APPENDIX I

Drainage Report

Dowling Road / Seward Highway Interchange Reconstruction

CFHWY00359

HYDRAULIC & HYDROLOGIC REPORT

FINAL

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Prepared by:

Lounsbury & Associates
5300 A Street
Anchorage, AK 99518
(907) 272-5451

Prepared for:

Alaska Department of
Transportation & Public Facilities
Central Region
4111 Aviation Avenue
Anchorage, AK 99519

NOTICE TO USERS

This report presents the preliminary results of the storm water drainage analysis performed for the proposed improvements to the Seward Highway and Dowling Road interchange. All work was performed in accordance with the most recent versions of the Alaska Department of Transportation and Public Facilities (DOT&PF) Alaska Highway Drainage Manual and the DOT&PF Preconstruction Manual, the Municipality of Anchorage (MOA) Project Management & Engineering (PM&E) Anchorage Stormwater Manual (ASM), and the National Pollutant Discharge Elimination System (NPDES) MS4 permit effective August 1st, 2015. Changes frequently occur during the evolution of the design process. Persons who may rely on the information contained in this document should consult with DOT&PF for the most current design decisions.

TABLE OF CONTENTS

FIGURES

TABLES

ABBREVIATIONS

DOT&PF -	Alaska Department of Transportation & Public Facilities
ASM -	Anchorage Stormwater Manual
ASSA -	AutoCAD Storm & Sanitary Sewer Analysis
cfs -	Cubic Feet Per Second
CMP -	Corrugated Metal Pipe
DSR -	Design Study Report
FEMA -	Federal Emergency Management Agency
ft -	Feet
H&H -	Hydrology & Hydraulics
HDPE -	High Density Polyethylene
M&O -	Maintenance & Operations
MOA -	Municipality of Anchorage
MS4 -	Municipal Separate Storm Sewer System
NPDES -	National Pollutant Discharge Elimination System
NRCS -	National Resources Conservation Service
PM&E -	Project Management & Engineering
ROW -	Right-of-Way
SCS -	Soil Conservation Service

1.0 Project Description

The Alaska Department of Transportation and Public Facilities (DOT&PF) are proposing to construct improvements at the Seward Highway and Dowling Road interchange. These improvements include a new bridge, larger roundabouts, additional capacity, and safety enhancements. Reconstructing the interchange and roundabouts will require a new stormwater network within the disturbed area of the project. This report analyzes the proposed drainage design and the effects on the existing system to determine if those effects conform to the National Pollutant Discharge Elimination System (NPDES) Municipal Separate Storm Sewer System (MS4) permit issued to the Municipality of Anchorage (MOA) and DOT&PF.

1.1 Location Map and Site Plan

The project limits extend along the Seward Highway between the 76th Avenue and Tudor Road interchanges, and along Dowling Road from the Juneau Street Right-of-Way (ROW) to Burlwood Street. The work includes portions of adjacent frontage roads Brayton Drive and Homer Drive and ramps in all four quadrants of the interchange. All stormwater network improvements are currently proposed within DOT&PF ROW. Figure 1 depicts the project Location and Vicinity Map.

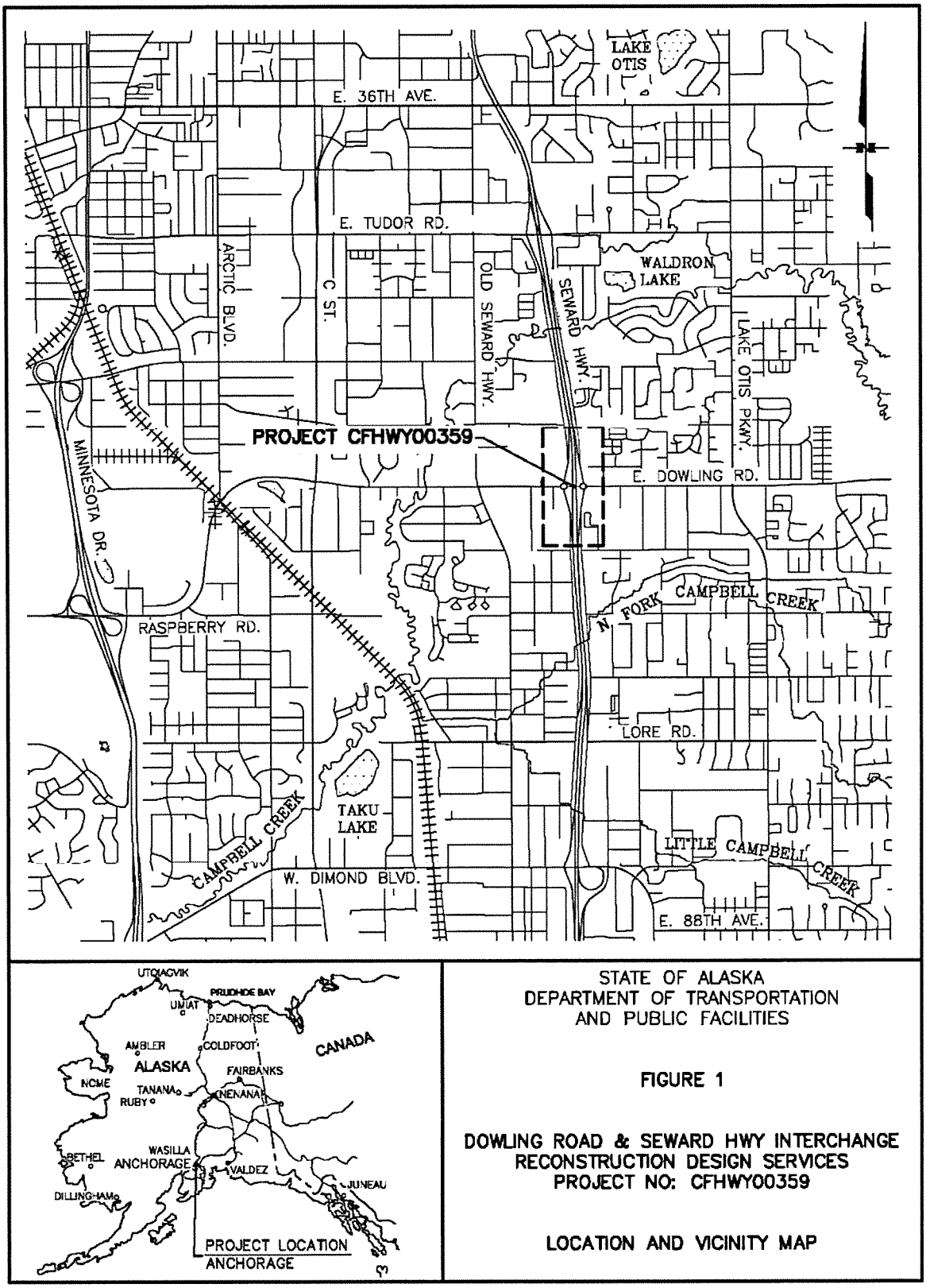

Figure 1 – Location and Vicinity Map

2.0 Drainage Basin Descriptions

This project is located within the Campbell Creek drainage basin. Stormwater in this basin is conveyed by a 24-inch diameter trunk line in Dowling Road that flows west in DOT&PF ROW, until it turns south into a 30-inch Corrugated Metal Pipe (CMP) located in an undeveloped MOA ROW called Juneau Street. The ROW runs along the east property line of the Kendall dealership on the corner of Old Seward and Dowling Road. It then heads west in 64th Avenue in a 48-inch diameter CMP to the outfall at Campbell Creek. The project is located outside a 500-year floodplain (Zone X) and there is no history of flooding for this site. This report focuses on the stormwater network in the Seward Highway and Dowling Road ROW.

2.1 Basin Size

The total contributing basin area is approximately 45.4 acres. The average basin elevation for the project is approximately 135 feet. Elevations within the Dowling Road ROW vary from 123 feet at the western project limits near the Juneau Street ROW, to 143 feet at the project limits at the eastern end. Elevations along the Seward Highway vary between 135 feet and 155 feet on the existing bridge over Dowling Road. The Existing Conditions Drainage Maps in Appendix B show the projects contributing basin.

2.2 Existing Conditions

2.2.1 Land Cover

The contributing drainage basin consists of commercial properties, roadways, grassed areas including the highway embankments and a soccer field at Polaris K-12 School, and some lightly wooded commercial lots to the northeast of the interchange. There is an existing gravel pad to the northeast of the interchange on Brayton Drive which is used for snow storage in the winter. During the spring months, some of the snow melt from this site flows into the drainage ditch along Brayton Drive and the northeast interchange ramp.

