McLean Creek (MC1) Dam

Updated Conceptual Design Report – Final – Vol 1 of 2

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EXECUTIVE SUMMARY

A Flood Mitigation Report completed in 2014 on behalf of Alberta Environment and Parks (AEP) identified the Springbank Off-Stream Reservoir (SR1) and McLean Creek Dam (MC1) as flood mitigation options for the Elbow River. In October 2015, the Government of Alberta announced they were proceeding with the SR1 project. As part of the required options assessment to supplement the Environmental Impact Assessment (EIA) submission for SR1, more detailed conceptual engineering and environmental work is being completed for MC1. This conceptual design report builds upon the McLean Creek Dry Dam concept originally proposed in the AMEC 2014 study [1]. The McLean Creek concept includes an earth-filled dam across the Elbow River valley, immediately upstream of McLean Creek. The concept is a dry dam, and would protect the downstream from a flood like the June 2013 flood but retain approximately 70,000 dam^3 and utilize the Glenmore capacity of 10,000 dam^3. This would provide a flood mitigation concept similar to the Spring Bank off stream reservoir design.

Opus was retained by Alberta Transportation in October 2016 to further develop the concept previously developed along with the associated construction cost estimate and schedule.

The dam would be approximately 50 meters in height (elevation 1429.0 m) and 350 meters in length at the river valley, with a total length of approximately 2300 meters.

The upstream portion of the dam will act as a cofferdam to control and divert the Elbow River around the construction area through two – six meter diameter gated tunnels in the right abutment. These tunnels will also maintain the level of the permanent pond impounded by the dam, and will control river flows downstream during operations. The diversion tunnels will be approximately four meters above the river bed, maintaining a permanent pond of approximately 180 acres approximately 15 meters deep, leaving approximately 35 meters above the pond surface to the top of the dam for flood storage.

The un-gated Service Spillway on the top of the dam at the left abutment is a concrete chute, approximately 40 meters wide at the top and 400 meters long. In the event of a flood in excess of the June 2013 event, the Service Spillway would allow water to overflow the dam to the Elbow River downstream of the dam.

Fish passage through the dam is provided via a short tunnel at the permanent pond elevation. Flows released through the fish passage facility would be kept relatively constant, and a concrete baffle system and possible periodic adjustments would be required to releases made through the tunnel to maintain water levels within a pre-selected reservoir range.

Operating this dam will be relatively simple as the only mechanisms are the gates for the Outlet Tunnels and the fish passage tunnel. During the passage of flood events, the fish tunnel gate will be closed. The gates in the low level Outlet Tunnels would initially be adjusted to limit releases through the tunnels to approximately 212 m^3/s, and these outflows would be maintained until the reservoir level surcharges to an elevation of 1424.4 meters. If the reservoir should rise to elevation 1424.4 meters, which is equivalent to passage of 2013 flood event, these gates would be further opened and additional flow would be released to prevent further rise of the pond elevation, if possible. For very large events, exceeding the 1000-year return period, the Outlet Tunnel gates would be fully opened and additional pond rise will occur. The Service Spillway would be activated for a flood greater than the 1000 year event. During passage of the project's Inflow Design Flood, the Probable Maximum Flood, the reservoir is expected to rise to elevation 1428.1 meters. Flood waters would be passed by the Outlet Tunnels, the Service Spillway and the Auxiliary Spillway in the right abutment. Thus, the dam is designed to safely pass the Probable Maximum Flood.

There is existing infrastructure located within the MC1 dam and reservoir footprint that would be impacted during a flood event, and as such would need to be relocated before the dam is fully constructed. Facilities requiring relocation include:

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- Elbow Valley Ranger Station (EVRS) and its water/wastewater treatment facilities,
- Approximately 10 kilometers of Highway 66 and the existing bridge over the Elbow River,
- McLean Creek Campground store,
- McLean Creek wastewater lift stations,
- 19 camping stalls at the McLean Creek Campground, and
- Various power and communication lines.

The existing EVRS, park camping and trails were reviewed to determine general function and usage in order to either mitigate or plan replacement of the facilities. The impacted facilities were considered in the plan to be either relocated or replaced on a like for like basis. The cost of demolition, or relocation and reclamation and/or new construction was considered and included.

Hemmera Envirochem Inc. was separately retained by Alberta Transportation to undertake various environmental investigation and assessments. The reclamation and compensation requirements have been included in the estimated costs.

The total Cost Opinion for the MC1 project, including the highway/bridge and facility relocation as well as the above- noted other requirements is estimated at **$406 Million** (including contingencies).

Two separate construction staging and delivery schedules were considered; one with a fall start and another with a spring start. The fall start construction schedule results in the shortest construction duration of less than four years total, and the facility would be fully operational for flood protection or ‘Substantially Complete’ after three spring seasons.

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1. Purpose of The Project

The primary purpose of the study is to re-evaluate the general concept of the McLean Creek Dam (MC1) option as well as the cost and construction schedule presented in the March 2014 Report by AMEC Foster Wheeler, Environment & Infrastructure (AMEC). The basic concept of an earthfill dam with a lower spillway, upper spillway and auxiliary outfall are elements that are progressed further in this study. Neither new locations nor completely new options were part of this study.

In order to achieve the primary purpose of this study, the following elements were reviewed, confirmed, or revised:

- Hydrological information and considerations
- Surficial and bedrock geology and hydrogeological information and considerations
- Conceptual engineering design, including providing more detail where required to confirm and optimize the engineering conceptual design of the various components for cost estimating
- Identification of potential borrow material sources with due consideration for environmental and regulatory requirements
- Assessment of the potential amount and level of debris carried by the flood waters
- Options to address fish passage
- A more in-depth construction schedule (one starting in spring and another in fall)
- A refined cost estimate based on this updated conceptual design

Opus was retained by Alberta Transportation in October 2016 to further develop the concept previously developed along with the associated construction cost estimate and schedule. The primary team members and their general areas of responsibility are presented below.

- Opus – overall project management of the team, construction cost estimate opinion, field and laboratory for drilling and coring
- Hatch – dam flood routing and structure, and geology
- North West Hydraulic – Probable maximum flood, hydrology and fish passage
- BGC – terrain analysis and bedload/debris assessment

1.1. Background

The MC1 study area is located on the Elbow River approximately 10 kilometers upstream of the Town of Bragg Creek, and immediately upstream of the confluence with McLean Creek, Figure 1. The purpose of the MC1 option, in combination with the Glenmore Reservoir, is to effectively manage floods having a magnitude of at least the 2013 flood event.

The MC1 concept consists of constructing an earth dam across the Elbow River to temporarily contain extreme flood flow until it can be safely released after the flood peak has passed. A Conceptual Design for this project was completed in May 2014 by AMEC and the results were presented in a May 2014 report (refer to Reference 1).

The major project components include a 50 meter high earth dam on the Elbow River and a gated concrete conduit outlet structure for managing normal and controllable flood flows, as well as an ungated chute spillway structure and an auxiliary earthen channel spillway to protect the dam from more extreme flood events. A permanent pond approximately 180 acres in size and 15 meters deep is planned for debris management.

Due to the area that would be inundated during a flood, the existing Highway 66 and bridge over the Elbow River would need to be relocated. Similarly, other infrastructure located within the proposed containment area, particularly Alberta Environment and Parks (AEP) infrastructure, would need to be relocated or otherwise modified to suit project needs. When completed, the dam system would be operated and maintained by AEP.

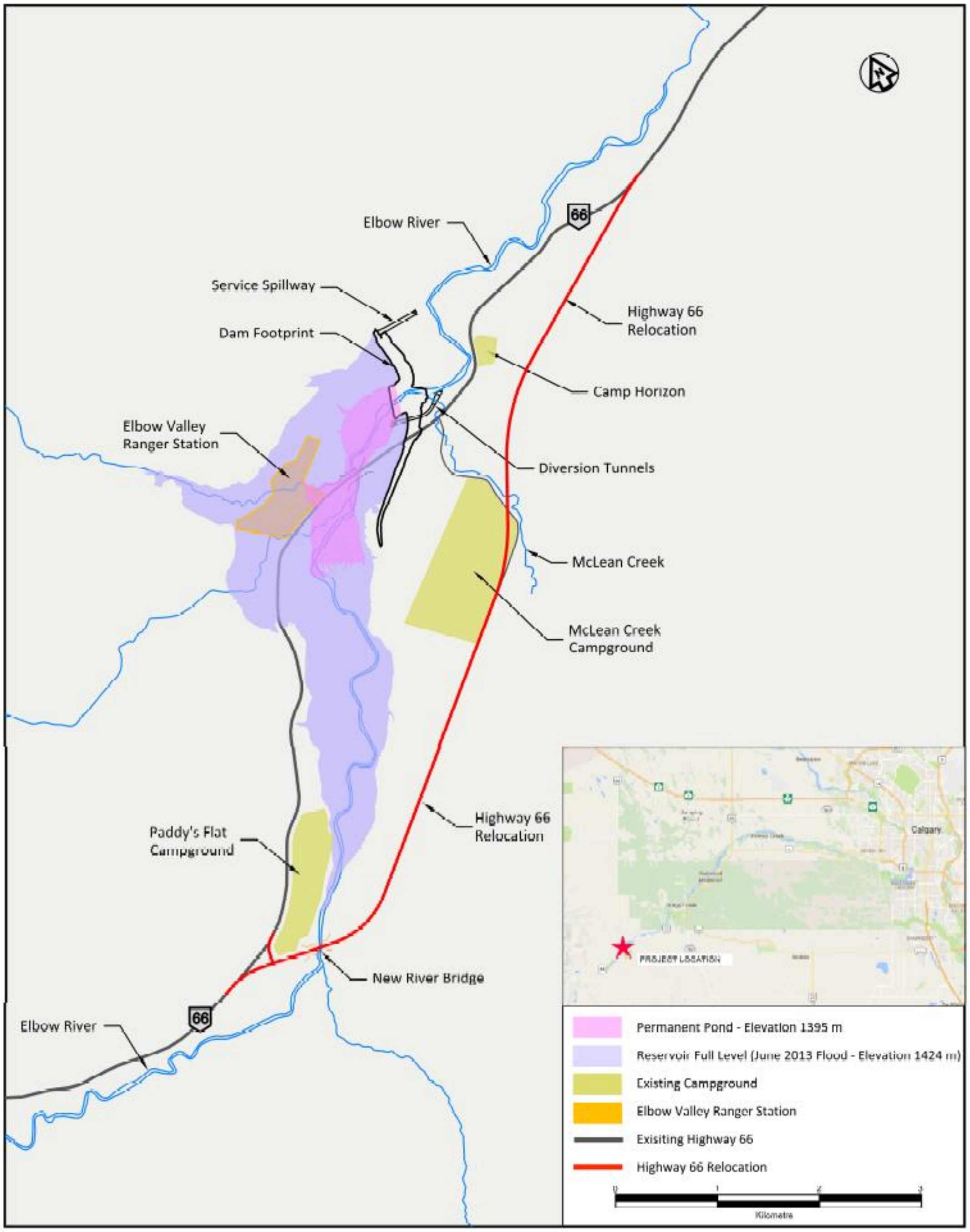

Figure 1 - Overall Site Plan

1.2. Design Criteria – Dam

The primary purpose of the dam is for flood attenuation for events as large as the June 2013 flood. The MC1 Dam would operate in conjunction with Glenmore Dam to attenuate flood events. The design criteria rationalized in the 2014 study is presented below. Additionally, Alberta Transportation has indicated that releasing 212 ^{3}/s during flood events was acceptable when utilizing the 10,000 dam^3 of storage space at the Glenmore Reservoir. The updated rationalization of flood volumes and hydrograph is presented in Section 2 of this report. The subsequent results of the flood routing and volume of water is presented in Section 6 of this report. Table 1.1 below provides the hydrological design values used in the AMEC (2014a) design.

Table 1.1: Operations Design Criteria (2014 Study)

Description (Peak Values)	Summer	Winter	Floods			
	July Mean	January Mean	20-year	100-year	500-year	PMF[1]
Peak reservoir inflow rate (m^3/s)	13.4	3.0	440	930	1,625	2,175
Permanent Outlet/Spillway Structure Outflow Rate (m^3/s)	13.4	3.0	250	260	636	780
Auxiliary Spillway Outflow Rate (m^3/s)	0	0	0	0	0	1,280
Reservoir Water surface elevation (m)	1399	1401.5	1407	1423	1426.5	1429
Total contained water volume (dam^3)	4000	5000	12000	47000	62000	72000

1.3. Design Criteria – Road and Bridge Relocation

The existing Highway 66 within the project area has a posted speed limit of 90km/h with two substandard curves. The volume of the existing highway is 1890 Average Annual Daily Traffic (AADT). The width of the pavement surface is 11.8 meters

The presence of a 50 meter high dam at the MC1 location, will result in the need to relocate 10 kilometers of the existing highway and is predominantly greenfield construction. The available options considered for relocating the highway result in a new bridge location south of the existing and a new highway location to the east.

Based on the highway classification and traffic volumes, the new road will be designed to Alberta Transportation's Highway Geometric Design Guide and have a design designation of RAU-211.8-110. The cross-sectional elements consist of 2 x 3.7 meter traffic lanes, 2.2 meter shoulders, 5H:1V side slopes, 4.0 meter ditch and 3H:1V back slopes. The basic right-of-way width will be 50 meters.

Based on the existing traffic volumes and high proportion of passenger and recreational vehicles the proposed pavement thickness for design and cost estimating will be based on 300 mm granular base course (GBC) and 100 mm asphaltic concrete pavement (ACP).

At the bridge crossing, the river channel is approximately 60 meters wide, set within a steep gorge about 12 meters deep. Clear-spanning the gorge to avoid significant earthworks and potential environmental effects within the river valley were considered in the concept update.

1.4. Design Criteria – Facility Relocation

For this conceptual design, the building sizes and usage were cataloged for the Elbow River Store and Elbow Valley Ranger Station (EVRS). Each building was assessed for relocation or salvage value and replacement. The details of the assessment are summarized in Section 8 of this report, and detailed in Appendix 8 [Facility Relocation].

The McLean Creek Campground will be impacted, and associated considerations were included to offset the impact by construction of 16 new stalls. While the campground could remain open through construction, there would be construction traffic and noise due to highway and bridge relocation construction activities, along with the Dam works. The wastewater lift stations at the Elbow River Store and Campground as well as the forcemain will be relocated.

1.5. Environmental Considerations

The potential environmental impacts created by the MC1 project were a consideration in the options review. Considerations included:

- using the existing McLean Creek campsite road,
- clear spanning the river,
- locating potential gravel and soil borrow pit and storage piles where there is no or limited trees,
- locating the temporary construction campsite and concrete plant in the existing Elbow River gravel pit,
- reclaiming all disturbed locations with topsoil and seeding to prevent erosion,
- remediating impacted soil from the operations at the EVRS, and
- provision of fish passage.

Hemmera Envirochem Inc. (*Hemmera*) was separately retained by Alberta Transportation to undertake various environmental investigations and assessments. Guidance for other environmental considerations were provided to Opus by Hemmera during this concept review based upon their investigation and assessment. The Hemmera report is not contained in this report but key items include:

- Vegetation and topsoil would be cleared from within the permanent pool area before inundation. Flood water retained behind the dam would recede to normal operating levels within two weeks following an extreme event like the 2013 flood. Dominant tree and plant species in the flood zone can tolerate this infrequent, short-term inundation. Less hardy plant species that are intolerant to flooding may not survive, and some areas may need to be revegetated after a flood event.
- Potential effects of the project on wildlife will vary among the different species that inhabit or use the project area. The Dam will create additional lake habitat, which will benefit diving waterfowl and other water birds and provide new wintering habitat for fish. Construction of the Dam and related works will, however, result in the removal of some wildlife habitat areas from active use and the alteration of habitat features in certain areas. These impacts can be reduced by minimizing the area of disturbance, reclaiming temporarily-used areas after construction, and identifying off-site habitat offsetting opportunities.
- The Dam will convert existing upstream riverine habitat into lake habitat, which could result in a change in the composition of fish species within the permanent pond area, with a relative increase in the prevalence of species that favour lake ecosystems or are more adept to environmental changes. Overall, the extent of fish habitat, relative to the area of existing riverine habitat, will increase. Creation of the permanent pond can also be expected to result in some new rearing and wintering habitat (i.e. increased ice cover during winter).

2. Hydrology Overview

2.1. Background

The primary objective of the AMEC 2014 study was to develop preliminary inflow design flood (IDF) (non-PMF) hydrographs for the conceptual design of the MC1 and SR1 dams on the Elbow River and a dam on the Bow River near Morley. The detailed hydrological review is contained in Appendix 1 [Hydrology Report]. Below is a summary of that material.

The approach undertaken by AMEC (2014a) for the Elbow River projects is briefly described as follows:

1. The flow records for the three WSC (Water Survey of Canada) stations near Glenmore Reservoir were combined to produce an extended data set covering the period of 1908-2013.
2. Flood frequency estimates for return periods of 2 through 1000 years were developed for annual maximum daily discharges, instantaneous peak discharges and annual maximum 1 through 7-day flood volumes, based on the combined 1908-2013 data set.
3. An “alternating block” approach (often used to develop synthetic design rainfall storms) was used to develop synthetic flood hydrographs with a 7-day duration for return periods of 20, 100 and 500 years. This approach has no relevance to observed hydrographs. As it embeds the maximum 1 through 7-day amounts in a single 7-day time series, it is generally expected to result in a hydrograph with conservatively high discharges off from the peak.
4. Relationships between the flood peaks and 7-day flood volumes with various return periods were developed. The relationships were used later to update the flood volumes from adjusted flood peak frequency estimates.
5. The flood peak frequency estimates were adjusted by multiplying them with factors that were intended to reflect the effects of three historical floods (which occurred prior to the systematic streamflow monitoring in 1879, 1897 and 1902) on the flood frequency analyses for the Bow River upstream of Elbow River. The factors ranged from 1.09 to 1.34 for the 5 through 1000-year floods, with 0.99 for the 2-year flood.
6. The final 7-day design flood volumes were estimated subsequently by using the relationships from Step 4 and the adjusted flood peaks. The hydrographs were then updated based on the adjusted flood volumes.
7. Given the proximity of Sites MC1 and SR1 to the Glenmore Reservoir, AMEC (2014a) considered the design values resulting from the above procedure applicable to the conceptual designs for both of the Elbow River dam projects, notwithstanding the noticeable differences among their drainage areas. The Elbow River drainage area near Glenmore Reservoir is about 75% and 42% greater than those at Sites MC1 and SR1, respectively.

Opus has based the current MC1 Dam conceptual design on the flood hydrographs resulting from the AMEC (2014a) study. Detailed review and assessment of the design values are presented in the following sections.

2.2. June 2013 Flood

As described above in Section 2.1, AMEC (2014a) developed 7-day and 20-, 100- and 500-year synthetic flood hydrographs using an “alternating block” approach. This approach is generally expected to be conservative because the maximum 1 through 7-day amounts are embedded in the resulting hydrographs. Since both of the design flood peaks and flood volumes adopted from AMEC (2014a) are considered acceptable for the MC1 project at the current conceptual design stage, we believe the hydrographs are also appropriate for the current design stage. The 200- and 1000-year flood hydrographs were developed by scaling up the 100- and 500-year hydrographs with the ratios between corresponding flood peaks, respectively. The resulting 200-year hydrograph is comparable with the June 2013 flood hydrograph for the Bragg Creek station (05BJ004) provided by WSC. It is recommended that the design flood peak discharges and 200 and 1000-year inflow hydrographs be fully re-developed should the project progress to detailed design.

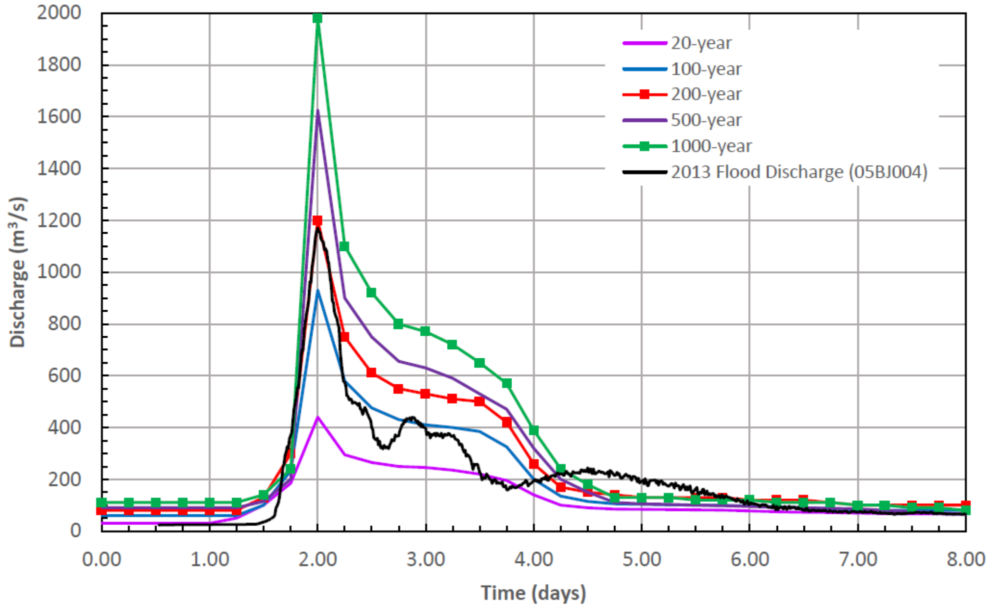

Figure 2.1: Design Flood Hydrographs Adopted for MC1 Project Conceptual Design

2.3. Probable Maximum Flood (PMF)

A PMF hydrograph for the Elbow River at the proposed diversion point on the Elbow River for the Springbank Off-stream Reservoir (SR1) project was developed by Stantec (2015a). That design PMF has a peak discharge of 2770 m^3/s and a 7-day volume of 362,000 dam^3. These values have been applied directly to Site MC1 to facilitate the conceptual design of the MC1 Dam project.

The total volume of the 48-hour PMP (probable maximum precipitation) over the SR1 basin (Scenario 1) is about 349,000 dam^3, which is smaller than the SR1 PMF volume. The snowmelt runoff volume input to the model for the 7-day modelling period (from starting of the PMP) is about 40,000 dam^3 (based on the HEC-HMS model developed by Stantec for Scenario 1). With the runoff coefficient value of 0.7 assumed by Stantec, this is equivalent to a total snowmelt (SWE – soil water equivalent) volume of 57,000 dam^3. The ratio of the SR1 PMF volume to the PMP plus the snowmelt volume input is 0.89, which appears relatively high but appropriate for an extreme event.

Based on these empirical checks, it appears reasonable to use the Stantec SR1 PMF without adjustment for the MC1 site as a preliminary estimate for the current conceptual design stage of the project.

2.4. Conclusions

The IDF hydrographs currently adopted for the MC1 Dam project are from AMEC (2014a). They appear to be generally acceptable for sizing high-flow discharge facilities (e.g. spillways) of the MC1 Dam at the current conceptual design stage.

The IDF hydrographs were from flood frequency analyses using the flow data of the Elbow River near the Glenmore Reservoir. Although they appear to be suitable for the current conceptual design stage of the MC1 project, we believe that these IDF hydrographs should be updated in subsequent design stages, based on a comprehensive, site-specific study in accordance with the Alberta Transportation guidelines on flood frequency analysis and extreme flood analysis. The 1000-year flood peak estimate from a flood frequency analysis could be higher than the adopted design value; however, estimates for such extreme floods are typically subject to significant uncertainties, and the current design value does not appear to be unreasonable when compared with our preliminary flood frequency estimates presented in this report. The 1000-year design flood peak should be re-evaluated in conjunction with PMF estimates.

It is reasonable to expect that the PMF for Site MC1 is in the range of 0.9 to 1.0 times the PMF for Site SR1. Using the PMF estimate for Site SR1 from Stantec (2015b) without adjustment is likely acceptable at the current conceptual design stage of the MC1 project.

If the MC1 project is progressed further, a detailed, site-specific PMF study is recommended. The study should be carried out in accordance with the Alberta Transportation guidelines, the Canadian Dam Association 2007 Dam Safety Guidelines (with 2013 revision) and industrial accepted practices. The detailed analysis may lead to a change in the PMF peak flow and/or volume, which could impact dam design.

3. Regional Geology Overview

The area of interest is located in the Rocky Mountain Foothills in the Foothills Parkland Sub-region of the Alberta Natural Regions classification system (Downing and Pettapiece, 2006). This is a narrow region east of the Rocky Mountains and south of Calgary, classified by open hilly terrain with extensive areas of hummocky and basal moraine, glaciofluvial and glaciolacustrine deposits along valleys (BGC, 2016).

The proposed project site is approximately 19 kilometers west of the McConnell Thrust Fault, which separates the foothills from the front ranges of the Rocky Mountains. A northwest-southeast trending thrust fault has been mapped crossing the Elbow River approximately 200 meters downstream of the confluence of the Elbow River and McLean Creek; no other thrust faults are mapped within the footprint of the dam or upstream for more than 9 kilometers (AMEC, 2014b).

3.1. Surficial Geology

Published surficial geology mapping indicates the Elbow River is underlain by coarse stream alluvium as valley bottom deposits generally consisting of gravelly sand to gravel, with cobbles and boulders. A glaciofluvial outwash plain is mapped on both sides of the Elbow River upstream of the dam (Bayrock and Reimchen, 1975). These materials consist of well sorted and rounded glaciofluvial gravel. The material present along McLean Creek is mapped as fine stream alluvium consisting of sand and clay with local minor gravel or organic material. West and east of McLean Creek, there is a moderately leeched till of the Cordilleran provenance, generally clay till with some sand and gravel. Most till deposits appear to be moderately well to well drained, with the exception of a terrace on the north side of the Elbow River where the presence of bogs suggests the ground water is shallow (BGC, 2016). The tills have some gravel and sand, but behave as low to intermediate plasticity clay tills. Bedrock is exposed in the Elbow River channel walls at the dam site and on both sides of McLean Creek near its outlet.

The surficial geology is shown in Figure 3.1 below, and is described in more detail in Appendix 2 [Terrain Analysis/Surficial Geology], which contains the terrain analysis report by BGC (2016).

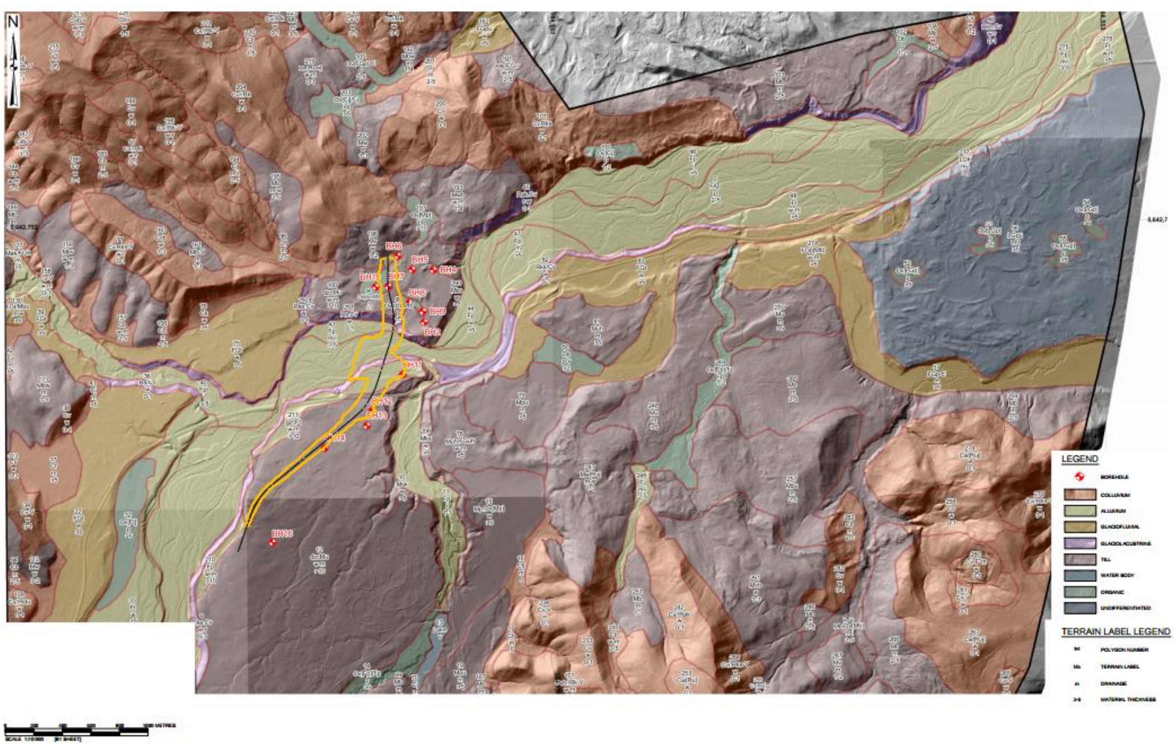

Figure 3.1 – Surficial Geology (also Drawing G-01 in Drawings Package)

3.2. Bedrock Geology

Near the project site, the bedrock is comprised of two Cretaceous-aged formations; the Wapiabi Formation to the south and west and the Brazeau Formation to the north and east (McMechan, 1995).

The Brazeau Formation is described as (Prior et al., 2013):

- Sandstone, laminated siltstone, and mudstone; chert- and quartzite–bearing, granule to pebble conglomerate (lower part); overlain by greenish-grey to dark grey mudstone, siltstone, and greenish-grey sandstone; thin coal and coaly shale beds; numerous thin bentonites (upper part).

The Wapiabi Formation is described as (Prior et al., 2013):

- Shale, mudstone, silty shale, argillaceous siltstone, and siltstone (some platy, some with rusty-brown weathering, some calcareous); local bentonite layers and siderite concretions (isolated or along horizons, locally abundant); includes fine grained, massive to cross-bedded sandstone of the Marshybank member (lower part of formation) and the fine- to coarse grained sandstone and argillaceous siltstone of the Chungo Member (upper part of formation); rare, thin chert-pebble layers; marine to locally non-marine.

The contact between the Wapiabi and Brazeau Formations is immediately downstream of the confluence between McLean Creek and the Elbow River (Prior, et al., 2013) as shown on Drawing G-02.

4. 2017 Investigation Program

The Geotechnical Investigation Report associated with the 2017 Investigative Program is presented in Appendix 3 [Dam Geotechnical] for the Dam and as a separate report for the Road and Bridge relocation in Appendix 4 [Road and Bridge Geotechnical]. A brief summary of the investigation and findings are provided below:

4.1. Investigative Program

The objective of the investigation was to determine the depth and nature of the surficial materials and the soil and bedrock properties required to advance the design to a conceptual level and prepare a cost estimate.

The geotechnical investigation comprised the following elements:

- Four geotechnical site reconnaissance assessments
- Surficial geology mapping (BGC Engineering)
- Geotechnical drilling of six boreholes
- Downhole geophysical surveys of the boreholes (Century Wireline)
- Permeability testing within the boreholes (Groundwater Information Technologies)
- Surficial geophysical surveys, including seismic refraction surveys on the left abutment and electrical resistivity tomography (ERT) on the right abutment (Advisian)
- Installation of vibrating wire piezometers (VWPs) in each borehole
- Excavation of 11 test pits within potential borrow areas
- Laboratory testing of soils and rock

4.1.1. Borehole Drilling and Testing Program

A total of six boreholes were drilled with mud rotary drilling equipment using a track-mounted rig on the south abutment (two holes) and in the river valley (one hole), and a heli-portable drill rig on the north abutment (three holes). This program provided the first samples of intact bedrock at the dam site, and insight into the nature and surface of the bedrock formations. The drilling was conducted from January 11 to February 23, 2017. The locations of the boreholes are shown on Drawings G01 to G03, with a geological profile provided in Drawing G03. Standard Penetration Tests (SPTs) were performed in some of the surficial soils to obtain penetration resistance and disturbed samples, and triple tube coring was performed in the bedrock to obtain continuous rock cores. Preliminary logging was performed in the field, and soil and rock samples were photographed, then preserved and sent to the Opus laboratory for detailed logging and laboratory testing.

Upon completion of the drilling, downhole geophysical testing was performed on five of six boreholes. BH17-01 did not penetrate the bedrock so no downhole geophysical testing was performed. The tools used included:

- Dipmeter with x-y caliper
- Natural Gamma
- Sonic
- Acoustic Televiewer
- Optical Televiewer
- Spontaneous Potential
- Density
- Neutron

From these tools the following data plots were obtained:

- Borehole wall roughness
- Natural gamma of the formation materials
- Dip and dip direction of bedding

- Porosity
- Density
- Acoustic image of borehole surface
- Optical image of borehole wall
- Formation resistivity
- Formation sonic velocity

Downhole geophysical testing was performed by Century Wireline, and their downhole logs are contained in Appendix 3 [Dam Geotechnical].

In-situ permeability testing was then performed using a packer system to isolate portions of the boreholes that were selected based on the downhole geophysics results. Packers were used to isolate specific intervals in the bedrock, and to facilitate infiltration tests in sand and gravel deposits atop the bedrock in some boreholes. The report by Groundwater Information Technologies on the permeability testing is also contained in Appendix 3 [Dam Geotechnical].

Vibrating wire piezometers were installed at selected intervals in boreholes, and the holes were backfilled with a cement-bentonite slurry.

4.1.2. Surface Geophysical Testing

During the drilling program, two unexpected conditions were encountered: Extensive gravel deposits beneath the till on the south abutment, and a very thick till deposit (24-29 meters) overlying a sand and gravel deposit of varying thickness atop the bedrock on the north abutment. To obtain a better understanding of the spatial extent and variability of these conditions, Advisian was contracted to conduct two different surface geophysical testing programs:

- On the south abutment, electrical resistivity tomography (ERT) was performed along two lines to provide insight into the thickness of the till and extent and thickness of the underlying gravel deposit
- On the north abutment, surface seismic testing was conducted along six different lines to obtain insight into the depth to the top of bedrock over the area of the north abutment

The locations of the surface geophysical testing lines are shown on Drawing G02, and the Advisian report is contained in Appendix 3 [Dam Geotechnical].

4.1.3. Borrow Area Investigation

A borrow investigation was conducted to develop a preliminary assessment of the types and quantities of fill available for dam construction. The investigation targeted potential borrow sources for both granular fill and till material. Granular borrow sources will be used for filter, drain and shell material, random fill, unclassified fill and selected granular fill for the MC1 Dam construction. Till will be used for impervious material for the dam core.

To determine potential borrow areas for dam construction, three preliminary studies were undertaken:

- Terrain analysis of an area within 5 kilometers of the dam site (BGC, 2016)
- Review of the report on the geotechnical investigation for the upgrade of Highway 66 (obtained from Alberta Transportation) by BBT Geotechnical Consultants Ltd. dated June 1981, titled “Subgrade soils investigation highway 66 (S.R. 553) west of Elbow Falls to junction Highway 22 station 11+400 to station 30+660
- Determination by Opus GIS staff of the potential areal extent of fluvial river deposits that would be covered by the FSL (full supply level) pond for various pond elevations, to examine where granular deposits were located that could be salvaged for dam construction.

Sampling in potential borrow areas was performed by test pitting using a medium sized wheeled backhoe with a maximum reach of 4.5 meters. Test pits locations are shown in Drawing G04, and profiles of the borrow areas are shown in Drawing G05.

4.1.4. Laboratory Testing

A laboratory testing program was conducted on soil and rock samples obtained from the drilling program, and samples obtained from the test pit program. Soil testing on borehole samples included particle size analysis, Atterberg Limits, and field moisture contents on selected samples. Analysis on test pit samples included the above as well as Standard Proctor Compaction testing. Tests provided insight into the nature of the soils on the site and data that could be compared to previous investigation results.

Bedrock testing included unconfined compressive strength tests on intact samples, and direct shear strength tests on joints of selected samples. The test results provided insight into the strength and deformation properties of intact rock and joints in the formations.

Shelby tube samples were also obtained during the borrow investigation. Selected specimens from these samples were sent to an external laboratory for consolidation testing. The results from these tests provide insight into the time dependent settlement characteristics of the clay till under the loading of the dam embankments.

4.2. Findings

The geotechnical investigations encountered unexpected ground conditions when compared to the previous results obtained by AMEC (2014b). The revised geological interpretation is presented on Drawing G02 and is described below.

4.2.1. Surficial Geology and Groundwater Conditions

4.2.1.1. North Abutment

On the north abutment, the depth to bedrock was much greater than expected. Under most of the dam foundation, the bedrock appears to be generally flat lying at approximately Elevation 1370 based on the drilling results and the seismic refraction survey results (Appendix 3). This is below the bedrock level of Elevation 1372, encountered in the river valley drilling.

The presence of exposed bedrock on the north river valley wall up to approximately Elevation 1400, and the apparent flat lying bedrock surface further north at Elevation 1370 has led to the conclusion that there is very likely a subsurface pillar or knob of rock that protrudes above Elevation 1370 meters between the Elbow River and seismic Line E and between Seismic Lines A and F (Drawing G02). The north side of this knob is located in close proximity to the contact between the Wapiabi and Brazeau Formations. This rock knob appears to be approximately 250 meters long and 50 meters wide at the upstream end widening to approximately 150 meters at the downstream end.

The north abutment ground surface is characterized by generally flat lying topography and much of it is overlain by muskeg. The clay till appears to be 20 to 30 meters thick and is underlain by an approximately 10 meter thick layer of gravelly sand with 10 – 20% fines, which sits atop the bedrock. Given the sandy and silty texture of the gravel, it is possible these materials are of glaciofluvial origin.

The data from the VWPs indicate the groundwater generally follows the topography in the area. The VWP data show that the piezometric head elevation reduces from 1409.4 meters to 1374.5 meters to 1373.2 meters progressing from BH17-07 to BH17-01 to BH17-02, respectively. Downward gradients are indicated in each borehole from the VWP data.

Two permeability tests undertaken in the gravel layer in the north abutment indicate moderate permeability (1×10^{-4} to 6×10^{-3} cm/s) with lower permeability in deposits that contained higher fines contents.

4.2.1.2. River Channel

The results from the borehole in the river valley indicate that bedrock in the river channel is overlain by fluvial silty sand and gravel up to six meters thick. Permeability testing was not undertaken within fluvial materials in the river channel.

The data from the VWPs indicate the piezometric head elevation in the upper VWP has increased by approximately 2.0 meters since installation, and on the last reading (April 18, 2017) was one metre above the surface elevation. Slight downward gradients are indicated. The increased pressure during the April 2017 reading indicates there is likely a layer of more fractured and permeable rock at the bedrock surface that is periodically subject to artesian pressure relative to the ground surface, and this layer is being pressurized by water seeping from the valley wall. The water in this layer seeps both upward to the ground surface, and downward to the underlying rock.

4.2.1.3. South Abutment

At the south abutment, drilling results show an approximately ten meters thick layer of glacial till overlying a three to six meter thick layer of gravel, which overlies bedrock. The ERT (electrical resistivity tomography) surveys, which cover a more extensive area indicate that the glacial till is between 2.5 and 9 meters thick and the underlying gravel layer is between 5 and 12 meters thick [Appendix 3]. The apparently thicker gravel to the southwest may be an artifact of edge effects of the ERT survey and should be verified.

Permeability testing was not undertaken within the granular layer at the south abutment.

The data from the VWPs indicate a piezometric head elevation of 1409.3 meters within the gravel unit, and downward gradients within the formation.

4.2.2. Bedrock Geology, Structure, Permeability and Strength

4.2.2.1. Brazeau Formation

The bedrock lithology encountered where the dam footprint ties into the high ground at the north end of the dam (BH17-07) is very different from that encountered in other boreholes. It is interpreted to be part of the Brazeau Formation, since it lies to the north of the contact shown on the regional geological mapping (Drawing G02). This bedrock consists of hard competent sandstone interlayered with weaker shale and siltstone. It has a very similar appearance to the bedrock exposed in the north bank of the Elbow River downstream of the spillway flip-bucket. Clay-infilled joints and occasional weak seams up to 5 mm thick were noted within the drill core. Because borehole BH17-01 did not penetrate the bedrock due to drilling difficulties, the location of the contact between the Brazeau and Wapiabi formations cannot be refined further; the contact is between BH17-02 and BH17-07.

The RQD (rock quality designation) encountered within the Brazeau Formation is variable, between 50 and 98%, however, the majority is between 50 and 70%. Overall the GSI (geological strength index) is conservatively estimated to be between 37 and 42. For heterogenous rock masses as observed downstream of the spillway flip-bucket, this would be indicative of interbedded sandstone and shale with smooth moderately weathered and altered discontinuity surfaces.

Hydraulic conductivities greater than 1×10^{-3} cm/s were encountered within the top of the bedrock in BH17-07, which is moderately weathered. Between 3 and 13 meters below the top of rock, permeability testing indicates that the hydraulic conductivities are between 6×10^{-5} and 6×10^{-4} cm/s. Open joints were encountered within the test intervals.

One major joint set was identified from the downhole geophysics within BH17-07. This joint set has an orientation of 36°/215°. This is consistent with the orientation of the bedding observed in the outcrop exposed downstream of the spillway flip-bucket.

Given the relatively small sample size a conservative estimate of the friction angle along bedrock discontinuities of 29° was selected for the Brazeau Formation. However, it should be noted that the Brazeau Formation rock is not likely to be exposed due to the depth of overburden materials at the north abutment, except possibly at the spillway flip-bucket location. The shear resistance of the rockmass for the Brazeau shale was estimated as a cohesion of 0.23 MPa and a friction angle of 42°, and for the Brazeau sandstone as a cohesion of 0.38 MPa and a friction angle of 57°.

4.2.2.2. Wapiabi Formation

The Wapiabi Formation is exposed within the Elbow River north and south channel walls. At the upstream end of the north abutment the bedding planes have a similar orientation (20°/060°) to bedding on the south abutment. The rock exposed on the surface of this outcrop contained open fractures and displays slight to moderate weathering. The bedding near the downstream end of this outcrop appears to rotate and the bedding is more steeply dipping, oriented at 80°/060°. In addition, on the south abutment, downstream of McLean Creek, the bedding has also rotated and is oriented at 78°/230. The contact between the Brazeau and Wapiabi Formations crosses the Elbow River immediately downstream of the confluence with McLean Creek. This rotation in bedding is very likely related to the regional thrust faulting as well.

Two joint sets were identified within the Wapiabi Formation from downhole geophysics in BH17-02, These joints sets are oriented at:

- Joint Set 1 - 22°/207°
- Joint Set 2 - 62°/049°

Joint Set 1 has a similar orientation to the joint set encountered within the Brazeau Formation. Joint Set 2 has a similar orientation to the steeply rotating bedding observed in the north channel wall at the downstream portion of the dam site. This indicates that borehole BH17-02 may be close to the contact between the Brazeau and the Wapiabi Formations.

In the remaining boreholes drilled within the Wapiabi Formation in the river channel and on the south abutment, the only discontinuity set noted is oriented at 13°/043° which is similar to the orientation noted in the bedding planes in the exposed bedrock at the south abutment, upstream of the confluence of the Elbow River and McLean Creek.

The RQDs encountered within the Wapiabi Formation are generally more than 85% and the GSI is generally between 65 and 75. A GSI in this range is indicative of blocky rock mass consisting of cubical blocks formed by three interconnecting discontinuity sets and slightly weathered discontinuity surfaces.

The bedrock encountered within the Wapiabi Formation contained few open joints, most containing clay infill less than 3 mm thick, except for weathered zones within the top 3 meters of bedrock. The hydraulic conductivities obtained for the Wapiabi Formation ranged from less than 1 x 10^{-5} cm/s to 2 x 10^{-4} cm/s. The exceptions are within weathered zones at the top of the rock where the hydraulic conductivities are greater than 1 x10^{-3} cm/s. There could also be localized zones with higher hydraulic conductivities. For example, one zone encountered within BH17-03 has a hydraulic conductivity between 2 x 10^{-4} and 6 x 10^{-3} cm/s.

Given the relatively small sample size, a conservative estimate of the friction angle along discontinuities of 34° was selected for the Wapiabi Formation. The shear resistance of the rock mass for the Wapiabi Formation was determined to have a cohesion of 1.20 MPa and a friction angle of 49°.

4.3. Borrow Sources

Based on the results of the BGC (2016) surficial geology map, the BBT (1981) borehole data, and site reconnaissance insight, five granular borrow sites (GD1 to GD5) and two till borrow sites (TD1, TD2) were selected, shown on Drawing G04. Test pits were dug in these areas to better define the nature of the material to complement the available borehole data. A third till borrow area was later identified (TD3) on the north abutment when drilling and seismic geophysical data indicated the till deposit was much thicker than originally anticipated.

The results of the test pit and drilling are summarized in Tables 4.2 and 4.3 on the following pages.

In addition to the test pitting program, a GIS study was performed to evaluate the volumes of granular material that would be submerged beneath the FSL pond. The intent of harvesting granular from the inundated valley beneath the FSL pond area is to both salvage the granular material prior to inundation as well as to increase the volume of the pond for water and/or future sediment deposition. The GIS study used a simplifying assumption that the bedrock elevation beneath the granular material was the same as the river bottom elevation, thus the calculated volume of granular material was the volume between the LiDAR-based ground surface and the river bottom. This is deemed a conservative volume estimate because borehole and test pit data indicate that in some areas the granular deposit is deeper than the current river bottom. Only the fluvial and glaciofluvial deposits were considered in this evaluation. Table 4.1 below shows the results of that study.

Table 4.1: Granular Volumes with Various FSL Pond Elevations

Pond Elevation (m)	Pond Area (m^2)	Granular Volume (m^3)
1406	932,183	693,131
1404	897,021	637,609
1402	838,441	577,667
1400	784,493	520,821
1398	705,602	443,380
1396	598,522	312,536
1394	488,518	258,416
1392	389,755	176,780
1390	286,855	138,242
1388	252,757	114,748
1386	227,167	102,040
1384	138,292	81,207
1382	83,161	50,072
1380	31,273	11,316

Table 4.2: Summary of Granular Deposit Laboratory Results

Borrow Area ID	Overburden Reject (m)		Granular Thickness (m)		Moisture Content (%)		Cobbles[1] (%)		Gravel[2] (%)		Sand (%)		Fines (%)	
	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Typical
GD1	0.8-1.8	1.3	2.8-4.2	>3.0	5-12	8	0-5	5	56-70	64	28-34	30	2-13	6
GD2	0.7-1.8	1.3	1.7-2.8	2.3	6-11	8	0-5	5	42-56	49	31-47	39	11-13	12
GD3	0.0-0.2	0	2.6-6.0	>3.0	3-7	5	30-60	40	76-84	80	15-23	19	1	1
GD4	0.0-0.3	0.3	1.2-4.0+	>3.0	3-10	7	10-40	20	64-66	65	24-28	26	6-12	9
GD5	0.4	0.4	3.1	3.1	2-9	5	0-10	10	58	58	34	34	8	8

[1] Based on visual estimates

[2] Excludes oversized material

Table 4.3: Summary of Till Deposit Laboratory Results

Borrow Area ID	Overburden Reject (m)		Till Thickness (m)		Moisture Content (%)		Cobbles[1] (%)		Gravel[2] (%)		Sand (%)		Fines (%)		Liquid Limit	Plastic Limit	Plasticity Index
	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Typical	Range	Range	Range
TD1	0.2-1.5	0.3	2.8-10.6	6.0	7-25	14	0-20	5	2-18	15	17-30	23	52-81	62	22-48	10-17	8-30
TD2	0.3-0.4	0.3	>3.3-4.2	>4.0	8-22	10	5-20	5	14-17	15	29-33	32	52-54	53	30-40	15	15-25
TD3	0-6.5	2.0	24-29	26	5-40	15	---	---	---	---	---	---	---	---	27-44	12-19	15-25

[1] Based on visual estimates
[2] Excludes oversized material

4.4. Highway and Roads

The MC1 project involves realignment of a portion of the existing Highway 66 adjacent to the Elbow River. The proposed alignment is shown in Drawing R01 to R03, and details are contained in Appendix 4 [Road and Bridge Geotechnical]. The total length of the proposed realignment is approximately 10 kilometers, starting from about 800 meters east of Canyon Creek Rd (SW limit) to the east end of Gooseberry Campground (NE limit). The proposed realignment will traverse along the east side of the Elbow River and the existing Highway 66. The realignment will

- cross the Elbow River near southwest project limit,
- traverse the land near the McLean Creek Campground for the central portion,
- cross McLean Creek just downstream of the fishing pond and
- cross undeveloped forested areas at the southwest and northeast portions.

Large cuts will be required at the approaches to the bridge across the Elbow River at the south west end of the realignment, at the northeast end where it joins Highway 66 and in the central portion at station 7+000. An approach fill will be required at the west end of the bridge.

Seven geotechnical boreholes were drilled at select locations along the proposed road realignment to evaluate the geotechnical characteristics, groundwater conditions, and to provide recommendations on cut and fill sections through soil and rock, and subgrade preparation. This study involved a drilling program and soil laboratory testing, as well as a desktop review of available road profiles, cross-sectional plans, terrain analysis, contour and LiDAR maps, aerial photographs and previous geotechnical studies for the existing Highway 66.

4.4.1. Subsurface Conditions

The borehole locations are shown on the alignment drawings Drawings R01 to R03. The boreholes drilled within the northeast portion of the realignment (Drawing R03) revealed clay till over the entire depth of the boreholes (BH16-07, 13.1 m; BH16-08, 17.8 m). At the McLean Creek crossing, the borehole revealed clay shale at shallow depth (3.0 meters) underlying a surficial fluvial gravel layer as shown in Figure 4.4 below. The thick clay till deposit at the northeast portion and the fluvial gravel within the McLean Creek were also identified in the terrain analysis map by BGC (2016).

The borehole drilled within the middle portion of the realignment (BH16-16) had a thin layer of clay till (to 1.3 meter depth) underlain by gravel which extended to the borehole termination depth of 3.75 meters (Drawing R02). This is generally in accord with the surficial geology observed in the south abutment from the ERT results (Section 4.2.1.3).

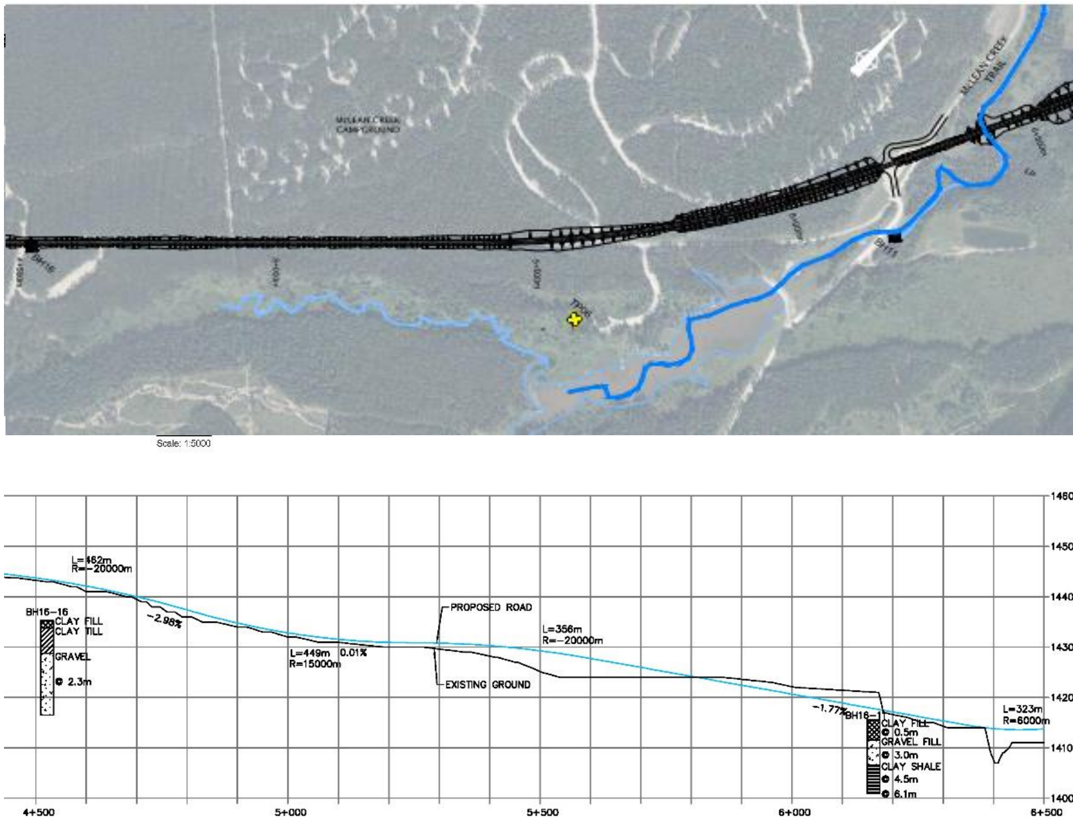

Figure 4.4 Subsurface Conditions - Drawing R02

The boreholes within the southwest portion (BH16-21, -23- -26; Drawing R01) primarily encountered a gravel deposit except for thin embedded layers of silt and clay. This is generally in accord with glaciofluvial deposits identified in the BGC (2016) terrain analysis report within the southwest portion of the proposed realignment. Exposed bedrock and fluvial deposits were also identified in the BGC (2016) map within the river channel and bottom of the west valley wall.

4.4.2. Discussion

Based on the desktop review, it was established that the proposed realignment will traverse mainly through till except at the northeast and southwest portions where it will pass through glaciofluvial deposits. Organic and/or fluvial deposits were identified adjacent to the south side of the central portion.

The potential geotechnical issues identified for the southwest portion of the road realignment are:

- Potential organic deposits along the road realignment
- Construction techniques through bedrock and gravel along the proposed cuts
- Long term stability of the proposed cut slopes, particularly through rocks or fluvial deposits

The middle portion of the realignment is designed to match the existing road grade, and no cut or fill sections are required. Construction will mainly involve grade widening and upgrading of the existing Husky Road. The potential geotechnical issue along the middle portion of the realignment will be the likely presence of organic deposits, particularly along the east side of the realignment.

The northeast portion of the realignment will start from the northeast end of the existing McLean Creek Trail, and will extend to the northeast crossing McLean Creek and traversing through the mainly undeveloped area to join the existing Highway 66 east of the Gooseberry Campground. The northeast realignment is generally designed at the natural ground level with less than 5 meters of cut and fill, except at the height of land at chainage 7+000.

4.5. Elbow River and McLean Creek Crossings

The southwest portion of the realignment connects the existing Highway 66 to the McLean Creek Trail to the east. This portion of the road will cross the Elbow River approximately 1 km from the existing Highway 66 where a high-level bridge approximately 200 meters long is proposed. A three-span bridge is proposed, with bridge piers located on the current river banks (Drawing S01 to S03). The proposed bridge abutment foundations will be supported on the native ground at the east valley slope. At the west valley slope, however, an 18 meter high approach embankment fill is proposed to support the bridge abutment foundations. The realignment will also consist of cut sections through the valley slopes at the west and east sides. The west cut will be approximately 14 meters deep near the west abutment, while the east cut will be 10 meters deep.

An approximately 10 meter high approach fill is proposed at the McLean Creek crossing. A bridge size culvert will be required below the proposed fill at the creek crossing. The installation of this bridge culvert is recommended to be constructed as per applicable sections of the Alberta Transportation (AT) standard specifications.

4.5.1. Subsurface Conditions

Two boreholes were drilled at the east side of the river valley crossing (BH16-21, -23) to depths in excess of the proposed cut depths. The subsurface conditions along the proposed eastern cut is expected to consist of glaciofluvial deposit (sand and gravel) underlain by bedrock, however the depth of the bedrock was not determined in the drilling program.

