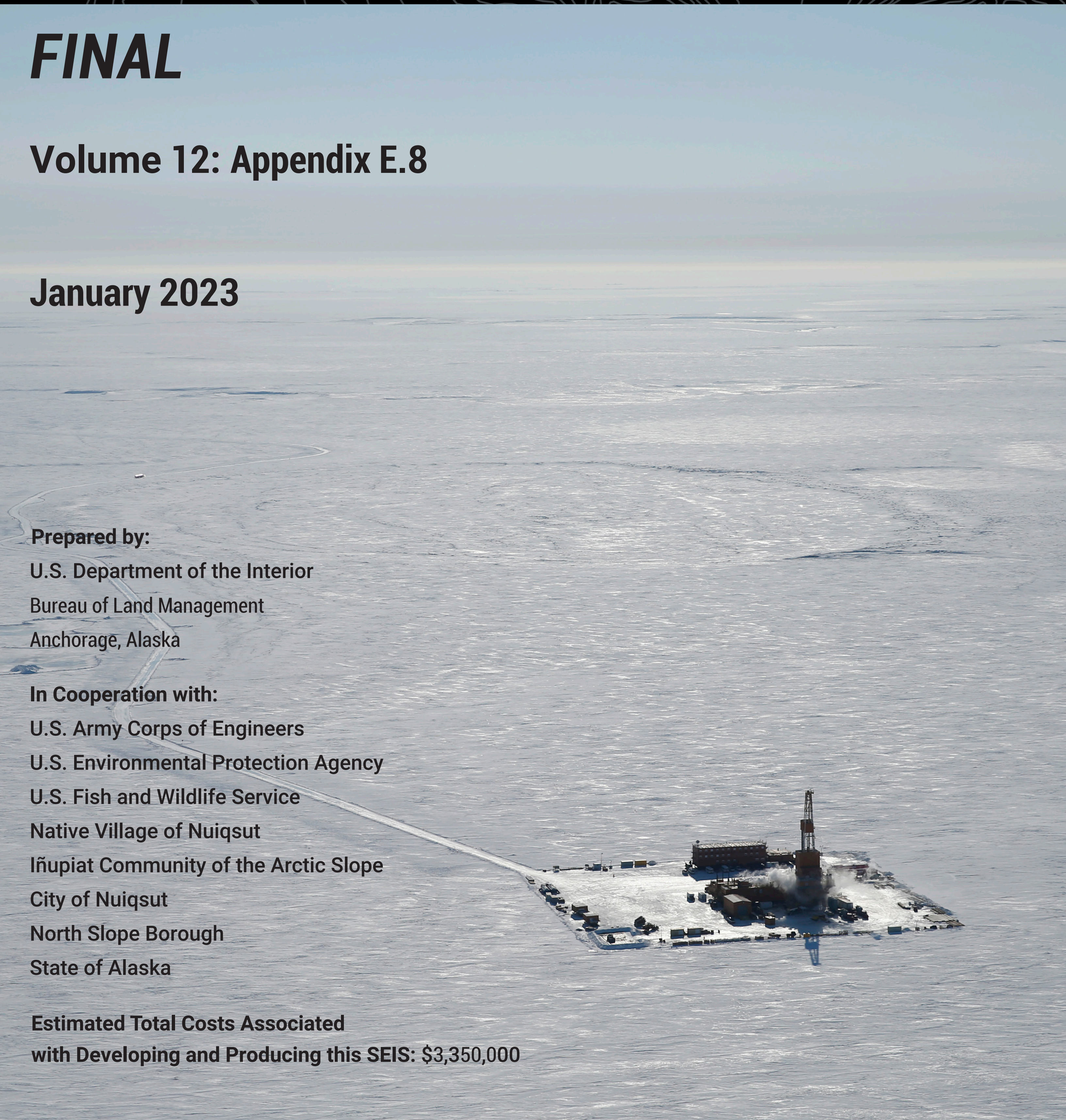
U.S. Department of the Interior
Bureau of Land Management

Willow Master Development Plan

Supplemental Environmental Impact Statement

FINAL



Volume 12: Appendix E.8

January 2023

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U.S. Department of the Interior
Bureau of Land Management
Anchorage, Alaska

In Cooperation with:

U.S. Army Corps of Engineers
U.S. Environmental Protection Agency
U.S. Fish and Wildlife Service
Native Village of Nuiqsut
Iñupiat Community of the Arctic Slope
City of Nuiqsut
North Slope Borough
State of Alaska

Estimated Total Costs Associated with Developing and Producing this SEIS: $3,350,000

Mission

To sustain the health, diversity, and productivity of the public lands for the future use and enjoyment of present and future generations.

Cover Photo Illustration: North Slope Alaska oil rig during winter drilling.
Photo by: Judy Patrick, courtesy of ConocoPhillips.

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Willow Master Development Plan

Appendix E.8
Water Resources Technical Appendix

January 2023

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Willow Master Development Plan

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List of Acronyms

cfs	cubic feet per second
CPAI	ConocoPhillips Alaska, Inc.
HDD	horizontal directional drilling
MBI	Michael Baker International
mm	millimeters
NAVD88	North American Vertical Datum of 1988
NWS	National Weather Service
Project	Willow Master Development Plan Project
RM	river mile
USGS	U.S. Geological Survey
VSM	vertical support member
WSE	water surface elevation

Glossary Terms

Bottom-fast ice – Ice that is attached to the waterbody or sea floor and is relatively uniform in composition and immobile during winter (also known as bedfast, ground-fast, fast, shorefast, or landfast ice).

Discharge – The rate at which a given volume of water passes a given location within a specific period of time (e.g., cubic feet per second or gallons per minute).

Rolligon – A type of wheeled, low-impact off-road vehicle frequently used on the North Slope for tundra or snow travel; it can be configured to suit a variety of industrial and construction needs.

Stage – The vertical height of the water above an established but usually arbitrary point. Sometimes zero stage corresponds to the riverbed but more often to just an arbitrary point.

Water surface elevation – The elevation of the water surface of a river, lake, or stream above an established reference or vertical datum.

1.0 WATER RESOURCES

1.1 General Flow Characteristics of Rivers and Streams in the Analysis Area

Freeze-up often begins with ice forming along the shoreline and ice pans floating down the river. As freeze-up continues, the ice cover spreads across the stream and in shallow locations the entire water column freezes. Stream flow during the winter on the North Slope is generally so low that it is not measurable and is often nonexistent. In late May or early June there is a rapid rise in **discharge** resulting from snowmelt runoff, a period generally referred to as spring breakup. More than half the annual discharge for a stream can occur during spring breakup, a period of several days to a few weeks. Extremely large areas can be inundated in a matter of days as a result of rapid snowmelt combined with ice- and snow-blocked channels. Most streams continue to flow through the summer but at substantially lower discharges. Rainstorms can increase streamflow temporarily, but they are seldom sufficient to produce a discharge comparable to that which occurs during the average spring breakup. Streamflow rapidly declines in most streams shortly after the onset of freeze-up in September and ceases in most streams by December.

1.1.1 Influence of Climate Change on Flow

Although climate change is occurring, it is unknown how it might impact flood-peak magnitude and frequency in the Arctic. The National Weather Service (NWS) evaluated the potential for statistically significant trends in the 1-day and 1-hour annual maximum daily precipitation data for Alaska (for stations that had at least 40 years of data), which are often used to predict flood-peak discharge (Perica, Kane et al. 2012). There was no trend in 1-hour annual maximum precipitation for the 12 stations with 40 years of record. Of the 154 stations with 40 years of 1-day annual maximum precipitation data, 85% had no statistically significant trends, 8% had a positive trend, and 7% had a negative trend. Spatial maps did not reveal any spatial cohesiveness in positive and negative trends.

U.S. Geological Survey (USGS) evaluated the flood-peak data set used to develop regression equations to predict flood-peak discharge throughout Alaska (Curran, Barth et al. 2016). Statistically significant trends were detected at 43 of the 387 stream gages evaluated. Of the 43 stream gages with significant trends, 22 had increasing trends and 21 had decreasing trends.

Although precipitation levels are projected to increase, the longer, warmer summers may increase evapotranspiration. An increase in evapotranspiration may result in a net loss in surface water by the end of the summer season, which could affect the size, depth, and areal extent of thaw lakes. Increases in winter precipitation may have some effect on lake recharge and peak snowmelt runoff in rivers and streams.

1.2 Hydrology of Rivers and Streams in the Willow Area

1.2.1 Colville River*

The Colville River is the largest north-flowing river in the U.S. and drains an area of about 23,600 square miles. It originates in the DeLong Mountains of the Brooks Range and generally follows a west-east flow corridor until reaching Umiat, where it turns north and flows into Harrison Bay in the Beaufort Sea.

Discharge and **stage** data are available for several locations on the Colville River. The closest gaging stations to Ocean Point (approximately river mile [RM] 46.5) are at Umiat (RM 117) and Monument 1 (RM 26.5), Figure 3.8.2. Although neither of these existing gages measures winter flow at Ocean Point, Umiat is more closely representative of Ocean Point than Monument 1 because Umiat is upstream of the influence of saltwater intrusion and tidal backwatering from the Colville River Delta and Monument 1 is not. Seventeen years of stage and discharge have been measured at the USGS Umiat gaging station 15875000 (Tables E.8.1 and E.8.2). The average monthly mean discharge at Umiat in winter (December through April) ranged from 83 to 4.1 cubic feet per second (cfs) from 2002 to 2021 (USGS 2022), as shown in Table E.8.1. (The range of mean monthly discharge for December through April was 132.2 to 0.0 cfs; Table E.8.1.) During that time, the minimum recorded average daily winter discharge varied from 0.0 cfs (2003 through 2009) to 20.0 cfs (2019) (USGS 2022). The annual spring peak discharge occurred between May 22 and June 10, with a median date of June 1. The time from the last day of minimum flow to the annual spring discharge varied between 12 and 47 days, with a median time of 24 days. The annual spring peak discharge varied from 73,000 to 268,000 cfs, with a median of 177,800 cfs. Note that the Colville River is more than 2,000 feet wide at Umiat and that by late winter the flow is contained to a very small channel within that width. In other words, the ice across 99% of the channel is frozen to the bottom, but somewhere within that width there is a very small channel with flow.

Table E.8.1. Colville River Mean Monthly Discharge (cubic feet per second) at Umiat*

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2002	ND	ND	ND	ND	ND	ND	ND	ND	21,030	7,221	844	100.1
2003	3.6	0	0	0	690	65,690	24,030	31,800	12,760	10,490	560	72.6
2004	6.9	2.2	0.2	0	40,890	24,940	15,310	24,870	12,060	557	142	56.6
2005	20.8	4.2	< 0.1	0	12,830	72,480	13,920	4,143	6,014	1,169	200	104.5
2006	18.4	0.1	0	0	22,010	37,120	21,940	33,560	6,229	2,667	325	80.0
2007	27.9	11.7	0.9	0	4,179	50,530	12,140	17,820	7,511	874	177	72.6
2008	21.1	0.7	0	0	17,260	46,530	12,900	10,770	1,867	560	207	72.9
2009	15.0	0	0	3.0	36,940	45,050	13,890	13,440	13,750	1,775	418	95.2
2010	36.5	13.9	1.7	0.5	17,280	48,760	10,370	15,720	6,213	1,248	454	132.2
2011	35.5	9.7	1.1	0.4	37,790	31,190	13,170	11,330	11,940	1,958	375	93.5
2012	29.2	11.0	1.9	0.5	16,680	41,910	16,970	14,860	27,440	3,678	145	45.9
2013	16.4	3.9	2.0	1.0	6,434	83,970	10,530	10,290	11,750	1,475	509	130.7
2014	25.9	9.3	6.0	6.0	33,290	72,180	29,820	10,130	16,140	1,215	217	89.9
2015	45.2	29.0	16.8	12.0	62,410	17,010	8,243	22,250	11,550	1,504	276	65.5
2016	24.4	10.1	5.7	2.8	47,460	32,660	14,540	27,290	15,310	4,868	405	64.4
2017	16.0	3.8	1.2	1.0	12,070	26,220	13,110	36,370	25,900	6,403	448	86.5
2018	24.9	11.9	7.1	6.0	12,220	47,610	26,970	30,330	23,280	3,122	343	67.1
2019	40.9	30.2	22.6	20.0	36,180	18,370	12,380	38,990	15,500	ND	ND	ND
2020	27.2	9.0	4.7	4.0	106,013	23,807	12,248	19,911	23,106	13,442.0	370.3	69.0
2021	21.7	7.8	2.6	2.1	9,792	34,387	24,607	21,238	27,565	ND	ND	ND
Average monthly mean discharge Sep 2002 to Sep 2021	**24.1**	**8.9**	**4.1**	**3.1**	**28,022.0**	**43,179.7**	**16,162.5**	**20,795.4**	**14,845.8**	**3,568.1**	**356.4**	**83.3**
Average monthly mean discharge Sep 2010 to Sep 2021	**27.9**	**12.3**	**6.5**	**5.1**	**34,576**	**39,029**	**16,599**	**22,090**	**17,975**	**3,891**	**354.2**	**84.5**

Source: USGS 2022
Note: ND (no data); < (less than); Sep (September). No incomplete data have been used for statistical calculations.

Table E.8.2. Summary of Annual Minimum and Spring Peak Discharge for the Colville River at Umiat*

Year	First Date of Minimum Flow (month/day)	Last Date of Minimum Flow (month/day)	Minimum Flow Discharge (cfs)	Annual Spring Peak Stage Date (month/day)	Annual Spring Peak Discharge (cfs)	Minimum Flow to Spring Peak Discharge (days)
2003	1/19	5/08	0	6/10	213,000	33
2004	3/06	5/09	0	5/24	222,000	15
2005	3/02	5/04	0	6/08	161,000	35
2006	2/04	5/09	0	5/30	173,000	21
2007	3/11	5/17	0	6/05	183,000	19
2008	2/07	5/16	0	5/28	108,000	12
2009	1/29	4/21	0	6/07	152,000	47
2010	3/20	5/19	0.5	6/01	186,000	13
2011	3/21	4/23	0.3	5/29	230,000	36
2012	3/22	5/15	0.5	6/02	177,000	18
2013	4/04	5/22	1.0	6/04	243,000	13
2014	3/01	5/05	6.0	5/31	195,000	26
2015	3/31	5/08	12.0	5/21	268,000	13
2016	4/12	4/30	2.5	5/25	193,000	25
2017	3/06	5/09	1.0	6/02	73,000[a]	24
2018	3/30	5/04	6.0	6/01	112,000	28
2019	3/24	5/02	20.0	5/25	135,000	23
2020	4/03	5/08	4.0	5/27	149,000	19
2021	4/04	4/28	2.0	6/07	99,800	40

Source: USGS 2022
Note: cfs (cubic feet per second)
[a] The peak discharge of 82,000 cfs occurred on 8/19.

From January 2003 through January 2009, mean monthly minimum winter flows of 0.0 cfs were recorded. From March 2010 to the present, no flows of 0.0 cfs have been recorded in the gaging station record. However, the lack of recorded 0.0 cfs flows may be due to the 2010 change in the USGS offices responsible for the site, including a difference in procedures and more frequent late-winter site visits (M. Schellekens [USGS], personal communication to Ken Karle, Hydraulic Mapping and Modeling. January 31, 2020).

Direct stream discharge measurements are required to create a gaging station rating curve, which converts stage (water height) into discharge. The USGS maintains a database of 155 discharge measurements made at Colville River Gaging Station 15875000 at Umiat between March 1, 1953, and October 18, 2019. December through April winter measurements are provided in Table E.8.3 (USGS 2022).

Table E.8.3. Winter Field Discharge Measurements at Umiat, U.S. Geological Survey Gaging Station 15875000

Measurement Number	Date	Streamflow (cfs)	Ice Cover	Measurement Rating[a]
1	4/1/1953	0	Yes	Unspecified
15	12/2/2003	197	Yes	Poor
16	2/23/2004	2.2	Yes	Poor
26	12/1/2004	85.1	Yes	Poor
27	1/12/2005	23.4	Yes	Fair
44	12/4/2006	118	Yes	Fair
45	1/22/2007	22.4	Yes	Poor
46	3/27/2007	0	Yes	Good
52	12/12/2007	81.0	Yes	Fair
64	1/18/2009	12.3	Yes	Poor
71	2/11/2010	17.4	Yes	Poor
77	3/4/2011	2.6	Yes	Poor
78	3/30/2011	0.3	Yes	Poor
89	3/4/2012	3.8	Yes	Fair
97	1/8/2013	21.7	Yes	Poor
98	3/2/2013	2.6	Yes	Poor
106	1/21/2014	17.9	Yes	Poor
107	3/1/2014	4.4	Yes	Poor
108	3/31/2014	6.4	Yes	Poor
116	1/12/2015	46.0	Yes	Poor
117	4/15/2015	11.9	Yes	Poor
124	1/26/2016	16.2	Yes	Poor
125	3/14/2016	5.4	Yes	Poor
126	4/18/2016	2.3	Yes	Fair
134	3/14/2017	1.0	Yes	Poor
141	1/16/2018	24.0	Yes	Poor
142	4/16/2018	5.7	Yes	Poor
149	2/10/2019	31.4	Yes	Poor
150	3/27/2019	19.7	Yes	Poor

Source: USGS 2022

Notes: cfs (cubic feet per second). Table shows all the published data from December through April data for the time period listed for USGS Gaging Station 15875000.

[a] The measurement rating is used to describe the relationship between stage (water surface elevation) and discharge. An equation is used to describe the curve, since it changes constantly as the riverbed changes. Winter measurements are not used to help construct the measurement rating curve, as the stage measurements are unreliable due to the presence of ice. The measurement rating is not a rating of the accuracy of the data.

Downstream from Umiat, the probability of having flow in every month of the year increases as the drainage area increases. Similarly, the magnitude of the flow is likely to increase roughly proportional to the drainage area increase. Thus, when the average monthly mean April flow is 3.1 cfs at Umiat, where the drainage area is approximately 13,860 square miles, the average monthly mean April flow may be 1.5 times than that near Nuiqsut (4.7 cfs), where the drainage area is 20,670 square miles. Therefore, the flow at Ocean Point is likely higher than the flow at Umiat.

