Mill Creek Park Dam Improvements Project

Engineer's Report

May 2015

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List of Abbreviations and Acronyms

af	Acre-feet
cfs	Flow rate in cubic feet per second
El.	Elevation
f'c	Compressive strength of concrete
fps	Velocity in feet per second
ft	Foot or feet
ft^2	Area in square feet
ft^3	Volume in cubic feet
ft^3/s	Flow rate in cubic feet per second
fy	Yield strength
HGL	Hydraulic Grade Line
hp	Horsepower
H:V	Ratio of horizontal to vertical slope
ID	Inside diameter
lbs	Pounds
lf	Linear feet
MP	Mile post
NWS	Normal water surface
OD	Outside diameter
psi	Pressure in pounds per square inch
Q	Flow rate
rpm	Revolutions per minute
TDH	Total design head
WS	Water surface

1.0 Introduction

The City of Cosmopolis (City) plans to restore the function and value of the Mill Creek Dam and pond which were breached during a storm in November 2008. The breach also caused the failure of the Mill Creek Park loop trail's footbridge. The proposed Mill Creek Park Dam Improvements Project will replace the breached dam with a concrete gravity dam of similar structure; re-grade and re-vegetate the impoundment pond area with native wetland vegetation; replace the failed footbridge with a new footbridge; and construct a new fish passage facility around the dam. The project implements priorities identified in the City's Hazard Mitigation Plan and the Mill Creek Multi-Objective Plan.

In efforts to address the impaired dam, pond, and footbridge, the City has pursued several avenues and funding opportunities since 2008. Immediately following the breach, sufficient funds were not available to repair the damaged facilities. In 2010, the City prepared its Hazard Mitigation Plan, which identified the Mill Creek Dam project as a high priority hazard mitigation measure to address storm and flood hazards resulting from the breach. In 2012, the City developed the Mill Creek Multi-Objective Plan to expand efforts to assess the entire Mill Creek system and identify opportunities for improvements throughout the watershed. The City's top priorities in the Mill Creek Multi-Objective Plan are to replace the dam, restore the pond, and replace the footbridge.

This Engineer's Report evaluates how to restore the function and value of the dam, pond, and footbridge that existed prior to the breach, in order to implement the priorities in the City's Hazard Mitigation Plan and Mill Creek Multi-Objective Plan; achieve the City's multiple objectives of providing recreation, flood hazard reduction, habitat restoration, and fish passage; and meet the current requirements of federal, state, and local regulatory agencies. This report addresses the conceptual configuration (type, size and location) of the dam replacement, pond restoration, new fish passage facilities, and footbridge replacement. The report also provides environmental information used to prepare and submit the project's long lead Individual Permit (IP) application to United States Army Corps of Engineers (USACE or Corps).

2.0 Background

The dam was breached during a prolonged storm event of several days of rain in November 2008, and was caused when a portion of the hillside above the dam slid and undermined the right abutment (looking downstream), resulting in a breach of the entire impoundment area and the failure of the existing footbridge. Emergency repairs by the City stabilized the erosion on the hillside above the dam and the creek channel.

Prior to the 2008 breach, the concrete dam created a fish barrier in Mill Creek, impounded approximately two acres within Mill Creek Park, and created a pond that also served for recreational fishing. The footbridge that was located above the dam was part of the park's recreational loop trail. The dam and pond also provided the City with some ability to attenuate stream flows from proceeding downstream through residential and commercial areas of Cosmopolis. Flooding downstream of the dam along G, H, and I Streets has occurred during storm events since the dam breach, particularly during record rainfall in January 2015; and, the culvert at C Street collapsed during a large storm event in November 2012. Photos of the project area are shown in Figure 1.

Figure 1. Photos of the Project Area

Photo 1. Downstream Face of Dam prior to Breach

Photo 2. Upstream Face of Dam Repairs following Breach

Photo 3. Downstream Face of Dam Repairs following Breach

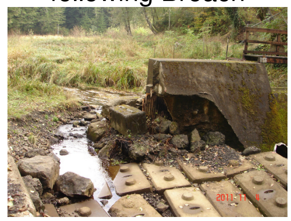


Photo 4. Downstream Face of Dam Repairs following Breach

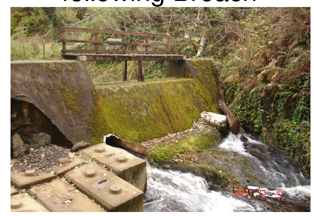


Photo 5. Mill Creek Pond Upstream of the Dam prior to Breach

Photo 6. Mill Creek through Former Pond Upstream of the Dam

Photo 7. Footbridge above Dam following Breach

Photo 8. Mill Creek Culvert Collapse at C Street

2.1 City's Mill Creek Multi-Objective Plan

This section summarizes the City's Mill Creek Multi-Objective Plan, originally prepared in 2012 and updated in 2013 and 2014.

Mill Creek flows through Mill Creek Park and through the center of the City of Cosmopolis, providing important recreational and natural resource amenities to the 1,500 citizens of Cosmopolis, and the greater Grays Harbor area due to its proximity to Highway 101. As a top priority of the Cosmopolis City Council and its citizens, the City prepared the Mill Creek Multi-Objective Plan to identify opportunities for improvements throughout the Mill Creek watershed.

The Multi-Objective Plan identifies, evaluates, and addresses four components of the Mill Creek stream system, resulting in a phased project implementation approach over multiple years and funding cycles. The Plan presents background and details of each component; the alternatives being considered and the City's preferred alternatives for each component; potential stakeholder partnerships; and potential funding sources. The City intends to update the Plan on a routine basis as more information and greater understanding of the improvement projects develop over time.

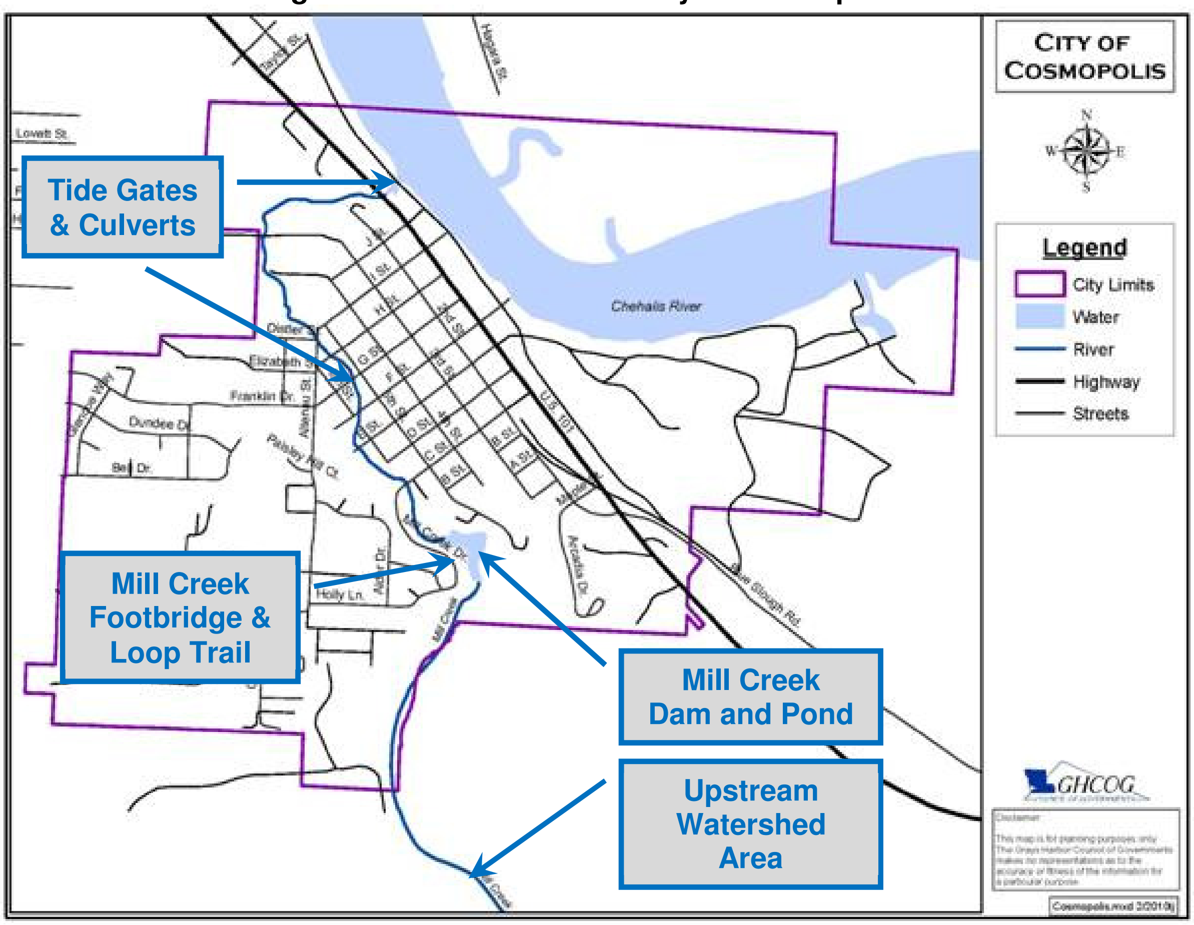
The four components of the Mill Creek stream system are described below and are shown on Figure 2:

- **Mill Creek Dam and Pond**: Prior to the 2008 breach, concrete dam impounded approximately two acres within Mill Creek Park and created a recreational fishing area. The pond was stocked with trout as the dam created a fish passage barrier. Resulting from the breach, the impoundment has reverted back to a stream channel. The City plans to replace the dam and the restore the pond.

- **Mill Creek Park Footbridge and Loop Trail**: The ½ mile loop trail is incomplete with the 2008 removal of the damaged footbridge that was located above the dam. The City plans to replace the footbridge and restore the loop trail, and desires to improve accessibility and add educational components to the loop trail.
- **Upstream Watershed**: Rebuilding the dam triggers the requirement to include fish passage around the dam, to the pond and upstream watershed. The City will investigate upstream watershed habitat improvements, particularly related to fisheries and other important species, in the future.
- **Tide Gates and Culverts**: Three tide gates are located at the discharge of Mill Creek to the Chehalis River, and at six different locations, fourteen 4-foot to 5-foot diameter culverts convey Mill Creek through the residential area of Cosmopolis from the Mill Creek Dam to the tide gates at the Chehalis River (the remainder of Mill Creek in this area is open channel). In the future, the City may assess the operation of the tide gates to improve stream and fisheries conditions in Mill Creek. Assessment of the culverts and stream channel capacity may also trigger upgrades to improve stream and fisheries conditions in Mill Creek.

The City plans to address Mill Creek improvements through stakeholder and funding partnerships. The City understands that multiple objectives must be achieved in the broader context of the entire Mill Creek stream system to provide the maximum stakeholder and funding benefit.

Figure 2. Mill Creek Stream System Components



2.2 Project Development

The Mill Creek Dam Improvements Project is being implemented in several phases using Task Orders. Brief descriptions are provided below of the task orders that have been completed and the task orders pending for completion:

- **Task Order 1 – Data Acquisition and Site Visit (Completed 2012)**
  Task Order 1 involved data collection and organization, field visit, identification and confirmation of alternatives to be evaluated, and development of a strategic plan that identified the overall "road map" for advancing the dam replacement project forward.
- **Task Order 2 – Initial Regulatory Consultation and Preliminary Design Criteria (Completed November 2013)**
  Meetings were held with regulatory agencies to discuss project goals, objectives, environmental compliance, and dam safety requirements for the replacement dam and pond. The intent was to establish a coordination process, identify studies, reports, and permits required by each agency, understand timelines and review processes, and initiate a sense of collaboration and teamwork. Initial regulatory consultations were conducted with the following:
  - Washington Department of Fish and Wildlife (WDFW): The key project element confirmed by WDFW is that fish passage will be required to be incorporated as part of the replacement dam.
  - Washington State Recreation and Conservation Office (RCO): RCO is the state agency responsible for managing the Federal Land and Water Conservation Fund (LWCF) grant which helped develop Mill Creek Park. As part of project improvements, RCO requires the recreation capacity of the park be retained, and not diminished through conversion to other uses, and that the outdoor recreational uses must be replaced with similar types of facilities. RCO concurred with the City's project intentions.
  - Washington State Department of Ecology Dam Safety Office (DSO): For the replacement dam, DSO requires verification of acceptable dam structural stability, soils (both embankment and foundation soils), type of structure, liquefaction and other seismic factors, and the project must establish the dam hazard classification to comply with DSO's 8-step decision framework to determine the design and performance goals.
  - Federal Emergency Management Agency (FEMA): The FEMA Region X Floodplain Management and Insurance Branch Chief suggested that it would only be necessary to meet with FEMA if the base flood elevations (BFE) shown on the City's existing Flood Insurance Rate Map (FIRM) were to be changed as a result of the project. If the BFEs are changed by the project, then the Risk Analysis Branch of the FEMA Region X office would need to be involved if the project results in any changes to the FIRM. The improvement project approach is to be consistent with and not modify the BFEs in the current FIRM, which was prepared when the dam was in place.
  - US Army Corps of Engineers Civil Works Regulatory Program and Permits (USACE): The USACE will require an individual permit based on the current site conditions. A nationwide permit is not available due to the amount of time that has lapsed since the 2008 event. Inundation of the impoundment area (Mill Creek Pond – the area immediately upstream of the dam) will be considered a "loss". The plan to rebuild the dam and inundate the former pond area will require an assessment of impacts to wetlands and the development of mitigating measures to decrease those impacts. Alternative designs for the dam and for the mitigating measures are required so the

project can demonstrate that the project represents the "least environmentally-damaging practicable alternative".

- **Task Order 2.1 – Initial Geotechnical, Environmental, and Survey Services (Completed November 2013)**
  To obtain information to bridge gaps between Task Orders 2 and 3, one day field visits, with associated reporting, were conducted focusing on geotechnical investigations near the dam, initial wetland identification, and base survey control and mapping. The data were used to complete Task Order 2 and served as the basis for related services under Task Order 3.
- **Task Order 3 – Field Data Collection and Engineer's Report (Current)**
  Field data were collected including geotechnical, environmental, and fisheries baseline data. This Engineer's Report evaluates how to restore the function and value of the dam, pond, and footbridge, including fish passage. The report addresses the configuration (type, size and location) of the improvements. The report also provides information necessary to prepare and submit the long lead individual permit application to USACE.
- **Task Order 4 - Final Design and Permitting (Future)**
  The project improvements will be developed into plans and specifications for bidding from construction contractors. Required permit applications will be prepared and submitted.
- **Task Order 5 –Bidding & Construction (Future)**
  Bid documents will be developed for the public construction contracting process.

3.0 Purpose of Project

Based on the City's strategic plans, multiple objectives, project development to date, and the project's identified regulatory requirements, the overall purposes of the Mill Creek Dam Improvements project are:

- Replace the breached dam with a concrete gravity dam of similar structure. The City has historically operated the dam and impoundment area for storage during rainfall/runoff periods and plans to restore this function with the replacement dam in order to provide one component of flood hazard reduction within the overall Mill Creek watershed. The replacement dam will meet DSO structural, seismic and hazard classification requirements.
- Restore the impoundment pond area by re-grading and re-vegetating with native wetland vegetation to improve wetland and aquatic habitat and restore recreational objectives. Recreational fishing in the pond area has been historically provided and the City plans to restore this recreational function with the added benefit of fish passage at the dam location. The restored pond will meet USACE Individual Permit requirements for impacts to wetlands and the development of mitigating measures to decrease those impacts. The restored pond will also meet RCO requirements to retain the recreation capacity of the park, not diminish through conversion to other uses, and replace outdoor recreational uses with similar types of facilities.
- Construct a new fish passage facility around the dam to improve aquatic habitat by addressing the fish barrier in Mill Creek at the project location. The new fish passage facility will meet WDFW requirements.
- Replace the failed footbridge with a new footbridge to restore recreational objectives. The new footbridge will also meet RCO requirements to retain the recreation capacity of

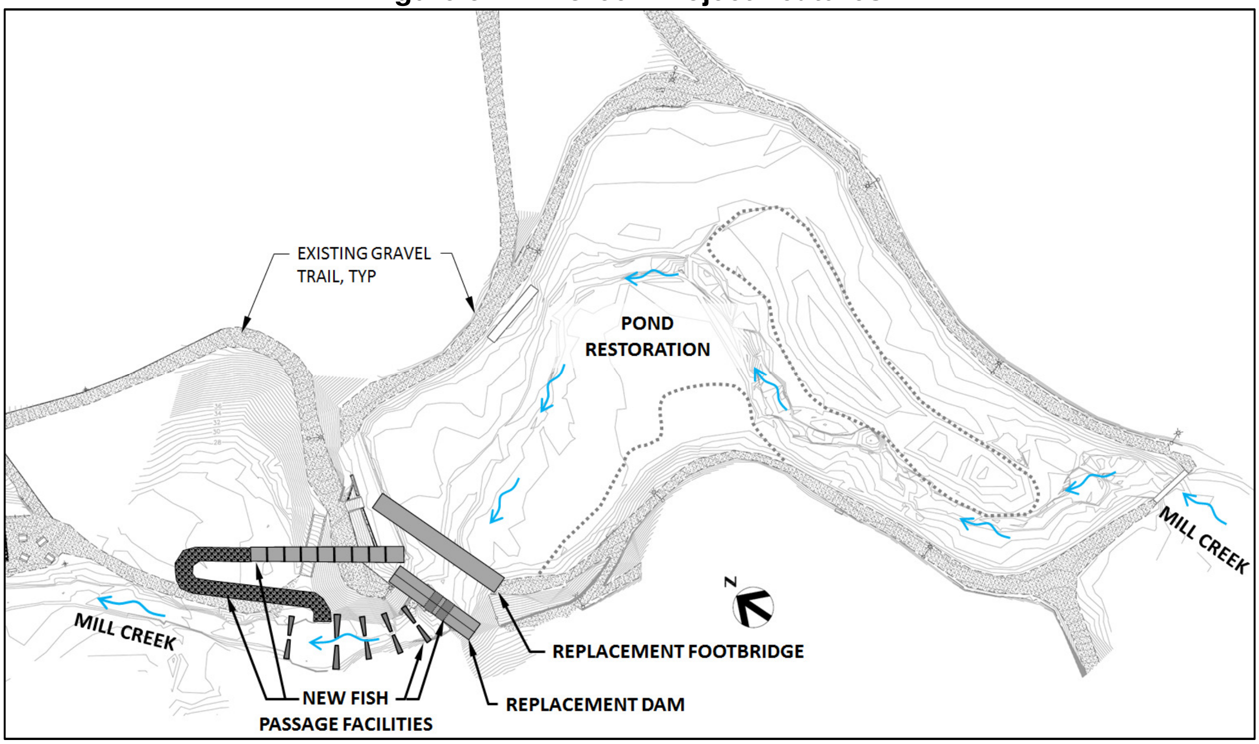
the park, not diminish through conversion to other uses, and replace outdoor recreational uses with similar types of facilities.

The conceptual replacement dam, pond restoration (grading and vegetation), and new fish passage facilities are being developed to be operational through the range of full impoundment pool and empty impoundment pool conditions, allowing for all beneficial project objectives of recreation, fish passage, and flood hazard reduction with habitat restoration to be incorporated.

4.0 Overview of Project Features

The proposed project's four primary features are shown on Figure 3 and include the replacement dam, pond restoration, new fish passage, and replacement footbridge.

Figure 3. Mill Creek Project Features



The project's conceptual drawings are included in Appendix A. The conceptual drawings include the overall site plan, and drawings for the dam improvements, fish passage facility, and pond grading and planting. Profiles, sections, and details have also been prepared for the dam, fish passage, and pond. Related plan sheets are included for the footbridge, demolition of existing structures, temporary erosion and sediment control, and stream bypass during construction.

Preparation of this Engineer's Report included development of technical memoranda for key project technical disciplines, including geotechnical and structural for the replacement dam, hydrology and hydraulics for the project, criteria and alternatives for the new fish passage facility, and alternatives for the replacement footbridge.

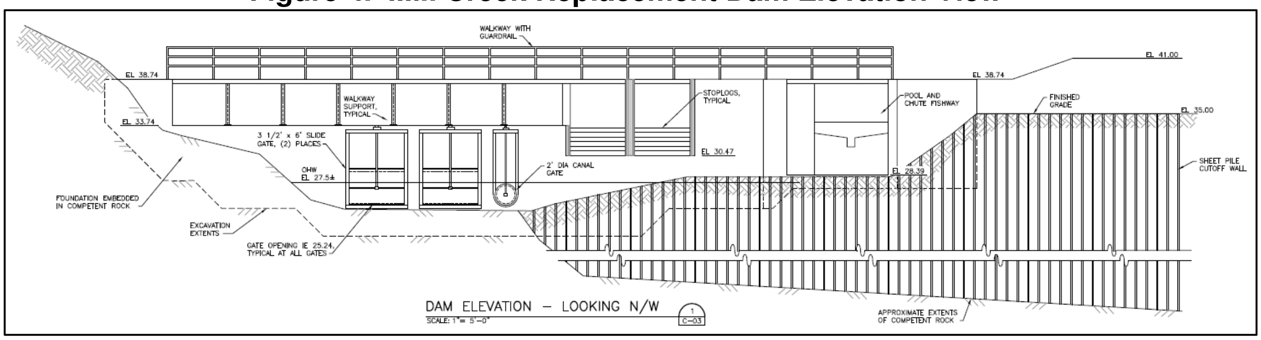
The conceptual plans and technical memoranda, along with environmental, biological, aquatics habitat, cultural resources, and wetland and stream assessments served as the basis for preparation and submittal of the Washington State Joint Aquatics Resources Permit Application (JARPA) Form to the United States Army Corps of Engineers (USACE) for an Individual Permit (IP) for the project.

These technical and environmental components are summarized in the sections below, with references to the technical memoranda and environmental documentation in the appendices.

5.0 Replacement Dam

The technical memorandum documenting the preliminary geotechnical and structural investigations for the replacement dam is presented in Appendix B. The purpose of these investigations was to document the soil and rock properties at the site and prepare the conceptual replacement dam layout to be in accordance with Washington Department of Ecology's Dam Safety Office (DSO) structural, seismic, and hazard classification requirements. Alternatives were identified including dam removal, concrete gravity dam with right abutment improvements (proposed project), earthen embankment dam, and popup dam (inflatable dam and bottom-hinged wicket gate dam). The upstream elevation view of the conceptual concrete gravity replacement dam is shown on Figure 4.

Figure 4. Mill Creek Replacement Dam Elevation View



The replacement dam is an anchored concrete dam founded on rock with a rock abutment on the left side (looking downstream) of the dam. The right side of the concrete dam would tie into the new fish passage structure to be constructed in the existing right abutment. The dam would have two upstream gated 3.5-foot-high by 6-foot-wide outlet structures at the base of the structure to pass instream winter flows and provide winter fish passage. The dam also would have a 2-foot gated pipe to allow controlled passage of summer flows. An ungated spillway would be provided across the left and center of the dam with two, 6-foot-wide spillway sections with provisions for stoplogs to control summer pool elevations. The fish passage structure would be constructed immediately to the right of the dam for full pool fish passage. A sheet pile cutoff wall that penetrates to the top of firm rock is proposed for the right abutment of the dam, extending beyond the fish passage facility as far as needed to control seepage through the abutment.

The following refinements and activities are envisioned as part of the final design phase of the replacement dam project:

- Additional geotechnical subsurface explorations and evaluation to determine top of rock and soil properties.
- Seepage, static and seismic analyses.
- Structural design of the replacement dam and fish passage facility.

6.0 Hydrology & Hydraulics

The technical memorandum documenting the hydrology and hydraulics (H&H) evaluation for the project is presented in Appendix C. The H&H task is intended to develop feasibility level information for the replacement dam following methods outlined by the Washington Department of Ecology's Dam Safety Office (DSO) in its Dam Safety Guidelines, to support the conceptual design of critical project elements including:

- Inflow Design Flood: The Mill Creek Dam inflow design flood was preliminarily estimated using runoff computed using a HEC-HMS model of the drainage area above the dam. Based upon the Hazard Classification, the design event was assumed to be the Probable Maximum Flood (PMF), following the occurrence of Probable Maximum Precipitation (PMP). The results are approximately 50 percent larger than the FEMA results at the 100-year flow level, indicating that parameters used in the HEC-HMS model of the watershed and the PMF results are likely to be conservative.
- Flood Reservoir Routing and Spillway Evaluation: The runoff from the inflow analysis was directly routed through the reservoir behind the replacement dam. Peak water surface elevations for the long duration PMF and the 100-Year flood were calculated using the HEC-HMS hydrologic model. The spillway routing results show that the maximum water surface elevation during the 100-year flood is 2.0 feet above the dam crest, and the maximum water surface associated with the PMF is 3.6 feet above the dam crest.
- Dam Break Inundation Analysis: DSO requires working through a decision framework process to determine the necessary design step, which ranges from Step 1 to Step 8 with increasingly more stringent requirements to satisfy at the higher steps. This project has been evaluated under an assumed design step of 8. Design Step 8 utilizes extreme design events and design loads to provide the extremely high levels of reliability needed to properly protect the public. The dam break inundation analysis includes the estimation of the dam break outflow hydrograph, routing the dam break hydrograph through the downstream creek channel, and estimation of the inundation levels and damages to downstream structures. A HEC-HMS model was developed to estimate the dam break hydrograph, and the downstream boundary condition was set at the FEMA 100-year Base Flood Elevation (BFE).
- Downstream Hazard Classification: Based on DSO guidelines, it is estimated that the Mill Creek Replacement Dam would have a Downstream Hazard Classification of High - 1B, based on the population located in the watershed downstream of the dam.

These analyses focused on compliance with DSO requirements for the replacement dam, and indicate conservative results. Historic stream flow data for Mill Creek are not available for comparison. While the 100-year and PMF will likely overwhelm the capacity of the pond, analyses will be expanded during final design to further evaluate and define the flood hazard reduction elements and operation of the dam and pond during various runoff events. The City plans to reestablish an operation and management approach for this multi-use facility that includes flood hazard reduction such that the pond can provide some mitigation of events that would provide benefits to the citizens and property downstream of the dam.

Because historic stream flow data for Mill Creek are not available, the City installed a stream gage in August 2014, located in Mill Creek within Mill Creek Park near the small footbridge that crosses the creek toward the upstream end of the impoundment area. Stream depth data are being collected for conversion to stream flow in order to provide baseline historic data, in correlation with the City's ongoing recording of rainfall data, to advance the hydrology and hydraulics analyses during final design.

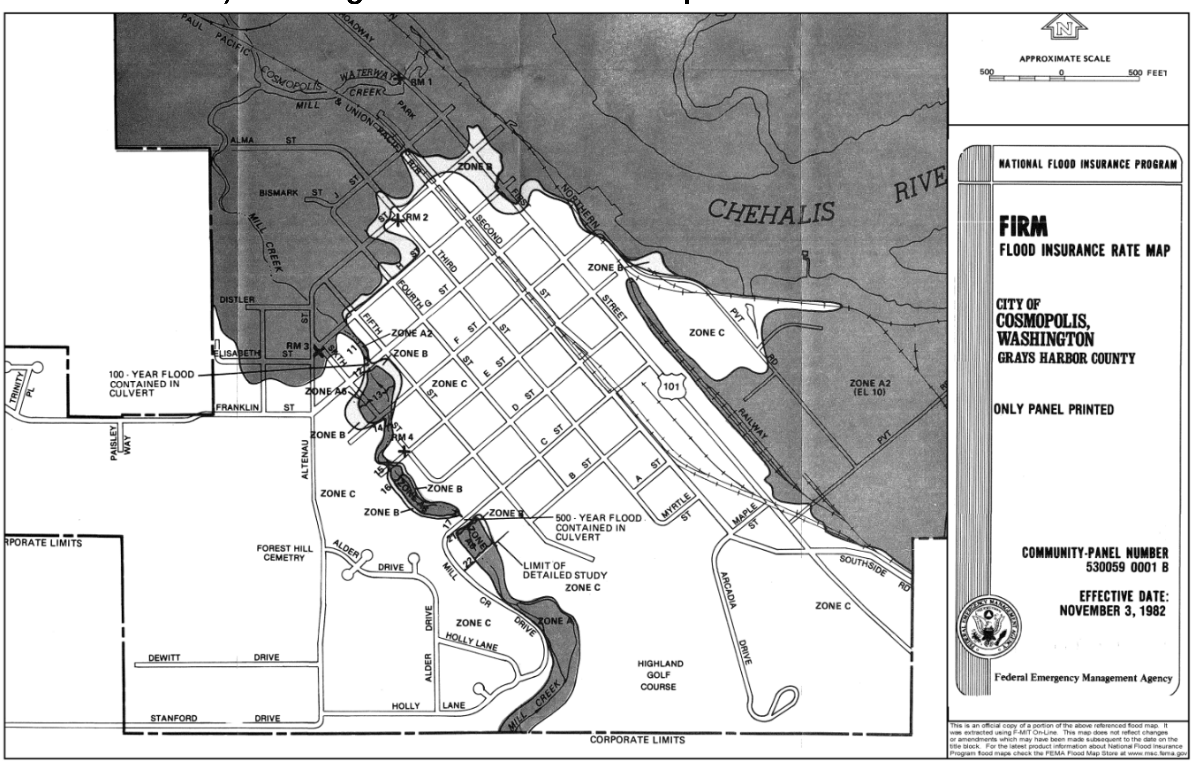
The following refinements and activities are envisioned as part of the final design phase of the replacement dam project:

- Develop the model to improve accuracy of downstream inundation using additional channel survey geometry.
- Incorporate Mill Creek stream gage data.
- Provide an analysis of flood routing when the pond is managed for flood hazard reduction
- Review draft FEMA information.

6.1 Background with FEMA

In September 1998, FEMA issued a Letter of Map Revision (LOMR) in response to the September 1997 request by the City of Aberdeen to revise the effective FIRM, and the affected portions of the Flood Insurance Study Report, to show the effects of construction of a flood control levee along the Chehalis River in the area of Mill Creek. The levee was constructed to provide protection from the flood having a 1-percent chance of being equaled or exceeded in any given year (base flood). Interior drainage analyses were completed by the US Army Corps of Engineers, Seattle District, to compute the ponding elevations landward of the levee along a number of creeks (including Mill Creek). The result of the modification increased the base flood elevation for the Chehalis River and decreased the base flood elevation and Special Flood Hazard Area of Mill Creek from elevation 10 feet (NGVD) to elevation 8 feet (NGVD) downstream of West Huntley Street (Figure 5) (FEMA, 1998).

Figure 5. City of Cosmopolis Flood Insurance Rate Map (Effective Date November 3 1982) Showing Mill Creek within the Special Flood Hazard Area



Any proposed modifications to Mill Creek as part of the project will need to be made in accordance with the FEMA regulations (National Flood Insurance Act of 1968, as amended, 42 U.S. Code 4001-4128, and 44 CFR Part 65) to ensure that base flood elevations do not increase.

7.0 Pond Restoration

Following the dam breach, reed canarygrass has become predominant in the Mill Creek Pond area; red alder saplings and small-fruited bulrush are also present in the pond area. The proposed project includes re-grading the pond area and planting with native wetland vegetation in order to improve the wetland and aquatic habitat and meet USACE requirements.

Pond re-grading would develop new contours. Vegetation within the pond grading footprint would be stripped and removed, and soil would be moved and removed as necessary to accomplish the grading plan. The existing channel location would be preserved in the newly graded pond area. Some excavated material would be redistributed to create habitat features such as the elevated margins for wetland plants and an island. The remaining excavated material including cleared vegetation would be hauled off site and disposed of in an approved location. The pond grading plan is shown on Figure 6 and the pond planting plan is shown on Figure 7.

Figure 6. Pond Grading Plan

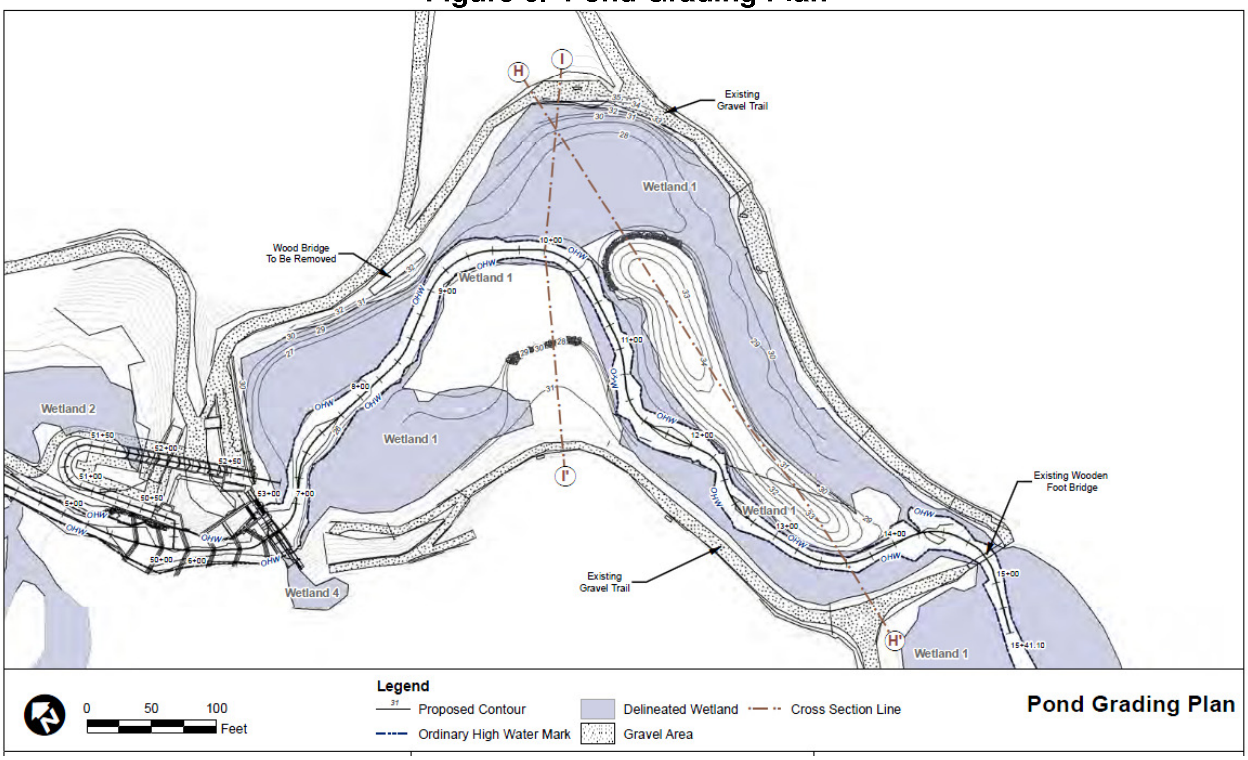
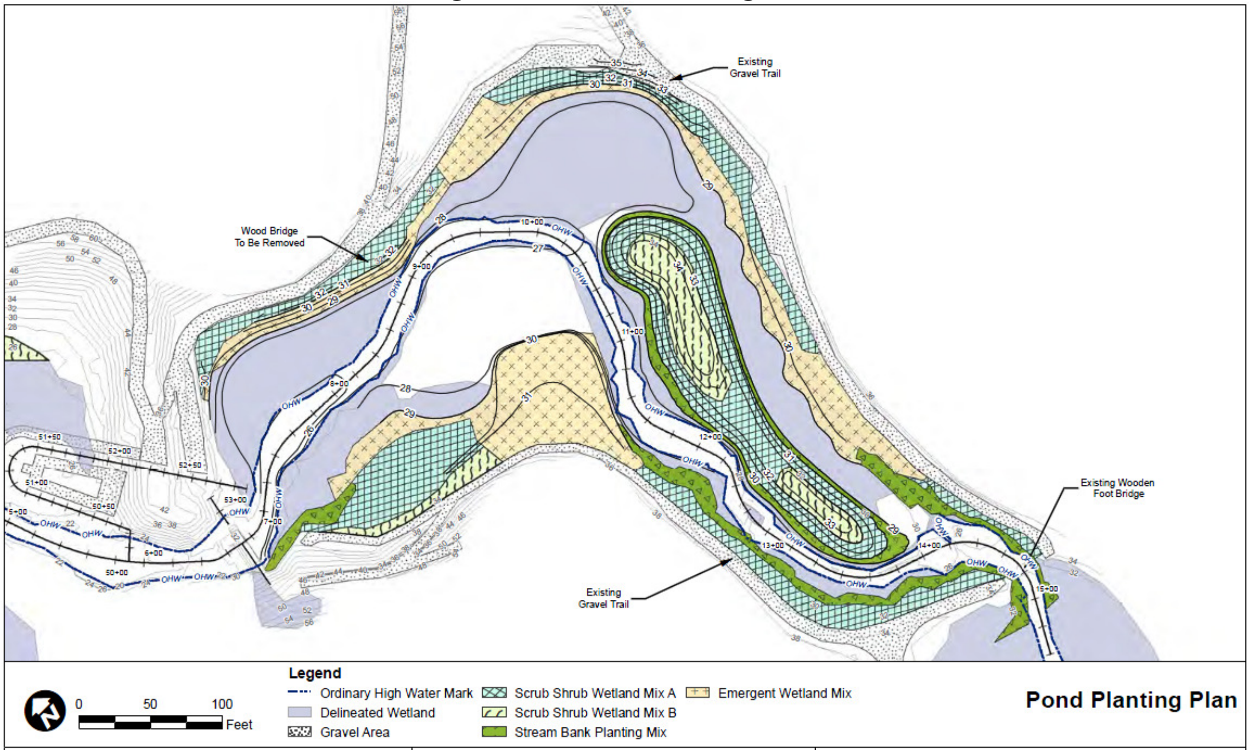


Figure 7. Pond Planting Plan

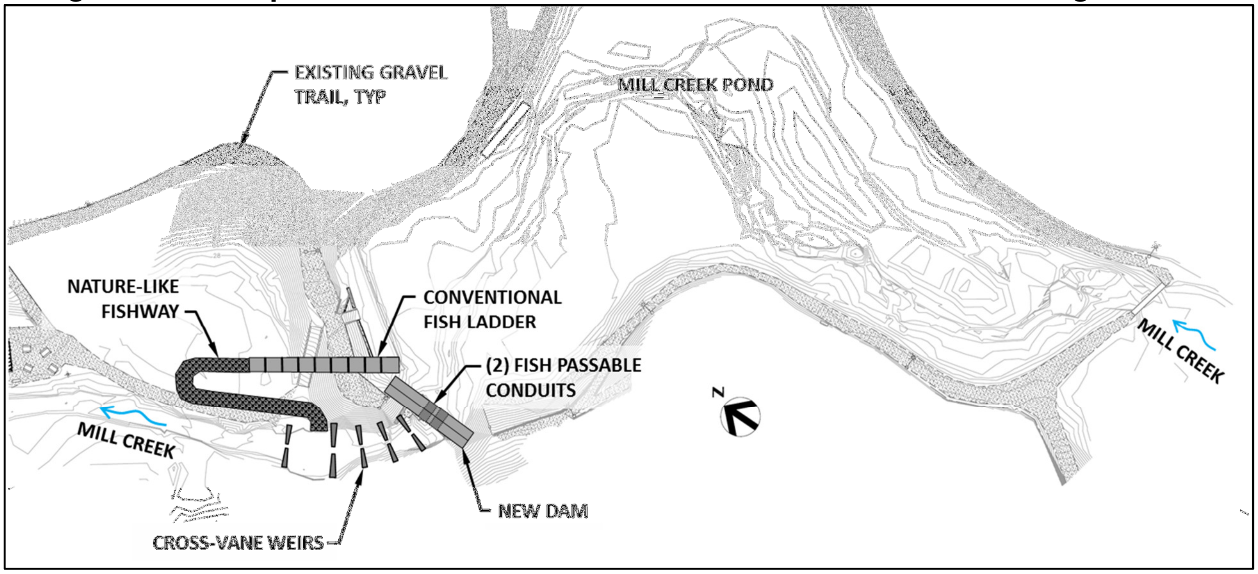


8.0 Fish Passage

The technical memorandum documenting the fish passage criteria and alternatives is presented in Appendix D. To be in accordance WDFW and other fisheries agency requirements, the tech memo documents the fisheries resources in Mill Creek, the selected species for fish passage facility design, fish passage design flows for targeted species, and the fish bypass and fishway design criteria (for the entrance, conduit, ladder, exit, debris rack, and temporary/interim passage facilities).

Conceptual level alternatives were evaluated including conventional fish ladder, nature-like fishway, trap and haul, fish lifts, reservoir bypass channel, and fish passage through dam (tunnel or conduit). The selected alternative is a combination fish ladder and fish passage tunnel through the dam due to the ability to provide fish passage under a wide range of operating conditions including full pool and no pool conditions. During full pool operations the combination fish ladder, which consists of a conventional fish ladder and nature-like fishway, provides fish passage. Fish passage would be provided via the conduit through the dam during periods when there is no reservoir pool. The concept schematic of the combination fish ladder and fish passage tunnel is shown on Figure 8.

Figure 8. Concept Schematic of Combination Fish Ladder and Fish Passage Tunnel



The following items are identified as next steps required in the project implementation process:

- Analyze Mill Creek hydraulics to refine design of dam fish passable conduits.
- Analyze Mill Creek hydraulics to establish downstream boundary condition of fishway.
- Analyze Mill Creek dam operations to establish upstream boundary condition of fishway during pond operations.
- Perform design reviews with the fisheries agencies as part of the environmental permitting process.

9.0 Replacement Footbridge

The technical memorandum documenting the footbridge alternatives is presented in Appendix E. This memorandum documents the initial evaluation for replacement of the failed footbridge, and identifies alternative locations, width, and structural material types, in order to meet RCO requirements to replace outdoor recreational uses with similar type facilities, and to meet AASHTO LRFD bridge design specifications and AASHTO guide specifications for pedestrian bridges.

Three alternative locations were evaluated, with the preferred location for the replacement footbridge being just upstream of the new dam spanning Mill Creek pond from the termini of the lower loop trail around the pond on the north and south sides.

The preferred footbridge width is approximately 11 feet, including 9 feet horizontal clear distance plus 2 feet for structural members and railings. The 9 feet of horizontal clear distance consists of a minimum of 6 feet for two-directional pedestrian travel plus 3 feet for standing along either side of the footbridge without disrupting travel. The horizontal clear distance would also provide sufficient clearance for an all-terrain vehicle (ATV) or similar sized vehicle to traverse the bridge for City park maintenance or for emergency response purposes.

Four structural material type alternatives are considered suitable for the replacement footbridge location, span length, and width, including:

- Timber beams with timber deck slab and railings
- Steel truss with cast-in-place concrete deck slab

- Steel beams with cast-in-place concrete deck slab
- Precast concrete beams with integral concrete deck slab

Characteristics of each alternative structure type are provided with regard to aesthetics, service life, maintenance, and construction costs. The preferred structure material type will be selected by the City during the final design phase of the project.

10.0 Environmental & Permitting

The environmental and permitting documentation prepared for the project is presented in Appendix F, and includes:

- Washington State Joint Aquatic Resources Permit Application (JARPA) Form: The JARPA is used to apply for federal, state, and local permits and approvals.
- Biological Evaluation and EFH Assessment: This assessment report documents that the project may affect but is not likely to adversely affect Endangered Species Act (ESA) listed species. The project would affect Essential Fish Habitat (EFH); however, the effects are temporary during construction and the addition of fish passage past the new dam will benefit EFH by providing increased access to habitat upstream.
- Fish and Aquatics Habitat Report: This report describes the existing fish use and aquatic habitat in Mill Creek upstream of the dam, and is intended to provide documentation of existing stream channel and fish habitat conditions in the project area and an assessment of changes from the fish passage component and the additional upstream area made available for fish use.
- Wetland and Stream Delineation Report: This report provides documentation of existing wetland and stream conditions in the project area to support federal, state, and local permitting for the project. Four wetlands totaling 4.11 acres in area were delineated within the project area and distinguished from adjoining uplands by the presence of indicators for wetland hydrology, hydric soils, and hydrophytic vegetation. Mill Creek is the only stream identified in the project area.
- Cultural Resources Assessment: This assessment consisted of background review, field investigation, and reporting. Background review determined the Area of Potential Effect (APE) to be in an area of moderate probability for historic properties. Field investigation included a pedestrian and subsurface survey. The only cultural resource recorded in the APE was the Mill Creek Dam, which was inventoried, but does not appear to have any qualities that would make it eligible for listing on the National Register of Historic Places and, as such, is not considered a historic property. Therefore, the project as proposed does not appear to have the potential to affect historic properties and no further cultural resources oversight is warranted.

The construction processes required to build the project would take place within wetlands, below the Ordinary High Water Mark (OHWM) of Mill Creek, and in the 100-year floodplain of Mill Creek. It is anticipated that construction of the proposed project would occur over a period of seven months from April through October. The USACE-established in-water work window for the Chehalis River and its tributaries from the mouth upstream to Porter Creek is June 1 through October 31. Per consultations with WDFW, the in-water work window of June 1 through September 30 would be used for this project. The environmental permits applicable to the project are listed in Table 1.