On the west side of the river valley, very difficult drilling conditions were encountered (sand and gravel with extensive cobbles and/or boulders) so a maximum drilling depth of 1.6 m was attained in BH16-26. The terrain analysis map by BGC (2016) showed glaciofluvial deposit at the valley slopes and narrow exposed bedrock surface at the bottom of the west valley slope. Thus bedrock is anticipated to be encountered on the west cut section.

The subsurface on the west side of the McLean Creek crossing (BH16-11) indicated bedrock at 3.0 meters depth overlain by fluvial deposits.

4.5.2. Discussions

Excavation into the bedrock on the west approach to the Elbow River bridge may require drilling and blasting. For preliminary concept /design purposes, the rock cuts are envisioned to be at a slope angle of about 60 degrees, and is similar to previous information for the area. The cuts through the glaciofluvial deposits would be about 4H:1V. It is also recommended to consider wide and deep ditches at the bottom of the cut slopes to intercept the anticipated rock falls and gravel debris.

At the Elbow River bridge, the proposed embankment fill should be keyed in to the existing valley slope by benching the existing slope one level at a time starting from the ditch bottom. Erosion protection such as rock riprap is recommended at the base of the proposed embankment fill and river channel. For preliminary design purposes, the proposed head slope of the approach fill could be steeper. The foundation for the proposed bridge abutment at the west valley side should extend into the native ground to minimize the down drag load and anticipated settlement of the embankment fill. In addition, the embankment fill should be planned well ahead of the bridge construction to allow the majority of the settlement to occur prior to the installation of the bridge abutment foundations.

The potential geotechnical issues identified for the bridge location are:

- Stability of the proposed cut slopes, particularly through rocks or fluvial deposits
- Stability of the existing valley slopes upon disturbance with the proposed construction area
- Stability of the embankment fill at the proposed bridge location
- Erosion protection at the base of the approach fill

5. Project Risk and Workshops

5.1. Summary of Workshops

The team adopted a risk-based design approach which focused the effort on the design element presenting the highest risk in order to complete the design concepts in a shorter period of time and reduce uncertainty related to the cost and schedule of construction. To do so, the team conducted the following series of workshops:

- Workshop 1: Risk Analysis, Information Phase and Team Alignment
- Workshop 2: Value Engineering and Risk Analysis
- Workshop 3: Value Engineering Evaluation Phase (Dam and Structures)
- Workshop 4: Value Engineering Evaluation Phase (Bridge, Roads and Structures Relocation)
- Workshop 5: Constructability Review and Risk Analysis

The project team members that participated at each workshop varied depending on the content and objectives. The primary core team members that attended most of the workshops were:

- Alberta Transportation
- Opus
- Hemmera
- Hatch
- Cascade Consultants
- Northwest Hydraulics

The details of the workshops and the outcomes were documented and are presented in Appendix 5 [Workshops and Value Engineering Reports]. A summary of the primary outcomes for each session is presented below.

5.2. Workshop 1: Risk Analysis, Information Phase and Team Alignment

This workshop was planned to facilitate the delivery of the project within a short period of time, with a focus on the major elements contributing to project cost, schedule and constructability.

The workshop's main objectives were:

- Identify information gaps
- Identify roles and responsibilities of the consulting team
- Conduct Risk Analysis and incorporate risk-based design concepts
- Identify action items to facilitate project delivery

A total of 21 participants attended the workshop on November 27-28, 2016. Participants represented the owner and consulting team.

The workshop's major outcomes include:

- Using risk-based design, the following items were identified as priorities for design advancement:
    - Dam and Structures
    - Schedule/Cost Estimating
    - Geotechnical
    - Facility Relocation
    - Roads and Bridge
    - Environmental
    - Hydrology
    - Operation

- Team alignment was achieved by identifying the roles and responsibilities for the various team members
- The team completed risk analysis for the project and identified mitigation strategies which were later adopted during the design development stage.

5.3. Workshop 2: Value Engineering and Risk Analysis

The second workshop's objective was to conduct value engineering to identify various options/creative ideas to further the project design and advance the development of the cost estimate and schedule. The workshop was conducted over two days, with a different focus on each day.

- Day One: Roads, bridges, and facilities relocation
- Day Two: Dam, structures, debris, and bedload design

A total of 25 participants attended the workshop on December 8-9, 2016.

The workshop's major outcomes:

- Day One—Road and Bridge, and facilities relocation: the team identified 15 options/design recommendations for the road and bridge components
- Day Two—Dam, structures, debris, and bedload design: the team identified 40 various options/ideas/design recommendations
- The team agreed to analyze and develop these ideas, which were then evaluated during the subsequent workshop, based on the following criteria:
    - Feasibility
    - High-level cost estimates
    - Impact on functionality and cost
- Facilitate preparation for the evaluation workshop (workshops 3 and 4) once all the required information is available. The technical team conducted the evaluation session to decide on the highest value option (for various components).

5.4. Workshop 3: Value Engineering (Evaluation Phase for Dam and Structures)

Following Workshop 2 (Value Engineering), the project team developed the design concepts to the level required for conducting the evaluation phase. The workshop to complete the evaluation phase for the dam and structures was conducted on January 27, 2017. This workshop comprised of a design meeting for the dam and structures working group.

The team members reviewed and commented on ideas from the Value Engineering workshop, and determined whether they would be implemented or if they were unfeasible. The team also identified various constructability issues that need to be addressed in the upcoming workshops in order to facilitate the design progress.

The workshop's major outcomes were:

- The design team discussed various components and recommended that the service spillway be on the left abutment
- The diversion during construction and low-level outlet is recommended to be twin tunnels, 5-6 meters in diameter on either the left or right abutment
- Tunnel construction to be used for the low-level outlet to reduce the risk associated with construction and a possible failure of the proposed box conduit through the left abutment soil
- Auxiliary spillway to be on the right abutment. It should also be widened as much as possible
- Upstream cofferdam to be designed for a 1-in-50-year storm event
- Envisioned the apparent buried gravel valley in the left abutment could be treated from the surface with jet grouting or other types of soil treatment

- The team reviewed value engineering ideas and evaluated them
- The team will optimize the dam configurations by considering the auxiliary spillway width, service spillway width, and low-level outlet tunnel diameter
- Consider an emergency relief spillway during construction of the upstream cofferdam to pass flood greater than 1-in-50 years

5.5. Workshop 4: Value Engineering (Evaluation Phase for Bridge and Road component)

Following Workshop 2 (Value Engineering) the project team developed the design to the level required for conducting the evaluation phase. The team met on February 9 to complete the evaluation phase for the bridge and road component and structures relocation.

The workshop began with reviewing the current design and various options. The reviewed sections were then grouped into three categories: South section, Middle section; and North section. The team then decided to eliminate some options for various reasons, including:

- Options D and E were eliminated due to high cost and lower design standards, as well as constructability issues
- Options A5 and A6 were eliminated due to the existence of better alternatives for a lower cost with similar functionality
- Options C1 and C4 were also eliminated due to the existence of better alternatives for a lower cost with similar functionality

The team proceeded with evaluating the identified options in each section (except the middle section, in which case only one option was evaluated). The evaluation process followed a structured analytical hierarchy process. The analysis resulted in selecting the highest value options. The selected options were: alignment A2-South Section, B1 and C3-North Section.

5.6. Workshop 5: Constructability Review and Risk Analysis

The workshop was conducted March 9-10, 2017. A total of 25 participants attended the workshop with the following main objectives:

- Present the current design
- Conduct a constructability review
- Update the project risk analysis

The workshop's major outcomes were:

- The current design as of March 9-10, 2017, was accepted by the project team and assumed frozen at this stage to facilitate the completion of the first cost estimation
- Various action items and information gaps were identified to complete cost estimation
- Risk analysis was updated, and the expected cost of risk was deemed to be approximately $58 million

6. Dam Design

6.1. Flood Routing Assessment

6.1.1. Introduction

A key part of the MC1 assessment involved investigation of the ability of the project to:

1. Mitigate the impacts of the design flood event (the 2013 event) on downstream infrastructure
2. Safely pass a much larger design event approaching the PMF for the site

To assess the project's ability to do so, a flood routing model was set up to simulate the passage of various flood events, ranging from 20-year flows up to the project PMF. All major discharge structures proposed for the McLean Creek project were incorporated into the simulation runs, including:

- Two, six meter diameter low level diversion/outlet tunnels at invert Elevation 1384 m (approximately four meters above river bed level)
- A service spillway 40 meters wide, with an ogee crest level at Elevation 1424.5 meters
- An auxiliary spillway (200 meters wide) with an overflow Elevation of 1426.1 meters

The results are summarized below for the preferred concept. Similar assessments were made for the AMEC 2014 concept using updated inflow hydrographs, storage curves and routing assumptions.

6.1.2. Methodology and Data

6.1.2.1. Routing Methodology

Using the inflow hydrographs and discharge capacity estimates developed by others, and described previously, the outflow and reservoir stage hydrographs were determined based on standard level pool routing techniques. Flows were routed according to the following formula:

$$\left[\left[\frac{I_2+I_1}{2}\right]-\left[\frac{O_2+O_1}{2}\right]\right]*\Delta T=S_2-S_1$$

where:	ΔT	=	Routing interval - s
	S_1 and S_2	=	Storage at the beginning and end of period – m^3
	I_1 and I_2	=	Inflow at the beginning and end of period - m^3/s
	O_1 and O_2	=	Outflow at beginning and end of period - m^3/s

6.1.2.2. Hydraulic Data

The hydraulic data available for the assessment included the following:

- Project inflows
- Project storage curve
- Discharge rating curves for each hydraulic structure

Each are described below.

6.1.2.3. Project Inflows

Project inflows were based on:

- Hydrological assessments originally conducted by AMEC in 2014 for this project
- A discharge hydrograph recorded at the Sarcee Bridge WSC gauge for the 2013 flood event - this inflow represents the flood volume that will need to be managed at the City of Calgary's Glenmore Reservoir, and provides a conservative estimate of inflows for the MC1 project
- A PMF study for the Elbow River conducted by Stantec as a part of the SC1 project in 2015

Figure 6.1 illustrates the final inflow hydrographs used in the design and evaluation of the MC1 concept(s).

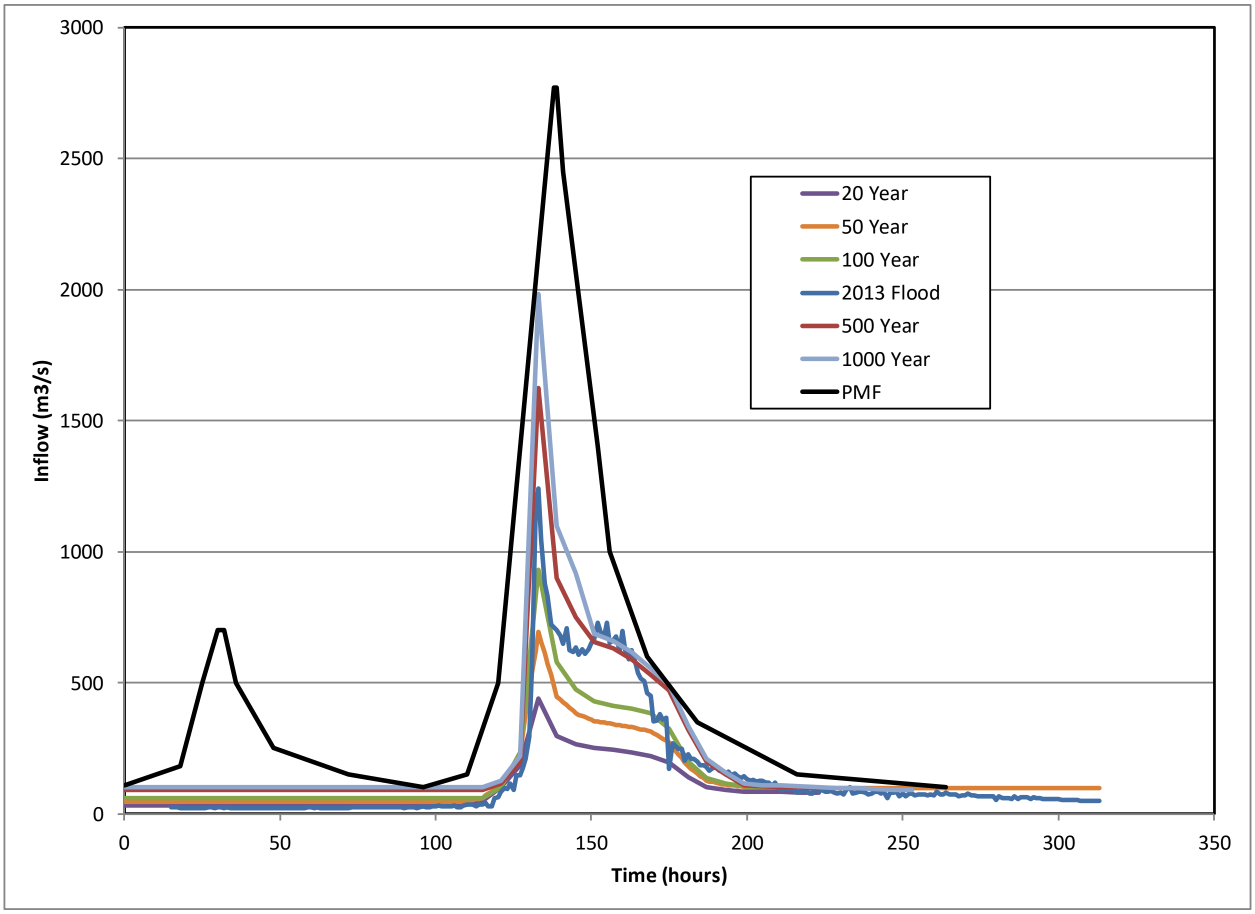

Figure 6.1: Project Inflows

6.1.2.4. Storage Relationship

The storage volume relationship for the MC1 project was discussed in Section 2.0 with details in Appendix 1. The final curve used in these routing studies is shown in Figure 6.2 below.

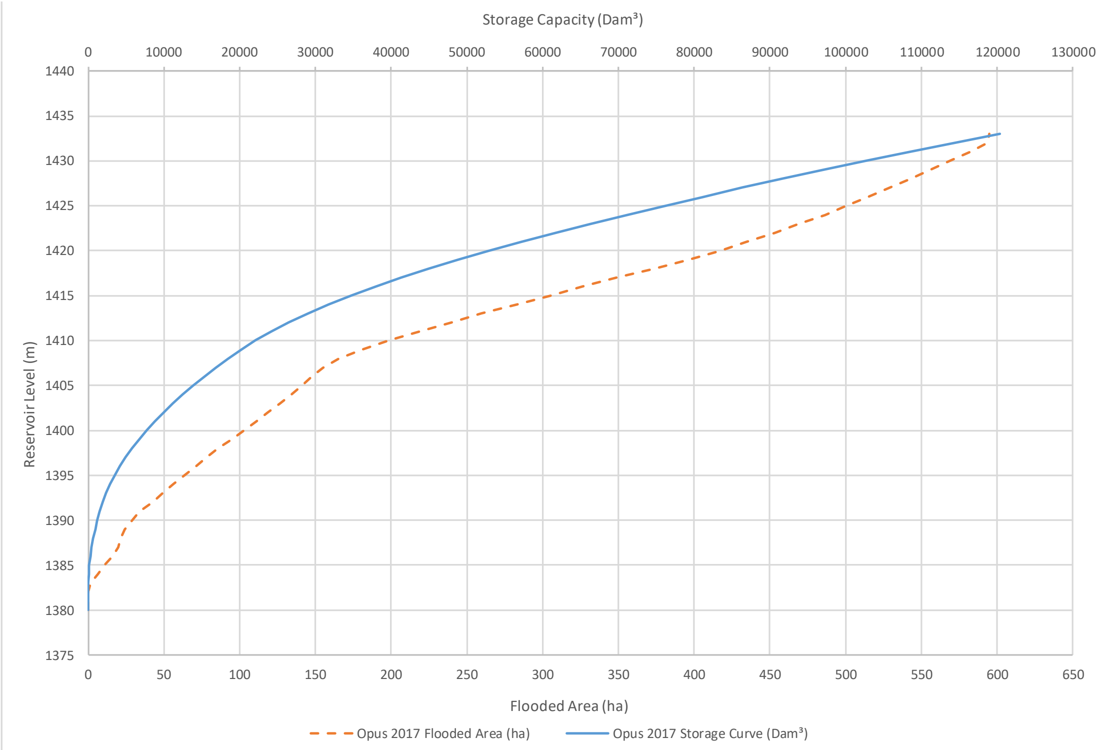

Figure 6.2: McLean Creek Stage - Storage Relationship

6.1.2.5. Discharge Rating Curves

The discharge capacity for the MC1 project is provided by (in order of mobilization of spill capacity):

- the twin six meter diameter diversion/low level outlet tunnels
- the 40 meter wide service spillway
- the 200 meter wide auxiliary spillway

Elevation-discharge rating curves were developed for each component using standard empirical techniques and methods.

The project's low-level outlet (and primary regulating structure) releases flow through two parallel six meter diameter tunnels. Flow through each tunnel outlet is regulated using a movable 4.7 x 4.7 meter vertical lift gate. The free discharge rating curve with both tunnels operating is shown in Figure 6.3.

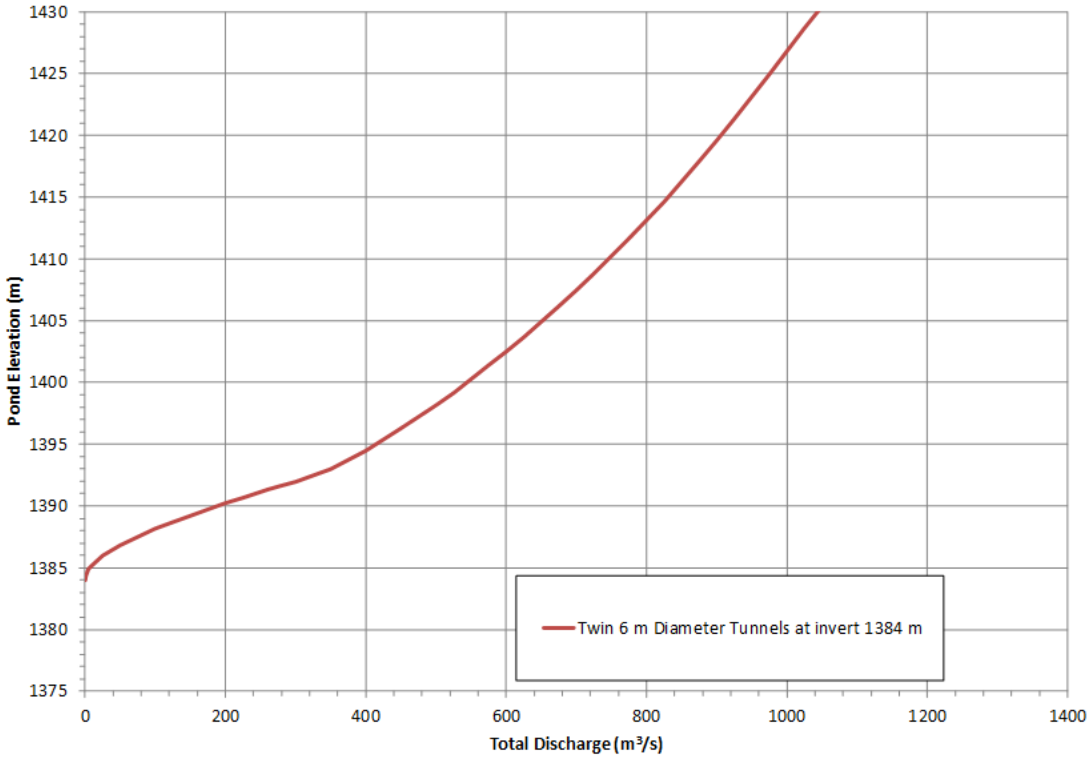

Figure 6.3: Rating Curve of Low Level Outlet Tunnels

The designed Service Spillway consists of an ungated free overflow ogee weir that is 40 meters wide. The crest elevation for the spillway ogee weir is Elevation 1424.5 meters, and it only conveys flood water for events greater in magnitude than the 2013 event. The spillway consists of a free overflow crest, a flat chute section (which converges in width from 40 meters at the upstream end, to a constant 28 meters), followed by a steep section of the chute which terminates in a flip-bucket where the chute meets the Elbow River. Finally, during very large floods which exceed the capacity of the service spillway and low-level outlet, the reservoir will increase until the water is released through the auxiliary spillway, located on the right abutment of the project. The auxiliary spillway consists of a low section in the right embankment dyke that is approximately 200 meters wide, with an invert elevation of 1426.1 meters (2.9 meters below the dam crest). The rating curves for both the service spillway and the auxiliary spillway are shown in Figure 6.4.

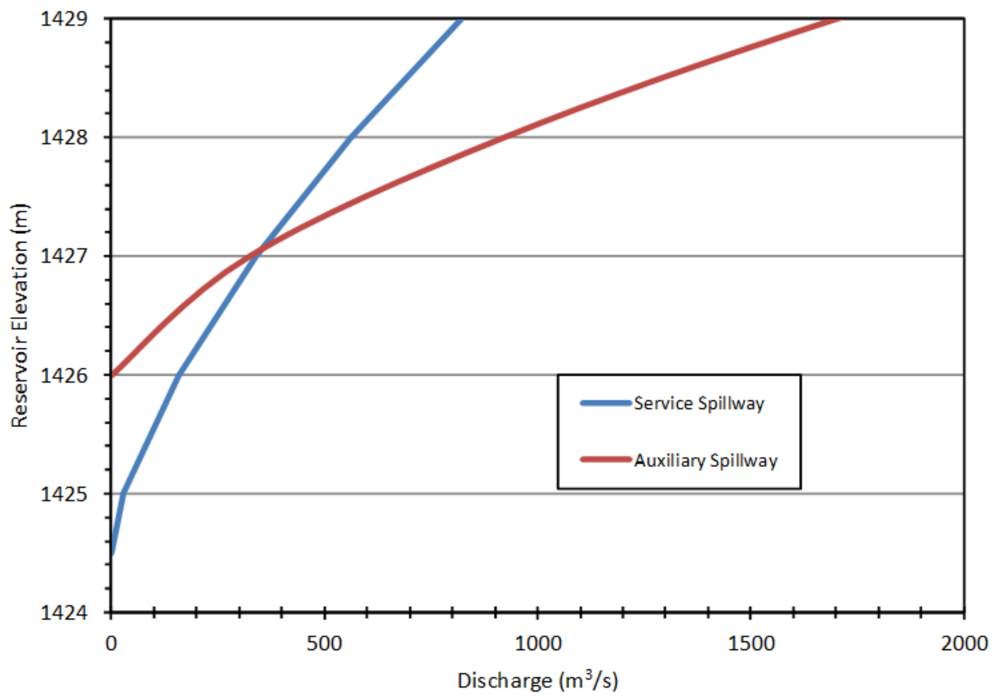

Figure 6.4: Rating Curves of Service Spillway and Auxiliary Spillway

6.1.3. Operational Scenarios During Construction

During the construction of the MC1 project, a two phased or two stage diversion strategy would be implemented:

- **Stage 1:** During this initial phase of diversion, all flow would remain in the natural river channel. The tunnels would be constructed during this initial phase, along with the service spillway. Initial work would also be started on the smaller embankment dykes located outside the river valley. A very small temporary cofferdam would be constructed at the entrance and exit to the tunnels in order to provide protection against high water levels that may occur at this time. The tunnels and gate assemblies would be completed during this stage. Figure 6.5 summarizes the natural water levels expected in the channel at the location of the tunnel inlets. As shown, the proposed invert level at the tunnel entrance (Elevation. 1384 meters) is actually above the natural 100-year flood level at this location.
- **Stage 2:** During Stage 2 of diversion, the river would be closed with a cofferdam at the upstream toe of the Main Dam, the tunnel gates would be fully lifted, and all river flow would be diverted through the open tunnels. The gates would remain lifted throughout this phase of diversion, and work would continue on the construction of the Main Dam and the adjacent embankment dykes. Should a major flood occur at this time, water levels would rise upstream of the cofferdam. Expected water levels immediately upstream of the tunnel inlets are as follows:
    - 20 year flood: Elevation 1393.8 meters
    - 50 year flood: Elevation 1398.1 meters
    - 100 year flood: Elevation 1402.3 meters
    - 1000 year flood: Elevation 1415.5 meters

The crest of the upstream cofferdam for the main embankment dam construction is set at elevation 1399.1 meters to provide protection against a flood event with an annual exceedance probability of 0.02 (50-year return period), with a one meter freeboard. The 1:50 year flood event was selected as the design event to limit the volume of water that could be stored behind the cofferdam, and released uncontrollably in the event that a larger flood occurs and causes overtopping of the cofferdam. For the 1:50 year flood event, the water stored behind the cofferdam would be limited to 6,700 dam^3, and the sudden release of this volume would be handled relatively easily by the Glenmore Reservoir.

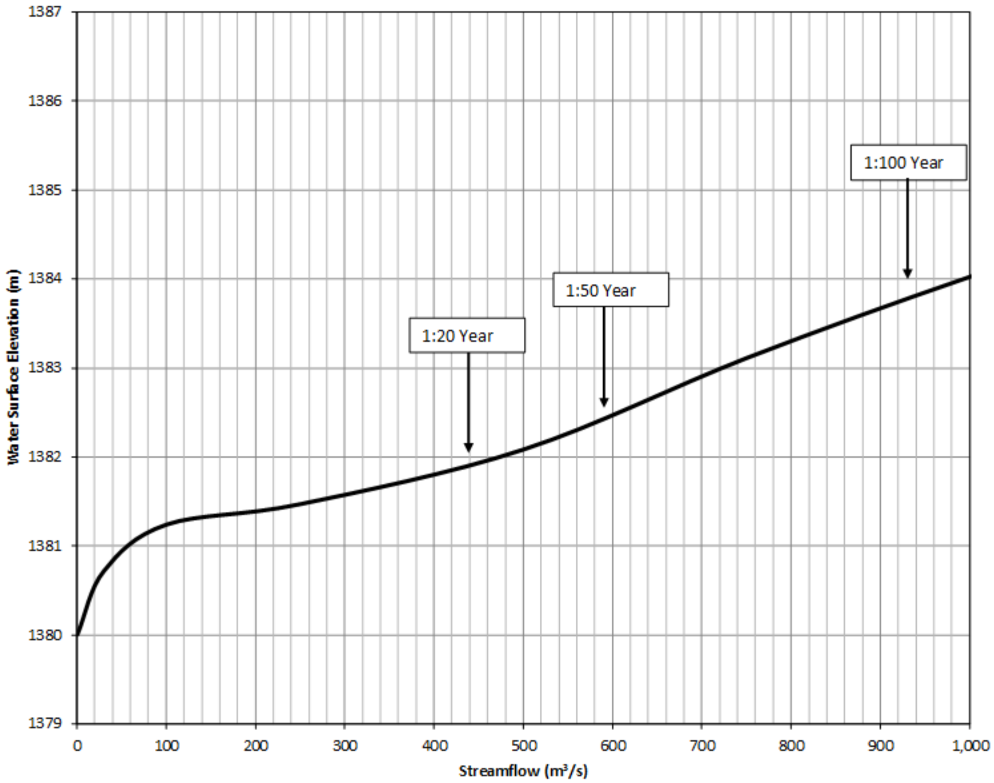

Figure 6.5: Natural Rating Curve at Tunnel Inlet Site

6.1.4. Post Project Operation – Flood Passage

6.1.4.1. Operational Assumptions

Once fully operational, the project will be required to pass various levels of flood during its operating life. The primary objective, from a flood handling standpoint, will be to temporarily store water by reducing/regulating outflows during a large flood event to a release rate that is manageable by downstream infrastructure (i.e. Glenmore Reservoir).

The MC1 concept has been developed considering the June 2013 flood equivalent as its design standard in terms of rate of inflow and flood volume, as it was for the AMEC (2014a) concept. Earlier studies by AMEC (2014a) identified a need to store up to 68,000 dam^3 of water during the passage of this event. This storage requirement was assessed taking into consideration that outflows at the Glenmore dam would likely be restricted to 170 m^3/s during such an event to protect downstream residents and infrastructure. Current analyses by Stantec for the SR1 site have identified a need to store a total volume of water of approximately 70,200 dam^3 at the SR1 site during the passage of the 2013 design event, assuming that an additional storage of 10,000 dam^3 is available at the Glenmore reservoir to also attenuate the flood. This more stringent criteria was also adopted in the design of the current MC1 concept. Outflows from the MC1 facility would be larger than the 170 m^3/s Glenmore dam outflow, and the differential in flow between the MC1 outflow and the Glenmore Dam outflow would then be stored within the City of Calgary's Glenmore reservoir. Over the approximately 65 hour period in which the 2013 inflows exceed the critical flow of 170 m^3/s, this would require that MC1 releases be limited to an average flow of approximately 212 m^3/s (i.e. the differential between these two flows equates to approximately 10,000 dam^3 over this period). The 10,000 dam^3 stored within the Glenmore dam, along with the approximately 70,000 dam3 stored within the MC1 facility would be sufficient to manage the 2013 event. The retention of approximately 70,000 dam^3 of storage above the project FSL El. 1395 m would require that the reservoir surcharge to El. 1424.4 m, based on the project storage volume curve.

Therefore, the operational strategy followed in simulating flood passage included the following steps:

- The starting reservoir level for all flood events at the MC1 project was set at the normal FSL of Elevation 1395 meters.
- As flows rise, it was assumed tunnel gates would be raised to match outflow with inflow until inflows began to exceed approximately 212 m^3/s. Initially, the gates would be raised in a single tunnel until flows through that tunnel reached 106 m^3/s, and following this, flows through the second tunnel would be mobilized until outflows through it also reached 106 m^3/s. As noted above, operation at this restricted outflow is necessary to help manage the incoming flood at the Glenmore Dam.
- At larger inflows, average outflows would be restricted to approximately 212 m^3/s, and the reservoir level will begin to rise as water is stored in the reservoir. The reservoir level would be allowed to rise to Elevation 1424.4 meters under this operating condition. This would temporary store up to 70,000 dam^3 of flood water.
- If the reservoir level continues to rise above el. 1424.4 meters, the tunnel gates would be opened further to prevent additional surcharge at the MC1 project. This mode of operation would continue until the gates were fully raised. When the gates are fully raised, the discharge capacity through both tunnels would approach 830 m^3/s. If inflows continue to cause the reservoir to rise, the service spillway will begin to operate once the reservoir level exceeds Elevation 1424.5 meters.
- If the reservoir level continues to rise, the auxiliary spillway will begin to pass flows once levels exceed Elevation 1426.1 meters.

6.1.4.2. Normal Flood Passage

An initial series of runs was undertaken to demonstrate the performance of the MC1 facility in passing smaller and more routine flood events such as the 20-, 50-, and 100-year floods. As noted above, in each of these runs, the reservoir level was maintained at Elevation 1395 meters initially, but allowed to surcharge above this level once inflows began to exceed 212 m^3/s. The resulting hydrographs are shown in Figures 6.6 to 6.8 below.

In reviewing each figure, the following general observations can be made:

- In each case, maximum reservoir outflows do not exceed the specified maximum of 212 m^3/s.
- For the 1:20 year flood, the maximum reservoir level reached is Elevation 1404.7 m, a surcharge of approximately 9.7 meters from the normal FSL. This would store approximately 9,900 dam^3 of the incoming flood volume. The level would remain above Elevation 1395 meters for approximately three days.
- For the 1:50 year flood, the maximum reservoir level reached is Elevation 1413.1 m, a surcharge of approximately 18.1 m from the normal FSL. This would store approximately 25,800 dam^3 of the incoming flood volume. The level would remain above 1395 meters for approximately six days.
- For the 1:100 year flood, the maximum reservoir level reached is Elevation 1419.8 meters, a surcharge of approximately 24.8 m from the normal FSL. This would store approximately 48,600 dam^3 of the incoming flood volume. The level would remain above Elevation1395 m for approximately 8.5 days.

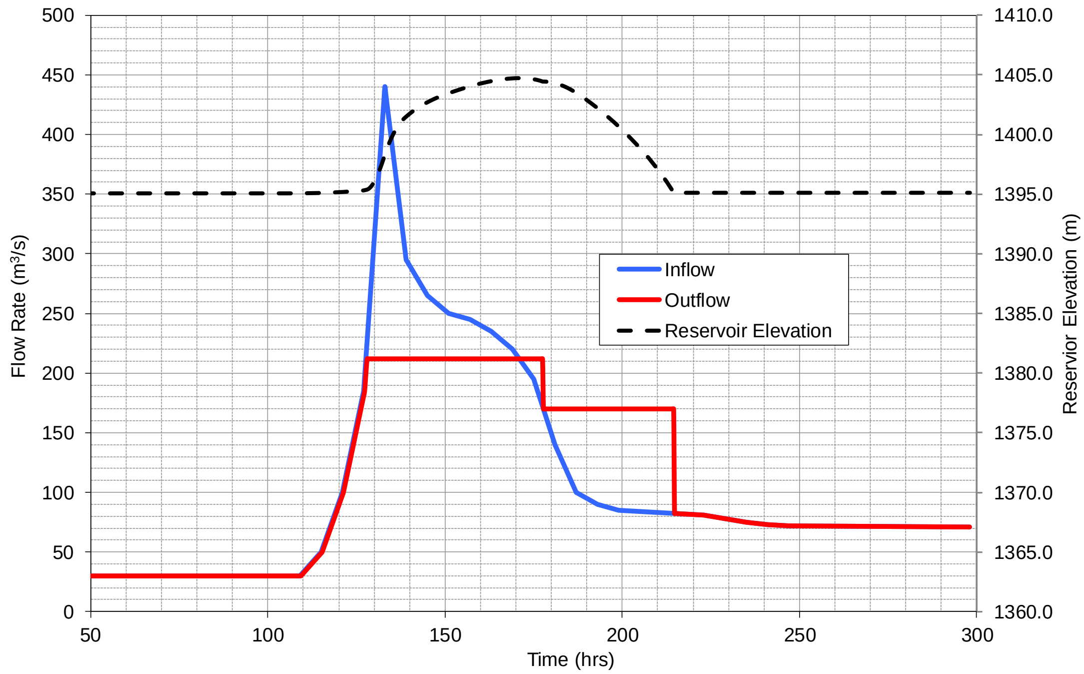

Figure 6.6: Flow Rates and Reservoir Pond Elevation vs Time During Passage of 1:20 Year Flood Event

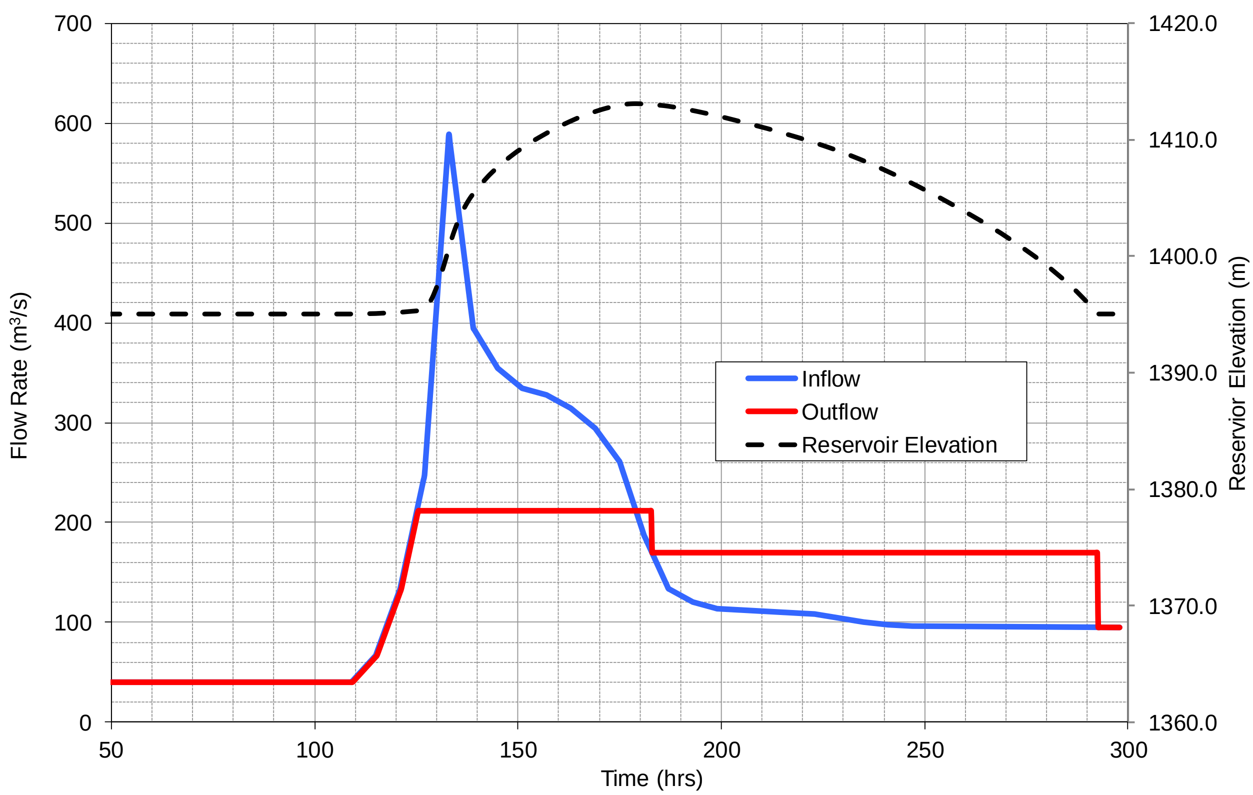

Figure 6.7: Flow Rates and Reservoir Pond Elevation vs Time During Passage of 1:50 Year Flood Event

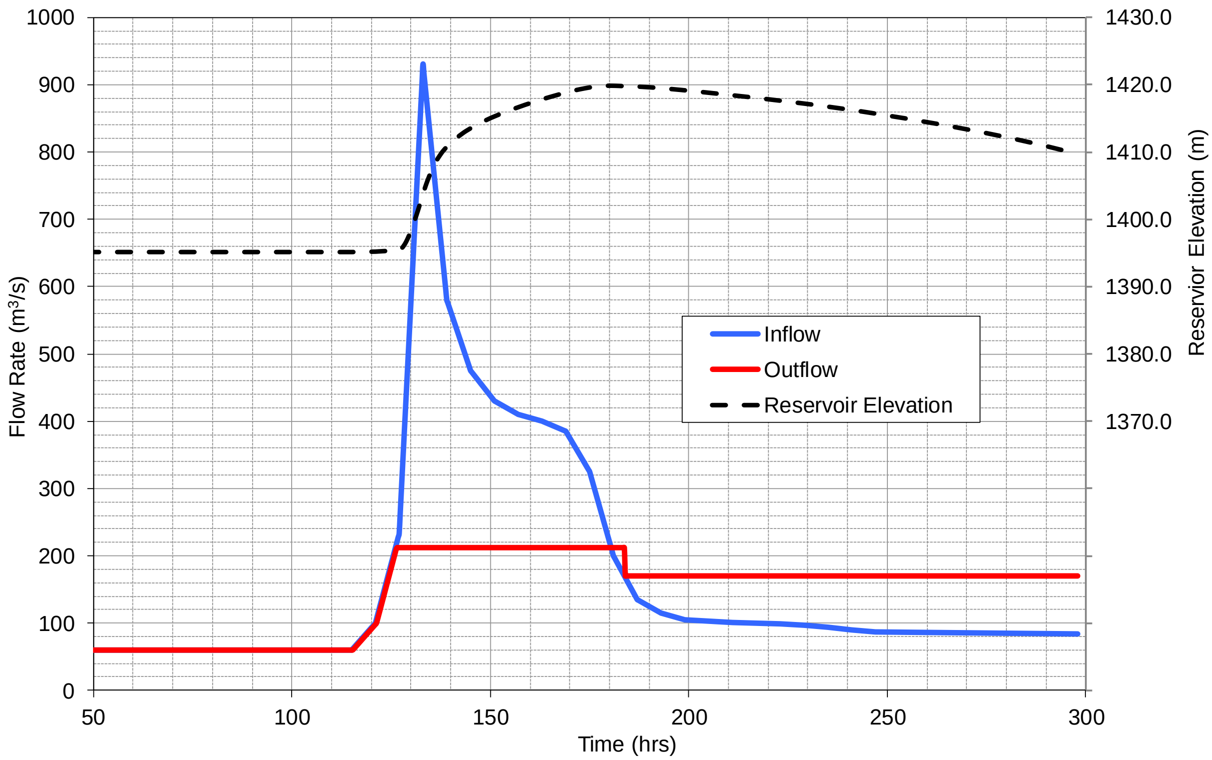

Figure 6.8: Flow Rates and Reservoir Pond Elevation vs Time During Passage of 1:100 Year Flood Event

6.1.4.3. 2013 Flood Event

The next run undertaken simulated performance of the dam during the passage of a flood event that would be equivalent in peak inflow and flood volume to that of the 2013 event. This is the design event for the MC1 facility in terms of its flood mitigation capabilities. The estimated return period for this event is approximately 200 years. The results of this routing scenario are shown on Figure 6.9. Routing of the flood event again resulted in an average outflow from the facility of approximately 212 m^3/s – all of which would be passed through the regulated tunnel outlets.

As can be seen, the maximum reservoir level reached was Elevation 1424.4 meters, the total flood volume stored was equal to the required 70,000 dam^3, and the total volume stored (including dead storage) would be 73,500 dam^3. The level would remain above Elevation 1395 m for approximately ten days. It should be noted that this would provide equivalent protection to the City of Calgary to that of the SR1 project.

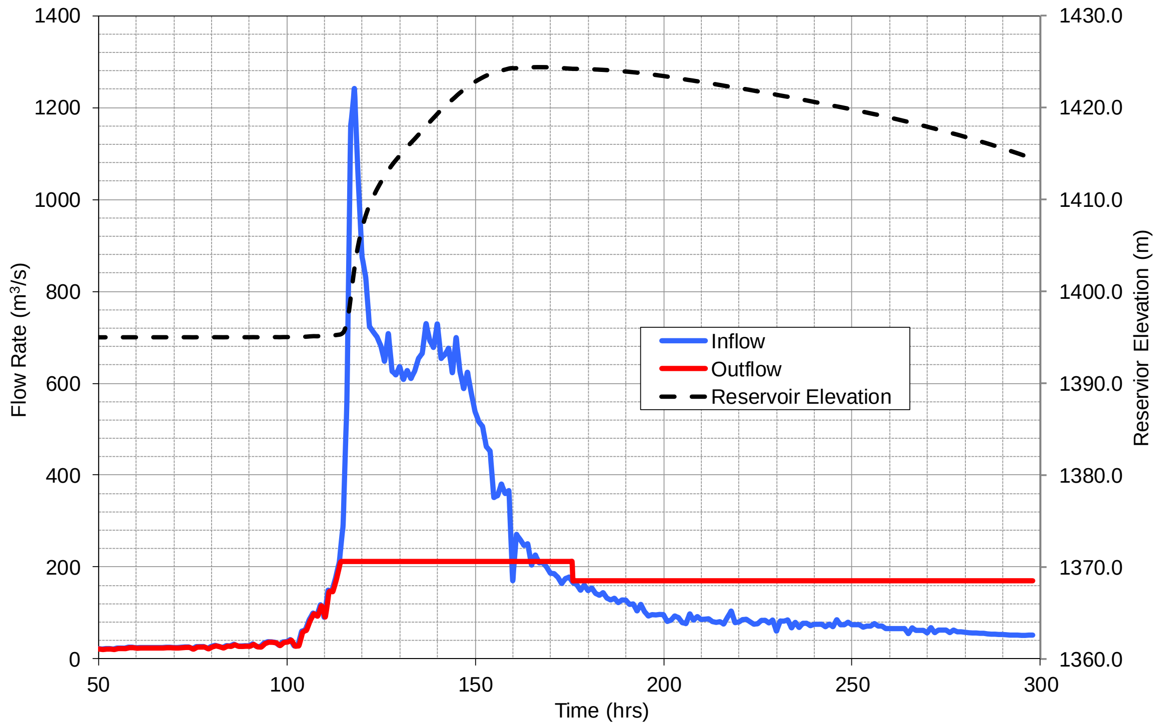

Figure 6.9: Flow Rates and Reservoir Pond Elevation vs Time During Passage of the 2013 (Design) Flood

6.1.4.4. 1000 Year Event

The 1000-year flood event was routed through the complete facility, using identical flood routing assumptions. The results of this routing scenario are shown on Figure 6.10.

The simulation results indicated the following:

- The reservoir would rise to a peak Elevation of 1424.5 m before dropping again. At this level, the total volume stored would be close to 73,600 dam^3
- Outflows through the tunnel would initially be limited to approximately 212 m^3/s, and would be held at this level for approximately 24 hours. Once the reservoir level begins to exceed Elevation 1424.4 meters, the gates would be opened to maintain the level at or below this important design elevation. This would result in a peak outflow from the MC1 facility of approximately 830 m^3/s.
- The simulation implies that the 1000-year flood could be managed without mobilizing the service spillway. Peak water levels would be just at the crest elevation of the ogee weir.
- Without the attenuating effect of the dam, flows from a 1000 year flood would be almost 2000 m^3/s, whereas with the dam, 70,100 dam^3 would be temporarily stored, and maximum flows downstream of the dam would be reduced to approximately 830 m^3/s rather than the almost 2000 m^3/s that would otherwise be experienced without the MC1 dam.

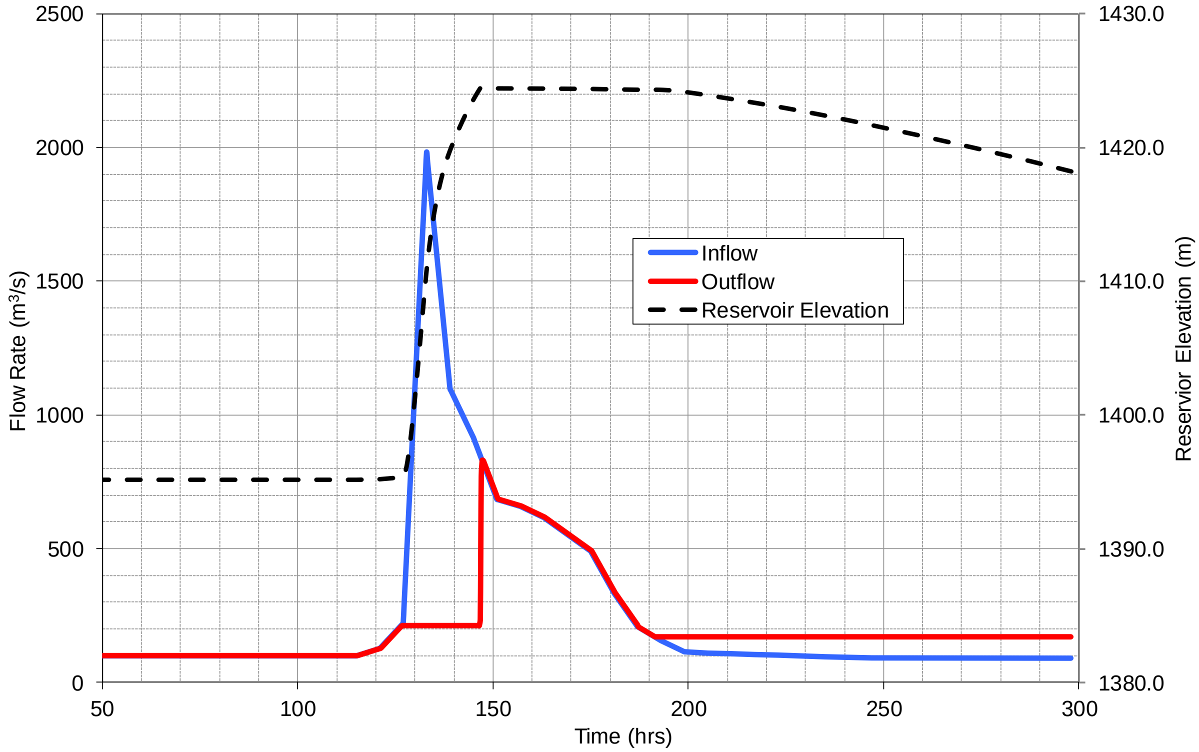

Figure 6.10: Flow Rates and Reservoir Pond Elevation vs Time During Passage of the 1:1000 Year Flood

6.1.4.5. PMF Event

A final run was then undertaken to evaluate the performance of the facility during passage of the project PMF. Using the model, the PMF hydrograph for the facility was routed through the complete facility, using identical flood routing assumptions. The results of this routing scenario are shown on Figure 6.11.

The run results indicated the following:

- The reservoir elevation would rise to a peak level of Elevation 1428.1 meters before beginning to drop again. At this level, the total volume stored would be just over 93,000 dam^3.
- Outflows through the tunnel would initially be held at 212 m^3/s during the passage of the initial snowmelt peak. The basin response to the PMF rainfall would require the tunnel gates to be fully opened, and the reservoir level would continue to climb, mobilizing first the service spillway, and after that, the auxiliary spillway.
- The peak outflow from the MC1 facility would reach 2600 m^3/s. Peak outflows through the tunnel would reach 1000 m^3/s, peak outflows through the service spillway would reach 600 m^3/s, and peak outflows through the auxiliary spillway would reach 1000 m^3/s. Flows through the auxiliary spillway would be required for approximately 24 hours.
- At the peak of the flood, the reservoir level would reach Elevation 1428.1 meters, and would be approximately 0.9 m below the crest of the dam, which has been set at Elevation 1429 meters. This amount of freeboard would be sufficient to contain any coincident wind and wave effects. Initial runup calculations were undertaken as a part of this study, based on extreme wind data available for the Springbank A meteorological station. This data suggests the 1:2 year windspeed at the facility would be approximately 70 km/hr from the critical West direction, and this would result in a runup of approximately 0.7 meters.

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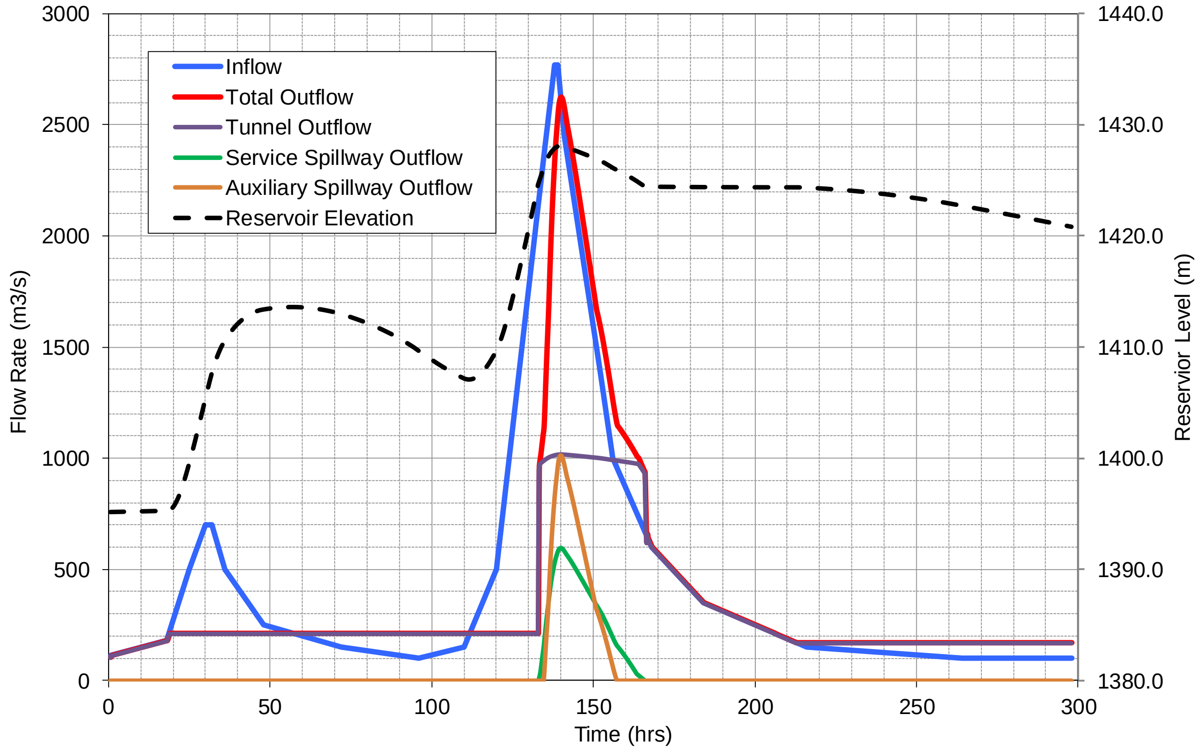

Figure 6.11: PMF Passage with 200 m Auxiliary Channel

6.1.5. Summary

In summary, a routing model has been developed and used to evaluate the hydraulic performance of the proposed flood mitigation scheme. The results of these runs are summarized in Table 6.1 below.

Table 6.1: Summary of Flood Passage

Description (Peak Values)	Floods				
	20-year	100-year	Jun-13	1000-year	PMF
Peak reservoir inflow (m^3/s)	440	930	1240	1984	2770
Tunnel outlet structure peak discharge rate (m^3/s)	212	212	212	830	1000
Service spillway peak discharge (m^3/s)	0	0	0	0	600
Auxiliary earth channel peak discharge (m^3/s)	0	0	0	0	1000
Maximum reservoir water surface elevation (m)	1404.7	1419.8	1424.4	1424.5	1428.1
Maximum total contained water volume (dam^3)	13,400	52,100	73,500	73,600	93,000

6.2. Dam and Hydraulic Structures

A number of alternatives were considered for the main dam; river diversion during construction and hydraulic structures to control and release flood events as well as normal river flows.

These are summarized in Appendix 6A [Dam and Hydraulic Structures].

The concluded general arrangement is shown below in Figure 6.2 and has the following primary flood control structures:

- Main dam located in the river valley and the left (north) and right (south) abutment dykes
- A service spillway located at the extreme north end of the left abutment dyke
- Twin diversion/low level outlet tunnels located in the right abutment
- An auxiliary spillway located near the south end of the right abutment dyke

Each structure is described in the following sections.

