HIDDEN LAKE DAM REMOVAL PROJECT
BASIS OF DESIGN REPORT

SHORELINE, WASHINGTON

Prepared for
City of Shoreline Public Works Department

Prepared by
Herrera Environmental Consultants, Inc.

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Note:
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Prepared for
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INTRODUCTION

This report documents the basis of various design decisions and design details for the Hidden Lake project, inclusive of both phases of planned construction: 1) dam removal and stream restoration through the lake area, and 2) culvert replacement. This report references other supporting documentation that has been prepared specific to hydrologic analysis and hydraulic modeling, geotechnical analyses, culvert replacement alternatives analysis, and environmental critical areas delineations. Construction to remove the dam impounding Hidden Lake and restore a natural channel and adjacent floodplains amid the drained lake bed is planned for summer and fall of 2022. This construction contract will also include trail improvements along the east side of the lake area. Construction to remove the existing Boeing Creek culverts beneath NW Innis Arden Way just downstream of the existing damsite and install a larger roadway crossing structure is planned for 2024. This construction work will include extending the restored stream channel through the roadway crossing, tying into the existing channel to the south.

Separating the project into two construction contracts is necessary for funding purposes and because it would likely be impossible to complete all construction work in a single permitted in-water work period (anticipated to be July 16 to September 30). If a construction contractor were to attempt to complete all in-water work required for dam removal, draining the lake, building a new stream channel, and replacing the existing culverts under NW Innis Arden Way in a 2.5-month period of time, the extent of construction activity amid the residential neighborhood surrounding the project site would be difficult to accomplish without prolonged road closures that would generally be overwhelming for the City to manage. Dam removal and stream restoration through the lake bed area will occur first because the rate of sediment filling in the existing lake creates some urgency to remove the lake outlet piping system before it could be clogged and induce all streamflow to pass over the dam crest. The dam was not designed to operate in that manner and could be vulnerable to failure in a flood event if most or all of the streamflow passes over its crest.

PROJECT BACKGROUND

Hidden Lake is a manmade water body located in the City of Shoreline, east of the intersection of NW Innis Arden Way and 10th Avenue NW, partially within Shoreview Park (Figure 1). The installation of a dam across Boeing Creek in the early 20th century created Hidden Lake for the purpose of recreational fishing amid Boeing family estate land. Decades later the lake had filled in completely. King County built a new dam and excavated the lakebed to re-create the lake in its current configuration in the mid-1990s, and that project design explicitly included a sediment trap within the lake to allow maintenance dredging to reduce sedimentation farther downstream in Boeing Creek.

Figure 1.
Vicinity Map for the Hidden Lake Dam Removal Project.

Following incorporation as a new city, the City of Shoreline (City) took over ownership and management of the dam and the sediment trap in the lake in the late 1990s. Hidden Lake currently traps larger volumes of sediment carried by Boeing Creek than King County had anticipated in the design, and thus has required repetitive, expensive dredging projects in order to maintain it. Shoreline's City Council decided to cease further dredging of Hidden Lake in 2013 and to explore options for management of the lake area as a result of that decision. That work led to a decision to remove the dam and the lake it creates.

During the course of analyzing alternatives for dam removal and stream restoration, the Shoreline City Council supported a staff recommendation to evaluate improvements downstream of the dam that could enable fish passage (focused on salmonids) where it is currently blocked in Boeing Creek. This direction from Council resulted in an analysis of the feasibility and conceptual design requirements for removing small dams across the creek channel between NW Innis Arden Way and Puget Sound (Herrera 2017a) and analysis of options and costs for replacing the existing 48-inch-diameter culverts beneath NW Innis Arden Way with a larger, fish-passable culvert or bridge structure. Based on the findings of that work, the City decided to remove the existing Boeing Creek culverts that are buried approximately 25 feet deep beneath NW Innis Arden Way and replace them with a much larger structure designed to pass sediment and floodborne debris.

The existing culverts are estimated to be approximately 60 years old based on the general age of housing stock in the surrounding area, and assuming the roadway was constructed in that timeframe. This age is approaching the typical design life of stream culverts built in that era, and a video inspection conducted in 2012 revealed that one of the culverts has multiple cracks and small holes in the pipe, including one in the pipe invert about 58 feet downstream of the culvert inlet (J. Featherstone, personal communication June 10, 2016). Furthermore, although they have not been rated by the Washington Department of Fish and Wildlife (WDFW) or others to determine their status with regard to WDFW's fish passage barrier assessment guidance, the existing culverts appear to be at least a partial barrier to upstream fish passage because their outlets are perched above a channel lined with large riprap stones that allow streamflow to pass through the voids in the riprap (see Figure 2). The flow in both culverts is also typically very shallow, which would hinder ability for fish to swim through them.

Replacing these culverts in combination with dam removal and stream restoration through the lake area represents an opportunity to: 1) save on design and permitting costs (via having the same project team complete the design and permitting work all together), 2) condense the duration of construction-related impacts in the neighborhood, given that culvert replacement is going to be needed in the coming years regardless, and 3) greatly reduce the risks of floodborne debris blocking the existing culvert inlets following dam removal.

Figure 2. Outlet Conditions of Boeing Creek Culverts Beneath NW Innis Arden Way.

The new structure to convey Boeing Creek flows through the NW Innis Arden Way right of way is being designed according to applicable fish passage design guidance to comply with Washington Administrative Code (WAC) section 220-110-070, Section 3(a), which says:

> "In fish bearing waters or waters upstream of a fish passage barrier (which can reasonably be expected to be corrected, and if corrected, fish presence would be reestablished), culverts shall be designed and installed so as not to impede fish passage."

There are two barriers in Boeing Creek downstream of the project area that completely block upstream fish passage, and the culvert beneath the railroad tracks at the creek mouth may be a partial barrier (Windward Environmental et al. 2013). However, the standard that has been established by the Washington Department of Fish and Wildlife and several Indian tribes in the Puget Sound area in recent years is to assume downstream barriers will be replaced in the future. Therefore, it is prudent to design the new roadway crossing structure to meet fish passage design criteria to assure successful project permitting.

A fundamental focus of the project design is to successfully accommodate sediment transport through the project area following construction so that sediment management issues are no longer a concern for the City in the project reach of Boeing Creek. The resultant increase in sediment deposition in downstream reaches of Boeing Creek and at its Puget Sound beach delta is also expected to improve fish habitat downstream of the project area. Other key drivers of the project design are:

- Portions of the project area are on privately owned residential parcels. Vegetation clearing, earthwork, planting, and privacy fencing on those parcels need to be designed to address the interests of each affected property owner, since easements on each of those properties are required for construction, and in ways that minimize the burden on the City to conduct future maintenance activities on private property.

- Construction of both project phases will require bypassing Boeing Creek flow through active work areas.

- Beaver can be expected to use the project area in the long term, and design of the new stream channel and riparian plantings needs to accommodate them in ways that do not induce unwanted outcomes such as unpredictable stream channel migration on to private property.

- There are several utilities in the NW Innis Arden Way right of way, some of which must be kept operational during construction activity and all of which will have to be replaced to some extent as a result of culvert replacement construction impacts.

- Trail improvements at the northeast edge of the project site will require temporary closure of existing trails that are currently used for public access to the Hidden Lake shoreline to prevent trail users from entering the construction site. While trail construction does not involve in-water work, it will be difficult to deliver trail construction materials to the work site once drive-in access across the former lake bed is no longer possible. Thus, it is likely that most of the trail construction work will need to be finished before the contractor “retreats” from the lake bed area towards NW Innis Arden Way near the closure of the in-water work window in the first construction contract.

Existing Conditions

Boeing Creek flows into the study area from the northeast, where it is impounded by a small earthen dam that maintains a near-constant water level in Hidden Lake. Lake outflows pass through a manhole structure and two pipes buried within the dam extending from that manhole structure to a concrete pad at the entrance to two 48-inch-diameter culverts beneath the fill embankment on which NW Innis Arden Way was built. Boeing Creek continues southwest of the study area downstream of the NW Innis Arden Way crossing and drains into Puget Sound approximately 0.7 mile downstream (Figure 1). Shoreview Park and Boeing Creek Park are located northeast and east of the study area and contain trails, sports fields, forested areas, and an off-leash dog park. Single-family residential development is located south, west, and northwest of the study area.

Rationale for Construction Phasing

Several design decisions and resultant design requirements hinge on whether removing the dam and restoring a free-flowing channel through the lake area is done before culvert replacement downstream of the damsite, or vice versa. The design team evaluated whether both of these phases of construction could be combined into a single construction contract, and deemed that doing so represents too much risk and would add to overall project cost. There is very limited available project site area for construction contractor staging and heavy equipment maneuvering until the lake is drained, such that the duration of road closure and neighborhood disturbance would be greater if both phases were combined in a single construction contract. Furthermore, a limited “fish window” of time (discussed later in this report) in which construction activities can occur in contact with Boeing Creek will be imposed in the project permits, and it would be very difficult, if not impossible, for a contractor to complete dam removal, stream restoration through the lake, and culvert replacement activities at lower elevations “touching” the stream within that 2.5-month period of time. If it were attempted, there is a high likelihood that some of the construction would need to be completed the following summer, with a variety of problems and neighborhood inconveniences arising as a result of halting construction progress mid-way through completion.

Therefore, it is prudent to plan for two separate phases of construction, each of which can be completed in a single dry season (generally in the months of May through September) when drier weather and low streamflow conditions can be counted on. The City completed an analysis of alternatives and conceptual design for dam removal and culvert replacement in early 2018, which concluded that dam removal should occur as the first phase, since ongoing sedimentation of the lake poses a risk to safe and reliable conveyance of flood flows through the dam and downstream of NW Innis Arden Way. The rate of lake sedimentation has been relatively low in recent years such that this risk is not as much of a driver of construction planning as it appeared to be during the course of the conceptual design development work, such that it could be possible to have dam removal and stream restoration through the lake area occur as the second phase of construction.

There are several tradeoffs in this decision associated with timing of removal of large trees that are valued by local residents and the City in general, how streamflow could be bypassed through the construction work zone, durations of road closures and temporary lane closures, potential blockage of the existing roadway culverts by floodborne debris, and costs (including whether the first phase of construction would need to build something that would thereafter be removed in the second phase of construction). Many of these considerations are linked to the stream elevation profile under existing and proposed conditions.

The dam and the lake outflow piping associated with it create a distinct, abrupt streamflow elevation drop. For reasons described later in this report, the restored stream channel through the road crossing will be several feet lower in elevation than the existing culverts, increasing the elevation drop. If culvert replacement construction occurs as the first phase, the new culvert or bridge “opening” in the roadway crossing would need to be backfilled with soil and/or streambed material for a depth of up to 6 feet greater than the finished project configuration through the length of the new culvert or bridge crossing so that the stream elevation entering the upstream side of the new roadway crossing structure matches the downstream toe of the dam, otherwise there would be a 6-foot-high waterfall at the toe of the dam that would be a major erosion risk requiring armoring countermeasures that would thereafter be removed in the second phase of construction. As described later in this report, the selected design approach for a new stream crossing of the roadway is a wide culvert structure buried in the road embankment. An armored waterfall just upstream of the entrance to this new structure would be a risky scenario until the second phase of construction commences, if a large flood event(s) were to occur before dam removal and stream channel construction through that area removed it. If instead the new culvert structure were backfilled to a depth of up to 6 feet above the eventual finished streambed surface to prevent the need for an armored waterfall, it would be difficult and time-consuming to remove that backfill in the enclosed structure in the second phase of construction, adding considerable cost to the project. Thus, the proposed sequencing is as follows:

- Phase 1 – drain the lake, remove the dam, construct a new stream channel through the former lake bed that connects to the existing concrete pad at the upstream entrance to the existing culverts, and install a new debris rack surrounding the culvert entrance area to prevent floodborne debris from plugging one or both culverts (a debris cage on top of a lake outflow manhole on the upstream side of the dam currently serves this function).

- Phase 2 – excavate into the roadway embankment to remove the existing culverts and the debris rack installed in Phase 1, install the new culvert structure and extend the new stream channel through it to the downstream side, and excavate to lower the streambed in the vicinity of the existing damsite and culvert entrance area to tie into the streambed elevation through the road crossing.

This phasing plan does not add undue cost, and should enable the least overall project costs, and reduces risks for the City.

Overview of Report Contents

The remainder of this report contains supporting information for the following topic areas:

- Stream channel bankfull width and elevation profile requirements – the bankfull width in a "reference reach" of Boeing Creek is a basis for sizing the new roadway crossing of Boeing Creek, which in turn affects grading of the new stream channel through the lake bed area; the existing stream channel dimensions and bed elevation at the upstream and downstream ends of the project area dictate the design configuration at these "tie in" points.

- Culvert replacement design configuration – design requirements for fish passage; bridge and culvert options considered; selected structure size and material; new structure foundation requirements; retaining walls needed to limit cost and encroachment on private property; and associated considerations for earthwork and maintaining operations of various utilities in the road right of way.

- Hydrologic analysis and hydraulic modeling – these results inform design of the new stream channel, log structure anchoring requirements, predicted inundation of restored floodplain areas that informs the design of wetland mitigation areas and riparian plantings, and flow bypass requirements during construction.

- New stream channel alignment and cross-sectional geometry, including retaining walls on each side of the stream in close proximity to the new roadway crossing structure.

- Flow bypass during construction – design requirements, options considered in the process of developing the bypass features shown in the preliminary design plans for each phase of construction, and considerations for construction contractor flexibility for means and methods of bypass system installation and operations.

- Temporary bypass of utilities in the NW Innis Arden Way right of way during culvert replacement construction of the – focusing mainly on maintaining sanitary sewer flow conveyance from east to west through the construction work zone.

- Trail design configuration to minimize environmental impacts and connect with existing trail.

Figures 3a and 3b show the proposed project improvements for the two phases of project construction: Hidden Lake Dam Removal and Boeing Creek Culvert Replacement.

Figure 3a.
Hidden Lake Dam Removal Project.

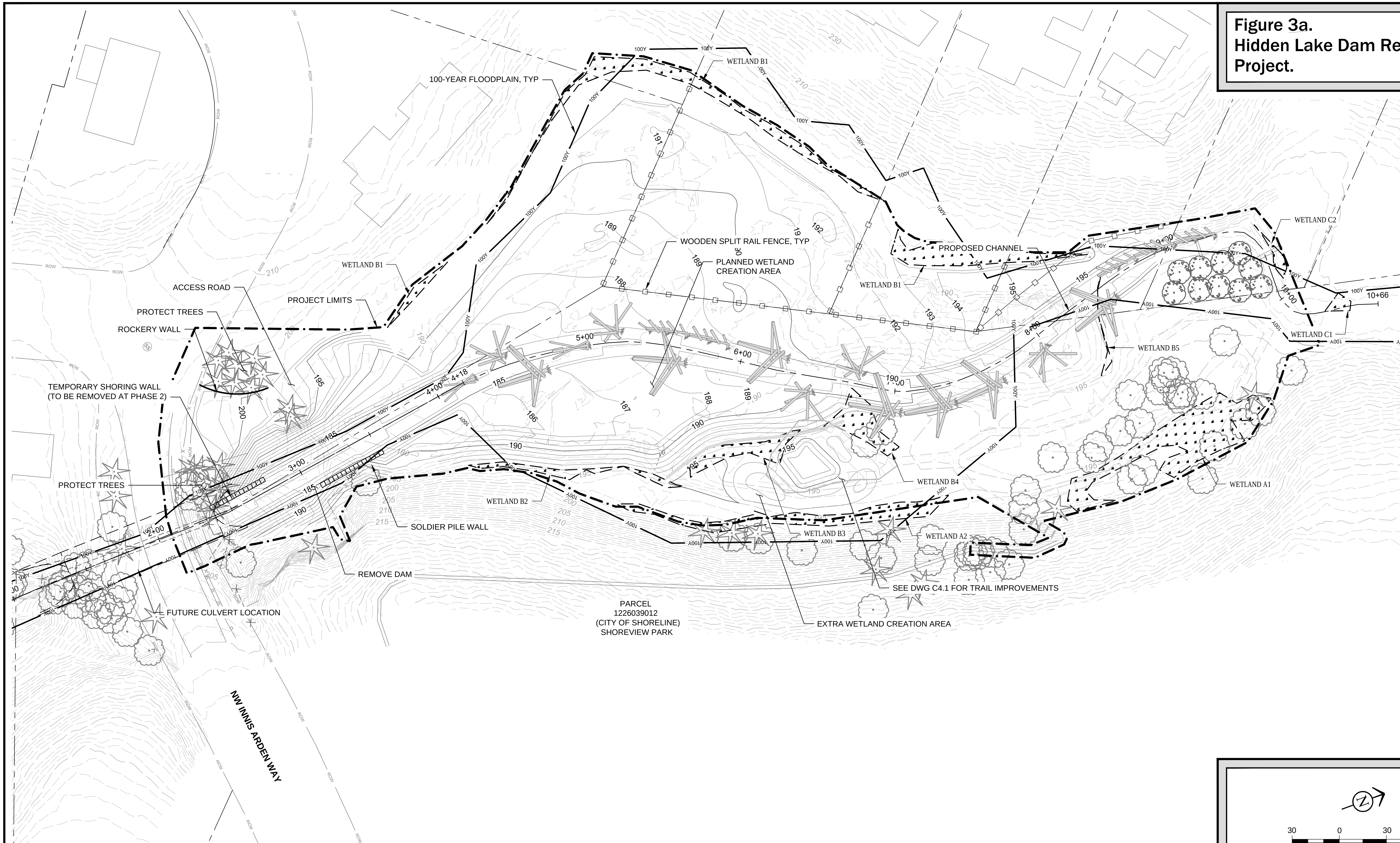

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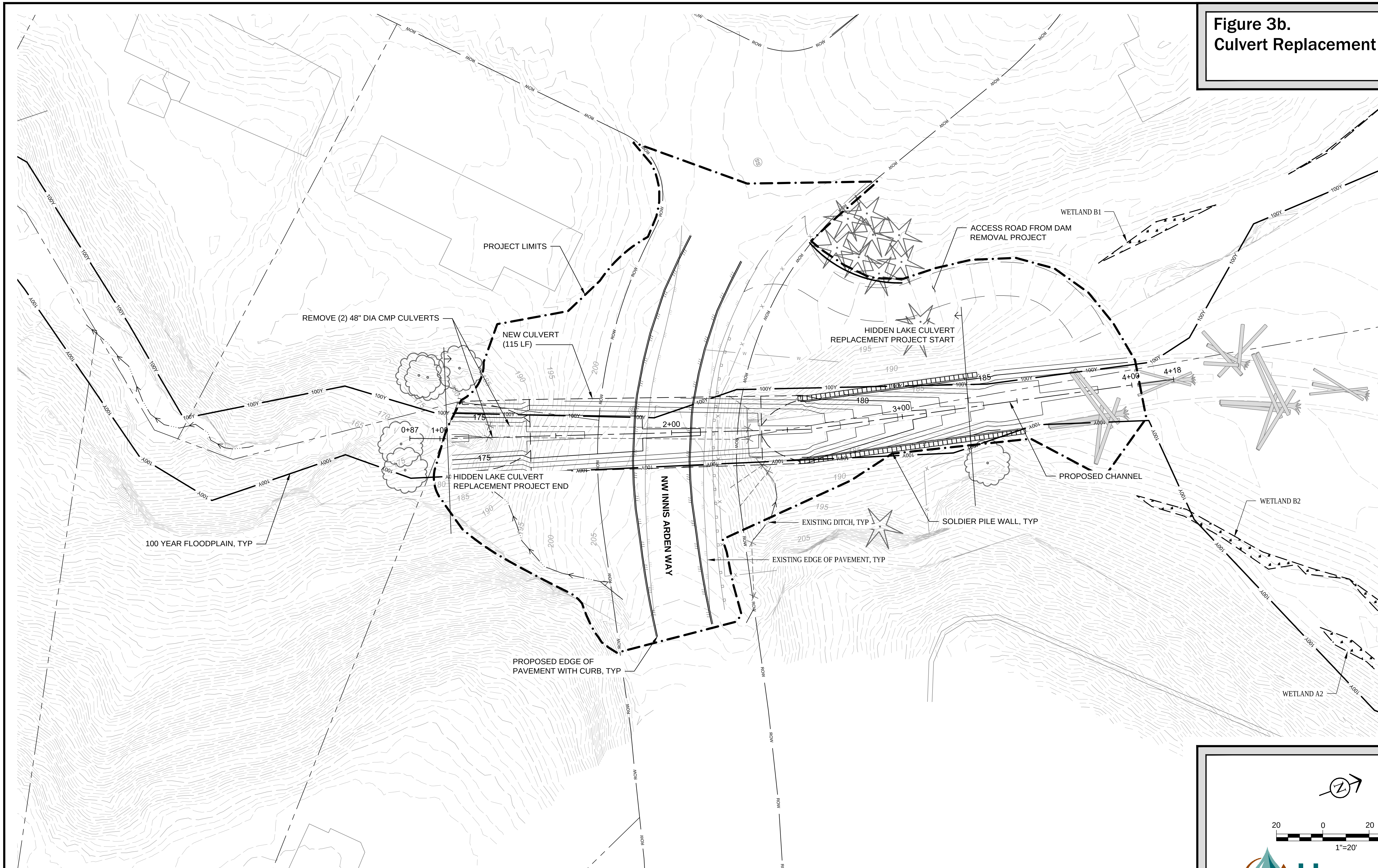
Figure 3b.
Culvert Replacement Project.



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STREAM CHANNEL BANKFULL WIDTH DETERMINATION AND SELECTION OF UPSTREAM AND DOWNSTREAM CHANNEL MODIFICATION LIMITS

BANKFULL WIDTH DETERMINATION

The bankfull width of Boeing Creek in a nearby, relatively undisturbed location provides the basis for defining the minimum width of a new culvert or bridge structure per the "stream simulation" design approach in the Water Crossing Design Guidelines (Barnard et al. 2013) prepared for WDFW's Washington State Aquatic Habitat Guidelines program. Project design team representatives conducted a site visit with Larry Fisher of WDFW on December 17, 2018 to visit a reference location in Boeing Creek in which to measure bankfull widths. Upstream of Hidden Lake the creek is unnaturally wide due to extensive erosion and past anthropogenic influences. Downstream of the project area the creek flows amid a forested ravine that is much more natural in character, but there is notably more elevation drop through this reach than there is in the project area and it is influenced by two manmade dams spanning the channel that are anadromous fish passage barriers (Herrera 2017a; Windward et al. 2013). One of those is a sheetpile dam that was historically used to impound flow for a diversion pipeline for Seattle Golf Club irrigation water supply. This dam is located approximately 900 feet downstream of NW Innis Arden Way. Roughly 300 feet downstream of NW Innis Arden Way there is a steep rock cascade made of riprap that was clearly placed to block the natural channel, for unknown reason(s) on private property. Both of these dams have altered the stream channel upstream of it (via the channel filling in with sediment) and immediately downstream of it (via plunge pool scour).

The stream channel gradient between the outlet of the existing culverts on the downstream side of NW Innis Arden Way and the downstream side of the Seattle Golf Club dam averages about 9 percent (Herrera 2017a), inclusive of the dams, which is far steeper than the new stream channel can possibly be within the project area, therefore this reach of the creek was excluded from consideration for bankfull width measurements. Also, Barnard et al. (2013) recommend avoiding reaches that have significant geomorphic barriers like those found in this reach and in the vicinity of Hidden Lake.

Thus, a reference location downstream of the Seattle Golf Club diversion dam was selected for bankfull width measurements. This location is shown in Figure 1. The stream channel gradient is approximately 2 percent in this area (Herrera 2017a). This gradient is comparable to the channel gradient approaching the upstream end of Hidden Lake, and is equivalent to the proposed stream channel gradient to be constructed through the project area except for one short section of steeper channel in the existing dam area.

The streamflow that passes through the reference reach is slightly different than passes through the project site due to additional drainage basin area that contributes runoff to Boeing Creek downstream of Hidden Lake, and also due to the limited flow attenuation that occurs in Hidden Lake. The bankfull flow depth generally corresponds to a 2-year recurrence peak flow (Barnard et al. 2013). The modeled 2-year peak flow in Boeing Creek where bankfull width measurements were obtained is 73.2 cubic feet per second (cfs), whereas the modeled 2-year peak flow in Boeing Creek where it approaches the upstream end of Hidden Lake is 72.1 cfs (Windward et al. 2013). These flow estimates, combined with similar channel gradient as will be constructed by the project, indicate that the selected location for bankfull width measurements is near optimal.

