FINAL HYDROLOGY REPORT

SECOND AND THIRD STREET REHABILITATION PROJECT

Manokotak, Alaska

Bristol Project No. 32150007

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ACRONYMS AND ABBREVIATIONS

°	degrees
%	percent
‘ or ft	feet
A	drainage area or pipe cross-sectional area
AC	acres
ADOT&PF	Alaska Department of Transportation and Public Facilities
AK	Alaska
BMP	Best Management Practices
Bristol	Bristol Engineering Services Corporation
C	Runoff Coefficient
cfs	cubic feet per second
CMP	corrugated metal pipe
Council	Manokotak Village Council
CSP	corrugated steel pipe
DCCED	Department of Commerce, Community, and Economic Development
F	Fahrenheit
FAA	Federal Aviation Administration
ft/ft	feet per foot
H:V	horizontal: vertical
I	Rainfall Intensity
IDF	Intensity-Duration-Frequency
in/hr	inches per hour
KMO	Manokotak
min	minutes or minimum
PE	Professional Engineer
PDS	Partial Duration Series
Q	flow
R	Hydraulic Radius
ROW or R/W	Right-of-way
S	slope
t_c	Time of Concentration

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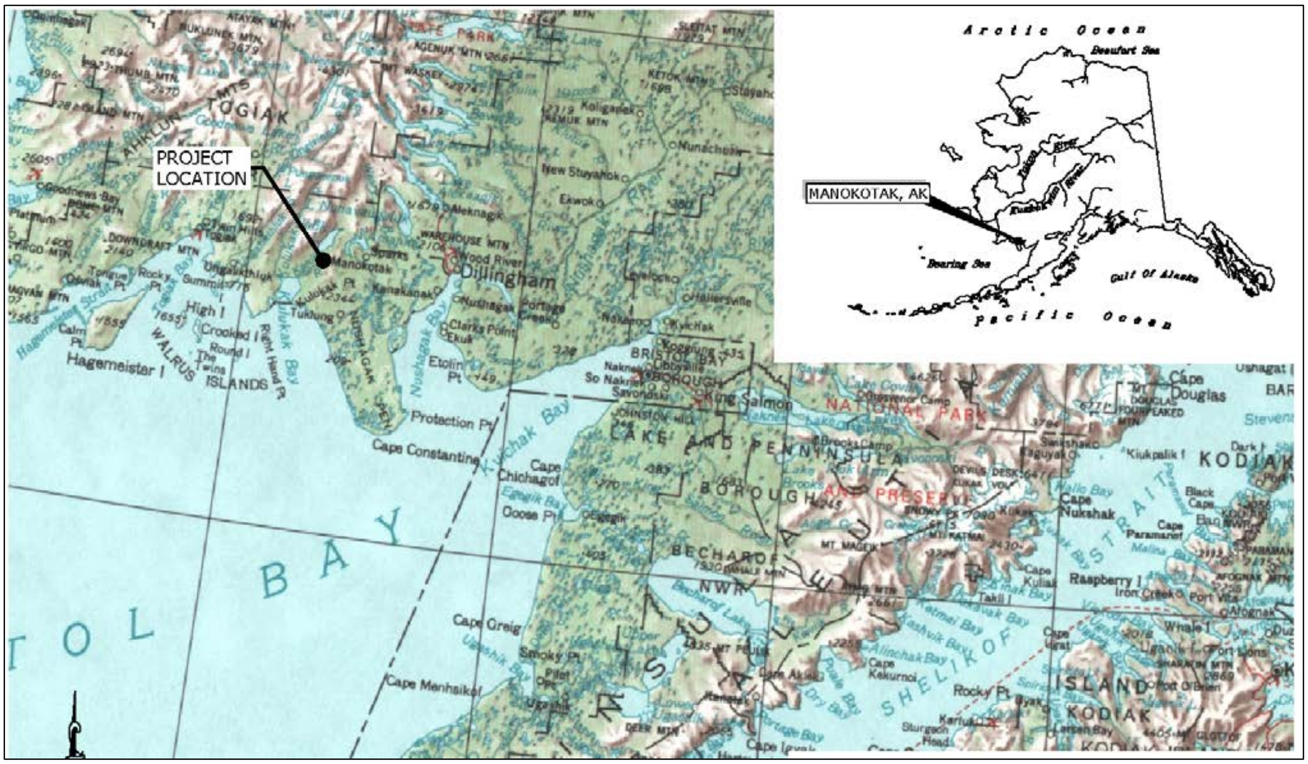
1.0 INTRODUCTION

Bristol Engineering Services Company, LLC (Bristol) is under contract with the Manokotak Village Council (Council) to develop a hydrology report for First Street, Second Street, Third Street, Alder Street, C Street, and Salmon Street in Manokotak, Alaska (Figure 1).

Figure 1 Location & Vicinity Map



This report details how Bristol determined storm water flow, culvert pipe sizing, and ditch capacity for the Second and Third Street Rehabilitation Project; it also summarizes climatic data, describes flow calculation methods, and recommends culvert sizes and locations.

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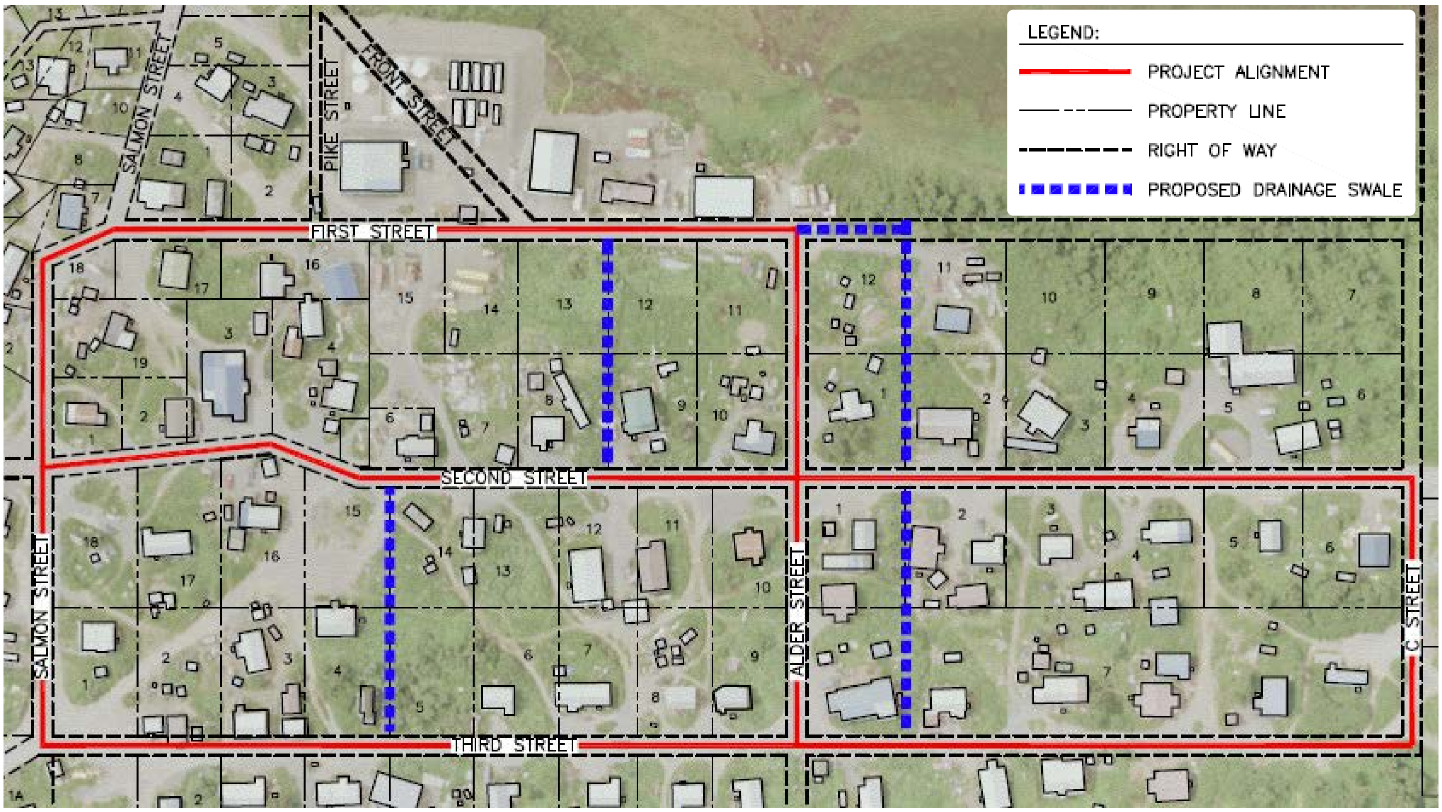
2.0 BACKGROUND

The project scope consists of rehabilitating approximately 0.9 miles of existing roads in Manokotak, shown below in Figure 2.

Figure 2 Project Area



Roads included in the design are as follows, with typical sections shown on Figure 3.

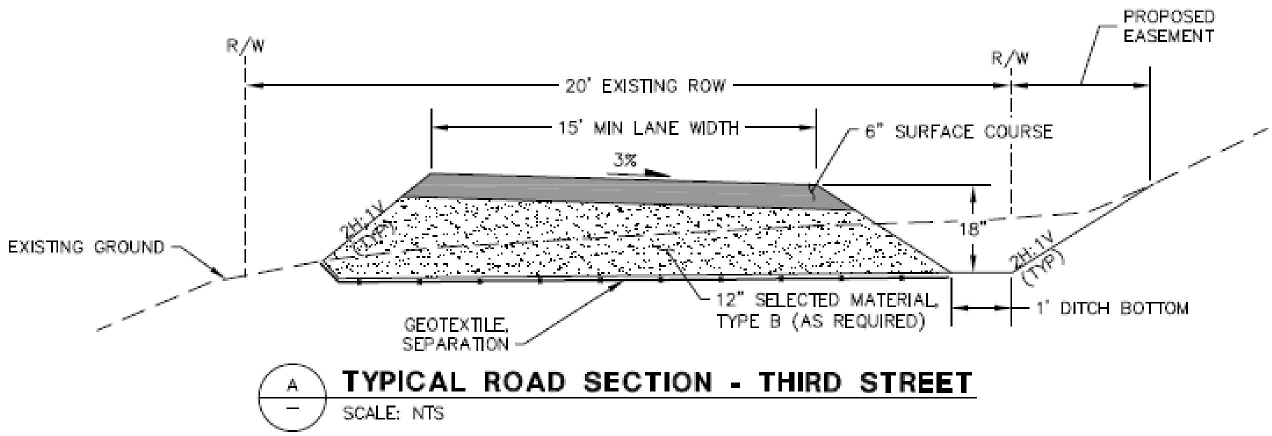
- First Street
  - 15-foot-wide cross-section with 1-foot-wide ditch.
  - Approximate Length: 820 feet.
- Second Street
  - 15-foot-wide cross-section with 1-foot-wide ditch.
  - Approximate Length: 1,390 feet.
- Third Street
  - 12-foot-wide cross-section with 1-foot-wide ditch and on street parking.
  - Approximate Length: 1,410 feet.

