Springbank Off-Stream Storage Project Preliminary Geotechnical Assessment Report

Volume I of 4

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Sign-off Sheet

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Abbreviations

ADSG	Alberta Dam Safety Guidelines
AEP	Alberta Environment and Parks
AEW	Alberta Environment and Water
APEGBC	Association of Professional Engineers and Geologists of British Columbia
ARC	Alberta Research Council
AT	Alberta Transportation
BAP	Best available practice
BP	Before present
BZF	Brazeau Formation
CDA	Canadian Geotechnical Society
CGS	Canadian Dam Association
CPT	Cone Penetration Testing
GSC	Geological Survey of Canada
CSF	Coalspur Formation
FOS	Factor of Safety
GoA	Government of Alberta
IDF	Inflow Design Flood
MA	Mega-annum (Million years)
MC1	McClean Creek Dam
NMD	Nomad Member
PHSA	Probabilistic Seismic Hazard Assessment

PMF	Probable Maximum Flood
PPF	Paskapoo Formation
QPO	Quantitative Performance Objectives
RVC	Rocky View County
SR1	Springbank Offstream Dam and Reservoir
SPT	Standard Penetration Test
USACE	United States Army Corp of Engineers
USBR	United States Bureau of Reclamation
WCSB	Western Canada Sedimentary Basin

1.0 INTRODUCTION

This Preliminary Geotechnical Assessment Report has been prepared by Stantec Consulting Ltd (Stantec) for Alberta Transportation (AT) to support the Preliminary Design and Environmental Impact Assessment (EIA) of the Springbank Off-Stream Storage Project (herein, referred to as the SR1 Project). It has been developed under the Terms of Reference (TOR) 0015997 and subsequent addendums (Government of Alberta (GoA), 2014).

This report summarizes the site characterization and preliminary geotechnical design undertaken for the Springbank Off-Stream Dam and Reservoir (SR1) Project. This project is a flood diversion and storage system proposed by the GoA to divert and temporarily store flood waters from the Elbow River Basin.

The project comprises a diversion structure, a diversion channel, earthfill dam and storage reservoir with no permanent pool. SR1 is designed to operate alongside the Glenmore Dam in Calgary to limit flood flows within the Elbow River downstream of the Glenmore Dam to less than 160 m^3/s during design flood events. During periods of operation, SR1 will divert and convey floodwaters 4 km to an off-stream storage reservoir located to the north of the Elbow River. The water will be retained in the reservoir until the flood event has passed and then will be released back into the river system in a controlled manner.

1.1 TERMS OF REFERENCE

This report contains the information to support the following preliminary design tasks as stated in TOR 0015997 (GoA, 2014):

- Conduct an inspection of the site;
- Review previous surveys and investigations, data, maps, aerial photos, record drawings and reports. Identify any need for additional investigations, studies and reports; and upon the Minister's approval, conduct additional geotechnical investigations and surveys that may be required for Final Design;
- Geotechnical investigations shall be completed to final design level[1];
- Identify major earthworks material sources for the project and conduct sufficient investigations to verify the quantity and quality of these sources;

[1] This has not been completed to the 'final design level' due to land access constraints and information gaps identified during design. This Assessment is based on the primary investigation data only. Recommendations for the Supplementary Investigation is presented in Section 18.0.

- Calculate quantities of surplus materials to be excavated and identify disposal locations;
- Complete the preliminary design of all major project components;
- Provide system operating criteria to be followed in the event of a project design criteria, as well as, operating criteria required to ensure the project is maintained in a state of flood readiness; and,
- Identify utilities and determine their relocation and modification requirements.

1.2 REPORT STRUCTURE

This report is divided into 21 sections with attachments. Sections 2 to 7 describe the geological site conditions and geotechnical characteristics of the SR1 Project Site. Sections 8 to 21 summarize the preliminary geotechnical design of the SR1 Project components.

In addition to the tasks defined in TOR 0015997 (GoA, 2014), this Report has been prepared in accordance with the guiding principles of the Canadian Dam Association (CDA) Dam Safety Guidelines (2007), the CDA Dam Safety Review Technical Bulletin (2016); and the Alberta Dam and Canal Safety Directive (AEP, 2018).

2.0 PROJECT DESCRIPTION

2.1 LOCATION

The SR1 Project Site is located within the Municipal District of Rocky View County (RVC) and is approximately 15 km west of the Calgary city limits and 10 km south of Cochrane (Figure 1). The Diversion Structure is located approximately 34 km upstream of the Glenmore Dam. The Storage Reservoir is located approximately 29 km upstream of the Glenmore Dam.

2.1.1 Coordinate System

The Coordinate System for the SR1 project is NAD 1983 3TM 114. The vertical datum is NGVD 1988.

2.1.2 Project Locations

Locations within the SR1 Project Site are defined using two approaches:

- Quarter-section Method: one-square-mile (2.6 km^2) sections defined by the Dominion Land Survey (DLS) in Western Canada. Each quarter-section within the SR1 Project Site was assigned a Land Parcel Number; and,
- Stations: The linear components of the dam system were assigned component specific station points. The Floodplain Berm was assigned Stations 0+500 to 1+900 m; the Diversion Channel was assigned Stations 10+000 to 14+700 m; and the Storage Dam was assigned Stations 20+000 to 24+000 m.

2.2 PHYSIOGRAPHY

2.2.1 Classification

The SR1 Project Site is located in the Eastern Foothills of the Rocky Mountains. This is part of the Okotoks Uplands District of the Western Benchlands Section of the Southern Alberta Uplands Physiographic Region (Pettapiece, 1986). This region is characterized by NNW–SSE trending ridges with intervening valleys. The Elbow River bounds the southern limit of the SR1 Project Site and comprises a broad post-glacial river valley and flood plain. The SR1 project site is bisected by Highway 22 and associated township roads. The original ground (OG) elevation ranges between approximately Elevation 1244 m at the western end of the diversion channel to Elevation 1185 m at an Unnamed Creek, labeled herein as the east unnamed creek, meandering through the reservoir footprint.

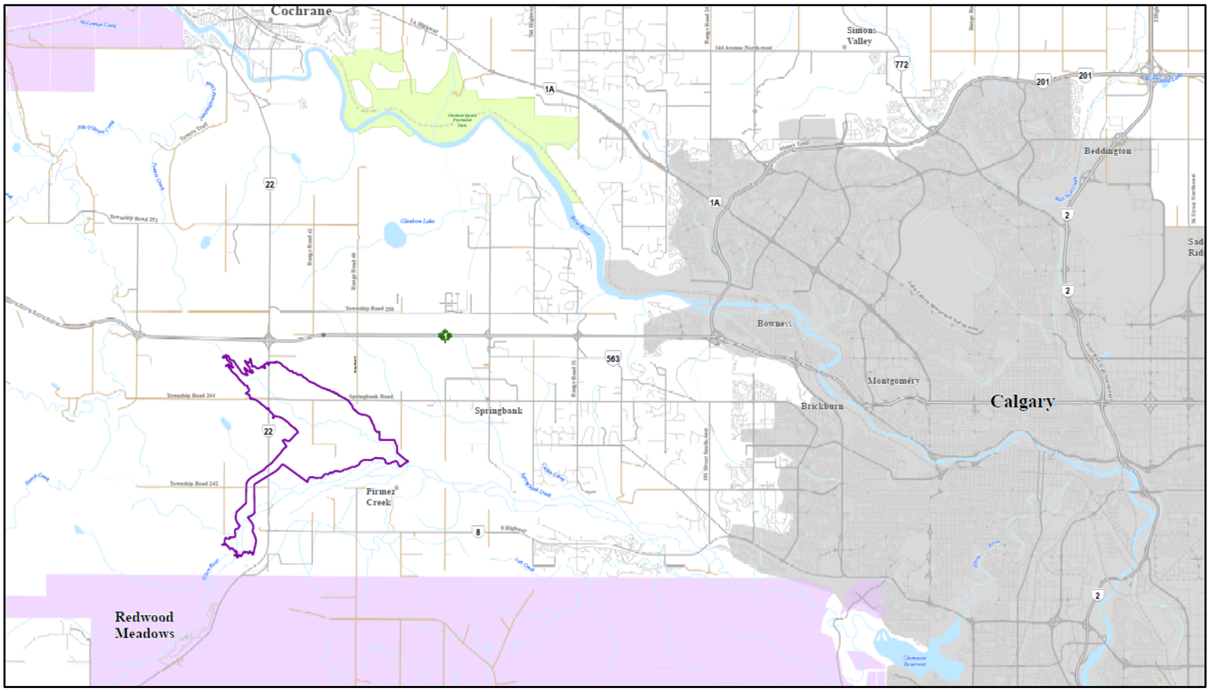


Figure 1. Location of the SR1 Project Site

2.2.2 Watercourses

The Elbow River is a tributary of the Bow River in the South Saskatchewan River basin in Southern Alberta. It originates at the Rae Glacier on the eastern slopes of the Rocky Mountains and flows 120 km to its confluence with the Bow River in downtown Calgary. The river drops approximately 1,062 m along its course, making it one of the steepest of its size in Alberta. Upstream of the SR1 Project Site, the Elbow River flows eastwards through the montane, alpine, and sub-alpine terrain of the Front Ranges and Foothills. It is a steep, single-thread stream at its headwaters in the Front Ranges then transitions into a multi-threaded, meandering river.

Two unnamed tributary creeks are present within the SR1 Project Site:

- A western creek flows in a NW-SE direction through the diversion channel at Station 12+200 m; and,
- The east unnamed creek flows in a NW-SE direction through the reservoir footprint.

2.3 SITE SELECTION

2.3.1 Feasibility Study

The site for the SR1 Project was proposed by Amec (2014) as part of their feasibility study for flood mitigation measures for the Bow River, Elbow River, and Oldman River basins.

2.3.2 Conceptual Design

Amec developed a conceptual design for the SR1 Dam System(2014). The key geotechnical components are summarized below:

2.3.2.1 Earthfill Dam

This comprised a 3 km long earthfill storage dam with a maximum height of 24 m. The embankment slopes were 3H:1V with 6 m wide berms *'resulting in average dam slopes of between 3H:1V and 4H:1V'* constructed with '*an impervious fill zone 1A compacted clay core and random compacted zone 2A fill upstream and downstream shells*'.

It was reported that the dam foundation would comprise 'a combination of lacustrine clay and clay till' and that 'previous experience with similar low to medium plastic soil subgrades indicates that subgrade deformations or increase in pore-water pressure due to embankment construction are not limiting factors for typical rates of embankment construction'.

The dam would comprise a zoned earthfill structure. It was reported that the locally available 'medium plastic to low plastic lacustrine clay and clay till soil will provide suitable borrow material for constructing the impervious fill zone 1A compacted core'; and that 'slope angles of 3H:1V for slopes formed of random zone 2A fill will provide adequate minimum factor of safety against slope instability for the approximately 24 m height of the main embankment – for an unsaturated slope condition'.

2.3.2.2 Diversion Channel

It was reported that the 'material excavated from the diversion channel will consist mostly of lacustrine silty clay and clayey silt, silty clay till, and bedrock of the Brazeau and Porcupine Hills Formations'; that 'the lacustrine and till deposits predominately consist of medium plastic silty clays with occasional instances of either low plastic or high plastic clays' and that it 'should be recognized that the number of boreholes drilled to date was limited to five locations due to restricted land access'. The 'bedrock in the project area generally consists of inter-bedded mudstone, siltstone and sandstone'.

It was assumed that 'slopes excavated to an angle of 3H:1V or flatter will provide a minimum 1.5 factor of safety against slope instability, assuming a 25 m high slope and considering that less than about 40 percent of slope height is below the groundwater table'.

2.3.2.3 Diversion Structure

It was reported that the 'fluvial sand and gravel deposits in the river channel will provide a stable subgrade both to support the diversion structure foundations, and to provide resistance to lateral loads during flood events. Local lacustrine clay and clay till deposits excavated from the adjacent diversion channel are generally of medium plasticity, and are suitable for use in constructing low permeability compacted backfill for headwalls and wing walls that extend into adjacent embankments or native soil abutments'.

2.3.2.4 Floodplain Berm

This would comprise a 'zoned fill with an impervious fill zone 1A compacted clay core and random compacted 2A fill upstream and downstream shells. Available local medium plastic to low plastic clay and clay till soil will provide suitable borrow material for constructing the impervious 1A compacted core. Local clay soil, as well as reworked bedrock or other excavated materials from the embankment subgrade or diversion channel excavation, will provide suitable material for construction of the upstream and downstream random fill zone 2A shells'.

The foundation was assumed to 'consist of a combination of fluvial sand and gravel deposits and clay/clay till soil. Removal of fine sand or silt overbank materials in the upper portion of the subgrade may be required in some areas prior to placing embankment fill to limit potential for piping below the embankment'.

2.4 PRELIMINARY DESIGN

Stantec has developed this preliminary design of the SR1 Project using the Conceptual Design (AMEC, 2014) as the basis for design. The key components of the current SR1 preliminary design used as the basis for this Preliminary Geotechnical Assessment are summarized in Section 2.4.2.

2.4.1 Design Flood Event

The SR1 Project is designed to meet the following criteria:

- Provide temporary storage capacity for flood waters similar to what was estimated during the 2013 flood event. The estimated peak flow during the 2013 event was 1,240 m^3/s; and,
- Reduce flows downstream of the Glenmore Reservoir to 170 m^3/s based on available flood storage of 10 M m^3 at Glenmore Reservoir.

2.4.2 General Arrangement

The SR1 Project will be constructed off-stream to the north of the Elbow River Valley. The components of the SR1 Project are presented in Figure 20-1 and will comprise the following[2]:

- Floodplain berm;
- Auxiliary spillway;
- Diversion Service spillway;
- Diversion inlet:
- Diversion channel linking the diversion structures to the reservoir;
- New highway bridge on Township Road 242;
- New highway bridge on Highway 22;
- Emergency spillway;
- Saddle dam;
- Diversion channel outlet;
- Off-stream storage earthfill dam.
- Reservoir area; and,
- Low level Outlet located in upland area near the east unnamed creek.

The SR1 project will also require the relocation of existing utilities, pipelines and highway assets within the project limits.

2.4.3 Dam Consequence Classification

A dam breach inundation study was completed and is provided with the Preliminary Design Report. This study evaluated potential failure scenarios and the consequences of failure of the Off-stream Storage Dam and the Diversion Structure as individual dams.

The Off-stream Storage Dam breach analysis results identify thousands of residential and commercial properties within the inundation zone. Based on the size of the population at risk a Hazard Classification of "Extreme" is justified for the Off-stream Storage Dam.

[2] Listed in an upstream to downstream flow direction.

Failure of the Diversion Structure during a flood event would produce minimal increases in discharge and water surface elevation. However, the breach wave caused by a failure of the Diversion Structure may carry concentrated debris that could damage Highway 22 which is located a short distance downstream. Based on the potential for high economic losses affecting infrastructure, a dam class of "High" is justified for the Diversion Structure.

3.0 SITE CHARACTERIZATION ACTIVITIES

The site characterization of the SR1 Project has been undertaken in accordance with the Alberta Dam and Canal Safety Directive (AEP, 2018); the Canadian Day Association (CDA) Dam Safety Guidelines (2007) and the Professional Practice Guideline 'Site Characterization for Dam Foundations in BC (APEGBC, 2016)[3]. This section summarizes the site characterization activities undertaken between 2014 and 2018.

3.1 DESK STUDY REVIEW

A desk-study was undertaken prior to the commencement of the 2016 field program. This was intended to develop a regional-scale site geology model (SGM), define the scope for the field program and support the preliminary geotechnical design. This comprised the following data sources.

3.1.1 Alberta Geological Survey Data

The geology of the SR1 project site has been mapped historically by the Alberta Research Council (ARC) and subsequently the Alberta Geological Survey (AGS) in partnership with the Alberta Energy Regulator (AER). The following AGS maps were reviewed for this report:

- Sheet 150: Surficial Geology of Alberta Foothills and Rocky Mountains (AGS, 1980);
- Sheet 204: Calgary Urban Area (generated from the original map in Moran, 1986);
- Sheet 207: Quaternary Geology of Southern Alberta (Shetsen, 1987);
- Sheet 236: Bedrock Geology of Alberta (Hamilton et al, 1999);
- Sheet 560: Geology of the Alberta Rocky Mountains and Foothills (Pană and Elgr, 2013);
- Sheet 600: Bedrock Geology of Alberta (Prior et al, 2013);
- Sheet 601: Surficial Geology of Alberta (Fenton et al, 2013);
- Sheet 602: Bedrock Topography (MacCormack et al, 2015b);
- Sheet 603: Sediment thickness (MacCormack, et al, 2015a); and,

[3] This document was developed following the independent review of the Mount Polley Tailings Dam Failure in 2014. This document outlines the appropriate standard of practice for all types of dams during the various phases of development, from conceptual through to design, construction, design updates and closure. This can be applied throughout Canada. Stantec consider this document to be a BAP.

- Sheet 604: Glacial landforms in Alberta (Atkinson, 2014).

3.1.2 Seismic Data

The following seismic data was reviewed for the Probabilistic Seismic Hazard Assessment (PSHA) for the SR1 Project:

- Alberta earthquake catalogue: September 2006 through December 2010 (Stern et al, 2013);
- Geological Survey of Canada (GSC) Open File 7576: 5th generation seismic hazard model input files as proposed to produce values for the 2015 national building code of Canada (Halcuk et al, 2014); and,
- Major Dams Seismic Hazard Assessment Report (Klohn Crippen Berger, 2007)

3.1.3 Published Literature and Case Studies

This Report cites a number of published reports, peer-reviewed technical papers and conference proceedings. A full list of these references can be founded in Section 19.0.

3.1.4 Terrain Mapping

The terrain and landforms associated with SR1 project site and surrounding area were reviewed using the following data sources:

- Bare-earth LiDAR (1 m and 15 m pixels) supplied by Airborne Imaging flown November 1, 2015;
- Bare-earth 1:20,000 Provincial Digital Elevation Model (DEM); and,
- Historical aerial photographs from 1927, 1949, 1962, 1974 and 1982.

3.2 SITE RECONNAISSANCE AND MAPPING

Commensurate with the start of the field program in 2016, the geotechnical team performed a series of site walkovers and geological mapping exercises.

3.2.1 Geological Mapping

Eighteen (18) outcrops within the SR1 Project Site were mapped during the 2016 field program (Figure 20-2). These are summarized in Table 1. Geological mapping was used to determine the lateral and vertical distribution of lithological units; orientation of the bedding planes; the number, orientation and spacing of jointing; and an estimate of the Geological strength index (GSI) of the exposed rock mass.

Table 1. Summary of Outcrops

Outcrop ID	Coordinates (UTM)	Description
OC1	5655614 N 676593 E	Approximately 120 m long outcrop along the northwest bank of the Elbow River; upstream of the Diversion Structure Inlet. Sub-vertical bedding of the Brazeau Formation and overlying glacigenic units are exposed.
OC2	5655763 N 676706 E	180 m section of outcrop along the northwest bank of the Elbow River at the location of the Diversion Structure Inlet. Sub-vertical bedding of the Brazeau Formation and overlying glacigenic units are exposed.
OC3	565595 N 676823 E	200 m section of outcrops along the northwest bank of the Elbow River; downstream of the Diversion Structure Inlet. Heavily folded rock mass and sub-vertical bedding of the Brazeau Formation and overlying glacigenic units are exposed. Possible location of the Brazeau Thrust.
OC4	5656148 N 677015 E	50 m section of outcrops along the Elbow River to the west of the Highway 22 Bridge. Sub-horizontal bedding of sandstone units within Brazeau Formation and overlying glacigenic units are exposed.
OC5	565631 N 677217 E	150 m long outcrop along the Elbow to the west of the Highway 22 Bridge Sub-horizontal bedding of sandstone units within Brazeau Formation and overlying glacigenic units are exposed.
OC6	5659203 N 677544 E	200 m long road cutting on the east side of Highway 22. The Brazeau-Coalspur Formation boundary (Entrance conglomerate) is exposed.
OC7	5658900 N 677717 E	150 m long outcrop of Coalspur Formation on a NW-SE trending ridge
OC8	5656948 N 677904 E	120 m section of outcrops along the north bank of an old channel on the Elbow River. Sub-horizontal bedding of sandstone units within Brazeau Formation and overlying glacigenic units are exposed.
OC9	565705 N 678080 E	70 m section of outcrops along the north bank of the Elbow River. Sub-horizontal bedding of sandstone units within Brazeau Formation and overlying glacigenic units are exposed.
OC10	5657084 N 678253 E	200 m section of outcrops along the north bank of the Elbow River. Sub-horizontal bedding of sandstone units within Brazeau Formation and overlying glacigenic units are exposed.
OC11	5657152 N 678608 E	220 m section of outcrops along the north bank of the Elbow River. Sub-horizontal bedding of sandstone units within Brazeau Formation and overlying glacigenic units are exposed.

Outcrop ID	Coordinates (UTM)	Description
OC12	565822 N 679352 E	130 m long outcrop of Coalspur Formation on a NW-SE trending ridge
OC13	5658003 N 680717 E	200 m long outcrop along the north bank of the Elbow River. Sub-horizontal bedding of Paskapoo Formation is exposed.
OC14	5658269 N 681003 E	300 m section of outcrops and landslides along north bank of an old channel on the Elbow River. Glacial lacustrine and glacial till units are exposed.
OC15	5658361 N 681461 E	220 m section of outcrops and landslides along the Elbow River. Glacial lacustrine and glacial till units are exposed.
OC16	5658358 N 681721 E	150 m section outcrops and landslides along the Elbow River. Glacial lacustrine and glacial till units (included the basal grey till) are exposed.
OC17	5658325 N 681955 E	50 m long outcrop along the Elbow River. Glacial lacustrine and glacial till units (included the basal grey till) are exposed.
OC18	565850 N 682203 E	20 m long outcrop along the East Unnamed Creek. Fluvial lag deposits are exposed.

3.2.2 Laser Scanning

The outcrops along the Elbow River to the west of Highway 22 (OC 1 to OC5) were scanned using terrestrial LIDAR. This was used to develop a digital outcrop model (DOM) where access was difficult for the geological mapping of the outcrops and undertake a rock mass assessment.

The outcrops were scanned using a C10 scanner. This instrument has a scan rate of 50,000/sec with each measurement having a quoted positional accuracy of 4-6 mm and simultaneously captures full color photos that are mapped onto the data for visualization. Scan locations were set up at 50 m spacing on the opposite bank of the Elbow River to ensure coverage. Data processing was undertaken using Cyclone and visualization of the 3D point cloud models was accomplished using TruView and ReCap.

3.3 FIELD PROGRAMS

3.3.1 2014 Feasibility Study

This geotechnical investigation was undertaken by Amec[4] between March and May 2014 as part of their feasibility study into flood protection projects on the Elbow River. The investigation was comprised of six (6) boreholes drilled at accessible sites within the project footprint (Figure 20-3). This was undertaken using solid- and hollow-stem auger methods. No rotary coring was performed. Laboratory testing included moisture content, Atterberg Limits, particle size distribution and water soluble sulfates testing.

[4] Now known as AmecFosterWheeler.

The findings of this investigation was summarized in Section 5 of the report by Amec Foster Wheeler (2014). An extract is presented below:

> *'The findings of the current preliminary geotechnical field investigation program indicate that subsurface soils in the area of the proposed diversion channel, earthen dam structure and reservoir at the Springbank site generally consist of medium plastic clay and clay till soil underlain by bedrock consisting of interlayered mudstone, sandstone and siltstone. Subsurface materials underlying the proposed diversion structure system are expected to consist primarily of fluvial sand and gravel deposits, while the subgrade underlying the floodplain berm is expected to consist of a mixture of clay, silt, sand and gravel soils. The soils encountered during the field investigation would be expected to be suitable as foundation materials for embankments and structures associated with the development. The clay and clay till soil are also suitable for use in embankment construction of the floodplain diversion channel fills and the main reservoir embankment."*

3.3.2 2016 Geotechnical Investigation

3.3.2.1 Field Program Approach

The field work plan was prepared by Stantec (2016) and was scheduled to accommodate the requirements of the Land Access Requirements negotiated between the AT and the landowners within the SR1 Project Site and restricted access periods (RAP) in the Elbow River.

3.3.2.2 Completed Activities

The initial field program started on March 21, 2016 and was completed on August 25, 2016. The laboratory testing was completed by December 2016. The fieldwork completed included:

- 135 boreholes using auger, sonic, ODEX and rotary coring;
- 20 Cone Penetration Test (CPT) locations at the dam and diversion channel footprint; and,
- Seismic refraction survey and Multichannel Analysis of Surface Waves (MASW) survey at the Diversion Structure and Low Level Outlet locations.

The location of the exploratory holes undertaken are summarized in Figures 20-4a and 20-4b, and the full results of the 2016 Geotechnical Investigation provided in the "Geotechnical Investigation Report," (2016) included in Attachment 3.1.

The factual data (borehole records, cone penetration testing (CPT) report, laboratory testing results, and geophysical survey reports) collected from this investigation was issued to AT in December 2016 in a Draft Geotechnical Investigation Report (Stantec, 2016). This report is included in Attachment 3.

The site works were supervised by Stantec personnel. All boreholes were logged on site by Stantec personnel using the Modified Unified Soils Classification System (MUSCS). The methodology for the investigation is presented below.

3.3.2.3 Auger Drilling

One hundred and seventeen (117) boreholes across the project site were drilled by All-Service Drilling Ltd using either hollow- or solid-stem auger methods. A maximum of three (3) drilling rigs were on site at any one time. The following rigs were used:

- Acker Soil-Max Junior: drill rig is mounted on 24" tracks and is set up for hollow- and solid-stem augering, direct push and rotary core operations. The length of the rig is 6.9 m (22.6 feet) with a mast-down height of 2.9 m (9.6 ft.).

- Acker Soil-Max: drill rig is mounted on a Go-Tract 1600 Carrier with 36" wide tracks and is set up for hollow- and solid-stem augering and rotary core operations. The length of the rig is 7.6 m (25 feet) with a mast-down height of 3.5 m (11.4 ft.);

- Diedrich D50: drill rig is mounted on 24" tracks and is set up for hollow- and solid-stem augering and rotary core operations. The length of the rig is 6.9 m (22.6 ft.) with a mast-down height of 2.9 m (9.6 ft.); and,

- Strata Start 10: drill rig is mounted on an International 4400 Truck and is set up for hollow- and solid-stem augering and rotary core operations. The length of the rig is 7.6 m (25 ft.) with a mast-down height of 3.7 m (12 ft.).

The intervals of sampling varied depending on the encountered unit, location and (in some cases) land access restrictions. The following sampling methodology was used for these boreholes:

- Undisturbed Shelby tube samples were obtained to provide specimens for laboratory testing. Sampling was in accordance with ASTM D1587 'Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes'. After the tube was removed from the boring, the Stantec field representative measured the recovery, visually classified the soil, and recorded the information on the boring logs;

- Standard Penetration Testing (SPT) sampling (continuous or at specific intervals) was performed to characterize soil stiffness and relative density. SPT specimens were used for subsequent laboratory index testing to assist in characterizing the soil profiles. The sampling was performed in accordance with ASTM D1586 'Standard Test Method of Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils'; and,

- Bulk samples of representative auger cuttings were collected and bagged for use in the laboratory testing program.

All collected samples were stored in moisture-tight containers and delivered daily to the Stantec laboratory in Calgary for testing.

The borehole logs developed from the auger drilling are presented in the Geotechnical Investigation Report (Stantec, 2016).

3.3.2.4 Sonic Coring

Sonic drilling methods were used in four (4) boreholes completed by Mobile Augers Ltd. This method facilitated the recovery of a continuous core in the glacigenic units beneath the proposed dam footprint. The borehole logs developed from the sonic coring are presented in the Geotechnical Investigation Report (Stantec, 2016).

3.3.2.5 ODEX

ODEX methods were used for drilling fourteen (14) boreholes, mostly within the fluvial gravel and cobble deposits associated with the Elbow River. ODEX is a trade-marked, down-hole air hammer system that advances a casing as the hole is drilled. Sampling was undertaken at depths by removing the eccentric drill bit and leaving the casing in place. When the borehole reached the target depth, the casing was retrieved and could be reused. This system does not use drilling mud and reduces the risk of borehole caving in fluvial channel gravels and cobbles.

The borehole logs developed from the ODEX drilling are presented in the Geotechnical Investigation Report (Stantec, 2016).

3.3.2.6 Rotary Coring

Ninety-seven (97) of the boreholes drilled using auger, ODEX and sonic methods were extended into bedrock using rotary coring. All-Service Drilling Ltd. used HQ3 triple-tube, wireline rock coring equipment to recover rock core specimens for logging and testing. This system uses a cable to retrieve the core barrel, which avoids the need for connecting rods. The core barrel assembly has inner and outer tubes. The inner tube collects the rock core sample during drilling and is independent of the outer tube. Coring was completed in accordance with the ASTM D2113 'Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation'.

The borehole logs developed from the rotary coring are presented in the Geotechnical Investigation Report (Stantec, 2016).

3.3.2.7 Cone Penetration Testing

CPTs were completed by ConeTec at twenty (20) locations around the dam footprint and in the diversion channel. Testing involved pushing a cone penetrometer into the ground at a constant rate to provide a continuous subsurface soil profile. The cone tip resistance (q_t), pore-water pressure (u), and sleeve friction (f_s) are measured as the cone is advanced. CPT was completed in accordance with the ASTM Standard D5778-07, 'Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils'. Pore water dissipation and seismic shear wave velocity tests were performed at selected locations.

The report supplied by ConeTec upon completion of the work is presented in the Geotechnical Investigation Report (Stantec, 2016).

3.3.2.8 Exploratory Hole Locations

The actual exploratory hole locations are presented in Figures 20-4a through 20-43. The boreholes are referenced by the SR1 project components as follows:

- D# – Storage Dam;
- DC# – Diversion Channel;
- DS# – River Structures (Service Spillway and Diversion Inlet);
- FB# – Floodplain Berm;
- BS# – Borrow Source;
- DB# - Debris Deflection Barrier;
- H# – Highway embankment and bridge; and,
- GW# – Groundwater well.

The borehole identification system follows a numerical sequence. Several boreholes proposed in the original Work Plan were deleted due to land access issues encountered during the program. The boreholes have not been renumbered, so there are some gaps in the numbers assigned to the completed boreholes.

3.3.3 2018 Supplemental Geotechnical Exploration

3.3.3.1 Completed Activities

A supplemental field exploration was performed in 2018 in two (2) mobilizations. The first mobilization was between April 21 and May 9, 2018. The first mobilization consisted of three (3) boreholes within the Elbow River (DB1 to DB3) for the proposed Debris Barrier and 11 boreholes and 6 Seismic Cone Penetration Test (SCPT) soundings within the dam footprint to assess proposed Low-Level Outlet alignment options.

The second mobilization was between September 24 and October 31, 2018. The second mobilization consisted of four (4) boreholes to further characterize the glaciolacustrine and glacial till units within the dam footprint, two (2) boreholes to assess an alternate LLO alignment and 14 test pits and trenches throughout the dam footprint. Delays were encountered during the second mobilization fieldwork due to inclement weather which required demobilization of the test-pitting excavator on October 4, 2018 and re-mobilizing on October 29, 2018.

The fieldwork completed for this scope of work includes:

- Three (3) boreholes within the Elbow River completed using solid stem auger, and rotary coring for the Debris Barrier.
- Thirteen (13) boreholes completed using solid stem auger, and rotary coring for the Low Level Outlet and characterization of glaciolacustrine materials.
- Six (6) SCPT soundings within the dam footprint along the proposed Low Level Outlet alignments.
- Fourteen (14) test pits and test trenches completed using a track mounted excavator within the dam footprint and LLO locations.

The location of the exploratory boreholes are summarized in Figure 20.4a through 20-4e, and full results of the 2018 geotechnical investigation are provided in the "Supplementary 2018 Geotechnical Investigation Report", 2019 included in Attachment 3.2.

3.3.3.2 Auger Drilling

For the 2018 drilling, two (2) drilling rigs were utilized; only one (1) drilling rig was on-site at any given time. The following rigs were used:

- Acker Soil-Max Junior: drill rig is mounted on 24" tracks and is set up for hollow and solid-stem augering, direct push, and rotary core operations. The length of the rig is 6.9 m (22.6 feet) with a mast-down height of 2.9 m (9.6 ft).

- Acker Soil-Max: drill rig is mounted on a Go-Tract 1600 Carrier with 36" wide tracks and is set up for hollow and solid-stem augering, and rotary core operations. The length of the rig is 7.6 m (25 feet) with a mast-down height of 3.5 m (11.4 ft);

The intervals of sampling varied depending on the encountered unit, location, and purpose of the borehole. The following sampling methodology was used for these boreholes:

- Undisturbed Shelby tube samples were obtained to provide specimens for laboratory testing. Sampling was in accordance with ASTM D1587 'Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes'. After the tube was removed from the boring, the Stantec field representative measured the recovery, visually classified the soil, recorded the information on the boring logs, and sealed the tube to retain moisture.

- Standard Penetration Testing (SPT) sampling (continuous or at specific intervals) was performed to characterize soil stiffness and relative density. SPT specimens were used for subsequent laboratory index testing to assist in characterizing the soil profiles. The sampling was performed in accordance with ASTM D1586 'Standard Test Method of Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils'.

- Bulk samples of representative auger cuttings were collected and bagged for use in the laboratory testing program.

All collected samples were stored in moisture-tight containers and delivered daily to the Stantec laboratory in Calgary for testing. Where bedrock was encountered during augering and SPT's, it has been identified as inferred on the borehole records.

3.3.3.3 Rotary Coring

All three (3) boreholes completed for the Debris Barrier and three (3) of the LLO boreholes were extended into bedrock using rotary coring following overburden drilling using a combination of solid and hollow-stem augering. All-Service Drilling Ltd. used HQ3 triple-tube, wireline rock coring equipment to recover rock core specimens for logging and testing. This system uses a cable to retrieve the core barrel, which avoids the need for connecting rods. The core barrel assembly has inner and outer tubes. The inner tube collects the rock core sample during drilling and is independent of the outer tube. Coring was completed in accordance with the ASTM D2113 'Standard Practice for Rock Core Drilling and Sampling of Rock for Site Investigation'.

3.3.3.4 Test Pitting

Test pitting was split into two mobilizations due to a significant snow event on October 2, 2018 that restricted access to site; the snowfall event was followed by a warming trend which resulted in soft ground conditions and required equipment demobilization to prevent unnecessary damage to access infrastructure.

Test pitting was carried out using a track mounted John Deere 135D excavator owned and operated by KLS during the first mobilization; a track mounted John Deere 210G rented and operated by GWRL was used during the second mobilization. Test pitting was carried out to the maximum reach of the excavator (typically 4.5 m), or the presence of sloughing materials which impeded further excavation. During test pitting, the topsoil was stripped and stockpiled for replacement following backfill of the test pit.

Crews entered the test pit while the excavation was shallower than 1.5 m in height, and soil conditions permitted. Following advancement beyond 1.5 m Stantec personnel logged the soil from the bucket of the excavator and spoil pile. Upon reaching the base of the test pit the excavation was backfilled with excavated materials and used a bucket to lightly compact materials.

3.3.3.5 Cone Penetration Testing

CPTs were completed by ConeTec at six (6) locations within the dam footprint and were completed adjacent to LLO boreholes. Testing involved pushing a cone penetrometer into the ground at a constant rate to provide a continuous subsurface soil profile. The cone tip resistance (qt), pore-water pressure (u), and sleeve friction (fs) are measured as the cone is advanced. CPT was completed in accordance with the ASTM Standard D5778-07, 'Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils'. Pore water dissipation and seismic shear wave velocity tests were performed at selected locations.

3.3.4 Laboratory Testing

Laboratory tests (Table 2) were conducted on selected soil and rock samples at the Stantec laboratory in Calgary. Advanced rock testing was undertaken on selected rock cores by Trican Well Service Ltd. Direct simple shear testing was undertaken on selected Shelby tube samples of the glaciolacustrine and glacial till deposits by Tetra Tech Inc. The test results are presented in the Geotechnical Investigation Report (Stantec, 2016) and the Supplemental 2018 Geotechnical Investigation Report (Stantec, 2018).

Table 2. List of Laboratory Testing

Test	Standard
Moisture content	ASTM D2216 / CSA A23.2-11A
Particle size distribution (sieve analysis and hydrometer)	ASTM D422
Atterberg limits	ASTM D4818 Method B – 1 Point
Hydraulic Conductivity test, flexible wall/falling head	ASTM D5084
Unit weight	ASTM D2166
Unconsolidated undrained triaxial	ASTM D2850
Consolidated undrained triaxial with Pore Pressure Measurements	ASTM D4767
Swell test	ASTM 4546 Method C
Direct shear	ASTM 3080
1D consolidation	ASTM D2435
Standard proctor	ASTM D698
Water soluble Sulphates	CSA A23.2-2B & 38
Specific gravity	ASTM D854
Crumb test	ASTM D6572
Double hydrometer	ASTM D4221
Pinhole test	ASTM D4647
Unconfined compressive strength test	ASTM D2938
Unconfined compressive strength with strain measurements	ASTM D2166
Point load test	ASTM D5731
Slake durability	ASTM D4644

3.3.5 Geophysical Survey

Seismic refraction and Multi-channel analysis of surface wave (MAWS) surveys were completed by DMT Geoservices Ltd at the diversion structure and low level outlet. The methodology, survey locations and results of the surveys can be found in the Geotechnical Investigation Report (Stantec, 2016).

3.3.6 Hydrogeological Investigation

The hydrogeological components of the intrusive investigation are discussed below and summarized in Figures 21-5a and 21-5b. The in-situ testing comprised:

- Standpipe piezometers were installed in 35 locations. Thirty of these were single well installations with five (5) comprising an upper and lower nested well;
- 37 single packer hydraulic conductivity tests were conducted in five (5) boreholes to determine the hydraulic conductivity of the bedrock. The equipment used for these tests, provided by All Service Drilling Ltd., consisted of pneumatic packer assembly and related accessory equipment. Surface calibration tests were completed on the equipment at the start of each test to determine the friction loss in the system. The tests were completed at the base of the borehole, as the borehole was advanced;
- 10 rising head tests were undertaken by Stantec Personnel to estimate the hydraulic conductivity adjacent to the well completion elevations; and,
- 30 Pore Pressure Dissipation Tests (PPDT) were undertaken during 15 of the CPT soundings.

3.3.7 Packer Testing

Packer permeability testing was carried out within the Debris Barrier boreholes (DB1 to DB3) to determine the permeability of the bedrock within the Elbow River. The equipment used for these tests, provided by All Service Drilling, consisted of pneumatic packer assembly and related accessory equipment. Due to complications with the equipment the packer seal would release part way through testing, and as such only five (5) of ten (10) test intervals provided valid permeability information. All Service attempted to diagnose the issues with the packer assembly, however they were unable to diagnose the main cause of the issue and packer testing was abandoned.

The tests were completed in a zone at the base of the borehole, as the borehole was advanced. The bedrock was tested in approximately 3 m long increments as the borehole was advanced. The results of these individual tests that were deemed a valid test are presented in the Supplemental 2018 Geotechnical Investigation Report (Stantec, 2018).

3.3.8 Groundwater Monitoring

In addition to the eight (8) standpipe piezometers previously installed on GoA lands, two (2) standpipe piezometers were installed along two of the LLO alignments during the May 2018 investigation. In addition to standpipe piezometers, a series of three (3) vibrating wire piezometers were installed in three (3) of the boreholes, one targeting the glaciolacustrine materials, one targeting the glacial till materials, and one at the glacial till and bedrock interface.

4.0 REGIONAL GEOLOGY MODEL

A Regional-scale Geology Model (RGM) has been developed for this Preliminary Geotechnical Assessment. This was based on the findings of the desk study review (Section 3.1) and terrain mapping (Section 3.2). The objectives (Fell et al, 2015; Parry et al, 2005) of the RGM is to identify and evaluate on a regional scale:

- Active geological processes and geohazards that have the potential to impact the SR1 project[5];
- Construction or operational activities associated with the SR1 Project that may change the existing geological, stress or hydrogeological conditions;
- Regional stratigraphy and geological structure;
- Geological units present, their inter-relationships and how the engineering geological and geotechnical properties of each unit may vary due to geological processes;
- Scale of variation and modern analogues for problematic lithological units;
- Current and previous stress regimes;
- Geological features such as faults, thrusts or landslides occurring at or near the site, but not exposed or recognizable at the site;
- The engineering geomorphological considerations of the site in terms of the regional stratigraphy, structure and geological history; and,
- Regional groundwater conditions.

The RGM was used to develop the scope of the 2016 field program (Section 3.4.2) and the subsequent Site-specific Engineering Geology Model, which is discussed in Section 5.

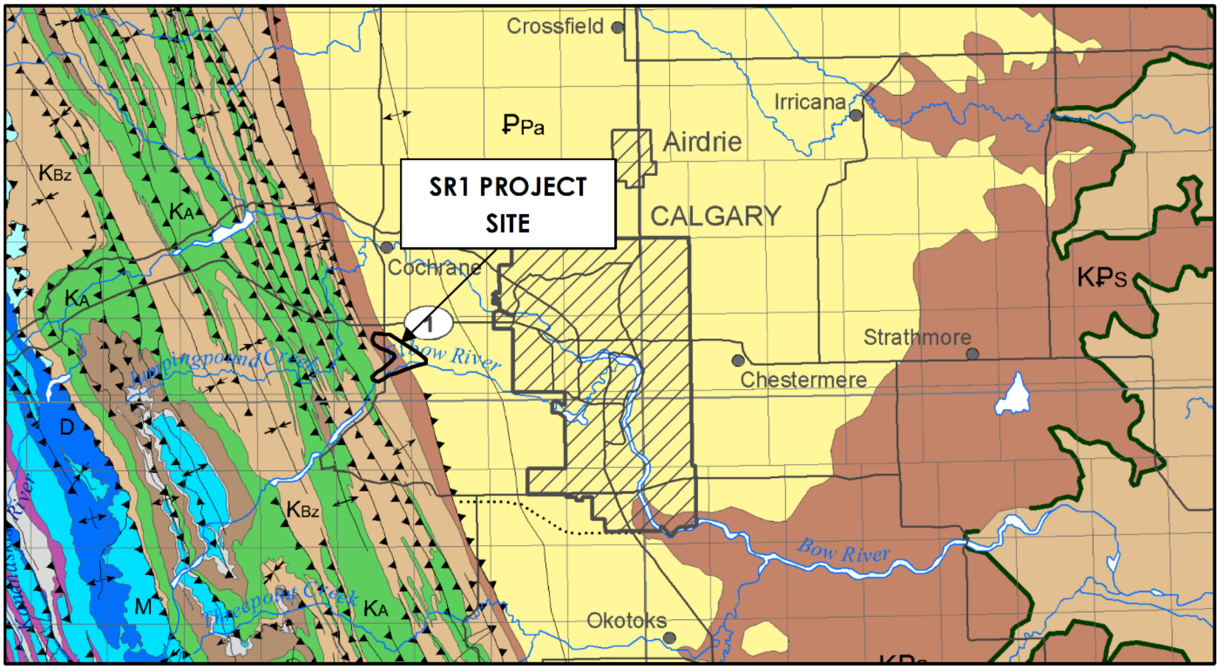
[5] Processes which may be active enough to affect a dam project include de-stressing, chemical weathering of soil/rocks, solution, creep/landslides, subsidence, pressure by groundwater, freezing, animal borrowing, vegetation, seismicity, volcanism and glaciation (Fell et al, 2015).

4.1 BEDROCK GEOLOGY

4.1.1 Regional Framework

The earliest bedrock map we found that included the SR1 Project Site was “Map 027: Geological Map of Alberta” produced by Green (1972) for the ARC. This was superseded by Map 236: Geological Map of Alberta (Hamilton et al. 1999), which depicted the bedrock geology at 1:1,000,000 scale and compiled new mapping by the AGS, the Geological Survey of Canada (GSC), and by the Canadian Society of Petroleum Geologists through the contribution of its membership to the Geological Atlas of the Western Canada Sedimentary Basin (WCSB).

This Preliminary Geotechnical Assessment utilized the most recent version, Sheet 600: Bedrock Geology of Alberta (Prior et al. 2013), which was jointly published by the AGS and AER. This map was updated using new interpretations, new mapping and 3D models of subsurface stratigraphy based on the interpretation of geophysical logs from oil and gas wells. An extract from Sheet 600 is reproduced in Figure 2.



PALEOGENE

Saunders Group

ꝒPa	**PASKAPOO FORMATION:** recessively weathering, grey to greenish-grey mudstone and siltstone with subordinate (although generally better exposed) pale grey, thick- to thin-bedded, commonly cross-stratified sandstone; minor conglomerate, mollusc coquina, and coal; nonmarine

UPPER CRETACEOUS and PALEOGENE

KꝒCp	**COALSPUR FORMATION:** sandstone (thin bedded to massive), siltstone, mudstone, and coal; subordinate conglomerate and bentonite; thick coal seams occur locally; extent poorly defined in some areas; nonmarine

UPPER CRETACEOUS

KBz	**BRAZEAU FORMATION:** sandstone, laminated siltstone, and olive-green mudstone; chert- and quartzite-bearing, granule to pebble conglomerate (lower part); overlain by greenish-grey to dark grey mudstone, siltstone, and greenish-grey sandstone; thin coal and coaly shale beds; numerous thin bentonites (upper part)
KA	**Alberta Group** **WAPIABI FORMATION:** shale, mudstone, silty shale, argillaceous siltstone, and siltstone (some platy, some with rusty-brown weathering, some calcareous); local bentonite layers and local siderite concretions (isolated or along horizons, locally abundant); includes fine-grained, massive to cross-bedded sandstone of the Marshybank Member (lower part of formation) and the fine- to coarse-grained sandstone and argillaceous siltstone of the Chungo Member (upper part of formation); rare, thin chert-pebble layers; marine to locally nonmarine

Figure 2. Regional Bedrock Geology (Extracted from AGS Map 600)

4.1.1.1 Western Canada Sedimentary Basin

The SR1 Project Site is located within the WSCB. This is a 1.4M km2 sedimentary basin that underlies Manitoba, southern Saskatchewan, Alberta, northeastern British Columbia and the southwest corner of the Northwest Territories. The basin thickness is wedge shaped with a maximum thickness of 6 km on the axis of the Alberta Syncline and tapers out eastwards towards a zero-edge at the Canadian Shield.

Mossop and Shetsen (1994) divided the WSCB into two successions that reflect different tectonic settings: Paleozoic to Jurassic-age platformal succession, dominated by carbonate rocks deposited on a stable craton. This is overlain by Mid-Jurassic to Paleocene foreland basin succession, dominated by clastic rocks formed during active margin orogenic activity.

4.1.1.2 Cordillerian Deformation Belt

The SR1 Project site is located within the eastern zone of the Cordillerian Deformation Belt. This is a northwest-tapering zone of thin-skinned, thrusts and faults developed between 115 Ma and 55 Ma during the Cordillerian Orogeny[6]. Post-orogenic differential erosion has resulted in high relief of the Southern Canadian Rockies and the eastern Foothills. The Southern Canadian Rockies are typically divided into the Front Ranges, Main Ranges and Western Ranges.

4.1.1.3 Regional Stratigraphy

The geology of the SR1 Project Site is underlain by Upper Cretaceous to Tertiary bedrock that was deposited in the Alberta Foreland Basin and subsequently deformed by the Laromide Orogeny. The encountered formations are summarized in Table 3. The published geographical extents of these formations are presented in Figure 2.

The stratigraphic nomenclature used for this table is based on Dawson's (1994) 'Central and northern foothills' and the Table of Formations for Central Mountains and Foothills' (AGS, 2015).

Table 3. Geology Stratigraphy

Group	Sequence	Formation / Member		Age (Ma)
Saunders	Entrance / Paskapoo	Paskapoo		56
		Coalspur	Upper	63
			Lower	
	Belly River / Edmonton	Brazeau	Upper	70
			Lower	78
Smoky	Wapiabi	Nomad Member		83

[6] Often referred to as the Laramide Orogeny, which occurred primarily in the USA during the Cretaceous Period.

4.1.2 Brazeau Formation

The Brazeau Formation (BZF) subcrops beneath the western portion of the SR1 Project Site (Figure 20-14). It underlies the floodplain berm, diversion structure and diversion channel between approximate Station 10+000 and 13+200 m.

The BZF is part of the Belly River-Edmonton sequence. The dominant lithology is mudstone, siltstone and fine grained sandstone. Coaly shale and coal beds are common. Natural Resources Canada (NRCAN, 2015) describe the BZF as a non-marine succession of inter-bedded mudstone, siltstone and fine-grained sandstones with subordinate but prominent coarser grained sandstone layers. The AGS (2015) currently sub-divides the BZF into lower and upper members.

AGS (2015) mapping indicates that the BZF subcrops immediately east of Nomad Marine (NMD) Shales, which comprise black shales inter-bedded with thickening upward, hummocky cross-bedded siltstone and sandstone.

The Lower BZF Member was described by Jerzykiewicz (1997) as a succession of fluvial sandstones inter-bedded within grey to olive-green flood-plain mudstones deposited on an erosional contact with the marine Nomad Member (NMD) of the Wapiabi Formation. At the Blackstone River Type Section, which is approximately 225 km NW of the SR1 Project Site, this unit comprises a stacked succession of channel sandstone units, siltstone and overbank mudstones. The overbank mudstones that separate the channels comprise 70 to 75 percent of the succession and are usually three times thicker than the channel sandstones. The channels are comprised of massive or low angle bedded sandstones and conglomerates. The sandstone is grey / greenish grey and can exhibit a salt-and-pepper appearance due to chert and lignitic fragments. The overbank deposits contain non-laminated, greenish grey to dark grey mudstone, silty to sandy mudstone, laminated siltstone, and very fine grained sandstone. Thin coal beds, coaly shale (NRCAN, 2015) and bentonite beds up to 0.2 m thick begin to appear in the upper portion of the lower BZF.

The Upper BZF Member comprises a coarsening-upward sequence of predominantly lacustrine mudstones inter-bedded with sandstones. The lower portion of this Member comprises mudstone and rhythmically interlaminated siltstone and Claystone of offshore lacustrine origin, inter-bedded with delta-front sandstone layers. The upper portion contains increased sand units comprising predominantly thick sheet-flood sandstone layers, inter-bedded with laminated lacustrine-type mudstone and siltstone. Channelized sandstones are rare (Jerzykiewicz, 1997).

4.1.3 Coalspur Formation

The Coalspur Formation (CSF) subcrops beneath the diversion channel between Station 13+200 and 14+700 m, the emergency spillway, diversion channel outlet, the west dam abutment and western portion of the dam footprint between approximate Station 20+000 to Station 21+400 m (Figure 20-14).

The boundary with the underlying BZF is the Entrance Conglomerate. Jerzykiewicz (1997) identified this boundary on the Highway 22 road cutting (OC6) and indicated that the adjacent ridge marked the eastern limit of the Cordilleran Deformation Belt. The boundary was extrapolated to the SE and bisects the diversion channel at approximate Station 13+200 m (Figure 2).

The boundary between the CSF and the overlying Paskapoo Formation (PPF) was not identified and is inferred from AGS Mapping (Prior et al. 2013). At a regional-level, Jerzykiewicz (1997) indicated that the boundary is identified as a prominent sandstone unit, which can be observed on the Bow River approximately 3 km upstream of the Highway 22 bridge in Cochrane and within the Jumpingpound Creek approximately 3.5 km southwest of Cochrane. It is likely that the ridge on which the west dam abutment will be constructed may represent this boundary.

The CSF is a sequence of inter-bedded mudstone, siltstone and fine grained sandstone with subordinate coarser grained sandstone layers and channel lag deposits. Although, this formation is known for its coal beds, these are typically absent in the central foothills between Cochrane and Turner Valley (Jerzykiewicz, 1992).

The lower portion of the CSF comprises predominantly mudstone with thick, fining upward layers of fluvial sandstone. The upper CSF comprises coarsening upward sequences of distributary channels and distributary mouth-bar sediments associated with lacustrine and swamp sediments (Jerzykiewicz, 1997).

4.1.4 Paskapoo Formation

The Paskapoo Formation (PPF) subcrops beneath the east dam abutment, the eastern portion of the dam footprint between approximate Station 21+400 and 24+000 m, the Low-level outlet and the reservoir (Figure 3).

The PPF is comprised of an inter-bedded non-marine sandstone, siltstone and mudstone with minor amounts of bentonite and coal (Lyster and Andriashek 2010). Jerzykiewicz (1997) indicated that thick mudstones predominate over fluvial channel sandstones characteristic of point bar deposition. The formation was divided into five lithological domains by Hamblin (2004) and three litho-stratigraphical members by Demchuk and Hill (1991).

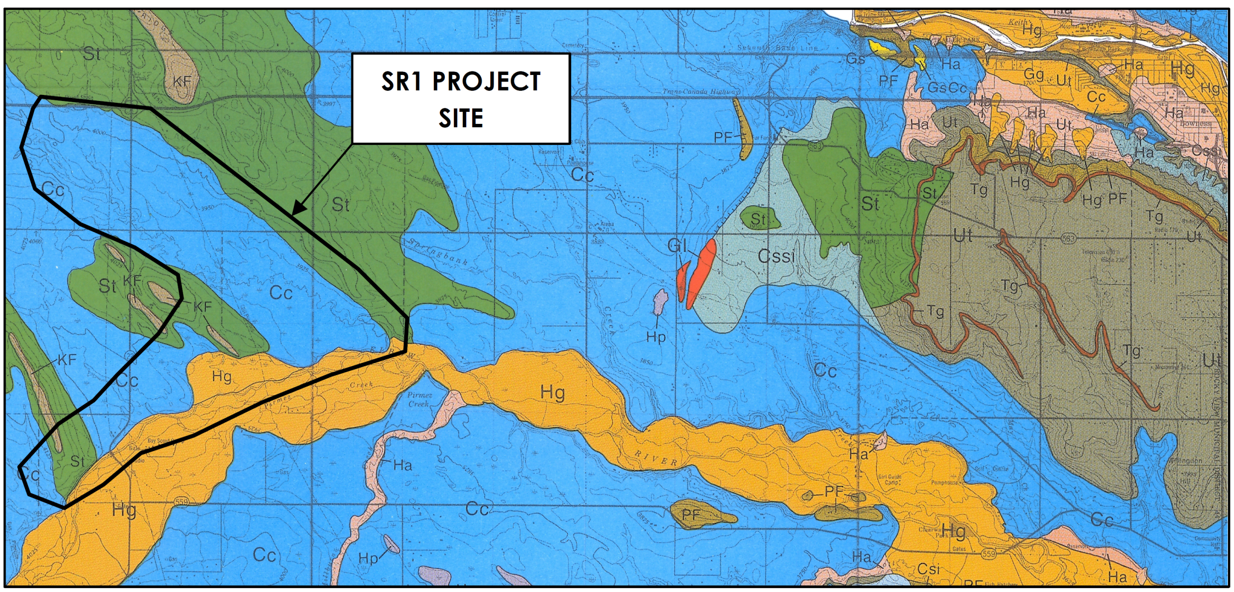
The SR1 Project Site is located within the Bow River Domain (Hamblin, 2004). This domain is dominated by thick mudstones with thick, fining upward, meandering channel sandstones but lacking well developed Paleosol or coal beds. Coal is absent and caliche debris occurs only as a lag deposit at the base of some fluvial channel deposits.

Demchuk and Hill (1991) divided the PPF into three members: the basal Haynes Member, the overlying Lacombe Member and the locally eroded Dalehurst Member. Based on the regional geological structure, it is likely that the SR1 project is underlain by the Haynes and Lacombe Members. The Haynes Member is approximately 50 m thick, and composed of medium- to coarse-grained sandstones of amalgamated fluvial channel deposits. The Lacombe Member comprises the majority of the PPF and is characterized by extensive siltstone and mudstone beds with isolated sandstone channel deposits. It has a maximum thickness of approximately 500 m along the western margin of the basin (Quartero et al, 2015).

4.2 SURFICIAL GEOLOGY

4.2.1 Framework

The surficial geology within Calgary was mapped at 1:100,000 scale as part of the ARC Bulletin 53 (Moran, 1986). Map 204, which shows the Calgary urban area at 1:50,000, was generated from the original hard copy map included in ARC Bulletin 53. This was developed using nomenclature proposed by Tharin (1960). The unit names applied by Tharin (1960) and Moran (1986) are still frequently used in the local geotechnical industry. An extract from Bulletin 53 is included in Figure 3.



Quaternary	Till	Clayey Lacustrine	Silty Lacustrine	Sandy Lacustrine	Ice Contact Fluvial Gravel	Fluvial Channel Sand	Fluvial Channel Gravel	Loamy Overbank and Fan Sediment
Post-Glacial Undivided							Hg	Ha
Glacial Undivided	Gt				Gl	Gs	Gg	
Calgary Formation		Cc	Csi	Cssi				
Crossfield Formation	Crt		Crsi	Crssi	Crl	Crs		
Balzac Formation	Bt		Bsi		Bl	Bs		
Lochend Formation	Lt	Lc			Ll			
Spy Hill Formation (Upper Unit)	Ut	Uc			Ul	Us		
Spy Hill Formation (Lower Unit)	St	Sc			Sl	Ss		

Figure 3. Surficial Geology of the SR1 Project Site (Extracted from AGS Map 204)

The Quaternary geology in Southern Alberta was mapped at a 1:500,000 scale by Shetsen (1987). This map differs from that produced by Moran (1986) as it was defined by age, genesis and lithologic type without naming the units. The surficial geology of Alberta was updated in 2013 by the AGS in Map 601 (Fenton et al. 2013). This included a compilation of previous surficial map data, which was edited for continuity and generalized for use at 1:1,000,000 scale. This map describes the glacigenic units within Alberta based on the depositional environment and landform assemblages. No stratigraphical names are applied to these units on a regional level. An extract from Map 601 is presented in Figure 4.

The accompanying Map 604 showed the distribution of glacial landforms in Alberta (Atkinson 2014). This was compiled from mapping data, research literature and an updated analysis of remote sensing data. An extract from Map 604 is presented in Figure 5.

The surficial geology of the SR1 Project Site comprises glacigenic units deposited during the Pleistocene Glaciations. Using Tharin (1960) and Moran's (1986) nomenclature, the site is blanketed by the Lower Spy Hill Till and the Calgary Formation. Using the AGS nomenclature the site is blanketed by undivided moraine and glacio-lacustrine deposits. The surficial units present within the SR1 Project Site are discussed below.

4.2.2 Glacial Till

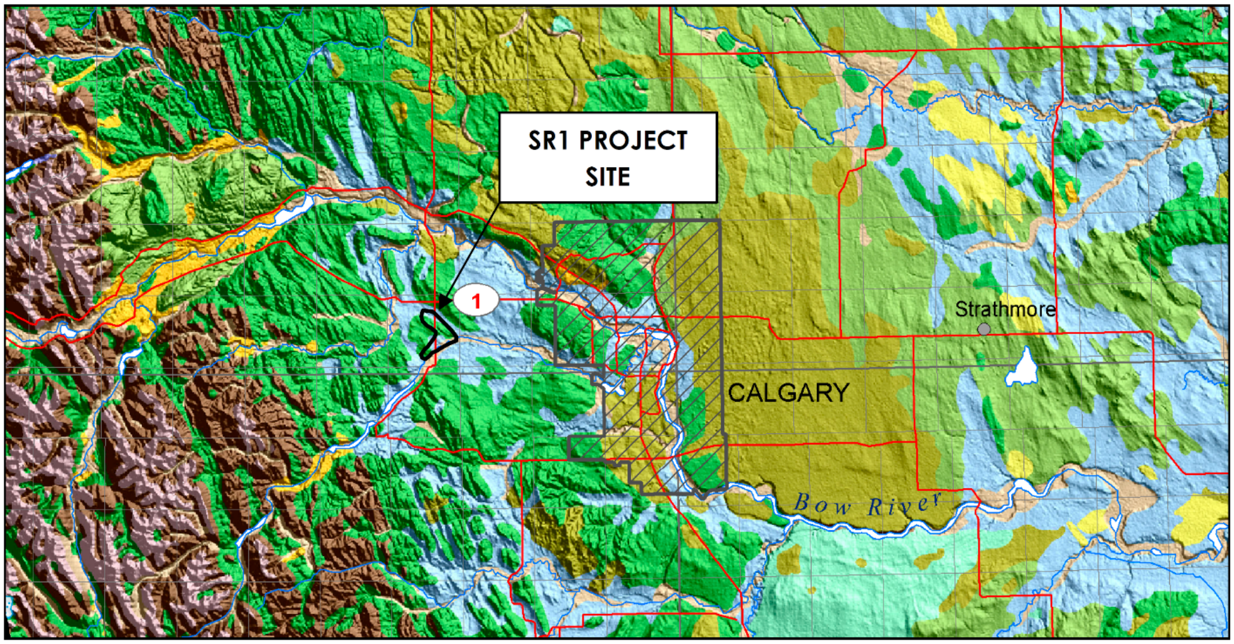
The Lower Spy Hill Till was described by Moran (1986) as '*hard, dark grey, sandy silty clay*'. Clasts comprise dark carbonates and pink / purple quartzites. Granitic clasts are rare or absent. This unit was also mapped in Canmore by Fisher (1999), where it comprised a '*stoney, clay matrix diamicton with rounded clasts, laterally the diamicton base is rich in angular sandstone clasts, and locally varies in thickness up to 4 m*'. Moran (1986) indicated that the thickness ranged between 0.3 and 17.1 m northwest of Calgary. South of Calgary, the thickness ranges between 0.3 and 18.3 m.

The AGS define the glacial till at the SR1 Project Site as 'undivided moraine'. This consists of 'Diamicton (till) deposited directly by glacial ice, and is a mixture of clay, silt, sand and minor pebbles, cobbles and boulders. Locally, this unit may contain blocks of bedrock, pre-existing stratified sediment and till, and/or lenses of glaciolacustrine and/or glaciofluvial sediment' (Fenton et al, 2013).

4.2.3 Glacial Lacustrine Units

The Glacial Lacustrine units within the SR1 Project Site were described by Moran (1986) as '*lacustrine offshore sediments*' comprising '*silt, clay and minor sand*'. These were named the Calgary Formation.

The AGS describe these units as 'fine-grained, distal sediments deposited in or along the margins of glacial lakes. These are a) offshore sediment; rhythmically laminated to massive fine sand, silt and clay, locally debris released from floating ice or b) Littoral and nearshore sediments; massive to stratified, well-sorted silty sand, pebbly sand and minor gravel' (Fenton et al, 2013).



Moraine: Diamicton (till) deposited directly by glacial ice with a mixture of clay, silt, and sand, as well as minor pebbles, cobbles, and boulders; characterized by a lack of distinctive topography. Locally, this unit may contain blocks of bedrock, stratified sediment, or lenses of glaciolacustrine and/or glaciofluvial sediment.

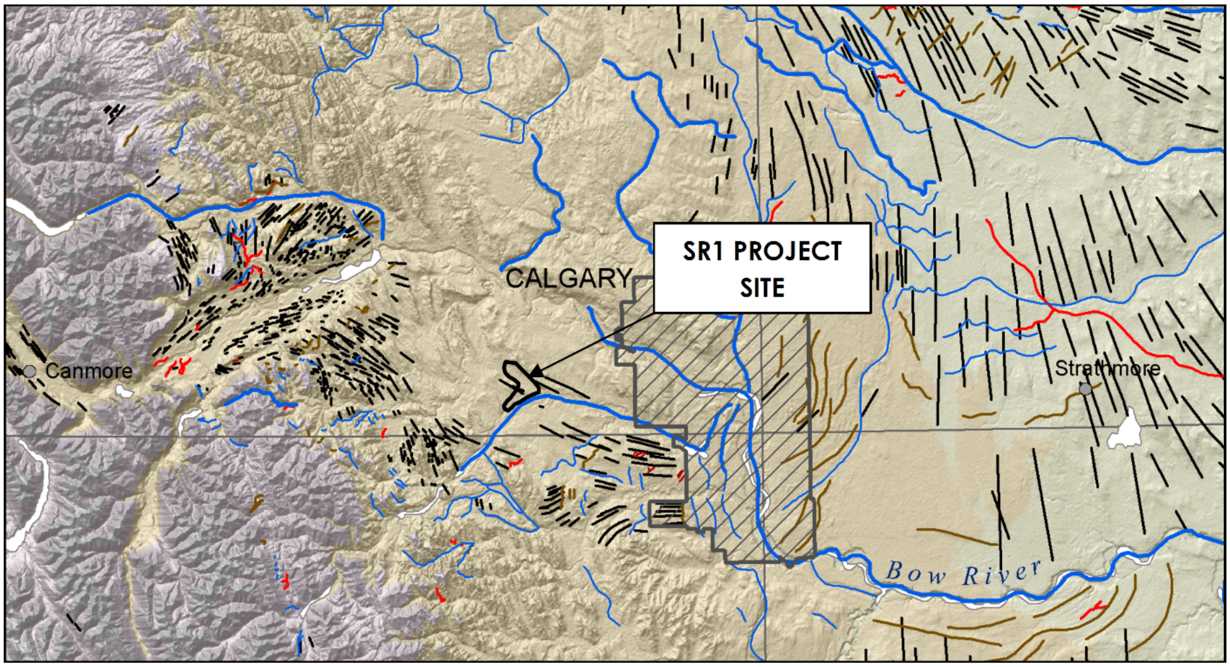
Glaciofluvial Deposits: Sediments deposited by glacial meltwater in subaerial, subaqueous, and subglacial environments; sediment ranges from massive to stratified, poor to well sorted, coarse to fine grained; in places, this unit includes till; may show evidence of ice melting (slumped structures).

Fluvial Deposits: Sediments deposited by streams and rivers; synonymous with alluvium; includes poorly to well-sorted, stratified to massive sand, gravel, silt, clay, and organic sediments occurring in channel and overbank deposits; in places, includes a significant component of colluvial deposits as these two units are inseparable at this map scale.

Glaciolacustrine Deposits: Sediments deposited in or along the margins of glacial lakes; includes a) offshore sediment; rhythmically laminated to massive fine sand, silt, and clay, locally containing debris released by the melting of floating ice; and b) littoral (nearshore) sediments; massive to stratified, well-sorted silty sand, pebbly sand, and minor gravel; occurs in beaches, bars, and deltas.

Stagnant Ice Moraine: Sediments resulting from the collapse and slumping of englacial and supraglacial debris due to the melting of buried stagnant ice at the glacier margin; sediment is mainly till but locally includes stratified glaciolacustrine or glaciofluvial sediments; characterized by low- to high-relief hummocky topography.

Figure 4. Regional Surficial Geology (Extracted from AGS Map 601)



Streamlined bedforms: Glacially streamlined lineations consisting of longitudinal ridges and furrows aligned parallel to ice-flow direction; includes depositional landforms mainly composed of till, and erosional landforms consisting of smoothed and elongated bedrock ridges; morphology varies from oval-shaped hills (drumlins) to elongate sediment ridges (flutes) and mega-scale lineations, comprising regional-scale, highly elongated ridges, with lengths ranging from 5–30 km and heights in the order of 3–10 m; landforms include sharply defined ridges to more subtle, low amplitude streamlining of the terrain, which imparts a distinctive grain to the landscape.

Crevasse-fill ridges: Linear, curvilinear, or chevron-form ridges, can be oriented perpendicular or parallel to ice-flow direction and may intersect to form a box-like pattern; ridges are typically 1–5 m high and 300–1500 m long; primarily composed of till but can contain sorted glacial sediments; formed by supraglacial sediments filling crevasses or subglacial sediments being squeezed into basal crevasses.

Moraine ridges: Ridges, linear to arcuate in plan-form, aligned perpendicular to ice flow; formed by the deposition or deformation of sediment at or close to the ice-sheet margin; morphologically may consist of single or assemblages of small, typically 500–1000 m long and 2–5 m high, linear to arcuate ridges recording the progressive retreat of the ice margin, to larger composite ridges that may contain tracts of stagnant-ice landforms on a regional scale, reflecting a combination of sediment deposition and deformation at the glacier margin, and melting out of masses of glacier ice that were either incorporated or buried beneath the moraine.

Eskers: Elongate to sinuous ridges of glaciofluvial sand and gravel; ridges are typically 5–10 m high and have lengths from 2–10 km; reflect deposition by meltwater in subglacial, englacial, or supraglacial channel networks.

Meltwater channels: Channels that have primarily been eroded by subglacial, supraglacial, or proglacial meltwater; exhibit steep and well-defined channel margins, and often show incised meanders, bifurcations or complex stream patterns; may have long straight reaches or are deeply incised relative to their widths and lengths; major meltwater channels consist of large channels that show evidence that their drainage pattern was influenced at a regional scale by the location of an ice-margin or subglacial drainage conduit; major meltwater channels often form the valleys of modern river systems; minor meltwater channels have shorter reaches generally less than 30 km long, and were eroded into local topographic features; commonly form networks which connect with major meltwater channels.

Figure 5. Regional Glacial Landforms (Extracted from AGS Map 604)

4.2.4 Fluvial Deposits

The fluvial deposits associated with the Elbow River are described by Moran (1986) as '*fluvial channel sediments*' with localized areas of overlying '*fluvial silt overbank*'. The fluvial channel sediments are gravel with minor sand.

The AGS describe these units as 'sediments deposited by streams and rivers; synonymous with alluvium; includes poorly to well-sorted, stratified to massive sand, gravel, silt, clay and organic sediments occurring in channel and overbank deposits' (Fenton et al, 2013).

4.3 REGIONAL GEOLOGICAL HISTORY

This section summarizes the geological history of the SR1 Project Site based on the desk-study review of published reports, maps and literature. The development of the geological history for a dam site is important as it provides supporting data for the:

- Characterization of glacigenic units based on depositional architecture and identified assemblages;
- Evaluation of the stress history of glacigenic units and bedrock;
- Identification of depositional environments and modern analogs for glacigenic units and bedrock; and,
- Estimation of the scale of variation for the foundation geology.

The following section has been divided into five (5) sub-sections that reflect significant geological events that control the geological history and geotechnical behavior of the SR1 project site.

- Cretaceous Period (145 to 66 Ma);
- Paleocene Epoch (66 to 56 Ma)
- Eocene to Pliocene Epochs (56 to 2.58 Ma);
- Pleistocene Epoch (2.58 Ma to 11,700 BP); and,
- Holocene Epoch (11,700 BP to present day).

4.3.1 Cretaceous Period (145 to 66 Ma)

4.3.1.1 83 to 78 Ma

Paleographic reconstruction indicates that the SR1 project site was covered by the Pakowki Sea transgression at this time (Dawson et al, 1994). The NDM Shales are thought to have been deposited in a peritidal backwater lagoon, connected to the open sea through tidal inlets and were periodically flooded by marine waters (Jerzykiewicz, 1997). The boundary between the NDM Layer and the overlying BZF is an erosional unconformity.

4.3.1.2 78 to 70 Ma

Paleographic reconstruction indicated that the region comprised fluvial and coastal plains with areas of lacustrine deposition between 78 and 76 Ma (Dawson et al, 1994). Coal swamps and the Bearpaw Sea were located to the east in modern day Saskatchewan. The channel sandstones were deposited by anastomosing, braided rivers and rapidly aggrading channels within a rapidly subsiding floodplain.

The Bearpaw Sea began to transgress westwards from 76 Ma, flooding the fluvial coastal plains and forming lacustrine and coal swamp environments as this moved westwards. The westernmost limit of the Bearpaw Sea, around 70 Ma, is located to the east of Calgary, thus maintaining a non-marine depositional setting for the SR1 Project Site at this time. The BZF was deposited at this time.

4.3.1.3 70 to 66 Ma

The CSF was deposited at this time and represents a continental depositional setting comprising a high gradient, alluvial fans indicative of thrusting associated with the Laromide Orogeny. Drainage comprised longitudinal trunk rivers and transverse tributaries, similar to modern alluvial foreland basins (Jerzykiewicz, 1997). Paleo-geographic reconstruction indicates a sub-aerial environment around 66 Ma (Dawson et al. 2004).

4.3.2 Paleocene Epoch (66 to 56 Ma)

The PPF was deposited at this time. Paleo-geographic reconstruction indicated a fluvial coastal plain with localized coal swamps and arid uplands by 63 Ma. The depositional environment around 56 Ma was an arid landscape with uplands and mountains to the west and a fluvial coastal plain to the east (Dawson et al. 2004).

4.3.3 Eocene to Pliocene Epochs (56 to 2 Ma)

Regional tectonic uplift and isostatic rebound resulted in the erosion of post-Paleocene sediments from the WCSB between 56 and 2 MA. During this period, the Cordillera, Interior Plains and the Canadian Shield were drained in a west to east direction by the continent-scale, Bell River System (McMillan 1997; Duk-Rodkin and Hughes 1994). The river system transported large volumes of sediment from the Rocky Mountains and Foothills towards the Hudson Bay and the Labrador Sea. It comprised a network of braided and meandering rivers (Leckie 2006), which deposited laterally continuous sheets of sands and gravels.

The onset of the Pleistocene Glaciations resulted in the re-alignment of the Bell River drainage network into the modern-day drainage system. Stalker (1961) proposed several pre- and inter-glacial river alignments through Alberta. The interglacial Elbow River broadly flowed along its current alignment but continued southeast at the Glenmore Reservoir site instead of its current alignment through Elbow Park. At the SR1 Project Site, Stalker (1961) inferred that there was a potential tributary.

The Tertiary Gravels are located to the east of the SR1 Project Site on Broadcast Hill. They are unlikely to be present on the SR1 Project Site but may be useful as an off-site borrow source.

4.3.4 Pleistocene Epoch (2 Ma to 11,700 BP)

The bedrock within the region is overlain by glacigenic units of Late Pleistocene age derived from the Wisconsinan Glacial Event (85,000 to 11,000 BP). Older Pre-Wisconsinan events may have converged within Calgary region but evidence is limited. Andriashek et al (2014) recognized at least four LIS glaciations in Alberta and Jackson et al (1989) surmised that four glacial episodes had occurred within the Bow and Highwood River Basins. Of the four, Episodes 1 to 3 are thought to have impacted the SR1 project site. Episode 4 was restricted to the Rockies and terminated at Castle Junction in the Bow Valley and is not discussed further.

4.3.4.1 Episodes 1 and 2

Episodes 1 and 2 are Pre-Wisconsinan glacial episodes inferred to have occurred within the Calgary region based on evidence from the Porcupine Hills and Waterton National Park areas in Southern Alberta. These events involved both the Cordillerian (CIS) and Laurentian Ice Sheets (LIS). Jackson et al. (1989) suggested that these early glacial episodes modified the drainage networks that existed at that time. Watercourses flowing eastwards from the Foothills were forced to drain southwards along the margins of the retreating LIS, initiating the north-south trending network of glacial channels in Calgary. Further east, the pre-glacial valleys were infilled with glacigenic materials, forcing watercourses to drain further south.

4.3.4.2 Episode 3

This episode occurred during the Wisconsinan event and had a significant impact on the landscape of the region. The Late Wisconsinan advance of the CIS and LIS started from a Middle Wisconsinan inter-stadial minimum between 27,000 and 20,000 BP.

The SR1 project site is located within the saddle of the CIS and LIS. This is the convergence zone between the CIS that advanced eastwards from the Rocky Mountains onto the Prairies and the LIS that advanced southwards from the Keewatin Sector. Figure 6 is an extract from Jackson and Andriashek (2010) shows the paleo-flowlines of the Cordillerian and Laurentian Ice sheets together with the Foothills Erratic Train. This Episode has been divided into a series of ice sheet advances and re-advances.

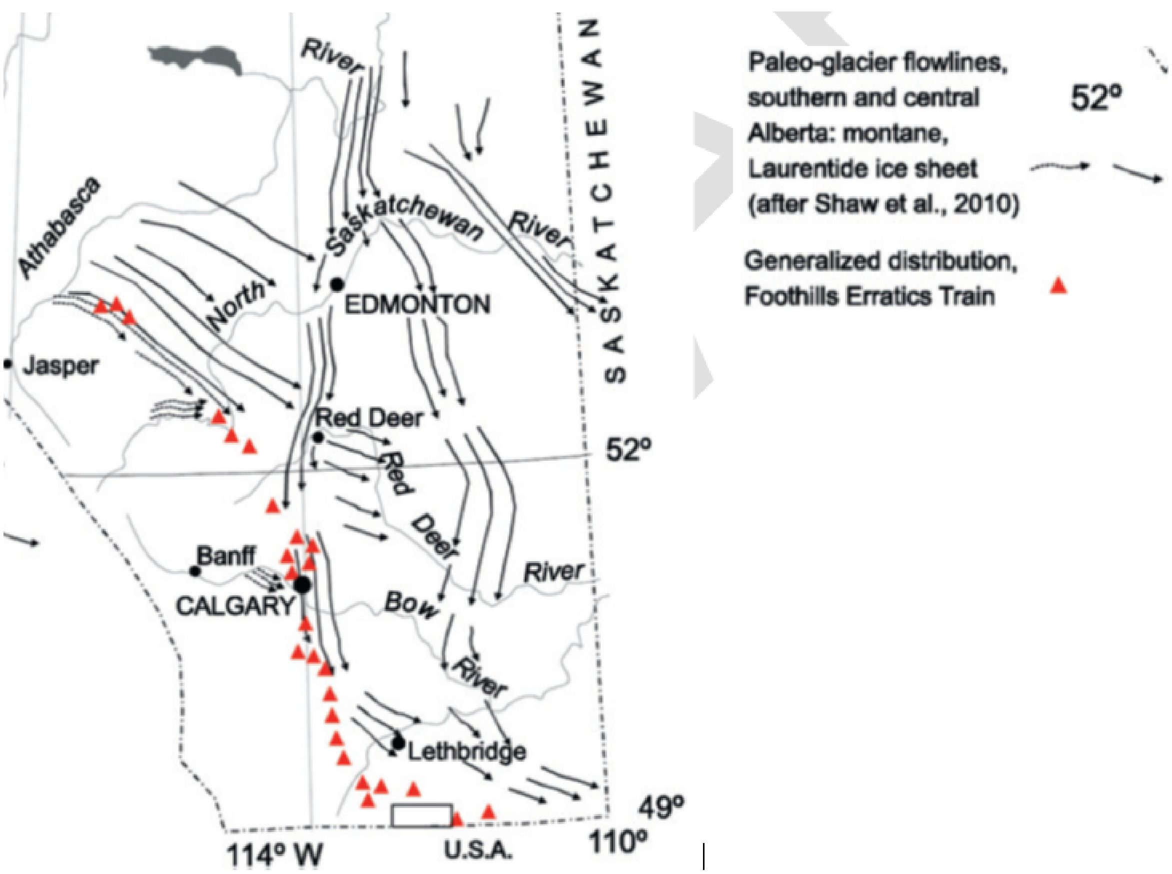


Figure 6. Paleo-flowlines of the Cordillerian and Laurentian Ice Sheets (Extracted from Jackson and Andriashek, 2010)

Cordillerian Bow Valley Advance

The CIS advanced from the Bow Valley and Kananaksis region. The Bow Valley Advance (BVA) across the SR1 project site can be reconstructed from a mappable flowset of streamlined glacial landforms visible on the bare-earth LIDAR and DEM for the region. This set of lineations (flutes, drumlins or streamlined bedrock) indicate that the ice-sheet flowed in a broad WNW-ESE direction from the Foothills, through the Tsuu T'ina Reserve towards Calgary (Figure 7).

Moran (1986) indicated that this BVA resulted in the deposition of the Lower Spy Hill Glacial Till unit within the western Calgary region and dated the BVA between 20,000 and 24,000 BP with its maximum limit between 23,000 and 24,000 BP.

Laurentian Advance

Streamlined bedforms mapped to the north and east of Calgary indicate that a lobate LIS advanced from the northeast into the Calgary region (Figure 5 and Figure 6). This ice lobe is likely to have diverged from the High Plains Ice Stream (HPIS). The HPIS is a 250 km long and between 50 and 85 km wide, regional-scale geomorphological imprint of a fast ice-stream that originated north of Edmonton, flowed southwards and terminated at Lethbridge (Evans et al. 2008; 2014).

The LIS converged with the CIS in the Calgary region. The maximum western extent of the LIS within the Bow River Valley was at Cochrane (Tharin, 1960; Moran, 1986). The LIDAR and DEM mapping indicate that the boundary was further to the west in Calgary.

The LIS and CIS continued to converge until the combined ice sheet reached Southern Alberta and Montana. This is considered to be the Last Glacial Maximum (LGM) and is thought to have occurred between 21,000 and 20,000 BP (Dyke 2004).

Deglaciation of the CIS-LIS Saddle

After the LGM, rapid deglaciation and suturing of the CIS-LIS saddle began to occur. This occurred along a narrow ice-free corridor to the east of the Rocky Mountains. The date by which it had retreated from Calgary is unknown. Dyke (2004) indicated that the CIS and the LIS were still merged until at least 18,000 BP and that a corridor was present in the Calgary region between 16,000 and 14,000 BP (Dyke, 2004). Gregoire et al (2012) indicated that the corridor was present by 15,000 BP. Gowan et al (2016) indicated that the LIS margin was located in central Alberta by 15,000 BP.