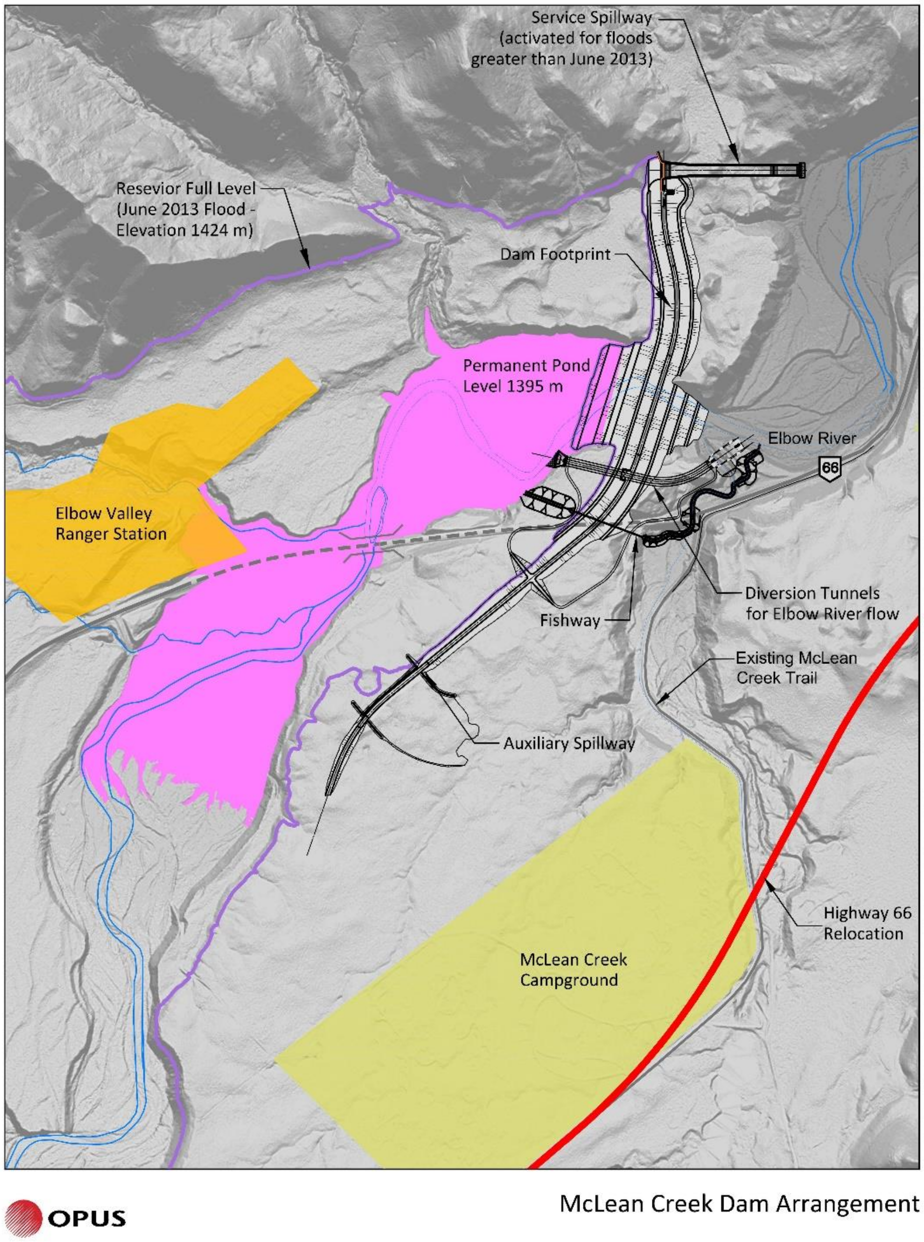

Figure 6.2 – Dam General Arrangement

6.2.1. Dam Foundation Bedrock and Soil

The initial AMEC 2014 study indicated a relatively simple site geology consisting of a bedrock river valley for the main dam location with clay till over bedrock for the abutment dykes. However, following the field and seismic testing (Section 4) deep gravel deposits have been identified in the abutments making the site geology complex. Particular details to address these gravel deposits with cut-off walls have been rationalized.

The foundation preparation in the river bed will consist of removal of existing riverbed alluvium to expose the top of bedrock under the footprint of the dam. Under the central core of the dam, weathered rock will be excavated to a depth of three meters to expose sound bedrock. Here the bedrock will be treated by slush grouting joints in the rock.

Based on the available information and experience, the ‘valley’ section would be excavated back into the slopes with benches at an average slope of 1H:6V in rock and 2.5H:1V in soil. A three meter wide bench will be created for every six meters vertical excavation of rock for the central core area for construction staging and equipment access (Drawing D03-A).

In addition to the foundation surface treatment, the bedrock will be drilled and grouted creating a single line grout curtain in the rock consisting of 20 meter deep primary and secondary grout holes (Drawing D03). A grouting program such as envisioned for this project is conventional for dam foundations, and is intended to seal any large open joints in the rock minimizing seepage through the dam foundation from upstream to downstream. Based on the available test drilling, it is anticipated the foundation rock has a low permeability and the grout losses due to adsorption will be low to very low.

6.2.2. Main Dam and Abutment Dykes

A conventional zoned earthfill dam with a central clay core, filters and shells has been selected for the main dam and abutment dykes (Drawing D02). Borrow pit investigations have indicated sufficient volumes of granular and clay till fill materials to construct the dam.

The core will be impervious fill (Zone 1A) clay till material derived from required excavations and from borrow sources close to the dam. The clay till has sufficient clay content to form a low permeability zone and yet have an adequate strength. The core has a bottom width of 35 meters where the core contacts the prepared rock foundation, and as such, the core to maximum head ratio is 0.7:1 (or 70%) which is a typical ratio of zoned earth dams.

For large dams, cores with a minimum width of 25% to 50% of the water head are recommended in the literature and the importance of following the precedent practice is emphasized. In this respect, the following criteria have been considered, as suggested by various sources (Reference 9):

- The minimum base width of the core to be 50% of the water head
- At interface, the minimum contact length to be 25% of the head

The core will be placed, after the foundation preparation indicated previously, in horizontal lifts of 0.3 m and compacted to 95% modified proctor density at 0% to 2% over optimum moisture content. Extensive moisture content conditioning of fill material will be required.

The bulk of the dam earthworks volume will consist of nearby granular material for the unclassified fill (Zone 2A) and pit run granular (Zone 4D). Borrow sources are shown on Drawings G04 and G05.

6.2.2.1. Dam Stability

The stability of dams, and hence the side slopes, is often controlled by foundation conditions. The sedimentary rocks of Alberta are known to contain horizontal and near-horizontal weak seams, often at residual strength due to past geologic activity such as glacial down cutting and subsequent stress relief and valley rebound. However, at the MC1 site, contiguous horizontal weak seams were not detected in the bedrock foundation from the cores recovered from drill holes in the river valley nor from the abutments. Furthermore, swelling clay minerals are not present in the bedrock. On this basis, it was determined that the MC1 design could be based on relatively high residual strength parameters for the bedrock bedding planes, i.e. ϕ^'=32°, rather than very low residual strength parameters.

Conventional stability analyses were done for the main dam as well as governing sections of the left and right abutment dykes. The analyses were performed using industry standard commercial software (i.e. Geostudio) and the results indicate the design safety factors meet and exceed the Canadian Dam Safety (CDA) requirements for an Extreme Hazard dam. Conventional overall dam side slopes of three horizontal to one vertical (3H:1V) were selected.

During the next design stage, a comprehensive triaxial test program will be required for the clay till material as well as further drilling and testing accompanied by appropriate additional engineering analyses in order to confirm the current design assumptions.

6.2.2.2. Cofferdam

During the first phase of construction, the two diversion tunnels will be constructed in the rock on the right bank of the river. During this time, the river will continue to flow in the natural river channel.

The second phase of dam construction will begin with the building of an upstream cofferdam across the river valley which will tie into each abutment. The cofferdam will divert the river flows through the diversion tunnels to protect construction of the dam during normal flows as well as from flooding during construction.

The upstream cofferdam will be constructed in two stages: firstly, a low fill will be placed to approximately five meters in height to divert the river through the diversion tunnels and to provide protection for the foundation grouting and placement of the initial Zone 1A Impervious Fill (clay till) on rock; and secondly, the main cofferdam fill will be placed to the full height of approximately 19 meters.

The upstream cofferdam will be a zoned earthfill embankment dam constructed to elevation 1399.1 meters or approximately 19 meters above the current river bed. The cofferdam will be incorporated into the body of the main dam, as shown on Drawing D02. This crest elevation of the cofferdam will provide protection against a 1:50 year flood event with one meter of freeboard. The cofferdam design features include unclassified fill (Zone 2A) and Pit Run Granular Fill (Zone 4D) with 3H:1V side slopes, an inclined upstream Impervious Zone 1A clay till blanket to retain upstream water, and an underlying Zone 3A Fine Filter. A single line, 20 meters deep grout curtain will be constructed into the rock foundation and tied into the upstream blanket if required to prevent seepage through the foundation beneath the dam.

6.2.3. Cut-Off Walls

The gravel beneath the clay till soil in the left and right abutments, while similar in composition and deposition, was evaluated separately for treatment given the difference in the elevation and depths.

6.2.3.1. Left Abutment

There is a deep buried bedrock valley in the left abutment of the dam that is approximately 650 meters wide and up to approximately 50 meters deep leaving a narrow rock pillar on the north slope of the Elbow River valley (Section 4.2). In this area, the upper soil is a relatively homogeneous clay till 25 to 30 meters thick overlying up to approximately ten meters of sand and gravel soil overlying sedimentary rock at the deepest part of the buried valley. The top of the rock rises steeply to the north of the service spillway (Drawing D03-A).

The sand and gravel at the base of the buried valley is believed to possess relatively high permeability and if left untreated significant seepage is expected to occur in this layer from upstream-to-downstream. Several treatment options were reviewed however given the depth, a plastic concrete cut-off wall is envisioned. Given the potential hydraulic pressure by the permanent pond and the temporary higher pressure when the dam is full with flood water, a cut-off wall approximately 0.8 meters wide and 350 meters in length is necessary to control seepage gradients in this permeable zone.

The construction technique is relatively conventional and widely used for controlling seepage for infrastructure. A deep trench will be excavated through overlying clay till and gravel with crane-mounted grab excavators. The excavation will be stabilized with a temporary bentonite slurry wall and filled with a plastic concrete-bentonite mixture in panel sections.

6.2.3.2. Right Abutment

The right abutment geology is also complex as indicated in the prior geologic summary. Test drilling and geophysics have indicated a mantel of glacial soils ranging from 5 to 20 meters thick overlying sedimentary rock along the 1400 meter length of the centerline of the right abutment dyke. Within the soil unit, the upper material consists of clayey till and there is an extensive high permeability layer of sand and gravel below the till overlying the bedrock. This layer could form a seepage path beneath the dam from upstream to downstream, and as such, it will be essential that a cut-off be constructed through the layer over much of the right abutment. The sand/gravel is above the elevation of the permanent pond and only temporary hydraulic pressure is expected during flood water retention. Given the potential hydraulic path and shallower depth, a cement/bentonite/soil slurry cut-off wall approximately 900 meters in length and extending to sound rock is considered a suitable treatment for the right abutment (Drawings D03-B).

The construction technique is relatively conventional and widely used for controlling seepage for infrastructure. A trench will be excavated approximately 0.8 meters wide through overlying clay till and gravel with conventional track mounted excavators equipped with a long-reach boom. The excavation will be stabilized with a temporary bentonite slurry and backfilled with a bentonite/soil mixture.

6.2.4. Service Spillway

The ungated Service Spillway will be positioned at the northern end of the left abutment and will feature a fixed crested ogee weir; a long chute down the existing slope to the Elbow River and a flip-bucket near the River (Drawing D04). The spillway will begin to release flood flow greater than the 1:1000 year flood event (greater than the June 2013 flood).

The spillway overflow weir will be an ‘ogee’ shape, 40 meters in width with a crest elevation of 1424.5 meters. The ogee will be constructed of mass concrete with cast-in-place reinforced concrete wing walls and an apron. A bridge across the spillway was deemed to be unnecessary, and a vehicle turnaround was placed on the south side by expanding the crest of the dam.

The spillway chute will be approximately 410 meters in length with a width of 40 meters near the crest and 28 meters over most of the length. The chute will have an interior height of three meters which will be sufficient to allow the dam to pass the PMF flow, including freeboard requirements. The entire chute structure will be founded on 2.5 meters of granular fill to provide frost protection to the foundation soils and to provide under-slab drainage for seepage as well as precipitation entering from the backfill.

A flip-bucket structure will be situated at the base of the chute, and its purpose will be to throw spill releases a sufficient distance from the structure into the plunge pool, where the energy of these flows will be dissipated as it re-joins the Elbow River. The flip-bucket will be 25 meters in width, between four meters and eight meters in thickness and constructed of mass concrete. Erosion protection at the outlet will consist of a concrete apron which will extend 15 meters horizontally from the flip-bucket, and a secant pile scour arresting concrete wall extending 15 meters vertically down into rock.

6.2.5. Low-Level/Diversion Tunnels

A low-level outlet, with a gate and tunnel through the rock was assessed to be cost comparable to an open cut and backfill for a conduit option. The tunnel option provides added benefits as it is less risky given the nature of tunnels with concrete in rock and long-term performance for seepage control.

Two six meter finished inside diameter tunnels constructed through the right abutment as shown on Drawing D05, will serve two purposes: to divert river flow around the construction of the main dam and to act as part of the permanent outlet facilities for flow during operation. The tunnels will also assist in passing extreme flood events along with the service and auxiliary spillways.

The inlet structure will be located immediately upstream of the toe of the main dam in rock with an invert elevation of 1384 meters, which is approximately four meters above the current river bed.

The inlet consists of a headwall, tunnel inlets and side wing walls all founded on rock. A portal face extending six meters above the tunnel obvert (crown) will be required to initiate tunneling and will be excavated by drilling and blasting in benches up slope.

Flow will be regulated by 4.7 meter x 4.7 meter vertical lift gates in each tunnel. A small fill extension upstream of the dam will provide access to the hoist house.

Stop logs are planned at the portals to isolate the main gate where maintenance or repair is required.

6.2.6. Stilling Basin and McLean Creek Diversion

The tunnel outlet will be located downstream of the toe of the main dam at about the location of the current outlet of McLean Creek into the Elbow River. The outlet will consist of an 80 meter long by

40 meter wide by 5 meter deep stilling basin constructed in rock. The stilling basin is required to dissipate energy from high velocity tunnel flows during normal operations and flood events. Riprap at the outlet of the stilling basin will provide erosion protection.

McLean Creek currently exits into the Elbow River at the location where the diversion tunnel stilling basin will be located. The creek channel will need to be re-aligned as shown on Drawing D05 in order to re-direct flow around the stilling basin location both during and after construction. The short length of the channel will be lined with riprap over bedding material to protect against erosion.

6.2.7. Auxiliary Spillway

An auxiliary spillway will be located near the end of the right abutment dyke to provide additional discharge capacity for very large flood events that may approach the PMF. The auxiliary spillway will be formed by constructing a weir to an elevation of 1426.1 meters with a width of 200 meters. The channel will be oriented to direct flow towards McLean Creek, and finished with topsoil and grass. The channel and weir have been designed to limit their overall head and unit discharge to help reduce erosion potential.

6.2.8. Other

The MC1 general arrangement impacts two features and modification will be needed for:

- McLean Creek - diversion near its outlet to the Elbow River
- Un-named drainage/creek on the left abutment north and through the Service Spillway footprint

6.3. Preliminary Reservoir Terrain Evaluation

The MC1 site area was assessed for geohazards that may affect the proposed reservoir and dam site. The report prepared by BGC is contained in Appendix 6B [Terrain (Landslide Assessment)]. This assessment included qualitative evaluation of terrain stability within the maximum area potentially inundated by reservoir infilling, defined as encompassing the PMF pond elevation of 1428 meters upstream of the dam.

The evaluation based on terrain mapping indicated about 8% of the study area as having a moderate or high relative susceptibility to landslide initiation following reservoir filling or rapid drawdown (terrain stability classes IV or V). For example, the scarp slope on both sides of the Elbow River floodplain was interpreted as being comprised of colluvium or glaciofluvial material with a terrain stability class of IV or V, or exposed bedrock. On the northwest side of Elbow River, moderately steep colluvium and rock slopes are present at the back of the terraces, and extend to below the 1,430 meter contour. These are mapped as terrain stability class IV and could be destabilized if water levels reach the toe of the slopes with sufficient depth to increase toe erosion.

The Beaver Flats Landslide Complex is regarded to have the most significant potential impact to downstream infrastructure. Some rotational slides with an area less than 20,000 m^2 were observed but such a small landslide was considered to have a low potential to disrupt the flow of the Elbow River or its tributaries.

The evaluation did not include field investigation and is considered preliminary.

6.4. Debris Assessment and Management

Debris assessment and management is reported in the BGC report in Appendix 6C [Bedload (Geomorphology)]. The MC1 dam will have an impact on the channel morphology and fluvial processes on the Elbow River. An assessment was completed to evaluate the potential rate of upstream sediment accumulation, as well as the likely downstream morphologic adjustments which may occur as a result of the MC1 dam construction.

The Elbow River has a watershed area of 702 km^2 upstream of the dam site and 1217 km^2 upstream of the Glenmore Reservoir. The surficial materials are generated both by thrust faulting in the Permian and Mesozoic strata in the area, and by glacially-deposited valley fills from the most recent glaciation. The high sediment supply in the watershed has contributed to the development of a low-order braided channel pattern in the vicinity of the MC1 Dam site, which is characterized by aggradation and lateral instability.

Damming typically reduces braiding intensity (i.e., the number of channels) and promotes a shift toward more stable (less complex) channel patterns. While the response is often spatially and temporally complex, the direction and magnitude of the response is a function of the sediment imbalance downstream of the dam, as well as the competence of downstream flows to erode the channel bed and banks.

Sediment is supplied to the MC1 Dam site by hillslope failures, tributaries, and bank and island erosion, and routed downstream by the flow. Bank erosion is a substantial sediment source, and contributed approximately 7.5 x 105 m^3 (2.0 x 10^6 tonnes) of sediment to a 7.6 kilometer long reach encompassing the MC1 Dam site during the 2013 flood event.

Various techniques and considerations were used to estimate the bedload transport into the permanent pond at MC1. These are presented in the report, and is estimated to range between 9,240 to 36,700 m^3 per year.

It is envisioned that the gravel currently within the permanent pond area will be utilized for dam construction which will provide additional space for the bedload transport for many decades. Some removal of deposited sediment many be required after a major flood event such as the June 2013. The low-level inlet structure will be fitted with removable trash racks to manage floating debris.

Consideration was also given to the management of floating debris that may collect at the project site. Given the conceptual level of this study, a formal debris assessment has not been conducted to determine the debris load that may be mobilized in the event of a large flood event on the Elbow River. This would require relatively extensive bank surveys in the upper river reach. However, floating booms may be implemented to collected floating debris. Additionally, the tunnels would be equipped with coarse trashracks to prevent large woody debris from entering each tunnel. Given the depth of the tunnels, it is expected that during a large flood event, most woody debris would remain floating at the reservoir surface, particularly as the reservoir began to surcharge. At the end of each large event, the extent and nature of any accumulated debris would need to be reviewed and any accumulated debris removed periodically. Removal of any debris that may collect at the tunnel entrance would require the use of a mobile crane/clam. Mobile crane access will be provided by a road from the top of the dam and a platform near the portal.

Given the long crest of the overflow spillway (40 meters), it is expected that any floating debris that may approach the operating spillway would simply be passed over the weir, down the chute, and passed downstream in to the Elbow River.

6.5. Fish Passage Overview

6.5.1. Background and Consideration

With the location of key fish habitats above and below the dam site, impacts to longitudinal connectivity are recognized as an issue. Fish movement and reduction of fish passage-related mortality and injury are required to help ensure the long-term viability of fish populations in the Elbow River.

While the mitigation design is based largely on conditions for upstream passage, the movement of fish downstream was also considered. Three possible options were identified by NHC in discussions with Opus and Hatch with respect to upstream fish passage options on the MC1 Dam:

1. No upstream passage structures, and fish are collected in a trap and truck operation at a dedicated fish collection facility
2. Alteration of the current dam discharge structures to allow upstream fish passage (e.g. modify the low-level outlet tunnels), along with modification of the dam operations
3. Development of a separate fish passage structure that would allow volitional movement for upstream migrant fish under normal dam operations

Scenario 2 was eliminated due to the high velocities and technical challenges to mitigate potential injury and mortality to fish.

Scenario 3 was determined to be the most effective solution for upstream passage given the operations and hydraulics of the diversion tunnel and downstream dissipater structure.

The report and details are presented in Appendix 6D [Fish Passage].

6.5.2. Fish Passage

In addition to the ecohydraulic and expected fish movement requirements, the fish passage structure design and operations must be compatible with the hydraulic and geotechnical design of the dam and expected dam operational scenarios. Based on the preliminary fish passage options, mitigation works, civil and geotechnical requirements, a separate free-surface fishway system is proposed.

The proposed fish passage structure would consist of the following:

1. Inlet Structure / Fishway Exit: a headworks inlet structure with submerged orifices or gates to control the total fishway discharge
2. Bypass Tunnel: A tunnel through the dam to connect the inlet structure to the downstream fishway
3. Nature-like Fishway: A 350 meter long, 5% grade, nature-like fishway down the downstream dam face providing the elevation gain required
4. Outlet Structure / Fishway Entrance: An outlet structure into the stilling basin for fish to enter the fish passage system

The preliminary concept is provided in Appendix 6D.

6.5.3. Fish Passage Operations

Flows released through the fish passage facility would be kept relatively constant, and periodic adjustments would be required to releases made through the tunnel to maintain water levels within a pre-selected reservoir range (assumed to be 1394 to 1395 m for this conceptual level of study). The water level within the fish passage tunnel would be maintained at approximately Elevation 1394 meters and flows into the fish passage facility would be regulated at the intake to maintain this flow throughout the expected range in the normal reservoir level. This would be done through the periodic adjustment of a series of baffle/orifice plates in the small intake structure at the entrance to the fishway tunnel.

6.6. Independent Geotechnical Review

An independent Geotechnical review was conducted by Bob Patrick. The commentary and response are provided in Appendix 6E [Geotechnical Peer Review].

One of the more notable inquiries related to the pore pressure in the Impervious Core fill during construction and longer term. The resulting review concluded that additional stability analysis is needed, if the project proceeds towards implementation. Notwithstanding the review, it was considered low risk that the medium plastic clay till planned to be utilized for the Impervious Core would require controlled construction placement rates nor impact the global stability of the dam. The stability analysis completed thus far was consistent for industry practice for a conceptual design.

The second item was a suggestion to consider an impermeable blanket upstream of the dam to cut-off seepage instead of the concrete and slurry cut-off walls. However, the design team considered that the site investigations completed for the updated conceptual design were insufficient to support such a design. Additionally, given the extreme hazard classification of the dam and the complex geology at the left abutment, a cut-off wall was chosen reflecting our assessment that robust solutions will likely be required for MC1.

6.7. Dam Operations

As discussed in section 6.1.4.1, during the passage of flood events, the tunnel gates will be lifted as required to prevent water levels on the reservoir from exceeding Elevation 1395 meters. However, in accordance with the flood management strategy for this facility, flow releases through the diversion tunnels would be restricted to an average flow of 212 m^3/s to limit releases to downstream areas. The fish ladder facility would be closed during major flood events. The reservoir level will be allowed to surcharge above Elevation 1395 meters for floods with inflows exceeding 212 m^3/s (approximately a 5- to 10-year event). If the reservoir should surcharge to Elevation 1424.4 meters (equivalent to passage of 2013 flood event), the diversion tunnel gates would be opened and additional flow released to prevent further surcharge if possible. For very large events, exceeding the 1000 year return period, the gates would be fully opened and additional reservoir surcharge will occur. During passage of the projects Inflow Design Flood, the PMF, the reservoir is expected to surcharge to Elevation 1428.1 meters.

The maintenance and non-flood monitoring requirements for the MC1 dam are envisioned to be minimal, i.e. <1 FTE, as there is no day-to-day operational management of the gates. Additionally, the mechanical workings for the gates are relatively simple and operations could be automated.

The only operational needs envisioned would be inspections to manage risk of debris, erosion, security and general inspection to confirm operational status.

It should be noted that the preliminary operating strategy for MC1 has focused primarily on flood management. However, the permanent storage of the facility can also be used to provide additional water supply in the event of an extreme drought. If needed, the projects 3,500 dam^3 permanent storage volume could be utilized to augment flow releases during a severe drought period. Depending on the value associated with this type of flow augmentation capability, it may even be desirable to increase the project permanent pool level. This could be assessed as a part of future optimization studies should the project advance past the conceptual level of study.

7. Highway and Bridge Relocation

7.1. Highway

7.1.1. Highway Geometry

With the MC1 Dam in place, a portion of Highway 66 and the bridge would be under the water level of the permanent pond. Opus undertook a planning and roadway concept review process utilizing a series of workshops to develop various options, constraints, constructability review and value engineering to determine the preferred option. In all, 17 options were developed and assessed and are presented in the detailed report contained in Appendix 7 [Road and Bridge]. The preferred bridge location was a function of comparing constructability, cost, standards and environmental impacts of two general scenarios:

a. the deep hill cuts in the river valley with a shorter bridge

b. a longer bridge that is higher above the river with less river valley cutting and earthworks

At the end of the evaluation, the selection team concluded that the longer bridge which minimized impacts to the campgrounds and environment, river and constructability challenges was preferred. The proposed alignment of the road is shown in Drawing R01 to R03.

Near the south end of the alignment, the proposed new Highway 66 horizontal alignment consists of a right hand 600 meter radius curve with 6.0% super-elevation and a left hand 800 meter radius curve with 5.4% super-elevation. A straight 700 meter section between the horizontal curves contains the proposed bridge structure spanning the Elbow River. The bridge is approximately 215 meters long. A straight section of road from kilometer 2+325 to kilometer 5+355 meters is located on an existing McLean Creek Trail of which approximately 1.0 kilometer is paved. Towards the north and crossing the McLean Creek Trail and McLean Creek the proposed horizontal alignment consists of a left hand 2000 meter radius curve with 3.0% super-elevation and a right hand 3000 meter radius curve with no super-elevation. Finally, at the tie into the existing Highway 66, the alignment has a 2000 meter radius curve with 3.0% super-elevation.

The new vertical profile is well above the PMF Elevation of 1428.1 meters.

Vertical Crest and Sag Curve K values of 100 and 55 respectively are met or exceeded by the proposed design. These meet the required comfort criteria (K= 30) for a 110 km/h design speed.

The minimum stopping sight distance criteria for crest curves of 235 meters (K = 45) is also met.

7.1.2. Cross Section

The new road will be designed to Alberta Transportation’s Highway Geometric Design Guide and have a design designation of RAU-211.8-110. The cross-sectional elements consist of 2 x 3.7 meter traffic lanes, 2.2 meter shoulders, 5H:1V side slopes, 4.0 meter ditch and 3H:1V back slopes. The basic right-of-way width will be 50 meters.

Based on the existing traffic volumes of 1890 AADT and high proportion of passenger and recreational vehicles, the proposed pavement thickness will be 400 millimeters consisting of 300 millimeter granular base course and 100 millimeter asphaltic concrete pavement.

7.1.3. Drainage

Stormwater drainage will be provided in the form of side run-off from the sealed carriageway onto the grassed earth embankments. Drainage ditches, running parallel to the new road and approximately 3 meters wide will convey the water to new centerline culverts.

No significant erosion control requirements are anticipated. Typical roadway construction best management practices implemented by Alberta Transportation is expected.

Culverts in the form of corrugated steel pipes will be provided across the new carriageway to convey stormwater from the ditches on the southern side of the carriageway to the drainage outfalls located on the north side adjacent to the Elbow River.

7.1.4. Geotechnical

A total of seven geotechnical boreholes were drilled between December 19 - 22, 2016 along the proposed road realignment. The depths of the boreholes ranged from 1.6 meters (west side of Elbow River) to more average depths of 16.5 meters.

7.1.4.1. Subsurface Soil Conditions

In the boreholes drilled east of McLean Creek within the northeast portion of the proposed realignment, clay till was encountered throughout the boreholes which were drilled to the maximum depth of 17.8 meters. At the McLean Creek crossing, the borehole revealed clay shale at shallow depth underlying a surficial fluvial gravel layer. Thick clay till deposit at the northeast portion and fluvial gravel within the McLean Creek were also identified.

In the borehole within the middle portion of the proposed realignment, a thin layer of clay till was encountered underlain by gravel which extended to the borehole termination depth of 3.75 meters.

In the boreholes drilled at the Elbow River valley within the southwest portion of the proposed realignment, gravel was encountered throughout the boreholes except thin embedded layers of silt and clay at some locations. The borehole at the west valley slope was drilled only to the depth of 1.6 meters due to the presence of cobbles or boulders causing auger refusal. On the other hand, the boreholes at the east side of the river valley were drilled to the depths of 15 meters and 16.5 meters. Glaciofluvial deposits were also identified in the BGC (2016) terrain analysis map within the southwest portion of the proposed realignment. Exposed bedrock and fluvial deposit were also identified in the BGC (2016) map within the river channel and bottom of west valley.

7.1.4.2. Earthwork Fills

If the exposed subgrades, after removal of deleterious materials, are less than 0.6 meters below the design subgrade level, the excavation or subcutting should be extended to 0.6 meters below the design subgrade level. Weak subgrade areas should be subcut and backfilled with suitable materials. Alternately, placement of geogrid over weak inorganic subgrade soil could be considered to provide additional support.

The approved exposed subgrade can be backfilled to the design subgrade elevation using suitable engineered materials. Where the backfill soils are highly plastic, the moisture content for compaction should be within a range of optimum to 3% above optimum. Also, where the backfill soils are predominately silt materials, the moisture content should be within a range of 3% below optimum to optimum.

The excavated clay till and glaciofluvial materials from the project site may be used as backfill for grading after separating organics, mixed topsoil, roots and other unsuitable materials. The materials should be free from organics, rubble, snow, ice, frozen lumps and stones greater than 150 millimeters in diameter.

Where embankments are to be placed and compacted on hillsides or where the existing embankments are extended, the side slopes should be denuded of all vegetation and topsoil, and benched one level at a time (starting at the ditch bottom) in order to obtain bonding between the new embankments and existing embankments or hillsides.

The proposed embankment sideslope flatter than 4H:1V is considered to be adequate for stability. If shallow groundwater is encountered at new embankment construction sites, stage construction of embankments should be followed in order to dissipate the excess pore water pressure due to the fill weight.

7.1.4.3. Earthwork Cuts

After the proposed design cuts, the exposed subgrades should be subcut further to 0.6 meters below the design subgrade level. As in the fill sections, the exposed subgrades after the design cuts should be inspected and approved by a qualified geotechnical engineer. Weak areas should be removed and backfilled with suitable materials. Alternately, geogrid reinforcement can be considered over weak inorganic subgrade soils depending on the specific site condition during construction.

Since groundwater is expected at some locations, dewatering may be required during excavations. If groundwater is encountered after the proposed design cuts, it is recommended to either use subsurface drainage systems or raise the road grade to achieve a minimum of one meter difference between the design subgrade level and water level.

Backslopes should be cut at slope of 4H:1V or flatter. If organics, very soft or loose soils, and/or high groundwater level are encountered in the excavated backslopes, their stability should be evaluated based on the site-specific conditions.

7.2. Elbow River Bridge

To limit the amount of earthworks at the approaches and the associated potential environmental impacts and construction costs, the high road option was retained for the bridge profile. The low option would have required large volumes of earth cuts that would need to be disposed offsite. An approach fill will be required at the west end of the bridge.

Additionally, for ease of site access and environmental reasons, the long span bridge option over the river was selected to avoid having piers in the water. A main span of 115 meters was considered enough to clear the steep river banks with an additional offset of 10 meters to the adjacent piers. Side spans of 50 meters were selected to tie into the roadway at the proposed ground level on each side of the approaches. Additional details are shown in Drawings S01 and S02.

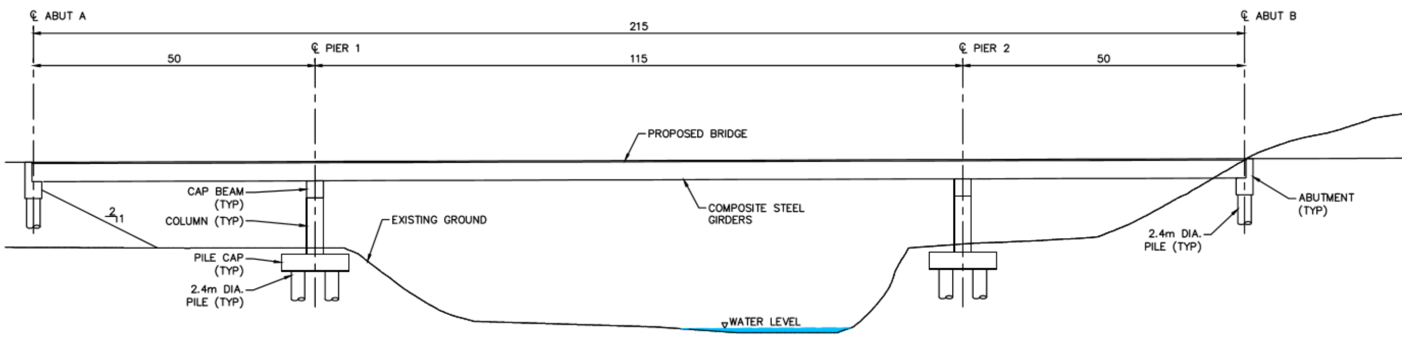

Figure 7.1: Proposed Bridge Profile

For the superstructure, it was decided that incremental launching would be the preferred construction method to accelerate construction time and given the limited existing access to the site. There is also a large flat area on the west side of the river that would be suitable for staging the launch. A relatively light superstructure consisting of four steel girders was selected for launching. Another option considered was with segmental concrete box girders using the balanced cantilever construction method. It was found, however, that this method would have been more time consuming because it would probably require cast-in-place segments. A reinforced concrete slab of 225 millimeters was selected for the deck, which could be either cast-in-place or precast panels. An overlay of 50 millimeters of asphalt with a waterproof membrane was proposed over the deck slab. Traffic barriers were proposed as TL-4 given the volume of traffic on the bridge. Construction sequencing is illustrated in Drawing S03.

The proposed foundations consist of two concrete abutments and two concrete piers. The abutments and piers were selected to be on large diameter bored piles to resist the vertical and horizontal loads during construction and service life. A spread footing at the piers would have had a larger footprint that would have required the piers to be pushed away from the river banks thus increasing the span.

7.2.1. Elbow River Bridge Hydrology

Various bridge location and elevations were considered during the options analysis. The preferred option will be a clear span of the river and will be significantly above the elevation of the PMF pond elevation at the bridge location.

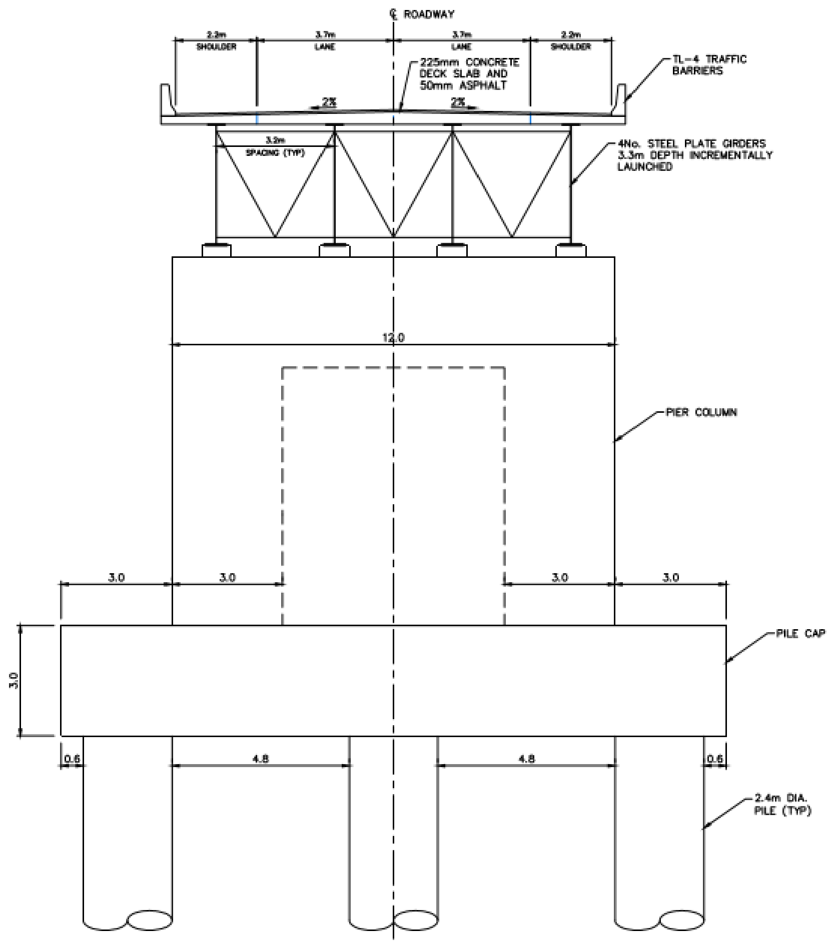

Figure 7.2: Bridge Cross Section

8. Facility Relocation

8.1. Introduction

The scope of the study was to generate a schedule of quantities to relocate or replace facilities that must remain operational during and/or after the MC1 dam is constructed. At the direction of the client, it has been assumed that all facilities not deemed to be completely abandon, will be replaced. Further, conceptually suitable sites for relocations are presented to quantify the conceptual requirements of the infrastructure to support these new sites. The quantities derived from the study and reported here are presented for cost estimating purposes at a conceptual level. The details are presented in Appendix 8 [Facility Relocation], with a summary presented below.

8.2. Existing Facilities in the Area

8.2.1. Elbow Valley Ranger Station (EVRS)

The EVRS consists of two compounds, the East Compound and the West Compound. The East Compound includes housing for rangers and campsite wardens, and support buildings for campsite operations. The West Compound includes a firefighting, search and rescue and maintenance base. Additionally, there is a sewage treatment plant that services EVRS, McLean Creek Campground and Camp Horizon.

8.2.2. McLean Creek Campground

McLean Creek Campground is a large camping area comprised of 170 campsites and is designed to accommodate off highway vehicle users. It includes a store, sewage disposal, toilets, tap water, playground and power.

8.2.3. Camp Horizon

Easter Seals Camp Horizon offers residential camps that provide outdoor adventure based programs for children and adults with disabilities and medical conditions.

8.3. Impact Analysis

8.3.1. Flood Impact

EVRS could initially be affected by flooding behind the cofferdam during MC1 Dam construction. Once MC1 is fully operational, EVRS will be affected by the permanent pond.

8.3.2. Highway 66 Relocation Impact

The proposed relocation of Highway 66 to support MC1 will affect the McLean Creek Campground.

8.3.3. Consequential Impact

The EVRS sewage treatment plant services McLean Creek Campground and Camp Horizon. Disruptions at EVRS could affect McLean Creek Campground and Camp Horizon.

8.4. Recommendations

8.4.1. Elbow Valley Ranger Station

The concept plan provides for the relocation of the EVRS and all its nonabandoned facilities to the proposed Gooseberry Station, downstream of MC1.

8.4.2. McLean Creek Campground

Build 16 new campsites and store at a suitable location. Construct a new access to the campground from the new Highway 66. Connect the existing campsite sewage dump location and sewer system to the new sewage treatment facility at the proposed Gooseberry Station with a new forcemain.

8.4.3. Camp Horizon

Connect the existing Camp Horizon sewer system to the new sewage treatment facility at the proposed Gooseberry Station with a new forcemain.

9. Land Requirements

The land for the entire project is located on lands under the control of various Ministries within the Government of Alberta:

- Crown Land - majority of the dam footprint and Highway 66 relocation
- Park Land - McLean Creek Campground area
- The Elbow River and McLean Creek bed and shores

10. Construction and Schedule

10.1. Mobilization and Camp/Laydown

A temporary camp with all services (water/sanitary/power/natural gas) is envisioned for about 100 people. It is shown on Drawings C01 to C04, and is located near the gravel pit west of the McLean Creek Campground.

The work force is expected to peak around 200 people. However, one could envision an average work force of 100 people over the three year period. During the summer periods, on average, a work force of 150 is expected.

Mobilization of the first crews, office and clearing and earth moving equipment is necessary in the first week to effectively accomplish the construction schedule. A relatively quick camp, office and concrete plant set up along with staffing is needed within the first four weeks to initiate the tunnel and bridge works.

10.2. Utilities and Temporary Bridges

There are several utilities such as telephone, electrical, water and wastewater lines that will require crossing plans and in some instances relocation. These are standard requirements and are not critical to the construction schedule.

Temporary bridges, such as a steel Bailey bridge, will be needed to cross the Elbow River during the initial upstream cofferdam and early service spillway work.

10.3. Dam Construction

The overall sequence of river diversion and dam construction will be as follows:

1. Construct a small cofferdam around the inlet of the diversion tunnels and divert the river around the cofferdam, and construct a small cofferdam around the outlet of the diversion tunnels
2. Construct the diversion tunnels including installation of gates, hoists, etc., and start the service spillway and cut-off walls
3. Divert the river through diversion tunnels constructing a cofferdam across the river downstream of the tunnel inlet
4. Construct the upstream cofferdam
5. Construct the main dam and complete the service spillway and auxiliary spillway
6. Begin operations

10.3.1. Dam Earthworks

The dam earthworks can be carried out with conventional earth moving equipment such as large rock trucks and backhoes. Some specialized equipment will be required for the cut off walls and bentonite/soil capping.

A general summary of the estimated 4.5 million m^3 of dam earthworks is as follows:

- topsoil removal and rock removal to prepare the entire dam and abutment footprint consists of approximately – 1,000,000 m^3
- Impervious Fill (Zone1A) - 732,000 m^3
- Unclassified Fill (Zone 2A) - 1,035,000 m^3
- Fine Filter and Drainage Layers (Zone 3A, 3B) - 785,000 m^3
- Pit Run Granular (Zone 4D) - 951,000 m^3

10.3.2. Tunneling and Blasting

The two, six meter diameter (finished) low level outlet tunnels are planned to be excavated using drill and blast methods, with conventional drills (Jumbos) to complete 70 to 90 - 90 millimeter diameter holes in the face of the tunnel, blasting with dynamite (Senatel Ultrex and Fortel Plus, and Detagel Presplit), and then mucking (removing the blasted rock).

The tunnel pilot round to start each tunnel will be a small short round advancing 2.5 meters followed by a slash (short advancement of less than 1/2 diameter) round the same length. The typical practice is to complete four slash rounds before beginning full face rounds that will advance the face of the tunnel by four to five meters, followed by mucking. Two types of full face rounds are envisioned one with perimeter holes drilled on 0.6 meter c/c spacing and each hole loaded. and another option with drill holes on 0.3 meter c/c spacing and load every other hole. This later technique should produce the least amount of over break and therefore require less concrete. A typical cross section of full round is shown in Figure 10.1 below.

The shaft will be constructed in a typical drop raise shaft round where the holes are drilled from the surface to the level of the top of the tunnel. When the tunnel excavation reaches past the shaft location, explosive charges are lowered down the holes and blasted to the surface in 3.0 meter lifts to break approximately 2.7 meters from the bottom up to the surface. This technique will permit blasting both shafts in a few days with mucking completed from the tunnel below.

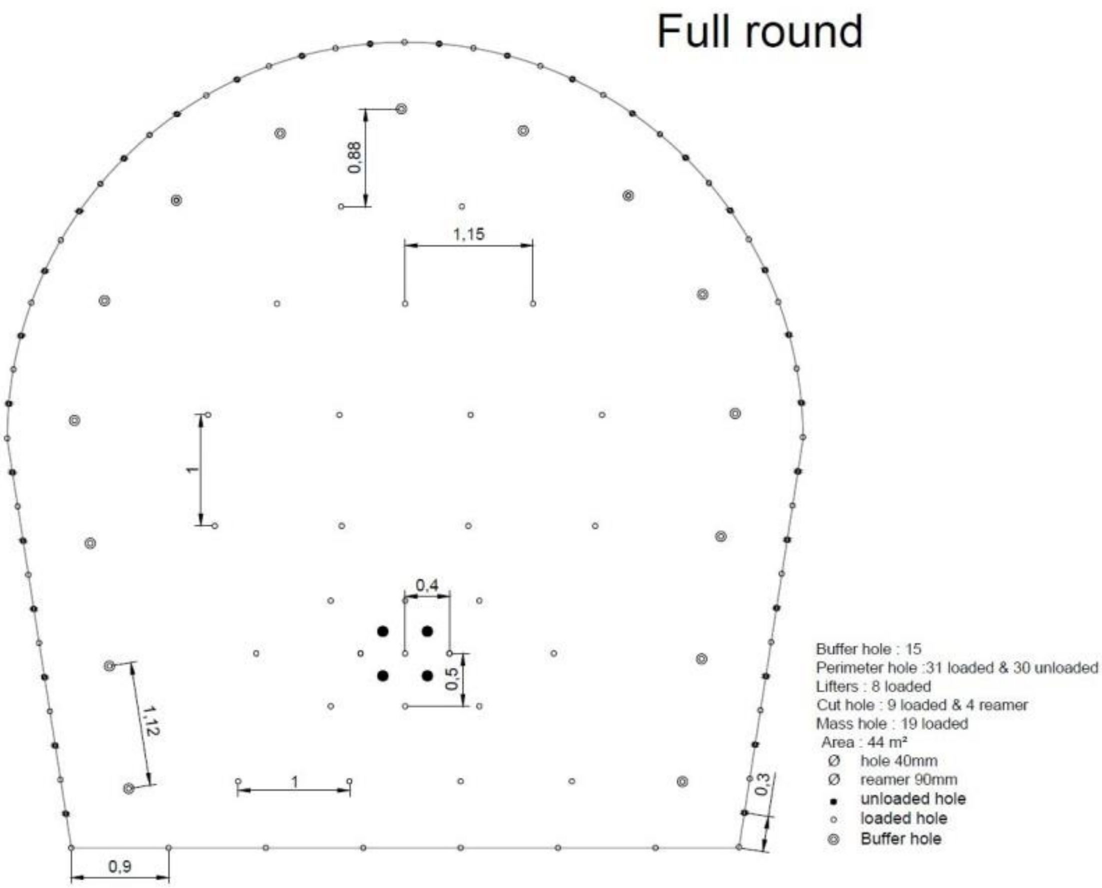

Figure 10.1: Typical Full Round Tunnel Cross Section

10.3.3. Concrete

Based on the site location and volume of concrete for the project, it is envisioned that an onsite batch plant will be the most efficient and cost effective solution. The batch plant would need to be calibrated and quality controlled to produce high quality materials. Production rates of 100 m^3/hour are reasonable for a small batch plant and is assumed in the construction schedule and cost estimate.

10.4. Highway and Road Construction

10.4.1. Earthworks

The volume of earthworks associated with the construction of the relocated Highway 66, based on the preliminary investigations to date is: total excavated material of 947,200 m^3, total fill material required is

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474,600 m^3 with a surplus material requiring disposal of 472,600 m^3. The earthworks volumes are calculated as 10% over and above the "most likely" earthworks volumes determined from InRoads (computer-aided three-dimensional design package).

The volumes presented represent the upper bound of materials as it has been assumed that earthworks will be undertaken on a cut-to-waste approach. Actual excess excavated material will be lower given that a cut-to-fill approach will be applied during detailed design.

A fill factor of 1.25 has been applied to the total volume of fill required, usable undercut has been assumed at 75% usable and the usable cut material generated is 75% usable.

10.4.2. Earth Borrow Sites

As all the material required for the road construction can be sourced from existing cuts, no borrow sites are required.

10.4.3. Surplus Fill Sites

During the earthmoving operations, excess excavated material will be placed at clearly defined surplus fill sites. These locations have been selected during the concept design stage. Based on observations of the site, the majority of the surplus fill sites will occupy pasture areas.

10.4.4. Paving

The proposed pavement structure consists of 300 millimeters of granular base course (GBC) and 100 millimeters of asphaltic concrete pavement (ACP). Approximately 105,000 tonnes of GBC will be required and 30,000 tonnes of ACP. Assuming production rates of 3,000 tonnes per day for GBC and 2,000 for ACP, the estimated number of construction days for these operations would be 35 days for GBC and 15 days for ACP.

10.4.5. Bridge Construction

The proposed bridge is a conventional steel superstructure and concrete substructure. The foundations consist of two concrete abutments and two concrete piers. The abutments and piers will be supported by large diameter bored piles to resist the vertical and horizontal loads.

It is envisioned that the bridge will be constructed firstly for the 50 meter long west span where site access and flat terrain can permit a quick start. Thus, the west end approach fill for the bridge must be initiated at the project start. This will provide the working space to incrementally launch the 115 meter long main span. This would permit other forces to be working on the east abutment and 50 meter side span.

10.5. Construction Schedule

The construction schedule for this project is relatively complex due to the critical path linkages of the tunnel, new bridge and the relocation of the EVRS. Long lead time order items such as bridge girders, gates and stop logs will need to be preordered prior to Year 1. The schedule developed includes the Construction Phase, and items such as detailed engineering design, environmental assessment and permits, land access and utility relocation coordination would need to occur prior to Year 1.

The construction schedule is aggressive and therefore 24 hour/7 days per week with shift work is envisioned. There will be winter slow down or 'partial shut-down' when only some construction elements can be progressed. The following assumptions were applied to the construction schedule:

- During normal periods of production, there will be two crews working 10 hour shifts
- Primary earthworks and concrete flat work for the spillway and bridge deck will occur between April 15 and November 15, which is the typical Southern Alberta construction period
- Approximately four inclement rain days per month are considered in the schedule through the April 15 to November 15 period

- The tunnel work will progress through the winter, as per the construction schedule
- Regardless of the spring or fall start, the critical path of the project elements remains the same

Two detailed construction schedules have been prepared and are presented in the Construction Schedule Appendix. The Fall Start Construction Schedule results in the shortest duration of less than 4 years (about 3.5 years total), and the facility is operational for flood protection after three spring seasons.

The Spring Start Construction Schedule results in a longer duration of 4.5 years and would be operational for flood protection after four spring seasons. This is simply due to the construction activities being impacted by the timing of the winter months when fill placement is impacted by freezing temperatures and the spring flood season. Additionally, there are schedule implications for the period in which the large thin concrete surfaces for the spillway slab and bridge deck can be placed.

The summary of the Fall Start Construction Season is as follows:

Year 1:

a. Construction preparation activities:
    i. Construction camps
    ii. Laydown areas
    iii. Spoil location
    iv. Clearing and timber salvage contract
b. Procure gates, hoists, stop logs and trash racks for Diversion Tunnels; procure superstructure of the bridge
c. Build the cut-off walls beneath the abutment dykes and divert traffic during construction
d. Construct the bridge substructure, rough-grading on the new highway
e. Begin Service Spillway construction
f. Construct the hoist shafts
g. Prepare portal and complete tunnel construction
h. Site preparation of new Gooseberry EVRS site and relocated
i. Preparation of the borrow areas, haul roads and temporary bridges
j. Utility relocation

Year 2:

a. Build upstream cofferdam (tunnel will be completed by then)
b. Foundation preparation and grouting for the central portion of the dam
c. Begin Main Dam fill placement
d. Continue work on the service spillway
e. Complete bridge superstructure and deck
f. Final grading of Highway 66 and paving
g. Complete the infrastructure relocation
h. Complete Highway 66 relocation
i. Insulate the core of the dam before winter

Year 3:

a. Complete the service spillway

b. Demolition of the existing highway bridge

c. Complete dam earthworks

d. Test and commission gates

Year 4

a. Reclamation, topsoil and seeding

b. Site restoration

10.6. Contracting and Construction Risks

The risk quantification session in Workshop 5 resulted in a total of six risk factors with the severity score of “serious”; these are:

1. Potential negative impact on the environment (including fish, wildlife, vegetation, etc.) or endangering species as a result of project construction disturbance, hindering natural connectivity for animals
2. If the contract documents do not provide equitable sharing of risks between the owner and contractor, then the contractor will include the cost of risk in the price
3. If the contractor defaults during construction
4. If the project's low level of design maturity results in perception of need for high contingency, then the project cost could be over-estimated
5. If the scope of infrastructure relocation is not strictly controlled, then additional functionality or expansion could be added in the new infrastructure (such as wider lane, larger facility, etc.)
6. Potential resistance to the project from stakeholders and land users, as a result of irreversible negative impact on the traditional land use

The project expected cost of risk totaled $58 Million. Details of the workshop is presented in Appendix 5.

11. Construction Cost Estimate Opinion

11.1. Assumptions

The construction cost estimate opinion is based on the details presented in the sections above and the drawings. Additionally, a number of assumptions were applied using expertise from the various team members for cost estimating project elements that have impacts to the construction schedule and costs. These elements would be engineered during the detailed design phase. Assumptions have been rationalized for the service spillway, tunnel works and other concrete related elements:

- concrete wall and slab thicknesses
- pile and sheet pile foundation requirements
- concrete pour size and panels
- rebar and water stop requirements
- proposed replacement facilities will be equal to existing operational facilities (ie. trails)

11.2. Process for Cost Estimating

11.2.1. Dam Construction

A work breakdown structure was developed for the construction of the MC1 Dam, and work packages were created for estimation. The construction cost estimate for the MC1 Dam concept was developed using a bottom-up, unit cost method of estimation. Each work package was broken down into tasks and priced by populating with resources and adding a proration of the indirect construction costs. Resources may include labour, equipment, crews (an amalgamation of labour and equipment), materials, subcontractors or consumables. Each unit cost was developed by applying a production rate to the labour, equipment or crew resources, subcontractor quotes and the supply of each unit of material, if applicable. HCSS HeavyBid software was used to build the MC1 Dam concept construction estimate.

11.2.2. Road Construction

A work breakdown structure was developed for the relocation of Highway 66, and work packages were created for estimation. The construction cost estimate for the relocation of Highway 66 was developed using an analogous (top-down) and parametric, unit cost method of estimation. Each work package was priced using available data (past unit rates, Alberta Transportation Unit Price Averages) and expert judgement (contractor input).

11.2.3. Facility Relocation

<u>Sewage Treatment Facility - Gooseberry</u>

The proposed sewage treatment facility at the conceptual Gooseberry Station was estimated by reviewing past tender data for similar work including Contract No. 09KC/OS092: Elbow Valley Ranger Station Wastewater System Upgrade.

<u>Water Treatment Facility - Gooseberry</u>

The proposed water treatment facility at the conceptual Gooseberry Station was estimated by reviewing past tender data for similar work including Contract No. 08KC/OS199: Elbow Maintenance Area and Fire Base - Water Treatment and Distribution System Upgrade.

<u>Site Civil Demolition and Construction</u>

Site civil demolition and construction at the Gooseberry Station, Elbow Valley Ranger Station and McLean Creek Campground was estimated by reviewing past tender data, Alberta Transportation unit prices and by utilizing a local civil construction contractor (Whissell Contracting Ltd.)

Building and Structural Demolition

Building demolition was estimated by applying RS Means Heavy Construction Cost Data to approximate building properties: type, foundation, area and volume.

A local structural moving contractor, McCann's Building Mover Ltd., was utilized for cost estimating of structural salvage, building sale and structural relocation.

Environmental Remediation

The cost estimation of environmental remediation of EVRS was supplied by Hemmera Environchem Inc. This cost item includes remediation of both the contaminated soil and the EVRS septic field.

New Building Construction

Building construction was estimated by applying RS Means Construction Cost Data to the square footage of the area of the proposed building.

11.2.4. Quantities and Unit Rates

The cost estimate opinion has been reviewed by a team of knowledgeable engineers. The following provides a summary of information and processes undertaken by the team to develop the cost estimate:

- The quantities have been calculated using MicroStation, with the major earthworks quantities confirmed independently using Civil 3D.
- Unit Rates for tunnel, cut-off wall, grouting and all concrete as well as all soils (gravel and mineral) for the dam have been determined through a bottom up approach similar to the process a contractor would develop for a bid. Quotes for cement, bentonite, concrete plants, equipment and labour, from various suppliers and sub-contractors were utilized to develop these costs and rates.
- Detailed pricing was obtained from an experienced specialist contractor for the concrete and slurry wall construction.
- The unit rates for the roadway and bridge have been rationalized through existing Alberta Transportation information and construction experience.
- The cost of the Infrastructure relocation (including reclamation of the existing site) has been based on a combination of direct pricing for building relocation, utilizing engineered buildings for the new building, engineering knowledge, and experience
- Information on contamination clean-up of $3.6 Million has been provided by Hemmera, and is included under the Infrastructure relocation.
- Information on the habitat compensation of $8.4 Million has been rationalized by Hemmera, based on the replacement cost to create new habitat that would be eliminated by the dam and permanent pond footprint.

