Appendix F

Conceptual Design of the McLean Creek Flood Storage Site

Southern Alberta Flood Recovery Task Force
Volume 4 – Flood Mitigation Measures

Appendix F – Elbow River Dam at McLean Creek

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1.0 ELBOW RIVER DAM AT McLEAN CREEK

1.1 Concept Description

The Elbow River Dam at McLean Creek (MC1) site was previously identified and investigated for flood mitigation as part of the *1986 Elbow River Floodplain Management Study* by WER Engineering Ltd., IBI Group, and ECOS Engineering. The site is located in the Green Zone on Crown Land approximately 10 km upstream of the Town of Bragg Creek, and immediately upstream of the confluence of McLean Creek with the Elbow River.

This project concept considers building an earth fill dam across the main stem of the Elbow River. It includes a combined concrete outlet/service spillway structure for discharging normal and flood flows, and includes an auxiliary earth cut channel spillway to protect the dam from extreme floods up to the probable maximum flood (PMF) event. The dam site and reservoir area are illustrated in **Drawing F1**.

The proposed earth fill dam (main embankment) traverses a river gorge which is approximately 110 m wide at the base and is steep walled for a height of about 28 m. The left abutment has a high knob-like feature falling away to an undulating plateau more-or-less equal to the height of the main gorge and then rising again to the northwest. The right abutment has a plateau at about the same elevation and then rises again to the southwest. The Kananaskis Country Highway 66 traverses the right abutment. The river valley itself bends sharply to the north-northeast at the dam site, facilitating the construction of an auxiliary earth channel spillway on the right bank. Similarly, the topography and river alignment are well suited for construction of a permanent outlet/spillway structure in the left valley abutment.

The permanent outlet/service spillway is a gated conduit structure with its intake invert located about 21 m above valley bottom. The structure concrete gates would typically be left in the wide open position thereby allowing free passage of river water with minimum reservoir level rise during normal flow conditions (i.e., non-flood). The gates would be strategically closed during flood events thereby holding back a significant portion of the flow in reservoir storage.
The concrete structure also serves as a service spillway designed to pass even more extreme flood events, if they ever occur, thereby protecting the dam from potential overtopping and associated catastrophic failure.

This conceptual design includes a small permanent pool in the valley bottom extending from river bottom elevation 1,379.0 m to the permanent outlet structure intake invert elevation 1,398.0 m, thereby permanently containing approximately 4,000 dam^3 of water as dead storage. This storage is intended to prevent incoming larger bottom sediment from plugging the intake area, and could also replace the previously existing Allen Bill Pond which was destroyed by the 2013 flood. There is no low level outlet to release the dead storage. Additional water could be contained above the dead storage El. 1,398.0 m (i.e., multi-use storage) by regulating the permanent outlet gates using pre-programmed automation methods, rather than leaving the gates in the wide open position as considered herein. The potential value and/or need for multi-use storage at this site should be evaluated as part of the future study.

2.0 HYDROLOGICAL OVERVIEW

2.1 Median and Mean Monthly Flows

Median winter and median annual flows for the Elbow River are approximately 3 and 10 m^3/s, respectively, as recorded at Alberta Environment and Sustainable Resource Development (ESRD) gauging station 05BJ004 (Elbow River at Bragg Creek). Mean monthly flows as recorded at station 05BJ004 are provided in **Table F2.1**.

Table F2.1
Elbow River Mean Monthly Flows
Gauging Station Elbow River at Bragg Creek

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Mean Flow (m^3/s)	3.0	2.9	3.2	4.7	14.6	25.2	13.4	9.3	8.1	6.5	4.7	3.7

The MC1 site is located approximately 10 km upstream of this gauging station, resulting in a 10% reduction in drainage area. The impact of this area's reduction on median and mean monthly flows has not been estimated as a part of this study, but will be much less than 10%.

2.2 Flood Flows

Frequency analysis of flood inflows into Glenmore Reservoir (i.e., 10 km downstream of the MC1 site as discussed herein) which was completed for this study resulted in instantaneous flood peak flow and 7-day flood volume estimates as summarized in **Table F2.2**. These estimates are considered to be representative of the upstream MC1 site (i.e., assumes minimal inflow between the MC1 site and Sarcee Bridge during extreme flood events generated in higher regions of the basin). Background information which provides the basis for flood estimates is documented separately in **Appendix C** of the main report. Estimates of the June 2013 flood instantaneous peak flow and total flood volume entering Glenmore Reservoir are included for comparison in **Table F2.2**.

Table F2.2
Elbow River Instantaneous Flood Peak and Runoff Volume Estimates

Annual Flood Probability (Return Period)	Instantaneous Peak Flow (m^3/s)	7-day Volume dam^3
5% Annual Exceedence Probability (AEP; 1:20-year)	440	83,000
1% AEP (1:100-year)	930	130,000
June 2013 Flood	1,260	154,000
0.2% AEP (1:500-year)	1,625	183,000

As indicated by **Table F2.2,** the June 2013 flood instantaneous peak flow and flood volumes were larger than the estimated 1% AEP flood but smaller than the 0.2% AEP flood. More detailed frequency analysis should be performed as part of future, more detailed design study.

2.3 Probable Maximum Flood

The PMF is defined as the most severe flood that may be reasonably expected to occur at a particular location. The PMF is normally evaluated by deterministic methods that maximize the various factors contributing to the generation of a flood. The probability of such a flood occurring is very rare (e.g., once in a million years).

A PMF hydrograph at Glenmore Reservoir was previously generated by ESRD and is included in the August 1986 *Elbow River Floodplain Management Study* by WER, IBI and ECOS. The PMF entering Glenmore Reservoir was estimated to have a flood peak value of 2,175 m^3/s and a 7-day volume of approximately 464,000 dam^3, which is approximately 3.0 times the volume of the 2013 flood. ESRD cautions:

> "…that these are preliminary estimates of PMF…subject to considerable error and that a detailed assessment….would be required prior to any detailed design."

3.0 GEOLOGICAL AND GEOTECHNICAL OVERVIEW

A preliminary subsurface field investigation was completed as a part of this study as documented in a separate report entitled *Preliminary Geotechnical Investigation Report, Elbow River Dam at McLean Creek* (AMEC, 2014).

In general terms, the Elbow River valley upstream from Bragg Creek is defined by the foothills and by bedrock exposures. Bedrock is of Upper Cretaceous age, consisting of sandstones, siltstones, and mudstones of the Brazeau Formation. Valley bottom materials in the area consist of terraced modern alluviums composed of boulder to cobbly sands and gravels with fine-grained back-water deposits. Materials at higher elevations include colluvial deposits, glacial drift (till), and outwash deposits. The thickness of valley bottom materials overlying the bedrock is likely to be only a few metres. The depth of glacial deposits over adjacent bedrock topography is highly variable. The site rock exposures indicate that thickly bedded sand-stone lies above the more thinly bedded siltstones and mudstones, and that the bedrock is dipping in an east to southeast direction at about 5 to 10 degrees. The right side topography above the edge of the gorge is likely nominally capped with glacial drift materials, the left gorge wall is capped with a substantial amount of glacial drift material.

4.0 FLOOD STORAGE VOLUME

4.1 Background Considerations

Significant residential development located along the Elbow River floodplain downstream of Glenmore Reservoir is at risk during extreme flood events. Pathway closures are required when Glenmore Reservoir flood discharge reaches 40 m^3/s. Modest overbank flooding of undeveloped areas starts at 120 m^3/s discharge. Widespread basement seepage occurs for discharges of 140 m^3/s. First residents are impacted at discharges of 170 m^3/s. Evacuation of residents is initiated at a discharge of 192 m^3/s.

The most recent Glenmore Reservoir storage capacity and flooded area curves which were produced by Klohn Crippen Berger in 2013 are illustrated on **Figure F4.1**. The existing Glenmore Reservoir storage is used to attenuate flood peaks thereby protecting downstream developments. If an extreme flood is forecast, the City of Calgary opens the Glenmore Reservoir low level DOW valves thereby drawing the reservoir down to provide flood storage for the incoming flood. Maximum permissible drawdown is 5 m below FSL El. 1,076.85 m which equates to a flood storage volume of 15,400 dam^3 (KCB Glenmore Bathymetric Survey, 2013). This drawdown could be accomplished in 25 hours at the maximum discharge rate of 170 m^3/s (maximum discharge before significant downstream flood damages start to occur). In reality a portion of this storage should be drawn down well in advance of an actual flood event forecast (e.g., in the spring when significant snow pack exists in the watershed). The 15,400 dam^3 draw down was successfully achieved in anticipation of the June 2013 flood. The City of Calgary needs to use caution when drawing the reservoir down in that if they draw down the Glenmore Reservoir and the forecast flood does not develop they can be left with insufficient water supply.

Bathymetric surveys by Klohn Cripper Berger for the City of Calgary indicate that Glenmore Reservoir may have lost approximately 17% of its storage volume since 1933 as a result of sediment transport into the reservoir. This process is ongoing.

