# Introduction

This appendix accompanies ESA's 60% Design Report for the Lowman Beach Park Shoreline Restoration. The purpose of this appendix is to document our analysis and methodologies and provide greater background for the technical reviewer. For completeness, this document may repeat some information already stated in the 60% Design Report.

## Existing Conditions

### Flow Path

Pelly Creek is a low elevation coastal stream that has been highly modified by urban development. The headwaters of the creek are piped, then it emerges to flow in a semi-natural channel through Pelly Place Natural Area, approximately 1,300 feet upstream of the park and outfall, before being routed into a long (> 600 feet) pipe along the southeastern side of Murray Ave SW. It crosses under Murray Ave SW to daylight again briefly in a ditch on a small City-owned parcel across the street from Lowman Beach Park before entering the final 405-foot long pipe sequence that carries it through the park to its outfall.

### Pipe Attributes

Pelly Creek currently flows through Lowman Beach Park in an 18" diameter concrete pipe, which was installed in 1973. The approximate elevations and slopes or the existing system are shown in **Table 1**, based on the as-built drawings from the original installation (Metropolitan Engineers, 1973). The original drawings show elevations in City of Seattle Datum. The conversion between City of Seattle Datum and NAVD 88 varies between 9.1 and 9.9 feet depending on the location in the City (City of Seattle, 2014). The pipe invert at the outlet was surveyed at elevation 8.19 feet NAVD 88 in 2017, but the pipe is broken just upslope of this location and the outlet has settled along with the existing seawall, so it is unlikely that this still corresponds to the as-built elevation (City of Seattle 2017). In the absence of a surveyed reference point, we followed City guidance which recommends adding 9.7 feet to convert from City of Seattle Datum to NAVD 88 (City of Seattle, 2014). These elevations should be understood to be approximate and should be verified in the field.

**TABLE 1: PIPE SYSTEM CHARACTERISTICS (METROPOLITAN ENGINEERS, 1973)**

| Inverts (upstream to downstream) | Elevation, City of Seattle Datum (ft) | Elevation, NAVD 88 (ft) | Downstream Segment Length (ft) | Downstream segment slope (%) |
|---|---|---|---|---|
| Inlet | 10.58 | 20.28 | 6.59 | 0.25 |
| Manhole C | 10.56 | 20.26 | 72.18 | 0.26 |
| Manhole B | 10.38 | 20.08 | 101.6 | 0.26 |
| Manhole A | 10.11 | 19.81 | 119.38 | 6.77 |
| Pipe angle | 2.00 | 11.70 | 102.97 | 2.41 |
| Outlet | -0.50 | 9.20 | | |

The ditch immediately upstream of the pipe inlet is trapezoidal and appears to be manmade rather than a naturally formed channel. It has a bankfull width of 4 feet and a bankfull depth of 6 inches. The slope approaching the inlet is very shallow, approximately 0.25%. The ditch is heavily overgrown with blackberry and ivy. There are signs of erosion and naturally-formed side channels in this reach. The topography to the north of the culvert inlet is very flat, with little to no natural storage capacity at the inlet. It appeared that in any high flow event, water would flow to the north, flooding the neighbors' yards before achieving enough depth to backwater the culvert inlet.

### Sediment and Debris Load

A trash rack at the pipe inlet prevents large debris from entering the pipe. Due to the upstream pipe network and the exceptionally low slope of the first three pipe reaches listed in Table 1, it is unlikely that Pelly Creek will carry a significant sediment load to the outfall. A common design guidance for stormwater systems is to maintain a minimum pipe slope of 0.5% in order to transport sediment and prevent clogging. We would expect any available sediment load to drop out in one of the manholes before reaching the outfall. Observed creek flows have been clear during all site visits.

### Utilities and Obstructions

Slightly to the north of the Pelly Creek culvert, and at greater depth, is a 66-inch municipal storm sewer outfall that extends several hundred feet offshore. Maintaining appropriate depths of cover over this pipe, protecting it from damage during construction, erosive creek flows, and wave action were all considerations in design. Other piped utilities are concentrated in the southern portion of the park and will not be affected by this project.

There are also two very large trees growing on the top of the existing Pelly Creek pipe. Preserving these trees and minimizing disruptions to their root systems was an important consideration in design.

## Hydrology

### Drainage Basin and Land Use

The current drainage basin of Pelly Creek is approximately 0.02 square miles (15.11 acres), mostly zoned SF 5000 (single family homes, minimum lot size 5000 square feet) with some inclusions of LR1 zoning (multifamily residential development, up to 3 units per lot) (City of Seattle, 2012). The basin also contains the undeveloped, 1-acre Pelly Creek Natural Area (King County iMap, 2019). **Table 2** shows the land uses in the basin from the C-CAP database (NOAA 2016). Based on these estimates, 21% to 41% of the basin area is impervious.