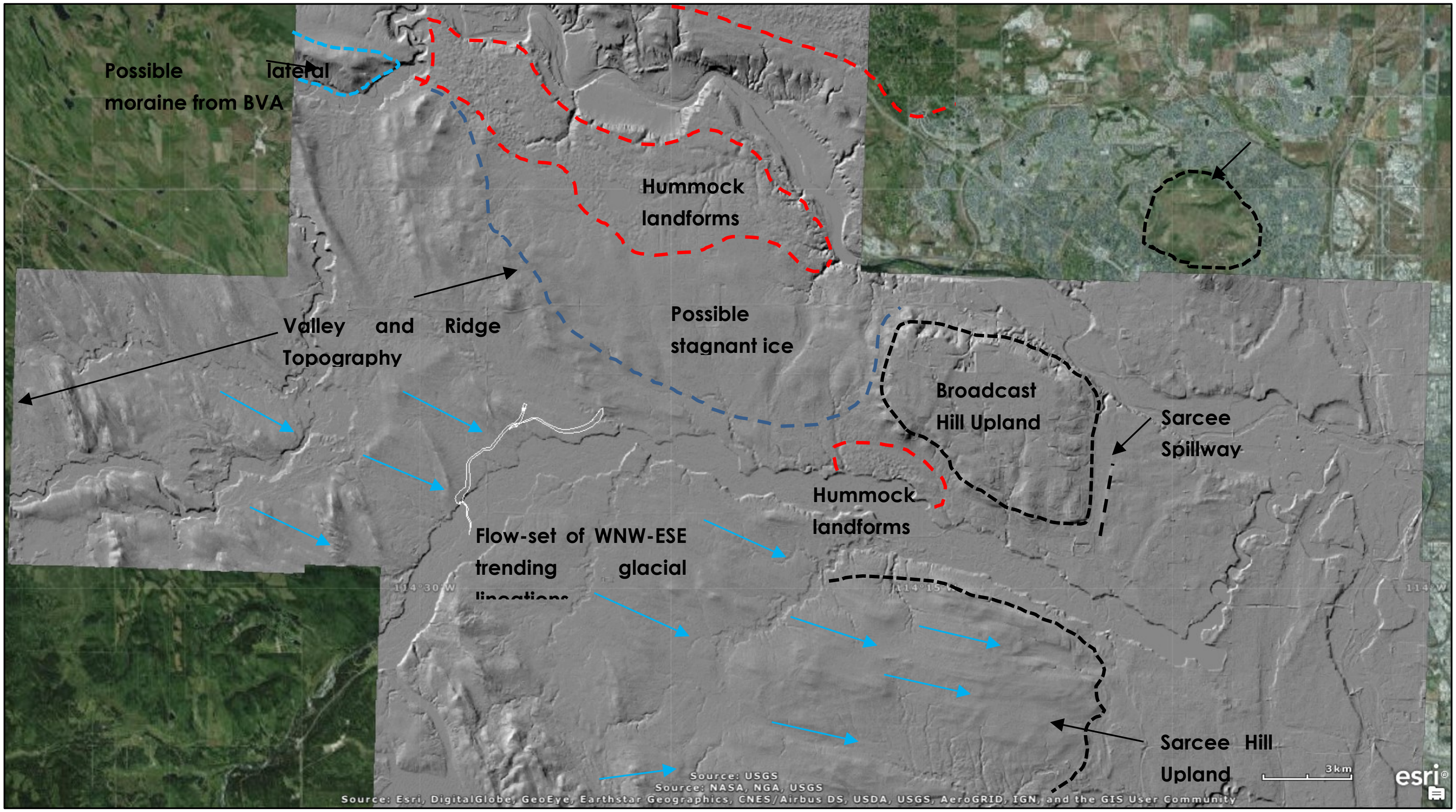


Figure 7. Geomorphological landforms from Regional Digital Elevation Model

Jackson et al. (1982) indicated that the Kananaskis Valley was free of ice by 13,500 BP and McDonald (1981) indicated that Kananaksis and Morley Flats were free of ice by 10,400 BP. Evans (1999) indicated that the dammed lake in Kananaksis was progressively drained by spillways (such as Sibbald Creek) eastwards into the Elbow River Drainage System before flowing northwards towards the Bow Valley after the ice had sufficiently retreated. This temporary spillway could be indicative of Stalkers (1961) tributary spur on a hypothesized interglacial Elbow River.

Glacial Lake Calgary

Deglaciation resulted in the development of transient proglacial lakes along the ice-free corridor. Proglacial lakes are dynamic and transient features of the post-glaciation Pleistocene landscape. Their evolution is influenced by the lake's relationship with the proximity of the ice margins. Ice-dammed lakes can empty frequently as new outlets become exposed due to deglaciation.

Glacial Lake Calgary was formed by the damming of eastern drainage paths by LIS. The lake formed between Calgary, Cochrane and Bragg Creek. The extents of Glacial lake Calgary were mapped by Fisher (1999); Moran (1986), Harris and Ciccone (1983) and Tharin (1960). The SR1 project site is located at the western margin of the 'final' lake position. The approximate extents of Glacial Lake Calgary based on Harris and Ciccone (1983) and Moran (1986) is presented in Figure 8.

Glacio-lacustrine depositional processes in the lake would have changed over time. When the CIS margin was near to the SR1 project site, sediment deposition would have been controlled by processes associated with the marginal ice cliffs or submerged ice ramps. Sedimentation could involve mass movements such as debris flows and slides, slumps and surge currents or ice rafted, supraglacial debris from the calving of icebergs. Landforms and sediments associated with this type of depositional environment would include poorly-sorted diamictons, mass-flow deposits, lacustrine muds and dropstones. Continued stagnation and westward retreat of the CIS margin would result in distal sedimentation. This would involve the deposition of fine, suspended sediment from streams and precursors to the Bow and Elbow River; lake underflows, interflows and overflows with ice-rafted debris from occasional icebergs.

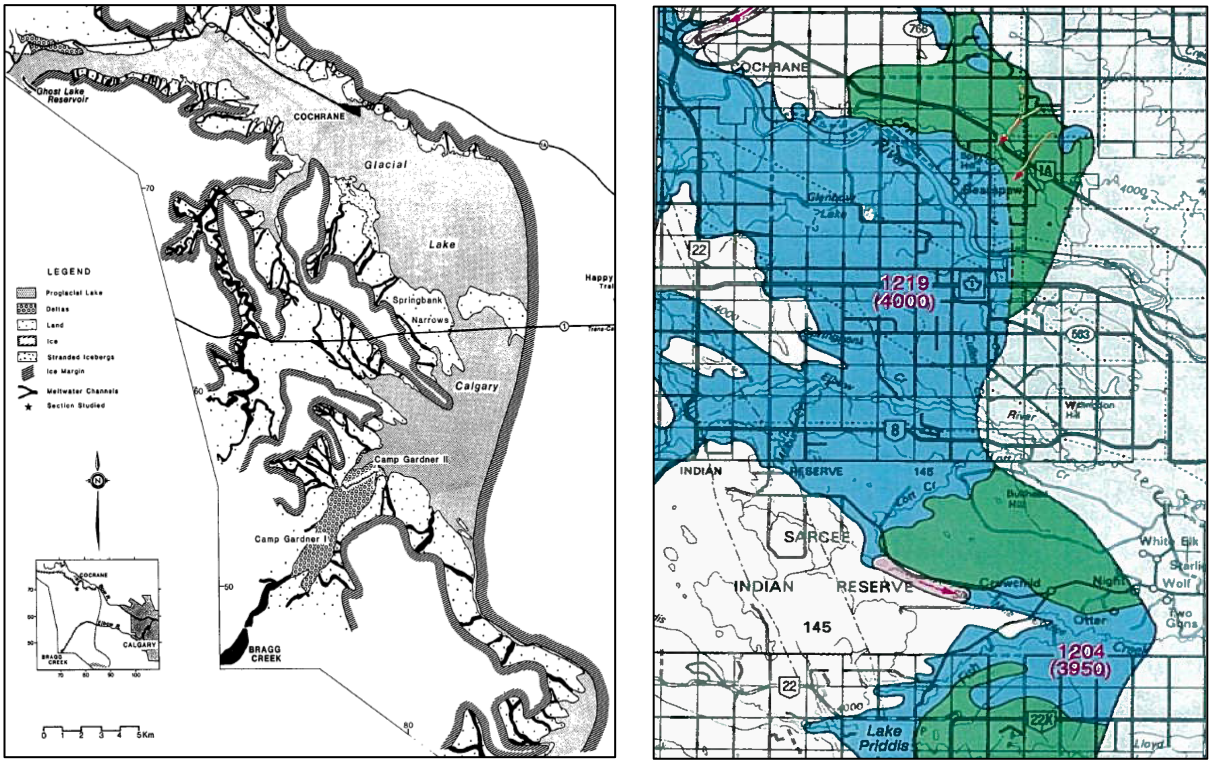


Figure 8. Approximate Extents of the Glacial Lake Calgary (Extract from Harris and Ciccone, 1983; and Moran, 1986)

Moran (1986) divided the retreat of the CIS and LIS within Calgary into seven (7), undated stages. These are presented in Figure 9 and summarized below.

- 1st Stage: SR1 project site is covered with 'debris-covered, stagnant glacial ice' derived from the deglaciation of the CIS. Glacial lakes formed to the south at Priddis and north at Cochrane (minimum elevation is Elevation 1273 m based on level of controlling lake outlet);
- 2nd Stage: the stagnant ice completed melted from the SR1 project site. Glacial Lake Calgary formed with a minimum elevation of Elevation 1219 m. The higher ground within the SR1 Project Site is not submerged by the lake. The LIS margin is immediately to the west of Calgary;

- 3rd Stage: the LIS margin retreated further east, exposing outlets at lower elevations. Glacial Lake Calgary retreated to the east and split into two lakes within the Bow and Elbow Valleys connected by the Sarcee Outlet. The minimum elevation is Elevation 1181 m in the Bow Valley and Elevation 1173 m in the Elbow Valley. The Elbow River formed and drains into the Elbow Valley Lake. The SR1 Project Site is now sub-aerial; and,
- 4th to 7th Stages: Further retreat of the ice-sheet and Glacial Lake Calgary to the east. The SR1 Project Site is still sub-aerial.

Ice-sheet Re-advances

A series of localized re-advances occurred during the deglaciation of the ice-sheets. Three re-advances of the LIS into Calgary were proposed by Moran (1986): Lochend, Balzac and Crossfield. During the re-advance, eastward drainage channels were blocked and pro-glacial lakes reformed within the Bow and Elbow River Valleys but these did not extend as far as the SR1 project site.

The recession of the BVA ice lobe was interrupted by re-advances: the Canmore Advance and the subsequent, smaller Eisenhower Re-Advances (Jackson et al. 1989; Evans et al. 1999). These events were restricted to the Rockies and did not extend as far as the SR1 Project Site.

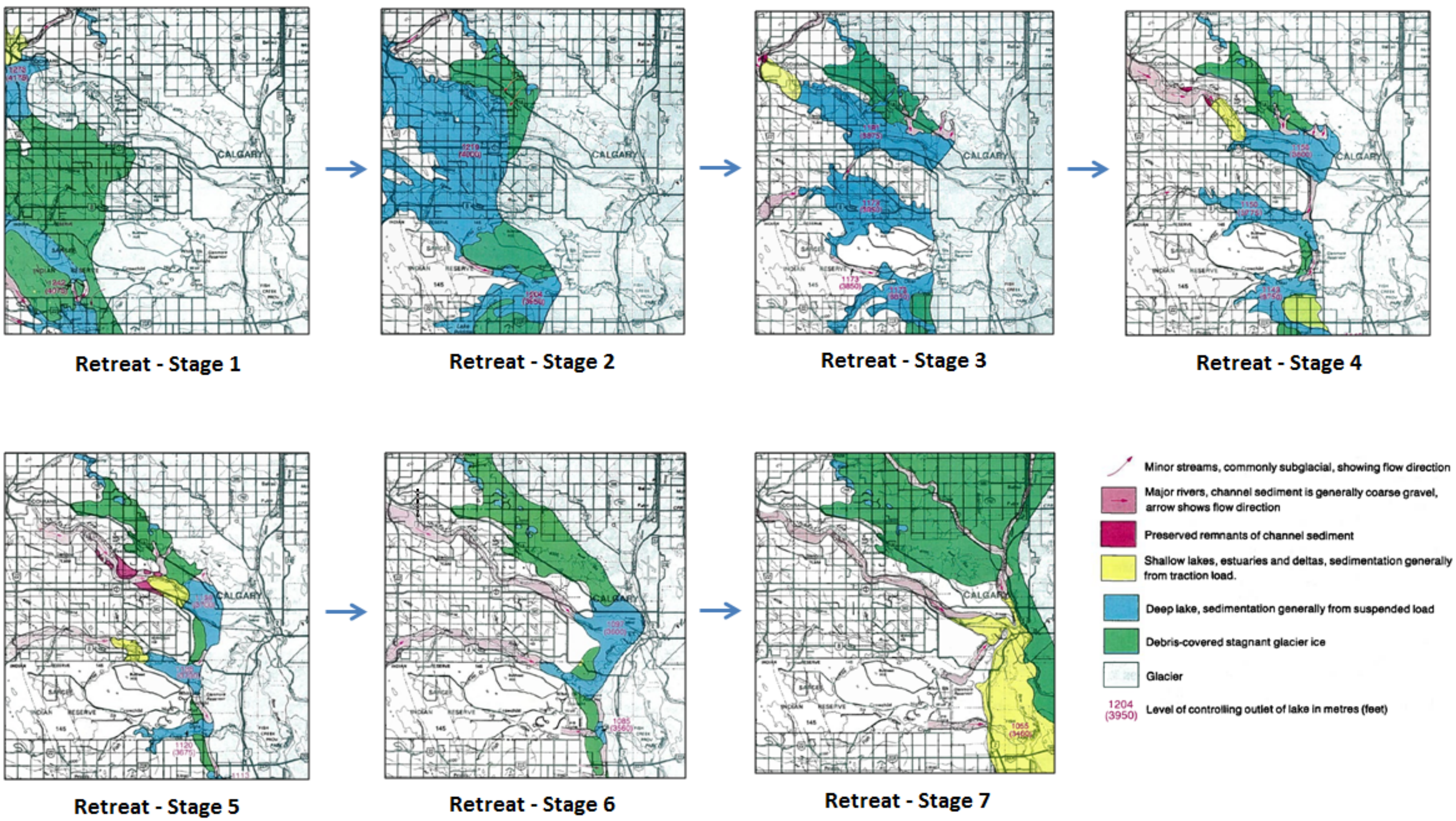


Figure 9. Retreat of the Cordillerian and Laurentian Ice Sheets (Extract from Moran, 1986)

4.3.5 Holocene Period (11,700 BP to Present Day)

This period was characterized by the post-glacial development of the Elbow River Valley and its tributaries through episodes of fluvial deposition and erosion. These valleys are currently over-sized compared to their present day bedload indicating that post-glacial flows were significantly higher.

There is limited literature regarding its formation, but the Elbow River likely developed in a similar manner as the Bow River. Jackson et al. (1982) proposed a para-glacial origin for the erosion of the Bow River Valley and deposition of the Bighill Creek Formation, whilst Oetelaar (2002) suggested that the Bow River Valley underwent two major episodes of fluvial deposition between 11,500 BP and the present day. The first episode occurred between 11,500 and 10,000 BP due to the decreasing rate of isostatic rebound, climate response, sparse vegetation and unstable para-glacial conditions. The unstable ice-marginal deposits in the Rockies were drained, eroded and transported into the Elbow River. Unstable landscapes were also present in the Foothills, further increasing the sediment load. This resulted in the formation of braided river systems and the deposition of coarse gravel. The second episode of fluvial deposition occurred between 9,000 and 5,000 BP following a brief period of erosion. From 5,000 BP to present day, the Elbow River may have reached 'an approximate state of equilibrium' with erosion more predominant than deposition.

The Unnamed creek runs through the reservoir area and dam footprint. The river valley that contains the unnamed creek is oversized for its current flow. It is likely that this was a post-glacial channel, which drained meltwater and para-glacial debris from stagnant ice moraines to the north.

4.4 STRUCTURAL GEOLOGY

4.4.1 Regional Model

The regional-scale geological structures present within the SR1 project site are presented in Figure 10. This is an extract from the AGS Map 560 (Pana and Elgr, 2013).

The geological structure within the Cordillerian deformation belt is dominated by thrust faults (Faure et al, 2004). The easternmost Front Ranges consist of imbricated, dipping thrust sheets involving Paleozoic carbonates. The abrupt transition between the Front Ranges and the Foothills is the region-scale McDonnell Thrust-Fault. The Foothills are of lower relief than the Front Ranges and comprise a 40 m wide zone of closely-spaced, low-displacement thrust faults involving Cretaceous bedrock (Osborn et al, 2002), including the BZF and CSF. The thrust faults typically bisect through the stratigraphical succession and juxtapose older strata over younger strata, resulting in stratigraphical repetitions (Faure et al, 2004) moving from west to east.

The AGS mapping indicates that the Brazeau thrust-fault bisects the SR1 project site location between the proposed diversion structure and the existing Highway 22 Bridge (Pana and Elgr, 2013). There is an unnamed thrust-fault between the CSF and PFF to the southeast of the SR1 Project Site. However, Faure et al (2004) grouped this into the Brazeau Thrust-Fault zone.

4.4.2 Brazeau Thrust-Fault

The Brazeau thrust fault is a regional-scale geological structure that extends over 375 km between the communities of Hinton and Highwood along a NNW-SSE strike. The hanging-wall is to the southwest and has thrust the Lower BZF to the surface. The thrust-fault bisects the SR1 project site location between the proposed diversion structure the existing Highway 22 Bridge[7].

Kinematic modelling by Faure et al (2004) indicated that the Brazeau Thrust Fault was developed by 61 Ma after deposition of the BZF, CSF and PPF. Further horizontal compression up to 55 Ma resulted in the decollement and thrusting of these units to higher elevations than existing during the present day. Post-orogenic differential erosion due to uplift resulted in the removal of these units.

A geological cross-section through the Foothills showing the thrust emplacement is presented in Figure 11. This is extracted from Faure et al (2004).

4.4.3 Folding

The AGS mapping (Pana and Elgr, 2013) identified two regional-scale folds within the SR1 project site:

- An unnamed anticline is mapped to the southwest of the Brazeau Thrust-Fault; and,
- The Calgary Syncline is located to the east of the SR1 Project Site.

Geological mapping of outcrops along the Elbow River suggest that an anticline may be present between OC5 and OC7. Small-scale folding can be observed in OC5.

[7] Pana and Elgr (2013) indicate that this structure is the boundary between the BZF and the CSF in this area. This contradicts the findings of Jerzykiewicz, (1997).

Regional Geology Model
December 8, 2020

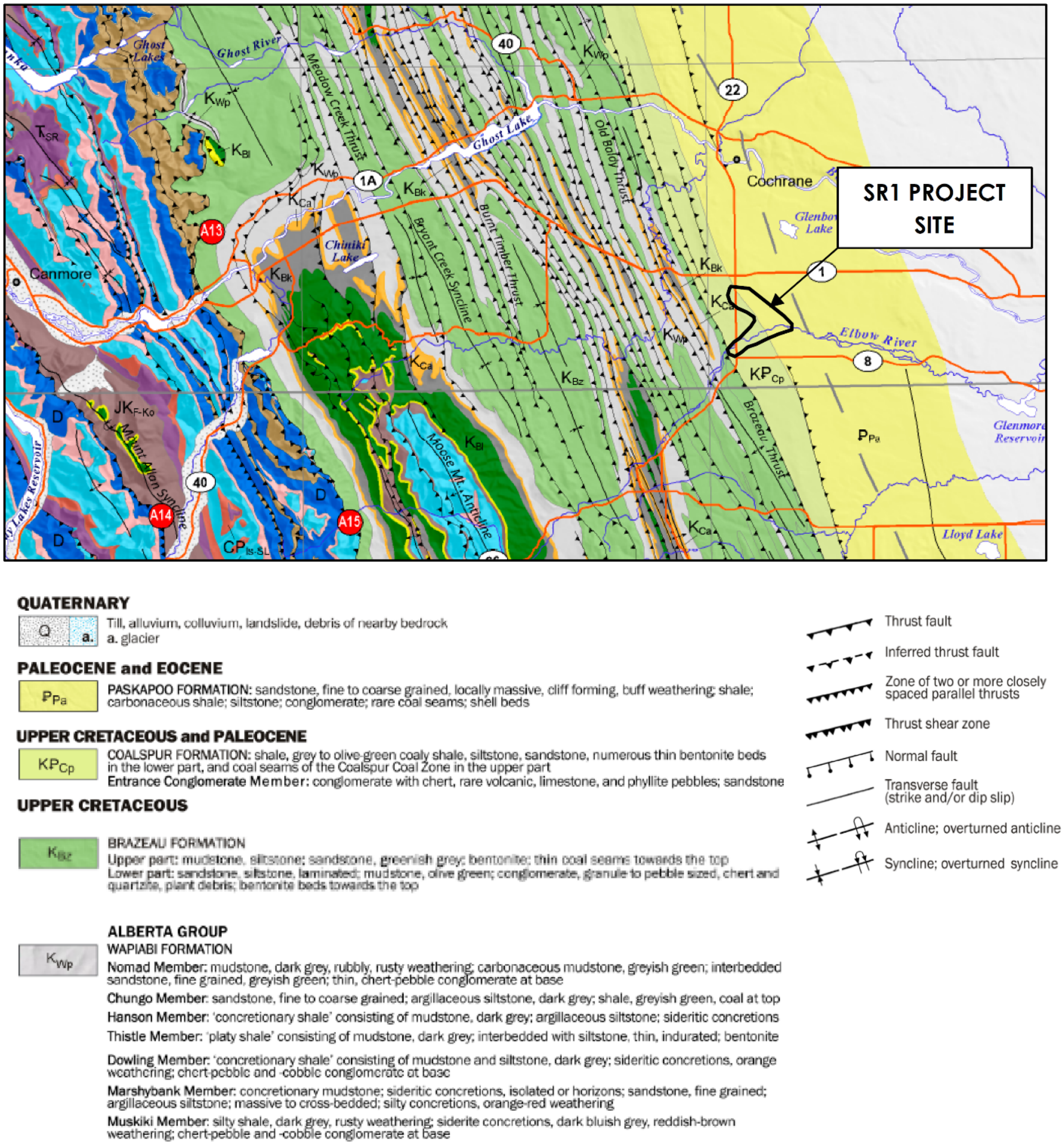


Figure 10. Regional Structural Geology (Extract from AGS Map 560)

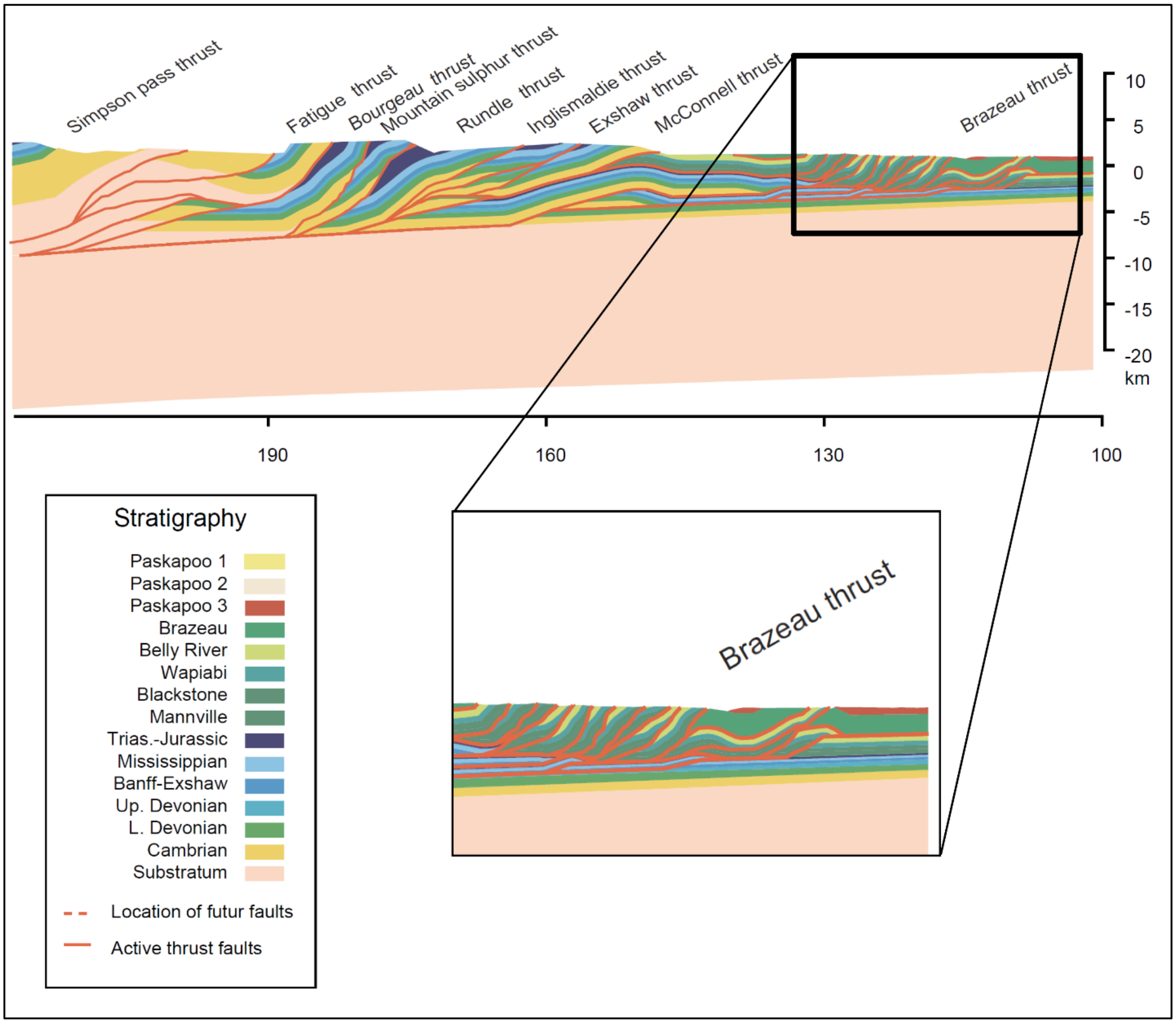


Figure 11. Thrust Emplacement in the Foothills Region (Extract from Faure et al, 2004)

4.5 SEISMOLOGY

The SR1 project site is considered to be in an area of low to moderate seismic activity. The location of historical seismic events within the region is presented in Figure 12. This shows the epicentral locations and magnitudes for recorded earthquakes of all magnitudes up to the end of the 2006.

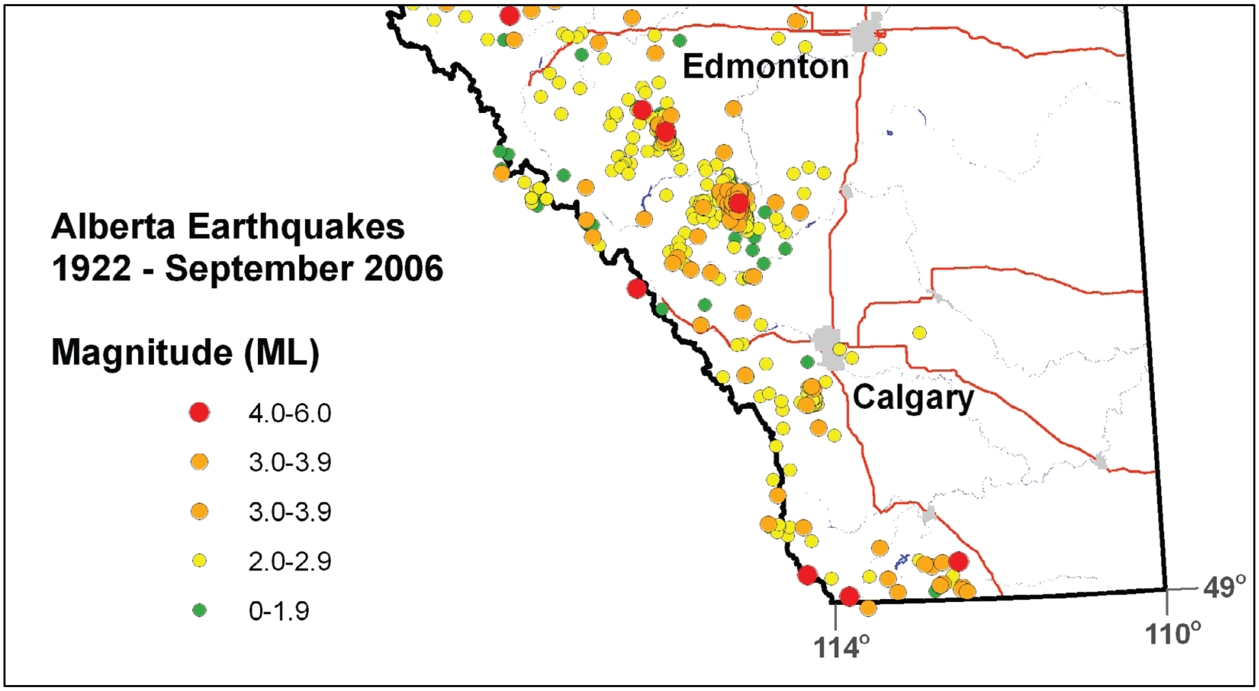


Figure 12. Earthquakes records in Earthquake Canada Database from 1922 to 2006 (extracted from Stern et al, 2013)

The AGS characterize Alberta as a transition from a relatively low-seismicity intraplate regime to a more active foreland belt. Seismic events in Alberta tend to be between micro (0 on the Richter, or local magnitude [ML] scale) and minor (3 ML) in size. Moderate earthquakes with a magnitude greater than 4 ML are rarer (AGS, 2016)

Induced seismicity is common in the foothills region of Southwestern Alberta. Induced seismicity in the foothills region has been associated with both hydraulic fracturing (i.e., "fracking") and waste injection activities associated with oil and gas extraction. Notable areas, where induced seismicity has been documented include the Crooked Lake Sequences to west of Fox Creek, the Brazeau River Cluster and the Rocky Mountain House Seismogenic Zone, located approximately 100 to 150 km northwest of Calgary, and the Cardston Earthquake Swarm located approximately 200 km southeast of Calgary.

4.6 HYDROGEOLOGY

A Regional Groundwater Assessment (RGA) was produced for the RVC by Hydrogeological Consultants (2002) and indicated that both surficial and bedrock aquifers occur within the SR1 project site.

4.6.1 Surficial Aquifers

The RGA (Hydrogeological Consultants, 2002) divides the surficial units within the RVC into two types: the lower surficial deposits comprise pre-glacial fluvial and lacustrine units; and the upper surficial deposits of the 'traditional glacial deposits of till and meltwater deposits'. Within these, three hydraulic components of the surficial aquifer can occur.

- Sand and gravel deposits of the lower surficial deposits. Pre-glacial deposits may exist within the SR1 project site but they have not been identified;
- Saturated pockets of sand and gravel in the upper surficial deposits; and,
- Unsaturated pockets of sand and gravel in the upper surficial deposits.

4.6.2 Bedrock Aquifers

The RGA for Rocky View (Hydrogeological Consultants, 2002) define two 'shallow bedrock' aquifers within the SR1 Project Site:

- The 'disturbed belt' Edmonton Group aquifer. This correlates with the permeable units of the Brazeau and Coalspur Formations. The apparent yields typically range between 10 and 75 m^3/day, although Figure 21 of the RGA shows localized zones of higher than 75 m^3/day near the dam footprint; and,
- The Dalehurst Member aquifer[8]. This is youngest stratigraphic member of the PPF and subcrops to the west of the 5th Meriden. This Member has a maximum thickness of 800 m within the RVC and is mostly composed of shale, siltstone with sandstone, bentonite and coal seams or zones. The apparent yields typically range between 10 and 75 m^3/day. Recharge to the bedrock aquifers within the RVC takes place from the overlying surficial deposits and from flow in the aquifer from outside the RVC.

Grasby et al (2008) divided the coarse-grained, water-bearing facies within the PPF into:

- Thick, stacked multi-storied units (channels) comprising fine to coarse-grained, fining upwards, well-sorted, quartz-chert sandstones with erosional bases. Units can be 3 to 12 m thick and stacked into 50 m thick successions with lateral extents over 100 m; and,
- Thinner units (crevasse splays), typically fair to well-sorted, very fine to fine-grained sandstones with erosional bases, horizontal and ripple laminations. They are typically less than 1 m thick (can be up to 3 m) and have lateral extents up to 50 m.

[8] It is unclear if this is the actually the Dalehurst Member.

The aquitard units comprise 'thin to thick units of greenish-grey, blocky and pedogenically altered, sandy to muddy siltstone with scattered thin fine sandstones beds, roots, wood fragments and caliche' (Grasby et al, 2008).

Grasby et al (2008) made a series of observations on the fracture distribution in the PPF: sandstone outcrops are typically characterized by sub-vertical fracture systems with orientation in NE-SW direction and that there is higher fracture density in thin beds.

4.7 GEOHAZARDS

Slope instability is a common geohazard along the rivers in both the Calgary region and Southern Alberta. Regional landslide susceptibility mapping by the AGS (2016) in Map 605 indicated that the susceptibility of the SR1 Project Site was typically low except for the Elbow River Valley Slopes, which were classified as medium to high. Thompson and Morgenstern (1977) attributed slope instability along the rivers in Alberta to over-steepening of river banks due to toe erosion on the outside of meander bends.

Mollard (1977) defined two types of rock landslide types in the physiographic regions associated with the SR1 project Site: (1) mountain slopes in the Cordilleran region of western Canada, chiefly steeply dipping bedded and foliated rocks; (2) valley sides in Upper Cretaceous argillaceous bedrock, mostly bentonitic marine clay shale, silty shale, and mudstone'. Rock-falls can occur within the BZF, CSF and PPF typically due to undercutting of non-durable layers or wedge failures.

Slope instability in the glacigenic units typically manifest as complex rotational and/or translational failures caused by river erosion or through man-made development, which increases the groundwater elevation, impedes drainage or surcharges the slopes. The stability of slopes in Calgary has been discussed by Osborn and Rajewicz (2008), De Lugt et al, (1993), Osborn (1986), Stepanek and Rodier (1980), Hardy et al (1980) and Osborn (1975).

5.0 SITE CHARACTERIZATION

This section summarizes site characterization of the SR1 Project Site and the development of the Site Engineering Geology Model. The aim of this model is to develop a 3D understanding of the engineering geology of the SR1 Project Site and its geotechnical behavior and properties. The model has been developed using the data collected from the site characterization activities summarized in Section 3.

The results of the field and laboratory testing as well as the process of selecting engineering design parameters are detailed in the "Springbank Off-stream Reservoir (SR1), Updated Geotechnical Materials Properties Design Basis Memorandum – Selection of Soil Material Properties." This memorandum is included in Attachment 5.

5.1 GEOLOGICAL PROFILES

Four (4) longitudinal geological profiles have been developed for this Preliminary Geotechnical Assessment. The location of these profiles are summarized in Figure 20-6. The Geological profiles are:

- Figure 20-7: Geological Profile A - Floodplain Berm and Diversion Structure;
- Figure 20-8: Geological Profile B – Diversion Channel; and,
- Figure 20-9: Geological Profile C – Off-Stream Storage Dam.

5.2 ALLUVIUM

The site characterization activities indicated that two assemblages of alluvium were present within the SR1 Project Site. The principal deposit was associated with the Elbow River Valley with less extensive deposits associated with the tributary creeks, of which the Unnamed Creek was the largest. The composition and engineering properties of these units are discussed below in Sections 5.2.1 and 5.2.2.

5.2.1 Elbow River Sub-Unit

This comprises an assemblage of coarse-grained and overbank alluvial deposits associated with the Elbow River. This sub-unit will be encountered beneath the floodplain berm, auxiliary spillway and the diversion service spillway. The extents of this unit is presented in Figure 20-10d.

This sub-unit was deposited in a broad, post-glacial, terraced river valley. The river valley is approximately 1.8 km wide in the western part of the project site. The active non-vegetated river channel is currently located in the northern limit of the river valley and is between 40 and 170 m wide. The river channel comprises a broad meandering, braided, gravel-river bed with longitudinal and traverse bars, abandoned channels, eroding river banks and localized vegetated, silt overbank deposits present on the established bars and terraces (Figure 13).

The particle size distribution of the gravel bed beneath the floodplain berm ranges between 53 and 79 percent gravel and 17 to 36 percent sand. Round-shapes cobbles of Front Range and Foothills-derived lithology are extensive. The fines content was less than 10 percent and typically comprises silt-sized particles.

This sub-unit was deposited directly onto the underlying BZF. The thickness ranged between 1.8 m on the gravel bars immediately adjacent to the active river channel to 4 m on terraces located approx. 350 m southeast of the active river channel.

Figure 13. Fluvial Morphology of the Elbow River

5.2.1.1 Drained Strength

SPT N values ranged between 13 and 100+ indicating the alluvium could be described as compact to very dense. The range of corresponding peak and critical state friction angles will vary considerably given the range of gradation, density and angularity of the bed load. A minimum critical state friction angle of 32° is recommended for design purposes.

5.2.1.2 Hydraulic Conductivity

No in-situ testing was undertaken on this unit due to the high gravel content. The hydraulic conductivity of this unit has been estimated as 1 x10-6 m/s for design purposes using the published literature for a clean sand of 'medium permeability' (Head, 1985). The use of the PSD data with empirical relationships, such as Hazen's and Kozeney's Formula is unreliable on gravels (Fell et al, 2015).

Figure 14. Diversion Structure Location

Figure 14 shows the location of the Diversion Structure looking upstream. This shows the gravel bed assemblage in the Elbow River. The silt overbank deposits and established vegetation can be seen on the left.

5.2.2 Unnamed Creek Sub-Unit

This comprises an assemblage of fluvial deposits associated with the Unnamed Creek. The creek is located in a sinuous-shaped, over-sized valley (Figure 15 and Figure 16). The gradient of the river valley base increases in steepness from northwest to the southeast with up to 4 m of downcutting at the dam footprint. The width of the valley ranges between approx. 110 and 170 m at the dam footprint. Further towards its confluence with the Elbow River, the valley becomes deeper but the width does not change significantly.

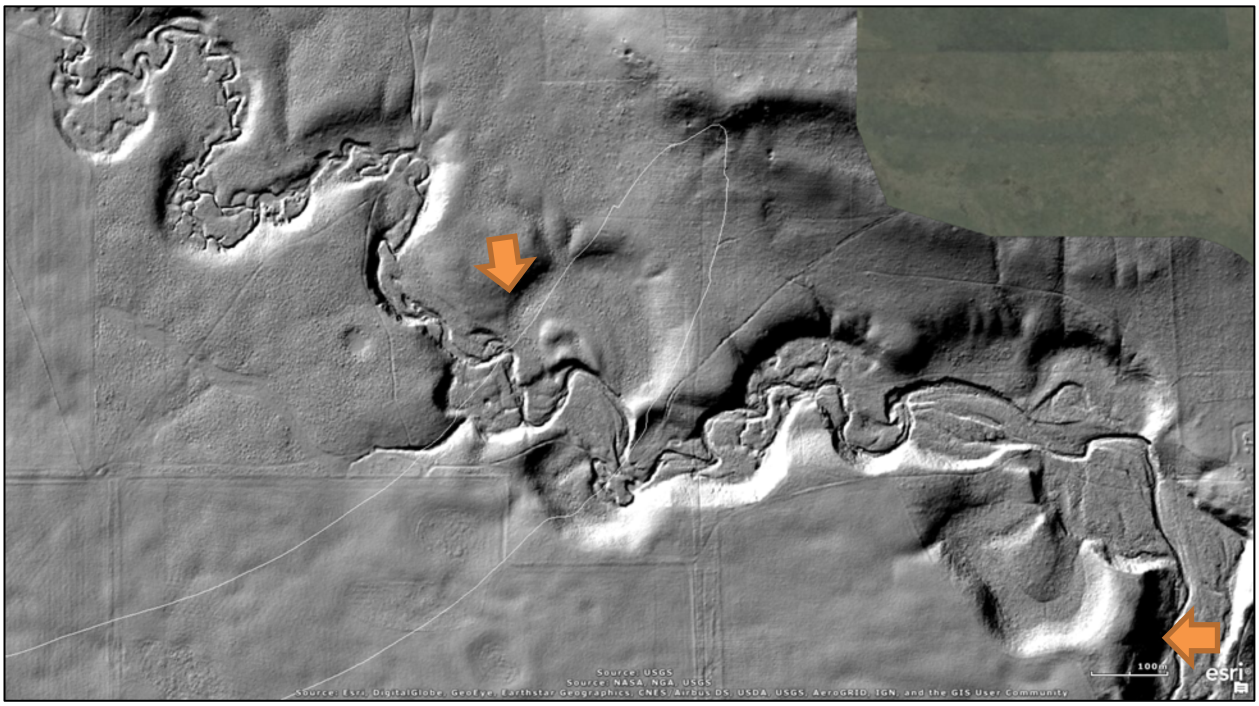


Figure 15. DEM of Unnamed Creek Valley

One-meter digital elevation model (DEM) of the Unnamed Creek Valley. The sinuous geomorphology of the over-sized, post-glacial valley and abandoned channels can be visualized. The current river channel is undersized compared to the existing valley. Arrows show locations and orientations of the photos in Figure 16 and Figure 17.

Figure 16. Unnamed Creek Valley Looking South

The exploratory hole data indicated that at the dam footprint, the valley is infilled with variable alluvial deposits. There is a basal unit of very dense to compact sand and gravel with frequent cobbles. This is between 2 and 2.5 m thick and deposited directly onto the PPF bedrock. This is overlain by localized deposits of overbank alluvium and organic deposits between 1.5 and 6 m thick. This comprises very stiff, brown, low to medium plasticity, silty clay with occasional sand, gravel and cobbles.

At OC18, which is located between the dam footprint and the confluence with the Elbow River, the alluvium comprised angular-shaped boulders of sandstone interbedded with round-shaped gravel and cobbles (Figure 17).

Figure 17. West Bank of Unnamed Creek River Valley showing angular blocks of sandstone interbedded within the alluvium.

5.2.2.1 Index Properties

The distribution of index properties and particle size with elevation and depth are presented in Figure 20-11.

5.2.2.2 Phase Relationships

The moisture content, specific gravity and unit weights for the glacigenic units are summarized in Table 4**.**

Table 4. Phase Relationships for Alluvium

Material Name	G_s	Moist w (%)	Sat w (%)	e	γ_d (kPa)	γ_m (kPa)	γ_{sat} (kPa)
Elbow River Alluvium	2.65	6.0	22.2	0.59	16.4	17.4	20.0
Unnamed Creek Alluvium	2.67	6.0	6.0	0.59	19.8	21.0	21.0

5.2.2.3 Stress History

No consolidation testing was undertaken on the overbank units due to the high sand and gravel contents. From a geological perspective, these units should be normally-consolidated given that they were deposited in a post-glacial setting; however, CPT profiling suggests that these units have an OCR greater than ten. This indicates that these units may have undergone mechanical over-consolidation due to desiccation, freeze-thaw and lowering of the water table.

5.2.2.4 Undrained Strength Profile

The S_U profile from the CPT indicated that the S_U of the silt overbank was typically greater than 150 kN/m^2. The CPT had limited penetration into the basal sand and gravel alluvium.

5.2.2.5 Hydraulic Conductivity

The characteristics of the alluvial sand and gravel layer varied significantly between the borings advanced along the creek channel. An average hydraulic conductivity, K_H of $1x10^{-6}$ m/s and K_H/K_V of 1.0 has been selected for the Preliminary Design. This is broadly representative of a clay gravel (GC), silty sand (SM) and clayey (SC) sand. Given the occurrence of these units within all of the boreholes, it assumed that this sand and gravel layer is laterally extensive, continuous and hydraulically-connected.

5.2.3 Selected Design Parameters

The following design parameters for the alluvial soils have been selected for design:

- Moist Unit Weight, γ_m = 17.4 kPa
- Minimum friction angle, ϕ' = 32°
- Average hydraulic conductivity, K_H = $1x10^{-6}$ m/s, K_H/K_V of 1.0

5.3 COLLOVIUM

The site characterization activities indicated that two assemblages of colluvium were present within the SR1 Project Site. The principal deposit was associated with the landslides and eroded, over-steepened slopes of the Elbow River Valley with less extensive deposits associated with the Storage Dam NE Ridge abutment. The composition and engineering properties of these are discussed below in Sections 5.3.1. and 5.3.2.

5.3.1 Elbow River

The site walkovers and aerial photographs have indicated that the natural slopes of the Elbow River Valley exhibit extensive slope instability, with complex rotation failures being the dominant failure mechanism. Rock-falls, debris flows and localized slumping are also present.

5.3.1.1 West of Highway 22

The Elbow River has post-glacially down-cut into the BZF and glacigenic units to form a series of steep-sided slopes modified by rock-falls and relict landslides. Frequently observed failure mechanisms include:

- Rock-falls of steeply dipping durable sandstone undercut by non-durable mudstones;
- Raveling of fissured mudstones, shales and desiccated clays;
- Relict rotational landslides within the glacigenic units; and,
- Debris flow within the glacigenic units located approx. 200 m downstream of the Diversion Sluiceway

The toe of the slumped soil and vegetation mass was 15 m wide and had locally displaced the river. The spherical backscarp was located above inferred bedrock.

Figure 18. Slope Instability observed on the Elbow River Valley to the west of the Highway 22 Road Bridge

5.3.1.2 East of Highway 22

The river valley is broad with localized shallow slopes to the east of the Highway 22 Road Bridge. Slope instability is limited to isolated slab rock-falls in the BZF and CSP due to undercutting of the sub-horizontal to horizontal sandstone units.

5.3.1.3 Downstream of the Dam

The top of the PPF bedrock dips below the current level of the Elbow River in this area resulting in 15 to 20 m high slopes comprised entirely of GL, UBT and LBGT. With no resistant bedrock controls at the toe, these slopes undergo frequent slope instability and toe erosion with complex rotation failures being the dominant failure mechanism. The slopes are currently a dynamic system, in which equilibrium is recurrently disturbed by channel migration. Based on observations from site, the following cycle of slope regression is proposed:

- Slope at equilibrium;
- Channel migration towards the slope;
- Initiation of erosion of the toe (disturbing agent);
- Over-steepening of the slope with subsequent rotational failure. This forms a localized circular back-scarp in plan-view divided by spines of steeper material (Figure 19);
- The steeper spines lose support and material gradually fail (Figure 20); and,
- Surface water, progressive slumping and establishment of vegetation further modify the slope until it’s reaches equilibrium.

Figure 19. Slope Instability observed in glacigenic units on the Elbow River Valley downstream of the dam

5.3.2 NE Abutment Ridge

A 6.8 m thick zone of colluvium was encountered on the SW face of the NE Abutment Ridge in D52. This was encountered within a rotary core between Elevation 1201.4 and 1194.7 m as it was assumed to be bedrock during the field work (Figure 20). This zone comprised angular and rounded, cobble-sized fragments of sandstone within a 'matrix' of silty sand. This was overlain by residual silt and clay deposits.

Figure 20. Zone of Colluvium Observed on the NE Abutment

This was not encountered in adjacent boreholes undertaken at lower elevations: BS1 and D51 indicating that this may occur in the upper slopes of the ridge only.

5.4 GLACIAL SOILS

The SR1 Project Site is blanketed with a widespread and complex assemblage of glacigenic deposits representative of subglacial and supraglacial depositional settings. The associated landforms, types, composition and engineering properties of these units are discussed below in Sections 5.4.1 to 5.4.3.9.

5.4.1 Landforms

The 1 m and 15 m-scale LIDAR DEM for the SR1 Project Site and surrounding area (Figure 7) indicate that the following glacial landforms are present:

- Flow-set of WNW-ESE lineations indicative of Cordillerian ice-flow from the Bow Valley. These could represent flutes, drumlins or streamlined bedrock;
- Hummocky terrain indicating stagnant ice is present to the northeast and southeast of the SR1 project site; and,
- Within the SR1 reservoir, circular depressions indicative of stagnant ice or grounded icebergs can be observed on the 1m-scale DEM.

5.4.2 Types

The site characterization activities have identified five (5) glacigenic units within the SR1 Project Site. For the purpose of this assessment, glacigenic units displaying a diamicton fabric are termed 'glacial tills'. A diamicton can be defined as a '*non-sorted or poorly sorted, unconsolidated sediment containing a wide range of particle sizes for which no genesis is presumed*' (Bennett and Glasser, 2009). The five (5) sub-units were classified based on observations from boreholes and outcrops such as changes in color, fabric, clast lithology and shape; index properties; particle size distribution and CPT profiling.

These five units are listed below and discussed further in Sections 5.4.2.1 to 5.4.2.5.

- Glacial-lacustrine (GL) clays and silts;
- Upper Brown Till (UBT);
- Brown-Grey Subglacial Till (BGST);
- Basal Granular Till (BGT); and,
- Lower Grey Subglacial Till (LGST).

The extents of each glacigenic unit are presented in Figure 20-10a, b and c. The distribution of index properties and particle size with elevation and depth are presented in Figure 20-12 and summarized below in Table 5.

Table 5. Summary of Index Properties and Composition for Glacigenic Sub-Units

Property	GL	UBT	BGST	BGT	LGST
Moisture content	17.7 - 34 (26.4)	11.8 - 26 (17.2)	3 –23.3 (13.4)		10 - 24.1 (15.1)
Clay content	37.7 - 93.7 (59)	1 - 56.6 (31.3)	3.7 – 49.4 (29)	4.7 - 12.7 (8.8)	11 - 43.4 (30)
Silt content	5.9 - 54 (37.8)	9 - 73 (46.4)	14 - 82.1 (42.5)	9.6 - 18.6 (14)	35.9 - 51.1 (43.3)
Sand content	0 - 10.5 (2.7)	3.1 - 33.4 (15.8)	2.3 - 56.9 (20.2)	25.1 - 34 (28.7)	10.9 - 23.7 (17.3)
Gravel content	0 - 11.7 (0.5)	0 - 58.8 (6.5)	0 - 40.8 (8.3)	38.9 - 59.9 (48.6)	1.7 - 26.1 (10.3)
Liquid limit	40 - 78 (55)	21 - 53 (33)	21 - 47 (33)	-	20 - 44 (33)
Plastic Index	22 - 62 (35)	-	3 - 30 (18)	-	5 - 28 (19)
Liquidity Index	0 - 0.6 (0.2)	-	-1.3 - 0.3 (-0.2)	-	-1.0 - 0.7 (0)
Activity	0.4 - 1.0 (0.6)	-	0.2 - 1.2 (0.6)	-	0.5 - 0.7 (0.6)

5.4.2.1 Glacio-Lacustrine Clay and Silt

The GL was encountered beneath the dam footprint and the diversion channel within the exploratory holes and the geological mapping of outcrops. The extents of this unit within the SR1 Project Site is presented in Figure 20-10a. It was always encountered at the top of the glacigenic sequence, near the existing ground level. SPT N values indicated that the density of this unit was ‘stiff to hard’ with typical values between 15 and 25.

The GL was typically encountered as olive brown to brown, medium to high-plastic, clay and silt. The thickness it ranged between 0.5 and 16 m. Photos of recovered samples are presented in Figure 21 and Figure 22. The distribution of index properties and particle size with elevation and depth for the GL is presented in Figure 20-12 and summarized in Table 5.

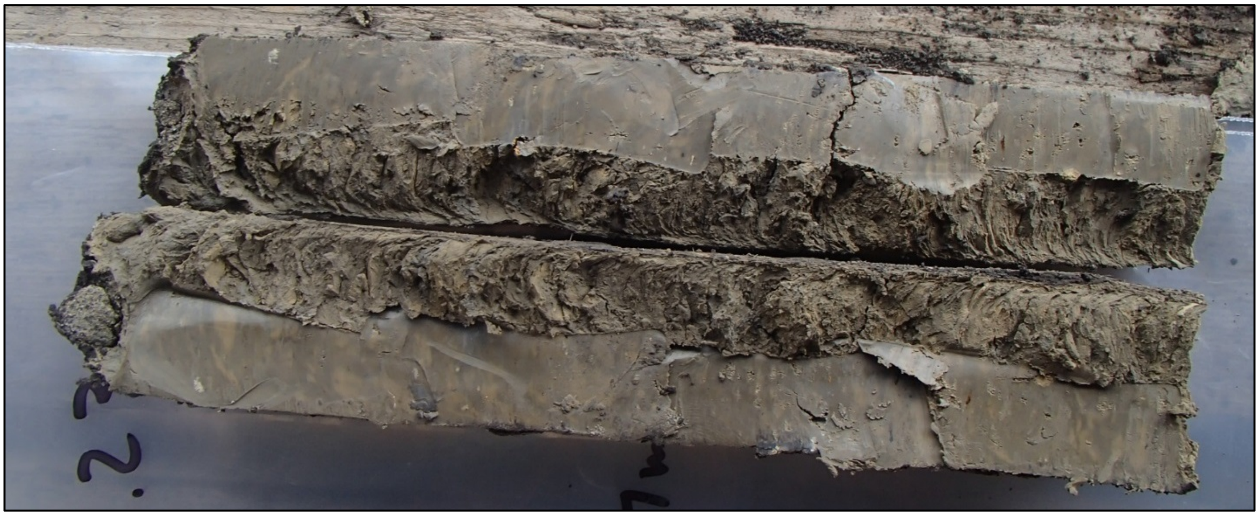


Figure 21. Sample of Glaciolacustrine Clay and Silt Recovered from 2.3 to 2.7 m depth in D20

Figure 22. Sample of Glaciolacustrine Clay and Silt Recovered from 4.8 to 5.3 m depth in D32

Index testing indicate that this unit was a medium to high plasticity clay with silt. The LL ranged between 41 and 78 percent with approx. 2/3 of the test results have a LL greater than 50. The PI ranged between 23 and 62 with the majority of the test results between 30 and 40. The LI was typically between 0 and 0.1.

The activity ranged between 0.4 and 1.0 with an average value of 0.6. This indicated that the GL ranged between inactive and normal based on Skempton (1953).

Clay was the dominant fraction typically comprising 50 and 70 percent. There were no trends with depth or elevation. The silt content was lower and typically ranged between 30 and 45 percent. There was typically less than five percent sand although this could reach 10 percent. There were rare gravel and cobble sized fragments.

The fabric of this unit was observed in the outcrops along the Elbow River downstream of the dam footprint and east of the Highway 22 Road Bridge (Figure 23 and Figure 24). The observations are summarized below:

- The diagnostic characteristic was a laminated-fabric typically encountered in the upper 2 to 3 m. This comprised undulating, alternating light and dark bands and the lack of gravel-sized clasts indicative of a diamicton. It is unknown if these bands represent alternating clay and silt layers; and,
- The contact with the underlying UBT was gradational.

Figure 23. Rhythmically-bedded Glacio-lacustrine clay and silt observed in the upper part of OC14.

Figure 24. Rhythmically-bedded Glacio-lacustrine clay and silt observed at the top of OC9.

This unit is likely to be representative of the Calgary Formation (Section 4.2.3). The fabric is indicative of a lake bottom environment with deposition controlled by underflows, interflows, overflows and occasional ice-bergs (Carrick and Tweed, 2013). This is the last phase of glacigenic deposition to affect the SR1 Project Site. The snouts of the Cordillerian and Laurentian Ice-sheets would have retreated to the west and east of the site at the time of deposition.

5.4.2.2 Upper Brown Till

This unit was encountered beneath the GL within the dam footprint and the eastern portion of the diversion channel. The extents of this unit are presented in Figure 20-10c. The UBT was typically encountered as an olive brown to brown, medium plastic, clay and silt with increased sand content with depth. SPT N values indicated that this unit was typically less dense than the underlying BGST and LGT. The distribution of index properties and particle size with elevation and depth for the UBT is presented in Figure 20-12.

This unit was compositionally different to the overlying GL. Index testing indicate that this unit was a medium plasticity silt with clay and sand. The LL was typically between 20 and 40 percent and decreased with depth. The PI was typically between 10 and 25. The LI was typically between -0.2 and 0.4.

The activity ranged between 0.4 and 1.0 with an average value of 0.6. This indicated that the GL ranged between inactive and normal based on Skempton (1953).

Silt was the dominant fraction typically comprising 35 and 50 percent. The clay content was more variable and ranged considerably, with values between 10 and 50 percent. The sand content ranged between 10 and 30 percent and there typically up to 10 % gravel, although higher contents up 59 percent were locally encountered.

The fabric of this unit was observed in the outcrops along the Elbow River downstream of the dam footprint and east of the Highway 22 Road Bridge. The observations are summarized below:

- In the outcrops, the UBT comprises a massive, matrix-supported, brown-olive brown diamicton. Weathered outcrops were light brown to cream in color. This fined upwards into the overlying GL. Localized winnowing was observed;
- A localized zone of stratification was observed in OC14. This stratified layer was interbedded within the diamicton fabric (Figure 26) and contained a drop-stone that visibly deformed the surrounding matrix; and,
- In OC16, the base of this unit contained rounded cobble-sized clasts (possible dropstones but unable to get close enough to confirm).

Figure 25. Recovered Sample of Upper Brown Till from 6.8 to 7.2 m in D17

Figure 26. Stratified Layer observed near the base of the Upper Brown Glacial Till in OC14

It is likely that this unit represents a waterlain till. This was deposited in a glacio-lacustrine lake shore environment with deposition controlled by submerged ice ramps, ice cliffs and calving ice-bergs.

5.4.2.3 Brown-Grey Subglacial Till

The BGST sub-unit was identified throughout the SR1 project site, in particular within the diversion channel. The extents of this unit are presented in Figure 20-10b. The BGST was typically encountered as a dark brown to grey, sandy, silty Clay with variable gravel content. A photo showing a typical sample of this material is shown in Figure 27. SPT N values indicated that the density of this unit was 'hard' with typically +50 blows.

Index testing indicated that this unit was low to medium plasticity silt with clay and sand. The LL was typically between 20 and 40 percent and decreased with depth. The PI was typically between 5 and 25. The LI was typically between 0 and -0.5 with outliers up to 0.8.

The activity ranged between 0.2 and 1.2 with an average value of 0.6. This indicated that the GL ranged between inactive and normal based on Skempton (1953).

Silt was the dominant fraction typically comprising 30 and 50 percent. The clay content was more variable and ranged considerably, with values between 10 and 40 percent. The sand content ranged between 10 and 30 percent and there was typically up to 20 percent gravel.

The fabric of this unit was observed in the outcrops along the Elbow River throughout the SR1 Project Site. The observations are summarized below:

- Thicker accumulations of the BGST were mapped to the west of the Highway 22 Bridge in OC1 to OC5 Figure 28;
- The BGST was weathered and altered to an olive-light brown color when exposed in the outcrops;
- The contact with the underlying bedrock is undulating and sharp;
- Clasts comprised rounded to sub-rounded cobble-sized fragments of sandstones and carbonates;
- The fabric of elongated clasts in OC5 was in a broad NW-SE direction; and,
- Localized, sub-horizontal clusters of cobble-sized fragments were encountered in OC5.

Figure 27. Typical Sample of the Brown-Grey Subglacial Till

Figure 28. Brown-Grey Subglacial Till Observed in OC5

It is likely that this unit represents a subglacial depositional environment indicative of the Lower Spy Hill Glacial Till. Glacial landforms mapped in the DEM for the SR1 Project Site indicate that this unit was deposited at the base of the Cordillerian Ice-Sheet that moved over the SR1 Project Site in a NW-SE direction.

5.4.2.4 Basal Granular Till

The BGT sub-unit was identified in the western portion of the proposed Diversion Channel between Station 10+000 and 10+600 m and near the Diversion Structure. The extents of this unit are presented in Figure 20-10c. The BGT was typically encountered as a brown, well-graded, sand and gravel with a variable fines content (see Figures 29 and 30). SPT N values indicated that the stiffness of this unit was 'hard' with typically +50 blows.

The fabric of this unit was observed in the outcrops along the Elbow River throughout the SR1 Project Site. The observations are summarized below:

- The BGT comprised a 0.5 to 1 m thick layer of a light grey to brown, clast-dominated diamicton.
- White and orange staining was observed in this layer; and,
- The contact between the underlying bedrock was sharp, undulating and infilled depressions in the bedrock.

Figure 29. Typical Sample of the Basal Granular Till

Figure 30. Basal Granular Till Overlying BSF at OC3

5.4.2.5 Lower Grey Subglacial Till

The LGST unit was identified beneath the dam footprint by the boreholes and geological mapping of OC16 and OC17. The extents of this unit are presented in Figure 20-10b. This unit was encountered above the PPF in the deepest portion of the valley between approximate Station 22+300 and Station 23+500 m. The top of this unit ranged between Elevation1187.7 and 1173.4 m. The thickness ranged between 1 and 9.3 m.

The index properties and particle size distribution was similar to the BGST[9]. The LL typically ranged between 30 and 40 percent. The PI ranged between 14 and 28. The LI was typically between 0 and -0.2, however, there were outlier values between 0.2 and 0.7.

This unit contained between 35 to 51 percent silt and 20 to 37 percent clay. The sand content was less than the BGST and ranged between 10 and 20 percent. There was typically up to 20 percent gravel.

[9] The dataset is smaller compared to the BGST.

The diagnostic characteristic of this unit was its grey colored matrix. The fabric of this unit was observed in two outcrops along the Elbow River in OC16 and OC17 (Figure 20-2). The observations are summarized below:

- Comprises a massive, matrix-supported, grey diamicton;
- The contact between the units was sharp, i.e. not gradational and undulating;
- Orthogonal-distributed fissures could be observed in OC16; and,
- Imbrication of clasts (approximate NW-SE direction) was observed in OC17.

Figure 31. Lower Brown Subglacial Till and Overlying Upper Brown Till in OC15

5.4.3 Insitu Engineering Properties

For this assessment, the five (5) glacigenic units have been grouped into two (2) geotechnical design units based on the index properties of each unit. It is assumed each group will have similar engineering characteristics and will behave in a similar manner for the slope stability and seepage analyses:

- The UBT, BGST, LGST, and BGT sub-units have been grouped together into a single glacial till (GT) unit; and,
- The GL unit is considered as a separate glacigenic unit.

5.4.3.1 Soil Testing

A summary of laboratory tests performed on samples obtained from the 2016 and 2018 geotechnical explorations is presented in Table 20.

Table 6. Summary of Laboratory Tests Performed (2016 and 2018 Explorations)

Laboratory Test	Diversion Structure	Floodplain Berm	Diversion Channel	Dam	Highway	Borrow Source	Hydro-geology	TOTAL
Atterburg Limits	7	4	117	224	20	37	35	444
Grainsize Analysis	10	4	110	228	19	37	35	443
Specific Gravity	5	---	1	---	---	---	---	6
Standard Proctor	1	---	16	1	4	10	1	33
Hydraulic Conductivity	---	---	13	17	2	3	1	36
Consolidation	---	---	2	42	---	---	---	45
Direct Shear	---	---	1	8	---	---	---	9
Unconfined Compressive Strength	---	---	4	4	1	---	---	9
Consolidated Undrained Triaxial	---	---	21	49	6	7	2	85
Unconsolidated Undrained Triaxial	---	---	5	6	---	---	---	11
Swell Test	---	---	1	---	---	---	---	1
Crumb Test	---	---	2	---	---	---	---	2
Pinhole Test	---	---	2	---	---	---	---	2

Table 6. Summary of Laboratory Tests Performed (2016 and 2018 Explorations) (Continued)

Laboratory Test	Diversion Structure	Floodplain Berm	Diversion Channel	Dam	Highway	Borrow Source	Hydro-geology	TOTAL
Double Hydrometer	---	---	2	---	---	---	---	2
Carbonate Content	---	---	3	7	---	---	---	10

5.4.3.2 Soil Classifications

Soil classifications were determined by the Unified Soil Classification System (USCS) (ASTM D2487) from laboratory testing consisting of Atterberg limits testing (ASTM D4318) and hydrometer grainsize analyses (ASTM D422). A summary of the USCS soil classifications determined for glacial lacustrine soil samples obtained in the different project zones is presented in Table 7.

Table 7. Summary of Glacial Lacustrine USCS Soil Classifications Grouped by Project Zone

Feature	Zone	Borings	Number of USCS Classifications	USCS Classifications
Floodplain Berm	1	FB3 thru FB7	0	No GL Soils
Diversion Structure	2	DS1 thru DS9	1	CL (1)
Diversion Channel	3	DC1 thru DC12	3	CH (2) CL (1)
	4	H10 thru H13	4	CH (1) CL (3)
	5	DC13 thru DC17	7	CH (7)
	6	H1 thru H4	1	CH (1)
	7	DC19 – DC24	8	CH (8)
	8	DC25 thru DC34	10	CH (8) CL (2)
Storage Dam	9	D1 thru D9	8	CH (7) CL (1)
	10	D10 thru D25	29	CH (26) CL (3)
	11	D26 thru D40, D42, D57 thru D63 GL1 thru GL4	57	CH (22) CL (34) SC(1)

Feature	Zone	Borings	Number of USCS Classifications	USCS Classifications
		LLO01 thru LLO06, LLO17 thru LLO18		
	12	D41 thru D50 LLO06 thru LLO07	0	No GL Soils
	13	D51 thru D52 LLO09 thru LLO16	4	CH (3) CL (1)
	Borrow	BS1 thru BS5	16	CH (6) CL (10)
	Geo Wells	GW1 thru GW11	16	CH (10) CL (6)
Total Site			164	CH (101) CL (62) SC (1)

Approximately 62 percent of the glacial lacustrine soil samples classified as CH (Clay of High plasticity) and 38 percent classified as CL (Clay of Low plasticity).

USCS soil classifications determined for glacial till soils from samples obtained from the different project zones are summarized in Table 8.

Table 8. Summary of Glacial Till USCS Soil Classifications Grouped by Project Zone

Feature	Zone	Borings	Number of USCS Classifications	USCS Classifications
Floodplain Berm	1	FB3 thru FB7	0	No GT Soils
Diversion Structure	2	DS1 thru DS9	9	CL (8) GM (1) *
Diversion Channel	3	DC1 thru DC12	42	CL (38) GM (2) * GC (1) * SC (1) *
	4	H10 thru H13	5	CL (5)
	5	DC13 thru DC17	12	CL (7) CL-ML (2) ML (1) SC-SM (1) SC (1)
	6	H1 thru H4	6	CL (4) ML (1) SC (1)
	7	DC19 – DC24	7	CL (4) CL-ML (2) SC (1)
	8	DC25 thru DC34	18	CL (15) CL-ML (2) GC (1) *

Feature	Zone	Borings	Number of USCS Classifications	USCS Classifications
Storage Dam	9	D1 thru D9	8	CL (7) CL-ML (1)
	10	D10 thru D24	23	CL (21) CL-ML (1) GC (1) *
	11	D26 thru D40, D42, D57 thru D63 GL1 thru GL4 LLO01 thru LLO06, LLO17 thru LLO18	38	CL (32) CL-ML (1) ML (2) SC (1) GW (2) *
	12	D41 thru D50 LLO06 thru LLO07	24	CL (14) SC (1) SM (1) GC (2)* GW (2)* GW-GM (1)* GM (3)*
	13	D51 thru D52 LLO09 thru LLO16	6	CL (6)
	Borrow	BS1 thru BS5	14	CL (14)
	Geo Wells	GW1 thru GW11	15	CL (14) CL-ML (1)
Total Site			227	CL (189) CL-ML (10) ML (4) SC-SM (1) SC (6)* SM(1) GC (5)* GM (6)* GW-GM (1) GW(4)*

*Basal Sand/Gravel Till Layer

Approximately 83 percent of the glacial lacustrine soil samples classified as CL (Clay of Low plasticity).

5.4.3.3 Unit Weight

Unit weight values were determined (ASTM D2167) for 88 undisturbed GT and GL samples. A summary of the insitu dry and moist unit weight values is presented in Figure 32 and Figure 33.

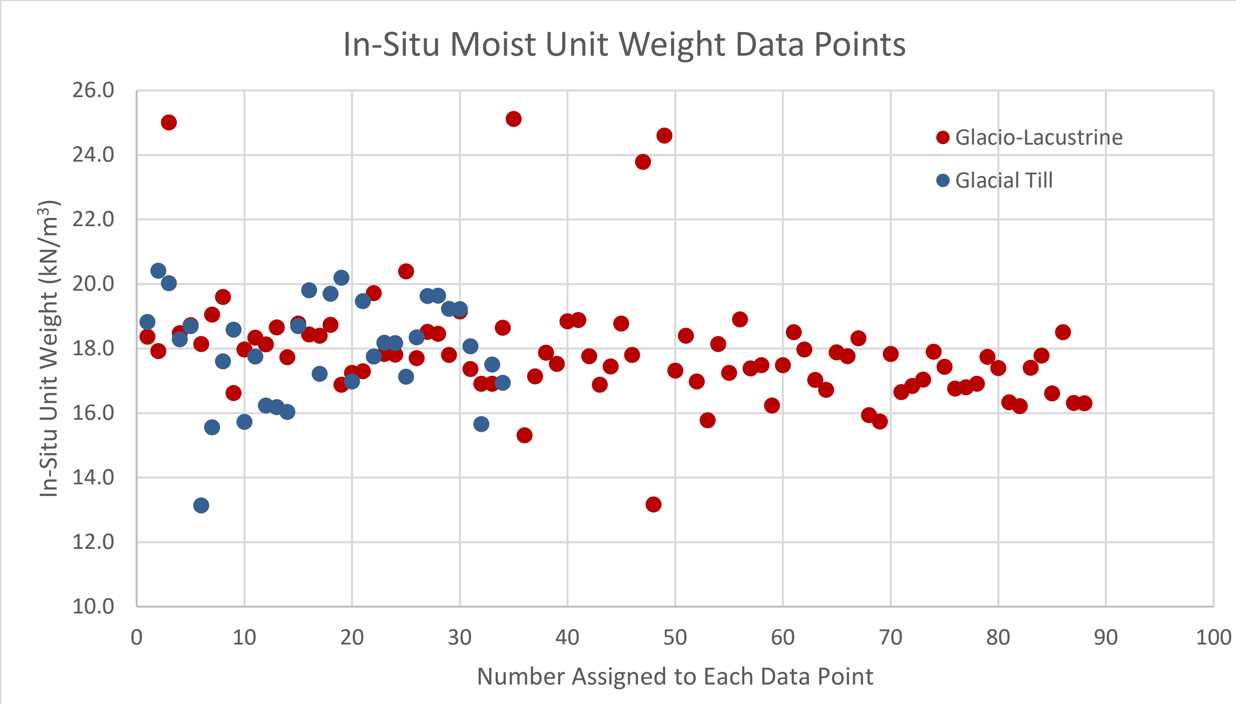


Figure 32. Project Wide In-Situ Unit Weight Values – GT and GL

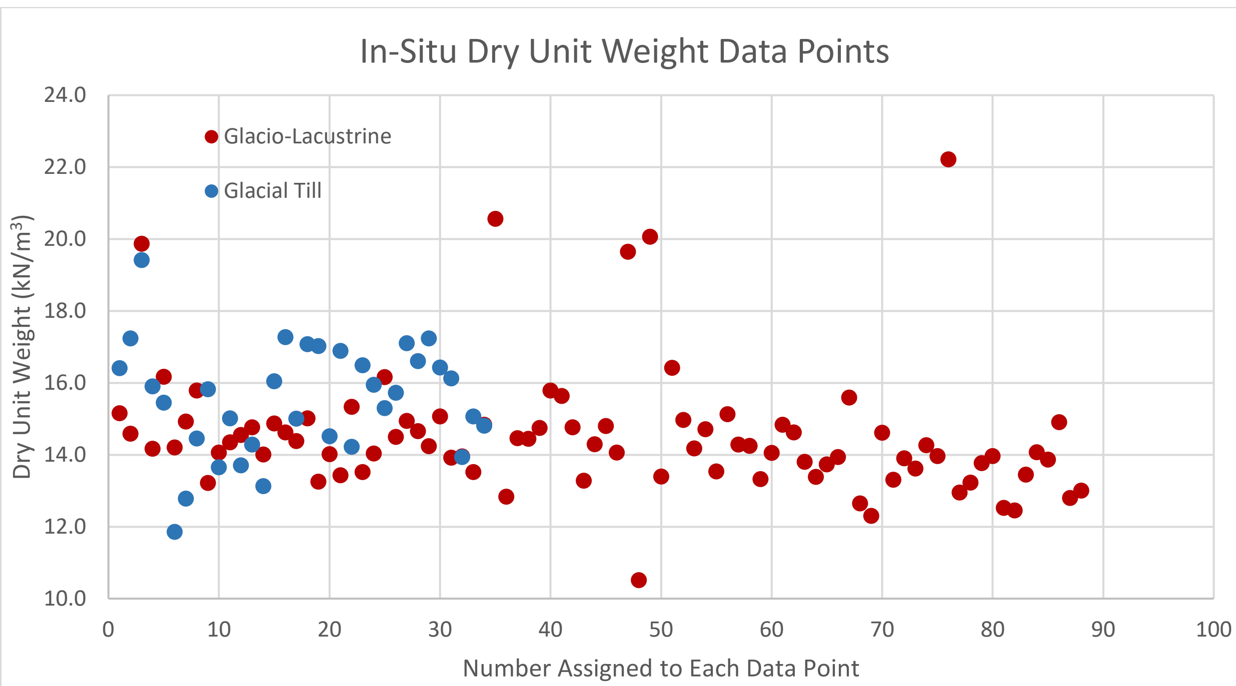


Figure 33. Project Wide Dry Unit Weight Values – GT and GL

5.4.3.4 Moisture Content

Project wide natural moisture content values from laboratory test results were reviewed. Plots for glacial lacustrine and glacial till soil sample natural moisture content are included in Figure 34 and Figure 35.

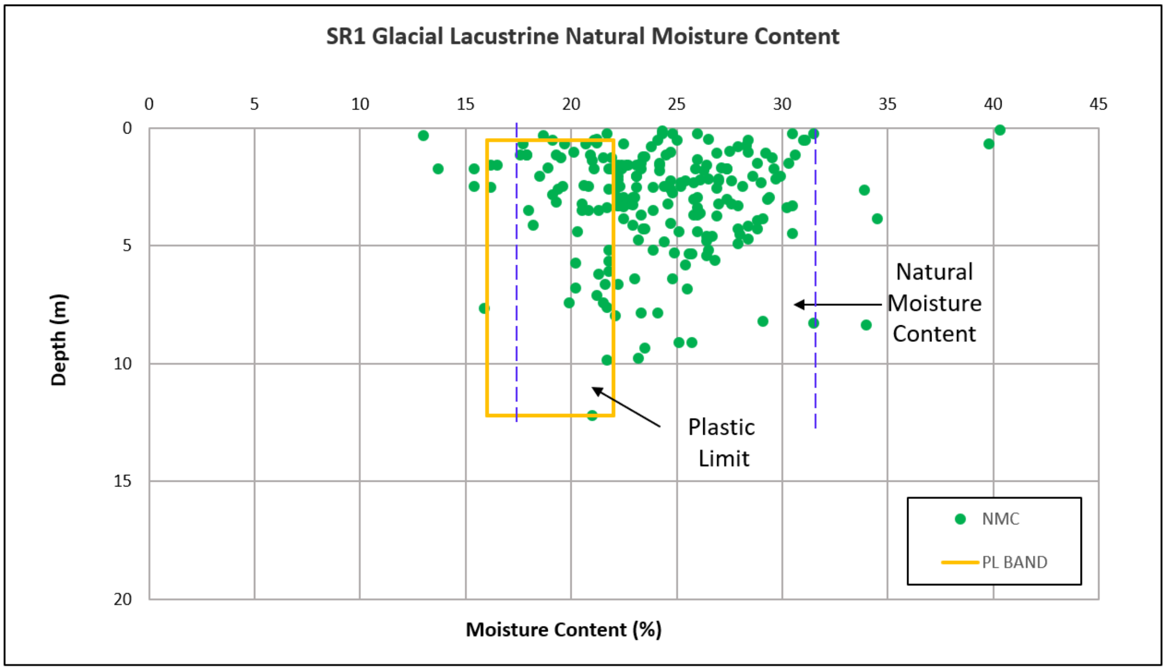


Figure 34. Glacial Lacustrine Natural Moisture Content

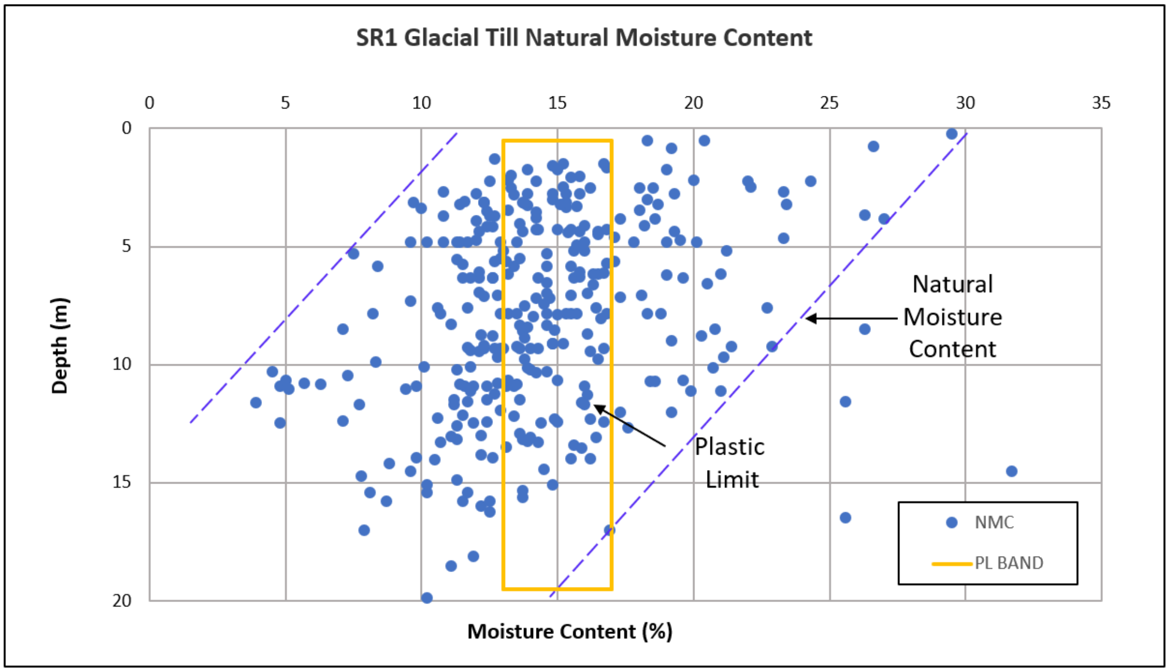


Figure 35. Glacial Till Natural Moisture Content

5.4.3.5 Consolidation Stress History

The yield stress and over-consolidation ratio (OCR) for the glacigenic deposits beneath the dam footprint have been estimated from 1D consolidation testing and CPT data. Forty-four (44) one-dimensional consolidation tests (ASTM D2435) were undertaken on undisturbed GL and GT samples. The yield stress (commonly referred to as the pre-consolidation stress) was calculated using the A. Casagrande (1936) Approach and used to derive the OCR, recompression index (C_r), and compression index (C_c) for each test.

Thirty-three (33) one-dimensional consolidation tests were undertaken on undisturbed GL samples. The data from the consolidation testing is presented in Table 9.

Table 9. Summary of Consolidation Test Results – Glacial Lacustrine

ID	Unit	Depth (m)	Pre-Consolidation Pressure (kPa)	In-situ Void Ratio	OCR	C_c	C_r
DC33	GL	3.00-3.60	190	0.894	3.2	0.25	0.09
DC34	GL	3.00-3.60	180	0.760	3.0	0.27	0.06
D2	GL	1.50-1.95	205	0.767	6.6	0.21	0.09
D12	GL	2.70-3.20	400	0.696	7.6	0.24	0.06
D14	GL	3.00-3.50	125	0.629	2.1	0.13	0.04
D14	GL	4.60-5.10	265	0.778	3.0	0.20	0.05
D14	GL	6.10-6.60	270	0.690	2.4	0.17	0.03
D16	GL	3.00-3.45	230	0.706	4.0	0.15	0.03
D20	GL	0.90-1.35	190	0.516	9.4	0.13	0.04
D20	GL	2.70-3.20	160	0.652	3.0	0.16	0.05
D20	GL	5.40-6.00	285	0.654	2.8	0.24	0.08
D20	GL	7.60-8.05	180	0.580	1.3	0.09	0.01
D28	GL	3.50-3.92	275	0.607	4.1	0.21	0.04
D30	GL	1.70-2.15	100	0.526	2.9	0.13	0.03
D30	GL	4.40-4.85	120	0.581	1.4	0.12	0.03
D36	GL	4.50-4.95	280	0.619	3.3	0.23	0.03
D51	GL	2.70-3.15	270	0.573	5.1	0.21	0.06
D59	GL	2.40-2.89	260	0.629	5.5	0.19	0.05
D60	GL	0.80-1.25	140	0.767	7.6	0.21	0.04
D68	GL	4.40-4.85	160	0.448	1.9	0.12	0.02
LLO05	GL	3.00-3.45	120	0.573	2.1	0.11	0.03
LLO12	GL	3.00-3.45	115	0.713	2.0	0.16	0.05
LLO17	GL	2.25-2.70	145	0.728	3.3	0.19	0.06
GL1A	GL	1.50-1.95	95	0.692	3.1	0.12	0.03
GL1A	GL	4.05-4.50	200	0.525	2.6	0.16	0.04
GL1A	GL	5.40-8.85	160	0.552	1.6	0.12	0.03
GL1A	GL	7.20-7.65	240	0.532	1.8	0.12	0.02

ID	Unit	Depth (m)	Pre-Consolidation Pressure (kPa)	In-situ Void Ratio	OCR	C_c	C_r
GL2	GL	1.95-2.40	155	0.570	4.0	0.13	0.03
GL2	GL	5.10-5.55	190	0.636	2.0	0.16	0.04
GL2	GL	7.35-7.80	210	0.594	1.5	0.13	0.03
GL2	GL	8.25-8.70	155	0.658	1.0	0.17	0.04
GL3	GL	3.25-3.70	150	0.537	2.4	0.14	0.03
GL4	GL	2.40-2.85	145	0.854	3.1	0.19	0.06

Project wide results of pre-consolidation pressure versus depth and over consolidation ratio versus depth for GL soils are included in Figure 36 and Figure 37.