Ocean Point is located at a distinct transition of the Colville River channel pattern. Starting approximately 40 miles upriver from Ocean Point, the Colville, joined by several tributaries within the reach (Anaktuvuk River, Kogosukruk River, and Kikiakrorak River), flows north in a wide floodplain with two dissimilar side-by-side channel patterns. The main channel system on the west side includes interconnected distributary channels within a sparsely vegetated floodplain that includes depositional longitudinal and transverse bars. On the right side, multiple smaller channels take the form of serpentine (scroll) meanders, with extensively developed riparian vegetation. Five miles upstream from Ocean Point, the river enters a sweeping 180-degree right-hand bend. At Ocean Point, the river transitions to a single meandering channel, although remnant abandoned channels are readily apparent in aerial imagery. The river remains primarily in a single channel for another 20 miles to the east and northeast before entering the Colville River Delta.

Available data specific to the Colville River at Ocean Point are summarized in Table 3.8.4. Although the data are limited, Ocean Point has been used as a **rolligon** crossing for a number of years by various users (users are described in Section 3.14, *Land Ownership and Use*) because the area is shallow and has the potential for **bottom-fast ice**.

Table E.8.4. Water Data for the Colville River at Ocean Point

Date	Flow or Ice Conditions	Water Temperature (degrees C)	Salinity (ppt)	Source
December 10, 2007	Ice not grounded, approximately 2 to 3 feet water depth under the ice.	NC	NC	J. Winters [ADF&G], personal communication to DOWL. January 16, 2020.
April 4, 2019	Grounded ice to 0.7-foot water depth, 0.5 to 6.2 feet ice thickness.	NC	NC	CPAI 2019b
September 5, 2019	28,900 cubic feet per second. Open channel conditions. Average water depth 5.7 feet.	9.8 to 10.0	0.1	MBI 2019
December 31, 2019[a]	Ice grounded near both banks. Floating ice thickness is 2.8 feet. Approximately 1.2 to 2.2 feet of water under the ice. Velocity is 0.15 to 0.25 feet per second.	0.1	0.2	CPAI 2019b
February 25, 2020[a]	Ice grounded at both banks and in the middle of the channel. Water columns are less than 1.3 feet deep. Floating ice thickness is 4.6 feet.	0.4	0.26	CPAI 2020, MBI 2020a
March 10, 2021	Ice not grounded, approximately 4.6 feet water depth under the ice. Floating ice thickness is 5.5 feet. Velocity is 0 feet per second.	-0.1	0.47	CPAI 2022; MBI 2021

Note: ADF&G (Alaska Department of Fish and Game); C (Celsius); CPAI (ConocoPhillips Alaska, Inc.); NC (not collected); ppt (parts per thousand). Data collected at similar, but not the same, locations near Ocean Point.
[a] More data for this date are provided in Table E.8.5.

Michael Baker International (MBI) collected field data at two potential crossing locations on the Colville River near Ocean Point (Figure E.8.1). Data included cross-sectional river bottom profiles, discharge, velocity, water depth, **water surface elevation** (WSE), site conditions, and general in situ water quality parameters (Michael Baker International 2020b). Soil active layer depths were also investigated for both banks of each crossing. Table E.8.5 summarizes the discharge measurements for Ocean Point at two locations and the coincident discharge at USGS Gaging Station 15875000 at Umiat.

Table E.8.5. Summary of Discharge Data Collected at Ocean Point in 2019, 2020, and 2021

Ocean Point Transect	Date	Time	Measured Width (feet)	Measured Area (square feet)	Average Velocity (feet/second)	Measured Discharge (cfs)	Coincident Discharge at USGS Gaging Station 15875000 (cfs)
1	September 5, 2019	2:50 p.m.	1,270	7,570	3.0	29,068	19,900
6 (8.5 miles downstream of Transect 1)	September 5, 2019	4:50 p.m.	1,803	6,189	2.83	28,874	19,600
1	December 31, 2019	12:00 p.m.	650	880	0.15	135	Unavailable
1	February 25, 2020	Unavailable	304	228	0.04	9	Unavailable
1	February 17, 2021	12:15 p.m.	450	495	0.03	13.8	Unavailable
1	March 10, 2021	11:17 a.m.	118	55	0.01	0.7	Unavailable

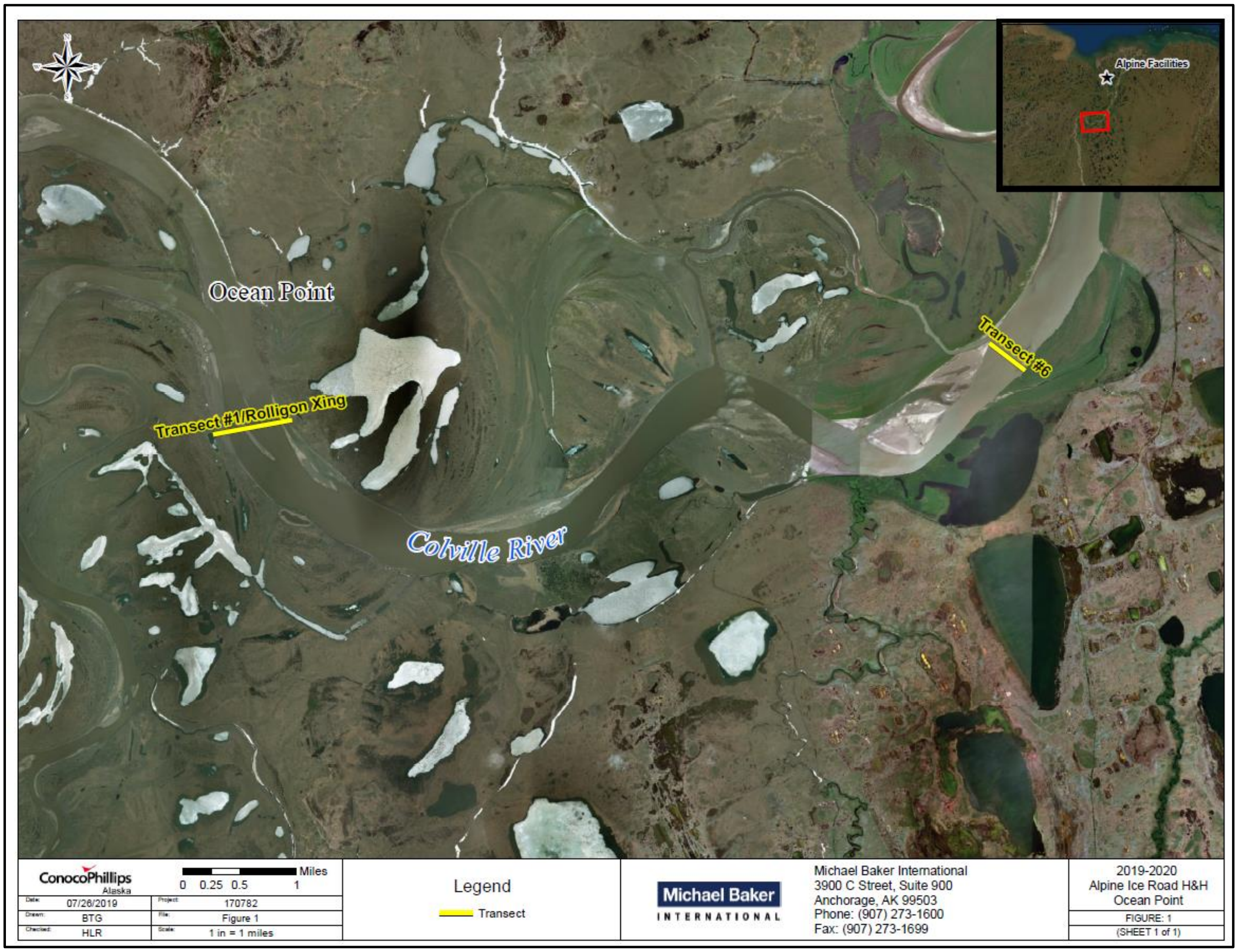
Source: CPAI 2022; MBI 2019, 2020b, 2021; USGS 2022
Note: cfs (cubic feet per second); USGS (U.S. Geological Survey).

Based on the data available for Ocean Point and Umiat, discharge at Ocean Point was estimated using the drainage-area ratio method (Emerson, Vecchia et al. 2005) commonly used to estimate individual streamflow discharges for sites where no streamflow data are available using data from one or more nearby gaging stations (Table E.8.6). More information on how this estimate was developed is in Karle (2020) and USGS (2022), provided as Appendix E.8B, *Ocean Point Technical Memorandums*.

Table E.8.6. Estimated Colville River Mean Monthly Discharge (cubic feet per second) at Ocean Point

Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
41.3	18.3	9.7	7.5	51,173	57,762	24,566	32,693	26,602	5,759	524.3	125.0

Note: Estimate based on mean monthly discharge at Umiat, 2010–2021 (USGS 2020) using the drainage-area ratio method (Emerson, Vecchia et al. 2005).



Source: MBI 2019

Figure E.8.1. Ocean Point Data Collection Locations

1.2.2 Fish Creek

Fish Creek has its headwater in the Arctic foothills and flows into Harrison Bay just east of the Colville River Delta. It has a drainage area of approximately 836 square miles, including its major tributaries: Judy (Kayyaaq) Creek, Judy (Iqalliqpik) Creek, and the Ublutuoch (Tiŋmiaqsiuġvik) River (Figure 3.8.1). The Willow Master Development Plan Project (Project) would cross or come near to all of these tributaries, which are described below.

The Project would cross Fish Creek at approximately RM 55.5, where the bankfull width is approximately 330 feet, the average bankfull depth is approximately 4.5 feet, and the depth to thalweg is approximately 6.4 feet (CPAI 2018b).

Spring breakup stage and discharge have been measured in Fish Creek for 17 years at RM 32.4 (Table E.8.7) (J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018), about 22.8 RMs downstream from the proposed infrastructure. During that time, water began to flow between May 12 and June 5, with a median date of May 27. The annual peak discharge occurred between May 23 and June 18, with a median date of June 9. In 6 out of 17 years the peak stage occurred earlier and was higher than the stage at the time of the peak discharge. The largest difference between the peak stage and the stage at the peak discharge was 1.51 feet. The time from the beginning of flow to the peak discharge varied between 6 and 24 days, with a median time of 11 days. The annual peak discharge varied from 2,040 to 5,400 cfs, with a median of 3,370 cfs. Freeze-up data were collected in 14 of the 17 years. During that time, freeze-up occurred between October 4 and October 30, with a median date of October 17.

Table E.8.7. Summary of Annual Peak Stage and Annual Peak Discharge for Fish Creek at River Mile 32.4

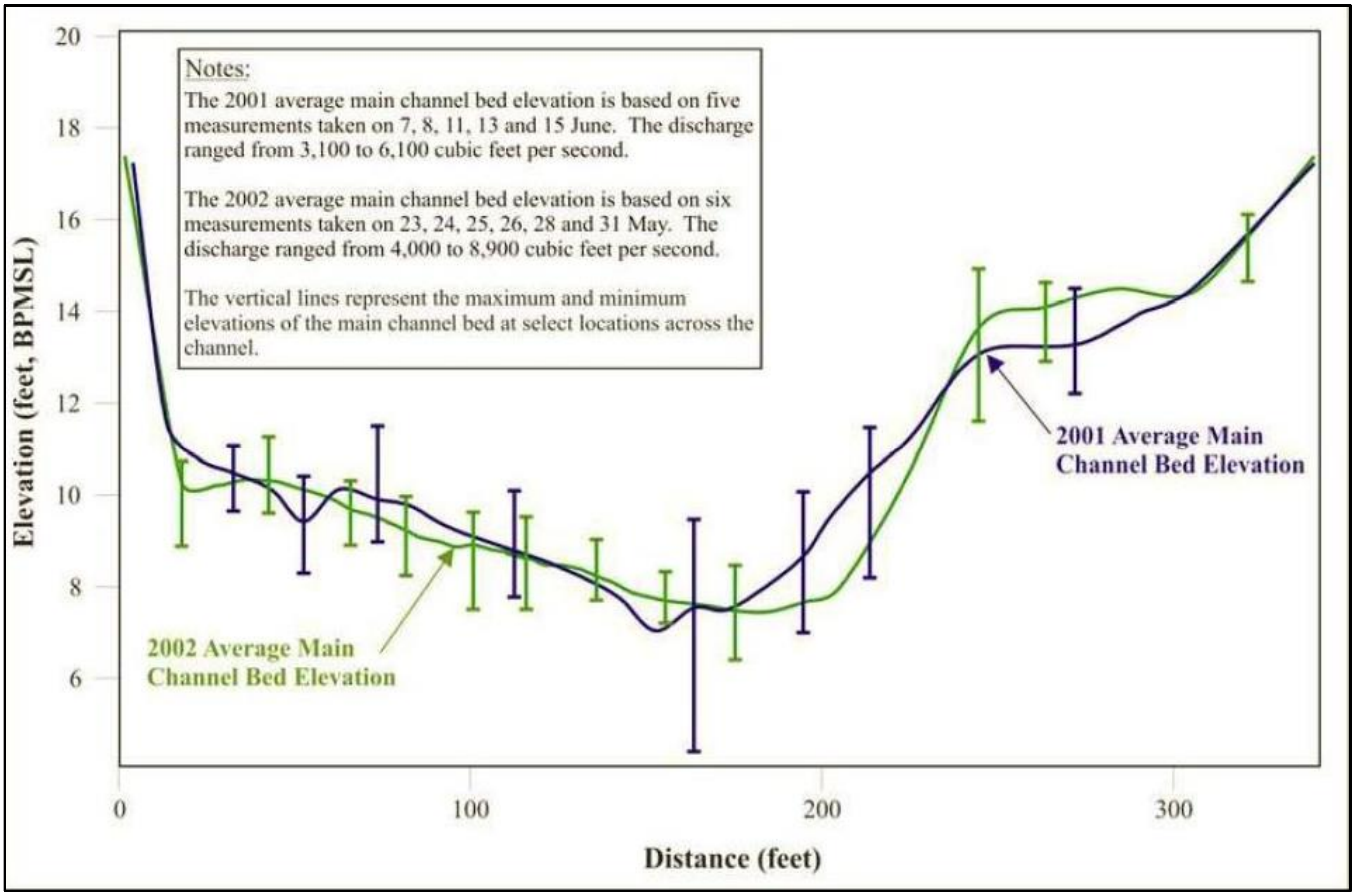
Year	Date Flow Begins (m/d)	Date of Freeze-Up (m/d)	Annual Peak Stage Date (m/d)	Annual Peak Stage (ft)	Annual Peak Stage Discharge (cfs)	Annual Peak Discharge Date (m/d)	Annual Peak Discharge Stage (ft)	Annual Peak Discharge (cfs)	Zero Flow to Peak Q (days)
2001	6/5	N/A	6/15	22.25	3,640	6/15	22.25	3,640	10
2002	5/17	N/A	5/27	22.42	3,685	5/27	22.42	3,685	10
2003	6/1	10/7 e	6/12	23.87	3,470	6/12	23.87	3,470	11
2004	6/2	10/30 e	6/9	23.48	4,410	6/9	23.48	4,410	7
2005	6/5	10/10 e	6/6	21.74	1,040	6/1	21.44	2,800	13
2006	5/27	10/16 e	6/12	21.72	3,170	6/12	21.72	3,170	16
2007	5/31	10/17 e	6/9	20.57	2,200	6/9	20.57	2,200	9
2008	5/23	10/4 e	6/6	20.12	2,270	6/6	20.12	2,270	14
2009	5/21	10/13	6/3	21.49	3,240	6/3	21.49	3,240	13
2010	6/1	10/8	6/9	23.50	3,730	6/9	23.50	3,730	8
2011	5/28	10/23	6/3	23.12	2,120	6/8	21.61	2,610	11
2012	5/25	10/20	6/6	22.25	2,720	6/11	21.93	3,510	17
2013	5/31	10/17	6/12	23.98	5,400	6/12	23.98	5,400	12
2014	5/15	10/17	5/20	22.35	2,290	6/8	21.77	3,370	24
2015	5/17	10/8	5/23	24.14	4,830	5/23	24.14	4,830	6
2016	5/12	10/21	5/27	20.10	1,470	5/31	20.08	2,040	19
2017	5/27	N/A	6/2	21.00	1,510	6/7	20.96	2,740	11

Source: J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018
Note: cfs (cubic feet per second); d (day); e (estimate); ft (feet); m (month); N/A (not available); Q (discharge). Coordinates of the site (NAD27): 70.2706, -151.8692.

The Fish Creek channel is relatively low gradient and highly sinuous. Undercut stream banks and bank sloughing are common along the outside of meander bends (URS Corporation 2003). The riverbed appears to be very mobile. The river banks and bed of Fish Creek are composed of a mixture of sand and silt, with a median riverbed grain size of 0.13 millimeter (mm) at RM 25.1 and 0.037 mm at RM 32.4 (URS Corporation 2001). During the 2001 spring breakup, the maximum observed change in riverbed elevation was 5 feet at RM 25.1 and 7 feet at RM 32.4 (URS Corporation 2001). During the 2002 spring breakup, the maximum observed change in riverbed elevation was 3 feet at RM 25.1 and 1 foot at RM 32.4 (URS Corporation 2003). Figures E.8.2 and E.8.3 present the average riverbed elevation in 2001 and 2002 at RM 25.1 and RM 32.4, respectively. Also shown is the extent of the deviations from the average during those years.