Four separate bankfull width measurements were obtained, in accordance with guidance presented in Appendix C of Barnard et al. (2013) for field indicators of bankfull conditions on the stream banks. Four different stream channel cross-section locations were measured. One of these locations was anomalously wide due to the local influence of a channel-spanning logjam and was subsequently discarded. The average of the other three measurements, 18 feet, was deemed an appropriate basis for reference bankfull width as documented in an email exchange with Larry Fisher a few days after that site visit. A copy of that email exchange is provided in Appendix A, along with photos of the channel where bankfull measurements were taken.

A bankfull width of 18 feet is therefore applied to analysis of the replacement crossing structure at NW Innis Arden Way. Hydraulic modeling described later in this report was a primary basis for sizing the new channel geometry upstream of the new structure.

Selection of Upstream and Downstream Project Limits

Upstream Limit

Due to the existing, relatively natural stream conditions at the upstream end of the lake, the upstream limit of project construction was selected to minimize impacts to upstream habitat conditions and stream gradient. Input from adjacent property owners was also taken into consideration with regard to any changes in stream channel alignment at the upstream end of the project area. The channel gradient approaching the lake from the northeast is consistent with the proposed grade of the new channel to be constructed through the existing lake bed (described later in this report). The upstream project limit coincides with the location where a temporary cofferdam will be constructed to bypass streamflows through the construction site in the existing lakebed, and the associated disturbance that cofferdam installation and removal will cause.

Downstream Limit

In July 2017, the project team completed a memorandum titled Concept Design Evaluation of Fish Passage Improvements in Lower Boeing Creek, for a roughly 1,100-foot-long reach of the creek from NW Innis Arden Way to downstream of the Seattle Golf Club diversion dam (Herrera 2017a). Results of this analysis indicated that successful implementation of lower Boeing Creek fish passage improvements would be very difficult as a City-led project, with extremely high costs, substantial risks, and many uncertainties. City staff concluded that such an approach would not be viable for restoring fish passage in lower Boeing Creek in conjunction with the Hidden Lake Dam Removal Project.

However, the Hidden Lake project design should not preclude potential future fish passage improvements downstream. The stream gradient downstream of the existing culverts increasingly steepens with steep ravine side slopes immediately adjacent to the channel. These conditions would require significant bank re-grading work (on private properties) to construct a stable, fish passable stream channel, with increasing difficulty of bank re-grading the farther downstream the connection point is into the existing stream bed. The downstream project limit was selected at the location shown in Figure 4, coinciding with a stable section of the existing channel bed and relatively wider banks, and also consistent with the reach scale gradient to support fish passage (Herrera 2017a). Riprap placed by King County in the past for channel stabilization forms somewhat of a sill at the outlet of a plunge pool at the existing culvert outlets. The plunge pool will be eliminated in the project design but this sill creates a durable tie-in point for the modified stream channel emanating from the new roadway crossing structure.

Figure 4. Downstream Project Limit.

CULVERT REPLACEMENT ALTERNATIVES ANALYSIS

For a bridge spanning the creek ravine to be viable, given realistic funding the City has access to for constructing the project, it would need to fit within the existing right of way limits to preclude the need for purchasing additional right of way. Additionally, the NW Innis Arden Way crossing of Boeing Creek is within a horizontal curve in the roadway alignment with limited driver sight distance. If a new bridge were constructed at this site, it would have to meet City standards, which would invoke improved sight distance requirements. Upgrading the roadway to current City standards would not be required if the road surfacing and width is rebuilt to match existing conditions over the top of a new culvert structure. In addition, due to the depth of the existing culvert, a new bridge with 2H:1V embankment slopes (per geotechnical engineering design recommendations described later in this report) on the sides of the stream channel would require a bridge span of about 140 feet. That bridge length would need to be curved to accommodate the curved roadway alignment. Access within the constrained site to construct a bridge of this length would be difficult and time-consuming. The estimated cost of a new bridge and modified approach roadways on each side of it is on the order of $1,400,000 to $1,600,000, not including the costs of excavating to remove the existing culverts and constructing a new stream channel through the crossing, the costs associated with a long duration of road closure for construction and temporary utilities maintenance in the work area, or the costs of “hanging” several utilities on the new bridge structure. Bridge construction-related costs would equate to at least $1,000,000 or more in total project cost compared to other structure options that were evaluated and described below. That added cost exceeds what the City can afford, and would preclude culvert replacement as part of the current project.

New Roadway Crossing Structure And Minimum Interior Dimensions

The proposed project design does not include a permanent access driveway on the south side of NW Innis Arden Way, because there is not sufficient space to provide such a driveway without impacting the home to the southwest. Thus, in the event any maintenance of the stream channel, new culvert structure, or a retaining wall is needed in the long term beneath the roadway or at the south end of the new structure, access to conduct that maintenance with any vehicle(s) or heavy equipment will need to be via the stream channel from north to south through the new structure. Based on recent experience that Herrera and Jacobs both were involved in with retrofitting some larger rock in the stream channel at the upstream end of a large box culvert that was built in Bellevue in 2014, head room to maneuver a medium-sized excavator inside the structure is worth designing for, to offer flexibility to get any needed maintenance done at least cost in the future. That interior head room above the channel bed is estimated to be 12 feet minimum, for a width of approximately 15 feet. Thus, the options evaluated for a new structure at the project site all have ability to provide a "clearance box" of 12 feet high by 15 feet wide in the center of the new structure cross-section, measured above the nominal channel bed elevation (which will fluctuate some within the base of that box shape due to irregular stream bed composition). The intent of the design is to not need such maintenance in the long term, but if it becomes necessary for any reason the extra interior height will represent a worthwhile insurance policy to prevent excessively expensive maintenance.

To reliably provide fish passage in the long term, the new crossing of Boeing Creek beneath NW Innis Arden Way is designed per the guidance presented in Barnard et al. (2013) and with reference to the Washington State Department of Transportation's (WSDOT's) Hydraulics Manual (WSDOT 2019a). In recent years WSDOT has completed hundreds of analyses of fish passage barrier culvert removals across the state, and as a result has honed its process for how to determine the appropriate replacement roadway crossing structure, and that process is presented in the Hydraulics Manual.

Barnard et al. (2013) spell out three basic options for a fish-passable structure: 1) a "no-slope" culvert, 2) a "stream simulation" culvert, and 3) a bridge. For bridges, the WSDOT Hydraulics Manual further defines confined bridges versus unconfined bridges. The width and gradient of Boeing Creek in the project reach are not conducive to the "no slope" option. Thus, the stream simulation culvert and bridge options were evaluated.

A stream simulation culvert is essentially a sufficiently wide and tall culvert that allows for sustenance of a natural channel bed through the entire length of the culvert. The minimum width of a stream simulation culvert is based on this simple equation: 1.2 x bankfull width + 2 feet (Barnard et al. 2013). Based on a bankfull width of 18 feet as described previously, the minimum interior width of a new culvert structure at this site is 24 feet. That width can be accommodated with the existing topography in the road crossing area, and there are a variety of culvert structure materials and shapes that could be used to build a 24-foot-wide structure beneath the roadway embankment fill, while accommodating the "clearance box" dimensions described above, followed by restoring the roadway in-kind. The stream channel within the culvert would need to be sloped, with a relatively flat slope preferred to avoid the need for engineering a non-deformable channel through it.

Barnard et al. (2013) establish a bankfull width of 15 feet as a general upper bound on applicability of the stream simulation design approach, with exceptions. Those exceptions are associated with stream-specific geomorphic characteristics and the length of the new culvert. Additionally, for new culverts greater than 20 feet in width, WSDOT internal guidelines recommend that the structure be considered a bridge and evaluated for seismic considerations. WSDOT (2019a) defines two types of bridges: confined and unconfined, with differentiation of each type based on the floodplain utilization ratio (FUR). The FUR is the width of the 100-year floodplain in the road crossing area relative to the stream channel bankfull width (Barnard et al. 2013). The 100-year floodplain width at the road crossing is derived based on existing conditions hydraulic model results documented in a memo contained in Appendix B of this report. The bankfull width immediately upstream and downstream of NW Innis Arden Way is estimated for this purpose based on 2-year flood modeling results (Appendix B) as opposed to measured bankfull width since the stream channel is highly modified with armored banks at each end of the existing culverts. Hydraulic model runs specific to this purpose "removed" the existing culverts to provide an indication of what the flow width and depth would be if the culverts were not altering conveyance capacity through the road crossing (which the existing culverts do in large flood events) per WSDOT (2019a) guidance.

The existing dam and the roadway embankment fill downstream of it appear to be located amid a natural constriction in the creek's valley, which is likely not a coincidence since dams and embankment fills in ravines are typically built at natural topographic constrictions to reduce cost. If the dam and roadway embankment were not present, the width of the Boeing Creek floodplain for over 200 feet length through this area would be very similar, such that if a wide enough culvert or bridge structure were in place the FUR is calculated to be 1. Hydraulic modeling confirms this – if the existing dam and culverts are ignored and a stream channel from the base elevation of the dam through a sufficiently wide and tall "notch" in the roadway embankment is assumed in their place, there is no simulated change in flow width, depth, or velocity between the damsite and the downstream side of the road. A FUR value of 1 is well below the threshold of an unconfined system, and therefore a confined bridge analysis is appropriate for the replacement structure.

Those confined conditions will be unchanged by project construction because site topography and vulnerability of destabilizing adjacent steep slopes significantly constrain widening the floodplain at the road crossing. For a confined bridge WSDOT (2019a) recommends applying a factor of safety to the reference bankfull width, which at minimum yields a structure width of 1.2 x bankfull width + 2 feet, or 24 feet at this site. The factor of safety should be increased to yield a greater width than this if the new structure could create excessive backwater conditions during flood events, flow velocities will differ greatly from velocities in the reference reach, significant sediment aggradation is expected at the crossing in the future, channel migration is expected at and near the roadway crossing, or the project designer has other reasons to increase the width. None of those considerations apply to this site, therefore the structure width does not need to be greater than 24 feet to satisfy WDFW and WSDOT guidance, subject to additional analysis of freeboard in the 100-year flood event. WSDOT (2019a) requires a minimum of 3 feet of freeboard above the 100-year flood level through any roadway crossing greater than 20 feet wide. Freeboard analysis for the proposed replacement structure is described later in this section.

In WSDOT's terminology, a bridge structure can be buried in an embankment supporting the roadway (i.e., "confined"), or it can span the stream without any embankment. For this project site, a buried structure is proposed for reasons explained above. To satisfy WSDOT (2019a) and WDFW (Barnard et al. 2013) requirements for a confined bridge, the structure width must be increased by 30 percent above the minimum if the structure length will be more than 10 times the width. The length of the new structure will be no greater than 135 feet based on the dimensions of the existing roadway embankment fill, unless it is extended farther to the north to reduce the length of an expensive retaining wall that will be needed on the west side of the modified stream channel entering the new structure. The new structure length will therefore be approximately 5 to 6 times the minimum required span width of 24 feet, and increasing the width is not necessary to satisfy the WSDOT and WDFW design guidance.

While WSDOT uses “bridge” nomenclature for a structure span width greater than 20 feet, for purposes of this project design and this report the term “culvert” is used hereafter for the new structure, since the best options for structure material are considered to be culvert products in the local construction supply industry.

Replacement Culvert Alignment and Length

The steep slopes adjacent to the stream on the north and south sides of NW Innis Arden Way, proximity of several private properties, and a desire to preserve as many mature trees as possible effectively constrain the alignment of the new culvert structure. Thus, the new structure will be installed parallel to and straddling the existing culverts. Straddling the existing culverts will enable using one or both of them to route streamflow through the work area as the roadway embankment is excavated above them.

The length of the new culvert was determined iteratively with consideration of excavation extents and costs, structure material cost, and the dimensions of headwalls and wing walls that would be needed and the corresponding costs of those features. Because the new culvert will be buried within the existing roadway embankment, the City will not require the roadway to be improved to current City design standards (i.e., inclusion of increased shoulder widths and adding a sidewalk) at the completion of project construction, but such improvements can be expected at some point in the future. Thus, the length of the new culvert needs to enable future widening of the roadway (with possible inclusion of a sidewalk) without need to modify (extend) the structure. The design therefore includes a rebuilt roadway embankment that is wider than the existing embankment at road level, and embankment side slopes similar to the existing slopes. This embankment configuration translates into a new culvert length of approximately 135 feet at stream channel level, and lesser length at the culvert crown (top) elevation depending on whether retaining walls are used to contain the embankment soil surrounding the upstream and downstream ends. The upstream end of it will be at least 15 feet upstream (north) of the existing culvert entrances, and the downstream end will be at about the same location as the existing culvert outlets.

Structure Material and Shape

A thorough analysis of culvert structure material options was performed, including pre-fabricated structures (made of concrete, aluminum, and steel) and a custom-made configuration that would use vertical shoring walls installed for excavation into the tall roadway embankment to serve as the permanent side walls of the new structure. The primary considerations in selecting a preferred material are an optimal combination of a dependably long life span coupled with least cost. The project team developed a variety of design concepts and corresponding cost estimates, with geotechnical engineering input and third-party cost estimating expert support. The chosen material and shape is a circular steel plate arch, comparable to Contech's BridgeCor® product. This product can provide durable, long-term performance while saving approximately $300,000 in cost compared to the next lowest cost precast concrete alternative, accounting for all construction work related to structure excavation, installation and backfill.

In January 2020 WSDOT updated its guidance for buried steel structures to account for potential corrosion and abrasion, requiring designers to perform a service life analysis to ensure a minimum service life of 75 years. Soil pH and resistivity provide a basis for understanding corrosion potential. HWA (2020) collected four soil samples amid the existing roadway embankment for this purpose. Laboratory analysis of those four samples indicates pH values ranging from 5.8 to 8.7 (mean of 7.1) and resistivity values ranging from 3,200 to 22,000 ohm-cm (mean of 10,050 ohm-cm) (HWA 2020).

The chart shown in Figure 5 is based on 18 gage galvanized steel (WSDOT 2020). Thicker gage steel can be used to increase the service life, via a greater thickness of steel that allows some corrosion to occur without compromising structure integrity. The thickness factor shown in the inset table in this chart is used to multiply the estimated service life of 18 gage steel, enabling selection of a steel thickness that achieves the desired service life. With an average pH of 7.1 and average resistivity of 10,050 ohm-cm, the service life of 18 gage steel is estimated to be about 33 years. By increasing the steel thickness to 5 gage, the estimated service life is approximately 140 years, exceeding WSDOT's guidance for minimum service life by several decades, and providing a dependably long-lasting solution for this site for the City of Shoreline.

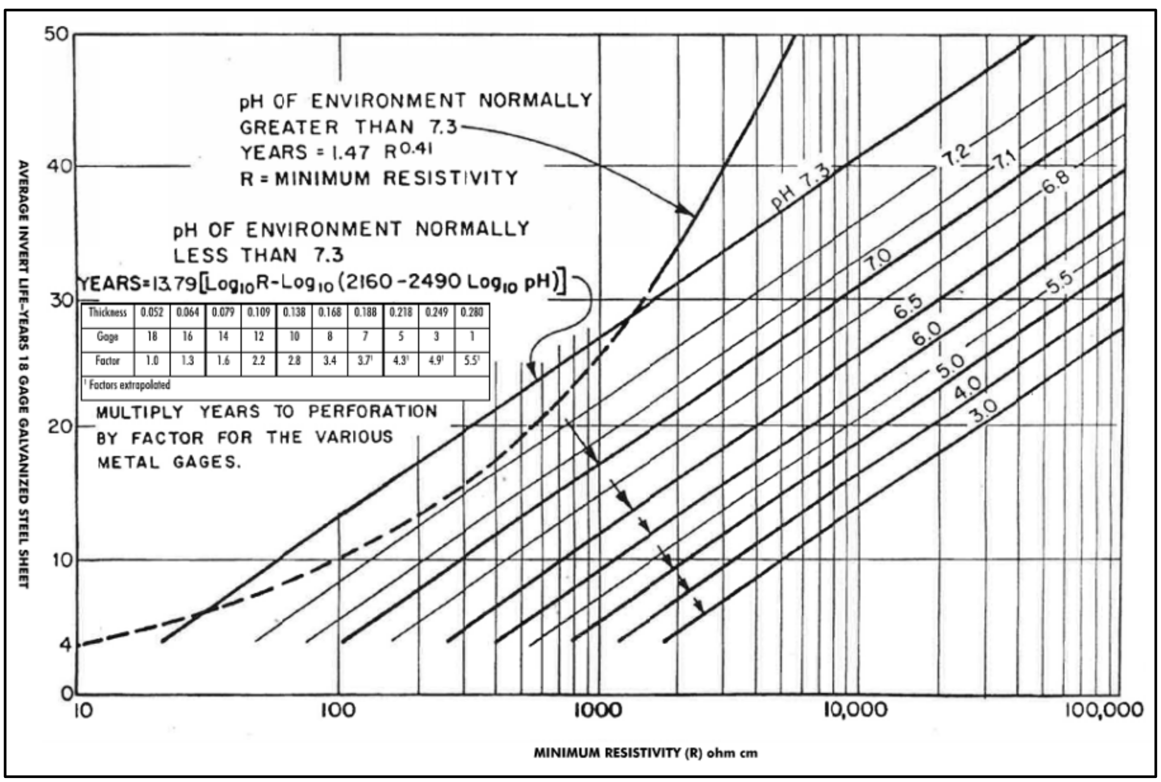

Figure 5. Galvanized Steel Structure Service Life as a Function of Soil pH and Resistivity.

Reasons that other culvert options were not selected include:

- The constrained site allows limited space for staging of equipment to complete the culvert removal and replacement work. The use of relatively heavy precast concrete culvert sections would require a larger crane to lift and place the precast units in the bottom of the excavation and likely result in higher costs and environmental impacts (e.g., tree removal) to provide a temporary construction access road along the stream channel.

- A custom-made culvert that uses excavation shoring walls as permanent side walls represents atypical construction and could result in higher construction bids while also inducing risks to the City if challenges emerge during construction that preclude the designed configuration from being constructible. Excavating wider to install a prefabricated structure in the bottom of the excavation prior to backfill is a much more common approach that construction contractors are familiar with.

- The potential for abrasion of the structure given that a lot of sediment is transported in Boeing Creek makes aluminum a potentially risky choice, as aluminum is more susceptible than steel to abrasion.

The proposed design cross-section for the new culvert is shown in Figure 6. This culvert size will comfortably fit below the elevation of several utilities that need to remain in service at the completion of construction in the road right of way.

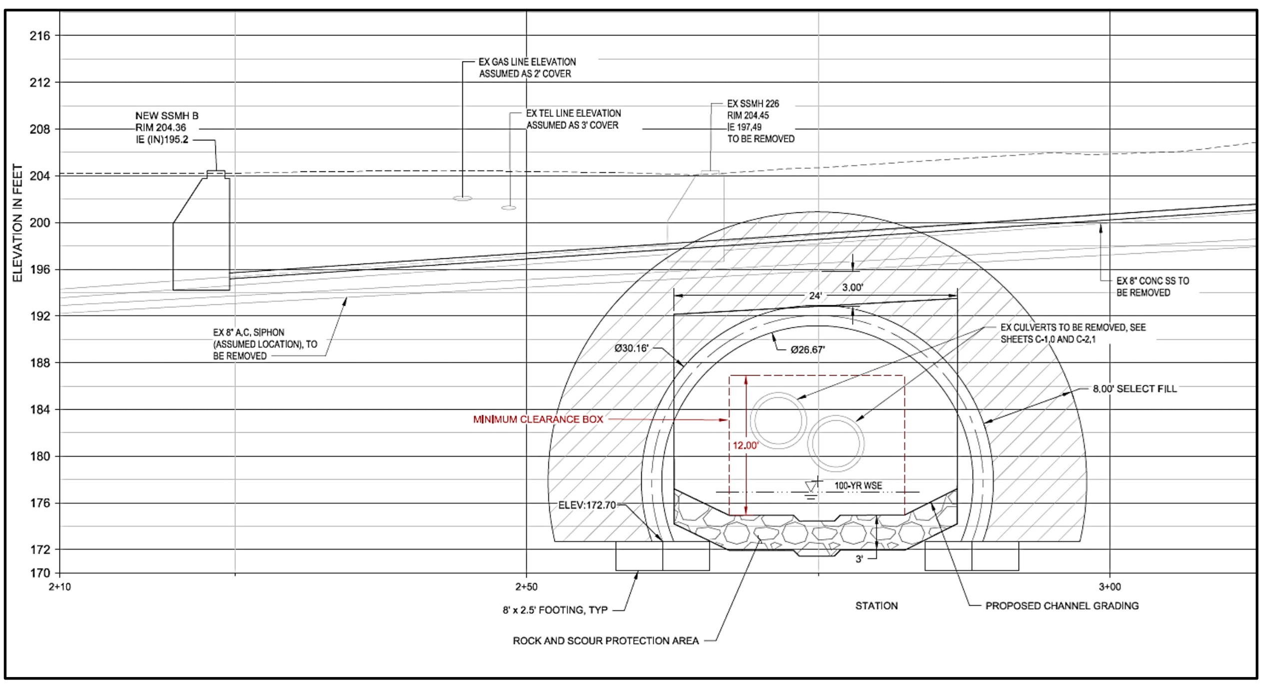

Figure 6. Confined Bridge Design Cross Section.

Headwalls and Wingwalls

At the upstream end of the new culvert, retaining walls are needed to prevent over-steepened ravine slopes to the east and west. Soldier piles with lagging in between the "H" piles are proposed as a cost-effective approach for these walls along the sides of the lowered streambed elevation as it approaches the road crossing to limit the width of excavation in relation to private property impacts on the west side and preventing destabilization of a steep hillslope on the east side.

The circular steel culvert material offers an opportunity to bevel the culvert shape as it extends out of the roadway embankment on each end to match the ground slope, alleviating the need for headwalls (above the culvert) or wingwalls (extending laterally from the culvert end) on either the upstream or downstream ends. To protect the steel culvert material that would otherwise be exposed to weathering and damage by natural elements, a concrete collar will be poured around the outer edge of the culvert where it extends above the stream bed and outward from the embankment fill above the stream bed. The beveled design with a concrete collar saves considerable cost on walls that would otherwise be needed and provides a durable solution for protecting the culvert ends. On the upstream side the soldier pile walls on both sides of the stream channel will connect to the concrete collar.

To minimize the excavation area and maintain stable slopes, the walls on each side of the stream channel on the upstream side of the road will be constructed from in front of the wall (within the stream channel footprint) without having to excavate and place backfill material behind the wall. This approach is partly to support a design objective of minimizing removal of mature trees and also to reduce costs for installation. These walls will be made of soldier piles that consist of steel piles placed in augered shafts that are filled with concrete. The wall facing will be constructed by excavating in front of the soldier piles using lagging and then casting a reinforced concrete wall facing attached to the soldier piles. Given the wall heights (up to about 12 feet maximum above the stream bed elevation), this is a cost-effective and constructible wall system that does not need to rely on lateral soil nails or ground anchors placed behind the wall to support the wall, which would add cost and increase the construction complexity, inducing risks for the City.

The finished surface of the soldier pile walls will be concrete fascia extending beneath the stream channel bed. The bottom elevation of the fascia is designed to be a minimum of 2 feet below the elevation of potential stream bed scour in accordance with the WSDOT Geotechnical Design Manual (WSDOT 2019b).

The first construction contract for dam removal and stream restoration through the drained lakebed will involve constructing a wall on the east side of the stream channel in the vicinity of the dam to maintain a stable slope after the dam is removed and to minimize removal of mature trees on the slope northeast of the dam, which is a priority for the City. Several design iterations for new stream channel grading north of the existing damsite were completed to spare as many mature trees as possible on the west side of the new channel and north of the cleared area used by the City for maintenance and inspection access to the dam. The adjacent property owner has expressed interest in saving those trees that are on their property. The stream alignment was shifted to the east to retain trees in that area, pushing it closer to a steep slope on the southeast edge of the lake and necessitating a wall to maintain slope stability. This wall on the east side of the channel will be permanent – during the subsequent construction for culvert replacement it will be extended south to a new headwall surrounding the confined bridge structure. This wall is needed in this same alignment for the culvert replacement phase of construction anyway, so does not represent an added cost to protect trees on the other side of the new stream channel.

