2.1	136.4	139.0	1.8	2.1	.0	.0
544	7,600	9,310	176	178	7,600	9,310	4.8	5.4	168.8	171.0	4.4	4.9	.0	.0
547	1,370	1,700	84	85	1,370	1,700	2.5	2.9	80.7	83.4	2.5	2.8	.0	.0
548	9,550	11,800	183	202	9,550	11,800	6.4	7.1	86.4	90.2	7.2	8.1	3.6	4.3
556	24,400	30,600	286	292	24,400	30,600	10.4	12.0	204.6	211.5	9.0	10.1	2.3	2.5
572	10,700	11,800	160	162	10,700	11,800	6.9	7.3	155.8	157.9	6.8	7.2	.0	.0
573	79,400	109,000	921	921	78,154	105,630	13.6	17.3	422.7	428.8	12.4	15.0	8.3	10.6
603	2,100	2,600	146	147	2,095	2,578	3.5	4.0	113.6	116.0	3.6	4.0	.6	.6
658	9,430	11,200	93	93	9,081	10,423	9.9	11.3	75.0	74.9	9.0	9.8	1.7	2.3
678	2,260	2,870	75	76	1,947	2,357	7.9	8.9	62.0	62.0	7.6	8.6	2.2	3.0
686	4,420	5,400	219	221	4,420	5,400	3.5	4.0	151.5	152.1	3.6	4.0	.8	1.0
687	37,600	48,200	1,299	1,301	37,600	48,200	6.3	7.4	928.6	930.1	7.1	8.1	1.4	1.6
690	3,090	4,360	106	109	3,090	5,620	9.4	13.6	26.4	26.4	8.8	14.4	12.0	11.6
742	32,600	42,900	415	415	31,656	41,313	13.8	16.3	463.1	466.5	12.7	15.0	.0	.0
832	2,130	3,840	34	34	1,651	2,746	6.3	9.3	42.2	48.6	4.9	6.6	.6	.8
833	3,890	4,860	76	76	3,571	4,383	8.6	9.8	61.5	65.9	5.9	6.5	1.9	1.9
999	7,020	10,200	236	241	7,015	10,164	5.4	6.9	157.4	162.8	5.2	6.4	1.5	1.8
1220	9,870	12,400	174	174	9,363	11,492	7.5	9.0	118.1	131.3	6.0	6.9	2.4	2.3
1261	25,100	28,500	368	370	25,100	28,500	8.0	8.6	295.1	295.7	7.8	8.3	1.1	1.2
1282	20,300	23,400	223	226	20,300	23,400	8.4	9.1	217.5	219.1	8.3	9.0	.0	.0
1283	10,300	12,000	165	165	9,799	10,580	6.6	10.1	134.6	148.3	6.9	8.1	1.2	1.9
1284	7,880	9,180	125	129	7,880	9,180	9.3	10.0	86.6	89.8	8.0	8.6	2.2	2.4
1329	7,910	11,800	83	98	7,910	11,800	7.7	8.9	76.1	87.1	6.9	8.3	.4	.6
1389	980	1,230	99	104	980	1,230	1.6	1.8	98.2	103.1	1.6	1.8	.0	.0

Table 5. Computed contraction-scour depths, and hydraulic variables used in computation for the 100-year and 500-year recurrence-interval flood discharges at selected bridge sites in Alaska—*Continued*

[**ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. **Discharge at bridge:** 100- and 500-year discharges were calculated using methodology of Jones and Fahl (1994). ft^3/s, cubic feet per second; ft, feet]