Figure F4.1

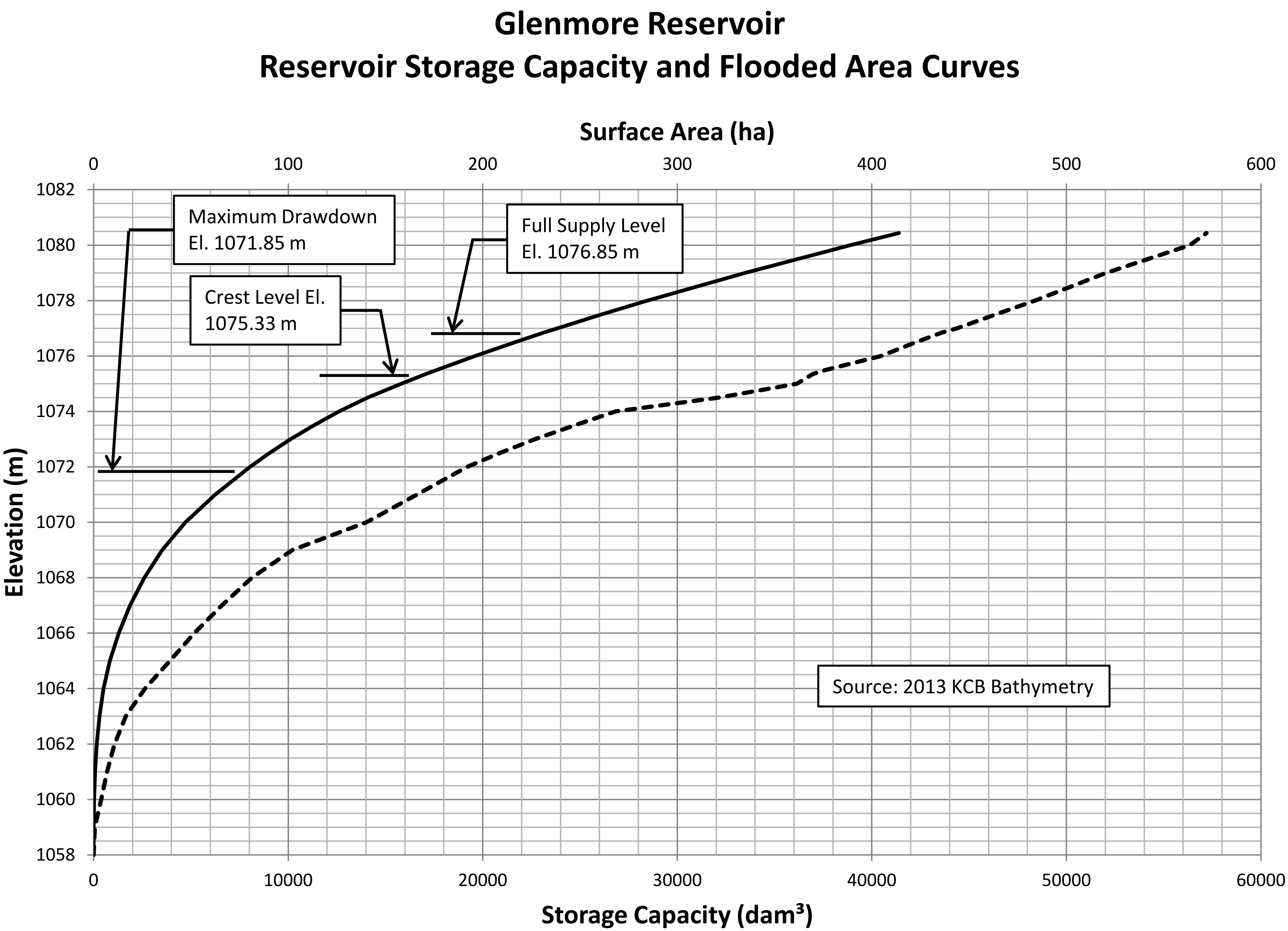

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Table F4.1 provides estimates of the flood volume required to prevent significant damages along the Elbow River downstream of Glenmore Reservoir, considering a continuous discharge of 170 m^3/s from the reservoir for the duration of the flood (i.e., discharge before first downstream residents are impacted by flood water).

Table F4.1
Required Reservoir Flood Storage Volume to Prevent Damages

Return Period (Years)	Minimum Storage Requirement
5% AEP (1:20-year)	16,800
1% AEP (1:100-year)	56,600
June 2013 Flood	83,000
0.2% AEP (1:500-year)	107,500

Based on the data presented in **Table F4.1,** one can conclude that the Glenmore Reservoir flood storage of 15,400 dam^3 is inadequate to prevent discharge from exceeding the 170 m^3/s value for flood events as small as the 5% AEP flood. The level of protection is even poorer if the City is not successful drawing Glenmore Reservoir down to its minimum El. 1,071.85 m prior to flood impact. It is therefore concluded that the existing level of protection to residences downstream of the Glenmore Reservoir is inadequate. That said, Glenmore Reservoir flood storage does provide significant flood peak attenuation and downstream development protection (e.g., as much as full protection for floods just smaller than 5% AEP, and successfully attenuated June 2013 flood inflow peak of 1,260 m^3/s to discharge of approximately 700 m^3/s).

4.2 Flood Protection Design Basis

The current Alberta minimum flood protection design standard is the 1% AEP flood, or alternatively can be based on a historical flood event (e.g., June 2013 flood). Increased protection should be considered based on economic assessment and/or when such an event would result in severe societal impact. As an example, the Red River floodway was originally sized to protect Winnipeg from the 0.2% AEP (1:500-year) flood event. It was later enlarged to provide 0.14% AEP (1:700-year) flood protection. Even greater protection was considered but costs were proven to be prohibitive.

The MC1 concept as presented herein was developed considering the 1% AEP minimum design standard (i.e., total flood storage requirement of 56,600 dam^3). As previously mentioned, Glenmore Reservoir can provide 15,400 dam^3 of that amount. As indicated in **Figure F4.2,** the remaining 41,200 dam^3 flood storage could be provided with MC1 storage reservoir water level of approximately El. 1,422.0 m. To account for operational inefficiencies a slightly higher 1% AEP El. 1,423.0 m has been used. This considers that none of the previously mentioned dead storage can be preleased. The conceptual design provides for a nominal 3.5 m additional storage above the 1% AEP El. 1,423.0 m (i.e., maximum allowable reservoir El. 1,426.5 m) resulting in a combined total flood storage capacity of 73,400 dam^3 (i.e., Glenmore and MC1 combined reservoir storage) prior to activation of the MC1 auxiliary earth channel spillway. Considering the project size presented in this conceptual design, a 2013 magnitude flood would still result in residential damages along the Elbow River floodplain downstream of Glenmore Reservoir, but these damages would be greatly reduced as compared to what was experienced in 2013. The MC1 project could be built to a higher level than investigated herein to provide enhanced flood protection (e.g., full containment for 2013 magnitude flood or larger). Alternatively, additional projects could be constructed to provide enhanced flood protection above that provided herein.

Figure F4.2

Elbow River Dam Site at McLean Creek (MC1)
Reservoir Storage Capacity and Flooded Area Curves