Table 1. Project Applicable Environmental Permits

Permit / Approval	Regulated Activity
FEDERAL	
US Army Corps of Engineers (USACE) local district Individual Permit (submitted April 4, 2015; Received by USACE on April 13, 2015)	• Wetland Fill • Stream fill • Dam repair/fish passage not authorized under any NWP
USFWS/NMFS ESA Compliance	• Federal Action (Corps Authorization)
STATE	
Washington Department of Ecology (Ecology) Section 401 Water Quality Certification (Individual) Coastal Zone Management Act	Triggered by Corps Individual Permit
Washington Department of Ecology (Ecology) NPDES	• Over an acre of earthwork triggers NPDES
DAHP National Historic Preservation Act Compliance/ Consultation	Federal Action (Corps Authorization)
Washington Department of Fish and Wildlife (WDFW) Hydraulic Project Approval (HPA)	In-water work in Mill Creek
Washington Department of Natural Resources (WDNR) Aquatic Lands Easement	NOT APPLICABLE – Reach of Mill Creek is not State Owned Aquatic Land
Washington State Recreation and Conservation Office Project Approval	Entire project – Mill Creek Park has been funded by Land and Water Conservation Fund and state bond funds, thus purpose of park cannot change
LOCAL	
City of Cosmopolis SEPA Checklist City permits	• Over 100CY fill/excavation triggers SEPA • SEPA review required for HPA

The following environmental and permitting activities will also be developed during the final design phase for the project:

Alternatives Analysis Report – As required for the USACE permit, a Section 404 3(b) Alternatives Evaluation is required. An increased level of effort is required for coordination and design activities to support preparation of permit applications triggering an Individual Permit. This includes more analysis of the 'practicable alternatives' to demonstrate compliance with the USACE Section 404(b)(1) Guidelines for Specification of Disposal Sites for Dredged or Fill Material (CFR 40 Part 230 Section 404(b)(1)). Subpart (a) of this Guideline stipulates the following:

"…with minor exception, no discharge of dredged or fill material shall be permitted if there is a practicable alternative to the proposed discharge which would have less adverse impact on the aquatic ecosystem, so long as the alternative does not have other significant adverse environmental consequences."

For the purpose of this requirement, practicable alternatives include, but are not limited to:

- Activities which do not involve a discharge of dredged or fill material into the waters of the United States or ocean waters;
- Discharges of dredged or fill material at other locations in water of the United States or ocean waters.

The practicable alternatives will be developed to demonstrate compliance with Section 404(b)(1). It is assumed that the alternatives analysis prepared for NEPA will provide the information necessary for evaluation under these Guidelines. The preliminary list of alternatives to be vetted for the practicable alternatives analysis to meet the project's purposes and objectives include:

- No build (do nothing or leave the site as is)
- Replace the dam at existing location and include fish passage (proposed project)
- Build new dam at another location within the Mill Creek watershed
- Build larger dam with increased storage
- Remove existing dam and restore creek for fish passage
- Install pumps to discharge Mill Creek to Chehalis River
- Build levees or floodwalls on Mill Creek
- Raise houses in 100-year floodplain of Mill Creek
- Flood proof existing structures in floodplain
- Low impact development retrofits in urban areas of Mill Creek
- Non structural elements (floodplain regulations)

11.0Next Steps

This Engineer's Report documents the conceptual development of the Mill Creek Dam Improvements Projects and sets the baseline for advancing the project to the final design phase of the project, followed by construction. The final design phase will include further development of the geotechnical, structural, civil, mechanical, hydrology & hydraulics, and environmental disciplines to acquire permits and prepare bid documents for construction.

Appendix A

Conceptual Project Drawings

STATE MAP

VICINITY MAP

LOCATION MAP

Permitting Drawings For:

City of Cosmopolis

Mill Creek Park Dam Improvements

HDR Project No.
171201

City of Cosmopolis, WA
MARCH 12, 2015

INDEX OF DRAWINGS

PERMITTING DRAWINGS

PROPOSED SITE PLAN SYMBOLOGY

SITE PLAN

Symbol	Description
x	PROPOSED TEMPORARY FENCE
x x x	BARBED WIRE OR CHAINLINK FENCE
100 YEAR	100 YEAR FLOOD BOUNDARY
	PERMANENT EASEMENT
	TEMPORARY OR CONSTRUCTION EASEMENT
	PROPERTY LINES
	RIGHT OF WAY LINES
	RIGHT OF WAY CENTERLINE
B-101	SOIL BORING
	CUT SLOPE
	FILL SLOPE
	CONTRUCTION STAGING AREA

EROSION CONTROL

Symbol	Description
	SEDIMENT FENCE
HVF	CONSTRUCTION HIGH VISIBILITY FENCE
	DIRECTION OF SURFACE RUNOFF
	CONTRUCTION ENTRANCE

EXISTING SITE PLAN SYMBOLOGY

SYMBOL	DESCRIPTION
	MONUMENT
	MONUMENT IN CASE
	REBAR & CAP
	SCRIBED "X"
	PK NAIL
	METAL SIGN POST
	WOOD SIGN POST
	SANITARY SEWER MANHOLE
	STORM DRAIN MANHOLE
	CATCH BASIN
WM	WATER METER
W	WATER VAULT
	GAS VALVE
TR	TRAFFIC JUNCTION BOX
E	ELECTRICAL VAULT
EL	ELECTRICAL JUNCTION BOX
PM	POWER METER
TC	TELECOMMUNICATION JUNCTION BOX
CT	CABLE TV JUNCTION BOX
	JUNCTION BOX
P	POWER VAULT
TEL	TELECOMMUNICATION RISER
P	UTILITY POLE
UG	UTILITY POLE W/CONDUITS TO UNDERGROUND
	GUY ANCHOR
	GATE POST
MB	MAIL BOX
	CONIFEROUS TREE
	DECIDUOUS TREE
	TEST HOLE (WITH FIELD IDENTIFIER)
	YARD LIGHT OR LUMINAIRE W/NO MAST
	GRATE INLET
CPEP	CORRUGATED POLYETHYLENE PIPE
RCP	REINFORCED CONCRETE PIPE
CIP	CAST IRON PIPE
CMP	CORRUGATED METAL PIPE
DIP	DUCTILE IRON PIPE
W	WATER MANHOLE
	WATER VALVE
OR	FIRE HYDRANT
	TREE CANOPY OR SHRUB LINE
	ASPHALT IN PLAN
	GRAVEL IN PLAN

LINETYPE	DESCRIPTION
	GENERIC GAS LINE
IRR	IRRIGATION
	STORM DRAIN
	SANITARY SEWER
FM	FORCEMAIN
WL	WETLAND BOUNDARY
UGE	UNDERGROUND POWER
UGF	UNDERGROUND FIBER OPTIC
UGT	UNDERGROUND TELEPHONE
UGTV	UNDERGROUND TELEVISION
OHE	OVERHEAD POWER
OHF	OVERHEAD FIBER OPTIC
OHT	OVERHEAD TELEPHONE
OHTV	OVERHEAD TELEVISION
20	MAJOR CONTOUR
21	MINOR CONTOUR
	DITCH OR STREAM CENTERLINE
OHW	ORDINARY HIGH WATER
WL	WETLAND BOUNDARY

GENERAL SYMBOLOGY

ARROW INDICATES DIRECTION OF PLAN NORTH

N

SECTION
0 10 20 40
SCALE IN FEET

SECTION LETTER
FLAG INDICATES DIRECTION OF SECTION CUT
X
XXX
* DRAWING WHERE SECTION VIEW IS LOCATED

SECTION CUT MARKER

SECTION LETTER
SECTION
SCALE: 1/4" = 1'-0"
0
00x-00
* DRAWING WHERE SECTION VIEW IS FIRST CUT

DETAIL NUMBER
X
XXX
* DRAWING WHERE DETAIL IS LOCATED

DETAIL MARKER

DETAIL NUMBER
DETAIL
SCALE: 1/4" = 1'-0"
0
00x-00
* DRAWING WHERE DETAIL WAS TAKEN

KEYNOTE OR SPECIFIC NOTE DESIGNATION

KEYNOTE NUMBER
#

PHOTO MARKER

#
###

MISCELLANEOUS

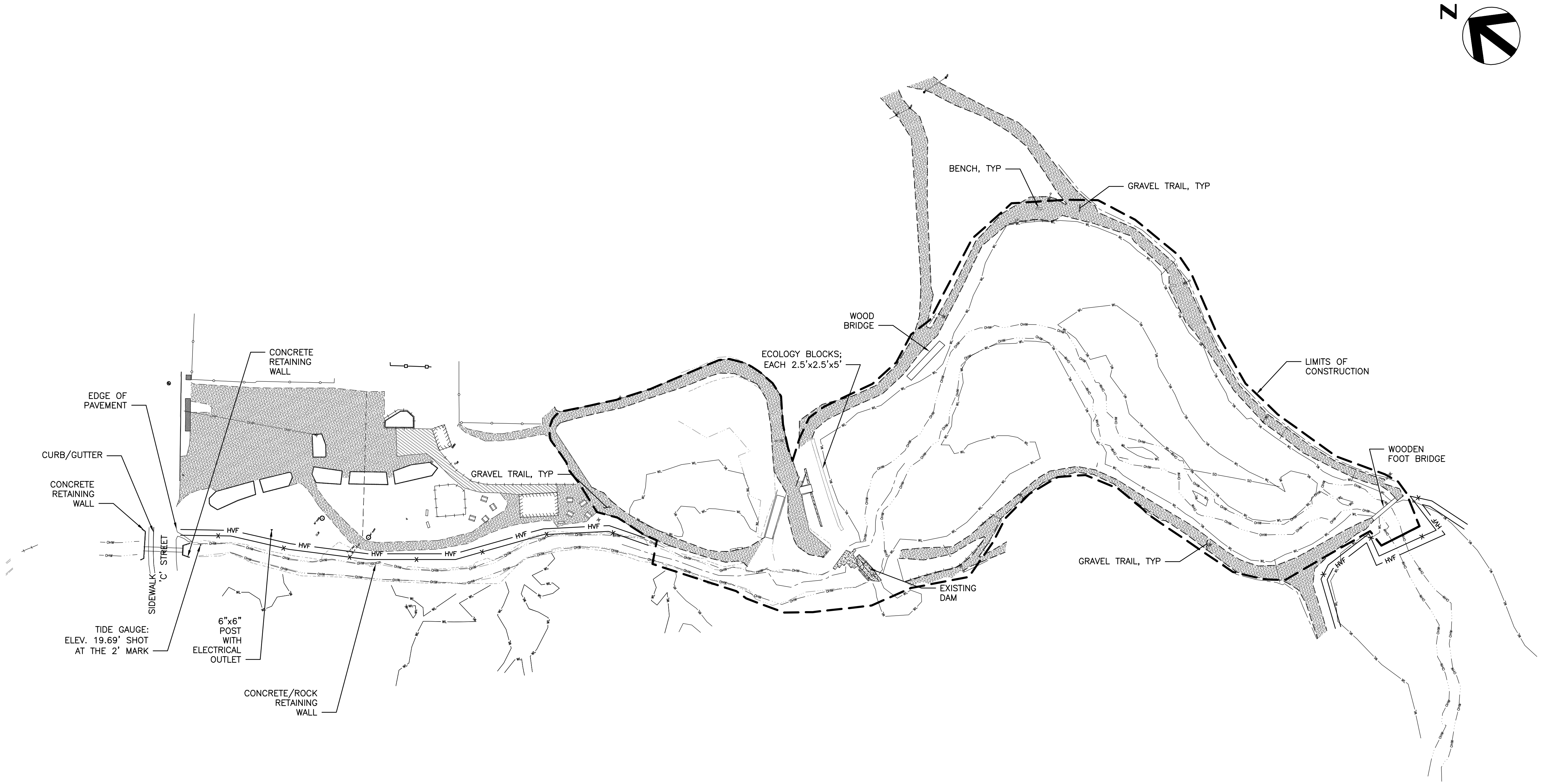
ABBREVIATIONS

ABC	AGGREGATE BASE COURSE
ABN	TO BE ABANDONED
ABND	ABANDONED
AC	ASBESTOS CEMENT
ACP	ASPHALT CONCRETE PAVEMENT
AFF	ABOVE FINISH FLOOR
ALUM	ALUMINUM
ANSI	AMERICAN NATIONAL STANDARD INSTITUTE
BF	BLIND FLANGE
BFV	BUTTERFLY VALVE
BV	BALL VALVE
℄	CENTER LINE
CB	CATCH BASIN
CAV	COMBINATION AIR RELEASE VALVE
CCP	CONCRETE CYLINDER PIPE
CDF	COMPACTED DENSITY FILL
CE	CONCRETE ENCASEMENT
CI	CAST IRON
CO	CLEAN OUT
CONC	CONCRETE
CONSTR.	CONSTRUCTION
CLR	CLEAR
CMP	CORRUGATED METAL PIPE
CMU	CONCRETE MASONRY UNIT
CTB	CONCRETE TREATED BASE
CV	CHECK VALVE
CU	COPPER
DEQ	DEPARTMENT OF ENVIRONMENTAL QUALITY
DF	DOUGLAS FIR
DIP	DUCTILE IRON PIPE
DR	DRAIN
D/W	DRIVEWAY
ED	EQUIPMENT DRAIN
EL	ELEVATION
ELB	ELBOW
EW	EACH WAY
EXIST	EXISTING
FCA	FLANGED COUPLING ADAPTER
FD	FLOOR DRAIN
FDN	FOUNDATION
FE	FLANGED END
FEXT	FIRE EXTINGUISHER
FF	FINISH FLOOR
FH	FIRE HYDRANT
FM	FLOW METER
GALVS	GALVANIZED STEEL
HPG	HIGH PRESSURE GAS
IE; INV	INVERT ELEVATION
LF	LINEAL FEET
MH	MANHOLE
MIN	MINIMUM
MIPT	MALE IRON PIPE THREAD
MJ	MECHANICAL JOINT
MON	MONITORING
NGVD	NATIONAL GEODETIC VERTICAL DATUM
NIC	NOT IN CONTRACT
OF	OVERFLOW PIPE
OHE	OVERHEAD ELECTRIC
PE	POLYETHYLENE/ PLAIN END
PNL	PANEL
POC	POINT OF CONNECTION
PVC	POLYVINYL CHLORIDE
PG	PRESSURE GAGE
PRV	PRESSURE REDUCING VALVE
PS	PIPE SUPPORT
(R)	HORIZONTAL STATION REFERENCE
R&R	REMOVE AND REPLACE PER EXISTING
RED	REDUCING, REDUCER
RR	RAILROAD
SAN	SANITARY
SIM	SIMILAR
SPEC	SPECIFICATION
SSMH	SANITARY SEWER MANHOLE
STA	STATION
STM	STORM
TYP	TYPICAL
UGE	UNDERGROUND ELECTRIC
UGT	UNDERGROUND TELEPHONE
UGTV	UNDERGROUND TV CABLE
UNO	UNLESS NOTED OTHERWISE
VIF	VERIFY LOCATION AND/OR FEATURES IN FIELD PRIOR TO CONDUCTING POTENTIALLY IMPACTED CONSTRUCTION ACTIVITY
N	NORTH
E	EAST
S	SOUTH
W	WATER, WEST

SHEET NOTES

1. THIS IS A STANDARD DRAWING SHOWING COMMON SYMBOLOGY. ALL SYMBOLS ARE NOT NECESSARILY USED ON THIS PROJECT.
2. SCREENING OR SHADING OF WORK IS USED TO INDICATE EXISTING COMPONENTS OR TO DEEMPHASIZE PROPOSED IMPROVEMENTS TO HIGHLIGHT SELECTED TRADE WORK.

PERMITTING DRAWINGS
NOT FOR CONSTRUCTION



LEGEND:

HVF — HIGH VISIBILITY FENCE

x — SILT FENCE

PLAN

0 30 60 120

SCALE IN FEET

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ISSUE	DATE	DESCRIPTION

PROJECT MANAGER: T. HUME
DESIGNED BY:
CHECKED BY:
DRAWN BY:
PROJECT NUMBER: 000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

TESC PLAN

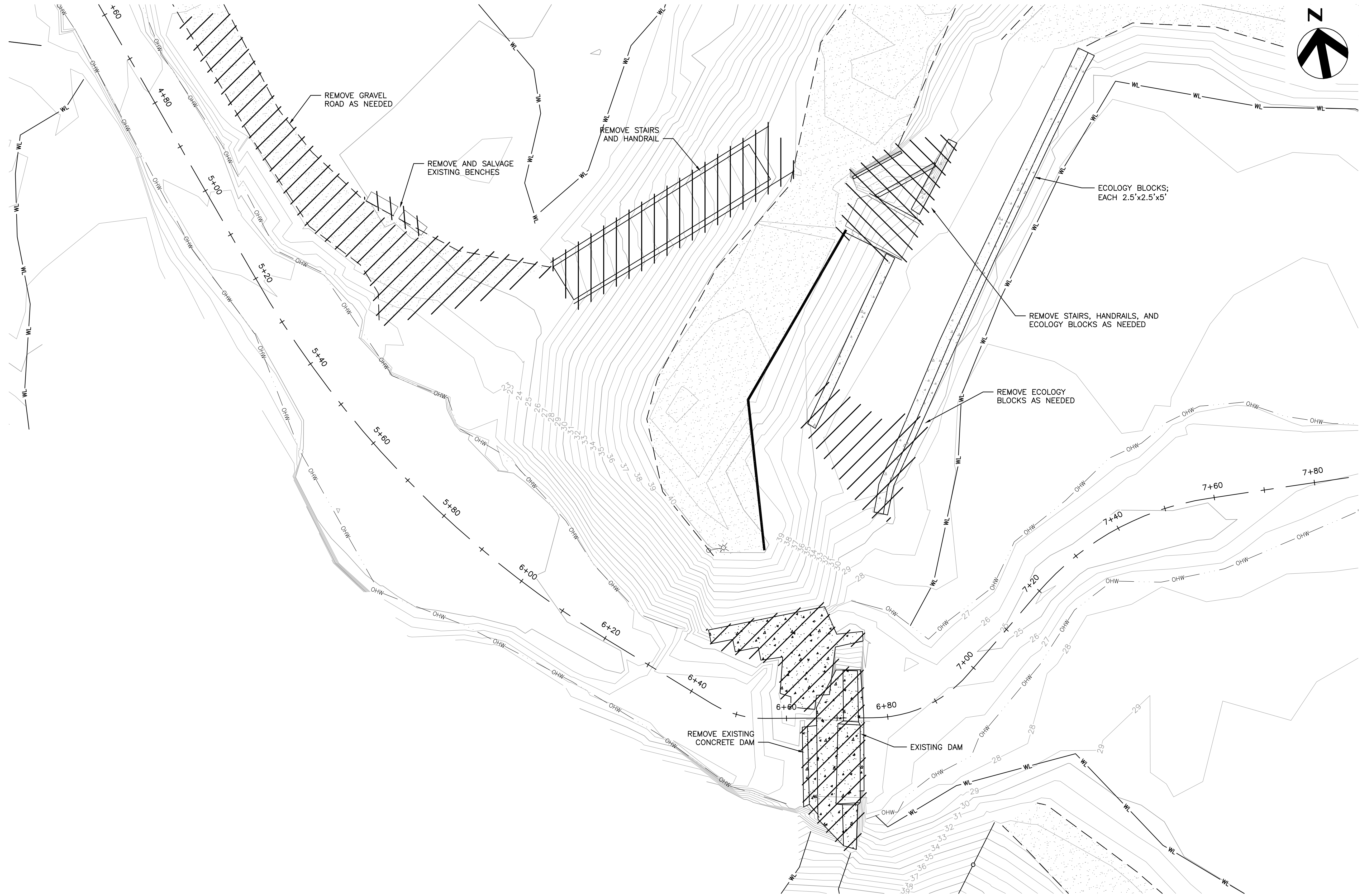
0 1" 2"

FILENAME G-02.dwg

SCALE AS NOTED

SHEET: 3 Of 16

G-02



PLAN
SCALE: 1" = 10'

PERMITTING DRAWINGS
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MILL CREEK PARK DAM IMPROVEMENTS

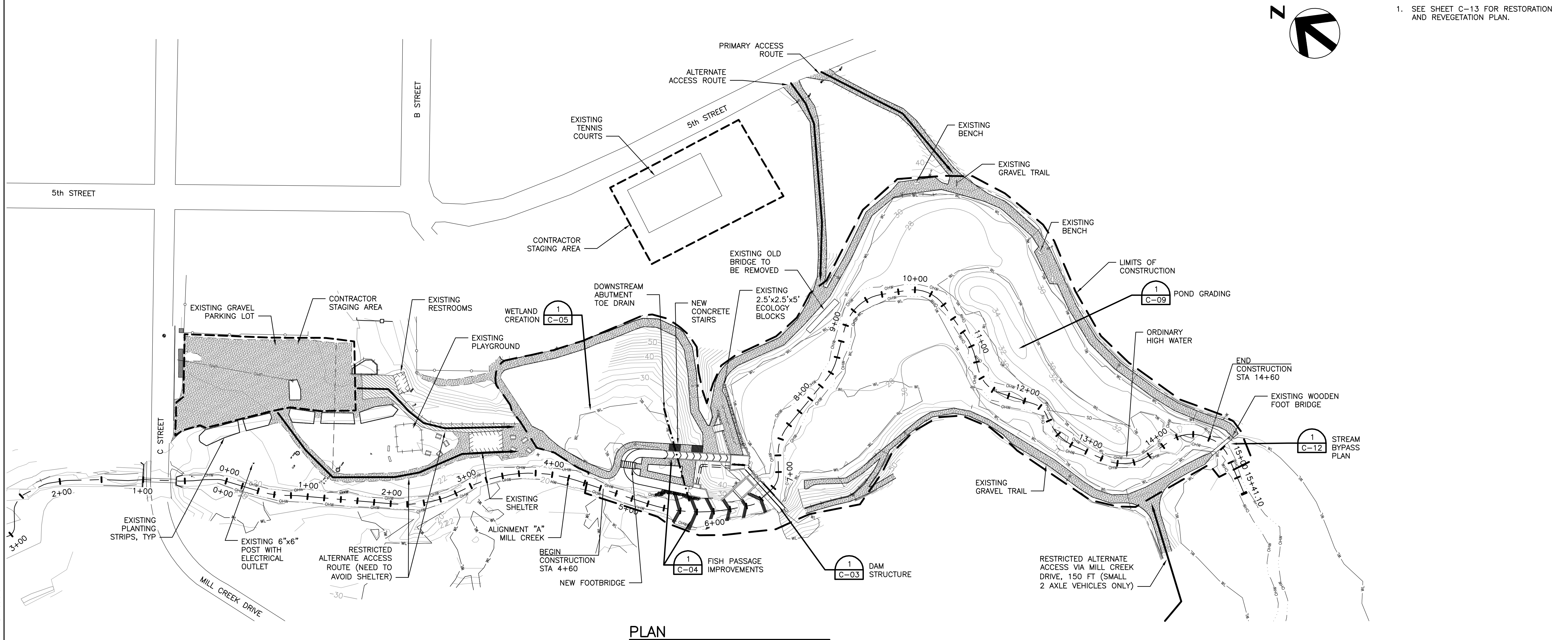
CITY OF COSMOPOLIS

DAM DEMOLITION PLAN

FILENAME	C-01.dwg
SCALE	AS NOTED

SHEET: 4 Of 16

C-01



PLAN

SCALE: 1" = 60'

PERMITTING DRAWINGS
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MILL CREEK PARK DAM IMPROVEMENTS

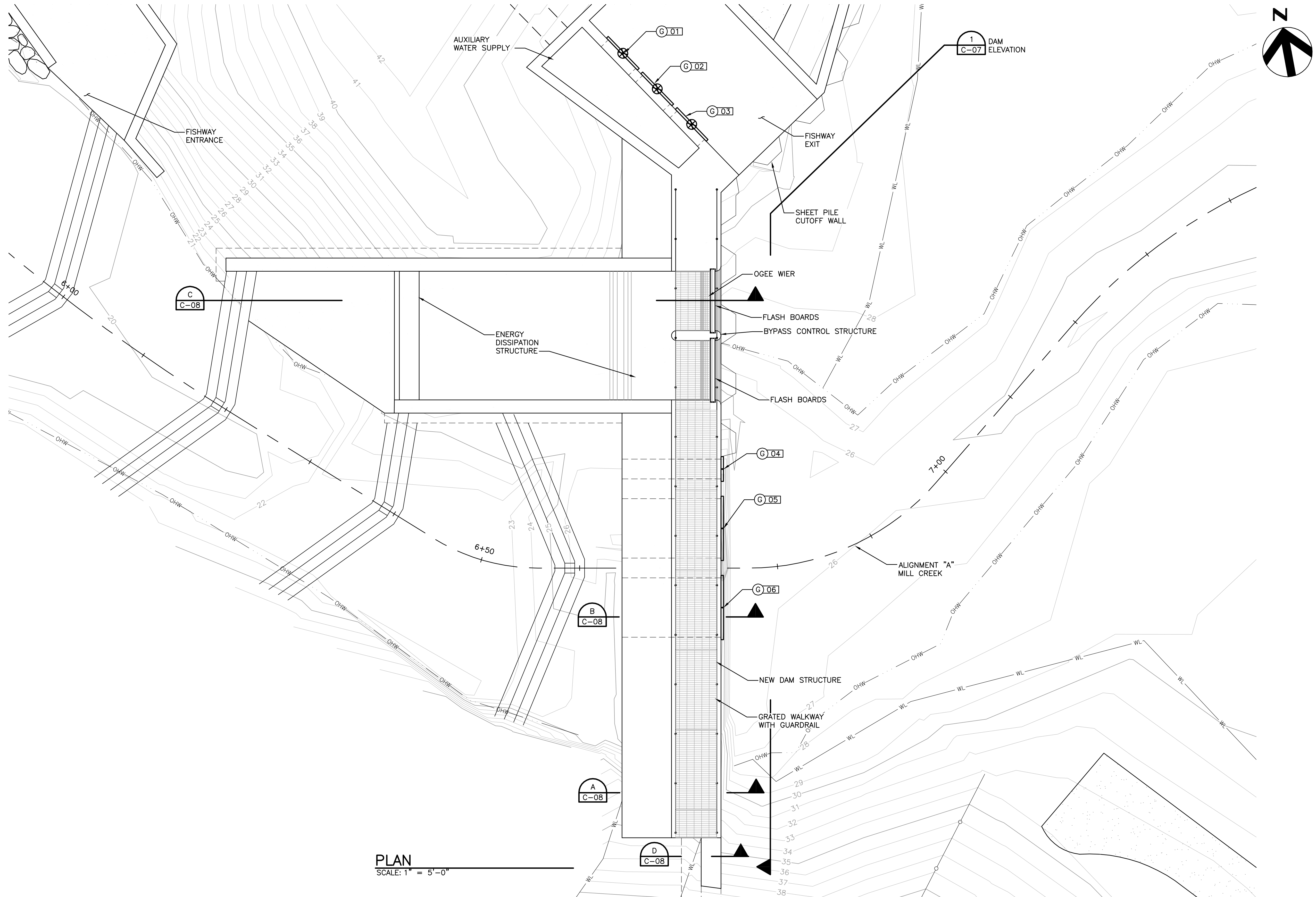
CITY OF COSMOPOLIS

OVERALL SITE PLAN

FILENAME C-02.dwg
SCALE AS NOTED

SHEET: 5 Of 16

C-02



PLAN
SCALE: 1" = 5'-0"

PERMITTING DRAWINGS
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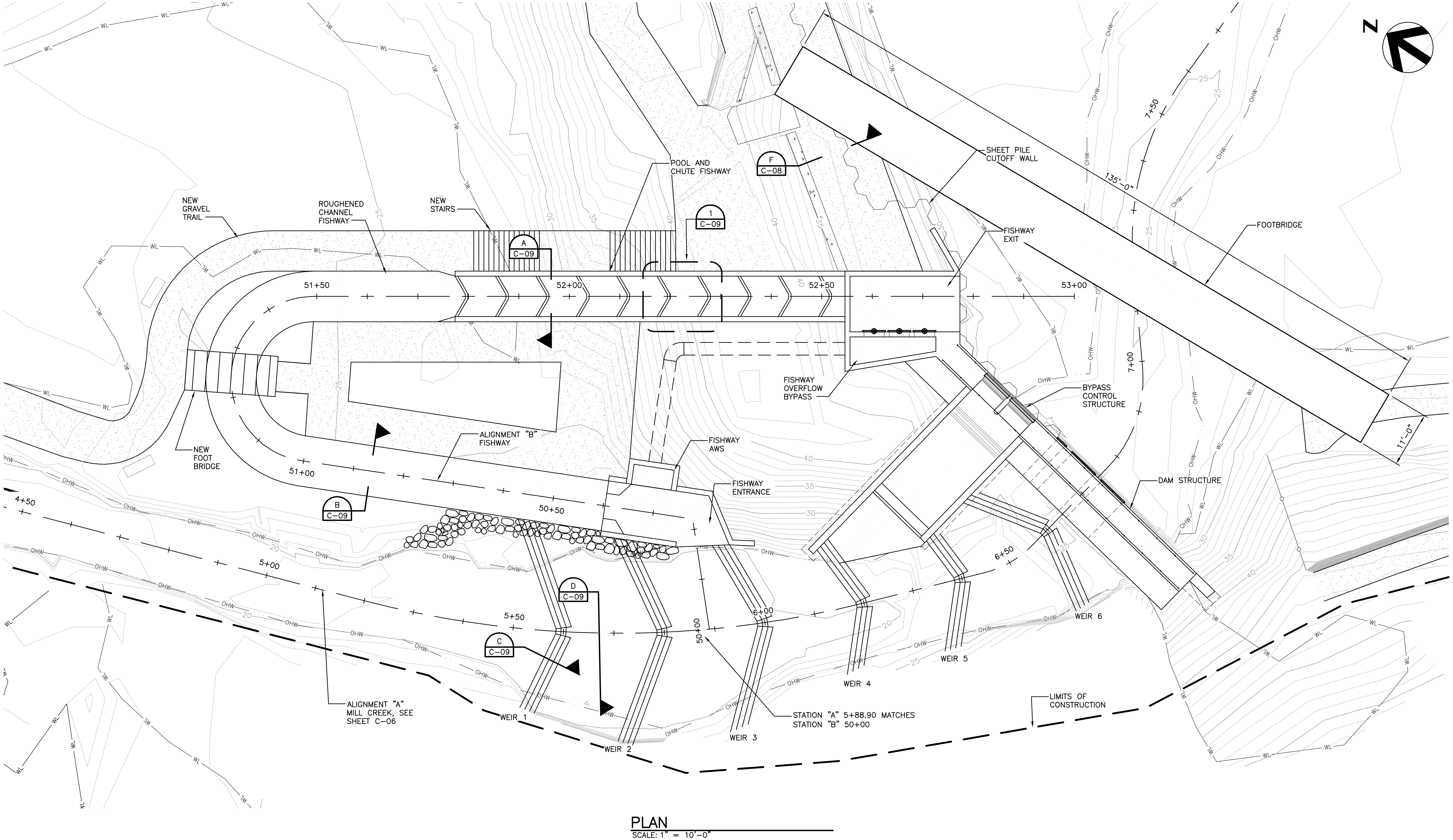
MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

DAM IMPROVEMENTS PLAN

FILENAME C-03.dwg
SCALE AS NOTED

SHEET: 6 Of 16
C-03



PLAN
SCALE: 1" = 10'-0"

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MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

FISH PASSAGE PLAN

FILENAME C-04.dwg
SCALE AS NOTED
SHEET: 7 Of 16
C-04

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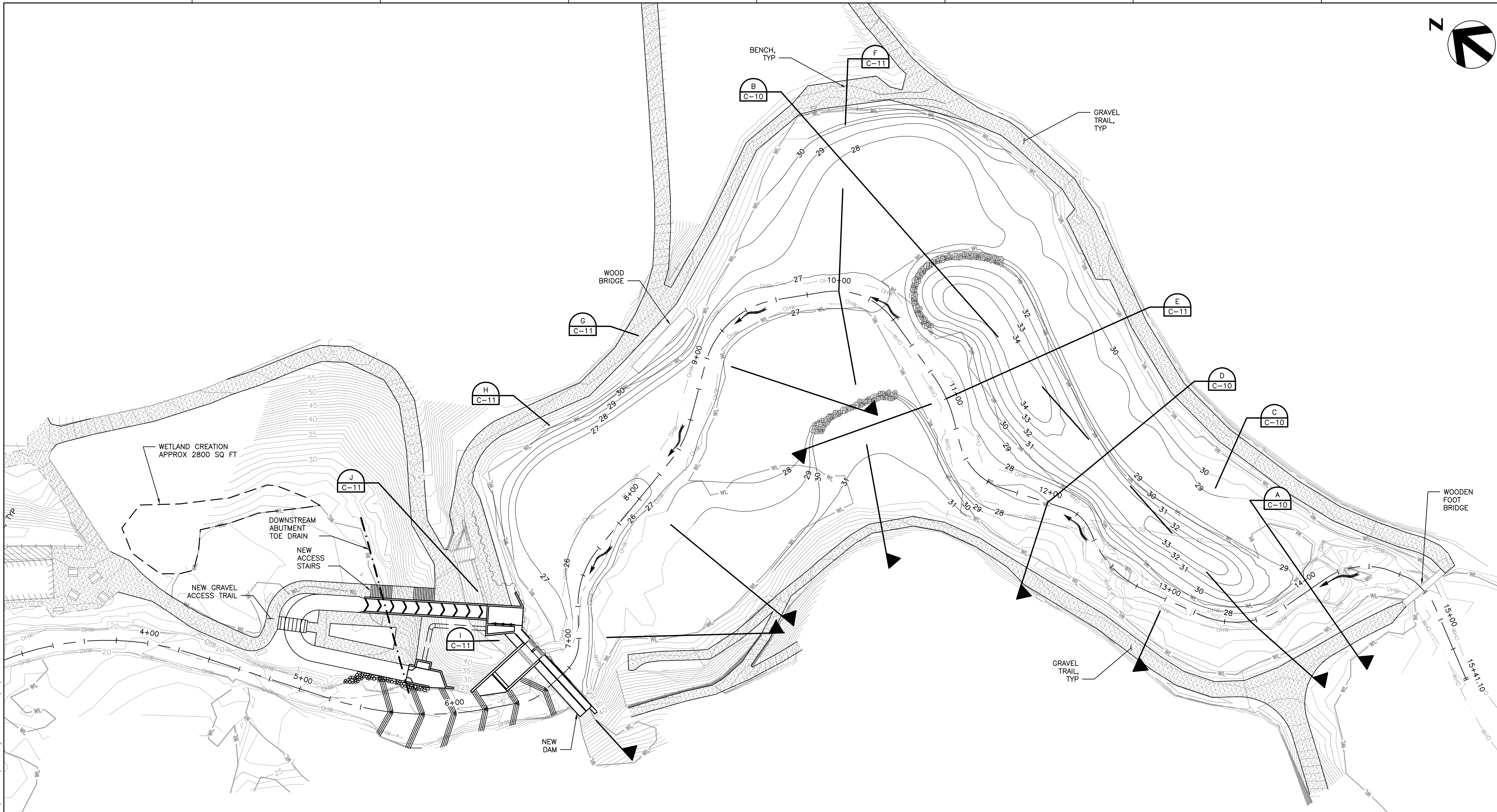

PLAN
SCALE: 1" = 30'-0"

PERMITTING DRAWINGS
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MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

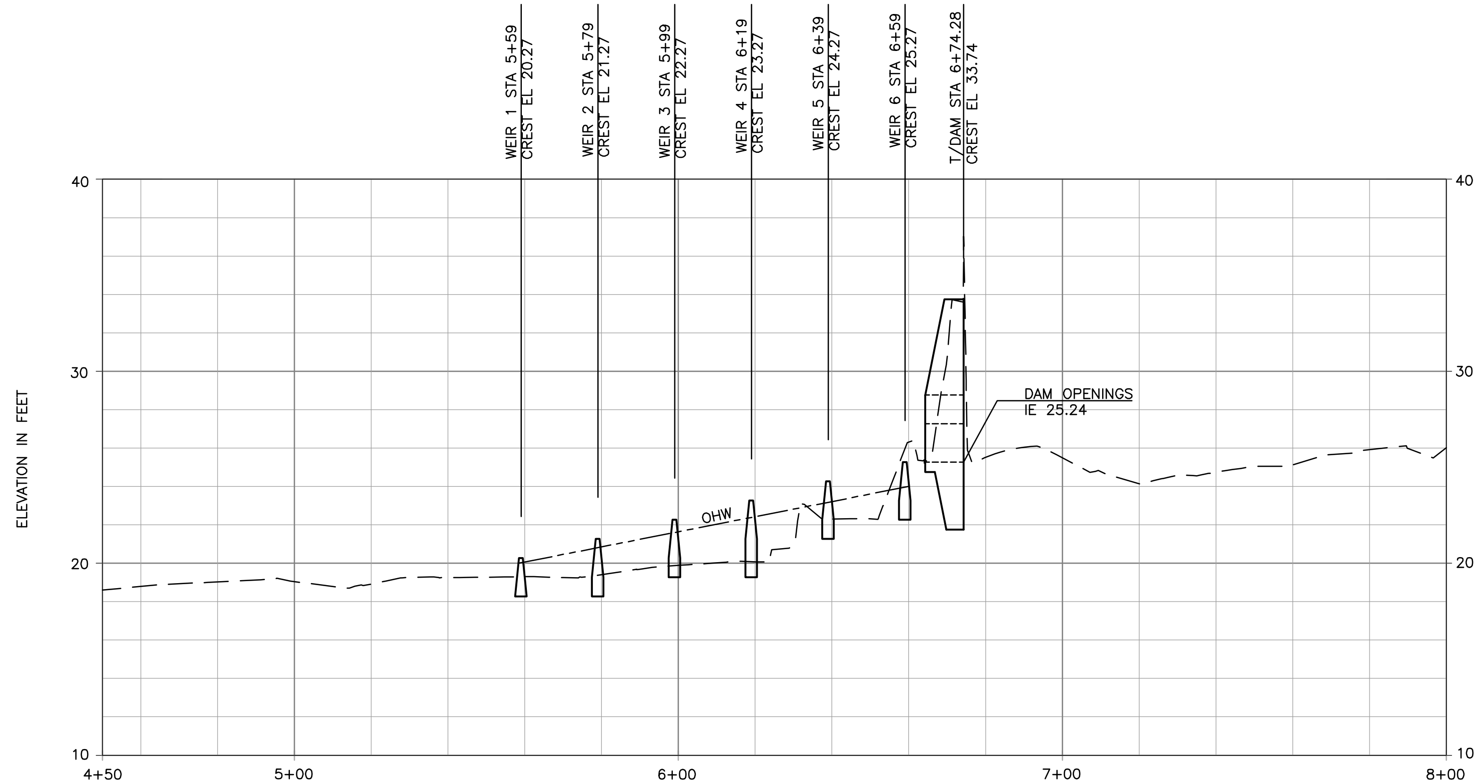
POND GRADING PLAN

0 1" 2"

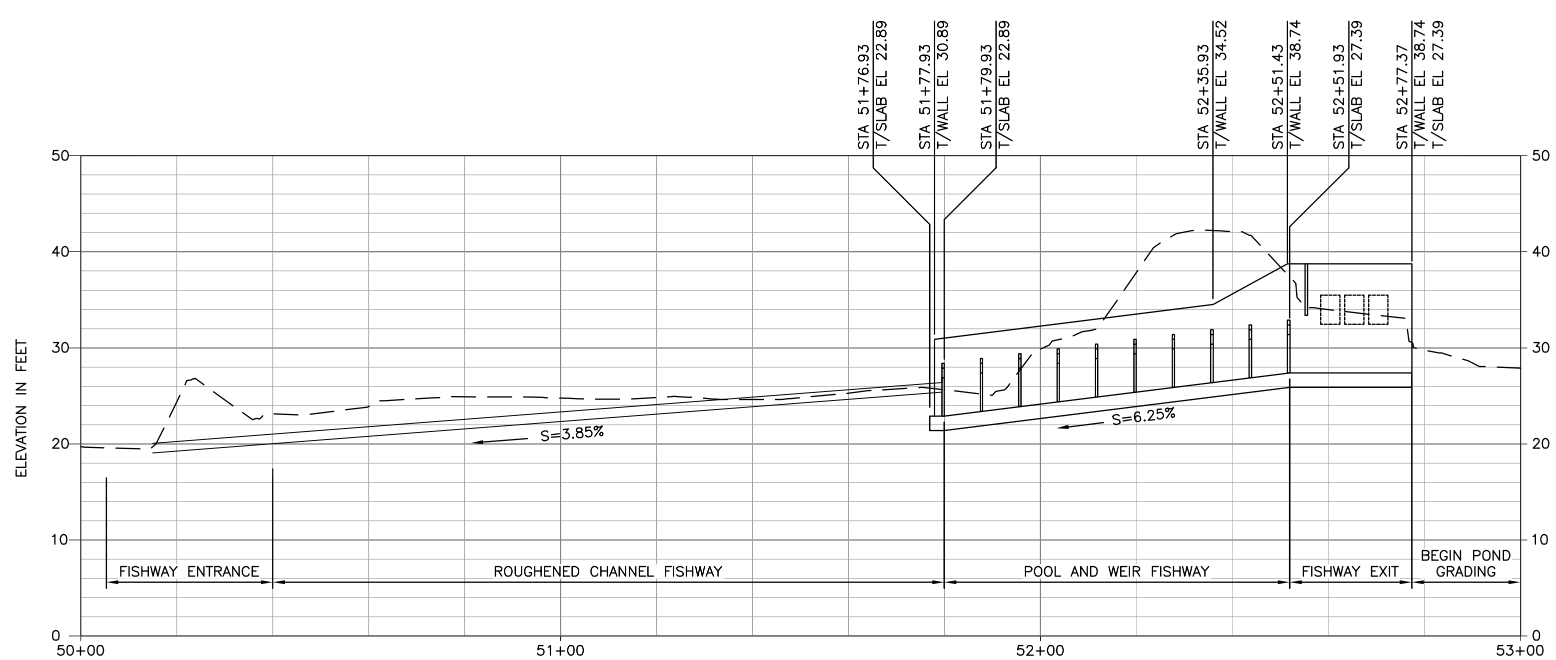
FILENAME	C-05.dwg
SCALE	AS NOTED

SHEET: 12 Of 16

C-05



ALIGNMENT "A" STATIONING
MILL CREEK PROFILE
SCALE: 1" = 25' H 1" = 5' V



ALIGNMENT "B" STATIONING
FISHWAY PROFILE
SCALE: 1" = 20' H 1" = 10' V

PERMITTING DRAWINGS
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HDR

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MILL CREEK PARK DAM IMPROVEMENTS

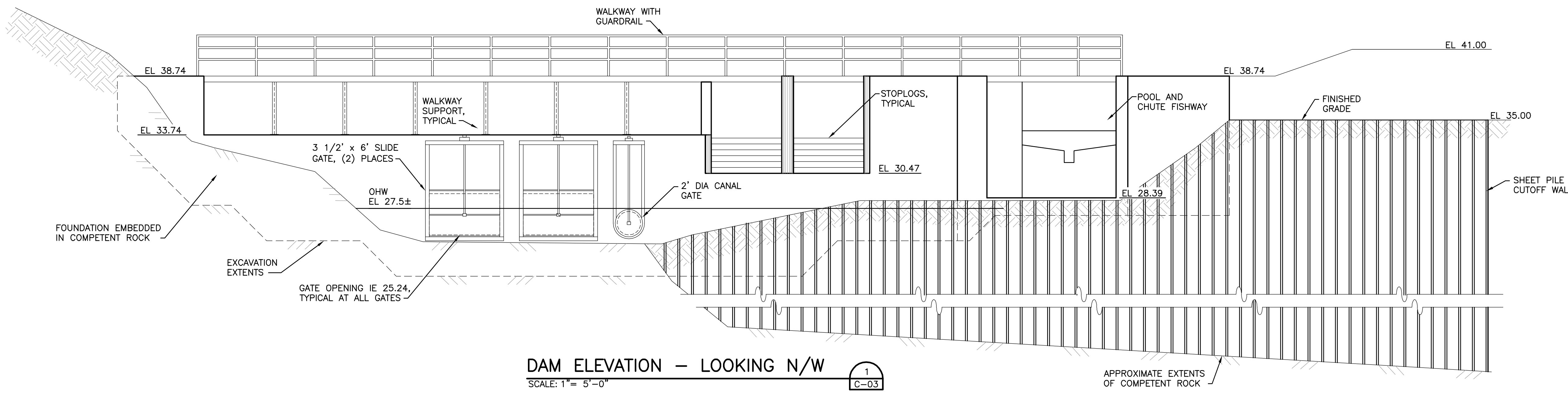
CITY OF COSMOPOLIS

MILL CREEK PROFILE

0 1" 2"