In addition to the above cost estimating process, the calculated unit costs were compared to similar dam project components, such as the spillway, cut off walls, and earthworks to other nearby projects in North America (ie. BC Hydro Site C).

11.3. Contingency

The total value of the contingency in a cost estimate can be significant dollars for a major project during the concept and planning stages. This updated conceptual design report included field investigation, laboratory testing and engineering analysis for the high risk and more costly items. However, many details have not been engineered and are still conceptual. In order to fairly assess the value of the contingency to carry in the cost estimate, the risk and related potential cost implication were reviewed from three different perspectives:

1. A modified range estimate exercise for the major project component was undertaken, the 'Expected' and '95th' percentile contingency ranged from $66 Million to $81 Million.
2. The project risk assessment completed in Workshop 5, indicated the expected cost of risk is about $58 Million.

3. Given the level of detail completed, the project was compared to the AACE International practice for cost estimate. The level of detail of the project design was considered to be Class 3 (see Table 11.1 below). A contingency of 20 to 25% was envisaged from this assessment.

Table 11.1: AACE Contingency Practice

	Primary Characteristic	*Secondary Characteristic*			
ESTIMATE CLASS	LEVEL OF PROJECT DEFINITION Expressed as % of complete definition	END USAGE Typical purpose of estimate	METHODOLOGY Typical estimating method	EXPECTED ACCURACY RANGE Typical variation in low and high ranges [a]	PREPARATION EFFORT Typical degree of effort relative to least cost index of 1 [b]
Class 5	0% to 2%	Concept Screening	Capacity Factored, Parametric Models, Judgment, or Analogy	L: -20% to -50% H: +30% to +100%	1
Class 4	1% to 15%	Study or Feasibility	Equipment Factored or Parametric Models	L: -15% to -30% H: +20% to +50%	2 to 4
Class 3	10% to 40%	Budget, Authorization, or Control	Semi-Detailed Unit Costs with Assembly Level Line Items	L: -10% to -20% H: +10% to +30%	3 to 10
Class 2	30% to 70%	Control or Bid/ Tender	Detailed Unit Cost with Forced Detailed Take-Off	L: -5% to -15% H: +5% to +20%	4 to 20
Class 1	50% to 100%	Check Estimate or Bid/Tender	Detailed Unit Cost with Detailed Take-Off	L: -3% to -10% H: +3% to +15%	5 to 100

Notes:
[a] The state of process technology and availability of applicable reference cost data affect the range markedly. The +/- value represents typical percentage variation of actual costs from the cost estimate after application of contingency (typically at a 50% level of confidence) for given scope.
[b] If the range index value of "1" represents 0.005% of project costs, then an index value of 100 represents 0.5%. Estimate preparation effort is highly dependent upon the size of the project and the quality of estimating data and tools.

In the final review, a 25% or $80 Million contingency was utilized in the cost estimate as it closely reflects the 95th percentile contingency assessment.

11.4. Construction Cost Estimate Opinion

The total construction cost estimate opinion for the MC1 project is **$406 Million**, as summarized in Table 11.2 below with the details in Appendix Cost Estimate. This amount includes dam and tunnel construction, spillways, fish passage and cut off walls, as well as other items as noted in the details. The cost estimate opinion includes the Highway 66 Elbow River major bridge and McLean Creek bridge culvert. The estimate allows for 25% contingency for unforeseen additional cost not anticipated at this time. An additional allowance of 20% has been included for the cost of the project management, detailed engineering, environmental assessment and permitting, stakeholder engagement and other work required by the consultants. The environmental costs for wetland and aquatic habitat mitigation, as well as the remediation of the EVRS provided by Hemmera have been included. The estimate is based on 2017 pricing for the 3.5 year project, with no escalation.

Table 11.2: MC1 Construction Cost Estimate Opinion		
General		
Mobilization	$	12,000,000
Care of Water	$	3,000,000
Total	**$**	**15,000,000**
Construction		
MC1 Dam including all structures	$	188,000,000
Highway 66 Relocation	$	34,341,000
Facility Relocation	$	22,853,000
Total	**$**	**245,194,000**
Environmental Habitat		
Wetland Compensation	$	708,000
Aquatic Habitat Management Plan	$	10,000,000
Total	**$**	**10,708,000**
SUBTOTAL CONSTRUCTION	**$**	**270,902,000**
Engineering/Environment/Engagement (20%)	$	54,181,000
Contingencies (25% including Engineering)	$	81,271,000
Total	**$**	**135,452,000**
Grand Total	**$**	**406,354,000**

Notes:

1. This Construction Estimate is based on the level of project information developed in the study
2. Unit prices are based on calculated information, historic bid data, past project experience and engineering judgment
3. The summary information is rounded to nearest $1000s
4. Based upon no escalation, similar to a contract bid price in the present calendar year

12. Future Engineering Considerations

12.1. Hydrology

It is reasonable to expect that the PMF for Site MC1 is in the range of 0.9 to 1.0 times the PMF for Site SR1. Using the PMF estimate for Site SR1 from Stantec (2015b) without adjustment is likely acceptable at the current conceptual design stage of the MC1 project. A detailed, site-specific PMF study is required for the MC1 project at the next design stage. The study should be carried out in accordance with the Alberta Transportation guidelines, the Canadian Dam Association 2007 Dam Safety Guidelines (with 2013 revision) and accepted engineering practices. The detailed analysis may lead to an increased PMF peak, which would in turn lead to a higher dam crest and increased dam construction costs.

12.2. Dam/Geotechnical

The dam considerations were based on the available information and project scope. Numerous other details must be considered should the MC1 project proceed to detailed design. Below are some of the considerations:

- Permeability and extent of the gravel in the left and right abutments and associated requirements for cut off walls or other seepage control measures
- Rock quality at the dam core interface and associated detailed seepage and stability analysis
- Rock quality and depth of cover for the fish passage tunnel
- Rock quality at the tunnel outlet location - needed to determine if the rock has sufficient strength to act as an unlined stilling basin
- Auxiliary spillway surface treatment requirements
- Rip rap consideration above the permanent pond elevation
- Operations requirements for debris management
- Further investigation and management of the potential landslide at Beaver Flats, 12 kilometers upstream of the MC1 dam site
- Bedload and river erosion monitoring program that involves a baseline air photo assessment prior to dam construction, regular unmanned aerial vehicle (UAV) surveys following construction, and longer term air photo analyses at 10 year intervals
- Additional recommendations are provided in Appendix 3 [Dam Geotechnical]
- Slope stability analyses

12.3. Other Infrastructure

To adequately plan and design for the future operation of the other impacted and future infrastructure at the MC1 project, the following items require additional effort:

- Needs assessment for future EVRS operations
- Detailed designs for water and waste water treatment facilities and associated infrastructure
- Considerations for future park and area usage

13. Closure

This report is based on, and limited by, the interpretation of the data, circumstances and conditions available at the time of the completion of the work as referenced throughout the report. It has been prepared in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.

It has been prepared for the exclusive use by Alberta Transportation and others may not rely upon this information contained herein.

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14. References

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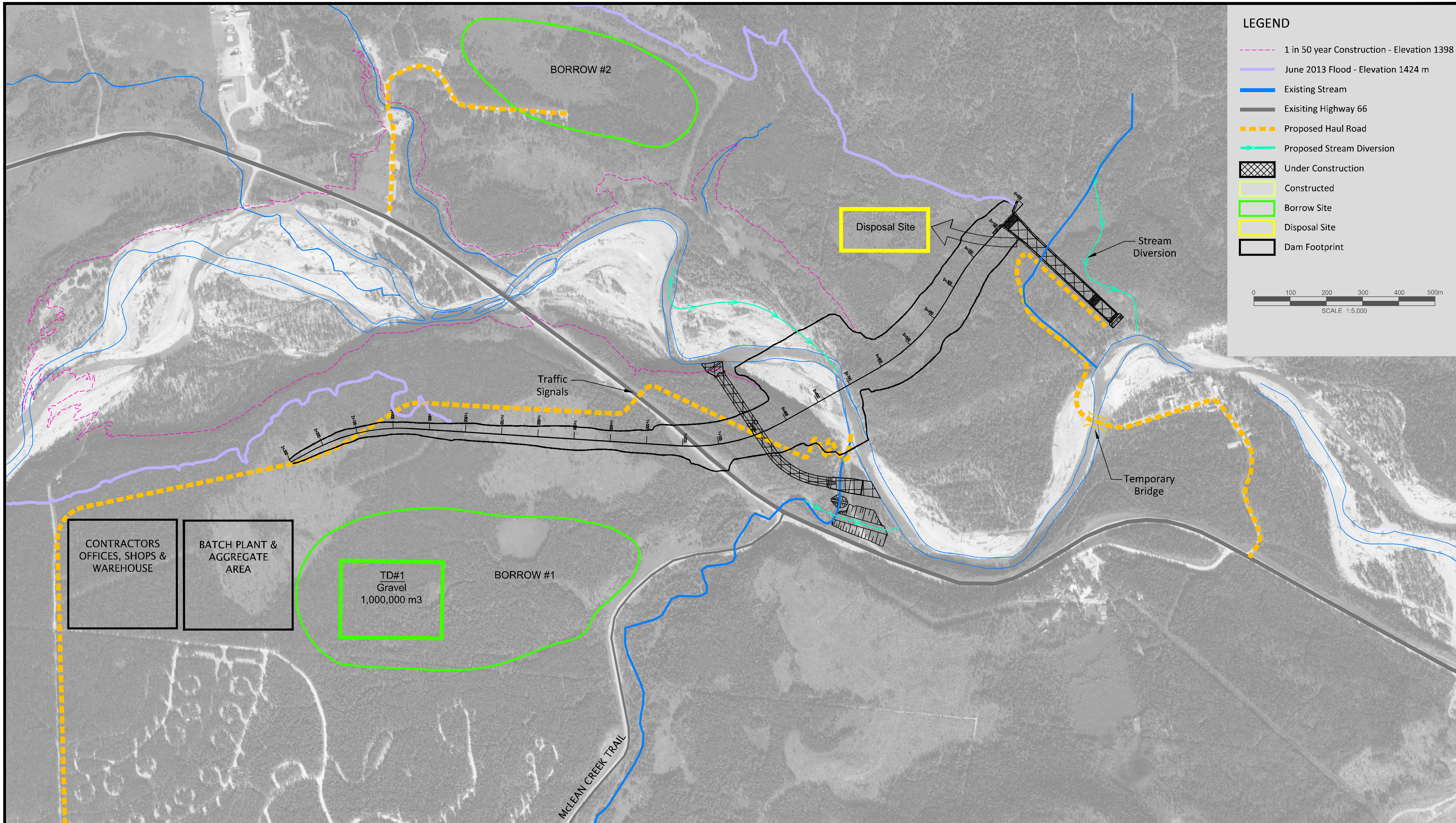
Drawings Package



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ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

MCLEAN CREEK

YEAR 1

LEGAL DESCRIPTION

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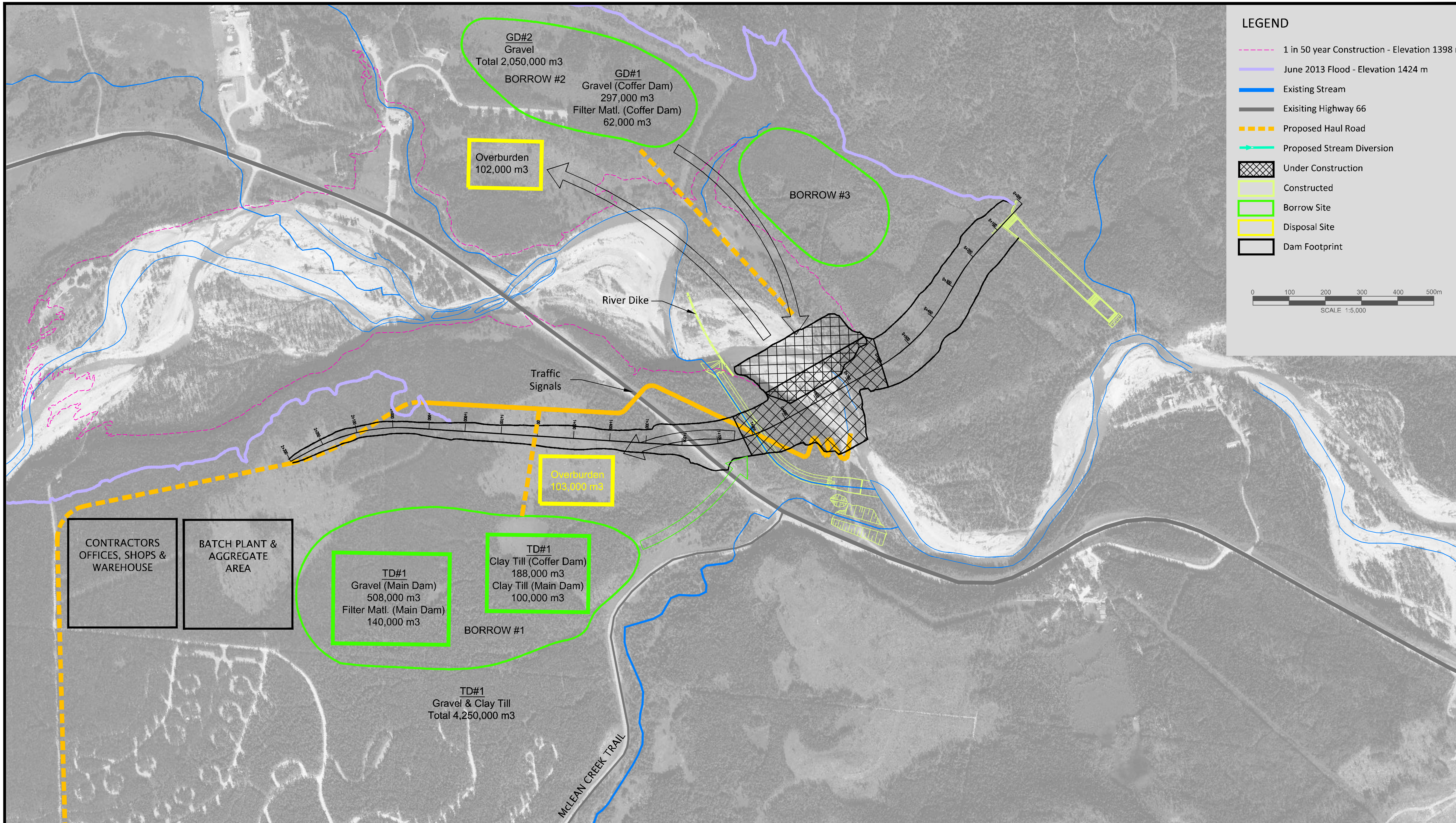

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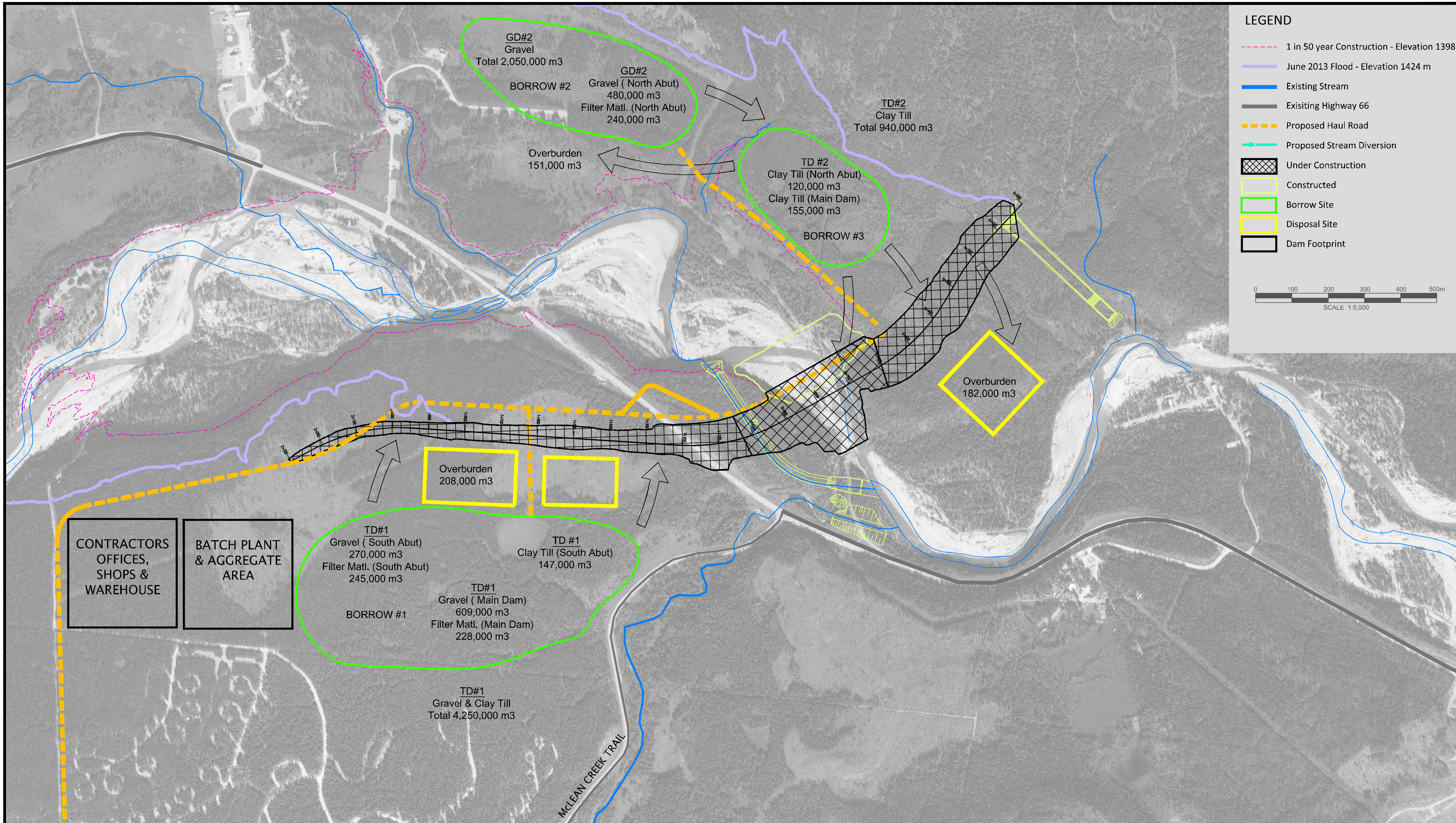

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MCLEAN CREEK

YEAR 3

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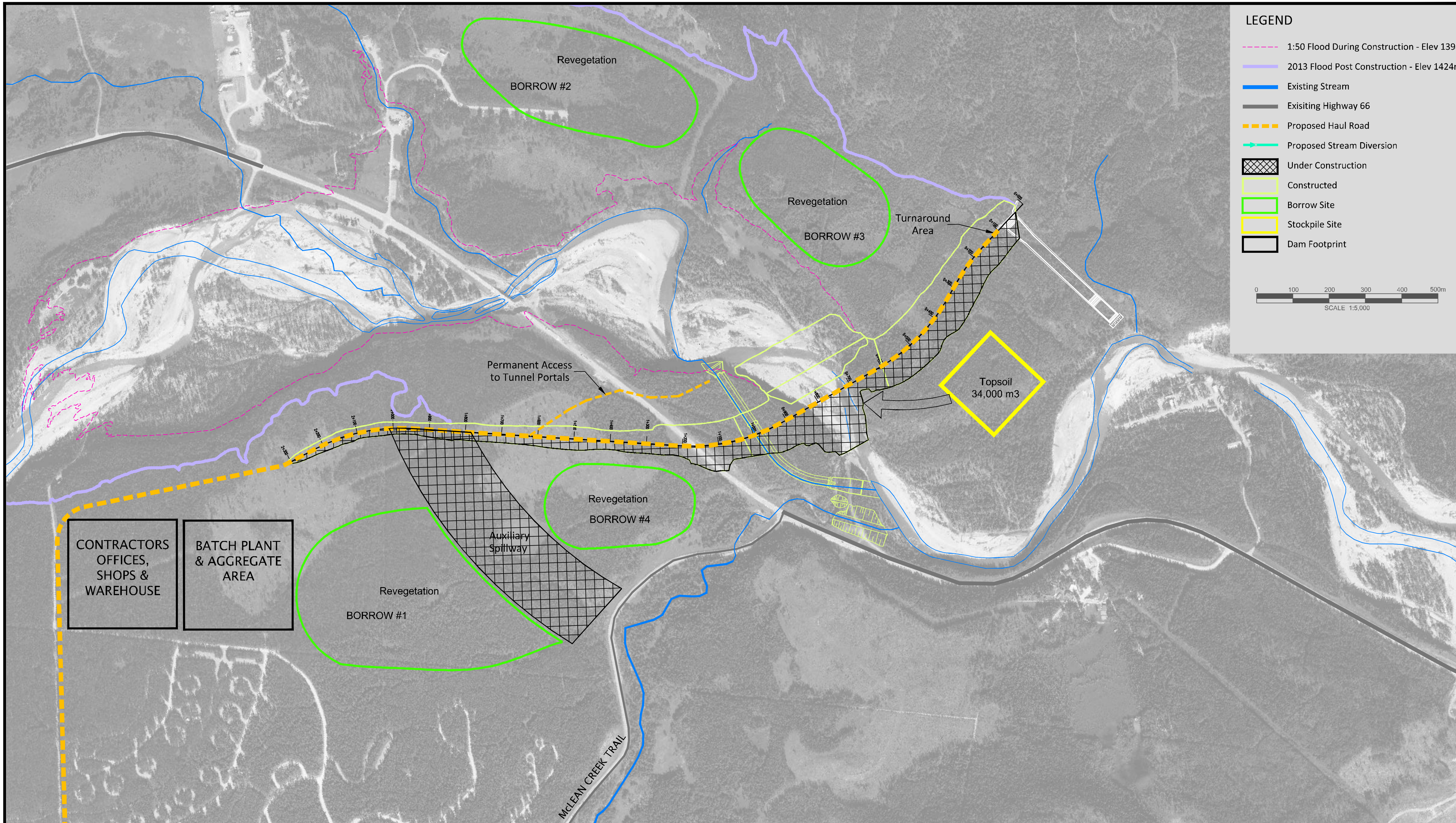

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CONSTRUCTION STAGING
FALL AWARD – YEAR 4

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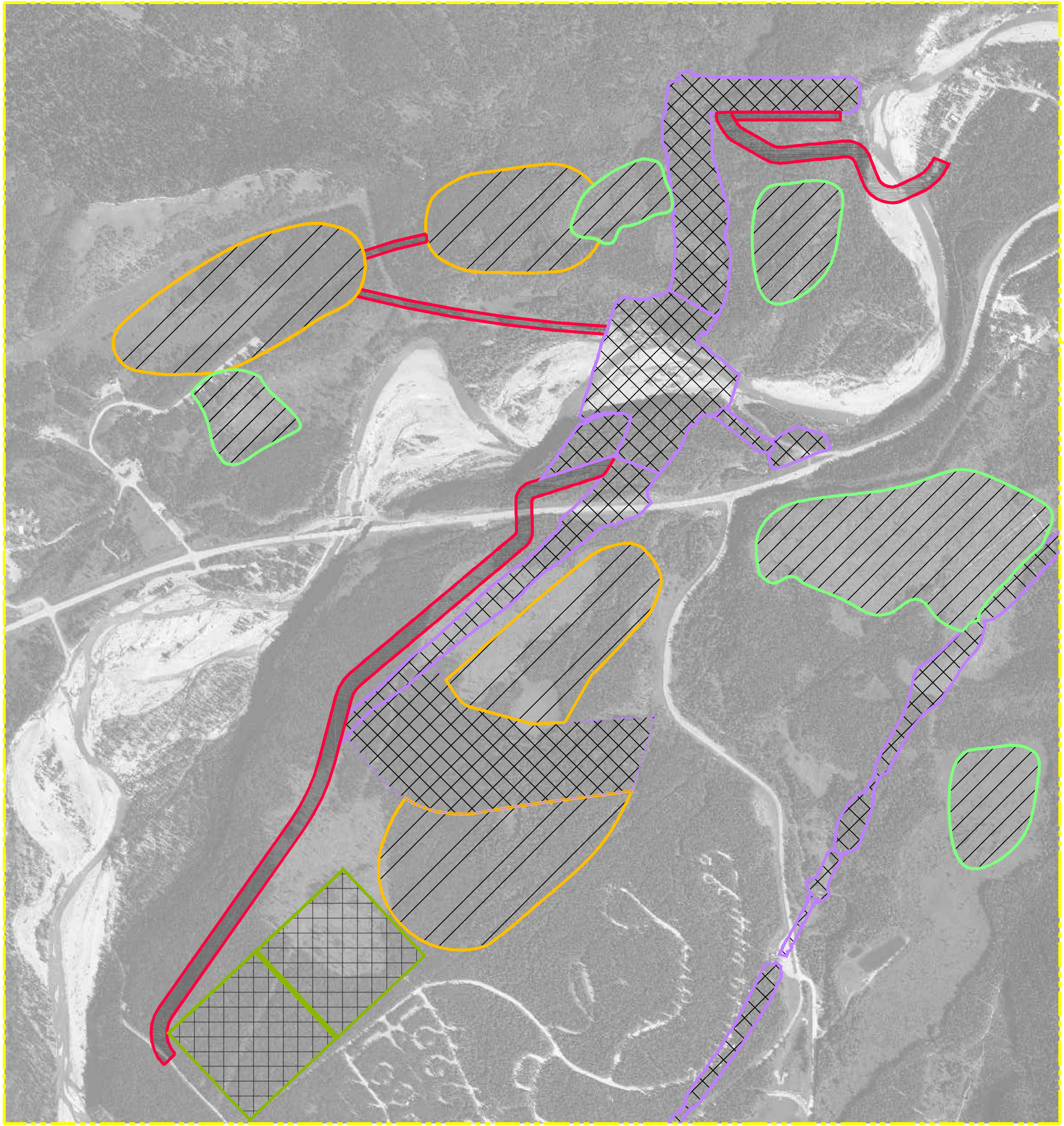

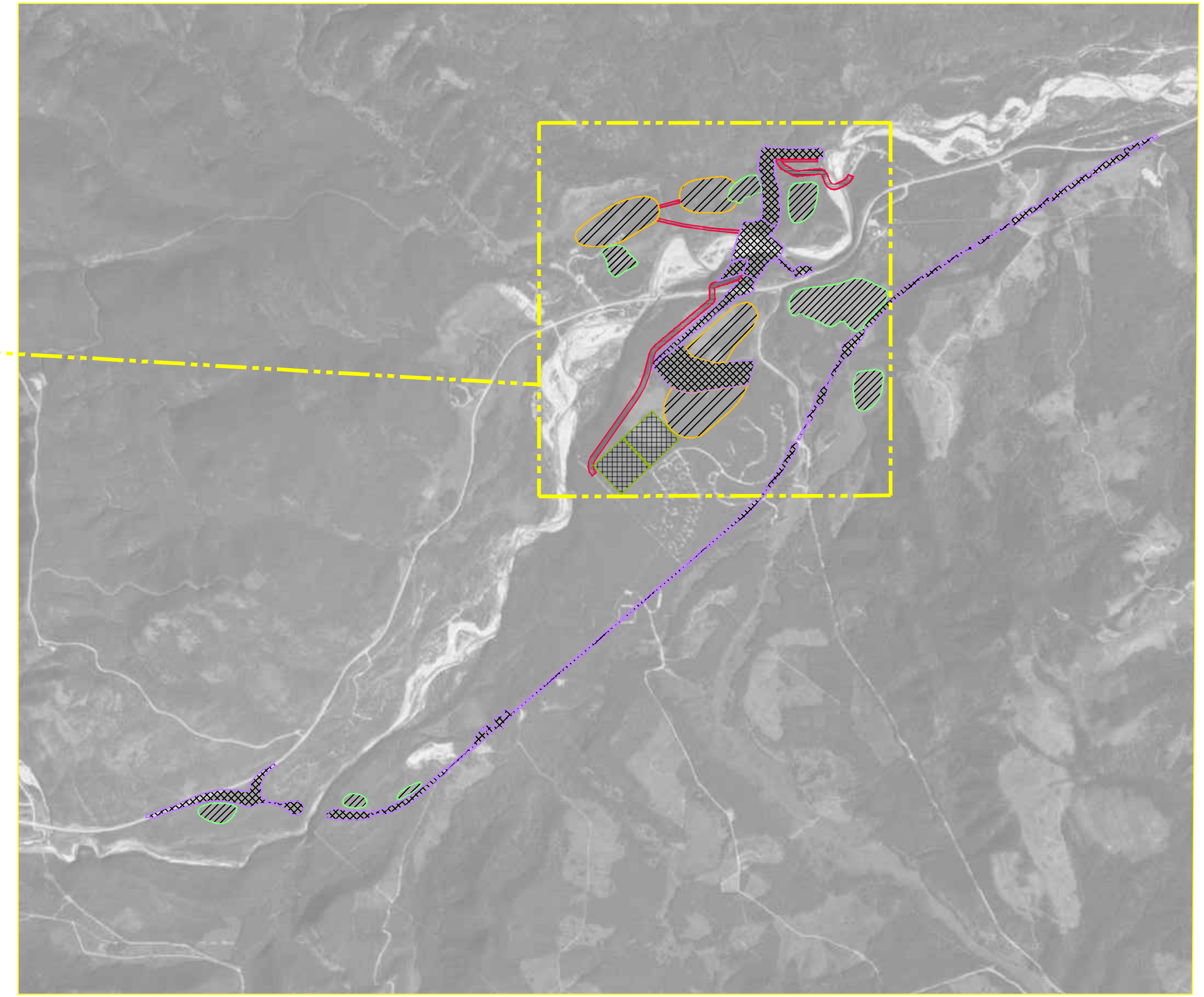

LEGEND

- Permanent Disturbed Areas
- Surplus Disturbed Area
- Borrow Site
- Temporary Haul Road
- Laydown Area

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SURPLUS MATERIAL STORAGE AND BORROW SITE

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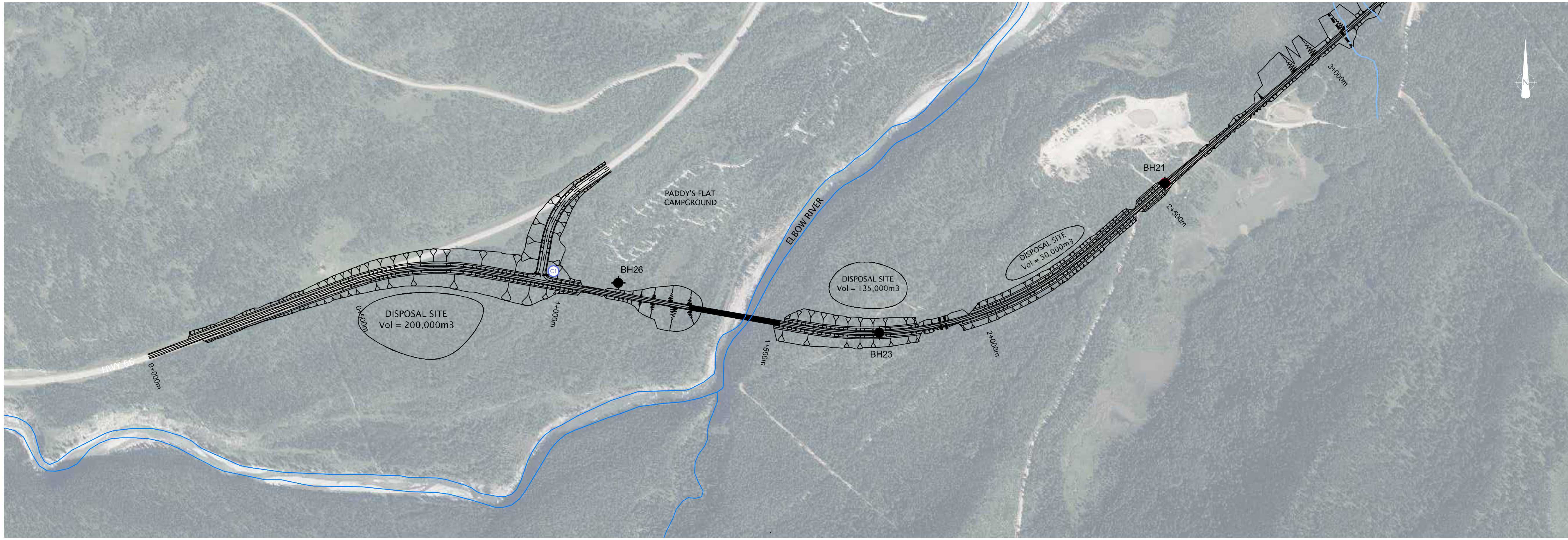

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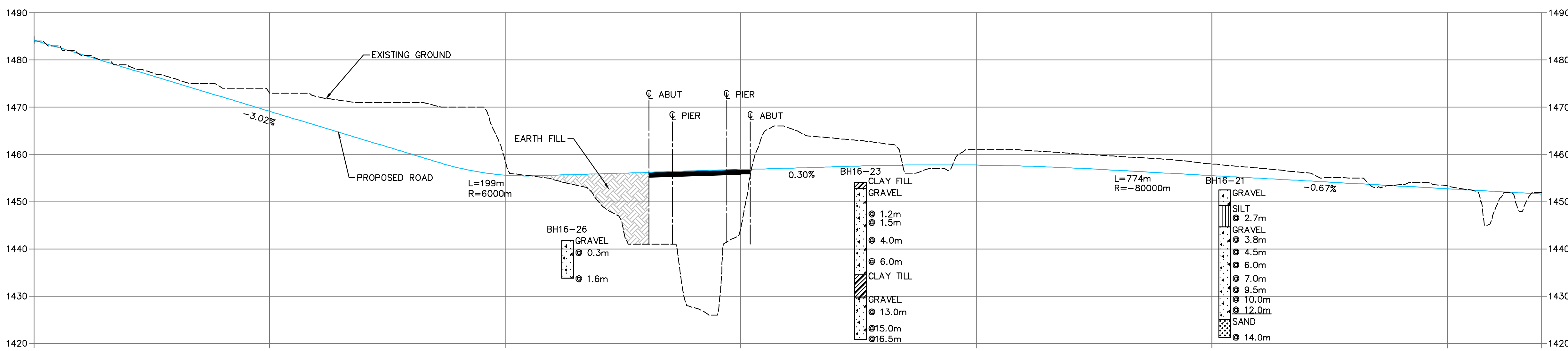

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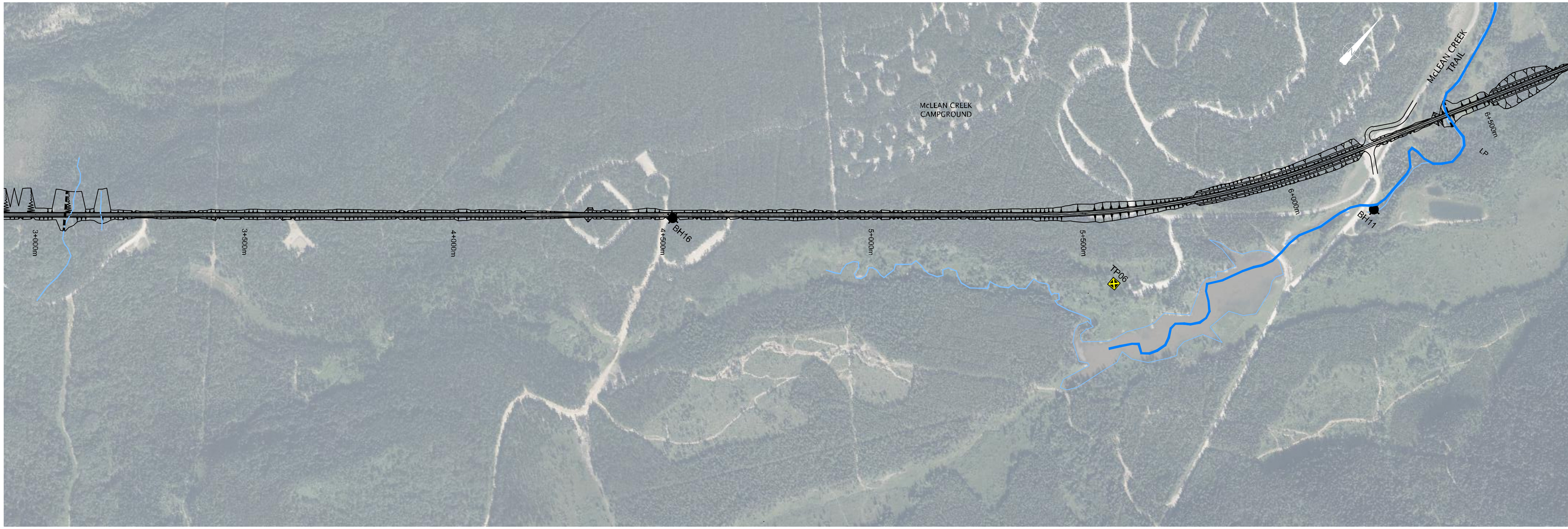
WATER MANAGEMENT SECTION

MCLEAN CREEK

ROAD PLAN AND PROFILE
SHEET 1 OF 3

DWG. No.: R01_B

DATE: MAR. 30, 2017



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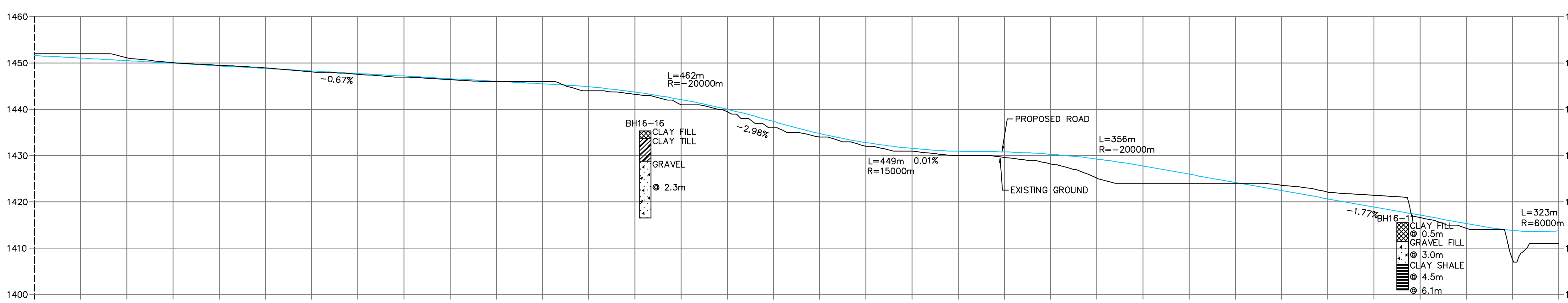

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WATER MANAGEMENT SECTION

MCLEAN CREEK

ROAD PLAN AND PROFILE
SHEET 2 OF 3

DWG. No.: R02

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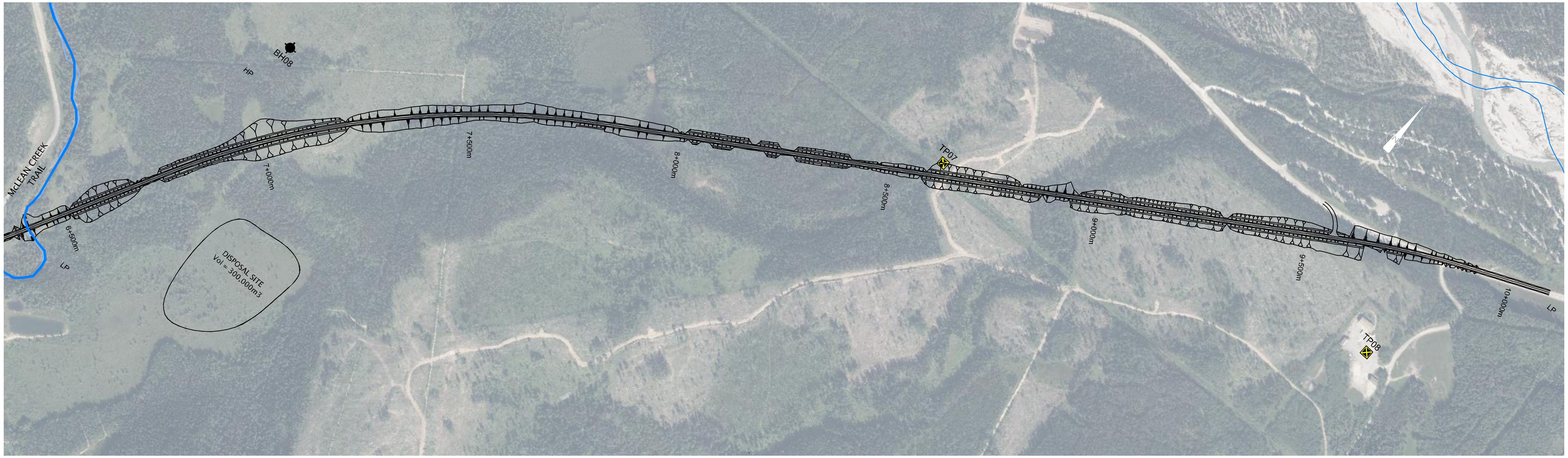

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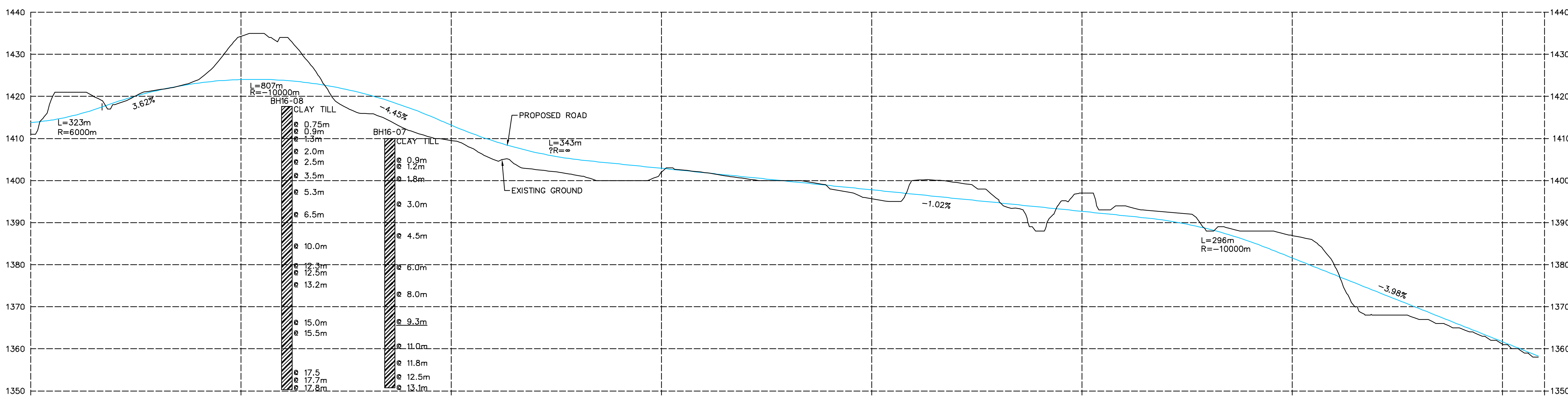

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WATER MANAGEMENT SECTION

MCLEAN CREEK

ROAD PLAN AND PROFILE
SHEET 3 OF 3

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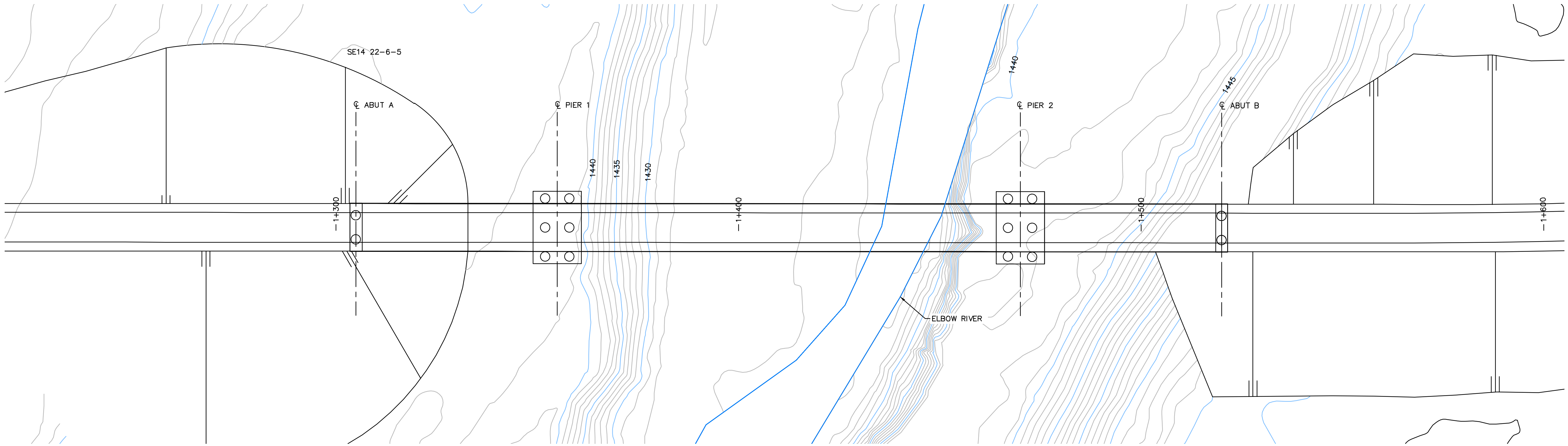

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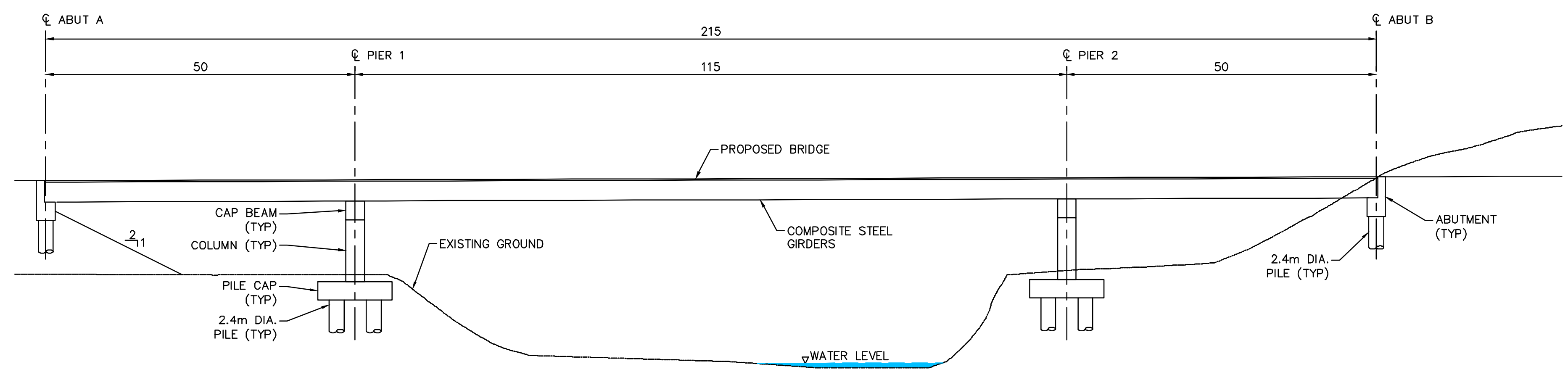

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ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

MCLEAN CREEK

PROPOSED BRIDGE
PLAN AND ELEVATION

DWG. No.: S01

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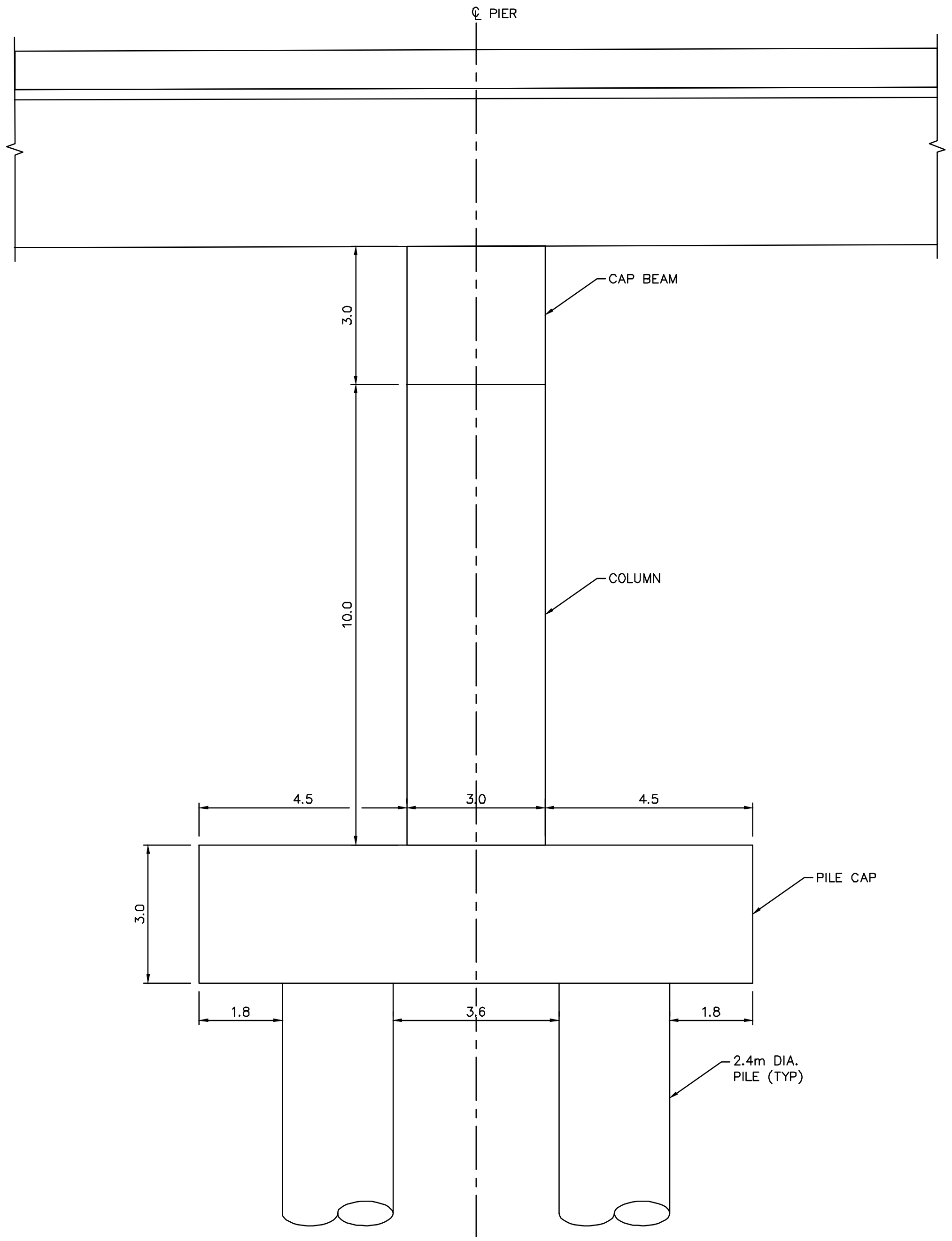

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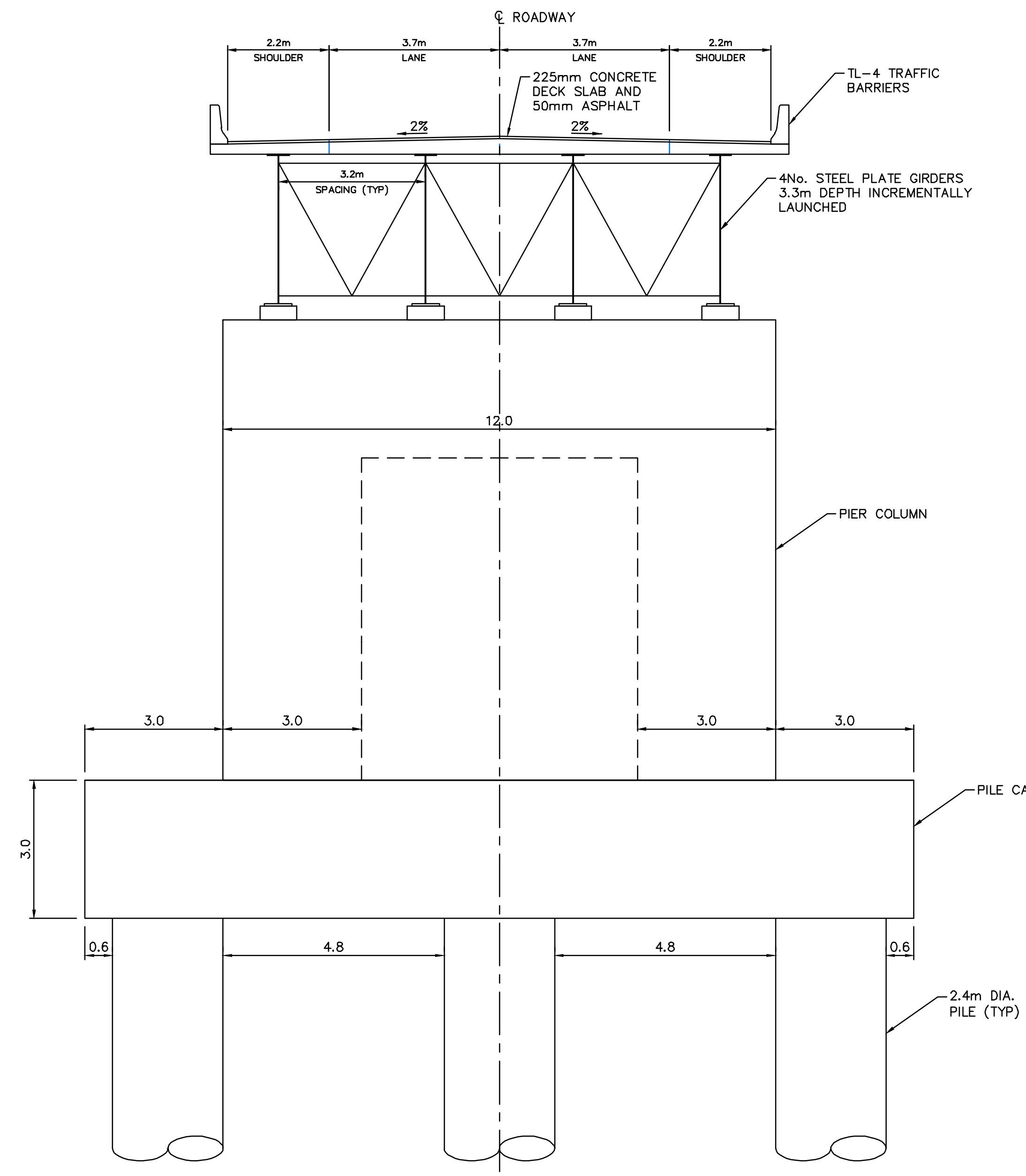


BRIDGE CROSS SECTION

Scale: 1:75(A1) 1:150(A3)