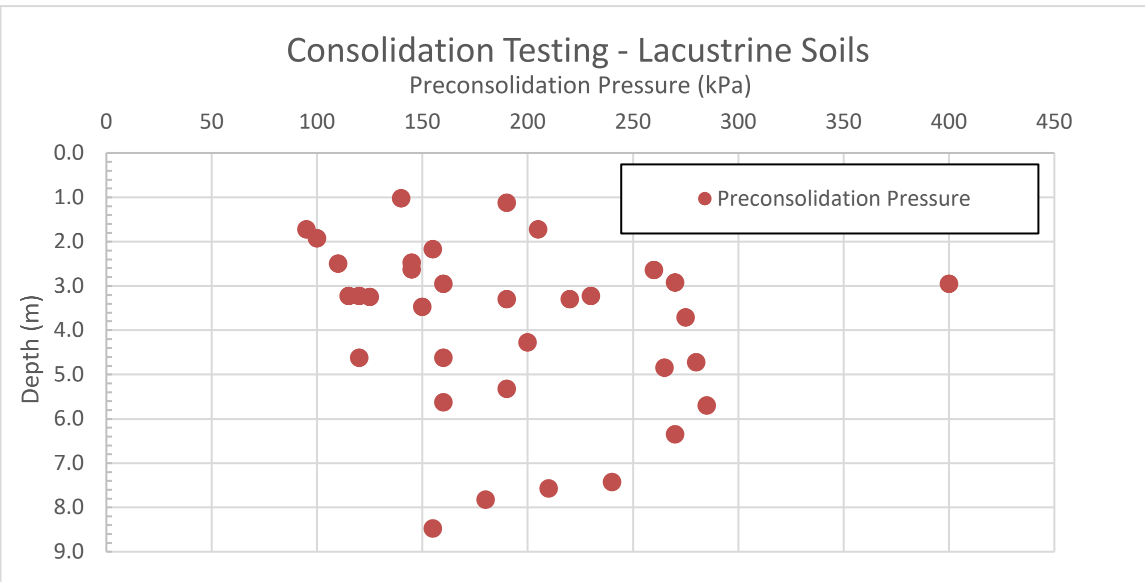


Figure 36. Pre-Consolidation Pressure Versus Depth - Glacial Lacustrine

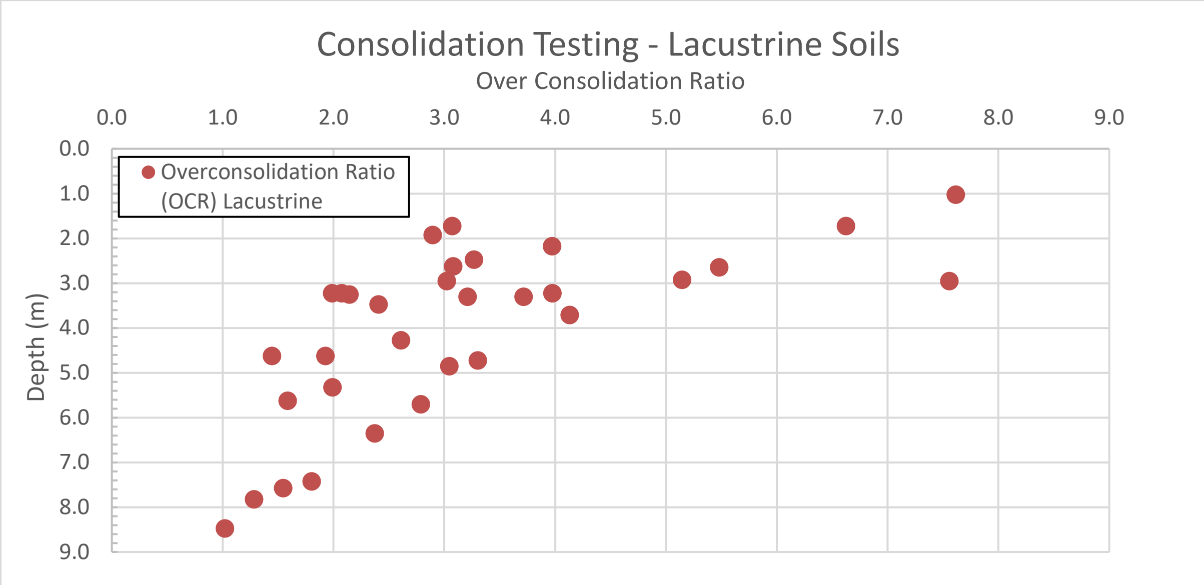


Figure 37. Over Consolidation Ratio with Depth - Glacial Lacustrine

Due to difficulty in obtaining undisturbed Shelby tube samples of glacial tills, only eleven one-dimensional consolidation tests were undertaken on undisturbed GT samples. The data from the consolidation testing is presented in Table 10.

Project wide results of pre-consolidation pressure versus depth and over consolidation ratio versus depth for GT soils are included in Figure 38 and Figure 39.

Table 10. Summary of Consolidation Test Results – Glacial Till

ID	Unit	Depth (m)	Yield Stress (kPa)	In-situ Void Ratio	OCR	C_c	C_r
D8	GT	1.80-2.30	310	0.612	8.4	0.15	0.03
D11	GT	1.60-1.90	245	0.669	7.8	0.18	0.06
D62	GT	4.60-5.05	87	0.489	1.0	0.15	0.02
D68	GT	7.60-8.10	200	0.588	1.4	0.17	0.03
LLO08	GT	4.60-5.05	115	0.576	1.3	0.15	0.04
LLO12	GT	4.60-5.05	120	0.505	1.4	0.09	0.02
LLO12	GT	7.60-8.05	120	0.504	0.9	0.13	0.02
LLO17	GT	4.05-4.50	110	0.467	1.4	0.08	0.02
GL1A	GT	10.90-11.35	205	0.516	1.0	0.08	0.02
GL2	GT	10.5-10.95	135	0.482	0.7	0.08	0.02
GL2	GT	11.55-12.00	90	0.304	0.4	0.08	0.02

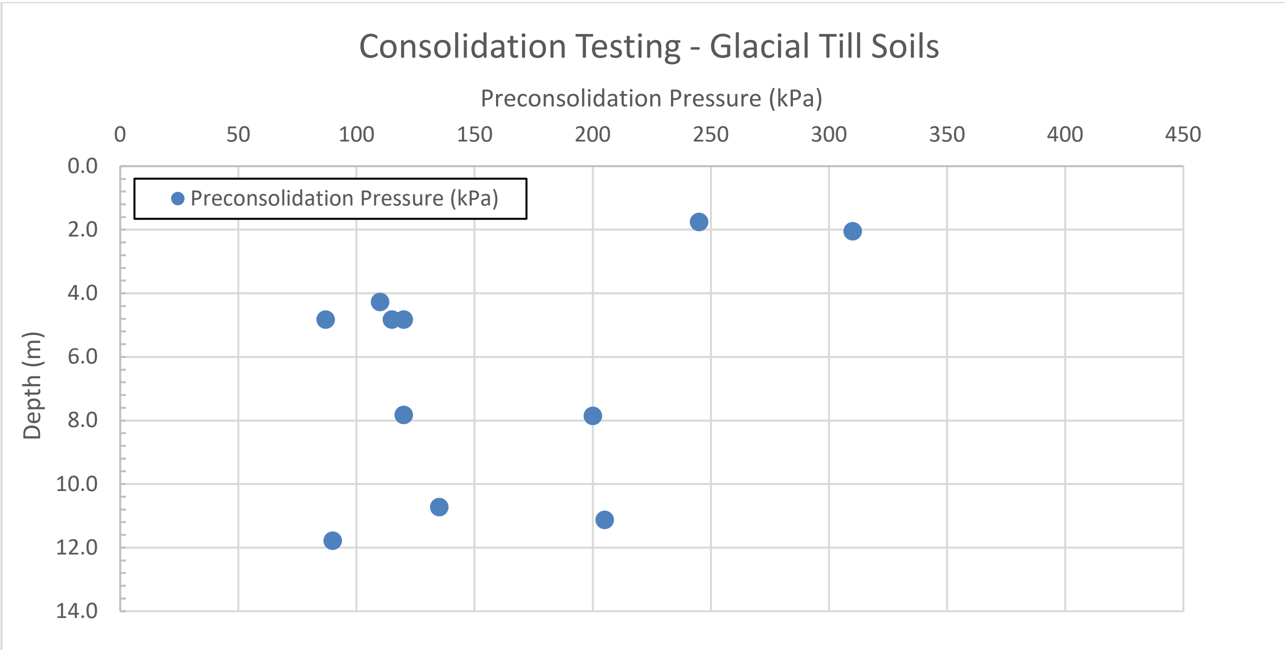


Figure 38. Pre-Consolidation Pressure Versus Depth - Glacial Till

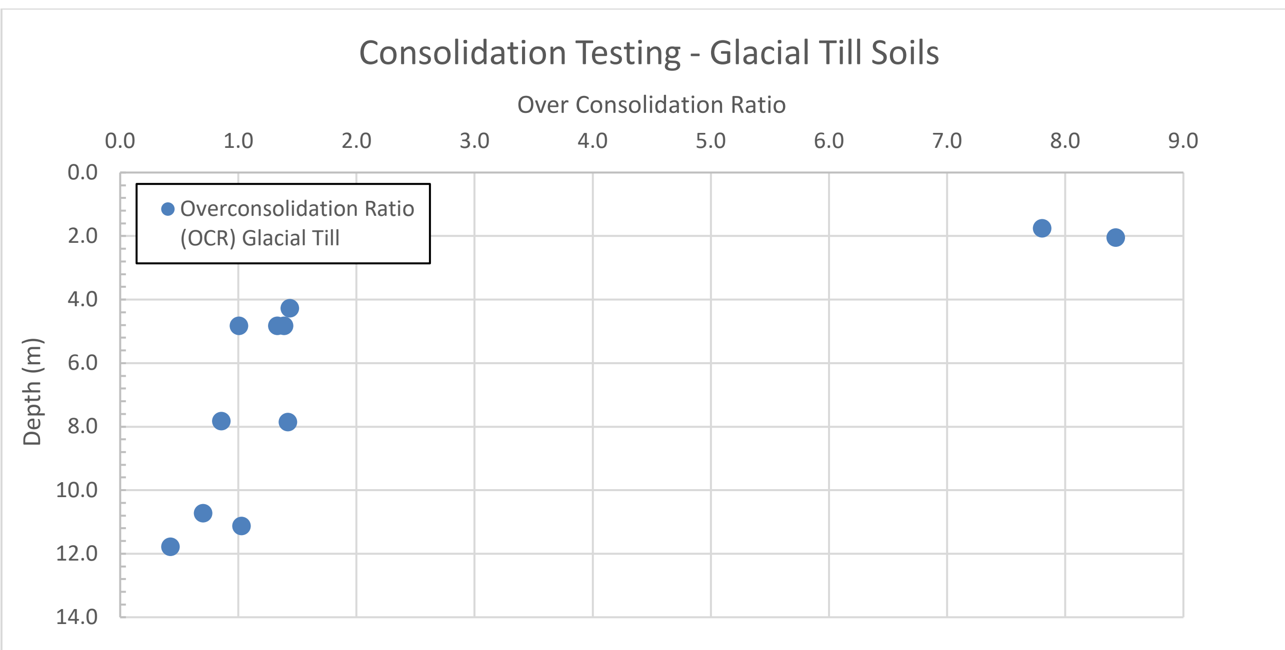


Figure 39. Over Consolidation Ratio Versus Depth - Glacial Till

The LGT, BGT and BGST accumulated in a subglacial depositional setting at the base of a fast-moving Cordillerian ice sheet (or lobe) originating from the Rocky Mountains. The likely mechanisms of accumulation include direct lodgment, subglacial melting, cavity deposition and subglacial deformation. Based on numerical climatic modelling and mapping of glacial erratics in the Porcupine Hills[10] , the Calgary region was overlain by at least 1 km of ice during the LGM (Gowan et al, 2016; Osborn et al, 2000). Walker (1971) suggested that the BVA ice was at least 900 m thick over the center of Bow Valley and occupied the lower 16 km of Kananaskis Valley, in which a dammed lake formed in the vicinity of the modern day Barrier Lake (Evans, 1999).

The UBT and overlying GL accumulated in supraglacial / ice-marginal setting indicative of glacial stagnation and the progressive retreat of the ice-sheet westwards. These units would have been deposited through subglacial and supraglacial melt-out of sediments, debris flows and from lacustrine deposition.

During the Holocene Epoch, para-glacial and periglacial processes would have occurred. The retreat of the ice sheets would have left inherently unstable deposits susceptible to slope instability and erosion by meltwater. Examples include:

- Mechanical over-consolidation of sediments through lowering of the water table due to release of glacial meltwaters and down-cutting of the Elbow River through the existing glacigenic units, and;
- Desiccation of the upper glacigenic units due freeze-thaw and periods of drought within the Holocene Period.

5.4.3.6 Hydraulic Conductivity

The in-situ hydraulic conductivity of the glacigenic units has been estimated from laboratory testing, dissipation testing in the CPT'S and groundwater well level observation testing.

A total of 21 falling head permeability tests (ASTM D5084) were performed on undisturbed GL and GT soil samples. The test results were reviewed and summarized by dam station limits and by soil type, and geometric means of the test results were calculated for each zone. Additionally, eight CPT field dissipation tests were reviewed to evaluate the horizontal hydraulic conductivity of the two predominant soil types.

The results of 14 falling head permeability tests performed on undisturbed GL soil samples are summarized in Table 11. The results of the four CPT field dissipation tests performed in GL are summarized in Table 12.

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[10] Approx.

Table 11. Summary of Permeability Values from Laboratory Testing – Glacial Lacustrine

Zone	Sample Type	Number of Falling Head Permeability Tests	Geometric Mean k_v (m/sec)
7	Undisturbed	1	1.90E-10
8	Undisturbed	1	7.70E-10
10	Undisturbed	3	2.00E-10
11	Undisturbed	6	3.42E-10
12	Undisturbed	2	1.80E-10
Geohydro	Undisturbed	1	3.20E-10
Total Site	Undisturbed	14	3.07E-10

Table 12. Summary of CPT Pore Pressure Dissipation Tests – Glacial Lacustrine

Zone	Number of CPT Pore Pressure Dissipation Tests	Geometric Mean k_h (m/sec)
8	1	1.24E-9
11	3	1.46E-10
Total Site	4	2.49E-10

The range of permeability values obtained from undisturbed GL soil samples from different project zones is presented graphically in Figure 40.

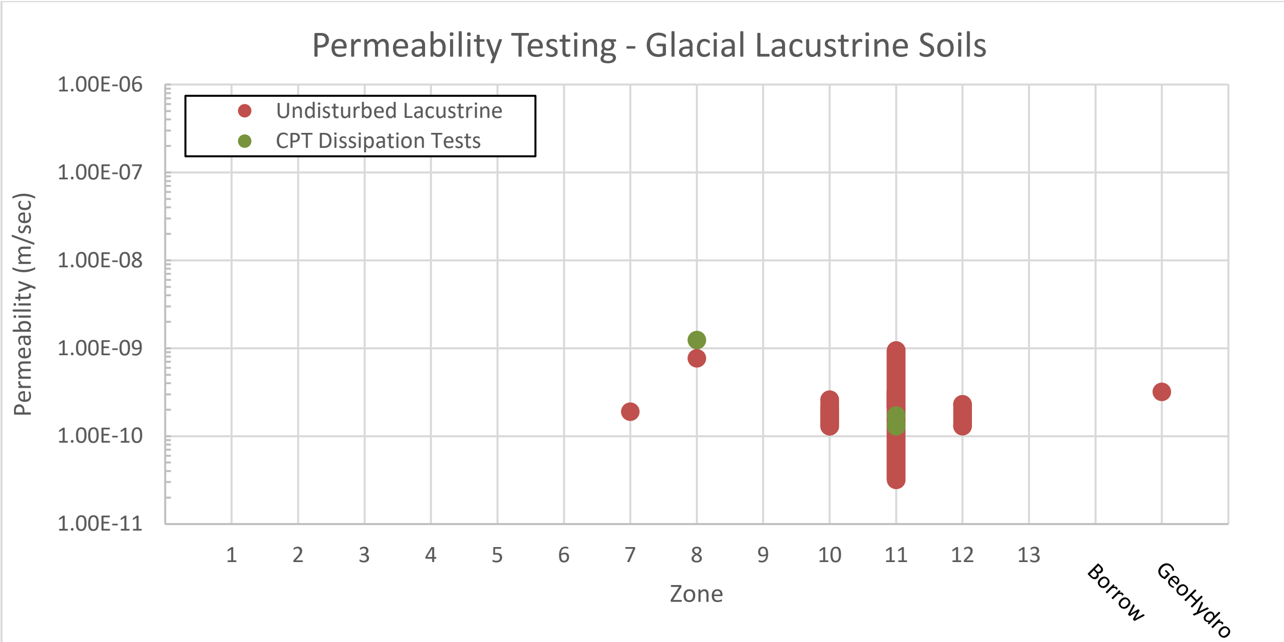


Figure 40. Spatial Distribution of Undisturbed Glacial Lacustrine Permeability Test Results

The results of seven falling head permeability tests performed on undisturbed GT soil samples are summarized in Table 13. The results of the four CPT field dissipation tests performed in GT soil are summarized in Table 14.

Table 13. Summary of Permeability Values from Laboratory Testing – Glacial Till

Zone	Sample Type	Number of Falling Head Permeability Tests	Geometric Mean k_v (m/sec)
3	Undisturbed	3	2.00E-10
7	Undisturbed	1	7.70E-10
9	Undisturbed	1	3.50E-10
10	Undisturbed	1	4.50E-11
12	Undisturbed	1	5.80E-11
Total Site	Undisturbed	7	2.61E-10

Table 14. Summary of CPT Pore Pressure Dissipation Tests – Glacial Till

Zone	Number of CPT Pore Pressure Dissipation Tests	Geometric Mean k_h (m/sec)
8	3	2.59E-8
11	1	3.03E-10
Total Site	4	8.51E-9

The range of permeability values obtained from undisturbed GT soil samples and CPT dissipation field tests from each project zone is presented graphically in Figure 41.

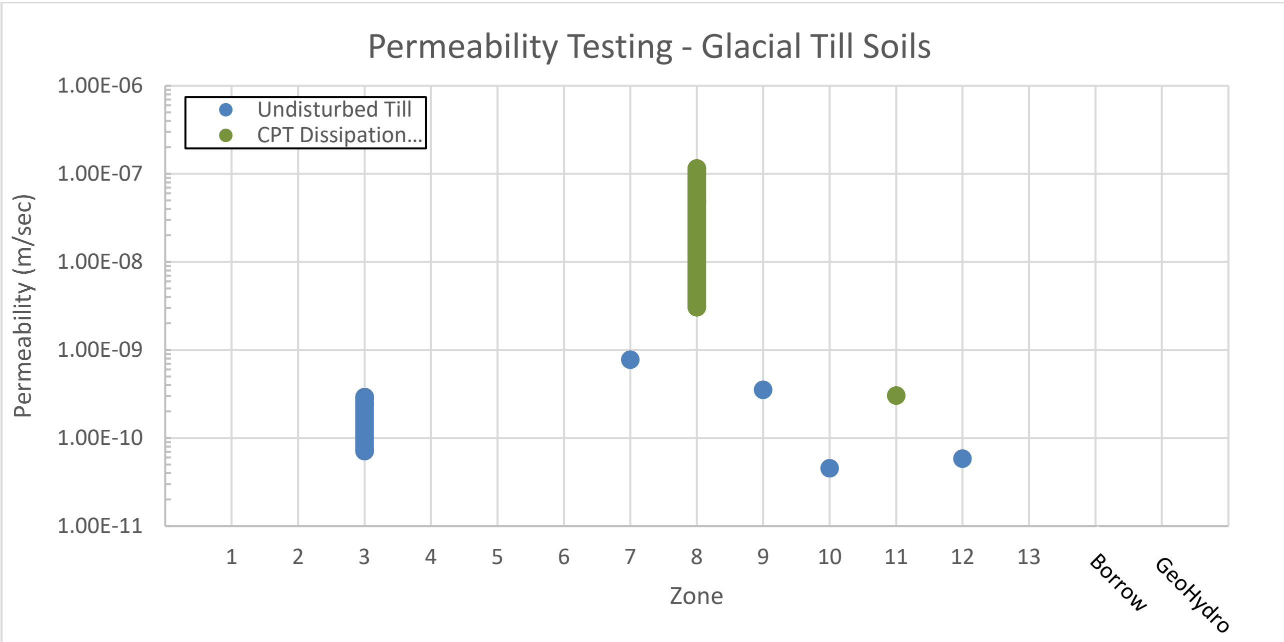


Figure 41. Spatial Distribution of Undisturbed Glacial Till Permeability Test Results

Three (3) slug tests were undertaken on the glacigenic units within the SR1 Project Site to determine the radial, i.e. horizontal hydraulic conductivity of the tested unit. These are summarized in Table 15.

Table 15. Results of Slug Testing

Borehole	Ground Elevation (masl)	Top of Screen (masl)	Base of Screen (masl)	Target Material	K_r Hvorslev Method (m/s)	K_r KGS Method (m/s)	K_r Bouwer-Rice (m/s)
BS3	1197.4	1191.3	1188.3	GT	N/C	2.35E-10	8.18E-10
GW9	1204.5	1200.2	1198.7	GL	5.32E-8	2.18E-7	N/C
GW10	1195.3	1183.1	1180.0	GT	2.52E-7	6.27E-10	N/C

Eight (8) pore pressure dissipation tests were undertaken on the glacigenic units in five (5) boreholes. The horizontal hydraulic conductivity values are summarized in Table 16.

Table 16. CPT Pore Pressure Dissipation Testing Results (GL and GT)

Borehole	Ground Elevation (masl)	Test Depth (m)	Test Elevation (masl)	Target Material	t_{50} (s)	k_h[1] (m/s)
DC26	1203.8	1.1	1202.7	GL	1339.0	1.2E-09
DC26	1203.8	3.7	1200.1	GT	648.0	3.1E-09
DC26	1203.8	9.5	1194.3	GT	70.2	4.9E-08
D26	1192.0	5.5	1186.5	GL	6491.0	1.7E-10
D26	1192.0	11.0	1181.0	GL/GT	4394.0	2.8E-10
D31	1191.0	5.8	1185.3	GL	8145.0	1.3E-10
D39	1191.4	12.1	1179.4	GT	2993.0	4.5E-10
D40	1191.1	4.3	1186.8	GL	7687.0	1.4E-10

The suite of laboratory and in-situ testing indicated that the GL and GT have similar hydraulic conductivities. Because of the depositional history of the GL, an anisotropic ratio of 3:1 was selected. From the horizontal mean of the laboratory falling head tests, a vertical hydraulic conductivity of 3x10-11 m/s was selected. Using the anisotropic ratio of 3:1, a horizontal hydraulic conductivity of 1x10-10 m/s was calculated for design purposes for the GL. The GT was assumed to be isotropic for this study, and a horizontal and vertical hydraulic conductivity of 3x10-10 m/s from the horizontal mean of the falling head laboratory tests was used for the analysis.

5.4.3.7 Drained Shear Strength

A total of 61 consolidated undrained triaxial (ASTM D4767) tests were performed on undisturbed GL and GT soil samples. A summary of the soil types subjected to consolidated undrained (CU) triaxial testing are presented in Table 17.

Table 17. Summary of Consolidated Undrained (CU) Triaxial Tests

Test Type	Soil Type	Sample Type	Completed
CU Triaxial Tests	Glacial Lacustrine	Undisturbed	39
	Glacial Till	Undisturbed	22
	Total		61

The p-q plot of the combined CU test data for 39 undisturbed GL soil samples is presented in Figure 42. The p'-q plot of the test results was used to develop design values. The intercept for c' was normalized to zero. The typical friction angles are based on lines of best fit.

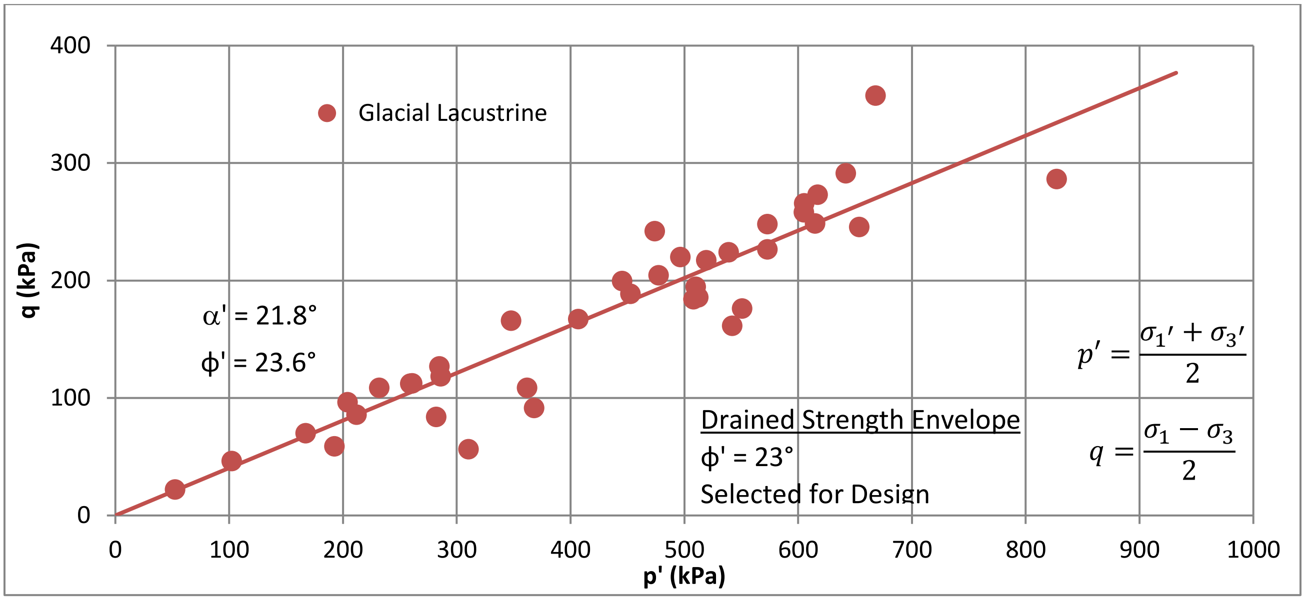


Figure 42. CU Triaxial Test Results - Undisturbed Glacial Lacustrine

CU triaxial tests performed on undisturbed glacial lacustrine specimens were typically performed until 20 percent axial strain was obtained. The CU test results typically produced a peak shear stress followed by a lower residual shear strength. Residual shear strengths either stabilized after 9 to 12 percent axial strain occurred or continued to decrease with additional strain.

The p-q plot of the combined CU test data for 22 undisturbed GL soil samples is presented in Figure 43.

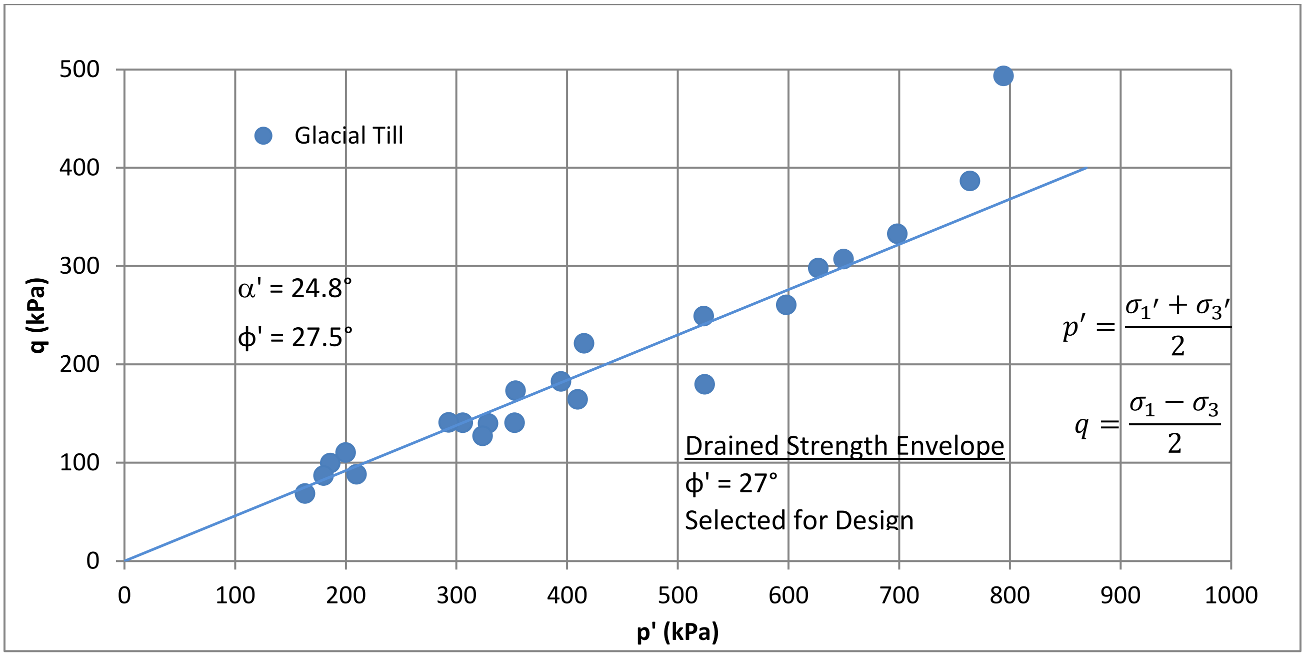


Figure 43. CU Triaxial Test Results - Undisturbed Glacial Till

CU triaxial tests performed on undisturbed glacial till specimens were typically performed until 20 percent axial strain was obtained. The CU test results typically produced a peak shear stress followed by a lower residual shear strength. Residual shear strengths either stabilized after 7 to 15 percent axial strain occurred or began to increase or decrease with additional strain.

Laboratory CU test results were reviewed to determine residual strength of the GT soil. The test results that clearly demonstrated a peak strength followed by a uniform residual strength yielded an average peak shear strength of 27 degrees and an average residual shear strength of 25 degrees after approximately 10 percent strain occurred.

5.4.3.8 Undrained Strength Profiles

The undrained strength (S_U) of the glacigenic units have been estimated using data from 15 CPT's and ten unconsolidated undrained triaxial tests.

Ten (10) unconsolidated undrained triaxial tests were undertaken on GL samples obtained from Shelby tubes at relatively shallow depth. The results of these are summarized below in Figure 44 and suggest that the S_u values range between 75 and 110 kN/m^2. Two tests were undertaken on samples of UBT and resulted in S_u values of 165 and 300 kN/m^2. These results are considered an index as it recognized that the S_u is influenced by sample disturbance, water content of testing, stress path and the test procedure (Fell et al, 2014; Ladd and DeGroot, 2004).

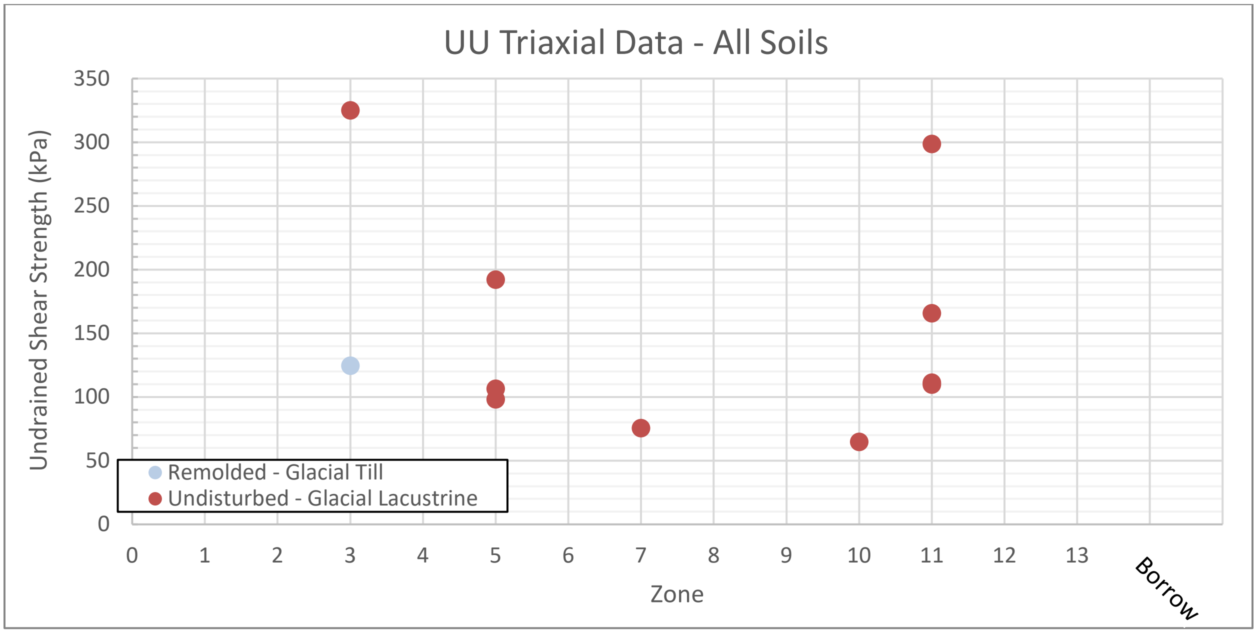


Figure 44. Undrained Shear Strength Results

CPT Data

Undrained strength (S_u) from CPT data was calculated using a N_{kt} value of 15. Kleven (1981) showed that for normally consolidated marine clays, the cone factor N_{kt} varied between 11 and 19 with an average value of 15. Aas et al. (1986) correlated plasticity index to N_{kt}. The results indicate that N_{kt} increases with increasing plasticity, ranging from 8 to 16 for plasticity indexes from 3 to 50%. A large number of studies have been performed resulting in N_{kt} values between 15 and 20 (ESOPT 1974). Based on this information, an N_{kt} value of 15 was chosen for the glacial lacustrine and glacial till soils.

Soil horizon breaks were determined using nearby soil borings where available and SBT values from the CPT data. The data was summarized by soil type for all samples and by geographic zone. Project wide results of S_u versus depth for glacial lacustrine are included in Figure 45.

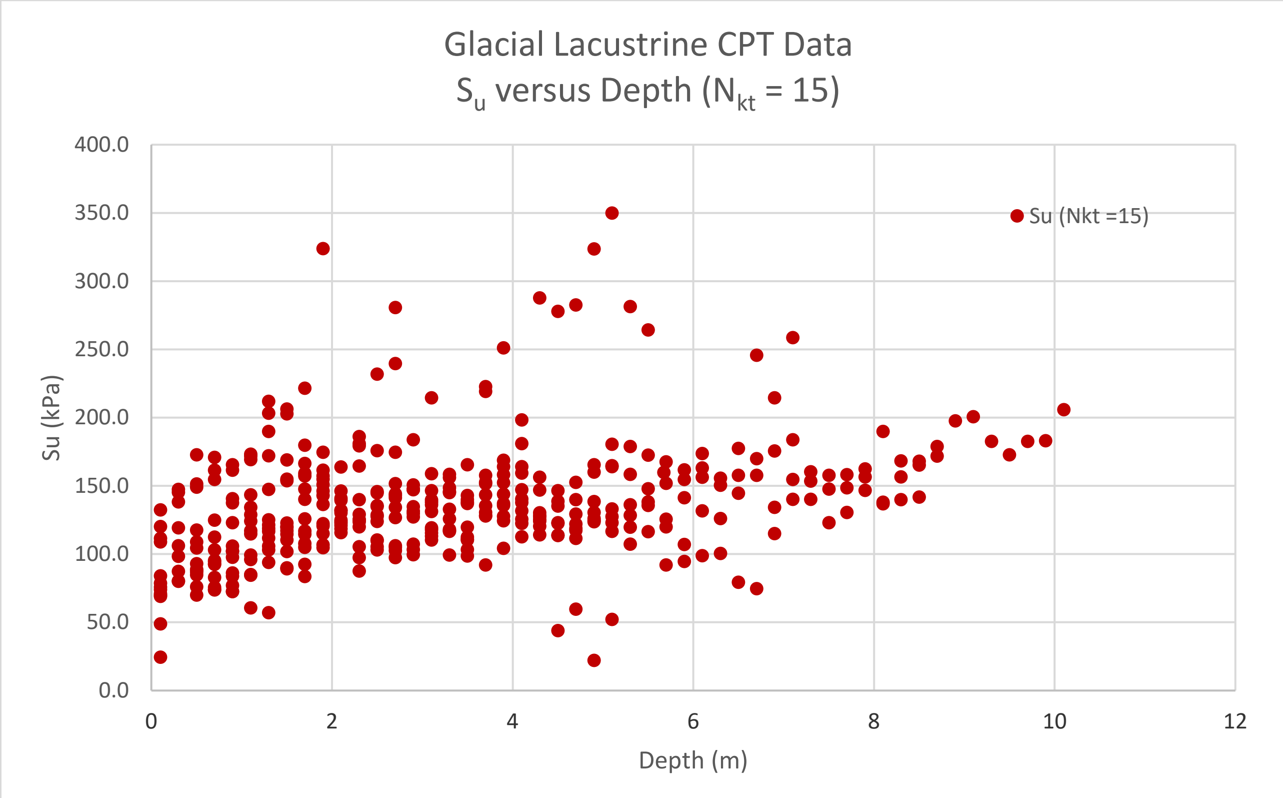


Figure 45. CPT Undrained Shear Strength versus Depth – Glacial Lacustrine – Project Wide

Project wide results of S_u versus depth for glacial till are included in Figure 46.

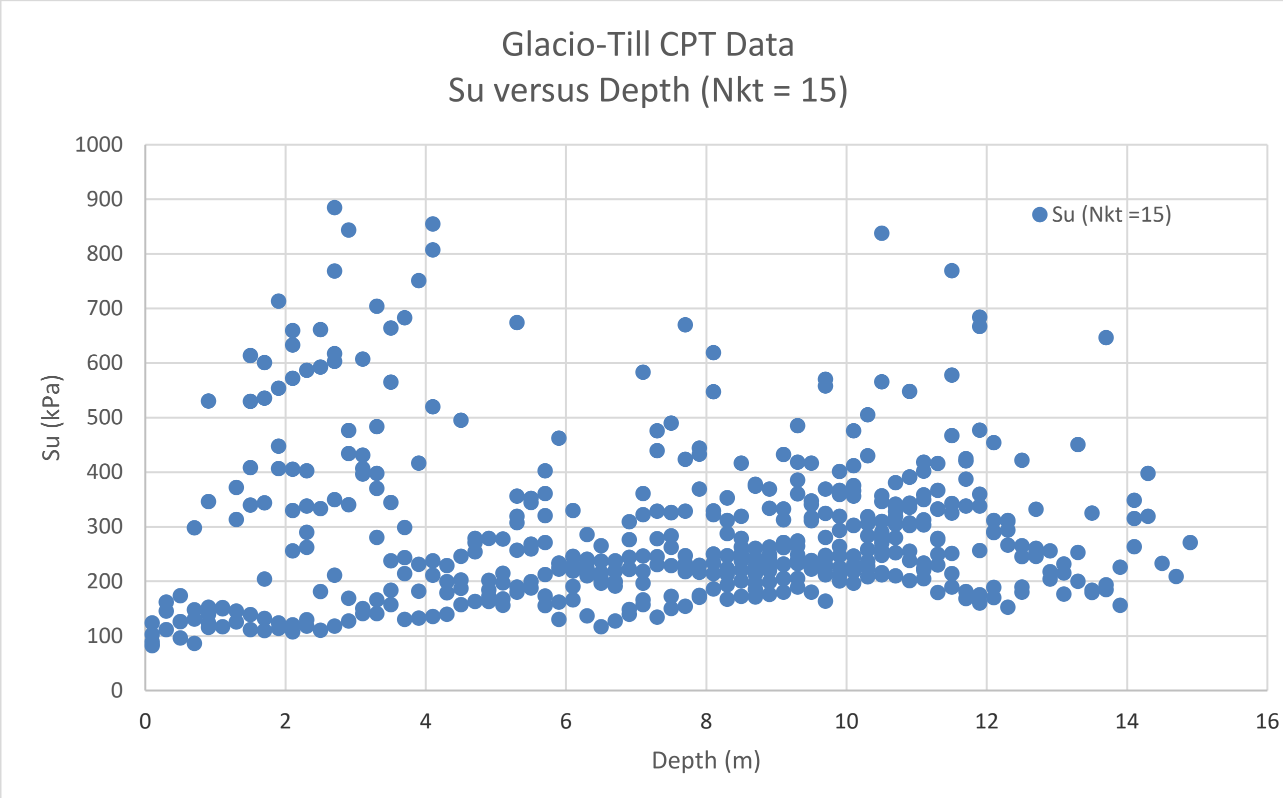


Figure 46. CPT Undrained Shear Strength versus Depth – Glacial Till – Project Wide

Undrained Strength Envelopes from CU Triaxial Tests

Using data from the CU tests, undrained strength envelopes were fit to plots of shear strength on the failure plane $(S_U)_{ff}$ versus specimen consolidation pressure (σ'_{con}) (FERC 2006; USBR 2011). The undrained strength envelope is represented by Mohr-Coulomb parameters (c and φ) as shown in Figure 47.

The undrained shear strength on the failure plane $(S_U)_{ff}$ was calculated for each CU test performed. In a triaxial compression test, failure develops on a plane that is oriented at (45° + ϕ'/2) above horizontal (Duncan et al. 2014). The shear stresses on the failure plane in a particular CU test can be determined using Mohr's circle and the equation shown in Figure 47.

Here, σ_{df} is the measured deviator stress at failure and ϕ'is the drained friction angle.

$$\boxed{(S_u)_{ff} = \frac{\sigma_{df}}{2} \cos\phi' = c + \sigma'_{con} \tan\phi}$$

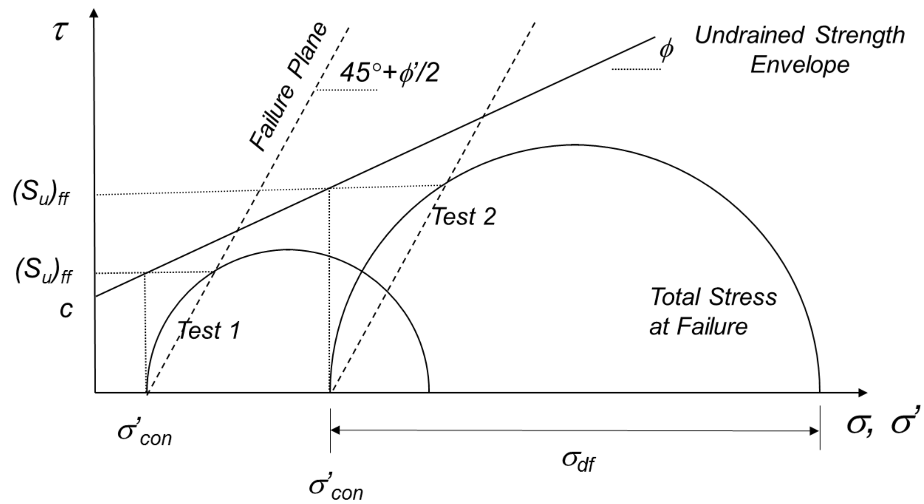


Figure 47. Undrained Strength Envelope Representing the Failure Plane Shear Stresses Measured in CU Triaxial Tests

The results of $(S_u)_{ff}$ versus (σ'_{con}) for 39 CU tests performed on undisturbed Glacial Lacustrine specimens are included in Figure 48. An undrained strength envelope fit through the data results in c = 15 kPa and ϕ = 20˚. Likewise, the results from 22 CU tests performed on undisturbed Glacial Till specimens shown in Figure 49 indicate values of c = 60 kPa and ϕ = 19˚. The higher strength of the undisturbed Glacial Till likely reflects, in part, the higher preconsolidation pressures in this glacial material. The undrained cohesion and friction angle values shown in Figures 48 and 49 were obtained by linear regression.

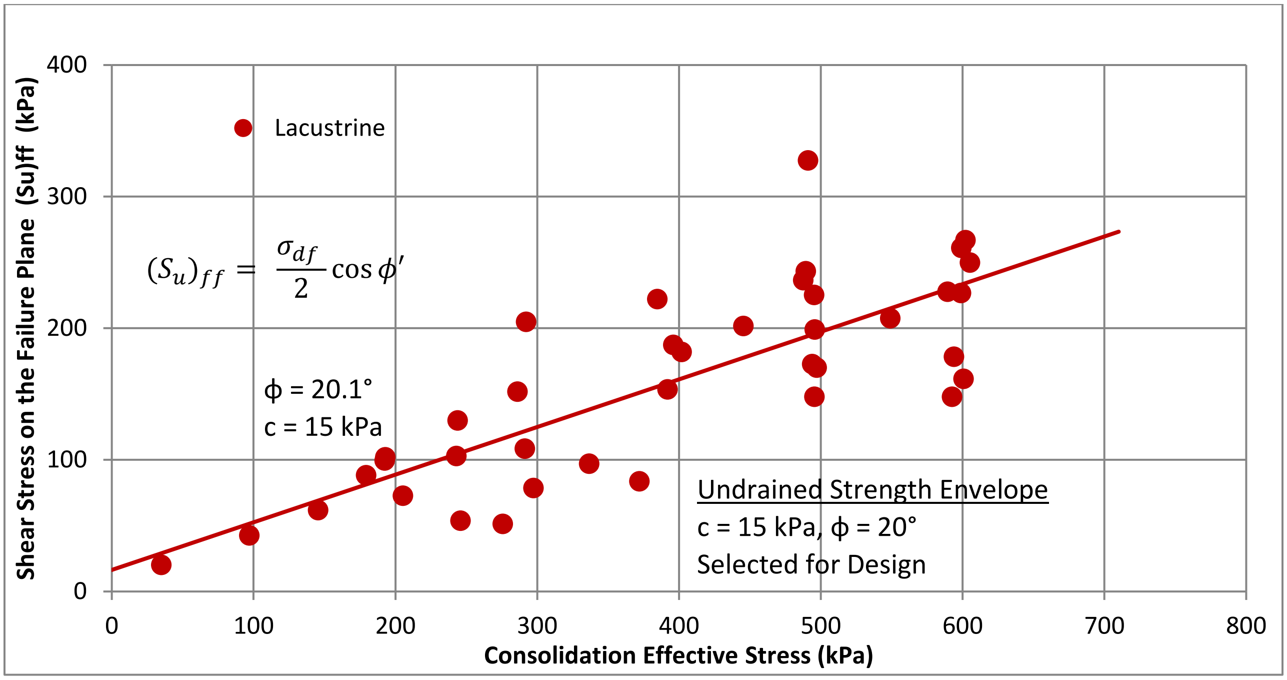


Figure 48. CU Triaxial Test Undrained Strength Envelope – Undisturbed Glacial Lacustrine

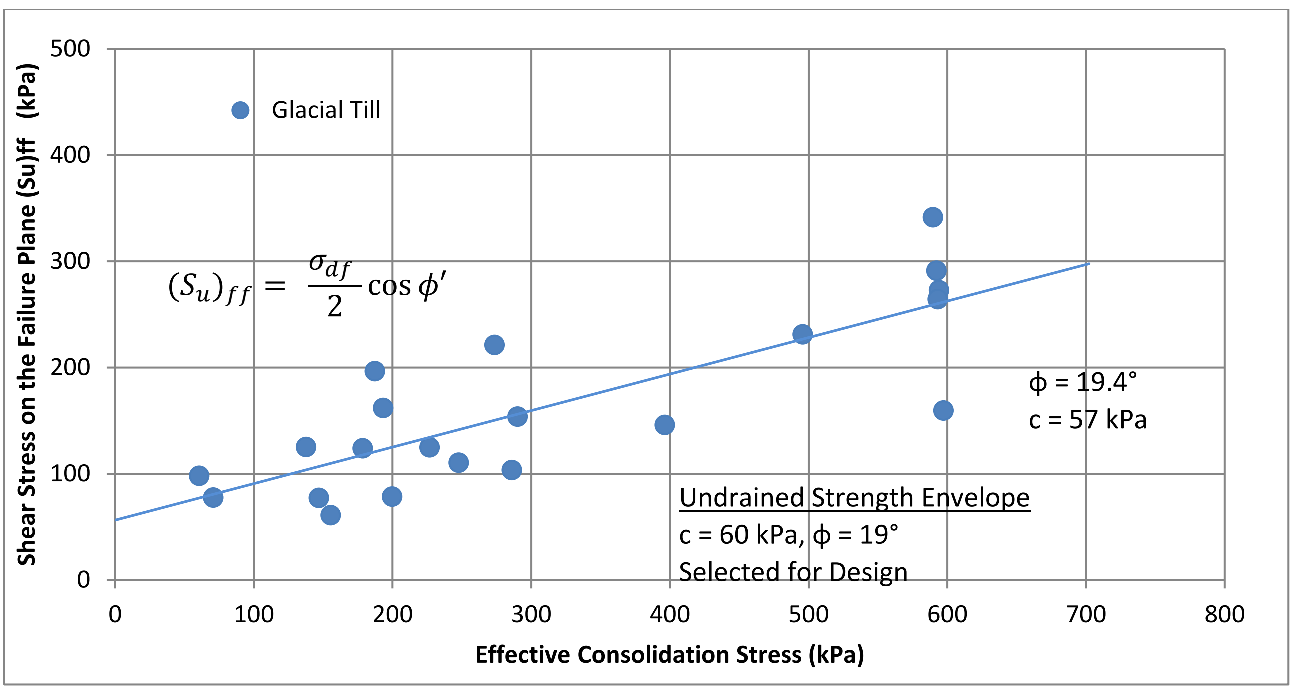


Figure 49. CU Triaxial Test Undrained Strength Envelope – Undisturbed Glacial Till

Direct Simple Shear Testing

General – Direct simple shear (DSS) tests were performed to establish S_u/σ_v' (c/p) ratios appropriate for undrained analyses of the lacustrine clay layer. These tests were performed in accordance with ASTM D 6528-17. The four initial (June 2018 program) tests were performed at a shearing rate of five percent per hour. Subsequent tests were performed at a shearing rate of one percent per hour. The tests were performed by the TetraTech laboratory in Richmond, BC. While the 2016 tube samples were cut and evaluated prior to shipment to Vancouver, the other DSS samples were sent as intact Shelby tube samples.

Normally Consolidated Tests – The conditions within the glacial lacustrine clay layer under the embankment dam will vary with both applied load and pore pressure. To characterize possible behavior, variations in consolidation stress were utilized in the DSS tests to replicate the impact of stress history on the soil. To model the normally consolidated condition, samples were run at relatively high confining stresses. The initial program used stresses slightly higher than the measured preconsolidation pressures. Later OCR 1.0 tests were run at a confining stress of 500 kPa, well above observed preconsolidation pressures within the glacial lacustrine clay soil at the SR1 storage dam site.

OCR = 2.0 Tests – The glacial lacustrine clay typically exhibits some overconsolidation, varying from high OCR values near the surface to 2.0 or less at depths of four metres and greater. To replicate higher OCR conditions, the preconsolidation stress at each requested DSS test location was estimated from adjacent 1-D consolidation data and relative depths. As a companion to each normally consolidated test, a second DSS was performed using a consolidation stress equal to one half of the estimated preconsolidation stress.

Test Results – The main testing program began in late October 2018 and was completed during January 2019. The high consolidation stress tests began right away. The OCR 2.0 testing was initiated when 1-D consolidation tests in adjacent areas had been completed and analyzed to assign DSS test parameters.

As testing progressed, difficulties developed with the low confining stress (OCR 2.0) tests. Due to sample shear stiffness and generally low-end friction, the samples were partially sliding along the interface with the filter stones / platens, rather than yielding with shear deformation. These tests were performed with roughened, grooved porous metallic filter stones, but the friction was not enough to overcome the sample shear stiffness at the low normal pressure. The OCR 2.0 tests were then modified by first consolidating the samples to the full estimated preconsolidation stress and then unloading back to the specified consolidation for shearing. This "seated" the samples into the platen, but some slippage still occurred.

Ultimately replacement platens were utilized. These were made of the same roughened porous metallic material but had raised fins designed to penetrate into the sample ends. The raised fin platens improved the test results, producing higher shear resistance when tested on adjacent samples, but some slippage still occurred as the fins sheared through the base of the sample.

Overall, the DSS tests have not provided a full measure of the undrained shear strength of the Glacial Lacustrine clay. In most tests, the measured response partly represents the slipping resistance of the interface between the soil and the DSS end platens. Despite the slippage, these tests are considered meaningful as they provide a lower-bound of available shear strength. Table 18 below provides the basic results of the DSS testing.

Table 18. Results of Direct Simple Shear Testing of Glacial Lacustrine Clay

Borehole	Sample	Depth (m)	σ_v'* (kPa)	$S_{u5\%}$ (kPa)	S_u/σ_v'	Test OCR**
LLO1	ST4	3.32	120	87	0.725	2.0
LLO1	ST4	3.34	270	85	0.315	1.0
LLO1	ST7	4.92	140	53	0.379	1.9
LLO1	ST7	4.94	300	82	0.273	1.0
D14	ST8	3.18	60	46	0.767	2.1
D14	ST8	3.22	500	140	0.280	1.0
D60	ST2	1.00	500	113	0.226	1.0
GL1A	ST2	1.30	45	49	1.089‡	2.0
GL1A	ST2	1.37	500	127	0.254	1.0
GL1A	ST5	2.69	55	57	1.036‡	2.0
GL1A	ST5	2.72	500	134	0.268	1.0
GL1A	ST8	3.88	66	85	1.288‡	2.0
GL1A	ST8	2.92	500	150	0.300	1.0
GL1A	ST11	5.24	77	98	1.273‡	2.0
GL1A	ST11	5.27	500	137	0.274	1.0
GL1A	ST14	6.67	112	88	0.786‡	2.0
GL1A	ST14	6.70	500	143	0.286	1.0
GL1A	ST17	8.03	132	96	0.727‡	2.0
GL1A	ST17	8.06	500	140	0.280	1.0
GL2	ST3	1.79	73	43	0.589	2.0
GL2	ST3	1.82	500	132	0.264	1.0
GL2	ST6	3.12	73	27	0.369	2.2
GL2	ST6	3.16	500	128	0.256	1.0
GL2	ST10	4.92	90	50	0.556‡	2.0
GL2	ST10	4.97	500	131	0.262	1.0
GL2	ST13	6.29	100	43	0.430	2.0
GL2	ST13	6.32	500	131	0.262	1.0
GL2	ST16	8.09	105	96	0.873‡	2.0
GL2	ST16	8.12	500	135	0.270	1.0
GL2	ST19	9.40	110	81	0.736‡	2.0
GL2	ST19	9.46	500	120	0.240	1.0
GL3A	ST5	3.08	68	52	0.765‡	2.0
GL3A	ST5	3.12	500	122	0.244	1.0
GL4	ST4	2.24	68	53	0.779‡	2.0
GL4	ST4	2.27	500	125	0.250	1.0
LLO17	ST4	2.10	68	31	0.456	2.0
LLO17	ST4	2.14	500	113	0.226	1.0

*Test consolidation stress applied during shearing. **σ_p' established from nearby 1-D consolidation testing. ‡Tests performed with raised rib platens.

OCR 1.0 tests: Count 19, Mean S_u/σ_v' = 0.265, Standard Deviation S_u/σ_v' = 0.0216
OCR 2.0 tests: Count 18, Mean S_u/σ_v' = 0.757, Standard Deviation S_u/σ_v' = 0.2701

The undrained strength ratio (S_u/σ_v') can be used to model the shear strength of a soil as a function of the effective vertical stress. The S_u/σ_v' value is dependent on the consolidation state of the soil where a constant value can be used for normally consolidated soils, and the ratio increases as OCR increases. A correlation between S_u/σ_v' and OCR was suggested by Ladd (1992) and is shown in **Equation 1**:

$$\frac{S_u}{\sigma_v'} = S * OCR^{0.8}$$ **Equation 1**

where:

S_u = Shear strength (assumes ϕ = 0)

σ_v' = Vertical effective pressure

S = Normally consolidated shear strength ratio

OCR = Overconsolidation Ratio

0.8 = empirical exponent

The results of the direct simple shear tests conducted on the undisturbed glacial lacustrine soil samples indicate an average S_u/σ_v' of 0.265 for normally consolidated soil samples, and an average S_u/σ_v' of 0.757 for soil samples with an OCR of 2. Refer to Figure 50 for a plot of all DSS results. Design strength ratios were selected as 0.265 and 0.757for OCR of 1.0 and 2.0, respectively.

A relationship is needed to calculate S_u/σ_v' for any OCR. This was accomplished by using the above values (selected from the DSS test results) and solving for the exponent in **Equation 1**. The resultant exponent was calculated as 1.5, somewhat higher the empirical value suggested by Ladd. The relationship used to calculate S_u/σ_v' for the stability analyses for the glacial lacustrine foundation soils is then:

$$\frac{S_u}{\sigma_v'} = 0.265 * OCR^{1.5}$$ **Equation 2**

This equation is depicted on the results of the direct simple shear tests shown in Figure 50 below. The selected equation results in 54% of the DSS test results above the S_u/σ_v' envelope.

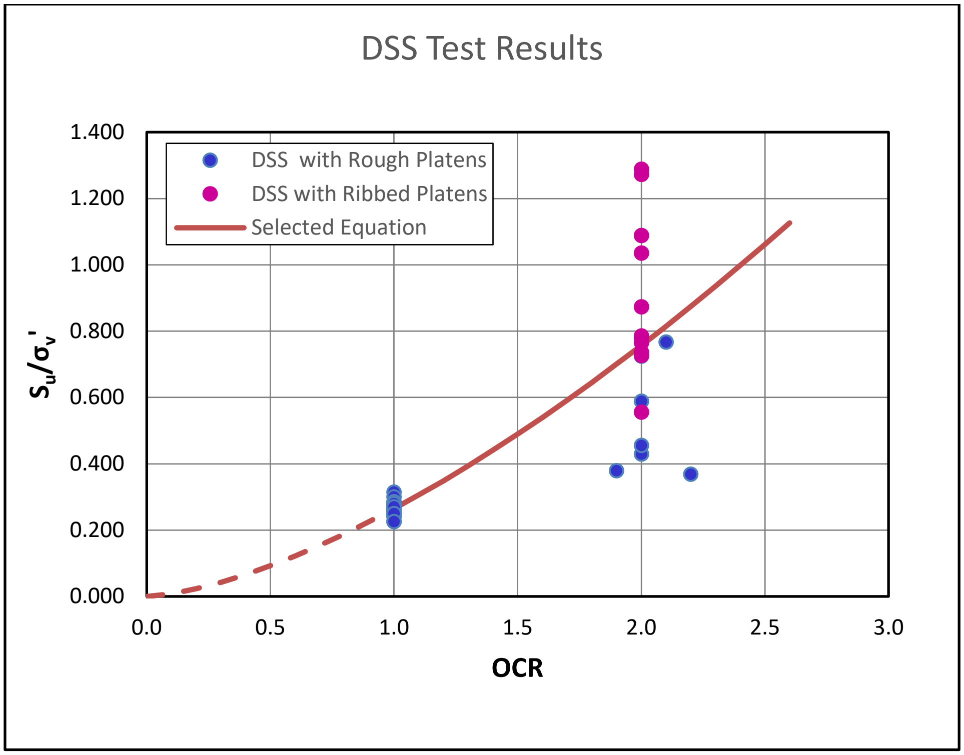


Figure 50. S_u/σ_v' versus OCR for the Glacial Lacustrine Foundation

5.4.3.9 Selected Design Parameters

Unit weights for in-situ soils were selected based on average results of laboratory testing and typical values from published sources. For the glacial lacustrine and glacial till soils, average laboratory unit weight test results were used to select the in-situ unit weight of 18 kN/m³ for both soils.

Table 19. Soil Density and Strength Parameters

Material Name	Unit Weight (kN/m³)
Embankment Shell	20
Embankment Core	20
Sand Drain	21
Glacial Lacustrine	18
Glacial Till	18
Gravel	22
Weathered Bedrock	21

The moisture content, specific gravity and unit weights from laboratory testing for the glacigenic units are summarized in Table 20.

Table 20. Phase Relationships for Glacigenic Units

Material Name	G_s	Moist w (%)	Sat w (%)	e	γ_d (kPa)	γ_m (kPa)	γ_{sat} (kPa)
Glacio-Lacustrine (GL)	2.70	24.0	30.6	0.83	14.5	18.0	18.9
Glacial Till (GT)	2.69	15.9	26.0	0.70	15.5	18.0	19.6

The laboratory falling head test results and the CPT data were used in selecting permeability values for seepage analyses. The permeability parameters selected for the analyses are presented in Table 21.

Table 21. Saturated Hydraulic Conductivity Parameters

Material Name	k_v (m/s)	k_h (m/s)	k_h/k_v
Embankment Shell	3.00E-10	3.00E-10	1.00
Core	3.00E-11	1.00E-10	3.33
Drain	1.00E-05	1.00E-05	1.00
Glacial Lacustrine	3.00E-11	1.00E-10	3.33
Glacial Till	3.00E-10	3.00E-10	1.00
Gravel	1.00E-06	1.00E-06	1.00
Weathered Bedrock	--	--	--

The drained strength parameters selected for the analyses are presented in Table 22.

Table 22. Selected Drained Soil Strength Parameters

Material Name	Drained Strength		Undrained Strength	
	Friction Angle (degrees)	Cohesion (kPa)	Friction Angle (degrees)	Cohesion (kPa)
Foundation Glacial Lacustrine	23	0	20	15
Foundation Glacial Lacustrine (alt. method)			$S_u/\sigma_v' = 0.265 * OCR^{1.5}$	
Foundation Glacial Till	27	0	19	60
Sand Drain	33	0	N/A[1]	N/A[1]
Alluvial Gravel	35	0	N/A[1]	N/A[1]
Weathered Rock	35	0	N/A[1]	N/A[1]

[1]Undrained strengths are equal to drained strengths

5.5 PASKAPOO FORMATION

5.5.1 Extents

The PPF was encountered beneath the dam footprint and the northeast abutment. The boundary between the CSP and PPF could not be identified but has been inferred to be located on the west abutment.

The top of the PPF bedrock varied across the dam footprint and is presented in Figure 20-14. The isopaches show that the top of bedrock typically reduces in elevation from west to east and towards the Elbow River**.**

5.5.2 Description

The PPF comprised an interbedded sequence of weathered clay, mudstones, sandstones and siltstones. The extent and distribution of individual lithology's was difficult to map using borehole data only. However, the rock mass of the PPF could be observed in OC13, which was exposed on the north bank of the Elbow River. This outcrop was approximately 195 m south of the downstream toe. OC13 indicated that the PPF comprised a cyclic sequence of thinly bedded, mudstones and sandstones. The bedding planes were dipping 10-15° NE (Figure 51). The individual beds could be observed laterally over 50 to 100 m before disappearing from view beneath the Elbow River. The sandstone units were 0.5 m thick beds of cream-colored sandstone with vertical jointing. The mudstones units were of similar thickness and were encountered as weathered to a fissile, light grey to olive brown, highly fractured mudstone

Sandstone units were predominant within the western portion of the dam footprint, whilst mudstones and claystones units were predominant in the eastern portion of the dam footprint.

Figure 51. Paskapoo Formation Observed in OC13

5.5.3 Identification of Weak Layers

Weak, slickensided clay-shale layers are frequently encountered in the WSCB. These are formed by tectonic processes, valley rebound and glaciotectonic deformation (Morgenstern, 1988). These layers have been found to influence the investigations, design, construction and performance of dams across the Rocky Mountain and Prairie Regions of Canada and the USA, including the Gardiner Dam in Saskatchewan (Jaspar, 1979), Syncrude's Mildred Lake Settling Basin (Nicol, 1994; Morgenstern, 1988); Oldman River Dam (Divachi et al, 1991), Paddle River Dam (Thiessen and Ramada, 1986; Thurber, 1989); Jackpine Mine External Tailings Facility (Bayliss et al, 2014), Nipawin Dam (Rivard, 2014), St Marys Dam (KCB, 2013) and Alameda Dam (Quinn et al, 2014).

The majority of the case studies were associated with Cretaceous high-plasticity marine clay units such as the Bearspaw and Clearwater Formations. PPF was formed in a fluvial coastal plain environment; however, mudstones are typically predominant and AGS and NRCAN geological mapping has indicated that bentonite layers exist in the PPF. The SR1 project site has also been subjected to glacial forces, tectonic forces due to the Laramide Orogeny, up to 2 km removal of overburden after the Laramide Orogeny and more recently, the down-cutting of the Elbow River and subsequent valley rebound.

At the Oldman Dam in Southern Alberta, weak mudstone layers were encountered in the Porcupine Hills Formation (local variation of the Paskapoo). Divachi et al (1991) indicated the following characteristics:

- Occurred along contacts between relatively strong and weak units, such as sandstones;
- Exhibit parallel bedding;
- Occur as single shear plans, groups of closely spaced sub-parallel shear-planes or as brecciated zones;
- Single shear planes are usually less than 2 mm thick and occasionally up to 10 mm thick. Groups can be up to 75 mm thick;
- Thin shear plans often contain fillings of silt and clay gouge;
- Commonly associated with thin, dark, carbonaceous claystone marker beds;
- Continuous for distances of 100's m, sometimes up to 1km;
- Curved shear plans or splays can occasionally develop off the main bedding plane shear. These splayed planes can form a braided network of shears within a claystone bed; and,
- Frequency is greatest in the upper 10 to 15 m of bedrock, possibly attributed to weathering.

Rotary drilling and geological mapping was undertaken to determine the presence of weak mudstone layers within the PPF. The following observations were made as part of this preliminary geotechnical assessment:

- Visual descriptions of recovered rock cores indicated that there is extensive weak, mudstone/claystone lithological unit beneath the dam footprint, particularly in the eastern portion;
- Laterally extensive claystone/mudstone layers were observed remotely in OC13. However, these could not be accessed and mapped in detail due to the Elbow River;
- Slickensides were occasionally encountered in the mudstone units. These were recorded in D52 at Elevation 1188.3 to 1186.4 m;
- The UCS tests indicated that the mudstone / claystone units had a compressive strength between 0.7 and 2 MPa;
- There was considerable scatter in the results of 4 direct shear tests performed on mudstone samples. Residual strengths are discussed in Section 5.5.4;
- Index testing on selected clay/mudstone layers indicated that the LL typically ranged between 35 and 44. However, one (1) test in D60 indicated that a high plasticity clay layer with a LL of 79 percent was present at 30.5 m below OG at an elevation of Elevation 1161. 5 m;

5.5.4 Residual Strength

It is common practice in Alberta and Saskatchewan to assume that only the residual effective strength can be mobilized for the stability of an earthfill dam unless geotechnical investigation proves otherwise (Morgenstern, 1988; Rivard, 2014). Given the presence of relatively weak clay and mudstone layers; the presence of glacial lineation's indicative of fast moving ice-streams and likely stress-relief due to post-Cretaceous removal of overburden, glacial erosion and isostatic rebound, this Preliminary Geotechnical Assessment has assumed that the residual strength is mobilized.

The $Ø'_R$ of the weak mudstone layers has been estimated using direct shear testing; comparison with other projects on similar foundations; and empirical relationships.

5.5.4.1 Direct Shear Tests

Four (4) direct shear tests could only be completed during this investigation. The results showed considerable scatter, which could be due to incomplete shearing during the laboratory and lithological heterogeneity. Individual $Ø'_R$ values were 4.1°, 18.4°, 24.6° and 36.8°.

In the absence of a significant site-specific trends, published direct shear test data from downtown Calgary (Ye-Loo et al, 2009) and from the Dickson Dam Geotechnical Investigation (UMA, 1979) were plotted for comparison on Figure 52. The average residual strength values assuming a zero intercept are:

- SR1 Geotechnical Investigation: $Ø'_R$ = 21°;
- Ye-Loo et al (2009): $Ø'_R$ = 15°;
- Tests undertaken by Golders Associates at the same downtown site as Ye-Loo et al (2009): $Ø'_R$ = 16°; and,
- Dickson Dam: $Ø'_R$ = 24°.

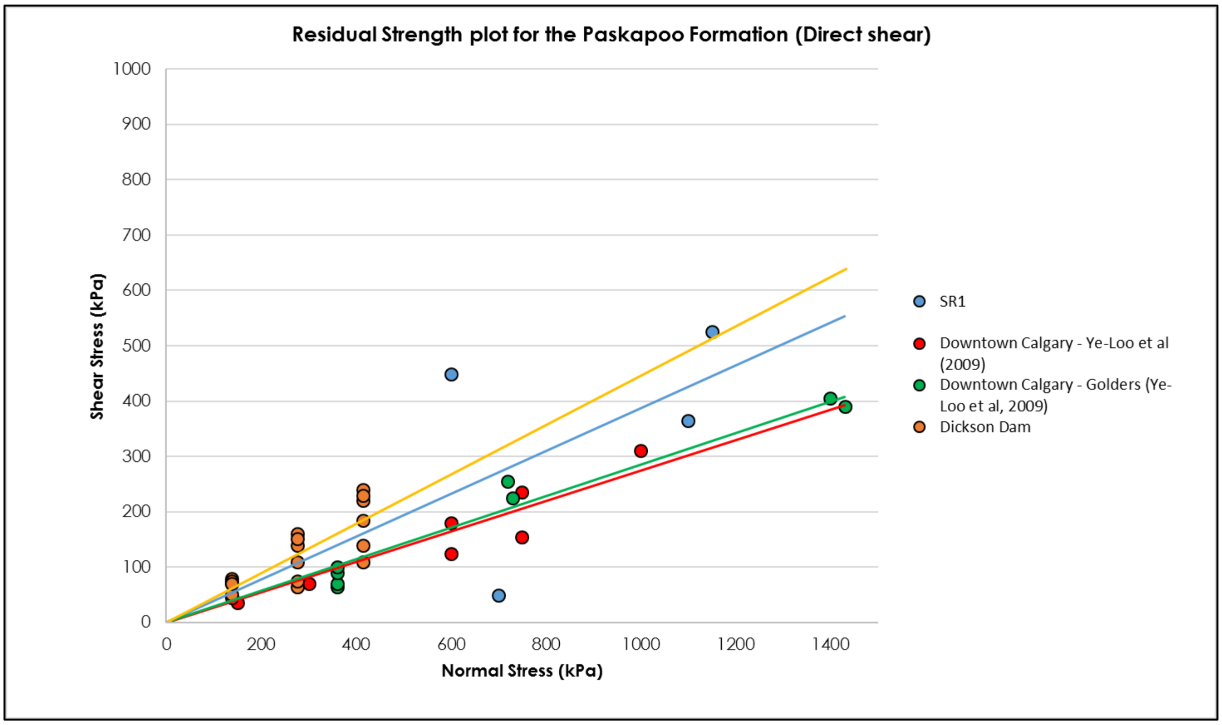


Figure 52. Residual Strength Plot for the Paskapoo Formation

5.5.4.2 Comparison with other projects on similar foundations

Six (6) extreme consequence dams have been constructed on the PPF to date. This includes the Glenmore Dam in Calgary (operated by the COC), the Ghost Dam, Barrier Dam and Brazeau Hydroelectric Dams (operated by TransAlta) and the Dickson and Oldman Dams (operated by the AEP). Examples of design parameters used for the PPF from these dams are summarized in Table 23. No data has been obtained from TransAlta for the Barrier, Brazeau and Ghost hydroelectric facilities.

Table 23. Design Parameters for Existing Dams Constructed on the Paskapoo Formation

Dam	Unit description	Phi (°)	Cohesion (kPa)	Source
Glenmore Dam	Bedrock – horizontal bedding planes	25	0	KCB (2014)
	Concrete rock interface	25	0	
	Cross bedding planes	50	0	
Dickson Dam	Claystone foundation	15 - 11.5	0	UMA (1979)
Oldman Dam	Claystone / mudstone	35	0	UMA (1989); Divachi et al (1991)
	Sandstone	25	0	
	Seam 1	17	0	
	Seam 2	12.5	0	
Oldman Dam Spillway	Mudstone	35	0	
	Sandstone	35	0	
	Seam R1	10.5	0	

Based on the above results, the $Ø'_R$ values adopted for existing designs range between 10.5° and 17°.

5.5.4.3 Empirical Relationships

Using the empirical relationships discussed previously in Section 5.5.4.2, the residual strength was estimated using a LL = 44 and 79 percent. The range of residual strength values are summarized in Table 24.

Table 24. Effective Strength Parameters for the Paskapoo Formation Based on Empirical Relationships

Liquid Limit	Stark and Hussain (2013)		Rivard (2014)
	Clay Content (%)	$Ø_R$ (400-700 kN/m^3)	
44 %	> 50	15-19°	10 - 23°
79 %		8 – 11°	5 - 8°

5.5.5 In-situ Permeability

The results of packer testing and groundwater testing in PPF units are summarized in Table 25 and Table 26 respectively. These indicate the in-situ hydraulic conductivity ranged between 6.5 x 10^{-5} and 6.1 x 10^{-8} m/s. This is comparable to a slightly permeable (widely to very widely spaced discontinuities) rock mass (Bell, 1992).

Table 25. Results of Packer Testing Based on Bedrock Type

Bedrock Type	Number of Tests	Minimum Permeability (m/s)	Maximum Permeability (m/s)	Average Permeability (m/s)
Mudstone	5	2.2E-7	4.3E-5	8.8E-6
Claystone	2	1.6E-7	2.3E-6	1.2E-6
Siltstone	5	4.2E-6	1.1E-7	1.6E-6
Sandstone	4	3.3E-7	2.8E-5	9.9E-6
Mixed lithology	20	6.1E-8	6.5E-5	5.4E-6

Table 26. Results of Slug Tests in Bedrock

ID	Ground Elevation (El. m)	Top of Screen (El. m)	Base of Screen (El. m)	K_r Hvorslev Method (m/s)	K_r KGS Method (m/s)
D51	1194.4	1165.4	1163.9	1.46E-5	N/C
GW1	1211.7	1199.5	1196.5	1.16E-6	2.33E-6
GW4	1204.3	1185.7	1182.6	8.77E-7	1.93E-6
GW6	1196.5	1177.6	1174.5	2.83E-9	3.84E-9
GW8	1216.7	1200.3	1198.1	6.25E-7	2.19E-6

5.5.6 Selected Design Parameters

Based on the above results, a $Ø'_R$ = 17.5° was adopted for the mudstone units in the PPF. This was primarily based on the lower bound direct shear tests from Ye-Loo et al (2009) with an adjustment for the field effects (+1° is typically used based on oil sands dam experience by Cameron, 2013). This value was supported by the relationship proposed by Stark and Hussain (2013), whilst the values published by Rivard (2014) exhibited considerable scatter and could not be meaningfully applied.

5.6 COALSPUR FORMATION

5.6.1 Extents

The CSF was encountered in the eastern portion of the Diversion Structure, beneath the Emergency Spillway and on the western abutment of the dam. The longitudinal bedrock profile of the CSF within the diversion channel is presented in Figure 22-8a, b and c..

5.6.2 Rock Mass Characteristics

The rock cores indicate that the CSF comprises an interbedded sequence of sandstones and mudstones. There are limited outcrops compared to the BZF but the lower part of the formation was observed in OC6 and 7 adjacent to the Highway 22x. The CSF was encountered as a gently dipping sequence of thin to medium bedded, sandstones interbedded with thin beds of mudstone. The beds dipped approx. 25 to 35° NE (Figure 53).

The Entrance Conglomerate, which is the boundary between the BZF and CSF (Jerzykiewicz, 1997) was identified in OC6 (Figure 54).

Figure 53. Sandstone beds within the Coalspur Formation at OC6.

Figure 54. Entrance Conglomerate observed in OC6

5.6.3 Strength Testing

No strength testing was undertaken on the CSF during this stage of the investigation. Testing will be undertaken for the emergency spillway in the next phase of investigation.

5.6.4 Hydraulic Conductivity

No testing was undertaken on the CSF during this stage of the investigation. Testing will be undertaken for the emergency spillway in the next phase of investigation.

5.6.5 Selected Design Parameters

Design parameters from the Paskapoo Formation in Section 5.5.6 will also be applied to the Coalspur Formation.

5.7 BRAZEAU FORMATION

5.7.1 Extents

The BZF was encountered beneath the floodplain berm, diversion structure and the diversion channel. The longitudinal bedrock profile of the BZF within the diversion channel is presented in Figures 21-8a, b and c. At the diversion structure and floodplain berm location, the elevation of the top of the BZF is a function of the glacial and fluvial erosional processes. The top of BZF is encountered between Elevation 1219.2 and 1221 m in the northern slopes of the river valley. Whereas, in the Elbow River Valley and active river bed, fluvial erosion has down-cut the BZF by approx. 10 to 12 m with the top of BZF ranging between Elevation 1207.5 m and Elevation 1210.9 m.

In the diversion channel, the top of the BZF reduces in elevation between Station 10+400 m (Elevation 1228.3 m) and 11+750 m (Elevation 1195.9 m). Between Station 11+750 and 13+200 m, the top of the BZF becomes more undulating and ranges between Elevation 1212.2 m and Elevation 1205.4 m.

5.7.2 Rock Mass Characteristics

The rock cores indicate that the BZF comprises an interbedded sequence of weathered clays, mudstones, clay-shale, siltstones and sandstones. Geological mapping of the outcrops along the Elbow River and Highway 22x have provided insight in the lithological distribution, rock mass characteristics and regional geological structure.

5.7.2.1 Outcrop 1: 5655614 N 676593 E

This is an approx. 120 m long outcrop along the northwest bank of the Elbow River; upstream of the Diversion Structure Inlet (Figure 55). The BZF was encountered as a sub-vertically bedded sequence of non-durable shales, carbonaceous shales, mudstones and siltstones interbedded with 1 to 2 m thick durable, fine grained sandstone beds (Figure 55). The bedding is typically sub-vertical (75 to 85°) with a dip-direction to the ENE (Figure 56).

The sandstone beds exhibited three joint sets: J1 was perpendicular to the bedding plane (B1), whilst J2 and J3 were parallel to B1. The shales exhibited a fissile, slightly folded fabric. The mudstones were highly fractured.

Figure 55. Outcrop (OC1) of Brazeau Formation upstream of the Diversion Structure

Figure 56. Sub-vertical bedding within the Brazeau Formation at OC1

5.7.2.2 Outcrop 2: 5655763 N 676706 E

This is an approx. 180 m outcrop along the northwest bank of the Elbow River at the location of the Diversion Structure Inlet (Figure 57). Due to the orientation of the bedding planes and river channel, the 'top' of the BZF beds could be observed in the slope.

Figure 57. Outcrop of Brazeau Formation at the Diversion Structure at OC2

5.7.2.3 Outcrop 3: 565595 N 676823 E

This is an approx. 200 m outcrop along the northwest bank of the Elbow River and is downstream of the Diversion Structure Inlet. The BZF was encountered as interbedded dark grey shales and pale grey to cream, fine-grained sandstones and siltstones. The rock mass contained small-scale chevron-folding, anticlinal folding and shear zones perpendicular to the bedding plane orientation (Figure 58).

Figure 58. Folded rock mass encountered in the Brazeau Formation at OC3

5.7.2.4 Outcrop 4 (5656148 N 677015 E)

This is an approx. 50 m outcrops along the northwest bank of the Elbow River. The BZF comprises gently dipping, medium to thickly bedded, pale grey to cream, fine sandstone. The dip of the bedding planes was typically between 20 and 45° SW.

Figure 59. Sandstone beds in the Brazeau Formation at OC4

5.7.2.5 Outcrop 5 (565631 N 677217 E)

This is an approx. 150 m outcrop along the northwest bank of the Elbow River. The BZF comprise gently to steeply dipping and folded, medium to thick bedded, pale grey to cream sandstone interbedded with dark grey mudstones. The rock mass comprises a shallow syncline with the bedding at the southern end of the syncline dipping between 30 and 70° NE and at the northern end of the outcrop dipping between 25 and 64 SW.

Figure 60. Folded sandstone beds in the Brazeau Formation at OC5

5.7.2.6 Outcrop 8 (5656948 N 677904 E)

This is an approx. 120 m outcrop along the north bank of an old channel on the Elbow River. The BZF comprises steeply dipping and folded thinly bedded, pale grey to cream sandstone interbedded with dark grey mudstones. The dip of the bedding planes was typically between 40 and 50° NE.

Figure 61. Steeply Dipping Sandstone and Mudstones Beds in the Brazeau Formation at OC8

5.7.2.7 Outcrop 9 (565705 N 678080 E)

This is an approx. 70 m outcrop along the north bank of the Elbow River. The BZF comprises predominantly gently dipping, thin to thick bedded, pale grey to cream sandstone interbedded with subordinate dark grey mudstones. The dip of the bedding planes was typically between 25 and 50° NE.

Figure 62. Gently Dipping Sandstone Beds in the Brazeau Formation at OC9

5.7.2.8 Outcrop 10 (5657084 N 678253 E)

This is an approx. 200 m outcrop along the north bank of the Elbow River. The BZF comprises predominantly gently dipping, thin to thick bedded, pale grey to cream sandstone interbedded with subordinate dark grey mudstones. The dip of the bedding planes was typically between 15 and 30° NE.