FILENAME C-06.dwg
SCALE AS NOTED

SHEET: 8 Of 16
C-06



DAM ELEVATION – LOOKING N/W
SCALE: 1"= 5'-0"
1 / C-03

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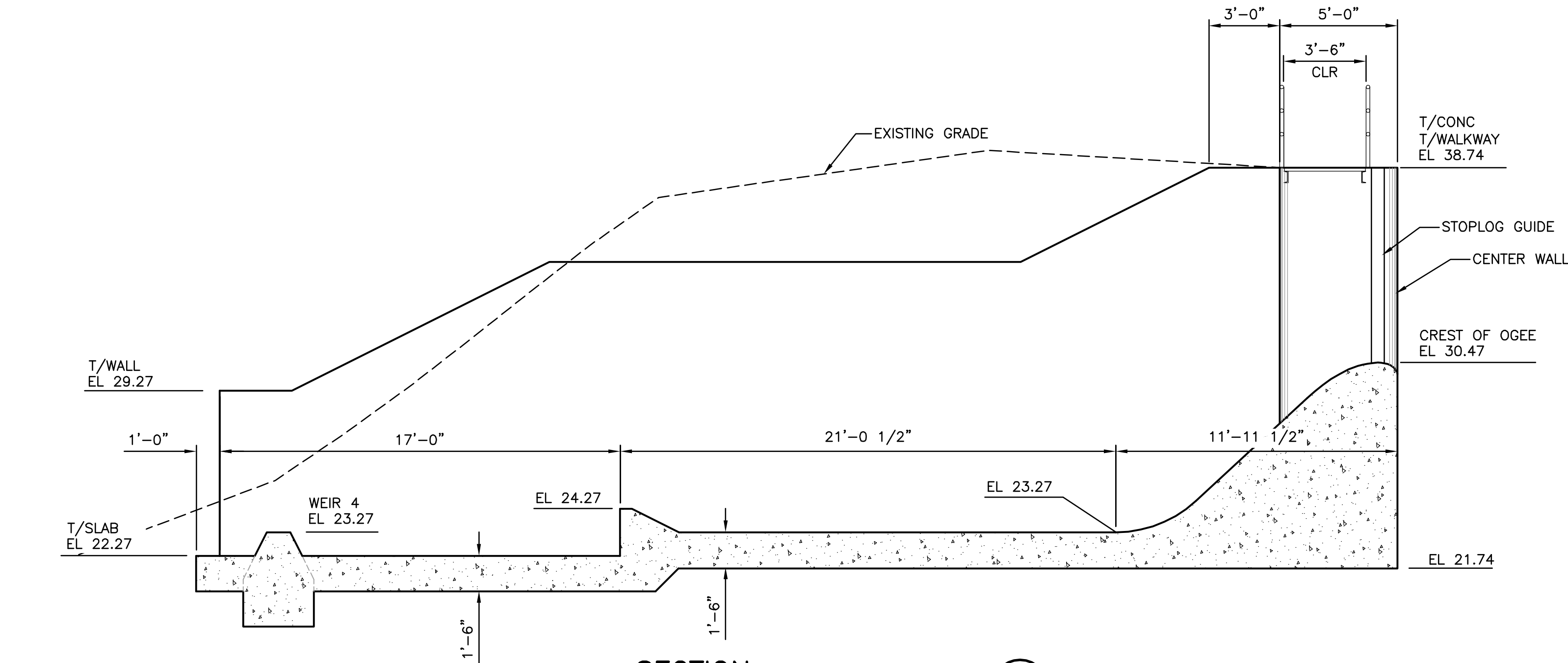
MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

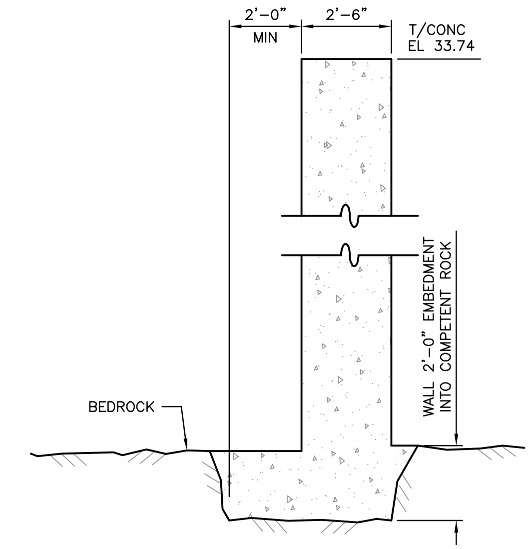
DAM ELEVATIONS AND SECTIONS 1

0 1" 2"

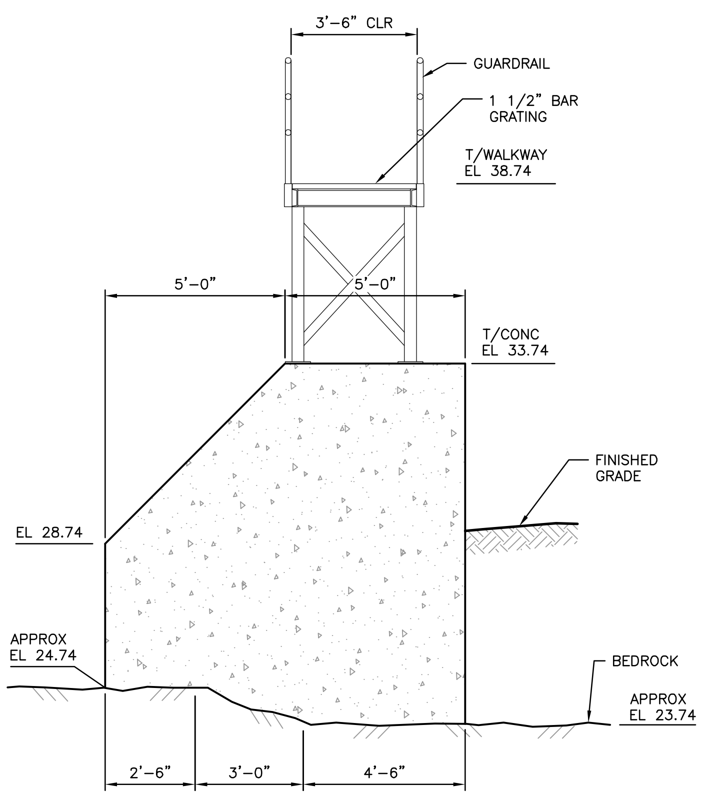
FILENAME C–07.dwg
SCALE AS NOTED



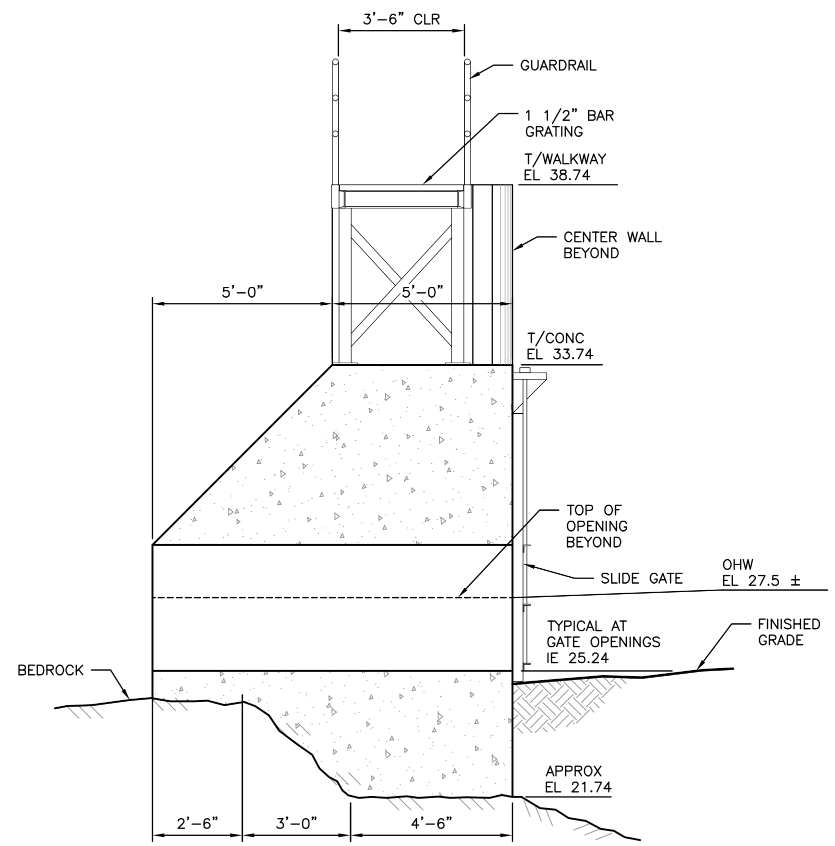
SECTION C
SCALE: 1/4" = 1'-0"
C-03



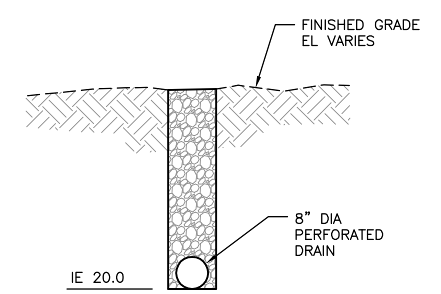
SECTION D
SCALE: 3/8" = 1'-0"
C-03



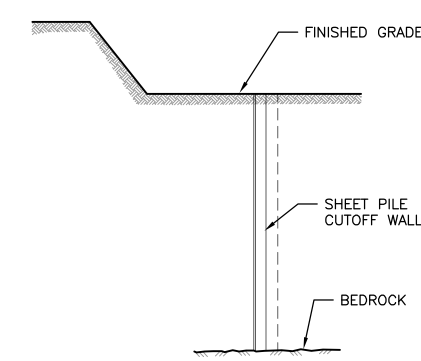
SECTION A
SCALE: 3/8" = 1'-0"
C-03



SECTION B
SCALE: 3/8" = 1'-0"
C-03



SECTION E
SCALE: 3/8" = 1'-0"
C-12



SECTION F
SCALE: 3/8" = 1'-0"
C-04

PERMITTING DRAWINGS
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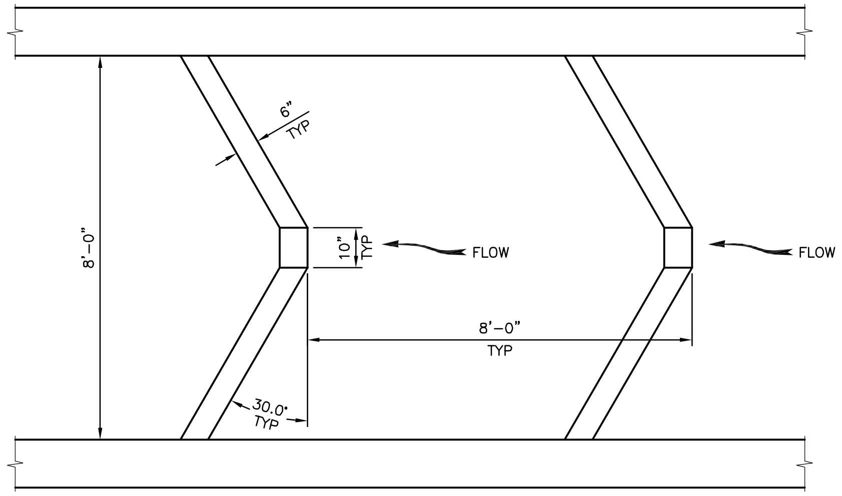
MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

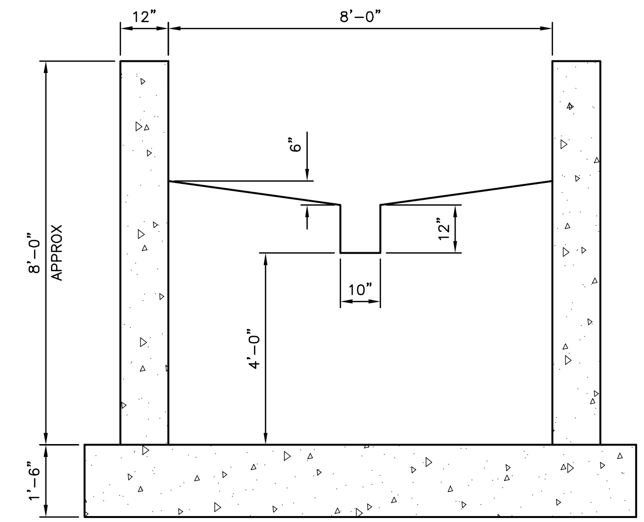
DAM ELEVATIONS AND SECTIONS 2

FILENAME C-08.dwg
SCALE AS NOTED

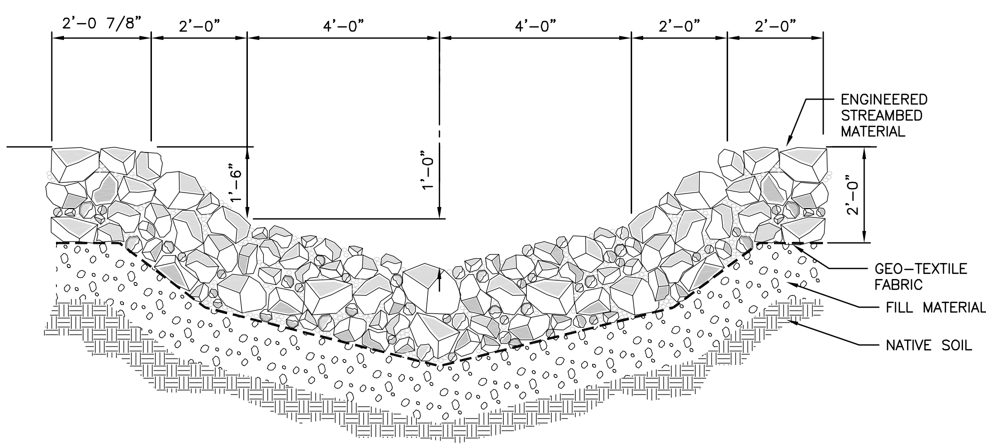
SHEET: 10 Of 16
C-08



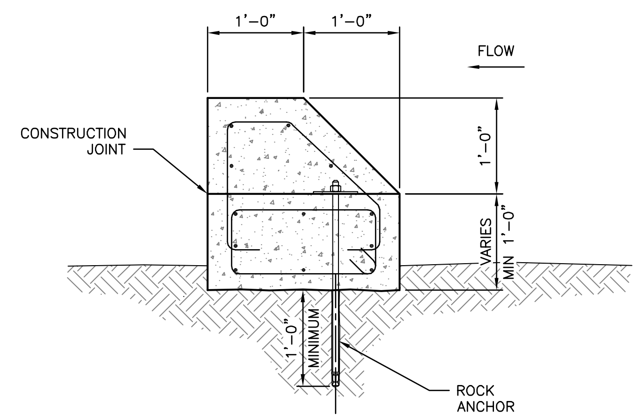
POOL AND CHUTE FISHWAY PARTIAL PLAN (1 / C-04)
SCALE: 1/2" = 1'-0"



POOL AND CHUTE FISHWAY SECTION (A / C-04)
SCALE: 1/2" = 1'-0"



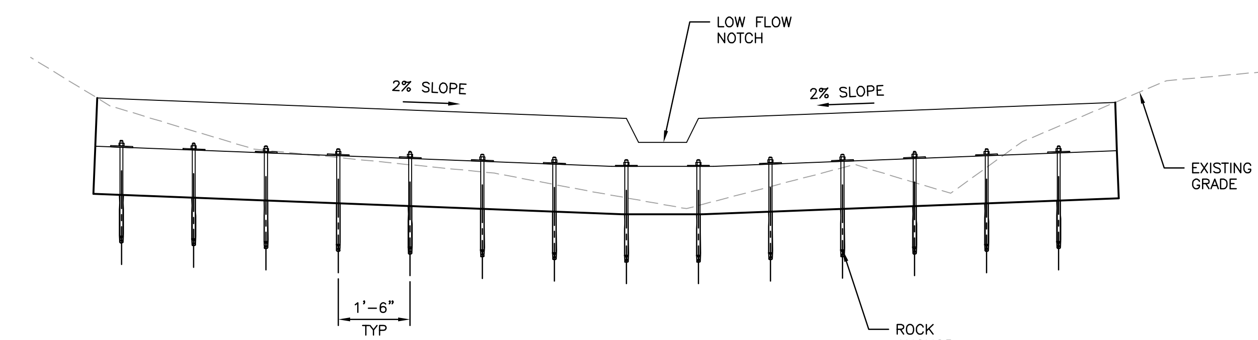
ROUGHENED CHANNEL SECTION (B / C-04)
SCALE: 1/2" = 1'-0"



TYPICAL INSTREAM WEIR SECTION (C / C-04)
SCALE: 1" = 1'-0"

NOTES:

1. WEIR WILL BE MORE NATURALISTIC BOULDER SHAPED.

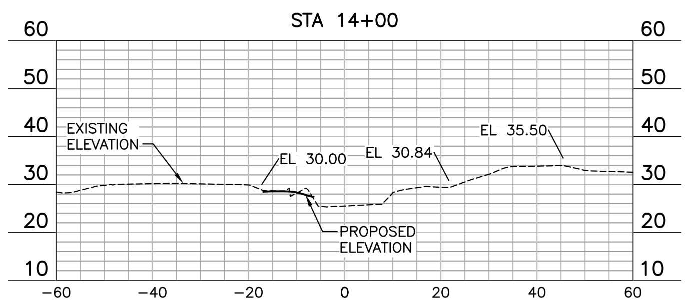


TYPICAL INSTREAM WEIR ELEVATION (D / C-04)
SCALE: 1/2" = 1'-0"

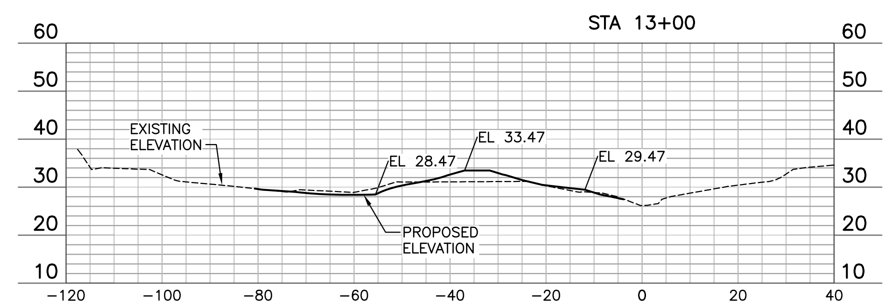
NOTES:

1. OHW ELEVATION VARIES. SEE MILL CREEK PROFILE ON SHEET C-06

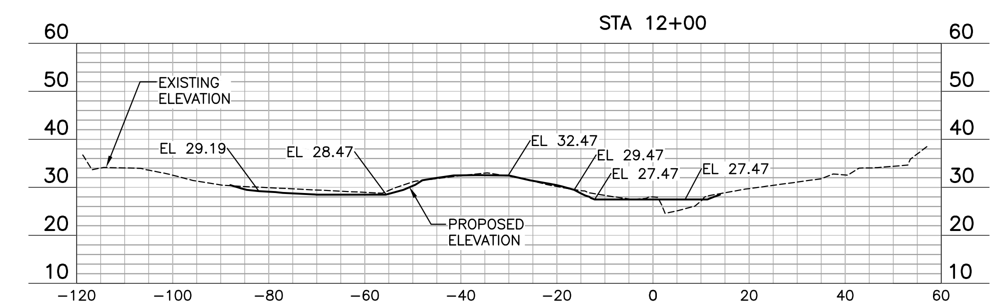
PERMITTING DRAWINGS
NOT FOR CONSTRUCTION



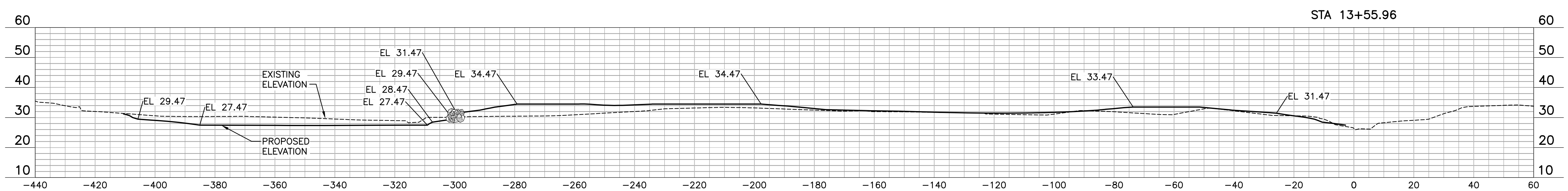
SECTION A C-05
SCALE: 1"=20'



SECTION C C-05
SCALE: 1"=20'



SECTION D C-05
SCALE: 1"=20'



SECTION B C-05
SCALE: 1"=20'

PERMITTING DRAWINGS
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HDR C:\pwworking\sea\d1128869\C-10.dwg PRINTED: 2/25/2015 3:49:23 PM BY: jecampbe

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PROJECT NUMBER:	000000000171201

ISSUE DATE DESCRIPTION

MILL CREEK PARK DAM IMPROVEMENTS

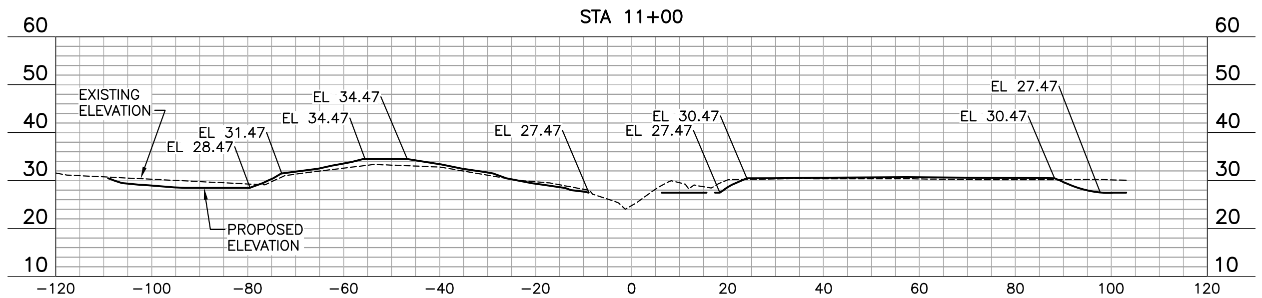
CITY OF COSMOPOLIS

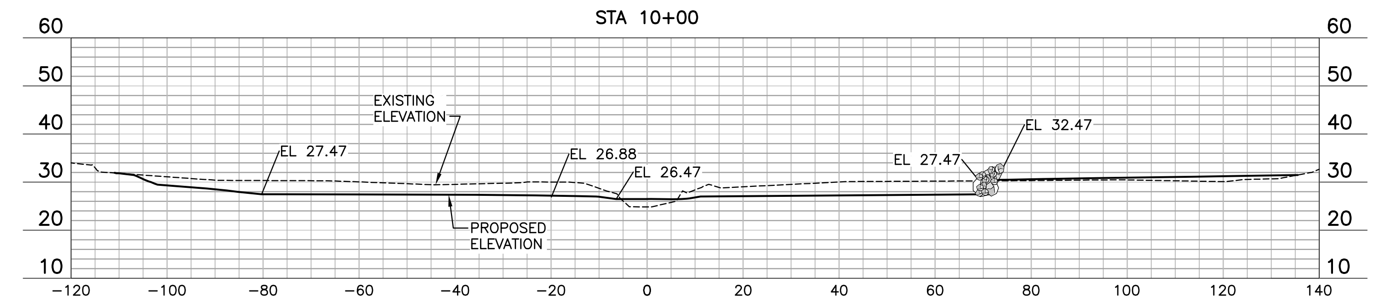
POND CROSS SECTIONS 1

0 1" 2"

FILENAME C-10.dwg
SCALE AS NOTED

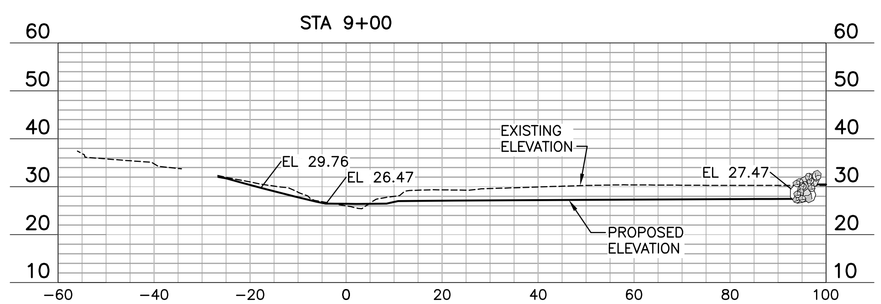
SHEET: 13 Of 16
C-10

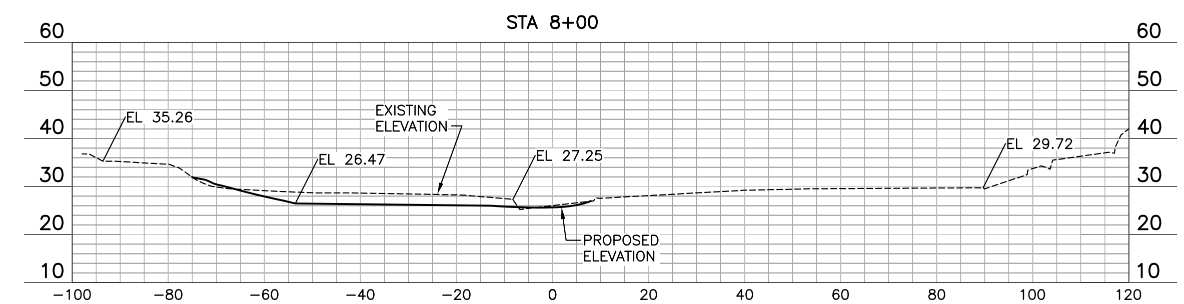




SECTION E C-05
SCALE: 1"=20'

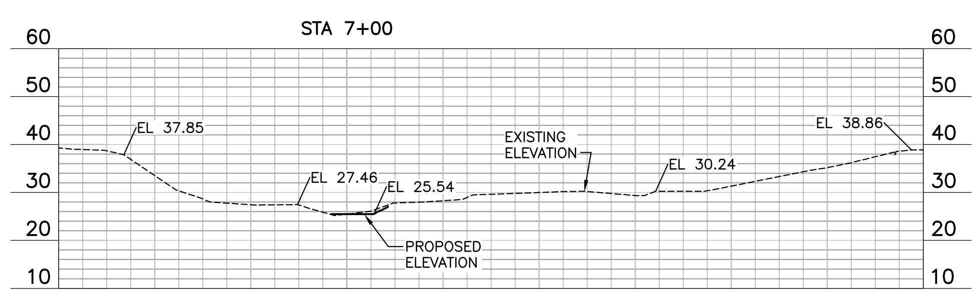
SECTION F C-05
SCALE: 1"=20'

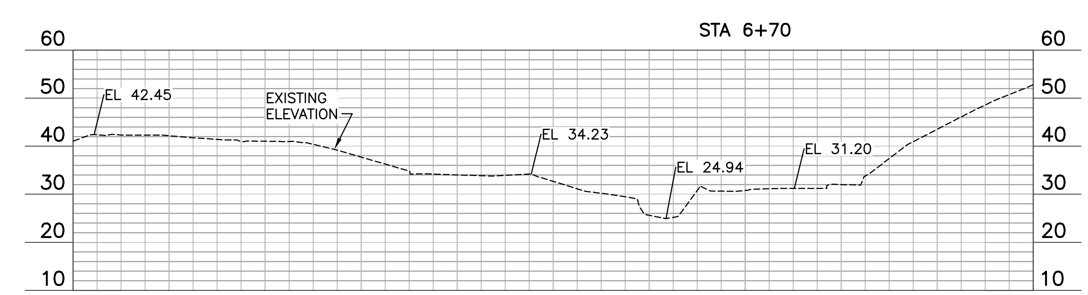




SECTION G C-05
SCALE: 1"=20'

SECTION H C-05
SCALE: 1"=20'





SECTION I C-05
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SECTION J C-05
SCALE: 1"=20'

PERMITTING DRAWINGS
NOT FOR CONSTRUCTION

HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER:	T. HUME
DESIGNED BY:	
CHECKED BY:	
DRAWN BY:	
PROJECT NUMBER:	000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

POND CROSS SECTIONS 2

0 1" 2"

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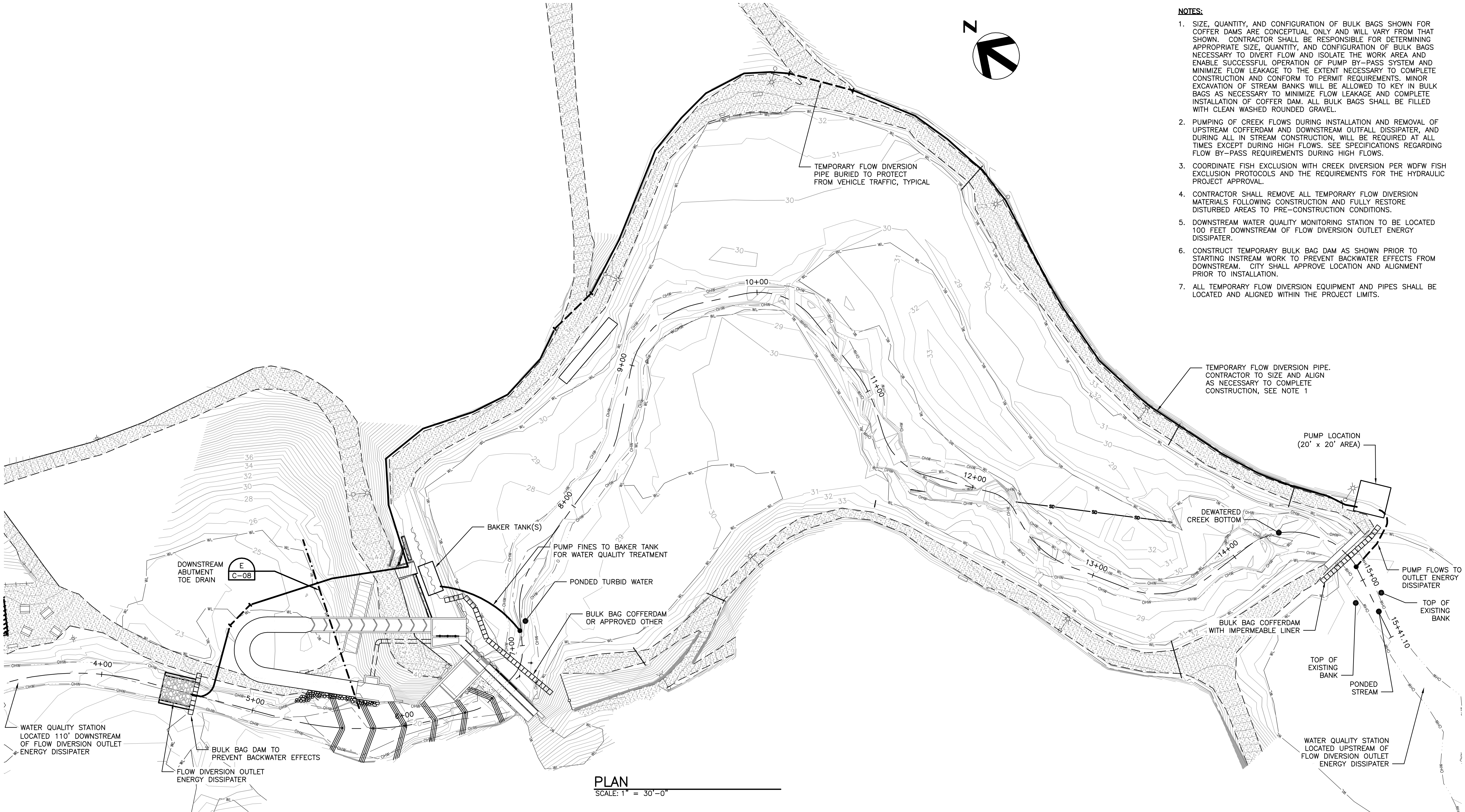
SHEET: 14 Of 16
C-11

NOTES:

1. SIZE, QUANTITY, AND CONFIGURATION OF BULK BAGS SHOWN FOR COFFER DAMS ARE CONCEPTUAL ONLY AND WILL VARY FROM THAT SHOWN. CONTRACTOR SHALL BE RESPONSIBLE FOR DETERMINING APPROPRIATE SIZE, QUANTITY, AND CONFIGURATION OF BULK BAGS NECESSARY TO DIVERT FLOW AND ISOLATE THE WORK AREA AND ENABLE SUCCESSFUL OPERATION OF PUMP BY–PASS SYSTEM AND MINIMIZE FLOW LEAKAGE TO THE EXTENT NECESSARY TO COMPLETE CONSTRUCTION AND CONFORM TO PERMIT REQUIREMENTS. MINOR EXCAVATION OF STREAM BANKS WILL BE ALLOWED TO KEY IN BULK BAGS AS NECESSARY TO MINIMIZE FLOW LEAKAGE AND COMPLETE INSTALLATION OF COFFER DAM. ALL BULK BAGS SHALL BE FILLED WITH CLEAN WASHED ROUNDED GRAVEL.
2. PUMPING OF CREEK FLOWS DURING INSTALLATION AND REMOVAL OF UPSTREAM COFFERDAM AND DOWNSTREAM OUTFALL DISSIPATER, AND DURING ALL IN STREAM CONSTRUCTION, WILL BE REQUIRED AT ALL TIMES EXCEPT DURING HIGH FLOWS. SEE SPECIFICATIONS REGARDING FLOW BY–PASS REQUIREMENTS DURING HIGH FLOWS.
3. COORDINATE FISH EXCLUSION WITH CREEK DIVERSION PER WDFW FISH EXCLUSION PROTOCOLS AND THE REQUIREMENTS FOR THE HYDRAULIC PROJECT APPROVAL.
4. CONTRACTOR SHALL REMOVE ALL TEMPORARY FLOW DIVERSION MATERIALS FOLLOWING CONSTRUCTION AND FULLY RESTORE DISTURBED AREAS TO PRE–CONSTRUCTION CONDITIONS.
5. DOWNSTREAM WATER QUALITY MONITORING STATION TO BE LOCATED 100 FEET DOWNSTREAM OF FLOW DIVERSION OUTLET ENERGY DISSIPATER.
6. CONSTRUCT TEMPORARY BULK BAG DAM AS SHOWN PRIOR TO STARTING INSTREAM WORK TO PREVENT BACKWATER EFFECTS FROM DOWNSTREAM. CITY SHALL APPROVE LOCATION AND ALIGNMENT PRIOR TO INSTALLATION.
7. ALL TEMPORARY FLOW DIVERSION EQUIPMENT AND PIPES SHALL BE LOCATED AND ALIGNED WITHIN THE PROJECT LIMITS.



PLAN
SCALE: 1" = 30'-0"

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HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER:	T. HUME
DESIGNED BY:	
CHECKED BY:	
DRAWN BY:	
PROJECT NUMBER:	000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS
CITY OF COSMOPOLIS Incorporated 1891

STREAM BYPASS PLAN

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SHEET: 15 Of 16

C-12

PLAN
SCALE: 1" = 30'-0"

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HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER:	T. HUME
DESIGNED BY:	
CHECKED BY:	
DRAWN BY:	
PROJECT NUMBER:	000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS
CITY OF COSMOPOLIS Incorporated 1891

PLANTING PLAN

0 1" 2"

FILENAME	C–13.dwg
SCALE	AS NOTED

SHEET: 16 Of 16
C-13

Appendix B

Geotechnical/Structural Technical Memorandum

Technical Memorandum

Date:	May 4, 2015
Project:	Mill Creek Dam Improvements Project
To:	Darrin Raines, Director of Public Works; City of Cosmopolis
From:	Tim Hume, Project Manager; HDR
Prepared by:	Richard Hannan, HDR
Reviewed by:	Chris Krivanec, HDR
Subject:	Geotechnical and Structural – Preliminary Investigations and Alternatives Evaluation

1.0 Introduction

The City of Cosmopolis is evaluating alternatives for the replacement of the failed Mill Creek Park Dam. Prior to its failure, the concrete gravity dam and a short sheet pile wall tied to an earthen embankment. The embankment formed the right abutment of the project (right side of the project when looking downstream). This embankment structure will be referred to as the "right abutment" in this memorandum. Historically, the dam impounded approximately 2 acres within Mill Creek Park. The dam was breached in November 2008 as a result of erosion at the contact area between the concrete gravity dam and the right abutment. Subsequent events caused additional erosion, requiring installation of ecology blocks to control erosion and stabilize the right abutment area. Preliminary subsurface geotechnical investigations have been performed to develop geotechnical site information, including soil and rock properties that are needed for conceptual design of a new concrete dam and fish passage facility.

2.0 Purpose and Scope

This memorandum documents the feasibility level geotechnical site investigation and the soil and rock properties used for the initial evaluation of alternatives for replacement of the failed dam. This memorandum also provides a brief discussion of identified alternatives and geotechnical and design issues related to each, and a preliminary design for the selected alternative.

2.1 Proposed Project

Based on an initial evaluation, the proposed project is an anchored concrete dam founded on rock with a rock abutment on the left side (looking downstream) of the dam. The right side of the concrete dam would tie into a new fish passage structure to be constructed in the existing right abutment. The dam would have two upstream gated 3.5-foot-high by 6-foot-wide outlet structures at the base of the structure to pass instream winter flows and provide winter fish passage. The dam also would have a 2-foot gated pipe to allow controlled passage of summer flows. An ungated spillway would be provided across the left and center of the dam with two, 6-foot-wide spillway sections with provisions for stoplogs to control summer pool elevations. A fish passage structure would be constructed immediately to the right of the dam for full pool fish passage. A sheet pile cutoff wall that penetrates to the top of firm rock is proposed for the right abutment of the dam, extending beyond the fish passage facility as far as needed to control seepage through the abutment.

3.0 Geotechnical Data

3.1 Literature Review

To assist in the geologic evaluation of the site, the Washington Department of Natural Resources Geologic Map GM-53 and the Washington Department of Natural Resources Geologic Map of Washington-Southwest Quadrant were reviewed. The Washington State Water Well database was also reviewed in an attempt to better determine the depth to rock. To evaluate seismic potential, the United States Geological Survey (USGS) Quaternary fault and fold database was reviewed. Probabilistic ground motions for the project site were obtained from the USGS 2008 Earthquake Hazard program website. A detailed list of references is included in Section 7.0.

3.2 Regional and Site Geology

The Chehalis River Basin including the Mill Creek Tributary is located in the northwestern portion of the Willapa Hills Physiographic region. The region is situated between the Olympic Mountains and Columbia River. The region’s rock formations are generally igneous or sedimentary rock formations which are not intensely deformed or altered but tend to have deep weathering profiles. The Washington State Department of Ecology, Natural Resources, Geologic Map of Washington-Southwest Quadrant shows bedrock at the site to be a marine sedimentary rock of Upper Miocene age (5 million to 7 million years old). The rock at the site is mapped as Montesano Formation, consisting of siltstone and sandstone, blue-gray in color when fresh, and containing feldspar and mica flakes. The map shows alluvial deposits or landslide material overlie the bedrock. Based on the regional geology and lack of mapped faults in the area, it is probable that the sandstone formation observed at the site is relatively horizontal. Observations made at the site and subsurface explorations show the rock to be weathered to a depth of 5 to 10 feet below the soil/rock interface.

The existing dam is located in a relatively small stream valley with bedrock forming both valley walls. The rock in the valley walls is relatively soft sandstone that is decomposed to a depth of 5 to 10 feet, and weathered below that. The existing dam is founded on the sandstone. The same sandstone was also encountered in the bore holes. The existing concrete dam is tied to the right abutment. The right abutment consists of an earthen embankment about 200 feet long that abuts the right valley wall. The existing concrete dam is founded on bedrock at approximately Elevation 24.5 feet and is keyed into rock on the left abutment. The right side of the concrete dam was keyed into the right abutment. Initial explorations show the top of bedrock under the right abutment dropping to the right. No information is available related to the earthen embankment that forms the right abutment. Based on initial exploration, it appears that the right abutment is composed of a silty sand with a relatively low permeability and was constructed across the original stream channel. The stream channel may have been located closer to the center or right side of the valley. Initial explorations suggest that the top of decomposed bedrock under the right abutment may drop to approximately Elevation -1.5 feet.

3.3 Seismicity

The U.S. Geological Survey (USGS) of 2008 National Seismic Hazard Mapping Project Probabilistic Seismic Hazard Analyses Interactive Deaggregation web site provided Peak Horizontal Ground Acceleration for selected Mean Return Times. Query results are provided Table 1 and Table 2.

Table 1. Query Results for Peak Horizontal Ground Acceleration at selected Mean Return Times

Mean Return Time (years)	Peak Horizontal Ground Acceleration (g)
108	0.11
475	0.29
975	0.43
2,475	0.70
4,950	0.95

Table 2. PSHA Results for 475-year Return Time

Period (seconds)	SA/PA (g)
0.0	0.29
0.1	0.55
0.2	0.62
0.3	0.55
0.5	0.43
1.0	0.23
2.0	0.11

The USGS Design Maps Summary Report for the site using the 2012 International Building Code is included in Attachment D.

3.4 Field Explorations and Testing

Subsurface exploratory explorations were conducted in two phases. Backhoe explorations were performed on Oct. 17, 2013. Backhoe explorations were discussed in Appendix D of the Technical Memorandum “Mill Creek Dam Improvements Project” dated November 25, 2013. Logs of the backhoe explorations are included in Attachment C.

Phase 2 explorations were performed April 7 and 8, 2014. Two borings were drilled at the Mill Creek Dam Site (BH-1 and BH-2). The boreholes were advanced using a truck mounted Mobile

B-59 drill rig with mud rotary drilling techniques. See Figure 1 “Site Map” for locations of the backhoe pits and borings.

Disturbed samples were obtained at 5-foot intervals with a split-spoon. The Standard Penetration Test (SPT), which is run when the split-spoon is driven, provides an indication of the relative consistency or density of the foundation soils. HQ wire-line coring methods were used in both borings to sample the sandstone bedrock. HQ coring consists of a 2.4-inch inner diameter triple-walled core barrel advanced in maximum 5-foot runs. Core samples were boxed and retained for further review.

BH-1 was drilled on the ecology block supported roadway on the upstream toe of the right abutment, approximately 10 feet from the southwest end of the roadway. The purpose of this boring was to evaluate the depth of the fill, and the strength and consistency of the soils underlying the dam. The boring was drilled to a depth of 54 feet, which included 19 feet of coring through the sandstone bedrock.

BH-2 was also drilled on the ecology block supported roadway on the upstream toe of the right abutment approximately 70 feet from the southwest end of the roadway. The purpose of this boring was to evaluate the depth of the fill, strength and consistency of the soils underlying the dam, and consistency in the elevation of the sandstone bedrock surface. The boring was drilled to a depth of 64.5 feet, which included 20 feet of coring through the sandstone bedrock.

The boreholes were continuously logged during drilling. The final logs were prepared based on a review of the field logs, an examination of the soil samples and laboratory testing. Attachment A contains copies of the field boring logs including a visual description of the materials, sampling intervals, SPT blow counts and laboratory data for each borehole. The explorations site plan is provided as Figure 1.

3.5 Geotechnical Laboratory Testing

Northwest Geotechnical, Inc. conducted laboratory index testing on selected samples from each of the geotechnical borings. Testing consisted of water content, Atterberg limits, and sieve analysis of the soil materials, and unconfined compressive strength of rock core samples. Laboratory data is located in Attachment B.

