AAFC/AESB HIGHFIELD DAM
SERVICE CONTRACT NO. 3
SPILLWAY PRE-DESIGN COMPLETION

35525

FINALREPORT

Prepared for:

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

Agriculture and Agri-Food Canada
Agri-Environment Services Branch

Date:
November 30, 2011

Report Prepared By:

nhc
northwest hydraulic consultants

In Association with:

MDH
ENGINEERED SOLUTIONS

DISCLAIMER

This document has been prepared by Northwest Hydraulic Consultants Ltd. in accordance with generally accepted engineering and geoscience practices and is intended for the exclusive use and benefit of the client for whom it was prepared and for the particular purpose for which it was prepared. No other warranty, expressed or implied, is made.

Northwest Hydraulic Consultants Ltd. and its officers, directors, employees, and agents assume no responsibility for the reliance upon this document or any of its contents by any party other than the client for whom the document was prepared. The contents of this document are not to be relied upon or used, in whole or in part, by or for the benefit of others without specific written authorization from Northwest Hydraulic Consultants Ltd. and our client.

EXECUTIVE SUMMARY

Highfield Dam is located in N36-15-11W3 on Rush Lake Creek, approximately 10 km south of Rush Lake, SK and 28 km east of Swift Current, SK. The original project was constructed in 1942 and is currently operated and maintained by the Agri-Environment Services Branch (AESB, formerly PFRA) of Agriculture and Agri-Food Canada (AAFC).

AESB/PFRA Dam Safety Reviews have indicated that the project's existing spillway and low-level outlet facilities cannot safely pass the project's estimated Inflow Design Flood (IDF) of 361 m^3/s. In order to address these deficiencies, AAFC/AESB conducted conceptual and preliminary level studies, as described in the Highfield Dam Project Rehabilitation Predesign Report (July 2009), which produced three spillway design alternatives, but did not provide a clear recommendation on the preferred alternative.

AESB retained Northwest Hydraulic Consultants (NHC), with subconsulting services provided by MDH Engineered Solutions Corp. (MDH), to complete the pre-design of the proposed spillway upgrade. This included reviewing new information available since completion of the previous study; updating and refining the three previously developed alternatives; developing three additional design alternatives; applying a systematic multi-criteria analysis (MCA) to compare all six alternatives; and, selecting the preferred spillway design alternative.

The selected alternative that resulted from the MCA was an earth spillway constructed on the east abutment. The proposed spillway upgrade will include construction of a 100 m wide earth-cut channel leading to a concrete stepped energy dissipating structure and rock apron, and a rip rap protected earth berm downstream from the dissipator to direct spillway flows away from the toe of the dam and into Rush Lake Creek. A bridge will be installed over the earth-cut spillway channel to provide operator access to the existing low level outlet controls, and the existing earth spillway located on the west abutment will be plugged and rendered inoperable.

With this spillway design, the top elevation of the embankment dam will need to be raised 0.9 m, from El. 724.8 m to El. 725.7 m, to meet the freeboard requirements as specified in the 2007 Canadian Dam Association (CDA) guidelines. The findings of the geotechnical assessments showed that in order to meet 2007 CDA guidelines for the overall factor of safety for dam stability, the maximum recommended slope for the downstream side of the embankment dam is 1 vertical to 6 horizontal (1V:6H). This is significantly less than the existing downstream slope, indicating that remedial work will be required for dam stability whether or not spillway upgrades are implemented. The existing low-level outlets at the east and west ends of the embankment will be extended downstream to accommodate the raised embankment and shallower embankment slope.

Side slopes of 1V:5H vertical are recommended in the earth-cut spillway channel in order to have long term stability of the slopes within the clay shale soils. Establishing vegetation within the spillway channel is also important for providing protection against erosion.

The majority of the construction can likely be completed in the dry, with possible diversion of the low-level outlet flows while the outlet structures are extended downstream, during construction of the energy dissipator structure and placement of the rock apron. Staging of the embankment construction may need to be staged over two construction seasons to ensure that pore water pressures resulting from the embankment loading through various stages of construction do not cause the existing dam stability to be compromised. Instrumentation will be required to monitor stability of the embankment, settlement, and porewater pressures during and after construction.

Additional geotechnical testing, including additional drilling along the length of the dam, consolidation testing of the dam embankment, and drilling test holes at the bridge structures, are recommended to better define the subsurface conditions prior to the detailed design stage.

The total capital cost estimated for the proposed spillway upgrade is $14.5 million; the annual maintenance cost is estimated at $225,000, and the annual operating cost is expected to be $5,000. The resulting net present value (NPV) for the selected alternative is estimated at $18.7 million.

Due to the high costs of implementing the spillway upgrades, it is recommended that a value engineering assessment be considered to evaluate the ratio of the spillway's function to its costs. While the basic function of the spillway is to safely pass the design flood, it is possible that by relaxing some of the design criteria established at the onset of the study, significant cost savings could be achieved.

CREDITS AND ACKNOWLEDGEMENTS

The authors would like to thank AESB for initiating this study. Technical liaison for the study was maintained with the following AESB personnel:

- Garth Haack — Project Manager
- Glenn McLaughlin — Manager of Capital Projects
- Wayne Carlson — Assistant Project Manager
- Carrie Wallace — Project Engineer

The following NHC personnel participated in the study:

- Brian Hughes, P.Eng. — Project Manager & Principal in Charge
- Edwin Zapel, P.E. — Senior Technical Reviewer
- Kara Hurtig, P.Eng. — Project Engineer

The following MDH personnel participated in the study:

- Darrell Mihial, P.Eng. — Senior Geo-Environmental Engineer
- P. Jorges Antunes, P.Eng. — Geotechnical Lead
- Fred Antunes — Civil Design / Costing Lead

TABLE OF CONTENTS

List of Tables ..... **vi**

List of Figures ..... **vi**

List of Appendices ..... **vi**

1 INTRODUCTION **1**

1.1 Project Background 1

2 TECHNICAL APPROACH **3**

3 INFORMATION REVIEW **4**

3.1 Hydrologic Design Criteria 4

3.2 Hydraulic Design Criteria 5

3.3 Geotechnical Design Criteria 7

3.4 Environmental Considerations 8

4 DEVELOPMENT OF DESIGN ALTERNATIVES **10**

4.1 Review of Original Three Design Alternatives 10

4.2 Development of Additional Alternatives 12

5 EVALUATION AND COMPARISON OF ALTERNATIVES **15**

5.1 Flood Routing Results 15

5.2 Cost Analysis Summary 16

5.3 Multi-Criteria Analysis 18

6 PRE-DESIGN FOR SELECTED ALTERNATIVE **21**

6.1 Pre-Design for Selected Alternative 21

6.2 Constructability Considerations 24

6.3 Construction Sequencing 27

6.4 Updated Cost Estimates for Selected Alternative 28

7 SUMMARY AND RECOMMENDATIONS **31**

8 REFERENCES **33**

List of Tables

Table 3.1 Design Floods for Highfield Dam Spillway Pre-Design
Table 3.2 Freeboard Requirements

Table 5.1 Flood Routing Results – Alternatives 1 to 6
Table 5.2 Summary of Pre-design Cost Estimates
Table 5.3 Summary of Multi-Criteria Analysis Primary Objectives and Weighting
Table 5.4 Summary of Multi-Criteria Analysis Results

Table 6.1 Flood Routing Results – Earth Spillway on East Abutment
Table 6.2 Quantities and Capital Cost Estimates Summary – Earth Spillway on East Abutment
Table 6.3 Summary of Cost Estimates – Earth Spillway on East Abutment

List of Figures

Figure 1 Existing Site Layout
Figure 2 Alternative 1 Ungated Labyrinth Weir – East (Plan)
Figure 3 Alternative 1 Ungated Labyrinth Weir – East (Details)
Figure 4 Alternative 2 Gated Spillway – East (Plan)
Figure 5 Alternative 2 Gated Spillway – East (Details)
Figure 6 Alternative 3 New Earth Spillway – East (Plan)
Figure 7 Alternative 3 New Earth Spillway – East (Details)
Figure 8 Alternative 4 Ungated Labyrinth Weir – West (Plan)
Figure 9 Alternative 4 Ungated Labyrinth Weir – West (Details).
Figure 10 Alternative 5 Gated Spillway – West (Plan)
Figure 11 Alternative 5 Gated Spillway – West (Details)
Figure 12 Alternative 6 New Earth Spillway – West (Plan)
Figure 13 Alternative 6 New Earth Spillway – West (Details)
Figure 14 Spillway Pre-Design - Earth Spillway on East Abutment (Plan)
Figure 15 Spillway Pre-Design - Earth Spillway on East Abutment (Details)
Figure 16 Spillway Pre-Design – Low Level Outlets
Figure 17 Spillway Discharge Curve - Earth Spillway on East Abutment

List of Appendices

Appendix A. Highfield Dam Background Information
Appendix B. Spillway Upgrade at Highfield Dam Final Report (MDH, November 2011)

1 INTRODUCTION

Northwest Hydraulic Consultants Ltd. (NHC) and MDH Engineered Solutions (MDH) were selected by Agri-Environment Services Branch (AESB) of Agriculture and Agri-Food Canada (AAFC) to complete the pre-design for a new spillway facility at Highfield Dam. The project was conducted in accordance with an agreement between AESB and NHC dated 10 November 2010.

1.1 PROJECT BACKGROUND

Highfield Dam is located in N36-15-11W3 on Rush Lake Creek, approximately 10 km south of Rush Lake, SK and 28 km east of Swift Current, SK. The original project was constructed in 1942 and is currently operated and maintained by the Agri-Environment Services Branch (AESB)[1]of Agriculture and Agri-Food Canada (AAFC).

The existing project is comprised of a 1040 m long earthfill embankment, a 20 m wide earth spillway channel located at the west end of the embankment, and low-level outlets at the west and east ends of the embankment. The embankment crest is set at El. 724.8 m and the Full Supply Level (FSL) is El. 723.0 m, providing 1.8 m of freeboard under normal operation. The existing low-level outlets have a combined capacity of approximately 17 m^3/s and the existing earth-cut spillway has a capacity of approximately 25.5 m^3/s (with 0.6 m of freeboard on the embankment), for an overall discharge capacity of 42.5 m^3/s.

AESB / PFRA Dam Safety Reviews have indicated that the project's existing spillway and outlet facilities cannot safely pass the project's Inflow Design Flood (IDF), which has been estimated at 361 m^3/s. In addition, the freeboard on the dam is inadequate by current dam safety standards. In order to address these deficiencies, AAFC/AESB conducted conceptual and preliminary level studies, as described in the Highfield Dam Project Rehabilitation Predesign Report (July 2009), which produced three spillway alternatives:

- Alternative 1: Labyrinth Spillway - New operating spillway located on east abutment consisting of an ungated labyrinth concrete chute spillway
- Alternative 2: Gated Spillway - New operating spillway located on east abutment consisting of a gated concrete chute spillway
- Alternative 3: Earth Spillway - New operating spillway located on east abutment consisting of an earth-cut spillway channel

The July 2009 report was prepared without the results of the geotechnical investigation that was completed in April 2010nor the rare plant, fish and wildlife study that was completed in

[1]AESB was formerly the Prairie Farm Rehabilitation Administration (PFRA)

December 2010. In addition, several design alternatives were not fully considered in the evaluation, and a single recommended option was not explicitly stated.

The purpose of the Spillway Pre-Design Completion Project, as presented in this report, is to use the new information available to review and update the designs for the three alternatives previously developed by AESB; to develop three additional design alternatives; and, to provide a clear recommendation for a preferred alternative with sufficient background information to proceed to final design for the project.

2 TECHNICAL APPROACH

The technical approach followed by NHC and MDH for the Highfield Dam Spillway Pre-Design Completion project included the following key tasks:

- Development and approval of the Multi-Criteria Analysis (MCA) process to be used in comparing the spillway alternatives;
- Review of existing information related to the project and the three alternatives previously developed by AESB;
- Refinement of the three previous design alternatives and development of three additional design alternatives;
- Development of conceptual-level designs and cost estimates for all six alternatives;
- Comparison of the six alternatives using the MCA, and selection of a single preferred alternative; and
- Completion of the pre-design for the selected alternative.

The NHC/MDH team worked closely with each other and with personnel from AESB throughout the duration of the project and drew on the combined experience of the project team in development of the alternative designs. The main roles and responsibilities of NHC and MDH for the study were:

- NHC – responsible for project management and administration, hydrology, hydraulic design and analysis, development and evaluation of alternatives
- MDH – responsible for geotechnical engineering, constructability and cost estimating for each of the alternatives

AESB personnel provided all relevant data used in the study, provided input to the MCA and design criteria, and reviewed all key design decisions. At the onset of the study, a kick-off meeting was held at AESB's offices in Regina, followed by a visit to the project site. Conference calls were generally held bi-weekly between AESB, NHC and MDH to discuss progress of the study and ensure clear and concise communication of information and objectives.

3 INFORMATION REVIEW

A review was conducted of the hydrologic, hydraulic, geotechnical and environmental data available for the project. The review included verification of the designs described in the Highfield Dam Project Rehabilitation Predesign Report (July 2009) with consideration for updated information. This included the Highfield Dam Geotechnical PreDesign Draft (April 2010); updated flooded area and reservoir capacity curves based on recent LiDAR survey data (provided by AESB in Dec 2010); and, the Rare Plant, Fish and Wildlife Study Report (Dec 2010). The results of the hydraulic review, geotechnical assessment and stability analysis were then used to determine the parameters and details for each of the alternatives and were incorporated into the cost estimates and comparisons of the alternatives.

3.1 HYDROLOGIC DESIGN CRITERIA

In reference to the Canadian Dam Association (CDA) Dam Safety Guidelines (2007), Highfield Dam would likely be classified as 'Significant' or 'High' in terms of consequence potential. The hazard rating is a result of the presence of the TransCanada Highway and Canadian Pacific Rail (CPR) mainline located approximately 8 km downstream, but the relatively few habitations in the flood risk area. As a result of its classification range, the Inflow Design Flood (IDF) was selected as the 1:1000 year return period and the Operating Spillway Design Flood (OSDF) was selected as the 1:200 year return period.

AESB conducted a detailed flood frequency analyses of both snowmelt and rainfall events in November 2007 to update the inflow hydrographs for Highfield Dam. The rainfall summer events were found to be larger than snowmelt events for return periods larger than 1:50, and as a result they were adopted for use in this study. The resulting peak flow and runoff volumes for these pertinent flood events are summarized in Table 3.1 below.[2]

Table 3.1 Design Floods for Highfield Dam Spillway Pre-Design

Design Flood	Return Period	Instantaneous Peak Flow (m^3/s)	Runoff Volume (dam^3)
OSDF	1:200	180	26,900
IDF	1:1000	361	60,300

[2]The hydrologic design criteria and design floods presented in this section were previously developed by AESB. The current project did not re-evaluate these parameters.

3.2 HYDRAULIC DESIGN CRITERIA

Existing Spillway Capacity

The existing Highfield Dam spillway is a 20 m wide earth spillway located at the west end of the dam embankment. The spillway channel invert is at El. 723.0 m, and the spillway control is a road set at El. 723.3 m that crosses the spillway channel in-line with the dam embankment crest. In addition to the earth spillway there are two existing low-level outlets: the west outlet that discharges into the Herbert Main Canal; and, the east outlet that discharges into Rush Lake Creek. A plan view of the existing site layout is shown in Figure 1. The existing spillway capacity and low-level outlet rating curves were provided by AESB for use in the spillway pre-design project and are included in Appendix A of this report[3].

According to the rating curves, the existing spillway capacity is approximately 50.3 m^3/s with the road in place and 58.4 m^3/s with the road eroded (with the reservoir level at the embankment crest elevation of 724.8 m). The maximum discharge through the east outlet is estimated at 11.6 m^3/s with the reservoir level at the top of the gate well (El. 724.54 m); and operation of the west outlet is limited by the capacity of the Herbert Main Canal, which is approximately 6 m^3/s. At the Full Supply Level (FSL) of El. 723.0 m, the west outlet can only be operated at a 50% gate opening.

A preliminary review of information provided for the TransCanada Highway bridge crossings over Rush Lake Creek located approximately 8 km downstream from the dam, and for the CPR mainline bridge located approximately 11 km downstream from the dam indicate that these crossing may not have sufficient capacity to pass the spillway outflows for the IDF design condition. AESB indicated that the capacities of these crossings should not dictate the allowable release from the spillway, and that adherence to the CDA Dam Safety Guidelines should be the overriding factor. It is recommended that the authorities responsible for these crossings be notified that the maximum design outflows from Highfield Dam will increase with implementation of the proposed spillway upgrades.

Tailwater rating data in Rush Lake Creek and Herbert Main canal were not available for use in the present study. However, an estimate of the tailwater rating curve was developed by NHC using an assumed average valley slope of between 0.0007 m/m and 0.0011 m/m, and assumed range for floodplain and channel Manning's n roughness coefficients of 0.045 to 0.06 and 0.04 to 0.05, respectively. For a peak spillway outflow of200 m^3/s, the tailwater was estimated to be between El. 718.0 m and 718.2 m.

Freeboard Requirements

The freeboard requirements for the current pre-design study were based on adherence to the CDA Dam Safety Guidelines (2007), which include:

---

[3]It should be noted that the low-level outlet ratings did not consider tailwater influence.

- no overtopping by 95% of the waves caused by the most critical wind with a frequency of 1:1000 year when the reservoir is at its maximum normal elevation.
- No overtopping by 95% of the waves caused by the most critical wind when the reservoir is at its maximum extreme level during passage of the IDF. The most critical wind was based on the dam's 'High' consequence classification, and was a wind with Annual Exceedance Probability (AEP) of 1/2.

Freeboard calculations were provided in the previous pre-design report by AAFC-AESB (July 2009). NHC reviewed these calculations using the CDA Dam Safety Guidelines (2007) and the wind speed, fetch and reservoir depth data provided in the previous pre-design report, which are summarized in Table 6.2 along with the resulting freeboard requirements.

Table 3.2 Freeboard Requirements

	Wind Speed	Effective Fetch Length	Average Reservoir Depth	Structure Slope	Wind Setup	Wave Runup	Allowance for Settlement	Minimum Freeboard
	(km/h)	(km)	(m)	H:1V	(m)	(m)	(m)	(m)
Normal Freeboard at FSL	140	1.7	3.7	3	0.143	1.44	0.1	1.7
Minimum Freeboard at IDF	79	1.7	6.8	3	0.025	0.58	0.1	0.7

The 2009 pre-design study completed by AESB used the OSDF (1:200 year flood event) freeboard requirement of 2.0 m as the governing condition for all three alternatives evaluated in that study. However, the OSDF freeboard is not a freeboard requirement included in the 2007 CDA Dam Safety Guidelines, so it was not used in the current evaluation. In addition, the resulting normal freeboard requirement at FSL as determined in the current study is approximately 1 m lower than previously estimated. The PFRA Dam Safety Guidelines (1987) had also governed the freeboard criteria at FSL.

This site was not considered to be at risk of settlement of dam crest due to earthquake, nor water level rise due to displacement caused by land slide. However, during the final design phase for the project, an allowance for potential settlement due to consolidation of foundation and embankment materials and degradation of the core due to frost action, weathering, and incidental loading such as traffic should be evaluated and accounted for in establishing the top of the dam crest. Settlement has been estimated herein as approximately 0.1 m.

Flood Routing

The flooded area and reservoir capacity curves have been updated since the July 2009 Predesign Report. The updated curves, developed by AESB in November 2010, were based on new LiDAR survey data collected in 2010 and are included in Appendix A of this report. These curves were used in the flood routing analysis, in combination with the inflow hydrographs from the November 2007 flood frequency analyses for Highfield Dam and the rating curves developed for the six spillway design alternatives, to develop spillway outflows for each alternative.

Velocity Threshold

During the design process, a limiting velocity criterion of 2 m/s was established for earth-cut spillway channels. This limiting velocity was considered to be the threshold velocity beyond which there would be potential for significant erosion within the channel, based on limiting velocities for plain grass (Hewlett, 1985).This criterion was utilized in developing the conceptual designs of the earth-cut channels for the pre-design alternatives.

3.3 GEOTECHNICAL DESIGN CRITERIA

MDH completed the geotechnical aspects of the spillway pre-design, including a stability analysis of the dam and the spillway design alternatives. Details of this work are presented in the "Spillway Upgrade at Highfield Dam Draft Report" (MDH, March 2011), which is included in Appendix B of this report.

Upon reviewing the existing geotechnical data for the Highfield Dam site it was determined by MDH that there were insufficient data to conduct reliable slope stability analyses of the dam, and it was recommended that additional geotechnical investigation be conducted. The scope of the additional geotechnical field work and laboratory testing consisted of:

- Drilling three deep stratigraphic boreholes for lithology and obtaining in situ soil samples for laboratory testing through the downstream face of the dam;
- Drilling two shallow boreholes within the existing earth spillway at the west end of the embankment;
- Installing three slope inclinometers within the dam boreholes to monitor displacement;
- Installing multiple stacked vibrating wire piezometers to monitor pore water pressures and the pressure gradient at each of the three dam boreholes;
- Performing laboratory testing on soil samples collected during the drilling program to obtain the material properties and geotechnical parameters required for the slope stability analyses, including at least one direct shear test;
- Obtaining base line readings for the slope inclinometers for future slope monitoring; and
- Obtaining initial pore water pressure readings from the vibrating wire piezometers to obtain porewater pressure profiles in the existing dam and foundation soils.

Prior to completing the geotechnical field work and slope stability analyses, preliminary assumptions for slope stability were required such that conceptual designs for the six alternatives could be developed. These assumptions included:

- The stable slope for the downstream face of the embankment was assumed to be 1V:8H for all alternatives. This was considered to be a worst-case scenario, and therefore a conservative assumption.
- The stable side slope for earth-cut channels was assumed to be 1V:5H based on preliminary geotechnical findings which showed the presence of clay shale soils.

Refinements to these criteria were made once the field work and laboratory analyses were completed. The refined criteria were then implemented in the pre-design for the selected alternative as described in Section 6 of this report.

3.4 ENVIRONMENTAL CONSIDERATIONS

Environmental Assessment

The Rare Plant, Wildlife, Fish and Habitat Assessment Final Report, which was issued in December 2010 (KGS Group, 2010), summarizes the biological assessments completed at the Highfield Dam site, identifies potential adverse environmental impacts associated with the proposed spillway upgrades, and recommends applicable mitigation measures. Project activities that were considered most likely to result in measurable disturbances include enlarging the dam footprint in order to raise the top of dam elevation and revise its downstream slope; clearing and trenching associated with development of a new spillway channel; and, clearing and excavation of a borrow pit area.

In addition to the potential environment effects typical with earth work projects of this type, additional key findings and recommendations from the KGS report include the following:

Vegetation

Five provincially “extremely rare” to “rare-uncommon” plant species were identified within the project area, but none are federally protected or considered species at risk in Saskatchewan, nor species with activity restrictions imposed by Saskatchewan Ministry of Environment. However, it was advised that the Saskatchewan Conservation Data Centre (SKCDC) be informed of the presence of these five plant species and that recommended set back restrictions are applied.

Birds

The chestnut collared longspur, which is listed as threatened by the Committee on the Status of Endangered Wildlife in Canada (COSEWIC),was observed nesting in the grassland fragments. Mitigation measures include ensuring that a portion of the grassland fragments are preserved and developing an appropriate rehabilitation plan for this section of the site that includes re-vegetation using a native plant seed mix. Other birds observed within the project area included loggerhead shrike, great blue heron, ferruginous hawks, colonial nesting birds, and American

bittern, but it was considered unlikely the SKCDC activity restrictions and restrictions dates will be required for this project, unless a nest is encountered.

Amphibians

The proposed project, especially enlarging the embankment, will extend into a portion of the habitat used by the northern leopard frog, which is listed as provincially "rare-uncommon" species and under COSEWIC a species of special concern. The quantity of habitat that would be impacted by the project was considered to be negligible compared to that available in the immediate surrounding area; however, adherence to appropriate setback distances for activity and restriction dates will be required.

Fish

No fish species of conservation concern were found within the proposed project area. However, it was recommended to limit construction activity when possible, in areas with higher valued fish habitat, and adhere to the Department of Fisheries and Ocean Canada (DFO) in-water work restrictions, which would be from April 1 to May 31 at the Highfield Dam (Southern Saskatchewan) based on the presence of spring spawning fish.

The Rare Plant, Wildlife, Fish and Habitat Assessment Final Report (KGS Group, 2010) should be referenced for detailed recommendations and applicable mitigation measures prior to any construction at the dam site.

4 DEVELOPMENT OF DESIGN ALTERNATIVES

4.1 REVIEW OF ORIGINAL THREE DESIGN ALTERNATIVES

The three design alternatives developed by AESB during the previous pre-design work (July 2009) were evaluated as part of the current study, and modifications were incorporated into all three alternatives, as described below. These alternatives, referred to as Alternatives 1 through 3, all included a new operating spillway located on the east abutment, an access bridge over the spillway channel, and plugging and abandoning the existing earth spillway. Updated conceptual design drawings for Alternatives 1, 2 and 3 are included as Figures 2 through 7.

Alternative 1 Labyrinth Weir –East *(Figures 2 and 3)*

Alternative 1 is comprised of a 500 m long earth-cut inlet channel leading to an ungated labyrinth weir that discharges into a concrete chute and stilling basin. Review of this design resulted in the following modifications to the inlet channel, labyrinth weir and the spillway chute:

- A berm at El. 723.3 m was added at the entrance to the inlet channel to restrict ponding and allow for vegetation to establish in the channel by keeping the channel dry until the reservoir level rises at least 0.3 m above the FSL. Vegetation such as a dense grass cover will improve the channel's resistance to erosion.
- The inlet channel side slopes were decreased from 1V:3H to 1V:5H based on preliminary geotechnical information.
- The floor of the inlet channel was raised from El. 721.5 m to 723.0 m to reduce excavation volumes.
- An additional cycle was added to the labyrinth weir to increase its capacity.
- The magnification ratio (length/width) of the labyrinth weir was increased from 1.85 to 5 to improve its efficiency and adhere to recommended design ratios (Falvey, 2003).
- The spillway width at the weir location was increased from 13 m to 20 m to accommodate the additional weir cycle and increased magnification ratio.
- The spillway chute was modified to expand from 20 m at the labyrinth weir to 25 m at the stilling basin to reduce flow velocities entering the stilling basin and maintain the hydraulic jump within the basin.
- The required top of dam (T.O.D.), based on flood routing and freeboard requirements, was increased from El. 726.5 m to El. 727.1m.

Alternative 2 Gated Spillway – East *(Figures 4 and 5)*

The Alternative 2 concept has many similarities to Alternative 1, except that the labyrinth weir is replaced with a gated control structure. Again, the Alternative 2 spillway would be located at the downstream end of a 500 m long earth-cut inlet channel on the east abutment and would discharge

into a concrete chute spillway leading to a concrete stilling basin. Review of this design resulted in the following modifications:

- A berm at El. 723.3 m was added at the entrance to the inlet channel to restrict ponding and allow for vegetation to establish in the channel by keeping the channel dry until the reservoir level rises at least 0.3 m above the FSL. Vegetation such as a dense grass cover will improve the channel's resistance to erosion.
- The inlet channel side slopes were decreased from 1V:3H to 1V:5H based on preliminary geotechnical information.
- The floor of the inlet channel was raised from El. 721.5 m to 723.0 m to reduce excavation volumes.
- The two 6 m wide x 2 m high vertical slide gates were replaced by 6 m wide x 1.5 m high radial gates (radial gates are considered less problematic during general operation and in freezing temperatures).
- The gate sill elevation was retained at El. 722.0 m; however, the revised design incorporates a concrete transition section from the inlet channel at El. 723.0 m to the horizontal gated section at El. 722.0 m.
- The concrete spillway chute was modified to expand from 13 m at the location of the gates to 25 m at the stilling basin to reduce flow velocities entering the stilling basin and maintain the hydraulic jump within the basin.
- The required T.O.D. elevation based on flood routing and freeboard requirements was increased from El. 726.2 m to El. 727.2 m.

Alternative 3 Earth Spillway – East *(Figures 6 and 7)*

Alternative 3 consists of a new operating earth spillway located on the east abutment. Review of this design resulted in the following revisions to the design:

- A berm at El. 723.3 m was added at the entrance to the spillway channel to restrict ponding and allow for vegetation to establish in the channel by keeping the channel dry until the reservoir level rises at least 0.3 m above the FSL. Vegetation such as a dense grass cover will improve the channel's resistance to erosion.
- The earth-cut side slopes were decreased from 1V:3H to 1V:5H based on preliminary geotechnical information.
- The spillway channel invert was modified from the previous invert at El. 724.5 m to a gradual (0.25%) sloping floor from El. 723.3 m at the entrance berm to El. 722.3 m at the energy dissipator structure.
- The rip rap apron at the downstream end of the spillway was replaced with a concrete stepped energy dissipator to improve reliability.

- The required T.O.D. elevation based on flood routing and freeboard requirements was reduced from El. 727.2 m to 725.7 m.

4.2 DEVELOPMENT OF ADDITIONAL ALTERNATIVES

NHC developed three design alternatives in addition to those described in the 2009 AESBP redesign study. Initially, a wide range of alternatives were considered for implementation at the Highfield Dam project, including:

- Modification of existing earth spillway on west side of the embankment
- Broad crested overflow spillway bridging Herbert Main Canal
- Drop inlet spillway at either the east or west end of the embankment
- Fuse gate or fuse plug spillways at either the east or west end of the embankment
- Stepped spillway incorporated into the crest of the existing embankment
- Labyrinth weir or gated spillway on the west side of embankment
- Rubber weir or Obermeyer spillway at either the east or west end of the embankment
- Siphon spillway at either the east or west end of the embankment

Preliminary screening of these alternatives was conducted jointly between AESB, NHC and MDH, and took into consideration hydraulic capacity, constructability, operation and maintenance, lifecycle costs and environmental impacts. Based on this preliminary screening, three alternative designs, referred to as Alternatives 4 through 6, were selected and conceptual level design details were developed for the alternatives.

To eliminate the need for an access bridge, as required for the east side alternatives, it was agreed that the additional alternatives would each be located on the west side of the embankment. In addition, the west side alternatives do not encroach on land outside of the boundary of land controlled by Canada, so associated costs with negotiating new land contracts would be eliminated. However, a significant limitation for the west side alternatives was the limited capacity of the Herbert Main Canal. Since the new spillway outflows could not be accommodated within the Herbert Main Canal, a concrete box culvert was used to convey the canal flows beneath the spillway channels that “bridge” the canal and direct spillway outflows onto the floodplain below the dam. Figures 8 through 13 include the preliminary conceptual design drawings for the three additional alternatives.

Alternative 4 Labyrinth Weir – West *(Figures 8 and 9)*

Alternative 4 consists of a new labyrinth weir spillway located on the west abutment leading to a shallow high-velocity concrete chute that bridges the Herbert Main Canal and terminates with a concrete stepped energy dissipator. Key features include:

- Earth-cut inlet channel and invert of concrete spillway set at El. 721.55 m

- Curved entrance walls leading to a 25 m wide, three-cycle labyrinth weir section located near the axis of the embankment dam with a weir crest elevation set at El. 723.0 m
- A 25 m wide by 180 m long curving concrete chute leading to a concrete stepped energy dissipator (1m high by 3 m long steps).
- Concrete box culvert (36 m long by 1 m high by 8 m wide) used to convey Herbert Main Canal flows beneath the spillway chute.
- Maintaining the existing earth spillway as an auxiliary spillway by installing a concrete box culvert (40 m long by 2m high by 8 m wide) to convey the Herbert Main Canal flows beneath a concrete chute and a stepped energy dissipator (similar to energy dissipator proposed for the labyrinth weir spillway).
- The required T.O.D. elevation based on flood routing and freeboard requirements is El. 725.2 m.

The weir for Alternative 4 is located up near the axis of the embankment to eliminate the need for a long inlet channel, such as the one used for Alternatives 1 and 2. In addition, the relatively low topography between the existing earth spillway and the west end of the embankment allows for the design of a relatively shallow high velocity chute downstream of the weir compared to that required for the east side alternatives.

Alternative 5 Gated Spillway – West *(Figures 10 and 11)*

Alternative 5 is similar to Alternative 4, but the labyrinth weir spillway is replaced with a gated control structure. Key features include:

- Earth-cut inlet channel excavated to El. 721.55 m
- Curved entrance walls leading to a 25 m wide, three-gate control weir located near the axis of the embankment dam. Each gate is 7.5 m wide by 1.5 high and the gate sill is set at El. 722.0 m.
- A 25 m wide by 180 m long curving concrete chute leading to a concrete stepped energy dissipator (1m high by 3 m long steps).
- Concrete box culvert (36 m long by 1 m high by 8 m wide) used to convey the Herbert Main Canal flows beneath the spillway chute.
- The existing earth spillway is maintained as an auxiliary spillway, as described for Alternative 4.
- The required T.O.D. elevation based on flood routing and freeboard requirements is El. 725.5 m.

The primary advantage of using a gated control weir is that a lower sill elevation can be achieved which allows the reservoir to be drawn down at the beginning of a flood event thereby reducing the reservoir surcharge during a flood event. In addition, when not in use, the gates allow additional ponding of water up to the top of the gates (El. 723.5 m).

Alternative 6 Earth Spillway – West *(Figures 12 and 13)*

Alternative 6 is comprised of enlarging the existing earth spillway on the west abutment to accommodate the IDF discharge. Key features of Alternative 6 include:

- A 100 m wide earth-cut spillway channel with 1V:5H side slopes following the same alignment as the existing earth spillway.
- A berm at El. 723.3 m installed at the entrance to the enlarged earth-cut channel to restrict ponding and allow for vegetation to establish in the channel by keeping the channel dry until the reservoir level rises at least 0.3 m above the FSL. Vegetation such as a dense grass cover will improve the channel's resistance to erosion.
- The spillway channel invert was modified from the existing invert at El. 723.0 m to a gradual (0.25%) sloping invert from El. 723.3 m at the entrance berm to El. 722.3 m where the spillway channel bridges the Herbert Main Canal.
- A concrete box culvert (190 m long by 2 m high by 8 m wide) installed along the base of the spillway to convey Herbert Main Canal flows beneath the spillway.
- A concrete slab bridging the Herbert Main Canal and a stepped energy dissipator (1m high by 3m long steps) used as the terminal structure for the spillway.
- The required T.O.D. elevation based on flood routing and freeboard requirements is El. 725.5 m.

Alternative 6 makes use of the existing earth spillway in order to reduce excavation requirements; however, the capacity limitations for the Herbert Main Canal make it necessary to bridge the canal.

5 EVALUATION AND COMPARISON OF ALTERNATIVES

5.1 FLOOD ROUTING RESULTS

A flood routing model was created by NHC using STELLA software (Version 4.4) to determine the maximum reservoir elevation and the resulting spillway outflow during passage of the 1:200 year (OSDF) and the 1:1000 year (IDF) rainfall events. Inputs to the model included the reservoir capacity curves (Nov 2010), the inflow hydrographs (from the November 2007 Highfield Dam flood frequency analyses), and spillway rating curves developed by NHC for each of the six design alternatives[4].

The reservoir level at the initiation of the flood routing analysis was assumed to be El. 723.0 m, and it was assumed that the east and west low level outlets were closed for the duration of the event[5]. It also assumed that for the two gated spillway alternatives (Alternatives 2 and 5) the spillway gates were fully opened at the beginning of the event, and hence no 'control' was provided by the gate itself in the analysis. Table 5.1 provides a summary of the flood routing results.

Table 5.1 Flood Routing Results – Alternatives 1 to 6

Alternative	OSDF (1:200 year)		IDF (1:1000 year)	
	Maximum Outflow (m^3/s)	Max Reservoir Level (m)	Maximum Outflow (m^3/s)	Max Reservoir Level (m)
1	44	725.1	154	726.4
2	43	725.1	146	726.5
3	115	724.4	267	725.0
4	118	723.7	209	724.5
5	94	723.8	182	724.8
6	99	724.2	257	724.8

[4]The ratings curves for the spillway channels were developed using the U.S. Army Corps of Engineers Hydrologic Engineering Center River Analysis System (HEC-RAS) software (Version 4.1).

[5]It is advisable to close the outlets during large flood events, since there is a potential for the outlets to become damaged or plugged. It is also considered a conservative assumption to close the outlets for the flood routing analysis, as this would produce a worst-case for reservoir surcharge.

The highest reservoir levels were produced by Alternatives 1 and 2, which in turn required the largest increase in the top of dam elevation to satisfy the freeboard requirements. The largest peak outflows resulting from the IDF occurs for Alternatives 3 and 6 - the two earth spillway alternatives. These maximum outflows were 267m^3/s and 257 m^3/s, respectively, while the peak outflows for the other alternatives ranged from 146 m^3/s to 209 m^3/s.

5.2 COST ANALYSIS SUMMARY

Conceptual level cost estimates were developed for all six alternatives. Table 5.2 shows a summary of the capital costs estimated for each of the six alternatives. These estimates included all significant components, including labour, materials, equipment, engineering, construction management and contractor's overhead and profit. Unit prices were based on AESB experience from work recently completed at Duncairn and Junction Dams as well as NHC's and MDH's experience with similar construction projects, the Saskatchewan Ministry of Highways and Infrastructure (SMHI) 2011 bid price trends, and commercially available construction cost data guides.

Estimated capital costs include 7 percent for detailed design engineering, 10 percent for construction engineering and a 20 percent contingency allowance to cover details of design and uncertainty in long-term operation and maintenance costs that cannot be fully addressed at the pre-design phase of the study. Design engineering would consist of preparation of design drawings and report, tender document preparation and tendering. Construction engineering costs include, inspection and testing services, project management, contractor management, surveying for initial layout and for quantities, and final reporting (as-built) on construction activities. Estimates of the operation and maintenance costs for each alternative were also developed. These long-term (lifecycle) costs were based on AESB experience and industry standards for similar water control structures. The net present value assumed a 5 percent discount rate over 100 years.

For the preliminary cost estimates, it was assumed that all concrete structures will be reinforced and formed in-place. It was also assumed that the unit cost for excavation includes material disposal on the embankment or spoil pile, and an assumed 1 km haul distance to a borrow source when the quantity of excavation is less than the required quantity of embankment material. The analysis did not consider whether the excavated material would be suitable for use in the embankment.

The concrete quantities had the most significant impact on the construction costs. The least expensive alternative was Alternative 3 (earth spillway on the east abutment), followed by Alternative 2 (labyrinth weir on the east abutment). Alternatives 4 through 6 included the highest volumes of concrete, because of the structures required to bridge the Herbert Main Canal, and were therefore the highest in terms of capital costs. The most costly alternative, Alternative 6 (earth spillway on the west abutment), significantly exceeded the costs of the other five alternatives, because of the large volume of concrete used for the box culvert, the slab bridging over the culvert and the stepped energy dissipator at the downstream end of the spillway.

Table 5.2 summarizes the total pre-design capital costs, maintenance costs, operating costs and net present value estimated for each alternative developed for use in the MCA (described in Section 5.3). The highest maintenance costs were associated with Alternative 6 due to the large concrete structures and long culvert at its downstream end; while the highest operating costs were associated with the two gated alternatives (Alternative 2 and 5). Alternative 3 resulted in the lowest costs for all categories.

Table 5.2 Summary of the Pre-Design Cost Estimates

Alternative	Capital Cost	Annual Maintenance Cost	Annual Operating Cost	Net Present Value of all Costs
1 - Labyrinth Weir - East side	$14,619,586	$220,166	$15,000	$18,641,340
2 - Gated Spillway - East side	$15,736,246	$256,688	$25,000	$21,378,086
3 - Earth Spillway - East side	$14,581,747	$226,658	$5,000	$18,741,174
4 - Labyrinth Weir - West Side	$16,822,203	$290,646	$10,000	$23,390,149
5 - Gated Concrete Spillway - West Side	$17,671,603	$334,382	$20,000	$25,740,638
6 - Earth Spillway - West Side	$21,383,207	$467,255	$5,000	$36,106,792

Notes:
Estimated capital costs include 7% for detailed design engineering, 10% for construction engineering and 20% contingency
Net present value assumes 5% discount rate over 100 years

Details of the geotechnical assessment and the cost estimates can be referenced in the Spillway Upgrade at Highfield Dam Final Report (MDH, November 2011) which is included as Appendix B of this report.

The above cost estimates reflect the updated costs, which include the revised downstream embankment slope of 1V:6H (previously set at 1V:8H), the use of reinforced concrete for key structures (in place of roller compacted concrete) and additional rip rap required on the upstream face of the embankment. The initial MCA was conducted using preliminary costs prior to incorporating these revisions; however, the ranking of the alternatives, from lowest to highest cost, remained unchanged.

5.3 MULTI-CRITERIA ANALYSIS

Comparison of the six spillway alternatives was based on a multi-criteria analysis (MCA), wherein the alternatives were scored according to a set of technical, environmental and economic objectives, and different weightings were assigned to each class of objectives. This method provided a systematic approach for comparing the alternatives and identifying a preferred alternative. It allowed for both technical and non-technical criteria to be included in the analysis.

The "scores" related to how well each design would perform for a given objective (that is, the highest score was assigned to the design alternative that best met the stated objective), while the "weights" were indicative of the relative importance of each objective (for example, objectives related to dam safety were given higher weighting than objectives related to environment). Multiplying the objective scores by the corresponding objective weights provided a weighted score for each objective. Then adding the weighted scores together for all objectives provided a total score for each alternative. Finally, comparing the total scores for each alternative, the alternatives were ranked from the preferred alternative to the least desirable alternative.

The MCA objectives and the weights were defined and agreed upon at the onset of the project in order to clearly identify the key project objectives prior to considering specific design concepts. Table 5.3 presents the main objectives and associated weighting values that were adopted for the Highfield MCA. The MCA was divided into five main objectives: technical, economic, environmental, transfer potential and secondary benefits. The technical objective was further subdivided into the following categories:

- Constructability – including material familiarity, time to construct, care of water, and land control required for construction and for operation;
- Reliability of performance;
- Resilience; and,
- Operator intervention – including availability of operator access and whether maintenance could be done in house.

Within the economic objective, construction costs were given one-half of the marks, and maintenance and operation costs were each weighted with one-quarter of the marks. The marks within the environmental objective were divided equally between acute impacts (during construction) and long-term impacts (post-construction).The transfer potential category considered operation and maintenance requirements along with environmental impacts due to operations (this objective was included to account for potential value in the event that the dam was transferred to a provincial entity or other agency in the future). Finally, the secondary benefits objective takes into consideration any additional opportunities that may come from installing a particular alternative such as increased storage for irrigation.

Table 5.3 Summary of Primary Multi-Criteria Analysis Objectives and Weighting

Objective	Weighting
Technical	30%
Economic	50%
Environmental	5%
Transfer Potential	10%
Secondary Benefits	5%
Evaluation Total	100%

In addition to the above MCA objectives, the following two objectives were defined as "requirements" for all alternatives:

- The alternative must pass the IDF with the CDA guideline freeboard (0.7 m below T.O.D.)
- The alternative must allow for operation of both gated low level outlets during passage of the flood by either providing access from the east side or automation

The initial MCA was completed separately by personnel at NHC, MDH and AESB. This provided an opportunity for all parties to contribute to the analysis and ensure that the outcome of the analysis was technically sound, unbiased and accepted by the project stakeholders. As noted previously, the initial MCA was conducted using the preliminary cost estimates, which did not account for the revised downstream embankment slope of 1V:6H, the use of reinforced concrete for key structures and additional rip rap required on the upstream face of the embankment. However, the order of the alternatives ranked from lowest to highest cost was unaffected by the updated cost estimates.

In comparing the rankings performed by the three project teams, the main differences were:

- NHC rated Alternatives 4 and 5 on the west abutment higher than either AESB or MDH
- AESB gave lower scores to the two gated options (Alternatives 2 and 5) than either NHC or MDH
- MDH's rankings demonstrated some preference for the east side alternatives by giving higher marks to Alternatives 1 and 2 and lower marks to Alternatives 4 and 6 than AESB or NHC

Despite these differences, the MCA results from all three organizations resulted in Alternative 3 as the preferred alternative. The outcome of the three MCA ratings was averaged and is presented in Table 5.4.

Table 5.4 Summary of Multi-Criteria Analysis Results

ALTERNATIVE	MULTI-CRITERIA SCORES					
	Technical	Economic	Environment	Transfer Potential	Secondary Benefits	OVERALL
1 - Labyrinth Weir - East side	74.7	83.5	75.0	61.6	50.0	76.6
2 - Gated Spillway - East side	51.3	66.8	65.0	45.1	100.0	61.5
3 - Earth Spillway - East side	79.2	99.8	65.0	91.4	50.0	**88.5**
4 - Labyrinth Weir - West Side	77.9	72.7	85.0	62.3	50.0	72.7
5 - Gated Spillway - West Side	53.1	62.1	70.0	41.0	50.0	57.1
6 - Earth Spillway - West Side	85.7	67.6	80.0	77.6	50.0	73.8
Weighting	30	50	5	10	5	

On completion of the MCA, a sensitivity analysis of the MCA process was performed. For example, it was investigated whether the elimination of the existing spillway from Alternatives 4 and 5 would have a significant enough impact on costs to change the outcome of the MCA. By not using the existing spillway as an auxiliary spillway, this would eliminate the need for the concrete structures at the downstream end of the spillway required for bridging the canal. This would result in a cost savings of approximately $ 2,500,000, and additional flood routing calculations for Alternatives 4 and 5 were completed to verify that additional dam height would not be required with the existing spillway plugged. However, the estimated savings in construction costs were not sufficient to affect the outcome of the MCA.

In reviewing the results of the MCA, it was noted that the operating costs may dominate the outcome for the economic component of the MCA and that costs may be quite sensitive to the accuracy of the operating costs. However, AESB indicated that the weighting of the operating and maintenance costs was considered appropriate in this case.

It was also noted that by providing a “perfect score” of 100 to the alternatives with the lowest cost and the highest secondary benefits, but not assigning perfect scores for any of the other objectives, the *effective weighting* of the economic and secondary benefits objectives are higher than indicated by the values of 50 and 5 used in the MCA. To check whether this could affect the outcome, the MCA results were further analyzed by assigning a “perfect score” of 100 to the alternatives that ranked best within the technical, environmental and transfer potential objectives, and pro-rating the remaining alternatives using the MCA scores outlined in Table 5.5. The additional analysis still resulted in Alternative 3 as the preferred alternative. On the basis of these results, Alternative 3 was recommended as the preferred alternative and was accepted by AESB as the selected alternative (via email Feb 27, 2011).

6 PRE-DESIGN FOR SELECTED ALTERNATIVE

6.1 PRE-DESIGN FOR SELECTED ALTERNATIVE

Once the selection of the earth spillway concept on the east abutment (Alternative 3) was approved by AESB, preliminary design details were developed. This included verifying and refining key dimensions for the spillway channel and energy dissipator and developing methods for implementing the embankment raise and extending the low-level outlets. The conceptual design drawings were updated and advanced to the pre-design level. These are included as Figures 14 to 16. The key features of the selected alternative include:

Earth-cut Spillway Channel and Energy Dissipator

- A 100 m wide by 570 m long earth-cut spillway channel.
- A berm with a top elevation of 723.3 m installed at the entrance to the earth-cut spillway channel to prevent ponding and allow for growth and maintenance of a dense grass cover within the channel by keeping it dry until the reservoir level rises at least 0.3 m above the FSL. It is still recommended that AESB maintain storage levels in the reservoir at or below FSL during normal operation to allow for sufficient freeboard and minimize the risks of damage due to wave action on the embankment. This design would not adversely impact current operations or frequency of operator visits. If a reduction in operator interference is desired, a method for automatically maintaining the FSL up to a threshold discharge could be explored during the final design phase.
- The spillway channel has a gradually sloping (0.25% along the centerline) channel invert from El. 723.3 m at the entrance berm to El. 722.3 m at the top of the concrete stepped energy dissipator.
- Side slopes of 1V:5H are recommended in the earth-cut spillway channel to ensure long term stability of the slopes within the clay shale soils.
- A stepped energy dissipator constructed of reinforced concrete supported by piles with 1 m high by 3 m long steps is located at the downstream end of the earth-cut channel.
- Vertical abutment walls flank the entrance to the energy dissipator, transitioning from the 1V:5H earth-cut side slopes to the vertical concrete walls at the energy dissipator structure. The top of wall elevation will be El. 725.7 m (until downstream of the first step) and it is anticipated that 0.46 m thick reinforced concrete walls will be used.
- For the pre-design, it was assumed that all concrete structures including the abutment walls will be supported by 0.4 m diameter piles spaced at 3.5 m and extending down to a minimum depth of 7.6 m.
- Rip rap will be placed downstream from the stepped energy dissipator as a horizontal apron at El. 715.5 m. It is anticipated that a minimum thickness of 1.2 m of 0.9 m diameter rip rap (Class V as described in Section 6.2 of this report) will be used, and that a 0.4 m granular

filter will underlie the rip rap. The apron will extend across the full width of the energy dissipator.

- An earth berm on the west side of the Rush Lake Creek channel will be required to direct spillway outflows away from the toe of the dam. As shown in Figure 14, the top of the erosion protection berm is set at El. 720.0 m with 1V:5H side slopes and protected by a 0.6 m thick layer of 0.4 m diameter rip rap (Class II). It is anticipated that spoil from the spillway excavation could be used to construct this berm.
- An access bridge will be installed over the earth-cut spillway channel in line with the dam centreline. It is anticipated that 25 m long spans will be used and piers will be required at each embankment toe, across the spillway channel and up the side slopes, as necessary. Rock protection at the base of the piers may be required to prevent local scour. The inclusion of the bridge was based on criteria set at the onset of the project. Due to its high cost, alternative access should be reviewed during the final design phase.
- The existing earth spillway on the west abutment should be plugged and rendered inoperable to restrict flows from damaging the Herbert Main Canal.

Embankment Upgrades

- The embankment dam will be raised by 0.9 m to El. 725.7 m to provide sufficient freeboard at the IDF (0.7 m of freeboard).
- The upstream face of the embankment will extend up on a 1V:3H slope and the downstream face of the embankment will be constructed at a 1V:6H slope as recommended by the geotechnical analysis.
- Rip rap will be required to protect the upstream face of the raised embankment from wave action. AESB has indicated that the existing upstream slope protection (0.3 m diameter rip rap over 0.3 m of bedding gravel) may be undersized. Over-steepening and exposed bedding gravel have been noted at various spots along the embankment down to approximately 1.4 m below FSL. The final design work should include a more detailed review of the gradation of the existing and proposed upstream slope protection. The pre-design uses a 0.6 m thick layer of 0.4 m diameter rip rap (Class II) as described in Section 6.2 of this report over 0.4 m of bedding gravel. The minimum operating level has been estimated at El. 721.5 m, so the bottom of the rip rap protection has been set at El. 721.2 m to account for wave rundown. Regular maintenance should include monitoring and repairs to exposed bedding gravel areas as needed.

Low Level Outlets

- The structural and geotechnical integrity of the existing outlet conduits should be evaluated in the final design phase to ensure that the low level outlets can support the additional load from the raised embankment. Boreholes will be required along the route of the outlets to conduct consolidation testing to establish the settlement magnitude, the differential settlement potential, and for strength testing.

- For the pre-design, it has been assumed that the existing low level outlets will be adequate to handle the additional embankment load that the dam raise will impose. The costs of cursory structural foundation evaluation were not included in the cost estimate. Figure 16 shows a section through both the east and west outlets.
- It is expected that the downstream section of the west outlet will need to be extended by approximately 30 m to accommodate the raised embankment alignment, while the length of the existing east outlet is considered adequate.[6]
- The west outlet would need to be terminated with a new stilling basin to for energy dissipation. The geotechnical integrity of the extended outlet should also be evaluated during final design.[7]
- For the purposes of the pre-design cost estimate, the west outlet extension has been estimated to be similar to the existing 1.5 m x 1.5 m, by 0.3 m thick reinforced concrete conduit. It has been assumed that the stilling basin will also be of reinforced concrete construction with a design similar to the existing basin, and would be supported on piles. The possibility of a simpler and less expensive impact basin could be considered for the west outlet, since its flow capacity is limited to 6 m^3/s (to accommodate downstream restrictions).
- The pre-design cost estimate assumes that the existing gates and hoisting systems would not need to be replaced or upgraded.

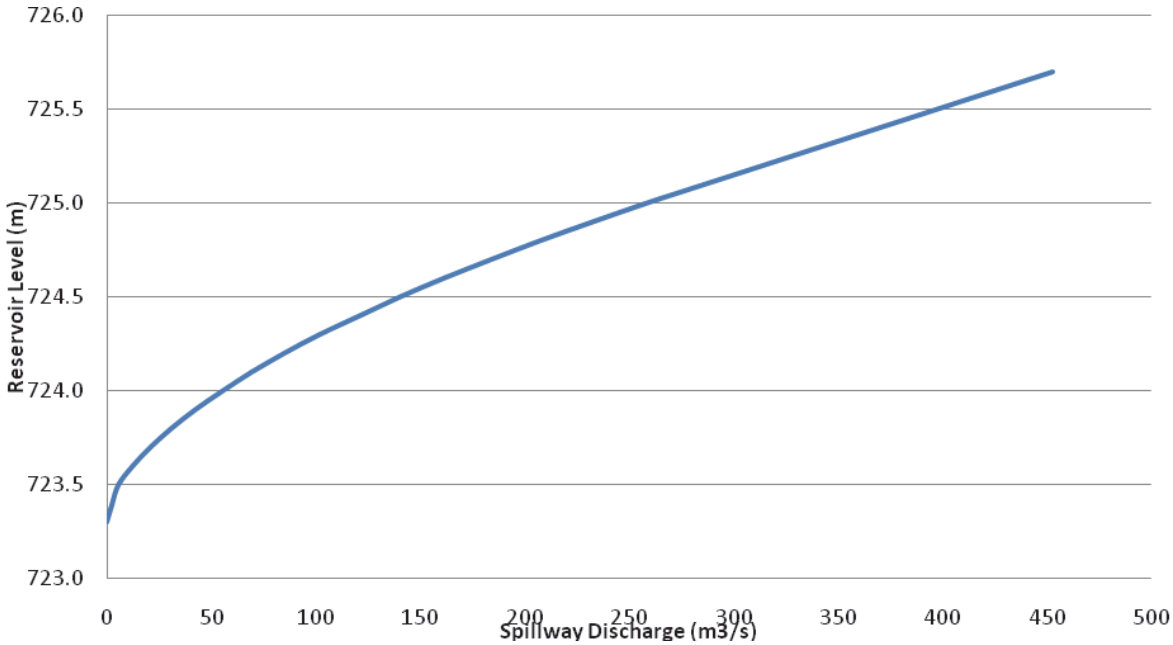
Flood Routing

Flood routing analysis was conducted for the refined design to confirm the maximum reservoir elevation and the resulting spillway outflow during passage of the 1:200 year (OSDF) and 1:1000 year (IDF) rainfall events. Inputs to the model included the reservoir capacity curves (Nov 2010), the inflow hydrographs and spillway rating curves developed by NHC. The rating curve for the proposed earth spillway on the east abutment was developed using HEC-RAS (Version 4.1) and is shown in Figure 17 shown below.

[6] In 1994, the east outlet's reinforced concrete conduit was extended 14 m using 1600 mm diameter corrugated steel pipe (CSP) and the stilling basin was replaced.

[7] In 1969, the foundation of the downstream portion of the east outlet was found to be soft and was subsequently supported by piles.

Figure 17 Spillway Discharge Curve – Earth Spillway on East Abutment



As with the previous flood routing analyses, the reservoir level at the initiation of the flood routing analysis was assumed to be El. 723.0 m, and it was assumed that the east and west low level outlets were closed for the duration of the event.

Table 6.1 provides a summary of the flood routing results for the selected alternative. The maximum reservoir level for the IDF event is 725.0 m and the peak outflow is 267 m^3/s. For the OSDF event, the maximum reservoir level is El. 724.4 m and the peak outflow is 115 m^3/s.

Table 6.1 Flood Routing Results – Earth Spillway on East Abutment

OSDF (1:200 year)		IDF (1:1000 year)	
Maximum Outflow (m^3/s)	**Max Reservoir Level (m)**	**Maximum Outflow (m^3/s)**	**Max Reservoir Level (m)**
115	724.4	267	725.0

6.2 CONSTRUCTABILITY CONSIDERATIONS

With consideration for the objectives identified during the development of the MCA, some constructability issues were identified and are discussed below.

Slope Stability

Based on the slope stability analyses conducted by MDH, it was determined that a minimum 1V:6H downstream slope would be required to meet the CDA (2007) guideline for the embankment's overall factor of safety, even without the implementation of spillway upgrades (i.e. no embankment raise nor spillway modifications).It is also recommended that a more intense drilling investigation be conducted along the length of the dam as part of the detailed design phase to further characterize the clay shales beneath the site, specifically to confirm the presence or absence of thin bentonite layers within the shale. Finally, a sand drainage blanket combined with a chimney drain(s) is recommended for installation as part of the dam expansion in order to reduce the build-up of pore pressures within the foundation soils due to the addition of fill soil for the construction of the upgraded dam embankment.

Settlement

The upper clay shale at this site shows evidence of being disturbed or reworked by glacial action and as such is not likely in its original state of consolidation and could be subject to some settlement. Consolidation testing is recommended in the detailed design stage of the dam embankment in order to estimate the magnitude of settlement.

Foundation for Bridge Structure

The proposed design includes road access for an operator over a 4.8 m wide bridge spanning the earth-cut spillway channel. The soils on the east side uplands generally consist of clay shale and a variable mantel of clay till, while the soils on the east abutment consist of the dam fill, which is a cohesive material, underlain by alluvium and clay shale. There is heaving potential of the highly plastic clay shales present near the proposed bridge location. It is expected that drilled cast-in-place concrete could likely be installed satisfactorily at this site; however, casing may be required in the upper portion of the piles that extend through the softer alluvium or in seepage zones in order to install them successfully. Alternatively, continuous flight auger (CFA), driven timber or steel (pipe or HP) piles could be used. These options would likely be more expensive than drilled cast-in-place concrete piles. It is recommended that additional test holes be drilled at the location of the bridge abutments and piers in order to define the subsurface conditions and obtain samples for laboratory testing required to complete the detailed design of the bridge foundation.

Spillway Slopes

Based on drilling conducted by AESB previously, the spillway on the east side will be constructed mainly within glacial clay till soil in the upper portion and clay shale in the lower portion. Also, the till, which is highly plastic in some areas and zones, is underlain by clay shale. The piezometric conditions within the east back slope are not known and they would have a significant influence on the stability of the proposed spillway. Without a detailed stability analysis, slopes of 1V:5H are recommended along the spillway. A side slope of 1V:3H may be possible in the final design, as suggested by PFRA (2003), but completion of a stability analysis based on a detailed geotechnical investigation of the soils and piezometric conditions in the east back slope would be required to

confirm this design. Boreholes would be also be required to characterize the borrow potential of the soils along the spillway.

For the final design phase, boreholes will also be required in the area of the stepped energy dissipator to characterize the subgrade and confirm if problematic soils and conditions are present, and a slope stability analysis should be completed. The stepped dissipator will likely require a piled foundation to support the floor and wall system. Cantilevered retaining walls could be considered in the final design phase depending on the outcome of the geotechnical investigation. Problematic soils, such as organics, desiccated clay and clay shale should be removed a minimum of 900 mm from the underside of the concrete slab and replaced with suitable, free draining granular materials in order to reduce settlement and swelling problems. A deeper depth of subcut may be required depending on the results of the geotechnical investigation required for the stepped dissipator design. A subdrain system may also be required if seepage conditions are encountered. In addition, a cut-off wall would be required near the upstream end of the concrete to earthen spillway transition. Detailed design of any subdrain system and the cut-off wall would be completed in the final design phase.

Erosion Protection

Establishing vegetation within the earth spillway is important for providing protection against erosion. Topsoil can be placed on the slopes and along the bottom of the earthen spillway and seeded with a deep rooting grass species capable of withstanding the local climate. Additional short–term erosion protection may be required to control any significant erosion during the period after construction and before vegetation is established. Turf reinforcement mats could be installed which aid in the establishment of vegetation and provide long–term erosion protection.

Rip rap will be used to protect the upstream face of the raised embankment, downstream from the stepped energy dissipator as a horizontal apron, and to protect the erosion protection berm. A U.S. Army Corps of Engineers General Guidance for Rip Rap Gradation (USACE 1980) was used to define the rip rap classification. The two classes of riprap included in the predesign are described below:

Class II (D_{50} = 0.4 m)

- 90% of stones shall range between 0.3 to 0.6 m (45 and 225 kg)
- The 50% size of the gradation shall be 0.4 m (90 kg)
- 10% of the stones may range between 0.2 to 0.3 m (12 and 45 kg)

Class V (D_{50} = 0.9 m)

- 90% of stones shall range between 0.5 to 0.8 m (160 and 820 kg)
- The 50% size of the gradation shall be 0.9 m (340 kg)
- 10% of the stones may range between 0.2 to 0.5 m (12 and 160 kg)

Rip rap shall be durable quarried or field stone of a quality suitable to ensure permanence in the structure and in the climate in which it is to be used. The stone shall be well graded within the required limits and shall be free from cracks, seams, and other defects that would unduly increase deterioration from natural causes. The inclusion of objectionable quantities of dirt, sand, clay, or rock fines will not be permitted. Occasional pieces of stone slightly larger than the maximum weight will be permitted provided the gradation and voids are not unduly affected and that surface tolerances are met. Neither the breadth nor the thickness of any piece of stone shall be less than one third of its length. Stone shall have a specific gravity between 2.50 and 2.80.

Flow Hydraulics

Significant quantities of debris, large vegetation, and sedimentation within the spillway channel has the potential to significantly affect the spillway capacity, so removal of these items should be considered as part of the spillway maintenance program.

A 1V:3H step profile is recommended for the stepped energy dissipator. A number of scale model and prototype studies have evaluated and provided design criteria for stepped chutes with moderate slopes from 2H:1V to 4H:1V that are typical of embankment dams (Chanson 1994; Frizell 1992; Ahmann and Zapel 2000; Gonzalez and Chanson 2007; Frizell 2009). However, it is recommended that numerical or physical modelling be conducted of the proposed spillway at Highfield Dam to assess flow patterns approaching the energy dissipator and its ability to effectively dissipate the energy at its downstream end. As a result of the curved alignment, there is potential for the flow super-elevation along the north side of the spillway, which may result in increasing the unit discharge above the typical maximum design value used for stepped dissipators. If non-uniform flow is present, the crest elevation of the first step can be adjusted to re-establish acceptable unit discharges. The costs of modelling the spillway are estimated to range between $50,000 and $200,000 depending on the level of detail desired.

6.3 CONSTRUCTION SEQUENCING

The main construction activities for the project will include the following:

- Mobilization and site preparation
- Spillway channel excavation and entrance berm construction
- Energy dissipator construction (including upstream concrete walls)
- Access bridge construction
- Extension of east and west low-level outlets
- Embankment raise
- Rock protection placement on upstream face of embankment, at bridge piers and downstream of energy dissipator

Prior to construction of the operating spillway on the east abutment, procurement of additional land will be required, since the spillway limits exceed the boundary of land controlled by Canada.

It is anticipated that the majority of the work can be completed in the dry; however, it may be necessary to divert flows from the east low-level outlet and isolate Rush Lake Creek during construction of the energy dissipator and the protection berm and placement of rock apron. Similarly, it will likely be necessary to divert flows from both low-level outlets during construction of the retaining walls required to extend these outlets. Short-term erosion protection, such as turf reinforcement mats, may be required to control erosion during construction and in the period immediately following construction before vegetation is established.

The Rare Plant, Wildlife, Fish and Habitat Assessment Final Report (KGS Group, 2010) should be referenced for detailed recommendations and applicable mitigation measures prior to any construction at the dam site. This will likely include adherence to DFO in-water work restrictions and appropriate setback distances for activity and restriction dates for the northern leopard frog.

The most challenging part of the construction sequencing is considered to be raising the dam embankment. Preliminary modelling indicates that staged construction will likely be required for the embankment to ensure that pore water pressures resulting from the embankment loading through various stages of construction do not cause the existing dam stability to be compromised. Timelines for staged construction of the embankment could range from 6 months up to two construction seasons. A detailed analysis of the required staging is recommended during detailed design. Instrumentation will be required to monitor stability of the embankment, settlement, and porewater pressures during and after construction. A detailed monitoring program will need to be defined at the detailed design stage.

6.4 Updated Cost Estimates for Selected Alternative

The construction cost estimates for the selected alternative were reviewed and updated, following the same process and using the same information as described previously (Section 5.2). The updated quantities and costs estimates for the Pre-design of Alternative 3 are summarized in Table 6.2. Table 6.3 summarizes the estimated total capital cost, annual maintenance cost, annual operating cost and net present value for the proposed earth spillway.

Table 6.2 Quantities and Capital Cost Estimates Summary – Earth Spillway on East Abutment

Description	Unit	Quantity	Price	Cost
Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00
Land Acquisition/Control	Hectare	10	$1,250.00	$12,500.00
Care of Water	Lump Sum	1	$50,000.00	$50,000.00
Stripping	m^2	135 232	$3.50	$473,312.00
Earth Excavation and Embankment Construction	m^3	360 000	$5.00	$1,800,000.00
Structure Excavation	m^3	850	$22.00	$18,700.00
Topsoil, Grading and Seeding	m^2	135 100	$8.50	$1,148,350.00
Traffic Gravel	m^3	575	$10.00	$5,748.64
Granular Filter	m^3	28 000	$50.00	$1,400,000.00
Rip Rap	m^3	16 000	$110.00	$1,760,000.00
Bedding Gravel	m^3	15 000	$40.00	$600,000.00
Structure Subdrainage System	Lump Sum		$200,000.00	
Energy Dissipator				
- Walls	m^3	100	$1,200.00	$120,000.00
- Slab	m^3	1 100	$850.00	$935,000.00
- Foundation (piles)	m^3	100	$1,200.00	$120,000.00
Canal Precast Concrete Bridge	Lump Sum	1	$1,800,000.00	$1,800,000.00
West Outlet Modifications	Lump Sum	1	$150,000.00	$150,000.00
			Contract Items Sub-Total	**$10,643,611**
			Construction Engineering (10.0%)	$1,064,361
			Design Engineering (7.0%)	$745,053
			Contingency (20.0%)	$2,128,722
			Total Cost	**$14,581,747**

Table 6.3 Summary of Cost Estimates – Earth Spillway on East Abutment

Capital Cost	Annual Maintenance Cost	Annual Operating Cost	Net Present Value of All Costs
$14,581,747	$226,658	$5,000	$18,741,174

Value Engineering Assessment

Given to the relatively high costs of implementing the spillway upgrades, it is recommended that a value engineering assessment be considered to evaluate the ratio of the spillway's function to its costs. While the basic function of the spillway is to safely pass the design flood, it is possible that by relaxing some of the design criteria established at the onset of the study, significant cost savings could be achieved.

For example, design criteria established at the onset of the study included not exceeding the capacity of the Herbert Main Canal to avoid damaging the canal. As a result, it was necessary to include a costly structure at the downstream end of the west side alternatives to bridge the canal. However, infrequent damage to the canal and the long term costs of repairing the canal in the event of a large flood event may be substantially less than the overall cost associated with bridging the canal. Similarly, given the relatively high costs associated with installing a bridge over the earth-cut spillway channel, it is recommended that the acceptance standard for this aspect of the work be re-evaluated and consideration be made for other access options for dam operation and maintenance.

7 SUMMARY AND RECOMMENDATIONS

The purpose of the Highfield Dam Spillway Pre-Design Completion Project was to develop and evaluate a spillway design alternative to safely pass the project's estimated Inflow Design Flood (IDF) of 361 m^3/s. Newly available information was applied to review three previously developed alternatives as presented in the Highfield Dam Project Rehabilitation Predesign Report (July 2009), and to develop three additional design alternatives. A systematic multi-criteria analysis (MCA) was then used to compare the six alternatives and select the preferred spillway design alternative.

The selected alternative is an operating earth spillway located on the east abutment. The proposed spillway is comprised of a 570 m long by 100 m wide earth spillway with 1V:5H side slopes leading to a concrete stepped energy dissipating structure discharging into Rush Lake Creek and the floodplain area downstream of the embankment dam. A rock apron and earth berm will be installed downstream of the energy dissipator to protect the toe of the dam from erosion.

The top of embankment will be raised by 0.9 m to El. 725.7 m and the downstream slope of the dam will be revised to 1V:6H to meet CDA guidelines for stability. The existing west low-level outlet will require a 30 m extension to the downstream toe of the raised embankment and the construction of a new stilling basin. A bridge will be installed over the earth-cut spillway channel to provide operator access to the low-level outlet controls and the existing earth-cut spillway on the west abutment will be plugged and rendered inoperable.

The maximum reservoir level predicted for the selected spillway design during the IDF event is 725.0 m and the peak outflow is 267 m^3/s. For the OSDF event (200-yr flood with peak inflow of 180 m^3/s), the maximum reservoir level is El. 724.4 m and the peak outflow is 115 m^3/s.

Prior to construction of the spillway, procurement of additional land will be required, since the spillway limits exceed the boundary of land controlled by Canada. In addition, the Rare Plant, Wildlife, Fish and Habitat Assessment Final Report (KGS Group, 2010) should be referenced for detailed recommendations and applicable mitigation measures prior to any construction at the dam site.

Construction of the upgrades may take up to two construction seasons such that the dam embankment construction can be staged adequately to ensure that porewater pressures resulting from the embankment loading through various stages of construction do not cause the stability of the existing dam to be compromised. It is anticipated that all construction work can be completed in the dry, with possible diversion of the flows from the east low-level outlet for construction of the stepped energy dissipator, rock apron and earth protection berm. Staging requirements would need to be confirmed using a more detailed analysis at the final design stage. In addition, instrumentation will be required to monitor stability of the embankment, settlement and porewater pressures during and after construction.

The total capital cost, annual maintenance cost, annual operating cost and net present value for the proposed earth spillway were estimated as shown below.

Capital Cost	Annual Maintenance Cost	Annual Operating Cost	Net Present Value of All Costs
$14,581,747	$226,658	$5,000	$18,741,174

Recommendations resulting from this study include:

- The maximum recommended slope for the downstream side of the embankment dam is 1V:6H to meet the CDA Guidelines for slope stability (independent of whether or not spillway upgrades are implemented).
- Side slopes of 1V:5H are recommended in the earth-cut spillway channel to ensure long term stability of the slopes within the clay shale soils.
- Establishing vegetation in the earth spillway is important for providing protection against erosion. Additional short–term erosion protection may be required to control any significant erosion during the period shortly after construction and before vegetation is established.
- Additional drilling along the length of the dam is recommended to further characterize the clay shales beneath the site, specifically to confirm the presence or absence of thin bentonite layers within the shale.
- A sand drainage blanket combined with a chimney drain(s) is recommended for installation as part of the dam expansion in order to reduce the build-up of pore pressures within the foundation soils due to the addition of fill soil for the construction of the upgraded dam embankment.
- Consolidation testing of the dam embankment is recommended in order to estimate the magnitude of settlement.
- Additional test holes should be drilled at the location of the bridge abutments and piers in order to define the subsurface conditions and obtain samples for laboratory testing required to complete the detailed design of the bridge foundation.
- Numerical or physical modelling of the spillway is recommended to confirm the approach conditions and performance of the stepped energy dissipator.
- The authorities responsible for the Trans Canada Highway and CP Rail crossings downstream of Highfield Dam should be notified that the maximum design outflows from Highfield Dam will increase with implementation of the proposed spillway upgrades.
- A value engineering assessment is recommended to determine if the capital costs for the spillway can be reduced while preserving the basic function of the spillway to protect the dam. It is possible that by relaxing some of the design criteria established at the onset of the study, significant cost savings could be achieved.

8 REFERENCES

AAFC/AESB (July 2009). “Highfield Dam Project Rehabilitation Predesign Report”.Water Supply Infrastructure Development Innovative Water Systems and Engineering Unit. Regina, Saskatchewan.

AAFC/AESB (Nov 2007). “Highfield Dam – Updated Flood Frequency Analysis Memorandum”. Surface Water Unit Water Planning and Sourcing Division. Regina, Saskatchewan.

Ahmann, M.L., and E.T. Zapel (March 200). "Stepped Spillways - A Dissolved Gas Abatement Alternative." Proceedings of the International Workshop on Hydraulics of Stepped Spillways. Zurich, Switzerland.

Chanson, Hubert (1994). “Hydraulics of Skimming Flows over Stepped Channels and Spillways”, Journal of Hydraulic Research, Vol. 32, No. 3.

CDA (2007) “Dam Safety Guidelines” Canadian Dam Association (ISBN 978-0-7726-5802-9).

Hewlett (1985) “Dikes and Revetments Design Maintenance and Safety Assessment”, Published by CIRIA; edited by Krystian Pilarczyk, Rijkswaterstaat; Hydraulic Engineering Division, Delft, 1998, pg 291.

KGS Group (Dec 2010) “Rare Plant, Wildlife, Fish and Habitat Assessments for the Rehabilitation of the Highfield Dam Project AAFC/AESB Service Contract No. 2 Final Report” Kontzamanis, Graumann Smith Macmillan Inc. (KGS Group), Winnipeg, Manitoba.

Falvey (2003) “Hydraulic Design of Labyrinth Weirs”, Published by American Society of Civil Engineers, United States.

Frizell, K.H. (1992). “Hydraulics of Stepped Spillways for RCC Dams and Dam Rehabilitations”, Bureau of Reclamation, Hydraulics Branch Office, PAP-595.

Frizell, K.H. (2009). “Stilling Basin Performance for Stepped Spillways of Mild to Steep Slopes Type III Basins”, 33rd IAHR Congress.

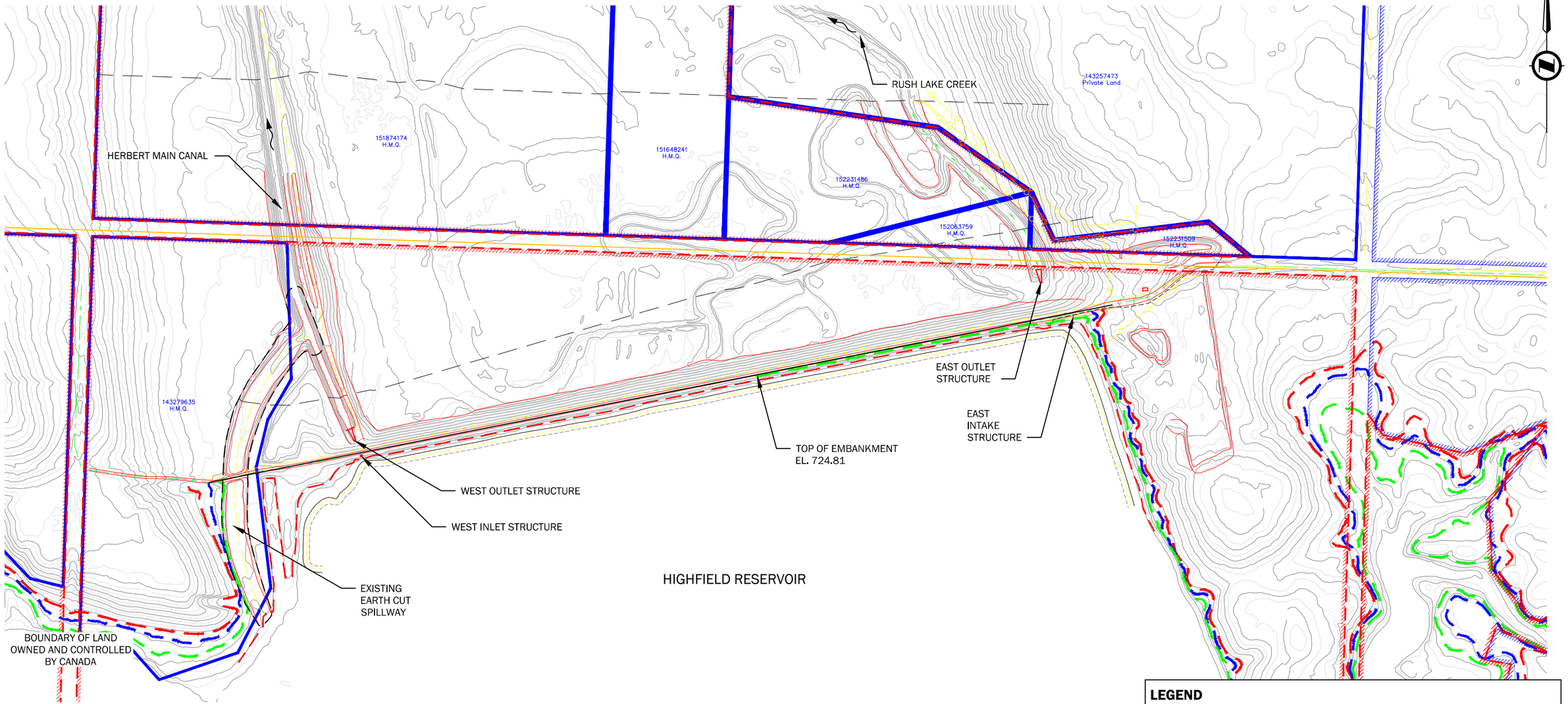
STELLA (January 2008), Software Version 4.4 created by Robert Webb, Australia.

Gonzalez, Carlos A. and Chanson, Hubert (2007). “Hydraulic Design of Stepped Spillways and Downstream Energy Dissipators for Embankment Dams”, Dam Engineering, Vol. XVVII, Issue 4.

HEC-RAS (January 2010), Hydrologic Engineering Center River Analysis System (HEC-RAS) software Version 4.1, U.S. Army Corps of Engineers.

USACE (1980). “General Guidance for Riprap Gradation (Pacific Northwest Rivers)”, US Army Corps of Engineers, Seattle District. Seattle, WA.

FIGURES



LEGEND	
	200 - YEAR RAINFALL FLOOD - EL. 724.41
	500 - YEAR RAINFALL FLOOD - EL. 725.18
	1000 - YEAR RAINFALL FLOOD - EL. 725.84
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

SITE PLAN

SCALE = 1:5000

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 20-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

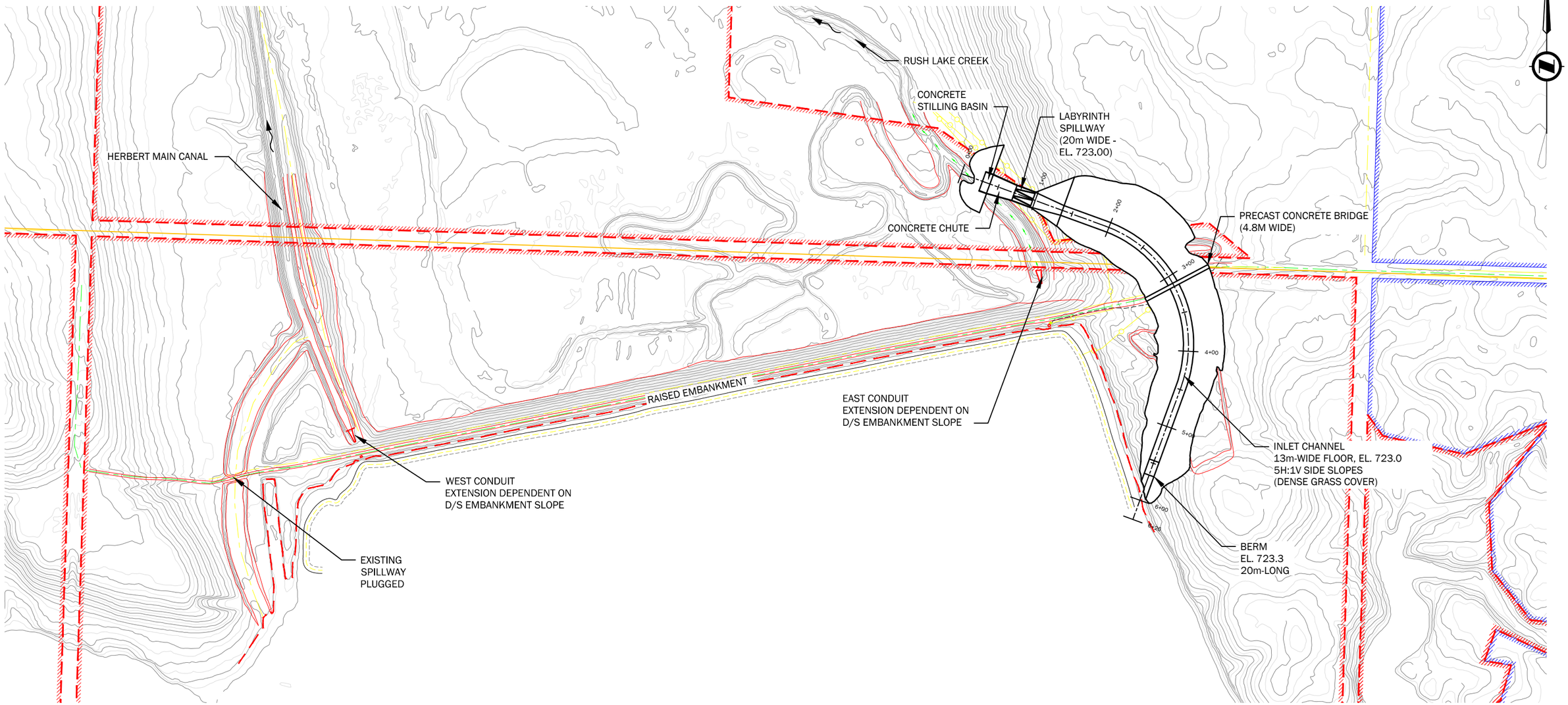
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 1

EXISTING SITE LAYOUT PLAN

Drawing No: 35525-001 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

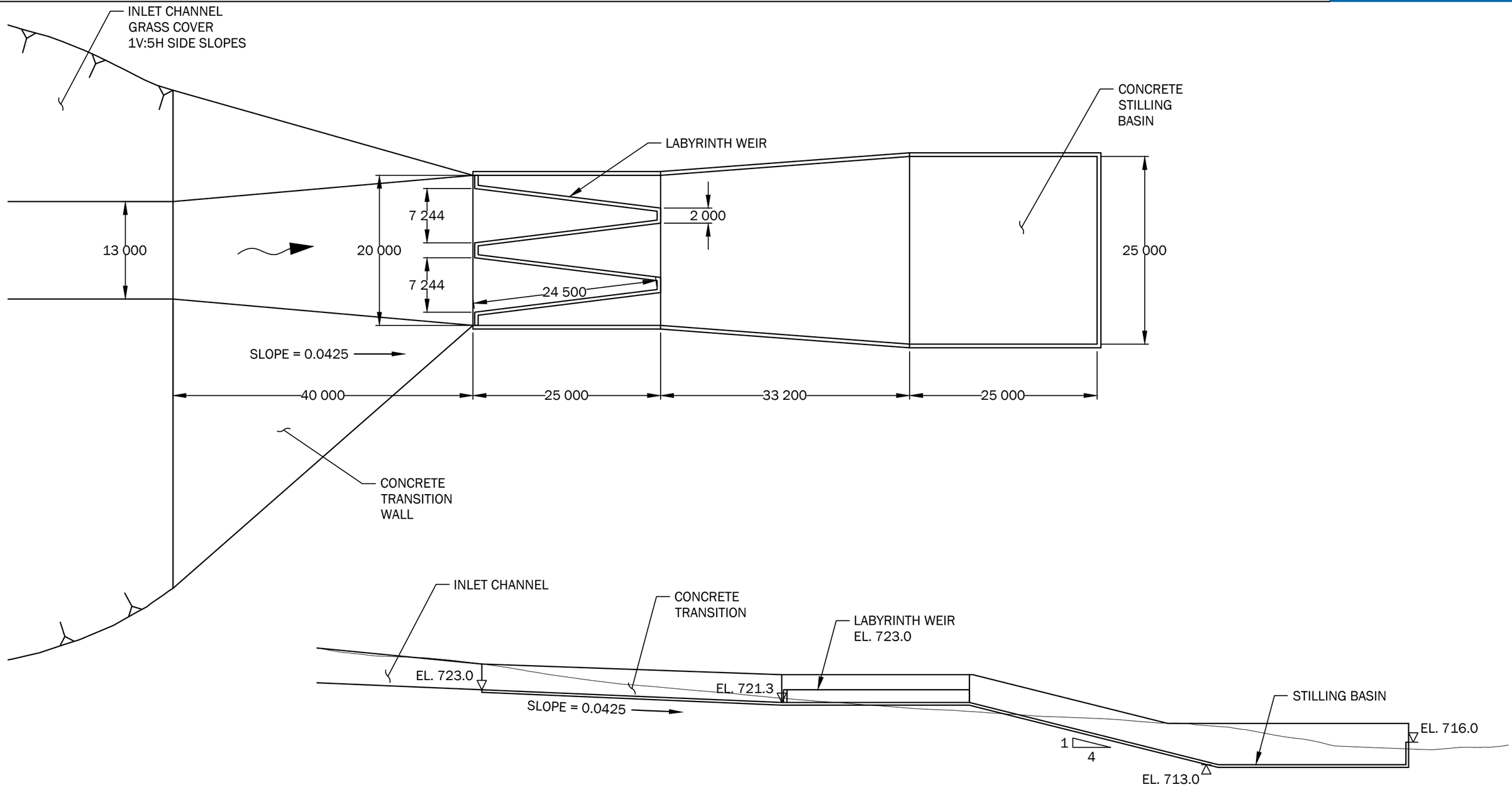
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 2

ALTERNATIVE 1
UNGATED LABYRINTH WEIR - EAST (PLAN)

Drawing No: 35525-002 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



LABYRINTH SPILLWAY AND AND SECTION

SCALE = 1:500

nhc northwest hydraulic consultants	Revisions:			Scale:	AS NOTED
	No:	Description:	By:	Date:	Designed By:	KIH
					Drawn by:	BXH
					Reviewed By:	BRH,ETZ
					Date:	JAN 21-2011
					Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

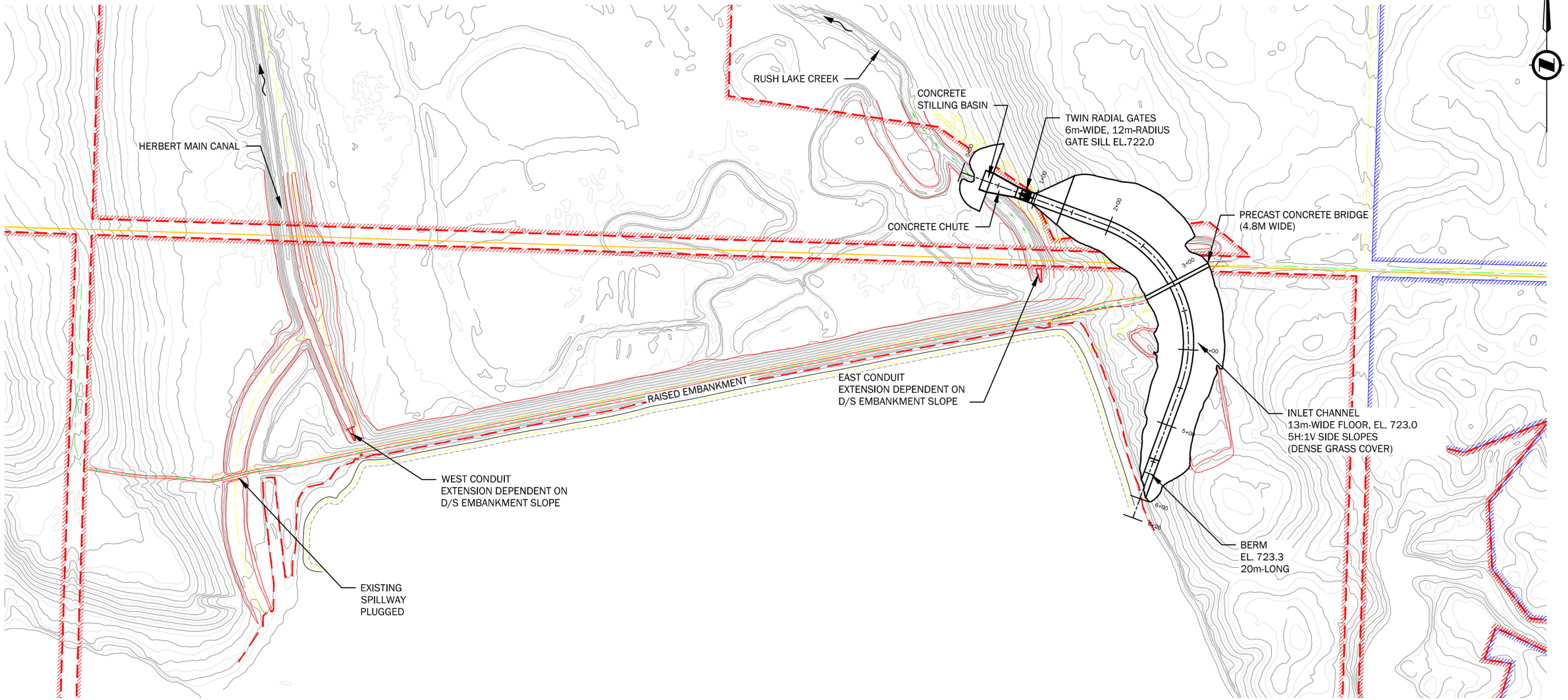
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 3

ALTERNATIVE 1
UNGATED LABYRINTH WEIR - EAST (DETAILS)

Drawing No: 35525-003 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

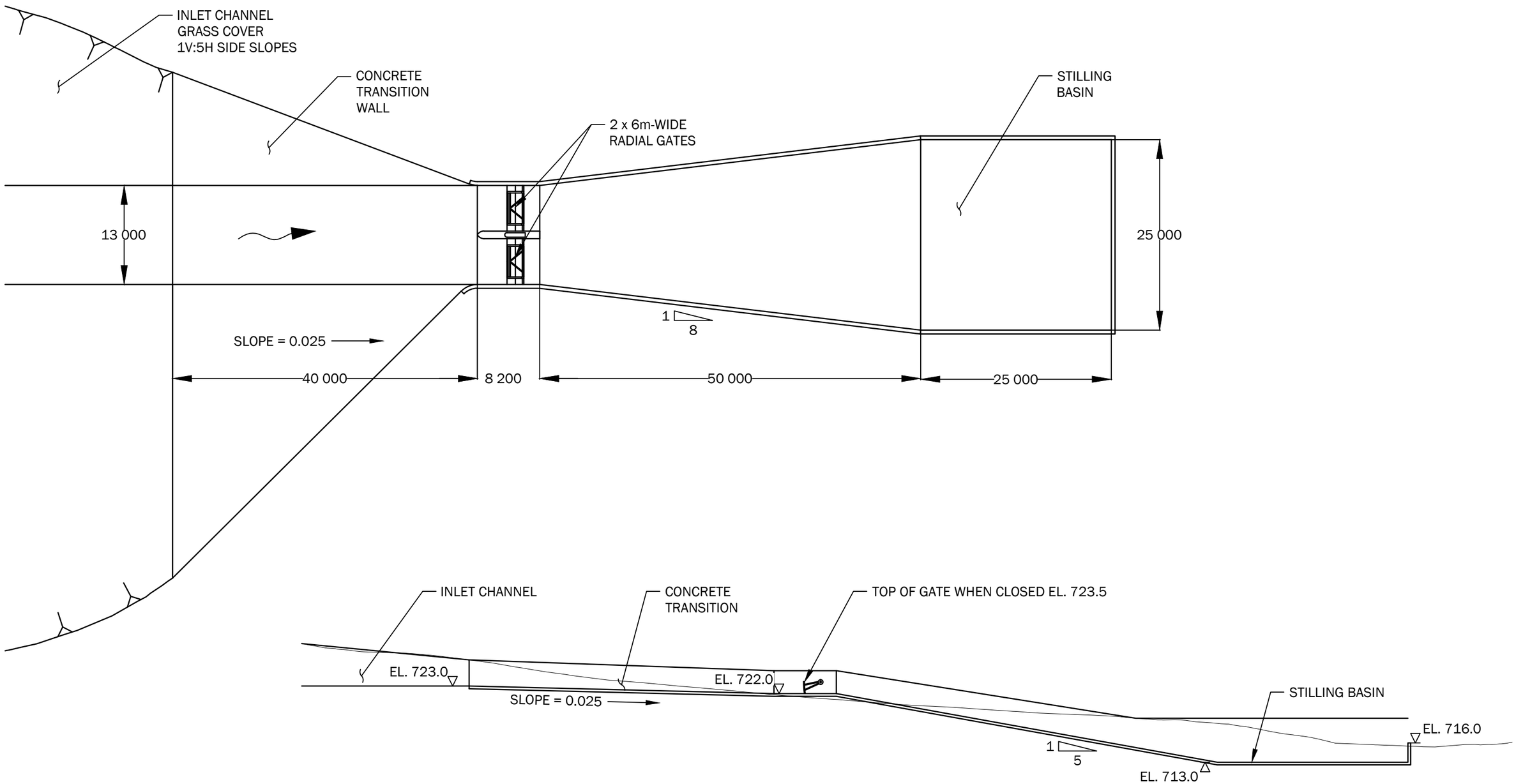
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 4

ALTERNATIVE 2
GATED SPILLWAY - EAST (PLAN)

Drawing No: 35525-004 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



GATED SPILLWAY PLAN AND SECTION

SCALE = 1:500

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

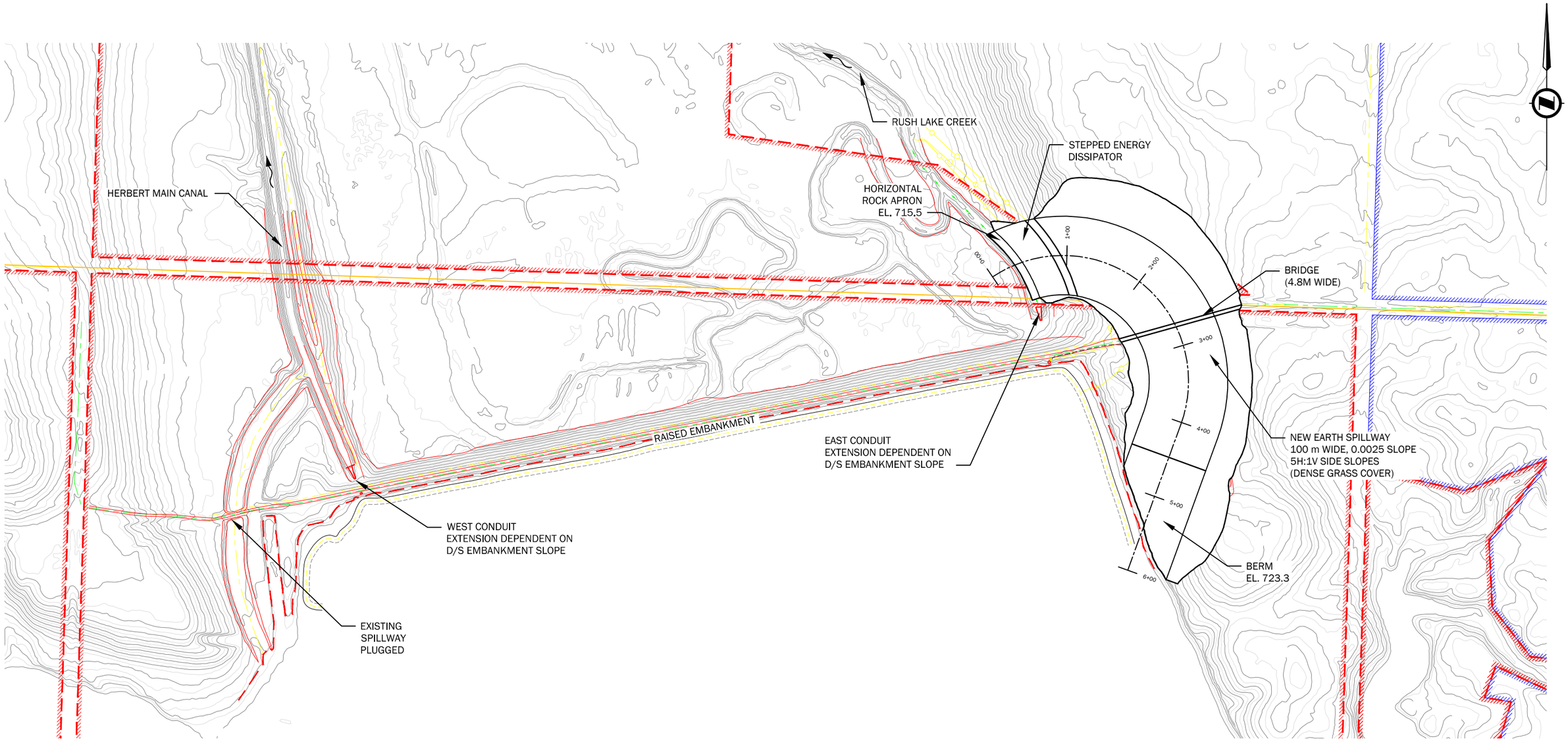
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 5

ALTERNATIVE 2
GATED SPILLWAY - EAST (DETAILS)

Drawing No: 35525-005 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
(Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

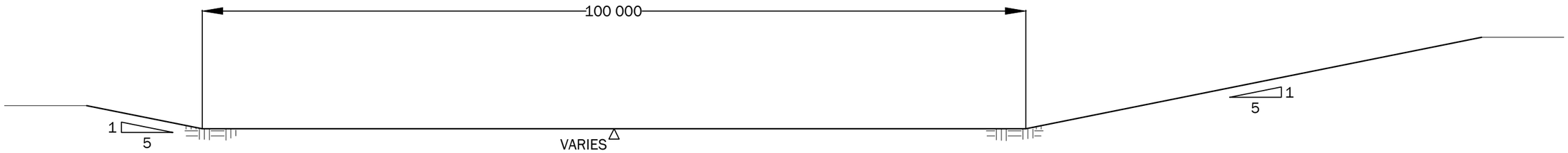
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 6