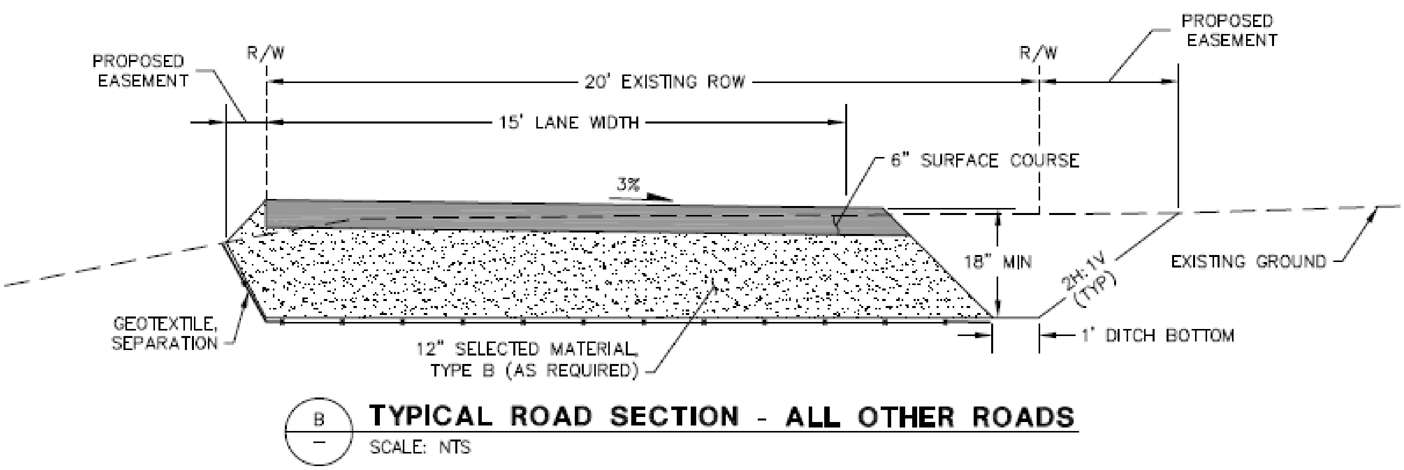
- Alder Street
  - 15-foot-wide cross-section with 1-foot-ditch.
  - Approximate Length: 470 feet.
- C Street
  - 15-foot-wide cross-section with 1-foot-ditch.
  - Approximate Length: 230 feet.
- Salmon Street
  - 15-foot-wide cross-section with 1-foot-ditch.
  - Approximate Length: 470 feet.

This design also includes new drainage features. Currently there are three year-round, drainage channels that flow from the no-name mountain, east of the Third Street, down through the project area. Runoff from these channels is a constant annoyance to surrounding residents. As a solution, new rock-filled drainage swales (Figure 4) will be constructed to convey runoff to ditches and storm piping. The swales will include a 12-inch perforated pipe beneath the rock to reduce standing water at the surface, as requested by the community to reduce safety risks for children. Rock-lined open channel swales will only be used at the drainage outlet point of the project area. Locations of the new swales (Figure 2) will be placed at lot lines when possible, minimizing right of way impacts and private land. Additionally, Type "A" field inlets will be installed at each road-crossing culvert to convey runoff from the drainage ditches to the culverts, while maintaining the required 12-inch minimum cover over the culverts. Type "A" field inlets are defined in the Alaska Department of Transportation and Public Facilities (ADOT&PF) Standard Drawings.

Finally, new on-street parking will alleviate the current congestion on Third Street. Eight-foot wide parking areas will be added to the west side of Third Street by installing a series of bin-walls, retaining walls, guardrails, and ramps leading to adjacent properties (Figure 5).

Figure 3 Typical Sections





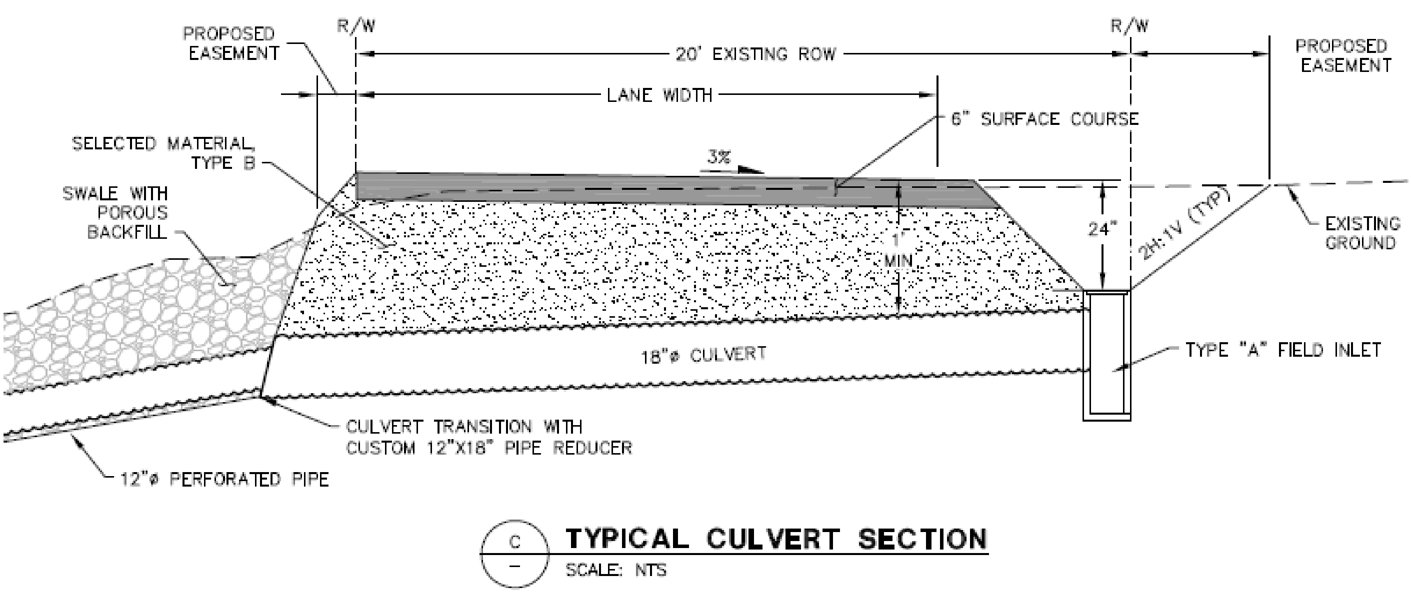
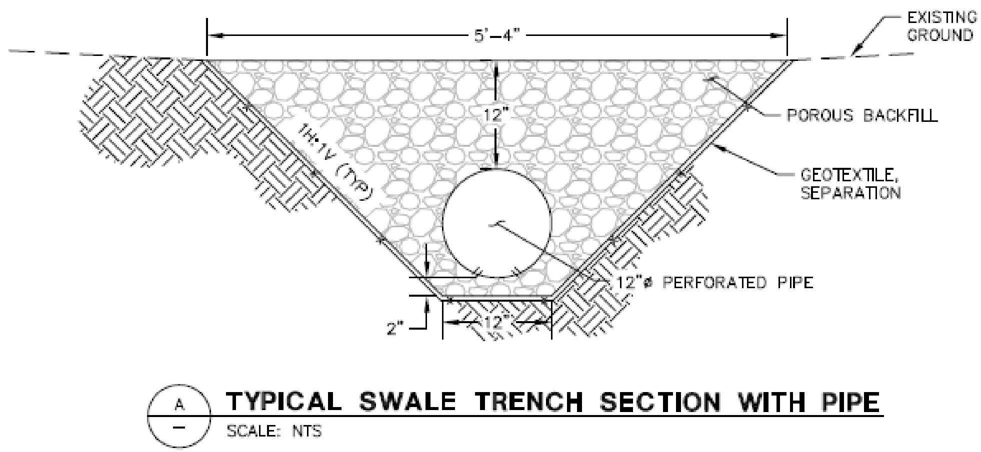