ADOT & PF No.	Discharge at bridge (ft^3/s)		Width of approach channel (ft)		Discharge at approach (ft^3/s)		Flow depth in approach (ft)		Width of channel at bridge (ft)		Depth of flow at bridge (ft)		Depth of contraction scour (ft)	
	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year
Phase 2 analysis														
233	22,100	27,100	255	255	21,261	26,405	13.9	13.7	141.0	138.6	14.6	12.8	6.5	6.4
242	4,220	5,220	108	112	4,220	5,220	5.1	5.8	84.9	87.7	5.8	6.4	.8	.9
254	186,000	206,000	1,555	1,560	186,000	206,000	14.4	15.3	936.7	938.8	14.4	15.0	5.0	5.3
255	83,200	103,000	379	379	82,257	101,340	19.2	21.9	455.9	469.6	17.4	19.6	.0	.0
355	36,000	42,000	368	368	34,914	40,012	14.2	15.4	319.5	323.2	15.0	15.9	1.6	1.9
527	50,600	64,900	673	673	50,502	64,695	9.7	11.5	446.7	451.1	13.1	14.6	2.7	3.1
535	6,920	8,710	149	160	6,920	8,706	5.3	6.5	77.0	80.6	6.3	7.1	2.6	3.2
539	79,400	104,000	1,567	1,636	79,400	104,000	9.9	11.5	410.2	412.6	17.3	17.8	12.0	14.4
557	22,100	28,500	333	348	22,100	28,500	7.5	8.6	333.8	348.8	7.5	8.5	.0	.0
558	19,400	25,100	319	321	19,400	25,090	6.8	8.0	319.3	325.4	6.2	7.2	.0	.0
574	18,400	22,200	430	435	18,400	22,200	8.2	9.0	264.6	266.2	8.2	8.8	2.7	3.0
608	1,610	2,070	77	80	1,610	2,070	3.1	3.4	96.1	97.5	2.1	2.4	.0	.0
609	1,220	1,750	96	102	1,220	1,750	1.6	1.9	66.8	69.1	2.2	2.7	.4	.5
694	25,400	30,300	290	310	25,400	30,300	11.2	11.9	252.9	267.8	11.6	12.3	.9	1.1
1025	4,590	6,520	52	52	4,443	6,207	7.0	8.7	130.0	141.5	5.9	7.3	.0	.0
1147	43,200	53,600	334	345	43,200	53,600	12.5	14.5	277.9	284.0	12.3	14.6	1.4	1.8
1255	3,800	4,710	87	91	3,415	4,189	6.9	7.6	90.5	97.5	4.7	5.4	.5	.5
1341	6,920	8,710	341	349	6,920	8,710	5.6	6.4	227.3	231.2	6.4	7.2	1.5	1.7
1513	1,830	2,330	51	53	1,792	2,160	5.5	6.8	30.0	30.0	7.3	8.4	2.2	3.3

Table 6. Computed pier-scour depths and hydraulic variables used in computation for the 100-year and 500-year recurrence-interval flood discharges at selected bridge sites in Alaska

[**ADOT & PF No.**: Alaska Department of Transportation and Public Facilities. $\mathbf{K_2}$: Correction factor for channel-flow angle of attack. ft, feet. –, no data]

ADOT & PF No.	Froude no. at bridge		Depth of flow at bridge (ft)		Pier nose shape	Angle of attack (degrees)	K_2	Pier width (ft)	Pier length (ft)	Depth of flow at pier (ft)		Depth of pier scour (ft)	
	100-year	500-year	100-year	500-year						100-year	500-year	100-year	500-year
Phase 1.5 analysis													
205	0.59	0.61	12.3	13.6	sharp	0	1.00	6.0	28.5	22.7	24.1	15.1	15.6
240	.17	.18	7.7	8.4	round	45	4.23	1.3	30.0	11.4	12.4	12.1	12.7
277	.39	.52	9.4	10.9	no pier	–	–	–	–	–	–	–	–
308	.71	.72	4.9	6.3	sharp	45	4.23	1.0	28.0	7.5	8.9	16.2	17.4
396	.36	.40	5.3	5.9	square	45	4.23	1.0	34.0	7.6	8.3	12.2	13.1
401	.40	.49	9.1	10.8	no pier	–	–	–	–	–	–	–	–
505	.13	.13	18.1	19.7	sharp	0	1.00	6.0	50.0	32.6	34.2	8.9	9.1
509	.93	.67	4.2	4.2	sharp	30	1.91	9.0	33.0	3.0	5.6	24.9	26.9
518	.49	.49	3.3	3.4	sharp	20	2.04	6.0	36.0	6.9	7.5	20.8	21.5
520	.61	.63	3.4	3.9	sharp	0	1.00	6.0	35.0	5.2	5.7	9.1	9.6
530	.61	.63	6.6	7.2	square	15	2.49	1.0	43.0	10.7	11.8	10.2	10.7
543	.70	.72	1.8	2.1	sharp	9	1.98	1.7	29.0	4.7	5.1	9.1	9.4
544	.68	.70	4.4	4.9	round	0	1.00	3.0	35.0	9.5	10.2	7.2	7.2
547	.67	.70	2.5	2.8	sharp	25	3.20	1.2	29.0	6.5	6.9	12.8	13.4
548	.75	.74	7.2	8.1	round	35	3.65	3.0	34.0	10.1	11.5	32.6	33.9
556	.76	.76	9.0	10.1	sharp	0	1.00	.8	35.2	15.3	17.8	4.0	4.2
572	.53	.55	6.8	7.2	sharp	0	1.00	8.0	31.0	12.5	12.9	14.1	14.5
573	.73	.73	12.4	15.0	sharp	0	1.00	4.0	20.0	20.1	23.2	12.2	12.8
603	.44	.46	3.6	4.0	round	20	1.72	3.0	12.0	5.9	6.4	10.1	10.6
658	.32	.41	9.0	9.7	no pier	–	–	–	–	–	–	–	–
678	.28	.31	7.6	8.6	square	0	1.00	1.0	1.0	13.8	15.6	3.5	3.8
686	.70	.72	3.6	4.0	square	0	1.00	.8	.8	5.5	6.0	3.3	3.4
687	.37	.39	7.1	8.1	sharp	24	3.13	1.5	20.0	9.9	10.9	13.0	13.8
690	.46	.49	8.8	14.4	no pier	–	–	–	–	–	–	–	–
742	.27	.28	12.7	15.0	sharp	0	1.00	1.8	25.0	16.6	19.0	4.3	4.6
832	.82	.38	4.9	6.3	no pier	–	–	–	–	–	–	–	–
833	.73	.73	5.9	6.5	no pier	–	–	–	–	–	–	–	–
999	.63	.65	5.2	6.4	sharp	30	3.50	.8	57.0	8.7	10.1	11.9	12.7
1220	.78	.78	6.0	6.9	round	0	1.00	1.3	35.0	8.0	9.1	3.1	3.1
1261	.69	.71	7.8	8.3	sharp	15	2.49	.8	25.0	10.6	11.1	9.4	9.7
1282	.69	.70	8.3	9.0	sharp/round	20	2.86	2.0	25.0	11.7	12.5	19.9	20.5
1283	.72	.84	6.9	6.3	sharp	0	1.00	.8	24.0	9.4	9.4	3.3	3.6
1284	.66	.66	8.0	8.6	sharp	0	1.00	.8	24.0	10.9	11.8	3.4	3.5
1329	.77	.76	6.9	8.3	sharp	5	1.59	2.0	40.0	11.1	13.8	10.3	11.0
1389	.85	.87	1.6	1.8	no pier	–	–	–	–	–	–	–	–

