



March 30, 2013

PRE-DESIGN REPORT

AAFC - Highfield Dam Rehabilitation

Submitted to:

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REPORT



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**FINAL REPORT – HIGHFIELD DAM SERVICES CONTRACT NO. 6:
PROJECT REHABILITATION PRE-DESIGN STUDY**

Dear Mr. Haack,

We are pleased to submit three hard copies of our final report on the Rehabilitation Pre-Design Study for the Highfield Dam, as well as a DVD containing a digital copy of the report to AAFC.

We thank you for commissioning Golder Associates Ltd. to undertake this interesting assignment. Please contact the undersigned should you require any clarification regarding this report submission.

Yours truly,

GOLDER ASSOCIATES LTD.



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Executive Summary

Agriculture and Agri-Food Canada engaged Golder Associates Ltd. to conduct a pre-design engineering study in support of the rehabilitation of Highfield Dam, located near Swift Current, Saskatchewan. Highfield Dam impounds one of a series of water supply and irrigation reservoirs in the Swift Current Irrigation Project. The reservoir is located on Rush Lake Creek and receives inflow both from the upstream catchment and diversion inflow from Swift Current Creek via the Swift Current Main Supply Canal and overland drainage from the surrounding watershed. Highfield Reservoir provides irrigation water to the Rush Lake and Herbert Irrigation Projects located downstream from Highfield Dam. The 1,040 m long earth fill dam has a crest elevation of 724.8 metres and impounds approximately 15,130 dam³ when the reservoir is operating at its Full Supply Level of 723.0 metres (above sea level). The dam discharges water by two low-level outlets and an earth spillway.

Highfield Dam was originally constructed by the Prairie Farm Rehabilitation Administration between 1941 and 1942. The dam underwent a sequence of raises until 1950. The low-level outlet works were refurbished in the early 1990's. Highfield Dam was notionally classified as 'High' consequence prior to the release of the Canadian Dam Association's Dam Safety Guidelines (CDA Guidelines) in 2007. A more recent evaluation downgraded the incremental consequence classification to 'Significant'. This evaluation coupled with recent dam safety reviews identified a number of non-compliances with the Canadian Dam Association Guidelines, most notably the following:

- insufficient freeboard;
- inadequate capacity to safely convey the Inflow Design Flood;
- questionable structural integrity of the low-level outlet structures; and
- unacceptable factors of safety against embankment failure.

The rehabilitation pre-design provides design provisions to raise the embankment by 0.9 metres to an elevation of 725.7 metres while providing a granular blanket drain at the downstream toe of the embankment to improve pore water pressure dissipation. This will improve the factors of safety against future embankment slope failure.

The West Low-Level Outlet will be slip lined with a smaller diameter high density polyethylene pipe and grout. This will improve the structural performance of the conduit with minimal disturbance to the dam and reduction in discharge capacity.

The existing East Low-Level Outlet will be decommissioned. A drop inlet spillway and low level outlet works will be constructed on the east abutment to function as the principle spillway and to facilitate low level riparian releases to Rush Lake Creek. This structure will feature a reinforced concrete drop inlet with a control sill at Full Supply Level, a 2,000 millimetres diameter high density polyethylene pipe as the spillway conduit, and a reinforced concrete stilling basin. The low level outlet component of the structure will feature a 1,500 millimetre intake conduit, a reinforced concrete gate well containing a slide gate and associated equipment. The 1,500 mm diameter conduit will discharge into the spillway conduit near the centreline of the embankment.

The existing West Spillway will be upgraded so that the required 200 year Inflow Design Flood can be safely passed in combination with the Drop Inlet Spillway. The upgraded spillway will include a fuse plug dyke near the



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embankment centreline, a rock lined drop chute to pass flood flows into the Herbert Main Canal and a gabion lined side channel spillway to release flows safely downstream to Rush Lake Creek. The fuse plug is designed to utilize the available storage capacity in the reservoir to attenuate flood flows up to and including the 100 year flood with freeboard. In excess of the 100 year flood, the fuse plug is designed to erode quickly. The rehabilitated arrangement will therefore reduce the operational frequency of the West Spillway. The Drop Inlet Spillway will operate so long as the reservoir is above Full Supply Level.

A small amount of reservoir land control will be required as a result of the embankment raise. About 1 hectare of land will need to be purchased near the dam to facilitate structure construction and borrow material sourcing. A 0.6 kilometre section of a rural municipal road crossing the reservoir near its upstream extent will be raised to prevent flooding during extreme events.

The pre-design project layout and anticipated construction footprint have been reviewed in consideration of environmental concerns and species at risk identified in the previous studies. In order to manage potential environmental risks and to meet Agriculture and Agri-Food's goal that, "no significant environmental effects result from the rehabilitation of the project," recommendations are provided herein on the implementation of various best management practices. These best management practices must be reviewed during the detailed design phase of work and throughout construction to incorporate any new observations or environmental concerns not previously identified.

The construction of the dam rehabilitation work will be conducted over three construction seasons based upon the current construction sequencing plan. The construction has an estimated capital cost of \$13,151,000, including a 20% contingency. This estimate is based on a Class "C" cost estimate reflecting the information and design available at the time of the analysis contained herein.



Study Limitations

This report has been prepared by Golder Associates Ltd. for the benefit of the client to whom it is addressed. The information and data contained herein represent Golder Associates Ltd.'s best professional judgment in light of the knowledge and information available to Golder Associates Ltd. at the time of preparation. Except as required by law, this report and the information and data contained herein are to be treated as confidential and may be used and relied upon only by the client, its officers and employees. Golder Associates Ltd. denies any liability whatsoever to other parties who may obtain access to this report for any injury, loss or damage suffered by such parties arising from their use of, or reliance upon, this report or any of its contents without the express written consent of Golder Associates Ltd. and the client.



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1.0 INTRODUCTION

Golder Associates Ltd. (Golder) was retained by Agriculture and Agri-Food Canada (AAFC) to conduct a pre-design study for the rehabilitation of the dam and outlet works associated with the Highfield Dam site. The design work described herein was completed under AAFC Service Contract No. 6.

AAFC's overall goal for the rehabilitation of Highfield Dam is to improve on the level of dam safety such that the risks associated with a dam failure are reduced to as low as reasonably practicable (ALARP). The Canadian Dam Association (CDA) Dam Safety Guidelines (CDA, 2007) served as the basis for improvements to the dam. AAFC has set a goal to ensure that there are no significant adverse effects on the environment that result from the rehabilitation.

This report should be read in conjunction with the Study Limitations included at the beginning of this document. The reader is specifically directed to this information as it is essential for the proper interpretation and usage of this report.

1.1 Project Description

Highfield Dam is owned and operated by AAFC and is situated along Rush Lake Creek, approximately 28 kilometres east of Swift Current, Saskatchewan in Section 36, Township 15, Range 11 W3M. Originally constructed by the Prairie Farm Rehabilitation Administration (PFRA) between 1941 and 1942, the dam underwent a sequence of raises until 1950 and hydraulic structure modifications in 1950, 1952 and as recently as 1991, to its present arrangement. For a detailed location plan of the Highfield Dam and reservoir, refer to Drawing Sheet No. 100 found in Appendix D.

The reservoir is supplied by runoff from the Swift Current Plateau as it drains into the Rush Lake Creek watershed. The reservoir also receives diverted water from Swift Current Creek. As one of the major reservoirs in the Swift Current Creek watershed, water released from the reservoir is used primarily to supply water for irrigation, municipal, domestic use, while the reservoir and surrounding area provide habitat and some recreational value. Downstream water users include the Herbert and Rush Lake Irrigation Projects.

The components of the existing dam include: an earthfill embankment approximately 10 m high with a crest length of 1,040 m at the existing Top of Dam (TOD) elevation of 724.8 m (above sea level); a gated low level outlet works near the east end of the dam, a gated low level outlet works near the west end of the dam, and an earth spillway around the west abutment. The Highfield Reservoir has a Full Supply Level (FSL) of 723.0 m and impounds 15,130 cubic decametres (dam^3) at FSL and 25,800 dam^3 at TOD. The corresponding reservoir surface areas are 5.2 km^2 and 6.6 km^2 , respectively.

The dam is a bulk earth fill structure with a cross section consisting of an impervious clay core, a granular (gravel) toe drain, and riprap armored upstream face. The original construction included a compacted clay key located beneath the upstream side of the dam. The existing dam has a crest width of 5 m with an upstream slope of 3.0 horizontal to 1.0 vertical (3H:1V), and a downstream slope of 3.5H:1V.

Releases from the reservoir supply the Rush Lake Creek via the East Low Level Outlet (East LLO) and the Herbert Irrigation Project main canal (Herbert Canal), via the West Low Level Outlet (West LLO). An earth-cut spillway, located beyond the west abutment of the dam, provides additional flood discharge capacity.



The existing East LLO consists of an inlet, cast-in-place 1.5 m square concrete conduit section, a 1.5 m diameter and 1.6 m diameter Corrugated Metal Pipe (CMP) conduit section, a concrete gate well located on the upstream side of the Highfield Dam embankment, and concrete outlet stilling basin. The gate well houses a slide gate that is manually operated by a gate actuator located at crest level. The total length of the structure is about 90 m.

The existing West LLO consists of a cast-in-place reinforced concrete inlet, with a 1.5 m square conduit section transitioning to a concrete outlet (stilling basin). A concrete gate well is located on the upstream side of the Highfield Dam embankment and houses a manually operated slide gate similar to the East LLO. The total length of the structure is about 32 m.

The West Spillway is located beyond the west abutment of Highfield Dam and is excavated through native materials. The grass lined channel has a trapezoidal cross-section with a bottom width of 20 m and side slopes of 3H:1V. The spillway operates as a free overflow spillway and discharge is initiated when the reservoir exceeds the highest invert elevation of the channel at about 723.3 m. The spillway discharges into Herbert Canal.

1.2 Dam Safety Issues

A Phase I Dam Safety Evaluation (DSE) was completed by PFRA in 1989 (PFRA, 1989). PFRA was using a three tier dam classification system – Low, Significant and High. Highfield Dam was classified as a High consequence dam at the time of the PFRA evaluation. The Safety of Dam Design Flood (analogous to Inflow Design Flood) was set as the Probable Maximum Flood (PMF). The evaluation noted the existing spillway could not pass this flood and further study was recommended to resolve the apparent deficiency. Other noted concerns were the uncertain condition of the two low level outlets, performance issues with under-design of the upstream slope protection on the dam, and inadequate provisions in the design and construction of the embankment for proper seepage control measures. The evaluation noted that adverse seepage potential exists for the embankment and foundation.

AAFC completed internal preliminary engineering studies (AAFC 2009) to address the deficiencies outlined in the PFRA evaluation. These studies assumed a High consequence dam classification under the CDA 1995 Dam Safety Guidelines (revised 1999) and determined the IDF to be the 1,000 year return period event. The IDF was defined as 361 m³/s at the time of that analysis. The study concluded that the existing spillway and outlet facilities could not safely convey the defined IDF.

AAFC engaged Northwest Hydraulic Consultants (NHC), with MDH Engineering Solutions (MDH) as a sub-consultant, to advance the preliminary engineering to the feasibility level. The terms of reference for the study were to assume a dam classification of High based on CDA 2007 guidelines and to assume the IDF as the 1,000 year return period. The resulting report assembled by NHC (NHC 2011), examined a number of spillway options and developed a feasibility-level cost estimate for a select option. The report and AAFC mutually concluded that the assumed consequence classification and IDF were driving a very high rehabilitation cost estimate. A recommendation was given to complete a formal review of the dam's classification.

This formal review was carried out by Golder in 2011. The incremental consequence classification of Highfield Dam was reassessed based upon the 2007 edition of the Canadian Dam Association Dam Safety Guidelines (CDA Guidelines) (Golder, 2011). This resulted in AAFC reclassifying Highfield Dam with an Incremental Consequence Classification (ICC) of "Significant" with a recommended IDF corresponding to the 200 year event.



AAFC prepared an Investment Analysis Report based on the dam safety and rehabilitation requirements for Highfield Dam. This report indicated that modifications to the Highfield Dam are economically warranted. The prescribed modifications are intended to reduce risks and the associated probability of dam failure, and liability of AAFC/Government of Canada as measured by the CDA Guidelines. The scope of work associated with this endeavor is described in Section 2.0.

A summary of deficiencies with respect to compliance with current codes and operational performance is provided below:

- inadequate provision of freeboard allowance;
- inadequate capacity to safely route, attenuate, and discharge the IDF;
- insufficient factors of safety regarding the embankment stability consistent with the 2007 CDA Guidelines;
- inadequate provisions in the design and construction of the embankment for proper seepage control measures; and
- the concrete conduit and concrete gate well do not meet current code requirements.

1.3 Format of this Report

This design report has been prepared to summarize the pre-engineering design process. The report is organized as follows:

- Section 2.0 describes the scope of work;
- Section 3.0 provides a discussion of the pre-design field review;
- Section 4.0 includes a summary of the design basis;
- Section 5.0 describes the development of alternative design concepts and the selection process leading to preliminary design;
- Section 6.0 summarizes the pre-engineering design;
- Section 7.0 provide a summary of environmental considerations;
- Section 8.0 provides a discussion of planning for engineering resources;
- Section 9.0 discusses construction planning (sequencing, effort, equipment);
- Section 10.0 provides a cost estimate and cash flow assessment; and
- Section 11.0 summarizes project risks and mitigation strategies.

1.4 Previous Studies

Previous studies commissioned by AAFC have been prepared to summarize the as-built characteristics, assess the status of the dam, and to provide recommendations to rehabilitate and/or upgrade the dam and associated works. These studies have been reviewed and the previous considerations and recommendations made have been incorporated into the development of the pre-design efforts that are the subject of this report.



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The previous studies are listed in Table 1.

Table 1: Previous Studies

Study Title	Author	Date	Type
Heritage Resources Impact Assessment Agriculture and Agri-Food Canada Highfield Dam Rehabilitation	Bison Historical Services Ltd.	July 2012	Heritage resources impact assessment
Dam Safety Rehabilitation Highfield Dam	AAFC	February 2012	Rehabilitation plan
Highfield Dam Embankment Foundation Assessment	Golder Associates Ltd.	January 2012	Condition assessment
Highfield Dam – Dam Classification and Hydro Technical Study	Golder Associates Ltd.	November 2011	Hydrotechnical assessment
Highfield Dam Service Contract No. 3 Spillway Pre-Design Completion, 35525	Northwest Hydraulic Consultants and MDH Engineered Solutions Corp.	December 2011	Pre-design study
Highfield Dam Project – 2010 Dam Safety Review	Wayne Carlson, AAFC	June 2011	Dam Safety Review (DSR)
Rare Plant, Fish and Habitat Assessments for the Rehabilitation of the Highfield Dam Project AAFC / AESB Service Contract No. 2 – Final Report	KGS Group Consulting Engineers	December 2010	Terrestrial and aquatics assessment
Highfield Dam – Updated Flood Frequency Analyses	Fred R.J. Martin, AAFC	November 2007	Hydrologic assessment
Rare Plant, Wildlife, Fish and Native Habitat Assessment of Cadillac and Highfield Reservoirs, Saskatchewan	Jacques Whitford Environment Limited	December 2003	Terrestrial assessment
Highfield Dam East Irrigation Outlet Works Concrete Condition Survey 1989	W. Demiak, AAFC	May 1990	Condition assessment
Report on Repairs to East Outlet Highfield Dam Swift Current Irrigation Project	G.T. Forsyth, AAFC	January 1970	Repair and construction

AAFC = Agriculture and Agri-Food Canada

Studies undertaken by and/or commissioned by AAFC have reached the following dam safety conclusions and/or identified dam safety deficiencies:

- Consequence classification – **Significant**, no loss of life;
- Embankment stability analysis – factors of safety below CDA Guidelines;
- Embankment piping design – existing embankment is not adequately designed to resist piping and internal erosion;



- IDF – the 200 year flood is appropriate for Highfield Dam based on the revised ‘Significant’ classification. The existing spillway system is unable to safely convey this IDF; the spillway requires modification to pass the revised IDF; and
- Outlet works – uncertainty regarding the long-term integrity of these structures.

1.5 Rehabilitation Goals and Objectives

The AAFC holds two primary goals regarding the rehabilitation of the Highfield Dam.

The first goal, as stated in the AAFC Highfield Dam Rehabilitation Project Plan and Investment Analysis document, is to reduce risks (liability) to AAFC and the Government of Canada associated with Highfield Dam. This will be achieved by undertaking the design and construction of modifications to Highfield Dam in accordance with the CDA Guidelines such that the modifications reduce the probability of dam failure. This will ultimately reduce the risk (liability) to AAFC and the Government of Canada to as low as reasonably practicable.

Project components that may be modified to meet the CDA Guidelines and current codes include:

- normal and minimum freeboard;
- capacity to safely contain and convey the IDF;
- embankment stability;
- design measures to ensure the long-term integrity of the embankment against piping failure; and
- long-term integrity of the low level outlet works.

In order to manage potential environmental risks and to meet AAFC’s second goal stating, “no significant environmental effects result from the rehabilitation of the project”, the pre-design included a review of all aspects of the project layout and anticipated construction footprint against relevant environmental concerns identified in previous studies. These previous studies include field biological studies and Heritage Resources Impact Assessment (HRIA) work completed by others. The outcome of this study is a list of recommendations of best management practices. These best management practices are to be reviewed through the detailed design phase of work and throughout construction in order to incorporate new observations and environmental concerns not yet identified.

2.0 SCOPE OF WORK

Golder has been engaged to complete the following engineering services in support of the feasibility-level preliminary engineering design for the overall rehabilitation of Highfield Dam.

The general study objectives for the rehabilitation of Highfield Dam are as follows:

- raise of the dam crest from 724.8 m to 725.7 m;
- modify the existing spillway system to safely convey flood events up to and including the 1 in 200 year IDF without compromising the dam;
- rehabilitate the East LLO and West LLO to provide safe operation under the new loading conditions of a raised dam, including those loads experienced during construction;



- apply best management practices through the design process as well as during construction to address potential adverse environmental effects; and
- comply with applicable regulatory requirements.

Golder developed a scope of work based on the AAFC objectives. The scope consisted of nine tasks, as follows:

- Task 1: East LLO works assessment and pre-design;
- Task 2: West LLO works assessment and pre-design;
- Task 3: Embankment pre-design;
- Task 4: Spillway pre-design;
- Task 5: Schedule and cash flow assessment;
- Task 6: Pre-design study report;
- Task 7: Project management and project coordination;
- Task 8: Meetings; and
- Task 9: 3D CADD pre-design engineering drawings.

This report summarizes the body of work associated with Tasks 1 to 5 and 9.

3.0 FIELD INSPECTION

A field inspection was carried out by Golder on August 27, 2012. The inspection included visual assessments only and did not include operational testing of mechanical (gate) equipment destructive/non-destructive material testing of the existing fill materials or the concrete structures. The approved scope limited the inspections to the downstream sections of the East and West LLOs. The allotted time was sufficient to complete an overall site review of the Highfield Dam and all its appurtenances.

The inspected elements included:

- the main embankment dam;
- toe areas;
- up/downstream slopes including armor protection;
- the West Spillway located beyond the west abutment of the dam; and
- a portion of Herbert Canal (± 100 m) downstream of the West LLO.

Messrs. Andrew Szojka and Ron Kitagawa carried out the field inspection scope of work. Both Andrew and Ron have valid Confined Space Certification. Golder administered the required site safety protocols per Golder's policies and procedures. AAFC staff, including Messrs. Garth Haack and Bob Mills, provided the AAFC entry procedures, documentation, and field support for entry into the East and West LLO conduits. Additional AAFC field support staff provided equipment and materials to ventilate and pump standing water out of the conduits.



Golder prepared a Health, Safety and Environment Plan (HaSEP) in advance of the site inspection. The HaSEP was reviewed together with a Field Level Hazard Assessment (FLHA) with all Golder and AAFC staff upon arriving to the site. Prior to the entry of each outlet conduit, the gate actuators were locked out using personalized locks. The keys were held by those entering the outlet conduits until completion of the downstream conduit inspections. Air monitor readings were recorded on Confined Space Documentation as per conduit entry protocols.

A field inspection checklist is included in Appendix A together with select photographs taken during the site visit.

3.1 East Low Level Outlet Downstream Conduit

The East LLO was inspected above ground together with the following drawings:

- Drawing 117912 – 1992 Record Drawing titled: Plan & Profile of the East Inlet Structure as of Oct/92;
- Drawing 117913 – Record drawing titled: Plan Showing East Gate well, Ladder Details & Misc. Metal Work as of Oct/92;
- As Constructed Drawing C116711 – Existing Structure Details, Contract 1 – Rehabilitation of East Irrigation Outlet Works, 1991; and
- As Constructed Drawing C116714 – Structural Details, Contract 1.

AAFC provided record documents of previous repairs and inspections, including:

- 1970 Report on Repairs to the East Outlet, Highfield Dam;
- 1985 Dam Safety Inspection Checklist;
- 2010 Highfield Inspection Checklist;
- 1989 Highfield Dam East Irrigation Outlet Works Concrete Condition Survey;
- 2010 Dam Safety Review (Table 1 and 2 Summary); and
- 2010 Photo Record.

The following summarizes the visual condition of the East LLO conduit:

- The conduit vent pipe extends through the concrete structure but is fully exposed within the gate well riser in a submerged condition. The vent pipe was assumed broken within the gate well riser section as water flowed continuously into the outlet conduit for the duration of the inspection. A follow up inspection carried out by AAFC independent of Golder verified that the vent pipe is 'badly corroded' through inside the gate well riser and supplemented the observation by photograph received by e-mail October 24, 2012.
- The gate seated well with observed spray leakage from the four corners of the gate.
- The downstream face of the gate showed signs of corrosion with rust nodules present over 30% to 50% of the face. The gate appeared in overall fair condition.
- The concrete portion of the downstream conduit presented a degree of deterioration associated with structures of the same age with visible cracks, pour lines, surface flaking revealing aggregate, and pop-offs



adjacent larger cracks of previous repairs. Water lines were visible together with iron oxide (rust color) stains, and minor soluble salt deposition (efflorescence).

- Previous crack repair areas appeared in fair shape with separation cracks forming between the original concrete and repair mortar. Spalls or pop-offs were noted with one area with exposed reinforcing steel.
- The concrete transitions to the Corrugated Metal Pipe (CMP) area appeared in fair condition. The CMP showed advanced surface oxidation primarily along the haunch; between the spring line and invert of the pipe. A short section of (older) CMP was asphalt coated while the longer (newer) CMP was galvanized with advanced surface oxidation (rust) development. Horizontal and vertical measurements recorded indicated no significant deformation of the CMP. One puncture of the CMP was found on the crown of the pipe at about 5.4 m up the pipe measured from the end of the CMP at the 'waterface of the new headwall' (refer to PFRA drawing C116714) at the upstream side of the outlet works stilling basin. The puncture is suspected to have occurred during construction.
- The transition to a larger diameter CMP near outlet appeared to be in fair condition. The CMP was observed to be in fair condition.
- The transition of the CMP to the outlet structure, which is exposed to the seasonal temperature variation of the environment, appeared to be in fair condition.
- The concrete outlet structure and stilling basin showed few signs of deterioration, deformation, displacement, or surface weathering and thus is in good condition. Minor scouring of the invert section was present and minor vertical cracking was observed in the basin walls. The stilling basin was submerged at the time of the inspection and was not inspected.
- The outlet channel was observed to be heavily vegetated with aquatic/wetland type plants (cattails). Armor protection provided to transition from the stilling basin to the original Rush Lake Creek channel appeared to be in good condition.

3.2 East Low Level Outlet Works Overall

The overall embankment above the East LLO alignment revealed no surface anomalies. Embankment placed over the lower section of the outlet conduit provided a level space for vehicle access to the outlet. No protective fence is provided around the concrete outlet structure and the structure walls marginally extend above the ground surface.

The gate well appeared in good condition with minor erosion of materials evident where the riser pipe of the gate well meets the upstream embankment face. The water level inside the gate well reflected the reservoir level and was reported by AAFC to be at an elevation of 722.42 m at the time of the inspections. The gate well was not accessed during the inspection. The vent pipe was visible in the gate well above the waterline. No ruptures, breaks or vent pipe separation were observed above water. The gate actuator mounted to the top of the gate well appeared well maintained, well painted, and operable.

The inspection did not include underwater inspections of the gate well or the intake section of the East LLO.

3.3 West Low Level Outlet Downstream Conduit

The West LLO was inspected above ground together with the following drawings:



- Drawing 156-D7 – 1952 Record Drawing titled: Reinforcing Details of West End Outlet Structure.
- Drawing 2753 C – 1971 As Constructed Drawing titled: West End Outlet Structure.
- Drawing 2754 C – 1953 As Constructed Drawing titled: West Outlet Structure Reinforcing Details.

AAFC provided supporting record documents of previous inspections, identical to the documents listed previously for the east low level outlet.

The following summarizes the visual condition of the west low level outlet conduit:

- The outlet conduit contained pooled water.
- The conduit vent pipe was dry and showed no signs of leakage.
- The gate appeared to be well seated with little leakage observed between the concrete and the gate.
- The downstream face of the gate showed signs of corrosion with rust nodules and staining vertically over approximately 30% of the gate shell surface. The overall condition of the gate appeared to be good.
- The concrete conduit presented a lower degree of deterioration compared to structures having the same age. Visible cracks, pour lines, surface flaking revealing aggregate, and occasional spalls were observed along the outlet conduit. Concrete joints showed no signs of separation or displacement and there was no observed leakage. The water line in the conduit was well defined and was observed to be at the spring line (half the depth) of the conduit. Iron oxide (rust color) stains, and minor soluble salt deposition (efflorescence) was observed.
- The transition area to the outlet was observed to be in good condition.
- The concrete outlet structure presents as the drawings illustrate with ribbed side walls and roughened invert to the stilling basin. The walls of the structure appeared weathered, particularly at the corners and bend in the outlet to the wing walls.
- The walls of the outlet structure terminate at ground level and the surrounding vegetation hides the top of the outlet structure. The West low level outlet structure also has a narrow transition section to the basin and was difficult to observe due to the heavy vegetation cover. Fencing was not present around the structure and is potentially a public safety hazard.
- Armor rock was observed to be piled beyond the concrete structure inside Herbert Canal. The purpose of the rock pile was surmised to provide energy dissipation at the transition point to the canal.
- The downstream canal banks were observed to be well vegetated with visible infilling of the canal invert as a consequence of the canal bank erosion processes.

3.4 West Low Level Outlet Works Overall

The embankment fill over the West LLO alignment showed no sign of deformations, displacements, cracks, or other surface anomalies. Access to the West LLO was provided by a trail from the crest of Highfield Dam and continued along the crest of Herbert Canal's right embankment.



The gate well appeared in good condition with armor displacement visible due to surface runoff concentrations adjacent the gate well structure. The gate well interior was not accessed; however the ladder access and gate stem appeared in good condition. Pour lines in the concrete were visible with minor honeycombing of the concrete surface. Algae were visible on the surface of the water inside the gate well and potentially may pose a hazard to entry by means of oxygen depletion. The gate actuator mounted to the top of the gate well appeared well maintained, well painted, and operable.

The inspection did not include underwater inspections and the lower (underwater) portion of the gate well or the intake section of the West LLO.

3.5 Highfield Dam – Main Embankment and Downstream Toe Area

Highfield Dam is accessible via rural road from the east abutment and by an un-maintained canal dyke crest trail on the west side. Access from the west would be essentially impassible during inclement weather except by way of special construction equipment. Both the right (east) and left (west) abutments are well maintained and have well-established vegetative cover. No gullies or erosion features were observed. Tree and brush were prevalent on the reservoir side but off of the main dam at both abutments. The East LLO and West LLO alignments are immediately adjacent the abutment areas. The dam did not present any surface anomalies along the LLO alignments.

The upstream and downstream slopes were uniform where visible above the reservoir level. Armor rock consisted primarily of rounded field stone with an estimated d_{50} of between 250 and 300 mm. The armor appears to have performed well despite some signs of beaching, gap grading, and voiding in some areas to the bedding material. The downstream slope was well vegetated with tall grasses. Some shrubs have encroached onto the downstream face of the dam near the toe. One animal burrow was observed.

The crest of the dam was surfaced with granular road topping, appeared well maintained and was free of vegetation. One minor depression was observed. Settlement anomalies, cracking or displacements were not observed along the dam crest.

The toe area of the dam was relatively dry at the time of inspection but showed signs of shallow groundwater seepage. Several oxbows with free water were observed extending to the toe of the dam. Heavy shrub growth was observed in a number of locations with some areas encroaching onto the lower portion of the dam as mentioned above. Care of water or preservation of watercourses will be a paramount concern along the toe of the dam. Excavation at the toe of the dam will require careful planning and consideration to manage water.

3.6 Earth Spillway

The earth-cut spillway was vegetated with well-established tall grass. Free roaming cattle were grazing near the reservoir at the inlet end of the spillway. The access road from the crest of Highfield Dam crosses the spillway at about the halfway point to the downstream point of discharge into Herbert Canal. The access road embankment is raised above the spillway channel invert and would impose a minor barrier to flow.

The spillway showed signs of having been operated in the past with vegetation gaps, exposed earth and an erosion gully has formed at the spillway point of discharge into Herbert Canal.



3.7 Herbert Canal

From record drawings provided in advance of the field inspection, Herbert Canal was constructed with a very shallow bed grade ($S = 0.00025$), a bed width of 3.0 m, and 2H:1V back slopes. The design capacity of the canal has been reported to be approximately $6.0 \text{ m}^3/\text{s}$ and provides an approximate freeboard of 0.9 m.

The Herbert Canal was inspected from Highfield Dam to approximately 100 m past the overflow spillway's point of discharge to the canal. The right bank of the canal (looking downstream) appeared overbuilt. The LiDAR survey contours and PRFA Drawing 45628 confirm that the highest fill elevation occurs nearly 200 m downstream of the crest of Highfield Dam nearly matching the existing design crest elevation of the dam. No explanation of the overbuilt portion of the canal embankment is noted in the records however it is surmised that the overbuilt section was required to direct flows along the canal and away from Highfield Dam (refer to Section 4.1.4 and 6.2.2.4). The right bank provides a vehicle trail for access to the downstream infrastructure. The canal banks were observed to be well-vegetated and the canal bed showed signs of sedimentation from bank erosion, particularly downstream of the overflow spillway's point of entry. The deposition was surmised to be the result of material erosion along the spillway alignment during operations.

The top of the right bank was observed to be higher near the spillway entry point and lower at Highfield Dam and points downstream of the spillway entry. This was later confirmed by during a review of the LiDAR information provided by AAFC.

3.8 Field Instrumentation

Field instrumentation was observed during the site visit however no piezometer readings were taken at the time of the inspection. Recent drilling activity was evident by the presence of bare and rough earth on the downstream face of the dam in addition to the invasion of weeds in contrast to the more uniform tall grass distribution along the majority of the dam face. Selected instruments have been protected from the potential of animal (cattle) disturbance using barrier fencing.

3.9 Post Inspection Low Level Outlet Conduit Surveys

AAFC completed a conduit survey after the completion of Golder site inspections. A copy of an annotated drawing was provided to Golder on October 1, 2012 and is attached in Appendix A. A review of the survey results suggests that there are no significant deviations from what is presented on the record drawings provided for both outlets nor what might compromise the intended operational hydraulics of the structures.

4.0 DESIGN CRITERIA

This section provides a definition of the project design criteria. It includes a discussion of supporting background information, functional requirements to be upheld during construction, performance objectives for the post-rehabilitation project, and instrumentation and monitoring requirements.

4.1 Background Information

4.1.1 Commentary on CDA 2007

The CDA guidelines were established to provide an industry standard for the care of dams. The guidelines were most recently revised in 2007. The revision included the definition of principles that are applicable to all dams in Canada. The document provides guidance with respect to processes and criteria that facilitate compliance with the guiding principles.



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Among the principles is the conservative practice to provide protection that is ALARP. The application of this principle relies on the classification of the dam based on the consequence of failure tempered by the application of sound engineering judgement.

The 2007 CDA Guidelines reflect a modern update to the CDA Guidelines of 1999. The entire document was fundamentally restructured and evolved over nearly three decades of collaboration with industry dam safety professionals and dam owners. The most notable change is seen in the Dam Classification categories of Table 2-1 of the CDA Guidelines that essentially increases the category numbers from the previous four to five.

AAFC is endeavouring to align the management of its dams in accordance with the CDA Guidelines. Highfield Dam, a structure designed and built between 1941 and 1942, and modified over decades of operation, has held a notional High classification until 2012 when the AAFC formally stated Highfield Dam to hold a consequence classification of 'Significant'. This revised classification was grounded through a rigorous and detailed ICC assessment that included hydraulic modeling and inundation mapping analysis for hypothetical dam breach scenarios of Highfield Dam (Golder 2011).

4.1.2 As-built Information

The as-built documents and drawings identified in Table 2 were provided by AAFC.

Table 2: As-built Records

Document Name	Type	Date	Author
Highfield Reservoir Existing Topography and Drill Hole Locations, 208524	Drawing	July 28, 2011	G. Haack, C. Hill (AESB)
Highfield Dam Location Plan, Profile and East and West Outlet Cross-Sections, 102819A	Drawings	February 5, 1987	PFRA
Highfield Dam Reservoir Topography West ½ Sec. 31-15-10W3 & Sec. 36-15-11W3 and Flooded Area and Capacity Curves, 92907	Drawing	August 1973	PFRA
Swift Current Irrigation Project general Plan for Highfield Reservoir, 1085-C-1	Drawing	April 1943	PFRA
Swift Current Irrigation Project Highfield Reservoir Outlet Duct & Outlet Control Structures, 2722	Drawing	April 1943	PFRA

PFRA = Prairie Farm Rehabilitation Administration

4.1.3 Hydrotechnical Analysis

The hydrotechnical analysis of Highfield Reservoir, (Golder 2011) completed in 2011 included a compilation of hydraulic, hydrologic, and survey data. The data was collected at the local road bridges and culvert crossings, major highway, railway bridge and culvert crossings, and all creek channels, canals, and floodplains located downstream of the dam structure.

Sources of information for a description of the reservoir and dam structure (including low level outlets) were obtained from AAFC. The compiled information includes the following:

- past study and design reports;
- design and as-built record drawings relating to the spillway and east and west low level outlet hydraulic capacities;



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- 1:50,000 scale topographic mapping with 10 m contour intervals;
- LiDAR mapping with 0.5 m contour intervals (collected in 2009); and
- Site information collected during a field reconnaissance on August 26-27, 2012.

AAFC completed an evaluation of Highfield Dam in 2009. The evaluation concluded that the existing spillway and outlet works do not have the capacity to safely convey the project's IDF. The IDF was estimated to have a peak flood discharge of $361 \text{ m}^3/\text{s}$, which is equivalent to the 1,000-year flood event.

The incremental consequence classification was revised in 2012. The revision was based on alignment with the 2007 CDA Guidelines. The consequence classification was reduced from 'HIGH' to 'SIGNIFICANT' as a result of the assessment. Following the reclassification of Highfield Dam, AAFC has developed rationale to adopt the 200-year rainfall event as the IDF for Highfield Dam.

AAFC completed the flood hydrology assessment for this project in 2007, updated in 2011. A copy of the complete analysis is included in Appendix B. The 200-year rainfall event generates an instantaneous peak inflow of $180 \text{ m}^3/\text{s}$ and produces a runoff volume of $2.69 \times 10^4 \text{ dam}^3$. The inflow rainfall hydrographs that were used in this study were provided by AAFC in 2011 and are shown in Figure 1.

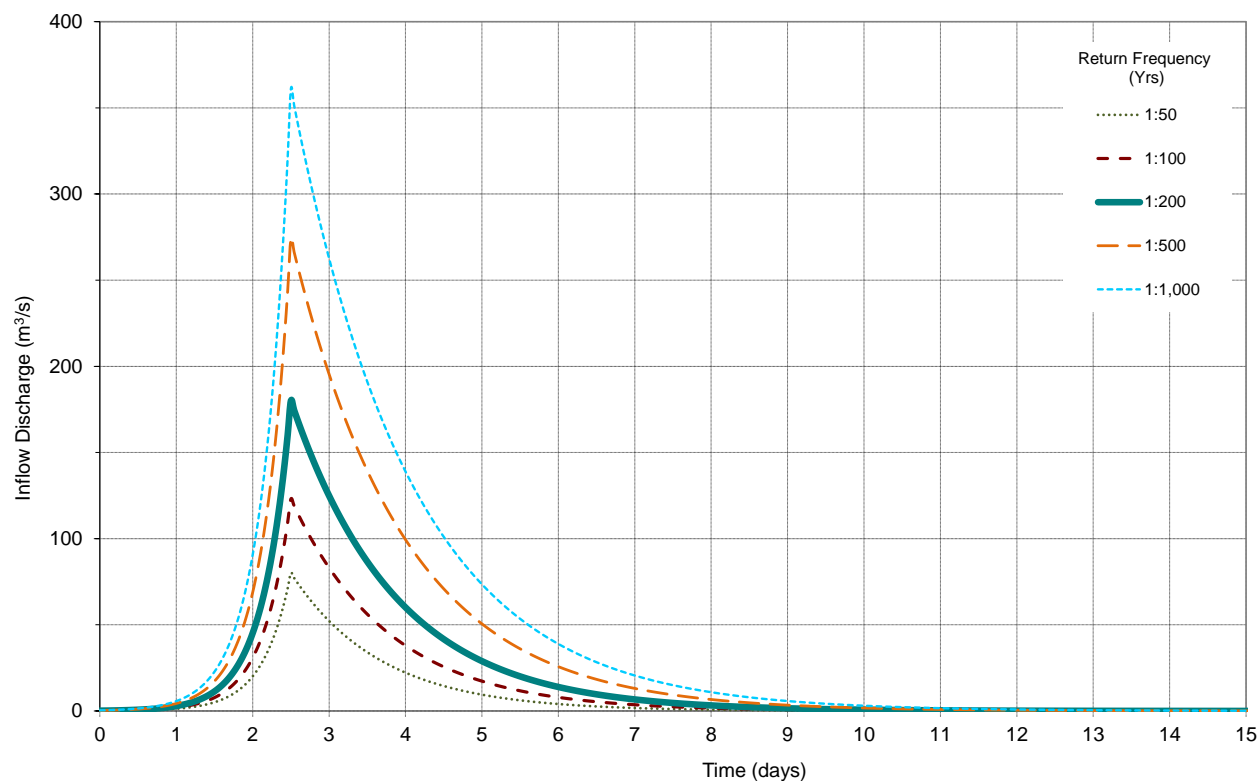


Figure 1: Rainfall Inflow Hydrographs to Highfield Reservoir (AAFC 2011)

AAFC provided elevation-storage-area curves for Highfield Reservoir, as developed in 2011. The elevation-storage curves are shown in Figure 2 (AAFC-AESB's Drawing No. 208533). The FSL of the reservoir



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is 723.0 m while the fall target level (FTL) is 720.8 m. The corresponding reservoir surface area and storage at the FSL are 5.2 km² and 15,130 dam³, respectively.

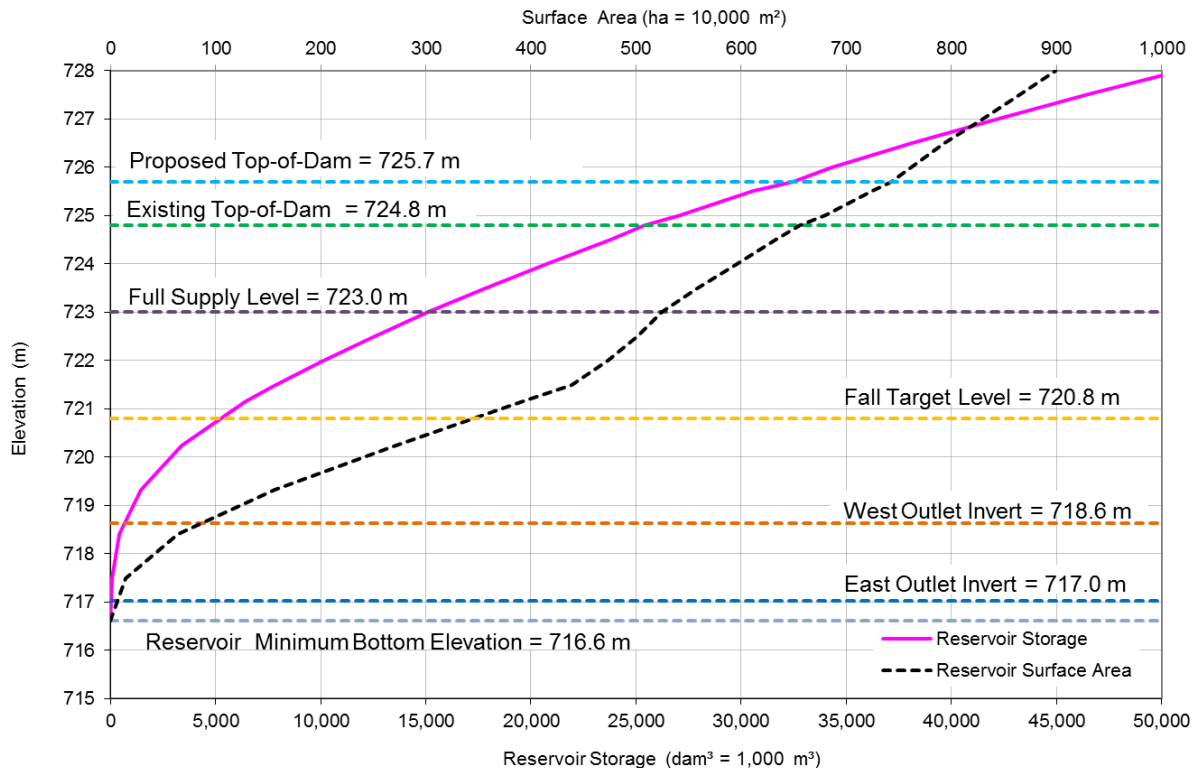


Figure 2: Highfield Reservoir Surface Area and Storage Curves

4.1.4 Freeboard Analysis

Golder completed a cursory freeboard analysis of Highfield Reservoir. This report includes both minimum and normal freeboard analyses. This analysis assumes that the proposed crest level is protected from overtopping by 95% of the waves caused by the critical wind when the reservoir is at its maximum extreme level.

The critical wind speed is dependent on the consequence class of the dam. The wind frequency that is used for the calculation of the freeboard is the 1 in 10 year event given that Highfield Dam has been assessed a classification of 'Significant'. A frequency analysis was completed for overland wind speed using the climate data from Swift Current.

A wave run up analysis was conducted based on the parameters presented in Table 3. Reservoir depths and fetch lengths were based on available topographic maps and reservoir stage-area curves. Wind speed was adjusted to reflect wind blowing over Highfield Reservoir. The analysis assumed that the upstream face of Highfield Dam is armored with riprap with a slope of 3H:1V. The analysis estimated a wave run up of 0.8 m.



Table 3: Cursory Freeboard Analysis for Highfield Dam

Parameter	Minimum Freeboard Analysis	Normal Freeboard Analysis
Return Period	1:10 years	1:1000 years
Overland Wind Speed	88 km/h	126 km/h
Adjusted Wind Speed	106 km/h	151 km/h
Upstream Side Slope	3H:1V	3H:1V
Fetch Length	1.5 km	1.5 km
Significant Wave Height	0.65 m	1.05 m
Wave Speed	3.6 m/s	4.2 m/s
Wind Setup	0.05 m	0.13 m
95% High Wave Run Up	0.7 m	1.2 m
Total Freeboard Requirement (Setup + Run up)	0.8 m	1.3 m

km/h = kilometres per hour; m = metres; km = kilometres; m/s = metres per second

The water level of the reservoir, including the wave run up, is 725.6 m. Assuming that the proposed TOD is set at 725.7 m, the resultant freeboard is 0.1 m. This conclusion suggests that the capacities of the outlet works and spillways of the modified dam structure are adequate based on the classification of Highfield Dam.

4.1.5 Reservoir Routing Analysis

Reservoir flood routing analyses were completed for the current and proposed spillway structures at the dam. Rating curves were developed for each case. The analyses assumed that the low level outlets were not operational during flood passage. For the proposed configuration, the analysis assumed that the fuse plug in the west spillway had been instantaneously eroded on reaching the 100 year flood reservoir level and the spillway control sill invert of 723.3 m governed.

A comparison of the combined rating curve is depicted in Figure 3.



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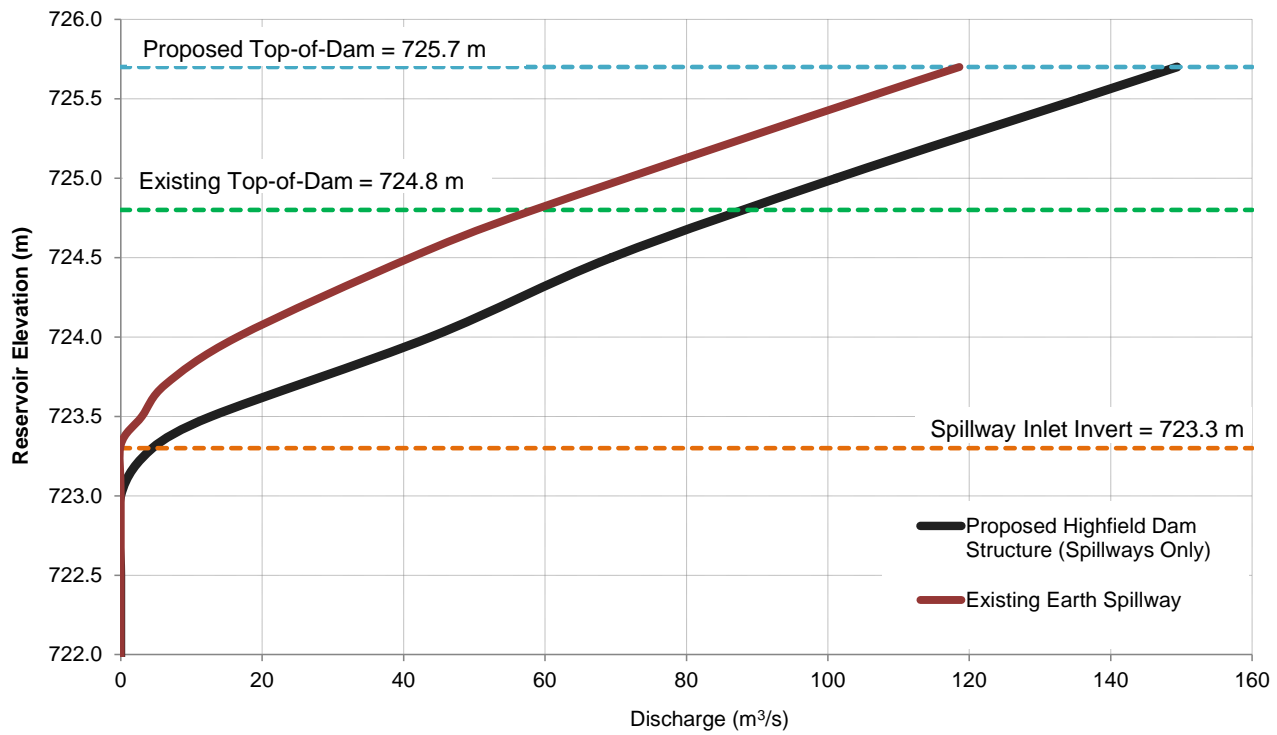


Figure 3: Spillway Rating Curves for the Existing Spillway and Proposed Modifications to Highfield Dam

The performance of the existing dam and the proposed dam structure was evaluated against the instantaneous peak flow expected to occur during the 200-year rainfall flood event (Q_{200}). AAFC provided the 200-year inflow hydrograph to Golder for this analysis.

A rating curve for the reservoir system was developed from the individual rating curves of the Drop Inlet Spillway and the West Spillway. The analysis used the stage-discharge relationship provided by AAFC. The reservoir routing analysis assumed a starting reservoir water level at FSL and operated up to the maximum reservoir level.

The wave run up on the upstream face of the dam is approximately 0.8 m assuming a face slope of 3:1 (refer to Section 4.1.4). The target reservoir level during the IDF must be below 724.9 m to provide adequate freeboard. The analysis suggests that proposed top of dam elevation of 725.7 m is sufficient for this purpose.

The discharge capacities of the existing and post-rehabilitation scenarios were assessed. A comparison of the discharge capacities is shown in Figure 4.



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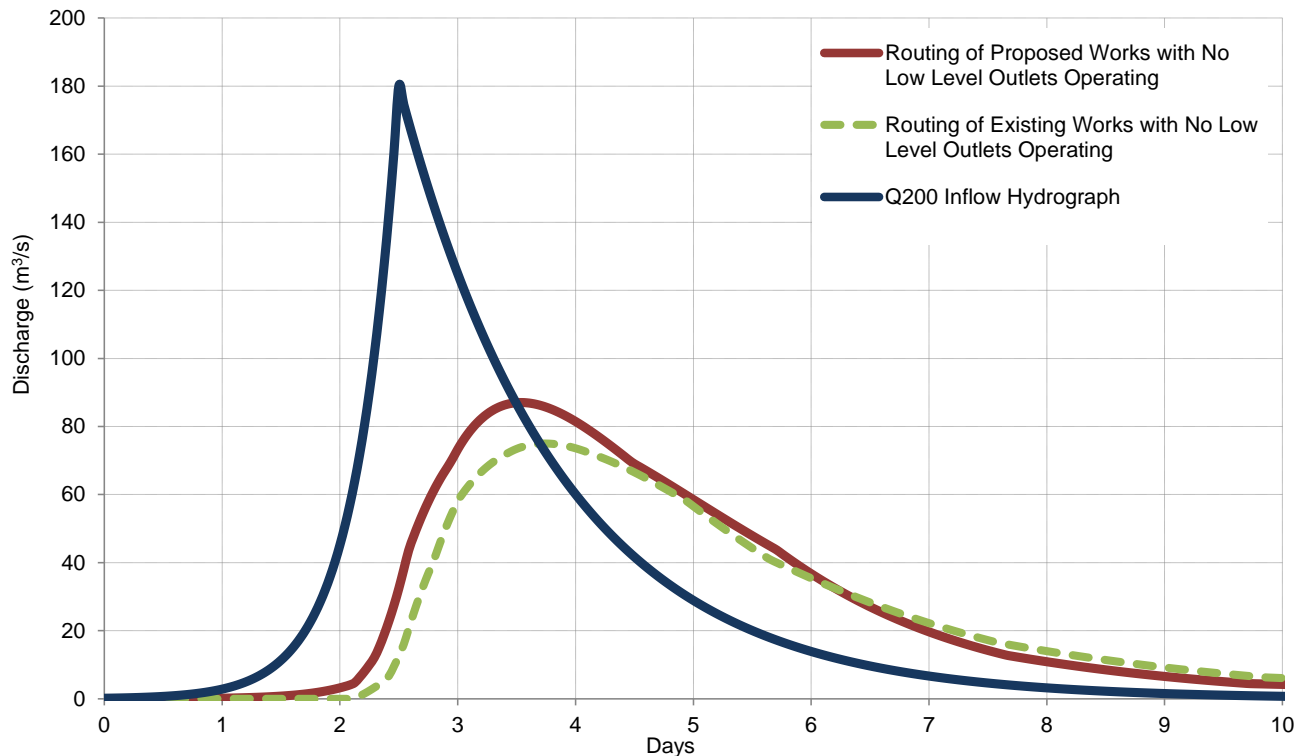


Figure 4: Inflows and Outflows of Reservoir Routing Analysis of Proposed Dam Outlets (West Spillway and Drop Inlet Spillway)

The maximum discharge that may be passed through the rehabilitated dam outlet works is 86.3 m³/s. This conclusion is based on the reservoir routing analysis for the IDF.