Figure 63. Sandstone Beds in the Brazeau Formation at OC10

5.7.2.9 Outcrop 11 (5657152 N 678608 E)

This is an approx. 220 m outcrop along the north bank of the Elbow River. The BZF comprises predominantly gently dipping, medium to thick bedded, pale grey to cream sandstone interbedded with subordinate dark grey mudstones. The dip of the bedding planes was typically between 15 and 30° NE.

Figure 64. Sandstone and Mudstone Beds in the Brazeau Formation at OC11

5.7.3 Strength Testing

Following the 2016 exploration, direct shear and UCS tests were undertaken on 10 bedrock samples by Trican Geological Solutions. Detailed results of the direct shear and unconfined compressive strength tests are presented in Attachment 3. These were obtained from the 'vertical boreholes' and are likely to reflect the strength perpendicular to the bedding planes.

The UCS values ranged between 1.22 MPa for mudstone samples to 37.41 MPa for shale and sandstone samples.

The following friction coefficients were obtained from the direct shear tests:

- Shale DSNF Friction Coeff. = 0.43 (Peak), 0.25 (Residual)
- Shale/Mudstone DSSS Friction Coeff. = 0.54 (Peak), 0.47 (Residual)
- Mudstone/Claystone DSSS Friction Coeff. = 0.82 (Peak), 0.58 (Residual)
- Mudstone/Shale DSINT Friction Coeff. = 0.54 (Peak), 0.43 (Residual)
- Mudstone DSINT Friction Coeff. = 0.34 (Peak), 1.16 (Residual)

In 2018, a drilling program consisting of three boreholes was performed at the Debris Barrier site. The borings were advanced with a track mounted drill rig, equipped with HQ rock coring tools, between April 17th and April 27th, 2018. Each of the boreholes was advanced to a depth of 35 m. The recovered rock core consists of claystone, mudstone, sandstone, shale and coal. In general, the rock was highly weathered in the upper three to five meters and fractured throughout. Bedding dipped from approximately 45 degrees to near vertical. Rock core samples were photographed, boxed and transported to Stantec's Calgary laboratory for storage. Typed logs of the boreholes are included in Attachment 3.1.

Selected samples were pulled from the completed rock core for strength testing. Rock core testing was performed by Stantec's Lexington Kentucky materials laboratory. Testing consisted of Slake Durability Index (SDI) testing (ASTM D-4644), Unconfined Compression with Elastic Property Measurement (ASTM D-7012D), Anchor Pull-Out testing (method based on USACE method-Lienhart & Stransky 1985) and Direct Shear (ASTM D-5607). Direct shear testing was performed on natural fractures, smooth sawn interfaces and intact samples. The soft and highly fractured nature of some rock material resulted in the inability to perform appropriate laboratory testing. Parameters utilized in analyses were adjusted to account for the upward bias from testing performed only on medium to strong samples. Laboratory testing results from the 2018 rock testing program are included in Attachment 3.3.

The results from each of the types of direct shear tests were rendered into plots of shear stress versus normal stress. The strength envelope (line) characterizing the data points obtained is described by a slope (friction angle) and intercept (cohesion). For unfilled joints in rock masses, the cohesion was taken as zero and the envelope was constructed through the plot origin. A summary of the results from each type of test is included in Table 27 below.

Table 27. Summary of Direct Shear Rock Tests

Test of Type	Test Sample	Drained Strength	
		Cohesion (kPa)	Friction Angle ϕ'
Direct Shear Natural Fracture	Shale 1	0	22.5°
	Shale 2	0	24.6°
	Shale 3	0	21.0°
	Siltstone	0	10.6°
Direct Shear Smooth Sawn	Shale	0	17.3°
	Siltstone	0	27.7°
Direct Shear Intact Sample (residual)	Shale 1	0	38.1°
	Shale 2	0	27.7°

The unconfined compressive strength tests performed produced peak compressive stresses. Several test samples had strain gages attached to establish elastic properties of the rock. The following values were obtained:

Table 28. Summary of Unconfined Compression Rock Tests

Test Sample	UC (MPa)	E (kPa)	Poissons Ratio
Shale	8.89	2.83 E6	0.59
Claystone	1.10	0.76 E6	0.30
Siltstone 1	29.1	8.55 E6	0.28
Siltstone 2	6.07	7.86 E6	0.56
Siltstone 3	24.4	-	-
Siltstone 4	16.4	4.00 E6	0.46
Siltstone 5	25.2	8.55 E6	0.32
Sandstone	59.4	17.2 E6	0.26

To establish the durability of the rock samples collected SDI testing was performed on selected samples. The following values were obtained:

Shale (6 samples) 42.9% to 95.5% (73.9% average)

Siltstone (1 sample) 99.2%

Claystone (1 sample) 8.6%

Full laboratory test results are included in Attachment 3.3.

5.7.4 Unit Weight

The unit weight of the BZF is summarized in Table 29.

Table 29. BZF Unit Weight

Borehole ID	Core Interval	Sample Elevation (m)	Unit Weight		Bedrock Description
			(kg/m³)	(KN/m³)	
FB6	RC10	1206.75	2562	25.12	Poor Quality Gray Sandstone
DC6	RC19	1207.93	2379	23.33	Fair Quality Gray Sandstone
DC6	RC20	1206.63	2428	23.81	Fair Quality Gray Sandstone
DC7A	RC6	1211.34	3059	30.00	Very Poor Quality Gray Siltstone

5.7.5 Identification of Weak Layers

DC7 and subsequent laboratory testing identified the presence of 1 to 2 m thick, high plasticity clay. Atterberg testing has indicated that it plots nearer the U-line suggesting the presence of montmorillonite, however, no x-ray diffraction testing has been undertaken to confirm this and local experience suggests that these high plastic layers tend to be typically 0.1 m thick.

5.7.6 Durability

Slake durability index (SDI) testing was performed on 15 bedrock samples in the BZF. The SDI values ranged from 0.0 to 97.8. Low SDI values of 0.0, 0.8, and 5.6 were obtained from three mudstone and bentonite samples, moderate SDI values ranging from 31.0 to 67.8 were obtained from eight samples of mudstone, claystone, shale and sandstone, and higher values ranging from 91.5 to 97.8 were obtained from three shale samples.

5.7.7 Artesian Pressures

Artesian conditions were encountered in the diversion channel footprint during the 2016 Investigation. These were encountered in DC01 and DC05. In DC01, artesian conditions were encountered within the upper 4 m of bedrock. The equalized elevation of the water was 2.5 m above OG at Elevation 1238.3 m. In DC05, artesian conditions were encountered within the bedrock (unknown level). The equalized elevation of the water was 0.3 m above OG at Elevation 1242.4 m.

Artesian conditions were not observed in adjacent boreholes DC3, DC4 and DS7 to DS8 indicating a localized condition.

5.7.8 Selected Design Parameters

Based on Stantec's general understanding of the bedrock and the results of the laboratory testing noted above recommendations were developed for bearing and sliding of the debris structure foundation elements. The following values are recommended:

Sliding Friction	24° (μ = 0.45)
Sliding Cohesion	0 kPa
Bearing Capacity	500 kPa

6.0 EMBANKMENT CHARACTERIZATION

This section summarizes the earthworks requirements for the SR1 Project. The classification of materials as common excavation, borrow excavation and rock excavation is summarized in the following sub-sections.

The AT uses a nomenclature system for designating and specifying earthwork materials on drawings. Additional designations can be developed based on the type of material, its gradation and permeability, if required.

The results of the field and laboratory testing as well as the process of selecting engineering design parameters are detailed in the "Springbank Off-stream Reservoir (SR1) – updated Geotechnical Materials Properties Design Basis Memorandum – Selection of Soil Material Properties." This memorandum is included in Attachment 5.

6.1 SPECIFICATIONS

6.1.1 Civil Works Master Specification for the Construction of Provincial Water Management Projects

The specifications are based on the Civil Works Master Specification (CWMS) for the Construction of Provincial Water Management Projects (AT, 2016). This is the standard specification used when developing contracts for the construction of medium to large water management projects (dams, spillways, canals, control structures, erosion abatement works, and flood control dykes) that are owned by the GoA. Section 2330 of the CWMS is used to specify the quality requirements for earthwork materials. Section 2315 of the CWMS is used to specify the requirements for excavation.

6.1.2 Water Control Structures – Selected Guidelines

The Water Control Structures – Selected Guidelines was prepared jointly by the AT and the AEP (2004) to provide guidelines for the design of water control structures on provincially-owned water projects, provide guidance on some of the factors to be considered and applicable references; and facilitate the transfer of information and standards of good practice among consulting engineering firms involved on provincially-owned water projects.

6.2 CLASSES OF EXCAVATION

6.2.1 Common Excavation

Common excavation is defined as the excavation of on-site soils required by the contract documents, excluding topsoil and subsoil stripping, borrow area excavation, and rock excavation. The diversion channel will be the primary source of common excavation for the project. There will also be common excavation required for the floodplain berm and diversion structures.

6.2.2 Rock Excavation

Rock excavation is defined as rock materials that 'in an unfrozen state, cannot be ripped into individual detached masses smaller than 1.5 m^3 in size with a single tooth ripper mounted on a Caterpillar D8 or equivalent Group 10 Crawler Tractor as outlined in the Alberta Roadbuilders and Heavy Construction Association Equipment Rental Rates Guide or boulders larger than 1.5 m^3 in size' (AT, 2006). Rock excavation will occur along the diversion channel, at the diversion channel inlet structure, and at the emergency spillway.

6.2.3 Borrow Excavation

Borrow excavation is defined as the excavation of soil materials in the specified borrow areas, excluding topsoil and subsoil stripping. Two (2) borrow sources have been identified within the reservoir footprint.

6.3 EMBANKMENT ZONE 1A

6.3.1 Definition

The CWMS define this material as native soils obtained from required excavations or specified borrow areas with:

- Maximum size of 150 mm. Cobbles larger than 80 mm should be removed when used within 1 m of structures and 0.6 mm of pipes;
- Minimum plasticity Index of 7 percent;
- Minimum 50 percent passing the 80µm sieve size;
- No high plasticity clays with a liquid limit greater than 50 percent; and,
- No organic, deleterious or frozen materials

The Water Control Structures Selected Guidelines define the impervious fill as 'normally used to provide a relatively impermeable barrier to reduce seepage'. In comparison to the CWMS, they state that '*in cases where high plasticity clay with a liquid limit greater than 50 percent is to be used as impervious fill, special care will be required during compaction to prevent the build-up of excessive lateral earth pressures on the structure*'.

6.3.2 Impervious Fill Zone 1A Applications

Impervious Fill Zone 1A will be required for :

- Floodplain Berm Core
- Saddle Dam
- Storage Dam Core
- Low Level Outlet Works (LLOW) Backfill, and
- Diversion Structure Backfill

6.3.3 Sources

6.3.3.1 Diversion Channel

It is anticipated that the majority of the GT fill excavated from the Diversion Channel consisting of low plasticity clay (LL<50) can be used as Impervious Fill Zone 1A. It is anticipated that highly plastic GL fill (LL>50 percent) excavated from the Diversion Channel will not be used as Impervious Fill Zone 1A.

6.3.3.2 Borrow Source 1

It is anticipated that the majority of the GT fill excavated from Borrow Source 1 consisting of low plasticity clay (LL<50) can be used as Impervious Fill Zone 1A. It is anticipated that highly plastic GL fill (LL>50 percent), encountered in Borrow Source 1 Boreholes BS3, BS4 and BS5, will not be used as Impervious Fill Zone 1A.

6.3.4 Supplemental Requirements

It is recommended that the CWMS definition for Impervious Fill Zone 1A (GT materials) be supplemented to include the following additional requirements:

- maximum size of 75 mm
- minimum plasticity index (PI) of 10 percent

6.4 EMBANKMENT ZONE 2A

6.4.1 Definition

The CWMS define this material as native soils obtained from required excavations or specified borrow area with:

- Maximum size of 150 mm
- No high plasticity clays with a liquid limit greater than 50 percent,
- No organic, deleterious or frozen materials

The Water Control Structures Selected Guidelines do not provide any guidance on this material.

6.4.2 Random Fill Zone 2A Subclasses

There will be three subclasses of Random Fill Zone 2A based on the planned materials which will be excavated from the diversion channel and borrow sources.

- 2A (1): Soil Embankment
- 2A (2): Non-durable Rock/Soil Embankment
- 2A (3): Rock Fill Embankment

6.4.3 Random Fill Zone 2A Applications

Random Fill Zone 2A will be required at:

- Floodplain Berm Downstream Shell
- Structure Backfill
- Off-Stream Storage Dam Shell
- Miscellaneous Backfill

6.4.4 Random Fill Zone 2A Supplemental Specification Requirements

Supplemental specifications to the CWMS Random Fill Zone 2A requirements will be necessary for the subclasses of material discussed in Section 12.8.

Random Fill Zone 2A (1): Select Soil embankment may include moderate to highly plastic glacio-lacustrine clay borrow soils or glacial till clay borrow soils placed in the embankment shell and compacted to a minimum of 95 percent of standard Proctor value and placed in maximum 200 mm (8 inch) lifts with an allowable moisture content ranging from minus two percent to plus two percent of optimum moisture content.

Random Fill Zone 2A (2): Non-durable rock/soil embankment shall consist of soil and weathered, non-durable bedrock (SDI<85) placed in maximum 200 mm (8 inch) lifts. Large rock fragments shall be broken down into pieces less than 150 mm (6 inches) in any dimension or removed from the lift. Non-durable rock shall be broken down and watered to the satisfaction of the engineer prior to compaction. All Zone 2A (2) materials shall be approved by the engineer and compacted to 95 percent of the standard Proctor value or as required by the engineer.

Random Fill Zone 2A (3): Rock Fill embankment shall consist of sound durable sandstone and shale rock fill within the embankment shell zones with a minimum Slake Durability Index (SDI) value of 85. The maximum lift thickness shall be 600 mm (24 inches) with a maximum particle size of 450 (18 inches).

6.4.5 Sources

6.4.5.1 Diversion Channel

The majority of the fill excavated from the diversion channel can be used as Random Fill Zone 2A. This will comprise GT (typically BGST) and GL. There were localized areas of GL and GT (typically UBT) which had a LL > 50 percent and therefore, cannot be used under the CWMS without special care considerations.

The BZF can be used as Random Fill Zone 2A; however, the localized zone of high-plasticity clays encountered in DC7 must be classified as waste fill.

6.4.5.2 Borrow Source

The majority of the fill excavated from Borrow Source 1 can be reused as Random Fill Zone 2A. This will comprise GT and GL. There were areas of GL and localized GT encountered in BS3, BS4 and BS5, which had a LL > 50 percent and therefore, cannot be used under the CWMS without special care considerations.

6.5 WASTE FILL

6.5.1 Definition

The CWMS define this material as native soils obtained from required excavations or specified borrow area that do not meet the requirements for Impervious Fill Zone 1A or Random Fill Zone 2A; and/or are excess quantities of Impervious Fill Zone 1A or Random Fill Zone 2A. Waste Fill will only be placed in designated stockpiles and will not be used as engineered fill in the floodplain berm or the off-stream storage dam.

It is assumed that some of the weathered rock or non-durable rock may be classified as waste due to comingling of durable and non-durable rock and soil during excavation and subsequent difficulty with placement of the comingled material.

6.5.2 Stockpile Locations

Two (2) stockpile locations have been identified within the SR1 Project Site as shown in the preliminary design plans. The stockpiles have been located to provide an adequate volume of material for completion of the project.

6.6 SAND AND GRAVEL FILL

6.6.1 Definition

The CWMS define this material as natural (not crushed) well graded sand with a trace of gravel. The particle size envelope is defined in Table 30.

Table 30. Particle Size Distribution for Filter Materials

Sieve Size	Percentage Passing by Mass		
	Fine Filter Zone 3A	Coarse Filter Zone 3B	Coarse Filter Zone 3B (alternative)
40 mm	-	100 %	-
28 mm	-	-	100 %
20 mm	-	80 – 100 %	75 – 100 %
10 mm	100 %	40 – 80 %	40 – 85 %
5 mm	90 – 100 %	5 – 40 %	5 – 50 %
2.5 mm	70 – 95 %	0 – 3 %	0 – 3 %
1.25 mm	50 – 80 %	-	-
630 μm	25 – 55 %	-	-
315 μm	10 – 25 %	-	-

Sieve Size	Percentage Passing by Mass		
	Fine Filter Zone 3A	Coarse Filter Zone 3B	Coarse Filter Zone 3B (alternative)
160 µm	0 – 10 %	-	-
80 µm	0 – 3 %	0 – 2 %	0 – 2 %

6.6.2 Filter and Drainage Requirements

The filter and drainage requirements are discussed in Section 12.7. Based on the filter calculations, Zone 3A may be adequate; however, testing will be required prior to construction using the glacio-lacustrine soil and proposed Zone 3A source to verify the ability of the sand to serve as a filter for the glacio-lacustrine soils.

6.6.3 Sources

6.6.3.1 On-Site

No on site sources for filter materials have been identified. The sand and gravel alluvium associated with the east unnamed creek and the Elbow River would require processing and environmental permitting. It is planned that the Zone 3A material will be purchased from an off-site source.

6.6.3.2 Off-Site

Off-site sources have not been identified at this point.

6.6.4 Assumed Design Parameters

Because the Fine Filter Zone 3A will be obtained from an off-site source which has not been determined, laboratory testing for design parameters was not conducted. Typical values from published guidance documents were used for this material.

6.6.4.1 Density

A saturated unit weight of 21 kPa was assumed for the analyses. Assuming a typical specific gravity of 2.65 for sand, the Fine Filter Zone 3A would have a dry density of 17.4 kPa.

6.6.4.2 Strength

Using NAVFAC 7.01 (1986), for a $\gamma_d = 17.4$ kN/m^3, the angle of internal friction for drained strength would be approximately 33 degrees for an SM and SP type-material. It is assumed that the sand will exhibit no cohesion. For the analysis, a drained friction angle of 30° was selected.

Due to the free-draining nature of the foundation material, the undrained shear strength parameters are assumed to be the same as the drained shear strength parameters.

6.6.4.3 Permeability

From Cedergren (1997), a clean fine sand has a typical permeability of 1x10^{-3} cm/sec. This value was used for the analysis for the Fine Filter Zone 3A.

6.7 REMOLDED GLACIAL TILL EMBANKMENT PROPERTIES

6.7.1 Testing

A glacial till earthwork assessment has been undertaken using laboratory test data from areas designated as common excavation and borrow sources. The quantity of data comprised:

- 24 Standard Proctor on GT samples
- 12 permeability tests on remolded GT samples.
- 17 consolidated undrained (CU) triaxial tests on remolded GT samples

6.7.2 Remolded Standard Proctor Density

Standard Proctor moisture-density tests (ASTM D698) were performed on 24 disturbed GT bag samples. Optimum moisture content and maximum dry density values obtained from standard Proctor tests are presented in Figure 65. Calculated maximum dry unit weight values are presented in Figure 66. Estimated embankment unit weight values for GT samples compacted to 98 percent of standard Proctor density at optimum moisture content plus two percent (OMC + 2%) are presented in Figure 67.

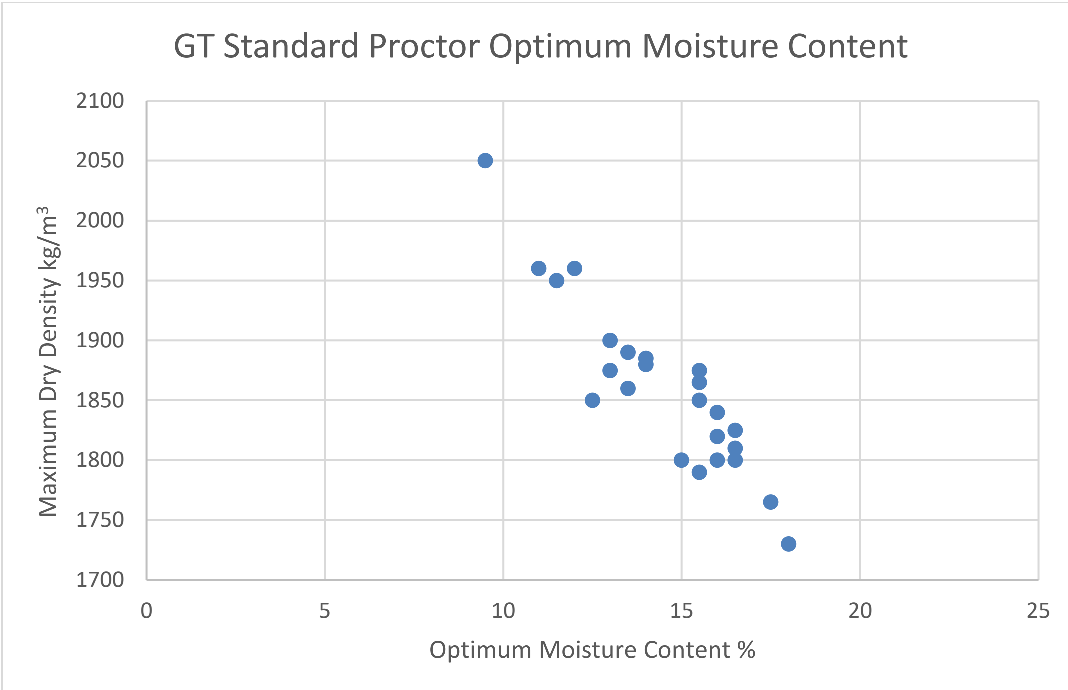


Figure 65. Glacial Till Standard Proctor Results

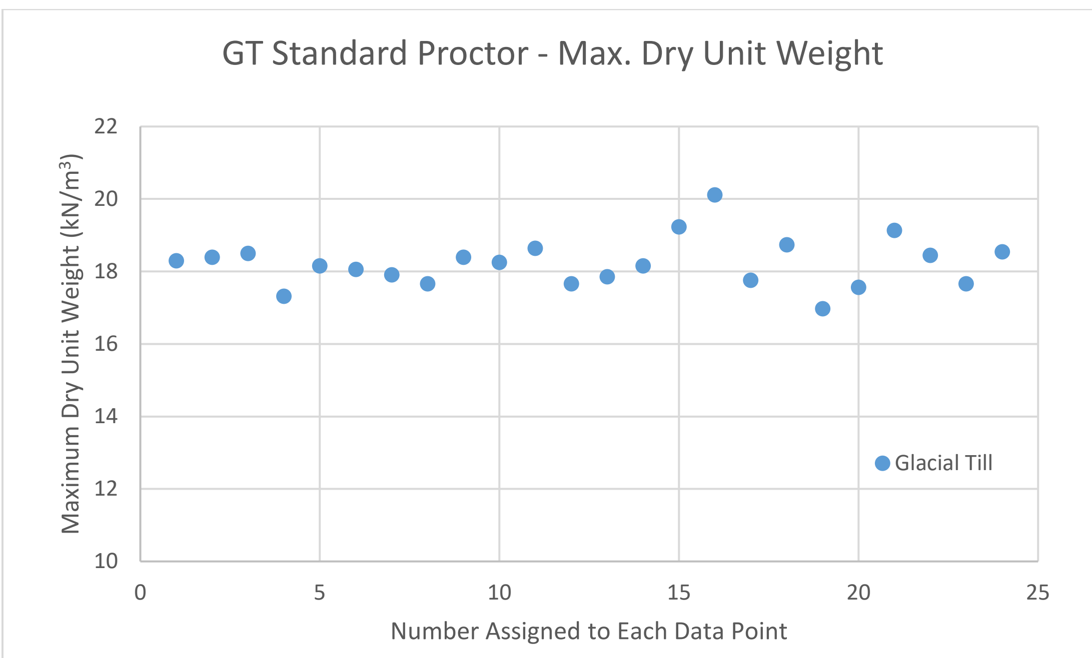


Figure 66. Glacial Till Standard Proctor – Maximum Dry Unit Weight

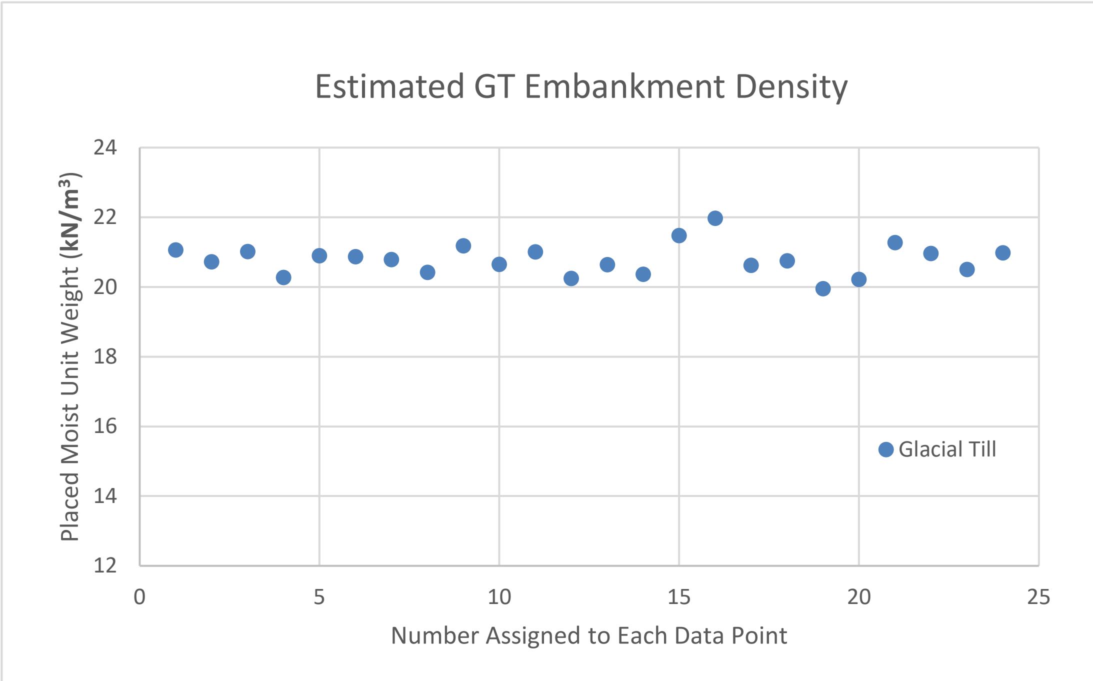


Figure 67. Estimated GT Embankment Density – 98% Proctor, OMC + 2 Percent

Based on the test results on remolded GT samples, the average maximum dry unit weight is 18.2 kN/m^3 and the average maximum dry density is 1858 kg/m^3. The average OMC is 14 percent. For GT embankment materials placed within Impervious Fill Zone 1 at 98 percent standard Proctor density and +2 percent of optimum, the estimated in-place unit weight is 20.8 kN/m^3 and the in-place density is 2120 kg/m^3. An average embankment unit weight value of approximately 20 kN/m^3 was chosen for all GT embankment materials in the analysis.

Project wide GT natural moisture content values from laboratory test results were compared to optimum moisture content values from the standard Proctor testing and reviewed for constructability purposes. A plot of project wide GT natural moisture content values and a window of optimum moisture content values is included in Figure 68. The data indicates that project wide in-situ GT moisture contents generally range from -7 percent to +7 percent of optimum moisture content.

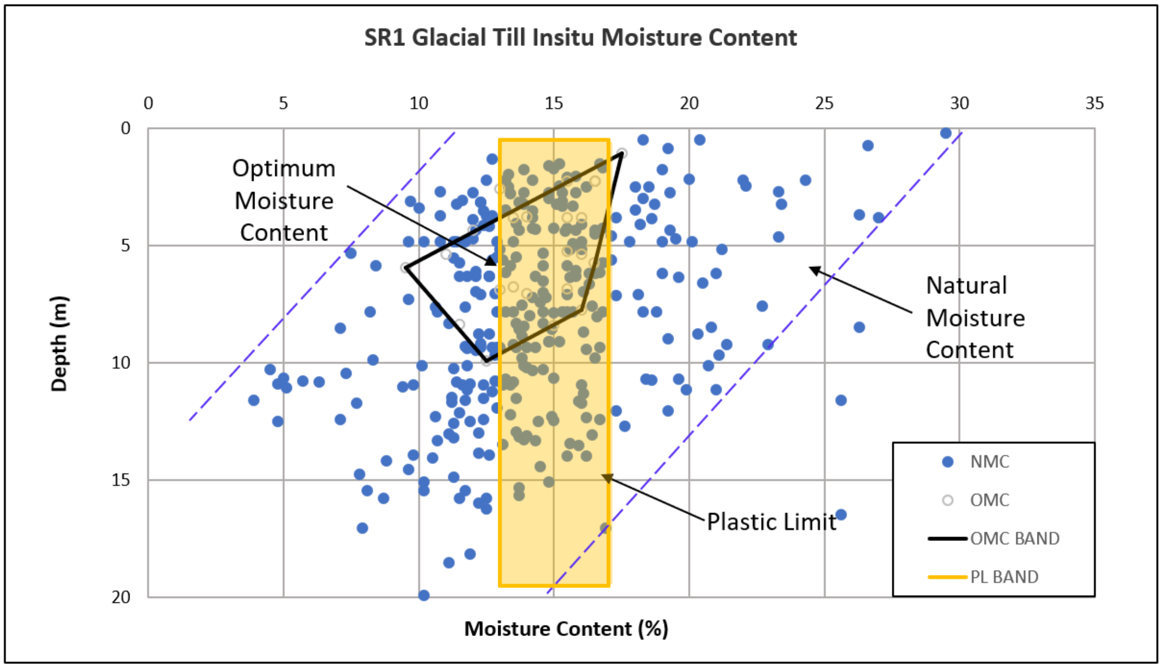


Figure 68. Glacial Lacustrine Natural and Optimum Moisture Contents

6.7.3 Hydraulic Conductivity

A total of 12 falling head permeability tests (ASTM D5084) were performed on remolded GT soil samples. The test results were reviewed and summarized by zone and by soil type, and geometric means of the test results were calculated for each zone. The results of the falling head permeability tests performed on remolded GT soil samples are summarized in Table 31.

Table 31. Summary of Permeability Values from Laboratory Testing – Glacial Till

Zone	Sample Type	Number of Falling Head Permeability Tests	Geometric Mean k_v (m/sec)
3	Remolded	4	1.17E-10
4	Remolded	2	4.42E-10
5	Remolded	1	1.50E-9
8	Remolded	1	8.90E-11
11	Remolded	2	6.05E-11
12	Remolded	1	5.10E-10
Borrow	Remolded	1	1.00E-9
Total Site	Remolded	12	3.81E-10

The range of permeability values obtained from remolded GT soil samples from each project zone is presented graphically in Figure 69.

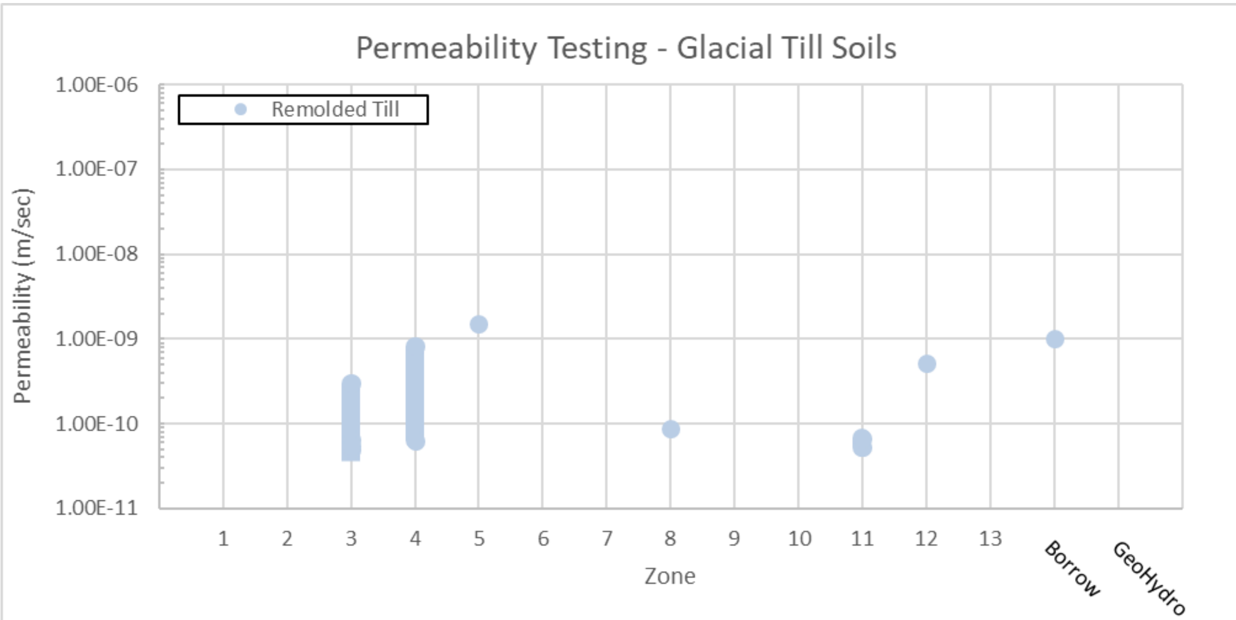


Figure 69 Spatial Distribution of Remolded Glacial Till Permeability Test Results

6.7.4 Drained Shear Strength

A total of 17 consolidated undrained triaxial (ASTM D4767) tests were performed on remolded GT soil samples. The test results were reviewed and summarized by zone and by soil type, and best fit angle of friction values were calculated for each zone. The range of best fit friction angles from single point CU tests performed on remolded GT soil samples from each project zone is presented in Table 32. A p-q plot of the combined CU test data for remolded GT soil samples is presented in Figure 70.

Table 32. Summary of Average Drained Shear Strength Values for Stability Analysis – Glacial Till

Feature	Zone	Sample Type	Number of CU Triaxial Tests	Best Fit Friction Angle (°)
Diversion Channel	3	Remolded	4	21.3
	4	Remolded	2	25.5
	6	Remolded	1	32.8
	8	Remolded	1	26.7
Storage Dam	11	Remolded	1	25.9
	12	Remolded	1	31.3
	13	Remolded	1	25.7
	Borrow	Remolded	6	28.4
Total Site		Remolded	17	27.7

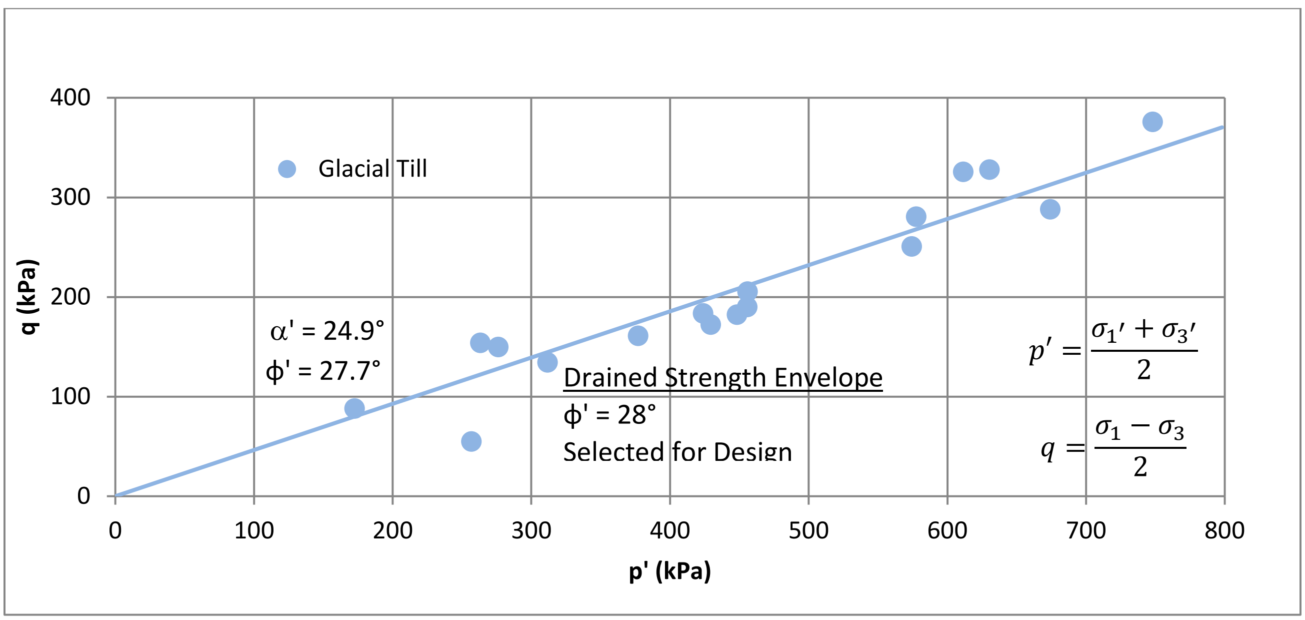


Figure 70. CU Triaxial Test Results - Remolded Glacial Till

CU triaxial tests performed on glacial till specimens were typically performed until 20 percent axial strain was obtained.

6.7.5 Undrained Shear Strength

Using data from the CU tests, undrained strength envelopes were fit to plots of shear strength on the failure plane $(S_U)_{ff}$ versus specimen consolidation pressure (σ'_{con}) (FERC 2006; USBR 2011). The undrained strength envelope is represented by Mohr-Coulomb parameters (c and φ) as shown in Figure 47.

The undrained shear strength on the failure plane $(S_U)_{ff}$ was calculated for each CU test performed. In a triaxial compression test, failure develops on a plane that is oriented at (45˚ + ϕ'/2) above horizontal (Duncan et al. 2014). The shear stresses on the failure plane in a particular CU test can be determined using Mohr's circle and the equation shown in Figure 47. Here, σ_{df} is the measured deviator stress at failure and ϕ'is the drained friction angle.

Results of $(S_U)_{ff}$ versus (σ'_{con}) for 17 CU tests on remolded Glacial Till samples are plotted in Figure 72. An undrained strength envelope fit through the remolded Glacial Till data results in c = 80 kPa and ϕ' = 19˚. The undrained cohesion and friction angle values shown in Figure 72 were obtained by linear regression.

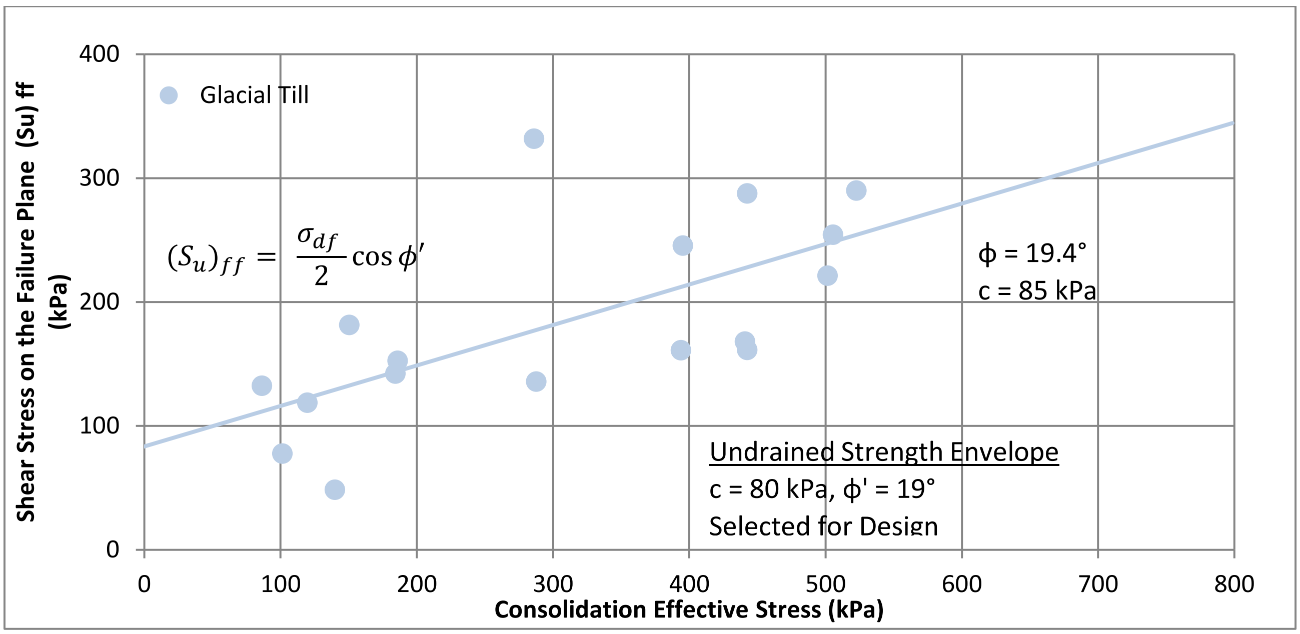


Figure 71. CU Triaxial Test Undrained Strength Envelope – Remolded Glacial Till

6.7.6 Selected GT Embankment Design Parameters

A summary of the selected design parameters for Glacial Till embankment materials is presented in Table 33.

Table 33. Summary of Selected Glacial Till Embankment Parameters

Glacial Till Design Parameter	Selected Design Value
Embankment Moist Unit Weight	20 kN/m^3
Embankment Moist Density	2120 kg/m^3
Hydraulic Conductivity (k)	3.81E-10 (m/sec)
Drained Shear Strength (ϕ')	28˚
Undrained Shear Strength (ϕ')	19˚
Undrained Cohesion (c)	80 kPa

6.8 REMOLDED GLACIAL LACUSTRINE ENGINEERING PROPERTIES

6.8.1 Remolded GL Engineering Properties

A glacial lacustrine earthwork assessment has been undertaken using laboratory test data from areas designated as common excavation and borrow sources. The quantity of data comprised:

- 9 Standard Proctor on GL samples
- 3 permeability tests on remolded GL samples.
- 5 consolidated undrained triaxial tests on remolded GL samples

6.8.2 Remolded Standard Proctor Density

Standard Proctor moisture-density tests (ASTM D698) were performed on 9 disturbed GL bag samples. Optimum moisture content and maximum dry density values obtained from standard Proctor tests are presented in Figure 73. Calculated maximum dry unit weight values are presented in Figure 74. Estimated embankment unit weight values for GT samples compacted to 98 percent of standard Proctor density at optimum moisture content plus two percent (OMC + 2%) are presented in Figure 75.

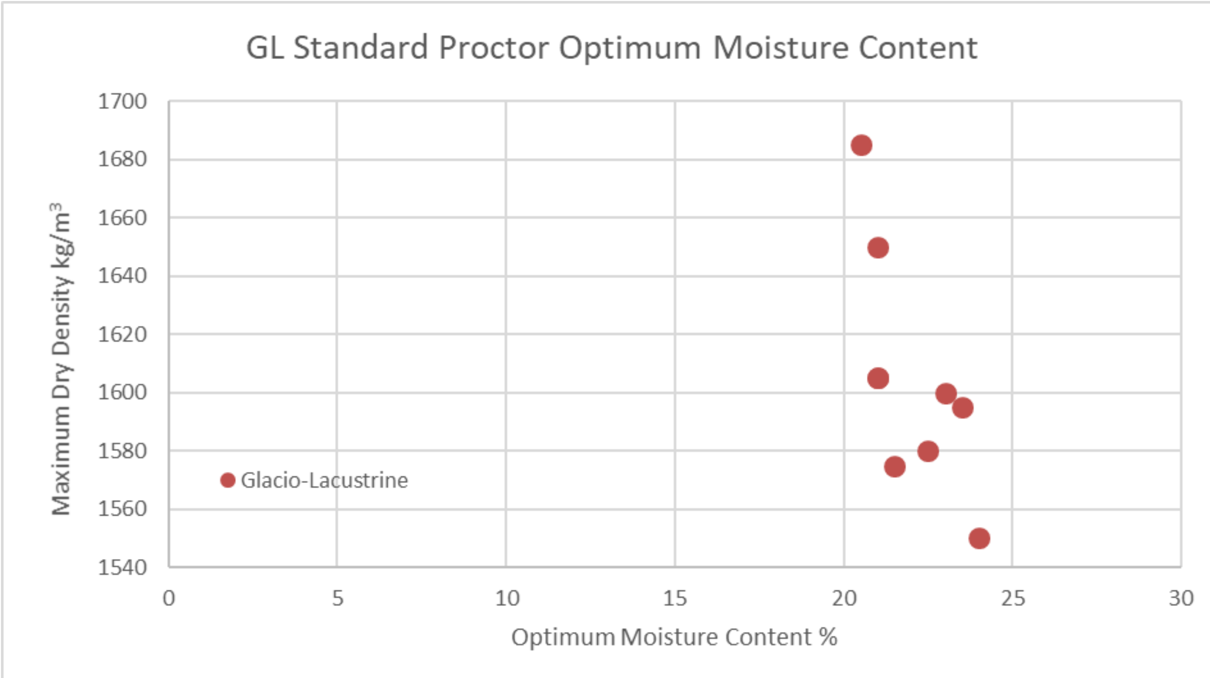


Figure 72. Glacial Lacustrine Standard Proctor Results

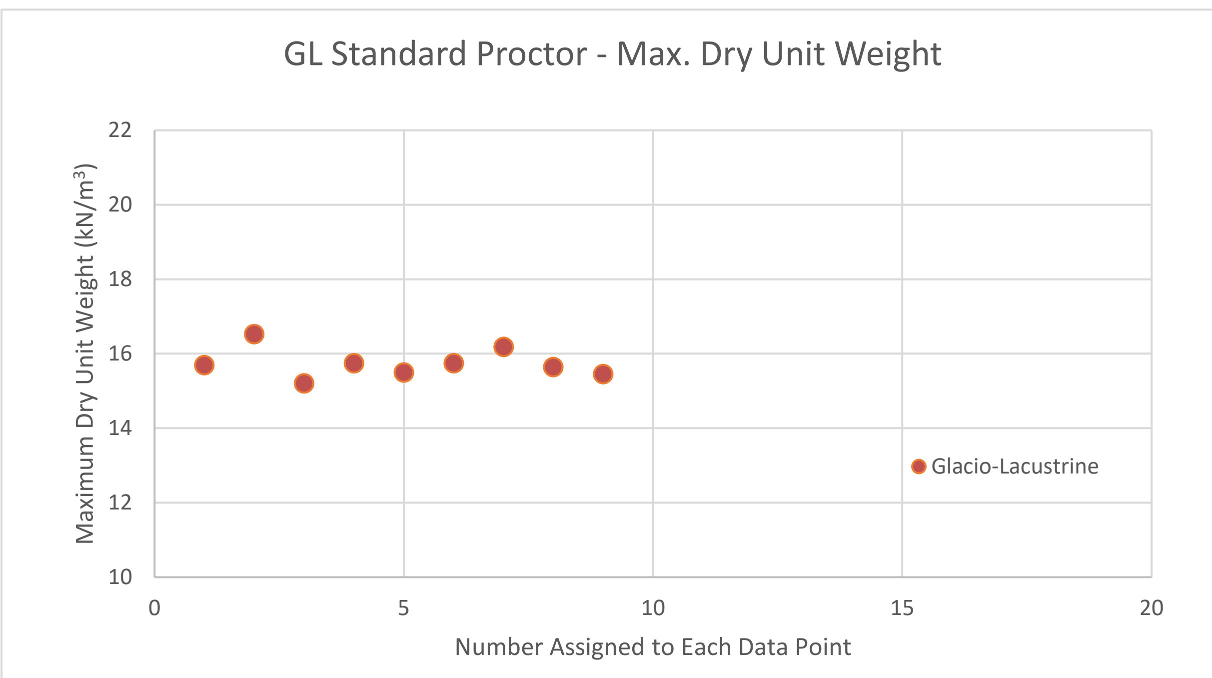


Figure 73. Glacial Lacustrine Standard Proctor – Maximum Dry Unit Weight

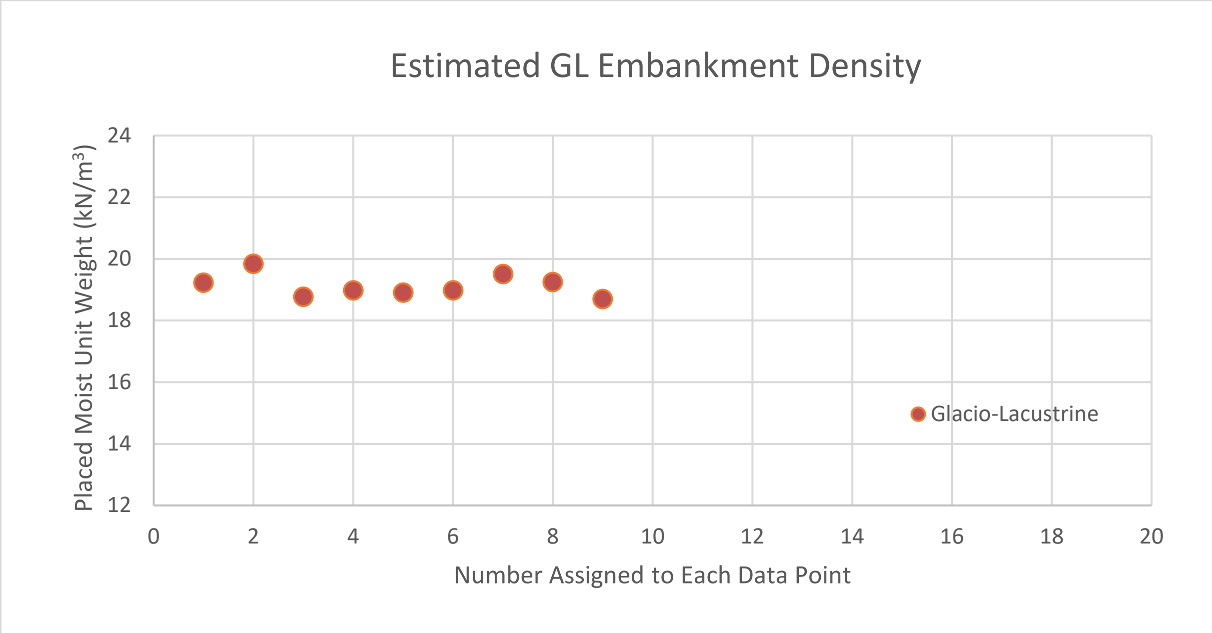


Figure 74. Estimated GL Embankment Density – 98% Proctor, OMC + 2 Percent

Based on the test results on remolded GL samples, the average maximum dry unit weight is 15.8 kN/m^3 and the average maximum dry density is 1605 kg/m^3. The average OMC is 22 percent. For GL embankment materials placed within Zone 2A at 98 percent standard Proctor density and +2 percent of optimum, the estimated in-place unit weight is 19.1 kN/m^3 and the in-place density is 1947 kg/m^3.

Project wide GL natural moisture content values from laboratory test results were compared to optimum moisture content values from the standard Proctor testing and reviewed for constructability purposes. A plot of project wide GL natural moisture content values and a window of optimum moisture content values is included in Figure 76. The data indicates that project wide in-situ GL moisture contents generally range from -4 percent to +10 percent of optimum moisture content.

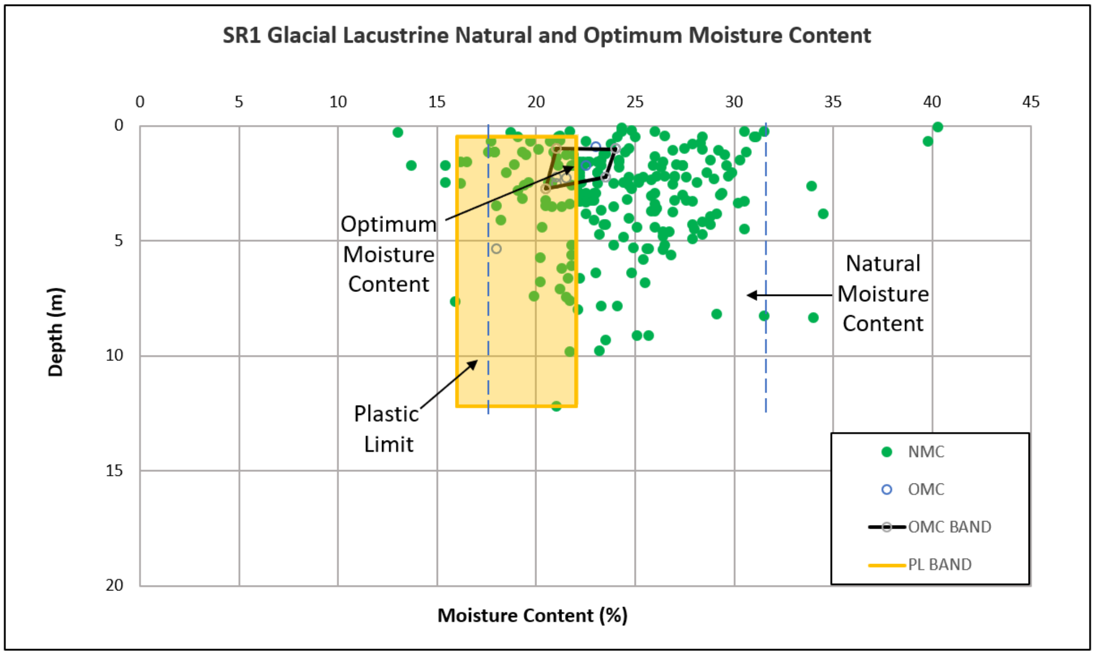


Figure 75. Glacial Lacustrine Natural and Optimum Moisture Contents

An average embankment unit weight value of approximately 20 kN/m^3 for the compacted GL materials was chosen for use in the analysis.

6.8.3 Hydraulic Conductivity

A total of three falling head permeability tests (ASTM D5084) were performed on remolded GL soil samples. The test results were reviewed and summarized by zone and by soil type, and geometric means of the test results were calculated for each zone. The results of the falling head permeability tests performed on remolded GL soil samples are summarized in Table 34. The range of permeability values obtained from remolded GL soil samples from each project zone is presented graphically in Figure 77.

Table 34. Summary of Permeability Values from Laboratory Testing – Remolded Glacial Lacustrine

Zone	Sample Type	Number of Falling Head Permeability Tests	Geometric Mean k_v (m/sec)
5	Remolded	1	1.10E-10
Borrow	Remolded	2	7.14E-10
Total Site	Remolded	3	5.12E-10

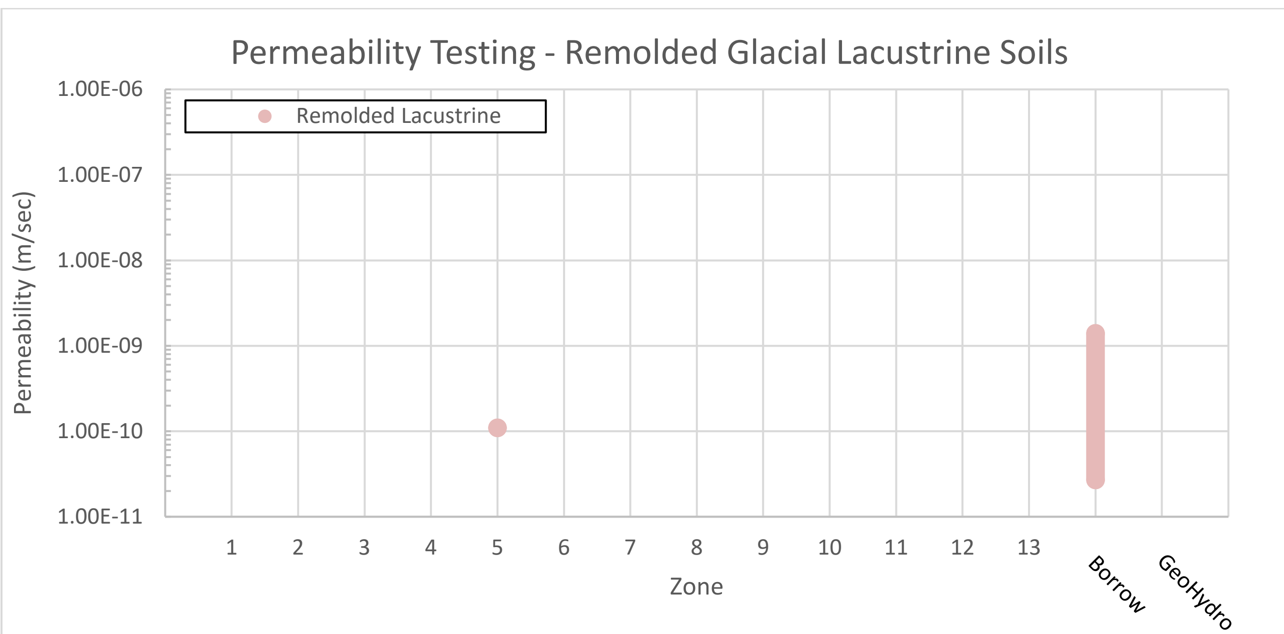


Figure 76. Spatial Distribution of Remolded Glacial Lacustrine Permeability Test Results

6.8.4 Drained Shear Strength

A total of five consolidated undrained triaxial (ASTM D4767) tests were performed on remolded GL soil samples. The test results were reviewed and summarized by zone and by soil type, and best fit angle of friction values were calculated for each zone. The range of best fit friction angles from single point CU tests performed on remolded GL soil samples from each project zone is presented in Table 35. A p-q plot of the combined CU test data for remolded GT soil samples is presented in Figure 78.

Table 35. Summary of Average Drained Shear Strength Values for Stability Analysis – GL Soils

Feature	Zone	Sample Type	Number of CU Triaxial Tests	Best Fit Friction Angle (°)
Diversion Channel	4	Remolded	1	27.9
	8	Remolded	1	21.6
Storage Dam	11	Remolded	1	25.1
	13	Remolded	2	24.5
Total Site		Remolded	5	24.3

The p'-q plot of the test results was used to develop design values. The intercept for c' was normalized to zero, and the typical friction angles are based on lines of best fit.

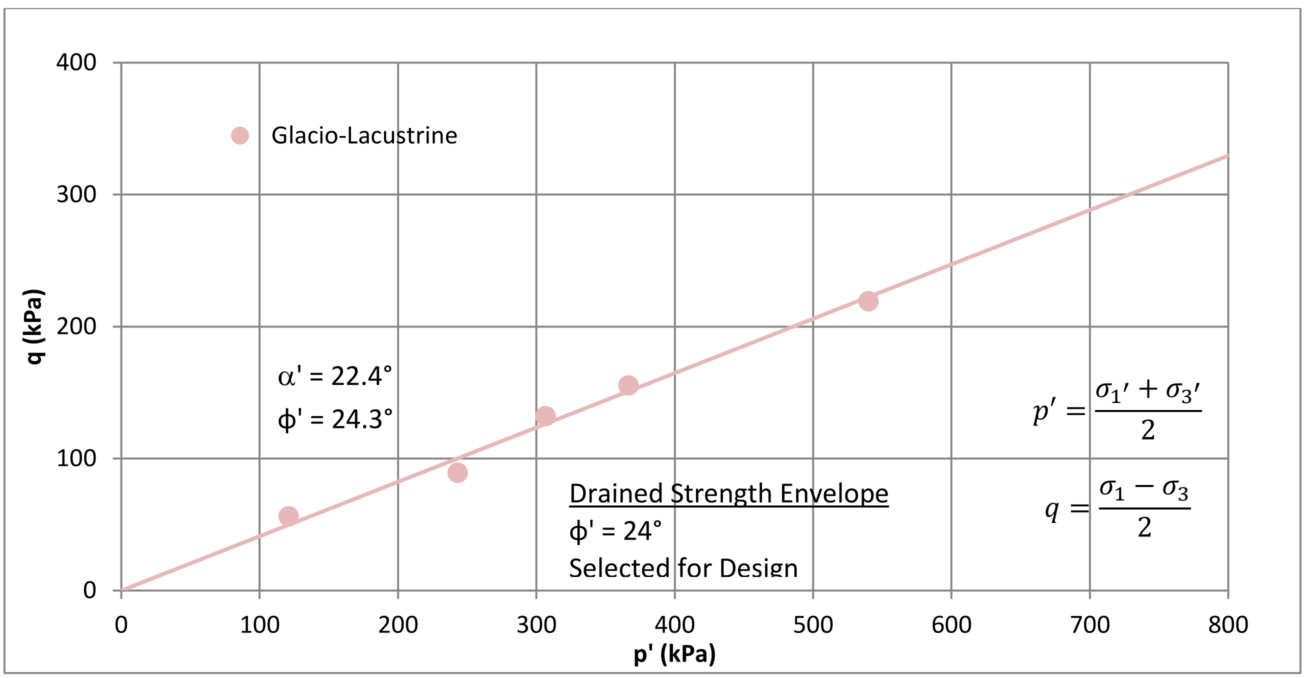


Figure 77. CU Triaxial Test Results – Remolded Glacial Lacustrine

CU triaxial tests performed on undisturbed glacial lacustrine specimens were typically performed until 20 percent axial strain was obtained.

6.8.5 Undrained Strengths

Results of $(S_u)_{ff}$ versus (σ'_{con}) for five CU tests on remolded Glacial Lacustrine samples are plotted in Figure 79. An undrained strength envelope fit through the remolded Glacial Lacustrine data results in c = 25 kPa and ϕ = 15˚. The undrained cohesion and friction angle values shown in Figure 79 were obtained by linear regression.

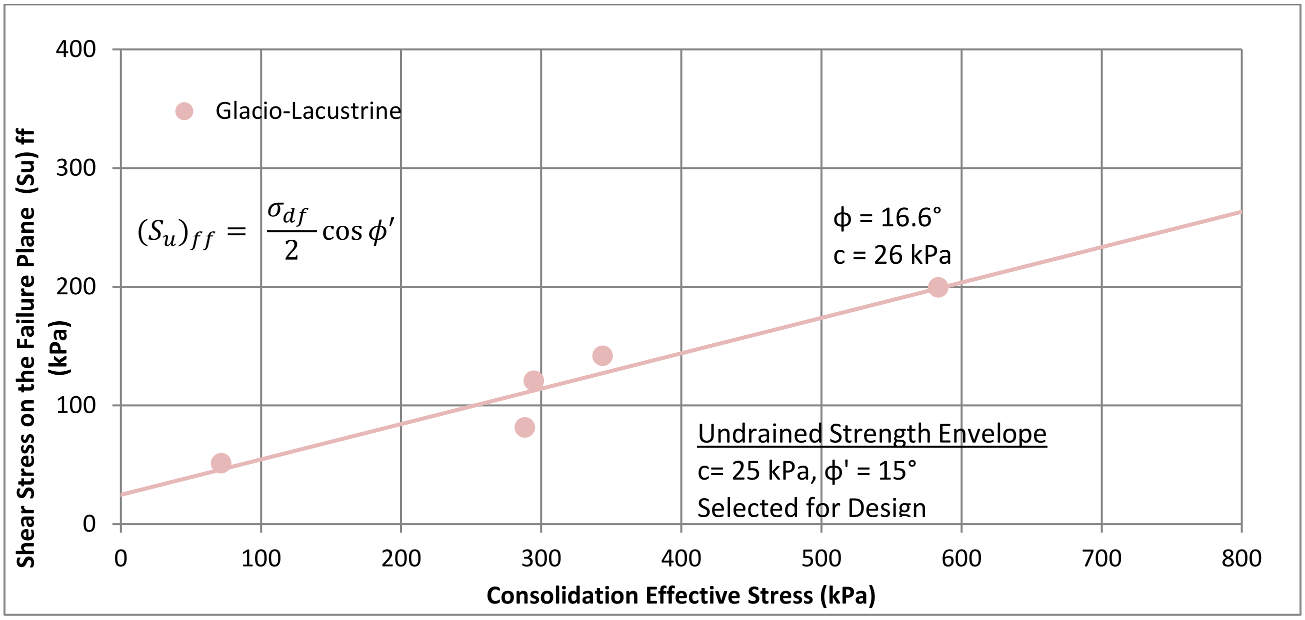


Figure 78. CU Triaxial Test Undrained Strength Envelope – Remolded Glacial Lacustrine

6.8.6 Selected Design Parameters

A summary of the selected design parameters for Glacial Till embankment materials is presented in Table 36.

Table 36. Summary of Selected Glacial Lacustrine Embankment Parameters

Glacial Till Design Parameter	Selected Design Value
Embankment Moist Unit Weight	20 kN/m^3
Embankment Moist Density	2120 kg/m^3
Hydraulic Conductivity (k)	5.12E-10 (m/sec)
Drained Shear Strength (ϕ')	24˚
Undrained Shear Strength (ϕ')	15˚
Undrained Cohesion (c)	25 kPa

7.0 PROBABILISTIC HAZARD SEISMIC ASSESSMENT

7.1 INTRODUCTION

The purpose of this Probabilistic Seismic Hazard Assessment (PHSA) was to define ground motion parameters for use in seismic design for the project. The seismic hazard assessment report (Stantec, 2017) is included in Attachment 7. Since the dam will be classified as an extreme consequence dam in accordance with CDA Dam Safety Guidelines (2007), the dam and associated appurtenant structures must be designed to resist an Earthquake Design Ground Motion (EDGM) with an Annual Exceedance Probability of 1/10,000.

7.2 SEISMICITY

The SR1 project site is situated in an area of low to moderate seismic activity. The site is located within the eastern limit of the Cordillerian deformation belt, which is characterized by closely spaced, low displacement NNW-SSE thrust faults. The Brazeau thrust fault is mapped as crossing the proposed diversion channel approximately 2 km west of the dam site. The review of published literature revealed no information with regards to known active faults in the project region. Accordingly, the seismic model for the project is based on areal sources rather than specific faults.

Induced seismicity is common in the foothills region of Southwestern Alberta. Induced seismicity in the foothills region has been associated with both hydraulic fracturing (i.e., “fracking”) and waste injection activities associated with oil and gas extraction.

7.3 ANALYSIS METHODOLOGY

The PSHA was performed using EqHaz software (Assatourians and Atkinson, 2013) which utilizes a Monte Carlo Simulation to generate a simulated earthquake catalogue, and computes the resulting earthquake motions using Ground Motion Prediction Equations (GMPEs).

Three different source types were included in the seismic hazard model: active crustal sources, a stable craton source, and a subduction interface source. Hazard contributions from sources more than 300 km from the site were excluded, with the exceptions of those from the Cascadia Interface Source.

The seismic hazard model incorporated appropriate GMPE suites for each of the source types. Maximum magnitudes for each seismic source were selected to reflect the information presented in the GSC Open File 7576, which documents the 2015 National Building Code of Canada (NBCC) seismic hazard model. However, the PSHA incorporated some modifications to the 2015 NBCC seismic hazard model.

7.4 EARTHQUAKE DESIGN GROUND MOTIONS

Based on measured shear wave velocity at the project site, EDGM values correspond to Vs30 = 425 m/s and Vs30 = 265 m/s, for the proposed diversion structure and dam, respectively. The Peak Ground Acceleration (PGA), Peak Ground Velocity (PGV) and 5 percent damped spectral accelerations for the horizontal component of the EDGM with an annual exceedance probability of 1/10,000 at the diversion structure and dam sites are presented in Table 37. Motions for intermediate periods may be estimated by linear interpolation.

Table 37. Horizontal EDGM Values for an Annual Exceedance Probability of 1/10,000 at the SR1 Project Site

Parameter	EDGM Values for an Annual Exceedance Probability of 1/10,000	
	Diversion Structure (Vs30 = 425 m/s)	Dam (Vs30 = 265 m/s)
PGV	17.5 cm/s	23.4 cm/s
PGA	0.26 g	0.28 g
Sa(0.05s)	0.34 g	0.33 g
Sa(0.1s)	0.54 g	0.47 g
Sa(0.2s)	0.63 g	0.64 g
Sa(0.3s)	0.54 g	0.69 g
Sa(0.5s)	0.35 g	0.52 g
Sa(1.0s)	0.18 g	0.26 g
Sa(2.0s)	0.072 g	0.11 g
Sa(5.0s)	0.022 g	0.034 g
Sa(10.0s)	0.0062 g	0.0087 g

The vertical to horizontal ratio for PGA, PGV and five percent damped spectral acceleration with an annual exceedance probability of 1/10,000 at the SR1 Project Site are presented in Table 38.

Table 38. Ratio of Vertical to Horizontal Ground Motions

Parameter	Ratio of Vertical to Horizontal Ground Motions	
	Diversion Structure (Vs30 = 425 m/s)	Dam (Vs30 = 265 m/s)
PGV	0.41	0.39
PGA	0.56	0.56
Sa(0.05s)	0.86	0.94
Sa(0.1s)	0.72	0.81
Sa(0.2s)	0.47	0.50
Sa(0.3s)	0.41	0.40
Sa(0.5s)	0.40	0.36
Sa(1.0s)	0.42	0.37
Sa(2.0s)	0.51	0.43
Sa(5.0s)	0.58	0.48
Sa(10.0s)	0.91	0.76

7.5 DE-AGGREGATION OF SEISMIC HAZARD

De-aggregation data provides information as to the source contribution of the seismic hazard at the site. Figure 79 and Figure 80 show de-aggregation plots for the 10,000 year PGA and T = 0.5s, respectively.

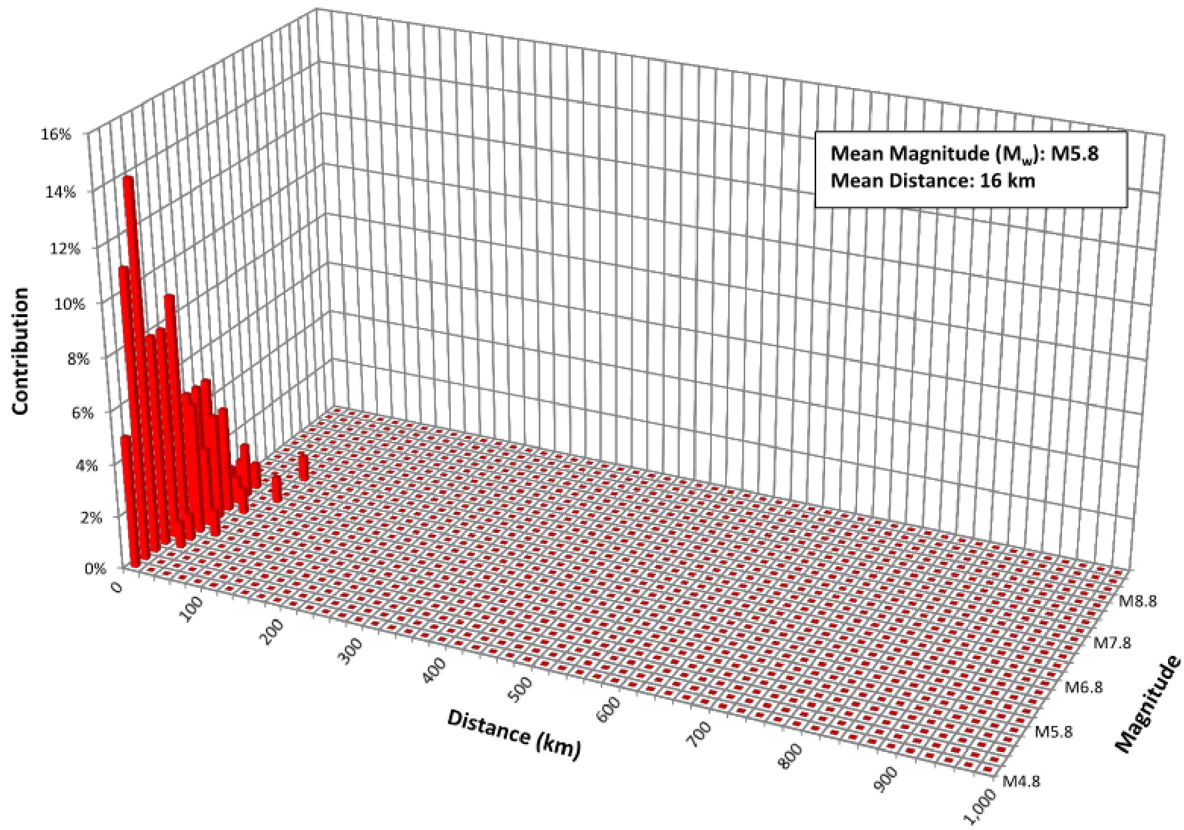


Figure 79. Deaggregation of 10,000 Year Return Period PGA

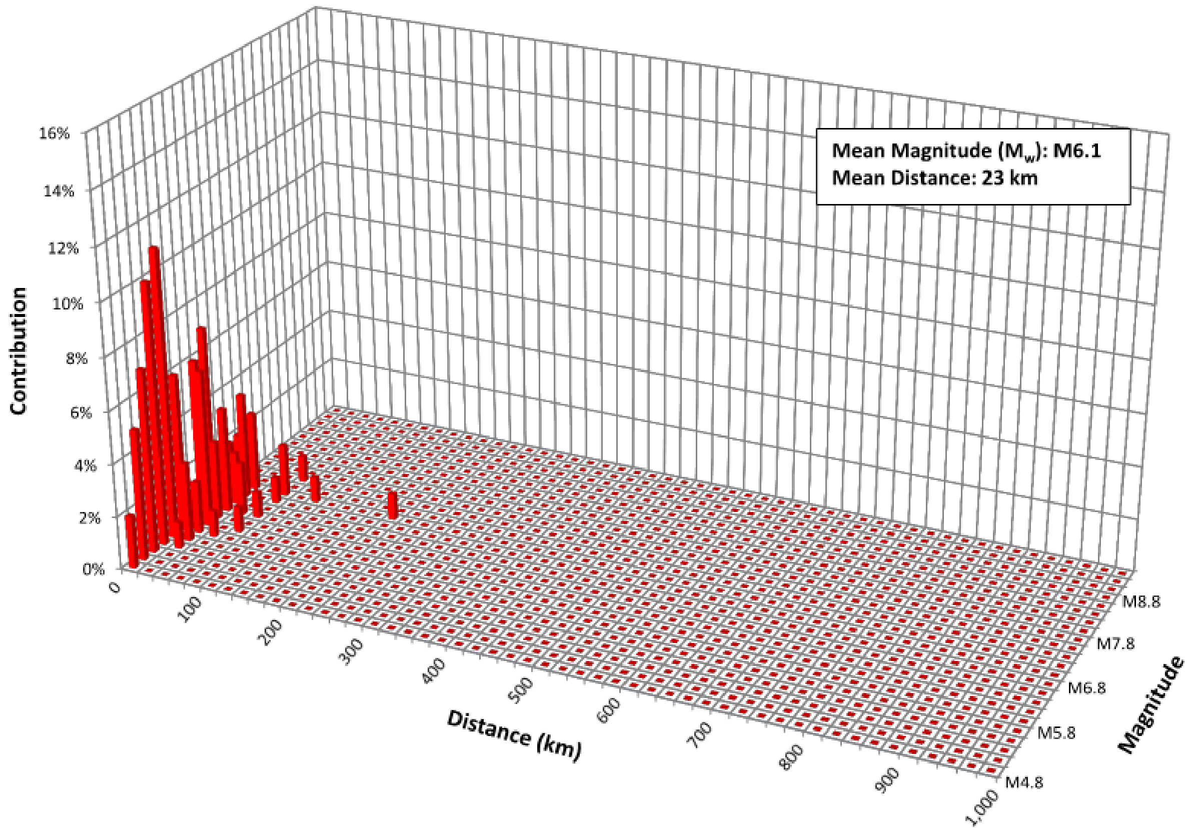


Figure 80. Deaggregation of 10,000 Year Return Period Sa(0.5s)

Based on the de-aggregation data, a magnitude of 6.0 would be appropriate for use in liquefaction or slope displacement analyses for the proposed dam.

7.6 EARTHQUAKE TIME HISTORIES

A suite of linearly scaled earthquake records was selected to approximately match the uniform hazard spectrum for an annual exceedance probability of 1/10,000 over a period range of 0.05 to 2.0 s. This is expected to encompass the period range applicable for the proposed dam and associated appurtenant structures. Motions were selected to represent the bedrock conditions in the vicinity of the dam site, which typically fall within the range of Site Class C (i.e., 360 m/s < Vs30 < 760 m/s). Earthquakes of all mechanisms (i.e. strike-slip, reverse, normal, etc.) were included in the earthquake search.

Eleven (11) sets of crustal earthquake records (i.e., three orthogonal components) from the Pacific Earthquake Engineering Research (PEER) Center's Ground Motion Database were selected to approximately match the target ground motion characteristics. The selected ground motion records with applicable scaling factors are presented in Table 39. All horizontal acceleration time histories should be used in the analysis.

Table 39. List of Selected Earthquake Motion Records

No.	RSN No.	Earthquake	Mw	Mechanism	Rjb (km)	Vs30 (m/s)	Scaling Factor (Horizontal)	Scaling Factor (Vertical)
1	212	Livermore-01 (1980)	5.8	strike slip	23.9	403	1.65	1.67
2	246	Mammoth Lakes-06 (1980)	5.94	strike slip	41.8	371	1.87	2.02
3	321	Mammoth Lakes-11 (1983)	5.31	strike slip	7.1	382	2.29	2.16
4	548	Chalfant Valley-02 (1986)	6.19	strike slip	21.6	371	1.35	1.12
5	671	Whittier Narrows-01 (1987)	5.99	reverse-oblique	31.6	508	2.00	2.19
6	3605	Lazio-Abruzzo_ Italy (1984)	5.8	Normal	20.0	437	2.24	1.53
7	3859	Chi-Chi_ Taiwan-05 (1999)	6.2	Reverse	53.0	438	2.47	2.12
8	4139	Parkfield-02_ CA (2004)	6	strike slip	9.5	417	1.06	1.30
9	6057	Big Bear-01 (1992)	6.46	strike slip	26.2	362	1.93	1.81
10	6060	Big Bear-01 (1992)	6.46	strike slip	40.9	368	1.81	1.39
11	6878	Joshua Tree_ CA (1992)	6.1	strike slip	21.4	368	1.48	1.45

8.0 PRELIMINARY DESIGN APPROACH

8.1 SCOPE

The following sections summarize the preliminary geotechnical design for the components of the SR1 Project Site. This has been divided into the following sections.

- Floodplain berm (Section 10.0)
- Auxiliary spillway (Section 10.0)
- Diversion structures: inlet and service spillway (Section 10.0);
- Diversion channel (Section 11.0)
- Diversion channel outlet (Section 11.0)
- Off-stream storage earthfill dam (Section 12.0); and,
- Low level outlet (Section 13.0)

The geotechnical and geological considerations for the reservoir rim and the utility diversions are provided in Sections 14.0 and 15.0, respectively.

8.2 DAM LIFECYCLE CONSIDERATIONS FOR DESIGN

8.2.1 During Construction

This preliminary design has assumed that the SR1 Project will be constructed in stages over a three (3) year period. This is based on pore pressure response within the foundation and embankment fill and tolerable vertical settlement and deformation at the toe. This approach requires:

1) The use of instrumentation program and design reviews during construction to monitor pore pressures and deformation during construction and winter breaks; and,
2) The implementation of contingency measures within the Construction Contract should monitoring indicate that the dam is not performing satisfactory during construction.

8.2.2 During Normal Operation

The SR1 dam is a dry earthfill dam that will temporarily retain a reservoir during the design flood event. Outside of the operation period, the reservoir will be dry and the dam will not retain water. The dam has been designed with a crest elevation of 1213.5 m. This is based on the Full Service Level (FSL) of Elevation 1210.8. The Inflow Design Flood (IDF) level for the reservoir is Elevation 1212.0 m and is based on the Probable Maximum Flood (PMF) routing calculations.

The operation sequence for a single flood event assumed for this design comprises:

1) Reservoir filling: hydrotechnical analysis has indicated that it will take approximately 2.5 days to fill the reservoir to the FSL of Elevation 1210.8 m[11] during the design flood event;
2) Reservoir impoundment at the IDF: the floodwater can be surcharged above the FSL for a maximum period of five (5) days. Drawdown of the reservoir level between Elevation 1212.0 m and Elevation 1210.8 m is directed through the emergency spillway; and,
3) Reservoir drawdown: The Operators will determine when the stored floodwater can be released in consultation with the City of Calgary. The reservoir will be drawdown from the FSL using the Low Level Outlet. The hydraulic capacity of the Low Level Outlet is 40 days; however, the drawdown period may vary due to riparian constraints, operational decision or unforeseen post-flood conditions downstream of the Low Level Outlet.

8.2.3 During First Fill

The first filling is recognized as a critical phase in the dam lifecycle. For a failure mode of internal erosion through the embankment, Foster et al (2000) found that 48 percent of dam failures and 26 percent of dam incidents occurred during the first fill.

While a limited commissioning test filling is currently proposed, significant filling will not occur until initial flood event containment. Accordingly, this Preliminary Assessment has assumed that the "first filling" will occur when the dam system is operated for the first time under the design flood conditions.

8.2.4 During Periods of Inactivity

The system will be operational but not functioning as designed. During this time, which could last months to years, the reservoir will be dry (no permanent pool) and the low level outlet is open allowing stormwater runoff from the catchment to drain into the unnamed creek.

[11] This assumes that minimal loss of reservoir capacity due to sedimentation and surface run-off from surrounding watershed.

This Preliminary Geotechnical Assessment has assumed that routine operations, inspections, monitoring and maintenance will be on going during periods of inactivity as per the Dam Safety Management System. For an extreme consequence structure, will comprise:

- Regular, ongoing inspections / investigations by the AEP; and.
- Dam, Safety Review (DSR) every five (5) years; and,
- Annual Performance Reviews when under construction or during years of operation.