NOTES	No.	REVISIONS	DATE	BY	CHK'D	APP'D	DESIGNER	CHECKER

ENGINEERING RECORD

DESCRIPTION	INITIALS	DATE
SURVEYED		
DESIGNED		
DRAWN	AA	2017-03-24
CHECKED	TC	2017-03-27
ISSUED FOR APPROVAL		
ISSUED FOR TENDER		
ISSUED FOR CONSTRUCTION		

CONSULTANT

OPUS

PERMIT TO PRACTICE
PERMIT NUMBER P 292
The Association of Professional Engineers and Geoscientists of Alberta

AS SHOWN

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

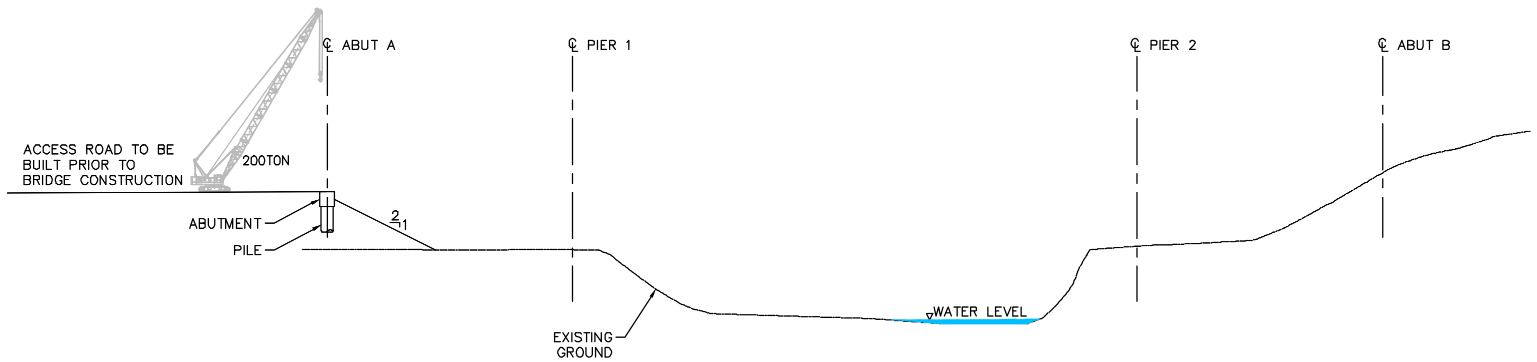
MCLEAN CREEK

PROPOSED BRIDGE
SECTIONS

DWG. No.: S02

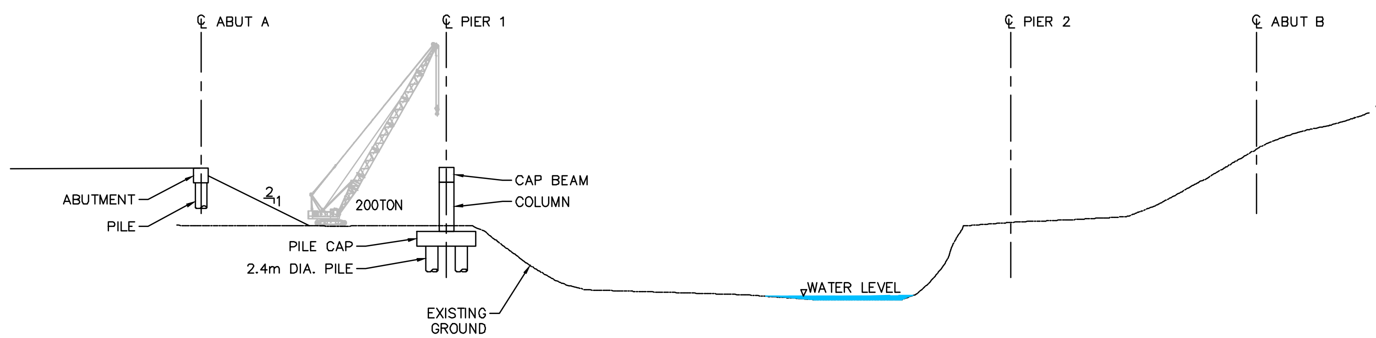
J:\39000\39001 ED60 McLennan Ross LLP\500CAD\515Bridge Plans\SP 3900160.A1 S01-S02 REV A GA and Cross Section.dwg

DATE: MAR. 30, 2017



STAGE 1

Scale: 1:750(A1) 1:1500(A3)



STAGE 2

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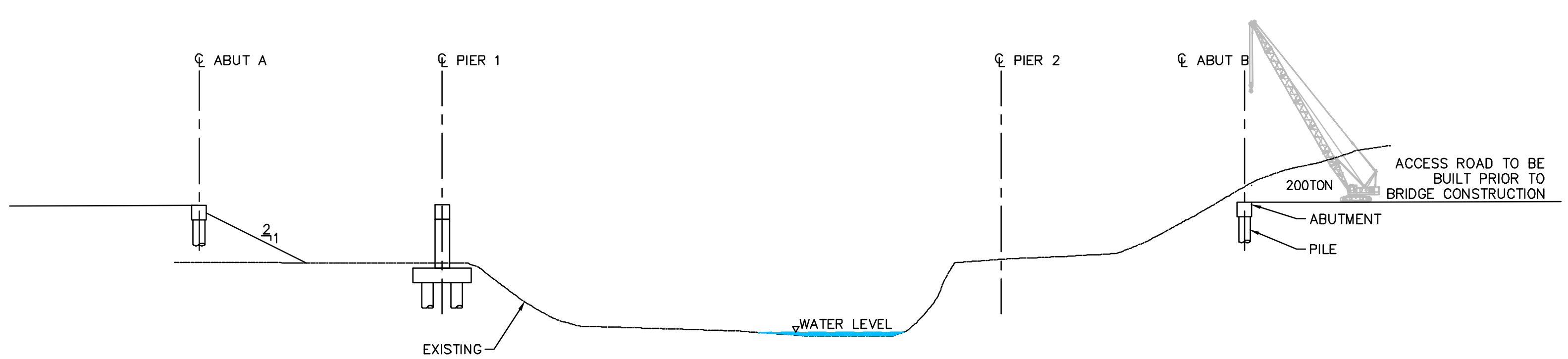

STAGE 3

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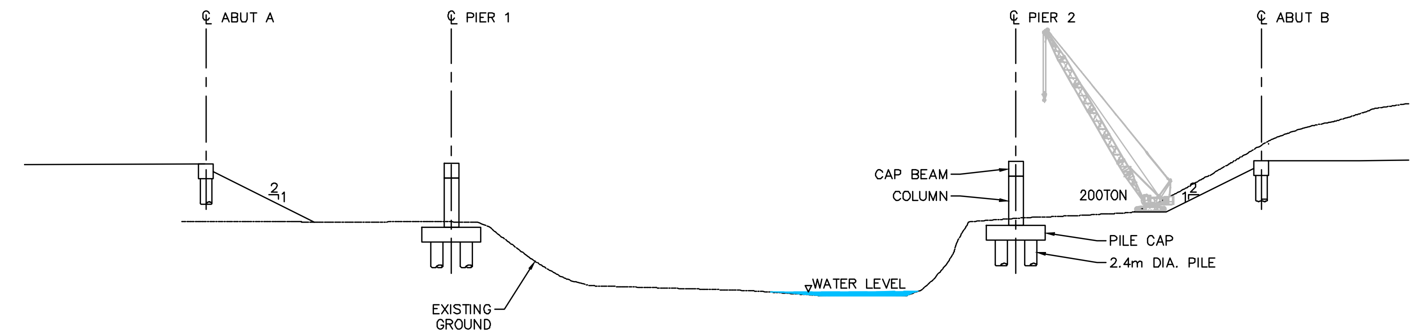

STAGE 4

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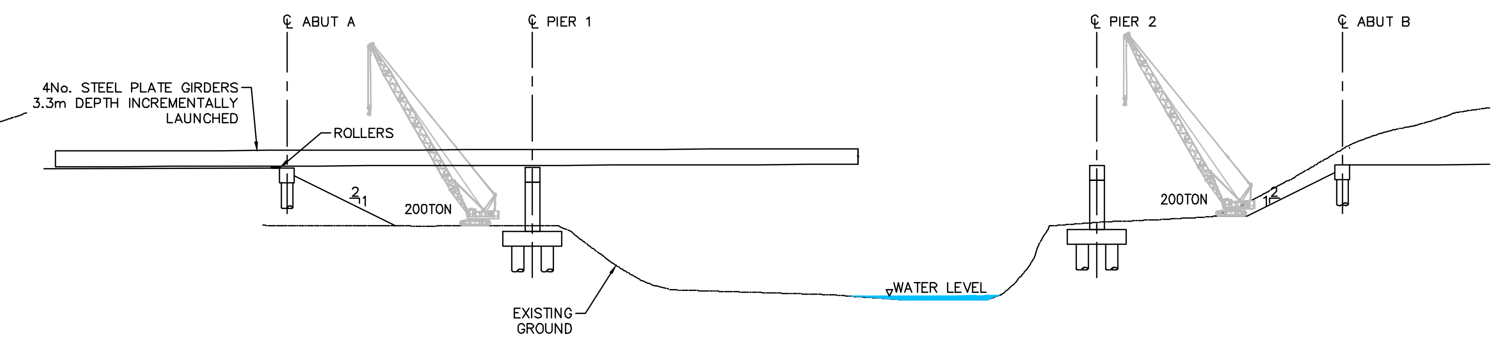
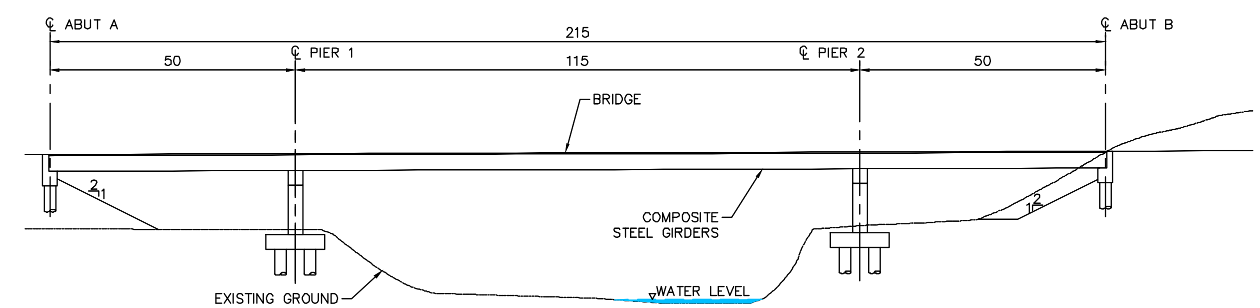


STAGE 5

Scale: 1:750(A1) 1:1500(A3)



STAGE 6

Scale: 1:750(A1) 1:1500(A3)

NOTES	No.	REVISIONS	DATE	BY	CHK'D	APP'D	DESIGNER	CHECKER

ENGINEERING RECORD

DESCRIPTION	INITIALS	DATE
SURVEYED		
DESIGNED		
DRAWN	AA	2017-03-27
CHECKED	TC	2017-03-27
ISSUED FOR APPROVAL		
ISSUED FOR TENDER		
ISSUED FOR CONSTRUCTION		

CONSULTANT

OPUS

PERMIT TO PRACTICE
PERMIT NUMBER P 292
The Association of Professional Engineers and Geoscientists of Alberta

AS SHOWN

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

MCLEAN CREEK

PROPOSED BRIDGE
CONSTRUCTION SEQUENCE

DWG. No.: S03

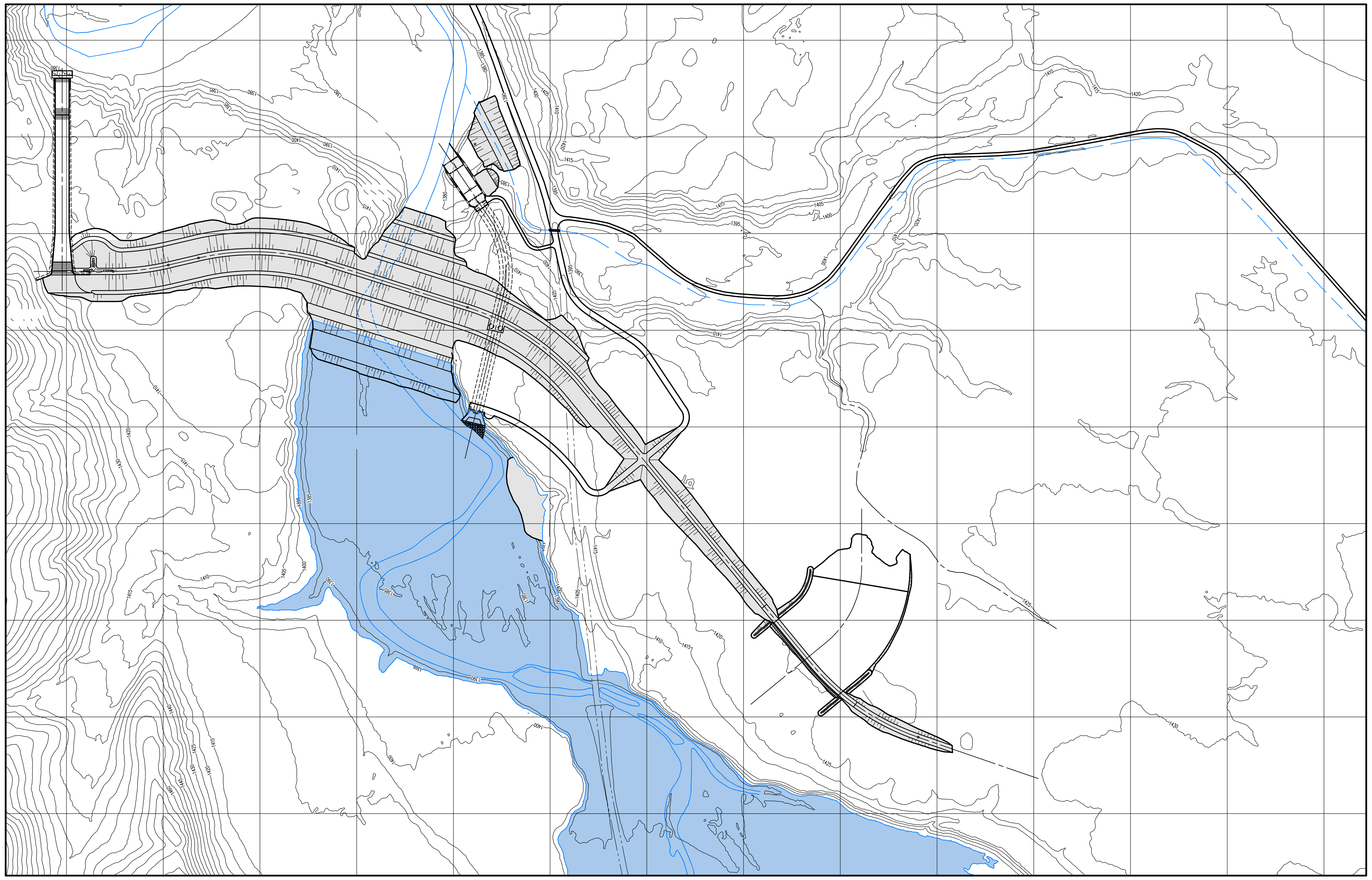
DATE: MAR. 30, 2017

J:\39000\39001 ED60 McLennan Ross LLP\500CAD\515Bridge Plans\SP 3900160.A1 S03 REV A Construction Sequence.dwg

T01
DWG. No.

McLEAN CREEK DAM MC1

ALBERTA TRANSPORTATION - WATER MANAGEMENT SECTION



0 100 200 300 400 500 METRES
SCALE 1:5

SITE PLAN MC1

LOCATION MAP

DRAWING LIST

TITLE	DRAWING NUMBER
DRAWING LIST AND KEY PLAN	T01
GENERAL ARRANGEMENT	D01
MAIN DAM AND ABUTMENT FILL TYPICAL CROSS SECTIONS	D02
FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PART PLAN AND PROFILE SHEET 1 OF 3	D03-A
FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PART PLAN AND PROFILE SHEET 2 OF 3	D03-B
FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PART PLAN AND PROFILE SHEET 3 OF 3	D03-C
SERVICE SPILLWAY PLAN AND PROFILE	D04
DIVERSION TUNNEL PLAN AND PROFILE	D05
TERRAIN MAP	G01
GEOLOGICAL PLAN	G02
GEOLOGICAL PROFILES	G03
BORROW AREA PLAN	G04
BORROW AREA PROFILES	G05

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME
SIGNATURE
ENG REG NUMBER
REVISION DATE
ORIGINAL DATE
REG. PROFESSIONAL

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0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10

REVISIONS

HATCH

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
DRAWING LIST
AND KEY PLAN

SCALE AS NOTED OR AS NOTED
DWG. No. T01
REV 0

SHEET SIZE: B1

$USERNAME$ $DATE$ $FILE$ $TIME$

0 60 120 180 240 300 METRES

SCALE: 1:3000 [B1 FRAME]

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME	
SIGNATURE	
ENG REG NUMBER	
REVISION DATE	
ORIGINAL DATE	
REG. PROFESSIONAL	

No.	DESCRIPTION	BY	CHK'D	DATE
0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10
REVISIONS				

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HATCH

OPUS STEWART WEIR

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1

MC1 GENERAL ARRANGEMENT

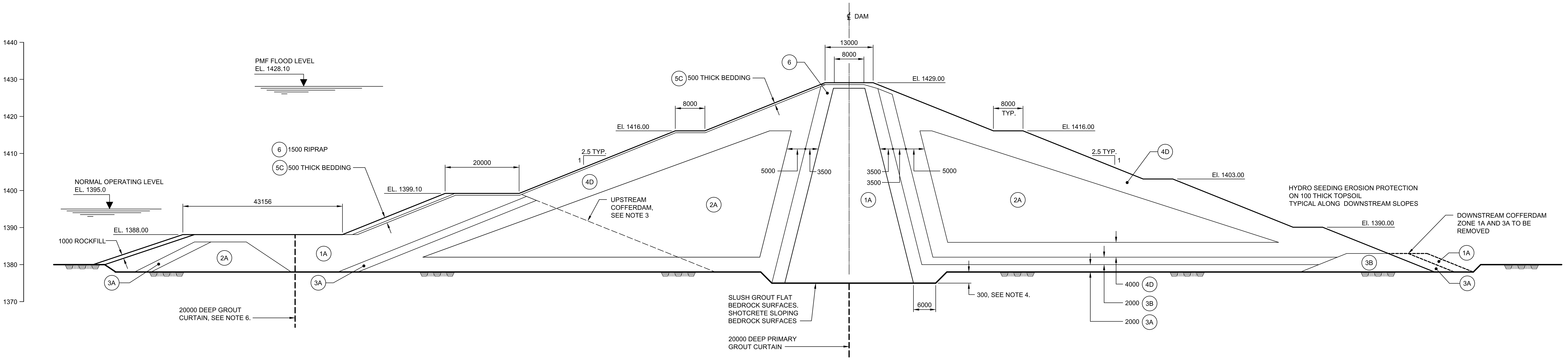
SCALE	DWG. No.	REV
AS NOTED OR AS NOTED	D01	0

SHEET SIZE: B1

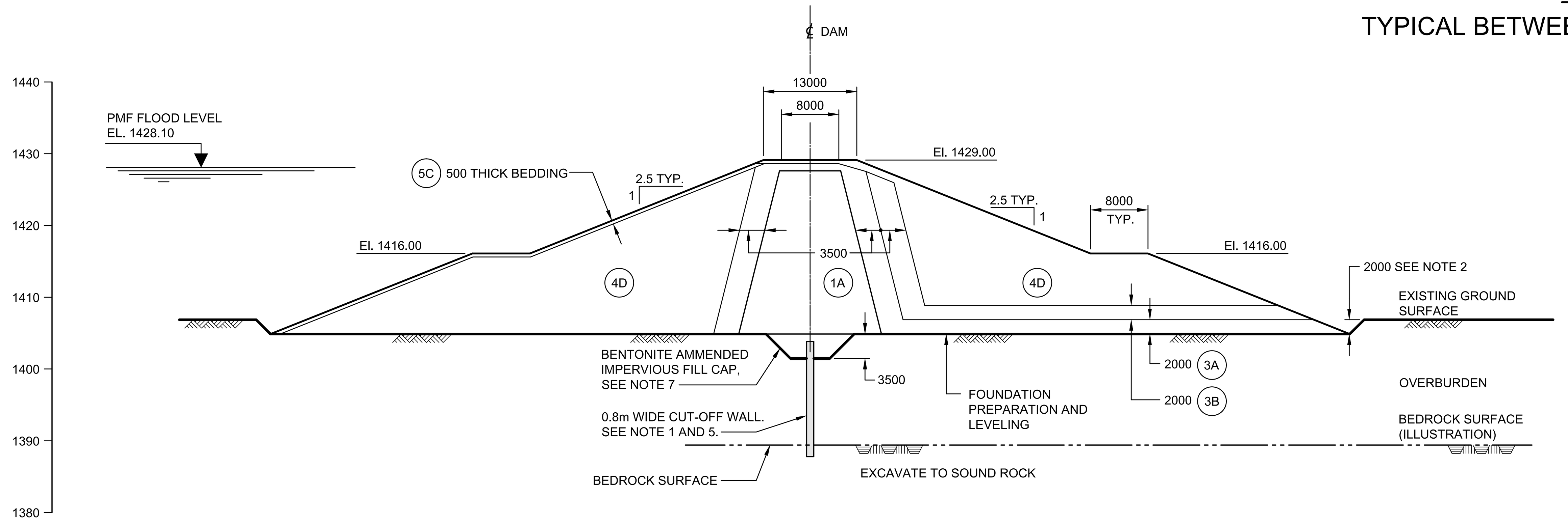
$USERNAME$ $DATE$ $TIME$ $FILE$

DWG. No. D02

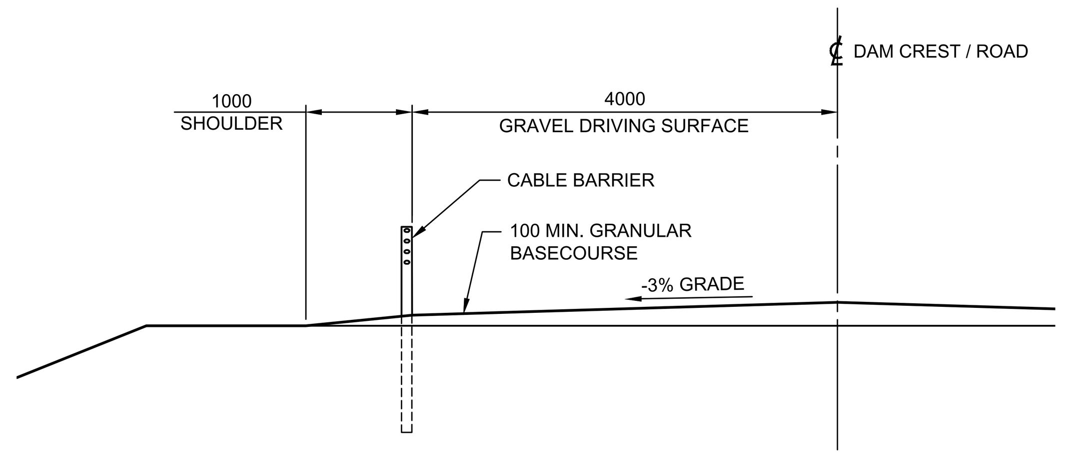
DIMENSIONS SHOWN IN MILLIMETRES
ELEVATIONS SHOWN IN METRES



SECTION A-A
TYPICAL BETWEEN CHAINAGES 0+550 & 0+950



SECTION B-B
TYPICAL BETWEEN CHAINAGES 0+100 & 0+550, 0+950



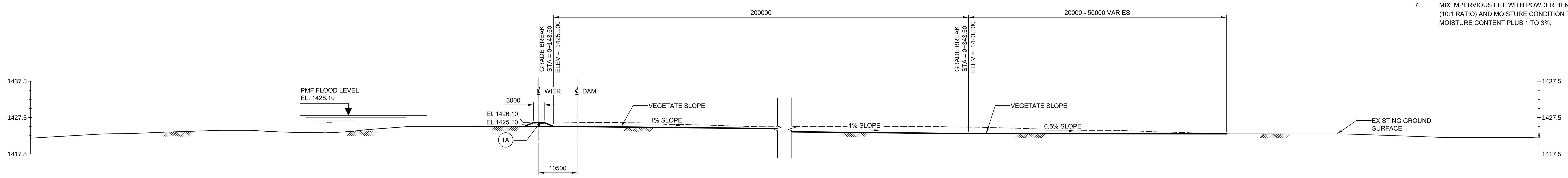
ROAD SURFACE DETAIL
TYPICAL ALONG LENGTH OF DAM CREST
SCALE 1:50

LEGEND

- 1A IMPERVIOUS FILL
- 2A UNCLASSIFIED FILL
- 3A FINE FILTER
- 3B DRAINAGE LAYER
- 4D PIT RUN GRANULAR
- 5C RIPRAP BEDDING / COBBLES
- 6 RIPRAP

NOTES:

1. CUT-OFF WALL WILL BE PLASTIC CONCRETE ON THE LEFT ABUTMENT AND CEMENT BENTONITE ON THE RIGHT ABUTMENT.
2. 2 METRE FOUNDATION EXCAVATION IS ASSUMED UNDER THE DAM FOOTPRINT ON OVERBURDEN.
3. UPSTREAM COFFERDAM ELEVATION IS SELECTED AT EL. 1399.1 WHICH IS EQUIVALENT TO THE 50 YEAR FLOOR LEVEL DURING CONSTRUCTION.
4. 3m ROCK EXCAVATION IS REQUIRED UNDER THE CORE AND FILTERS.
5. ON LEFT ABUTMENT, CUT-OFF WALL WILL BE CONSTRUCTED IN 3.5 METRE PANELS USING BENTONITE SLURRY FOR STABILIZATION. PLASTIC CONCRETE WILL BE TREMIED IN PANELS REPLACING SLURRY. ON RIGHT ABUTMENT, CEMENT BENTONITE SLURRY WALL WILL BE USED FOR STABILIZATION AND FINAL CUT-OFF WALL.
6. THE GROUT CURTAIN TO BE INSTALLED FROM EL. 1388.0 IF REQUIRED.
7. MIX IMPERVIOUS FILL WITH POWDER BENTONITE (10:1 RATIO) AND MOISTURE CONDITION TO OPTIMUM MOISTURE CONTENT PLUS 1 TO 3%.



SECTION C-C
AUXILIARY SPILLWAY CENTERLINE PROFILE

0 10 20 30 40 50 METRES
SCALE: 1:500 [B1 SHEET]

DRAWING No.	DRAWING TITLE
-	
REFERENCE DRAWINGS	

NAME	
SIGNATURE	
ENG REG NUMBER	
REVISION DATE	
ORIGINAL DATE	
REG. PROFESSIONAL	

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No.	DESCRIPTION	BY	CHK'D	DATE
0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10
REVISIONS				

HATCH

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

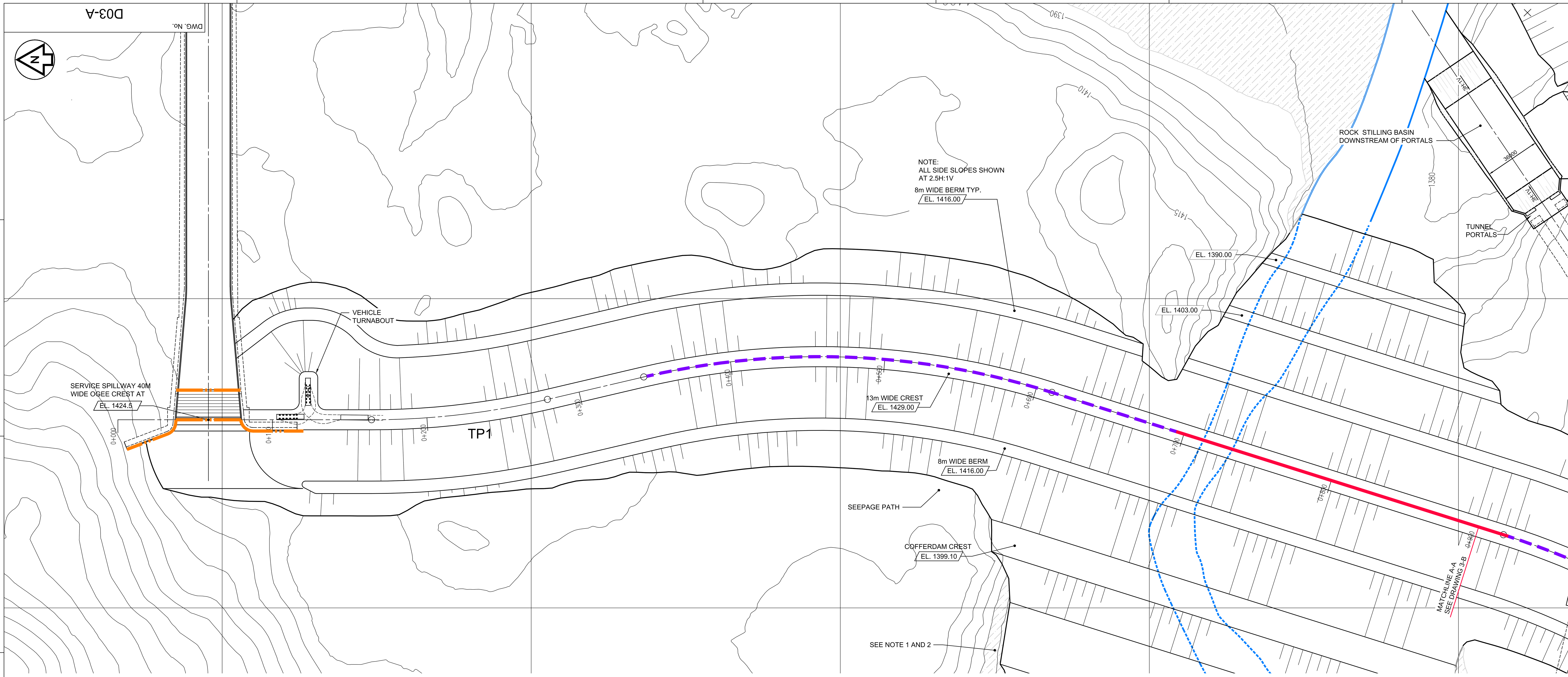
McLEAN CREEK DAM MC1
MAIN DAM AND ABUTMENT FILL
TYPICAL CROSS SECTIONS

SCALE AS NOTED OR AS NOTED | DWG. No. D02 | REV 0

SHEET SIZE: B1

$USERNAMES $DATES $FILES $TIMES

DIMENSIONS SHOWN IN MILLIMETRES
ELEVATIONS SHOWN IN METRES



PLAN AT STA 0+000 TO 0+900

SCALE 1:1250

NOTES:

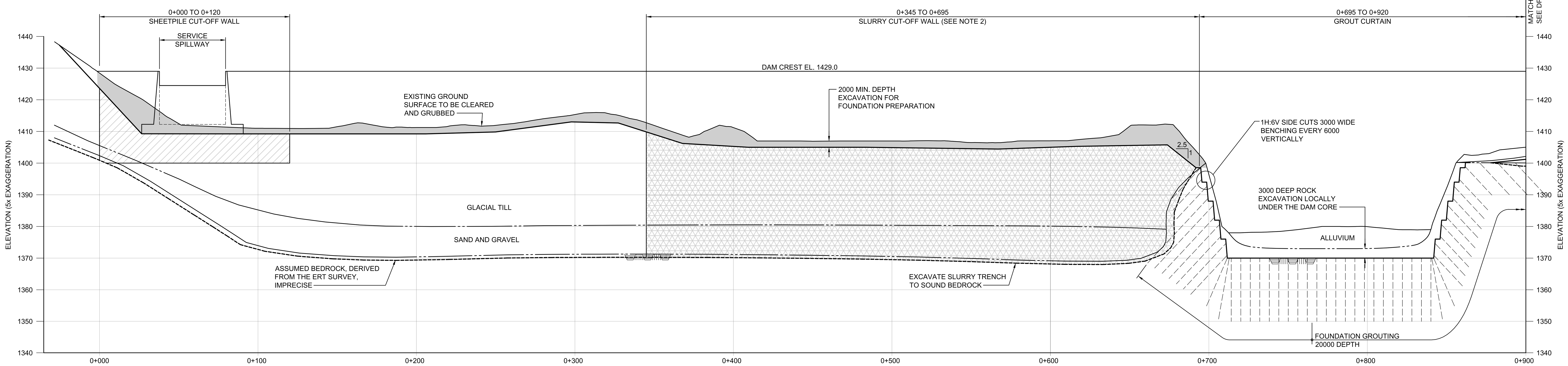
1. SHADED REGION REPRESENTS SUSPECTED DAYLIGHTING OF SAND AND GRAVEL LAYER. IT IS EXPECTED THAT SEEPAGE WILL OCCUR THROUGH THIS LAYER. TEST TRENCHES ARE TO BE EXCAVATED PRIOR TO FINAL DESIGN TO PINPOINT THE LAYER' LOCATION. CONSTRUCT IMPERVIOUS FILL BLANKET OVER EXPOSED SAND AND GRAVEL WITHIN A DISTANCE OF 12 TIMES THE HYDRAULIC HEAD ACTING ON THE DAM.
2. MAINTAIN A MINIMUM SEEPAGE PATH OF 625m IN SAND AND GRAVEL LAYER.

LEGEND - PLAN

- SHEETPILE COLLAR
- SLURRY CUT-OFF WALL
- GROUT CURTAIN
- DAYLIGHT OF SAND AND GRAVEL BLANKET (SEE NOTE 1)

LEGEND - PROFILE

- 2000 MIN. DEEP EXCAVATION FOR FOUNDATION PREPARATION
- SLURRY CUT-OFF WALL
- SHEETPILE COLLAR



FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PROFILE STA 0+000 TO 0+900

SCALE 1:1250 (HORIZONTAL) / 1:625 (VERTICAL)

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

HORIZONTAL 1:1250 0 25 50 75 100 125 METRES
VERTICAL 1:625 0 12.5 25 37.5 50 62.5 METRES

NAME
SIGNATURE
ENG REG NUMBER
REVISION DATE
ORIGINAL DATE
REG. PROFESSIONAL

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HATCH

No.	DESCRIPTION	BY	CHK'D	DATE
0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10

REVISIONS

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

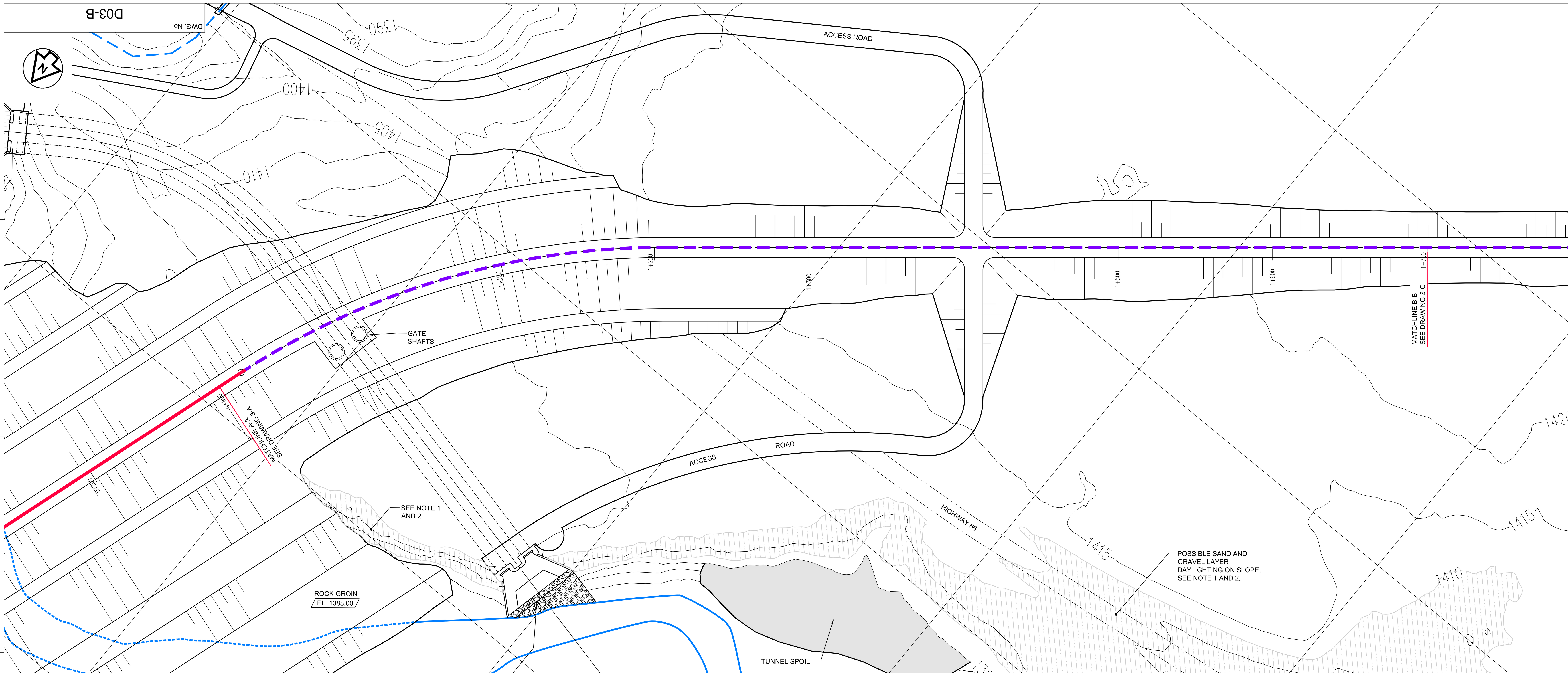
ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PART PLAN AND PROFILE SHEET 1 OF 3

SCALE AS NOTED OR AS NOTED | DWG. No. D03-A | REV 0

SHEET SIZE: B1

DIMENSIONS SHOWN IN MILLIMETRES
ELEVATIONS SHOWN IN METRES



PLAN AT STA 0+900 TO 1+700

SCALE 1:1250

NOTES:

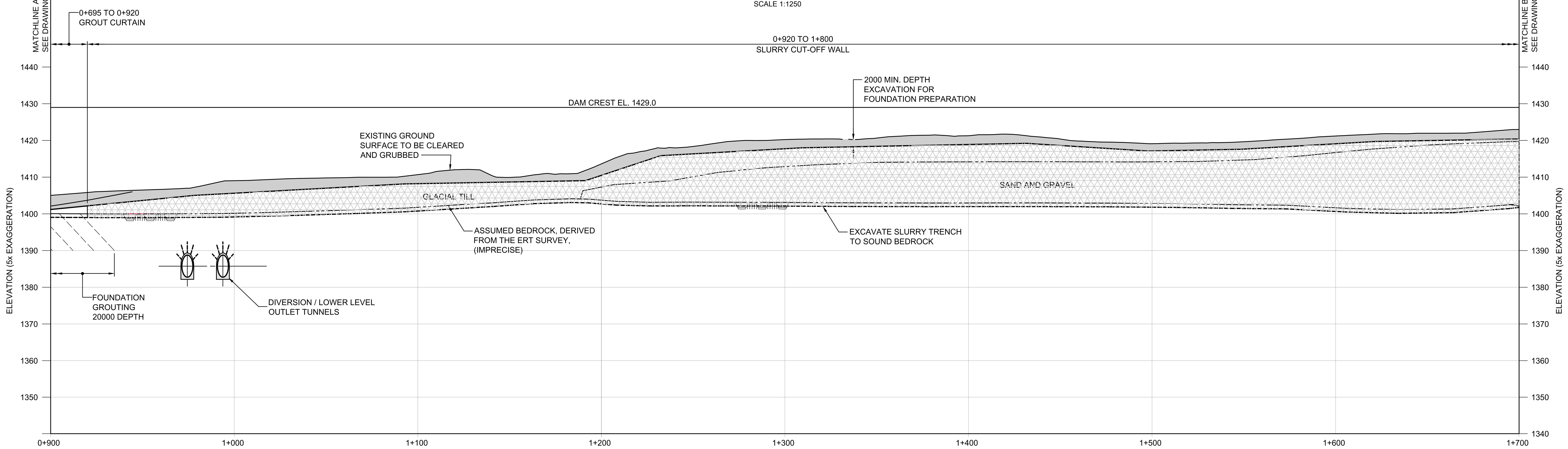
1. SHADED REGION REPRESENTS SUSPECTED DAYLIGHTING OF SAND AND GRAVEL LAYER. IT IS EXPECTED THAT SEEPAGE WILL OCCUR THROUGH THIS LAYER. TEST TRENCHES ARE TO BE EXCAVATED PRIOR TO FINAL DESIGN TO PINPOINT THE LAYER' LOCATION. CONSTRUCT IMPERVIOUS FILL BLANKET OVER EXPOSED SAND AND GRAVEL WITHIN A DISTANCE OF 12 TIMES THE HYDRAULIC HEAD ACTING ON THE DAM.
2. MAINTAIN A MINIMUM SEEPAGE PATH OF 625m IN SAND AND GRAVEL LAYER.

LEGEND - PLAN

- SLURRY CUT-OFF WALL
- GROUT CURTAIN
- DAYLIGHT OF SAND AND GRAVEL BLANKET (SEE NOTE 1)

LEGEND - PROFILE

- 2000 MIN. DEEP EXCAVATION FOR FOUNDATION PREPARATION
- SLURRY CUT-OFF WALL
- SHEETPILE COLLAR



FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PROFILE STA 0+900 TO 1+700

SCALE 1:1250 (HORIZONTAL) / 1:625 (VERTICAL)

HORIZONTAL 1:1250 0 25 50 75 100 125 METRES
VERTICAL 1:625 0 12.5 25 37.5 50 62.5 METRES

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME
SIGNATURE
ENG REG NUMBER
REVISION DATE
ORIGINAL DATE
REG. PROFESSIONAL

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No.	DESCRIPTION	BY	CHK'D	DATE
0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10

REVISIONS

HATCH

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

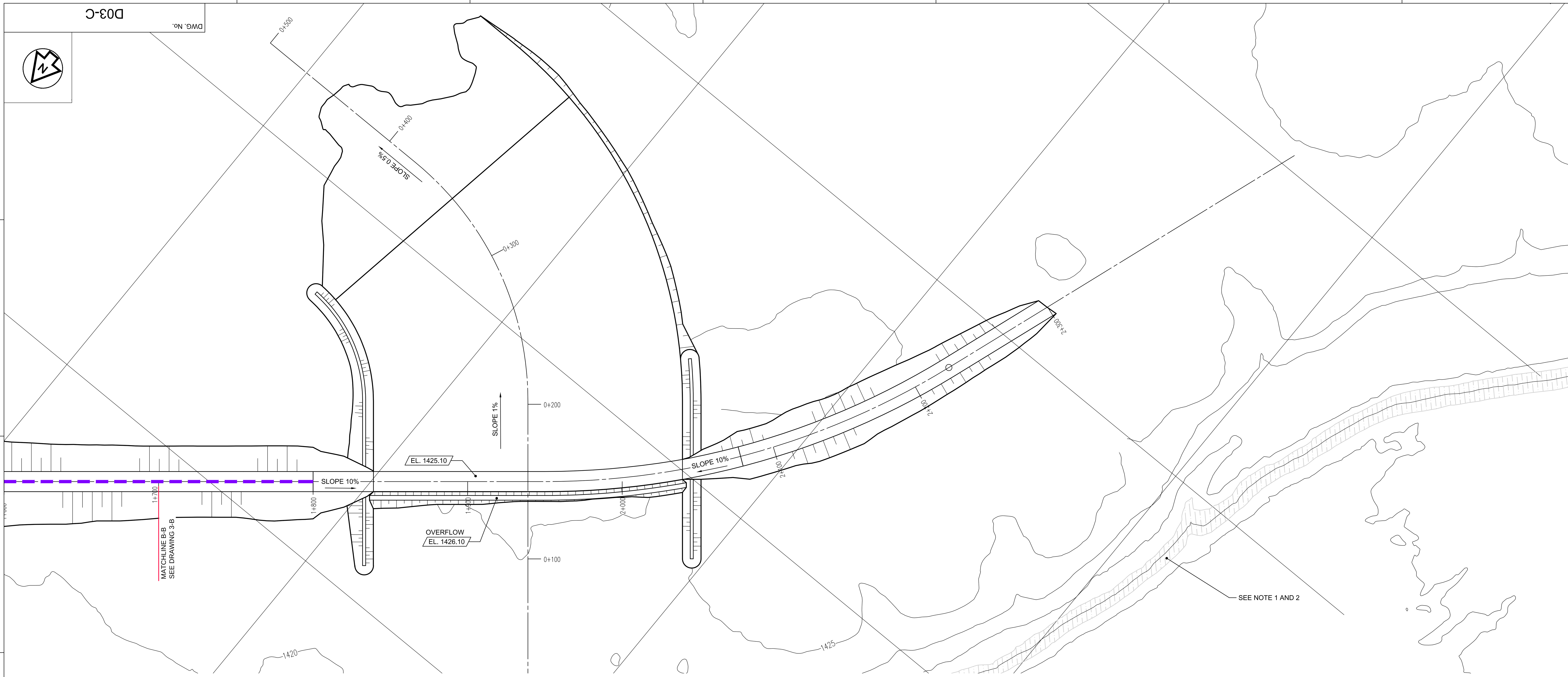
McLEAN CREEK DAM MC1
FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PART PLAN AND PROFILE SHEET 2 OF 3

SCALE AS NOTED | DWG. No. D03-B | REV 0

SHEET SIZE: B1

$USERNAMES $DATES $FILES $TIMES

DIMENSIONS SHOWN IN MILLIMETRES
ELEVATIONS SHOWN IN METRES



PLAN AT 1+700 TO 2+300

SCALE 1:1250

NOTES:

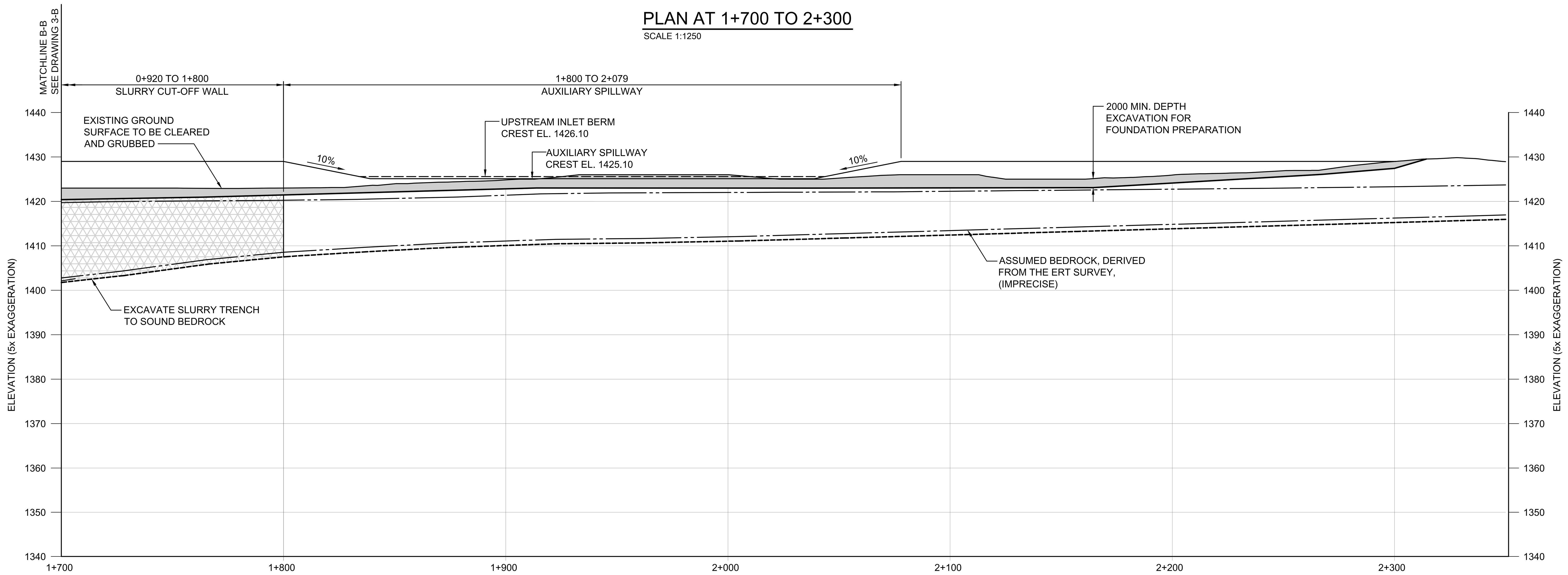
1. SHADED REGION REPRESENTS SUSPECTED DAYLIGHTING OF SAND AND GRAVEL LAYER. IT IS EXPECTED THAT SEEPAGE WILL OCCUR THROUGH THIS LAYER. TEST TRENCHES ARE TO BE EXCAVATED PRIOR TO FINAL DESIGN TO PINPOINT THE LAYER' LOCATION. CONSTRUCT IMPERVIOUS FILL BLANKET OVER EXPOSED SAND AND GRAVEL WITHIN A DISTANCE OF 12 TIMES THE HYDRAULIC HEAD ACTING ON THE DAM.
2. MAINTAIN A MINIMUM SEEPAGE PATH OF 625m IN SAND AND GRAVEL LAYER.