Figure 4 Typical Swale Sections



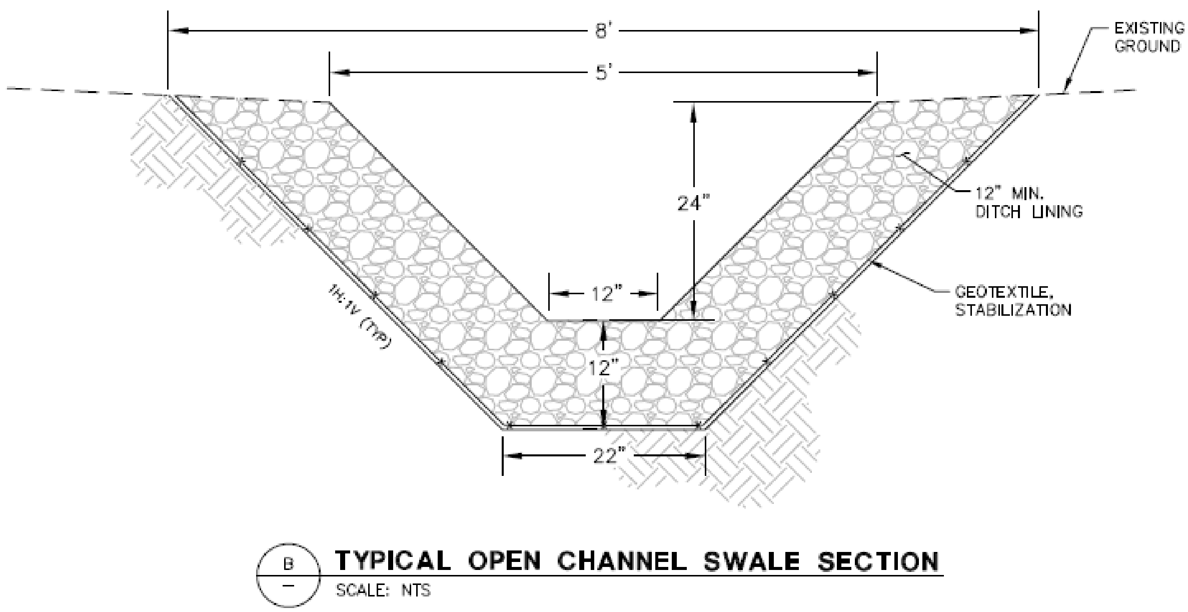
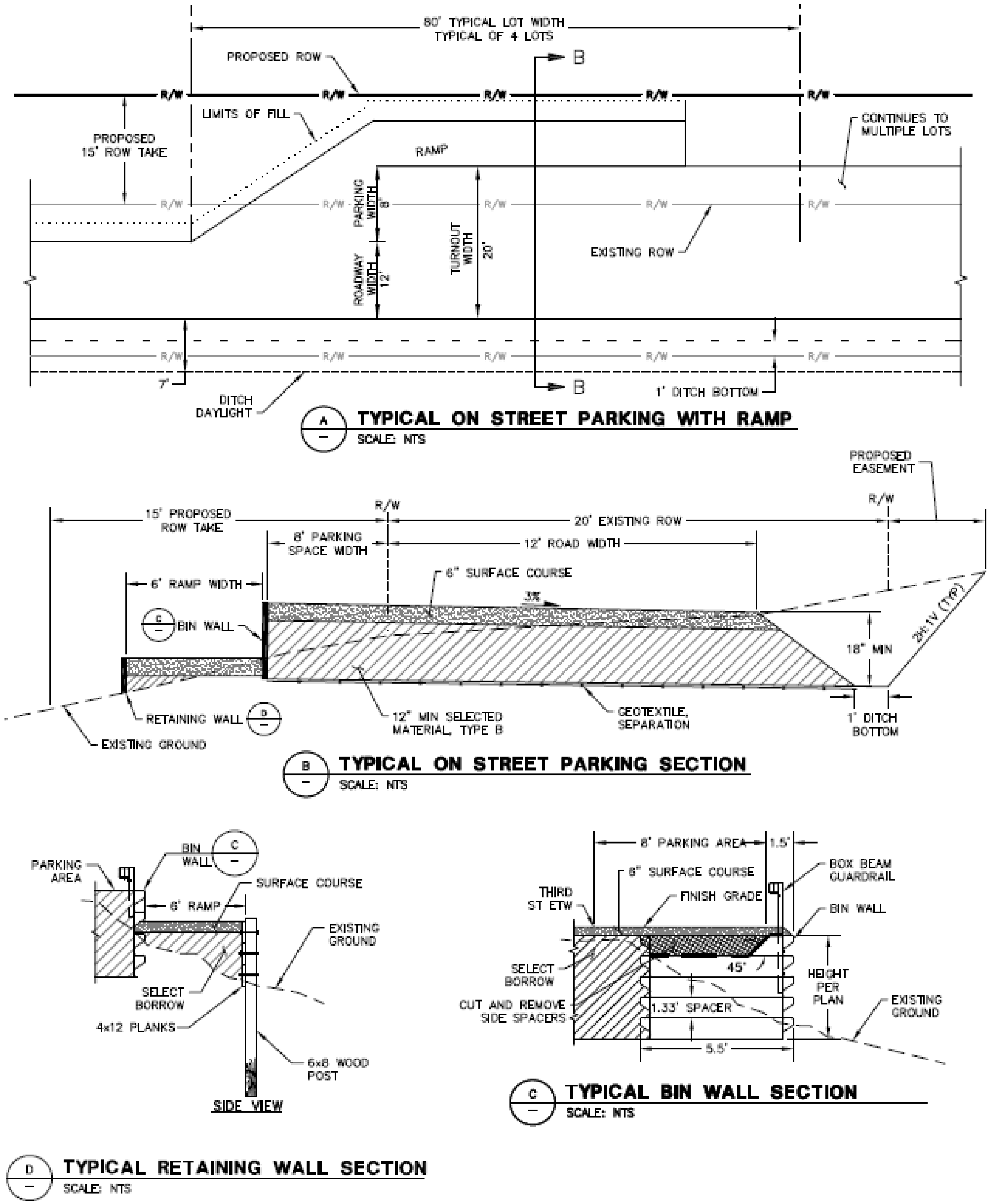


Figure 5 On-Street Parking Details



2.1 LOCATION AND CLIMATE

Manokotak is located 25 miles southwest of Dillingham on the Igushik River. It lies 347 miles southwest of Anchorage. Manokotak is 58 degrees (°) 59 minutes (') and 23 seconds (") north latitude and 159 degrees (°) 2 minutes (') and 57 seconds west longitude, in Section 12, Township 14 South, Range 59 West, of the Seward Meridian. Manokotak is located in the Bristol Bay Recording District, and encompasses 36.4 square miles of land and 0.9 square miles of water.

Manokotak is located in a climatic transition zone. The primary influence is maritime, although the arctic climate affects the region. Average summer temperatures range from 40 to 70 °F; winter temperatures average from 4 to 30 °F. Fog and high winds exist periodically through the year. The river is ice-free from June through mid-November (Alaska Department of Commerce, Community, and Economic Development, 2014). Average annual precipitation in the area is 26 inches, including 82 inches of snowfall (Western Regional Climate Center, 2014).

2.2 SITE VISIT

On October 2nd, 2014 Professional Engineer Isaac Pearson performed an initial site inspection of the existing road conditions. The weather was sunny during the investigation. The engineer photographed all routes and evaluated current road conditions and drainage patterns.

A 95% plan-in-hand review was conducted by Bristol on May 21, 2018 to observe structures, utilities, and other features observed within the project footprint and cross-reference existing conditions to the plans.

3.0 HYDROLOGY ANALYSIS

As part of this project, Bristol performed a hydrological analysis of the watershed, which affects the existing road system. The information required to perform the analysis was gathered from the road corridor topographical survey, topographical maps and available reference material. To complete the analysis Bristol determined the desired rainfall intensity return event, and determined the drainage basin sizes and characteristics. The individual drainage basin characteristics included size, location, land type, slope, and flow length. Bristol engineers used the Rational Method to estimate peak runoff for drainage swale and storm drain pipe sizing:

$$Q_P = CiA$$

Q_P = peak runoff quantity, ft^3/s

C = runoff coefficient, dimensionless

i = rainfall intensity, in/hr

A = drainage area, acres

There are four basic assumptions for using the Rational Equation:

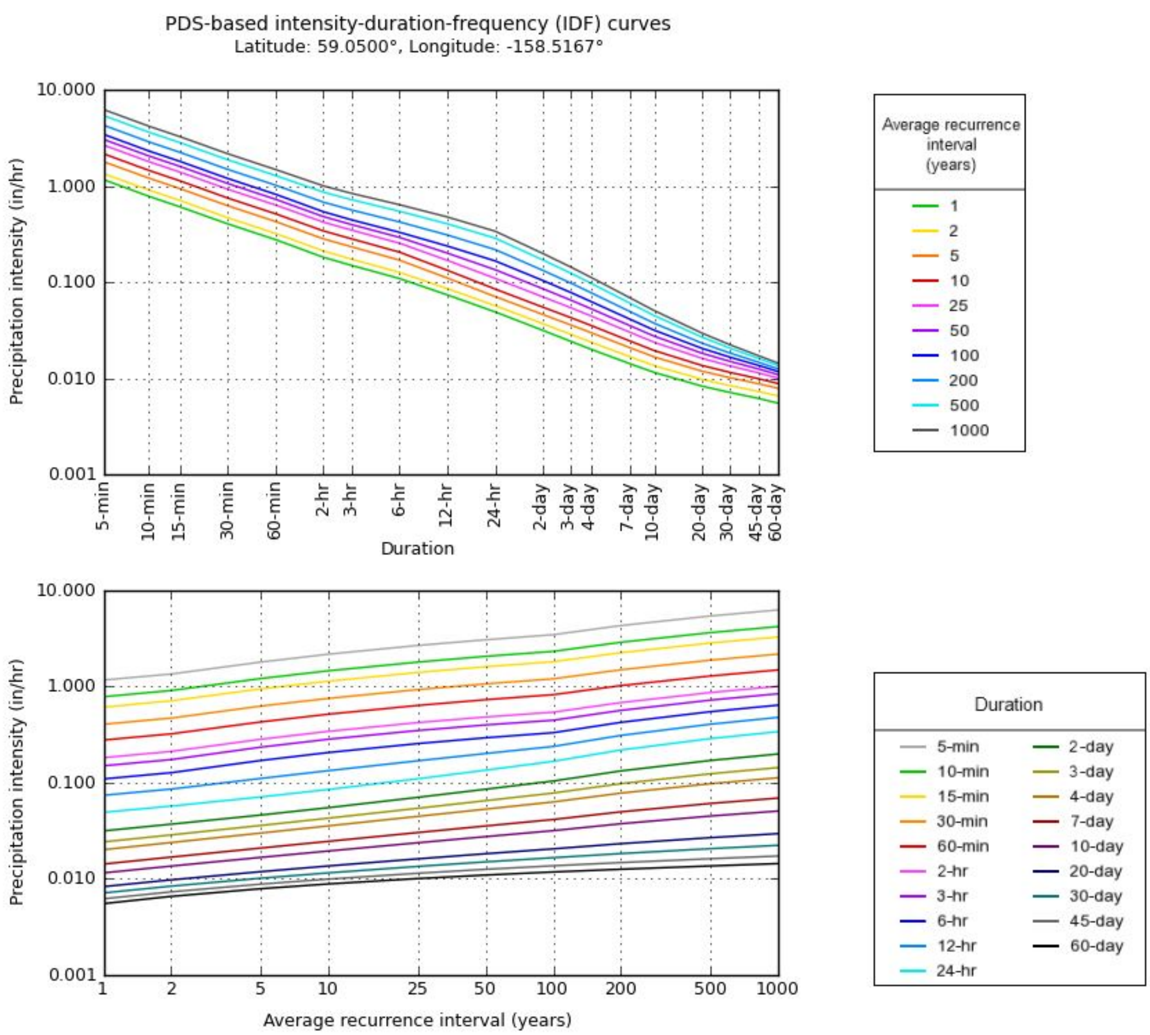
1. The rainfall intensity is constant for a time interval at least equal to the time of concentration;
2. The runoff is a maximum when the rainfall intensity lasts as long as the time of concentration;
3. The runoff coefficient remains constant during the storm event; and
4. The watershed area does not change during the storm.

3.1 RAINFALL INTENSITY RETURN EVENT

In most hydrological analyses, the rainfall intensity of a specific return event is used to model the runoff characteristics of the watershed. However, in this case Bristol also considered the spring melt waters as a contributing factor to the 24-hour storm events. The average annual snowfall of 82 inches was converted to a water equivalent of 8.2 inches based on a typical conversion ratio of 10 inches of snow equal to 1 inch of water. Bristol assumed that the 8.2-inch water equivalent was released over a period of 20 melting days, which added 0.41 inches of rainfall to the storm event in a 24-hour period.