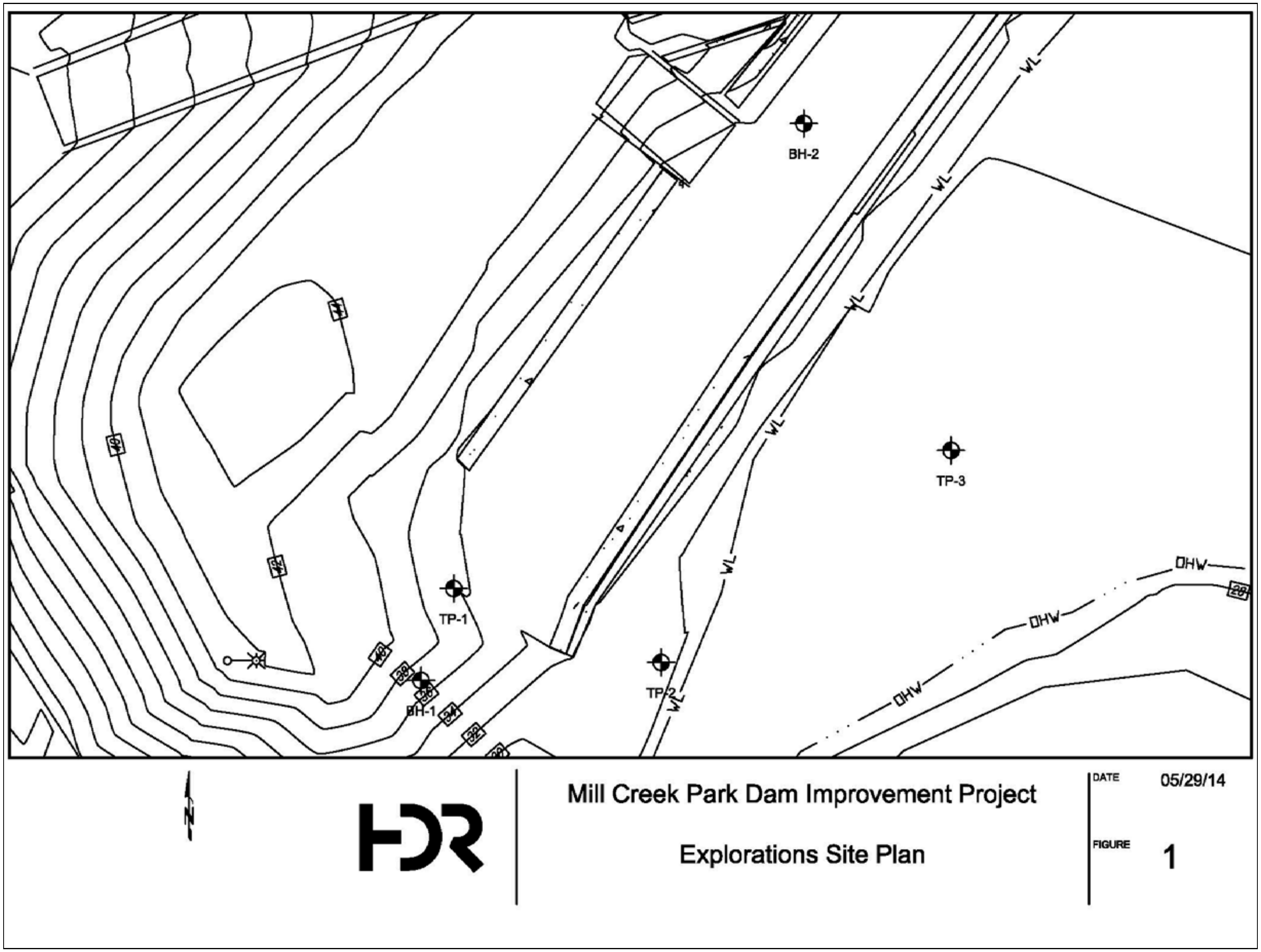


Figure 1. Explorations Site Plan

3.6 Engineering Property Characterization of the Soil

3.6.1 Liquefaction Potential

Based on initial testing, the soil materials sampled from the right abutment are probably not liquefiable. Testing of the soil indicated that the native overburden materials range from sandy silt to silty sand with the silt content (percent passing the No. 200 sieve) ranging form about 30% to 80%. The plasticity index (PI) for the silts generally ranges from 7 to 9, with liquid limits (LL) of all greater than 39%. Additional explorations, sampling, and laboratory testing are needed to better define the nature of the materials forming the right abutment. The additional bore holes need to extend from the top of the abutment to bedrock.

Materials with more than 15% silt material and with LL greater than 35% are not considered liquefiable, (Seed, H.B. and Idriss, I.M. 1982. Monograph: Ground Motions and Soil Liquefaction During Earthquakes, Earthquake Engineering Research Center). Research by Boulanger, Ross W, and IM Idress 2004 (“Evaluating the Potential for Liquefaction or Cyclic Failure of Silts & Calys” Boulanger, Ross W, and IM Idress, Department of Civil & Environmental Engineering, College of Engineering, University of California at Davis, December 2004) classify materials with a PI 7 as sand-like and considered liquefiable, while those with a PI greater than 7 are classified as clay-like. Based on the above criteria the silty sand, sandy silt materials encountered in the two borings are more likely to be susceptible to cyclic softening but probably not susceptible to liquefaction. If sands with less than 15 percent silt or clay are encountered in future explorations, they likely would be liquefiable.

Even though liquefaction may not occur, significant deformation of the abutment should be anticipated during a seismic event. See Section 3.3 for a discussion on seismic loading. Stability analyses have not been performed on the right abutment, but based on the geometry of the abutment it probably does not meet criteria for static or dynamic stability. Despite its probable low factor of safety for stability, the geometry of the right abutment combined with the relatively low elevation of the proposed pool relative to the elevation of the top of the abutment would likely prevent the loss of pool assuming it is at or near normal pool elevation. Analysis of the right abutment will be necessary during the next phase of this project to determine if mitigation is required to meet safety criteria for the project during a seismic event.

3.6.2 Permeability

Laboratory permeability tests were not performed on sampled right abutment materials. The permeability values selected are based on a variety of published sources of information including values developed through extensive testing. A range of permeabilities was presented in Table 7 of the Big Creek Dam No. 1 and No. 2 Preliminary Geotechnical Investigations and Seismic Evaluation, City of Newport, Oregon (HDR 2013). Based on silt contents greater than 28 percent a horizontal permeability of 10^{-4} centimeters/second and a vertical permeability of 10^{-5} centimeters/second were selected. During the design phase of this project, a seepage analysis will be necessary in order to determine the degree of seepage mitigation that will be required to allow the project to meet normal seepage criteria.

3.6.3 Static Strength

Based on previous experience with similar silty sands and sandy silts the following strength parameters were selected:

Shear strength c = 1.2 psi

Angle of Internal Friction Ø = 24°

3.7 Engineering Property Characterization of the Rock

The boring logs of the rock materials indicate a Rock Quality Designation (RQD) of between 0 and 30 with most being 0. Laboratory Unconfined Compressive strengths ranged from 149 psi to 163 psi (1.03 to 1.12 Megapascal [MPa])

3.7.1 Unit Weight

Unit Weight for the rock core was not measured in the laboratory but the rock is assumed to have a unit weight of approximately 140 lb/cubic foot.

3.7.2 Static Strength

Rock strength properties (taken from Table B-2, EM1110-1-2908) for rock with an unconfined compressive strength of between 1 and 5 MPA, and an RQD less than 25 are as follows:

Shear strength c = 14.5 psi

Angle of Internal Friction Ø = 15°

Allowable Bearing capacity

EM1110-1-2908 provides an empirical value of 10 tons per square foot for the allowable bearing capacity for rock with an RQD of 0.

3.8 Site Observations and Findings

The following summarizes observations and findings:

- The rock at the site is a very soft sandstone but has sufficient strength and durability for the structures planned for the site.
- The surface of the rock on the valley floor is highest under the existing concrete dam and drops off to the north.
- Both the north and south valley wall are composed of sandstone similar to that observed in the bore holes. The rock on the north side appears to be much more deeply weathered than the south side.
- Naturally occurring overburden materials recovered in the boring appears to be a non-liquefiable silty sand or sandy silt.
- Due to the relatively high bedrock forming the concrete dam foundation and extending under the right abutment, it is probable that a dam founded on bedrock could be economically constructed at this location. A sheetpile or other type of cutoff wall probably

would be needed for a short distance into the right abutment to control potential seepage around a new concrete dam section.

3.9 Conclusion

Due to the relatively high bedrock forming the foundation of the existing concrete dam and extending under the right abutment, the site appeared to be suitable for construction of a concrete dam and fish passage structures that would be stable during a design earthquake and not have excessive seepage around or under the structure.

Because of the relatively rapid decrease in bedrock elevation in the upstream and downstream direction, a structure with a small footprint would likely be the most economical solution. An embankment-type dam would probably not be the most economical solution. A concrete gravity dam or other concrete type structure constructed in the same location as the existing structure would likely be the most economical and require the least amount of foundation excavation and dewatering effort.

4.0 Alternatives

4.1 Evaluation Criteria

Alternatives were developed during the initial site visit and first design workshop based on the project requirements of providing flood hazard reduction and fish passage during both a full pool and low pool conditions. The alternatives were evaluated based on safety, constructability, ability to meet project requirements, and comparable construction cost. Alternatives advanced for initial evaluation were:

- Dam removal
- Concrete Gravity Dam with Right Abutment Improvements
- Earthen Embankment Dam
- Popup Dams (inflatable dams and bottom-hinged wicker gate dams)

The alternatives evaluated are discussed below.

4.2 Alternatives Evaluation

4.2.1 Dam Removal

4.2.1.1 Description

This alternative returns the site to the condition that existed prior to construction of the concrete gravity dam and requires removal of the existing concrete structure, armoring the right abutment, and significant regrading of the stream channel. Some regrading of the abutments would be required to return slopes to a stable condition.

4.2.1.2 Evaluation

- Flood hazard reduction is not met

- Fish passage is met
- Safety is only met if additional work is performed to stabilize both abutments and to armor the right abutment. Failure to armor the right abutment would allow erosion of the abutment and migration of the channel to the north. As the channel migrates to the north and downcuts an additional 8 to 10 feet below the existing channel elevation.
- Removal of the existing dam and regrading of the left abutment would require stream diversion. Regrading and armoring of the right abutment could be accomplished with minimal effort after the existing dam is removed and the stream diverted to the area of the existing dam.

4.2.2 Concrete Gravity Dam with Right Abutment Improvements

4.2.2.1 Description

This alternative would replace the existing concrete dam with a concrete control structure on the left side of the project. The control structure would have two, 6-foot-wide by 3.5-foot-high openings at the bottom of the structure fitted with slide gates. These openings would allow at-grade passage of winter flows, and a full pool in the summer with the gates closed. A 2-foot diameter gated outlet pipe would be used to control normal flows through the project. The spillway structure designed for normal high flows would be located to the right side of the 6-foot-wide slide gates and adjacent to the fish passage structure. Because the structures are constructed of concrete, overtopping by the Probable Maximum Flood (PMF) is allowed which would significantly reduce the required height of the structure. Some form of seepage and stability mitigation will likely be required for the right abutment area to meet current design standards. It is likely that the mitigation would consist of placement of excess materials at the downstream toe area and installation of a sheet pile cutoff in the upstream portion of the abutment.

4.2.2.2 Evaluation

- The requirement for flood hazard reduction is met.
- The requirement for fish passage is met.
- The requirement for safety is met.
- The project could be constructed without unreasonable difficulty but the constraints on access and working area would require some additional effort on the part of the contractor. Stream diversion and in-water periods would need to be included in the construction schedule.

4.2.3 Earthen Embankment Dam

4.2.3.1 Description

This alternative would require removal of the existing dam and construction of an embankment dam with sufficient height to prevent overtopping in the event of a PMF and construction of a spillway with sufficient capacity to pass the PMF. This alternative would also require a fish passage structure.

4.2.3.2 Evaluation

- The requirement for flood hazard reduction is met
- The requirement for fish passage is met
- The requirement for safety is met
- The ability to construct this alternative would be difficult if the embankment were constructed in the same area as the existing concrete gravity dam. To prevent overtopping of the embankment dam during a PMF the earthen embankment structure would need to be several feet taller than the existing structure concrete dam, and have a very wide spillway. Due to the limited space available, it would be difficult to fit the two structures into the space available unless the spillway section were placed on the left side the existing earthen embankment and fish passage placed adjacent to the spillway. Preliminary calculations suggest that this configuration would leave no room for the embankment dam, and would resemble the concrete dam alternative that has been developed.

4.2.4 Popup Dams (inflatable dams and bottom-hinged wicker gate dams)

4.2.4.1 Description

This alternative would replace the concrete control and spillway structures with a popup structure that could be raised to the required elevation for summertime storage or flood storage or lowered to allow winter flood flows to pass through the project.

4.2.4.2 Evaluation

- The requirement for flood hazard reduction is met
- The requirement for fish passage is met
- The requirement for safety is met
- The alternative would have most of the same constraints as the concrete alternative. Operation and maintenance of this alternative was assessed to be more difficult than the other alternatives, and the cost of construction was determined to be greater than the other alternatives.

5.0 Alternative Design

Based on the evaluation of the alternatives, the concrete dam alternative was selected by the design team for final consideration due to its ability to better meet the following project requirements:

- The ability to incorporate fish passage
- Ease of operation
- Probable lowest construction cost and life cycle cost

5.1 Structural Design

Structural design of the dam and its associated structures has not been performed during this phase of the project. However, a preliminary pseudo-static stability analysis was performed for the gravity section under the following loading conditions:

- Normal flow
- 100-year flood
- 500-year flood
- Probable Maximum Flood
- Operational Based Earthquake (OBE)
- Maximum Design Earthquake (MDE)
- OBE, No Water
- Construction Case.

Two 1-foot wide sections of the dam were analyzed: the first through the solid mass of the dam structure and the second through the gated section. Stability calculations and assumptions are provided and summarized in Attachment E.

5.2 Stability Analysis Criteria

Dam Gravity Section

Top of Block Elevation	33.74
Upstream Bottom of Dam	24.74
Downstream Bottom of Dam	24.74
Base Elevation	23.74
Width of Section	1.00'
Overall Cross Section Width Under Study	1.00'
Width of Foundation (Heel to Toe)	10.00'

River Water Elevations

Normal Head Water Elevation	Normal	33.74
Normal Tail Water Elevation	Normal	26.70
Differential Head		7.04'
500-year Flood Head Water Elevation		35.80
500-year Flood Tail Water Elevation		29.10
Differential Head		6.70'
100-year Flood Head Water Elevation		35.30
100-year Flood Tail Water Elevation		28.70
Differential Head		6.60'

Soil Data

Upstream Soil Friction Angle		15.0 deg*
Upstream Soil Unit Weight (Dry)		125 pcf**
Upstream Soil Unit Weight (Submerged)		65 pcf
Active Lat. Soil Pressure Coefficient Ka = (1-sin f)/(1+sin f)		0.59
Downstream Soil Friction Angle		15.0 deg
Downstream Soil Unit Weight (Dry)		125 pcf
Downstream Soil Unit Weight (Submerged)		65 pcf
Passive Lat. Soil Pressure Coeff. Kp = (1-sin f)/(1+sin f)		1.70
Allowable Bearing Pressure	Normal Loading	20 ksf***
Allowable Bearing Pressure	Seismic Loading	26.60 ksf
Shear Strength (Cohesion Resistance at Base)	14.5 psi =	2.088 ksf
Underlying Foundation Material Angle of Friction		15.0 deg
Coefficient Friction (Soil/Concrete)		0.27
Soil/Soil Friction Factor		0.27

* deg=degrees
** pcf=pounds per cubic foot
*** ksf=kips per square foot

Seismic Data

Operating Basis Earthquake Peak Acceleration (OBE)	Horizontal	0.25g*
Operating Basis Earthquake Peak Acceleration (OBE)	Vertical	0.16g
Maximum Design Earthquake Peak Acceleration (MDE)	Horizontal	0.84g
Maximum Design Earthquake Peak Acceleration (MDE)	Vertical	0.56g

* g=grams

Required Factors of Safeties

FERC and DOE Criteria are provided below.

Resultant		**Location at Base**	**Required Factor of Safety**
Normal	Usual	Middle 1/3	2.0
500-year Flood	Unusual	Middle ½	1.5
Earthquake (OBE)	Extreme	Within Base	1.0
Earthquake (MDE)	Extreme	Within Base	1.0
FS Floatation			1.1

Structural

Water unit Weight	0.0624 pcf
Concrete Unit Weight	0.15
Concrete Compressive Strength (Lean Concrete)	3,000 psi*
Reinforced Concrete Compressive Strength	4,000 psi
Active Lateral Soil Pressure	73.60 pcf
Passive Lateral Soil Pressure on Downstream Conc Face	212.30 pcf
Passive Lateral Soil Pressure on Downstream Conc Face	142 pcf
Uplift Pressure	See Diagram
Wind Pressure	0
Ice Load	0
Thickness of Ice	0
Active Lateral Seismic Soil Pressure	Mononobe-Okabe Equation
Lateral Hydrodynamic Pressure	Calculate using Westergaard's Formula

* psi=pounds per square inch

5.3 Conclusions

The following conclusions have been drawn based on the analysis performed:

- Based on the above assumptions and criteria, the gravity section without any opening is stable for all loading. There is some tension under the base due to the 500-year Flood and PMF event and for the Maximum Design Earthquake (MDE). That is an earthquake that has a 5 percent chance of being exceeded in a 50-year-return period of 950 years). This would be further evaluated at later design stage and anchors may be provided if needed. This usually is expected for these events.
- Based on the above assumptions and criteria, the gravity section with gate openings is stable for all loading conditions. There is some tension under the base due to the 500-year Flood and PMF event and for both OBE and the MDE earthquakes. Again, this would be further evaluated at later design stage and anchors maybe provided if needed.

6.0 Next Phase

The following additional work is recommended as part of the final design of this project:

- Right abutment explorations to better define top of rock and determine soil properties of the right abutment materials and the materials underlying the abutment.
- One boring downstream of the right abutment to determine top of rock and soil properties of the materials on which the fish facility will be founded.
- Evaluation of the foot bridge foundation area.
- Seepage analysis of the right abutment to define limits of the seepage cutoff and assist in stability analysis.
- Static and seismic stability analysis of the right abutment.
- Seismic settlement evaluation of the right abutment.

- Verify seismic level of shaking for the operational based event, design based event, and maximum credible earthquake event.
- Structural design of selected alternative.
- Structural design of fish passage facility.

7.0 Limitations

This memo was prepared for the exclusive use of the City of Cosmopolis in the planning and design of this project. Information contained herein should not be used for other sites or purposes. The work was done in general accordance with and to a level generally acceptable for planning level studies. No other warranty, expressed or implied, is made.

The conclusions and recommendations contained in this memo are based on site conditions as they existed at the time of the site visit and limited explorations. Soil conditions at other locations may differ from conditions occurring at the test pit (Attachment C) and boring locations (Attachment A).

8.0 References

Boulanger, Ross W and Idriss, M. 2004. Evaluating the Potential for Liquefaction or Cyclic Failure of Silts and Clays. December 2004.

California Board of Senior Consultants [BOSC], 2010. Natomas Levee Improvement Program Board of Senior Consultants, Draft Recommended Initial Permeability Values for Seepage Analyses of Levees in the Natomas Basin (Table). January 31, 2010.

HDR Engineering, Inc., 2013. Final Big Creek Dam No. 1 and No. 2, Preliminary Geotechnical Investigation and Seismic Evaluation. February 2013.

Schuster, Eric J., 2005. Geologic Map of Washington State, Geologic Map GM-53, Washington Division of Geology and Earth Resources. 2005.

Seed, H.B. and Idriss, I.M. 1982. Ground Motions and Soil Liquefaction during Earthquakes. EERI Monograph, Berkeley, California.

U.S. Army Corps of Engineers, 1994. EM 1110-2-2908 – Engineering and Design – Rock Foundations, 30 November 1994.

U.S. Army Corps of Engineers, 2003. EM 1110-2-1902 - Engineering and Design – Slope Stability, October 31, 2003.

U.S. Geological Survey (USGS), 2002. National Seismic Hazard Mapping Project, Probabilistic Hazard Look-up by Latitude/Longitude. Accessed May 7, 2013. http://eqhazmape.usgs.gov

U.S. Geological Survey (USGS), 2006. Quaternary fault and fold data base for the United States. Accessed on May 7 2013. http://earthquake.usgs/regional/qfaults/wa/index.php

Washington Department of Ecology, Water Well Log Viewer. Accessed May 6, 2013. https://fortress.wa.gov/ecy/waterresources/map/WCLSWebMap/WellConstructionMapSearch.aspx

Attachment A

Boring Logs

HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204
Telephone: (503) 423-3700
Fax: (503) 423-3737

BORING NUMBER BH-1

CLIENT City of Cosmopolis
PROJECT NAME Mill Creek Park Dam
PROJECT NUMBER 002-171201-010
PROJECT LOCATION Cosmopolis, WA
DATE STARTED 4/7/14 **COMPLETED** 4/7/14
GROUND ELEVATION 34.5 ft **HOLE SIZE** 5" inches
DRILLING CONTRACTOR Cascade Drilling (Mobile B-59)
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING --- Could not determine depth due to drilling metho
LOGGED BY Nick Clark **CHECKED BY** Rich Hannan
AT END OF DRILLING ---
NOTES Standard SPT, calibrated hammer
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		POORLY GRADED GRAVEL, (GP) gray, dense (fill)								
5		SANDY SILT, (ML) gray, wet, very soft, low plasticity (alluvium)	SPT SS-1-1	100	4-2-1 (3)					
			SPT SS-1-2	0	3-1-1 (2)					
10			SPT SS-1-3	0	0-0-1 (1)	61	41	34	7	63
15			SPT SS-1-4	0	4-3-2 (5)					
20		SILTY SAND, (SM) gray, wet, very loose, medium sand (alluvium)	SPT SS-1-5	22	0-0-1 (1)					
25			SPT SS-1-6	100	0-0-2 (2)		39	31	8	
30		SILTY SAND, (SM) light gray to white, moist, very dense, weak cementation (decomposed sandstone)	SPT SS-1-7	100	50/3"					
35										

(Continued Next Page)

HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204
Telephone: (503) 423-3700
Fax: (503) 423-3737

BORING NUMBER BH-1

CLIENT City of Cosmopolis
PROJECT NAME Mill Creek Park Dam
PROJECT NUMBER 002-171201-010
PROJECT LOCATION Cosmopolis, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS LIQUID LIMIT	PLASTIC LIMIT	PLASTICITY INDEX	FINES CONTENT (%)
35		SANDSTONE, gray very close to close fracture spacing, decomposed to highly weathered, extremely weak (R0) to weak (R2), scatter shell fragments, joints are open, rough and irregular (tertiary marine sedimentary rock)	Run 1	0	50/5"					
40			Run 2	31 (0)						
45		44 - 49 ft.: Scattered, thin beds of moderately weathered, medium hard (R3) to hard (R4), very close to close jointed, with scattered shell fragments	Run 3	40 (8)						
50			Run 4	65 (30)						

Bottom of borehole at 54.0 feet.

HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204
Telephone: (503) 423-3700
Fax: (503) 423-3737

BORING NUMBER BH-2

CLIENT City of Cosmopolis
PROJECT NAME Mill Creek Park Dam
PROJECT NUMBER 002-171201-010
PROJECT LOCATION Cosmopolis, WA
DATE STARTED 4/8/14 **COMPLETED** 4/8/14
GROUND ELEVATION 34.5 ft **HOLE SIZE** 5" inches
DRILLING CONTRACTOR Cascade Drilling (Mobile B-59)
GROUND WATER LEVELS:
DRILLING METHOD Mud Rotary
AT TIME OF DRILLING --- Could not determine depth due to drilling metho
LOGGED BY Nick Clark **CHECKED BY** Rich Hannan
AT END OF DRILLING ---
NOTES Standard SPT, calibrated hammer
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		POORLY GRADED GRAVEL, (GP) gray, dense (fill)								
5		SILT, (MH) gray, wet, very soft, medium plasticity, trace organics (alluvium)	SPT SS-2-1	100	1-1-1 (2)					
10			SPT SS-2-2	100	0-0-0 (0)		56	35	21	
15			SPT SS-2-3	100	0-0-0 (0)					
20			SPT SS-2-4	100	0-0-0 (0)					
25		Large wood fragments at 25 ft.	SPT SS-2-5	100	0-0-0 (0)	66	56	47	9	80
		SILTY SAND, (SM) blue/gray to brown/tan, wet, very loose, medium sand (alluvium)								
30			SPT SS-2-6	17	6-1-2 (3)					
35										

(Continued Next Page)

HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204
Telephone: (503) 423-3700
Fax: (503) 423-3737

BORING NUMBER BH-2

CLIENT City of Cosmopolis **PROJECT NAME** Mill Creek Park Dam

PROJECT NUMBER 002-171201-010 **PROJECT LOCATION** Cosmopolis, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
35		SILTY SAND, (SM) blue/gray to brown/tan, wet, dense to very dense, fine sand, some mica (decomposed sandstone)	SS-2-7	100	3-21-17 (38)					28
40			SS-2-8	33	31-50					
45		SANDSTONE gray, close fracture spacing, decomposed to highly weathered, extremely weak (R0), joints are open, rough and irregular (tertiary marine sedimentary rock)	Run 1	47 (0)	50					
50			Run 2	57 (20)						
55			Run 3	47 (0)						
60			Run 4	59 (0)						

Bottom of borehole at 64.5 feet.

Attachment B

Northwest Geotech, Inc. Laboratory Testing

Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97370 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To:	Mr. Richard Hannan, P.E., R.P.G., C.E.G. HDR Engineering, Inc. 1001 SW 5th Avenue, Suite 1800 Portland, Oregon 97204	**Date:** **Lab No:**	4/21/14 14-128
Project:	Laboratory Testing	**Project No.:**	2179.1.1

Report of: Atterberg limits, amount of material passing the number 200 sieve, sieve analysis, and compressive strength of rock cores

Sample Identification

NGI completed Atterberg limits, amount of material passing the number 200 sieve, sieve analysis, and compressive strength of rock cores testing on samples delivered to our laboratory. The samples were delivered by a HDR Engineering, Inc. representative on April 11, 2014. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following tables and attached page.

Laboratory Testing

Atterberg Limits (ASTM D4318)			
Sample ID	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**
SS-3-3 @ 10 – 11.5 ft.	41	34	7
SS-3-6 @ 25 – 26.5 ft.	39	31	8
SS-4-2 @ 10 – 11.5 ft.	56	35	21
SS-4-5 @ 25 – 26.5 ft.	56	47	9

Amount of Material Finer than the No. 200 Sieve (ASTM D1140)		
Sample ID	**Moisture Content (%)**	**Percent Passing the No. 200 Sieve**
SS-3-3 @ 10 – 11.5 ft.	60.9	63.0
SS-4-5 @ 25 – 26.5 ft.	65.7	80.1

Copies: Addressee

Northwest Geotech, Inc.
9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To:	Mr. Richard Hannan, P.E., R.P.G., C.E.G. HDR Engineering, Inc. 1001 SW 5th Avenue, Suite 1800 Portland, Oregon 97204	**Date:** **Lab No:**	4/21/14 14-128
Project:	Laboratory Testing	**Project No.:**	2179.1.1

Laboratory Testing

Sieve Analysis of Aggregate (ASTM C136/ C117)	
Sieve Size	**SS-4-7 @ 35 – 36.5 ft. Percent Passing**
⅜"	100
¼"	99
#4	98
#8	96
#10	95
#16	94
#30	92
#40	88
#50	74
#100	43
#200	27.9

Compressive Strength of Intact Rock Core Specimens (ASTM D 7012 Method C)				
Sample ID	**Diameter (inches)**	**Height (inches)**	**Rate of Loading (%/min)**	**Uniaxial Compressive Strength (psi)**
BH4 Run 4 @ 61 – 61.5 ft.	2.29	3.00	0.2	149

Compressive Strength of Intact Rock Core Specimens (ASTM D 7012 Method C)				
Sample ID	**Diameter (inches)**	**Height (inches)**	**Rate of Loading (%/min)**	**Uniaxial Compressive Strength (psi)**
BH4 Run 4 @ 64 – 64.5 ft.	2.28	2.91	0.2	163

 REVIEWED BY: Bridgett Adame

Attachment C

Test Pit Logs

HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204
Telephone: (503) 423-3700
Fax: (503) 423-3737

TEST PIT NUMBER TP-1

CLIENT City of Cosmopolis
PROJECT NAME Mill Creek Park Dam
PROJECT NUMBER
PROJECT LOCATION Cosmopolis, WA
DATE STARTED 10/10/13 **COMPLETED** 10/10/13
GROUND ELEVATION 38 ft **TEST PIT SIZE** 3 x 8 inches
EXCAVATION CONTRACTOR City of Cosmopolis
GROUND WATER LEVELS:
EXCAVATION METHOD
AT TIME OF EXCAVATION ---
LOGGED BY Rich Hannan **CHECKED BY** Nick Clark
AT END OF EXCAVATION ---
NOTES
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION	Depth (ft)	Elevation (ft)
0.0–3.7		GP		POORLY GRADED GRAVEL, (GP) gray, moist, dense (fill)	3.7	34.3
3.7–7.2		SM		SILTY SAND, (SM) blue / gray to brown / tan, moist, loose, fine sand, organics (alluvium)	7.2	30.8
7.2–9.7		SM		SILTY SAND, (SM) blue, wet, loose, some rock fragments (decomposed sandstone)	9.7	28.3

Depth scale: 0.0, 2.5, 5.0, 7.5

Refusal at 11.7 feet.
Bottom of test pit at 9.7 feet.

HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204
Telephone: (503) 423-3700
Fax: (503) 423-3737

TEST PIT NUMBER TP-2

CLIENT City of Cosmopolis
PROJECT NAME Mill Creek Park Dam
PROJECT NUMBER
PROJECT LOCATION Cosmopolis, WA
DATE STARTED 10/10/13 **COMPLETED** 10/10/13
GROUND ELEVATION 32 ft **TEST PIT SIZE** 3 x 8 inches
EXCAVATION CONTRACTOR City of Cosmopolis
GROUND WATER LEVELS:
EXCAVATION METHOD
AT TIME OF EXCAVATION 10.00 ft / Elev 22.00 ft
LOGGED BY Rich Hannan **CHECKED BY** Nick Clark
AT END OF EXCAVATION ---
NOTES
AFTER EXCAVATION ---

DEPTH (ft)	SAMPLE TYPE NUMBER	U.S.C.S.	GRAPHIC LOG	MATERIAL DESCRIPTION
0.0		SM		SILTY SAND, (SM) brown to blue / gray, wet, loose, fine sand, organics (alluvium)
2.5				
5.0				
7.5				light brown organic silty sand with tree fragments; approximately 18 in. thick
10.0				▽
				11.6 … 20.4

Refusal at 16.6 feet.
Bottom of test pit at 11.6 feet.

Attachment D

USGS Design Maps Summary Report (9/19/2014)

Deaggregation of seismic Hazard for multiple Periods of Spectral Acceleration (from USGS National Seismic Hazard mapping Project , 2008 version)				
Event	Return Period (years)	Spectral Period (seconds)	Horizontal Acceleration (g)	Vertical Acceleration (g)
OBE	144	0.0	0.131	0.087
OBE	144	0.1	0.246	0.164
OBE	144	0.2	0.276	0.184
OBE	144	0.3	0.238	0.159
OBE	144	0.5	0.182	0.121
OBE	144	1.0	0.090	0.060
OBE	144	2.0	0.038	0.025
MDE	950	0.0	0.432	0.288
MDE	950	0.1	0.839	0.560
MDE	950	0.2	0.934	0.623
MDE	950	0.3	0.842	0.562
MDE	950	0.5	0.670	0.447
MDE	950	1.0	0.375	0.250
MDE	950	2.0	0.192	0.128
MCE	2475	0.0	0.699	0.466
MCE	2475	0.1	1.408	0.939
MCE	2475	0.2	1.506	1.005
MCE	2475	0.3	1.370	0.914
MCE	2475	0.5	1.069	0.713
MCE	2475	1.0	0.637	0.425
MCE	2475	2.0	0.369	0.246

OBE - Operational Based Earthquake - An earthquake that can reasonably be expected to occure within the service life of the project (50% probability in 100 years - a return period of 144 years)

MDE - Maximum Design Earthquake - An earthquake that has a 5 percent chance of being period exceeded in a 50 year - Return period of 950 years

MCE - The greatest eathquake that can reasonably be expected to be generated on a specific source, on the basis of seismological and geologic evidence. This event has not been determined but has been assumed to be an event with a return period of 2475 years.

USGS Design Maps Summary Report

User–Specified Input

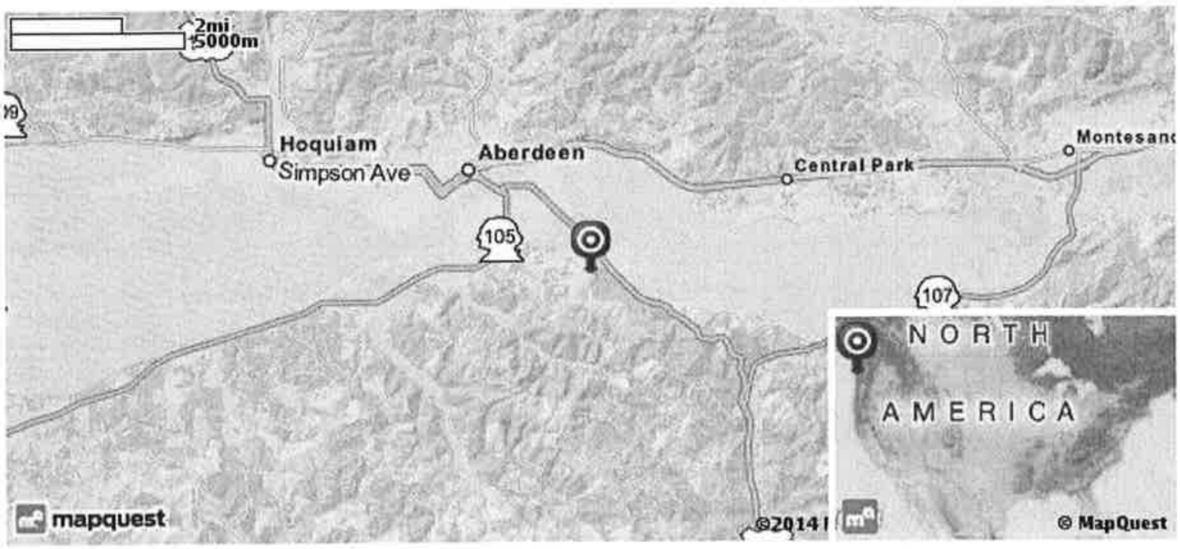
Report Title Cosmopolis Mill Creek
Fri September 19, 2014 14:48:04 UTC

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 46.95°N, 123.773°W

Site Soil Classification Site Class C – "Very Dense Soil and Soft Rock"

Risk Category I/II/III

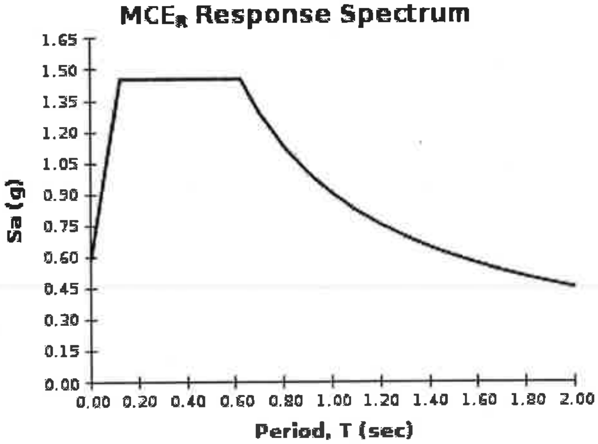


USGS–Provided Output

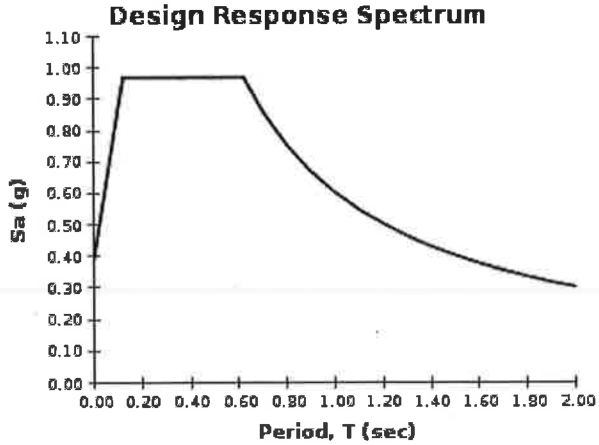
S_S =	1.453 g	S_{MS} =	1.453 g	S_{DS} =	0.969 g
S_1 =	0.696 g	S_{M1} =	0.905 g	S_{D1} =	0.603 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



Design Response Spectrum



USGS Design Maps Detailed Report

2012 International Building Code (46.95°N, 123.773°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 1613.3.1 — Mapped acceleration parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2012 International Building Code are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 1613.3.3.

From Figure 1613.3.1(1)[1]	S_S = 1.453 g
From Figure 1613.3.1(2)[2]	S_1 = 0.696 g

Section 1613.3.2 — Site class definitions

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Section 1613.

2010 ASCE-7 Standard – Table 20.3-1
SITE CLASS DEFINITIONS

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	≥2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 1613.3.3 — Site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters

TABLE 1613.3.3(1)
VALUES OF SITE COEFFICIENT F_a

Site Class	Mapped Spectral Response Acceleration at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_S

For Site Class = C and S_S = 1.453 g, F_a = 1.000

TABLE 1613.3.3(2)
VALUES OF SITE COEFFICIENT F_v

Site Class	Mapped Spectral Response Acceleration at 1-s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	$\boxed{1.3}$
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight-line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.696 g, F_v = 1.300

Equation (16-37): $S_{MS} = F_aS_S = 1.000 \times 1.453 = 1.453$ g

---

Equation (16-38): $S_{M1} = F_vS_1 = 1.300 \times 0.696 = 0.905$ g

---

Section 1613.3.4 — Design spectral response acceleration parameters

Equation (16-39): $S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.453 = 0.969$ g

---

Equation (16-40): $S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.905 = 0.603$ g

---

Section 1613.3.5 — Determination of seismic design category

TABLE 1613.3.5(1)
SEISMIC DESIGN CATEGORY BASED ON SHORT-PERIOD (0.2 second) RESPONSE ACCELERATION

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 0.969 g, Seismic Design Category = D

TABLE 1613.3.5(2)
SEISMIC DESIGN CATEGORY BASED ON 1-SECOND PERIOD RESPONSE ACCELERATION

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.603 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ "the more severe design category in accordance with Table 1613.3.5(1) or 1613.3.5(2)" = D

Note: See Section 1613.3.5.1 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 1613.3.1(1)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(1).pdf
2. *Figure 1613.3.1(2)*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/IBC-2012-Fig1613p3p1(2).pdf

Attachment E

Structural Analysis – Design Calculations

ONE COMPANY Many Solutions	Project: City of Cosmopolis	Computed: NS	Date: 09/10/14
	Subject: Seismic Design	Checked:	Date:
	Task: Cosmopolis Dam	Page: 1	of: 1
	Job #: 171201	No:	

Seismic Design:

Mapped maximum earth quake spectral response acceleration:

Short period response acceleration	S_S 0.2sec	1.453	USGS maps summary report
Long period response acceleration	S_1 1.0 sec	0.696	USGS maps summary report
Soil Site Class		C	Richard Hannan E-Mail dated 09/18/14
Short period site coefficient	F_a =	1.000	
Long period site coefficient	F_v =	1.300	

Design Spectral Acceleration:

	S_{DS}=(2/3)S_{MS}	0.969	USGS maps summary report
	S_{D1}=(2/3)S_{M1}	0.603	USGS maps summary report

Design Response Spectrum:

	C_t	0.020	ASCE7 Table 12.8-2 pg 90
Height of Building	hn	8.00	ft
	x	0.75	ASCE7 Table 12.8-2 pg 90
	Ta=Ct*Hn^x	0.10	ASCE 12.8-7
	T_o=0.2*(S_{D1}/S_{DS})	0.12	ASCE 11.4.5
	Ts=S_{D1}/S_{DS}	0.62	ASCE11.4.5
	TL=	16.00	ASCE Fig 22-12, Pg 224

Risk Category: III — ASCE7 Table 1.5-1 pg 2

Seismic Use Group:

I_e 1.25 — ASCE7 Table 1.5-2 pg 5

Seismic Design Category

Short period Design Category	D	ASCE7 Table 11.6-1 pg. 67
1-S Period Response Design Category	D	ASCE7 Table 11.6-2 pg. 67

Response modification Coefficient **R** 2 — ASCE7 Table 15.4-2 pg. 73
(Flat bottom ground supported tanks with reinforced non sliding base)

12.8 Equivalent Lateral Force Procedure

For T<=T_L	Csmax=S_{D1}/(T*(R/I))	3.96	ASCE7 Eq. (12.8-3) pg 89
For T>T_L	Csmax=S_{D1}*T_L/((T^2)*(R/I))	666.45	ASCE7 Eq. (12.8-4) pg 89
C_S Design	$C_{SDESIGN}$=(S_{DS}/(R/I))	0.61	ASCE7 Eq. (12.8-2) pg 89
	Csmin=0.01	0.01	ASCE7 Eq. (12.8-5) pg 90
	Csmin=0.044S_{DS}I	0.053	ASCE7 Eq. (12.8-5) pg 91
If S_1=>0.6g	$C_S$min=(0.05*S_1)/(R/I)	0.022	ASCE7 Eq. (12.8-6) pg 90

Seismic Base shear	**V=C_s*W=**	**0.61 *W**	ASCE7 Eq.12.8-1

15.4.2 Rigid Non-Building Structure

Seismic Base shear	**V=.3*(S_{DS}*W*I_{e})**	**0.36 *W**	ASCE7 Eq. (15.4-5) pg 144

Mean Return Time (years)	Peak Horizontal Ground Acceleration (g)	
108	0.11	
475	0.29	
975	0.43	
2475	0.7	Rich Hannan Tue 9/16/2014 2:42 PM
4,950	0.95	

Rich Hannan: Thu 9/18/2014 4:22 PM
Mark see attached table of Spectral Acceleration vs Period for seismic events with return times of 144 years, 975 years, and 2475 years. As discussed the 144 year event is a good estimate for the operational based earthquake (no damage), the 975 year event is a good estimate for Maximum Design Earthquake (no failure) and I have included a 2475 year return event to represent the Maximum Credible Earthquake. **For our project my feeling is that we should stick with the OBE and DBE for our design.**

Deaggregation of seismic Hazard for multiple Periods of Spectral Acceleration
(from USGS National Seismic Hazard mapping Project , 2008 version)

Event	Return Period (years)	Spectral Period (seconds)	Horizontal Acceleration (g)	Vertical Acceleration (g)
OBE	**144**	**0.1**	**0.246**	**0.164**
MDE	**950**	**0.1**	**0.839**	**0.560**
MCE	**2475**	**0.1**	**1.408**	**0.939**

Rich Hannan 9/18/2014 4:28 PM

Deaggregation of seismic Hazard for multiple Periods of Spectral Acceleration
(from USGS National Seismic Hazard mapping Project , 2008 version)

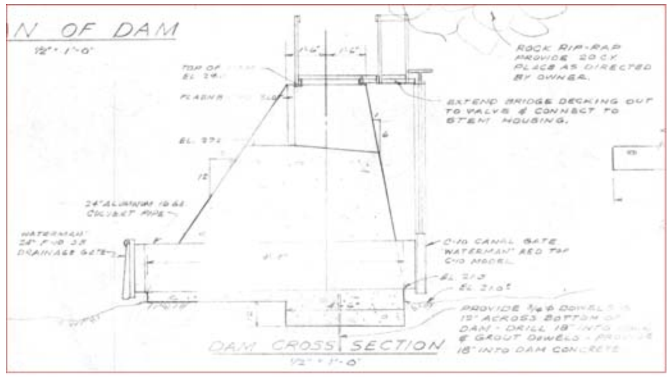
Event	Return Period (years)	Spectral Period (seconds)	Horizontal Acceleration (g)	Vertical Acceleration (g)			
OBE	144	0.0	0.131	0.087			
OBE	**144**	**0.1**	**0.246**	**0.164**			
OBE	144	0.2	0.276	0.184			
OBE	144	0.3	0.238	0.159			
OBE	144	0.5	0.182	0.121			
OBE	144	1.0	0.090	0.060			
OBE	144	2.0	0.038	0.025			
MDE	950	0.0	0.432	0.288			
MDE	**950**	**0.1**	**0.839**	**0.560**			
MDE	950	0.2	0.934	0.623			
MDE	950	0.3	0.842	0.562			
MDE	950	0.5	0.670	0.447			
MDE	950	1.0	0.375	0.250			
MDE	950	2.0	0.192	0.128			
MCE	2475	0.0	0.699	0.466			
MCE	**2475**	**0.1**	**1.408**	**0.939**			
MCE	2475	0.2	1.506	1.005			
MCE	2475	0.3	1.370	0.914			
MCE	2475	0.5	1.069	0.713			
MCE	2475	1.0	0.637	0.425			
MCE	2475	2.0	0.369	0.246			

OBE - Operational Based Earthquake - An earthquake that can reasonably be expected to occur within the service life of the project (50% probability in 100 years - a return period of 144 years)

MDE - Maximum Design Earthquake - An earthquake that has a 5 percent chance of being period exceeded in a 50 year - Return period of 950 years

MCE - The greatest earthquake that can reasonably be expected to be generated on a specific source, on the basis of seismological and geologic evidence. This event has not been determined but has been assumed to be an event with a return period of 2475 years.