There are mapped wetlands in the northwest quadrant of the Seward Highway and Dowling Road interchange. The construction of a new ramp in this quadrant and improvements to Homer drive will result in some impact to these wetlands. The drainage design in this area after construction will reflect the pre-development conditions. Runoff from the frontage roads and ramps will be directed along the curb and gutter to the south and north to stormwater systems in Dowling Road and 56th Avenue, respectively.

Manning's Roughness Coefficients and National Resources Conservation Service (NRCS) Soil Conservation Service (SCS) Curve Numbers were assigned to the sub-basins in accordance with the Anchorage Stormwater Manual (ASM) Volume 1. Curve numbers from hydrologic soil group C were used in this analysis. Table 1 and Table 2 summarize the roughness coefficients and curve numbers which were applied within this basin:

N Value	Surface Description
0.011	Smooth asphalt
0.15	Short grass prairie
0.24	Dense grasses (lawns)

Table 1 - Mannings Roughness Coefficients for Overland and Sheet Flow

LAND COVER TYPE	CURVE NUMBER
Streets & Roads	
Paved; curbs and storm sewers	98
Paved; open ditch including right-of-way	92
Gravel including right-of-way	89
Dirt, including right-of-way	87
Other Impervious Surfaces	98
Lawn	
Steep Slopes (S>6%)	86
Moderate Slopes (2%<S<6%)	79
Flat Slopes (S<2%)	74
Natural Forest	
Fair (some forest litter)	73
Natural Brush	
Poor condition (< 50% ground cover)	77
Fair condition (50% - 75% ground cover)	70

Table 2 - NRCS (SCS) Curve Numbers

2.2.2 LAND USE

Land uses within the basin are primarily developed commercial lots adjacent to the roadway. There are two existing car sales facilities within the basin – Kendall Volkswagen Porsche Audi near Dowling and Old Seward, and Frontier Auto Sales in the southwestern quadrant of the interchange. There are also two auto repair facilities and a car rental establishment. Most of the basin acreage is impervious.

There is a soccer field in the southeast corner of the basin adjacent to the New Seward Highway north-bound off-ramp. This soccer field, the grassed highway embankments, and one undeveloped lot on the north side of Dowling Road near the western project limits account for the majority of grassed or pervious landscape within the basin.

2.2.3 EXISTING SUB-BASIN SUMMARY

Table 3 summarizes the inputs assigned to the sub-basins throughout the drainage basin considered in the analysis of the existing conditions. Weighted SCS Curve numbers were calculated by means of weighted area and use of the SCS Curve numbers mentioned previously.

Sub-basin Name	Acres	Manning (n)	Sheet Flow		Shallow Concentrated Flow		T.O.C. (minutes)	Weighted SCS Curve #
			Length (ft)	Slope (%)	Length (ft)	Slope (%)		
Sub-03	0.26	0.011	67	2	119	2	1:59	98.00
Sub-04	0.52	0.011	150	1	245	2	4:41	96.80
Sub-05	1.25	0.011	150	1	200	0.5	5:35	98.00
Sub-06	0.52	0.011	100	2	60	1	2:17	95.60
Sub-07	0.09	0.011	39	2	89	1	1:34	98.00
Sub-08	0.15	0.011	62	2	78	1	1:52	96.80
Sub-09	0.51	0.011	150	1	46	0.5	3:48	98.00
Sub-10	0.14	0.011	33	2	99	0.5	1:53	93.20
Sub-12 Subarea A	1.53	0.011	150	1	199	0.5	12:31	94.25
Sub-12 Subarea B		0.25	53	1	146	0.3		
Sub-13	0.56	0.011	31	2	234	0.3	4:13	93.20
Sub-14	0.18	0.011	87	1	-	-	2:07	98.00
Sub-15	0.59	0.011	150	1	88	1	3:59	98.00
Sub-16	0.17	0.011	29	1.6	147	0.7	2:10	95.60
Sub-17	1.66	0.011	150	0.5	144	2	5:22	95.60
Sub-20	0.74	0.011	30	1	355	0.5	6:29	92.00
Sub-21	1.23	0.011	60	1	420	0.5	6:26	98.00
Sub-22	1.41	0.011	150	0.5	235	1	6:44	94.40
Sub-23	0.11	0.011	31	1.6	159	0.4	2:49	98.00
Sub-24	0.15	0.011	40	2	169	0.5	2:49	98.00
Sub-25	1.53	0.011	150	0.5	260	0.6	7:04	89.00
Sub-26	1.13	0.011	150	0.7	132	0.7	5:04	96.80
Sub-27	0.76	0.011	150	0.6	272	0.6	6:53	96.80
Sub-28	0.75	0.011	150	0.5	30	0.5	4:39	92.00
Sub-29	1.22	0.011	150	0.5	120	1	5:17	96.95
Sub-30	0.04	0.011	54	2.9	-	-	0:56	98.00
Sub-32	0.09	0.011	30	2.5	96	0.9	1:27	98.00
Sub-33	0.94	0.011	150	0.8	102	0.8	4:30	86.80
Sub-34	0.84	0.011	22	4.3	849	0.9	7:43	96.80
Sub-35	0.06	0.011	27	1.5	78	2.2	1:08	98.00

Table 3 - Existing Drainage Basin Summary Table

Sub-basin Name	Acres	Manning (n)	Sheet Flow		Shallow Concentrated Flow		T.O.C. (minutes)	Weighted SCS Curve #
			Length (ft)	Slope (%)	Length (ft)	Slope (%)		
Sub-36 Subarea A	0.13	0.15	40	1.5	62	1.1	4:31	90.80
Sub-36 Subarea B		0.011	16	0.5	62	1.1		
Sub-37 Subarea A	0.09	0.15	42	1.8	50	1.3	4:02	89.60
Sub-37 Subarea B		0.011	14	1.6	36	2.8		
Sub-38	1.46	0.011	52	2	720	1.5	5:53	94.40
Sub-39	1.97	0.011	75	3.2	747	1.1	9:07	88.40
Sub-40	0.99	0.15	140	16	170	2.5	9:26	85.65
Sub-41	0.14	0.011	24	2.4	121	1.2	1:26	98.00
Sub-42 Subarea A	0.42	0.011	17	1.4	216	1	5:55	92.00
Sub-42 Subarea B		0.24	111	19	26	0.7		
Sub-43	2.81	0.011	150	4.8	718	1.5	8:15	83.00
Sub-44	1.64	0.24	20	10	580	1.3	8:43	78.80
Sub-45	2.12	0.011	79	2.5	811	1.2	9:36	87.20
Sub-46	0.68	0.011	46	2.5	722	0.9	7:07	98.00
Sub-47	0.19	0.011	31	1.4	192	1.2	2:14	98.00
Sub-48	0.18	0.15	33	1.4	97	1.6	7:30	89.60
Sub-49 Subarea A	0.28	0.15	74	3.2	-	-	5:32	92.00
Sub-49 Subarea B		0.011	30	0.8	125	2.4		
Sub-50	0.33	0.011	29	2.2	261	1.4	2:26	92.00
Sub-51	0.58	0.011	44	2.5	510	1.8	3:58	98.00
Sub-52	4.01	0.15	150	1	280	1	33:03	87.00
Sub-53	1.60	0.011	73	0.5	627	2.7	5:33	89.00
Sub-54 Subarea A	0.97	0.011	150	0.5	-	-	12:59	93.20
Sub-54 Subarea B		0.24	29	0.2	198	1.2		
Sub-55	0.50	0.15	150	0.5	66	1.3	36:11	74.00
Sub-56	1.10	0.15	150	0.5	170	2	37:41	74.00
Sub-57	0.56	0.15	150	0.5	110	4.7	36:01	74.00
Sub-58	0.32	0.15	150	0.5	20	0.5	35:29	74.00
Sub-59	0.33	0.15	28	3.3	173	0.1	10:25	74.00
Sub-60	0.36	0.011	35	3	296	2	2:23	95.60
Sub-61	0.08	0.011	35	3.1	102	2	1:14	98.00
Sub-62	0.42	0.011	68	0.4	152	1.7	3:47	94.40
Sub-63	0.51	0.15	51	0.4	242	1.8	17:32	83.60
Sub-64	1.96	0.011	135	3	285	1.7	3:43	89.00

Table 3 - Existing Drainage Basin Summary Table (continued)

2.3 Proposed Conditions

2.3.1 Land Cover

No changes will occur to the project area outside of the ROW boundaries. An increase in imperviousness will occur for the Dowling Road roundabouts. The roundabouts are proposed to be larger than the existing ones which will increase their paving surface area. No other significant changes to pervious or impervious land cover will occur within the corridor.

2.3.2 Land Use

No land use changes will occur within the basin as a result of this project. Offsite changes in land use will not impact the proposed stormwater network because the Municipality will require any developments to manage their own stormwater to pre-development flow rates.