HYDROLOGIC ANALYSIS AND HYDRAULIC MODELING

Hydrologic Data

At the outset of the current project design effort, flows in Boeing Creek had not been gaged for decades and a partially calibrated creek basin hydrologic model developed for the Boeing Creek Basin Plan (Windward Environmental et al. 2013) represented the only available flow information for the project design to make use of (Herrera 2016). With several years until construction was anticipated, Herrera installed a staff gage in Boeing Creek immediately upstream of Hidden Lake to collect water level data beginning in September 2016. As of the time this report was written, the staff gage was continuing to collect data. Manual flow (discharge) estimates were obtained at several points in time from late 2016 through 2019 at a range of discharges (i.e., varying magnitudes of storm events in the basin) to enable developing a "rating curve" that converts stage (water level) to discharge. The rating curve allows extrapolation of discharges at higher water levels than have occurred during the staff gage record with increasing confidence as more data points are added to the rating curve.

Flood Flows

The project design needs to consider a range of flood flow characteristics to optimize project outcomes. The 100-year flood flow was used as a primary basis for estimating peak water depths and flow velocities for design of the new stream channel, and for evaluating potential changes in flooding characteristics within and downstream of the project area after construction. The Basin Plan modeling yielded peak flow estimates of 72 cubic feet per second (cfs), 161 cfs, and 227 cfs in Boeing Creek approaching the upstream end of Hidden Lake in the 2-, 25-, and 100-year recurrence flood events, respectively, under existing drainage basin land use conditions (Windward et al. 2013). Because the drainage basin is substantially developed already, and the City is expected to require stringent onsite stormwater management controls for future land development and redevelopment projects to prevent worsening of flooding and erosion conditions in Boeing Creek, drainage basin runoff conditions associated with future land development were not evaluated to inform project design.

The highest discharge that is estimated to have occurred since the Boeing Creek staff gage was installed in 2016 just upstream of Hidden Lake is about 66 cubic feet per second (cfs), based on the current rating curve, in the early afternoon of December 20, 2019 suggesting the runoff event was slightly less than a 2-year event. Conveniently, King County has operated a rain gage at Shoreline Community College (called 04u–Boeing Creek Rain Gauge) since October 1989. This local gage is the best known source of publicly available rainfall data in the Boeing Creek basin, and measured rainfall at this location provides a good indication of the relative discharge that occurs at the Hidden Lake project site. The rain gage record that King County shares online was evaluated for several days through the afternoon of December 20, 2019. A total rainfall depth of 2.96 inches was recorded in a 26-hour period preceding when the peak water level (and thus discharge) was recorded at the staff gage at 1:50 pm on December 20th. That amount of rain is very close to a 25-year, 24-hour duration hypothetical storm according to isopluvial maps presented in the King County Surface Water Design Manual (King County 2016). The heavy rainfall should have produced a large stream discharge on December 20, however, the precipitation on December 20 was bimodal with a 12-hour lag between precipitation peaks and this can dampen the peak stream flow response in an urbanized, "flashy" system. The peak discharge at Piper's Creek, a nearby basin just to the south of Boeing Creek, in that same December 2019 storm event was also just below a 2-year event (177 cfs flow compared to an approximate 2-year flow of 180 cfs). Therefore, it is possible the bimodal character of the precipitation hyetograph in this particular storm resulted in a dampened runoff response, which is not surprising since both gages are near the downstream ends of their respective basins.

City staff inspected the Hidden Lake dam area on the afternoon of December 20, 2019 and did not observe flow over the dam spillway, although high-energy flow conditions were occurring at the culvert entrances downstream of the dam (John Featherstone, personal communication). Hydraulic modeling suggests that the dam does not overtop until a flow magnitude slightly greater than a 2-year event occurs, so it is likely that a peak discharge of 66 cfs entering Hidden Lake is a reasonable estimate for the December 2019 storm event, though that may be slightly lower than actually occurred. This analysis indicates that the basin runoff modeling documented by Windward Environmental et al. (2013) may be slightly conservative, but conservative is good for project design.

The City recently completed an assessment of future climate impacts and associated vulnerabilities that included analysis of potential changes in precipitation characteristics at Boeing Creek Rain Gauge 04u. Based on information developed by the University of Washington Climate Impacts Group, this study found that storm intensities and total precipitation amounts will likely increase in the wet season (in the months of October through April), resulting in increased Boeing Creek flood flows in the 2080s compared to flows that occurred in the 1980s. These changes will likely be gradual in the decades ahead, but the trend can be predicted as of now (Herrera 2019). However, this study's findings do not clearly indicate that the 100-year flood peak flow to use for project design should be increased by a factor of safety. The basin size is such that large storm events on the order of 1 to 2 days in duration can produce the largest flows in the creek. The analysis of future precipitation changes at Boeing Creek Rain Gauge 04u indicates that the 24- and 48-hour duration precipitation depths with 100-year return period frequency are expected to increase by 6 and 10 percent, respectively, in the 2080s compared to the 1980s. Because the results of existing conditions drainage basin runoff modeling presented in Windward et al. (2013) appear to be slightly conservative, the existing conditions flood flow estimates were not increased to account for future climate change effects.

Appendix B contains a memo documenting hydraulic modeling performed for design development based on the flow estimates documented in Windward et al. (2013).

Low Flows for Design of Habitat Features and Sizing of Flow Bypass System During Construction

The staff gage data collected since September 2016 provide reliable information for design of project elements associated with smaller flood events and base flows. The hydraulic model was run for the 2-year recurrence peak flow to inform design of the inset / low flow channel within the new stream channel. As described later in this report, the staff gage data obtained in the months of July through September in 2017, 2018, 2019, and 2020, converted to flow rates, provide a reliable basis for sizing the streamflow bypass system for each phase of construction.

Hydraulic Model Results

The hydraulic analysis of existing lake outflow conditions and the proposed project conditions (following the first phase of construction and then following completion of culvert replacement construction) was performed using the two-dimensional finite volume RiverFlow2D Plus hydrodynamic model. This model was selected for its ability to evaluate lateral distribution of flows and velocity vectors as a result of the proposed change in overbank flow area and unsteady nature of the flood events simulated that would otherwise not be captured using a one-dimensional modeling approach. Analysis of overbank and lateral flows is important to assess new creek channel design parameters and to accurately simulate effects downstream of the project site. The required RiverFlow2D model inputs included a topographic surface (i.e., a digital elevation model [DEM]), hydraulic boundary conditions, and hydraulic roughness (Manning's "n") values for the creek channel, floodplain, dam, culverts, and lake outlet pipes. The memo in Appendix B provides details on those model inputs.

The new stream channel created in the first phase of construction will tie into the concrete pad at the entrance to the existing culverts under NW Innis Arden Way. Until they are replaced in a subsequent construction contract, the existing culverts will continue to influence flood flow conveyance capacity. Along with installation of a much wider and taller culvert beneath NW Innis Arden Way, the channel elevation profile will be lowered in this area during the second phase of construction, and thus the characteristics of flood flow routing through the road crossing will change relative to interim conditions before the second phase of construction occurs. The hydraulic model was used to assess the differences in flow conditions in this part of the project area, and downstream, for these different points in time.

Model output results depicting flow depths and velocities are presented in Appendix B. A summary of those results follows:

- The gradient of the new stream channel will promote relatively high velocity, shallow-depth flow conditions, even in the 100-year flood event once the new roadway crossing structure is in place. However, until they are removed the existing culverts will constrain flood flow capacity, causing a backwater effect radiating north into the downstream portion of the former lake bed area. This backwater effect considerably reduces peak flow velocities over a distance of approximately 230 feet in the 100-year flood. The model results for existing conditions and following completion of the first phase of construction indicate peak 100-year flow depths of about 7.8 feet just upstream of the existing culverts. That backwater effect will be eliminated following completion of the culvert replacement after Phase 2 construction. In the existing condition this backwater would also occur in an extreme flood event, but the effect is dampened by the ability of the lake water level to rise slightly as flows continue to pass through the lake outlet piping system before the dam spillway is engaged. Related to these model results, photos taken during the December 2019 flood event (which as described previously was approximately a 2-year recurrence flood event) show turbulent water at the entrance to the culverts with a depth of about 5 feet.

- Widespread overbank flooding is not expected on either side of the new stream channel. To promote frequent overbank flooding into the floodplain area created on the east side of the new channel in the former lake bed, the left (east) bank was lowered in the design in the vicinity of Station 5+00 to Station 6+25 in the design plans.

- With elimination of the lake and construction of a new stream channel in the bed of the lake, the peak water surface elevations upstream of NW Innis Arden Way will be lower than in existing conditions after Phase 2 construction. See Table B1 in Appendix B.

- The simulated change in peak water surface elevations in the 100-year flood event downstream of the project site is insignificant. Appendix B provides an assessment of downstream hydraulic effects. Pre- and post-project hydrographs were extracted from the hydraulic model output to evaluate this possibility. The post-project model results do not indicate any significant changes to the downstream peak discharge or slope of the hydraulic grade line. The only change observed in this comparison was a 12- to 20-minute shift in the timing of the downstream hydrograph, which is to be expected with removal of the lake and its outlet piping, which have minor flow attenuation effects. With no significant change in slope, shape, or peak flow in the downstream hydrographs in the post-project condition compared to the pre-project (existing) condition, no change to the water surface elevations or velocities are anticipated downstream of the new culvert in the 100-year flood event. These results suggest that the lake in its current condition does not significantly attenuate flood flows, and supports a determination that there will be no adverse downstream flooding-related impacts after Phase 1 or Phase 2 construction. Thus, no mitigation associated with hydraulic effects is planned downstream as part of the project design.

The 100-year flood model results were used for scour calculations, sizing new stream channel substrate, determining the new culvert structure span at an elevation equivalent to the 100-year peak water surface plus 3 feet of freeboard (per WSDOT [2019a] guidance for design of confined bridges), and log structure stability calculations.

New Stream Channel Alignment, Cross-Sectional Geometry, Potential Scour, and Substrate Sizing

The stream design is based on “natural” cross sections that were identified upstream and downstream of the project area. The proposed streambed for the dam removal phase will hold a 2 percent slope for a channel length of 635 feet through the existing lake bed mimicking upstream conditions. As the new channel nears the existing damsite it will be graded to maintain a 0.5 percent slope for a length of about 65 feet in the first phase of construction and will end at the concrete splash pad at the entrance to the existing culverts under NW Innis Arden Way. In the culvert replacement phase of construction, the splash pad will be removed and the 0.5 percent slope will be regraded down to an 8 percent slope for a length of 75 feet to just upstream of the entrance to the new culvert. This steepened slope is required to match downstream grades and to emulate likely historic stream conditions as the stream cut though historic landslide debris in this reach of Boeing Creek (i.e., where the existing dam was conveniently located). The slope through the new culvert will hold a 2 percent grade for a length of about 145 feet to the downstream project limit. It is desirable to have a relatively flat slope through the new roadway crossing structure to reduce flow velocities and associated scour potential, and a 2 percent slope through the new culvert was chosen to mimic upstream conditions for transient sediment transport purposes. Since installing wood habitat features inside the new culvert poses potential for significant long-term maintenance needs, the stream design through the new structure will use a combination of large boulders (in clustered “bands” that deflect flow) and coarse streambed material to hold the stream bed in place while providing low flow habitat complexity that does not need maintenance.

A floodplain bench will be graded in on the left (east) bank through the lake bed to provide wetland habitat and refuge for fish during high flow events. Additionally, the floodplain bench will encourage formation of wetland conditions over time to help mitigate for impacts to existing lakeshore wetlands associated with project construction, discussed in the Impact Mitigation section below.

Streambed Material Design

The streambed mix was designed according to the Washington Department of Fish and Wildlife (WDFW) design standards, and using predicted flow characteristics in the hydraulic modeling output to establish input parameters for the WDFW sizing calculations. For the steep section of the new stream channel profile, 8 percent between the existing damsite and the upstream entrance to the new culvert structure, design guidance for “roughened channel” mixes was followed (Barnard et al. 2013). A formal pebble count was not conducted for the determination of stream bed material sizing, but streambed material was observed during a site visit, on October 27, 2016, and observations were used to inform streambed material sizing calculations.

Appendix C presents the streambed material sizing calculations and results. The specified streambed material will consist of a well graded boulder, cobble, gravel, and sediment mixture to provide beneficial fish habitat, reduce potential for stream bed material mobilization in high flows, and prevent the stream from flowing subsurface. The streambed mix will extend above the new channel bed for a height of a few feet on the channel banks to reduce erosion potential during high flows.

Log Structures

Fifteen habitat log structures and two log revetment structures will be placed in a 450-foot-long reach of the new stream channel through the lake bed to provide habitat complexity and maintain the design alignment of the channel, which is on Shoreview Park land to the maximum extent possible. Habitat log structures will deflect flows to alternating sides of the constructed channel so the thalweg meanders, while also encouraging formation of pools. An 80-foot-long log revetment will be installed along the right (west) bank at the upstream end of the project, where channel grading begins, to provide erosion protection for the adjacent steep slope in high flow events. The stream currently takes a sharp turn in this same location, but low flow velocities prevail due to backwater from the lake. When the lake is drained for project construction the hydraulic gradient will steepen in this area, and higher velocity flow could induce bank erosion if the bank is not reinforced. The logs in this log revetment structure will extend into the bank via excavation to place them followed by backfill of the buried ends of the logs with soil and large rock. Rootwads will extend from the revetment face into the stream channel.

A 55-foot-long log revetment will be installed on the right (west) bank of the new channel across from a floodplain wetland creation area to reduce erosive pressure on the right bank while encouraging flow to engage the wetland creation area on the left bank during rain events.

Log structure design calculations to assure resistance to buoyancy and drag forces are provided in Appendix D. The 100-year flood event hydraulic conditions simulated with the model described later in this report yielded the flow depth and velocity parameters used in these calculations.

Streamflow Bypass During Construction

Based upon flow monitoring data that Herrera has collected for the City since September 2016 just upstream of Hidden Lake, flows in Boeing Creek are expected to be in the range of 1 to 4 cfs during construction of any project features in and near the stream in mid to late summer, coinciding with the permitted in-water work period. A flow rate greater than 4 cfs could occur if a large storm event occurs in the Boeing Creek basin at this time of year, though it would be rare for greater than 4 cfs to be sustained for more than a few hours based on the streamflow measurements obtained since 2016. Boeing Creek flow will be bypassed through the active work area for the duration of both phases of project construction, as needed until the construction activity is no longer occurring near the stream (such as when final backfill of the new culvert structure occurs, and when planting occurs after all grading work is complete). This will likely be accomplished via routing streamflow through a temporary pipe, with a cofferdam as needed at the upstream end and an outlet energy dissipater at the downstream end to prevent erosion of the existing stream channel. If fish are present in the stream during bypass activities, appropriate screening will be implemented at the bypass inlet per NMFS and WDFW guidelines.

The first phase of construction has more complicated streamflow bypassing to be done, because the lake needs to be drained first, and streamflow will be moving through the lake as it is drained. The construction specifications will require the construction contractor to prepare and submit a streamflow bypass plan in advance of any bypass work on the ground, and the available streamflow data will be provided in the contract documents to inform their planning. Herrera has found on numerous projects in streams and rivers in the past decade that prescribing a flow bypass plan for the contractor to implement is not preferable because doing so does not lead to the contractor truly understanding and respecting what is going to be needed for successful flow bypassing in conformance with permitting requirements and it puts unnecessary risk on the project owner if the prescribed plan does not work very well. One viable way that the design team has developed for streamflow bypass in each phase of construction is described below.

The lake outlet manhole has a buried pipe attached to it from the north, with a flange valve in the manhole interior that can potentially be activated to partially drain the lake to reduce the volume and depth of lake water that will be drained upon removing the dam. This pipe and the flange valve it connects to were installed when King County rebuilt Hidden Lake in the mid-1990s as a mechanism for routing streamflow through the construction work area. However, that valve has not been operated in over 20 years, and the condition of the pipe leading to it from the north is unknown. Thus, relying on these existing features without prior testing of the ability to readily use them during dam removal project construction is a risk. An alternative is for the contractor to dismantle the existing lake outlet manhole structure incrementally from the top down, creating a rudimentary weir at each incremental lowering of the elevation at which lake water can spill into the manhole. This could be done either with a submersible concrete saw or with a jackhammer or similar equipment that can break the manhole apart slowly from the top down. The construction specifications will require gradual lowering of the lake level so a sudden surge of water is not routed downstream, where it could cause unwanted increases in turbidity and/or channel erosion.

Bypassing clean stream flow for discharge downstream of the proposed work area will minimize turbidity impacts from construction activity. After the lake is mostly drained, the design plan for the first phase of construction includes installing a pump system at a low point in the lake bed (northwest of the dam) to complete lake dewatering while a cofferdam is installed at the upstream project limit. The design includes a gravity pipe (sized to convey at least 4 cfs) extending from that cofferdam to the existing concrete pad at the upstream entrance to the culverts under NW Innis Arden Way to diffuse energy and prevent erosion. During the second phase of construction, streamflow can be routed through one or both of the existing culverts as the excavation occurs down to the elevation of those culverts. Once those culverts are ready to be removed, a cofferdam will be placed in the constructed stream channel north of where the channel elevation will be lowered, and the flow will be routed through a gravity pipe (once again sized to convey at least 4 cfs) with a durable energy dissipater at the pipe outlet into the existing stream channel beyond the downstream end of proposed stream modification work. The energy dissipater could be made of existing riprap stones that will be removed from the stream channel in that area.

As noted above, the construction specifications for each phase of construction will require the contractor to submit a detailed plan for the streamflow bypass system, to be sized and maintained to convey at least 4 cfs. The stream gage data collected in mid to late summer in 2017, 2018, 2019, and 2020 indicate that a flow exceeding 4 cfs typically does not occur until fall rains become frequent in October. If a storm event occurs in the Boeing Creek basin during construction that raises flow levels to a rate that exceeds the streamflow bypass system capacity, the contractor will be required to remove equipment and loose material from the work area and allow excess flow to pass through the work area, then clean up afterward and commence construction once again after the flood has receded.

SEDIMENT TRANSPORT AND DEPOSITION

Hydraulic modeling results for proposed conditions following construction indicate that sediment carried in Boeing Creek into the project area from upstream in the basin should reliably be transported downstream in Boeing Creek toward its mouth at Puget Sound, without ability for significant deposition within the project area that could notably change anticipated project performance. The relatively shallow depth of flow in the restored stream channel and high flow velocities during flood events will not allow large volumes of sediment deposition in the channel or adjacent floodplain areas, unless a large landslide occurs upstream of the project area that causes massive deposition of sediment in the project area. The project cannot be designed differently to alleviate that potential.

TEMPORARY BYPASS OF EXISTING UTILITIES

There are several utilities in the NW Innis Arden Way right of way at the location of the creek crossing. Utilities that cannot be taken out of service for the duration of construction will either need to be rerouted around the construction area or supported and protected to ensure continuous operation during and after excavation for the culvert removal and replacement. All of the utilities will need to be replaced and put back in service after construction. Existing utilities, all of which are buried in the roadway embankment, include:

- Gas – 4″ diameter MPE iron pipe gas main
- Telecommunications – two parallel lines, assumed to be 2″-4″ diameter PVC

- Sanitary sewer - 8″ diameter concrete pipe
- Sanitary sewer siphon - 8″ diameter asphalt concrete pipe
- Water – 8″ cast iron pipe

Removal and replacement of the gas line will be done by Puget Sound Energy (PSE), the utility owner. The gas line will be cut and capped on either side of the excavation, and new pipe will be installed at the completion of construction. During construction, the contractor will need to coordinate with PSE on access, relocation plans, etc.

Temporary relocation of the existing telecommunication lines will also be done by the utility owner, presumed to be Frontier Telecom or Comcast Cable (this will need to be confirmed in advance of construction). The telecommunication conduit will be replaced at the completion of construction. Similar to the gas line, the City's construction contractor will need to coordinate with the utility owner.

The sewer lines, including a gravity pipe and siphon pipe, serve an estimated 100 single-family residences (SFRs) to the east of the project vicinity and the flow cannot be shut off during construction. In addition, in order to maintain gravity flow in the gravity pipe, the existing alignment and elevation profile of that sewer line cannot change appreciably through the project work area. Thus, bypass pumping will likely be required to maintain sewer flow from east to west through the work area, unless the contractor can resolve a way to maintain a gravity sewer line across the wide excavation needed for culvert replacement. The sewer line and siphon are owned by the Ronald Wastewater District. New sewer pipe will be installed as the excavation of the culvert is backfilled.

Flow estimates for bypass pumping were provided by the City and CHS Engineers, which has done extensive design work for Ronald Wastewater District in the past. The design flow estimates are 100 gallons per minute (gpm) for the 8 inch siphon and 25 gpm for the 8 inch gravity main. These estimates are supported by Herrera's previous calculations to estimate wastewater flows in the gravity pipe on a per capita basis, which were done prior to obtaining design flow information from the City and resulted in similar flow numbers.

The water main is owned by Seattle Public Utilities (SPU) and can be temporarily shut off in the project area via cutting and capping during construction. The water line will be isolated in the area of construction, cut at each side of the culvert excavation, capped, and replaced after installation of the new culvert and roadway embankment backfill is nearing completion. Water supply to nearby residents can be temporarily rerouted, except for one single-family residential property on the north side of NW Innis Arden Way and west of the creek, which has a side service located near the construction area. The exact location of this water side service line will be mapped during final design, and if it is within the cut and cap limits of the water line in the right of way, SPU will verify that the service is active, and if so, a temporary service will need to be provided. Work on the SPU water line will be supervised by SPU Operations.

Trail Design

The existing trails in Shoreview Park (those that are maintained for public use, as opposed to rudimentary trails through brush that have not been sanctioned) are typically dirt-surfaced and about 3 to 4 feet wide on average. A section of trail with stair steps near the project site is shown in Figure 7.

The trail improvements are being funded partly by a grant the City obtained from the Washington State Recreation and Conservation Office, under the Land and Water Conservation Fund program. Per the stated design intents in this grant contract, the restored and improved trail segments are intended to be similar in character to existing trails nearby in Shoreview Park that are not designed and maintained to meet requirements of the Americans with Disabilities Act (ADA).

Figure 7. Existing Trail in Shoreview Park Near Hidden Lake.

The Hidden Lake Loop Trail will be realigned as part of the dam removal phase of construction in the northeast edge of the project area and improved using a combination of at-grade (emulating existing nearby trails), turnpike (with wooden framing to elevate the trail surface slightly above adjacent grade), and boardwalk design elements. The new trail will lead to viewing platforms overlooking the restored stream channel and adjacent floodplain and wetlands. Most of the new trail length will be at-grade, with no surfacing material over native soil. Where the trail crosses over wetlands or sensitive areas, the trail will be on a wooden boardwalk supported on 1.5-inch-diameter pipes, installed by hand, to reduce project impacts. Where a new trail section traverses continuously sloped terrain, a turnpike design detail applies as shown in the preliminary design plans.

The proposed alignment of new trail sections was selected based on minimizing impact to sensitive areas, reducing potential for trespassing onto private property, and providing interactive education for the public. Trail construction will require delivery of boardwalk and turnpike wood material to the work site, but thereafter the construction can all be done by hand, and thus the timing to complete trail construction is relatively flexible compared to the remainder of the dam removal phase of construction work.

The City is interested in minimizing the potential for any water quality impacts due to materials used in project construction. The design minimizes use of pressure-treated lumber, only using it where it will important for longevity while not being subject to foot traffic. Metal hardware and fasteners will be made of hot dip galvanized steel with the addition of a duplex system coating after fabrication for long-term prevention of corrosion. Galvanized steel coating is necessary to minimize potential leaching of zinc into the creek. The locations of the metal hardware and fasteners will inherently be protected from exposure to precipitation.