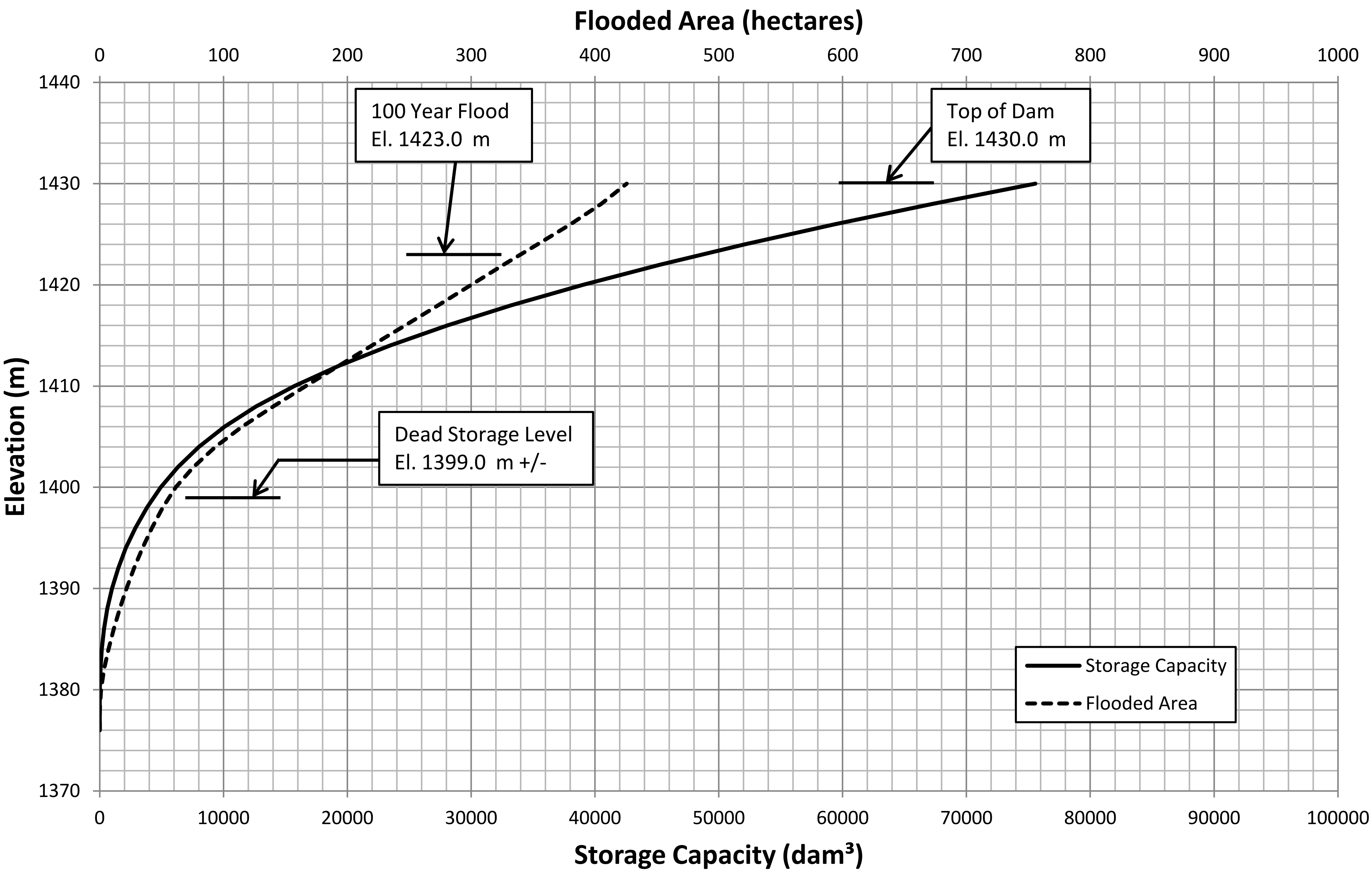

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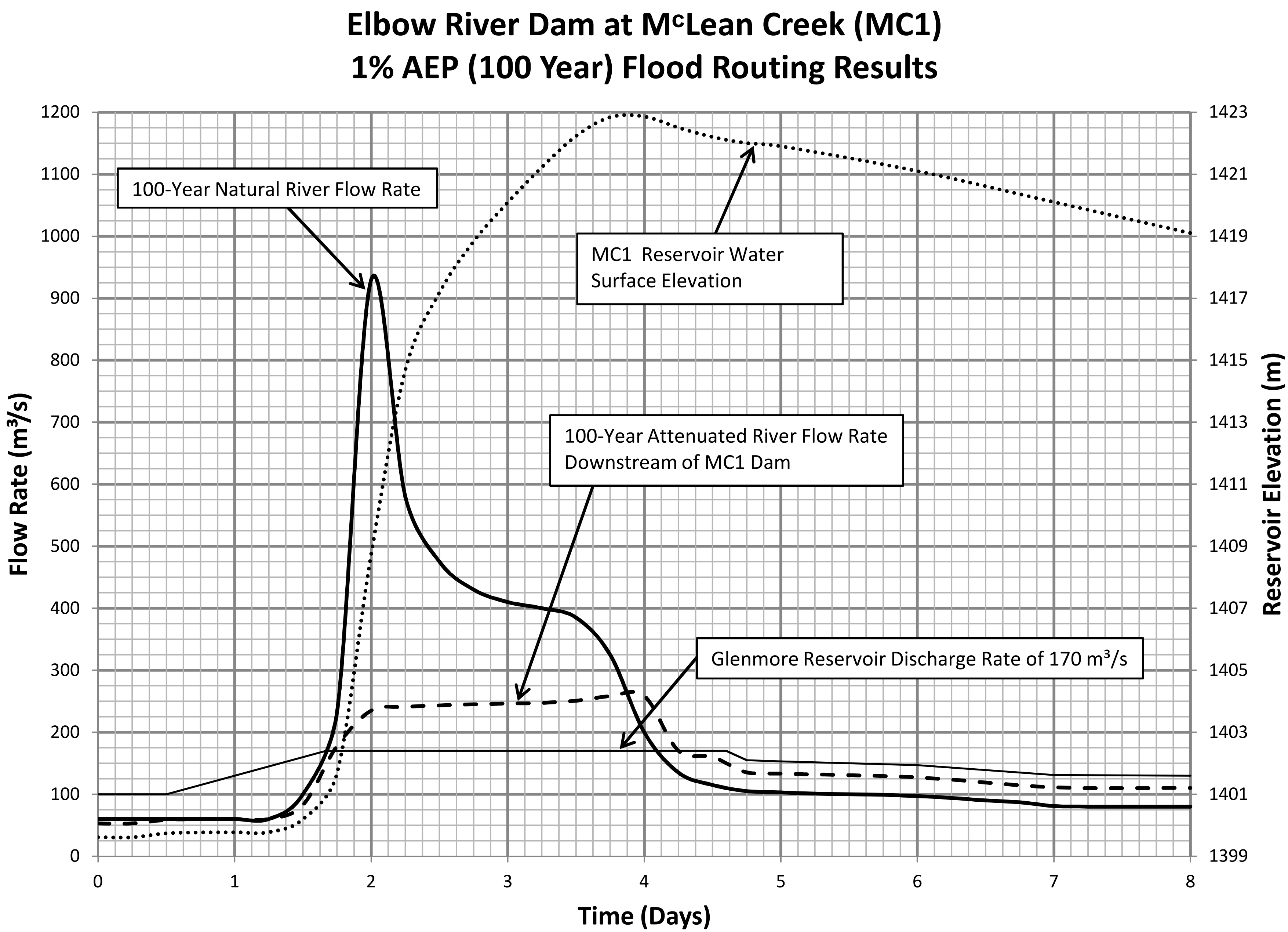
Figure F4.2 area and capacity curves were developed based on contours developed from the Canadian Digital Elevation data (CDED) illustrated on **Drawings F1**. These area and capacity curves should be updated in future design using more accurate LiDAR data.

Figure F4.3 illustrates the potential flood flow reduction benefits of the MC1 and Glenmore Reservoir storage when managing the 1% AEP flood. The figure illustrates that the MC1 peak inflow rate of 930 m^3/s is reduced to a peak discharge of 260 m^3/s downstream of the MC1 reservoir. This resulting 260 m^3/s flow rate is absorbed in Glenmore Reservoir storage. The resulting peak discharge from Glenmore Reservoir is 170 m^3/s; the maximum allowable discharge prior to residential damage.

The following additional observations are made with respect to **Figure F4.3**:

- The inflow hydrograph peaks rise rapidly emphasizing the need for improved flood forecasting methods.
- The MC1 structure gates need to be closed rapidly after the MC1 reservoir has stopped rising (i.e., inflow peak has passed and inflow rate exceeds outflow rate just before 4 days into the event) otherwise Glenmore Reservoir storage will be inadequate
- The above-noted operational considerations support building the project to greater than the 1% AEP return period protection level (i.e., increased volume and diversion rate) and/or constructing additional flood protection projects.

Figure F4.3



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5.0 PROJECT DESIGN

5.1 General

Pertinent structure data established for conceptual design and described in this report section are provided in **Table F5.1**.

Table F5.1
Elbow River Dam at McLean Creek (MC1) Pertinent Structure Data

Flood Storage Reservoir	
Dead Storage Volume	4,000 dam^3
Dead Storage Elevation	1,398.0
100-year Flood Storage Volume	41,200 dam^3
100-year Reservoir Flood Elevation	1,423.0 m
Main Embankment	
Dam Protection Design Flood	Probable Maximum Flood
Top of Dam Elevation	1,430.0 m
Maximum Dam Height	50 m
Probable Maximum Flood Elevation	1429.0 m
Freeboard above Probable Maximum Flood	1.0 m
Combined Permanent Outlet/Service Spillway Structure	
Conduit System	6 side by side openings
Intake Invert Elevation	1398.0 m
Size of Conduits	3.0 m wide x 3.0 m high each
Length of Conduit System	240 m
Gatewell Tower Height	32 m
Size of Gates	6 gates at 2.7 m wide x 3.0 m high
Length of Downstream Chute	160 m
Length of Stilling Basin	28.0 m
Width of Chute and Basin	23.0 m
Service Spillway Design Flood	0.2% AEP Period *
Maximum Service Spillway Outflow	780 m^3/s
Auxiliary Earth Channel Spillway	
Upstream Invert Elevation	1,426.0 m
Bottom Width	100 m
Fuse Plug Crest Elevation	1,426.5 m
Maximum Design Outflow	1,280 m^3/s

* Prior to activation of Auxiliary Earth Channel spillway.

5.2 Storage Dam and Reservoir

5.2.1 General

Drawing F2 illustrates the conceptual design for the main dam embankment, given the estimated geotechnical conditions, the perceived available construction materials, and the configuration of the valley cross-section at this site. The proposed 50 m high dam section considers a 10 m top width, an interior impervious core and outer random earth fill shells. Embankment slopes are estimated at 3H:1V with 10 m wide berms at 10 m vertical intervals resulting in average dam slopes of 4H:1V. As compared to a simple 4H:1V slope the berms provide the advantages of facilitating inspection, maintenance, and access to geotechnical instruments. They can also be used to build a temporary ditch system to facilitate surface water management following construction until a good grass cover is established. The need, width and spacing of such berms should be further evaluated as part of future design. The conceptual design includes an interior sand filter and drainage system, and upstream rock riprap slope protection. Rock riprap has been provided in the lower active reservoir zone (i.e., up to El. 1405.0 m). It is also provided in the top 10 m zone to protect the dam from potential failure considering the unlikely event of a PMF combined with a 50% AEP wind event. This upper zone riprap can be covered with topsoil and seeded to provide a more desirable landscape appearance. Consideration should also be given to using a more erosion-resistent impervious 1A zone material in the upstream shell/upstream dam surface to reduce flood storage wave damage risk. The extent of these features will be better established as part of more detailed future design.

During construction flood risk is always a major concern particularly when building a dam upstream of a major population centre. Allowance has been made in the cost estimate to incorporate cofferdams and other works (e.g., temporary low level conduits and additional emergency earth cut spillways) to protect a partially constructred dam from overtopping which could result in dam failure and associated catastrophic downstream damages within the City of Calgary. Finite details cannot be established until more detailed geotechnical information has been obtained.

McLean Creek presently flows into the Elbow River directly below the downstream toe of the dam; consequently it would have to be re-routed through a low height of ground such that the flow is directed away from the toe of the main embankment. The extent of relocation would be minimized by providing riprap protection at the embankment toe in this area.

5.2.2 Geotechnical Design Details

The dam embankment has been divided into three sections based on topography and subsurface conditions, namely:

- The valley section of dam in the Elbow River gorge, which is the highest section of the dam at approximately 50 m, and extends from a steep wall with bedrock exposure on the left abutment to a shallower soil slope on the right abutment area.
- The left section of the dam, which extends to the left from the river gorge along an upland area to meet with higher ground about 700 m from the Elbow River. The dam height through the left section is roughly 20 to 25 m, with variations according to the local topography.

- The right section of the dam, which extends to the right from the river gorge approximately southeast along an upland area to meet higher ground about 1,400 m south of the Elbow River. The dam height through the right section is roughly 20 m, also with variations according to local topography.

Valley Section

Subgrade conditions in the valley section include: exposed heavily jointed bedrock overlain with clay till at the left abutment, fluvial sand and gravel as well as silt and sand floodplain deposits in the valley floor and silty sandy floodplain deposits at the right abutment. Extensive excavation into the left abutment will be necessary to establish a secure interface between the constructed embankment and the native bedrock and clay till. Along the valley bottom, excavation of a core trench through the granular fluvial material to expose the underlying bedrock will be required to reduce seepage and piping potential below the impervious core of the dam. Based on the heavily jointed structure of the bedrock exposed at the left side of the gorge, it is expected that a grout curtain will be required below the impervious core of the dam. The borehole drilled approximately 150 m to the right of the existing river channel encountered approximately 6 m of sand and silt overlying the bedrock, whereas approximately 300 m farther to the right (SE) clay till was encountered from ground surface to approximately 10 m depth. In order to minimize potential for piping beneath the dam, the core trench should be excavated through surficial sand and silt deposits, and extend to the right to join with the clay till.