PSEUDO STATIC STABILITY ANALYSIS



CITY OF COSMO POLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK **1-FOOT WIDE**

Date Edited	3/23/2015
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS
Cells in red color are input information
Shaded cell indicate a need for confirmation

Notes:

Geotechnical & Seismic Criteria based on Desk top Study by Richard Hannan

If Gate Open Subtract Water Pressure above EL 24.74 **n** **Gate is Closed During Flood**

	COMMENTS				NOTES
Non Overflow Section		NVD-88	SPU		
Top of Block EL		33.74			Drawings C-08 Dated 2-19-15
Upstream Bott of Dam		24.74			Drawings
Downstream Bott of Dam		24.74			Drawings
Base EL		23.74			Drawings
Width of Section		1.00'	0%		
Width of Pier under study (if Studying Pier by itself)		1.00'	100%		Drawings
Overall Cross Section Width Under Study		1.00'	feet		
Width of Foundation (Heel to Toe)		10.00'			Gravity Section
River Water Elevations					
Normal Head Water Elevation (250 cfs)	Normal	33.74			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
Normal Tail Water Elevation (250 cfs)	Normal	26.70			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
Differential Head		7.04'			
500-Yr Flood Head Water Elevation (16,000 cfs)	500-Year Flood	35.80			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
500-Yr Flood Tail Water Elevation (16,000 cfs)	500-Year Flood	29.10			WIS, Basis of Design Report for Dam Stability analysis Feb 2001
Differential Head		6.70'			
100 Year Flood Head Water Elevation -	Standard Flood	35.30			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
100 Year Flood Tail Water Elevation -	Standard Flood	28.70			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
Differential Head		6.60'	feet		
Soil					
Upstream Soil Friction Angle		15.0 deg			Richard Hannan E-Mail dated 05/07/14
Upstream Soil Unit Weight (Dry)		125	pcf	125 pcf	Assumed
Upstream Soil Unit Weight (Submerged)		65	pcf		Assumed
Active Lateral Soil Pressure Coeff. Ka = (1-sin φ)/(1+sin φ)		0.59			MHH (Rankine)
Downstream Soil Friction Angle		15.0 deg	deg		Richard Hannan E-Mail dated 05/07/14

Downstream Soil Unit Weight (Dry)		125	pcf	125 pcf	
Downstream Soil Unit Weight (Submerged)		65	pcf		
Passive Lateral Soil Pressure Coeff. Kp = (1+sin φ)/(1-sin φ)		1.70			MHH (Rankine)
Allowable Bearing Pressure	Normal Loading	**20**	ksf		Richard Hannan E-Mail dated 05/07/14
Allowable Bearing Pressure	Seismic Loading	**26.60**	ksf	Normal *1.33	Assumed
Shear Strength (Cohesion Resistance at Base)		**14.5**	psi		Richard Hannan E-Mail dated 05/07/14
		2.088	ksf		
Underlying Foundation Material Angle of Friction		**15.0 deg**			Richard Hannan E-Mail dated 05/07/14
Coefficent Friction (Soil/Concrete)		**0.27**			Richard Hannan E-Mail dated 05/07/14
Soil/Soil Friction Factor		0.27			
EARTHQUAKE					
Seismic Zone		**N.A.**			
Operating Basis Earthquake Peak Acceleration (OBE)	Horizontal	**0.25g**			USGS National Seismic Hazard mapping Project , 2008 version
Operating Basis Earthquake Peak Acceleration (OBE)	Vertical	**0.16g**			USGS National Seismic Hazard mapping Project , 2008 version
Maximum Design Earthquake Peak Acceleration (MDE)	Horizontal	**0.84g**	N.A.		USGS National Seismic Hazard mapping Project , 2008 version
Maximum Design Earthquake Peak Acceleration (MDE)	Vertical	**0.56g**			USGS National Seismic Hazard mapping Project , 2008 version
Required Factors of Safeties					
FERC and DOE Criteria		FERC & DOE Resultant Location at Base	FERC & DOE Min. Sliding FS		*From Corps of Engineers EM 1110-2-2200 (30 Jun 95) Table 4-1 (Stability Stress Criteria)*
Load Condition					
Normal	Usual	Middle 1/3	**2**		Washington Group, Basis of Design Report, 3/1/2001
500-Year Flood	Unusual	Middle 1/2	**1.25**		Washington Group, Basis of Design Report, 3/1/2001
Earthquake (OBE)	Extreme	Within Base	**1**		Washington Group, Basis of Design Report, 3/1/2001
Earthquake (MDE)	Extreme	Within Base	**1**		N.A.
FS Floatation			**1.1**		Washington Group, Basis of Design Report, 3/1/2001
Structural					
Water unit Weight		**0.0624**	k/cf		
Concrete Unit Weight		**0.15**	k/cf		
Concrete Compressive Strength (Lean Concrete)		3000	psi		
Reinforced Concrete Compressive Strength		**4000**	psi		
Active Lateral Soil Pressure		73.60	*H psf		MHH
Passive Lateral Soil Pressure on Downstream Conc Face		212.30	*H psf		MHH (No Factor of Safety)
Passive Lateral Soil Pressure on Downstream Conc Face		142	*H psf		MHH (With 1.5 Factor of Safety)
Uplift Pressure		See Diagram	*L psf		
Wind Pressure		0	psf		
Ice Load		0	psf		
Thickness of Ice		0	feet		
Active Lateral Seismic Soil Pressure		Mononobe-Okabe Equation			
Lateral Hydrodynamic Pressure		Calculate using Westergaard's Formula			
Seepage					
Permeability range (at B-3)		.01 to .034	cm/sec		Geo Engineers indicated that more analysis is needed
Average Lowest Permeability		.01 to .1	cm/sec		Geo Engineers

RESULTS

Condition 1	**Normal**	**-**	
Normal	Required	Calculated	
FS - Floating	**1.10**	4.28	**OK**
FS - Sliding			
Uplift, No Cohesion	**2**	1.00	**No Good**
No Uplift, No Cohesion	**2**	1.33	**No Good**
Uplift, w/ Cohesion	**2**	7.61	**OK**
No Uplift, w/ Cohesion	**2**	7.94	**OK**
Overturning			
Resultant Location	**1.67**	0.29	**OK**
	Middle 1/3		
Bearing Under Heel	**20.00**	1.06 ksf	**OK**
Bearing Under Toe	**20.00**	1.49 ksf	**OK**

Condition 2	**Flood**	**-**		**Condition 5**			
100 yr flood - gates closed	Required	Calculated		***500 yr flood - gates closed***	Required	Calculated	
FS - Floating	**1.10**	3.23	**OK**	FS - Floating	**1.10**	3.08	**OK**
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1.25**	0.50	**No Good**	Uplift, No Cohesion	**1.25**	0.42	**No Good**
No Uplift, No Cohesion	**1.25**	0.83	**No Good**	No Uplift, No Cohesion	**1.25**	0.74	**No Good**
Uplift, w/ Cohesion	**2**	5.47	**OK**	Uplift, w/ Cohesion	**1.25**	4.99	**OK**
No Uplift, w/ Cohesion	**2**	5.79	**OK**	No Uplift, w/ Cohesion	**1.25**	5.31	**OK**
Overturning				Overturning			
Resultant Location	**2.50**	0.66	**OK**	Resultant Location	**2.50**	0.88	**OK**
	Middle 1/2				**Middle 1/2**		
Bearing Under Heel	**20.00**	0.68 ksf	**OK**	Bearing Under Heel	**20.00**	0.52 ksf	**OK**
Bearing Under Toe	**20.00**	1.56 ksf	**OK**	Bearing Under Toe	**20.00**	1.68 ksf	**OK**

Condition 3	**OBE**			**Condition 6**	**MDE**		
Normal OBE	Required	Calculated		***Normal MDE***	Required	Calculated	
FS - Floating	**1.10**	4.28	**OK**	FS - Floating	1.10	4.28	**OK**
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1**	0.40	**No Good**	Uplift, No Cohesion	**1**	0.16	**No Good**
No Uplift, No Cohesion	**1**	0.54	**No Good**	No Uplift, No Cohesion	**1**	0.22	**No Good**
Uplift, w/ Cohesion	**2**	3.13	**OK**	Uplift, w/ Cohesion	**1.25**	1.30	**OK**
No Uplift, w/ Cohesion	**2**	3.28	**OK**	No Uplift, w/ Cohesion	**1.25**	1.36	**OK**
Overturning				Overturning			
Resultant Location	**10.00**	1.83	**OK**	Resultant Location	**10.00**	5.54	**OK**
	Within Base				**Within Base**		
Bearing Under Heel	**26.60**	-0.12 ksf	**Tension**	Bearing Under Heel	**26.60**	-2.96 ksf	**Tension**
Bearing Under Toe	**26.60**	2.67 ksf	**OK**	Bearing Under Toe	**26.60**	5.51 ksf	**OK**

Condition 4	**OBE**	**No Water**		**Condition 7**	**Construction**	**No Water**	
No Water OBE	Required	Calculated		***No water construction***	Required	Calculated	
FS - Floating	**NA**	NA		FS - Floating	**NA**	NA	
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1**	1.05	**OK**	Uplift, No Cohesion	**2**	111.21	**OK**
No Uplift, No Cohesion	**1**	1.06	**OK**	No Uplift, No Cohesion	**2**	112.09	**OK**
Uplift, w/ Cohesion	**2**	6.39	**OK**	Uplift, w/ Cohesion	**1.25**	678.61	**OK**
No Uplift, w/ Cohesion	**2**	6.40	**OK**	No Uplift, w/ Cohesion	**1.25**	679.49	**OK**
Overturning				Overturning			
Resultant Location	**10.00**	0.38	**OK**	Resultant Location	**1.67**	0.67	**OK**
	Within Base				**Middle 1/3**		
Bearing Under Heel	**26.60**	1.21 ksf	**OK**	Bearing Under Heel	**20.00**	2.20 ksf	**OK**
Bearing Under Toe	**26.60**	1.92 ksf	**OK**	Bearing Under Toe	**20.00**	0.93 ksf	**OK**

Other Factors of Safeties

COE	COE Foundation Bearing Pressure
2	≤ Allowable
1.7	≤ Allowable
1.3	≤ 1.33* Allowable
1	
1.1	

CITY OF COSMO POLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK **1-FOOT WIDE**

Date Edited	2/19/2014
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS
Cells in red color are input information
Shaded cell indicate a need for confirmation

Design Summary & Calculations Water Level Earthquake	Condition 1 Normal	Condition 2 100 Year Flood	Condition 3 Normal OBE	Condition 4 No Water OBE	Condition 5 500 Year Flood	Condition 6 Normal MDE	Condition 7 No Water Construction
Forces							
Structure Weight	kips	kips	kips	kips	kips	kips	kips
Super Stucturre	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Concrete Weight	14.67	14.67	14.67	14.67	14.67	14.67	14.67
Water Weight							
W6 US Water Weight	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W7 - DS Water Weight	0.95	0.49	0.95	0.00	0.59	0.95	0.00
Water Pressure							
Upstream Force	3.12	4.17	3.12	0.00	4.54	3.12	0.00
Downstream Force	-0.27	-0.77	-0.27	0.00	-0.90	-0.27	0.00
If Gate is Open During Flood, Subtract Pressure on Gate abve EL 24.74	0.00	0.00	0.00	**0.00**	0.00	0.00	**0.00**
Uplift							
U (Total)	-3.89	-5.00	-3.89	0.00	-5.28	-3.89	0.00
Silt Pressure							
Upstream Force	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Downstream Force	-0.11	-0.11	-0.11	-0.11	-0.11	-0.11	-0.11
Earthquake							
Super Structure	0.00	0.00	0.25	0.25	0.00	0.84	0.00
Concrete Block	0.00	0.00	3.61	3.61	0.00	12.31	0.00
Inertial Water Upst. Rect.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Inertial Water Downs. Rect.	0.00	0.00	-0.16	0.00	0.00	-0.53	0.00
US Add. Slit Force	0.00	0.00	0.01	0.01	0.00	0.04	0.00
DS Add. Silt Force	0.00	0.00	0.01	0.01	0.00	0.04	0.00
US Add. Water Force	0.00	0.00	0.72	0.00	0.00	2.47	0.00
DS Add. Water Force	0.00	0.00	0.03	0.00	0.00	0.12	0.00
Soil Under Foundation (for Sliding Only)							
Is there any Soil Block being Pushed Foundation key…? (Y,N)	**y**	**N**	**N**	**N**	**N**	**N**	**N**
Weight of Soil Block Under Foundation Being Pushed	0.33	0.00	0.00	0.00	0.00	0.00	0.00
Moment Arms to Toe of Dam							
Structure Weight							
Calcualted in Gravity Section							
Water Weight							
W6 - US Block CG (X-dir)	-10.00	-10.00	-10.00	0.00	-10.00	-10.00	0.00
W7 - US Block CG (X-dir)	-1.84	-1.32	-1.84	0.00	-1.45	-1.84	0.00
Water Pressure							
Upstream CG (Y-dir)	2.33	2.85	2.33	0.00	3.02	2.33	0.00
Downstream CG (Y-dir)	-0.01	0.65	-0.01	0.00	0.79	-0.01	0.00
If Gate is Open During Flood, Subtract Pressure on Gate abve EL 24.74	0.00	2.52	0.00	0.00	2.69	0.00	0.00
Uplift							
Calcualted in Gate Section							
Silt Pressure							
Upstream CG (Y-dir)	0.33	0.33	0.33	0.33	0.33	0.33	0.33
Downstream CG (Y-dir)	0.67	0.67	0.67	0.67	0.67	0.67	0.67

CITY OF COSMO POLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK **1-FOOT WIDE**

Date Edited 2/19/2014
Date Created February 19, 2014
Prepared By Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS

Cells in red color are input information
Shaded cell indicate a need for confirmation

Design Summary & Calculations	Condition 1	Condition 2	Condition 3	Condition 4	Condition 5	Condition 6	Condition 7
Water Level	Normal	100 Year	Normal	No Water	500 Year	Normal	No Water
Earthquake		Flood	OBE	OBE	Flood	MDE	Construction
Earthquake							
Superstructure -CG (Y-dir)	10.53	10.53	10.53	10.53	10.53	10.53	10.53
Concrete Block - CG (Y-dir)	3.85	3.85	3.85	3.85	3.85	3.85	3.85
Inertial Water Upst. Rect.	4.60	0.00	4.60	0.00	0.00	4.60	0.00
Inertial Water Downs. Rect.	0.00	0.00	1.78	0.00	0.00	1.78	0.00
US Add. Slit Force - CG (Y-dir)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DS Add. Silt Force - CG (Y-dir)	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	0.50
US Add. Water Force - CG (Y-dir)	0.00	0.00	4.60	0.00	0.00	4.60	0.00
DS Add. Water Force - CG (Y-dir)	0.00	0.00	1.78	0.00	0.00	1.78	0.00
Floatation							
Vertical Structure Weight (Do Not Include D/S Apron)	15.67	15.67	15.67	15.67	15.67	15.67	15.67
Vertical Water Weight	0.95	0.49	0.95	0.00	0.59	0.95	0.00
Sum of Downward Vertical Forces (Do Not Include D/S Apron)	16.62	16.16	16.62	15.67	16.26	16.62	15.67
Sum of Upward Vertical Forces	3.89	5.00	3.89	0.00	5.28	3.89	0.00
Factor of Safety for Floatation	**4.28**	**3.23**	**4.28**	**NA**	**3.08**	**4.28**	**NA**
Overturning	Normal	100 Year	Normal	No Water	500 Year	Normal	No Water
Moment		Flood	OBE	OBE	Flood	MDE	Construction
Super Structure	-7.50	-7.50	-7.50	-7.50	-7.50	-7.50	-7.50
Conc. Rect.-W1	-81.48	-81.48	-81.48	-81.48	-81.48	-81.48	-81.48
W6 US Water Weight	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W7 - DS Water Weight	-1.76	-0.65	-1.76	0.00	-0.86	-1.76	0.00
Upstream Force	7.28	11.90	7.28	0.00	13.70	7.28	0.00
Downstream Force	0.00	-0.50	0.00	0.00	-0.71	0.00	0.00
If Gate is Open During Flood, Subtract Pressure on Gate abve EL 24.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Uplift (Total)	23.36	28.68	23.36	0.00	30.14	23.36	0.00
Silt Upstream Force	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Silt Downstream Force	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07
Super Structure (Seismic)	0.00	0.00	2.59	2.59	0.00	8.83	0.00
Concrete Block (Seismic)	0.00	0.00	13.91	13.91	0.00	47.44	0.00
Inertial Water Upst. Rect. (Seismic)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Inertial Water Downs. Rect. (Seismic)	0.00	0.00	-0.28	0.00	0.00	-0.95	0.00
US Add. Slit Force (Seismic)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DS Add. Silt Force (Seismic)	0.00	0.00	0.00	0.00	0.00	-0.02	0.00
US Add. Water Force (Seismic)	0.00	0.00	3.33	0.00	0.00	11.37	0.00
DS Add. Water Force (Seismic)	0.00	0.00	0.06	0.00	0.00	0.21	0.00
OVERTURNING							
M_{OVT} (Gate Closed)	30.64	40.58	50.25	16.50	43.84	97.53	0.00
M_{OVT} (Gate Open)	30.64	40.58	50.25	16.50	43.84	97.53	0.00
$M_{RESISTANCE}$	90.68	89.06	90.68	88.92	89.07	90.68	88.92
FS OVT (Gate Closed)	**2.96**	**2.19**	**1.80**	**5.39**	**2.03**	**0.93**	**N/A**
FS $_{OVT}$ (Gate Open)	**2.96**	**2.19**	**1.80**	**5.39**	**2.03**	**0.93**	**N/A**

CITY OF COSMO POLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK	**1-FOOT WIDE**
Date Edited	2/19/2014
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS
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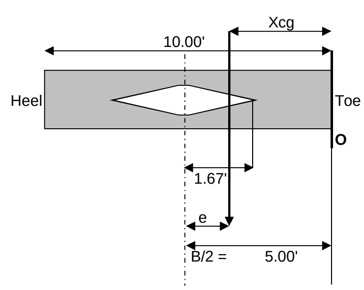
Design Summary & Calculations	Condition 1	Condition 2	Condition 3	Condition 4	Condition 5	Condition 6	Condition 7
Water Level	Normal	100 Year	Normal	No Water	500 Year	Normal	No Water
Earthquake		Flood	OBE	OBE	Flood	MDE	Construction
Sliding							
Sliding Force (Gate Closed)	3.16	4.21	7.63	3.91	4.57	18.43	0.04
Sliding Force (Gate Open)	3.16	4.21	7.63	3.91	4.57	18.43	0.04
Slide Resisting Force without Uplift	4.20	3.49	4.11	4.12	3.39	4.11	4.12
Slide Resisting Force with Uplift	3.15	2.14	3.06	4.12	1.96	3.06	4.12
Cohesion Force	20.88	20.88	20.88	20.88	20.88	20.88	20.88
FS - Uplift, No Cohesion (Gate Closed)	**1.00**	**0.50**	**0.40**	**1.05**	**0.42**	**0.16**	**111.21**
Gate Open	1.00	0.51	0.40	1.06	0.43	0.17	112.09
FS - No Uplift, No Cohesion	**1.33**	**0.83**	**0.54**	**1.06**	**0.74**	**0.22**	**112.09**
Gate Open	1.33	0.83	0.54	1.06	0.74	0.22	112.09
FS - Uplift, w/ Cohesion (Gate Closed)	**7.61**	**5.47**	**3.13**	**6.39**	**4.99**	**1.30**	**678.61**
Gate Open	7.61	5.47	3.14	6.40	4.99	1.30	679.49
FS - No Uplift, w/ Cohesion	**7.94**	**5.79**	**3.28**	**6.40**	**5.31**	**1.36**	**679.49**
Gate Open	7.94	5.79	3.28	6.40	5.31	1.36	679.49
Bearing Pressure (assume flotation)							
Sum of Vertical Forces (R)	12.74	11.16	12.74	15.67	10.98	12.74	15.67
Area of the Base (sf)	10.00	10.00	10.00	10.00	10.00	10.00	10.00
Half Base Width	5.00	5.00	5.00	5.00	5.00	5.00	5.00
Distance from Overturning Point to R (Xcg) "Closed"	4.71	4.34	3.17	4.62	4.12	-0.54	5.67
Distance from CG to Resultant Force, e "Closed"	0.29	0.66	1.83	0.38	0.88	5.54	-0.67
Distance from Overturning Point to R (Xcg) "Open"	4.71	4.34	3.17	4.62	4.12	-0.54	5.67
Distance from CG to Resultant Force, e "Open"	0.29	0.66	1.83	0.38	0.88	5.54	-0.67
B/6	1.67	1.67	1.67	1.67	1.67	1.67	1.67
Structural Modulus (ft^3)	16.67	16.67	16.67	16.67	16.67	16.67	16.67
Bearing Pressure Due Vertical Load (P/A) (ksf) "Closed, Open"	1.27	1.12	1.27	1.57	1.10	1.27	1.57
Bearing Pressure Due Moment (P*e/S) (ksf) "Closed"	0.22	0.44	1.40	0.36	0.58	4.23	-0.63
Bearing Pressure Due Moment (P*e/S) (ksf) "Open"	0.22	0.44	1.40	0.36	0.58	4.23	-0.63
	Normal	100 Year	OBE	OBE	500 Year	MDE	Construction
	Heel Toe	Heel Toe	Heel Toe	Heel Toe	Heel Toe	Heel Toe	Heel Toe
Bearing Conditions	-------- --------	-------- --------	++++ --------	-------- --------	-------- --------	++++ --------	-------- --------
Minimum Bearing Pressure (ksf) Under Heel "Closed	1.06	0.68	**-0.12**	1.21	0.52	**-2.96**	2.20
"Open"	1.06	0.68	-0.12	1.21	0.52	-2.96	2.20
Maximum Bearing Pressure (ksf) Under Toe "Closed"	1.49	1.56	2.67	1.92	1.68	5.51	0.93
"Open"	1.49	1.56	2.67	1.92	1.68	5.51	0.93

* Assuming there is no rigid connection between the D/S Apron and the Ogee or Piers

All Base Under Compression
Tension Under Heel



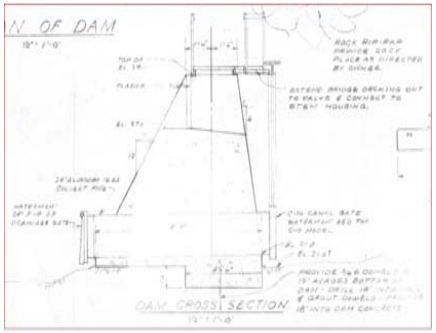
CITY OF COSMO POLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

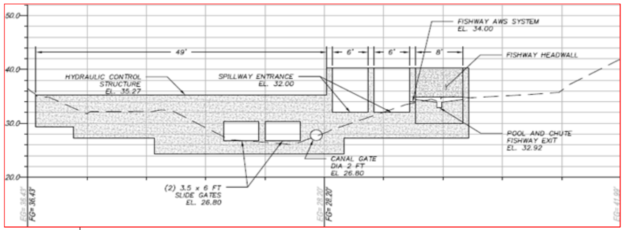
EXISTING BLOCK	**1-FOOT WIDE**
Date Edited	3/23/2015
Date Created	July 31, 2006
Prepared By	Mark H. Hijazi, P.E.

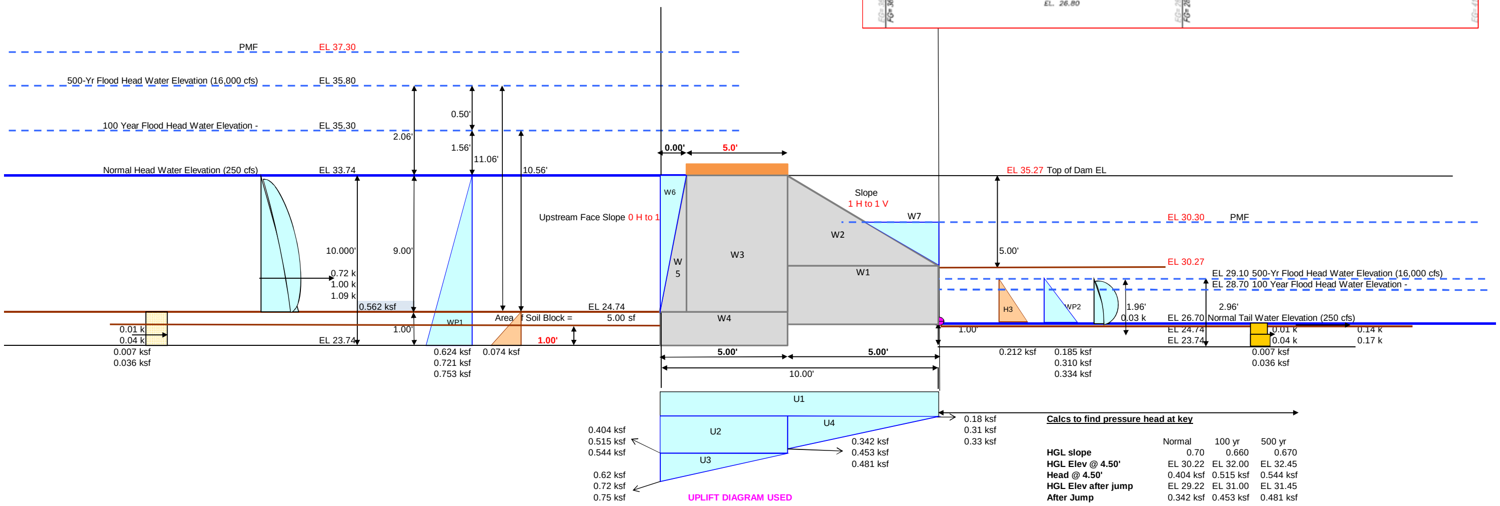
DESIGN ASSUMPTIONS
Cells in red color are input information
Shaded cell indicate a need for confirmation

Input

Section Width Under Study		**1.00'**		
Water unit Weight		62.4	pcf	
Concrete Unit Weight		150	pcf	
Concrete Compressive Strength		3000	psi	
Soil Unit Weight (Submerged)		65	k/cf	
Active Lateral Soil Pressure		73.60	*H psf	MHH
Passive Lateral Soil Pressure on Downstream Cutoff Walls		212.30	*H psf	MHH
Allowable Bearing Pressure	Normal Loading	20	ksf	
Allowable Bearing Pressure	Seismic Loading	26.6	ksf	
Uplift Pressure		See Diagram	*L psf	MHH
Earthquake Peak Acceleration (OBE) Horizontal		**0.25g**		
Earthquake Peak Acceleration (OBE) Vertical		**0.16g**		
Earthquake Peak Acceleration (MDE) Horizontal		0.84g		
Earthquake Peak Acceleration (MDE) Vertical		0.56g		
Active Lateral Seismic Soil Pressure (OBE)		7.06	*H psf	Mononobe-Okabe Equation
Active Lateral Seismic Soil Pressure (MDE)		35.50	*H psf	Mononobe-Okabe Equation
Coeff of Friction (Soil/Concrete)		0.27	Email from Rich Hannan	
Friction Angle (Soil/Soil)	15	0.27	=Tan 15	MHH







Calcs to find pressure head at key

	Normal	100 yr	500 yr
HGL slope	0.70	0.660	0.670
HGL Elev @ 4.50'	EL 30.22	EL 32.00	EL 32.45
Head @ 4.50'	0.404 ksf	0.515 ksf	0.544 ksf
HGL Elev after jump	EL 29.22	EL 31.00	EL 31.45
After Jump	0.342 ksf	0.453 ksf	0.481 ksf

Vertical Forces									
				V	**Direction**	**Xo**	**Yo**	Mo (X)	Mo (Y)
Concrete Structure				**kips**		**feet**	**feet**		k-ft
Steel Supper Structure, Steel I-Beams, Railing and Grating		WB1		1.00	Down	7.50'	10.53'	7.50'	10.53'
Concrete Block									
	Conc. Rect.	W1		4.15	Down	2.50'	2.77'	10.37'	11.47'
	Conc. Tria.	W2		1.88	Down	3.33'	1.67'	6.25'	3.13'
	Conc. Rect.	W3		7.90	Down	7.50'	5.27'	59.23'	41.58'
	Conc. Rect.	W4		0.75	Down	7.50'	0.50'	5.63'	0.38'
	Conc. Tria.	W5		0.00	Up	10.00'	3.51'	0.00'	0.00'
				14.67				81.48'	56.55'
					X_{cg}	5.55'	Y_{cg}		3.85'
Water (Normal)	Upst.Tri	W6		0.00	Up	10.00'			
	Downs. Tri	W7		0.95	Down	1.84'			
Water (100 yr flood)	Upst.Tri	W6		0.00	Up	10.00'			
	Downs. Tri	W7		0.49	Down	1.32'			
Water (500 yr flood)	Upst.Tri	W6		0.00	Up	10.00'			
	Downs. Tri	W7		0.59	Down	1.45'			
Uplift	Uplift Diagram and Calculations are shown above, Designer is to select which uplift condition to apply								

Seismic Inertia Due to Vertical Component of Earthquake

			OBE	**MDE**	**Xo**
					feet
Super Structure			-0.16	-0.56	7.50'
Conc. Rect.	W1		0.00	0.00	2.50'
Conc. Tria.	W2		-0.68	-2.32	3.33'
Conc. Rect.	W3		-0.31	-1.05	7.50'
Conc. Rect.	W4		-1.30	-4.42	7.50'
Conc. Tria.	W5		-0.12	-0.42	10.00'
Water (Normal)					
	Upst.Tri	W6	0.00	0.00	10.00'
	Downs. Tri	W7	-0.16	-0.53	1.84'

Horizontal Forces

			H	**Direction**	**Yo**
			kips		**feet**
Soil	Active Soil Upst.	SS1	0.037	===>	0.33'
	Passive. Soil Dwnst.	SS2	-0.106	<===	0.67'
			-0.069	<=====	
Hydrostatic (Normal)	Hydrostatic Upst.	WP1	3.120	===>	2.33'
	Hydrostatic Dwnst.	WP2	-0.273	<===	-0.01'
			2.847	=====>	
Hydrostatic (100 yr flood)	Hydrostatic Upst.	WP1	4.169	===>	2.85'
If Gate Open Subtract Water Pressure above EL 24.74	Gate is Closed During Flood		0.000	<===	2.52'
	Hydrostatic Dwnst.	WP2	-0.768	<===	0.65'
			3.402	=====>	
Hydrostatic (500 yr flood)	Hydrostatic Upst.	WP1	4.538	===>	3.02'
If Gate Open Subtract Water Pressure above EL 24.74	Gate is Closed During Flood		0.000	<===	2.69'
	Hydrostatic Dwnst.	WP2	-0.896	<===	0.79'
			3.641	=====>	
Seismic (OBE)					
Structure	Super Structure		0.25	===>	10.53'
	Concrete Block		3.61	===>	3.85'
			3.85	===>	
Water Normal WSEL	Water Upstream		0.72	===>	4.60'
	Water Downstream		0.03	===>	1.78'
			4.61		
Silt	Upstream		0.0071	===>	0.00'
	Downstream		0.007	===>	0.5
			0.0141	=====>	
Seismic (MDE)					
Structure Seismic (MDE) (calculated as a Ratio MDE/OBE)					
	Super Structure		0.84	===>	10.53'
	Concrete Block		12.31	===>	3.85'
	Soil Upstream		0.04	===>	0.50'
	Soil Downstream		0.04	===>	0.50'
			13.22		
Water Normal WSEL	Water Upstream		2.47	===>	4.60'
	Water Downstream		0.12	===>	1.78'
			15.81		
Silt	Upstream		0.0355	===>	0.00'
	Downstream		0.036	===>	0.5
			0.0710	=====>	

Full Uplift Diagram								
No Seepage								
No Drain		**HGL EL**	**FND EL**	**Pressure**	**U**	**Direction**	**Xo**	**M_o**
Normal WSEL		**ft**	**ft**	**ft**	**kips**		**feet**	
	Block							
	U1	26.70	24.74	0.18	-1.85	Up	5.00	9.24
	U2	30.22	23.74	0.22	-1.10	Up	7.50	8.24
	U3	33.74	23.74	0.22	-0.55	Up	8.33	4.58
	U4	29.22	24.74	0.16	-0.39	Up	3.33	1.31
	Uplift Pressure CG				**-3.89**	**Up**		23.36

Full Uplift Diagram								
No Seepage								
No Drain		**HGL EL**	**FND EL**	**Pressure**	**U**	**Direction**	**Xo**	**M_o**
100 yr Flood		**ft**	**ft**	**ft**	**kips**		**feet**	
	Block							
	U1	28.70	24.74	0.31	-3.10	Up	5.00	15.48
	U2	32.00	23.74	0.21	-1.03	Up	7.50	7.72
	U3	35.30	20.00	0.21	-0.51	Up	8.33	4.29
	U4	31.00	24.74	0.14	-0.36	Up	3.33	1.20
	Uplift Pressure CG				**-5.00**	**Up**		28.68

Full Uplift Diagram								
No Seepage								
No Drain		**HGL EL**	**FND EL**	**Pressure**	**U**	**Direction**	**Xo**	**M_o**
500 yr Flood		**ft**	**ft**	**ft**	**kips**		**feet**	
	Block							
	U1	29.10	24.74	0.33	-3.34	Up	5.00	16.72
	U2	32.45	23.74	0.21	-1.05	Up	7.50	7.84
	U3	35.80	20.00	0.21	-0.52	Up	8.33	4.35
	U4	31.45	24.74	0.15	-0.37	Up	3.33	1.22
	Uplift Pressure CG				**-5.28**	**Up**		30.14

RESULTS - Strip Through the Dam (No Openings)

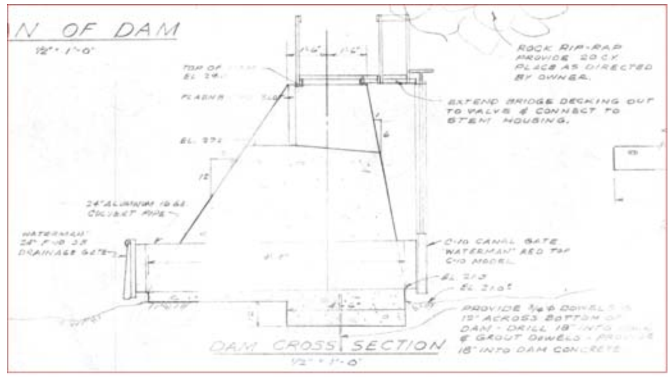
Condition 1	Normal	-	
Normal	Required	Calculated	
FS - Floating	**1.10**	4.28	**OK**
FS - Sliding			
Uplift, No Cohesion	**2**	1.00	**No Good**
No Uplift, No Cohesion	**2**	1.33	**No Good**
Uplift, w/ Cohesion	**2**	7.61	**OK**
No Uplift, w/ Cohesion	**2**	7.94	**OK**
Overturning			
Resultant Location	**1.67**	0.29	**OK**
	Middle 1/3		
Bearing Under Heel	**20.00**	1.06 ksf	**OK**
Bearing Under Toe	**20.00**	1.49 ksf	**OK**

Condition 2	Flood	-		Condition 5	Flood			Condition 8	Flood		
100 yr flood - gates closed	Required	Calculated		*500 yr flood - gates closed*	Required	Calculated		*PMF yr flood - gates closed*	Required	Calculated	
FS - Floating	**1.10**	3.23	**OK**	FS - Floating	**1.10**	2.17	**OK**	FS - Floating	**1.10**	2.72	**OK**
FS - Sliding				FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1.25**	0.50	**No Good**	Uplift, No Cohesion	**1.25**	0.14	**No Good**	Uplift, No Cohesion	**1.25**	0.24	**No Good**
No Uplift, No Cohesion	**1.25**	0.83	**No Good**	No Uplift, No Cohesion	**1.25**	0.46	**No Good**	No Uplift, No Cohesion	**1.25**	0.53	**No Good**
Uplift, w/ Cohesion	**2**	5.47	**OK**	Uplift, w/ Cohesion	**1.25**	4.71	**OK**	Uplift, w/ Cohesion	**1.25**	3.85	**OK**
No Uplift, w/ Cohesion	**2**	5.79	**OK**	No Uplift, w/ Cohesion	**1.25**	5.02	**OK**	No Uplift, w/ Cohesion	**1.25**	4.14	**OK**
Overturning				Overturning				Overturning			
Resultant Location	**2.50**	0.66	**OK**	Resultant Location	**2.50**	2.01	**OK**	Resultant Location	**2.50**	1.73	**OK**
	Middle 1/2				**Middle 1/2**				**Middle 1/2**		
Bearing Under Heel	**20.00**	0.68 ksf	**OK**	Bearing Under Heel	**20.00**	-0.13 ksf	**Tension**	Bearing Under Heel	**20.00**	-0.04 ksf	**Tension**
Bearing Under Toe	**20.00**	1.56 ksf	**OK**	Bearing Under Toe	**20.00**	1.36 ksf	**OK**	Bearing Under Toe	**20.00**	2.14 ksf	**OK**

Condition 3	OBE			Condition 6	MDE		
Normal OBE	Required	Calculated		*Normal MDE*	Required	Calculated	
FS - Floating	**1.10**	4.28	**OK**	FS - Floating	1.10	3.03	**OK**
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1**	0.40	**No Good**	Uplift, No Cohesion	**1**	0.12	**No Good**
No Uplift, No Cohesion	**1**	0.54	**No Good**	No Uplift, No Cohesion	**1**	0.20	**No Good**
Uplift, w/ Cohesion	**2**	3.13	**OK**	Uplift, w/ Cohesion	**1.25**	1.57	**OK**
No Uplift, w/ Cohesion	**2**	3.28	**OK**	No Uplift, w/ Cohesion	**1.25**	1.65	**OK**
Overturning				Overturning			
Resultant Location	**10.00**	1.83	**OK**	Resultant Location	**10.00**	7.28	**OK**
	Within Base				**Within Base**		
Bearing Under Heel	**26.60**	-0.12 ksf	**Tension**	Bearing Under Heel	**26.60**	-2.66 ksf	**Tension**
Bearing Under Toe	**26.60**	2.67 ksf	**OK**	Bearing Under Toe	**26.60**	4.25 ksf	**OK**

Condition 4	OBE	No Water		Condition 7	Construction	No Water	
No Water OBE	Required	Calculated		*No water construction*	Required	Calculated	
FS - Floating	**NA**	NA		FS - Floating	**NA**	NA	
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1**	1.05	**OK**	Uplift, No Cohesion	**2**	76.04	**OK**
No Uplift, No Cohesion	**1**	1.06	**OK**	No Uplift, No Cohesion	**2**	76.64	**OK**
Uplift, w/ Cohesion	**2**	6.39	**OK**	Uplift, w/ Cohesion	**1.25**	643.44	**OK**
No Uplift, w/ Cohesion	**2**	6.40	**OK**	No Uplift, w/ Cohesion	**1.25**	644.04	**OK**
Overturning				Overturning			
Resultant Location	**10.00**	0.38	**OK**	Resultant Location	**1.67**	0.73	**OK**
	Within Base				**Middle 1/3**		
Bearing Under Heel	**26.60**	1.21 ksf	**OK**	Bearing Under Heel	**20.00**	1.56 ksf	**OK**
Bearing Under Toe	**26.60**	1.92 ksf	**OK**	Bearing Under Toe	**20.00**	0.61 ksf	**OK**

PSEUDO STATIC STABILITY ANALYSIS



CITY OF COSMOPOLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK **1-FOOT WIDE**

Date Edited	3/23/2015
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS

Cells in red color are input information
Shaded cell indicate a need for confirmation

Notes:

Geotechnical & Seismic Criteria based on Desk top Study by Richard Hannan

If Gate Open Subtract Water Pressure above EL 24.74 **n** **Gate is Closed During Flood**

	COMMENTS				NOTES
Non Overflow Section		**NVD-88**	**SPU**		
Top of Block EL		**33.74**			Drawings
Upstream Bott of Dam		**24.74**			Drawings
Downstream Bott of Dam		**24.74**			Drawings
Base EL		**23.74**			Drawings
Width of Section		**1.00'**	0%		
Width of Pier under study (if Studying Pier by itself)		**1.00'**	100%		Drawings
Overall Cross Section Width Under Study		**1.00'**	**feet**		
Width of Foundation (Heel to Toe)		**10.00'**			Gravity Section
River Water Elevations					
Normal Head Water Elevation (250 cfs)	Normal	**33.74**			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
Normal Tail Water Elevation (250 cfs)	Normal	**26.70**			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
Differential Head		**7.04'**			
500-Yr Flood Head Water Elevation (16,000 cfs)	500-Year Flood	**35.80**			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
500-Yr Flood Tail Water Elevation (16,000 cfs)	500-Year Flood	**29.10**			WIS, Basis of Design Report for Dam Stability analysis Feb 2001
Differential Head		**6.70'**			
100 Year Flood Head Water Elevation -	Standard Flood	**35.30**			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
100 Year Flood Tail Water Elevation -	Standard Flood	**28.70**			Mill Creek Dam replacement hydrology and hydraulics analysis draft (9/9/2014)
Differential Head		**6.60'**	feet		
Soil					
Upstream Soil Friction Angle		15.0 deg			Richard Hannan E-Mail dated 05/07/14
Upstream Soil Unit Weight (Dry)		125	pcf	125 pcf	Assumed
Upstream Soil Unit Weight (Submerged)		65	pcf		Assumed
Active Lateral Soil Pressure Coeff. Ka = (1-sin φ)/(1+sin φ)		0.59			MHH (Rankine)
Downstream Soil Friction Angle		15.0 deg	deg		Richard Hannan E-Mail dated 05/07/14

Downstream Soil Unit Weight (Dry)		125	pcf	125 pcf	
Downstream Soil Unit Weight (Submerged)		65	pcf		
Passive Lateral Soil Pressure Coeff. Ka = (1+sin φ)/(1-sin φ)		1.70			MHH (Rankine)
Allowable Bearing Pressure	Normal Loading	**20**	ksf		Richard Hannan E-Mail dated 05/07/14
Allowable Bearing Pressure	Seismic Loading	**26.60**	ksf	Normal *1.33	Assumed
Shear Strength (Cohesion Resistance at Base)		**14.5**	psi		Richard Hannan E-Mail dated 05/07/14
		2.088	ksf		
Underlying Foundation Material Angle of Friction		**15.0 deg**			Richard Hannan E-Mail dated 05/07/14
Coefficient Friction (Soil/Concrete)		**0.27**			Richard Hannan E-Mail dated 05/07/14
Soil/Soil Friction Factor		0.27			
EARTHQUAKE					
Seismic Zone		**N.A.**			
Operating Basis Earthquake Peak Acceleration (OBE)	Horizontal	**0.25g**			USGS National Seismic Hazard mapping Project , 2008 version
Operating Basis Earthquake Peak Acceleration (OBE)	Vertical	**0.16g**			USGS National Seismic Hazard mapping Project , 2008 version
Maximum Design Earthquake Peak Acceleration (MDE)	Horizontal	**0.84g**	N.A.		USGS National Seismic Hazard mapping Project , 2008 version
Maximum Design Earthquake Peak Acceleration (MDE)	Vertical	**0.56g**			USGS National Seismic Hazard mapping Project , 2008 version
Required Factors of Safeties					
FERC and DOE Criteria		FERC & DOE Resultant Location at Base	FERC & DOE Min. Sliding FS		*From Corps of Engineers EM 1110-2-2200 (30 Jun 95) Table 4-1 (Stability Stress Criteria)*
Load Condition					
Normal	Usual	Middle 1/3	**2**		Washington Group, Basis of Design Report, 3/1/2001
500-Year Flood	Unusual	Middle 1/2	**1.25**		Washington Group, Basis of Design Report, 3/1/2001
Earthquake (OBE)	Extreme	Within Base	**1**		Washington Group, Basis of Design Report, 3/1/2001
Earthquake (MDE)	Extreme	Within Base	**1**		N.A.
FS Floatation			**1.1**		Washington Group, Basis of Design Report, 3/1/2001
Structural					
Water unit Weight		**0.0624**	k/cf		
Concrete Unit Weight		**0.1006**	k/cf		Equivalent Mass with Gates = .1006
Concrete Compressive Strength (Lean Concrete)		3000	psi		
Reinforced Concrete Compressive Strength		**4000**	psi		
Active Lateral Soil Pressure		73.60	*H psf		MHH
Passive Lateral Soil Pressure on Downstream Conc Face		212.30	*H psf		MHH (No Factor of Safety)
Passive Lateral Soil Pressure on Downstream Conc Face		142	*H psf		MHH (With 1.5 Factor of Safety)
Uplift Pressure		See Diagram	*L psf		
Wind Pressure		0	psf		
Ice Load		0	psf		
Thickness of Ice		0	feet		
Active Lateral Seismic Soil Pressure		Mononobe-Okabe Equation			
Lateral Hydrodynamic Pressure		Calculate using Westergaard's Formula			
Seepage					
Permeability range (at B-3)		.01 to .034	cm/sec		Geo Engineers indicated that more analysis is needed
Average Lowest Permeability		.01 to .1	cm/sec		Geo Engineers

RESULTS

Condition 1	**Normal**	**-**	
Normal	Required	Calculated	
FS - Floating	**1.10**	3.03	**OK**
FS - Sliding			
Uplift, No Cohesion	**2**	0.58	**No Good**
No Uplift, No Cohesion	**2**	0.92	**No Good**
Uplift, w/ Cohesion	**2**	7.20	**OK**
No Uplift, w/ Cohesion	**2**	7.53	**OK**
Overturning			
Resultant Location	**1.67**	0.80	**OK**
	Middle 1/3		
Bearing Under Heel	**20.00**	0.41 ksf	**OK**
Bearing Under Toe	**20.00**	1.17 ksf	**OK**