LEGEND - PLAN

- SLURRY CUT-OFF WALL
- DAYLIGHT OF SAND AND GRAVEL BLANKET (SEE NOTE 1)

LEGEND - PROFILE

- 2000 MIN. DEEP EXCAVATION FOR FOUNDATION PREPARATION
- SLURRY CUT-OFF WALL



FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PROFILE STA 1+700 TO 2+300

SCALE 1:1250 (HORIZONTAL) / 1:625 (VERTICAL)

HORIZONTAL 1:1250 0 25 50 75 100 125 METRES
VERTICAL 1:625 0 12.5 25 37.5 50 62.5 METRES

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME

SIGNATURE

ENG REG NUMBER

REVISION DATE

ORIGINAL DATE

REG. PROFESSIONAL

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No.	DESCRIPTION	BY	CHK'D	DATE
0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10

REVISIONS

HATCH

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
FOUNDATION EXCAVATION CUT OFF WALL AND GROUTING PART PLAN AND PROFILE SHEET 3 OF 3

SCALE AS NOTED OR AS NOTED | DWG. No. D03-C | REV 0

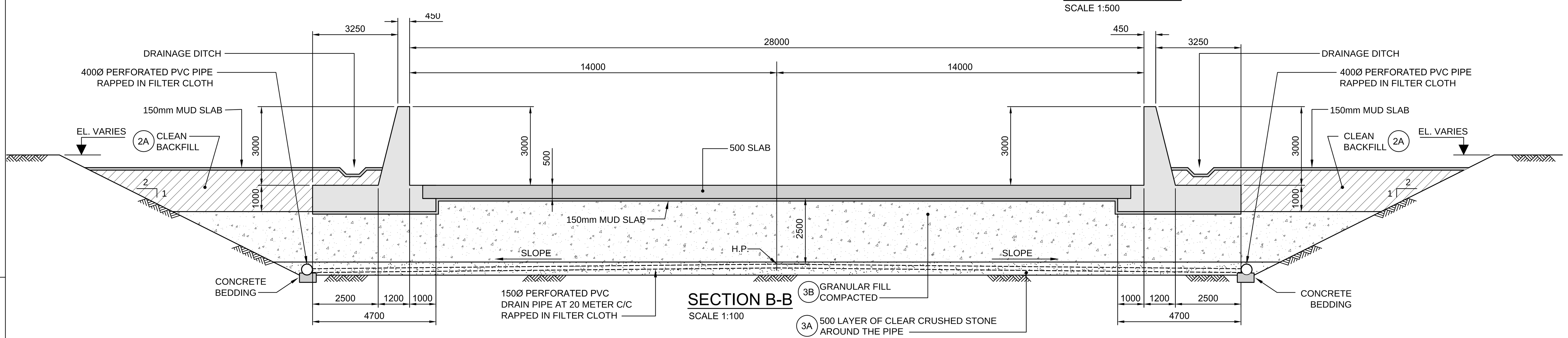
SHEET SIZE: B1

$USERNAMES $DATES $FILES $TIMES

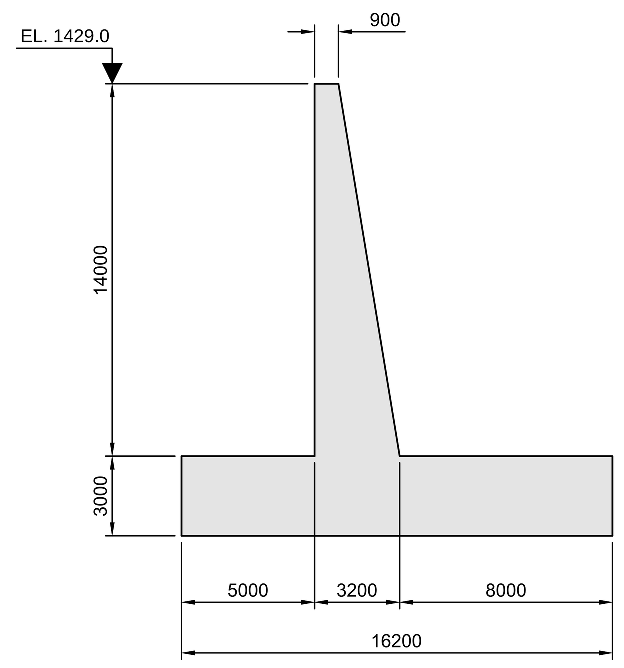
DIMENSIONS SHOWN IN MILLIMETRES
ELEVATIONS SHOWN IN METRES

PLAN
SCALE 1:500

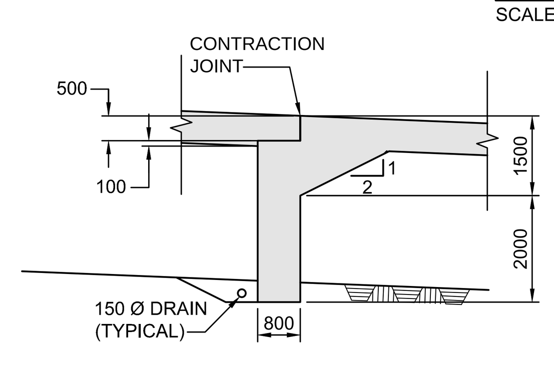
SECTION A - A
SCALE 1:500



SECTION B-B
SCALE 1:100



SECTION C-C
SCALE 1:200



DETAIL 1
SCALE 1:100

SCALE: 1:500 (0, 10, 20, 30, 40, 50 METRES)

LEGEND

- 2A UNCLASSIFIED FILL
- 3A FINE FILTER
- 3B DRAINAGE LAYER

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME
SIGNATURE
ENG REG NUMBER
REVISION DATE
ORIGINAL DATE
REG. PROFESSIONAL

No.	DESCRIPTION	BY	CHK'D	DATE
0	ISSUED AS BASIS OF ESTIMATE	KK	DP	17-07-10

REVISIONS

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HATCH

OPUS STEWART WEIR

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

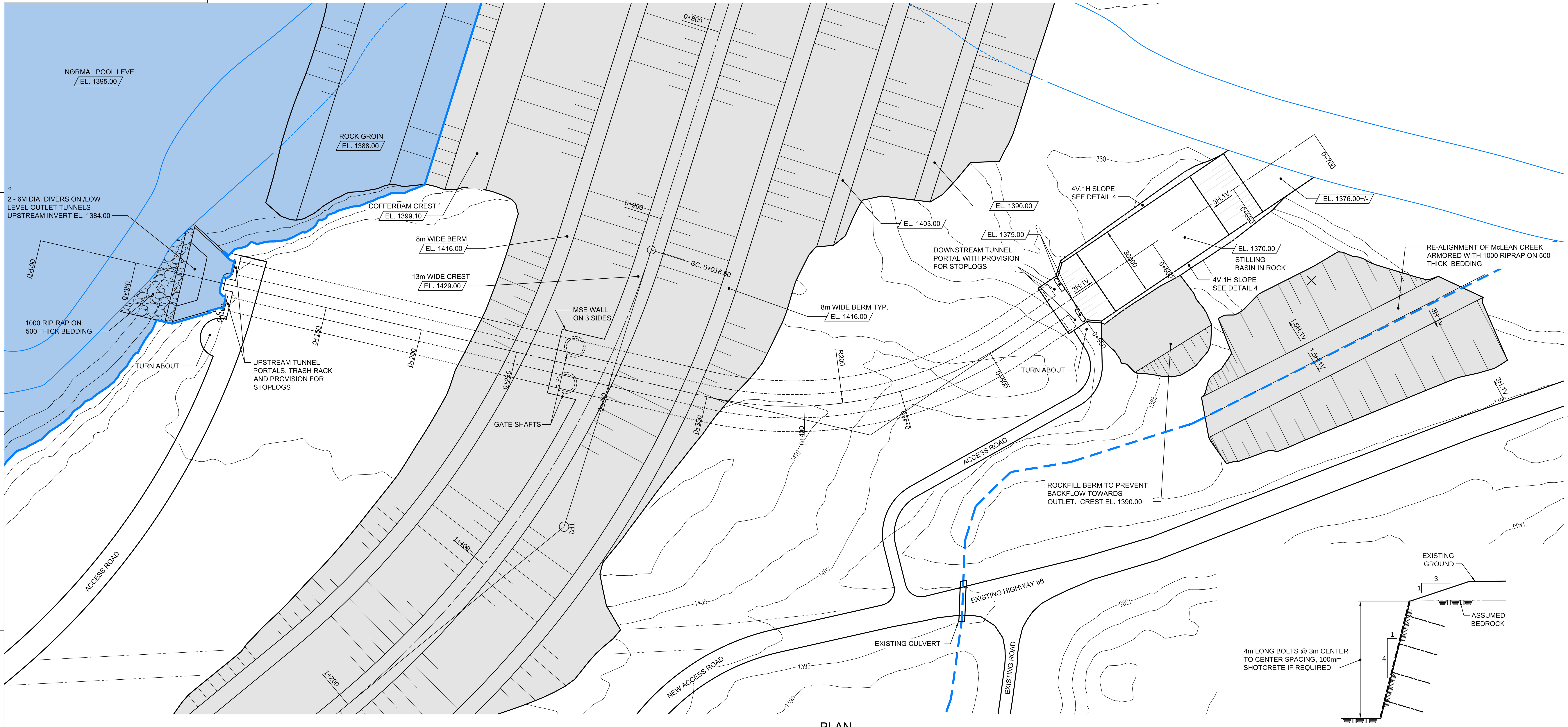
DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
SERVICE SPILLWAY
PLAN AND PROFILE

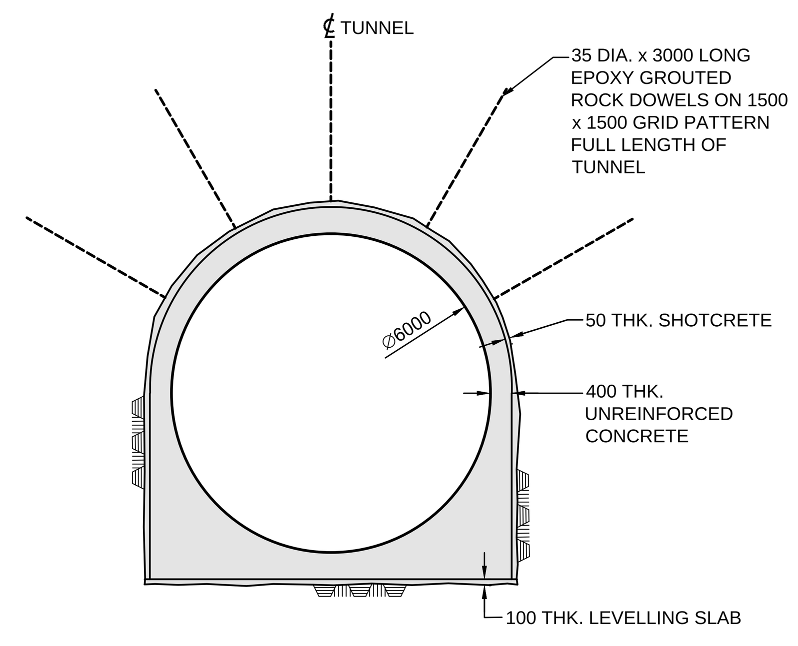
SCALE AS NOTED OR AS NOTED | DWG. No. D04 | REV 0

SHEET SIZE: B1



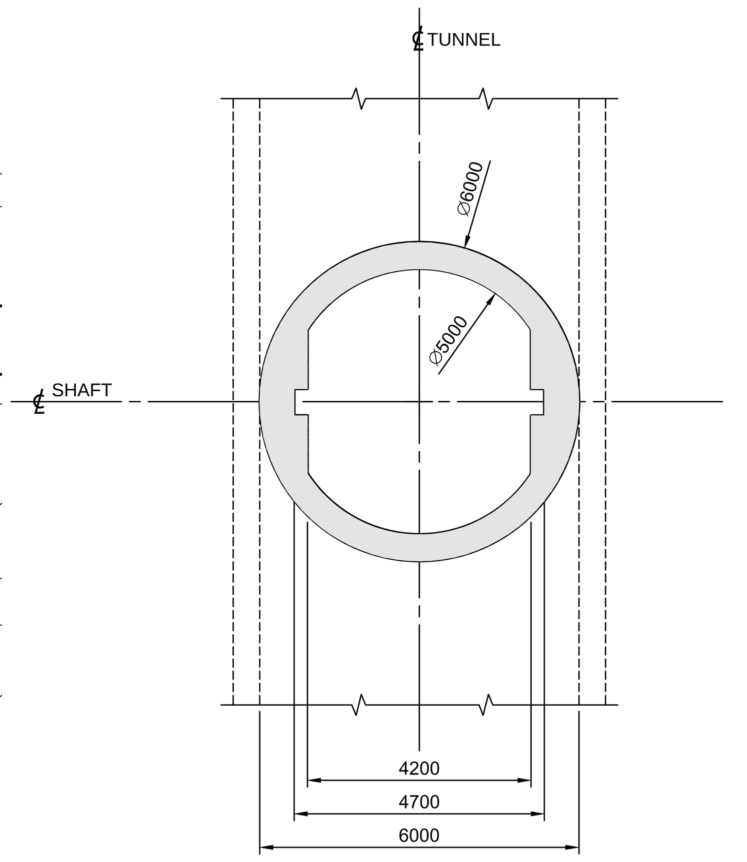
PLAN

SCALE: 1:1000



DETAIL 1 - TYP. TUNNEL SECTION

SCALE: 1:100



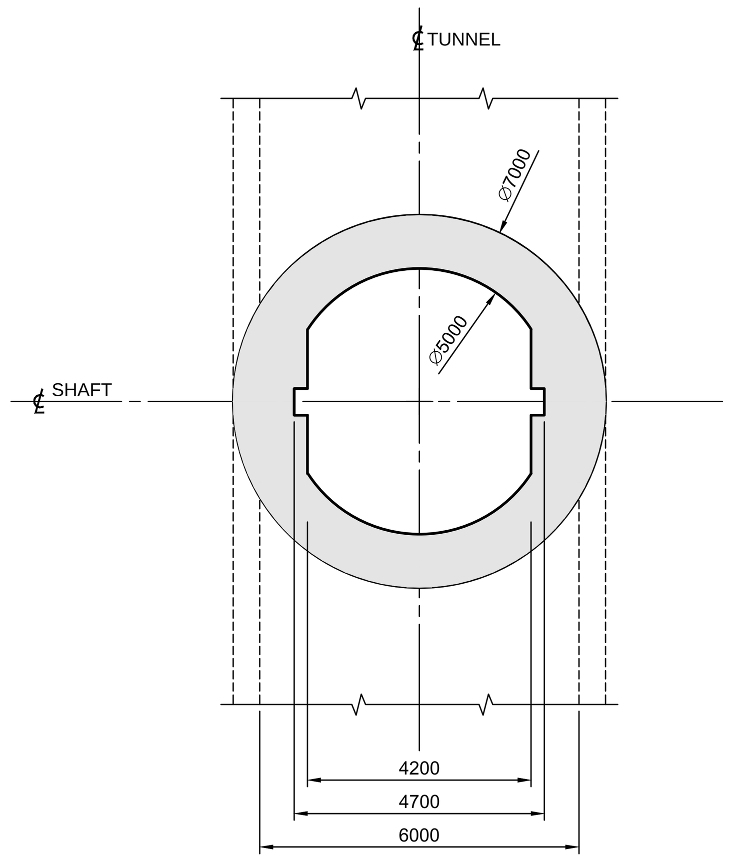
DETAIL 2 - TYP. SHAFT SECTION

4700 x 4700 ROLLER GATE IN CONCRETE LINED SHAFT

SCALE: 1:100

DETAIL 4 - TYP. SLOPE IN ROCK STILLING BASIN

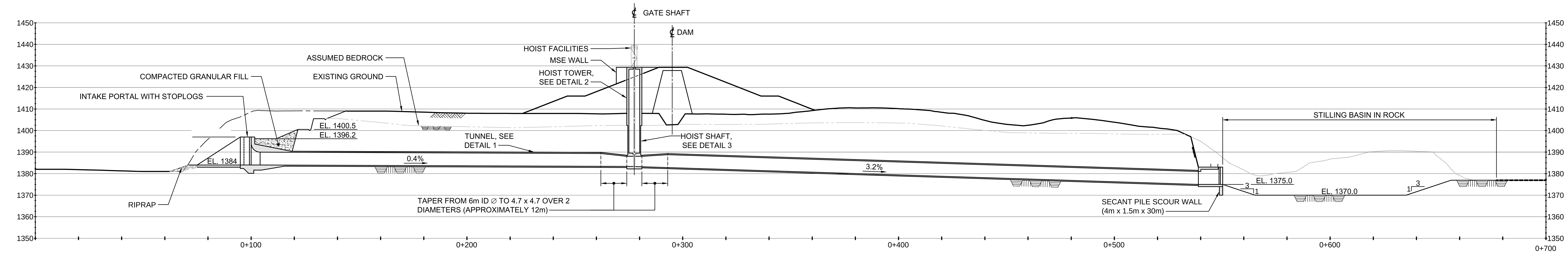
SCALE: 1:100



DETAIL 3 TYP. TOWER SECTION

4700 x 4700 ROLLER GATE IN CONCRETE HOIST TOWER AT DAM CREST ELEVATION

SCALE: 1:100



PROFILE

SCALE: 1:1000

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1:1000 0 20 40 60 80 100 METRES

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME	
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CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	-		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

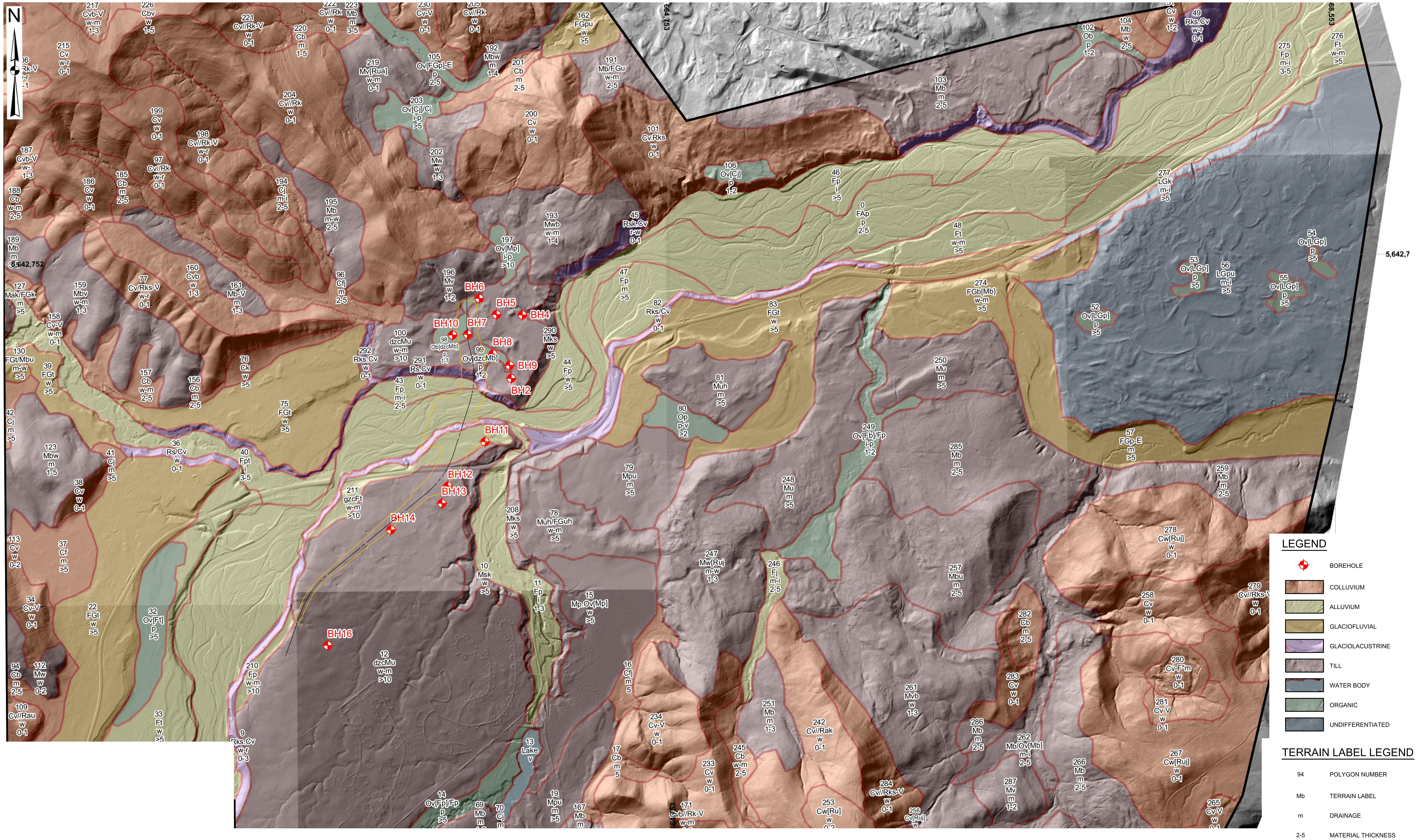
OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
DIVERSION TUNNEL
PLAN AND PROFILE

SCALE	DWG. No.	REV
AS NOTED	D05	0

SHEET SIZE: B1



0 200 400 600 800 1000 METRES

SCALE 1:10 000 [B1 SHEET]

DRAWING No.	DRAWING TITLE
-	
REFERENCE DRAWINGS	

NAME

SIGNATURE

ENG REG NUMBER

REVISION DATE

ORIGINAL DATE

REG. PROFESSIONAL

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REVISIONS

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DESIGN COORD.	-		
RESP. ENG.	P FRIZ		
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PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

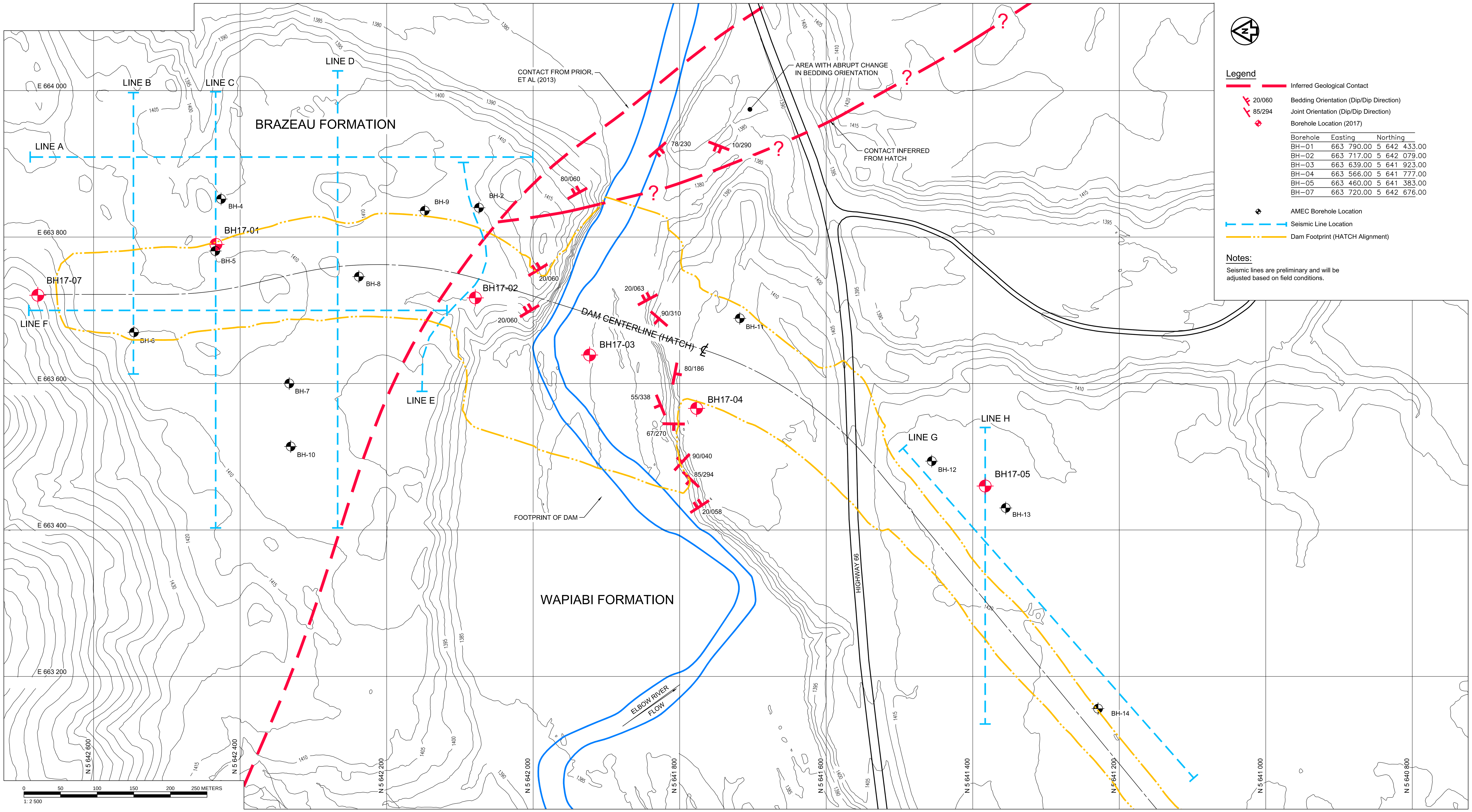
McLEAN CREEK DAM MC1

TERRAIN MAP

SCALE 1:10 000 OR AS NOTED | DWG. No. GO1 | REV 0

SHEET SIZE: B1

DWG. No. GO1



Legend

- Inferred Geological Contact
- 20/060 Bedding Orientation (Dip/Dip Direction)
- 85/294 Joint Orientation (Dip/Dip Direction)
- Borehole Location (2017)

Borehole	Easting	Northing
BH–01	663 790.00	5 642 433.00
BH–02	663 717.00	5 642 079.00
BH–03	663 639.00	5 641 923.00
BH–04	663 566.00	5 641 777.00
BH–05	663 460.00	5 641 383.00
BH–07	663 720.00	5 642 676.00

- AMEC Borehole Location
- Seismic Line Location
- Dam Footprint (HATCH Alignment)

Notes:

Seismic lines are preliminary and will be adjusted based on field conditions.

DRAWING No.	DRAWING TITLE	
REFERENCE DRAWINGS		

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OPUS STEWART WEIR

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LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1

GEOLOGICAL PLAN

SCALE AS NOTED OR AS NOTED

DWG. No. G02

REV 0

SHEET SIZE: B1

$USERNAME$ $DATE$ $FILE$ $TIME$

DWG. No. G02

DWG. No. G03

DRAWING No.	DRAWING TITLE
REFERENCE DRAWINGS	

NAME	
SIGNATURE	
ENG REG NUMBER	
REVISION DATE	
ORIGINAL DATE	
REG. PROFESSIONAL	

No.	DESCRIPTION	BY	CHK'D	DATE
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HATCH

OPUS STEWART WEIR

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DESIGN COORD.	-		
RESP. ENG.	P FRIZ		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

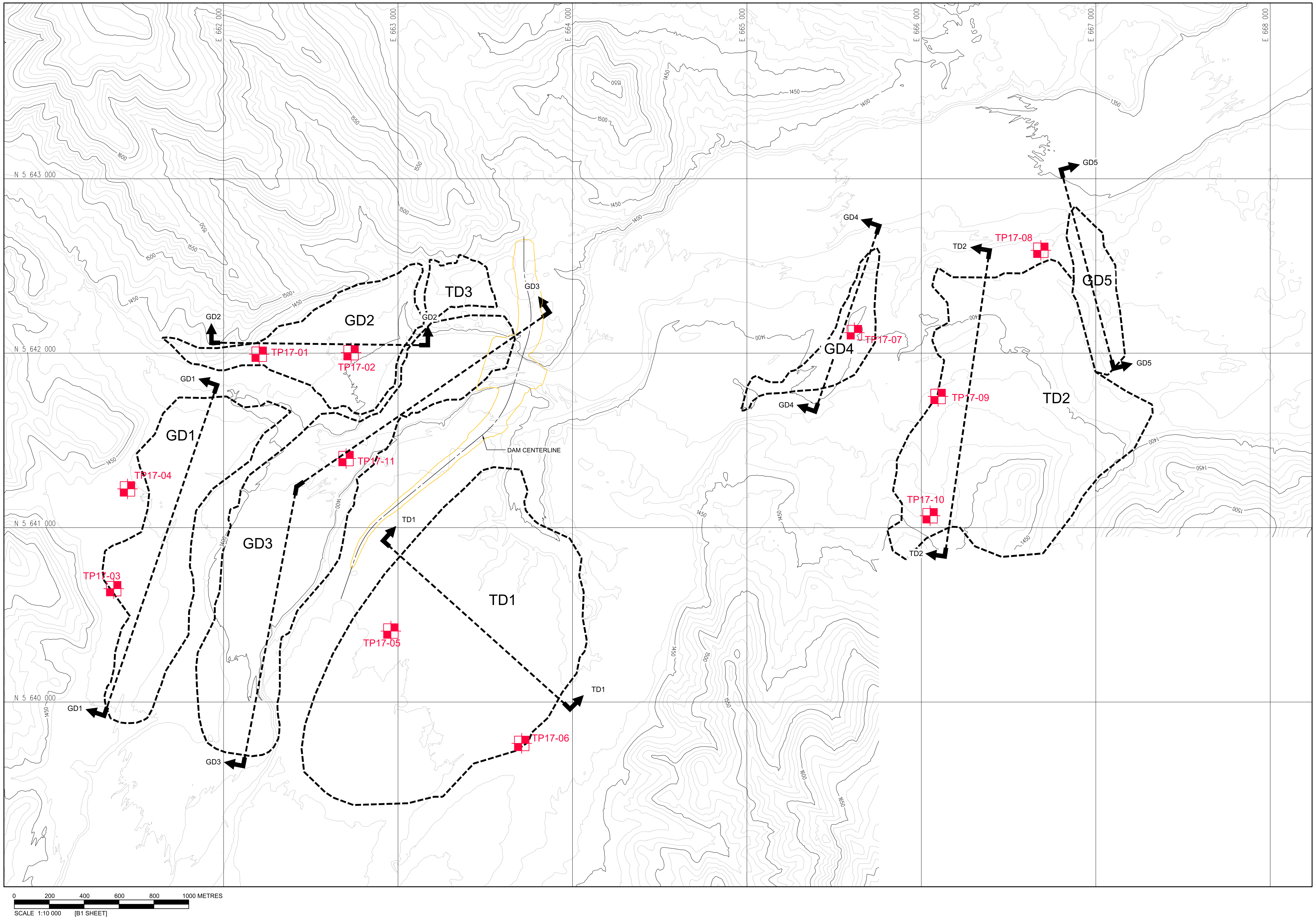
McLEAN CREEK DAM MC1

GEOLOGICAL PROFILE

SCALE AS NOTED OR AS NOTED | DWG. No. G03 | REV 0

SHEET SIZE: B1

$USERNAMES $DATES $FILES $TIMES



DIMENSIONS SHOWN IN MILLIMETRES
ELEVATIONS SHOWN IN METRES

NOTES

FOR LONGITUDINAL PROFILES OF BORROW AREAS SEE DRAWING G05

LEGEND

- - - - BORROW AREA BOUNDARY
TD TILL DEPOSITS (2 AREAS)
GD GRANULAR DEPOSITS (5 AREAS)
TP17-01 TEST PIT (HATCH, 2017)

TEST PIT COORDINATES

ID.	NORTHING	EASTING
TP17-01	5 641 994	662 204
TP17-02	5 642 003	662 731
TP17-03	5 640 650	661 370
TP17-04	5 641 222	661 449
TP17-05	5 640 407	662 959
TP17-06	5 639 763	663 709
TP17-07	5 642 118	665 616
TP17-08	5 642 590	666 685
TP17-09	5 641 752	666 096
TP17-10	5 641 068	666 050
TP17-11	5 641 396	662 702

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ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
BORROW AREAS
LOCATIONS

SCALE 1:10 000 OR AS NOTED

DWG. No. G04

REV 0

DRAWING No.	DRAWING TITLE

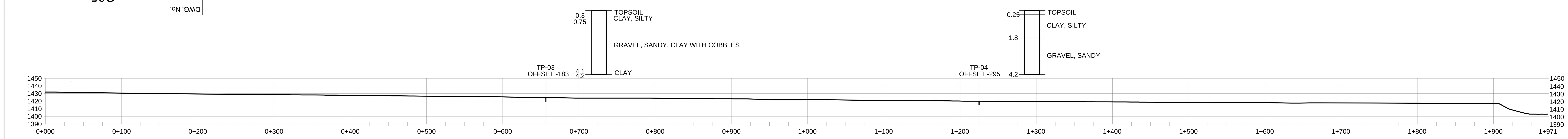
REFERENCE DRAWINGS

SHEET SIZE: B1

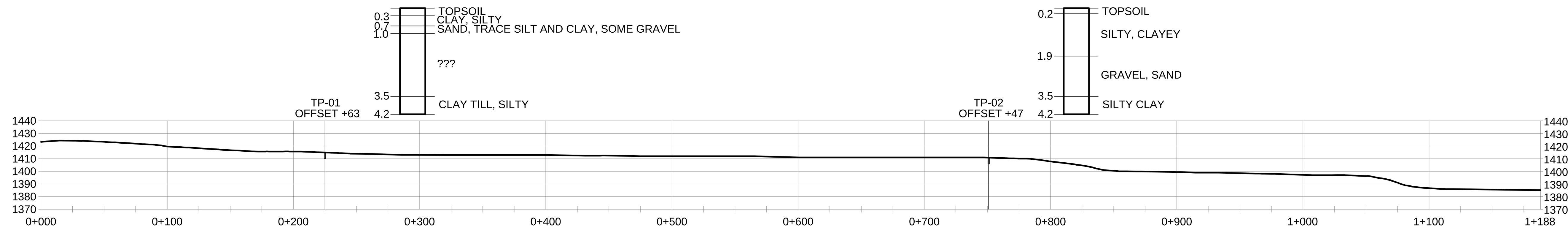
DWG. No. G04

NOTE

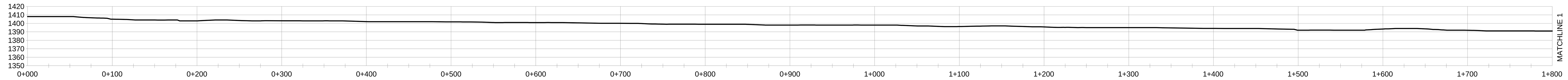
FOR LOCATION OF LONGITUDINAL PROFILES SEE DRAWING G04



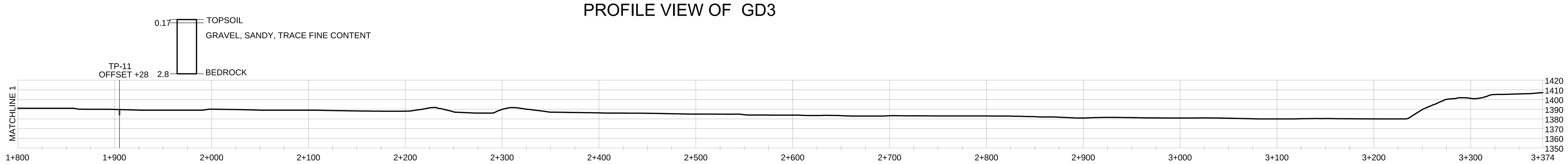
PROFILE VIEW OF GD1



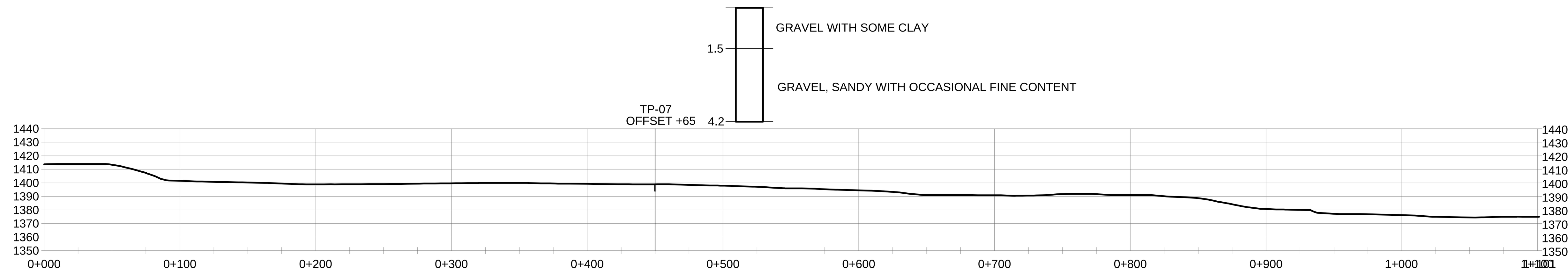
PROFILE VIEW OF GD2



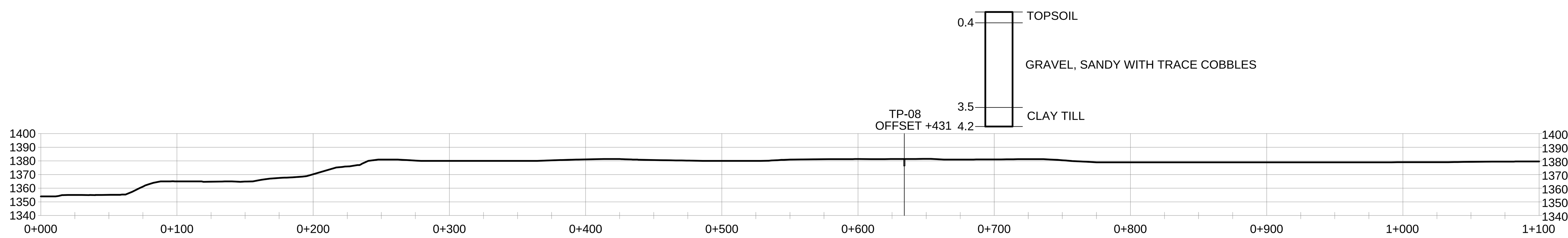
PROFILE VIEW OF GD3



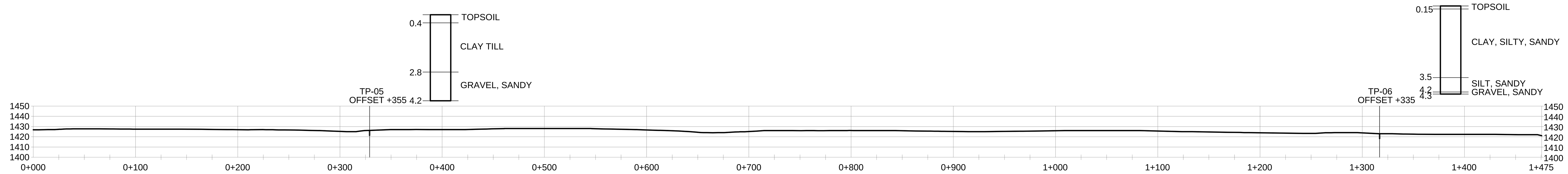
PROFILE VIEW OF GD3 (CONTINUED)



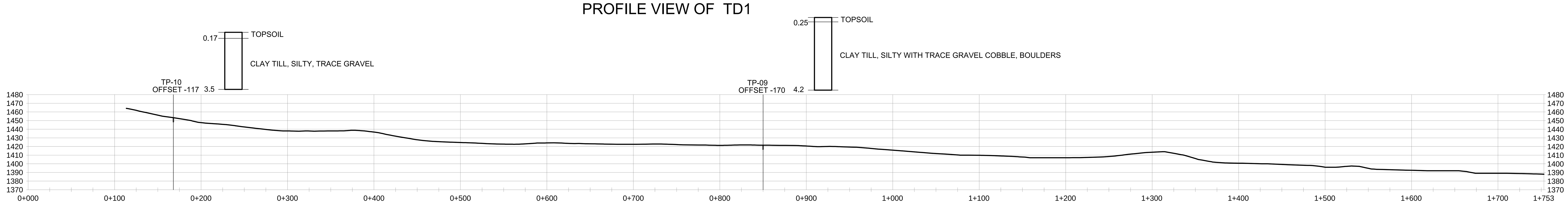
PROFILE VIEW OF GD4



PROFILE VIEW OF GD5



PROFILE VIEW OF TD1



PROFILE VIEW OF TD2

0 50 100 150 200 250 METRES

SCALE 1:2500 [B1 SHEET]

DRAWING No.	DRAWING TITLE
-	

REFERENCE DRAWINGS

NAME
SIGNATURE
ENG REG NUMBER
REVISION DATE
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REVISIONS

HATCH

ROLE	NAME	SIGNATURE	DATE
DRAFTSPERSON	D PARKES	NR	2017-02-27
DESIGNER	-	NR	-
CHECKER	P ASHAYER		
DESIGN COORD.	-		
RESP. ENG.	P FRIZ		
LEAD DISC. ENG.	S HINCHBERGER		
ENG. MANAGER	-		
PROJ. MANAGER	D PHELPS		

DRAWING APPROVAL STATUS: ISSUED FOR BASIS OF ESTIMATE

OPUS STEWART WEIR

ALBERTA TRANSPORTATION
WATER MANAGEMENT SECTION

McLEAN CREEK DAM MC1
BORROW AREAS
LONGITUDINAL PROFILES

SCALE 1:2500 OR AS NOTED | DWG. No. G05 | REV 0

SHEET SIZE: B1

$USERNAME$ $DATE$ $FILE$ $TIME$

Cost Estimate

APPENDIX A: Construction Cost Estimate Opinion

August 21, 2017

OPUS

MC1 Cost Opinion		
General		
Mobilization	$	12,000,000
Care of Water	$	3,000,000
Total	**$**	**15,000,000**
Construction		
MC1 Dam	$	188,000,000
Highway 66 Relocation	$	34,341,000
Facility Relocation	$	22,853,000
Total	**$**	**245,194,000**
Environmental Habitat		
Wetland Compensation	$	708,000
Aquatic Habitat Management Plan	$	10,000,000
Total	**$**	**10,708,000**
SUBTOTAL CONSTRUCTION	**$**	**270,902,000**
Engineering/Environment/Engagement (20%)	$	54,181,000
Contingencies (25% including Engineering)	$	81,271,000
Total	**$**	**135,452,000**
Grand Total	**$**	**406,354,000**

Notes:

1. This Construction Estimate is based on the level of project information developed in the study.
2. Unit prices are based on calculated information, historic bid data, past project experience, and engineering judgement
3. The summary information is rounded to nearest $1000s

SUMMARY ESTIMATE

MC1 DAM CONSTRUCTION			File:	S39001
			Date:	21-Aug-2017
	UNIT	ESTIMATE QUANTITIY	UNIT PRICE	TOTAL
MAIN DAM				
Site Preparation				
Clearing (other than Hwy.)	HA	41	$5,600.00	$229,600
Clearing &Timber Salvage	HA	62	$9,800.00	$607,600
Strip & Stockpile Topsoil	M3	220,000	$6.00	$1,320,000
Recontour, topsoil, seeding	M3	160,000	$6.00	$960,000
Common Excavation	M3	641,000	$6.00	$3,846,000
Rock Excavation for benches	M3	12,000	$28.00	$336,000
Common Excav. Reroute McLean Crk.	M3	80,000	$4.00	$320,000
Common Excav. Reroute SW creek	M3	3,500	$18.00	$63,000
Fills				
Zone 1A- Impervious	M3	732,000	$10.00	$7,320,000
Zone 2A- Unclassified Fill	M3	1,035,000	$10.00	$10,350,000
Zone 3A- Fine Filter	M3	448,000	$20.00	$8,960,000
Zone 3B- Drainage Layer	M3	338,000	$20.00	$6,760,000
Zone 4D - Pit Run Granular	M3	951,000	$13.00	$12,363,000
Zone 5C- Riprap Bedding/Cobbles	M3	60,000	$65.00	$3,900,000
Zone 6- Rip Rap	M3	30,500	$200.00	$6,100,000
Granular Base Course (Crest)	T	4,600	$45.00	$207,000
Gravel Surfacing (Crest)	T	1,500	$45.00	$67,500
Miscellaneous				
Topsoil & Seeding- DS Slope	M2	120,000	$6.00	$720,000
Cable Barrier	M	4,400	$300.00	$1,320,000
Cable Barrier End Terminals	EA	4	$7,500.00	$30,000
Security Fence at Turnaround	M	500	$280.00	$140,000
Instrumentation	LS	1	$700,000.00	$700,000
Safety Boom Across River	LS	1	$32,000.00	$32,000
Drilling and Grouting				
75mm x 10m Grout Holes- C'dam- OB	M	1,500	$210.00	$315,000
75mm x 20m Grout Holes- C'dam	M	3,000	$325.00	$975,000
75mm x 20m Grout Holes- Dam	M	6,000	$280.00	$1,680,000
Packers	EA	1,600	$140.00	$224,000
Washing & Pressure Testing	HR	320	$140.00	$44,800
Cement Take (No Sand)	TO	140	$420.00	$58,800
Cut-Off Walls				
Left Abut.- Plastic Concrete- PFC	M2	12,600	$1,500.00	$18,900,000
Left Abut. Bentonite COW Cap	M3	9,250	$100.00	$925,000
Right Abut.- Soil/Slurry Wall - PFC	M2	11,000	$600.00	$6,600,000
Right Abut. Bentonite COW Cap	M3	23,250	$100.00	$2,325,000
SUB-TOTAL, MAIN DAM				**$98,699,300**

SUMMARY ESTIMATE

MC1 DAM CONSTRUCTION			File:	S39001
			Date:	21-Aug-2017
	UNIT	ESTIMATE QUANTITIY	UNIT PRICE	TOTAL
DIVERSION TUNNELS/LOW LEVEL OUTLET				
Tunnels				
Excavation- 7m x 7m nominal	M3	35,000	$145.00	$5,075,000
Rock bolts- 35mm x 3m- Epoxy Grouted	EA	2,800	$140.00	$392,000
Wire Mesh-	M2	9,300	$28.00	$260,400
Shotcrete- 50mm thick	M2	15,200	$41.00	$623,200
32 MPa Concrete Lining	M3	11,100	$700.00	$7,770,000
Contact grouting	M	840	$100.00	$84,000
Inlet & Outlet Structures				
Common Excavation	M3	80,000	$6.00	$480,000
Rock Excavation	M3	69,200	$28.00	$1,937,600
Rock Bolts & Dowels	LS	1	$350,000.00	$350,000
32 MPa Concrete	M3	12,600	$730.00	$9,198,000
Re-Bar- Supply, cut, bend, place	TO	630	$3,500.00	$2,205,000
Stilling Basin Excavation				
Common Excav.- River Re-alignment	M3	80,000	$6.00	$480,000
Rock Excav. for Stilling Basin	M3	44,000	$28.00	$1,232,000
Zone 5C- Riprap Bedding/Cobbles	M3	1,000	$65.00	$65,000
Rip Rap DS of Stilling Basin	M3	1,600	$200.00	$320,000
Gate Shafts				
Rock Excavation	M3	1,300	$28.00	$36,400
Rock Bolts- 35mm x 3m, Epoxy Grouted	EA	300	$280.00	$84,000
Wire Mesh- 50x50mm	M2	660	$28.00	$18,480
Shotcrete- 50mm Thick	M2	660	$35.00	$23,100
Gate Shaft Retaining wall/Access pad	M2	875	$1,150.00	$1,006,250
32 MPa Concrete	M3	1,700	$845.00	$1,436,500
Re-Bar- Supply, cut, bend, place	TO	120	$3,500.00	$420,000
Gates, Guides & Hoists				
Gates Supply	LS	1	$700,000.00	$700,000
Stop Logs & Lifting Beam- Supply	LS	1	$420,000.00	$420,000
Gate Hoist & Structure- Supply	LS	1	$910,000.00	$910,000
Trash Racks- Supply	LS	1	$210,000.00	$210,000
Install Guides & Gates	LS	1	$700,000.00	$700,000
S & I Gen. Set for hoist		1	$140,000.00	$140,000
SUB-TOTAL, DIVERSION TUNNELS/LOW LEVEL OUTLET				**$36,576,930**
FISH PASSAGE TUNNEL				
Tunnel				
Excavation and Temporary Support - 3. 6 m by 3.8 m nominal	M	260	$4,850.00	$1,261,000
Tunnel Concrete Lining & Contact Grouting	M	260	$5,125.00	$1,332,500
Inlet & Outlet Excavation				
Common Excavation	M3	95,372	$6.00	$572,232
Rock Excavation - upstream only	M3	13,160	$28.00	$368,480
Engineering Fill - US & DS	M3	7,148	$10.00	$71,480

OPUS

SUMMARY ESTIMATE

MC1 DAM CONSTRUCTION			File:	S39001
			Date:	21-Aug-2017
	UNIT	ESTIMATE QUANTITIY	UNIT PRICE	TOTAL
Geotextile - US & DS	M2	7,148	$9.00	$64,332
Place 50 cm Cobbles for Natural River	M3	3,574	$40.00	$142,960
Recontour, topsoil, seeding	M3	3,808	$6.00	$22,848
Earth Fill - DS only	M3	11,406	$6.00	$68,436
Rip Rap - DS only	M3	413	$150.00	$61,950
Gate Structure	LS	1	$850,000.00	$850,000
Gate, Sill, Guides, Hoist, Shelter	LS	1	$500,000.00	$500,000
Hoist Power Supply	LS	1	$20,000.00	$20,000
SUB-TOTAL				**$5,336,218**

SUMMARY ESTIMATE

MC1 DAM CONSTRUCTION			File:	S39001
			Date:	21-Aug-2017
	UNIT	ESTIMATE QUANTITIY	UNIT PRICE	TOTAL
SERVICE SPILLWAY				
Earthworks/Cut-off Walls				
Common Excavation	M3	84,000	$6.00	$504,000
Crushed Stone- Drainage Piping	M3	7,200	$14.00	$100,800
Granular Fill Under Slab	M3	35,000	$14.00	$490,000
Clean Backfill (Outside Walls)	M3	8,700	$14.00	$121,800
Drainage Headers-400mm PVC Pipe	M	760	$110.00	$83,600
Drainage Piping-150mm Perforated PVC	M	720	$90.00	$64,800
Spillway - Sheet Pile -PFC	M2	1,510	$1,100.00	$1,661,000
Spillway - Secant Piles -PFC	M2	1,500	$2,050.00	$3,075,000
Concrete				
32 MPa Concrete- Ftgs, Walls, Ogee, Flip	M3	41,000	$730.00	$29,930,000
Re-Bar- Supply, Cut, Bend, Place	T	2,000	$3,500.00	$7,000,000
Service Spillway Mudslab	M3	3,000	$730.00	$2,190,000
Lean Concrete Slab- Outside Walls	M3	1,200	$560.00	$672,000
SUB-TOTAL SERVICE SPILLWAY				**$45,893,000**
AUXILIARY SPILLWAY				
Common Excavation	M3	158,000	$6.00	$948,000
Strip & Stockpile Topsoil	M3	30,000	$6.00	$180,000
Recontour, topsoil, seeding	M3	60,000	$6.00	$360,000
RipRap US of Sheet pile Wall	M3	0	$280.00	$0
Fine Filter material- 500mm thick	M3	0	$14.00	$0
Drainage Layer- 500mm thick	M3	0	$14.00	$0
RCC- 600mm thick	M3	0	$420.00	$0
Articulated Concrete Mats	M2	0	$250.00	$0
Auxiliary Spillway - Sheet Pile - PFC	M2		$850.00	$0
SUB-TOTAL, AUXILIARY SPILLWAY				**$1,488,000.00**
TOTAL: DAM & HYDRAULIC STRUCTURES			**Total**	$187,993,448
			Round Total	**$188,000,000**

SUMMARY ESTIMATE

HWY 66 RELOCATION ROADWORK			File:	S-39001
			Date:	29-Apr-2017
		ESTIMATE	UNIT	
	UNIT	QUANTITY	PRICE	TOTAL
MOBILIZATION				
Site Establishment	lump sum	1	$1,030,590.00	$1,030,590
ENVIRONMENTAL				
Erosion Control Barrier (Silt Fence)	m	5,000	$16.00	$80,000
EARTHWORKS				
Clearing	ha	35	$4,000.00	$140,000
Clearing and Timber Salvage	ha	35	$7,000.00	$245,000
Channel Excavation	m^3	600	$10.00	$6,000
Common Excavation	m^3	1,281,600	$4.00	$5,126,400
Topsoil Placement	m^2	251,900	$0.90	$226,710
Drill Seeding	ha	25	$900.00	$22,500
DRAINAGE				
Culverts - Supply and Install (900 mm dia. C.S.P.)	m	260	$650.00	$169,000
Granular Backfill - Culverts	t	550	$30.00	$16,500
SURFACING				
Supply of Aggregates - No option	t	135,900	$4.50	$611,550
Preparing Subgrade Surface	m^2	166,200	$1.60	$265,920
Granular Base Course - Des. 2 Class. 25	t	105,200	$23.00	$2,419,600
Asphalt Concrete Pavement - EPS Mix Type M1 (PG 46-34)	t	30,100	$68.00	$2,046,800
Gravel Surfacing, Des. 4, Class 20	t	1,430	$30.00	$42,900
Cutting of Pavement	m	30	$30.00	$900
TRAFFIC SERVICES				
<u>Road Signs & Supports</u>				
Supply and Install Signs (inc Posts	LS	1	$100,000.00	$100,000
<u>Fences</u>				
New Fence - Supply and Install - Class B	km	20	$10,000.00	$200,000
<u>Barriers</u>				
Strong Post W-Beam Guardrail - Supply and Install	m	400	$200.00	$80,000
Impact Attenuator (NCHRP 350/MASH 2009 TL-3) - Supply and Install	ea	4	$3,500.00	$14,000
OTHER CONSTRUCTION COSTS				
Demolish Existing Hwy Bridge	lump sum	1	$500,000.00	$500,000
Remove BF #78714 and BF#78664 (60m)	lump sum	1	$100,000.00	$100,000
Obliterate Existing Highway Pavement 1.4 km	lump sum	1	$20,000.00	$20,000
Road Construction Costs:			**Total**	**$13,464,370**
			Round Total	**$13,465,000**

SUMMARY ESTIMATE

HWY 66 BRIDGE WORK			File:	S-39001
			Date:	29-Apr-2017
Description	UNIT	ESTIMATE QUANTITIY	UNIT RATE	TOTAL
MOBILIZATION				
Site Establishment	lump sum	1	$2,083,041	$2,083,041
SUBSTRUCTURE				
Concrete - Pier Caps	Cubic Metre	220	$1,000	$220,000
Concrete - Pier Columns	Cubic Metre	540	$1,000	$540,000
Concrete - Pier Pile Caps	Cubic Metre	1300	$1,000	$1,300,000
Concrete - Pier Drilled Shafts	Cubic Metre	1090	$1,000	$1,090,000
Concrete - Abutment Walls	Cubic Metre	220	$1,000	$220,000
Concrete - Abutment Drilled Shafts	Cubic Metre	190	$1,000	$190,000
Rebar - Piers	Kilograms	630000	$5.00	$3,150,000
Rebar - Abutments	Kilograms	82000	$5.00	$410,000
SUPERSTRUCTURE - COMPOSITE STEEL GIRDERS				
Steel Superstructure - Fabrication, Supply, Erection	Tonnes	1400	$6,000.00	$8,400,000
Concrete - Deck, Parapets and Approach Slabs	Cubic Metre	850	$1,000.00	$850,000
Rebar - Deck, Parapets and Approach Slabs	Kilograms	170000	$5.00	$850,000
GRADING & PAVING				
Site Preparation	lump sum	1	$300,000.00	$300,000
Earthworks at West Abutment	Cubic Metre	1300	$150.00	$195,000
Supply and Placement of Asphalt Pavement	Tonnes	620	$150.00	$93,000
MISCELLANEOUS				
Expansion Joints	L.S.	2	$20,000.00	$40,000
Pot Bearings for Main Girders	L.S.	16	$10,000.00	$160,000
Rip Rap	Cubic Metre	1900	$150.00	$285,000
McLean Creek Bridge File	L.S.	1	$500,000.00	$500,000
Bridge Construction Costs:			**Total**	$20,876,041
			Round Total	**$20,876,000**

SUMMARY ESTIMATE

Facility Relocation			**File:**	S-39001
			Date:	11-Apr-2017
Task	Unit	Quantity	Item Rate	Total
DEMOLITION				
EVRS Demolition / Salvage				
Water/Waste Water				
Manhole	ea	17	$ 5,140.00	$ 87,380
Lift Station Salvage	ea	3	$ 2,000.00	$ 6,000
Water/Waste Water Line	m	2500	$ 30.00	$ 75,000
Water Treatment Plant Salvage	LS	1	$ 75,000.00	$ 75,000
Sewage Treatment Plant Salvage	LS	1	$ 50,000.00	$ 50,000
Buildings				
Split Level Houses	EA	5	$ 7,950.00	$ 39,750
Res Garages	EA	5	$ 3,360.00	$ 16,800
Bungalow	EA	1	$ 7,950.00	$ 7,950
Campsite Warden Workshop	EA	1	$ 13,600.00	$ 13,600
Campsite Warden Cabin	EA	1	$ 7,600.00	$ 7,600
Firefighter Cabins	EA	7	$ 12,130.00	$ 84,910
Firefighter Dining Hall	EA	1	$ 31,130.00	$ 31,130
Historical Ranger Building	EA	1	$ 113,840.00	$ 113,840
Ranger Building	EA	1	$ 152,140.00	$ 152,140
Cold Storage	EA	1	$ 76,160.00	$ 76,160
Christian Ventures Building	EA	1	$ 39,290.00	$ 39,290
Dumping	T KM	458400	$ 1.65	$ 756,360
Misc				
Fueling Station Salvage	LS	1	$ 25,000.00	$ 25,000
Helipad Demolition	LS	1	$ 25,000.00	$ 25,000
Weather Station Salvage	LS	1	$ 10,000.00	$ 10,000
Site Restoration	LS	1	$ 1,000,000.00	$ 1,000,000
Various Line Removal	LS	1	$ 120,000.00	$ 120,000
Environmental Remediation	LS	1	$ 3,620,371.00	$ 3,620,371
McLean Creek Campground Demolition				
Camp Site Demolition	EA	1	$ 30,000.00	$ 30,000
Store Relocation	EA	1	$ 40,000.00	$ 40,000

SUMMARY ESTIMATE

Facility Relocation			**File:**	S-39001
			Date:	11-Apr-2017
Task	Unit	Quantity	Item Rate	Total
NEW CONSTRUCTION				
Gooseberry Station				
Clearing & Grubbing	Ha	2.9	$ 25,000.00	$ 72,500
Stripping	m3	30000	$ 4.00	$ 120,000
Earthworks	m3	200000	$ 6.00	$ 1,200,000
Gravel Road Surfacing	m2	10000	$ 5.00	$ 50,000
Helipad				
Excavation	m3	290	$ 10.00	$ 2,900
GBC	t	280	$ 45.00	$ 12,600
Concrete	m3	68	$ 1,200.00	$ 81,600
Thermal Plast Marking	LS	1	$ 8,000.00	$ 8,000
Sewage Treatment				
Field	LS	1	$ 340,000.00	$ 340,000
Treatment Facility	LS	1	$ 380,000.00	$ 380,000
Services	ea	19	$ 3,500.00	$ 66,500
Sewer Transmission Lines	lm	1125	$ 160.00	$ 180,000
Manholes	vm	50	$ 1,300.00	$ 65,000
Lift Stations	ea	3	$ 150,000.00	$ 450,000
Forcemain Offsite	lm	1800	$ 165.00	$ 297,000
Water				
Well	LS	1	$ 100,000.00	$ 100,000
Treatment Facility/Pumphouse	LS	1	$ 500,000.00	$ 500,000
Services	ea	20	$ 2,500.00	$ 50,000
Water Transmission Lines	lm	1200	$ 150.00	$ 180,000
Instrumentation	LS	1	$ 475,000.00	$ 475,000
Fueling Station Relocation				
Pumps	LS	1	$ 125,000.00	$ 125,000
Tanks	LS	1	$ 35,000.00	$ 35,000
Concrete Pad	LS	1	$ 25,000.00	$ 25,000
Buildings				
Housing	EA	6	$ 230,000.00	$ 1,380,000
Firefighter Cabins	EA	9	$ 353,000.00	$ 3,177,000
Cost per SF	ft2	2615	$ 135.00	
Firefighter Dining Hall	LS	1	$ 588,000.00	$ 588,000
Cost per SF	ft2	3920	$ 150.00	
Ranger Building	LS	1	$ 1,688,000.00	$ 1,688,000
Cost per SF	ft2	10890	$ 155.00	
Christian Ventures Building	LS	1	$ 340,000.00	$ 340,000
Cost per SF	ft2	2723	$ 125.00	
Cold Storage	LS	1	$ 410,000.00	$ 410,000
Cost per SF	ft2	5445	$ 76.00	
Campsite Warden Office and Shop	LS	1	$ 505,000.00	$ 505,000
Cost per SF	ft2	3267	$ 155.00	
Recreation				

SUMMARY ESTIMATE

Facility Relocation			File:	S-39001
			Date:	11-Apr-2017
Task	Unit	Quantity	Item Rate	Total
Trails and Track	LS	1	$ 125,000.00	$ 125,000
Basket Ball Court	LS	1	$ 100,000.00	$ 100,000
Fencing				
Gates	EA	3	$ 8,500.00	$ 25,500
Fence	m	800	$ 100.00	$ 80,000
Com, Power and Propane				
Underground Power	LS	1	$ 240,000.00	$ 240,000
Underground Com	LS	1	$ 160,000.00	$ 160,000
Propane Line	LS	1	$ 420,000.00	$ 420,000
Tank Relocation	LS	1	$ 25,000.00	$ 25,000
Generator	LS	1	$ 135,000.00	$ 135,000
McLean Creek Campground				
Store Relocation	LS	1	$ 625,000.00	$ 625,000
New Sites	EA	16	$ 20,000.00	$ 320,000
Clearing	Ha	1.8	$ 25,000.00	$ 45,000
Stripping	m3	3000	$ 4.00	$ 12,000
Earthworks	m3	20000	$ 6.00	$ 120,000
Paving and Base	m2	13500	$ 75.00	$ 1,012,500
			Total	$ 22,852,381
			Round Total	**$ 22,853,000**

AACE International Recommended Practice No. 18R-97

COST ESTIMATE CLASSIFICATION SYSTEM – AS APPLIED IN ENGINEERING, PROCUREMENT, AND CONSTRUCTION FOR THE PROCESS INDUSTRIES

TCM Framework: 7.3 – Cost Estimating and Budgeting

Rev. March 1, 2016

Note: As AACE International Recommended Practices evolve over time, please refer to www.aacei.org for the latest revisions.