**TABLE 2: C-CAP REGIONAL LAND COVER (NOAA 2016)**

| Land Use | Area (Acres) |
|---|---|
| Forest | 4.58 |
| Low Intensity Developed (21% to 49% Impervious) | 7.43 |
| Medium Intensity Developed (50% to 79% Impervious) | 2.88 |
| High Intensity Developed (80% to 100% Impervious) | 0.22 |
| **TOTAL** | **15.11** |

It is unlikely that Pelly Creek's historical watershed was very large. GIS analyses of the surrounding topography suggest a maximum historical watershed area of 0.8 square miles (512 acres), although this area may have fed several small streams. Flows from much of the basin are now piped to other outfalls.

The average annual precipitation in this basin is approximately 38 inches per year (USGS 2019; Ecology 2012)

## Flow Estimation

Pelly Creek is ungauged, and no measured flow data is available. ESA simulated flows through three different methods to estimate potential flows in the project area. The methods had a high level of agreement, and design flow was selected by compositing the results. Design flow slection is discussed in more detail in the Design section of this document.

### SWMM

Seattle Public Utilities (SPU) provided their uncalibrated SWMM model of the drainage. The model has a basin area of 17.06 acres, including some area upstream of the Pelly Creek Natural Area and the modeled drainage area is 24.5% impervious.

**SWMM Flows - cfs (SPU 2016)**

| | |
|---|---|
| 2 yr | 2.4 |
| 5 yr | 3.5 |
| 10 yr | 4.0 |
| 25 yr | 5.1 |
| 50 yr | 5.3 |
| 100 yr | 5.4 |

Other outputs of this model include a maximum flow depth of 6 inches in the pipe and a maximum outfall velocity of 10.0 ft/s. The outfall pipe never surcharged for any of the flows in the simulation period (SPU 2016).

### WWHM

ESA developed a Western Washington Hydrology Model (WWHM) of the site based on a basin area of 15.11 square feet and the land uses described above (Ecology 2012). For modeling, we used the most conservative value for impervious area in each land use class for a total of 41% impervious area in the basin.

**WWHM Flows - cfs**

| | |
|---|---|
| 2 yr | 2.7 |
| 5 yr | 3.4 |
| 10 yr | 4.0 |
| 25 yr | 4.7 |
| 50 yr | 5.3 |
| 100 yr | 5.8 |

## HY8

ESA modeled the existing culvert alignment in HY-8 and found a maximum pipe capacity of 5.8 cfs before flows began overtopping the road. Since there are no reported drainage issues in this location, this serves as an upper bound on the flows that might be expected in this system. Pipe capacity in this system is controlled primarily by the sharp bend in the pipe slope just downstream of the Manhole A. The HY8 model doesn't account for channel conditions upstream of the pipe inlet. Consequently, at higher flows, the model was predicting a submerged inlet and pressure flow through the pipe. However, as previously discussed there is no area for flow to pond deeper than 6-12 inches at the pipe inlet before beginning to flow overland, so it is not realistic that the inlet would ever backwater or experience pressurized flow. Modeled outlet flow velocities at the maximum discharge were 9.3 feet per second and flow depth at the outlet and tailwater was less than an inch.

### Design Flows

After reviewing the assembled flow data, ESA selected a design flow of 6 cfs as a conservative (high) estimate of the 100-year flow within the range of the modelled estimates. While there are is no daily flow data from which to estimate low flows, based on field observations we estimate summer low flow to be less than 1 cfs.

### Bankfull Width

We couldn't identify an unaltered reference reach of Pelly Creek where bankfull width could be measured. Instead, ESA explored a variety of methods to establish the appropriate width for the restored reach.

We first referred back to historical conditions. A 1927 survey of the Lowman Beach Park shows a highly meandering creek channel which varies from approximately 3 to 6 feet wide at top of bank. However, as previously discussed, it is likely that the historical channel served a larger watershed.

The bankfull width regression equation provided in the 2013 WDFW Stream Crossing guidelines relates bankfull width to watershed area and annual precipitation through the following regression (WDFW 2013):

Bankfull width = 0.95 x watershed area $^{0.45}$ x average annual precipitation $^{0.61}$

Based on this equation, we would expect a bankfull width of approximately 1.6 feet. In later stages of analysis, we found this dimension insufficient to contain the design flow.

Bankfull width was set at 5 feet for design based on channel hydraulics and the need for the channel to carry the design flow with a factor of safety. This is within the range observed in the 1927 survey. Channel dimensions are discussed in more detail in the Design section, below.

## 60% Design

This section refers to ESA's 60% design plans. The reader is encouraged to refer to the plans for a better understanding of the features described.