The maximum reservoir elevation for the rehabilitated dam scenario is 724.8 m. This is 0.2 m lower than the reservoir level under the existing (non-rehabilitated) dam scenario (see Figure 5). The available freeboard will be 0.9 m assuming that the post-rehabilitation TOD elevation is 725.7 m. The analysis results in a maximum discharge through the West Spillway and the Drop Inlet Spillway of 58.4 m³/s and 29.0 m³/s, respectively, when the reservoir is at the maximum reservoir elevation.

Initial reservoir levels below FSL will provide additional storage for flood inflows. Spillway operations will only begin when reservoir levels exceed FSL as the Drop Inlet Spillway crest is set to FSL. The West Spillway and Fuse Plug arrangement will permit flood flows to attenuate as the crest of the Fuse Plug is set to erode only after a 100 year flood inflow is exceeded (assuming the reservoir is at FSL at the start of the flood inflow). The Fuse Plug is designed to erode and provide sufficient capacity to safely convey the IDF. With the rehabilitated dam arrangement, the West Spillway is anticipated to operate less frequently.

The maximum velocity generated by the East LLO drop inlet structure is approximately 9.2 m/s as the pipe discharges to the East LLO stilling basin. The West Spillway discharges to the Herbert Canal with velocities ranging up to 2.9 m/s during the peak flows of the IDF while velocities in the Herbert Canal are anticipated to range up to about 1.2 m/s.



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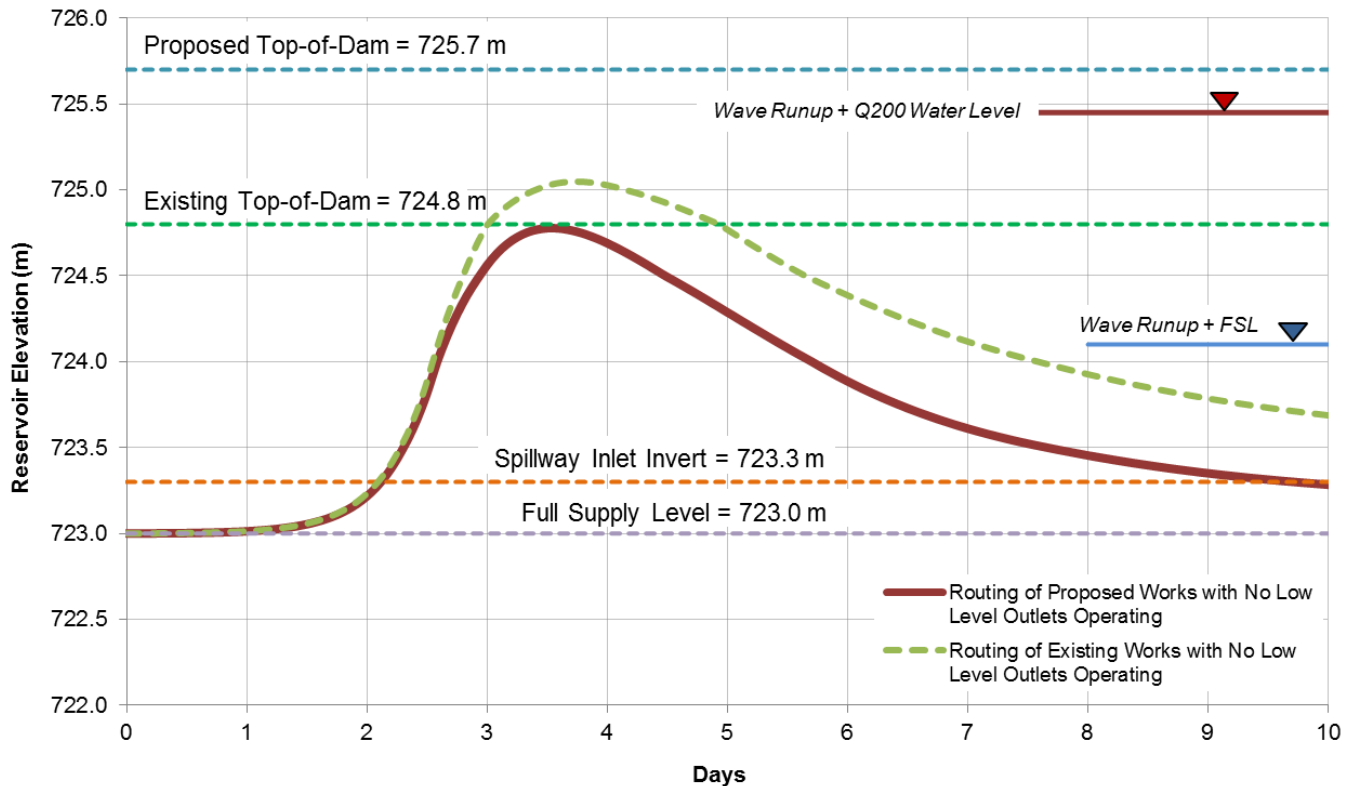


Figure 5: Reservoir Elevations of Reservoir Routing Analysis of Proposed Dam Outlets (West Spillway and Drop Inlet Spillway)

4.1.6 Geotechnical Data

Highfield Dam is located in the Rush Lake Creek alluvial floodplain. This area is characterized by surficial stratified deposits of sand, silt, and clay overlying an eroded glacial till plain. The Saskatoon Group (till and stratified drift) is typically less than 10 m in thickness and underlain by the Bearpaw Formation within the vicinity of the project (Maathuis & Simpson 2007). The Bearpaw Formation is characterized by soft, gray, non-calcareous marine silt and clay materials which are commonly known as shale.

MDH Engineering Solutions Corp. (MDH) reviewed the geotechnical aspects of Highfield Dam and various pre-design spillway alternatives as a subconsultant to NHC (NHC 2011, Appendix B). The final NHC report summarizes the general subsurface conditions and provides a summary of the general sequence of strata beneath the dam. These strata are described as follows in descending order:

- Post-Glacial Deposits:
 - Topsoil;
 - Alluvium; and
 - Possible eolian (wind-blown) deposits on the west upland.
- Glacial Deposits:



- Saskatoon Group (surficial stratified drift, Battleford Till, and Floral Till); and
- Sutherland Group.
- Bedrock:
 - Bearpaw Formation:
 - Aquadell Member;
 - Cruikshank Member; and
 - Snakebite Member.
 - Judith River Formation.

The NHC report indicated that the existing geotechnical information was insufficient to complete an embankment stability assessment of the dam. Golder was then engaged to complete a foundation and stability assessment of Highfield Dam in 2011.

A foundation assessment of Highfield Dam was completed in 2012 (Golder, 2012). This assessment included an evaluation of the consolidation and compressibility parameters of the alluvial foundation materials and identified key geotechnical considerations for the construction of remedial embankment works. The *Phase I Dam Safety Evaluation Report* for Highfield Dam, completed by the Prairie Farm Rehabilitation Administration (PFRA) in 1989 identified several deficiencies in the Embankment and Foundation conditions, including but not limited to poor compaction and high water contents in the embankment fill and inadequate seepage control measures. These issues were considered in the analysis for the foundation assessment.

The foundation assessment included the evaluation of a series of remedial embankment configurations. The study assessed cost, constructability, and minimization of impacts to wetland habitat, and recommended the following embankment geometry:

- construct dam embankment above an elevation of 720.4 m at a 3.5H:1V slope;
- construct dam embankment below an elevation of 720.4 m at a 5H:1V slope;
- berm width of 5 m at elevation 720.4 m;
- base of berm extending approximately 21.8 m from the existing toe; and
- construction of a granular blanket drain with a minimum thickness of 1.2 under the new fill.

The 2012 foundation assessment considered the rationale to stage construction to allow suitable time for pore water pressures within the embankment to dissipate to following the placement of new fill material. Golder devised a preliminary construction sequence to allow pore water pressures to dissipate based on transient embankment seepage analysis. The embankment remediation should be constructed in two stages with the first consisting of the construction of the toe berm followed by the embankment raise. The 2012 SEEP/W model by Golder predicted that a minimum lag time of 30 days was required to fully dissipate increased pore water pressures caused by the placement of fill on the dam for the first stage of construction (i.e., toe berm to elevation 720.4 m). However, the seepage model assumed that the Stage 1 fill was placed instantaneously. Since the fill



placement will occur in compacted lifts, some pore water dissipation will occur as construction progresses. The time required for pore water pressure dissipation will depend on the method and timing of fill placement during the first stage. Monitoring of the instrumentation during construction will be important to maintain adequate stability of the embankment and will dictate the actual rate of fill placement.

AAFC has expressed a desire to spread the capital expenditure over two years. This schedule will assist in reducing risks associated with dissipation of pore water pressures within the embankment.

4.1.7 Settlement and Deformation Analysis

A settlement and deformation analysis was conducted as part of the 2011 dam foundation assessment (Golder 2012). The evaluation utilized the GeoStudio 2007 SEEP/W and SIGMA/W finite element software.

The analysis concluded that the maximum deformation is expected to occur at the toe of the existing dam where the greatest depth of fill will be placed over existing ground. The modelling estimated that the maximum settlement due to consolidation at the toe of the dam may be 330 mm. The model indicated that the alluvial foundation materials will reach 90% consolidation within 1.3 years following the total placement of fill material.

The analysis also suggested that settlements up to 115 mm may be expected at the centre of the dam. This may be compensated for by overbuilding the dam crest or providing an allowance for repair and backfilling of the dam over time.

Overbuilding with additional fill material was recommended for all areas of fill placement to account for potential settlement following construction activities.

4.1.8 Embankment Stability Analysis

The stability of the dam was assessed using the GeoStudio SLOPE/W analysis software. The 2007 CDA Guidelines require a minimum factor of safety of 1.3 for satisfactory performance of upstream and downstream slopes for short-term conditions immediately following construction and a factor of safety of 1.5 is required for the downstream slope stability for the long-term condition where the reservoir level is at normal operating levels.

The baseline stability model, consisting of the existing dam cross section and known or assumed material properties, confirmed that the existing dam configuration fails to meet the stability factor of safety requirements set forth in CDA 2007.

Various remedial embankment configurations were modelled and a multi-stage dam raise consisting of 5H:1V slopes and a granular toe berm achieved satisfactory factors of safety for the aforementioned short- and long-term conditions.

This embankment configuration was further evaluated against predicted seismic and design storm loading conditions. Using a peak ground acceleration (PGA) of $0.062 \times$ the natural acceleration due to gravity ($g = 9.81 \text{ m/s}^2$) the modelling indicated a factor of safety of 1.10 (1.0 required). With a reservoir water level of about 725.0 m, the modeling indicated a factor of safety of 1.51 (1.5 required). From the reservoir routing analysis presented in Section 4.1.5, the maximum reservoir level during the 200 year IDF is 724.8 m. This suggests that the factor of safety stated above is marginally conservative.



4.2 Water Management Requirements during Construction

Functional water management requirements provide guidelines for the Contractor to adhere to during construction. These conditions maintain the level of service requirements for downstream water users, provision of environmental requirements, and the maintenance of dam safety.

The following functional requirements must be adhered to during construction:

- The reservoir will be operated normally during the spring runoff prior to and during the construction period. The reservoir will be operated to store water to its FSL and provide water supply. Excess flood flows will be passed using the existing East LLO, and/or the West Spillway.
- The existing East LLO shall not be decommissioned until the rehabilitated West Spillway and East Spillway and new East LLO are commissioned.
- The East LLO must deliver 2,500 dam³ once per year from spring run-off inflows to the non-intensive Rush Lake back flood in late March to early April over four to five days (i.e., 500-625 dam³/day) via Rush Lake Creek.
- Water must be delivered to the downstream irrigators per the AAFC established schedule. It is understood the West LLO must deliver 5,926 dam³ over twenty days (296 dam³/day) to the Rush Lake Irrigation Project (RLIP) from late May to mid-June through Herbert Canal. The West LLO riparian needs have been assumed to be non-existent as Herbert Canal is man-made. The West LLO may be used as an operating spillway during flood events prior to the planned Highfield Dam rehabilitation project being completed and then reliance on the West LLO is intended to be removed during flood events.
- The East LLO must supply a total irrigation volume of 504 dam³ (5.6 dam³/day) from mid-May to mid-August. The East LLO riparian flows should be maintained without interruption.
- The reservoir must not be drawn down below the FTL (720.8 m).
- The regulatory agency in Saskatchewan is the Saskatchewan Water Security Agency (WSA), formerly known as the Saskatchewan Watershed Authority (SWA). The WSA is responsible for all aspects of water management including allocation to downstream users. AAFC will be required to obtain a 'Permit to Construct' from the WSA prior to the rehabilitation. It is also recommended that AAFC discuss the planned water management with WSA and local stakeholder groups prior to formal application.
- It is recommended that the flood inflow risks for critical periods of construction be re-evaluated during final design and when construction schedule modifications are made to the spillway or outlet structures.
- The tender and construction documents must clearly establish roles and responsibilities for reservoir management and operations during construction, including communication protocols, how normal and abnormal dam operation are to be undertaken, and should include planned gate lockouts when workers may be inside the low-level outlet conduits.



4.3 Operating Performance Criteria

4.3.1 West Low-Level Outlet

The role of the West LLO is to supply water to Herbert Canal. The West LLO should be capable of delivering an average discharge of $3.4 \text{ m}^3/\text{s}$ at the FTL of 720.8 m based on the requirements of the Rush Lake Irrigation Project. If there is insufficient water in the Highfield Reservoir to meet the needs, water may be diverted from Swift Current Creek to the Highfield Reservoir by way of the Swift Current Main Canal.

The West LLO must be equipped with a gate that may be set to permit incremental rates of discharge. The West LLO must also be provided with a means to isolate the gate and majority of the conduit for maintenance purposes (i.e. stop logs).

4.3.2 East Low-Level Outlet

The East LLO serves two functional roles – it supplies water to Rush Lake Creek for riparian and irrigation needs and it serves as the primary spillway for Highfield Dam.

AAFC has defined the following key design criteria:

- discharge capacity of $6.85 \text{ m}^3/\text{s}$ at the Fall Target Level (720.8 m);
- peak capacity of $8.5 \text{ m}^3/\text{s}$ at FSL (723.0 m); and
- minimum capacity of $0.28 \text{ m}^3/\text{s}$.

The East LLO must be capable of discharging at the rates described above at key times throughout operations.

The East LLO must have the capability to incrementally throttle flows while the outlet is not operating in spill mode. This may be facilitated using an adjustable gate. The East LLO must also be provided with a means to isolate the gate and majority of the conduit for maintenance purposes (i.e., stop logs).

4.4 Instrumentation and Performance Monitoring

It is anticipated that the existing vibrating wire piezometers will be maintained to monitor pore water pressures within the embankment and the dam foundation before, during, and after construction. The existing instrumentation at the dam also includes three slope inclinometers installed at mid-slope locations on the downstream face of the existing embankment. These were installed in combination with vibrating wire piezometers. It is anticipated the slope inclinometers will be useful in monitoring for possible movements in the embankment and foundation during the toe berm phase of construction; however, it is anticipated that routing of compaction equipment during the dam raise construction may render the slope inclinometers useless. It is anticipated that the vibrating wire piezometers will be operational after construction. Additional instrumentation may be required to adequately assess stability of the dam during the construction phase.

The need for additional instrumentation and frequency of monitoring during construction will be assessed during the detailed engineering phase. It is anticipated that more frequent readings and post-processing will be required during the active placement and compaction of fill material on the existing dam structure.

Trigger levels that initiate remedial action include the following criteria:

- absolute threshold numbers;



- unexpected change (i.e. numbers that have been static that suddenly change);
- rates of change; and
- engineering judgement of trends.

Trigger levels specific to the construction phase will be developed during the detailed engineering design phase. Appropriate remedial actions and/or emergency response protocols will be incorporated into the construction specifications following detailed design.

5.0 ASSESSMENT AND ANALYSIS

5.1 Cursory Structural Analysis

A cursory structural analysis was completed of the East and West LLO structures. This work is associated with raised embankment loading and considerations on the long-term useful life of components. The analysis was performed based on a review of the available data; including construction record drawings, inspection information, the rehabilitation report, and photos. Details of the cursory structural analyses are contained in Appendix C.

5.1.1 East Low-Level Outlet Structural Analysis

The East LLO is located near the east abutment of Highfield Dam and serves as one of the three discharge facilities for Highfield Dam. The East LLO consists of a series of structural components as per following order:

- reinforced concrete inlet structure, built in 1942 and modified in 1952;
- reinforced concrete upstream conduit with square shaped cross section, built in 1942;
- gate well structure, built in 1952, with additional modifications such as ladder and access hatch in 1992;
- reinforced concrete downstream conduit with square shaped cross section, which is a continuation of the upstream conduit, built in 1942;
- corrugated steel pipe (CSP) upstream conduit extension, built in 1950 and replaced in 1969;
- reinforced concrete stilling basin structure, built in 1951, which is no longer in service;
- CSP downstream conduit extension, built in 1991; and
- reinforced concrete stilling basin that serves as an outlet.

Some components of the reinforced concrete structure, such as jointing materials, have a shorter life and will require maintenance or replacement. Microclimate exposures, atmospheric zones, and submergence affect the life cycle. Attention should be given to its remaining life of the structure considering the age of the reinforced concrete conduit and the condition of its components. The American Concrete Institute (ACI 350R) states that when all relevant loading conditions are considered, the design should provide safety and serviceability with a life expectancy of 50 to 60 years for structural concrete.

When it comes to CSP, static potential readings could reveal the rate of corrosion between the soil and the culvert to establish the conditions. This type of information is a realistic method for estimating the remaining life and predicting the future performance of a metal component.



Corrosion protection is an electrical method of preventing soil-side corrosion and arresting corrosion on existing culverts. Cost tends to be the overriding consideration when deciding whether or not to provide corrosion protection. Industry practice is to avoid placement of cathodic protection on existing culverts that are structurally inadequate or are already in an advanced stage of corrosion; therefore, structural adequacy and corrosion status should be determined prior to implementation of corrosion protection. Alternative coatings or materials are also viable options and the same comparison of merits should be considered for those methods (the cost versus the estimated increase in life expectancy).

CSA A23.3 suggests conducting a thorough field investigation in order to perform a strength evaluation by analytical means. The field investigation should include:

- dimensions and details of members as-built condition;
- properties of the materials; and
- pertinent conditions of the existing structure.

This analysis is a cursory review and based on information provided by the AAFC. Soil properties were assumed after a review of the Highfield Dam Embankment Foundation Assessment Report produced by Golder issued in 2012. The assumptions are listed as follows:

Concrete:

- $f'_c = 10.3 \text{ MPa to } 19.4 \text{ MPa}$, carried out analysis for a range of compressive concrete strengths and compared the results to PFRA 1989, Table 3. The range of compressive strength values stated is understood to be from the 1952 outlet structure. In the absence of specific test results for the 1942 structure, when compared against the 18.4 MPa value used for the West LLO and not knowing what measure of quality control was provided at the time, this range is considered reasonable.
- $f_y = 276 \text{ MPa}$ (Concrete Reinforcing Steel Institute (CRSI) 1996).
- reinforcement details, as per drawing records.

Corrugated Steel Pipe (CSP):

- Assumed a 12 Gauge (0.1046" thick), 1,524 mm diameter, no thickness readings available on records. Assumed weakest profile for analysis for both CSP and multi-plate CSP at downstream and upstream.
- Poisson's Ratio = 0.33.

Soil Properties, taken from Table 6 of the foundation assessment report (Golder 2012) and consistent with Table 7-14 in Appendix B of the NHC report (NHC 2011):

- $\gamma = 21 \text{ kN/m}^3$, $\phi' = 25^\circ$, $c' \text{ (cohesion)} = 7 \text{ kPa}$ for embankment fill.
- $\gamma = 21 \text{ kN/m}^3$, $\phi' = 20^\circ$, $c' \text{ (cohesion)} = 0 \text{ kPa}$ for original embankment fill.

For the alluvial soils at the Highfield Dam under the East Low Level Outlet Conduit, the following assumptions were recommended:

- Allowable bearing capacity: 46 kPa to 60 kPa.



- Coefficient of subgrade reaction, $k_s \sim 5.5$ MPa/m (soils are soft to firm silty clays; typical values range from 0 – 5 MPa/m for soft clays, and 5 – 10 MPa/m for firm clays – (Canadian Geotechnical Society, 2006).

5.1.2 East Low-Level Outlet Assessment Result

The reinforced concrete conduit has about 27% of capacity requirement of the current codes based on latest edition of National Building Code of Canada (NBCC) (National Research Council, 2010). This statement considers the loading condition induced by the proposed raised embankment. The conduit is estimated to have about 36% of capacity requirement of the current code under the present loading condition.

Division A of Appendix A in the NBCC suggests that the successful application of code requirements to an existing construction becomes a matter of balancing the cost of implementing a requirement with the relative importance of that requirement to the overall code objectives. The degree to which any particular requirement can be relaxed without affecting the intended level of safety of the code requires considerable judgment on the part of both the designer and the authority with jurisdiction in each case.

- The existing CSP would theoretically be able to sustain the load increase of the embankment, assuming the condition of the pipe has not been compromised. Quantitative inspection records that include documentation about thickness readings, the quality of the coating, and the type of coating, especially at the downstream CSP would need to be completed to confirm the validity of the theory.
- Golder's geotechnical team has noted that the alluvial subgrade soils at the site are highly variable with a mixture of saturated clays, silty clay, sand, and silt. These soils are very susceptible to frost heave and settlement, and are not great foundation materials. This type of subgrade, coupled with a provision to add more fill, could produce settlement effects and result in a redistribution of loads.

If the existing East LLO is to be considered for modification, the following actions are recommended:

- Obtain more accurate information regarding material properties and construction records, and validate the analysis of the reinforced concrete conduit.
- Review or determine the risks and costs associated with culvert failure in the future.

The accuracy of given information, particularly in regards to the as-built records and material properties, may be limited. Golder made assumptions to carry out the analysis given the deficiencies in the available construction records at the time of analysis. A more rigorous analysis is necessary following any field data acquisition that may be considered as part of final or detailed design work.

A hydraulic analysis of the East LLO with a sleeved conduit indicated that the rehabilitated structure will not meet the required discharge capacity. The East LLO will be decommissioned and replaced by a new LLO and spillway combination located in the east abutment of Highfield Dam.

5.1.3 West Low-Level Outlet Cursory Analysis

The West LLO is located near the west abutment of the Highfield Dam and serves as one of the three discharge facilities for Highfield Dam. The West LLO is a reinforced concrete structure built in 1951 that includes a submerged intake and upstream conduit, a gate well riser accessed from the crest of the Highfield Dam to permit gate actuation and irrigation release control, and a downstream conduit and outlet structure. The existing West LLO was last visually inspected by the AAFC in 2010 and no significant defects were logged.



Similar considerations regarding the life span of structural components and field investigations of the East LLO also apply to the West LLO (refer to Section 5.1.1).

The following assumptions apply to the review of the West LLO:

Concrete:

- $f'_c = 18.4$ MPa, air entrained but low at 4% (AAFC 2012a, page 54).
- $f_y = 276$ Mpa (Concrete Reinforcing Steel Institute (CRSI) 1996).
- Reinforcement details, as per drawing records.

Soil Properties:

- As noted in Section 5.1.1.

The recently completed subsurface investigations did not investigate the extent of shale present beneath the West LLO; however, shale bedrock is expected to be near the ground surface at the West LLO:

- Historical reports recommend an allowable bearing capacity of 400 kPa for the shale.
- Typical k_s values for very stiff clay would be 30 – 80 MPa/m (Canadian Geotechnical Society, 2006); $k_s \sim 48$ MPa/m would be consistent with the historical bearing capacity for the shale.

5.1.4 West Low-Level Outlet Assessment Result

The reinforced concrete conduit has about 40% of capacity requirement of the current NBCC. This statement reflects the loading condition induced by the proposed raised embankment. The conduit is estimated to have about 50% of capacity requirement of the current code under the present loading condition.

The West LLO structure requires modifications to accommodate the extension of the embankment fill over the stilling basin. A cover slab addition was considered during pre-design and an assessment was completed regarding the impact of the additional loads to the structure. The results revealed an overstressing of the bottom slab. Golder recommends therefore, that the conduit sleeve pipe intended for the rehabilitation of the West LLO concrete conduit, be extended into the existing stilling basin and the annular area around the pipe backfilled with concrete to the top of the stilling basin walls. This will create a monolithic outlet structure and allow fill to be placed over the structure without additional remedial measures. However, the result will be a shortened stilling basin that will require consideration during detailed design.

5.2 Alternatives Analysis

A range of alternative concepts were developed following the site review. These alternatives were evaluated and compared to determine the optimal concept to be advanced to pre-design.

The alternative concepts considered for the rehabilitation of Highfield Dam are summarized in Table 4.



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Table 4: Alternative Rehabilitation Concepts

Case	East LLO	West LLO	Spillway	Other
Base Case	-	-	-	No changes
Option 1	Sleeve existing conduit	Sleeve existing conduit	Alter spillway to accommodate larger flow	-
Option 2	Sleeve existing conduit	Sleeve existing conduit	No change	Add auxiliary spillway on east side
Option 3	Replace East LLO with larger capacity conduit/spillway	Sleeve existing conduit	Alter spillway to accommodate larger flow	-
Option 4	Replace East LLO with larger capacity conduit/spillway	Sleeve existing conduit	No change	-
Option 5	Decommission existing conduit	Sleeve existing conduit	Alter spillway to accommodate larger flow	Construct new conduit/spillway in the east abutment

LLO = Low Level Outlet

Golder conducted a cursory review of the alternatives and Option 2 and 5 were selected to advance for further analysis. The primary distinguishing feature between the two options is the handling of the East LLO: removal of the existing East LLO and replacing it in the same location (Option 2) versus decommissioning the existing East LLO and replacing it with a new structure in the east abutment (Option 5).

These options were assessed using a modified Kepner-Tregoe (K-T) analysis to rapidly screen the options in terms of key development accounts (project economics, technical, project risk, environmental/regulatory, and operational). Each criterion was subdivided into a number of sub-accounts. Each sub-account was assigned a weighting value based on the apparent importance. The sum of the weighting values total to 100. The options were compared for each sub-account and assessed scores ranging from 0 (most preferred) to 10 (least preferred). The sub-account scores were then multiplied by the weighting values to determine the relative performance or preference for a specific component within a more general account.

The K-T analysis process involved members of Golder's technical design team as well as representatives from AAFC. This engagement process resulted in the selection of a single design concept that was advanced through preliminary design. The results of the modified K-T analysis are shown in Table 5.

A hydraulic analysis of the East LLO indicated that the minimum capacity could not be achieved due to the reduced cross-sectional area available to convey flows. For this reasons Options 2 and 5 were eliminated from consideration after the alternatives analysis.

Option 5 was selected for further study based on the results of the alternatives assessment described above.



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Table 5: Modified K-T Analysis

Account	Weighting	Assigned Sub-Account Scoring		Tabulated Sub-Account Scoring		Comments
		Option 2	Option 5	Option 2	Option 5	
Project Economics	45			280	170	-
Capital cost	30	6	4	180	120	Complete replacement likely higher capital cost; based on qualitative and collective experience.
Life cycle or maintenance cost	10	6	4	60	40	Option 2 assumed higher due to conduit through dam vs. abutment plus higher head/earth pressures.
Construction cash flow	5	8	2	40	10	Option 2 limits window to first year and winter construction. Option 5 presents deferral of capital to later years, if desired.
Technical	25			130	120	
Construction logistics / staging	10	8	2	80	20	Option 5 offers flexibility in schedule.
Availability / sourcing of materials	5	4	6	20	30	Option 2 – potential re-use of materials; less borrow. Option 5 – potential for spoil generation and requirement for select/processed backfill.
Pipe foundation conditions and support	5	3	7	15	35	Option 2 – known foundation conditions. Option 5 – need confirmatory ground investigations and foundation treatment.
Pipe selection and design	5	3	7	15	35	Option 2 – straight pipe; one specification; deeper placement. Option 5 – longer pipe; possible bends; shallower placement.



Table 5: Modified K-T Analysis (continued)

Account	Weighting	Assigned Sub-Account Scoring		Tabulated Sub-Account Scoring		Comments
		Option 2	Option 5	Option 2	Option 5	
Risk	15			115	35	
Schedule	5	10	0	50	0	Option 2 – risks schedule over-run and potential spill through earth cut spillway during spring melt or cofferdam needs to be constructed higher. Option 5 – offers mitigating options if schedule is compromised.
Water management	5	10	0	50	0	Option 2 risks inability to meet downstream stakeholder needs if schedule exceeded.
Geotechnical	5	3	7	15	35	Option 5 bears geotechnical uncertainty.
Environmental / Regulatory	5			35	15	
Care of water (permits and approvals)	5	7	3	35	15	Option 5 presents less time in-stream; approval may be more streamlined.
Operational	10			100	0	
Water management to Saskatchewan Watershed Authority	10	10	0	100	0	Option 5 meets downstream obligations year round.
Total	100			560	340	
Rank				2	1	

6.0 PRE-DESIGN ENGINEERING

The pre-design engineering stage is intended to define design, sequencing, and cost of the rehabilitation of Highfield Dam to a level that is suitable to inform AAFC's decision-making process with respect to capital investment, cash flow timing, and design certainty.

A high-level summary of the differences between the existing condition and the proposed post-rehabilitation scenario is provided in Table 6 through Table 10.

A comparison of key reservoir details is provided in Table 6. The table shows the resultant changes in key elevations, reservoir inundation, and capacity when the dam crest is raised. Note that the FSL remains constant in both cases.



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Table 6: Comparison of Key Reservoir Details

Metric	Existing Condition	Post-Rehabilitation Scenario
Full Supply Level (FSL)	723.0 m	723.0 m
Fall Target Level (FTL)	720.8 m	720.8 m
Top of Dam (TOD)	724.8 m	725.7 m
Reservoir inundation at FSL	5.2 km ²	5.2 km ²
Reservoir inundation at TOD	6.6 km ²	7.4 km ²
Reservoir capacity at FSL	15,130 dam ³	15,130 dam ³
Reservoir capacity at FTL	5,240 dam ³	5,240 dam ³
Reservoir capacity at TOD	25,430 dam ³	32,430 dam ³

A comparison of embankment details is provided in Table 7. The existing condition of the dam provides insufficient freeboard while the modified dam provides a 100 mm allowance including a provision for wave run-up. The geometry of the dam has also been modified with a longer crest length and a toe berm to help stabilize the embankment.

Table 7: Comparison of Embankment Details

Metric	Existing Condition	Post-Rehabilitation Scenario
Freeboard allowance	-1.0 m (insufficient at current dam crest elevation)	0.1 m
Embankment crest length	1,040 m	1,060 m
Embankment crest width	4.3 m (5.0 m with riprap)	6.0 m
Upstream face slope	3H:1V	3H:1V
Downstream face slope	3.5H:1V	5H:1V (toe berm face) 3.5H:1V (upper dam face)

m = metres

A comparison of design and performance for the East LLO is provided in Table 8. Note that the original design included a bottom outlet conduit whereas the modified design incorporates a submerged low-level inlet as well as a drop inlet spillway. The modified configuration sees a reduction in maximum discharge at FSL but it provides a significant increase discharge capacity when the reservoir level exceeds the FSL.

Table 8: Comparison of East LLO Details

Metric	Existing Condition	Post-Rehabilitation Scenario
Outlet type	Bottom outlet conduit	Bottom outlet conduit (low-level inlet) Drop inlet spillway
Regulation mode	Submerged flow / vertical slide gate	Low-level inlet: submerged flow / vertical slide gate Drop inlet: weir flow / conduit capacity
East LLO invert	717.03 m	717.0 m (riparian flow inlet) 723.0 m (drop inlet spillway inlet)
Maximum discharge at FSL	10.1 m ³ /s	8.5 m ³ /s
Maximum discharge at TOD	11.6 m ³ /s (top of gate well)	31 m ³ /s

m³/s = cubic metres per second; m = metres



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A comparison of design and performance for the West LLO is provided in Table 9. Both designs utilize a submerged bottom inlet; however, the capacity of the modified conduit has been reduced due to the smaller diameter sleeve that is installed within the conduit. The modified configuration is able to meet the downstream discharge requirements as set forth by AAFC.

Table 9: Comparison of West LLO Details

Metric	Existing Condition	Post-Rehabilitation Scenario
Outlet type	Bottom outlet conduit	Bottom outlet conduit
Regulation mode	Submerged flow / vertical slide gate	Submerged flow / vertical slide gate
West LLO invert	718.64 m	718.6 m
Maximum discharge at FSL	10.3 m ³ /s	5.8 m ³ /s
Maximum discharge at TOD	12.7 m ³ /s	7 m ³ /s

LLO = Low Level Outlet; FSL = full supply level; TOD = top of dam; m = metres; m³/s = cubic metres per second

A comparison of design and performance for the West Spillway is provided in Table 10. Both designs utilize an open channel spillway; however, the capacity of the modified spillway has been increased through refinements to the spillway channel and drop structure. The modified works also include an erodible fuse plug dam equipped with a fixed elevation sill. This allows the reservoir to exceed the FSL without triggering the West spillway until the spillway crest threshold is reached and the fuse plug erodes.

Table 10: Comparison of West Spillway Details

Metric	Existing Condition	Post-Rehabilitation Scenario
Spillway type	Free overflow; open channel	Erodible fuse plug; open channel
Spillway crest	723.08 m	724.9 m (erodible crest) 723.3 m (fixed sill)
Spillway capacity at TOD	58.4 m ³ /s	119 m ³ /s

TOD = top of dam; m = metres; m³/s = cubic metres per second

6.1 Embankment

The existing embankment is constructed as a uniform earth fill dam. The manner of construction was consistent with the state of practice at the time. However, modern standards utilize soil zones within dams. The zoning is intended to reduce the phreatic line within the dam while controlling the hydraulic gradient. The zone gradations are selected through the application of filter criteria between adjacent soil zones. Proper application of the filter criteria and material selection mitigate the potential migration (erosion or movement) of fines between adjacent zone materials. This reduces the potential formation of preferential seepage paths or a “pipe” through the embankment due to progressive erosion, or “worming”, back to the reservoir. From a review of AAFC documentation, this has been noted as a concern as early as the 1989 Phase 1 DSE that states: “The design and construction of the embankment did not include proper seepage control measures and, as such, a relatively high seepage potential exists for the embankment and foundation.”

The rehabilitation of the Highfield Dam embankment will include work to raise, strengthen, and improve seepage control measures and to improve freeboard during the passage of flood events. The raised cross section of the embankment will be facilitated by an extension of the existing 3:1 upstream slope, a 6 m wide crest set at EL. 725.7 m (0.9 m raise), a 3.5:1 downstream slope to a 5.0 m wide berm at EL. 720.4, and a 5:1 slope to the valley bottom.



The embankment will be constructed in two phases to allow for settlement and pore water pressure dissipation. Year 1 will see the construction of a toe berm which will consist of 1.2 m thick (minimum) free draining granular material and common clay fill source from local borrow on-site. Minor repairs to the upstream slope protection on the existing embankment will also be completed in Year 1. Year 2 will see the completion of the embankment raise with common clay fill, placement of riprap slope armoring on the raised upstream slope, road gravel on the crest and landscaping, top soiling and seeding the downstream slopes.

The 2012 foundation conditions and embankment geometry assessment lead to the development of an embankment design concept that satisfies the dam safety requirements described in the 2007 CDA Guidelines. The analysis suggested that a multi-stage embankment raise with a 3.5H:1V upper slope and a 5 m wide granular toe berm with a 5H:1V would yield acceptable stability. The calculated factors of safety for the proposed embankment geometric design are compared to the CDA 2007 requirements in Table 11.

Table 11: Embankment Stability – Calculated Factors of Safety

Loading Condition	Calculated Factor of Safety (Golder 2012)	Required Factor of Safety (CDA 2007)
Seismic loading = 0.062 g	1.10	1.0
Reservoir @ EL. 725 m (ref. Section 4.1.7)	1.51	1.5

g = gravitational acceleration (9.81 m/s^2); m = metres; CDA = Canadian Dam Associates; Golder Associates Ltd. (Golder)

6.1.1 Dam Cross-section Geometry

The existing top-of-dam elevation is 724.8 m. The proposed rehabilitation activities include raising the dam by 0.9 m to a crest elevation of 725.7 m. The dam raise is intended to address insufficient normal freeboard capacity. Additional modification is required to comply with the minimum factors of safety against slope instability as prescribed by the 2007 CDA Guidelines.

The foundation assessment report (Golder 2012) presented a number of recommended modifications to the existing dam cross section in order to rehabilitate Highfield Dam such that it conforms to the requirements of CDA 2007.

The recommendations are as follows:

- The embankment raise should occur in two stages. The first is to include the construction of a toe berm and the second will consist of raising the dam to an elevation of 725.7 m. The toe berm should extend along the entire length of the dam to improve seepage control and pore water pressure in the dam.
- The top of the Stage 1 embankment fill (toe berm) should be constructed to an elevation of 720.4 m with a face slope of 5H:1V.
- The toe berm should have a bench width of 5 m at an elevation of 720.4 m. The base of the toe berm should extend approximately 21.8 m beyond the toe of the existing dam.
- The toe berm should include the construction of a granular blanket drain with a minimum thickness of 1.2 m under the new fill.
- The Stage 2 embankment fill (bulk fill) should be constructed with a face slope of 3.5H:1V. Stage 2 consists of all embankment fill material placed above an elevation of 720.4 m.



The proposed modifications to the existing dam cross section are shown on Drawing Sheet No. 410 in Appendix D.

6.1.2 Toe Berm

A berm will be constructed at the downstream toe of the existing dam. This berm will consist of free-draining granular material and common fill sourced from local borrow. The berm will provide a suitable foundation for the placement of additional material that will facilitate an increase in the height of the dam.

The construction of the toe berm will require a measure of initial site preparation including the removal of topsoil and other materials that are not amenable to sound foundation construction. Once stripped, the foundation will be topped with a biaxial geogrid and non-woven geotextile fabric to provide stability and separation, respectively. The geosynthetic materials will allow construction traffic to navigate the across the soft soils known to exist along the downstream toe of Highfield Dam. The stripping and placement of drain material could be carried out before spring thaw to reduce complications associated with wet ground conditions; however, it is recognized that this may not be possible due to timing constraints. It is assumed that excavation and backfilling will be carried out using backhoes and trucks progressing across the toe of the dam in sections. Based on this assumption, potential problems with instability or seepage will be limited to one section at a time and can be addressed if or when they occur.

A 1.2 m thick zone of free-draining granular material will be placed to act as a toe drain. It is anticipated that the first lift will be relatively thick (in the range of 0.5 m) to provide a trafficable surface, with the remainder placed and compacted in 150 mm lifts. Non-woven geotextile fabric will be placed to provide separation at all interfaces between the drain material and adjacent embankment fill (original or new). Alternatives to the use of geo-textiles were examined, including the use of concrete sand as the granular drain, and the costing and material supply using geotextiles appeared to be more favorable. This may be re-examined during detailed design.

Seepage is expected to occur at the toe of the existing dam embankment during construction. The seepage rate is expected to decrease as the reservoir levels decrease. Placement of fill for the dam raise will cause flow rates to temporarily increase, but they are expected to dissipate once the drain is in place and steady state conditions are re-established. Most of the current seepage through the dam is obscured by vegetation and wetlands at the downstream toe of the dam. Removal of the vegetation and topsoil prior to construction will result in a more noticeable accumulation of seepage at the toe of the dam. Pumping may be required to remove water that accumulates.

Review of piezometric data between October 2011 and February 2013 (SNC Lavalin, 2013) indicates that the pressures measured in the vibrating wire piezometers are influenced by reservoir levels (provided by AAFC on March 8, 2013). The maximum change in the piezometric measurements does not vary more than 8 kPa, which correlates to an increase in pressure head of less than 1 m. Based on this information and review of the previous analysis by Golder in 2012, it is expected that maintaining the reservoir levels at or below the full supply level during construction will not negatively affect the overall stability of the dam. Reducing the reservoir to minimum levels required will reduce seepage rates and pore water pressures at the toe of the dam if they are a concern during construction.

Pore water pressures within the embankment must be monitored before, during, and after the placement of fill. Pore water pressures must be allowed to dissipate during fill placement. The 2012 SEEP/W model by Golder predicted that a minimum lag time of 30 days was required to fully dissipate increased pore water pressures



caused by the placement of fill on the dam for each stage of construction. However, the seepage model assumed that the Stage 1 fill was placed instantaneously. Since the fill placement will occur in compacted lifts, some pore water dissipation will occur as construction progresses. Placement of additional fill prior to the dissipation of the pore water pressures may cause an increased rate of settlement, slope instability, and additional seepage at the toe of the dam. Monitoring of the instrumentation during construction will be important to maintain adequate stability of the embankment during construction.

Review of the piezometric data indicates that pressures experienced at the toe of the dam between November 2011 and February 2013 were greater than that predicted by the 2011 SEEP/W model by Golder. The 2011 SEEP/W model assumed an average hydraulic conductivity for the alluvial soils below the dam, which provided a general prediction of the groundwater flow and soil behaviour. The alluvial soils at the site contain discontinuous seams of sand, silt, and clay which affect the movement of water and the generation of pressures; as such, prediction of pore water pressures at specific piezometer locations is difficult. Installation of additional monitoring devices, including but not limited to slope inclinometers and additional piezometers, may be required. Further analysis should be carried out during detailed design of the dam raise to determine the threshold values required to be used during construction.

6.1.3 Embankment Fill

Additional fill material will be required to raise the dam to an elevation of 725.7 m. The second phase of fill placement may occur once it has been demonstrated that higher pore water pressures induced by the placement of the toe berm fill have dissipated below an acceptable reading.

The bulk fill may be locally-sourced clay provided that it meets engineering specifications to be defined during detailed engineering design. The material will be placed in 150 mm compacted lifts. Pore water pressures must be monitored to confirm that adequate stability of the dam is maintained throughout construction.

It is recommended that additional fill be placed on the dam crest to account for potential future settlement. An 'overbuild' of 150 mm is assumed for all areas of the dam.

Topsoil should be placed on the completed downstream dam face. The face should be seeded to establish a vegetated surface as soon as practicable to reduce the potential for surface erosion.

A gravelled traffic surface will be provided along the crest of the dam to facilitate maintenance and surveillance of the dam during operations.

6.1.4 Upstream Face Armoring

The field assessment completed in August of 2012 suggests that the upstream armor has generally performed well given the age of the structure. The existing armor consists of field stone and is sub-rounded with a ratio of the long axis to the thickness (aspect), of less than 2. The armor appears gap graded in areas along the length of the embankment with observations of occasional void and loss of bedding. The upstream face also shows signs of wave action with armor displacement. This is particularly visible in the area surrounding the West LLO. The visual assessment for gradation of the existing armor presents an estimated nominal dimension (d_{50}) of 250 - 300 mm with sizes ranging from about 75 mm to the occasional stone as large as 650 mm in diameter. Record drawings appear to indicate the nominal diameter of armor is 300 mm. This is consistent with field observations.



The upstream face of the dam will increase in length with the addition of the dam raise fill. The raised embankment face must be armored with riprap to resist degradation by potential wave action during flood events and other erosive processes. The rehabilitated upstream dam face will consist of three armor zones as described further below.

The first riprap zone is below FSL and is assumed to remain unchanged; however, spot rehabilitation of deficient areas within this zone may be required. Full rehabilitation has not been considered in the pre-design however a provisional sum for spot rehabilitation has been included in the cost estimate.

The second riprap zone will consist of an 800 mm thick layer of riprap consisting of a blend of existing riprap ($d_{50} = 300$ mm) and imported riprap ($d_{50} = 400$ mm) each assumed to make up 50% of the total required thickness of the armor. The nominal dimension of the riprap armor was selected using Hudson's Equation (Hudson 1961) with a consideration for resisting the 1 in 1000 year wave run-up. The existing armor will be removed from the dam face and blended with the imported riprap. A two-stage filter consisting of a 200 mm thick layer of well-graded granular bedding and a 150 mm thick sand filter will be placed on the exposed dam face prior to the placement of the blended riprap armor.

The upper, raised portion of the dam is expected to experience lesser erosive forces corresponding to a 200 year reservoir water level combined with a 1 in 10 year wave run up event. Hudson's Equation suggests using armor with a nominal dimension of 300 mm; correspondingly, a 600 mm thick layer of $d_{50} = 300$ mm armor will be placed in this zone. The granular bedding and a sand filter is identical to that described for the previous armor zone.

The upstream face will be groomed to provide a smooth transition between the new and existing riprap armor. The areas surrounding the East LLO and West LLO gate well structures will receive additional armor as the areas surrounding the risers have demonstrated to be prone to erosion from wave action and to surface runoff.

The selected thickness of armor is selected as 2.0 times the selected d_{50} . There are published documents (Smith, 1995) that have used 1.5 times the selected d_{50} and may be considered during detailed design. This will translate to a measurable savings in the cost to import field stone to the site.

6.1.5 Borrow Source Evaluation

6.1.5.1 General

AAFC has identified a potential fine grained soil borrow source for the embankment component of the Highfield Dam rehabilitation. The potential borrow source is located adjacent to the east abutment of the dam in Legal Subdivision (LSD) 1 and 2 of Section 1, Township 16, Range 11 W3M. The scope of work for pre-design includes the provision of recommendations for embankment construction, recommendations for borrow area management, and the confirmation of the borrow source and suitability for embankment borrow materials. The recommendations for embankment construction are addressed in Section 6.1.3 – Embankment Fill. The following sections address the latter two scope items.

6.1.5.2 Borrow Area Management

The potential borrow area encompasses the upper portion of the Rush Lake Creek valley extending down the valley slope. Access to the proposed borrow area is from the east dam access road and by future construction access traversing north of the east abutment area. The general location of the borrow area is shown on Drawing



Sheet No. 300 – Site Plan. The pre-design identified a trapezoidal borrow configuration to parallel the valley alignment. This arrangement will permit:

- ease of access – the borrow area is immediately north of the east abutment, construction staging areas and waste areas (south of the east abutment);
- cost effective material extraction and processing – the proximity to the valley slope will enable the development of a borrow extraction face. The borrow face permits mixing across material horizons or the loading of haul vehicles from upper and lower tiers;
- handling and transport – the proximity of the borrow area to both the lower proposed berm and crest facilitates the development of a haul circuit across the dam site;
- borrow area management (care of water) and site reclamation – the borrow location and topography will be conducive to preventing run-on and directing runoff for site water management. The topography will permit reclamation of the area to better mimic the natural valley slopes and features including naturalized drainage courses.

Notable concerns identified in study documentation provided by AAFC include the proximity of Archeological Site EbNu-29 and areas of observed occurrences of wildlife species and vegetation. Both EbNu-29 and the areas of observed occurrences are outside the borrow area footprint. The HRIA study (Bison 2012) conducted a foot survey in and around the potential borrow area and did not identify any cultural material. They concluded “no heritage sites will be impacted by the proposed development”. Biological surveys completed by KGS in 2010 and Jacques Whitford in 2003 noted potential effects on native vegetation habitat by construction activities. Construction will be confined to specific areas to limit disturbances as much as possible to the surrounding vegetation communities and follow best management practices (refer to Section 7.3).

6.1.5.3 Borrow Source Confirmation and Suitability

The Highfield Dam is located in the Rush Lake Creek alluvial floodplain, which is characterized by surficial stratified deposits of sand, silt and clay overlying the eroded glacial till plain. In the vicinity of the reservoir, the till and stratified drift, also known as the Saskatoon Group, is generally less than 10 m in thickness and is underlain by the Bearpaw Formation (Maathuis, 2007). The Bearpaw formation is generally characterized by soft, grey, non-calcareous marine silt and clay materials which are commonly known as shale.

AAFC carried out field drilling programs at the Highfield Dam in 1991, 2002, 2009, and 2010. Boreholes closest to the proposed borrow area are boreholes C52, C53, C54, and BH40. All four boreholes were drilled in the fall months of 2009, classified the near surface soils as Clay (CI) till, and were noted to be dry the same day they were drilled. Later drilling programs carried out by NHC/MDH in early 2011 and by Golder in late 2011 did not drill boreholes in or near the borrow area.

The soil classifications for the boreholes near the proposed borrow area suggest the likelihood of the near surface availability of Clay (CI) soils and their suitability for use in the construction of the dam rehabilitation to be high. In order to qualify the suitability, Golder recommends a borrow area test pit program consisting of three (3) to five (5) excavations be carried out to provide a visual assessment and classification and to provide bulk sample collection for index testing (moisture, plasticity, grain size analysis by hydrometer) and proctors. The test pits will also help verify the areal extent of suitable materials and assist in the evaluation of potential waste.



6.1.5.4 Granular Material Sources

The rehabilitation of the Highfield Dam will require granular bedding/filter material, granular material for the construction of the toe berm, gabion rock for spillways, and rip rap for shoreline armor. A granular material source on site is not likely to be of suitable quality or quantity. Granular sources of material are likely best procured from area suppliers. Rip rap and armor for gabions is likely to be field stone and sourced from the region and require early (long lead time) procurement, particularly for larger volume requirements.