Based on the materials encountered in the boreholes drilled along the proposed left and right embankment areas, the locally available low to medium plastic clay till soil is suitable for constructing both impervious 1A fill, and random 2A fill. Excavated bedrock would be suitable for random 2A fill, provided particle size could be managed to accommodate controlled compaction. Since the majority of excavation in bedrock would be limited to the interface of the embankment fill with the left abutment, it is expected that the volume of bedrock excavation will be small relative to the volume of borrow required for construction of the embankment. Borrow areas have yet to be identified, but it appears that a sufficient quantity of clay till, similar to that encountered in the boreholes, is available in the area. Haulage distances will depend on whether borrow is sourced within the future inundation area and flood storage area, or from alternative sites.

Excavation of the clay till soil can be undertaken with conventional equipment such as loaders, hydraulic excavators and scrapers. The same equipment can be used to excavate the typically sandy, silty floodplain deposits, with consideration that transport of such materials is limited especially where heavy wheeled equipment is involved. The exposed sandstone on the left side of the gorge is moderately strong but heavily jointed. Excavation of weathered jointed sandstone and siltstone is likely possible with large hydraulic excavators equipped with rock teeth and/or hydraulic breakers, and also possibly with large dozers and rippers. However, for less weathered bedrock having more widely spaced joints, drilling and blasting could be required. Additional drilling, with coring of the bedrock, during the detailed stage of design will provide information regarding the need for drill and blast techniques. Even unweathered mudstone, and weaker components of the siltstone and sandstone, can normally be excavated with large hydraulic excavators and dozers equipped with rippers.

Embankment slope angles of 4H:1V average (including benches) for slopes formed of random 2A fill will provide a minimum factor of safety of greater than 1.5 against slope instability for the approximately 50 m height of the valley section of the embankment – for the condition of a permanent upstream pool. Assessment of a rapid drawdown condition from the permanent pool dead storage water elevation 1,398.0 m was not conducted since there is no outlet for the permanent pool. A factor of safety against slope instability of approximately 1.4 was calculated for the valley section of the embankment, assuming: overall 4H:1V slopes including benches and saturated soil conditions below the 1% AEP floodwater elevation. The assumption regarding embankment saturation is considered conservative, since the flood waters would be unlikely to remain against the embankment long enough to establish more than the initial transient stages of seepage through the embankment.

Previous experience with similar bedrock foundation subgrades indicates that subgrade deformations or increase in porewater pressure due to embankment construction should not be limiting factors for typical rates of embankment construction. However, porewater pressures in the foundation bedrock should be monitored during construction of the embankment.

Left and Right Sections

Stripping will be required to remove organic soil overlying the clay till. At some locations stripping of pockets of loose silt or sand from areas underlying the impervious core may also be required. Based on the boreholes drilled at the site, the subgrade conditions underlying the left and right sections of the embankment consist primarily of stiff to hard clay till. At two borehole locations along the right embankment area the clay till was underlain by a layer of gravel and sand at a depth of approximately 10 m. During the detailed design additional site drilling should be undertaken to determine the extent of the gravel layer and whether it approaches ground surface farther upstream within the reservoir area.

As discussed above for the Valley Section of the embankment, the locally available low to medium plastic clay till soil is suitable for constructing both impervious fill, and random fill. Excavated bedrock would be suitable for random fill, provided particle size could be managed to accommodate controlled compaction. Borrow areas have yet to be identified, but it appears that a sufficient quantity of clay till, similar to that encountered in the boreholes, is available in the area. Haulage distances will depend on whether borrow is sourced within the future inundation area and flood storage area, or from alternative sites.

Excavation of the clay till soil can be undertaken with conventional equipment such as loaders, hydraulic excavators and scrapers.

Embankment slope angles of 4H:1V average (including benches) for slopes formed of random 2A fill will provide a minimum factor of safety of approximately 2.3 against slope instability for the approximate 20 m to 25 m height of the left and right embankment sections – for the condition of a permanent upstream pool at elevation of 1,399 m. A factor of safety against slope instability of approximately 1.4 was calculated for the left and right sections of the embankment, assuming: overall 4H:1V slopes, including benches, and saturated soil conditions below the 100-year floodwater elevation. The assumption regarding embankment saturation is considered conservative, since the flood waters would be unlikely to remain against the embankment long enough to establish more than the initial transient stages of seepage through the embankment.

Soil moisture contents measured in the clay till soils that will form the foundation subgrade for the left and right embankment sections were generally near the plastic limit for the soil. Previous experience with similar clay till foundation subgrades indicates that subgrade deformations or increase in porewater pressure due to embankment construction should not be limiting factors for typical rates of embankment construction.

5.2.3 Construction Material Sources

As already discussed, materials suitable for the impervious core and random shell sections of the zoned embankment fill are expected to come from necessary excavations and nearby borrow sources. It is currently estimated that abundant valley bottom granular materials would be available for construction. The most easily exploited deposit appears to be on the left side of the river downstream of the site. Granular materials required for dam filters, drains, and rock riprap bedding would need to be processed. Pitrun gradation may be suitable for pervious fill zones and structure backfill. Rock riprap and cobble armour protection would need to be brought in from off-site sources.

5.3 Permanent Outlet/Service Spillway Structure

A combined permanent outlet/service spillway structure is proposed in the left abutment as illustrated in **Drawing F2**. This reinforced concrete structure consists of a six bay gated conduit system constructed within the main embankment on the left abutment plateau, and discharging into a concrete chute which terminates in a hydraulic jump stilling basin. The structure concept is illustrated in **Drawing F3**. A gatewell would be provided just upstream of the dam centerline, which would be equipped with heavy duty cast iron sluice gates, one for each of the six conduit bays. The proposed gates are standard pre-engineered products. They are robust and low maintenance. Gate system control buildings and controls automation have been allowed for in the design and cost estimate. Structure details are provided in **Drawing F4**.

The 0.2% AEP design event was selected for the service spillway design flood. The permanent outlet/spillway structure is designed to pass this flood with minimal to no damage at the project site, and prior to operation of the auxiliary earth channel spillway. The two spillways (service and auxiliary) can pass larger floods up to the PMF, but significant damage in the way of erosion along the auxiliary spillway flow path is anticipated should such an event occur. The “no damage” service spillway design flood is typically selected to be between the 0.2% AEP event and 0.05% AEP event considering factors including cost and the extent of potential damages should a larger flood ever occur. The 0.2% AEP event was initially selected for this conceptual design considering that bedrock, or other relatively erosion-resistant materials, could be present in the area of the proposed auxiliary earth channel spillway, and that the additional cost to upgrade the permanent spillway to manage a larger flood would be significant. The spillway system design and structure design flood event were further reviewed after subsurface soils information was obtained late in the conceptual design process. The results of those investigations are addressed in **Section 8.2**.

The combined permanent outlet/spillway structure provides several distinct advantages as compared to the separate low level outlet and service spillway structures previously proposed in the 1986 study report. The advantages include:

1. Upstream reservoir level fluctuations would be much less as the combined structure has six conduit bays rather than the previously proposed two low level outlet conduit bays. The permanent summer pond levels could be maintained between El. 1398.5 and 1400.0 m for up to approximately the 10% AEP flood event. This is a significant advantage over the previously proposed system (1986 study report) which would result in significant annual reservoir surface level fluctuation.
2. Winter operation could consider closing five of the six sluice gates, and including a 3 m high weir gate in the sixth gatewell tower. This weir gate would hold the winter pond level just above El. 1401.0 m without gate regulation, keeping the conduit inlet structure submerged thereby avoiding potential ice buildup issues.
3. The combined structure cost is significantly lower than the cost of two separate structures.
4. Gate control on six bays will provide significant additional flood protection for floods larger than the 1% AEP flood (i.e., the flood of 2013). Gate operating rules would be pre-conceived. Simple operations would be devised considering potential variations of extreme floods.
5. The Elbow River has significant amounts of larger bottom sediment (e.g., cobble size and larger) which is transported along the channel bottom as a result of the relatively steep river gradient and associated high water velocities which occur during extreme floods. The bottom sediment will naturally stop moving when it enters the now proposed dead storage area as a result of the water velocity rapidly reducing to near zero in this area.

Excavation for the outlet/spillway structure will range from approximately 10 to 12 m depth for the multiple conduit section, to between about 5 and 18 m for the outlet chute section. It is expected that the majority of the excavation will encounter stiff to very stiff clay till. However, bedrock and fluvial deposits will likely be present in the excavation for the outlet chute near the Elbow River. The local clay till soil will provide stable subgrade support for the conduit, and for the chute foundation. The clay till is also suitable for construction of impervious backfill around headwalls, cutoff walls and side walls for the chute.

As an option to the combined concrete conduit outlet/spillway concept presented herein, there is potential at this site for an outlet structure to be tunnelled in the left abutment. Based on the information in hand, the conduit system is estimated to be better suited to the perceived site conditions and project requirements. This can be further evaluated as part of future study.

5.4 Auxiliary Earth Channel Spillway

The auxiliary spillway is envisioned to be an earth cut channel which would connect into an upland area from where extreme flood water would make its way to McLean Creek, thereby bypassing the dam site. The spillway would have a 100 m bottom width, with 3 horizontal to 1 vertical side slopes, and an upstream invert El. 1426.0 m. A small 0.5 m high fuse plug is included at its upstream end. The spillway channel invert should be founded in a relatively hard erosion resistant material (e.g., bedrock or stiff clay till) to ensure its integrity during an extreme flood event. The combined permanent outlet/spillway structure has been sized to manage all floods up to the 0.2% AEP event, prior to activation of the auxiliary spillway. The probability of this earth channel spillway structure being activated is therefore very low.

The available subsurface information near the proposed auxiliary spillway location indicates that for a channel bed elevation of approximately 1,426 m, the channel invert and sideslopes would be excavated primarily in clay till soil. Under no-flow conditions the sideslopes of the channel would be stable at a design sideslope angle of 3H:1V. Significant erosion through the channel area and downstream McLean Creek would occur during an extreme flood event which activates the auxiliary earth channel spillway. As previously noted the probability of this channel being activated is very low. Additional geotechnical investigations are required to better establish design requirements and a preferred location for this spillway.