For this analysis, a 10-year 24-hour return event was used. The rainfall intensity in Manokotak for the design storm is 0.085 inches/hour as derived from the partial duration series (PDS)-based intensity-duration-frequency (IDF) curves shown below in Figure 6. These graphs were obtained from the National Oceanic and Atmospheric Administration's National Weather Service, Hydrometeorological Design Studies Center, Precipitation Frequency Data Server.

Figure 6 PDS-based IDF Curves (Dillingham, AK)



This rainfall intensity equates to a total rainfall of 2.04 inches over the 24-hour storm period. For worst-case scenarios, 0.41 inches of melt water will be added to spring thaw calculations. The rainfall data for the design and spring thaw storm event is shown in Table 1.

Table 1 Rainfall Intensities for 24-hour Storm Events

Event	Rainfall (inches)	Intensity (in/hr)
10 Year	2.04	0.09
10 Year, with Spring Thaw	2.45	0.10

3.2 DRAINAGE BASIN CHARACTERISTICS

The watershed was divided into 6 drainage basins. The drainage basins (Figure 7) were determined by the local topography, naturally occurring drainage channels, and the area's potential impact to the road improvement project. Storm water runoff from these areas requires channelization via swales and culverts.

Once the drainage basins were determined, their areas were calculated with AutoCAD and each basin was assigned a runoff coefficient based upon the typical surface coverage within the basins. Bristol used the Alaska Storm Water Guide (2011) runoff coefficients to determine the average coefficients, which are listed in Table 2.

Table 2 Runoff Coefficients

Surface	Runoff Coefficient (C)
Gravel Road	0.55
Forest	0.14
Grassed Area	0.35
Impervious Surface	0.85

Bristol also analyzed the project drainage basins assuming the worst-case scenario of a spring thaw when all ground is still frozen and functioning like a smooth impermeable surface. A runoff coefficient value of 0.95 is appropriate for this assumption.

3.2.1 Size

The size of each area is measured in acres. Measurements were determined using AutoCAD Civil 3D software based on mapping and data collected by Bristol.

3.2.2 Location

The drainage basin locations were chosen based on the land type, their relationship to the discharge point, elevations of the surrounding areas, and the type of water conveyance features in areas. Water conveyance methods include overland flow or a defined channel.

3.2.3 Slope

The measure of slope for a given drainage basin is defined as the amount of fall in elevation divided by length over which the fall occurs. The values of the slope for each drainage basin were determined using existing topography based on Manokotak community mapping (DCCED, 2003)

3.2.4 Flow Length

The flow length of a drainage basin is defined as the longest distance a drop of water would have to traverse the sub-area in a straight line. Distances were measured with AutoCAD software.

3.2.5 Time of Concentration

The travel time was calculated using the US Federal Aviation Administration (FAA) formula:

$$t_c = [1.8 * (1.1 - C) * (L_o)^{1/2}] / (S)^{1/3}$$

t_c = time of concentration, minutes

C = rational method runoff coefficient, dimensionless

L_o = length to collection point, feet

S = slope, percent

3.3 RESULTS

Tables 3 presents the runoff coefficient numbers and drainage basin areas used to calculate the average runoff coefficient for the project area.

Table 3 Drainage Basin Composite Runoff Coefficients

Drainage Basin	Gravel Area (AC)	C Value	Impervious Area (AC)	C Value	Lawn Area (AC)	C Value	Forest Area (AC)	C Value	Total Area (AC)	**Composite C Value**
A	0.38	0.55	0.23	0.85	0	0.35	26.36	0.14	26.97	**0.15**
B	0.21	0.55	0.11	0.85	0	0.35	8.22	0.14	8.55	**0.16**
C	1.70	0.55	0.50	0.85	3	0.35	0.00	0.14	5.09	**0.47**
D	1.00	0.55	0.52	0.85	2	0.35	0.00	0.14	3.89	**0.47**
E	1.10	0.55	0.45	0.85	3	0.35	0.00	0.14	4.27	**0.45**

Using the rainfall intensity, drainage basin areas, and the runoff coefficients, the estimated flow rates can be determined using the rational method. Tables 4 and 5 display flow rates in each basin for design storm and spring thaw events, respectively.

Table 4 Design Basin Flow Rate Determination

Drainage Basin	Area (acre)	Dominant Soil Type	C Value	I (in/hr)	Q = CIA (cfs)
A	26.97	Forest	0.15	0.09	0.37
B	8.55	Forest	0.16	0.09	0.12
C	5.09	Lawn	0.47	0.09	0.21
D	3.89	Lawn	0.47	0.09	0.16
E	4.27	Lawn	0.45	0.09	0.17
Total	**48.8**				**1.04**

Table 5 Spring Thaw Basin Flow Rate Determination

Drainage Basin	Area (acre)	Dominant Soil Type	C Value	I (in/hr)	Q = CIA (cfs)
A	26.97	Forest	0.95	0.10	2.56
B	8.55	Forest	0.95	0.10	0.81
C	5.09	Lawn	0.95	0.10	0.48
D	3.89	Lawn	0.95	0.10	0.37
E	4.27	Lawn	0.95	0.10	0.41
Total	**48.8**				**4.63**

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4.0 DITCHING AND CULVERT CAPACITY ANALYSIS

One of the goals of this project is to improve the drainage patterns by placing culverts, roadside ditches, and rock-filled drainage swales at appropriate locations with installation for arctic conditions.

To determine the culvert locations, Bristol used the information computed during the hydrological analysis. The watershed analysis determined how the existing road corridors affected surface flow.

4.1 ROADSIDE DITCHING

The existing road system consists of drainage ditching and road-crossing culverts that direct runoff west where it continues down gradient. The existing ditches are inadequately sized to convey the amount of runoff experienced during spring-breakup. The ditching and culvert layout is also ineffective, resulting in drainage paths that cut through properties and cause standing water in yards and along road shoulders. To mitigate the ponding issue, roadside ditching will be re-graded to ensure proper conveyance to culverts and outlets. Ditches will be seeded with a native mix to reduce erosion. Culverts will also be addressed, as described in Section 4.2.

Due to narrow right-of-way and existing structures, roadside ditching was minimized where needed to keep roadway cut and fill limits within right-of-way and away from structures while still providing adequate conveyance and capacity for storm water runoff. As shown on Figure 3, the proposed ditches will be 12-inches wide and 18-inches deep with 2H:1V side slopes. The ditches will need to be widened to 24-inches wide at locations of the Type ‘A’ Inlets. Ditch slopes will follow the road profile vertical grades.

4.2 ROAD-CROSSING CULVERTS

The project area currently contains 14 existing culverts; most of the culverts are corrugated steel pipe (CSP), between 12 and 24 inches in diameter. Several culverts along the project corridor are plugged, or have collapsed and no longer function properly.

All existing culverts will be removed, and six new culverts are proposed along the alignments at points where they will function most efficiently (such as at low points in the road profile), modeled on the Manning Equation. Road-crossing culvert locations are shown on Figures 7 and 8. Both road-crossing culverts and driveway culverts will be included in the design (driveway culverts are discussed in Section 4.4, and swales are discussed in Section 5.0).

The Rational Method was first used to determine the runoff flow rate at each outlet point (road-crossing culvert and connected swale), for design and spring thaw flows, as shown on Tables 6 and 7, respectively. Then, the Manning Equation was used to verify the proposed culvert sizes.

Table 6 Culvert/Swale Flow Rate for Design Storm

Outlet	Contributing Drainage Basin	Runoff Coefficient (C)	Area Acres (A)	Rainfall Intensity (I)	Flow Rate cfs (Q)
Culvert 1/ Drainage Swale I	A	0.15	27.0	0.09	0.37
				Total	**0.37**
Culvert 2 Drainage Swale II	B	0.16	8.5	0.09	0.12
				Total	**0.12**
Culvert 3/ Drainage Swale III	A	0.15	27.0	0.09	0.37
	C	0.47	5.1	0.09	0.21
				Total	**0.58**
Culvert 4/ Drainage Swale IV	B	0.16	8.5	0.09	0.12
	D	0.47	3.9	0.09	0.16
				Total	**0.29**
Culvert 5/ Drainage Swale V	A	0.15	27.0	0.09	0.37
	C	0.47	5.1	0.09	0.21
	E	0.45	4.3	0.09	0.17
				Total	**0.76**

Table 7 Culvert / Swale Flow Rate for Spring Thaw

Outlet	Contributing Drainage Basin	Runoff Coefficient (C)	Area Acres (A)	Rainfall Intensity (I)	Flow Rate cfs (Q)
Culvert 1/ Drainage Swale I	A	0.95	27.0	0.10	2.56
				Total	**2.56**
Culvert 2 Drainage Swale II	B	0.95	8.5	0.10	0.81
				Total	**0.81**
Culvert 3/ Drainage Swale III	A	0.95	27.0	0.10	2.56
	C	0.95	5.1	0.10	0.48
				Total	**3.05**
Culvert 4/ Drainage Swale IV	B	0.95	8.5	0.10	0.81
	D	0.95	3.9	0.10	0.37
				Total	**1.18**
Culvert 5/ Drainage Swale V	A	0.95	27.0	0.10	2.56
	C	0.95	5.1	0.10	0.48
	E	0.95	4.3	0.10	0.41
				Total	**3.45**

The flow capacity for each culvert size was determined based on the Manning Equation and the pipe's cross-sectional dimension, material, flow depth, and slope. The Manning Equation and the parameters used in the modeling are as follows:

$Q = (1.49/n) * A * (R)^{2/3} * (S)^{1/2}$

Q = flow rate, cubic feet per second

n = Manning coefficient, dimensionless

A = area, square feet

R = hydraulic radius, feet

S = slope, percent

- Pipe Size
  - 18-inch diameter
- Pipe Slope
  - Varies, per plan
- Manning Coefficient
  - A value of 0.025 (corrugated metal pipe) was used in all cases

Using a 1% slope, the Manning's Equation calculation indicated 18-inch diameter culverts will be required. Once the project design was finalized, the Manning's Equation was used to verify the flow capacity at each culvert using the design slope. Table 8 shows the actual maximum flow calculated for each culvert (Q_{Actual}), assuming the pipes are flowing 100% full.

Table 8 Road-Crossing Culvert Flow Capacities

Culvert No. Per Plan	Culvert No.	Route No.	Slope Per Plan	Max. Culvert Flow (cfs) (Q_{Actual})	Spring Thaw Flow (cfs) ($Q_{Required}$)	Is $Q_{Actual} \geq Q_{Required}$?
P3-2	1	R3	0.99%	5.45	2.56	Yes
P3-6	2	R3	5.72%	13.10	0.81	Yes
P2-3	3	R2	0.37%	3.33	3.05	Yes
P2-5	4	R2	2.00%	7.75	1.18	Yes
P1-2	5	R1	0.40%	3.46	3.45	Yes

This flow rate was then compared against the spring thaw flow ($Q_{Required}$) to ensure the proposed culverts are adequately sized. According to the calculations, each proposed culvert will provide capacity required for the drainage system.