Table 6. Computed pier-scour depths and hydraulic variables used in computation for the 100-year and 500-year recurrence-interval flood discharges at selected bridge sites in Alaska—*Continued*

[**ADOT & PF No.**: Alaska Department of Transportation and Public Facilities. **K_2**: Correction factor for channel-flow angle of attack. ft, feet. –, no data]

ADOT & PF No.	Froude no. at bridge		Depth of flow at bridge (ft)		Pier nose shape	Angle of attack (degrees)	K_2	Pier width (ft)	Pier length (ft)	Depth of flow at pier (ft)		Depth of pier scour (ft)	
	100-year	500-year	100-year	500-year						100-year	500-year	100-year	500-year
Phase 2 analysis													
233	0.43	0.61	14.6	12.8	sharp	10	2.07	0.8	28.0	20.5	19.3	8.1	9.2
242	.59	.60	5.8	6.4	sharp	0	1.00	1.5	34.0	8.7	9.5	4.4	4.5
254	.61	.64	14.4	15.0	sharp	0	1.00	7.0	28.0	25.3	25.9	17.6	18.1
255	.44	.44	17.4	19.6	sharp	7	1.61	4.5	40.0	24.0	26.8	20.1	20.9
355	.34	.36	15.0	15.9	sharp	0	1.00	4.0	40.0	18.3	19.4	8.5	8.9
527	.45	.48	13.1	14.6	round	10	1.77	3.7	30.0	19.0	20.5	18.0	19.0
535	.80	.77	6.3	7.1	round	0	1.00	5.0	32.5	10.4	11.8	15.0	15.0
539	.46	.57	17.3	17.8	sharp	0	1.00	4.3	26.0	26.0	26.7	11.4	12.7
557	.57	.56	7.5	8.5	sharp	0	1.00	2.5	68.0	14.1	15.5	7.1	7.3
558	.58	.59	6.2	7.2	circular	0	1.00	5.0	5.0	11.7	12.9	11.7	12.2
574	.47	.49	8.2	8.8	sharp	0	1.00	3.0	38.0	11.1	11.9	6.8	7.1
608	.63	.63	2.1	2.4	square	0	1.00	3.4	24.0	3.1	3.5	6.5	6.8
609	.71	.70	2.2	2.7	group cyl.	0	1.00	1.5	24.0	3.1	3.9	3.7	3.9
694	.41	.42	11.6	12.3	round	0	1.00	5.0	19.5	15.1	16.5	11.0	11.5
1025	.42	.40	5.9	7.3	circular	0	1.00	3.0	3.0	8.6	10.5	6.6	6.9
1147	.56	.54	12.3	14.6	round	0	1.00	3.0	22.0	18.9	21.4	7.2	7.2
1255	.65	.62	4.7	5.4	sharp	20	2.86	.8	24.0	7.5	8.5	9.4	9.6
1341	.30	.31	6.4	7.2	circular cyl.	0	1.00	6.0	6	8.35	9.26	8.8	9.3
1513	.16	.17	7.3	8.4	No pier	–	–	–	–	–	–	–	–