5.5 Reservoir Operations

A summary of proposed gate operations, and resulting reservoir water levels, and retained water volume, considering both normal summer and winter flow, and flood conditions is provided in **Table F5.2**. The table data indicates that considering the 1% AEP flood event results in a peak MC1 outflow of 260 m^3/s. This flow can be safely passed through Bragg Creek as it exists (i.e., without dykes). Similarly, this discharge can be further reduced to a peak flow-rate of 170 m^3/s downstream of Glenmore Reservoir by using available flood storage which can be made available at Glenmore Reservoir. This 170 m^3/s flow rate has been established as the no damage flow rate for the Elbow River downstream of Glenmore Reservoir. The table similarly indicates significant flood reduction for the 0.2% AEP flood event (i.e., 1,625 m^3/s inflow reduced to 636 m^3/s outflow from MC1). The dam offers little protection against the PMF should such an event occur, because the flood volume is very large (i.e., inflow peak of 2,175 m^3/s reduced to outflow peak of 2,060 m^3/s). The volume and flow rate estimates provided in **Table F5.2** are based on preliminary flood hydrograph and reservoir capacity estimates. These estimates will be updated after a more detailed hydrologic assessment is completed as part of the future design.

Table F5.2
Elbow River Dam at McLean Creek (MC1)
Pertinent Operations Data

Description (Peak Values)	Summer	Winter	Floods			
	July Mean	January Mean	20-year	100-year	500-year	PMF
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)	1,399.0	1,401.5	1,407.0	1,423.0	1,426.5	1,429.0
Total Contained Water Volume (dam^3)	4,000	5,000	12,000	47,000	62,000	72,000

The levels provided in this table are based on gate operations as follows:

1. All sluice gates wide open for normal summer flows.
2. 5 sluice gates closed and 1 weir gate in place for winter flows.
3. 4 of 6 sluice gates strategically closed if flood flow causes reservoir to rise to El. 1404.0 m and reservoir is still rising (i.e. >5% AEP event).
4. Strategically start reopening gates if reservoir reaches El. 1423.0 m (1% AEP event) and is still rising.
5. Rapidly reclose gates after MC1 reservoir level stops rising (i.e., inflow peak has passed and inflow rate exceeds outflow rate).

6.0 EXISTING INFRASTRUCTURE IMPACTS

The proposed project is located within the Green Zone and is located entirely on Crown Land. Highway 66 and numerous existing recreational facilities will be impacted by the proposed project.

The resulting reservoir will inundate a portion of existing Kananaskis Highway 66 including a bridge crossing of the Elbow River. A potential highway and bridge relocation route around the south side of the reservoir is illustrated on **Drawing F1**. Additional study is required to establish a preferred route. It may be desirable to retain a portion of the existing Highway 66 to provide access from the west, to existing and/or new facilities along the north side of the reservoir impoundment area.

The dam and reservoir area is characterized by fairly intensive recreational use, including day use and extended activities, covering all four seasons. The existing recreational facilities' locations are illustrated on **Drawing F1** and are discussed below:

- The Paddy's Flat recreational area borders the Elbow River on the north side bank and is adjacent to the flood plain. There are two campgrounds within this area, the first is a group camping facility while the second offers public camping for both tent and trailers.
  The campgrounds offer standard serviced campsites with water, vault toilets, fire pits, and tables. Paddy's Flat is a seasonal use site only (May to October) with a total of 98 public campsites. The campgrounds are above the 1% AEP flood level; however, some impacts are anticipated as a result of the Highway 66 relocation.
- River Cove is a group camping facility only. The facility is on the north side, adjacent to the Elbow River within the flood area, and features the usual picnic tables, water, fire pits, and vault toilets. Relocation or removal would be required.
- Allen Bill Pond was a combination hiking trailhead and day use picnic site located on the north side of the Elbow River, and south of existing Highway 66 immediately upstream of the Elbow River Bridge. The pond was stocked with rainbow trout and was a popular fishing site. This pond was destroyed during the 2013 flood. The proposed McLean Creek dam site permanent pond dead storage could serve similar recreational purposes.
- Station Flats is a hiking and horseback trailhead. Located on the north side of Highway 66, there is a small gravelled parking lot and vault toilets. Highway 66 provided access to this area. That access from the east will no longer exist.

- The Elbow Ranger Station is located on the north side of Highway 66 along Ranger Creek, and these facilities would be affected. The existing facilities include a large maintenance compound, a station office building which houses three departments (Alberta Forestry Services, Alberta Parks and Recreation, Alberta Fish and Wildlife), a dining hall, 8 seasonal bunk houses, 11 permanent residences, 2 mobile homes, and 1 cold compound storage building. It is not known to what extent these facilities are currently used, if at all. Requirements would need to be established and the station relocated or dismantled.

7.0 ENVIRONMENTAL AND REGULATORY OVERVIEW

The proposed project is located within the Green Zone and is on Crown land. Project components would directly affect the Elbow River and its associated riparian land. Environmental concerns to be addressed in the project design include:

- Hydrogeology – effects of ponded water on groundwater resources, including aquifers.
- Water quality and quantity – effects of potential changes in upstream (ponded water) and downstream flows, sediment load, and water quality parameters.
- Fisheries – potential for effects on fish and fish habitat, including possible populations of brook trout, brown trout, bull trout, cutthroat trout, longnose dace, mountain whitefish, rainbow trout, and white sucker. Bull trout and native, genetically pure westslope cutthroat trout are listed as “species of special concern” and “threatened” by Alberta’s Endangered Species Conservation Committee, respectively. Populations of native, genetically pure westslope cutthroat trout are also listed federally under the *Species at Risk Act* (SARA). While westslope cutthroat trout populations that are genetically pure occur in part so Canyon, Silvester, and Prairie creeks, which are tributaries of the Elbow River upstream of the McLean creek confluence, no native strains of westslope cutthroat trout have been reported in the Elbow River.
- Soils – effects of ponding and changes in flows on soils and potential for soil erosion.
- Wildlife – provincially designated Key Wildlife and Biodiversity zones are located along the Elbow River, which impose potential timing and construction constraints for the proposed project. Potential effects may occur to species using the zone, including grizzly bear, harlequin duck, and wolverine. Grizzly bear are listed as “at risk” provincially and as “special concern” under COSEWIC federally. Harlequin Duck are “sensitive” provincially. Wolverine are listed as “may be at risk” provincially and “special concern” federally under COSEWIC. None of these three species are listed under SARA. Wildlife habitat and movement patterns may be altered in proximity to the project.
- Vegetation – potential effects on vegetation will include rare non-vascular plants, which were reported in the 1960s in this area and have buffer areas around known locations. These buffer areas overlap the proposed project location.
- Traditional and non-traditional land use – potential effects include access; changes in recreational use as the Elbow River Provincial Recreational Area overlaps the proposed project location; changes in traffic patterns; and aesthetic concerns. The potential project site may be located within the Stoney Nakoda and Tsuu T’ina First Nations traditional territories.

The proposed project would require a license under the *Water Act*, which is administered by the ESRD. The project triggers Alberta Regulation 111/93 EPEA Environmental Assessment (Mandatory and Exempted Activities) Regulation, which requires an environmental impact assessment (EIA) be completed for a dam greater than 15 m in height. A water management project that requires an EIA triggers a Natural Resources Conservation Board (NRCB) review. Typically environmental studies to support the EIA would include a minimum of 1 year of site-specific data.

The proponent would submit its project application with its supporting EIA to ESRD, which makes a determination of completeness. Once deemed complete, the NRCB review process would involve a public hearing as part of its review. The NRCB and ESRD have a history of working cooperatively on environmental reviews of this kind. The ESRD/NRCB process could take between 18 to 24 months to complete. At the completion of the process, the NRCB sends its determination to cabinet, which reviews the report and issues its final approval decision.

In addition to the ESRD and NRCB, several other provincial and federal departments will have regulatory roles for the proposed project. These processes can generally occur in parallel with the ESRD/NRCB review, as much of the information required for them supports the environmental review. For example, pre-development and post-development aquatic environmental assessments would be necessary as part of the application for approval under the *Water Act*. Specific authorizations and permits would be obtained subsequent to the ESRD/NRCB decision, if the project was approved.

An overview of the regulatory process is shown in **Table F7.1**.

Table F7.1
Regulatory Process Overview

Regulator	Legislation	Requirements/Process	Schedule
Provincial			
ESRD	EPEA Environmental Assessment Mandatory and Exempted Activities Regulation 111/93	Under EPEA an EIA is required for a dam greater than 15 m in height, as specified in the Mandatory and Exempted Activities Regulation.	18 to 24 months (with data collection and surveys 30 to 36 months)
NRCB	*Natural Resources Conservation Board Act*	The NRCB review process is triggered when a water management project requires an EIA.	
ESRD	*Alberta Water Act*	Authorization	Variable
	Alberta Water Act	Licence and approval	Variable
	Public Lands Act	Dispositions following the Environmental Field Report (EFR) process	5-8 months
Alberta Culture (AC)	*Historical Resources Act*	Application for clearance	Depends on requirements; for historic resources impact assessment, expect 4 to 6 months from initial application for clearance.
Federal			
Fisheries and Oceans Canada (DFO)		Authorization pursuant to the *Fisheries Act* (habitat and fish passage)	90 days post-filing, providing submission is complete.
Miscellaneous Federal Acts		*Migratory Birds Convention Act* (MBCA)	
		Species at Risk Act (SARA)	*n/a*

As currently conceptualised, the proposed project is not listed in the *Regulations Designating Physical Activities*, under the *Canadian Environmental Assessment Act* (CEAA). It does not result in a reservoir with a surface area that would exceed the annual mean surface area of a water body by 1,500 ha or more.

8.0 CONSTRUCTION COST ESTIMATE AND PROJECT SCHEDULE

8.1 Project Cost Estimate

A detailed cost estimate is provided in **Table F8.1**. The project cost is estimated to be $239,581,000. The estimate provided herein is based on 2012 construction price data. Year 2012 prices were used considering that 2013 construction prices are skewed as a result of abnormal activity which resulted from the June 2013 flood event. It is assumed that the construction of MC1 would take place in a more competitive environment for contractors and suppliers, and as such the 2012 prices are considered indicative of realistic project cost. Additional subsurface soils investigations are required to better establish the concept details presented herein. More detailed hydrological assessment and topographic data are required to better establish the size of required works. A contingency allowance of 25% has been included in an effort to account for additional costs which could result from future additional information and the results of more detailed design work. No allowance is included for escalation until the time of construction.