8.3 DESIGN GUIDANCE

The design basis for each component of the SR1 Project has been prepared in accordance with the following guidelines:

- Alberta Dam and Canal Safety Directive (AEP, 2018);
- AT Engineering Consultant Guidelines for Highway, Bridge and Water Projects, Volume 1 – Design and Tender;
- Canadian Dam Association (CDA) Dam Safety Guidelines (2007);
- Canada Foundation Engineering Manual 4th edition (Canadian Geotechnical Society (CGS), 2006);
- Civil Works Master Specifications for Construction of Provincials Water Management Projects (AT, 2006 with revisions up to 2016); and,
- Water Control Structures - Selected Design Guidelines (AT / AEP, 2004).

Where the above documents did not provide adequate basis for design, industry accepted standards and Best Available Practices (BAP) were adopted. This included:

- Experience of dam design and construction from the Prairie Farm Rehabilitation Administration (PFRA) documented in Rivard (2014);
- Experience of previous AEP dam design and construction in Alberta (Dickson, Oldman, Twin Valley, Paddle River Dams);
- Technical guidance from the International Committee on Large Dams (ICOLD), United States Society on Dams (USSD), United States Army Corps of Engineers (USACE) and United States Bureau of Reclamation (USBR); and,
- Experience of levee design and construction in Europe and North America.

8.4 DISCUSSION OF STABILITY DESIGN CASES

8.4.1 Water-Retention Earthfill Dams

The CDA Dam Safety Guidelines (2007) define the range of loading conditions to be typically considered in assessing the stability of a dam as:

- Normal Operating conditions: defined as the conditions for which the dam is 'expected to experience during normal operations'. This includes the 'normal maximum operating steady state conditions and long-term slope stability';
- Unusual Loading Conditions: 'may occur on an exceptional basis; they stress the structure more, in specific ways, than the normal conditions'. This includes 'end of construction pore pressures, extreme operational hydrological conditions and severe wave action'; and,
- Extreme Loading Conditions: 'correspond to highly improbable events, which, if they to occur, would be considered emergencies'. This includes 'extreme earthquake and hydrologic loading that result in the minimum residual freeboard'.

Based on Section 8.2, the SR1 Project does not conform to the above typical loading conditions. The function, i.e. the normal operating condition, of the constructed dam system is really an unusual or potentially, an extreme transient loading condition. The development of a suitable stability design basis based on these unique operating conditions and the traditional standards-based approach to design are discussed below:

8.4.1.1 Canadian Dam Safety Guidelines

The CDA Guidelines (2013) define five (5) 'generally-accepted' minimum FOS for various embankment dam loadings (Table 40). This is based on the traditional standards-bases approach to dam safety.

Table 40. CDA Stability Guidelines

Loading Condition		Minimum FOS	Slope
Static	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 - 1.3	Upstream
Seismic	Pseudo-static	1.0	Upstream and downstream
	Post-earthquake	1.2 -1.3	Upstream and downstream

These were developed for conventional water or tailings retention structures that have a permanent reservoir (with operational fluctuations as required), which develops steady-state conditions over time after first filling. As discussed in Section 8.4.1, the CDA guidelines does not cover all of the operating conditions of the SR1 Dam System.

The CDA Dam Safety Guidelines (2013) does not include any prescriptive FOS for a temporary flood loading condition. The guidelines state that the '*distribution of pore pressures in the entire continuum of the dam system should be evaluated for different loading cases and stages, such as the following: under transient loading, such as rapid or sudden drawdown, floods, and earthquakes*' (CDA 2007). Given the ambiguity over this statement, Stantec has reviewed internationally accepted design documentation to identify suitable FOS for a traditional standards-based approach.

8.4.1.2 United Kingdom – Environment Agency

Guidance on the design, operation and adaptation of reservoirs for flood storage was developed in the United Kingdom by the Environment Agency and Department of Environment, Food and Rural Affairs (2016). This does not contain any specific geotechnical design codes for dam embankments used as flood storage reservoirs. It is also states that 'there are currently no international standards which specifically apply to flood storage reservoirs, but general dam engineering principles and standards should be applied'. Reference is made the Eurocode 7 design guidelines, USBR design standards, the USACE engineering manuals, the International Levee Handbook (CIRIA, 2013) and to the 'Application of Eurocode 7 to the Design of Flood Embankments (CIRIA, 2014).

The International Levee Handbook (2013) is the result of a joint research project between CIRIA (UK), Ministry of Ecology (France) and the USACE (USA) that identified the best practices in the safety assessment, design, construction and management of levees across Europe and North America. This document references the approaches defined in the Eurocode 7, USACE engineering manuals and the Department of Water Resources (DWR) in California.

The Design of Flood Embankments (CIRIA, 2014) describes the application of Eurocode 7 to the design of flood embankments in the United Kingdom. The methodology is based on partial factors instead of the global FOS method typically used in Canada and the USA. Therefore, it is not discussed further in this Preliminary Geotechnical Assessment.

8.4.1.3 California Department of Water Resources

The Urban Levee Design Criteria produced by DWR (2012) provides the following design guidance on levees.

Intermittently Loaded Levees

The minimum FOS is 1.4 for downstream failure surfaces that intersect the crest based on the design water surface elevation, and 1.2 based on phreatic surface at the hydraulic top of levee. The phreatic surface is assumed to be steady state but a lower surface can be considered based on the design hydrograph, fill type, geometry and performance history. For river stage loadings, the steady state phreatic surface has to be justified through transient seepage analysis, hydraulic data and field performance data.

Frequently Loaded Levees

This is defined as a levee that experiences a water surface elevation of 0.33 m (1 ft.) higher than the corresponding elevation on the downstream toe for at least once a day for more than 36 days per year on average. They follow the guidance in EM 1110-2-1913 (USACE, 2000) which states that '*embankments that are subject to water loading for prolonged periods (longer than normal flood protection requirements) or permanently should be designed in accordance with earth dam criteria rather than the levee criteria given herein*'.

The minimum FOS is 1.5 for downstream failure surfaces assuming steady state seepage at the design water surface elevation, and 1.3 based on a phreatic surface at the hydraulic top of levee. A phreatic surface lower than the steady-state condition defined by the Seep/W analysis cannot be used.

8.4.1.4 Embankment Dams Design Standard - United States Bureau of Reclamation

Section 4 of the Embankment Dam Design Standards No. 13 (USBR, 2011) provides the following guidance for evaluating the static stability of dams under flood loading:

'If the phreatic surface under flood loading is significantly different (higher) from that of the steady-state condition for the active conservation pool, then the stability under this (higher phreatic surface) condition should be analyzed'.

'A phreatic surface should be estimated for the maximum reservoir level. The maximum reservoir level may occur from a surcharge pool that drains relatively quickly or from a flood control pool that is not to be released for several months. The hydraulic properties (permeability) of materials in the upper part of the embankment affected by the reservoir fluctuations should be evaluated to determine whether a steady-state or transient analysis should be made when estimating the position of the phreatic surface'.

'If the phreatic surface is significantly different (higher) from that of the steady-state condition for the active conservation pool, then the stability under this (higher phreatic surface) condition should be analyzed.'

'For the operational conditions, a factor of safety of 1.2 for assumed steady-state seepage conditions under maximum reservoir water level during a probable maximum flood event would be justified if the duration of high flood pool is relatively short and the reservoir operations call for draining the flood storage quickly using spillway and outlet works facilities at the dam site, and restoring the reservoir to the active conservation pool'.

'A higher factor of safety (approaching 1.5) might be required if the duration of flood storage above the active conservation pool is long and could potentially result in phreatic surface which is significantly higher than the steady-state phreatic surface under the active conservation pool.'

The document does not quantify the terms 'relatively short' and 'draining of the flood storage quickly'. However, for the purpose of this Preliminary Geotechnical Assessment, we have assumed that the floodwater will be retained for relatively short period of time.

8.4.1.5 United States Army Corps of Engineers

The US Army Corp of Engineers guidelines (USACE, 2003) provide design criteria for a 'surcharge pool loading condition:

'The surcharge pool is considered a temporary pool, higher than the storage pool, that adds a load to the driving force but often does not persist long enough to establish a steady seepage condition. The stability of the downstream slope should be analyzed at maximum surcharge pool.'

In this loading condition, the minimum FOS is 1.4 for a maximum surcharge pool load case. This assumes that the maximum surcharge pool is modelled as a surcharge thrust on the dam and that the pore pressures are assumed to be those developed under normal storage conditions, which in the case of the SR1 project, is assumed to be at ground level.

8.4.1.6 Recommended Design Basis

Based on the above discussions, we have recommended that the slope stability assessment of the main dam, saddle dam and floodplain berm be undertaken using the CDA guidelines with the temporary operating condition modelled using the approaches recommended by the USBR and USACE (Table 41).

Table 41. Recommended Design Load Cases for Off-Stream Storage Dam, Saddle Dam, and Floodplain Berm

Load Case	Reference	Reservoir	Foundation Behavior	Pore Pressures	FOS
End of Construction	CDA	None	Undrained strength parameters;	Phreatic surface in foundation	1.3
End construction – multi-year construction	CDA	None	Undrained strength ratio (c/p) in the GL		1.3
	CDA, PFRA	None	Drained strength parameters	Phreatic surface modelled in the foundation and B-bar applied to the foundation and embankment fill	1.3
Not operational - long Term	CDA	None	Drained strength parameters	Phreatic surface in foundation	1.5
Operation - Design Flood	USBR	IDF	Drained strength parameters	Steady state seepage in embankment dam;	1.2
	USCAE	IDF	Undrained strength parameters	Flood pool modelled as a surcharge; phreatic surface in foundation	1.4
Rapid Drawdown	CDA	IDF	Undrained strength parameters	Multi-stage phreatic surface from reservoir	1.2
Seismic – Pseudostatic	CDA	IDF		Flood pool modelled as a surcharge; phreatic surface in foundation	1.0
Seismic – Post Earthquake	CDA	IDF	Residual strengths in liquefied units		1.2

8.4.2 Diversion Channel

The consequences of a slope failure will depend on whether the diversion channel is in operation, the location of the failure along the channel length, and reservoir level at the time of failure. This assessment has made the assumptions:

- At full design capacity, the diversion channel will be in operation for a maximum of 2.5 days, i.e. until the reservoir has filled up. Whilst, it could be operated at lower flows rates for a longer period, this assessment has assumed that the excavated slopes will not become saturated during that time;

- The critical scenario will be a failure immediately prior to a design flood event. This assumes that a section of the side-slope fails during a rainfall event prior to a flood and 1) puts critical downstream infrastructure and the general public at risk; and 2) does not give sufficient time for maintenance work to take place before the SR1 Dam System is activated and becomes operational. The consequences of this type of failure could be partial damming of the channel resulting in back-up or re-routing of the flood waters, effectively making the SR1 dam system non-functional; and,
- For failures during a design flood event, it is likely that the debris from a localized slope failure would be swept along by the diverted waters. The modelling of the flood waters in a limiting equilibrium model would act as a toe surcharge and not reflect worse-case conditions. Therefore, this condition has not been analyzed.

Table 42. Recommended Design Load Case for the Diversion Channel Slope Stability

Load Case	Reference	Reservoir	Foundation Behavior	Pore Pressures	FOS
Long Term	CFEM	None	Drained strength parameters	Phreatic surface	1.5

8.4.3 Structures

The design cases for the structures are in accordance with the CDA Dam Safety Guidelines and CFEM 4th edition (2006).

8.5 STABILITY ANALYSIS METHODOLOGY

8.5.1 Geology Profiles

The geology profiles used for the Slope/W and Seep/W analyses were developed using the RGM and SGM developed in Sections 4 and 5.

8.5.2 Methodology

The slope stability of the main and saddle dams, floodplain berm, diversion channel and diversion structure excavations has been assessed using conventional 2D limiting equilibrium analysis. Numerical modelling, (finite element analysis) was used only to estimate construction period pore pressure on sections with thick GL foundation layers.

The analysis was undertaken using the limiting equilibrium modelling software, Slope/W (part of the Geostudio 2012® suite). The analysis was undertaken using the following generalized methodology:

- The Morgenstern-Price Method was used to identify the critical failure surface;
- No negative pore pressures were allowed to generate in the analysis (suction was capped at 0 kN/m^2);
- Optimization of the failure surface was applied to the critical failure surface. Judgement was applied for the resultant surface as this method can produce kinematically-implausible slip surface shapes;
- Rapid drawdown of the main dam, saddle dam and floodplain berm was modelled using the built-in feature in Slope/W, which is based on the method developed by Duncan et al (1990) and Duncan and Wright (2005);
- The surcharge pressure from the flood event in the USACE Method is produced by applying a surcharge equivalent to the height of column of reservoir water at the IDF multiplied by the unit weight of water; and
- Analysis of the pseudo-static design case was undertaken using the built-in multi-stage feature in Slope/W. Analysis of the post-earthquake design case was undertaken using a single-stage analysis with reduced strengths in Slope/W.

8.5.3 Pore Pressure Response

8.5.3.1 End of Construction

The pore pressure response of the earthfill, GL, and GT units was modelled within the Slope/W analysis using estimated pore pressures at the time of analysis. The pore pressures were estimated using both the simplified B-bar methodology and the finite element analysis method. The development of construction period pore pressures from these two methods are discussed in Section 12.5.3.

The $\overline{B}$ values in the Slope/W analysis are summarized in Table 43.

Table 43. B-bar Values assumed for the End of Construction

Material	Location	Years Loaded	B-bar for Case 3 End of Year 3 Construction					
			Station 20+000	Station 21+050	Station 21+750	Station 22+500	Station 22+990	Station 23+175
GT Foundation	Crest	1, 2 & 3	0.10	0.10	0.15	0.45	0.4	0.4
	Slopes	1 & 2	0.10	0.10	0.15	0.3	0.4	0.3
	Toe	1	0.10	0.10	0.1	0.15	0.3	0.2
GL Foundation	Crest	1, 2 & 3	0.15	0.15	0.3	0.8	-	-
	Slopes	1 & 2	0.15	0.15	0.3	0.45	-	-
	Toe	1	0.15	0.15	0.25	0.3	-	-
GT Embankment Core	Year 1	1, 2 & 3	-	-	-	0.5	-	0.5
	Year 2	2 & 3	-	-	0.4	0.4	0.55	0.4
	Year 3	3	0.0	0.0	0.0	0.0	0.0	0.0

8.5.3.2 During Impoundment

Full saturation of earthfill dams and steady state conditions can take decades to develop in Alberta (Chin , 1994). Given the likely short duration of impoundment, it is recognized that the earthfill dam will likely not become saturated during operation. However, as part of this assessment, we have considered the USBR design case which assumes stead state conditions with a FOS of 1.2 (Table 41).

8.5.4 Preliminary Design Parameters

The slope stability input parameters for the preliminary design of each component are summarized in Table 44.

Table 44. Slope Stability Analysis Input Parameters

Unit	Component	Unit Weight	Drained Friction Angle (deg)	Undrained Friction Angle(deg)	Undrained Cohesion (kPa)
Impervious 1A Fill	Main dam, saddle dam, Floodplain Berm	20	28	19	80
Random 2A Fill	Main dam, saddle dam, Floodplain Berm	20	24	15	25
Riprap	Floodplain Berm	22	38	N/A(1)	N/A(1)
Rock Toe	Main Dam	20	33	N/A(1)	N/A(1)
Elbow River Alluvium	Floodplain berm, diversion structure	20	27	N/A(1)	N/A(1)
Unnamed Creek Alluvium	Main dam	22	35	N/A(1)	N/A(1)
Glacio-lacustrine	Main dam, saddle dam, reservoir	18	23	20	15
Glacio-lacustrine (alt. method)				$S_u/\sigma_v' = 0.265 * OCR^{1.5}$	
Glacial Till		18	27	19	60
Weathered Bedrock	Main Dam	21	35	N/A[1]	N/A[1]
Brazeau Formation	Diversion channel	21	Curved strength envelope (Hoek-Brown model), see Table 56	N/A[1]	N/A[1]

[1]Undrained strengths are equal to drained strengths

8.5.5 Seismic Analysis Methodology

8.5.5.1 Liquefaction Susceptibility

Liquefaction susceptibility of the foundation units was evaluated using criteria developed by Seed et al (2003), Bray and Sancio (2006) and Idriss and Boulanger (2008).

8.5.5.2 Pseudostatic Analysis

A pseudo-static screening analysis was undertaken in Slope/W to assess the seismic stability of the main dam, saddle dam and earthfill dam. The seismic stability was assessed for a 1:10,000-year return period seismic event using PGA values derived from the PHSA for the diversion structure and dam locations. An earthquake design ground motion of 0.5 PGA was applied to the pseudo-static stability analysis (Hynes-Griffin and Franklin, 1984).

The probability of the 1:10,000-year design earthquake occurring when the SR1 system is functioning during a flood event with the establishment of a steady state seepage condition was considered to be extremely low. Therefore, the pseudo-static analysis assumed that the reservoir applied a surcharge only.

For undrained loading, a composite, bilinear drained-undrained strength envelope was used. The undrained strengths of the composite envelope were reduced to 80 percent of the static undrained strength (Hynes-Griffin and Franklin, 1984).

8.5.5.3 Displacement Analysis

The deformations of profiles which have a FOS > 1.0 were estimated using the slope displacement analysis proposed by Bray and Travasarou (2007). The FOS > 1.0 was considered a threshold value and was only used to indicate the requirement for further analysis.

8.5.5.4 Post-Earthquake

The stability of the main dam, saddle dam and floodplain berm was evaluated for the static conditions immediately following the cessation of the earthquake motions.

8.5.6 Seepage Analysis

8.5.6.1 Methodology

2D seepage analysis has been undertaken on the floodplain berm, diversion channel and off-stream storage dam. This was undertaken using the finite element modelling software, Seep/W (part of the Geostudio 2012® suite).

Modelling approaches used for specific components of the dam system and design cases are discussed below:

- This Preliminary Geotechnical Assessment has only considered steady state conditions. No transient analysis was undertaken at this stage;
- Boundary conditions were set for all structures using planned IDF or PMF flood elevations, existing groundwater elevations from piezometers and monitoring wells, river elevations, or anticipated groundwater elevations following construction. Where a pipe or relief well was modeled, a zero pressure boundary condition or total head boundary condition at the pipe elevation was used.

The input parameters for the Seep/W analyses are summarized in Table 45.

Table 45. Seepage Analysis Input Parameters

Material Name	Seepage Model Inputs	
	k_h (m/s)	k_v/k_h
Impervious Fill (1A)	1.00E-10	0.30
Random Fill (2A)	3.00E-10	1.00
Fine Filter (3A)	1.00E-05	1.00
Crushed Rock	1.00E-05	1.00
Riprap	3.00E-04	1.00
Alluvium	1.00E-06	1.00
Glaciolacustrine	1.00E-10	0.30
Glacial Till	3.00E-10	1.00
Bedrock	1.00E-08	0.15

8.5.6.2 Internal Erosion Assessments

The hydraulic gradients predicted in the seepage model were considered to evaluate the potential for piping at the seepage exit, in the area of the downstream toe of the off-stream storage dam and floodplain berm.

Terzaghi et al. (1996) identify and describe two principal modes of piping failure: (a) piping due to erosion and (b) piping due to heave. The conditions leading to piping and heave at a seepage exit can be reviewed analytically and a factor of safety can be assessed. Internal piping due to erosion defies theoretical treatment, making it impractical to quantify a factor of safety. Piping due to erosion cannot be analyzed numerically.

Safety factors for piping due to heave, defined in terms of the seepage exit gradient, are described below.

Where water seeps vertically upward to a relatively level surface, the factor of safety against piping at the seepage exit (FOS_{exit}) is evaluated in terms of the vertical exit gradient. The analysis is based on the effective stress in the soil in the area of the seepage exit.

Heave and piping occurs when the upward gradient is equal to the critical gradient (i_{crit}) from USACE, 1986:

$$\Delta H = H\left(\frac{1}{c'}\right)\log\frac{p_0+\Delta p}{p_0}$$ **Equation 3**

The factor of safety for the exit seepage is then given by:

$$FS_{exit} = \frac{i_{crit}}{i}$$ **Equation 4**

A FOS equal to 3.0 is generally required at the downstream toe of a dam (USACE, 1986)

8.5.7 Settlement Estimates

The settlement of the foundations and earthfill materials for the main dam, saddle dam and Floodplain Berm were estimated using mythologies described in CFEM (2006).

Equation 5 was used to calculate settlement in sand and gravel layers.

$$\Delta H = H\left(\frac{1}{c'}\right)\log\frac{p_0+\Delta p}{p_0}$$ **Equation 5**

Equation 6 was used to calculate settlement in normally consolidated clay layers.

$$\Delta H = H\left(\frac{C_c}{1+e_o}\right)\log\frac{p_o+\Delta p}{p_o}$$ **Equation 6**

Equation 7 was used to calculate settlement in over consolidated clay layers.

$$\Delta H = \frac{H}{1+e_0}\left(C_r \log\frac{p_c}{p_o} + C_c \log\frac{p_f}{p_c}\right) \qquad \textbf{Equation 7}$$

where:

H = thickness of subdivided layer

c' = bearing capacity index (from SPT N-values)

p_o = initial effective overburden stress at center of subdivided layer

p_c = preconsolidation pressure from Casagrande procedure

p_f = final effective vertical stress at center of subdivided layer

e_o = initial void ratio

C_r = recompression index

C_c = compression index

9.0 FLOODPLAIN BERM PRELIMINARY DESIGN

9.1 DESCRIPTION

The Floodplain Berm is a 1.2 km long earthfill embankment located to the southeast of the diversion structure. It will be constructed on the alluvial terraces adjacent to the active Elbow River channel. The purpose of the Floodplain Berm is to prevent floodwaters from flanking the service spillway when the gates are closed and direct flow to the diversion structure and channel.

The Floodplain Berm has a 6 m wide crest with an elevation that ranges between 1221.5 m at the tie-in with original ground and 1218.4 m at the Auxiliary Spillway. The maximum embankment height along the length of the berm is 5.5 m.

For the Floodplain Berm, two typical sections were analyzed.

- Station 0+600 to 0+900 m: Typical Section A;
- Station 0+900 to 1+600 m: Typical Section B.

Typical Section A comprises a homogenous earthfill embankment constructed with Impervious 1A Fill (Figure 83). The upstream slopes are 3H:1V with no upstream riprap protection. The 'structural' geometry of the downstream slope is 3.2H:1V but this may be flattened during construction to include surplus Impervious 1A or Random 2A Fill. Foundation preparation will include a 0.5 m clean-up.

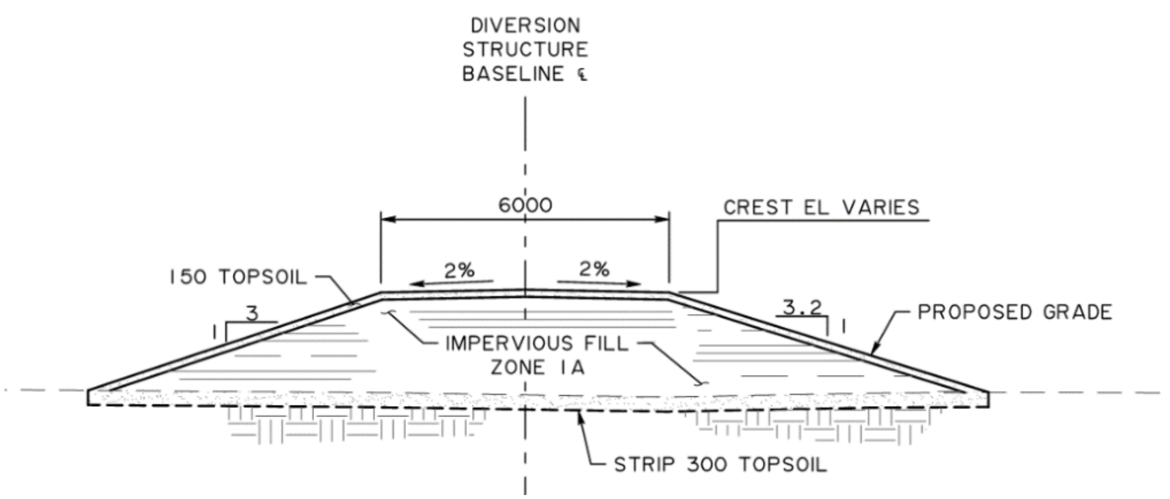


Figure 81. Floodplain Berm Typical Section A Configuration

Typical Section B comprises a zoned earthfill embankment constructed with a core of Impervious 1A Fill and 3A Filter on the downstream side of the core (Figure 82). The upstream slopes are 3H:1V with riprap protection. The 'structural' geometry of the downstream slope is 3.2H:1V but this may be flattened during construction to accommodate surplus Impervious 1A or Random 2A Fill.

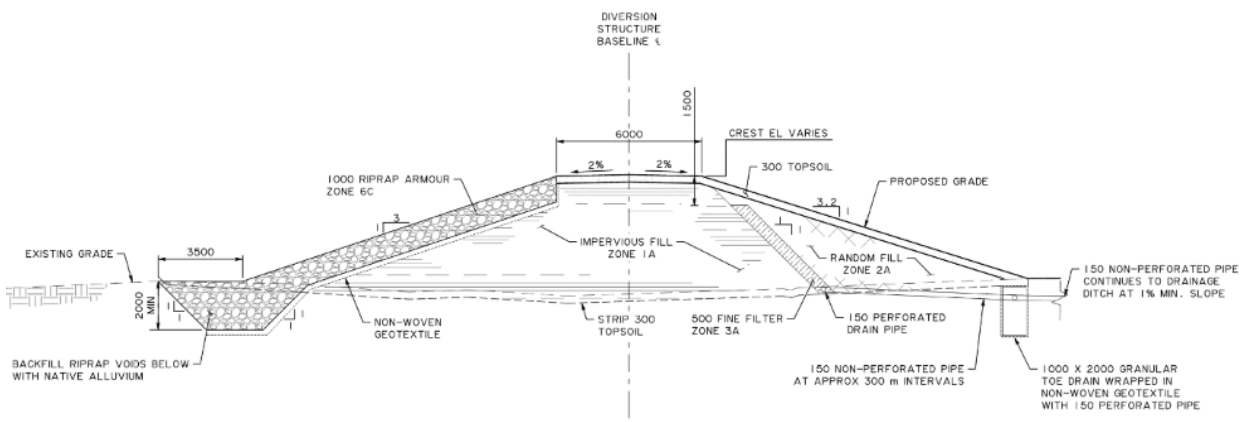


Figure 82. Floodplain Berm Typical Section B Configuration

9.2 DESIGN CONSIDERATIONS

The Floodplain Berm is a 'high' consequence structure based on the CDA Dam Safety Guidelines (2007).

9.3 SITE-SPECIFIC GROUND CONDITIONS

Five (5) boreholes were undertaken beneath or adjacent to the floodplain berm footprint. Access was constrained by dense vegetation and the location of abandoned river channels. The boreholes indicated that the top of the BZF bedrock ranged between Elevation 1207.2 (FB7) and 1210.4 m (FB3). This comprised steeply dipping, weak, interbedded mudstones, siltstone and sandstones (Section 5.7). The upper 3 m was typically highly weathered and poor quality. The BZF was overlain by 3.4 to 4.9 m thick layer of Elbow River alluvium sub-unit (Section 5.2.1).

9.4 SEEPAGE ANALYSIS

The objective of the seepage analysis was to model the phreatic surface, flow and hydraulic gradients across the selected profiles under the long term, flood operations, and Max IDF conditions.

9.4.1 Profiles

Seep/W analysis was undertaken on the two profiles (Table 46).

Table 46. Profiles Analyzed at the Floodplain Berm

Profile	Notes
St. 0+900	Represents the segment of the berm from St. 0+600 (beginning of berm) to 0+900. The berm is between 0 to 2 m high with foundation soils between 0 to 5 m thick. Foundation consists of fluvial material (gravel and sand) underlain by bedrock. Weak weathered mudstone or claystone present above top of competent bedrock.
St. 1+600	Represents the segment of the berm, from St. 0+900 to 1+628. The height of the berm is between 2 to 6 m high. Weak weathered mudstone present above top of competent bedrock.

9.4.2 Floodwater Elevations

The seepage analysis was conducted following the methodology discussed in Section 8.5.6. The floodplain berm will function as a dry structure with no pool for normal conditions. However, seepage analyses were performed to determine the steady-state phreatic surface at pool elevations of Flood of Record (FoR) Operations Condition and the Diversion Structure IDF (IDF-DS) events. The headwater and tailwater elevations were obtained from the 2D hydraulic model directly at the applicable cross section stations along the floodplain berm. Some assumptions were made about tailwater elevations for the Normal and Flood Operations load cases, where the ground surface was above the tail water elevation obtained from the Hydraulic Model. The headwater and tailwater pools were modeled as fixed head boundary conditions. The specific boundary condition elevations used for each section are included in Table 47 below.

Table 47. Headwater and Tailwater Elevations – Floodplain Berm and RCC Auxiliary Spillway

Cross Section Station	Normal[1]		FoR Operations		Max IDF-DS	
	Headwater (m)	Tailwater (m)	Headwater (m)	Tailwater (m)	Headwater (m)	Tailwater (m)
0+900	1217.4	1216.6	1219.2	1218.4	1219.7	1218.9
1+600	1213.6	1211.1	1216.1	1212.1	1217.4	1213.8

1. Normal headwater and tailwater conditions are below the ground surface elevation.

9.4.3 Foundation Heave

The exit gradients from the Seep/W model were evaluated at the downstream toe of the berm to a depth of approx. 1.5 m for the FoR Operations and IDF-DS event. A critical exit gradient of 1.04 was calculated using a void ratio (e) of 0.59 and a specific gravity (G_s) of 2.65 for the alluvial foundation material. The FOS was then determined. The results are presented in Table 48 and plots of the seepage analysis results are included in Attachment 1.

Table 48. Factors of Safety Against Piping Due to Heave

Cross Section Station	Calculated Exit Seepage Gradient		Factor of Safety Against Piping Due to Heave	
	FoR Operations[1]	Max IDF-DS	FoR Operations	Max IDF-DS
0+900	N/A	0.023	N/A	45
1+600	N/A	0.029	N/A	35

1. There is no tailwater pool during the FoR operations flood event.

Based on the results of the seepage analysis, adequate factors of safety are predicted for piping due to heave using the typical sections.

9.5 SLOPE STABILITY ANALYSIS

9.5.1 Profiles

The same profiles used for the Seep/W analysis were modelled for the stability analysis.

9.5.2 Stability Load Cases

The load cases evaluated are described in Table 49 below.

Table 49- Critical Design Load Cases for the Floodplain Berm

Load Case	Reference	Headwater & Tailwater	Foundation Behavior	Pore Pressures	FOS
End of Construction	CDA	Existing	Undrained strength parameters	Phreatic surface in foundation	1.3
No Pool - long Term	CDA	Existing	Drained strength parameters	Phreatic surface in foundation	1.5
Operation - Design Flood	USBR	Flood of Record and IDF-DS	Drained strength parameters	Steady state seepage in embankment	1.2
	USACE	Flood of Record and IDF-DS	Undrained strength parameters	Flood pool modelled as a surcharge; phreatic surface in foundation	1.4
Rapid Drawdown	CDA	Flood of Record and IDF-DS	Undrained strength parameters	Multi-stage phreatic surface from headpond	1.2
Seismic – Pseudostatic	CDA	Flood of Record and IDF-DS	Short Term, Undrained Seismic Parameters	Flood pool modelled as a surcharge; phreatic surface in foundation	1.0[1]
Seismic – Post Earthquake	CDA	Flood of Record and IDF-DS	Short Term, Undrained Seismic Parameters	Flood pool modelled as a surcharge; phreatic surface in foundation	1.2

1. Used to trigger deformation analysis only.

9.5.3 Results

The results of the slope stability analysis for each load case for the proposed floodplain berm are presented in Table 50. The Slope/W outputs are included in Attachment 9.1.

Table 50. Summary of Stability Analysis – Floodplain Berm

Load Case	Section	Factors of Safety					
		Normal		FoR Operations		IDF-DS	
		DS	US	DS	US	DS	US
End of Construction - Total Stress Analysis (Target FOS = 1.3)	0+900	2.2	1.9	--	--	--	--
	1+600	1.5	1.6	--	--	--	--
Long Term Drained (Target FOS = 1.5)	0+900	2.2	1.9	--	--	--	--
	1+600	1.5	1.6	--	--	--	--
Flood Load - USBR Method (Target FOS = 1.2)	0+900	--	--	2.0	--	1.8	--
	1+600	--	--	1.5	--	1.2	--
Flood Load - USACE Method (Target FOS = 1.4)	0+900	--	--	2.2	--	2.2	--
	1+600	--	--	1.5	--	1.5	--
Rapid Drawdown (Target FOS = 1.2)	0+900	--	--	--	1.9	--	1.9
	1+600	--	--	--	1.6	--	1.6
Seismic - Pseudostatic (Target FOS = 1.0)	0+900	--	--	1.4	1.4	1.4	1.6
	1+600	--	--	1.0	1.3	1.0	1.9
Seismic - Post Earthquake (Target FOS = 1.2)	0+900	2.2	1.9	--	--	--	--
	1+600	1.5	1.6	--	--	--	--

The stability analyses showed adequate factors of safety for each load case. Detailed discussions of the analyses are included in Attachment 9.

9.6 SETTLEMENT ANALYSIS

Settlement analysis of the alluvium foundation was undertaken at 200 m spacings between Station 0+800 and 1+600 m. The total settlement due to embankment loading ranged from 11 mm at Station 0+800 to 24 mm at Station 1+600. The estimated settlement is presented below in Table 51.

Table 51. Total Estimated Foundation Soil Settlement Below Floodplain Berm

Station	Floodplain Berm Height (m)	Thickness of Alluvium (m)	Foundation Settlement (mm)
0+800	1.3	4.0	11
1+000	2.2	4.0	15
1+200	3.6	4.0	20
1+400	4.0	4.0	21
1+600	5.4	4.0	24

Most of the settlement estimated for the Floodplain Berm will occur during the embankment construction. This settlement will be made up with additional fill as the embankment reaches the crest.

The results of the analysis show that settlement following completion of the embankment construction is anticipated to be negligible along the centerline of the Floodplain Berm. No overbuild is recommended for the Floodplain Berm.

10.0 DIVERSION STRUCTURES PRELIMINARY DESIGN

10.1 DESCRIPTION

The diversion structures consist of the service spillway, auxiliary spillway, and debris deflection barrier within the Elbow River floodplain and the diversion inlet structure set into the left abutment of the river (Figure 83).

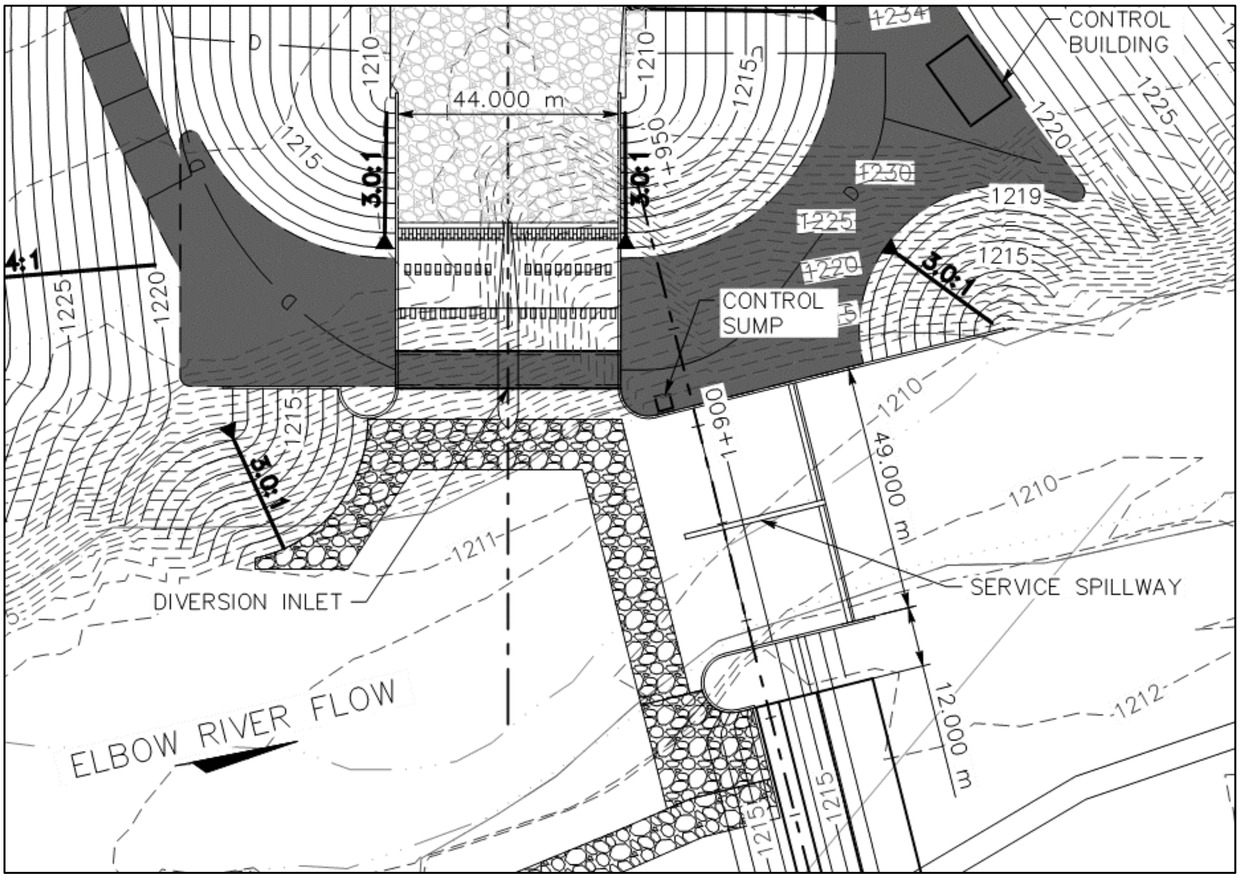


Figure 83. Diversion Structures Location

10.2 SITE SPECIFIC GROUND CONDITIONS

Ten boreholes were undertaken within the active river channel of the Elbow River at the location of the diversion structure and debris deflection barrier. The surficial units comprised 0.7 to 2.4 m thick layer of alluvium underlain by the BZF bedrock. The BZF comprised steeply-bedded, interbedded mudstones, siltstones and sandstones. The recovered boreholes indicated that the upper 1 to 3 m was highly weathered. The top of the BZF bedrock ranges between El. 1208.2 to 1210.1 m.

Three boreholes (DS6 to DS9) were undertaken on the crest of the river valley slopes. These locations were approx. 23 m above the active river channel. The surficial deposits comprised localized GL underlain by GT. The top of the BZF bedrock was higher than the active river channel and ranged between El. 1219.2 and 1221 m.

10.3 FOUNDATION RECOMMENDATIONS

10.3.1 Bearing Capacity Parameters

The foundations for the diversion spillway and inlets will be founded directly onto the BZF bedrock. The geological mapping and borehole descriptions (Section 5.7) indicate that the BZF rock mass is highly weathered and fractured. The argillaceous units (mudstones, carbonaceous shales, siltstones, clay-shales) were considerably weaker and less durable than the sandstone beds. This Preliminary Geotechnical Assessment has assumed that a relatively competent foundation subgrades ranges between El. 1205.6 m (DS9) and El. 1208.7 m (DS1). The foundation is likely to comprise predominantly up to 90% argillaceous units, whilst the sandstone beds are likely to comprise 10%.

The allowable bearing capacity of the BZF is summarized in Table 52. This is based on the UCS values obtained from the rock cores. Settlement of rock bearing structural elements was considered as negligible.

Table 52. Allowable Bearing Capacity of Intact Rock (without Cohesion) - Diversion Structures

Bedrock Type	Percent Bedrock Type Below Bearing	Typical Unconfined Compressive Strength MPa	Cohesion Mpa	Estimated Basic Friction Angle (phi)	Ultimate Bearing Capacity [1] kPa	Allowable Bearing Capacity FOS = 3.0 kPa
Shale	30	20.7	0	29	2,627	876
Mudstone	40	5.5	0	24	1,510	503
Claystone	20	17.2	0	24	1,510	503
Sandstone	10	24.1	0	32	3,668	1,220

[1] Derived from USACE EM 1110-1-2908, *Rock Foundations*, Equation 6-1, 1994.

The allowable bearing capacity for the composite bedrock is 620 kPa. The recommended bedrock friction angle is 26° with an interface cohesion of 0 kPa.

The Drained cross bed shear strength and cohesion parameters for the BZF were estimated using the Generalized Hoek and Brown Failure Criterion (1988) using rock mass data (Table 53).

Table 53. SR1 Diversion Structure – Cross Bed Shear Strength Parameters

Boring	Percent Bedrock Type Below Bearing	Typical Unconfined Compressive Strength MPa	Hoek-Brown Coefficients			Estimated Cross Bed Friction Angle	Estimated Cross Bed Cohesion (kPa)
			Mi Value	GSI Value	D Value		
Shale	30	34.1	6	35	0.5	39.2	153
Mudstone	40	1.9	4	30	0.5	12.0	34
Claystone	20	1.9	4	30	0.5	13.4	39
Sandstone	10	24.1	13	55	0.5	50.5	257

The recommended bedrock cross bed friction angle is 24 degrees, and the recommended cross bed cohesion is 90 kPa.

The recommended coefficient of sliding friction (μ) are 0.51 for the GT and 0.45 for the BZF.

The Modulus of Subgrade Reaction (k) was estimated from the from AFM TM 5-809-12 (1987). These are:

- GL: 27 MN/m^3
- GT: 34 MN/m^3
- Weathered BZF: 81.5 MN/m^3
- Competent BZF: 136 MN/m^3

10.3.2 Under Seepage

Seep/W analysis indicated that the flows within the BZF foundation is likely to be less than one l/s.

10.3.3 Foundation Treatment

Pressure grouting is recommended to reduce the permeability of the BZF immediately below the diversion structures to depth of 25 m. The preliminary grouting arrangement consists of a single row of pressure grouted rock core boreholes spaced approximately 1 m apart along the centerline of the diversion structure water control elements. Supplemental (secondary and tertiary) grouting boreholes will be recommended between boreholes where significant grout takes are observed in the primary grout holes.

10.3.4 Frost Depth

A determination of frost depth penetration for the diversion structures was estimated using the Modified Berggren Equation as outlined in the CFEM (2006). Regional climatic data for the regional Springbank weather station was obtained from the Canadian Climate Normal Station Data (1981 to 2010) website.

A frost depth of 1.96 meters was calculated for the Springbank region. A summary of climatic data and other parameters used in the frost depth calculations is presented in Table 54. Detailed calculations are included in Attachment 10.1.

Table 54. Parameters Used for Frost Depth Calculations

Frost Depth Parameter	Value
Mean Annual Air Temperature (MAAT)	3.1 °C
Average Annual Duration of Freezing Period	64 days
Average Ground Surface Freezing Index	1,046.9 °C days
50 Year Return Design Freezing Index[1]	1,700 °C days
Foundation Soil Dry Density	1,491 kg/m³
Thermal Conductivity of Fine Grained Soils (k_f)	95.0 KJ/day per m K
Calculated Frost Depth	1.96 m

10.4 RETAINING WALL LOADS

10.4.1 Recommended Design Parameters

To maximize re-use of available fills, it was assumed that the retaining walls adjacent to the service spillway and diversion inlet will be backfilled with GT from the diversion channel excavation. The parameters used for the analysis are:

- Saturated Unit Weight (Ysat) = 22.0 kN/m3
- Moist Unit Weight (Ymoist) = 20.0 kN/m3
- Effective Friction Angle (Φeff) = 27°
- Rankine At Rest Earth Pressure Coefficient (Ko) = 0.55
- Rankine Active Earth Pressure Coefficient (Ka) = 0.38
- Rankine Passive Earth Pressure Coefficient (Kp) = 2.66
- Permeability (kv) = 3.00 E -10 m/s

At-rest earth pressures have been used in the design of retaining walls to limit wall movements.

10.4.2 Seismic Wall Loading

The seismic loading for the retaining walls are based on procedures contained in USACE EM 1110-2-2100 (2005)-Stability Analysis of Concrete Structures, and USACE EM 1110-2-2502 (1989)-Retaining and Flood Walls.

For retaining walls that are able to yield laterally during an earthquake, the calculation of increased earth pressures induced by earthquakes can be approximated by the Mononobe-Okabe (M-O) pseudo-static approach. While this approach was originally intended for cohesionless backfill, recent studies (NCHRP, 2008) have shown that cohesion will decrease the seismic earth pressure. Therefore, using the M-O approach with the GT will provide a conservative estimate of seismic earth pressure.

As discussed in USACE EM 1110-2-2100 (2005), the horizontal seismic coefficient for evaluation of earthquake loading should be equal to 2/3 the effective PGA. The 2008 NCHRP report recommends a percentage of the PGA to be used for the horizontal seismic coefficient.

Based on the PSHA (Stantec, 2017), the free field PGA for the 1/10,000-year seismic event is 0.26g at the Diversion Structure. Accordingly, the recommended horizontal seismic coefficient (K_h) for the M-O analysis is as follows:

$$K_h = 2/3 \times 0.26g = 0.17g$$

10.5 DEBRIS DEFLECTION BARRIER FOUNDATION ANALYSES

Foundation analyses for the debris barrier supports were completed to size the foundation elements. The following analyses were conducted based on the superstructure and debris barrier design as of 2019 Preliminary Design. This debris barrier design consists of two foundation elements: Support A (caissons on river side) and Support B (spread footing on bluff side). The foundations for the debris barrier will be founded directly onto the Brazeau (BZF) bedrock. A cross section view of the debris barrier design is shown in Figure 84. All foundation analysis performed on the Debris Barrier were based on the assumption that no scour below the top of rock would occur. The top of rock surface will be protected by rip rap or other means.

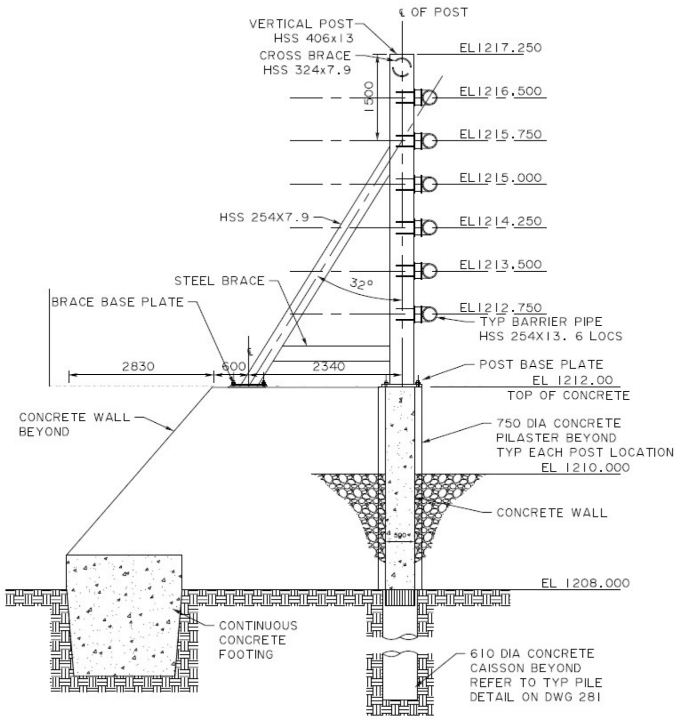


Figure 84 Typical Section of Debris Barrier and Foundation

10.5.1 Provided Load Conditions

To facilitate establishment of appropriate distribution of foundation loading between the respective elements lateral spring constants were utilized in the structural analysis. The following constants were provided:

Support A shafts (each shaft):	32,000 kN/m lateral	38,000 kN-m/radian rotation
Support B footings (2.5 m length):	70,000 kN/m lateral	Fixed for rotation

The Support A stiffnesses were developed based on preliminary runs of Lpile with the 600 mm shaft in the BZF bedrock. The Support B lateral stiffness was developed using a composite of an assumed passive wedge resistance and an estimated foundation shear stiffness in the soft bedrock. The Support B footing was assumed to be fixed against rotation, confirmed in the analysis by the resultant location within the kern.

The appropriate load amounts for the debris barrier foundation analyses were then developed using the structural analyses program SAP 2000 including structural elements (i.e. steel superstructure, concrete support structure, etc.) above the anticipated bedrock surface. The load conditions provided included factored loads, service loads, and dead loads of the debris barrier.

10.5.2 Bearing Materials

As discussed in Section 5.7, the geological mapping and borehole descriptions indicate that the BZF rock mass is highly weathered and fractured. The argillaceous units (mudstones, carbonaceous shales, siltstones, clay-shales) were considerably weaker and less durable than the sandstone beds. This Preliminary Geotechnical Assessment has assumed that a relatively-competent foundation subgrades ranges between El. 1205.6 m (DS9) and El. 1208.7 m (DS1). The foundation is likely to comprise predominantly up to 90% argillaceous units, whilst the sandstone beds are likely to comprise 10%.

The allowable bearing capacity of the BZF is summarized in Table 55. This is based on the UCS values obtained from laboratory tests performed on the rock core samples. Refer to Section 5 for additional information on BZF rock core testing.

Table 55 Allowable Bearing Capacity of Intact Rock (without Cohesion) - Diversion Structures

Bedrock Type	Percent Bedrock Type Below Bearing	Typical Unconfined Compressive Strength MPa	Cohesion Mpa	Estimated Basic Friction Angle (phi)	Ultimate Bearing Capacity [1] kPa	Allowable Bearing Capacity FOS = 3.0 kPa
Shale	30	20.7	0	29	2,627	876
Mudstone	40	5.5	0	24	1,510	503
Claystone	20	17.2	0	24	1,510	503
Sandstone	10	24.1	0	32	3,668	1,220

[1] Derived from USACE EM 1110-1-2908, *Rock Foundations*, Equation 6-1, 1994.

10.5.3 Drilled Shaft Analyses

Drilled shaft analyses were conducted for the Support A foundation elements. The Support A drilled shafts were assessed in the SAP 2000 program with 2.5 meters center to center spacing. The analyses, including the complete set of parameters and assumptions used are presented in Attachment 10. The following describes the analysis process used and provides a summary of the results. Based on the analyses conducted, it is anticipated that the provided downward loading will be the controlling load case.

10.5.3.1 Side Friction Capacity

The estimated side friction resistance in rock was calculated using the uniaxial compressive strength of the rock as presented in the United States Federal Highway Administration Drilled Shaft Manual (FHWA, 2018). The core samples subjected to unconfined compressive strength tests were collected during the Stantec April2018 geotechnical exploration. The uniaxial compressive strength used to calculate the side friction was the typical unconfined compressive strength from the Mudstone presented in Table 55. Additional reduction factors were applied to the calculation to compensate for the potentially caving rock.

10.5.3.2 Point Bearing Capacity

The estimated ultimate point resistance in rock was calculated using the Goodman, 1980 equation. The Goodman (1980) equation utilizes the unconfined compressive strength and drained friction angle to estimate the point bearing capacity. Using the Goodman (1980) equation, the estimated point bearing capacity was calculated to be approximately 4,300 kPa and is consistent with values used locally for this material.

10.5.3.3 Lateral Capacity

The lateral capacity of the drilled shaft was assessed using the Ensoft, Inc. computer program Lpile. The lateral capacity and deflection of the drilled shaft was assessed using factored, and service loads, respectively. The results of the Lpile analyses indicate that the lateral deformation of the drilled shaft foundation will be negligible under the provided lateral service loads with the deflections less than 1.0 mm. The printouts from the Lpile program are included in Attachment 10.

10.5.3.4 Adfreeze Calculations

Based on the frost depth of approximately two (2) meters and large exposed perimeter area of the debris barrier due to the continuous web walls, substantial uplift forces on the foundation elements due to adfreeze are possible. A simple adfreeze calculation was performed for drilled shaft foundation elements due to concerns for pile ratcheting.

The adfreeze calculation used the approximate perimeter area of the debris barrier concrete supports and the estimated frost developed presented in the 2017 Preliminary Geotechnical Assessment Report. The anticipated adfreeze force per perimeter area was determined from the guidance in Canadian Foundation engineering manual (CFEM, 2006) which states that the average adfreeze bond stress is approximately 65 kPa for fine-grained soils frozen in contact with concrete.

Using the sizing of the drilled shaft foundation elements from the bearing capacity analyses, the estimated adfreeze force is calculated to be resisted and the adfreeze loading is not anticipated to be the controlling load case.

10.5.3.5 Uplift Capacity

The load cases provided and assessed were for downward loading conditions only. The adfreeze condition produced uplift on the shafts, but the shaft lengths developed for the downward loading were sufficient to resist the adfreeze loading. If uplift loading from other conditions is anticipated, additional analyses may be required.

10.5.4 Continuous Spread Footing Analyses

Continuous footing analyses were conducted for the Support B foundation element. The Support B continuous strip footing was assessed with a tributary area corresponding to the 2.5 meters center to center spacing. The analyses including the complete set of parameters and assumptions used are presented in Attachment 10. The following is a brief description of the analyses conducted.

10.5.4.1 Bearing Capacity

The bearing capacity analyses for the continuous spread footing was completed using a traditional spread footing analyses and the recommended bearing capacities in Table XX. The spread footing analyses was conducted for a 1 meter by 1 meter foundation element. This foundation size was chosen to accommodate for the uncertainties of the bedrock formation near the soil/rock interface along the alignment.

Based on our analyses, a 1 meter wide by 1 meter deep spread will be sufficient for the loading conditions provided.

10.5.4.2 Sliding Resistance

Sliding resistance of the continuous footing was evaluated for the load conditions provided. The sliding resistance is resisted by the passive earth pressures acting on the side of the retaining wall as well as the base friction between the bedrock and concrete interface. The values used for the coefficient of sliding was presented in the internal Geotechnical Parameters Memo (9/2019).

Based on the sliding resistance calculations conducted, a 1 meter by 1 meter continuous spread footing will be sufficient to resist the provided lateral forces.

10.5.4.3 Adfreeze Calculations

It has been assumed that the continuous footing bearing on rock will not be subjected to this ratcheting effect due to the size and shape of the foundation element.

11.0 DIVERSION CHANNEL PRELIMINARY DESIGN

11.1 DESCRIPTION

The diversion channel is approximately 4.7 km long with a 24 m bottom width and 3H:1V soil side slopes and 2H:1V rock side slopes. The diversion channel will be constructed predominantly by excavation with two (2) zones of side-long topography requiring an embankment section on the eastern side. Typical configurations are provided in Figure 85.

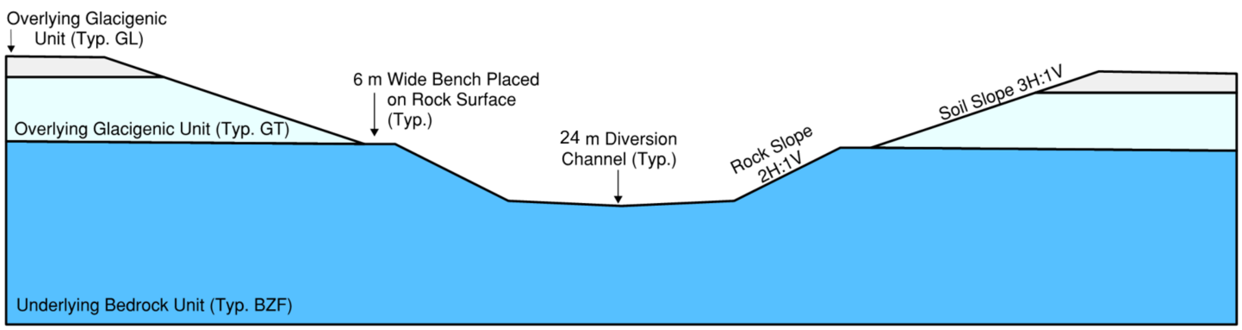


Figure 85. Typical Diversion Channel Sections

11.2 SITE SPECIFIC GROUND CONDITIONS

11.2.1 Generalized Ground Conditions

The anticipated ground conditions are presented in Figure 20-8. These are likely to comprise:

- Station 10+000 to 11+100 m: excavation within BZF and overlying GT and GL units
- Station 11+100 to 12+100 m: excavation within GT and GL units
- Station 12+100 to 13+400 m: excavation within BZF and overlying GT and GL units
- Station 13+400 to 14+570 m: excavation within CSF and overlying GT and GL units

11.2.2 Groundwater

Existing groundwater conditions were determined by reviewing piezometer data and CPT data. Additionally, groundwater elevations encountered in boreholes during drilling were reviewed. Piezometers were installed at six (6) locations along the Diversion Channel, with two (2) of these locations having nested piezometers with one in the soil overburden and one in the bedrock. These piezometers have been monitored from initial drilling in summer 2016 through spring 2017. During this time period only minor level changes were observed. Of the six (6) piezometers installed in the soil overburden, four (4) have depth to water of less than 4.3 m, with an average depth to water of 3.3 m for these instruments. The other two (2), in DC6 and DC7A at approximate Station 10+500 m have an average depth to water of 14.5 m. CPT DC26 indicated depth to groundwater of approximately 2.5 meters at the time of drilling.

For the slope stability analysis at Stations 10+150 m, 10+400 m, and 11+000 m, the assumed groundwater elevation was taken as 15 meters below the crest of the channel based on the piezometer readings in DC6 and DC7A. For the other sections analyzed, the groundwater elevation was assumed to be three (3) meters below the crest based on the other five piezometers installed along the channel.

The effects of constructing the Diversion Channel on the local groundwater regime are difficult to estimate. To gain a better understanding, seepage analyses were conducted using the finite element modelling software, Seep/W (part of the Geostudio 2016® suite). This evaluation was performed in the soil overburden. While the bedrock is believed to have significant permeability through joint sets and beds, the highly variable nature of their orientations makes modeling unmeaningful. Accordingly, the top of rock was assumed to be an impervious boundary. The analyses used an assumed boundary condition for the up-gradient direction. However, without a permanent source such as a pool behind a dam, the appropriate upstream boundary condition is difficult to determine. Based on the results of these seepage analyses, using a total head boundary condition of approximately 3 meters below the ground surface 150 meters from the crest of the channel, the phreatic surface typically daylighted approximately one-third up the soil slope face. Based on this analysis, the assumed phreatic surface through the soil for locations downstream of Station 11+000 m was assumed to occur near the slope surface at the one-third point up from the channel bottom or top of bedrock, if present.

11.2.3 Brazeau Formation Strength Models

The BZF underlies the Diversion Channel between Station 10+000 and 13+200 m. A simplified Hoek-Brown approach was selected to evaluate the stability of the slopes to be excavated in bedrock. The BZF was modelled in the GeoStudio program SLOPE/W using a shear stress / normal stress function. The function for the BZF were estimated using the Generalized Hoek-Brown Criterion (Table 56).

Table 56. Input Parameters for Generalized Hoek-Brown Criterion

Unit	Unconfined Compressive Strength (MPa)	Geological Strength Index (GSI)	Intact Rock Parameter (m_i)	Disturbance Factor (D)
BZF	3.0	35	6	0.0

11.3 STABILITY ANALYSIS

11.3.1 Profiles

This assessment modeled seven (7) profiles perpendicular to the Diversion Channel alignment:

- Station 10+150 m – this represents the channel inlet from the Elbow River downstream of the diversion structure with a benched profile to account for the proposed access roads on either side of the channel;
- Station 10+400 m – this represents a 15 m deep excavation into bedrock with 20 m of overlying glacigenic units;
- Station 11+000 m – this represents the utility crossing for the Nova Chemicals and Pengrowth pipelines and the AltaLink overhead transmission line;
- Station 11+400 m – this represents a 14 m deep excavation entirely in soil with soil beneath the flowline of the Diversion Channel;
- Station 11+900 m – this represents the utility crossing at the TransCanada Pipelines location;
- Station 12+400 m – this represents a 10 m deep excavation in soil and bedrock with 7 m tall soil slopes, and bedrock occurring at the flowline of the Diversion Channel;
- Station 14+000 m – this represents an excavation predominantly into bedrock with a small amount of soil overburden (less than 5 m).

11.3.2 Minimum Failure Depth

The factor of safety presented in the analyses below are based on analysis defined minimum 2 m deep failure surface. Shallower failure surfaces are possible and will result in a reduced factor of safety. An infinite slope analysis results in a 1.3 factor of safety within the soil slopes.

It is Stantec's opinion that a failure surface 2 m deep or less reflects a maintenance type failure. These maintenance type failures require repairs but do not, in general, place operation of the facility at risk.

11.3.3 Groundwater Control in Stability Analyses

When the factor of safety for a profile was below 1.5 for assumed groundwater conditions as discussed in Section 11.2.2., groundwater control measures were modeled to determine if adequate factors of safety could be met. The groundwater control was modeled by drawing the piezometric line to a node 12 meters from the toe of the diversion channel or along the bedrock/soil interface (if the toe of the diversion channel is in bedrock), as shown below in Figure 86. The potential groundwater control techniques to achieve this piezometric line drawdown are discussed in Section 11.5.3.

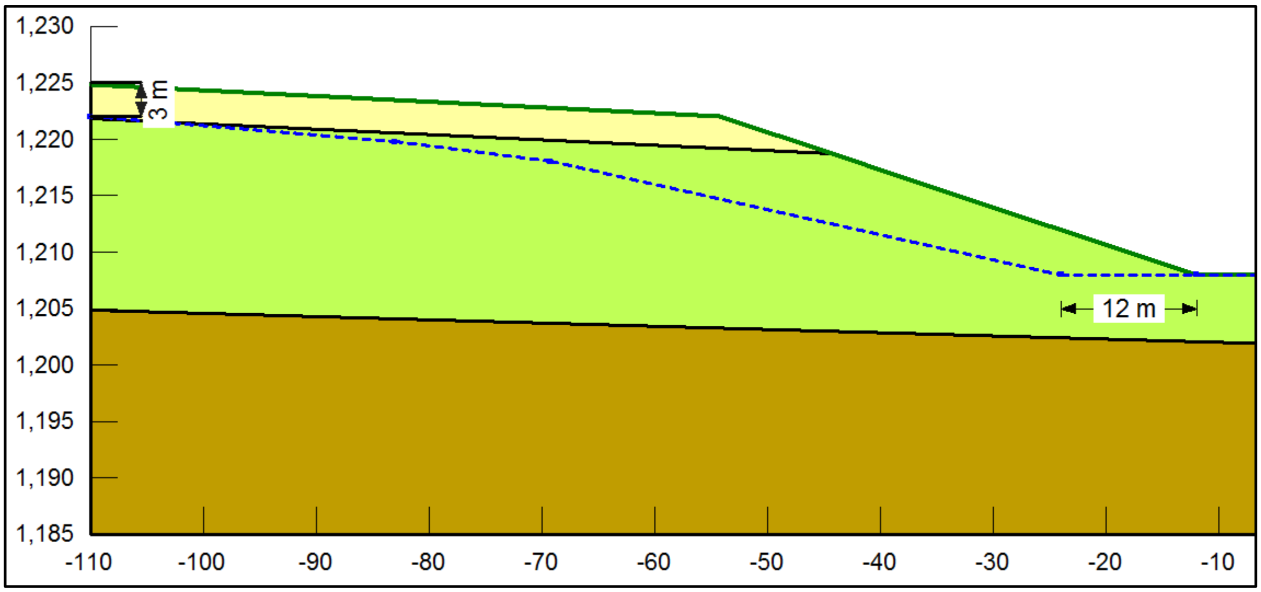


Figure 86. Generalized Groundwater Control Measure

11.3.4 Slope Stability Results

Slope stability results for each cross section are included in the sections below. Plots of SLOPE/W results are included in Attachment 11.1. A reference number is provided for each analysis included in the slope stability plots.

11.3.4.1 Station 10+150 m Analysis

This model analyzed the updated entrance near the diversion structure and the new benched structure due to the access road. The results are summarized below in Table 57. The section is displayed in Figure 87.

Table 57. Summary of Results for Station 10+150 m

Ref:	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 15 m below ground level - No control of groundwater	Rotational failure through the glacigenic (GL and GT) units	1.6
2	Measured/Inferred - Approximately 15 m below ground level - No control of groundwater	Rotational failure through the BZF Formation	2.1

Based on piezometer data, groundwater is approximately 15 m deep in the overlying glacigenic units. The analyses show that the desired FOS is achieved without groundwater control when the soil slopes excavated to 3H:1V with one bench and the rock slope is excavated to 2H:1V.

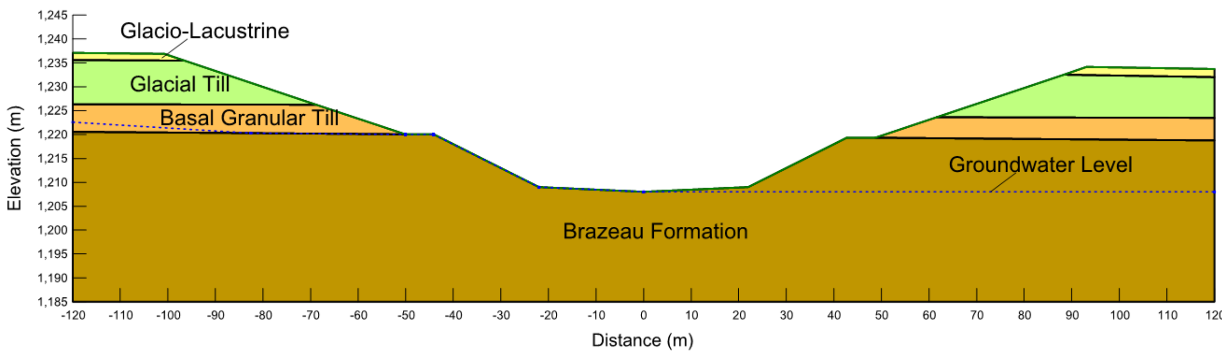


Figure 87. Station 10+150 Section

11.3.4.2 Station 10+400 m Analysis

The model at Station 10+400 analyzed the deepest excavation within the Diversion Channel, with nearly equal depths of soil and rock. The results are summarized below in Table 58. The section is displayed in Figure 88.

Table 58. Summary of Results for Station 10+400 m

Ref	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 15 m below ground level - No control of groundwater	Rotational failure through the glacigenic (GL and GT) units.	1.5
2	Measured/Inferred - Approximately 15 m below ground level - No control of groundwater	Rotational failure through the BZF Formation	1.7

The analyses show that the desired FOS is achieved without groundwater control when the soil slopes excavated to 3H:1V and the rock slope is excavated to 2H:1V.

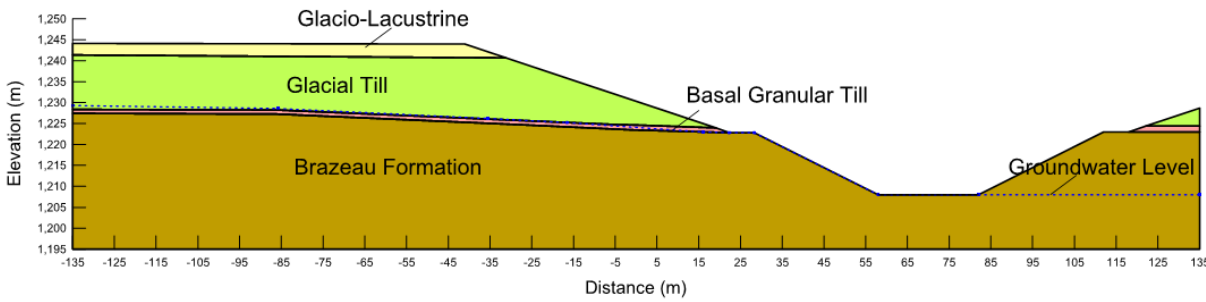


Figure 88. Station 10+400 Section

11.3.4.3 Station 11+000 m Analysis

This model analyzed an excavation within the glacigenic units at the location of two (2) pipelines and one (1) overhead transmission line crossing of the Diversion Channel. The results are summarized below in Table 59. The section is displayed in Figure 89.

Table 59. Summary of Results for Station 11+000 m

Ref	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 15 m below ground level - No control of GW	Rotational failure through the glacigenic (GL and GT) units.	1.5
2	Measured/Inferred - Approximately 15 m below ground level - No control of GW	Failure forced into BZF Formation Rotational failure through the glacigenic (GL and GT) units	2.1

Based on piezometer data, groundwater is approximately 15 m deep in the overlying glacigenic units. The analyses show that the desired FOS is achieved with groundwater control when the soil slopes and the rock slopes are excavated to 3H:1V and 2H:1V, respectively.

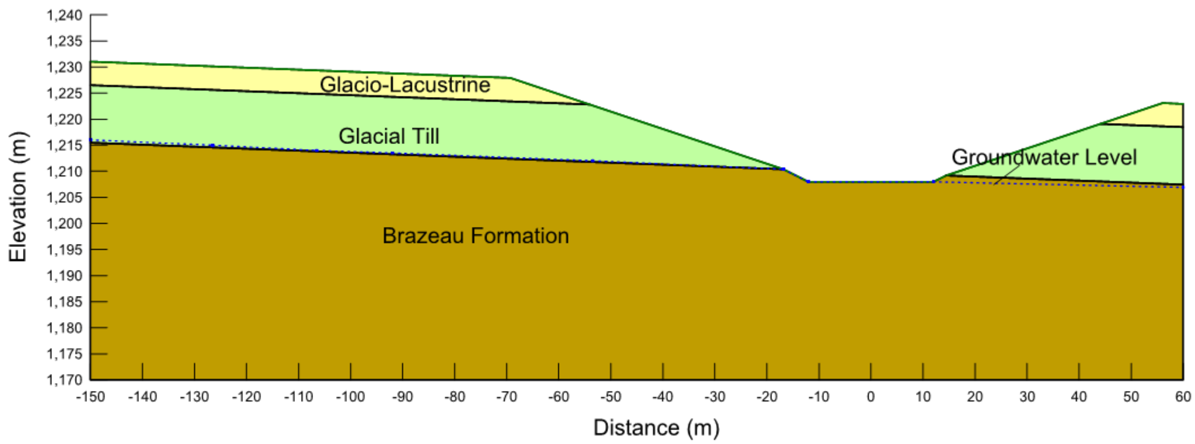


Figure 89. Sta. 11+000 Section

11.3.4.4 Station 11+400 m Analysis

This model analyzed an excavation within the glacigenic units. The results are summarized below in Table 60. The section is displayed in Figure 90.

Table 60. Summary of Results for Station 11+400 m

Ref	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 3 m below ground level - No control of GW	Rotational failure through the glacigenic (GT) unit	0.8
2	Measured/Inferred - Approximately 3 m below ground level - GW control before toe of soil slope	Deep rotational failure through the glacigenic (GL and GT) units	1.5

Groundwater control is necessary to achieve a factor of safety of 1.5. The groundwater control measure would need to lower the phreatic surface 12 m from the toe of the diversion channel and 3 m below the ground surface. Potential groundwater control measures are discussed in Section 11.5.3.

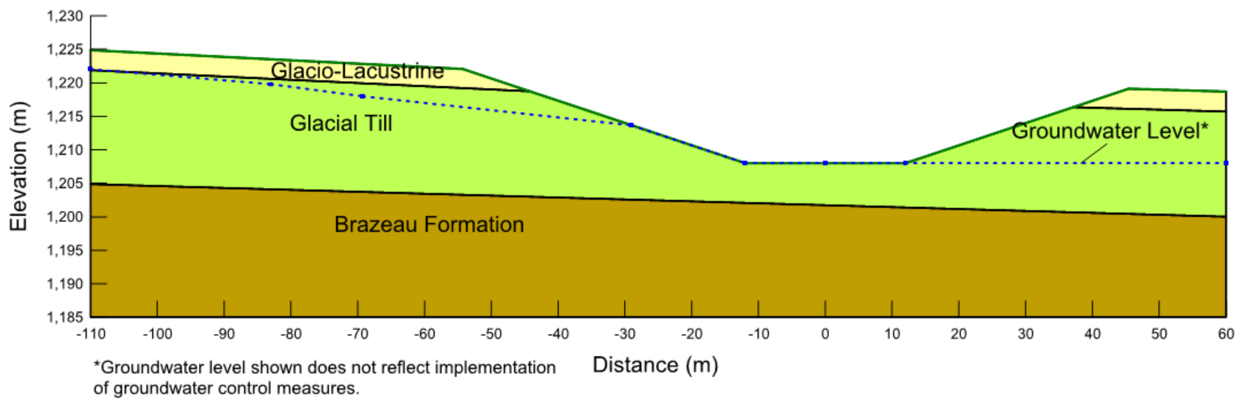


Figure 90. Station 11+400 Section

11.3.4.5 Station 11+900 m Analysis

This model analyzed an excavation within the glacigenic units at the location of two (2) pipeline crossings of the Diversion Channel. The results are summarized below in Table 61. The section is displayed in Figure 91.

Table 61. Summary of Results for Station 11+900 m

Ref	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 3 m below surface - No Control of GWr	Rotational failure through the glacigenic (GL and GT) units.	0.9
2	Measured/Inferred - Approximately 3 m below surface - GW control before toe of slope	Rotational failure through the glacigenic (GL and GT) units.	1.5

Groundwater control is necessary to achieve a factor of safety of 1.5. The groundwater control measure would need to lower the phreatic surface 12 m from the toe of the Diversion Channel and 3 m below the ground surface. Potential groundwater control measures are discussed in Section 11.5.3.

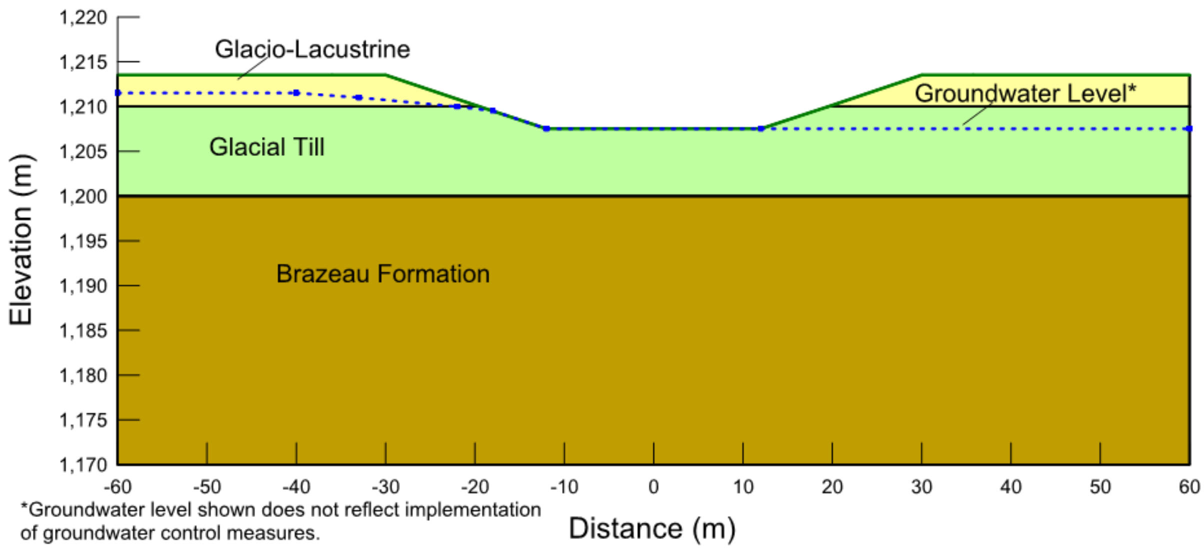


Figure 91. Station 11+900 Section

11.3.4.6 Station 12+400 m Analysis

This model analyzed an excavation within the glacigenic units and bedrock in the lower slope of the excavation and under the flowline of the channel. This location is also near the Diversion Channel crossing at Highway 22. Slope stability analyses used the phreatic conditions from the seepage analyses to calculate the factor of safety. For this section, the critical stability drivers are the thickness of GL unit and minimum failure depth. The results are summarized below in Table 62. The section is displayed in Figure 92.

Table 62. Summary of Results for Station 12+400 m

Ref	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 3 m below surface - No Control of Water	Rotational failure through the glacigenic (GL and GT) units.	1.1
2	Measured/Inferred - Approximately 3 m below surface - GW control before toe of soil slope	Rotational failure through the glacigenic (GL and GT) units.	1.5
3	Measured/Inferred - Approximately 3 m below surface - No Control of Water	Failure forced into BZF Formation Rotational failure through the glacigenic (GL and GT) units	1.7

Groundwater control is necessary to achieve a factor of safety of 1.5. The groundwater control measure would need to lower the phreatic surface 12 m from the toe of the diversion channel and 3 m below the ground surface. Potential groundwater control measures are discussed in Section 11.5.3.

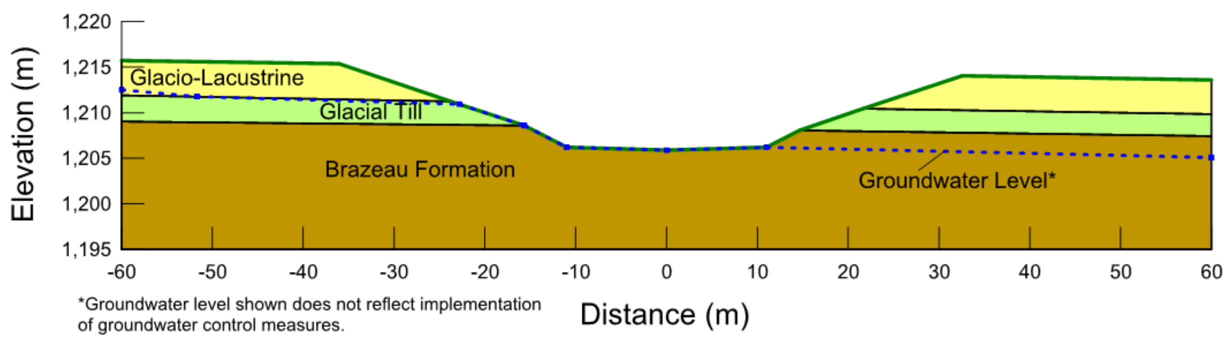


Figure 92. Station 12+400 Section

11.3.4.7 Station 14+000 m Analysis

The model analyzed at Station 14+000 m requires an excavation nearly entirely in the CSF bedrock. Groundwater was assumed to generally follow the soil/rock interface. It was assumed that the entire rock mass was saturated. The results are summarized below in Table 63. The section is displayed in Figure 93.

Table 63. Summary of Results for Station 14+000 m

Ref	Groundwater Conditions	Failure Mechanism	FOS
1	Measured/Inferred - Approximately 3 m below surface - No Control of Water	Rotational failure through the glacigenic (GT) unit above the CSF unit.	1.7
2	Measured/Inferred - Approximately 3 m below surface - No Control of Water	Rotational failure through the BZF Formation	1.8

With the rock mass modeled using the Hoek-Brown strength parameters, adequate FOS was achieved with 2H:1V rock side slopes. Note the bedrock strengths developed for the BZF were used in this analysis. While the rock here may be of slightly better quality than the Brazeau rock evaluated, it consists of similar subcomponents and it was judged reasonable to use similar strength parameters.

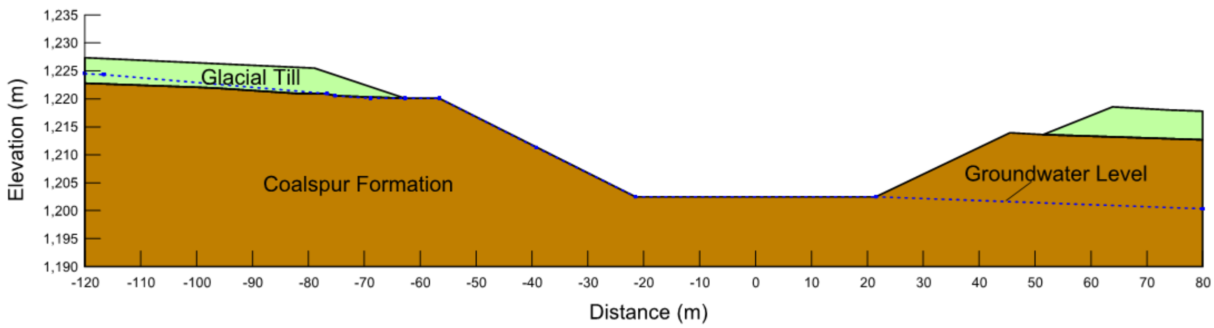


Figure 93. Sta. 14+000 Section

11.4 ROCK MASS EVALUATION

The geological mapping, borehole data and subsequent slope stability analysis have indicated that the BZF may be problematic for slope stability. Factors of safety for failures through the rock mass are greater than 1.5 in all cases; however, slopes steeper than the 2H:1V currently proposed are not recommended due to the increased potential for kinematic block and joint failures, with erosion of the mudstone layers and failure of sandstone blocks.

11.5 GROUNDWATER CONTROL

Groundwater control will be required in certain locations. It is difficult to predict the effect of the Diversion Channel excavation on the local groundwater regime, especially given the irregular nature of the BZF bedding and jointing and how the bedrock and soil groundwater regimes interact with each other. Based on the analyses conducted, locations represented by Station 11+400 m, where the soil slopes are relatively tall with soil beneath the flowline of the channel are likely to require groundwater control. This is most likely to occur from Station 11+100 to 12+100 m; however, it may not be necessary at all locations. Groundwater control may also be required at other locations along the channel. The nature of the groundwater control measure will be driven by the amount and persistence of water encountered at various locations.

It is recommended that a series of piezometers be installed near the proposed upstream crest of the Diversion Channel. These piezometers will be used to monitor the effects of the Diversion Channel excavation on the groundwater level. Assumptions from the seepage analyses can then be verified during construction and groundwater control can be implemented in a revised and more efficient manner.

Groundwater conditions can change with time and should be monitored as part of the facility operation and maintenance program. If groundwater levels rise, risk of sloughing and slope failures increase. Proper monitoring should allow observation of conditions requiring mitigation, if any, prior to sloughs or slope failures developing.