ALTERNATIVE 3
NEW EARTH SPILLWAY - EAST (PLAN)

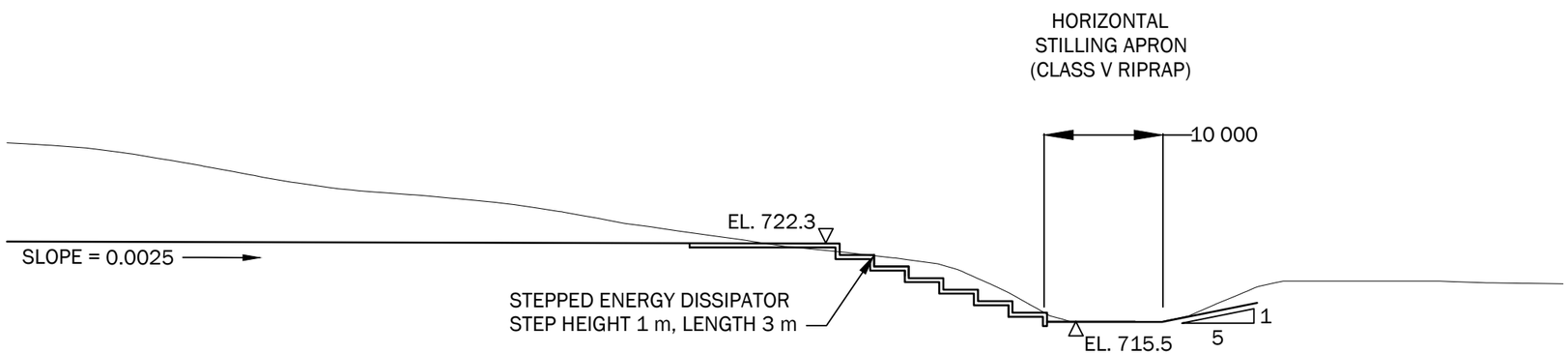
Drawing No: 35525-006 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



TYPICAL SECTION - NEW EARTH SPILLWAY

SCALE = 1:500



TYPICAL PROFILE - STEPPED ENERGY DISSIPATOR

SCALE = 1:500

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

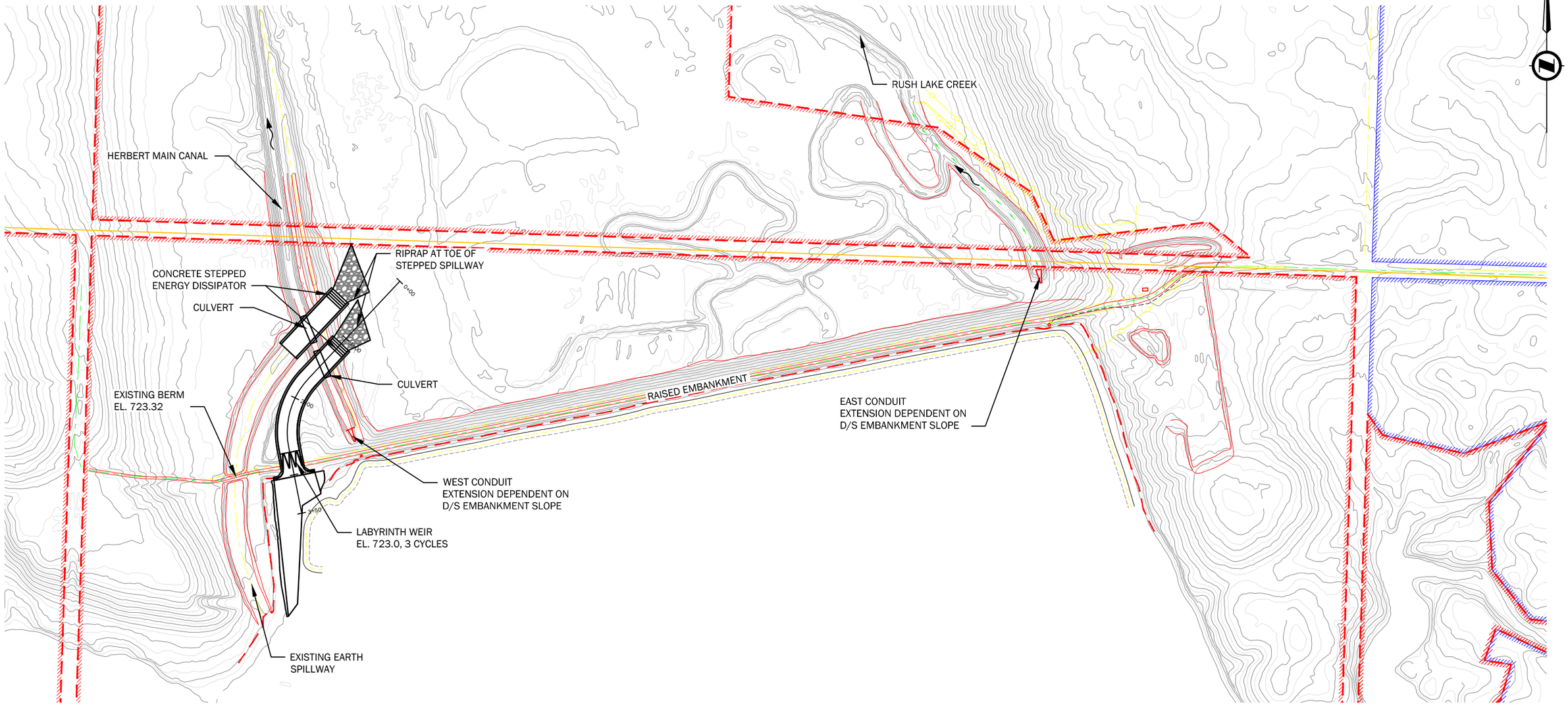
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 7

ALTERNATIVE 3
NEW EARTH SPILLWAY - EAST (DETAILS)

Drawing No: 35525-007 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********** (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

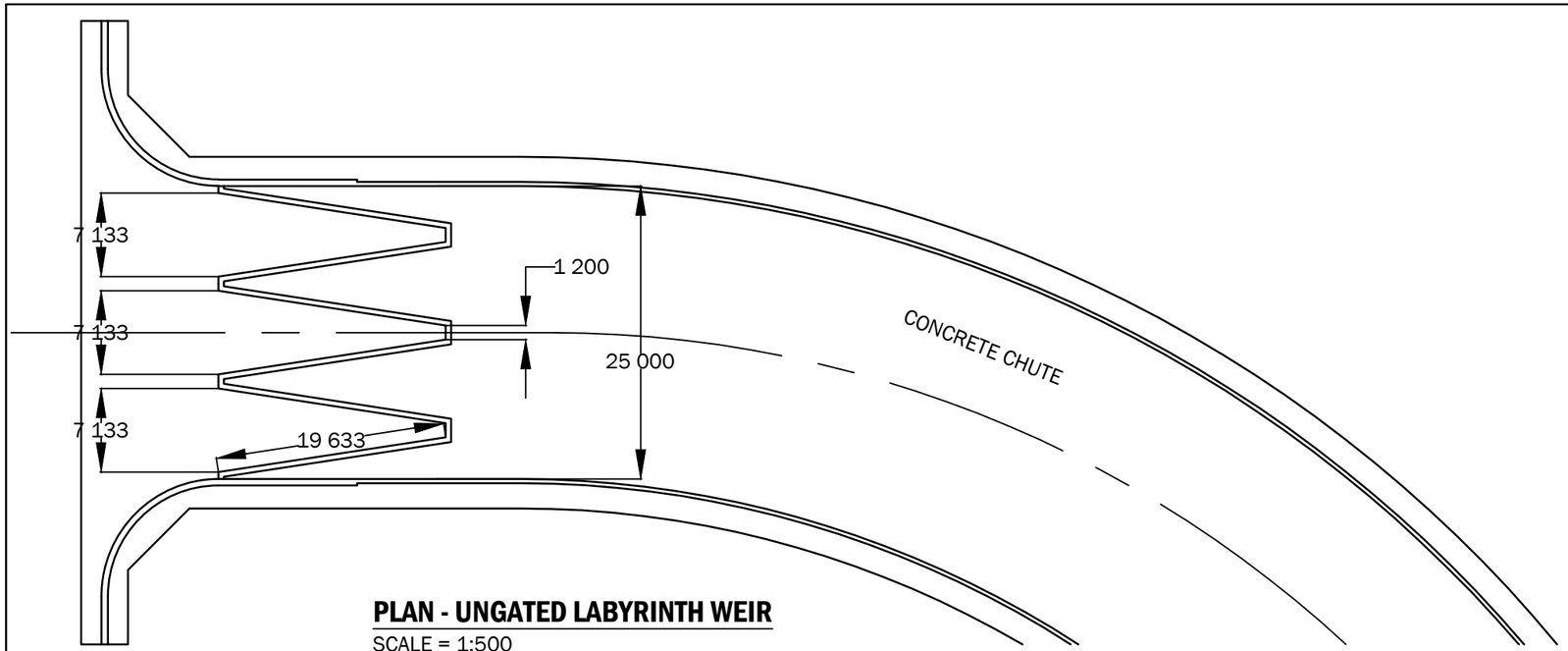
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 8

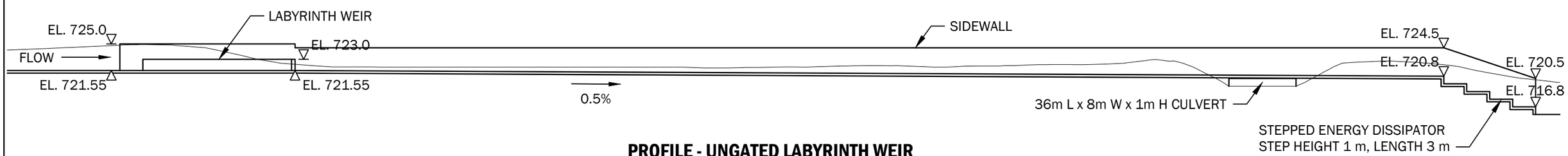
ALTERNATIVE 4
UNGATED LABYRINTH WEIR - WEST (PLAN)

Drawing No: 35525-008 Rev: 0

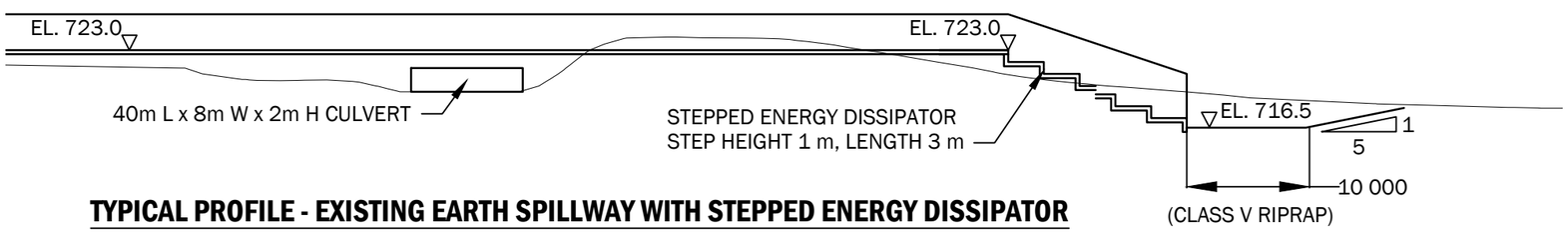
30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



PLAN - UNGATED LABYRINTH WEIR
SCALE = 1:500



PROFILE - UNGATED LABYRINTH WEIR
SCALE = 1:500



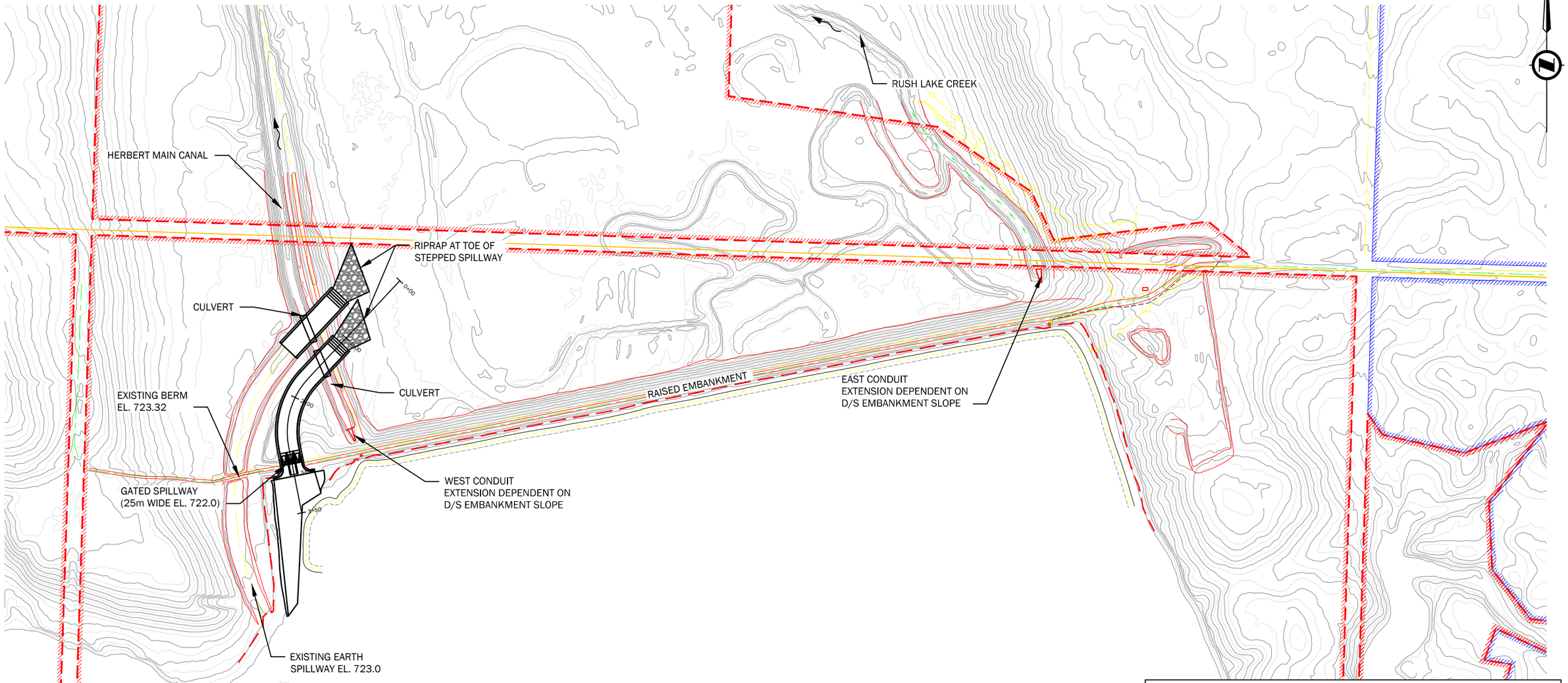
TYPICAL PROFILE - EXISTING EARTH SPILLWAY WITH STEPPED ENERGY DISSIPATOR
SCALE = 1:500

nhc northwest hydraulic consultants
30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada
AESB – Agri-Environment Service Branch

HIGHFIELD DAM SPILLWAY PRE-DESIGN
FIGURE 9
ALTERNATIVE 4
UNGATED LABYRINTH WEIR - WEST (DETAILS)
Drawing No: 35525-011 Rev: 0



LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

SITE PLAN

SCALE = 1:5000

nhc northwest hydraulic consultants

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

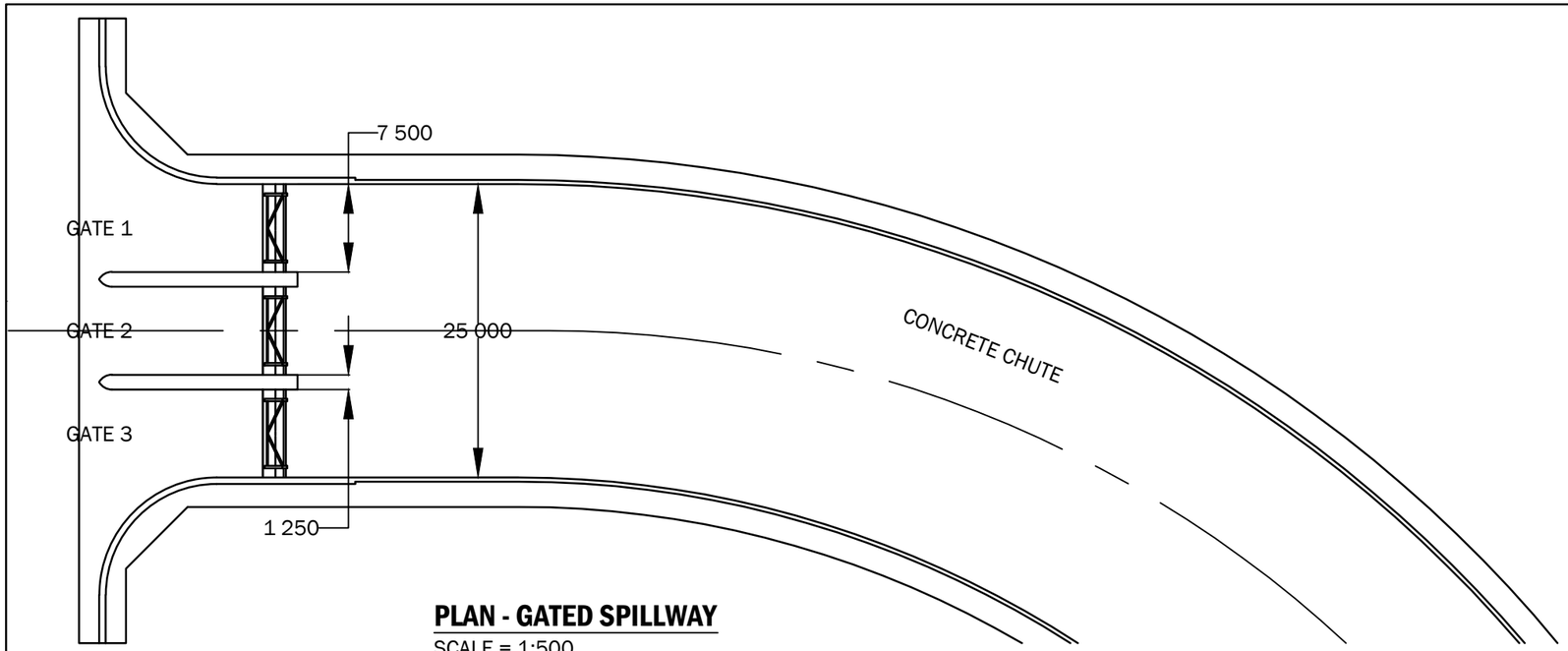
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 10

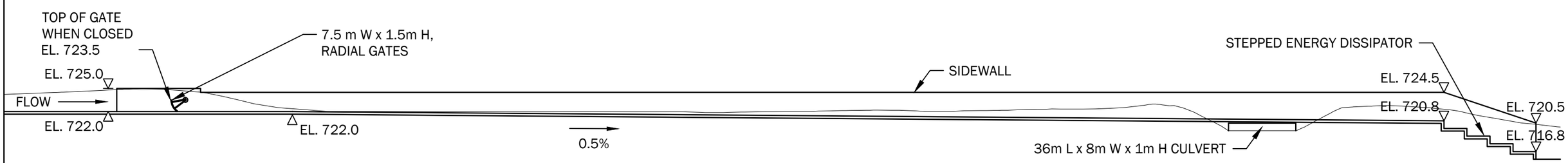
ALTERNATIVE 5
GATED SPILLWAY - WEST (PLAN)

Drawing No: 35525-010 Rev: 0

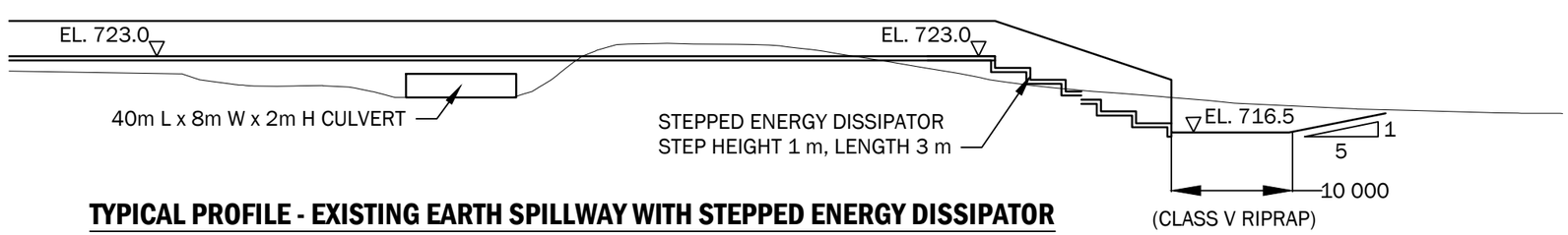
30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



PLAN - GATED SPILLWAY
SCALE = 1:500



PROFILE - GATED SPILLWAY
SCALE = 1:500



TYPICAL PROFILE - EXISTING EARTH SPILLWAY WITH STEPPED ENERGY DISSIPATOR
SCALE = 1:500

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

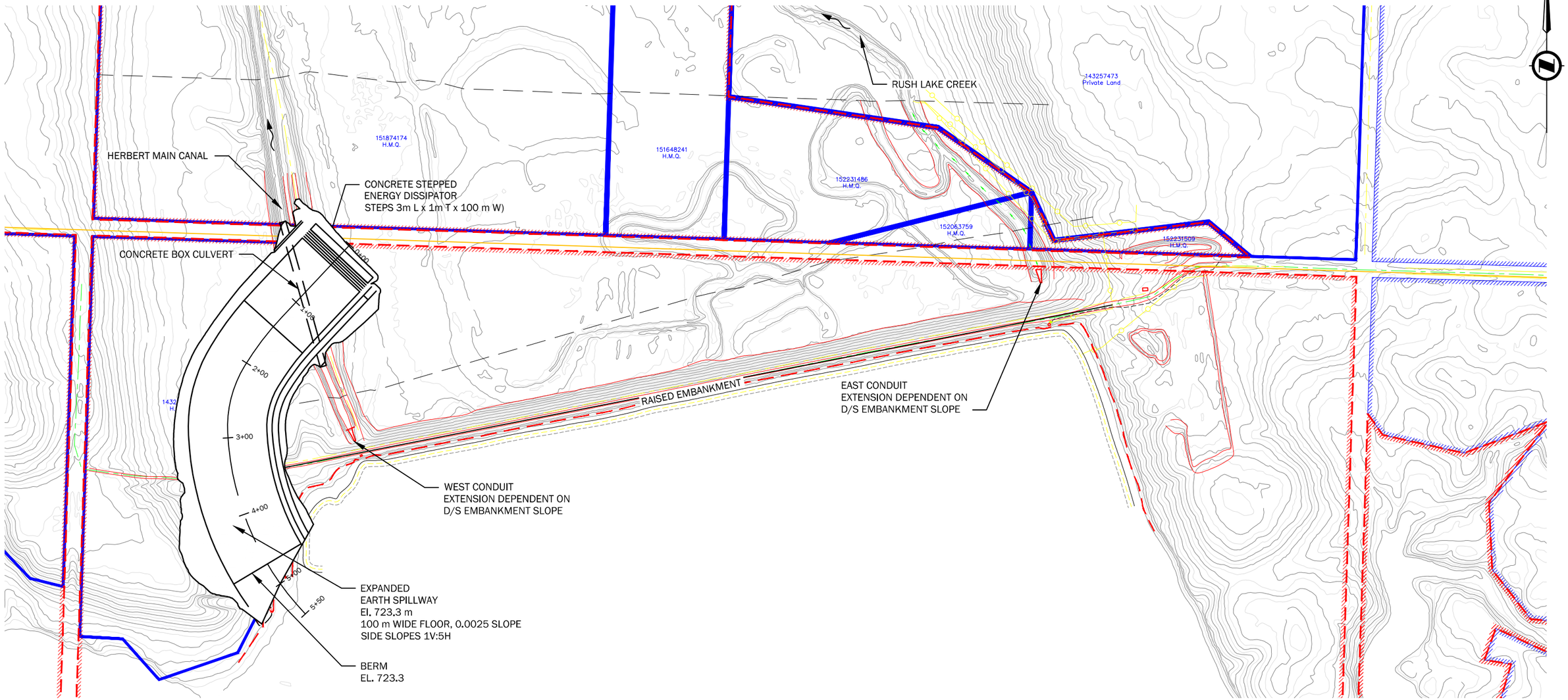
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 11

ALTERNATIVE 5
GATED SPILLWAY - WEST (DETAILS)

Drawing No: 35525-011 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

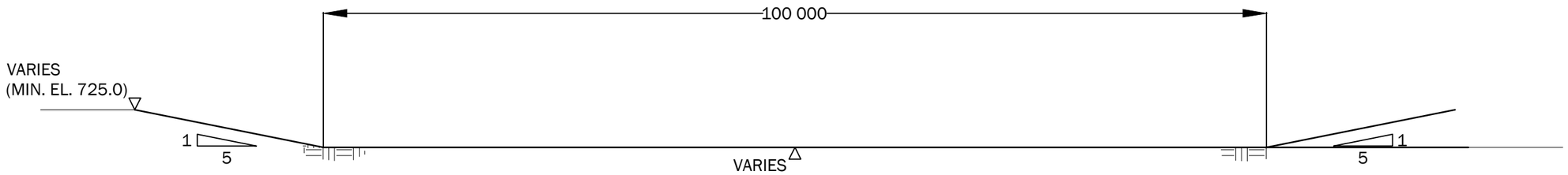
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 12

ALTERNATIVE 6
NEW EARTH SPILLWAY - WEST (PLAN)

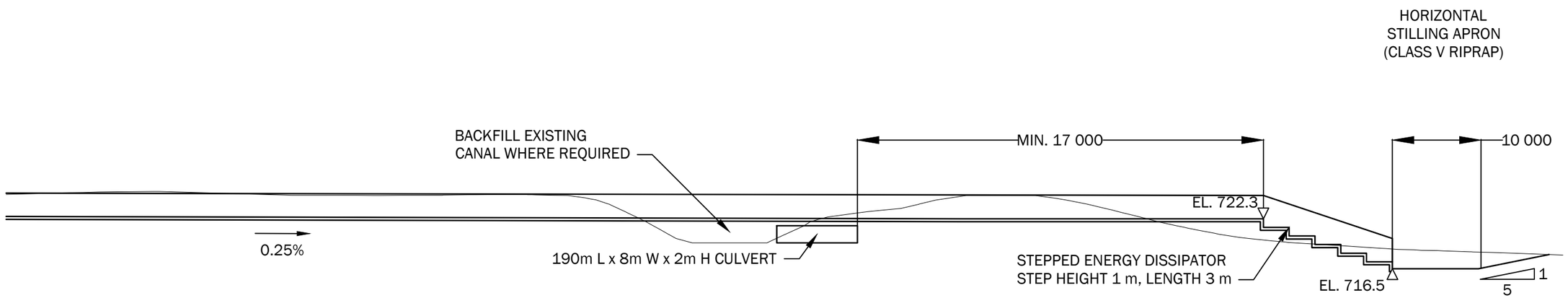
Drawing No: 35525-012 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



TYPICAL SECTION - NEW EARTH SPILLWAY

SCALE = 1:500



TYPICAL PROFILE - NEW EARTH SPILLWAY WITH STEPPED ENERGY DISSIPATOR

SCALE = 1:500

nhc northwest hydraulic consultants

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

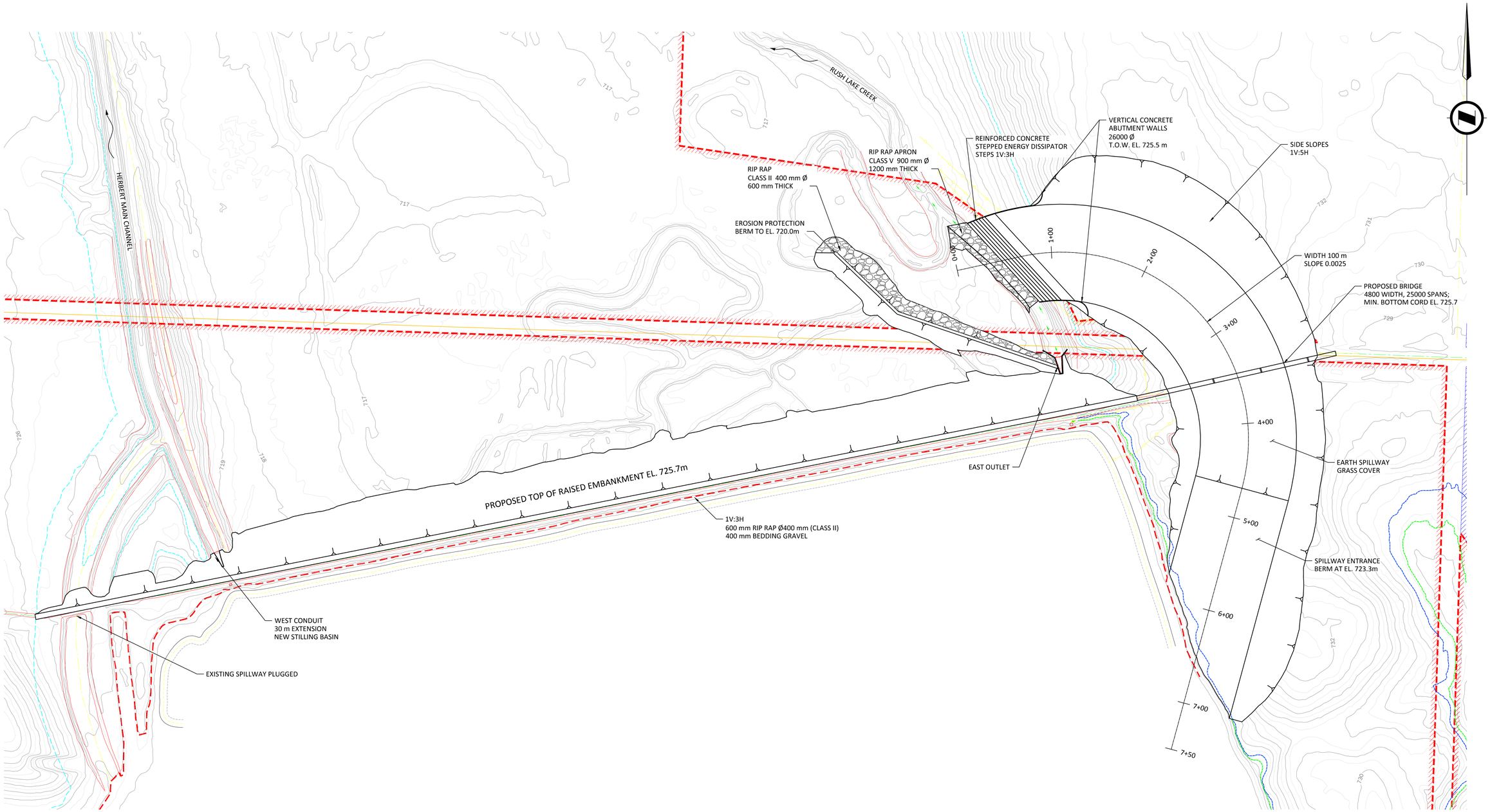
AESB – Agri-Environment Service Branch

HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 13

ALTERNATIVE 6
NEW EARTH SPILLWAY - WEST (DETAILS)

Drawing No: 35525-013 Rev: 0



LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
	MAXIMUM RESERVOIR LEVEL AT IDF EL. 725.2m
	MAXIMUM RESERVOIR LEVEL AT OSDF EL. 724.4m

SITE PLAN

SCALE: 1:2000

0 40 80 120 160 200

DISTANCE IN METRES

(1 cm = 40 m)

nhc northwest hydraulic consultants

Revisions:			
No:	Description:	By:	Date:
A	ISSUED FOR INFORMATION	KEH	MAR 2011

Scale:	AS NOTED
Designed By:	KIH
Drawn by:	KEH
Reviewed By:	BRH
Date:	MAR 2011
Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

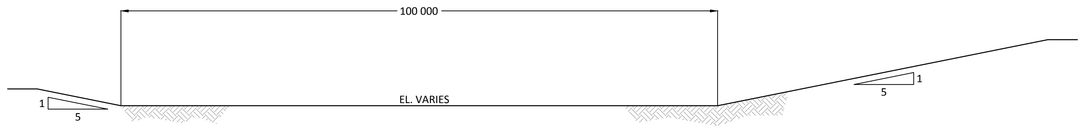
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 14

SPILLWAY PREDESIGN
EARTH SPILLWAY ON EAST ABUTMENT (PLAN)

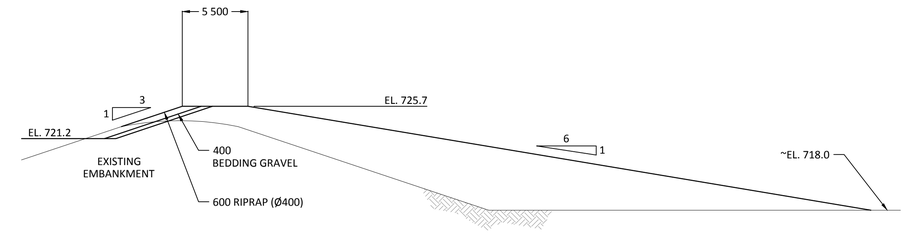
Drawing No: 35525-007 Rev: A

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



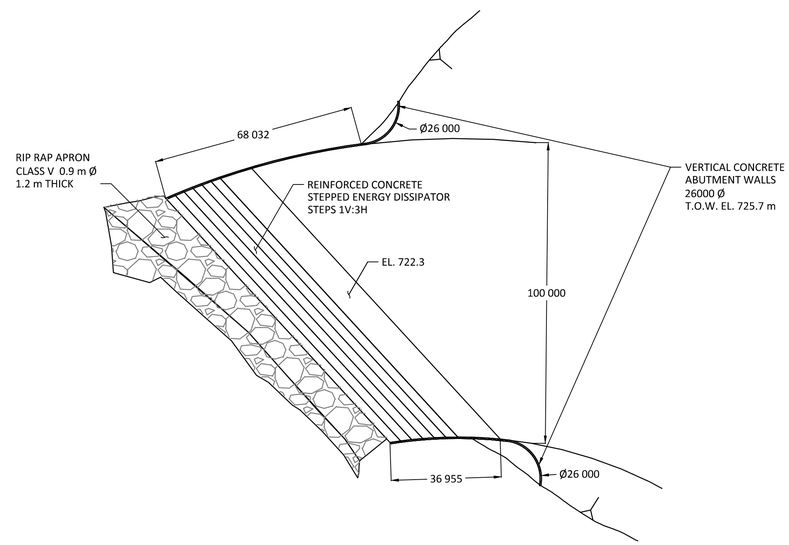
TYPICAL SECTION - EARTH SPILLWAY ON EAST ABUTMENT

SCALE: 1:500



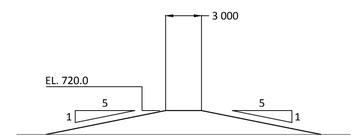
TYPICAL SECTION - RAISED EMBANKMENT

SCALE: 1:250



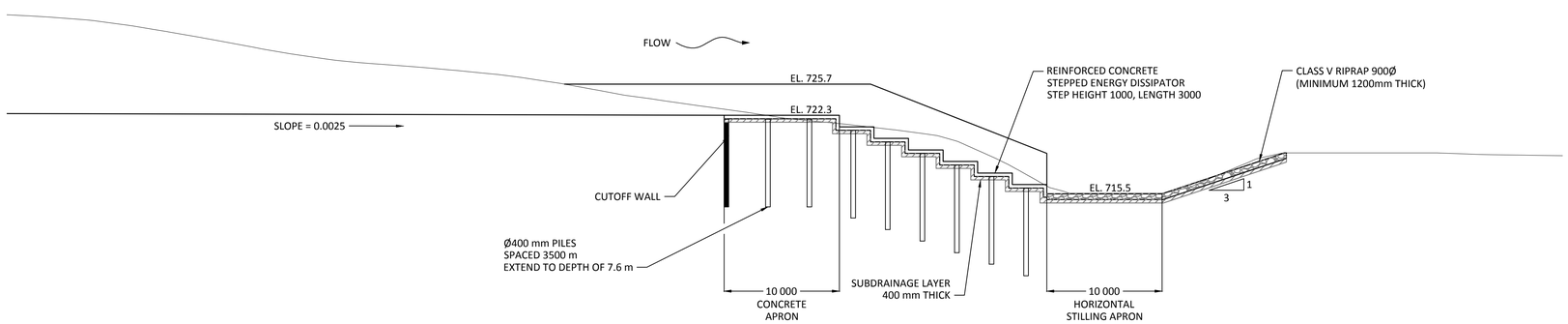
STEPPED ENERGY DISSIPATOR

SCALE: 1:1000



TYPICAL SECTION - EROSION PROTECTION PLAN

SCALE: 1:250



TYPICAL PROFILE - STEPPED ENERGY DISSIPATOR

SCALE: 1:250

nhc northwest hydraulic consultants

Revisions:			
No:	Description:	By:	Date:
A	ISSUED FOR INFORMATION	KEH	MAR 2011

Scale:	AS NOTED
Designed By:	KIH
Drawn by:	KEH
Reviewed By:	BRH
Date:	MAR 2011
Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

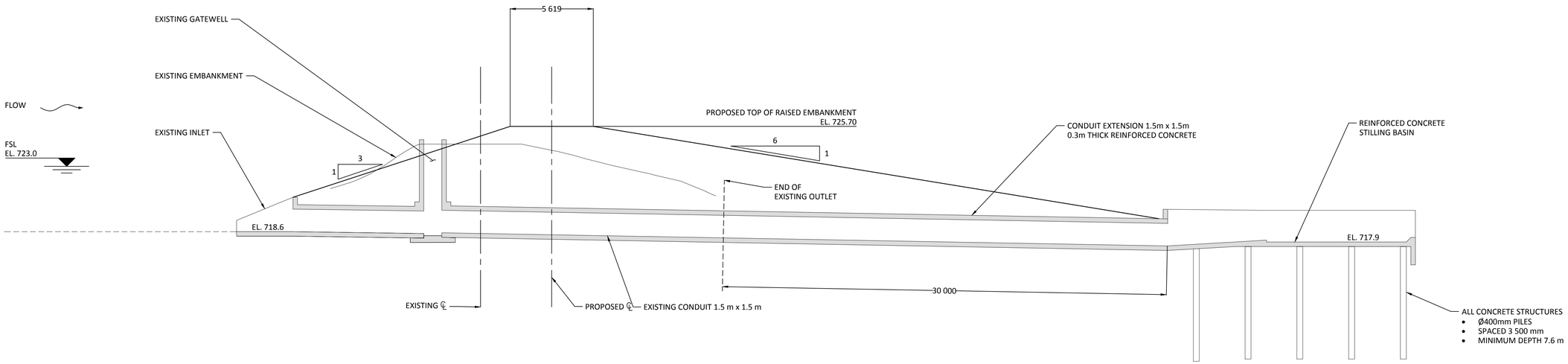
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 15

SPILLWAY PREDESIGN
EARTH SPILLWAY ON EAST ABUTMENT (DETAILS)

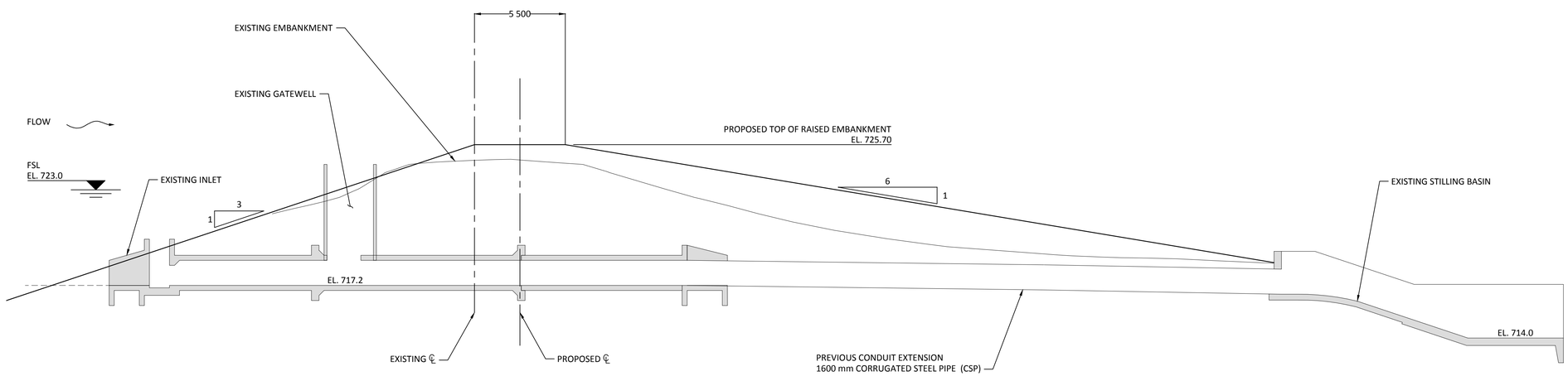
Drawing No: 35525-007 Rev: A

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



WEST OUTLET SECTION

SCALE: 1:150



EAST OUTLET SECTION

SCALE: 1:150

nhc northwest hydraulic consultants
30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com

Revisions:			
No:	Description:	By:	Date:
A	ISSUED FOR INFORMATION	KEH	MAR 2011

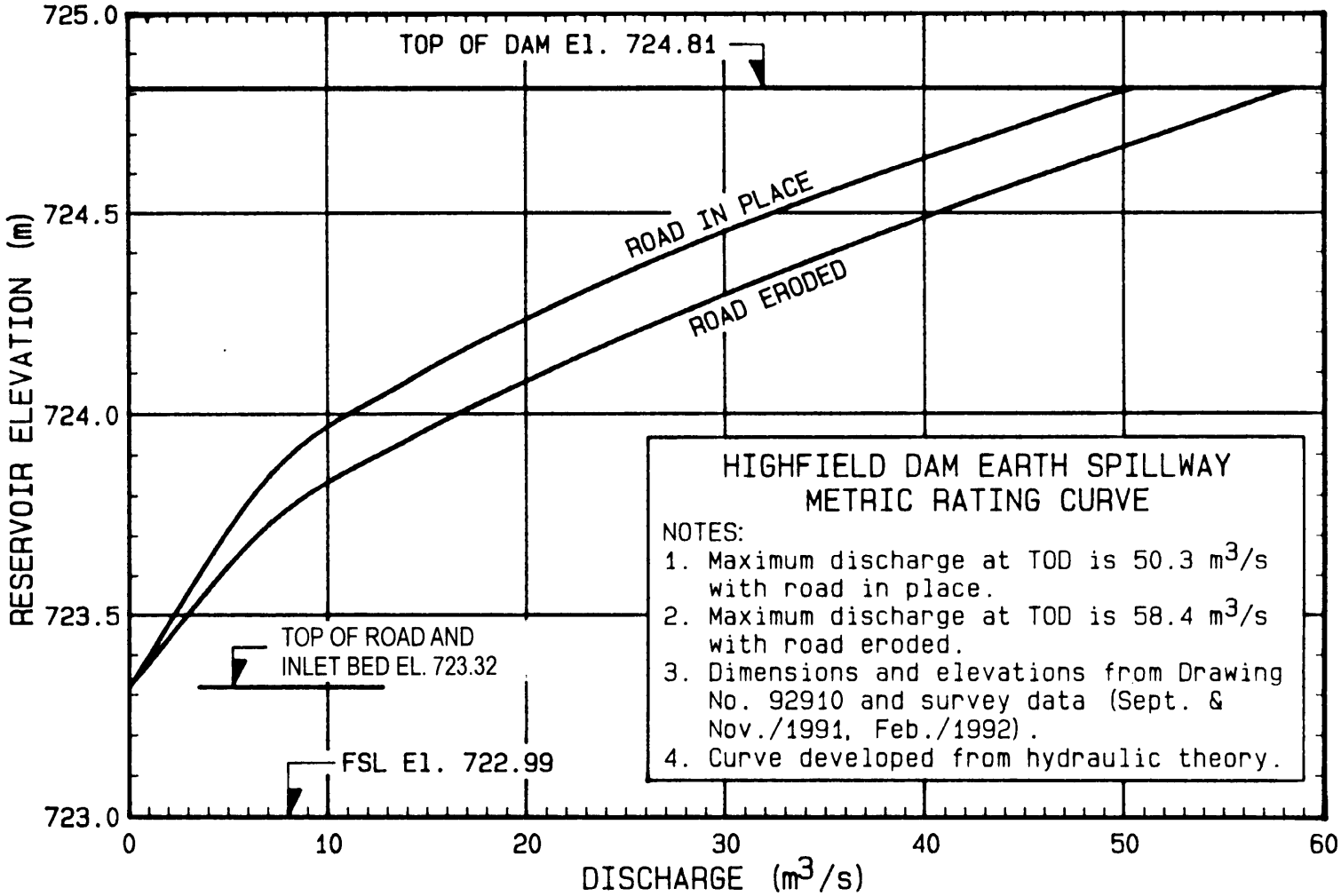
Scale:	AS NOTED
Designed By:	KIH
Drawn by:	KEH
Reviewed By:	BRH
Date:	NOV 2011
Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada
AESB – Agri-Environment Service Branch

HIGHFIELD DAM SPILLWAY PRE-DESIGN
FIGURE 16
SPILLWAY PREDESIGN
LOW LEVEL OUTLETS
Drawing No: 35525-007 Rev: A

APPENDIX A

HIGHFIELD DAM BACKGROUND INFORMATION



Designed
Drawn
Checked
Submitted
Date APR 16/92
Approved
Director, Development Service
Date 92/04/27

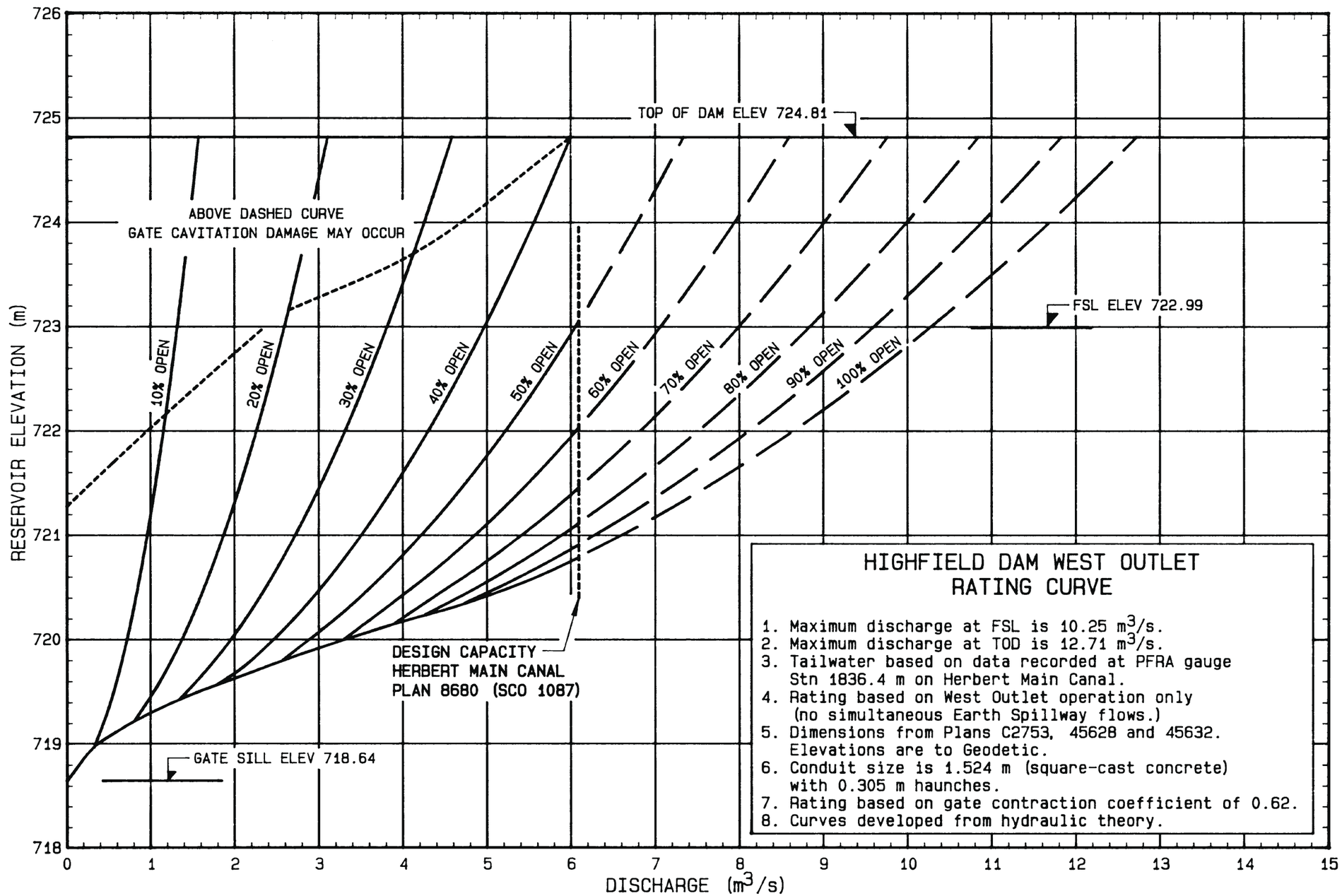
Agriculture Canada
Prairie Farm Rehabilitation Administration
Administration du Rétablissement agricole des Prairies
Development Service

SUPERCEDES DRAWING No. 116036

HIGHFIELD DAM

EARTH SPILLWAY
RATING CURVE (Metric)

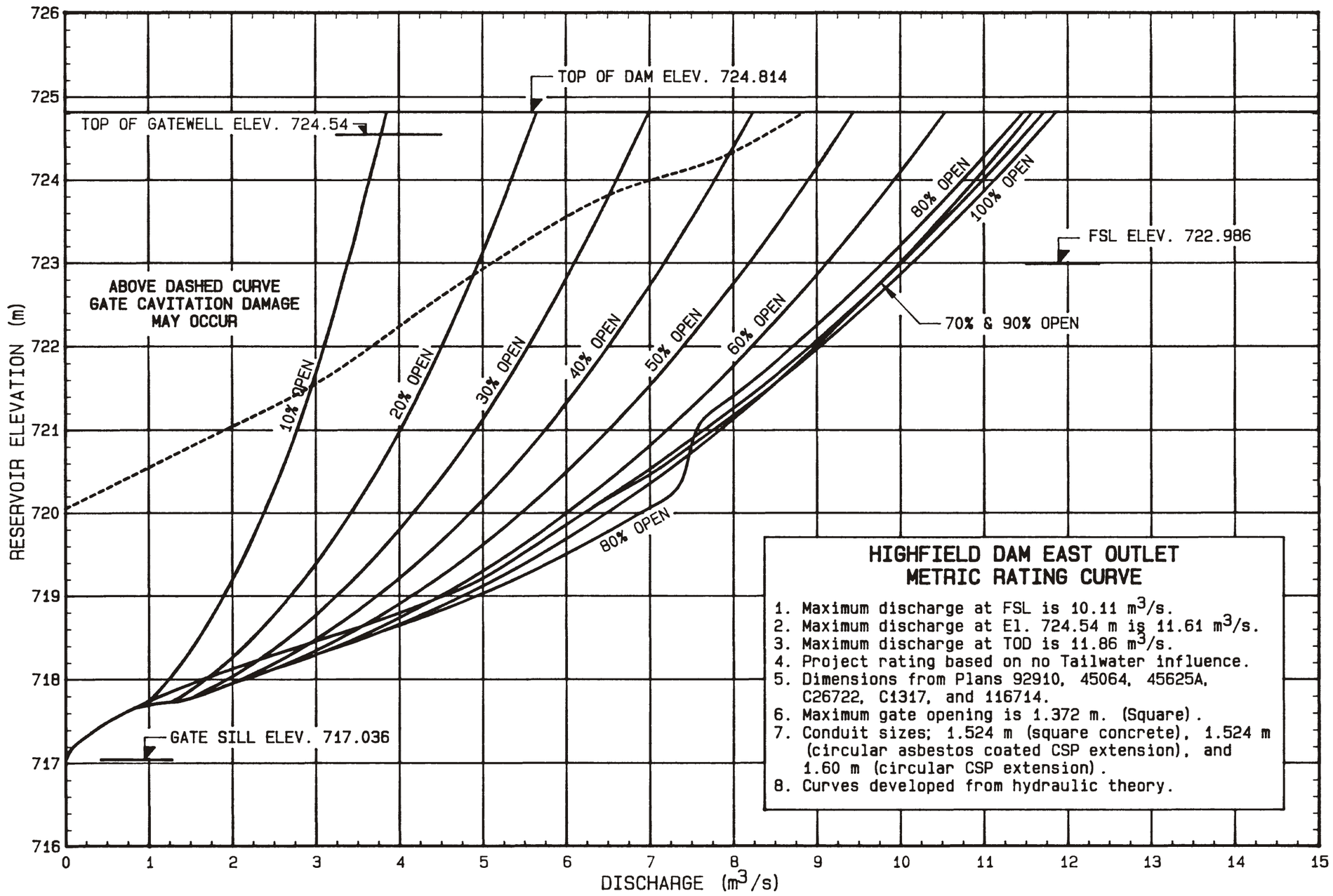
Scale
Date APR/92
PFRA No.
116036A



HIGHFIELD DAM WEST OUTLET RATING CURVE

1. Maximum discharge at FSL is 10.25 m^3/s.
2. Maximum discharge at TOD is 12.71 m^3/s.
3. Tailwater based on data recorded at PFRA gauge Stn 1836.4 m on Herbert Main Canal.
4. Rating based on West Outlet operation only (no simultaneous Earth Spillway flows.)
5. Dimensions from Plans C2753, 45628 and 45632. Elevations are to Geodetic.
6. Conduit size is 1.524 m (square-cast concrete) with 0.305 m haunches.
7. Rating based on gate contraction coefficient of 0.62.
8. Curves developed from hydraulic theory.

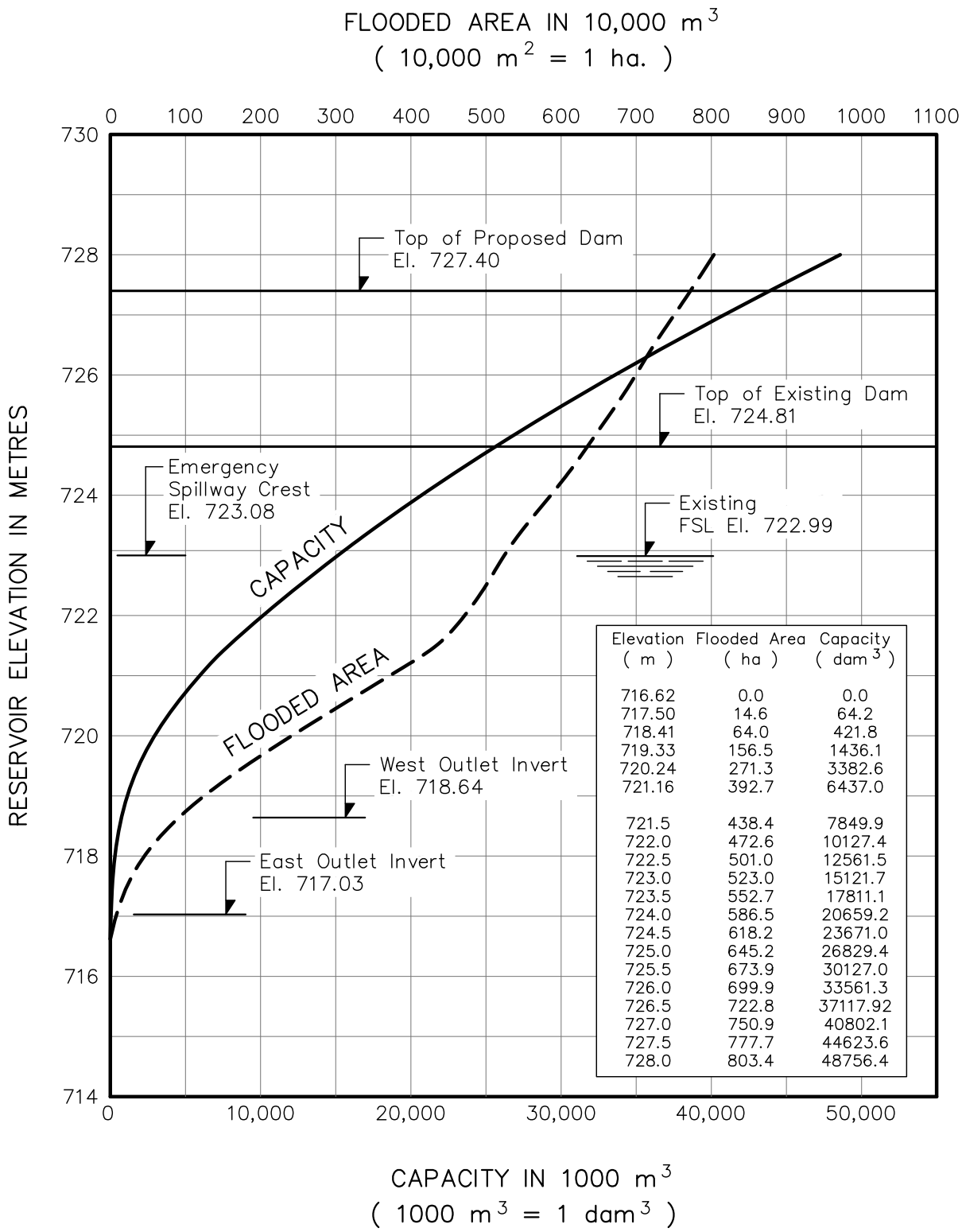
Designed RBW BAH	Submitted Brad H Hand	Agriculture Canada	HIGHFIELD DAM
Drawn RBW	Date APR 16/92	Prairie Farm Rehabilitation Administration / Administation du Rétablissement agricole des Prairies	**WEST OUTLET**
Checked WCB	Approved Director, Development Service	Development Service	**RATING CURVE (Metric)**
	Date 92/04/30		Scale / Date APR./92 / PFRA No. 116032



HIGHFIELD DAM EAST OUTLET METRIC RATING CURVE

1. Maximum discharge at FSL is 10.11 m^3/s.
2. Maximum discharge at El. 724.54 m is 11.61 m^3/s.
3. Maximum discharge at TOD is 11.86 m^3/s.
4. Project rating based on no Tailwater influence.
5. Dimensions from Plans 92910, 45064, 45625A, C26722, C1317, and 116714.
6. Maximum gate opening is 1.372 m. (Square).
7. Conduit sizes; 1.524 m (square concrete), 1.524 m (circular asbestos coated CSP extension), and 1.60 m (circular CSP extension).
8. Curves developed from hydraulic theory.

Designed BAH RBW	Submitted	Agriculture Canada	HIGHFIELD DAM
Drawn RBW	Date APR. 16/92	Prairie Farm Rehabilitation Administration / Administation du Rétablissement agricole des Prairies	EAST OUTLET RATING CURVE (Metric)
Checked WCL	Approved for Director, Development Service Date 92/04/30	Development Service	Scale / Date APR./92 / PFRA No. / 116034

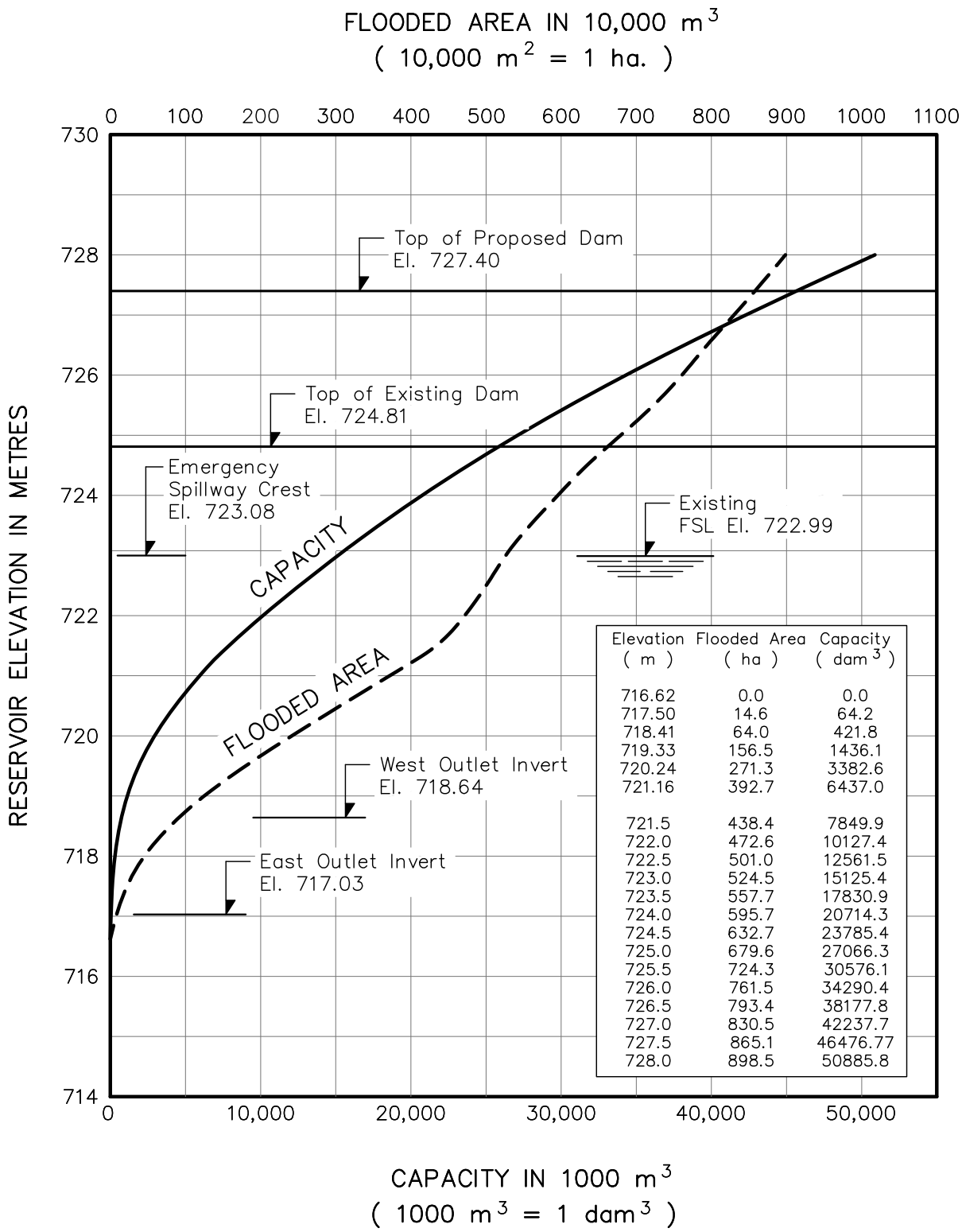


Elevation (m)	Flooded Area (ha)	Capacity (dam^3)
716.62	0.0	0.0
717.50	14.6	64.2
718.41	64.0	421.8
719.33	156.5	1436.1
720.24	271.3	3382.6
721.16	392.7	6437.0
721.5	438.4	7849.9
722.0	472.6	10127.4
722.5	501.0	12561.5
723.0	523.0	15121.7
723.5	552.7	17811.1
724.0	586.5	20659.2
724.5	618.2	23671.0
725.0	645.2	26829.4
725.5	673.9	30127.0
726.0	699.9	33561.3
726.5	722.8	37117.92
727.0	750.9	40802.1
727.5	777.7	44623.6
728.0	803.4	48756.4

1. Elevations – flooded area from El. 716.62 to 721.16 taken PFRA Plan 44295.
2. Elevations – flooded area from El. 721.5 to 728.0 based on Lidar Survey calculations.
3. Flooded Area and Capacity curves based on North/South Access Road in place with NO culverts.

Designed	Approved	Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	HIGHFIELD DAM
Drawn	Position Title	PFRA ARAP	CAPACITY AND FLOODED AREA CURVES MAIN EMBANKMENT TO N/S ACCESS ROAD
Checked	Date	Prairie Farm Rehabilitation Administration / Administration du retablissement agricole des Prairies	Scale: As Shown; Date: Nov./2010; PFRA No.: ******

Figure 1



Elevation (m)	Flooded Area (ha)	Capacity (dam^3)
716.62	0.0	0.0
717.50	14.6	64.2
718.41	64.0	421.8
719.33	156.5	1436.1
720.24	271.3	3382.6
721.16	392.7	6437.0
721.5	438.4	7849.9
722.0	472.6	10127.4
722.5	501.0	12561.5
723.0	524.5	15125.4
723.5	557.7	17830.9
724.0	595.7	20714.3
724.5	632.7	23785.4
725.0	679.6	27066.3
725.5	724.3	30576.1
726.0	761.5	34290.4
726.5	793.4	38177.8
727.0	830.5	42237.7
727.5	865.1	46476.77
728.0	898.5	50885.8

1. Elevations – flooded area from El. 716.62 to 721.16 taken PFRA Plan 44295.
2. Elevations – flooded area from El. 721.5 to 728.0 based on Lidar Survey calculations.
3. Flooded Area and Capacity curves based on North / South Access Raod in place with culverts.

Designed	Approved	Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	HIGHFIELD DAM
Drawn	Position Title	PFRA ARAP	CAPACITY AND FLOODED AREA CURVES MAIN EMBANKMENT TO DYKE
Checked	Date	Prairie Farm Rehabilitation Administration / Administration du retablissement agricole des Prairies	Scale: As Shown; Date: Nov./2010; PFRA No.: ******

Figure 2

APPENDIX B

SPILLWAY UPGRADE AT HIGHFIELD DAM
FINAL REPORT

MDH

Spillway Upgrade at Highfield Dam

Final Report

Prepared for:

30 Gostick Place

North Vancouver, B.C.

V7M 3G3

R2488-265010

November 2011

Executive Summary

Northwest Hydraulics Consultants (NHC) retained the services of MDH Engineered Solutions Corp. (MDH) to complete the geotechnical aspects of the spillway pre-design, including stability analyses of the dam incorporating the various spillway alternative designs.

Highfield Dam is located approximately 10 km south of Rush Lake, Saskatchewan on Rushlake Creek and is operated and maintained by the Agri-Environment Services Branch (AESB) of Agriculture and Agri-Food Canada (AAFC). The existing Highfield Dam/Reservoir project is comprised of a 1040 m long earth embankment, a 20 m wide earth-cut spillway located at the west end of the embankment, and low-level outlet conduits at the west and east ends of the embankment.

The scope of work for completion of the pre-design study for Highfield Dam spillway included five primary tasks, as described below. MDH was responsible for geotechnical engineering, constructability and cost estimating for the six spillway alternatives that would be designed by NHC. The five tasks were as follows:

1. Information review
2. Development of additional alternatives
3. Evaluation and comparison of alternatives
4. Development of the preliminary design for the selected alternative
5. Preparation of the pre-design report

Upon reviewing the existing geotechnical data for the Highfield Dam site it was determined by MDH that there was insufficient data to conduct a reliable slope stability analyses of the dam and recommended that additional geotechnical investigation be conducted. The scope of the additional geotechnical field work and laboratory testing that was conducted consisted of:

1. Drilling three deep stratigraphic boreholes for lithology and obtain in situ soil samples for laboratory testing through the downstream face of the dam and two shallow boreholes within the existing earth spillway on the west end of the dam;
2. Installation of three slope inclinometers to monitor any displacement;
3. Installation of multiple stacked vibrating wire piezometers (attached to the outside of the slope inclinometer casing or a 1” sacrificial PVC pipe) to monitor pore water pressures and the pressure gradient at each of the three borehole locations in the downstream face of the dam;

4. Performing laboratory testing on soil samples collected during the drilling program to obtain the material properties and geotechnical parameters required for the slope stability analysis, including at least one direct shear test;
5. Completion of a survey of the instrumentation to provide NAD83 co-ordinates;
6. Obtaining base line readings for the slope inclinometers (SI) installed as part of this study for future slope monitoring; and
7. Obtain initial pore water pressure readings from the vibrating wire piezometers installed as part of this study to obtain pore water pressure profiles in the dam and foundation soils

Six spillway alternatives were presented by NHC as follows:

1. Ungated labyrinth weir spillway on the east abutment;
2. Gated spillway on the east abutment;
3. Earth spillway on the east abutment;
4. Ungated labyrinth weir spillway on the west abutment;
5. Gated spillway on the west abutment; and
6. Earth spillway on the west abutment.

All six spillway alternatives require the existing dam height to be increased. The results of the slope stability analyses showed that the existing dam does not meet the Canadian Dam Association (CDA) 2007 Guidelines for overall factor of safety (FS). The CDA Guidelines (2007) specify a FS of 1.5, the existing dam has a factor of safety of approximately 1.2. Increasing the dam height to accommodate any of the six alternatives will further decrease the FS. Slope stability analyses were conducted using the proposed dam height for each of the six alternatives. Based on the analyses, it was determined that a minimum 6:1 downstream slope would be required to meet the CDA (2007) Guideline for overall factor of safety.

MDH conducted analyses of the costs of each of the six spillway alternatives based on the pre-design completed by NHC. The summary of costs is as follows:

Alternative	Capital Cost	Annual Maintenance Cost	Annual Operating Cost	Net Present Value of all Costs
1 - Labrynth Weir - East side	$14,619,586	$220,166	$15,000	$18,641,340
2 - Gated Spillway - East side	$15,736,246	$256,688	$25,000	$21,378,086
3 - Earth Spillway - East side	$14,581,747	$226,658	$5,000	$18,741,174
4 - Labrynth Weir - West Side	$16,822,203	$290,646	$10,000	$23,390,149
5 - Gated Concrete Spillway - West Side	$17,671,603	$334,382	$20,000	$25,740,638
6 - Earth Spillway - West Side	$21,383,207	$467,255	$5,000	$36,106,792

Notes:
Capital costs Include 17 percent for detailed design and construction engineering and 20 percent contingency
Net present value assumes 5 percent discount rate over 100 years

The costs showed that Alternative #3 which was an earth spillway constructed on the east side of the existing dam was least expensive.

NHC subsequently conducted a multi-criteria analysis (MCA) on the six spillway alternatives. Alternative #3 – the earth spillway on the east side of the existing dam scored highest on the MCA and was selected as the preferred alternative.

MDH provided preliminary geotechnical recommendations for the preferred alternative. The key recommendations were:

Slope Stability

Based on the proposed dam height for Alternative #3 of 725.7 masl, the recommended downstream slope is 6:1 and the recommended upstream slope is 3:1. MDH also recommended that additional investigation be conducted during the detailed design phase to further characterize the clay shales beneath the site, specifically to confirm the presence or absence of thin bentonite layers within the shale.

MDH recommends that a sand drainage blanket combined with a chimney drain(s) be installed as part of the dam expansion in order to reduce the build-up of pore pressures within the foundation soils due to the addition of fill soil for the construction of the upgraded dam embankment.

Preliminary modelling showed that staged construction will likely be required for the dam embankment to ensure that pore pressures during construction due not cause the existing dam to be compromised. An analysis of the required staging is recommended during detailed design.

Settlement

The upper clay shale at this site shows evidence of being disturbed or reworked by glacial action and as such is not likely in its original state of consolidation and could be subject to some settlement. Settlement cannot be reliably quantified. Estimates of settlement can be made using the results of consolidation testing on the foundation soils. Consolidation testing is recommended in the detailed design stage of the dam embankment in order to estimate the magnitude of settlement.

Foundation for Bridge Structure

The only structure to be constructed for Alternative 3 is the bridge across the spillway. The soils on the east side uplands generally consisted of clay shale. The soils on the east abutment of the existing dam consisted of the dam fill, which as cohesive material, underlain by alluvium and clay shale. It is expected that drilled cast-in-place concrete piles could likely be installed satisfactorily at this site and would be the least expensive option. Casing may be required in the upper portion of the piles that extend through the softer alluvium in order to install them successfully. Alternatively, continuous flight auger (CFA), driven timber or steel (pipe or HP) piles could be used. These options would likely be more expensive than drilled cast-in-place concrete piles. The type of pile would also be dependent upon the loads expected.

It is recommended that additional test holes be drilled at the location of the bridge abutments and piers in order to define the subsurface conditions and obtain samples for laboratory testing required to complete the detailed design of the bridge foundation.

Spillway Slopes

Based on drilling conducted by PFRA previously, the spillway on the east side will be constructed mainly within glacial clay till soil in the upper portion and clay shale in the lower portion. Also, the till is underlain by clay shale. Slopes of 5 horizontal to 1 vertical are recommended in the earth spillway in order to have long term stability of the slopes within the clay shale soils.

Erosion Protection

Establishing vegetation on the back–slopes should provide adequate erosion protection. Topsoil can be placed on the slopes and seeded with a deep rooting grass species capable of withstanding the local climate. Additional short–term erosion protection may be required to control any significant erosion during the period after construction and before vegetation is established. Turf reinforcement mats could be installed which aid in the establishment of vegetation and provide long–term erosion protection.

Alternative erosion control measures such as soil cement were not considered because of the shallow spillway slopes. However, if the spillway slope is steeper than proposed, alternative measures may have to be considered.

MDH can provide a detailed erosion control plan as part of the design phase.

Preliminary Design of Riprap Erosion Protection for the Spillway Outlet

Stone riprap should be placed at the spillway exit and in the stilling basin to prevent erosion. The rock riprap should be angular or sub–angular and as near to equi–dimensional as practical.

A 300 mm granular filter should underlie the riprap over the entire area. A cut-off wall will likely be required at the end of the channel to avoid water flow and piping of fines from under the rip-rap or concrete chute structure. The design of the cut-off wall can be done in the detailed design stage.

Instrumentation and Monitoring

Instrumentation will be required to monitor stability of the embankment, settlement, and porewater pressures during and after construction.

Three SIs were installed under this investigation along the downstream face of the dam. When the dam embankment is raised, the SI's will have to be raised by adding sections onto the existing casing as the embankment fill is placed. Protective steel casing should be installed to protect the SIs from damage. An adequate monitoring program will need to be defined at the detailed design stage.

Settlement instrumentation in the form of a special horizontal tube should be installed in the embankment to monitor consolidation of the foundation soils and embankment earth fill. The

horizontal tubes will provide a settlement profile beneath the entire embankment. The tubes can be read using a Shape Acceleration Array (SAA). SAA is a rope-like array of sensors and microprocessors that fits into a small (27 mm ID) casing. Any deformation that moves the casing is accurately measured as a change in shape of the SAA. Vibrations may also be measured, at multiple points.

Vibrating wire piezometers (VWPs) were installed in the three boreholes drilled along the embankment. Similar to the SI's, the VWP wires will have to be extended up through the embankment during construction when the embankment is raised. The VWPs should be in place prior to fill placement and sand drainage layer installation so existing conditions can be determined, and the effectiveness of the sand drain can be measured. The VWPs would require an adequately designed monitoring program at the detailed design stage.

Table of Contents

Executive Summary i
Table of Contents vii
List of Figures ix
List of Tables x
List of Appendices xi
1.0 Introduction 1
1.1 General 1
1.2 Objectives and Scope of Work 5
2.0 Background 7
3.0 Field Investigation 8
3.1 Borehole Drilling, Field Testing, Logging and Sampling 8
3.2 VW Piezometers and SI Casing Installation 9
4.0 Site Characterization 11
4.1 Physiographic Setting 11
4.1.1 Soils 11
4.1.2 Rushlake Creek Characteristics 12
4.2 Subsurface Conditions 12
4.2.1 Regional Geology 12
4.2.1.1 Development of the Neidpath Channel 19
4.2.2 Regional Hydrogeology 22
4.2.2.1 Bedrock Aquifers 22
4.2.2.2 Glacial Aquifers 24
4.2.3 Local Geology 24
4.2.3.1 Topsoil 28
4.2.3.2 Fill 28
4.2.3.3 Alluvium 28
4.2.3.4 Saskatoon Group Surficial Stratified Drift 29
4.2.3.5 Glacial Till 29
4.2.3.6 Clay Shale 30
4.2.4 Landslides 32
4.2.5 Local Hydrogeology 33
4.2.5.1 Piezometric Conditions Within the Alluvium 33
4.2.6 Piezometric Conditions Within the Dam Fill 34

4.2.6.1 Piezometric Conditions Within Glacial Tills on the Uplands and Valley Walls 34
4.2.6.2 Piezometric Conditions Within the Bearpaw Formation 35
4.3 Laboratory Testing 35
4.3.1 Basic Soil Characteristic Tests 35
4.3.2 Clay Shale Effective Shear Strength Parameters 36
5.0 Spillway Alternatives 37
6.0 Slope Stability Analyses 38
6.1 Surface Topography 39
6.2 Stratigraphy 42
6.3 Material Properties 42
6.3.1 Stability Parameters 42
6.3.2 Seepage Parameters 43
6.4 Hydraulic Conditions 44
6.5 Seismic Conditions 44
6.6 Failure Mechanisms 45
6.7 Method of Analysis 46
7.0 Slope Stability Analysis Results 47
7.1 Piezometric Model Calibration 47
7.2 Slope Stability Results 48
7.2.1 Existing Height Analysis 48
7.2.2 Scenario 1: Normal Operation at Full Supply Level (FSL) 49
7.2.3 Scenario 2: Probable Maximum Flood (PMF) 49
7.2.4 Scenario 3: Rapid Drawdown 50
7.2.5 Scenario 4: Seismic Considerations 51
7.2.6 Summary of Results for Existing Height 52
7.2.7 Upgraded Height Analysis 53
7.2.8 Scenario 1: Normal Operation 53
7.2.9 Scenario 2: Flood Event (1:1000) 56
7.2.10 Scenario 3: Rapid Drawdown 56
7.2.11 Scenario 4: Seismic Consideration 58
7.2.12 Summary of Results for Upgraded Height 59
7.3 Sensitivity Analysis 60
8.0 Cost Analyses for Spillway Alternatives 63
9.0 Geotechnical Considerations for Selected Spillway Alternative 65

9.1 Selected Alternative 65
9.2 Embankment Slope Stability Considerations 65
9.2.1 Recommended Embankment Slope 65
9.2.2 Sand and Chimney Drain 66
9.2.3 Staged Construction 68
9.3 Embankment Settlement 69
9.4 Embankment Construction Difficulties 70
9.5 Foundation for Bridge 71
9.6 East Side Spillway 72
9.6.1 Spillway Slopes 72
9.6.2 Spillway Subgrade 72
9.6.3 Spillway Terminal Structure 73
9.6.4 Spillway Outlet Deflector Berm 73
9.7 Erosion Protection 74
9.8 Preliminary Design of Riprap Erosion Protection for the Spillway Outlet 74
9.9 East and West Low Level Outlets 75
9.10 Instrumentation and Monitoring 75
9.10.1 Slope Stability 76
9.10.2 Settlement 76
9.10.3 Porewater Pressure 76
10.0 Closure 77
11.0 References 79

List of Figures

Figure 1-1 – Site Location 3
Figure 1-2 – Site Plan 4
Figure 4-1 - Schematic, overall stratigraphy of the Swift Current area, adapted from Maathuis and Simpson (2007) 13
Figure 4-2 - Schematic Quaternary stratigraphy of the Swift Current area, adapted from Maathuis and Simpson (2007). 14
Figure 4-3 - Generalized section of the Bearpaw Formation from the South Saskatchewan River Valley, adapted from Caldwell (1968) 15
Figure 4-4 – Cross Section I-I' (From Maathius and Simpson, 2007) 17
Figure 4-5 - Common glaciotectonic structures arranged according to their typical horizontal scale. (adapted from Occhietti, 1973) 18
Figure 4-6 – Regional Topographical Features 21

Figure 4-7 – Regional Aquifers 23
Figure 4-8 – 1940's cross section through Highfield dam 26
Figure 4-9 - Piezometric conditions at Highfield Dam, February to June, 2011. 33
Figure 6-1 - Forces Acting Upon a Slip Surface 38
Figure 6-2 – Cross Sections Through Dam 41
Figure 7-1 – Plot between calculated *Fs* vs. time (days) due to rapid drawdown. 51
Figure 7-2 - Sensitivity Results for Seismic Coeifficient at Existing Height 52
Figure 7-3- Slip Surface Detail of Dam Height of 725.7 masl (6:1 Slope) 55
Figure 7-4 - Slip Surface Detail of Dam Height of 727.1 masl (6:1 Slope) 55
Figure 7-5 - Rapid Drawdown Results for Upgraded Alternatives 57
Figure 7-6 - Sensitivity Results for Seismic Coefficient at Upgraded Height (727.1 masl) 59
Figure 7-7 - Sensitivity Analysis Results for Cross-section A-A' 61
Figure 7-8 - Sensitivity Analysis Results for Cross-section B-B' 62
Figure 7-9 - Sensitivity Analysis Results for Cross-section C-C' 62
Figure 9-1 - Fs of a 6:1 Slope With Sand Drain (725.7 masl Height) 67
Figure 9-2 - Fs of a 6:1 Slope Without Sand Drain (725.7 masl Height) 68

List of Tables

Table 3-1 – Borehole completion summary. 9
Table 3-2 – Summary of VW piezometer installation details 10
Table 4-1 – Summary of subsurface information 25
Table 4-2 - Piezometric conditions within the alluvium as reported in the PFRA Dam toe boreholes, September, 2009 34
Table 4-3 –Summary of the laboratory testing program 36
Table 4-4 – Clay shale effective shear strength parameters 36
Table 6-1 – Slope stability factors of safety for static conditions (2007 CDA Guidelines) 39
Table 6-2 - Slope stability requirements for seismic conditions (2007 CDA Guidelines). 39
Table 6-3 - Material Properties used in the stability analysis 43
Table 6-4 - Potential Failure Surfaces 45
Table 7-1 - Hydraulic Conductivity Values 47
Table 7-2 - Hydraulic Conductivity Calibration Results 47
Table 7-3 - Slope Stability Scenario Details 48
Table 7-4 - Slope Stability Results for Existing Height at FSL 49
Table 7-5 - Slope Stability Results of Existing Height at PMF 50

Table 7-6 - Seismic Stability Results for Existing Height 52
Table 7-7- Dam Height for Each Alternative 53
Table 7-8 - FSL and PMF Vales for Each Alternative 54
Table 7-9 - Slope Stability Results for Upgraded Heights at FSL 54
Table 7-10 - Slope Stability Results for Upgraded Heights at PMF 56
Table 7-11 - Rapid Drawdown Functions 57
Table 7-12 - Seismic Stability Results for Upgraded Height (727.1 masl) 58
Table 7-13 - Slope Stability Results of 6:1 Slope 60
Table 7-14 - Sensitivity Analysis Material Property Ranges 60
Table 8-1 – Highfield Dam Spillway Upgrade Financial Analyses Summary 64
Table 9-1 - Stability Analysis Summary for Existing Height (724.5 masl) 65
Table 9-2 - Stability Analysis Summary for Upgraded Height (725.7 masl) 65

List of Appendices

Appendix A – Schematic of VW Piezometer Installations
Appendix B –VW Piezometer Calibration Sheets
Appendix C – MDH Borehole Logs
Appendix D – PFRA Borehole Logs
Appendix E – Piezometer Field Data
Appendix F – Laboratory Testing Results
Appendix G – Spillway Alternatives
Appendix H – Detailed Cost Spreadsheets for Spillway Alternatives

1.0 Introduction

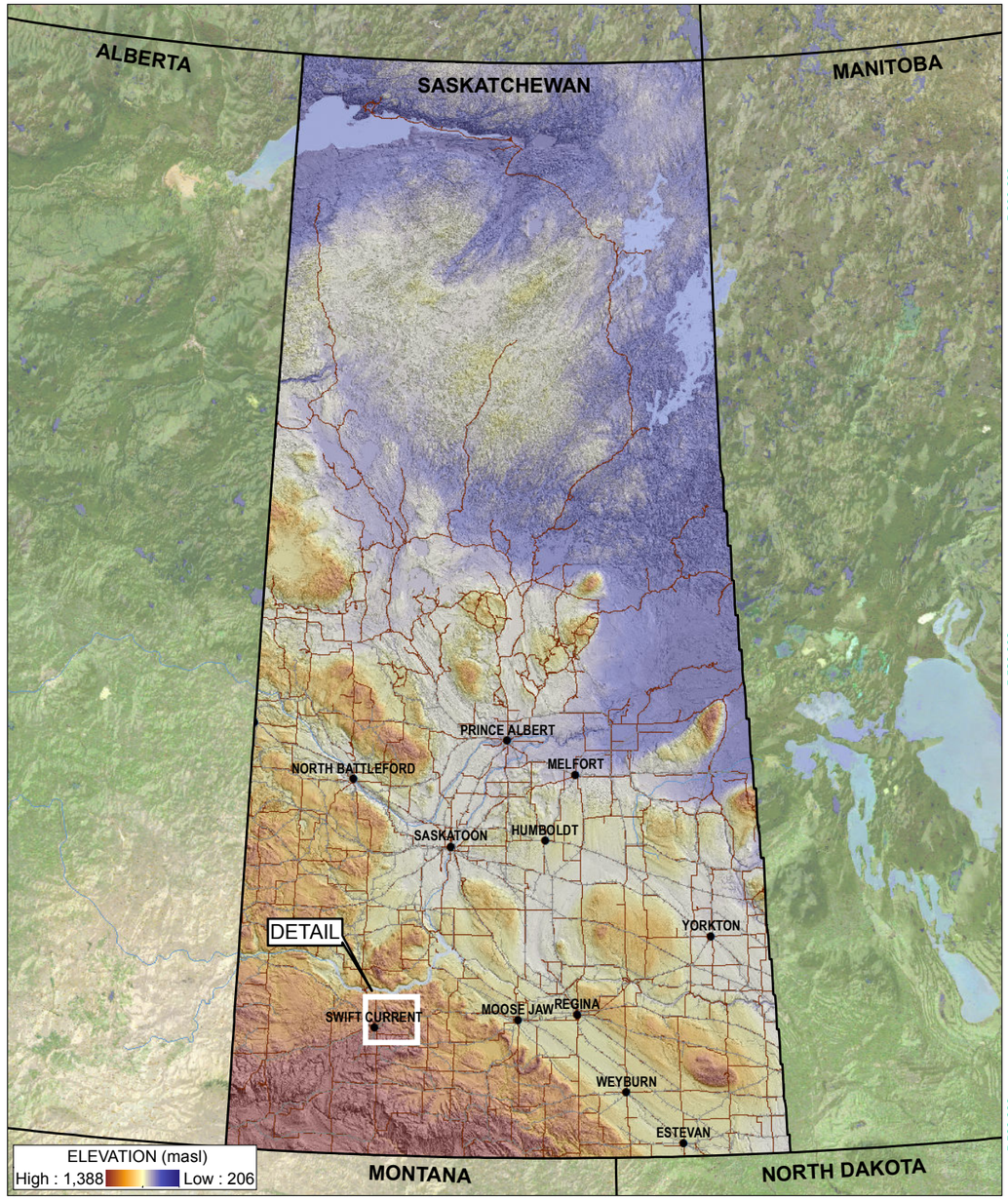
1.1 General

Northwest Hydraulics Consultants (NHC) was retained by the Agri-Environmental Services Branch (AESB) (formerly Prairie Farm Rehabilitation Administration (PFRA)) of Agriculture and Agri-food Canada (AAFC) to complete the Spillway Pre-design for the Highfield Dam located south of Rush Lake, Saskatchewan. NHC retained the services of MDH Engineered Solutions Corp. (MDH) to complete the geotechnical aspects of the spillway pre-design, including stability analyses of the dam incorporating the various spillway alternative designs.

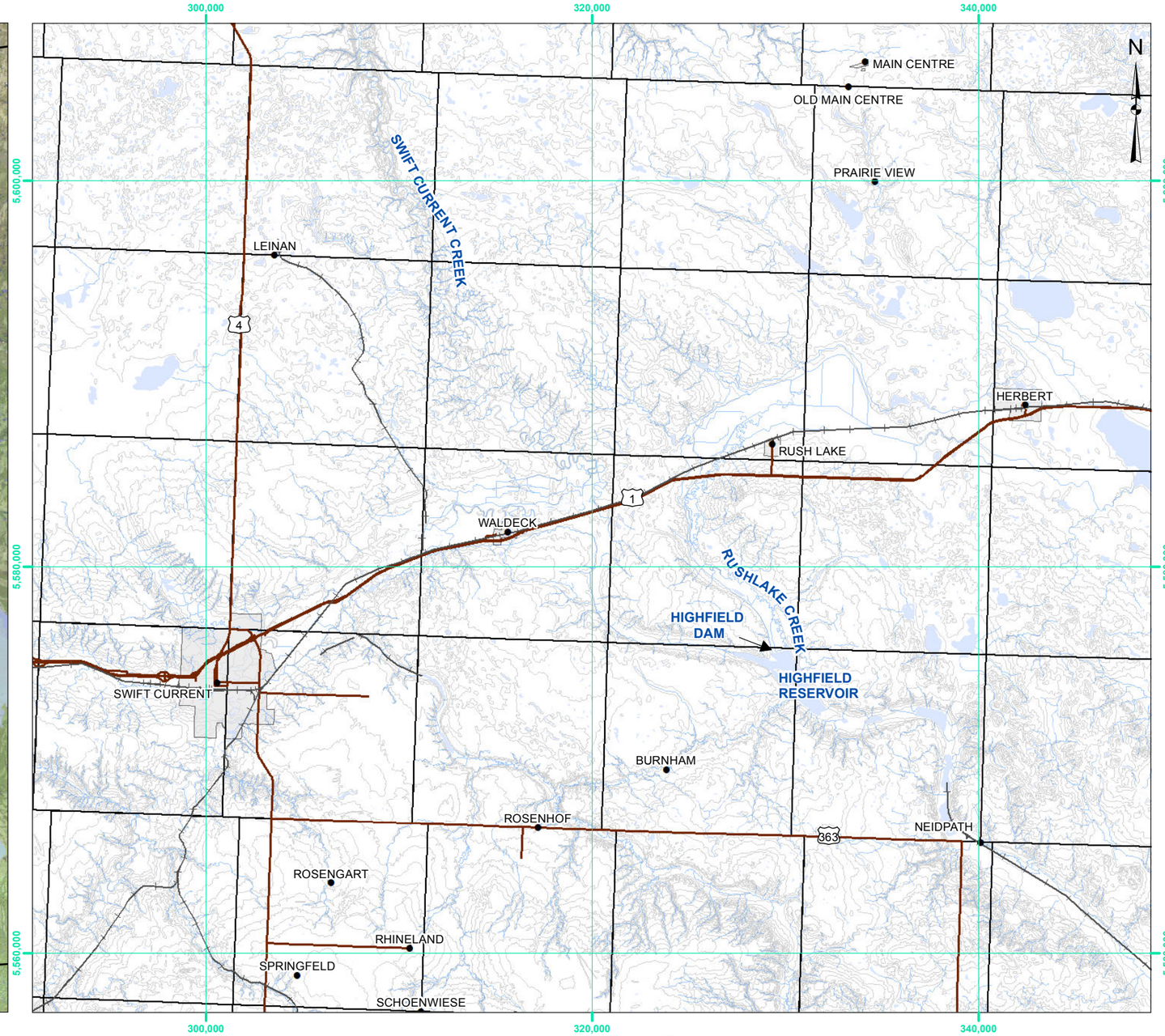
Highfield Dam is located approximately 10 km south of Rush Lake, Saskatchewan on Rushlake Creek and is operated and maintained by the AESB (Figure 1-1). The existing Highfield Dam/Reservoir project is comprised of a 1,040 m long earth embankment, a 20 m wide earth-cut spillway located at the west end of the embankment, and low-level outlet conduits at the west and east ends of the embankment (Figure 1-2). The embankment crest is set at El. 724.8 metres above sea level (masl) and the Full Supply Level (FSL) is El. 723.0 masl, providing 1.8 m of freeboard under normal operation. The existing low-level outlets have a combined capacity of approximately 17 m^3/s and the existing earth-cut spillway has a capacity of approximately 25.5 m^3/s (with 0.6 m of freeboard on the embankment), for an overall discharge capacity of 42.5 m^3/s. AESB Dam Safety Reviews have indicated that the project's existing spillway and conduit facilities cannot safely pass the project's Inflow Design Flood (IDF), which has been estimated at 361 m^3/s. In addition, the freeboard on the dam is inadequate by current dam safety standards, and a minimum 1.4 m raise of the embankment is necessary to ensure acceptable freeboard allowances. In order to address these deficiencies, AESB conducted conceptual and preliminary level studies, as described in the Highfield Dam Project Rehabilitation Pre-design Report (July 2009), for three spillway alternatives:

- Alternative 1: Ungated Spillway - New operating spillway located on east abutment consisting of an ungated labyrinth concrete chute spillway;
- Alternative 2: Gated Spillway - New operating spillway located on east abutment consisting of a gated concrete chute spillway; and
- Alternative 3: Earth Spillway - New operating spillway located on east abutment consisting of an earth spillway.

Each of these alternatives were designed to safely pass the Operating Spillway Design Flood (OSDF) of 180 m3/s with 2.0 m of freeboard (to prevent overtopping by wave action) and the



PROVINCE SCALE: 1:6,500,000



DETAIL SCALE: 1:250,000

Legend

- COMMUNITY
- RAILWAY
- MAJOR HIGHWAY
- WATER COURSE
- WATER BODY
- URBAN MUNICIPALITIES
- 25 METRE TOPOGRAPHIC CONTOUR

NOTES:
1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 13N.

DRAFT

SCALE	AS SHOWN	DATE
DESIGN BY		
DRAWN BY	S. LONG, GIS Cert.	07-MAR-11
APPROVED BY		

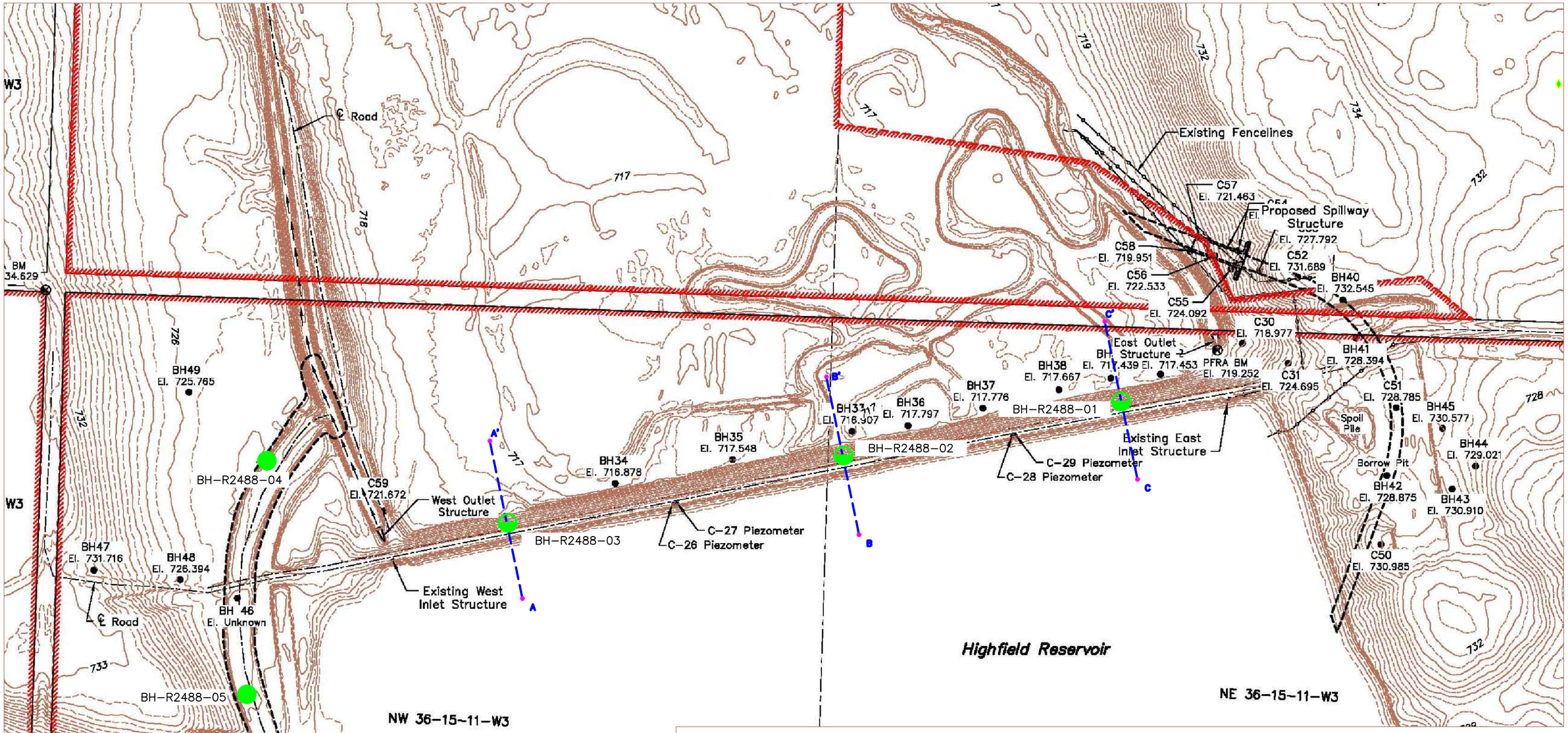
CLIENT: nhc northwest hydraulic consultants

PRODUCED BY: MDH ENGINEERED SOLUTIONS

TITLE: LOCATION PLAN

PROJECT No.	R2488-265010	FIG. No.	1-1
DRAWING No.	R2488-21-4		

Path: P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dams_GIS\2_Drawings\R2488-21-04 (Location Map).mxd



LEGEND

- MDH BOREHOLE
- MDH BOREHOLE WITH PIEZOMETERS
- BOREHOLE DRILLED BY OTHERS
- CROSS SECTION X — — — — X'

NOTE: SOURCE DRAWING INFORMATION OBTAINED FROM CANADA AGRI-ENVIRONMENTAL SERVICES BRANCH DRAWING:
HIGHFIELD RESERVOIR – MAIN WORKS REHABILITATION
EXISTING TOPOGRAPHY AND DRILL HOLE LOCATIONS – SHEET 1 of 3

CLIENT: nhc northwest hydraulic consultants

PRODUCED BY: MDH ENGINEERED SOLUTIONS

REFERENCE	
DWG. No.	DESCRIPTION

REVISIONS						
No.	DATE	DESCRIPTION	SUP.	DRN.	CHK.	APP.