Geotechnical Analysis and Recommendations

Geotechnical analyses were performed to assess site conditions, geologic hazards, seismic design considerations, slope stability, and inform culvert replacement options. The methods and results are presented in a report prepared by HWA GeoSciences. The report also includes recommendations for dam removal, channel grading, scour and erosion protection, bearing pressures on retaining walls, and installation of the new roadway crossing structure and log structures (HWA 2020).

Geotechnical findings and recommendations that directly affect the project design and that are not discussed elsewhere in this report are summarized below.

- Permanent slopes adjacent to the stream channel in the area where the dam is removed should be no steeper than 2H:1V, due to liquefaction potential in the native soils and a need to prevent ground disturbance in close proximity to oversteepened slopes that could be vulnerable to failure.

- With the proposed elevation of the finished stream channel, it is not possible to have a 2H:1V side slope extending to the west and southwest of the damsite without removing all of the mature trees in that area and forcing the site access roadway to be located farther to the west, where it would require removal of additional mature trees on private property. Thus, both phases of project design include a wall on the west side of the stream channel extending north of the NW Innis Arden Way embankment to contain the extents of grading in this part of the project site.

- On the east side of the new channel in the dam removal area, the newly graded side slope should tie into relatively flat formed in colluvium. Do not disturb the ground east of this area close to the base of a near-vertical slope formed in advanced outwash and glaciolacustrine soils. This recommendation led to inclusion of a permanent retaining wall in the design on the east side of the stream channel extending approximately 25 feet north of the existing lake outlet control manhole. Expect groundwater to be encountered at mid-depth of culvert replacement excavation on the east side and be prepared to manage groundwater during the remainder of the excavation and include a permanent subsurface drainage feature(s) to reduce groundwater contact with new confined bridge structure.

- Excavation for culvert removal and replacement can be accomplished with a combination of sloping and temporary shoring. Shoring via soldier piles and lagging between piles is viable, with the bearing pressure on the shoring limited to the pressure anticipated during construction. Where shoring is not used in the culvert removal excavation, the slope face should be no steeper than 1.5H:1V, and the slope may need to be flatter than that if groundwater seepage is encountered. It makes sense to plan for shoring of most or all of the east side of the culvert removal excavation, since seepage along the top of the glaciolacustrine soil layer is expected. That layer is much deeper on the west side of the proposed excavation, and thus open cut excavation is more viable on the west side.

- The foundation for the new culvert structure can be concrete spread footings placed on the native glaciolacustrine soil, which is very firm.

- Permanent retaining walls on each side of the stream channel should be constructed using soldier piles and lagging, with inclusion of reliable drainage measures on the landward side of each wall to prevent hydrostatic pressure from building up against the wall.

- The maximum slope on finished stream channel banks should be no steeper than 2H:1V unless armoring or a bioengineered treatment is used to prevent erosion and sloughing.

- Excavations to bury logs included in log revetment structures and potentially in habitat log structures could encounter groundwater. Backfilling of buried log ends should be done with dry (potentially imported) soil as opposed to backfilling with wet soil in such excavations. Wet soils excavated onsite can be dried out and reused selectively.

- Where channel bank erosion cannot be allowed due to the need to constrain channel alignment, and a log revetment is not included in the design to prevent erosion, place riprap at the toe of bank below the ordinary high water level and a permeable ballast rock layer behind/beneath the riprap.

Design Features to Completely Mitigate Environmental Impacts Onsite

The proposed project will result in temporary and permanent impacts to wetlands, streams, significant trees, and critical area buffers. Impacts will be mitigated per the Mitigation Plan included in the Critical Areas Report (Herrera 2020). Mitigation elements are expected to improve water quality, hydrology, and habitat through the construction of new wetlands and establishment of native vegetation.

All impacts will be mitigated on site through re-establishment of the natural flow of Boeing Creek and associated wetland and riparian plantings. New wetland and riparian areas will be established in the drained lakebed through grading and revegetation. In existing conditions, there are approximately 9,203 square feet (0.21 acre) of wetlands in the project area. The proposed project will result in approximately 18,665 square feet (0.43 acre) of wetlands in the project area. In existing conditions, Boeing Creek has a length of approximately 685 linear feet, as measured from the upstream end of the lake to the dam. From the dam to the outlet of the existing culverts beneath NW Innis Arden Way there is effectively no existing stream channel. The proposed project will increase stream length to approximately 840 linear feet by removing the dam and creating a sinuous Boeing Creek channel in the bed of Hidden Lake and extending through the NW Innis Arden Way crossing.

As described previously, hydraulic modeling indicates that the project will not increase peak flows during major flood events in Boeing Creek downstream of NW Innis Arden Way. Therefore, the project will not increase potential for stream channel erosion downstream, and will almost certainly reduce the potential for erosion due to promoting transport of sediments in streamflows into the lower reaches of Boeing Creek. The increased sediment transport relative to existing conditions will enable the stream to “heal” in areas where channel downcutting has been occurring in recent decades, and the increased transport of sediments to the creek delta at Puget Sound is expected to increase the size of the delta, resulting in beneficial effects for salmon forage fish habitat (Herrera 2017b).

The overall functional lifts created by the project will compensate for all project impacts on trees, streams, wetlands, and their buffers. The project is expected to have a positive cumulative effect on functions in critical areas and critical area buffers, including water quality, hydrology, and habitat improvement.

REFERENCES

Barnard, R.J., J. Johnson, P. Brooks, K.M. Bates, B. Heiner, J.P. Klavas, D.C. Ponder, P.D. Smith, and P.D. Powers. 2013. Water Crossing Design Guidelines. Washington Department of Fish and Wildlife.

King County. 2016. King County, Washington Surface Water Design Manual. King County Department of Natural Resources and Parks.

Herrera. 2016. Draft Technical Memorandum: Hydrologic, Hydraulic, and Geomorphic Analysis. Prepared for City of Shoreline Public Works Department by Herrera Environmental Consultants, Inc., Seattle, Washington. January 5.

Herrera. 2017a. Draft Technical Memorandum: Concept Design Evaluation of Fish Passage Improvements in Lower Boeing Creek. Prepared for City of Shoreline Public Works Department by Herrera Environmental Consultants, Inc., Seattle, Washington. July 7.

Herrera. 2017b. Potential Nearshore Habitat Gains Analysis: Boeing Creek Delta. Prepared for City by Herrera Environmental Consultants, Inc., Seattle, Washington. February 3.

Herrera. 2019. City of Shoreline Climate Trends and Predicted Impacts. Summary Memorandum Prepared for the City of Shoreline by Herrera Environmental Consultants, Inc., Seattle, Washington. November 21.

Herrera. 2020. Draft Critical Areas Report and Mitigation Plan—Hidden Lake Dam Removal Project. Prepared for City of Shoreline Public Works Department by Herrera Environmental Consultants, Inc., Seattle, Washington. January 23.

HWA. 2020. Geotechnical Report: Hidden Lake Dam Removal and Stream Restoration Project, Shoreline, Washington. Prepared for Herrera Environmental Consultants, Inc., and City of Shoreline by HWA GeoSciences Inc., Bothell, Washington. January 22.

Windward Environmental LLC, Osborn Consulting Incorporated, and The Watershed Company. 2013. Boeing Creek Basin Plan. Prepared for City of Shoreline by Windward Environmental LLC, Osborn Consulting Incorporated, and The Watershed Company.

WSDOT. 2019a. Hydraulics Manual (M23-03.06). Washington State Department of Transportation, Environmental and Engineering Programs, Hydraulics Office.

WSDOT. 2019b. Geotechnical Design Manual (M46-03). Washington State Department of Transportation, Materials Laboratory, Engineering and Regional Operations.

APPENDIX A

Bankfull Width Determination

Bankfull width measurements were completed at four Boeing Creek channel cross-section locations within a total of stream length of approximately 200 feet on December 17, 2018. The measurements were led by Jeff Parsons, senior geomorphologist with Herrera, and Larry Fisher of WDFW. Valerie Wu of Herrera and John Featherstone of the City of Shoreline joined to observe and discuss stream characteristics relevant to this determination. Bankfull width indicators evaluated in the field were based on guidance in the Water Crossing Design Guidelines (Barnard et al. 2013), for situations where there is no defined overbank floodplain area adjacent to the stream channel (as is the case at the NW Innis Arden Way crossing as well), including the following:

- A change in vegetation
- A change in the particle size of stream bank material
- Undercuts in the bank
- Evidence of sediment and waterborne debris deposition on the banks
- Stain lines on boulders

Streambed substrate was not sampled during this field work because the stream characteristics are completely artificial on both sides of NW Innis Arden Way at the project site, and thus there is no ability to compare substrate at the culvert replacement site with substrate in the reference reach where bankfull width measurements were obtained. However, it is important to consider whether the stream substrate characteristics at the project site after construction will be similar to those where bankfull width measurements were taken. Larry Fisher confirmed during this field work that the channel substrate observed in the reference reach is typical of what can be expected in the project area following construction based on observations of Boeing Creek upstream and downstream of the project area.

Photos taken during this field work, indicating prevailing channel conditions in the reach of the creek where bankfull width was determined, are provided below. The flow rate in the creek was relatively low on this date, making it easy to observe bank conditions and navigate through the stream corridor to select specific locations to obtain width measurements using a measuring tape.

APPENDIX B

Hydraulic Modeling Memorandum

INTRODUCTION

This technical memorandum presents the methods and results of hydraulic modeling performed in support of design of the Hidden Lake Dam Removal Project. This memorandum references a technical memorandum prepared in 2016 documenting hydrologic, hydraulic and geomorphic analyses of project alternatives (Herrera 2016), for which the hydraulic model of existing conditions cited in this memorandum was originally created. The results of the modeling described herein provide information for use in design of the proposed new stream channel and culvert, including substrate sizing, log structure stability analyses, and scour potential, and also support assessment of the project's effects on flood flow conveyance within and downstream of the project site.

BOEING CREEK BASIN HYDROLOGY

The hydrologic analysis completed for purposes of hydraulic modeling of Hidden Lake project alternatives (Herrera 2016) was reassessed to determine if adjustments are needed in the peak flow rates and associated storm runoff hydrographs to use in modeling for project design. That assessment resulted in a decision to continue using the streamflow hydrographs for the 2- and 100-year flood events presented in Herrera (2016), as they are reasonably conservative. Additional information supporting this decision is provided in the basis of design report (Herrera 2021). Those hydrographs are shown in Figure B-1. In addition, a low flow rate was modeled to provide information to use in designing the stream channel for typical habitat conditions. This low flow discharge was assumed to be 2.7 cubic feet per second, which represents an average base flow from May to October that is not associated with storm events. The model results for this low flow are useful for assessing water depths and velocities that sustain instream fish habitat (such as size of pools that can be maintained near a log structure in the dry season).

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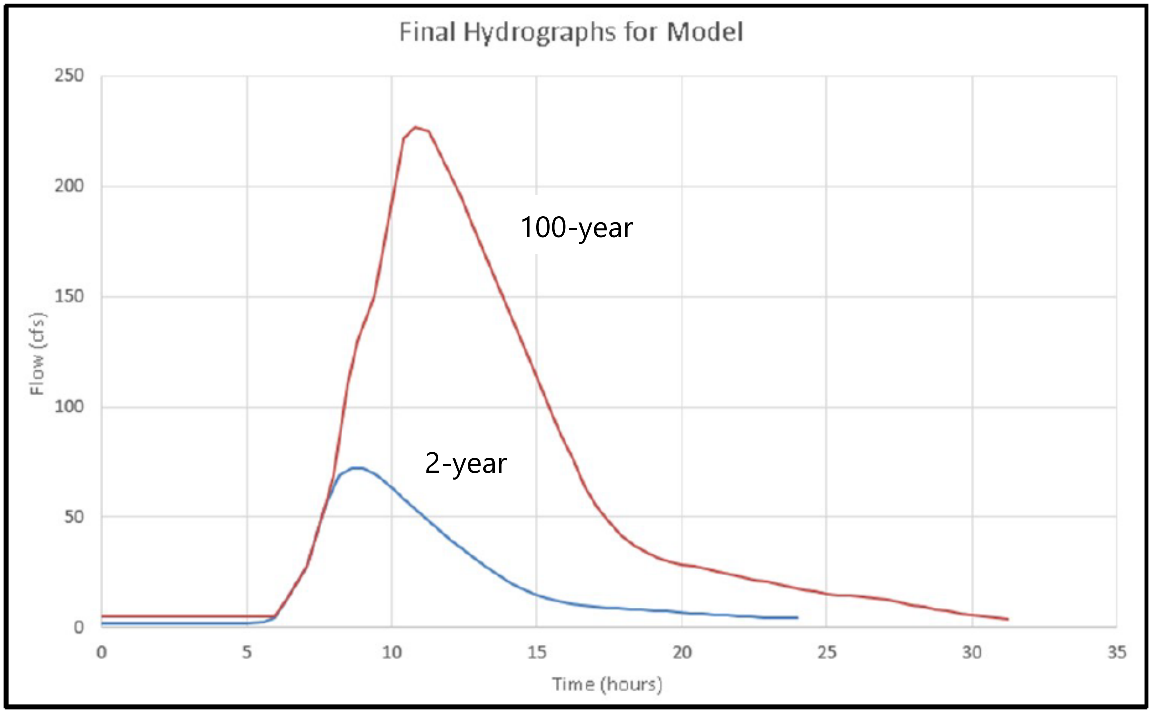


Figure B-1. Boeing Creek Hydrographs Used in Hydraulic Modeling for Project Design.

HYDRAULIC ANALYSIS

The hydraulic modeling was performed using the two-dimensional finite volume RiverFlow2D Plus hydrodynamic model. This model was selected for its ability to evaluate lateral distribution of flows and velocity vectors as a result of the change in overbank flow area and unsteady nature of the flood events simulated that would otherwise not be captured using a one-dimensional modeling approach (Herrera 2016). Analysis of overbank and lateral flows is important to assess new creek channel design parameters and to accurately simulate effects downstream of the existing dam site. The required RiverFlow2D model inputs included a topographic surface (i.e., a digital elevation model [DEM]), hydraulic boundary conditions, and hydraulic roughness (Manning's "n") values for the creek channel, floodplain, dam, and pipes.

The RiverFlow2D model of existing (pre-project) streamflow conditions through and downstream of the project area presented in Herrera (2016) was retained with no changes. The existing conditions model was modified to evaluate post-project conditions that will occur with phased project construction. Phase 1 will remove the existing dam, drain the lake, and construct a new stream channel through the drained lake bed and dam site, with the channel tying in to the elevation of the existing concrete pad at the upstream entrance to the two existing Boeing Creek culverts under NW Innis Arden Way. Phase 2 will occur a few years after Phase 1 is complete, and will remove the existing culverts under the road and replace them with a 24-foot-wide culvert structure. In this second phase of construction the stream channel elevation profile will be deepened from approximately the location of the existing dam site through the new roadway crossing, and the lowered and restored stream channel will connect to the existing stream channel south of the road.

The design of both phases of the project needs to be tailored to anticipated hydraulic conditions. It is important to evaluate hydraulic conditions following the first phase of construction to understand how streamflow will move through the site with the existing roadway culverts having lesser flood flow conveyance capacity than the replacement culvert structure will.

PROPOSED CONDITIONS MODEL DEVELOPMENT

The upstream and downstream boundary conditions were not changed for either of the proposed conditions models. The topography of the site was modified for the model of the proposed Phase 1 project conditions, with the proposed channel and floodplain surface adjusted to reflect the site configuration presented in the 60 percent complete design plans. A second model of the proposed Phase 2 project conditions was prepared with additional topographic alterations reflecting the lowered channel elevation through the NW Innis Arden Way crossing, geometry of proposed side walls at the new culvert entrance and exit, and a 24-foot-wide circular culvert with streambed material placed in the bottom and extending above peak flood levels along the interior culvert sides, such that all flood flows modeled are in contact with the new streambed through the replacement culvert structure. The Manning's roughness ("n") parameters input to the proposed conditions models for stream channel and floodplain areas were based on values that correspond to proposed instream log structures, coarse streambed material, and dense riparian vegetation in the restored floodplain areas on the east and west sides of the new channel through the existing lake bed (Chow 1959).

RESULTS

Results for the Existing Conditions, Phase 1 proposed conditions, and Phase 2 proposed conditions hydraulic models are presented in Attachments B1, B2, and B3, respectively. As mentioned above, the existing conditions results are the same as presented in Herrera (2016). Table B-1 summarizes the simulated water surface elevations at several cross-sections through the length of the project site for each of these scenarios (before and after phased construction). Figure B-2 shows the locations of these cross-sections. The station numbering assigned to the cross-sections is the same as is being used in the project design plans, and several of the cross-sections included in Table B-1 are the same as used to present new stream channel and floodplain grading information in the project design plans (those with a C, D, etc. letter identifier).

Table B-1. Simulated Peak Water Surface Elevations in a 100-Year Flood Event Along Stream Centerline.							
Design Stationing	**Description Tied to Existing Condition**	**Existing Lake or Stream Bed Elevation (feet)**	**Existing Conditions Peak Water Surface Elevation (Feet)**	**Proposed Phase 1 Stream Bed Elevation (feet)**	**Peak Water Surface Elevation After Phase 1 Construction (feet)**	**Proposed Phase 2 Stream Bed Elevation (feet)**	**Peak Water Surface Elevation After Phase 2 Construction (feet)**
0+00	Downstream of project limits	155.79	160.25	155.79	160.18	155.82	160.20
1+00	Beyond existing culvert outlets	171.81	175.69	171.81	175.30	173.35	175.71
2+50	Near downstream toe of existing dam	182.17	188.64	181.54	189.36	175.96	178.64
3+10	Section C – upstream edge of existing dam	196.45	197.26	181.88	189.55	180.69	183.56
3+50	Section D – south end of existing lake	189.74	197.34	182.71	189.58	182.77	185.82
4+00	Section E – mid lake	187.48	197.34	183.68	189.60	183.78	186.12
4+50	Section F – mid lake	187.77	197.33	184.57	189.61	184.57	187.00
5+50	Section H – mid lake	188.46	197.33	186.60	189.61	186.60	188.56
7+00	Section K – mid lake	193.08	197.33	189.92	191.49	189.92	191.43
8+50	Section N – just upstream of north lake shore	194.70	197.33	192.57	194.83	192.57	194.85
9+50	Upstream of lake	193.88	197.84	193.88	197.72	193.88	197.83
11+00	Upstream of project limits	198.65	200.64	196.96	199.29	196.96	199.24

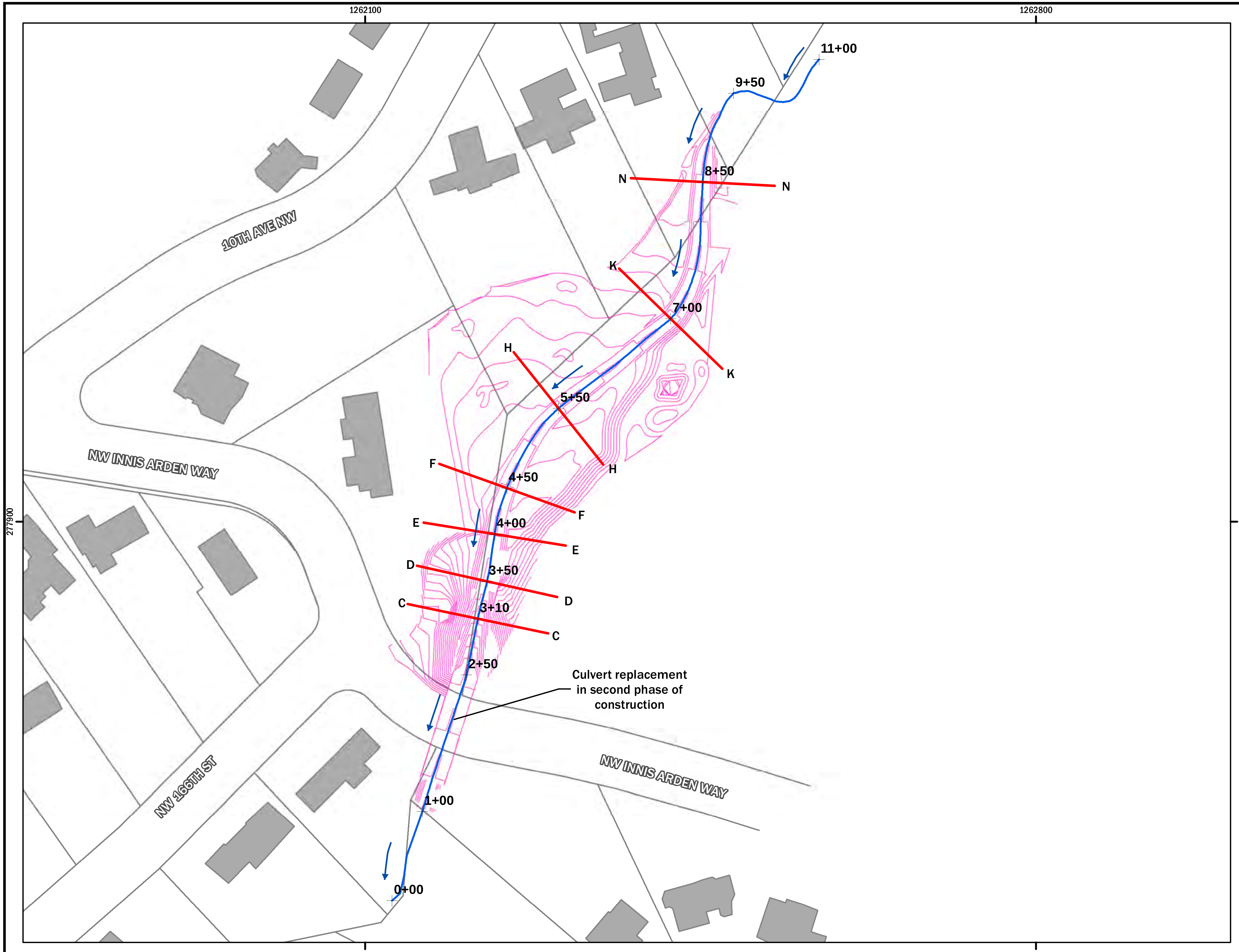


**Figure B-2.
Cross-section Locations for Comparison of Modeled Water Surface Elevations.**

Legend

- Proposed channel alignment
- Flow direction
- Cross-section
- Station
- Parcel
- Building
- Phase 1 and 2 - Proposed Grading

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

The results for proposed conditions following Phase 1 and then Phase 2 of construction indicate that there will be no noticeable changes in flow depths or velocities upstream or downstream of the project area. The interim conclusions regarding such impacts presented in Herrera (2016) still stand.

Phase 1 Results

Phase 1 model results suggest a backwater condition will form upstream of the existing culverts in an extreme flood (if such a flood occurs before the culverts are removed in the second phase of construction, as seen in the 100-year flood model results), beginning with about 0.8 foot of increased peak water level in the 100-year flood in the vicinity of the concrete pad at the culvert entrances and extending upstream through a portion of the former lake bed. This is due to the limited flow conveyance capacity of the existing culverts beneath NW Innis Arden Way. This backwater and resulting water surface rise would only occur in the short reach between the existing culvert inlets and the area where the dam will be removed. Throughout the drained lake area water surface elevations will decrease in the 100-year flood. Following Phase 1 construction the prevailing flow velocities in a major flood event will drop significantly as the flow approaches the existing culverts (see results in Attachment B2) due to this backwater effect. Some sediment deposition may occur in the channel in this area until Phase 2 is constructed. Deposition would likely be minor within the first decade (if it takes that long to build Phase 2) and most of the deposition would likely occur in the area that would require excavation as part of the Phase 2 channel connection at the head of the Phase 2 "roughened channel" section upstream of the new culvert structure.

Phase 2 Results

Following Phase 2 construction, the model results show no backwater effect upstream of the new culvert under NW Innis Arden Way, which is expected due to the much larger size of that proposed structure compared to the existing culverts. Simulated flow depths and velocities upstream of station 5+00 are effectively identical in the Phase 1 and Phase 2 model results (upstream of the Phase 1 backwater effect). The proposed channel within the lake bed is sloped at 2% and the simulated flow velocities in that portion of the new channel exceed 7 to 8 feet per second in the 100-year flood event. In the 2-year flood event the simulated velocities are relatively high at 4 to 5 feet per second in this area of the site. Those high velocities suggest that coarse gravel and cobble augmentation of the stream bed will be required to establish an initially stable channel. Flow velocities of this magnitude can erode sands and fine gravels but will only induce minor erosion if the streambed is composed of coarser gravels and cobbles. The relatively high flow velocities will also induce scour below log structures. Scour countermeasures such as larger cobble (used in the streambed mix) may be required 2 to 3 feet below the log structures to mitigate for scour to minimize potential to undermine a log structure. Some scour is desired since it creates pool habitat near logs that can provide cover for juvenile fish and resting areas for adult salmonids moving upstream to spawn.