6.2 Spillway System

The proposed spillway system for the rehabilitated Highfield Dam project will consist of a drop inlet pipe spillway on the east abutment, referred to herein as the East Spillway, and modifications to the existing earth spillway on the west abutment, referred to herein as the West Spillway. A new East LLO will be constructed with a drop inlet spillway. The existing East LLO will be decommissioned at the end of the project.

The East and West Spillways are discussed in Sections 6.2.1 and 6.2.2 respectively.

6.2.1 East Spillway and Low Level Outlet

A drop inlet pipe spillway is proposed for the East Spillway. Components of the drop inlet spillway include:

- a 2.8 m square reinforced concrete riser;
- a 100 m long, 2.0 m diameter HDPE conduit; and
- a conventional diverging wall outlet structure of reinforced concrete.

The new East LLO will be constructed with a drop inlet spillway and will include a 1.5 m HDPE intake conduit and a reinforced concrete wet-well constructed near the crest of the dam. The wet-well will contain a 1.5 m square slide gate and gate actuator. The new East LLO will join the drop inlet conduit about 34 m downstream of the inlet riser by way of a short conduit section, pipe transition fittings and concrete thrust block. A floating log boom will be installed to identify the works for boater safety.

The drop inlet will be constructed in dry conditions within the east abutment of the existing dam. The construction area for the new works may be isolated using a cofferdam built from sand bags placed over existing ground or excavated subgrade. The sand bags may be constructed to the existing dam crest elevation. The existing East LLOs and West Spillway will be used for flood management. Other cofferdam arrangements may be considered as part of detailed design. If lower cofferdams are considered during detailed design, a review of the flood risks during construction is recommended.

The excavation will likely require seepage and runoff control by pumping. As such, the overall stability of the excavation is recommended to be re-evaluated particularly if the design, construction schedule or water management operations change. Refer to Section 7 for a list of best management practices that must be considered through detailed design and construction.

The drop inlet structure may be founded on native soil or shale provided that the foundation has been prepared to achieve the desired performance requirements (to be defined in detailed engineering design). The riser structure will be cast-in-place.



The 2.0 m diameter double-walled HDPE pipe will be laid in a trench excavated through the east abutment. Seepage collars and a filter diaphragm will be provided to reduce the likelihood of piping failure through the bedding and soil backfill around the outer circumference of the conduit (to be specified in detailed engineering design). The pipe will discharge into a concrete stilling basin to dissipate energy and prevent erosion at the toe of the dam.

The cofferdam will be removed once the East Spillway drop inlet, the new East LLO gate well, conduit, and stilling basin have been constructed and backfilled. Stop logs will be installed in the East LLO gate well to prevent discharges through the conduit prior to the completion of the East LLO. The East LLO pipe extending into the reservoir will be bolted to the flanged connection provided near the upstream side of the wet-well under wet conditions, and the pipe will be backfilled with clean gravel. The stop logs may be removed once the pipe segment has been secured.

The components of the proposed East LLO are described in the following sections.

6.2.1.1 Drop Inlet Structure

The details of the drop inlet structure are shown on the 500 series pre-design drawing sheets. The drop inlet structure consists of a square concrete chamber and an outlet pipe. The key design parameters of the drop inlet structure that were used in the hydraulic analysis are outlined below:

- a 2.8 m square concrete drop inlet;
- effective weir length is approximately 10.9 m (four-way drop inlet);
- smoothed entrance crest curve radius of 0.4 m to reduce entrance losses;
- bottom invert elevation is set at 717.0 m and the crest of the drop inlet structure is positioned at FSL (723.0 m), resulting in a concrete drop inlet structure with a height of 6.0 m; and
- a 2,000 mm diameter HDPE pipe (roughness of 0.0015 mm) with a length of 100 m exits the drop inlet structure and runs at a constant slope of 0.5% before connecting to a stilling basin above Rush Lake Creek (outlet invert is 716.5 m).

The discharge coefficient over the weir was determined by a relationship proposed by C.D. Smith (Smith, 1995) for drop inlet structures based on the design parameters provided above. Discharge coefficients for this structure are higher than an ogee crest weir and range from 2.4 to 3.0 depending on the reservoir elevation.

The rating curve of the drop inlet structure is a combination of the weir flow that is exhibited from the drop inlet structure and the pipe flow exhibited in the pipe which outlets to Rush Lake Creek. The rating curve for the proposed drop inlet structure (without the riparian flow) is shown in Figure 6.

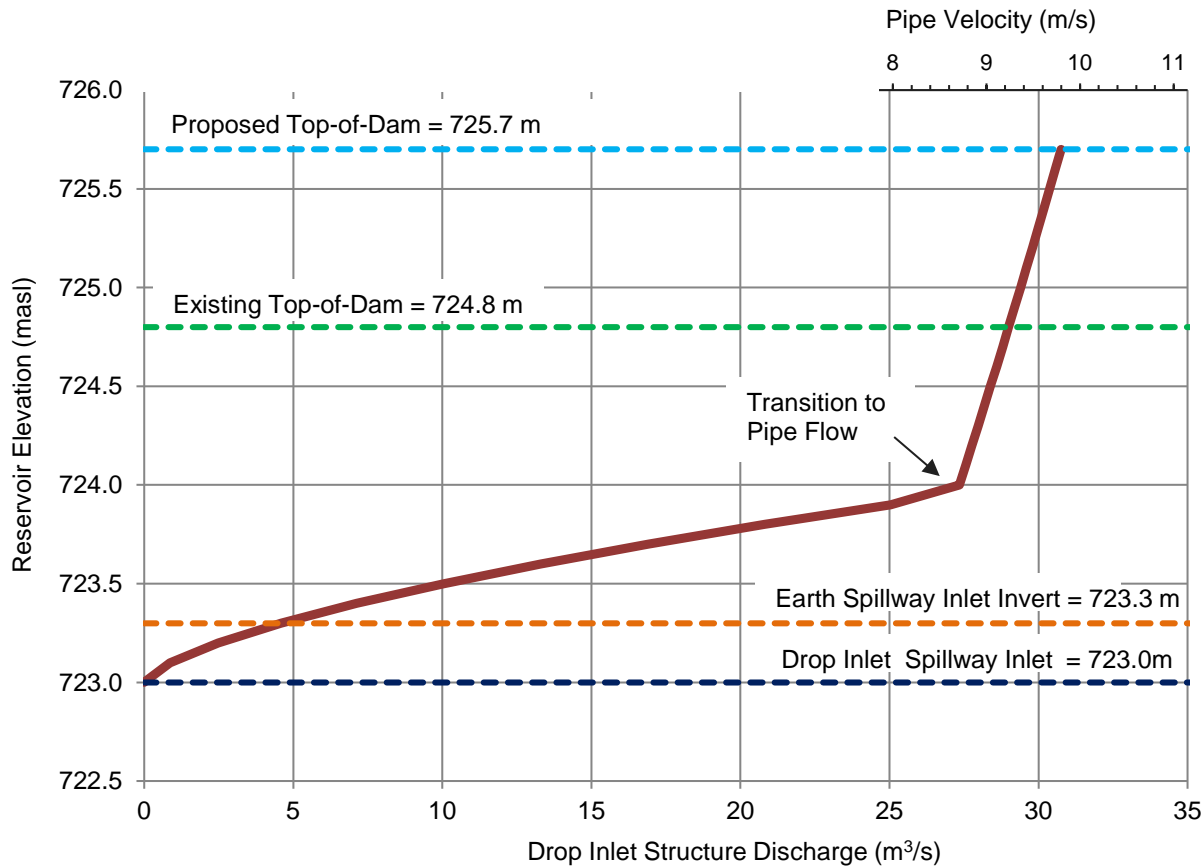


Figure 6: Design Rating Curve for the East Spillway Drop Inlet Structure and Pipe Velocity

At an approximate discharge of $27.3 \text{ m}^3/\text{s}$ the weir flows from the drop inlet structure 'drown out' and the flow becomes controlled by capacity of the downstream pipe (outlet control). The velocities in the pipe become very high when discharges exceed $27.3 \text{ m}^3/\text{s}$. This is a result of the flow becoming regulated by the outlet pipe characteristics (i.e. slope, pipe material, length).

Measures to protect against erosion and scour are required at the outlet to Rush Lake Creek due to the high exit velocities (approximately 9 to 10 m/s) generated during peak discharge. A stilling basin will be provided to dissipate energy and reduce velocities prior to discharging into Rush Lake Creek.

6.2.1.2 Conveyance Conduit

A new conveyance pipeline will be constructed to transfer water through Highfield Dam. A 100 m long reach of 2,000 mm diameter double-walled HDPE pipe (Weholite) will be required. Weholite has been selected based on its favourable strength-to-weight ratio, resistance to corrosion and biological growth, leak resistance, and ease of installation.

A concrete seepage collar or similar seepage intercept will be required at the point where the pipe enters the body of the earth fill section of Highfield Dam. The collar is intended to mitigate internal erosion and backward piping erosion dam failure mechanisms.



The HDPE conduit will be placed in a trench that will be excavated through the east abutment of Highfield Dam. Sandbagging may be utilized to isolate the area on the upstream side of the excavation. The placement of pipe should be in accordance with common installation standards (i.e. D2321, *Standard practice for Underground Installation of Thermoplastic Pipe for Sewers and Other Gravity Flow Applications*). The trench should be sub-excavated below the design grade of the pipe to allow for the placement of a suitable foundation and pipe bedding material. Side-wall bracings should be provided as required to maintain safe working conditions as per local regulatory requirements.

Based on AAFC record borehole (BH) data drilled August 2002 and provided in the NHC 2011 report, clay shale bedrock is anticipated to be near the existing ground surface and as shallow as 1 m based on BH C30 and C31. The proposed conveyance conduit alignment is likely to be required trenched through clay shale bedrock. The current understanding of the bedrock geology suggests that slaking (disintegration of fine-grained rock) may be a consideration for fresh bedrock that is exposed to the atmospheric conditions during construction based on past testing undertaken by AAFC. It is recommended that slake durability tests be conducted on core samples during later geotechnical site investigations. This will help to prescribe appropriate mitigation measures prior to construction. The degradation of the rock must be managed by appropriate rock reinforcement and retention techniques which may include covering with a cement-sand grout, pneumatically-applied mortar, or concrete (Robin Fell, 2005). A simple mud slab could be poured; however, sealing trench walls may require more elaborate measures. Concrete cradles should be placed at regular intervals along the length of the pipe. The cradles may reduce the likelihood of internal erosion and backward erosion piping. The number of seepage collars required depends on the length of pipe and the projection of the collars normal to the long axis of the pipe (Linsley, 2002).

A filter diaphragm should be constructed midway between the centerline of the embankment dam and the downstream toe. A filter diaphragm is a zone of well-graded, clean sand constructed around the conduit. The diaphragm intercepts water that may flow through compacted fill surrounding conduits or water that may flow along the interface between the conduit and surrounding fill (USDA – NRCS, 2007). Note that the diaphragm has not been designed during the pre-design phase. Likewise, the detailed design must evaluate the potential for seepage along the conveyance pipeline and consider mitigation strategies including material backfill. Bedrock treatment/protection during construction may only be properly assessed following more intensive investigations employing boreholes and test trench investigations along the pipe alignment as part of final design.

Pipe segments should be handled as per the manufacturer's instructions. The pipe segments may be joined by manual extrusion welding or automatic welding. Initial backfill should consist of angular, well-graded material (structural fill) placed in 150-300 mm lifts placed evenly on both sides of the pipe. Backfill material into the haunch area should be shovelled and hand-tamped to ensure that no voids remain. The structural fill should be placed and compacted in lifts to a minimum of 300 mm above the crown of the pipe. No material should be machine compacted directly over the pipe until the backfill has exceeded 300 mm above the crown of the pipe. The horizontal and vertical pipe dimensions should be monitored throughout fill placement.

Compacted common fill may be used to complete the backfill of the trench using standard specifications for compacted impervious embankment applicable to dam construction.



6.2.1.3 Stilling Basin

A concrete stilling basin will be required to dissipate energy and reduce exit velocities as water exits from the East LLO conduit. The stilling basin design is based upon the design approach presented in the 1995 metric edition of the publication titled *Hydraulic Structures* (Smith, 1995). The basin is approximately 25 m long flaring from 2.0 m wide at the end of the conduit to 9.0 m wide at the outfall to Rush Lake Creek.

The basin includes seven (7) chute blocks at the end of a 3H:1V drop section, eight (8) baffle blocks in the bottom basin, and a ramped end sill at the downstream end of the structure. The basin design reflects the tailwater curve developed as part of a recent hydrotechnical assessment of Highfield Dam (Golder, 2011). The rating curve was developed using a steady-state hydraulic model (HEC_RAS). The rating curve indicates that the tailwater elevation in Rush Lake Creek at the design discharge of 32 m³/s is 718.0 m. The basin floor will be constructed below the natural depth of Rush Lake Creek to contain the hydraulic jump under high flow conditions.

The stilling basin will be a standard monolithic structural design. A conventional under-slab drainage system has been assumed in the pre-design although this may be re-evaluated during detailed design. Under-slab drainage is incorporated for potential seepage control, to dissipate hydrostatic pressures, or uplift pressures that occur during hydraulic jump operation. The stilling basin may be founded on clay shale material which could be subject to swelling pressures or deterioration due to weathering and exposure to water. Over-excavation of foundation was assumed in the order of 1.2 m during pre-design and the under-slab backfill was assumed to consist of engineered granular fill. The seepage, drainage, and foundation protection measures are recommended for reassessment during detailed design.

Standard fence details have been assumed for public safety. A short excavated channel will join the structure to the existing Rush Lake Creek. A 50 m section of this channel will include riprap armor protection.

Plan and section views of the East LLO stilling basin are shown as Section 5 on Drawing Sheet 515.

6.2.1.4 Low-level Intake

The East Low-Level inlet will consist of a 1,500 mm diameter HDPE pipe that projects into Highfield Reservoir. The upstream end of the pipe will be secured by a pair of H-piles driven to design capacity or refusal in the lake sediments and foundation materials beneath the reservoir. Angle iron brackets will prevent vertical movement. The pipe will be allowed to translate along the long axis of the pipe to allow for thermal expansion/shrinkage during construction and operations.

The inlet side of the pipe will be equipped with a flanged end to allow divers to bolt a plate bulkhead onto the opening to facilitate downstream surveillance and maintenance.

A gate well equipped with upstream and downstream slide gates will be constructed as shown on Detail 1 on Drawing Sheet 510. The downstream slide gate will be installed in a positive seating position. This allows for the complete isolation of the downstream outlet works when reservoir water levels are below FSL. The secondary, upstream gate is provided to isolate the gate well chamber so the primary gate may be inspected and serviced. The gate well will be accessible via an at-grade ramp from the rehabilitated embankment crest. The low-level pipe will tie into the primary East Spillway conduit immediately downstream from the gate well.



The gate and conduit will allow the necessary riparian flow requirements to be managed based on a range of gate apertures (% gate opened). A rating curve was developed under the assumption that there will be no submergence of the gate within the gate well structure. It has been assumed that the low-level gate will be fully closed when the reservoir reaches FSL to facilitate the analysis. Actual gate operation and performance will be assessed during detailed design.

A rating curve accounting for a range of gate openings was developed based on the following assumptions:

- water depths in the concrete drop inlet structure based on discharges through the outlet pipe (length = 100 m, pipe diameter = 2,000 mm, pipe slope = 0.5%, Manning's roughness coefficient = 0.009);
- low level HDPE pipe which has a length of 25 m, a slope of 0%, and an invert elevation of 717.0 m; and
- exit losses based on rapid contraction and expansion from various sized openings.

A series of rating curves of the East LLO for various gate opening conditions is presented on Figure 7. This configuration meets the required discharge criteria.

An approximate gate opening of 65% corresponds to a discharge of $8.5 \text{ m}^3/\text{s}$ at FSL. An approximate gate opening of 70% is required for a discharge of $6.85 \text{ m}^3/\text{s}$ when the reservoir is at the FTL.

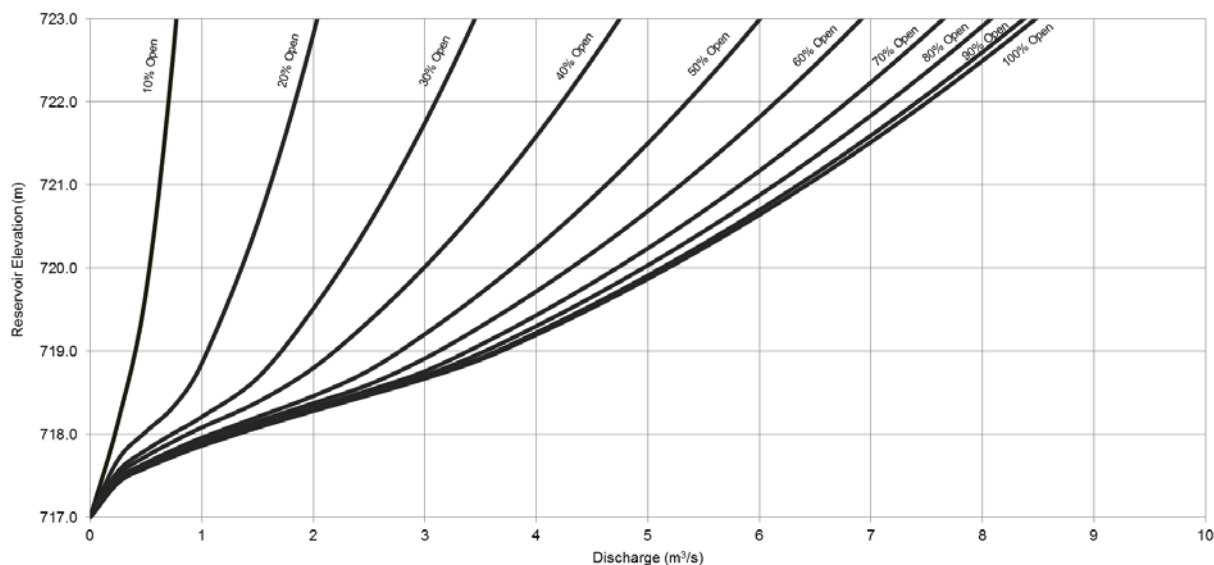


Figure 7: Rating Curves for the East Low Level Outlet Controlled by a Gate Under Various Opening Conditions

6.2.2 West Spillway

The proposed West Spillway is a modification of the existing earth spillway. The West Spillway will be a 25 m wide earth cut channel that will include the following key modifications/components:

- improvements to the upstream spillway reach (including a pilot channel to drain water away from the upstream toe of the fuse plug);
- a new erodible fuse plug equipped with a control sill at the approximate mid-point of the spillway channel;



- a rock-lined chute to convey flows from the spillway channel to Herbert Canal; and
- a side channel relief spillway to pass overflow in Herbert Canal into the valley floor at a safe distance from the toe of Highfield Dam.

The existing earth-cut overflow spillway provides the capacity for handling large flow events. The spillway conveys flood water from the reservoir to the main Herbert canal. The initial 200 m of the spillway is generally flat but follows a gradual slope past the existing Highfield Dam centreline to discharge down the cut slope of the main Herbert canal along the Rush Lake Creek's western valley slopes. The west and east low level outlets currently can serve as secondary spillways but have limited capacity and have manually operated gates. Figure 3 shows a plot of the outflow rating curve for the existing Highfield Dam Earth Spillway based on AAFC's Drawing No. 116036A dated April 1992.

The dimensions of this existing earth-cut spillway (dimensions and elevations were developed from Drawing No. 92910 and ground survey data from 1992) were compared with available surface information (from 0.5 m contour data provided by AAFC). The elevation difference between the design drawings and developed surface ranges from 0.2 to 0.4 m which is mainly because the developed design surface was based on 0.5 m contours. Since the provided AAFC rating curve was developed on the basis of the initial design and a detailed ground survey, it is expected that this rating curve is representative of the current spillway capacity.

From the site visit the spillway was noted to be maintained in a reasonable condition, was fully grassed, and there was little indication of usage other than some erosion gullying on the spillway outlet into Herbert Canal. Cattle freely roam and graze in the area as it offers ease of access to drinking water from the reservoir. Impressions left by cattle hooves and trails are evident on the spillway side slopes to natural ground. These minor observed anomalies will not adversely affect the passage of flood flows. For the purposes of this analysis the developed rating curve from AAFC was deemed suitable for any flood routing exercises.

The existing maximum discharge at the existing top of dam is $58.4 \text{ m}^3/\text{s}$ if the road that crosses the spillway along the dam crest is eroded. The rating curve for the AAFC developed rating curve was extrapolated and a maximum discharge of $119 \text{ m}^3/\text{s}$ was estimated on a proposed top of dam of 725.7 m.

On examining the existing spillway centreline profile provided by LiDAR topography, the existing ground varies in elevation and the control sill to the spillway is not clearly discernible. The highest ground elevation appears to be 723.3 m, with the predominant ground elevation immediately before the Highfield Dam centreline as 723.0 m.

Downstream of the control sill, the channel bed slope will immediately fall away at 5% for a distance of 10 m but transition to a flatter 0.5% bed slope. The 5% slope will be armored with rock having a nominal diameter of 400 mm and placed to a thickness of 600 mm with a granular bedding layer 150 mm thick providing a transition and filter to the native soils. The 0.5% slope to the rock lined spillway is anticipated to be grass lined.

The existing access road crossing of the spillway is located along the centreline of Highfield Dam. A new access road will be reconstructed to align with the proposed dam centreline but remain level to the existing spillway channel bed elevation at the point of crossing. A nominal 75 mm thick road bed material provided for the crossing is not anticipated to create conveyance issues during flood passage.

Presently, flood flows conveyed down the overflow spillway will enter the canal and quickly exceed the design capacity of the canal. If the flood flows approach the IDF, insufficient conveyance capacity of the canal will



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generate backwater to the West LLO and overtop the canal embankment nearest the Highfield Dam; a low point identified by LiDAR topography. The risk to scour the dam at the toe of the west abutment is high and considered unacceptable.

The rating curves for the existing earth spillway and the proposed West Spillway are shown on Figure 8.

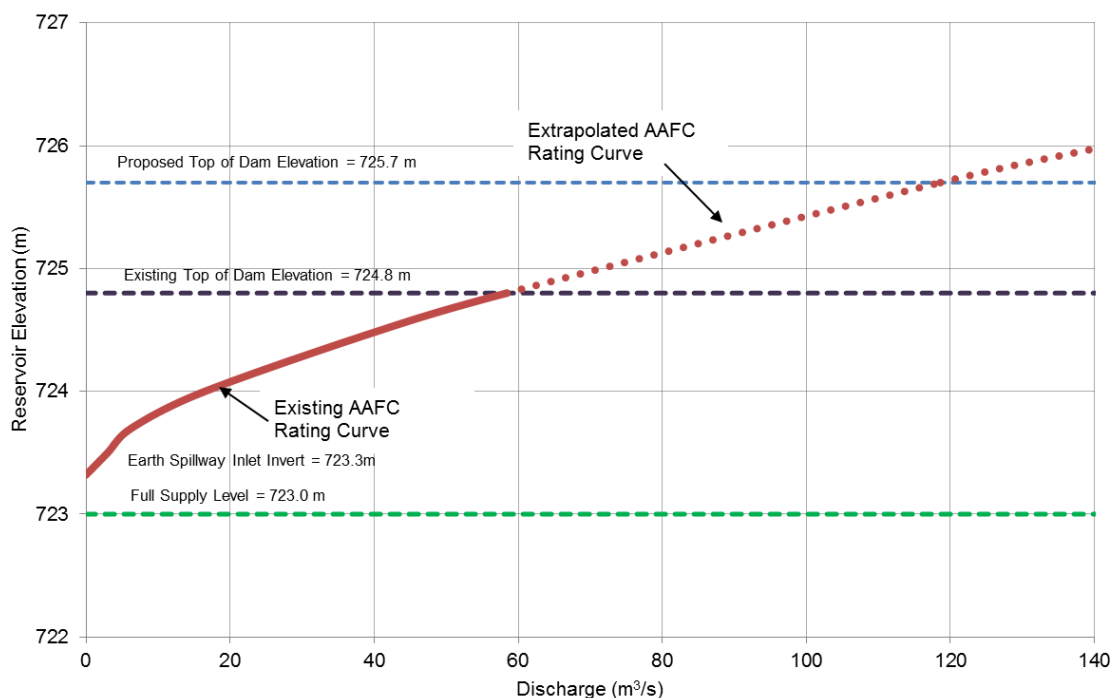


Figure 8: Existing and Compared Rating Curves for the West Spillway

6.2.2.1 Improvements to the Channel

The remedial measures proposed for the overflow spillway involve trimming the spillway channel to a new true dimension bed width of 25 m and 3H:1V side slopes. A new control sill will be located about 40 m upstream of the proposed Highfield Dam centreline set at an elevation of 723.3 m and will consist of a buried gabion basket with a lean mix concrete cap.

The overflow spillway inlet channel bed slope will essentially be zero at an elevation of 723.0 m and is anticipated to be grass lined. A pilot channel graded back to the reservoir will be excavated to facilitate back drainage to the reservoir when reservoir levels fall below FSL.

The spillway bed slope beyond the fuse plug arrangement is 0.5% and is intended to be grass lined up to the head of the rock-lined chute drop structure that will convey water from the spillway to Herbert Canal.

6.2.2.2 Fuse Plug Dam

A fuse plug arrangement capable of utilizing the remaining storage of the reservoir during lower flood frequency events is considered an economic alternative to more elaborate concrete structures. A fuse plug maintains valuable water storage while reducing the frequency of spillway operation and the potential costs that can be



incurred following an event. Fuse plugs are generally designed to operate for floods with a frequency of no less than 100 years.

A fuse plug arrangement set over the control sill location has been considered in the pre-design based on Highfield Dam's IDF corresponding to a 1 in 200 year peak flood event. The fuse plug would erode after exceeding the 100 year inflow. The fuse plug would have a crest elevation of 724.5 m and pilot channel elevation of 724.3 m to retain the 100 year flood. The maximum reservoir water level during the 100 year flood is 724.25 m, assuming the reservoir is at FSL prior the flood event. The pilot channel is necessary in order to concentrate the flows and initiate back scour over the fuse plug that will eventually erode the entire fuse plug.

The fuse plug will be constructed of natural earthen materials. The pre-design upstream section of the fuse plug presents armor protection with a nominal diameter of 200 mm and includes bedding material. The internal structure will consist of an inclined impervious core and reverse transition filter arrangement downstream leading to an erodible granular fill material that makes up the majority of the embankment fill. The granular material is intended to be non-trafficable to operation traffic. Barriers are recommended to prevent cattle from damaging the fuse plug.

The gradations of the materials selected within the fuse plug will determine the rate at which the fuse plug section will erode. The fuse plug is intended to erode quickly and completely to reveal the concrete control sill that will behave as a broad crested weir. The fuse plug will activate upon reaching the 100 year peak flood event. During this event, the fuse plug will overtop, progressively eroding the fuse plug to activate the full discharge capacity of the West Spillway. The overflow spillway will then convey flows in excess of the 100 year flood up to the IDF while leaving sufficient freeboard to the crest of the dam (see Section 4.1.4 regarding the freeboard analysis).

6.2.2.3 *Rock-lined Chute*

The rock-lined chute drop structure is designed using gabion baskets/mats and naturally available graded rock fill. The overflow spillway drop will convey water from the spillway channel at an elevation of 721.87 m to Herbert Canal invert at an elevation of 719.50 m over a 10% spillway slope and channel slope length of about 24.5 m.

Gabion mats provide erosion protection within the chute drop structure. The gabion mats have a design thickness of 300 mm and will be filled with stone having a D_{50} of 150 mm. The gabion mats have been designed to resist internal damage up to a flow velocity of 6.4 m/s. The mats could handle higher flow velocities (up to 8.0 m/s) with some internal movement of the gabion rock; however, the overall mats would remain intact.

The rock-lined chute is designed to convey the 200-year IDF. Gabion baskets (1,000 mm thick) will be installed beneath the gabion mats to anchor the mats and to resist sliding. Three gabion baskets will be installed at equal spacing from the weir to the bottom of the chute. The baskets will be attached to the mats to prevent movement of the mats during high flows.

A basin nominally 10 times the depth of flow within Herbert Canal at IDF is provided and gabion mats extended around the junction of overflow spillway chute to Herbert Canal and the opposing side of Herbert Canal where field observations revealed some scour features as a consequence of past spillway operations.

The gabion mats and erosion protection on the side slopes will be required to the maximum of the 200-year flood event depth over the weir with freeboard.



6.2.2.4 Herbert Canal Improvements

Herbert Canal was designed for the conveyance of irrigation releases from Highfield Reservoir via the West LLO. The canal has been constructed along the west side of the Rush Lake Creek valley. The canal is a side-hill channel with material excavated from the canal used to construct the east (downslope) embankment of the canal. The canal is trapezoidal in section. The canal's irrigation design capacity is $6 \text{ m}^3/\text{s}$ with 0.9 m of freeboard to the top of the canal banks. The canal was designed with a 3.0 m wide bed and a bed slope of 0.00025 (m/m) and 2H:1V side slopes.

The Herbert Canal will receive flood flows from the overflow spillway. Flood routing exercises carried out during pre-design (see Section 4.1.5) support the need for the overbuilt section of the Herbert canal's right bank near the spillway. However, the analysis also demonstrates that if the flood flows approach the IDF, insufficient conveyance capacity of the canal will generate backwater to the West LLO and likely overtop the canal embankment nearest the Highfield Dam. This low point was confirmed by field visual information, the LiDAR survey and record PRFA Drawing 45628. The risk to scour the dam at the toe of the west abutment is high and is considered unacceptable.

The pre-design includes a side channel spillway excavated through the east bank of Herbert Canal. A control sill set at an elevation of 720.23 m will direct excess flow from Herbert Canal into the Rush Lake Creek valley. The control sill and side channel spillway length is 100 m with side slopes 3H:1V. The control sill is proposed to consist of a gabion basket with a lean concrete sill. The bed slope downstream of the control sill is 3%, will include a 0.3 m thick gabion mat extending 5 m down the channel and transition to a grass lined channel. Flow that passes through the side channel spillway will flow overland to Rush Lake Creek.

The pre-design includes in-canal modifications to Herbert Canal. From early field inspections, the canal shows signs of infilling with sloughs and slope ravel that are predominantly grass covered. The right bank of the canal is eroded with oversteep slopes at the entry point of spillway flows to Herbert Canal. The pre-design will reshape the banks of the canal to 2H:1V and expand the bed width of the canal to 5 m. The canal modification was assumed to extend to a point about 25 m downstream of the side channel. This match point may be re-evaluated or confirmed during detailed design. The bed slope of Herbert Canal will be maintained as intended by the original design.

6.3 Decommissioning of Existing East LLO

The existing East LLO will be decommissioned once the new East LLO and spillway are operational. A bulkhead will be affixed to the upstream inlet of the East LLO. Decommissioning may commence once the East LLO gate well and conduits have been dewatered and the space certified safe for entry. The interior of the East LLO conduit should be pressure washed to ensure good contact with infill grout. The hardware from within the gate well should be removed and salvaged.

The East LLO conduit will be backfilled with non-shrink grout. The conduit and gate well should be isolated into cells and backfilled in stages to ensure a good seal. Each cell should be sealed to facilitate pressure grouting. Perforated PVC pipes should be provided to allow the isolated cells to vent. The soil-structure interaction will be restored once the East LLO has been backfilled.

The stilling basin may be demolished or left in place.



6.4 West Low-Level Outlet

Currently the West LLO serves two purposes:

- provide irrigation releases to the Rush Lake Irrigation Project via Herbert Main Canal; and
- to operate as a second operating spillway during low frequency storm inflow runoff scenarios. The West LLO is not intended for use during extreme storm inflow events.

The West LLO consists of a 1.5 m square cast reinforced concrete conduit with 0.305 m × 0.152 m haunches and a gate sill elevation of 718.64 m. The West LLO discharges to the Herbert Canal and experiences the tailwater effects based on the water elevations in the canal. The design capacity of the Herbert Canal is approximately 6.0 m³/s with a canal freeboard of 0.9 m based on record drawings. The West LLO is capable of discharges in excess of 6.0 m³/s; however, these flows would overflow the Herbert Canal banks and eventually flow overland to Rush Lake Creek.

The field review concluded that the West LLO is in relatively good condition. The 1992 rating curve provided by AAFC required minor adjustments due to the observed increased roughness inside the concrete conduit and the gates. It is not expected that adjusting the roughness of the inside conduit walls would lead to a significant improvements in the hydraulic capacity due to the size of the outlet.

The rating curve was extrapolated to the proposed TOD (725.7 m) as the existing rating curve is only estimated up to the current TOD (724.8 m). The rating curve for the West LLO is shown in Figure 10. The rating curve reflects a fully open gate. The original rating curve is shown in Drawing 116032 provided by AAFC. The maximum discharge of the West LLO with gates fully open is 10.3 m³/s at FSL (723.0 m) and 12.7 m³/s at the existing TOD.

A review of the structural integrity of the West LLO suggests that it is not compliant with modern building codes (see Section 5.1). The structural integrity will be treated by sleeving the existing conduit with a smaller diameter CSP and filling the annular space between the CSP and existing conduit with non-shrink grout. While CSP sleeving has been assumed for this report, alternative materials and techniques, such as HDPE sliplining, may be considering in final design.

Sleeving will reduce the width and section of the outlet from a 1.5 m box to a 1.2 m diameter circular section. The design rating curve for the West LLO that was provided by AAFC was adjusted to account for this reduction. A rating curve for a fully opened gate was developed for the purposes of this report only. It is expected that gate operations will need to be investigated to determine new operating rules for the modified West LLO.

To develop the modified rating curve for the West LLO the following assumptions were made:

- existing pipe sleeved with a CSP pipe with a length of approximately 40 m;
- maintains the same slope (0.8%) as the existing West LLO;
- the sleeved conduit has a Manning's roughness value of 0.020; and
- Tail-water depths and velocities were based on a rating curve developed for Herbert Canal which assumed a bed slope of 0.2%, a prismatic trapezoidal channel, and a Manning's roughness value of 0.04 as identified by Golder (2011).



The adjusted rating curve for the West LLO is shown on Figure 9. The maximum discharge of the West LLO will be reduced from 12 m³/s to 7 m³/s at the current top of dam elevation. The maximum discharge of the modified West LLO is 7.5 m³/s based on the proposed top of dam elevation and assuming the gate remains fully opened. The gate operating rules will need to be revised as Herbert Canal is limited by its design capacity of about 6 m³/s. Gate operations and analysis of gate openings to achieve desired flows will require further detailed analysis and is not covered in this pre-design study. The new rating curve is capable of meeting the discharge requirements from AAFC outlined in Section 4.3.

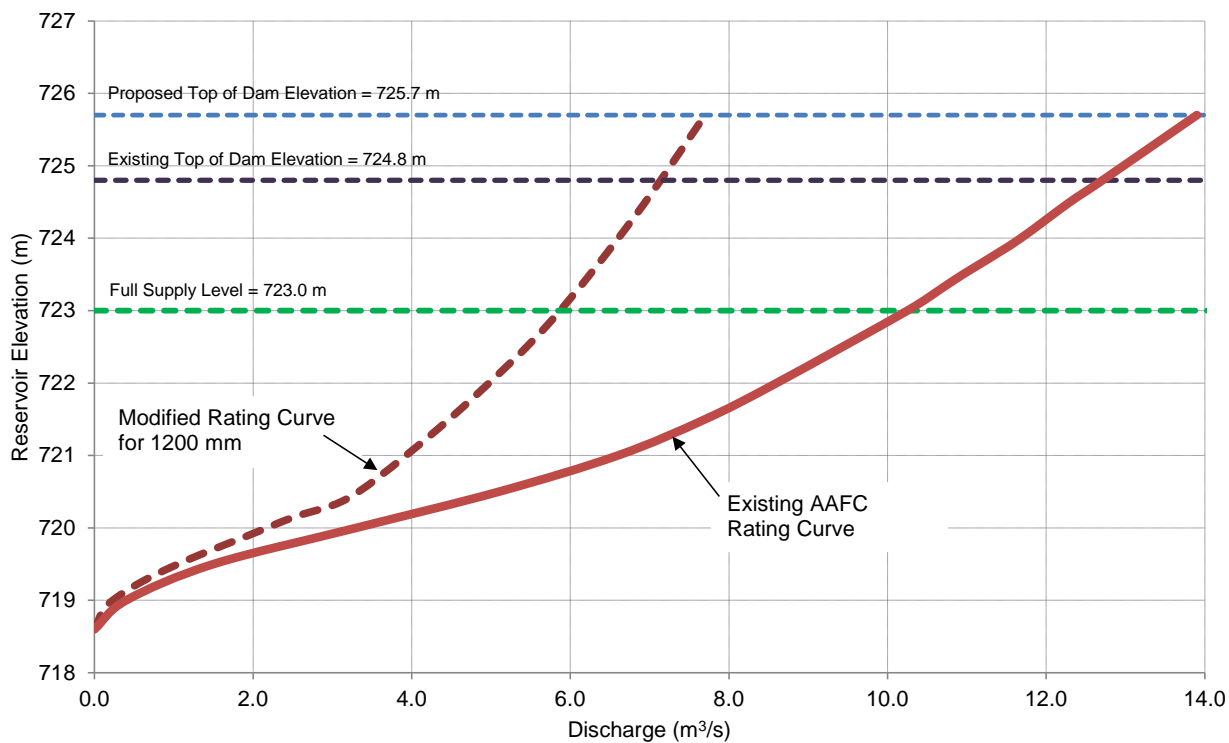


Figure 9: Modified West LLO Rating Curve Based on Reduction to 1.2 m CSP Pipe with Gate Fully Opened

6.4.1 Conduit Rehabilitation

The existing reinforced concrete conduit will be sleeved with round 1,200 mm diameter corrugated steel pipe (CSP). The annular area between the CSP and the existing conduit will be backfilled with low-strength flowable grout.

It is recommended that the rehabilitation of the West LLO conduit be conducted in dry conditions. To achieve this, a bulkhead should be installed and the interior space should be certified as safe to work in.

Two pipe reaches will be required. The two CSP segments will sleeve the conduit reaches located upstream and downstream of the existing gate well structure. The upstream reach of CSP will be pulled into the West LLO prior to placing the bulkhead while the downstream reach of CSP will be slid into place using plastic/steel or wood runners once the bulkhead has been certified. Perforated PVC pipes will be placed in each conduit reach to provide ventilation during the grout backfill procedure.



Individual lengths of the CSP will be connected using external couplers. Hook bolts will be used to center the pipe within the existing conduit. Inspection holes will be provided along the full length of pipe. These will be used to confirm the placement of grout backfill.

The flowable grout fill will be placed along the full length of the CSP liners. The grout will fill the voids between the existing conduit and the CSP to prevent further deformation or collapse by re-establishing soil-pipe interaction. The grout infill reduces the potential for joint leakage while providing additional strength around the CSP.

The method of grout placement is at the discretion of the Contractor, subject to the review and approval of the Owner's Engineer; however, it is advised that grouting be staged to ensure complete infill and a good set. As the slope of the existing conduit is not conducive to a gravity fill approach, a staged pipe fill approach may be used. This technique involves plugging the annular area between the conduit and CSP at both ends of the conduit and utilizing grouting nipples that are installed in the CSP. Filling will be staged to discharge grout at specific intervals along the conduit until it reaches a specific pressure. A PVC pipe will be installed to provide ventilation of the conduit.

The grout fill should be placed continuously to prevent cold joints and bonds during grouting.

Details of the West LLO conduit rehabilitation are depicted as Sections 2 and B on Sheet 610 of the pre-design drawing set that accompanies this report.

6.4.2 Gate Well Rehabilitation

The West LLO gate well rehabilitation will consist of structural upgrades to the existing concrete riser including the replacement of the existing stop log slots and slide gate works. The existing air vent will also be modified to accommodate the changes to the structure and conduit. A ramp and/or stairway will be provided to provide access from the raised dam crest to the gate well.

The West LLO inlet will be isolated using a bulkhead, allowing the West LLO conduit and gate well to de-water. Once the bulkhead is certified, the existing interior works of the West LLO gate well will be removed. The existing concrete structure will remain; however, vertically-oriented steel plates will be inserted into the space and welded to form a continuous steel sleeve within the gate well. The plate will be offset from the existing inner dimensions of the gate well and the remaining space will be in-filled with concrete. Portals will be cut into the plate such that the upstream and downstream reaches of the West LLO conduit sleeves may integrate with the gate well.

The reconditioned gate well will be equipped with new stop log slots with cut-to-fit timber stop logs.

The entrance to the downstream reach of the West LLO conduit will be provided with a 1,200 mm × 1,200 mm galvanized steel slide gate equipped with a manual gate operator. This gate will completely isolate the downstream reach of the West LLO for inspection or maintenance. The gate may also be operated at incremental heights to throttle the rate of release through the West LLO.

Cathodic protection will be required to reduce the likelihood of corrosion of metal components within the gate well and West LLO conduits.



This work will extend the service life of the structure while complying with current CDA Guidelines and NBCC requirements.

Details of the West LLO gate well rehabilitation are depicted as Sections 1 and A on Sheet 610 of the pre-design drawing set that accompanies this report.

6.4.3 Outlet Structure

The existing West LLO includes a concrete stilling basin at the downstream end of the conduit. The field review noted that the structure is in generally good condition and does not currently require rehabilitation. To facilitate the dam raise however, the outlet requires modification as the toe of the raised embankment will extend beyond the upstream end of the stilling basin.

The CSP liner from the West LLO conduit will project into the stilling basin. The annular area outside of the pipe will be backfilled with reinforced concrete even with the stilling basin walls. This will allow fill to be placed over the structure such that the embankment may be raised.

It has been assumed that the stilling basin will continue to function as an energy dissipator due to the reduction in flow through the West LLO; however, the hydraulics of the stilling basin should be re-assessed during detailed design to confirm this assumption.

Details of the modifications to the stilling basin are depicted on Sheet 610 of the pre-design drawing set that accompanies this report.

6.5 Ancillary Works and Other Considerations

The following sections describe the proposed ancillary works and other considerations in support of the rehabilitation of Highfield Dam.

6.5.1 Construction Access, Laydown, and Staging

Preliminary construction access, laydown, and staging areas have been identified on the drawings. During detailed design, the requirements may be refined. Construction access was considered to be predominantly from the east side of the dam crest road. This road is accessed via a network of local roads from Highway 1, east of Swift Current. Web-based, aerial photographs highlight a minimum of one bridge crossing. This crossing may have load restrictions and/or seasonal closures. These constraints have not been assessed. It was assumed that most loads to site would be legal highway widths.

The mobilization of construction equipment may require wide loads. It appears that site access could be achieved from the west side of the dam; however, the access was not considered the main access to the construction site due to the unmaintained nature of the access. The barge access would be via boat ramps to the reservoir. East access to the site will be affected during the installation of the east conduit. West access across the spillway will be affected during the west spillway rehabilitation. A temporary access road was considered on the toe berm following its construction. This would allow loaded construction vehicles along the toe and return across the crest of the dam unloaded. Loaded construction traffic on top of the dam should be limited to the extent practical. This will not be feasible during the riprap installation on the upstream face of the dam. Instrument reading frequencies must be reviewed and adjusted regularly to accommodate changes in construction activities and loading conditions.



The detailed design specifications may consider requesting the Contractor to temporarily “bridge” over the existing East LLO or non-rehabilitated West LLO conduits.

The Prime Contractor will be responsible for implementing on-site safety protocols as established by the contract. AAFC will be required to obtain meet these established requirements in order to access the site.

Stockpiles for materials are dependent on the Contractor's means, methods, and techniques. Just-in-time delivery of materials can prevent double-handling. Key components of the laydown and staging areas which should be considered are:

- topsoil and organics stockpile;
- riprap harvesting stockpiles;
- unsuitable material (spoil) stockpiles;
- borrow material zones;
- excavated native materials stockpiled locally for reuse;
- Gabion rock stockpiles;
- laydown areas for HDPE pipe, reinforcing steel, formwork, structural H-piles, geotextiles, miscellaneous metals, bulkheads, sediment and erosion control materials, miscellaneous consumables, etc.;
- seasonal snow stockpiles, if required;
- dedicated refuelling zone off the reservoir for Contractor's equipment;
- mobile soil testing lab and resident engineer's office; and
- contractor's wash-cart, office facilities, generator with fuel tank, and rescue boat.

6.5.2 Land Control

An examination of the current and required land holdings around the reservoir was undertaken as part of the Highfield Dam rehabilitation. The proponent needs to demonstrate land control for lands that might be occupied or flooded as a result of the proponent's project proposal in order to meet provincial regulatory requirements to obtain a permit to construct and operate from the Saskatchewan Water Security Agency (WSA). It is expected that outright ownership of land below FSL will be required and land control - via ownership or flood / seepage easement will be required to the maximum extent of reservoir inundation resulting from the proponent's reservoir operation. In view of AAFC's long-term desire to divest itself of water control structures, AAFC has decided to attempt to obtain land control to the proposed top of dam elevation of 725.7 m.

AAFC's review of land control has found that Canada owns all reservoir lands below FSL with the possible exception of some minor areas near the mouth of Rush Lake Creek and the Swift Current Main Canal at the upper end of the reservoir. The review also found that Canada does not have all land control to meet the top of dam standard mentioned above. The land control review found that there could be as much as 121 ha of land below the top of dam contour that is not owned or controlled by Canada; however, approximately 81 ha of that total is located in an area south east of the upstream extent of the reservoir. It has been assumed that an abandoned railway embankment could minimize flooding in that area. AAFC has assumed that land control



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would not be required based on the assumption about the railway embankment and the rare nature of the IDF. As a result, the net shortfall would require the control of approximately 40 ha, most of which would be not cultivated or is marginal land. Approximately 1 ha of land will be required at the Highfield Dam site to facilitate the construction of the new works including borrow and other construction areas. Summary details are tabulated in Table 12 and shown on Figure 10.

Table 12: Proposed Top of Dam Elevation by Quarter Section

Land Parcel	Quarter Section	Quarter Label	Area (hectares)
86b0b8e1-31b1-4114-a9c1-5d780f700c93	NE	NE-23-15-11-W3	3.109
381f7ce4-8b21-41ec-8e23-360232889c3f	NW	NW-20-15-10-W3	3.412
09dca6e8-4fa5-4e56-aab4-ca69cfa2e428	NW	NW-19-15-10-W3	0.210
90c6ea80-b65e-4bdb-b76a-5697c60b3aaa	NE	NE-30-15-10-W3	0.494
1acbf36b-6a0e-40e0-94f9-6c424a86c413	SE	SE-28-15-10-W3	1.554
e4ba1510-e90e-45c9-b0fc-0bd30c779ec9	SW	SW-29-15-10-W3	1.371
91f3e268-f454-49ce-94b1-2eab3c25226d	SE	SE-35-15-11-W3	0.215
4af7c0c0-cf78-4c2a-8d0d-1ef6e4924799	NE	NE-34-15-11-W3	2.825
22053f06-3d0e-42aa-9c84-3caf66e86629	NE	NE-20-15-10-W3	21.513
878ddae3-eb31-4140-8412-dff3229e9951	NE	NE-33-15-11-W3	0.686
f926ac7f-9009-42a7-a07f-9122f1d5419c	NW	NW-34-15-11-W3	3.012
18fc2712-2a27-4d6c-9929-6dc04c69146d	NE	NE-21-15-10-W3	31.289
12cce26e-62dd-42ed-b0c7-cbe0cb0def31	NW	NW-25-15-11-W3	0.184
1a8551f7-0328-4714-a01a-d7820592fc18	NW	NW-21-15-10-W3	28.517
bba0e378-5ed2-4be4-8493-0f3bdfee1fec	SW	SW-03-16-11-W3	2.713
cba52f51-a9c7-49ac-b368-3e0a60037374	NW	NW-31-15-10-W3	1.786
4bba9cc2-ba36-4c52-a900-7f825cad203a	SE	SE-04-16-11-W3	1.161
c64524b6-1b29-4899-a624-33c93ff06416	NW	NW-24-15-11-W3	6.243
28aea0ca-b88e-431c-bc6b-317c379a2639	SE	SE-25-15-11-W3	0.071
a4656395-faeb-49dd-aca4-79fc369345df	SW	SW-25-15-11-W3	8.909
039746e8-446a-42da-86ef-30984dcf6277	NE	NE-19-15-10-W3	0.581
dabadee7-f2fc-4e73-bccc-c5b6f2425f6b	NW	NW-35-15-11-W3	1.255



6.5.3 Road Improvements

The upstream end of Highfield Reservoir is currently crossed by an all-weather gravelled farm access road which is between SE30-15-10W3 and NE19-15-10W3. The available information shows that the road is crossed by one 0.9 m diameter corrugated metal pipe (CMP) culvert about 19.8 m long (PFRA Plan 96207). The age and condition of the culvert is unknown.

The road is predominately at an elevation of 724.4 m and thus below top of dam based on an AAFC centreline survey of this road in the early 2000's. AAFC has not reviewed this matter with the local RM of Coulee No. 136. This should be undertaken as part of the final design of the project. It has been assumed approximately 0.6 km of this road will need to be raised to the top of dam elevation. In the absence of more detailed information, the cost of this road raising cannot be estimated to the feasibility level consistent with this report. Based on the limited information, Golder's preliminary cost estimate for the road raise is \$128,000. Further information will be required at the final design stage to complete the design of this road.

6.5.4 Environmental Site Assessment

Treasury Board Real Property Environment Policy requires that before acquiring real property, departments must ascertain the environmental condition of the property and determine whether it is or can be made environmentally compatible with its intended use. This is called an Environmental Site Assessment (ESA). Depending on the nature of the land control, there can be up to three phases of an ESA:

- Phase I ESA - A non-intrusive investigation to identify potential liabilities associated with contaminants in soil, sediment, ground or surface water through site inspection and historical review. The guidelines to conduct a Phase I ESA are in accordance with the Canadian Standards Association (CSA) Standard Z768-01.
- Phase II ESA - An intrusive investigation following a Phase I ESA to identify and determine the nature and extent of potential contamination.
- Phase III ESA - Site Remediation.

During the final design phase, AAFC will be required complete an ESA prior to entering into the land acquisition phase of the project. The ESA are usually undertaken by a service arrangement between AAFC and Public Works and Government Services Canada (PWGSC). Other activities AAFC will have to undertake will include legal surveys, land appraisals, negotiations for land acquisition and legal requirements for transfer of title.