REVISIONS						
No.	DATE	DESCRIPTION	SUP.	DRN.	CHK.	APP.

DRAWING STATUS	DATE
PRELIMINARY	
DESIGN REPORT	
APPROVED FOR TENDER	
APPROVED FOR CONSTRUCTION	
RECORD OF CONSTRUCTION	

SCALE	1 : 4,000 AT 11 x 17	DATE
DESIGN BY	D. MIHIAL, M.A.Sc., P.Eng.	16-MAR-11
DRAWN BY	S. RUSSELL, B.Sc.	16-MAR-11
CHECKED BY		
APPROVED BY		

TITLE: HIGHFIELD DAM BOREHOLE AND CROSS SECTION LOCATIONS

PROJECT No. R2488-265010 | FIG. No. 1-2

DRAWING No. R2488-91-1 | REV.

IDF (361 m3/s) with 0.6 m of freeboard. In addition, preliminary construction cost estimates were developed for each alternative. However, the July 2009 report was prepared without the results of the preliminary geotechnical investigation (completed in April 2010) or the ongoing the rare plant, fish and wildlife study (completed in December 2010). In addition, an error in the design of the weir height results in a significant change to the estimated volumes of concrete and earthwork, and hence cost. Finally, several feasible design alternatives were not included in the evaluation of alternatives, and although the report clearly favoured the labyrinth option, a single recommended option is not explicitly stated. Based on this, AAFC considered the July 2009 report to be incomplete. The purpose of the current Spillway Pre-Design Completion Project is to address these concerns and provide a clear recommendation for a preferred alternative, with sufficient background information to proceed to final design for the project.

1.2 Objectives and Scope of Work

On October 26, 2010, NHC in association with MDH submitted a proposal for the Highfield Dam Spillway Pre-design Completion and were awarded the contract.

In accordance with the proposal, the scope of work for completion of the pre-design study for Highfield Dam spillway included five primary tasks, as described below. MDH was responsible for geotechnical engineering, constructability and cost estimating for the six spillway alternatives that would be designed by NHC. The five tasks were as follows:

1. Information review
2. Development of additional alternatives
3. Evaluation and comparison of alternatives
4. Development of the preliminary design for the selected alternative
5. Preparation of the pre-design report

Upon reviewing the existing geotechnical data for the Highfield Dam site it was determined by MDH that there was insufficient data to conduct a reliable slope stability analyses of the dam. Previous investigations include boreholes at the crest and toe of the dam, but no boreholes had been documented on the downstream slope of the dam resulting in a critical gap of information. As well, no testing had been conducted to determine shear strength parameters for the soils within and below the dam. MDH subsequently recommended that additional geotechnical investigation be conducted. The scope of the additional geotechnical field work and laboratory testing that was conducted consisted of:

1. Drilling three deep stratigraphic boreholes for lithology and obtain in situ soil samples for laboratory testing through the downstream face of the dam and two shallow boreholes within the existing earth spillway on the west end of the dam;
2. Installation of three slope inclinometers to monitor any displacement;
3. Installation of multiple stacked vibrating wire piezometers (attached to the outside of the slope inclinometer casing or a 25 mm diameter sacrificial PVC pipe) to monitor pore water pressures and the pressure gradient at each of the three borehole locations in the downstream face of the dam;
4. Performing laboratory testing on soil samples collected during the drilling program to obtain the material properties and geotechnical parameters required for the slope stability analysis, including at least one direct shear test;

5. Completion of a survey of the instrumentation to provide NAD83 co-ordinates;
6. Obtaining base line readings for the slope inclinometers installed as part of this study for future slope monitoring; and
7. Obtain initial pore water pressure readings from the vibrating wire piezometers installed as part of this study to obtain pore water pressure profiles in the dam and foundation soils.

The scope-of-work did not include an assessment of the condition of the existing east and west low-level outlets.

2.0 Background

Runoff from Rushlake Creek and flows diverted from Swift Current Creek at Swift Current Weir via the Swift Current Main Canal are stored in Highfield Reservoir for releases to the Herbert and Rush Lake Irrigation Projects. Releases to the Herbert Irrigation Project are made through the West Outlet Works via the Herbert Main Supply Canal, which serves approximately 900 ha of privately irrigated land. Releases to the Rush Lake Irrigation Project are made through the East Outlet Works via the Rushlake Creek channel, which serves approximately 2200 ha of irrigated land owned by Prairie Farm Rehabilitation Administration (PFRA).

Highfield Dam, an earth fill structure, was originally constructed in 1941 by the Prairie Farm Rehabilitation Administration (PFRA). The dam was originally constructed to elevation 723 masl. The dam was raised several times between 1942 and 1950 to its present elevation of 724.8 masl.

The east irrigation low-level outlet works was constructed at the same time as the original embankment and consisted of a 1.5 m square concrete conduit with an upstream gate-well. In 1951, due to raising of the dam, a length of 1,524 mm diameter corrugated steel pipe (CSP) supported by timber piles was added to the downstream end of the conduit and a reinforced concrete energy dissipating outlet structure was constructed at the conduit outlet. Ice damage to the upstream gatewell resulted in construction of a central gatewell in 1952. In 1969 cracks in the downstream section of concrete conduit were repaired and the CSP was replaced. In 1983 and 1988, holes through the sloping portion of the outlet structure floor slab were repaired.

The west irrigation low-level outlet works was constructed in 1952 and consists of a 1.5 m square reinforced concrete conduit with a central gatewell and reinforced concrete inlet and energy converting outlet structure.

3.0 Field Investigation

3.1 Borehole Drilling, Field Testing, Logging and Sampling

Prior to drilling, MDH had Carson Energy Services Ltd. (CES) in Swift Current prepare working pads for drilling on the downstream slope of the dam. The working pads were constructed by cutting an approximately 9.1 m long by 3.0 m wide notch into the dam at each borehole location and using the excavated soil to extend the pad a further 3.0 m beyond the face of the dam slope, effectively making a 9.1 m by 6.1 m working pad. The slopes were restored by CES by replacing the excavated soil after completion of the installation of the VW piezometers and SI casing.

Borehole drilling was conducted between 07 and 17 January 2011. The field investigation program involved drilling three boreholes located on the downstream slope of the dam and two boreholes located along the centreline of the existing spillway situated on the west side of the dam. The boreholes were drilled using a compact, track-mounted sonic drill rig operated by Boart Longyear Canada from Calgary, AB. Boreholes R2488-01 to R2488-03 were drilled to depths varying from 21.6 metres below ground surface (mbgs) to 24.3 mbgs, depending on the geological units encountered and soil conditions. The drill rig was equipped with a 102 mm diameter drill bit attached to the bottom of a core barrel from which continuous core soil samples were recovered. Water was used to lubricate the hole during drilling. Steel casing (152 mm in diameter in 1.5 m long segments) was used to protect the hole from sloughing in while the drill bit and core barrel was advanced inside the casing. Boreholes R2488-04 and R2488-05 were drilled to a depth of 10.7 mbgs using only the drill bits attached to the core barrel (i.e. no outside casing was required).

At each 1.5 m interval during drilling, the core barrel carrying the soil sample was extracted from the borehole and the core samples were recovered by vibrating the samples into a plastic bag. The samples were then laid on a table and visually logged by MDH personnel at the time of drilling. Relatively undisturbed core samples were repacked and labelled for laboratory testing. Disturbed samples of interesting materials (e.g. bentonite) were bagged and labelled for laboratory review. Split spoon samples from the Standard Penetration Tests (SPT) were also wrapped and labelled after being visually examined.

In addition, gamma radioactive particle counts were logged for the boreholes drilled on the downstream slope of the dam using a digital geophysical logging system (Mount Sopris Matrix) after completion of visual logging.

SPTs were conducted at approximately 3 m intervals and pocket penetrometer tests were conducted at approximately 1.5 m intervals during drilling.

A summary of the borehole completion information is provided in **Table 3-1**. The borehole locations are shown on Figure 1-2.

Table 3-1 – Borehole completion summary.

Borehole Number	Date Drilled	Borehole Depth (m)	Borehole GPS Location			Instrumentation
			Northing	Easting	Elevation	
R2488-01	07-Jan-11	21.6	5575736	330051	720	VW piezometers and SI casing
R2488-02	12-Jan-11	22.9	5575678	329759	720	VW piezometers and SI casing
R2488-03	14-Jan-11	24.3	5575611	329406	726	VW piezometers and SI casing
R2488-04	17-Jan-11	10.7	5575673	329155	729	none
R2488-05	17-Jan-11	10.7	5575428	329134	722	none

3.2 VW Piezometers and SI Casing Installation

To provide pore water pressure information for the stability analysis and long-term monitoring, three VW piezometers, manufactured by Durham Geo Slope Indicator (DGSI), were installed in a stacked fashion at each of the three boreholes drilled on the downstream slope of the dam. Before each installation, the VW piezometers were visually examined to ensure that there was no damage to them. The label on each VW piezometer was checked against the number on the wire cable. A zero reading of each VW piezometer was recorded using a VW Data Recorder before installation.

The direct grout-in method was used to install the VW piezometers. The VW piezometers were first wrapped in geotextile and bound with electrical tape for protection prior to being lowered into the borehole. The piezometers were then attached to SI casing at desired depths based on identified stratigraphic units and were lowered into the borehole. The piezometers were grouted into place with cement-bentonite grout. A summary of the VW piezometer installation details is presented in Table 3-2. A schematic of the VW piezometer installations are provided in

Appendix A. The VW piezometers were calibrated by the manufacturer and the calibration sheets are provided in Appendix B.

Table 3-2 – Summary of VW piezometer installation details.

Borehole Number	Vibrating Wire Piezometer		Piezometer Tip Installation Depth	Elevation of Piezometer Tip	Target Stratigraphic Unit
	ID	S/N	mbgs	(masl)	
R2488-01	R2488-01A	10-5594	16.6	704.7	vulcanic ash
	R2488-01B	10-5593	12.8	708.5	contact, ox & unox shale
	R2488-01C	10-5452	10.1	711.2	base of silt
R2488-02	R2488-02A	10-4651	16.9	704.9	vulcanic ash
	R2488-02B	10-5573	12.6	709.2	base of sand/silt
	R2488-02C	10-5449	5.6	716.2	base of fill
R2488-03	R2488-03A	10-4718	20.3	701.9	vulcanic ash
	R2488-03B	10-4652	14.5	707.7	contact, ox & unox shale
	R2488-03C	10-5574	11.3	710.9	vulcanic ash
	R2488-03D	10-5453	5.4	716.8	base of fill

As mentioned above, the installation of SI casing in the boreholes was conducted simultaneously with the installation of the VW piezometers. The SI casing, manufactured by Roctest Ltd., was made of ABS pipe with an 85 mm outside diameter. Four orthogonal grooves are orientated longitudinally along the inside surface of the casing for running the inclinometer probe. A typical SI casing connection included a bottom cap, casing and a top cap. During the installation, the bottom cap and adjacent sections of SI casing were connected by pushing them together and rotating a pre-installed threaded locking collar with o-ring. The casings were filled with water to resist the buoyancy encountered when lowering the SI casing into the borehole. Once the SI casing was lowered into the borehole, one set of grooves was aligned parallel to the longitudinal axis of the dam. Weights were then added on top of the casing to eliminate the effect of increased buoyancy during grouting. A schematic showing the SI casing installation details is provided in Appendix A.

4.0 Site Characterization

4.1 Physiographic Setting

The Highfield Dam and its reservoir are situated in the Rushlake Creek Valley approximately 10 km south of the Village of Rush Lake. The dam site is situated at the boundary between the Cypress Hills Uplands and Missouri Coteau Upland Physiographic Sections, both of which are part of the Alberta High Plains Physiographical Region. The site is contained within the Vermilion Hills Physiographic Subsection of the Missouri Coteau Upland. The Swift Current Escarpment Physiographical Subsection of the Cypress Hills Upland is situated to the west and south of the site and the Rush Lake Moraine Physiographical Subsection of the Missouri Coteau Upland is situated to the north of the site.

The Vermilion Hills are characterized by Ayres et al (1985) as being gently to moderately rolling moraines (glacial landforms) with local glacio-fluvial and glacio-lacustrine deposits, and with elevations ranging from 730 masl to 780 masl and occasionally to 825 masl. The Vermilion Hills with has limited external drainage to the Swift Current and Rushlake Creeks. The adjacent Swift Current Escarpment is characterized by Ayres et al (1985) as sloping to the east from an elevation of approximately 825 masl to approximately 730 masl along its eastern boundary. Furthermore the Swift Current Escarpment has gently undulating to steeply sloping moraines.

4.1.1 Soils

The surficial soils (pedology) surrounding the Highfield Dam were mapped and reported by Ayres et al (1985). The alluvium, soils map symbol Av6:c, in the channel immediately to the north of the Dam is unpatterned with the predominately clay (surficial) soil present being gleysolic (developed in poor drainage conditions) and is carbonated and/or salinized, probably due to groundwater discharge. The surficial soils mapped on the east and west valley walls are Hillwash Complex (Hw), a complex soil derived from colluvium and eroded materials. On the upland to the east of the Dam the surficial soils are mapped as glacio-fluvial and glacio-lacustrine (Hatton, Ht, sandy, and Fox Valley, Fx2, silty) in origin and are usually less than 1.2 m thick, and which unconformably overlies glacial till. On the upland to the west of the Dam the surficial soil is mapped as Fox Valley (Fx1), again a glacio-lacustrine deposit some 1.2 m or less in thickness, which unconformably overlies glacial till.

4.1.2 Rushlake Creek Characteristics

Rushlake Creek is a meandering, underfit stream developed in a former meltwater channel, the Neidpath Channel, in a sidehill position. The present Rushlake Creek flows northwest from the Dam. Downstream from the Dam the Rushlake Creek flows into the Neidpath Channel from the southwest, entering the south end of the Highfield Reservoir, where the creek turns to the northwest.

The Rushlake Creek Valley is incised some 35 m below the surrounding uplands at the Dam site. The slope of the valley floor to the north is in the order of 0.1 percent. The approximate slope of the valley walls just downstream of the dam is 2.5 degrees and 6.9 degrees, for the west and east valley slopes, respectively.

4.2 Subsurface Conditions

4.2.1 Regional Geology

The geological deposits and groundwater resources in the Swift Current area are documented in the Saskatchewan Research Council (SRC) report, Maathuis and Simpson (2007). The complete schematic stratigraphy of the region is provided in Figure 4-1 and for the Quaternary in Figure 4-2 both adapted from Maathuis and Simpson (2007).

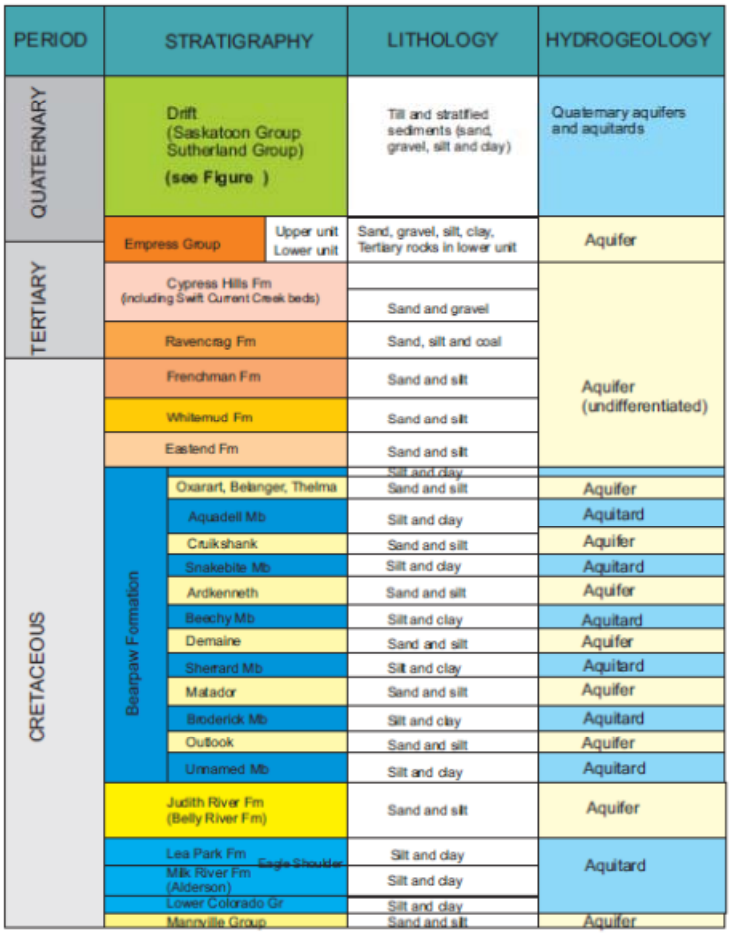


Figure 4-1 - Schematic, overall stratigraphy of the Swift Current area, adapted from Maathuis and Simpson (2007)

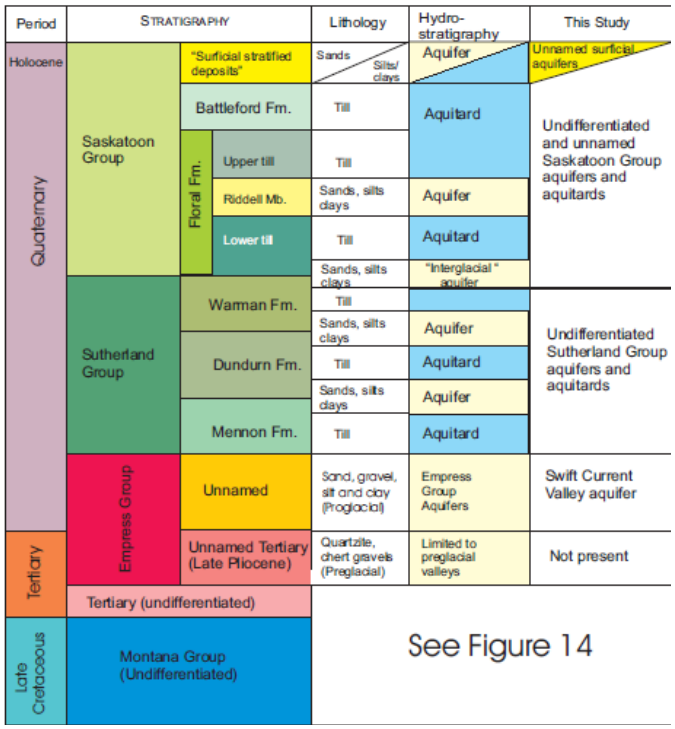
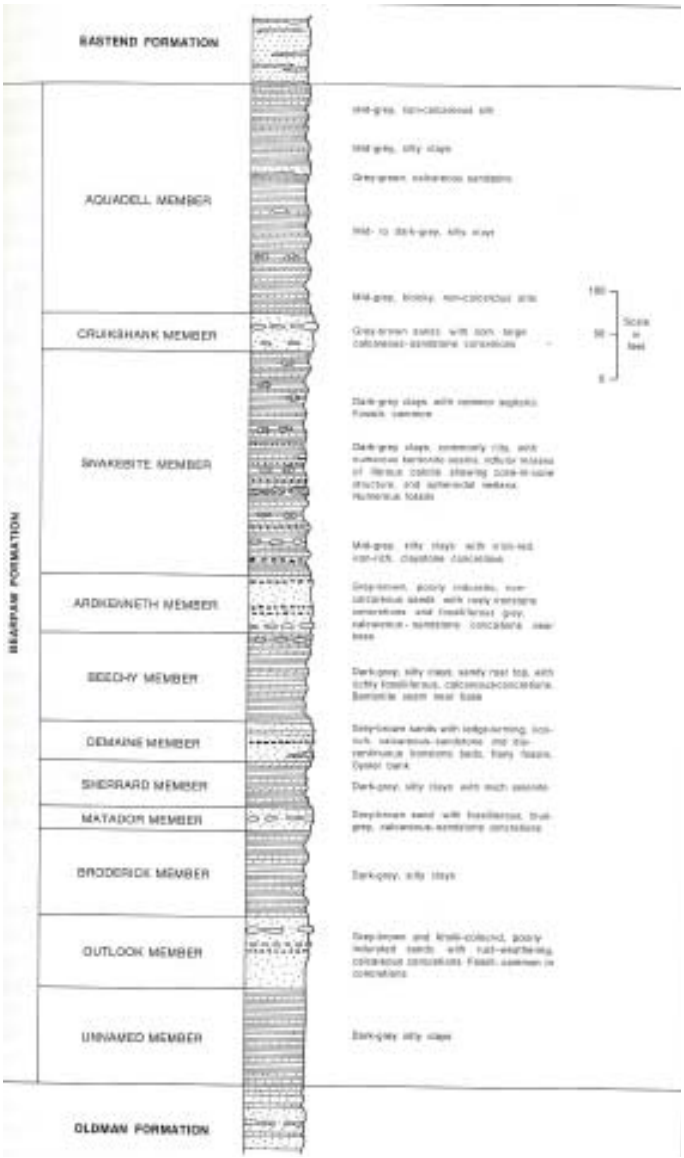


Figure 4-2 - Schematic Quaternary stratigraphy of the Swift Current area, adapted from Maathuis and Simpson (2007).

The most significant bedrock formations that underlay the site are the Bearpaw Formation, and its Aquadell and Snakebite Members. A schematic section of the Bearpaw Formation from the South Saskatchewan River Valley is shown in Figure 4-3, from Caldwell (1968). The Snakebite Member is generally characterized by Caldwell (1968) as fossiliferous, bentonitic, dark-grey clays and silty clays with numerous bentonite seams (yellowish) and occasional nodular calcite deposits. The Aquadell Member is similar to the Snakebite Member with the exception that it has few bentonite seams, or none at all, and may contain sandstone layers.

Figure 4-3 - Generalized section of the Bearpaw Formation from the South Saskatchewan River Valley, adapted from Caldwell (1968)

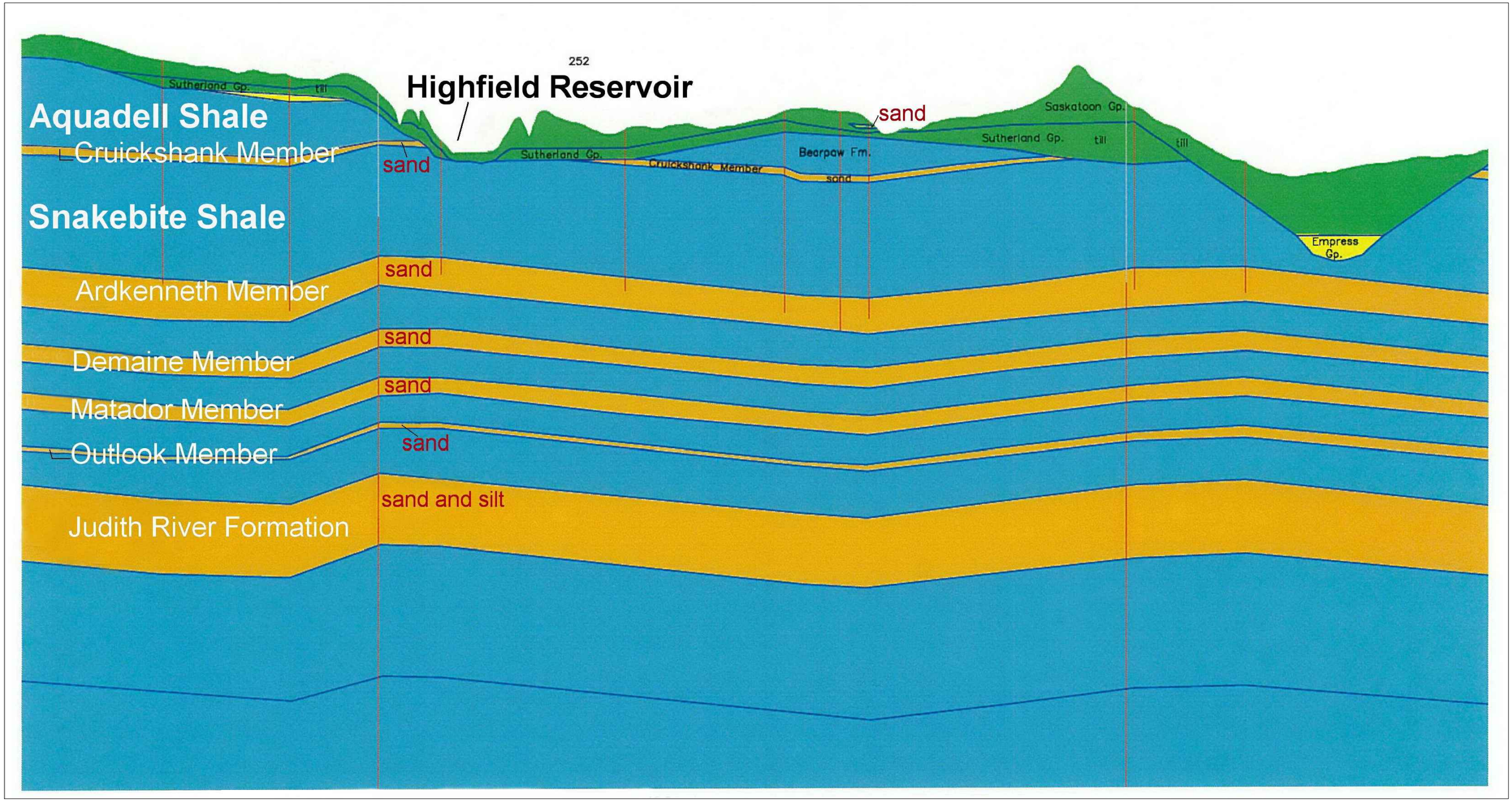


The regional stratigraphy in the vicinity of Highfield Dam Cross is shown in cross section I-I' (Maathuis and Simpson (2007), presented in Figure 4-4. This cross section shows that the Rushlake Creek Valley is eroded into the Bearpaw Formation's Snakebite Member (it underlies the Cruickshank Member), approximate bedrock contact elevation 715 masl, with the Aquadell Member (it overlies the Cruickshank Member) present in the valley walls.

The glacial deposits mapped as being present in the area are the Saskatoon and Sutherland Groups. These are composed of tills (aquitards) with occasionally granular intertill (between

glacial members) and/or entratill (within glacial members) deposits (aquifers) and when present these granular deposits can be of wide extent.

The glacial advances and readvances over the study area also developed ice thrust features due to glaciotectonics, Kupsch (1962), Sauer (1978) and Aber and Ber, (2007). The scale of glaciotectonic structures is illustrated in the schematic diagram, Figure 4-5, adapted from Aber and Ber (2007), and they are measured in millimetres to hundreds of kilometres.



No.	REVISION	SCALE NOT TO SCALE		DATE
		DESIGN BY	D. MIHIAL, M.A.Sc., P.Eng.	21-MAR-11
		DRAWN BY	S. RUSSELL, B.Sc.	21-MAR-11
		APPROVED BY		

CLIENT
nhc northwest hydraulic consultants

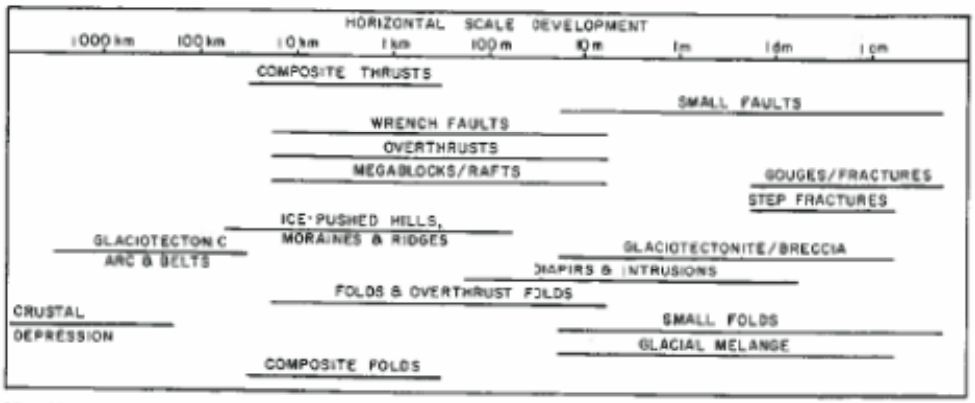
PRODUCED BY
MDH ENGINEERED SOLUTIONS

TITLE
SRC CROSS SECTION I–I'

PROJECT No.	R2488-265010	FIG. No. 4-4
DRAWING No.	R2488-63-2	REV.

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\2. Drafting\2. Drawings\R2488-63-2.dwg

Figure 4-5 - Common glaciotectonic structures arranged according to their typical horizontal scale. (adapted from Occhietti, 1973)



Glaciotectonics severely affected the bedrock clay shale deposits on the prairies as these deposits are generally weak compared to other strata. As a result the presence of ice thrust glacially-disturbed bedrock results in added complexity in the assessment of a site's geology. Typically, a glacial advance directly towards an upland results in severe ice thrusting, an example being the Dirt Hills ice thrust features near Moose Jaw (Christiansen and Sauer, 1989). The Wisconsinan glacial lobes that flowed across the Highfield Dam site are known to have flowed predominately from the northwest as they curved around the Cypress Hills Upland and not directly upslope from the northeast, Christiansen (1959), Klassen (2002), and Ross et al (2009). As a consequence, the glaciotectonics expected at the Highfield Dam site would be significant, but would likely be significantly less than those observed at the Dirt Hills.

Glaciotectonic structures would be anticipated at Highfield Dam as the regional slope is increasing to the southwest, towards the Cypress Hills, which resulted in the southwest-ward advancing glacier developing strong compressive subsurface stress on the underlying strata. The upper portion of the Bearpaw Formation at the site, to some undetermined depth, would be expected to be disturbed with slickensides, breccias, fractures, and gouge zones; all pre-existing shear zones with lower, often residual, shear strengths. Other indications of disturbed clay shale and pre-existing shears is a substantial increase in water content and decreased density which are typically associated with zones with high Atterberg limits, frequently bentonite seams. In addition, disturbed clay shale can have unconformities, such as intercalated (inserted) till or other bedrock or non-bedrock strata, plus folds and faults within a bedrock clay shale.

4.2.1.1 Development of the Neidpath Channel

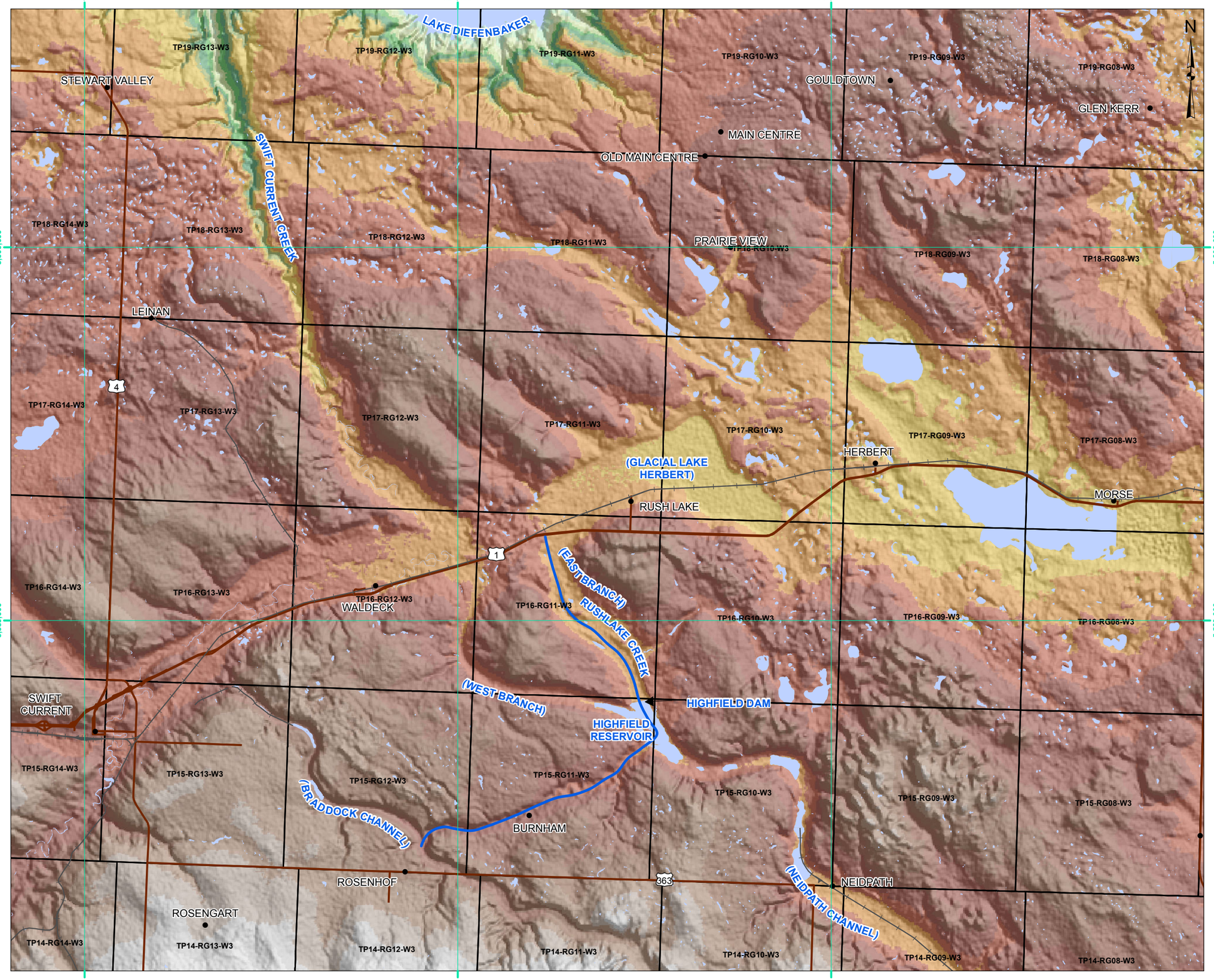
The Highfield Dam is situated in the Rushlake Creek Valley which was created by glacial meltwater which flowed south-easterly during the Wisconsinian deglaciation in the order of 15.5k years before present (BP), Christiansen (1959). The Rushlake Creek is the modern underfit stream that developed in the glacial meltwater channel after the meltwater flows ceased. The present day Rushlake Creek now flows northwest-ward, in the opposite direction to that of the glacial meltwater flows, which were southeast-ward.

The name given to the glacial meltwater channel, present day Rushlake Creek Valley, in which the Highfield Dam is situated, is the Neidpath Channel. At the Dam site, the Neidpath Channel has a “Y” shape. The two branches of the “Y” are referred to as the Western and Eastern Neidpath Branches for the purposes of this report and the stem of the “Y” is simply the Neidpath Channel. These are shown on Figure 4-6 According to Christiansen (1959) the Western Neidpath Branch and the Neidpath Channel developed after the Braddock Meltwater Channel, to the west, had ceased to function as a meltwater channel. The abandonment of the Braddock Channel was due to the Laurentide Ice Sheet having retreated northeast to a new position and subsequent meltwater flows had then begun to incise the Western Neidpath Branch and the Neidpath Channel as a sidehill, northwest to southeast trending, meltwater channel. A sidehill meltwater channel develops across the natural drainage direction, to the northeast in this case, as the glacier forms one side of the channel and blocks the natural drainage direction, in this case the northwest side. From Highfield Dam the Western Neidpath Branch/Neidpath Channel flowed south-eastward as it drained Glacial Lake Bigstick, situated just north of Swift Current, into a series of glacial lakes that developed along the southern margin of the glacier: Glacial Lakes: Kincaid, Gravelbourg and Old Wives Lake, Christiansen (1959), Christiansen (1979) and Klassen (2002). Later the glacier moved further to the northeast and the portion of the Western Neidpath Branch from Highfield Reservoir to the northwest (Town of Waldeck) was abandoned as the Eastern Neidpath Branch developed as a result of Glacial Lake Herbert developing and meltwater flows from it then incised the Eastern Neidpath Branch down to the Highfield Dam site and there it connected to the pre-existing Neidpath Channel. The Eastern Neidpath Branch connected Glacial Lake Herbert in the north to the before mentioned southern glacial lakes. Glacial Lake Herbert may have extended at some time(s) in its history some distance into the Eastern Neidpath Branch, or further, as the alluvium at the vicinity of the Dam is quite silty and clayey suggesting a glacio-lacustrine origin without strong erosive flows from north to south. As well, surficial glacio-lacustrine deposits (Ayres et al, 1985) on the uplands adjacent to the Dam

also indicate a relatively still water (lake) environment surrounded the site at some time, likely as part of an extended Glacial Lake Herbert. Christiansen (1959) indicates that a relatively level (flat) grade between the northern and southern glacial lakes is likely the reason that the Neidpath Channel and its branches were not deeply incised below the upland.

The western arm of the present day Rushlake Creek flows from the remnant of the Braddock Chanel eastward over to the Neidpath Channel and joins at the south end of the Highfield Reservoir. This western arm likely developed late in the Braddock Channel's history as the glacier retreated north-eastward.

The depth to which a meltwater channel is incised, how quickly it is incised, its gradient, meltwater flow volumes and floods, and the rate of erosion and/or alluvium deposition post glacially, are all factors in the development of landslides along the valley walls. The relatively shallow incision of the Neidpath Channel into the underlying Bearpaw Formation may be a contributing factor in the stability of the valley slopes at the Dam site.



LEGEND

- COMMUNITY
- RAILWAY
- MAJOR HIGHWAY

ELEVATION (masl)

High : 890 — Low : 550

NOTES:

1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 13N.
2. DIGITAL ELEVATION MODEL DERIVED FROM CANADIAN DIGITAL ELEVATION DATA (CDED) AND IS VERTICALLY ACCURATE TO + OR – 10 METERS.
3. GLACIAL MELTWATER CHANNELS AND GLACIAL LAKE BASIN INDICATED BY BRACKETS ON THE PLAN.

DRAFT

TITLE	REGIONAL TOPOGRAPHICAL FEATURES		
PROJECT No.	R2488-265010	FIG. No.	4-6
DRAWING No.	R2488-23-1		
CLIENT	nhc northwest hydraulic consultants		
PRODUCED BY	MDH ENGINEERED SOLUTIONS		
SCALE	1:200,000	DATE	
DESIGN BY			
DRAWN BY	S. LONG, GIS Cert.		08-MAR-11
APPROVED BY			

Path: P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\3. GIS\2. Drawings\R2488-23-1 (Topographic Map).mxd

4.2.2 Regional Hydrogeology

The regional aquifers are shown in Figure 4-7.

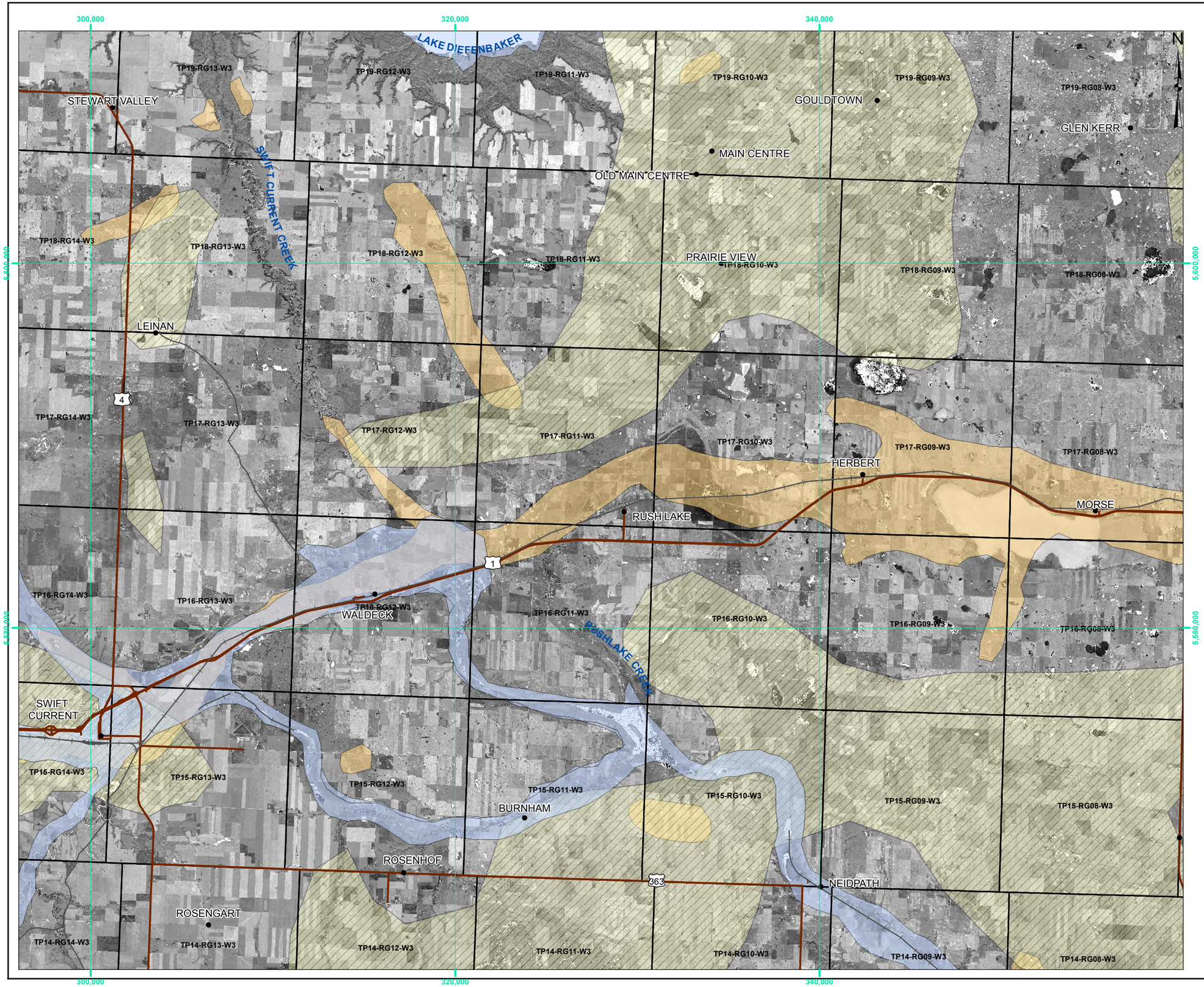
4.2.2.1 Bedrock Aquifers

The bedrock aquifers that are present in the Swift Current region are, in ascending order, the Judith River Aquifer, Bearpaw Formation aquifers and the Eastend-Cypress Hills Aquifer. The Eastend-Cypress Hills Aquifer is not mapped by Maathuis and Simpson (2007) as being present at the site. The Highfield Dam site is underlain by the Judith River Aquifer and five of the Bearpaw Formation aquifers as follows (in ascending order):

1. Outlook Aquifer, encountered at approximate elevation 460 masl,
2. Matador Aquifer, encountered at approximate elevation 500 masl;
3. Demaine Aquifer, encountered at approximate elevation 550 masl;
4. Ardkenneth Aquifer, encountered at approximate elevation 620 masl; and
5. Cruickshank Aquifer, encountered at approximate elevations 710 masl to 730 masl, which is close to the valley bottom elevation, 717.5 masl at the Dam.

The Judith River Aquifer, a major groundwater resource in the area, is encountered at approximately elevation 400 masl, some 325 mbgs at the dam site. The piezometric head in the Judith River Aquifer is not know in the vicinity of the Dam with the only reported data, elevation 700 masl from Maathuis and Simpson (2007), being from some 30 km south and southeast of the Dam. The Judith River Aquifer would not impact the Highfield Dam given its depth (in the order of 325 mbgs), the intervening Bearpaw Formation Aquifers, and an estimated piezometric level within the Judith River Aquifer of approximately 700 masl or less at the Dam site.

Only the Ardkenneth and Cruickshank Aquifers are deemed to impact or be impacted by the Highfield Dam and its reservoir. The estimated water level of the Ardkenneth Aquifer in the vicinity of Highfield Dam is approximate elevation 688 masl, which is well below the valley floor approximate elevation 717.5 masl. The Highfield Dam reservoir should be a source of recharge for the Ardkenneth Aquifer. The Cruickshank Aquifer could possibly discharge into the Rushlake Creek Valley and, if this was the case it could negatively impact the stability of the valley slopes.



LEGEND

- COMMUNITY
- RAILWAY
- MAJOR HIGHWAY
- ALUVIAL AQUIFER
- CRUIKSHARK AQUIFER
- EMPRESS GROUP

NOTES:

1. COORDINATE SYSTEM: NAD 1983 UTM ZONE 13N.
2. THE FOLLOWING AQUIFERS HAVE BEEN OMITTED FROM THE MAP FOR CLARITY AND ALSO BECAUSE THEY ARE EXTENSIVE (COVER MOST OF THE MAP AREA), IN ASCENDING ORDER THEY ARE:

   1. JUDITH RIVER AQUIFER

   2. BEARPAW FORMATION
      a. OUTLOOK AQUIFER;
      b. MATADOR AQUIFER;
      c. DEMAINE AQUIFER; AND
      d. ARDKENNETH AQUIFER

DRAFT

TITLE	REGIONAL AQUIFERS		
PROJECT No.	R2488-265010	FIG. No.	4-7
DRAWING No.	R2488-24-1		
CLIENT	nhc northwest hydraulic consultants		
PRODUCED BY	MDH ENGINEERED SOLUTIONS		
SCALE	1:200,000	DATE	
DESIGN BY			
DRAWN BY	S. LONG, GIS Cert.		08-MAR-11
APPROVED BY			

Path: P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\3. GIS\2. Drawings\R2488-24-1 (Aquifer Map).mxd

4.2.2.2 Glacial Aquifers

The Empress Formation Aquifer, a major bedrock/glacial aquifer in Saskatchewan, is not mapped by Maathuis and Simpson (2007) as being present in the immediate vicinity of Highfield Dam. A minor Empress Formation Aquifer of limited areal extent, approximately 2 km to 3 km in diameter, is mapped some 5 km to the southeast of the dam.

Aquifers of the Sutherland or Saskatoon Groups are also not mapped by Maathuis and Simpson (2007) as being present in the study area. However, minor glacial aquifers of limited extent may be present.

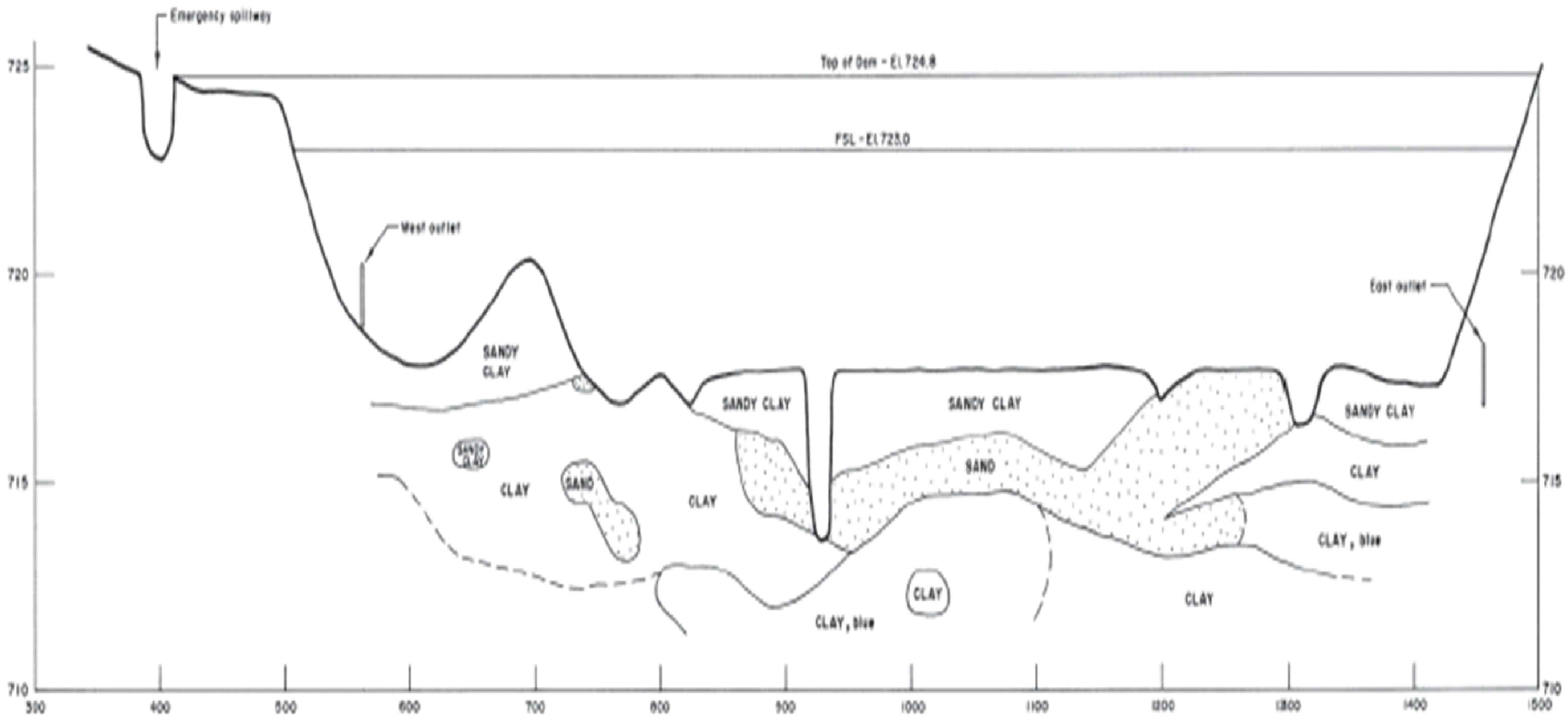
Sand deposits within the Rushlake Creek Valley alluvium would likely be considered aquifers. Maathuis and Simpson (2007) map the western portion of Rushlake Creek, the northwest portion of the glacial Neidpath Meltwater Channel (from the Town of Waldeck to the Dam), the Rushlake Creek Valley south of the Dam, and the Wiwa Creek Valley as all having surficial, glacio-fluvial deposits, which are therefore potential aquifers. However, from Highfield Dam northward no major glacio-fluvial deposits are mapped by Maathuis and Simpson (2007) in the Rushlake Creek Valley.

4.2.3 Local Geology

The strata encountered in the present MDH investigation as well as those encountered in recent PFRA investigations are summarized in Table 4-1 as well as on the borehole logs from MDH's recent field investigation in Appendix C and from historical investigations conducted by PFRA presented in Appendix D. A compilation of the gamma logs for the three boreholes drilled by MDH along the Dam's downstream side slope, Boreholes R2488-01, R2488-02 and R2488-03, are provided in Appendix C. The 1940's PFRA borehole logs were not available but a cross section complied by PFRA from them is shown in Figure 4-8.

Table 4-1 – Summary of subsurface information

Area of Interest	MDH Boreholes	PFRA Boreholes	BH Elevation masl	Dam Fill	Glacial Materials		Alluvium	Bedrock		Ground Water	Remarks
					Glacial Till	Stratified Drift		Oxidized Shale	Unoxidized Shale		
West Side of Dam	R2488-04		723.1					x	x	no	Existing west spillway
	R2488-05		723.0					x	x		
		BH46	723.0					x			
		BH47	731.7		x			x			
		BH48	726.4		x	x		x			
		BH49	725.8		x			x			
		C60			x	x					aggregate source
Along the Axis of the Dam (Valley Floor)	R2488-01		721.3	x			x	x	x	Y	Drilled on Dam downstream side slope
	R2488-02		721.8	x			x	x	x		
	R2488-03		722.2	x			x	x	x		
		C26	724.9	x			x	x	x		standpipe piezometers through Dam crest
		C27	724.8	x			x	x	x		
		C28	724.4	x			x	x	x		
		C29	725.2	x			x	x	x		
		C59	721.7	x			x	x		N	by west outlet
		BH34	716.9				x	x		Y	
		BH35	717.5				x	x			
		BH33	716.9				x	x			flowing hole
		BH36	717.8				x				
		BH37	717.8				x	? (likley)			
		BH38	717.7				x	x			
		BH39	717.4				x	x			
		BH32	717.5		x		x				part of shale tongue
		C24	720.1	x			x	x			near East outfall
East Side of Dam		C25	718.2		x	x		x		Y	near East outfall
		C30	719.0		x			x			
		C31	724.7	x				x			East abutment area
		C61			x	x (dry)					deep bench mark
		BH43	730.9		x					N	
		BH44	729.0		x						
		BH45	730.6		x						
		C50	731.0		x			x			
		BH42	728.9		x			x		Y	
		C51	728.8		x			x		N	
		BH41	728.4		x	x (wet)		x			seepage
		BH40	732.5		x			x			
		C52	731.7		x	x		? (likley)			cobbles
		C53	727.8		x	x (dry)		x			
		C54	725.3		x	x		x			
		C55	724.1		x	x		x			
		C56	722.5		x	x		x			
		C57	721.5		x	x (dry)		x			
		C58	720.0		x	x (dry)		x			
		C62	720.8		x	x		x		?	
		C63	719.4		x	x		x		Y	



PROFILE DAM ℄

No.	REVISION	SCALE NOT TO SCALE		DATE
		DESIGN BY	D. MIHIAL, M.A.Sc., P.Eng.	21-MAR-11
		DRAWN BY	S. RUSSELL, B.Sc.	21-MAR-11
		APPROVED BY		

CLIENT
nhc northwest hydraulic consultants

PRODUCED BY
MDH ENGINEERED SOLUTIONS

TITLE
PFRA CROSS SECTION ALONG THE AXIS OF HIGHFIELD DAM

PROJECT No.	R2488-265010	FIG. No. 4-8
DRAWING No.	R2488-63-2	REV.

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\2. Drafting\2. Drawings\R2488-63-2.dwg

The general sequence of strata present at the Dam and its vicinity, are described in descending order, are:

- Post Glacial Deposits:
  - Topsoil;
  - Alluvium deposits within the Rushlake Creek Valley. The shallow alluvium would be recent, post-glacial deposits while the deeper alluvium would have been deposited proglacial and therefore considered as part of the Saskatoon Group Surficial Stratified Drift. The full depth of the alluvium will all be considered as Surficial Stratified Drift as there is limited information available on which to differentiate the two deposits, plus they are conformable.
  - Possibly eolian (windblown) deposits on the west upland.

- Glacial Deposits:
  - Saskatoon Group:
    - Surficial Stratified Drift. As noted above the Surficial Stratified Drift forms a significant thickness of the alluvium. In addition, Surficial Stratified Drift deposits are mapped by Ayres et al (1985) as mantling the uplands flanking the Dam. This includes water lain deposits, glacio-fluvial and glacio-lacustrine, on the west and east sides.
    - Battleford Till;
    - Floral Till.
  - Sutherland Group

- Bedrock:
  - Bearpaw Formation:
    - Aquadell Member, silty shale aquitard;
    - Cruickshank Member, sandy aquifer;
    - Snakebite Member, silty shale aquitard.
  - Judith River Formation

The strata encountered are discussed in detail in the following subsections.

4.2.3.1 Topsoil

No surficial topsoil deposits were encountered in the MDH boreholes and only one topsoil layer was noted on a PFRA borehole log. An approximately 150 mm thick topsoil layer is shown under the Herbert Canal spoil/fill in PFRA Borehole C59 near the Dam's west outlet.

Organic soils were encountered intermixed within the dam's fill and also within the alluvium underlying the Dam in both the boreholes drilled for this investigation and for PFRA previous investigations.

4.2.3.2 Fill

Fill was encountered in all three boreholes drilled along the downstream side slope for this investigation, Boreholes R2488-01 to R2488-03. The depth of fill encountered varied from 6.5 m, 5.6 m to 5.2 m, in Boreholes R2488-01, R2488-02 and R2488-03, respectively. The depth of fill in the two PFRA boreholes drilled in 2002 through the centre of the Dam, C26 and C28, were estimated from their logs as being approximately 5.8 m and 4.5 m, respectively.

The fill can generally be characterization as medium plastic clay with occasional highly plastic clay zones. Some lenses and pockets of organic soils were encountered in the fill as well as a few granular lenses. The laboratory test results indicated average plastic and liquid limits of 18.6 (±3.59) percent and 46.7 (±3.54) percent, respectively, with the standard deviations shown in brackets. The average water content in the fill was 21.9 (±5.3) percent with a range of 14.2 to 34.2 percent. The water contents in the fill generally increased with depth. The Atterberg limit and water content results are similar to those shown on the PFRA borehole logs.

The fill generally was in a stiff to firm condition near the top of the boreholes and softened with depth. The SPT results had an average blow count (N) value of 7.2 (±3.0), which would correlate to an unconfined compression test result of approximately 92 kPa.

4.2.3.3 Alluvium

The alluvium encountered in the three MDH boreholes, R2488-01 to R2488-03, varied in thickness from 3.5 m, 6.3 m and 0 m, Boreholes R2488-01, R2488-02 and R2488-03, respectively. The alluvium encountered in the FPRA Boreholes, C26 and C28, drilled through the Dam, as well as the toe Boreholes: C32 to 39 and Borehole C59, varied considerably across the valley floor and ranged from 2.4 to 10.4 m in thickness.

The alluvium is composed of a wide range of soils from granular to highly plastic clays that are interlayered, interbedded, and interlaminated in locations. In addition, the alluvium has seams, layers and pockets of organics as well as zones of organic rich soils. Some very deep, soft and highly compressible organic deposits could be encountered within the old creek channel(s) and oxbows.

Standard Penetration Tests performed within the alluvium had blow counts (N) ranging from 0 to 13. The N = 0 blow count was recorded in a very loose sand layer at a depth of 10.9 m in Borehole R2488-02.

The granular alluvium was generally poorly graded, fine grained and silty and sometimes clayey, with only very occasional trace amounts of gravel being encountered. It was in a loose to medium dense and wet condition. The granular alluvium would be considered as liquefiable.

The silty alluvium was sandy and/or clayey and often with some organics. It was generally in a wet and soft to very soft condition. Layers within the silty alluvium could be liquefiable depending on its clay content.

The clayey alluvium was generally silty with some to little sand and would range from low plastic to highly plastic. It was in a very moist to wet and soft condition.

4.2.3.4 Saskatoon Group Surficial Stratified Drift

Besides the alluvium described above some Surficial Stratified Drift was recorded in PFRA Boreholes C60 and C48 on the west upland. In Borehole C60 1.5 m of slightly gravelly silty sand in a damp condition was overlain by approximately 0.4 m of very sandy, clayey gravel and cobbles. In Borehole C48 a 1 m layer of lacustrine medium plastic clay overlies approximately 2 m of slightly gravelly poorly graded, silty sand that is stated as being in a “dry” condition.

4.2.3.5 Glacial Till

As previously noted the glacial till encountered at the site cannot, as yet, be classified into either the Saskatoon or the Sutherland Groups.

The MDH boreholes did not encounter any naturally deposited glacial tills. The glacial till deposits reported in previous PFRA investigations varied considerably in thickness.

On the west flank, Boreholes C47 to C49 and C60, the glacial till encountered was approximately 2.5 m or less in thickness while on the east flank the glacial till reaches thicknesses of 11 m as described in the PFRA boreholes.

The glacial till on the west upland was typically medium plastic and very stiff to hard. It was described in PFRA reports as being significantly dry of its optimum water content.

The glacial till encountered in the PFRA boreholes on the east upland and along the centreline of the 2002 proposed spillway were generally medium plastic and sometimes highly plastic just above the contact with the underlying clay shale where the clay shale and glacial till have been reworked by the glaciers. The east side glacial till was typically very stiff to hard with a water content that increased with depth. The upper, approximately 2 m of the glacial till was described in PFRA reports as being dry of its optimum water content. Occasional stratified drift, either silty sands or sandy silts were encountered within the east side glacial till deposit(s). The stratified drift was generally damp and sometimes moist or wet. A 0.3 m thick, water bearing, poorly graded gravel layer was encountered immediately above the clay shale in PFRA Borehole C25.

4.2.3.6 Clay Shale

Members of the Bearpaw Formation were encountered at the Dam site. Whether the Aquadell Formation clay shale deposit, the granular Cruickshank Aquifer and/or the Snakebite Member are present is not certain as there has not been a deep stratigraphic borehole drilled on either the east or west uplands. Ideally two deep stratigraphic boreholes would be drilled on the east and west uplands, respectively, in order to establish the stratigraphy at the site. The boreholes would be drilled to a bedrock marker, the shallowest being the Ardkenneth Aquifer.

The Bearpaw Formation encountered was composed of clay shale with occasional: layers of sandstone, laminations of bentonite and beds of volcanic ash. The presence of sandstone layers and volcanic ash may indicate that the Bearpaw Formation encountered was the Aquadell Member. Volcanic ash has been reported in the area. A 0.6 m thick volcanic ash layer, whose properties were those of silt, was encountered in a PFRA auger hole drilled for a proposed dyke across the south end of the Highfield Reservoir as reported by Iverson (1948). A volcanic ash bed is reported by Worcester (1950) near Waldeck within the Bearpaw Formation, thickness in the order of 2.4 m as noted by Crawford (1951). A 1.8 m thick volcanic ash bed at Neidpath, presumably within the Bearpaw Formation, was reported by Crawford (1951).

The clay shale encountered was highly plastic, moist and occasionally moist to very moist near its contact with the overlying glacial deposits. The clay shale was oxidized to varying depths which are shown on MDH's borehole logs provided in Appendix C. The upper portion of the clay shale, to some undetermined depth, was reworked by glaciations (glaciotectonics) as previous described and exhibited breccias, fractures, disturbed laminations, slickensides and also intercalated glacial till and unoxidized clay shale within oxidized clay shale.

The oxidized clay shale encountered within Boreholes R2488-01 to R2488-03, under the Dam had average plastic and liquid limits of 28.4 (±3.44) and 72.9 (±6.07), respectively and a water content average of 37.3 (±7.7) percent with a dry density average of 1,282 kg/m^3 (±140). The unoxidized clay shale encountered under the Dam had average plastic and liquid limits of 27.9 (±3.5) and 76.0 (±9.7), respectively and a water content average of 37.9 (±9.34) percent and its dry density averaged 1,424 kg/m^3 (±87). The SPT blow counts, N, for the oxidized and unoxidized clay shale under the Dam averaged 19.3 and 16.2, respectively.

The oxidized clay shale encountered in the existing west spillway, Boreholes R2488-04 and R2488-05, had water contents that averaged 35.7 (±9.18) percent (%) and its dry density averaged 1,348 kg/m^3 (±118). Approximately one metre of unoxidized clay shale was encountered in each of these boreholes underlying the oxidized clay shale. The water content of the unoxidized clay shale encountered was about 32.7 percent and the dry density was approximately 1,383 kg/m^3. The SPT blow counts, N, for the oxidized clay shale averaged 24.8 (±15.6) and 18 (one test) for the unoxidized clay shale.

The volcanic ash, when encountered, was found to be non-plastic and silt like as evidenced by three Atterberg limit tests performed on samples from Boreholes R2499-01 and R2488-03. Five natural moisture content tests performed on the ash gave an average of 56.4 percent (%) and ranged from 48.6 percent (%) to 66.3 percent (%). The ash was generally light grey in colour and tended to be somewhat crumbly, being neither a hard brittle material (desiccated), nor soft and soupy (flowing).

Evidence of glaciotectonics, ice thrusting, was encountered in the boreholes drilled for this investigation and within boreholes drilled in previous PFRA investigation. The oxidized clay shale in all three MDH boreholes, Boreholes R2488-01 to R2488-03, was reworked with occasional breccias, distorted laminations and fracture zones being observed. In Borehole R2488-03 an intercalated (inserted) glacial till was encountered within the clay shale at a depth

of 9.3 mbgs, elevation 712.85 masl. Glacial till was intermixed in the upper portion of the clay shale in Borehole R2488-02. In addition, volcanic ash deposits which should have been found at the same contact depth within the three boreholes were unaligned. Some of the misalignment could be due to the effects of landsliding along the west valley wall, see Subsection 4.2.4. Unoxidized clay shale was also found within oxidized clay shale in Borehole R2488-04, drilled within the existing west side spillway, at a depth of 8.8 mbgs (below the spillway floor) to 9.0 mbgs, elevation 714.3 masl to 714.1 masl.

The PFRA cross section, Figure 4-8, shows significant glaciotectonics across the Dam site due to the following:

- The upper part of the clay shale was described in numerous PFRA boreholes as being reworked. On the uplands adjacent to the Dam, the Atterberg limits of the glacial till immediately above the clay shale are typically highly plastic while the upper clay shale Atterberg limits are often quite low as compared to deeper clay shale, an indication that this contact zone between glacial till and clay shale has been reworked. In addition clay shale fragments are often reported intermixed within the glacial till deposits;
- Sand deposits were occasionally encountered within the clay shale;
- A possibly vertical contact between oxidized and unoxidized clay shale; and
- Unoxidized clay shale within oxidized clay shale and unoxidized clay shale overlying oxidized clay shale.

4.2.4 Landslides

A review of some air photos of the area reveals that landslides have developed along some sections of the Rushlake Creek and Wiwa Creek Valley walls, the Neidpath Channel. The Highfield Dam itself was constructed with its west abutment on a slump block as indicated on air photos taken prior to when the Dam was constructed, and a “shale tongue”, where shale from a landslide is intruded into a glacial or alluvial deposit, is mapped by PFRA within the alluvium under the east abutment.

The landslides most probably developed within bentonitic rich zones of the Snakebite and/or Aquadell Members of the Bearpaw Formation and triggered by the down cutting of the Neidpath Channel. Landslides that develop in the Bearpaw Formation tend to be translational,

retrogressive landslides that can have multiple slump blocks; all developed along planar shear zones within highly bentonitic or almost pure bentonite laminations/seams.

4.2.5 Local Hydrogeology

The piezometric conditions measured under the Dam are plotted in Figure 4-9, and the field data is provided in Appendix E.

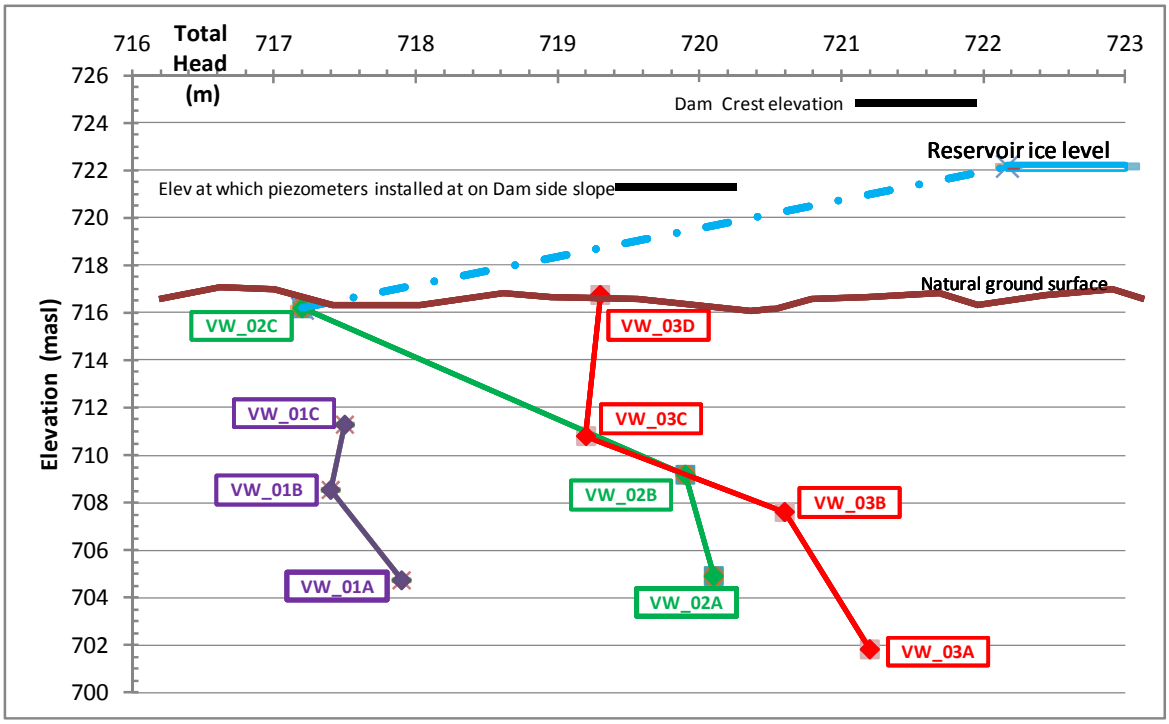


Figure 4-9 - Piezometric conditions at Highfield Dam, February to June, 2011.

Figure 4-9 shows that the groundwater at the site has an upward gradient. The up-gradient of the groundwater flow will have a negative impact on slope stability that will have to be addressed during detailed design and construction.

4.2.5.1 Piezometric Conditions Within the Alluvium

A flowing PFRA borehole, Borehole C33, was located near the centre of the valley. This borehole is likely intercepting a confined aquifer within the alluvium. The groundwater levels recorded in the other PFRA boreholes, Boreholes C32 to C39, were within approximately 1 m of the ground surface, approximate elevation 717.5 masl, as summarized in Table 4-2.

Table 4-2 - Piezometric conditions within the alluvium as reported in the PFRA Dam toe boreholes, September, 2009

PFRA BH Number	Natural Ground Elevation (masl)	Depth Drilled (m)	Thickness of Alluvium Over Shale (m)	Shale Contact Elevation (masl)	Depth to Groundwater (mbgs)	Groundwater Elevation (masl)	Date Measured	Remarks
C59	719.5	7.5	2.4	717.1	dry		7-Oct-09	2.2 m of fill over natural grd.
BH34	716.9	7.0	6.0	710.9	0.72	716.2	22-Sep-09	
BH35	717.5	8.5	8.5+		1.35	716.2	22-Sep-09	
BH33	716.9	10.0	10.0+		flowing		N/A	drilled 09 Sept. 2009
BH36	717.8	8.5	8.5+		0.29	717.5	23-Sep-09	
BH37	717.8	7.0	7.0+		0.75	717.1	23-Sep-09	shale fragments at bottom
BH38	717.7	7.0	6.5	711.2	0.82	716.9	23-Sep-09	
BH39	717.4	7.0	6.0	711.4	0.88	716.5	23-Sep-09	
BH32	717.5	7.0	4.0	712.0	0.95	716.6	9-Sep-09	Part of shale "tongue". Glacial Till, 1.5 m thick, encountered under alluvium and over shale
	717.7	. = average elevation				**716.7**	. =average groundwater elevation	

Notes:

1. Boreholes listed from west to east and their locations are shown on Figure __
2. BH C59, depth to shale is calculated from the natural ground surface

4.2.6 Piezometric Conditions Within the Dam Fill

The piezometric level within the Dam fill is indicated by piezometers: R2488-02C and R2488-03D, which are located close to the bottom of the fill. The piezometric level is approximately elevation 718.3 masl, and ranges from 717.2 masl to 719.3 masl. This piezometric level is approximately 6.6 m below the crest of the Dam (elevation 724.9 masl). The Highfield Reservoir ice level elevation was 722.17 masl at the time of the survey. The piezometric level within the Dam fill would be expected to fluctuate in relationship to the Highfield Reservoir level.

4.2.6.1 Piezometric Conditions Within Glacial Tills on the Uplands and Valley Walls

The five PFRA boreholes drilled on the west upland area, Boreholes C46 to 49, drilled to 4 mbgs to 5 mbgs, and C60, drilled to 10 mbgs, were all reported as being dry at the time of drilling them. The west upland may have a low watertable due to its location between the Western Neidpath and Eastern Neidpath Branches. If this is the case, it could have positive implications for the west valley wall stability.

A groundwater source(s) may be present within the east upland/valley slope. Three PFRA boreholes, Boreholes C25, C42 and C63 both had some groundwater observed within them when they were drilled, yet other boreholes were observed to be dry. The location of Borehole

C25 is not known with certainty, but as it was drilled for the 1991 west outlet concrete repair project presumably it is located near the west outlet outfall. In Borehole C25 seepage was observed with the glacial till at a depth of approximately 2 mbgs, elevation 716.2 masl and at a depth of 2.6 mbgs, elevation 715.6 masl, a 0.3 m thick wet gravel layer was encountered above the clay shale contact. Borehole C42 is located approximately 60 m upstream of the Dam, adjacent to the Reservoir, and it is not reported as having encountered any granular seams or lenses. The groundwater level reported in Borehole C42 is approximately elevation 721.24 masl which may correlate to the reservoir water level. Borehole C63 is located approximately 60 m downstream of the Dam and C63 and an approximately 2 m thick moist silty sand layer (SM) was encountered only 0.9 mbgs, some 1.5 m above the clay shale contact with a medium plastic clay layer intervening. The presence of groundwater within some boreholes or at some locations along the east side may be due to:

- Isolated water bearing strata;
- The influence of the Highfield Reservoir, which might explain the seepage observed in both Boreholes C25 and C42;
- The presence of the Cruickshank Aquifer.

4.2.6.2 Piezometric Conditions Within the Bearpaw Formation

The piezometric data from the vibrating wire piezometers installed in Boreholes R2488-01 to R2488-03, shows that there is an upward gradient from within the Snakebite Member of the Bearpaw Formation towards the surface (see Figure 4-9). An upward gradient would have negative implications for the stability within the Bearpaw Formation under the Dam. While there is an upward gradient, little flow would be expected from the underlying aquifers as the hydraulic conductivity of the clay shale aquitards is very low, with the possible exception that the Cruickshank Aquifer may be discharging within the lower valley slopes. Seepage along joint or fracture systems would be possible.

4.3 Laboratory Testing

4.3.1 Basic Soil Characteristic Tests

The laboratory testing program was directed at determining the index and engineering properties of the soils encountered at the project location. All the samples obtained were stored in the moisture room at MDH's geotechnical laboratory in Saskatoon, SK. The laboratory tests

were performed in accordance with ASTM test procedures. The laboratory testing program is summarized in Table 4-3.

Table 4-3 –Summary of the laboratory testing program

Laboratory Test	ASTM Standard	Number of Tests
Atterberg Limits	ASTM D4318	26
Direct Shear	ASTM	1
Water Content	ASTM D2216	107
Grain size analysis including hydrometer	ASTM D422	3
Unit Weigh (bulk density)	ASTM D7263	31

The detailed laboratory test results are provided in Appendix F. The water content, Atterberg limits, and dry density test results are also annotated on MDH's borehole logs presented in Appendix C.

4.3.2 Clay Shale Effective Shear Strength Parameters

Effective shear strength parameters, namely effective cohesion (c') and the angle of effective shearing resistance (internal friction angle) (ϕ') for the clay shale were obtained from the direct shear test (ASTM D3080-90). One sample from the oxidized clay shale, with Atterberg limits of liquid limit of 89.1% and plastic limit of 28.5% and a natural water content of 34.4%, encountered in Borehole R2488-03 was tested in a remoulded state so a peak strength is not obtained. The shear strength values are summarized in Table 4-4.

Table 4-4 – Clay shale effective shear strength parameters

Borehole Number	Sample Number	Depth mbgs	Elevation masl	Peak		Residual		Grain Size		Remarks
				Effective Cohesion c' (kPa)	Effective Angle of Shearing Resistance ϕ'	Effective Cohesion c' (kPa)	Effective Angle of Shearing Resistance ϕ'	Sand (%)	Silt and clay (%)	
BH2488-03	FGL-61A	10.8	711.4	na		0	25	23	77	remolded

5.0 Spillway Alternatives

Six alternatives for the new spillway design were presented by NHC. They are as follows:

1. Ungated labyrinth weir spillway on the east abutment;
2. Gated spillway on the east abutment;
3. Earth spillway on the east abutment;
4. Ungated labyrinth weir spillway on the west abutment;
5. Gated spillway on the west abutment; and
6. Earth spillway on the west abutment.

Drawings showing the details of the six alternatives are included in Appendix G.

6.0 Slope Stability Analyses

A geotechnical analysis software package, GeoStudio 2007, was used for the numerical modelling. This software package is capable of conducting seepage, stress and slope stability modelling in one software package.

Two of the GeoStudio 2007 software packages; SLOPE/W and SEEP/W were used in this analysis. SEEP/W is a finite element software package, which was utilized to compute the pore water pressures from steady-state analysis due to varying reservoir elevation. SLOPE/W is a limit equilibrium analysis software package, which was coupled with SEEP/W to incorporate pore water pressures in the stability analysis. SLOPE/W utilizes limit equilibrium theory for calculating the factor of safety (*Fs*).

The factor of safety is defined as the ratio of resisting forces (M_R) to the driving forces (M_A) as shown in Figure 6-1. The resisting forces are primarily the mobilized shear strength (S_{UMOB}) along the length of failure plane being analyzed (*L*). Driving forces are the weight of the embankment (*W*) and the thrust from the embankment (P_{fill}). On the upstream face of the embankment the hydrostatic force is added to the resisting forces.

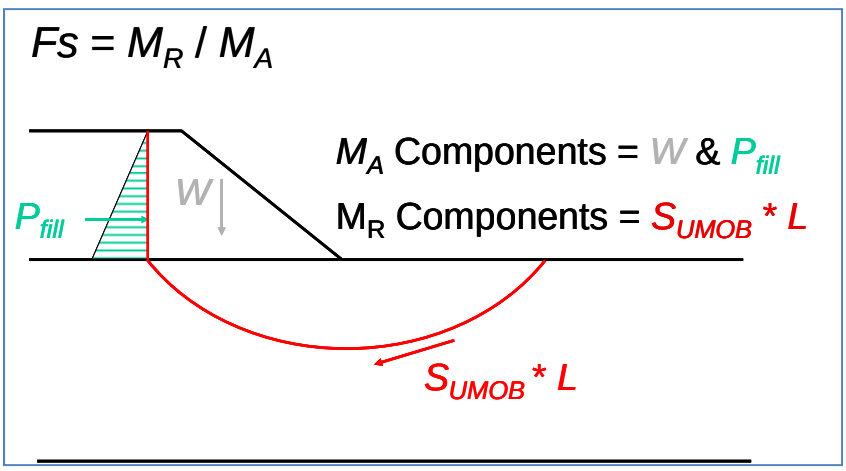


Figure 6-1 - Forces Acting Upon a Slip Surface

A pre-design stability analysis was completed on the upstream and downstream dam face at the three instrument lines to determine a theoretical factor of safety (*Fs*) at different dam elevations. The locations of the stability sections utilized in this investigation are shown on Figure 1-2.

The stability analysis required defining the following items:

1) Surface topography;
2) Stratigraphy;
3) Material properties;
4) Hydraulic conditions (pore water pressures);
5) Seismic conditions;
6) Failure mechanism; and
7) Method of analysis.

The objective of the numerical modelling was to determine the Factor of Safety (*Fs*) for the existing dam and evaluate the *Fs* against the dam stability requirements outlined in the 2007 CDA Guidelines. The slope stability requirements under static and seismic conditions from the 2007 CDA Guidelines are provided in Table 6-1 and Table 6-2.

Table 6-1 – Slope stability factors of safety for static conditions (2007 CDA Guidelines).

Loading Conditions	Minimum Fs Note 1	Slope
End of construction before reservoir filling	1.3	Upstream and Downstream
Long-term (steady state seepage, normal reservoir level)	1.5	Downstream
Full or partial rapid drawdown	1.2 to 1.3 Note 2	Upstream

Note 1. Factor of Safety (Fs) is the factor required to reduce operational shear strength parameters in order to bring a potential sliding mass into a state of limiting equilibrium, using generally accepted methods of analysis.
Note 2. Higher *Fs* may be required if drawdown occurs relatively frequently during normal operation.

Table 6-2 - Slope stability requirements for seismic conditions (2007 CDA Guidelines).

Loading Conditions	Minimum Fs	Slope
Pseudo-static	1	Upstream and Downstream
Post-earthquake	1.2 - 1.3	Upstream and Downstream

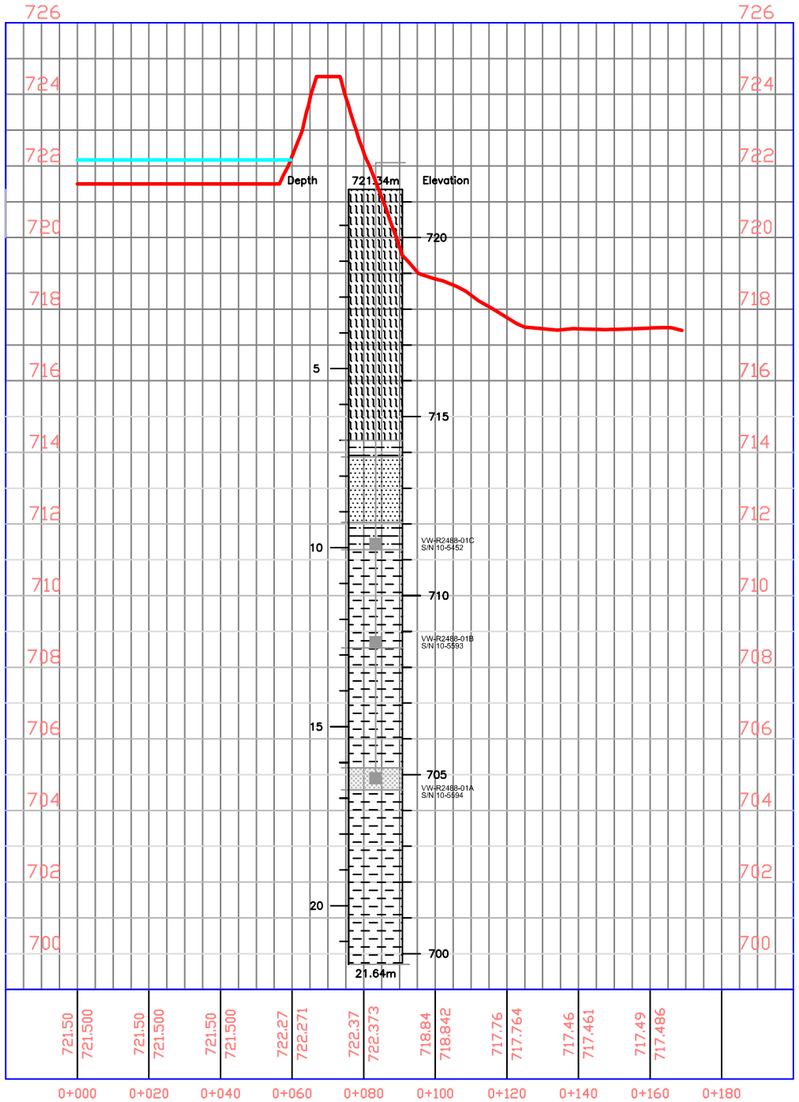
The scope of the work was to evaluate the *Fs* for different reservoir elevations, rapid reservoir drawdown, and seismic loading.

6.1 Surface Topography

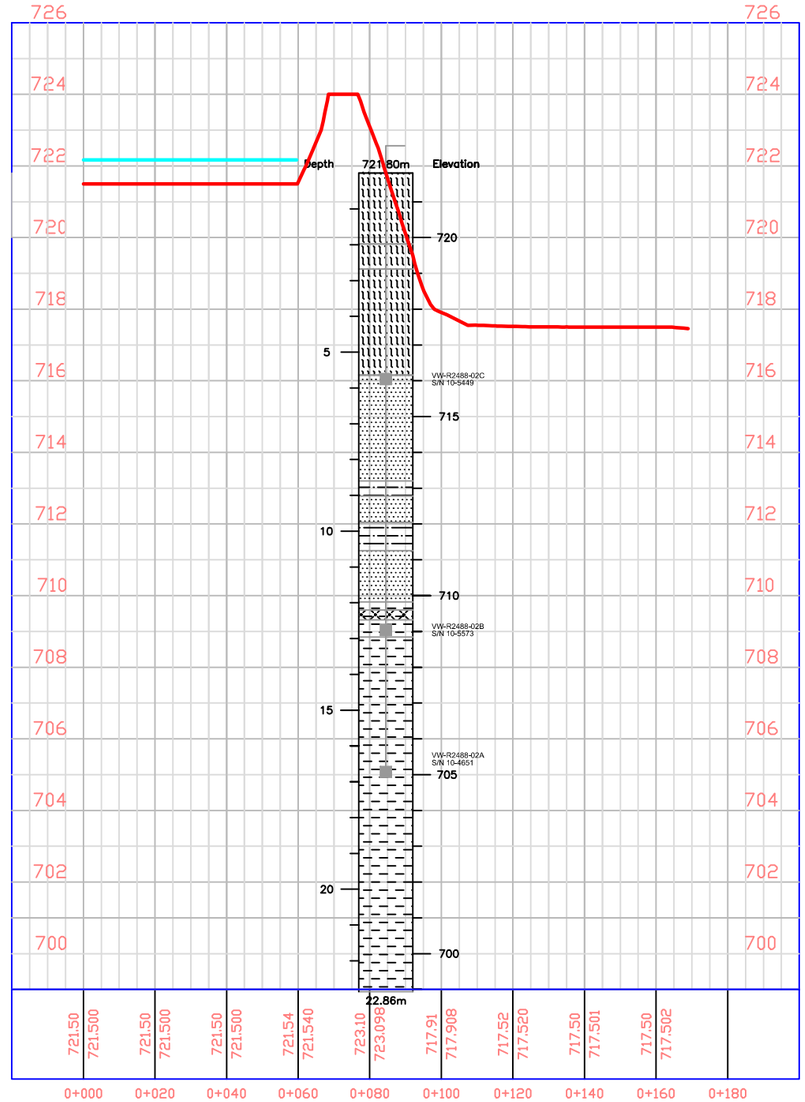
The surface topography and existing geometry was inferred from LIDAR and previous topographical survey data provided by AESB.

Three cross-sections (A-A, B-B' and C-C') corresponding to the three boreholes that were drilled along the downstream slope of the dam, were evaluated for the stability analysis. The locations of the three cross-sections are shown on Figure 1-2. Individual cross-sections showing the detail at each section along with the monitoring instrumentation are shown in Figure 6-2.

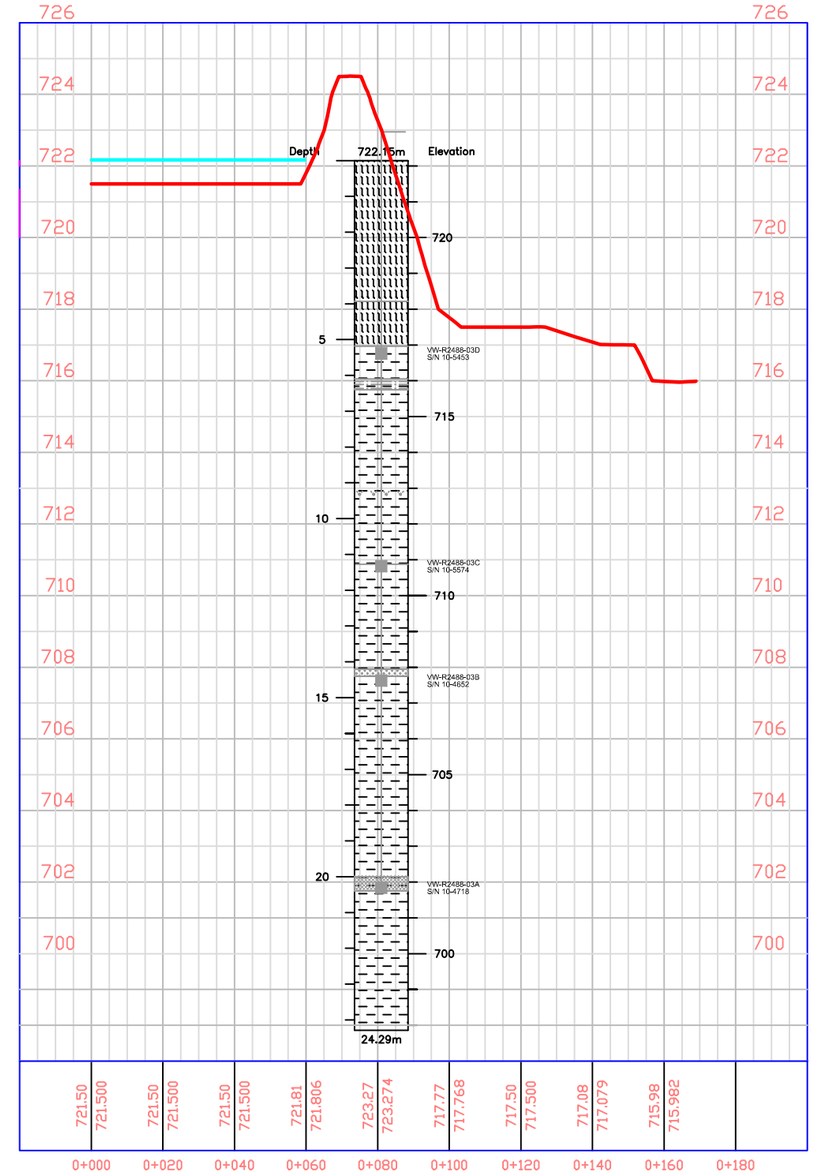
A-A′ WITH R2488-BH-01



B-B′ WITH R2488-BH-02



C-C′ WITH R2488-BH-03



LEGEND

No.	REVISION	SCALE NTS		DATE
		DESIGN BY	D. MIHIAL, M.A.Sc., P.Eng.	17-MAR-11
		DRAWN BY	S. RUSSELL, B.Sc.	17-MAR-11
		APPROVED BY		

CLIENT
nhc
northwest hydraulic consultants

PRODUCED BY
MDH ENGINEERED SOLUTIONS

TITLE
CROSS SECTIONS WITH BOREHOLES

PROJECT No. 2488-265010 | FIG. No.

DRAWING No. R2488-63-1 | REV.

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\2. Drafting\2. Drawings\R2488-91-1 and 63-1.dwg

6.2 Stratigraphy

Information obtained from MDH's recent stratigraphic drilling investigation and from AESB's historic borehole logs were utilized to construct the stratigraphy for the numerical models. The general stratigraphy of the dam and underlying soils was divided into five (5) primary units:

- Fill;
- Alluvium;
- Intact Oxidized Shale;
- Disturbed (Reduced) Strength Oxidized Shale;
- Intact Unoxidized Shale.

6.3 Material Properties

6.3.1 Stability Parameters

The stability analysis included in this report required inputs for bulk unit weight (γ_b), effective friction angle (ϕ') and apparent cohesion intercept (c'). Table 6-3 summarizes the material parameters used in this study.

The upper few meters of the Cretaceous Bearpaw Formation shale in southern Saskatchewan have typically been sheared and softened to a residual strength state by glacial erosion. The value of the residual friction angle has been reported as low as 6.5° to 8° (Sauer, 1985). MDH conducted a direct shear test on a sample within the disturbed shale which yielded a residual friction angle of 25°. It was felt that this value did not represent the disturbed shale because of the high sand content that was noted in the sample. The sample was tested because it had the highest liquid limit of all the samples tested. The high liquid limit of the sample is indicative of a bentonitic soil. The disturbed Cretaceous shale was overconsolidated due to the recompression applied by subsequent glaciations (Sauer, 1993). It is generally accepted that overconsolidated clay will have a cohesion intercept. Consequently, in the case of the disturbed and recompressed clay shale at the site an effective cohesion intercept of 2 kPa was assumed.

The disturbed shale lies on a foundation of undisturbed, unoxidized shale which has historically been considered completely intact. The intact shale of the Bearpaw Formation is closely related to the intact shale of the Lea Park Formation. The value of the residual friction angle and the effective cohesion for the intact shale of the Lea Park Formation has been reported to be

approximately 20° and 10 kPa, respectively (Sauer, 1999). For the purposes of this analysis, the shear strength properties of the Bearpaw Formation intact shale were considered analogous to those of the Lea Park Formation. Recent stratigraphic investigations have revealed the presence of a thin bentonite seam cutting through the interior of the undisturbed, unoxidized shale formation. This discontinuity is of interest to the stability analysis due to potential slip surface created by the bentonite layer. Little information is available for the shear strength parameters of the bentonite seam. For analysis purposes, the bentonite layer was assumed to have properties similar to those of the disturbed clay shale unit. As well, the friction angle for the disturbed clay shale was taken as 10° a value between the residual and peak friction angles reported by Sauer(1193, 1999).

Table 6-3 - Material Properties used in the stability analysis

Material	Unit Weight γ (kN/m^3)	Cohesion c' (kPa)	Phi ϕ (Degrees)
Fill	21	7	25
Soft Fill	21	0	20
Designed Sand Draiange Layer	20	0	30
Alluvial Deposits	20	0	20
Oxidized Clay Shale (Normal Strength)	22	10	15
Oxidized Clay Shale (Reduced Strength)	22	2	10
Unoxidized Clay Shale	Modelled as impenetrable		

Note: A residual strength for the clay shale, typically c' = 0 kPa and ϕ' = 8º, was not used in the analysis, rather the reduced strength parameters provided in Table 6-3 were used instead.

6.3.2 Seepage Parameters

Specifying the hydraulic conductivity (*K*) value is one of the most complex processes in numerical modelling, since the value of K for a single material is generally:

- Lognormal (ranges over many orders of magnitude);
- Stochastic in nature (varies greatly through space);
- Directional (varies in vertical and horizontal direction);
- Scale dependant (field and laboratory values); and
- Non-linear dependence on the water content (unsaturated flow).

The purpose of finite element seepage analysis is to provide a pore water pressure profile for the slope stability analysis and not to provide the actual magnitude of flow. Therefore, the hydraulic conductivity values were systematically adjusted to calibrate the seepage model to match the field pore water pressure (head) values obtained from the instrumentation installed in the field investigation.

Engineering judgement was utilized in assigning the generic soil-water characteristic curves (available in GeoStudio software) to the respective materials based on the lithology encountered during the drilling investigation and the grain size analysis of the samples obtained during the drilling program.

6.4 Hydraulic Conditions

The finite element modelling software SEEP/W was coupled with limit equilibrium analysis software SLOPE/W to compute pore water pressures due to the varying reservoir elevation. The numerical model finite element mesh was constructed in such a way that a specific node exists at the (x,y) coordinate at the tip of each piezometer. Readings from the three vibrating wire piezometers installed on the dam were obtained in February 2011 when the reservoir ice elevation was measured to be approximately 722.2 masl.

The SEEP/W models were calibrated to real-time data obtained from vibrating wire piezometers. Models were calibrated by systematically adjusting the hydraulic conductivity values and boundary conditions in order to match the pore water pressure (head) values measured in the field. Numerical calibration is an iterative process, which involves several trial-and-error simulations to match the numerical solution to the field data. However, it is important not to force the numerical solution, which might result in a non-realistic output. Once calibrated, the models were used to predict the pore water pressure profile for different reservoir elevations. The predicted pore water pressures were then imported into SLOPE/W for the stability analysis.

6.5 Seismic Conditions

The 2007 CDA Guidelines require the *Fs* to be greater than or equal to 1.0 under pseudo-static loading conditions. SLOPE/W requires a peak ground acceleration value in order to conduct the pseudo-static analysis. Richter and Nuttli scale readings were obtained within a 500 km radius of Regina, Saskatchewan from the Natural Resources Canada website which is: http://earthquakescanada.nrcan.gc.ca. Sixty seismic events were recorded in this area from

1985 to the present. A statistical analysis was performed on the data to calculate the mean and standard deviation:

- Mean seismic event → 3.0 (Nuttli scale), standard deviation → 0.400

The maximum seismic event equals 4.0 (Nuttli scale) with seismic values reported to equal 3.5 near the southern part of Saskatchewan and US border. The corresponding peak ground acceleration was found to be:

- Corresponding peak ground acceleration → 0.059g (where g = 9.81m/s^2)

"The difficulty with the pseudo static approach is that in reality the seismic acceleration only acts for a very short moment in time during the earthquake shaking. The factor of safety in reality varies dramatically both above and below static factor of safety. The factor of safety may even momentarily fall below 1.0, but this does not mean the slope will necessarily collapse." (Krahn, 2007).

6.6 Failure Mechanisms

Identifying an appropriate failure mechanism can play a critical role in developing the stability model. This typically requires analyzing the soils beneath the Dam. Slip surfaces are sometimes circular (rotational) in nature but they can become translational due to the presence of a weak layer(s). The weak layer then becomes the controlling layer as it dictates the shape and location of the critical slip surface. The slip surface will become composite (translational) because of the weak layer, which will result in large lateral movement along a planar slip surface within the weak layer. In analyzing the three cross-sections being analyzed, the weak layers identified to be analyzed as failure mechanisms are provided in Table 6-4.

Table 6-4 - Potential Failure Surfaces

Cross-Section	Elevation (masl)	Depth (mbgs)	Details
A-A'	708.7	8.8	Oxidized / Unoxidized shale contact
B-B'	708.8	8.7	Oxidized / Unoxidized shale contact
C-C'	710.7	6.8	Oxidized / Unoxidized shale contact

6.7 Method of Analysis

For the analysis conducted, MDH utilized a limit equilibrium software package (SLOPE/W) available in Geostudio 2007. This specialty software was developed by a Canadian-based company named Geo-Slope International. SLOPE/W utilizes limit equilibrium theory for modelling slope stability and has the ability to use various methods of analysis. The method used for this analysis was Morgenstern and Price with a half sine side force function. This method satisfies both the moment and force equilibrium equations.

7.0 Slope Stability Analysis Results

7.1 Piezometric Model Calibration

The SEEP/W models were calibrated to data from vibrating wire piezometers installed along the three modelled cross-sections. For the purpose of pre-design stability analysis, hydrostatic conditions were assumed, so the target calibration value was derived by taking the average of all the piezometers installed in the borehole. When calibrating the seepage model for pre-design analysis, a maximum acceptable error of 15% of the head loss across the modelled geometry was utilized. Several trial-and-error seepage models were undertaken to closely match the head data obtained from the vibrating wire piezometers installed as part of the field investigation. The hydraulic material properties obtained are summarized in Table 7-1 and the corresponding results in Table 7-2.