On May 26, 2002, the discharge, suspended sediment load, and bedload were all measured at RM 25.1. The discharge was 8,900 cfs (the same as the annual peak discharge recorded the day before); the bedload was 423 tons per day; the suspended sediment load was 8,400 tons per day; and the total sediment load was computed to be 8,800 tons per day (URS Corporation 2003). The concentration of suspended sediment was 349 milligrams per liter. Approximately 6.1% of the bedload was composed of organic material (URS Corporation 2003). The median diameter of the mineral portion of the bedload was 0.12 mm and the specific gravity of the mineral portion of the bedload was 2.640 (URS Corporation 2003).

The daily changes in the channel bed that were recorded during the 2001 and 2002 breakups suggest that the bed is easily eroded, moved, and shaped by the flow (URS Corporation 2003). The interaction of the water-sediment mixture and the sand bed can create different bed configurations, such as ripples, dunes, transition, and antidunes. The type of bed form present affects both the hydraulic roughness and the rate of sediment transport, which affects the water velocity, the depth of the scour, and the WSE. At RM 25.1, dunes are probably present at discharges of 3,100 to 4,800 cfs (URS Corporation 2003). At discharges between 6,100 and 8,900 cfs, both dunes and antidunes are probably present (URS Corporation 2003). The antidunes are probably confined to the deepest and/or fastest portions of the channel (URS Corporation 2003). As the discharge increases beyond 6,100 cfs, the portion of the bed covered by antidunes is likely to increase (URS Corporation 2003). At RM 32.4, both ripples and dunes are probably present at discharges of 1,500 to 2,300 cfs (URS Corporation 2003). At discharges between 3,100 and 3,700 cfs, dunes are probably the predominant bed form.



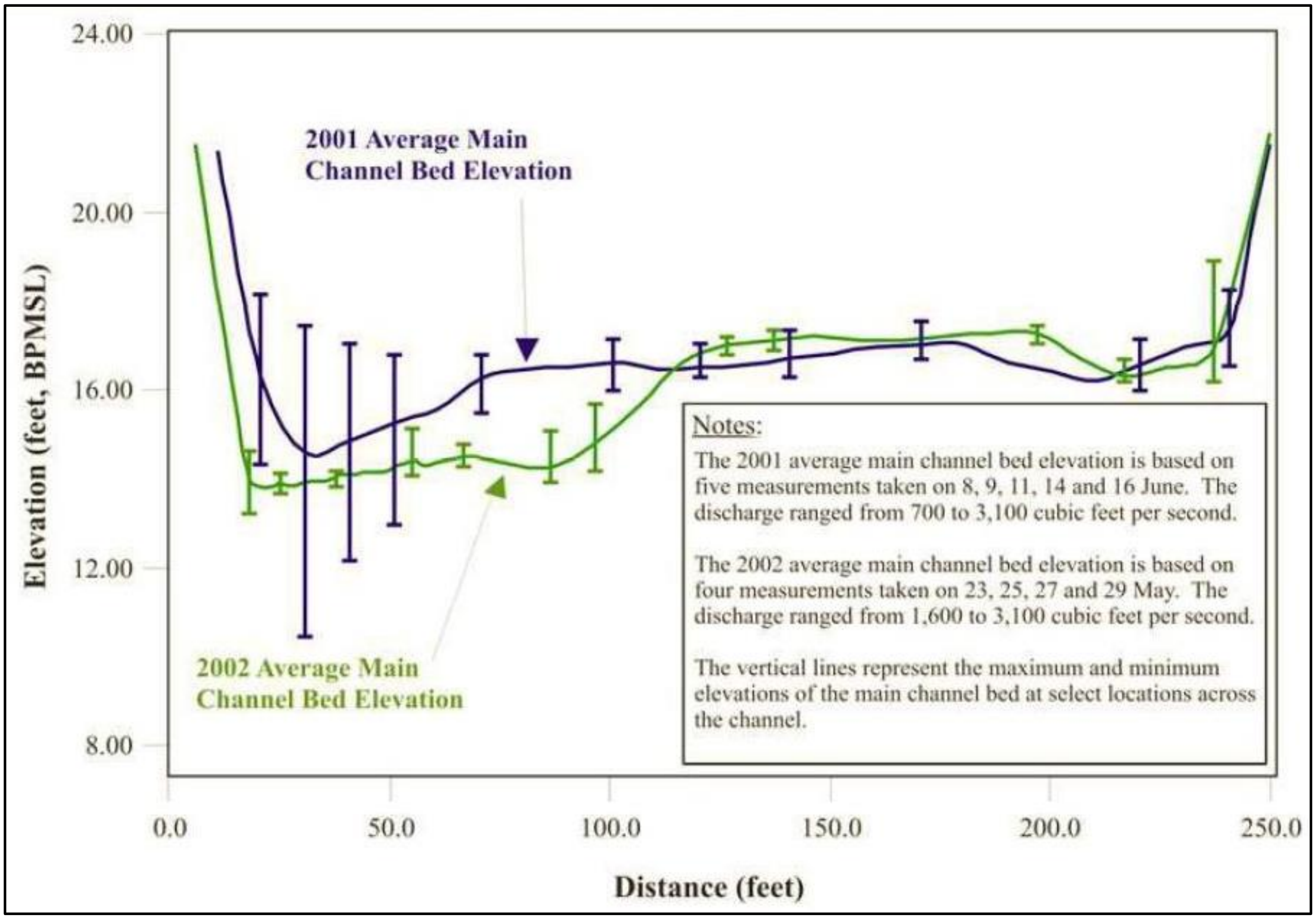
Source: URS Corporation 2003

Figure E.8.2. Average Riverbed Elevation in Fish (Iqalliqpik) Creek at River Mile 25.1, 2001 and 2002

Discharge and water surface slope measurements, along with surveyed cross-sections and a water surface profile model, were used to estimate hydraulic roughness in the channel on a particular day during spring breakup using data collected in both 2001 and 2002. At RM 25.1, the channel hydraulic roughness on the day of the measurements was 0.021 in both 2001 and 2002 (URS Corporation 2003). At RM 32.4, the channel hydraulic roughness on the day of the measurements was 0.028 in 2001 and 0.030 in 2002 (URS Corporation 2003). At RM 43.3, the channel hydraulic roughness on the day of the measurements was 0.027 in both 2001 and 2002. Although the values probably change from day to day during breakup and from year to year, the computed values are within the range of values one would expect when dunes and antidunes are present on the riverbed (0.014–0.035). Computations of hydraulic roughness based on measured discharge and water surface slope, and normal depth computations, on 5 to 6 days during breakup in both 2001 and 2002 suggested a slightly bigger range in hydraulic roughness values, but the values are still within the range one would expect when dunes and antidunes are present (URS Corporation 2003).

Seventeen years of summer flow data is available for Fish Creek at RM 32.4 (J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018) . A summary of the available mean monthly discharge data is provided in Table E.8.8.

In 2018, a monitoring site was established at RM 55.5 (Michael Baker Jr. Inc. 2018). Observations during the 2018 spring breakup indicated the peak stage (46.25 feet [North American Vertical Datum of 1988]) occurred 0.5 hour after the peak discharge (4,400 cfs; WSE 46.03 feet NAVD88) and at a time when the channel was not impacted by snow or ice within the channel at the monitoring site (Michael Baker Jr. Inc. 2018). This suggests that the peak stage was due to backwater, possibly due to an ice jam downstream. Prior to the peak discharge, WSEs at the monitoring site had been impacted by snow and ice in the channel and an ice jam (Michael Baker Jr. Inc. 2018). It was also noted that the riverbed was mobile during spring breakup (Michael Baker Jr. Inc. 2018). Figure E.8.4 presents a cross-section of the channel showing the discharge measurement. In general, the WSE decreased throughout the summer but increased in early September in response to a rain event (Michael Baker Jr. Inc. 2018). Maximum and minimum summer WSEs were 43.17 feet NAVD88 (fall rainfall peak) and 40.74 feet NAVD88.



Source: URS Corporation 2003

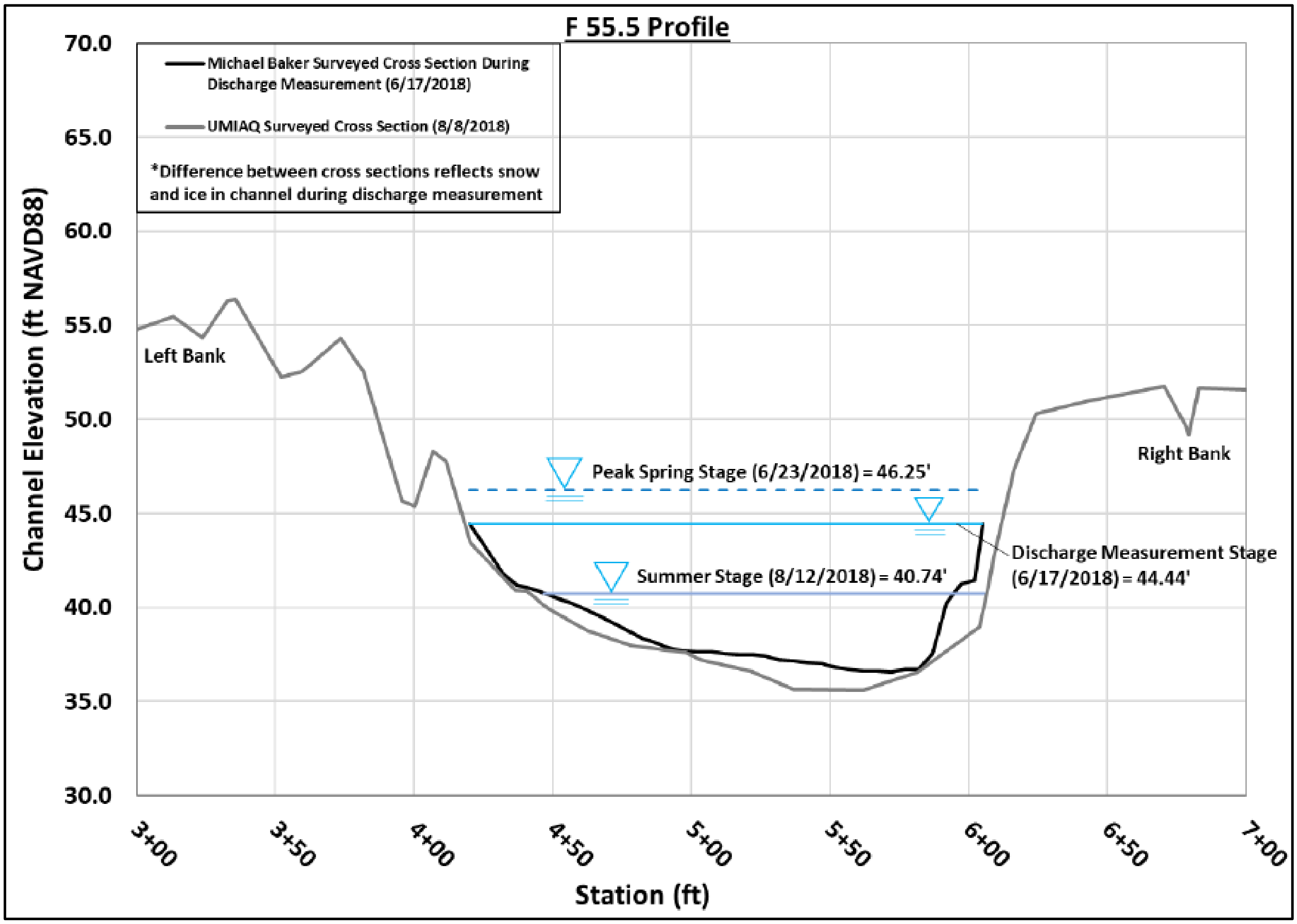
Figure E.8.3. Average Riverbed Elevation in Fish Creek at River Mile 32.4, 2001 and 2002

Table E.8.8. Mean Monthly Discharge (cubic feet per second) in Fish Creek at River Mile 32.4

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2001	–	–	–	–	–	1,761	697	412	298	242	208	173
2002	137	104	70	35	808	1,118	526	252	259	230	199	168
2003	137	107	77	47	16	1,620	633	391	341	173	25	0
2004	0	0	0	0	0	2,311	732	331	298	196	38	0
2005	0	0	0	0	0	1,484	750	282	171	44	6	0.2
2006	0	0	0	0	47	1,643	555	298	210	132	40	2
2007	0	0	0	0	0	10,004	259	66	37	12	0.1	0
2008	0	0	0	0	112	911	224	113	73	17	0	0
2009	0	0	0	0	432	1,684	405	179	196	63	5	0
2010	0	0	0	0	0	1,719	532	321	191	59	3	0
2011	0	0	0	0	37	1,600	437	206	185	120	28	2
2012	0	0	0	0	15	1,748	459	240	256	185	25	0
2013	0	0	0	0	0.6	2,617	803	439	386	293	27	0
2014	0	0	0	0	753	2,014	877	353	282	190	31	0.7
2015	0	0	0	0	1424	1,637	402	203	165	62	19	0.6
2016	0	0	0	0	325	1,085	372	245	518	352	45	1
2017	0	0	0	0	91	1,555	486	619	846	806	262	14

Source: J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018
Note: "–" (no data).

Observations during the 2019 spring breakup indicated the peak stage of 44.71 feet NAVD88 and an estimated peak discharge of 5,100 cfs, both on May 28. Summer stage levels generally remained below peak spring stage. During a late summer precipitation event, stage crested at levels observed near the end of spring breakup. The minimum recorded summer stage was 40.08 feet NAVD88 on July 20, and the highest recorded summer stage was 42.59 feet NAVD88 on August 29 (Michael Baker International 2020b).



Source: Michael Baker Jr. Inc. 2018

Figure E.8.4. Cross-Section on Fish Creek at River Mile 55.5

Table E.8.9 presents flood-peak magnitude and frequency estimates for Fish Creek at RM 55.5 based on the Curran et al. (2003) USGS 2003 regression equations (Michael Baker Jr. Inc. 2018).

Table E.8.9. Flood Magnitude and Frequency in Fish Creek at River Mile 55.5

Percent Chance of Exceedance in Any Given Year (%)	Recurrence Interval (years)	Annual Peak Discharge (cfs)
50	2	10,400
20	5	15,200
10	10	18,200
4	25	21,800
2	50	24,400
1	100	26,900

Source: Michael Baker Jr. Inc. 2018

Spring breakup observations have also been made at the following sites:

- RM 0.7 in 2001 (URS Corporation 2001), 2002 (URS Corporation 2003), 2005 (Michael Baker International 2005), and 2006 (Michael Baker International 2007)
- RM 10.3 in 2005 (Michael Baker International 2005) and 2006 (Michael Baker International 2007)
- RM 11.7 in 2001 (URS Corporation 2001) and 2002 (URS Corporation 2003)
- RM 12.6 in 2001 (URS Corporation 2001) and 2002 (URS Corporation 2003)
- RM 18.4 in 2001 (URS Corporation 2001) and 2002 (URS Corporation 2003)
- RM 25.1 in 2005 (Michael Baker International 2005) and 2006 (Michael Baker International 2007)
- RM 32.4 in 2005 (Michael Baker International 2005) and 2006 (Michael Baker International 2007)
- RM 43.3 in 2001 (URS Corporation 2001) and 2002 (URS Corporation 2003)

Hydraulic designs on Fish Creek should consider the flood-peak data that have been collected on Fish Creek at RM 32.4, the highly mobile bed, the impact of ice and snow on annual peak WSEs, and the riverbed forms and hydraulic roughness likely to be present at the design discharge. In developing flood-peak magnitude and frequency estimates on streams in the Fish Creek basin, the 17 years of data collected at RM 32.4 should be considered. Single-station flood-peak magnitude and frequency analyses could be conducted with these data to estimate the flood-peak magnitude and frequencies at RM 32.4. A best estimate of the flood-peak magnitude and frequency at RM 32.4 could then be developed from a weighted average based on the uncertainty associated with estimates from each of two methods: the single-station frequency analysis and the Shell regression equations[1] (Arctic Hydrological Consultants and ERM 2015). The weighted average estimate would then be extrapolated to other locations within the basins as a proportion of the Shell regression equation estimate.

Since the hydraulic roughness is changing throughout spring breakup, when designing structures on this river it would be prudent to consider a range of hydraulic roughness values. Higher hydraulic roughness values will provide estimates with high WSEs and lower velocities. Lower hydraulic roughness values will provide estimates with lower WSEs and higher velocities. Both conditions are important when designing structures within the channel and floodplain.

1.2.2.1 Willow Creek 8

Willow Creek 8 is a tributary of Fish Creek. It has a meandering, incised channel with intermittent deep, beaded pools (Michael Baker Jr. Inc. 2018). The infield road for all action alternatives would cross Willow Creek 8 at the MBI TBD_6 and SW22 monitoring sites, about 1.7 and 3 RMs upstream of the Fish Creek confluence, respectively (Michael Baker Jr. Inc. 2018). At the SW22 crossing, Willow Creek 8 has a poorly defined channel in a low-lying area of polygon troughs connecting Lake M0305 to an unnamed lake to the south (Michael Baker Jr. Inc. 2018). At TBD_6, the Willow Creek 8 channel is incised and well defined. At TBD_6, the bankfull width is approximately 32 feet and the average bankfull depth is approximately 4.8 feet (CPAI 2018b). Monitoring sites TBD_6 and SW22 were established in 2018.