7.0 ENVIRONMENTAL CONSIDERATIONS

7.1 Goals and Objectives

The AAFC has initiated a project to rehabilitate Highfield Dam with a focus on improving dam safety and reducing the risks (liability) to AAFC/Government of Canada imposed by the dam. This AAFC goal is expressed in an investment statement that summarized work carried out regarding Highfield Dam to date and provides a business case to move the project forward. A secondary AAFC goal is to ensure that potentially significant environmental effects resulting from the proposed rehabilitation are mitigated. AAFC's objective is to carry out the necessary design and construction steps to modify Highfield Dam in accordance with the *Canadian Environmental Assessment Act, 2012* (CEAA 2012) and other applicable federal and provincial legislations. Potential risks to the physical, biological, or human environment must be identified such that mitigation measures



can be incorporated into the project and accounted for through a compliance monitoring program that includes best management practices that will be incorporated by AAFC.

7.2 Status of Previous Work

The CEAA 2012 sets out the requirements of environmental assessments and focuses on environmental effects that are within federal jurisdiction, including fish and other aquatic species, aquatic habitat, migratory birds, federal lands, cross-boundary (provincial or international) effects, effects that impact Aboriginal peoples, and any other environmental effects that impact federal decisions. An Environmental Effects Evaluation (EEE) must be undertaken in order to demonstrate compliance with Section 67 of the CEAA (2012). Under Section 6 of the Schedule to the federal Regulations Designating Physical Activities (CEAA 2012), an environmental assessment would be required for a project involving the construction, operation, decommissioning and abandonment of a dam or dyke that would result in the creation of a reservoir with a surface area that would exceed the annual mean surface area of a natural water body by 1,500 ha or more, or an expansion of a dam or dyke that would result in an increase in the surface area of a reservoir of more than 35% is a designated project. However, the Project, as currently proposed, will require the rehabilitation to an existing dam as part of its ongoing maintenance, and as such is not subject to the provisions of the *CEAA (2012)*.

AAFC engaged three consultants to complete environmental baseline studies to support an EEE. These baseline studies include two rare plant, wildlife, fish, and habitat assessments (in 2003 and 2010), and a Heritage Resources Impact Assessment (HRIA). The baseline studies reflect publically available guidelines and information provided by the Committee on the Status of Endangered Wildlife in Canada (COSEWIC), the federal *Species at Risk Act* (SARA), the Saskatchewan Conservation Data Centre (SKCDC) databases, and Saskatchewan Ministry of the Environment (MOE).

7.2.1 December 2003 – Biological Survey

Jacques Whitford Environment Ltd. (Jacques Whitford) completed a rare plant, wildlife, fish and native habitat assessment of Highfield Dam in 2003. The limits of the assessment included the reservoir, areas potentially impacted as part of construction efforts around the dam and ancillary works, and areas downstream including the Rush Lake Creek. The assessment made use of historical and provincial records. The assessment report was presented in three parts with conclusions presented in Table 13.



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Table 13: 2003 Jacques Whitford Study Components Summary

Fish and Fish Habitat	<ul style="list-style-type: none">■ Diverse habitat within the reservoir consists of islands, bays, and large aquatic plant beds.■ Reservoir stocked with northern pike, walleye, and lake whitefish. Only northern pike and yellow perch species have established sustained populations.■ The reservoir is eutrophic and has experienced fish kill in the past over winters during low reservoir levels (low dissolved oxygen).■ The water quality data for Rush Lake Creek is similar to the Highfield reservoir. Both waterbodies contain dense to sparse aquatic vegetation, and flows with sufficient depths to potentially support northern pike and yellow perch.
Rare Plant and Native Habitat (Baseline)	<ul style="list-style-type: none">■ Sixty (60) percent of the habitats surveyed at the Highfield Dam site were native.■ Habitat types included Rush Flats, Saline Wet Meadow, and Native Grassland (or Grassland Fragments) that were dominated by native species.■ Crested wheatgrass has the highest (continuous) cover of exotic species.■ Diverse habitats in the Highfield Dam area may include federally-listed or provincially tracked plant species (none found previously).■ Provincially tracked plant species least mouseltail (<i>Myosurus minimus</i>) and narrow-leaved plantain (<i>Plantago elongata</i>) were identified about 700 m downstream of Highfield Dam.■ Slender Mouse-ear Cress (<i>Halimolobos virgata</i>) was listed as 'Threatened' by COSEWIC. Suitable habitat for this species is present at Highfield Dam.
Wildlife and Wildlife Habitat (Baseline)	<ul style="list-style-type: none">■ Diverse habitats exist and may contain provincially or federally rare or endangered wildlife (none found previously).■ Federally listed wildlife sightings included a breeding pair of loggerhead shrikes (<i>Lanius ludovicianus</i>) about 100 m downstream of Highfield Dam.■ Richardson's ground squirrels were noted in the area.■ Waterfowl and songbirds were observed in the area.■ Twelve (12) provincially tracked songbirds listed as 'sensitive to population decline' were observed previously in the area.■ One northern leopard frog (<i>Lithobates pipiens</i>) was observed at Highfield Dam and is listed as 'Special Concern'■ Plains spadefoot toad (<i>Spea bombifrons</i>) and western hognose snake (<i>Heterodon nasicus</i>) are provincially tracked species that may occur at Highfield Dam.

7.2.2 December 2010 – Biological Survey

KGS Group completed a rare plant, wildlife, fish, and native habitat assessment of Highfield Dam in 2010. The KGS assessment was prepared as a supplement to the earlier Jacques Whitford assessment and was intended to present the most current data available for the project site. The assessment emphasized the downstream areas of the dam along Rush Lake Creek, further identifying existing fish and fish habitat, and documented species that had not been previously observed and reported.

The identification of two additional habitat types, namely: the reservoir (RES) including shoreline habitat along the reservoir, and wetland (WET) that includes the areas within and draining into the Rush Lake Creek are notable departures from the Jacques Whitford assessment. KGS notes that portions of the vegetation



communities within the native grassland habitat mentioned by Jacques Whitford may be highly degraded areas and so are more appropriately named as Grassland Fragment (GF) areas.

The northwest shoreline of Highfield Reservoir (from Highfield Dam extending 180 m along the shore) consists almost entirely of sand with varying levels of gravel, rock and cobble that could be used as spawning habitat for white sucker and walleye.

Rush Lake Creek has a number of natural (beaver) and physical (the Highfield Dam) barriers to passage. Low flow trickle areas and stranded pools, back flooded stagnant water, low dissolved oxygen concentrations and point source water quality alterations from agricultural runoff result in unsuitable fish habitat. KGS suggests that the development of fish passage should not be a necessary component of the Highfield Dam rehabilitation based on current constraint conditions.

No large bodied fish were observed in Rush Lake Creek; however, fathead minnow (*Pimephales promelas*) were observed. Yellow perch (*Perca flavescens*), northern pike (*Esox lucius*), white sucker (*Catostomus commersonii*), and walleye (*Sander vitreus*) were identified in the Highfield Reservoir. No federally-listed or provincially tracked fish species were identified.

Three additional provincially tracked plant species were identified within the Highfield Dam area in addition to the least mouseling (*Myosurus minimus*) and narrow leaved plantain (*Plantago elongata*) identified previously by Jacques Whitford in 2003:

- Heart-leaved buttercup (*Ranunculus cardiophyllus*);
- Stream bank wheatgrass (*Elymus lanceolatus var riparius*); and
- Water weed (*Elodea canadensis*).

None of the five provincially tracked species documented in 2003 and 2010 are federally-listed under COSEWIC and are therefore, not federally protected or considered species at risk in Saskatchewan. However, depending on the species and extent of the local population, activity restrictions may be imposed by MOE. Two of these species, stream bank wheatgrass and lance leaved plantain, are no longer tracked provincially by MOE.

In addition to the loggerhead shrike (previously observed in 2003), chestnut collard longspur (*Calcarius ornatus*) and ferruginous hawks (*Buteo regalis*) were also observed in 2010 and are federally-listed as 'threatened' avian species under COSEWIC.

During the 2010 assessment, there were no federally-listed or provincially tracked mammal species identified. No reptiles were observed; however, two species of amphibian were encountered within the wetland habitat, including the northern leopard frog.

7.2.3 July 2012 – Heritage Resource Impact Assessment (HRIA)

Bison Historical Services Ltd. (2012) completed a HRIA for the Highfield Dam site under Archaeological Resource Investigation Permit No. 12-153 issued by the Heritage Conservation Branch (Ministry of Parks, Culture and Sport). The HRIA was completed using a pedestrian reconnaissance and visual inspection program for all portions of the proposed borrow area immediately north of the east abutment of Highfield Dam, while shovel tests were judgementally excavated in areas where buried heritage resources could be encountered.



One previously recorded heritage resource was identified by the Heritage Conservation Branch as being in potential conflict with the proposed borrow area. EbNu-1 was reported in 1964 by the landowner (Mr. Henry Theissen) and documented as a Precontact burial. The original description of the site places its location outside of the borrow area. Bison Historical Services Ltd. did not encounter any evidence of EbNu-1 within the proposed borrow area or in the area immediately to the north.

One previously unrecorded heritage resource was identified during the HRIA and documented as EbNu-29, a Precontact Artifact Scatter consisting of Lithic tools. No other artifacts were observed around the identified site. Bison Historical Services Ltd. (2012) stated that EbNu-29 is 'deemed to have limited heritage resource significance' and given its location well outside the Highfield Dam remediation project footprint, no further work at EbNu-29 was recommended.

The HRIA report prepared by Bison Historical Services Ltd. (2012) recommended the project be granted Heritage Property Act clearance as 'no heritage sites will be impacted by the proposed development'. AAFC received the clearance letter from the Saskatchewan Heritage Conservation Branch on August 7, 2012 (File No. 12-1156).

7.3 Pre-Design Environmental Strategy

Golder has completed the pre-design having reviewed the rare plant, wildlife, fish and habitat assessment and HRIA reports prepared by others. The project layout and anticipated construction footprint have been reviewed in consideration of environmental concerns and species at risk identified in the previous studies. In order to manage potential environmental risks and to meet AAFC's goal that, "no significant environmental effects result from the rehabilitation of the project", Golder recommends AAFC implement various best management practices as provided herein. Golder recommends these best management practices be reviewed during the detailed design phase of work and throughout construction to incorporate any new observations or environmental concerns not previously identified.

Table 14 through Table 17 provide a summary of the potential adverse environmental effects for each of the environmental component (fish and fish habitat, rare plants and native habitat, wildlife and wildlife habitat and heritage resources), as well as the general techniques and best management practices to be considered during and/or after the rehabilitation of the Project.



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Table 14: Potential Adverse Environmental Effects and Best Management Practices Concerning Fish and Aquatic Habitat

Environmental Component	Potential Adverse Environmental Effect	Proposed Best Management Practice
Fish	Disturbance to fish populations	<ul style="list-style-type: none"> ■ In-water work may require a <i>Fisheries Act</i> Authorization or a Letter of Advice from Fisheries and Oceans Canada (DFO). ■ Provincial Aquatic Habitat Protection Permits and Special Collection Permits may also be required. ■ One or more fish salvage operations may need to be completed to remove fish from isolated work areas before work may proceed. ■ Cofferdams may be required to isolate areas of in-water works. ■ A Total Suspended Sediment (TSS) monitoring program may be needed. ■ Environmental monitors, with experience managing TSS programs, may be required to be on site during in-water construction periods. ■ Spring spawning timing windows, for no in water work, for northern pike and yellow perch from April 1 to May 31 will be adhered to.
Fish Habitat	Disturbance to fish habitat	<ul style="list-style-type: none"> ■ In-water work may require a <i>Fisheries Act</i> Authorization or a Letter of Advice from Fisheries and Oceans Canada (DFO). ■ Any construction activities adjacent to water bodies will have to have erosion control measures in place to prevent sediment from entering the water body. ■ Any areas of in-water work will have to be isolated from the rest of the water body, possibly with cofferdams and turbidity curtains. ■ Machinery is not to enter the water body at any time. ■ Machinery used should be clean and free of leaks. ■ Measures should be in place to prevent spills of fuels or oils from entering the water body.



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Table 15: Potential Adverse Environmental Effects and Best Management Practices Concerning Vegetation

Environmental Component	Potential Adverse Environmental Effect	Proposed Best Management Practice
Vegetation	Disturbance of vegetation communities.	<ul style="list-style-type: none"> ■ Although portions of the study area have been previously disturbed (e.g., cultivated or modified with introduced perennial forage species), native vegetation communities will be affected by construction activities. Construction will be confined to a specific area within the study area to limit disturbances as much as possible to the surrounding vegetation communities. ■ A limited amount of clearing of shrubs and trees may be required to facilitate construction. Clearing will include removal and mulching of woody material, shrubs, and trees. Natural propagation and regeneration are expected to enhance recovery of these species. ■ Sod and topsoil will be stripped and salvaged from the construction areas to retain the propagules and seed bank found within. ■ Back slopes, side-slopes, and other areas where ground disturbance is necessary within the construction area will be recontoured to a stable profile, and salvaged topsoil will be spread over these areas. Micro-variations, such as track imprints or other small surface undulations, can be left to create safe sites for germinating plant species. ■ Natural plant regeneration will be promoted. However, following the recontouring of areas of exposed soils within the construction area, these areas will be seeded with an approved native seed mixture to promote surface stability, mitigate sheet wash, rill, or gully formation, wind/water erosion, and enhance aesthetics.
	Loss and/or disturbance to listed plant species and habitat.	<ul style="list-style-type: none"> ■ Five provincially tracked plant species have been previously identified within the study area, ■ Pre-construction surveys will be completed and the locations of where the plant species were previously identified will be re-visited and will be flagged for avoidance. ■ If the plant species are identified during the pre-construction surveys, the Saskatchewan MOE will be contacted to determine what specific mitigation measures should be implemented. This would include the use of temporary fencing or barricades to avoid disturbances to the plants, activity setback distances or salvage and relocation of the individual plants outside of the area of disturbance on a similar habitat type. Transplanting to suitable/similar adjacent habitat should be considered only after other options have been considered. A scientific permit/permission from MOE is required prior to transplant plants.
	Potential for introduction of noxious and nuisance weeds.	<ul style="list-style-type: none"> ■ Machinery will be cleaned prior to entrance into the study area and again before leaving the study area. Certified seed mixtures will be used for revegetation in consultation with AAFC. ■ If used for reclamation, hay/straw mulches will be obtained from a local source and inspected for noxious weeds prior to use. ■ Should weedy intrusions or spread occur that resulted directly from the Project, they will be promptly addressed using reasonable control measures that are determined in consultation with the appropriate regulator.

MOE = Ministry of Environment; AAFC = Agriculture and Agri-Food Canada



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Table 16: Potential Adverse Environmental Effects and Proposed Best Management Practices Concerning Wildlife

Environmental Component	Potential Adverse Environmental Effect	Proposed Best Management Practice
Wildlife Habitat	Loss or alteration of wildlife habitat.	<ul style="list-style-type: none"> ■ Although portions of the study area have been previously disturbed, construction will occur within areas that retain natural vegetation communities (e.g., grass and forb communities, shrub and tree communities, wetland, and riparian habitats). To limit potential disturbances to wildlife habitat as much as possible, construction will be confined to specific areas within the approximate area of interest (e.g., raised embankments and toe berm, west and east spillways, east outlets). ■ No unique wildlife habitats will be lost as a result of the Project. ■ Baseline field surveys were completed in 2003 and 2010 to identify sensitive habitat locations for avoidance or mitigation. Potential sensitive habitat (e.g., native grassland, steep valley side slopes, riparian areas) will be avoided as much as possible within the approximate area of interest. ■ All areas disturbed will be reclaimed to promote re-establishment of habitat types that existed prior to construction. ■ Vegetation regeneration and encroachment will be encouraged within the study area to assist in the re-habilitation of useable habitat.
	Nest sites could be affected.	<ul style="list-style-type: none"> ■ Construction will be confined to a specific area within the study area to limit disturbances to potential nesting and rearing habitat. ■ Construction activities may occur during the spring/summer period on native grassland and riparian areas (within the study area) and may potentially affect grassland and riparian nesting migratory bird species. If construction occurs during the sensitive nesting and breeding period (prior to August 31), a wildlife monitor may be required by Environment Canada (as part of their condition of approval) to complete a nest search of the construction area and adjacent habitats. A recommended activity restriction guideline of 50 m will be adhered to should an occupied nest site of a migratory bird be encountered, per standard recommendations for addressing potential effects to nesting migratory birds protected under the <i>Migratory Birds Convention Act</i> (P. Gregoire, pers. comm. 2012). ■ Removal of shrub and treed habitat to accommodate construction may potentially affect shrub-nesting migratory bird species (e.g., loggerhead shrikes) and raptor species (e.g., ferruginous hawks). To avoid disturbance to these species during the nesting period, shrubs and trees should be removed during the fall/winter period and mulched prior to the early spring (April 15) when these species typically return to set up nesting territories. If this is not possible, Activity Restriction Guidelines (Saskatchewan MOE 2013) will be adhered to should an occupied nest site be encountered. These guidelines outline recommended setback distances for particular listed wildlife species in Saskatchewan whereby specific activities must avoid known nesting locations for a specific distance. For example, the recommended setback distance that activities must avoid a loggerhead shrike nest by is 500 m.



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Table 16: Potential Adverse Environmental Effects and Proposed Best Management Practices Concerning Wildlife (continued)

Environmental Component	Potential Adverse Environmental Effect	Proposed Best Management Practice
Wildlife (cont.)	Nest sites could be affected (continued).	<ul style="list-style-type: none"> Depending on the timing of construction in the spring/summer period and the extent of construction a wildlife monitor may be required by Environment Canada to complete a nest search within 200 m of the construction area to identify occupied ground nests (e.g., Sprague's pipit nests). Activity Restriction Guidelines (Saskatchewan MOE 2013) will be adhered to should an occupied nest site be encountered. These guidelines outline recommended setback distances for particular listed wildlife species in Saskatchewan whereby specific activities must avoid known nesting locations for a specific distance. For example, the recommended setback distance that activities must avoid a Sprague's pipit nest by is 200 m.
	Construction activities may cause temporary displacement of wildlife or restrict wildlife movement.	<ul style="list-style-type: none"> Construction will be completed in an expeditious manner as safety allows. Construction will be confined to specific areas within the study area to limit disturbances to wildlife. Alternative and similar habitat is readily available outside of the study area that can be used by temporarily displaced wildlife. Resident wildlife in the area may have habituated to the existing land use (i.e., ranching, crop production, and highway use). Construction activities will be completed in such a manner that will not create barriers for wildlife (i.e., will allow connectivity between habitat types) or result in additional restrictions to wildlife movement other than those that currently occur with the present land use (i.e., existing infrastructure and transportation corridors). Potential barriers to wildlife, particularly small mammals, reptiles and amphibians, are expected to be confined to the Project footprint, likely at site-specific locations where the Project components bisect native habitat patches (e.g., Rush Lake Creek).
	Increased vehicle-wildlife collisions.	<ul style="list-style-type: none"> Equipment operators and construction personnel will be instructed to be aware of wildlife in the study area. Safe speed limits will be enforced during construction.
	Wildlife harassment and habituation.	<ul style="list-style-type: none"> Project personnel will be instructed to keep a clean work area and to not to feed or otherwise harass wildlife encountered. Garbage and wastes will be properly disposed of to avoid attracting scavenger species.
Listed Wildlife Species	Potential disturbance to listed wildlife species.	<ul style="list-style-type: none"> Unique or sensitive habitat types that may be used by listed wildlife species will be avoided as much as possible and construction activities will be confined to a specific area within the study area to limit the spatial extent of potential disturbances to listed wildlife species. <p>Baseline field surveys were completed in 2003 and 2010 to identify sensitive habitat and potential nesting and breeding locations by listed wildlife species and loggerhead shrike, chestnut collared longspur and ferruginous hawk were federally listed wildlife species were identified. If construction occurs during the sensitive nesting and breeding period (prior to August 31), a wildlife monitor may be required by Environment Canada (as part of their condition of approval) to complete a nest search of the construction area and adjacent habitats. Activity Restriction Guidelines (Saskatchewan MOE 2013) will be adhered to should an occupied nest site be encountered.</p>



AAFC - HIGHFIELD DAM REHABILITATION

Table 16: Potential Adverse Environmental Effects and Proposed Best Management Practices Concerning Wildlife (continued)

Environmental Component	Potential Adverse Environmental Effect	Proposed Best Management Practice
Listed Wildlife Species	Potential disturbance to listed wildlife species.	<ul style="list-style-type: none"> Northern leopard frogs were identified in the study area during the previous baseline field surveys, and could be affected by construction activities. Where possible, a 400 m buffer will be maintained year-round from seasonal wet areas that have suitable breeding habitat for this species. Further, a 400 m buffer will be maintained around wetlands known to provide wintering sites for the period of September through May to protect frogs moving to and from wintering sites, and during hibernation at these sites. It is understood that the oxbows and standing water areas within 50 m of the proposed toe berm will be partially or wholly de-watered to facilitate construction activities. It is our understanding that these areas typically maintain shallow water levels and generally freeze to the bottom during the winter season. For this reason, these areas are not considered suitable as overwintering habitat for northern leopard frogs, and therefore may not warrant any mitigation measures to occur prior to construction. However, if these oxbows and standing water areas do require de-watering prior to excavation, an environmental monitor will be on-site to capture and re-locate northern leopard frogs (if present) during the de-watering process.

MOE = Ministry of Environment; m = metres

Table 17: Potential Adverse Environmental Effects and Proposed Best Management Practices Concerning Heritage Resources

Environmental Component	Potential Adverse Environmental Effect	Proposed Best Management Practice
Heritage Resources	Disturbance to previously undiscovered archaeological sites during construction	<ul style="list-style-type: none"> An HRIA was conducted for the Project; no known sites are in conflict with the Project. In the event that unanticipated archaeological materials or features (including but not limited to, hearth features, lithic, ceramic and faunal artifacts, and human remains) are encountered during construction activities, it is recommended that all work in the immediate area cease and the Heritage Conservation Branch be contacted for further guidance.

HRIA = Heritage Resources Impact Assessment

7.4 Sediment and Erosion Control

The Canadian Master Specifications and industry standard practice for construction contracts is to include a 'Care of Water' or 'Preservation of Watercourses' specification. The specification requires the Contractor to submit a Care of Water, or Preservation of Watercourses Plan that is inclusive of an Erosion and Sediment Control Strategy. The plan is a required submission by the Contractor for approval prior to commencing construction activities. The erosion and sediment control practices are designed using site specific mitigation and control strategies to prevent deleterious materials/substances from affecting a watercourse. Measures are to be site specific and may include, but are not limited to, simple silt fencing, fabric rolls, sediment ponds, and pumping strategies.



7.5 Environmental Monitor

An Environmental Monitor (EM) should be provided at key times throughout construction. It is anticipated that an EM will provide a key interface between the construction activities and the AAFC, including interface with Saskatchewan MOE. The frequency of EM involvement is anticipated to be more intensive during project start-up, and if construction overlaps with the sensitive nesting and breeding period for wildlife or sensitive spawning periods for fish. The EM is anticipated to document sensitive species of vegetation and wildlife encountered within the construction footprint.

Monitoring may be adjusted following critical periods and may include inspection and reporting performance of erosion and sediment control activities and mitigation measures implemented by the Contractor and recommendations for improvements based on these findings. Monitoring is recommended to include TSS observations in the reservoir and in Rush Lake Creek before, during, and after in-water construction.

7.6 Post Construction Monitoring

Disturbed areas should be inspected in the spring following construction activities to evaluate the re-establishment of the native plant communities, and identify the presence of noxious weeds that may have resulted from the construction. The success of the soil stabilization efforts that were implemented should be assessed, and determine the necessity for any follow-up work, including complementary seeding. To prevent damages due to over-grazing, and to allow the re-establishment of the native plant communities, the native grassland areas of the disturbed areas should be fenced off for a period of at least one year following construction. The fence should be left in place until a self-sustaining, live-plant cover establishes (i.e., minimum of 50% to 70% live cover).

8.0 PLANNING FOR ENGINEERING RESOURCES

Future engineering-related activities are required to advance the project through detailed design and ultimately into construction.

Recommendations for future engineering activities include the following:

- Infill surveys – establishment of ground control, current reservoir bathymetry, survey information of miscellaneous areas not currently available or of insufficient accuracy.
- Infill geotechnical investigations – required to validate the foundation materials along the East Spillway and to validate the material properties, areal extent and waste potential of the potential borrow area as described in Section 6.1.5 – Borrow Source Evaluation. This data will be used to inform the detailed design process and reduce risks associated with unknown or unanticipated ground conditions.
- Material (riprap, engineered soil products that are not available locally, geosynthetic products, gabion baskets and mats, etc.) and equipment procurement.
- Resident engineering during construction.



9.0 CONSTRUCTION PLANNING

9.1 Construction Sequencing

The general construction sequencing consists of a three year construction period. The rationale for sequencing the construction over three years is driven by the constraints associated with the dissipation of pore water pressure during fill placement on the dam as well as operating requirements that necessitate the availability of irrigation water for downstream uses.

Year 1 work consists of the following:

- construction of the toe berm;
- west LLO rehabilitation;
- west spillway and Herbert Canal upgrades; and
- upstream dam facing bedding and riprap (stage 1).

Year 2 work consists of the following:

- construction of new East LLO and primary spillway (drop inlet);
- decommissioning of existing East LLO; and
- dam raise including completion of upstream slope protection, road gravel, landscaping and seeding.

Year 3 work consists of an upgrade to the local road near the south extent of Highfield Reservoir.

Detailed commentary and a complete schedule are provided in Section 9.2.

9.2 Pre-Design Construction Schedule

The project schedule for the detailed design and dam rehabilitation construction work was prepared in consultation with AAFC during the pre-design stage.

The rationale for the schedule has considered and assumed the following:

- The completion of the detailed design inputs shall be completed prior to the commencement of detailed design. This includes the permits and approvals necessary to complete the task (i.e. geotechnical site investigations, topographic survey, and environmental delineation). Information obtained from these activities may impact the schedule.
- A non-exhaustive list of regulatory and permitting requirements has been assumed. These requirements should be verified by AAFC. AAFC holds the responsibility to procure all necessary permits and approvals necessary to carry out the work.
- It has been assumed that the Loggerhead Shrike resides close to the project site and therefore woody undergrowth shall be removed in September prior to initiating construction.
- It has been assumed that the Northern Leopard Frog does not impact the project's critical path and relocation activities may be carried out in advance of the construction work, as required.



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- The Contractor is granted adequate time to obtain reviewed submittals, procure materials, and long lead items between contract award and mobilization.
- Riprap armor material supply is obtained as a separate contract to minimize potential schedule impacts. The award would occur prior to the fall to allow bidders to access farmer's fields to collect riprap armor if they rely on this supply method.
- Assumed the definition of Herbert Canal is "man-made" and therefore not subject to the Department of Fisheries and Oceans (DFO) requirements nor has riparian flow requirements.
- Assumed work on the existing west spillway is completed with Herbert Canal improvements. This takes advantage of their relative proximity and the use of gabion blocks and mats (common use of equipment and skilled labour).
- The existing East LLO will be maintained and can deliver riparian flows until the east auxiliary spillway and riparian inlet are commissioned. Once those tasks have been completed, the existing East LLO may be decommissioned.
- AAFC land acquisition shall be completed prior to the need to access borrow areas.
- PWGSC public tendering and procurement processes.
- Provision of normal irrigation requirements to downstream stakeholders and reservoir management.
- Hydrologic analysis of summer and fall rainfall events; assessment of normal snowmelt and spring runoff periods.
- Maintaining one operational low level outlet after the irrigation requirements have been met.
- Maintaining an operational existing west spillway and Herbert Main Canal for weir and overland spillway works.
- Geotechnical stipulations regarding settlement and construction during the embankment raise.
- Minimizing loaded equipment access to the top of the dam.
- Sequencing construction activities around the two existing low level outlets with structural limitations.
- Availability of construction materials such as native borrow material and riprap.
- Typical weather events not including flood events.
- The drop inlet spillway is constructed behind an earthen plug during a time of higher water in the reservoir (June/July). Any opportunity to modify the schedule should review placing this work in a fourth quarter of a year.
- Assumed the "reservoir road crossing embankment raise" is scheduled for a third construction year to allow AAFC to obtain municipality approvals. This activity was scheduled in a warm season to avoid road bans and the additional costs of winter construction.



Future modifications to the project schedule will have an impact on the project's estimated costs and AAFC should consider updating the cost estimate to reflect project schedule changes. The project schedule is shown on Figure 11.

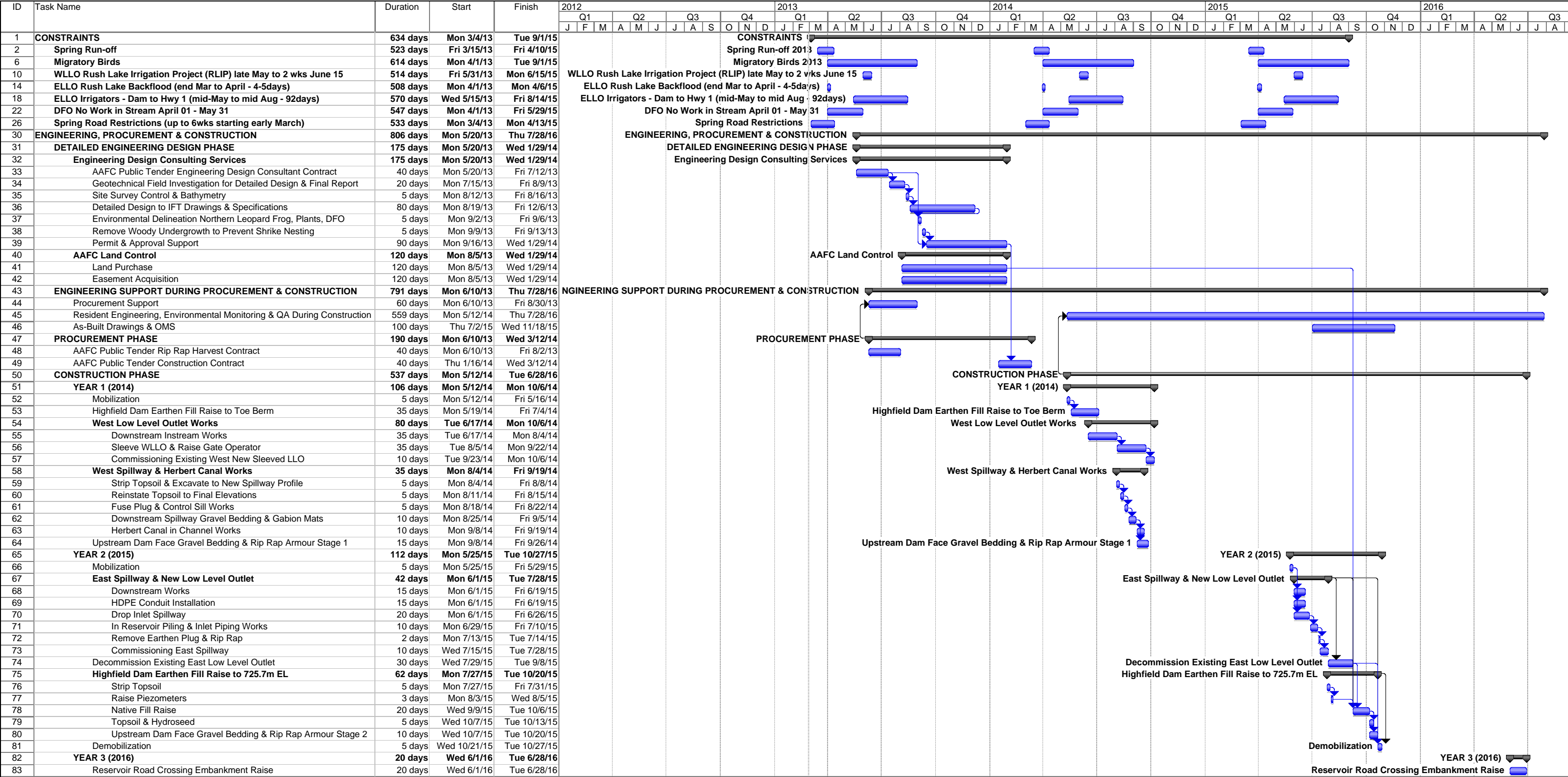
10.0 COST ESTIMATE


AAFC have requested a Class "C" construction estimate with a contingency of 20%. The Class "C" estimate is based on schematic design drawings, design/construction experience and market conditions. The estimate was based on material take-offs and application of unit costs. Golder provided the raw material takeoffs for the estimate. Golder consulted the RS Means historical database to provide cost and productivity rates as well.

The design was progressed to a level consistent with the project maturity necessary for a Class "C" estimate. Assumptions were made on the types of material specifications if they were not listed on the drawings. A project schedule was prepared as a basis for the Class "C" estimate. In consultation with AAFC, the dam rehabilitation works will span three years. The project schedule's logic includes the following activities and assumptions:

- completion of detailed design inputs, (e.g. geotechnical, survey, environmental delineation);
- regulatory and permitting requirements;
- land acquisition;
- tendering process;
- provision of normal irrigation requirements to downstream stakeholders and reservoir management;
- maintaining one operational low level outlet after the irrigation requirements have been met;
- maintaining an operational existing west spillway during months where potential flood events may occur (i.e. rehabilitation starting in August and finishing prior to the following spring run-off);
- geotechnical stipulations regarding settlement and construction during the embankment raise;
- minimizing loaded equipment on the top of the dam;
- sequencing construction activities around the two existing low level outlets with structural limitations;
- sourcing of long lead time materials; and
- typical weather not including flood events.

Modifications of the project schedule will have an impact on the project's estimated costs and AAFC should consider updating the cost estimate to reflect project schedule changes.





DRAFT

REVISION B

Project: AAFC Highfield Dam
Date: Wed 2/27/13

Task

Split

Progress

Milestone

Summary

Project Summary

External Tasks

External Milestone

Deadline





10.1 Assumptions and Exclusions

10.1.1 Assumptions

The following assumptions have been made to facilitate the completion of the cost estimate:

- mobilization and demobilization of local Contractor equipment and site facilities to the proposed sites at Highfield Dam;
- timely availability of all required materials for construction;
- no provision for escalation is included in the estimate;
- no allowance is included for any enablement works that may be required to access the site including weight restrictions on bridge structures, road bans and/or temporary soil/dumping facilities, (e.g., site support, labour, equipment siltation controls, etc.);
- all spoils and excess materials can be disposed within the project site or dump site that is located in close proximity to the worksite (i.e., within 2 km of the working area) with no tippage cost to the Contractor;
- the project will be spread over three (3) construction seasons with mobilization and demobilization costs;
- an advance procurement process will be undertaken to harvest the large quantities of rip-rap for stockpile onsite;
- the load restrictions for equipment on the dam site will be practically interpreted at the project site by the consultant;
- survey information and site data will be provided by the client in a form and quantity that is adequate for design and construction purposes;
- site-specific training will not be required for any activities on the site;
- full unhindered access to the site for the duration of the project operations;
- no site conflict with other Contractors during the construction work;
- Federal government Labour Condition provisions including Fair Wage Schedule have been assumed;
- no allowance has been made for any potential delays resulting from inclement and unseasonal weather, including flood and frost;
- no special requirements for personal protective equipment (PPE) beyond CSA approved hard hats, ear plugs, footwear, and visibility vests;
- no groundwater contamination is present;
- no requirement for imported top soil, backfill material and native cover; and
- required area to be excavated does not require ripping, blasting or other special considerations.



10.1.2 Exclusions

General exclusions include the following:

- No allowance has been included for obtaining any permits or permissions to undertake the work. It has been assumed that all required permits will be obtained by others. City, Town, Provincial and Municipal permits are excluded. No allowance for permit delays.
- No allowance has been included for the removal or disposal of any hazardous, contaminated, or toxic materials encountered.
- No allowance has been included for any enablement works or upgrades to the site other than those described herein.
- No allowance has been included for dewatering, well pointing etc. The estimate assumes water can be managed with conventional sump pumps and discharge hoses. Costs associated with special sediment controls, filtration, etc. are not included in the estimate.
- Taxes and duties are excluded.
- No allowances have been included for geotechnical related requirements including slope/trench stability, rock scaling, and ground improvement.
- No legal costs included.
- No escalation costs included.
- No snow removal costs.
- Costs related to the underground flow of contaminants in the groundwater (i.e., no groundwater release).
- Costs related to any degradation in the surface and groundwater flows.
- Additional costs due to the final geotechnical investigations impacting the detailed design, (e.g. piles, etc.).

10.2 Material Take-offs and Engineer's Cost Estimate

A description of the methods and sources used for determining all material, labour and subcontract pricing is provided below:

- Equipment Pricing: based on rates obtained from local Contractors who have recent experience with the type of work.
- Materials: based on recent supplier budgetary pricing for material elements that form part of the anticipated scope of work. Where applicable, we assumed larger material volumes would qualify for discounts and a premium may apply for small material volumes.
- Labour Hours and Productivity: the labour hours and production rates are based on performing the work during non-winter months to avoid the additional costs and inefficiencies associated with working during this time. The work may be impacted by other site constraints, weather, environmental considerations, wildlife habitat, fisheries windows, and spring run-off periods so the client should review the actual construction



schedule and revise the cost estimate accordingly. Assumed construction work shifts of ten (10) hours per day based on a six (6) day work week.

- Wage Rates and Crew Mixes: based on rates recently obtained from local Contractors who are competent and familiar with this type of work.
- Pricing Factors: material supply prices are considered to be budgetary only. Material pricing is subject to the pricing of global raw materials and the availability at the time of purchase. Golder excluded any cost considerations for material pricing fluctuations which the client should consider for future estimates as the project definition and maturity develops. Additional cost information is from RS Means 2012 with location at Moose Jaw, Saskatchewan.
- Indirect Pricing: construction-related indirect costs have been included in the Class "C" estimate and are embedded into the unit rates.
- Contractor Overhead and Profit: Contractor overhead and profit has been estimated at 18% of the total construction cost before the application of contingency.
- Allowances:
 - Engineering, resident inspection and quality assurance costs were included as a lump sum allowance.
 - A contingency of 20% was included in the cost estimate.
- Location Factor: the site is located close to Swift Current, Saskatchewan and it was assumed that there is an availability of competent Contractors and Subcontractors with no shortage of a skilled labour force. It was assumed that the project would be competitively bid.

A detailed breakdown of the material take-offs and associated costs are provided in the Pre-Design Class C Estimate table in Appendix E. The table, and all rates provided, include a 20% contingency.

Should the site conditions and design scope differ from this report, or the assumptions and exclusions change, additional or different rehabilitation activities may be required, which will necessitate revisions to this basis of cost estimate.

Further investigation, analysis and constructability reviews should be completed to develop the final detailed design to provide a more accurate representation of the cost estimate range based on an increase in project definition.

The Class "C" cost estimate for the proposed Highfield Dam rehabilitation has been developed at a pre-design level and is based upon incomplete information. Actual costs may vary from those presented in this estimate.

10.3 Cash Flow Assessment

A Class "C" estimate and project construction schedule were developed in support of the pre-design efforts presented herein. An annual cash flow was developed to provide an indication of the necessary AAFC budget cash flows. Engineering in the first budget year (2013-2014) reflects final design engineering (estimated at \$500,000), site surveys and bathymetry work, geotechnical investigations (boreholes, test excavations, and materials testing), environmental assessment support, permits and approvals support, and contract tender support. Construction engineering and inspection (resident engineering/site monitoring) costs make up the



balance of Engineering and are distributed over the later three years presented. Any changes to the estimate or schedule will necessitate an update to the proposed budget cash flow. The cash flows for each AAFC fiscal year include a 20% contingency and an additional 15%, to provide a construction estimate with a level of confidence of 80% (P80), based on the level of design maturity. The cash flow is presented in Table 18.

Table 18: Project Cash Flow (AAFC Budget Cycle April 01-March 31)

	2013-2014	2014-2015	2015-2016	2016-2017
Engineering	\$1,104,000	\$345,000	\$276,000	\$55,000
Rip Rap Contract	\$1,875,000	-	-	-
Construction		\$6,546,000	\$2,794,000	\$153,000
Permits	AAFC	AAFC	-	-
Land Control	\$2,500	AAFC	-	-
Other	AAFC	AAFC	AAFC	AAFC
Subtotal	\$2,982,000	\$6,891,000	\$3,070,000	\$208,000
Cumulative Total	\$2,982,000	\$9,873,000	\$12,943,000	\$13,151,000

Note: All values rounded (\$ Canadian 2012). AAFC to provide costs where noted. No costs have been provided for land easements.
AAFC = Agriculture and Agri-Food Canada

The total project cost is \$13,151,000 including a +20% contingency.

11.0 RISK ANALYSIS

Golder has conducted a preliminary risk analysis to identify and categorize various risks that may impact the design, construction, operation, and serviceability of Highfield Dam.

11.1 Risk Register

Each risk element/event contained within the register includes the identification of key elements that are affected by a given risk, the affected project phase, the category of risk, and possible consequence.

Project phases, in sequence from early to late stages, include:

- Phase I: Identification – project stage at which time the initial motivation that drives the need to complete the project is defined.
- Phase II: Definition – project stage at which time performance criteria and functional requirements for the project are defined.
- Phase III: Design – project stage at which time concepts are designed and evaluated in terms of functionality, constructability, cost, and schedule.
- Phase IV: Implementation – project stage at which time a Contractor plans, constructs, and commissions work associated with the designs prepared during Phase III.
- Phase V: Operation – project stage at which time the Contractor returns control of the facility to AAFC.

Note that most opportunities to mitigate potential risks may be found and exploited in the first three project phases. Early mitigation is often less costly to the proponent than remedial actions that occur during the latter two project phases.



Potential risk categories include:

- project delivery risks – events that may impede the contractor's ability to complete the project as directed;
- business risks – events that may impede AAFC's capability to successfully honour business commitments (i.e. ability to provide required quantities of water to downstream stakeholders); and
- schedule risks – events that may incur an adverse effect to the timely completion or operation of the project.

Consequence types include the following:

- Safety, Public – the event may put the public or their property at risk of damage, injury, or death;
- Safety, Worker – the event may put either AAFC or the contractor's staff or property at risk of damage, injury, or death;
- Environmental – the event may cause adverse impacts or damage to the local or downstream environment;
- Financial – the event may trigger a financial loss to AAFC or downstream stakeholders;
- Reliability, Water Supply – the event may affect AAFC's ability to deliver water to meet the needs of downstream water users; and
- Reputation – the event may harm the reputation of AAFC / Government of Canada, its Contractors, or its consultants.

The risk register is included in Appendix F.

A thorough evaluation of project risks has not been conducted at this stage. It is recommended that a detailed evaluation be conducted during detailed engineering design.

11.2 Risk Management

There are a number of project and construction risk events that should be analyzed by AAFC and maintained in a risk register for ongoing risk management purposes. Risk events may impact schedule and costs; therefore, they should be managed proactively by AAFC utilizing industry standard risk management techniques to identify, evaluate, assess, and mitigate risks. There may also be positive risk events or opportunities presented during risk management planning that may be exploited.

A non-exhaustive list of project and construction risk events includes the following:

- public tender periods exceeding "normal" AAFC durations;
- flood event occurs during construction;
- irrigators do not receive water allocations due to construction issues;
- dam breach or dam safety compromised;
- a project construction safety incident resulting in a fatality as the work involves higher consequence construction tasks;



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- activities for detailed design inputs are delayed by permits and approvals to complete the same (geotechnical, survey, environmental delineation);
- accurate survey information for all scope areas is required to confirm the client supplied contour intervals which may increase or decrease costs;
- the record and AAFC-supplied information is not accurate;
- multiple general contractors may have schedule conflicts (if this strategy was used);
- labour availability and Contractor competence is not adequate;
- pore water pressure dissipation and settlement are delayed, increasing the duration between the two phase dam embankment raise;
- an unsuccessful West LLO liner installation delays second phase embankment fill;
- East LLO or West LLO conduit structural failure;
- the lowering of the Highfield reservoir to facilitate construction efforts requires consultation with the WSA in order to engage the appropriate stakeholders that rely on water releases from the reservoir. Lowering the reservoir risks over winter fish mortality due to low dissolved oxygen and risks to meeting irrigation demands;
- barge-mounted, pile-driving rig has a hydraulic oil spill into the reservoir;
- nesting migratory birds onsite;
- other environmental incidents regarding regulatory or permitting authorities;
- land acquisition is more expensive, delayed or not negotiated successfully for borrow material (if required);
- land easements for flooding are more expensive than anticipated;
- farmer's cattle or the public enter the construction areas resulting in an incident;
- inability to secure rip-rap armor in enough quantity or with schedule impacts;
- road bans or local bridge load restrictions on access roads to site; and
- the reservoir road crossing embankment raise could be completed in an earlier construction year if permits and municipal approvals were obtained, which may reduce construction and resident engineering costs.

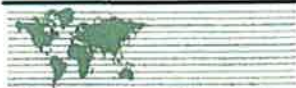


12.0 CLOSURE

The report presented herein has been compiled based on Golder's understanding of the objectives set forth by AAFC, the service and performance requirements contained within 2007 CDA Guidelines, and Golder's assessment of the existing structures as observed on August 27, 2012

The preliminary design work is the result of the efforts of several designers and subject matter experts who are signatory to this report. Golder holds a current Certificate of Authorization in the Province of Saskatchewan (Number 230) and is a member-in-good-standing of APEGS.

Please contact Andrew Szojka should you have any questions or comments related to the contents of this report.



Report Signature Page

GOLDER ASSOCIATES LTD.

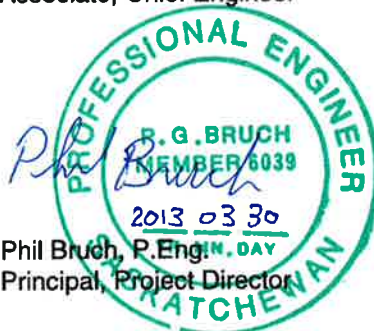
Corey Aurala
Design Coordinator

Andrew Szojka
Associate, Project Manager

RK/CMA/AS/xx



Ron Kitagawa, P.Eng.
Associate, Chief Engineer



Phil Bruch, P.Eng.
Principal, Project Director

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Association of Professional Engineers & Geoscientists of Saskatchewan		
CERTIFICATE OF AUTHORIZATION		
Golder Associates Ltd.		
Number C0230		
Permission to Consult held by:		
Discipline	Sk. Reg. No.	Signature
GEOTECHNICAL	6039	Phil Bruch



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Table 19: Drawings Provided by Agriculture and Agri-Food Canada

Drawing No.	Date	Title	Issued	File No.
PR-1244	1952	Highfield Dam Reinforcing Details of West End Outlet Structure	PFRA	156-D7
PR-750	1951	Highfield Dam Improvements to East Outlet As Constructed	PFRA	C1317
PR-1020	1951	Highfield Dam East Outlet Stilling Basin – As Constructed	PFRA	C1589
XL-406	1971	Highfield Dam West End Outlet Structure – As Constructed	PFRA	C2753
XI-407	1953	Highfield Dam West Outlet Structure Reinforcing Details – As Constructed	PFRA	2754-D74
8680A	1971	Herbert Canal Key Plan of Herbert Main Canal Showing Structure Locations and Canal Elements	PFRA	
1085-C-1	1943	General Plan for Highfield Reservoir	PFRA	21383A
C26722	1941	Highfield Reservoir Outlet Duct and Outlet Control Structure	PFRA	26722
42948	1974	Location Map	PFRA	
2-1489	1952	Highfield Dam East Outlet Gate Installation Drawing	ARMCO	45064
45628	1973	Profile of Herbert Main Canal 0+00 – 24+00	PFRA	
90025 Fig. 1	1970	Highfield Dam East Outlet Control Structure Testhole Results & Condition Prior to 1969 Repairs	PFRA	
90026 Fig 2	1969	Highfield Dam Repair to East Outlet Structure Crack in Concrete at Station 0+25	PFRA	
90027 Fig 3	1969	Highfield Dam Repair to East Outlet Structure Crack in Concrete at Station 0+10	PFRA	
90028 Fig 4	1969	Highfield Dam Repair to East Outlet Structure Joint in Concrete at Station 0+00	PFRA	



AAFC - HIGHFIELD DAM REHABILITATION

Table 19: Drawings Provided by Agriculture and Agri-Food Canada (continued)

Drawing No.	Date	Title	Issued	File No.
92907	1973	Reservoir Topography West ½ Sec.31-15-10W3 & Sec.36-15-11W3 and Flooded Area Capacity Curves	PFRA	
92908	1973	Reservoir Topography West ½ Sec.19, NW ¼ Sec.20, SW ¼ Sec 29 & Sec 30- 15-10W3 & NW ¼ Sec. 24 & Sec. 25-15-11W3	PFRA	
92909	1973	Reservoir Topography N ½ Sec 34 & NW,NE,SE ¼'s Sec 35-15-11W3	PFRA	
92910	1973	Embankment and Spillway Profiles, Typical Cross Sections & Outlet Structure Cross Sections, Structure Discharge Rating Curves	PFRA	
92911	1973	Road Profiles and Typical Cross Sections Of Reservoir Crossings Along the E.BDY of Sec19 & 30-15-10W3 & E.BDY of Sec34-15- 11W3	PFRA	
92912	1973	Reservoir Sedimentation Surveys Referenced Sedimentation Cross Sections	PFRA	
102819A	1987	Location Plan, Profile and East and West Outlet Cross Sections	PFRA	
116032	1992	Highfield Dam West Outlet Rating Curve	PFRA	
116033	1992	Highfield Dam West Outlet Rating Curve (Imperial)	PFRA	
116034	1992	Highfield Dam East Outlet Rating Curve (Metric)	PFRA	
116035	1992	Highfield Dam East Outlet Rating Curve (Imperial)	PFRA	
116036A	1992	Highfield Dam Earth Spillway Rating Curve (Metric)	PFRA	
116037A	1992	Highfield Dam Earth Spillway Rating Curve (Imperial)	PFRA	
C116711	1991	Highfield Dam Contract 1 – Rehabilitation of East Irrigation Outlet Works Existing Structure Details – As Constructed	PFRA	
C116714	1991	Highfield Dam Contract 1 – Rehabilitation of East Irrigation Outlet Works Structure Details Sheet 1 of 2 – As Constructed	PFRA	
C117912	1993	Highfield Dam Plan and Centreline Profile Of East Inlet Structure As of Oct/92	PFRA	
117913	1992	Highfield Dam Plan Showing East Gatewell Ladder Details & Misc. Metal Work As of Oct/92	PFRA	
208524	2011	Highfield Reservoir Existing Topography and Drill Hole Locations Sheet 1 of 6	AESB- DGSA	
208525	2011	Highfield Reservoir CL Profile Main Dam Embankment and Drill Hole Logs – Sta. 0 to Sta. 300 Sheet 2 of 6	AESB- DGSA	



AAFC - HIGHFIELD DAM REHABILITATION

Table 19: Drawings Provided by Agriculture and Agri-Food Canada (continued)

Drawing No.	Date	Title	Issued	File No.
208526	2011	Highfield Reservoir CL Profile Main Dam Embankment and Drill Hole Logs – Sta. 300 to Sta. 600 Sheet 3 of 6	AESB- DGSA	
208527	2011	Highfield Reservoir CL Profile Main Dam Embankment and Drill Hole Logs – Sta. 600 to Sta. 900 Sheet 4 of 6	AESB- DGSA	
208528	2011	Highfield Reservoir CL Profile Main Dam Embankment and Drill Hole Logs – Sta. 900 to Sta. 1200 Sheet 5 of 6	AESB- DGSA	
208529	2011	Highfield Reservoir CL Profile Main Dam Embankment and Drill Hole Logs – Sta. 1200 to Sta. 1500 Sheet 6 of 6	AESB- DGSA	
208533	2011	Highfield Reservoir Capacity and Flood Area Curves Highfield Reservoir	AESB- DGSA	



APPENDIX A

Field Inspection



SITE INSPECTION: HIGHFIELD DAM

INSPECTED BY : Andrew Szojka; Ron Kitagawa (Golder)

INSPECTION DATE August 27, 2012

CLIENT PM : Garth Haack (AAFC)

PROJECT SITE : Agriculture & Agri-Food Canada: Highfield Dam

PROJECT No. 1213450020

PRE-DESIGN STUDY (SCC#6) – GENERAL SITE INSPECTION

Site Conditions

Temperature, °C			General	Reservoir Level, m
Min	Mean	Max	Clear and sunny, some scattered clouds, light to moderate wind from the SSE, no precipitation.	722.442m *
11.5	18.6	30		

* Provided by e-mail from the AAFC September 4, 2012 based on water level recorder (Calibrated August 9, 2012).