Table 7-1 - Hydraulic Conductivity Values

Material	Hydraulic Conductivity
Fill Material	5.00E-08
Alluvial Deposits	1.00E-06
Shale	1.00E-09

Table 7-2 - Hydraulic Conductivity Calibration Results

Cross-section	Piezometer ID	Actual Field Readings		Simulated (masl)	Difference (m)	Percent Change
		Actual	Average			
A-A'	R2488-01A	718.43	718.13	718.8	0.67	14.18%
	R2488-01B	717.97				
	R2488-01C	718.00				
B-B'	R2488-02A	720.72	719.78	720	0.22	4.58%
	R2488-02B	720.45				
	R2488-02C	718.19				
C-C'	R2488-03A	722.17	721.08	720.7	-0.38	-8.00%
	R2488-03B	721.73				
	R2488-03C	720.09				
	R2488-03D	720.31				

It was not possible to accurately match the simulated head values to the measured head values in all piezometers due to heterogeneity in the actual materials that is not simulated in the seepage model. However, the models were successfully calibrated to a level sufficient for a pre-design stability analysis. Continued monitoring of the piezometers will help create a better understanding of the pore water pressure profile in the dam for varying reservoir elevations.

7.2 Slope Stability Results

The slope stability analysis was completed in two separate analyses. The first analysis focused on the existing height of the dam and also determined what slope upgrade would be required to meet current CDA guidelines. The second analysis focused on the stability of the dam upon upgrading the height. The varying height increases were based on pre-design alternatives provided by NHC as discussed previously in Section 5.0.

Each analysis took into consideration four different scenarios. Each scenario has specific *Fs* requirements identified in the CDA guidelines. The four scenarios and their required *Fs* are summarized in Table 7-3.

Table 7-3 - Slope Stability Scenario Details

Scenario	Reservoir Head Conditions	Fs Required
Level (FSL)	2.7 m freeboard	1.5
2) Probable Maximum Flood (PMF)	0.6 m freeboard	1.3
3) Rapid drawdown	Rapid drawdown from IDF to spillway	1.3
4) Seismic considerations	2.7 m freeboard	1.0

Under the static full supply level (FSL) and probably maximum flood (PMF) conditions the numerical modelling simulation involved a coupled slope stability and steady-state seepage analysis and the downstream face was analyzed. The rapid drawdown scenarios utilized transient seepage analysis coupled with a slope stability analysis and the stability of the upstream face was analyzed.

7.2.1 Existing Height Analysis

MDH utilized the most recent LiDAR data provided by NHC to determine that the elevation of the existing dam is approximately 724.5 masl with a downstream slope of 3:1. The toe of the dam was approximated at 717.5 masl which makes for a total height of 7 m. The analysis of the

existing dam height included analyzing varying slopes in order to determine what slope would be required to meet current CDA guidelines even if no dam raise were to be conducted.

7.2.2 Scenario 1: Normal Operation at Full Supply Level (FSL)

During the field investigation it was determined that the current dam reservoir elevation is approximately 722.2 masl. Although this only provides for 2.3 m of freeboard, it was considered to be normal operation for the purpose of the analysis on the existing dam conditions. Under these conditions, the stability of the downstream slope for each cross-section was analyzed for varying slopes of 3:1 to 8:1. The results are provided in Table 7-4.

Table 7-4 - Slope Stability Results for Existing Height at FSL

Fs of Current Height (724.5 masl) Normal Operation			
Slope	**Cross-Section**		
	A-A	**B-B**	**C-C**
3:1 (existing)	1.23	1.22	1.19
5:1	1.43	1.42	1.43
6:1	1.54	1.52	1.56
7:1	1.65	1.62	1.69
8:1	1.76	1.72	1.82

The results indicate that the existing structure does not meet current CDA guidelines and would require an upgrade to at least 6:1 in order to be in compliance.

7.2.3 Scenario 2: Probable Maximum Flood (PMF)

For this analysis, static head level of the reservoir was assumed to be 723.9 masl allowing for 0.6 m of freeboard on the existing dam height. Each cross-section was analyzed under static conditions with varying slopes between 3:1 to 8:1. The results are provided in Table 7-5

Table 7-5 - Slope Stability Results of Existing Height at PMF

Fs of Current Height (724.5 masl) Flood Event			
Slope	Cross-Section		
	A-A	B-B	C-C
3:1 (existing)	1.16	1.10	1.10
5:1	1.37	1.33	1.35
6:1	1.48	1.43	1.47
7:1	1.59	1.52	1.61
8:1	1.70	1.63	1.74

The results indicate that the existing structure does not meet current CDA guidelines and would require an upgrade to at least 5:1 in order to be in compliance.

7.2.4 Scenario 3: Rapid Drawdown

The rapid drawdown scenario evaluates the change in *Fs* due to a sudden change in reservoir level from the flood level the spillway inlet elevation. In the case of the existing structure, the analysis assumed a flood level of 724.5 masl, and an inlet level of 723.0 masl. The duration of the rapid drawdown was based on information provided by NHC. Figure 7-1 provides a plot between the calculated *Fs* and time (in days) for a rapid drawdown scenario for the existing dam height for each cross-section. The downstream slope has no impact on the *Fs* in this scenario, so it was arbitrarily selected at 6:1.

The calculated *Fs* for the dam's upstream face dropped sharply during the rapid drawdown period, as shown in Figure 7-1. There was a reduction in *Fs* because for a short period of time the hydrostatic force provided by the reservoir is removed, but the effective stress state in the soil does not change. When the effective stress does not change, the shear resistance to sliding remains unchanged, but the loss of the hydrostatic force results in the factor of safety being reduced.

Since the results of the rapid drawdown simulation are based on a transient seepage analysis, these results are to be used only as general guidelines. The calculated *Fs* and time plots are only provided to discuss the behaviour of a predicted failure due to rapid drawdown and help AESB understand the risk associated with rapid reservoir level drawdowns. These plots should not be used to make decisions regarding the rate at which the rapid drawdown can occur. If

possible, AESB should avoid situations in operating the dam that result in a rapid reservoir drawdown.

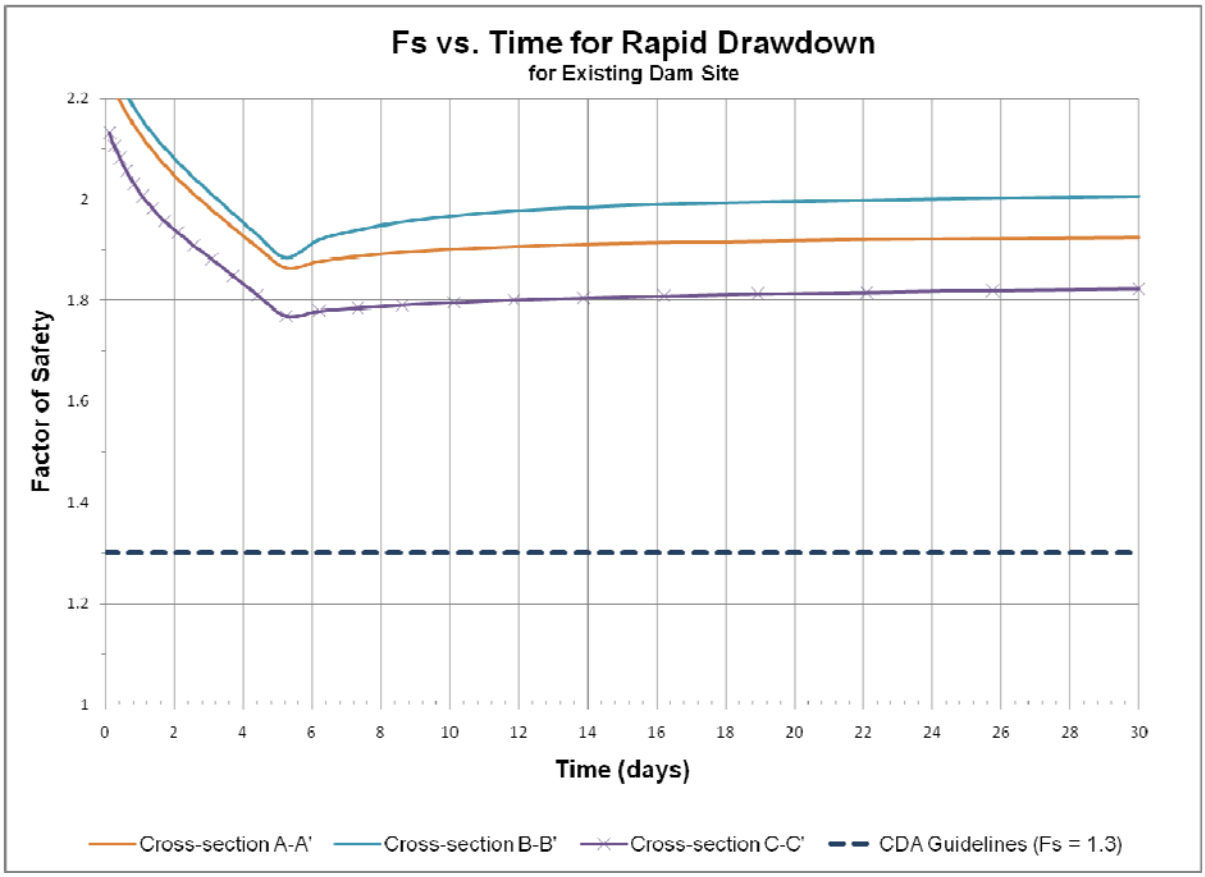


Figure 7-1 – Plot between calculated *Fs* vs. time (days) due to rapid drawdown.

7.2.5 Scenario 4: Seismic Considerations

The peak ground acceleration (pga) was determined to be 0.059g (Section 4.5). A pseudo-static analysis using this value was conducted on each cross-section using both FSL and PMF conditions. The results are summarized in Table 7-6. Additional sensitivity analysis was conducted to determine the impact of varying pga values which is presented in Figure 7-2. Using the existing historical pga, the calculated *Fs* was found to be below 1.0 under both reservoir head conditions.

Table 7-6 - Seismic Stability Results for Existing Height

Cross-Section	Reservoir Head	
	FSL	PMF
A-A'	0.96	0.91
B-B'	0.97	0.89
C-C'	0.94	0.87

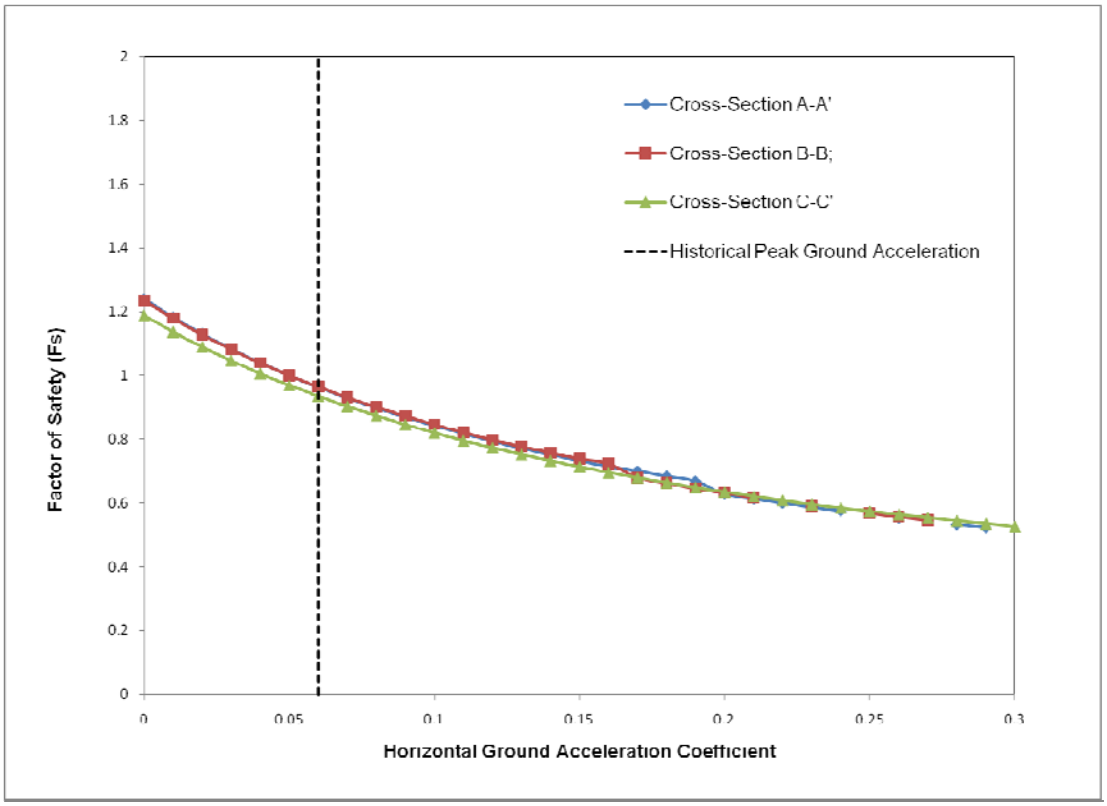


Figure 7-2 - Sensitivity Results for Seismic Coeifficient at Existing Height

7.2.6 Summary of Results for Existing Height

The *Fs* of Highfield dam at its current height and configuration under normal operating conditions is estimated to be as low as 1.19. The analysis also indicates that a flood event which brings the reservoir elevation within 0.6 m of the top of the dam would reduce the *Fs* to as low as 1.10. Under pseudo-static conditions, the resultant *Fs* is below 1.0.

These values do not comply with current 2007 CDA guidelines outlined in Section 4.0. In order to meet CDA guidelines, a slope of 6:1 would need to be constructed even if no increase in

height occurred. Modifying the slope to 6:1 would increase the *Fs* to 1.50 under normal operation and 1.40 for PMF levels.

7.2.7 Upgraded Height Analysis

To analyze the stability of upgrading the height of the dam, MDH considered the heights outlined in Table 7-7 based on the pre-design alternatives provided by NHC.

Table 7-7- Dam Height for Each Alternative

Upgrade Alternative Description	Required Embankment Elevation (masl)	Total Height (m)
Existing Earth Cut Spillway	726.9	9.4
Alternative 1 - Ungated Labyrinth Weir on East Side	727.1	8.6
Alternative 2 - Gated Trapezoidal Weir on East Side	727.2	9.6
Alternative 3 - Earth Cut Spillway on East Side	725.7	8.2
Alternative 4 - Ungated Labyrinth Weir + Existing Earth Spillway	725.2	8.2
Alternative 5 - Gated Trapezoidal Weir on East Side + Existing Earth Spillway	725.5	8.2
Alternative 6 - Enlarged Earth Cut Spillway on West Side	725.5	8.2

Each cross-section and height was analyzed for the FSL and PMF scenarios. In order to simplify the rapid drawdown seismic scenarios, the analysis was limited to one cross-section. The predicted phreatic water surface for each height configuration and scenario was simulated using the calibrated models as discussed in Section 6.1.

7.2.8 Scenario 1: Normal Operation

Based on the freeboard requirement of 2.7 m the full supply level for each height alternative is defined in Table 7-8. Under normal operating conditions, the *Fs* required according to CDA guidelines is 1.5. Using the FSL, each cross-section was analyzed at each potential design height using various slopes and the results are provided in Table 7-9

Table 7-8 - FSL and PMF Vales for Each Alternative

Alternative	Dam Height (masl)	FSL (masl)	PMF (masl)
Existing	726.90	724.20	726.30
1	727.10	723.40	725.50
2	727.20	724.40	726.50
3	725.70	723.00	725.10
4	725.20	723.00	725.10
5	725.50	723.00	725.10
6	725.50	723.00	725.10

Table 7-9 - Slope Stability Results for Upgraded Heights at FSL

Slope	Cross-Section A-A'				Cross-Section B-B'				Cross-Section C-C'			
	Upgraded Dam Height (masl) and Fs				Upgraded Dam Height (masl) and Fs				Upgraded Dam Height (masl) and Fs			
	725.7	726.1	726.9	727.1	725.7	726.1	726.9	727.1	725.7	726.1	726.9	727.1
3:1	1.17	1.15	1.14	1.13	1.22	1.20	1.17	1.16	1.15	1.12	1.09	1.09
5:1	1.42	1.37	1.35	1.35	1.41	1.40	1.38	1.37	1.41	1.38	1.36	1.35
6:1	1.53	1.48	1.47	1.47	1.52	1.51	1.49	1.49	1.55	1.51	1.50	1.49
7:1	1.62	1.60	1.59	1.59	1.63	1.61	1.60	1.60	1.68	1.64	1.64	1.64
8:1	1.75	1.72	1.71	1.70	1.74	1.74	1.72	1.71	1.81	1.78	1.78	1.78
9:1	1.85	1.84	1.83	1.82	1.86	1.86	1.83	1.83	1.95	1.93	1.92	1.92

The results show that despite an increase in height, the required slope to meet a *Fs* of 1.5 is still 6:1. The height variation within this range (725.7 – 727.1 masl) had very little impact on the *Fs* which is a result of the weak layer governing the stability of the dam. As the height of the dam increases, the radius of critical slip surface also increases. The increased radius results in an increased amount of oxidized shale at reduced strength being mobilized along the shear plane. This is illustrated in Figure 7-3 and Figure 7-4 which compares the dam height of 725.7 masl and 727.1 masl both with a downstream slope of 6:1. At 725.7 masl, the critical slip surface radius is 37.5 m which results in mobilizing 35 m of oxidized shale at residual strength within the lateral zone of the shear plane. Upon increasing the height of the dam, the radius of the critical slip surface increases to 42.0 m and approximately 45 m of oxidized shale at reduced strength is being mobilized within the shear plane.

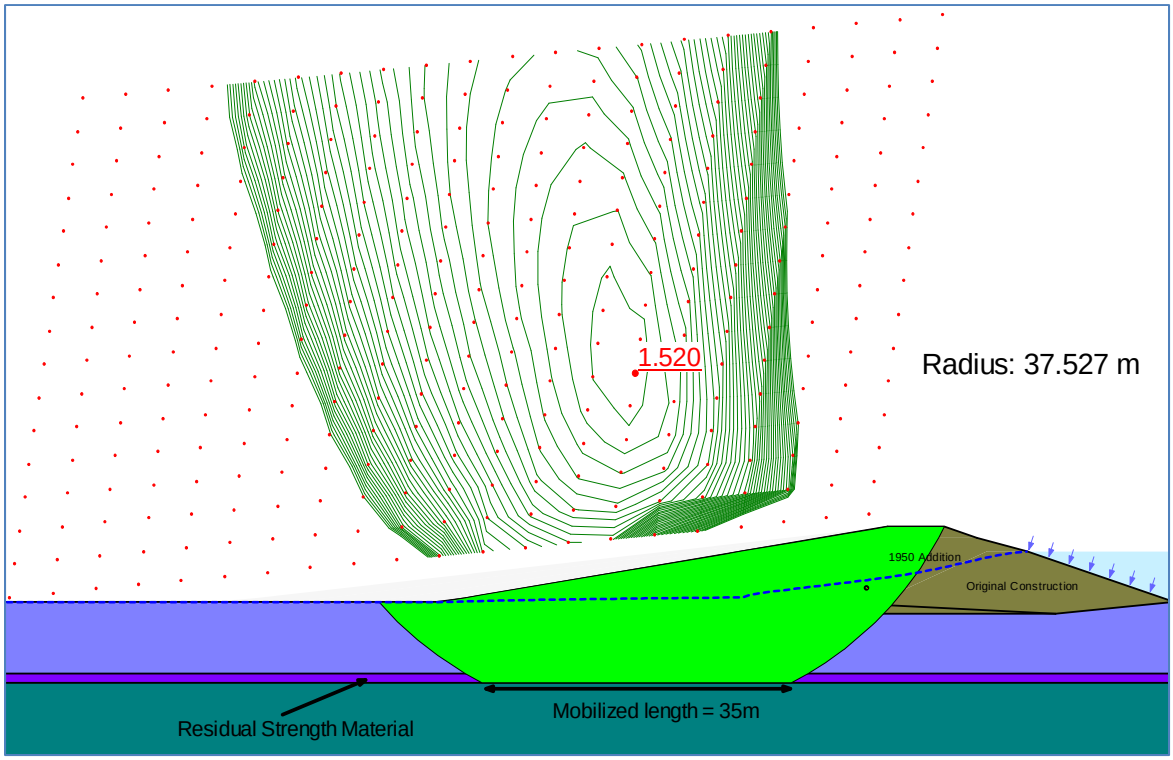


Figure 7-3- Slip Surface Detail of Dam Height of 725.7 masl (6:1 Slope)

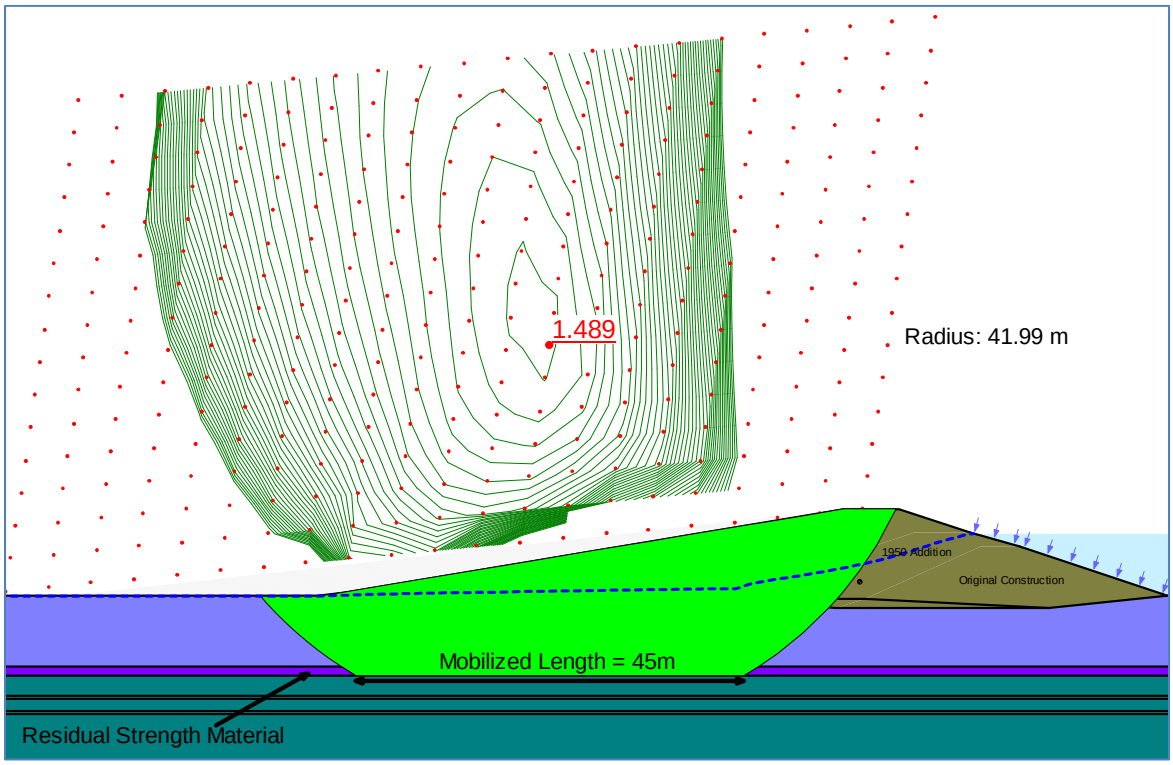


Figure 7-4 - Slip Surface Detail of Dam Height of 727.1 masl (6:1 Slope)

7.2.9 Scenario 2: Flood Event (1:1000)

Each height alternative was evaluated for each cross-section using a 0.6 m freeboard to represent a PMF (refer to Table 7-8 above). The results of this analysis are provided in Table 7-10.

Table 7-10 - Slope Stability Results for Upgraded Heights at PMF

Slope	Cross-Section A-A'				Cross-Section B-B'				Cross-Section C-C'			
	Upgraded Dam Height (masl) and Fs				Upgraded Dam Height (masl) and Fs				Upgraded Dam Height (masl) and Fs			
	725.7	726.1	726.9	727.1	725.7	726.1	726.9	727.1	725.7	726.1	726.9	727.1
3:1	1.10	1.11	1.09	1.09	1.16	1.15	1.13	1.12	1.06	1.04	1.02	1.02
5:1	1.36	1.33	1.31	1.31	1.35	1.35	1.33	1.32	1.32	1.30	1.29	1.28
6:1	1.47	1.44	1.43	1.43	1.46	1.45	1.44	1.44	1.46	1.43	1.42	1.42
7:1	1.56	1.56	1.55	1.55	1.57	1.56	1.55	1.56	1.59	1.57	1.56	1.57
8:1	1.70	1.68	1.67	1.67	1.68	1.69	1.67	1.67	1.73	1.70	1.71	1.71
9:1	1.81	1.80	1.79	1.79	1.80	1.80	1.79	1.79	1.86	1.85	1.85	1.85

The results indicate that in order to satisfy a *Fs* of 1.30 during peak flood conditions, a slope of at least 5:1 is required. It should be noted that these results are strongly influenced by the assumption that an effective sand drain will be constructed under the embankment addition. The sand drain reduces the porewater pressures within the fill which increases the overall stability. An analysis to demonstrate the impact of a sand drain is presented in Section 6.4 of this report.

7.2.10 Scenario 3: Rapid Drawdown

Based on the rapid drawdown analysis results of the existing structure, the rapid drawdown analysis for the upgraded scenarios was limited to Cross-section C-C' and was conducted for each alternative. The drawdown functions for each alternative were based on reservoir head generated by a 1:000 year flood event, and then drawing down to the inlet level of each alternative. To accurately define the rapid drawdown function for each alternative, MDH obtained hydrograph information from NHC. The drawdown amount and time for each alternative is summarized in Table 7-11. Using these parameters to define each rapid drawdown function, the results obtained are presented in Figure 7-5.

As noted above, these plots should not be used to make decisions regarding the rate at which the rapid drawdown can occur. If possible, AESB should avoid situations in operating the dam that result in a rapid reservoir drawdown.

Table 7-11 - Rapid Drawdown Functions

Alternative Number	1:1000 Peak Level (masl)	Inlet Elevation (masl)	Drawdown Duration (Days)
Alternative 0	726.26	723.00	8.71
Alternative 1	725.47	723.00	6.42
Alternative 2	726.50	723.00	11.75
Alternative 3	725.00	723.33	12.11
Alternative 4	724.50	722.00	12.92
Alternative 5	724.82	722.00	17.70
Alternative 6	724.83	722.00	15.10

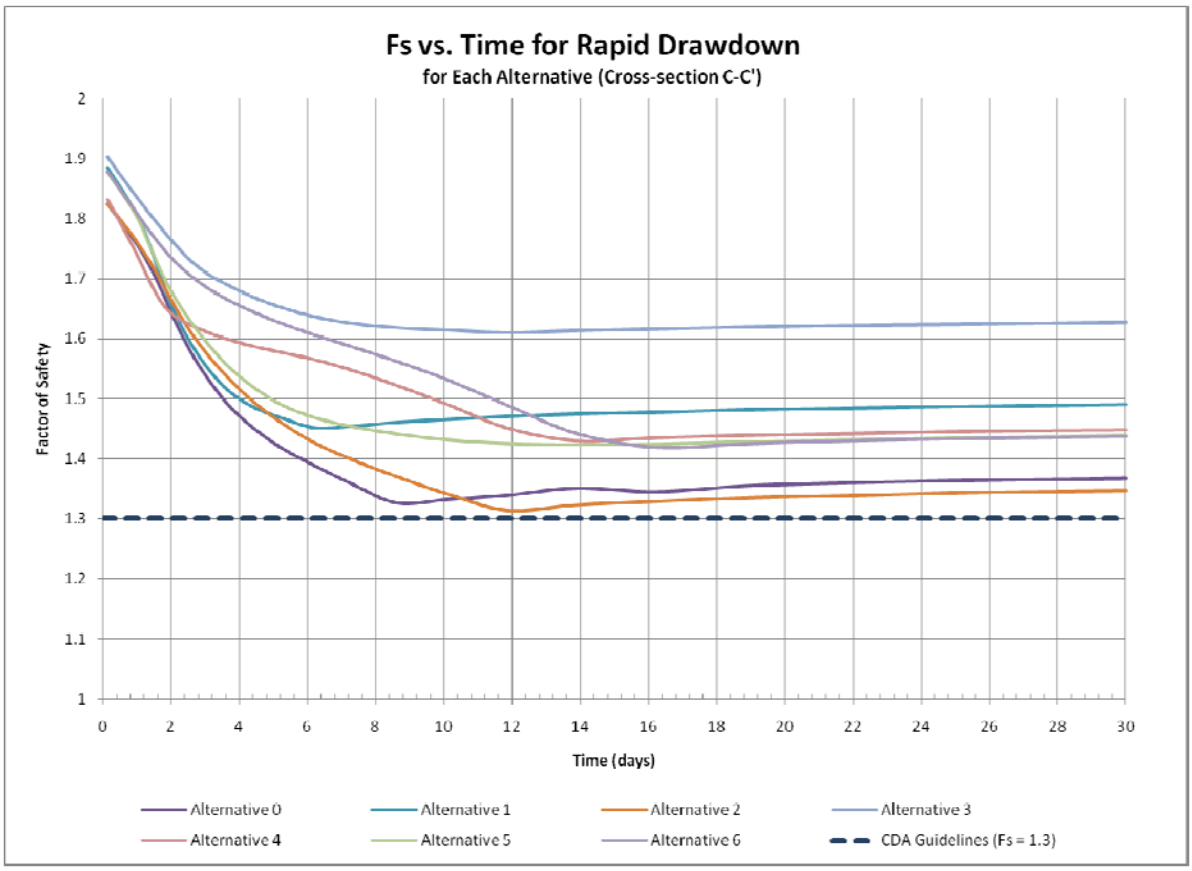


Figure 7-5 - Rapid Drawdown Results for Upgraded Alternatives

7.2.11 Scenario 4: Seismic Consideration

For simplification, the seismic analysis was limited to one height and slope configuration for all three cross-sections and analyzed the downstream slope. The chosen height and downstream slope were 727.1 masl and 6:1 respectively. These parameters were chosen based on the above results which indicate that a 6:1 slope will be favoured. The results of the pseudo-static analysis are summarized in Table 7-12.

Table 7-12 - Seismic Stability Results for Upgraded Height (727.1 masl)

Cross-Section	Reservoir Head	
	FSL	PMF
A-A'	1.03	0.99
B-B'	1.05	1.01
C-C'	1.04	0.99

Using the existing historical pga, the calculated *Fs* was found to be at or above 1.0 for all cross-sections in the FSL scenario. Additional sensitivity analysis was conducted using FSL conditions to determine the impact of varying pga values and the results are presented in Figure 7-6.

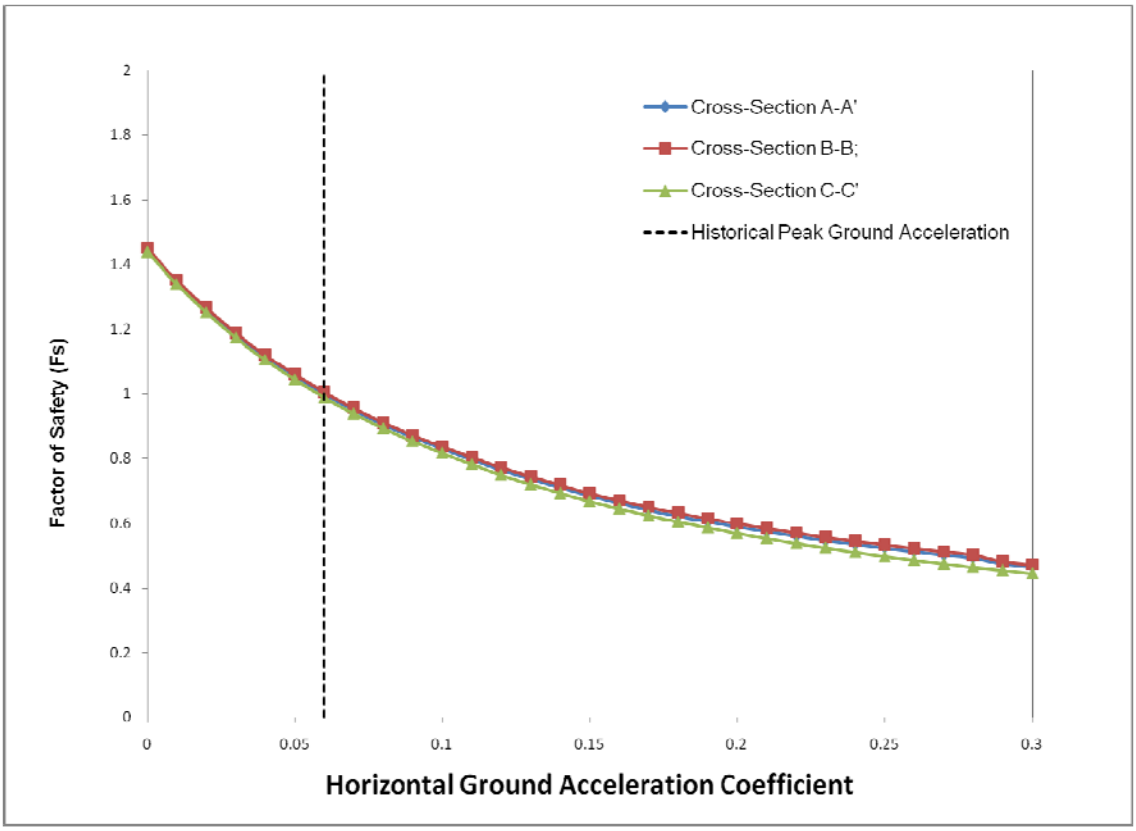


Figure 7-6 - Sensitivity Results for Seismic Coefficient at Upgraded Height (727.1 masl)

7.2.12 Summary of Results for Upgraded Height

The analysis conducted on various dam upgrade configurations indicates that regardless of the selected alternative, a slope of 6:1 is required to satisfy current CDA guidelines. Table 7-13 summarizes the results for each cross-section and alternative height using a 6:1 slope.

Table 7-13 - Slope Stability Results of 6:1 Slope

Fs Results for Slope of 6:1						
Piezo Conditions	A-A'		B-B'		C-C'	
	Dam Height	Fs	Dam Height	Fs	Dam Height	Fs
FSL (Criteria = 1.50)	725.7	1.53	725.7	1.52	725.7	1.55
	726.1	1.48	726.1	1.51	726.1	1.51
	726.9	1.47	726.9	1.49	726.9	1.50
	727.1	1.47	727.1	1.49	727.1	1.49
PMF (Criteria = 1.30)	725.7	1.47	725.7	1.46	725.7	1.46
	726.1	1.44	726.1	1.45	726.1	1.43
	726.9	1.43	726.9	1.44	726.9	1.42
	727.1	1.43	727.1	1.44	727.1	1.42

7.3 Sensitivity Analysis

A sensitivity analysis is used to help obtain a better understanding of the influence of particular parameters in a stability analysis. In a sensitivity analysis the parameters are selected in an orderly fashion between a defined minimum and maximum range using a uniform probability distribution function. The characteristic of a uniform distribution is that all values have an equal probability of occurrence. Table 7-14 provides the summary of sensitivity analysis parameters.

Table 7-14 - Sensitivity Analysis Material Property Ranges

Material	Unit Weight γ (kN/m^3)			Cohesion c' (kPa)			Phi ϕ (Degrees)		
	Mean	Min	Max	Mean	Min	Max	Mean	Min	Max
Fill	21	19	23	7	2	12	25	20	30
Soft Fill	21	19	23	5	0	10	20	15	25
Sand Filter	20	18	22	-			30	25	35
Alluvial Deposits	20	18	22	5	0	10	20	15	25
Oxidized Clay Shale (Peak)	22	20	24	10	5	15	15	10	20
Oxidized Clay Shale (Residual)	22	20	24	5	0	10	10	5	15
Unoxidized Clay Shale	Modelled as impenetrable								

The results of the sensitivity analysis performed on Cross-section A-A', Cross-section B-B' and Cross-section C-C' are shown in Figure 7-7, Figure 7-8, and Figure 7-9, respectively. The results are presented as a sensitivity plot in which all three parameters (weight, friction angle and cohesion) are normalized to a value ranging between 0.0 and 1.0. Zero means the lowest value and 1.0 means the highest value. For example, zero for alluvial deposits, means a unit weight (γ_b) of 18 kN/m^3 and an effective friction angle (ϕ') of 15°. Similarly a value of 1.0 for

alluvial deposits corresponds with a γ_b of 22 kN/m^3 and ϕ' of 25°. By comparing the slope of all the lines on the sensitivity plots, it was found that the calculated *Fs* for all three cross-sections is most sensitive to the reduced friction angle of the oxidized clay shale

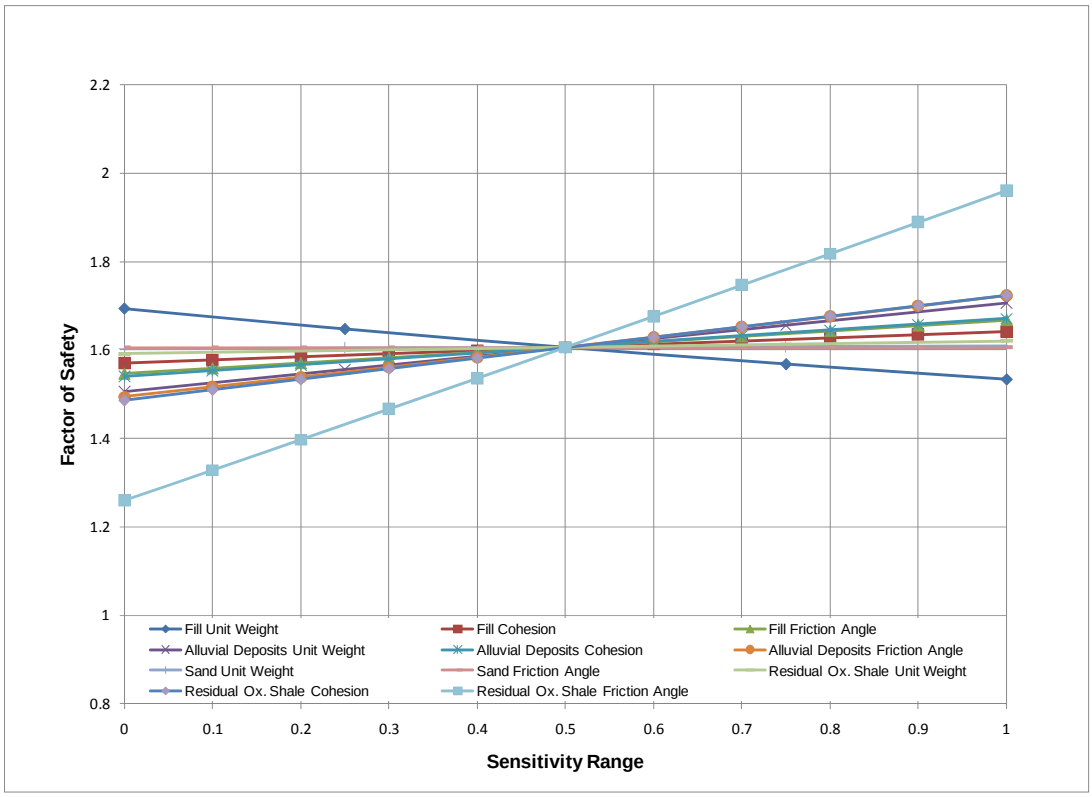


Figure 7-7 - Sensitivity Analysis Results for Cross-section A-A'

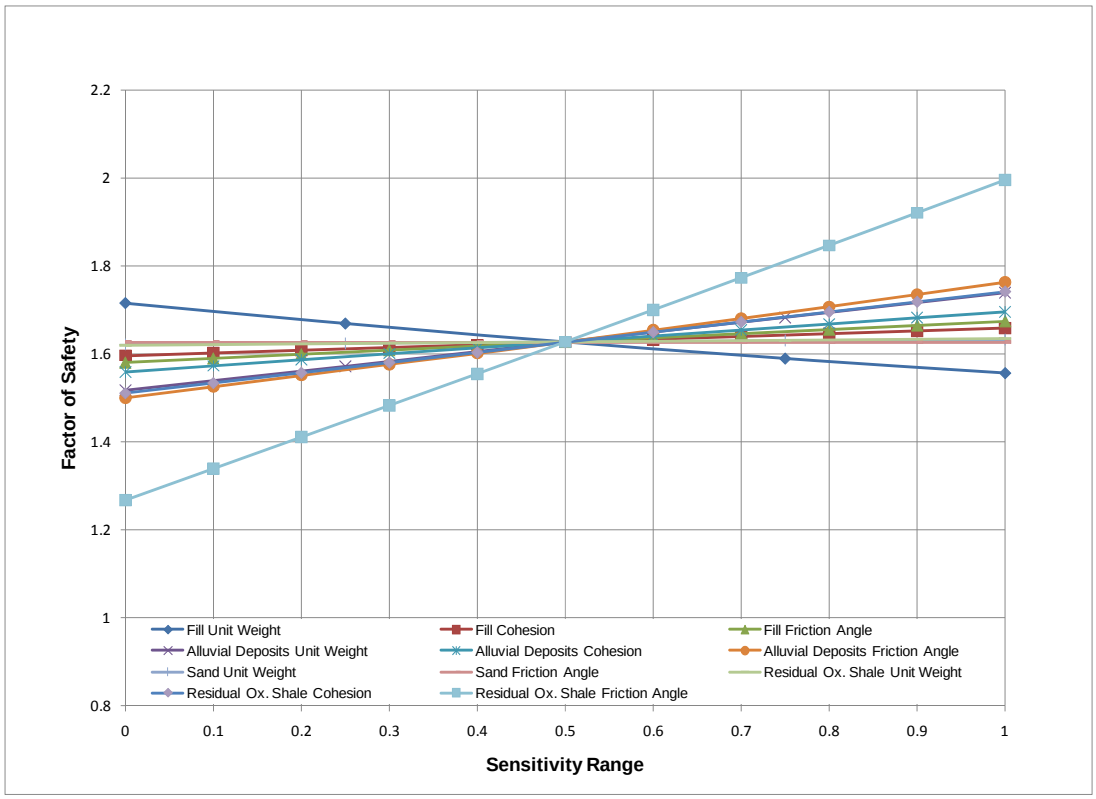


Figure 7-8 - Sensitivity Analysis Results for Cross-section B-B'

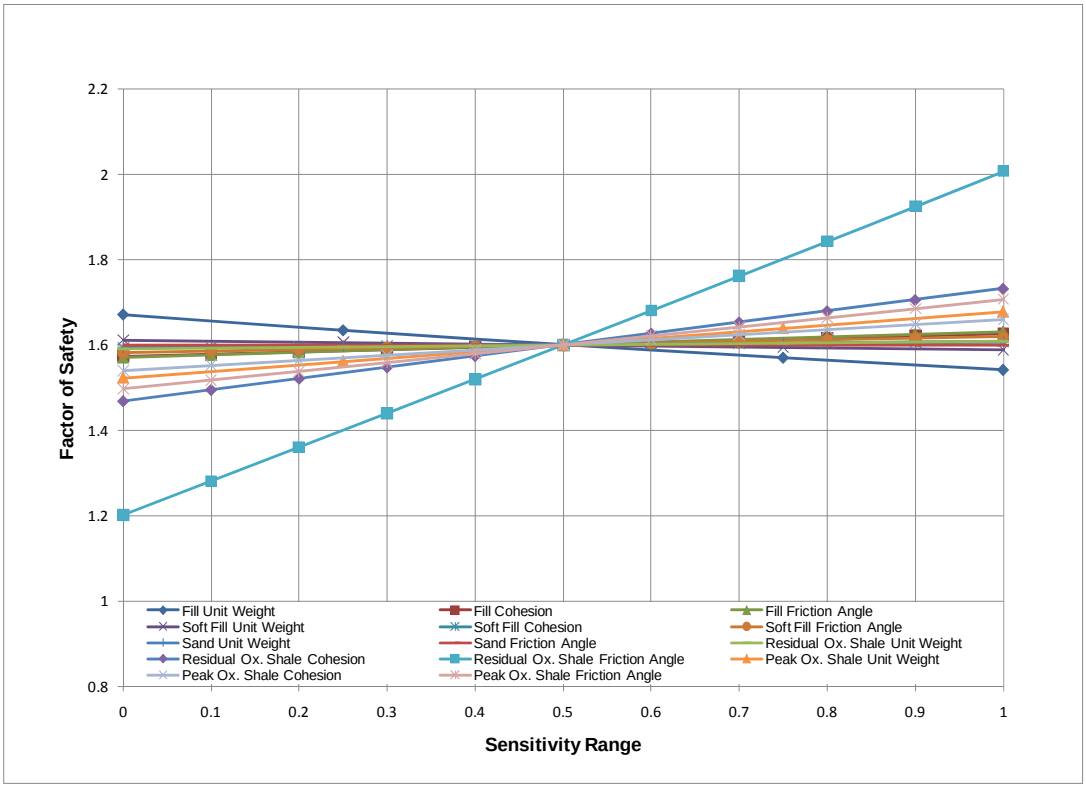


Figure 7-9 - Sensitivity Analysis Results for Cross-section C-C'

8.0 Cost Analyses for Spillway Alternatives

As part of the evaluation of the six alternatives that NHC conducted, MDH conducted the cost analyses. Initial capital, operating and maintenance costs were calculated for each of the six alternatives presented in Section 5.0. Capital costs were derived using the pre-design drawings provided by NHC which are included in Appendix G. MDH calculated earth and concrete volumes from the pre-design drawings and based the unit costs for the various items in each alternative on the Saskatchewan Ministry of Highways and Infrastructure (SMHI) 2011 bid price trends and on recent projects that MDH has conducted. For all of the options it was assumed that where excavation was required that all of the excavated material could be reused in the embankment as fill. In reality, there will likely be some excavated material that will not be suitable for use in the embankment. It was assumed that the balance of the required material for the embankment raise in each alternative would come from a nearby borrow source. A 1 km maximum haul was assumed for the borrow material. For Alternative 3 there is no haul required because there is greater excavation volume required for the spillway construction that is needed for fill to raise the embankment. In this case, the excess excavated material was assumed to be stockpiled on-site.

Engineering costs have been included in each estimate. Engineering costs have been broken down into design engineering and construction engineering costs. Design engineering would consist of preparation of design drawings and report, tender document preparation and tendering. Construction engineering costs include, inspection and testing services, project management, contractor management, surveying for initial layout and for quantities, final reporting (as-built) on construction activities.

In developing the life cycle costs for each option, operating and maintenance costs were based on those provided in an email on December 20, 2010 from Glenn McLaughlin of by AESB for the various types of spillways used in the six alternatives. A discount rate of 5 percent was used in developing the net present value for each alternative.

A summary of the costs (initial capital, operating and maintenance) required to implement each of the alternatives are provided in Table 8-1. The detailed cost spreadsheets are included in Appendix H.

Table 8-1 – Highfield Dam Spillway Upgrade Financial Analyses Summary

Alternative	Capital Cost	Annual Maintenance Cost	Annual Operating Cost	Net Present Value of all Costs
1 - Labrynth Weir - East side	$14,619,586	$220,166	$15,000	$18,641,340
2 - Gated Spillway - East side	$15,736,246	$256,688	$25,000	$21,378,086
3 - Earth Spillway - East side	$14,581,747	$226,658	$5,000	$18,741,174
4 - Labrynth Weir - West Side	$16,822,203	$290,646	$10,000	$23,390,149
5 - Gated Concrete Spillway - West Side	$17,671,603	$334,382	$20,000	$25,740,638
6 - Earth Spillway - West Side	$21,383,207	$467,255	$5,000	$36,106,792

Notes:

Capital costs Include 17 percent for detailed design and construction engineering and 20 percent contingency

Net present value assumes 5 percent discount rate over 100 years

As shown in Table 8-1, Alternative #3, the earth embankment on the east abutment had the lowest capital, operating and maintenance costs.

9.0 Geotechnical Considerations for Selected Spillway Alternative

9.1 Selected Alternative

Based on the multi-criteria analyses (MCA) that was conducted by NHC it was understood that Alternative #3 was the selected alternative. The geotechnical pre-design presented in the following subsections was conducted on this alternative.

9.2 Embankment Slope Stability Considerations

9.2.1 Recommended Embankment Slope

A numerical modelling analysis was completed to calculate the factor of safety (*Fs*) of the Dam under normal operation (different reservoir levels), as well as special transient loading conditions (rapid reservoir drawdown and earthquakes) as described in Sections 6 and 7 of this report.

At its existing height and slope, Highfield Dam does not meet CDA guidelines. The results for each scenario within this analysis are summarized in Table 9-1.

Table 9-1 - Stability Analysis Summary for Existing Height (724.5 masl)

Scenario	Critical Fs	Required Fs
1) Normal operation at Full Supply Level (FSL)	1.19	1.5
2) Probable Maximum Flood (PMF)	1.1	1.3
3) Rapid drawdown (Upstream Face)	1.77	1.3
4) Seismic considerations	0.87	1.0

The analysis conducted on the upgrade alternatives indicates that a 6:1 downstream slope is required which would yield the following results for each scenario:

Table 9-2 - Stability Analysis Summary for Upgraded Height (725.7 masl)

Scenario	Critical Fs	Required Fs
1) Normal operation at Full Supply Level (FSL)	1.53	1.5
2) Probable Maximum Flood (PMF)	1.47	1.3
3) Rapid drawdown (Upstream Face)	1.61	1.3
4) Seismic considerations	1.01	1.0

Based on the slope stability assessment and results presented in Sections 6 and 7 of this report, to meet the CDA guideline of a long term factor of safety of 1.5, it is recommended that an embankment slope of 6 horizontal to 1 vertical be used on the downstream side and 3 horizontal to 1 vertical be used on the upstream side. This recommendation would apply to the existing embankment; even if it is left in its present configuration (i.e. no embankment raise or spillway modifications are done). MDHs investigation included a minimal number of test holes. Evidence of bentonite pockets were encountered. It is recommended that a more intensive drilling investigation be conducted along the length of the dam to further characterize the clay shale, specifically to determine if any thin bentonite layers exist. It is recommended that drilling be conducted using a continuous core method such as sonic drilling. The drilling program should also include deep stratigraphic borehole(s) in order to firmly establish the stratigraphy at this site.

9.2.2 Sand and Chimney Drain

To minimize porewater development within the additional fill material that will be used to raise the embankment, a sand drain should be constructed beneath this material connected to a chimney drain(s) constructed through the embankment fill. A brief analysis was conducted to demonstrate the impact of a sand drain beneath the additional fill. Using a fully constructed height of 725.7 masl, and a downstream slope of 6:1, two scenarios were analyzed. The first model had no sand drain constructed, and the second included a fully functional sand drain. Without the sand drain the resultant *Fs* was 1.39, and with the sand drain the resultant Fs was 1.55. This equates to an 11.5% increase in *Fs* by constructing a sand drain. It is also worth noting that if an effective sand drain was not present, a 6:1 slope would no longer meet CDA guidelines. The results of this comparison are provided in Figure 9-1 and Figure 9-2, which illustrates the influence that the sand drain has on the phreatic surface throughout the additional material.

The preliminary design thickness of the sand drain would be approximately 0.6 m, but this should be confirmed in the detailed design stage along with the chimney drain size and configuration.

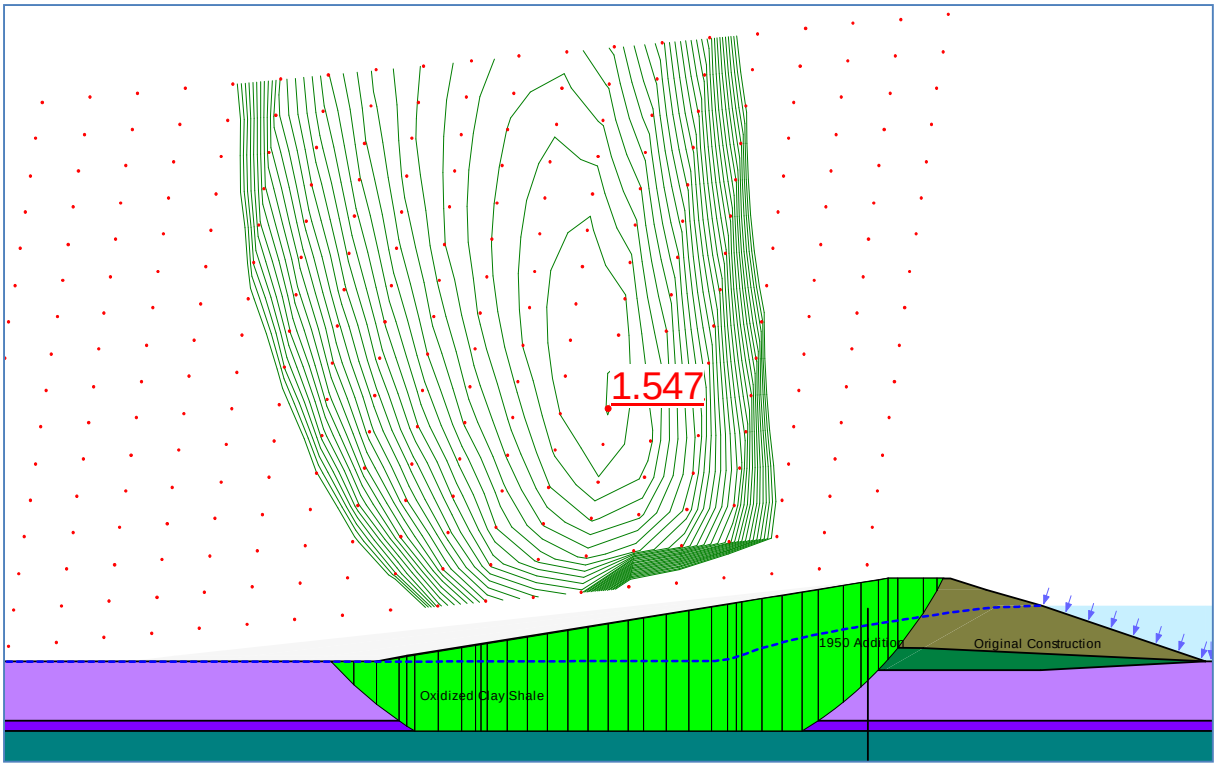


Figure 9-1 - Fs of a 6:1 Slope With Sand Drain (725.7 masl Height)

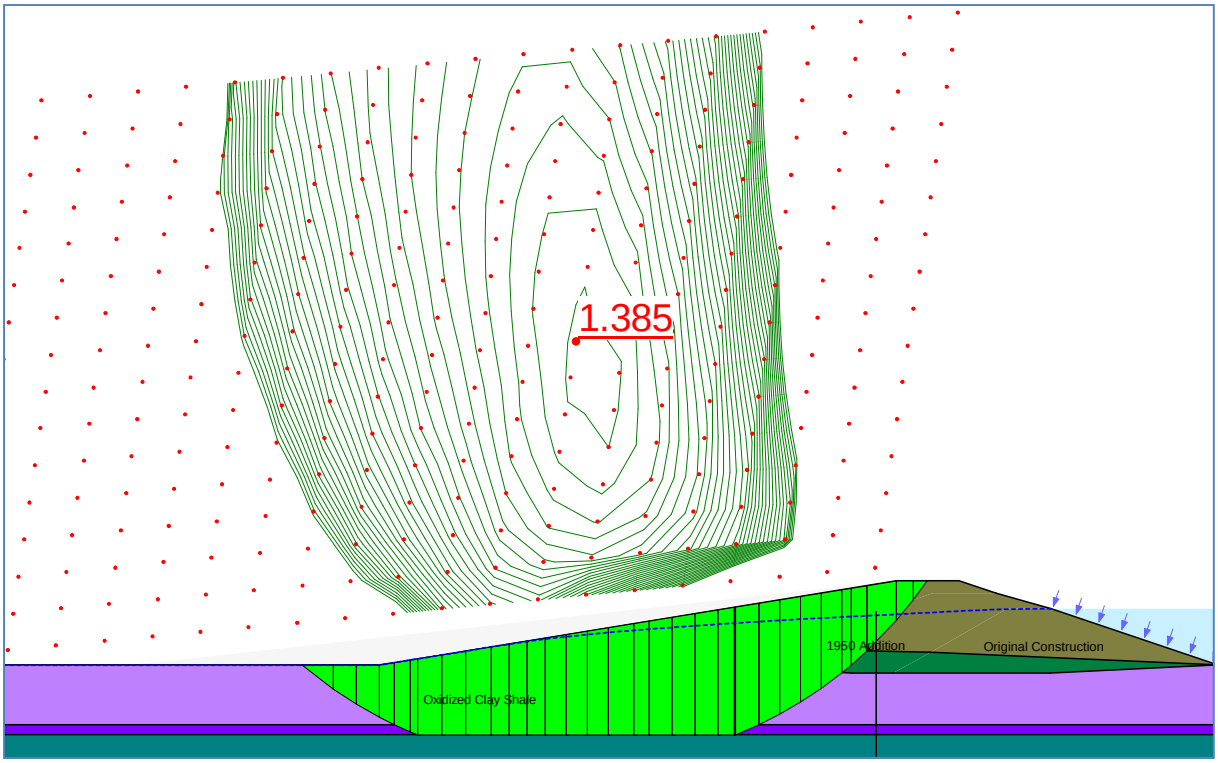


Figure 9-2 - Fs of a 6:1 Slope Without Sand Drain (725.7 masl Height)

9.2.3 Staged Construction

The CDA guidelines state that *Fs* of the dam immediately after construction should not be lower than 1.3. A brief preliminary analysis was conducted to simulate the construction scenario of the recommended upgraded slope. MDH recommends that a more in depth analysis be conducted during the detailed design phase to develop detailed construction guidelines; however this analysis provides preliminary insight as to whether or not staged construction will be required.

As the embankment height increases, the porewater pressures within the foundation soils increases due to increased loading. To simplify the porewater pressure load increase relationship, a porewater pressure coefficient was utilized. This coefficient is called B-bar and is represented by the symbol $\overline{B}$. This is a critical parameter because its value will dictate when construction of the embankment will need to be stopped.

This parameter is a unit less and is defined by the following relationship:

$$\overline{B} = \frac{\Delta u}{\Delta \sigma}$$

Where Δu is represents the change in pore water pressure and $\Delta \sigma$ is the change in effective stress. The effective stress is a known parameter since it is merely a function of the fill material height and unit weight. As a result, by monitoring the pore water pressure during construction (giving us a value for Δu), a $\overline{B}$ value can be determined. The implementation of $\overline{B}$ is important since it allows the numerical modelling software to take into consideration the changing pore water pressures conditions resulting from the embankment loading throughout the various stages of construction. This relationship can also be utilized to simulate long-term stability.

A typical $\overline{B}$ value that is used as standard practice for predicting porewater pressure loads during construction is 0.7. Using this value, an analysis was conducted of a fully upgraded dam height of 725.7 masl, using recommended downstream slope of 6:1. The results of the numerical model indicate that if the entire additional material were to be applied instantaneously, the increase in porewater pressure would result in reducing the *Fs* of the final structure to 1.00. As porewater pressures dissipated, the *Fs* would increase to 1.5, however a *Fs* of 1.0 immediately after construction would not be acceptable.

The analysis confirmed that staged construction will most likely be required. Staging might require up to two construction seasons. More detailed guidelines on this aspect would need to be developed during the detailed design phase.

9.3 Embankment Settlement

Settlement is the vertical component of soil deformation beneath the load under consideration. All imposed loads on soils will cause some settlement due to “elastic compression” of the foundation soils. This settlement occurs relatively rapidly (within days) and is termed “elastic” or “immediate” settlement and is a result of the rearrangement of the soil particles due to the applied load. Consolidation settlement, or those that are time-dependent and take months to years to develop and are a result of the expulsion of water from within the soil mass. In addition, the fill material used to construct the embankment will settlement under its own self weight. The combination of elastic, consolidation and fill settlement will need to be accounted for in the earth works requirements. Differential settlement could be a serious issue for the Dam

embankment and precise monitoring of the settlements will be required, and rehabilitation of the embankment may be required if excessive settlements occur. Alternatively, the final design could include measures to reduce total and differential settlements.

The soils beneath the embankment consist of alluvium (normally consolidated clays, silts and sands) over clay shale. The alluvium at this site was very soft (SPT, N, values of 0 in some cases). As such, it will be highly susceptible to settlement and this should be confirmed with consolidation tests. Clay shale, in its original position, is normally a highly consolidated material and would be subject to little or no settlement unless the preconsolidation pressure was exceeded. The preconsolidation pressure would not be exceeded with the proposed dam raise. However, the upper clay shale at this site shows evidence of being disturbed or reworked by glacial action and as such is not likely in its original state of overconsolidation and could be subject to some settlement.

Settlement cannot be reliably quantified. Estimates of settlement can be made using the results of consolidation testing on the foundation soils. Consolidation testing was not conducted as part of this pre-design study and is recommended in the detailed design stage of the Dam embankment in order to estimate the magnitude of settlement.

9.4 Embankment Construction Difficulties

Two major difficulties for raising the Dam embankment have already been discussed: stability concerns due to generation of high pore pressures within the subgrade, which may require staged construction, and excessive subgrade settlements. Another major challenge will be construction of the embankment over the soft alluvial deposit, downstream of the existing embankment, which has oxbows, soft soil conditions, high groundwater, and substantial depths of organic soils. Additional boreholes and possibly some test pits will be required to characterize the Rushlake Creek alluvium in the embankment area and to complete the final design.

The construction issues related to the soft alluvial deposit that will need to be addressed in the detailed design phase are, in no particular order:

1. What construction equipment and methodology can be successfully used? This may require construction of a test pad prior to the start of construction in order to prove out or modify the contractor's proposed methodology and choice of equipment.

2. What depth of organics will need to be stripped off and where to stockpile the organics for later reuse that will not create stability problems.
3. How best to construction the sand drain and embankment; for example, by end dumping the fill and spreading it out with wide pad bulldozers.
4. Would a thicker granular subdrain layer aid construction of the overlying fill by providing a stronger working platform?
5. How best to sequence the work and what areas of the embankment to work on in order to reduce excess pore pressure build up in the subgrade.
6. Will there be a need for:
    a. geotextiles, geoweb cells, and/or geogrids to facilitate construction, aid drainage, prevent pumping of silty soils, and/or reinforce the embankment. This will also require development of geosynthetics specifications and where to place the geosynthetics layer(s).
    b. vertical drains to accelerate consolidation;
    c. stone columns to reinforce the embankment, aid drainage, and reduce the embankment slope from 6 horizontal to 1 vertical to a steeper ratio.
7. Compaction specifications and how best to achieve the specified density and moisture content.
8. Material selection and optimal use of borrow sources.

9.5 Foundation for Bridge

The only structure to be constructed for Alternative 3 is the bridge across the spillway. As no boreholes were drilled for the proposed bridge in the present study therefore only a generalized design can be offered until a detailed geotechnical investigation is conducted.

The soils on the east side uplands generally consisted of clay shale and a variable mantel of clay till. The soils on the east abutment of the existing dam consisted of the dam fill, which is a cohesive material, underlain by alluvium and clay shale. Footings would have to be constructed to below the frost line which would be at least 2.0 mbgs and would likely be subject to some heave potential of the highly plastic clay shales that are present. Based on these conditions, it is expected that drilled cast-in-place concrete piles could likely be installed satisfactorily at this site and would be the least expensive option. Casing may be required in the upper portion of the piles that extend through the softer alluvium or in seepage zones in order to install them successfully. Alternatively, continuous flight auger (CFA), driven timber or steel (pipe or HP)

piles could be used. These options would likely be more expensive than drilled cast-in-place concrete piles. The type of pile would also be dependent upon the loads expected.

It is recommended that additional test holes be drilled at the location of the bridge abutments and piers in order to define the subsurface conditions and obtain samples for laboratory testing required to complete the detailed design of the bridge foundation.

9.6 East Side Spillway

9.6.1 Spillway Slopes

No boreholes were drilled in the east back slope area in this present study. Based on drilling previously conducted by PFRA, the spillway on the east side will be constructed mainly within glacial clay till soil in the upper portion and clay shale in the lower portion. Also, the till, which is highly plastic in some areas and zones, is underlain by clay shale. The piezometric conditions within the east back slope are not known and they would have a significant influence on the stability of the proposed spillway. Because of limited information a detailed slope stability analysis was not performed.

Without a detailed stability analysis slopes of 5 horizontal to 1 vertical are recommended along the spillway. This is the prudent approach as there is limited information to base a decision on, only the knowledge that clay shale and some depth of highly plastic clay till foundation soils are present and also because there is limited piezometric data. A side slope of 3 horizontal to 1 vertical may be possible in the final design, as suggested by PFRA (2003), but this will require a stability analysis be completed based on a detailed geotechnical investigation of the soils and piezometric conditions in the east back slope. Boreholes would be also be required to characterize the borrow potential of the soils along the spillway. Once the boreholes are drilled a spillway profile should be plotted up as well as several cross sections to aid the geotechnical assessment. The acceptable borrow generated from the spillway excavation would be used as engineered fill to complete the Dam grade raise and widening; therefore the final slope determination will be a combination of stability and borrow requirements.

9.6.2 Spillway Subgrade

The spillway is to be an earth structure. Excavation of the spillway will encountered a variety of soils most of which will be suitable as fill, but which will likely require moisture conditioning. The topsoil would be stripped, stockpiled and later reused to dress the subgrade. The topsoil on the

east side of the Dam is however relatively thin, and additional organic material, possibly from the Rushlake Creek Valley bottom, may need to be imported. The side slopes and floor of the spillway will need to be protected from erosion which is discussed in Subsection 9.7. Some subcutting of the subgrade may be required if problematic soils or conditions are encountered. The clay shale that is known to be present, at relatively shallow depths within portions of the east back slope, will not support vegetation if it forms the subgrade surface and will need to be amended or removed and replaced to promote vegetation growth.

9.6.3 Spillway Terminal Structure

A concrete spillway energy dissipating terminal (outlet) will be required for Alternative 3. For the final design phase boreholes will be required in the spillway outlet area to characterize the subgrade and confirm if problematic soils and conditions are present. Also a slope stability analysis of the spillway outlet should be completed for the final design phase. The spillway outlet will likely require a piled foundation to support the spillway floor and wall system. Cantilevered retaining walls could be considered in the final design phase depending on the outcome of the geotechnical investigation. Problematic soils, such as organics, desiccated clay, clay shale, should be removed a minimum of 900 mm from the underside of the concrete slab and replaced with suitable, free draining granular materials in order to reduce settlement and swelling problems. A deeper depth of subcut may be required depending on the results of the geotechnical investigation required for the spillway outlet design. A subdrain system may also be required if seepage conditions are encountered. In addition, a cut-off wall would be required near the upstream concrete spillway to earthen spillway transition. Detailed design of any subdrain system and the cut-off wall would be completed in the final design phase.

9.6.4 Spillway Outlet Deflector Berm

A spillway outlet deflector berm may be required depending on the final hydraulic design. The dimensions, location and orientation of a deflector berm would be established in the hydraulic final design phase. If the deflector berm is required then it should be constructed using the methods used to construct the main Dam embankment grade raise and extension. Some boreholes would be required in the location of the proposed deflector berm. The problems previously noted for the Dam embankment grade raise and extension would also apply to the construction of the deflector berm. The deflector berm would require robust riprap erosion protection which would be designed in the final design phase.

9.7 Erosion Protection

Establishing vegetation on the back–slopes and along the bottom of the earthen spillway should provide adequate erosion protection. Topsoil can be placed on the slopes and spillway floor and seeded with a deep rooting grass species capable of withstanding the local climate. Additional short–term erosion protection may be required to control any significant erosion during the period after construction and before vegetation is established. Turf reinforcement mats could be installed which aid in the establishment of vegetation and provide long–term erosion protection.

Alternative erosion control measures such as soil cement were not considered because of the shallow spillway slopes. However, if the spillway slope is steeper than proposed, alternative measures may have to be considered.

MDH can provide a detailed erosion control plan as part of the design phase.

9.8 Preliminary Design of Riprap Erosion Protection for the Spillway Outlet

Stone riprap should be placed at the spillway exit and in the stilling basin to prevent erosion. The rock riprap should be angular or sub–angular and as near to equi–dimensional as practical.

The longevity of the spillway may be significantly improved if a granular filter is placed under the riprap. Severe eddies might otherwise lead to surficial slumping and eventually degradation of the entire spillway channel. A 300 mm granular filter should underlie the riprap over the entire area. A cut-off wall will likely be required at the end of the channel to avoid water flow and piping of fines from under the rip-rap or concrete chute structure. The design of the cut off wall can be done in the detailed design stage.

It has been assumed that a suitable source of material will be located near the construction site. Commercially available erosion protection methods are also available, although these products have not been considered during the design process. If no suitable source of stone riprap can be located, MDH can provide services related to the design of alternate erosion protection for the abutments.

MDH can provide a detailed rip rap design and specification as part of the erosion protection plan during the detailed design phase.

9.9 East and West Low Level Outlets

As previously mentioned the scope of work for this present study did not include an inspection or assessment of the existing condition of the east and west low-level outlets. Raising the embankment will also necessitate increasing the width of the Dam and therefore increasing the length of the west outlet only as well as increasing the load on the existing portion of the outlets. The east outlet was extended to accommodate the 1940's early 1950's Dam grade raises and widening. The east outlet extension was founded on a timber pile foundation due to soft subgrade problems and over time it experienced serious cracking that was repaired in 1969, as reported by PFRA. Similar problems may occur if the proposed Dam grade raise and widening are constructed. Some issues that will need to be addressed in the final design phase are then:

1. What is the condition of the existing low-level outlets? Are there repairs to be made to either of them before they are extended? Do they require installation of an inner liner due to deterioration?
2. What materials should be used to extend them?
3. What are the subgrade conditions along the outlets' routes?
4. What magnitude of settlement can be expected along the extended outlets and is there means to alleviate or reduce these settlements and/or the differential settlements?
5. Will grade raise result in structural distress due to increased loading of the existing outlet or its extension?
6. Do the outlet extensions require a foundation system to reduce settlements and/or structural distress? If so what type of foundation system would be most appropriate, piles, geosynthetics?

To adequately address the above issues in the final design phase boreholes will be required along the route of the outlets. Samples should be obtained from these boreholes, for consolidation testing, particularly of the alluvium, in order to establish the settlement magnitude and the differential settlement potential, and also for strength testing.

9.10 Instrumentation and Monitoring

Instrumentation will be required to monitor stability of the embankment, settlement, and porewater pressures during and after construction. The following recommendations are based on the method of installation and construction sequencing. Monitoring frequency is initially high but diminishes with time. Potential locations will be discussed; however, exact locations of

instruments will have to be considered closer to construction to ensure the instruments safety and functionality during construction and post construction.

9.10.1 Slope Stability

Three SIs were installed under this investigation along the downstream face of the dam. When the dam embankment is raised, the SI's will have to be raised by adding sections onto the existing casing as the embankment fill is placed. Protective steel casing should be installed to protect the SIs from damage. An adequate monitoring program will need to be defined at the detailed design stage.

9.10.2 Settlement

Settlement instrumentation in the form of a special horizontal tube should be installed in the embankment to monitor consolidation of the foundation soils and embankment earth fill. The settlement instrumentation should be located approximately where the current SI's are along the length of the dam; however, this may require adjustment in the field based on accessibility, field observations and safety of the instrumentation. The horizontal tubes will provide a settlement profile beneath the entire embankment. The tubes can be read using a Shape Acceleration Array (SAA). SAA is a rope-like array of sensors and microprocessors that fits into a small (27 mm ID) casing. Any deformation that moves the casing is accurately measured as a change in shape of the SAA. Vibrations may also be measured, at multiple points.

9.10.3 Porewater Pressure

VWPs were installed in the three boreholes drilled along the embankment. Similar to the SI's, the VWP wires will have to be extended up through the embankment during construction when the embankment is raised. The VWPs are necessary to monitor the development of excess porewater pressures during construction and could form an integral part of monitoring long–term performance of the embankment drainage system. The VWPs should be in place prior to fill placement and sand drainage layer installation so existing conditions can be determined, and the effectiveness of the sand drain can be measured. The piezometer leads may be protected in steel pipes. Similar to the SI's, the VWPs would require an adequately designed monitoring program at the detailed design stage. The VWPs can be set up to include real–time monitoring with alarms if desired.

10.0 Closure

MDH Engineered Solutions Corp., hereinafter collectively referred to as “MDH”, has exercised reasonable skill, care and diligence in preparing this report. MDH will not be liable under any circumstances for the direct or indirect damages incurred by any individual or entity due to the contents of this report, omissions and/or errors within, or use thereof, including damages resulting from loss of data, loss of profits, loss of use, interruption of business, indirect, special, incidental or consequential damages, even if advised of the possibility of such damage. This limitation of liability will apply regardless of the form of action, whether in contract or tort, including negligence.

MDH has prepared this report for the exclusive use of Northwest Hydraulics Consultants and the Agri-Environmental Services Branch of Agriculture and Agri-Food Canada and does not accept any responsibility for the use of this report for any purpose other than intended. Any alternative use, reliance on, or decisions made based on this document are the responsibility of the alternative user or third party. MDH accepts no responsibility to any third party for the whole or part of the contents and exercise no duty of care in relation to this report. MDH accepts no responsibility for damages suffered by any third party as a result of decisions made or actions based on this report.

Should you have any questions or comments please contact us.

Regards,

MDH Engineered Solutions Corp.

Association of Professional Engineers
And Geoscientists of Saskatchewan
Certificate of Authorization number 662

Doug Charleson, M.Eng., P.Eng.

Darrell Mihial, M.A.Sc., P.Eng.

Jorge Antunes, M.Sc., P.Eng.

11.0 References

Aber, J.S., and Ber, A., 2007. "Glaciotectonism." Developments in Quaternary Science Series 6, Elsevier Publishers, UK.

Ayres, K.W., Acton, D.F., and Ellis, J.G., 1985. "The Soils of the Swift Current, 72J, Saskatchewan." Saskatchewan Institute of Pedology Report S6.

Caldwell, W.G.E., 1968. "The late Cretaceous Bearpaw Formation in the South Saskatchewan River Valley." Saskatchewan Research Council, Geology Division, Report No. 8.

Canadian Dam Association, 2007. Dam Safety Guidelines.

Christiansen, E.A., 1959. "Glacial Geology of the Swift Current Area, Saskatchewan." Saskatchewan Research Council, Dept. Of Mineral Resources, Geological Science Branch, Sedimentary Geology Division, Report No. 32.

Christiansen, E.A., 1979. "The Wisconsinan deglaciation of southern Saskatchewan and adjacent areas." Canadian Journal of Earth Sciences, Vol. 16, pp. 913-938.

Christiansen, E.A., and Sauer, E.K., 1997. "The Dirt Hills structure: an ice-thrust feature in southern Saskatchewan, Canada." Canadian Journal of Earth Sciences, Vol. 34, pp. 76-85.

Crawford, L.W., 1951. "An investigation of the properties of Saskatchewan volcanic ash." A MSc thesis submitted to the Dept. of chemistry, Univ. of Saskatchewan .

Iverson, N.L., 1948. "Highfield Dykes: E ½ Sec. 25-15-11, and Sec. 30-15-10, W. 3rd." PFRA internal report.

Jaspar, J.L., and Peters, N., 1979. "Foundation performance of Gardiner Dam." Canadian Geotechnical Journal, Vol. 16, pp. 758-788.

Klassen, R.W., 2002. "Surficial Geology of the Cypress Lake and Wood Mountain Map Areas, Southwestern Saskatchewan." Geological Survey of Canada, Bulletin 562.

Keith Consulting, 1975. Basis of Design Report for Saskatchewan Power Corporation Proposed Poplar River Power Station Dam Section 16 & 17, TWP1, RGE 26, W2nd. File No. WD 7021.

Krahn, 2007. Stability Modelling with SLOPE/W – An Engineering Methodology.

Krahn, 2007. Seepage Modelling with SEEP/W – An Engineering Methodology.

Kupsch, W.O., 1962. "Ice-thrust ridges in western Canada." Journal of Geology, Volume 70, pp. 582-590.

Maathuis, H., and Simpson, M., 2007. "Groundwater Resources of the Swift Current (72J) Area, Saskatchewan." Saskatchewan Research Council Publication No. 12178-1E07.

MDH, 2007. Morrison Dam Riprap Study (MDH Reference No. – M1376-130007).

MDH, 2008. Dam Safety Review at Morrison Dam – Draft Hydrology Report (MDH Reference No. – M1454-130008).

MDH, 2009. Dam Safety Review at Morrison Dam – Stratigraphic Drilling Investigation and Instrumentation (MDH Reference No. – M1454-130009) - Report Pending.

Occhietti, S., 1973. "Les structures et déformations engendrées par les glaciers – Essai de mise au point." Revue Géographique de Montréal 27, pp. 1052-1056.

Ross, M., Campbell, J.E., Parent, M., and Adams, R.S., 2009. "Palaeo-ice streams and the subglacial landscape mosaic of the North American mid-continental prairies." Boreas, Volume 38, pp. 421-439.

Sauer, E.K., 1978. "The engineering significance of glacial ice-thrusting." Canadian Geotechnical Journal, Vol. 15, No. 4, pp. 457-472.

Vanapalli, S.K., Fredlund, D.G., Pufahl, D.E., and Clifton, A.W., 1996. Model for the Prediction of Shear Strength with respect to Soil Suction. Canadian Geotechnical Journal, Vol. 33, pp. 379-392.

Worchester, W.G., 1950. "Clay resources of Saskatchewan." Dept. of Mineral Resources, Technical and Economic Series, Rpt. No. 7.

Appendix A

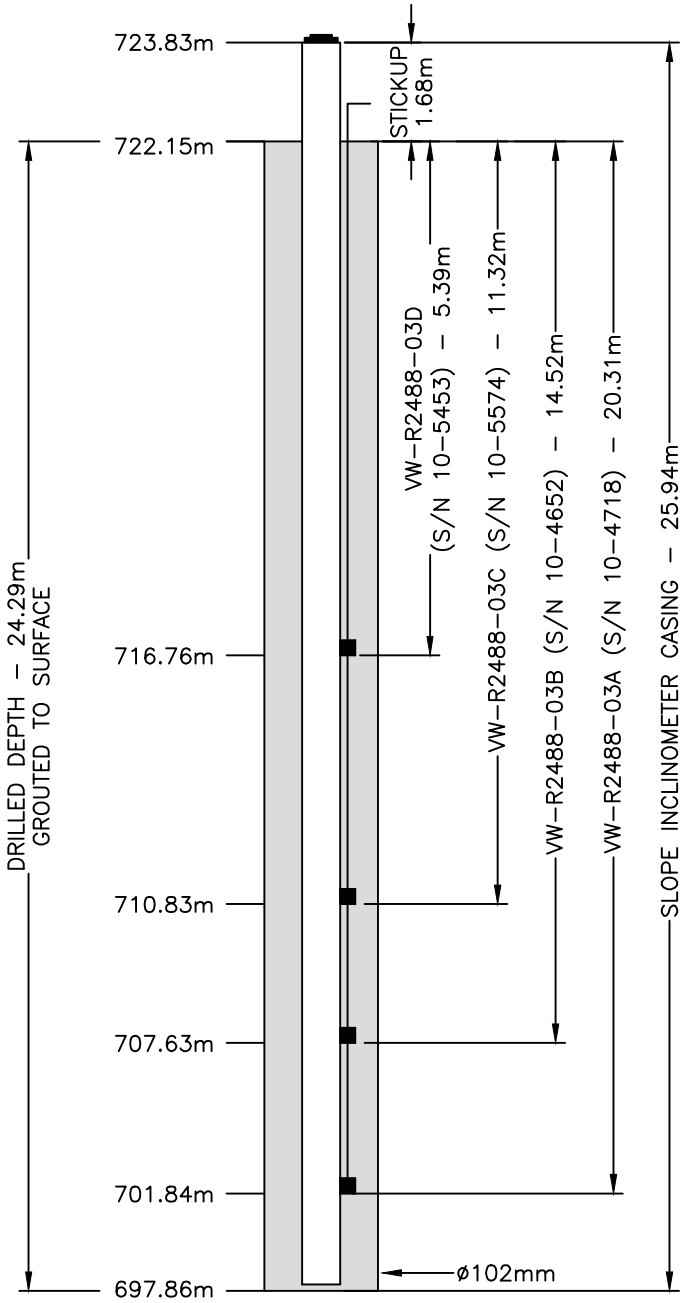
Schematic of VW Piezometer Installations

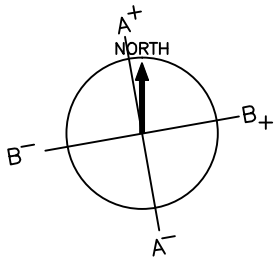
SLOPE INCLINOMETER SI-R2488-03 AND VIBRATING WIRES SI-VW-R2488-03A THROUCH D NORTHWEST HYDRAULIC CONSULTANTS - HIGHFIELD DAM

2011

5575607.69 N 329407.68 E
NAD 83 ZONE 13
NW36-15-11-W3
72J06

DRAFT





A^+ AXIS NOTCHED

SLOPE INCLINOMETER CASING SPECIFICATIONS:
- Rocktest GEO-LOK ABS 85 mm diameter
- Quick coupling threaded locking collar with o-ring

VIBRATING WIRE SPECIFICATIONS:
- VW-2488-03A
  Slope Indicator Part # 52611034 (S/N 10-4718)
  Cable Length - 45 m
  100 psi
- VW-2488-03B
  Slope Indicator Part # 52611033 (S/N 10-4652)
  Cable Length - 30 m
  100 psi
- VW-2488-03C
  Slope Indicator Part # 52611024 (S/N 10-5574)
  Cable Length - 30 m
  50 psi
- VW-2488-03D
  Slope Indicator Part # 52611030 (S/N 10-5453)
  Cable Length - 15 m
  100 psi

GROUT SPECIFICATIONS:
- cement-bentonite (91%-9% by weight)
- mixed to specific gravity of approximately 1.7

NOTES: 1. VW PIEZOMETERS ATTACHED TO SI CASING.

All depths are expressed in metres above or below natural ground surface, unless otherwise indicated. All elevations are expressed in metres above sea level.

SUPERVISOR	F. LIU, M.A.Sc., P.Eng.	CLIENT	nhc northwest hydraulic consultants
CONTRACTOR	BOART LONGYEAR	PRODUCED BY	MDH ENGINEERED SOLUTIONS
OPERATOR	S. MADDEN/S. WURZ	APPROVED BY	
TYPE OF DRILL RIG	BL 100C SONIC	DRAWN BY	S. RUSSELL, B.Sc.
DATE INSTALLED	14-JAN-11 TO 16-JAN-11	PROJECT No.	R2488-265010
		SCALE	NOT TO SCALE
		DATE	24-JAN-11

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\2. Drafting\1. Boreholes\VW-SI-2488-03.dwg

Appendix B

VW Piezometer Calibration Sheets

VW Piezometer Calibration Certificate

Serial #: 10-5594
Range : 100 psi
Cable Length: 30 m
Date of Calibration: 9/30/2010

Part #: 52611033
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.716834E-4	2.256710E-2	1.276407E+3
psi	-2.490057E-5	3.273081E-3	1.851272E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.275216E+3	2.285765E-2	2.717458E-2	-1.719388E-4	5.937360E-5	-2.233941E-3
psi	1.849480E+2	3.315105E-3	3.941201E-3	-2.493674E-5	8.611109E-6	-3.239943E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.5 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2793.0	0.02	0.1	-0.02
10.00	68.9	2718.6	9.99	68.9	0.01
20.00	137.9	2642.0	19.96	137.6	0.04
30.00	206.8	2562.6	29.99	206.8	0.01
40.00	275.8	2480.7	40.01	275.9	-0.01
50.00	344.7	2396.1	50.01	344.8	-0.01
60.00	413.7	2308.3	60.01	413.8	-0.01
70.00	482.6	2216.9	70.01	482.7	-0.01
80.00	551.6	2121.4	80.01	551.6	-0.01
90.00	620.5	2021.3	90.01	620.6	-0.01
100.00	689.5	1916.0	99.99	689.4	0.01

VW Piezometer Calibration Certificate

Serial #: 10-5593
Range : 100 psi
Cable Length: 30 m
Date of Calibration: 9/30/2010

Part #: 52611033
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.789069E-4	5.512037E-2	1.244815E+3
psi	-2.594825E-5	7.994534E-3	1.805452E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.241701E+3	5.587802E-2	1.892297E-1	-1.791706E-4	2.637479E-5	-3.772972E-3
psi	1.800871E+2	8.104136E-3	2.744448E-2	-2.598558E-5	3.825205E-6	-5.472041E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.3 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2796.0	0.04	0.3	-0.04
10.00	68.9	2722.5	9.98	68.8	0.02
20.00	137.9	2646.4	19.98	137.8	0.02
30.00	206.8	2567.8	29.98	206.7	0.02
40.00	275.8	2486.5	39.99	275.7	0.01
50.00	344.7	2402.2	50.01	344.8	-0.01
60.00	413.7	2314.8	60.01	413.8	-0.01
70.00	482.6	2223.7	70.01	482.7	-0.01
80.00	551.6	2128.4	80.01	551.6	-0.01
90.00	620.5	2028.4	90.00	620.5	0.00
100.00	689.5	1922.8	99.98	689.3	0.02

VW Piezometer Calibration Certificate

Serial #: 10-5452
Range : 100 psi
Cable Length: 15 m
Date of Calibration: 9/22/2010

Part #: 52611030
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-2.152016E-4	2.861201E-2	1.638170E+3
psi	-3.121236E-5	4.149822E-3	2.375965E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.637993E+3	2.867133E-2	2.755524E-3	-2.153162E-4	3.424328E-5	-2.439103E-3
psi	2.375625E+2	4.158278E-3	3.996409E-4	-3.122788E-5	4.966393E-6	-3.537495E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.2 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2826.2	0.02	0.1	-0.02
10.00	68.9	2767.6	10.01	69.0	-0.01
20.00	137.9	2707.8	19.98	137.8	0.02
30.00	206.8	2646.4	29.98	206.7	0.02
40.00	275.8	2583.5	39.99	275.7	0.01
50.00	344.7	2518.9	50.01	344.8	-0.01
60.00	413.7	2452.7	60.01	413.8	-0.01
70.00	482.6	2384.6	70.01	482.7	-0.01
80.00	551.6	2314.4	80.01	551.6	-0.01
90.00	620.5	2242.0	90.01	620.6	-0.01
100.00	689.5	2167.3	99.98	689.3	0.02

VW Piezometer Calibration Certificate

Serial #: 10-4651
Range : 100 psi
Cable Length: 30 m
Date of Calibration: 8/6/2010

Part #: 52611033
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.928653E-4	4.359411E-2	1.426774E+3
psi	-2.797275E-5	6.322792E-3	2.069361E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.427608E+3	4.131443E-2	1.052742E-1	-1.925583E-4	5.155894E-5	-2.164258E-3
psi	2.070498E+2	5.991940E-3	1.526819E-2	-2.792724E-5	7.477729E-6	-3.138880E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.5 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2835.1	0.02	0.1	-0.02
10.00	68.9	2768.8	10.00	68.9	0.00
20.00	137.9	2700.8	19.97	137.7	0.03
30.00	206.8	2630.6	30.00	206.8	0.00
40.00	275.8	2558.4	40.02	275.9	-0.02
50.00	344.7	2484.3	50.00	344.7	0.00
60.00	413.7	2407.7	60.00	413.7	0.00
70.00	482.6	2328.4	70.01	482.7	-0.01
80.00	551.6	2246.2	80.00	551.6	0.00
90.00	620.5	2160.7	90.00	620.5	0.00
100.00	689.5	2071.6	99.99	689.4	0.01

VW Piezometer Calibration Certificate

Serial #: 10-5573
Range : 50 psi
Cable Length: 30 m
Date of Calibration: 9/30/2010

Part #: 52611024
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.359379E-4	6.137277E-3	1.071635E+3
psi	-1.971613E-5	8.901368E-4	1.554275E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	-3.889623E+1	7.719380E-1	1.181961E+1	-2.681930E-4	-3.840785E-3	-4.569868E-2
psi	-5.641222E+0	1.119562E-1	1.714229E+0	-3.889673E-5	-5.570391E-4	-6.627800E-3

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.5 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2830.5	-0.01	-0.1	0.03
5.00	34.5	2784.8	5.01	34.5	-0.01
10.00	68.9	2738.5	10.01	69.0	-0.01
15.00	103.4	2691.3	15.02	103.6	-0.03
20.00	137.9	2643.5	20.00	137.9	0.00
25.00	172.4	2594.8	24.99	172.3	0.02
30.00	206.8	2545.0	29.99	206.8	0.02
35.00	241.3	2494.3	34.98	241.2	0.03
40.00	275.8	2442.2	40.01	275.9	-0.02
45.00	310.3	2389.2	45.01	310.3	-0.02
50.00	344.7	2335.1	50.00	344.7	0.00

VW Piezometer Calibration Certificate

Serial #: 10-5449
Range : 100 psi
Cable Length: 15 m
Date of Calibration: 9/22/2010

Part #: 52611030
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.674306E-4	2.486756E-3	1.373290E+3
psi	-2.428375E-5	3.606735E-4	1.991789E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.372162E+3	1.942841E-3	1.049590E-1	-1.674449E-4	4.356277E-5	-2.187244E-3
psi	1.990082E+2	2.817753E-4	1.522248E-2	-2.428497E-5	6.318023E-6	-3.172217E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.2 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2871.3	0.01	0.1	-0.01
10.00	68.9	2798.6	9.99	68.9	0.01
20.00	137.9	2723.9	19.98	137.8	0.02
30.00	206.8	2646.9	30.00	206.8	0.00
40.00	275.8	2567.8	39.99	275.7	0.01
50.00	344.7	2486.0	50.00	344.7	0.00
60.00	413.7	2401.4	60.01	413.8	-0.01
70.00	482.6	2313.8	70.01	482.7	-0.01
80.00	551.6	2222.7	80.01	551.6	-0.01
90.00	620.5	2127.7	90.01	620.6	-0.01
100.00	689.5	2028.5	99.99	689.4	0.01

VW Piezometer Calibration Certificate

Serial #: 10-4718
Range : 100 psi
Cable Length: 45 m
Date of Calibration: 8/9/2010

Part #: 52611034
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.636054E-4	7.976751E-3	1.312098E+3
psi	-2.372896E-5	1.156930E-3	1.903037E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.310633E+3	7.512558E-3	1.421739E-1	-1.636268E-4	3.704521E-5	-2.829747E-3
psi	1.900845E+2	1.089566E-3	2.061986E-2	-2.373123E-5	5.372765E-6	-4.104057E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.2 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2856.1	0.04	0.3	-0.04
10.00	68.9	2781.1	9.99	68.9	0.01
20.00	137.9	2703.7	19.97	137.7	0.03
30.00	206.8	2623.6	30.01	206.9	-0.01
40.00	275.8	2542.0	39.91	275.2	0.09
50.00	344.7	2456.0	50.01	344.8	-0.01
60.00	413.7	2367.7	60.02	413.8	-0.02
70.00	482.6	2276.0	70.02	482.8	-0.02
80.00	551.6	2180.3	80.03	551.8	-0.03
90.00	620.5	2079.9	90.06	620.9	-0.06
100.00	689.5	1976.0	99.94	689.1	0.06

VW Piezometer Calibration Certificate

Serial #: 10-4652
Range : 100 psi
Cable Length: 30 m
Date of Calibration: 8/6/2010

Part #: 52611033
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.877990E-4	5.985137E-3	1.500924E+3
psi	-2.723794E-5	8.680708E-4	2.176906E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.504691E+3	1.904072E-3	2.994431E-2	-1.872490E-4	9.562489E-5	-4.333672E-3
psi	2.182293E+2	2.761526E-4	4.342902E-3	-2.715721E-5	1.386873E-5	-6.285239E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.4 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2842.9	0.02	0.1	-0.02
10.00	68.9	2777.4	9.99	68.9	0.01
20.00	137.9	2710.1	19.99	137.8	0.01
30.00	206.8	2641.0	30.00	206.8	0.00
40.00	275.8	2570.2	39.99	275.7	0.01
50.00	344.7	2497.3	49.99	344.7	0.01
60.00	413.7	2422.1	60.00	413.7	0.00
70.00	482.6	2344.5	70.01	482.7	-0.01
80.00	551.6	2264.2	80.02	551.7	-0.02
90.00	620.5	2181.1	90.01	620.6	-0.01
100.00	689.5	2094.8	99.98	689.3	0.02

VW Piezometer Calibration Certificate

Serial #: 10-5574
Range : 50 psi
Cable Length: 30 m
Date of Calibration: 9/30/2010

Part #: 52611024
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.357053E-4	3.647153E-2	9.592993E+2
psi	-1.968239E-5	5.289749E-3	1.391346E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	9.624055E+2	3.228410E-2	1.662239E-1	-1.348804E-4	1.516503E-6	-1.247193E-3
psi	1.395802E+2	4.682248E-3	2.410789E-2	-1.956206E-5	2.199424E-7	-1.808837E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.4 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2796.6	-0.01	-0.1	0.01
5.00	34.5	2748.4	5.00	34.5	0.00
10.00	68.9	2699.3	10.00	68.9	-0.01
15.00	103.4	2649.2	15.01	103.5	-0.02
20.00	137.9	2598.3	20.00	137.9	0.00
25.00	172.4	2546.3	24.99	172.3	0.02
30.00	206.8	2493.1	29.99	206.8	0.03
35.00	241.3	2438.5	35.00	241.3	0.01
40.00	275.8	2382.4	40.02	275.9	-0.05
45.00	310.3	2325.3	45.01	310.3	-0.02
50.00	344.7	2266.8	49.99	344.7	0.02

VW Piezometer Calibration Certificate

Serial #: 10-5453
Range : 100 psi
Cable Length: 15 m
Date of Calibration: 9/22/2010

Part #: 52611030
Cable Part # : 50613524
Calibrated by: KB
Note:

ABC Calibration Factors

	A	B	C
kPa	-1.591758E-4	-5.689660E-3	1.253550E+3
psi	-2.308650E-5	-8.252155E-4	1.818121E+2

Pressure in kPa/psi = $(A \times Hz^2) + (B \times Hz) + C$, where Hz is frequency in Hertz.

TI Calibration Factors

	C0	C1	C2	C3	C4	C5
kPa	1.252665E+3	-5.630325E-3	4.276170E-2	-1.593009E-4	3.525179E-5	-1.740928E-3
psi	1.816773E+2	-8.165809E-4	6.201842E-3	-2.310383E-5	5.112660E-6	-2.524914E-4

Pressure in kPa/psi = $C0 + (C1 \times Hz) + (C2 \times T) + (C3 \times Hz^2) + (C4 \times Hz \times T) + (C5 \times T^2)$

Where Hz is the frequency reading in Hertz and T is the Thermistor reading in degrees C.
TI factors are calculated from temperatures at 5.0, 15.0 and 25.0 degrees C.
Applied pressure and temperature are NIST traceable.

Summary of Test Results at 15°C

Thermistor reading is 14.6 °C.

Applied Pressure is referenced to 1 atm. Calculated Pressure uses ABC Calibration factors.