Condition 2	**Flood**	**-**		**Condition 5**			
100 yr flood - gates closed	Required	Calculated		***500 yr flood - gates closed***	Required	Calculated	
FS - Floating	**1.10**	2.27	**OK**	FS - Floating	**1.10**	2.17	**OK**
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1.25**	0.20	**No Good**	Uplift, No Cohesion	**1.25**	0.14	**No Good**
No Uplift, No Cohesion	**1.25**	0.52	**No Good**	No Uplift, No Cohesion	**1.25**	0.46	**No Good**
Uplift, w/ Cohesion	**2**	5.16	**OK**	Uplift, w/ Cohesion	**1.25**	4.71	**OK**
No Uplift, w/ Cohesion	**2**	5.48	**OK**	No Uplift, w/ Cohesion	**1.25**	5.02	**OK**
Overturning				Overturning			
Resultant Location	**2.50**	1.58	**OK**	Resultant Location	**2.50**	2.01	**OK**
	Middle 1/2				**Middle 1/2**		
Bearing Under Heel	**20.00**	0.03 ksf	**OK**	Bearing Under Heel	**20.00**	-0.13 ksf	**Tension**
Bearing Under Toe	**20.00**	1.23 ksf	**OK**	Bearing Under Toe	**20.00**	1.36 ksf	**OK**

Condition 3	**OBE**			**Condition 6**	**MDE**		
Normal OBE	Required	Calculated		***Normal MDE***	Required	Calculated	
FS - Floating	**1.10**	3.03	**OK**	FS - Floating	1.10	3.03	**OK**
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1**	0.27	**No Good**	Uplift, No Cohesion	**1**	0.12	**No Good**
No Uplift, No Cohesion	**1**	0.44	**No Good**	No Uplift, No Cohesion	**1**	0.20	**No Good**
Uplift, w/ Cohesion	**2**	3.51	**OK**	Uplift, w/ Cohesion	**1.25**	1.57	**OK**
No Uplift, w/ Cohesion	**2**	3.68	**OK**	No Uplift, w/ Cohesion	**1.25**	1.65	**OK**
Overturning				Overturning			
Resultant Location	**10.00**	2.70	**OK**	Resultant Location	**10.00**	7.28	**OK**
	Within Base				**Within Base**		
Bearing Under Heel	**26.60**	-0.49 ksf	**Tension**	Bearing Under Heel	**26.60**	-2.66 ksf	**Tension**
Bearing Under Toe	**26.60**	2.07 ksf	**OK**	Bearing Under Toe	**26.60**	4.25 ksf	**OK**

Condition 4	**OBE**	**No Water**		**Condition 7**	**Construction**	**No Water**	
No Water OBE	Required	Calculated		***No water construction***	Required	Calculated	
FS - Floating	**NA**	NA		FS - Floating	**NA**	NA	
FS - Sliding				FS - Sliding			
Uplift, No Cohesion	**1**	1.03	**OK**	Uplift, No Cohesion	**2**	76.04	**OK**
No Uplift, No Cohesion	**1**	1.04	**OK**	No Uplift, No Cohesion	**2**	76.64	**OK**
Uplift, w/ Cohesion	**2**	8.71	**OK**	Uplift, w/ Cohesion	**1.25**	643.44	**OK**
No Uplift, w/ Cohesion	**2**	8.72	**OK**	No Uplift, w/ Cohesion	**1.25**	644.04	**OK**
Overturning				Overturning			
Resultant Location	**10.00**	0.37	**OK**	Resultant Location	**1.67**	0.73	**OK**
	Within Base				**Middle 1/3**		
Bearing Under Heel	**26.60**	0.84 ksf	**OK**	Bearing Under Heel	**20.00**	1.56 ksf	**OK**
Bearing Under Toe	**26.60**	1.33 ksf	**OK**	Bearing Under Toe	**20.00**	0.61 ksf	**OK**

Other Factors of Safeties

COE	COE Foundation Bearing Pressure
2	≤ Allowable
1.7	≤ Allowable
1.3	≤ 1.33* Allowable
1	
1.1	

CITY OF COSMOPOLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK	**1-FOOT WIDE**
Date Edited	2/19/2014
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS
Cells in red color are input information
Shaded cell indicate a need for confirmation

Design Summary & Calculations	Condition 1	Condition 2	Condition 3	Condition 4	Condition 5	Condition 6	Condition 7
Water Level	Normal	100 Year Flood	Normal	No Water	500 Year Flood	Normal	No Water Construction
Earthquake			OBE	OBE		MDE	
Forces							
Structure Weight	kips	kips	kips	kips	kips	kips	kips
Super Structure	1.00	1.00	1.00	1.00	1.00	1.00	1.00
Concrete Weight	9.84	9.84	9.84	9.84	9.84	9.84	9.84
Water Weight							
W6 US Water Weight	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W7 - DS Water Weight	0.95	0.49	0.95	0.00	0.59	0.95	0.00
Water Pressure							
Upstream Force	3.12	4.17	3.12	0.00	4.54	3.12	0.00
Downstream Force	-0.27	-0.77	-0.27	0.00	-0.90	-0.27	0.00
If Gate is Open During Flood, Subtract Pressure on Gate abve EL 24.74	0.00	0.00	0.00	**0.00**	0.00	0.00	**0.00**
Uplift							
U (Total)	-3.89	-5.00	-3.89	0.00	-5.28	-3.89	0.00
Silt Pressure							
Upstream Force	0.04	0.04	0.04	0.04	0.04	0.04	0.04
Downstream Force	-0.11	-0.11	-0.11	-0.11	-0.11	-0.11	-0.11
Earthquake							
Super Structure	0.00	0.00	0.25	0.25	0.00	0.84	0.00
Concrete Block	0.00	0.00	2.42	2.42	0.00	8.25	0.00
Inertial Water Upst. Rect.	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Inertial Water Downs. Rect.	0.00	0.00	-0.16	0.00	0.00	-0.53	0.00
US Add. Slit Force	0.00	0.00	0.01	0.01	0.00	0.04	0.00
DS Add. Silt Force	0.00	0.00	0.01	0.01	0.00	0.04	0.00
US Add. Water Force	0.00	0.00	0.72	0.00	0.00	2.47	0.00
DS Add. Water Force	0.00	0.00	0.03	0.00	0.00	0.12	0.00
Soil Under Foundation (for Sliding Only)							
Is there any Soil Block being Pushed Foundation key…? (Y,N)	**y**	**N**	**N**	**N**	**N**	**N**	**N**
Weight of Soil Block Under Foundation Being Pushed	0.33	0.00	0.00	0.00	0.00	0.00	0.00
Moment Arms to Toe of Dam							
Structure Weight							
Calculated in Gravity Section							
Water Weight							
W6 - US Block CG (X-dir)	-10.00	-10.00	-10.00	0.00	-10.00	-10.00	0.00
W7 - US Block CG (X-dir)	-1.84	-1.32	-1.84	0.00	-1.45	-1.84	0.00
Water Pressure							
Upstream CG (Y-dir)	2.33	2.85	2.33	0.00	3.02	2.33	0.00
Downstream CG (Y-dir)	-0.01	0.65	-0.01	0.00	0.79	-0.01	0.00
If Gate is Open During Flood, Subtract Pressure on Gate abve EL 24.74	0.00	2.52	0.00	0.00	2.69	0.00	0.00
Uplift							
Calculated in Gate Section							
Silt Pressure							
Upstream CG (Y-dir)	0.33	0.33	0.33	0.33	0.33	0.33	0.33
Downstream CG (Y-dir)	0.67	0.67	0.67	0.67	0.67	0.67	0.67

CITY OF COSMOPOLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK **1-FOOT WIDE**

Date Edited	2/19/2014
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS

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Shaded cell indicate a need for confirmation

Design Summary & Calculations Water Level Earthquake	Condition 1 Normal	Condition 2 100 Year Flood	Condition 3 Normal OBE	Condition 4 No Water OBE	Condition 5 500 Year Flood	Condition 6 Normal MDE	Condition 7 No Water Construction
Earthquake							
Superstructure -CG (Y-dir)	10.53	10.53	10.53	10.53	10.53	10.53	10.53
Concrete Block - CG (Y-dir)	3.85	3.85	3.85	3.85	3.85	3.85	3.85
Inertial Water Upst. Rect.	4.60	0.00	4.60	0.00	0.00	4.60	0.00
Inertial Water Downs. Rect.	0.00	0.00	1.78	0.00	0.00	1.78	0.00
US Add. Slit Force - CG (Y-dir)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DS Add. Silt Force - CG (Y-dir)	-0.50	-0.50	-0.50	-0.50	-0.50	-0.50	0.50
US Add. Water Force - CG (Y-dir)	0.00	0.00	4.60	0.00	0.00	4.60	0.00
DS Add. Water Force - CG (Y-dir)	0.00	0.00	1.78	0.00	0.00	1.78	0.00
Floatation							
Vertical Structure Weight (Do Not Include D/S Apron)	10.84	10.84	10.84	10.84	10.84	10.84	10.84
Vertical Water Weight	0.95	0.49	0.95	0.00	0.59	0.95	0.00
Sum of Downward Vertical Forces (Do Not Include D/S Apron)	11.79	11.33	11.79	10.84	11.43	11.79	10.84
Sum of Upward Vertical Forces	3.89	5.00	3.89	0.00	5.28	3.89	0.00
Factor of Safety for Floatation	**3.03**	**2.27**	**3.03**	**NA**	**2.17**	**3.03**	**NA**
Overturning	Normal	100 Year	Normal	No Water	500 Year	Normal	No Water
Moment		Flood	OBE	OBE	Flood	MDE	Construction
Super Structure	-7.50	-7.50	-7.50	-7.50	-7.50	-7.50	-7.50
Conc. Rect.-W1	-54.64	-54.64	-54.64	-54.64	-54.64	-54.64	-54.64
W6 US Water Weight	0.00	0.00	0.00	0.00	0.00	0.00	0.00
W7 - DS Water Weight	-1.76	-0.65	-1.76	0.00	-0.86	-1.76	0.00
Upstream Force	7.28	11.90	7.28	0.00	13.70	7.28	0.00
Downstream Force	0.00	-0.50	0.00	0.00	-0.71	0.00	0.00
If Gate is Open During Flood, Subtract Pressure on Gate abve EL 24.74	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Uplift (Total)	23.36	28.68	23.36	0.00	30.14	23.36	0.00
Silt Upstream Force	0.01	0.01	0.01	0.01	0.01	0.01	0.01
Silt Downstream Force	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07	-0.07
Super Structure (Seismic)	0.00	0.00	2.59	2.59	0.00	8.83	0.00
Concrete Block (Seismic)	0.00	0.00	9.33	9.33	0.00	31.82	0.00
Inertial Water Upst. Rect. (Seismic)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Inertial Water Downs. Rect. (Seismic)	0.00	0.00	-0.28	0.00	0.00	-0.95	0.00
US Add. Slit Force (Seismic)	0.00	0.00	0.00	0.00	0.00	0.00	0.00
DS Add. Silt Force (Seismic)	0.00	0.00	0.00	0.00	0.00	-0.02	0.00
US Add. Water Force (Seismic)	0.00	0.00	3.33	0.00	0.00	11.37	0.00
DS Add. Water Force (Seismic)	0.00	0.00	0.06	0.00	0.00	0.21	0.00
OVERTURNING							
M_{OVT} (Gate Closed)	30.64	40.58	45.67	11.92	43.84	81.90	0.00
M_{OVT} (Gate Open)	30.64	40.58	45.67	11.92	43.84	81.90	0.00
$M_{RESISTANCE}$	63.85	62.23	63.85	62.08	62.24	63.85	62.08
FS OVT (Gate Closed)	**2.08**	**1.53**	**1.40**	**5.21**	**1.42**	**0.78**	**N/A**
FS_{OVT} (Gate Open)	**2.08**	**1.53**	**1.40**	**5.21**	**1.42**	**0.78**	**N/A**

CITY OF COSMOPOLIS
MILL CREEK PARK DAM-STABILITY ANALYSIS CALCULATIONS

EXISTING BLOCK	**1-FOOT WIDE**
Date Edited	2/19/2014
Date Created	February 19, 2014
Prepared By	Mark H. Hijazi, P.E.

DESIGN ASSUMPTIONS
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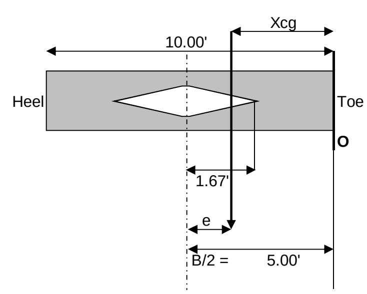
Shaded cell indicate a need for confirmation

Design Summary & Calculations	Condition 1	Condition 2	Condition 3	Condition 4	Condition 5	Condition 6	Condition 7
Water Level	Normal	100 Year	Normal	No Water	500 Year	Normal	No Water
Earthquake		Flood	OBE	OBE	Flood	MDE	Construction
Sliding							
Sliding Force (Gate Closed)	3.16	4.21	6.44	2.72	4.57	14.38	0.04
Sliding Force (Gate Open)	3.16	4.21	6.44	2.72	4.57	14.38	0.04
Slide Resisting Force without Uplift	2.89	2.18	2.80	2.82	2.08	2.80	2.82
Slide Resisting Force with Uplift	1.84	0.84	1.75	2.82	0.66	1.75	2.82
Cohesion Force	20.88	20.88	20.88	20.88	20.88	20.88	20.88
FS - Uplift, No Cohesion (Gate Closed)	**0.58**	**0.20**	**0.27**	**1.03**	**0.14**	**0.12**	**76.04**
Gate Open	0.58	0.20	0.27	1.04	0.14	0.12	76.64
FS - No Uplift, No Cohesion	**0.92**	**0.52**	**0.44**	**1.04**	**0.46**	**0.20**	**76.64**
Gate Open	0.92	0.52	0.44	1.04	0.46	0.20	76.64
FS - Uplift, w/ Cohesion (Gate Closed)	**7.20**	**5.16**	**3.51**	**8.71**	**4.71**	**1.57**	**643.44**
Gate Open	7.20	5.16	3.51	8.72	4.71	1.57	644.04
FS - No Uplift, w/ Cohesion	**7.53**	**5.48**	**3.68**	**8.72**	**5.02**	**1.65**	**644.04**
Gate Open	7.53	5.48	3.68	8.72	5.02	1.65	644.04
Bearing Pressure (assume flotation)							
Sum of Vertical Forces (R)	7.91	6.33	7.91	10.84	6.15	7.91	10.84
Area of the Base (sf)	10.00	10.00	10.00	10.00	10.00	10.00	10.00
Half Base Width	5.00	5.00	5.00	5.00	5.00	5.00	5.00
Distance from Overturning Point to R (Xcg) "Closed"	4.20	3.42	2.30	4.63	2.99	-2.28	5.73
Distance from CG to Resultant Force, e "Closed"	0.80	1.58	2.70	0.37	2.01	7.28	-0.73
Distance from Overturning Point to R (Xcg) "Open"	4.20	3.42	2.30	4.63	2.99	-2.28	5.73
Distance from CG to Resultant Force, e "Open"	0.80	1.58	2.70	0.37	2.01	7.28	-0.73
B/6	1.67	1.67	1.67	1.67	1.67	1.67	1.67
Structural Modulus (ft^3)	16.67	16.67	16.67	16.67	16.67	16.67	16.67
Bearing Pressure Due Vertical Load (P/A) (ksf) "Closed, Open"	0.79	0.63	0.79	1.08	0.62	0.79	1.08
Bearing Pressure Due Moment (P*e/S) (ksf) "Closed"	0.38	0.60	1.28	0.24	0.74	3.45	-0.47
Bearing Pressure Due Moment (P*e/S) (ksf) "Open"	0.38	0.60	1.28	0.24	0.74	3.45	-0.47
	Normal	100 Year	OBE	OBE	500 Year	MDE	Construction
	Heel Toe	Heel Toe	Heel Toe	Heel Toe	Heel Toe	Heel Toe	Heel Toe
Bearing Conditions	-------- --------	-------- --------	++++ --------	-------- --------	++++ --------	++++ --------	-------- --------
Minimum Bearing Pressure (ksf) Under Heel "Closed	0.41	0.03	-0.49	0.84	-0.13	-2.66	1.56
"Open"	0.41	0.03	-0.49	0.84	-0.13	-2.66	1.56
Maximum Bearing Pressure (ksf) Under Toe "Closed"	1.17	1.23	2.07	1.33	1.36	4.25	0.61
"Open"	1.17	1.23	2.07	1.33	1.36	4.25	0.61

* Assuming there is no rigid connection between the D/S Apron and the Ogee or Piers

All Base Under Compression
Tension Under Heel



RESULTS - Strip Through the Dam at the Gate Opening 3.5x6

Manually Calculated the Mass with gate and changes the Unit Weight of Concrete to come up with the reduced Mass

Condition 1	**Normal**	**-**	
Normal	Required	Calculated	
FS - Floating	**1.10**	3.03	**OK**
FS - Sliding			
Uplift, No Cohesion	**2**	0.58	**No Good**
No Uplift, No Cohesion	**2**	0.92	**No Good**
Uplift, w/ Cohesion	**2**	7.20	**OK**
No Uplift, w/ Cohesion	**2**	7.53	**OK**
Overturning			
Resultant Location	**1.67**	0.80	**OK**
	Middle 1/3		
Bearing Under Heel	**20.00**	0.41 ksf	**OK**
Bearing Under Toe	**20.00**	1.17 ksf	**OK**

Condition 2	**Flood**	**-**	
100 yr flood - gates closed	Required	Calculated	
FS - Floating	**1.10**	2.27	**OK**
FS - Sliding			
Uplift, No Cohesion	**1.25**	0.20	**No Good**
No Uplift, No Cohesion	**1.25**	0.52	**No Good**
Uplift, w/ Cohesion	**2**	5.16	**OK**
No Uplift, w/ Cohesion	**2**	5.48	**OK**
Overturning			
Resultant Location	**2.50**	1.58	**OK**
	Middle 1/2		
Bearing Under Heel	**20.00**	0.03 ksf	**OK**
Bearing Under Toe	**20.00**	1.23 ksf	**OK**

Condition 5	**Flood**		
500 yr flood - gates closed	Required	Calculated	
FS - Floating	**1.10**	2.00	**OK**
FS - Sliding			
Uplift, No Cohesion	**1.25**	0.17	**No Good**
No Uplift, No Cohesion	**1.25**	0.48	**No Good**
Uplift, w/ Cohesion	**1.25**	5.05	**OK**
No Uplift, w/ Cohesion	**1.25**	5.36	**OK**
Overturning			
Resultant Location	**2.64**	2.37	**OK**
	Middle 1/2		
Bearing Under Heel	**20.00**	-0.16 ksf	**Tension**
Bearing Under Toe	**20.00**	1.12 ksf	**OK**

Condition 8	**Flood**		
PMF yr flood - gates closed	Required	Calculated	
FS - Floating	**1.10**	1.93	**OK**
FS - Sliding			
Uplift, No Cohesion	**1.25**	0.01	**No Good**
No Uplift, No Cohesion	**1.25**	0.30	**No Good**
Uplift, w/ Cohesion	**1.25**	3.63	**OK**
No Uplift, w/ Cohesion	**1.25**	3.92	**OK**
Overturning			
Resultant Location	**2.50**	3.67	**No Good**
	Middle 1/2		
Bearing Under Heel	**20.00**	-0.68 ksf	**Tension**
Bearing Under Toe	**20.00**	1.82 ksf	**OK**

Condition 3	**OBE**		
Normal OBE	Required	Calculated	
FS - Floating	**1.10**	3.03	**OK**
FS - Sliding			
Uplift, No Cohesion	**1**	0.27	**No Good**
No Uplift, No Cohesion	**1**	0.44	**No Good**
Uplift, w/ Cohesion	**2**	3.51	**OK**
No Uplift, w/ Cohesion	**2**	3.68	**OK**
Overturning			
Resultant Location	**10.00**	2.70	**OK**
	Within Base		
Bearing Under Heel	**26.60**	-0.49 ksf	**Tension**
Bearing Under Toe	**26.60**	2.07 ksf	**OK**

Condition 6	**MDE**		
Normal MDE	Required	Calculated	
FS - Floating	1.10	3.07	**OK**
FS - Sliding			
Uplift, No Cohesion	**1**	0.14	**No Good**
No Uplift, No Cohesion	**1**	0.22	**No Good**
Uplift, w/ Cohesion	**1.25**	2.02	**OK**
No Uplift, w/ Cohesion	**1.25**	2.09	**OK**
Overturning			
Resultant Location	**10.58**	5.79	**OK**
	Within Base		
Bearing Under Heel	**26.60**	-1.47 ksf	**Tension**
Bearing Under Toe	**26.60**	2.75 ksf	**OK**

Condition 4	**OBE**	**No Water**	
No Water OBE	Required	Calculated	
FS - Floating	**NA**	NA	
FS - Sliding			
Uplift, No Cohesion	**1**	1.03	**OK**
No Uplift, No Cohesion	**1**	1.04	**OK**
Uplift, w/ Cohesion	**2**	8.71	**OK**
No Uplift, w/ Cohesion	**2**	8.72	**OK**
Overturning			
Resultant Location	**10.00**	0.37	**OK**
	Within Base		
Bearing Under Heel	**26.60**	0.84 ksf	**OK**
Bearing Under Toe	**26.60**	1.33 ksf	**OK**

Condition 7	**Construction**	**No Water**	
No water construction	Required	Calculated	
FS - Floating	**NA**	NA	
FS - Sliding			
Uplift, No Cohesion	**2**	27.63	**OK**
No Uplift, No Cohesion	**2**	27.85	**OK**
Uplift, w/ Cohesion	**1.25**	284.05	**OK**
No Uplift, w/ Cohesion	**1.25**	284.27	**OK**
Overturning			
Resultant Location	**1.76**	0.52	**OK**
	Middle 1/3		
Bearing Under Heel	**20.00**	1.14 ksf	**OK**
Bearing Under Toe	**20.00**	0.62 ksf	**OK**

Appendix C

Hydrology & Hydraulics Technical Memorandum

Technical Memorandum

Date: May 5, 2015

Project: Mill Creek Dam Improvements Project

To: Darrin Raines, Director of Public Works; City of Cosmopolis

From: Tim Hume, Project Manager; HDR

Prepared By: Steve Thurin, Dave Minner, HDR

Reviewed By: Stan Schweissing, HDR

Subject: Hydrology and Hydraulics

1.0 Introduction

The City of Cosmopolis is evaluating alternatives for the replacement of the failed Mill Creek Park Dam. Prior to its failure, the concrete gravity dam and earthen right abutment impounded approximately 10 acre-feet of water within Mill Creek Park. The dam was breached in November 2008 as a result of erosion at the contact area between the concrete gravity dam and the right abutment. Subsequent events caused additional erosion, requiring the installation of ecology blocks to control erosion and stabilize the right abutment area. Figure 1 shows the project location.

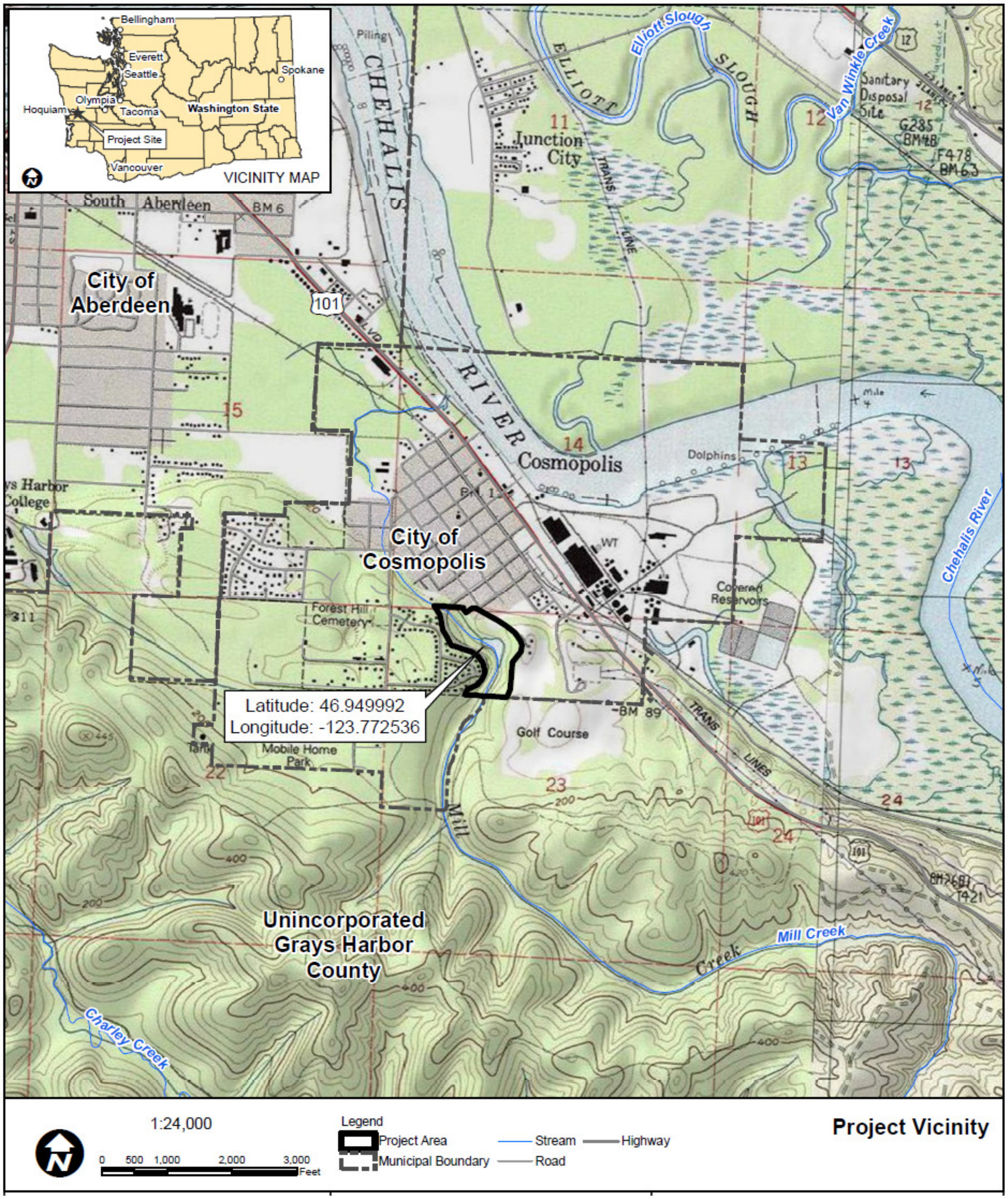


Figure 1. Project Location

2.0 Purpose and Scope

The hydrology and hydraulics (H&H) task of the Mill Creek Dam Replacement Engineering Report is intended to develop feasibility level information to support the conceptual design of critical project elements consistent with Washington Department of Ecology's Dam Safety Office (DSO) guidelines. This technical memorandum (TM) documents four sub-tasks:

- Inflow Design Flood
- Flood Reservoir Routing and Spillway Evaluation
- Dam Break Inundation Analysis
- Downstream Hazard Classification

3.0 Hydrology & Hydraulics

Conceptual design and H&H calculations were performed using methods outlined by the Washington Department of Ecology's Dam Safety Office (DSO). DSO provides dam design and guidance in its Dam Safety Guidelines. The guidance is intended to provide a broad perspective on design philosophy, engineering design considerations, and engineering and construction practices primarily focused on earthen embankments. The Dam Safety Guidelines do not discuss concrete structures in any depth due to the unique design problems that should be addressed by specialty firms well versed and qualified to formulate a suitable design.

3.1 Hydrology and Hydraulic Engineering

3.1.1 Inflow Design Flood

The Mill Creek Dam inflow design flood was preliminarily estimated using runoff computed using a HEC HMS model of the drainage area above the dam. The drainage area boundary was derived from USGS elevation data (StreamStats) and was estimated at 1.67 square miles. The storm drainage network does not appear to impact basin size. No additional development was assumed in the upper basin.

Based upon the Hazard Classification, the design event was assumed to be the Probable Maximum Flood (PMF), following the occurrence of Probable Maximum Precipitation (PMP).

Short, Intermediate, and Long duration PMP storms were developed based upon DSO guidelines outlined in Technical Note 3. The design storms are summarized in Table 1.

Table 1. Summary of DSO PMP Storms

PMP Storm Type	Total (in)	Peak (in/hr)
Short	5.30	11.83
Intermediate	12.29	3.86
Long	24.89	2.93

HEC HMS was used to generate the runoff volumes associated with the three PMP storm types. The SCS method was used. Curve numbers were estimated using AMC II conditions, Group B soil (Per SSURGO). Land Cover was estimated from a recent aerial and was mostly timber and golf course. The pervious composite curve number was estimated to be 66, and 1% of the basin is assumed to be directly connected impervious area. Basin lag time of 2.8 hours was estimated by assumptions about sheet, shallow concentrated and concentrated flow paths in the upper basin. Slopes were estimated from USGS contours. Channel geometry and shape were assumed because access to the upper basin was not permitted, nor were any measurements taken upstream. No reach routing was included in the HMS model. Study results are presented in Figure 2 and summarized in Table 2.

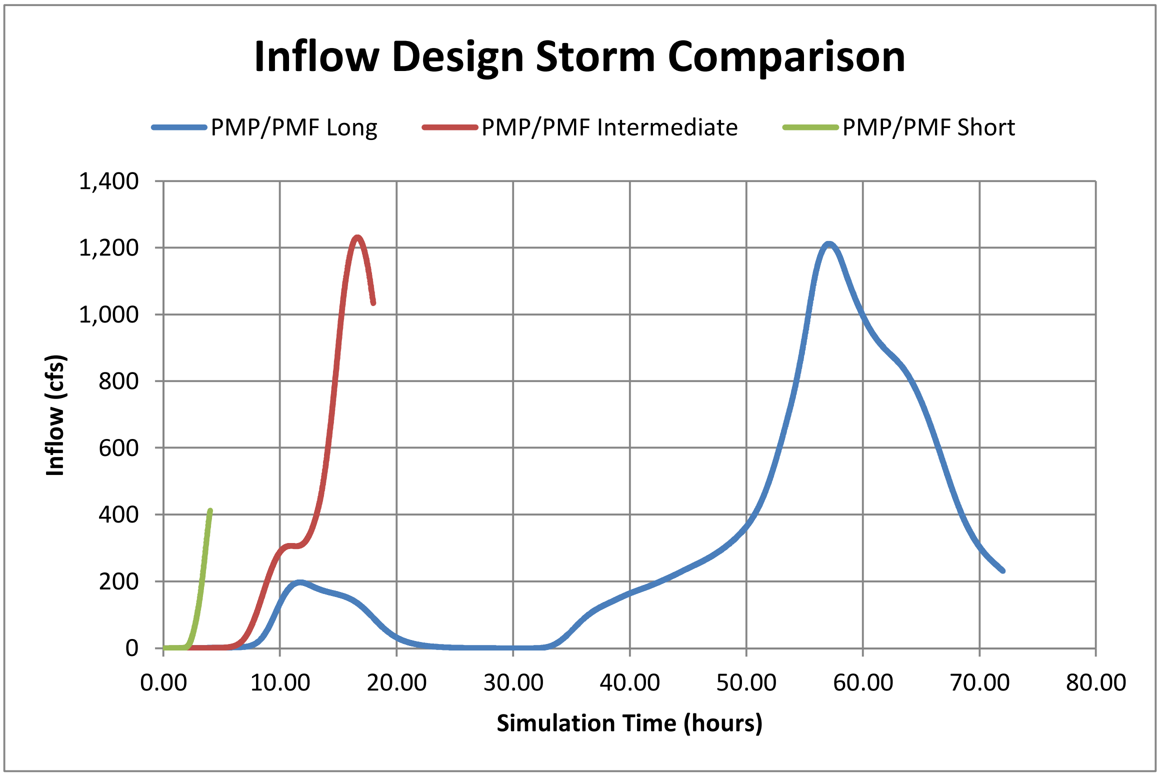


Figure 2. Comparison of Short, Intermediate and Long Duration PMP/PMF Inflows Developed from DSO Tech Note 3 and HEC-HMS

Table 2. Summary of Peak Inflow Results

Flood Condition	Storm Duration	Peak Flow (cfs)	Total Volume (acre-feet)
PMF	Short	538	25.7
	Intermediate	1,230	501
	Long	1,211	1,704
100-Year	Short	86	3.6
	Intermediate	486	168
	Long	499	654

For comparison purposes, the peak flow associated with the previous FEMA flood study of the watershed are shown in Table 3. The current study results are approximately 50 percent larger than the FEMA results at the 100-year flow level. This would indicate that parameters used in the HEC-HMS model of the watershed are likely to be conservative. Similarly, this indicates that the PMF results are likely conservative.

Table 3. Summary of FEMA Peak Flows (not used)

FEMA Recurrence Interval	Peak Flow (cfs)
10-Year	144
100-Year	331
500-Year	530

3.1.2 Flood Reservoir Routing and Spillway Evaluation

The runoff from the inflow analysis described in the preceding section was directly routed through the reservoir behind the replacement dam. Peak water surface elevations for the long duration PMF and the 100-Year flood were calculated using the HEC-HMS hydrologic model. Peak water surfaces were calculated assuming the initial water surface in the reservoir pool was 33.74 ft (full pool), the slide gates were closed, and the flashboards installed. The proposed spillway was modeled with an effective weir length of 53.25 ft and invert elevation of 33.74 feet. A weir coefficient (Cd) of 3.33 was assumed. An existing stage-storage curve was generated for Mill Creek Pond using survey and LiDAR Data (Figure 3). Results from the flood reservoir routing and spillway evaluation are summarized in Table 4.

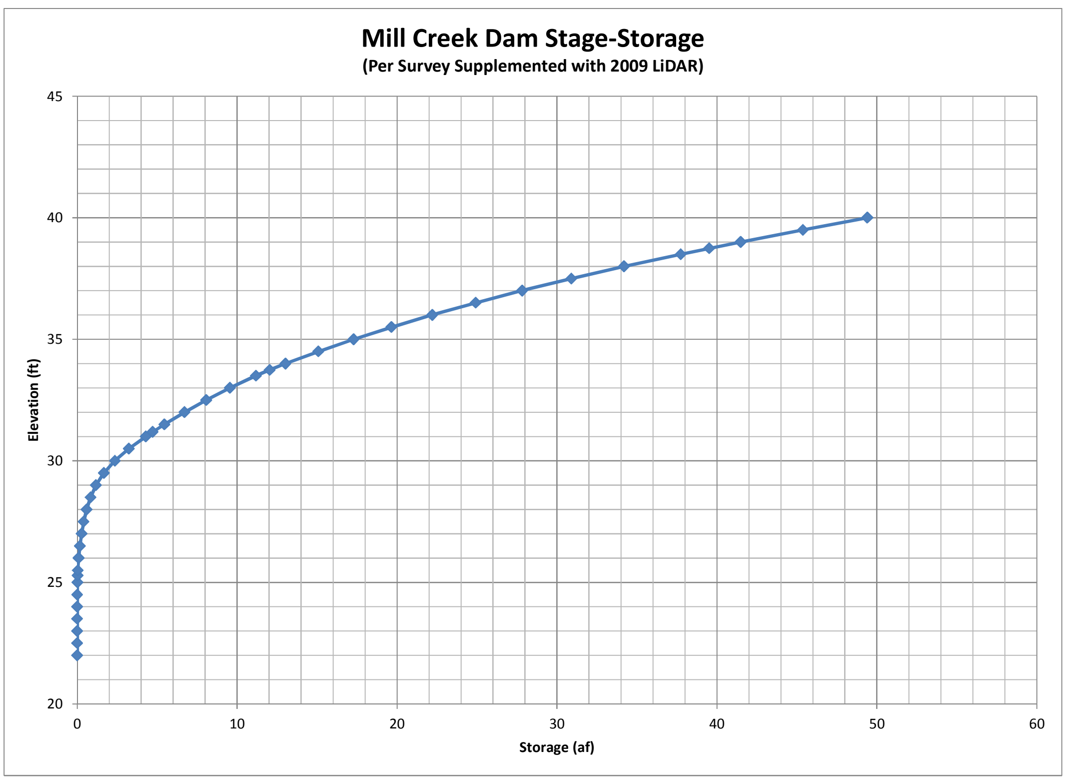


Figure 3. Existing Stage-Storage Curve for Mill Creek Pond.

Table 4. Summary of Flood Routing and Spillway Evaluation

Scenario	Inflow (cfs)	Outflow (cfs)	Reservoir Elevation (Feet, NAVD 88)	Reservoir Storage (Acre-Feet)
PMF (Long Duration)	1,211	1,211	37.3	29.9
100-Year (Long Duration)	499	499	35.7	20.8

The spillway routing results show that the maximum water surface elevation during the 100-year flood is 35.7 feet (2.0 feet above the dam crest), and the maximum water surface associated with the PMF is 37.3 feet (3.6 feet above the dam crest).

3.2 Design Step Determination

Identification of DSO requirements depends on working through a decision framework process to determine the necessary design step. The design steps range from Step 1 to Step 8 (Figure 4) with increasingly more stringent requirements to satisfy at the higher steps. Design Step 1 is applicable where the downstream consequences of failure would be minimal and there would be no potential for loss of life. Design Step 8 is applicable where the consequences of dam failure could be catastrophic with hundreds of lives at risk. Design Step 8 utilizes extreme design events and design loads to provide the extremely high levels of reliability needed to properly protect the public. Determination of the appropriate step depends on the results of a dam break inundation analysis and the appropriate downstream flood hazard category, which are discussed below. This project has been evaluated under an assumed design step of 8.

1/500 AEP 1	2	3	4	5	6	7	8 Theoretical Max. Event
D E S I G N S T E P							

10^{-3} 10^{-4} 10^{-5} 10^{-6}

Design Performance Goal – Annual Exceedance Probability

Figure 4. Design Step Format for Design and Performance Goals

3.2.1 Dam Break Inundation Analysis

A dam break inundation analysis includes the estimation of the dam break outflow hydrograph, routing the dam break hydrograph through the downstream creek channel and estimation of the inundation levels and damages to downstream structures.

A HEC-HMS model was developed to estimate the dam break hydrograph. Figures 5 and 6 provide the location and layout of the proposed dam structure. The following assumptions were made regarding the breach geometry:

- Breach Width: 40 ft
- Breach Side slopes: Vertical
- Time of Breach: 0.3 hr
- Breach coincides with the peak inflow of the 100-year flood event (Long Duration Storm)
- Reservoir Pool Elevation: 33.74 ft (Full Pool)
- All gates closed, flashboards installed

The dam break analysis estimates a peak discharge of 1,835 cfs at the dam, resulting from a maximum pond water surface elevation of 35.7 ft (approximately 2 feet above the dam crest) and a maximum storage of 20.8 acre-feet (Figure 7).

Simplified methods recommended by DSO Technical Note 1 were applied to route the dam break outflow hydrograph downstream and estimate the extent of inundation. Attenuated peak discharge downstream of the dam break was estimated using Figure 5a from DSO Tech Note 1 (Figure 8). Mill Creek Dam is approximately 1 mile upstream from the confluence with Grays Harbor, so the reduction in peak flow is relatively small[1].

Velocity for use in the inundation analysis was determined from Table 7 from DSO Tech Note 1 (Shown in Table 5). The estimated bed slope was 22 ft/mile and was derived from LiDAR. It was assumed Mill Creek is a Type 1 Main Channel. The resulting assumed velocity was 5 ft/sec.

A steady state 1-D HEC-RAS model was created to estimate the water surface elevations associated with the dam break. Cross sections were derived from LiDAR and extend from the Dam downstream to J Street. A constant flow of 1,800 cfs was used in the model (approximately the peak flow determined from the dam break analysis). The downstream boundary condition was set at the FEMA 100-YR Base Flood Elevation (BFE) of 12 ft. The BFE is influenced by the tidal conditions present in Grays Harbor. The approximate extent of inundation is shown in Figure 9. A summary of the dam break inundation results is provided in Table 6. The inundation is likely to be somewhat conservative given the short duration of the peak flow.

[1] Because the storage volume in the reservoir (approximately 20.8 acre-feet during the 100-year flood) is very small compared with the peak outflow, the actual peak flow attenuation associated with the downstream routing of the outflow hydrograph would be expected to be very high. This is further demonstrated in that the volume of inundation shown on Figure 9 is approximately 74 acre-feet. Using DSO Figure 5a attenuation, and steady flow backwater analysis will tend to make the inundation results conservative.