Special Considerations

- Golder Health, Safety and Environment Plan
- Field Level Hazard Assessment
- Confined Space Certification and Documentation (Plan), including Confined Space Entry Forms & Permits
- Fall Protection Training
- Gate actuator lockout locks
- AAFC provided “Highfield Dam – Low Level Outlet Works: Procedures for Inspection of Downstream Conduits”
- Dewatering pumps, air ventilation equipment, ladders, ropes (recovery system), redundant air monitors.

Special Note

This inspection does not constitute a dam safety inspection. The purpose of this inspection is to gather generalized site reconnaissance information over the entire dam site with specific focus on water conveyance structures. Water conveyance structures include the East and West Low Level Outlets, ELLO and WLLO, respectively, the West (earth cut emergency/overflow) Spillway and a portion of the Herbert canal. General observations are recorded regarding the main dam and downslope / toe areas to record features and observations from an environmental, civil design and construction perspective. The inspections were carried out having reviewed available record drawings and inspection reports, construction records pertaining specifically to the ELLO, and anecdotal information provided by the AAFC before and during the site inspection.

Dewatering and ventilation assistance was provided by AAFC staff. Assistance for confined space entry was provided by Garth Haack and Bob Mills of AAFC.

Follow-up surveys were completed by AAFC (see attached).



SITE INSPECTION: HIGHFIELD DAM

Inspection Notes

Area	Element	Observation
Main Dam	Site Access	<ul style="list-style-type: none"> • Travel about 31 km east from Swift Current along Highway No. 1 • Travel south about 7 km by rural road to Highfield dam • Good access to right abutment of the dam • Good signage at the dam; site was gated but not locked at the time
	Abutments	<ul style="list-style-type: none"> • Good, well established vegetation • No erosion gullies or major erosion features along either abutment • some shrubs and trees adjacent the abutments on reservoir side • Good signage at the dam; site was gated but not locked at the time
	Upstream slope	<ul style="list-style-type: none"> • reservoir side armor visible with some beaching • Armor appears gap graded • Armor consists of rounded field stone (est. $d_{50} = 250 - 300$ mm) • Some voiding and erosion to bedding material observed • Vegetation growth in upper 1 m to crest, primarily tall grass
	Downstream slope	<ul style="list-style-type: none"> • Good vegetation consisting of tall grass • Some shrub growth advancing from the toe up onto the dam slope • Toe areas dry enough to walk across however past moisture evident • Some saturated areas and ponded water near and at oxbow areas • Construction through wet areas will required dewatering measures • Construction care of water or protection of watercourses paramount • Instrumentation will require protection and raising during construction • Livestock freely roam the area and will require controls • Downstream slope appears generally uniform • some minor overstep areas near dam to toe transitions • one animal burrow found
	Crest	<ul style="list-style-type: none"> • Generally uniform visually with one minor depression / tire rut area • Crest covered in thin veneer of granular topping • Good vegetation control on crest • Crest accessible from both sides of the dam • The crest is not accessible from the west during spillway operations • No at surface anomalies along either LLO alignment
	Toe	<ul style="list-style-type: none"> • Travel about 31 km east from Swift Current along Highway No. 1 • Travel south about 7 km by rural road to Highfield dam • Good access to right abutment of the dam • Good signage at the dam; site was gated but not locked at the time



SITE INSPECTION: HIGHFIELD DAM

East Low Level Outlet	Intake	<ul style="list-style-type: none"> • submerged, not visible
	Gatewell	<ul style="list-style-type: none"> • Access from crest • Multiple locks and lock types observed • Danger signage observed • Gate actuator in good condition, well painted, and locked • Minor runoff erosion around gatewell riser • Inside gate well, air vent pipe observed corroded
	Low Level Conduit	<ul style="list-style-type: none"> • Poor gate seal across the sill, good seal on the sides and across the top of the gate. Gate in generally fair condition. • Air vent leakage heavy from gatewell (ventilation not possible) • Combined gate sill leakage and air vent leakage yields ~100 mm water depth at CMP outlet to concrete outlet stilling basin • Main gate downstream plate face predominated by rust nodules • Concrete conduit shows evidence of concrete pour lines, cracking, crack repairs, and cold joint cracks, most seeping or show signs of prior seepage with the development of deposits (efflorescence) and oxide stains. No iron bacterial observed • Some concrete pop-off areas and honeycombing with one pop-off revealing a short section of reinforcing steel • Concrete crack repair areas show signs of re-cracking w/seepage • Concrete invert shows signs of surface concrete paste erosion but to a degree not out of the ordinary for a structure of its age • transition area to 1.5 m diameter CMP in fair condition • CMP is galvanized for a short distance and shows signs of surface rusting where galvanization has worn away; primarily below the centre and around the invert • Asphalt coated and riveted CMP appears in fair condition • One section of CMP is highly oxidized • Transition from 1.5 m diameter to 1.6 m diameter culvert in good condition (swallow nest observed) • One puncture observed near the crown and close to the CMP outlet to the stilling basin; surmised to have occurred during construction • Junction of CMP to concrete stilling basin is in good condition
	Concrete Stilling Basin	<ul style="list-style-type: none"> • Constructed in 1991; concrete is in good condition; no fencing provided • Minor cracks observed on sidewalls with no deformations or opening • Invert transition to stilling basin appears to be in good condition • Stilling basin completely submerged; basin features not visible • Armor at outlet appeared sparse around stilling basin wing walls but



SITE INSPECTION: HIGHFIELD DAM

		good within the transition area to Rush Lake Creek; bulrush in creek
West Low Level Outlet	Intake	<ul style="list-style-type: none"> submerged, not visible
	Gatewell	<ul style="list-style-type: none"> Access from crest Access panel and gate actuator locked Danger signage observed Gate actuator in good condition, well painted Minor runoff erosion around gatewell riser Water in gate well covered in algae (potential O₂ depleted area)
	Low Level Conduit	<ul style="list-style-type: none"> Good gate seal across the sill with minor leakage at bottom corners and good seal on the sides and across the top of the gate No air vent water leakage and good confined space ventilation provided Roughly half of the gate's downstream plate shell face shows evidence of oxidation Concrete conduit generally in sound condition for the structure's age showing normal evidence of concrete pour lines, minor cracking and cold joint cracks, minor or signs of prior seepage with minor efflorescence and oxide stains. No iron bacterial present Some concrete pop-off areas and honeycombing Concrete invert shows signs of surface concrete paste erosion but to a degree not out of the ordinary for a structure of its age Transition to concrete stilling basin is in good condition Maximum operating water line visible inside the entire length of the concrete conduit
	Concrete Stilling Basin	<ul style="list-style-type: none"> The concrete outlet transitional stilling basin shows signs of its age Invert erosion observed Earth fill to the top of the outlet basin walls poses risk with no fencing Stilling basin partially submerged at time of inspection Armor at outlet appeared over-piled. Record photographs show rock armor inside the concrete transition area – surmise this rock has washed out and piled in place just outside of the stilling basin Transition area to Herbert canal shows signs of bank sloughing and dominated by tall grass vegetation
Overflow (earth cut) Spillway	Inlet channel	<ul style="list-style-type: none"> Heavy tall grass vegetation Cattle roaming freely; likely best access to reservoir for drinking Trapezoid channel shape visible Evidence of standing water Road crossing at dam centreline slightly raised above channel invert



SITE INSPECTION: HIGHFIELD DAM

	Outlet channel	<ul style="list-style-type: none">• Trapezoidal channel clearly evident with tall grass vegetation
	Confluence w/Herbert Canal	<ul style="list-style-type: none">• Evidence of spillway operation or preferential area drainage outlet point to Herbert canal by channel erosion/scour features and a delta of sediments deposited in the Herbert canal• the opposite side of the Herbert canal (right bank) appears scoured by inflows from the spillway
Herbert Canal	WLLO to just beyond the spillway point of confluence	<ul style="list-style-type: none">• shallow bed slope, no armor on side slopes• historical and some recent bank sloughing evident• Tall grass vegetation predominant over most of the side slopes except through the spillway confluence area to a distance downstream along the Herbert canal• infilling of sediment from spillway or local drainage inflow areas are creating meandering patterns along the invert at low flows – this may pose longer term maintenance issues• Overbuild of the right embankment to Herbert canal nearest the spillway (conforms to available record drawings)

Photographs taken during the inspection follow. Select photographs are annotated. Contact sheets are provided for the remaining photograph collection.



PHOTO RECORD - HIGHFIELD DAM: EAST LOW LEVEL OUTLET



East LLO air vent leakage from gatewell riser at main control gate.



Efflorescence along vertical crack. Oxide stains, seepage



View towards main gate, air vent and gate leakage. Pour lines and oxide stains visible.



Cracking, seepage and efflorescence.



Poor gate seat along invert. Gate skin shows rust nodules over entire surface.



Seepage, oxide staining, and efflorescence along cracks and pour lines.



PHOTO RECORD - HIGHFIELD DAM: EAST LOW LEVEL OUTLET



Existing crack with separation (left side looking downstream).



Concrete to CMP transition. Galvanized CMP w/advanced surface oxidation (rust).



Wall Crack with separation (right side looking downstream).



Rust over 60 % of the surface, primarily along invert.



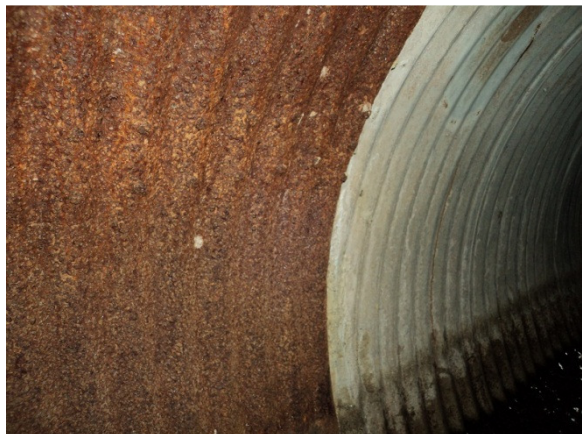
Wall crack with rebar stub end showing (oxidized)



Riveted and coated CMP w/



PHOTO RECORD - HIGHFIELD DAM: EAST LOW LEVEL OUTLET



Extent of rusting between sections. Coating failure.



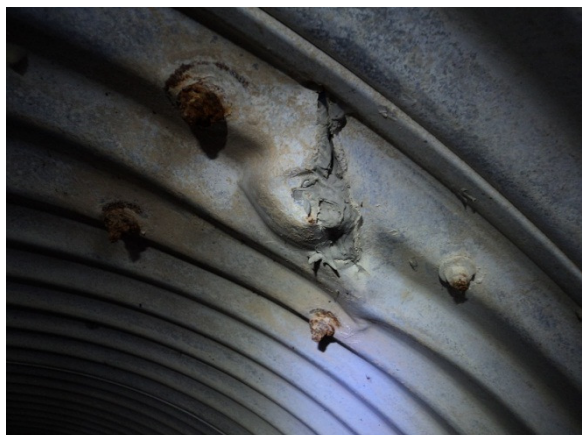
View upstream to 1.5 m to 1.6 m diameter CMP.



Transition from 1.5 m dia. to 1.6 m dia. CMP. Note swallow nest.



AAFC photo illustrates badly corroded air vent at waterline. Photo taken October 2012.



Puncture near crown of 1.6 m dia. CMP. Suspect occurred during construction.



Close-up shows corrosion and break through rupture. Photo taken October 2012



PHOTO RECORD - HIGHFIELD DAM: EAST LOW LEVEL OUTLET



View looking downstream along stilling basin. No protective fence.



East LLO gatewell CMP riser and armor condition.



View looking upstream toward concrete outlet stilling basin. Note basin armor. Concrete in good condition.



View of vegetation inside Rush Lake Creek at East LLO outlet.



East LLO gatewell at existing dam crest.



View from crest looking downstream along Rush Lake Creek and the East LLO outlet structure.



PHOTO RECORD - HIGHFIELD DAM: WEST LOW LEVEL OUTLET



Waterface transition from box culvert pipe section to West LLO stilling basin.



Overall view towards gate shows waterline staining, overall box condition, invert wear.



Good gate seating with minor leakage at right and left bottom corners of gate.



Honeycombing and crack formation..



Overall condition of gate appears good with oxidization of centre. Roof vent dry.



Crack and spall with minor seepage and efflorescence staining.



PHOTO RECORD - HIGHFIELD DAM: WEST LOW LEVEL OUTLET



Overall view back toward gate to illustrate general condition of West LLO conduit



West LLO gatewell view from top of gatewell. Algae growth may produce oxygen deficient atmosphere.



West LLO gatewell w/gate actuator lockout. Shows vent pipe and confined space entry ventilation.



Rock pile at end of West LLO transition to the Herbert Canal. Note general condition of canal banks.



West LLO transition wall to stilling basin w/roughened wall for energy dissipation.



View towards West LLO transition section. Note existing ground to top of wall. No protective fence.



PHOTO RECORD - HIGHFIELD DAM: MAIN DAM



Rural road access and project signage on approach to east abutment of the Highfield dam and East LLO.



General view looking west across upstream face armor protection and vegetation.



Overall view looking toward east abutment area to proposed East LLO and drop inlet spillway location.



General view of armor looking east toward the East LLO area. Note rounded field stone armor 3-400 mm



General view looking west from east abutment and existing East LLO alignment to wet dam toe area.



General view looking west across Highfield dam crest. Note granular topping and very flat terrain.



PHOTO RECORD - HIGHFIELD DAM: MAIN DAM



General view looks east at downstream slope and toe area vegetation cover.



General view of downstream toe area of the Highfield dam w/oxbows, vegetation and standing water.



Newly installed instrumentation and protection from cattle interaction.



General view looking east from the West LLO and Herbert canal access trail. Note toe area vegetation.



Evidence of borrowing animals in the downstream dam face.



Standing water in oxbows abutting to the toe of the Highfield dam.



PHOTO RECORD - HIGHFIELD DAM: SPILLWAY & HERBERT CANAL



West Spillway looking toward the Highfield reservoir. Free roaming cattle.



General view looking downstream along West Spillway noting vegetation cover and trapezoid shape.



View looks west across west spillway along dam crest centreline access road.



View looking north from the dam crest along West LLO alignment and discharge to Herbert Canal.



View looking downstream along west spillway from centreline access road.



View looking back toward the West LLO from right embankment overbuild and confluence w/spillway.



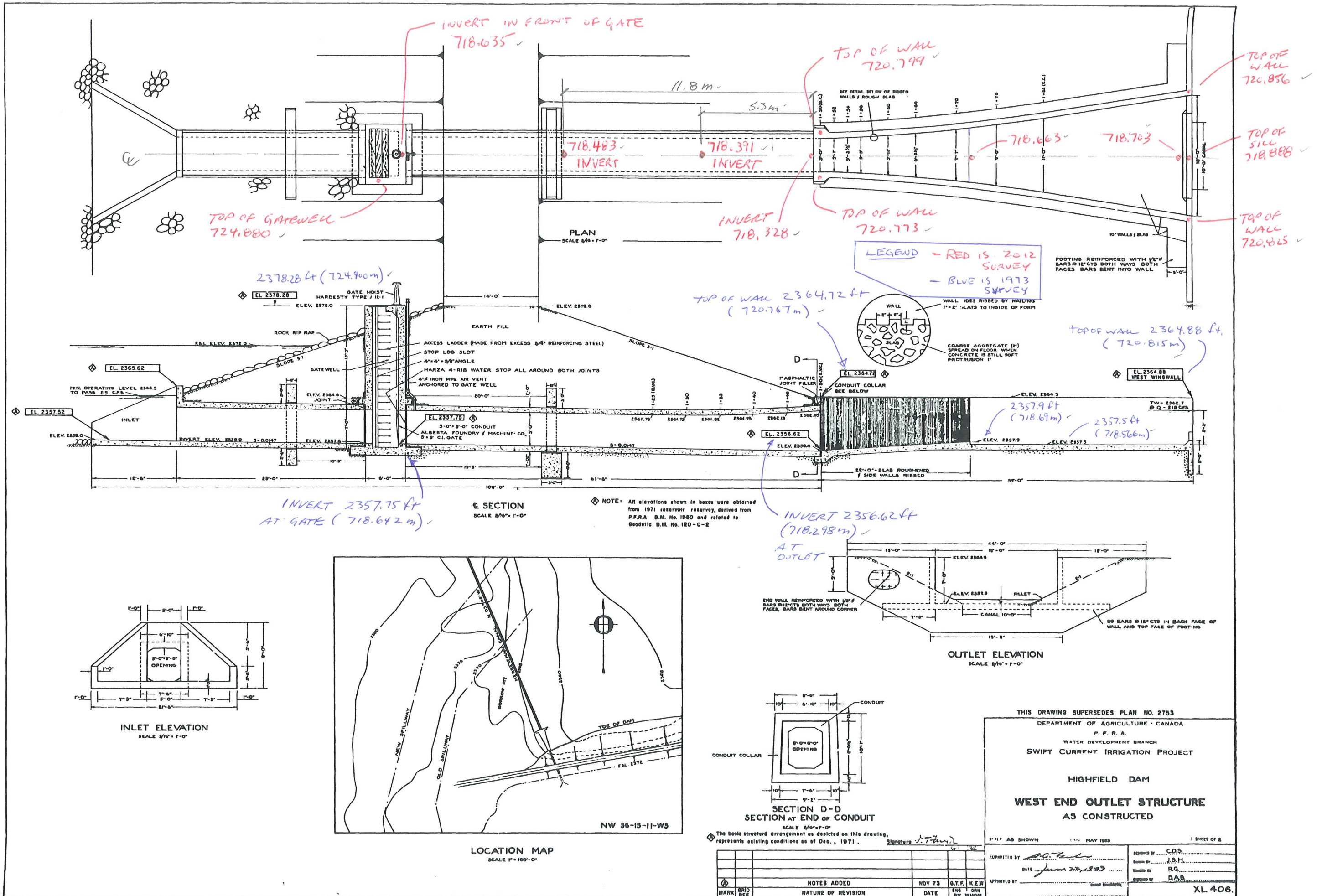
PHOTO RECORD - HIGHFIELD DAM: SPILLWAY & HERBERT CANAL

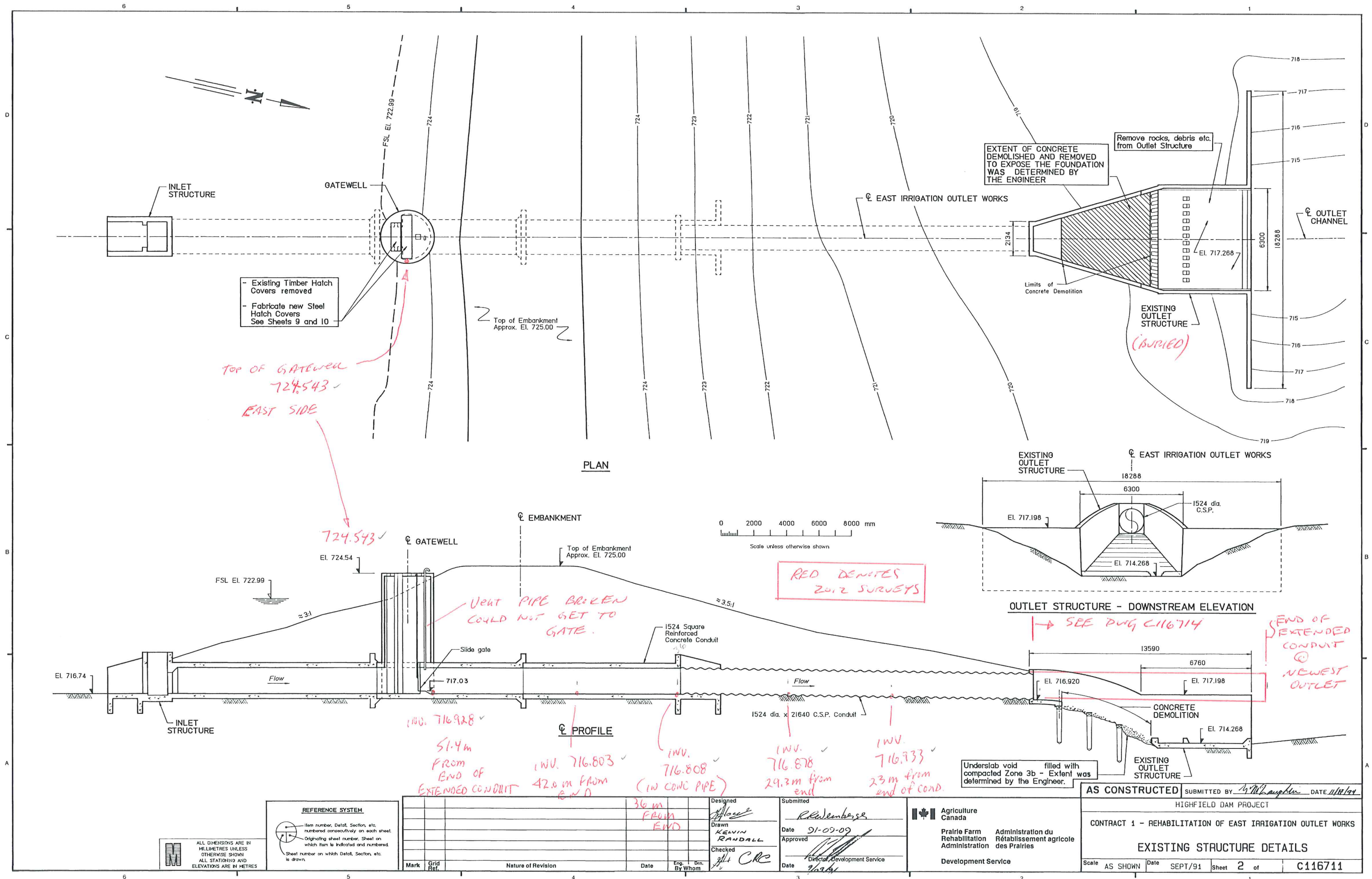


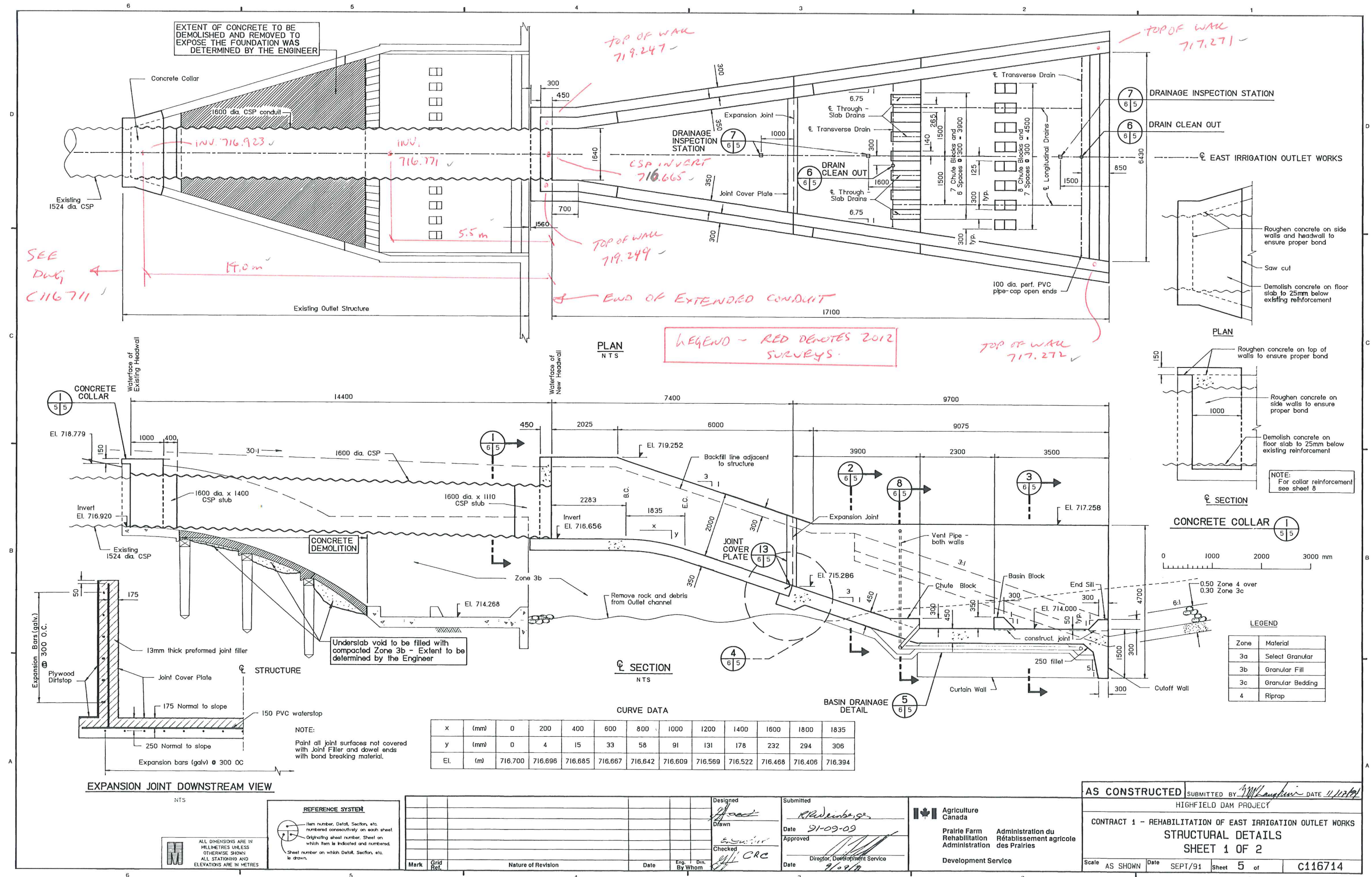
View looking west toward West Spillway confluence with the Herbert Canal. Note erosion and scour at point of entry.



View looking downstream from West Spillway confluence with the Herbert Canal. Note bank erosion, sloughing, infilling, bare earth areas and vegetation.







HIGHFIELD DAM CONDUIT DIMENSIONS AND INVERT ELEVATIONS

SURVEY AUGUST 28, 2012

EAST OUTLET

Distance From Outlet End of Conduit (m)	Height (in)	Width(in)	Invert Elevation (m)	Comment
0.0	65	61	716.665	Outlet end Csp pipe
5.5	64	62	716.771	Csp pipe
14.0	60	58	716.923	Csp pipe
23.0	62	57	716.933	Csp pipe
29.3	63	56	716.878	Csp pipe
36.0	61	61	716.808	Concrete pipe
42.0	62	60	716.803	Concrete pipe
51.4	61	61	716.928	Concrete pipe approx. 1 m from gate (leak at vent pipe)

WEST OUTLET

Distance From Outlet End of Conduit (m)	Height (in)	Width (in)	Invert Elevation (m)	Comment
0.0	72	60.5	718.328	Outlet end of conc pipe
5.3	60	60	718.391	Conc pipe
11.8	60	60	718.483	Conc pipe
19.6	60	60	718.635	Gate – Conc pipe



APPENDIX B

2007 AAFC Flood Hydrology Study



MEMORANDUM

NOTE DE SERVICE

TO
À

Garth I. Haack
Manager, Planning Unit
AAFC Operated Projects
Regina, Saskatchewan

FROM
DE

Fred R. J. Martin, P.Eng.
Manager, Surface Water Unit
Water Planning & Sourcing Division
AAFC, Regina

SECURITY - CLASSIFICATION - DE SÉCURITÉ

N/A

OUR FILE - NOTRE RÉFÉRENCE

4547:928-7H4

YOUR FILE - VOTRE RÉFÉRENCE

N/A

DATE

November 21, 2007

SUBJECT
OBJET

Highfield Dam – Updated Flood Frequency Analyses

In response to a request by Glenn McLaughlin via an e-mail to Ron Woodvine dated June 22, 2007, I have conducted detailed flood frequency analyses of both snowmelt and rainfall events for the purpose of reassessing and updating the inflow hydrographs for Highfield Dam. The results of the analyses are provided in Table 1.

Table 1

Highfield Dam – Updated Inflow Estimates

Return Period	Snowmelt (Spring) Events		Rainfall (Summer) Events	
	Instantaneous Peak Flow (m ³ /s)	Runoff Volume (dam ³)	Instantaneous Peak Flow (m ³ /s)	Runoff Volume (dam ³)
1:2	13	5,200	1.2	82
1:5	30	11,200	7.6	696
1:10	41	15,100	19	2,020
1:20	53	19,200	39	4,610
1:50	71	24,700	80	10,600
1:100	85	29,200	123	17,400
1:200	101	33,900	180	26,900
1:500	121	40,100	274	43,600
1:1,000	139	45,500	361	60,300

Associated snowmelt and rainfall hydrographs are attached.

With the exception of rainfall runoff volumes at relatively large return periods (i.e. 1:500 and greater), the magnitude of both the flood peaks and the runoff volumes for rainfall and snowmelt events are somewhat

lower than values determined in the previous analysis (August 2003). While the results of the previous analysis are considered to be appropriate for the database (i.e. a 36-year period 1966-2001) that was available at that time, the primary reason for this difference in the magnitude of the values is that the current analyses are based on six additional years of data (a 19% increase in the database period). Furthermore, all relationships (i.e. MDM versus instantaneous and MDM versus runoff volume) were revised using the current available database 1965-2006. The additional years of record contained the 5th largest snowmelt event (MDM of 19.3 m³/s) in 2003 and the largest recorded rainfall event (MDM of 37.3 m³/s) in 2005.

Snowmelt Events

The snowmelt analysis was based solely on recorded maximum daily mean (MDM) flows of Rushlake Creek Above Highfield Reservoir (05JC004) for the period 1965-2006. No attempt was made to extend the database by correlation with data for stations in other basins because there did not appear to be any hydrologically-similar basins in the vicinity for which data was available for extension purposes. The resulting array of recorded (1965-2006) MDMs at 05JC004 was analyzed using Environment Canada's Consolidated Frequency Analysis (CFA) program to determine the most appropriate frequency distribution. While the Three-Parameter Lognormal frequency distribution provided the best statistical data fit of the four distributions (Generalized Extreme Value, Three-Parameter Lognormal, Log Pearson Type III, and Wakeby) that were examined, it was not a satisfactory fit from a visual perspective. Thus, a frequency curve was "drawn by eye" that fit the plotted data and provided realistic values at larger return periods.

Snowmelt inflow hydrographs to Highfield Reservoir were developed from the derived parameters at station 05JC004 located upstream of Highfield Reservoir. MDMs for nominal flood events (i.e. 1:2 to 1:1,000) at 05JC004 were multiplied by an effective drainage area ratio to a nominal exponent of 0.75 to obtain inflow MDMs to Highfield Reservoir. These inflow MDMs were subsequently multiplied by the corresponding instantaneous/MDM flow ratio developed for 05JC004 and a Fuller Factor adjustment ratio to obtain instantaneous flows to Highfield Reservoir for nominal flood events. Runoff volumes for associated nominal flood events at 05JC004 (as determined using the MDM versus runoff volume relationship for 05JC004) were multiplied by a corresponding contributing drainage area ratio (linear relationship between the effective drainage area ratio at a 1:2 event and the gross drainage area ratio at a 1:500 event on arithmetic probability paper) to obtain runoff volumes for nominal flood events at Highfield Reservoir. A typical rising limb "k" value (11.531/day) was determined by assessing the rising limbs of the six largest snowmelt runoff events recorded at 05JC004. An in-house program HYDROGRAPH was used to generate the nominal inflow hydrographs using the derived rising limb "k" value (a corresponding value of 1.107 was used for a hydrograph coordinate interval of one hour), the estimated instantaneous flow, and the estimated runoff volumes at Highfield Reservoir.

Rainfall Events

The rainfall analysis was based solely on recorded maximum daily mean (MDM) flows of Rushlake Creek Above Highfield Reservoir (05JC004) for the period 1965-2006. No attempt was made to extend the database by correlation with data for stations in other basins because there did not appear to be any hydrologically-similar basins in the vicinity for which data was available for extension purposes. The resulting array of recorded (1965-2006) MDMs at 05JC004 was analyzed using Environment Canada's Consolidated Frequency Analysis (CFA) program to determine the most appropriate frequency distribution. While the Log Pearson Type III frequency distribution provided the best visual and statistical data fit of the four distributions (Generalized Extreme Value, Three-Parameter Lognormal, Log Pearson Type III, and Wakeby) that were examined, it was not a satisfactory fit because the frequency curve was concave up. Thus, a frequency curve was "drawn by eye" that fit the plotted data and provided realistic values at larger return periods.

Rainfall inflow hydrographs to Highfield Reservoir were developed from the derived parameters at station 05JC004 located upstream of Highfield Reservoir. MDMs for nominal flood events (i.e. 1:2 to 1:1,000) at 05JC004 were multiplied by an effective drainage area ratio to a nominal exponent of 0.75 to obtain inflow

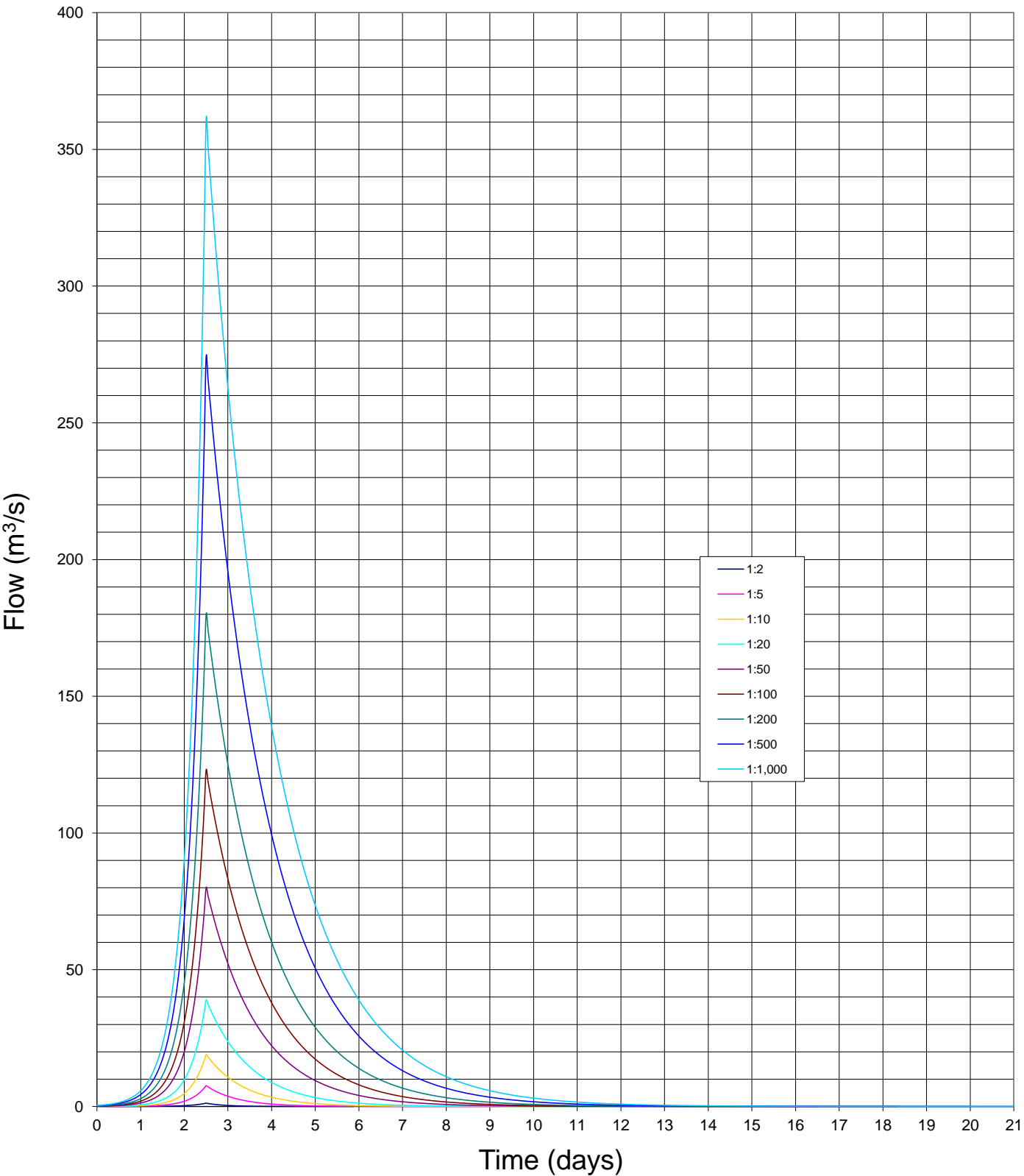
MDMs to Highfield Reservoir. These inflow MDMs were subsequently multiplied by the corresponding instantaneous/MDM flow ratio developed for 05JC004 and a Fuller Factor adjustment ratio to obtain instantaneous flows to Highfield Reservoir for nominal flood events. Runoff volumes for associated nominal flood events at 05JC004 (as determined using the MDM versus runoff volume relationship for 05JC004) were multiplied by a corresponding contributing drainage area ratio (linear relationship between the effective drainage area ratio at a 1:2 event and the gross drainage area ratio at a 1:500 event on arithmetic probability paper) to obtain runoff volumes for nominal flood events at Highfield Reservoir. A typical rising limb "k" value (15.893/day) was determined by assessing the rising limbs of the six largest rainfall runoff events recorded at 05JC004. An in-house program HYDROGRAPH was used to generate the nominal inflow hydrographs using the derived rising limb "k" value (a corresponding value of 1.122 was assumed for a hydrograph coordinate interval of one hour), the estimated instantaneous flow, and the estimated runoff volumes at Highfield Reservoir.

Fred R. J. Martin
Manager, Surface Water Unit

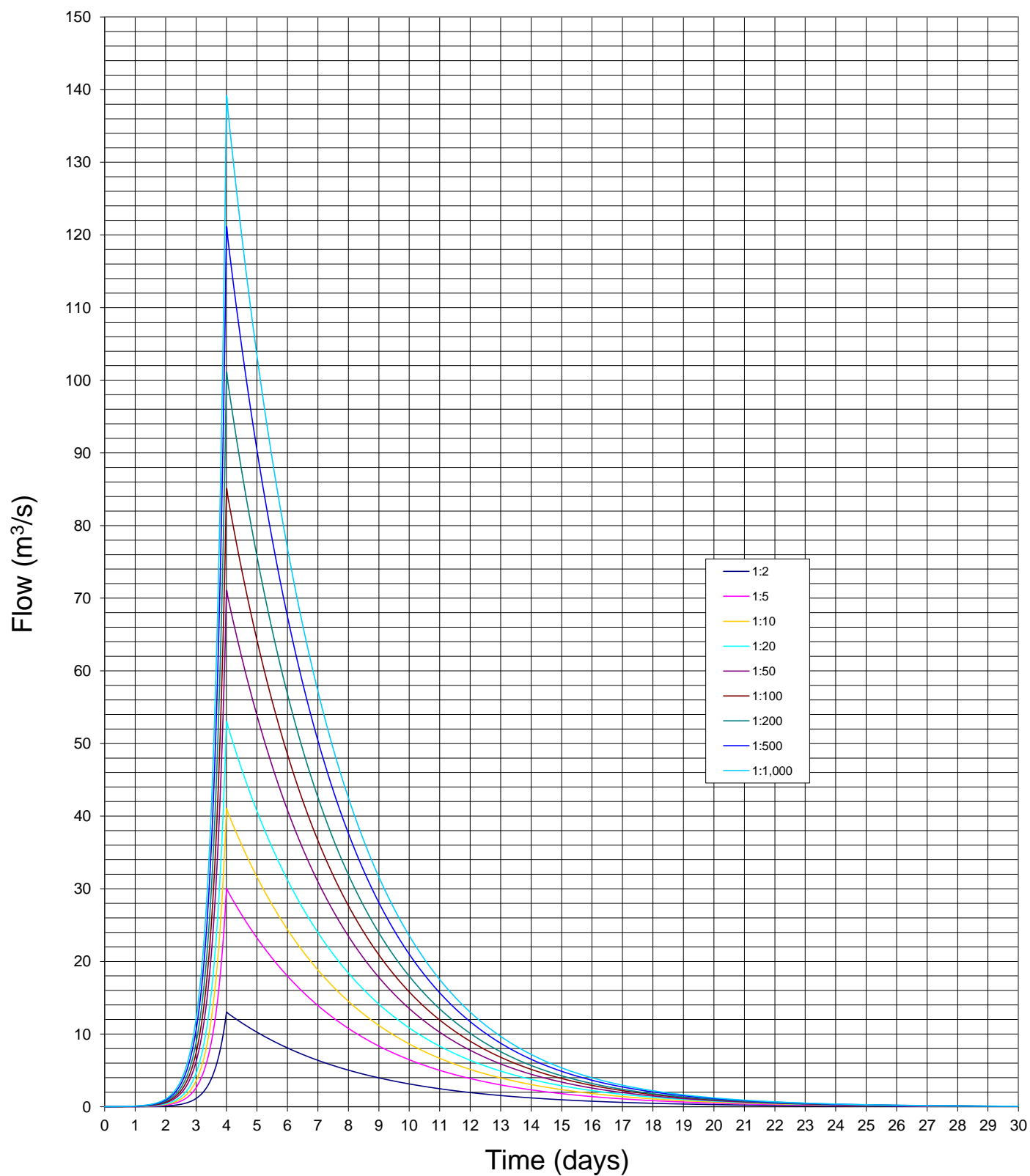
Attachments

cc: R. Woodvine
G. McLaughlin

Rainfall Inflow Hydrographs to Highfield Reservoir



Snowmelt Inflow Hydrographs to Highfield Reservoir





APPENDIX C

Structural Analyses

DATE Sept 25, 2012**PROJECT No.** 12-1345-0020**TO** Andrew Szjoka
Golder Associates (Civil Design Group)**CC** Max Abtahi**FROM** Kamran Kambakhshfard**EMAIL** kamran_Kambakhshfard@golder.com**STRUCTURAL ASSESSMENT OF EAST LOW LEVEL OUTLET**

Background

This memorandum provides an assessment of the east low level outlet structure as it has been identified in the Technical Proposal portion of the Highfield Dam Service Contract No.6 Project Rehabilitation Pre-Design Study. This work is a cursory analysis, associated with raised embankment loading. It will form a portion of the Pre-Design activity package, and is labeled "Task 1" in the above referenced proposal. The analysis was performed based on a review of the available data; including construction record drawings, inspection information, the rehabilitation report, and photos. The east low level outlet is located near the east abutment of the Highfield Dam and serves as a one of the three (3) discharge facilities for Highfield Dam. The other two would be the West Low Level Outlet structure and Spillway. The East Low Level Outlet consists of the series of structural components as per following order;

- Reinforced concrete inlet structure, built in 1942 and modified in 1952;
- Reinforced concrete upstream conduit with square shaped cross section, built in 1942;
- Gate well structure, built in 1952, with additional modifications such as ladder and access hatch in 1992;
- Reinforced concrete downstream conduit with square shaped cross section, which is a continuation of the upstream conduit, built in 1942;
- Corrugated steel pipe (CSP) upstream conduit extension, built in 1950 and replaced in 1969;
- Reinforced concrete stilling basin structure, built in 1951, which is no longer in service;
- Corrugated steel pipe (CSP) downstream conduit extension, built in 1991; and
- Reinforced concrete stilling basin that serves as an outlet.

General Discussion

In general some components of the Reinforced Concrete structure, such as jointing materials, have a shorter life and will require maintenance or replacement. Microclimate exposures, atmospheric zones, and submergence



affect the life cycle. Considering the age of the reinforced concrete conduit and the condition of its components, attention should be given to its remaining life. The American Concrete Institute (ACI 350R) states that when all relevant loading conditions are considered, the design should provide safety and serviceability, with a life expectancy of 50 to 60 years for structural concrete.

When it comes to corrugated steel pipe (CSP), static potential readings could reveal the rate of corrosion between the soil and the culvert to establish the conditions. This type of information is a realistic method for estimating the remaining life and predicting the future performance of a metal component.

Corrosion protection (CP) is an electrical method of preventing soil-side corrosion and arresting corrosion on existing culverts. Cost tends to be the overriding consideration when deciding whether or not to provide CP. The question one should ask is whether the cost of providing and maintaining CP is less than the cost of prematurely replacing the culvert. Industry practice is not to place cathodic protection on existing culverts that are structurally inadequate, or are already in an advanced stage of corrosion. Therefore, structural adequacy and corrosion status should be determined prior to implementation of CP to begin with. Alternative coatings or materials are also options and the same comparison of merits should be considered for those methods (the cost versus the estimated increase in life expectancy).

The gate well structure should be raised to match the top elevation of the dam embankment. It was visually inspected in 2010 and has been rated as “poor” with heavy rust on the vent, stem, and stem guide brackets, and is severely leaking. The cost of a replacement would probably be less than repairs and modification of the existing one, and consequently deliver a better life expectancy, but this requires a proper assessment.

CAN CSA A23.3 suggest conducting a thorough field investigation of the following to perform a strength evaluation by analytical means:

- Dimensions and details of members As-Built condition
- Properties of the materials
- Pertinent conditions of the existing structure

Assumptions

Concrete:

- $f'_c = 10.3\text{Mpa}$ to 19.4Mpa , carried out analysis for a range of crushing concrete strength and compared the results (Ref. Concrete Condition Survey, table 3)
- $f_y = 276\text{Mpa}$ (Ref. Engineering Evaluation of Reinforcement In old Reinforced Concrete Structures, CRSI, attached)
- Reinforcement details, as per drawing records

Corrugated Steel Pipe (CSP):

- Assumed a 12 Gauge (0.1046" thick), 1524 diameter, no thickness readings available on records. Assumed weakest profile for analysis for both CSP and multi-plate CSP at downstream and upstream.
- Poisson's Ratio = 0.33

Soil Properties:

- $\gamma = 21 \text{ kN/m}^3$, $\phi = 25$, $C' \text{ (cohesion)} = 7 \text{ kPa}$ for Embankment Fill
- $\gamma = 21 \text{ kN/m}^3$, $\phi = 25$, $C' \text{ (cohesion)} = 0 \text{ kPa}$ for Embankment Fill (Original, Ref. Highfield Foundation Assessment Report, table 6, page. 6)

For the alluvial soils at the Highfield Dam under the East Low Level Outlet Conduit, the following assumptions were recommended:

- Allowable bearing capacity: 46kPa to 60kPa
- Coefficient of subgrade reaction, $k_s \sim 5.5 \text{ MPa/m}$ (Soils are soft to firm silty clays; typical values range from 0 – 5 MPa/m for soft clays, and 5 – 10 MPa/m for firm clays – Canadian Foundation Engineering Manual, 4th edition)

Assessment Results

Based on latest edition of NBC (National Building Code of Canada) and Canadian design code, the reinforced concrete conduit has about 27% of capacity requirement of the current codes. This statement is result of the analysis due to the loading condition induced by raising embankment. The conduit is estimated to have about 36% of capacity requirement of the current code under the present loading condition.

The NBC (National Building Code of Canada) in Division A, Appendix A of its explanatory material suggests that the successful application of Code requirements to existing construction becomes a matter of balancing the cost of implementing a requirement with the relative importance of that requirement to the overall Code objectives. The degree to which any particular requirement can be relaxed without affecting the intended level of safety of the Code requires considerable judgment on the part of both the designer and the authority with jurisdiction in each case.

- Corrugated steel pipe (CSP) would theoretically be able to sustain the load increase of the embankment, assuming the condition of the pipe has not been compromised. Quantitative inspection records that include documentation about thickness readings, the quality of the coating, and the type of coating, especially at the downstream CSP would need to be completed to confirm the validity of the theory.
- Golder's geotechnical team has noted that the alluvial subgrade soils at the site are highly variable with a mixture of saturated clays, silty clay, sand, and silt. These soils are very susceptible to frost heave and settlement, and are not great foundation materials. Furthermore, this type of subgrade coupled with a provision to add more fill could produce settlement effects and result in a redistribution of loads.

Recommendations

- Obtain more accurate information regarding material properties and construction records, and validate the analysis of the reinforced concrete conduit.
- Review or determine the risks and costs associated with culvert failure in the future.

Limitations

- There are some limitations in the accuracy of given information, particularly in regards to the as-built records and material properties. We had to make a number of assumptions in order to carry out the analysis, since there are deficiencies in the available construction records at the time of analysis. The

results shall be verified and confirmed once there is an opportunity to obtain more accurate information, and to conduct a through review of the record drawings, material testing, and measurements.