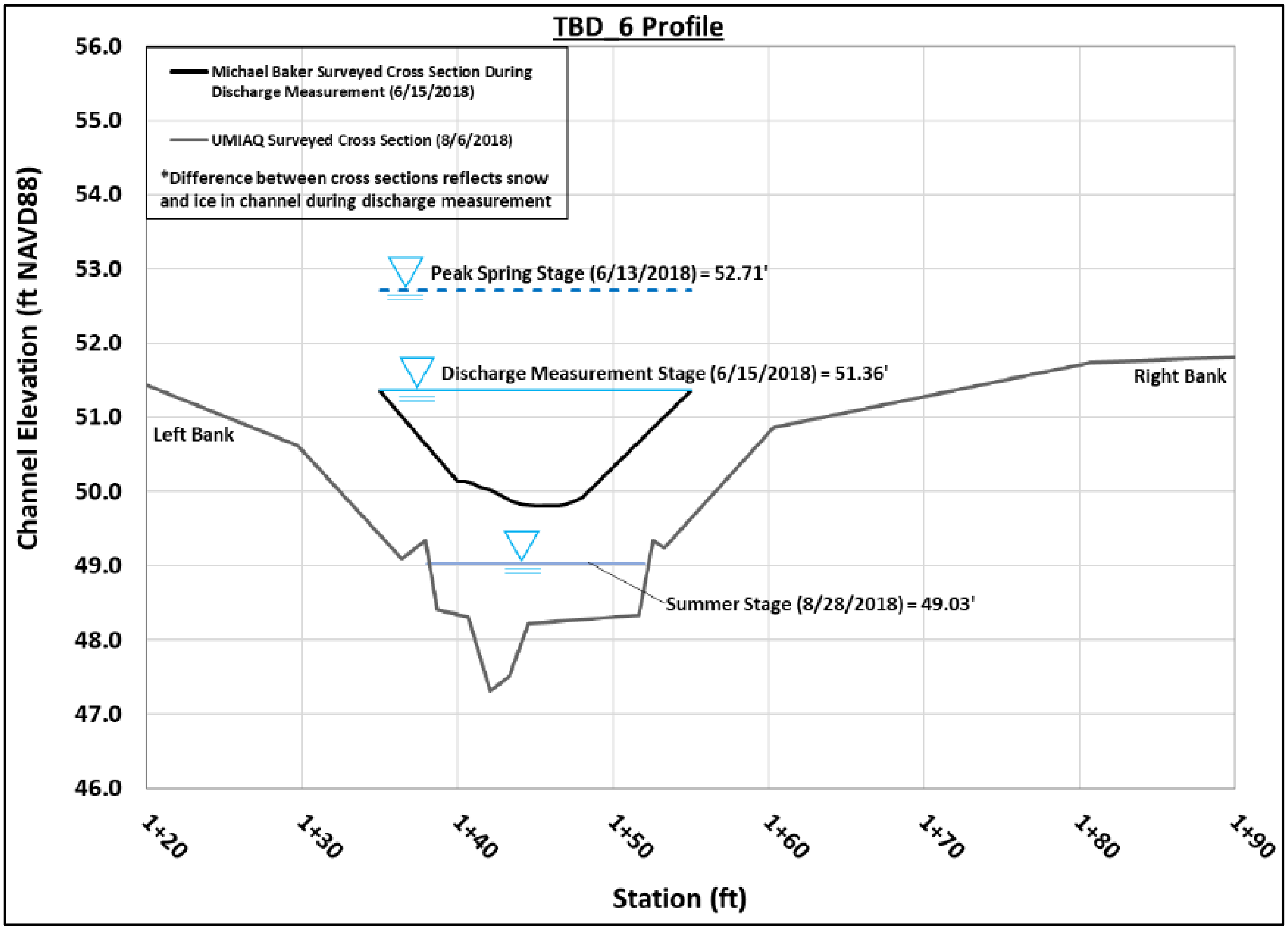
Due to low relief and the wide area of possible flow paths, the SW22 gage was not placed in the main flow path, and neither peak stage nor peak discharge information was collected during the 2018 spring breakup (Michael Baker Jr. Inc. 2018). At TBD_6, the peak stage was 52.71 feet NAVD88 and occurred on June 13. At the time of the peak stage there was snow and ice in the channel and overbank flooding (Michael Baker Jr. Inc. 2018). It is likely that the peak stage occurred prior to the peak discharge (Michael Baker Jr. Inc. 2018). The date and magnitude of the peak discharge were not recorded.

Figure E.8.5 shows a cross-section of the channel at TBD_6, including a cross-section from a June 15, 2018, discharge measurement, and the 2018 spring peak stage. The difference in the cross-sections, and the difference between the June 13 and 15 WSEs, is an indication of the magnitude of the impact of snow and ice on the peak stage and during the likely time of the peak discharge.

In general, the stage at TBD_6 fell throughout the summer except for fluctuations due to summer precipitation events (Michael Baker Jr. Inc. 2018). At the end of the summer monitoring season, the stage increased due to a late summer precipitation event (Michael Baker Jr. Inc. 2018). However, the stage remained well below the spring breakup peak stage throughout the summer (Michael Baker Jr. Inc. 2018). The maximum and minimum summer stages at TBD_6 were 50.18 feet and 49.03 feet NAVD88, respectively (Michael Baker Jr. Inc. 2018).

During the 2019 spring breakup, the TBD_6 peak stage was 53.72 feet NAVD88 on May 29. A discharge of 90 cfs was measured on May 30. The measured summer stage levels remained well below the spring breakup peak stage. The stage fluctuations reflected summer precipitation events. The minimum recorded summer stage was 49.07 feet NAVD88 on July 30 and the highest recorded summer stage was 50.96 feet NAVD88 on August 28 (Michael Baker International 2020b).

[1] The Shell regression equations are suggested rather than the 2003 USGS regression equations because considerably more North Slope river data were used to prepare the Shell regression equations than the USGS regression equations.



Source: Michael Baker Jr. Inc. 2018

Figure E.8.5. Cross-Section of Willow Creek 8 at Monitoring Site TBD_6

1.2.2.2 Judy (Iqalliqpik) Creek

Judy (Iqalliqpik) Creek has its headwater in the Arctic foothills and flows into Fish (Iqalliqpik) Creek at RM 26. Much of the Project infrastructure would be within the Judy (Iqalliqpik) Creek basin; Alternatives B (Proponent's Project) and D (Disconnected Access) would cross the main stem of Judy (Iqalliqpik) Creek at approximately RM 21.4 (Michael Baker Jr. Inc. 2018). At RM 21.4, the bankfull width is approximately 175 feet and the average bankfull depth is approximately 2.0 feet (CPAI 2018b). Several tributaries of Judy (Iqalliqpik) Creek are also crossed by the infrastructure: Judy (Kayyaaq) Creek, Willow Creek 1, Willow Creek 2, Willow Creek 3, Willow Creek 4, and Willow Creek 4A.

The spring breakup stage and discharge have been measured on the main stem of Judy (Iqalliqpik) Creek for 17 years at RM 7 (J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018) , about 13.3 RMs downstream from the proposed infrastructure (Table E.8.10). The date on which water began to flow during that time was between May 11 and June 5, with a median date of May 26. The annual peak discharge occurred between May 18 and June 10, with a median date of June 5. In 6 out of 17 years the peak stage occurred earlier and was higher than the stage at the time of the peak discharge. The largest difference was 2.39 feet. The time from the beginning of flow to the peak discharge varied between 1 and 12 days, with a median time of 8 days. The annual peak discharge varied from 2,250 to 9,210 cfs, with a median of 4,770 cfs. Freeze-up data were collected in 14 of the 17 years. During that time, freeze-up occurred between September 20 and October 11, with a median date of September 26.

Judy (Iqalliqpik) Creek has a relatively low gradient and a highly sinuous channel. Undercut stream banks and bank sloughing are common along the outside of meander bends (URS Corporation 2003). The Judy (Iqalliqpik) Creek riverbed appears to be very mobile. The river banks and bed are composed of a mixture of sand and silt, with a median riverbed grain size of 0.17 mm at RM 7 (URS Corporation 2001). During the 2001 spring breakup, the maximum observed change in riverbed elevation at RM 7 was 5 feet (URS Corporation 2001). During the 2002 spring breakup, the maximum observed change in riverbed elevation at RM 7 was 2 feet (URS Corporation 2003). Figure E.8.6 presents the average riverbed elevation in 2001 and 2002 at RM 7 and the deviations from average during those years.

Table E.8.10. Summary of Annual Peak Stage and Discharge for Judy (Iqalliqpik) Creek at River Mile 7

Year	Date Flow Begins (m/d)	Date of Freeze-Up (m/d)	Annual Peak Stage Date (m/d)	Annual Peak Stage (ft)	Annual Peak Stage Discharge (cfs)	Annual Peak Discharge Date (m/d)	Annual Peak Discharge Stage (ft)	Annual Peak Discharge (cfs)	Zero Flow to Peak Q (days)
2001	6/5	N/A	6/10	27.11	N/A	6/10	27.11	5,590	5
2002	5/18	N/A	5/25	26.81	N/A	5/25	26.81	7,150	7
2003	5/31	9/25	6/6	28.00	N/A	6/6	25.61	4,720	7
2004	5/18	9/26	5/26	28.55	N/A	6/5	26.62	4,770	8
2005	6/2	9/26	6/6	27.47	N/A	6/10	25.99	4,400	8
2006	5/26	10/5	5/30	26.00	N/A	6/7	24.97	3,930	12
2007	5/26	9/23	6/5	25.40	N/A	6/5	25.40	4,560	10
2008	5/22	9/29	5/29	24.93	N/A	5/29	24.93	3,850	7
2009	5/18	9/23	5/27	25.16	N/A	5/28	24.78	2,250	10
2010	6/2	9/26	6/8	27.95	N/A	6/8	27.95	9,210	6
2011	5/30	10/1	5/31	30.05	N/A	5/31	29.66	5,480	1
2012	5/26	10/9	6/5	26.86	N/A	6/5	26.86	6,950	10
2013	5/31	9/26	6/9	26.86	N/A	6/9	26.86	6,300	10
2014	5/14	10/10	5/18	30.07	N/A	5/18	30.07	5,410	4
2015	5/18	9/20	5/22	29.21	N/A	5/22	29.21	5,990	4
2016	5/11	10/11	5/22	26.21	N/A	5/22	26.21	4,010	11
2017	5/26	N/A	6/3	25.85	N/A	6/3	25.85	4,070	8

Source: J. Aldrich (Arctic Hydrologic Consultants), personal communication to Richard Kemnitz (BLM). September 11, 2018

Note: cfs (cubic feet per second); d (day); e (estimate); ft (feet); m (month); N/A (not available); Q (discharge); RM (river mile). The coordinates of the site (NAD27): 70.2206, -151.8352).

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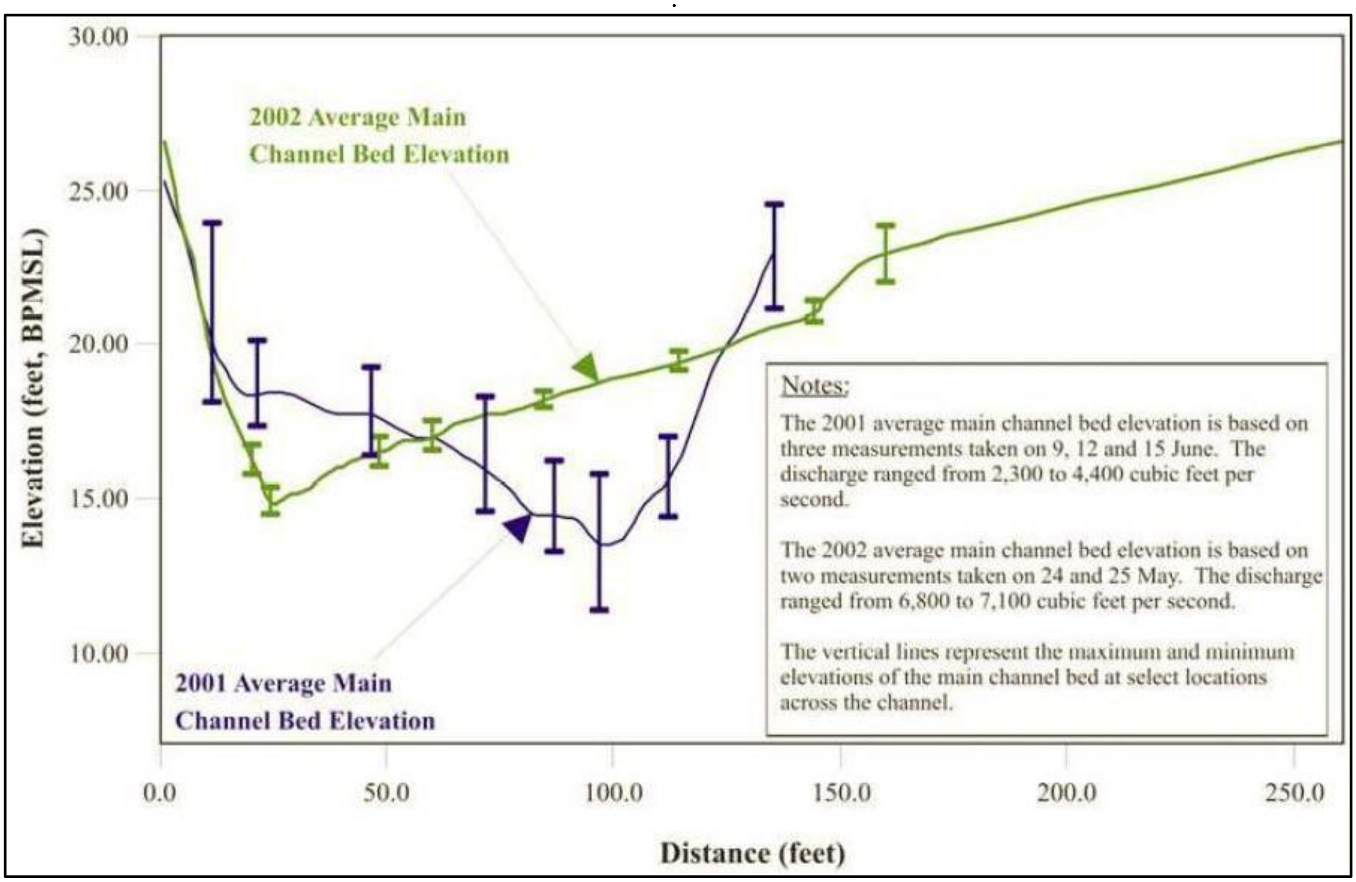


Source: URS Corporation 2003

Figure E.8.6. Average Riverbed Elevation for Judy (Iqalliqpik) Creek at River Mile 7, 2001 and 2002

The daily changes in the channel bed that were recorded during the 2001 and 2002 breakups suggest that the bed is easily eroded, moved, and shaped by the flow (URS Corporation 2003). At RM 7, dunes are probably present at discharges on the order of 2,300 cfs (URS Corporation 2003). At discharges between 3,200 and 7,000 cfs, both dunes and antidunes are probably present (URS Corporation 2003). The antidunes are probably confined to the deepest and/or the fastest portions of the channel (URS Corporation 2003). At discharges above 7,000 cfs, it is likely that antidunes cover the bed (URS Corporation 2003).

Discharge and water surface slope measurements, along with surveyed cross-sections and a water surface profile model, were used to estimate hydraulic roughness in the channel on a particular day during spring breakup using data collected in both 2001 and 2002. At RM 7 the channel hydraulic roughness on the day of the measurements was 0.014 in 2001 and 0.024 in 2002 (URS Corporation 2003). At RM 13.8 the channel hydraulic roughness on the day of the measurements was 0.020 in 2001 and 0.024 in 2002 (URS Corporation 2003). Although the values probably change from day to day during breakup and from year to year, the computed values are within the range of values one would expect when dunes and antidunes are present on the riverbed (0.014–0.035). Computations of hydraulic roughness based on measured discharge and water surface slope, and normal depth computations, at RM 7 on several different days suggest that in 2001 the hydraulic roughness during ice- and snow-impacted conditions varied from 0.022 to 0.028 (URS Corporation 2003). Similar computations during open-water conditions in 2001 and 2002 suggest that the hydraulic roughness varies from 0.13 to 0.022.

Seventeen years of summer flow data is available for Judy (Iqalliqpik) Creek at RM 7 (J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018) . A summary of the available mean monthly discharge data is provided in Table E.8.11.

Table E.8.11. Mean Monthly Discharge (cubic feet per second) in Judy (Iqalliqpik) Creek at River Mile 7

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2001	–	–	–	–	–	1,448	175	175	176	129	78	26
2002	0	0	0	0	1,273	492	285	166	155	110	66	22
2003	0	0	0	0	1	1,306	307	171	214	60	0.9	0
2004	0	0	0	0	493	1,786	263	155	221	51	3	0
2005	0	0	0	0	0	1,717	271	72	63	13	0	0
2006	0	0	0	0	93	1,559	164	133	85	38	4	0
2007	0	0	0	0	1	879	65	21	14	2	0	0
2008	0	0	0	0	334	775	91	65	42	4	0	0
2009	0	0	0	0	513	904	103	90	166	38	3	0
2010	0	0	0	0	0	1,718	149	220	113	18	1	0
2011	0	0	0	0	250	1,473	167	81	151	65	3	0
2012	0	0	0	0	64	1,785	132	82	161	86	3	0
2013	0	0	0	0	6	2,537	264	170	186	93	8	0
2014	0	0	0	0	1,044	1,469	310	134	166	85	8	0
2015	0	0	0	0	1,268	650	128	89	110	12	0	0
2016	0	0	0	0	977	570	106	139	358	308	41	0
2017	0	0	0	0	165	1,557	144	512	753	600	73	3

Source: J. Aldrich (Arctic Hydrologic Consultants), personal communication to Richard Kemnitz (BLM). September 11, 2018
Note: "–" (no data).

At RM 13.8, spring breakup peak WSEs have been measured periodically since 2001 (Table E.8.12).

Table E.8.12. Historical Peak Stage in Judy (Iqalliqpik) Creek at River Mile 13.8

Year	Peak Stage (feet BPMSL)	Date
2019	35.81	5/27
2018	37.09	6/6
2017	34.68	6/4
2006	35.56	5/30
2005	37.25	6/4
2004	–	–
2003	36.58	6/6
2002	35.86	5/25
2001	39.66	6/7

Note: "–" (no data); BPMSL (British Petroleum Mean Sea Level). Table adapted from Table 4.3 in Michael Baker Jr. Inc. (2018).