To increase the flood protection above the 1% AEP, to the 2013 flood-of-record level, would require the dam crest level raised by approximately 4 m to El. 1,434.0 m, and would result in an additional cost of approximately $55 million. This amount includes contingency and engineering allowances.

8.2 Geotechnical Investigation Cost Allowances

The results of geotechnical investigations completed near the end of this conceptual design process indicated that the auxiliary spillway area consists of clay till soils. Based on this limited information, these soils are considered suitable for auxiliary earth channel spillway design but could be less erosion resistant than what was assumed for the conceptual designs presented herein. The potential consequences of these geotechnical results could include the need for a higher design standard for the service spillway (e.g., 0.1% flood passage rather than 0.2% flood passage prior to activation of the auxiliary spillway channel) and additional protection works within the auxiliary earth channel spillway. Additional nominal allowances of $18 million and $9 million were therefore included in the cost estimate for potential modifications to the service spillway structure and auxiliary earth channel spillway designs presented herein, respectively, should they be required. The amounts allow for a larger service spillway structure (i.e., more conduits) than presented in the conceptual design drawings and the inclusion of a roller compacted concrete weir drop to manage potential erosion within the auxiliary earth channel spillway. Although it has not yet been proven that these features are required, it is considered prudent to allow for them in the cost estimate at this time.

Table F8.1
Elbow River Dam at McLean Creek (MC1) Cost Estimate

Item	Unit	Quantity	Unit Price	Extension
General				
Mob./Demobilization	lump sum	1	$10,000,000.00	$10,000,000
Care of Water	lump sum	1	$8,000,000.00	$8,000,000
Clearing & Timber Salvage	hectares	60	$12,000.00	$720,000
Haul Roads	km	10	$300,000.00	$3,000,000
Power Line Relocation	lump sum	lump sum	$400,000.00	$400,000
Ranger Station Removal	lump sum	lump sum	$1,200,000.00	$1,200,000
Topsoil/Seeding etc.	m^2	1,200,000	$1.50	$1,800,000
	Subtotal General			**$25,120,000**
Main Dam Embankment				
Stripping	m^3	200,000	$6.00	$1,200,000
Rock Excavation	m^3	20,000	$20.00	$400,000
Common Excavation	m^3	20,000	$5.50	$110,000
Borrow Excavation	m^3	3,900,000	$5.50	$21,450,000
Overhaul	m^3km	3,900,000	$1.50	$5,850,000
Impervious Fill	m^3	1,800,000	$1.50	$2,700,000
Random Fill	m^3	1,700,000	$1.40	$2,380,000
Fine Filter	m^3	152,000	$80.00	$12,160,000
Coarse Filter	m^3	19,000	$80.00	$1,520,000
Pitrun Gravel	m^3	120,000	$20.00	$2,400,000
Rock Riprap	m^3	38,000	$130.00	$4,940,000
Bedding Gravel	m^3	19,000	$60.00	$1,140,000
Geotechnical Instruments	lump sum	1	$800,000.00	$800,000
Grout Curtain	lump sum	1	$2,000,000.00	$2,000,000
	Subtotal Main Dam			**$59,050,000**

Combined Outlet/Service Spillway Structure				
Stripping	m^3	7,200	$6.00	$43,200
Common Excavation	m^3	600,000	$5.50	$3,300,000
Structure Fill	m^3	20,000	$30.00	$600,000
Reinforced Concrete	m^3	25,000	$1,000.00	$25,800,000
Fine Filter	m^3	2,700	$90.00	$243,000
Coarse Filter	m^3	1,900	$90.00	$171,000
Piping System	lump sum	1	$400,000.00	$400,000
Rock Riprap	m^3	1,900	$130.00	$247,000
Bedding Gravel	m^3	600	$70.00	$42,000
Gate/Hoist Systems	each	6	$560,000.00	$3,360,000
Superstructure	lump sum	lump sum	$90,000.00	$90,000
Controls/Instrumentation	lump sum	lump sum	$300,000.00	$300,000
Electrical/Mechanical	lump sum	lump sum	$500,000.00	$500,000
	Subtotal Structure			**$34,296,000**
Auxiliary Earth Channel Spilllway				
Stripping	m^3	7,200	$6.00	$43,000
Common Excavation	m^3	100,000	$6.00	$600,000
Fuse Plug System	m^3	200	$60.00	$12,000
	Subtotal Auxiliary Spillway			**$655,000**
Highway 66 Relocation				
Grading	km	8	$600,000.00	$4,800,000
Base/Pavement	km	8	$750,000.00	$6,000,000
Elbow River Bridge	lump sum	lump sum	$4,000,000.00	$4,000,000
Mclean Creek Crossing	lump sum	lump sum	$800,000.00	$800,000
	Subtotal Highway 66			**$15,600,000**
Spillway System Allowances Considering May 2014 Geotechnical Investigations				
Service Spillway	lump sum	lump sum	$16,000,000	$16,000,000
Auxiliary Spillway	lump sum	lump sum	$9,000,000	$9,000,000
	Subtotal Spillway Design Upgrader			**$25,000,000**
	SUBTOTAL CONSTRUCTION			**$159,721,000**
	-Contingencies (25%)			$39,930,000
	Subtotal Construction and Contingencies			**$199,651,000**
	-Engineering/Environmental (20%)			$39,930,000
	TOTAL CONSTRUCTION			**$239,581,000**

8.3 Project Schedule and Contracts

Studies to date indicate that the proposed project is feasible. A potential project schedule moving forward would consider both preliminary engineering and environmental impact assessment proceeding on parallel but linked paths, and followed by a detailed design–build or a detailed design-bid-build process.

A number of issues need to be resolved in order to proceed with preliminary design and environmental impact assessment. These include:

- Establishing the level of flood protection to be provided by the project (e.g. 1% AEP flood, 2013 record flood, or larger); and
- Establishing the need for and amount of dead storage and/or multi-use storage, if any.

Stakeholder involvement is required to better define project issues and potential solutions. Initiating stakeholder involvement and gaining land access need to be initial priorities.

Design-build or design-bid-build contracting procedures can be considered for project detailed design and construction. Design-build considers that the work is both designed and built by one project team. Design-bid-build considers that a team is selected to design the project, it then goes to public tender, and is constructed by the successful bidder. Design-build process can result in a reduced time schedule, but the design-bid-build process is considered to be more conventional and appropriate for this project type. The MC1 project could be tendered as one major construction contract, or alternatively divided into two or more contracts. At this time a minimum of two contracts is recommended. One contract would address construction of all dam site works. Bridge and road works would be included in the second contract. The two contract areas do not overlap and could proceed simultaneously. The multiple contract concept would provide smaller local contractors opportunity to bid some of this work and could allow earlier initiation of some portions of project construction.

The project schedule is dependent on factors including cash flow, land access, environmental studies and regulatory processes, subsurface field investigations and engineering design, and construction. As previously mentioned, design can proceed parallel with environmental studies and regulatory processes which could require 30 to 36 months to complete. Construction will require a minimum two calendar years, but a 3-year process is preferred considering the size of this project. Of course the government would need to weigh the risk of additional flood damage against the preferred longer construction period. Construction could proceed year-round, taking advantage of both summer and winter seasons. Most of the work would be performed in the spring through fall period; however, significant quantities of work could be completed in the winter. Special measures would be required for winter construction including heating and hoarding for concrete and continuous 24-hour per day earthfill operations. A project schedule can be developed but requires additional owner input.

9.0 CLOSURE

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

Yours truly,

AMEC Environment & Infrastructure *Reviewed by:*

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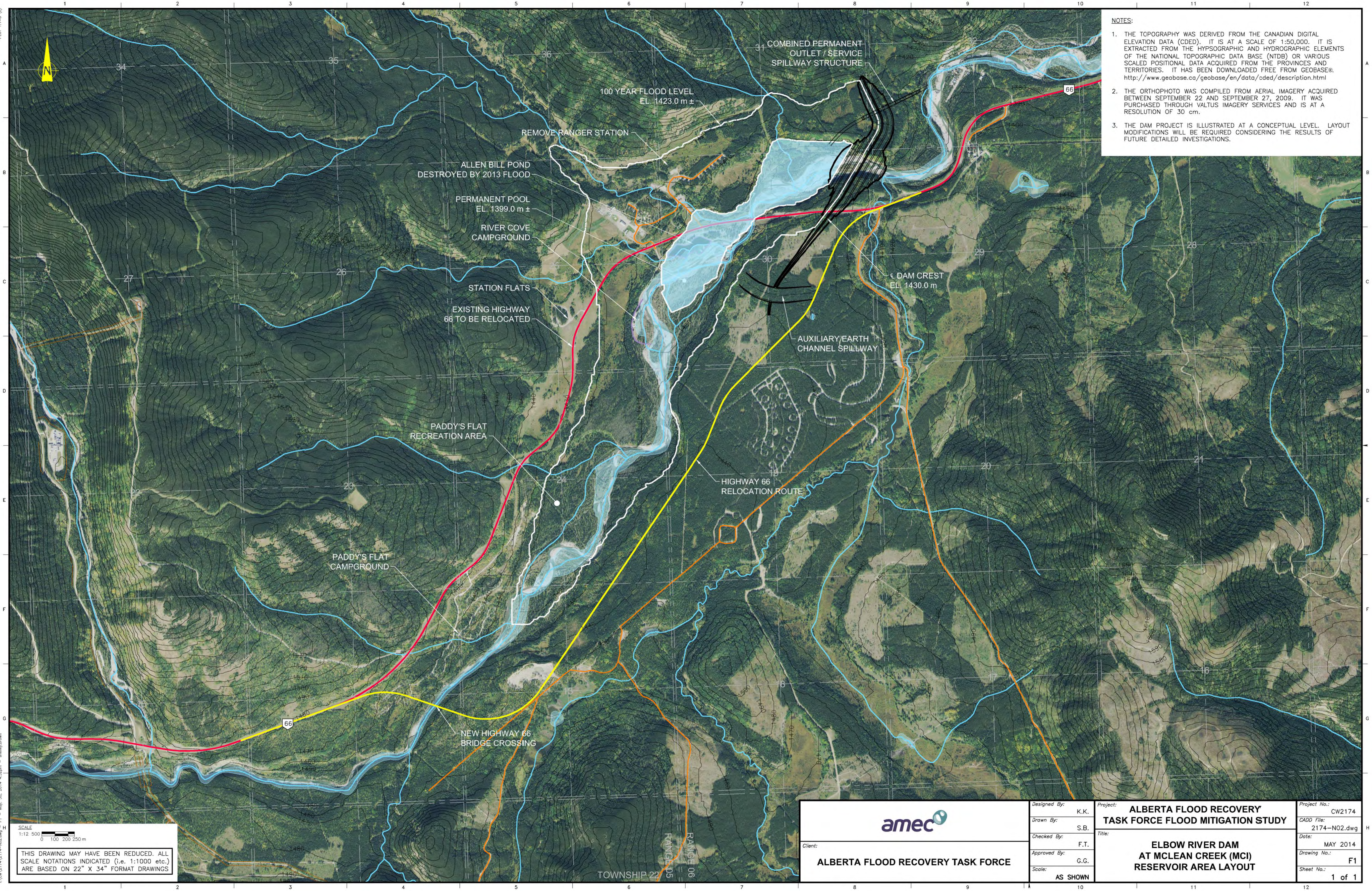
Permit to Practice No. P-4546