Applied (psi)	Equivalent (kPa)	Frequency (Hz)	Calculated (psi)	Calculated (kPa)	Error (%FS)
0.00	0.0	2788.5	0.00	0.0	0.00
10.00	68.9	2710.1	10.01	69.0	-0.01
20.00	137.9	2629.6	20.00	137.9	0.00
30.00	206.8	2546.6	29.99	206.8	0.01
40.00	275.8	2460.7	39.99	275.7	0.01
50.00	344.7	2371.7	49.99	344.7	0.01
60.00	413.7	2279.2	60.00	413.7	0.00
70.00	482.6	2182.9	70.00	482.6	0.00
80.00	551.6	2082.2	80.00	551.6	0.00
90.00	620.5	1976.4	90.00	620.5	0.00
100.00	689.5	1864.8	99.99	689.4	0.01

Appendix C

MDH Borehole Logs

BOREHOLE BH-R2488-01
NORTHWEST HYDRAULIC CONSULTANTS - HIGHFIELD DAM
2011

5575734.82 N 330050.94 E
NAD 83 ZONE 13
NE36-15-11-W3
072J06

FILL: till, clayey, silty, some sand to sandy, trace fine to medium gravel, trace cobble, olive brown (2.5Y 4/3 moist) to dark olive brown(2.5Y 3/3 moist), oxidized, firm to stiff, medium plasticity, moist, calcareous, trace Fe staining, trace rootlets, trace organics
- from 0 – 1.5 m: hard
- from 3.3 m to 3.4 m: organic soil, black
- from 3.4 m to 3.5 m: sandy, olive gray (5Y 5/2 moist)
- from 3.5 m to 4.6 m: mottled olive brown (2.5Y 4/3 moist) and dark gray (5Y 4/1 moist), firm to soft
- below 6.0 m: moist to very moist
- at 6.2 m: sand lenses

SILT: some clay to clayey, some fine sand, olive (5Y 4/3 moist) to olive gray, (5Y 4/2 moist) oxidized, soft, very moist, low plasticity, calcareous

SAND: silty, trace to some clay, trace gravel, olive gray (5Y 4/2 moist) to dark olive gray (5Y 3/2 moist), oxidized, wet, loose to medium dense, calcareous

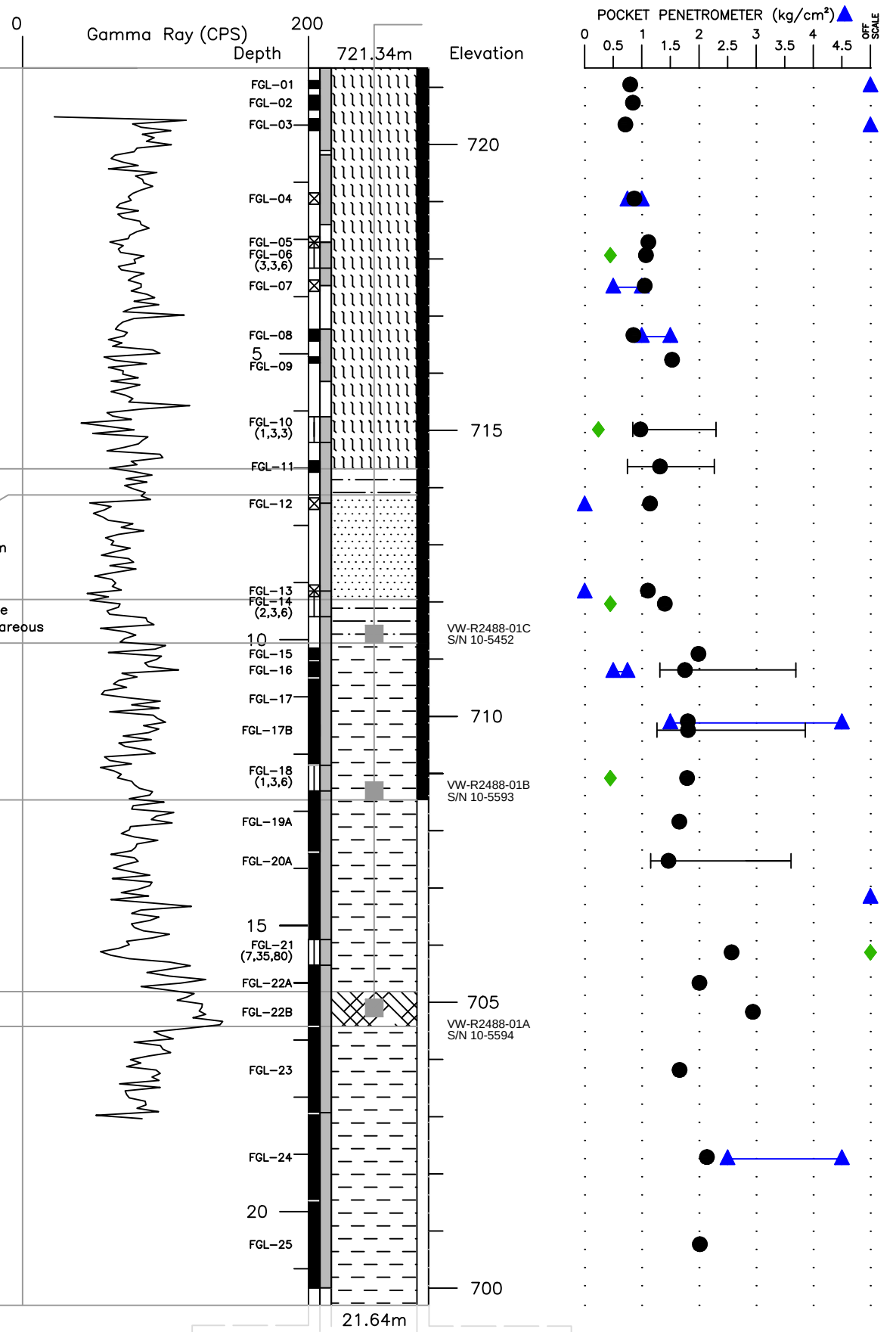
SILT: some clay to clayey, some fine sand, olive (5Y 4/3 moist) to olive gray (5Y 4/2 moist), oxidized, soft, low plasticity, very moist, calcareous

CLAY SHALE (disturbed): silty, sandy, olive (5Y 4/3 moist) to olive gray (5Y 4/2 moist), oxidized, firm to very stiff, high plasticity, moist, non–calcareous, Fe staining, trace gypsum
- at 11.6 m: trace yellowish bentonite
- at 12.4 m: trace yellowish bentonite

CLAY SHALE: silty, some sand, very dark gray (5Y 3/1, moist), unoxidized, stiff to hard, high plasticity, moist, non–calcareous, silt pockets and inclusions, trace gypsum, fissured and jointed
- from 15.5 m to 16.2 m: some blocky siltstone layers

VOLCANIC ASH: light gray (5Y 7/1 moist), non–plastic

CLAY SHALE: silty, some sand, very dark gray (5Y 3/1, moist), unoxidized, stiff to very stiff, high plasticity, moist, non–calcareous, fissured and jointed, silt pockets



SAMPLE LEGEND
- SPLIT SPOON
- CORE
- BAGGED

OXIDATION LEGEND
- OXIDIZED ZONE
- UNOXIDIZED ZONE

MATERIAL RECOVERY LEGEND
- RECOVERED MATERIAL
- NO MATERIAL

DRAFT

LIMITATION

This drill log is a summary of the conditions estimated by the field personnel at the specific location at the time of drilling. The conditions and properties described above will vary between locations and may vary with time.

FOR INCLINOMETER AND PIEZOMETER INFORMATION SEE DWG: SI–VW–R2488–01.dwg

SUPERVISOR	F. LIU, M.A.Sc., P.Eng.	CUTTING SAMPLE INTERVAL 1.52m(5.0ft)		COND. WATER µsiemens/cm	
LOGGED BY	F. LIU, M.A.Sc., P.Eng.	TYPE OF DRILL RIG BL 100C SONIC		COND. MUD	
GEOLOGY BY	N/A	TYPE OF LOGGER MOUNT SOPRIS MATRIX		SPECIFIC GRAVITY	
CONTRACTOR	BOART LONGYEAR CANADA	ABANDONMENT GROUTED WITH VW PIEZOMETER		NG RATE	
OPERATOR	S. MADDEN/S. WURZ	BIT SIZE 4 inch	INTERVAL 0.0m–21.3m	LOGGING SPEED	GAMMA 3.6m/min
DATE DRILLED	07–JAN–11	BIT SIZE 6 inch	INTERVAL 21.3m–21.6m		RES

CLIENT: nhc northwest hydraulic consultants

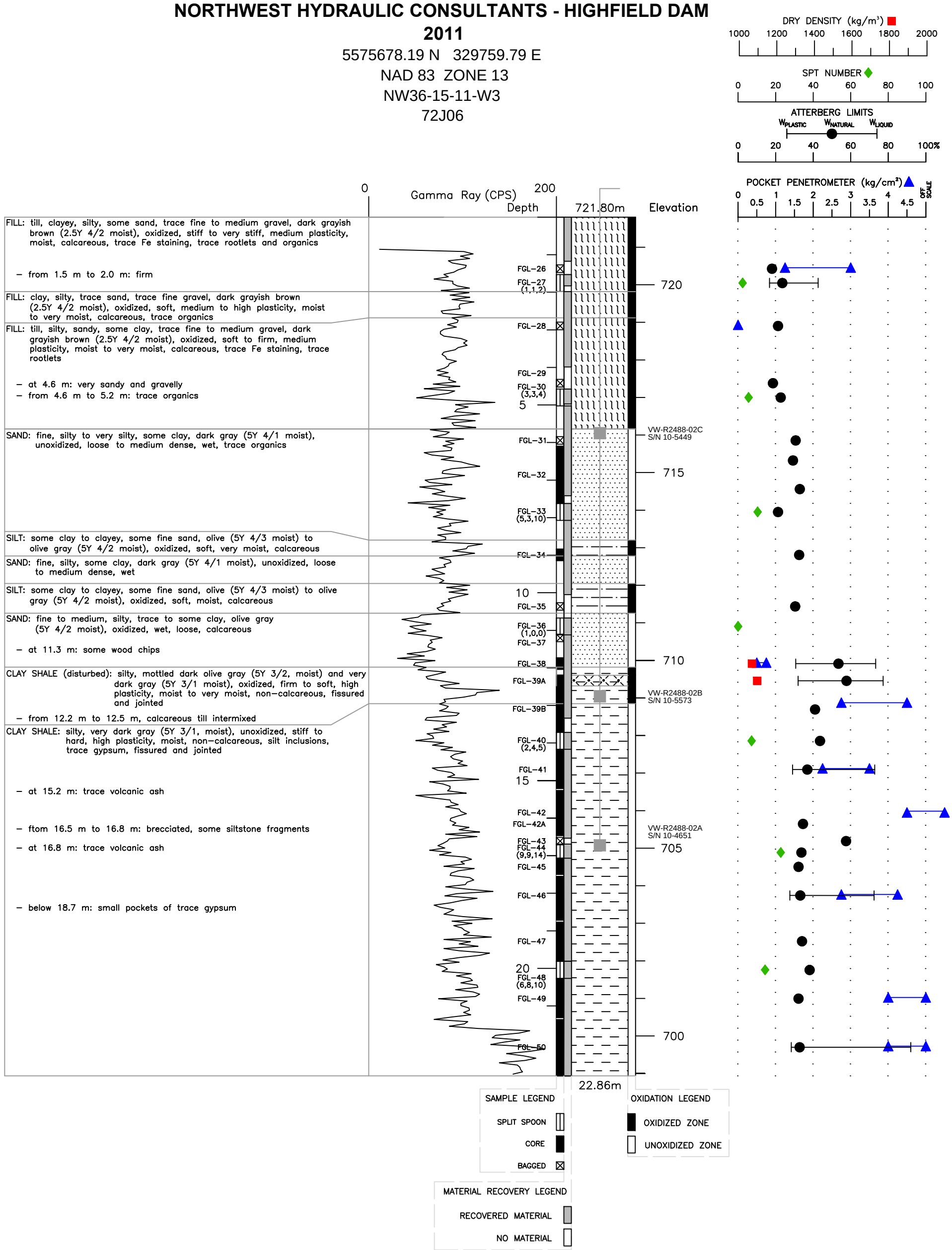
PRODUCED BY: MDH ENGINEERED SOLUTIONS

APPROVED BY	J. ANTUNES, M.Sc., P.Eng.		
DRAWN BY	S. RUSSELL, B.Sc.		
PROJECT No.	R2488–265010		
SCALE	1:100	DATE	21–JAN–11

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488–265010 Spillway Upgrade at Highfield Dam\2. Drafting\1. Boreholes\BH–R2488–01.dwg

BOREHOLE BH-R2488-02
NORTHWEST HYDRAULIC CONSULTANTS - HIGHFIELD DAM
2011

5575678.19 N 329759.79 E
NAD 83 ZONE 13
NW36-15-11-W3
72J06



DRAFT

NOTES: 1.
2.

LIMITATION

This drill log is a summary of the conditions estimated by the field personnel at the specific location at the time of drilling. The conditions and properties described above will vary between locations and may vary with time.

FOR INCLINOMETER AND PIEZOMETER INFORMATION SEE DWG: SI–VW–R2488–02.dwg

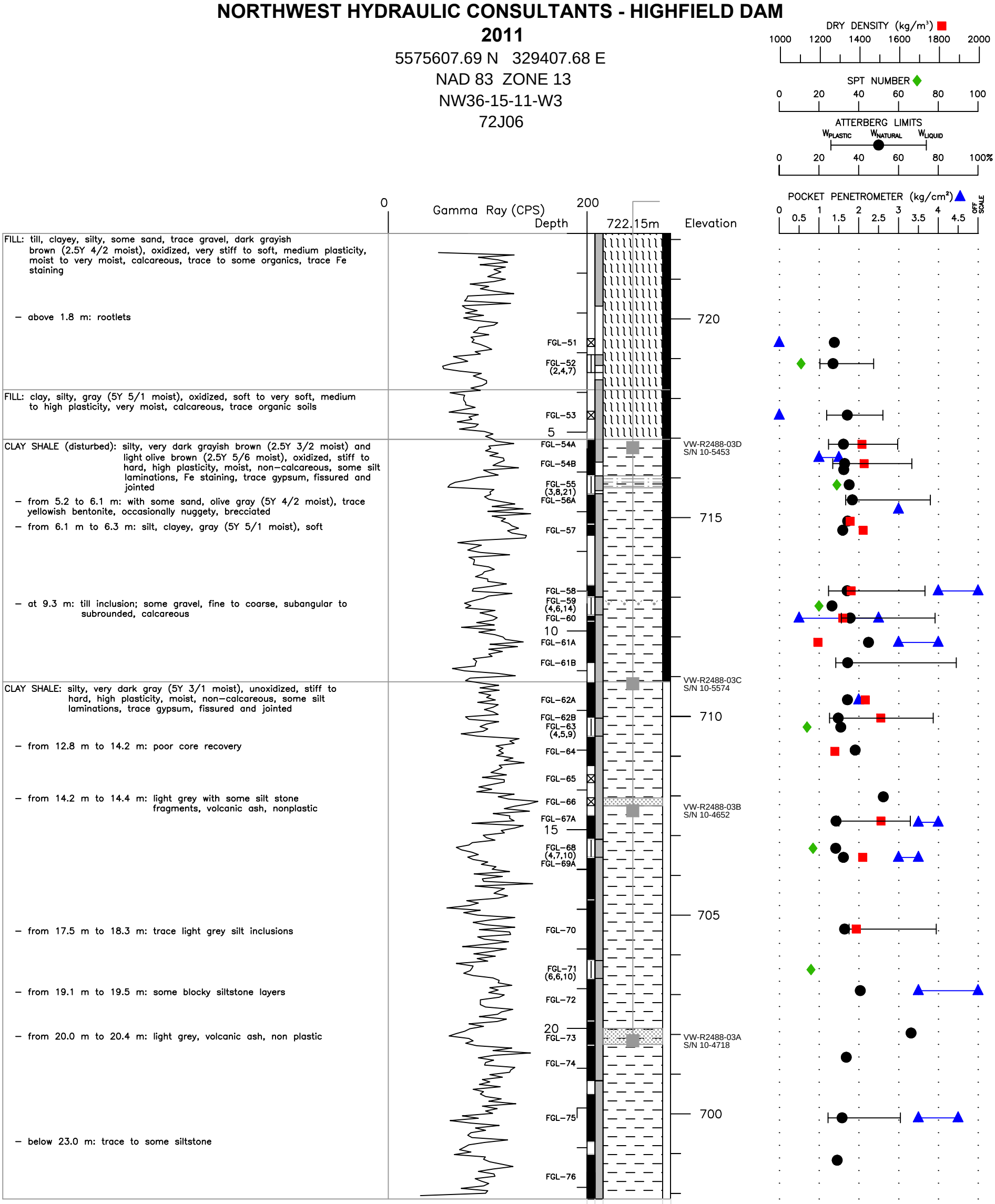
SUPERVISOR	F. LIU, M.A.Sc., P.Eng.	CUTTING SAMPLE INTERVAL 1.52m and 3.05m		COND. WATER µsiemens/cm	
LOGGED BY	F. LIU, M.A.Sc., P.Eng.	TYPE OF DRILL RIG BL 100C SONIC		COND. MUD	
GEOLOGY BY	N/A	TYPE OF LOGGER MOUNT SOPRIS MATRIX		SPECIFIC GRAVITY	
CONTRACTOR	BOART LONGYEAR CANADA	ABANDONMENT GROUTED WITH VW PIEZOMETER		NG RATE	
OPERATOR	S. MADDEN/S. WURZ	BIT SIZE 4 inch	INTERVAL 0.0m–22.9m	LOGGING SPEED	GAMMA 3.6m/min
DATE DRILLED	12–JAN–11	BIT SIZE	INTERVAL		RES N/A

CLIENT	nhc northwest hydraulic consultants
PRODUCED BY	MDH ENGINEERED SOLUTIONS
APPROVED BY	J. ANTUNES, M.Sc., P.Eng.
DRAWN BY	S. RUSSELL, B.Sc.
PROJECT No.	R2488–265010
SCALE	1:100
DATE	21–JAN–11

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488–265010 Spillway Upgrade at Highfield Dam\2. Drafting\1. Boreholes\BH–2488–02.dwg

BOREHOLE BH-R2488-03
NORTHWEST HYDRAULIC CONSULTANTS - HIGHFIELD DAM
2011

5575607.69 N 329407.68 E
NAD 83 ZONE 13
NW36-15-11-W3
72J06



FILL: till, clayey, silty, some sand, trace gravel, dark grayish brown (2.5Y 4/2 moist), oxidized, very stiff to soft, medium plasticity, moist to very moist, calcareous, trace to some organics, trace Fe staining

– above 1.8 m: rootlets

FILL: clay, silty, gray (5Y 5/1 moist), oxidized, soft to very soft, medium to high plasticity, very moist, calcareous, trace organic soils

CLAY SHALE (disturbed): silty, very dark grayish brown (2.5Y 3/2 moist) and light olive brown (2.5Y 5/6 moist), oxidized, stiff to hard, high plasticity, moist, non–calcareous, some silt laminations, Fe staining, trace gypsum, fissured and jointed

– from 5.2 to 6.1 m: with some sand, olive gray (5Y 4/2 moist), trace yellowish bentonite, occasionally nuggety, brecciated

– from 6.1 m to 6.3 m: silt, clayey, gray (5Y 5/1 moist), soft

– at 9.3 m: till inclusion; some gravel, fine to coarse, subangular to subrounded, calcareous

CLAY SHALE: silty, very dark gray (5Y 3/1 moist), unoxidized, stiff to hard, high plasticity, moist, non–calcareous, some silt laminations, trace gypsum, fissured and jointed

– from 12.8 m to 14.2 m: poor core recovery

– from 14.2 m to 14.4 m: light grey with some silt stone fragments, volcanic ash, nonplastic

– from 17.5 m to 18.3 m: trace light grey silt inclusions

– from 19.1 m to 19.5 m: some blocky siltstone layers

– from 20.0 m to 20.4 m: light grey, volcanic ash, non plastic

– below 23.0 m: trace to some siltstone

SAMPLE LEGEND
SPLIT SPOON
CORE
BAGGED

OXIDATION LEGEND
OXIDIZED ZONE
UNOXIDIZED ZONE

MATERIAL RECOVERY LEGEND
RECOVERED MATERIAL
NO MATERIAL

DRAFT

NOTES: 1.
2.

LIMITATION

This drill log is a summary of the conditions estimated by the field personnel at the specific location at the time of drilling. The conditions and properties described above will vary between locations and may vary with time.

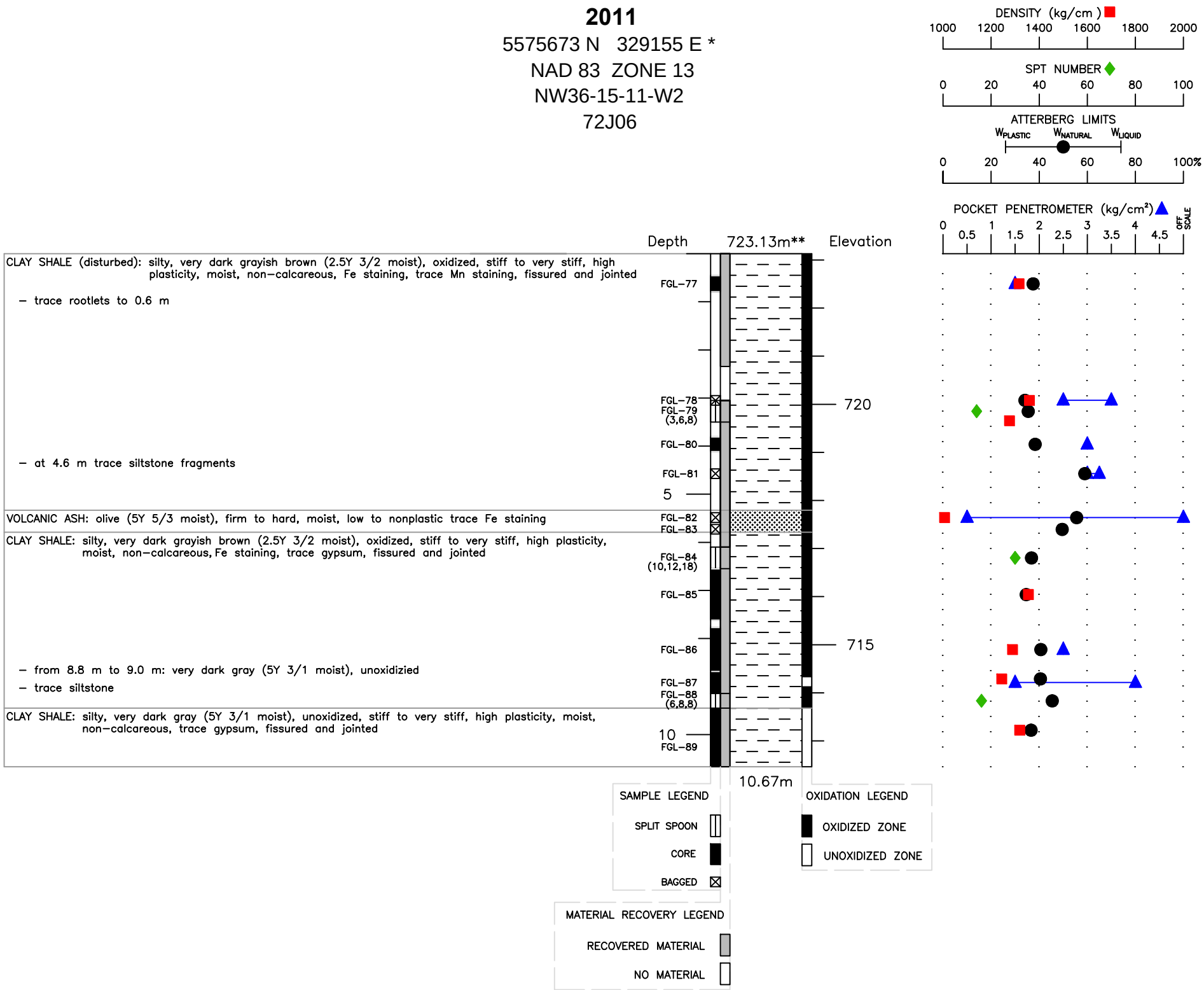
FOR INCLINOMETER AND PIEZOMETER INFORMATION SEE DWG: SI–VW–R2488–03.dwg

SUPERVISOR	F. LIU, M.A.Sc., P.Eng.	CUTTING SAMPLE INTERVAL 1.52m and 3.05m		COND. WATER µsiemens/cm		CLIENT nhc northwest hydraulic consultants	PRODUCED BY MDH ENGINEERED SOLUTIONS
LOGGED BY	F. LIU, M.A.Sc., P.Eng.	TYPE OF DRILL RIG BL 100C SONIC		COND. MUD			
GEOLOGY BY	N/A	TYPE OF LOGGER MOUNT SOPRIS MATRIX		SPECIFIC GRAVITY		APPROVED BY J. ANTUNES, M.Sc., P.Eng.	
CONTRACTOR	BOART LONGYEAR CANADA	ABANDONMENT GROUTED WITH VW PIEZOMETER		NG RATE		DRAWN BY S. RUSSELL, B.Sc.	
OPERATOR	S. MADDEN/S. WURZ	BIT SIZE 4 inch	INTERVAL 0.0m–24.3m	LOGGING SPEED	GAMMA 3.6m/min	PROJECT No. R2488–265010	
DATE DRILLED	14–JAN–11	BIT SIZE 6 inch	INTERVAL 0.0m–24.3m		RES N/A	SCALE 1:100	DATE 21–JAN–11

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488–265010 Spillway Upgrade at Highfield Dam\2. Drafting\1. Boreholes\BH–2488–03.dwg

BOREHOLE BH-R2488-04
NORTHWEST HYDRAULIC CONSULTANTS - HIGHFIELD DAM
2011

5575673 N 329155 E *
NAD 83 ZONE 13
NW36-15-11-W2
72J06



DRAFT

NOTES: * UTM COORDINATES OBTAINED HANDHELD GPS
** ELEVATION ESTIMATED USING EXISTING CONTOURS AND HANDHELD GPS NORTHING AND EASTING

LIMITATION

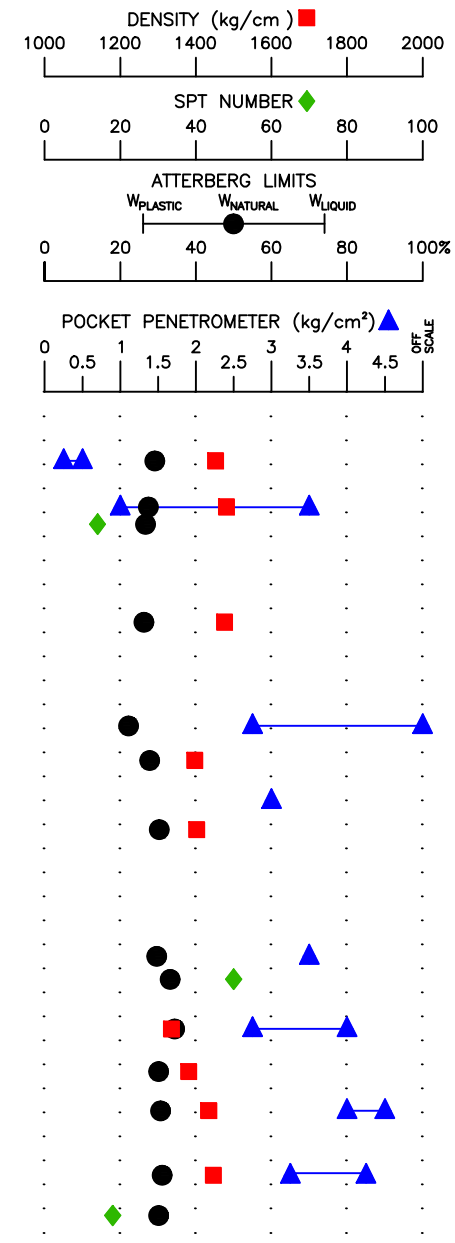
This drill log is a summary of the conditions estimated by the field personnel at the specific location at the time of drilling. The conditions and properties described above will vary between locations and may vary with time.

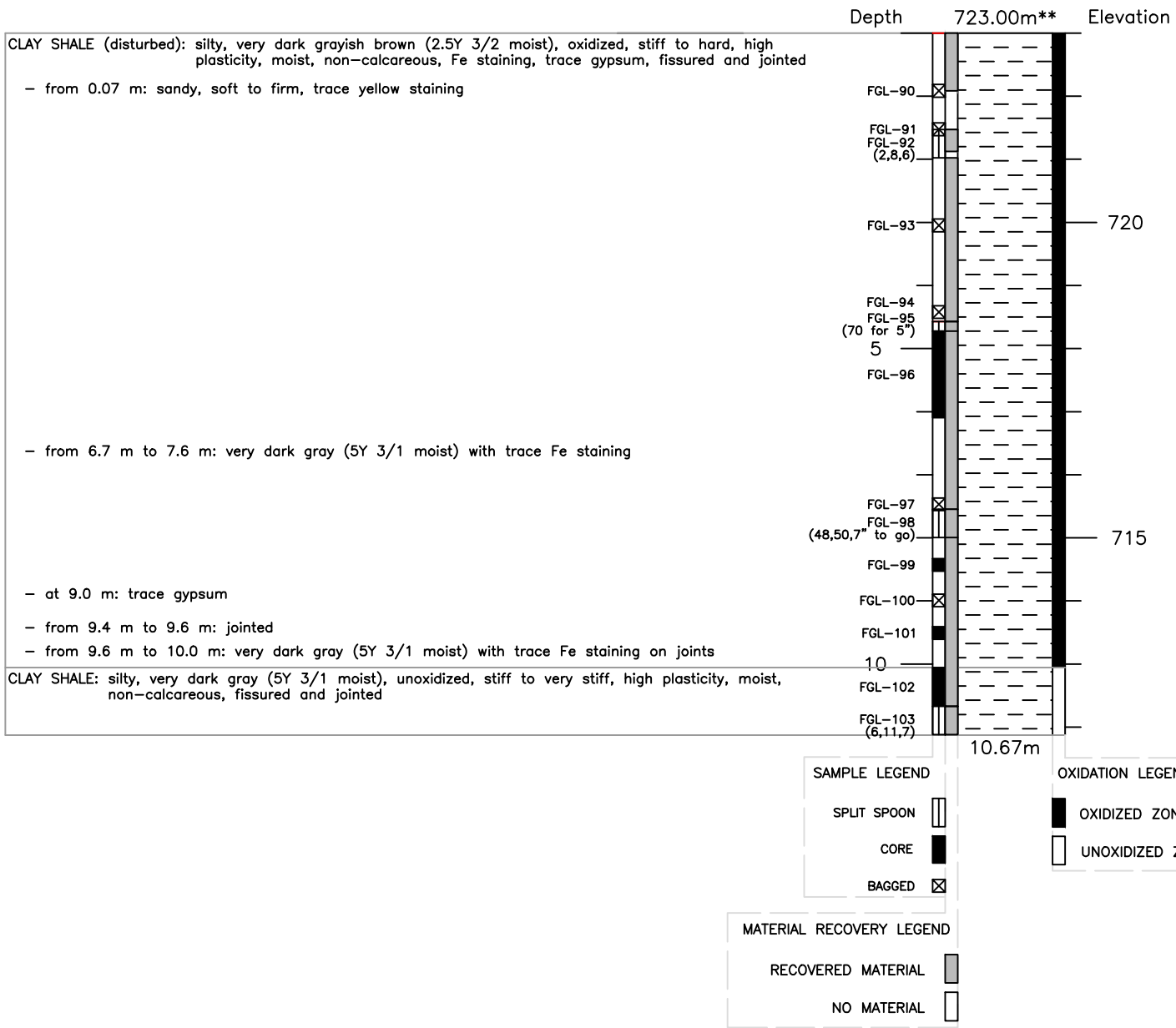
SUPERVISOR	F. LIU, M.A.Sc., P.Eng.	CUTTING SAMPLE INTERVAL 1.52m and 3.05m		COND. WATER µsiemens/cm		CLIENT nhc northwest hydraulic consultants	PRODUCED BY MDH ENGINEERED SOLUTIONS
LOGGED BY	F. LIU, M.A.Sc., P.Eng.	TYPE OF DRILL RIG BL 100C SONIC		COND. MUD			
GEOLOGY BY	N/A	TYPE OF LOGGER MOUNT SOPRIS MATRIX		SPECIFIC GRAVITY		APPROVED BY J. ANTUNES, M.Sc., P.Eng.	
CONTRACTOR	BOART LONGYEAR CANADA	ABANDONMENT GROUTED WITH VW PIEZOMETER		NG RATE		DRAWN BY S. RUSSELL, B.Sc.	
OPERATOR	S. MADDEN/S. WURZ	BIT SIZE 4 inch	INTERVAL 0.0m–10.7m	LOGGING SPEED	GAMMA N/A	PROJECT No. R2488–265010	
DATE DRILLED	17–JAN–11	BIT SIZE 6 inch	INTERVAL N/A		RES N/A	SCALE 1:100	DATE 21–JAN–11

ACADDWG:\\P:\NorthWest Hydraulic Consultants\R2488-265010 Spillway Upgrade at Highfield Dam\2. Drafting\1. Boreholes\BH-2488-04.dwg

BOREHOLE BH-R2488-05
NORTHWEST HYDRAULIC CONSULTANTS - HIGHFIELD DAM
2011

5575428 N 329134 E*
NAD 83 ZONE 13
NW36-15-11-W3
72J06





DRAFT

NOTES: * UTM COORDINATES OBTAINED HANDHELD GPS
** ELEVATION ESTIMATED USING EXISTING CONTOURS AND HANDHELD GPS NORTHING AND EASTING

LIMITATION

This drill log is a summary of the conditions estimated by the field personnel at the specific location at the time of drilling. The conditions and properties described above will vary between locations and may vary with time.

SUPERVISOR	F. LIU, M.A.Sc., P.Eng.	CUTTING SAMPLE INTERVAL	1.52m and 3.05m	COND. WATER	µsiemens/cm	CLIENT nhc northwest hydraulic consultants	PRODUCED BY MDH ENGINEERED SOLUTIONS
LOGGED BY	F. LIU, M.A.Sc., P.Eng.	TYPE OF DRILL RIG	BL 100C SONIC	COND. MUD			
GEOLOGY BY	N/A	TYPE OF LOGGER	MOUNT SOPRIS MATRIX	SPECIFIC GRAVITY		APPROVED BY	J. ANTUNES, M.Sc., P.Eng.
CONTRACTOR	BOART LONGYEAR CANADA	ABANDONMENT	GROUTED WITH VW PIEZOMETER	NG RATE		DRAWN BY	S. RUSSELL, B.Sc.
OPERATOR	S. MADDEN/S. WURZ	BIT SIZE 4 inch	INTERVAL 0.0m–21.3m	LOGGING SPEED	GAMMA N/A	PROJECT No.	R2488–265010
DATE DRILLED	17–JAN–11	BIT SIZE 6 inch	INTERVAL 21.3m–21.6m		RES N/A	SCALE 1:100	DATE 21–JAN–11

Appendix D

PFRA Borehole Logs

LEGEND

- TOPSOIL – or no sample
- PEAT – Peat
- COBBLE – Cobbles or Boulders
- GW – Well–graded gravel
- GP – Poorly graded gravel
- GM – Silty gravel
- GC – Clayey gravel
- SW – Well–graded sand
- SP – Poorly graded sand
- SM – Silty sand
- SC – Clayey sand
- ML – Silt, low plasticity
- CL – Clay, low plasticity
- OL – Organic silt or clay, low plasticity
- CI – Clay, medium plasticity
- MH – Silt, high plasticity
- CH – Clay, high plasticity
- OH – Organic silt or clay, high plasticity
- TILL (CL)
- TILL (CI)
- TILL (CH)
- CLAY SHALE or SHALE
- SILTSTONE
- SANDSTONE
- CLAY SHALE & SANDSTONE
- LIMESTONE

* Unless otherwise indicated :

- * Gradation of granular soil assumed to be fine to coarse
- * All overburden is alluvial or assumed alluvial
- * All overburden is brown or assumed brown
- * All bedrock is grey or assumed grey
- * All soils are inorganic or assumed inorganic

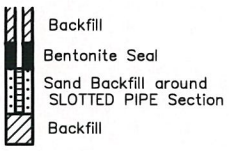
▼ – Last recorded water level

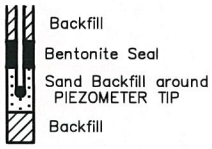
6 Jun 92 – 2.2m– Date – depth of water level from ground surface

wl – Water loss

(fn) – Field notes

PIEZOMETER INSTALLATIONS –





– Water content – %

– Mass density of soil (wet density) – t/m^3

– Plastic limit (W_p) and liquid (W_L) at ends of line shown on water content scale

– Divisions between low, medium, and high plasticity for clays only occur at liquid limits of 30 and 50

■ – Unconfined compressive strength – kPa

– Compressive strength using pocket penetrometers (PP) in kPa

– All PP > 1000 shown to right of plot

15–10–0.09–5 or 5–0.18–7

- – (When three numbers shown, no cobbles present)
- P200 – % fines passing No. 200 sieve (0.075 mm)
- D10 – 10% size in mm
- % Gravel – 75 mm to No. 4 sieve (4.75 mm)
- % Cobbles – 200 mm to 75 mm
- Boulders – > 200 mm

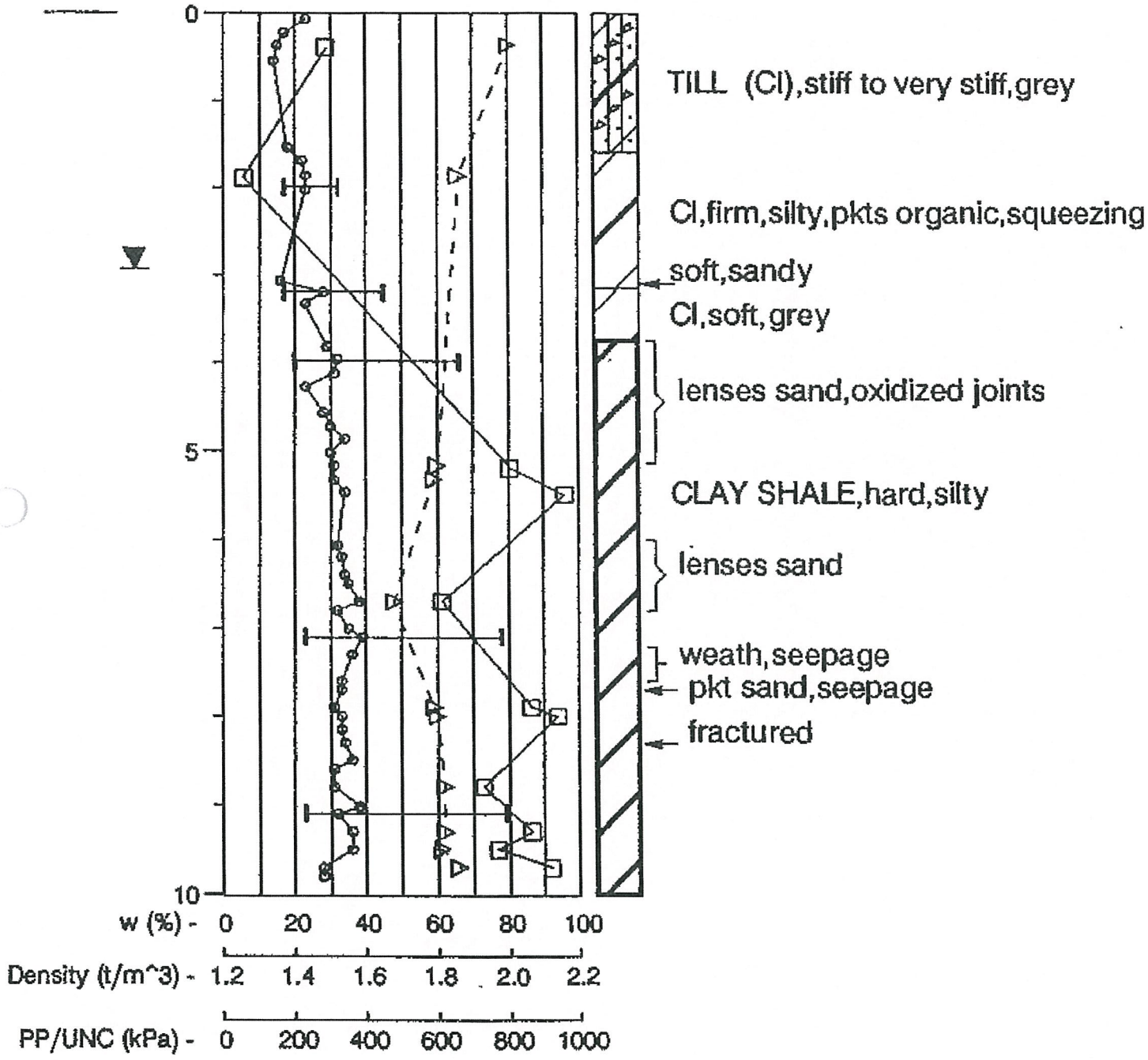
7.8	pH	Hydrogen ion concentration
0.06	SO_4	Water soluble sulphates – %
SPT=55		Standard Penetration Test – blows per 0.3 metres

"C" revision – Feb. 1994
"B" revision – April 1993
"A" revision – Sept. 1992

Designed	Approved	Agriculture and Agri–Food Canada / Agriculture et Agroalimentaire Canada	GEOTECHNICAL DIVISION
Drawn	Position Title	PFRA ARAP	LEGEND
Checked	Date	Prairie Farm Rehabilitation Administration / Administration du retablissement agricole des Prairies	Scale: As Shown; Date: Sept./92; PFRA No. 103807C

C24

EL. 720.1 m

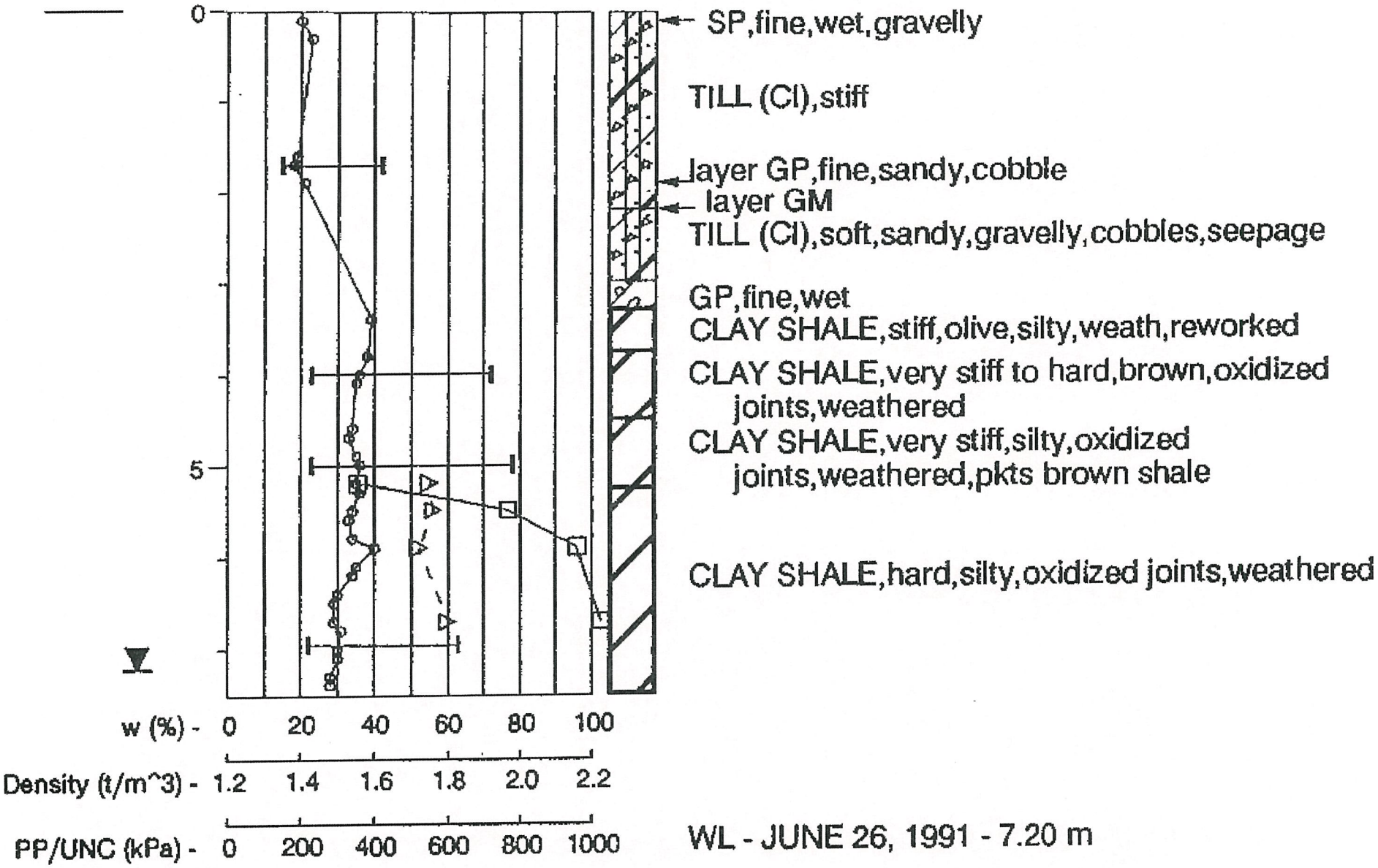


Drilled - June 26, 1991 - 10.0 m

WL - 26 JUNE,1991 - 2.95 m

C25

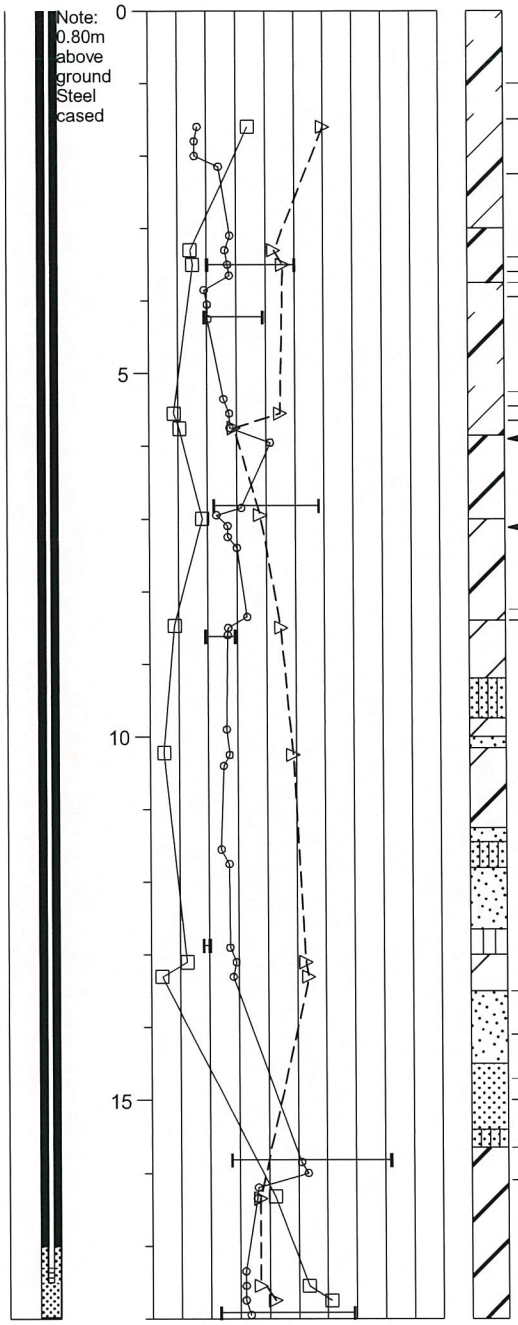
EL. 718.2 m



Drilled - June 26, 1991 - 7.5 m

C26

EL 724.869 m



Drilled - August 21, 2002 - 18 m

CI, very stiff, Till, (FILL)
(fn) hard, lenses & seams oxides
lenses sand

CH, stiff, lenses sand, (FILL)
pkts organic
lenses sand, seams CH, pkts & seams organic

CI, very stiff, Till, (FILL)

firm, pkts organic
firm, numerous pkts & seams organic
layer organic
CH, stiff, pkts organic

layer SM

CI, stiff, pkts & seams organic

firm, grey
CL, firm, layers & pkts SM, (fn) seepage

(fn) SM, fine, wet, seepage
CL, soft, lenses & pkts sand, (fn) seepage
SPSM, fine, wet, lumps clay, (fn) seepage
CI, firm, grey, layers & seams sand, (fn) seepage

SPSM, fine, wet, grey, (fn) seepage
SM, fine, wet, grey, seams organic, (fn) seepage
SPSM, fine, wet, grey, (fn) seepage

ML, very soft, grey, pkts sand, seams organic
CI, soft, grey, pkts sand
(fn) numerous layers CI
SPSM, fine, wet, (fn) seepage
seams & lenses clay
SP, fine, wet, (fn) seepage
(fn) SM, coarse, wet, frags clay shale
small frags Bentonite

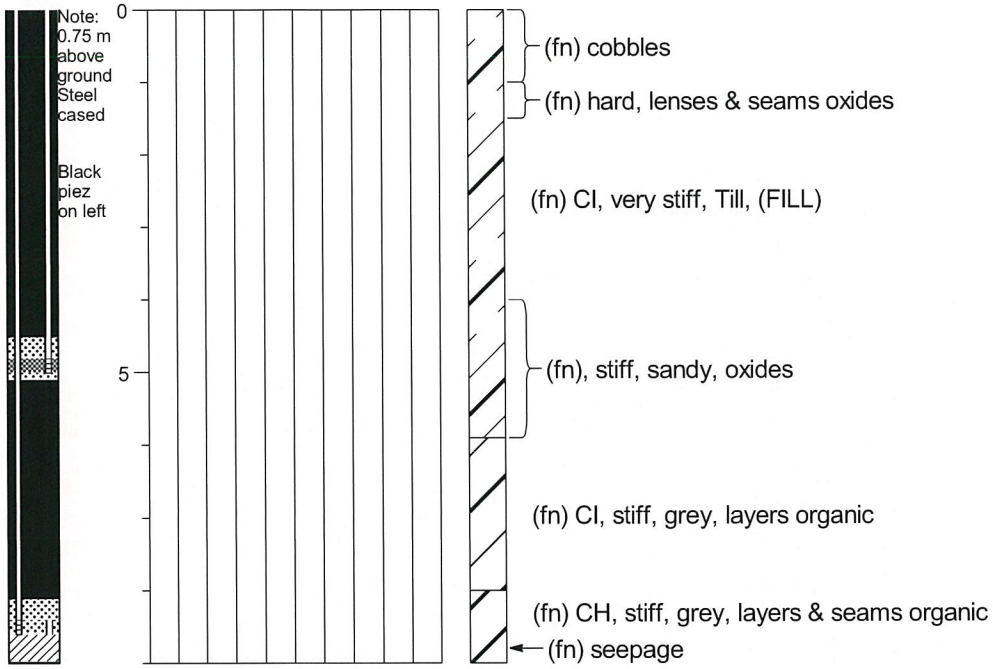
CLAY SHALE, hard, pkts ML

WL - October 24, 2002 - 5.54m
WL - August 25, 2003 - 5.79m
WL - September 13, 2004 - 5.45m
WL - June 18, 2009 - 5.33m
GPS: UTM 12, 329582.47 E, 5575629.03 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, E of West Outlet Structure, Top of Dam**				**C26**
	Drawn/Checked: **WB/EH/WR/VK**	**2010/03/09**	**4547:928:7H4**	**356555**	**1 of 1**

C27

EL 724.833 m



w (%) - 0 20 40 60 80 100

Density (t/m^3) - 1.4 1.6 1.8 2.0 2.2 2.4

UNC (kPa) - 0 200 400 600 800 1000

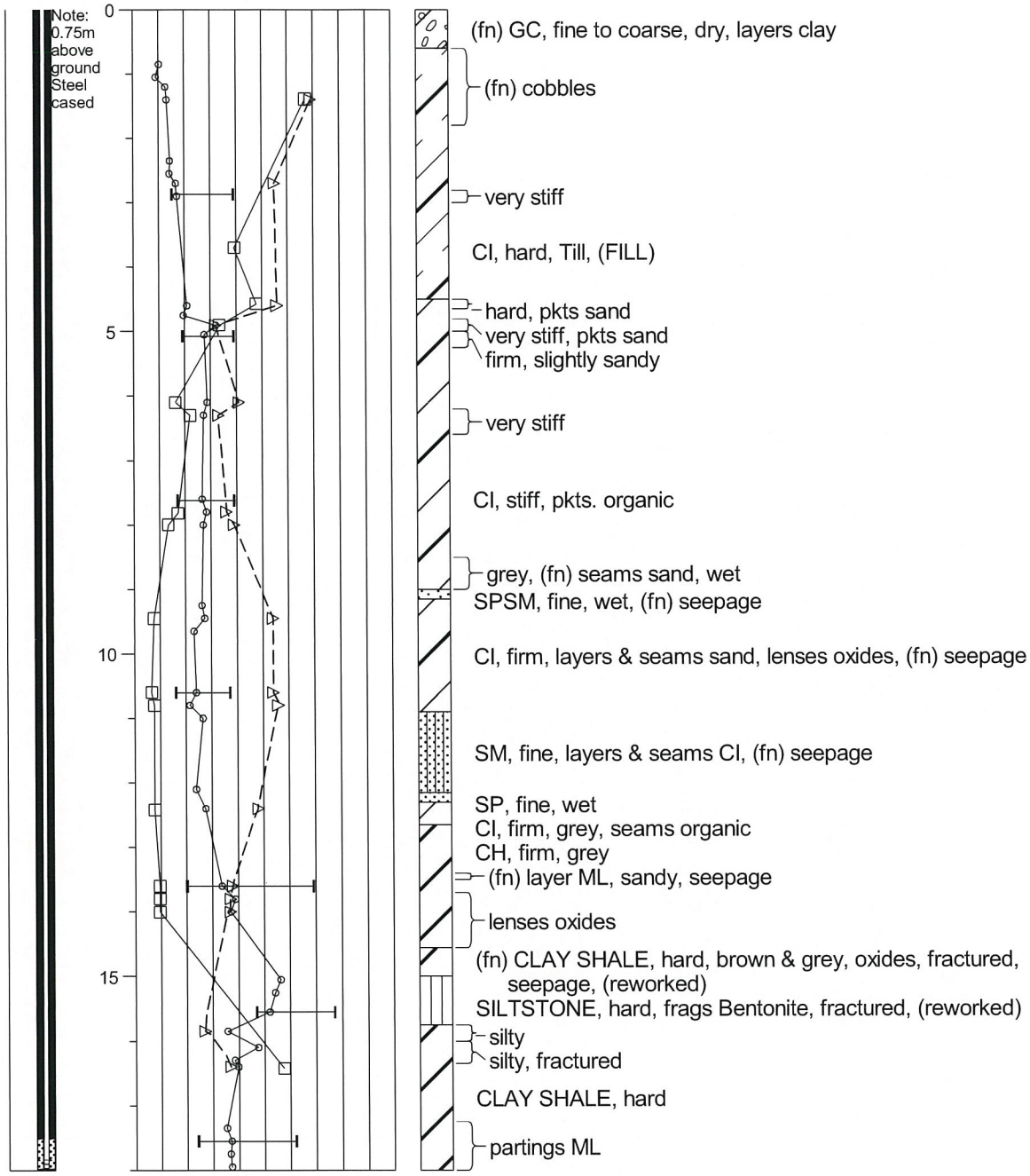
Drilled - August 21, 2002 - 9 m

WL - October 24, 2002 - B - 6.61m, W - 5.06m
WL - August 25, 2003 - B - 7.28m, W - 4.25m
WL - September 13, 2004 - B - 6.67m, W - 4.61m
WL - September 17, 2007 - B - 6.61m, W - 3.90m
WL - June 18, 2009 - B - 6.61m, W - 5.06m
GPS: UTM 12, 329583.42 E, 5575629.21 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, E of West Outlet Structure, Top of Dam**				**C27**
	Drawn/Checked: **WB/EH/WR/VK**	**2010/03/09**	**4547:928:7H4**	**356556**	**1 of 1**

C28

EL 724.43 m



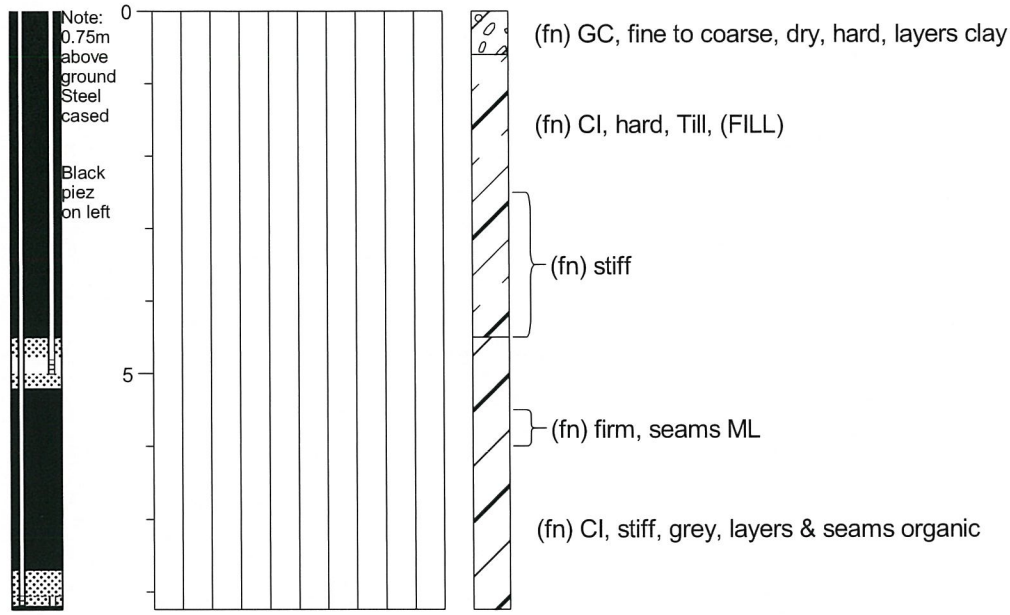
Drilled - August 22, 2002 - 18 m

WL - October 24, 2002 - 5.96m
WL - August 25, 2003 - 6.22m
WL - September 13, 2004 - 5.88m
WL - September 17, 2007 - 5.56m
WL - June 18, 2009 - 5.55m
GPS: UTM 12, 329937.76 E, 5575699.77 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, W of East Outlet Structure, Top of Dam**				**C28**
	Drawn/Checked: **WB/EH/WR/VK**	**2010/03/09**	**4547:928:7H4**	**356557**	**1 of 1**

C29

EL 725.192 m



w (%) - 0 20 40 60 80 100

Density (t/m^3) - 1.4 1.6 1.8 2.0 2.2 2.4

UNC (kPa) - 0 200 400 600 800 1000

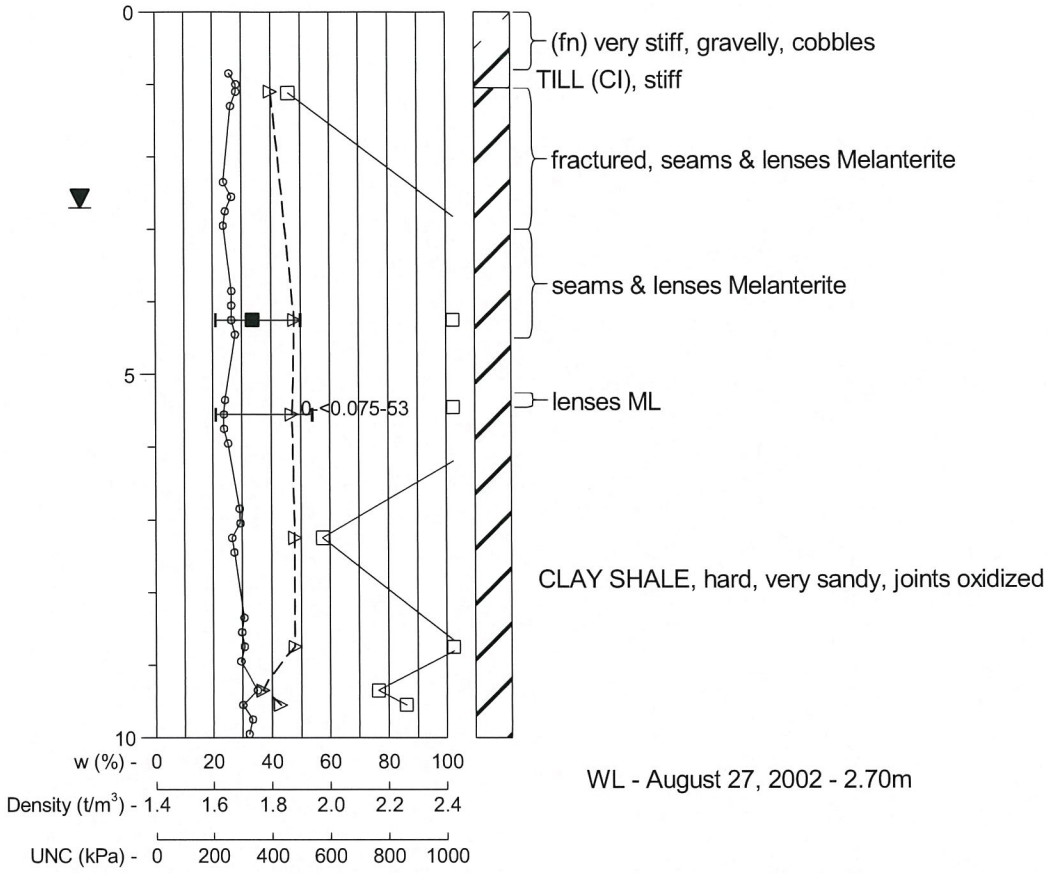
Drilled - August 23, 2002 - 8.25 m

WL - October 24, 2002 - B - 5.48m, W - 5.04m
WL - August 25, 2003 - B - 6.50m, W - 3.97m
WL - September 13, 2004 - B - 5.26m, W - 4.34m
WL - September 17, 2007 - B - 5.13m, W - 3.71m
WL - June 18, 2009 - B - 5.33m, W - 4.61m
GPS: UTM 12, 329938.67 E, 5575700.01 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, W of East Outlet Structure, Top of Dam**				**C29**
	Drawn/Checked: **WB/EH/WR/VK**	**2010/03/09**	**4547:928:7H4**	**356558**	**1 of 1**

C30

EL 718.977 m



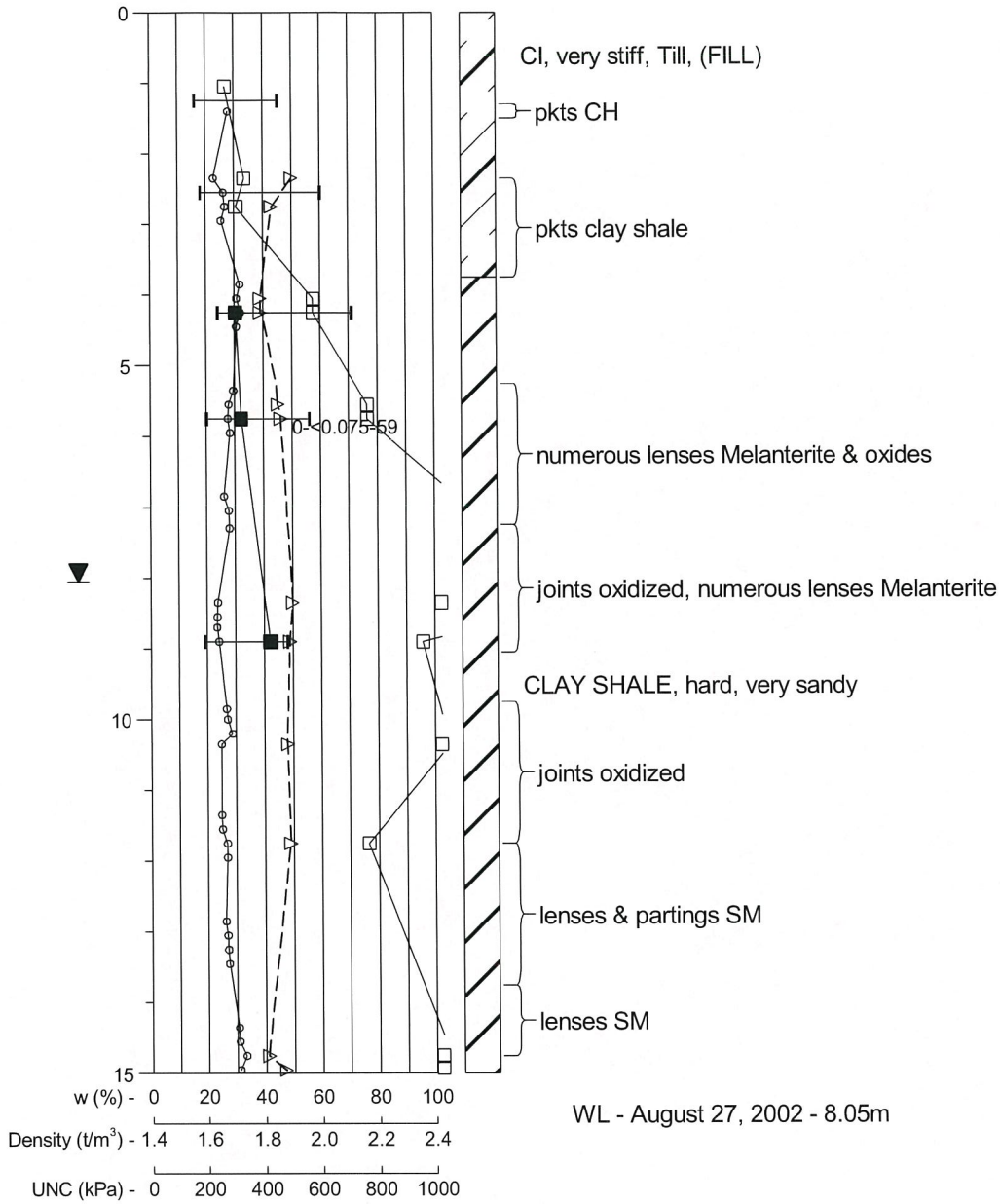
WL - August 27, 2002 - 2.70m

Drilled - August 27, 2002 - 10 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, E of East Outlet Structure**				**C30**
	Drawn/Checked: **WB/EH/WR/VK**	**2010/03/09**	**4547:928:7H4**	**356559**	**1 of 1**

C31

EL 724.695 m



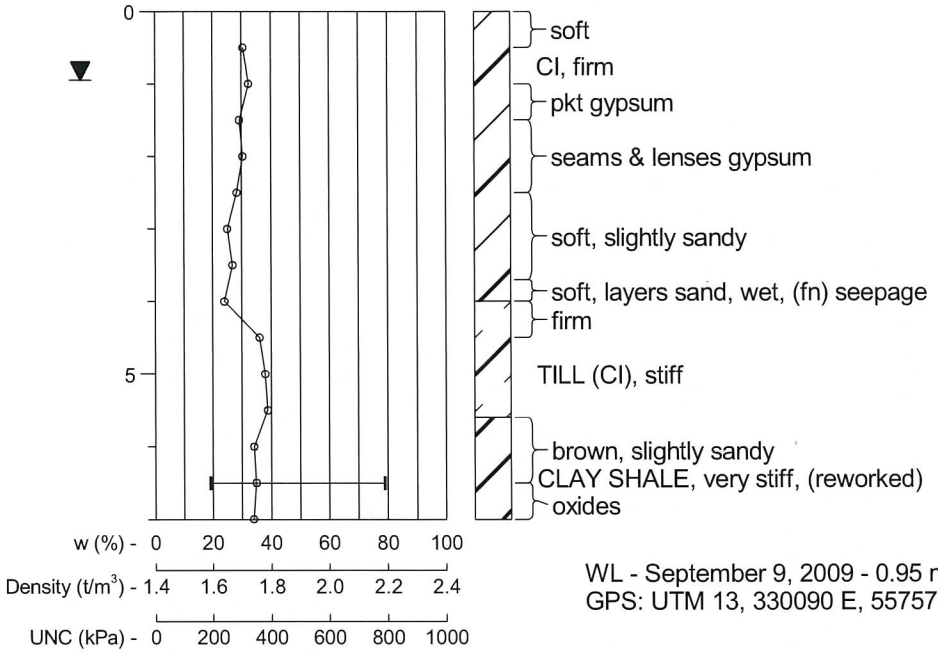
WL - August 27, 2002 - 8.05m

Drilled - August 26, 2002 - 15 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, d/s Dam C/L**				**C31**
	Drawn/Checked: **WB/EH/WR/VK**	**2010/03/09**	**4547:928:7H4**	**356560**	**1 of 1**

BH32

EL 717.453 m



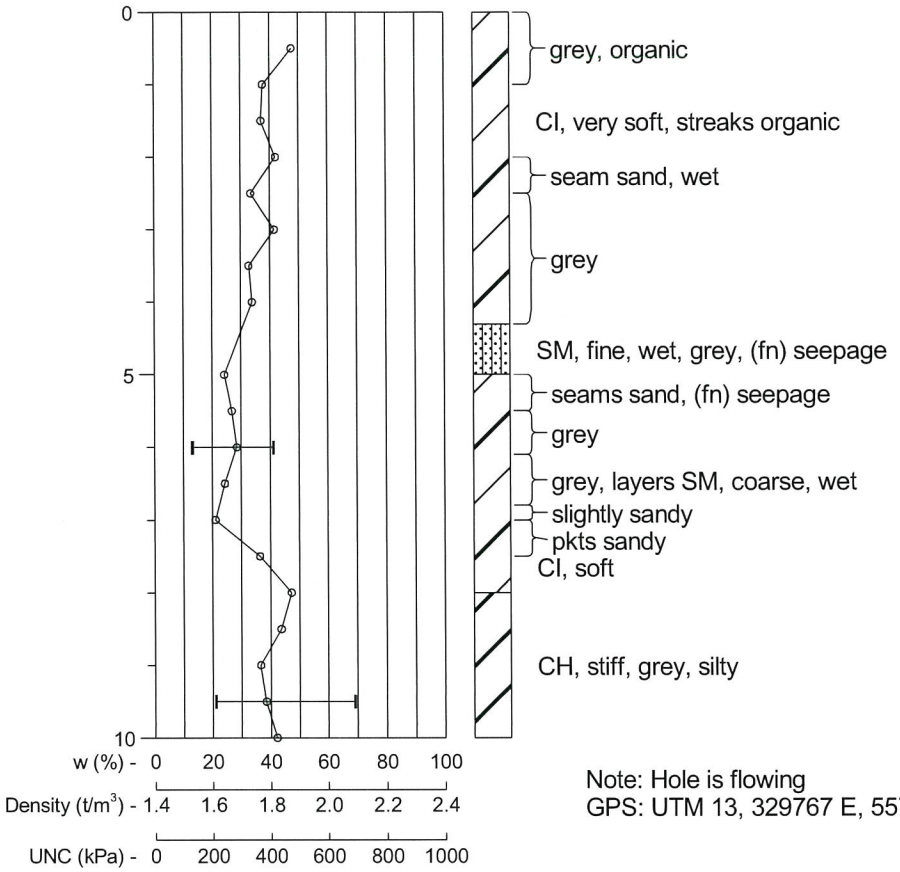
WL - September 9, 2009 - 0.95 m
GPS: UTM 13, 330090 E, 5575764 N

Drilled - September 9, 2009 - 7 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** NE 36-15-11 W3, Toe of Dam				**BH32**
	Drawn/Checked: **RP/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358911**	**1 of 1**

BH33

EL 716.907 m



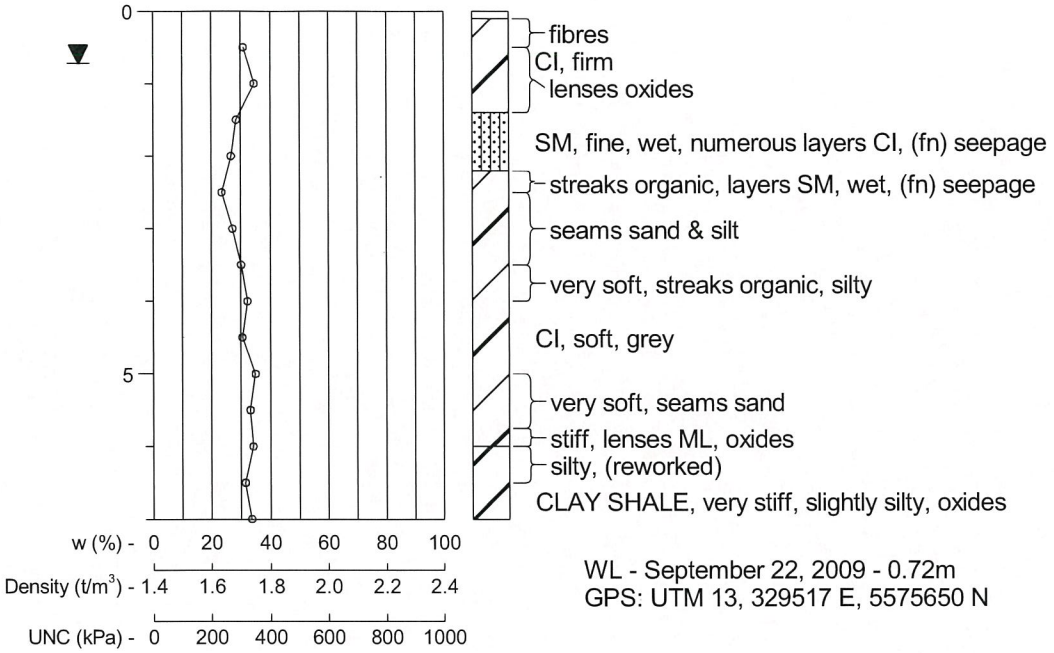
Note: Hole is flowing
GPS: UTM 13, 329767 E, 5575705 N

Drilled - September 9, 2009 - 10 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Toe of Dam**				**BH33**
	Drawn/Checked: **RP/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358912**	**1 of 1**

BH34

EL 716.878 m



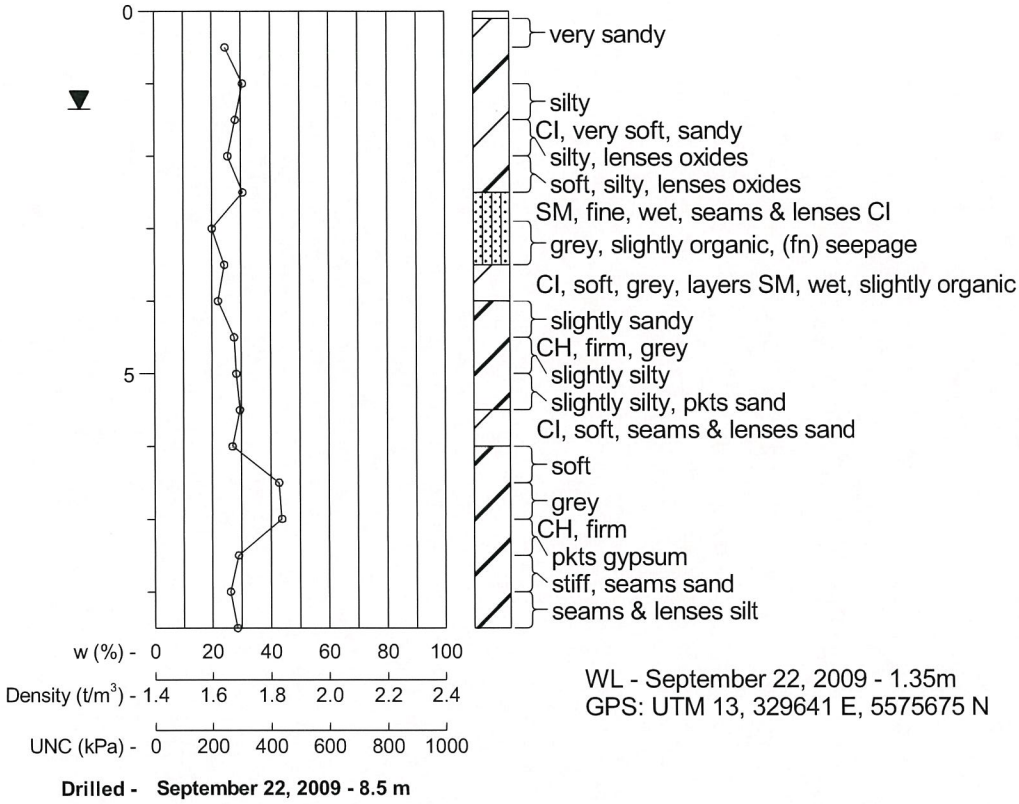
WL - September 22, 2009 - 0.72m
GPS: UTM 13, 329517 E, 5575650 N

Drilled - September 22, 2009 - 7 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Toe of Dam**				**BH34**
	Drawn/Checked: **NB/GC/VK**	**2010/02/15**	**4547:928:7H4**	**358913**	**1 of 1**

BH35

EL 717.548 m



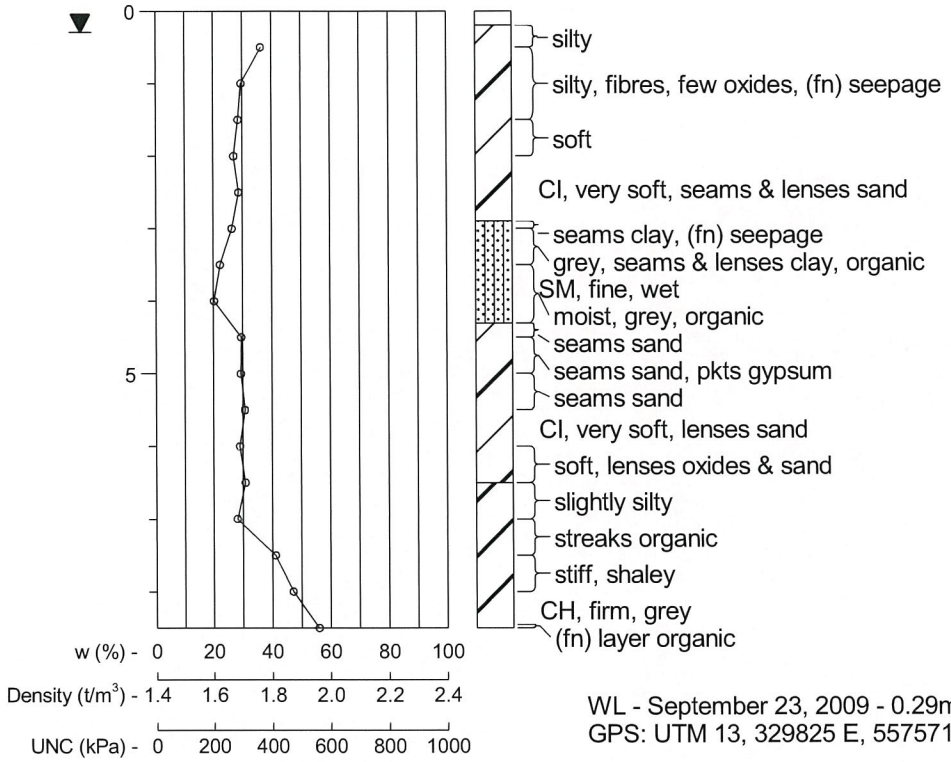
Drilled - September 22, 2009 - 8.5 m

WL - September 22, 2009 - 1.35m
GPS: UTM 13, 329641 E, 5575675 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Toe of Dam**				**BH35**
	Drawn/Checked: **NB/GC/VK**	**2010/02/15**	**4547:928:7H4**	**358914**	**1 of 1**

BH36

EL 717.797 m



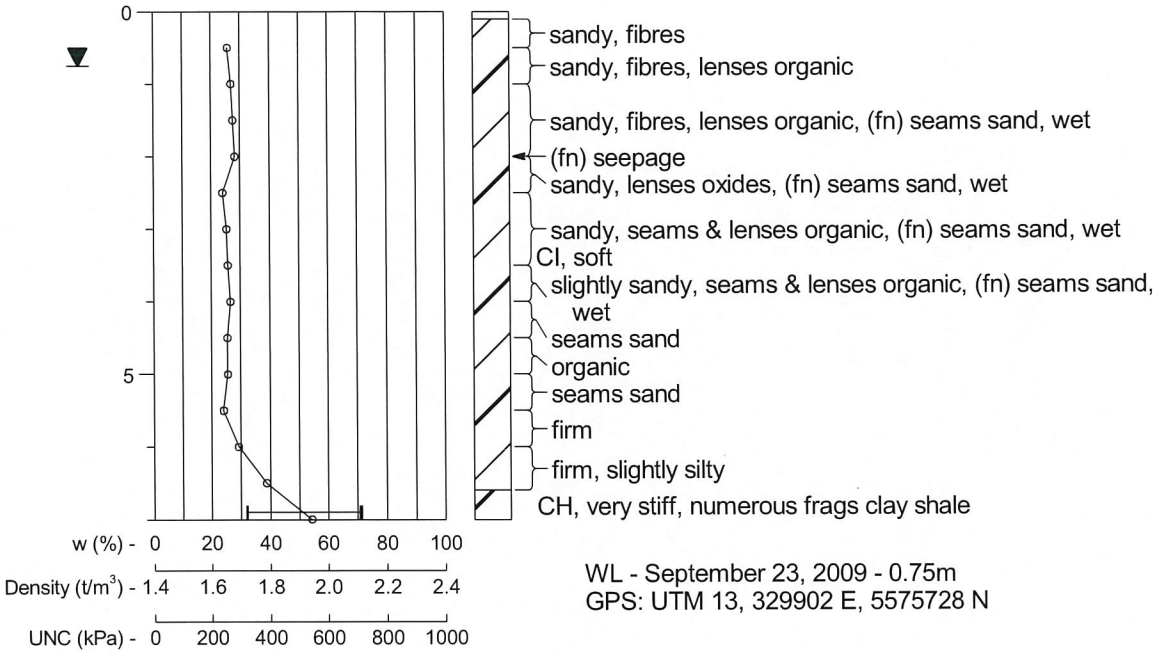
Drilled - September 22, 2009 - 8.5 m

WL - September 23, 2009 - 0.29m
GPS: UTM 13, 329825 E, 5575710 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Toe of Dam**				**BH36**
	Drawn/Checked: **NB/GC/VK**	**2010/02/15**	**4547:928:7H4**	**358915**	**1 of 1**

BH37

EL 717.776 m

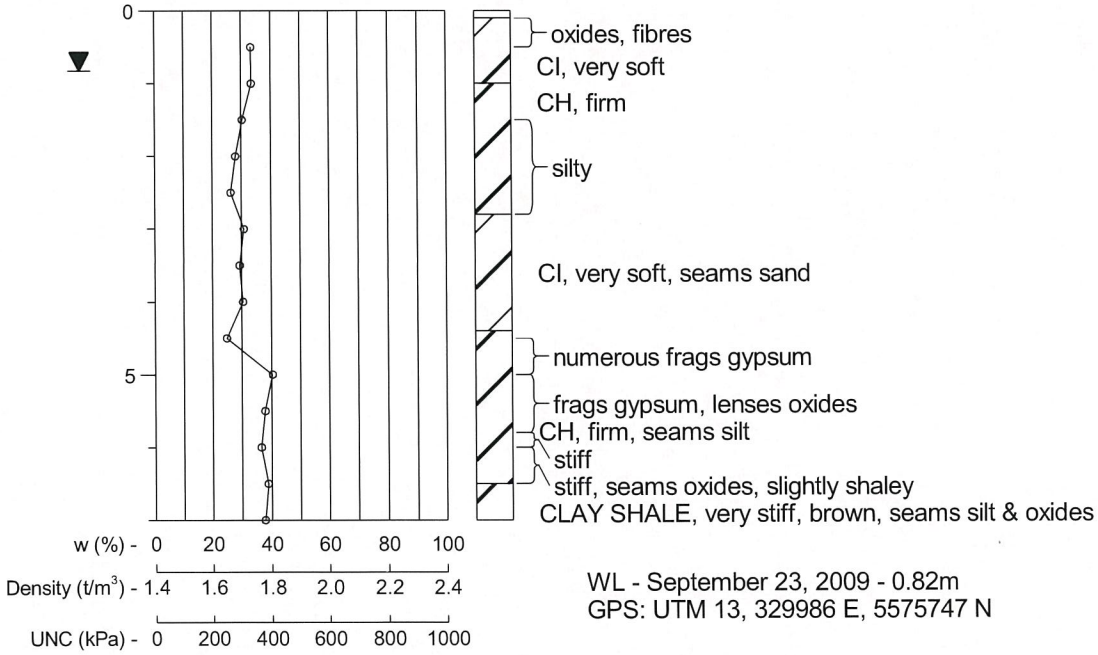


Drilled - September 22, 2009 - 7 m

WL - September 23, 2009 - 0.75m
GPS: UTM 13, 329902 E, 5575728 N

BH38

EL 717.667 m



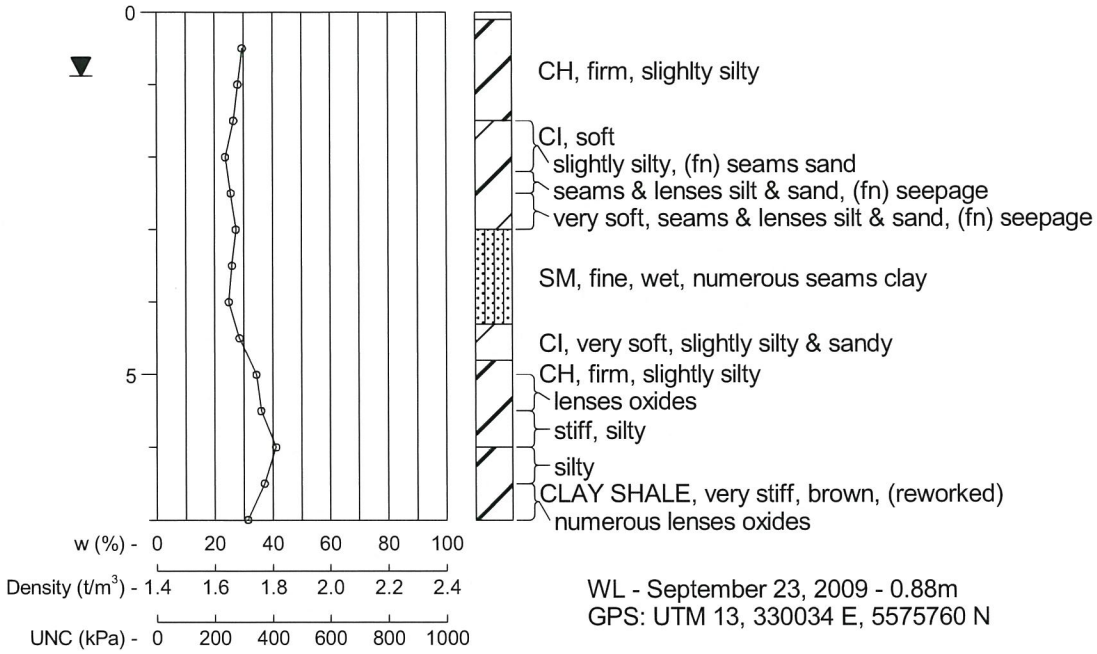
Drilled - September 23, 2009 - 7 m

WL - September 23, 2009 - 0.82m
GPS: UTM 13, 329986 E, 5575747 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Toe of Dam, d/s C/L**			
	Drawn/Checked: **NB/KB/VK**	**2010/03/09**	**4547:928:7H4**	**358916**

BH39

EL 717.439 m



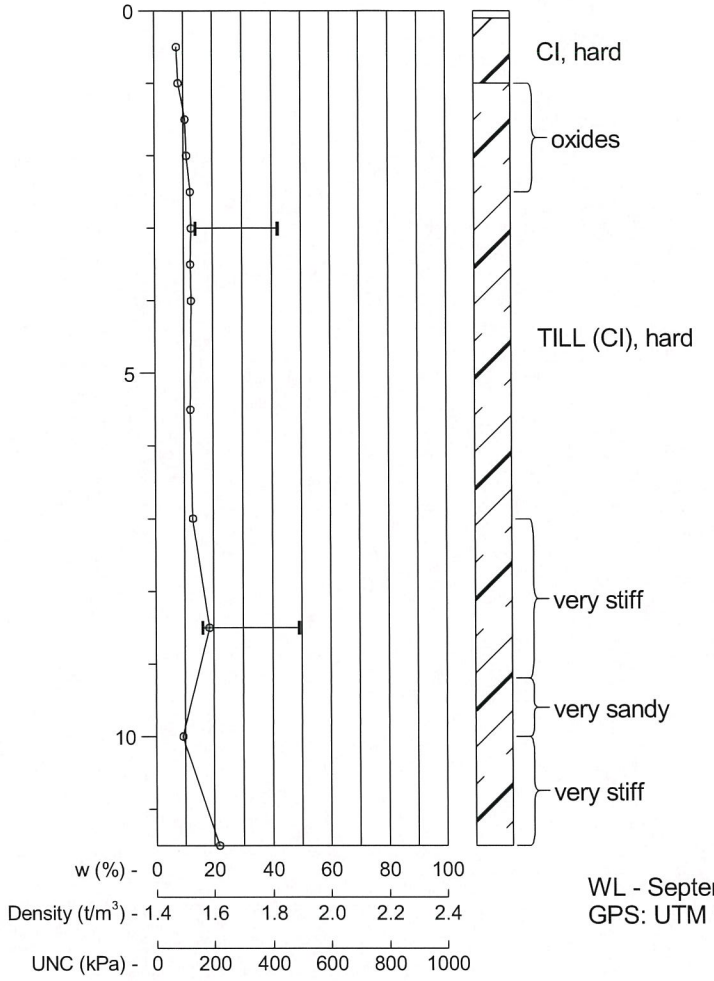
Drilled - September 23, 2009 - 7 m

WL - September 23, 2009 - 0.88m
GPS: UTM 13, 330034 E, 5575760 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Toe of Dam, d/s dam C/L**				**BH39**
	Drawn/Checked: **NB/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358917**	**1 of 1**

BH40

EL 732.545 m



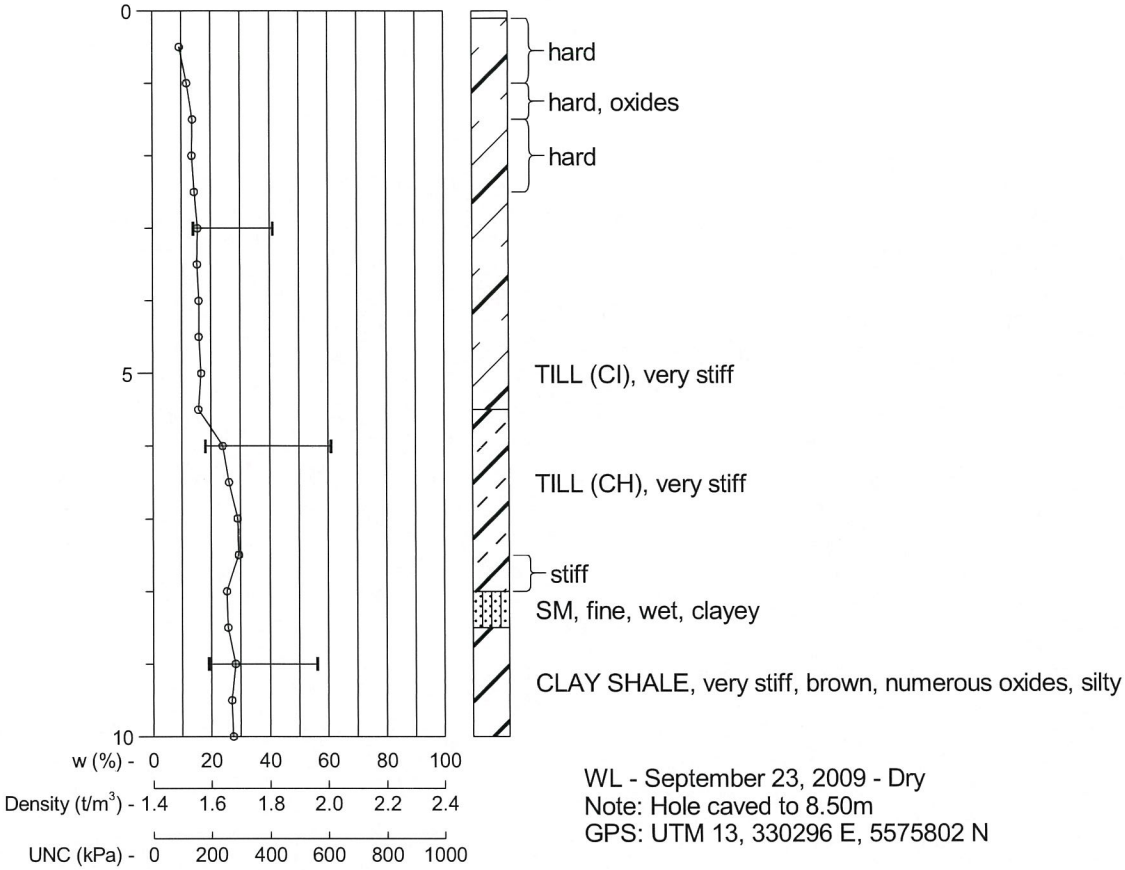
WL - September 23, 2009 - Dry
GPS: UTM 13, 330280 E, 5575842 N

Drilled - September 23, 2009 - 11.5 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** NE 36-15-11 W3, Spillway C/L				**BH40**
	Drawn/Checked: NB/KB/VK	2010/03/22	4547:928:7H4	358918	1 of 1

BH41

EL 728.394 m



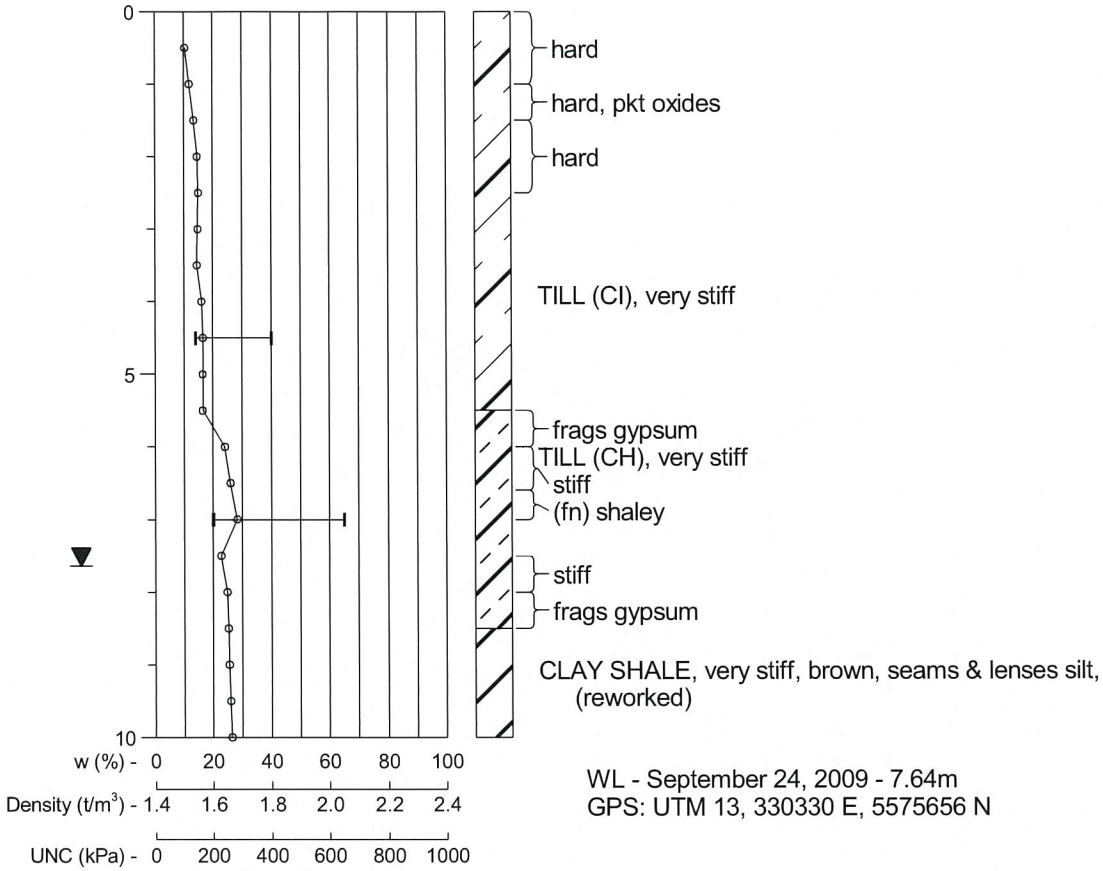
Drilled - September 23, 2009 - 10 m

WL - September 23, 2009 - Dry
Note: Hole caved to 8.50m
GPS: UTM 13, 330296 E, 5575802 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** NE 36-15-11 W3, Spillway C/L				**BH41**
	Drawn/Checked: NB/KB/VK	2010/03/22	4547:928:7H4	358919	1 of 1

BH42

EL 728.875 m



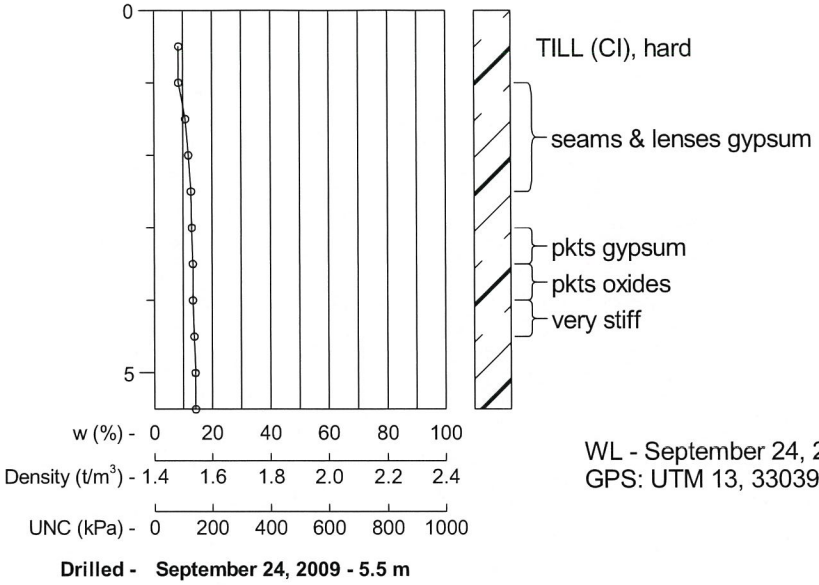
WL - September 24, 2009 - 7.64m
GPS: UTM 13, 330330 E, 5575656 N

Drilled - September 24, 2009 - 10 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** NE 36-15-11 W3, Spillway C/L				**BH42**
	Drawn/Checked: NB/KB/VK	2010/03/22	4547:928:7H4	358920	1 of 1