Observations made during the 2018 spring breakup at RM 13.8 indicated the peak stage (37.09 feet NAVD88) occurred prior to the peak discharge (4,100 cfs; WSE 36.37 feet NAVD88). On the day of the peak discharge, some intermittent ice floes were observed and considerable snow was present along each bank, but no bottom-fast ice was observed (Michael Baker Jr. Inc. 2018). It was also noted that the riverbed was mobile on both the day of the peak discharge and 10 days after the peak discharge, and that on the later date a moving bed velocity averaging 0.7 feet per second was observed (Michael Baker Jr. Inc. 2018). In 2019, recorded stage data revealed multiple spikes followed by declines in stage, indicating ice jams and associated backwater releases upstream of the J13.8 reach.

At RM 21.4, spring breakup monitoring was conducted in 2017, 2018, and 2019 (CPAI 2018a; Michael Baker International 2020a; Michael Baker Jr. Inc. 2018). In 2017, the peak stage was recorded as 90.2 feet (arbitrary datum; [CPAI 2018a]); in 2018, the peak stage was recorded as 51.24 feet NAVD88 (Michael Baker Jr. Inc. 2018); and in 2019, the peak stage was recorded as 49.80 feet NAVD88 (Michael Baker International 2020a). In 2018, it was noted that the channel bed was highly mobile during spring breakup (Michael Baker Jr. Inc. 2018). Summer stage was measured in 2018 and indicated that the stage fluctuated with precipitation, but water levels remained below the peak spring breakup stage (Michael Baker Jr. Inc. 2018). The stage increased at the end of the summer monitoring period due to a late summer precipitation event. Maximum and minimum summer WSEs in 2018 were 47.49 feet NAVD88 (fall rainfall peak) and 44.78 feet NAVD88. In 2019, a late summer precipitation event caused the stage to crest to levels observed near the end of spring breakup. The peak summer stage was 49.8 feet on May 27.

Table E.8.13 presents flood-peak magnitude and frequency estimates for Judy (Iqalliqpik) Creek at RM 13.8 based on the Curran et al. (2003) USGS 2003 regression equations (Michael Baker Jr. Inc. 2018).

Table E.8.13. Flood Magnitude and Frequency in Judy (Iqalliqpik) Creek at River Mile 13.8

Percent Chance of Exceedance in Any Given Year (%)	Recurrence Interval (years)	Annual Peak Discharge (cubic feet per second)
50	2	7,400
20	5	10,900
10	10	13,100
4	25	15,800
2	50	17,700
1	100	19,500

Source: Michael Baker Jr. Inc. 2018

Spring breakup observations have also been made at the following sites:

- RM 16.5 in 2017 (CPAI 2018a)
- RM 31.0 in 2001 (URS Corporation 2001)

Hydraulic designs on Judy (Iqalliqpik) Creek should consider the flood-peak data that have been collected on Judy (Iqalliqpik) Creek at RM 7, the highly mobile bed, the impact of ice and snow on annual peak WSEs, and the riverbed forms and hydraulic roughness likely to be present at the design discharge. In developing flood-peak magnitude and frequency estimates on streams in the Judy (Iqalliqpik) Creek basin, the 17 years of data collected at RM 7 should be considered. A single-station flood-peak magnitude and frequency analyses could be conducted with these data to estimate the flood-peak magnitude and frequencies at RM 7. A best estimate of the flood-peak magnitude and frequency at RM 7 could then be developed from a weighted average, based on the uncertainty associated with estimates from each of two methods: the single-station frequency analysis, and the Shell regression equations[2] (Arctic Hydrological Consultants and ERM 2015). The weighted average estimate would then be extrapolated to other locations within the basins as a proportion of the Shell regression equation estimate.

Since the hydraulic roughness is changing throughout spring breakup, when designing structures on this river it would be prudent to consider a range of hydraulic roughness values. Higher hydraulic roughness values would provide estimates with higher WSEs and lower velocities. Lower hydraulic roughness values would provide estimates with lower WSEs and higher velocities. Both conditions are important when designing structures within the channel and the floodplain.

1.2.2.2.1 Judy (Kayyaaq) Creek

Judy (Kayyaaq) Creek is a tributary to Judy (Iqalliqpik) Creek. It has a highly sinuous and incised channel: over 8 feet from the top of the bank to the streambed and typically about 30 feet wide (Michael Baker Jr. Inc. 2018).

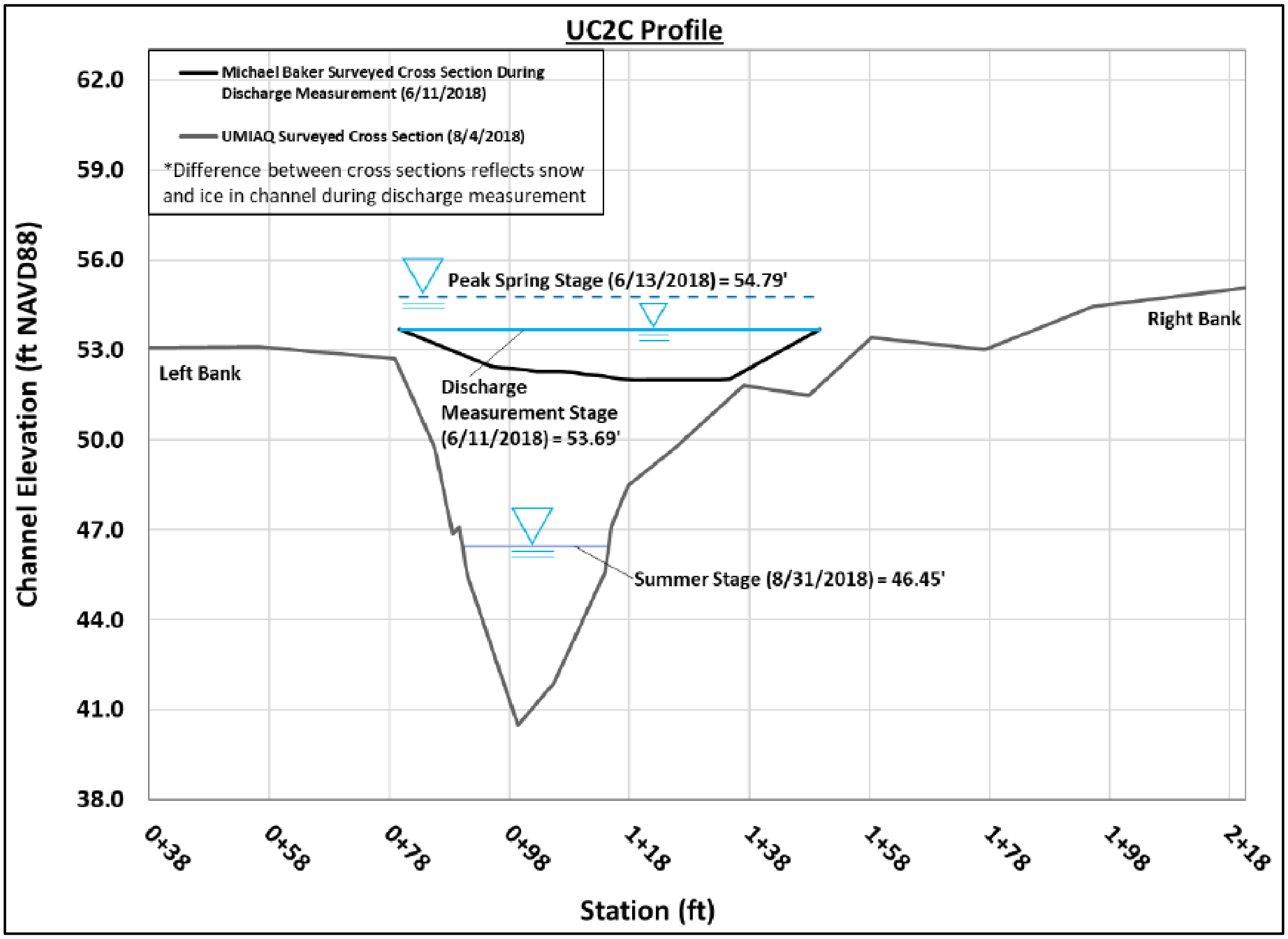
[2] The Shell regression equations are suggested rather than the 2003 USGS regression equations because considerably more North Slope river data was used to prepare the Shell regression equations than the USGS regression equations.

The UC2A, UC2B and UC2C gaging stations were established at approximately RM 8.4, 10.2, and 13.0, respectively (Michael Baker Jr. Inc. 2017). The UC2C gaging station is located where the infield road (for all action alternatives) would cross Judy (Kayyaaq) Creek (Michael Baker Jr. Inc. 2017), about 13 miles upstream from the confluence with Judy (Iqalliqpik) Creek. At RM 13.0 (UC2C gage) the bankfull width is approximately 20 feet and the average bankfull depth is approximately 5.5 feet (CPAI 2018b). Spring breakup and the summer stage have been monitored in both 2017 and 2018.

In both 2017 and 2018, the channel was full of wind-blown snow prior to the start of breakup (Michael Baker Jr. Inc. 2017, 2018). In 2017, it was reported that water began flowing on top of the drifted snow at all of the monitoring stations and then cut a channel down through the wind-blown snow (Michael Baker Jr. Inc. 2017). It was also stated that in 2017 the peak stage at all of the monitoring stations was elevated above bankfull by snow and ice in the channel and that the peak stage probably did not occur at the same time as the peak discharge (Michael Baker Jr. Inc. 2017). At UC2C the peak stage in 2017 was 99.88 feet (arbitrary datum) and occurred on May 30 (Michael Baker Jr. Inc. 2017). In 2018, the peak stage at UC2C was 54.78 feet NAVD88 and occurred on June 13 (Michael Baker Jr. Inc. 2018). In 2018, the peak stage was believed to have occurred at the same time as the peak discharge (Michael Baker Jr. Inc. 2018). At the time of the peak stage, “overbank flooding and minimal impedance from snow” was reported (Michael Baker Jr. Inc. 2018). However, since an observer could probably not have seen through 13-plus feet of water (Figure E.8.7), it seems unknown whether or not the peak stage and/or the stage at the peak discharge were impacted by snow and ice in the bottom of the channel. No estimate for the 2018 peak discharge was provided (Michael Baker Jr. Inc. 2018). Bankfull conditions with some overbank flooding in low-lying areas persisted through at least June 18.

Figure E.8.7 presents a surveyed cross-section at UC2C and a cross-section taken during a spring breakup discharge measurement (Michael Baker Jr. Inc. 2018). The difference between the cross-sections, and the difference between the WSE’s on June 11 and 13, represents the impact of snow and ice in the channel on the WSE.

In both 2017 and 2018, the summer stage fluctuated with precipitation, but water levels remained below the spring breakup peak stage. The maximum and minimum stages recorded at UC2C during summer 2017 were 93.1 feet and 90.85 feet, respectively (both based on an arbitrary datum). The maximum and minimum stages recorded at UC2C during the summer of 2018 were 47.81 feet and 46.45 feet NAVD88, respectively. In both years, the stage increased in the beginning of September as a result of precipitation events.



Source: Michael Baker Jr. Inc. 2018

Figure E.8.7. Cross-Section of Judy (Kayyaaq) Creek at Gaging Station UC2C

1.2.2.2.2 Willow Creek 1

Willow Creek 1 is a tributary of Judy (Iqalliqpik) Creek. Alternatives B (Proponent's Proposal) and C (Disconnected Infield Roads) would cross Willow Creek 1 between Lake R0060 and Lake M0016, which is also where the W1S monitoring site is located in a poorly defined, low-lying area (Michael Baker Jr. Inc. 2018).

The 2018 spring breakup peak stage at W1S was 79.16 feet NAVD88 and occurred on June 6 (Michael Baker Jr. Inc. 2018). The 2019 spring breakup peak stage was 79.25 feet NAVD88 and occurred on May 28 (Michael Baker International 2020a). Throughout the entire breakup monitoring periods for both 2019 and 2020, no distinguishable channel or discernible flow was identified near W1S, and the peak stage was probably the result of ponded local melt (Michael Baker International 2020a; Michael Baker Jr. Inc. 2018). During the summer, small stage fluctuations associated with summer precipitation were recorded, but water levels remained below the spring breakup peak stage (Michael Baker Jr. Inc. 2018). The 2018 maximum and minimum summer stages at W1S were 78.59 feet NAVD88 and 78.39 feet NAVD88, respectively (Michael Baker Jr. Inc. 2018). During summer 2018, no defined channel or flow was observed, only standing water (Michael Baker Jr. Inc. 2018).

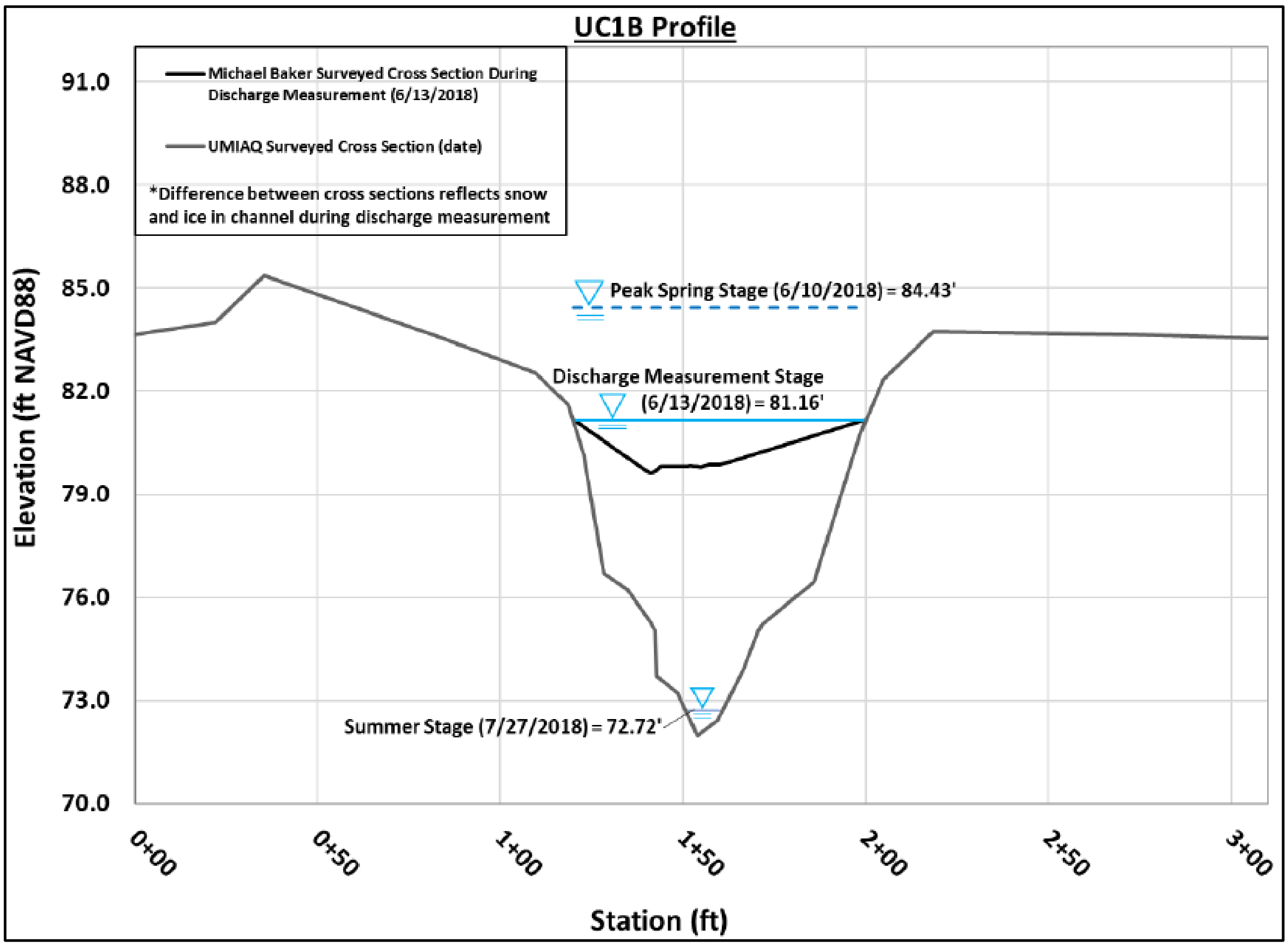
1.2.2.2.3 Willow Creek 2

Willow Creek 2 is a tributary of Judy (Iqalliqpik) Creek. Willow Creek 2 has a highly sinuous, deeply incised, beaded channel (Michael Baker Jr. Inc. 2018). It is over 10 feet from the top of the bank to the streambed and has a typical channel width of 20 feet (Michael Baker Jr. Inc. 2017). Alternatives B (Proponent's Proposal) and C (Disconnected Infield Roads) would cross Willow Creek 2 at RM 4.5, and the UC1B monitoring site is located on Willow Creek 2 at the proposed crossing (Michael Baker Jr. Inc. 2018). At RM 4.5, the bankfull width is approximately 4.5 feet and the average bankfull depth is approximately 2.5 feet (CPAI 2018b). Spring breakup and summer stage were monitored at UC1B in 2017, 2018, and 2019.

In 2017, 2018, and 2019, the channel was full of wind-blown snow prior to the start of breakup (Michael Baker International 2020a; Michael Baker Jr. Inc. 2017, 2018). In all 3 years, it was reported that water began flowing on top of the drifted snow and then cut a channel down through the wind-blown snow (Michael Baker International 2020a; Michael Baker Jr. Inc. 2017, 2018). In all 3 years, peak stage was reportedly affected by snow and ice in the channel, and peak stage did not coincide with the peak discharge (Michael Baker International 2020a; Michael Baker Jr. Inc. 2017, 2018). In 2017, the peak stage at UC1B occurred on May 30 at 96.87 feet (arbitrary datum) (Michael Baker Jr. Inc. 2017). In 2018, the peak stage at UC1B occurred on June 10 at 84.42 feet NAVD88 (Michael Baker Jr. Inc. 2018). A spring peak discharge was not recorded in either year. In 2019, the peak stage at CU1B occurred on May 26. The measured discharge on June 1 was 110 cfs (Michael Baker International 2020a).