4.3 FIELD INLETS

Field inlets were added to the design due to narrow right-of-way constraints. In order to keep road cut and fill limits within the right-of-way and away from existing structures, the height of the proposed structural section is limited. The road section design includes 6-inches of surface course over 12-inches of borrow material, which provides a total section depth of 18-inches. The depth of the roadside drainage ditch is also 18-inches. However, an 18-inch diameter road-crossing culvert with the invert set at the bottom of the roadside ditch would essentially have no cover beneath the road. The minimum required cover is 12-inches. Therefore, Bristol proposes to install ADOT&PF Type "A" field inlets at the inlet of each road-crossing culvert (See Figure 3). The field inlets will be approximately 2-feet wide by 3-feet long by 4-feet deep. They will be installed at the ditch bottom with an 18-inch minimum sump below the culvert invert elevation.

4.4 DRIVEWAY CULVERTS

A similar issue of providing a minimum 12-inch cover was encountered when designing the driveway culverts. For the driveway culverts, 12-inch diameter CMP culverts will be adequate to convey design storm and spring thaw runoff. However, a 12-inch diameter culvert under an 18-inch tall driveway section would only provide 6-inches of cover.

In locations where 12-inches of cover could not be achieved, special ditching or approach sections were utilized. For example, at high-points in the road profile where flows were minimal, a small 6-inch deep v-ditch was used to cross the driveway instead of a culvert. In other areas, additional fill and special driveway grading was implemented.

Driveway culvert capacities were verified on the design plans using the Manning's equation for 12-inch diameter CMPs and culvert slopes per plan. Through this process, five culverts were identified that did not provide adequate flow at with their design slopes, so slopes were adjusted as needed to increase the flow capacity. For these five culverts, ditches will need to be field fit at the culvert inverts since the required culvert slope and ditch slope (same as the road profile slope) do not match exactly.

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5.0 DRAINAGE SWALE ANALYSIS

New drainage swales will convey water from roadside ditching along property lines and away from essential infrastructure. Figures 7 and 8 show the locations of the five proposed drainage swales. Storm runoff from culverts and ditching will be directed to the swales to convey storm water to the project discharge points (Swale IV and Culvert 5).

Originally, the swale design consisted of trapezoidal open channels with riprap lining. However, the community was concerned about open water within the swales because children frequently play in yards near the proposed swales. Therefore, Bristol proposed to construct rock-filled trenches with a perforated pipe at the bottom to convey water beneath the rock layer, similar to a French-drain. The rock will be washed drain rock, or ditch-lining grade riprap with a high enough void ratio to allow water to trickle through the trench and into the pipe. The majority of the runoff will be contained within the pipe, but some storage will be contained within the voids of the rock. Open channel rock-lined swales will only be used for Swale V and at the end of Swale IV, at the drainage outfall point of the project area.

Rock-filled swales with perforated pipe will have a 12-inch bottom width, and will be 26-inches deep to accommodate a 12-inch diameter pipe with 2-inches of drain rock below the pipe and 12-inches above. Side slopes of 1H:1V were used to reduce the total top width of the swale and impacts to residential properties, which will be approximately 5-feet wide, as shown on Figure 5, Detail A. Rock-lined open channel swales will be 12-inches wide by 24-inches deep, with 1H:1H side slopes, and lined with rock 12-inches thick, as shown on Figure 5, Detail B.

The flow capacity for each drainage swale was determined based on Manning's Equation and the swales bottom width, side slope, material, flow depth, slope, and pipe diameter (if applicable).

The Manning Equation parameters used in the modeling are as follows:

$Q = (1.49/n) * A * (R)^{2/3} * (S)^{1/2}$

Q = flow rate, cubic feet per second

n = Manning coefficient, dimensionless

A = area, square feet

R = hydraulic radius, feet

S = slope, percent

- Rock-filled swale with pipe (see Figure 5, Detail A)
  - Bottom Width: 1 foot
  - Side Slope: 1H:1V
  - Depth: 26-inches
  - Pipe Size: 12-inch diameter
  - Manning Coefficient: 0.025 (corrugated metal pipe)
- Open channel rock-lined swale (see Figure 5, Detail B)
  - Bottom Width: 1 foot
  - Side Slope: 1H:1V
  - Depth: 24-inches
  - Manning Coefficient: 0.035 (earth channel; stony, cobbles)
- Swale Slope
  - The minimum design slope for each swale was used, which ranges from 0.4% to 7.6% depending on the swale profile.

Table 9 shows the minimum flow capacity using swale design parameters listed above and the minimum slope per plan for each portion of swale. The flow rate shown only includes the flow within the pipe, and does not include the water stored within the drain rock of the swale. The minimum flow rate within the pipe should be greater than or equal to the flow from the upstream culvert, as verified in the Table 9.

Table 9 Perforated Pipe Flow Capacities

Swale No.	Swale Type	Minimum Slope	Minimum Flow Rate (ft^3/sec)	Upstream Culvert Spring Thaw Flow (cfs) – Per Table 7	Upstream Culvert No.
I	Rock filled swale with perforated pipe	4.1%	3.77	2.56	1
II	Rock filled swale with perforated pipe	7.6%	5.13	0.81	2
III	Rock filled swale with perforated pipe	5.1%	4.21	3.05	3
IV	Rock filled swale with perforated pipe	4.0%	3.72	1.18	4
V	Rock lined open channel swale	0.4%	7.76	3.45	5

According to Table 9, the proposed swale designs will be adequate to convey spring thaw runoff for all swales.

Time of concentration (t_c) is the time required for an entire watershed to contribute to runoff at the point of interest for hydraulic design; this time is calculated as the time for runoff to flow from the most hydraulically remote point of the drainage area to the point under investigation. Tables 10 and 11 summarize the total time of concentration for each discharge point for design and spring thaw events, respectively. Locations for reaches can be found on Figure 7.

Table 10 Design Time of Concentration

Discharge Point	Contributing Reaches	Post Construction t_c (min)
Culvert 5	A1, A2, C1, C2, E1, E2	86.8
	t_c	**86.8**
Drainage Swale IV	B1, B2, D1, D2	39.8
	t_c	**39.8**

Table 11 Spring Thaw Time of Concentration

Discharge Point	Contributing Reaches	Post Construction t_c (min)
Culvert 5	A1, A2, C1, C2, E1, E2	20.9
	t_c	**20.9**
Drainage Swale IV	B1, B2, D1, D2	8.8
	t_c	**8.8**

6.0 DRAINAGE FACILITY MAINTENANCE

The proposed road-side ditching, culverts, and drainage swales should be inspected annually and with maintenance activities as needed. Recommended maintenance activities are as follows:

- Inspect all drainage facilities at least once per year, typically at the end of spring breakup.
- Road-side ditches will require removal of sediment buildup and re-establishment of vegetation, as needed.
- Driveway culverts will require removal of debris buildup and may require de-icing.
- Type ‘A’ inlet sediment traps/sumps will require cleanout on a yearly basis.
- Swales should also be inspected during the regular ditch maintenance cycle, and may require removal of organic material buildup on the surface of drain rock as well as flushing/cleaning of below grade pipe.

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7.0 CONCLUSIONS AND RECOMMENDATIONS

The existing drainage system within the project area is not adequate to handle the design storm and snow melt as defined by this report due to lack of defined reaches and failed structures. The design of proposed drainage features for the Second and Third Street Rehabilitation Project was based off the findings of this analysis.

Bristol recommends developing a new watershed management system as outlined in this report. This system will consist of above grade roadside ditches, field inlets, road-crossing culverts, driveway culverts, and rock-filled trenches with perforated pipe.

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8.0 REFERENCES

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APPENDIX A - FIGURES

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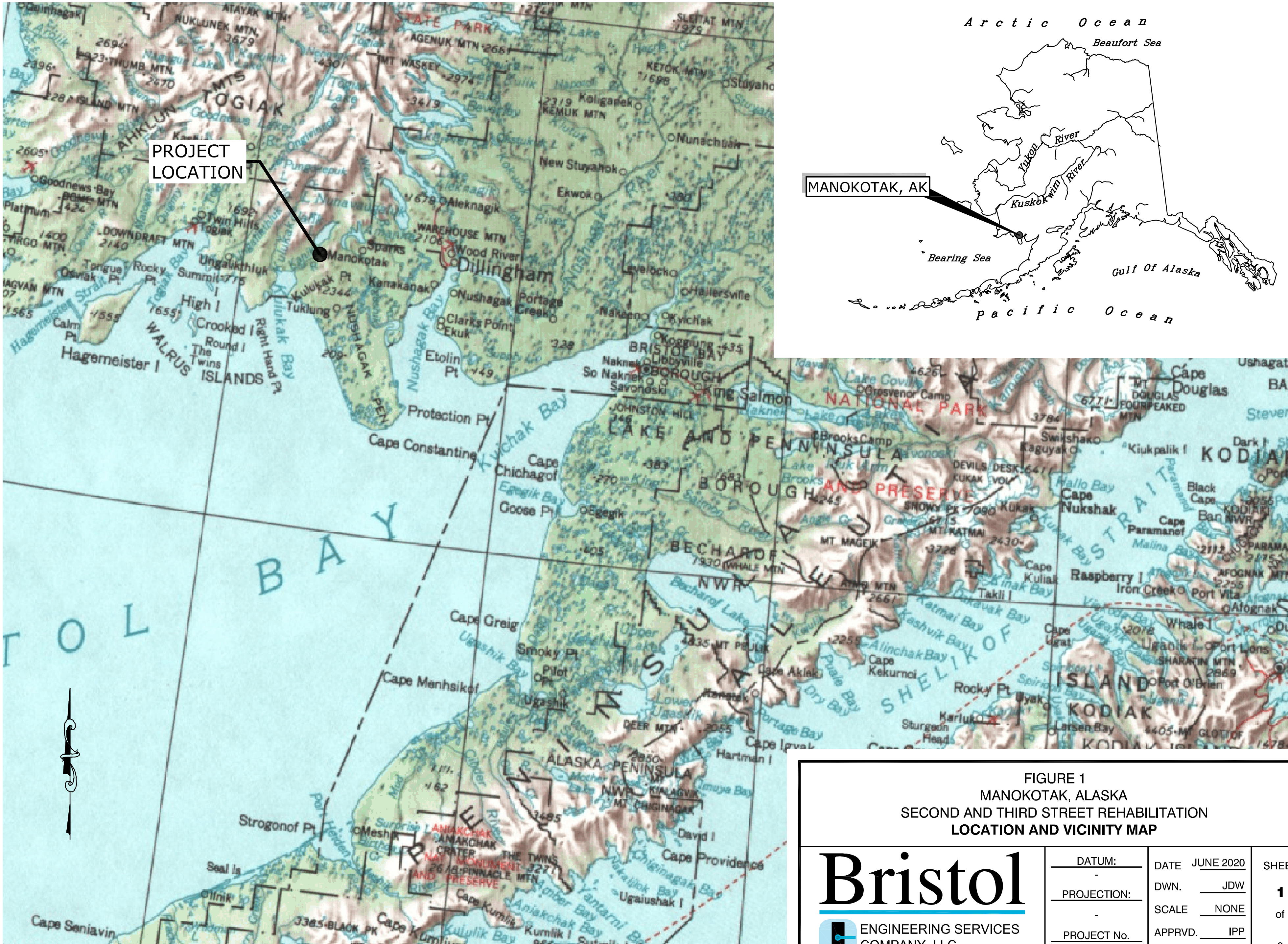