BH43

EL 730.910 m

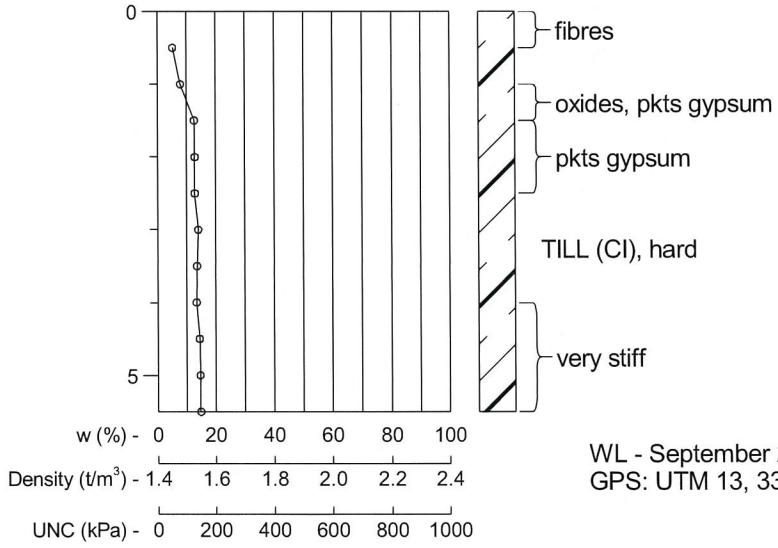


Drilled - September 24, 2009 - 5.5 m

WL - September 24, 2009 - Dry
GPS: UTM 13, 330399 E, 5575644 N

BH44

EL 729.021 m



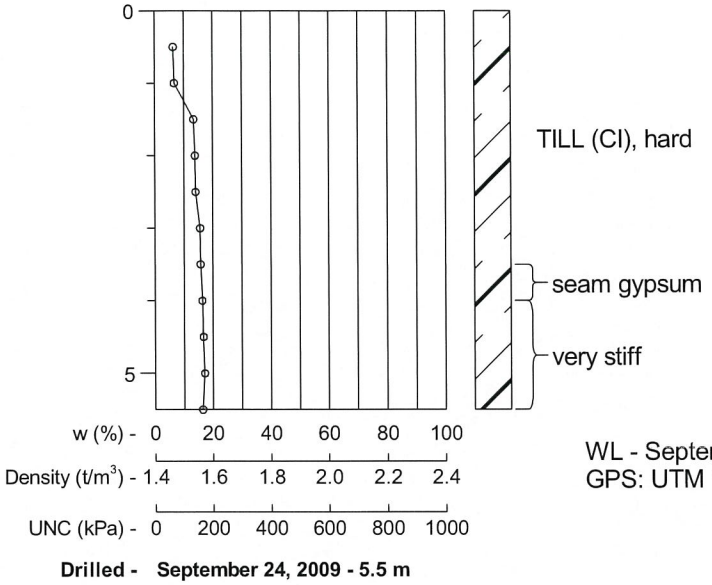
Drilled - September 24, 2009 - 5.5 m

WL - September 24, 2009 - Dry
GPS: UTM 13, 330426 E, 5575668 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Borrow Area, East Abutment**			
	Drawn/Checked: **NB/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358921**

BH45

EL 730.577 m

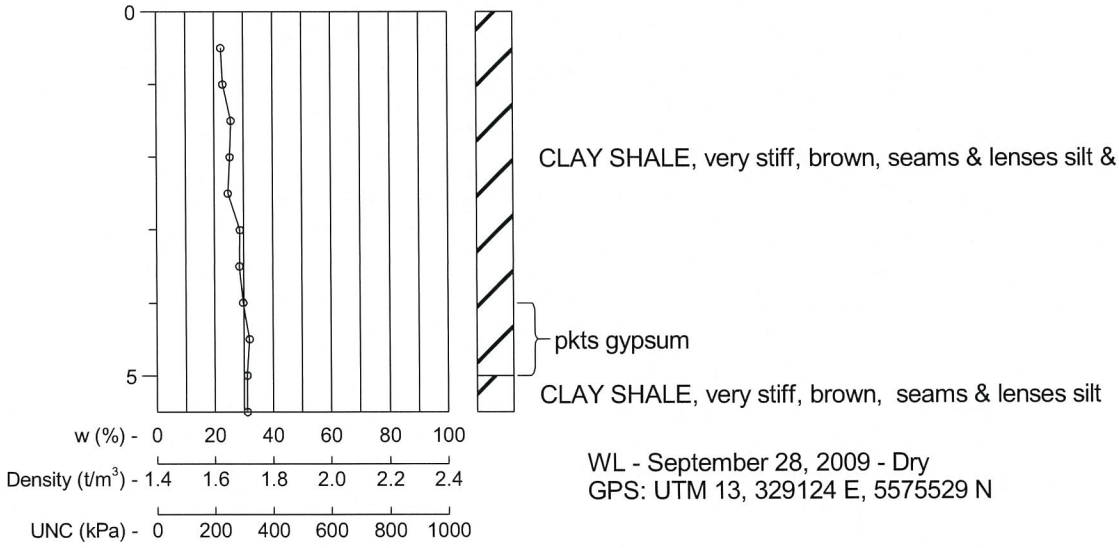


Drilled - September 24, 2009 - 5.5 m

WL - September 24, 2009 - Dry
GPS: UTM 13, 330387 E, 5575709 N

BH46

EL m



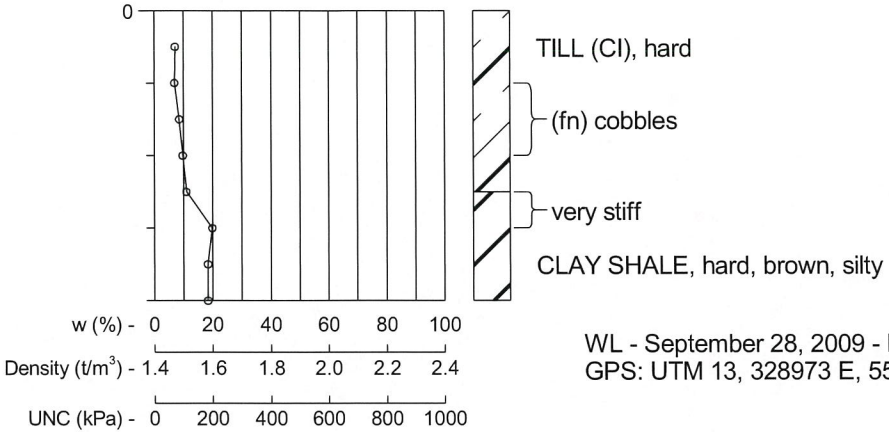
Drilled - September 28, 2009 - 5.5 m

WL - September 28, 2009 - Dry
GPS: UTM 13, 329124 E, 5575529 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Borrow Area, East Abutment**			
	Drawn/Checked: **NB/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358922**

BH47

EL 731.716 m

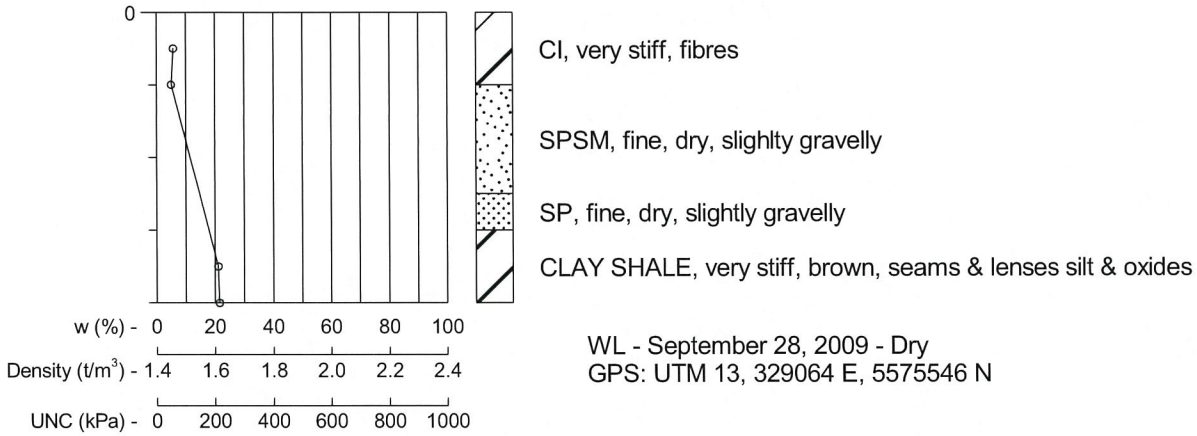


WL - September 28, 2009 - Dry
GPS: UTM 13, 328973 E, 5575558 N

Drilled - September 28, 2009 - 4 m

BH48

EL 726.394 m

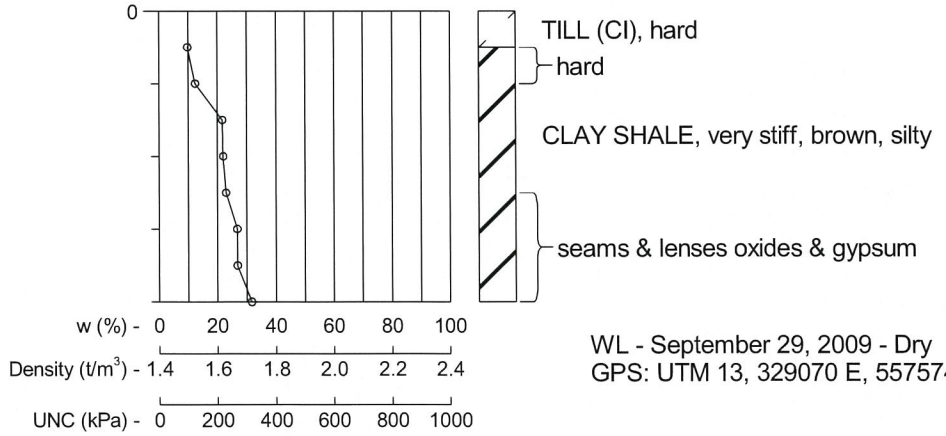


WL - September 28, 2009 - Dry
GPS: UTM 13, 329064 E, 5575546 N

Drilled - September 28, 2009 - 4 m

BH49

EL 725.765 m



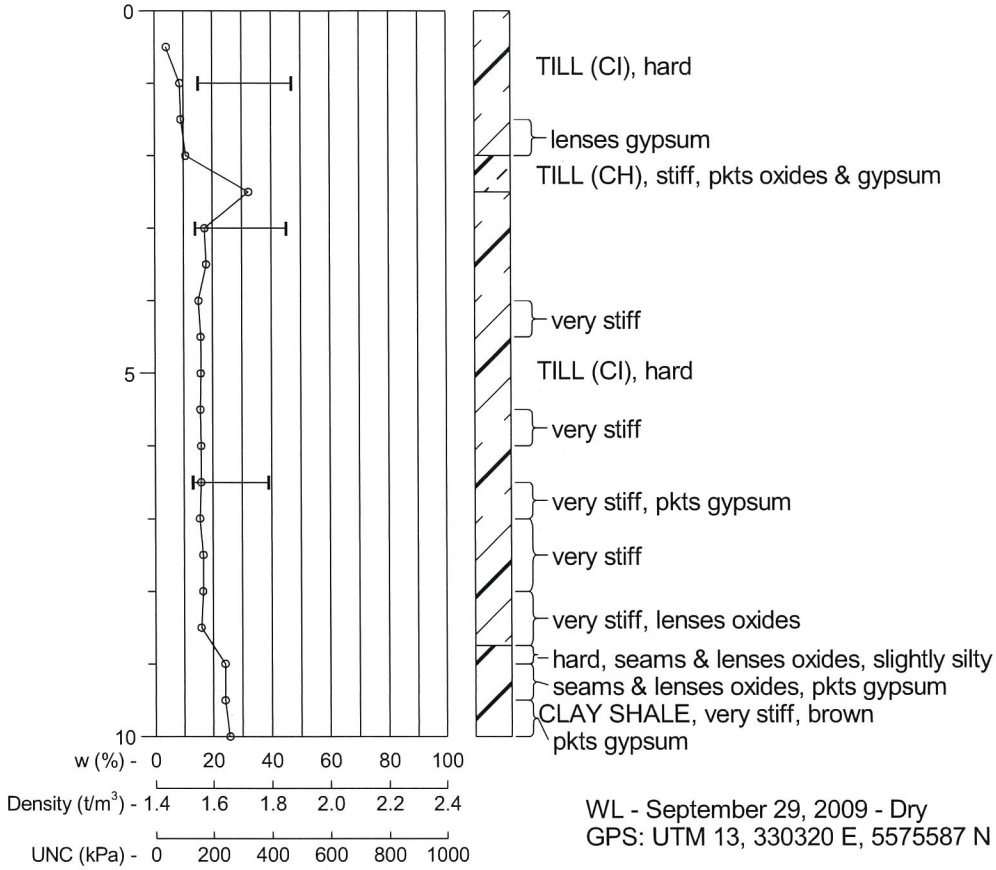
WL - September 29, 2009 - Dry
GPS: UTM 13, 329070 E, 5575746 N

Drilled - September 29, 2009 - 4 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Borrow Area, West Abutment**			
	Drawn/Checked: **NB/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358923**

C50

EL 730.985 m



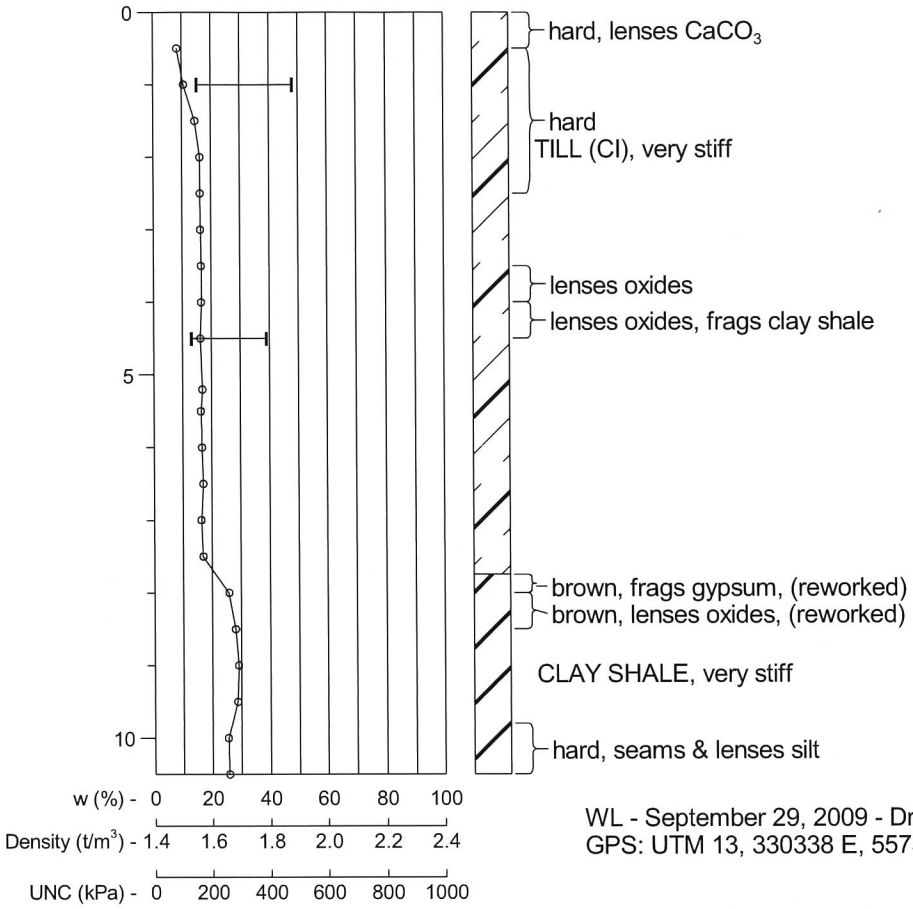
WL - September 29, 2009 - Dry
GPS: UTM 13, 330320 E, 5575587 N

Drilled - September 29, 2009 - 10 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Spillway Channel C/L, East Abutment**				**C50**
	Drawn/Checked: **NB/KB/VK**	**2010/03/22**	**4547:928:7H4**	**358924**	**1 of 1**

C51

EL 728.785 m



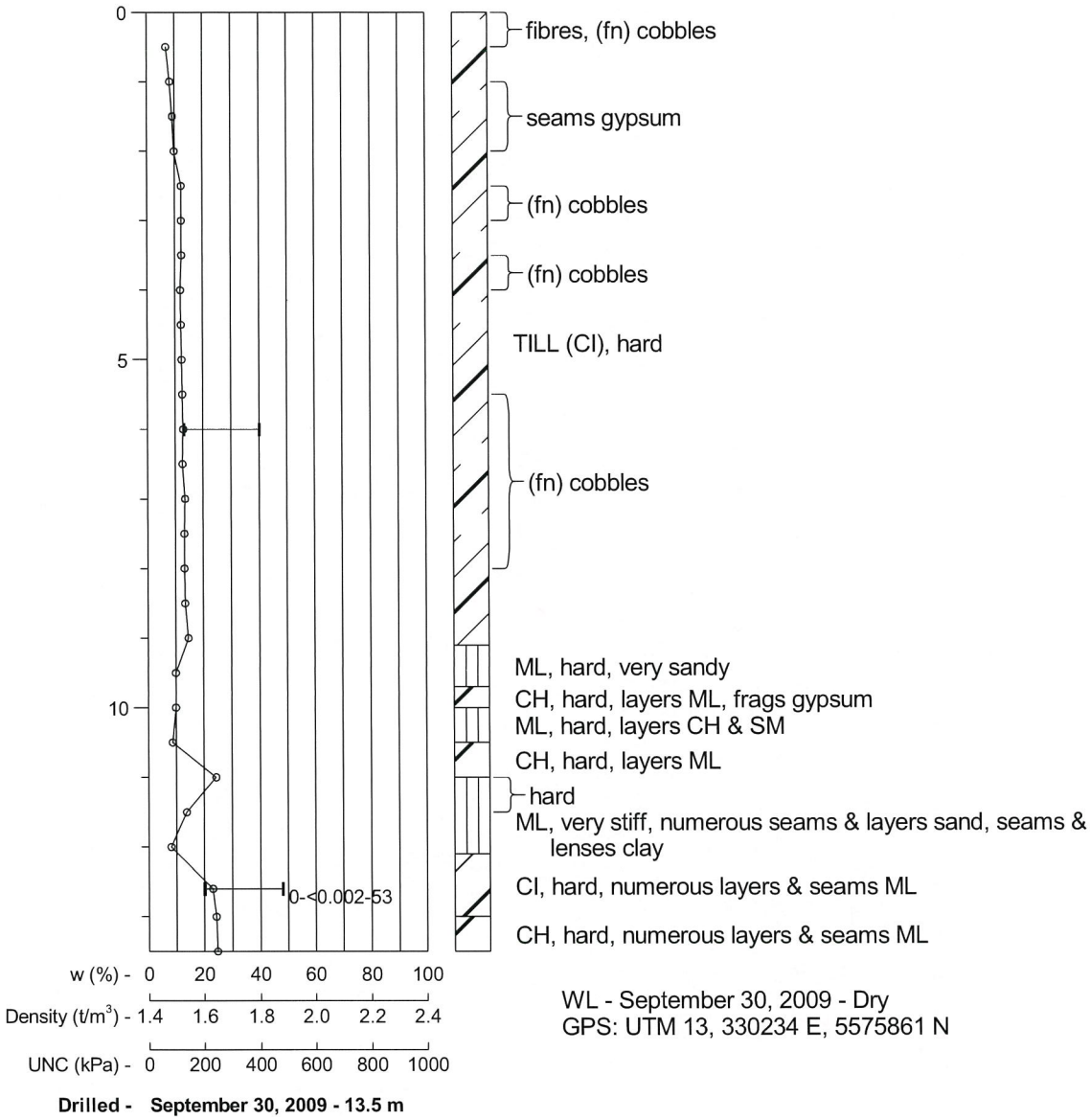
WL - September 29, 2009 - Dry
GPS: UTM 13, 330338 E, 5575730 N

Drilled - September 29, 2009 - 10.5 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Spillway Channel C/L, East Abutment**				**C51**
	Drawn/Checked: **NB/KB/VK**	**2010/03/22**	**4547:928:7H4**	**358925**	**1 of 1**

C52

EL 731.689 m



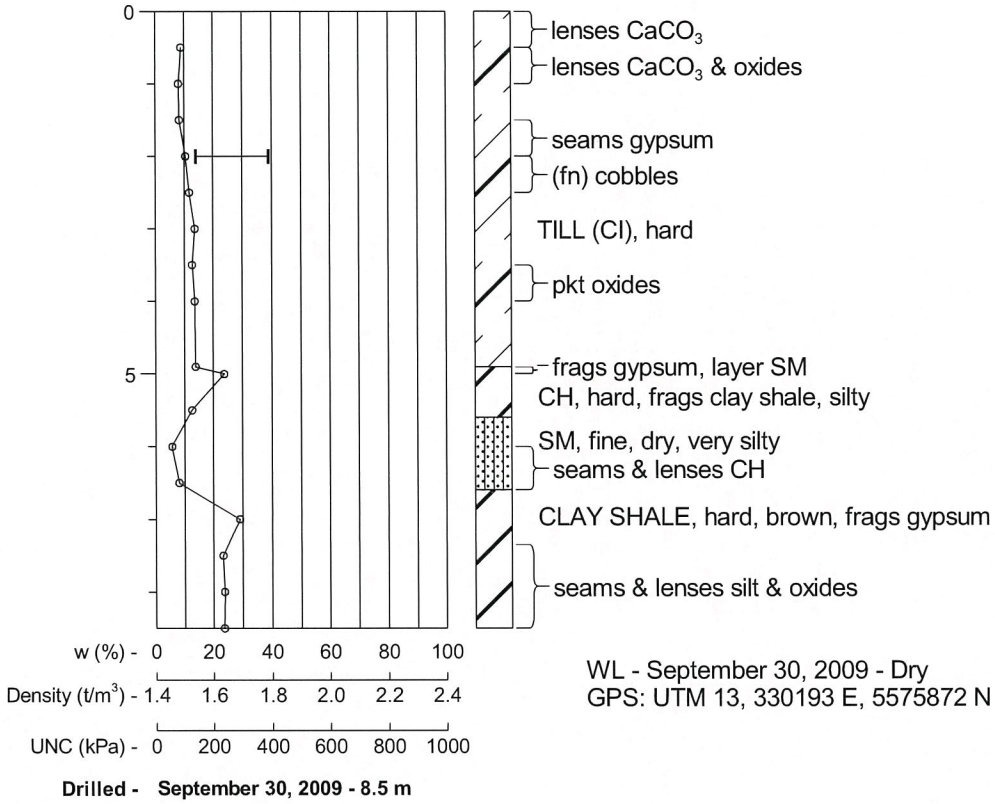
Drilled - September 30, 2009 - 13.5 m

WL - September 30, 2009 - Dry
GPS: UTM 13, 330234 E, 5575861 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Spillway Channel C/L, East Abutment**				**C52**
	Drawn/Checked: **NB/KB/VK**	**2010/03/22**	**4547:928:7H4**	**358926**	**1 of 1**

C53

EL 727.792 m



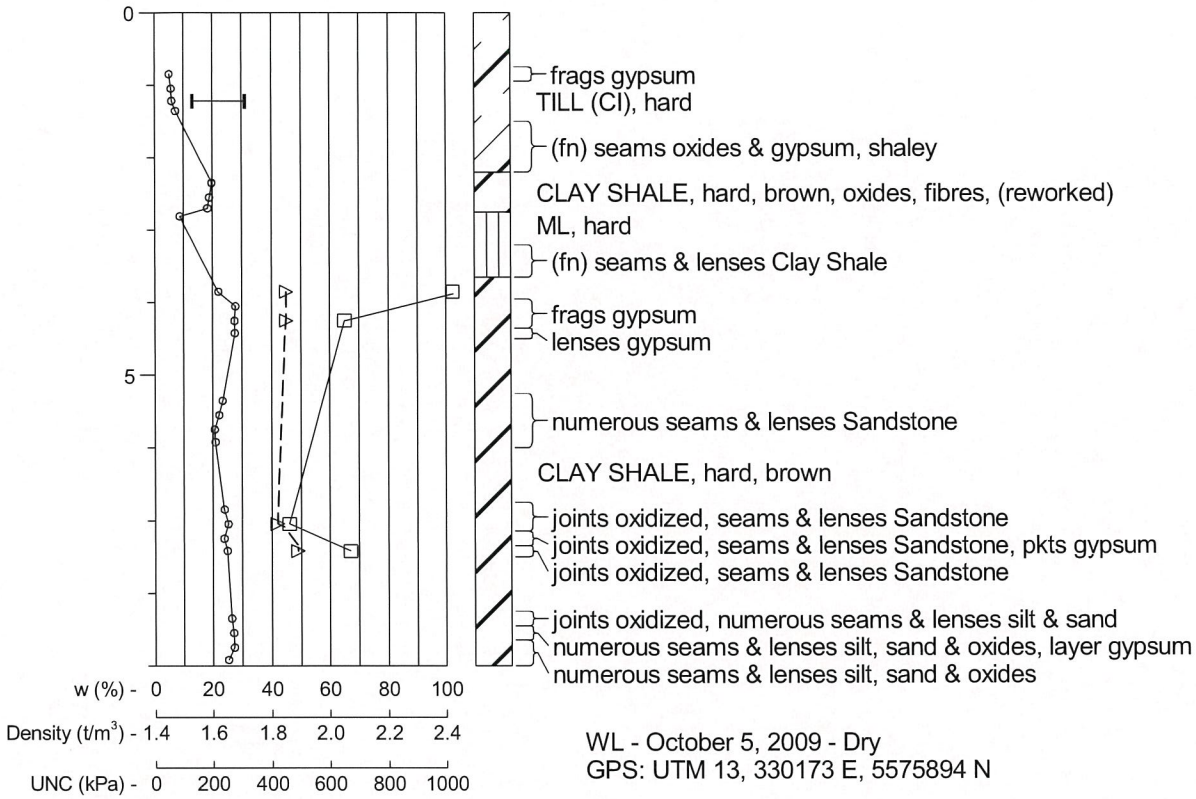
WL - September 30, 2009 - Dry
GPS: UTM 13, 330193 E, 5575872 N

Drilled - September 30, 2009 - 8.5 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Spillway Channel C/L**				**C53**
	Drawn/Checked: **NB/KB/VK**	**2010/03/22**	**4547:928:7H4**	**358927**	**1 of 1**

C54

EL 725.260 m



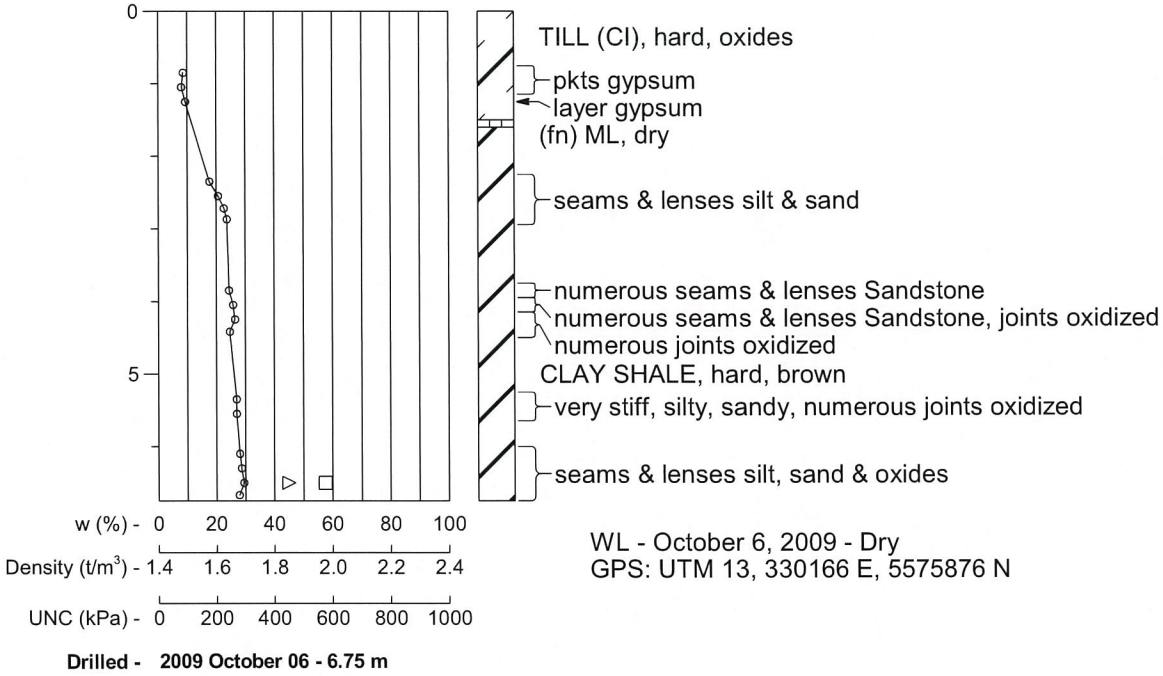
Drilled - October 5, 2009 - 9 m

WL - October 5, 2009 - Dry
GPS: UTM 13, 330173 E, 5575894 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Spillway Structure, East Abutment**				**C54**
	Drawn/Checked: **NB/KB/VK**	**2010/03/22**	**4547:928:7H4**	**358928**	**1 of 1**

C55

EL 724.092 m

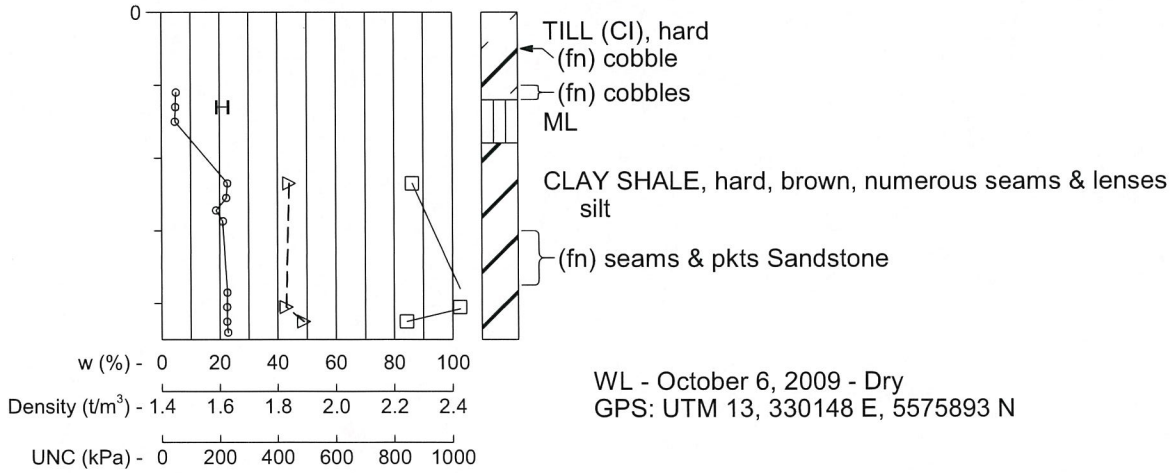


WL - October 6, 2009 - Dry
GPS: UTM 13, 330166 E, 5575876 N

Drilled - 2009 October 06 - 6.75 m

C56

EL 722.533 m



WL - October 6, 2009 - Dry
GPS: UTM 13, 330148 E, 5575893 N

Drilled - 2009 October 06 - 4.5 m

Agriculture and Agri-Food Canada | Agriculture et Agroalimentaire Canada

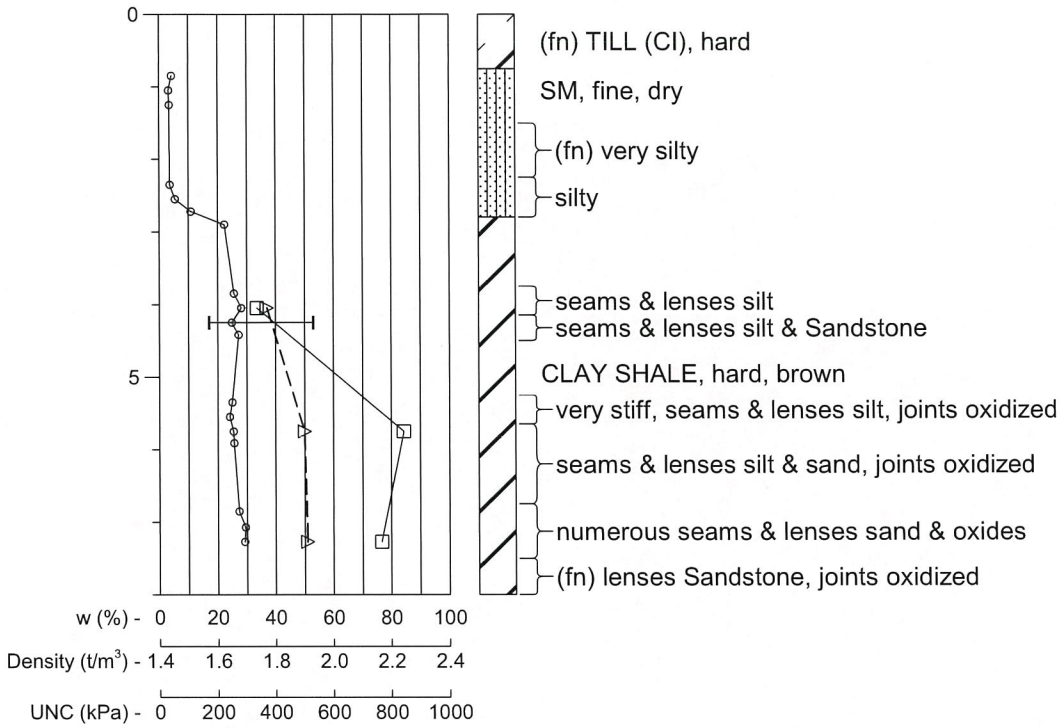
Highfield Dam

NE 36-15-11 W3, Proposed Spillway Structure

Drawn/Checked:	NB/KB/VK	2010/03/09	4547:928:7H4	358929

C57

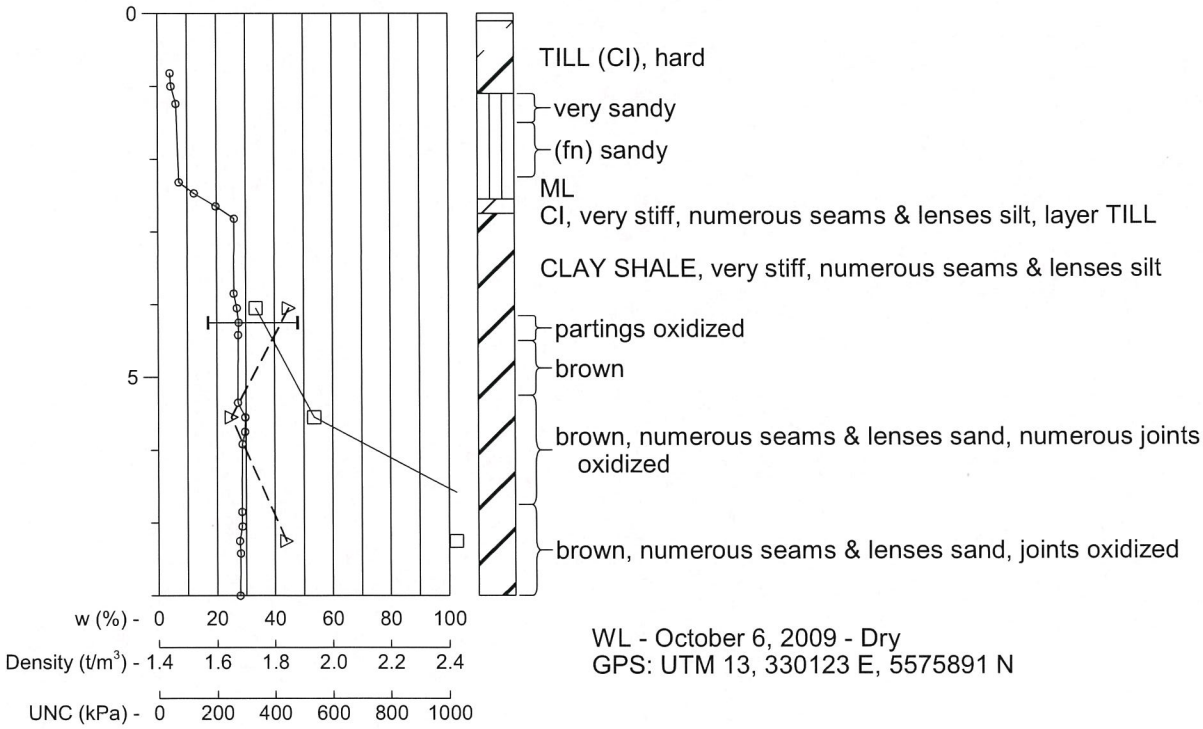
EL 721.463 m



Drilled - 2009 October 06 - 8 m

C58

EL 719.951 m



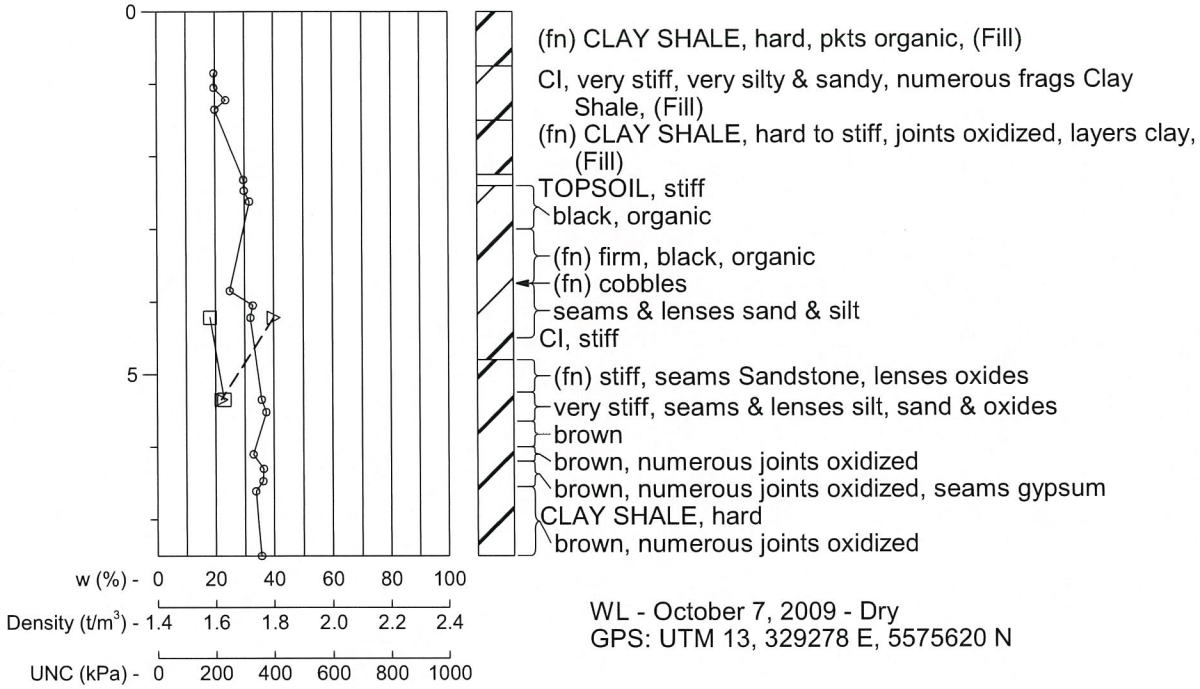
WL - October 6, 2009 - Dry
GPS: UTM 13, 330123 E, 5575891 N

Drilled - 2009 October 06 - 8 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Proposed Spillway Structure C/L**			
	Drawn/Checked: **NB/KB/VK**	**2010/03/09**	**4547:928:7H4**	**358930**

C59

EL 721.672 m



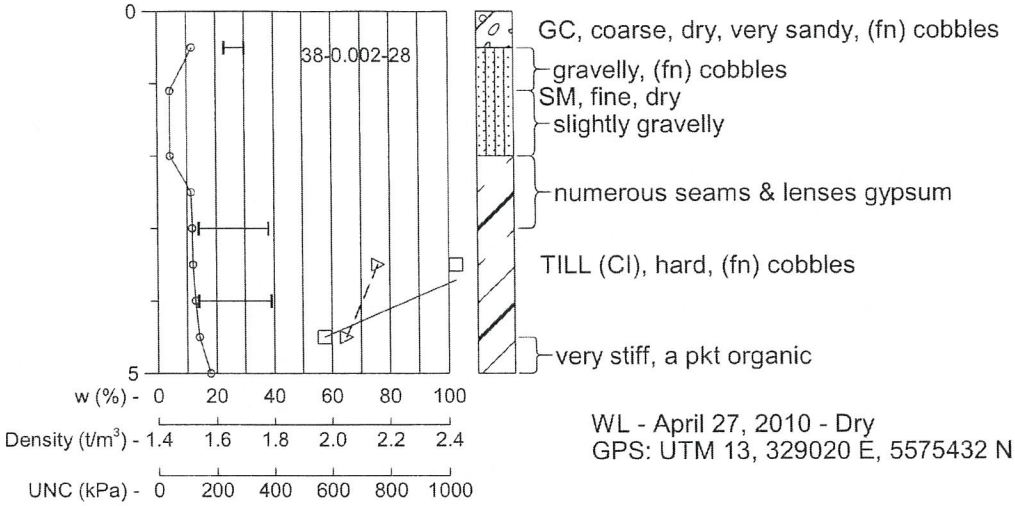
WL - October 7, 2009 - Dry
GPS: UTM 13, 329278 E, 5575620 N

Drilled - 2009 October 07 - 7.5 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Canal Inlet Structure, West Abutment**				**C59**
	Drawn/Checked: **NB/KB/VK**	**2010/02/15**	**4547:928:7H4**	**358931**	**1 of 1**

C60

EL m



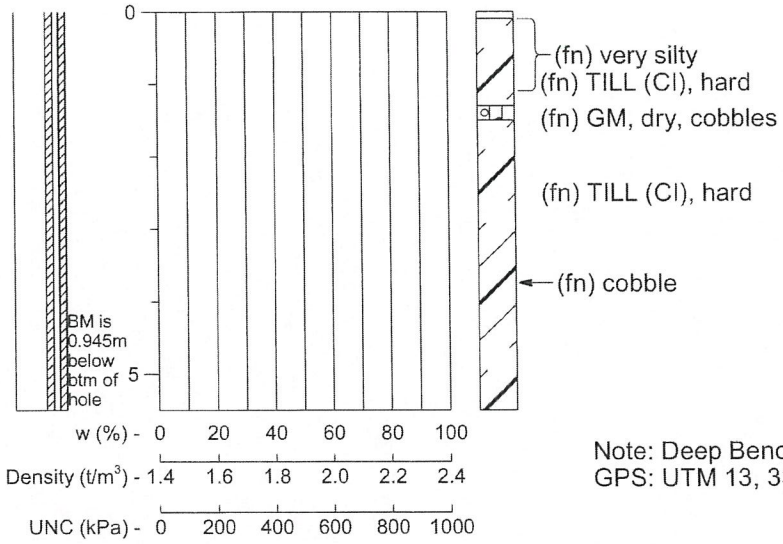
WL - April 27, 2010 - Dry
GPS: UTM 13, 329020 E, 5575432 N

Drilled - 2010 April 27 - 5 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Dam West Abutment**				**C60**
	Drawn/Checked: NB/KB/VK	2010/05/07	4547:928:7H4	358980	1 of 1

C61

EL m



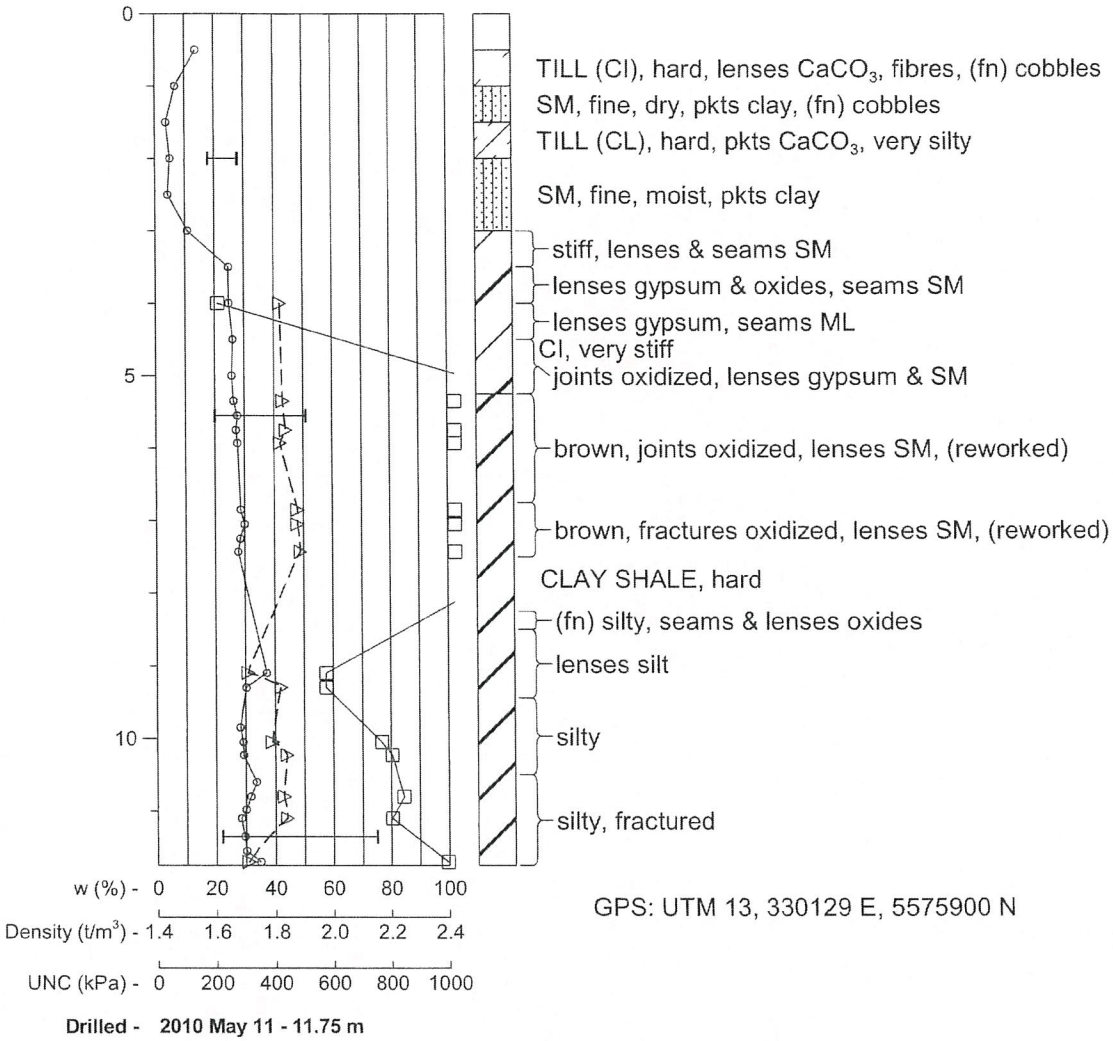
Drilled - 2010 April 27 - 5.5 m

Note: Deep Bench Mark
GPS: UTM 13, 330414 E, 5575775 N

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **NE 36-15-11 W3, Dam East Abutment**				**C61**
	Drawn/Checked: **NB/KB/VK**	**2010/05/07**	**4547:928:7H4**	**358981**	**1 of 1**

C62

EL m



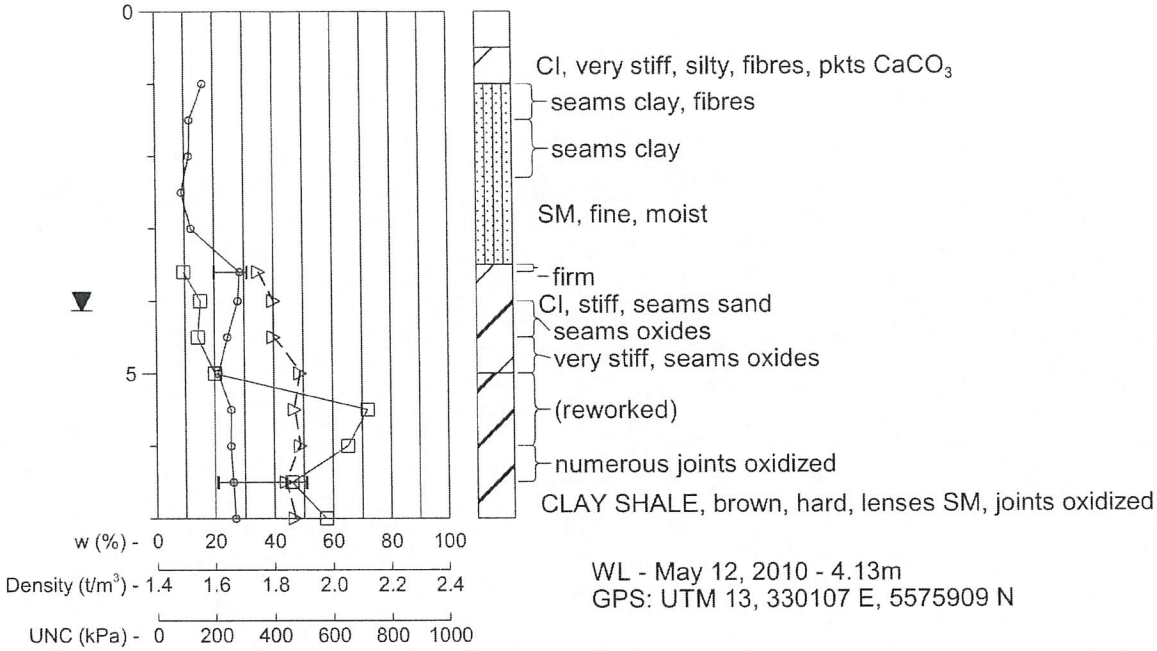
GPS: UTM 13, 330129 E, 5575900 N

Drilled - 2010 May 11 - 11.75 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **SE 01-16-11-W3, Proposed Spillway**				**C62**
	Drawn/Checked: **BK/NB/VK**	**2010/05/25**	**4547:928:7H4**	**358982**	**1 of 1**

C63

EL m



WL - May 12, 2010 - 4.13m
GPS: UTM 13, 330107 E, 5575909 N

Drilled - 2010 May 11 - 7 m

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada	**Highfield Dam** **SE 01-16-11-W3, Proposed Spillway**				**C63**
	Drawn/Checked: **BK/NB/VK**	**2010/05/25**	**4547:928:7H4**	**358983**	**1 of 1**

Appendix E

VW Piezometer Field Data

Borehole ID	Vibrating Wire Piezometer		VW Piezometer Readings				Difference (m)
			2/3/2011		3/2/2011		
	ID	S/N	Head (m)	^{0}C	Head (m)	^{0}C	
R2488-01	R2488-01A	10-5594	718.45	8.6	718.43	8.4	-0.02
	R2488-01B	10-5593	718.01	8.4	717.97	8.4	-0.04
	R2488-01C	10-5452	718.03	7.9	718.00	7.8	-0.04
R2488-02	R2488-02A	10-4651	720.66	8	720.72	8	0.06
	R2488-02B	10-5573	720.35	7.8	720.45	7.8	0.10
	R2488-02C	10-5449	718.17	7.4	718.19	7	0.02
R2488-03	R2488-03A	10-4718	722.07	7.2	722.17	7.10	0.10
	R2488-03B	10-4652	721.80	7.5	721.73	7.4	-0.07
	R2488-03C	10-5574	720.05	7.2	720.09	7.4	0.04
	R2488-03D	10-5453	720.24	7	720.31	6.9	0.07

Appendix F

Laboratory Testing Results

Atterberg Limits Laboratory Reports

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-01
Technician:	TW
Date:	4-Feb-2011

Sample: FGL-10 at 20-21.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A37	T9B	
Tare Wt, g	13.8	14.38	
Wet + Tare, g	20.36	20.92	
Dry + Tare, g	19.40	20.00	average
M%	17.1%	16.4%	16.8%

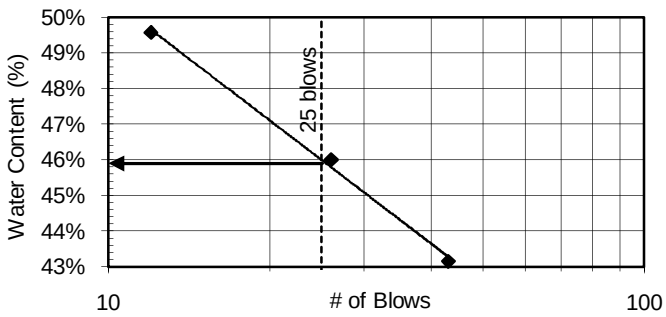
Liquid Limit (method A)

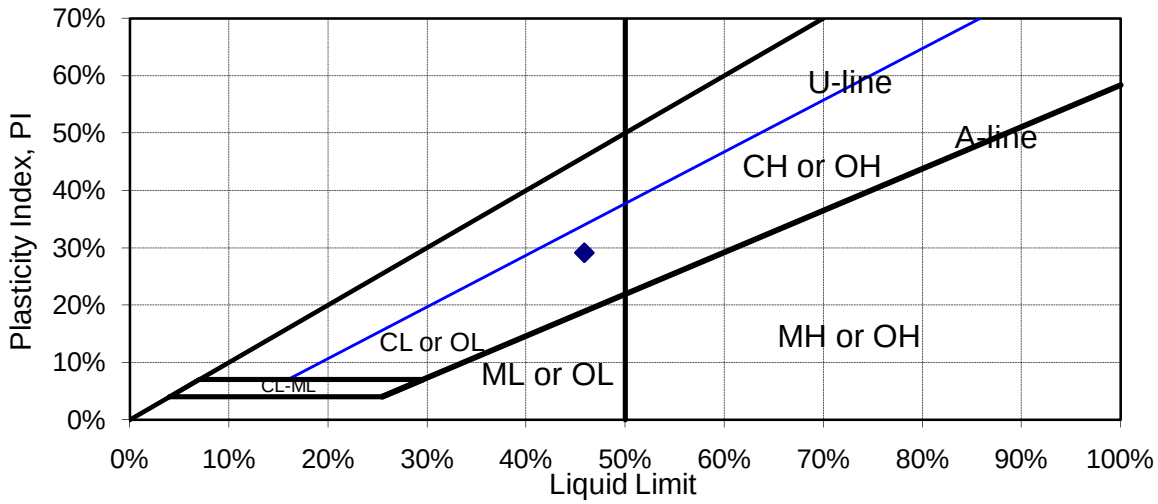
# of Blows	12	26	43
Tare #	T6	MSS	T100
Tare Wt, g	14.09	14.72	14.57
Wet + tare, g	19.34	19.29	20.64
Dry + tare, g	17.60	17.85	18.81
Water content	49.6%	46.0%	43.2%

SUMMARY

Plastic Limit:	16.8%
Liquid Limit:	45.9%
Plasticity Index:	29.1%
Classification:	CL

Natural Water Content: 19.5%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-01
Technician:	TW
Date:	4-Feb-2011

Sample: FGL-11 at 22.58-23.17' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	1J	15A	
Tare Wt, g	14.65	14.56	
Wet + Tare, g	21.92	20.98	
Dry + Tare, g	20.96	20.15	average
M%	15.2%	14.8%	15.0%

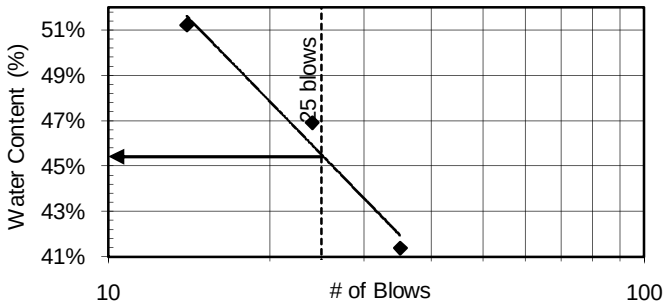
Liquid Limit (method A)

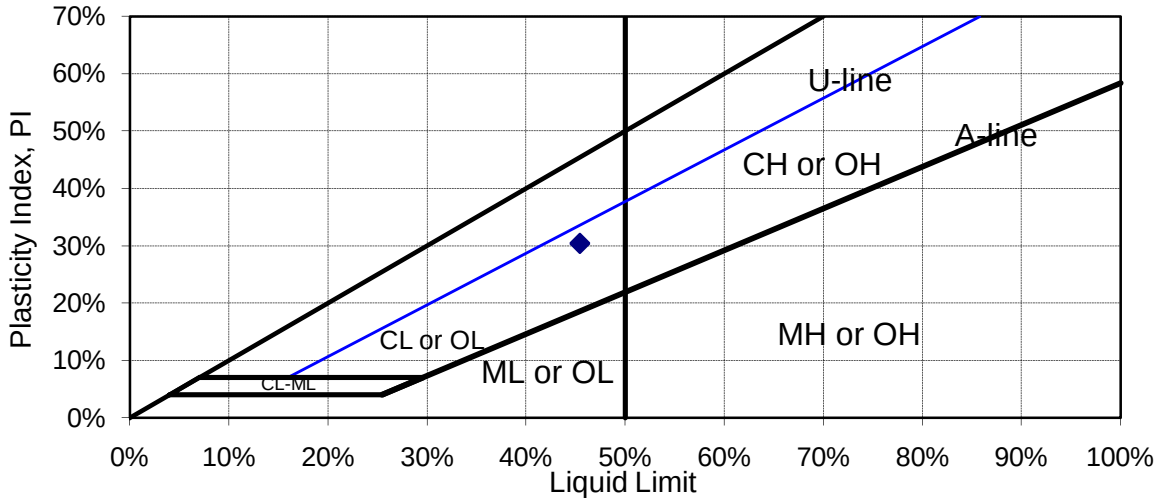
# of Blows	14	24	35
Tare #	20A	103	TLL
Tare Wt, g	14.47	17.47	14.16
Wet + tare, g	18.22	22.92	20.14
Dry + tare, g	16.95	21.18	18.39
Water content	51.2%	46.9%	41.4%

SUMMARY

Plastic Limit:	15.0%
Liquid Limit:	45.4%
Plasticity Index:	30.4%
Classification:	CL

Natural Water Content: 26.3%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-01
Technician:	TW
Date:	4-Feb-2011

Sample: FGL-16 at 34.1-35' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	PA	29A	
Tare Wt, g	14.12	14.45	
Wet + Tare, g	19.11	19.75	
Dry + Tare, g	18.05	18.67	average
M%	27.0%	25.6%	26.3%

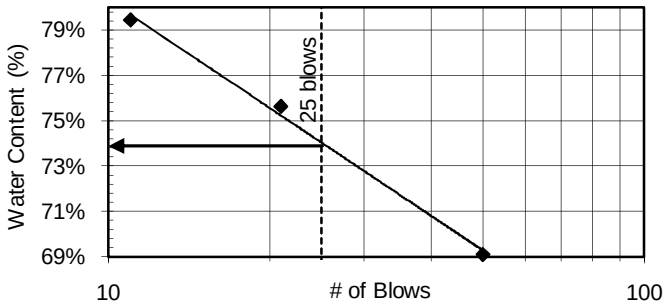
Liquid Limit (method A)

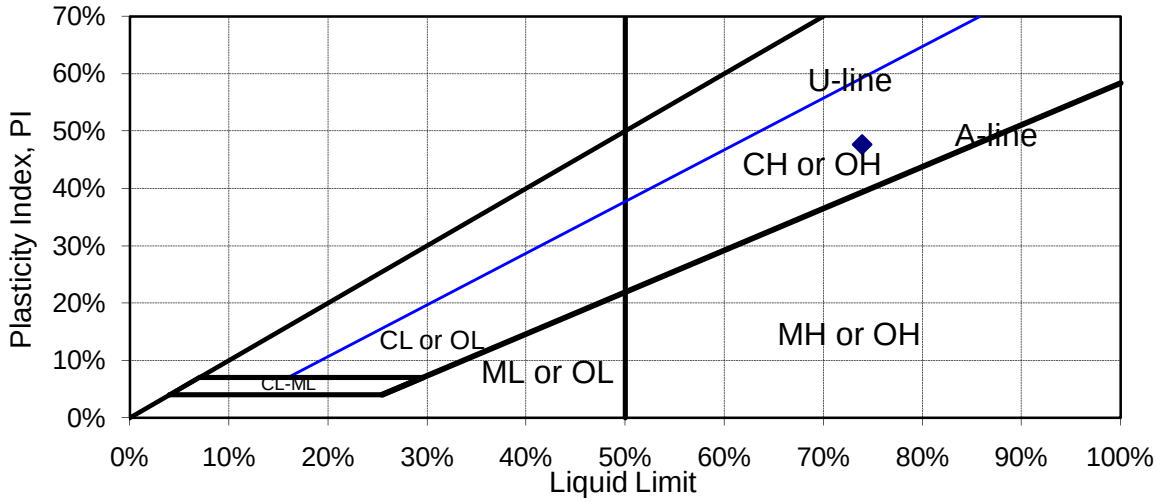
# of Blows	11	21	50
Tare #	44A	B4	39A
Tare Wt, g	14.50	14.46	14.39
Wet + tare, g	21.39	20.08	18.33
Dry + tare, g	18.34	17.66	16.72
Water content	79.4%	75.6%	69.1%

SUMMARY

Plastic Limit:	26.3%
Liquid Limit:	73.9%
Plasticity Index:	47.6%
Classification:	CH

Natural Water Content: 35.1%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-01
Technician:	TW
Date:	4-Feb-2011

Sample: FGL-17B at 38' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	PA	29A	
Tare Wt, g	14.12	14.45	
Wet + Tare, g	19.21	18.53	
Dry + Tare, g	18.18	17.71	average
M%	25.4%	25.2%	25.3%

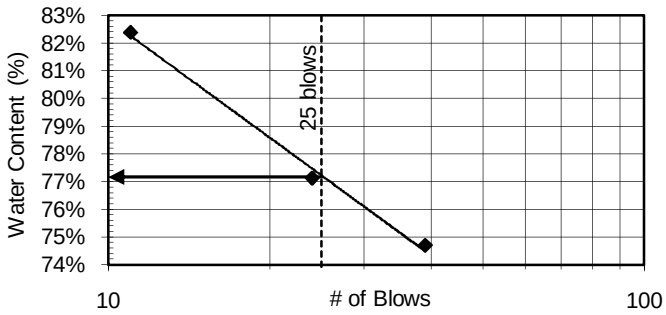
Liquid Limit (method A)

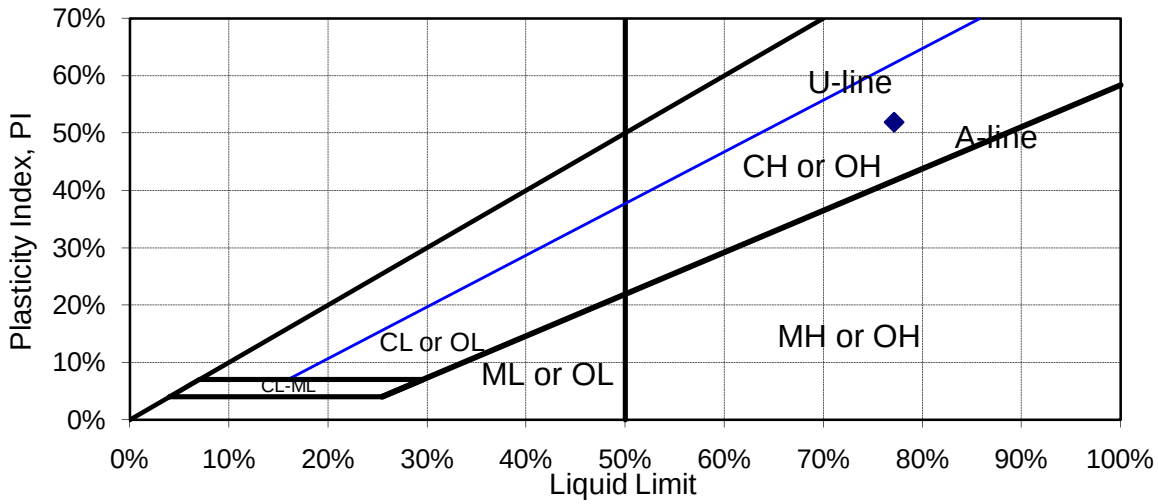
# of Blows	11	24	39
Tare #	36A	15A	T6
Tare Wt, g	14.57	14.55	14.08
Wet + tare, g	18.09	19.97	19.81
Dry + tare, g	16.50	17.61	17.36
Water content	82.4%	77.1%	74.7%

SUMMARY

Plastic Limit:	25.3%
Liquid Limit:	77.2%
Plasticity Index:	51.9%
Classification:	CH

Natural Water Content: 36.1%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-01
Technician:	AB
Date:	17-Feb-2011

Sample: FGL-20A at 45.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A18	A47	
Tare Wt, g	13.51	13.66	
Wet + Tare, g	19.77	20.28	
Dry + Tare, g	18.62	19.01	average
M%	22.5%	23.7%	23.1%

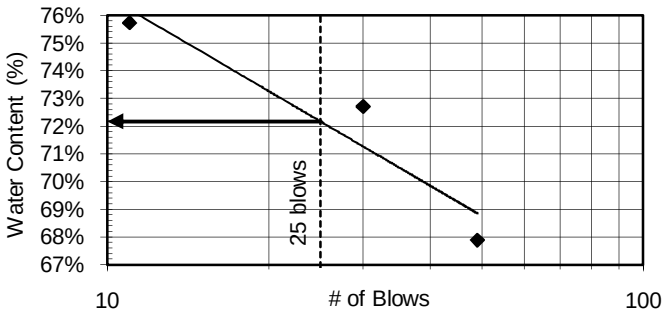
Liquid Limit (method A)

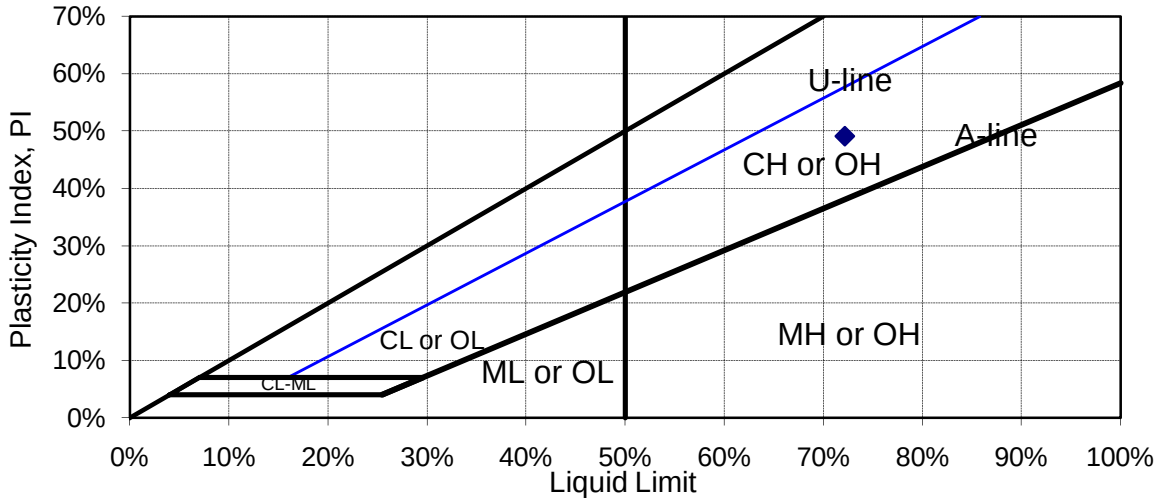
# of Blows	11	30	49
Tare #	MVP	6A	1A
Tare Wt, g	14.54	14.24	14.29
Wet + tare, g	22.94	23.29	23.02
Dry + tare, g	19.32	19.48	19.49
Water content	75.7%	72.7%	67.9%

SUMMARY

Plastic Limit:	23.1%
Liquid Limit:	72.2%
Plasticity Index:	49.1%
Classification:	CH

Natural Water Content: 29.3%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-01
Technician:	TW
Date:	3-Feb-2011

Sample: FGL-22B (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: N/A

Plastic Limit

Tare #			
Tare Wt, g			
Wet + Tare, g			
Dry + Tare, g			average
M%			

Liquid Limit (method A)

# of Blows				
Tare #				
Tare Wt, g				
Wet + tare, g				
Dry + tare, g				
Water content				

Non-Plastic

SUMMARY

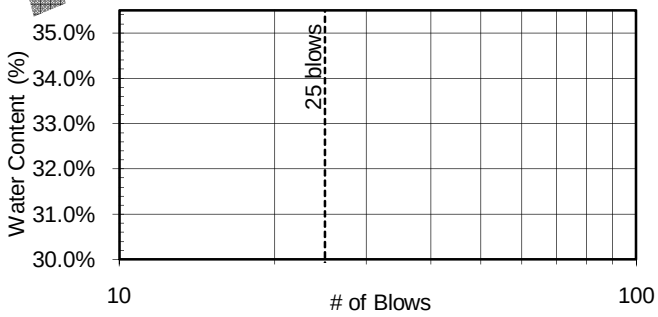
Plastic Limit:

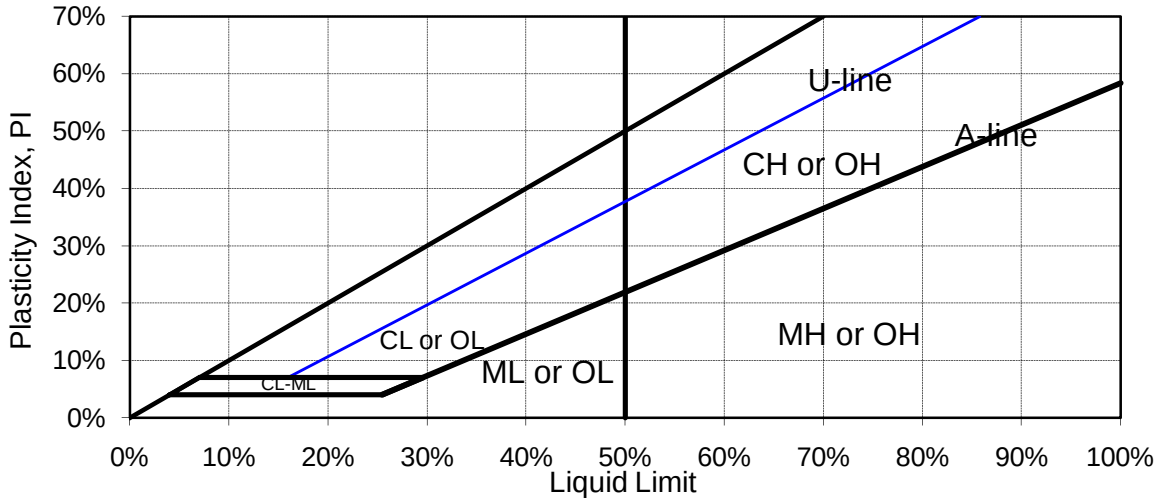
Liquid Limit:

Plasticity Index:

Classification:

Natural Water Content: N/A





Comments: Non-Plastic

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-02
Technician:	TW
Date:	1-Feb-2011

Sample: FGL-27 at 5-6.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	B2	PPE	
Tare Wt, g	14.23	14.22	
Wet + Tare, g	19.59	19.26	
Dry + Tare, g	18.82	18.54	average
M%	16.8%	16.7%	16.7%

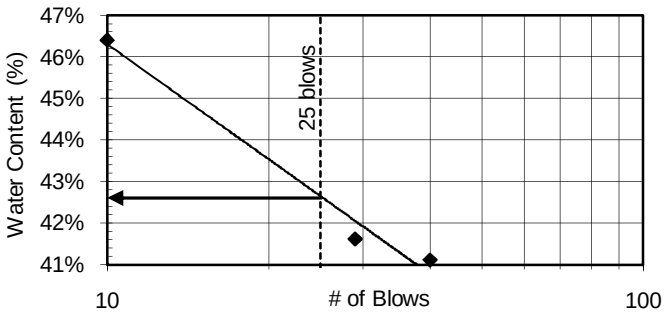
Liquid Limit (method A)

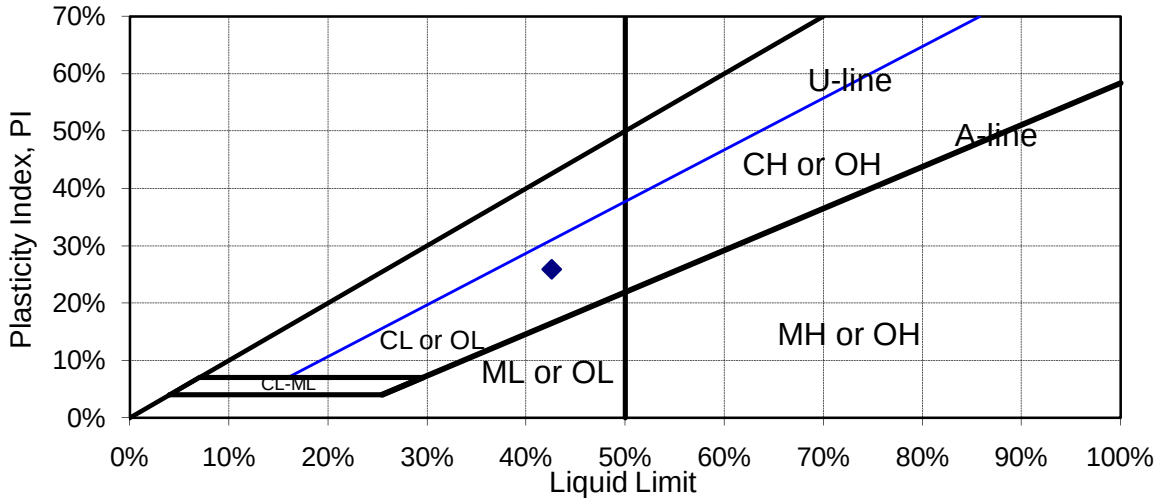
# of Blows	10	29	40
Tare #	63A	B7	R2
Tare Wt, g	14.42	14.47	14.20
Wet + tare, g	22.97	20.97	20.31
Dry + tare, g	20.26	19.06	18.53
Water content	46.4%	41.6%	41.1%

SUMMARY

Plastic Limit:	16.7%
Liquid Limit:	42.6%
Plasticity Index:	25.9%
Classification:	CL

Natural Water Content: 23.6%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-02
Technician:	TW
Date:	1-Feb-2011

Sample: FGL-38 at 38.5-39.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A47	TAK	
Tare Wt, g	13.66	14.27	
Wet + Tare, g	20.29	19.51	
Dry + Tare, g	18.71	18.30	average
M%	31.3%	30.0%	30.7%

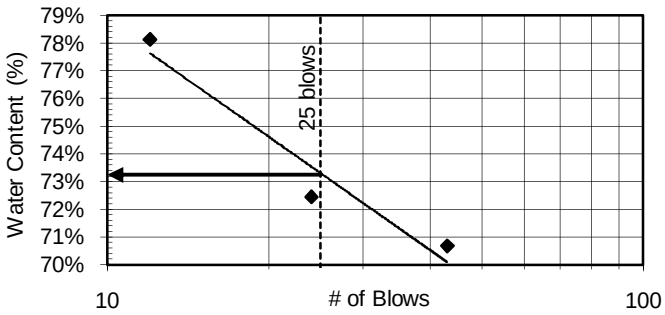
Liquid Limit (method A)

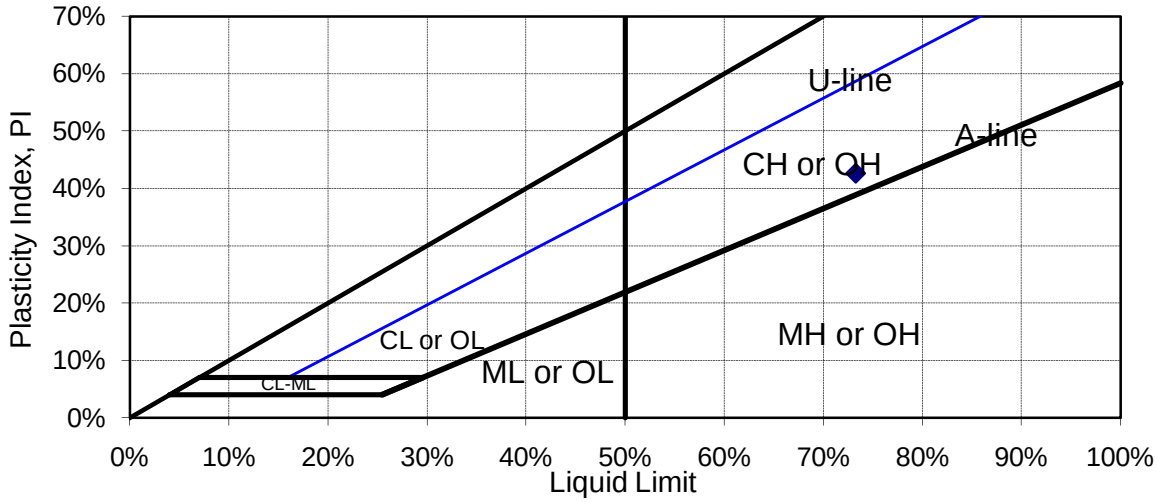
# of Blows	12	24	43
Tare #	70A	B4	66A
Tare Wt, g	14.34	14.46	14.48
Wet + tare, g	25.01	20.22	23.39
Dry + tare, g	20.33	17.80	19.70
Water content	78.1%	72.5%	70.7%

SUMMARY

Plastic Limit:	30.7%
Liquid Limit:	73.3%
Plasticity Index:	42.6%
Classification:	CH

Natural Water Content: 53.5%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-02
Technician:	TW
Date:	28-Feb-2011

Sample: FGL-39A at 40-41' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	36A	20A	
Tare Wt, g	14.56	14.48	
Wet + Tare, g	18.69	21.72	
Dry + Tare, g	17.68	19.98	average
M%	32.4%	31.6%	32.0%

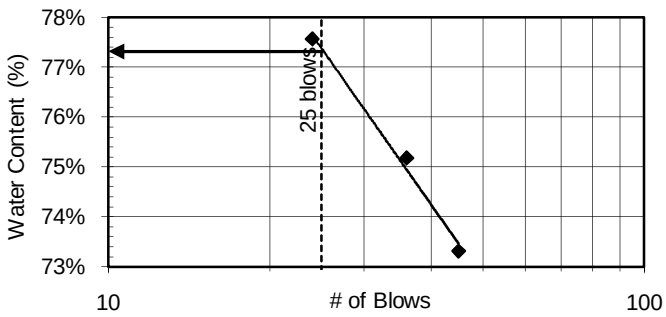
Liquid Limit (method A)

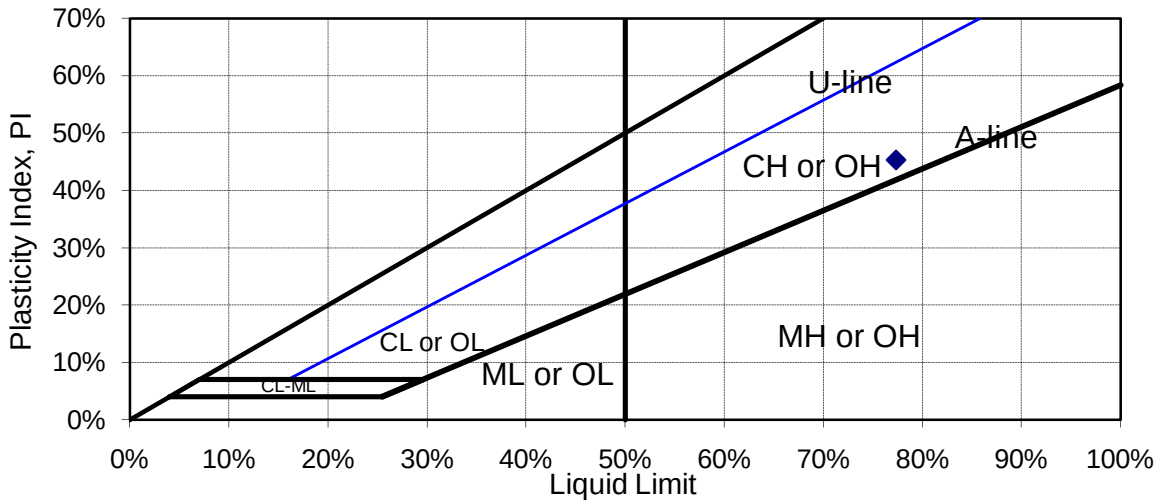
# of Blows	24	36	45
Tare #	TAK	21A	A35
Tare Wt, g	14.28	14.64	13.19
Wet + tare, g	22.59	21.98	24.75
Dry + tare, g	18.96	18.83	19.86
Water content	77.6%	75.2%	73.3%

SUMMARY

Plastic Limit:	32.0%
Liquid Limit:	77.3%
Plasticity Index:	45.3%
Classification:	CH

Natural Water Content: 57.8%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-02
Technician:	AB
Date:	22-Feb-2011

Sample: FGL-41 at 48' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A26	A21	
Tare Wt, g	12.68	13.21	
Wet + Tare, g	20.62	19.30	
Dry + Tare, g	18.84	17.93	average
M%	28.9%	29.0%	29.0%

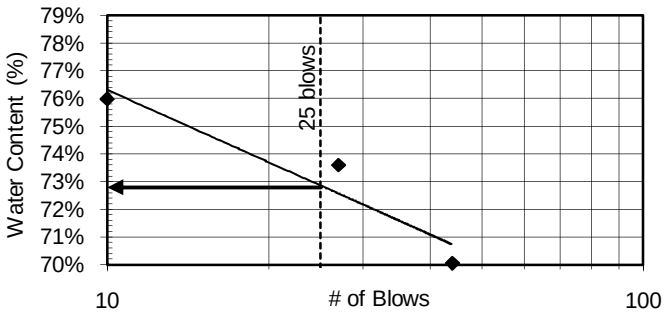
Liquid Limit (method A)

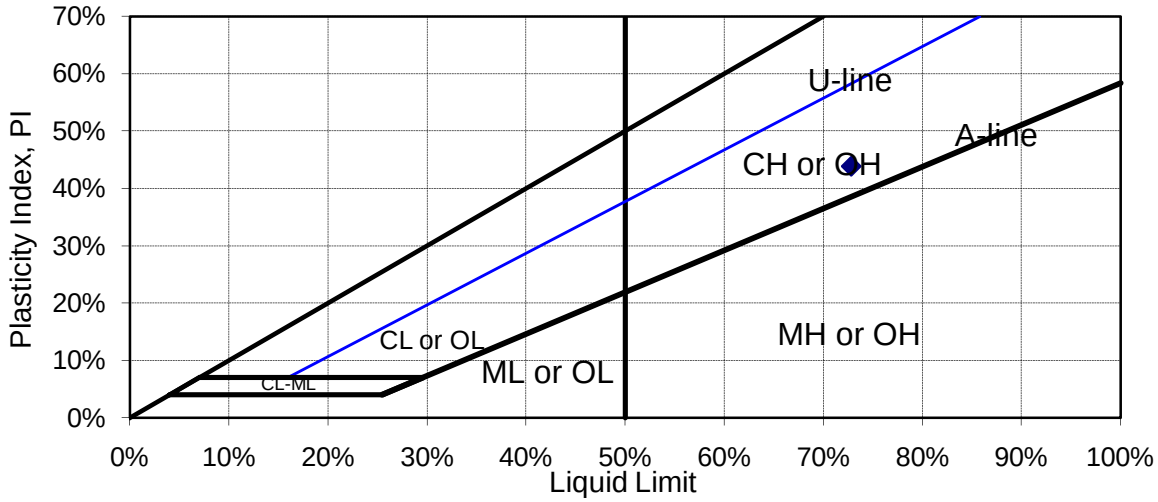
# of Blows	10	27	44
Tare #	23A	PA	64A
Tare Wt, g	14.33	14.11	14.51
Wet + tare, g	22.97	19.63	21.55
Dry + tare, g	19.24	17.29	18.65
Water content	76.0%	73.6%	70.0%

SUMMARY

Plastic Limit:	29.0%
Liquid Limit:	72.8%
Plasticity Index:	43.8%
Classification:	CH

Natural Water Content: 36.9%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-02
Technician:	AB
Date:	16-Feb-2011

Sample: FGL-46 at 61' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A9	A37	
Tare Wt, g	13.2	13.79	
Wet + Tare, g	23.16	20.77	
Dry + Tare, g	20.99	19.27	average
M%	27.9%	27.4%	27.6%

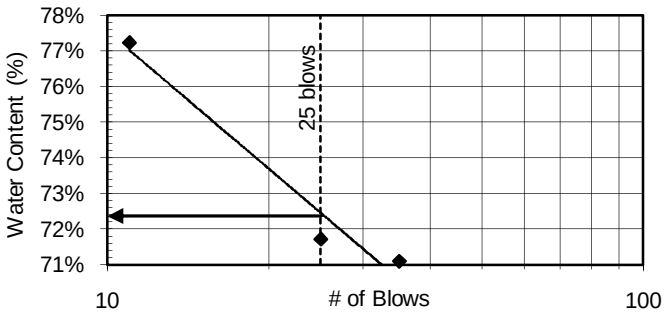
Liquid Limit (method A)

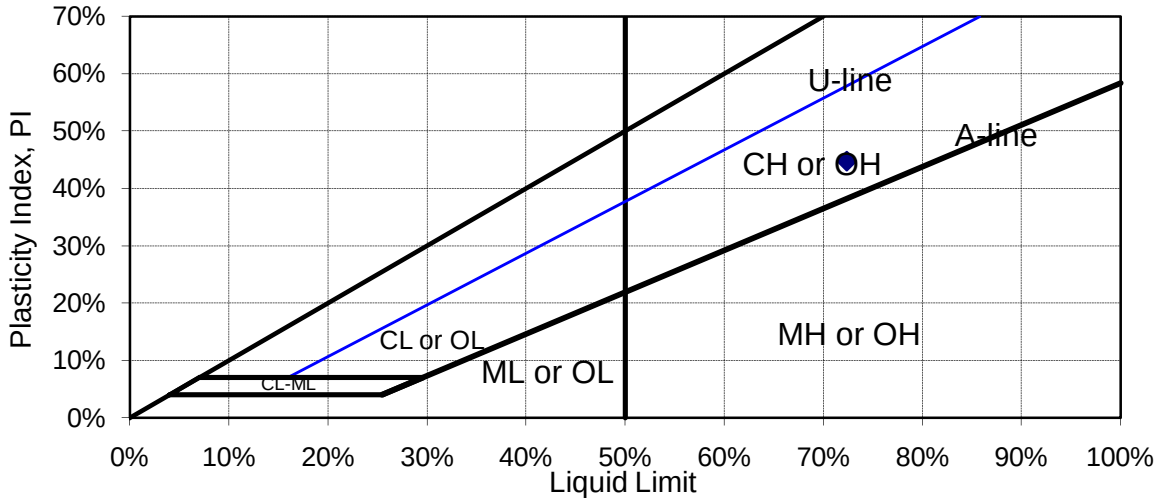
# of Blows	11	25	35
Tare #	B6	YAN	T6
Tare Wt, g	14.53	14.45	14.09
Wet + tare, g	31.03	23.62	25.40
Dry + tare, g	23.84	19.79	20.70
Water content	77.2%	71.7%	71.1%

SUMMARY

Plastic Limit:	27.6%
Liquid Limit:	72.4%
Plasticity Index:	44.8%
Classification:	CH

Natural Water Content: 33.1%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-02
Technician:	AB
Date:	17-Feb-2011

Sample: FGL-50 at 72' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A35	A24	
Tare Wt, g	13.19	13.02	
Wet + Tare, g	21.12	21.62	
Dry + Tare, g	19.37	19.72	average
M%	28.3%	28.4%	28.3%

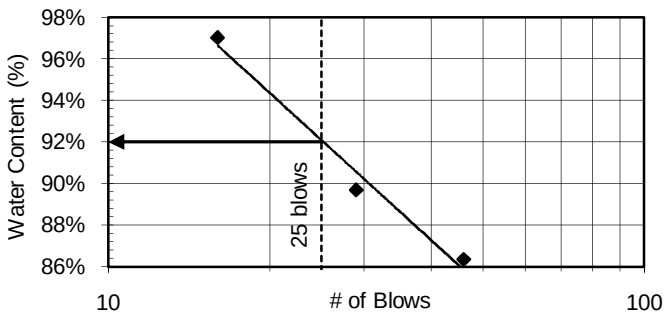
Liquid Limit (method A)

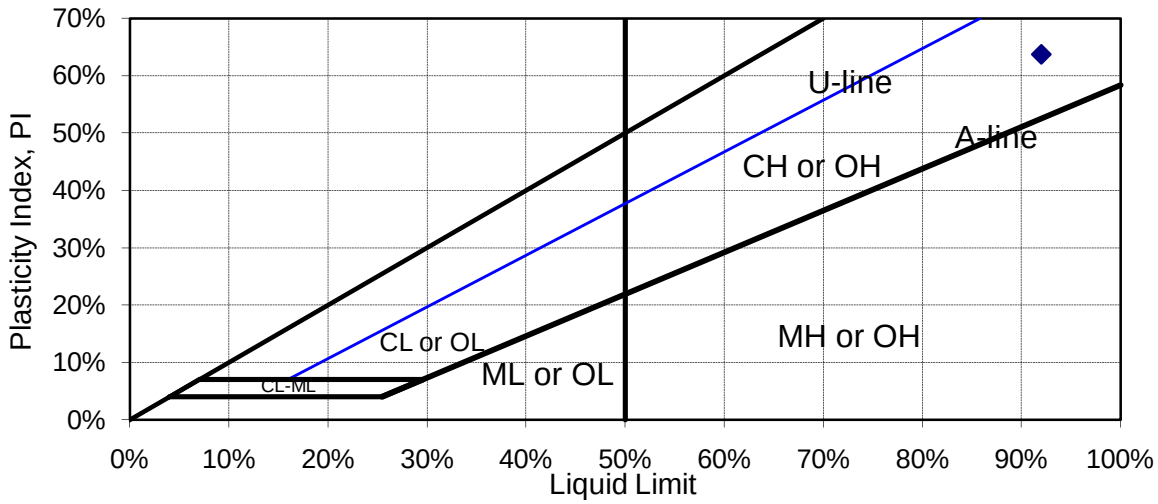
# of Blows	16	29	46
Tare #	21A	AD7	44A
Tare Wt, g	14.63	14.33	14.51
Wet + tare, g	24.54	25.18	21.61
Dry + tare, g	19.66	20.05	18.32
Water content	97.0%	89.7%	86.4%

SUMMARY

Plastic Limit:	28.3%
Liquid Limit:	92.0%
Plasticity Index:	63.7%
Classification:	CH

Natural Water Content: 32.8%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG/TW
Date:	1-Feb-2011

Sample: FGL-52 at 10-11.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	6A	46A	
Tare Wt, g	14.23	14.14	
Wet + Tare, g	19.44	20.26	
Dry + Tare, g	18.55	19.22	average
M%	20.6%	20.5%	20.5%

Liquid Limit (method A)

# of Blows	11	27	46
Tare #	47A	38A	13A
Tare Wt, g	14.45	14.56	14.09
Wet + tare, g	20.55	20.64	20.59
Dry + tare, g	18.46	18.72	18.57
Water content	52.1%	46.2%	45.1%

SUMMARY

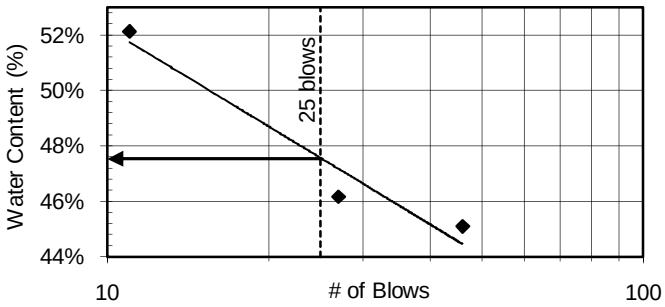
Plastic Limit: 20.5%

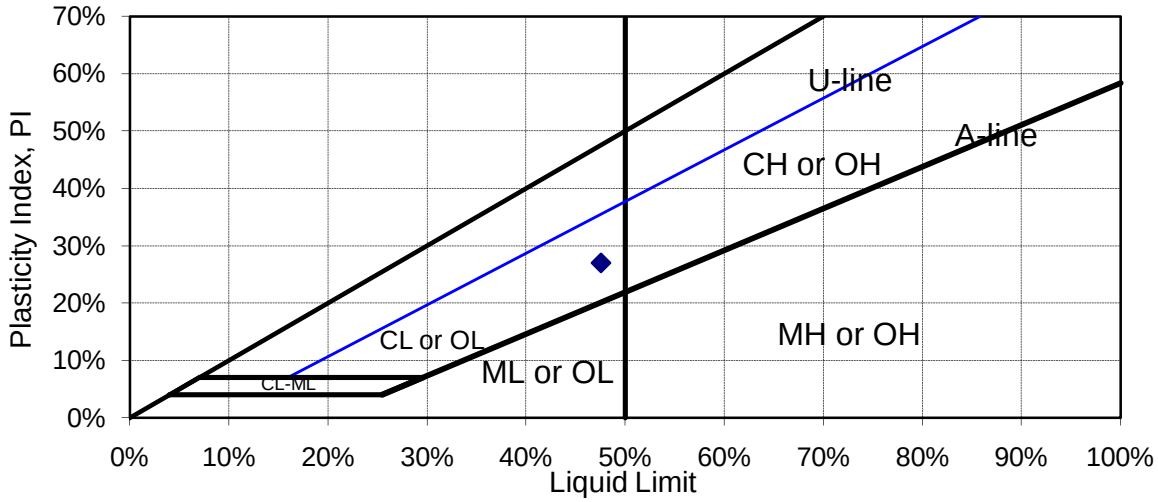
Liquid Limit: 47.5%

Plasticity Index: 27.0%

Classification: CL

Natural Water Content: 27.0%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-53 at 15' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	13A	B7	
Tare Wt, g	14.08	14.47	
Wet + Tare, g	20.88	20.97	
Dry + Tare, g	19.56	19.72	average
M%	24.1%	23.8%	23.9%

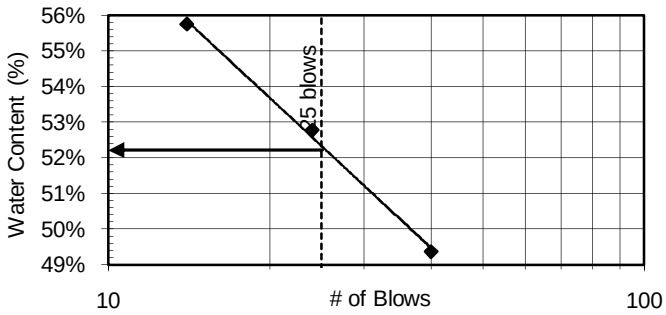
Liquid Limit (method A)

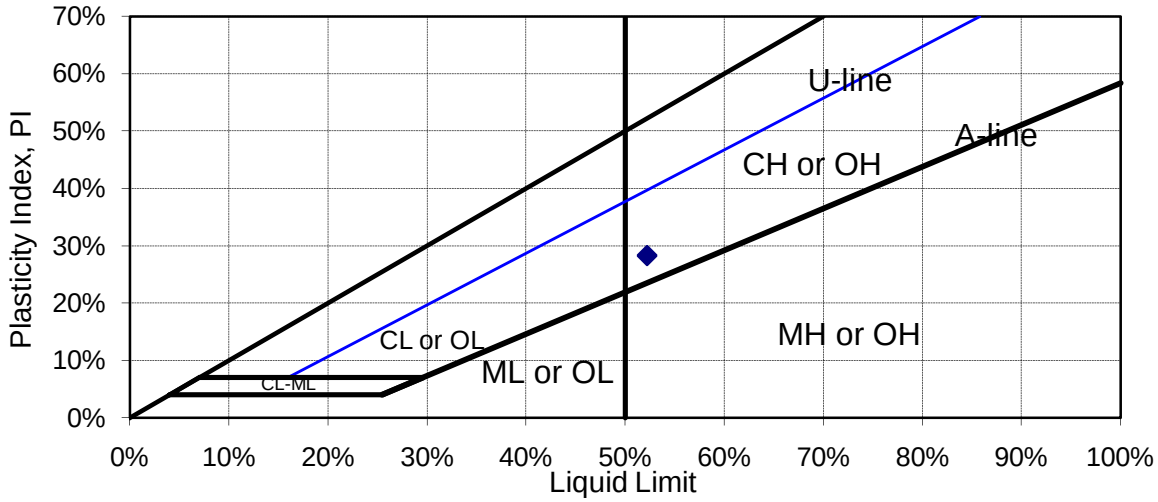
# of Blows	14	24	40
Tare #	63A	46A	R2
Tare Wt, g	14.42	14.14	14.19
Wet + tare, g	23.22	21.84	21.24
Dry + tare, g	20.07	19.18	18.91
Water content	55.8%	52.8%	49.4%

SUMMARY

Plastic Limit:	23.9%
Liquid Limit:	52.2%
Plasticity Index:	28.3%
Classification:	CH

Natural Water Content: 34.2%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-54A at 17.4' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	MVP	70A	
Tare Wt, g	14.53	14.34	
Wet + Tare, g	19.14	18.99	
Dry + Tare, g	18.24	18.05	average
M%	24.3%	25.3%	24.8%

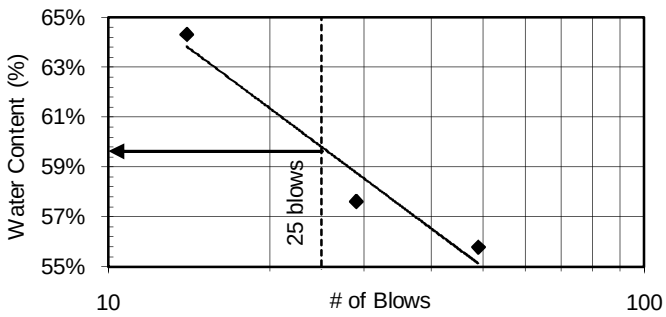
Liquid Limit (method A)