Figure E.8.8 presents a surveyed cross-section at UC1B and a cross-section taken during a spring breakup discharge measurement (Michael Baker Jr. Inc. 2018). The difference between the cross-sections, and the difference between the WSEs on June 11 and 13, represents the impact of snow and ice in the channel on the WSE.

In all 3 years, the summer stage fluctuated with precipitation, but water levels remained below the spring breakup peak stage. The maximum and minimum stages recorded at UC1B during summer 2017 were 84.63 feet and 83.01 feet, respectively (both based on an arbitrary datum) (Michael Baker Jr. Inc. 2017). The maximum and minimum stages recorded at UC1B during summer 2018 were 74.43 feet and 72.72 feet NAVD88, respectively (Michael Baker Jr. Inc. 2018). The maximum and minimum stages recorded at UC1B during summer 2019 were 75.2 feet and 72.83 feet NAVD88, respectively (Michael Baker International 2020a).



Source: Michael Baker Jr. Inc. 2018

Figure E.8.8. Cross-Section of Willow Creek 2 at Monitoring Site UC1B

1.2.2.2.4 Willow Creek 3

Willow Creek 3 is a tributary of Judy (Iqalliqpik) Creek. The infield road for all action alternatives would cross Willow Creek 3 between Lake M0015 and Lake R0055, which is also where the W3S monitoring site is located in a poorly defined, low-lying area (Michael Baker Jr. Inc. 2018). At W3S, the bankfull width is approximately 18 feet and the average bankfull depth is approximately 2.0 feet (CPAI 2018b). The Willow Creek 3 basin is also where the constructed freshwater reservoir would be located for all action alternatives. The constructed freshwater reservoir would divert water from Lake M0015.

The 2018 spring breakup peak stage at W3S was 84.13 feet NAVD88 and occurred on June 4 (Michael Baker Jr. Inc. 2018). The peak stage was affected by ice and snow but may have been the result of pooled local melt rather than flowing water (Michael Baker Jr. Inc. 2018). Eight days later (stage about 83.65 feet NAVD88), areas inundated by snowmelt and low-velocity flow were observed (Michael Baker Jr. Inc. 2018). During summer, small stage fluctuations associated with summer precipitation were recorded, but water levels remained below the spring breakup peak stage (Michael Baker Jr. Inc. 2018). The maximum and minimum summer stages at W3S were 83.40 feet and 82.86 feet NAVD88, respectively (Michael Baker Jr. Inc. 2018). Low-velocity flow through a poorly defined, ephemeral channel was observed on July 9 (Michael Baker Jr. Inc. 2018).

The 2019 spring breakup peak stage at WS3 was 88.49 feet NAVD88 and occurred on June 2 (Michael Baker International 2020a). Aerial observations at the time showed widespread meltwater and saturated snow across the Willow Creek 3 drainage, with no defined drainage channel (Michael Baker International 2020a). Discharge during spring breakup was measured twice. The May 30 discharge measurement of 5 cfs was classified as poor based on the influence of ice and snow in the channel. The June 2 discharge measurement of 16 cfs was classified as fair after water had receded from the peak stage and multiple flow paths had been established in the snow (Michael Baker International 2020a). During summer, water levels remained below the spring breakup stage and minimal stage fluctuations with summer precipitation events were recorded. The maximum and minimum summer stages at W3S were 87.78 feet and 87.24 feet NAVD88, respectively (Michael Baker International 2020a).

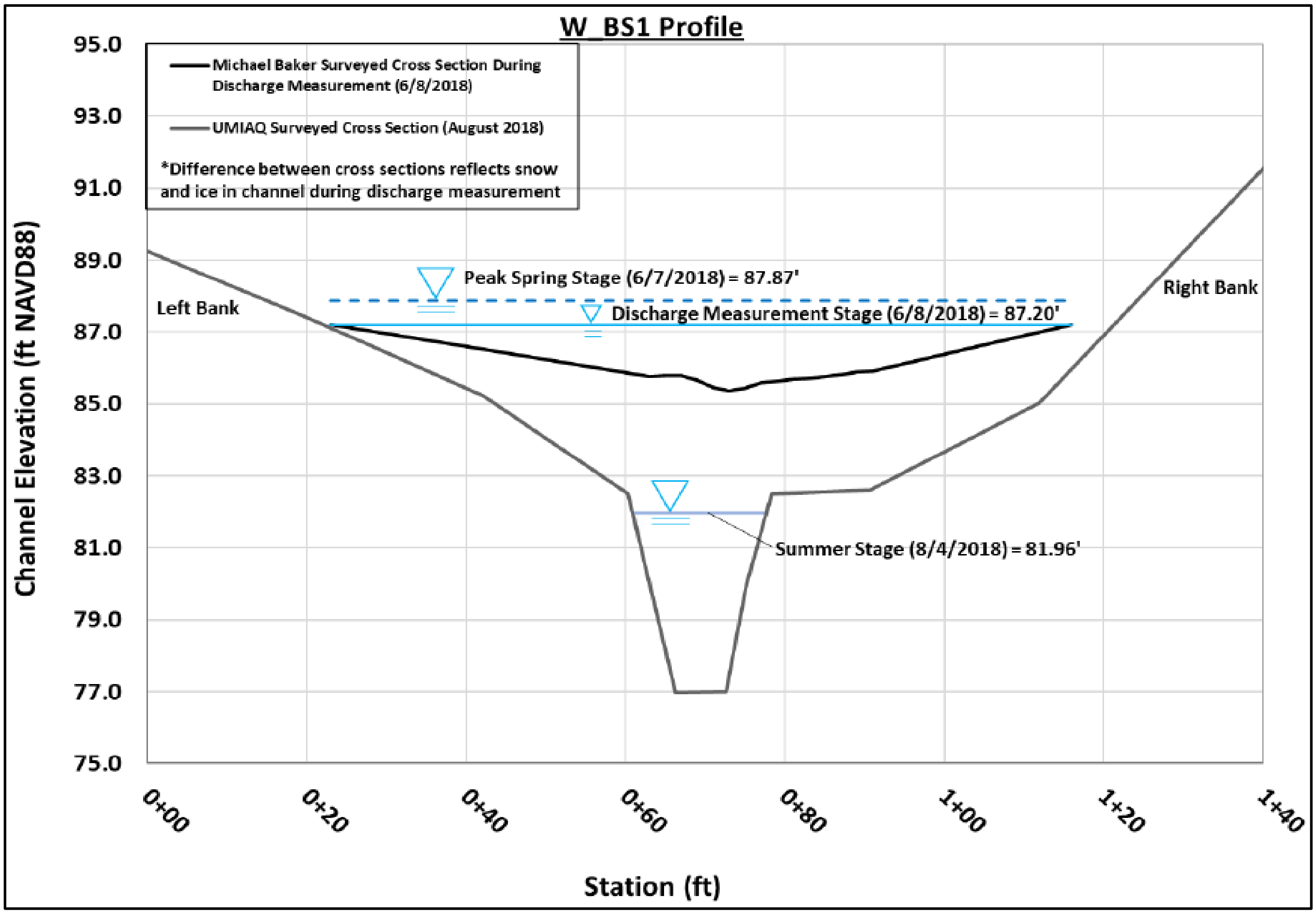
1.2.2.2.5 Willow Creek 4

Willow Creek 4 is a tributary of Judy (Iqalliqpik) Creek. It has an incised channel with intermittent, deep, beaded pools (Michael Baker Jr. Inc. 2018). The infield road for all action alternatives would cross Willow Creek 4 at RM 9, which is also the location of the W_BS1 monitoring site. At RM 9, the bankfull width is approximately 26 feet and the average bankfull depth is approximately 2.7 feet (CPAI 2018b). The W4 monitoring site is located at RM 5.2, adjacent to the Bear Tooth drill site 3/Willow Processing Facility pad.

The 2018 spring breakup peak stage at W_BS1 was 87.87 feet NAVD88 and occurred on June 7 (Michael Baker Jr. Inc. 2018). The 2018 spring breakup peak stage at W4 was 96.38 feet (arbitrary datum) and also occurred on June 7 (Michael Baker Jr. Inc. 2018). Both peaks occurred after a short, rapid rise in the WSE of 1.5 to 2 feet, and snow and ice within the channel affected the peak WSE at both sites. The timing and magnitude of the peak discharge were not recorded.

The 2019 spring breakup peak stage at W_BS1 was 87.38 feet NAVD88 and occurred on May 26 (Michael Baker International 2020a). The 2019 spring breakup peak stage at W4 was 94.21 feet (arbitrary datum) and occurred on May 26. The upstream gage, W_BS1, recorded the peak stage about 3 hours prior to the peak stage at the downstream gage, W4 (Michael Baker International 2020a).

Figure E.8.9 presents a surveyed cross-section at W_BS1 and a cross-section taken during a spring breakup discharge measurement (Michael Baker Jr. Inc. 2018). The difference between the cross-sections, and the difference between the WSE's on June 11 and 13, represents the impact of snow and ice in the channel on the WSE.



Source: Michael Baker Jr. Inc. 2018

Figure E.8.9. Cross-Section of Willow Creek 4 at Monitoring Site W_BS1

During the summers of both 2018 and 2019, the stage fluctuated with summer precipitation at both monitoring sites, but the water levels remained well below the spring breakup peak stage (Michael Baker International 2020a; Michael Baker Jr. Inc. 2018). The stage at the end of the summer monitoring season for both years increased due to late summer precipitation. The maximum and minimum summer stages at W4 for 2018 were 87.96 feet (arbitrary datum) and 85.11 feet (arbitrary datum), respectively (Michael Baker Jr. Inc. 2018), and for 2019 were 86.47 feet and 84.99 feet (arbitrary datum), respectively (Michael Baker International 2020a). The maximum and minimum summer stages at W_BS1 for 2018 were 83.79 feet and 81.96 feet NAVD88, respectively (Michael Baker Jr. Inc. 2018), and for 2019 were 85.46 feet and 82.29 feet (arbitrary datum), respectively (Michael Baker International 2020a).

1.2.2.2.6 Willow Creek 4A

Willow Creek 4A is a tributary of Willow Creek 4. The infield road for all action alternatives would cross Willow Creek 4A at MBI Monitoring Site W_S1, established in 2018. The channel near W_S1 is beaded and has defined banks. It has a bankfull width of approximately 24 feet and an average bankfull depth of approximately 4.5 feet (CPAI 2018b).

The 2018 spring breakup peak stage at W_S1 was 101.93 feet NAVD88 and occurred on June 8 (Michael Baker Jr. Inc. 2018). It was affected by snow and ice in the channel (Michael Baker Jr. Inc. 2018). At the time of the peak stage, the meltwater was confined by saturated snow, and the stage rose 1.5 feet in about 3 hours (Michael Baker Jr. Inc. 2018). The timing and magnitude of the peak discharge were not recorded.

In general, the stage fell throughout the summer except for fluctuations due to summer precipitation events (Michael Baker Jr. Inc. 2018). At the end of the summer monitoring season, the stage increased due to a late summer precipitation event (Michael Baker Jr. Inc. 2018). However, the stage remained well below the spring breakup peak stage throughout the summer (Michael Baker Jr. Inc. 2018). The maximum and minimum summer stages at W_S1 were 98.67 feet and 98.22 feet NAVD88, respectively (Michael Baker Jr. Inc. 2018).

The 2019 spring breakup peak stage at W_S1 was 101.89 feet NAVD88 on May 27 (Michael Baker International 2020a). Minor overbank flooding was noted in low-lying areas and adjacent polygon troughs, with stranded ice above the reach of the bank.

Summer stage levels fell except for fluctuations due to summer precipitation events. The stage increased to a maximum level of 99.68 feet on August 29 due to a notable precipitation event and the minimum stage was 98.77 feet on July 18 (Michael Baker International 2020a).

1.2.2.3 Ublutuoch (Tiŋmiaqsiuġvik) River

The Ublutuoch (Tiŋmiaqsiuġvik) River has its entire drainage basin on the Arctic Coastal Plain and flows into Fish (Iqalliqpik) Creek at RM 10. It has a drainage area of approximately 248 square miles, of which approximately 15% is covered by lakes (URS Corporation 2003). Two gravel mine site options are located in the Ublutuoch (Tiŋmiaqsiuġvik) River drainage basin, one on each side of the Ublutuoch (Tiŋmiaqsiuġvik) River. The downstream boundary of the gravel mine site analysis area would cross the Ublutuoch (Tiŋmiaqsiuġvik) River at approximately RM 13.9.

Spring breakup stage and discharge have been measured on the main stem of the Ublutuoch (Tiŋmiaqsiuġvik) River for 17 years at RM 13.7, about 0.2 RM downstream from the downstream boundary of the gravel mine site study area (Table E.8.14). During that time, water began to flow between May 17 and June 8, with a median date of May 30. The annual peak discharge occurred between May 19 and June 9, with a median date of June 5. In 9 out of 17 years the peak stage occurred earlier and was higher than the stage at the time of the peak discharge. The largest difference was 1.82 feet in 2005. The time from the beginning of flow to the peak discharge varied between 1 and 7 days, with a median time of 3 days. The annual peak discharge varied from 55 to 3,200 cfs, with a median of 1,700 cfs. Freeze-up data were collected in 7 of the 17 years. During that time, freeze-up occurred between September 26 and October 21, with a median date of October 8.

The Ublutuoch (Tiŋmiaqsiuġvik) River has a relatively low gradient and highly sinuous channel. In the vicinity of RM 13.7 the channel is incised within relatively steep upper banks that are vegetated with dense brush (URS Corporation 2003). The lower portion of the channel consists of a relatively flat bench located approximately 10 to 15 feet below the top of the upper banks (URS Corporation 2003). A 2- to 3-foot-deep × 15- to 20-foot-wide low-water channel is located in the bottom of the otherwise vegetated channel (URS Corporation 2003). The riverbed is composed of sand and gravel, with a median diameter of 7.0 mm (URS Corporation 2003).

At the time of the 2001 and 2002 spring peak WSE and discharge, the water was flowing on snow within the channel. A comparison of riverbed elevation on various dates during the 2002 breakup at RM 13.7 is shown in Figure E.8.10, and 2001 and 2002 riverbed elevations at the time of the peak discharge are presented in Figure E.8.11.

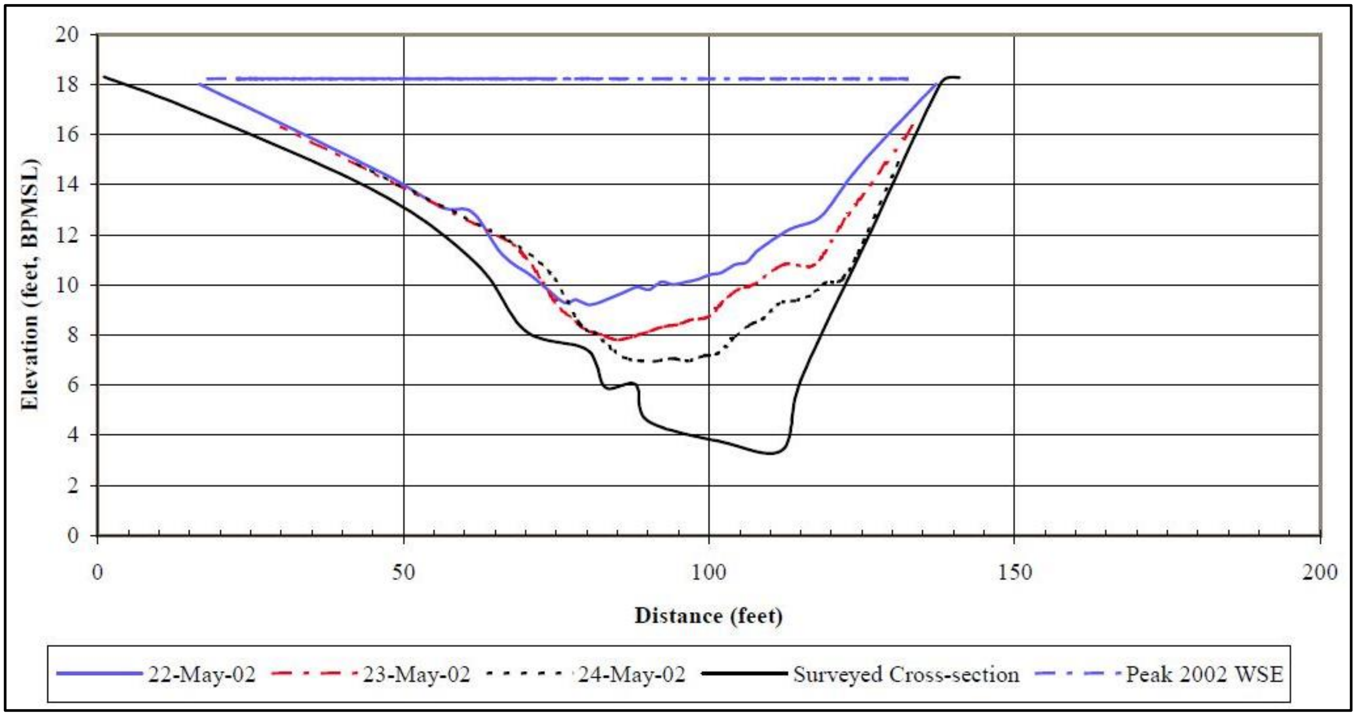
Table E.8.14. Summary of Annual Peak Stage and Discharge for the Ublutuoch (Tiŋmiaqsiuġvik) River at River Mile 13.7