2.3.3 Proposed Sub-basin Summary

Table 4 summarizes the inputs assigned to the sub-basins throughout the drainage basin considered in the analysis of the proposed conditions. Weighted SCS Curve numbers were calculated by means of weighted area and use of the SCS Curve numbers mentioned previously.

Sub-basin Name	Acres	Manning (n)	Sheet Flow		Shallow Concentrated Flow		T.O.C. (minutes)	Weighted SCS Curve #
			Length (ft)	Slope (%)	Length (ft)	Slope (%)		
Sub-03	0.26	0.011	67	2	119	2	5:00	98.00
Sub-04	0.52	0.011	150	1	245	2	5:00	96.80
Sub-05	1.25	0.011	150	1	200	0.5	5:34	98.00
Sub-06	0.52	0.011	100	2	60	1	5:00	95.60
Sub-07	0.09	0.011	39	2	89	1	5:00	98.00
Sub-08	0.15	0.011	62	2	78	1	5:00	96.80
Sub-09	0.51	0.011	150	1	46	0.5	5:00	98.00
Sub-10	0.14	0.011	33	2	99	0.5	5:00	93.20
Sub-12	1.68	0.011	150	1	204	0.5	5:37	95.50
Sub-13	0.58	0.011	30	2.1	247	0.5	5:00	94.40
Sub-15	0.61	0.011	150	1	92	0.6	5:00	98.00
Sub-17	1.66	0.011	150	0.5	144	2	5:21	95.60
Sub-20	0.69	0.011	30	1	355	0.5	6:28	92.00
Sub-21	1.13	0.011	60	1	420	0.5	6:25	98.00
Sub-22	1.37	0.011	150	0.5	235	1	6:44	94.40
Sub-23	0.21	0.011	47	2.1	162	0.5	5:00	98.00
Sub-25	2.07	0.011	150	0.8	232	0.6	6:01	90.80

Table 4 - Proposed Drainage Basin Summary Table

Subbasin Name	Acres	Manning (n)	Sheet Flow		Shallow Concentrated Flow		T.O.C. (minutes)	Weighted SCS Curve #
			Length (ft)	Slope (%)	Length (ft)	Slope (%)		
Sub-26 Subarea A	2.13	0.011	150	0.7	130	0.4	5:00	95.60
Sub-26 Subarea B		0.011	41	1.3	217	1.2		
Sub-28	0.65	0.011	150	0.5	30	0.5	5:00	92.00
Sub-29	1.02	0.011	150	0.5	120	1	5:17	95.90
Sub-52	3.77	0.15	150	1	362	1	34:40	87.00
Sub-53	1.19	0.011	73	0.5	441	3.6	5:00	92.60
Sub-54 Subarea A	0.92	0.011	150	0.5	-	-	12:58	93.20
Sub-54 Subarea B		0.24	29	0.2	198	1.2		
Sub-55	0.47	0.15	150	0.5	66	1.3	35:58	74.00
Sub-56	1.00	0.15	150	0.5	204	1.5	38:46	74.00
Sub-58	0.32	0.15	150	0.5	20	0.5	35:29	74.00
Sub-59	0.92	0.15	150	0.5	187	1.2	36:42	74.00
Sub-62	0.44	0.011	68	0.4	152	1.7	5:00	88.40
Sub-63	0.68	0.15	51	0.5	377	1.7	17:03	86.00
Sub-64	1.84	0.011	135	3	287	1.6	5:00	91.25
Sub-65	0.33	0.011	42	2.1	253	0.6	5:00	98.00
Sub-66	0.27	0.011	64	2	150	0.4	5:00	98.00
Sub-67	0.99	0.15	150	0.9	128	1.5	28:22	92.00
Sub-69	0.26	0.011	43	1.6	151	0.4	5:00	98.00
Sub-71	0.22	0.011	41	1.6	123	0.4	5:00	98.00
Sub-72	0.85	0.011	46	2	720	0.9	7:10	98.00
Sub-73	2.90	0.011	60	3.8	674	1.5	7:01	84.80
Sub-74	1.45	0.011	31	1.2	573	1.5	6:03	78.80
Sub-75	0.38	0.011	40	1.5	432	1.5	5:00	98.00
Sub-76	0.37	0.011	48	3.2	133	1.5	5:00	98.00
Sub-77	0.40	0.011	80	1.5	428	1	5:11	92.00
Sub-78	0.36	0.011	53	3.3	205	2.4	5:00	98.00
Sub-80	0.57	0.011	47	2.0	184	1.6	5:00	98.00
Sub-81	0.31	0.011	80	1.9	363	1.1	5:00	92.00
Sub-82	3.08	0.011	64	2.6	875	1	10:51	90.80
Sub-83	2.65	0.011	65	2.6	817	1.5	8:32	86.00
Sub-84	1.18	0.011	51	2	751	1.5	6:04	94.40

Table 4 - Proposed Drainage Basin Summary Table (continued)

3.0 Existing Conditions Site Plan

3.1 Seward Highway and Associated Ramps

Grades along the Seward Highway in the project corridor vary between 0.5% across the existing bridge to 2.4% on the north side of the bridge. Grassed embankments along the highway are 3:1 slopes to the green space between the associated ramps or frontage roads and the highway mainline.

The existing facilities at the Seward Highway and Dowling Road interchange were reconstructed in 2017 and 2018 to current MS4 Standards as part of the Dowling Road to Dimond Boulevard Reconstruction project. The new improvements provide pre-treatment of stormwater runoff by routing water along infiltration trenches within the internal green spaces of the ramps. A grassed swale also provides pre-treatment on the east side of Brayton Drive. Beehive inlets at the ends of the trenches/swales route this runoff to the stormwater system in Dowling Road. Check dams are also used throughout.

DOT&PF Maintenance identified ponding in the northeast quadrant of the interchange outside the roundabout and northeast ramp. This is due to the volume of runoff contributed by the snowmelt from the snow storage area to the northeast. During the Seward Highway: Diamond Boulevard to Dowling Road Reconstruction, Phase I project, a pipe and beehive inlet catch basin were installed in the ROW within the problem area. DOT&PF Maintenance reported that this inlet successfully alleviated the ponding in this location, a new cross culvert will be incorporated with the proposed interchange reconstruction to continue to convey snow storage runoff. The Maintenance site visit summary can be reviewed in Appendix A.

3.2 Dowling Road

Grades along Dowling Road vary from 1.5% east of the Dowling roundabouts to 0.3% on the west side of the roundabouts. Runoff flows from east to west in the project vicinity. Runoff is conveyed along curb and gutter and into curb inlets. All runoff is then routed through the 24-inch stormwater system to the Juneau Street ROW and then south towards the outfall at Campbell Creek.

The Dowling Road stormwater system between the Old Seward Highway and the Juneau Street ROW will not be replaced by this project; it is outside of the project impacts. That portion of the drainage network has been included in this analysis since it outfalls into the Juneau Street ROW and impacts the drainage capacity of the Dowling Road stormwater system.

Appendix B shows the Existing Conditions Drainage Maps.

4.0 Proposed Conditions Site Plan/Grading & Drainage Plan

4.1 Project Impacts

The Seward Highway is being reconstructed to accommodate six lanes of mainline traffic over Dowling Road. A new bridge and reconstructed ramps and frontage roads are required to accommodate the mainline reconstruction. Dowling Road is being reconstructed to include larger roundabout terminals at the interchange. Dowling Road is also being realigned to the north to avoid impacting the overhead electrical line that runs in an east-west orientation on the south side of Dowling Road. The realignment of Dowling Road will start at the Juneau Street ROW and continue east through the roundabouts to Meadow Street. Pavement areas will increase on the Seward Highway ramps and within the proposed Dowling Road roundabouts. Ramp locations will be realigned to match the new roundabout location. Curb locations will change which will require relocation of a majority of the stormwater system in the corridor. A total of 4,136 linear feet of stormwater pipe, 16 catch basins, 6 catch basin manholes, 10 manholes, 9 field inlets, one oil and grit separator, and 5 infiltration basins are proposed for improvements.

The existing Dowling Road stormwater system outside of the project limits is under 20 years in age and may continue to service the corridor. Appendix A contains a project site inspection memorandum that details the existing system in the late fall of 2018. Additionally, a couple existing manholes were inspected by Lounsbury in early spring of 2019. That inspection can also be reviewed in Appendix A.

4.2 Proposed Stormwater Improvements

MOA Design Storm Depths from the ASM Table 4.2-1 were used to analyze the contributing watershed. The proposed improvements will meet MOA and DOT&PF requirements for Conveyance Design, Water Quality Treatment, and Extended Detention. The proposed stormwater system decreases the peak runoff during all design storm events. Detailed discussions of the stormwater requirements are contained in Section 7.0, and hydrologic and hydraulic computations are detailed in Section 8.0 of this report.