11.5.1 Localized Inflow within the Glacigenic Units

Glacigenic units can contain water-bearing pockets, lenses and layers of granular materials not identified by boreholes. These can affect the slope performance and maintenance of the Diversion Channel. The excavation will be monitored during construction and localized zones of inflow will require site-specific treatment depending on the flow volume and extent.

11.5.2 Artesian Groundwater

The artesian pressures encountered in the BZF between Stations 10+000 and 10+600 m will require control during excavation. It is currently unknown if this will dissipate or will be seasonally or continuously recharged. The excavation will be monitored during construction and site-specific treatment will be installed depending on the flow volume and extent.

11.5.3 Groundwater Control Measures

At cross sections where a factor of safety was below the design target of 1.5, groundwater control measures were modeled to lower the phreatic surface 12 m from the toe of the diversion channel and 3 m below the excavated channel side slope ground surface. This groundwater control measure was successful in increasing the factor of safety to the required value at all sections that did not meet the initial FOS of 1.5. It is Stantec's experience that these slope conditions, without groundwater control, can result in sloughing and bulging; and significant slope failures would not be uncommon.

Groundwater control may consist of a toe buttress where the toe of the slope is excavated and replaced with free-draining crushed stone. The excavation would need to be at least 12 meters into the slope from the channel bottom or bottom of soil based on the stability analyses. Another potential groundwater control measure would be a vertical drain with outlet pipes spaced sufficiently to allow discharge of the groundwater flow. The drain would also need to be located 12 meters into the slope from the channel bottom or bottom of soil.

The necessary groundwater control will be selected based on actual field conditions when excavation is occurring. Piezometer readings and site observations will be utilized to determine how the existing groundwater levels react to the excavation of the Diversion Channel.

12.0 STORAGE DAM PRELIMINARY GEOTECHNICAL DESIGN

12.1 GENERAL FACILITY DESCRIPTION

The Off-stream Storage Dam (Dam) will be a 3.7 km long, zoned earthfill structure constructed across the existing valley and Unnamed Creek. The maximum height of the Dam is 29 m with a crest elevation of 1213.5 m. It comprises the following components:

- Impervious core constructed from Impervious 1A fill comprised of GT units. This will be constructed with 1V:1H slopes on the downstream side and 1V:1.5H on the upstream side. The top of the core is El. 1212.5 m.
- 1 m thick filter constructed from Fine Filter 3A fill adjacent to the downstream face of the core. This will extend along the top of the prepared original ground to the downstream toe.
- Upstream and downstream embankment shells constructed from Random 2A fill. The downstream and upstream shell will comprise 3.5H:1V side slopes with 10 m wide benches spaced 10 m vertically.
- Rock toe buttresses on upstream and downstream toes in the location of the thickest foundation soils (Sta. 21+800 to Sta. 22+800).
- Vertical downstream toe drain.

Typical sections of the Dam without and with the rock toe zones are presented in Figure 94 and Figure 95 respectively.

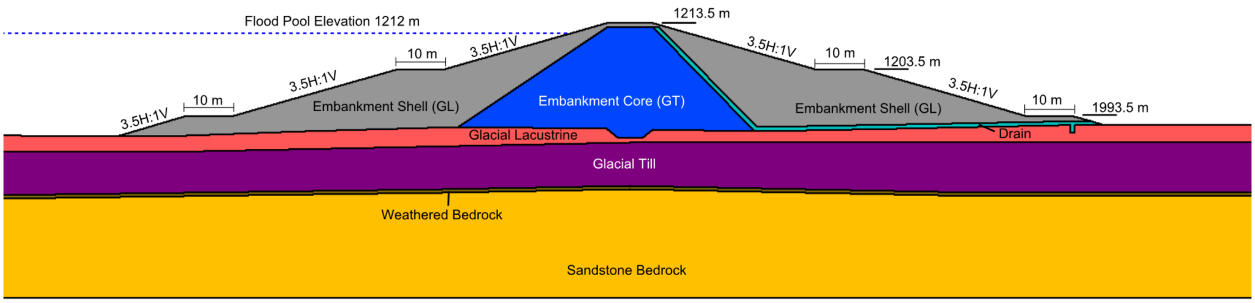


Figure 94 Storage Dam Typical Section

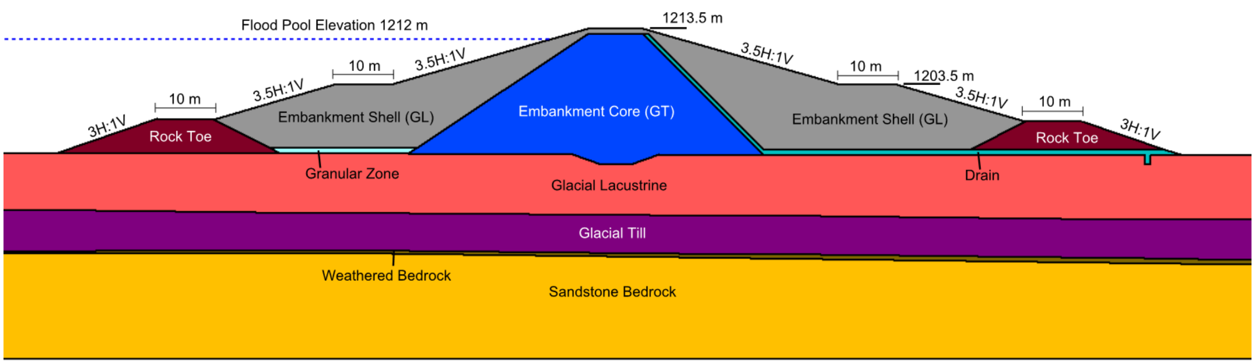


Figure 95 Storage Dam Typical Section, with rock toes

12.2 OVERVIEW OF GEOLOGY AND SOIL CONDITIONS

The Dam is located within a broad valley that has been infilled with glacigenic units on top of the PPF bedrock. The glacigenic units comprise variable thicknesses and extents of Glacial Lacustrine clay and Glacial clay Till (UBT and LGST). The overall thickness of the glacigenic units varies across the valley. It is 3.7 m thick at the western abutment (D3) and 19.8 m thick between Sta 22+600 and 22+900 m (D30 and D60). The thickness of the glacigenic units increases further towards the Elbow River, with a maximum thickness of 25.9 m encountered in D27. Descriptions of the site soils are covered in detail in Section 5.4.

12.3 DESIGN CRITERIA AND LOADING CONDITIONS

Design guidance is discussed in Section 8.3. The Canadian Dam Association *Dam Safety Guidelines* were used as the primary criteria with supplemental input from various other Canadian and U.S. agencies.

The selected Loading Conditions and associated criteria for slope stability Factor of Safety are presented in Table 64 below.

Table 64. Recommended Design Load Cases for Off-Stream Storage Dam

Load Case	Reference	Reservoir	Foundation Behavior	Pore Pressures	FOS
End of Construction	CDA	None	Undrained strength parameters;	Phreatic surface in foundation	1.3
End construction – multi-year construction	CDA	None	Undrained strength ratio (c/p) in the GL		1.3
	CDA, PFRA	None	Drained strength parameters	Phreatic surface modelled in the foundation plus pore pressure from B-bar or FEM	1.3
Not operational - long Term	CDA	None	Drained strength parameters	Phreatic surface in foundation	1.5
Operation - Design Flood	USBR	IDF	Drained strength parameters	Steady state seepage in embankment dam;	1.2
	USCAE	IDF	Undrained strength parameters	Flood pool modelled as a surcharge; phreatic surface in foundation	1.4
Rapid Drawdown	CDA	IDF	Undrained strength parameters	Multi-stage phreatic surface from reservoir	1.2
Seismic – Pseudostatic	CDA	IDF	80% of Undrained strength parameters	Flood pool as surcharge; phreatic level in foundation	1.0*
Seismic – Post Earthquake	CDA	IDF	80% of Undrained strength parameters	Flood pool as surcharge; phreatic level in foundation	1.2

*Not a design criteria. For FOS less than 1.0 seismic deformation analysis performed.

12.4 SEEPAGE ANALYSIS

12.4.1 Boundary Conditions

The Dam will function as a dry dam with no pool for normal conditions. However, Seep/W analysis was undertaken to determine the steady-state phreatic surface at the IDF for the USBR flood pool analysis.

The Seep/W analysis was undertaken to determine the steady-state phreatic surface at the IDF pool elevation of 1212 m. Potential seepage face boundary conditions were applied to the downstream face. To reduce edge-boundary effects, seepage model extents were expanded at least 100 m beyond the downstream toe of the dam slope.

12.4.2 Piping Factor of Safety Results

Assuming steady state conditions, the critical exit gradients at the toe of the storage dam were assessed for the IDF pool level (el. 1212). These are presented in Table 65. Plots from SEEP/W presenting the results of the seepage analyses and the exit gradient calculations are included in Attachment 12.1.

Table 65. Factors of Safety against Piping due to Heave

Cross Section	Maximum Exit Gradient	Factor of Safety Against Piping Due to Heave
20+000 (saddle dam)	0.273	3.8
21+050	0.300	3.5
21+750	0.347	3.0
22+500	0.333	3.1
22+990	0.333	3.1
23+175 (no treatment)	3.714	0.3
23+175 (with treatment)[1]	0.143	7.9

1. Analyzed seepage treatments discussed in the following section.

The analysis indicates that under full IDF pool steady-state conditions, adequate FOS are likely for piping due to heave.

12.4.3 Seepage Control within the Unnamed Creek

The geotechnical investigation indicated that the Unnamed Creek is an undersized river valley infilled with fluvial materials (sands and gravels) overlain by glacial till. The fluvial materials are consistently present in borings and test pits performed in the Unnamed Creek. The hydraulic conductivity of the fluvial materials is relatively high. It is possible that hydraulic conductivity may exist between the fluvial materials and the reservoir, which could result in unacceptable factors of safety again piping. To mitigate against this, seepage control measures were evaluated.

12.4.3.1 Potential Design Solutions

Data from the geotechnical investigation near the creek show that the fluvial materials located in this area are typically overlain by a low permeability glacial till layer. However, it is plausible that the fluvial materials extend to the surface at some locations, which could result in significant seepage flows through the Dam foundation. In these seepage analyses, the models were modified to represent a direct hydraulic connection of the reservoir to the fluvial materials. The glacial till material was removed from the model within the pool area, and the entire foundation zone in this region was modeled as fluvial materials. The following seepage control measures were considered:

- Option 1: 2 m thick upstream seepage blanket.
- Option 2: Cut-off key trench extended into the PPF bedrock. This would replace the alluvium with low-permeability engineered clay.
- Option 3: Vertical drains installed into the alluvium to intercept underflows. This is connected to a horizontal drainage blanket.
- Option 4: Pressure Relief System consisting of wells or a trench drain extended through the clay to the alluvium at the downstream toe.

Options 1 and 2 were considered potentially effective but uneconomical if implemented to the full extent required to reduce risk of high toe exit gradients to an acceptable level. Seep/W analysis was undertaken for Options 3 and 4.

12.4.3.2 Analysis Methodology

The cross section at Station 23+175 was modified to model a direct connection from the reservoir to the fluvial gravels. This connection was modeled at the upstream toe. The effects of the downstream vertical drain and relief wells were then evaluated.

The proposed section for the dam includes a 3-meter deep vertical toe drain 6 meters from the downstream toe. In the unnamed creek, the data from the geotechnical exploration shows the top of the fluvial materials ranges in depth below existing ground surface from 7.3 meters (D46) to 3.7 meters (D45).

The proposed design is shown in Figure 96. The vertical toe drain was extended one meter into the fluvial materials in order to provide a seepage path connection from the fluvial gravels to the horizontal blanket drain. Also, the vertical drain was moved towards the center of the dam, and was modeled one meter from the inflection of the chimney drain and blanket drain on the downstream face of the embankment core.

The relief well option was modeled by placing a total head boundary condition in the fluvial material just past the downstream toe of the dam, using a ground surface elevation of 1183.2 m as the fixed head. The seepage model is shown in Figure 96.

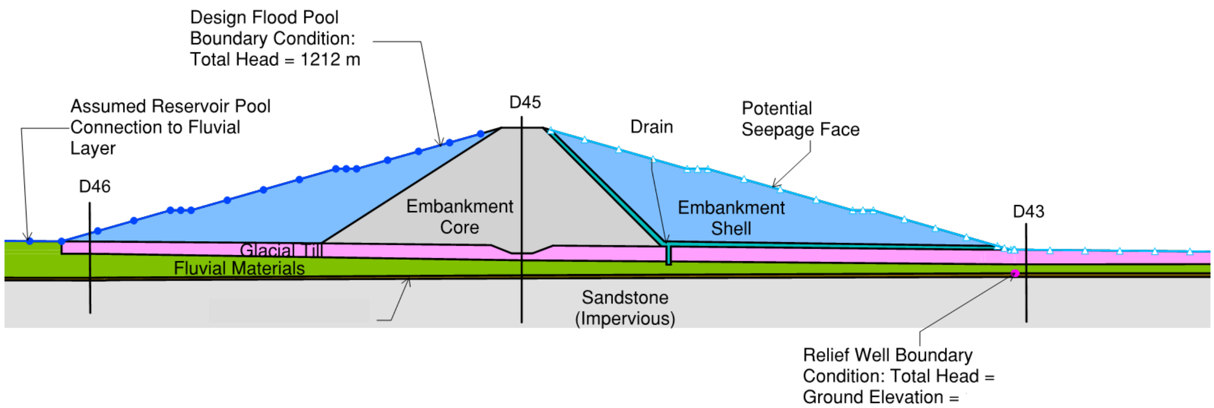


Figure 96. Unnamed Creek Foundation Treatment Seepage Model

12.4.3.3 Results

The seepage regime was analyzed for two scenarios. The first scenario included no seepage treatment. The second scenario included a drain extending into the fluvial materials, and a relief well at the downstream toe.

The first scenario where no seepage treatments were modeled resulted in vertical exit gradients of 3.7 at the downstream toe of the dam, resulting in an unacceptable piping factor of safety of 0.3. The second scenario, with a drain extending into the fluvial materials, and a relief well at the downstream toe, resulted in vertical exit gradients of 0.14 and a piping factor of safety of 7.9.

12.4.3.4 Conclusions and Recommendations

The analysis shows that the combination of a vertical drain extending into the fluvial materials and relief wells at the downstream toe will provide acceptable factors of safety against piping at the downstream toe near the Unnamed Creek. A series of individual relief wells installed as seepage treatments may reduce vertical exit gradients at the downstream toe of the dam. However, these seepage analyses model generalized subsurface conditions, based on the available geotechnical data. The actual conditions of the seepage regime under the dam are variable due to the random depositional characteristics of the fluvial materials. Due to the critical nature of adequate piping performance, it is recommended that the two discussed seepage treatments are constructed.

Piezometers should be installed in the fluvial materials and glacial till near the Unnamed Creek to monitor pore pressures during operation. During the commissioning first partial filling of the reservoir, both the piezometers and relief wells should be monitored and analyzed to determine if there is a significant direct connection to the fluvial materials from the pool. Relief wells typically require consistent maintenance. Accordingly, it is recommended that a French drain style "relief trench" extending down into the fluvial zone be constructed across the full Unnamed Creek bottom areas.

12.5 STABILITY ANALYSIS

12.5.1 General / Analysis Approach and Assumptions

The general assumptions for the design of the Off-Stream Storage Dam – including specifically, slope stability analysis considerations - were discussed in Section 8. Additional relevant information and the results obtained from the various stability evaluations are included in the following sections.

12.5.1.1 Selection of Analysis Sections

The SR1 Storage Dam will be approximately 3.7 kilometers long (Figure 97). It begins with a low saddle dam and a relatively low section on the west end. Around the midpoint in the length, the height increases noticeably, reaching a maximum at the Unnamed Creek 400 m from the east end. The tallest section (Station 23+175) is approximately 29 m in height.

To cover the range of embankment configurations six cross sections were selected for analyses. The section locations are indicated in Figure 97.

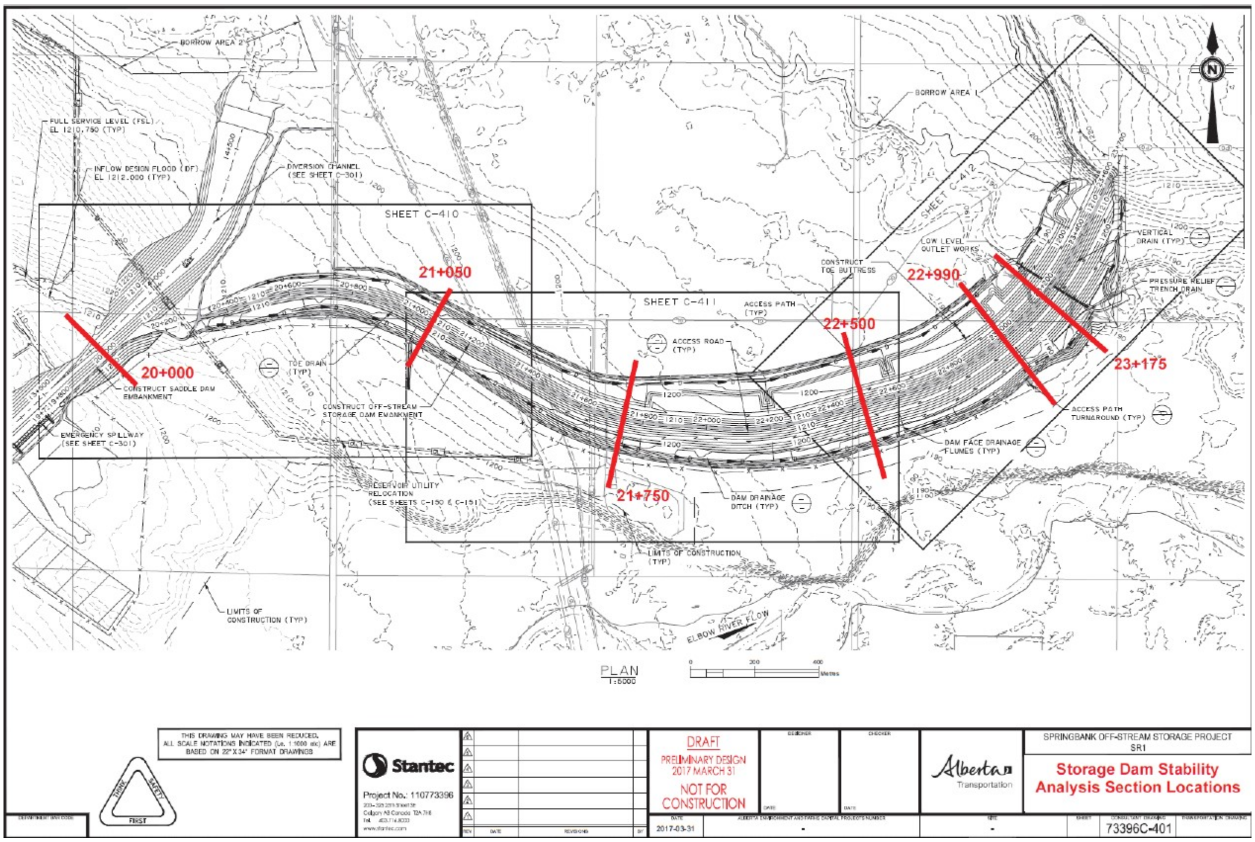


Figure 97. General Analysis Cross Section Locations

Analyses at each of these locations for the relevant loading conditions were performed. The selected section locations were chosen to represent various segments of the embankment dam. The general geologic profile of the Storage Dam is presented in Figure 98 below.

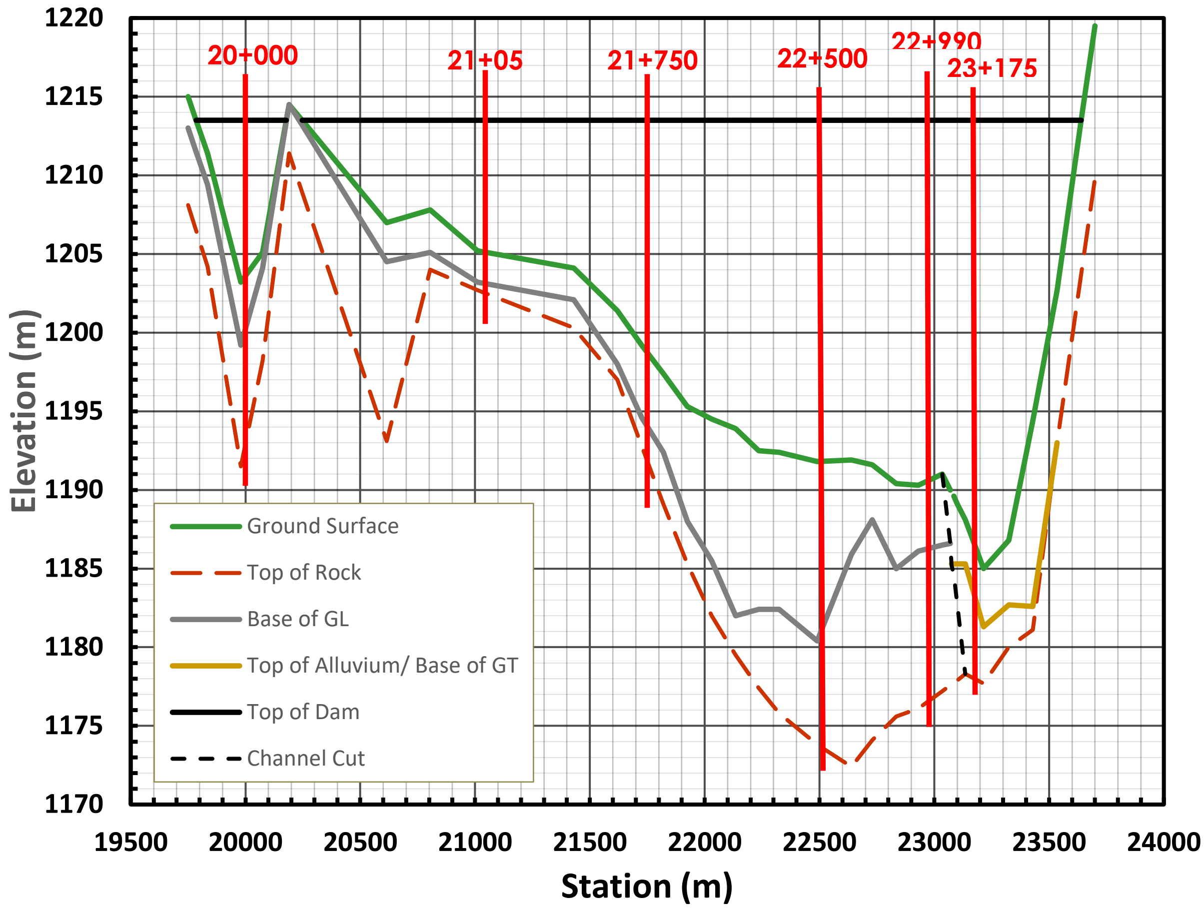


Figure 98. Storage Dam Foundation Profile showing Cross Section Locations

The greatest embankment height occurs near Station 23+200, but at this location the GL soil is absent. The greatest foundation soil thickness occurs at Station 22+650. Much of the foundation soil at this location is Glacial Clay Till (GT). The greatest thickness of GL foundation soil occurs between Station 22+100 and Station 22+500. Within this range the embankment is tallest at Station 22+500. This was selected as the critical cross section for GL soil foundation performance. At Station 22+500 the embankment is 22 m tall and the depth to bedrock is 18 m, with approximately 11.5 m of GL soil and 6.5 m of GT soil. The Station 23+175 section was selected to represent the tallest section. At this station the embankment is 29 m tall, but the foundation soil is only approximately 7 m thick.

12.5.2 Summary of Material Properties

Table 66 below summaries the shear strength properties used in the basic stability analyses performed.

Table 66. Material Parameters

Material Name	Unit Weight (kN/m^3)	Drained Strength		Undrained Strength	
		Cohesion (kPa)	Friction Angle (degrees)	Cohesion (kPa)	Friction Angle (degrees)
Embankment Shell (GL)	20	0	24	25	15
Embankment Core (GT)	20	0	28	80	19
Foundation Glacial Lacustrine	18	0	23	15	20
Foundation GL (alt. method)				$S_U/\sigma_V' = 0.265 * OCR^{1.5}$	
Foundation Glacial Till	18	0	27	60	19
Sand Drain	21	0	33	-	-
Rock Toe	20	0	33	-	-
Weathered Bedrock	21	0	35	-	-

12.5.3 Pore Pressure Response during Construction

As embankment is constructed, excess pore pressures will develop within the foundation and previously placed embankment material. Excess pore pressures will reduce the stability of the structure until enough time has elapsed for the pressures to dissipate. The rates of pore pressure generation and dissipation depend on many factors including the initial soil conditions, moisture content, soil compressibility, permeability, drainage paths, applied stress path, and loading sequence. With many factors impacting the change in pore pressure over time, it is difficult to predict the response with confidence. Initially, general site and soil characteristics and past case histories in similar situations were reviewed. Using this and other information, the pore pressure response was then evaluated using two general methods, the Simplified B-bar Method and the Finite Element Analysis Method.

12.5.3.1 Skempton's B-parameter versus B-Bar Values

Two related parameters are often used to characterize pore pressure response of soils. The B parameter, developed by A. W. Skempton (1954), describes the immediate change in pore pressure with uniform changes in confining stress ($\Delta\sigma_1 = \Delta\sigma_2 = \Delta\sigma_3$). The B parameter is commonly used in laboratory testing to gauge the degree of saturation in a soil sample. As the B parameter approaches one (as measured by the relationship between chamber pressure and sample pore pressure changes) the degree of saturation is assumed to approach 100 percent. This parameter only applies for the immediate response to changes in stress. It does not provide for changes (dissipation) in pore pressures over time after the change in stress.

The B-bar value is generally considered an "effective" value of Skempton's B parameter for a specific application. In practice, B-bar relates the change in pore pressure to changes in vertical stress ($\Delta\sigma_v$), usually without an explicit consideration for the applied shear stress. B-bar may be used to represent only the immediate pore pressure change or it may combine immediate change with consideration for pore pressure dissipation over the time of consideration. B-bar may also include some spatial coverage; that is, it may represent the average pore pressure response across a particular horizontal and vertical zone of soil.

Different engineers/authors have used the B-bar value to represent these varying factors, which is useful in characterizing a specific problem. However, the uniqueness of each situation makes it difficult to apply the results from one case history to other evaluations. In particular, the different rates of pore pressure dissipation, which varies significantly with soil thickness and drainage paths, complicates the comparison of B-bar values from various sites.

While the B-bar approach has limitations, it provides for simplified analyses of complex pore pressure problems and is commonly used in the analysis of pore pressures during staged construction. Case histories reviewed typically describe the pore pressure response in terms of the B-bar value. The B-bar methodology was used for the initial SR1 undrained analyses and for comparison with the more complex finite element analysis method.

12.5.3.2 Pore Pressures from Simplified B-Bar Method

12.5.3.2.1 General B-Bar Parameter Theory

The B-bar value is defined as the ratio of change in pore pressure over the applied vertical load. A B-bar value of 1.0 means the entire applied load is transferred to the pore water and 0.0 means no change in pore pressure from the applied load.

The B-bar value is commonly used to estimate remaining excess pore pressure at different times of interest. If pore pressure dissipation occurs in the time between load application and the time of interest, the lower B-bar value will represent some combination of the initial and time related response. As times of interest vary widely between different applications, comparison of B-bar values from differing situations can become problematic

12.5.3.2.2 Simplified B-Bar Parameter Analysis

The soil permeability, load application rate/timing, drainage paths and general configuration collectively influence the pore pressure increase and dissipation rates. The values of B-bar utilized in the analysis were estimated based on a combination of computations and judgement considering the available information. The relationship between soil permeability and the coefficient of consolidation (c_v) was also considered. Additionally, documented case histories from dams constructed on lacustrine / alluvial soils in the Canadian Prairie region were reviewed to judge the reasonableness of values selected for the SR1 embankment dam.

Values were selected for each soil zone at the time of the analysis to represent what portion of the added vertical load might remain as excess pore pressure. An analysis program (in this instance GeoStudio/SlopeW) was used to analyze stability using pore pressures equal to the static value plus the residual percentage of the added vertical load. The analysis is based on drained soil strength parameters and the excess pore pressures derived from the B-bar values.

12.5.3.2.3 Summary of B-bar Values from Simplified Approach

To perform the simplified B-bar analysis, the cross section being evaluated was divided into zones. The foundation soil was divided by soil type and year of embankment loading (year 1, year 1 & 2 or year 1, 2 & 3). The embankment was divided into years of construction. The soil zonation is illustrated in Figure 99. This simplified B-bar analysis was performed only on the pore pressure condition existing at the end of year-3 construction.

B-bar parameters were selected for each zone – foundation and embankment – for the areas loaded in construction seasons one, two, and three. The embankment was further divided into core and shell zones, while the foundation was divided between the glacial lacustrine clay and the glacial clay till layers. The selected B-bar values are presented in Table 67 below. The B-bar parameter values used for the end of construction season three Load Case analysis are illustrated graphically in Figure 99.

Table 67. B-bar Parameters Used for Slope Stability Analyses

Material	Location	Years Loaded	B-bar for Case 3 End of Year 3 Construction					
			Station 20+000	Station 21+050	Station 21+750	Station 22+500	Station 22+990	Station 23+175
GT Foundation	Crest	1, 2 & 3	0.10	0.10	0.15	0.45	0.4	0.4
	Slopes	1 & 2	0.10	0.10	0.15	0.3	0.4	0.3
	Toe	1	0.10	0.10	0.1	0.15	0.3	0.2
GL Foundation	Crest	1, 2 & 3	0.15	0.15	0.3	0.8	-	-
	Slopes	1 & 2	0.15	0.15	0.3	0.45	-	-
	Toe	1	0.15	0.15	0.25	0.3	-	-
GT Embankment Core	Year 1	1, 2 & 3	-	-	-	0.5	-	0.5
	Year 2	2 & 3	-	-	0.4	0.4	0.55	0.4
	Year 3	3	0.0	0.0	0.0	0.0	0.0	0.0

Table 67. B-bar Parameters Used for Slope Stability Analyses (Continued)

Material	Location	Years Loaded	B-bar for Case 3 End of Year 3 Construction					
			Station 20+000	Station 21+050	Station 21+750	Station 22+500	Station 22+990	Station 23+175
GL Embankment Shell	Year 1	1, 2 & 3	-	-	-	0.18	-	0.18
	Year 2	2 & 3	-	-	0.18	0.15	0.18	0.15
	Year 3	3	0.0	0.0	0.0	0.0	0.0	0.0
Rock Toe	Year 1	1	0.0	-	-	0.0	-	-

Specific zones of the embankment were then identified in GeoStudio to be "added weight" - or the source of the stress increase – to be applied to the soil below. For design this approach is performed iteratively with revised section configurations until an acceptable slope stability factor of safety (1.3) was achieved.

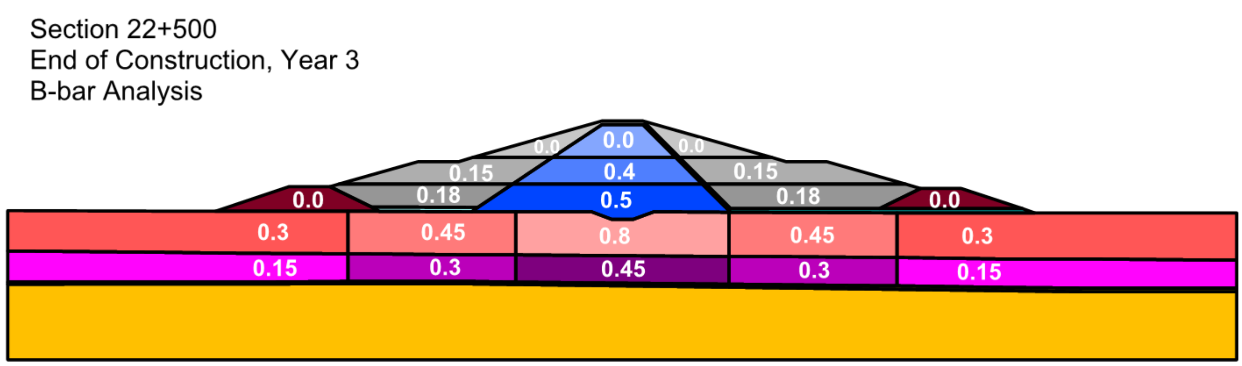


Figure 99. Simplified B-bar Analysis Cross Section

12.5.3.3 Pressures from Finite Element Method

12.5.3.3.1 General Description of Approach

A finite element model was developed to better represent two-dimensional drainage effects, as well as load increment and time step mechanics. A model of the design cross section was developed for analysis in PLAXIS 2D. The finite element model did not use a coupled effective stress and pore pressure formulation. Instead, the initial pore pressure response was estimated separately and input into the PLAXIS model at specific stages as the embankment height was raised. The dissipation of excess pore pressures was computed by PLAXIS through the time steps.

12.5.3.3.2 Plaxis Model

Three embankment cross sections - with significant glacial lacustrine foundation layers - were selected to analyze. The selection of the cross sections is described in Section 12.5.1. The sections selected included Station 21+750, Station 22+500 and 22+990. The cross section at Station 22+500 is used as a typical analysis in the following sections / figures. The solution mesh is shown in Figure 100.

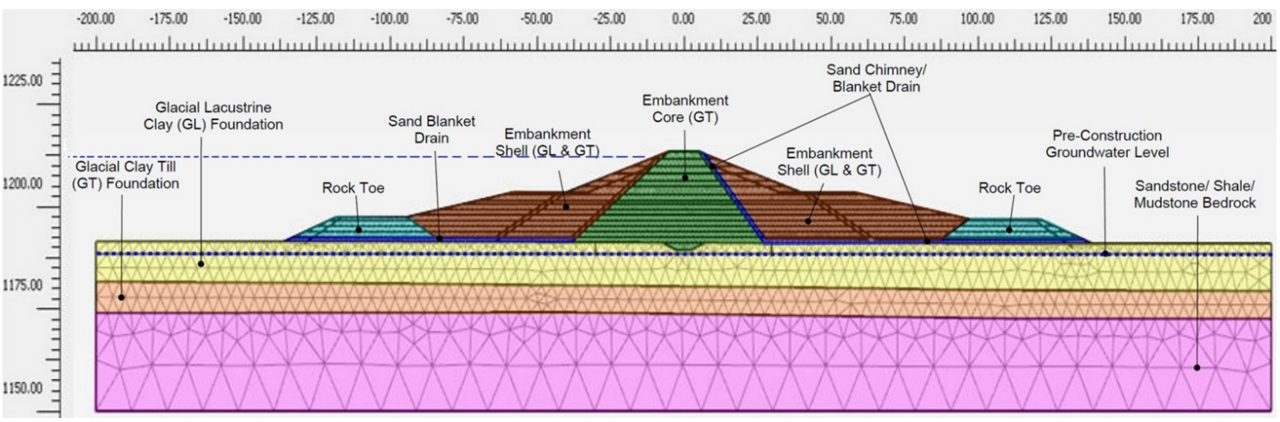


Figure 100. PLAXIS Model Configuration

The large elements at the bottom of the model are bedrock. The three soil layers above – from the top down - represent glacial lacustrine clay above the preconstruction groundwater table (3 m deep), glacial lacustrine clay below the groundwater table, and glacial clay till.

12.5.3.3.3 Purpose for Plaxis Analysis

The finite element model analysis is intended to overcome the uncertainties associated with pore pressure dissipation in the B-bar pore pressure evaluation methodology. The function of the Plaxis model was only to characterize the direction and rate of pore pressure dissipation. The pore pressures obtained from the PLAXIS analysis were then mapped to limit equilibrium slope stability analyses completed in GeoStudio's Slope/W.

12.5.3.3.4 Model Soil Properties

The various soil materials in and under the embankment need to be represented with specific constitutive model parameters. Based on each soil zone's characteristics, three soil models were considered. The model parameters represent the stiffness and shear strength of each soil. Permeability parameters are also needed to model pore pressure dissipation.

A linear elastic model was used for the bedrock. The Mohr-Coulomb constitutive model was used for all soil zones in the embankment and foundation. This model includes a linear response up to the failure condition and then a fully plastic response for strains above the failure load. The pore pressure dissipation rate is sensitive to the stiffness (modulus) assigned to the material.

Furthermore, because overconsolidated clays demonstrate a change in stiffness as the loading changes, the soil moduli were varied to match the observed soil performance. This method better accounts for soil behavior over a range of stress, as compared to a single compression modulus. This refinement was applied for both the Glacial Lacustrine and Glacial Till soils. Development of the varying moduli is described in the following paragraphs.

12.5.3.4 Review of Soil Preconsolidation Data

The relevant soil compressibility parameters and associated laboratory test results were reviewed in detail. Specifically, sample location relative to the critical embankment analysis section was evaluated, as well as disturbance indices for the laboratory test specimens. Data points which were geographically remote, or which exhibited significant disturbance were removed from the data set used for establishing soil properties. However, even with the remote and disturbed sample test results removed, the preconsolidation stresses exhibit substantial scatter. To provide more consistent input to the model, the preconsolidation pressure was set equal to the current (preconstruction) vertical effective stress plus a constant value. Based on the applicable 1-D consolidation test results, an offset value of 130 kPa was selected for the preconsolidation stress in the GL foundation soil and a value of 65 kPa was selected for the GT foundation soil. The OCR then varies from large values near the ground surface to lesser values at increasing depth.

12.5.3.5 Selected Soil Compressibility

The results from the 1-D consolidation tests were used to model the compressibility of the foundation soils under increasing load steps. Specifically, the foundation soil was divided into horizontal layers – four for the glacial lacustrine clay and two for the glacial clay till. For each layer the change in height (consolidation) was established as successive load applications increased the vertical effective stress. The ratio of the incremental change in height relative to the change in effective stress was computed as the tangent compression modulus. The values established are presented graphically in Figure 101. Note that modulus increases with load until the preconsolidation pressure is exceeded, where there is a large drop in the soil stiffness.

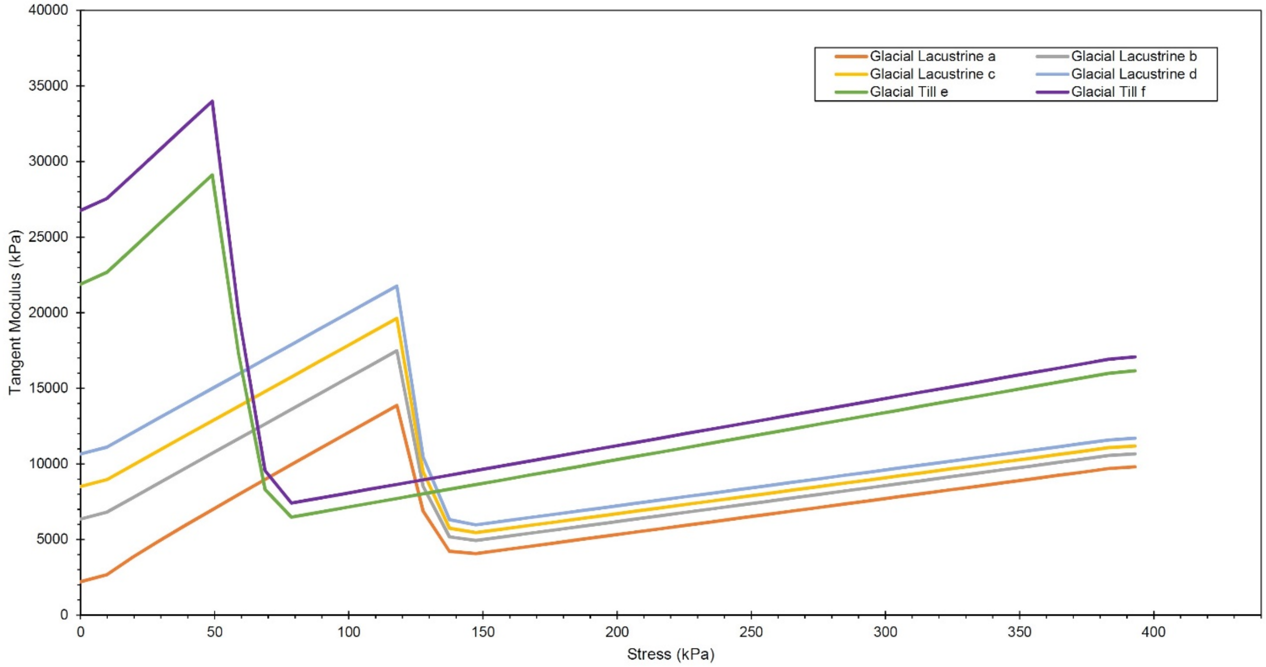


Figure 101. Variation in Compression Modulus with Confining Stress

To incorporate this variation in moduli into the PLAXIS model, the element stresses were evaluated at each analysis stage and the moduli were adjusted for the successive stage. The effective stresses were established separately for the zones of each soil layer under the embankment crest, under the slopes and under the toes. Note that the soil stiffness impacts the consolidation behavior, with the excess pore pressures dissipating more quickly from stiffer soil zones.

The other soil properties utilized within PLAXIS for these soil elements are presented in Table 68.

Table 68. Mohr-Coulomb Model Properties used in the PLAXIS Model

Material	c' (kPa)	ϕ'	e_o
Glacial Clay Till Core	0	28	0.52
Glacial Lacustrine Clay Shell	0	24	0.73
In Situ Glacial Lacustrine Clay	0	23	0.74
In Situ Glacial Clay Till	0	27	0.52
Granular Drain/Rock Toes	0	33	0.50

The underlying sandstone/shale bedrock was modeled with Linear Elastic Model elements. The bedrock elements were assigned an elastic modulus (E) of 600 MPa and a Poisson's ratio (ν) of 0.1.

Soil permeabilities are presented in the Material Properties Design Basis Memorandum included in Attachment 5. Those values represent laboratory measurements. The permeability of in situ soils (natural deposits or embankment fills) are often variable and higher than determined from laboratory test samples. To realistically represent the expected field conditions, permeabilities were selected based on a review of the laboratory tests, field tests, and engineering experience with field performance. These values, presented in Table 69, were then used in the PLAXIS analyses.

Table 69. Soil Permeabilities used in the PLAXIS Model

Material	Horizontal Permeability		Vertical Permeability		K_h/k_v
	m/sec	m/day	m/sec	m/day	
In Situ Glacial Lacustrine Clay	2.5E-09	2.2E-04	5.0E-10	4.3E-05	5
In Situ Glacial Clay Till	1.5E-09	1.3E-05	5.0E-10	4.3E-05	3
Sandstone/Shale Bedrock	3.0E-08	2.6E-03	3.0E-08	2.6E-03	1
Glacial Clay Till Core	2.5E-09	2.2E-04	5.0E-10	4.3E-05	5
Glacial Lacustrine Clay Shells	5.0E-09	4.3E-04	1.0E-09	8.6E-05	5
Granular Drains	3.0E-06	2.6E-01	3.0E-06	2.6E-01	1

12.5.3.5.1 Model Loading Stages / Parameter Adjustments

The PLAXIS model was set up to replicate the staged construction of the embankment over the planned thee construction seasons. For convenience the construction was divided into monthly loads. The stages considered are presented in Table 70. The construction sequence is illustrated graphically in Figure 102. The same completed elevations for year-1, year-2 and year-3 were used as were used in the Simplified B-bar Analysis method.

Table 70. Embankment Loading Stages

Year	Month	Stage	Fill Elevation.	Notes
1	May	0	1191.2	Existing Ground Elev.
1	June	1	1192.6	
1	July	2	1193.9	
1	August	3	1195.3	
1	September	4	1196.6	
1	October	5	1198.0	
				Year 1 Shutdown
2	May	6	1198.0	
2	June	7	1199.3	
2	July	8	1200.6	
2	August	9	1201.9	
2	September	10	1203.2	
2	October	11	1204.5	Case 1 End Yr 2 Construction
				Year 2 Shutdown
3	May	12	1204.5	Case 2 Intermediate Flood
3	June	13	1206.3	
3	July	14	1208.1	
3	August	15	1209.9	
3	September	16	1211.7	
3	October	17	1213.5	Case 3 End Yr 3 Construction
				Year 3 Shutdown – (fill complete)
4	May	18	1213.5	Case 4 Full Design Flood

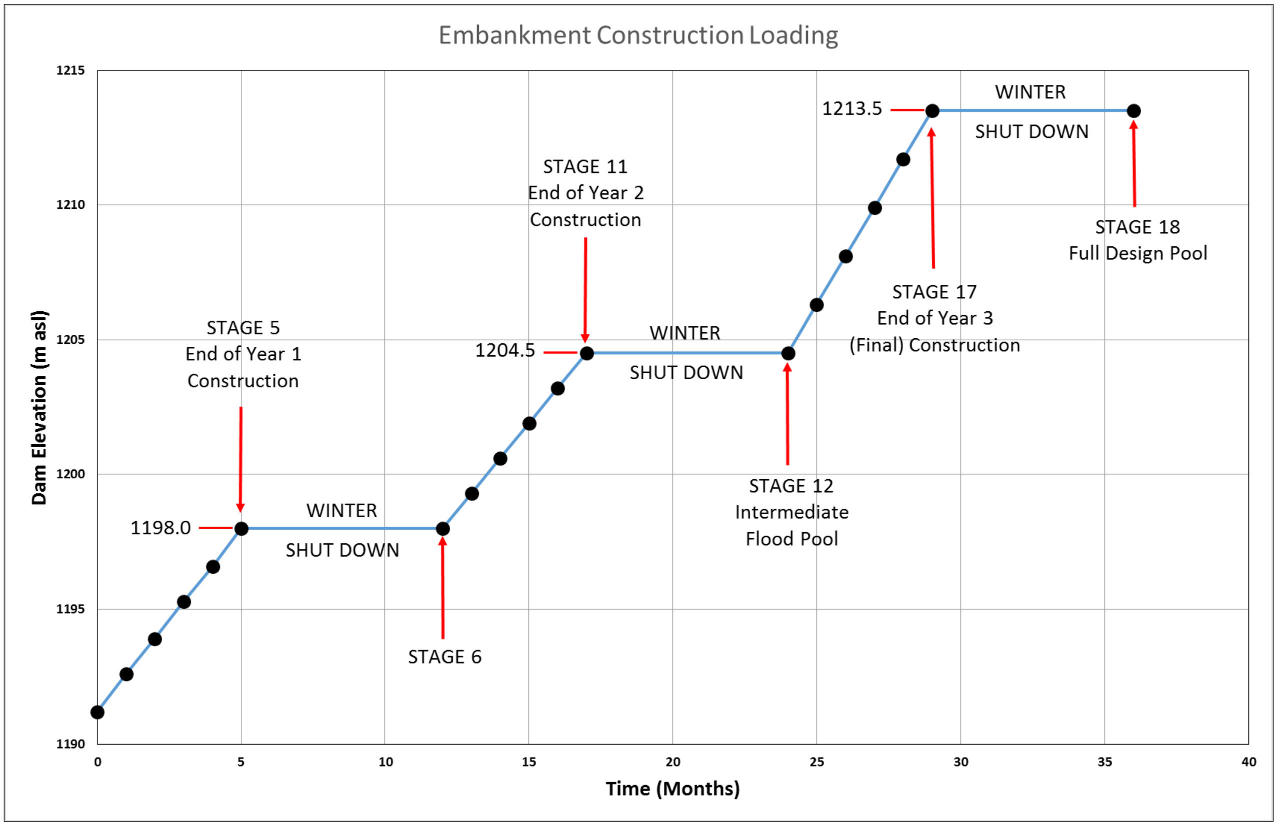


Figure 102. Sequence of Construction Embankment Loading

The PLAXIS model is developed in horizontal stages representing how the embankment will be constructed over time. Each of the horizontal layers is a single loading / time stage comprising a month of embankment construction. The model configuration at significant stages are illustrated in Figure 103 (end of Year -1 construction), Figure 104 (end of Year-2 construction), and Figure 105 (end of Year-3 (and final) construction). At each of these stages, construction is halted for seven months (winter shutdown) but pore pressures will continue to redistribute and dissipate.

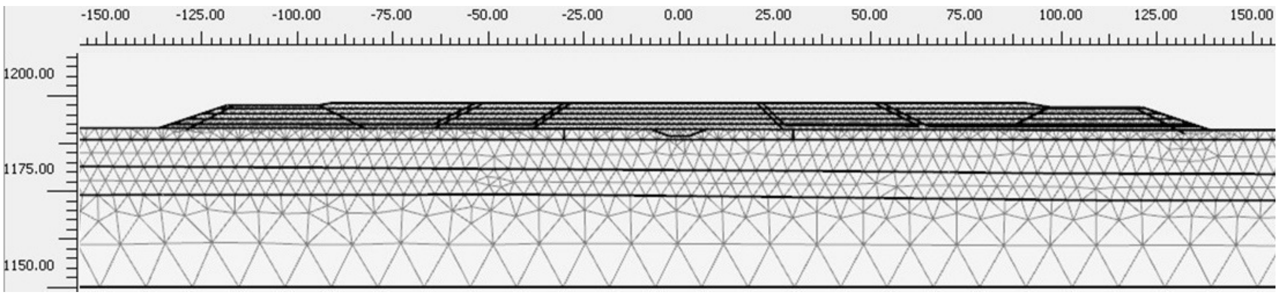


Figure 103. PLAXIS Model at End of Year 1

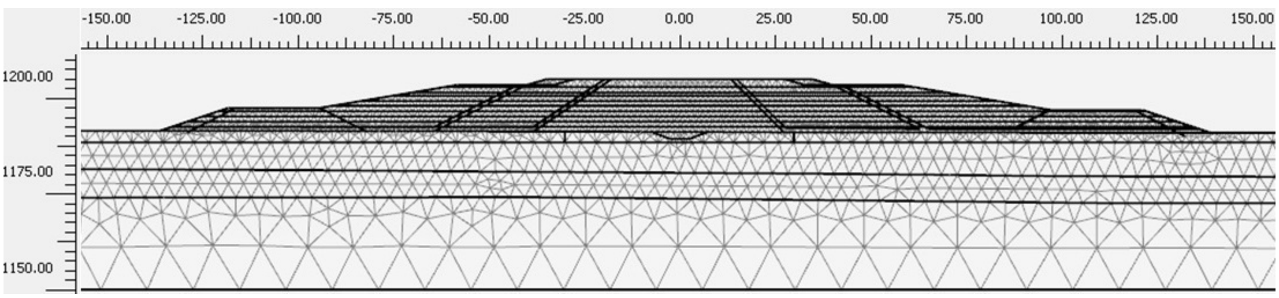


Figure 104. PLAXIS Model at End of Year 2

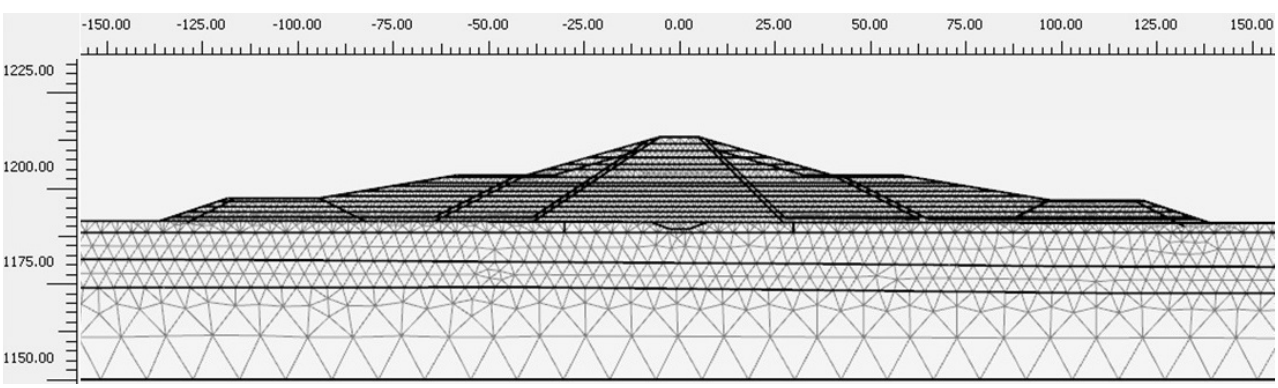


Figure 105. PLAXIS Model Configuration at Embankment Completion (Year 3)

One useful capability of the PLAXIS software is the ability to modify element properties at specified steps in the analysis process. Because the pore pressure response is expected to change with increasing embankment load, the construction was further divided into "Stage Groups" for pore pressure response or B Parameter adjustment. Specifically, five different sets of B parameters were provided for the following stage groups; Stages 0 to 3, Stages 4 to 7, Stages 8 to 10, Stages 11 to 14, and Stages 15 to 18.

Furthermore, the unsaturated glacial lacustrine clay foundation, the placed glacial clay till core zone, and the placed glacial lacustrine clay shell zone were each evaluated separately. These soils zones are subject to greater pore pressure response as the soil approaches saturation when compressed under load. Appropriate, updated B parameter values were provided for each material zone each time the model parameters were revised.

As discussed in Section 12.5.3.5 the moduli of the foundation soils was also adjusted at each analysis stage, to better represent the compressibility as the loading changes.

12.5.3.5.2 Initial Pore Pressure Response

As additional lifts of embankment are constructed, the underlying soils experience increased load. The soil is compressed by this added load, and the fluid in the pores (voids) picks up a portion of that load.

In the Simplified B-bar analysis method the portion of the new load carried by the pore water was represented by a B-bar value. This B-bar value incorporates both the initial response and the dissipation up to the time of interest. Hence, the B-bar value is generally related to an estimate of the remaining excess pore water pressure at some time of interest.

In the finite element analysis, the immediate pore pressure response is input to the model. The finite element computational code then models dissipation of the excess pressures as it processes successive analysis time steps. Accordingly, the parameter required to estimate the pore pressure response for the PLAXIS model is different than that used in the B-bar approach. For the PLAXIS analysis, the B parameter is used without consideration for dissipation rates.

Skempton (1954) related the immediate change in pore pressure (Δu) with changes in confining stress: $\Delta u = B\ (\Delta\sigma_c)$. For the PLAXIS model, the change in pore pressure was related to the change in applied vertical stress:

$$\Delta u = B\ (\Delta\sigma_v)$$

For saturated soils the B parameter is assumed to equal 1.0. The challenge comes when estimating the B parameter for unsaturated zones within the foundation and embankment soil.

The method used for establishing initial pore pressure response in unsaturated zones is based on the equation for change in pore pressure in unsaturated soils developed by J.W. Hilf and described by Fredlund et al. (2012).

12.5.3.5.3 Immediate Pore Pressure in Saturated Soil Zones

The in situ Glacial Till (GT) and the lower Glacial Lacustrine (GL) clays (below 3 m in depth) are saturated. In these foundation zones, the B parameter is assumed to be 1.0. The pore pressure response in saturated GL clays is critical for stability of the section, as shown later.

12.5.3.5.4 Immediate Pore Pressure Response in Unsaturated Soil

The pore pressure response of the unsaturated soils was modeled using the method developed by J. W. Hilf (1948) and described by Fredlund et al. (2012). Hilf's method accounts for the effects of both the water and the air in the soil pore space.

The derivation is based on the results of one-dimensional oedometer tests on compacted soil, Boyle's law, and Henry's law. Hilf's equation provides pore fluid pressure response as a function of the change in soil porosity.

$$\Delta \mathrm{u}_a = \left[\frac{\Delta n}{(1 - \mathrm{S}_0) * \mathrm{n}_0 \ + \ \mathrm{h} * \mathrm{S}_0 * \mathrm{n}_0 - \Delta \mathrm{n}]}\right] * u_{a0} \qquad \textbf{Equation 8}$$

Where:

Δu_a = Change in pore fluid pressure

u_{a0} = Initial air pressure (absolute)

Δn = Change in porosity

n_0 = Initial porosity

S_0 = Initial degree of saturation

h = Henry's constant (0.026 for 4°C soil)

This equation is used with steps in applied load that cause increments of porosity changes. The initial air pressure, porosity, and saturation are each determined for the start of the increment.

Hilf's pore pressure response can be plotted as function of porosity in the unsaturated soil. The 1-D soil consolidation response can be plotted in terms of porosity and effective stress. Here, the porosity is assumed to be a unique function of the effective stress (assumes no soil unloading). For a given porosity, the effective stress and pore pressure can then be summed to obtain the total stress at that porosity. From this computation, the increase in pore pressure can be estimated as a function of total stress. Regardless of the theoretical curves, when an initially unsaturated zone reaches the fully saturated condition, the B parameter is assumed to become 1.0.

These relationships, as developed for the unsaturated glacial lacustrine clay foundation are illustrated in Figure 106 and Figure 107. The slope of the curve in Figure 107 is equal to B, which approaches 1.0 at applied pressures greater than about 900 kPa. Similar relationships were developed for both the Glacial Till and Lacustrine Clay embankment soils.

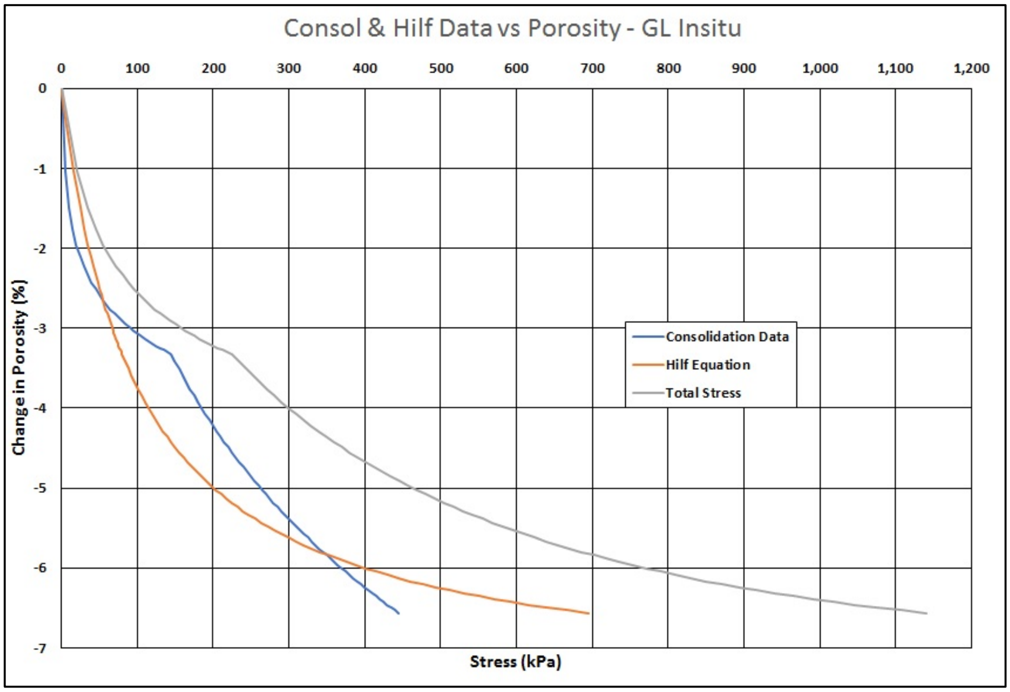


Figure 106. Hilf Equation and Consolidation Data for Unsaturated GL Soil

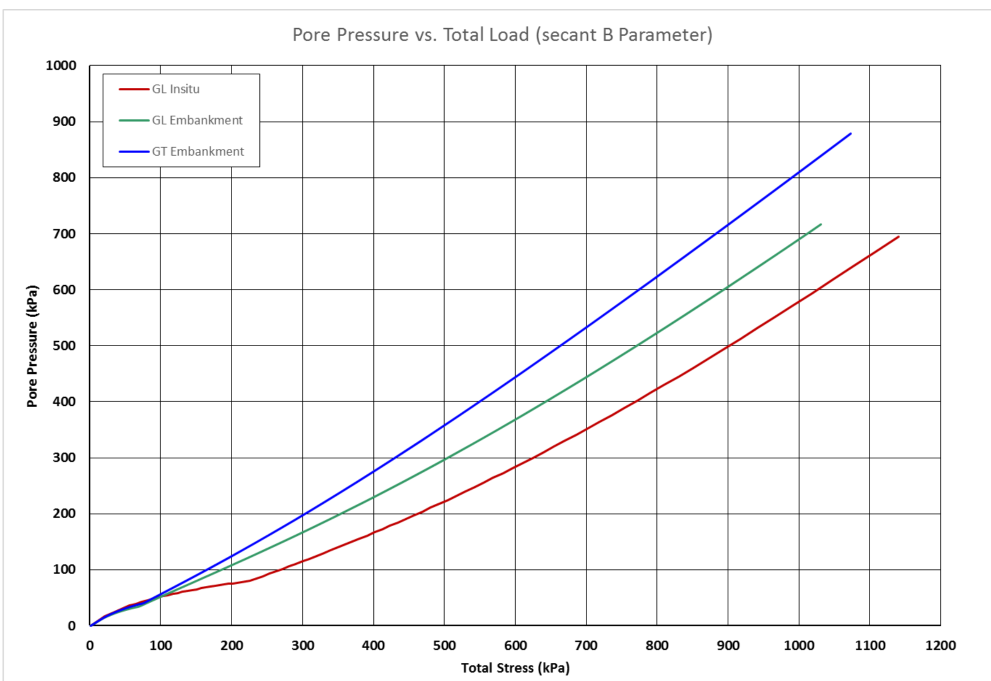


Figure 107. Excess Pore Pressure Predicted for Applied Total Stress in Unsaturated Soils

For comparison, the tangent B parameters from both methods are plotted against degree of saturation in Figure 108. The notes on the figure illustrate the increase in saturation (and B parameter) of the near-surface foundation soil as the embankment construction progresses.

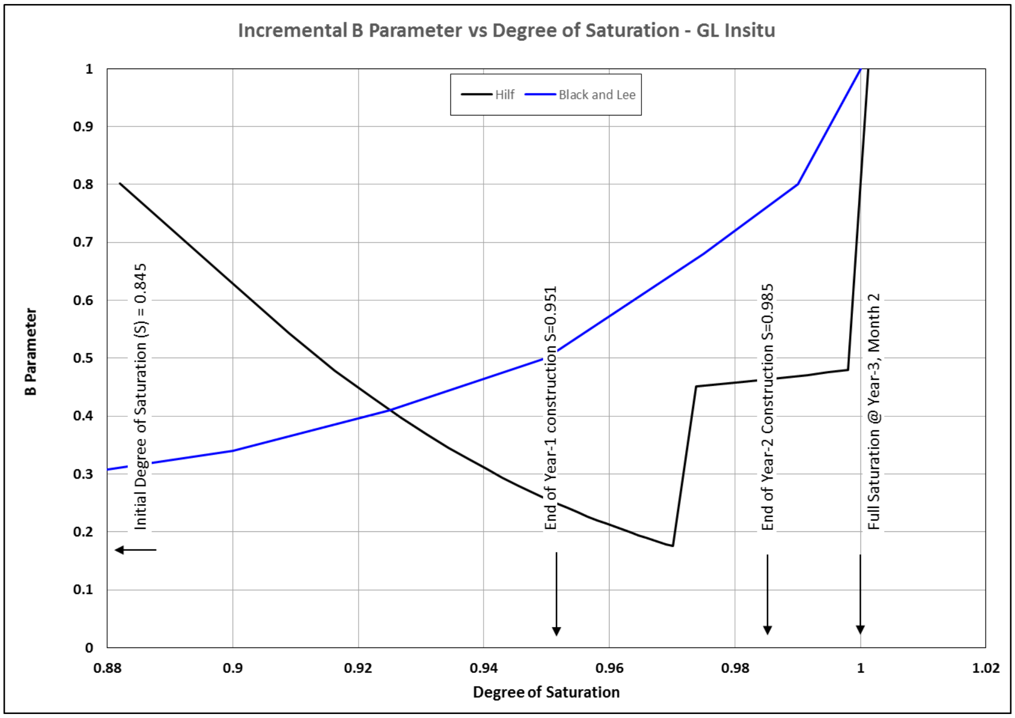


Figure 108. Relationship of B parameter and Degree of Saturation in GL Soil

12.5.3.5.5 Section Configuration Adjustments

During the design process, some trial cross sections were not stable, as indicated by excessive deformations in the Plaxis model. Modifications to the embankment cross section were implemented to improve stability.

Initially, toe buttresses were added on both the upstream and downstream slopes. Then buttresses were implemented as wider stability benches (10 m in place of 5 m). Additionally, to shorten drainage paths, a drainage zone (blanket or finger drain configuration) was added beneath the upstream shell. Ultimately, rock zones were also added to both upstream and downstream toes.

For the current evaluation, both the embankment core (glacial clay till) and shells (glacial lacustrine clay) were assumed to be placed at a moisture content two percentage points above the Proctor optimum moisture content. High soil moisture contents are expected in the borrow pits, and this slightly higher moisture may facilitate embankment construction.

12.5.3.5.6 Plaxis Factor of Safety Check

The PLAXIS program can compute factors of safety for slope stability. PLAXIS performs an effective stress analysis, using drained strength parameters and estimated pore water pressures. The factor of safety calculation is accomplished by incremental reduction in material strengths until instability occurs. At this point the program compares the specified model strengths to the reduced strengths to establish the factor of safety. This definition for the safety factor is the same as used in conventional limit equilibrium analyses, but the methods used to compute the internal soil stresses differ. This feature was used to compare the PLAXIS model results to the safety factors computed with GeoStudios SLOPE/W. Although the PLAXIS and SLOPE/W operate very differently, when checked, the factors of safety were generally within about 10 percent of each other.

12.5.3.5.7 Pore Pressure Output

At any given stage of the PLAXIS analysis, selected data can be exported for use in GeoStudio. Data provided for each model element includes; X location, Y location, excess pore pressure, total pore pressure, and total stress. This data was processed and passed to the corresponding GeoStudio analysis model, as described in Section 12.5.3.6.1.

The PLAXIS output was also processed and plotted to understand the distribution and dissipation of the excess pore pressures. Examples showing the change in pore pressure during the construction process are provided in Figure 109 and Figure 110. The results show higher pore pressures at the end of each construction season, increasing as the dam is raised (Stages 5, 11, and 17), followed by dissipation over the winter shutdown periods (Stages 6, 12, 18). Note that Figure 109 and Figure 110 show only the GL foundation soil zone.

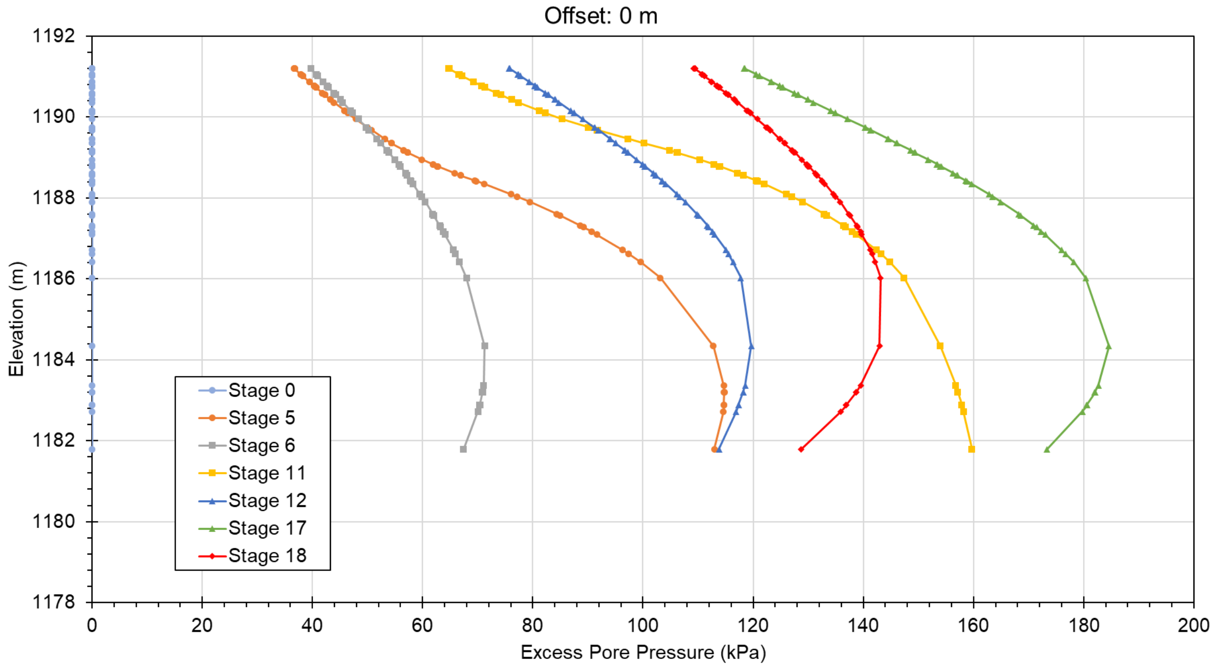


Figure 109. Excess Pressure in GL Foundation during Construction at Embankment Centerline

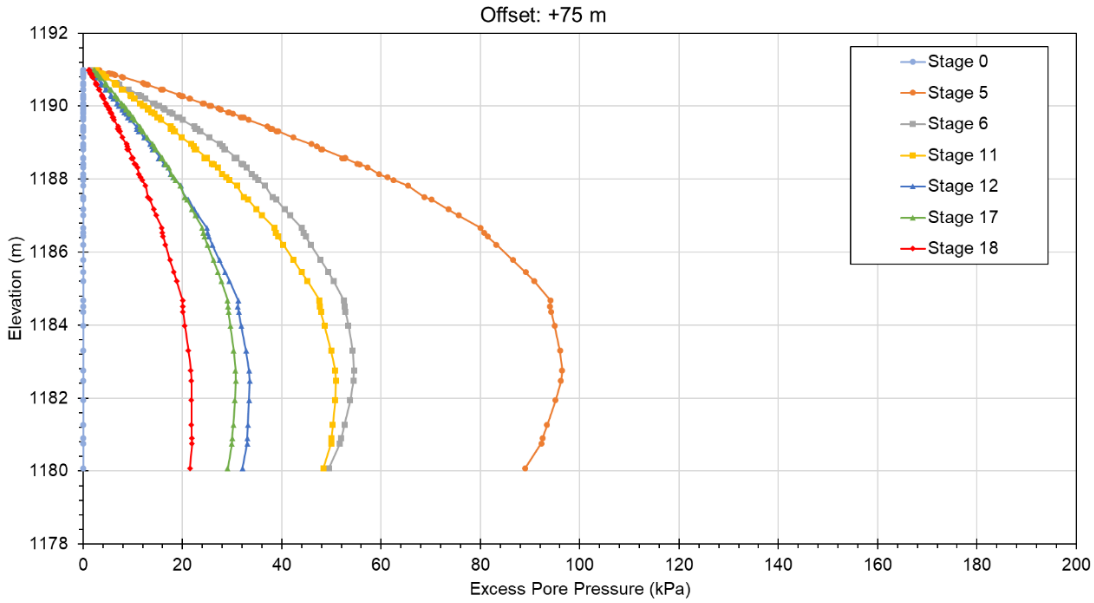


Figure 110. Excess Pore Pressure in GL Foundation during Construction at Downstream Toe

12.5.3.6 Construction Period Soil Strengths

To appropriately model the glacial lacustrine soil strengths during the time during and shortly following the construction of the embankment an undrained strength model was developed. This model is based on the S_u/σ_v' ratios derived from direct simple shear laboratory testing. The development of these soil strengths is described in the following paragraphs.

12.5.3.6.1 General Description of Approach

The undrained strength for the Glacial Lacustrine foundation soil was modeled using S_u/σ_v' ratios derived from direct simple shear laboratory testing. The S_u/σ_v' ratio is assumed to vary with overconsolidation ratio (OCR), which will change in the foundation GL soil as the embankment is constructed and consolidation occurs. To account for the variation in OCR and S_u/σ_v' at each point in the GL, the undrained shear strength was computed at grid points and modeled in GeoStudio as a spatial Mohr-Coulomb function. In the slope stability analyses, the grid point strengths are interpolated to obtain the strength on the base of each slice.

The cohesion (with ϕ equal to zero) was calculated at each point using S_u/σ_v' computed for the current OCR. The calculations were completed at the grid points where the stress and pore pressure were computed in PLAXIS. To calculate cohesion, the following steps were taken at each grid point for the time of interest for the construction period cases:

1. Export pore water pressure (u) and total vertical stress (σ_v) from PLAXIS for points within the foundation glacial lacustrine soil.

2. Calculate the current effective vertical stress (σ_v') at each point using the stresses and pore pressures from PLAXIS ($\sigma_v - u$).

3. Calculate the preconsolidation pressure at each point based on depth below the existing, original ground surface
4. Calculate the current OCR as the preconsolidation pressure divided by the current vertical effective stress (OCR = σ_p'/σ_v'). If the computed σ_v' is greater than σ_p', then OCR = 1.0.
5. Determine the S_u/σ_v' ratio at each point based on the current OCR and the relationship in Section 12.5.3.6.2.1.
6. Calculate undrained shear strength at each point by multiplying the S_u/σ_v' ratio by the current effective vertical stress (See Section 12.5.3.6.3).

The completion of these steps resulted in undrained shear strengths for each grid point in the glacial lacustrine foundation soils. These values (cohesion) were used in the spatial Mohr-Coulomb function within the GeoStudio Analysis process. Details for certain steps are discussed in the following sections.

12.5.3.6.2 Glacial Lacustrine Preconsolidation pressure

As discussed in Section 12.5.3.4 the preconsolidation pressures were determined as the preconstruction effective stress plus a fixed added stress. Based on the applicable 1-D consolidation test results, an offset value of 130 kPa was selected for the preconsolidation stress in the GL foundation soil and a value of 65 kPa was selected for the GT foundation soil.

12.5.3.6.2.1 Glacial Lacustrine S_U/σ_V' and OCR

The undrained strength ratio (S_u/σ_v') can be used to model the shear strength of a soil as a function of the effective vertical stress. The S_u/σ_v' value is dependent on the consolidation state of the soil where a constant value can be used for normally consolidated soils, and the ratio increases as OCR increases. A correlation between S_u/σ_v' and OCR was suggested by Ladd (1992) and is shown in Figure 110:

$$\frac{S_u}{\sigma_v'} = S * OCR^{0.8}$$ **Equation 9**

where:

S_u = Shear strength (assumes ϕ = 0)

σ_v' = Vertical effective pressure

S = Normally consolidated shear strength ratio

OCR = Overconsolidation Ratio

0.8 = empirical exponent

The results of the direct simple shear tests conducted on the undisturbed glacial lacustrine soil samples are presented in the Geotechnical Materials Properties Memo included in Attachment 5. That data indicates an average S_u/σ_v' of 0.265 for normally consolidated soil samples, and an average S_u/σ_v' of 0.757 for soil samples with an OCR of 2. Refer to Figure 111 for a plot of all DSS results. Design strength ratios were selected as 0.265 and 0.757, respectively, for OCR of 1.0 and 2.0.

A relationship is needed to calculate S_u/σ_v' for any OCR. This was accomplished by using the above values (selected from the DSS test results) and solving for the exponent in Figure 111. The resultant exponent was calculated as 1.5, somewhat higher than the empirical value suggested by Ladd. The relationship used to calculate S_u/σ_v' for the stability analyses for the glacial lacustrine foundation soils is then:

$$\frac{S_u}{\sigma_v'} = 0.265 * OCR^{1.5}$$ **Equation 10**

This equation is depicted on the results of the direct simple shear tests shown in Figure 111 below. The selected equation results in 54% of the DSS test results above the S_u/σ_v' envelope.

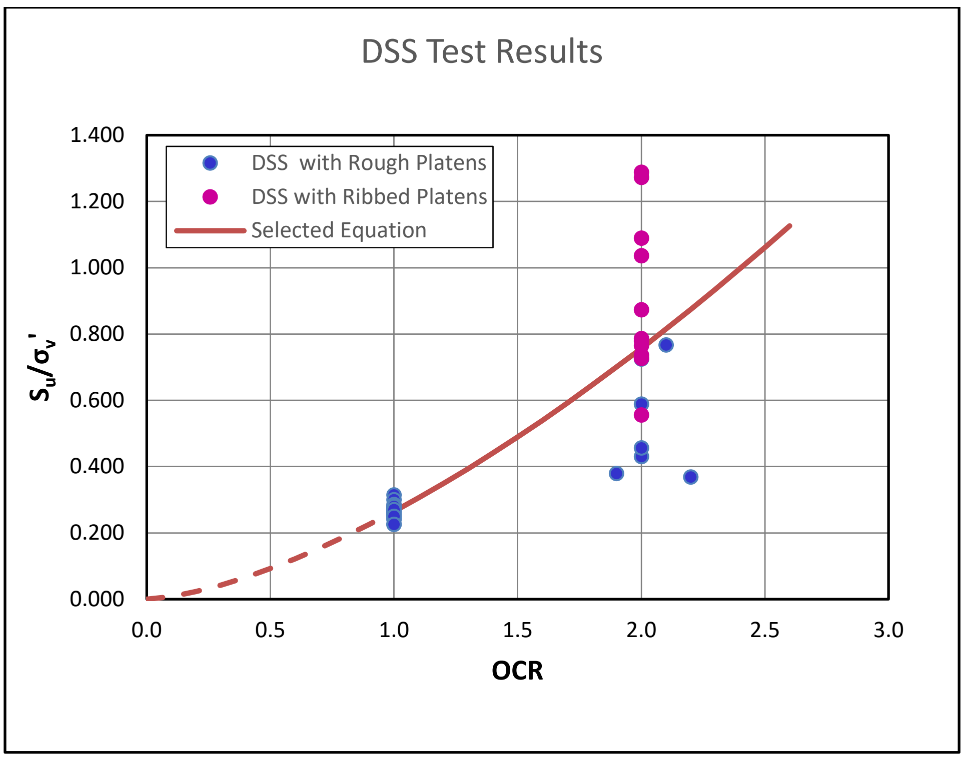


Figure 111. Su/σv' versus OCR for the Glacial Lacustrine Foundation

12.5.3.6.3 Glacial Lacustrine S_U/σ_V' and OCR

The steps noted in Section 12.5.3.6.1 were carried out using a spreadsheet and the PLAXIS output for pore water pressure and total vertical stress at each grid point. The spreadsheet calculated the current effective vertical stress (from the PLAXIS output) and depth below the original ground surface. Using the relationship described in Section 12.5.3.6.2, the preconsolidation pressure was calculated from the depth below the original surface. The OCR was then determined, and the S_u/σ_v' value was calculated using **Equation 8**. The undrained strength was calculated by multiplying S_u/σ_v' by the effective vertical stress. An example row from the spreadsheet is shown in Table 71.

Table 71. Example Strength Calculation

From PLAXIS				Calculated					
X (m)	Y (m)	Total Vertical Stress, σ_y (kPa)	U_{Total} (kPa)	Effective Vertical Stress, σ'_y (kPa)	Depth below Original Ground(m)	Preconsolidation Pressure (kPa)	Current OCR	S_u/σ_v'	S_u (kPa)
9.04	1188.29	429.15	309.60	119.55	2.9	134	1.12	0.314	37.6

The x, y, and S_u values for all points in the glacial lacustrine layer were then input into GeoStudio using a spatial Mohr-Coulomb method to model the strength of the foundation soil. A contour map of the computed variation in undrained strength is included in Figure 112. Note that the spatial function was only used in the glacial lacustrine foundation layer, so other material layers appear as zero cohesion in Figure 112.

The combination of the varying vertical effective stress and preconsolidation stress results in an undrained shear strength that varies between about 20 and 60 kPa in the glacial lacustrine layer. Furthermore, the plot in Figure 112 shows that the undrained strength is not significantly larger under the center of the embankment. This is because the induced, excess pore pressures are relatively high at the end of construction, due to relatively slow dissipation and lack of consolidation in these low-permeability materials.

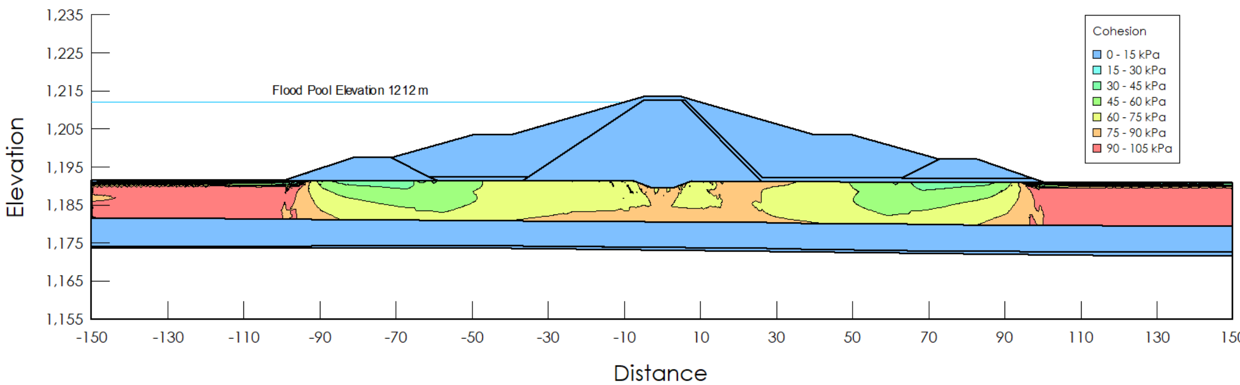


Figure 112. Spatial Contour Map of Undrained Shear Strengths in the Glacial Lacustrine Foundation Soil, end of Year 3 Construction

Data was extracted from the spatial shear strength information and plotted along vertical columns to illustrate changes in strength with depth and time. Columns were evaluated at the embankment centerline, at the approximate bench locations and near the toes. Data was included for the beginning and end of each construction season. A full set of plots are included in Attachment 12.1. Samples are provided in Figure 113 and Figure 114 below.