Year	Date Flow Begins (m/d)	Date of Freeze-Up (m/d)	Annual Peak Stage Date (m/d)	Annual Peak Stage (ft)	Annual Peak Stage Discharge (cfs)	Annual Peak Discharge Date (m/d)	Annual Peak Discharge Stage (ft)	Annual Peak Discharge (cfs)	Zero Flow to Peak Q (days)
2001	6/8	N/A	6/9	18.09	N/A	6/9	18.09	2,200	1
2002	5/19 e	N/A	5/22	18.22	N/A	5/22	18.22	2,000	3
2003	6/5	N/A	6/6	19.30	N/A	6/7	18.34	1,600	2
2004	6/1	N/A	6/5	19.55	N/A	6/5	19.55	2,400	4
2005	6/5	N/A	6/6	19.23	N/A	6/9	17.41	1,520	4
2006	6/1 e	N/A	6/4	16.67	N/A	6/6	15.04	1,250	5
2007	6/3	N/A	6/5	17.35	N/A	6/5	16.84	1,520	2
2008	5/27	N/A	5/29	17.42	N/A	5/29	16.85	955	2
2009	5/25	10/8	5/28	18.90	N/A	5/28	18.34	1,700	3
2010	6/5	9/27	6/7	19.68	N/A	6/7	19.68	3,200	2
2011	5/30	N/A	6/1	19.17	N/A	6/3	17.91	1,960	4
2012	5/30	10/11	6/5	18.33	N/A	6/5	18.33	2,130	6
2013	6/2	10/4	6/5	19.29	N/A	6/9	18.47	2,440	7
2014	5/17	10/11	5/19	18.61	N/A	5/19	18.61	1,270	2

Year	Date Flow Begins (m/d)	Date of Freeze-Up (m/d)	Annual Peak Stage Date (m/d)	Annual Peak Stage (ft)	Annual Peak Stage Discharge (cfs)	Annual Peak Discharge Date (m/d)	Annual Peak Discharge Stage (ft)	Annual Peak Discharge (cfs)	Zero Flow to Peak Q (days)
2015	5/20	9/26	5/22	19.91	N/A	5/23	19.26	2,440	3
2016	5/22	10/21	5/24	17.76	N/A	5/24	17.76	1,150	2
2017	5/28	N/A	5/31	16.69	N/A	5/31	16.69	1,380	3

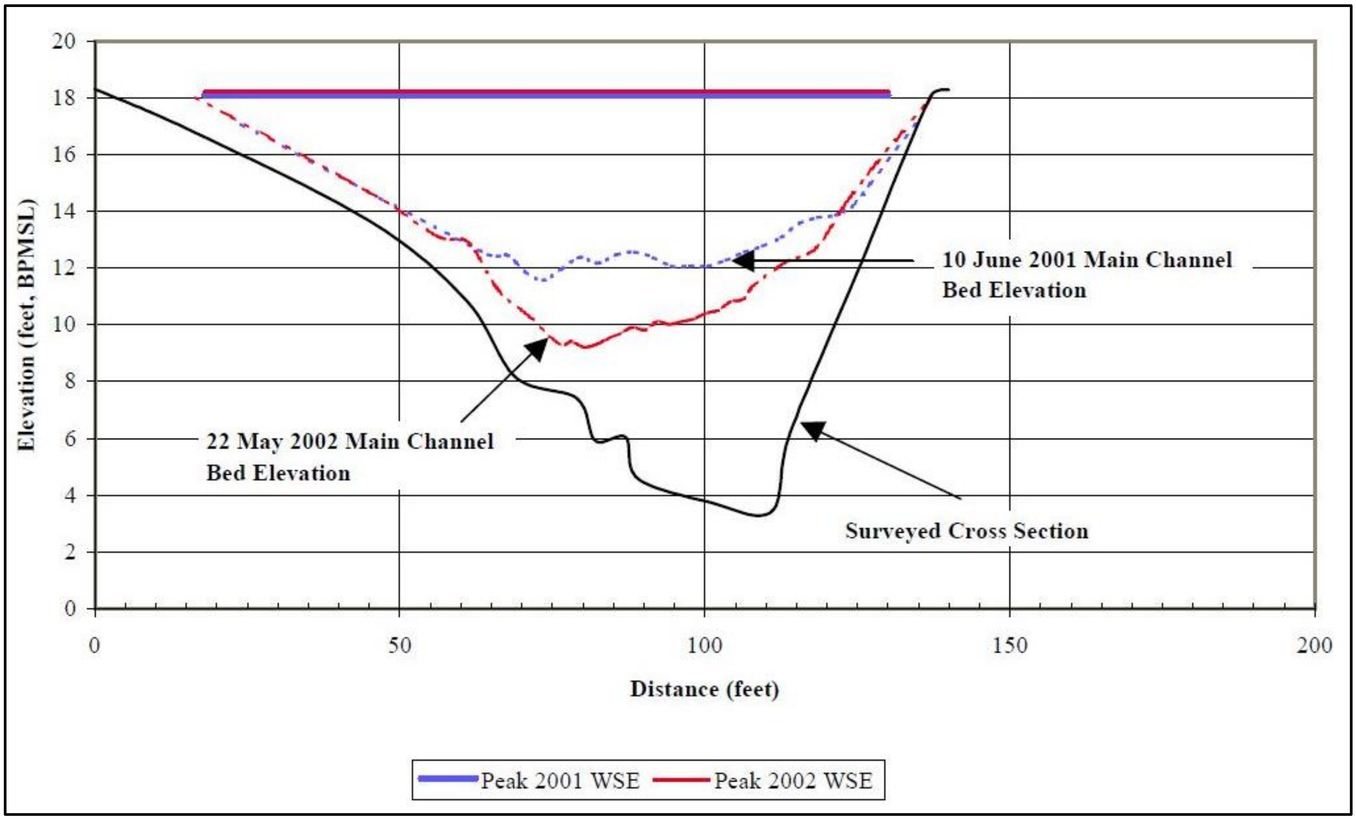
Source: J. Aldrich (Arctic Hydrologic Consultants), personal communication to Richard Kemnitz (BLM). September 11, 2018

Note: cfs (cubic feet per second); d (day); e (estimate); ft (feet); m (month); N/A (not available); Q (discharge); RM (river mile). The coordinates of the site (NAD83): 70.24316, -151.29693.



Source: URS Corporation 2003

Figure E.8.10. Effect of Snow and Ice in 2002 on Channel Cross-Section at River Mile 13.7



Source: URS Corporation 2003

Figure E.8.11. Comparison of 2001 and 2002 Cross-Sections at Peak Discharge at River Mile 13.7

Discharge and water surface slope measurements, along with surveyed cross-sections and a water surface profile model, were used to estimate hydraulic roughness in the channel on a particular day during the 2002 spring breakup. At RM 8 and RM 13.7, the channel hydraulic roughness on the day of the measurements, when ice and snow were impacting the hydraulic conditions, was 0.012 and 0.021, respectively (URS Corporation 2003). Computations of hydraulic roughness based on measured discharge and water surface slope and normal depth computations at RM 13.7 on each of 3 days in 2001 and 2002 during ice- and snow-impacted conditions varied from 0.019 to 0.025, with a median of 0.023 (URS Corporation 2001, 2003).

Seventeen years of summer flow data is available for the Ublutuoch (Tiŋmiaqsiuġvik) River at RM 13.7 (J. Aldrich [Arctic Hydrologic Consultants], personal communication to Richard Kemnitz [BLM]. September 11, 2018) . A summary of the available mean monthly discharge data is provided in Table E.8.15.

Table E.8.15. Mean Monthly Discharge (cubic feet per second) in the Ublutuoch (Tiŋmiaqsiuġvik) River at River Mile 13.7

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2001	0	0	0	0	0	435	47	45	38	27	16	5
2002	0	0	0	0	377	133	80	24	24	17	10	3
2003	0	0	0	0	0	389	112	57	52	6	0.5	0
2004	0	0	0	0	0	827	69	21	32	6	0.3	0
2005	0	0	0	0	0	467	78	13	7	2	0	0
2006	0	0	0	0	0	434	36	25	16	9	1	0
2007	0	0	0	0	0	283	18	2	0.5	0	0	0
2008	0	0	0	0	101	223	15	7	3	0.6	0	0
2009	0	0	0	0	241	456	27	12	31	15	4	0.6
2010	0	0	0	0	0	596	54	54	25	7	0.5	0
2011	0	0	0	0	11	628	33	10	12	7	0.8	0
2012	0	0	0	0	0.2	535	37	10	12	9	5	0.3
2013	0	0	0	0	0	857	72	26	30	8	2	0.1
2014	0	0	0	0	359	441	84	25	38	38	6	0.6
2015	0	0	0	0	438	208	18	14	16	2	0.2	0
2016	0	0	0	0	184	181	24	22	91	87	10	3

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2017	0	0	0	0	92	367	18	78	200	150	23	0.1

Source: J. Aldrich (Arctic Hydrologic Consultants), personal communication to Richard Kemnitz (BLM). September 11, 2018

At RM 14.5 (MBI Monitoring Site UB14.5) and RM 15.5 (MBI Monitoring Site UB15.5), the spring breakup stage and the extent of flooding was monitored in 2018 and 2019 (Michael Baker International 2020a; Michael Baker Jr. Inc. 2018). RM 14.5 is just downstream of the mouth of Bill's Creek, and RM 15.5 is just upstream. MBI (2018) also monitored the stage and extent of flooding on Bill's Creek, at Monitoring Site BC1. All of these sites are within the gravel mine site analysis area.

At UB14.5, the channel is incised and deep and fills with wind-blown snow during winter (Michael Baker Jr. Inc. 2018). During the 2018 spring breakup, the peak stage was 20.20 feet (adjusted for NAVD88 in 2020) and occurred on June 9. Pictures of the monitoring site on the day of the peak stage suggest that the peak stage was affected by snow and ice. During the 2019 spring breakup, the peak stage was 19.23 feet NAVD88 and occurred on May 29 (Michael Baker International 2020a).

At UB15.5, the channel is incised and deep and fills with wind-blown snow during the winter (Michael Baker Jr. Inc. 2018). During the 2018 spring breakup, the peak stage was 23.49 feet (adjusted for NAVD88 in 2020) and occurred on June 8. Pictures of the monitoring site on the day of the peak stage suggest that the peak stage was affected by snow and ice. During the 2019 spring breakup, the peak stage was 22.46 feet NAVD88 and occurred on May 26 (Michael Baker International 2020a).

Bill's Creek is a beaded channel consisting of large beads connected by deeply incised, narrow grass-lined channels with its headwaters in an area of small lakes (Michael Baker Jr. Inc. 2018). Wind-blown snow fills much of the drainage during the winter (Michael Baker Jr. Inc. 2018). During the 2018 spring breakup, the peak stage at BC1 was 41.85 feet (adjusted to NAVD88) and occurred on June 11. Based on the description of the conditions at the time of the peak stage (Michael Baker Jr. Inc. 2018), the peak stage was affected by snow and ice in the channel. The summer stage fluctuated with precipitation events but remained below the peak breakup stage (Michael Baker Jr. Inc. 2018). The stage increased at the end of the summer monitoring period as a result of late summer precipitation (Michael Baker Jr. Inc. 2018). The maximum and minimum summer stages were 88.67 feet and 87.01 feet (arbitrary datum), respectively (Michael Baker Jr. Inc. 2018).

During the 2019 spring breakup, the peak stage at BC1 was 39.78 feet NAVD88 and occurred on May 23. The peak stage was affected by snow and ice in the channel (Michael Baker International 2020a).

Spring breakup observations have also been made at the following sites:

- RM 6.8 in 2003, 2004, 2005, 2006, 2009, 2010, 2011, and 2013 (CPAI 2018a)
- RM 8.0 in 2002 (URS Corporation 2003)
- RM 13.5 in 2001 (URS Corporation 2001) and 2002 (URS Corporation 2003)

Hydraulic designs on the Ublutuoch (Tiŋmiaqsiuġvik) River should consider the flood-peak data that have been collected at RM 13.7, the impact of snow and ice at the time of the annual peak discharge, the impact of snow and ice on the annual peak WSE, and the hydraulic roughness likely to be present at the time of the design discharge. In developing flood-peak magnitude and frequency estimates on streams in the Ublutuoch (Tiŋmiaqsiuġvik) River basin, the 17 years of data collected at RM 13.7 should be considered. A single-station flood-peak magnitude and frequency analyses could be conducted with these data to estimate the flood-peak magnitude and frequencies at RM 13.7. A best estimate of the flood-peak magnitude and frequency at RM 13.7 could then be developed from a weighted average, based on the uncertainty associated with estimates from each of two methods: the single-station frequency analysis and the Shell regression equations[3] (Arctic Hydrological Consultants and ERM 2015). The weighted average estimate would then be extrapolated to other locations within the basin as a proportion of the Shell regression equation estimate.

Since the hydraulic roughness is changing throughout spring breakup, when designing structures on this river it would be prudent to consider a range of hydraulic roughness values. Higher hydraulic roughness values will provide estimates with higher WSEs and lower velocities. Lower hydraulic roughness values will provide estimates with lower WSEs and higher velocities. Both conditions are important when designing structures within

[3] The Shell regression equations are suggested rather than the 2003 USGS regression equations because considerably more North Slope river data was used to prepare the Shell regression equations than the USGS regression equations.

the channel and the floodplain. Additionally, snow blockage at the time of the peak discharge seems to be an annual occurrence and should be considered when estimating design WSEs.

1.2.3 Kalikpik River

The Kalikpik River originates in a complex network of lakes, approximately 15 miles south of Teshekpuk Lake, and flows into Harrison Bay northwest of Fish (Iqalliqpik) Creek (Michael Baker Jr. Inc. 2018). The river has a relatively low gradient, a highly sinuous channel, and the channel bed and banks consist predominantly of silt and sand (Michael Baker Jr. Inc. 2018). The most downstream end of the proposed infrastructure comes close to the Kalikpik River, about 17.5 RMs upstream from the coast (RM 17.5).

In 2018 and 2019, the stage was monitored during spring breakup at Kal 1 (Michael Baker Jr. Inc. 2018), about 21.8 RMs upstream from the coast. In 2018, the channel was full of windblown snow prior to the start of breakup (Michael Baker Jr. Inc. 2018). The peak stage occurred on June 11 at an elevation of 50.30 feet NAVD88 and was affected by snow and ice conditions (Michael Baker Jr. Inc. 2018). Snow remained along the banks and large ice floes were present in the channel for a couple of days following the peak stage (Michael Baker Jr. Inc. 2018). A second, smaller rise in the stage was observed on June 16 and may have been coincident with the peak discharge (Michael Baker Jr. Inc. 2018). A discharge of 320 cfs was measured at a stage of 48.18 feet NAVD88 on June 16 at 4:00 p.m. The stage was just below bankfull (Michael Baker Jr. Inc. 2018). No ice or snow was observed in the channel, but saturated snow remained along the south bank just above the water surface (Michael Baker Jr. Inc. 2018).

In 2019, the peak stage of 49.44 feet NAVD88 occurred on May 26, and was likely elevated by large quantities of saturated snow and bottom-fast channel ice (Michael Baker International 2020a). A discharge of 245 cfs was measured at a stage of 48.94 feet NAVD88 on May 30 (Michael Baker International 2020a).

For 2018 and 2019, MBI continued to monitor the stage during summers. The stage fluctuated throughout summer as a result of precipitation events but remained below the spring breakup peak stage (Michael Baker Jr. Inc. 2018). For both summers, later summer precipitation events led to increased stage levels that were slightly higher than the stage during the discharge measurement near the end of the summer monitoring period (Michael Baker International 2020a; Michael Baker Jr. Inc. 2018). The highest summer stage levels were 47.10 feet in 2018 and 47.91 feet in 2019 (Michael Baker International 2020a; Michael Baker Jr. Inc. 2018).

At Kal 1, the bankfull width is approximately 140 feet, the average bankfull depth is approximately 3 feet, and the thalweg depth is approximately 8 feet (CPAI 2018b).

1.3 Environmental Consequences

1.3.1 In-Water Structures

1.3.1.1 Bridge Crossings

The potential impacts to streams crossed by bridges during the life of the structure include the following:

- Increased backwater on the upstream side of the bridge
- Increased riverbed erosion within the bridge opening
- Increased riverbed and bank erosion downstream from the bridge
- Increased sediment deposition downstream from the bridge
- Increased sediment transport within and downstream from the bridge
- A change in channel morphology downstream from the bridge

The impact of a bridge on the stream being crossed is directly related to the criteria used to design the bridge and the extent to which the bridge is constructed according to the design. Some of the most important factors related to the hydraulic design of bridges on the North Slope include 1) the frequency of the design event in relation to the anticipated life of the structure; 2) the reliability of the computed magnitude and frequency of the design event; 3) the impact of snow and ice (including ice floes) at the time of the design event and during events with a smaller discharge than the design event; and 4) the reliability of the hydraulic computations used to estimate WSE and velocity, riverbed scour, and bank erosion. With regard to the frequency of the design event, the probability that the design event will not be exceeded during the life of the structure should be considered.

All bridges would be designed to maintain bottom chord clearance of 4 feet above the 100-year base flood elevation and at least 3 feet above the highest documented flood elevation. Table E.8.16 presents the relationship between the average return period of the design event and the probability that the design event will not be exceeded during various lengths of time. Note that the probability that the design event will not be exceeded decreases as the life of the structure increases. Based on the life of past structures on the North Slope, it seems very likely that the life of the structures could be greater than 40 or 50 years. A culvert or bridge based on a 100-year flood design that is likely to be in place for 50 years before removal or replacement would have a 61% chance that the design flood would not be exceeded one or more times during the life of the structure (i.e., 39% chance that design flood would be exceeded). As shown, although it is more likely that the design life will not be exceeded during the life of the Project, there is still a 39% chance it could be. This section describes the potential effects of bridges.

Table E.8.16. Theoretical Probability That the Design Event Will Not Be Exceeded in a Specified Number of Years

Design Event (average return period in years)	10 years	20 years	30 years	40 year	50 years	60 years	70 years
25	66%	44%	29%	20%	13%	8%	6%
50	82%	67%	55%	45%	36%	30%	24%
100	**90%**	**82%**	**74%**	**67%**	**61%**	**55%**	**49%**
200	95%	90%	86%	82%	78%	74%	70%
500	98%	96%	94%	92%	90%	89%	87%

Note: **Bold** denotes the design life of bridges for the Project. The difference between the theoretical probability and the actual probability is the accuracy of the design events' predicted probability of occurrence. For instance, if the design discharge is supposed to be a 100-year event but actually has an average return period of 90 years, the theoretical probability that the design event will not be exceeded will be higher than what is experienced.