4.2.1 Green Infrastructure Improvements

Five shallow infiltration basins are proposed within the green space surrounding the on- and off-ramp areas. The proposed basins are no deeper than 3 feet, with beehive inlets set to retain eight to 12-inches of runoff within the bottom of the basins. They are proposed to have 5:1 side slopes. All runoff contributing to these basins will flow through grassed swales or filter strips to achieve pre-treatment of road sediments.

No percolation tests have been conducted within the proposed basin areas, but previous geotechnical studies conducted at the site approximate that the water table is eight feet deep in the area which should support the proposed shallow infiltration basins. An under-drain is proposed to insure they empty fully.

The proposed basins provide the detention/retention in the system necessary to meet MOA Extended Detention goals and treatment of the half-inch Water Quality design storm. Additional details of basin design and water quality treatment can be reviewed in Sections 5.0 and 7.0 of this report.

4.2.2 WATER QUALITY IMPROVEMENTS

Runoff in Dowling Road that does not get treated in the infiltration basins will require treatment. To treat this runoff, an oil and grit separator is the proposed solution. A Rinker Stormceptor STC 7200 is a treatment unit that can fit in-line with the proposed stormwater improvements. The treatment unit can be constructed in available ROW and will include a bypass line to isolate the unit for maintenance. A gate will be installed on the bypass line to open and close it as needed for maintenance.

The Proposed Conditions Drainage Maps in Appendix C show the project basin with the proposed storm water system constructed. The system shifts north with the Dowling Road improvements but does not change the overall basin size. The figures show the drainage sub-basins, flow paths, stormwater system (proposed and existing), and infiltration basins along the project corridor.

5.0 Stormwater Controls Construction Considerations Plan

Three types of stormwater controls have been proposed to meet water quality treatment requirements for the project, these include vegetated swales/filter strips, infiltration basins and an oil and grit separator. The swales and infiltration basins will treat most surface runoff from the Seward Highway and associated ramps. The oil and grit separator will treat the Dowling Road stormwater system during the required water quality design storm.

5.1 Vegetated Swales and Grass Filter Strips

Vegetated swales and grass filter strips will provide pre-treatment for runoff before it enters the proposed infiltration basins. Appendix D contains details to construct and maintain the vegetated swales and filter strips proposed for the site.

During construction, the contractor should avoid compacting the soils in the proposed swale areas. Additionally, the swales should not be used as sediment control facilities during construction.

Maintenance of the proposed swales and filter strips include:

- routine and post-storm event inspections;
- appropriate watering and mowing;
- reseeding and vegetation replacement as necessary;
- sediment and trash removal as needed.

5.2 Infiltration Basins

The infiltration basins have been sized to treat all runoff from their contributing areas by retaining the Water Quality Treatment design storm. The infiltration basins also provide

extended detention during the 1-year design storm. The three proposed infiltration basins east of the highway overpass are designed as two feet deep, while the two basins proposed west of the highway are proposed to be three feet deep. "D4-Pond" accepts the greatest volume of runoff of the five basins.

All basin side slopes are proposed to be 5:1. The basins have been designed shallow based on the water table only being eight feet deep in the proposed basin areas. All basins will have field inlets in them set with a top elevation to retain 8 inches of runoff in the bottom of the basins. A minimum infiltration rate of 0.5 inches per hour has been assumed for the basins. Infiltration rates may be higher when the basins are fully vegetated. Under these design conditions, the basins are economical designs that will require minimal maintenance.

The site map figures in Appendix C show the infiltration basin names, locations, and shapes. Appendix D contains basic details to construct and maintain the infiltration basins proposed for the site. Figures 2 and 3 show the infiltration basin depths during the 10- and 100-year design storms. Under the proposed conditions, the basins have been sized correctly to provide retention of the Extended Detention design storm. The basins have a minimum freeboard of one foot during the 10-year design storm and do not overtop during the 100-year design storm.