NOTES:

1. THE TOPOGRAPHY WAS DERIVED FROM THE CANADIAN DIGITAL ELEVATION DATA (CDED). IT IS AT A SCALE OF 1:50,000. IT IS EXTRACTED FROM THE HYPSOGRAPHIC AND HYDROGRAPHIC ELEMENTS OF THE NATIONAL TOPOGRAPHIC DATA BASE (NTDB) OR VARIOUS SCALED POSITIONAL DATA ACQUIRED FROM THE PROVINCES AND TERRITORIES. IT HAS BEEN DOWNLOADED FREE FROM GEOBASE®. http://www.geobase.ca/geobase/en/data/cded/description.html
2. THE ORTHOPHOTO WAS COMPILED FROM AERIAL IMAGERY ACQUIRED BETWEEN SEPTEMBER 22 AND SEPTEMBER 27, 2009. IT WAS PURCHASED THROUGH VALTUS IMAGERY SERVICES AND IS AT A RESOLUTION OF 30 cm.
3. THE DAM PROJECT IS ILLUSTRATED AT A CONCEPTUAL LEVEL. LAYOUT MODIFICATIONS WILL BE REQUIRED CONSIDERING THE RESULTS OF FUTURE DETAILED INVESTIGATIONS.

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc.) ARE BASED ON 22" X 34" FORMAT DRAWINGS

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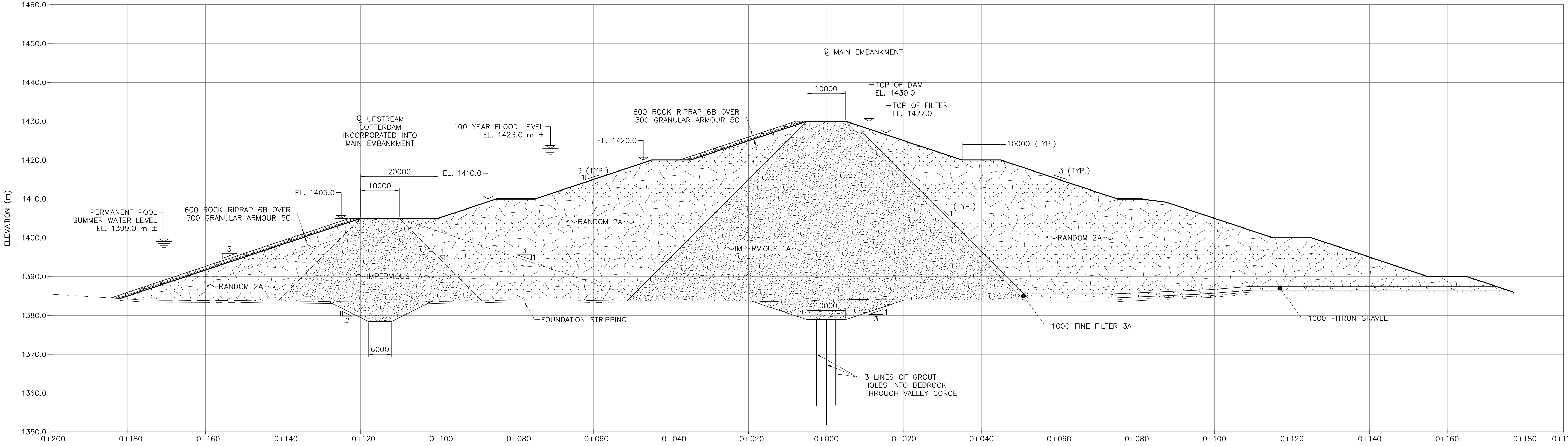
	Drawn By: S.B.		CADD File: 2174–N02.dwg
Client: ALBERTA FLOOD RECOVERY TASK FORCE	Checked By: F.T.	Title: ELBOW RIVER DAM AT MCLEAN CREEK (MCI) RESERVOIR AREA LAYOUT	Date: MAY 2014
	Approved By: G.G.		Drawing No.: F1
	Scale: AS SHOWN		Sheet No.: 1 of 1



MAIN DAM SECTION

SCALE 1:500

SCALE 1:500 0 5 10 m

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc.) ARE BASED ON 22" X 34" FORMAT DRAWINGS

REV	D	M	Y	ISSUE/REVISION DESCRIPTION	ENG.	APPR.
A	00	00	00	ISSUED FOR CLIENT REVIEW	X.X.	X.X.

amec

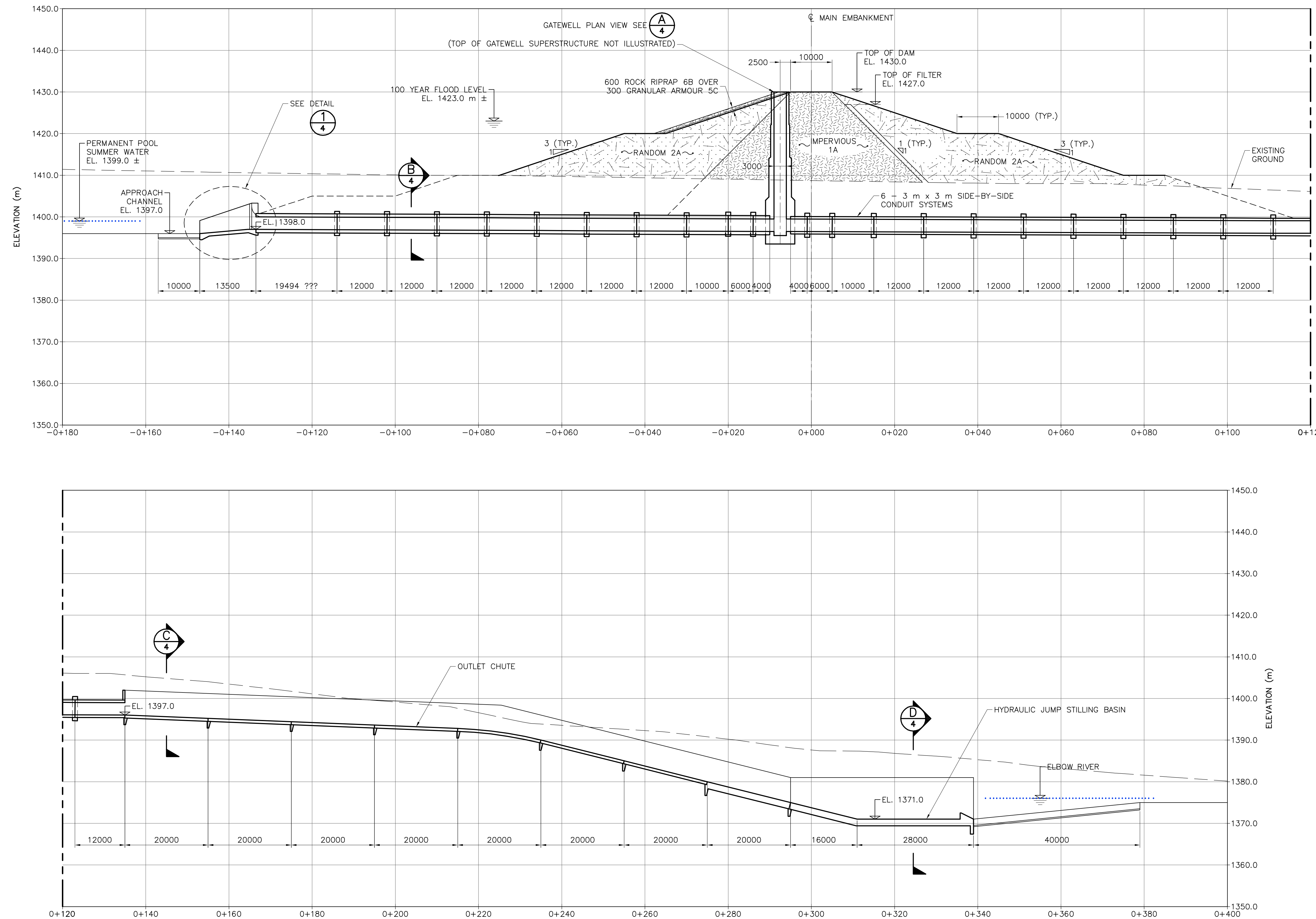
Client: ALBERTA FLOOD RECOVERY TASK FORCE

Designed By: K.K.
Drawn By: M.H.
Checked By: F.T.
Approved By: G.G.
Scale: AS SHOWN

Project: ALBERTA FLOOD RECOVERY TASK FORCE FLOOD MITIGATION STUDY

Title: ELBOW RIVER DAM AT MCLEAN CREEK (MCI) DAM SECTION

Project No.: CW2174
CADD File: 2174–N02.dwg
Date: MAY 2014
Drawing No.: F2
Sheet No.: 1 of 1