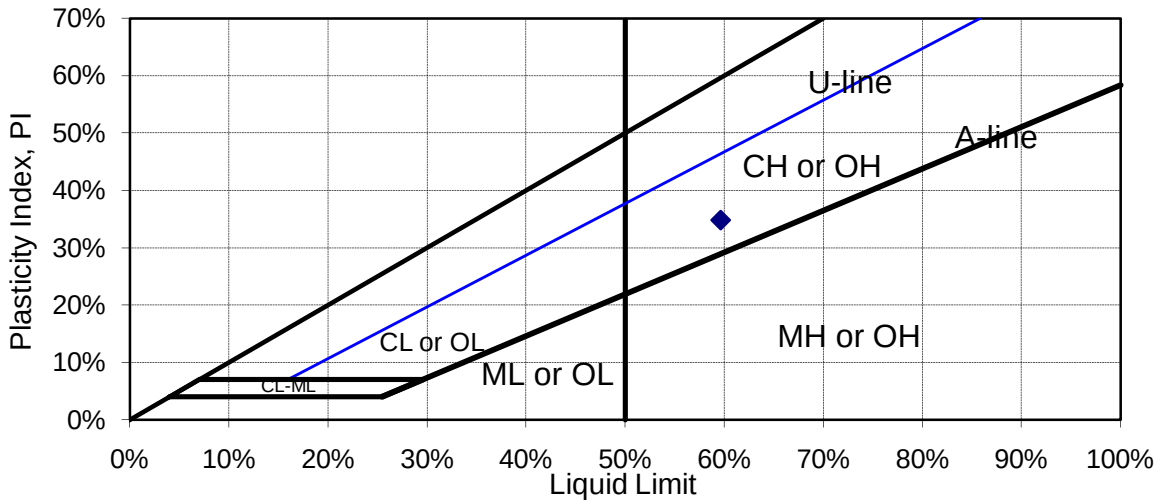
# of Blows	14	29	49
Tare #	66A	44A	A18
Tare Wt, g	14.48	14.50	13.51
Wet + tare, g	23.78	23.91	20.91
Dry + tare, g	20.14	20.47	18.26
Water content	64.3%	57.6%	55.8%

SUMMARY

Plastic Limit:	24.8%
Liquid Limit:	59.6%
Plasticity Index:	34.8%
Classification:	CH

Natural Water Content: 32.3%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-54B at 19' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	47A	6A	
Tare Wt, g	14.47	14.24	
Wet + Tare, g	20.10	19.85	
Dry + Tare, g	18.91	18.66	average
M%	26.8%	26.9%	26.9%

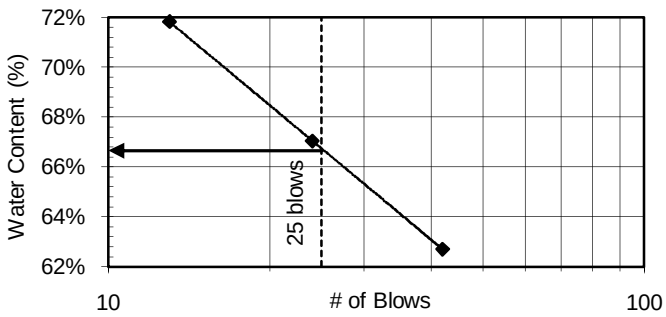
Liquid Limit (method A)

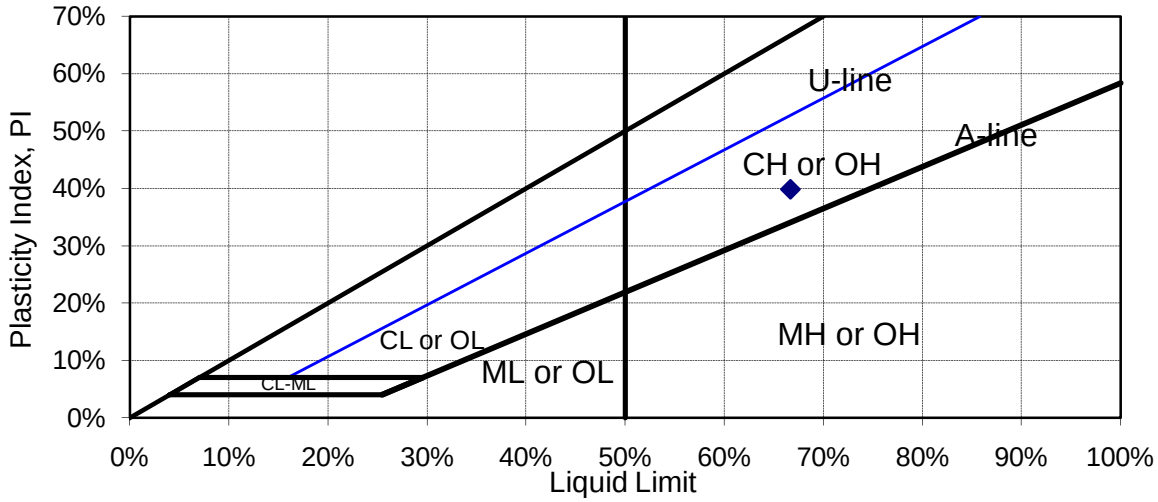
# of Blows	13	24	42
Tare #	TAK	38A	66A
Tare Wt, g	14.26	14.56	14.49
Wet + tare, g	22.37	21.91	21.34
Dry + tare, g	18.98	18.96	18.70
Water content	71.8%	67.0%	62.7%

SUMMARY

Plastic Limit:	26.9%
Liquid Limit:	66.7%
Plasticity Index:	39.8%
Classification:	CH

Natural Water Content: 32.8%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-56A at 22' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	65A	T7M	
Tare Wt, g	14.35	14.53	
Wet + Tare, g	19.96	20.52	
Dry + Tare, g	18.56	19.01	average
M%	33.3%	33.7%	33.5%

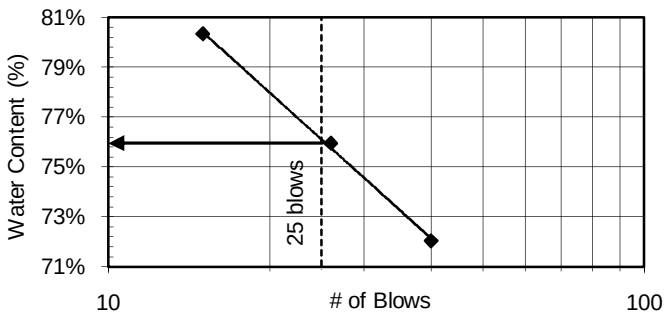
Liquid Limit (method A)

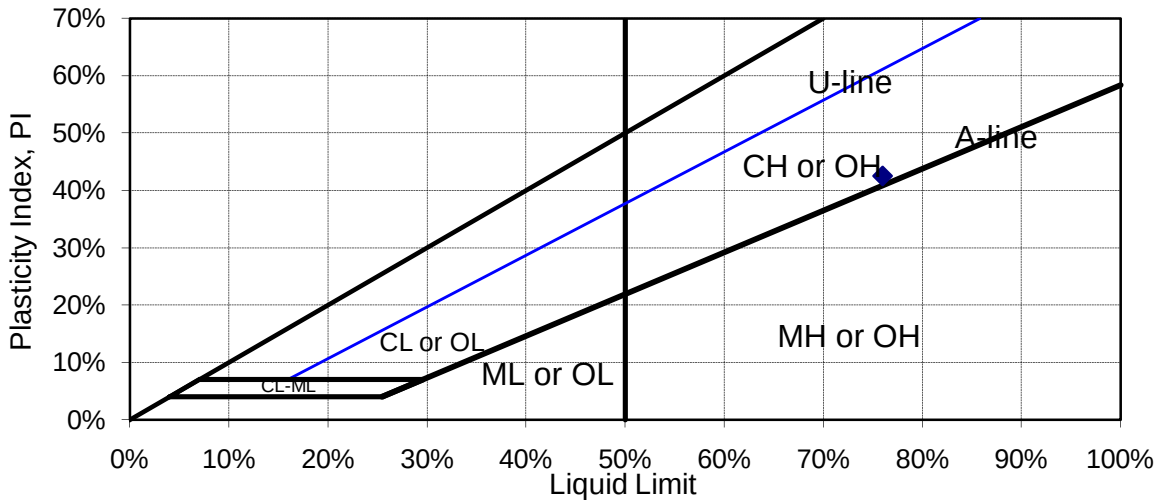
# of Blows	15	26	40
Tare #	23A	21A	40A
Tare Wt, g	14.33	14.64	14.41
Wet + tare, g	23.22	21.66	20.93
Dry + tare, g	19.26	18.63	18.20
Water content	80.3%	75.9%	72.0%

SUMMARY

Plastic Limit:	33.5%
Liquid Limit:	76.0%
Plasticity Index:	42.5%
Classification:	CH

Natural Water Content: 36.8%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	TW
Date:	3-Feb-2011

Sample: FGL-58 at 29-30' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	55A	TLL	
Tare Wt, g	14.16	14.17	
Wet + Tare, g	18.44	17.83	
Dry + Tare, g	17.59	17.10	average
M%	24.8%	24.9%	24.8%

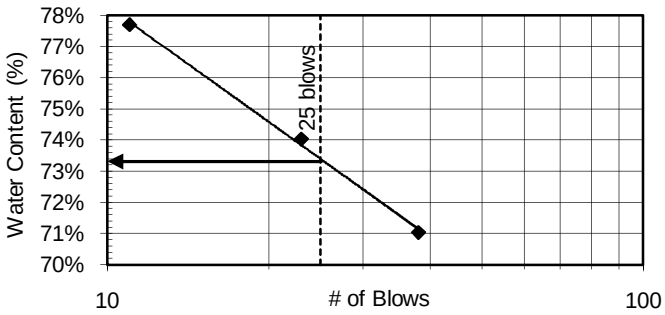
Liquid Limit (method A)

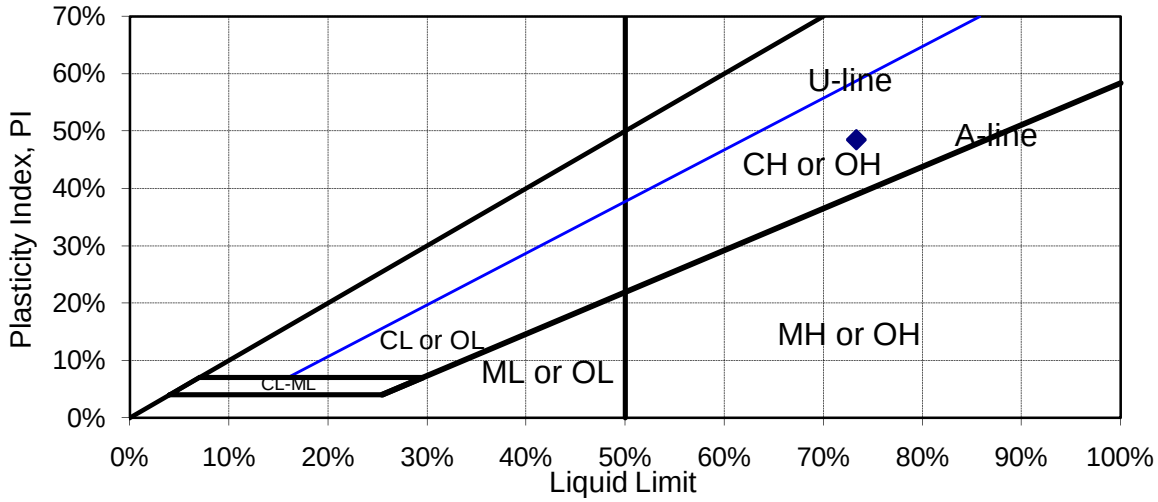
# of Blows	11	23	38
Tare #	BA	B4	B2
Tare Wt, g	14.09	14.47	14.24
Wet + tare, g	19.03	18.96	19.85
Dry + tare, g	16.87	17.05	17.52
Water content	77.7%	74.0%	71.0%

SUMMARY

Plastic Limit:	24.8%
Liquid Limit:	73.3%
Plasticity Index:	48.5%
Classification:	CH

Natural Water Content: 34.2%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-60 at 31.5-32' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	B4	20A	
Tare Wt, g	14.46	14.46	
Wet + Tare, g	19.95	18.43	
Dry + Tare, g	18.66	17.47	average
M%	30.7%	31.9%	31.3%

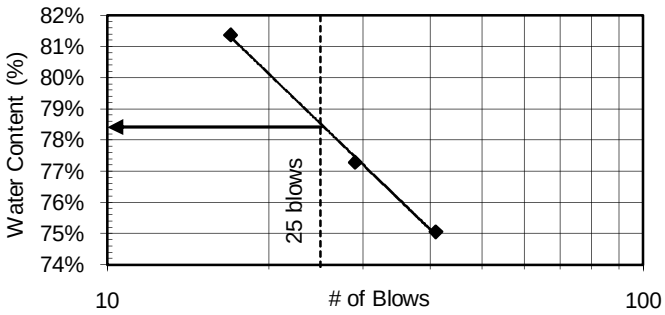
Liquid Limit (method A)

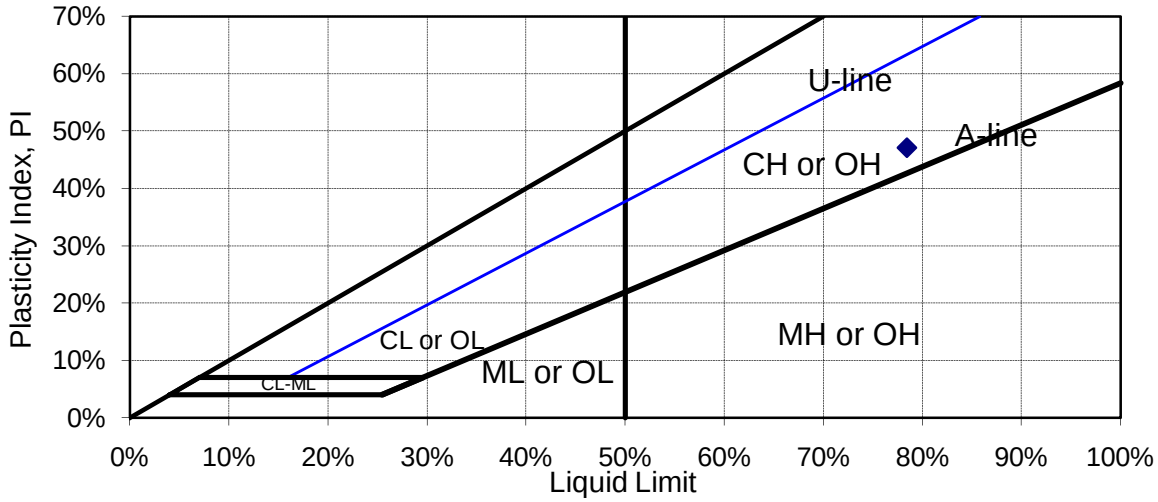
# of Blows	17	29	41
Tare #	PPE	38A	B2
Tare Wt, g	14.22	14.57	14.23
Wet + tare, g	21.42	22.14	21.81
Dry + tare, g	18.19	18.84	18.56
Water content	81.4%	77.3%	75.1%

SUMMARY

Plastic Limit:	31.3%
Liquid Limit:	78.4%
Plasticity Index:	47.1%
Classification:	CH

Natural Water Content: 35.6%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-61B (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	6A	15A	
Tare Wt, g	14.24	14.56	
Wet + Tare, g	19.32	18.77	
Dry + Tare, g	18.20	17.83	average
M%	28.3%	28.7%	28.5%

Liquid Limit (method A)

# of Blows	19	28	46
Tare #	TAK	2J	47A
Tare Wt, g	14.27	14.40	14.47
Wet + tare, g	22.07	22.20	20.62
Dry + tare, g	18.31	18.57	17.85
Water content	93.1%	87.1%	82.0%

SUMMARY

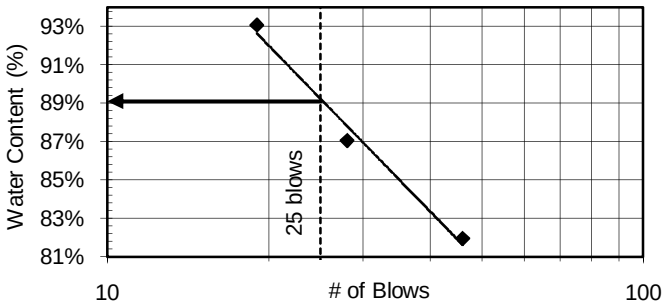
Plastic Limit: 28.5%

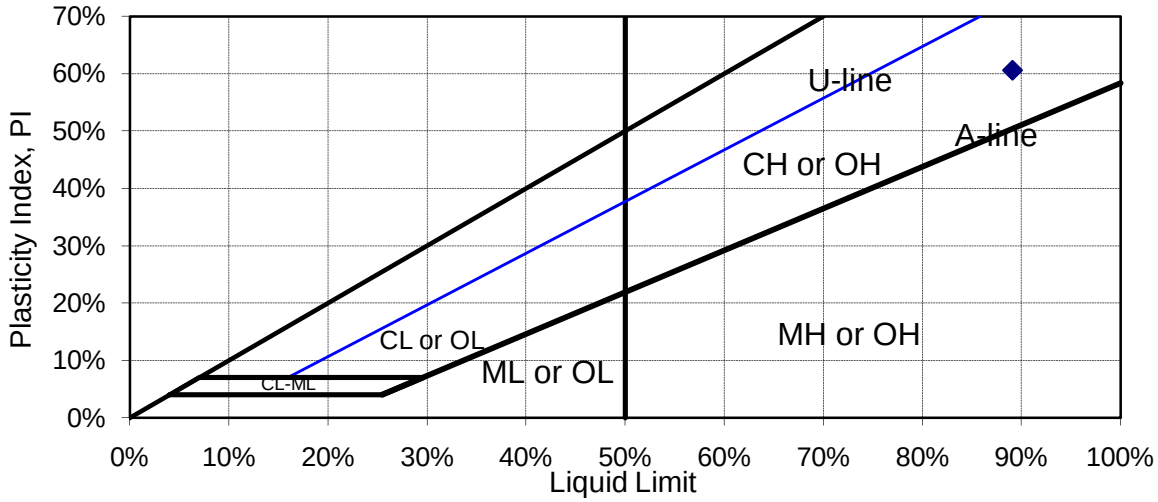
Liquid Limit: 89.1%

Plasticity Index: 60.6%

Classification: CH

Natural Water Content: 34.4%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-62B at 40' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	65A	70A	
Tare Wt, g	14.34	14.34	
Wet + Tare, g	20.67	20.12	
Dry + Tare, g	19.43	18.92	average
M%	24.4%	26.2%	25.3%

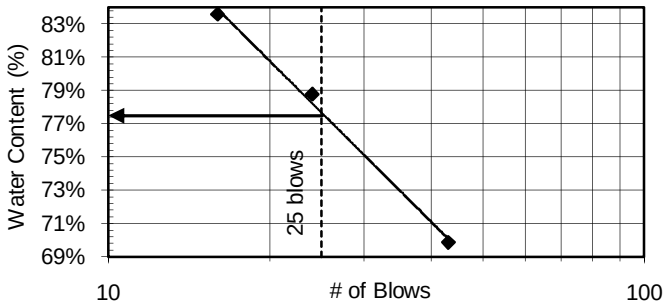
Liquid Limit (method A)

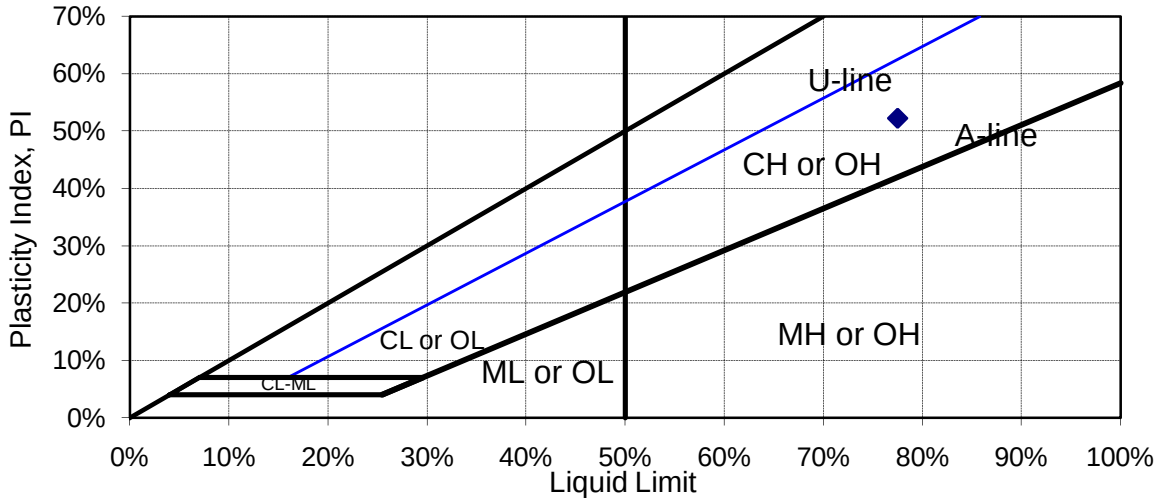
# of Blows	16	24	43
Tare #	23A	B4	2J
Tare Wt, g	14.30	14.46	14.40
Wet + tare, g	23.68	23.72	23.42
Dry + tare, g	19.41	19.64	19.71
Water content	83.6%	78.8%	69.9%

SUMMARY

Plastic Limit:	25.3%
Liquid Limit:	77.5%
Plasticity Index:	52.2%
Classification:	CH

Natural Water Content: 29.7%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	RG
Date:	31-Jan-2011

Sample: FGL-66 at 46.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: N/A

Plastic Limit

Tare #			
Tare Wt, g			
Wet + Tare, g			
Dry + Tare, g			average
M%			

Liquid Limit (method A)

# of Blows				
Tare #				
Tare Wt, g				
Wet + tare, g				
Dry + tare, g				
Water content				

Non-Plastic

SUMMARY

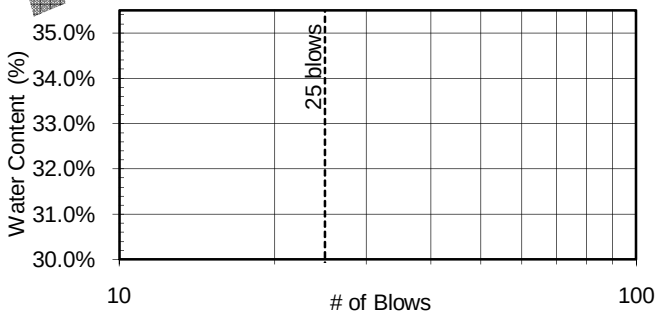
Plastic Limit:

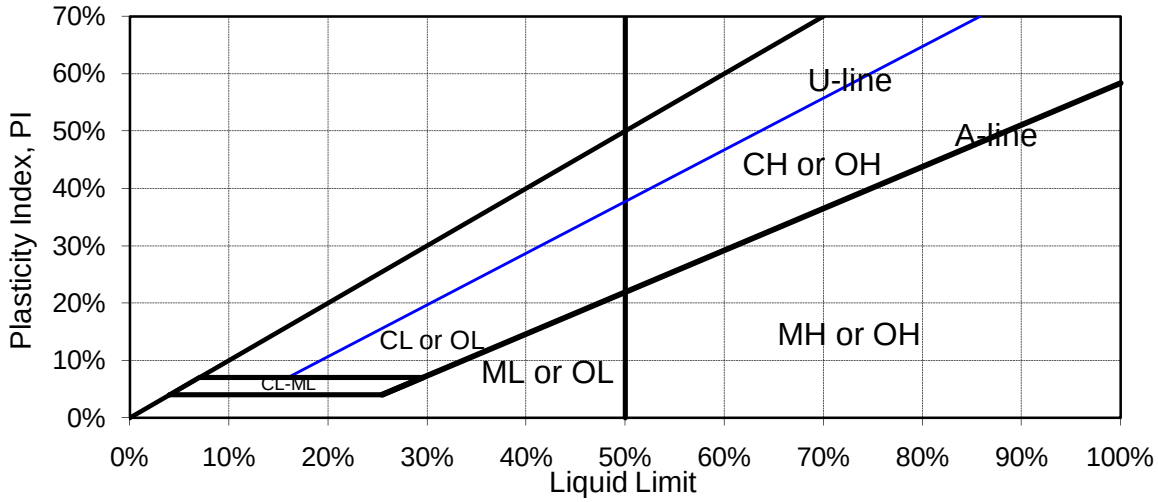
Liquid Limit:

Plasticity Index:

Classification:

Natural Water Content: 52.4%





Comments: Too silty too roll plastic

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	DCH
Date:	16-Feb-2011

Sample: FGL-67B at 48.5' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	1J	54A	
Tare Wt, g	14.65	14.53	
Wet + Tare, g	19.09	18.90	
Dry + Tare, g	18.07	17.89	average
M%	29.8%	30.1%	29.9%

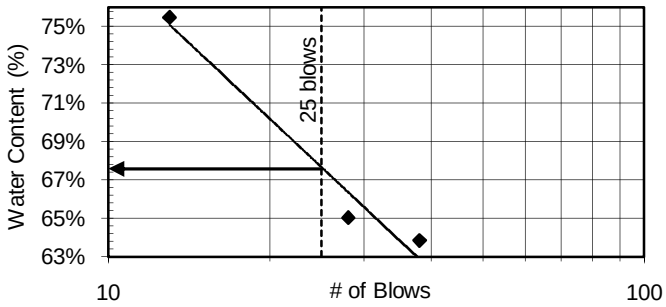
Liquid Limit (method A)

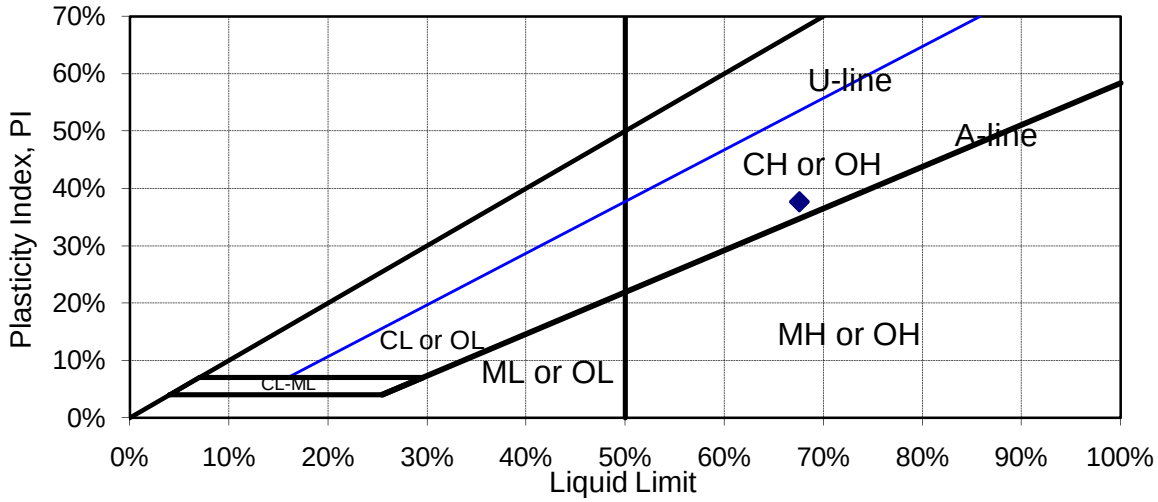
# of Blows	13	28	38
Tare #	B2	38A	A17
Tare Wt, g	14.24	14.57	12.98
Wet + tare, g	19.82	19.95	18.60
Dry + tare, g	17.42	17.83	16.41
Water content	75.5%	65.0%	63.8%

SUMMARY

Plastic Limit:	29.9%
Liquid Limit:	67.6%
Plasticity Index:	37.7%
Classification:	CH

Natural Water Content: 28.6%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	DCH
Date:	16-Feb-2011

Sample: FGL-70A at 62' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	40A	37A	
Tare Wt, g	14.4	14.10	
Wet + Tare, g	19.84	19.63	
Dry + Tare, g	18.42	18.20	average
M%	35.3%	34.9%	35.1%

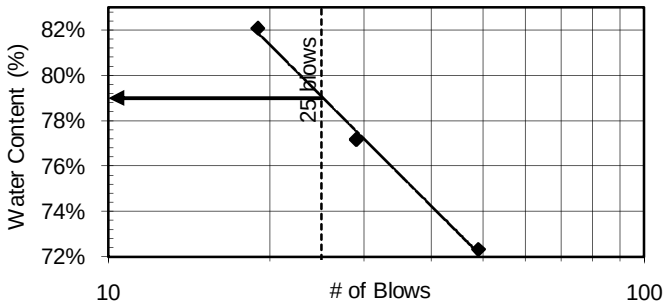
Liquid Limit (method A)

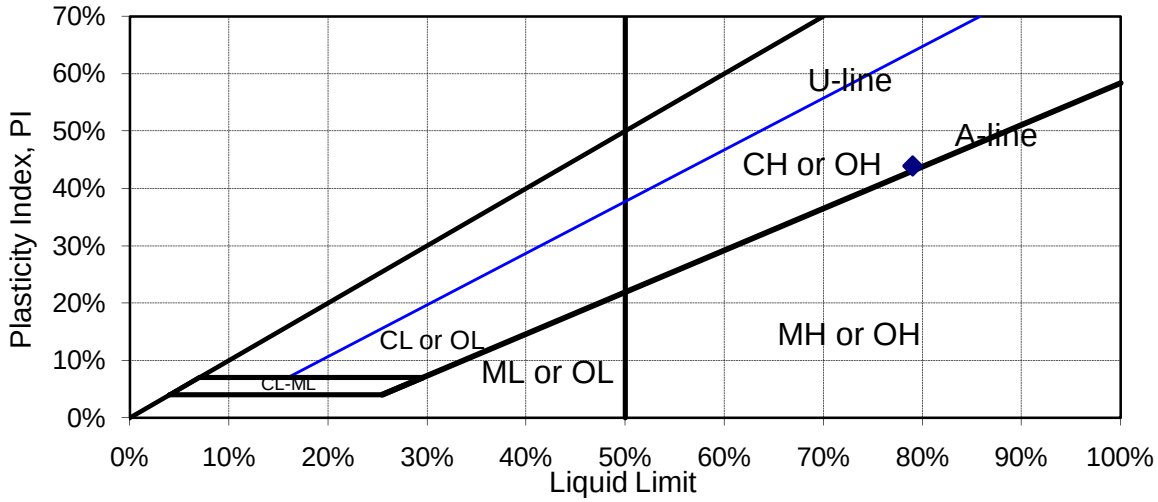
# of Blows	19	29	49
Tare #	A31	9A	PPE
Tare Wt, g	13.73	14.32	14.22
Wet + tare, g	19.52	21.46	19.70
Dry + tare, g	16.91	18.35	17.40
Water content	82.1%	77.2%	72.3%

SUMMARY

Plastic Limit:	35.1%
Liquid Limit:	79.0%
Plasticity Index:	43.9%
Classification:	CH

Natural Water Content: 32.9%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	DCH
Date:	16-Feb-2011

Sample: FGL-73 (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: N/A

Plastic Limit

Tare #			
Tare Wt, g			
Wet + Tare, g			
Dry + Tare, g			average
M%			

Liquid Limit (method A)

# of Blows				
Tare #				
Tare Wt, g				
Wet + tare, g				
Dry + tare, g				
Water content				

Non-Plastic

SUMMARY

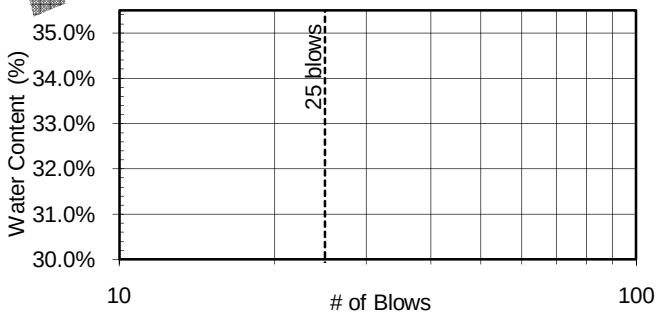
Plastic Limit:

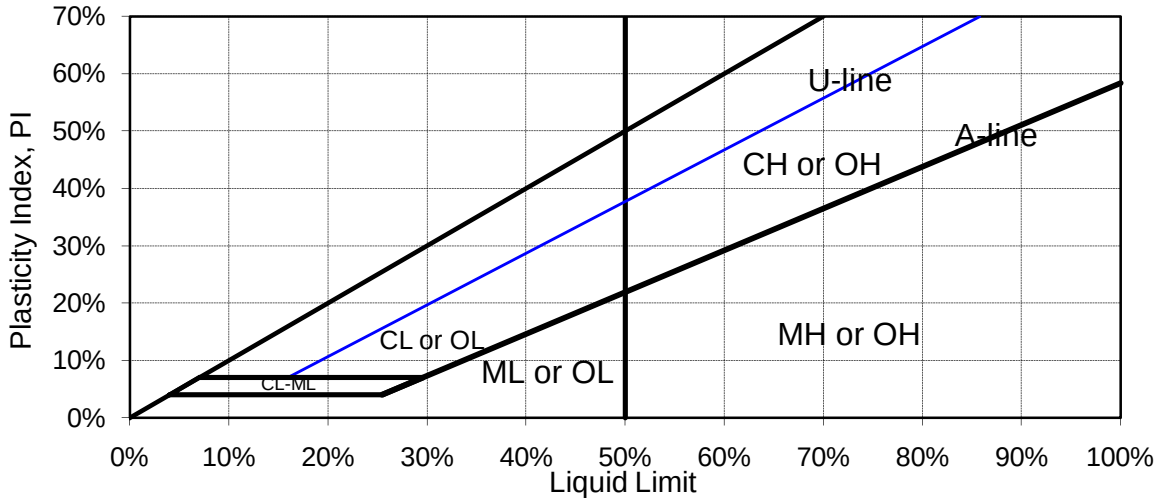
Liquid Limit:

Plasticity Index:

Classification:

Natural Water Content: 66.3%





Comments: Non-Plastic

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

ATTERBERG LIMITS TEST REPORT

(Test Reference: ASTM D 4318)

MDH ENGINEERED SOLUTIONS

Client:	Northwest Hydraulic Consultants
Project	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488-03
Technician:	DCH
Date:	16-Feb-2011

Sample: FGL-75 at 73' (air-dried)

Percentage of sample retained on 425-um (No. 40) sieve: NA

Plastic Limit

Tare #	A39	A47	
Tare Wt, g	13.78	13.66	
Wet + Tare, g	21.64	20.79	
Dry + Tare, g	20.04	19.43	average
M%	25.6%	23.6%	24.6%

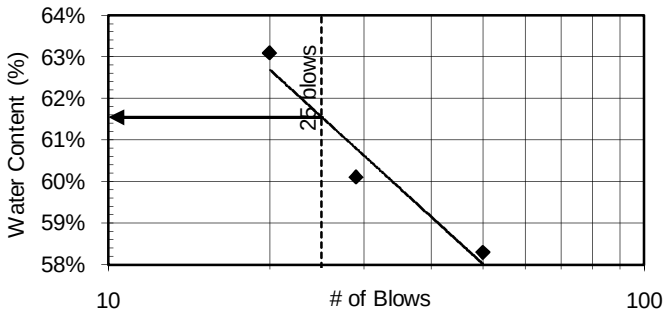
Liquid Limit (method A)

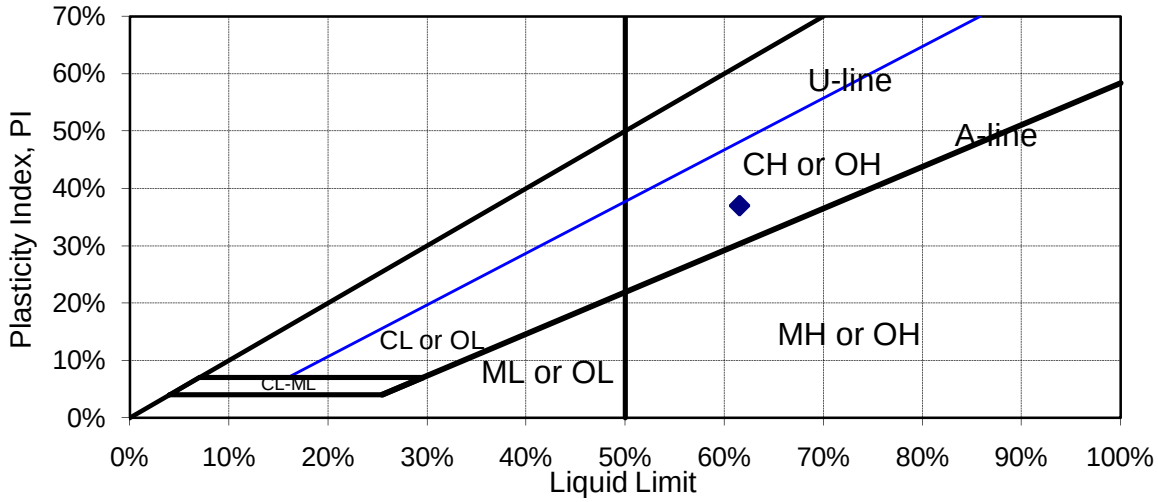
# of Blows	20	29	50
Tare #	2S	PA	10A
Tare Wt, g	14.12	14.12	14.36
Wet + tare, g	19.91	20.06	21.04
Dry + tare, g	17.67	17.83	18.58
Water content	63.1%	60.1%	58.3%

SUMMARY

Plastic Limit:	24.6%
Liquid Limit:	61.6%
Plasticity Index:	37.0%
Classification:	CH

Natural Water Content: 31.6%





Comments: -

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

Moisture Content Laboratory Reports

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech RG

Date: 27-Jan-11

Sample #	FGL-01	FGL-02	FGL-03	FGL-04	FGL-05	FGL-06
Test Hole #	01	01	01	01	01	01
Depth	0.75-1.17'	1.58-2.42'	2.92-3.58'	7.5'	10'	10-11.5'
Tare #	CC23	AA08	CC36	CC9	BB08	ZZ7
Tare Mass (g)	37.67	35.96	37.87	37.82	37.78	37.61
Wet sample + tare (g)	176.46	158.12	170.72	157.13	148.15	175.30
Dry sample + tare (g)	157.38	140.49	154.16	139.51	128.03	151.02
Wt. Dry sample (g)	119.71	104.53	116.29	101.69	90.25	113.41
Water Content (%)	15.94	16.87	14.24	17.33	22.29	21.41
Sample #	FGL-07	FGL-08	FGL-09	FGL-10	FGL-11	FGL-12
Test Hole #	01	01	01	01	01	01
Depth	12.5'	15-15.67'	16.58-16.92'	20-21.5'	22.58'-23.17'	24.5-25.5'
Tare #	BB14	AA18	PP8	BB31	PP32	CC27
Tare Mass (g)	37.68	37.49	88.03	37.95	37.81	37.77
Wet sample + tare (g)	159.12	185.57	142.15	136.66	139.65	190.09
Dry sample + tare (g)	138.05	164.01	129.49	120.59	118.43	161.76
Wt. Dry sample (g)	100.37	126.52	41.46	82.64	80.62	123.99
Water Content (%)	20.99	17.04	30.54	19.45	26.32	22.85
Sample #	FGL-13	FGL-14	FGL-15	FGL-16	FGL-17	FGL-18
Test Hole #	01	01	01	01	01	01
Depth	30'	30-31.5'	33.33-33.92'	34.1-35'	35-36'	40-41.5'
Tare #	PP37	AA15	BB15	CC28	BB18	C02
Tare Mass (g)	37.72	37.97	37.88	37.61	37.63	37.71
Wet sample + tare (g)	205.34	156.25	141.84	152.56	164.09	115.29
Dry sample + tare (g)	175.04	130.38	112.25	122.71	130.56	94.83
Wt. Dry sample (g)	137.32	92.41	74.37	85.1	92.93	57.12
Water Content (%)	22.07	27.99	39.79	35.08	36.08	35.82

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech RG

Date: 27-Jan-11

Sample #	FGL-19A	FGL-20A	FGL-21	FGL-22A	FGL-22B	FGL-23
Test Hole #	01	01	01	01	01	01
Depth	42.5'	45.5'	50-51'	50-55'	50-55'	57.5'
Tare #	PP21	BB40	CC37	PP20	CC14	BB26
Tare Mass (g)	38.01	37.63	37.72	37.52	37.69	37.57
Wet sample + tare (g)	192.82	157.60	137.56	129.42	148.95	179.73
Dry sample + tare (g)	154.33	130.41	103.68	103.1	107.76	144.34
Wt. Dry sample (g)	116.32	92.78	65.96	65.58	70.07	106.77
Water Content (%)	33.09	29.31	51.36	40.13	58.78	33.15
Sample #	FGL-24	FGL-25				
Test Hole #	01	01				
Depth	61'	67.5'				
Tare #	BB41	BB03				
Tare Mass (g)	37.99	37.64				
Wet sample + tare (g)	171.86	182.08				
Dry sample + tare (g)	131.79	140.63				
Wt. Dry sample (g)	93.8	102.99				
Water Content (%)	42.72	40.25				
Sample #						
Test Hole #						
Depth						
Tare #						
Tare Mass (g)						
Wet sample + tare (g)						
Dry sample + tare (g)						
Wt. Dry sample (g)						
Water Content (%)						

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech RG

Date: 27-Jan-11

Sample #	FGL-26	FGL-27	FGL-28	FGL-29	FGL-30	FGL-31
Test Hole #	02	02	02	02	02	02
Depth	4-5'	5-6.5'	9-10'	14-15'	15-16.5'	19-20'
Tare #	XBW	R7A	CC8	BB3	PP36	CC50
Tare Mass (g)	62.02	68.44	37.71	37.54	37.97	37.87
Wet sample + tare (g)	169.95	135.05	136.96	99.44	138.76	126.07
Dry sample + tare (g)	153.4	122.35	119.54	89.75	120.11	105.36
Wt. Dry sample (g)	91.38	53.91	81.83	52.21	82.14	67.49
Water Content (%)	18.11	23.56	21.29	18.56	22.71	30.69
Sample #	FGL-32A	FGL-32B	FGL-33	FGL-34	FGL-35	FGL-38
Test Hole #	02	02	02	02	02	02
Depth	20-22.5'	22.5-25'	25-26.5'	29-30'	33-35'	38.5-39.5'
Tare #	PP11	AA14	BB28	BB23	PP26	DD4
Tare Mass (g)	37.84	35.44	38.07	38.07	37.54	37.69
Wet sample + tare (g)	186.21	162.73	118.25	157.35	88.25	130.55
Dry sample + tare (g)	152.61	131.24	104.18	128.05	76.41	98.18
Wt. Dry sample (g)	114.77	95.8	66.11	89.98	38.87	60.49
Water Content (%)	29.28	32.87	21.28	32.56	30.46	53.51
Sample #	FGL-39A	FGL-39B	FGL-40	FGL-41A	FGL-42A	FGL-43
Test Hole #	02	02	02	02	02	02
Depth	40-41'	43'	45-46.5'	48'	53'	54-55'
Tare #	BB12	BB35	BB01	CC20	PP3	PP5
Tare Mass (g)	38.11	37.87	37.88	37.74	37.73	37.58
Wet sample + tare (g)	162.02	198.05	125.2	183.38	132.04	138.88
Dry sample + tare (g)	116.66	151.46	98.65	144.11	107.78	101.87
Wt. Dry sample (g)	78.55	113.59	60.77	106.37	70.05	64.29
Water Content (%)	57.75	41.02	43.69	36.92	34.63	57.57

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech RG

Date: 27-Jan-11

Sample #	FGL-44	FGL-45	FGL-46	FGL-47	FGL-48	FGL-49
Test Hole #	02	02	02	02	02	02
Depth	55-56'	58'	61'	65'	65-66.5'	69.5'
Tare #	BB06	CC26	BB25	BB4	AA19	BB30
Tare Mass (g)	37.82	37.38	37.72	37.50	37.97	37.97
Wet sample + tare (g)	198.65	181.63	161.26	183.61	132.30	172.30
Dry sample + tare (g)	158.03	146.46	130.52	146.5	106.27	139.59
Wt. Dry sample (g)	120.21	109.08	92.8	109	68.3	101.62
Water Content (%)	33.79	32.24	33.13	34.05	38.11	32.19
Sample #	FGL-50					
Test Hole #	02					
Depth	72'					
Tare #	AA06					
Tare Mass (g)	36.08					
Wet sample + tare (g)	157.11					
Dry sample + tare (g)	127.22					
Wt. Dry sample (g)	91.14					
Water Content (%)	32.80					
Sample #						
Test Hole #						
Depth						
Tare #						
Tare Mass (g)						
Wet sample + tare (g)						
Dry sample + tare (g)						
Wt. Dry sample (g)						
Water Content (%)						

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech SC

Date: 27-Jan-11

Sample #	FGL-51	FGL-52	FGL-53	FGL-54A	FGL-54B	FGL-54C
Test Hole #	03	03	03	03	03	03
Depth	5-6'	10-11.5'	15'	17.4'	19'	19.5'
Tare #	BB06	CC28	PP5	BB12	PP32	AA08
Tare Mass (g)	37.83	37.61	37.59	38.11	37.82	35.91
Wet sample + tare (g)	120.61	84.03	121.28	160.79	146.97	177.77
Dry sample + tare (g)	102.66	74.15	99.96	130.83	119.95	143.03
Wt. Dry sample (g)	64.83	36.54	62.37	92.72	82.13	107.12
Water Content (%)	27.69	27.04	34.18	32.31	32.90	32.43
Sample #	FGL-55	FGL-56A	FGL-56B	FGL-57	FGL-58	FGL-59
Test Hole #	03	03	03	03	03	03
Depth		22'	23'9"	24-25'	29-30'	30-31.5'
Tare #	PP8	BB15	BB8	CC9	BB14	CC27
Tare Mass (g)	38.03	37.88	37.81	37.82	37.68	37.77
Wet sample + tare (g)	132.67	101.09	137.85	122.66	134.74	149.04
Dry sample + tare (g)	108.04	84.1	112.24	102.14	110.03	125.73
Wt. Dry sample (g)	70.01	46.22	74.43	64.32	72.35	87.96
Water Content (%)	35.18	36.76	34.41	31.90	34.15	26.50
Sample #	FGL-60	FGL-61A	FGL-61B	FGL-62A	FGL-62B	FGL-63
Test Hole #	03	03	03	03	03	03
Depth	31.5-32'			38.5'	40'	40-41.5'
Tare #	PP3	BB31	CC36	PP37	AA15	BB18
Tare Mass (g)	37.75	37.96	37.89	37.68	37.94	37.62
Wet sample + tare (g)	153.48	169.45	135.81	135.81	158.05	168.21
Dry sample + tare (g)	123.07	128.73	110.76	110.76	130.55	137.34
Wt. Dry sample (g)	85.32	90.77	72.87	73.08	92.61	99.72
Water Content (%)	35.64	44.86	34.38	34.28	29.69	30.96

Comments: No depths listed on lab request form for FGL-55, 61A and 61B

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech RG

Date: 27-Jan-11

Sample #	FGL-64	FGL-66	FGL-67A	FGL-68	FGL-69A	FGL-70A
Test Hole #	03	03	03	03	03	03
Depth	41.5-44'	46.5'	48.5'	50-51.5'	51.5'	62'
Tare #	CC41	PP4	BB23	ZZ7	PP26	AA14
Tare Mass (g)	37.81	37.70	38.07	37.65	37.55	35.44
Wet sample + tare (g)	102.32	94.10	106.16	150.36	180.75	176.71
Dry sample + tare (g)	84.51	74.72	91.02	125.45	145.82	141.77
Wt. Dry sample (g)	46.7	37.02	52.95	87.8	108.27	106.33
Water Content (%)	38.14	52.35	28.59	28.37	32.26	32.86
Sample #	FGL-72	FGL-73	FGL-74	FGL-75	FGL-76A	
Test Hole #	03	03	03	03	03	
Depth	62.5'		68'	73'	76.5'	
Tare #	BB01	BB28	AA18	CC20	BB35	
Tare Mass (g)	37.89	38.07	37.48	37.76	37.85	
Wet sample + tare (g)	172.86	119.04	123.21	182.29	90.43	
Dry sample + tare (g)	133.76	86.76	101.59	147.58	78.57	
Wt. Dry sample (g)	95.87	48.69	64.11	109.82	40.72	
Water Content (%)	40.78	66.30	33.72	31.61	29.13	
Sample #						
Test Hole #						
Depth						
Tare #						
Tare Mass (g)						
Wet sample + tare (g)						
Dry sample + tare (g)						
Wt. Dry sample (g)						
Water Content (%)						

Comments: No depth listed for FGL-73 on lab request sheet

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech SC

Date: 27-Jan-11

Sample #	FGL-77	FGL-78	FGL-79	FGL-80	FGL-81	FGL-82
Test Hole #	04	04	04	04	04	04
Depth	1.5-2.6'	10'	10-11.5'	12.5-13.5'	13.5-16.5'	17.5-18.5'
Tare #	PP24	BB7	CC25	CC39	PP28	H1
Tare Mass (g)	37.83	37.75	37.29	37.53	37.73	37.76
Wet sample + tare (g)	151.27	152.19	97.07	201.92	142.87	85.16
Dry sample + tare (g)	120.35	123.11	81.43	156.34	103.87	68.22
Wt. Dry sample (g)	82.52	85.36	44.14	118.81	66.14	30.46
Water Content (%)	37.47	34.07	35.43	38.36	58.97	55.61
Sample #	FGL-83	FGL-84	FGL-85A	FGL-86	FGL-87A	FGL-88
Test Hole #	04	04	04	04	04	04
Depth	18.8'	20-21.5'	21.5-25'	25.5-28.5'	29'	30-31'
Tare #	H8R	AA17	5S5	BB11	DD8	PP29
Tare Mass (g)	37.78	35.59	37.78	37.85	37.94	37.74
Wet sample + tare (g)	83.82	110.88	177.57	191.99	115.77	92.89
Dry sample + tare (g)	68.56	90.62	141.63	147.42	93.32	75.66
Wt. Dry sample (g)	30.78	55.03	103.85	109.57	55.38	37.92
Water Content (%)	49.58	36.82	34.61	40.68	40.54	45.44
Sample #	FGL-89					
Test Hole #	04					
Depth	32.5'					
Tare #	PP14					
Tare Mass (g)	37.80					
Wet sample + tare (g)	78.61					
Dry sample + tare (g)	67.67					
Wt. Dry sample (g)	29.87					
Water Content (%)	36.63					

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS

Client: NorthWest Hydraulic Consultants

Project: Spillway Upgrade at Highfield Dam

MDH # R2488

Tech SC

Date: 27-Jan-11

Sample #	FGL-90	FGL-91	FGL-92	FGL-93	FGL-94	FGL-96A
Test Hole #	05	05	05	05	05	05
Depth	3'	5'	5-6.5'	10'	14-15'	16'
Tare #	AA13	PP35	PP10	ZZ3	PP6	BB10
Tare Mass (g)	35.43	37.90	37.66	37.45	37.76	37.57
Wet sample + tare (g)	132.12	99.94	94.03	139.50	100.60	180.10
Dry sample + tare (g)	110.28	86.57	82.14	118.25	89.16	149.1
Wt. Dry sample (g)	74.85	48.67	44.48	80.8	51.4	111.53
Water Content (%)	29.18	27.47	26.73	26.30	22.26	27.80

Sample #	FGL-96B	FGL-97	FGL-98	FGL-99	FGL-100	FGL-101
Test Hole #	05	05	05	05	05	05
Depth	19'	24-25'	25-26'	27-28'	29.5'	30-31.5'
Tare #	BB32	BB6	CC43	BB34	PP12	AA23
Tare Mass (g)	37.96	37.74	37.42	38.04	38.04	38.36
Wet sample + tare (g)	184.21	116.4	115.84	93.15	161.84	105.11
Dry sample + tare (g)	150.18	98.4	96.28	79.03	133.14	89.42
Wt. Dry sample (g)	112.22	60.66	58.86	40.99	95.1	51.06
Water Content (%)	30.32	29.67	33.23	34.45	30.18	30.73

Sample #	FGL-102	FGL-103				
Test Hole #	05	05				
Depth	34'	35-36.5'				
Tare #	BB29	PP7				
Tare Mass (g)	37.75	37.64				
Wet sample + tare (g)	150.25	144.02				
Dry sample + tare (g)	123.53	119.33				
Wt. Dry sample (g)	85.78	81.69				
Water Content (%)	31.15	30.22				

Comments:

Moisture Content & Bulk Density Laboratory Reports

MDH ENGINEERED SOLUTIONS

WATER CONTENTS & BULK DENSITIES

Client:	NorthWest Hydraulic Consultants
Project:	Spillway Upgrade at Highfield Dam
MDH #	R2488-04
Tech:	TW
Date:	24-Feb-11

Sample #	FGL-39A	FGL-38				
Tare #	BB12	DD4				
Tare Mass (g)	38.11	37.69				
Wet sample + tare (g)	162.02	130.55				
Dry sample + tare (g)	116.66	98.18				
Wt. Dry sample (g)	78.55	60.49				
Water Content (%)	57.75	53.51				
Bulk Density						
Sample #	FGL-39A	FGL-38				
Mass of sample in air (g):	99.57	157.61				
Mass of sample + wax in air (g):	104.29	165.20				
Mass of sample + wax in water (g):	41.71	61.12				
Wet density (kg/m^3):	1738	1649				
Dry density (kg/m^3):	1102	1074				
Sample #						
Tare #						
Tare Mass (g)						
Wet sample + tare (g)						
Dry sample + tare (g)						
Wt. Dry sample (g)						
Water Content (%)						
Bulk Density						
Sample #						
Mass of sample in air (g):						
Mass of sample + wax in air (g):						
Mass of sample + wax in water (g):						
Wet density (kg/m^3):						
Dry density (kg/m^3):						

Comments: _____

MDH ENGINEERED SOLUTIONS

WATER CONTENTS & BULK DENSITIES

Client:	NorthWest Hydraulic Consultants
Project:	Spillway Upgrade at Highfield Dam
MDH #	R2488-03
Tech:	SC
Date:	28-Jan-11

Sample #	FGL-54A	FGL-54B	FGL-56B	FGL-57	FGL-58	FGL-60
Tare #	BB12	PP32	BB8	CC9	BB14	PP3
Tare Mass (g)	38.11	37.82	37.81	37.82	37.68	37.75
Wet sample + tare (g)	160.79	146.97	137.85	122.66	134.74	153.48
Dry sample + tare (g)	130.83	119.95	112.24	102.14	110.03	123.07
Wt. Dry sample (g)	92.72	82.13	74.43	64.32	72.35	85.32
Water Content (%)	32.31	32.90	34.41	31.90	34.15	35.64
Bulk Density						
Sample #	FGL-54A	FGL-54B	FGL-56B	FGL-57	FGL-58	FGL-60
Mass of sample in air (g):	183.61	180.80	122.38	170.08	181.34	145.49
Mass of sample + wax in air (g):	193.94	194.09	131.61	178.80	190.95	154.00
Mass of sample + wax in water (g):	84.31	83.85	54.15	78.33	80.89	63.20
Wet density (kg/m^3):	1873	1897	1824	1876	1827	1791
Dry density (kg/m^3):	1416	1427	1357	1422	1362	1320

Sample #	FGL-61A	FGL-62A	FGL-62B	FGL-64	FGL-67A	FGL69A
Tare #	BB31	PP37	AA15	CC41	BB28	PP26
Tare Mass (g)	37.96	37.68	37.94	37.81	38.07	37.55
Wet sample + tare (g)	169.45	135.81	158.05	102.32	106.16	180.75
Dry sample + tare (g)	128.73	110.76	130.55	84.51	91.02	145.82
Wt. Dry sample (g)	90.77	73.08	92.61	46.7	52.95	108.27
Water Content (%)	44.86	34.28	29.69	38.14	28.59	32.26
Bulk Density						
Sample #	FGL-61A	FGL-62A	FGL-62B	FGL-64	FGL-67A	FGL69A
Mass of sample in air (g):	132.59	159.86	170.05	116.12	156.86	121.46
Mass of sample + wax in air (g):	138.64	171.21	179.49	124.90	164.39	128.63
Mass of sample + wax in water (g):	55.28	75.38	82.10	49.34	75.26	55.90
Wet density (kg/m^3):	1732	1924	1959	1768	1944	1878
Dry density (kg/m^3):	1195	1433	1511	1280	1512	1420

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS & BULK DENSITIES

Client:	NorthWest Hydraulic Consultants
Project:	Spillway Upgrade at Highfield Dam
MDH #	R2488-03
Tech:	SC
Date:	28-Jan-11

Sample #	FGL-70A					
Tare #	AA14					
Tare Mass (g)	35.44					
Wet sample + tare (g)	176.71					
Dry sample + tare (g)	141.77					
Wt. Dry sample (g)	106.33					
Water Content (%)	32.86					
Bulk Density						
Sample #	FGL-54A					
Mass of sample in air (g):	168.86					
Mass of sample + wax in air (g):	178.06					
Mass of sample + wax in water (g):	76.18					
Wet density (kg/m^3):	1845					
Dry density (kg/m^3):	1388					
Sample #						
Tare #						
Tare Mass (g)						
Wet sample + tare (g)						
Dry sample + tare (g)						
Wt. Dry sample (g)						
Water Content (%)						
Bulk Density						
Sample #						
Mass of sample in air (g):						
Mass of sample + wax in air (g):						
Mass of sample + wax in water (g):						
Wet density (kg/m^3):						
Dry density (kg/m^3):						

Comments: _____

MDH ENGINEERED SOLUTIONS

WATER CONTENTS & BULK DENSITIES

Client: NorthWest Hydraulic Consultants
Project: Spillway Upgrade at Highfield Dam
MDH # R2488-04
Tech: CG
Date: 1-Feb-11

Sample #	FGL-77	FGL-78	FGL-80	FGL-82	FGL-85A	FGL-86
Tare #	PP24	BB7	CC39	H1	5S5	BB11
Tare Mass (g)	37.83	37.75	37.53	37.76	37.78	37.85
Wet sample + tare (g)	151.27	152.19	201.92	85.16	177.57	191.99
Dry sample + tare (g)	120.35	123.11	156.34	68.22	141.63	147.42
Wt. Dry sample (g)	82.52	85.36	118.81	30.46	103.85	109.57
Water Content (%)	37.47	34.07	38.36	55.61	34.61	40.68
Bulk Density						
Sample #	FGL-77	FGL-78	FGL-80	FGL-82	FGL-85A	FGL-86
Mass of sample in air (g):	220.34	184.33	154.36	163.02	140.97	181.84
Mass of sample + wax in air (g):	256.98	215.49	181.34	190.49	164.93	225.79
Mass of sample + wax in water (g):	93.97	79.30	64.75	55.45	60.70	76.15
Wet density (kg/m^3):	1808	1822	1789	1565	1823	1814
Dry density (kg/m^3):	1316	1359	1293	1006	1355	1289

Sample #	FGL-87A	FGL-89				
Tare #	DD8	PP14				
Tare Mass (g)	37.94	37.80				
Wet sample + tare (g)	115.77	78.61				
Dry sample + tare (g)	93.32	67.67				
Wt. Dry sample (g)	55.38	29.87				
Water Content (%)	40.54	36.63				
Bulk Density						
Sample #	FGL-87A	FGL-89				
Mass of sample in air (g):	132.61	137.18				
Mass of sample + wax in air (g):	157.27	167.28				
Mass of sample + wax in water (g):	53.73	57.33				
Wet density (kg/m^3):	1749	1802				
Dry density (kg/m^3):	1244	1319				

Comments:

MDH ENGINEERED SOLUTIONS

WATER CONTENTS & BULK DENSITIES

Client:	NorthWest Hydraulic Consultants
Project:	Spillway Upgrade at Highfield Dam
MDH #	R2488-05
Tech:	CG
Date:	1-Feb-11

Sample #	FGL-90	FGL-91	FGL-93	FGL-96A	FGL-96B	FGL-99
Tare #	AA13	PP35	ZZ3	BB10	BB32	BB34
Tare Mass (g)	35.43	37.90	37.45	37.57	37.96	38.04
Wet sample + tare (g)	132.12	99.94	139.50	180.10	184.21	93.15
Dry sample + tare (g)	110.28	86.57	118.25	149.10	150.18	79.03
Wt. Dry sample (g)	74.85	48.67	80.8	111.53	112.22	40.99
Water Content (%)	29.18	27.47	26.30	27.80	30.32	34.45
Bulk Density						
Sample #	FGL-90	FGL-91	FGL-93	FGL-96A	FGL-96B	FGL-99
Mass of sample in air (g):	253.55	284.26	599.19	121.74	176.64	134.99
Mass of sample + wax in air (g):	283.05	315.18	665.99	144.14	219.66	175.48
Mass of sample + wax in water (g):	114.70	129.86	269.41	50.80	74.64	54.79
Wet density (kg/m^3):	1875	1888	1864	1786	1827	1795
Dry density (kg/m^3):	1452	1481	1476	1397	1402	1335
Sample #	FGL-100	FGL-101	FGL-102			
Tare #	PP12	AA23	BB29			
Tare Mass (g)	38.04	38.36	37.75			
Wet sample + tare (g)	161.84	105.11	150.25			
Dry sample + tare (g)	133.14	89.42	123.53			
Wt. Dry sample (g)	95.1	51.06	85.78			
Water Content (%)	30.18	30.73	31.15			
Bulk Density						
Sample #	FGL-100	FGL-101	FGL-102			
Mass of sample in air (g):	127.33	135.87	137.20			
Mass of sample + wax in air (g):	153.29	161.25	158.31			
Mass of sample + wax in water (g):	53.30	60.26	62.25			
Wet density (kg/m^3):	1798	1875	1897			
Dry density (kg/m^3):	1381	1434	1446			

Comments:

Hydrometer Laboratory Reports

PARTICLE-SIZE ANALYSIS REPORT

(Test Reference: ASTM D 422)

Sieve Analysis Sieve	Diameter (mm)	% Finer
3"	76.2	100
2"	50.8	100
1"	25.4	100
3/4"	19.1	100
3/8"	9.5	100
# 4	4.75	100
# 10	2.00	99
# 20	0.850	96
# 40	0.425	93
# 60	0.250	91
# 100	0.150	80
# 200	0.075	59
Hydrometer Analysis	0.0635	54.5
	0.0460	47.9
Dispersing agent:	0.0329	44.8
Sodium Hexametaphosphate	0.0234	43.0
	0.0167	39.4
Dosage of dispersing agent:	0.0123	38.6
40 g/L	0.0088	34.2
	0.0041	29.1
	0.0038	28.3
	0.0029	27.3
	0.0023	24.1
	0.0013	21.1

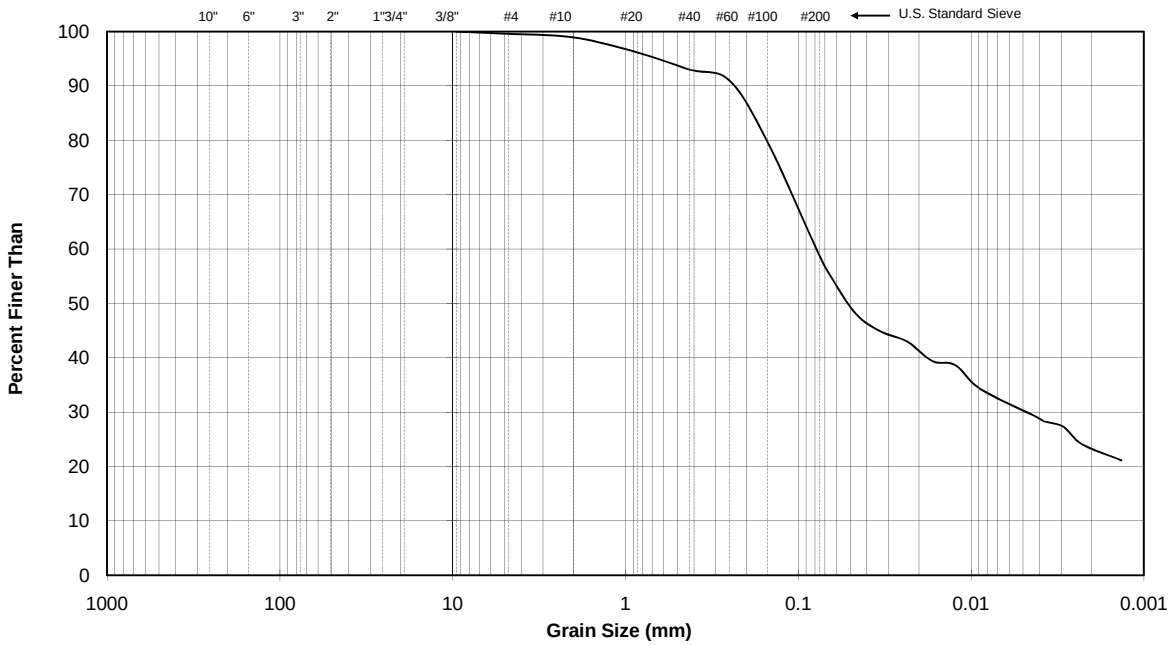
MDH ENGINEERED SOLUTIONS

CLIENT:	Northwest Hydraulic Consultants
PROJECT:	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488
SAMPLE:	FGL-53 @15'
DATE:	4-Feb-11

PARTICLE SIZE DISTRIBUTION SUMMARY

% GRAVEL	0
% SAND	41
% FINES (SILT, CLAY)	59

COMMENTS:



BOULDERS	COBBLES	GRAVEL		SAND			FINES (SILT, CLAY)
		Coarse	Fine	Coarse	Medium	Fine	

Unified Soil Classification System

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

PARTICLE-SIZE ANALYSIS REPORT

(Test Reference: ASTM D 422)

Sieve Analysis	Sieve	Diameter (mm)	% Finer
	3"	76.2	100
	2"	50.8	100
	1"	25.4	100
	3/4"	19.1	100
	3/8"	9.5	100
	# 4	4.75	100
	# 10	2.00	100
	# 20	0.850	100
	# 40	0.425	99
	# 60	0.250	98
	# 100	0.150	83
	# 200	0.075	70
Hydrometer Analysis		0.0623	64.0
		0.0456	55.0
Dispersing agent:		0.0330	49.1
Sodium Hexametaphosphate		0.0236	45.5
		0.0168	44.3
Dosage of dispersing agent:		0.0124	41.2
40 g/L		0.0088	37.6
		0.0063	34.2
		0.0044	30.9
		0.0030	27.2
		0.0022	24.2
		0.0012	20.2

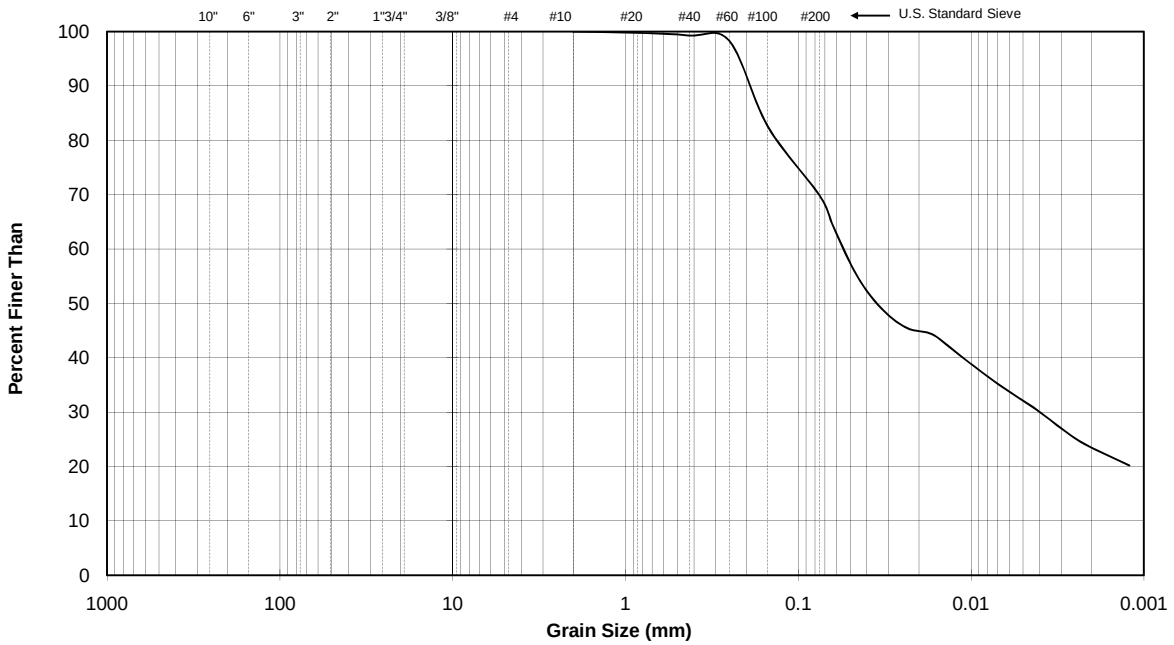
MDH ENGINEERED SOLUTIONS

CLIENT:	Northwest Hydraulic Consultants
PROJECT:	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488
SAMPLE:	FGL-54
DATE:	28-Feb-11

PARTICLE SIZE DISTRIBUTION SUMMARY

% GRAVEL	0
% SAND	30
% FINES (SILT, CLAY)	70

COMMENTS:



BOULDERS	COBBLES	GRAVEL		SAND			FINES (SILT, CLAY)
		Coarse	Fine	Coarse	Medium	Fine	

Unified Soil Classification System

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

PARTICLE-SIZE ANALYSIS REPORT

(Test Reference: ASTM D 422)

Sieve Analysis Sieve	Diameter (mm)	% Finer
3"	76.2	100
2"	50.8	100
1"	25.4	100
3/4"	19.1	100
3/8"	9.5	100
# 4	4.75	100
# 10	2.00	100
# 20	0.850	100
# 40	0.425	99
# 60	0.250	99
# 100	0.150	86
# 200	0.075	77
Hydrometer Analysis	0.0594	71.8
	0.0436	63.3
Dispersing agent:	0.0316	56.6
Sodium Hexametaphosphate	0.0227	52.4
	0.0162	50.4
Dosage of dispersing agent:	0.0120	45.6
40 g/L	0.0084	42.6
	0.0061	38.9
	0.0044	36.2
	0.0031	31.7
	0.0022	27.4
	0.0012	22.8

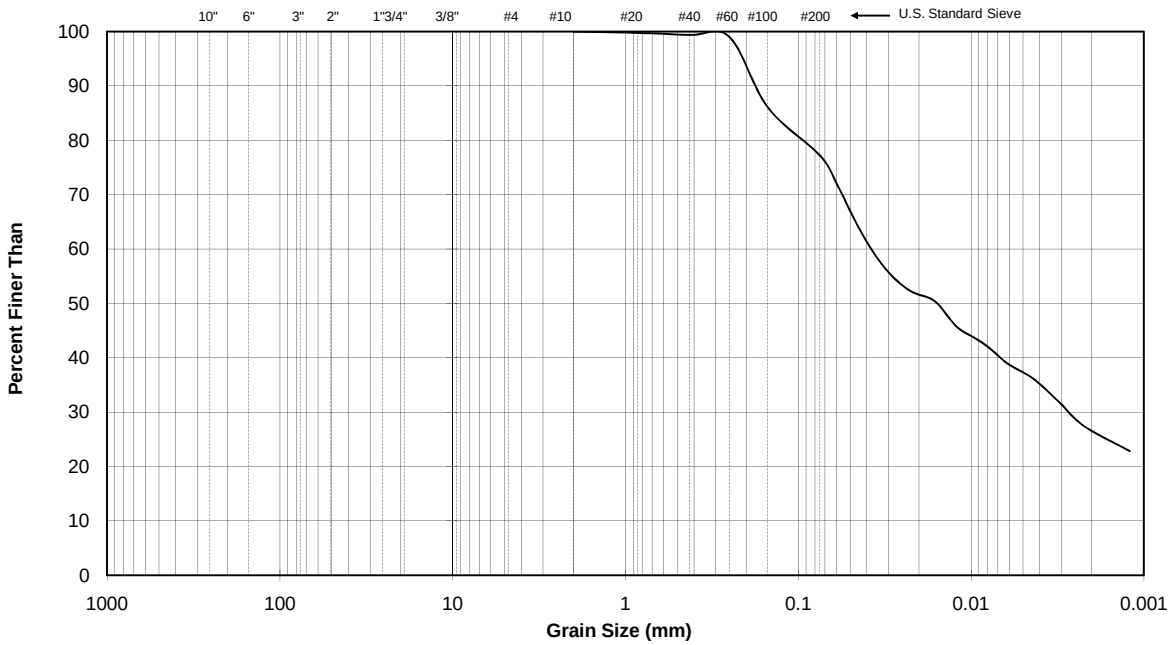
MDH ENGINEERED SOLUTIONS

CLIENT:	Northwest Hydraulic Consultants
PROJECT:	Spillway Upgrade at Highfield Dam
MDH Job No:	R2488
SAMPLE:	FGL-61B
DATE:	28-Feb-11

PARTICLE SIZE DISTRIBUTION SUMMARY

% GRAVEL	0
% SAND	23
% FINES (SILT, CLAY)	77

COMMENTS:



BOULDERS	COBBLES	GRAVEL		SAND			FINES (SILT, CLAY)
		Coarse	Fine	Coarse	Medium	Fine	

Unified Soil Classification System

The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

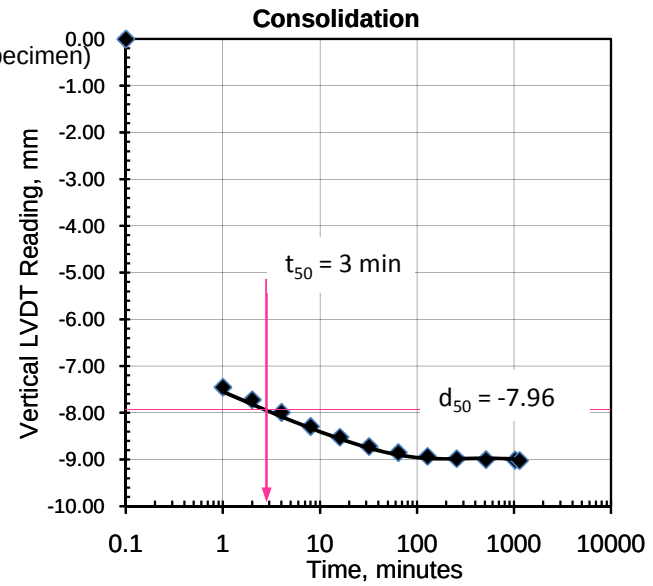
Direct Shear Laboratory Reports

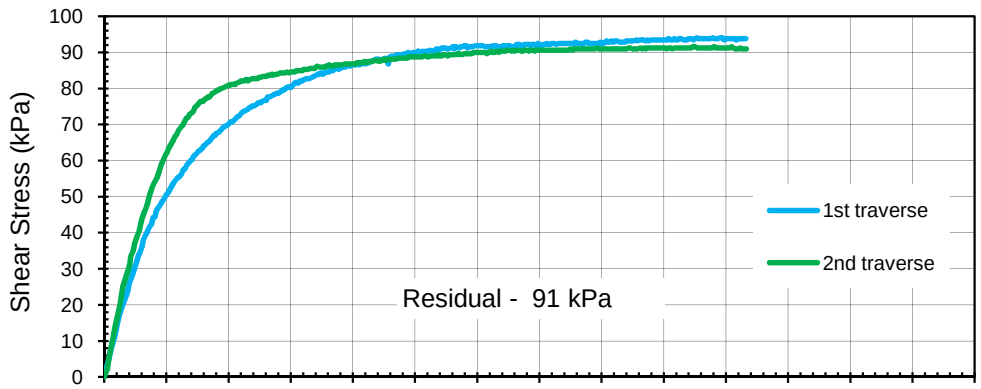
Client:	Northwest Hydraulic Consultant
Project:	Highfield Dam
MDH Job No:	R2488
Date:	March 07/11
Tested by	TH

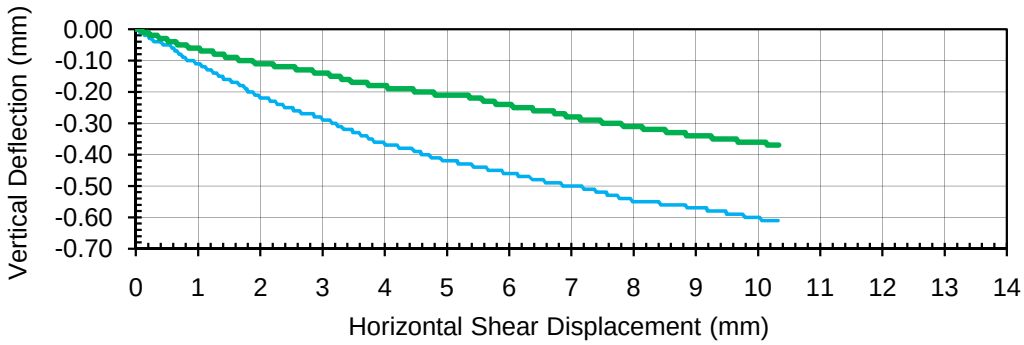
Soil Description	-
Boring #	R2488-03
Sample #	FGL-61B Remolded (3rd test specimen)
Depth of sample	-
Test Equipment	KW 7

	Initial	End of Test
Water Content, (%)	32.2%	38.6%
Diameter (mm)	63.5	63.5
Height (mm)	27.95	17
Wt. of Specimen (g)	105.39	98.93
Wet density (kg/m^3)	1190	1837
Dry density (kg/m^3)	900	1326

Normal Load, (kg)	65.83
Normal Stress, (kPa)	203.8
Shearing Rate, (in/min)	0.0003







The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

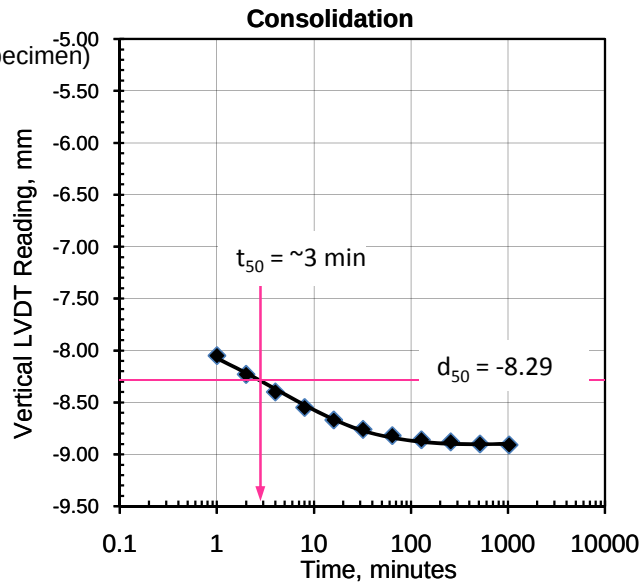
Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

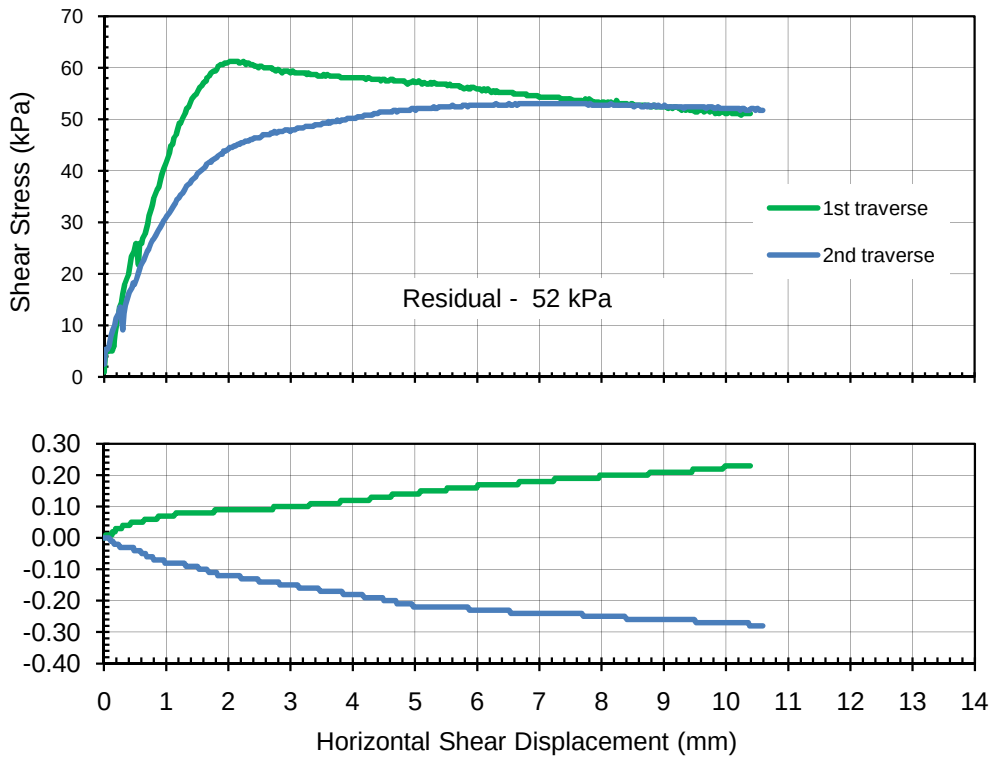
Client:	Northwest Hydraulic Consultant
Project:	Highfield Dam
MDH Job No:	R2488
Date:	March 14/11
Tested by	TH

Soil Description	-
Boring #	R2488-03
Sample #	FGL-61B Remolded (2nd test specimen)
Depth of sample	-
Test Equipment	KW 7

	Initial	End of Test
Water Content, (%)	31.6%	39.1%
Diameter (mm)	63.5	63.5
Height (mm)	27.18	16.04
Wt. of Specimen (g)	97.51	92.53
Wet density (kg/m^3)	1133	1821
Dry density (kg/m^3)	861	1309

Normal Load, (kg)	32.42
Normal Stress, (kPa)	100.4
Shearing Rate, (in/min)	0.0003





The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

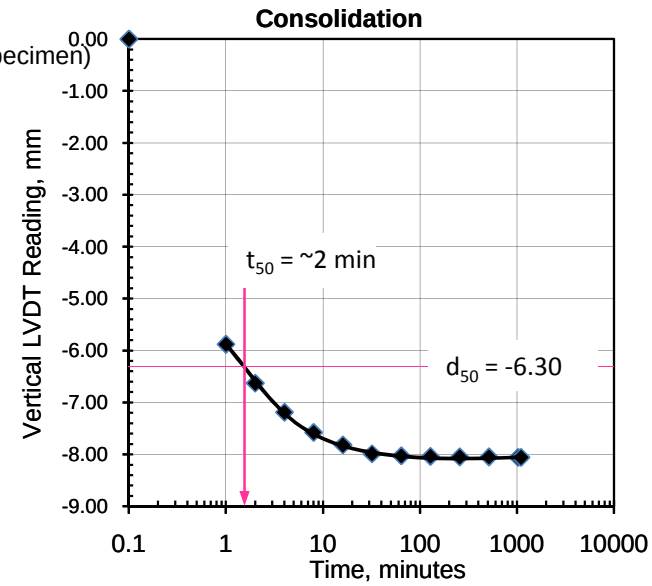
Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.

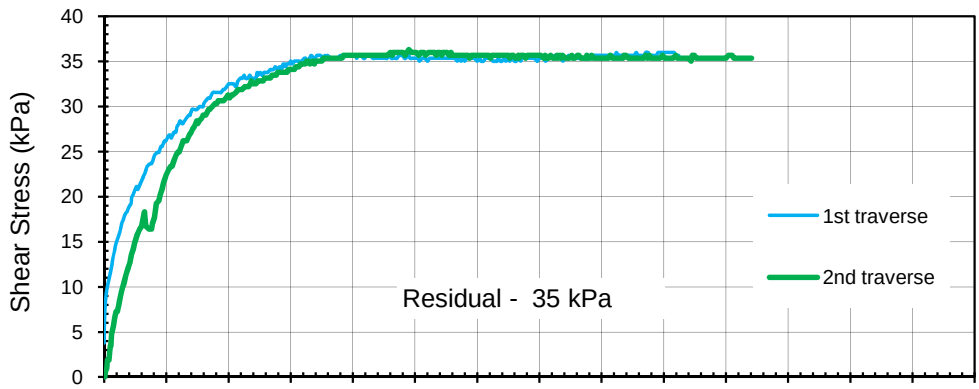
Client:	Northwest Hydraulic Consultant
Project:	Highfield Dam
MDH Job No:	R2488
Date:	March 01/11
Tested by	TH

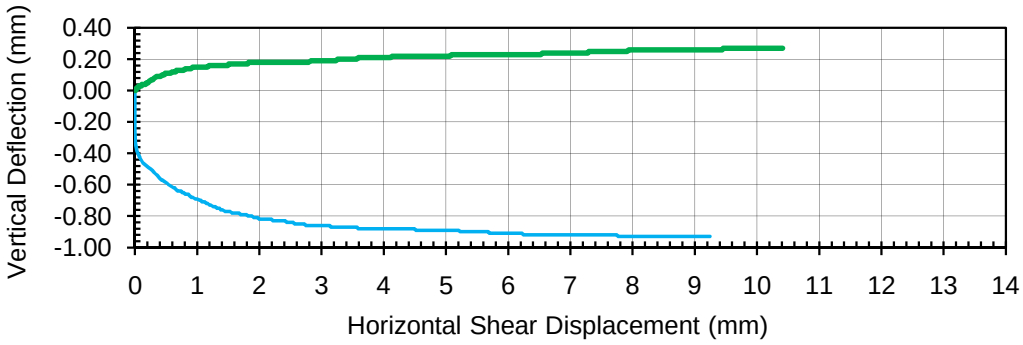
Soil Description	-
Boring #	R2488-03
Sample #	FGL-61B Remolded (1st test specimen)
Depth of sample	-
Test Equipment	KW 7

	Initial	End of Test
Water Content, (%)	30.3%	45.8%
Diameter (mm)	63.5	63.5
Height (mm)	29.21	16.98
Wt. of Specimen (g)	88.85	94.23
Wet density (kg/m^3)	960	1752
Dry density (kg/m^3)	737	1201

Normal Load, (kg)	16.23
Normal Stress, (kPa)	50.2
Shearing Rate, (in/min)	0.0003





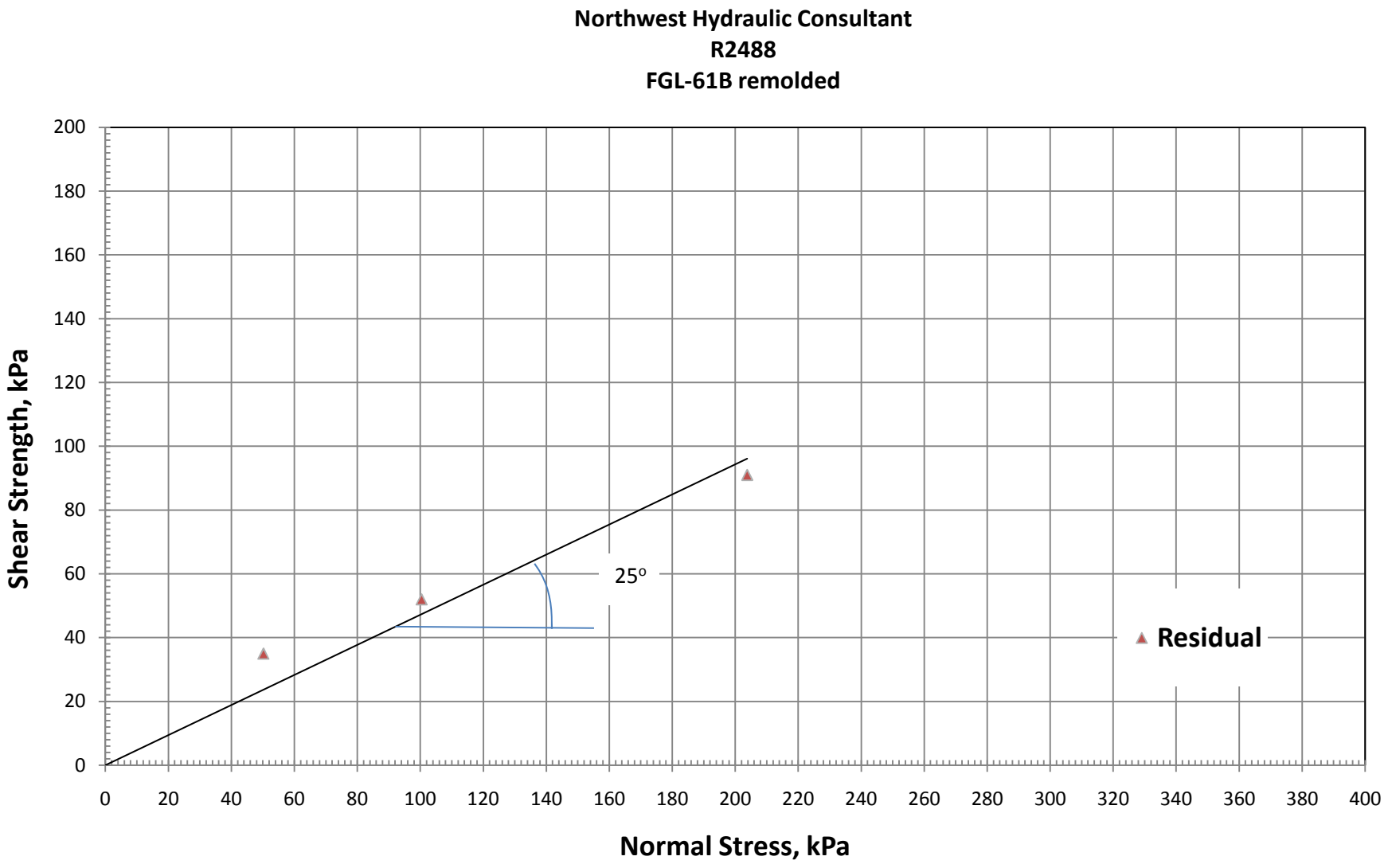


The testing services reported here have been performed in accordance with accepted local industry standards.

The results presented are for the sole use of the designated client only.

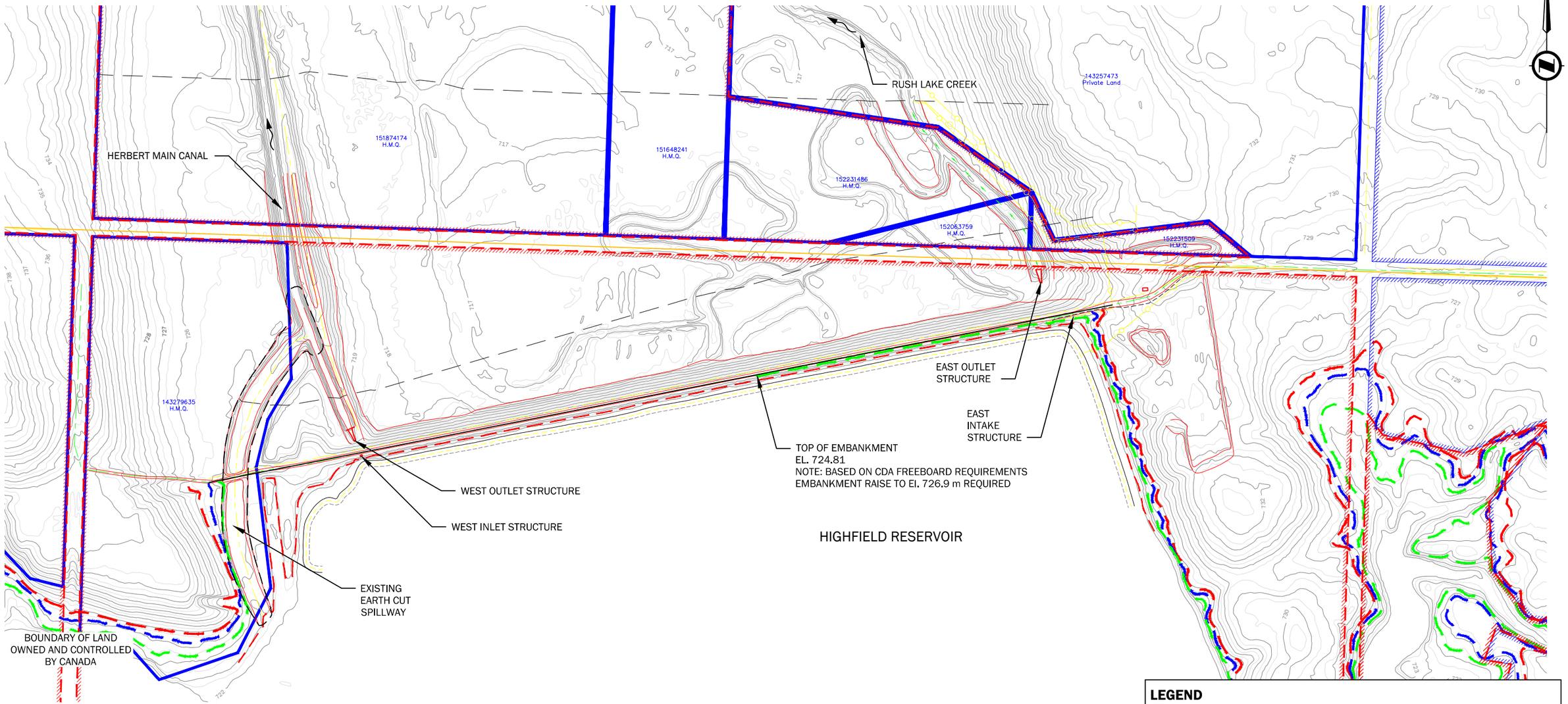
This report constitutes a testing service only. It does not represent any interpretation or opinion regarding specification compliance or material suitability.

Engineering interpretation will be provided by MDH Engineered Solutions Corp upon request.