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COST ESTIMATE CLASSIFICATION SYSTEM – AS APPLIED IN ENGINEERING, PROCUREMENT, AND CONSTRUCTION FOR THE PROCESS INDUSTRIES

TCM Framework: 7.3 – Cost Estimating and Budgeting

March 1, 2016

PURPOSE

As a recommended practice of AACE International, the *Cost Estimate Classification System* provides guidelines for applying the general principles of estimate classification to project cost estimates (i.e., cost estimates that are used to evaluate, approve, and/or fund projects). The *Cost Estimate Classification System* maps the phases and stages of project cost estimating together with a generic project scope definition maturity and quality matrix, which can be applied across a wide variety of process industries.

This addendum to the generic recommended practice (17R-97) provides guidelines for applying the principles of estimate classification specifically to project estimates for engineering, procurement, and construction (EPC) work for the process industries. This addendum supplements the generic recommended practice by providing:

- A section that further defines classification concepts as they apply to the process industries.
- A chart that maps the extent and maturity of estimate input information (project definition deliverables) against the class of estimate.

As with the generic recommended practice, the intent of this addendum is to improve communications among all of the stakeholders involved with preparing, evaluating, and using project cost estimates specifically for the process industries.

The overall purpose of this recommended practice is to provide the process industry with a project definition deliverable maturity matrix that is not provided in 17R-97. It also provides an approximate representation of the relationship of specific design input data and design deliverable maturity to the estimate accuracy and methodology used to produce the cost estimate. The estimate accuracy range is driven by many other variables and risks, so the maturity and quality of the scope definition available at the time of the estimate is not the sole determinate of accuracy; risk analysis is required for that purpose.

This document is intended to provide a guideline, not a standard. It is understood that each enterprise may have its own project and estimating processes and terminology, and may classify estimates in particular ways. This guideline provides a generic and generally acceptable classification system for process industries that can be used as a basis to compare against. This addendum should allow each user to better assess, define, and communicate their own processes and standards in the light of generally-accepted cost engineering practice.

INTRODUCTION

For the purposes of this addendum, the term “process industries” is assumed to include firms involved with the manufacturing and production of chemicals, petrochemicals, and hydrocarbon processing. The common thread among these industries (for the purpose of estimate classification) is their reliance on process flow diagrams (PFDs) and piping and instrument diagrams (P&IDs) as primary scope defining documents. These documents are key deliverables in determining the degree of project definition, and thus the extent and maturity of estimate input information.

Estimates for process facilities center on mechanical and chemical process equipment, and they have significant amounts of piping, instrumentation, and process controls involved. As such, this addendum may apply to portions of other industries, such as pharmaceutical, utility, water treatment, metallurgical, converting, and similar industries.

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This addendum specifically does not address cost estimate classification in non-process industries such as commercial building construction, environmental remediation, transportation infrastructure, hydropower, "dry" processes such as assembly and manufacturing, "soft asset" production such as software development, and similar industries. It also does not specifically address estimates for the exploration, production, or transportation of mining or hydrocarbon materials, although it may apply to some of the intermediate processing steps in these systems.

The cost estimates covered by this addendum are for engineering, procurement, and construction (EPC) work only. It does not cover estimates for the products manufactured by the process facilities, or for research and development work in support of the process industries. This guideline does not cover the significant building construction that may be a part of process plants.

This guideline reflects generally-accepted cost engineering practices. This RP was based upon the practices of a wide range of companies in the process industries from around the world, as well as published references and standards. Company and public standards were solicited and reviewed, and the practices were found to have significant commonalities. These classifications are also supported by empirical process industry research of systemic risks and their correlation with cost growth and schedule slip[8].

COST ESTIMATE CLASSIFICATION MATRIX FOR THE PROCESS INDUSTRIES

A purpose of cost estimate classification is to align the estimating process with project stage-gate scope development and decision making processes.

Table 1 provides a summary of the characteristics of the five estimate classes. The maturity level of project definition is the sole determining (i.e., primary) characteristic of class. In Table 1, the maturity is roughly indicated by a percentage of complete definition; however, it is the maturity of the defining deliverables that is the determinant, not the percent. The specific deliverables, and their maturity or status are provided in Table 3. The other characteristics are secondary and are generally correlated with the maturity level of project definition deliverables, as discussed in the generic RP [2]. The post sanction classes (Class 1 and 2) are only indirectly covered where new funding is indicated. Again, the characteristics are typical and may vary depending on the circumstances.

March 1, 2016

	Primary Characteristic	Secondary Characteristic		
ESTIMATE CLASS	**MATURITY LEVEL OF PROJECT DEFINITION DELIVERABLES** Expressed as % of complete definition	**END USAGE** Typical purpose of estimate	**METHODOLOGY** Typical estimating method	**EXPECTED ACCURACY RANGE** Typical variation in low and high ranges
Class 5	0% to 2%	Concept screening	Capacity factored, parametric models, judgment, or analogy	L: -20% to -50% H: +30% to +100%
Class 4	1% to 15%	Study or feasibility	Equipment factored or parametric models	L: -15% to -30% H: +20% to +50%
Class 3	10% to 40%	Budget authorization or control	Semi-detailed unit costs with assembly level line items	L: -10% to -20% H: +10% to +30%
Class 2	30% to 75%	Control or bid/tender	Detailed unit cost with forced detailed take-off	L: -5% to -15% H: +5% to +20%
Class 1	65% to 100%	Check estimate or bid/tender	Detailed unit cost with detailed take-off	L: -3% to -10% H: +3% to +15%

Table 1 – Cost Estimate Classification Matrix for Process Industries

This matrix and guideline outline an estimate classification system that is specific to the process industries. Refer to the generic estimate classification RP[1] for a general matrix that is non-industry specific, or to other addendums for guidelines that will provide more detailed information for application in other specific industries. These will provide additional information, particularly the project definition deliverable maturity matrix which determines the class in those particular industries.

Table 1 illustrates typical ranges of accuracy ranges that are associated with the process industries. The +/- value represents typical percentage variation of actual costs from the cost estimate after application of contingency (typically to achieve a 50% probability of project overrun versus underrun) for given scope. Depending on the technical and project deliverables (and other variables) and risks associated with each estimate, the accuracy range for any particular estimate is expected to fall into the ranges identified (although extreme risks can lead to wider ranges).

In addition to the degree of project definition, estimate accuracy is also driven by other systemic risks such as:

- Level of non-familiar technology in the project.
- Complexity of the project.
- Quality of reference cost estimating data.
- Quality of assumptions used in preparing the estimate.
- Experience and skill level of the estimator.
- Estimating techniques employed.
- Time and level of effort budgeted to prepare the estimate.
- Unique/remote nature of project locations and the lack of reference data for these locations.
- The accuracy of the composition of the input and output process streams.

Systemic risks such as these are often the primary driver of accuracy, especially during the early stages of project definition. As project definition progresses, project-specific risks (e.g. risk events) become more prevalent and also drive the accuracy range[3]. Another concern in estimates is potential pressure for a predetermined value that may result in a biased estimate. The goal should be to always have an unbiased and objective estimate. The stated estimate ranges are dependent on this premise and a realistic view of the project.

Failure to appropriately address systemic risks (e.g. technical complexity) during risk analysis impacts the resulting probability distribution of the estimate costs, and therefore the interpretation of estimate accuracy.

Another way to look at the variability associated with estimate accuracy ranges is shown in Figure 1. Depending upon the technical complexity of the project, the availability of appropriate cost reference information, the degree of project definition, and the inclusion of appropriate contingency determination, a typical Class 5 estimate for a process industry project may have an accuracy range as broad as -50% to +100%, or as narrow as -20% to +30%.

Figure 1 also illustrates that the estimating accuracy ranges overlap the estimate classes. There are cases where a Class 5 estimate for a particular project may be as accurate as a Class 3 estimate for a different project. For example, similar accuracy ranges may occur if the Class 5 estimate of one project that is based on a repeat project with good cost history and data and, whereas the Class 3 estimate for another is for a project involving new technology. It is for this reason that Table 1 provides ranges of accuracy range values. This allows application of the specific circumstances inherent in a project, and an industry sector, to provide realistic estimate class accuracy range percentages. While a target range may be expected of a particular estimate, the accuracy range is determined through risk analysis of the specific project and is never pre-determined. AACE has recommended practices that address contingency determination and risk analysis methods.

If contingency has been addressed appropriately, approximately 80% of projects should fall within the ranges shown in Figure 1. However, this does not preclude a specific actual project result from falling inside or outside of the bands shown in Figure 1 indicating the expected accuracy ranges.

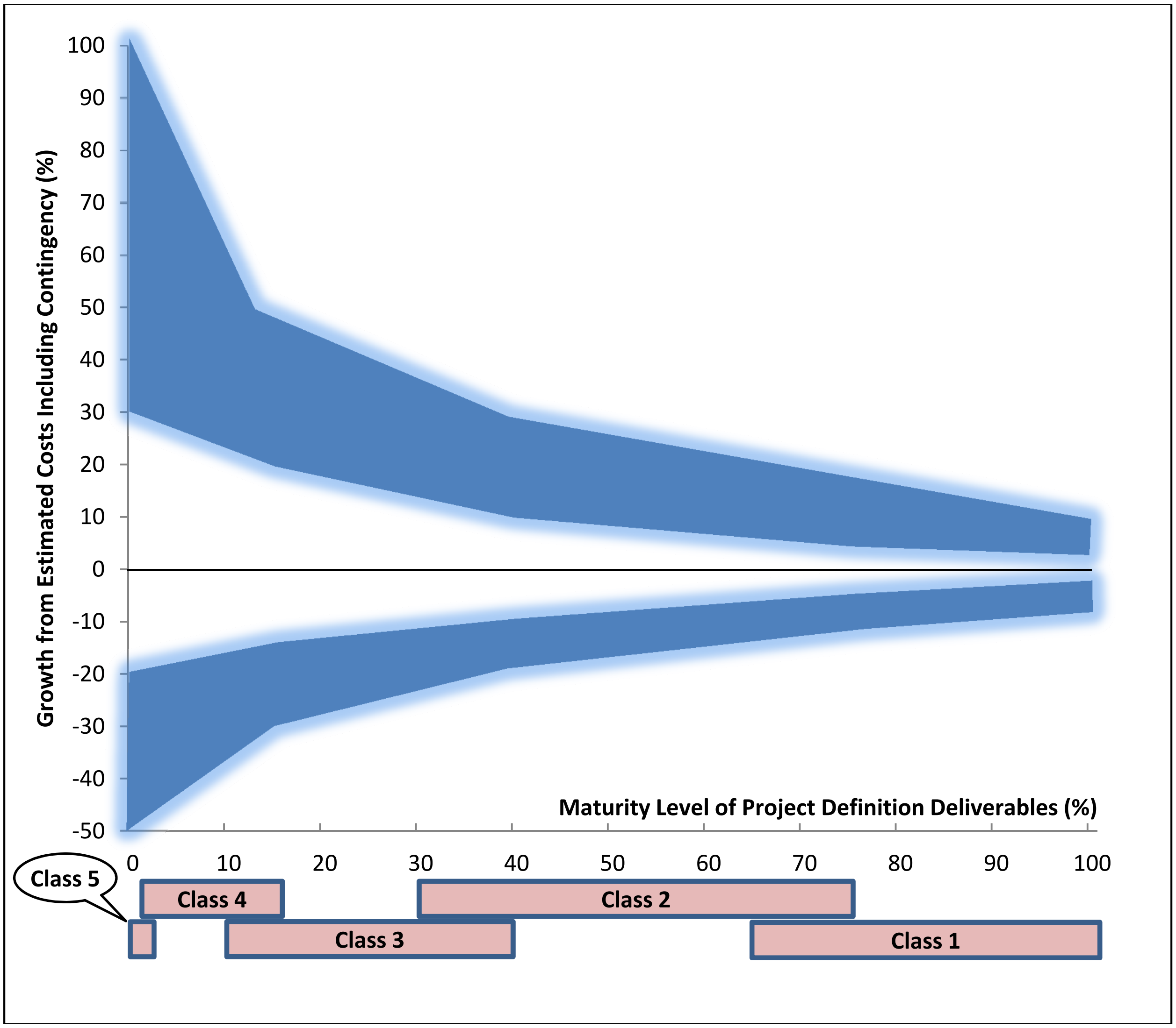


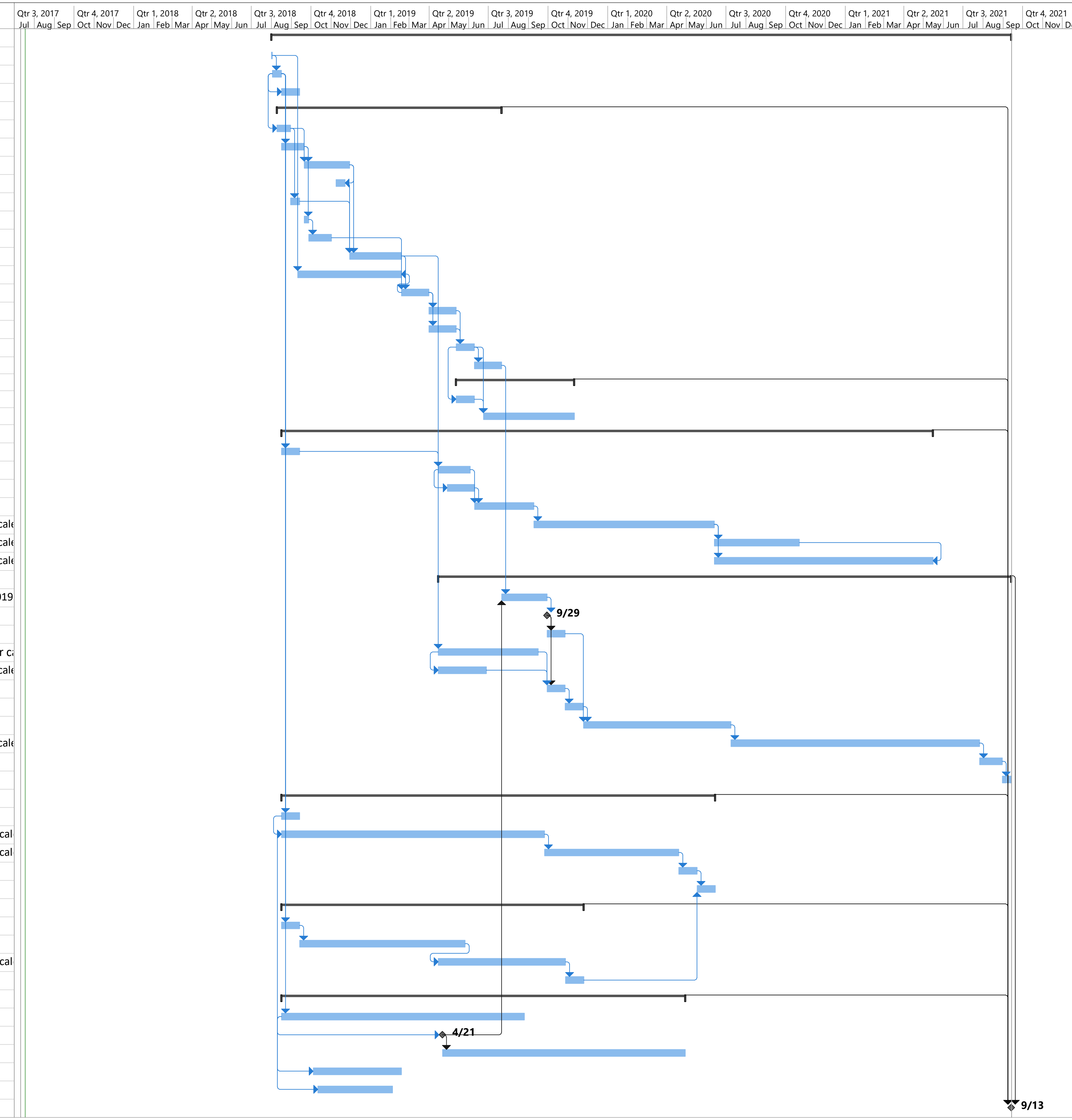
Figure 1 – Example of the Variability in Accuracy Ranges for a Process Industry Estimate

DETERMINATION OF THE COST ESTIMATE CLASS

The cost estimator makes the determination of the estimate class based upon the maturity level of project definition based on the status of specific key planning and design deliverables. The percent design completion may be correlated with the status, but the percentage should not be used as the estimate class determinant. While the determination of the status (and hence the estimate class) is somewhat subjective, having standards for the design input data, completeness and quality of the design deliverables will serve to make the determination more objective.

Construction Schedules

ID		Task Name	Duration	Start	Finish	Predecessors	Notes
1		**Elbow River Mclean Creek Damsite MC1 Construction- Fall Award**	**158.49 wks**	**Wed 8/1/18**	**Mon 9/13/21**		
2		Contracts Awarded	0.2 wks	Wed 8/1/18	Thu 8/2/18		
3		Initial Site Preparation	2 wks	Thu 8/2/18	Thu 8/16/18	2	
4		Install Camp, Garage and Laydown	4 wks	Thu 8/16/18	Thu 9/13/18	3SS+2 wks	
5		**Tunnel**	**48 wks**	**Thu 8/9/18**	**Sun 7/21/19**		
6		Excavation& Bolting of U/S Portal	3 wks	Thu 8/9/18	Thu 8/30/18	3SS+1 wk	
7		Dry House, Jumbo Garage, Water, Electricity, Permits, etc.	5 wks	Thu 8/16/18	Thu 9/20/18	3	
8		Tunnelling at U/S Portal	10 wks	Thu 9/20/18	Thu 11/29/1	6,7	
9		Drill & Blast Drop Raises	2 wks	Thu 11/8/18	Thu 11/22/1	8FF-1 wk	
10		Excavation & Bolting of D/S Portal	2 wks	Thu 8/30/18	Thu 9/13/18	6	
11		Erect & Calibrate Shotcrete Batch Plant	1 wk	Thu 9/20/18	Thu 9/27/18	7	
12		Erect & Calibrate Concrete Plant	5 wks	Thu 9/27/18	Thu 11/1/18	11	
13		Tunnelling at D/S Portal	10 wks	Thu 11/29/1	Sun 2/17/19	8,10,9	
14		Tunnel Forms Fabrication and Assembly	5 mons	Mon 9/10/18	Sun 2/17/19	15SF,2	
15		Concrete U/S Half of Tunnels (inc. grouting)	6 wks	Sun 2/17/19	Sun 3/31/19	13,12	
16		Concrete U/S Portal Structure	6 wks	Sun 3/31/19	Sun 5/12/19	15	
17		Concrete D/S Half of Tunnels (inc. grouting)	6 wks	Sun 3/31/19	Sun 5/12/19	15	
18		Concrete D/S Portal Structure	4 wks	Sun 5/12/19	Sun 6/9/19	16,17	
19		Concrete the two gate shafts to the top of the dam	6 wks	Sun 6/9/19	Sun 7/21/19	18	
20		**Fish Passage Tunnel**	**26 wks**	**Sun 5/12/19**	**Sun 11/10/1**		
21		Inlet & Outlet Excavation	4 wks	Sun 5/12/19	Sun 6/9/19	18SS	3wks-5wks
22		Tunnel Excavation, Lining, and Grouting	20 wks	Sun 6/23/19	Sun 11/10/1	18FS+2 wks,21	15wks-25wks
23		**Service Spillway**	**139.09 wks**	**Thu 8/16/18**	**Sat 5/15/21**		
24		Access Roads to Spillway and Borrow Pits	4 wks	Thu 8/16/18	Thu 9/13/18	3	
25		Stripping and Excavation	7 wks	Mon 4/15/1	Sun 6/2/19	24	start on Apr.15 2019
26		Sheet piling 1500 m2	6 wks	Mon 4/29/1	Sun 6/9/19	25SS+2 wks	
27		Place Granular for Slab & Footings & Mudslab	13 wks	Mon 6/10/1	Sun 9/8/19	26,25	
28		Secant piling	18 wks	Mon 9/9/19	Fri 6/12/20	27	frost & wet weather cal
29		Ogee, Flip Bucket Concrete	17 wks	Sat 6/13/20	Wed 10/21/2	28	frost & wet weather cal
30		Walls and Slab Concreting	25 wks	Sat 6/13/20	Sat 5/15/21	29FF,28	frost & wet weather cal
31		**Dam**	**123.2 wks**	**Mon 4/15/1**	**Mon 9/13/21**		
32		Coffer Dam 539,000 m3	10 wks	Sun 7/21/19	Sun 9/29/19	19,56	must finish by Nov 2019
33		Diversion	0 days	Sun 9/29/19	Sun 9/29/19	32	
34		Grouting Program off Coffer Dam	4 wks	Sun 9/29/19	Sun 10/27/1	33	
35		Bentonite Cut-Off Walls Left Abutment, inc. capping	20 wks	Mon 4/15/1	Sun 9/15/19	13	frost & wet weather c
36		Bentonite Cut-Off Walls Right Abutment, inc. capping	10 wks	Mon 4/15/1	Thu 6/27/19	35SS	frost & wet weather cal
37		River Channel Rock Trimming	4 wks	Sun 9/29/19	Sun 10/27/1	33,35,36	
38		River Channel Rock Grouting	4 wks	Sun 10/27/1	Sun 11/24/1	37	
39		Place Fill to el. 1413.5 (1,417,000 m3)	31 wks	Sun 11/24/1	Wed 7/8/20	38,34	
40		Balance of Fill to el. 1429 (1,625,000 m3)	31 wks	Wed 7/8/20	Mon 7/26/21	39	frost & wet weather cal
41		Install Hoist Structure& Gates	5 wks	Mon 7/26/21	Mon 8/30/21	40	
42		Guard Rail	2 wks	Mon 8/30/21	Mon 9/13/21	41	
43		**Highway Relocation**	**92.57 wks**	**Thu 8/16/18**	**Sun 6/14/20**		
44		Clearing	4 wks	Thu 8/16/18	Thu 9/13/18	3	
45		Earthworks	34 wks	Thu 8/16/18	Wed 9/25/1	44SS	Frost & wet weather cal
46		Pavement	8 wks	Wed 9/25/1	Sun 4/19/20	45	Frost & wet weather cal
47		Finishings	4 wks	Sun 4/19/20	Sun 5/17/20	46	
48		Reclaim	4 wks	Sun 5/17/20	Sun 6/14/20	53,47	
49		**Bridge**	**65.09 wks**	**Thu 8/16/18**	**Sun 11/24/1**		
50		Access and Staging Area	4 wks	Thu 8/16/18	Thu 9/13/18	3	
51		Substructure	35 wks	Thu 9/13/18	Sun 5/26/19	50	
52		Superstructure	26 wks	Mon 4/15/1	Sun 10/27/1	51FS-8 wks	Frost & wet weather cal
53		Finishings	4 wks	Mon 10/28/1	Sun 11/24/1	52	
54		**Facilities Relocation**	**86 wks**	**Thu 8/16/18**	**Wed 4/29/2**		
55		Build Gooseberry Ranger Station	52 wks	Thu 8/16/18	Sun 8/25/19	3	
56		Gooseberry Station Online	0 days	Sun 4/21/19	Sun 4/21/19	55SS+34 wks	
57		Demolish Elbow Valley Ranger Station	52 wks	Sun 4/21/19	Wed 4/29/2	56	
58		McLean Creek Store Relocation	18 wks	Thu 10/4/18	Sun 2/17/19	45SS+7 wks	
59		McLean Creek Campground Lot Replacement	15 wks	Thu 10/11/1	Sun 2/3/19	55SS+8 wks	
60		**Project Construction Completion**	**0 wks**	**Mon 9/13/21**	**Mon 9/13/21**	**5,23,31,43,49,54,2**	



Project: MC1 Fall Award Schedu
Date: Tue 7/18/17

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone	Manual Progress	
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline		
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Progress		

MCLEAN CREEK DAMSITE MC1 PROJECT

Technical Memorandum (DRAFT) on Schedule Range Estimating

1.0 INTRODUCTION

Palisade's @RISK for Project as an "add-in" for Microsoft Project, has been employed to conduct schedule range estimating for the MC1 project. The Monte-Carlo algorithm takes samples with each distribution based on the density of probability implied by the shape and area. Each time a sample is taken, it is entered into the plan so that the MS Project can recalculate the finish date.

2.0 MODELLING INPUTS

Beta distribution was applied to the inputs for the project schedule. The Beta functions are a set of flexible functions for generating an uncertain quantity known to be between given minimum and maximum values. The resulting density function can be symmetric or skewed in either direction, depending on the values of the distribution's two shape parameters. The beta distribution is an alternative to other popular bounded distributions, such as the triangular distribution, whose "most likely" parameter is easier to understand and estimate, yet the beta distribution provides somewhat more flexibility because it has two shape parameters that can be manipulated. In other words, the beta distribution provides a slightly wider variety of shapes.

Details on the inputs distributions are provided in Appendix 4.

The final project schedule used as the inputs to the Monte-Carlo Simulation is attached as Appendix 1. Assumptions made in this schedule include:

- Frost season occurs annually between November 16 – April 14.
- Wet weather season occurs annually in June, July and August. "Weather sensitive" activities are off one day per week in the wet weather season.
- "Weather sensitive" activities are noted in the project schedule file.
- The current high-level schedule assumes no resource limitations on any activities, no site layout constraints, no delays from permitting/approvals/etc. applications.
- All construction crews are expected to work seven days per week, and take only Christmas-New Year break (between December 24 and January 2 annually).
- Logics between activities are noted in the project schedule file.

3.0 MODELING OUTPUTS

The modeling outputs of key construction components are summarized in Table 1 below. Details on the Project Overall Finish Date (Item #2 in Table 1) are provided in Figure 1 below. Details and figures of all other components are provided in Appendix 2.

Table 1. Summary of Range Estimating Results on MC1 Project Schedule (Fall Award)

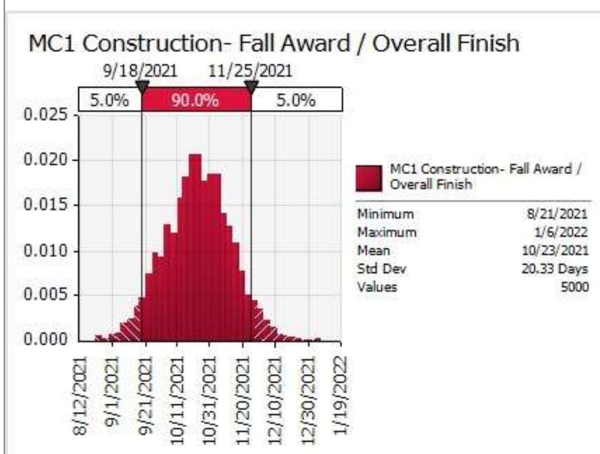
No.	Schedule Item	Min.	Max.	95 Percentile Value
1	**Project Total** Duration	155.26 weeks	173.48 weeks	168.91 weeks
2	**Project Overall** Finish Date	8/21/2021	1/6/2022	11/25/2021

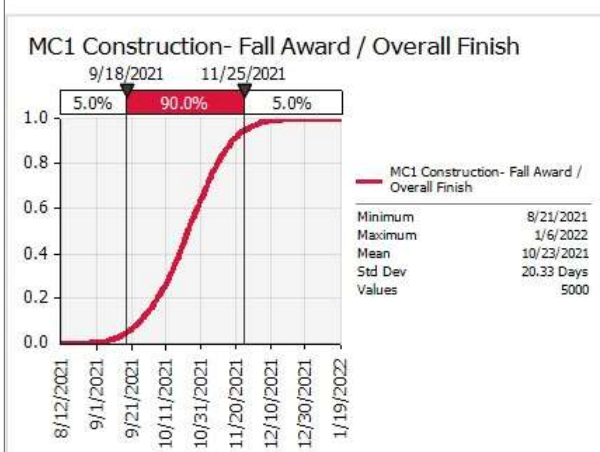
No.	Schedule Item	Min.	Max.	95 Percentile Value
3	**Tunnel** Duration	47.76 weeks	58.87 weeks	56.24 weeks
4	**Tunnel** Finish Date	7/19/2019	10/5/2019	9/17/2019
5	**Service Spillway** Duration	113.25 weeks	148.09 weeks	144.80 weeks
6	**Service Spillway** Finish Date	11/10/2020	7/24/2021	6/30/2021
7	**Dam** Duration	119.83 weeks	138.47 weeks	133.45 weeks
8	**Dam** Finish Date	8/20/2021	1/8/2022	11/24/2021
9	**Highway Relocation** Duration	65.06 weeks	117.02 weeks	109.57 weeks
10	**Highway Relocation** Finish Date	12/2/2019	12/13/2020	10/17/2020
11	**Bridge Relocation** Duration	60.51 weeks	112.45 weeks	105.46 weeks
12	**Bridge Relocation** Finish Date	11/2/2019	11/9/2020	9/18/2020
13	**Facility Relocation** Duration	72.05 weeks	87.29 weeks	85.81 weeks
14	**Facility Relocation** Finish Date	1/23/2020	5/19/2020	5/4/2020

@RISK Output Report for MC1 Construction- Fall Award / Overall Finish

Performed By: EHAN

Date: Monday, July 17, 2017 1:10:25 PM





Simulation Summary Information	
Workbook Name	MC1 Schedule Fall Award.xlsx
Number of Simulations	1
Number of Iterations	5000
Number of Inputs	90
Number of Outputs	20
Sampling Type	Latin Hypercube
Simulation Start Time	7/17/2017 12:50
Simulation Duration	00:03:05
Random # Generator	Mersenne Twister
Random Seed	1882790125

Summary Statistics for MC1 Construction- Fall Awa

Statistics		Percentile	
Minimum	Sat 8/21/21	5%	Sat 9/18/21
Maximum	Thu 1/6/22	10%	Sun 9/26/21
Mean	Sat 10/23/21	15%	Fri 10/1/21
Std Dev	20.32891299	20%	Tue 10/5/21
Variance	413.2647034	25%	Sat 10/9/21
Skewness	-0.040528615	30%	Wed 10/13/21
Kurtosis	2.965054516	35%	Sat 10/16/21
Median	Sat 10/23/21	40%	Mon 10/18/21
Mode	Tue 11/2/21	45%	Thu 10/21/21
Left X	Sat 9/18/21	50%	Sat 10/23/21
Left P	5%	55%	Tue 10/26/21
Right X	Thu 11/25/21	60%	Thu 10/28/21
Right P	95%	65%	Sun 10/31/21
Diff X	67.7625	70%	Wed 11/3/21
Diff P	90%	75%	Fri 11/5/21
#Errors	0	80%	Tue 11/9/21
Filter Min	Off	85%	Sat 11/13/21
Filter Max	Off	90%	Wed 11/17/21
#Filtered	0	95%	Thu 11/25/21

Figure 1. Range Estimating Output – Project Overall Finish Date

4.0 CRITICAL PATH

A Probabilistic Gantt chart was generated from the model and attached as Appendix 3. The probabilistic Gantt chart displays a Gantt chart that has bars showing probabilistic dates for task start and finish. The follow information is indicated for each activity:

- Base case (deterministic) start/finish dates – shown in blue and black horizontal bars
- User-defined confidence interval (90%) start/finish dates (5%-95%) – shown in pink horizontal bars
- Minimum and Maximum (100%) start/finish dates – shown in red horizontal bars
- Average (50%) start/finish dates – shown in red diamonds

The probabilistic Gantt chart can be used to prepare human resource, material, etc. for the specific task.

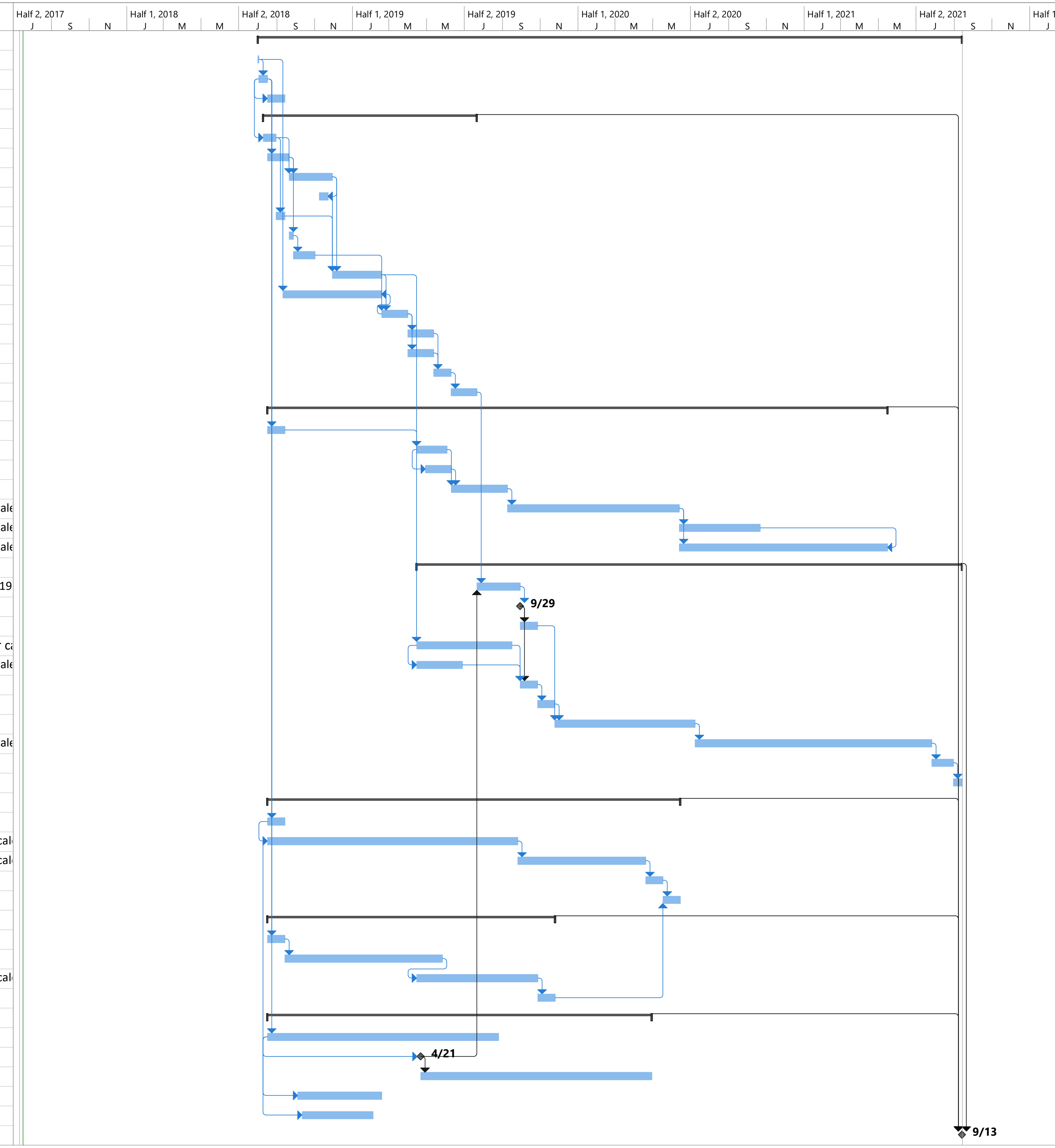
Critical Index (CI) for each task is also displayed on the probabilistic Gantt chart. The Critical Index is the percentage of iterations from the simulation that the task is on the critical path of the project. It allows the project team to rate the importance of tasks relative to schedule risk. The tasks with CI greater than 50% are highlighted in the Gantt chart, and are also summarized in Table 2 below.

Table 2. Schedule Tasks with CI Greater than 50%

Task ID	Task Name	Critical Index %
2	Contracts Awarded	100%
3	Initial Site Preparation	88.94%
7	Dry House, Jumbo Garage, Water, Electricity, Permits, etc.	88.94%
8	Tunnelling at U/S Portal	88.94%
13	Tunnelling at D/S Portal	88.94%
15	Concrete U/S Half of Tunnels (inc. grouting)	88.94%
17	Concrete D/S Half of Tunnels (inc. grouting)	53.14%
18	Concrete D/S Portal Structure	88.94%
19	Concrete the two gate shafts to the top of the dam	88.94%
29	Coffer Dam 539,000 m3	88.94%
30	Diversion	88.94%
34	River Channel Rock Trimming	90.64%
35	River Channel Rock Grouting	90.64%
36	Place Fill to el. 1413.5 (1,417,000 m3)	90.64%
37	Balance of Fill to el. 1429 (1,625,000 m3)	100%
38	Install Hoist Structure& Gates	100%
39	Guard Rail	100%
57	Project Construction Completion	100%

APPENDIX 1. MC1 FALL AWARD SCHEDULE

ID	Task Name	Duration	Start	Finish	Predecessors	Notes
1	**Elbow River Mclean Creek Damsite MC1 Construction- Fall Award**	**158.49 wks**	**8/1/18**	**9/13/21**		
2	Contracts Awarded	0.2 wks	8/1/18	8/2/18		
3	Initial Site Preparation	2 wks	8/2/18	8/16/18	2	
4	Install Camp, Garage and Laydown	4 wks	8/16/18	9/13/18	3SS+2 wks	
5	**Tunnel**	**48 wks**	**8/9/18**	**7/21/19**		
6	Excavation& Bolting of U/S Portal	3 wks	8/9/18	8/30/18	3SS+1 wk	
7	Dry House, Jumbo Garage, Water, Electricity, Permits, etc.	5 wks	8/16/18	9/20/18	3	
8	Tunnelling at U/S Portal	10 wks	9/20/18	11/29/18	6,7	
9	Drill & Blast Drop Raises	2 wks	11/8/18	11/22/18	8FF-1 wk	
10	Excavation & Bolting of D/S Portal	2 wks	8/30/18	9/13/18	6	
11	Erect & Calibrate Shotcrete Batch Plant	1 wk	9/20/18	9/27/18	7	
12	Erect & Calibrate Concrete Plant	5 wks	9/27/18	11/1/18	11	
13	Tunnelling at D/S Portal	10 wks	11/29/18	2/17/19	8,10,9	
14	Tunnel Forms Fabrication and Assembly	5 mons	9/10/18	2/17/19	15SF,2	
15	Concrete U/S Half of Tunnels (inc. grouting)	6 wks	2/17/19	3/31/19	13,12	
16	Concrete U/S Portal Structure	6 wks	3/31/19	5/12/19	15	
17	Concrete D/S Half of Tunnels (inc. grouting)	6 wks	3/31/19	5/12/19	15	
18	Concrete D/S Portal Structure	4 wks	5/12/19	6/9/19	16,17	
19	Concrete the two gate shafts to the top of the dam	6 wks	6/9/19	7/21/19	18	
20	**Service Spillway**	**139.09 wks**	**8/16/18**	**5/15/21**		
21	Access Roads to Spillway and Borrow Pits	4 wks	8/16/18	9/13/18	3	
22	Stripping and Excavation	7 wks	4/15/19	6/2/19	21	start on Apr.15 2019
23	Sheet piling 1500 m2	6 wks	4/29/19	6/9/19	22SS+2 wks	
24	Place Granular for Slab & Footings & Mudslab	13 wks	6/10/19	9/8/19	23,22	
25	Secant piling	18 wks	9/9/19	6/12/20	24	frost & wet weather cale
26	Ogee, Flip Bucket Concrete	17 wks	6/13/20	10/21/20	25	frost & wet weather cale
27	Walls and Slab Concreting	25 wks	6/13/20	5/15/21	26FF,25	frost & wet weather cale
28	**Dam**	**123.2 wks**	**4/15/19**	**9/13/21**		
29	Coffer Dam 539,000 m3	10 wks	7/21/19	9/29/19	19,53	must finish by Nov 2019
30	Diversion	0 days	9/29/19	9/29/19	29	
31	Grouting Program off Coffer Dam	4 wks	9/29/19	10/27/19	30	
32	Bentonite Cut-Off Walls Left Abutment, inc. capping	20 wks	4/15/19	9/15/19	13	frost & wet weather ca
33	Bentonite Cut-Off Walls Right Abutment, inc. capping	10 wks	4/15/19	6/27/19	32SS	frost & wet weather cale
34	River Channel Rock Trimming	4 wks	9/29/19	10/27/19	30,32,33	
35	River Channel Rock Grouting	4 wks	10/27/19	11/24/19	34	
36	Place Fill to el. 1413.5 (1,417,000 m3)	31 wks	11/24/19	7/8/20	35,31	
37	Balance of Fill to el. 1429 (1,625,000 m3)	31 wks	7/8/20	7/26/21	36	frost & wet weather cale
38	Install Hoist Structure& Gates	5 wks	7/26/21	8/30/21	37	
39	Guard Rail	2 wks	8/30/21	9/13/21	38	
40	**Highway Relocation**	**92.57 wks**	**8/16/18**	**6/14/20**		
41	Clearing	4 wks	8/16/18	9/13/18	3	
42	Earthworks	34 wks	8/16/18	9/25/19	41SS	Frost & wet weather cal
43	Pavement	8 wks	9/25/19	4/19/20	42	Frost & wet weather cal
44	Finishings	4 wks	4/19/20	5/17/20	43	
45	Reclaim	4 wks	5/17/20	6/14/20	50,44	
46	**Bridge**	**65.09 wks**	**8/16/18**	**11/24/19**		
47	Access and Staging Area	4 wks	8/16/18	9/13/18	3	
48	Substructure	35 wks	9/13/18	5/26/19	47	
49	Superstructure	26 wks	4/15/19	10/27/19	48FS-8 wks	Frost & wet weather cal
50	Finishings	4 wks	10/28/19	11/24/19	49	
51	**Facilities Relocation**	**86 wks**	**8/16/18**	**4/29/20**		
52	Build Gooseberry Ranger Station	52 wks	8/16/18	8/25/19	3	
53	Gooseberry Station Online	0 days	4/21/19	4/21/19	52SS+34 wks	
54	Demolish Elbow Valley Ranger Station	52 wks	4/21/19	4/29/20	53	
55	McLean Creek Store Relocation	18 wks	10/4/18	2/17/19	42SS+7 wks	
56	McLean Creek Campground Lot Replacement	15 wks	10/11/18	2/3/19	52SS+8 wks	
57	**Project Construction Completion**	**0 wks**	**9/13/21**	**9/13/21**	**5,20,28,40,46,51**	

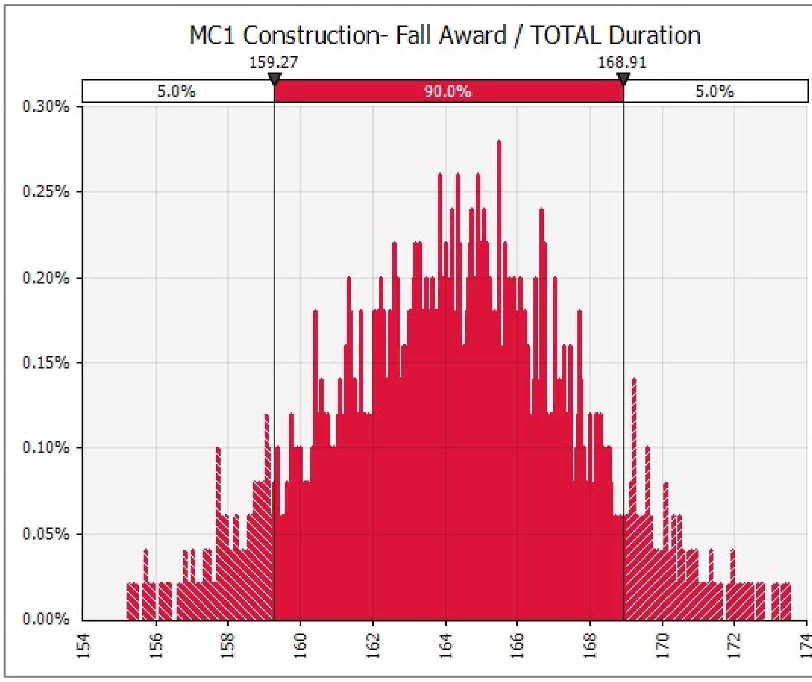


Project: MC1 Construction Sche
Date: July 16, 2017

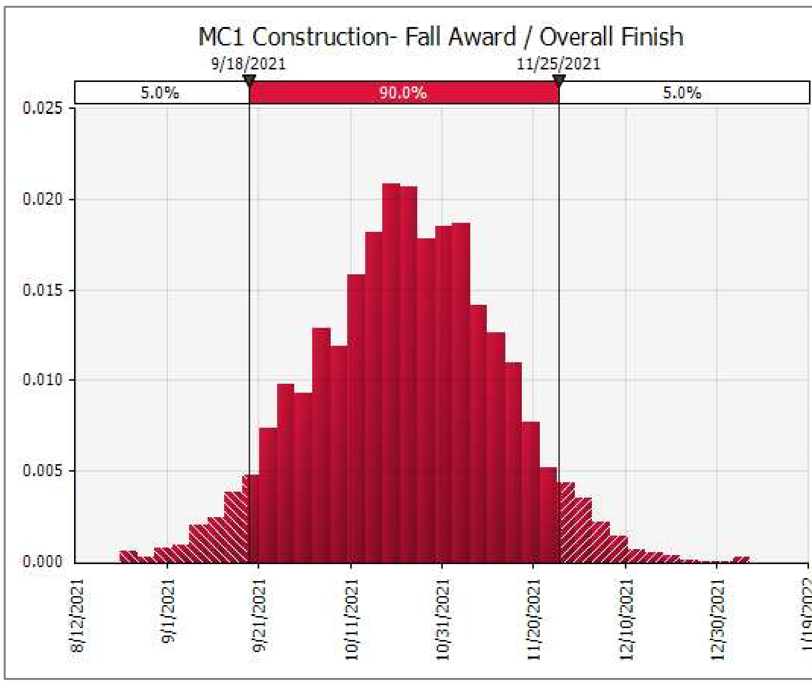
Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone	Manual Progress
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline	
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Progress	

APPENDIX 2. MODEL OUTPUTS

A2-1. Project Overall

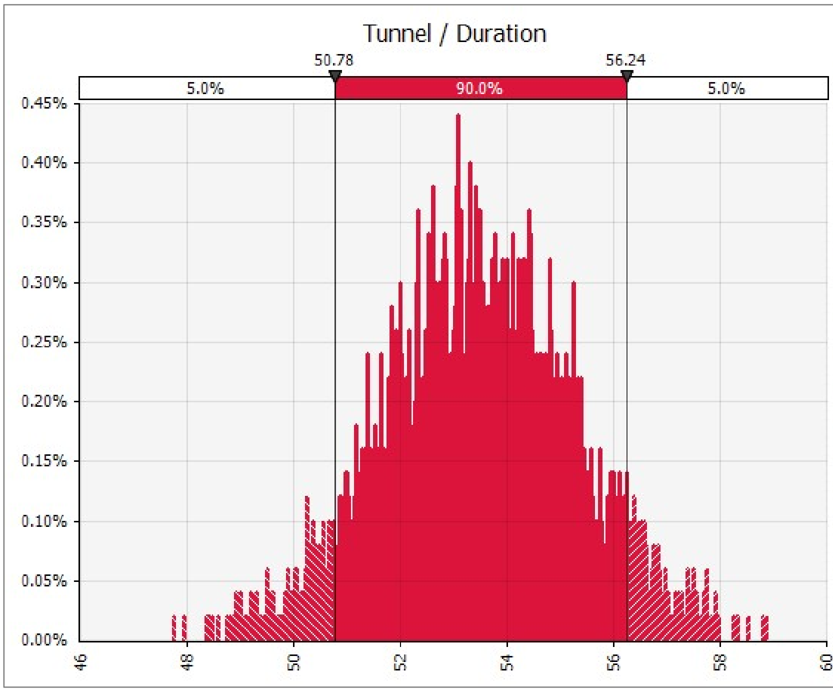


	MC1 Construction- Fall Award / TOTAL Duration
5%	159.270
10%	160.360
15%	161.100
20%	161.700
25%	162.260
30%	162.720
35%	163.150
40%	163.520
45%	163.890
50%	164.230
55%	164.600
60%	164.990
65%	165.360
70%	165.720
75%	166.100
80%	166.630
85%	167.170
90%	167.850
95%	168.910
99%	170.790

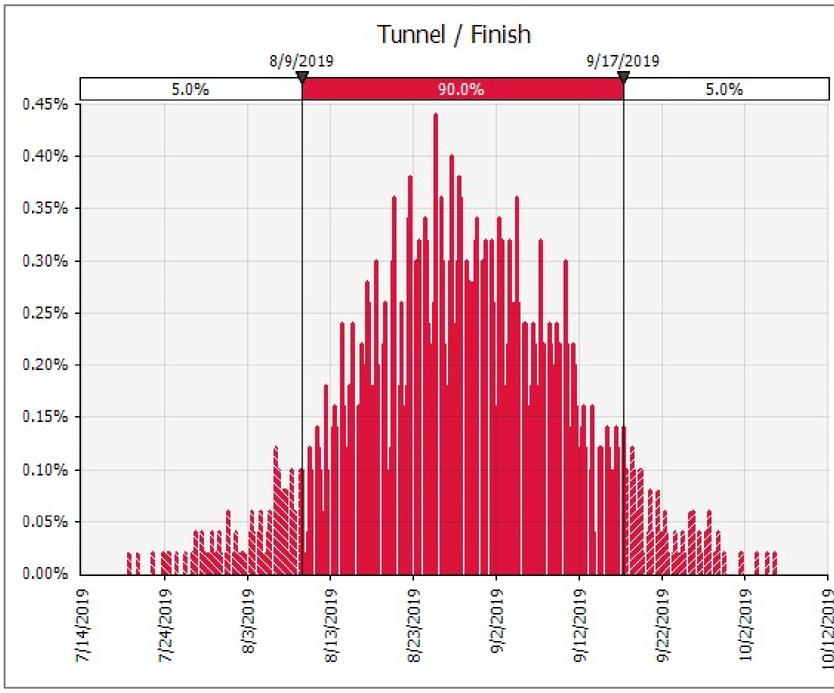


	MC1 Construction- Fall Award / Overall Finish
Cell	Tasks!F2
Minimum	8/21/2021
Maximum	1/6/2022
Mean	10/23/2021
90% CI	± 0.473
Mode	11/2/2021
Median	10/23/2021
Std Dev	20.33 Days
Skewness	-0.0405
Kurtosis	2.9651
Values	5000
Errors	0
Filtered	0
Left X	9/18/2021
Left P	5.0%
Right X	11/25/2021
Right P	95.0%
Dif. X	67.76
Dif. P	90.0%
1%	9/6/2021

A2-2. Tunnel

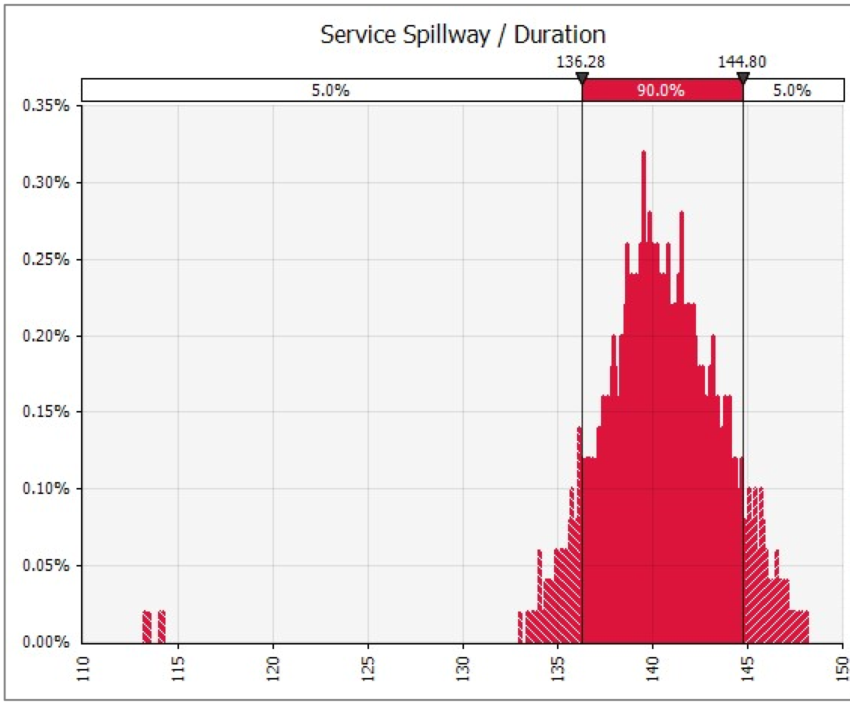


	Tunnel / Duration
Dif. X	5.460
Dif. P	90.0%
1%	49.570
5%	50.780
10%	51.390
15%	51.800
20%	52.110
25%	52.410
30%	52.670
35%	52.880
40%	53.110
45%	53.320
50%	53.520
55%	53.730
60%	53.940
65%	54.150
70%	54.390
75%	54.620
80%	54.900
85%	55.220
90%	55.610
95%	56.240
99%	57.350

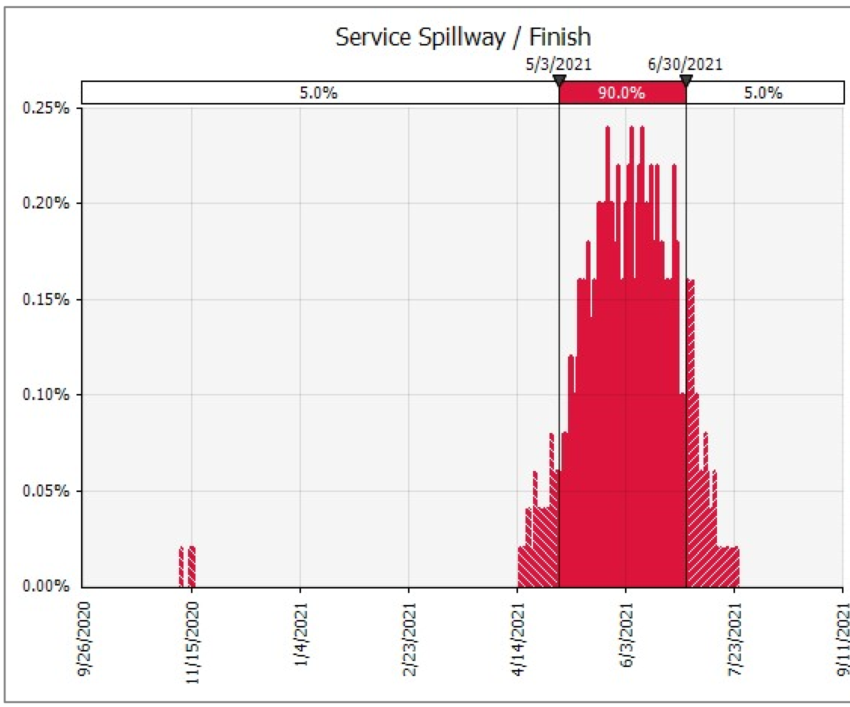


	Tunnel / Finish
Cell	Tasks!F6
Minimum	7/19/2019 3:12 PM
Maximum	10/5/2019 12:54 PM
Mean	8/29/2019 12:44 AM
90% CI	± 0.268
Mode	8/25/2019 4:54 PM
Median	8/29/2019 8:24 AM
Std Dev	11.518 Days
Skewness	-0.0463
Kurtosis	2.9281
Values	5000
Errors	0
Filtered	0
Left X	8/9/2019
Left P	5.0%
Right X	9/17/2019
Right P	95.0%
Dif. X	38.675
Dif. P	90.0%
1%	8/1/2019 11:54 AM
5%	8/9/2019 4:36 PM
10%	8/14/2019 9:18 AM
15%	8/16/2019 6:00 PM