References:

1. CAN CSA S6-06 – Canadian Highway Design Bridge Code.
2. CAN CSA A23.3-04 – Design of Concrete Structures.
3. ACI350R-06-American Concrete Institute, “Code Requirements for Environmental Engineering Concrete Structures and Commentary.”
4. CSPI-Corrugated Steel Pipe Institute “Handbook of Steel Drainage and Highway Construction.”
5. CRSI-Concrete Reinforcing Steel Institute Engineering Date Report Number 48 “Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures.”

Kamran Kambakhshfard, PEng.
Senior Structural Engineer

DATE Sept 25, 2012**PROJECT No.** 12-1345-0020**TO** Andrew Szjoka
Golder Associates (Civil Design Group)**CC** Max Abtahi**FROM** Kamran Kambakhshfard**EMAIL** kamran_Kambakhshfard@golder.com**STRUCTURAL ASSESSMENT OF WEST LOW LEVEL OUTLET**

Background

This memorandum provides an assessment of the west low level outlet structure as it has been identified in the Technical Proposal in the Highfield Dam Service Contract No.6 project rehabilitation Pre-Design Study. The scope of work is a cursory analysis associated with raised embankment loading. It will form a portion of the Pre-Design activity package, labelled "Task 2" in the above referenced proposal. This analysis has been performed based on a review of the available data, including the construction record drawings, inspection information, the rehabilitation report, and photos. The West Low Level Outlet is located near the west abutment of the Highfield Dam and serves as one of the three (3) discharge facilities for Highfield Dam. The other two are the East Low Level Outlet structure and the Spillway. The West Low Level Outlet is a reinforced concrete structure built in 1951 that includes an inlet and conduit structure.

General Discussion

In general some components of the reinforced concrete structure, such as jointing materials, have a shorter life than others and require maintenance or replacement sooner than others. There are microclimate exposures, atmospheric zones, and submergence that affect the life cycle. Considering the age of the reinforced concrete conduit and the condition of its components, prime attention should be given to its remaining life. The American Concrete Institute (ACI 350R) states that when all relevant loading conditions are considered, the design should provide safety and serviceability, with a life expectancy of 50 to 60 years for structural concrete.

The gate well structure should be raised to match the top elevation of the dam embankment. It was visually inspected in 2010 and no significant defects were logged.

CAN CSA A23.3 suggests conducting a thorough field investigation of the following to perform a strength evaluation by analytical means:

- Dimensions and details of members As-built condition
- Properties of the materials
- Pertinent conditions of the existing structure



Assumptions

Concrete:

- $f'_c = 18.4\text{MPa}$, air entrained but low at 4% (Ref. Power Point Presentation, page 54)
- $f_y = 276\text{MPa}$ (Ref. Engineering Evaluation of Reinforcement In old Reinforced Concrete Structures, CRSI, attached)
- Reinforcement details, as per drawing records

Soil Properties:

- $\gamma = 21\text{ kN/m}^3$, $\phi = 25$, C' (cohesion) = 7 kPa for Embankment Fill
- $\gamma = 21\text{ kN/m}^3$, $\phi = 25$, C' (cohesion) = 0 kPa for Embankment Fill (Original, Ref. Highfield Foundation Assessment Report, table 6, page. 6)

The geotechnical department did not investigate the shale at the site in great detail, however it is expected to be near the ground surface at the west outlet:

- Historical reports recommend an allowable bearing capacity of 400kPa for the shale.

Typical k_s values for very stiff clay would be 30 – 80 MPa/m (Canadian Foundation Engineering Manual, 4th Edition). $k_s \sim 48\text{ MPa/m}$ would be consistent with the historical bearing capacity for the shale.

Assessment Results

Based on latest edition of NBC (National Building Code of Canada) and Canadian design code, the reinforced concrete conduit has about 40% of capacity requirement of the current codes. This statement is result of the analysis due to the loading condition induced by raising embankment. The conduit is estimated to have about 50% of capacity requirement of the current code under the present loading condition.

NBC (National Building Code of Canada) in Division A, Appendix A of its explanatory material suggests that the successful application of Code requirements to existing construction becomes a matter of balancing the cost of implementing a requirement with the relative importance of that requirement to the overall Code objectives. The degree to which any particular requirement can be relaxed without affecting the intended level of safety of the Code requires considerable judgment on the part of both the designer and the authority having jurisdiction.

The outlet needs to be capped with a cover slab due to the extension of the embankment and according to primary objective of the scope (see attached details). The existing structure has been assessed for the impact of the additions, and the results show an overstressing of the bottom slab. We recommend that the bottom slab be reinforced at the detail stage.

Recommendations

- Obtain more accurate information about material properties and construction records, and validate the analysis of the reinforced concrete conduit.
- Review or determine the risks and costs associated with culvert failure in the future.

Limitations

There are limitations in the accuracy of the given information, particularly with regard to the as-built records and material properties. Since there were deficiencies in the available construction records at the time of analysis, we had to make a number of assumptions. The results shall be verified and confirmed once there is an opportunity to obtain more accurate information, and to conduct a through review of the record drawings, material testing, and measurements.

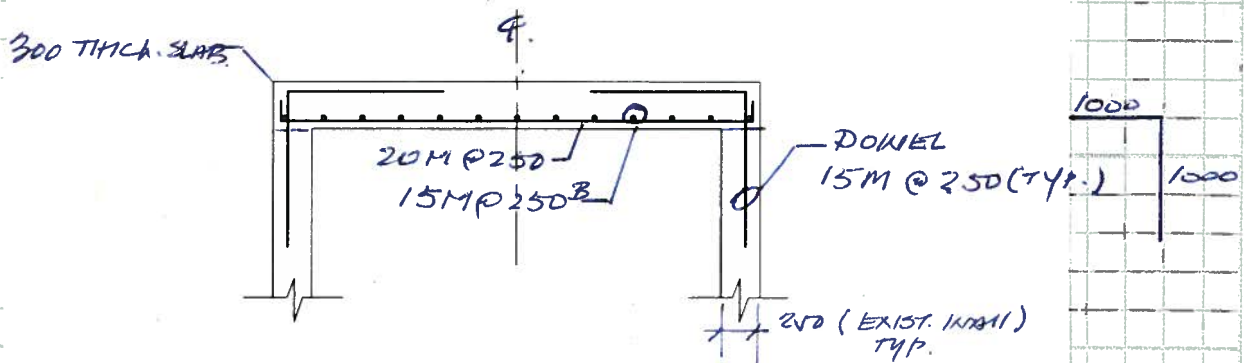
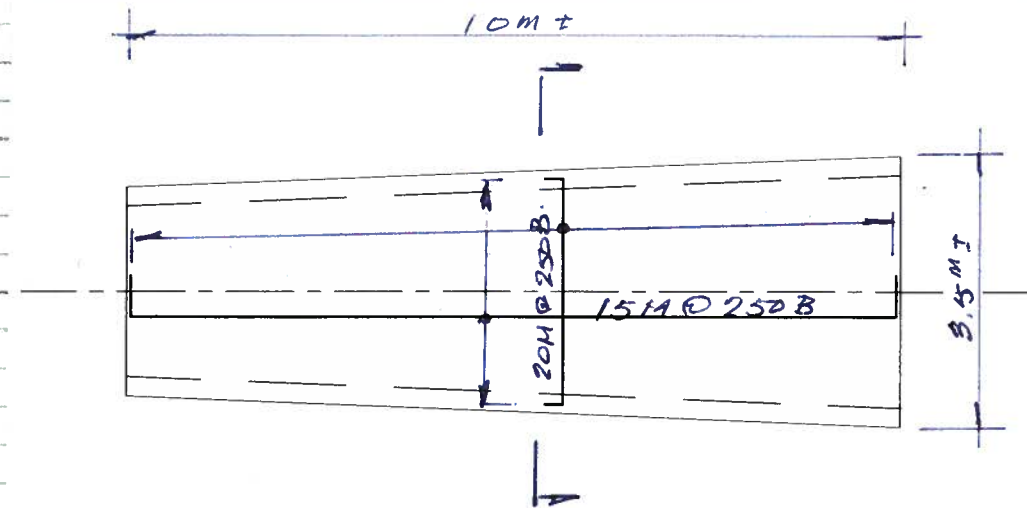
References:

1. CAN CSA S6-06 – Canadian Highway Design Bridge Code.
2. CAN CSA A23.3-04 – Design of Concrete Structures.
3. ACI350R-06-American Concrete Institute, "Code Requirements for Environmental Engineering Concrete Structures and Commentary."
4. CSPI-Corrugated Steel Pipe Institute "Handbook of Steel Drainage and Highway Construction."
5. CRSI-Concrete Reinforcing Steel Institute Engineering Date Report Number 48 "Evaluation of Reinforcing Bars in Old Reinforced Concrete Structures."

Kamran Kambakhshfard ,PEng.
Senior Structural Engineer



Subject <i>WEST LL. OUTLET COVER SLAB DETAIL</i>		
Job No.	Made by	Date
Ref.	Checked	Sheet
	Reviewed	of



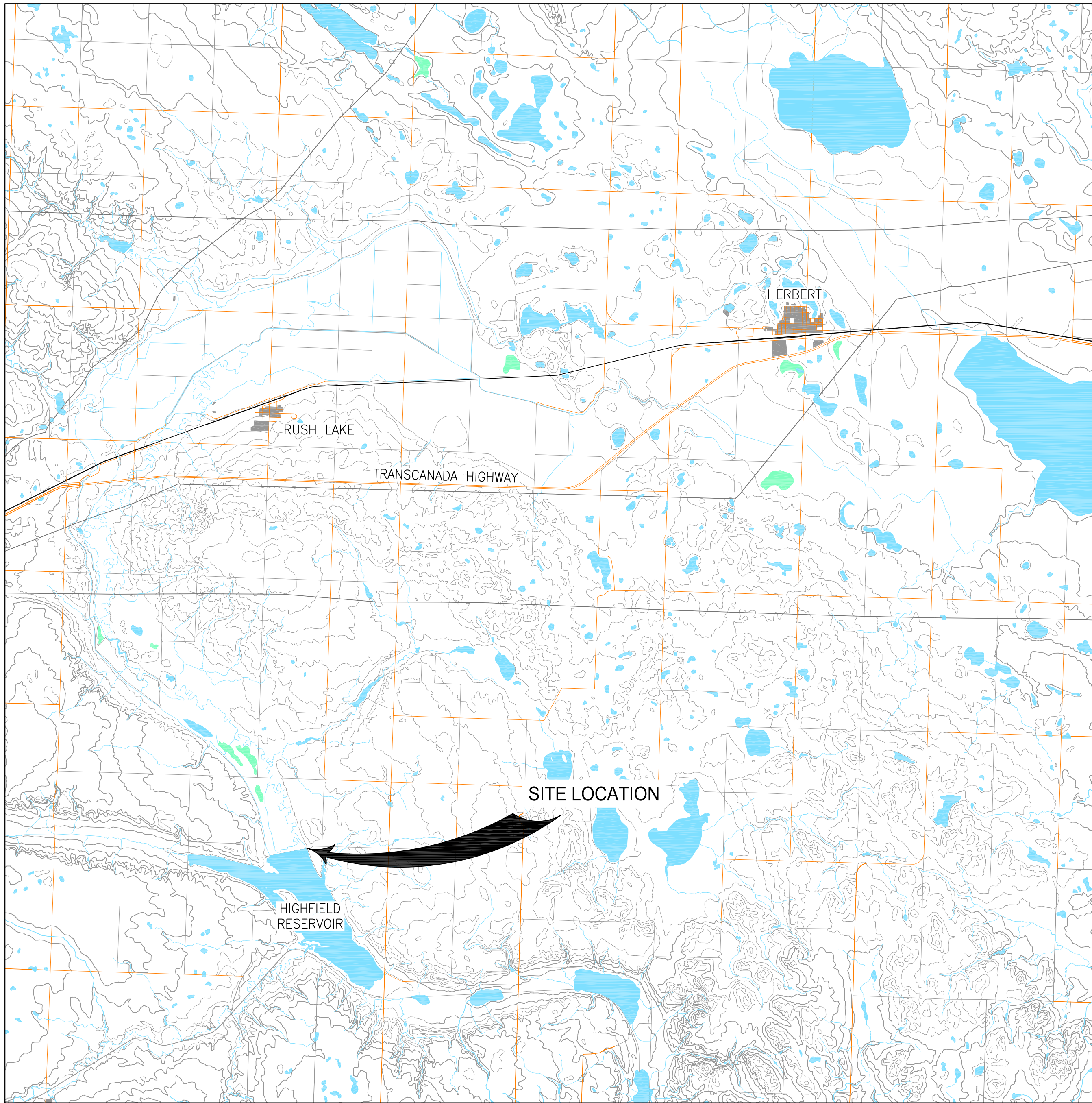
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APPENDIX D

Pre-Engineering Design Drawings

HIGHFIELD DAM REHABILITATION PROJECT PRE-DESIGN STUDY



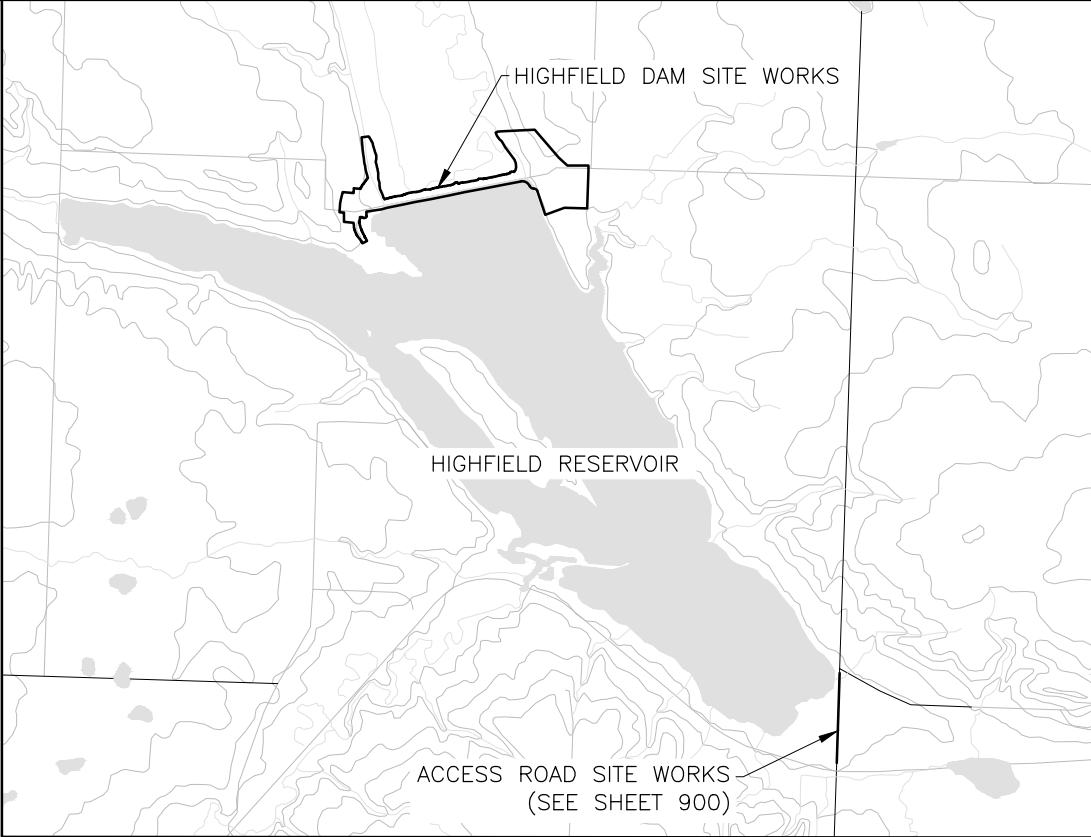
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Sheet List Table	
Sheet Number	Sheet Title
100	COVER SHEET, PROJECT KEY AND LOCATION PLAN
200	GENERAL ARRANGEMENT
300	SITE PLAN
400	RAISED EMBANKMENT PLAN AND PROFILE
410	RAISED EMBANKMENT SECTIONS AND DETAILS
500	EAST LOW LEVEL OUTLET AND SPILLWAY PLAN AND PROFILE
505	EAST LOW LEVEL OUTLET AND SPILLWAY CONDUIT PLAN AND PROFILE
510	EAST LOW LEVEL OUTLET AND SPILLWAY SECTIONS AND DETAILS (1 OF 2)
515	EAST LOW LEVEL OUTLET AND SPILLWAY SECTIONS AND DETAILS (2 OF 2)
600	WEST LOW LEVEL OUTLET PLAN AND PROFILE
610	WEST LOW LEVEL OUTLET SECTIONS AND DETAILS
700	WEST SPILLWAY PLAN AND PROFILE
710	WEST SPILLWAY SECTIONS
720	WEST SPILLWAY ROCK LINED CHUTE PLAN AND PROFILE
730	WEST SPILLWAY ROCK LINED CHUTE SECTIONS AND DETAILS
800	HERBERT CANAL - SIDE CHANNEL SPILLWAY PLAN AND SECTIONS
810	HERBERT CANAL ROAD REALIGNMENT PLAN, PROFILE AND SECTION
900	NORTH - SOUTH ACCESS ROAD PLAN AND PROFILE

REFERENCE
TOPOGRAPHIC MAP 072J/06 OBTAINED FROM CANMATRIX © 2007 HER MAJESTY THE
QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES. COORDINATE
SYSTEM: UTM NAD83 ZONE 13. KEY MAP IMAGE OF SASKATCHEWAN OBTAINED FROM YELLOWMAPS.COM. © 2001 HER
MAJESTY THE QUEEN IN RIGHT OF CANADA; DEPARTMENT OF NATURAL RESOURCES.

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PRE-DESIGN
NOT FOR
CONSTRUCTION

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LEGEND	
	EXISTING GROUND CONTOURS (0.5 m INTERVAL)
	EXISTING WATER BODY
	MINOR ROAD
	FENCE
	CROWN LAND
	POTENTIAL AND PROPOSED AREAS OF DISTURBANCE
	CONSTRUCTION LIMITS
	ALIGNMENT CENTERLINE
	FSL (EL. 723.0)
ABBREVIATIONS FSL = FULL SUPPLY LEVEL	

- NOTES
- DIMENSIONS ARE IN MILLIMETRES, ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.
 - USE OF THIS BORROW AREA SUBJECT TO RESULTS OF GEOTECHNICAL INVESTIGATION.

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON 22"x34" FORMAT DRAWINGS

- REFERENCE
- EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY McELHANNEY CONSULTING SERVICES LTD.

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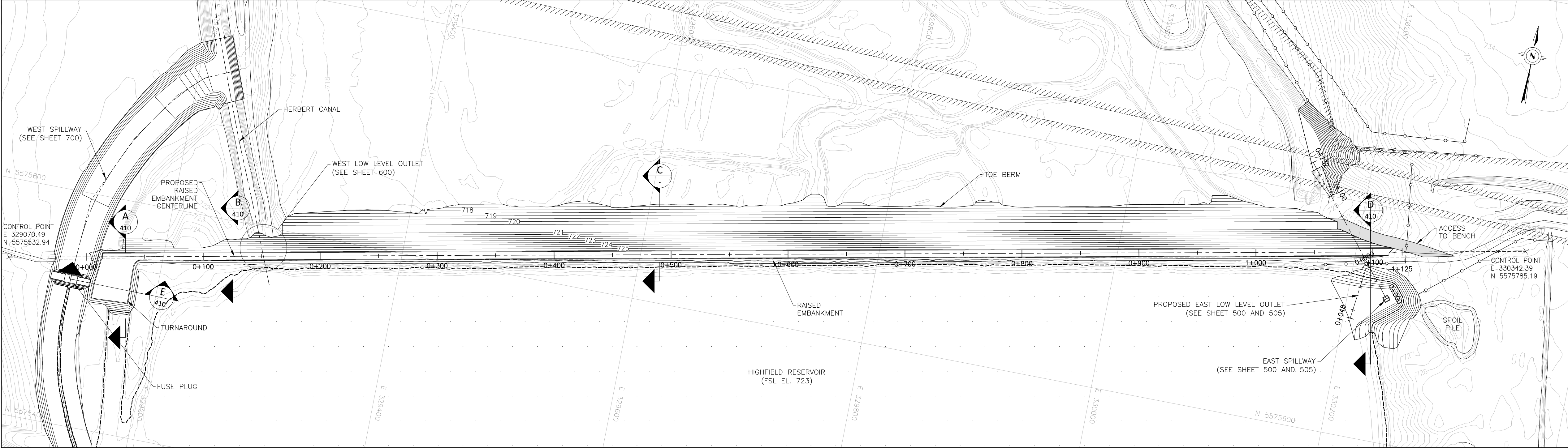


AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM

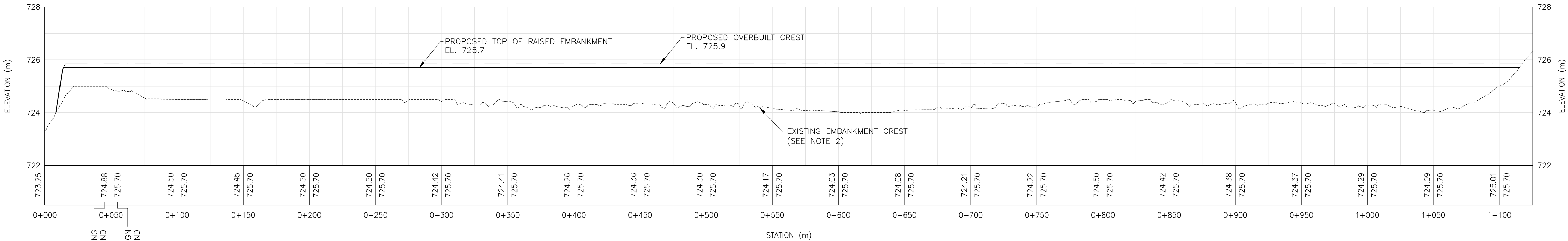
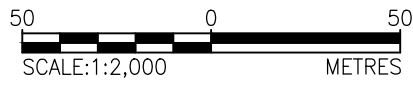
REHABILITATION PROJECT
PRE-DESIGN STUDY

TITLE
SITE PLAN

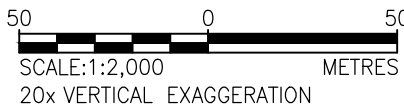
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PLAN - RAISED EMBANKMENT



PROFILE - RAISED EMBANKMENT



LEGEND

- EXISTING GROUND CONTOURS (0.5 m INTERVAL)
- EXISTING WATER BODY
- MINOR ROAD
- CROWN LAND
- FENCE LINE
- ALIGNMENT CENTERLINE
- FSL (EL. 723.0)

ABBREVIATIONS
FSL = FULL SUPPLY LEVEL

NOTES

- DIMENSIONS ARE IN MILLIMETRES, ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.
- EXISTING GROUND PROFILE REFLECTS EXISTING CONTOUR INFORMATION (0.5 m INTERVAL) PROVIDED BY THE CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE OF CURRENT FIELD CONDITIONS.
- OVERBUILT CREST NOT TO BE CONSIDERED AN OPERATION AND MAINTENANCE ISSUE.

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED. (i.e. 1:1000 etc) ARE BASED ON 22"x34" FORMAT DRAWINGS

REFERENCE

- EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY McELHANNEY CONSULTING SERVICES LTD.

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Golder Associates
Calgary, Alberta

Agriculture and Agri-Food Canada

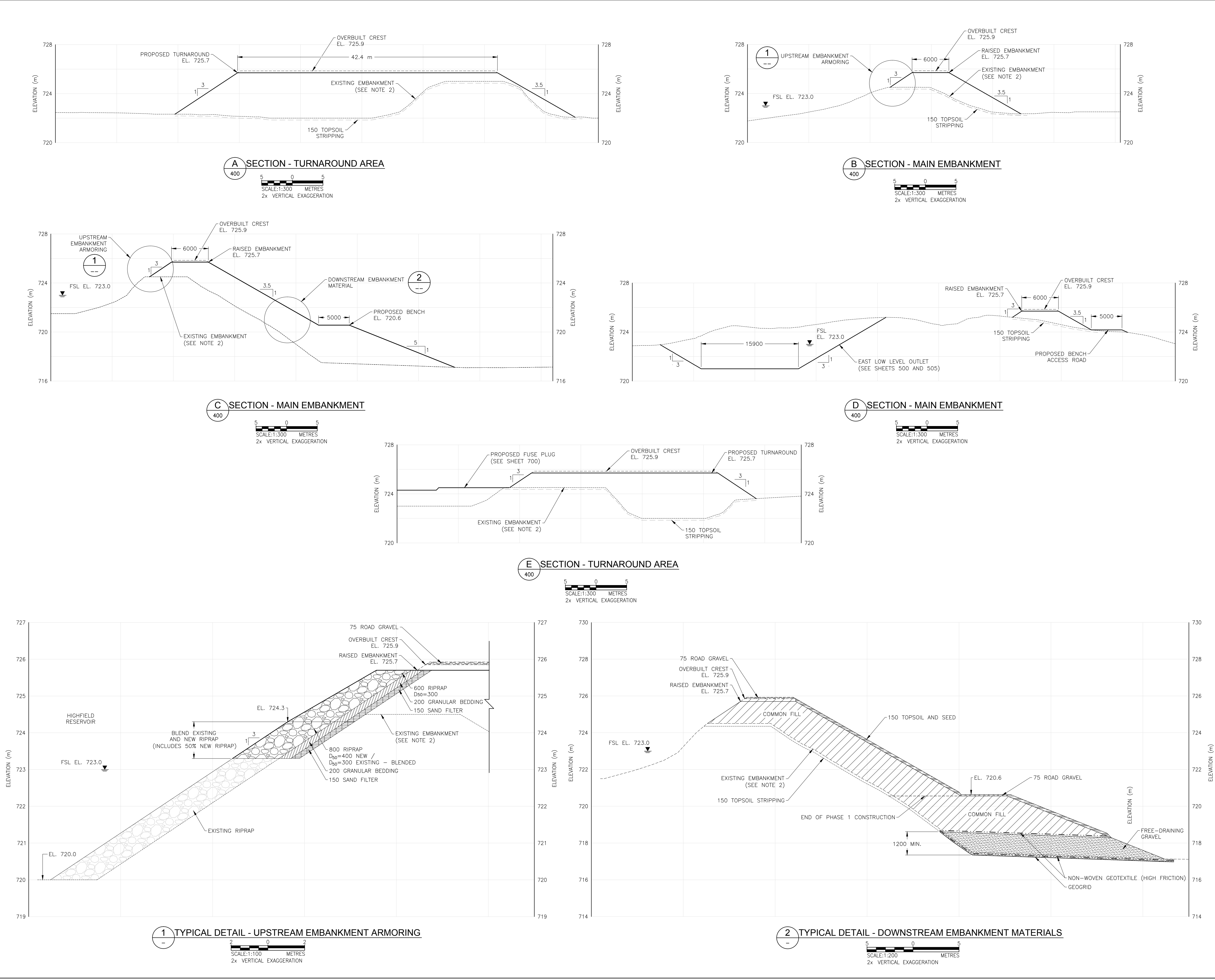
**AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM**

**REHABILITATION PROJECT
PRE-DESIGN STUDY**

TITLE

RAISED EMBANKMENT PLAN AND PROFILE

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LEGEND

ABBREVIATIONS
FSL = FULL SUPPLY LEVEL

NOTES
1. DIMENSIONS ARE IN MILLIMETRES. ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.
2. EXISTING GROUND PROFILE REFLECTS EXISTING CONTOUR INFORMATION (0.5m INTERVAL) PROVIDED BY CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE OF CURRENT FIELD CONDITIONS.

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON 22"x34" FORMAT DRAWINGS

REFERENCE
1. EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY McELHANNY CONSULTING SERVICES LTD.

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Golder Associates
Calgary, Alberta

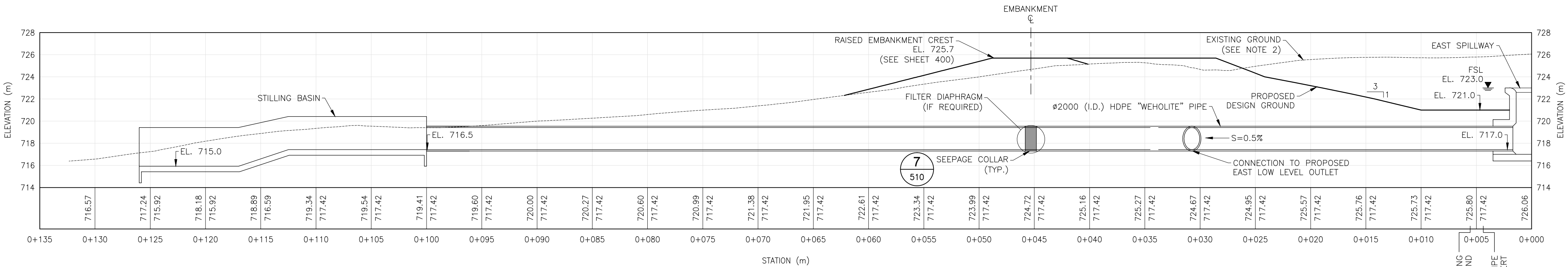
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AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM

REHABILITATION PROJECT
PRE-DESIGN STUDY

TITLE
RAISED EMBANKMENT
SECTIONS AND DETAILS

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PROFILE - EAST LOW LEVEL OUTLET AND SPILLWAY CONDUIT



- | NOTES | |
|-------|---|
| 1. | DIMENSIONS ARE IN MILLIMETRES, ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED. |
| 2. | EXISTING GROUND PROFILE REFLECTS EXISTING CONTOUR INFORMATION (0.5 m INTERVAL) PROVIDED BY THE CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE OF CURRENT FIELD CONDITIONS. |
| 3. | BATHYMETRIC INFORMATION NOT PROVIDED. GROUND BELOW WATER LEVEL INFERRED. |

REFERENCE	
1.	EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY McELHANNEY CONSULTING SERVICES LTD.

NAD 83 UTM ZONE 13

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	DATE								

ENGINEER'S SEAL	PERMIT STAMP



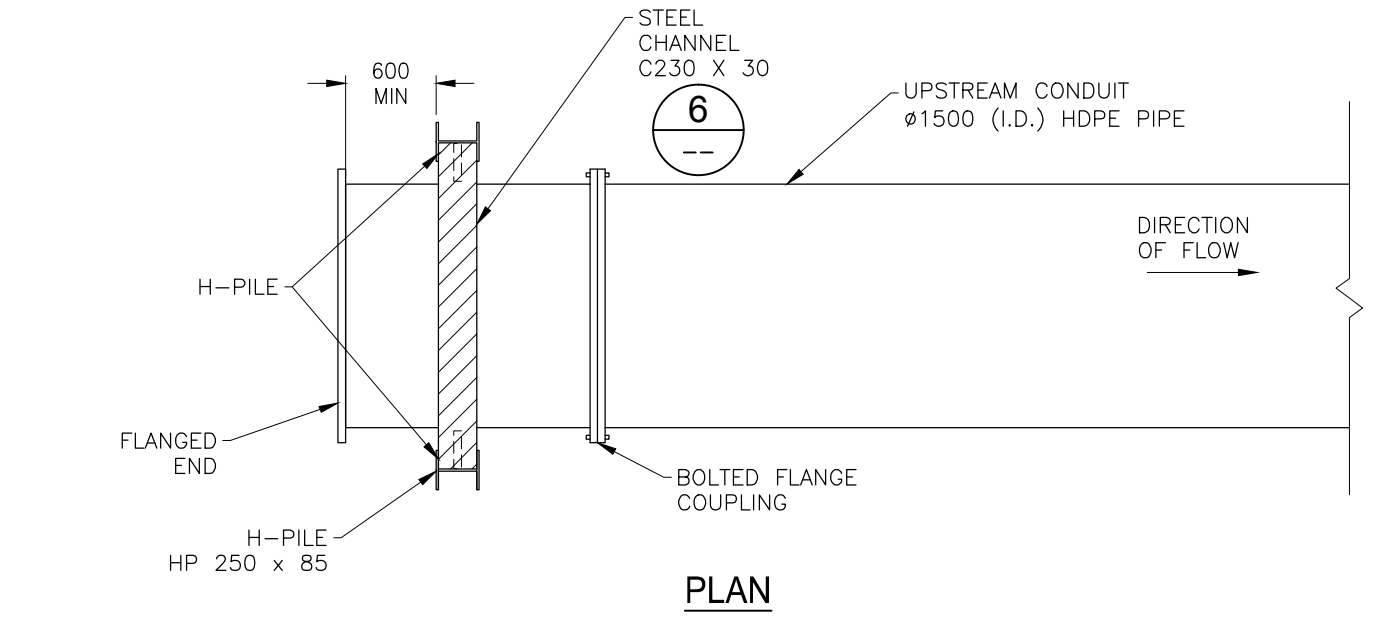
AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM

REHABILITATION PROJECT PRE-DESIGN STUDY

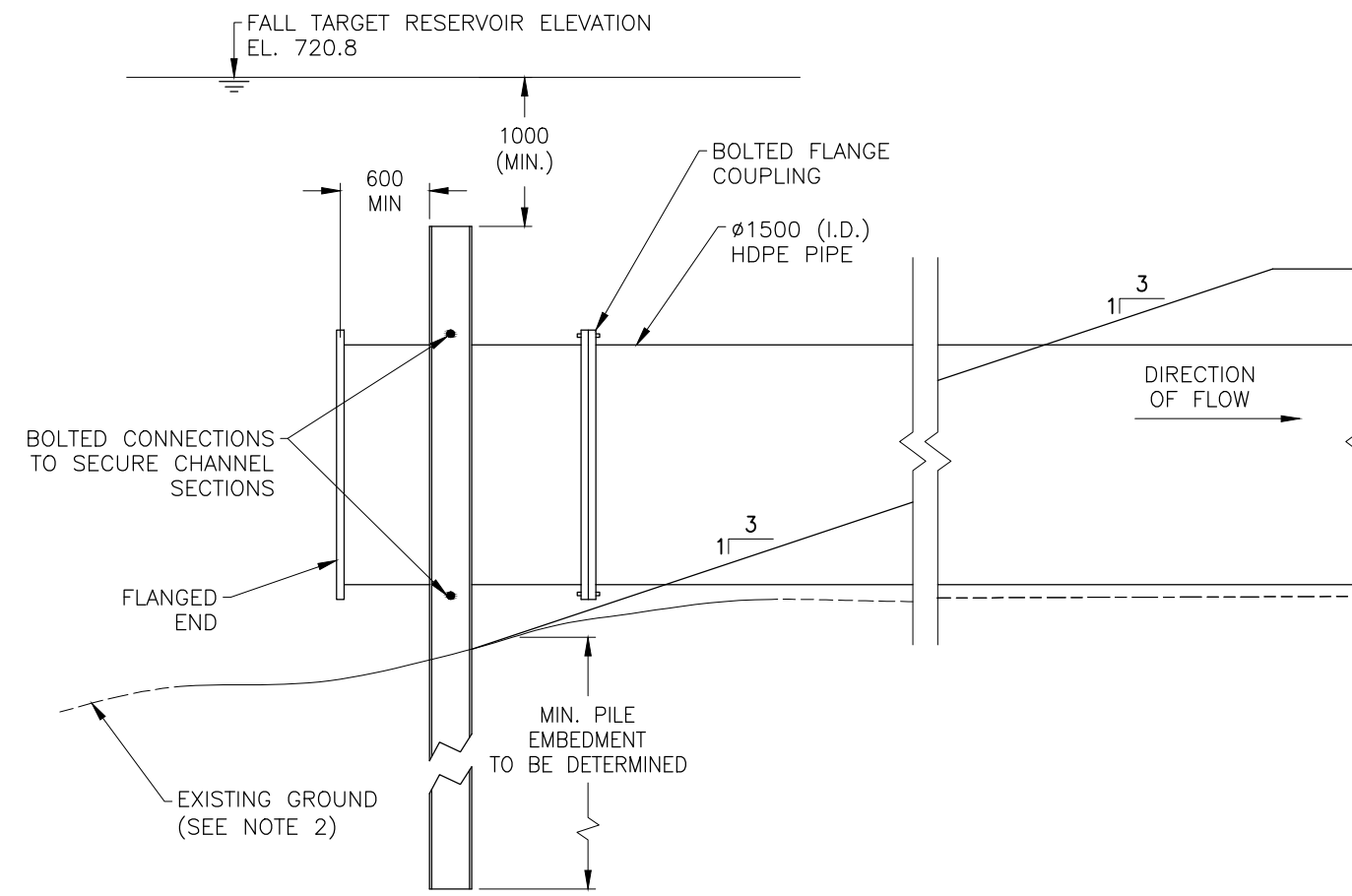
TITLE

EAST LOW LEVEL OUTLET AND
SPILLWAY CONDUIT PLAN AND PROFILE

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SHEET NO. 505	DRAWING NO. 1213450020DW015		REV. 0

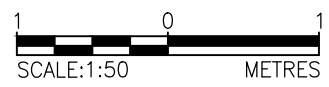
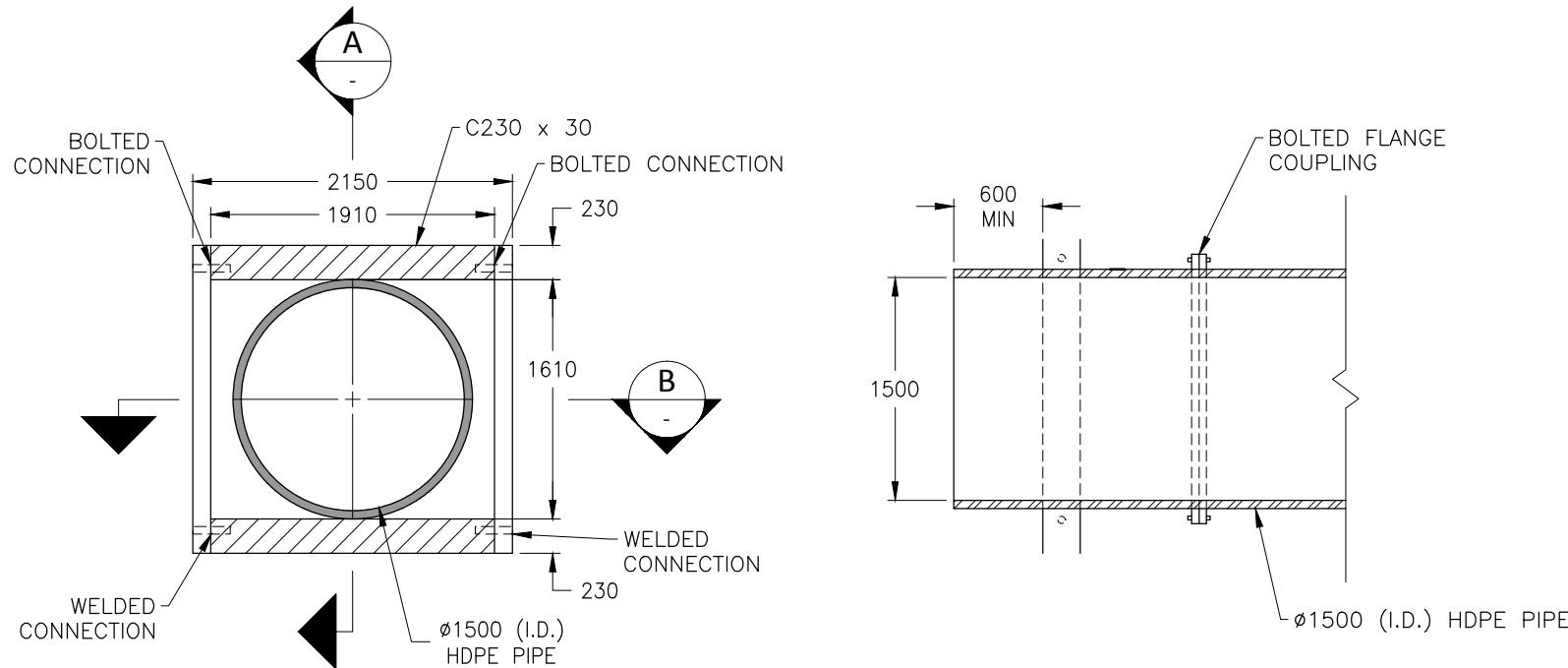


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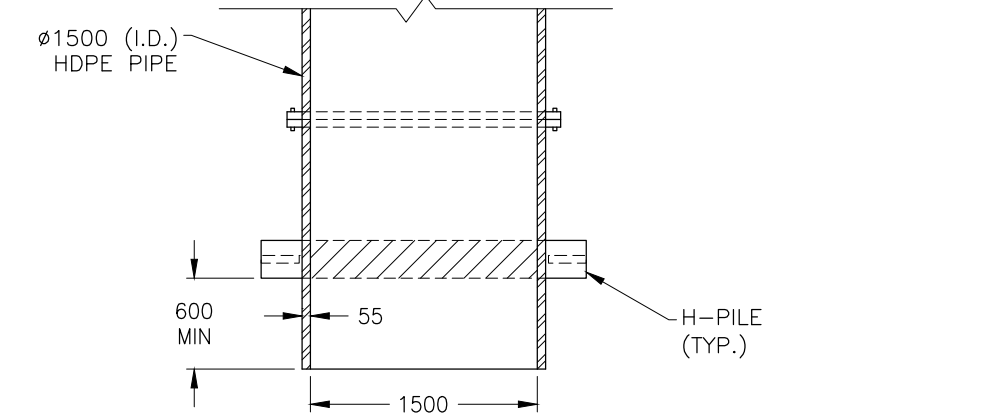


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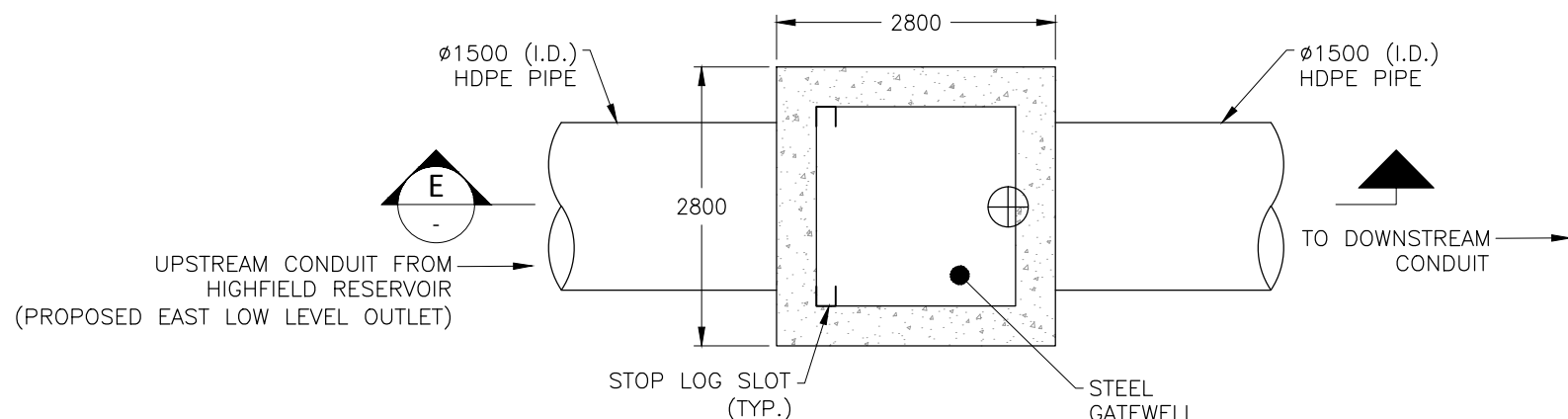
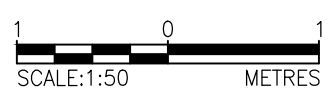
1 DETAIL - PROPOSED EAST LOW LEVEL OUTLET



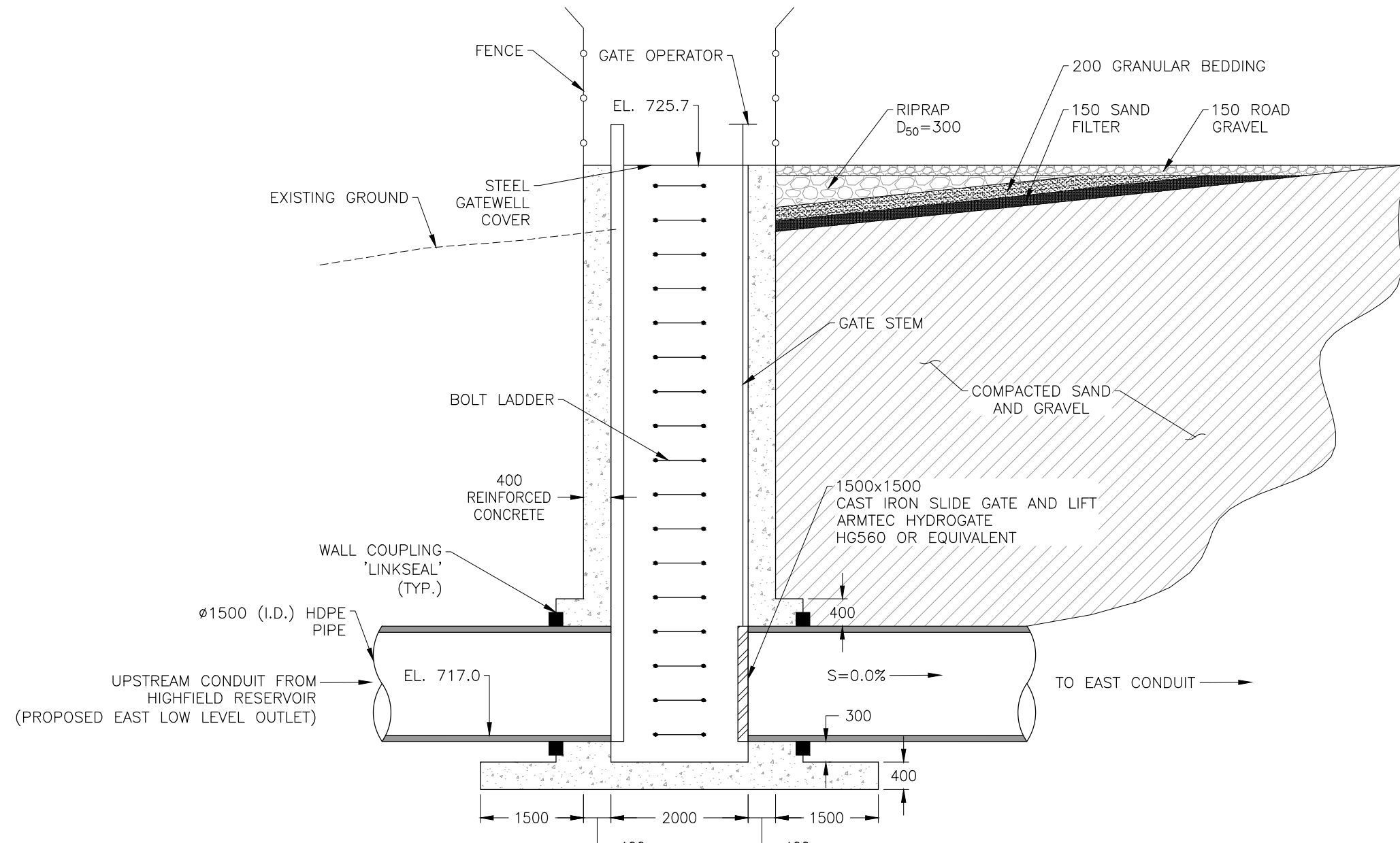
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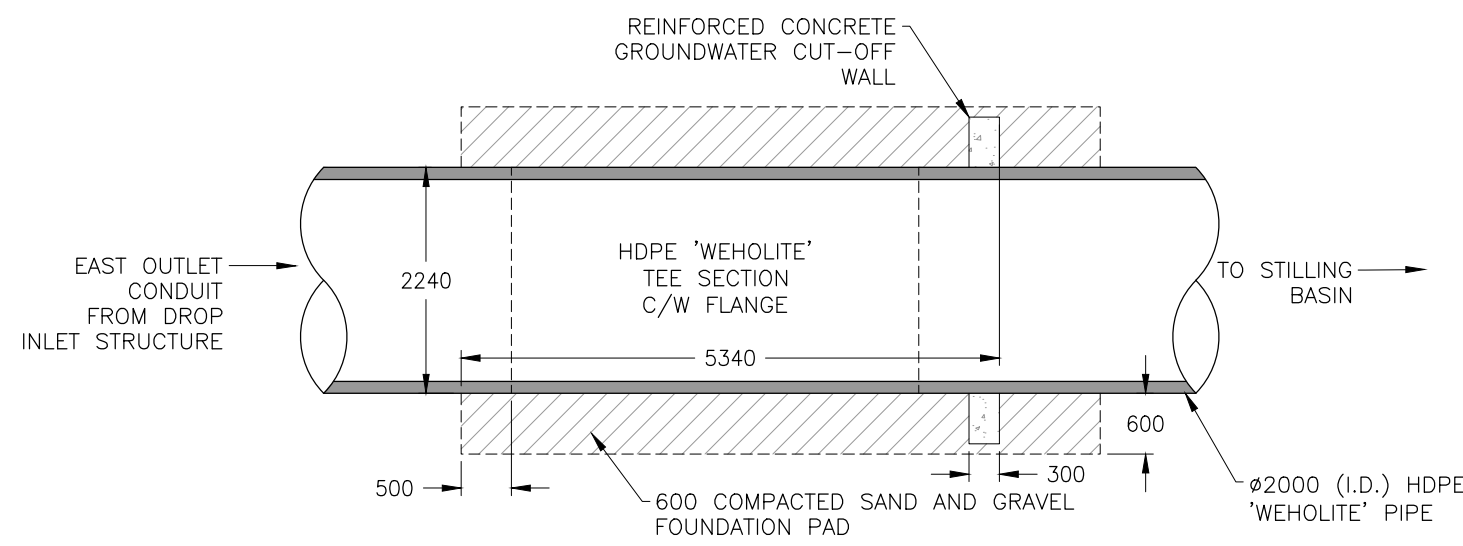
B SECTION - PROPOSED EAST LOW LEVEL OUTLET