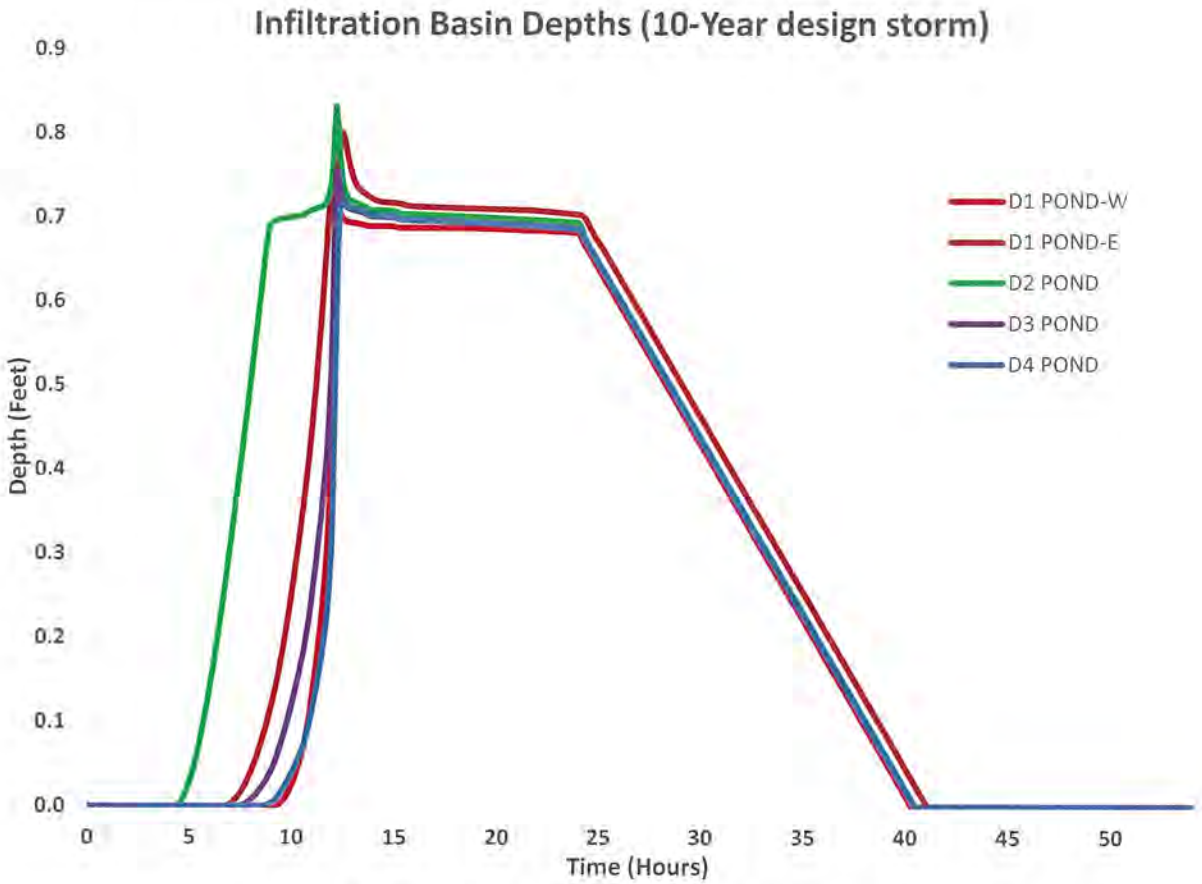

Figure 2 – Infiltration Basin Depths During the 10-year Design Storm

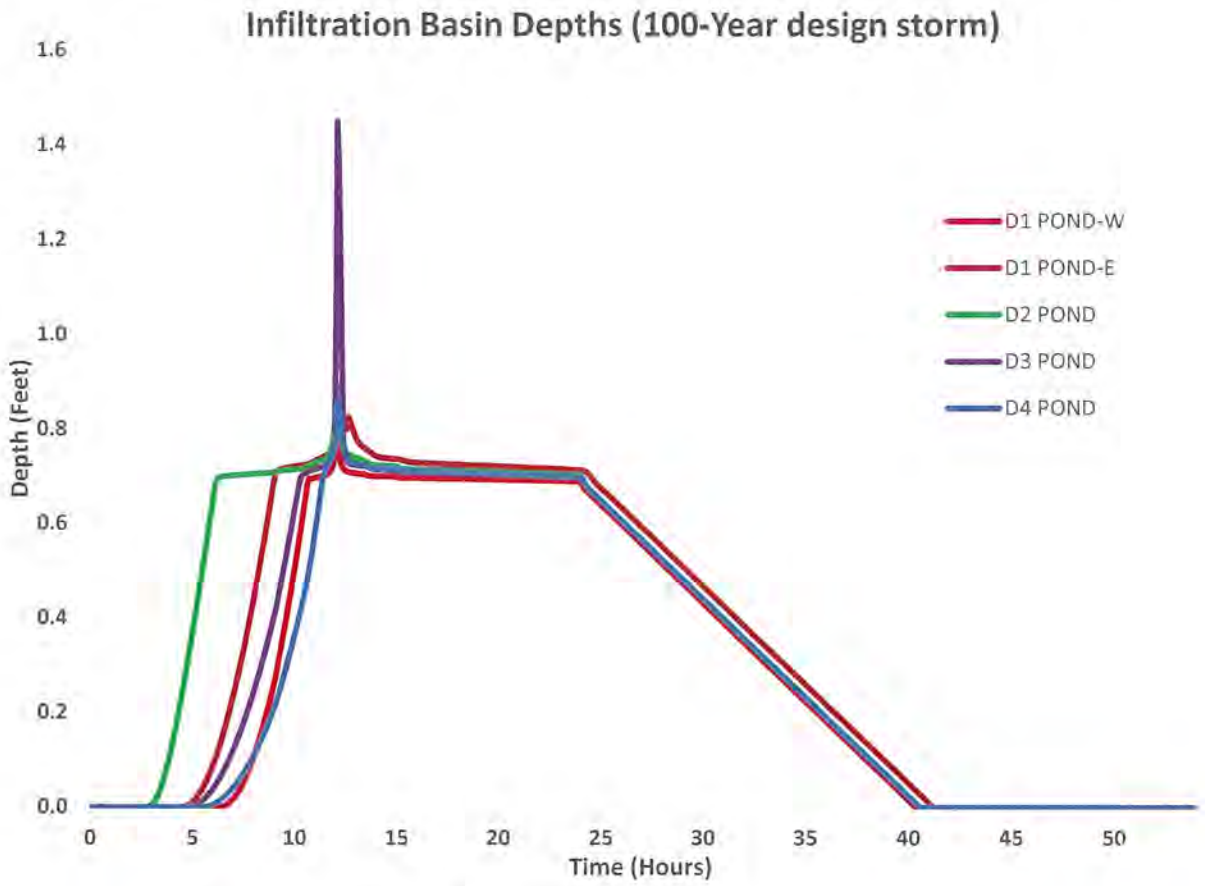

Figure 3 – Infiltration Basin Depths During the 100-year Design Storm

During construction, the contractor should avoid compacting soils in the areas of the proposed infiltration basins. Additionally, no runoff should enter the infiltration basin areas to prevent deposition and tightening of existing soils. Immediately following basin construction, the basins should be stabilized with appropriate vegetation. Additional construction details can be reviewed in Appendix D.

Maintenance will be required for proper operation of infiltration basins. The use and maintenance of the pre-treatment swales and filter strips will minimize the maintenance requirements for the basins. Sediment removal within the basin should occur when sediment is dry enough that it cracks and easily separates from the basin floor. Vegetation should be maintained to control weed growth and maintain the health of the vegetation in the basin. Additional maintenance considerations and recommended frequencies can be reviewed in Appendix D.

5.3 Oil and Grit Separator

A Rinker Stormceptor STC 7200, or equivalent, oil and grit separator is proposed to treat the Dowling Road stormwater system. Figure 4 in Section 7.1 shows the runoff hydrograph for the Water Quality Treatment design storm.

This unit can treat the peak flow during the Water Quality design storm (2.25 cubic feet per second (cfs)) and has hydrocarbon and sediment capacities large enough to minimize required maintenance for the structure. Details of the treatment unit and treatment capacities can be reviewed in Appendix E.

The project disturbance area ends where the drainage network heads south in the Juneau Street ROW. The 18-year old stormwater network south of that point will not be reconstructed. The water quality treatment unit has been sized to treat the full Dowling Road stormwater network east of the Juneau Street ROW except for the last two catch basins in front of Juneau Street. The additional treatment volume of the infiltration basins is an acceptable substitute for lack of treatment of those two locations, as they do not exceed 20% of the site's impervious areas per section 3.3.2.1 of the ASM. Other options were reviewed for treatment unit locations and orientations; see Section 10.3.2 for this discussion.

The treatment unit has been located in Dowling Road east of the project tie-in at the Juneau Street ROW. Additionally, a bypass line will be designed that allows the treatment unit to be isolated for maintenance cleaning and inspections.

No special considerations are required during construction. Regular maintenance is required. Water quality treatment units should be inspected at least annually, and sediment removed bi-annually. See Appendix E for additional maintenance details on oil and grit separators.

6.0 Soils Information

Geotechnical information from the Preliminary Structural Foundation Engineering Report for this project, dated November 2019, was used as the basis of design assumptions for the proposed infiltration basins. This design assumes the water table is 8 feet below the existing ground surface and infiltration rates will be a minimum of 0.5 inches per hour.

Underdrains are proposed within the basins to insure they empty completely. This will prevent the basins from creating saturated areas of influence that impact adjacent slopes or roadways. Additionally, the basins are located a minimum of 10 feet from any slopes or roadways.

Excerpts from the Preliminary Structural Foundation Engineering Report can be found in Appendix F. The study shows the boring locations conducted around the Dowling interchange and their associated bore logs. According to the logs, the underlying strata is a layer of peat on top of sandy silts. The water table is listed as 8-10 feet deep in the applicable borings.

7.0 Compliance with Stormwater Management Requirements

The proposed stormwater improvements comply with DOT&PF and Municipal stormwater management requirements. The following is a discussion of each requirement for the different design storms and how the project meets the targeted goals.

7.1 Requirement 1 - Water Quality Treatment

Infiltration Basins

Water quality treatment is provided by the proposed infiltration basins and oil and grit separator. To meet this requirement, treatment must be provided for runoff generated from the first 0.52 inches of rainfall from a 24-hour rainfall event preceded by 48 hours of no precipitation. The proposed water quality treatment design storm for this project totals 0.55 inches once the orographic factor of 1.05 is applied (Reference Figure 4.2-3 in the ASM).

The infiltration basins have been proposed in the available green space area of the project to treat the runoff coming from the Seward Highway and associated on- and off-ramps. Runoff that falls on the highway, in the green space and parts of the ramps is conveyed to the infiltration basins. Along the way, runoff is filtered through vegetated swales and grassed filter strips before entering the basins. Once inside the infiltration basins, runoff is consumed by vegetation or infiltrates into the ground. During the smaller rain events, most of the runoff will be consumed by vegetation within the basin.

The infiltration basins have been designed to retain water in them before spilling into the field inlets set 8 inches off the bottoms of the basins. During the Water Quality Treatment design storm, all runoff is detained in the bottom of the basins. This ensures that all runoff from the basin contributing areas is treated by not being conveyed into the Dowling Road stormwater system. Infiltration, evapotranspiration, and underdrains will safeguard that the basins empty between rain events.

Oil and Grit Separator

Because there is no available space for green infrastructure in the ROW, the Dowling Road stormwater system will be treated with an oil and grit separator near the project outfall location. The proposed treatment unit has been sized to treat the project area runoff.

The project outfall occurs where the Dowling Road stormwater network turns south into the Juneau Street ROW (Pipe EP-10). The treatment unit will be installed in a structure just upstream of that outfall (Manhole S20-4). The peak runoff for the stormwater system through that manhole during the Water Quality Treatment design storm is 2.25 cfs as shown in Figure 4. According to the peak treatment capacities for Stormceptor units (shown in Appendix E), an STC 7200, or equivalent, will be required to treat this peak runoff rate.