FIGURE 1
MANOKOTAK, ALASKA
SECOND AND THIRD STREET REHABILITATION
LOCATION AND VICINITY MAP

Bristol
ENGINEERING SERVICES COMPANY, LLC

DATUM: -	DATE	JUNE 2020	SHEET
PROJECTION: -	DWN.	JDW	1
PROJECT No. 32150007	SCALE	NONE	of
	APPRVD.	IPP	9

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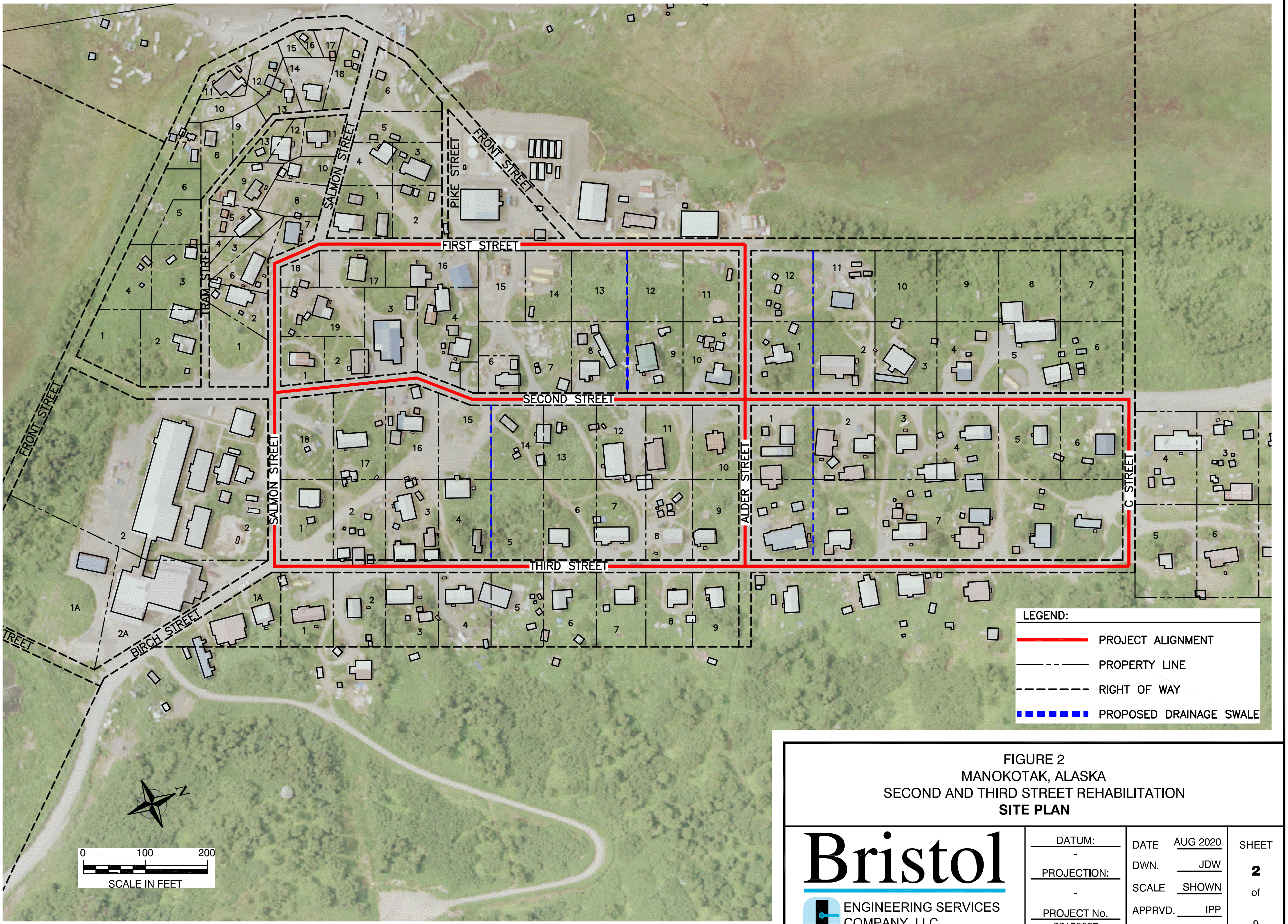
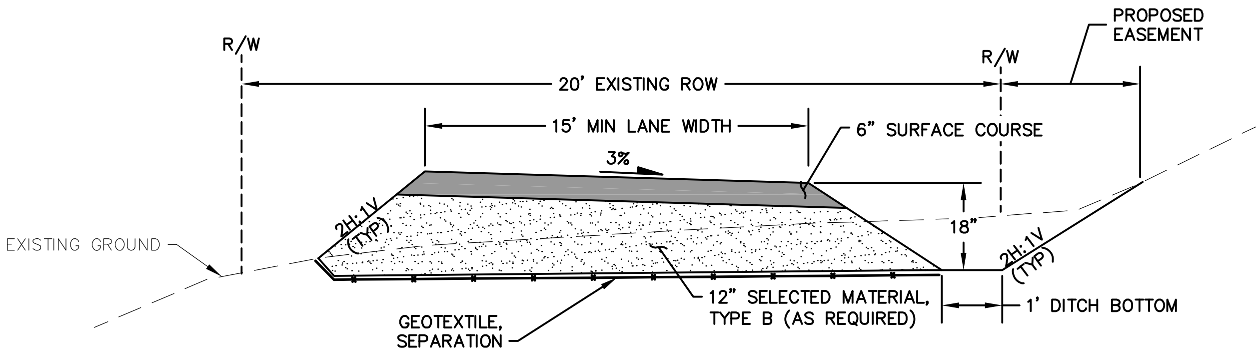


FIGURE 2
MANOKOTAK, ALASKA
SECOND AND THIRD STREET REHABILITATION
SITE PLAN

Bristol
ENGINEERING SERVICES COMPANY, LLC

DATUM: -	DATE AUG 2020	SHEET
PROJECTION: -	DWN. JDW	2
	SCALE SHOWN	of
PROJECT No. 32150007	APPRVD. IPP	9



A – TYPICAL ROAD SECTION - THIRD STREET

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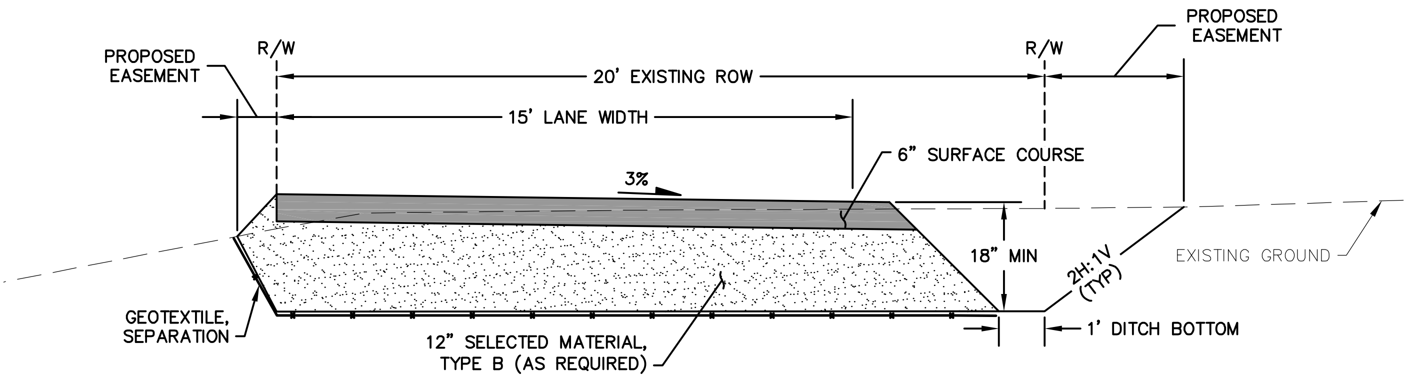
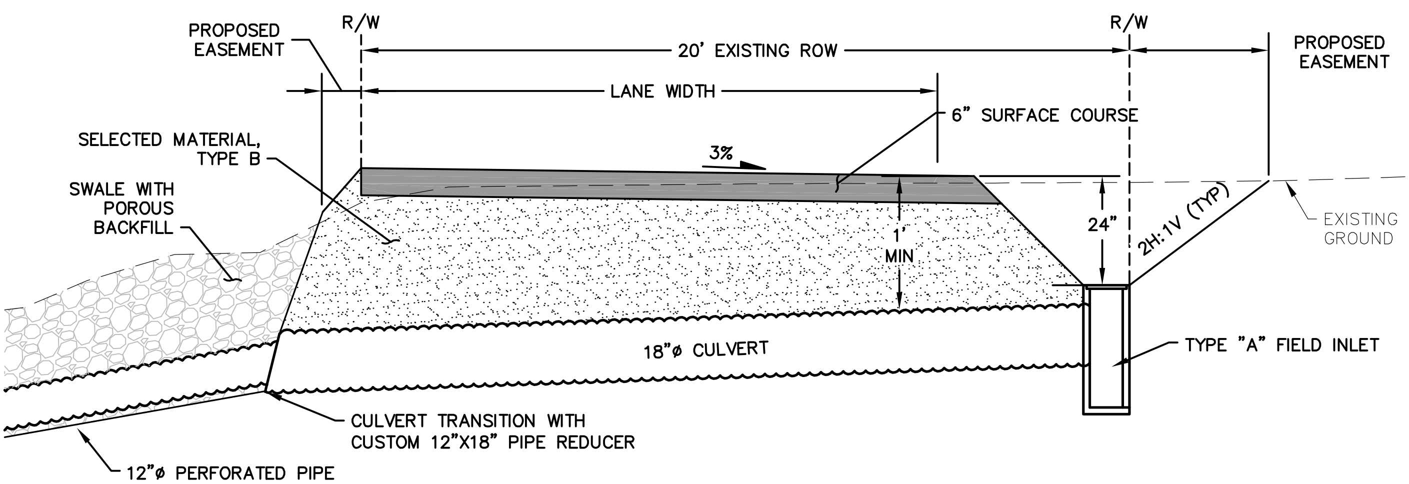