Channel Design Hydraulics

Based on these model results, the new stream channel design geometry was adjusted to function like a 3-stage channel with the first two stages (a low flow channel and a small terrace/bench above it) containing most of the 2-year flood peak flow. Flows greater than the 2-year flood event can overtop more of the second stage channel width and occupy the third stage area, which is adjacent floodplain in selected areas of the site. The two-stage channel width was established as 15 feet wide based on the model results, to allow for an inset floodplain bench and for the low flow channel to "wiggle" around within the 15-foot channel. The new stream bank is higher in the design along the right (west) bank near the upstream end of the project site to keep the channel and most flow in the park property. The model results show no overbank flooding along the right bank near the upstream end of the project area on private property. Farther downstream of that point, the stream channel design, as informed by iterative model runs, will allow flood water to spill into the restored floodplain over both the right and left banks.

Hydraulic model results for the low flow and 2-year flood flow suggest that the designed channel geometry meets the desired sediment transport and habitat functions. Simulated low flow velocities in the channel vary from 1 to 2.5 feet per second, and there are large areas where the model results indicate variable velocities ranging from 0.5 to 1.5 feet per second, which provide ideal edge habitat for fish along the channel length. The 2-year flood flow velocities of 4 to 5 feet per second in the main portion of the channel will start to move gravels, so the channel will likely be dynamic within the prescribed initial channel geometry. That is desirable, so the new channel is not artificially "locked in" to a low flow path. The model results indicate peak flow velocities on the second stage inset floodplain will range between 0.5 to 2 feet per second, providing flood refugia and good habitat even during larger flows. More importantly, this distribution of velocities suggest some dynamics in the main part of the channel but only minor erosion risks along the second stage (edges) of the channel, and the velocity distribution is very similar to the velocity distribution observed in the model results in the existing creek channel upstream of the project area.

Downstream Effects Assessment

The model results after Phase 2 construction show no change in peak water levels or flow velocities in the 2-year and 100-year flood events downstream of the project area. Compared to existing conditions, those same results show lowered flood water levels throughout the project area upstream of the new culvert.

Beyond evaluating potential changes in peak water surface elevations and flow velocities downstream of the project site, potential changes in the timing of the peak of a flood wave downstream of the new culvert were assessed. With the removal of any dam-like feature, changes to upstream flood storage can change the timing of downstream flooding, which can thereby change peak flow elevations and velocities. Figures B-3 and B-4 present a comparison of the pre- and post-project simulated flood hydrographs downstream of the project area for the

2-year and 100-year flood events, respectively. For each of these flood events the hydrograph is predicted to be similar as in existing conditions with a slight shift such that the flooding peak will occur between 12 to 20 minutes earlier. However, for both flood hydrographs analyzed the peak discharges are nearly identical to existing conditions (actually a slight decrease in peak flow for proposed conditions). The relatively similar slope and shape of the hydrographs indicates that the ramping up and down of the flood hydrograph through the project area will not be changed by dam removal and culvert replacement. This also indicates that the peak flow velocities in the 2- and 100-year flood events will be effectively unchanged compared to pre-project (existing) conditions.

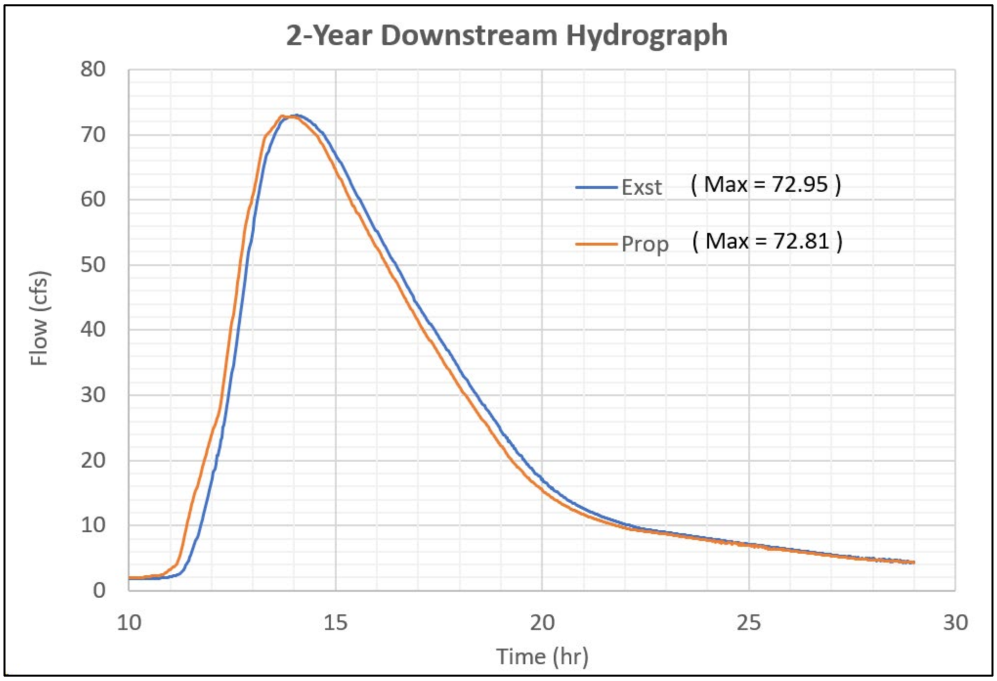


Figure B-3. Downstream Flood Hydrograph Comparison (2-Year Flood).

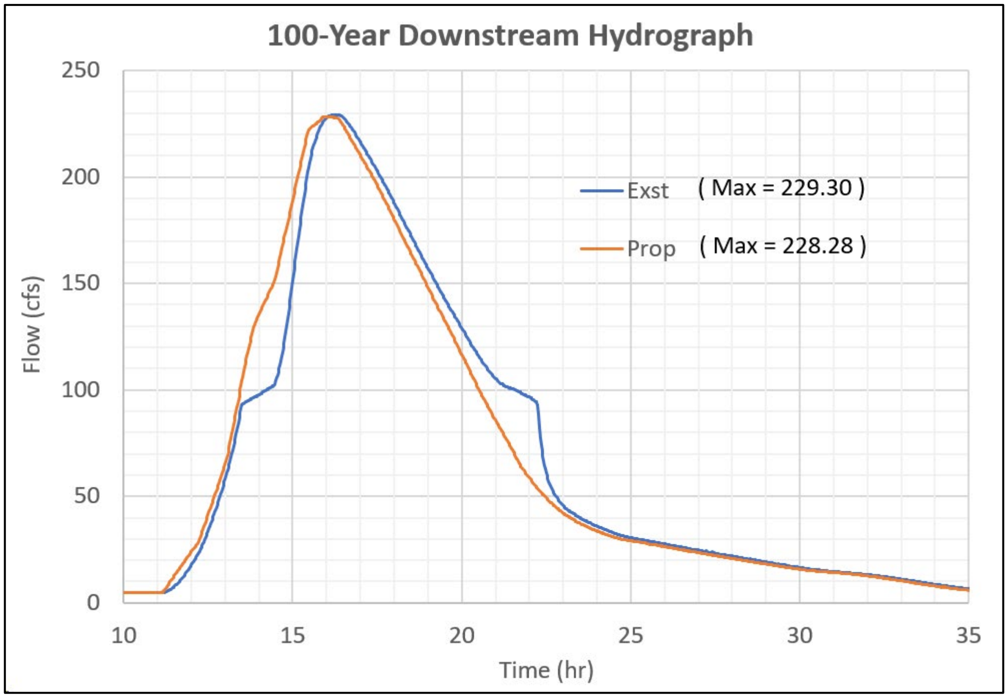


Figure B-4. Downstream Flood Hydrograph Comparison (100-Year Flood).

Figure B-4 shows a slight change in the 100-year flood hydrograph shape at approximately 100 cubic feet per second (cfs) that is a typical signature of a floodplain storage component. Under existing conditions, the model results indicate that the left (eastern) bank upstream of NW Innis Arden Way overtops when the discharge reaches about 100 cfs. This area will not overtop under proposed conditions. However, this storage component appears to be insignificant for the 100-year flow under existing conditions (which makes sense given that the flat area that flood flow can occupy east of the dam is not large), and thus eliminating that storage effect will not induce a significant change to the peak flow and only a small timing shift in the peak of the hydrograph.

The flood hydrograph comparisons in Figures B-3 and B-4 suggest that the lake has negligible flood storage capacity in larger flood events. The findings described here also suggest that existing erosion or flood hazards downstream of the project area are unlikely to change as a result of the project, regardless of whether future basin development, climate change, or other factors cause changes in the magnitude of flood events in Boeing Creek.

In summary, the model results presented in Attachments B and C to this memo provide a basis to be confident in the proposed channel design to meet project objectives, while not causing any adverse effects upstream and downstream of the project area.

REFERENCES

Chow, V.T. 1959. Open-Channel Hydraulics. McGraw-Hill, Inc.

Herrera. 2016. Technical Memorandum: Hydrologic, Hydraulic, and Geomorphic Analysis, Hidden Lake Dam Removal Project, Shoreline, Washington (draft). Prepared for the City of Shoreline by Herrera Environmental Consultants, Seattle, Washington. January 5.

Herrera. 2021. Hidden Lake Dam Removal Project Basis of Design Report, Shoreline, Washington. Prepared for City of Shoreline Public Works Department by Herrera Environmental Consultants, Seattle, Washington. August 21.

ATTACHMENT A

Existing Conditions Hydraulic Model Results

**Existing Depth (ft) 2YR Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.**

Legend

Contour (Ex. WSE 2 YR)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

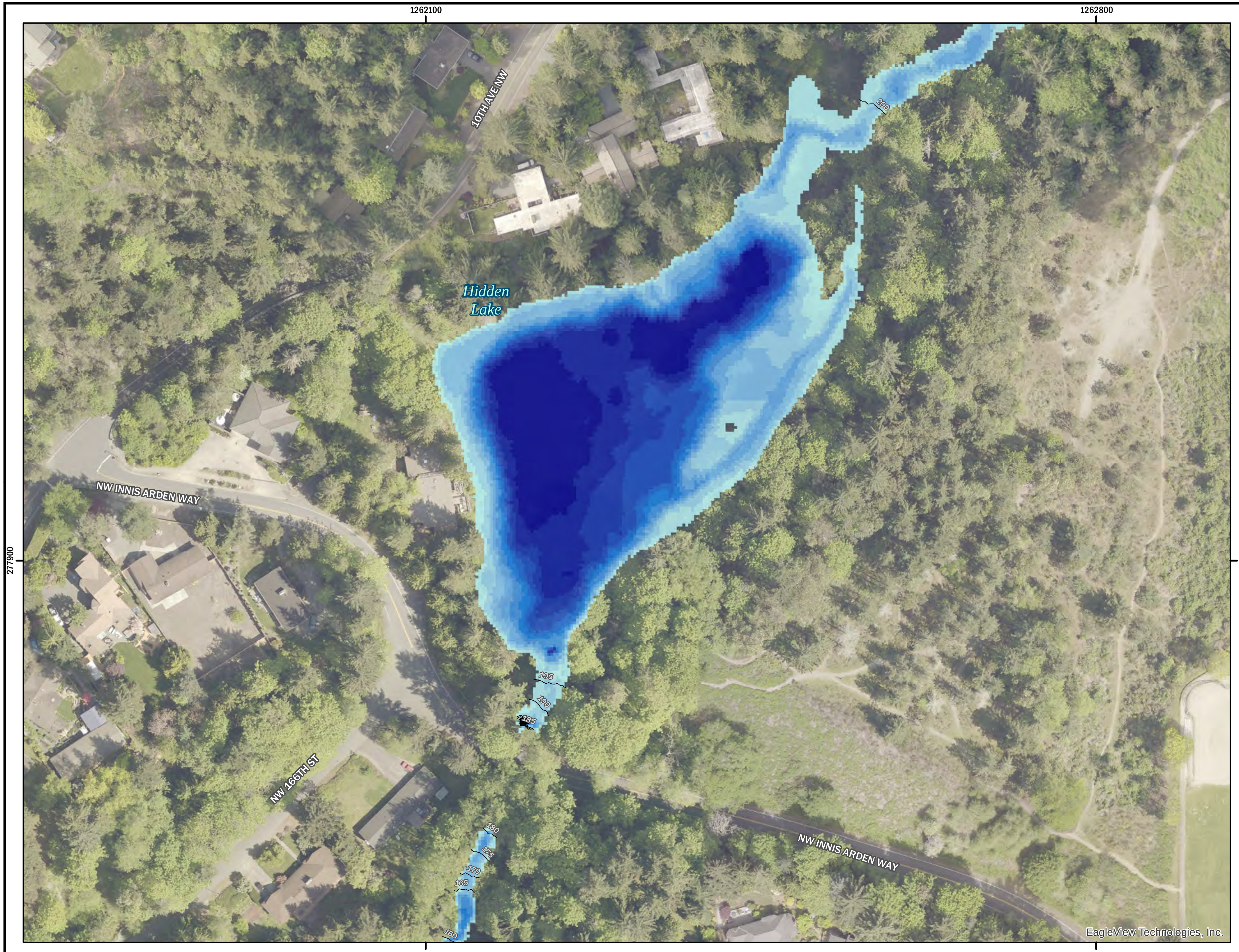
N

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200514\FigX_Depth_20200514.mxd (5/18/2020)



Existing Depth (ft) 100YR Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.

Legend

60 Contour (Ex. WSE 100 YR)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

N

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200514\FigX_Depth_20200514.mxd (5/18/2020)

Existing Depth (ft) Low Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

Contour (Ex. WSE Low Flow)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

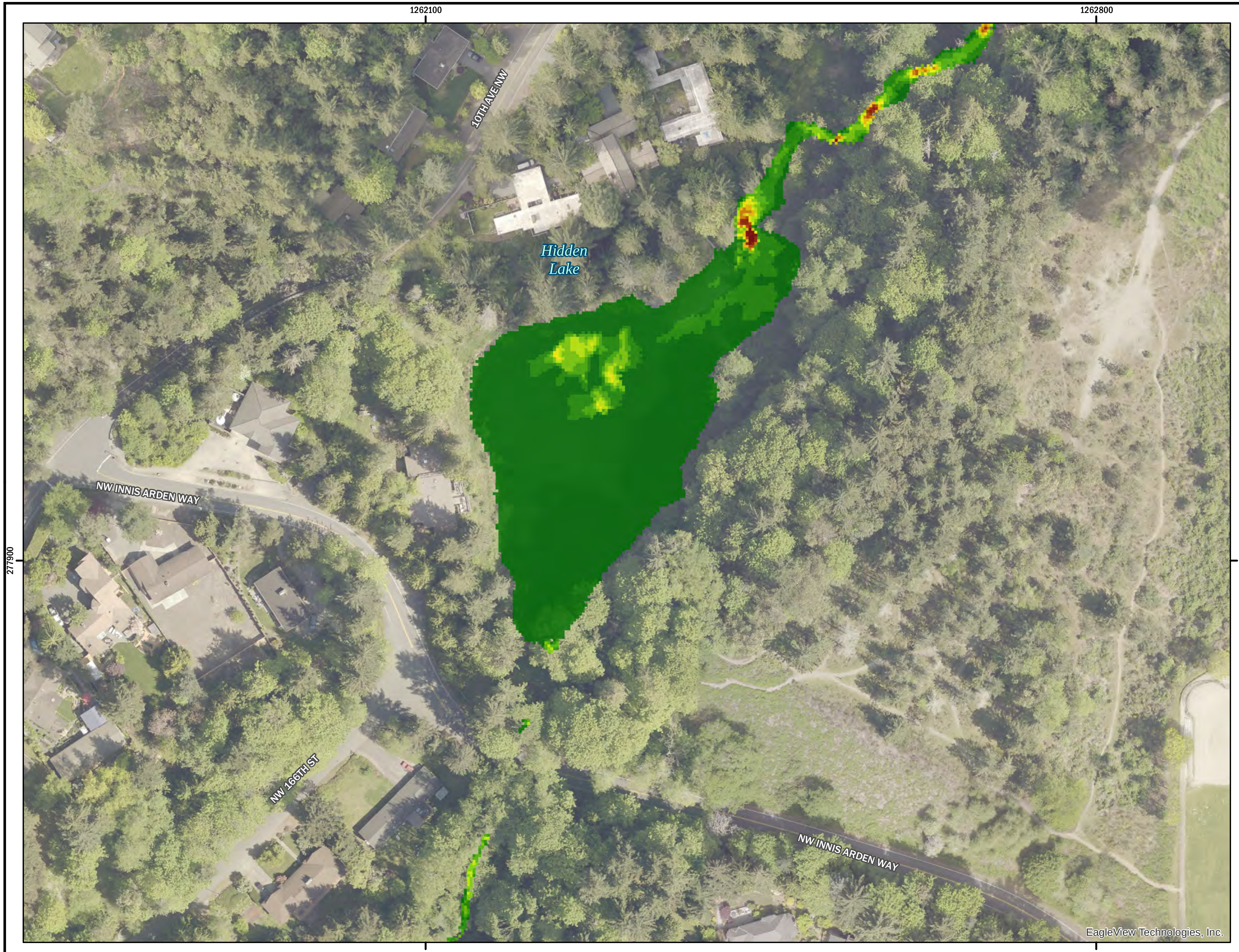
0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200514\FigX_Depth_20200514.mxd (5/18/2020)

Existing Velocities (ft/s) 2YR Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.



Legend

Velocity (ft/s)

- <= 0.5
- 0.5 to 1
- 1 to 1.5
- 1.5 to 2
- 2 to 2.5
- 2.5 to 3
- 3 to 3.5
- 3.5 to 4
- 4 to 4.5
- 4.5 to 5
- >5

N

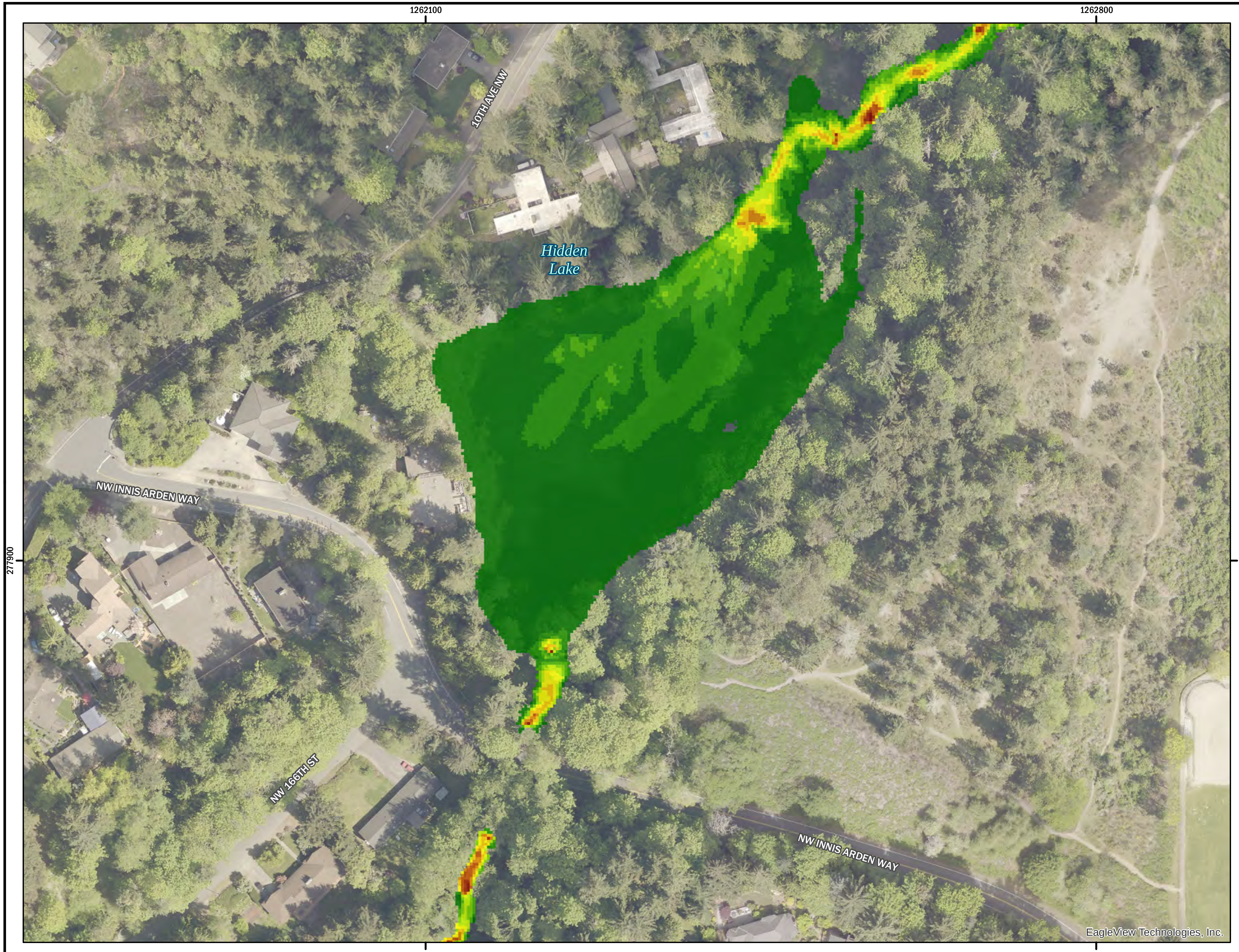
0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200514\FigX_Velo_20200514.mxd (5/18/2020)



Existing Velocities (ft/s) 100YR Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.

Legend

Velocity (ft/s)

- < 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 9
- 9 to 10
- > 10

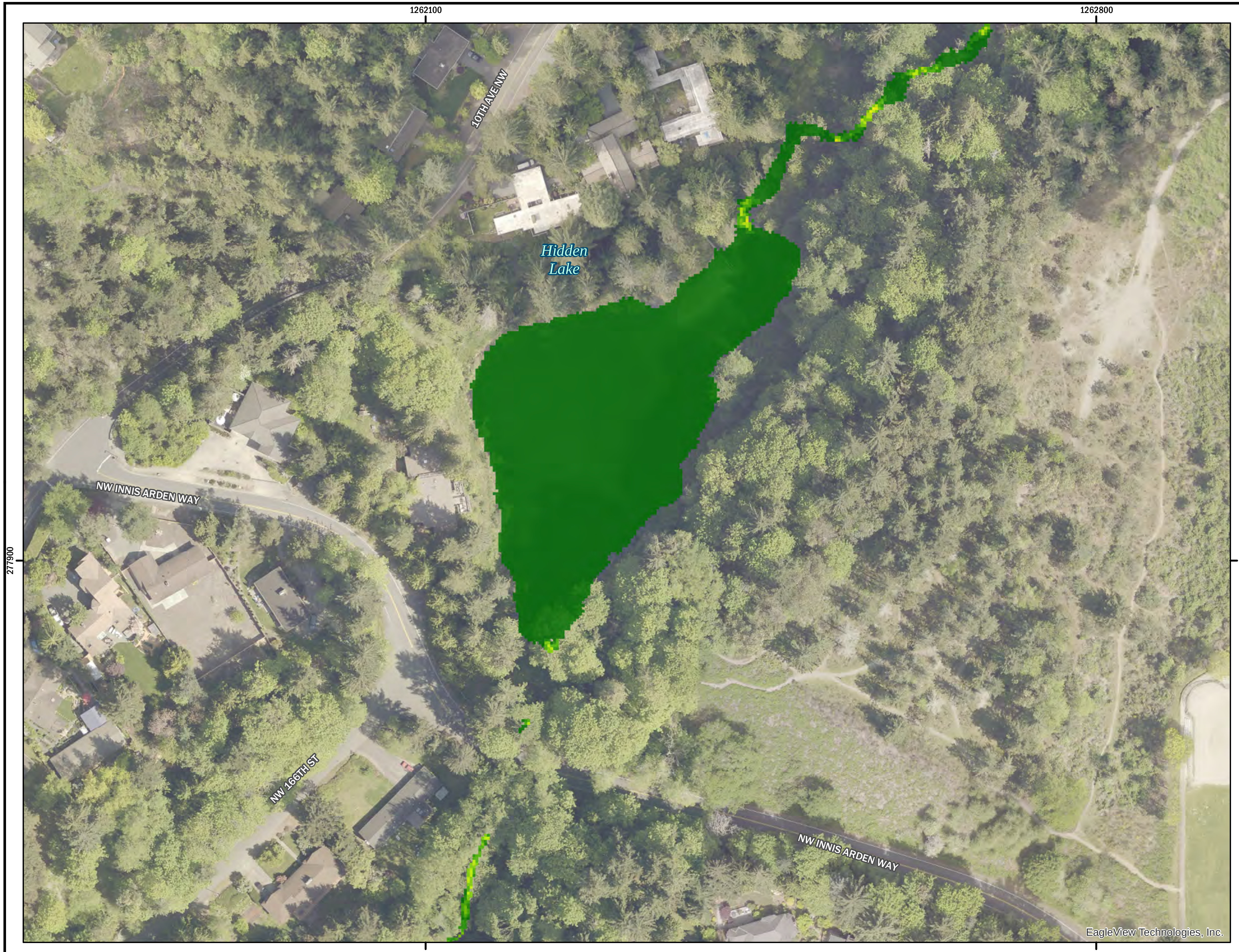
N

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

Existing Velocities (ft/s) Low Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.