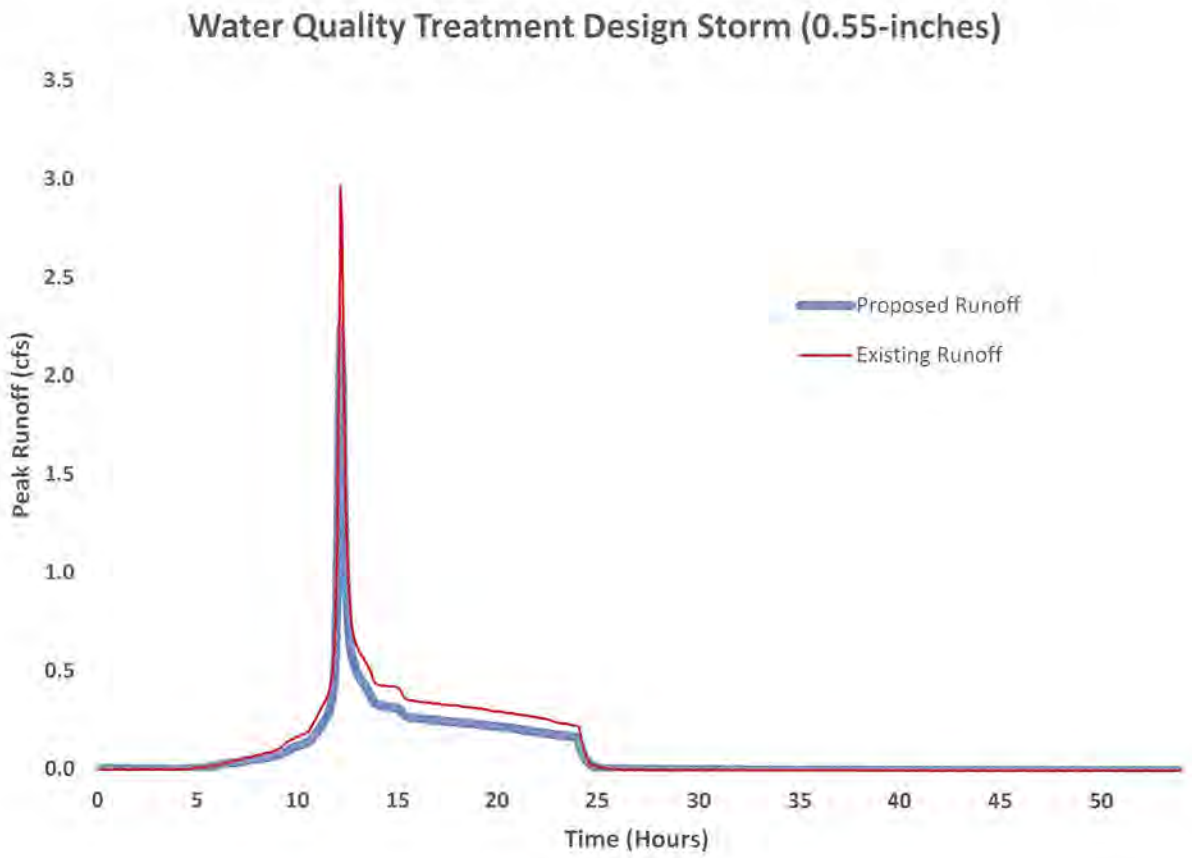

Figure 4 – Water Quality Treatment Design Storm Hydrograph

The treatment units have the ability to bypass storms larger than the Water Quality Treatment design storm internally. However, DOT&PF prefers to construct a bypass line that allows the Maintenance & Operations (M&O) crews to isolate the treatment unit and clean it without the worry of runoff flowing through the unit during maintenance efforts.

7.2 Requirement 2 - Extended Detention

As noted in DOT&PF's Memorandum regarding Department facilities within the Municipality (see Appendix G), Extended Detention is not required as part of this project. However, it should be noted that this project meets this requirement. The infiltration basins provide enough retention that the peak system outflow is reduced in comparison to existing conditions. The existing conditions peak runoff rate during the 1-year design storm is 10.4 cfs and the peak runoff rate for post conditions is 9.8 cfs.

7.3 Requirement 3 - Conveyance

10-Year Exceedance Probability

Table 1120-1 of the Alaska Preconstruction Manual was used to select the design storm for conveyance criteria. The stormwater system can be classified as a "Storm sewer feeder line" resulting in using the 10-year Exceedance Probability for conveyance design. The system conveys the storm without overtopping any manholes. Several 24-inch mainline pipes surcharge for up to 43 minutes in the AutoCAD Storm & Sanitary Sewer Analysis (ASSA) computer model. Surcharging of closed pipes is acceptable as long as surcharge heights remain at least one foot below top of structure. The proposed 24-inch HDPE pipes match the size of the existing system. Perforated 18-inch pipes will be used as underdrains within the infiltration basins. Appendix H shows the pipe profiles and maximum water depths anticipated during the 10- and 100-year design storms. Refer to the profile titled "10-year Conveyance Design Storm Profile" for a hydraulic profile of this design storm. The existing and proposed hydrograph for the 10-year design storm can be seen in Figure 5.

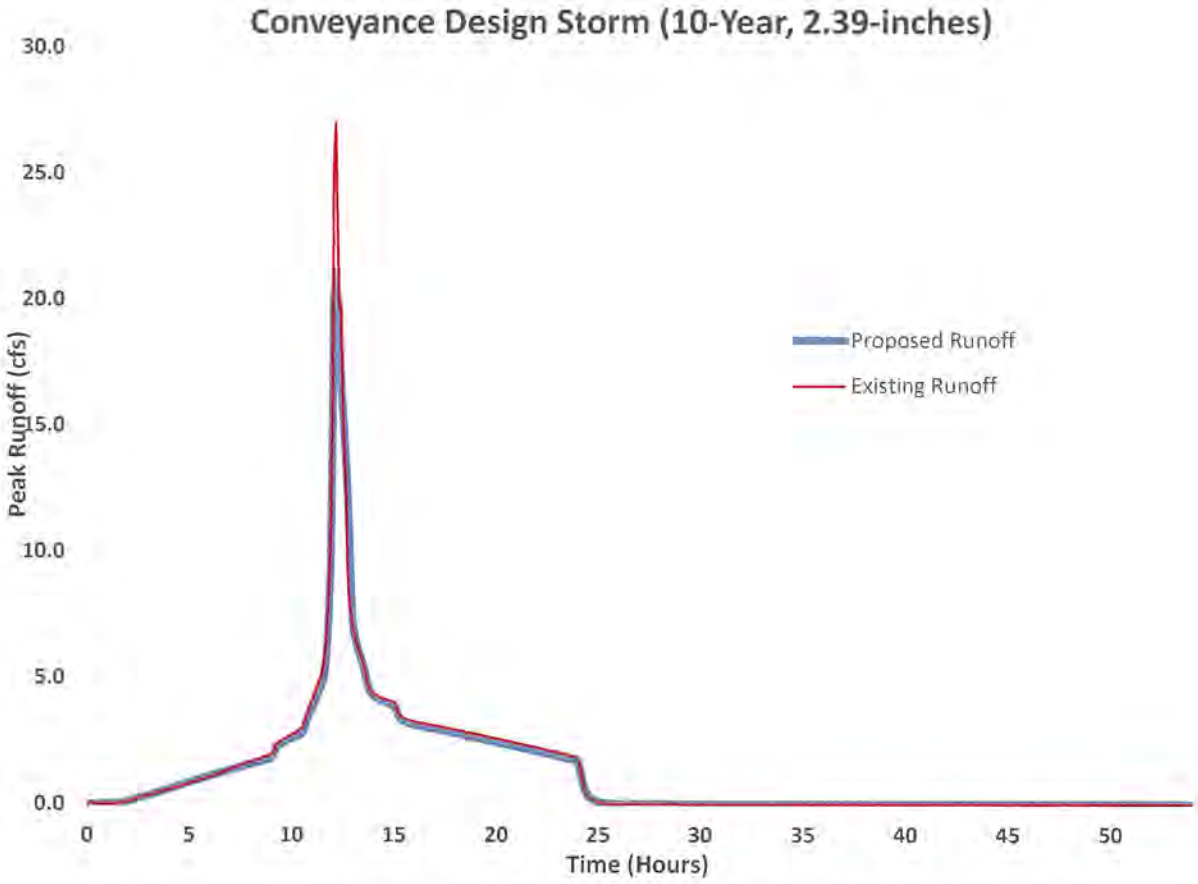

Figure 5 – 10-Year Conveyance Design Storm Hydrograph

25-Year Exceedance Probability

It was recognized that DOT&PF may want to review the 25-year Exceedance Probability if they considered the project stormwater network to be classified as a "Trunk Sewer Line" from Table 1120-1. The hydraulic profile of the Dowling Road mainline can be viewed in Appendix H. As can be seen in the profile, the proposed system still conveys the 25-year design storm without overtopping the manholes. However, the surcharge height in the manholes comes within 1 inch of the top of two structures. The 25-year design storm does not meet the surcharging recommendations noted previously for the 10-year design storm. The existing and proposed hydrograph for the 25-year design storm can be seen in Figure 6.