B – TYPICAL ROAD SECTION - ALL OTHER ROADS

SCALE: NTS



C – TYPICAL CULVERT SECTION

SCALE: NTS

FIGURE 3
MANOKOTAK, ALASKA
SECOND AND THIRD STREET REHABILITATION PROJECT
TYPICAL SECTIONS

Bristol
ENGINEERING SERVICES COMPANY, LLC

DATUM: -	DATE	AUG 2020	SHEET
PROJECTION: -	DWN.	JDW	3
PROJECT No. 32150007	SCALE	SHOWN	of
	APPRVD.	IPP	6

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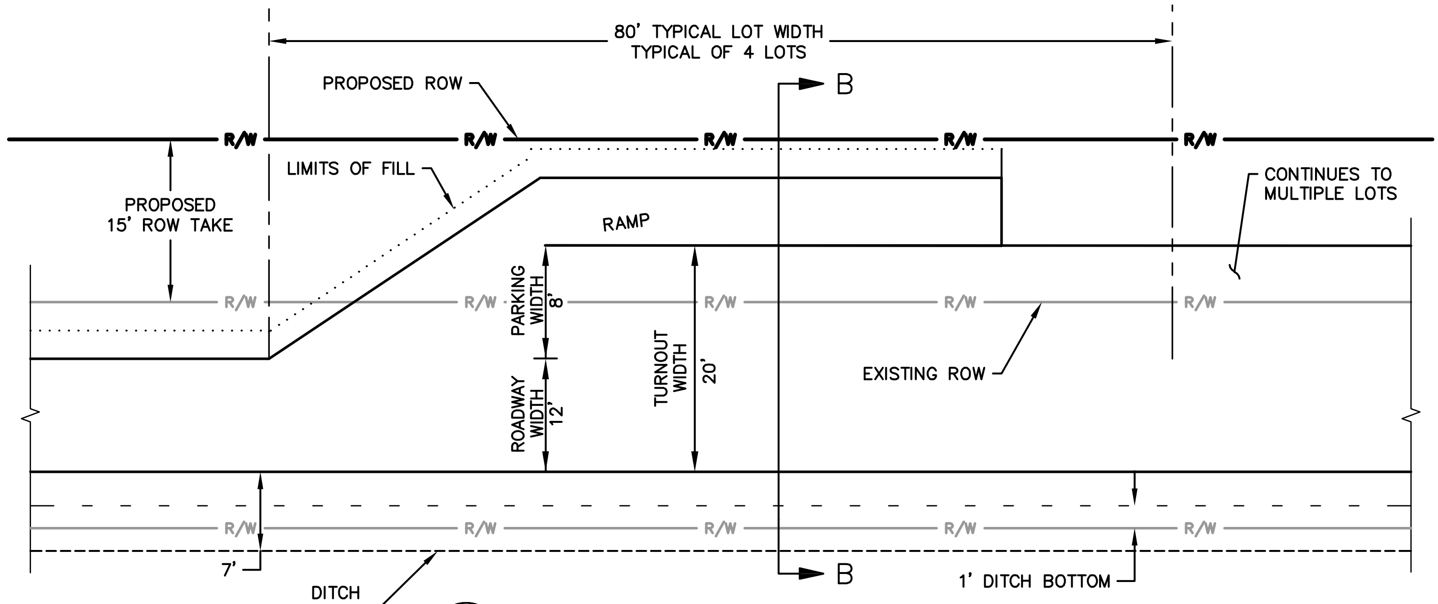
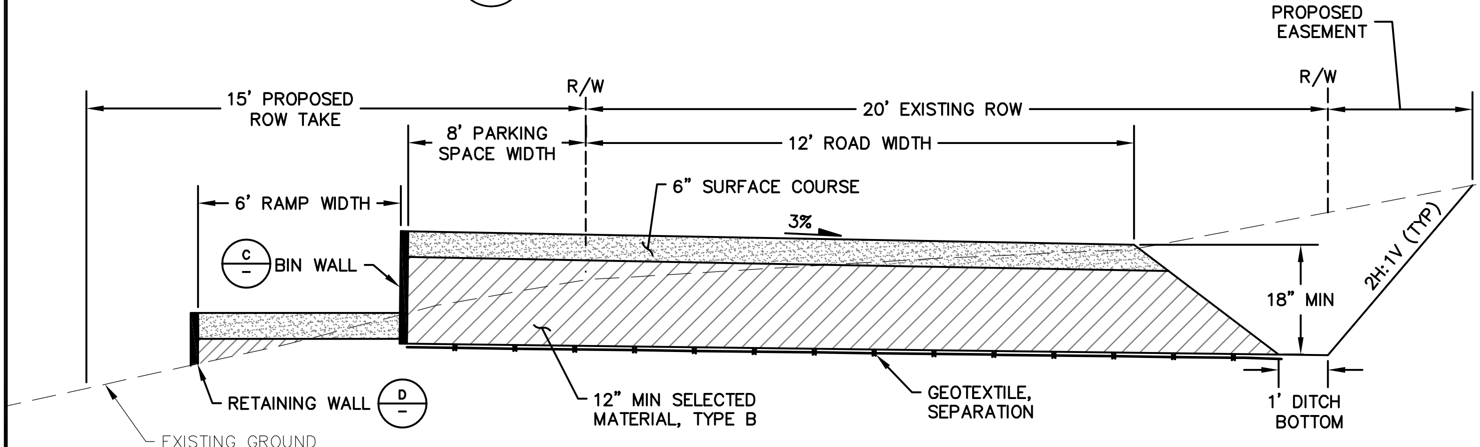


A — TYPICAL ON STREET PARKING WITH RAMP

SCALE: NTS



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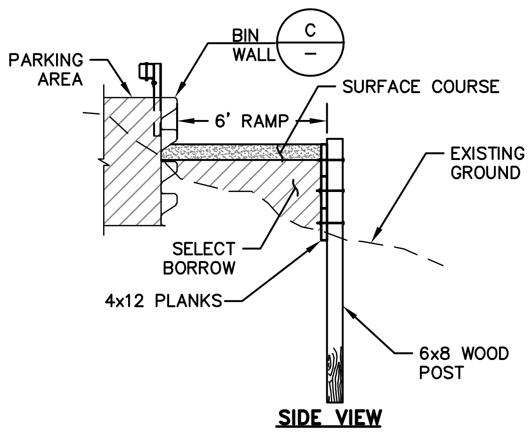
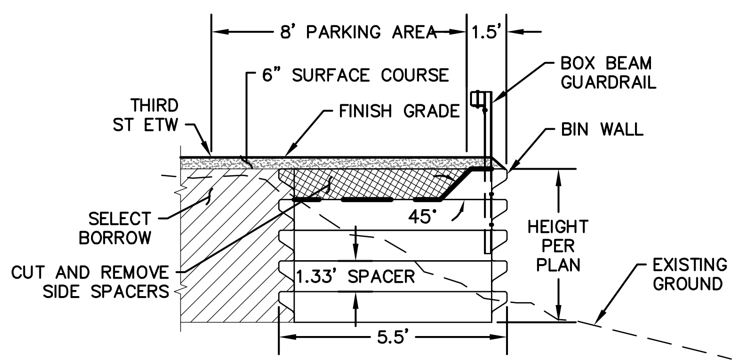
SCALE: NTS





C — TYPICAL BIN WALL SECTION

SCALE: NTS

D — TYPICAL RETAINING WALL SECTION

SCALE: NTS

FIGURE 4
MANOKOTAK, ALASKA
SECOND AND THIRD STREET REHABILITATION PROJECT
TYPICAL SECTIONS

Bristol
ENGINEERING SERVICES
COMPANY, LLC

DATUM: -	DATE AUG 2020	SHEET 4 of 6	
PROJECTION: -	DWN. JDW		
PROJECT No. 32150007	SCALE SHOWN		
	APPRVD. IPP		

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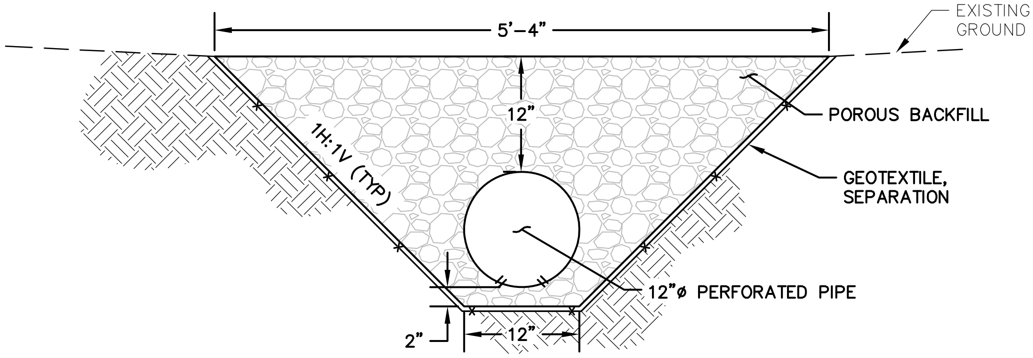
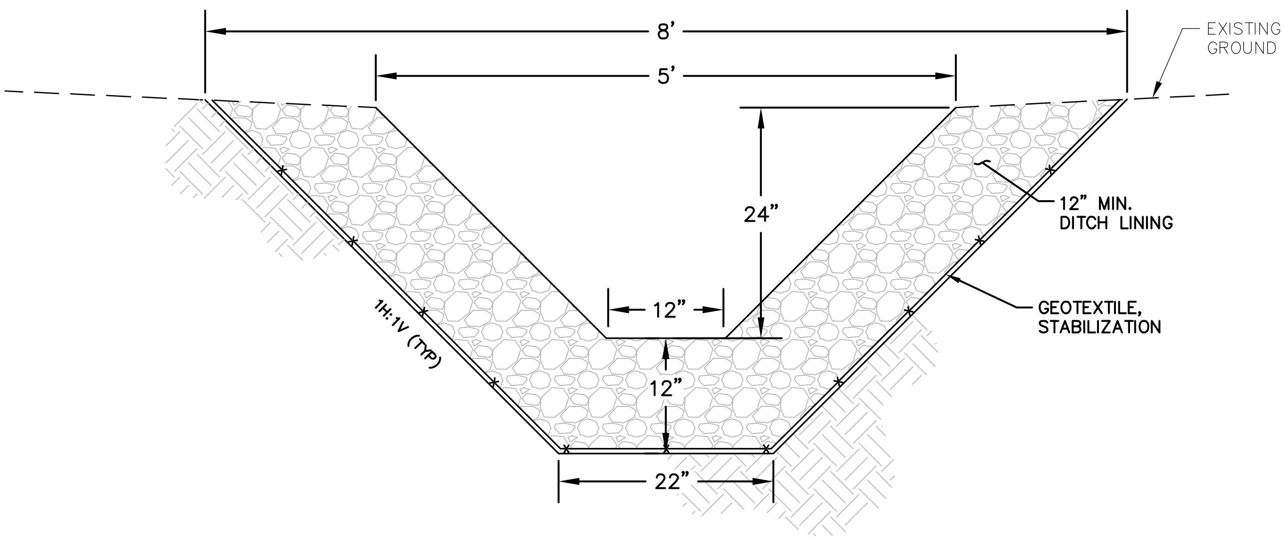