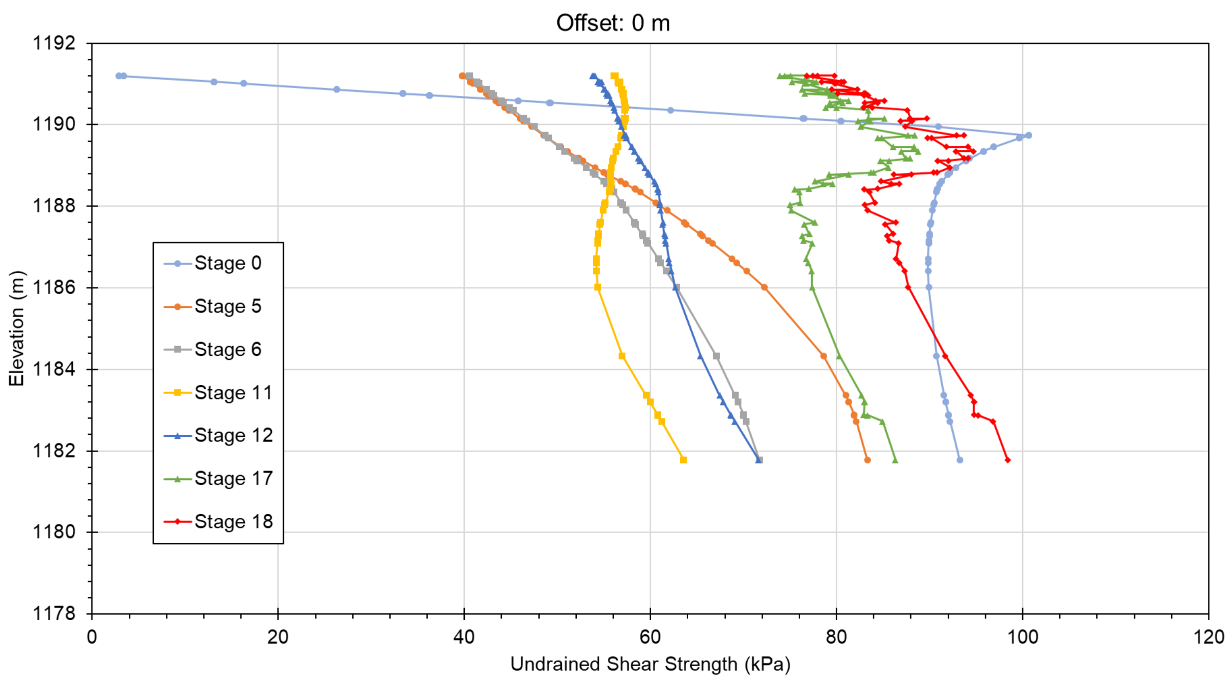


Figure 113. Change in Shear Strength during Construction at Embankment Centerline

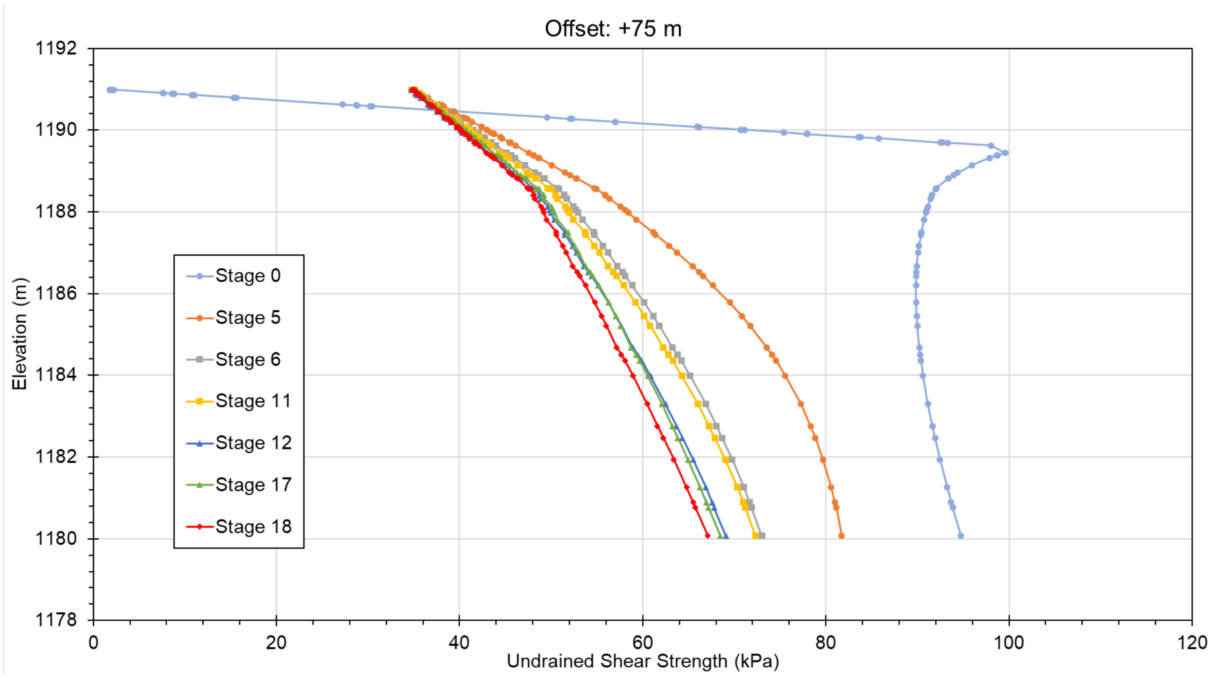


Figure 114. Change in Shear Strength during Construction at Downstream Toe

Considering these curves, it is apparent that the undrained shear strength at most depths initially decreases as the embankment is constructed. This is counterintuitive, as we would expect the undrained shear strength to increase as consolidation occurs. Eventually, the strength begins to increase once the current vertical effective stress exceeds the preconsolidation stress (that is, once the soil becomes normally consolidated).

An evaluation was undertaken to better understand why the noted reduction in strength occurs with increasing load. Computationally, the use of **Equation 10** with an exponent greater than one results in high shear strength ratios at low effective stresses (high OCR). The strength ratio and undrained shear strength then drop as the soil moves toward a normally consolidated condition at increasing effective stresses. This phenomenon does not occur with exponents less than unity, as typically expected for clayey soils. The physical reason for the high exponent (m = 1.5) for the glacial lacustrine soil is unknown but is supported by the available DSS data.

The exponential relationship for strength ratio (**Equation 10**) may introduce unrealistic trends, so another strength model was considered. Mesri (Terzaghi et al. 1996) suggested that undrained shear strength could be characterized as a fraction of the preconsolidation stress. Based on the normally consolidated ($\sigma'_v = \sigma'_p$) DSS test results, $S_u = 0.265\ \sigma'_p$ for the GL soil. At preconsolidation stresses less than about 150 kPa, this relationship underpredicts the DSS tests on overconsolidated samples. A minimum shear strength of 40 kPa was thus established. Stability of the embankment cross section was then re-evaluated using $S_u = 0.265\ \sigma'_p$, with a minimum S_u = 40 kPa, in the GL layer.

Stability results from the two soil strength models produced very similar results, with factors of safety which differed by one-half percent, or less, for the critical analyses.

12.5.3.7 Stability Analysis with Construction Period Pore Pressures

12.5.3.7.1 Construction Period Load Cases

To address the construction period slope stability, four load cases were evaluated. Specifically, two load cases representing conditions at the completion of construction season two (of three total) were evaluated for "end" of construction and also for the occurrence of an intermediate level flood containment event following the winter shutdown. The remaining two load cases represent conditions at the completion of construction season three, evaluated for the end of this construction season and for a design flood containment event immediately following construction, but prior to full dissipation of the elevated pore pressures. For the finite element method analysis method both drained and undrained stability analyses were completed for each load case.

The load cases and target Factor of Safety criteria for each analysis are summarized in Table 72. The same criteria were used for both the end of construction year 2 and the end of construction year 3. Timing for the application of each case was discussed in Section 12.5.3.5.1.

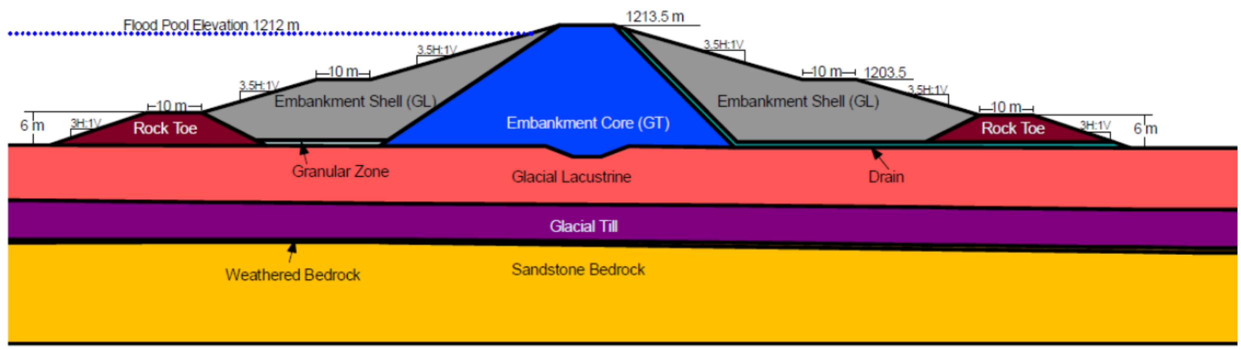
Table 72. Construction Period Load Cases

Case No.	Type	Year	Pool Elev. (m)	Analysis Direction	Target FOS
1	End of Construction	2	None	Both	1.3 (CDA)
2	Flood	2	1201.1	Downstream	1.4 (USACE)
3	End of Construction	3	None	Both	1.3 (CDA)
4	Flood	3	1212.0	Downstream	1.4 (USACE)

12.5.3.7.2 Critical Cross Section

Construction period slope stability was evaluated at three cross sections, at Station 21+750, at Station 22+500 and at Station 22+990. These locations were determined based on the segments of the embankment with significant GL foundation zones.

As discussed in Section 12.5.3.5.5, rock toes were added to obtain stability in the PLAXIS model at Station 22+500. Rock toes were not required at the other stations due to the decreased thickness of the GL soil. The 10 m bench widths were required for all analyzed cross sections. The Station 22+500 cross section - used in the stability analysis of the thickest GL foundation - is shown in Figure 115.



Note: Existing grade varies from 1191.0 to 1191.6 m at this section.

Figure 115. Critical Cross Section for the Stability Analysis

12.5.3.7.3 Approach

The analysis was completed with GeoStudio SLOPE/W using the "Spatial Function" option to represent the pore water pressure distribution predicted with PLAXIS. An example pore water pressure contour map from the end of construction year 3 is shown in Figure 116.

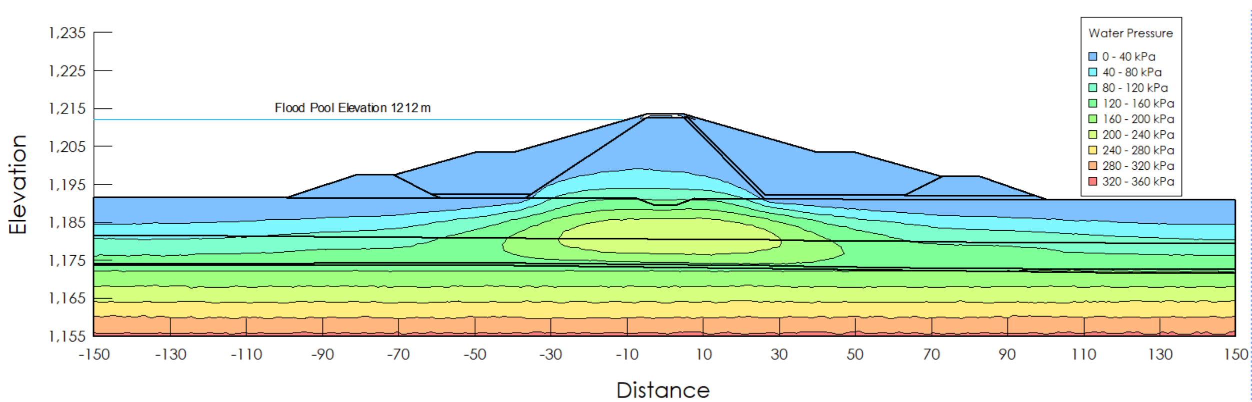


Figure 116. Porewater Pressure Spatial Function Contour Map, End of Year 3

Each of the four load cases was evaluated using both drained and undrained strength parameters. For the undrained strength for the glacial lacustrine foundation soil, S_u/σ_v' ratios were used to determine the undrained shear strength at each grid point. Shear strengths at the base of each vertical slice was then interpolated using the spatial function in SLOPE/W as discussed in Section 12.5.3.6.3. For the other materials, undrained strengths were modeled as a bilinear (drained/undrained) envelope. Selection of these strength parameters are documented in the Geotechnical Material Properties discussions in Section 5 and Section 6. Strength parameters used in the analyses are summarized in Table 66.

12.5.3.7.4 Stability Results Using Construction Pore Pressures

The critical sections were evaluated with 3.5H:1V exterior side, and with 10 m wide benches. The Station 22+500 section has a rock toe at the bottom of each slope and granular drainage zones under both shells. The toes are 6 m tall with 10 m top widths. The granular zones are 1 m thick blanket or closely spaced fingers. The upstream zone will only be used for construction drainage and will be constructed of local sourced material. The downstream blanket and associated chimney drain will be constructed of filter sand.

Failures in both the upstream and downstream directions were evaluated for the end of construction load cases (Case 1 and 3) and failures in the downstream direction only were evaluated for the flood load cases (Case 2 and 4).

Graphical output from the SLOPE/W analyses are presented in Attachment 12. The results of the analyses are summarized in Table 73. The failures tended to be large, deep translational failures that follow the glacial lacustrine foundation layer, for the upstream and downstream directions.

Table 73. Construction Period Slope Stability Analysis Results

Case	Station	Failure Direction	Factor of Safety			
			Design Criteria	B-bar Analysis	PLAXIS/GeoStudio Analysis	
				Drained		Undrained
1 - End of Construction Year 2	21+750	Downstream	1.3	-	1.7	1.9
		Upstream	1.3	-	1.3	1.4
	22+500	Downstream	1.3	1.6	1.4	1.4
		Upstream	1.3	1.6	1.4	1.4
	22+990	Downstream	1.3	-	1.4	1.4
		Upstream	1.3	-	1.3	1.3
2 - Flood Year 2	21+750	Downstream	1.4	-	-	2.0
	22+500	Downstream	1.4	-	-	1.5
	22+990	Downstream	1.4	-	-	1.5
3 - End of Construction Year 3	21+750	Downstream	1.3	1.5	1.5	1.5
		Upstream	1.3	1.3	1.3	1.3
	22+500	Downstream	1.3	1.3	1.5	1.3
		Upstream	1.3	1.3	1.3	1.3
	22+990	Downstream	1.3	1.3	1.4	1.4
		Upstream	1.3	1.3	1.3	1.3
4 - Flood Year 3	21+750	Downstream	1.4	-	-	1.5
	22+500	Downstream	1.4	-	-	1.4
	22+990	Downstream	1.4	-	-	1.4

12.5.4 Summary of Basic Slope Stability Analysis Results

The non-construction period stability analyses were performed in accordance with the standards and methodology discussed in Section 8.5 . The results of the Slope/W analysis for each design case are presented in Table 74. This is based on the storage dam geometry summarized in Section 12.1. The outputs of the stability analyses from SLOPE/W are included in Attachment 12.1.

Table 74. Stability Analyses Results – Recommended Embankment Section

Load Case	Section	Factors of Safety	
		Upstream	Downstream
End of Construction – Undrained Analysis (Target FOS = 1.2)	20+000	1.5	1.6
	21+050	1.6	1.6
	21+750	1.3	1.5
	22+500	1.3	1.3
	22+990	1.3	1.4
	23+175	1.6	1.6
End of Construction – Drained Analysis B-bar Pore pressures (Target FOS = 1.3)	20+000	1.4	1.6
	21+050	1.5	1.6
	21+750	1.3	1.5
	22+500	1.3	1.3
	22+990	1.3	1.3
	23+175	1.6	1.6
End of Construction – Drained Analysis Plaxis Pore pressures (Target FOS = 1.3)	20+000	-	-
	21+050	-	-
	21+750	1.3	1.5
	22+500	1.3	1.5
	22+990	1.3	1.4
	23+175	-	-
Long Term Drained (Target FOS = 1.5)	20+000	1.5	1.6
	21+050	1.6	1.6
	21+750	1.6	1.6
	22+500	1.6	1.6
	22+990	1.6	1.6
	23+175	1.6	1.6
Flood Load – USBR Method (Target FOS = 1.2)	20+000	-	1.3
	21+050	-	1.3
	21+750	-	1.5
	22+500	-	1.4
	22+990	-	1.6
	23+175	-	1.6
Flood Load – USACE Method (Target FOS = 1.4)	20+000	-	1.6
	21+050	-	1.6
	21+750	-	1.5
	22+500	-	1.4
	22+990	-	1.4
	23+175	-	1.6

Load Case	Section	Factors of Safety	
		Upstream	Downstream
Rapid Drawdown (Target FOS = 1.2)	20+000	1.3	-
	21+050	1.4	-
	21+750	1.5	-
	22+500	1.2	-
	22+990	1.2	-
	23+175	1.4	-
Seismic - Pseudostatic (Target FOS = 1.0)	20+000	1.1	1.0
	21+050	1.1	1.1
	21+750	0.9	1.0
	22+500	0.7	0.7
	22+990	1.0	1.0
	23+175	1.0	0.9
Seismic – Post Earthquake (Target FOS = 1.2)	20+000	1.5	1.6
	21+050	1.6	1.6
	21+750	1.4	1.6
	22+500	1.2	1.2
	22+990	1.6	1.6
	23+175	1.6	1.5

1) Seepage analysis for the USBR flood load method was conducted without the vertical drain near the core or relief system for 23+175. These features would reduce the pore pressures and improve the slope stability of this section.

12.5.5 Special Considerations

In addition to the basic and construction slope stability analysis, unique conditions were considered at several locations. These conditions included Stability of the Station 20+000 Saddle Dam, a more detailed Rapid Drawdown evaluation, and the possibility of a two-year embankment construction in the vicinity of the LLOW conduit. The following sections provide discussions of these considerations. The seismic deformation analysis is also discussed under the Special Considerations section.

12.5.5.1 Saddle Dam

The Saddle Dam to be constructed from Station 19+800 through Station 20+200 occurs adjacent to the Diversion Channel. For hydraulic considerations it is necessary that the channel slopes remain constant at 3 horizontal to 1 vertical. The saddle dam embankment is only approximately 10 m tall, but the relatively low strength glacial lacustrine clay shell was found to have inadequate stability at the channel's 3 horizontal to 1 vertical slope. Accordingly, the cross section was modified to provide a stable configuration on the upstream (channel) side. This consisted of replacement of the typical soil shell zone material with a minimum 1.5 m thickness of free draining granular material. Channel flow velocities / scour also dictated the use of some rip rap protection, with the greater rock thickness requirement driving the final slope configuration.

12.5.5.2 Rapid Drawdown

The rapid drawdown analysis presented in the basic stability analysis utilized the Duncan – Wright (2005) method contained within the SlopeW software. This methodology assumes a fully saturated embankment as the initial condition and applies the undrained soil strengths during the drawdown process. It generally provides a conservative result for typical permanent pool reservoir drawdown conditions. However, the SR1 Off-Stream Storage Dam will never contain a permanent reservoir pool. The design assumptions associated with the SR1 project call for the flood event pool to be emptied promptly after filling. This may result in two differences from normal wet reservoir conditions, which contradict implicit assumptions in the Duncan-Wright method.

First, the embankment will likely never experience a steady state seepage (fully saturated) condition. Given the very low permeability clays to be used for the embankment dam construction, and the limited duration when the reservoir is holding water, the seepage front which develops should only extend a relatively short distance into the embankment face.

Second, with no permanent pool, the upstream face of the embankment dam will remain exposed to the elements. It is expected that significant freeze-thaw action as well as desiccation drying will occur in the surficial layer of the soil. This weathering action, in addition to consistent wetting and drying due to regular pool fluctuations, could result in crack development and a general deterioration of soil strength and an increase in permeability.

To more closely model the performance of this surficial layer a transient seepage analysis was performed. Several slope stability analyses were performed, using undrained shear strengths and pore water pressures from regular intervals of the transient seepage analysis. In other undrained stability analyses, composite drained-undrained shear strength envelopes were implemented, such that the selected shear strength is the lesser of the drained or undrained shear strength. This is judged to be a conservative method of approximating the nonlinear undrained strength behavior. This approach was judged to be overly conservative in these transient stability analyses, because the applied shear strength approaches zero, as stress approaches zero. Even very soft clays should exhibit some small, but nonzero, undrained shear strength. Peck et. Al (1974) suggest that very soft clays (clays capable of being penetrated several centimeters with one's fist) have an undrained shear strength of about 12 kPa. In the transient stability analyses, near-surface soil layers were modeled with a bilinear strength envelopes; however, since the undrained shear strength is not expected to approach zero at zero stress, the modeled shear strength was not permitted to drop below 12 kPa.

The resulting model was analyzed for pool hold durations of 0-days, 30-days, 45-days, 60-days, 90-days and 120-days. The pool was assumed to fill to a flood elevation of 1210.75 m in three days. The rate of drawdown was modeled as 40 days (maximum rate LLOW conduit can accommodate), 80 days, 120 days, 240 days, 360 days and 720 days. The stability of the slope was evaluated for each seepage time step and the results compiled in a time versus factor of safety plot. To better understand the slope stability results, separate searches were performed on the upper, middle and lower slopes. An additional search was performed over the entire slope length. The results were combined to provide a composite minimum factor of safety envelope. An example is shown in Figure 117, and a full set of plots is shown in Attachment 12.1. The lowest value occurring was then considered the critical factor of safety for the combination of pool hold duration and drawdown rate.

The transient analyses were performed on both the Station 22+500 section and the Station 22+990 station (LLOW location). The Station 22+990 section, which lacked the rock toes was found to be the more critical. Results of the Station 22+990 analyses are presented in Table 75 below.

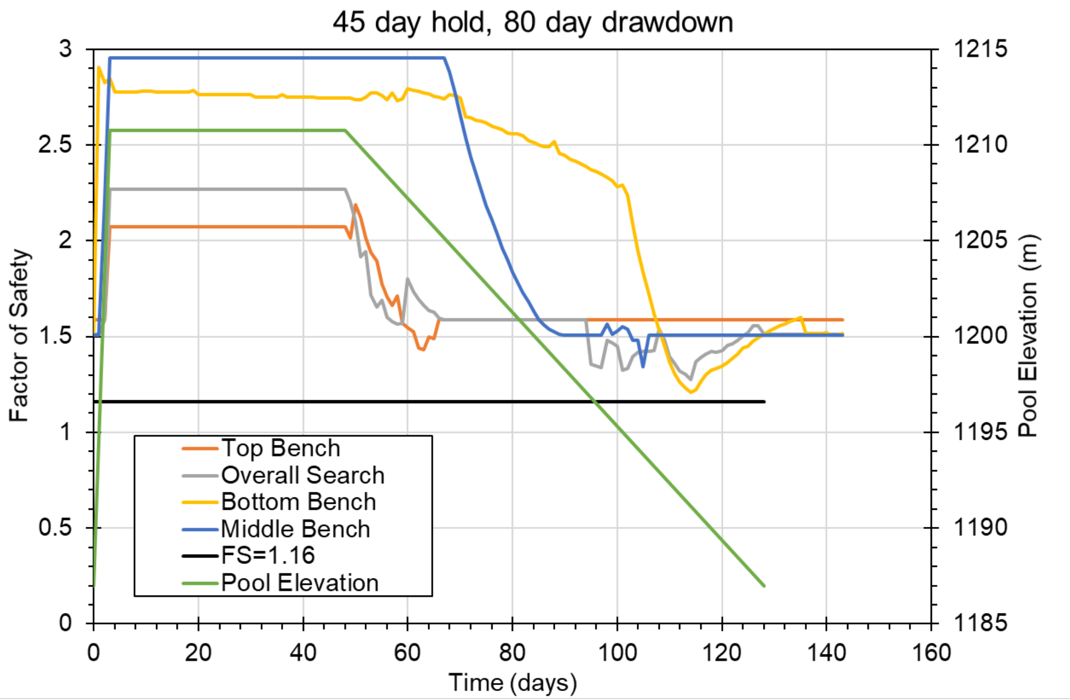
Table 75. Transient Pool Drawdown Slope Stability Factor of Safety (Station 22+990)

Drawdown Rate		Duration of Pool Hold (days)					
m/day	**Total days**	**0**	**30**	**45**	**60**	**90**	**180**
0.6	40	1.4	1.2	1.2	1.2	**1.1**	**1.0**
0.3	80	1.4	1.3	1.2	1.2	**1.1**	**1.0**
0.2	120	1.4	1.3	1.3	1.2	**1.1**	**1.0**
0.1	240	1.4	1.3	1.3	1.3	1.3	1.2
0.07	360	1.4	1.3	1.3	1.3	1.3	1.2
0.03	720	1.4	1.3	1.3	1.3	1.3	1.3

Note: Those results which do not achieve the factor of safety criteria of 1.2 are shown in red text.

On the basis of these results the pool drawdown criteria has been established as the following:

1) Pool holds up to 60 days — drawdown at 0.6 m /day
2) Pool holds greater than 60 days — drawdown at 0.1 m /day



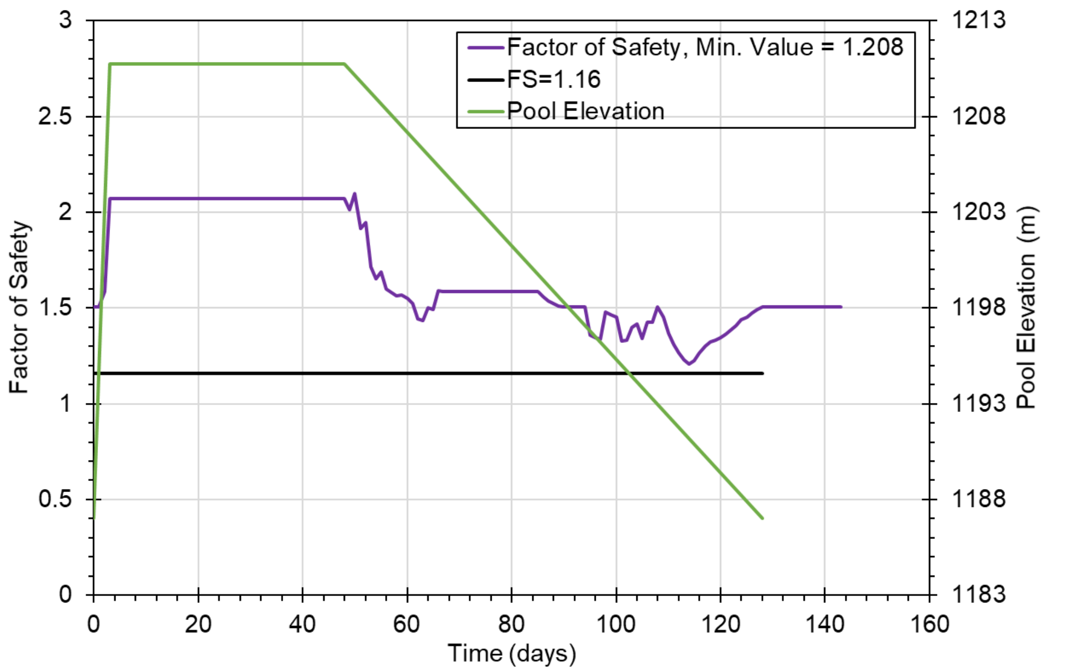


Figure 117. Example plot of transient stability results, showing results from individual searches (top) and a composite envelope of critical values at each time step (bottom)

12.5.5.3 Two-Year Embankment Construction Beside LLOW Conduit

The embankment construction, which is planned to occur over three construction seasons, will be critical to the overall schedule for the project. However, the LLOW conduit will take six to eight months to construct and during the time of its construction, work on the embankment over and directly adjacent to it will not be possible. The embankment three-year construction schedule is driven by the required time rate of construction over the glacial lacustrine clay (GL). To overcome the time rate constraint the area adjacent to the LLOW conduit will have the layer of GL soil removed and replaced with glacial clay till soil (GT). The GT soil will allow a much more rapid rate of construction, supporting the two-year embankment construction schedule required over the LLOW conduit.

To better understand the impacts of not removing the GL soil an additional analysis was performed assuming the GL layer remained in place and that the full embankment height was constructed in years 2 and 3 (2 years of construction). The results of this analysis are provided in Table 76 below.

Table 76. Construction Period Slope Stability by LLOW with GL Soil Retained

Case	Station	Failure Direction	Factor of Safety		
			Design Criteria	PLAXIS/GeoStudio Analysis	
				Drained	Undrained
End of Construction Year 2 (1 yr dam construction)	22+990	Downstream	1.3	1.4	1.4
		Upstream	1.3	1.3	1.3
Flood Year 2	22+990	Downstream	1.4	-	1.5
End of Construction Year 3 (2 yrs dam construction)	22+990	Downstream	1.3	1.4	1.4
		Upstream	1.3	**1.2**	**1.2**
Flood Year 3	22+990	Downstream	1.4	-	**1.3**

The results indicate that the two-year embankment construction narrowly misses the factor of safety criteria at the end of Construction Year 3. This section (Station 22+990) was modeled in GeoStudio as a two-dimensional figure, which implies a generally prismatic extension of the section geometry for a meaningful length. In reality the stability of this location of the embankment dam will be substantially impacted by the three-dimensional nature of the construction around the LLOW trench and backfill. It is expected that the actual field factor of safety – reflecting the realities of the geometry – will be greater than these results indicate.

The general stability analyses for Station 22+990 were based on the assumption that either 1) three construction seasons would be available for embankment construction at this location or 2) that the GL soil would be removed (undercut and replaced with GT soil) back from the sides of the LLOW trench for a distance sufficient to provide for all the late-start (year 1 & year 2 embankment placed during year 2) embankment loading stresses would fall onto the replaced GT soil rather than the GL soil. Based on the likely existing GL soil depth this distance is expected to be approximately 75 m in width.

If it becomes necessary to construct the embankment at this location in two seasons, it is recommended that GL soil be removed and replaced with GL soil. However, with the close factor of safety results and the significant three-dimensional effects at play, an area of replacement less than the 75 metres is recommended. Provided that a full length and section trapezoidal cut – with similar width top and bottom - is made with 4 horizontal to 1 vertical side slopes, and that the full GL cut is replaced with GT soil during construction season one, a 20 metre replacement cut width is recommended to support full embankment construction in seasons two and three.

12.5.5.4 Seismic Analysis

The Slope/W pseudostatic stability analyses indicated that a FOS of 1.0 was not achieved for Stations 21+750 m, 22+500 m and 23+175 m. Therefore, the magnitude of earthquake-induced permanent deformations was estimated using a Newmark sliding block analysis (Newmark, 1965). This was undertaken for the 3.5 horizontal to 1.0 vertical slope configuration. This analysis requires estimation of the yield acceleration from pseudostatic analyses and acceleration time histories from ground response analyses. The yield acceleration (k_h) to achieve a FOS = 1.0 for the storage dam was determined using Slope/W. The ground response analysis is summarized in Section 7 and in the Project PHSA Report (Stantec, 2017).

Modulus reduction and damping curves developed by Ishibashi and Zhang (1993) were used to model the nonlinear, strain dependent behavior of the foundation units and earthfill. The Ishibashi and Zhang (1993) modulus reduction and damping curves are functions of PI and mean effective stress (σ'_m).

The scaled earthquake time histories were input at the top of bedrock and the ground response analysis was used to propagate the earthquake motion from the bedrock, vertically through the foundation and earthfill dam. A ground response analysis was performed at the downstream toe, mid-slope and crest of the dam at Stations 21+750 m, 22+500 m and 23+175 m. Shear stress time histories were obtained from the ground response analyses at the location of the critical slip surface at the downstream toe of the dam, crest, and mid-slope at each cross section.

The resultant deformations derived from the Newmark Analysis are summarized in Table 77. The calculations for the Newmark deformation analysis for the 11 ground motions at each of the three cross sections are presented in Attachment 12.3.

Table 77. Deformation Analysis Results

Section	Newmark Deformation (mm) 3H:1V
21+750	40
22+500	230
23+175	20

The maximum deformation was 230 mm (0.23 m). This is significantly below the maximum accepted value of 1 m (CDA, 2007).

12.6 SETTLEMENT ANALYSIS

Settlement calculations were performed at ten embankment centerline locations between Station 20+600 and Station 23+400 where the embankment fill thicknesses are expected to vary from 6.6 m to 29.2 m. Settlement parameters used in the analyses are presented in Table 9 and Table 10 of Section 5.4.3.5.

12.6.1 Foundation Settlement Results

Total settlement estimates of the foundation soils due to embankment loading range from 144 mm at Station 21+050 to 1035 mm at Station 22+600. Settlement estimates of each soil horizon at each of the ten centerline dam locations is presented below in Table 78. Settlement figures and calculations are included in Attachment 12.2.

Table 78. Total Estimated Foundation Soil Settlement Below Storage Dam Embankment

	Embank	Glacial-lacustrine		Glacial Till		Gravel Layer		All Layers
Station	Height m	Thickness m	Settlement mm	Thickness m	Settlement mm	Thickness m	Settlement mm	Thickness m
20+600	7.4	1.9	98	11.0	50	N/A	N/A	**148**
21+050	6.6	2.8	140	1.0	5	N/A	N/A	**145**
21+350	7.0	3.0	112	1.5	25	N/A	N/A	**137**
21+650	13.7	4.3	159	2.1	53	N/A	N/A	**212**
21+975	18.3	11.3	544	1.8	78	N/A	N/A	**622**
22+290	20.4	10.1	477	5.2	237	N/A	N/A	**714**
22+600	21.5	6.1	371	13.7	664	N/A	N/A	**1035**
22+925	23.5	4.5	242	9.1	572	N/A	N/A	**814**
23+175	29.2	0	0	2.9	424	3.6	15	**439**
23+440	19.6	1.3	95	11.3	657	1.7	7	**759**

12.6.2 Estimated Embankment Settlement

Approximately 200 mm of settlement is estimated to occur within the embankment core at the tallest section (Station 22+600) based on Hunter, G; Fell, R, (2003). Prorated settlement values within the embankment were used for the other dam sections.

12.6.3 Total Settlement of Embankment Crest

Using the results of the soil foundation settlement analyses, a range of settlement values along the profile of the dam was calculated to determine the required overfill for the storage dam. The proposed overfill amount to include in the preliminary design was based on the following assumptions:

- 80 percent of the calculated foundation settlement is expected to occur after the embankment is constructed based on the B-Bar values.
- In addition to foundation settlement, the compacted embankment core is expected to settle approximately 200 mm at the tallest embankment section after construction (Hunter, G; Fell, R, (2003). Embankment core settlement was prorated to shorter embankment sections.
- The minimum anticipated foundation settlement was estimated to be 75 percent of the total settlement to account for more than twenty percent of foundation settlement during construction.
- The maximum anticipated foundation settlement was estimated to be 125 percent of the total settlement to account for less than twenty percent of foundation settlement during construction

The calculated minimum and maximum settlement values for each analyzed section are presented in Figure 118.

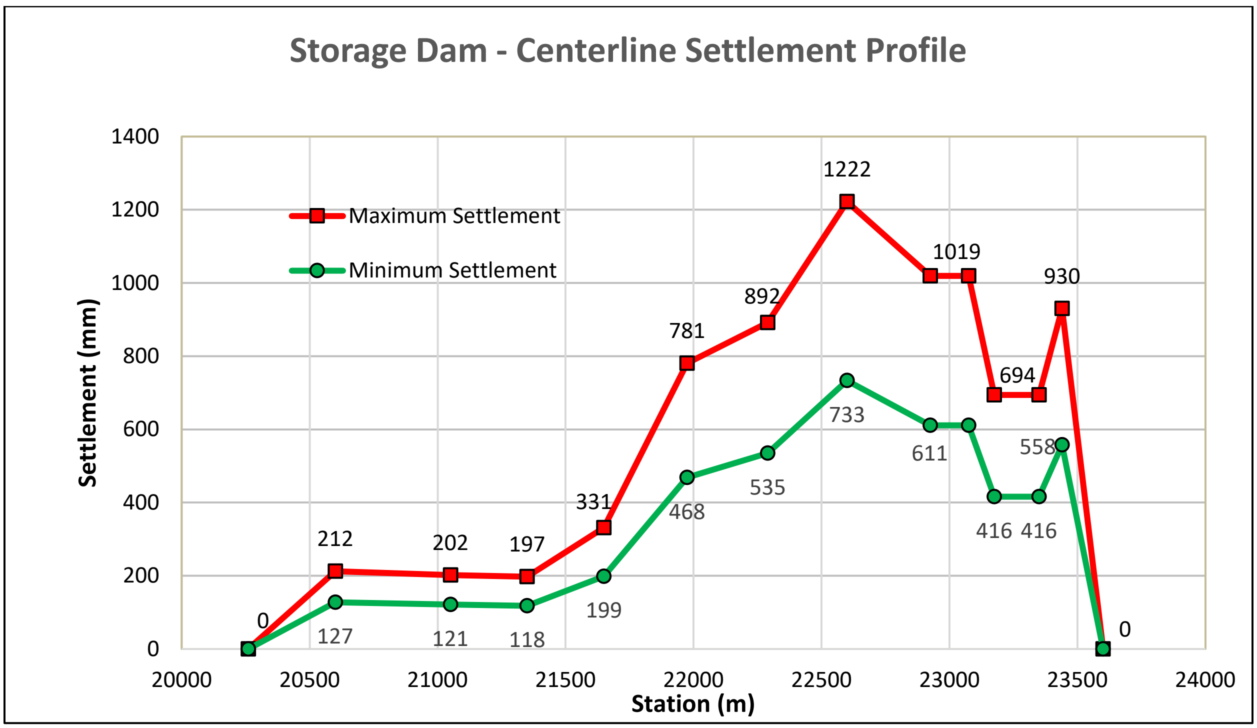


Figure 118. Storage Dam Settlement Ranges

Recommended embankment overfill values were established conservatively as approximately two-thirds of the difference between the minimum and maximum estimated settlement values (approximately 108 percent of the calculated value). A generalized overfill profile for embankment overfill design is presented in Figure 119.

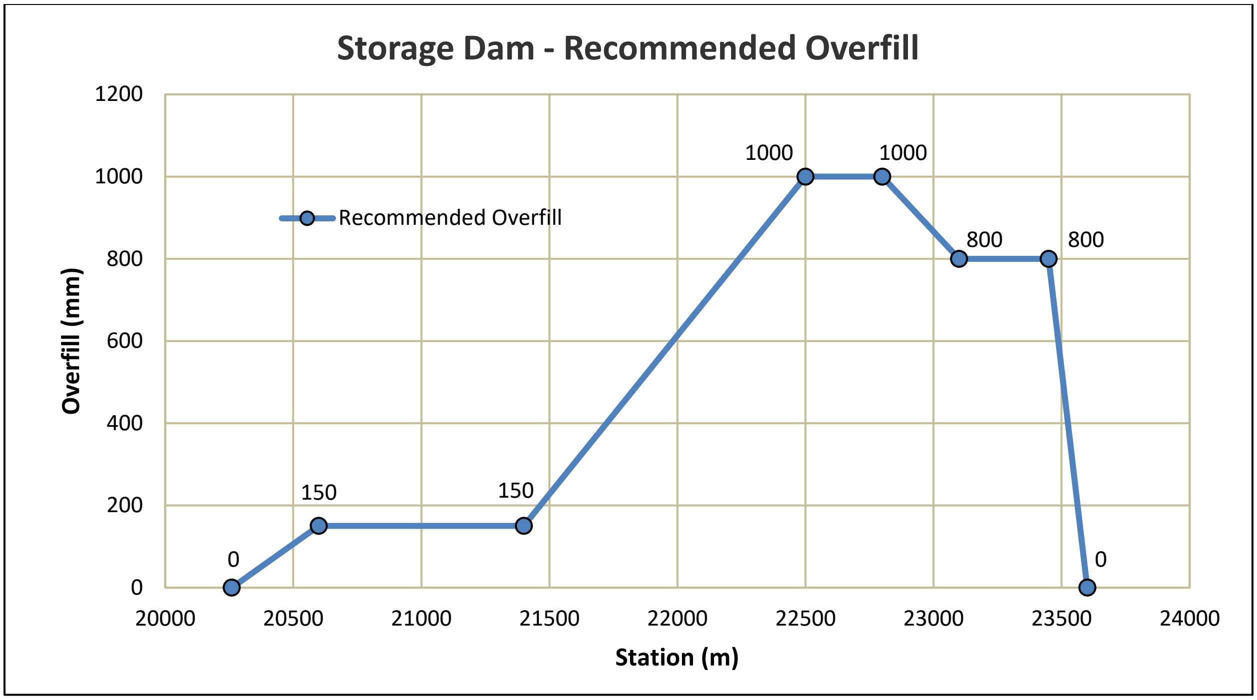


Figure 119. Recommended Storage Dam Overfill

12.7 FILTER DESIGN

Filter gradations were selected for the interface between the chimney drain for the dam core and for the interface between the blanket drain and underlying GL on the downstream side of the dam. The dam core will be comprised of low plasticity clay soil with a USCS classification of CL. The foundation soils are comprised of low and high plasticity clay soil with USCS classifications CL and CH.

12.7.1 Filter Design Methodology

Filter design was performed in accordance with USACE procedures as published in EM 1110-2-2300 (1994). The core will be constructed of Impervious Fill Zone 1A material. The existing foundation soil under the dam is typically GL with GT within the limits of the unnamed creek. Gradations for the dam core soil zone were obtained from samples from a proposed borrow area upstream of the dam and from the Diversion Channel. A total of 13 gradations were used to characterize the dam core. Gradations of the dam foundation soil were obtained from boring samples. A total of seven gradations were used to characterize the downstream foundation soil.

The coarse and fine limits of the base soil gradations were used to develop gradation limits for a filter material that will provide containment of the base soil and allow water drainage. An extrapolation of the fine portion of the base soil gradations was necessary to develop the filter criteria. The base soil gradations for the core, and foundation materials are presented in Table 79, and Table 80, respectively. Gradation plots are provided in Attachment 12.4.

Table 79. Core Soil Gradation

Core Soil Gradation			
Opening (mm)	**Sieve No.**	**Upper (Coarser) Limit (%)**	**Lower (Finer) Limit (%)**
25	-	100.00	
12.5	-	93.9	100.00
9.5	-	91.9	100.00
4.75	4	88.6	99.50
2.36	8	86.0	98.4
2	10	85.4	98.2
1.18	16	83.6	97.8
0.6	30	81.6	97.2
0.3	50	79.4	96.2
0.15	100	74.4	95.9
0.075	200	64.0	91.6
0.053	270	63.0	86.0
0.02	-	45.0	76.0
0.006	-	30.0	63.0
0.0005	-	15.00	57.00
0.00007	-	12.00	30.00
0.000002	-	5.00	15.00

Table 80. Foundation Soil Gradation

Foundation Soil Gradation			
Opening (mm)	Sieve No.	Upper (Coarser) Limit (%)	Lower (Finer) Limit (%)
25	-	100.0	-
9.5	-	94.0	-
4.75	4	93.5	100.0
2.36	8	92.8	99.9
2.00	10	92.7	99.8
1.18	16	92.3	99.7
0.600	30	91.8	99.6
0.300	50	91.1	99.5
0.150	100	89.5	99.4
0.075	200	86.5	99.0
0.028	-	82.4	96.5
0.01	-	67.0	94.0
0.00022	-	15.0	36.0
0.00017	-	14.5	31.5
0.000003	-	6.5	15.0

Filter design for the GT core soil was performed in accordance with USACE publication EM 1110-2-2300. Thirteen gradation/hydrometer tests were used to characterize the GT borrow soils for the core, and fine and coarse gradations were developed to define the gradation envelope for the core soil.

The GT core soil is classified best as USACE soil category 2. The soil 2 category, consisting of clays with 40 to 85 percent passing a #200 sieve, defines a filter D_{15} size of $\leq$ 0.7 mm. The Fine Filter 3A material D_{15} size ranges from 0.18 mm to 0.40 mm and passes stability filter criteria.

Filter criteria was also performed for an USACE category 1 soil consisting of clays with 85 percent passing a #200 sieve. This criteria requires a D_{15} (filter) size $\leq 9 \times d_{85}$ (core soil). The Fine Filter 3A material D_{15} size is less than the required 1.8 mm and 0.48 mm values and passes soil 1 category stability filter criteria.

USACE permeability criteria requires a D15 (filter) / d15 (soil) value $\geq$ 3 to 5. This value for the Fine Filter 3A and core soil is 150 or greater and passes the permeability criteria.

The CWMS defined Fine Filter Zone 3A material will provide a suitable filter for the core material. The CWMS gradation of the Fine Filter Zone 3A material is presented in Table 81. Filter calculations are provided in Attachment 12.4.

Table 81. Chimney and Blanket Drain/Filter

Fine Filter Zone 3A			
Opening (mm)	**Sieve No.**	**Upper Limit (%)**	**Lower Limit (%)**
10		100	-
5		90	100
2.5		70	95
1.25		50	80
0.630		25	55
0.315		10	25
0.160	#100	0	10
0.080	#200	0	3

While the Fine Filter Zone 3A sand will protect the GT core zone adequately, based on USACE filter guidelines, when compared to the foundation GL soils, portions of the Fine Filter Zone 3A material may be too coarse between the 0.2 mm and 1.0 mm grain sizes. However, according to Cedergren (1997), for some CL and CH soils, the 15 percent size of the filter criteria may be as great as 0.4 mm, and that filter design may be based on filtration tests where gradation curves are not approximately parallel. Based on these exceptions, we recommend that filtration tests be performed to evaluate if the readily available Fine Filter Zone 3A material would provide a suitable filter for the dam foundation materials below the drain. Should the tests on actual filter material indicate that the sand is not acceptable, either a thin layer (300 mm) of GT soil should be placed beneath the drain or an alternative (finer) filter sand should be located. A significant cost savings is likely to be realized if the standard Fine Filter Zone 3A material can be used as a suitable filter material.

12.7.2 Drain Flow Capacity

The approximate volumetric flow capacity of the chimney drain/filter was calculated as the product of the estimated hydraulic conductivity of the filter material (10E-05 m/s), and a gravity flow gradient with a 1-meter thick filter layer inclined at 1H:1V on the downstream side of the dam core. The approximate volumetric flow capacity of the chimney drain/filter was calculated as approximately 7E-6 m^3/s.

The approximate volumetric flow capacity of the blanket drain/filter was calculated as approximately 1E-7 m^3/s assuming a hydraulic conductivity of 10E-05 m/s for the filter material with a 1-meter thick filter zone.

The anticipated flow from the SEEP/W analysis for the typical section outside the unnamed creek is approximately 1E-9 m^3/s. The filter/drain capacities exceed the estimated maximum flows by a factor of 100.

12.8 EARTHWORK MATERIAL REQUIREMENTS

The embankment dam will be constructed of zones with specified material types and requirements for particle sizes, permeability, moisture content, and compaction. The Civil Works Master Specifications (CWMS) provide material and placement requirements for the following applicable embankment zones:

- Impervious Fill Zone 1A – Impervious Embankment Core and Key Trench
- Random Fill Zone 2A – Embankment Shell (Upstream and Downstream)
- Fine Filter Zone 3A – Sand / Fine Filter Material
- Coarse Filter Zone 3B – Gravel / Coarse Filter Material

These zones are discussed further in Section 6.0. The CWMS requirements for zoned materials and placement are summarized in Table 82 below.

Table 82. Alberta Transportation CWMS Embankment Zone Material and Placement Requirements

Embankment Zone	Soil Type	Max. Particle Size	Permeability	Moisture Content Limits[1]	Compaction
Impervious Fill Zone 1A	CL (PI > 7)	150 mm Min. 50% passing 80µm sieve	N/A	-2%. to +1%	200 mm max. lifts; ≥97% Std. Proctor Min. six passes roller
Random Fill Zone 2A	CL, MH, ML, CL-ML, SC, or SM-SC	150 mm	N/A	-2%. to +1%	200 mm max. lifts; ≥95% Std. Proctor Min. four passes roller
Zone 3A Fine Filter Zone	SW	Well graded sand Additional requirements on gradation	N/A	N/A	300 mm max. lifts; Min. two passes vibratory roller
Zone 3B Coarse Filter Zone	GW	Well graded gravel with sand. Additional requirements on the gradation	N/A	N/A	300 mm max. lifts; Min. two passes vibratory roller

1) Range of moisture content (%) below to above optimum as determined by standard Proctor compaction effort.

It is recommended that the CWMS be amended to allow the reuse of fill obtained from the common excavation and minimize waste. The recommended modifications to the CWMS requirements for zoned materials for use in the off-stream storage dam are summarized in Table 83 below. Changes from the CWMS standard specifications are bold and italicized. Zone 2A has been divided into subclasses based on the anticipated excavated soils from the diversion channel.

Table 83. Recommended Modifications to Alberta Transportation CWMS Embankment Fill Zone Requirements for the Storage Dam

Embankment Zone	Soil Type	Max. Particle Size	Permeability	Moisture	Compaction
Impervious Zone 1A - Embankment Core	***CL (PI > 10 & LL<50)***	***75 mm*** *(3-inch)*	***1.0E-7 cm/s*** *(maximum)*	***-1% to+3%***	200 mm (8-inch) max. lifts; ***95%*** *Std. Proctor*
Zone 2A(1)- Select Soil Embankment Shell (Soil)	*CH, CL, MH, ML, CL-ML, SC, or SM-SC*	***125 mm*** *(5-inch)*	N/A	***-2% to+2%***	*200 mm (8-inch) max. lifts; 95% Std. Proctor*
Zone 2A(2) – Non-Durable Rock and Soil Embankment Shell	*Non-Durable Rock (SDI < 85)*	*300 mm (6-inch)*	*N/A*	*Water added as directed by* engineer	*200 mm (8-inch) max. lifts; vibratory roller as directed by geotechnical engineer*
Zone 2A(3)- Embankment Rock Toes	*Durable Rock (SDI ≥85)*	*450 mm (18-inch) Max. 10% Fines*	*N/A*	*N/A*	*600 mm (24-inch) max. lifts; Two passes vibratory roller*
Zone 2A(3) - Rockfill Embankment Shell (Rock)	*Durable Rock (SDI ≥85)*	*450 mm (18-inch) Max. 10% Fines*	*N/A*	*N/A*	*600 mm (24-inch) max. lifts; Two passes vibratory roller*
Zone 3A Fine Filter Zone - Sand Embankment Filter	SW	Well graded sand Additional requirements on gradation	*1.0E-4 cm/sec (minimum)*	N/A	200 mm (8-inch) max. lifts; Two passes vibratory roller

Embankment Zone	Soil Type	Max. Particle Size	Permeability	Moisture	Compaction
Zone 3B Coarse Filter Zone – Gravel Embankment Filter	GW	Additional requirements on the gradation	*1.0E-4 cm/sec (minimum)*	N/A	300 mm (12-inch) max. lifts; Two passes vibratory roller

Note: Recommended modifications to specifications are shown in bold italicized. All specifications assign Standard Proctor compaction effort.

Recommended modifications to the CWMS requirements for zoned materials and placement include allowance of moderate to highly plastic glacio-lacustrine clay soils placed in the embankment shells with an allowable moisture content ranging from minus two percent to plus two percent. Zone 1A soil is recommended for soil moisture content ranging from minus one percent to plus three percent of Proctor optimum moisture. A new embankment Zone 2A(3) has been specified for durable rock fill within the embankment shell zones with a minimum SDI value of 85.

12.9 FOUNDATION PREPARATION

To decrease the potential for transverse cracking due to differential settlement of the embankment soil against steep foundation slopes, the foundation surface must be prepared. The planned foundation treatment consists of flattening all existing slopes within the dam footprint to 4H:1V or flatter perpendicular to the storage dam centerline and 4H:1V or flatter parallel to the storage dam centerline. This will generally need to occur within the vicinity of the Unnamed Creek, where the existing creek banks are approximately 3H:1V or steeper.

Additional foundation treatment will consist of stripping topsoil and performing proof rolls on the exposed foundation prior to placement of embankment materials. The proof roll shall be conducted with a loaded 10-wheel tandem-axle dump truck weighing not less than 15 tons or other vehicle approved by the responsible engineer. Soft spots or areas of excessive pumping or rutting shall be excavated and replaced with compacted backfill until a satisfactory foundation is achieved.

Unexpected soil conditions or geotechnical issues, such as pre-existing fill areas, organic materials, different soils, trash, wet spots, springs, etc. shall be identified during stripping and foundation preparation. The responsible engineer shall determine remedial measures to address these issues prior to embankment soil placement.

Prior to embankment placement, the responsible engineer shall inspect the foundation for acceptance.

12.10 CONSTRUCTION MONITORING PROGRAM

The purpose of the monitoring program is to monitor the performance of the storage dam during construction and compare its observed geotechnical behavior against the expected behavior.

The continuation of the instruments previously used during construction for the operational dam safety management program will be assessed after the construction has been completed and the dam is commissioned for operation.

12.10.1 Recommended Instrumentation

The proposed instrumentation layout is presented in Figure 20-18. The program will comprise the following:

- Piezometers for measuring pore pressures within the earthfill dam and foundation units. The piezometers will be a combination of vibrating wire piezometers and open standpipe piezometers. These will be installed with the screened tips within the 1st season earthfill, Unnamed Creek alluvium, GL, GT and mudstone units on the PPF. The piezometers will be provided in rows along specific sections so the upstream, centerline and downstream zones can be monitored individually. Nested instruments will be used to provide the respective pore pressures in the different formation at a single horizontal location.

- Slope inclinometers for measuring horizontal displacement within the earthfill dam and foundation units. These will be installed in rows along specific sections together with the piezometers.

- Sondex settlement systems for measurement settlement within the earthfill dam and foundation units. These will be installed in the same boreholes as the slope inclinometers. These systems utilize rings installed at increments within the borehole along corrugated drainage pipe placed outside the slope inclinometer casing. The corrugated pipe is free to compress and settle with the embankment and foundation soils. The sondex settlement systems allow for monitoring of total settlement similar to survey monuments but also provide the ability to determine the soil zones where the settlement is occurring

- Surface Displacement Monuments for monitoring movement of the dam surface. These will be installed at regular intervals on the crest and upstream and downstream benches. These monuments will be located (horizontal and vertical position) with survey grade instruments. Follow-up measurements will be taken at regular intervals – as noted in the Dam Safety Monitoring Plan – and when any issue or concerns relative to structure movements occur.

- Flow monitoring weirs to monitor any seepage within the drains and relief wells / trenches.
- Laser scanning locations are proposed to allow for development of 4D as-built records for dam safety documentation and to measure deformations.

12.10.2 Quantitative Performance Objectives

The objective of the monitoring program will be to verify concurrence of design assumptions relative to embankment movements and pore pressures with the actual quantities which occur during construction. This will be accomplished by tabulating anticipated values at different periods during construction and expected rates of change. The tabulated values will be continuously compared to actual readings.

12.10.3 Basis for Evaluating Future Dam Performance

Where the actual measured parameters (pore pressures, deformation and seepage flow) fall within the range of expected values from the geotechnical analyses, the structure will be deemed to be performing as designed. Pore pressure increases measured in the field and the rate of dissipation will be compared to the B-bar and FEM estimated values used in the design. The measured settlement will be compared to the anticipated settlement from the analyses. When measured parameters fall outside of the expected ranges (+/- 10 percent), engineering evaluation will be required to assess performance. Additional analyses and modifications to the structure or operations plan may be necessary.

13.0 LOW LEVEL OUTLET PRELIMINARY DESIGN

The Low Level Outlet (LLO) will be constructed west of the unnamed creek valley at approximate Station 23+022. The LLO is a semi-circular concrete pipe with an intake structure with sluice gate on the upstream end and an outlet structure on the downstream end. Figure 120 shows the plan and profile of the LLO structure.

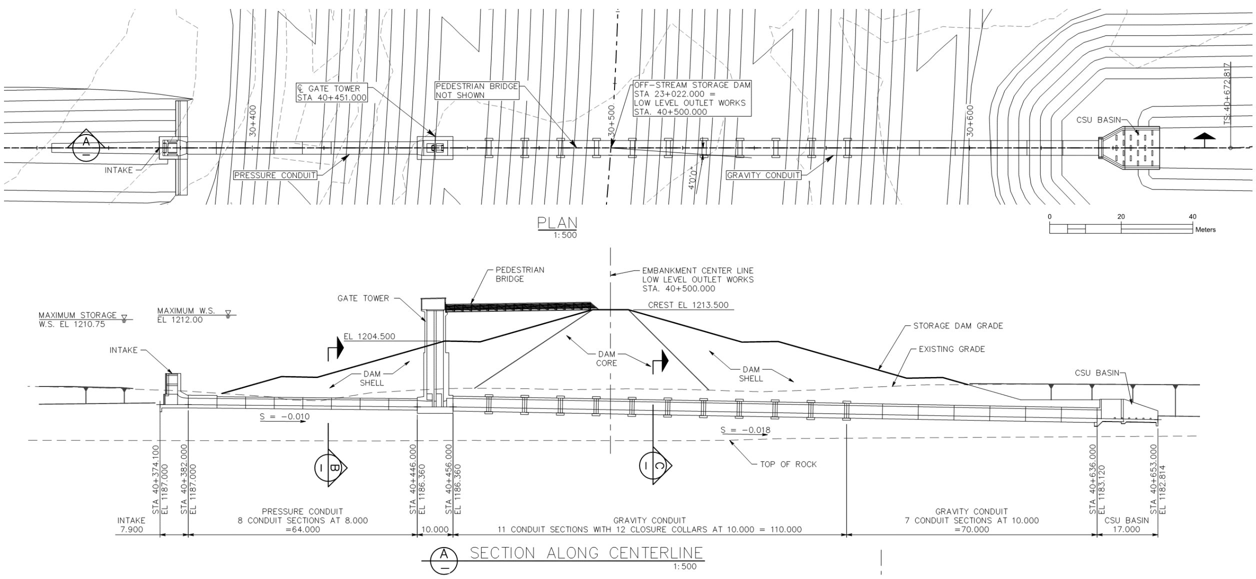


Figure 120. Low Level Outlet

13.1 SITE SPECIFIC GROUND CONDITIONS

Boreholes (D36, D58, D62, LLO03 and LLO08) were drilled along the low level outlet alignment near St. 23+022. The soils units ranged in depth from 7.3 m to 10.0 m. Top of rock elevations ranged from El. 1175.5 m to 1178.3 m. This is approximately 5 to 10 m below the low level outlet foundation. The bedrock encountered consists of sandstone and mudstone of the PPF. The top of bedrock typically consists of a layer of highly weathered, poor quality bedrock ranging in thickness from 1.5 m to 5.5 m.

13.2 RECOMMENDED GEOTECHNICAL PARAMETERS

Soil excavation will be required to construct the low level outlet conduit on a soil bearing foundation. Recommended soil and bedrock parameters to be used in the design of the low level outlet were selected based on project wide laboratory testing and are presented below.

- Allowable bearing capacity (q_a) to be determined based on final structure configuration.

- Effective Friction Angle of Soil)(φ)
  - Lean Clay Glacial Till with Sand: Φ = 27 degrees
- Effective cohesion (c)
  - c = 0 kPa (both soil layers)
- Coefficient of sliding friction (μ)
  - Lean Clay Glacial Till with Sand: μ = 0.51
- Settlement
  - See settlement analyses in Section 13.3
- Modulus of Subgrade Reaction (k) from AFM TM-5-809-12 (1987):
  - Lean Clay Glacial Till with Sand: 34 MN/m^3 (125.2 lb/in^3)

Due to potential settlement problems associated with partial soil and partial rock bearing structural elements, all individual structural elements of the low level outlet should be supported entirely by soil bearing foundations.

The low level outlet components will be backfilled with embankment core and shell soils obtained from the diversion channel excavation. The following embankment soil parameters to be used in the design of the low level outlet structure were developed based on laboratory testing and standard correlations.

Embankment Backfill Soil Parameters:

- γ_{sat} = 20.0 kN/m^3
- γ_{moist} = 20.0 kN/m^3
- Φ_{eff} = 24° (Embankment Shell)
- Φ_{eff} = 28° (Embankment Core)
- Rankine At Rest Earth Pressure Coefficient (K_o) = 0.59 (Embankment Shell)
- Rankine At Rest Earth Pressure Coefficient (K_o) = 0.53 (Embankment Core)
- Rankine Active Earth Pressure Coefficient (K_a) = 0.42 (Embankment Shell)
- Rankine Active Earth Pressure Coefficient (K_a) = 0.36 (Embankment Core)
- Rankine Passive Earth Pressure Coefficient (K_p)= 2.37 (Embankment Shell)
- Rankine Passive Earth Pressure Coefficient (K_p) = 2.77 (Embankment Core)

- Permeability k_v

  = 3.00 E -10 m/sec (Embankment Shell)
  = 3.00 E -11 m/sec (Embankment Core)
  = 3.00 E -10 m/sec (Lean Clay Till Foundation Soils)
  = 1.00 E -06 m/sec (Gravel with Sand Foundation Soils)

- Seepage Parameters and Uplift Assumptions
  - Relatively minor but unknown uplift due to non-steady state conditions upon first filling/short-term pool

- Frost Considerations
  - Recommended design frost depth = 2.0 meters
  - Non-frost susceptible backfill should consist of gravel and clean sands

13.3 SETTLEMENT ANALYSIS/SETTLEMENT PROFILE

13.3.1 Design

Settlement analyses were performed along the low level outlet alignment (Station 23+022). The dam cross section is presented in Figure 121. The proposed section consists of 3.5H:1V sideslopes with 10 metre wide horizontal benches located every 10 vertical meters. The interior of the dam consists of a low permeability embankment core and exterior embankment shells.

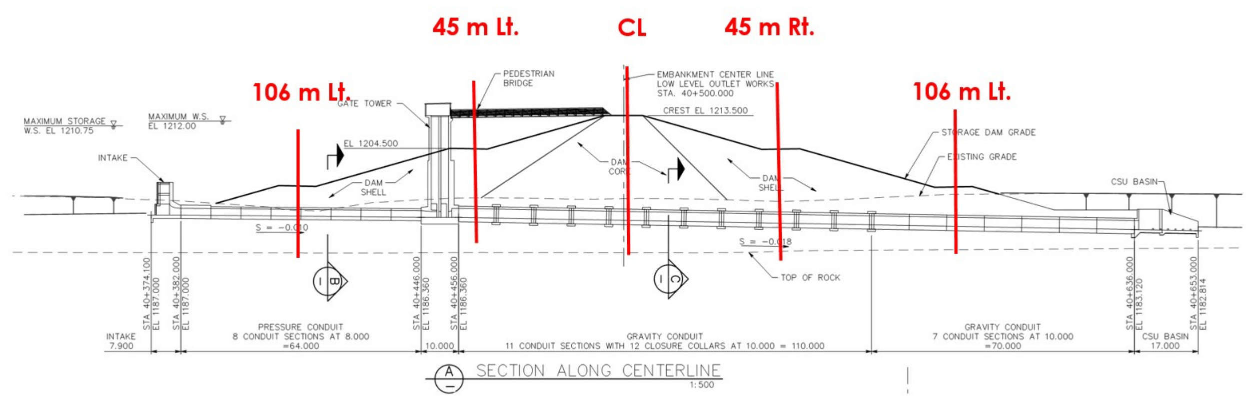


Figure 121. Storage Dam Section at Low Level Outlet

13.3.2 Laboratory Consolidation Test Results

One-dimensional consolidation tests were performed on undisturbed GT and GL samples obtained in the vicinity (borings D36, D62 and LLO08) of the low level outlet. A summary of the consolidation test results is presented in Table 84.

Table 84. Consolidation Test Results

Boring	Sample	Soil Type	Sample Depth (m)	Preconsolidation Pressure (kPa)	OCR	Cc	Cr
D36	ST10	GL	4.50-4.95	280	3.3	0.23	0.03
D62	ST6	GT	4.60-5.05	88	1.0	0.15	0.02
LLO08	ST7	GT	4.60-5.05	115	1.3	0.15	0.04

13.3.3 Settlement Analyses

An embankment crest elevation of 1213.5 m was used to calculate foundation settlements beneath the embankment at the low-level outlet spillway pipe location (Station 23+022) using the guidance from CFEM (2006).

Five soil columns (Figure 121) were analyzed below the storage dam crest and below the upstream and downstream slope benches located at elevations 1193.5 m and 1203.5 m. The results of the analyses are included in Table 85. Settlement calculations and figures are included in Attachment 13.1.

It is anticipated that the spillway pipe will experience less settlement than the full embankment section due to the foundation soil excavation required to install the low-level outlet spillway pipe. The estimated settlements below the spillway outlet pipe analyzed along the low-level outlet section are included in Table 85.

Table 85. Total Estimated Foundation Soil Settlement Below Spillway Outlet Pipe

LLO Station	Embankment Height m	Lean Clay with Sand Layer (GT)	
		Thickness m	Settlement mm
139 m Lt.	0	10.3	0
106 m Lt.	6.0	6.3	254
45 m Lt.	12.8	7.8	245
CL	23.3	7.0	411
45 m Rt.	11.8	5.3	183
106 m Rt.	6.0	6.1	126
139 m Rt.	0	6.1	0

The estimated foundation soil settlement beneath the spillway pipe due to embankment loading ranges from zero at the upstream and downstream toe to 411 mm below the dam centerline. A spillway conduit camber of approximately 50 % of the maximum estimated settlement value below the spillway pipe is recommended for design. Therefore, a maximum conduit camber of 200 mm is recommended for the spillway pipe. The spillway profile should be designed with a smooth form such as an arc with the camber heights based on approximately 50 % of the estimated settlement values presented in Table 85. The profile should be designed to be sloped to drain at the end of construction through the end of long term settlements.

The conduit foundation should be inspected and approved by a geotechnical engineer prior to conduit installation. Soft, yielding, or unsuitable soils should be removed, undercut and backfilled with suitable embankment soils under the supervision of a geotechnical engineer. The backfill material should be compacted following the same requirements as the earth embankment (Section 6.3).

The spillway conduit installation will require an excavation ranging in depth from 3.9 meters to approximately 7.6 meters below existing grade. The width between the excavation sides and the conduit must be wide enough to allow power compaction equipment to operate beside the conduit during backfilling. Details of the embankment foundation grading and preparation will be provided in the Civil Plans.

13.4 LATERAL EARTH MOVEMENT

13.4.1 Introduction

Extension of the low level outlet conduit could be caused by lateral deformation of the embankment. Two methods have been used to estimate the potential lateral dam deformation and extension of the LLO: Duncan and Walker (1984) and the United States Department of Agriculture (USDA, 1969). These methods and the predicted lateral deformations are described in the following sections.

13.4.2 Walker and Duncan (1984)

Lateral deformation or bulging of embankment dams is more common in embankments constructed with impervious materials compacted at moisture exceeding optimum moisture content (Walker and Duncan, 1984). In addition to contributing to conduit extension, excessive lateral deformations can lead to lateral cracking along the embankment slopes.

Walker and Duncan (1984) state that lateral bulging is most prevalent for embankments constructed on rigid, unyielding foundations. Embankments constructed on yielding foundations (soil) settle during construction, causing an inward rotation of the slope surface that could negate lateral deformation (Walker and Duncan, 1984).

The simplified procedure for estimating the extent of lateral bulging at the end of construction developed by Walker and Duncan (1984) was intended for dams with rigid, unyielding foundations. The SR1 foundation would not be considered rigid. Therefore, the predicted lateral deformations from this simplified procedure may over estimate lateral deformation.

Inputs to the simplified procedure include the end of construction factor of safety, the embankment side slopes, embankment height, compacted unit weight, degree of saturation of the embankment fill, the unconsolidated, undrained shear strength of the embankment fill at a depth of 3/4H below the dam crest, and the strain level at 50 percent of the unconsolidated, undrained embankment shear strength. The simplified procedure provides an estimate of the end of construction lateral deformation at the mid-height of the dam.

The lateral bulging in this procedure is estimated from the following equation and parameters:

$\Delta x = K_s K_m E_{50}(YH^2/CF^2)$ = Lateral bulging at the mid-height of the dam

K_s = Embankment saturation factor that varies from 0.95 for 100 percent saturation to 0.35 for 75 percent saturation

K_m = Embankment side slope factor = 0.9 for 3.5H:1V Side Slope

E_{50} = Strain level at which 50 percent of the undrained strength is mobilized

Y = Unit weight of embankment material = 20.0 kN/m^3

H = Embankment height = 29.0 m

C = Undrained Unconsolidated (UU) shear strength with a confining stress equal to ¼ of the mean vertical stress in the dam

F = Factor of safety at the end of construction = 1.3

The reasonably well known parameters used in this procedure include Km, Y, H, and F, and the values of these four parameters are provided above. Some of the parameters that are not currently well known include Ks, E50, and C. A best estimate and a conservative estimate has been made for these three parameters based on our experience with similar materials. These parameter estimates and corresponding estimates for lateral bulging are provided in Table 86 below.

Table 86. Lateral Bulging Parameters and Estimates

Parameter	Best Estimate	Conservative Estimate
K_s	85% Saturation (Ks = 0.59)	100% Saturation (Ks = 0.95)
E_{50}	0.01	0.03
C	120	70
Δx = Lateral bulging at the mid-height of the dam (m)	0.45	2.4

As shown in Table 86, the estimated lateral bulging at the mid-height of the dam estimated using this procedure varied from approximately 0.45 m for the best estimate and 2.4 m for the conservative estimate.

According to the USBR (2011), lateral deformation patterns are maximum on the dam side slopes and approximately mid-height in the dam, and decrease to a lower value at the base of the dam. Based on the USBR diagrams showing lateral deformation patterns (USBR, 2011), a 2/3 reduction in the mid-height lateral deformation appears to be a conservative estimate of the lateral deformation at the base of the dam. 1/3 of the conservative estimate of lateral deformation is equal to 0.8 m). This value of lateral deformation is assumed to occur from the crest to the upstream toe of the dam, and from the crest to the downstream toe of the dam.

Including the headwall joint, there are 14 joints in the low level outlet between the crest and the upstream inlet, and 14 joints in the low level outlet between the crest and the downstream outlet. Assuming the lateral deformation is linearly distributed along the length of the low level outlet, lateral extension at each joint would be approximately 60 mm. If the lateral deformation is not linearly distributed along the length of the low level outlet, the lateral extension at some of the joints could be larger than 60 mm.

13.4.3 United States Department of Agriculture (USDA, 1969)

The USDA published a simplified method for calculating the required joint extensibility for conduits at the base of a dam (USDA, 1969). The input parameters for this simplified method include foundation settlement, dam width, dam height, compressible foundation thickness, embankment unit weight, undrained shear strength of embankment material, and conduit dimensions. This procedure provides estimates of joint opening due to conduit rotation and lateral strain.

The required Joint extensibility using this procedure is estimated with the equations and parameters shown below. Note that this design procedure has been developed using specific parameters that require Imperial units. SI units cannot be used directly in the calculations. However, the final results are converted to SI units as shown below.

Two values of the undrained shear strength for the foundation material have been used in this procedure to provide a best estimate and a conservative estimate of the required joint extensibility. The parameters and equations used in this design procedure are presented below.

J = Required joint extensibility = $g_s + g_r + S$

g_s = Maximum probable joint opening due to foundation and embankment strain

g_r = Probable joint opening due to joint rotation

S = Safety margin (0.5 inches minimum)

H = Embankment height = 96 feet

B = Embankment width = 748 feet

d = Depth of compressible foundation = 21.5 feet

Y_m = Moist unit weight of embankment material = 127 lb/ft^3

s = Undrained shear strength of foundation at end of construction = 2,090 psf (Best Estimate), 1,500 psf (Conservative Estimate)

δ = Maximum anticipated settlement of foundation = 1.4 feet

D = Inside diameter of conduit = 9.2 feet

D_o = Outside diameter of conduit = 14.9 feet

L = Length of monolithic conduit section = 29.5 feet

R_1 = Theoretical ratio of maximum unit horizontal strain to average unit vertical strain – from chart below (function of B, d, and H) = 0.082

R_2 = Correction factor = $2Y_mHd/sB + 0.1$ = 0.44 (Best Estimate), 0.57 (Conservative Estimate)

ϵ_{hm} = Maximum unit horizontal strain = $R_1R_2\delta/d$ = 0.0023(Best Estimate), 0.003(Conservative Estimate)

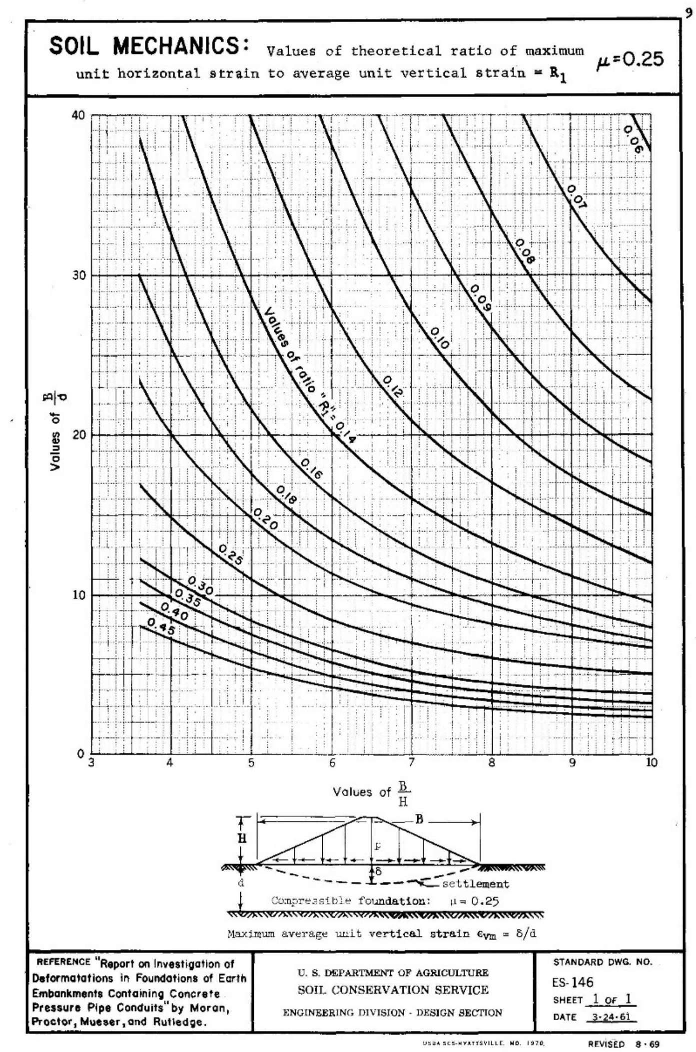
$g_s = \epsilon_{hm}DL$ = 0.83 inches (Best Estimate), 1.07 inches (Conservative Estimate)

$g_r = 2.5D_o\ \delta/B$ = 0.84 inches

S = Safety margin = 0.5 inches

$J = g_s + g_r + S$ = 56 mm (2.2 inches) (Best Estimate)

$J = g_s + g_r + S$ = 61 mm (2.4 inches) (Conservative Estimate)



[redacted]

Figure 122. Storage Dam Section at Low Level Outlet

As shown above, the estimated required joint extensibility using this simplified design procedure is 56 mm (2.2 inches) using our best estimate of the parameters and 61 mm (2.4 inches) for our conservative estimate of undrained shear strength. Based on the various unknowns and variability of the other parameters used in the simplified design procedure, we consider that it is appropriate to apply a safety factor of two to the conservative estimate. Accordingly, the estimated required joint extensibility using this design procedure is 120 mm.

13.4.4 Conclusions

The two simplified methods used to estimate lateral extension of the LLO and lateral dam displacement predicted lateral deformation of 70 mm and joint opening of 61 mm. The similarity in the results using two simplified methods provide some confidence that the estimated lateral extension is reasonable. Based on the various unknowns and variability of the parameters used in the simplified design procedures, we consider that it is appropriate to apply a safety factor of two to the estimates. Accordingly, the estimated joint extension that should be incorporated into the design of the LLO is 140 mm.

13.5 CONSTRUCTION REQUIREMENTS

During construction, the excavation and foundation preparation for the low level outlet must be reviewed by responsible engineer prior to construction. If conditions are different than those anticipated and used in the analyses, modifications or foundation improvement measures may be required. This may include over excavation and replacement with suitable fill material, concrete, or flowable fill.

Special care will be required when compacting the embankment soils around the Low Level Outlet to reduce the risk for piping along the conduit. Also, a diaphragm filter has been included around the conduit downstream of the low permeability core to intercept seepage along the conduit and protect against piping.

14.0 RESERVOIR

The planned reservoir for the off-stream storage dam has an approximate surface area of 7,900,000 m^2. The reservoir will inundate Springbank Road when the system is operational and mitigating a flood event. The Highway 22 will be raised as part of the project to avoid flooding during operations.

14.1 GEOLOGICAL INFORMATION

Geotechnical information from the 13 hydrogeology (GW1 to GW13) and five borrow site boreholes (BS1 to BS5) was reviewed to characterize the ground conditions within the reservoir site. These boreholes indicate that the stratigraphy is similar to that encountered beneath the storage dam. In the lower areas of the reservoir (below approximate El. 1200 m), approximately 3 to 5 m thick GL is underlain by GT. Above El. 1200 m, there is no GL and only GT is present. The bedrock extents for the project site are shown on Figure 20-14.

The Unnamed Creek runs through the proposed reservoir. It is assumed that alluvium similar to that encountered within the storage dam footprint would continue upstream. However, the valley appears to become broader and shallower.

14.2 POTENTIAL FOR RIM INSTABILITY

The reservoir rim has relatively flat slopes (4 to 8 percent) with some steeper slopes (up to 4H:1V) along the eastern edge near the storage dam and along the western edge north of the diversion channel outlet. Four cross sections were analyzed for slope stability, two along the eastern rim and two along the western rim. Section locations are included in Figure 123.

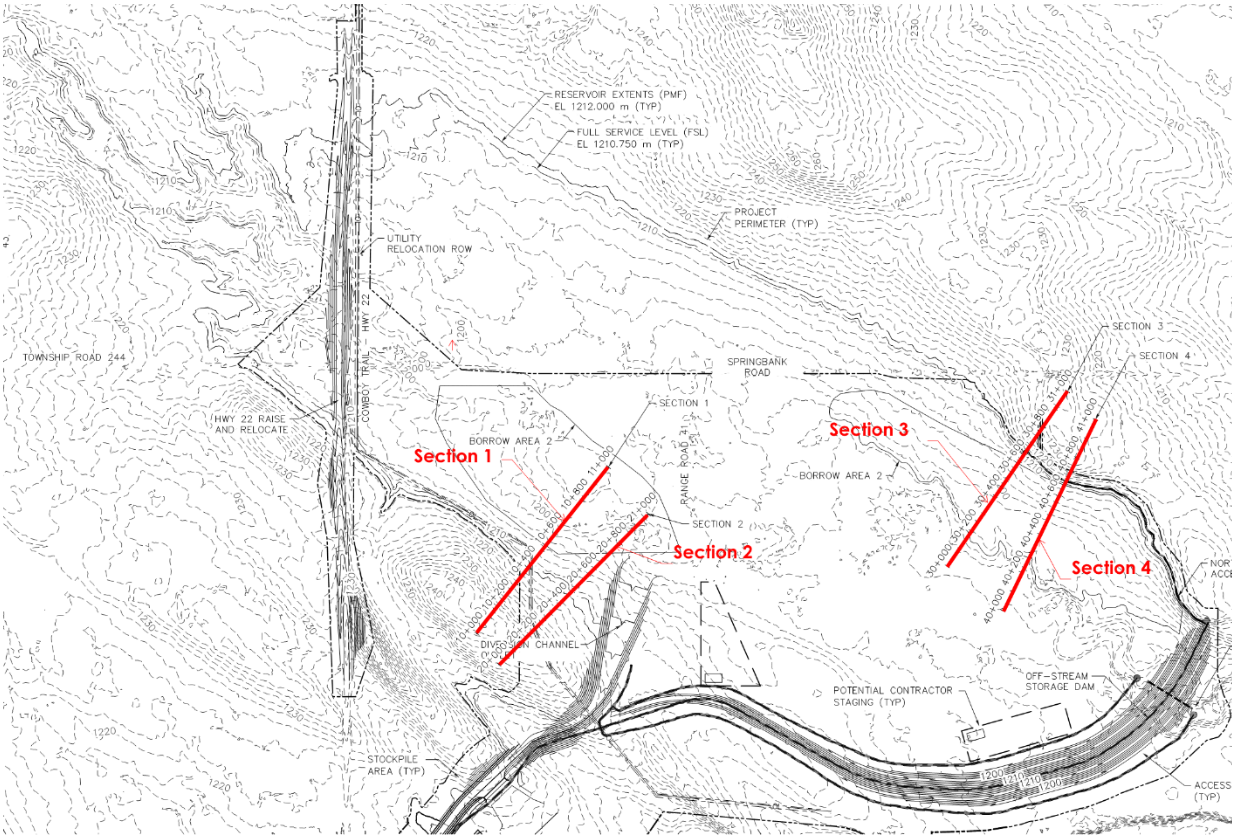


Figure 123. Reservoir Rim Cross Section Locations

Failures through both the glacigenic units and underlying PPF bedrock were evaluated. Because of the relatively flat slopes, the resultant factor of safety was 1.7 or greater for all failure modes analyzed.