During floods in which the cross-sectional area of the flow is restricted by the bridge, water would back up behind the bridge. The difference between the unrestricted WSE and the restricted WSE on the upstream side of the bridge is called backwater. The magnitude of the backwater would depend upon the amount of constriction presented by bridge or road embankments and would usually become larger with larger flood events.
The maximum increase in WSE generally occurs at a location upstream from the bridge, about equal in distance to about one-half the total length of the embankment obstructing the flow of water. The upstream extent of the backwater is a function of both the magnitude of the constriction and the slope of the stream. The duration of the backwater would be somewhat less than the duration of the flood. Backwater is generally a concern if it causes a structure (such as an upstream pipeline) or another resource to be damaged by the inundation created as a result of the backwater.

The more a bridge restricts the flow (i.e., the greater the backwater), the higher the velocity through the bridge. At a particular discharge, if the velocity through the bridge exceeds the velocity that would have occurred prior to construction of the bridge, and the bed material is mobile at that velocity, it is likely that the depth of the scour would be greater than would have occurred prior to bridge construction. Similarly, if the velocity downstream from the bridge is greater than the velocity that would have occurred prior to bridge construction, it is possible that bank erosion would be more severe than would have occurred. With increased erosion comes increased sediment transport and increased sediment deposition. An increase in erosion and deposition can lead to a change in channel morphology. If the bridge abutments or pier piles are undermined by scour, the bridge may collapse. Scour is historically one of the most common causes of bridge failure in North America (Cook 2014). However, scour is not a problem if it is correctly addressed during the design of the bridge.

1.3.1.2 Culverts

The potential impacts to streams crossed by culverts during the life of the structure include the following:

- Increased backwater on the upstream side of the culvert
- Increased riverbed and bank erosion downstream from the culvert
- Increased sediment deposition downstream from the culvert
- Increased sediment transport downstream from the culvert
- A change in channel morphology downstream from the culvert

The impact of the culvert on the stream being crossed is directly related to the criteria used to design the culvert and the extent to which the culvert is constructed according to the design. The size, layout, and quantity of Project culverts would be based on site-specific conditions in order to pass the 50-year flood event with a headwater elevation not exceeding the top of the culvert (headwater to diameter ratio of 1 or less). Some of the most important factors related to the hydraulic design of culverts on the North Slope include 1) the frequency of the design event in relation to the anticipated life of the structure; 2) the reliability of the computed magnitude and frequency of the design event; 3) the impact of snow and ice (including ice floes) at the time of the design event and during events with a smaller discharge than the design event; 4) the reliability of the hydraulic computations used to estimate WSE and velocity, riverbed scour, and bank erosion; and 5) the reliability of the topographic and flow information used to located the culvert. With regard to the frequency of the design event, see the discussion in Section 2.5.3.2.1, *Bridges*. A culvert based on a 50-year flood design that is likely to be in place for 50 years before removal or replacement would have a 36% chance that the design flood would not be exceeded one or more times during the life of the structure (i.e., 64% chance that design flood would be exceeded).

During floods in which the cross-sectional area of the flow is restricted by the culvert, water would back up behind the culvert. The magnitude of the backwater would depend upon the amount of constriction presented by the culvert. See discussion in Section 2.5.3.2.1 for additional information.

The more the culvert restricts streamflow (i.e., the greater the backwater), the higher the velocity through the culvert. The higher the velocity through the culvert, the more likely it is that riverbed erosion (scour) and bank erosion would occur at the culvert outlet and downstream from the culvert. With increased erosion comes increased sediment transport and increased sediment deposition. An increase in erosion and deposition can lead to a change in channel morphology.

1.3.2 Pipelines

All of the pipeline waterbody crossings would be aboveground on vertical support members (VSMs) except for the Colville River crossing, which would be installed 70 feet below the river channel using horizontal directional drilling (HDD).

1.3.2.1 Aboveground Crossings

As water passes around VSMs, at an aboveground crossing there is the potential for an increase in velocity and scour. This may result in erosion at the VSM and sediment deposition downstream from the VSM. If ice floes or debris build up on a VSM, the scour at the VSM could be greater than anticipated and could compromise the integrity of the VSM and thus the pipeline.

If water, floating ice, or debris comes in contact with the aboveground pipeline, the pipeline could be ruptured. It is unknown to what flood event or ice condition the pipeline crossings would be designed.

Where an aboveground pipeline crossing is immediately upstream from a road crossing (either a bridge or a culvert), backwater from the road during the pipeline design event should be considered when setting the bottom of the pipe elevation. Additionally, if the road is designed for a smaller flood than the pipeline, the changes in hydraulic conditions at the pipeline as a result of the road wash-out should be considered (i.e., changes in location of the concentrated flow and the impact on erosion at the VSM).

Where an aboveground pipeline crossing is immediately downstream from a road crossing (either a bridge or a culvert), the impact of the road on where water will be flowing and the velocity of the water at the pipeline VSM should be considered. Additionally, if the road is designed for a smaller flood than the pipeline, the changes in hydraulic conditions at the pipeline as a result of the road wash-out should be considered (i.e., changes in the location of the concentrated flow and the impact on erosion at the VSM).

1.3.2.2 Belowground Crossings

Design of the HDD crossing should consider the likely scour depth during all floods up to and including the design flood and the likely channel migration over the life of the crossing. It is unknown to what flood event the HDD crossing would be designed.

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Willow Master Development Plan

Appendix E.8B
Ocean Point Technical Memorandums

January 2023

Appendix E.8B1
Ocean Point Technical Memorandum, May 26, 2020

Appendix E.8B1
Ocean Point Technical Memorandum, January 31, 2022

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HYDRAULIC MAPPING AND MODELING
Kenneth F. Karle, P.E.
1091 West Chena Hills Drive, Fairbanks, AK 99709

May 26, 2020

Ocean Point Technical Memorandum

To: E. Leyla Arsan, DOWL

From: Kenneth Karle, P.E.

Subject: Ocean Point Monthly Mean Discharge

An EPA SDEIS reviewer recommended that, as there are no flow data available for the Colville River at Ocean Point, a representative 'synthetic dataset' could be developed for the Ocean Point crossing, using discharge data from the Umiat gaging station. This memo describes the methodology for conducting such an analysis, and includes a table listing average monthly discharge estimates for the Ocean Point crossing.

The drainage-area ratio method suggested by EPA to develop an Ocean Point discharge dataset is indeed commonly used to estimate both flood frequency magnitudes, and individual streamflow discharges, for sites where no streamflow data are available using data from one or more nearby gaging stations (Emerson et al., 2005). The method is intuitive and straightforward to implement and is in widespread use by analysts and managers of surface-water resources. It's often used for locations where no supporting discharge data are available to confirm the validity or develop some type of bias correction to account for differences in watershed characteristics.

A simple ratio of watershed areas upstream of the point of interest is used to estimate flood magnitudes of ungaged sites on gaged streams. The drainage area ratio equation is:

$$Q_u = \frac{Q_g \times A_u}{A_g}$$

Where

- Q_u = ungaged area flow statistic
- Q_g = gaged area flow statistic
- A_u = ungaged area
- A_g = gaged area

In a memo dated November 16, 2018, Jim Aldrich (Arctic Hydrologic Consultants) compiled a table of Colville River Mean Monthly Flow at Umiat, AK, using data from the USGS gaging station 15875000. I updated the table in February 2020; see Table 1.

Note that in every year from 2002 to 2009, there was at least one month from February to April with an average discharge of 0 cfs. Starting in 2010, there were no more '0 cfs' months, and average winter monthly discharge values increased significantly for the period from 2010 to 2019. There are several possible explanations for this. Ongoing climate change on the Alaskan North Slope, with drastically increased temperatures, is well documented. Warmer winters will result in increased winter discharge. Matt Schellekens, the chief hydrologist of the USGS Fairbanks office, noted that prior to the mid-1990s, winter flow was never observed in the Sagavanirktok River. Now, flow is almost always observed and often it is quite a bit (M. Schellekens, personal communication, January 31, 2020).

A second explanation is that slight differences in procedures were used for two different periods. From 2003 to 2009, the site was operated from the USGS Anchorage field office. During that time, there were not many late winter visits, and flow was assumed to go to zero. Since 2010 the gage has been operated from the USGS Fairbanks field office. The Fairbanks hydrographers “usually spent a lot of time in late March or April hunting around the river reach near the gage and almost always found/find at least one or two very small open leads of water seeping out of the downstream end of a gravel bar or two” (M. Schellekens, personal communication, January 31, 2020).

The EPA reviewer noted the increase in winter flows and recommended that only the last 10 years of the Umiat discharge data should be used for the area-ratio analysis, as using mean discharges from the entire period of record “will likely underestimate the discharge at Ocean Point…”

The drainage area for USGS Umiat gaging station 15875000 is 13,860 mi^2. The drainage area upstream of the proposed Ocean Point ice bridge crossing is estimated at 20,580 mi^2. The drainage area ratio (Ocean Point/Umiat) is 1.48.

As a check on the validity of using the drainage area ratio method, I compared a discharge measurement made at Ocean Point to gaged flow at Umiat. CPAI measured a discharge flow rate of 29,000 cfs at Ocean Point on 9/5/2019 at 250 pm. The average flow velocity was 3 ft/sec. Accounting for travel time downstream, the related upstream discharge at Umiat on 9/4/2019 at 1050 am was 23,000 cfs. The Ocean Point flow was approximately 1.3 times greater than the Umiat flow. One data-pair point set is not statistically significant. However, it does imply some reassurance for using the drainage-are method for flow estimates.

Table 1 includes the mean value of the mean monthly discharge values at Umiat for two periods: 2003-2019, and 2010-2019.

I conducted an area ratio analysis to estimate flows at Ocean Point using the mean value of the mean monthly flows for the period 2010-2019, and a drainage area ratio (ungaged/gage) of 1.48. See Table 2.

Table 1. Colville River mean monthly discharge (cfs) at Umiat.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2002									21,030	7,221	844.3	100.1
2003	3.55	0	0	0	690	65,690	24,030	31,800	12,760	10,490	560	72.6
2004	6.87	2.17	0.161	0	40,890	24,940	15,310	24,870	12,060	556.5	142.3	56.6
2005	20.8	4.23	0.016	0	12,830	72,480	13,920	4,143	6,014	1,169	200	104.5
2006	18.4	0.107	0	0	22,010	37,120	21,940	33,560	6,229	2,667	324.7	80
2007	27.9	11.7	0.887	0	4,179	50,530	12,140	17,820	7,511	873.5	177	72.6
2008	21.1	0.724	0	0	17,260	46,530	12,900	10,770	1,867	560	207	72.9
2009	15	0	0	3.03	36,940	45,050	13,890	13,440	13,750	1,775	418	95.2
2010	36.5	13.9	1.65	0.5	17,280	48,760	10,370	15,720	6,213	1,248	454	132.2
2011	35.5	9.66	1.07	0.37	37,790	31,190	13,170	11,330	11,940	1,958	375	93.5
2012	29.2	11	1.92	0.5	16,680	41,910	16,970	14,860	27,440	3,678	145.3	45.9
2013	16.4	3.93	2	1.02	6,434	83,970	10,530	10,290	11,750	1,475	509.3	130.7
2014	25.9	9.25	6	6	33,290	72,180	29,820	10,130	16,140	1,215	216.7	89.9
2015	45.2	29	16.8	12	62,410	17,010	8,243	22,250	11,550	1,504	275.7	65.5
2016	24.4	10.1	5.71	2.75	47,460	32,660	14,540	27,290	15,310	4,868	404.7	64.4
2017	16	3.79	1.16	1	12,070	26,220	13,110	36,370	25,900	6,403	447.9	86.5
2018	24.9	11.9	7.14	6.00	12,220	47,610	26,970	30,330	23,280	3,122	342.9	67.1
2019	40.9	30.2	22.6	20.0	36,180	18,370	12,380	38,990	15,500			
Mean of Monthly Discharge- Sept 2002-Sept 2019	24.0	8.9	3.9	3.1	24,500	44,800	15,900	20,800	13,700	2,990	356.0	84.0
Mean of Monthly Discharge- Jan 2010- Sept 2019	29.5	13.3	6.6	5.0	28,181	41,988	15,610	21,756	16,502	2,830	352.4	86.2

Table 2. Estimated Colville River mean monthly discharge (cfs) at Ocean Point, based on mean monthly discharge at Umiat 2010-2019.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Estimated Mean Monthly Discharge	43.7	19.7	9.8	7.4	41,710	62,140	23,100	32,200	24,420	4190	521.6	127.6

Numerous factors will affect the relationship between discharge and drainage area. For example, if the watershed characteristics of the upper watershed, such as the ratio of mountainous area to lowlands, were significantly different than those of the additional downstream drainage area, then the flow relationship may not be linear. Such a relationship could potentially be improved by investigating regional statistics, regression, and rainfall-runoff modeling (bias correction). That type of additional analysis generally leads to the development of an exponent for the drainage area ratio. But that type of data is obviously scarce and probably not worth pursuing.

Another consideration is that this analysis does not account for other conditions that may affect flow rates at Ocean Point. For example, surface flow passing Umiat may be forced downstream into a gravel bed flow condition due to a blocked channel. Surface flow may also end up in storage as ice until warming temperatures occur. Conversely, groundwater seeps between Umiat and Ocean Point may lead to larger flows downstream than predicted by the drainage area ratio. The consensus of opinion from Jim Aldrich, Matt Schellekens (USGS), and Richard Kemnitz (BLM retired) is that there is probably surface flow in the Colville River downstream of Umiat in every month of the year.

As noted elsewhere, the best course of action to characterize winter flows at Ocean Point will be to conduct field observations and measurements during the winter months at the Ocean Point crossing for the next several years. However, until such field measurements are made, the flow statistics in Table 2 can be used, with caution, to provide an estimate of the magnitude of winter flows for the Ocean Point crossing.

Please let me know if you have additional questions or need more information.

Ken

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MEMORANDUM

TO: Zach Huff, E.I.

FROM: Euan-Angus MacLeod, P.E.

DATE: November 22, 2022

SUBJECT: Ocean Point Monthly Mean Discharge - Update

This memorandum provides an update to average monthly discharge estimates for the Ocean Point crossing of the Colville River originally presented in the May 26, 2020, Ocean Point Technical Memorandum prepared by Kenneth Karle, P.E. (2020 Memo). This memorandum includes updates to Table 1 and Table 2 from the 2020 Memo using additional flow data available for USGS gaging station 15875000 at Umiat from January 2020 through September 2022. The same drainage-area ratio methodology described in the 2020 Memo was used to update Table 2, which provides estimated mean monthly discharge for the Coville River at Ocean Point.

Table 1. Colville River mean monthly discharge (cfs) at Umiat.

Year	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
2002	No Data	No Data	No Data	No Data	No Data	No Data	No Data	No Data	21,030	7,221	844.3	100.1
2003	3.55	0	0	0	690	65,690	24,030	31,800	12,760	10,490	560	72.6
2004	6.87	2.17	0.161	0	40,890	24,940	15,310	24,870	12,060	556.5	142.3	56.6
2005	20.8	4.23	0.016	0	12,830	72,480	13,920	4,143	6,014	1,169	200	104.5
2006	18.4	0.107	0	0	22,010	37,120	21,940	33,560	6,229	2,667	324.7	80
2007	27.9	11.7	0.887	0	4,179	50,530	12,140	17,820	7,511	873.5	177	72.6
2008	21.1	0.724	0	0	17,260	46,530	12,900	10,770	1,867	560	207	72.9
2009	15	0	0	3.03	36,940	45,050	13,890	13,440	13,750	1,775	418	95.2
2010	36.5	13.9	1.65	0.5	17,280	48,760	10,370	15,720	6,213	1,248	454	132.2
2011	35.5	9.66	1.07	0.37	37,790	31,190	13,170	11,330	11,940	1,958	375	93.5
2012	29.2	11	1.92	0.5	16,680	41,910	16,970	14,860	27,440	3,678	145.3	45.9
2013	16.4	3.93	2	1.02	6,434	83,970	10,530	10,290	11,750	1,475	509.3	130.7
2014	25.9	9.25	6	6	33,290	72,180	29,820	10,130	16,140	1,215	216.7	89.9
2015	45.2	29	16.8	12	62,410	17,010	8,243	22,250	11,550	1,504	275.7	65.5
2016	24.4	10.1	5.71	2.75	47,460	32,660	14,540	27,290	15,310	4,868	404.7	64.4
2017	16	3.79	1.16	1	12,070	26,220	13,110	36,370	25,900	6,403	447.9	86.5
2018	24.9	11.9	7.14	6.00	12,220	47,610	26,970	30,330	23,280	3,122	342.9	67.1
2019	40.9	30.2	22.6	20.0	36,180	18,370	12,380	38,990	15,500	0	0	0
2020	27.2	9.0	4.7	4.0	106,013	23,807	12,248	19,911	23,106	13,442	370.3	69.0
2021	21.7	7.8	2.6	2.1	9,792	34,387	24,607	21,238	27,565	No Data	No Data	No Data
2022	23.1	14.0	9.2	6.0	63,728	43,319	12,872	25,002	12,977	No Data*	No Data	No Data
Mean of Monthly Discharge-Sept 2002-Sept 2022	24.0	9.1	4.2	3.3	29,807	43,187	15,998	21,006	14,757	3,380	337.6	78.9
Mean of Monthly Discharge-Sep 2010-Sept 2022	27.5	12.5	6.7	5.1	37,006	39,386	16,288	22,333	18,538	3,767	308.8	71.3

*** Contained provisional data subject to revision at time of writing**

Table 2. Estimated Colville River mean monthly discharge (cfs) at Ocean Point, based on mean monthly discharge at Umiat 2010-2022.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Estimated Mean Monthly Discharge	40.8	18.5	10.0	7.6	54,768	58,291	24,107	33,052	27,437	5,574	475.0	105.5