Legend

Velocity (ft/s)

- <= 0.5
- 0.5 to 1
- 1 to 1.5
- 1.5 to 2
- 2 to 2.5
- 2.5 to 3
- 3 to 3.5
- 3.5 to 4
- 4 to 4.5
- 4.5 to 5
- >5

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

ATTACHMENT B

Proposed Conditions Model Results Following Phase 1 of Construction

Phase 1 - Proposed Depth (ft)
Low Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 - Proposed Grading
- Contour (Prop. WSE Low Flow)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Phase 1 - Proposed Velocities (ft/s)
Low Flow Event Hidden Lake Dam
Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 - Proposed Grading

Velocity (ft/s)

- <= 0.5
- 0.5 to 1
- 1 to 1.5
- 1.5 to 2
- 2 to 2.5
- 2.5 to 3
- 3 to 3.5
- 3.5 to 4
- 4 to 4.5
- 4.5 to 5
- >5

N

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Phase 1 Proposed Depth (ft)
2YR Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 - Proposed Grading
- 60 Contour (Prop. WSE 2 YR)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

N

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Phase 1 - Proposed Velocities (ft/s)
2YR Flow Event Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 - Proposed Grading

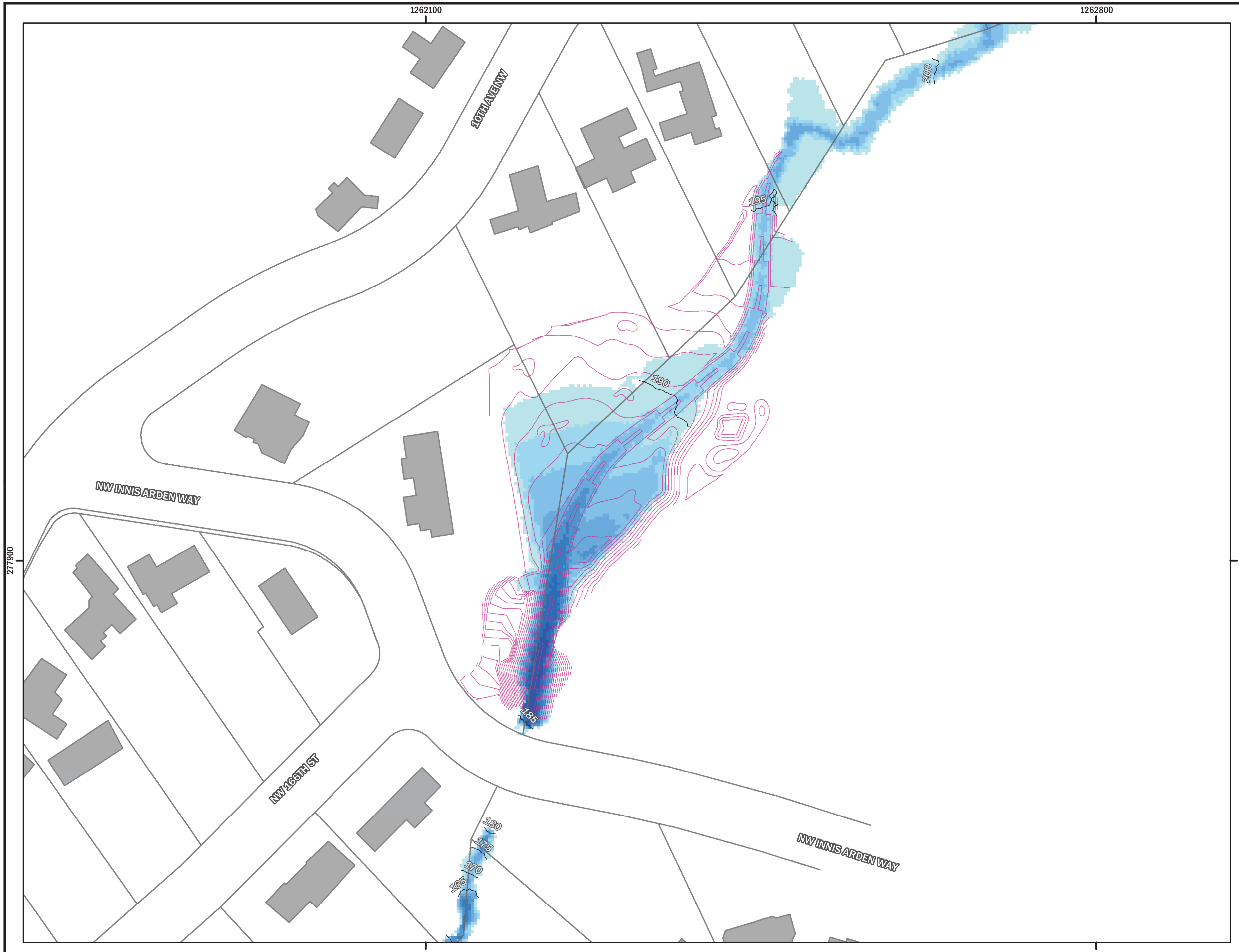
Velocity (ft/s)

- <= 0.5
- 0.5 to 1
- 1 to 1.5
- 1.5 to 2
- 2 to 2.5
- 2.5 to 3
- 3 to 3.5
- 3.5 to 4
- 4 to 4.5
- 4.5 to 5
- >5

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)



Phase 1 Proposed Depth (ft)
100YR Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 - Proposed Grading
- Contour (Prop. WSE 100 YR)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Phase 1 - Proposed Velocities (ft/s) 100YR Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 - Proposed Grading

Velocity (ft/s)

- < 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 9
- 9 to 10
- > 10

N

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

ATTACHMENT C

Proposed Conditions Model Results Following Phase 2 of Construction

Phase 2 - Proposed Depth (ft)
Low Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

Parcel	
Building	
Phase 1 and 2 - Proposed Grading	
Contour (Prop. WSE Low Flow)	

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200818_BODReport\FigX_PHASE2_PropDepth_20200818.mxd (8/19/2020)

Phase 2 - Proposed Velocities (ft/s)
Low Flow Event Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 and 2 - Proposed Grading

Velocity (ft/s)

- <= 0.5
- 0.5 to 1
- 1 to 1.5
- 1.5 to 2
- 2 to 2.5
- 2.5 to 3
- 3 to 3.5
- 3.5 to 4
- 4 to 4.5
- 4.5 to 5
- >5

0 50 100 200
Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

Phase 2 Proposed Depth (ft)
2YR Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 and 2 - Proposed Grading
- 60 Contour (Prop. WSE 2 YR)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

N

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200818_BODReport\FigX_PHASE2_PropDepth_20200818.mxd (3/10/2021)

Phase 2 - Proposed Velocities (ft/s) 2YR Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 and 2 - Proposed Grading

Velocity (ft/s)

- <= 0.5
- 0.5 to 1
- 1 to 1.5
- 1.5 to 2
- 2 to 2.5
- 2.5 to 3
- 3 to 3.5
- 3.5 to 4
- 4 to 4.5
- 4.5 to 5
- >5

N

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

Phase 2 Proposed Depth (ft)
100YR Flow Event
Hidden Lake Dam Removal Project
Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 and 2 - Proposed Grading
- 60 Contour (Prop. WSE 100 YR)

Depth (ft)

- <= 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 8
- 9 to 10
- > 10

N

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington
State Plane North (ft)

Produced By: GIS
Project: K:\Projects\Y2018\18-06771-000\Project\GISWorking\Modeling\ModelingPlots_20200818_BODReport\FigX_PHASE2_PropDepth_20200818.mxd (3/10/2021)

Phase 2 - Proposed Velocities (ft/s) 100YR Flow Event Hidden Lake Dam Removal Project Shoreline, Washington.

Legend

- Parcel
- Building
- Phase 1 and 2 - Proposed Grading

Velocity (ft/s)

- < 1
- 1 to 2
- 2 to 3
- 3 to 4
- 4 to 5
- 5 to 6
- 6 to 7
- 7 to 8
- 8 to 9
- 9 to 10
- > 10

N

0 50 100 200

Feet

HERRERA

Aerial: King County (2019)
Coordinates: NAD 1983 Washington State Plane North (ft)

APPENDIX C

Streambed Material Sizing Calculations

WDFW Water Crossing Guidelines (2013)

Step 1 - Assumed culvert span

15 = Assumed Channel Width at Bankfull (ft)

Step 2 - Size Bed Material

Eqn 3-5, USACE EM 1110-2-1601

$$D_{30} = \frac{1.95\ S^{0.555}\ q^{2/3}}{g^{1/3}}$$

INPUT Variables

227.3 = 100-yr flow (cfs)
0.02 = Bed Slope of roughened cascade (ft/ft)
1.5 = Size factor of safety

OUTPUT (Calcs)

15.15 = q (unit discharge $ft^3/s/ft$)
0.56 = $D_{30,}$ using USACE Eqn 3-5, accounts for size factor of safety (cell B
0.84 = D_{84} assuming 1.5x $D_{30,}$ Fuller Thompson Eqn 6.5

Step 3 - Check Culvert Span

1.19 = Assumed larges particle size (ft)
4.76 = 4 times largest particle
YES <= Is culvert span greater than 4 times the largest particle size?

Step 4 - Create Material Gradation

See spreadsheet tab "Material Gradation Calcs"

Step 5 - Calculate EDF

Manning's Equation

$$V = \frac{k}{n} R_h^{2/3} \cdot S^{1/2}$$

INPUT Variables

25.2 = Fish passage Design flow (10% exced = 10-yr flow (cfs))

0.695 = depth estimate to match calculated Q to the right=>
(iterative input approach - red arrow)

0.07 = assumed Mannings' n value to use for calculations
(based on output from 3 equations to the right ==>>>)

OUTPUT (Calcs)

18.108 = WP (wetted perimeter) in feet (assumes 2:1 side slopes)
11.391 = A (area in ft^2)
0.629 = R (hydraulic radius)
25.174 = Q calculated to match fish passage flow
2.210 = V (velocity)

0.094 = n from Limerinos' equation
0.095 = n from Jarrette's equation
0.061 = n from Mussetter's equation

EDF = γQS/A *Equation 6.3, 2013 WDFW Water Crossing Guidelines*

2.760904 = EDF

Table 6.4, WDFW Water Crossing Design Guidelines 201

Slope ft/ft	Max EDF ft-lb/ft³/sec
0.02	5
0.04	10
0.06	15
0.08	20
0.12	30
0.16	40
0.20	50

Calculations using n-value equations

EDF Calc for 15 feet wide

Table 6.4: Recommended relationship between roughened channel slope and maximum EDF.

Slope ft/ft	Max EDF ft-lb/ft³/sec	EDF (calculated)	Depth	Velocity	n (assumed)	
0.015	3.75	2.08	0.692	1.801	0.060	
0.02	5	2.76	0.695	2.212	0.070	<= Design

$EDF_{(calc)}$ < Max Allowable EDF ✓

Bed Roughness n- value calculator (WDFW Water Crossing Design Guidelines 2013)

Limerinos' equation

$n = 0.0926R^{1/6} / [1.16 + 2\log(R/D_{84})]$ **Equation 6.6**

$$n = \frac{0.0926R^{1/6}}{1.16 + 2.00\log\frac{R}{d_{84}}}$$

$n = 0.1129R^{1/6} / [1.16 + 2\log(R/D_{84})]$ for SI Units
and referenced in Cha

0.629 = R is hydraulic radius (ft)
0.84 = D_{84} is dimension of 84th percentile particle
0.75 = R/D_{84} data (based on $0.9 < R/D_{84} < 69$)
n = 0.094 (applicable data range is $0.02 < n < 0.107$)

Jarrette's equation

$n = 0.39S_f^{0.38}R^{-0.16}$ **Equation 6.7**

$n = 0.39S_f^{0.38}R^{-0.16}$. for SI Units (ref Stream Hydrology- Introduc
and referenced in Chapter 3 of th

$n = 0.32 \cdot Sf^{0.38} \cdot R^{-0.16}$ in S.I. units

0.629 = R is hydraulic radius (ft)
0.02 = S_f is Bed Slope (based on $0.002 < S_f < 0.04$, but up to 0.0825 may be ok)
0.74767 = R/D_{84} data (based on $0.4 < R/D_{84} < 11$)
n = 0.095 (applicable data range is $0.03 < n < 0.142$ and velocity <3ft/s)

Mussetter's equation

$(8/f)^{1/2} = R^{1/6}/(ng)^{1/2}$ **Equation 6.5**
$(8/f)^{1/2} = 1.11(d_m/D_{84})^{0.46} (D_{84}/D_{50})^{-0.85} Sf^{0.39}$ **Equation 6.8**

$R^{1/6}/(ng)^{1/2} = 1.11(d_m/D_{84})^{0.46} (D_{84}/D_{50})^{-0.85} Sf^{0.39}$

$1.49R^{0.17}/(n)(g)^{0.5} = (8/f)^{1/2} = 1.11(y/D_{84})^{0.46} (D_{84}/D_{50})^{-0.85} S_f^{0.}$

32.2 = g is gravity (32.2 ft/s^2)
0.629 = R is hydraulic radius
0.695 = d_m is mean depth
0.841 = D_{84} is dimension of 84^{th} percentile particle
0.701 = D_{50} is dimension of 50^{th} percentile particle
0.02 = S_f is Bed Slope (based on $0.0054 < S_f < 0.168$)
0.7477 = R/D_{84} data (based on $0.25 < R/D_{84} < 3.72$)

n = 0.061 (applicable data range is $0.036 < n < 4.2$)

More References:

http://www.sciencedirect.com/science/article/pii/S0169555X97000111
Good one => http://www.jstor.org/pss/3673667

An Evaluation of Methods for Estimating Manning's n in Small Mountain Streams
W. Andrew Marcus, Keith Roberts, Leslie Harvey and Gary Tackman
Mountain Research and Development
Vol. 12, No. 3 (Aug., 1992), pp. 227-239
(article consists of 13 pages)

recommends Jarrett's equation

Material Gradation

INPUT Variables

0.84 = D_{84} (assuming 1.5x D_{30})
0.84 = P (percentage of the mixture smaller than D84)
0.50 = n (assume maximum density mix of rounded material, see below)

OUTPUT (Calcs)

1.19 =D_{100} (calculation from the Fuller-Thompson equation)

Chapter 3
Riprap Protection

EM 1110-2-1601
Change 1
30 Jun 94

Table 3-1
Gradations for Riprap Placement in the Dry, Low-Turbulence Zones

Limits of Stone Weight, lb[1], for Percent Lighter by Weight

D_{100}(max)	100		50		15		D_{30}(min
in.	Max	Min	Max[2]	Min	Max[2]	Min	ft
Specific Weight = 155 pcf							
9	34	14	10	7	5	2	0.37
12	81	32	24	16	12	5	0.48
15	159	63	47	32	23	10	0.61
18	274	110	81	55	41	17	0.73
21	435	174	129	87	64	27	0.85
24	649	260	192	130	96	41	0.97
27	924	370	274	185	137	58	1.10
30	1,268	507	376	254	188	79	1.22
33	1,688	675	500	338	250	105	1.34
36	2,191	877	649	438	325	137	1.46
42	3,480	1,392	1,031	696	516	217	1.70
48	5,194	2,078	1,539	1,039	769	325	1.95
54	7,396	2,958	2,191	1,479	1,096	462	2.19

Fuller-Thompson equation:
P=(d/D100)n
where:
d = particle size of interest, mm (ft)
P= percentage of the mixture smaller than d
D100= largest size material in the mix, mm (ft)
n = parameter that determines how fine the resulting mix will be.
A value of 0.5 produces a maximum density mix when particles are round
This equation can be rearranged to find any particle size, for example:
D16 = 0.321/nD50
D5 = 0.101/nD50
When distribution is calculated by a pebble count, D100, D84, D50 of the reference reach are taken directly from the surface pebble count, and smaller grain sizes are determined through use of the Fuller-Thompson equation (6.5). This is based on D50, and creates a simulated bed mix.

Fuller-Thompson equation:

$$P=(d/D_{100})^n$$

Equation 6.5

where:

d = particle size of interest, mm (ft)
P= percentage of the mixture smaller than d
D_{100}= largest size material in the mix, mm (ft)
n = parameter that determines how fine the resulting mix will be. A value of 0.5 produces a maximum density mix when particles are round

This equation can be rearranged to find any particle size, for example:

$$D_{16} = 0.32^{1/n}D_{50}$$

$$D_5 = 0.10^{1/n}D_{50}$$

When distribution is calculated by a pebble count, D_{100}, D_{84}, D_{50} of the reference reach are taken directly from the surface pebble count, and smaller grain sizes are determined through use of the Fuller-Thompson equation (6.5). This is based on D_{50}, and creates a simulated bed mix. (This application has not been field tested, and professional judgment is recommended).

WDFW Water Crossing Guidelines (2013)

Step 1 - Assumed culvert span

12 = Assumed Channel Width at Bankfull (ft)

Step 2 - Size Bed Material

INPUT Variables

227.3 = 100-yr flow (cfs)
0.08 = Bed Slope of roughened cascade (ft/ft)
1.5 = Size factor of safety

OUTPUT (Calcs)

$$D_{30} = \frac{1.95\ S^{0.555}\ q^{2/3}}{g^{1/3}}$$

18.94 = q (unit discharge $ft^3/s/ft$)
1.40 = D_{30} using U.S. Army Corps of Engineers reference, EM 1110-2-160
2.11 = D_{84} assuming 1.5x D_{30}

Step 3 - Check Culvert Span

2.5 = Assumed larges particle size (ft)
10 = 4 times largest particle
YES **<= Is culvert span greater than 4 times the largest particle size?**

Step 4 - Create Material Gradation

See spreadsheet tab "Material Gradation Calcs"

Step 5 - Calculate EDF

INPUT Variables

25.2 = Fish passage Design flow (10% exced = 10-yr flow (cfs))

0.853 = depth estimate to match calculated Q to the right=>
(iterative input approach - red arrow)

0.16 = assumed Mannings' n value to use for calculations
(based on output from 3 equations to the right ==>>>)

EDF = γQS/A *Equation 6.8*

10.76008 = EDF

OUTPUT (Calcs)

15.815 = WP (wetted perimeter) in feet (assumes 2:1 side slopes)
11.691 = A (area in ft^2)
0.739 = R (hydraulic radius)
25.177 = Q calculated to match fish passage flow
2.153 = V (velocity)

$$V = \frac{k}{n} R_h^{2/3} \cdot S^{1/2}$$

0.352 = n from Limerinos' equation
0.157 = n from Jarrette's equation
0.149 = n from Mussetter's equation

Calculations using n-value equations

EDF Calc for 10 feet wide

Table 6.4: Recommended relationship between roughened channel slope and maximum EDF.

Slope ft/ft	Max EDF ft-lb/ft³/sec	EDF (calculated)	Depth	Velocity	n (assumed)
0.08	20	11.22	0.943	2.250	0.160
0.10	25	14.49	0.920	2.330	0.170

EDF Calc for 12 feet wide

Table 6.4: Recommended relationship between roughened channel slope and maximum EDF.

Slope ft/ft	Max EDF ft-lb/ft³/sec	EDF (calculated)	Depth	Velocity	n (assumed)	
0.08	20	10.76	0.853	2.153	0.160	<= design EDF $_{(calc)}$ > Max Allowable EDF ✓
0.10	25	13.91	0.828	2.227	0.170	

Tool to Assess Gravel and Rock Gradations

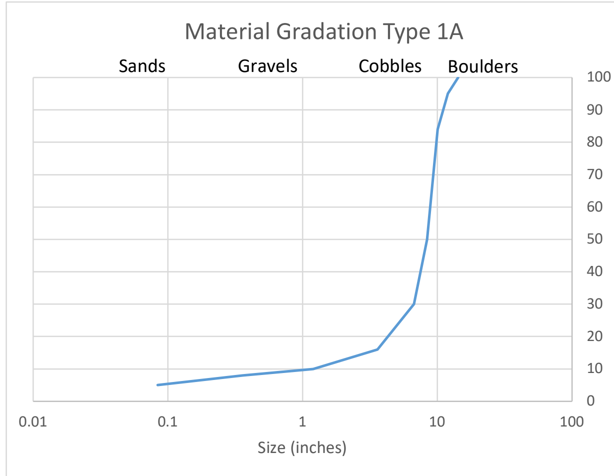
Type 1a (2% slope):

grain size (inch)	percent passing	size (ft)		
14.28	**100**	**1.19**		
12	95	1	D_{84}/D_{100}	0.705882
10.08	**84**	**0.84**	D_{84}/D_{50}	1.2
8.4	**50**	0.7	D_{84}/D_{16}	2.8
6.72	**30**	**0.56**		
3.6	16	0.3		
1.2	10	0.1		
0.36	8	0.03		
0.084	5	0.007		

Red value from USACE Roughened Channel Calcs

Grading coeff 1.68 $$S_{grad} = \frac{\left(\frac{D_{84}}{D_{50}} + \frac{D_{50}}{D_{16}}\right)}{2}$$

Sorting coeff (Folk) 0.29 $$s_{F\&W} = \frac{\phi_{84} - \phi_{16}}{4} + \frac{\phi_{95} - \phi_{5}}{6.6}$$



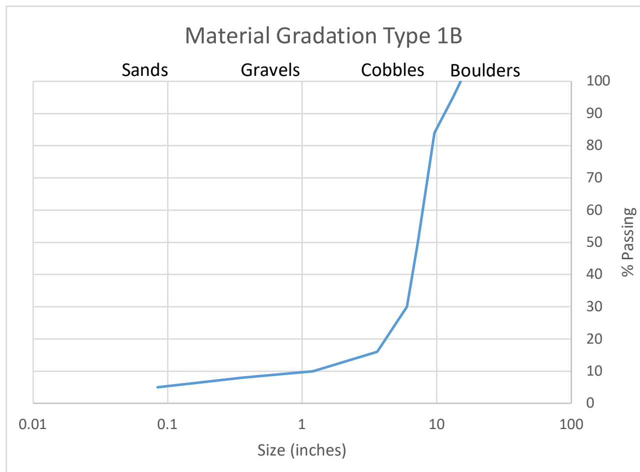
Type 1b (more well-graded at 2% slope):

grainsize (inch)	percent passing	size feet		
15	**100**	1.25		
13.2	95	1.1	D_{84}/D_{100}	0.64
9.6	**84**	0.8	D_{84}/D_{50}	1.333333
7.2	**50**	0.6	D_{84}/D_{16}	2.666667
6	**30**	**0.5**		
3.6	16	0.3		
1.2	10	0.1		
0.36	8	0.03		
0.084	5	0.007		