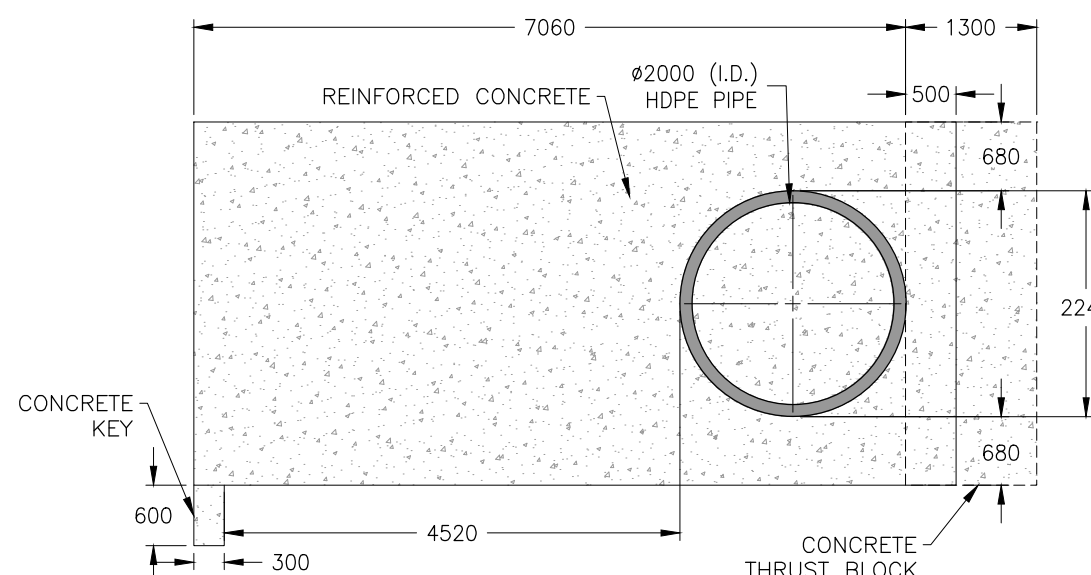
7 DETAIL - CONCRETE GATEWELL



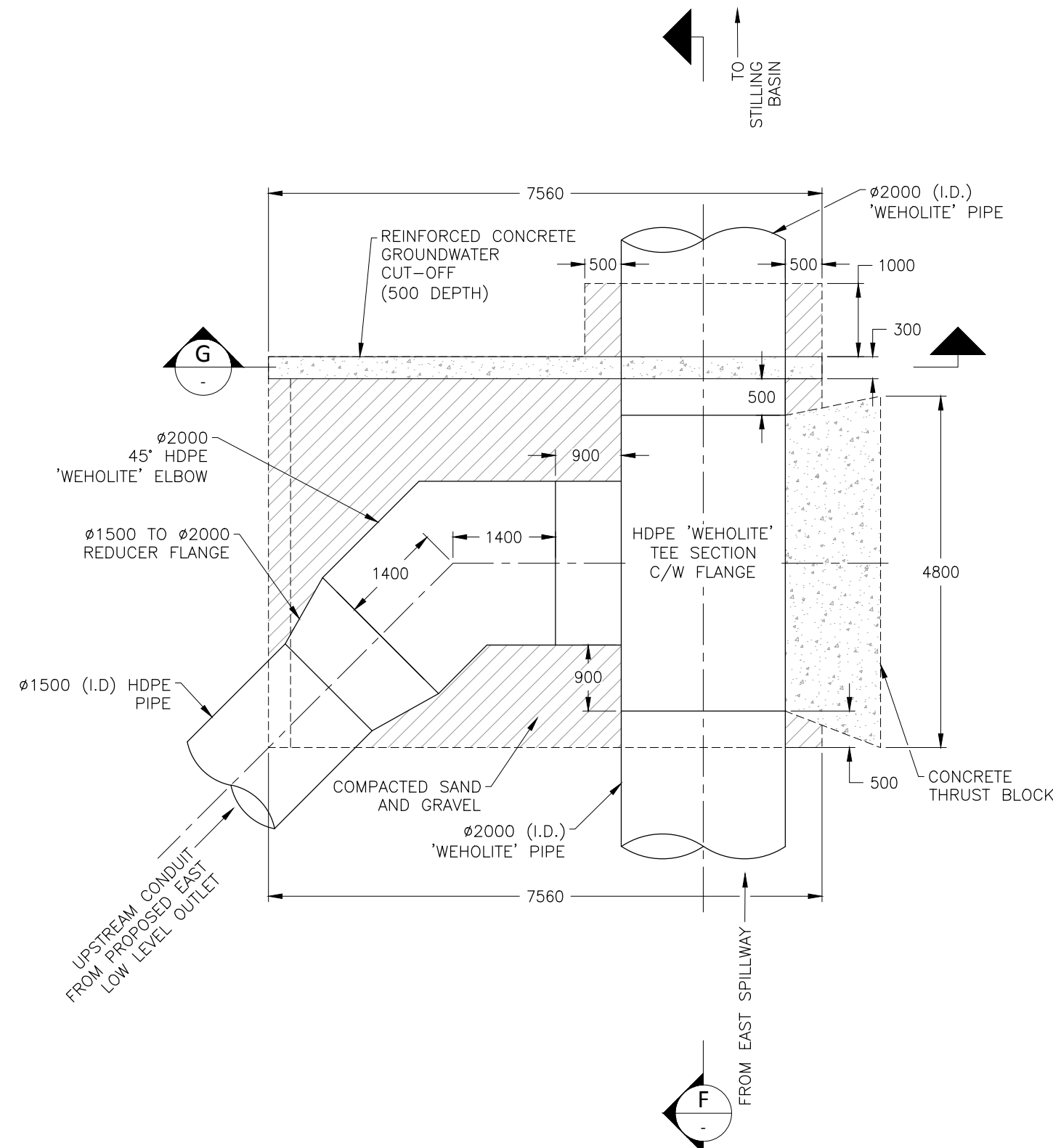
E SECTION - CONCRETE GATEWELL



F SECTION - GROUNDWATER CUT-OFF WALL



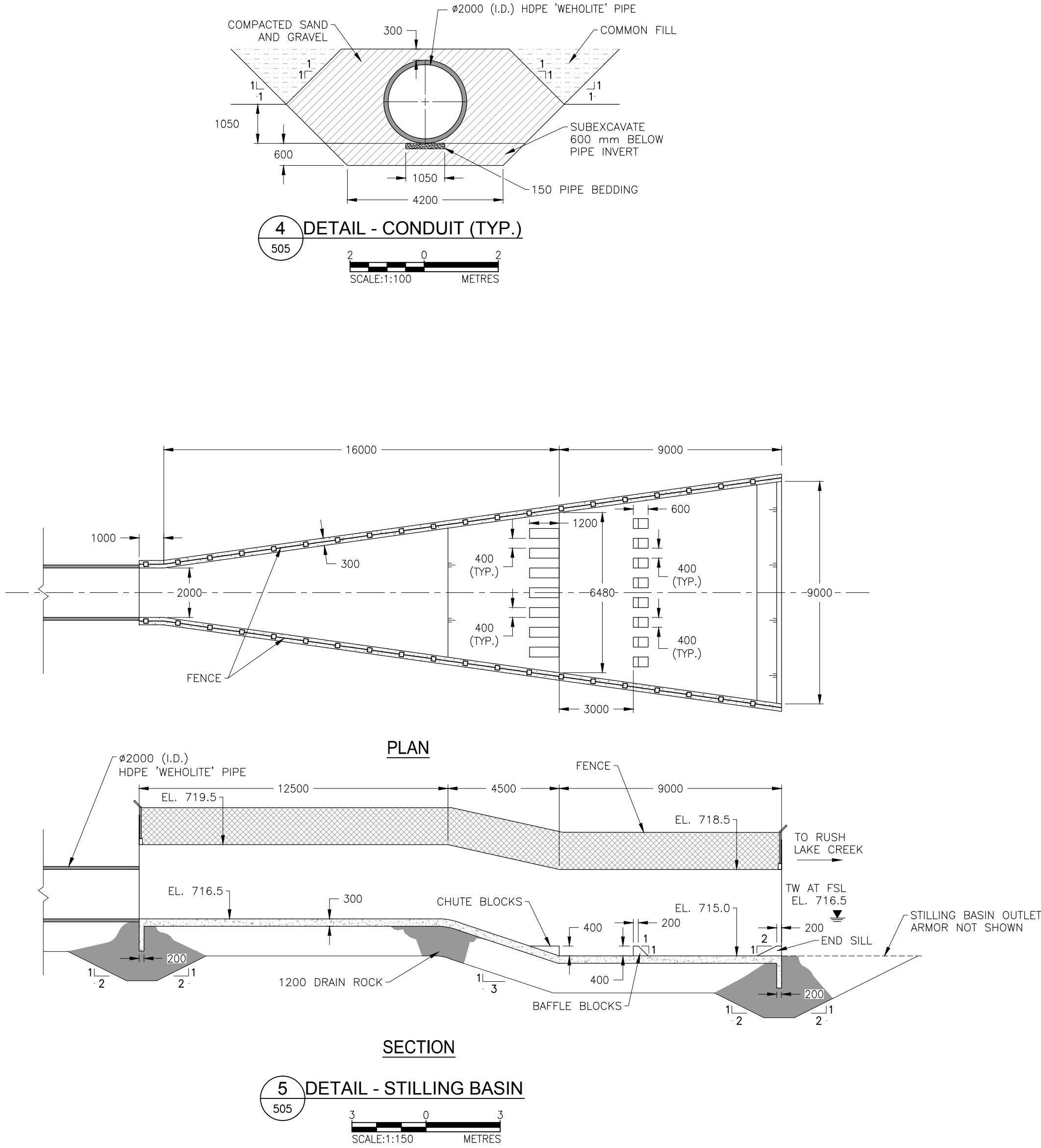
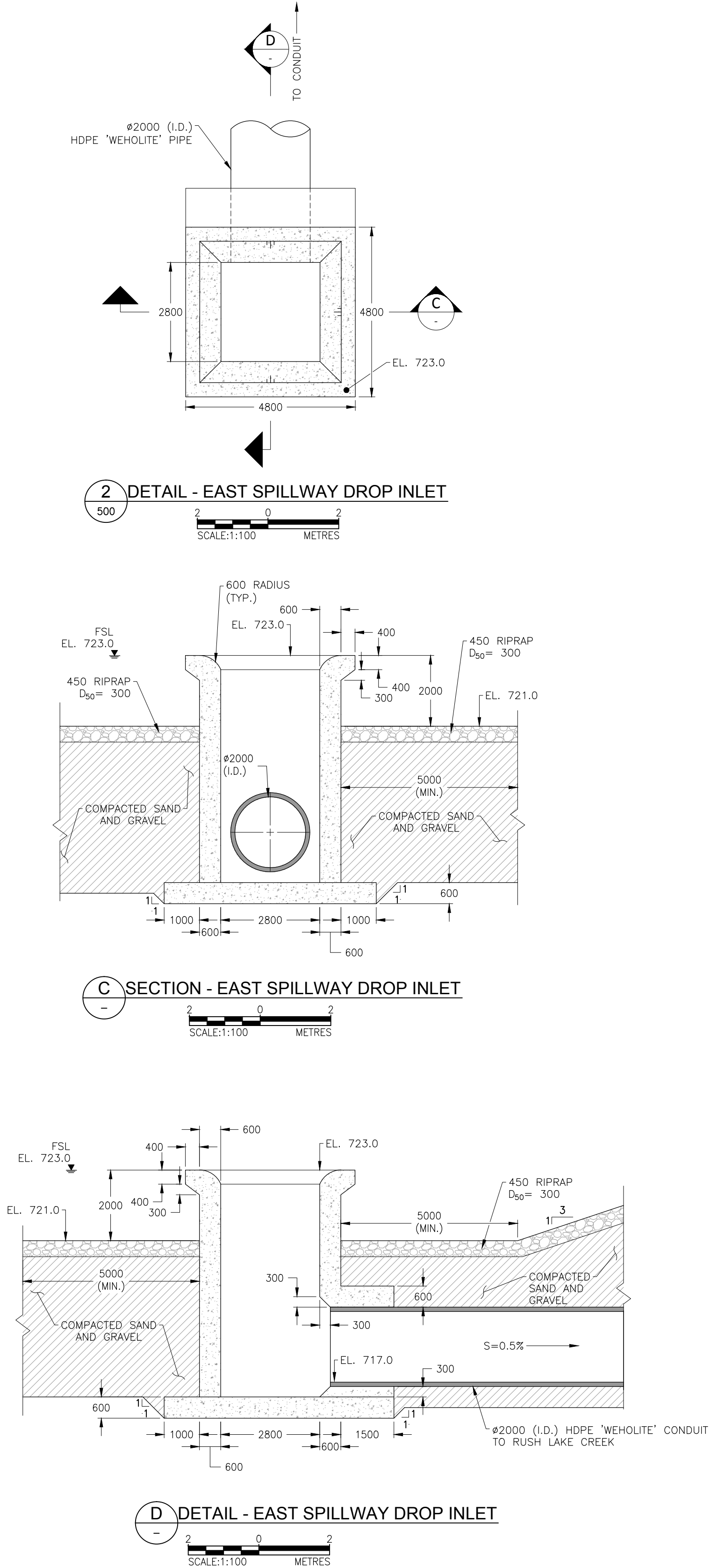
G SECTION - GROUNDWATER CUT-OFF WALL



8 DETAIL - JUNCTION BLOCK



LEGEND									
ABBREVIATIONS FSL = FULL SUPPLY LEVEL TW = TAIL WATER									
NOTES									
1. DIMENSIONS ARE IN MILLIMETRES. ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.									
2. EXISTING GROUND PROFILE REFLECTS EXISTING CONTOURS (0.5) INTERVAL PROVIDED BY CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE BY CURRENT FIELD CONDITIONS.									
3. EXTEND ARMOR/BALLAST TO RIPARIAN FLOW INLET.									
THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON 22"X34" FORMAT DRAWINGS									
REFERENCE									
NAD 83 UTM ZONE 13									
0	13	03	28	ISSUED FOR PRE-DESIGN		AS	DW	CA	AKS
REV	yy	mm	dd	DESCRIPTION		BY	CHK	ENG	APP
			DATE						
ENGINEER'S SEAL					PERMIT STAMP				
ISSUED FOR PRE-DESIGN NOT FOR CONSTRUCTION									
 Golder Associates Calgary, Alberta					 Agriculture and Agri-Food Canada				
AGRICULTURE AND AGRI-FOOD CANADA HIGHFIELD DAM									
REHABILITATION PROJECT PRE-DESIGN STUDY									
TITLE									
EAST LOW LEVEL OUTLET AND SPILLWAY SECTIONS AND DETAILS (1 OF 2)									
DRAWN BY AS		CHECKED BY DW		ENGINEERED CA		APPROVED AKS			
SHEET NO. 510		DRAWING NO.		1213450020DW011				REV. 0	



LEGEND

ABBREVIATIONS

FSL = FULL SUPPLY LEVEL

TW = TAIL WATER

NOTES

1. DIMENSIONS ARE IN MILLIMETRES, ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON 22"X34" FORMAT DRAWINGS

REFERENCE


NAD 83 UTM ZONE 13

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	DATE								
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Agriculture and Agri-Food Canada

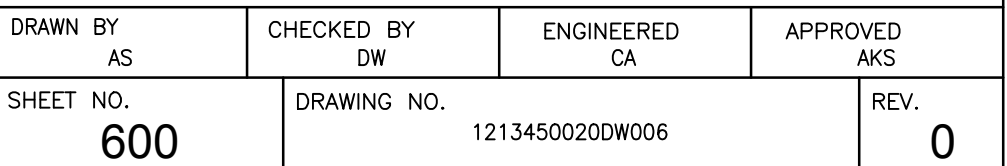
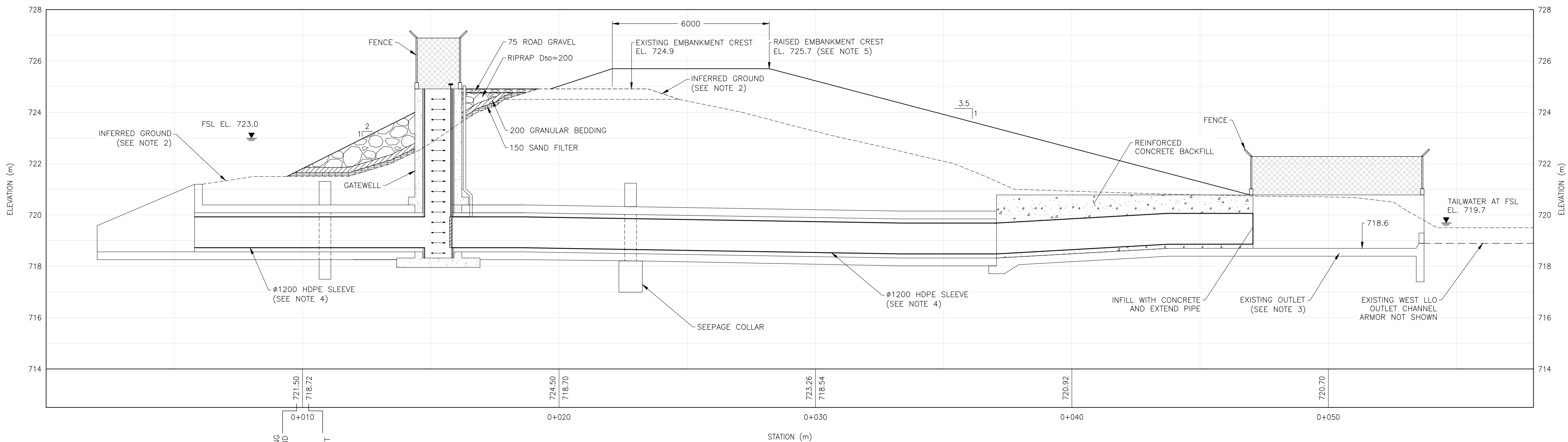
AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM

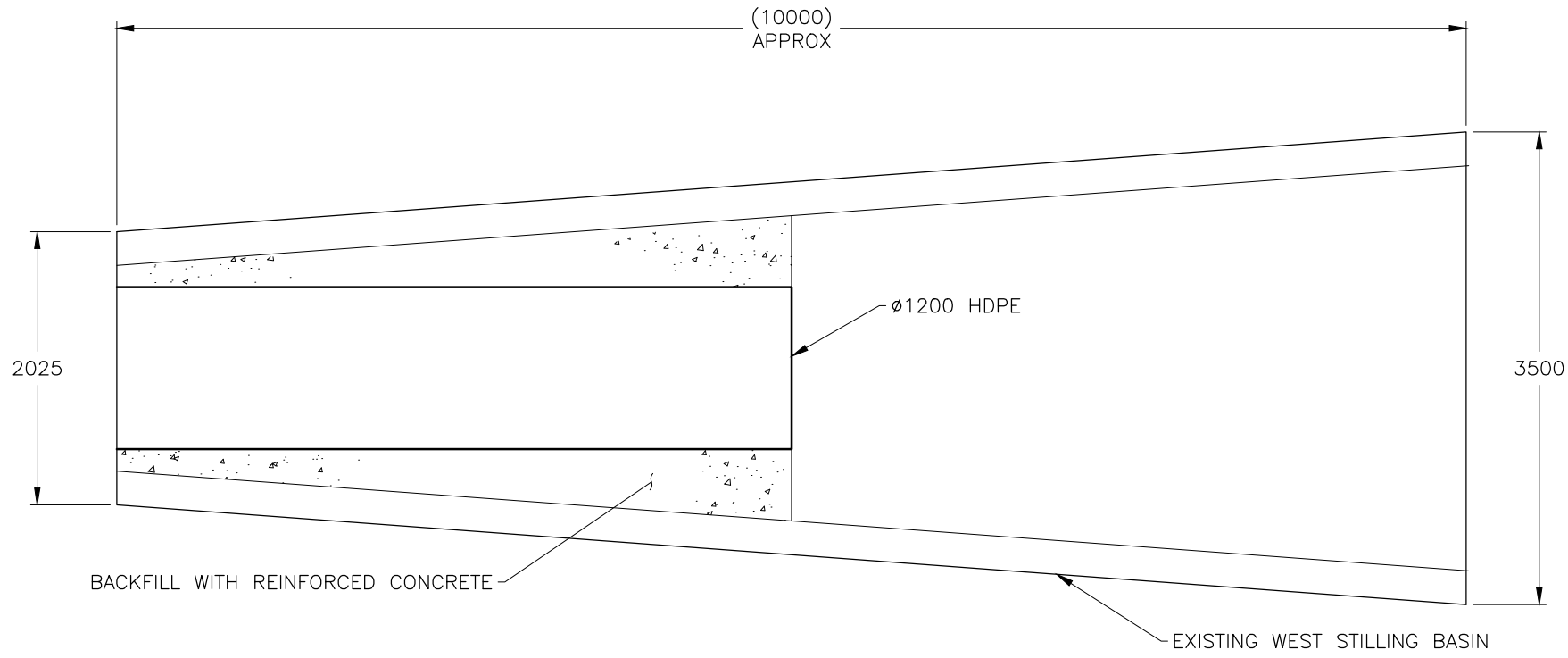
REHABILITATION PROJECT
PRE-DESIGN STUDY

TITLE

EAST LOW LEVEL OUTLET AND SPILLWAY
SECTIONS AND DETAILS (2 OF 2)

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SHEET NO. 515	DRAWING NO. 1213450020DW018		REV. 0





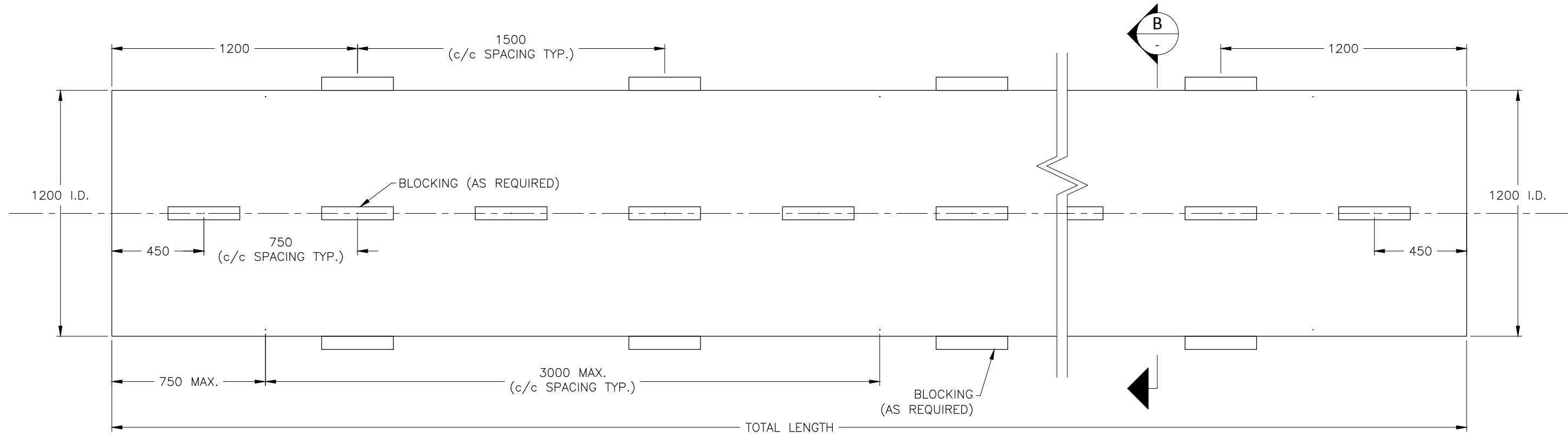
3 DETAIL - OUTLET

600

SCALE:1:50

0

METRES



2 DETAIL - WEST LOW LEVEL OUTLET

600

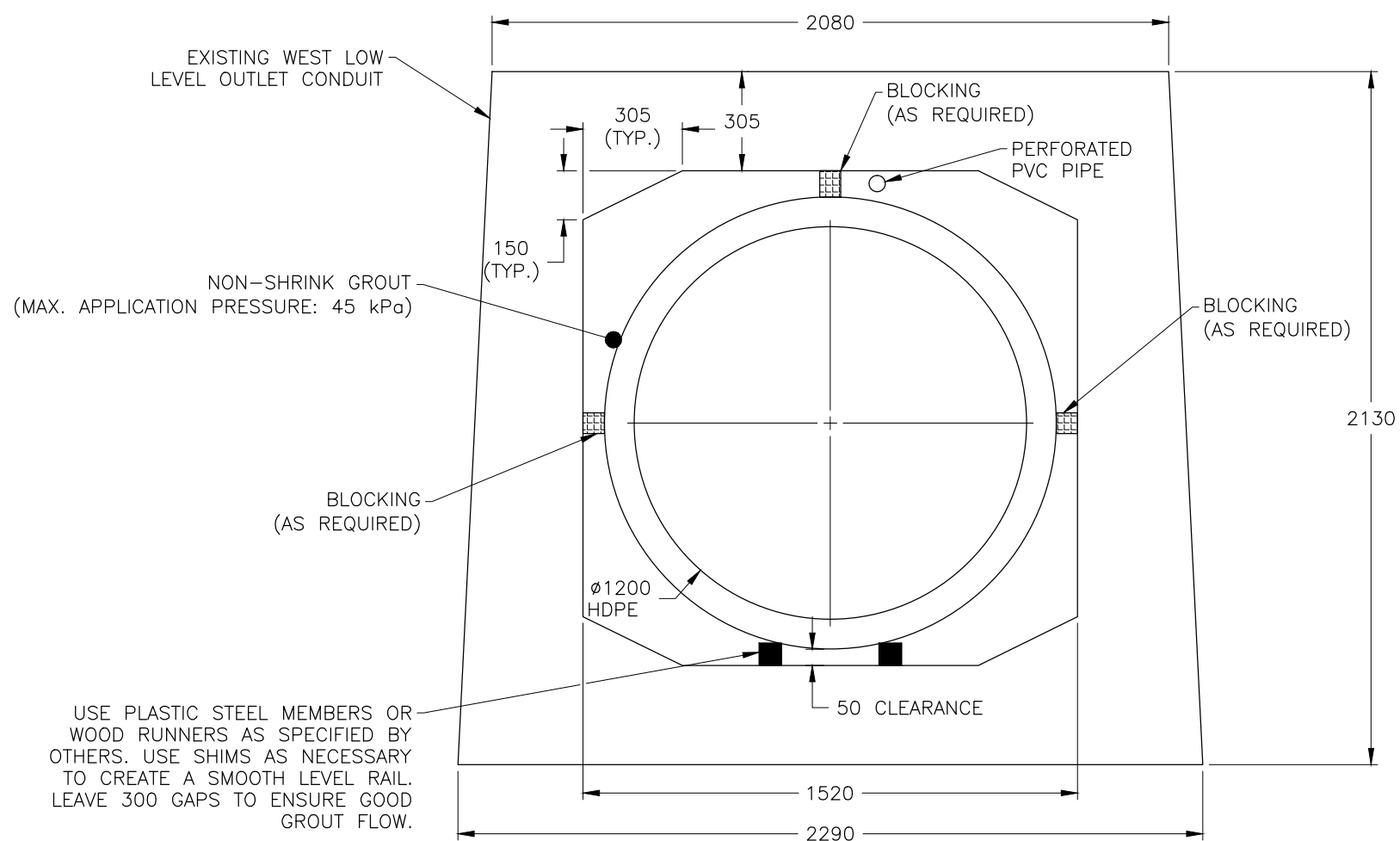
SCALE:1:20

0.5

0

0.5

METRES



B SECTION - WEST LOW LEVEL OUTLET

600

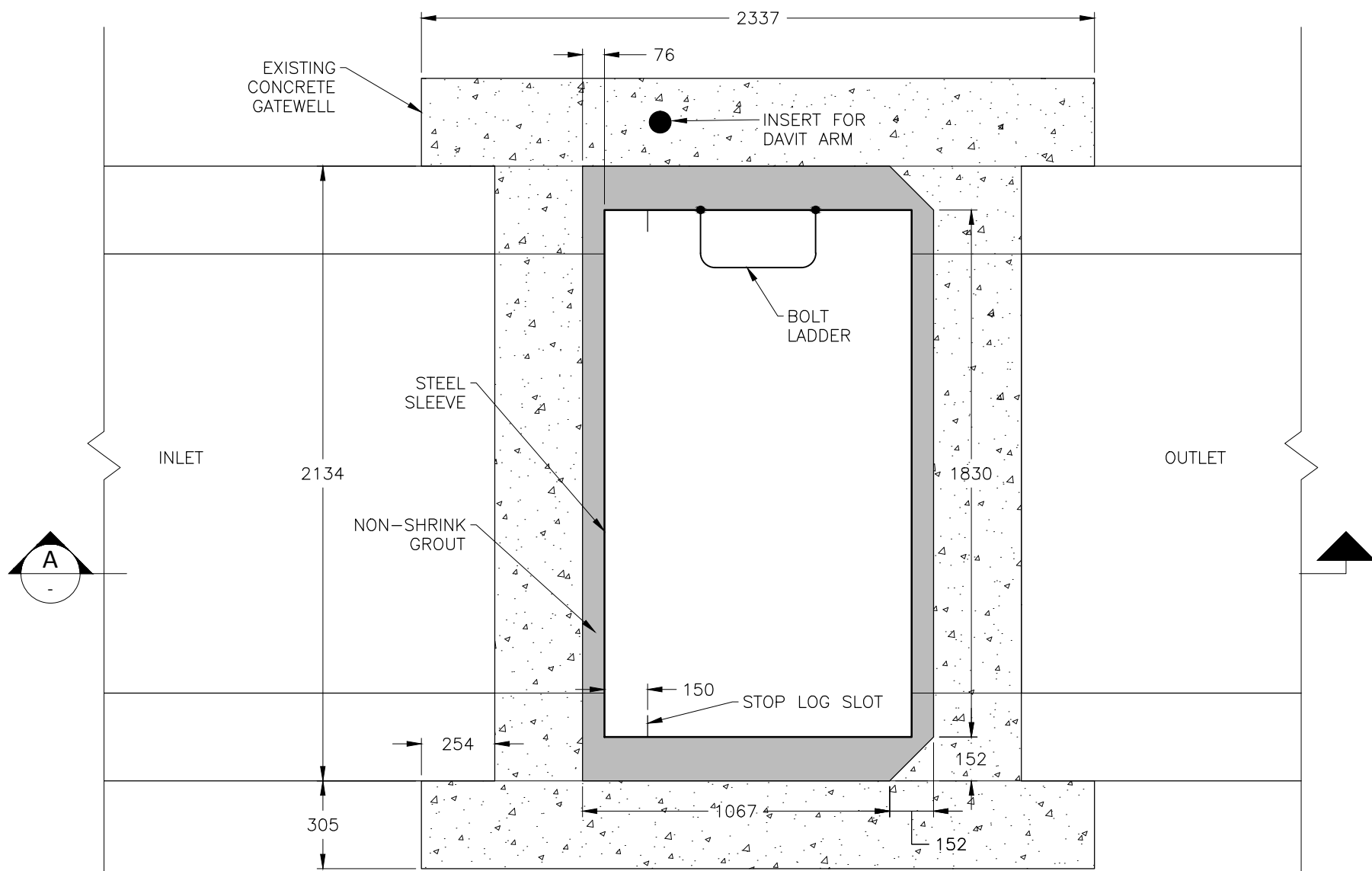
SCALE:1:20

0.5

0

0.5

METRES



1 DETAIL - WEST LOW LEVEL OUTLET

600

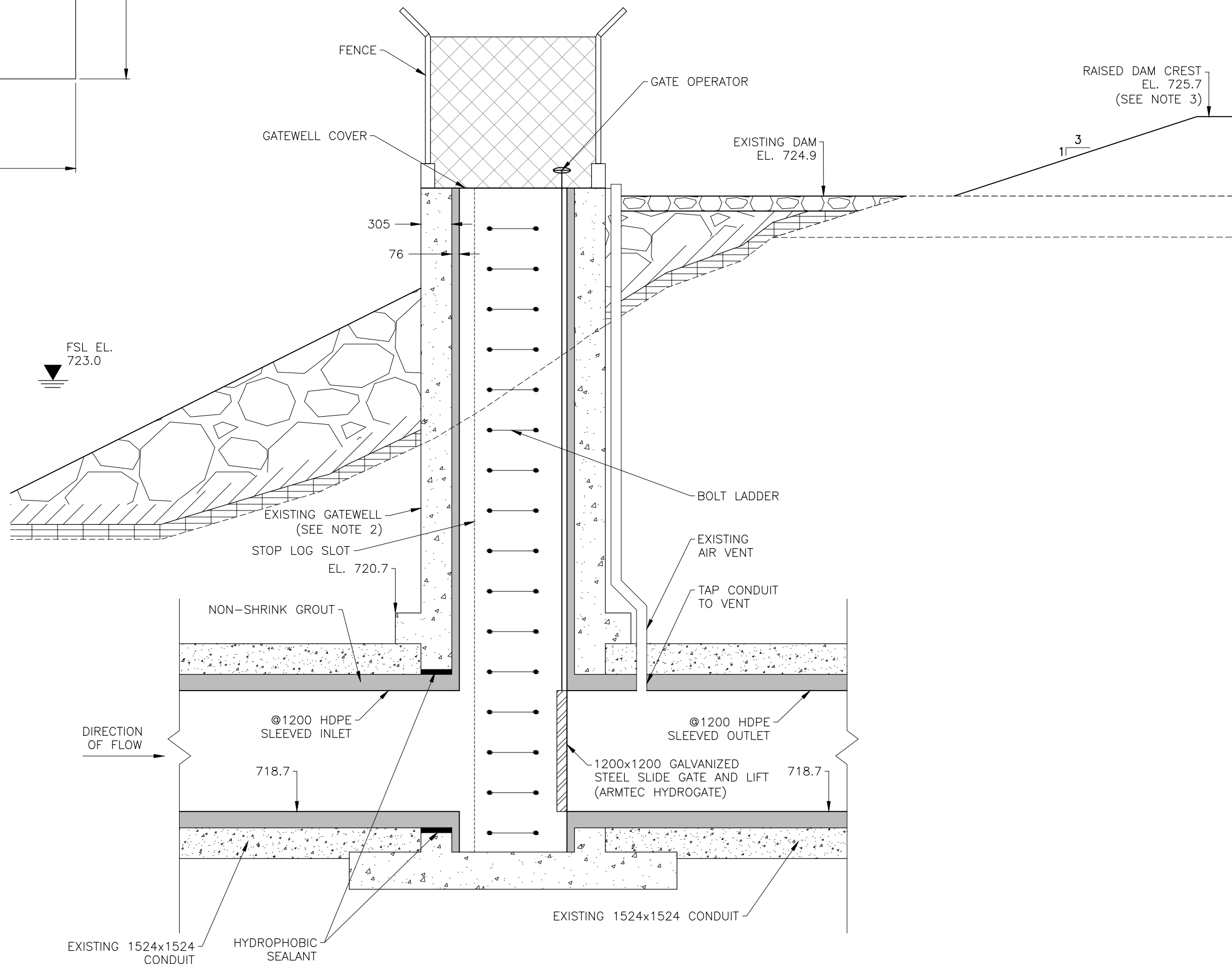
SCALE:1:20

0.5

0

0.5

METRES



A SECTION - GATE WELL

600

SCALE:1:40

1

0

1

METRES

LEGEND

ABBREVIATIONS
FSL = FULL SUPPLY LEVEL
CSP = CORRUGATED STEEL PIPE

NOTES

1. DIMENSIONS ARE IN MILLIMETRES. ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.
2. EXISTING STRUCTURE DIMENSIONS ARE APPROXIMATE, SEE REFERENCE 1.
3. OVERBUILT CREST LINE AT EL. 725.8 NOT SHOWN.
4. MAXIMUM ALLOWABLE GROUT PRESSURE: 45 kPa.
5. IF NECESSARY SECURE LINER BY BLOCKING OR OTHER SUITABLE MEASURE TO PREVENT PIPE FROM FLOATING DURING GROUT APPLICATION.
6. GROUT ANNULAR VOID BY PUMPING OR GRAVITY METHOD WITH CEMENTITIOUS GROUT

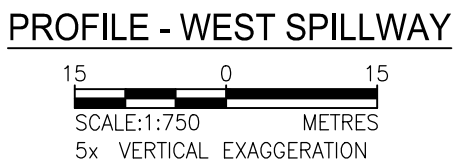
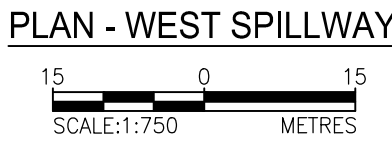
THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON 22"x34" FORMAT DRAWINGS

REFERENCE

1. EXISTING STRUCTURES DIGITIZED FROM SCANNED DEPARTMENT OF AGRICULTURE CANADA, WATER DEVELOPMENT BRANCH, SWIFT CURRENT IRRIGATION PROJECT, HIGHFIELD DAM AS-BUILT DRAWINGS. DRAWING NUMBERS: C2753, 117912. DIMENSIONS ARE APPROXIMATE.
2. EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY McELHANNEY CONSULTING SERVICES LTD.

NAD 83 UTM ZONE 13

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LEGEND

EXISTING GROUND CONTOURS (0.5 m INTERVAL)

ALIGNMENT CENTERLINE

FSL (EL. 723.0)

DIRECTION OF FLOW

ABBREVIATIONS

FSL = FULL SUPPLY LEVEL

NOTES

1. DIMENSIONS ARE IN MILLIMETRES. ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.

2. EXISTING GROUND PROFILE REFLECTS EXISTING CONTOUR INFORMATION (0.5m INTERVAL) PROVIDED BY CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE OF CURRENT FIELD CONDITIONS.

3. EXISTING STRUCTURE DIMENSIONS ARE APPROXIMATE, SEE REFERENCE 2.

THIS DRAWING MAY HAVE BEEN REDUCED. ALL SCALE NOTATIONS INDICATED (i.e. 1:1000 etc) ARE BASED ON 22"X34" FORMAT DRAWINGS

REFERENCE

1. EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY MCELHANNAY CONSULTING SERVICES LTD.

2. EXISTING STRUCTURES DIGITIZED FROM SCANNED DEPARTMENT OF AGRICULTURE CANADA, WATER DEVELOPMENT BRANCH, SWIFT CURRENT IRRIGATION PROJECT, HIGHFIELD DAM AS-BUILT DRAWINGS. DRAWING NUMBERS: C2753, 117812. DIMENSIONS ARE APPROXIMATE.

NAD 83 UTM ZONE 13

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REV	yy	mm	dd	DESCRIPTION			BY	CHK	ENG	APP
				DATE						
ENGINEER'S SEAL							PERMIT STAMP			

ISSUED FOR

PRE-DESIGN

NOT FOR

CONSTRUCTION

Golder Associates

Calgary, Alberta

Canada

Agriculture and Agri-Food Canada

AGRICULTURE AND AGRI-FOOD CANADA

HIGHFIELD DAM

REHABILITATION PROJECT

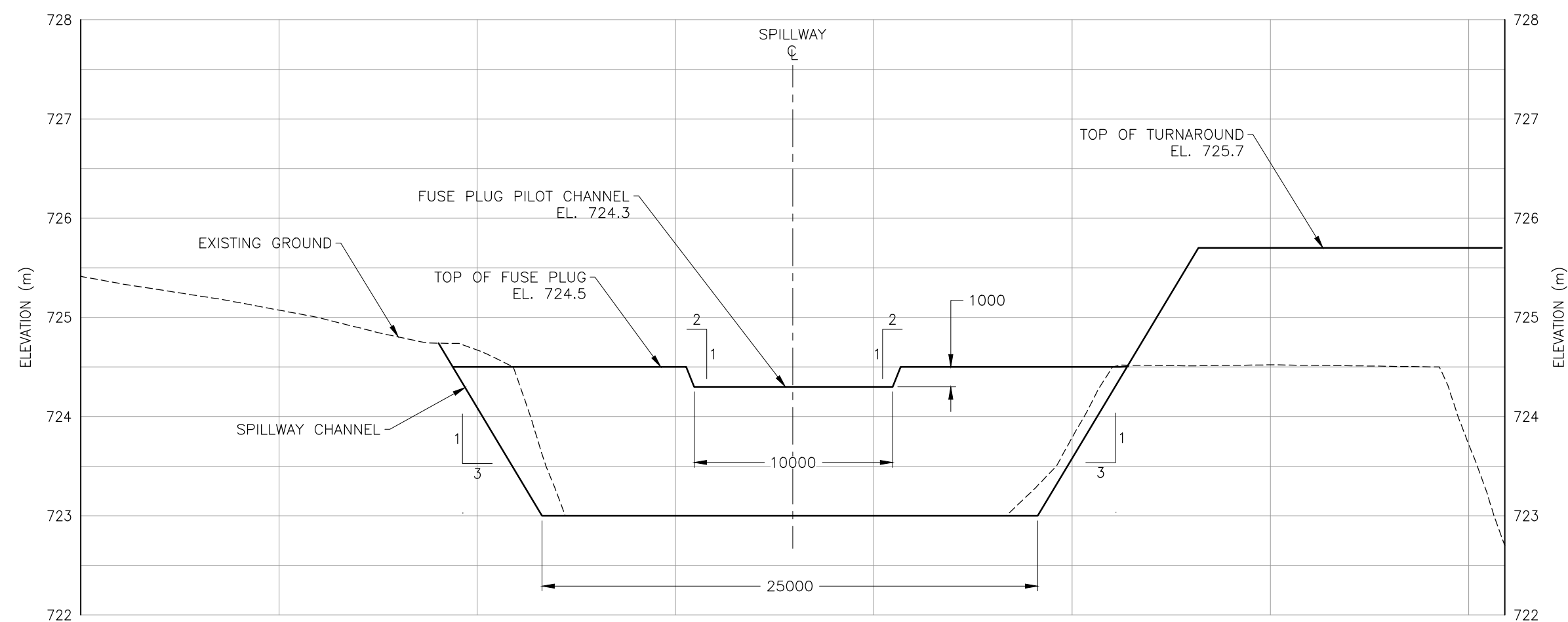
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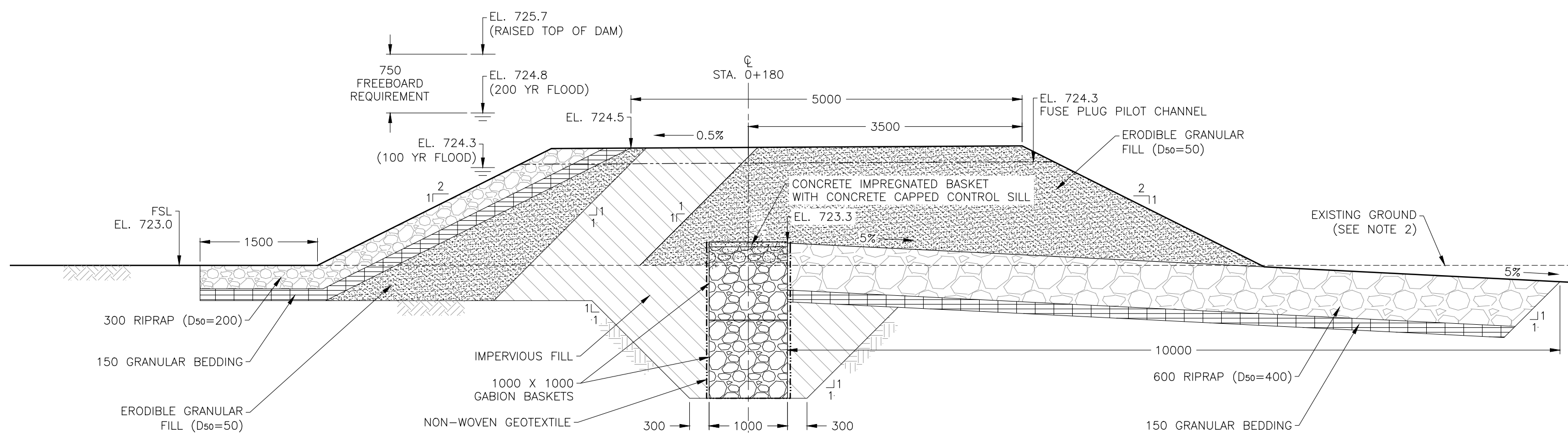
WEST SPILLWAY

PLAN AND PROFILE

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AS	DW	CA	AKS
SHEET NO.	DRAWING NO.	REV.	
700	1213450020DW007	0	



B SECTION - WEST SPILLWAY



D SECTION - FUSE PLUG

ABBREVIATIONS
FSL = FULL SUPPLY LEVEL

1. DIMENSIONS ARE IN MILLIMETRES. ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.

1. EXISTING GROUND PROFILE REFLECTS EXISTING CONTOUR INFORMATION (0.5m INTERVAL) PROVIDED BY CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE OF CURRENT FIELD CONDITIONS.

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1. EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY McELHANNEY CONSULTING SERVICES LTD.

NAD 83 UTM ZONE 13

[illegible]

ENGINEER'S SEAL

PERMIT STAMP

ISSUED FOR
PRE-DESIGN
NOT FOR
CONSTRUCTION



AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM

REHABILITATION PROJECT
PRE-DESIGN STUDY

TITLE

WEST SPILLWAY SECTIONS

DRAWN BY
AS

CHECKED BY
DW

ENGINEER
CA

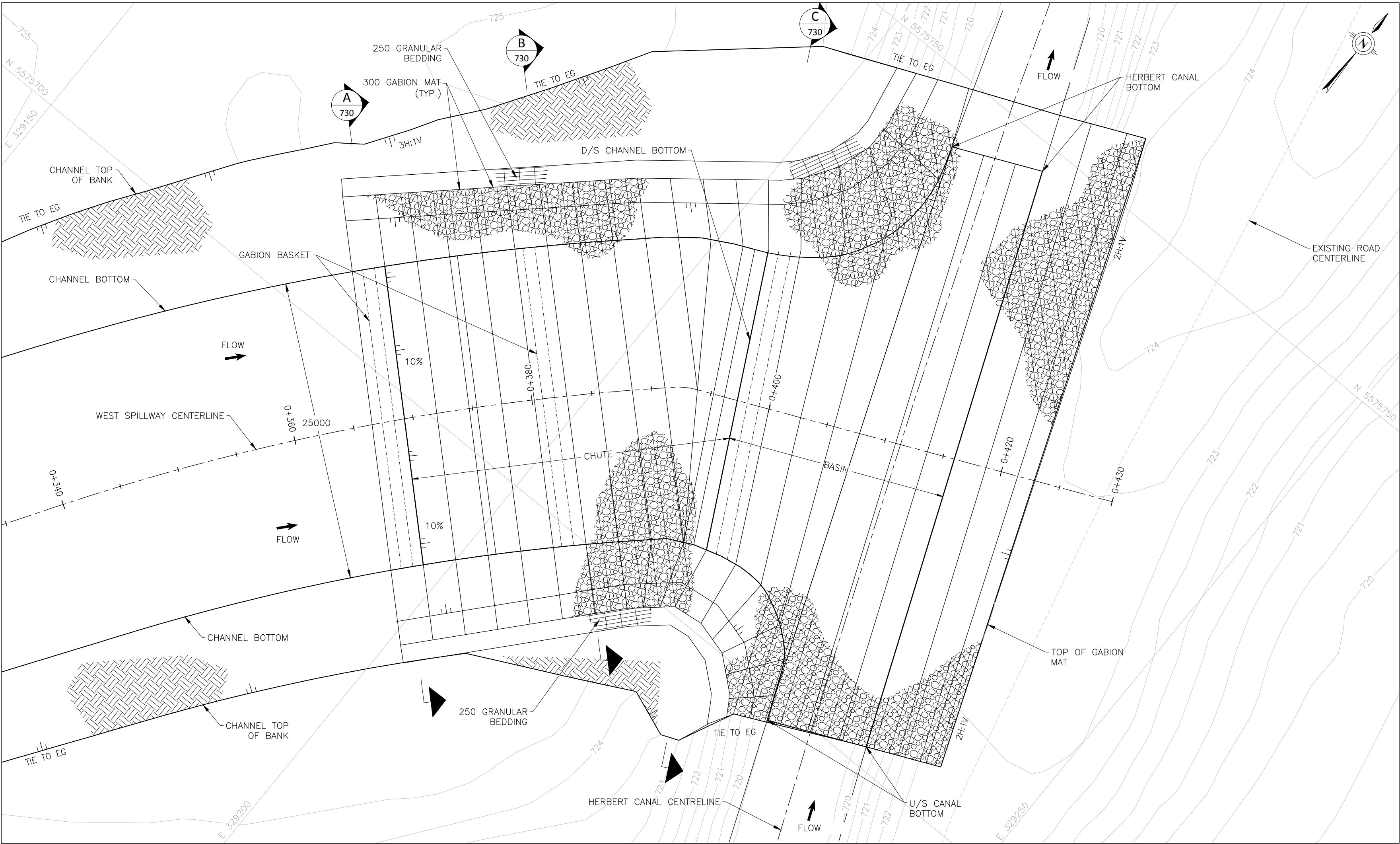
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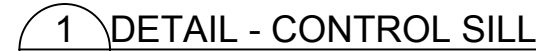
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REV. 0





DRAWN BY AS	CHECKED BY DW	ENGINEERED CA	APPROVED AKS
SHEET NO. 730	DRAWING NO. 1213450020DW013		REV. 0



 EXISTING GROUND CONTOURS (0.5 m INTERVAL)
 MINOR ROAD
 ALIGNMENT CENTERLINE
 DIRECTION OF FLOW
 CROWN LAND

1. DIMENSIONS ARE IN MILLIMETRES. ELEVATIONS, STATIONS AND COORDINATES ARE IN METRES, UNLESS OTHERWISE NOTED.
2. EXISTING GROUND PROFILE REFLECTS EXISTING CONTOUR INFORMATION (0.5m INTERVAL) PROVIDED BY CLIENT. EXISTING GROUND PROFILE MAY NOT BE REPRESENTATIVE OF CURRENT FIELD CONDITIONS.

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1. EXISTING GROUND CONTOURS DEVELOPED FROM FALL 2009 LIDAR PROVIDED BY MCELHANNEY CONSULTING SERVICES LTD.