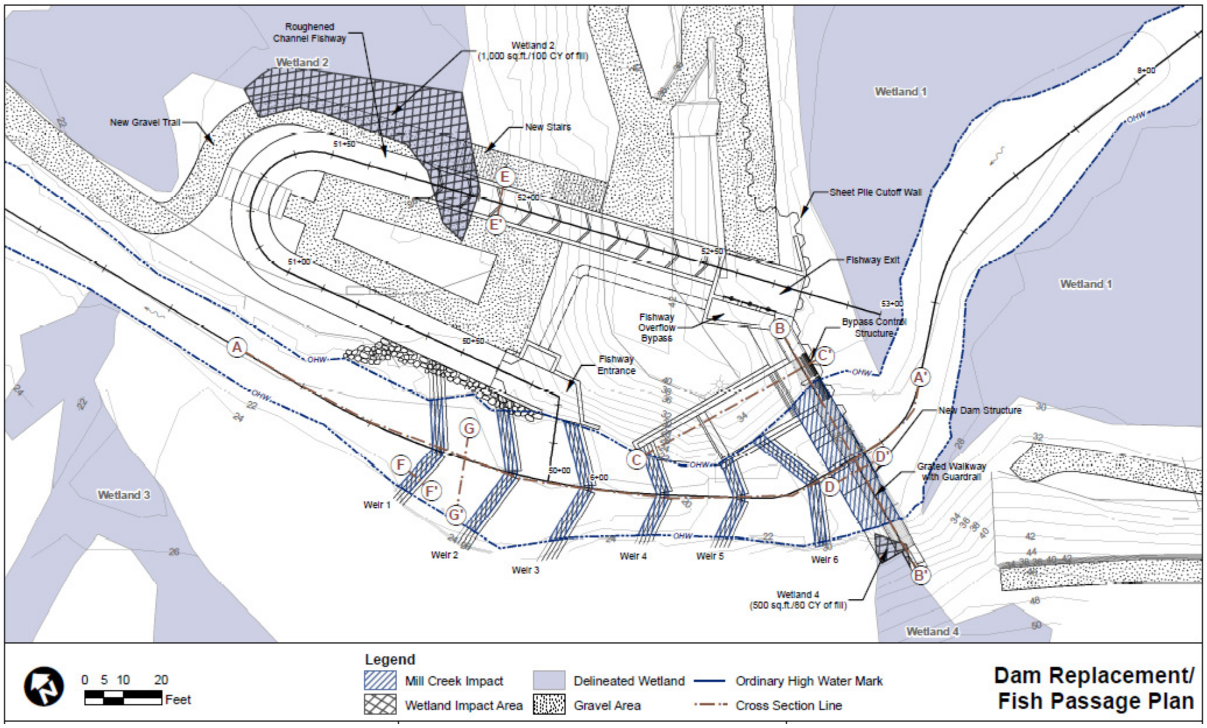


Figure 5. Proposed Dam and Fish Ladder Layout

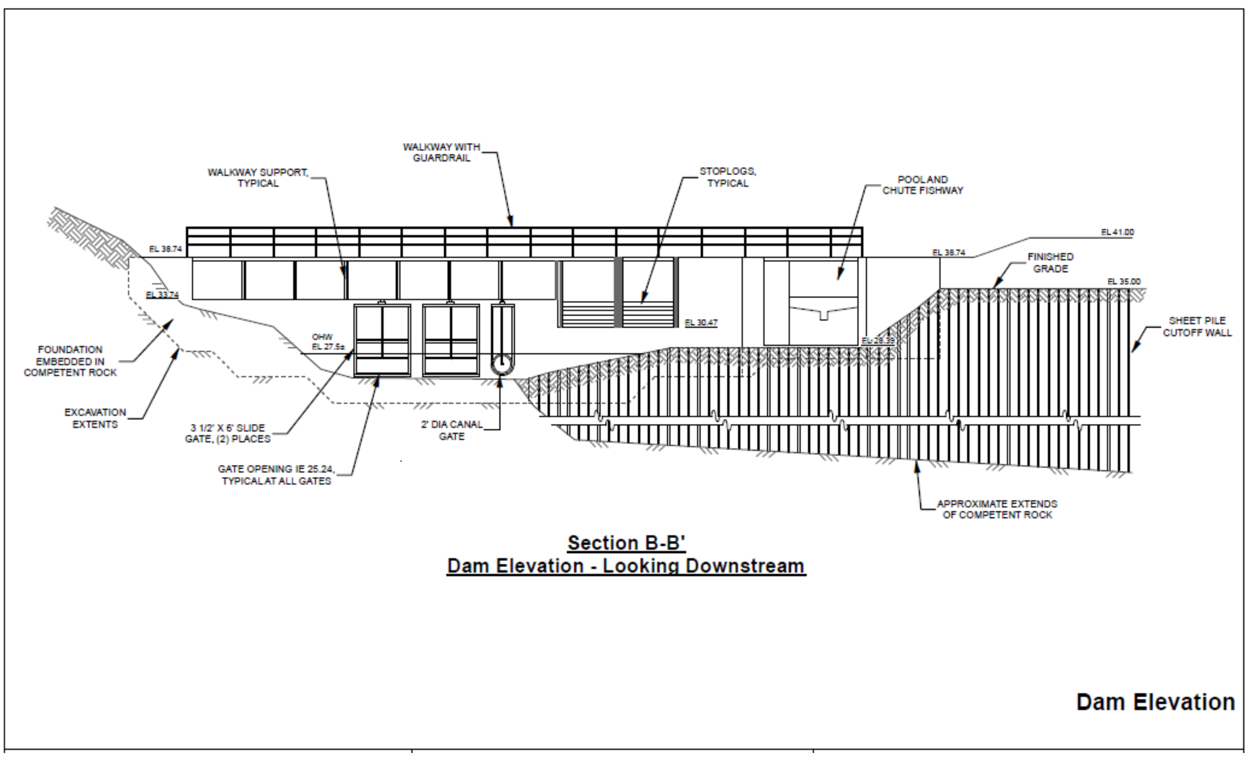


Figure 6. Proposed Dam Elevation

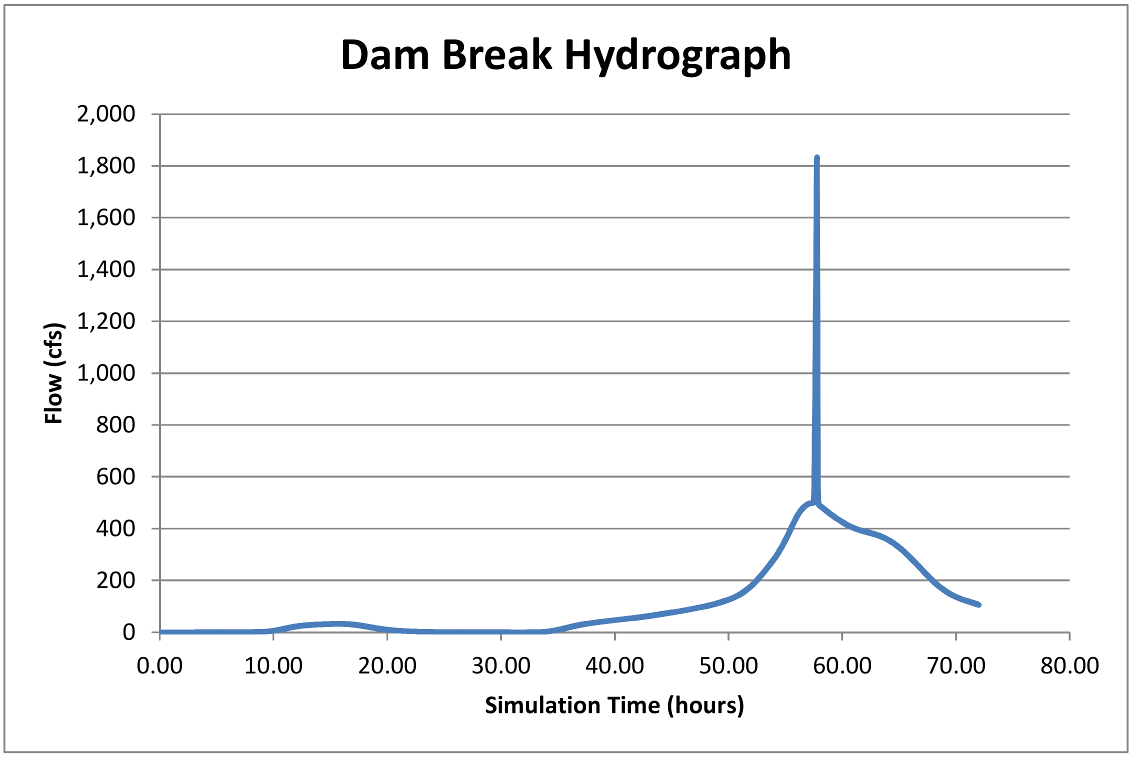


Figure 7. Simulated Dam Break Hydrograph from HEC-HMS at Full Pool during 100-YR Long Duration Storm

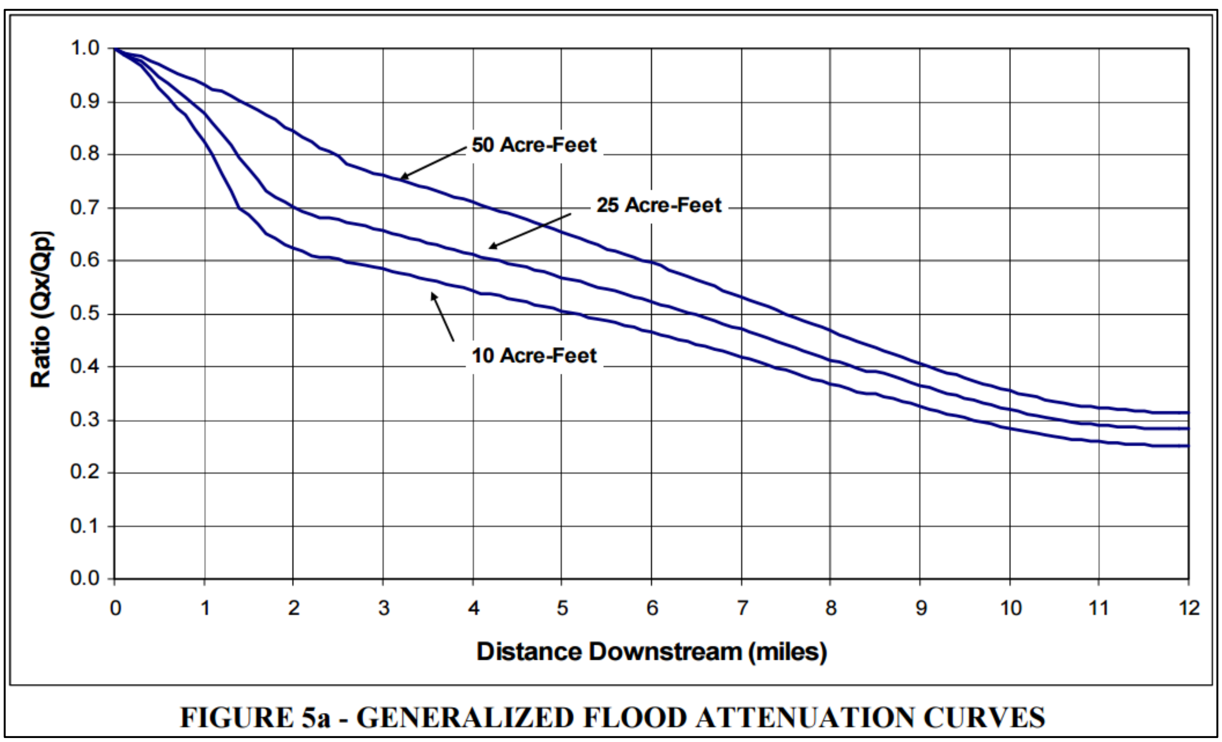


Figure 8. Figure 5a from DSO Tech Note 1 Used for Downstream Routing (25 Acre-Feet Curve).

Table 5. Table 7 from DSO Tech Note 1 Used to Determine Representative Velocity for Estimating Inundation from Dam Break Floods.

TABLE 7 - REPRESENTATIVE VELOCITIES FOR USE IN ESTIMATING INUNDATION FROM DAM BREAK FLOODS

TYPE 1 MAIN CHANNEL - GRAVEL OVERBANKS - GRASS, PASTURE		TYPE 2 MAIN CHANNEL - GRAVEL, COBBLES OVERBANKS - IRREGULAR, BRUSH, SCATTERED SHRUBS		TYPE 3 MAIN CHANNEL GRAVEL COBBLES, BOULDERS OVERBANKS WOODED	
BEDSLOPE (ft/mi)	VELOCITY (ft/sec)	BEDSLOPE (ft/mi)	VELOCITY (ft/sec)	BEDSLOPE (ft/ml)	VELOCITY (ft/sec)
5	2.4	5	1.7	5	1.4
10	3.4	10	2.4	10	1.9
15	4.1	15	3.0	15	2.4
20	4.8	20	3.5	20	2.7
30	5.8	30	4.2	30	3.3
40	6.7	40	4.9	40	3.8
60	8.2	60	6.0	60	4.7
80	9.5	80	6.9	80	5.4
100	10.6	100	7.7	100	6.1
200	12.0	200	10.9	200	8.6
300	12.0	300	12.0	300	10.5
400 or greater	12.0	400 or greater	12.0	400 or greater	12.0

Table 6. Summary of Dam Break Routing and Inundation Results

Location	Distance From Dam (miles)	Qx/Qp	Qx (cfs)	Velocity (ft/s)	Flow Area = Q\V (sq ft.)	Approximate Water Surface Elevation (ft, NAVD 88)	Approximate Cumulative Upstream Volume (acre-feet)
Dam	0	1.00	1,835		367	30.1	-
C Street	0.14	0.99	1,817		363	27.0	11.3
D Street	0.22	0.98	1,798		360	23.0	17.5
E Street	0.28	0.97	1,780		356	21.3	23.9
F Street	0.37	0.96	1,762	5	352	18.6	35.3
G Street	0.44	0.96	1,762		352	16.8	45.4
H Street	0.50	0.95	1,743		349	14.7	54.6
I Street	0.57	0.94	1,725		345	13.8	65.4
J Street	0.63	0.93	1,707		341	12.0	74.0

Note: Water surface elevations and volumes were estimated for a flow rate of 1,835 cfs using HEC-RAS.

Figure 9. Preliminary Dam Break Inundation Area assuming failure during the 100-Year Long Duration Storm.

3.2.2 Downstream Hazard Classification

DSO Tech Note 1 presents a methodology for estimating the hazard classification of existing and proposed dams. DSO defines downstream hazard as "the potential loss of life or property damage downstream of a dam from floodwaters released at the dam or waters released by partial or complete failure of the dam". The Mill Creek Replacement Dam will not have the flexibility to release floodwaters in excess of the routed inflow to the reservoir; therefore the only potential hazard is from a dam failure.

"Downstream Hazard Classification does not correspond to the condition of the dam or appurtenant works, nor the anticipated performance or operation of the dam. Rather, it is descriptive of the setting in areas downstream of the dam and is an index of the relative magnitude of the potential consequences to human life and development should a particular dam fail." (DSO)

DSO estimates the downstream hazard based on the following factors:

1. Population at Risk
2. Economic Loss
3. Environmental Damages

Population at Risk
DSO guidance suggests that an inundation depth of 1 foot or more at a given dwelling, worksite or temporary use area can indicate a hazard to life. With regard to estimating the population at risk below a given dam, it is common practice to use a value of 3 persons per inhabited dwelling. A population of seven or more individuals is categorized as a High hazard potential.

Economic Loss
DSO guidance suggests that three to ten inhabited structures, with some industry and work sites would designate a High hazard classification with respect to economic loss. Also, an effect on primary highways and rail lines would be reason for designating a dam as High hazard.

Environmental Damages
Water quality degradation of more than a limited amount due to a potential dam failure would result in a dam being classified as High hazard. Environmental damages could result from the failure of a dam where the reservoir contains materials which may be harmful to human or aquatic life or stream habitat. This category also includes streams with fisheries of regional significance where channel scour and sediment deposition are likely to result from a dam break flood.

The area immediately downstream of the Mill Creek Replacement Dam is City park, with no significant inhabited structures. Approximately 600 feet downstream of the dam, Mill Creek crosses under C Street, and enters an urbanized area of single family houses for the next one-half mile. There are between 35 and 40 houses on the immediate bank of Mill Creek in this reach. For the next 0.4 mile, the channel flows through essentially undeveloped land, although there are industrial properties in relatively close proximity. Approximately 1.1 miles downstream of the dam, Mill Creek crosses US Highway 101 and a railway line, and then enters the Chehalis River, within the Grays Harbor estuary.

A review of the dam failure inundation area estimated in Section 1 indicates that dozens of structures could potentially be inundated to a depth of 1 foot or more following an assumed worst-case failure of the Mill Creek Replacement Dam. A dam failure during a 100-year flood would likely endanger more than 30 and less than 300 people. Additionally, a dam failure could potentially affect US Highway 101, and the nearby railroad. Industrial and commercial properties also likely exist within the potential inundation area. It is estimated that the chance of significant environmental damage is relatively small.

Overall, it is estimated that the Mill Creek Replacement Dam would have a Downstream Hazard Classification of High - 1B.

4.0 Future Phases

The following refinements and additional work are recommended as part of the final design of the replacement dam.

- Develop a 2D HEC-RAS Model to provide more accurate routing of the dam break outflow hydrograph resulting downstream inundation
- Obtain additional channel survey to refine hydraulic analysis
- Analyze stream gage data to develop typical monthly operations
- Review currently draft FEMA maps, assuming they have been approved in the interim
- Provide an analysis of flood routing when the pond is managed for flood hazard reduction

5.0 Limitations

The analysis and results presented above are preliminary, commensurate with a Conceptual Level design analysis. Specific aspects will need revision as the design progresses and additional information becomes available. The hydrologic and hydraulic calculations used approximate methods, as recommended by DSO guidance.

Appendix D

Fish Passage Technical Memorandum

Technical Memorandum

Date:	Tuesday, May 05, 2015
Project:	Mill Creek Dam Improvements Project
To:	Darrin Raines, Director of Public Works, City of Cosmopolis
From:	Tim Hume, Project Manager, HDR
Prepared by:	Shaun Bevan, HDR
Reviewed by	Mike Garello, HDR
Subject:	Fish Passage Criteria and Assessment of Alternatives

Introduction

The City of Cosmopolis has retained the services of HDR to investigate opportunities to restore the flood attenuation and recreational values of Mill Creek Dam. As part of this investigation, it was recognized that fish passage facilities may be required to restore and maintain upstream and downstream fish passage for fish species potentially present in the lower reaches of Mill Creek.

Purpose and Objective of this Technical Memorandum

The objective of this Technical Memorandum (TM) is to perform a preliminary assessment of fish passage options for the overall Mill Creek Dam Improvement Project and select an alternative for further design and incorporation. This TM also provides key design considerations and criteria that can be used as the initial framework for further design development and preparation of construction documents.

Fisheries Resources

Prior to the dam breach in 2008, Mill Creek pond upstream of the dam was used as a recreational fishing area for youth. The pond was stocked seasonally with rainbow trout. Since the dam breach, the extent to which remnant rainbow trout may potentially still inhabit the creek is unknown.

The only salmonid species documented to inhabit Mill Creek is coho salmon (*Onchorhynchus kisutch*) (Smith and Wegner 2001; WDFW 2014). Coho typically spend 1 to 2 years rearing in freshwater before migrating out to the ocean in the spring between March and June. During this time they inhabit pools and areas with cover from woody debris or other structure. In winter, coho juveniles can move both upstream and downstream into pools and off-channel areas.

Coho remain in the ocean for 1 to 2 years before returning to spawn from November through January and sometimes into February (Smith and Wegner 2001). The young hatch in about 6 to 8 weeks, and emerge from the redd 2 to 3 weeks after that (Wydoski and Whitney 2003). The juvenile outmigration occurs in the spring from March through June. Coho are highly tolerant of degraded habitat and are commonly found in residential areas and streams channeled through ditches (Wydoski and Whitney 2003).

Prior to the breach of the dam, the dam posed an impassible barrier and marked the upstream extent of coho presence. The stream channel within Mill Creek park between the site of the dam and C street contains habitat suitable for spawning, and several redds have been located in this reach.

Selected Species for Design

The targeted fish species that will be used to establish fish passage design criteria for the project is primarily coho, but fish passage structures will also be designed to accommodate steelhead, coastal cutthroat, and bull trout. General characteristics of targeted fish species are presented in Table 1. The swim speeds of fish species are broken into three categories: sustained, prolonged, and burst. Sustained speeds can be maintained for long periods of time, on the order of hours, prolonged speeds can be maintained for minutes, and bursts speeds are a single effort maintained for 5 to 10 seconds.

Table 1. General characteristics of selected species occurring in Mill Creek (Bell 1991).

Targeted Fish Species	General Characteristics
Coho Salmon	• Typical weight range 5 to 20 lbs • Spend 2 years in the ocean • Reach maturity at 3 years • Adults have burst swimming speeds of 11 to 21 ft/s, prolonged speeds of 4 to 11 ft/s, and sustained speeds of 0 to 4 ft/s.
Steelhead	• Typical weight range 5 to 30 lbs • Spend 1 to 4 years in the ocean • Reach maturity at 3 to 6 years • Adults have burst swimming speeds of 14.5 to 26.5 ft/s, prolonged speeds of 5 to 14.5 ft/s, and sustained speeds of 0 to 5 ft/s
Coastal Cutthroat	• Typical weight range for sea-run 0.5 to 4 lbs and 0.25 to 5 lbs for resident • Sea-run spend 0.5 to 1 years in the ocean • Sea-run reach maturity at 2 to 5 years and 3 to 4 for residents • Adults have burst swimming speeds of 6 to 13.5 ft/s, prolonged speeds of 2.5 to 6 ft/s, and sustained speeds of 0 to 2.5 ft/s
Bull Trout	• Typical occurrence and physical parameters for bull trout in the Chehalis River are currently being evaluated by others and will be incorporated as the information becomes available. • Bull trout have been known to exhibit both fluvial and ad-fluvial migration behaviors depending on basin characteristics. • Very little information is available pertaining to the swimming capabilities of bull trout. Adults have documented "sprint" at speeds of 4 to 7 ft/s (Mesa et al. 2008).

Fish Passage Design Flows

Upstream fish passage assessments and designs at impediments typically use fisheries resource agencies' guidelines for determining a range of fish passage flows. The design objective is to provide suitable hydraulic conditions over a range of reasonable streamflows that the target fish are expected to migrate upstream. This range of streamflows is defined by the low and high fish passage flows.

Fish passage design flow criteria are most often based on exceedance calculations of daily mean flows. The exceedance flows statistically represent the flow equaled or exceeded certain percentages of the time.

NMFS (2011) requires the high fish passage design flow to be the mean daily average streamflow that is exceeded 5% of the time during periods when migrating fish are typically present, while WDFW (2000) suggests 10% exceedance flow. NMFS (2011) requires a low fish passage design flow equal to the mean daily average streamflow that is exceeded 95% of the time during periods when migrating fish are typically present. WDFW does not have a specific low flow guideline.

The fish passage facilities will be designed to facilitate conditions which promote passage throughout the range of anticipated migration flows: the lowest of the low fish passage design flows through the highest of the high fish passage design flows which represents the range of targeted fish species and life stages. The resulting low fish passage design flow is 1 cfs and the high fish passage design flow is approximately 115 cfs. The facility will need to meet fish passage design criteria throughout this range of flows. Once flows exceed the high fish passage design flow or are below the low fish passage design flow, compliance with fish passage criteria is typically not expected by the agencies.

Fish Passage Design Criteria

This section identifies specific design criteria or references specific sources of design criteria relevant to the development of fish passage concepts and designs. Specific criteria relative to facility hydraulics must be met to assure compliance with regulatory requirements and are typically only applied during detailed design and are only generally considered here for the basic identification and development of potential fish passage options. Fisheries criteria are typically guidelines providing a range of values or, in some instances, a specific value for design that should be met but can also be adjusted in combination with other criteria in light of site-specific conditions. Site-specific biological and physical rationale for not meeting criteria may be required and different values that support better performance or solve site-specific issues may be necessary during the development of the design. The list of criteria following the next paragraph is not intended to be an all-inclusive list of criteria for design, but is used to guide option and alternative formulation.

Note that the terms "entrance" and "exit" are used for notation of fish passage features in reference to the direction that a fish is travelling. In the case of fish ladders, because they are typically designed for adult upstream passage, the fish ladder entrance refers to the downstream end or hydraulic outlet, and the fish ladder exit refers to the upstream end or hydraulic inlet. In the case of fish bypass features, because they are typically designed for juvenile (and sometimes post-spawn adults) downstream passage, the bypass entrance refers to the upstream end or hydraulic inlet, and the bypass exit refers to the downstream end or hydraulic outlet. Terminology in the following criteria subsections follows standard terminology for fish ladders (upstream passage) and fish bypasses (downstream passage), depending upon which type of passage facility is being addressed.

Fish Bypass Criteria

Bypass systems are designed to facilitate both juvenile and adult fish downstream passage back to the river system, typically around a diversion or fish screen system, in a manner that minimizes risk of injury and delay. Fish bypass systems typically contain three major components; the bypass entrance, conduit, and exit.

Bypass Entrance Criteria

- Flow Control – Independent flow control should be provided at each bypass entrance (NMFS 2011).
- Travel Time – Fish are to enter a bypass within 60 seconds of exposure to any length of screen (NMFS 2011 and WDFW 2000a).
- Velocity – Bypass entrance velocity must be greater than 110% of the maximum screen-sweeping velocity. Velocity should not decrease between the screen terminus and bypass entrance and should accelerate gradually (NMFS 2011 and WDFW 2000a).
- Acceleration – The flow should not decelerate and should not exceed an acceleration rate of 0.2 feet per second per foot of travel (NMFS 2011).
- Lighting – Ambient lighting is required at the entrance to the bypass flow control (NMFS 2011).
- Dimensions – Bypass entrance should be a minimum of 18 inches wide and its height must extend from floor of the screen to water surface (NMFS 2011 and WDFW 2000a). For weirs used in bypass systems that have diversions greater than 25 cfs, a minimum weir depth of 1 foot should be maintained throughout the smolt out-migration period (NMFS 2011).
- Juvenile Capture Velocity – A minimum velocity of 8 ft/s is a common design threshold used in situations that require the capture of juvenile salmonids. Experience with current projects will be considered if a bypass system becomes part of the facility design.

Bypass Conduit Criteria

- Materials and fittings – Smooth pipes, joints and other interior surfaces are required to minimize turbulence and the potential for fish injury. Closure valves should not be used within the bypass pipe (NMFS 2011 and WDFW 2000a).
- Flow Transitions – Pumping if fish are within the bypass system is not allowed. If site conditions permit, bypass flows should be open channel (NMFS 2011 and WDFW 2000a). Where site conditions don’t permit open channel bypass flows, a bypass pipe may be used. WDFW criteria states that bypass pipes should not be pressurized, while NMFS criteria states that pressures within bypass pipes must be equal to or above atmospheric pressure. NMFS criteria also states that transitions from pressurized to non-pressurized (or vice-versa)

should be avoided within the pipe. Free-fall of fish within a pipe or enclosed conduit within the bypass system is not allowed (NFMS 2011).

- Bypass Flow – Bypass flow should be approximately 5% of the total screened flow (NMFS 2011). Based on professional judgment, this proportion may be considered a minimum. Higher bypass flow proportions will be considered if a bypass is included in the design.
- Velocity – NMFS criteria states the bypass pipe should be designed to have velocities between 6 and 12 ft/s, however higher velocities can be approved with special attention to pipe and joint smoothness (NMFS 2011). WDFW requires bypass pipe velocity to not exceed 30 ft/s (WDFW 2000a).
- Geometry – NMFS only requires the open channel or pipe diameter to be sized based on bypass flow and slope in order to meet other bypass conduit criteria. WDFW states the bypass diameter should be a minimum of 2 feet (WDFW 2000a).
- Bends – The ratio of bypass centerline to pipe diameter must be 5 or greater, and larger ratios may be required for super-critical velocities (NMFS 2011 and WDFW 2000a).
- Depth – NMFS criteria requires a minimum depth of at least 40% of the bypass pipe diameter, unless otherwise approved (NMFS 2011). WDFW states that depth should be maintained at 9 inches or greater (WDFW 2000a).
- Hydraulic Jump – Hydraulic jumps should not occur within the pipe (NMFS 2011 and WDFW 2000a).

Bypass Exit Criteria

- Velocity – The outfall impact velocity, the velocity of the bypass flow entering the river, should not exceed 25 ft/s (NFMS 2011 and WDFW 2000a).
- Location – The outfall should be located in an area with strong downstream currents, at least 4 ft/s, free of eddies, reverse flow, or likely predator habitat. The outfall should also be located in an area with sufficient depth to avoid fish injuries (NMFS 2011 and WDFW 2000a).
- Adult Attraction – The bypass outfall must be designed to avoid the attraction of upstream migrants. Upstream migrants might leap at the outfall, therefore provisions for minimizing risk to injury or stranding on the bank must be included in the outfall design (NMFS 2011 and WDFW 2000a).

Fishway Criteria

Upstream fish passage designs at dams use widely recognized fishway design guidelines and references and are traditionally designed for the adult fish life stage. There are three major components to a fishway: the fishway entrance, fish ladder, and fishway exit. The fishway entrance's primary objective is to maximize fish attraction. The fish ladder's primary objective is to provide hydraulic conditions that promote fish passage up and around a passage barrier like a dam. The fishway exit's primary function is to maintain hydraulic conditions suitable for fish passage for the range of forebay or reservoir water surface elevations. The design criteria specific to each component are presented below.

Fishway Entrance

- Entrance Location – The entrance located should be based on site-specific operations and streamflow characteristics. Entrances must be placed in locations where fish can easily locate the attraction flow. Multiple entrances may be required if the site has multiple locations where fish hold (NMFS 2011 and WDFW 2000b).
- Entrance Geometry – The entrance should have a minimum width of 4 feet and depth of 6 feet (NMFS 2011)
- Entrance Head Differential– The head differential at the entrance should be maintained between 1.0 and 1.5 feet (NMFS 2011 and WDFW 2000b).
- Attraction Flow – Minimum 5% to 10% of high fish passage design flow (NMFS 2011). WDFW has no specific fishway attraction flow criteria but states that flow must be adequate to compete with spillway or powerhouse flows for attraction of fish. Auxiliary water systems may be used to increase the fishway entrance attraction flow.

Fish Ladder Design

- Head Differential – The hydraulic drop between each pool within the fish ladder must be a maximum of 1 foot (NMFS 2011 and WDFW 2000b).
- Minimum Pool Dimensions – Minimum of 8 feet long, 6 feet wide and 5 feet deep (NMFS 2011).
- Energy Dissipation Factor (EDF) – Each pool volume should be sized to have a maximum energy dissipation factor of 4 ft-lb/sec/ft^3. Only the volume of the pool having active flow and contributing to energy dissipation should be included in the energy dissipation calculation (NMFS 2011 and WDFW 2000b).
- Minimum Depth Over Weirs – Overflow weirs in fishways should have 1 foot of flow depth over weirs (NMFS 2011 and WDFW 2000b).
- Turning pools – Turning pools are required at each location where the fishway bends more than 90°. Turning pools should be at least double the length of the designed standard pool measured along the centerline (NMFS 2011). Bends should also be designed to eliminate upwelling in the corners (WDFW 2000b).
- Orifice Dimensions – NMFS criteria states orifices should be a minimum of 15 inches high and 12 inches wide (NMFS 2011). WDFW criteria recommend a minimum size of 18 inches high and 15 inches wide (WDFW 2000b).
- Freeboard – Freeboard must be a minimum of 3 feet within the fish ladder at the high design flow (NMFS 2011 and WDFW 2000b).
- Lighting – The use of ambient lighting throughout the entire fishway is preferred. Abrupt lighting changes within the fishway are not allowed (NMFS 2011 and WDFW 2000b).

Fishway Exit

- Head Differential – The fishway exit head differential should range from 0.25 to 1.0 feet (NMFS 2011). In order to accommodate forebay fluctuations this may require the use of adjustable weirs, multiple exits at different elevations, or other engineered solutions that accommodate forebay fluctuations.
- Length – A minimum channel length of two standard ladder pools should be incorporated upstream of the exit control (NMFS 2011).

- Location – The exit should be located along the shoreline at a location with similar depths to those within the fishway and with velocities less than 4.0 ft/s. Exits should be located well upstream of spillways, sluiceways, and powerhouses to minimize the risk of being swept downstream.
- Debris Rack – Coarse trash racks should be installed at the fishway exit and must be oriented at a deflection angle greater than 45° relative to the river flow (NMFS 2011 and WDFW 2000b).

Upstream Passage through Conduits and Channels Criteria

One of two typical design approaches may be considered when providing upstream passage through a conduit such as a tunnel, culvert or similar type structure: stream simulation or hydraulic design. In the stream simulation approach the conduit is placed at the same or similar slope as the natural stream and includes streambed material similar to or slightly coarser than the surrounding reach. Hydraulic design commonly includes the use of baffles, weirs, or roughened channels within the conduit. These two design approaches are summarized in greater detail below.

Stream Simulation Approach

The stream simulation approach is typically appropriate for systems with bank widths less than 15 feet but can be applied to larger systems. Using this type of approach, the inside width of a proposed conduit should be approximately equal to 1.2 times the bankfull width plus 2 feet. When the length-to-width ratio exceeds 10, the conduit width should be increased by approximately 30% (WDFW 2013). Such a conduit should be buried in the streambed 30-50% of its rise (WDFW 2013). Channels with slopes less than 4% should have a conduit bed with pool-riffle morphology, while channel slopes greater than 4% should have a conduit bed with cascade or step-pool morphology (WDFW 2013). The slope ratio (ratio of the conduit slope to the natural channel slope) must be less than 1.25 in order to utilize the stream simulation approach. If the slope ratio exceeds 1.25, the hydraulic design approach is required (WDFW 2013). Conduit slopes significantly different from the natural channel slope should also be avoided, as this can create a deposition or degradation zone.

Hydraulic Design Approach

The hydraulic design approach is most appropriate for exceptionally long conduits or conduits that exceed the maximum stream simulation slope ratio of 1.25. A minimum depth for a conduit without sediment should be 0.8 feet at the low fish passage design flow and there is no minimum depth requirement for conduits with natural bed sediment (WDFW 2013). The conduit should be designed to have maximum hydraulic drops of 0.8 feet with a maximum EDF of 5.0 ft-lb/ft^3-sec for baffles and less than 250 times the water surface slope for roughened channels (WDFW 2013).

Baffles are only typically used in exceptionally long conduits with a maximum recommended slope of 3.5% (WDFW 2013). A minimum of 5 feet of headroom should be provided for maintenance, and 6 feet for conduits in excess of 200 feet (WDFW 2013). Inspection and maintenance of a hydraulic design conduit is required on a regular basis to ensure criteria are met. WDFW (2013) suggests that conduits greater than 200 feet in length should have a maximum velocity of 2 ft/s, conduits between 100 and 200 feet long should have a maximum velocity of 3 ft/s, and conduits between 10 and 100 feet long should have a maximum velocity of 4 ft/s.

Debris Rack Criteria

Debris racks are commonly used to exclude large debris from entering fish passage facilities. Debris rack openings should be a minimum of 8 inches clear, or 12 inches clear if adult Chinook are present. NMFS criteria states that approach velocity should be less than 1.5 ft/s, while WDFW criteria states the maximum should be 2.0 ft/s. Debris racks should be sloped at 1:5 or flatter to assist with manual cleaning. In systems with coarse floating debris, debris booms or other provisions must be incorporated into the debris rack design (NMFS 2011 and WDFW 2000b).

Temporary/Interim Passage Facilities

An interim fish passage plan must be prepared and submitted to NMFS for approval if construction of an artificial impediment is scheduled during periods when migrating fish are present. Interim passage facilities must meet all regular facility criteria unless approved by NMFS (NMFS 2011).

Alternative Development

Concept level alternatives considered for evaluation at the proposed project location were developed by performing the following activities:

- Identification of applicable fish passage technologies (options),
- Evaluation of fish passage options and determination of fata flaw,
- Selection of options for further evaluation, and
- Formulation of alternatives using one or more fish passage options.

Potential Fish Passage Options

The following paragraphs provide a brief overview of potential fish passage options that were considered during alternative development.

Conventional Fish Ladder

A conventional fish ladder is typically a fabricated structure used to facilitate passage of fish over or around an obstacle, dam, or other migration barrier. Typically, fish ladders consist of a series of ascending pools that must be “climbed” or jumped by the fish. A series of pools contained within the water passage acts to incrementally divide the height of the passage and to dissipate the energy in the water, thereby enabling fish to gradually climb the height required to pass over the obstacle. The number of pools contained within the fish ladder depends on the climb required to pass over the obstacle and the vertical height between pools which is dependent upon fish swimming capability.

Although there are multiple variations, the three most common conventional fish ladders are pool and weir, baffle (denil, Alaskan steeppass, or other baffle configurations), and vertical-slot.

Additionally, two or more types of ladders can be used in combination to create a combination ladder.

Fishway guidelines for the State of Washington are presented in the document titled "Draft Fishway Guidelines for Washington State," prepared by WDFW (2000a). Although these guidelines were developed primarily for anadromous salmonids, many of the guidelines are applicable to resident fish species such as rainbow trout. Design criteria generally accepted by WDFW are included in the guideline document; however, design criteria and requirements for a specific site or facility must be verified directly with WDFW.

The type, size, and complexity of the fish ladder are largely formulated from four primary factors:

- The total vertical hydraulic differential that a fish will need to ascend or descend,
- The minimum and maximum flow that will be conveyed down the fish ladder,
- The swimming performance and condition of target fish species, and
- Fishway entrance and exit conditions.

In general, fishways require a narrow range of depth fluctuations in the fishway exit (upstream end) to operate successfully, typically less than 10 feet. The variation in forebay versus tailwater elevations is an important design element and limits success of various types of fishways. The greater the fluctuation observed, the more difficult it is to provide upstream passage successfully over the range of anticipated migration flows without a series of added appurtenances such as complex hydraulic controls or multiple exits.

Nature-Like Fishway

Nature-like fishways are designed to mimic steep natural channels with gradients that typically range from 1 to 5 percent. They provide a roughened series of profile control features that maintain multiple fish pathways throughout the range of anticipated design flows. Nature-like fishways can be configured in a number of ways but typically incorporate rock weirs, rock ramps, rock chutes, log weirs, and other features that mimic natural steep channels.

Design of a nature-like fishway requires careful analysis of bed substrate, scour, and sediment mobilization in order to determine the required size and gradation of imported bed substrate materials. Nature-like fishways can often be a viable fish passage alternative, especially for smaller passage impediments up to 5 to 10 vertical feet.

Trap and Haul

In general, the trap and haul option could include the collection, transfer, and release of fish in perpetuity of facility operations. These operations may be performed at a range of frequencies dependent upon the presence and migration tendencies of target fish species. Trap and haul facilities are not typically considered unless implementation of a conventional or nature-like fishway is identified as an impractical or unviable project option.

Trap and haul facilities typically include a channel-spanning passage barrier, ladder, and holding facilities. Fish migrating upstream can be collected at a system analogous to a short fish ladder leading to a collection pool from which fish are removed, or a picket barrier or fish weir placed at a suitable location downstream of the Mill Creek dam site. Picket weirs can be designed as permanent structures or to be installed and removed on a seasonal basis.

Costs for the trap and haul are composed of the initial investment in collection and transferring equipment, but are dominated more so by the ongoing operational costs required for continuing operations. Given the potential viability of a conventional or nature-like fishway at the project location as well as consideration of potential ongoing operations and maintenance costs, trap and haul facilities were not identified as a potential option for this project.

Fish Lifts

There were two basic types of mechanical lifts evaluated as potential fish passage options: fish locks and fish elevators (inclusive of fish pumps). A fish lock consists of holding chambers at the upstream and downstream faces of a dam linked by a sloping or vertical shaft which is filled with water when migrating fish enter the downstream chamber. The efficiency of such a fish facility depends mainly on the behavior of the fish and ability to attract fish to the entrance. Fish must remain in the downstream and central pool throughout the attraction phase, follow the rising water during the filling stage, and then swim upstream as the center lock empties. Fish elevators operate similarly where fish swim into a holding pool or hopper that closes prior to lifting. The hopper is then lifted like an elevator until it reaches a holding pen or flume where the fish are released upstream. Both options require adequate entrance conditions and control of attraction flow throughout operation.

Both types of fish lifts described above require substantial inputs of electrical power and a high level of effort to maintain and operate. Larinier (2007) reports that fish lifts suffer the following disadvantages compared to ladders: higher operating and maintenance costs, more chance of breaking down, and a higher risk of damage to fish. Fish lifts are not typically considered unless implementation of a conventional or nature-like fishway is identified as an impractical or unviable project option. Therefore, it was not considered for further evaluation and alternative development.

Reservoir Bypass

A reservoir bypass channel consists of an open channel or pipe that completely bypasses the hydraulic influence of the reservoir. The primary advantage of a reservoir bypass is that upstream and downstream passage is improved by eliminating the impact of potential forebay fluctuations and migration delays within the reservoir due to disorientation. For example, outmigrating fish tend to follow hydraulic streamlines moving downstream. Those streamlines and hydraulic patterns are lost when they encounter a large impoundment of water. Here, fish tend to lose their ability to orient themselves downstream to continue their migration. When routed around the impoundment, hydraulic streamlines are more consistent with downstream flow of water and therefore fish are able to continue their downstream migration without delay. In general, the upstream passage facility would exit at the downstream end of the bypass and a fish barrier would route fish to the bypass at the head of the reservoir.

Fish Passage Tunnel or Conduit through Dam

Fish passage could be provided through a tunnel or conduit specifically designed for fish passage through the dam. In this option, the river would freely pass under the dam and have hydraulics conducive to fish passage. In order to form a pool upstream of the dam, the fish passable conduits would have to be closed, effectively blocking fish passage through the conduits.

No Fish Passage

This option includes the implementation of a new dam without any fish passage facilities. The presence of the dam would block access of any fish species from migrating upstream. Likely, resident fish remaining upstream could migrate downstream over the dam during high flow events.

Summary of Options Not Selected for Further Evaluation

Upon cross-comparison of each potential option with the site-specific characteristics, a number of potential fish passage options were not selected for further consideration and alternative formulation. These options and the basis of why they were not selected are presented in Table 2.

Table 2. Summary of options not selected for further evaluation.

Option	Reason Not Selected
Trap and Haul Facility	• High long term O&M costs, • Requires special permits and training for fish trapping and handling, • Likely not acceptable by resource agencies given the viability of other passage options, and • Volitional passage is highly preferred over trap and haul when practical.
Fish Lift Facility	• High capital cost. • Likely not acceptable by resource agencies given the viability of other passage options for this project. • Volitional passage is highly preferred over trap and haul when practical.
No Fish Passage	• Fish passage required by WDFW.

Summary of Concept Level Alternatives

Several concept level alternatives were formulated using the fish passage options selected for further evaluation. A summary of each concept is provided in the following sections.

Alternative A – Conventional Fish Ladder

Alternative A includes a conventional fish ladder from the base of the dam to the reservoir forebay. Conventional fish ladders are one of the most common fish passage technologies used for dams of this size. The fish ladder would provide passage over a hydraulic differential of approximately 10 feet from the fish entrance to the fish exit.

The fish ladder would include a fishway entrance, pool and weir or pool and chute, and fishway exit. Assuming a pool size of 8 feet long, a hydraulic drop over each weir of between 0.5 and 1 feet, a fishway entrance of 20 feet and fishway exit of 20 feet, the fish ladder would be approximately 120 to 200 feet long (depending on the design hydraulic drop at each weir).

This alternative would include design and construction of the following primary project elements:

- Fish ladder entrance near base of new dam structure.
- Pool and chute or pool and weir fish ladder to reservoir.
- Fish ladder exit designed for typical full pool operating conditions.
- No fish passage provisions provided for low pool operation conditions.

One of the primary limitations of this alternative is its inability to handle a range of pool conditions and the complete inability to provide fish passage during times that no pool exists.

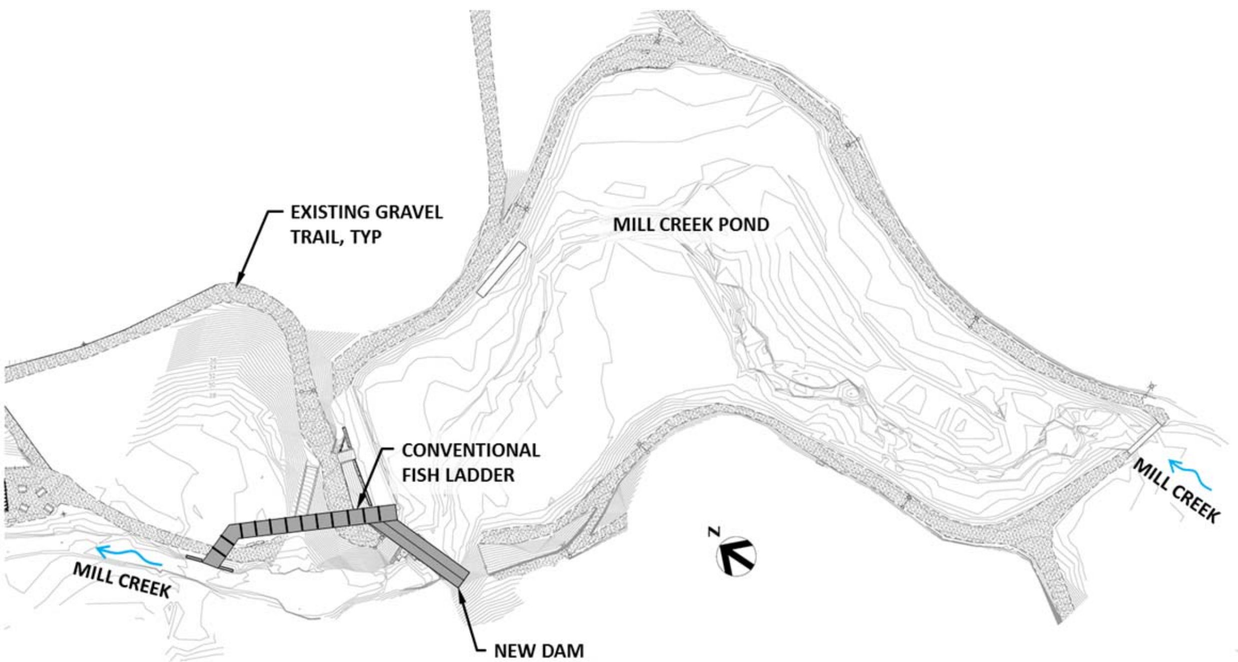


Figure 1. Concept schematic of Alternative A.

Alternative B – Nature-Like Fishway

Alternative B is similar to Alternative A, in that it provides fish passage from the base of the dam to the reservoir forebay. However, Alternative A includes a nature-like fishway instead of a conventional fishway to provide fish passage over a hydraulic differential of approximately 10 feet from the dam base to the forebay.

Nature-like fishways are used primarily at smaller fish passage barriers because they are longer than conventional fish ladders. They are typically designed with a slope of 1% to 5%, which would results in a fishway length of between 200 and 1,000 feet.

This alternative would include design and construction of the following primary project elements:

- Fishway entrance near base of dam structure.
- Nature-like fishway to reservoir.
- Fishway exit (hydraulic inlet) designed to divert the appropriate flow down the fishway at the full pool operating conditions.
- No fish passage provisions provided for low pool operation conditions.

Similar to Alternative A, the two primary limitations of Alternative B are its inability to handle a range of pool conditions and the complete inability to provide fish passage during times that no pool exists. An additional limitation is the potential size (length) of the nature-like fishway and the difficulty of fitting the nature-like fishway within the site geometrically.

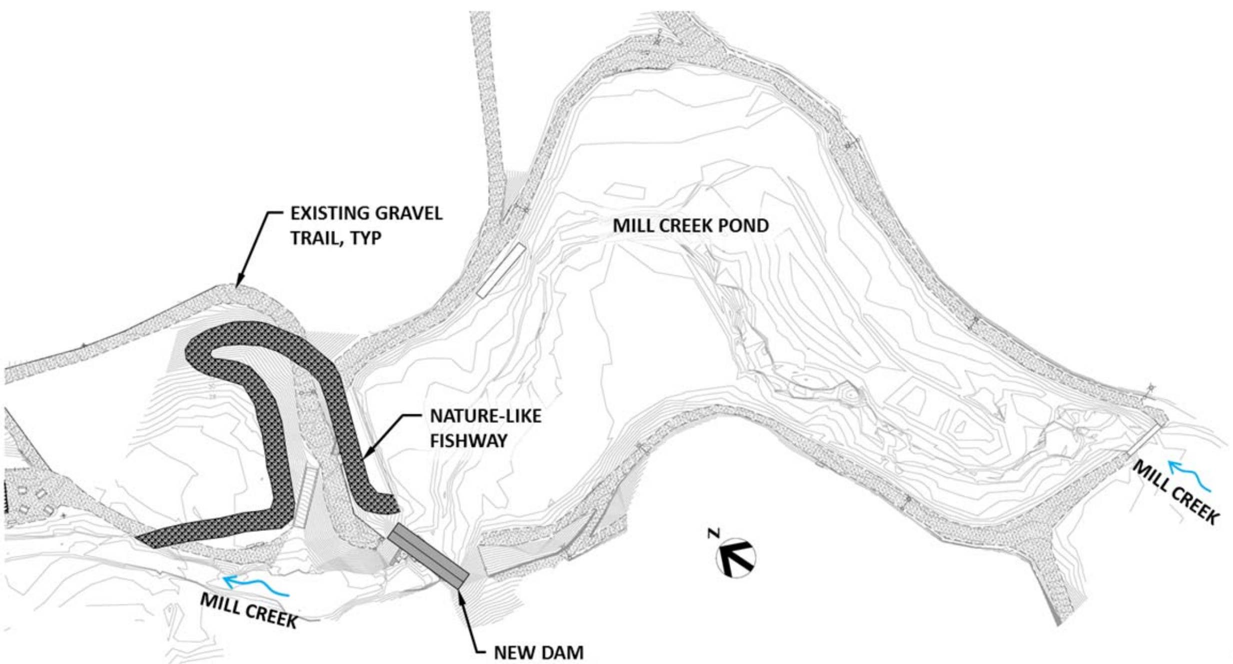


Figure 2. Concept schematic of Alternative B.