Northwest Hydraulic Consultant
R2488
FGL-61B remolded
Shear Strength, kPa
Normal Stress, kPa
0
20
40
60
80
100
120
140
160
180
200
220
240
260
280
300
320
340
360
380
400
25°
Residual

Appendix G

Spillway Alternatives



LEGEND	
	200 - YEAR RAINFALL FLOOD - EL. 724.41
	500 - YEAR RAINFALL FLOOD - EL. 725.18
	1000 - YEAR RAINFALL FLOOD - EL. 725.84
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

SITE PLAN

SCALE = 1:5000

nhc northwest hydraulic consultants

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 20-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

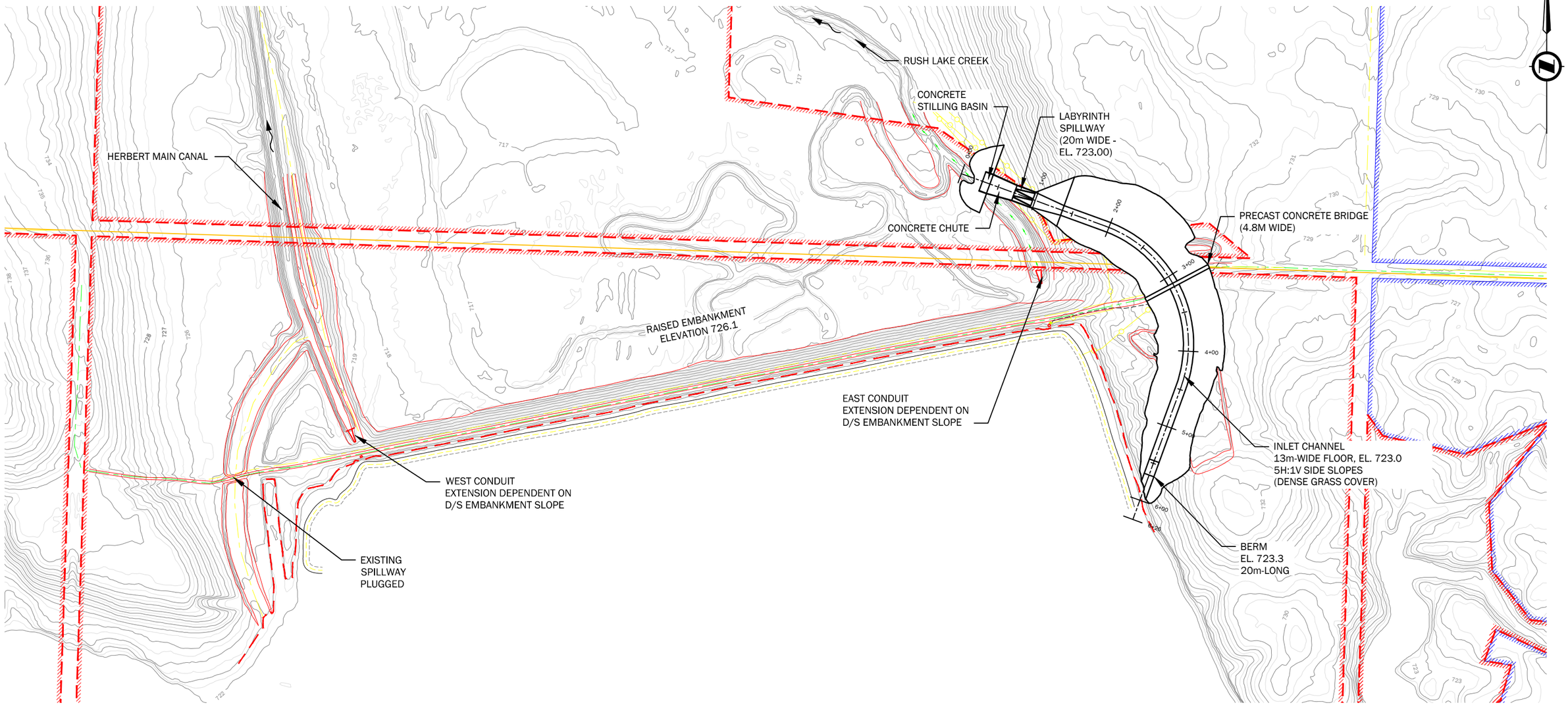
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 1

EXISTING SITE LAYOUT PLAN

Drawing No: 35525-001 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

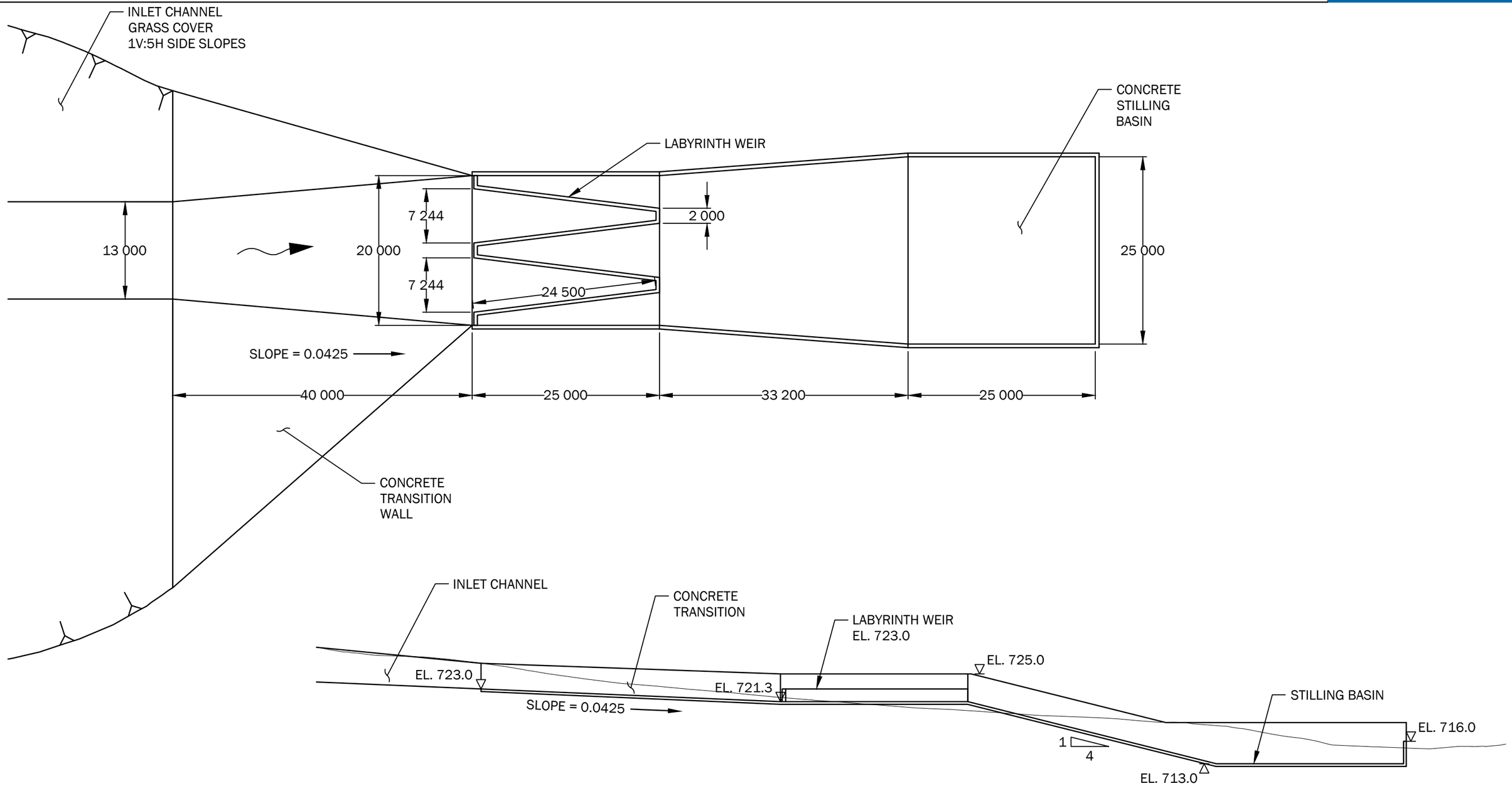
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 2

ALTERNATIVE 1
UNGATED LABYRINTH WEIR - EAST (PLAN)

Drawing No: 35525-002 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



LABYRINTH SPILLWAY AND AND SECTION

SCALE = 1:500

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

nhc northwest hydraulic consultants

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

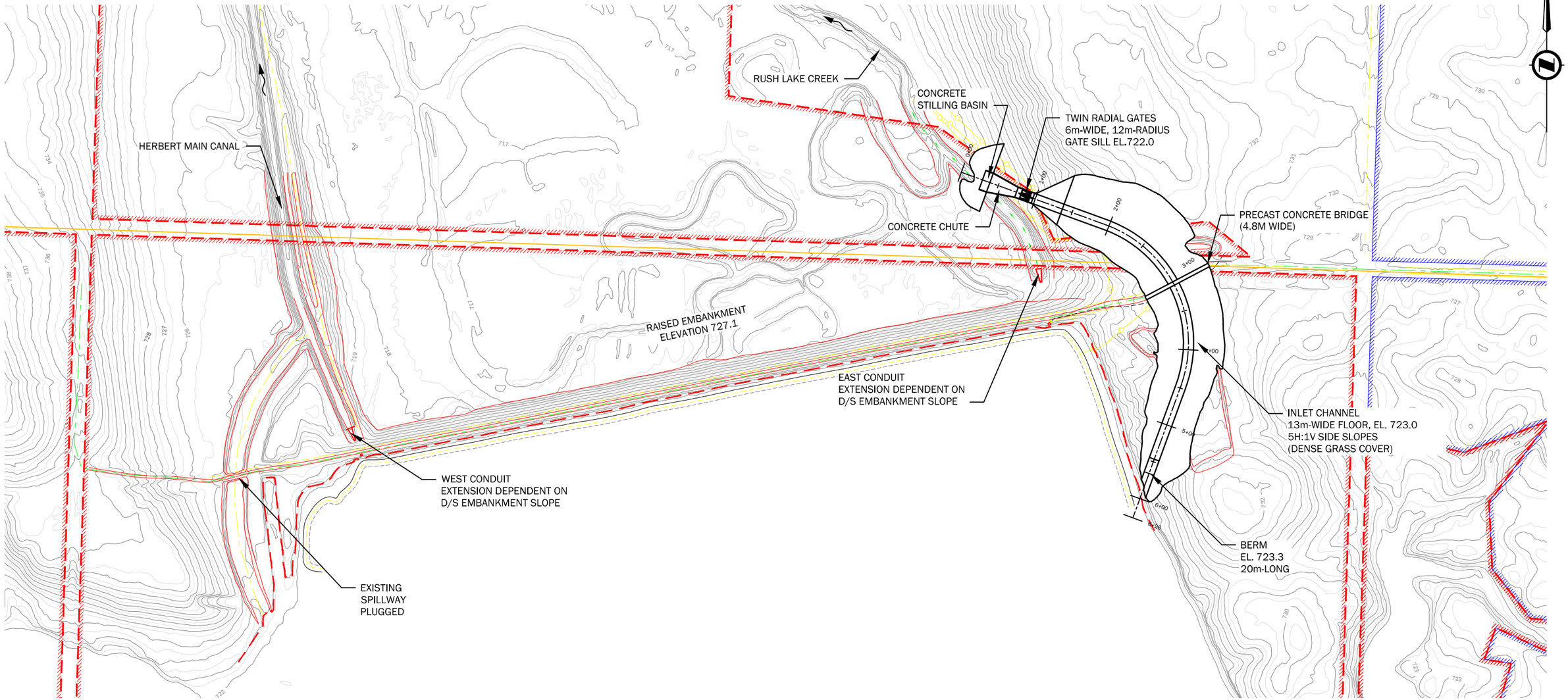
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 3

ALTERNATIVE 1
UNGATED LABYRINTH WEIR - EAST (DETAILS)

Drawing No: 35525-003 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

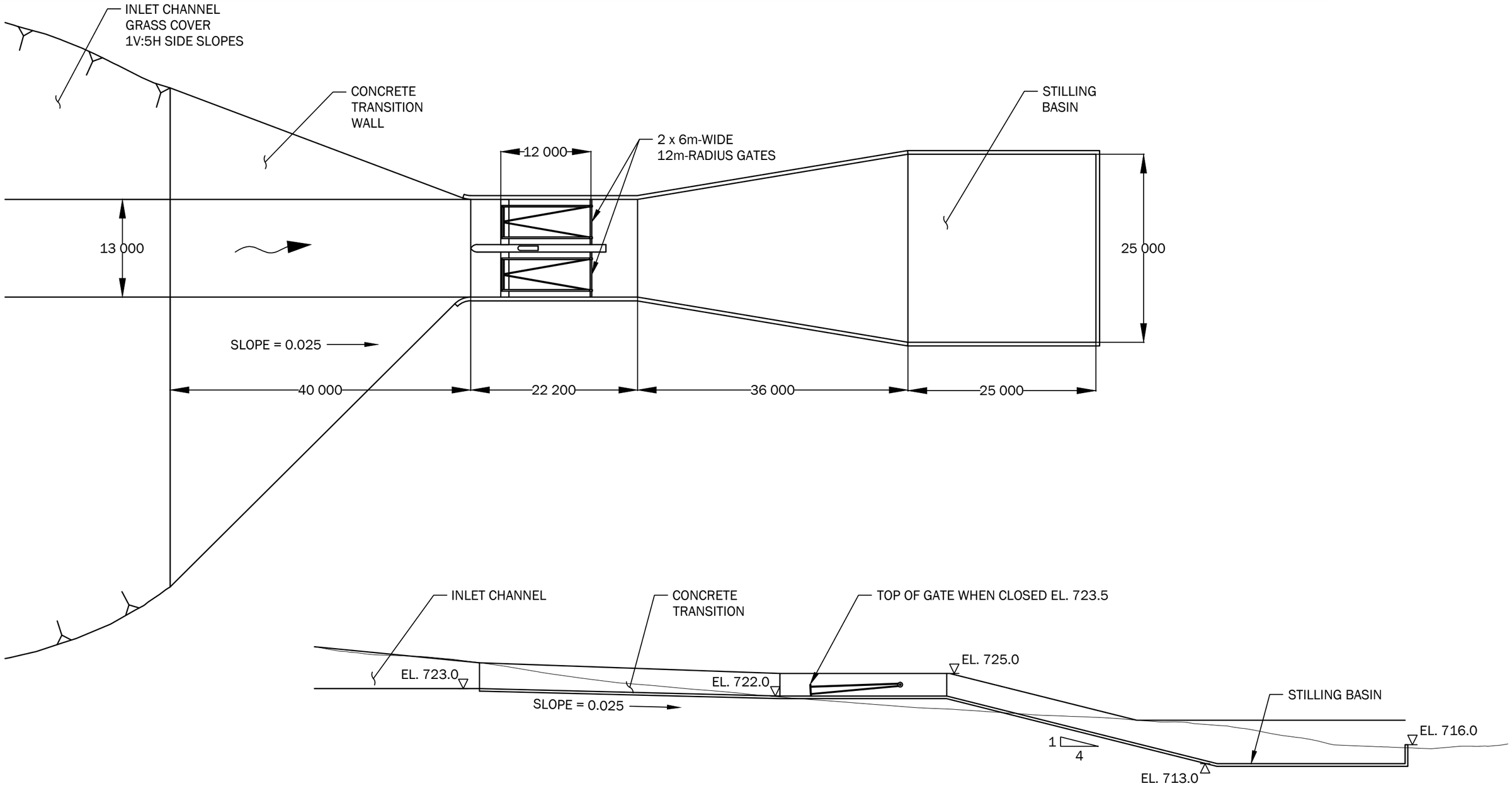
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 4

ALTERNATIVE 2
GATED SPILLWAY - EAST (PLAN)

Drawing No: 35525-004 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



GATED SPILLWAY PLAN AND SECTION

SCALE = 1:500

nhc northwest hydraulic consultants	Revisions:			Scale:	AS NOTED
	No:	Description:	By:	Date:	Designed By:	KIH
					Drawn by:	BXH
					Reviewed By:	BRH, ETZ
					Date:	JAN 21-2011
					Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

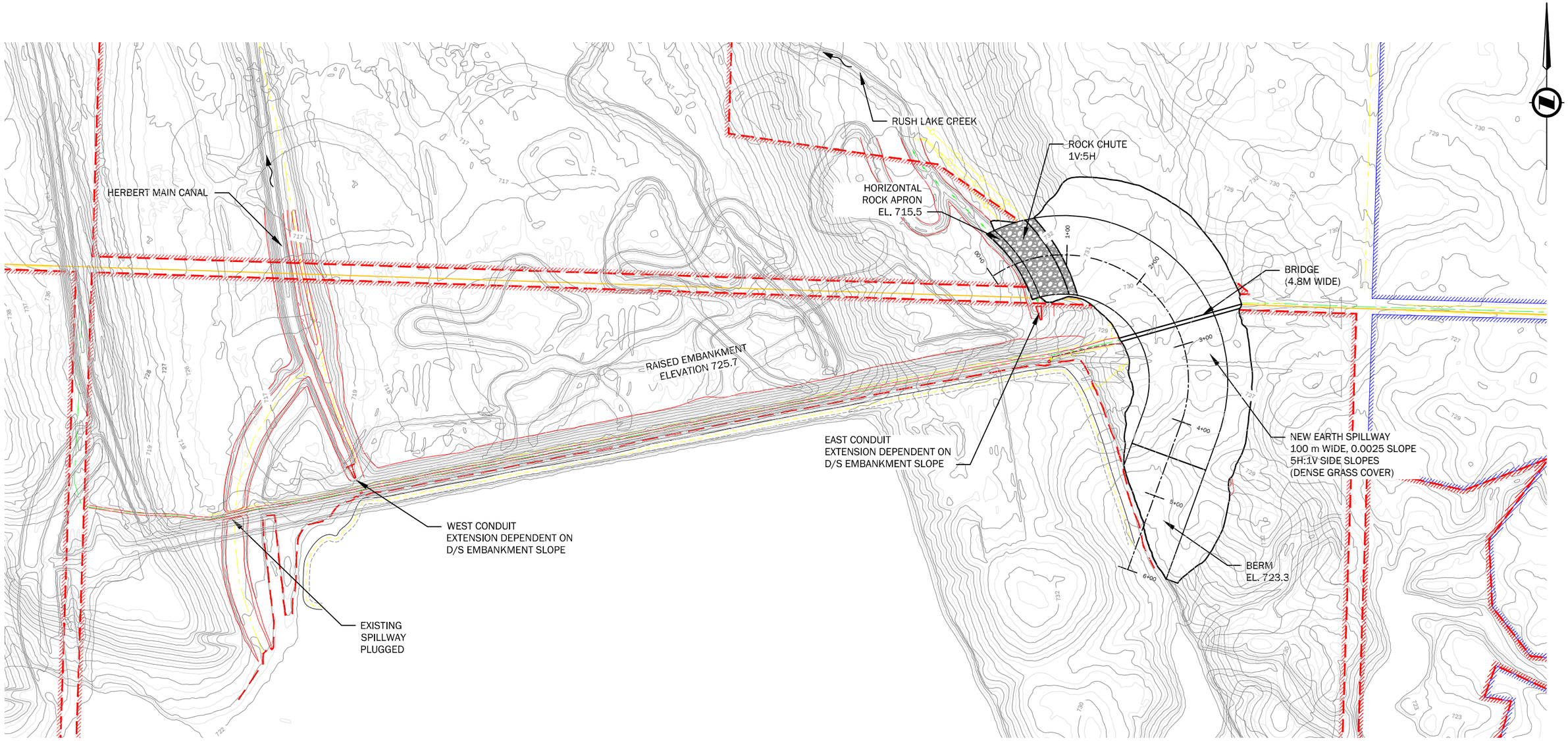
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 5

ALTERNATIVE 2
GATED SPILLWAY - EAST (DETAILS)

Drawing No: 35525-005 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

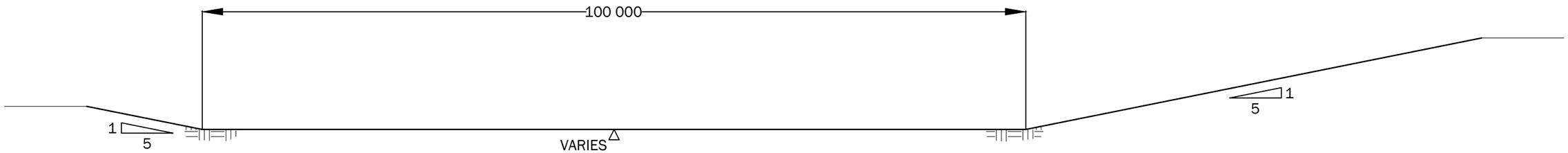
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 6

ALTERNATIVE 3
NEW EARTH SPILLWAY - EAST (PLAN)

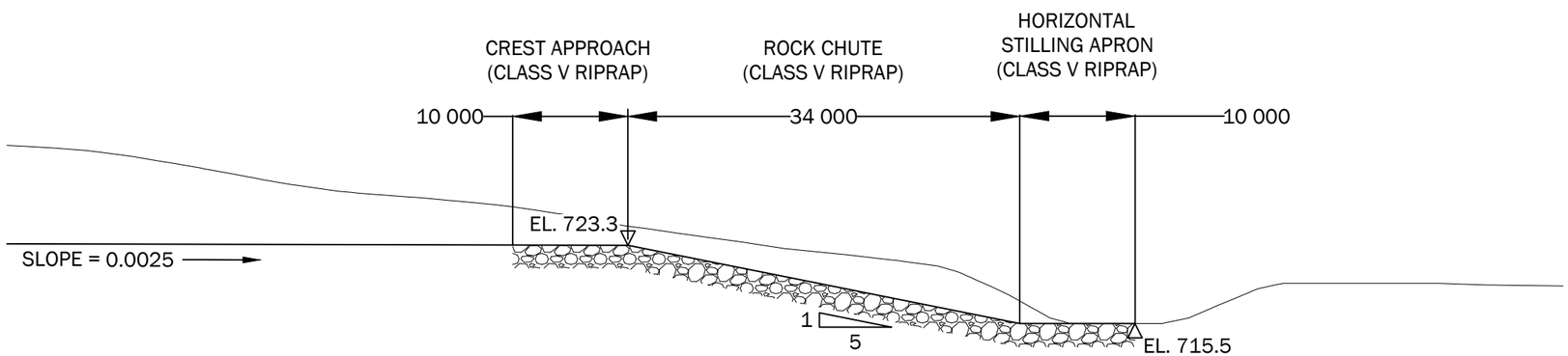
Drawing No: 35525-006 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



TYPICAL SECTION - NEW EARTH SPILLWAY

SCALE = 1:500



TYPICAL PROFILE - ROCK CHUTE SPILLWAY WITH STILLING APRON

SCALE = 1:500

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

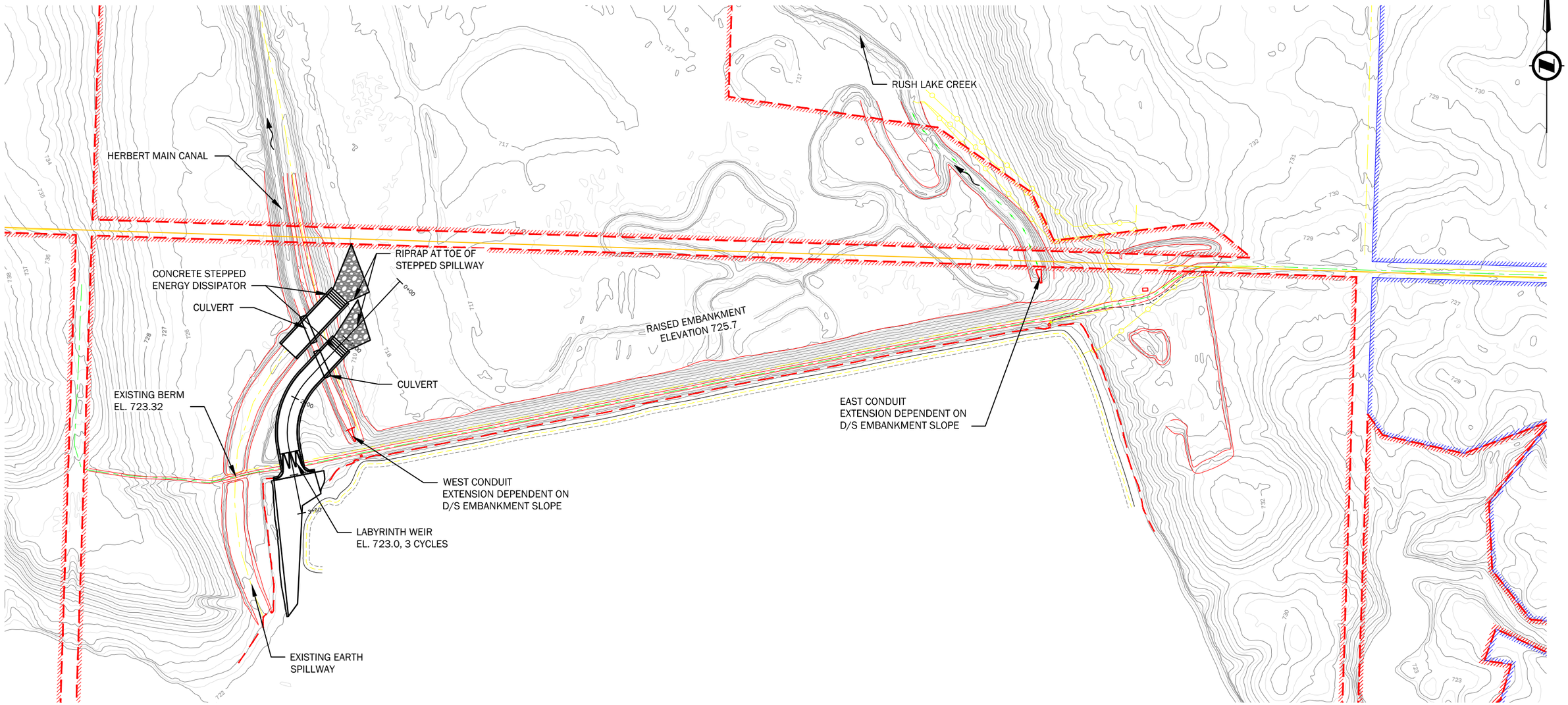
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 7

ALTERNATIVE 3
NEW EARTH SPILLWAY - EAST (DETAILS)

Drawing No: 35525-007 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
(Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

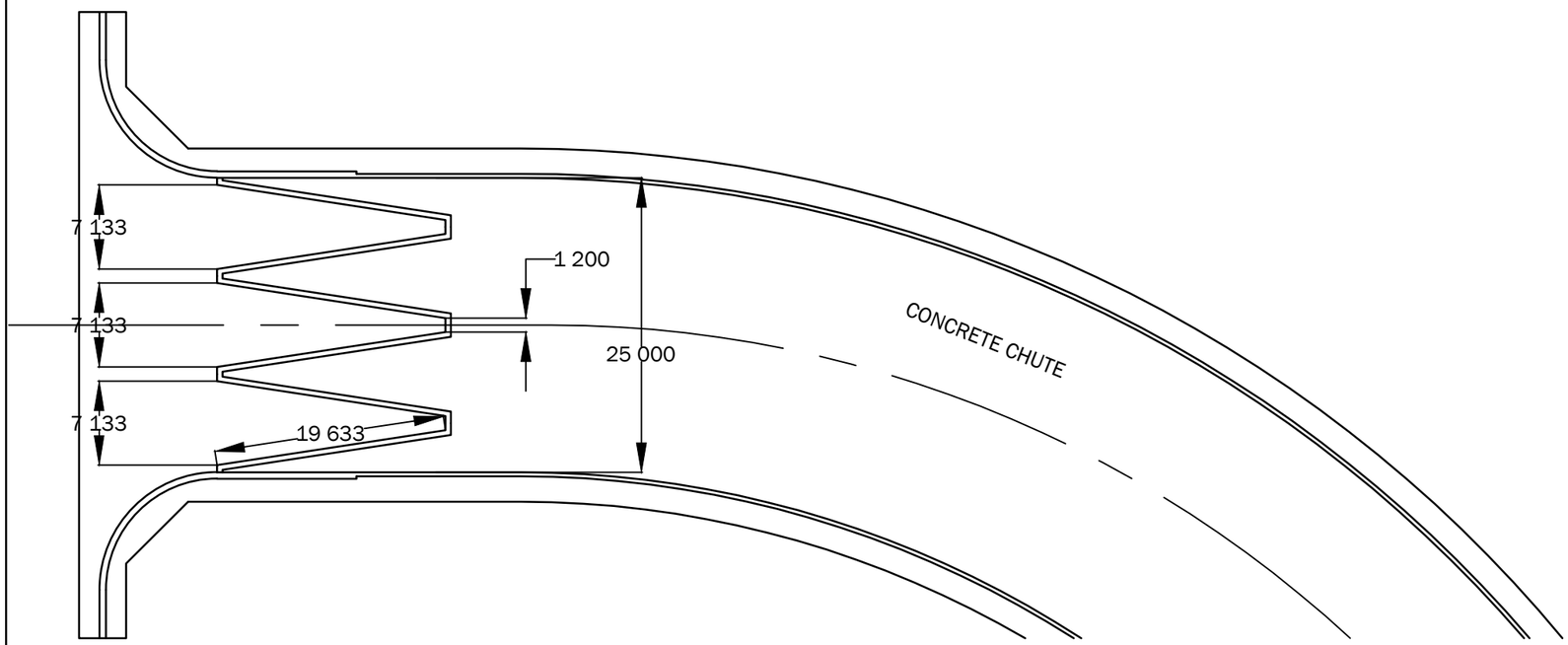
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 8

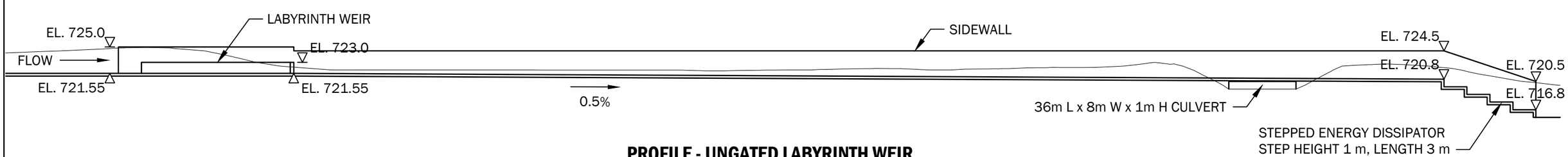
ALTERNATIVE 4
UNGATED LABYRINTH WEIR - WEST (PLAN)

Drawing No: 35525-008 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



PLAN - UNGATED LABYRINTH WEIR
SCALE = 1:500



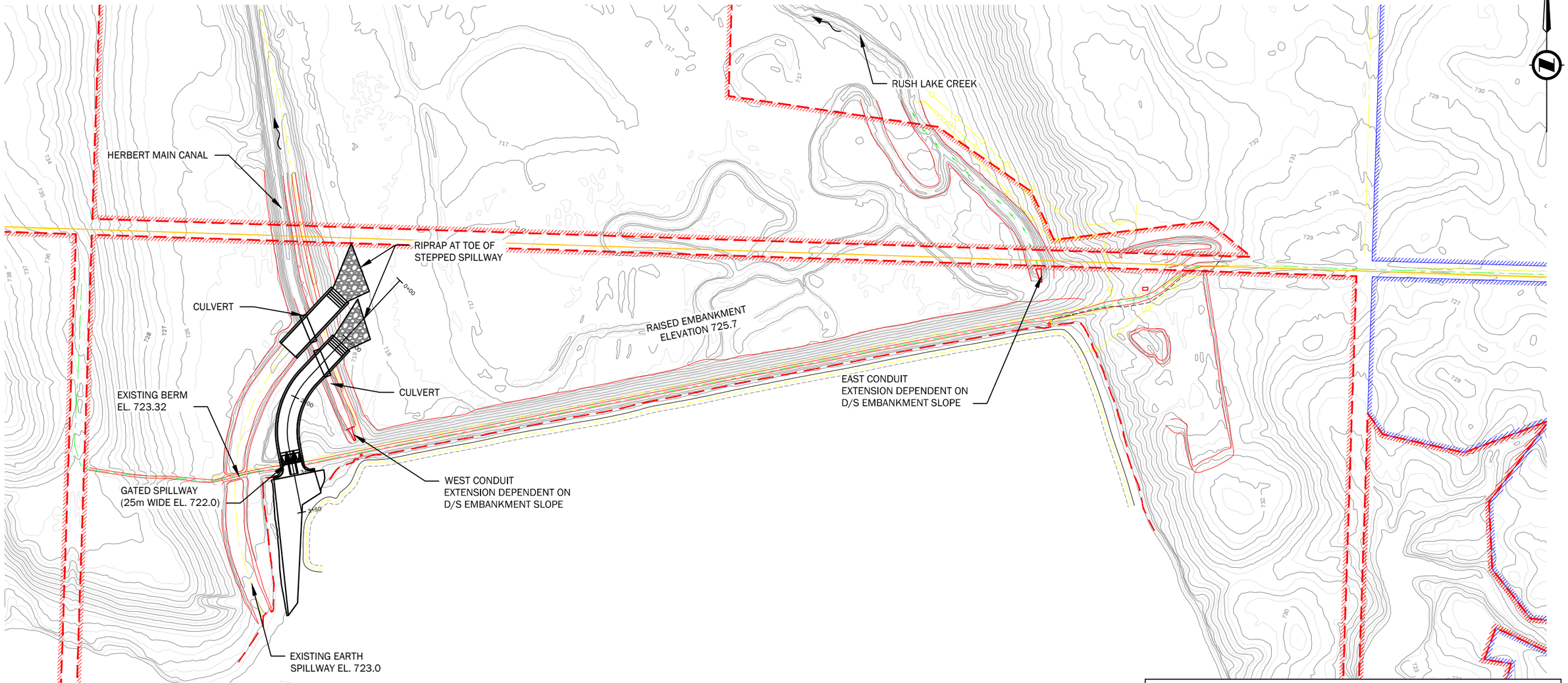
PROFILE - UNGATED LABYRINTH WEIR
SCALE = 1:500

nhc northwest hydraulic consultants
30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH, ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada
AESB – Agri-Environment Service Branch

HIGHFIELD DAM SPILLWAY PRE-DESIGN
FIGURE 9
ALTERNATIVE 4
UNGATED LABYRINTH WEIR - WEST (DETAILS)
Drawing No: 35525-011 Rev: 0



LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

SITE PLAN

SCALE = 1:5000

nhc northwest hydraulic consultants

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

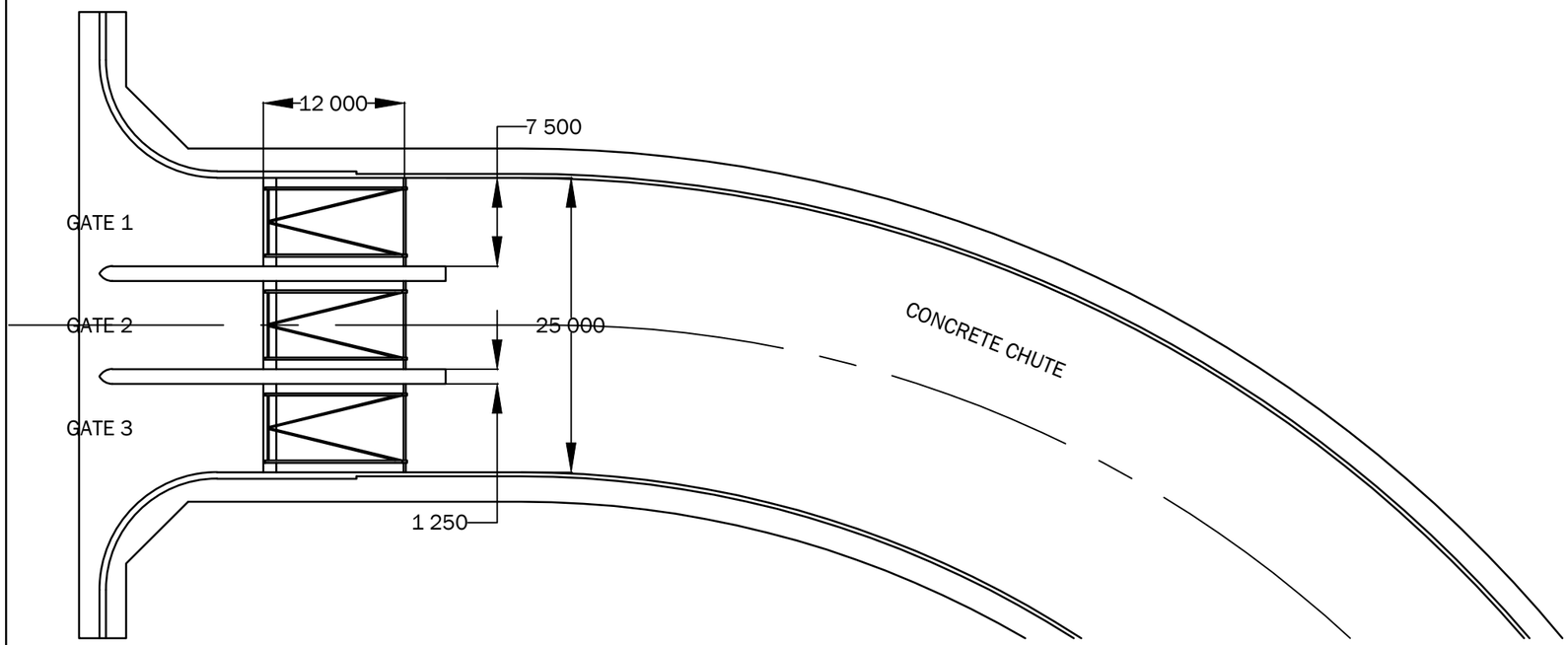
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 10

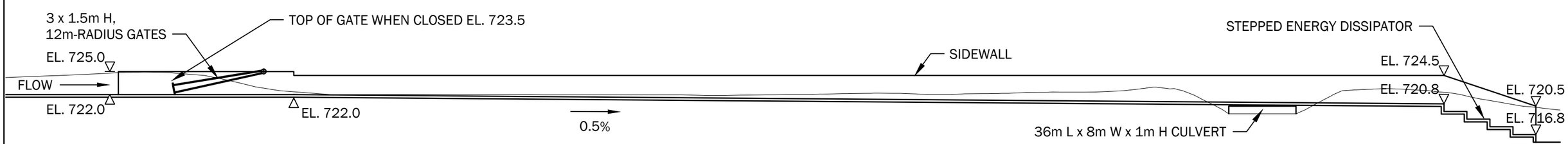
ALTERNATIVE 5
GATED SPILLWAY - WEST (PLAN)

Drawing No: 35525-010 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



PLAN - GATED SPILLWAY
SCALE = 1:500



PROFILE - GATED SPILLWAY
SCALE = 1:500

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

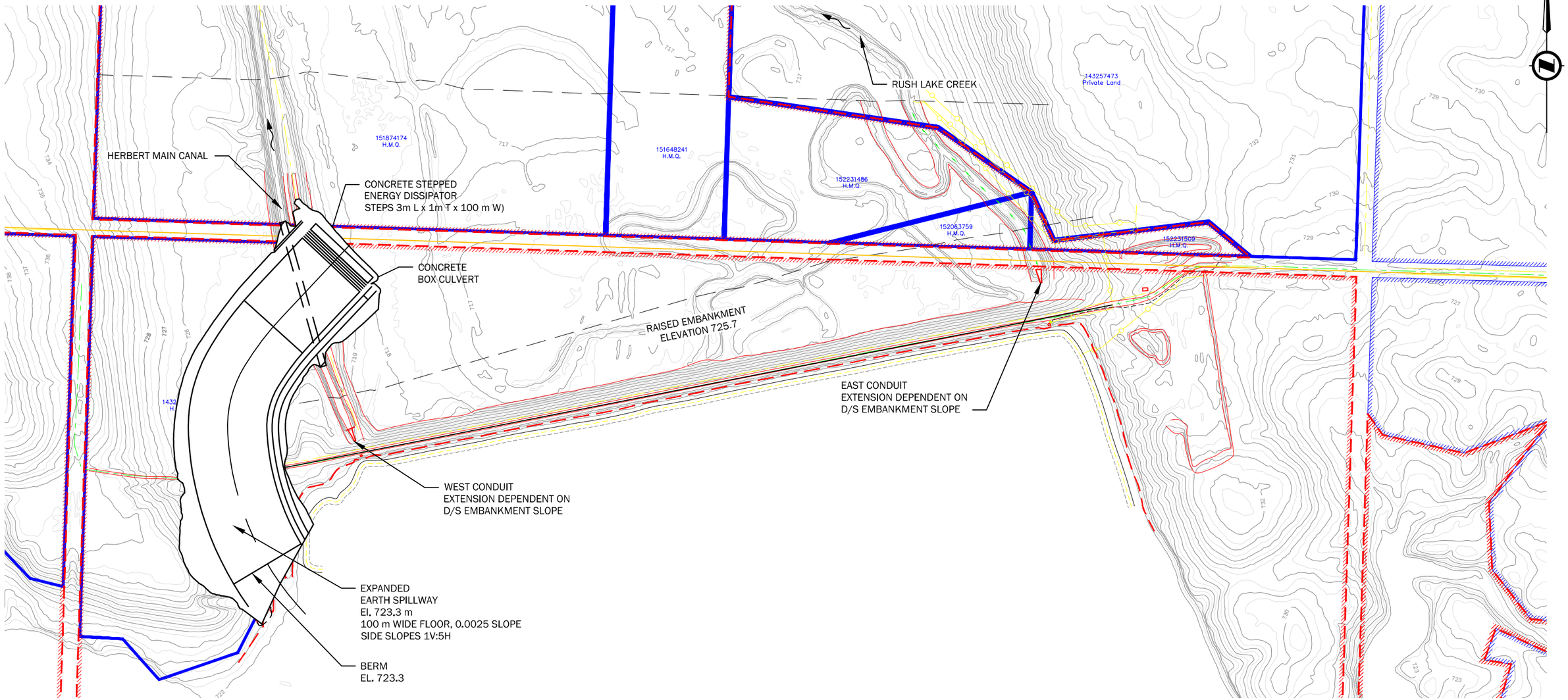
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 11

ALTERNATIVE 5
GATED SPILLWAY - WEST (DETAILS)

Drawing No: 35525-011 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



SITE PLAN

SCALE = 1:5000

LEGEND	
	BOUNDARY OF LAND OWNED OR CONTROLLED BY CANADA
********* (Canada/Private)	PARCEL NUMBER AND OWNER

nhc northwest hydraulic consultants

Revisions:					
No:	Description:	By:	Date:	Scale:	AS NOTED
				Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH,ETZ
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

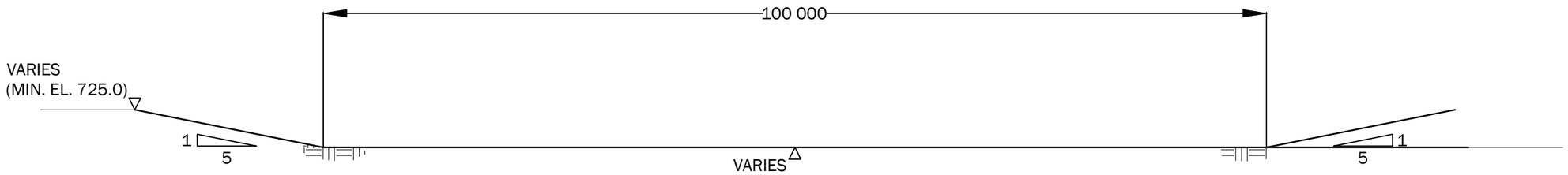
HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 12

ALTERNATIVE 6
NEW EARTH SPILLWAY - WEST (PLAN)

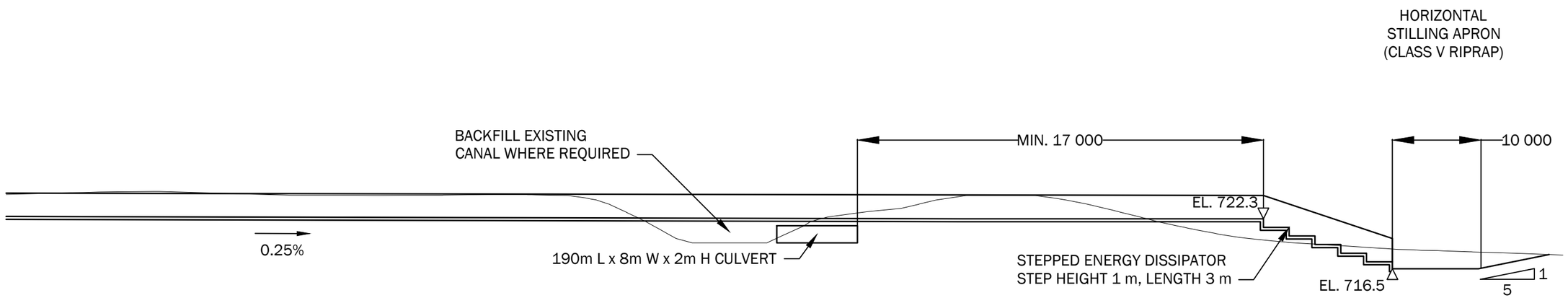
Drawing No: 35525-012 Rev: 0

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com



TYPICAL SECTION - NEW EARTH SPILLWAY

SCALE = 1:500



TYPICAL PROFILE - NEW EARTH SPILLWAY WITH STEPPED ENERGY DISSIPATOR

SCALE = 1:500

nhc northwest hydraulic consultants

30 Gostick Place, North Vancouver, B.C. V7M 3G3 Canada Office: 604.980.6011 Fax: 604.980.9264 www.nhcweb.com

Revisions:				Scale:	AS NOTED
No:	Description:	By:	Date:	Designed By:	KIH
				Drawn by:	BXH
				Reviewed By:	BRH
				Date:	JAN 21-2011
				Project No:	35525

Agriculture and Agri-Food Canada / Agriculture et Agroalimentaire Canada

AESB – Agri-Environment Service Branch

HIGHFIELD DAM SPILLWAY PRE-DESIGN

FIGURE 13

ALTERNATIVE 6
NEW EARTH SPILLWAY - WEST (DETAILS)

Drawing No: 35525-013 Rev: 0

Appendix H

Detailed Costs for Spillway Alternatives

MDH
ENGINEERED SOLUTIONS

Quantities and Cost Estimate Sheet

Contract No.:	R2488
Type of Work:	Dam Raise and Spillway Replacement
Region:	Highfield Dam
Prepared By:	Darrell Mihial
Date:	November 28, 2011

Project Information	
Project Number	Estimated Cost
Total	

Location	From km	To km	Length
		Total	m

Project Description: Cost estimate for Alternative 1, the ungated labyrinth weir, for the spillway upgrade and dam raising. Assumes embankment elevation of 727.1 masl and a downstream slope of 6:1.

Remarks:

Contract Items

Item No.	Spec. No.	Description	Unit	Quantity	Pre-Tender Price	Pre-Tender Cost	At Award Price	At Award Cost
		Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00		
		Land Acquisition/Control	Hectare	10	$1,250.00	$12,500.00		
		Care of Water	Lump Sum	1	$200,000.00	$200,000.00		
		Stripping	m^2	98 152	$3.50	$343,532.00		
		Earth Excavation and Embankment Construction	m^3	291 000	$5.00	$1,455,000.00		
		Structure Excavation	m^3	2 000	$22.00	$44,000.00		
		Haul of Borrow Material	m^3hm	1 450 000	$0.40	$580,000.00		
		Topsoil, Grading and Seeding	m^2	95 700	$8.50	$813,450.00		
		Traffic Gravel	m^3	575	$10.00	$5,748.64		
		Granular Filter	m^3	31 000	$50.00	$1,550,000.00		
		Rip Rap	m^3	12 000	$110.00	$1,320,000.00		
		Bedding Gravel	m^3	15 000	$40.00	$600,000.00		
		Structure Subdrainage System	Lump Sum	1	$250,000.00	$250,000.00		
		Reinforced Concrete Stilling Basin						
		- Walls	m^3	260	$1,200.00	$312,000.00		
		- Slab	m^3	310	$850.00	$263,500.00		
		- Foundation (piles)	m^3	75	$1,200.00	$90,000.00		
		Reinforced Concrete Chute						
		- Walls	m^3	130	$1,200.00	$156,000.00		
		- Slab	m^3	250	$850.00	$212,500.00		
		- Foundation (piles)	m^3	100	$1,200.00	$120,000.00		
		Reinforced Concrete Labrynth Weir						
		- Walls	m^3	155	$1,200.00	$186,000.00		
		- Slab	m^3	175	$850.00	$148,750.00		
		- Foundation (piles)	m^3	60	$1,200.00	$72,000.00		
		Reinforced Concrete Transition						
		- Slab	m^3	225	$850.00	$191,250.00		
		- Foundation (piles)	m^3	100	$1,200.00	$120,000.00		
		Metal Work	Lump Sum	1	$25,000.00	$25,000.00		
		Canal Precast Concrete Bridge	Lump Sum	1	$1,000,000.00	$1,000,000.00		
		West Outlet Modifications	Lump Sum	1	$200,000.00	$200,000.00		
		East Outlet Modifications	Lump Sum	1	$150,000.00	$150,000.00		
		Contract Items Sub-Total				**$10,671,230.64**		

Sundries

Item	Cost
Sundries Total	
Materials	
Material Sub-Total	
Freight/Miscellaneous	
Materials Total	

Budget Items

Budget Type	Year: Prev. Years	2008	2009	Total
1. Internal Budget Target				
2. Pre-Tender Estimate				
3. At Award Budget				

Pre-tender Total Added Project Cost	$3,948,355

	Difference ($)	% Difference
Contract Items At Award vs. Pre-Tender		

Summary of Estimates

Expenditure Category	Pre-Tender	At Award
Contract Items (Sub-Total(not inc. Site Occ.) + OCI)	$10,671,231	
Force Account		
Materials		
Sundries		
Construcion Engineering (10.0%)	$1,067,123	
Design Engineering (7.0%)	$746,986	
Testing Services		
AESB Forces		
ROW Property		
ROW Surveys		
Contingency (20.0%)	$2,134,246	
Total Cost	**$14,619,586**	

Other Contract Items (OCI)

Item	Cost
Other Contract Items Total	

Force Account

Item	Cost
General	
Force Account Total	

MDH ENGINEERED SOLUTIONS
232-1911 E Truesdale Drive
Regina, SK S4V 2N1
Ph: (306) 546-4220
Fax: (306) 546-4262

Highfield Dam Spillway Replacement
Alternative #1
Financial Analysis

Year	Description or Type of Repairs	Capital Costs	Maintenance Costs	Operating Costs
Net Present Value (Discount Rate = 5.00%) = $18,641,340		$14,197,750.16	$4,160,156.62	$283,432.94
1		$14,422,686.19	$0.00	$0.00
2		$0.00	$220,166.19	$15,000.00
3		$0.00	$220,166.19	$15,000.00
4		$0.00	$220,166.19	$15,000.00
5		$0.00	$220,166.19	$15,000.00
6		$0.00	$220,166.19	$15,000.00
7		$0.00	$220,166.19	$15,000.00
8		$0.00	$220,166.19	$15,000.00
9		$0.00	$220,166.19	$15,000.00
10		$0.00	$220,166.19	$15,000.00
11		$0.00	$220,166.19	$15,000.00
12		$0.00	$220,166.19	$15,000.00
13		$0.00	$220,166.19	$15,000.00
14		$0.00	$220,166.19	$15,000.00
15		$0.00	$220,166.19	$15,000.00
16		$0.00	$220,166.19	$15,000.00
17		$0.00	$220,166.19	$15,000.00
18		$0.00	$220,166.19	$15,000.00
19		$0.00	$220,166.19	$15,000.00
20		$0.00	$220,166.19	$15,000.00
21		$0.00	$220,166.19	$15,000.00
22		$0.00	$220,166.19	$15,000.00
23		$0.00	$220,166.19	$15,000.00
24		$0.00	$220,166.19	$15,000.00
25		$0.00	$220,166.19	$15,000.00
26		$0.00	$220,166.19	$15,000.00
27		$0.00	$220,166.19	$15,000.00
28		$0.00	$220,166.19	$15,000.00
29		$0.00	$220,166.19	$15,000.00
30		$0.00	$220,166.19	$15,000.00
31		$0.00	$220,166.19	$15,000.00
32		$0.00	$220,166.19	$15,000.00
33		$0.00	$220,166.19	$15,000.00
34		$0.00	$220,166.19	$15,000.00
35		$0.00	$220,166.19	$15,000.00
36		$0.00	$220,166.19	$15,000.00
37		$0.00	$220,166.19	$15,000.00
38		$0.00	$220,166.19	$15,000.00
39		$0.00	$220,166.19	$15,000.00
40		$0.00	$220,166.19	$15,000.00
41		$0.00	$220,166.19	$15,000.00
42		$0.00	$220,166.19	$15,000.00
43		$0.00	$220,166.19	$15,000.00
44		$0.00	$220,166.19	$15,000.00
45		$0.00	$220,166.19	$15,000.00
46		$0.00	$220,166.19	$15,000.00
47		$0.00	$220,166.19	$15,000.00
48		$0.00	$220,166.19	$15,000.00

49		$0.00	$220,166.19	$15,000.00
50		$0.00	$220,166.19	$15,000.00
51	new concrete spillway	$5,561,132.50	$220,166.19	$15,000.00
52		$0.00	$220,166.19	$15,000.00
53		$0.00	$220,166.19	$15,000.00
54		$0.00	$220,166.19	$15,000.00
55		$0.00	$220,166.19	$15,000.00
56		$0.00	$220,166.19	$15,000.00
57		$0.00	$220,166.19	$15,000.00
58		$0.00	$220,166.19	$15,000.00
59		$0.00	$220,166.19	$15,000.00
60		$0.00	$220,166.19	$15,000.00
61		$0.00	$220,166.19	$15,000.00
62		$0.00	$220,166.19	$15,000.00
63		$0.00	$220,166.19	$15,000.00
64		$0.00	$220,166.19	$15,000.00
65		$0.00	$220,166.19	$15,000.00
66		$0.00	$220,166.19	$15,000.00
67		$0.00	$220,166.19	$15,000.00
68		$0.00	$220,166.19	$15,000.00
69		$0.00	$220,166.19	$15,000.00
70		$0.00	$220,166.19	$15,000.00
71		$0.00	$220,166.19	$15,000.00
72		$0.00	$220,166.19	$15,000.00
73		$0.00	$220,166.19	$15,000.00
74		$0.00	$220,166.19	$15,000.00
75		$0.00	$220,166.19	$15,000.00
76		$0.00	$220,166.19	$15,000.00
77		$0.00	$220,166.19	$15,000.00
78		$0.00	$220,166.19	$15,000.00
79		$0.00	$220,166.19	$15,000.00
80		$0.00	$220,166.19	$15,000.00
81		$0.00	$220,166.19	$15,000.00
82		$0.00	$220,166.19	$15,000.00
83		$0.00	$220,166.19	$15,000.00
84		$0.00	$220,166.19	$15,000.00
85		$0.00	$220,166.19	$15,000.00
86		$0.00	$220,166.19	$15,000.00
87		$0.00	$220,166.19	$15,000.00
88		$0.00	$220,166.19	$15,000.00
89		$0.00	$220,166.19	$15,000.00
90		$0.00	$220,166.19	$15,000.00
91		$0.00	$220,166.19	$15,000.00
92		$0.00	$220,166.19	$15,000.00
93		$0.00	$220,166.19	$15,000.00
94		$0.00	$220,166.19	$15,000.00
95		$0.00	$220,166.19	$15,000.00
96		$0.00	$220,166.19	$15,000.00
97		$0.00	$220,166.19	$15,000.00
98		$0.00	$220,166.19	$15,000.00
99		$0.00	$220,166.19	$15,000.00
100		$0.00	$220,166.19	$15,000.00
Total		$19,983,818.69	$21,796,452.50	$1,485,000.00

MDH
ENGINEERED SOLUTIONS

Quantities and Cost Estimate Sheet

Contract No.:	R2488
Type of Work:	Dam Raise and Spillway Replacement
Region:	Highfield Dam
Prepared By:	Darrell Mihial
Date:	November 28, 2011

Project Information	
Project Number	Estimated Cost
Total	

Location	From km	To km	Length
		Total	m

Project Description: Cost estimate for Alternative 2, the gated concrete spillway, for the spillway upgrade and dam raising. Assumes embankment elevation of 727.2 masl and a downstream slope of 6:1.

Remarks:

Contract Items

Item No.	Spec. No.	Description	Unit	Quantity	Pre-Tender Price	Pre-Tender Cost	At Award Price	At Award Cost
		Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00		
		Land Acquisition/Control	Hectare	10	$1,250.00	$12,500.00		
		Care of Water	Lump Sum	1	$200,000.00	$200,000.00		
		Stripping	m^2	104 392	$3.50	$365,372.00		
		Earth Excavation and Embankment Construction	m^3	291 000	$5.00	$1,455,000.00		
		Structure Excavation	m^3	2 000	$22.00	$44,000.00		
		Haul of Borrow Material	m^3hm	1 450 000	$0.40	$580,000.00		
		Topsoil, Grading and Seeding	m^2	101 940	$8.50	$866,490.00		
		Traffic Gravel	m^3	575	$10.00	$5,748.64		
		Granular Filter	m^3	36 000	$50.00	$1,800,000.00		
		Rip Rap	m^3	12 000	$110.00	$1,320,000.00		
		Bedding Gravel	m^3	15 000	$40.00	$600,000.00		
		Structure Subdrainage System	Lump Sum	1	$200,000.00	$200,000.00		
		Reinforced Concrete Stilling Basin						
		- Walls	m^3	260	$1,200.00	$312,000.00		
		- Slab	m^3	310	$850.00	$263,500.00		
		- Foundation (piles)	m^3	75	$1,200.00	$90,000.00		
		Reinforced Concrete Chute						
		- Walls	m^3	130	$1,200.00	$156,000.00		
		- Slab	m^3	250	$850.00	$212,500.00		
		- Foundation (piles)	m^3	100	$1,200.00	$120,000.00		
		Reinforced Concrete Labrynth Weir						
		- Walls	m^3		$1,200.00	$1,200.00		
		- Slab	m^3	175	$850.00	$148,750.00		
		- Foundation (piles)	m^3	60	$1,200.00	$72,000.00		
		Reinforced Concrete Transition						
		- Slab	m^3	225	$850.00	$191,250.00		
		- Foundation (piles)	m^3	100	$1,200.00	$120,000.00		
		Metal Work	Lump Sum	1	$200,000.00	$200,000.00		
		Canal Precast Concrete Bridge	Lump Sum	1	$1,000,000.00	$1,000,000.00		
		West Outlet Modifications	Lump Sum	1	$200,000.00	$200,000.00		
		East Outlet Modifications	Lump Sum	1	$200,000.00	$200,000.00		
		Spillway Gates	Lump Sum	2	$250,000.00	$500,000.00		
		Contract Items Sub-Total				**$11,486,310.64**		

Sundries

Item	Cost
Sundries Total	

Materials			
Material Sub-Total			
Freight/Miscellaneous			
Materials Total			

Budget Items

Budget Type	Year: Prev. Years	2008	2009	Total
1. Internal Budget Target				
2. Pre-Tender Estimate				
3. At Award Budget				

Pre-tender Total Added Project Cost	$4,249,935

	Difference ($)	% Difference
Contract Items At Award vs. Pre-Tender		

Summary of Estimates

Expenditure Category	Pre-Tender	At Award
Contract Items (Sub-Total(not inc. Site Occ.) + OCI)	$11,486,311	
Force Account		
Materials		
Sundries		
Construcion Engineering (10.0%)	$1,148,631	
Design Engineering (7.0%)	$804,042	
Testing Services		
AESB forces		
ROW Property		
ROW Surveys		
Contingency (20.0%)	$2,297,262	
Total Cost	**$15,736,246**	

Other Contract Items (OCI)

Item	Cost
Other Contract Items Total	

Force Account

Item	Cost
General	
Force Account Total	

MDH ENGINEERED SOLUTIONS
232-1911 E Truesdale Drive
Regina, SK S4V 2N1
Ph: (306) 546-4220
Fax: (306) 546-4262

Highfield Dam Spillway Replacement
Alternative #2
Financial Analysis

Year	Description or Type of Repairs	Capital Costs	Maintenance Costs	Operating Costs
Net Present Value (Discount Rate = 5.00%) = $21,378,086		$16,055,440.06	$4,850,258.14	$472,388.23
1		$16,086,991.14	$0.00	$0.00
2		$0.00	$256,688.13	$25,000.00
3		$0.00	$256,688.13	$25,000.00
4		$0.00	$256,688.13	$25,000.00
5		$0.00	$256,688.13	$25,000.00
6		$0.00	$256,688.13	$25,000.00
7		$0.00	$256,688.13	$25,000.00
8		$0.00	$256,688.13	$25,000.00
9		$0.00	$256,688.13	$25,000.00
10		$0.00	$256,688.13	$25,000.00
11		$0.00	$256,688.13	$25,000.00
12		$0.00	$256,688.13	$25,000.00
13		$0.00	$256,688.13	$25,000.00
14		$0.00	$256,688.13	$25,000.00
15		$0.00	$256,688.13	$25,000.00
16		$0.00	$256,688.13	$25,000.00
17		$0.00	$256,688.13	$25,000.00
18		$0.00	$256,688.13	$25,000.00
19		$0.00	$256,688.13	$25,000.00
20		$0.00	$256,688.13	$25,000.00
21		$0.00	$256,688.13	$25,000.00
22		$0.00	$256,688.13	$25,000.00
23		$0.00	$256,688.13	$25,000.00
24		$0.00	$256,688.13	$25,000.00
25		$0.00	$256,688.13	$25,000.00
26	New spillway gates	$770,000.00	$256,688.13	$25,000.00
27		$0.00	$256,688.13	$25,000.00
28		$0.00	$256,688.13	$25,000.00
29		$0.00	$256,688.13	$25,000.00
30		$0.00	$256,688.13	$25,000.00
31		$0.00	$256,688.13	$25,000.00
32		$0.00	$256,688.13	$25,000.00
33		$0.00	$256,688.13	$25,000.00
34		$0.00	$256,688.13	$25,000.00
35		$0.00	$256,688.13	$25,000.00
36		$0.00	$256,688.13	$25,000.00
37		$0.00	$256,688.13	$25,000.00
38		$0.00	$256,688.13	$25,000.00
39		$0.00	$256,688.13	$25,000.00
40		$0.00	$256,688.13	$25,000.00
41		$0.00	$256,688.13	$25,000.00
42		$0.00	$256,688.13	$25,000.00
43		$0.00	$256,688.13	$25,000.00
44		$0.00	$256,688.13	$25,000.00
45		$0.00	$256,688.13	$25,000.00
46		$0.00	$256,688.13	$25,000.00
47		$0.00	$256,688.13	$25,000.00
48		$0.00	$256,688.13	$25,000.00

49		$0.00	$256,688.13	$25,000.00
50		$0.00	$256,688.13	$25,000.00
51	new concrete spillway and gates	$6,009,022.25	$256,688.13	$25,000.00
52		$0.00	$256,688.13	$25,000.00
53		$0.00	$256,688.13	$25,000.00
54		$0.00	$256,688.13	$25,000.00
55		$0.00	$256,688.13	$25,000.00
56		$0.00	$256,688.13	$25,000.00
57		$0.00	$256,688.13	$25,000.00
58		$0.00	$256,688.13	$25,000.00
59		$0.00	$256,688.13	$25,000.00
60		$0.00	$256,688.13	$25,000.00
61		$0.00	$256,688.13	$25,000.00
62		$0.00	$256,688.13	$25,000.00
63		$0.00	$256,688.13	$25,000.00
64		$0.00	$256,688.13	$25,000.00
65		$0.00	$256,688.13	$25,000.00
66		$0.00	$256,688.13	$25,000.00
67		$0.00	$256,688.13	$25,000.00
68		$0.00	$256,688.13	$25,000.00
69		$0.00	$256,688.13	$25,000.00
70		$0.00	$256,688.13	$25,000.00
71		$0.00	$256,688.13	$25,000.00
72		$0.00	$256,688.13	$25,000.00
73		$0.00	$256,688.13	$25,000.00
74		$0.00	$256,688.13	$25,000.00
75		$0.00	$256,688.13	$25,000.00
76	New spillway gates	$770,000.00	$256,688.13	$25,000.00
77		$0.00	$256,688.13	$25,000.00
78		$0.00	$256,688.13	$25,000.00
79		$0.00	$256,688.13	$25,000.00
80		$0.00	$256,688.13	$25,000.00
81		$0.00	$256,688.13	$25,000.00
82		$0.00	$256,688.13	$25,000.00
83		$0.00	$256,688.13	$25,000.00
84		$0.00	$256,688.13	$25,000.00
85		$0.00	$256,688.13	$25,000.00
86		$0.00	$256,688.13	$25,000.00
87		$0.00	$256,688.13	$25,000.00
88		$0.00	$256,688.13	$25,000.00
89		$0.00	$256,688.13	$25,000.00
90		$0.00	$256,688.13	$25,000.00
91		$0.00	$256,688.13	$25,000.00
92		$0.00	$256,688.13	$25,000.00
93		$0.00	$256,688.13	$25,000.00
94		$0.00	$256,688.13	$25,000.00
95		$0.00	$256,688.13	$25,000.00
96		$0.00	$256,688.13	$25,000.00
97		$0.00	$256,688.13	$25,000.00
98		$0.00	$256,688.13	$25,000.00
99		$0.00	$256,688.13	$25,000.00
100		$0.00	$256,688.13	$25,000.00
Total		$23,636,013.39	$25,412,125.25	$2,475,000.00

MDH ENGINEERED SOLUTIONS

Quantities and Cost Estimate Sheet

Contract No.:	R2488
Type of Work:	Dam Raise and Spillway Replacement
Region:	Highfield Dam
Prepared By:	Darrell Mihial
Date:	November 30, 2011

Project Information	
Project Number	Estimated Cost
Total	

Location	From km	To km	Length
		Total	m

Project Description: Cost estimate for Alternative 3, the earth spillway on the east side, for the spillway upgrade and dam raising. Assumes embankment elevation of 725.7 masl and a downstream slope of 6:1.

Remarks:

Contract Items

Item No.	Spec. No.	Description	Unit	Quantity	Pre-Tender Price	Pre-Tender Cost	At Award Price	At Award Cost
		Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00		
		Land Acquisition/Control	Hectare	10	$1,250.00	$12,500.00		
		Care of Water	Lump Sum	1	$50,000.00	$50,000.00		
		Stripping	m^2	135 232	$3.50	$473,312.00		
		Earth Excavation and Embankment Constructio	m^3	360 000	$5.00	$1,800,000.00		
		Structure Excavation	m^3	850	$22.00	$18,700.00		
		Haul of Borrow Material	m^3hm		$0.40			
		Topsoil, Grading and Seeding	m^2	135 100	$8.50	$1,148,350.00		
		Traffic Gravel	m^3	575	$10.00	$5,748.64		
		Granular Filter	m^3	28 000	$50.00	$1,400,000.00		
		Rip Rap	m^3	16 000	$110.00	$1,760,000.00		
		Bedding Gravel	m^3	15 000	$40.00	$600,000.00		
		Structure Subdrainage System	Lump Sum		$200,000.00			
		Energy Dissipator						
		- Walls	m^3	100	$1,200.00	$120,000.00		
		- Slab	m^3	1 100	$850.00	$935,000.00		
		- Foundation (piles)	m^3	100	$1,200.00	$120,000.00		
		Reinforced Concrete Chute						
		- Walls	m^3		$1,200.00			
		- Slab	m^3		$850.00			
		- Foundation (piles)	m^3		$1,200.00			
		Reinforced Concrete Labrynth Weir						
		- Walls	m^3		$1,200.00			
		- Slab	m^3		$850.00			
		- Foundation (piles)	m^3		$1,200.00			
		Reinforced Concrete Transition						
		- Slab	m^3		$850.00			
		- Foundation (piles)	m^3		$1,200.00			
		Metal Work	Lump Sum	1				
		Canal Precast Concrete Bridge	Lump Sum	1	$1,800,000.00	$1,800,000.00		
		West Outlet Modifications	Lump Sum	1	$150,000.00	$150,000.00		
		Spillway Gates	Lump Sum		$250,000.00			
		Contract Items Sub-Total				**$10,643,610.64**		

Sundries

Item	Cost
Sundries Total	

Materials			
Material Sub-Total			
Freight/Miscellaneous			
Materials Total			

Budget Items

Budget Type	Year Prev. Years	2008	2009	Total
1. Internal Budget Target				
2. Pre-Tender Estimate				
3. At Award Budget				

Pre-tender Total Added Project Cost	$3,938,136

	Difference ($)	% Difference
Contract Items At Award vs. Pre-Tender		

Summary of Estimates

Expenditure Category	Pre-Tender	At Award
Contract Items (Sub-Total(not inc. Site Occ.) + OCI)	$10,643,611	
Force Account		
Materials		
Sundries		
Construcion Engineering (10.0%)	$1,064,361	
Design Engineering (7.0%)	$745,053	
Testing Services		
AESB Forces		
ROW Property		
ROW Surveys		
Contingency (20.0%)	$2,128,722	
Total Cost	**$14,581,747**	

Other Contract Items (OCI)

Item	Cost
Other Contract Items Total	

Force Account

Item	Cost
General	
Force Account Total	

MDH ENGINEERED SOLUTIONS
232-1911 E Truesdale Drive
Regina, SK S4V 2N1
Ph: (306) 546-4220
Fax: (306) 546-4262

Highfield Dam Spillway Replacement
Alternative #3
Financial Analysis

Year	Description or Type of Repairs	Capital Costs	Maintenance Costs	Operating Costs
Net Present Value (Discount Rate = 5.00%) = $18,741,174		$14,363,870.61	$4,282,825.40	$94,477.65
1		$15,082,064.14	$0.00	$0.00
2		$0.00	$226,658.13	$5,000.00
3		$0.00	$226,658.13	$5,000.00
4		$0.00	$226,658.13	$5,000.00
5		$0.00	$226,658.13	$5,000.00
6		$0.00	$226,658.13	$5,000.00
7		$0.00	$226,658.13	$5,000.00
8		$0.00	$226,658.13	$5,000.00
9		$0.00	$226,658.13	$5,000.00
10		$0.00	$226,658.13	$5,000.00
11		$0.00	$226,658.13	$5,000.00
12		$0.00	$226,658.13	$5,000.00
13		$0.00	$226,658.13	$5,000.00
14		$0.00	$226,658.13	$5,000.00
15		$0.00	$226,658.13	$5,000.00
16		$0.00	$226,658.13	$5,000.00
17		$0.00	$226,658.13	$5,000.00
18		$0.00	$226,658.13	$5,000.00
19		$0.00	$226,658.13	$5,000.00
20		$0.00	$226,658.13	$5,000.00
21		$0.00	$226,658.13	$5,000.00
22		$0.00	$226,658.13	$5,000.00
23		$0.00	$226,658.13	$5,000.00
24		$0.00	$226,658.13	$5,000.00
25		$0.00	$226,658.13	$5,000.00
26		$0.00	$226,658.13	$5,000.00
27		$0.00	$226,658.13	$5,000.00
28		$0.00	$226,658.13	$5,000.00
29		$0.00	$226,658.13	$5,000.00
30		$0.00	$226,658.13	$5,000.00
31		$0.00	$226,658.13	$5,000.00
32		$0.00	$226,658.13	$5,000.00
33		$0.00	$226,658.13	$5,000.00
34		$0.00	$226,658.13	$5,000.00
35		$0.00	$226,658.13	$5,000.00
36		$0.00	$226,658.13	$5,000.00
37		$0.00	$226,658.13	$5,000.00
38		$0.00	$226,658.13	$5,000.00
39		$0.00	$226,658.13	$5,000.00
40		$0.00	$226,658.13	$5,000.00
41		$0.00	$226,658.13	$5,000.00
42		$0.00	$226,658.13	$5,000.00
43		$0.00	$226,658.13	$5,000.00
44		$0.00	$226,658.13	$5,000.00
45		$0.00	$226,658.13	$5,000.00
46		$0.00	$226,658.13	$5,000.00
47		$0.00	$226,658.13	$5,000.00
48		$0.00	$226,658.13	$5,000.00

49		$0.00	$226,658.13	$5,000.00
50		$0.00	$226,658.13	$5,000.00
51		$0.00	$226,658.13	$5,000.00
52		$0.00	$226,658.13	$5,000.00
53		$0.00	$226,658.13	$5,000.00
54		$0.00	$226,658.13	$5,000.00
55		$0.00	$226,658.13	$5,000.00
56		$0.00	$226,658.13	$5,000.00
57		$0.00	$226,658.13	$5,000.00
58		$0.00	$226,658.13	$5,000.00
59		$0.00	$226,658.13	$5,000.00
60		$0.00	$226,658.13	$5,000.00
61		$0.00	$226,658.13	$5,000.00
62		$0.00	$226,658.13	$5,000.00
63		$0.00	$226,658.13	$5,000.00
64		$0.00	$226,658.13	$5,000.00
65		$0.00	$226,658.13	$5,000.00
66		$0.00	$226,658.13	$5,000.00
67		$0.00	$226,658.13	$5,000.00
68		$0.00	$226,658.13	$5,000.00
69		$0.00	$226,658.13	$5,000.00
70		$0.00	$226,658.13	$5,000.00
71		$0.00	$226,658.13	$5,000.00
72		$0.00	$226,658.13	$5,000.00
73		$0.00	$226,658.13	$5,000.00
74		$0.00	$226,658.13	$5,000.00
75		$0.00	$226,658.13	$5,000.00
76		$0.00	$226,658.13	$5,000.00
77		$0.00	$226,658.13	$5,000.00
78		$0.00	$226,658.13	$5,000.00
79		$0.00	$226,658.13	$5,000.00
80		$0.00	$226,658.13	$5,000.00
81		$0.00	$226,658.13	$5,000.00
82		$0.00	$226,658.13	$5,000.00
83		$0.00	$226,658.13	$5,000.00
84		$0.00	$226,658.13	$5,000.00
85		$0.00	$226,658.13	$5,000.00
86		$0.00	$226,658.13	$5,000.00
87		$0.00	$226,658.13	$5,000.00
88		$0.00	$226,658.13	$5,000.00
89		$0.00	$226,658.13	$5,000.00
90		$0.00	$226,658.13	$5,000.00
91		$0.00	$226,658.13	$5,000.00
92		$0.00	$226,658.13	$5,000.00
93		$0.00	$226,658.13	$5,000.00
94		$0.00	$226,658.13	$5,000.00
95		$0.00	$226,658.13	$5,000.00
96		$0.00	$226,658.13	$5,000.00
97		$0.00	$226,658.13	$5,000.00
98		$0.00	$226,658.13	$5,000.00
99		$0.00	$226,658.13	$5,000.00
100		$0.00	$226,658.13	$5,000.00
Total		$15,082,064.14	$22,439,155.25	$495,000.00

MDH
ENGINEERED SOLUTIONS

Quantities and Cost Estimate Sheet

Contract No.:	R2488
Type of Work:	Dam Raise and Spillway Replacement
Region:	Highfield Dam
Prepared By:	Darrell Mihial
Date:	November 28, 2011

Project Information	
Project Number	Estimated Cost
Total	

Location	From km	To km	Length
		Total	m

Project Description: Cost estimate for Alternative 4, the ungated labyrinth weir on the west side, for the spillway upgrade and dam raising. Assumes embankment elevation of 725.2 masl and a downstream slope of 6:1.

Remarks:

Contract Items

Item No.	Spec. No.	Description	Unit	Quantity	Pre-Tender Price	Pre-Tender Cost	At Award Price	At Award Cost
		Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00		
		Land Acquisition/Control	Hectare		$1,250.00			
		Care of Water	Lump Sum	1	$50,000.00	$50,000.00		
		Stripping	m^2	89 552	$3.50	$313,432.00		
		Earth Excavation and Embankment Construction	m^3	165 000	$5.00	$825,000.00		
		Structure Excavation	m^3	7 000	$22.00	$154,000.00	$50,000.00	
		Haul of Borrow Material	m^3hm	1 310 000	$0.40	$524,000.00		
		Topsoil, Grading and Seeding	m^2	88 300	$8.50	$750,550.00		
		Traffic Gravel	m^3	575	$10.00	$5,748.64		
		Granular Filter	m^3	30 576	$50.00	$1,528,800.00		
		Rip Rap	m^3	19 000	$110.00	$2,090,000.00	$125,000.00	
		Bedding Gravel	m^3	15 000	$40.00	$600,000.00		
		Structure Subdrainage System	Lump Sum	1	$400,000.00	$400,000.00	$250,000.00	
		East Chute and Energy Dissipator						
		- Walls	m^3	525	$1,200.00	$630,000.00		
		- Slab	m^3	1 300	$850.00	$1,105,000.00		
		- Foundation (piles)	m^3	326	$1,200.00	$391,200.00		
		West Chute and Energy Dissipator						
		- Walls	m^3	300	$1,200.00	$360,000.00	$360,000.00	
		- Slab	m^3	850	$850.00	$722,500.00	$722,500.00	
		- Foundation (piles)	m^3	190	$1,200.00	$228,000.00	$228,000.00	
		Reinforced Concrete Labrynth Weir						
		- Walls	m^3	150	$1,200.00	$180,000.00		
		- Slab	m^3	175	$850.00	$148,750.00		
		- Foundation (piles)	m^3	60	$1,200.00	$72,000.00		
		West Outlet Modifications	Lump Sum	1	$150,000.00	$150,000.00		
		Culvert	Lump Sum	1	$350,000.00	$350,000.00	$350,000.00	
		Culvert	Lump Sum	1	$450,000.00	$450,000.00		
		Contract Items Sub-Total				**$12,278,980.64**		

Sundries

Item	Cost
Sundries Total	

Materials			
Material Sub-Total			
Freight/Miscellaneous			
Materials Total			

Budget Items

Budget Type	Year: Prev. Years	2008	2009	Total
1. Internal Budget Target				🟦
2. Pre-Tender Estimate				
3. At Award Budget				

Pre-tender Total Added Project Cost	$4,543,223

	Difference ($)	% Difference
Contract Items At Award vs. Pre-Tender		

Summary of Estimates

Expenditure Category	Pre-Tender	At Award
Contract Items (Sub-Total(not inc. Site Occ.) + OCI)	$12,278,981	
Force Account		
Materials		
Sundries		
Construcion Engineering (10.0%)	$1,227,898	
Design Engineering (7.0%)	$859,529	
Testing Services		
AESB Forces		
ROW Property		
ROW Surveys		
Contingency (20.0%)	$2,455,796	
Total Cost	**$16,822,203**	

Other Contract Items (OCI)

Item	Cost
Other Contract Items Total	

Force Account

Item	Cost
General	
Force Account Total	

MDH ENGINEERED SOLUTIONS
232-1911 E Truesdale Drive
Regina, SK S4V 2N1
Ph: (306) 546-4220
Fax: (306) 546-4262

Highfield Dam Spillway Replacement
Alternative #4
Financial Analysis

Year	Description or Type of Repairs	Capital Costs	Maintenance Costs	Operating Costs
Net Present Value (Discount Rate = 5.00%) = $23,390,149		$17,709,285.32	$5,491,908.18	$188,955.29
1		$17,594,516.39	$0.00	$0.00
2		$0.00	$290,645.90	$10,000.00
3		$0.00	$290,645.90	$10,000.00
4		$0.00	$290,645.90	$10,000.00
5		$0.00	$290,645.90	$10,000.00
6		$0.00	$290,645.90	$10,000.00
7		$0.00	$290,645.90	$10,000.00
8		$0.00	$290,645.90	$10,000.00
9		$0.00	$290,645.90	$10,000.00
10		$0.00	$290,645.90	$10,000.00
11		$0.00	$290,645.90	$10,000.00
12		$0.00	$290,645.90	$10,000.00
13		$0.00	$290,645.90	$10,000.00
14		$0.00	$290,645.90	$10,000.00
15		$0.00	$290,645.90	$10,000.00
16		$0.00	$290,645.90	$10,000.00
17		$0.00	$290,645.90	$10,000.00
18		$0.00	$290,645.90	$10,000.00
19		$0.00	$290,645.90	$10,000.00
20		$0.00	$290,645.90	$10,000.00
21		$0.00	$290,645.90	$10,000.00
22		$0.00	$290,645.90	$10,000.00
23		$0.00	$290,645.90	$10,000.00
24		$0.00	$290,645.90	$10,000.00
25		$0.00	$290,645.90	$10,000.00
26		$0.00	$290,645.90	$10,000.00
27		$0.00	$290,645.90	$10,000.00
28		$0.00	$290,645.90	$10,000.00
29		$0.00	$290,645.90	$10,000.00
30		$0.00	$290,645.90	$10,000.00
31		$0.00	$290,645.90	$10,000.00
32		$0.00	$290,645.90	$10,000.00
33		$0.00	$290,645.90	$10,000.00
34		$0.00	$290,645.90	$10,000.00
35		$0.00	$290,645.90	$10,000.00
36		$0.00	$290,645.90	$10,000.00
37		$0.00	$290,645.90	$10,000.00
38		$0.00	$290,645.90	$10,000.00
39		$0.00	$290,645.90	$10,000.00
40		$0.00	$290,645.90	$10,000.00
41		$0.00	$290,645.90	$10,000.00
42		$0.00	$290,645.90	$10,000.00
43		$0.00	$290,645.90	$10,000.00
44		$0.00	$290,645.90	$10,000.00
45		$0.00	$290,645.90	$10,000.00
46		$0.00	$290,645.90	$10,000.00
47		$0.00	$290,645.90	$10,000.00
48		$0.00	$290,645.90	$10,000.00

49		$0.00	$290,645.90	$10,000.00
50		$0.00	$290,645.90	$10,000.00
51	new concrete spillway, culverts and rip rap	$11,470,074.00	$290,645.90	$10,000.00
52		$0.00	$290,645.90	$10,000.00
53		$0.00	$290,645.90	$10,000.00
54		$0.00	$290,645.90	$10,000.00
55		$0.00	$290,645.90	$10,000.00
56		$0.00	$290,645.90	$10,000.00
57		$0.00	$290,645.90	$10,000.00
58		$0.00	$290,645.90	$10,000.00
59		$0.00	$290,645.90	$10,000.00
60		$0.00	$290,645.90	$10,000.00
61		$0.00	$290,645.90	$10,000.00
62		$0.00	$290,645.90	$10,000.00
63		$0.00	$290,645.90	$10,000.00
64		$0.00	$290,645.90	$10,000.00
65		$0.00	$290,645.90	$10,000.00
66		$0.00	$290,645.90	$10,000.00
67		$0.00	$290,645.90	$10,000.00
68		$0.00	$290,645.90	$10,000.00
69		$0.00	$290,645.90	$10,000.00
70		$0.00	$290,645.90	$10,000.00
71		$0.00	$290,645.90	$10,000.00
72		$0.00	$290,645.90	$10,000.00
73		$0.00	$290,645.90	$10,000.00
74		$0.00	$290,645.90	$10,000.00
75		$0.00	$290,645.90	$10,000.00
76		$0.00	$290,645.90	$10,000.00
77		$0.00	$290,645.90	$10,000.00
78		$0.00	$290,645.90	$10,000.00
79		$0.00	$290,645.90	$10,000.00
80		$0.00	$290,645.90	$10,000.00
81		$0.00	$290,645.90	$10,000.00
82		$0.00	$290,645.90	$10,000.00
83		$0.00	$290,645.90	$10,000.00
84		$0.00	$290,645.90	$10,000.00
85		$0.00	$290,645.90	$10,000.00
86		$0.00	$290,645.90	$10,000.00
87		$0.00	$290,645.90	$10,000.00
88		$0.00	$290,645.90	$10,000.00
89		$0.00	$290,645.90	$10,000.00
90		$0.00	$290,645.90	$10,000.00
91		$0.00	$290,645.90	$10,000.00
92		$0.00	$290,645.90	$10,000.00
93		$0.00	$290,645.90	$10,000.00
94		$0.00	$290,645.90	$10,000.00
95		$0.00	$290,645.90	$10,000.00
96		$0.00	$290,645.90	$10,000.00
97		$0.00	$290,645.90	$10,000.00
98		$0.00	$290,645.90	$10,000.00
99		$0.00	$290,645.90	$10,000.00
100		$0.00	$290,645.90	$10,000.00
Total		$29,064,590.39	$28,773,944.48	$990,000.00

Quantities and Cost Estimate Sheet

MDH ENGINEERED SOLUTIONS

Contract No.: R2488
Type of Work: Dam Raise and Spillway Replacement
Region: Highfield Dam
Prepared By: Darrell Mihial
Date: November 28, 2011

Project Description: Cost estimate for Alternative 5, the gated concrete spillway on the west side, for the spillway upgrade and dam raising. Assumes embankment elevation of 725.5 masl and a downstream slope of 6:1.

Project Information

Project Number	Estimated Cost
Total	

Location	From km	To km	Length
		Total	m

Remarks:

Contract Items

Item No.	Spec. No.	Description	Unit	Quantity	Pre-Tender Price	Pre-Tender Cost	At Award Price	At Award Cost
		Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00		
		Land Acquisition/Control	Hectare		$1,250.00			
		Care of Water	Lump Sum	1	$50,000.00	$50,000.00		
		Stripping	m^2	89 552	$3.50	$313,432.00		
		Earth Excavation and Embankment Construction	m^3	165 000	$5.00	$825,000.00		
		Structure Excavation	m^3	7 000	$22.00	$154,000.00		
		Haul of Borrow Material	m^3hm	1 310 000	$0.40	$524,000.00		
		Topsoil, Grading and Seeding	m^2	88 300	$8.50	$750,550.00		
		Traffic Gravel	m^3	575	$10.00	$5,748.64		
		Granular Filter	m^3	30 576	$50.00	$1,528,800.00		
		Rip Rap	m^3	19 000	$110.00	$2,090,000.00		
		Bedding Gravel	m^3	15 000	$40.00	$600,000.00		
		Structure Subdrainage System	Lump Sum	1	$400,000.00	$400,000.00		
		East Chute						
		- Walls	m^3	525	$1,200.00	$630,000.00		
		- Slab	m^3	1 300	$850.00	$1,105,000.00		
		- Foundation (piles)	m^3	326	$1,200.00	$391,200.00		
		West Chute						
		- Walls	m^3	300	$1,200.00	$360,000.00		
		- Slab	m^3	850	$850.00	$722,500.00		
		- Foundation (piles)	m^3	190	$1,200.00	$228,000.00		
		Reinforced Concrete Labrynth Weir						
		- Walls	m^3		$1,200.00			
		- Slab	m^3	175	$850.00	$148,750.00		
		- Foundation (piles)	m^3	60	$1,200.00	$72,000.00		
		West Outlet Modifications	Lump Sum	1	$150,000.00	$150,000.00		
		Culvert	Lump Sum	1	$350,000.00	$350,000.00		
		Culvert	Lump Sum	1	$450,000.00	$450,000.00		
		Spillway Gates	Lump Sum	2	$300,000.00	$600,000.00		
		metalwork	Lump Sum	1	$200,000.00	$200,000.00		
		Contract Items Sub-Total				**$12,898,980.64**		

Sundries

Item	Cost
Sundries Total	
Materials	
Material Sub-Total	
Freight/Miscellaneous	
Materials Total	

Budget Items

Budget Type	Prev. Years	Year 2008	Year 2009	Total
1. Internal Budget Target				
2. Pre-Tender Estimate				
3. At Award Budget				

Pre-tender Total Added Project Cost	$4,772,623
Contract Items At Award vs. Pre-Tender	Difference ($) / % Difference

Summary of Estimates

Expenditure Category	Pre-Tender	At Award
Contract Items (Sub-Total(not inc. Site Occ.) + OCI)	$12,898,981	
Force Account		
Materials		
Sundries		
Construcion Engineering (10.0%)	$1,289,898	
Design Engineering (7.0%)	$902,929	
Testing Services		
AESB Forces		
ROW Property		
ROW Surveys		
Contingency (20.0%)	$2,579,796	
Total Cost	**$17,671,603**	

Other Contract Items (OCI)

Item	Cost
Other Contract Items Total	

Force Account

Item	Cost
General	

MDH ENGINEERED SOLUTIONS
232-1911 E Truesdale Drive
Regina, SK S4V 2N1
Ph: (306) 546-4220
Fax: (306) 546-4262

Highfield Dam Spillway Replacement
Alternative #4
Financial Analysis

Year	Description or Type of Repairs	Capital Costs	Maintenance Costs	Operating Costs
Net Present Value (Discount Rate = 5.00%) = $25,740,638		$19,044,404.50	$6,318,323.04	$377,910.58
1		$18,549,316.39	$0.00	$0.00
2		$0.00	$334,381.90	$20,000.00
3		$0.00	$334,381.90	$20,000.00
4		$0.00	$334,381.90	$20,000.00
5		$0.00	$334,381.90	$20,000.00
6		$0.00	$334,381.90	$20,000.00
7		$0.00	$334,381.90	$20,000.00
8		$0.00	$334,381.90	$20,000.00
9		$0.00	$334,381.90	$20,000.00
10		$0.00	$334,381.90	$20,000.00
11		$0.00	$334,381.90	$20,000.00
12		$0.00	$334,381.90	$20,000.00
13		$0.00	$334,381.90	$20,000.00
14		$0.00	$334,381.90	$20,000.00
15		$0.00	$334,381.90	$20,000.00
16		$0.00	$334,381.90	$20,000.00
17		$0.00	$334,381.90	$20,000.00
18		$0.00	$334,381.90	$20,000.00
19		$0.00	$334,381.90	$20,000.00
20		$0.00	$334,381.90	$20,000.00
21		$0.00	$334,381.90	$20,000.00
22		$0.00	$334,381.90	$20,000.00
23		$0.00	$334,381.90	$20,000.00
24		$0.00	$334,381.90	$20,000.00
25		$0.00	$334,381.90	$20,000.00
26	new Gates	$1,232,000.00	$334,381.90	$20,000.00
27		$0.00	$334,381.90	$20,000.00
28		$0.00	$334,381.90	$20,000.00
29		$0.00	$334,381.90	$20,000.00
30		$0.00	$334,381.90	$20,000.00
31		$0.00	$334,381.90	$20,000.00
32		$0.00	$334,381.90	$20,000.00
33		$0.00	$334,381.90	$20,000.00
34		$0.00	$334,381.90	$20,000.00
35		$0.00	$334,381.90	$20,000.00
36		$0.00	$334,381.90	$20,000.00
37		$0.00	$334,381.90	$20,000.00
38		$0.00	$334,381.90	$20,000.00
39		$0.00	$334,381.90	$20,000.00
40		$0.00	$334,381.90	$20,000.00
41		$0.00	$334,381.90	$20,000.00
42		$0.00	$334,381.90	$20,000.00
43		$0.00	$334,381.90	$20,000.00
44		$0.00	$334,381.90	$20,000.00
45		$0.00	$334,381.90	$20,000.00
46		$0.00	$334,381.90	$20,000.00
47		$0.00	$334,381.90	$20,000.00
48		$0.00	$334,381.90	$20,000.00

49		$0.00	$334,381.90	$20,000.00
50		$0.00	$334,381.90	$20,000.00
51	new concrete spillway, gates, culverts and rip rap	$12,424,874.00	$334,381.90	$20,000.00
52		$0.00	$334,381.90	$20,000.00
53		$0.00	$334,381.90	$20,000.00
54		$0.00	$334,381.90	$20,000.00
55		$0.00	$334,381.90	$20,000.00
56		$0.00	$334,381.90	$20,000.00
57		$0.00	$334,381.90	$20,000.00
58		$0.00	$334,381.90	$20,000.00
59		$0.00	$334,381.90	$20,000.00
60		$0.00	$334,381.90	$20,000.00
61		$0.00	$334,381.90	$20,000.00
62		$0.00	$334,381.90	$20,000.00
63		$0.00	$334,381.90	$20,000.00
64		$0.00	$334,381.90	$20,000.00
65		$0.00	$334,381.90	$20,000.00
66		$0.00	$334,381.90	$20,000.00
67		$0.00	$334,381.90	$20,000.00
68		$0.00	$334,381.90	$20,000.00
69		$0.00	$334,381.90	$20,000.00
70		$0.00	$334,381.90	$20,000.00
71		$0.00	$334,381.90	$20,000.00
72		$0.00	$334,381.90	$20,000.00
73		$0.00	$334,381.90	$20,000.00
74		$0.00	$334,381.90	$20,000.00
75		$0.00	$334,381.90	$20,000.00
76		$0.00	$334,381.90	$20,000.00
77		$0.00	$334,381.90	$20,000.00
78		$0.00	$334,381.90	$20,000.00
79		$0.00	$334,381.90	$20,000.00
80		$0.00	$334,381.90	$20,000.00
81		$0.00	$334,381.90	$20,000.00
82		$0.00	$334,381.90	$20,000.00
83		$0.00	$334,381.90	$20,000.00
84		$0.00	$334,381.90	$20,000.00
85		$0.00	$334,381.90	$20,000.00
86		$0.00	$334,381.90	$20,000.00
87		$0.00	$334,381.90	$20,000.00
88		$0.00	$334,381.90	$20,000.00
89		$0.00	$334,381.90	$20,000.00
90		$0.00	$334,381.90	$20,000.00
91		$0.00	$334,381.90	$20,000.00
92		$0.00	$334,381.90	$20,000.00
93		$0.00	$334,381.90	$20,000.00
94		$0.00	$334,381.90	$20,000.00
95		$0.00	$334,381.90	$20,000.00
96		$0.00	$334,381.90	$20,000.00
97		$0.00	$334,381.90	$20,000.00
98		$0.00	$334,381.90	$20,000.00
99		$0.00	$334,381.90	$20,000.00
100		$0.00	$334,381.90	$20,000.00
Total		$32,206,190.39	$33,103,808.48	$1,980,000.00

Quantities and Cost Estimate Sheet

MDH ENGINEERED SOLUTIONS

Contract No.:	R2488
Type of Work:	Dam Raise and Spillway Replacement
Region:	Highfield Dam
Prepared By:	Darrell Mihial
Date:	November 28, 2011

Project Description: Cost estimate for Alternative 6, the earth spillway on the west side, for the spillway upgrade and dam raising. Assumes embankment elevation of 725.5 masl and a downstream slope of 6:1.

Project Information

Project Number	Estimated Cost
Total	

Location	From km	To km	Length
		Total	_____ m

Remarks:

Contract Items

Item No.	Spec. No.	Description	Unit	Quantity	Pre-Tender Price	Pre-Tender Cost	At Award Price	At Award Cost
		Mobilization and Demobilization	Lump Sum	1	250,000.00	$250,000.00		
		Land Acquisition/Control	Hectare		$1,250.00			
		Care of Water	Lump Sum	1	$50,000.00	$50,000.00		
		Stripping	m^2	143 152	$3.50	$501,032.00		
		Earth Excavation and Embankment Construction	m^3	165 000	$5.00	$825,000.00		
		Structure Excavation	m^3	20 000	$22.00	$440,000.00		
		Haul of Borrow Material	m^3hm	1 310 000	$0.40	$524,000.00		
		Topsoil, Grading and Seeding	m^2	144 400	$8.50	$1,227,400.00		
		Traffic Gravel	m^3	575	$10.00	$5,748.64		
		Granular Filter	m^3	28 800	$50.00	$1,440,000.00		
		Rip Rap	m^3	18 000	$110.00	$1,980,000.00		
		Bedding Gravel	m^3	15 000	$40.00	$600,000.00		
		Structure Subdrainage System	Lump Sum	1	$500,000.00	$500,000.00		
		Stepped energy Dissipator						
		- Walls	m^3	311	$1,200.00	$373,200.00		
		- Slab	m^3	4 140	$850.00	$3,519,000.00		
		- foudnation	m^3	1 019	$1,200.00	$1,222,800.00		
		Concrete box culvert	Lump Sum	1	$2,000,000.00	$2,000,000.00		
		West Outlet Modifications	Lump Sum	1	$150,000.00	$150,000.00		
		Contract Items Sub-Total				**$15,608,180.64**		

Sundries

Item	Cost
Sundries Total	
Materials	
Material Sub-Total	
Freight/Miscellaneous	
Materials Total	

Budget Items

Budget Type	Prev. Years	2008	2009	Total
1. Internal Budget Target				
2. Pre-Tender Estimate				
3. At Award Budget				

		Difference ($)	% Difference
Pre-tender Total Added Project Cost	$5,775,027		
Contract Items At Award vs. Pre-Tender			

Summary of Estimates

Expenditure Category	Pre-Tender	At Award
Contract Items (Sub-Total(not inc. Site Occ.) + OCI)	$15,608,181	
Force Account		
Materials		
Sundries		
Construcion Engineering (10.0%)	$1,560,818	
Design Engineering (7.0%)	$1,092,573	
Testing Services		
AESB Forces		
ROW Property		
ROW Surveys		
Contingency (20.0%)	$3,121,636	
Total Cost	**$21,383,207**	

Other Contract Items (OCI)

Item	Cost
Other Contract Items Total	

Force Account

Item	Cost
General	
Force Account Total	

MDH ENGINEERED SOLUTIONS
232-1911 E Truesdale Drive
Regina, SK S4V 2N1
Ph: (306) 546-4220
Fax: (306) 546-4262

Highfield Dam Spillway Replacement
Alternative #6
Financial Analysis

Year	Description or Type of Repairs	Capital Costs	Maintenance Costs	Operating Costs
Net Present Value (Discount Rate = 5.00%) = $36,106,792		$27,183,278.49	$8,829,035.83	$94,477.65
1		$26,805,326.60	$0.00	$0.00
2		$0.00	$467,255.28	$5,000.00
3		$0.00	$467,255.28	$5,000.00
4		$0.00	$467,255.28	$5,000.00
5		$0.00	$467,255.28	$5,000.00
6		$0.00	$467,255.28	$5,000.00
7		$0.00	$467,255.28	$5,000.00
8		$0.00	$467,255.28	$5,000.00
9		$0.00	$467,255.28	$5,000.00
10		$0.00	$467,255.28	$5,000.00
11		$0.00	$467,255.28	$5,000.00
12		$0.00	$467,255.28	$5,000.00
13		$0.00	$467,255.28	$5,000.00
14		$0.00	$467,255.28	$5,000.00
15		$0.00	$467,255.28	$5,000.00
16		$0.00	$467,255.28	$5,000.00
17		$0.00	$467,255.28	$5,000.00
18		$0.00	$467,255.28	$5,000.00
19		$0.00	$467,255.28	$5,000.00
20		$0.00	$467,255.28	$5,000.00
21		$0.00	$467,255.28	$5,000.00
22		$0.00	$467,255.28	$5,000.00
23		$0.00	$467,255.28	$5,000.00
24		$0.00	$467,255.28	$5,000.00
25		$0.00	$467,255.28	$5,000.00
26		$0.00	$467,255.28	$5,000.00
27		$0.00	$467,255.28	$5,000.00
28		$0.00	$467,255.28	$5,000.00
29		$0.00	$467,255.28	$5,000.00
30		$0.00	$467,255.28	$5,000.00
31		$0.00	$467,255.28	$5,000.00
32		$0.00	$467,255.28	$5,000.00
33		$0.00	$467,255.28	$5,000.00
34		$0.00	$467,255.28	$5,000.00
35		$0.00	$467,255.28	$5,000.00
36		$0.00	$467,255.28	$5,000.00
37		$0.00	$467,255.28	$5,000.00
38		$0.00	$467,255.28	$5,000.00
39		$0.00	$467,255.28	$5,000.00
40		$0.00	$467,255.28	$5,000.00
41		$0.00	$467,255.28	$5,000.00
42		$0.00	$467,255.28	$5,000.00
43		$0.00	$467,255.28	$5,000.00
44		$0.00	$467,255.28	$5,000.00
45		$0.00	$467,255.28	$5,000.00
46		$0.00	$467,255.28	$5,000.00
47		$0.00	$467,255.28	$5,000.00
48		$0.00	$467,255.28	$5,000.00

49		$0.00	$467,255.28	$5,000.00
50		$0.00	$467,255.28	$5,000.00
51	new concrete, culvert and rip rap	$19,920,201.53	$467,255.28	$5,000.00
52		$0.00	$467,255.28	$5,000.00
53		$0.00	$467,255.28	$5,000.00
54		$0.00	$467,255.28	$5,000.00
55		$0.00	$467,255.28	$5,000.00
56		$0.00	$467,255.28	$5,000.00
57		$0.00	$467,255.28	$5,000.00
58		$0.00	$467,255.28	$5,000.00
59		$0.00	$467,255.28	$5,000.00
60		$0.00	$467,255.28	$5,000.00
61		$0.00	$467,255.28	$5,000.00
62		$0.00	$467,255.28	$5,000.00
63		$0.00	$467,255.28	$5,000.00
64		$0.00	$467,255.28	$5,000.00
65		$0.00	$467,255.28	$5,000.00
66		$0.00	$467,255.28	$5,000.00
67		$0.00	$467,255.28	$5,000.00
68		$0.00	$467,255.28	$5,000.00
69		$0.00	$467,255.28	$5,000.00
70		$0.00	$467,255.28	$5,000.00
71		$0.00	$467,255.28	$5,000.00
72		$0.00	$467,255.28	$5,000.00
73		$0.00	$467,255.28	$5,000.00
74		$0.00	$467,255.28	$5,000.00
75		$0.00	$467,255.28	$5,000.00
76		$0.00	$467,255.28	$5,000.00
77		$0.00	$467,255.28	$5,000.00
78		$0.00	$467,255.28	$5,000.00
79		$0.00	$467,255.28	$5,000.00
80		$0.00	$467,255.28	$5,000.00
81		$0.00	$467,255.28	$5,000.00
82		$0.00	$467,255.28	$5,000.00
83		$0.00	$467,255.28	$5,000.00
84		$0.00	$467,255.28	$5,000.00
85		$0.00	$467,255.28	$5,000.00
86		$0.00	$467,255.28	$5,000.00
87		$0.00	$467,255.28	$5,000.00
88		$0.00	$467,255.28	$5,000.00
89		$0.00	$467,255.28	$5,000.00
90		$0.00	$467,255.28	$5,000.00
91		$0.00	$467,255.28	$5,000.00
92		$0.00	$467,255.28	$5,000.00
93		$0.00	$467,255.28	$5,000.00
94		$0.00	$467,255.28	$5,000.00
95		$0.00	$467,255.28	$5,000.00
96		$0.00	$467,255.28	$5,000.00
97		$0.00	$467,255.28	$5,000.00
98		$0.00	$467,255.28	$5,000.00
99		$0.00	$467,255.28	$5,000.00
100		$0.00	$467,255.28	$5,000.00
Total		$46,725,528.13	$46,258,272.85	$495,000.00