Red value from USACE Roughened Channel Calcs

Grading coeff 1.63

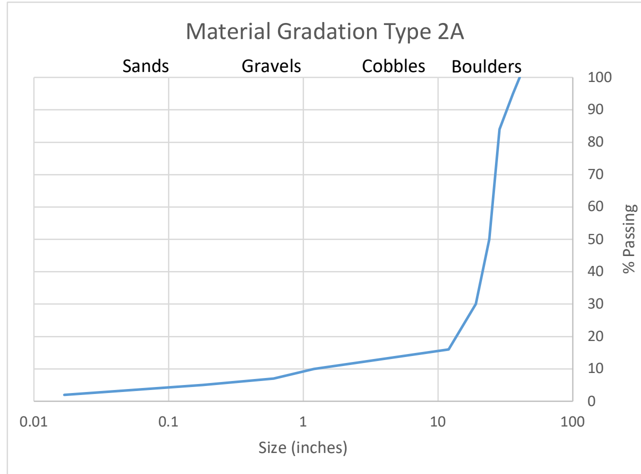
Sorting coeff (Folk) 0.29



Type 2a (8% slope):

grainsize (inch)	percent passing	size feet
40.44	**100**	**3.37**
36	95	3
28.56	**84**	**2.38**
24	**50**	2
19.08	**30**	**1.59**
12	16	1
1.2	10	0.1
0.6	7	0.05
0.18	5	0.015
0.0168	2	0.0014

D_{84}/D_{100}	0.706231
D_{84}/D_{50}	1.19
D_{84}/D_{16}	2.38



Red value from USACE Roughened Channel Calcs

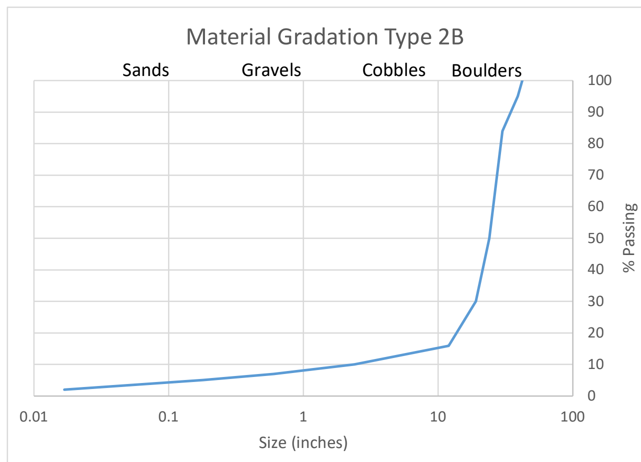
Grading coeff 6.00

Sorting coeff (Folk) 0.84

Type 2b (8% slope):

grainsize (inch)	percent passing	size feet
42	**100**	3.5
39	95	3.25
30	**84**	2.5
24	**50**	2
19.08	**30**	**1.59**
12	16	1
2.4	10	0.2
0.6	7	0.05
0.18	5	0.015
0.0168	2	0.0014

D_{84}/D_{100}	0.714286
D_{84}/D_{50}	1.25
D_{84}/D_{16}	2.5



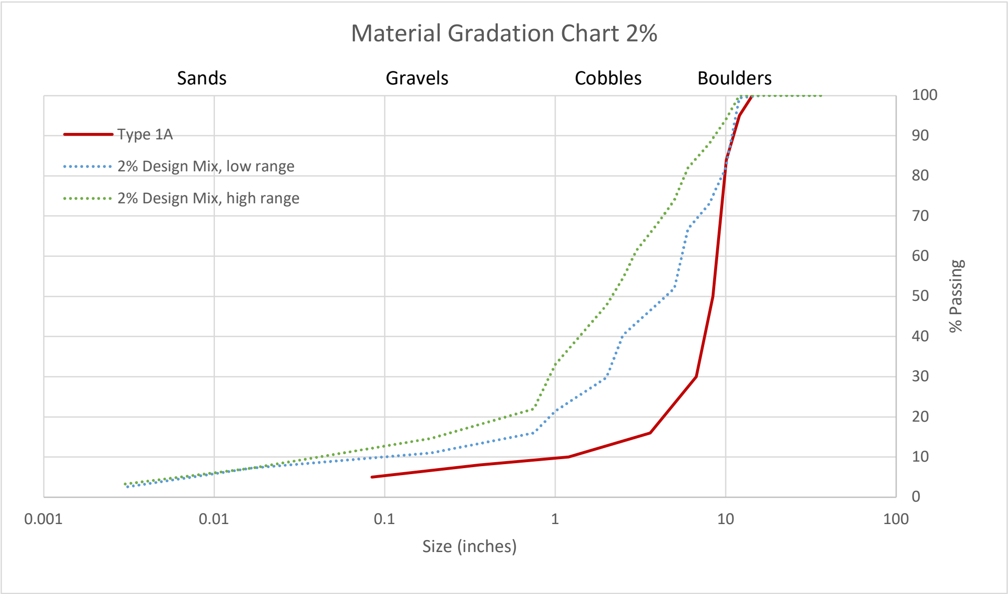
Red value from USACE Roughened Channel Calcs

Grading coeff 3.50

Sorting coeff (Folk) 0.83

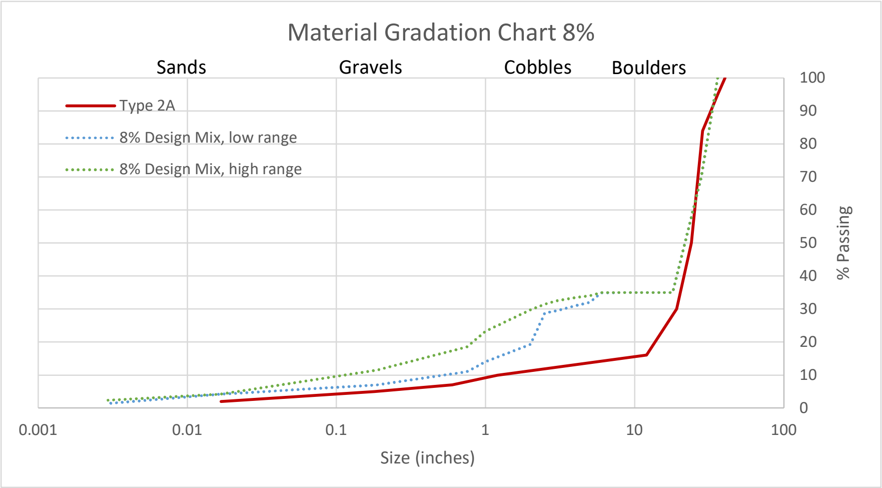
Final 2% Design mix

size (inch)	% Passing low	high
18	100	100
16	100	100
12	99.4	100
8	73	88
4	48.2	68.2
2.5	40.4	54.6
2	29.8	47.8
1	21.4	33
0.18	11	14.6
0.0168	7.2	7.2



Final 8% Design Mix

	size (inch)	% Passing low	high
	36	100	100
	28	100	100
	18	100	100
	16	100	100
	12	100	100
	8	100	100
	4	100	100
	2.5	99	100
No 4	2	65	95
No 40	1	50	85
No 200	0.18	26	44
	0.0168	16	16



Final Design Mix-- Proportions of Existing WSDOT Standard Mixes

	2% Design	8% Design
3 Man	0%	30%
2 Man	0%	35%
12" Cobble	60%	0%
8" Cobble	0%	0%
6" Cobble	20%	10%
Streambed Sediment	20%	25%
check	100%	100%

<-------- Adjust these percentages to change the design mix

Size	3 man bou	2 man bou	12" Cobble*		8" Cobble*		6" Cobble*		Streambed Sediment*		2% Design		8% Design	
	% passing	% passing	% Passing	% Passing	% Passing	% Passing Lo	% Passing	% Passing	% Passing	% Passing Lower	% Passing Upper	% Passing Lower	% Passing Upper	% Passing Lower
36	100	100	100	100	100	100	100	100	100	100	100	100	100	100
28	0	100	100	100	100	100	100	100	100	100	100	100	70	70
18	0	0	100	100	100	100	100	100	100	100	100	100	35	35
16	0	0	100	100	100	100	100	100	100	100	100	100	35	35
12	0	0	**100**	**99**	100	100	100	100	100	100	100	99.4	35	35
10	0	0	**90**	**70**	100	100	100	100	100	100	94	82	35	35
8	0	0	80	55	**100**	**99**	100	100	100	100	88	73	35	35
6	0	0	70	45	**90**	**70**	**100**	**99**	100	100	82	66.8	35	34.9
5	0	0	**60**	**30**	85	60	**90**	**70**	100	100	74	52	34	32
4	0	0	52	27	80	50	85	60	100	100	68.2	48.2	33.5	31
3	0	0	44	24	75	40	75	45	100	100	61.4	43.4	32.5	29.5
2.5	0	0	36	21	70	35	65	40	**100**	**99**	54.6	40.4	31.5	28.75
2	0	0	28	18	**60**	**30**	**60**	**30**	**95**	**65**	47.8	29.8	29.75	19.25
1	0	0	20	14	35	20	20	15	**85**	**50**	33	21.4	23.25	14
0.75	0	0	**10**	**10**	**10**	**10**	**10**	**10**	70	40	22	16	18.5	11
0.187	0	0	8	8	6	6	5	5	**44**	**26**	14.6	11	11.5	7
0.0165	0	0	6	6	4	4	2	2	**16**	**16**	7.2	7.2	4.2	4.2
0.0029	0	0	2	2	2	2	1	1	**9**	**5**	3.2	2.4	2.35	1.35

*WSDOT Standard mix, bold numbers indicate vaules pulled from the WSDOT standard mixes.

APPENDIX D

Log Structure Design Calculations

Log Revetment

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW **Checked By:** IBM
Completed On: 7/22/2020 **Checked On:** 7/22/2020

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
R11	13	10	X	3	10	30	32.0	62.4	324	633	308	925
L11	15	10		1	12	12	32.0	62.4	393	766	373	373
L12	15	15		2	18	37	32.0	62.4	589	1,149	560	1,119
L13	15	20		1	25	25	32.0	62.4	785	1,532	746	746
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
Racking	6	15		0	3	0	32.0	62.4	94	184	90	0
Totals						104	w/o racking				without racking	3,163
						104	w/racking				with racking	3,163
						4	cy with racking				% of total buoyant force due to racking	0.0%
						3	cy with racking within log ballast zone					

Structure Ballast Requirements

Recurrence Flow	Alluvium & Riprap Specific Weight	Water Specific Weight	Net/Buoyant Alluvium & Riprap Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Dry Ballast Weight	Ballast Volume Requirement	Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
	lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
100	134	62.4	71.8	1.5	1,943	2,802	48	2	2.05	23	35	Yes
2	134	62.4	71.8	1.5	1,210	3,534	43	2	2.05	21	35	Yes
100- Phase 1	134	62.4	71.8	1.5	4,745	0	66	2	2.05	32	35	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

Recurance Interval	Velocity	Depth of Flow	% of Structure Interacting with Flow	Net Buoyant Force
Yr	ft/s	ft	%	lb_f
100	6	2	41%	-4698
2	3.5	1.5	26%	-5395
100- Phase 1	6	4.5	100%	-2032

Specific Weight - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Specific Weight - Riprap

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
165.0	165.0	62.4	102.6

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

Log Revetment

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW
Completed On: 7/22/2020
Checked By: IBM
Checked On: 7/22/2020

Friction Coefficients

Wood	Clean Wood	0.25 - 0.5
Wood	Wet Wood	0.2
Wood	Clean Metal	0.2 - 0.6
Wood	Wet Metals	0.2
Wood	Stone	0.2 - 0.4
Wood	Concrete	0.62
Wood	Brick	0.6

https://www.engineeringtoolbox.com/friction-coefficients-d_778.html

friction coefficient= 0.2

Number of Friction Points: 0 no piles

Recurance Interval	FS Pure Buoyancy	Bouyant force to be resisted by the friction force on the piles to achieve dsired FS	F_{drag}	$F_{friction}$, Acting on all piles	Net Buoyant Force (includes friction)	FS Adjusted (includes friction)
Yr		lb_f	lb_f	lb_f	lb_f	
100	2.49	-3,048	580	0	48	12.09
2	2.71	-3,640	445	0	43	4.47

Note:
Friction is acting at 8 points total on the three piles in the bank log structure, on the three piles.
3 points on the upstream pile, 1 point on the middle pile, and 4 points on the downstream pile.

Habitat Type 1

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW **Checked By:** IBM
Completed On: 7/22/2020 **Checked On:** 7/22/2020

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
R2	21	20	X	1	53	53	32.0	62.4	1,693	3,302	1,609	1,609
L1	21	15		1	36	36	32.0	62.4	1,155	2,251	1,097	1,097
L2	21	20		1	48	48	32.0	62.4	1,539	3,002	1,462	1,462
L3	21	25		1	60	60	32.0	62.4	1,924	3,752	1,828	1,828
L4	21	30		1	72	72	32.0	62.4	2,309	4,503	2,194	2,194
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
Racking	6	15		0	3	0	32.0	62.4	94	184	90	0
Totals						269	w/o racking				without racking	8,190
						269	w/racking				with racking	8,190
						10	cy with racking				% of total buoyant force due to racking	0.0%
						7	cy with racking within log ballast zone					

Structure Ballast Requirements

Recurrence Flow	Saturated Alluvium & Riprap Specific Weight	Water Specific Weight	Net/Buoyant Alluvium & Riprap Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Dry Ballast Weight	Net Ballast Volume Requirement	Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
	lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
100	134	62.4	71.8	1.25	11,374	-1,137	150	6	1.6	94	109	Yes
2	134	62.4	71.8	1.25	5,388	4,849	111	4	1.6	70	109	Yes
100- Phase 1	134	62.4	71.8	1.25	10,237	0	143	5	1.6	89	109	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

Recurance Interval	Velocity	Depth of Flow	% of Structure Interacting with Flow	Net Buoyant Force
Yr	ft/s	ft	%	lb_f
100	6	2	111%	-3134
2	3.5	1.5	53%	-9502
100- Phase 1	6	4.5	100%	-4344

Specific Weight - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Specific Weight - Riprap

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
165.0	165.0	62.4	102.6

Assumptions
10% of volume for log w/out rootwad added to same size of log with rootwad

Habitat Type 1

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW
Completed On: 7/22/2020
Checked By: IBM
Checked On: 7/22/2020

Friction Coefficients

Wood	Clean Wood	0.25 - 0.5
Wood	Wet Wood	0.2
Wood	Clean Metal	0.2 - 0.6
Wood	Wet Metals	0.2
Wood	Stone	0.2 - 0.4
Wood	Concrete	0.62
Wood	Brick	0.6

https://www.engineeringtoolbox.com/friction-coefficients-d_778.html

friction coefficient= 0.2

Number of Friction Points: 1 no piles

Recurance Interval Yr	FS Pure Buoyancy	Bouyant force to be resisted by the friction force on the piles to achieve dsired FS lb_f	F_{drag} lb_f	$F_{friction}$, Acting on all piles lb_f	Net Buoyant Force (includes friction) lb_f	FS Adjusted (includes friction)
100	1.38	-3,073	1,880	376	-226	2.06
2	2.16	-7,911	1,439	288	-177	47.59

Note:
Friction is acting at 8 points total on the three piles in the bank log structure, on the three piles.
3 points on the upstream pile, 1 point on the middle pile, and 4 points on the downstream pile.

Habitat Type 2

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW **Checked By:** IBM
Completed On: 7/22/2020 **Checked On:** 7/22/2020

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
R5	21	35	X	1	93	93	32.0	62.4	2,963	5,778	2,815	2,815
L3	21	25		1	60	60	32.0	62.4	1,924	3,752	1,828	1,828
L4	21	30		4	72	289	32.0	62.4	2,309	4,503	2,194	8,774
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
Racking	6	15		0	3	0	32.0	62.4	94	184	90	0
Totals						441	w/o racking				without racking	13,418
						441	w/racking				with racking	13,418
						16	cy with racking				% of total buoyant force due to racking	0.0%
						12	cy with racking within log ballast zone					

Structure Ballast Requirements

Recurrence Flow	Saturated Alluvium & Riprap Specific Weight	Water Specific Weight	Net/Buoyant Alluvium & Riprap Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Dry Ballast Weight	Net Ballast Volume Requirement	Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
	lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
100	134	62.4	71.8	1.5	10,968	9,158	221	8	2.4	91	149	Yes
2	134	62.4	71.8	1.5	5,484	14,642	185	7	2.4	76	149	Yes
100- Phase 1	134	62.4	71.8	1.5	20,126	0	280	10	2.4	115	149	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

Specific Weight - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Specific Weight - Riprap

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
165.0	165.0	62.4	102.6

Recurance Interval	Velocity	Depth of Flow	% of Structure Interacting with Flow	Net Buoyant Force
Yr	ft/s	ft	%	lb_f
100	6	2	54%	-22711
2	3.5	1.5	27%	-28840
100- Phase 1	6	4.5	100%	-12475

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

Habitat Type 2

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW
Completed On: 7/22/2020
Checked By: IBM
Checked On: 7/22/2020

Friction Coefficients

Wood	Clean Wood	0.25 - 0.5
Wood	Wet Wood	0.2
Wood	Clean Metal	0.2 - 0.6
Wood	Wet Metals	0.2
Wood	Stone	0.2 - 0.4
Wood	Concrete	0.62
Wood	Brick	0.6

https://www.engineeringtoolbox.com/friction-coefficients-d_778.html

friction coefficient= 0.2

Number of Friction Points: 0 no piles

Recurance Interval Yr	FS Pure Buoyancy	Bouyant force to be resisted by the friction force on the piles to achieve dsired FS lb_f	F_{drag} lb_f	$F_{friction}$, Acting on all piles lb_f	Net Buoyant Force (includes friction) lb_f	FS Adjusted (includes friction)
100	2.69	-17,425	2,124	0	221	28.61
2	3.15	-21,857	1,626	0	185	5.65

Note:
Friction is acting at 8 points total on the three piles in the bank log structure, on the three piles.
3 points on the upstream pile, 1 point on the middle pile, and 4 points on the downstream pile.

Habitat Type 3

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW **Checked By:** IBM
Completed On: 7/22/2020 **Checked On:** 7/22/2020

Structure Buoyancy Calcs

Log Type	Avg Diameter	Length	Rootwad	Logs Per Structure	Individual Log Volume	Total Log Volume	Log Specific Weight	Water Specific Weight	Individual Log Weight	Individual Log Buoyant Force	Net Buoyant Force Per Log	Total Log Buoyant Force
-	in	ft	-	No.	ft^3	ft^3	lb_f/ft^3	lb_f/ft^3	lb_f	lb_f	lb_f	lb_f
R3	21	25	X	1	66	66	32.0	62.4	2,117	4,127	2,011	2,011
L3	21	25		2	60	120	32.0	62.4	1,924	3,752	1,828	3,656
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
					0	0	32.0	62.4	0	0	0	0
Racking	6	15		0	3	0	32.0	62.4	94	184	90	0
Totals						186	w/o racking				without racking	5,667
						186	w/racking				with racking	5,667
						7	cy with racking				% of total buoyant force due to racking	0.0%
						5	cy with racking within log ballast zone					

Structure Ballast Requirements

Recurrence Flow	Saturated Alluvium & Riprap Specific Weight	Water Specific Weight	Net/Buoyant Alluvium & Riprap Specific Weight	Factor of Safety	Submerged Ballast Weight Requirement	Dry Ballast Weight	Net Ballast Volume Requirement	Ballast Volume Requirement	Min Avg Depth of Ballast Over Each Log	Required Plan View Area of Backfill	Approximate Plan View Area of Backfill	Ok?
	lb_f/ft^3	lb_f/ft^3	lb_f/ft^3	-	lb_f	lb_f	ft^3	yd^3	ft	ft^2	ft^2	-
100	134	62.4	71.8	1.5	5,484	3,016	99	4	2.3	43	79	Yes
2	134	62.4	71.8	1.5	5,484	3,016	99	4	2.3	43	79	Yes
100- Phase 1	134	62.4	71.8	1.5	8,500	0	118	4	2.3	52	79	Yes

Density - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
kg/m3	kg/m3	kg/m3	kg/m3
2000	2150	1000	1150

Recurance Interval	Velocity	Depth of Flow	% of Structure Interacting with Flow	Net Buoyant Force
Yr	ft/s	ft	%	lb_f
100	6	2	65%	-11247
2	3.5	1.5	65%	-11247
100- Phase 1	6	4.5	100%	-7261

Specific Weight - Sands and Gravels

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
124.9	134.2	62.4	71.8

Specific Weight - Riprap

ρ(dry)	ρ(sat)	ρ(water)	ρ(buoyant)
lbf/ft3	lbf/ft3	lbf/ft3	lbf/ft3
165.0	165.0	62.4	102.6

Assumptions

10% of volume for log w/out rootwad added to same size of log with rootwad

Habitat Type 3

Project: Hidden Lake Dam Removal
Project #: 18-06771-000
Completed By: VW
Completed On: 7/22/2020
Checked By: IBM
Checked On: 7/22/2020

Friction Coefficients

Wood	Clean Wood	0.25 - 0.5
Wood	Wet Wood	0.2
Wood	Clean Metal	0.2 - 0.6
Wood	Wet Metals	0.2
Wood	Stone	0.2 - 0.4
Wood	Concrete	0.62
Wood	Brick	0.6

https://www.engineeringtoolbox.com/friction-coefficients-d_778.html

friction coefficient= 0.2

Number of Friction Points: 0 no piles

Recurance Interval Yr	FS Pure Buoyancy	Bouyant force to be resisted by the friction force on the piles to achieve dsired FS lb_f	F_{drag} lb_f	$F_{friction}$, Acting on all piles lb_f	Net Buoyant Force (includes friction) lb_f	FS Adjusted (includes friction)
100	2.98	-10,135	883	0	99	10.48
2	2.98	-10,135	677	0	99	10.48

Note:
Friction is acting at 8 points total on the three piles in the bank log structure, on the three piles.
3 points on the upstream pile, 1 point on the middle pile, and 4 points on the downstream pile.