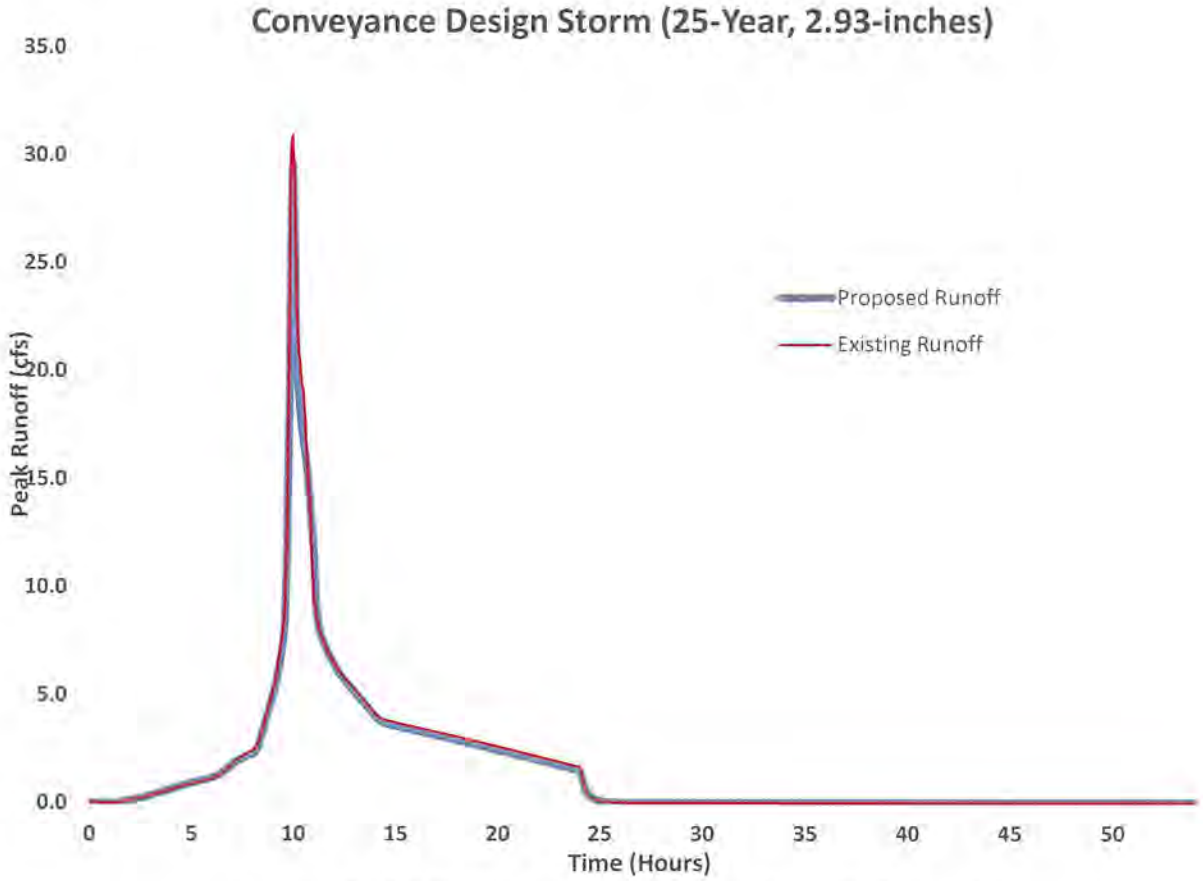

Figure 6 – 25-Year Conveyance Design Storm Hydrograph

The catch basin points outside of DOT&PF ROW on the south side of Dowling Road in Greenwood and Rosewood Streets will pond several inches during the 25-year design storm. Under existing conditions, many of the manholes in the corridor overtop the lids during this design storm. The proposed stormwater network improves the surcharging condition of the system in comparison to the existing condition.

Discussion of Upsizing Mainline Pipes

Upsizing of some of the mainline pipes to 30-inch diameters instead of the currently proposed 24-inch diameters may cause flooding of the system downstream. A Downstream Impact Analysis has not been conducted for this project, but it is known that upsizing the mainline pipes increases the peak runoff during the 100-year design storm at the project outfall by approximately 25%. This increase in peak runoff may require the downstream system to be upsized. For this reason, it is recommended to build the proposed system using 24-inch mainline pipe and not upsizing to 30-inch.

7.4 Requirement 4 - Detention and Peak Flow Control

As noted in DOT&PF's Memorandum regarding Department facilities within the Municipality (see Appendix G), Detention and Peak Flow Control is not required as part of this project. However, this project will meet the requirements for the ASM's Option 2. It decreases the 10-year design storm peak flow by 5% compared to existing conditions. As shown in Figure 7 below, the proposed stormwater system also reduces the 100-year design storm by 2.5%. Considering a 24-inch stormwater mainline, the downstream system will not be negatively impacted by the proposed improvements of this project.

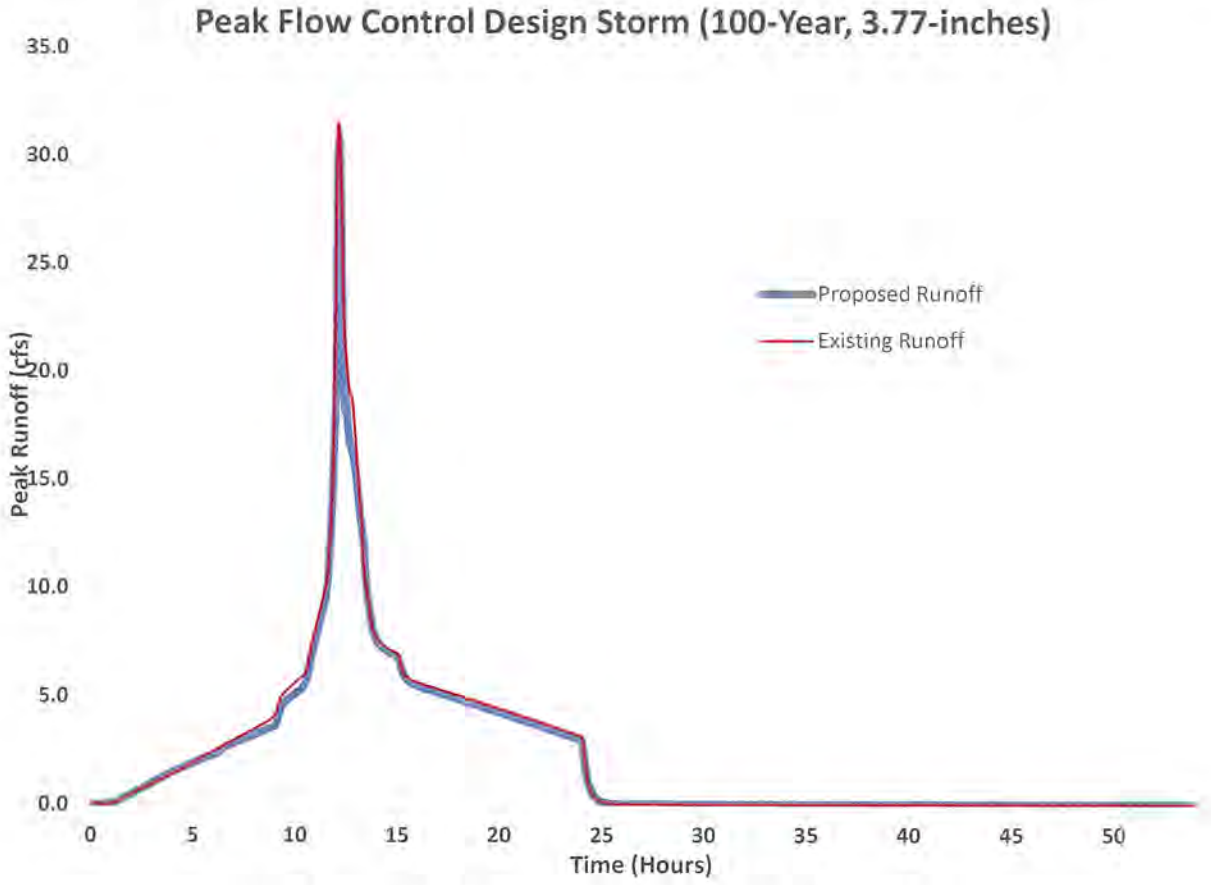

Figure 7 – Peak Flow Control Storm Hydrograph

7.5 Requirement 5 – Downstream Impact Analysis

As noted in DOT&PF's Memorandum regarding Department facilities within the Municipality (see Appendix G), a Downstream Impact Analysis is not required as part of this project. This requirement is met under Municipal code.

7.6 Requirement 6 – Wetland Compliance

Section 404 permits will be obtained for interchange roadway work that impacts the adjacent wetlands. No additional drainage analyses are necessary for wetland work because the proposed stormwater network will not discharge runoff to any wetlands downstream of the project. The contractor will protect adjacent wetlands using proper BMP's during construction.

7.7 Requirement 7 – Operation and Maintenance Plan

As noted in DOT&PF's Memorandum regarding Department facilities within the Municipality (see Appendix G), an Operations and Maintenance Plan is not required as part of this project.

7.8 Requirement 8 – Stormwater Management Report

This report has been prepared as outlined in the Municipality's ASM. It will be submitted to DOT&PF for review and approval. Any pertinent information that is required in the Alaska Preconstruction Manual for H&H Reports that is not covered in the ASM Report Outline is discussed in Section 10.0 *Miscellaneous*.

8.0 Hydrologic and Hydraulic Computations

AutoCAD Storm & Sanitary Sewer Analysis (ASSA) computer software was used to model the project area under existing and proposed conditions. Inputs for the models are tabulated in Section 2.0. Existing and Proposed Site Maps can be reviewed in Appendices B and C, respectively. The hydrology runoff, Hydraulic Routing and Exfiltration specifications used in the ASSA model can be reviewed below.