NAD 83 UTM ZONE 13

0	13	03	28	ISSUED FOR PRE-DESIGN	AS	DW	CA	AKS	
REV	yy	mm	dd	DESCRIPTION	BY	CHK	ENG	APP	
	DATE								

ENGINEER'S SEAL

PERMIT STAMP

ISSUED FOR
PRE-DESIGN
NOT FOR
CONSTRUCTION



AGRICULTURE AND AGRI-FOOD CANADA
HIGHFIELD DAM

REHABILITATION PROJECT PRE-DESIGN STUDY

TITLE

HERBERT CANAL - SIDE CHANNEL
SPILLWAY PLAN AND SECTIONS

DRAWN BY
AS

CHECKED BY
DW

ENGINEERED
CA

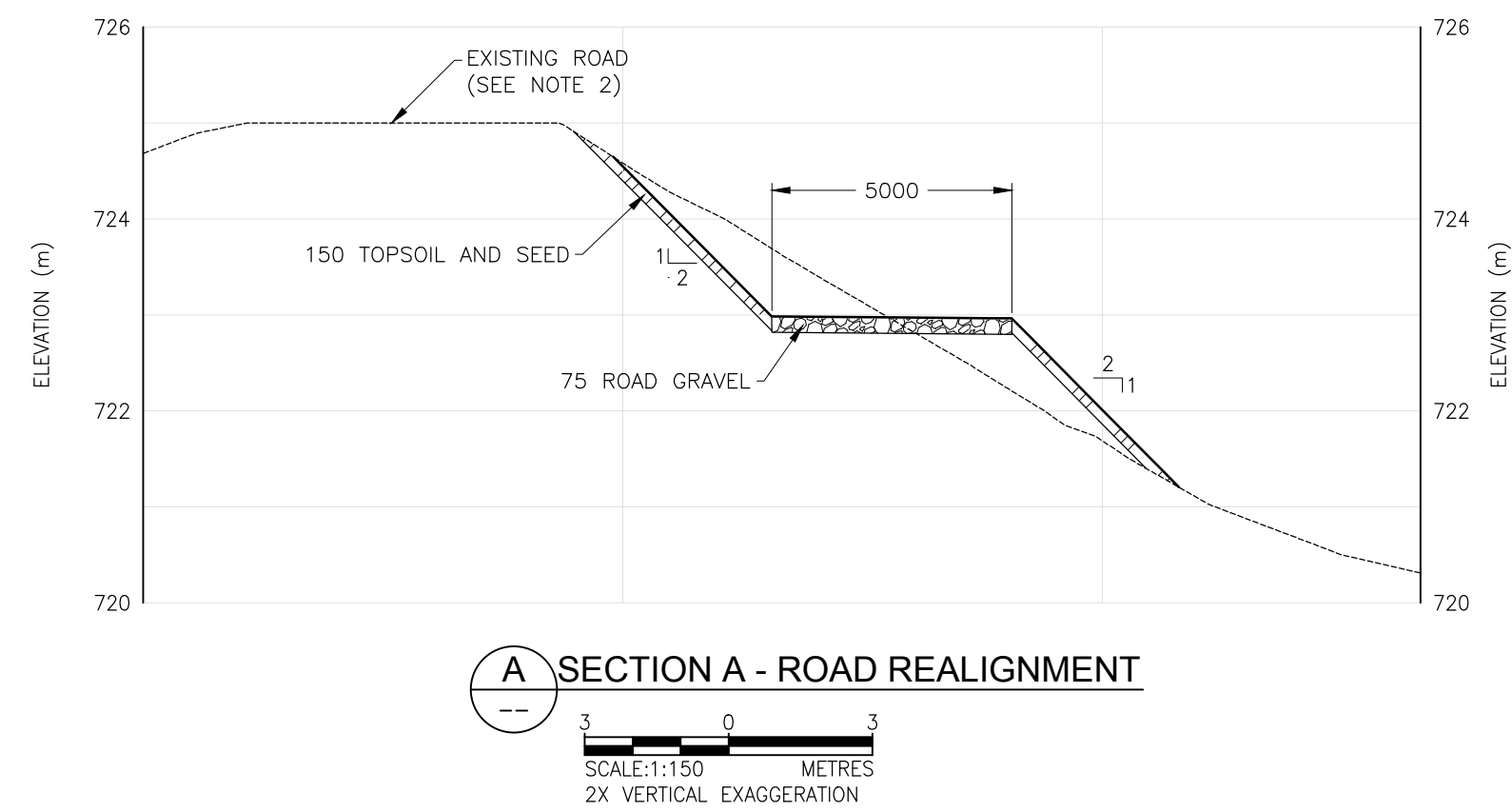
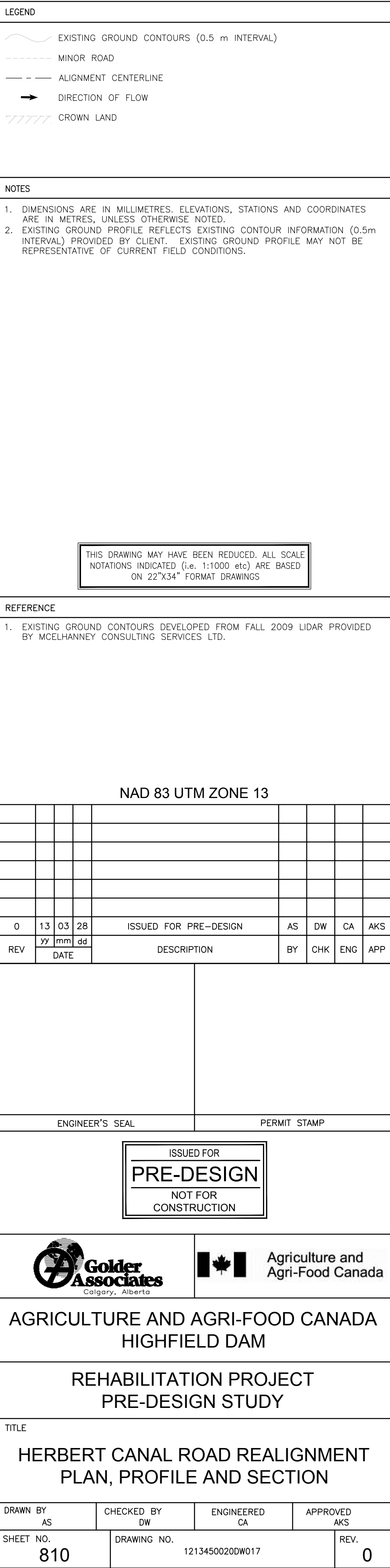
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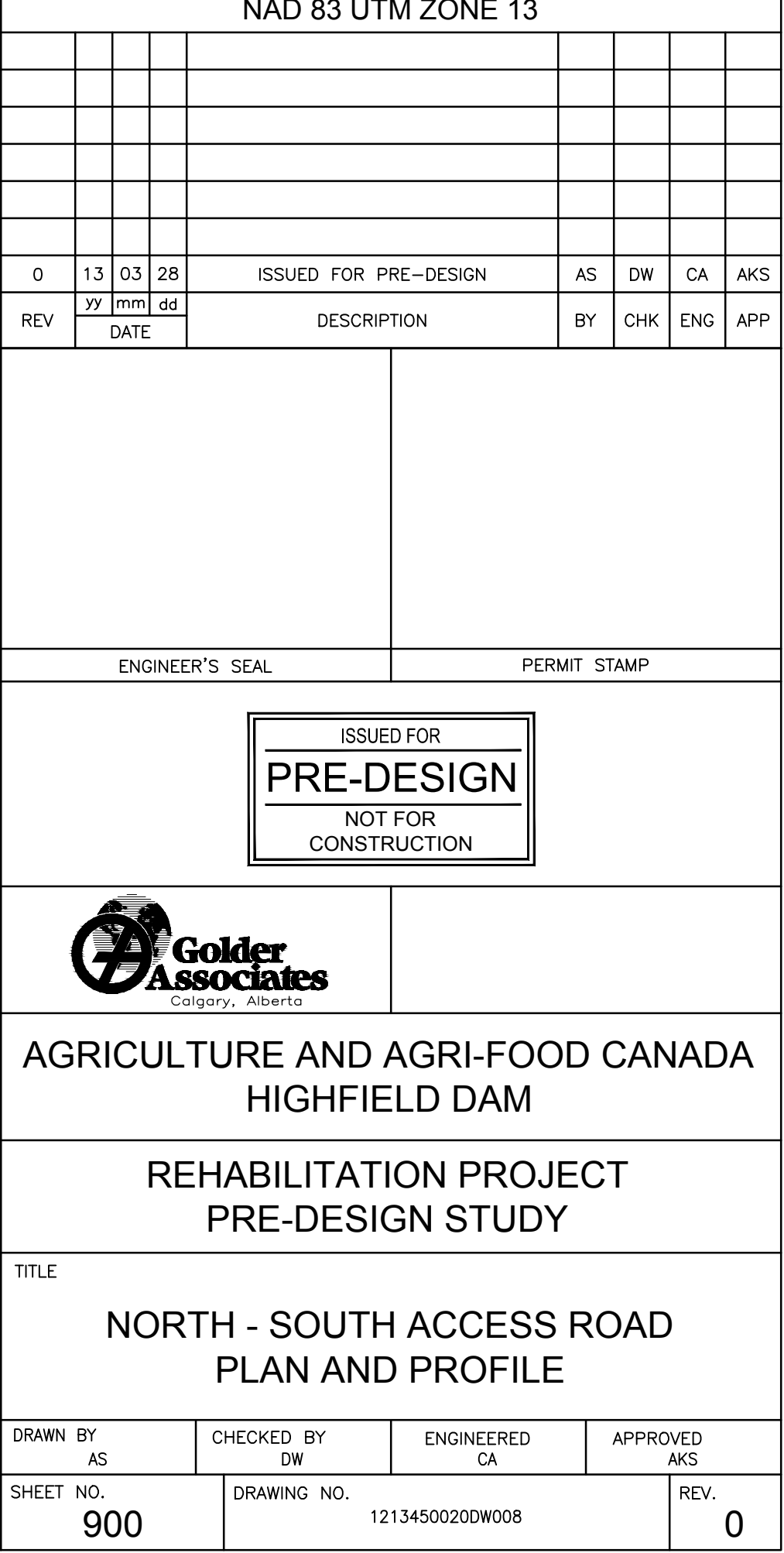
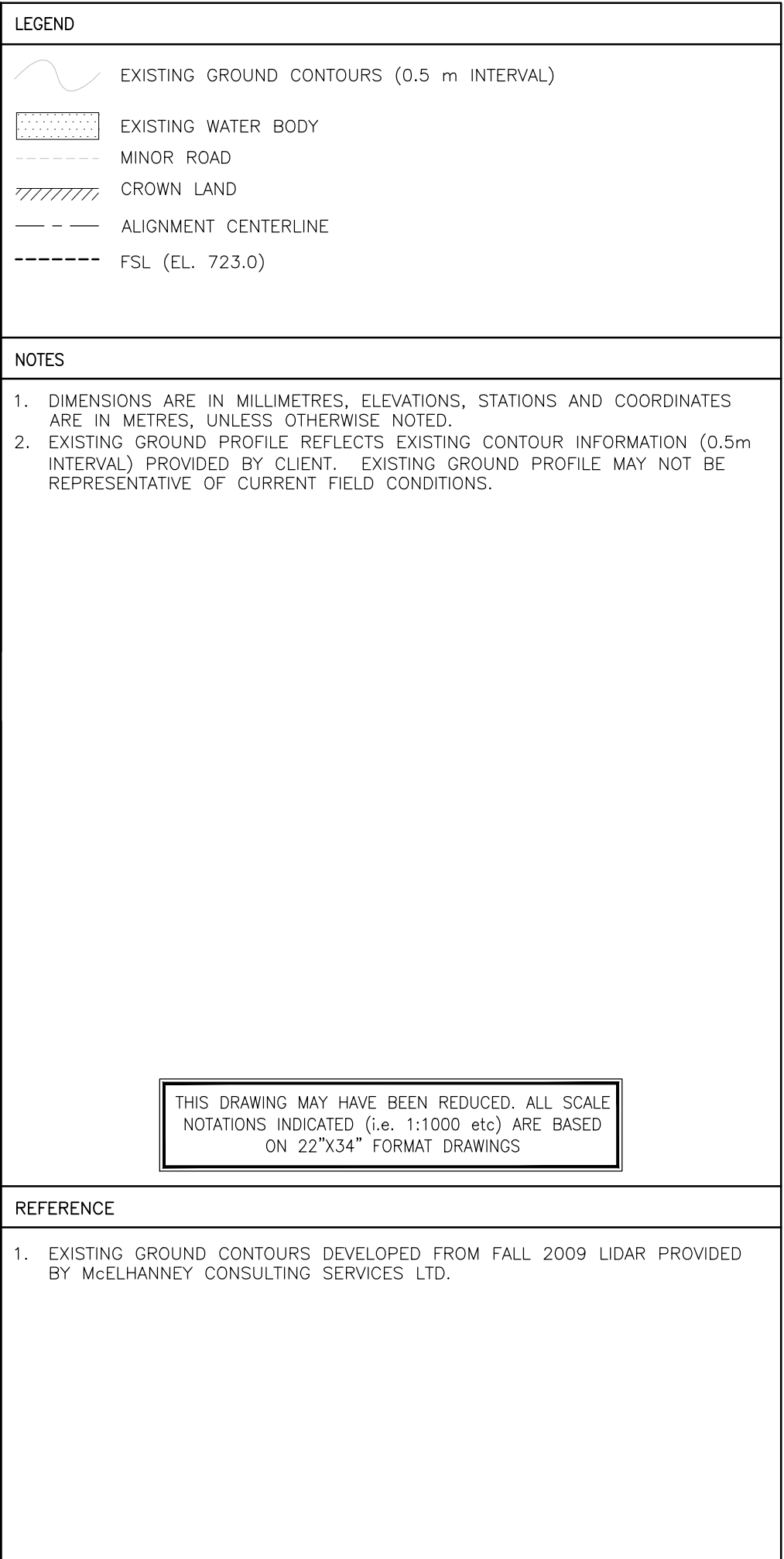
SHEET NO. 90

DRAWING NO.

1213450020DW016

REV







APPENDIX E

Cost Estimate



Bid Item	Bid Description	Description	Quantity	Unit	Rate	\$ (2012)
1000 TOE BERM						\$ 4,285,999
101	WATER, EROSION AND SEDIMENT CONTROL	Silt Control	2,400	M	\$5.98	\$ 14,354
101	WATER, EROSION AND SEDIMENT CONTROL	Silt Fence	2,400	M	\$4.07	\$ 9,770
102	STRIP AND STOCKPILE TOPSOIL	Subgrade Prep	3,735	M3	\$9.53	\$ 35,601
102	STRIP AND STOCKPILE TOPSOIL	Excavation - To Stockpile On-site	3,735	M3	\$4.05	\$ 15,109
102	STRIP AND STOCKPILE TOPSOIL	Pad Preparation	3,735	M3	\$15.26	\$ 56,980
103	DRAIN OXBOW AND WET AREAS	Pumping	1	LS	\$8,904.74	\$ 8,905
104	SELECT CLEAR AND GRUB (inc. snow)	Clear & Grub - Lump Sum	1	LS	\$4,392.09	\$ 4,392
105	EXCAVATE ORGANICS	Trucking to Disposal Site - Organics	3,735	M3	\$4.56	\$ 17,021
106	SUPPLY AND INSTALL GEOGRID	Geogrid	19,500	M2	\$6.67	\$ 130,162
106	SUPPLY AND INSTALL GEOTEXTILE	Geotextile	30,000	M2	\$10.00	\$ 300,000
107	GRANULAR MATERIAL	Fill - Import Aggregates	27,164	M3	\$107.96	\$ 2,932,726
108	PLACE GRANULAR FOR TOE BERM	Pad Preparation	21,033	M3	\$11.58	\$ 243,464
108	PLACE GRANULAR FOR TOE BERM	Subgrade Prep	21,033	M3	\$9.53	\$ 200,463
109	NATIVE BACKFILL	Fill - Native	23,907	M3	\$11.58	\$ 276,732
110	75MM GRAVEL ON FIRST BENCH	Fill - Import Aggregates	350	M3	\$107.96	\$ 37,786
110	75MM GRAVEL ON FIRST BENCH	Pad Preparation	350	M3	\$7.23	\$ 2,532
2000 WEST LOW LEVEL OUTLET						\$ 770,267
WEST LOW LEVEL OUTLET						\$ 384,522
201	FABRICATE AND SUPPLY U/S BULKHEAD	FORMWORK	1	LS	\$34,196.40	\$ 34,196
201	FABRICATE AND SUPPLY U/S BULKHEAD	REBAR	3,600	KG	\$6.11	\$ 21,983
201	FABRICATE AND SUPPLY U/S BULKHEAD	CONCRETE	1	LS	\$59,359.74	\$ 59,360
201	FABRICATE AND SUPPLY U/S BULKHEAD	EMBEDDED METALWORK	1	LS	\$8,142.00	\$ 8,142
202	CONFINED SPACE INSPECTION & CERT	Supervision	1	LS	\$4,124.09	\$ 4,124
202	CONFINED SPACE INSPECTION & CERT	Environmental Plans / Inspections	1	LS	\$5,116.92	\$ 5,117
204	DIVER PRE SURVEY AND BULKHEAD INSTALL AND REMOVE	INSTALL CONCRETE BULK HEAD	1	LS	\$25,158.78	\$ 25,159
204	DIVER PRE SURVEY AND BULKHEAD INSTALL AND REMOVE	REMOVE CONCRETE BULK HEAD	1	LS	\$10,584.60	\$ 10,585
205	DEWATER CONDUIT AND CLEAN	Pumping	1	LS	\$12,860.58	\$ 12,861
205	DEWATER CONDUIT AND CLEAN	Grinding	120	M	\$238.86	\$ 28,663
206	INSTALL HORIZONTAL LINER AND GROUT	STRUCTURES	62	M	\$1,811.40	\$ 112,307
207	GROUT AND CURE ANNULUS	CONCRETE	75	M3	\$827.01	\$ 62,025
212 BULKHEAD						\$ 8,057
212	BULKHEAD INSPECTION AND CERT	Supervision	1	LS	\$4,124.09	\$ 4,124
212	BULKHEAD INSPECTION AND CERT	Environmental Plans / Inspections	1	LS	\$3,933.16	\$ 3,933
209 GATE WELL						\$ 205,180
220	REMOVE EX INTERNAL WORKS IN GATE WELL	REMOVALS	1	LS	\$23,545.87	\$ 23,546
221	FABRICATE & SUPPLY STEEL LINER	SHAFTS	1	LS	\$96,000.00	\$ 96,000
222	INSTALL VERTICAL GATE WELL, LINER, GROUT & CURE	INSTALL METAL WORKS	1	LS	\$21,600.00	\$ 21,600
223	CAGED LADDER	EMBEDDED METALWORK	7	M	\$395.01	\$ 2,765
224	SLIDE GATE	Main	1	LS	\$28,583.55	\$ 28,584
225	GATE STEM	Main	6	M	\$545.49	\$ 3,273
226	GATE OPERATOR	Main	1	LS	\$13,113.75	\$ 13,114
227	STOP LOG SLOTS	Main	1	LS	\$7,433.11	\$ 7,433
228	STOP LOGS	Main	1	LS	\$5,557.63	\$ 5,558
230	GROUT & CURE IN ANNULUS	CONCRETE	4	M3	\$827.01	\$ 3,308
210 OUTLET BULK CONCRETE						\$ 74,365
240	CONCRETE	FORMWORK	1	LS	\$24,000.00	\$ 24,000
240	CONCRETE	CONCRETE	58	M3	\$694.40	\$ 40,275
241	20M REBAR	REBAR 20M	184	KG	\$6.11	\$ 1,125
242	10M REBAR	REBAR 10M	289	KG	\$6.11	\$ 1,765
243	FENCING SUPPLY AND INSTALL	CHAIN LINK 6FT	24	M	\$300.00	\$ 7,200
211 CONDUIT						\$ 98,144
250	SUPPLY LINER	1200MM HDPE	40	M	\$1,800.00	\$ 72,000
251	PERFORATED PVC PIPE	Main	33	M	\$34.16	\$ 1,127
355	PLASTIC STEEL MEMBERS	EMBEDDED METALWORK	40	EA	\$75.61	\$ 3,025
356	GROUT & CURE - GROUT IN ANNULUS	CONCRETE	40	M3	\$549.80	\$ 21,992
3000 DECOMMISSION EAST LOW LEVEL OUTLET						\$ 167,553
301	FABRICATE AND SUPPLY U/S BULKHEAD	FORMWORK	1	LS	\$13,027.20	\$ 13,027
301	FABRICATE AND SUPPLY U/S BULKHEAD	REBAR	1,500	KG	\$6.11	\$ 9,160
301	FABRICATE AND SUPPLY U/S BULKHEAD	CONCRETE	1	LS	\$9,954.73	\$ 9,955
301	FABRICATE AND SUPPLY U/S BULKHEAD	EMBEDDED METALWORK	1	LS	\$8,142.00	\$ 8,142
302	DIVER PRE SURVEY AND BULKHEAD INSTALL	INSTALL CONCRETE BULK HEAD	1	LS	\$17,684.42	\$ 17,684
303	DEWATER CONDUIT AND CLEAN	Pumping	1	LS	\$7,339.23	\$ 7,339
303	DEWATER CONDUIT AND CLEAN	Grinding	20	M	\$238.82	\$ 4,776
304	CONDUIT AND GATEWAY LEAN MIX FILL	CONCRETE	192	M3	\$479.51	\$ 92,066
305	BULKHEAD INSPECTION & CERT	Supervision	1	LS	\$2,062.04	\$ 2,062
305	BULKHEAD INSPECTION & CERT	Environmental Plans / Inspections	1	LS	\$3,341.28	\$ 3,341

Class C Estimate

4000 WEST SPILLWAY					\$	447,719
401 STRIP TOPSOIL AND VEGETATION	Excavation - To Stockpile On-site	2,700	M3	\$9.10	\$	24,575
402 SIDE SLOPE PREP & CHANNEL PREP & COMPACT	Subgrade Prep	2,700	M3	\$11.44	\$	30,879
403 GABION BASKET - FUSE PLUG	Gabion Walls	68	M3	\$409.80	\$	27,867
404 BACKFILL NATIVE FILL PLUG	Fill - Native	18	M3	\$9.71	\$	175
405 GRANULAR FILL (D50=200)	Excavate	45	M3	\$27.13	\$	1,221
405 GRANULAR FILL (D50=200)	Machine Placed Rip-Rap	45	M3	\$82.26	\$	3,702
405 GRANULAR FILL (D50=200)	Haul On-site	45	M3	\$209.30	\$	9,419
405 GRANULAR FILL (D50=200)	Spread and Compact On-site	45	M3	\$12.59	\$	567
406 GRANULAR BEDDING 150MM THK IMPORT	Fill - Import Aggregates	70	M3	\$105.85	\$	7,409
406 GRANULAR BEDDING 150MM THK IMPORT	Haul On-site	70	M3	\$209.30	\$	14,651
406 GRANULAR BEDDING 150MM THK IMPORT	Spread and Compact On-site	70	M3	\$12.53	\$	877
407 GRANULAR FILL (D50=50) 300THK ARMOUR	Excavate	325	M3	\$27.13	\$	8,816
407 GRANULAR FILL (D50=50) 300THK ARMOUR	Machine Placed Rip-Rap	325	M3	\$82.26	\$	26,736
407 GRANULAR FILL (D50=50) 300THK ARMOUR	Supply and Haul to On-site Stockpile	325	M3	\$241.87	\$	78,606
407 GRANULAR FILL (D50=50) 300THK ARMOUR	Spread and Compact On-site	325	M3	\$12.54	\$	4,076
408 GRANULAR FILL (D50=400) ARMOUR RIP-RAP	Excavate	150	M3	\$27.13	\$	4,069
408 GRANULAR FILL (D50=400) ARMOUR RIP-RAP	Machine Placed Rip-Rap	150	M3	\$82.26	\$	12,340
408 GRANULAR FILL (D50=400) ARMOUR RIP-RAP	Haul On-site	150	M3	\$274.43	\$	41,165
408 GRANULAR FILL (D50=400) ARMOUR RIP-RAP	Spread and Compact On-site	150	M3	\$12.53	\$	1,880
409 IMPERVIOUS FILL NATIVE On-site	Impervious Fill	220	M3	\$17.86	\$	3,929
410 BACKFILL TOPSOIL AND HYDROSEED	Topsoil - Supply & Place	1,400	M3	\$2.70	\$	3,774
410 BACKFILL TOPSOIL AND HYDROSEED	Seed & Fertilize / Hydroseed	1,400	M3	\$1.09	\$	1,522
411 GABION BASKET - DROP STRUCTURE	Gabion Walls	111	M3	\$409.80	\$	45,488
412 NON-WOVEN GEOTEXTILE (DROP STRUCTURE)	Geotextile	2,500	M2	\$5.53	\$	13,826
413 PILOT CHANNEL EXCAVATE	Excavate	200	M3	\$9.03	\$	1,807
414 GRANULAR FILL (DROP STRUCTURE)	Fill - Import Aggregates	620	M3	\$105.85	\$	65,625
414 GRANULAR FILL (DROP STRUCTURE)	Machine Placed Rip-Rap	620	M3	\$9.87	\$	6,120
415 SUB EXCAVATE SPILLWAY	Excavate	730	M3	\$9.04	\$	6,600
5000 DAM RAISE					\$	3,118,246
501 STRIP TOPSOIL AND STOCKPILE	Subgrade Prep	520	M3	\$9.53	\$	4,953
501 STRIP TOPSOIL AND STOCKPILE	Excavation - To Stockpile On-site	520	M3	\$4.05	\$	2,104
501 STRIP TOPSOIL AND STOCKPILE	Pad Preparation	520	M3	\$15.26	\$	7,935
502 RAISE PIEZOMETERS	Piezometer	5	EA	\$2,852.35	\$	14,262
503 BACKFILL NATIVE MATERIALS FROM On-site STOCKPILE	Subgrade Prep	49,500	M3	\$14.30	\$	707,652
503 BACKFILL NATIVE MATERIALS FROM On-site STOCKPILE	Excavation - To Stockpile On-site	49,500	M3	\$4.05	\$	200,244
504 PLACE TOPSOIL AND SEED	Topsoil - Supply & Place	4,549	M3	\$2.69	\$	12,259
504 PLACE TOPSOIL AND SEED	Seed & Fertilize / Hydroseed	4,549	M3	\$1.09	\$	4,946
505 EXCAVATE EXISTING ROAD GRAVEL TO On-site STOCKPILE	Fill - Import Aggregates	1,650	M3	\$107.96	\$	178,139
505 EXCAVATE EXISTING ROAD GRAVEL TO On-site STOCKPILE	Pad Preparation	1,650	M3	\$7.24	\$	11,940
505 EXCAVATE EXISTING ROAD GRAVEL TO On-site STOCKPILE	Subgrade Prep	1,650	M3	\$9.53	\$	15,726
506 PLACE TOP EMBANKMENT ROAD GRAVEL 75MM	Fill - Import Aggregates	505	M3	\$105.85	\$	53,452
506 PLACE TOP EMBANKMENT ROAD GRAVEL 75MM	Pad Preparation	505	M3	\$7.23	\$	3,653
508 GRANULAR BEDDING	Fill - Import Aggregates	1,595	M3	\$105.85	\$	168,824
508 GRANULAR BEDDING	Pad Preparation	1,595	M3	\$7.23	\$	11,538
509 OVERBUILD NATIVE BY 150MM	Fill - Native	1,000	M3	\$14.47	\$	14,469
510 SAND FILTER 150MM	Subgrade Prep	1,200	M3	\$127.05	\$	152,464
507 UPSTREAM RIP-RAP NEW ARMOUR (D50=400)	Supply Rip-Rap to On-site Stockpile	1330	M3	\$304.57	\$	405,079
513 UPSTREAM RIP-RAP (D50=300) 300THK ARMOUR	Excavate Exist Rip-Rap to Stockpile - Manual	1330	M3	\$36.73	\$	48,854
513 UPSTREAM RIP-RAP BLEND 300 & 400 THK ARMOUR	Mix 300 & 400 Rip-Rap & Place	2660	M3	\$94.60	\$	251,646
513 UPSTREAM RIP-RAP (D50=300) 300THK ARMOUR	Supply Rip-Rap to On-site Stockpile	3260	M3	\$241.87	\$	788,483
513 UPSTREAM RIP-RAP (D50=300) 300THK ARMOUR	Haul On-site	3260	M3	\$5.75	\$	18,738
513 UPSTREAM RIP-RAP (D50=300) 300THK ARMOUR	Spread and Compact On-site	3260	M3	\$12.54	\$	40,886

6000 NEW EAST LOW LEVEL OUTLET & EAST SPILLWAY					\$	1,961,326
GENERAL					\$	786,886
601 WATER, EROSION AND SEDIMENT CONTROL	Silt Control	500	M	\$5.98	\$	2,991
601 WATER, EROSION AND SEDIMENT CONTROL	Silt Fence	500	M	\$4.07	\$	2,036
602 STRIP TOPSOIL TO ON-SITE STOCKPILE	Subgrade Prep	320	M3	\$9.54	\$	3,052
602 STRIP TOPSOIL TO ON-SITE STOCKPILE	Excavation - To Stockpile On-site	320	M3	\$4.05	\$	1,296
602 STRIP TOPSOIL TO ON-SITE STOCKPILE	Pad Preparation	320	M3	\$15.25	\$	4,880
603 BULK EXCAVATION OF NATIVE MATERIAL	Excavation - Cut to Fill	6,500	M3	\$6.00	\$	39,016
605 INSTALL PLUG & DEWATERING	Excavation - Cut to Fill	40	M3	\$36.32	\$	1,453
605 INSTALL PLUG & DEWATERING	Pumping	1	LS	\$27,513.45	\$	27,513
607 BACKFILL NATIVE MATERIAL	Fill - Native	10,000	M3	\$9.65	\$	96,460
611 RIP-RAP TO INLET & AROUND DROP INLET STRUCTURE	Machine Placed Rip-Rap	2,000	M3	\$82.26	\$	164,527
611 RIP-RAP TO INLET & AROUND DROP INLET STRUCTURE	Haul On-site	2,000	M3	\$209.30	\$	418,595
611 RIP-RAP TO INLET & AROUND DROP INLET STRUCTURE	Spread and Compact On-site	2,000	M3	\$12.53	\$	25,068
NEW EAST LOW LEVEL OUTLET GATE WELL CONSTRUCTION					\$	127,337
CONCRETE	CONCRETE	55	M3	\$966.86	\$	53,177
BACKFILL ENGINEERED MATERIAL	Fill - Native	135	M3	\$9.65	\$	1,302
BACKFILL ENGINEERED MATERIAL	Fill - Import Aggregates	125	M3	\$107.96	\$	13,495
631 SLIDE GATE	Main	1	LS	\$28,583.55	\$	28,584
632 GATE STEM	Main	8	M	\$545.23	\$	4,525
633 GATE OPERATOR	Main	1	LS	\$13,113.75	\$	13,114
BOLT LADDER	EMBEDDED METALWORK	150	M	\$9.60	\$	1,440
FENCING SUPPLY AND INSTALL	CHAIN LINK 6FT	12	M	\$300.00	\$	3,600
GATEWELL COVER	FIBREGLASS	1	LS	\$900.00	\$	900
WALL COUPLINGS	LINKSEAL	2	LS	\$3,600.00	\$	7,200
640 EAST LOW LEVEL INLET CONDUIT					\$	199,031
641 SUPPLY 1500MM HDPE PIPE	Main	42	M	\$2,002.93	\$	84,123
642 INSTALL 1500MM HDPE PIPE	Main	42	M	\$80.06	\$	3,363
643 RUBBER SLEEVE	Main	42	M	\$407.10	\$	17,098
644 SUPPORT CHANNEL	STEEL C230 X 30	5	M	\$500.40	\$	2,502
645 BOLTED FLANGE COUPLING (INLET)	EMBEDDED METALWORK	1	EA	\$7,908.26	\$	7,908
646 HP 250x85	Steel	35	M	\$382.67	\$	13,394
646 HP 250x85	PILES & PILE DRIVING	35	M	\$1,595.61	\$	55,846
647 BACKFILL ENGINEERED MATERIAL	Fill - Native	135	M3	\$9.65	\$	1,302
647 BACKFILL ENGINEERED MATERIAL	Fill - Import Aggregates	125	M3	\$107.96	\$	13,495
630 EAST SPILLWAY & PIPE					\$	160,808
634 CONCRETE	CONCRETE	92	M3	\$966.86	\$	88,951
635 BACKFILL ENGINEERED MATERIAL	Fill - Native	611	M3	\$9.64	\$	5,893
635 BACKFILL ENGINEERED MATERIAL	Fill - Import Aggregates	611	M3	\$107.96	\$	65,964
650 OUTLET CONDUIT					\$	392,110
651 SUPPLY 2000MM HDPE "WEHOLITE"	Main	50	PCS	\$828.60	\$	41,430
652 INSTALL 2000MM HDPE WEHOLITE	Main	98	M	\$266.86	\$	26,152
652i SUPPLY & INSTALL Y-FITTING	1200MM TO 2000MM	1	LS	\$42,000.00	\$	42,000
653 CONCRETE (OUTLET STRUCTURE)	CONCRETE	75	M3	\$966.86	\$	72,514
654 BACKFILL ENGINEERED MATERIAL	Fill - Native	1,417	M3	\$9.65	\$	13,669
654 BACKFILL ENGINEERED MATERIAL	Fill - Import Aggregates	1,417	M3	\$107.96	\$	152,981
655 BEDDING SAND	Fill - Import Aggregates	16	M3	\$151.39	\$	2,422
656 BACKFILL GENERAL FILL	Fill - Native	4,244	M3	\$9.65	\$	40,941
STILLING BASIN & CHANNEL IMPROVEMENTS					\$	295,153
617 FENCE REMOVAL AND REINSTATEMENT	Remove Fence	400	M	\$8.82	\$	3,528
617 FENCE REMOVAL AND REINSTATEMENT	Reinstall Fence	400	M	\$14.11	\$	5,644
613 EXCAVATION OF OUTLET CHANNEL	Excavation - Cut to Fill	4,600	M3	\$9.08	\$	41,752
604 MUDSLAB BASE	Fill - Native	40	M3	\$9.71	\$	388
604 MUDSLAB BASE	Haul On-site	40	M3	\$209.29	\$	8,372
604 MUDSLAB BASE	Spread and Compact On-site	40	M3	\$25.07	\$	1,003
612 CONSTRUCT WINGWALLS AND TRANSITION	FORMWORK	60	M2	\$252.39	\$	15,144
612 CONSTRUCT WINGWALLS AND TRANSITION	REBAR	1,200	KG	\$6.47	\$	7,762
612 CONSTRUCT WINGWALLS AND TRANSITION	CONCRETE	12	M3	\$1,067.48	\$	12,810
612 CONSTRUCT WINGWALLS AND TRANSITION	EMBEDDED METALWORK	500	KG	\$20.17	\$	10,085
614i GRANULAR BEDDING	Fill - Import Aggregates	100	M3	\$105.85	\$	10,585
614i GRANULAR BEDDING	Pad Preparation	100	M3	\$7.23	\$	723
615 FINALIZE ROAD OVER EMBANKMENT AND CONDUIT	Soil Stabilization	72	M3	\$50.14	\$	3,610
614 OUTLET RIP-RAP AND RESERVOIR SIDESLOPE	Machine Placed Rip-Rap (d50=250)	500	M3	\$82.26	\$	41,131
614 OUTLET RIP-RAP AND RESERVOIR SIDESLOPE	Haul On-site	500	M3	\$209.29	\$	104,647
614 OUTLET RIP-RAP AND RESERVOIR SIDESLOPE	Spread and Compact On-site	500	M3	\$12.53	\$	6,267
610 REMOVE PLUG TO TIE IN TO RESERVOIR	REMOVE CONCRETE BULK HEAD	1	LS	\$10,584.60	\$	10,585
616 TOPSOIL AND SEED INCLUDING BORROW AREAS	Topsoil - Supply & Place	2,940	M3	\$2.69	\$	7,922
616 TOPSOIL AND SEED INCLUDING BORROW AREAS	Seed & Fertilize / Hydroseed	2,940	M3	\$1.09	\$	3,196
6200 ACCESS ROAD					\$	153,410
820 GRAVEL	Fill - Import Aggregates	1,066	M3	\$107.96	\$	115,088
820 GRAVEL	Subgrade Prep	1,066	M3	\$11.44	\$	12,192
821 CULVERT - 1200MM CSP	Main	24	M	\$461.72	\$	11,081
822 END TREATMENTS	HEADWALLS & OUTFALLS	2	EA	\$7,524.49	\$	15,049



Class C Estimate

6300 HERBERT CANAL SIDE CHANNEL WORKS					\$	379,564
901 ROADWAY	Subgrade Prep	10,000	M3	\$9.53	\$	95,304
901 ROADWAY	Fill - Import Aggregates	10,000	M3	\$2.00	\$	19,973
901 ROADWAY	Pad Preparation	10,000	M3	\$0.13	\$	1,268
903 SIDE CHANNEL	Subgrade Prep	2,000	M3	\$9.53	\$	19,064
903 SIDE CHANNEL	Excavation - To Stockpile On-site	2,000	M3	\$4.04	\$	8,090
903 SIDE CHANNEL	Pad Preparation	2,000	M3	\$15.25	\$	30,509
904 NON-WOVEN GEOTEXTILE	Geotextile	1,120	M2	\$6.12	\$	6,850
905 GRANULAR FILTER	Fill - Import Aggregates	280	M3	\$139.43	\$	39,041
905 GRANULAR FILTER	Pad Preparation	280	M3	\$7.24	\$	2,027
906 GABION MATTRESS	Gabion Walls	336	M3	\$362.58	\$	121,826
907 GABION BASKET	Gabion Walls	112	M3	\$317.96	\$	35,612
9999 INDIRECTS					\$	84,677
99999 INDIRECTS	Mobilize Equipment - Phase 1 & 2	1	LS	\$65,136.00	\$	65,136
99999 INDIRECTS	Site Facilities	1	LS	\$19,540.80	\$	19,541
ENGINEERING, RESIDENT & QA					\$	1,780,200
Engineering, Resident & QA in Budget Year 2013/14		1	LS	\$1,104,000	\$	1,104,000
Engineering, Resident & QA in Budget Year 2014/15		1	LS	\$345,000	\$	345,000
Engineering, Resident & QA in Budget Year 2015/16		1	LS	\$276,000	\$	276,000
Engineering, Resident & QA in Budget Year 2016/17		1	LS	\$55,200	\$	55,200
AAFC LAND PURCHASE					\$	2,500
North East Corner of Dam Borrow Area (AAFC Rate Provided)		1	Hec	\$2,500	\$	2,500
TOTAL PROJECT COST					\$	13,151,000
*includes 20% contingency						

SUMMARY TABLE					\$	13,151,000
1000 TOE BERM					\$	4,286,000
2000 WEST LOW LEVEL OUTLET					\$	770,000
3000 DECOMMISSION EAST LOW LEVEL OUTLET					\$	167,600
4000 WEST SPILLWAY					\$	447,700
5000 DAM RAISE					\$	3,118,200
6000 NEW EAST LOW LEVEL OUTLET & EAST SPILLWAY					\$	1,961,300
6200 ACCESS ROAD					\$	153,400
6300 HERBERT CANAL SIDE CHANNEL WORKS					\$	379,600
9999 INDIRECTS					\$	84,700
ENGINEERING, RESIDENT & QA					\$	1,780,200
AAFC LAND PURCHASE					\$	2,500



APPENDIX F

Preliminary Risk Register



AAFC HIGHFIELD DAM
REHABILITATION
12-1345-0020

Probability (P)		Severity					Risk Classifications:	
		5	4	3	2	1		
		Catastrophic	Major	Significant	Minor	Insignificant		
5	Almost Certain	25	20	15	10	5	20-25 =	Very High Risk
4	Likely	20	16	12	8	4	15-16 =	High Risk
3	Possible	15	12	9	6	3	8,9,10 or 12 =	Moderate Risk
2	Unlikely	10	8	6	4	2	4,5 or 6 =	Low Risk
1	Rare	5	4	3	2	1	1,2 or 3 =	Very Low Risk

Risk Register

No.	Key Element	Category	Project Phase	Event	Possible Consequence	Severity	Probability	Uncontrolled Risk Rating	Options Considered to Mitigate Risk	Severity	Probability	Mitigated Rating	Bid Item Affected
	WLLO, ELLO	Business Risk	Identification	Concrete reinforcement in existing structures not assessed	Safety - Public	4	3	12	Destructive (coring) / non-destructive testing (GPR)	4	2	8	
	Reservoir system	Project Delivery Risk	Identification	Not accounting for all downstream water users	Reliability - Customer	4	2	8	AAFC to follow up with confirmation of water users / licensees	2	2	4	
	Reservoir system	Business Risk	Definition	Assessment of siltation in reservoir	Reliability - Water Supply	3	3	9	Bathymetric survey to quantify / dredging to rehabilitate	3	2	6	
	Embankment Dam	Project Delivery Risk	Design	Low confidence in LiDAR survey data	Financial	3	4	12	Ground control survey tied into SLS/CLS monuments	1	4	4	
	Reservoir system	Business Risk	Definition	Statistical error in hydrometric data	Reliability - Water Supply	4	3	12				0	
	Embankment Dam	Project Delivery Risk	Identification	Failure to identify adverse foundation conditions	Safety - Public	5	3	15	Infill geotechnical investigations	5	2	10	
	Reservoir system	Business Risk	Operation	Extreme climate variability	Reliability - Water Supply	5	3	15	Review and revise climate data periodically	5	2	10	
	Reservoir system	Business Risk	Design	Changes in existing area-storage reservoir curves since 2011 due to sedimentation	Reliability - Water Supply	4	3	12	Current bathymetric survey	4	2	8	
	Reservoir system	Business Risk	Identification	Not accounting for snowmelt event or ice formation thus, limiting the storage in reservoir.	Reliability - Water Supply	4	3	12	Analyse climate data, synthesize short term record to predict future snow / ice accumulation	4	2	8	
	Reservoir system	Project Delivery Risk	Definition	Validity of rating curves provided by AAFC	Financial	3	3	9	Independent development of rating cruves	3	2	6	
	Embankment Dam	Project Delivery Risk	Definition	Assumption of the correct dam classification	Safety - Public	4	3	12	Independent confirmation of dam classification	3	2	6	
	Embankment Dam	Business Risk	Definition	Selection of appropriate model parameters for stability analyses	Safety - Public	4	3	12	Model validation (representative samples that are lab tested)	4	1	4	
	Reservoir system	Business Risk	Definition	Reservoir routing assumptions (timing/quantities)	Reliability - Water Supply	3	4	12	Routing analysis	3	2	6	
	Reservoir system	Business Risk	Definition	Assumption of hydraulic coefficients	Reliability - Water Supply	4	4	16	Validate hydraulic coefficients (model testing)	4	2	8	
	Embankment Dam	Business Risk	Definition	Definition of design seismic event	Safety - Public	5	3	15	Confirm local seismicity and adjust dam consequence classification accordingly	5	2	10	
	Reservoir system	Business Risk	Definition	Definition of design flood event	Safety - Public	5	3	15	Confirm flood frequency analysis and adjust design accordingly	5	2	10	
	Reservoir system	Business Risk	Definition	Hydraulic capacity may be lower due to outdated design curves provided by AAFC.	Reliability - Water Supply	5	3	15	Confirm capacity curves during detailed design	5	2	10	
	Dam, ELLO, WLLO, Spillway	Project Delivery Risk	Implementation	Availability of labour	Financial	4	4	16	Provide enough time to procure an adequately trained work force	3	3	9	
	Dam, ELLO, WLLO, Spillway	Project Delivery Risk	Implementation	Availability of equipment	Financial	4	4	16	Provide enough time to procure the correct equipment	3	3	9	
	Dam, Spillway	Project Delivery Risk	Implementation	Availability of d50 = 400 mm riprap	Financial	4	4	16	Procure material in advance	2	2	4	
	Dam, spillway, ELLO	Project Delivery Risk	Implementation	Availability of granular material	Financial	4	4	16	Procure material in advance	2	2	4	
	Dam, spillway, ELLO, WLLO, Herbert Canal	Schedule Risk	Implementation	Seasonal delays due to adverse weather	Financial	4	4	16	Implement weather-preparedness plan; schedule work during seasons with more predictably favourable weather	2	4	8	
	ELLO, WLLO, Herbert Canal	Schedule Risk	Implementation	Work outside of aquatic timing windows	Environmental	4	4	16	Implement tight control on tasks that may impact downstream habitat	4	3	12	



AAFC HIGHFIELD DAM
REHABILITATION
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Probability (P)		Severity					Risk Classifications:	
		5	4	3	2	1		
		Catastrophic	Major	Significant	Minor	Insignificant		
5	Almost Certain	25	20	15	10	5	20-25 =	Very High Risk
4	Likely	20	16	12	8	4	15-16 =	High Risk
3	Possible	15	12	9	6	3	8,9,10 or 12 =	Moderate Risk
2	Unlikely	10	8	6	4	2	4,5 or 6 =	Low Risk
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Risk Register

No.	Key Element	Category	Project Phase	Event	Possible Consequence	Severity	Probability	Uncontrolled Risk Rating	Options Considered to Mitigate Risk	Severity	Probability	Mitigated Rating	Bid Item Affected
	Dam, ELLO, Herbert Canal	Schedule Risk	Implementation	Work outside of migratory bird timing windows	Environmental	4	4	16	Clear brush in advance of migratory bird window	4	3	12	
	Dam, spillway, ELLO, WLLO	Business Risk	Implementation	HSE events	Safety - Worker	5	5	25	Implement site controls, pre-work hazard assessments, bulkhead/confined space entry certification, safe work practices	5	2	10	
	Dam, spillway, ELLO, WLLO	Project Delivery Risk	Implementation	Unforeseen construction conditions	Financial	4	4	16	Confirm observations of past site reviews and note/adapt to any changes that have occurred following those reviews	3	3	9	
	Dam, spillway, ELLO, WLLO	Project Delivery Risk	Implementation	Regulatory risk - change in policy or design requirements	Financial	3	3	9	Adopt a forward-looking assessment and design approach to anticipate future regulatory changes	3	2	6	
	ELLO	Project Delivery Risk	Implementation	Slaking of native bedrock (potential seepage pathways through dam)	Safety - Public	5	4	20	Seal / cover exposed bedrock to prevent deterioration under atmospheric conditions	2	4	8	
	Dam, Herbert Canal	Business Risk	Operation	Adverse visual impact	Reputation	3	4	12	Adopt harmonious design, landscape surrounding area and revegetate with native grasses	1	3	3	
	Dam	Project Delivery Risk	Definition	Prediction of pore water dissipation	Safety - Public	5	4	20	Assess pore water pressure trigger levels and monitor to respond quickly to potential embankment instability	5	2	10	
	Dam, spillway, ELLO, WLLO	Schedule Risk	Implementation	Delays in project sequencing	Reputation	4	4	16	Employ tight contruction project management; possibly implement contractor penalties for failure to deliver	4	2	8	
	Reservoir system	Business Risk	Implementation	Failure to meet downstream water requirements	Reliability - Water Supply	5	4	20	Prepare measures to ensure that adequate water will be available to meet downstream requirements	4	2	8	
	ELLO, WLLO, Herbert Canal	Business Risk	Implementation	Failure to comply with DFO operating windows	Environmental	5	4	20	Engage with DFO to address any in-stream works; apply for HADDs if work must occur outside of timing windows	4	2	8	
	Dam, Spillway, ELLO, WLLO, Herbert Canal	Schedule Risk	Design	Design revision	Financial	5	5	25	Early client-consultant-contractor engagement to reduce risk of design changes later in project development	3	3	9	
	Dam, spillway, ELLO, WLLO, Herbert Canal	Business Risk	Identification	Proximity of material supply	Financial	4	4	16	Identify materials with long lead times / long distances to haul. Optimize design to reduce material requirements. Procure early to ensure supply.	3	3	9	
	Reservoir system	Business Risk	Identification	Land acquisition	Financial	4	3	12	Identify land to be acquired early and initiate negotiations / purchase agreements	3	3	9	
		Schedule Risk	Definition	AAFC large project tendering process delays	Financial	4	4	16	Provide enough time in design > construction schedule to allow for AAFC/PWGSC tendering and procurement processes	2	4	8	
	Dam	Business Risk	Identification	Embankment settlement	Safety - Public	4	3	12	Provide overbuild during dam construction to account for anticipated total/differential settlement	2	3	6	
	Dam	Business Risk	Identification	Availability of on-site borrow	Financial	4	4	16	Assess availability of borrow and ensure provate land acquisition/agreements are in place prior to construction	2	3	6	
		Project Delivery Risk	Identification	Seasonal closure of local roads	Financial	3	4	12	Identify potential road closers early and develop project schedule accordingly	3	2	6	
		Business Risk	Identification	Soil / groundwater contamination	Financial	3	3	9	Conduct Phase I / II environmental site assessment to identify any contaminants that may affect construction	3	1	3	
		Project Delivery Risk	Implementation	Permits / licenses / approvals	Financial	4	4	16	Ensure that all permitting and regulatory processes are initiated / completed prior to construction	4	1	4	
		Project Delivery Risk	Identification	Identification, delineation, and clearance of any areas of archaeological/heritage significance.	Reputation	3	3	9	Conduct heritage resource assessment / avoid identified areas or catelog and relocate artifacts	1	3	3	
	Dam, Herbert Canal	Project Delivery Risk	Identification	Presence of Northern leopard frog in project footprint	Environmental	4	3	12	Live trap and relocate frogs; dewater habitat outside of frog mating and overwintering seasons	3	3	9	
	Dam, Herbert Canal, local borrow areas	Project Delivery Risk	Identification	Presence of Shrike (migratory bird)	Environmental	4	3	12	Clear brush in advance of migratory bird window	3	3	9	
	Dam, Herbert Canal, spillway, ELLO	Project Delivery Risk	Implementation	Winter construction	Financial	3	4	12	Align project construction schedule to minimize winter work	2	4	8	
		Project Delivery Risk	Implementation	Dam breach during construction	Safety - Public	5	4	20	Monitor pore water pressures during embankment raise; ensure cofferdam integrity (if required); provide one operational spillway at all times	5	3	15	
	ELLO, WLLO	Project Delivery Risk	Implementation	WLLO / ELLO structural failure under loading	Safety - Worker	5	4	20	Restrict loaded traffic over ELLO and WLLO sluiceways; limit worker activity inside pre-rehabilitation sluiceway	5	2	10	
	ELLO	Schedule Risk	Identification	Availability of barge for pile driving (ELLO)	Financial	3	3	9	Identify barge/pile driving subcontractor early	2	3	6	
	Reservoir system	Business Risk	Implementation	Cattle control	Environmental	3	3	9	Fence off active contruction areas to limit cattle-human-equipment interactions.	3	2	6	

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