Table 7. Computed pressure-scour depths and hydraulic variables used in computation for the 100-year and 500-year recurrence-interval flood discharges at selected bridge sites in Alaska

[**ADOT & PF No.:** Alaska Department of Transportation and Public Facilities. –, no pressure flow for 100-year discharge. ft, feet; ft^3/s, cubic feet per second; ft^2, square feet; ft/s, feet per second]

ADOT & PF No.	Depth at approach (ft)		Discharge in approach (ft^3/s)		Area of approach (ft^2)		Approach velocity (ft/s)		Depth of overflow (ft)		Average depth at bridge (ft)	Median particle diameter (ft)	Incipient motion velocity (ft/s)		Pressure scour (ft)	
	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year	100-year	500-year			100-year	500-year	100-year	500-year
401	–	8.9	–	4,781	–	701	–	6.8	–	0	9.0	0.03281	–	5.3	–	8.1
658	11.1	11.2	8,824	10,449	1,024	1,035	8.6	10.1	3.5	4.0	6.0	.03281	5.5	5.5	0.0	.2
678	7.8	8.9	1,953	2,355	589	677	3.3	3.5	0	.6	6.5	.03281	5.2	5.3	2.1	.2

Table 8. Computed pier-stem scour depths, and hydraulic variables used in computation for the 100-year and 500-year recurrence-interval flood discharges at selected bridge sites in Alaska

[**ADOT & PF No.**: Alaska Department of Transportation and Public Facilities. **K_{hpier}**: Coefficient to account for height of pier base above streambed and shielding effect of footing. **K_1:** Correction factor for pier-nose shape. **K_2:** Correction factor for channel-flow angle of attack. **D_{84}:** particle size for which 84 percent are finer by weight. ft, feet; ft/s, feet per second]

ADOT & PF No.	Average depth of approach (ft)		K_{hpier}		Distance between front edge of pile cap and pier (ft)	Width of pier (ft)	Height of pier stem above bed (ft)		Maximum approach velocity (ft/s)		K_1	K_2	Depth of pier stem scour (ft)	
	100-year	500-year	100-year	500-year			100-year	500-year					100-year	500-year
205	22.7	24.1	0.0	0.0	1.3	6.0	4.8	5.3	16.0	17.0	0.9	1.0	0.3	0.4
505	32.6	34.2	.1	.1	6.0	6.0	13.0	13.0	4.1	4.3	.9	1.0	.4	.4
544	9.5	10.2	.0	.0	1.7	3.0	2.3	2.4	11.9	12.7	1.0	1.0	.3	.3
572	6.5	7.3	.1	.1	5.3	3.8	11.0	11.1	10.7	11.1	.9	1.0	.7	.7
539	26.0	26.7	.1	.1	5.0	4.3	1.1	6.4	13.3	16.7	.9	1.0	.9	1.7
694	15.1	16.5	.3	.3	8.5	5.0	3.3	3.5	9.0	9.6	1.0	1.0	2.8	2.9

ADOT & PF No.	Velocity at footing (ft/s)		Adjusted flow velocity (ft/s)		Width of pile cap (ft)	Distance from bed to top of footing after scour (ft)		D_{84} of bed material (ft)	Adjusted depth of flow (ft)		Depth of footing scour (ft)		Depth of complex pier scour (ft)	
	100-year	500-year	100-year	500-year		100-year	500-year		100-year	500-year	100-year	500-year	100-year	500-year
205	12.6	13.5	15.9	16.9	23.0	5.0	5.5	0.2	22.9	24.3	26.6	27.7	26.9	28.1
505	3.6	3.7	4.0	4.3	16.0	13.2	13.2	.2	32.8	34.4	13.9	14.2	14.3	14.6
544	9.2	9.9	11.7	12.6	10.0	2.4	2.5	.2	9.7	10.4	13.7	14.1	13.9	14.4
572	11.0	11.3	10.2	10.6	8.0	11.3	11.4	.2	6.9	7.6	14.1	14.3	14.8	15.0
539	8.5	13.5	13.1	16.2	24.0	1.6	7.2	.2	26.4	27.5	19.4	29.5	20.3	31.2
694	6.8	7.2	8.3	8.8	15.0	4.7	5.0	.2	16.5	18.0	17.0	17.6	19.8	20.5

Manuscript approved for publication, April 28, 2004

Prepared by the Publishing Group, U.S. Geological Survey,
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Summary and Comparison of Multiphase Streambed Scour Analysis at Selected Bridge Sites in Alaska

SIR 2004–5066

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