A2-3. Service Spillway

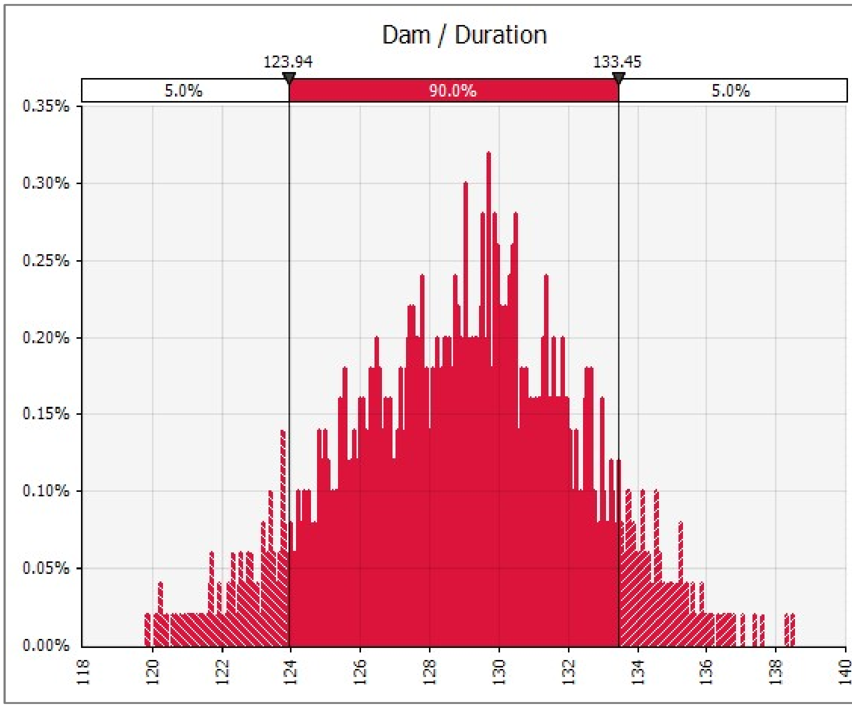


	Service Spillway / Duration
Dif. P	90.0%
1%	134.730
5%	136.280
10%	137.100
15%	137.700
20%	138.180
25%	138.590
30%	138.960
35%	139.330
40%	139.640
45%	139.950
50%	140.260
55%	140.630
60%	140.960
65%	141.350
70%	141.760
75%	142.160
80%	142.640
85%	143.190
90%	143.870
95%	144.800
99%	146.260

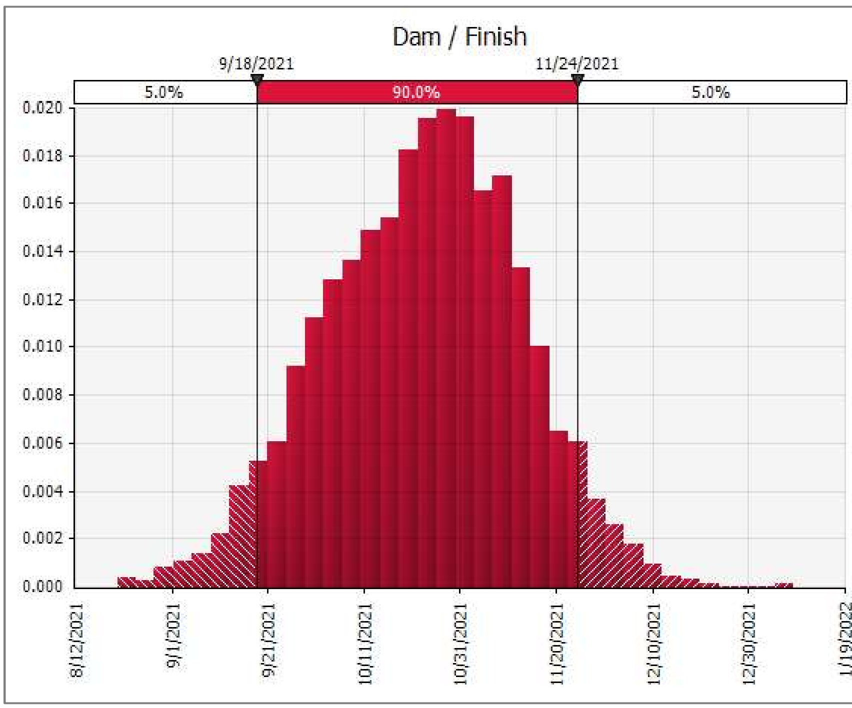


	Service Spillway / Finish
Cell	Tasks!F21
Minimum	11/10/2020
Maximum	7/24/2021
Mean	5/31/2021
90% CI	± 0.429
Mode	5/25/2021
Median	5/30/2021
Std Dev	18.45 Days
Skewness	-0.8639
Kurtosis	12.8893
Values	5000
Errors	0
Filtered	0
Left X	5/3/2021
Left P	5.0%
Right X	6/30/2021
Right P	95.0%
Dif. X	58.15
Dif. P	90.0%
1%	4/22/2021
5%	5/3/2021
10%	5/8/2021
15%	5/12/2021

A2-4. Dam

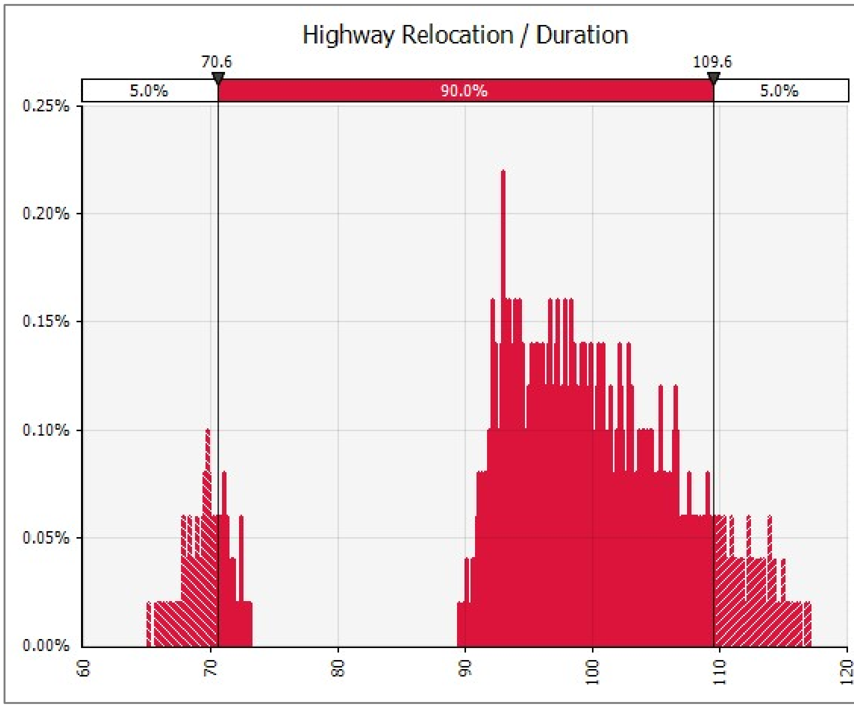


	Dam / Duration
Dif. X	9.510
Dif. P	90.0%
1%	122.000
5%	123.940
10%	125.090
15%	125.810
20%	126.370
25%	126.870
30%	127.370
35%	127.780
40%	128.250
45%	128.650
50%	129.040
55%	129.410
60%	129.770
65%	130.130
70%	130.510
75%	130.960
80%	131.370
85%	131.830
90%	132.510
95%	133.450
99%	135.170

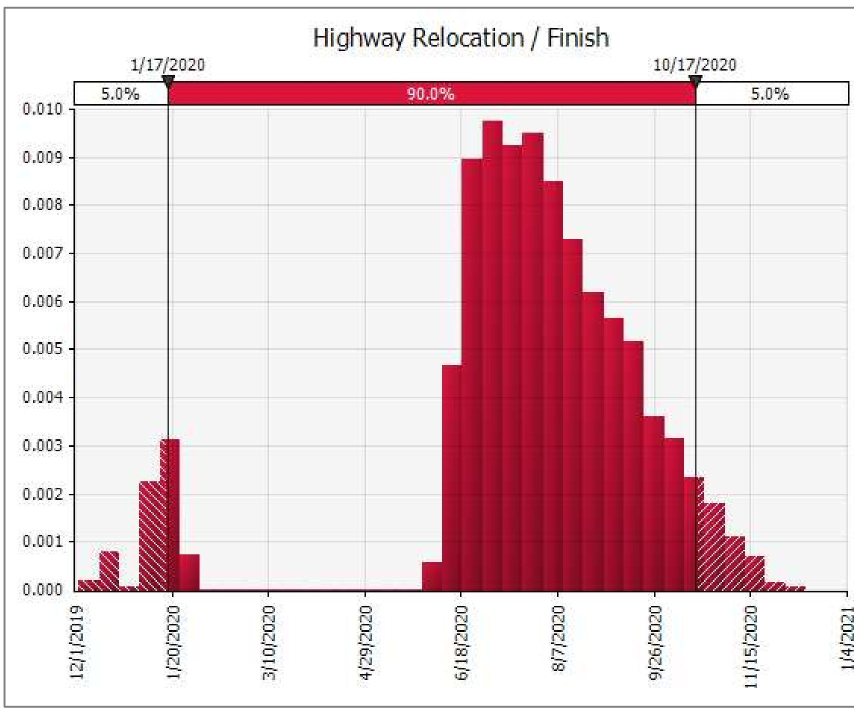


	Dam / Finish
Cell	Tasks!F29
Minimum	8/20/2021
Maximum	1/8/2022
Mean	10/23/2021
90% CI	± 0.470
Mode	11/9/2021
Median	10/24/2021
Std Dev	20.22 Days
Skewness	-0.1307
Kurtosis	2.8269
Values	5000
Errors	0
Filtered	0
Left X	9/18/2021
Left P	5.0%
Right X	11/24/2021
Right P	95.0%
Dif. X	66.83
Dif. P	90.0%
1%	9/4/2021
5%	9/18/2021
10%	9/26/2021
15%	10/1/2021

A2-5. Highway Relocation

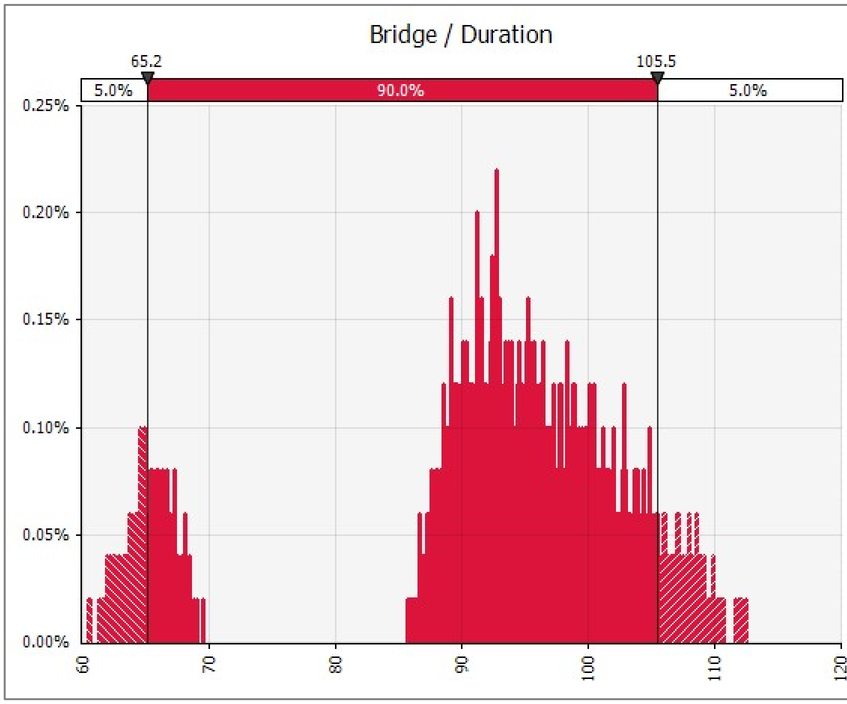


	Highway Relocation / Duration
1%	68.380
5%	70.590
10%	91.630
15%	92.760
20%	93.510
25%	94.170
30%	94.930
35%	95.710
40%	96.510
45%	97.200
50%	97.960
55%	98.710
60%	99.590
65%	100.570
70%	101.570
75%	102.750
80%	103.940
85%	105.330
90%	107.070
95%	109.570
99%	112.970

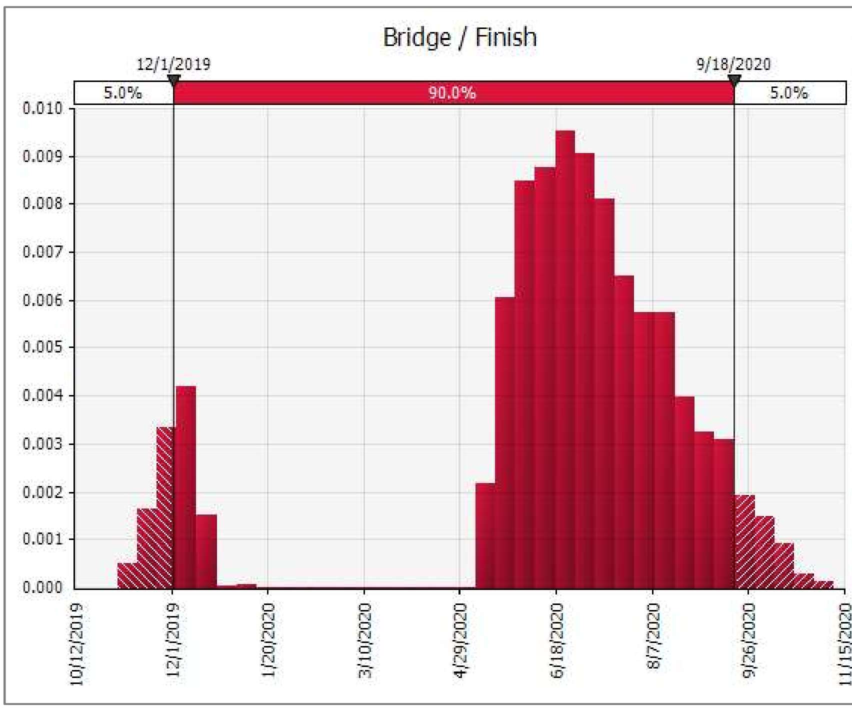


	Highway Relocation / Finish
Cell	Tasks!F41
Minimum	12/2/2019
Maximum	12/13/2020
Mean	7/23/2020
90% CI	± 1.54
Mode	6/23/2020
Median	7/28/2020
Std Dev	66.22 Days
Skewness	-1.5429
Kurtosis	6.0473
Values	5000
Errors	0
Filtered	0
Left X	1/17/2020
Left P	5.0%
Right X	10/17/2020
Right P	95.0%
Dif. X	273.89
Dif. P	90.0%
1%	12/23/2019
5%	1/17/2020
10%	6/13/2020
15%	6/21/2020

A2-6. Bridge

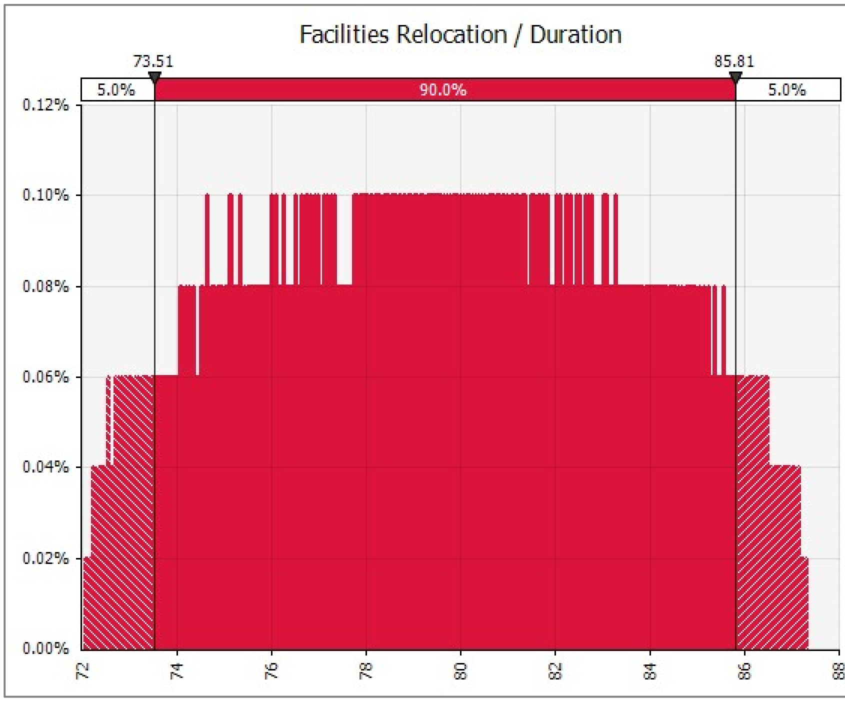


	Bridge / Duration
Dif. X	40.290
Dif. P	90.0%
1%	63.000
5%	65.170
10%	67.050
15%	88.030
20%	89.120
25%	90.050
30%	90.840
35%	91.670
40%	92.450
45%	93.170
50%	93.940
55%	94.690
60%	95.560
65%	96.470
70%	97.560
75%	98.680
80%	99.910
85%	101.220
90%	103.050
95%	105.460
99%	108.750

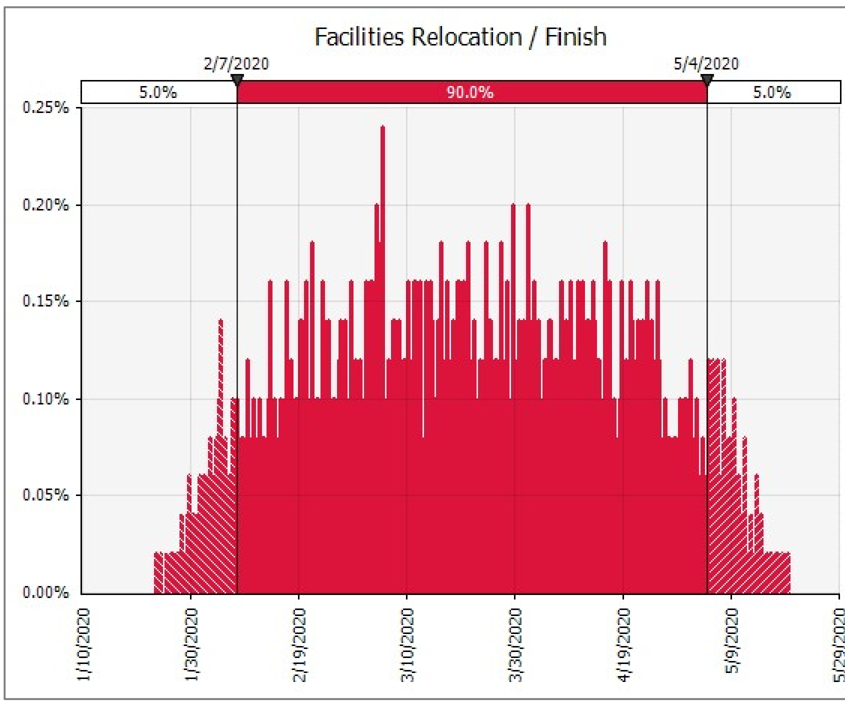


	Bridge / Finish
Cell	Tasks!F47
Minimum	11/2/2019
Maximum	11/9/2020
Mean	6/16/2020
90% CI	± 1.86
Mode	5/30/2020
Median	6/30/2020
Std Dev	80.07 Days
Skewness	-1.4774
Kurtosis	4.6614
Values	5000
Errors	0
Filtered	0
Left X	12/1/2019
Left P	5.0%
Right X	9/18/2020
Right P	95.0%
Dif. X	291.89
Dif. P	90.0%
1%	11/16/2019
5%	12/1/2019
10%	12/13/2019
15%	5/19/2020

A2-7. Facility Relocation

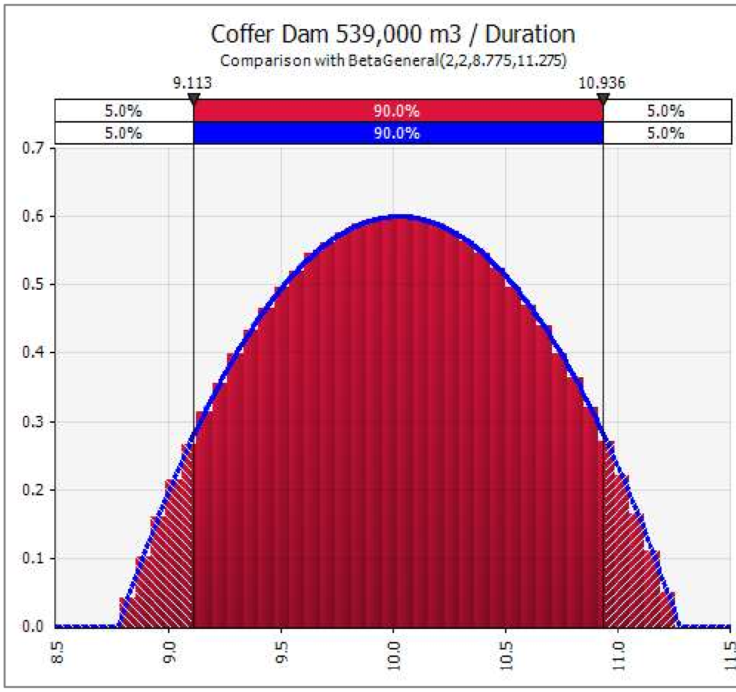


	Facilities Relocation / Duration
1%	72.530
5%	73.510
10%	74.410
15%	75.190
20%	75.910
25%	76.580
30%	77.220
35%	77.840
40%	78.460
45%	79.060
50%	79.660
55%	80.260
60%	80.870
65%	81.480
70%	82.100
75%	82.750
80%	83.420
85%	84.130
90%	84.910
95%	85.810
99%	86.800

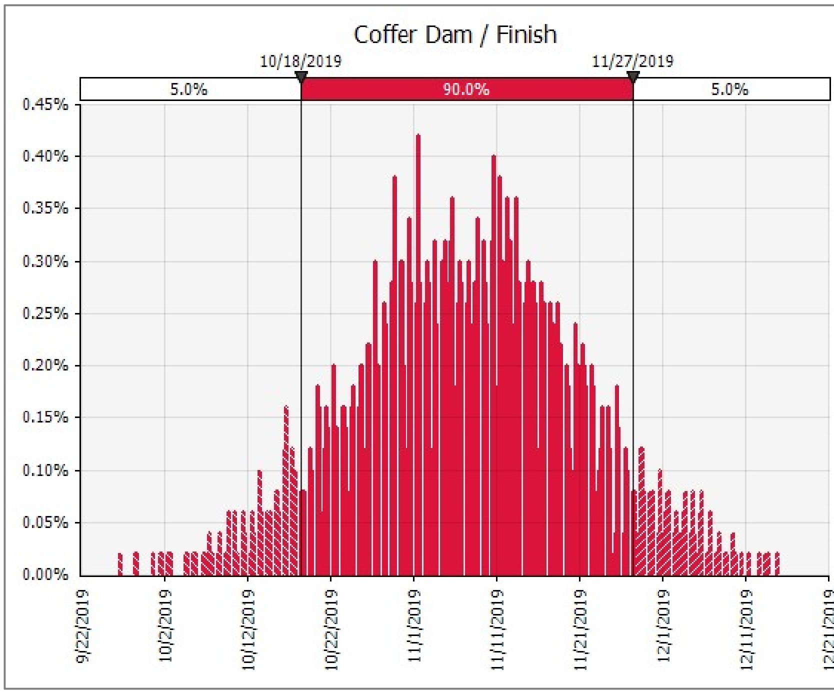


	Facilities Relocation / Finish
Cell	Tasks!F52
Minimum	1/23/2020
Maximum	5/19/2020
Mean	3/22/2020
90% CI	± 0.626
Mode	3/5/2020
Median	3/21/2020
Std Dev	26.90 Days
Skewness	0.0040
Kurtosis	2.0391
Values	5000
Errors	0
Filtered	0
Left X	2/7/2020
Left P	5.0%
Right X	5/4/2020
Right P	95.0%
Dif. X	87.03
Dif. P	90.0%
1%	1/31/2020
5%	2/7/2020
10%	2/14/2020
15%	2/19/2020

A2-8 Coffer Dam



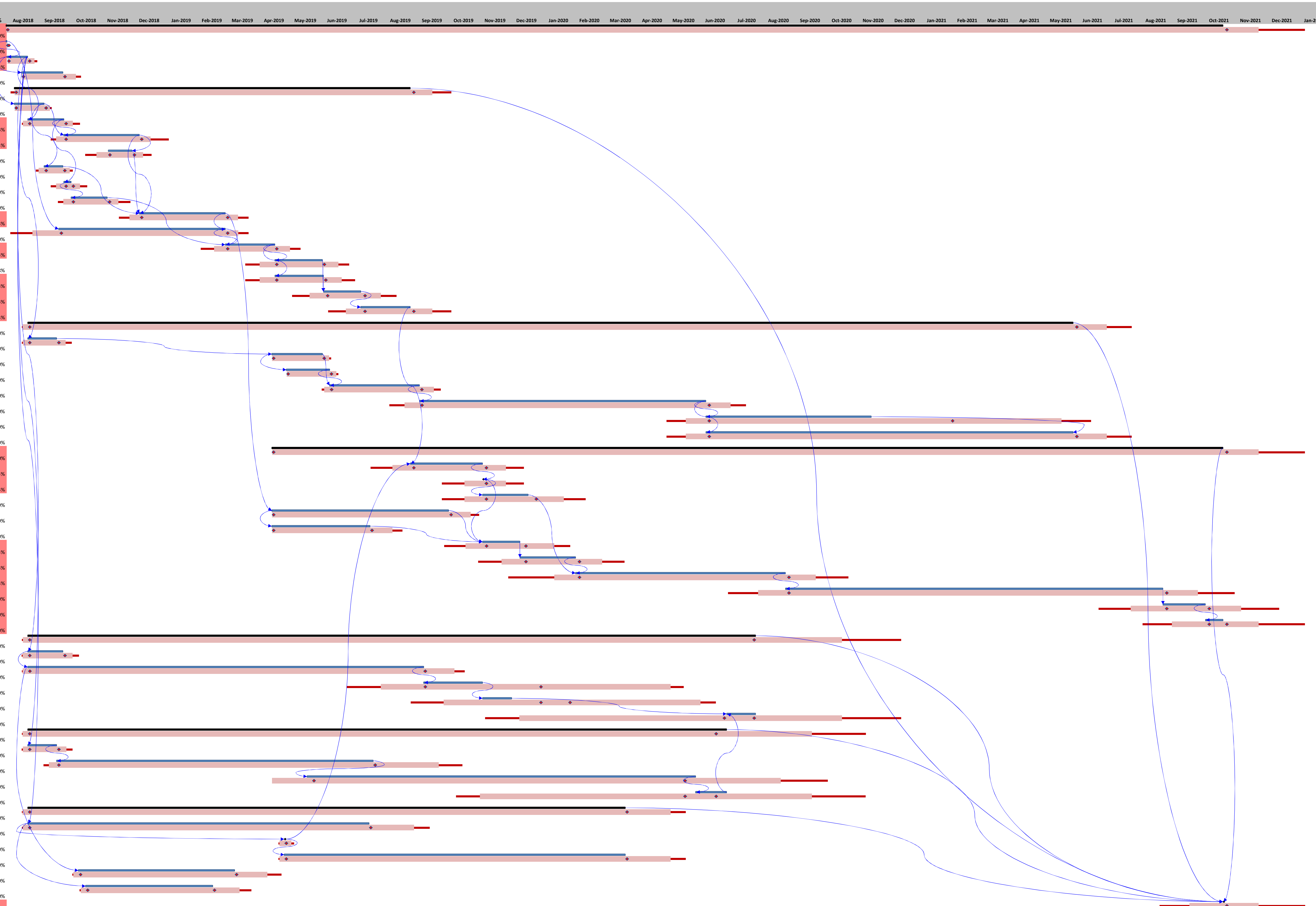
	Coffer Dam 539,000 m3 / Duration	BetaGeneral (2,2,8.775,11.27
1%	8.9221	8.9223
5%	9.1131	9.1134
10%	9.2643	9.2645
15%	9.3858	9.3860
20%	9.4926	9.4929
25%	9.5908	9.5909
30%	9.6831	9.6831
35%	9.7714	9.7715
40%	9.8571	9.8573
45%	9.9413	9.9415
50%	10.0249	10.0250
55%	10.1082	10.1085
60%	10.1926	10.1927
65%	10.2784	10.2785
70%	10.3667	10.3669
75%	10.4589	10.4591
80%	10.5568	10.5571
85%	10.6636	10.6640
90%	10.7851	10.7855
95%	10.9364	10.9366
99%	11.1271	11.1277



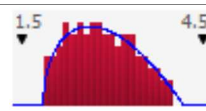
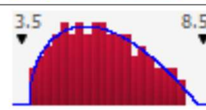
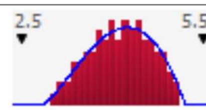
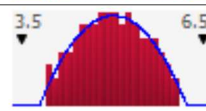
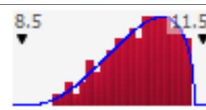
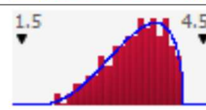
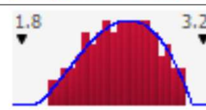
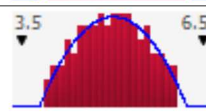
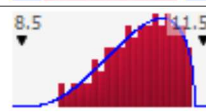
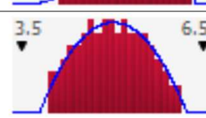
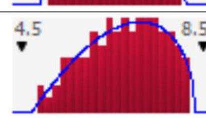
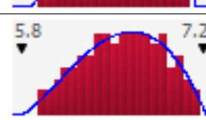
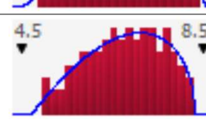
	Coffer Dam / Finish
Cell	Tasks!F30
Minimum	9/26/2019 4:06 PM
Maximum	12/14/2019 5:06 PM
Mean	11/7/2019 4:56 AM
90% CI	± 0.283
Mode	11/1/2019 1:06 PM
Median	11/7/2019 11:54 AM
Std Dev	12.156 Days
Skewness	-0.0429
Kurtosis	2.8873
Values	5000
Errors	0
Filtered	0
Left X	10/18/2019
Left P	5.0%
Right X	11/27/2019
Right P	95.0%
Dif. X	40.075
Dif. P	90.0%
1%	10/9/2019 5:42 PM
5%	10/18/2019 8:54 AM
10%	10/22/2019 10:12 AM
15%	10/25/2019 12:24 PM

APPENDIX 3. PROBABILISTIC GANTT CHART

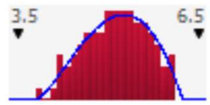
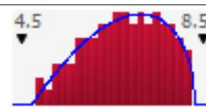
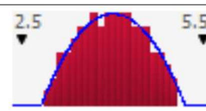
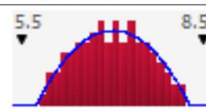
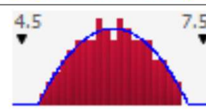
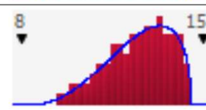
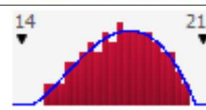
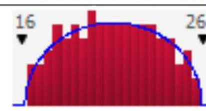
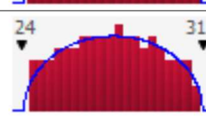
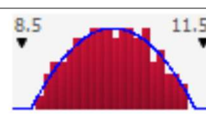
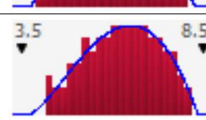
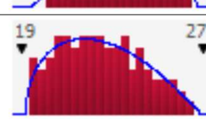
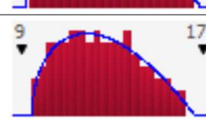
ID	Task Name	Deterministic Start	Deterministic Finish	Critical Index%
1	**Elbow River Mclean Creek Damsite MC1 Construc**	**Wed 8/1/18**	**Thu 10/21/21**	100%
2	Contracts Awarded	Wed 8/1/18	Thu 8/2/18	100%
3	Initial Site Preparation	Thu 8/2/18	Wed 8/22/18	88.94%
4	Install Camp, Garage and Laydown	Thu 8/16/18	Tue 9/25/18	0%
5	**Tunnel**	**Thu 8/9/18**	**Tue 8/27/19**	0%
6	Excavation& Bolting of U/S Porta	Thu 8/9/18	Fri 9/7/18	0%
7	Dry House, Jumbo Garage, Wate	Wed 8/22/18	Wed 9/26/18	88.94%
8	Tunnelling at U/S Portal	Wed 9/26/18	Sat 12/8/18	88.94%
9	Drill & Blast Drop Raises	Thu 11/8/18	Sat 12/1/18	0%
10	Excavation & Bolting of D/S Porta	Fri 9/7/18	Tue 9/25/18	0%
11	Erect & Calibrate Shotcrete Batcl	Wed 9/26/18	Wed 10/3/18	0%
12	Erect & Calibrate Concrete Plant	Wed 10/3/18	Wed 11/7/18	0%
13	Tunnelling at D/S Portal	Sat 12/8/18	Fri 3/1/19	88.94%
14	Tunnel Forms Fabrication and As	Fri 9/21/18	Fri 3/1/19	0%
15	Concrete U/S Half of Tunnels (inc	Fri 3/1/19	Thu 4/18/19	88.94%
16	Concrete U/S Portal Structure	Thu 4/18/19	Mon 6/3/19	36.18%
17	Concrete D/S Half of Tunnels (inc	Thu 4/18/19	Tue 6/4/19	53.14%
18	Concrete D/S Portal Structure	Tue 6/4/19	Wed 7/10/19	88.94%
19	Concrete the two gate shafts to t	Wed 7/10/19	Tue 8/27/19	88.94%
20	**Service Spillway**	**Wed 8/22/18**	**Sat 5/29/21**	0%
21	Access Roads to Spillway and Bor	Wed 8/22/18	Wed 9/19/18	0%
22	Stripping and Excavation	Mon 4/15/19	Mon 6/3/19	0%
23	Sheet piling 1500 m2	Mon 4/29/19	Mon 6/10/19	0%
24	Place Granular for Slab & Footing	Mon 6/10/19	Thu 9/5/19	0%
25	Secant piling	Thu 9/5/19	Mon 6/8/20	0%
26	Ogee, Flip Bucket Concrete	Mon 6/8/20	Sun 11/15/20	0%
27	Walls and Slab Concreting	Mon 6/8/20	Sat 5/29/21	0%
28	**Dam**	**Mon 4/15/19**	**Thu 10/21/21**	100%
29	Coffer Dam 539,000 m3	Tue 8/27/19	Tue 11/5/19	88.94%
30	Diversion	Tue 11/5/19	Tue 11/5/19	88.94%
31	Grouting Program off Coffer Dam	Tue 11/5/19	Thu 12/19/19	0%
32	Bentonite Cut-Off Walls Left Abu	Mon 4/15/19	Thu 10/3/19	1.70%
33	Bentonite Cut-Off Walls Right Ab	Mon 4/15/19	Fri 7/19/19	0%
34	River Channel Rock Trimming	Tue 11/5/19	Wed 12/11/19	90.64%
35	River Channel Rock Grouting	Wed 12/11/19	Mon 2/3/20	90.64%
36	Place Fill to el. 1413.5 (1,417,00(	Mon 2/3/20	Mon 8/24/20	90.64%
37	Balance of Fill to el. 1429 (1,625,	Mon 8/24/20	Tue 8/24/21	100%
38	Install Hoist Structure& Gates	Tue 8/24/21	Mon 10/4/21	100%
39	Guard Rail	Mon 10/4/21	Thu 10/21/21	100%
40	**Highway Relocation**	**Wed 8/22/18**	**Sun 7/26/20**	0%
41	Clearing	Wed 8/22/18	Tue 9/25/18	0%
42	Earthworks	Wed 8/22/18	Mon 9/9/19	0%
43	Pavement	Mon 9/9/19	Tue 11/5/19	0%
44	Finishings	Tue 11/5/19	Tue 12/3/19	0%
45	Reclaim	Sun 6/28/20	Sun 7/26/20	0%
46	**Bridge**	**Wed 8/22/18**	**Sun 6/28/20**	0%
47	Access and Staging Area	Wed 8/22/18	Wed 9/19/18	0%
48	Substructure	Wed 9/19/18	Mon 7/22/19	0%
49	Superstructure	Sun 5/19/19	Fri 5/29/20	0%
50	Finishings	Fri 5/29/20	Sun 6/28/20	0%
51	**Facilities Relocation**	**Wed 8/22/18**	**Sun 3/22/20**	0%
52	Build Gooseberry Ranger Station	Wed 8/22/18	Thu 7/18/19	0%
53	Gooseberry Station Online	Sat 4/27/19	Sat 4/27/19	0%
54	Demolish Elbow Valley Ranger St	Sat 4/27/19	Sun 3/22/20	0%
55	McLean Creek Store Relocation	Wed 10/10/18	Sun 3/10/19	0%
56	McLean Creek Campground Lot F	Wed 10/17/18	Sun 2/17/19	0%
57	Project Construction Completion	Thu 10/21/21	Thu 10/21/21	100%



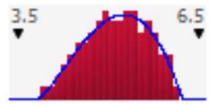
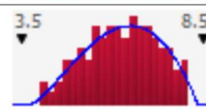
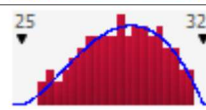
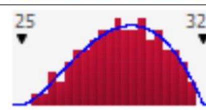
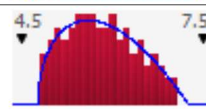
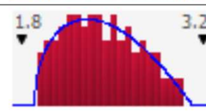
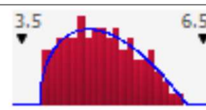
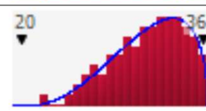
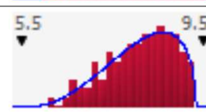
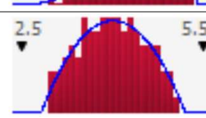
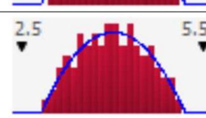
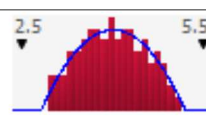
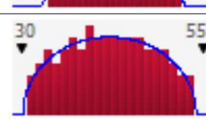
APPENDIX 4. DISTRIBUTIONS DERIVED FROM VARIOUS PARAMETERS

@RISK Input Results					Unit: Weeks
Name	Graph	Min	Mean	Max	95%
Initial Site Preparation / Distribution		1.98	2.87	3.97	3.71
Install Camp, Garage and Laydown / Distribution		3.92	5.74	7.90	7.48
Category: Tunnel					
Excavation& Bolting of U/S Portal		3.08	4.15	5.07	4.86
Dry House, Jumbo Garage, Water, Electricity, Permits, etc.		4.01	5.02	6.04	5.85
Tunnelling at U/S Portal / Distribution		9.09	10.44	11.24	11.13
Drill & Blast Drop Raises / Distribution		2.14	3.38	4.09	3.97
Excavation & Bolting of D/S Portal / Distribution		2.05	2.58	3.04	2.93
Erect & Calibrate Concrete Plant / Distribution		3.99	5.03	6.02	5.84
Tunnelling at D/S Portal / Distribution		9.19	10.44	11.24	11.14
Tunnel Forms Fabrication and Assembly / Distribution		4.03m	5.03m	6.10m	5.82m
Concrete U/S Half of Tunnels (inc. grouting) / Distribution		4.99	6.77	8.12	7.94
Concrete U/S Portal Structure / Distribution		5.97	6.59	7.14	7.01
Concrete D/S Half of Tunnels (inc. grouting) / Distribution		5.11	6.77	8.19	7.92

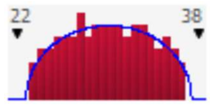
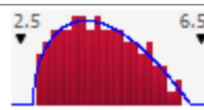
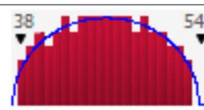
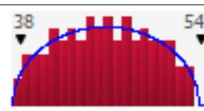
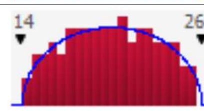
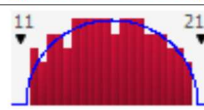
@RISK Input Results **Unit: Weeks**

Name	Graph	Min	Mean	Max	95%
Concrete D/S Portal Structure / Distribution		3.91	5.15	6.06	5.87
Concrete the two gate shafts to the top of the dam / Distribution		4.99	6.77	8.14	7.93
Category: Service Spillway					
Access Roads to Spillway and Borrow Pits / Distribution		2.97	4.02	5.03	4.82
Stripping and Excavation / Distribution		5.90	7.03	8.16	7.87
Sheet piling 1500 m2 / Distribution		4.90	6.02	7.08	6.86
Place Granular for Slab & Footings & Mud slab / Distribution		9.56	12.49	14.32	14.03
Secant piling / Distribution		15.11	17.89	20.30	19.79
Ogee, Flip Bucket Concrete / Distribution		16.74	21.10	25.58	24.72
Walls and Slab Concreting / Distribution		24.57	27.56	30.68	30.09
Category: Dam					
Coffer Dam 539,000 m3 / Distribution		8.85	10.03	11.18	10.92
Grouting Program off Coffer Dam / Distribution		4.33	6.29	8.06	7.67
Bentonite Cut-Off Walls Left Abutment, inc. capping / Distribution		19.65	22.57	26.44	25.35
Bentonite Cut-Off Walls Right Abutment, inc. capping / Distribution		9.77	12.60	16.05	15.18

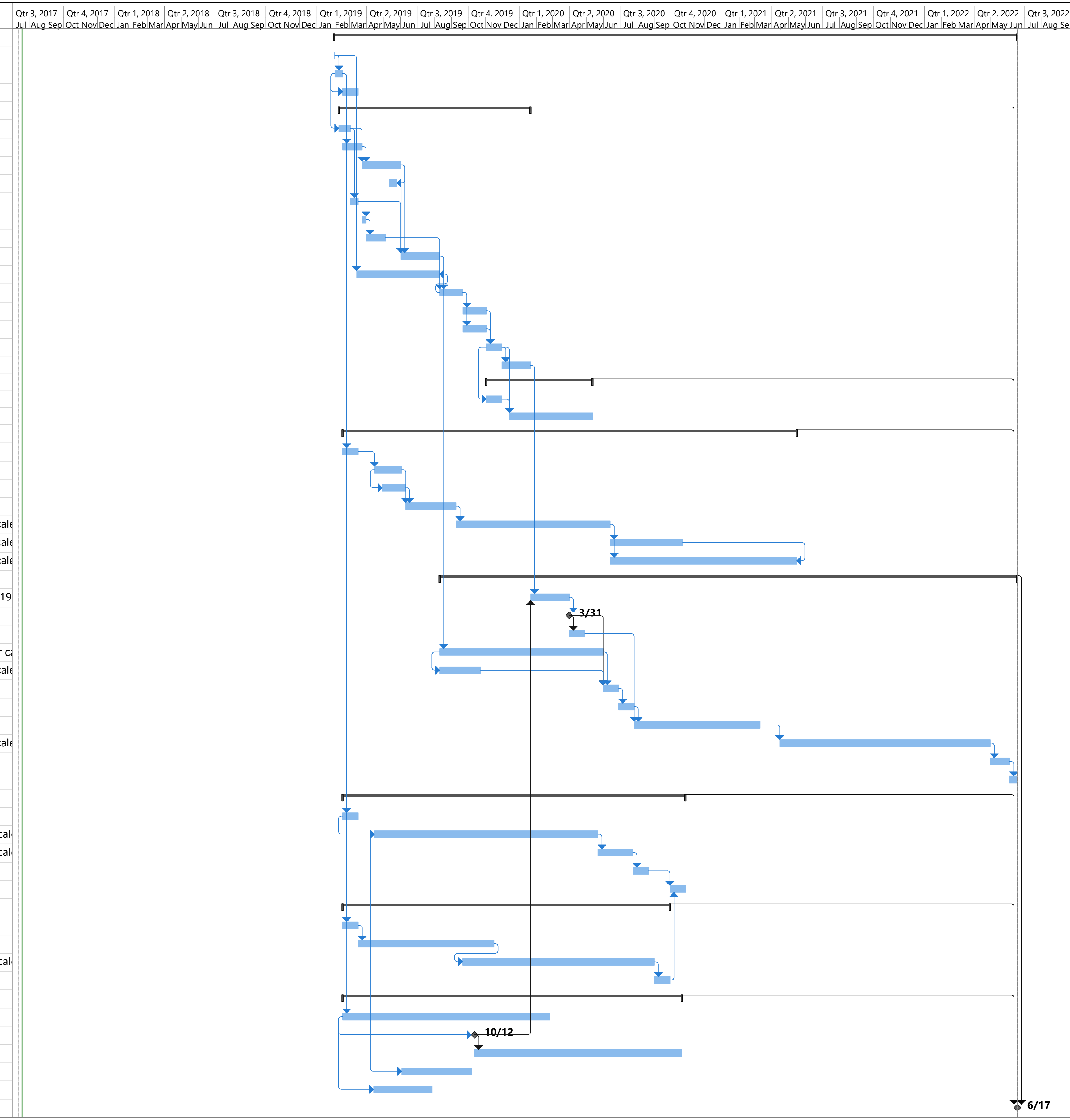
@RISK Input Results **Unit: Weeks**

Name	Graph	Min	Mean	Max	95%
River Channel Rock Trimming / Distribution		3.98	5.15	6.11	5.87
River Channel Rock Grouting / Distribution		4.21	6.29	7.99	7.67
Place Fill to el. 1413.5 (1,417,000 m3) / Distribution		25.93	28.92	31.50	31.04
Balance of Fill to el. 1429 (1,625,000 m3) / Distribution		25.67	28.92	31.56	30.99
Install Hoist Structure& Gates / Distribution		4.93	5.86	7.04	6.78
Guard Rail / Distribution		1.97	2.43	3.02	2.87
Category: Highway Relocation					
Clearing / Distribution		3.96	4.86	6.11	5.76
Earthworks / Distribution		22.15	31.06	35.84	35.07
Pavement / Distribution		6.21	8.10	9.20	9.04
Finishings / Distribution		3.05	4.03	5.06	4.81
Reclaim / Distribution		2.96	4.03	5.09	4.82
Category: Bridge					
Access and Staging Area / Distribution		3.04	4.03	5.08	4.81
Substructure / Distribution		31.80	42.26	53.04	51.07

@RISK Input Results **Unit: Weeks**

Name	Graph	Min	Mean	Max	95%
Superstructure / Distribution		23.78	30.15	36.52	35.55
Finishings / Distribution		2.99	4.31	6.01	5.58
Category: Facilities Relocation					
Build Gooseberry Ranger Station / Distribution		38.51	45.66	52.83	51.77
Demolish Elbow Valley Ranger Station / Distribution		38.12	45.67	53.02	51.78
McLean Creek Store Relocation / Distribution		14.69	20.12	25.37	24.49
McLean Creek Campground Lot Replacement / Distribution		11.99	16.10	20.27	19.53

ID		Task Name	Duration	Start	Finish	Predecessors	Notes
1		**Elbow River Mclean Creek Damsite MC1 Construction- Spring Awa**	**171.86 wks**	**Fri 2/1/19**	**Fri 6/17/22**		
2		Contracts Awarded	0.2 wks	Fri 2/1/19	Sat 2/2/19		
3		Initial Site Preparation	2 wks	Sat 2/2/19	Sat 2/16/19	2	
4		Install Camp, Garage and Laydown	4 wks	Sat 2/16/19	Sat 3/16/19	3SS+2 wks	
5		**Tunnel**	**48 wks**	**Sat 2/9/19**	**Tue 1/21/20**		
6		Excavation& Bolting of U/S Portal	3 wks	Sat 2/9/19	Sat 3/2/19	3SS+1 wk	
7		Dry House, Jumbo Garage, Water, Electricity, Permits, etc.	5 wks	Sat 2/16/19	Sat 3/23/19	3	
8		Tunnelling at U/S Portal	10 wks	Sat 3/23/19	Sat 6/1/19	6,7	
9		Drill & Blast Drop Raises	2 wks	Sat 5/11/19	Sat 5/25/19	8FF-1 wk	
10		Excavation & Bolting of D/S Portal	2 wks	Sat 3/2/19	Sat 3/16/19	6	
11		Erect & Calibrate Shotcrete Batch Plant	1 wk	Sat 3/23/19	Sat 3/30/19	7	
12		Erect & Calibrate Concrete Plant	5 wks	Sat 3/30/19	Sat 5/4/19	11	
13		Tunnelling at D/S Portal	10 wks	Sat 6/1/19	Sat 8/10/19	8,10,9	
14		Tunnel Forms Fabrication and Assembly	5 mons	Wed 3/13/19	Sat 8/10/19	15SF,2	
15		Concrete U/S Half of Tunnels (inc. grouting)	6 wks	Sat 8/10/19	Sat 9/21/19	13,12	
16		Concrete U/S Portal Structure	6 wks	Sat 9/21/19	Sat 11/2/19	15	
17		Concrete D/S Half of Tunnels (inc. grouting)	6 wks	Sat 9/21/19	Sat 11/2/19	15	
18		Concrete D/S Portal Structure	4 wks	Sat 11/2/19	Sat 11/30/19	16,17	
19		Concrete the two gate shafts to the top of the dam	6 wks	Sat 11/30/19	Tue 1/21/20	18	
20		**Fish Passage Tunnel**	**26 wks**	**Sat 11/2/19**	**Tue 5/12/20**		
21		Inlet & Outlet Excavation	4 wks	Sat 11/2/19	Sat 11/30/19	18SS	3wks-5wks
22		Tunnel Excavation, Lining, and Grouting	20 wks	Sat 12/14/19	Tue 5/12/20	18FS+2 wks,21	15wks-25wks
23		**Service Spillway**	**114.23 wks**	**Sat 2/16/19**	**Sat 5/15/21**		
24		Access Roads to Spillway and Borrow Pits	4 wks	Sat 2/16/19	Sat 3/16/19	3	
25		Stripping and Excavation	7 wks	Mon 4/15/19	Sun 6/2/19	24	start on Apr.15 2019
26		Sheet piling 1500 m2	6 wks	Mon 4/29/19	Sun 6/9/19	25SS+2 wks	
27		Place Granular for Slab & Footings & Mudslab	13 wks	Mon 6/10/19	Sun 9/8/19	26,25	
28		Secant piling	18 wks	Mon 9/9/19	Fri 6/12/20	27	frost & wet weather cale
29		Ogee, Flip Bucket Concrete	17 wks	Sat 6/13/20	Wed 10/21/2	28	frost & wet weather cale
30		Walls and Slab Concreting	25 wks	Sat 6/13/20	Sat 5/15/21	29FF,28	frost & wet weather cale
31		**Dam**	**144.66 wks**	**Sat 8/10/19**	**Fri 6/17/22**		
32		Coffer Dam 539,000 m3	10 wks	Tue 1/21/20	Tue 3/31/20	19,56	must finish by Nov 2019
33		Diversion	0 days	Tue 3/31/20	Tue 3/31/20	32	
34		Grouting Program off Coffer Dam	4 wks	Tue 3/31/20	Tue 4/28/20	33	
35		Bentonite Cut-Off Walls Left Abutment, inc. capping	20 wks	Sat 8/10/19	Sun 5/31/20	13	frost & wet weather ca
36		Bentonite Cut-Off Walls Right Abutment, inc. capping	10 wks	Sat 8/10/19	Wed 10/23/1	35SS	frost & wet weather cale
37		River Channel Rock Trimming	4 wks	Sun 5/31/20	Sun 6/28/20	33,35,36	
38		River Channel Rock Grouting	4 wks	Sun 6/28/20	Sun 7/26/20	37	
39		Place Fill to el. 1413.5 (1,417,000 m3)	31 wks	Sun 7/26/20	Wed 3/10/21	38,34	
40		Balance of Fill to el. 1429 (1,625,000 m3)	31 wks	Thu 4/15/21	Fri 4/29/22	39	frost & wet weather cale
41		Install Hoist Structure& Gates	5 wks	Sat 4/30/22	Fri 6/3/22	40	
42		Guard Rail	2 wks	Sat 6/4/22	Fri 6/17/22	41	
43		**Highway Relocation**	**86.94 wks**	**Sat 2/16/19**	**Mon 10/26/2**		
44		Clearing	4 wks	Sat 2/16/19	Sat 3/16/19	3	
45		Earthworks	34 wks	Mon 4/15/19	Thu 5/21/20	44SS	Frost & wet weather cal
46		Pavement	8 wks	Fri 5/22/20	Thu 7/23/20	45	Frost & wet weather cal
47		Finishings	4 wks	Fri 7/24/20	Thu 8/20/20	46	
48		Reclaim	4 wks	Tue 9/29/20	Mon 10/26/2	53,47	
49		**Bridge**	**82.94 wks**	**Sat 2/16/19**	**Mon 9/28/20**		
50		Access and Staging Area	4 wks	Sat 2/16/19	Sat 3/16/19	3	
51		Substructure	35 wks	Sat 3/16/19	Sat 11/16/19	50	
52		Superstructure	26 wks	Sat 9/21/19	Mon 8/31/20	51FS-8 wks	Frost & wet weather cal
53		Finishings	4 wks	Tue 9/1/20	Mon 9/28/20	52	
54		**Facilities Relocation**	**86 wks**	**Sat 2/16/19**	**Tue 10/20/20**		
55		Build Gooseberry Ranger Station	52 wks	Sat 2/16/19	Tue 2/25/20	3	
56		Gooseberry Station Online	0 days	Sat 10/12/19	Sat 10/12/19	55SS+34 wks	
57		Demolish Elbow Valley Ranger Station	52 wks	Sat 10/12/19	Tue 10/20/20	56	
58		McLean Creek Store Relocation	18 wks	Mon 6/3/19	Sun 10/6/19	45SS+7 wks	
59		McLean Creek Campground Lot Replacement	15 wks	Sat 4/13/19	Sat 7/27/19	55SS+8 wks	
60		**Project Construction Completion**	**0 wks**	**Fri 6/17/22**	**Fri 6/17/22**	**5,23,31,43,49,54,20**	



Project: MC1 Spring Award Sch
Date: Tue 7/18/17

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Progress

Manual Progress

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