Log Revetment

	Log Type	Log Diameter (in)	Log Length (ft)	Log Depth (ft)	Percent Cover	Area of Buried Log (ft)	Area Waited Depth (ft)
1	L13	15	20	2.33	50.0%	12.5	23.333333
2	R11	13	10	2.33	70.0%	7.583333333	16.333333
3	R11	13	10	2.33	70.0%	7.583333333	16.333333
4	R11	13	10	2.33	70.0%	7.583333333	16.333333
5	L12	13	15	1.25	0.0%	0	0
6	L12	13	15	1.25	0.0%	0	0
7	L11	15	10	1.25	80.0%	0	0

Area of Buried Logs (ft^2) 35.25
Area Weighted Average Depth (ft) 2.05

Note:
These calculations do not include the rootwads.
No chain included on these structures, the piles do not contribute to the soil cover

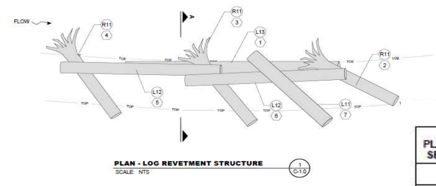
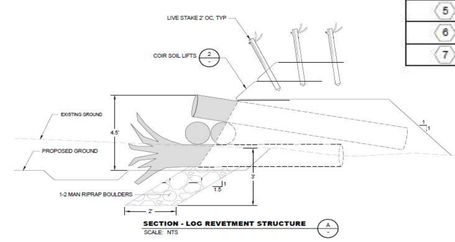


TABLE - LOG REVETMENT STRUCTURE LOG SCHEDULE:

LOG PLACEMENT SEQUENCE	LOG ID #	DIAMETER (IN)	LENGTH (FT)	ROOTWAD
1	L13	14-16	20	NO
2-4	R11	12-14	10	YES
5	L12	12-14	15	NO
6	L12	12-14	15	NO
7	L11	14-16	10	NO



Habitat Type 1

	Log Type	Log Diameter (in)	Log Length (ft)	Log Depth (ft)	Percent Cover	Area of Buried Log (ft)	Area Waited Depth (ft)
1	R2	21	20	4	60.0%	21	48
2	L4	21	30	3	60.0%	31.5	54
3	L2	21	20	2	50.0%	17.5	20
4	L3	21	25	2	60.0%	26.25	30
5	L1	21	15	3	50.0%	13.125	22.5

Area of Buried Logs (ft^2) 109.375
Area Weighted Average Depth (ft) 1.60

Note:
These calculations do not include the rootwads.
No chain included on these structures, the piles do not contribute to the soil cover

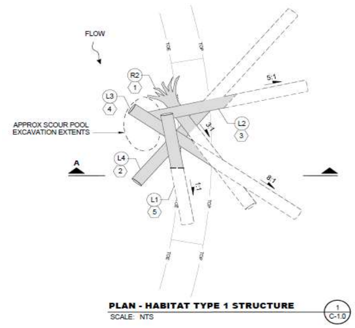
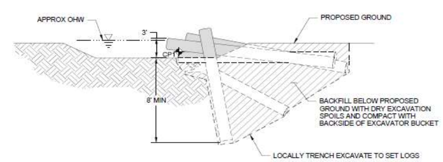


TABLE - HABITAT TYPE 1 STRUCTURE LOG SCHEDULE:

LOG PLACEMENT SEQUENCE	LOG ID #	DIAMETER (IN)	LENGTH (FT)	ROOTWAD
1	R2	18-24	20	YES
2	L4	18-24	30	NO
3	L2	18-24	20	NO
4	L3	18-24	25	NO
5	L1	18-24	15	NO



Habitat Type 2

	Log Type	Log Diameter (in)	Log Length (ft)	Log Depth (ft)	Percent Cover	Area of Buried Log (ft)	Area Waited Depth (ft)
1	R5	18	35	3	50.0%	26.25	52.5
2	L4	18	30	3	60.0%	27	54
3	L4	18	30	4	60.0%	27	72
4	L4	18	30	4	50.0%	22.5	60
5	L4	18	30	4	60.0%	27	72
6	L3	18	25	4	50.0%	18.75	50

Area of Buried Logs (ft^2) 148.5
Area Weighted Average Depth (ft) 2.43

Note:
These calculations do not include the rootwads.
No chain included on these structures, the piles do not contribute to the soil cover

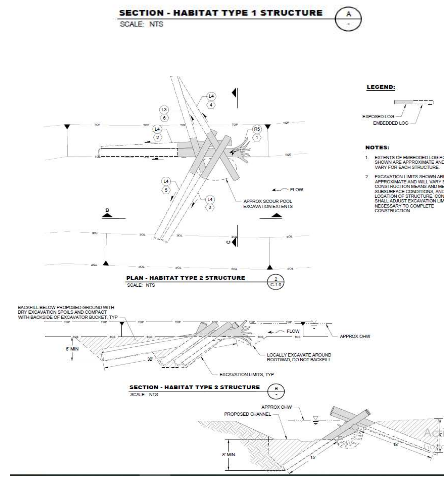


TABLE - HABITAT TYPE 2 STRUCTURE LOG SCHEDULE:

LOG PLACEMENT SEQUENCE	LOG ID #	DIAMETER (IN)	LENGTH (FT)	ROOTWAD
1	R5	18-24	35	YES
2	L4	18-24	30	NO
3	L4	18-24	30	NO
4	L4	18-24	30	NO
5	L4	18-24	30	NO
6	L3	18-24	25	NO

Habitat Type 3

	Log Type	Log Diameter (in)	Log Length (ft)	Log Depth (ft)	Percent Cover	Area of Buried Log (ft)	Area Waited Depth (ft)
1	R3	21	25	4	60.0%	26.25	60
2	L3	21	25	4	60.0%	26.25	60
3	L3	21	25	4	60.0%	26.25	60

Area of Buried Logs (ft^2) 78.75
Area Weighted Average Depth (ft) 2.29

Note:
These calculations do not include the rootwads.
No chain included on these structures, the piles do not contribute to the soil cover

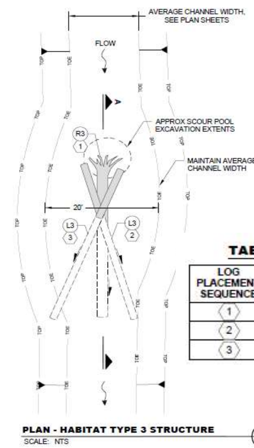
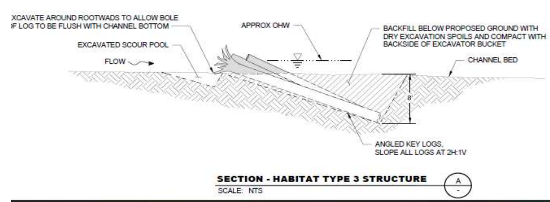


TABLE - HABITAT TYPE 3 STRUCTURE LOG SCHEDULE:

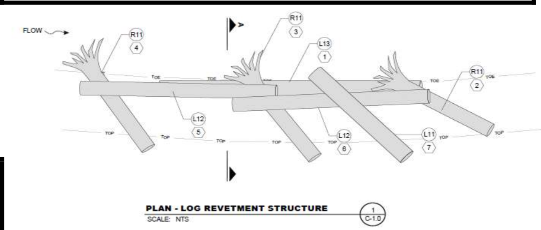
LOG PLACEMENT SEQUENCE	LOG ID #	DIAMETER (IN)	LENGTH (FT)	ROOTWAD
1	R3	18-24	25	YES
2	L3	18-24	25	NO
3	L3	18-24	25	NO



Input Date : 7-Jul-20
Designer : Valerie Wu
Checked by: IBM
Proj # 18-06771-000

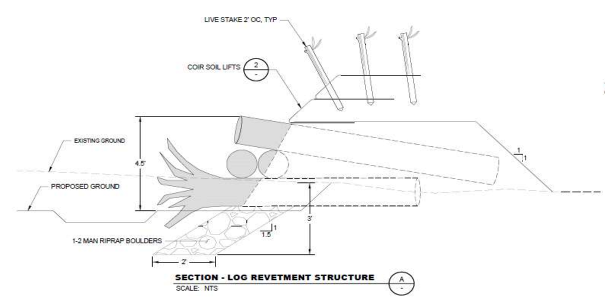
INPUT SUMMARY FOR CONSTRICTION SCOUR
FOR REVETMENT STRUCTURE

symbol	variable	unit	Mean Value	Notes
y_0	existing depth in contracted section before scour	ft	2.58	Input from 100-yr 5/14/20 modeling results
y_1	average depth in upstream main channel	ft	2.5	Input from 100-yr 5/14/20 modeling results
Q_1	flow in upstream main channel	ft^3/s	227	100-yr flow from the draft H & H report
Q_2	flow in contracted channel	ft^3/s	227	100-yr flow from the draft H & H report
W_1	channel bottom width of upstream main channel	ft	12	
W_2	channel bottom width in contracted section	ft	9	
D_{50}	median diameter of bed material	mm	25.4	1 inch median diameter per the selected design 2% streambed mix
S	Slope of energy grade line of main channel	ft/ft	0.02	



OUTPUT SUMMARY FOR CONSTRICTION SCOUR
FOR REVETMENT STRUCTURE

Output Summary Table		
Laursen Equation	0.43	ft
Parker Equation	0.52	ft
Average	0.47	ft



symbol	variable	unit	have data (what resource)	need data (how to get)
y_0	existing depth in contracted section before scour	m	HEC-RAS	verify with field measurements
y_1	average depth in upstream main channel	m	HEC-RAS	verify with field measurements
Q_1	flow in upstream main channel	m^3/s	gauge, HEC-RAS	
Q_2	flow in contracted channel	m^3/s	HEC-RAS	
W_1	channel bottom width of upstream main channel	m	LIDAR	
W_2	channel bottom width in contracted section	m	LIDAR, HEC-RAS	
D_m	diameter of smallest nontransportable particle in bed material	m	11 sediment samples (BoR)	Assume D100
D_{50}	median diameter of bed material	m	11 sediment samples (BoR)	
D_{95}	grain size for which 95% of bed material is finer	m	11 sediment samples (BoR)	
ω	settling velocity of bed material based on D_{50}	m/s		Table 5.4 in Julien; Dietrich curves
K_1	Exponent			Table 5.5
S	Slope of energy grade line of main channel	m/m	HEC-RAS	

$$\omega = \sqrt{(G - 1)gD_{50}}$$

20-Jul-20

CONTRACTION SCOUR (Bank Structures)

100 - year Hidden Lake 1

INPUT

existing depth at site where constriction will occur (before scour) =	y_0 =	0.786384	m
average depth in upstream main channel=	y_1 =	0.762	m
flow in upstream main channel=	Q_1 =	6.4	m^3/s
flow in contracted channel=	Q_2 =	6.4	m^3/s
channel bottom width of upstream main channel=	W_1 =	3.6576	m
channel bottom width in contracted section=	W_2 =	2.7432	m
slope of energy grade line of main channel=	S =	0.02	m/m
median diameter of bed material=	D_{50}=	0.0254	m
acceleration of gravity=	g =	9.81	m/s^2
specific gravity=	G =	2.65	

Determination of live-bed or clear-water scour

Live-bed: $V > V_c$

Clear-water: $V < V_c$

velocity for determining type of scour =	V =	2.50	m/s
your estimated velocity =	Ve =	2.5	m/s
Velocity= (Q/(y1*W1))=	Vp =	2.31	m/s
Grain size for Vc=	D_x=	0.0254	m
Critical Velocity = $V_{cD_x} = 6.19 y_1^{1/6} D_x^{1/3}$	V_c=	1.74	m/s
type of scour =	**live-bed scour**		

2.2-2.75 per Patti's email

LIVE-BED SCOUR

Laursen's 1960 equation (modified) [FHWA 2001]

$$\frac{y_2}{y_1} = \left(\frac{Q_2}{Q_1}\right)^{6/7} \left(\frac{W_1}{W_2}\right)^{k_1} \qquad d_{cs} = y_2 - y_0$$

$u_* = \sqrt{g y_1 S}$ $\quad \omega = \sqrt{(G-1) g D_{50}}$ fall velocity can also be found in table 5.4 in Julien, 1998

Mode of Bed Material Transport	k_1	u_*/ω
Mostly bedload	0.`59	`<0.50
Some suspended load	0.64`	0.50 to 2.0
Mostly suspended load	0.69	>2.0

OUTPUT

fall velocity of bed material based on D_{50}=	ω=	0.64	m/s
shear velocity in the upstream section=	u_*=	0.39	m/s
	u_*/ω=	0.60	
mode of bed material transport factor=	k_1=	0.64	
average depth in contracted section after scour=	y_2=	0.92	m
average contraction scour depth=	**d_{cs}=**	**0.13**	**m**

LIVE-BED SCOUR

Parker (1981) equation [Melville and Coleman 2000]

$$\frac{y_2}{y_1} = \left(\frac{W_1}{W_2}\right)^{k_4}$$

k_4=exponent that can vary from 0.675 to 0.825

k_{4min}= 0.675

k_{4max}= 0.825

OUTPUT

average depth in contracted section after scour=	y_{2min}=	0.93	m
average depth in contracted section after scour=	y_{2max}=	0.97	m
average contraction scour depth (using k4=0.675)=	**d_{csmin}=**	**0.14**	**m**
average contraction scour depth (using k4=0.825)=	**d_{csmax}=**	**0.18**	**m**
average=		0.16	

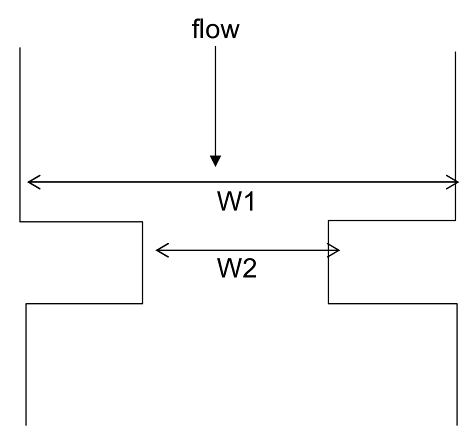
CLEAR-WATER SCOUR

Laursen's 1963 equation [FHWA 2001]

$$y_2 = \left[\frac{0.025\, Q_2^2}{D_m^{2/3} W_2^2}\right]^{3/7} \qquad d_{cs} = y_2 - y_0$$

OUTPUT

average depth in contracted section after scour=	y_2=	1.11	m
average contraction scour depth =	**d_{cs}=**	**0.32**	**m**



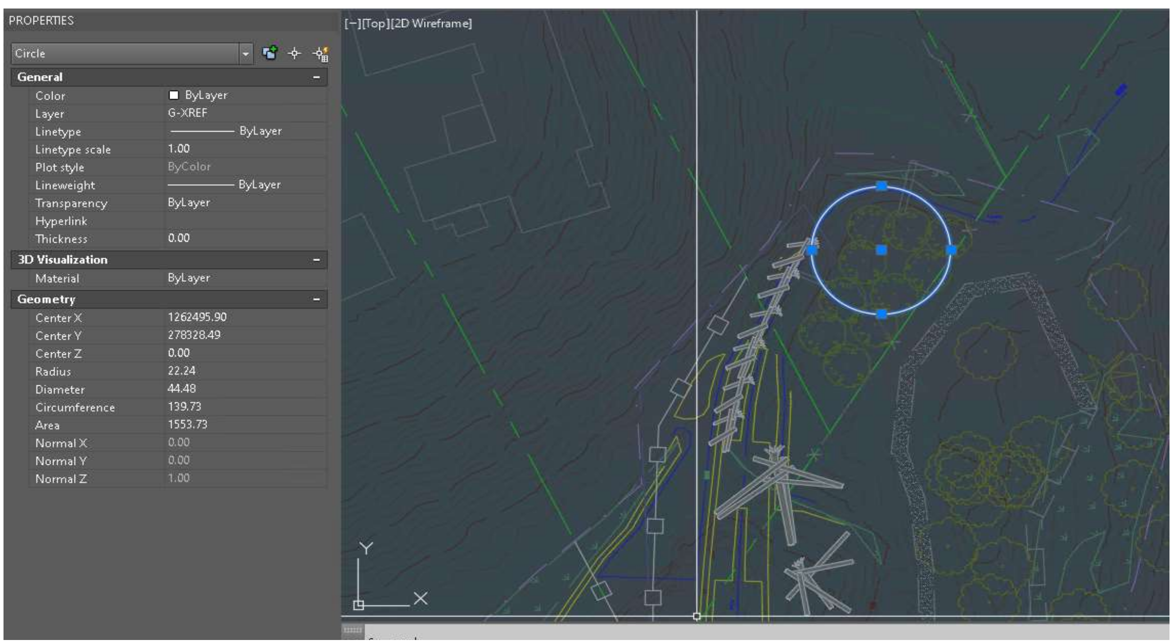
Input Date : 14-May-20
Designer : Valerie Wu
Checked by: IBM 15-May-20
Proj # 18-06771-000

OUTPUT SUMMARY FOR BEND SCOUR
Upstream of Revetment Structure

INPUT VARIABLES FOR CALCULATIONS				
approach flow depth =	y=	2.5	ft	Input from 10
flow depth at point of scour =	d=	2.5	ft	
radius of curvature at channel centreline =	R_C=	22	ft	measured in C
Width of flow =	W =	10	ft	Assumes bank

OUTPUT SUMMARY FOR BEND SCOUR
Upstream of Revetment Structure

Esimated scour below existing bed		
Thorne Equation		1.92 ft
Maynord Equation		1.80 ft
	Average	1.86 ft



REVETMENT BEND SCOUR

INPUT			
approach flow depth =	y=	2.50	ft
flow depth at point of scour =	d=	2.50	ft
radius of curvature at channel centreline =	R_C=	22.00	ft
Width of flow =	W =	10.00	ft

Thorne Equation (Thorne et al. 1997; cited in WDFW, 2002)

$$d_{as} = y\left[1.07 - \log\left(\frac{R_c}{W} - 2\right)\right] \quad for \;\; 2 < \frac{Rc}{W} < 22$$

OUTPUT			
	Rc/W =	2.20	
bend scour=	**d_{as}=**	**1.92**	**ft**

INPUT			
approach flow depth=	y=	2.50	ft
flow depth at point of scour =	d=	2.50	ft
radius of curvature at channel centreline =	R_C=	22.00	ft
Width of flow =	W =	10.00	ft

Maynord Equation (Maynord 1996)

$$d_{as} = y\left[1.8 - 0.051\left(\frac{R_c}{W}\right) + 0.0084\left(\frac{W}{y}\right)\right] \quad for \;\; 1.5 < \frac{Rc}{W} < 10 \quad and \quad 20 < \frac{W}{y} < 125$$

OUTPUT			
	Rc/W =	2.20	
bend scour=	**d_{as}=**	1.80	**ft**

Input Date : 14-May-20
Designer : Valerie Wu
Checked by: IBM

KEY
Green Text= Fill in data
Red Text= Calculated data

OUTPUT SUMMARY FOR LOCAL ABUTMENT SCOUR
Hidden Lake Culvert

INPUT VARIABLES FOR CALCULATIONS				Notes
approach flow depth (y_1 Froehlich+Ahmed, y_o NHCRP, y Lui)=	y=	2.5	ft	100-year depth (at XS 312.79) immediately upstream of the culvert
approach flow depth **on floodplain** (Froehlich)=	y_a=	2.5	ft	Assume y_a=y flow is contained in a trapezoidal channel
length of embankment=	L=	4	ft	top width minus culvert width
Effective length of flow obstruction (Froehlich, HIRE)=	L'=	3.0	ft	Assume 2/3 of L
velocity upstream of structure=	V=	4.09	ft/s	
Effective velocity (obstructed by abutment)=	V_e=	5.45	ft/s	
abutment shape coefficient (Froehlich, HIRE)=	K_1=	0.55		
angle of structure to flow=	θ=	90	degrees	
unobstructed channel width (Ahmed, NHCRP)=	W_1=	20	ft	
constricted channel width (NHCRP)=	W_2=	17	ft	
coefficient for abutment shape (Liu et Al.)=	K_L=	1		
correction factor for influence of channel bend (Ahmed)=	K_p=	1		
correction factor for influence of shape of structure (Ahmed)=	K_s=	1		
correction factor for influence of angle of attack (Ahmed)=	K_a=	1		
correction factor for influence of porosity (Ahmed)=	K_n=	0.9		
Effective flow (obstructed by abutment)=	Q_e =	41	ft^3/s	Assume effective flow is proportional to the ((XS area-unobstructed XS area)/ XS area)*Q
upstream flow=	Q =	204	ft^3/s	
upstream unit discharge (Q/W_1)=	q =	10.2	ft^2/s	
NHCRP Specific Inputs and Variables				
width of **floodplain**=	B_f=	3.0	ft	Assume B_f=L
constricted channel width=	W_2=	17	ft	
constricted unit discharge=	q_2=	12.0	ft^2/s	
flow depth prior to scour=	y_0=	2.5	ft	
Graphical factor determination				
type of abutment (Select from menu)=	Type=	wingwall		
type of scour calculation=	Type=	Live-bed		
unit discharge ratio=	q_2/q=	1.18		
graphically determined correction factor=	α=	1.15	*use lookup figures =>	Assumes the abutment is being scoured and decreasing in length http://onlinepubs.trb.org/onlinepubs/n
figure needed to determine correction factor=	Figure	8.1		
Additional Information	DO NOT NEED TO FILL OUT THIS SECTION			
Is critical shear stress known for the floodplain soil?		no		
particle size with 50 percent finer	D50=	0.058333333	ft	
K_u		11.17	English units	
NA		5	lb/ft^2	
NA		1.2	lb/ft^3	

Lookup Table (in following Tabs)		
Abutment type	Scour	Figure
Spill Through	Live-bed	8.9
Spill Through	Clear-water	8.11
Wingwall	Live-bed	8.10
Wingwall	Clear-water	8.12

OUTPUT SUMMARY FOR LOCAL ABUTMENT SCOUR
Hidden Lake Culvert

Output Summary Table			
Froehlich Equation [FHWA 2001]	4.99	ft	<= from HEC-18
Liu et. Al. Equation [Hoffmans and Verheij, 1997]	2.33	ft	
HIRE Equation		ft	<= from HEC-18
Ahmed Equation [Hoffmans and Verheij, 1997]	4.12	ft	
NCHRP 24-20 Equation	0.80	ft	<= from HEC-18
Average	3.06	ft	

HIRE Check
7.7 <= HIRE Scour (ft)
L / y_1 = 1.6 <= if < 25, then HIRE is not
yes Do you wish to exclude HIRE

LOCAL ABUTMENT SCOUR (Culvert Structure)

INPUT

approach flow depth (y_1 Froehlich+Ahmed, y_o NHCRP, y Lui)=	y=	0.76	m
approach flow depth **on floodplain** (Froehlich)=	y_a=	0.76	m
length of embankment=	L=	1.22	m
Effective length of flow obstruction (Froehlich, HIRE)=	L'=	0.91	m
velocity upstream of structure=	V=	1.25	m/s
effective velocity (obstructed/floodplain)=	V_e=	1.66	m/s
abutment shape coefficient (Froehlich, HIRE)=	K_1=	0.55	
angle of structure to flow=	θ=	90	degrees
unobstructed channel width=	W_1=	6.10	m
coefficient for abutment shape (Liu)=	K_L=	1	
correction factor for influence of channel bend (Ahmed)=	K_p=	1	
correction factor for influence of shape of structure (Ahmed)=	K_s=	1	
correction factor for influence of angle of attack (Ahmed)=	K_a=	1	
correction factor for influence of porosity (Ahmed)=	K_n=	0.9	
Unit discharge (Q/W_1)	q =	0.95	m^2/s
acceleration of gravity=	g=	9.81	m/s^2
NHCRP Specific Inputs			
width of floodplain=	B_f=	0.9	m
constricted unit discharge=	q_2=	1.1	m^2/s
flow depth prior to scour=	y_0=	0.8	m
Graphical factor determination			
type of abutment (Select from menu)=	Type=	wingwall	
type of scour calculation=	Type=	Live-bed	
unit discharge ratio=	q_2/q=	1.18	x-axis
graphically determined correction factor=	α=	1.15	y-axis
Additional Information		**DO NOT NEED TO FILL OUT THIS SECTION**	
Is critical shear stress known for the floodplain soil?		no	
particle size with 50 percent finer	D50=	0.01778	m
K_u		6.19	SI units
NA		239	Pa
NA		189	N/m^3

COMMON OUTPUT

angle of embankment to flow=	K_2=	1.00	
Froude number (upstream)=	Fr=	0.46	
Froude number (Approach Floodplain)=	Fr'=	0.61	m

Froehlich Equation (1989) for Live-bed scour [FHWA 2001]

$$\frac{d_{as}}{y_a} = 2.27\, K_1 K_2 \left(\frac{L'}{y_a}\right)^{0.43} Fr^{0.61} + 1.0$$

$$K_2 = \left(\frac{\theta}{90}\right)^{0.13}$$

θ<90° if embankment points dstr; θ>90° if embankment points upstr.

local abutment scour=	d_{as}=	1.52	m

Ahmed (1953) [Hoffmans and Verheij 1997]

$$d_{as} + y_1 = K_A K'_A \left[\frac{q}{1-m}\right]^{2/3} \qquad m = L/W_1$$

$$K_A = 2K_P K_S K_\alpha K_\eta \qquad K'_A = 2.14\, g^{-1/3}$$

OUTPUT

unit discharge=	q=	0.95	m^2/s
	m=	0.20	
local abutment scour=	d_{as}=	1.25	m

Liu, et al. (1961) [Hoffmans and Verheij 1997]

$$d_{as} = K_L\, y \left(\frac{L}{y}\right)^{0.4} Fr^{0.33}$$

OUTPUT

local abutment scour=	d_{as}=	0.71	m

HIRE Equation

$$\frac{y_s}{y_1} = 4\, Fr^{0.33} \frac{K_1}{0.55} K_2$$

OUTPUT

local abutment scour=	d_{as}=	2.35	m

NCHRP 24-20 Equation

$$y_{max} = \alpha_A y_c \text{ or } y_{max} = \alpha_B y_c$$

$$y_s = y_{max} - y_0$$

Selected flow depth including scour=	**yc=**	**0.88**	**m**
Live-bed flow depth including scour	yc=	0.88	m
Clear-water (Method 1) flow depth including scour	yc=	0.73	m
Clear-water (Method 2) flow depth including scour	yc=	0.01	m
ymax=	ymax=	1.01	m
local abutment scour=	y_s=	0.25	m

Lookup Table (Do not change)

Live-bed	Yes	0.88
Live-bed	No	0.88
Clear-water	No	0.73
Clear-water	Yes	0.01