Alternative C – Fish Passage Conduit or Tunnel Through Dam

Alternative C includes fish passable conduits (or tunnels) through the dam. The conduits would effectively be designed as fish passable culverts, providing fish passage when a reservoir pool is not present.

This alternative would include design and construction of the following primary project elements:

- Two openings designed to facilitate fish passage conditions through the center of the new dam structure.
- Installation of instream cross-vane weirs to facilitate fish passage to the base of the dam during flow through conditions.

The primary disadvantage to this alternative is that it only provides fish passage during periods when there is no reservoir pool upstream of the dam. Fish passage is only provided when the dam is freely passing flow through the conduit similarly to a culvert.

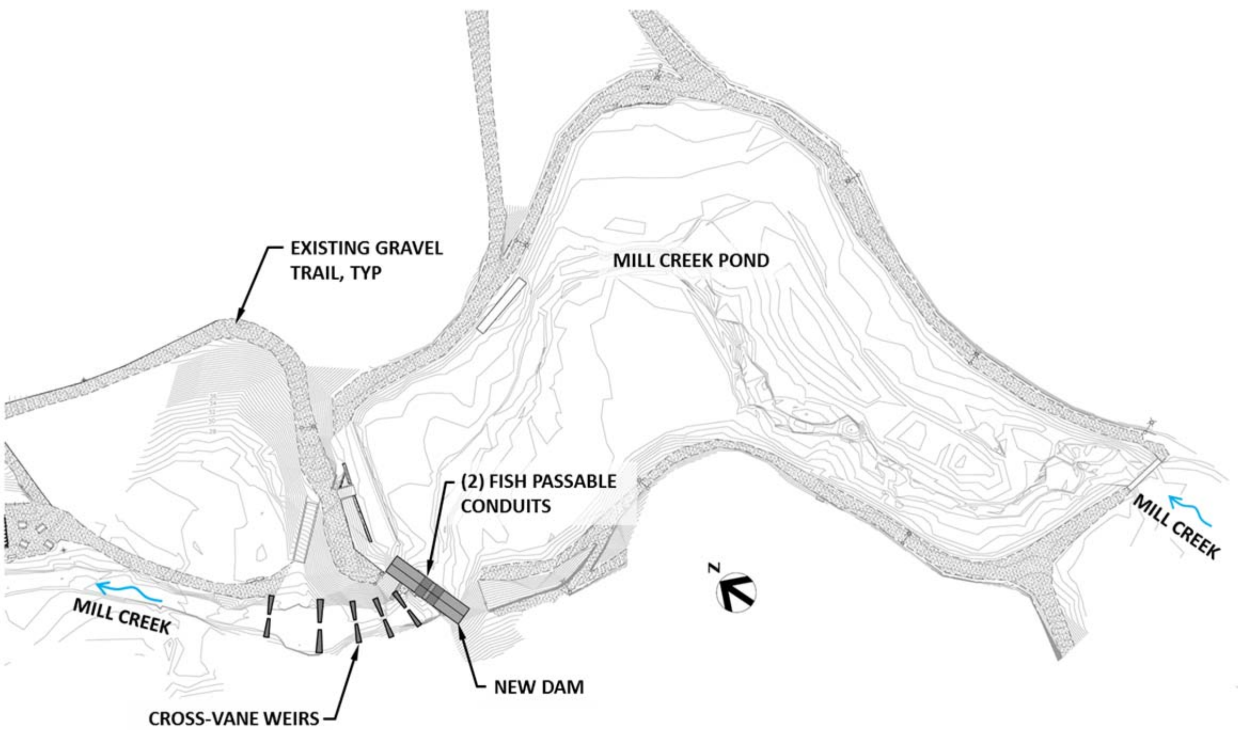


Figure 3. Concept schematic of Alternative C.

Alternative D – Combination Fish Ladder and Fish Passage Tunnel
Alternative D combines the previous three alternatives, providing fish passage during both full pool and no pool conditions. During full pool operations the combination fish ladder, which consists of a conventional fish ladder and nature-like fishway, provides fish passage. Fish passage would be provided through conduit through the dam during periods when there is no reservoir pool.

Assuming the conventional fish ladder accommodates half the hydraulic differential, 5 feet, and the nature-like fishway accommodates the other 5 feet, the overall fishway would be approximately 300 feet. This assumes the nature-like fishway is designed with a 3% slope and that the conventional fishway is designed with 8 foot long pools with a 0.5 foot hydraulic drop over each weir and 20 feet for both the fishway entrance and exit structures.

This alternative would include design and construction of the following primary project elements:

- Fish ladder entrance approximately 50 ft downstream of new dam structure.
- Combination nature-like and pool and chute fish ladder to reservoir.
- Fish ladder exit designed for typical full pool operating conditions.
- Two openings designed to facilitate fish passage conditions through the center of the new dam structure.
- Installation of instream cross-vane weirs to facilitate fish passage to the base of the dam during flow through conditions.

The primary disadvantage to this alternative would be the cost. This alternative has a relative cost higher than the previous three alternatives due to the incorporation of multiple fish passage features. However, this alternative provides passage during both full pool and no pool conditions.

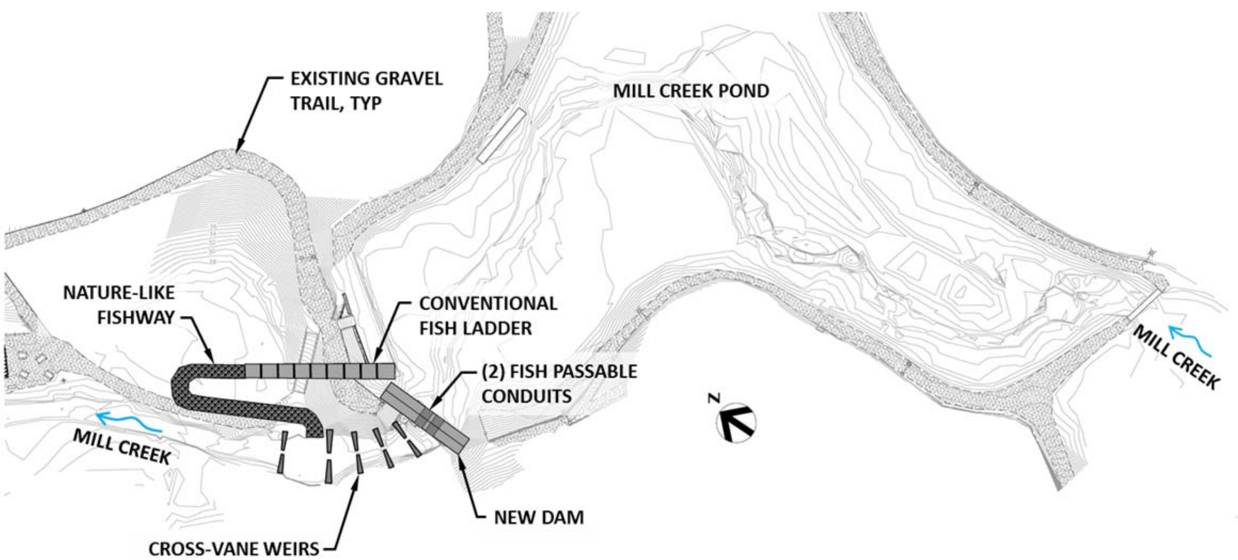


Figure 4. Concept schematic of Alternative D.

Alternative E – Fish Ladder and Reservoir Bypass

Alternative E provides fish passage via a conventional fish ladder and reservoir bypass from the base of the dam to a location upstream of the reservoir.

This alternative would include design and construction of the following primary project elements:

- Fish ladder entrance near base of new dam structure.
- Pool and chute or pool and weir fish ladder to reservoir bypass.
- Reservoir bypass channel to head of reservoir.
- Creek flow bifurcation structure at head of reservoir.

The primary advantage of this alternative is that its ability to pass fish is not affected by reservoir operations. The primary limitation is the difficulty to effectively bifurcate flow at the head of the reservoir. Further, it is difficult to route all downstream migrating fish back through the reservoir bypass, as many may likely be swept over the bifurcation sill and into the reservoir. These fish would remain in the reservoir until removed or salvaged during low flow conditions. Flow bifurcation would likely require a cross-channel sill in order to split the flow as desired between the bypass channel and the river channel. This added complexity increases the alternative cost as well as operation and maintenance requirements. Another limitation is that a reservoir bypass requires a relatively large footprint. The reservoir bypass channel would be designed similar to a nature-like fishway (channel slope between 1% and 5%) and would require a width similar to the existing footpath. Therefore, the footpath would either need to be removed, or a large amount of grading would be required in order to fit both the footpath and reservoir bypass from the dam to the head of reservoir.

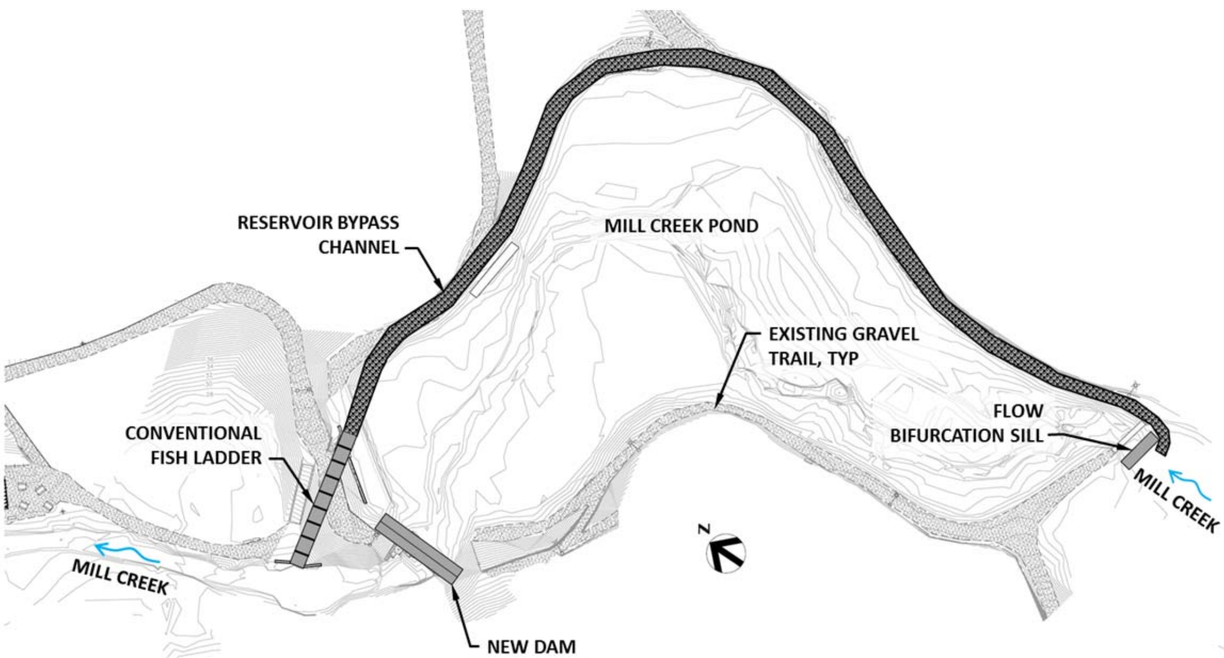


Figure 5. Concept schematic of Alternative E.

Comparison of Alternatives

Each alternative concept was qualitatively evaluated based on its ability to meet the project objectives. Results from the evaluation were used to score and rank each of the potential alternatives. Evaluation of the alternatives included the following activities:

- Identification of evaluation factors to be used as a uniform basis of comparison;
- Development of screening matrix and rating of alternatives based upon how well they are perceived to meet the constraints of the established evaluation factors;
- Ranking of alternatives based upon their apparent ability to meet project objectives; and
- Preparation of order of magnitude project construction, operation and maintenance, and implementation costs.

Discussion and conclusions resulting from these activities are summarized in the following paragraphs.

Evaluation Factors

Multiple evaluation factors were initially listed and, after a few trial evaluations, the number of evaluation factors was narrowed down to five that appeared to adequately reflect the objectives of the proposed project. The five evaluation factors used in the final evaluation process are provided below in their relative order of priority for this project:

1. Perceived Effectiveness – The intent of this selection factor compares the relative ability of each potential option to meet project objectives within regulatory guidelines over the range of foreseeable hydraulic conditions. An option that is believed to have a higher likelihood of meeting project objectives is given a higher rating than one than may not.
2. Meets Operation and Maintenance Objectives – This selection factor compares the relative level of effort, accessibility, reliability, and complexity of anticipated operation and maintenance activities. An option which requires complex operation and maintenance activities with a high level of anticipated effort with difficult access would rate lower than an option which requires less complex operation and maintenance effort with more favorable access.
3. Proven Technology – The intent of this selection factor is to measure the relative level of perceived agency acceptance. An option which incorporates a proven standard of the industry approach would rate higher than an approach that would be recognized as unproven or “experimental”.
4. Constructability – This selection factor compares the relative level of complexity to construct a potential option. Construction elements considered include scheduling within instream construction windows, interaction with the existing facility, basic construction methods, as well as the number and complexity of construction elements.
5. Capital Cost – This selection factor is a qualitative comparison of the relative order of magnitude construction costs. Construction costs are assumed to include the general conditions, mobilization, earthwork, purchase of materials, installation of all project components, start-up, demonstration, and commissioning of the proposed facility. An

option with a higher anticipated construction cost would rate lower than an option with a lower anticipated construction cost.

Rating Scale

Each alternative was given a rating of 1 to 3 for each evaluation factor. A rating of 3 indicates an alternative is favorable and meets all evaluation factor requirements identified. A rating of 1 indicates that the alternative is unfavorable with respect to that evaluation factor criterion.

Rating Results

Ratings were applied to each alternative for each evaluation factor. Results of the evaluation are presented in Table 3.

Table 3. Evaluation results of each concept level alternative.

	Alternative A	Alternative B	Alternative C	Alternative D	Alternative E
Effectiveness	1	1	1	3	2
O&M	2	2	3	2	1
Proven Technology	3	3	2	2	2
Constructability	3	1	3	3	2
Capital Cost	2	2	3	1	1
Total	**11**	**9**	**12**	**11**	**8**

The evaluation resulted in the following ranking of alternatives:

1. Alternative C – Fish Passage Conduit or Tunnel Through Dam
2. Alternative D – Combination Fish Ladder and Fish Passage Tunnel
3. Alternative A – Conventional Fish Ladder
4. Alternative B – Nature-Like Fishway
5. Alternative E – Fish Ladder and Reservoir Bypass

Alternative Selected for Further Consideration

Alternatives A, C, and D were all evaluated to be feasible options and had very similar total evaluation ratings. Ultimately, Alternative D was selected due to its ability to provide fish passage under a larger range of operating conditions than Alternative A and C. This is reflected in the “perceived effectiveness” rating, where Alternative D was rated the highest. Alternative D provides the flexibility to provide passage in both reservoir operation scenarios, ultimately providing the most flexibility in the dam operations without impacting fish passage. The proposed layout for Alternative D is provided in Attachment A of this document.

Next Steps

The following items are identified as next steps required in the project implementation process.

- Analyze Mill Creek hydraulics to refine design of dam fish passable conduits.
- Analyze Mill Creek hydraulics to establish downstream boundary condition of fishway.
- Analyze Mill Creek dam operations to establish upstream boundary condition of fishway during pond operations.
- Complete preparation of design documentation and construction documents in collaboration with the City and stakeholder requirements.
- Perform design reviews with the fisheries agencies as part of the environmental permitting process.

References and Citations

Bell, M. 1991. Fisheries Handbook of Engineering Requirements and Biological Criteria. U.S. Army Crops of Engineers, North Pacific Division, Portland, Oregon.

NMFS (National Marine Fisheries Service). 2011. Anadromous Salmonid Passage Facility Design. NMFS, Northwest Region, Portland, Oregon.

Smith, C.J., and M. Wegner. 2001. Salmon and steelhead limiting factors: Chehalis basin and nearby drainages Water Resource Inventory Areas 22 and 23. Washington State Conservation Commission Final Report.

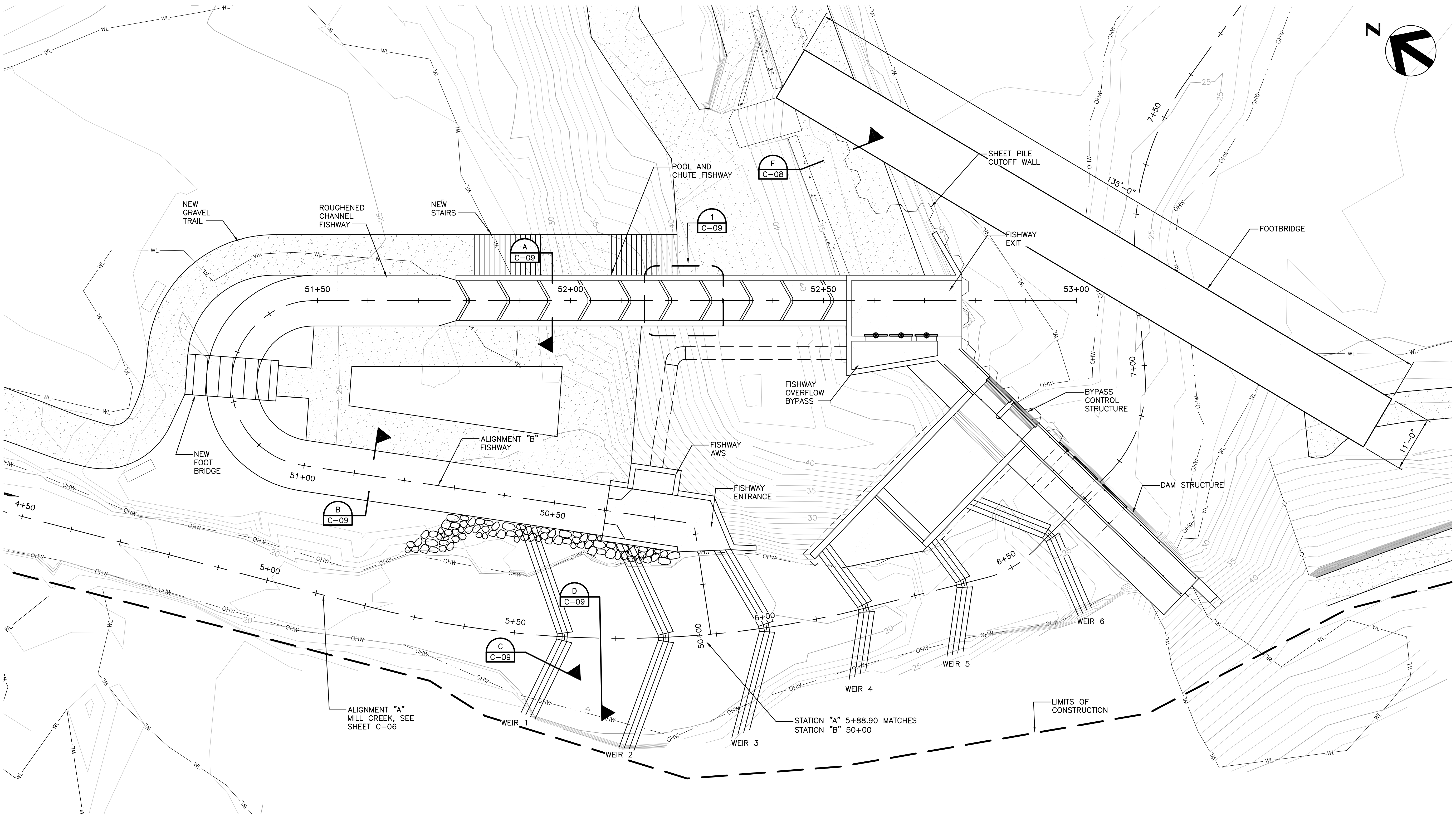
WDFW (Washington Department of Fish and Wildlife). 2000a. Draft Fish Protection Screen Guidelines for Washington State.

WDFW. 2000b. Draft Fishway Guidelines for Washington State.

WDFW. 2013. Water Crossings Design Guidelines.

WDFW Salmonscape. 2014. Fish Distribution Maps. [Online] Available: http://wdfw.wa.gov/mapping/salmonscape. Accessed January 2014.

Attachment A – Overall Layout of Alternative D



PLAN
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PERMITTING DRAWINGS
NOT FOR CONSTRUCTION

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ISSUE	DATE	DESCRIPTION

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DESIGNED BY:
CHECKED BY:
DRAWN BY:
PROJECT NUMBER: 000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

FISH PASSAGE PLAN

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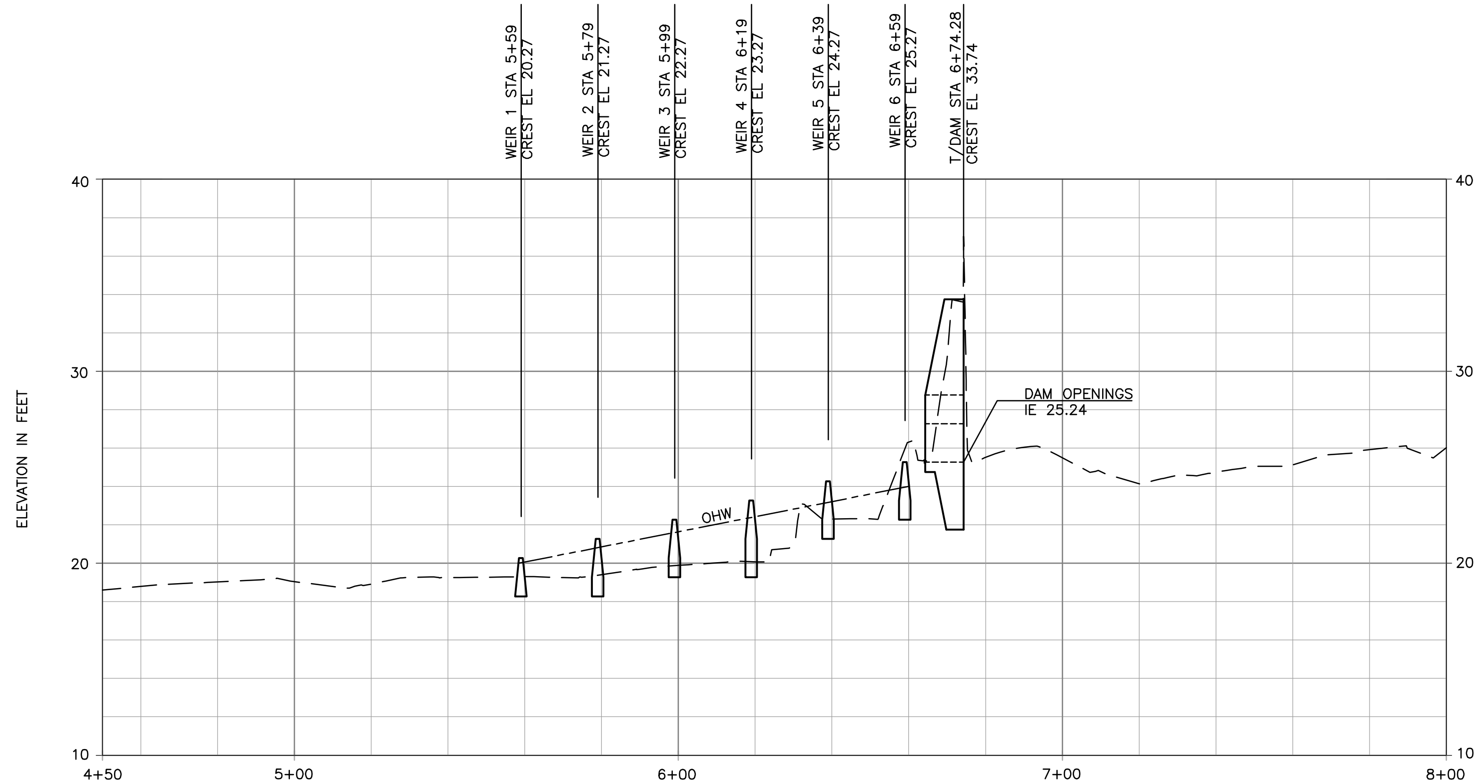
SHEET: 7 Of 16
C-04



ALIGNMENT "A" STATIONING
MILL CREEK PROFILE
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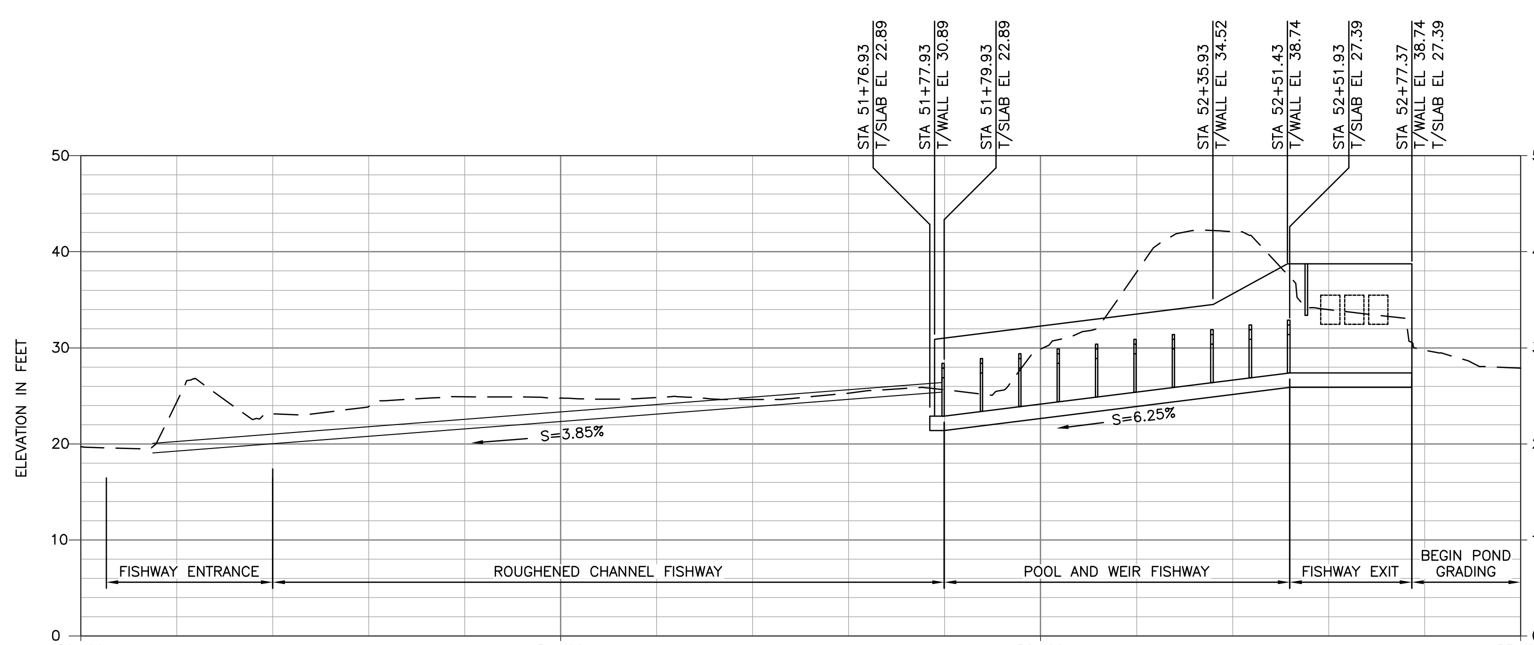

ALIGNMENT "B" STATIONING
FISHWAY PROFILE
SCALE: 1" = 20' H 1" = 10' V

PERMITTING DRAWINGS
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HDR

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER:	T. HUME
DESIGNED BY:	
CHECKED BY:	
DRAWN BY:	
PROJECT NUMBER:	000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF COSMOPOLIS

MILL CREEK PROFILE

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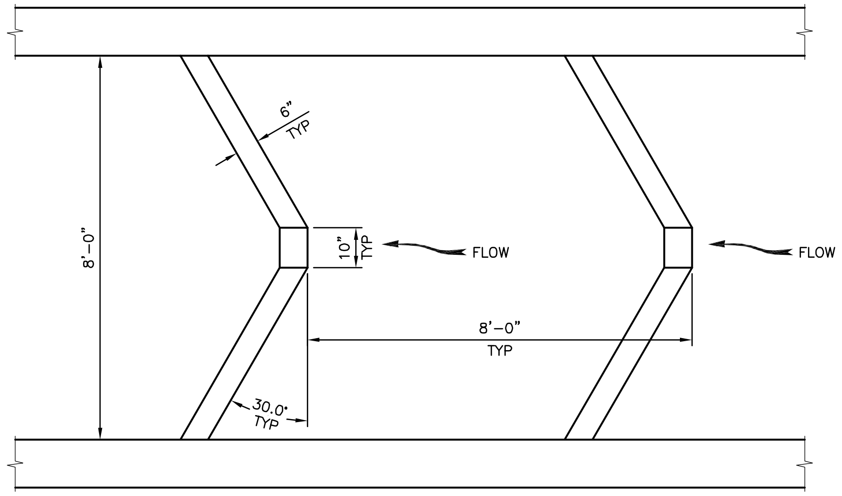

POOL AND CHUTE
FISHWAY PARTIAL PLAN
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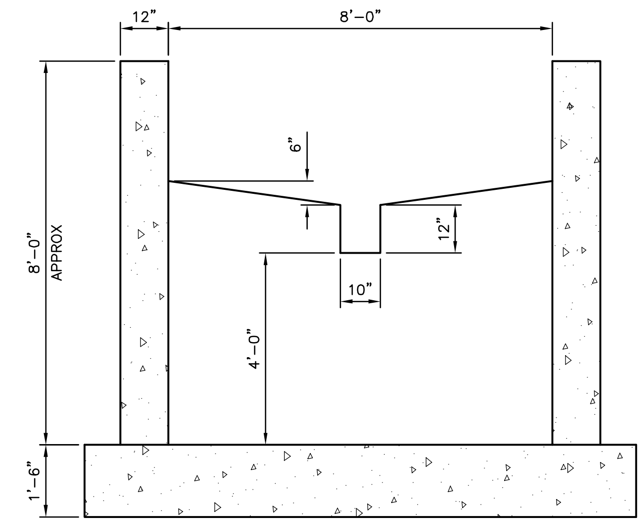

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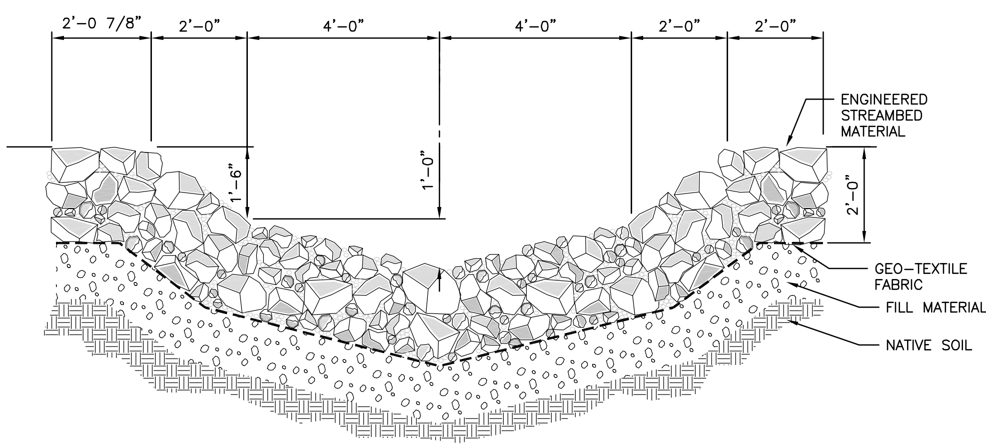

ROUGHENED CHANNEL SECTION
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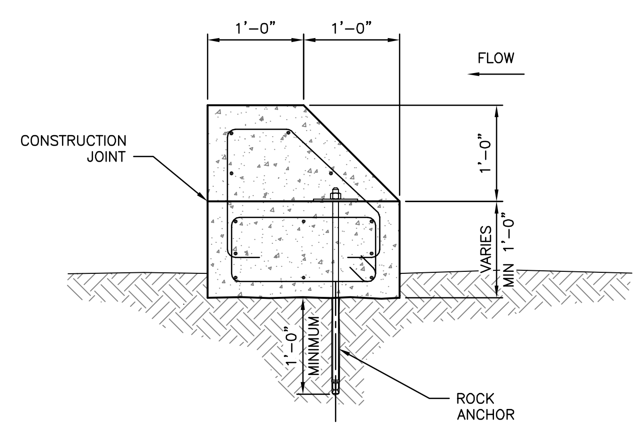
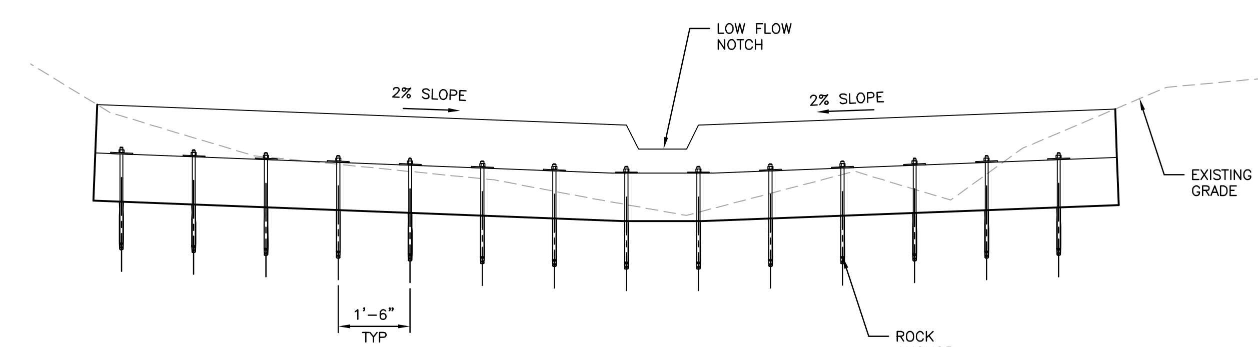


TYPICAL INSTREAM
WEIR SECTION
SCALE: 1" = 1'-0"
C
C-04

NOTES:

1. WEIR WILL BE MORE NATURALISTIC BOULDER SHAPED.



TYPICAL INSTREAM
WEIR ELEVATION
SCALE: 1/2" = 1'-0"
D
C-04

NOTES:

1. OHW ELEVATION VARIES. SEE MILL CREEK PROFILE ON SHEET C-06

PERMITTING DRAWINGS
NOT FOR CONSTRUCTION

ISSUE	DATE	DESCRIPTION

PROJECT MANAGER:	T. HUME
DESIGNED BY:	
CHECKED BY:	
DRAWN BY:	
PROJECT NUMBER:	000000000171201

MILL CREEK PARK DAM IMPROVEMENTS

CITY OF
COSMOPOLIS

FISH PASSAGE SECTIONS
AND DETAILS

FILENAME C-09.dwg
SCALE AS NOTED

SHEET: 11 Of 16
C-09

Appendix E

Footbridge Technical Memorandum

Technical Memorandum

Date: April 21, 2015
Project: Mill Creek Dam Improvements Project
To: Darrin Raines, Director of Public Works; City of Cosmopolis
From: Tim Hume, Project Manager; HDR
Prepared by: Tony Messmer, HDR

Reviewed by: Mike Lamont, HDR

Subject: Footbridge

1.0 Introduction

The City of Cosmopolis is planning the replacement of the failed Mill Creek Park Dam. The dam was breached in November 2008 as a result of erosion between the concrete gravity dam and the right abutment. The breach was caused by a large alder tree that fell from the hillside above the dam after several days of heavy rain. The fallen alder caused the hillside to become unstable and slide into the sheet piling of the dam, causing the breach to occur. A footbridge located above the dam was part of the pond's loop trail and also failed during the dam breach, and the City plans replacement of the footbridge concurrent with dam replacement.

2.0 Purpose and Scope

This memorandum documents the initial evaluation for replacement of the failed footbridge, and provides a brief discussion of identified alternative locations, width, and structural material types.

3.0 Footbridge Location Alternatives

The previous footbridge was located slightly downstream of the existing dam, providing a means across Mill Creek, and completing the loop trail around Mill Creek Pond. In order to provide the same loop trail functionality, locations considered for the new footbridge are in the vicinity of the dam. Figure 1 shows three locations considered for the replacement footbridge.

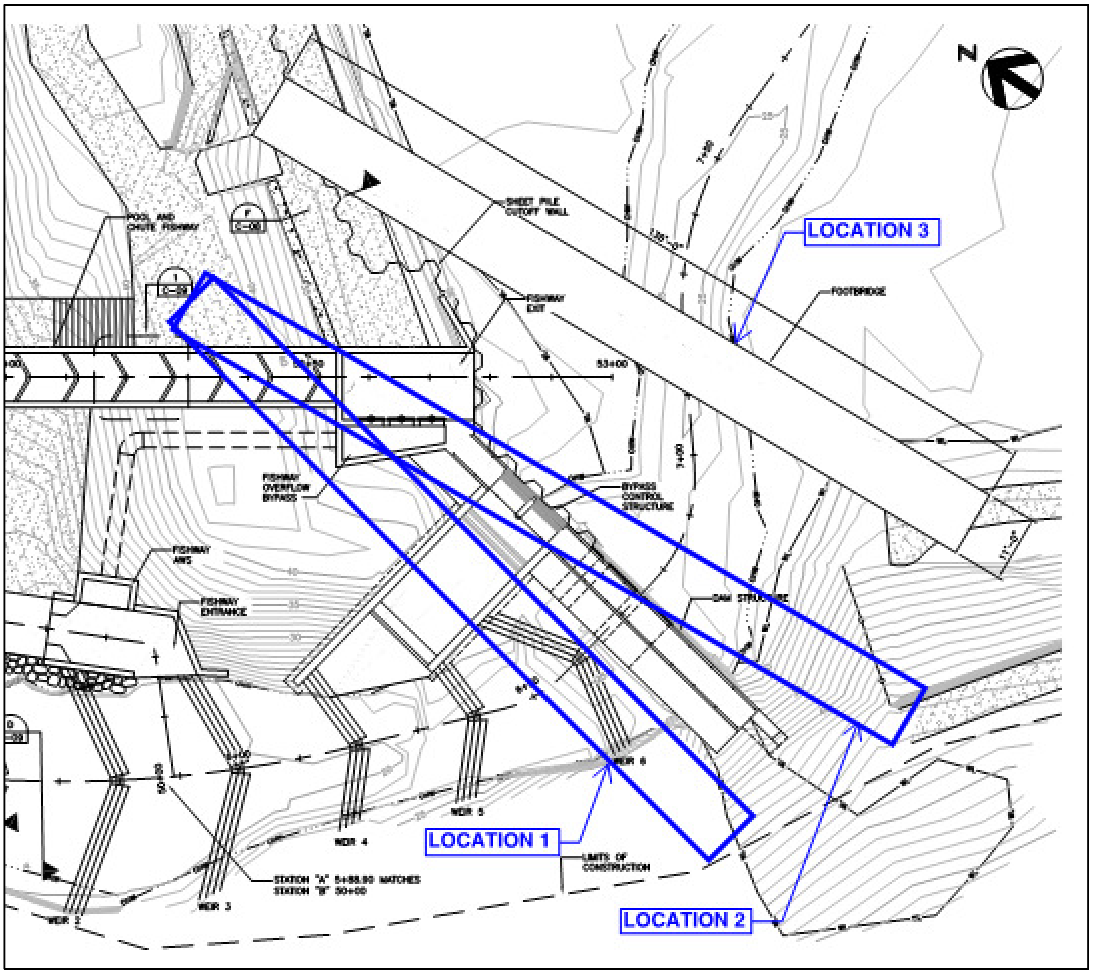


Figure 1: Footbridge Locations Considered

Location 1 – Downstream of the Dam (Previous Footbridge Location)

Location 1, near the location of the previous footbridge, would span Mill Creek just downstream of the dam from the existing embankment on the north side to the existing natural embankment on the south side.

The length of the new footbridge would be longer than the previous bridge due to erosion of the embankment on the north side, and resulting from construction of the new dam and fishway. On the south side, the existing steep slope embankment at what would be the south footbridge abutment would lead to increased foundation costs and construction difficulty, in addition to constraints for restoring trail access to the footbridge through a delineated wetland area.

Location 2 – Above the Dam

Location 2 would span Mill Creek above the dam from the existing embankment on the north side to the existing natural embankment on the south side. The location of the north footbridge abutment would be virtually the same as that of Location 1. However, the location of the south footbridge abutment would be shifted east to a location where the slope of the existing ground is not as steep as the Location 1 south abutment, reducing concerns associated with increased foundation costs and construction difficulty, and avoiding the delineated wetland area.

Since this footbridge location would be situated above the dam, approximately seven feet of vertical clearance between the top of the dam and the bottom of the footbridge structure would be needed to provide sufficient headroom for City staff to operate and maintain the dam. The elevation of the walkway on top of the new dam is 38.74 feet and the existing ground at the north footbridge abutment is elevation 41.00 feet, resulting in inadequate vertical clearance. Although less challenging than Location 1, providing trail access to the footbridge at the south abutment would also be difficult due to the existing embankment slope.

Location 3 – Upstream of Dam

Location 3 would cross Mill Creek upstream of the new dam, spanning from the terminus of the lower loop trail around Mill Creek Pond on the north and south sides.

The elevation of the bottom of the footbridge would be set at the same elevation as the top of the new dam, elevation 38.74 feet, which will require minor re-grading around the north and south footbridge abutments where the existing ground is currently about elevation 34 feet.

The length of the footbridge at Location 3 would be approximately 135 feet with the south abutment being placed at the end of the lower trail on the south side of Mill Creek Pond and the north abutment being placed clear of the new fishway exit, while allowing access along the west side of the north footbridge abutment for City staff to operate and maintain the new dam and fishway.

Preferred Location

The preferred location for the replacement footbridge is Location 3, upstream of the new dam.

4.0 Footbridge Width

The preferred width of the new footbridge is assumed to be approximately 11 feet, including 9 feet of horizontal clear distance plus 2 feet for structural members and railings.

The 9 feet of horizontal clear distance consists of a minimum of 6 feet for two-directional pedestrian travel plus 3 feet for standing along either side of the footbridge without disrupting travel. The 6 feet of two-directional pedestrian travel is typically required to provide 3 feet clearance for each direction of travel. In addition, the 9 feet horizontal clear distance would also provide sufficient clearance for an all-terrain vehicle (ATV) or similar smaller vehicle to traverse the bridge for City park maintenance or for emergency response purposes.

5.0 Footbridge Structural Material Type Alternatives

Four structure types are considered suitable for this footbridge location, span length, and bridge width. The characteristics of each alternative structure type are provided below with regard to aesthetics, service life, maintenance, and construction costs.

Timber Beams with Timber Deck Slab and Railings

- Aesthetics – suitable for the park environment
- Service Life - estimated to be 40 to 50 years
- Maintenance – maintenance costs are associated with re-painting or re-applying protective coating for the timber elements.
- Construction Cost – would require multiple spans thus cost varies depending on pier locations and span length

Steel Truss with Cast-in-Place Concrete Deck Slab

- Aesthetics – typically considered aesthetically pleasing
- Service Life - Estimated to be up to 75 years
- Maintenance – maintenance costs are associated with re-painting the steel truss and railings every 20 to 30 years
- Construction Cost – estimated to range from $150 to $180 per square foot.

Steel Beams with Cast-in-Place Concrete Deck Slab

- Aesthetics – can be improved by using special architectural railings
- Service Life - estimated to be up to 75 years
- Maintenance – maintenance costs are associated with re-painting the steel beams every 30 to 40 years, and re-painting the steel railings every 20 to 30 years.
- Construction Cost – estimated to be $125 to $150 per square foot

Precast Concrete Beams with Integral Concrete Deck Slab

- Aesthetics – can be improved by using special architectural railings
- Service Life - estimated to be up to 75 years
- Maintenance – maintenance costs would be associated with re-painting steel railings if they are used (concrete railings can also be used)
- Construction Cost – estimated to be $100 to $120 per square foot.

Preferred Structure Type

The preferred structure type will be selected by the City during the final design phase of the project.

6.0 Design Criteria

The design of the footbridge will be in accordance with the following, as appropriate:

- AASHTO LRFD Bridge Design Specifications and AASHTO Guide Specifications for Pedestrian Bridges
- International Building Code
- American Institute of Steel Construction
- American Welding Society

Design live loading will consist of standard pedestrian loading, as defined by the AASHTO LRFD Bridge Design Specifications, and an ATV or other similar City maintenance vehicle defined by the City.

7.0 Next Steps

The following activities will be performed as part of the final design of this project:

- Alternatives analysis of footbridge width and structure type.
- City's selection of preferred footbridge
- Final design of footbridge

Appendix F

Environmental & Permitting Documents

Bound Separately

- *Washington State Joint Aquatic Resources Permit Application (JARPA) Form*
- *Biological Evaluation and EFH Assessment*
- *Fish and Aquatics Habitat Report*
- *Wetland and Stream Delineation Report*
- *Cultural Resources Assessment*