- Hydrology Method: TR-20
- Time of Concentration Method: TR-55
- Link Routing Method: Hydrodynamic
- Force Main Equation: Hazen-Williams
- Exfiltration Method: Horton, Free Surface Area

The design storms were developed using Table 4.2-1 and 4.2-3 of the ASM. Pertinent excerpts of the ASM are shown in Appendix I. Table 1120-1 of the Highway Drainage Manual was then used to choose the applicable design storms to meet DOT&PF requirements for the different stormwater evaluations. The design storms used to model the stormwater system proposed for this project can be reviewed below in Table 5.

Design Requirement	Design Storm	Exceedance Probability	Application	Anchorage and Eagle River Total Depths* (inches)
Conveyance Design	10-year, 24-hour	10 years (10%)	Storm Sewer Feeder Lines	2.39
	25-year, 24-hour	25 years (4%)	All Other Trunk Storm Sewer Lines	2.93
Water Quality Treatment	90th Percentile/ 24-hour	n/a	Green Infrastructure, Water Quality Treatment	0.55
Extended Detention**	1-year, 24-hour	1 year (100%)	Channel Protection	1.09
Peak Flow Control**	100-year, 24-hour	100 years (1%)	Peak Control/Channel Protection	3.77
*Depth after orographic factor applied from ASM Figure 4.2-3.				
**Not required per Alaska Highway Preconstruction Manual.				

Table 5 – Design Storm Summary Table

Computation Output Reports can be viewed in Appendices J and K for the different design storms. The reports detail all inputs and outputs related to the modeled stormwater system. Runoff hydrographs were shown in Section 7.0 for the applicable design storms. The hydrographs detail the before and after runoff rates for each design storm at the project outlet (Pipe EP-10 in the Juneau Street ROW).

Maximum hydraulic grade profiles can be reviewed in Appendix H for the 10-, 25- and 100-year design storms to show the amount of backwater being experienced by the stormwater system. As can be seen in those figures, the proposed system can convey the 10-year design storm without ponding in the roadway and almost conveys the 100-year design storm but experiences shallow ponding in the roadway for a short duration (3 minutes in the ASSA model).

9.0 Operation and Maintenance Plan

DOT&PF will maintain an O&M Plan for the proposed stormwater facilities. Suggested construction techniques and maintenance details were discussed in Section 5.0.

10.0 Miscellaneous

This report complies with Section 4.5.2 of the Alaska Highway Drainage Manual. Additional items that require detail are discussed as follows:

10.1 Icing Problems

Icing of the proposed stormwater facilities is not anticipated to be a problem. Infiltration Basins will overflow into the proposed field inlets if the runoff depth is over 8 inches during lower winter flows. The proposed underground stormwater pipes have adequate cover, 4 feet or more, throughout much of the project with a few exceptions. Where depth of cover is less than 4 feet, insulation is proposed over the pipe to prevent icing.

10.2 Design Flood Frequency Summary

Per Table 1120-1 of the Alaska Highway Preconstruction Manual, the 10-year design storm was used to size the conveyance capacity of the proposed stormwater network. The 24-hour design storm used has a depth of 2.39 inches. Peak flow control was also analyzed using the 100-year design storm with a depth of 3.77 inches. These storm depths were derived using the methods described in Chapter 4 of the Anchorage Stormwater Manual.

The analysis determined the system can convey the required design storm. No ponding occurs during the 10-year design storm. Minor ponding along one side street occurs within the roadway during the 25-year design storm for 8 minutes. Under existing conditions, the ponding in the side streets occurred for approximately 30 minutes. The proposed system alleviates some of the ponding in the corridor during the 25- and 100-year design storms.

Upsizing of the existing mainline was discussed in Section 7.3. Without a Downstream Analysis being conducted, it is not recommended to upsize the proposed stormwater mainline to a 30-inch diameter pipe.

10.3 Alternate Designs

10.3.1 Green Infrastructure

Infiltration Basins have been proposed on this project because they have economical construction costs and reduce the impacts to the roadway structural system. Other options are available such as underground retention systems or perforated stormwater pipes that store/infiltrate runoff during conveyance. These systems are more costly or have the ability to impact the road subgrade more than the proposed infiltration basins. Initial cost, maintenance considerations, and roadway impacts are the reason that infiltration basins have been proposed as the preferred alternative for green infrastructure on this project.

10.3.2 Water Quality Treatment Location

Several locations were reviewed before choosing the location of the Dowling Road oil and grit separator. The currently proposed location treats all of the project corridor except for the last two catch basins on both the north and south side of Dowling Road where the system turns south into the Juneau Street ROW. Catch basin oil and grit separators could be installed at those two locations to treat all of the project corridor runoff however, the proposed infiltration basins provide excess water quality treatment by retaining more than the water quality treatment design storm volume. This offsets the need to install the additional catch basin treatment units.

Placing a larger oil and grit separator unit within the Juneau Street ROW was also considered. The unit would treat the entire Dowling Road stormwater network (including the stormwater network the project does not replace) but placement of the larger unit

within the ROW is a challenge and additional ROW easements would be required. The Juneau Street ROW contains storm drain, water and sewer mains that would require relocation.

10.4 Floodplain Encroachment

No floodplain encroachment occurs as a result of this project. The project is outside of the 500-year floodplain. The applicable Federal Emergency Management Agency (FEMA) Floodplain Map can be reviewed in Appendix L. The proposed stormwater improvements maintain the existing runoff rates for the 100-year design storm and therefore, do not create floodplain impacts to the project corridor or downstream areas. Some of the side streets experience localized ponding in the roadway during the 25- and 100-year design storms. Under existing conditions, the localized ponding occurs for approximately 30 minutes. After the proposed stormwater improvements are constructed, the localized ponding occurs for 3-11 minutes. The proposed improvements alleviate some of the localized ponding issues in the surrounding roadways during the larger design storms.

10.5 Surcharged Pipes

The analysis results indicate that some mainline pipes will surcharge during the design storm events. Pipe suppliers were contacted regarding the structural integrity of pipes under surcharged conditions and the potential to require pipe banding. Suppliers responded that HDPE pipes with standard bell and spigot joints can withstand up to 10 pounds per square inch (psi) of pressure at the joints. During the 100-year flood, the maximum surcharged depth for any pipe is 9.3 feet from the invert of a 24-inch pipe. This results in a pressure of 4 psi at the invert of the pipe, which is well under the maximum design pressure.

11.0 Conclusion

The proposed stormwater network analyzed in this report meets the requirements laid out in the joint DOT&PF/MOA NPDES MS4 permit. Green Infrastructure has been proposed as infiltration basins around the highway interchange. Water Quality Treatment is achieved through use of the infiltration basins and the proposed oil and grit separator. The proposed stormwater network can convey the required design storms and does not impact the downstream capacity of the system. Multiple options are available to meet the MS4 permit requirements, but the proposed system is economically feasible and does not create saturated roadway structural sections which could reduce the design life of the project.

12.0 Hydrologic and Hydraulic Summary Table

Flood Frequency (year)	10	25	100
Exceedance Probability (%)	10	4	1
Discharge (cfs)	21.6	29.9	31.9
Water Surface Elevation (ft)	*	*	*
Anticipated Additional Backwater (ft)	0	0	0
Scour (ft)	n/a	n/a	n/a
Contributing Drainage Area: 45.1 Acres Hydraulic Capacity of Downstream Pipe (EP-10): 19.37 cfs The system surcharges during the 100-year exceedance event but experiences minimal ponding at the surface.			

*See Hydraulic Grade Lines in Appendix H.

Table 6 – H&H Summary Table

13.0 Certification

This Hydrology & Hydraulic (H&H) Report was completed by a licensed professional engineer.

Prepared By:

Scott Pulice, P.E.

REFERENCES

1. State of Alaska Department of Transportation & Public Facilities, *Alaska Preconstruction Manual*, 2012
2. State of Alaska Department of Transportation & Public Facilities, *Highway Drainage Manual*, June 2006.
3. Federal Emergency Management Agency, Map Service Center, Website: https://msc.fema.gov/portal/advanceSearch.
4. Municipality of Anchorage, *Anchorage Stormwater Manual Volume 1 Management & Design Criteria*, December 2017.
5. State of Alaska Department of Transportation & Public Facilities, *Preliminary Structural Foundation Engineering Report Seward Highway Dowling Road Undercrossing Bridge No. 1324*, November 2019
6. Contech Engineered Solutions, Stormceptor Systems, Website: https://www.conteches.com/stormwater-management/treatment/stormceptor-systems.

APPENDICES:

Submitted under seperate cover