A
–
TYPICAL SWALE TRENCH SECTION WITH PIPE
SCALE: NTS



B
–
TYPICAL OPEN CHANNEL SWALE SECTION
SCALE: NTS

FIGURE 5
MANOKOTAK, ALASKA
SECOND AND THIRD STREET REHABILITATION PROJECT
TYPICAL SWALE SECTIONS

Bristol
ENGINEERING SERVICES COMPANY, LLC

DATUM:	DATE	AUG 2020	SHEET
-	DWN.	JDW	5
PROJECTION:	SCALE	SHOWN	of
-	APPRVD.	IPP	6
PROJECT No. 32150007			

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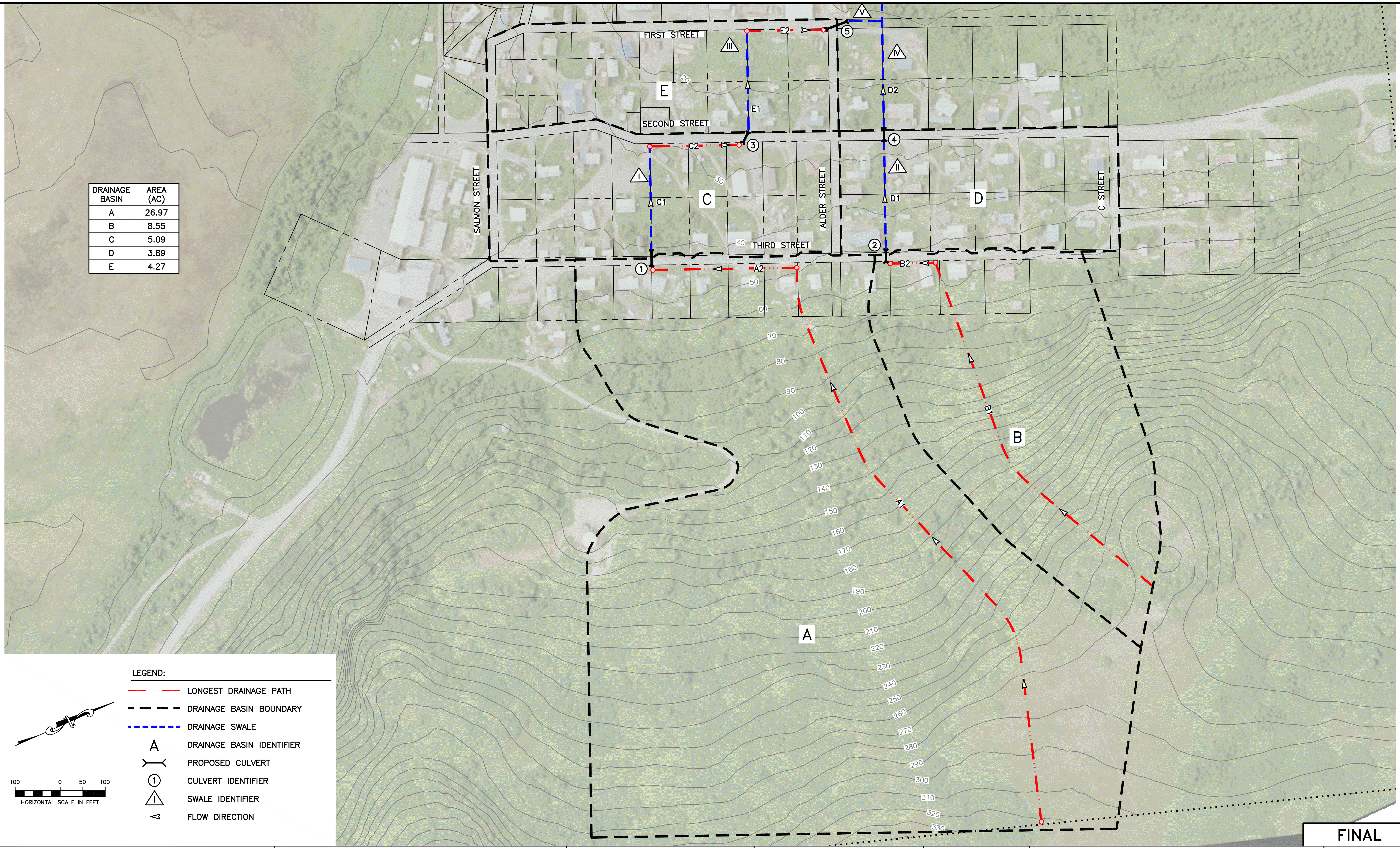

DRAINAGE BASIN	AREA (AC)
A	26.97
B	8.55
C	5.09
D	3.89
E	4.27

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REVISIONS

NO.	DATE	BY	DESCRIPTION

REVISIONS

NO.	DATE	BY	DESCRIPTION

Project No. 32150007

Bristol
ENGINEERING SERVICES COMPANY, LLC
Phone (907) 563-0013 Fax (907) 563-6713

SECOND AND THIRD STREET REHABILITATION PROJECT
MANOKOTAK, ALASKA

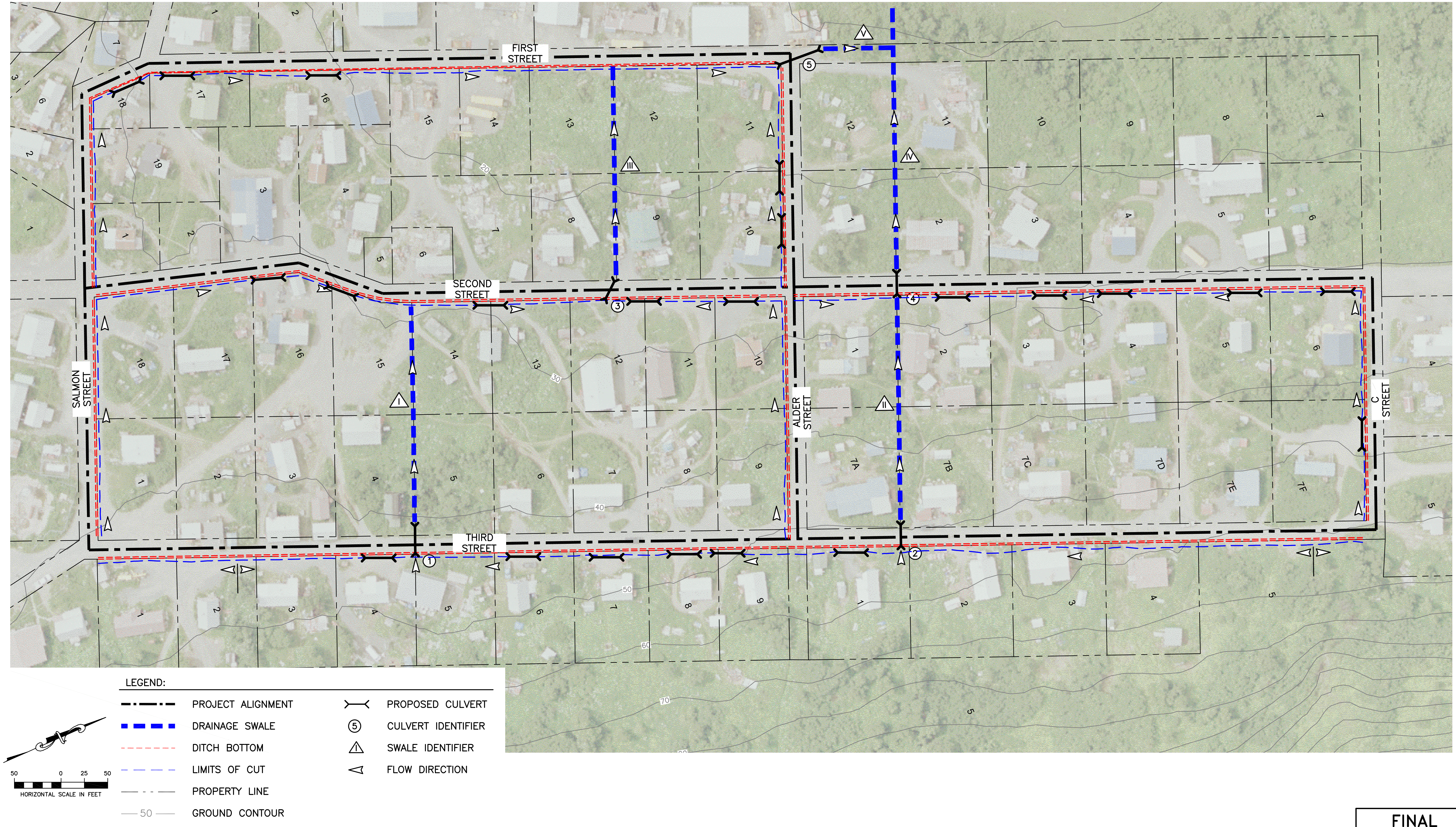
HYDROLOGY MAPPING
BASIN LOCATIONS

FINAL

SHEET NO.

FIG 7

SCALE: SHOWN | DESIGNED: JDW | CHECKED: IPP | DRAWN: JDW | DATE: AUG 2020 | SHEET 7 OF 8



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REVISIONS			
NO.	DATE	BY	DESCRIPTION

REVISIONS			
NO.	DATE	BY	DESCRIPTION

Project No. 32150007

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FINAL

SECOND AND THIRD STREET REHABILITATION PROJECT
MANOKOTAK, ALASKA

HYDROLOGY MAPPING
DRAINAGE SWALES AND CULVERTS

SHEET NO.
FIG 8

SCALE: SHOWN	DESIGNED: JDW	CHECKED: IPP	DRAWN: JDW	DATE: AUG 2020	SHEET 8 OF 8