℄ PROFILE COMBINED PERMANENT OUTLET / SERVICE SPILLWAY STRUCTURE

SCALE 1:500

SCALE 1:500 0 5 10 m

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc.) ARE BASED ON 22" X 34" FORMAT DRAWINGS

REV	D	M	Y	ISSUE/REVISION DESCRIPTION	ENG.	APPR.
A	00	00	00	ISSUED FOR CLIENT REVIEW	X.X.	X.X.

amec

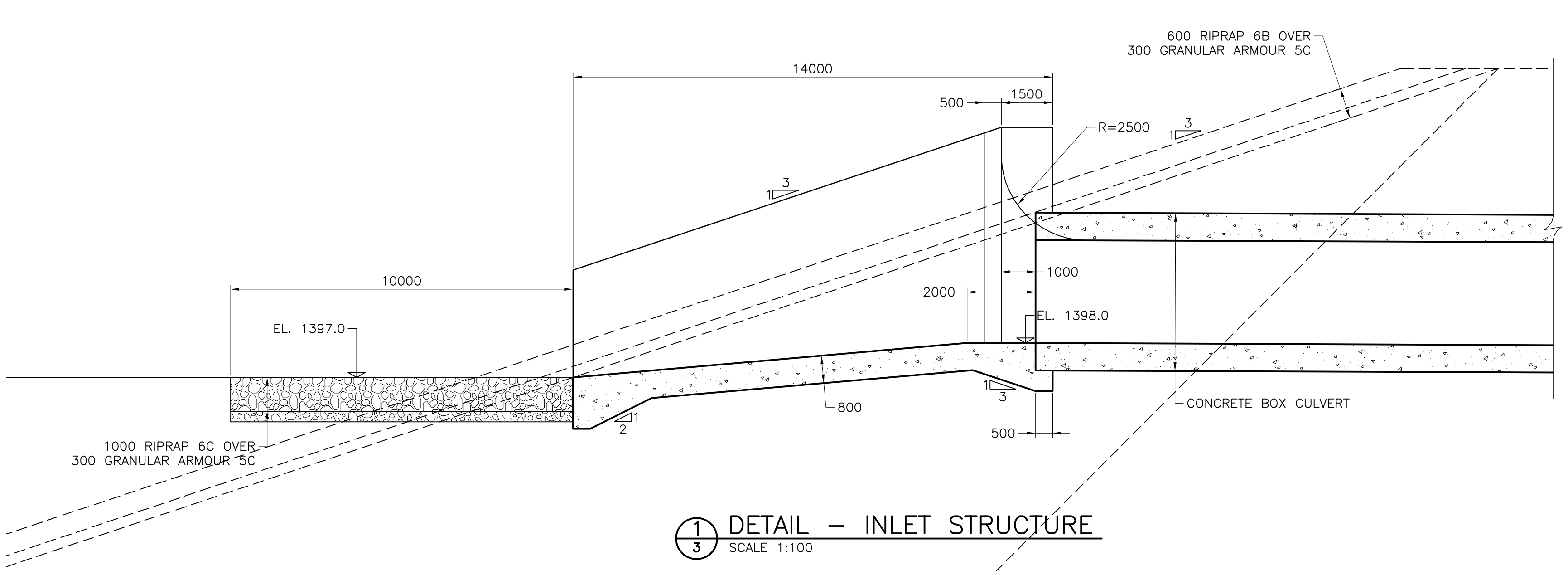
Client: ALBERTA FLOOD RECOVERY TASK FORCE

Designed By: K.K.
Drawn By: S.B.
Checked By: F.T.
Approved By: G.G.
Scale: AS SHOWN

Project: ALBERTA FLOOD RECOVERY TASK FORCE FLOOD MITIGATION STUDY

Title: ELBOW RIVER DAM AT MCLEAN CREEK (MCI) COMBINED PERMANENT OUTLET / SERVICE SPILLWAY STRUCTURE PROFILE

Project No.: CW2174
CADD File: 2174–N02.dwg
Date: MAY 2014
Drawing No.: F3
Sheet No.: 1 of 1



DETAIL – INLET STRUCTURE (1/3)
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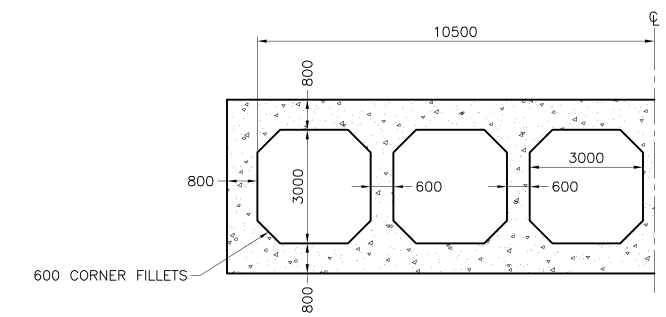

SECTION – CONDUIT SYSTEM (B/3)
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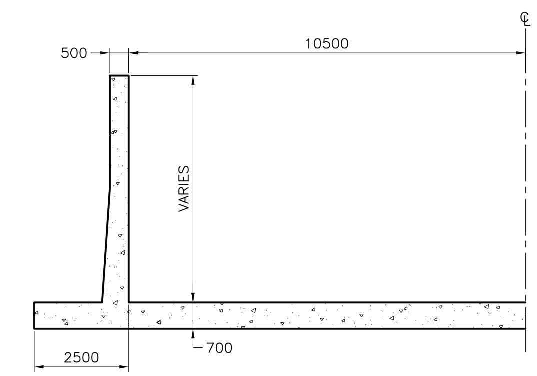
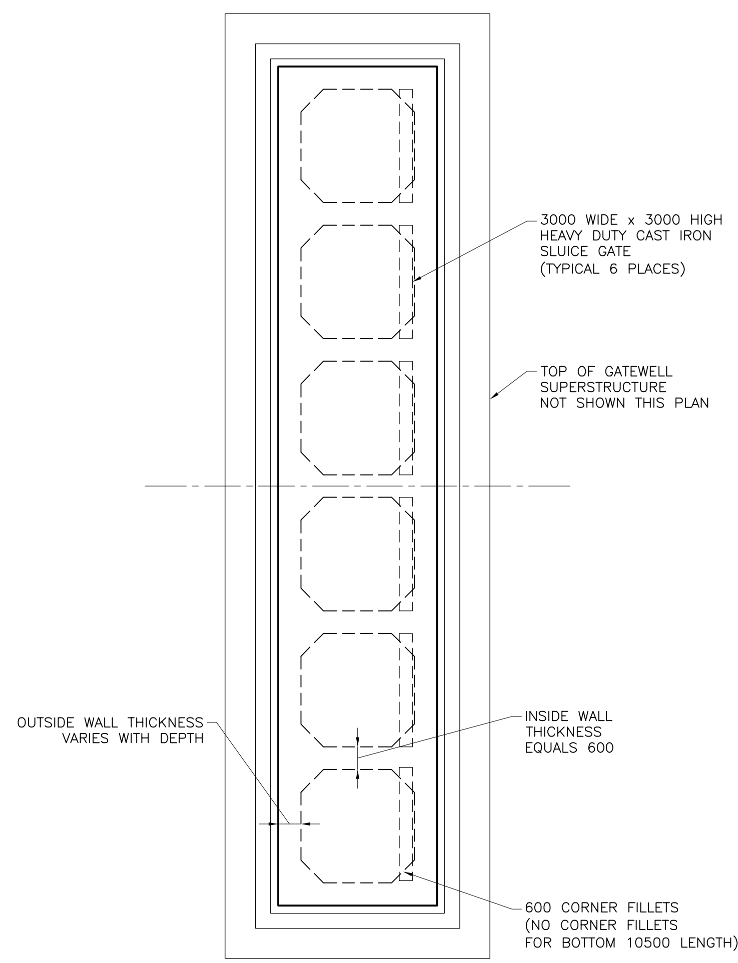
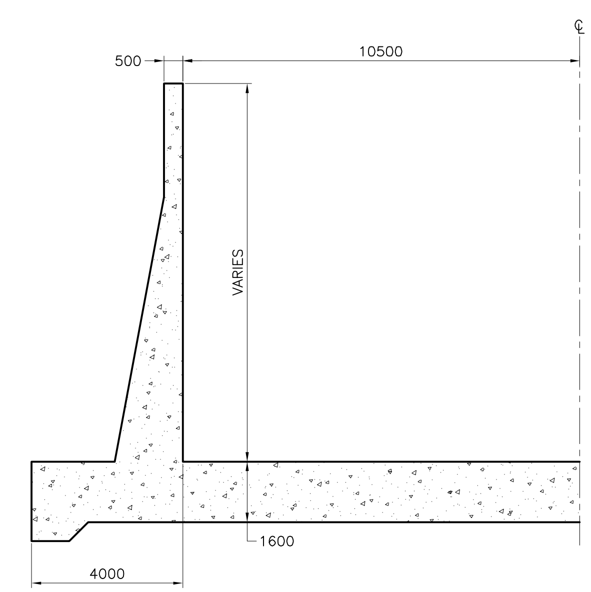


SECTION – OUTLET CHUTE (C/3)
SCALE 1:100



PLAN VIEW – GATEWELL (A/3)
SCALE 1:100



SECTION – STILLING BASIN (D/3)
SCALE 1:100

SCALE 1:100 0 1 2m

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc.) ARE BASED ON 22" X 34" FORMAT DRAWINGS

REV	D	M	Y	ISSUE/REVISION DESCRIPTION	ENG.	APPR.
A	00	00	00	ISSUED FOR CLIENT REVIEW	X.X.	X.X.

amec

Client: ALBERTA FLOOD RECOVERY TASK FORCE

Designed By:	K.K.
Drawn By:	S.B.
Checked By:	F.T.
Approved By:	G.G.
Scale:	AS SHOWN
Project:	ALBERTA FLOOD RECOVERY TASK FORCE FLOOD MITIGATION STUDY
Title:	ELBOW RIVER DAM AT MCLEAN CREEK (MCI) COMBINED PERMANENT OUTLET / SERVICE SPILLWAY STRUCTURE DETAILS
Project No.:	CW2174
CADD File:	2174–B02.dwg
Date:	MARCH 2014
Drawing No.:	F4
Sheet No.:	1 of 1