### Pipe Modifications

The daylight location for Pelly Creek was chosen base on the depth of the pipe beneath the ground surface and the desire to minimize disturbance of the existing large trees. We initially considered a daylight at the edge of the sidewalk to maximize the open channel length within the park. However, the pipe burial depth would have required substantial retaining walls at the outlet, which would be neither cost efficient, aesthetically pleasing, or safe for the public. It would also have necessitated removal of the trees.

To daylight the creek at the desired location, the existing pipe is being cut approximately 34.25 pipe-feet downstream of Manhole A and replaced with two pipe segments at a gentler slope. The new pipe directs flow to the south, away from the buried 66” outfall and towards the center of the park. A new 48” manhole will be installed at each pipe joint to enable cleanout and inspections. The first pipe segment will be 56 feet long and have a slope of 3%. The second pipe segment, leading to the opening, will be 30 feet long at a 2.5% slope, see sheet C9.

## Channel Dimensions

The channel has a 6.5-percent slope from the outlet to the start of the backshore. This slope was constrained by the location of the pipe opening and the elevation of the back beach, and could not be adjusted as part of design. Based on Manning’s Equation, a trapezoidal channel with a bottom width of 1 foot, bankfull width of 5 feet, a bankfull depth of 1 foot, and 5:3 side slopes (horizontal:vertical), would carry 6 cfs with approximately 5 inches of freeboard. This additional capacity is desirable because it allows for potential future restoration work higher in the watershed, which could return additional flows to the creek. These flows are currently piped to other outfalls. The bottom of the channel will be slightly sloped towards the center to concentrate low flows at the thalweg. Our analysis indicates that summer low flows would be one or two inches deep.

Across the back beach, the channel is expected to be very mobile and only a pilot channel will be initially graded, with the same dimensions, but a 0.2% slope. Fines will be washed into the beach sediments in this area to prevent flows from going subsurface as soon as they reach the beach. No channel will be graded into the shore face below MHHW as the creek will make its own channel in this zone.

## Substrate Sizing

Mannings Equation predicts velocities in the channel will be 5 feet/s at the 100-year flow event. The Ibash method for rock sizing yields a $D_{50}$ of 2 inches (FHWA 2012). To provide protection at high flows and the ability for the stream to shape its channel at lower flows, two layers of rock were used in channel construction. The upper 6 inches will be 4-inch diameter cobble ($D_{50}$ of 1.5 inches) with an 8-inch layer of 8-inch diameter cobble ($D_{50}$ of 3 inches) beneath. WSDOT standard mixes will be used due to their consistency and ready availability.

## Energy Dissipation

We also used HY-8 to model flow velocities at the culvert outlet, which yielded the following values for velocity and Froude number at the culvert outlet.

**HY-8 RESULTS**

| | Velocity (ft/s) | Outlet Depth (ft) | Froude # |
|---|---|---|---|
| 2 yr | 6.37 | 0.08 | 2.48 |
| 5 yr | 6.80 | 0.09 | 2.35 |
| 10 yr | 7.10 | 0.10 | 2.21 |
| 25 yr | 7.42 | 0.11 | 2.10 |
| 50 yr | 7.66 | 0.12 | 1.98 |
| 100 yr | 7.83 | 0.12 | 2.03 |

These velocities were higher than what was expected for the channel, and flow at the pipe outlet is always supercritical. To reduce energy and avoid having to oversize streambed sediment throughout the entire channel, an energy dissipation pool lined with riprap was added to the design. The system was selected to be the minimum required solution, and was designed based on FHWA HEC-14 guidance (FHWA 2006). This resulted in a pool 6 inches deeper than the channel thalweg by 4.5 feet long, with a 1.5-foot downstream apron. Flows leaving the pool were 0.3 feet deep with a velocity of 3.4 feet/s. This is lower than the channel velocity at normal depth, so it considered an acceptable exit velocity for the structure. The structure will be constructed of a 3-foot thick blanket of WSDOT rock for scour and erosion protection class A ($D_{100}$ = 18 inches, $D_{50}$ = 8 inches).

## Utility Protection

The minimum depth of cover over the 66-inch storm sewer occurs through the back beach area. Cover over the pipe in this area will be 4 feet of beach material, which is readily erodible at the design flow. To reduce the risk of exposing and damaging the storm sewer, a band of buried riprap will be installed on the back beach approximately 15 feet in front of the storm sewer. This riprap will serve as a hard line of protection should the stream channel shift to the north and cause erosion in this area. No riprap was extended into the shore face because design grade it that section involves placement of fill, rather than excavation, resulting in sufficient depths of cover to protect the pipe. Additionally, the slope in that area is expected to be naturally dynamic, and we did not want the rock to become exposed.

# Conclusions

While it isn’t possible to fully restore Pelly Creek to pre-settlement conditions, this project daylights over 100 feet of previously piped creek, provides freshwater flow across the shore face, and allows the public to experience the interactions of fluvial and coastal systems on this site for the first time in nearly 50 years.

# References

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