14.3 POTENTIAL FOR LEAKAGE

Given the infrequent occasions when the pool with contain significant and the generally low hydraulic conductivity of the prevalent GL and GT, it is not anticipated that leakage out of the reservoir footprint will become an issue.

15.0 UTILITY DIVERSIONS

The SR1 Project site covers a relatively large area. Within this area, numerous utilities will conflict with the proposed construction and/or operation of the flood control system. Conflicting utilities will be relocated as a part of the overall project. The utilities include oil and natural gas pipelines, above and below grade power lines and various communication lines. Residential water service lines also exist in some locations. While the individual utility operating organizations will remove and reconstruct the lines, in certain instances the details used in those relocations are significant to the performance of the SR1 Project.

15.1 RELOCATION PROCEDURES AND ROUTING

The utilities that will remain after project construction will be located over, through or around the SR1 corridor. The majority of these utilities will be reconstructed near their current alignment, with adjustments as required to pass over/under the new surfaces or as needed to facilitate the actual change over from old to new elements. In the vicinity of the Storage Dam, however, the utilities will be removed from the embankment dam's footprint and will be rerouted to a location beyond the end of the dam.

15.2 RELOCATION INSTALLATION DETAILS

Two methods will be used for buried utility relocation, cut / cover direct burial and Horizontal Directional Drilling (HDD) boring. At the present time the direct burial methodology is anticipated for all relocations, except for the three oil pipelines currently located near Storage Dam Station 21+300 to 21+500.

HDD Bore - The oil pipe lines currently under the dam will be relocated to a point beyond the western limit of the storage dam. While the specific details will be established by the operating company, the conceptual installation consists of a bore passing below both the Diversion Channel excavation and the Storage Dam Baseline in the area of Diversion Channel Station 14+000 and Storage Dam Station 20+300. The bore will be approximately 600 m long, beginning on the slope below the storage dam baseline and ending on the hill side above the channel. The entry slope is 21.3 percent (12 degrees) for 118 m, the bore then passes along 251 m of an arc with a radius of 600 m, then an exit slope of 21.3 percent for 240 m to the exit point. The bore would be about 49 m below the existing surface at the channel centerline. This places the pipes approximately 28 m below the channel flowline elevation. The bore could consist of either a single bore large enough for all three pipes or two parallel bores with the pipes installed separately.

To ensure that the pipe bore(s) will not result in leaks from the flood reservoir pool, the bore will need to pass through low permeability mudstone / claystone for the saturated length and/or will need to be post installation pressure grouted. Additional drilling is currently proposed to identify formations to be encountered at the HDD bore location.

The presence of the existing pipelines within the embankment dam footprint will be a dam safety issue. These pipes will have to be removed and their trenches carefully prepared and backfilled according to project specifications prior to subsequent embankment construction in that area.

Cut / Cover Installations - The other buried utilities, which occur along the Diversion Channel alignment, are expected to be installed below the proposed channel section using a direct cut and cover method. The relocated pipelines and/or cables will be placed in excavated surface trenches. The installation will be similar to typical utility installations, except for the bedding and backfill requirements in the vicinity of the channel and dam.

To ensure that significant seepage and/or scour will not become a concern, all utilities will be installed with a minimum cover of 2 meters to the channel flowline. The backfill material around the conduits or pipes will be select material, placed and compacted in thin lifts. Additionally, the surface in the base and lower portions of the Diversion Channel will need to provide sufficient resistance to scour. Depending on final channel lining plans, additional armoring may be required at some locations.

Rock excavation will be required in many locations to install the utilities in a trench.

15.3 GENERAL CONSIDERATIONS

The design and details for the relocated pipelines will be prepared by the operating utilities, or their consultants. However, the nature of the installations are critical to the performance of the SR1 Project elements. Accordingly, the design and installation specifications for utility relocations must be reviewed and approved, and the construction process will have to be observed by SR1 project to verify specification compliance. Cathodic protection may be required to ensure significant deterioration of the piping system does not occur in the vicinity of critical SR1 Project elements.

16.0 TRANSPORTATION INFRASTRUCTURE

Thirteen borings were advanced for the two bridges (Highway 22 and Township Road 242) and the Highway 22 embankment construction necessary for the SR1 project. The two bridges are located at approximate Diversion Channel Stations 11+460 and 12+445. Geotechnical information obtained from these borings was used, where appropriate, in developing the geological model and geotechnical parameters used for the dams and diversion design.

17.0 CONSTRUCTABILITY AND SCHEDULE

Evaluation of the likely excavation material quantities indicates that the total amounts of each material type (potentially supplemented by designated borrow areas) will provide the required quantity of embankment construction.

17.1 AVAILABILITY AND USABILITY OF MATERIALS

The majority of the earth and rock materials may be broken into four general categories, moderate to high plasticity clay (glacio lacustrine clay), low plasticity clay (glacial clay till), durable rock (predominantly sandstone) and non-durable rock (predominantly mudstones and shales). The overall soil quantity and the division between main soil types can be characterized by the completed borings. However the split between durable and non-durable rock is difficult to establish with confidence prior to mass excavation. Additionally, the practicality for removing these materials separately for different uses is not yet well understood.

The design of the embankment dam utilizes the low plasticity glacial clay till for the low permeability core zone, while the glacial lacustrine clay will be placed in the upstream and downstream shells. Acceptable non-durable and/or durable rock material may also be incorporated into the embankment shells. A portion of the durable rock will be used to construct the rock toe zones. Some of the glacial clay till will be transported across the river to construct the core of the Flood Plain Berm and the glacial till will be used as backfill around the various concrete structures. The available rock material and glacial soil will be used for the Highway 22 embankment across the reservoir pool area.

Based on the current projections, it appears that some soil (lacustrine clay and/or glacial till) may need to be sourced from the identified borrow area and that some amount of non-durable and/or mixed rock material may be placed in the identified spoil storage areas.

17.2 SEQUENCING OF EXCAVATION AND EMBANKMENT CONSTRUCTION

Although the volume of materials available from the proposed excavations appear to generally match the requirements for the embankment zones, the sequencing of the two efforts may not align. The embankment must be constructed as a combined unit (all zones) and must generally be built upward in horizontal stages for its full length. Likewise the excavation will start at the ground surface and proceed downward and horizontally. Requirement for positive drainage of the excavation may limit where the work can progress, unless active pumping is utilized to remove ground water and surface water inflow and maintain water free excavations.

The earthwork contractor will be responsible for sourcing and approximately placing the designated materials. Careful preplanning will be required and the plan will have to be adaptable to variations in the material coming from the excavation. Temporary stockpiles may be required.

17.3 SEASONAL LIMITATIONS ON EARTHWORK CONSTRUCTION

The channel excavation may be conducted in most any weather condition, although very wet or cold periods may impact contractor productivity. However, the embankment construction will require careful soil placement and compaction during acceptable weather conditions. Overly moist or frozen materials will not be permitted in the embankments. Accordingly, embankment construction will be limited to dry and above freezing periods. This requirement will likely reduce the period for placement of dam material to around five or six months each year.

17.4 RATE OF STORAGE DAM EMBANKMENT CONSTRUCTION

The Storage Dam footprint is largely on glacio lacustrine clay soil. This soil, while moderately overconsolidated, is relatively slow draining and will be subject to excess pore pressure buildup as the embankment construction progresses. The lacustrine clay soil has a relatively low shear strength which, in conjunction with high pore pressures, puts the construction at risk of foundation failure should embankment loading occur too fast. A stability analysis has been performed to check construction stability. Monitoring of instrumentation during construction will allow confirmation that pore pressures remain within an acceptable range.

The design addresses the rate of loading stability issues in two ways; 1) rock toe zones are included to improve slope stability and 2) the embankment construction is designated to be performed in three distinct phases during three separate construction seasons. The entire embankment, including all internal zones is to be constructed to elevation 1198 during the first construction season, constructed to elevation 1204.5 m during the second construction season with the balance of the embankment and the toe buttress completed during the third construction season.

It should be noted that a foundation / slope failure of the Storage Dam embankment during construction would not represent any Dam Safety risk. It would, however have the potential to pose a life-safety risk to construction staff and could seriously impact project schedules and budgets.

18.0 SUPPLEMENTARY INVESTIGATION

Some areas of the project site were not available for equipment access during the field exploration due to property access constraints. Additionally, the geotechnical fieldwork occurred before the full development of the preliminary design. As the design progressed, structures and features were revised, critical areas and added design drivers were identified, and subsequent data gaps were noted. A supplemental geotechnical exploration will be required to address data gaps and verify critical assumptions prior to completion of the final design. The necessary additional work is discussed for each element in the subsections below. While generally consistent soils occur at the site, critical variations in the thicknesses and properties were determined to be significant.

18.1 UPSTREAM TOE OF DAM – STATION 21+000 TO STATION 22+500

Eleven additional borings are planned as shown in Figure 124 below. The goal of these borings is to identify depth to rock, determine the thickness of the glacio-lacustrine layer, and determine the presence of any materials different from the current assumptions of the foundation soils in this area. Sampling should be conducted to determine soil horizon breaks and classify soils encountered.

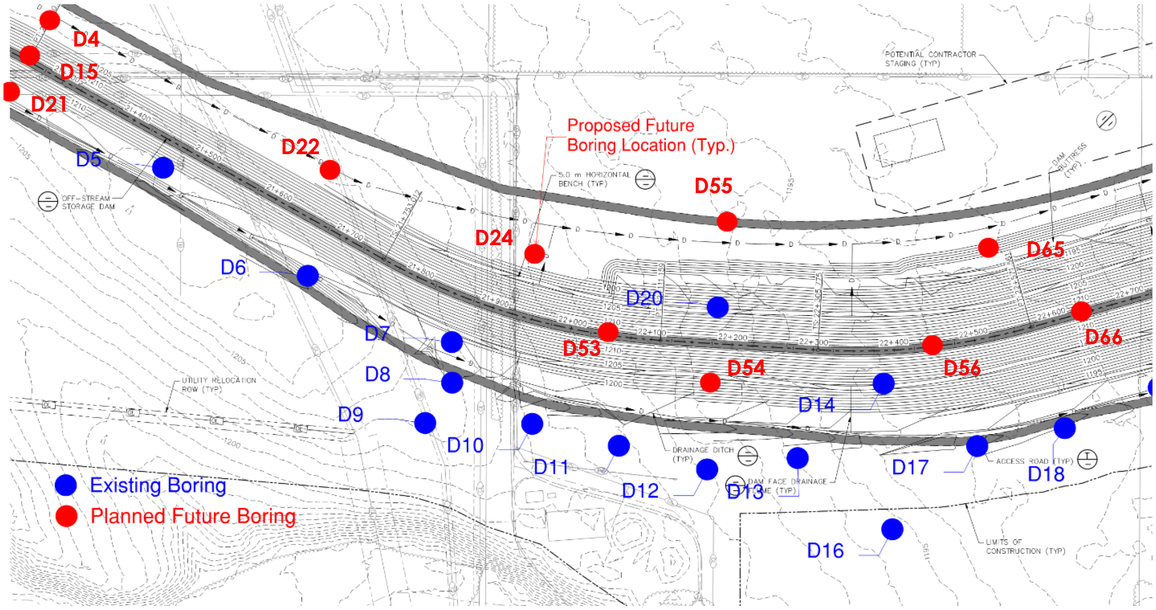


Figure 124. Off-Stream Storage Dam Additional Borings

18.2 EMERGENCY SPILLWAY

A total of twelve additional borings are planned along two possible locations as shown on Figure 125 below. The purpose of the borings is to determine depth to rock for the emergency spillway foundation and characterize the soil/rock materials in the outlet channel of the spillway for erosion control requirements.

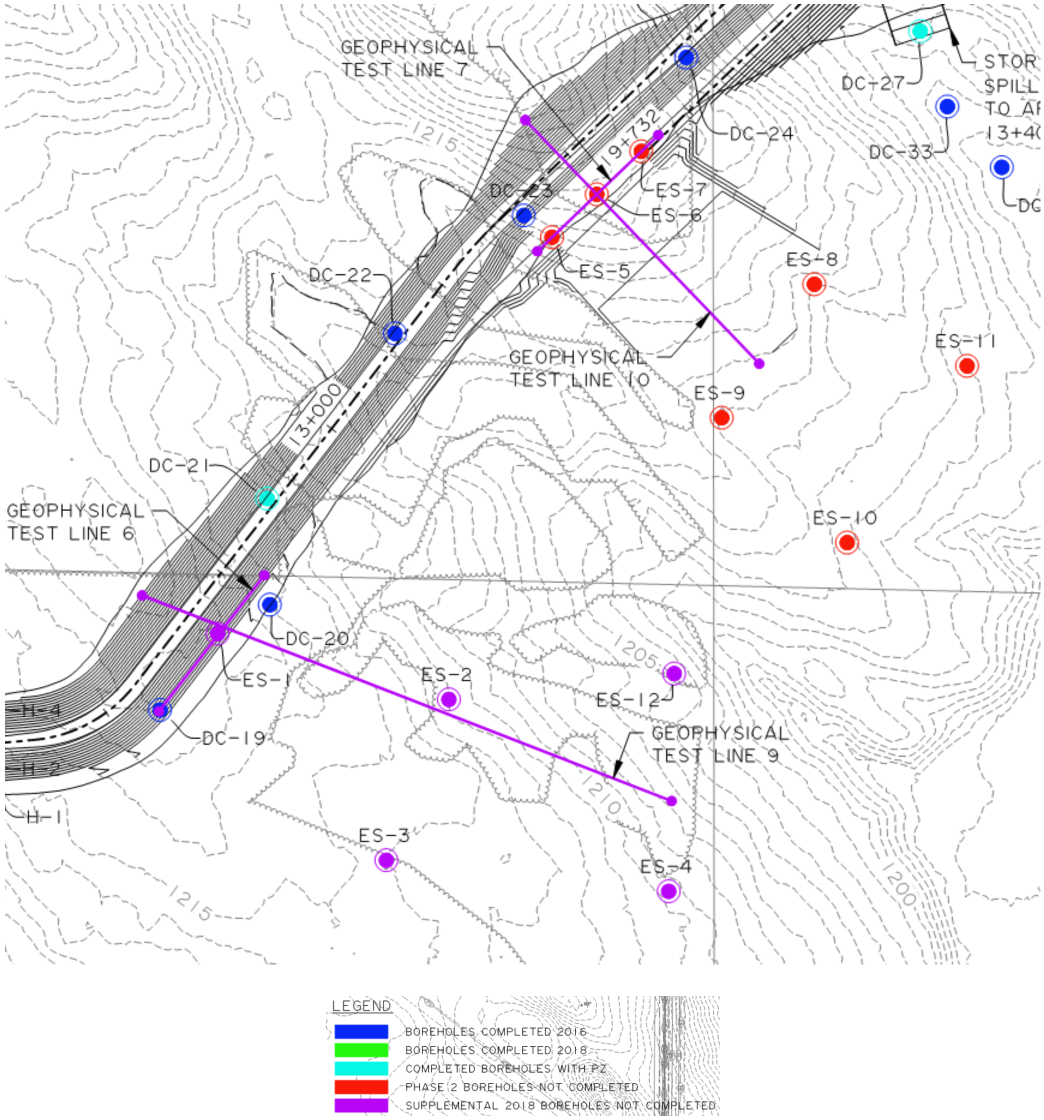


Figure 125. Emergency Spillway Additional Borings

18.3 DIVERSION CHANNEL OUTLET

An additional two borings are proposed for the RCC overlay location to determine foundation material characteristics and depth to rock. Two other borings are proposed along the HDD bore crossing under the diversion channel. These borings are shown in Figure 126 below.

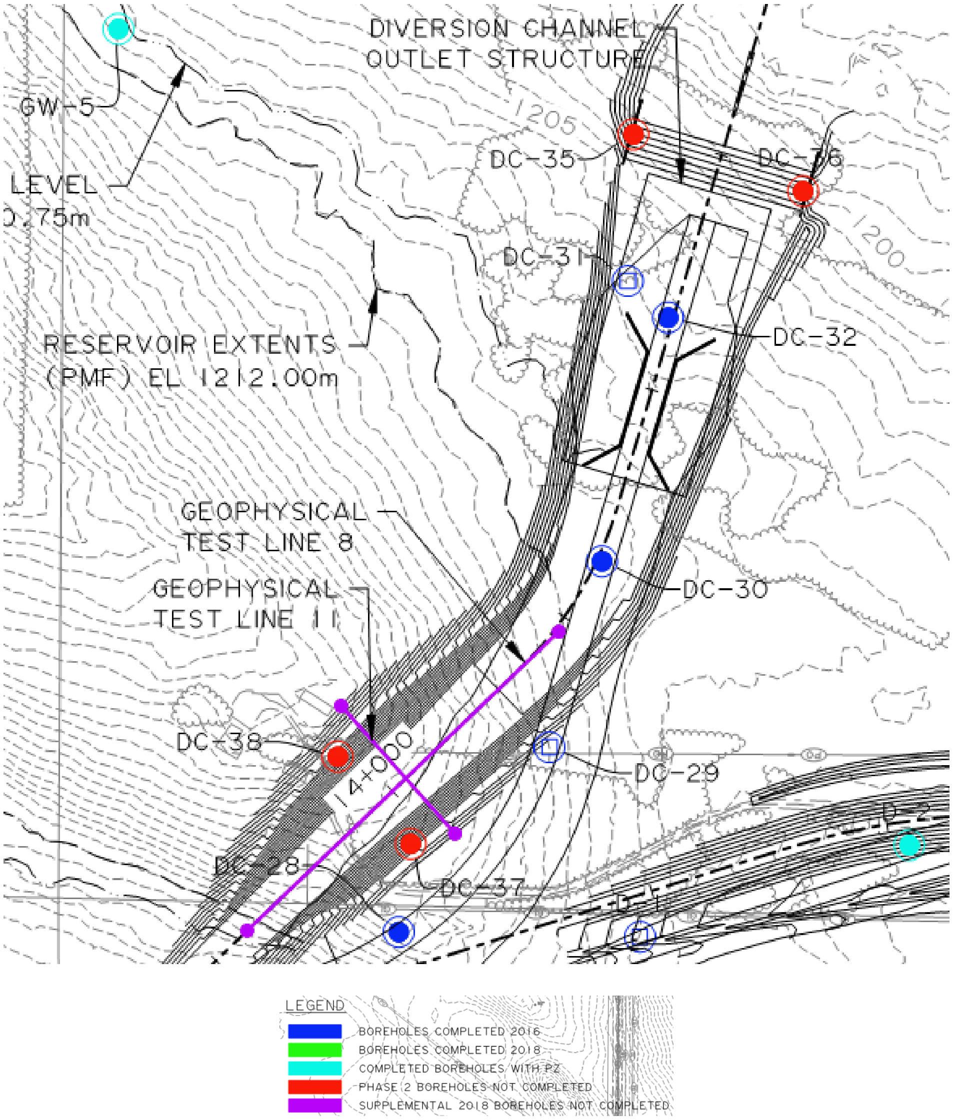


Figure 126. Diversion Outlet Additional Borings

18.4 BORROW SOURCE

Eight additional borings are proposed at multiple potential borrow source locations across the project area. These borings are shown in the following four figures.

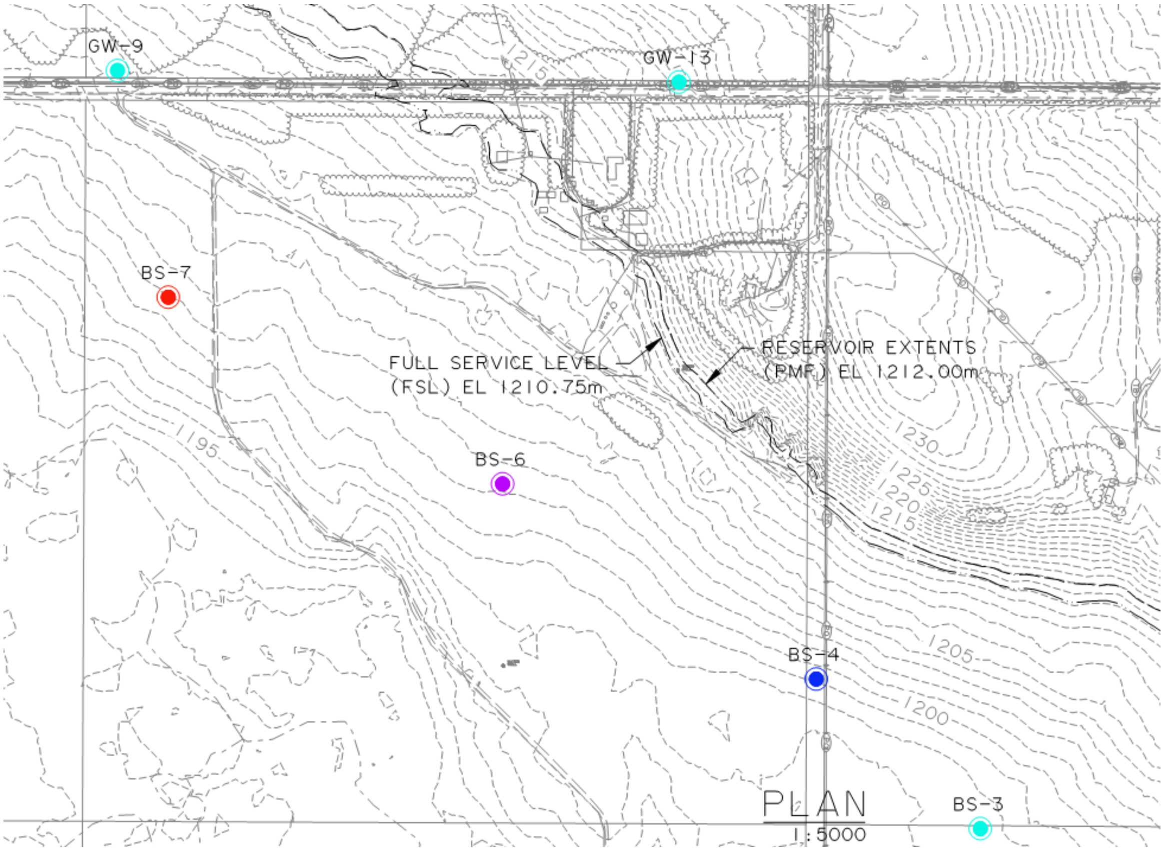


Figure 127. Additional Borrow Source Boring BS-6 and BS-7

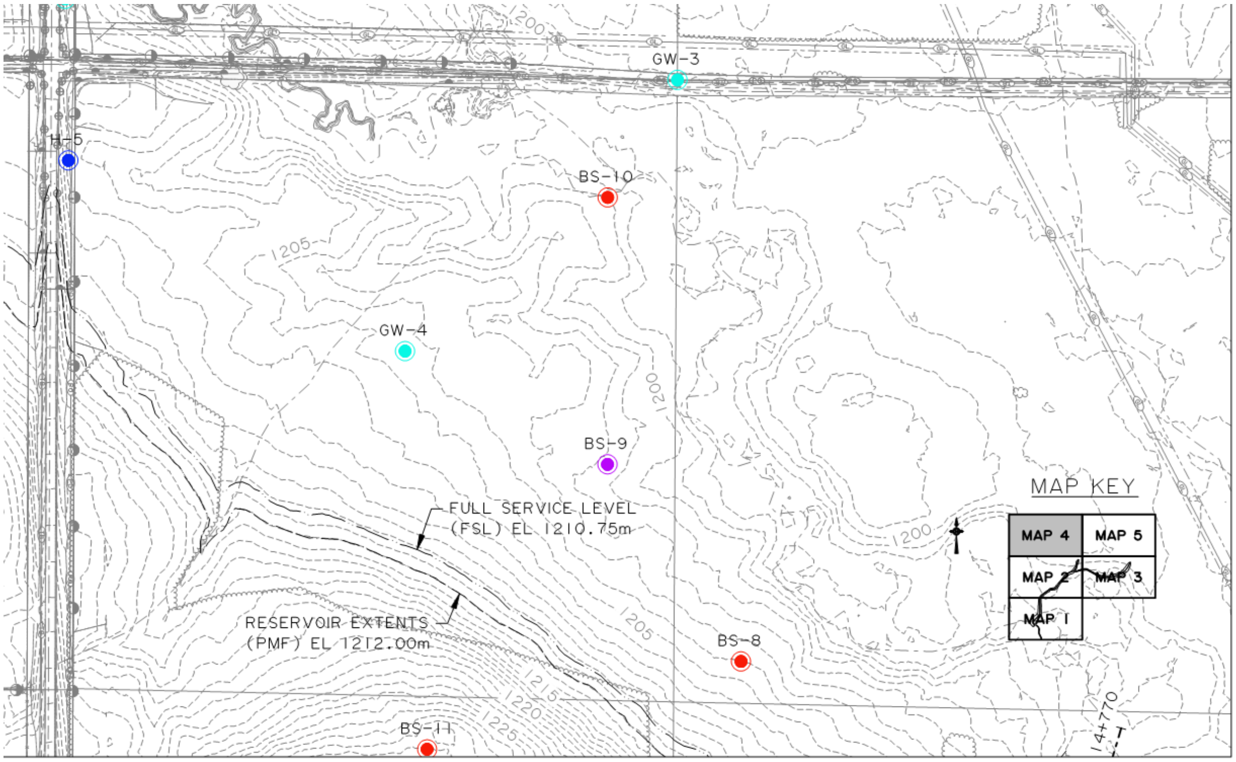


Figure 128. **Additional Borrow Source Boring BS-8 through BS-11**

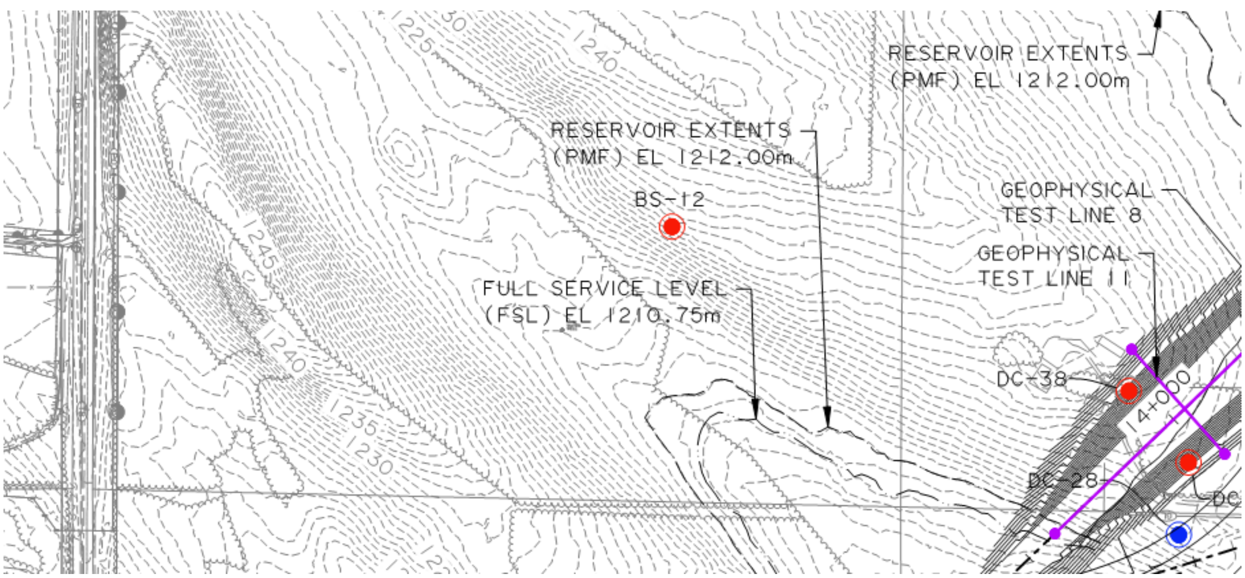


Figure 129. **Additional Borrow Source Boring BS-12**

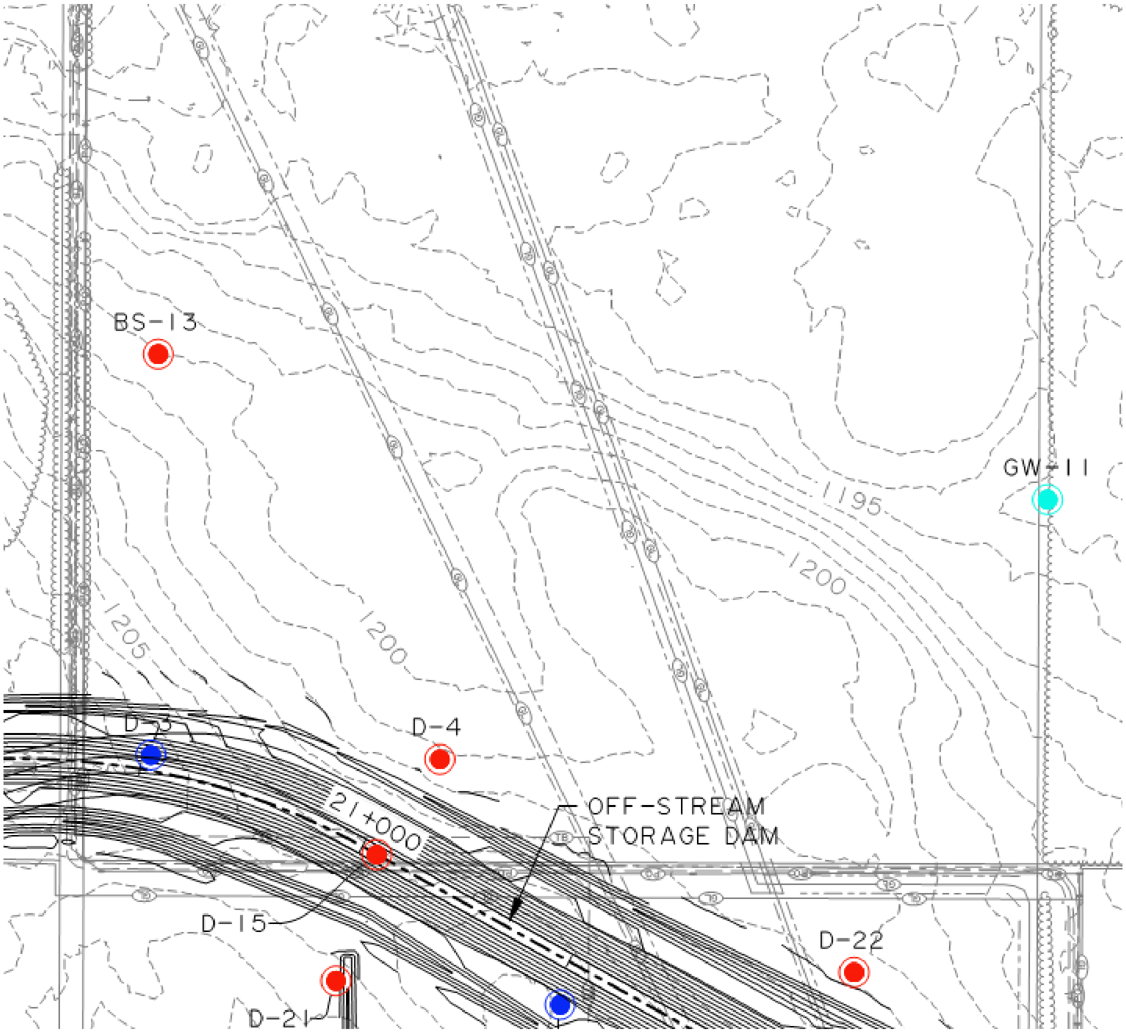


Figure 130. Additional Borrow Source Boring BS-13

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20.0 FIGURES

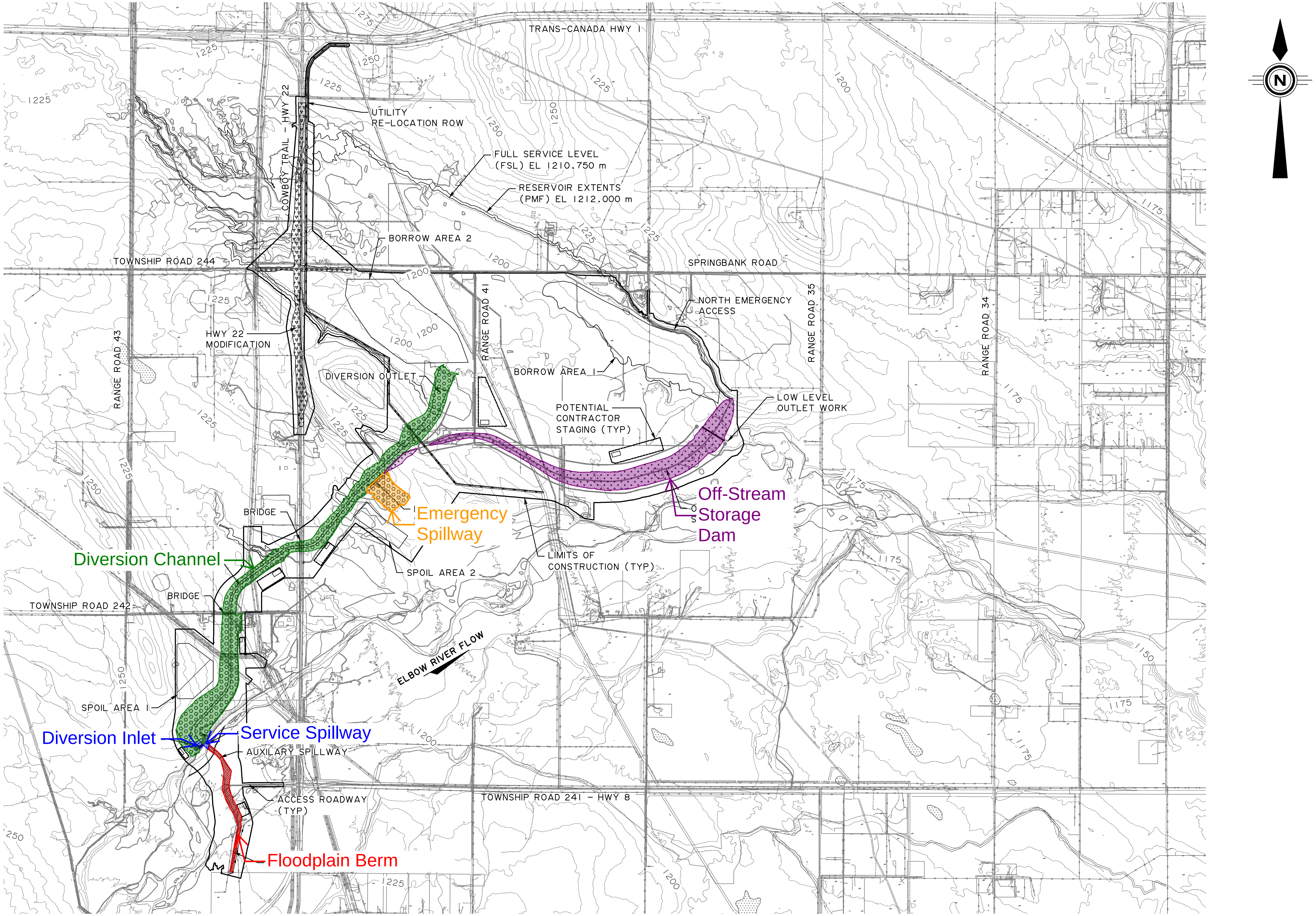

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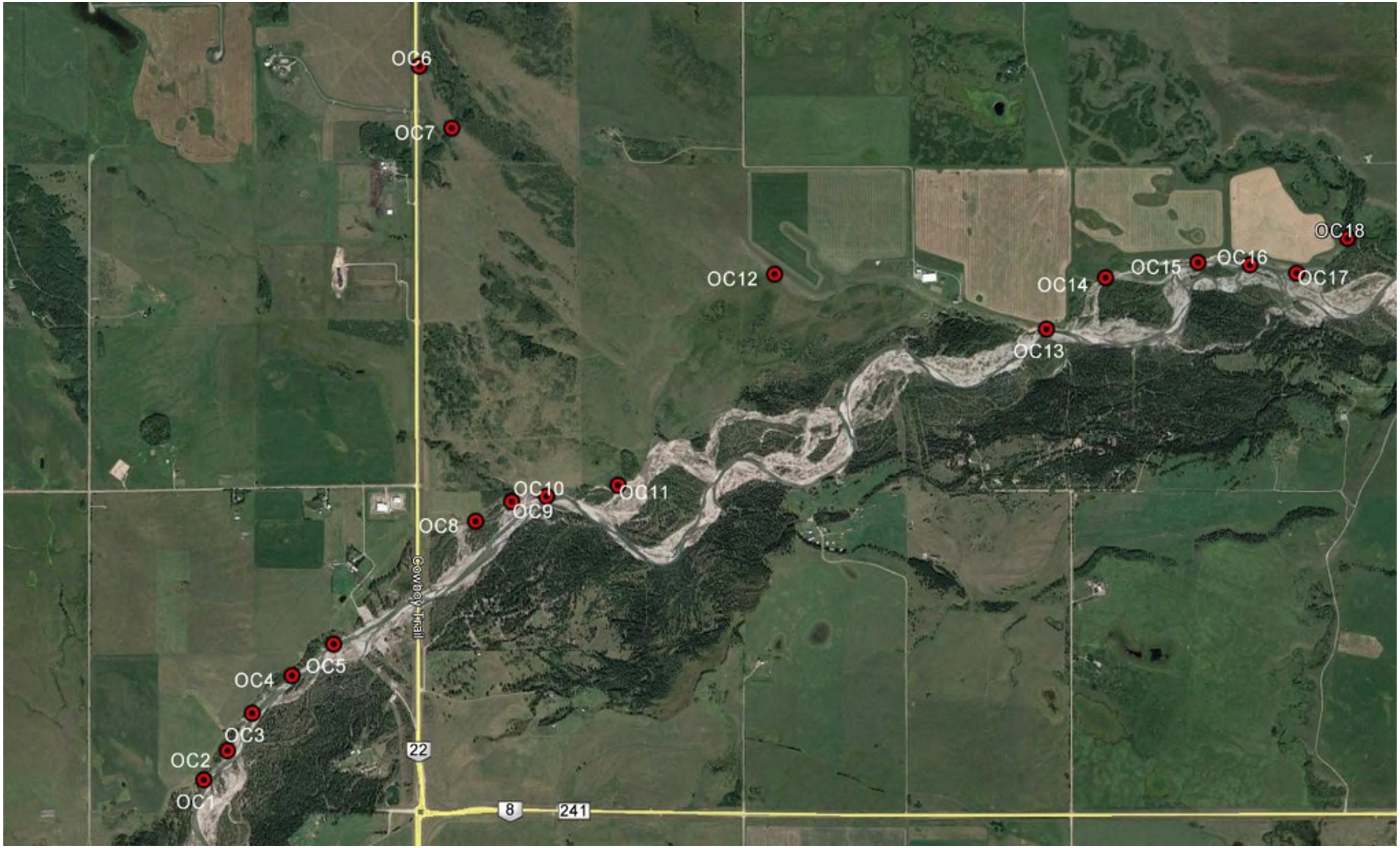
Figure No.
20 -1

Title
General Arrangement Plan



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Outcrop Location

Notes

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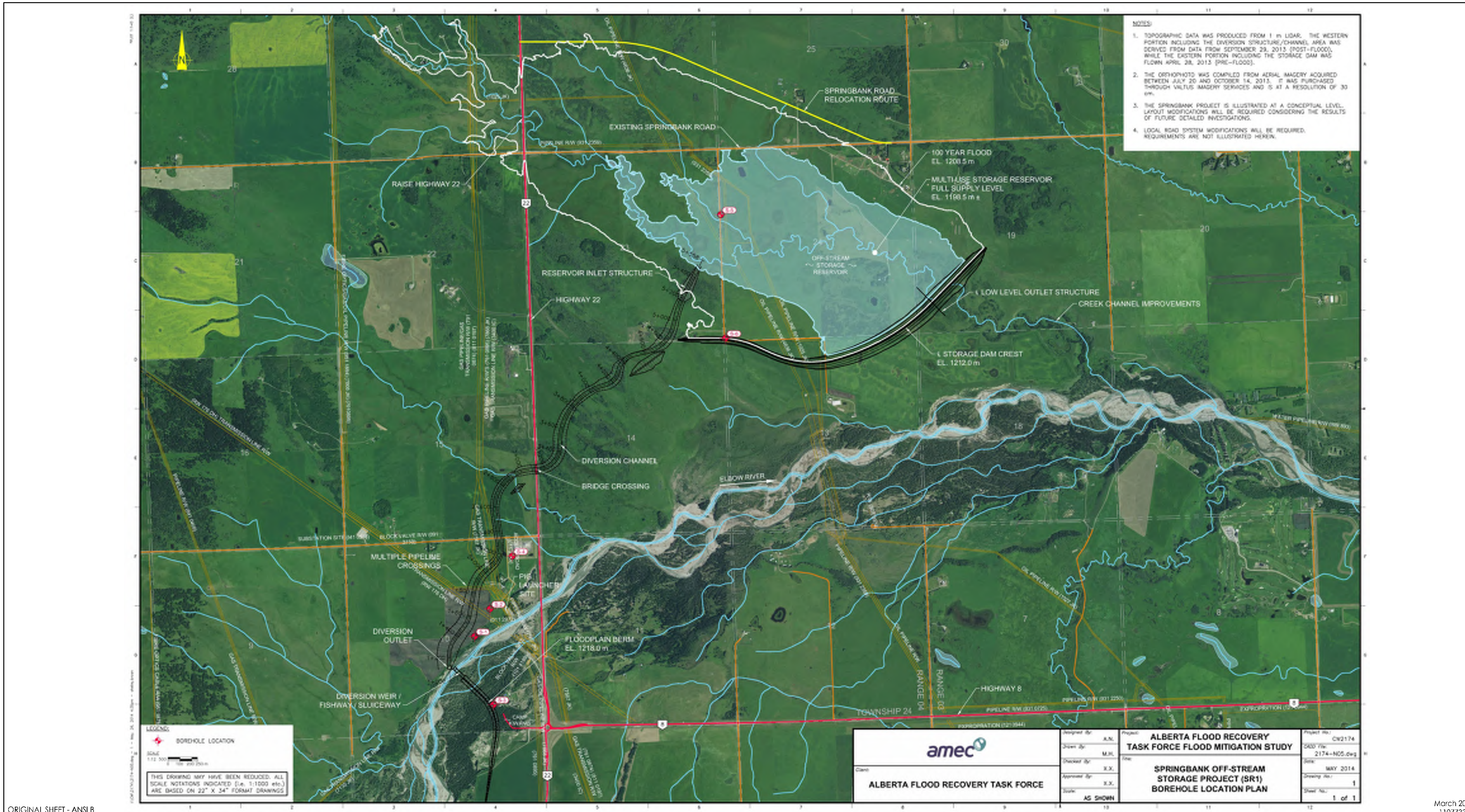
Figure No.

20 -2

Title

Outcrop Locations



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Notes

Borings advanced by AMEC during the feasibility phase of the project. Boring location map not to scale.

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Figure No.

20.-3

Title

2014 Feasibility Study -
Exploratory Hole Location Plan

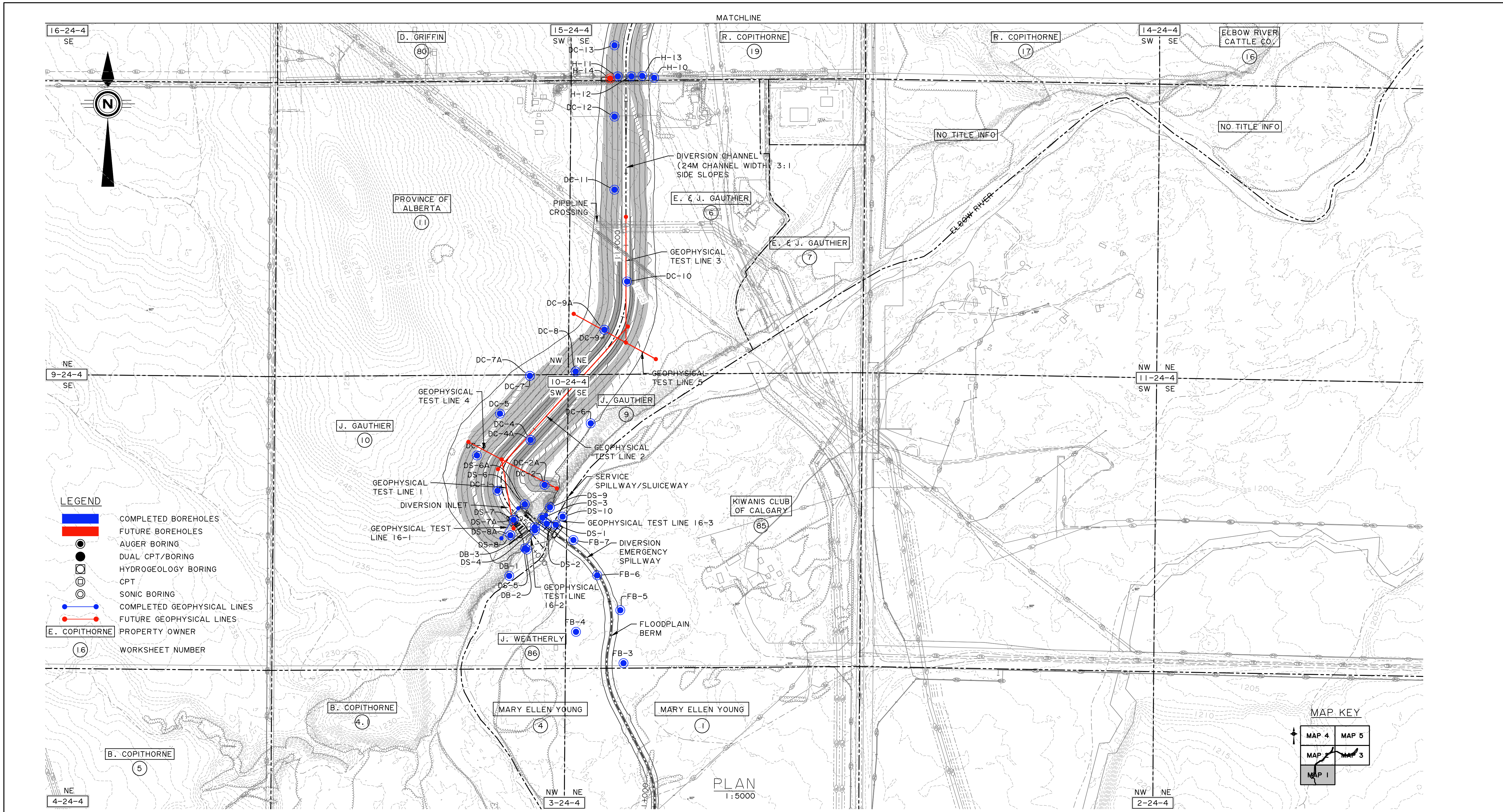

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Figure No.
20-4a

Title
Exploratory Hole Location Plan

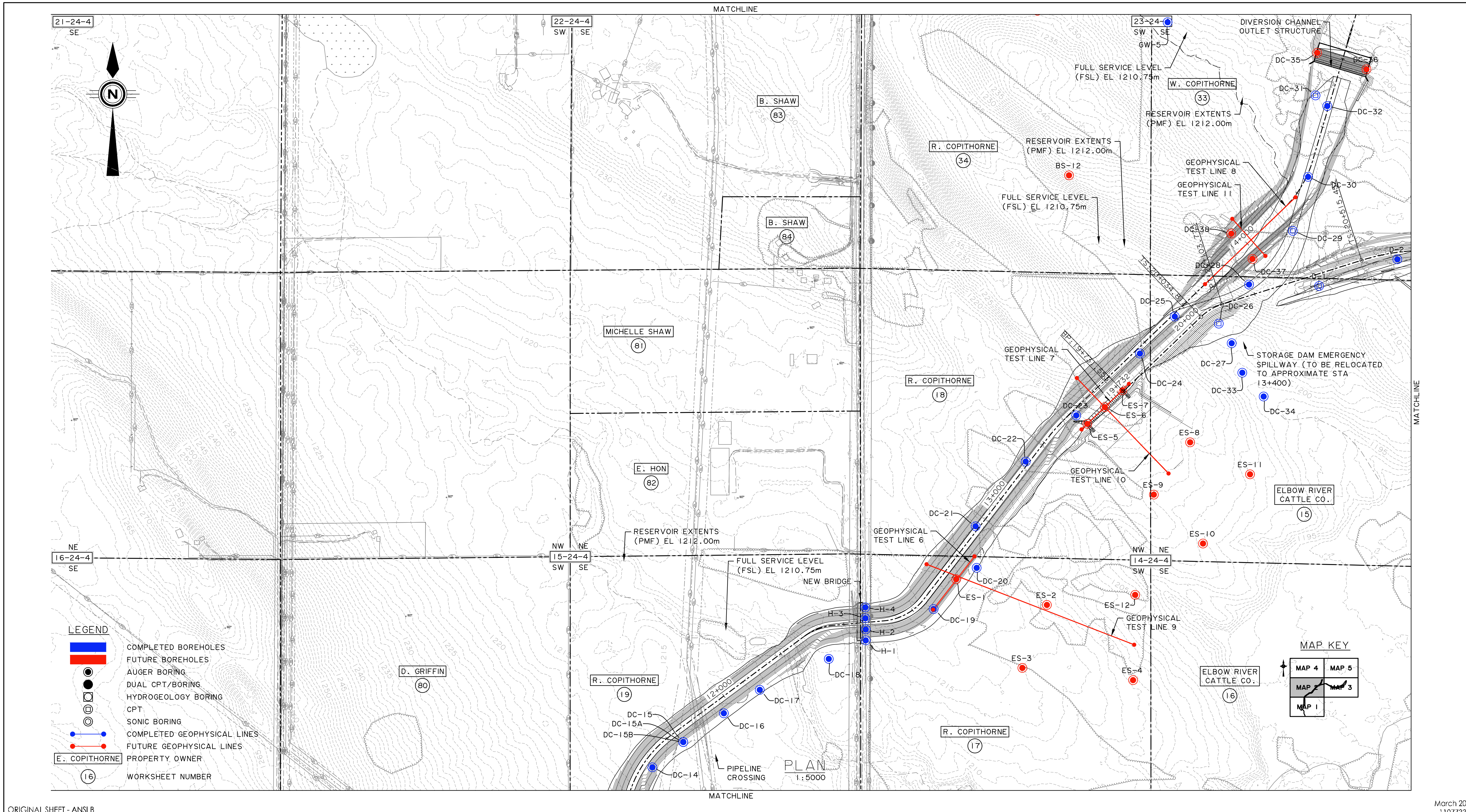

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Figure No.

20-4b

Title

Exploratory Hole Location Plan

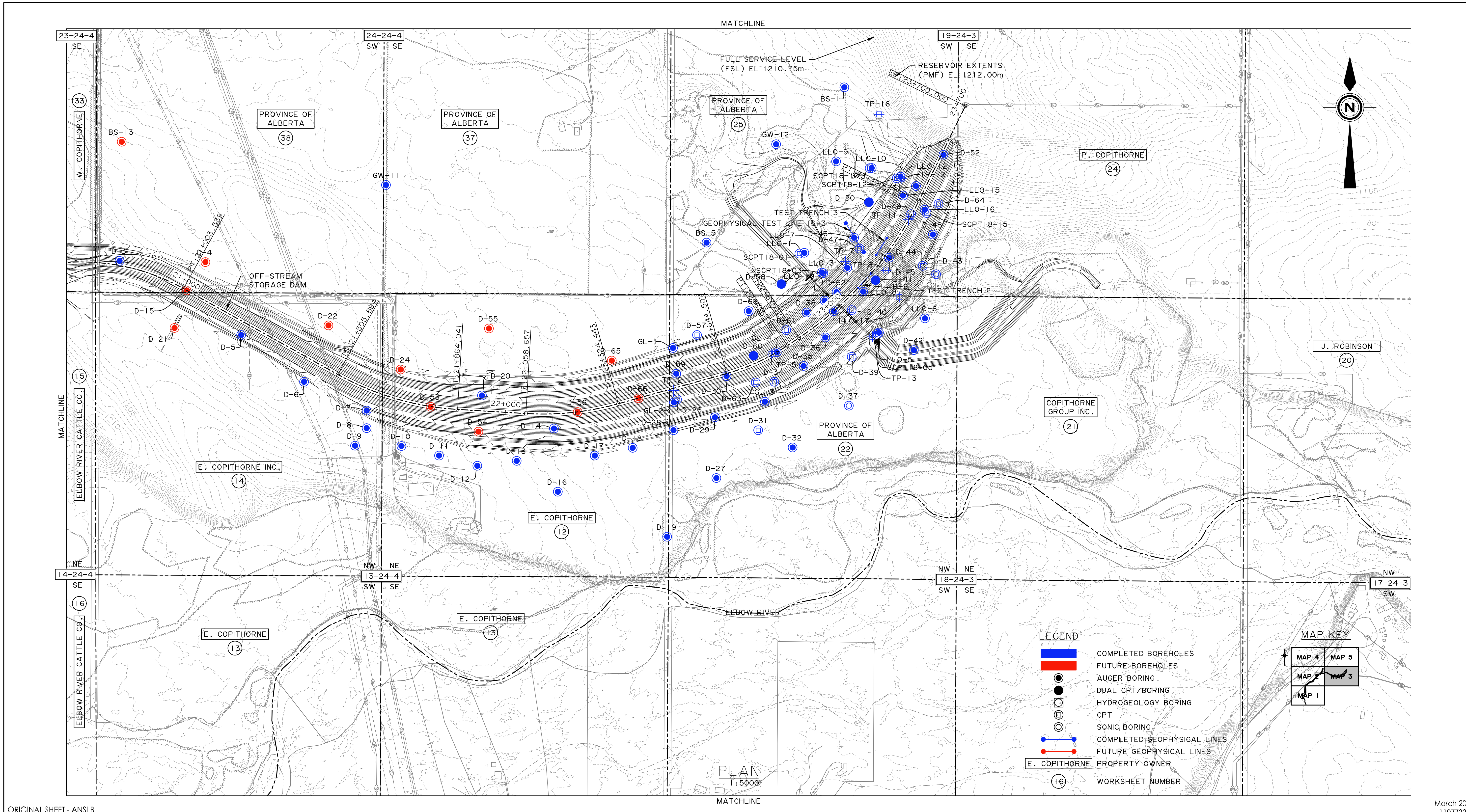

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Figure No.

20-4c

Title

Exploratory Hole Location Plan

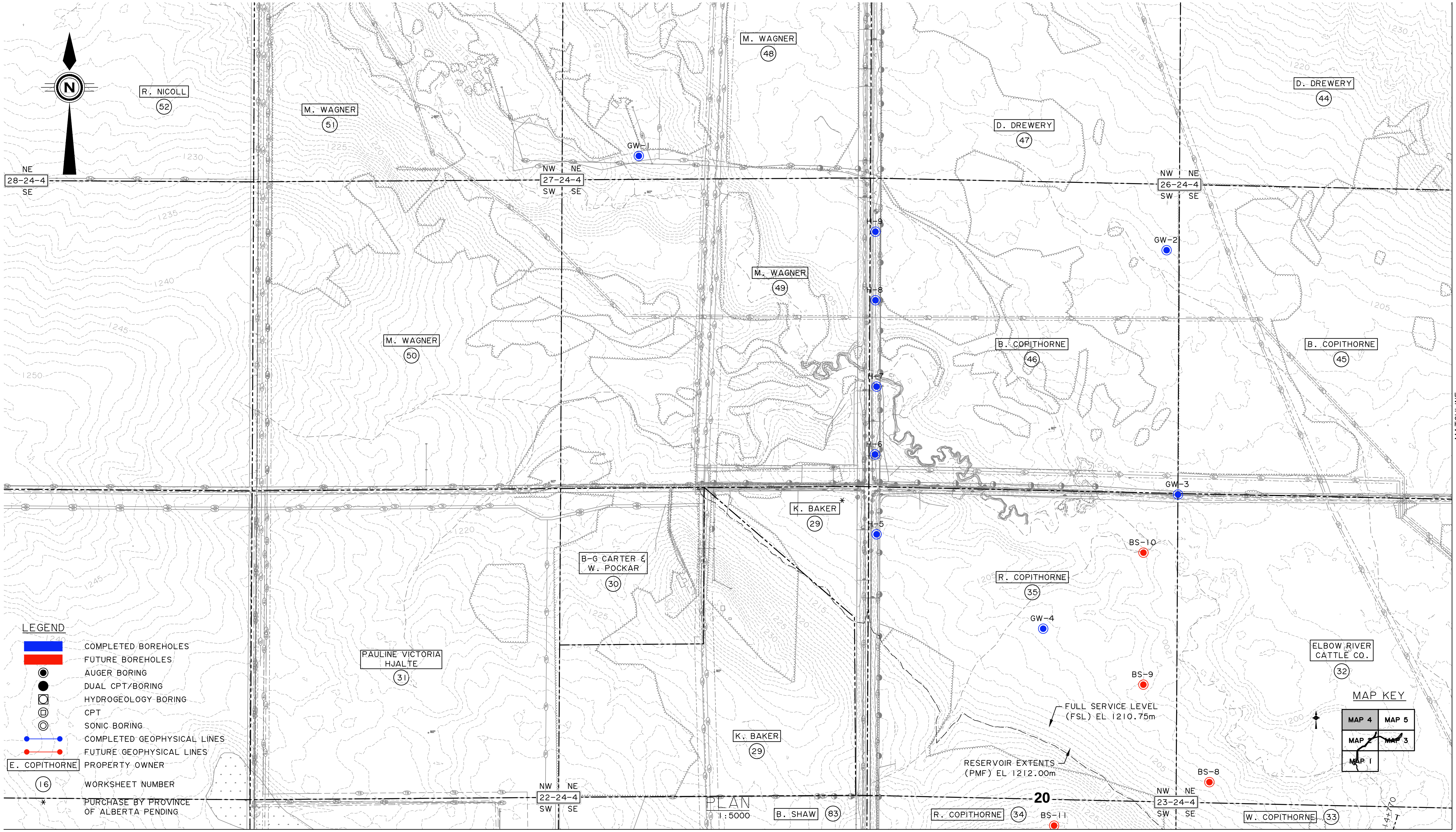

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Figure No.

20.-4d

Title

Exploratory Hole Location Plan

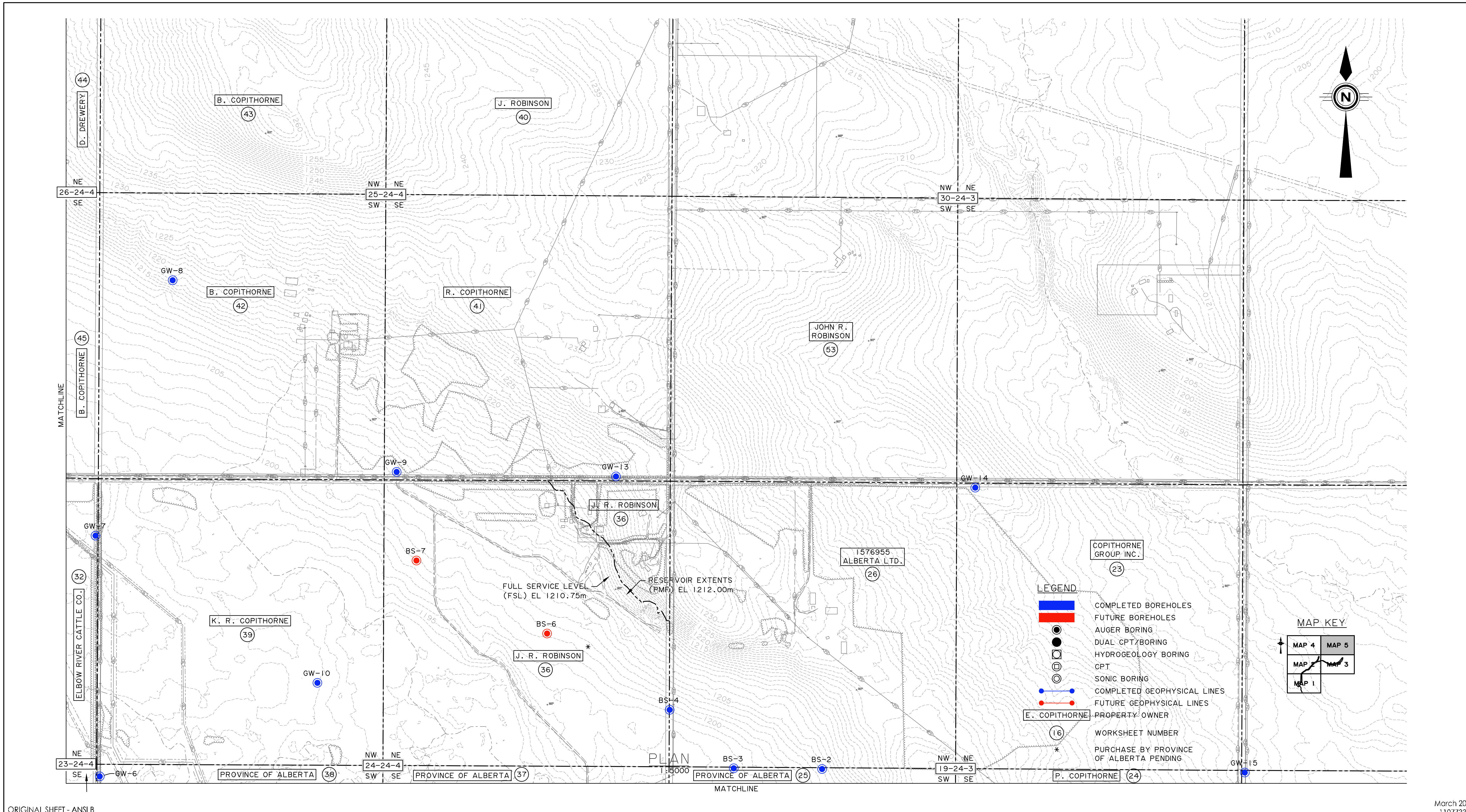

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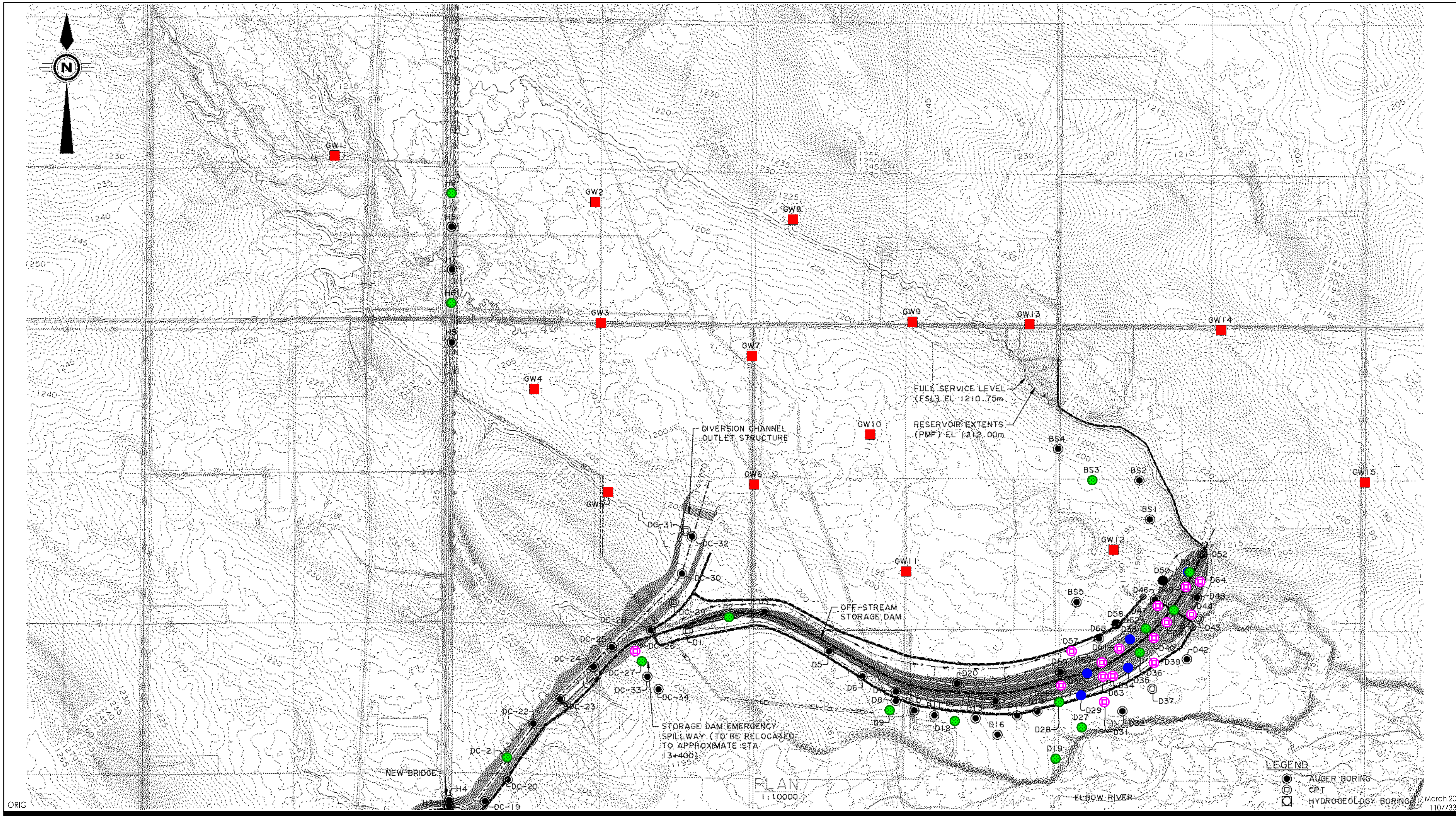
Figure No.

20-4e

Title

Exploratory Hole Location Plan



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- Packer Testing
- 2016 Hydrogeology Well
- Piezometer
- CPT Dissipation Testing

Notes

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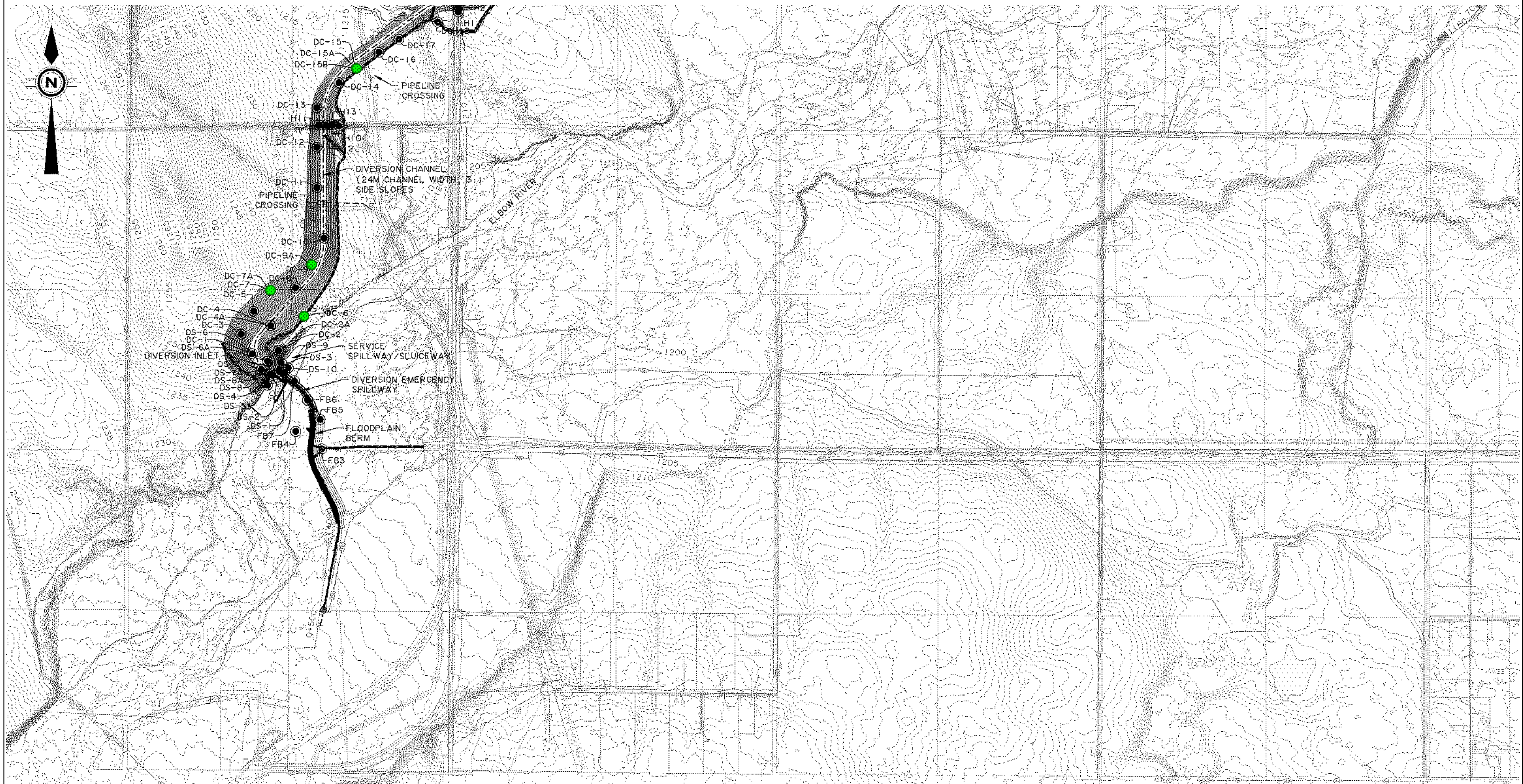
Preliminary Geotechnical Design Report

Figure No.
20-5a

Title
Hydrogeology Information



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Legend

- Packer Testing
- 2016 Hydrogeology Well
- Piezometer
- CPT Dissipation Testing

Notes

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Alberta Transportation
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Figure No.
20-5b

Title
Hydrogeology Information

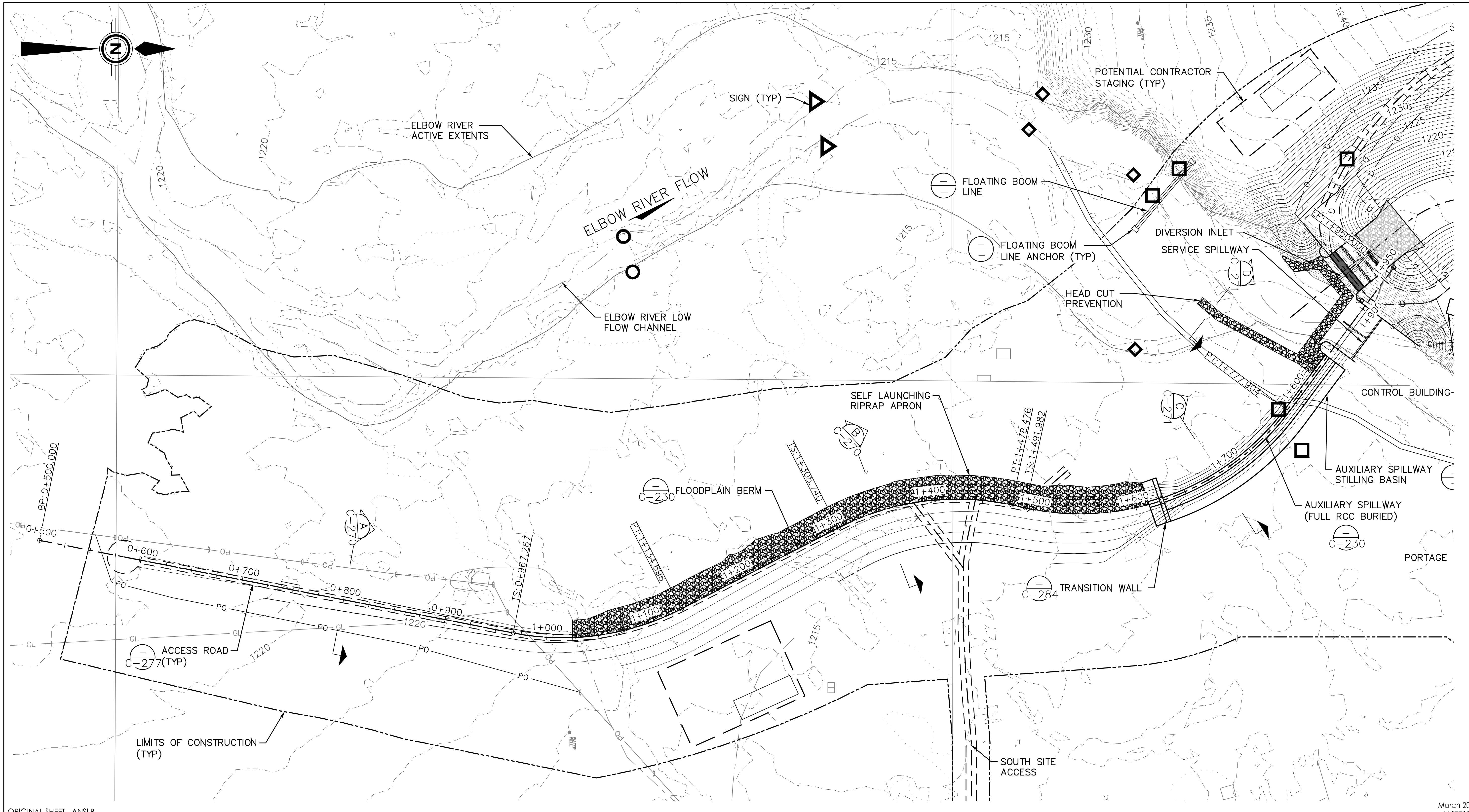

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Figure No.
20-6a

Title
Geological Profile Locations

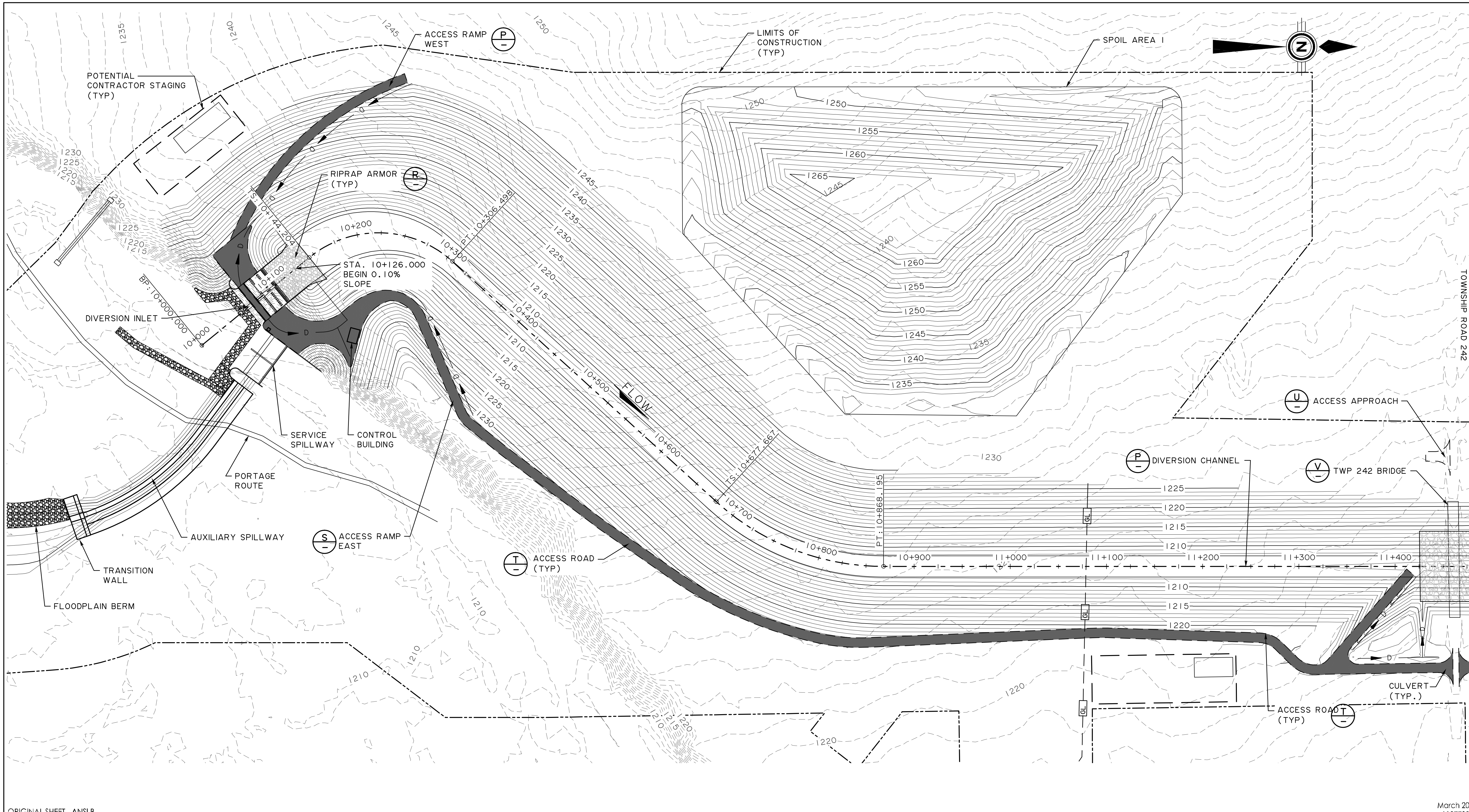

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Figure No.

20-6b

Title

Geological Profile Locations

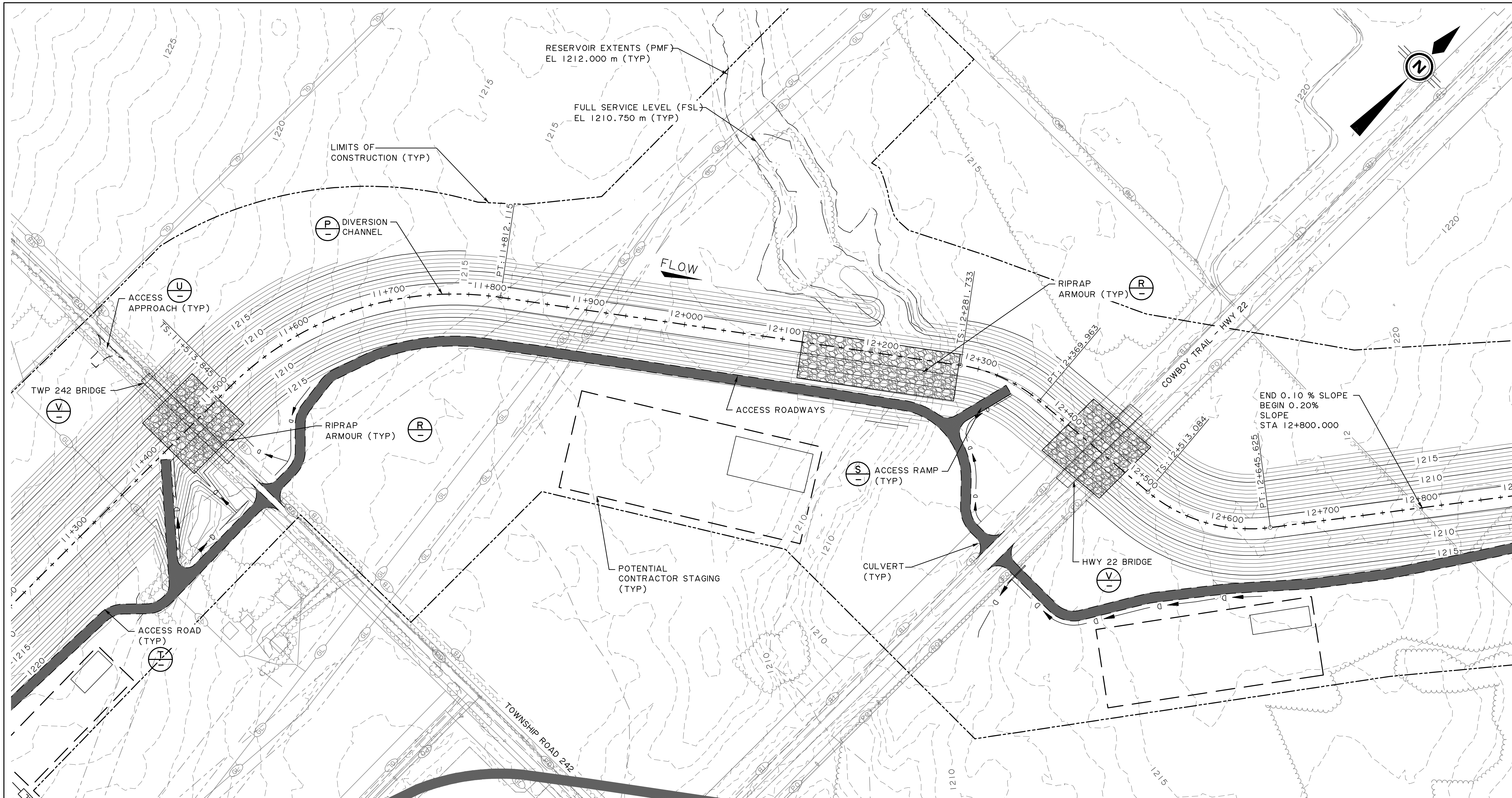

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Figure No.

20-6c

Title

Geological Profile Locations

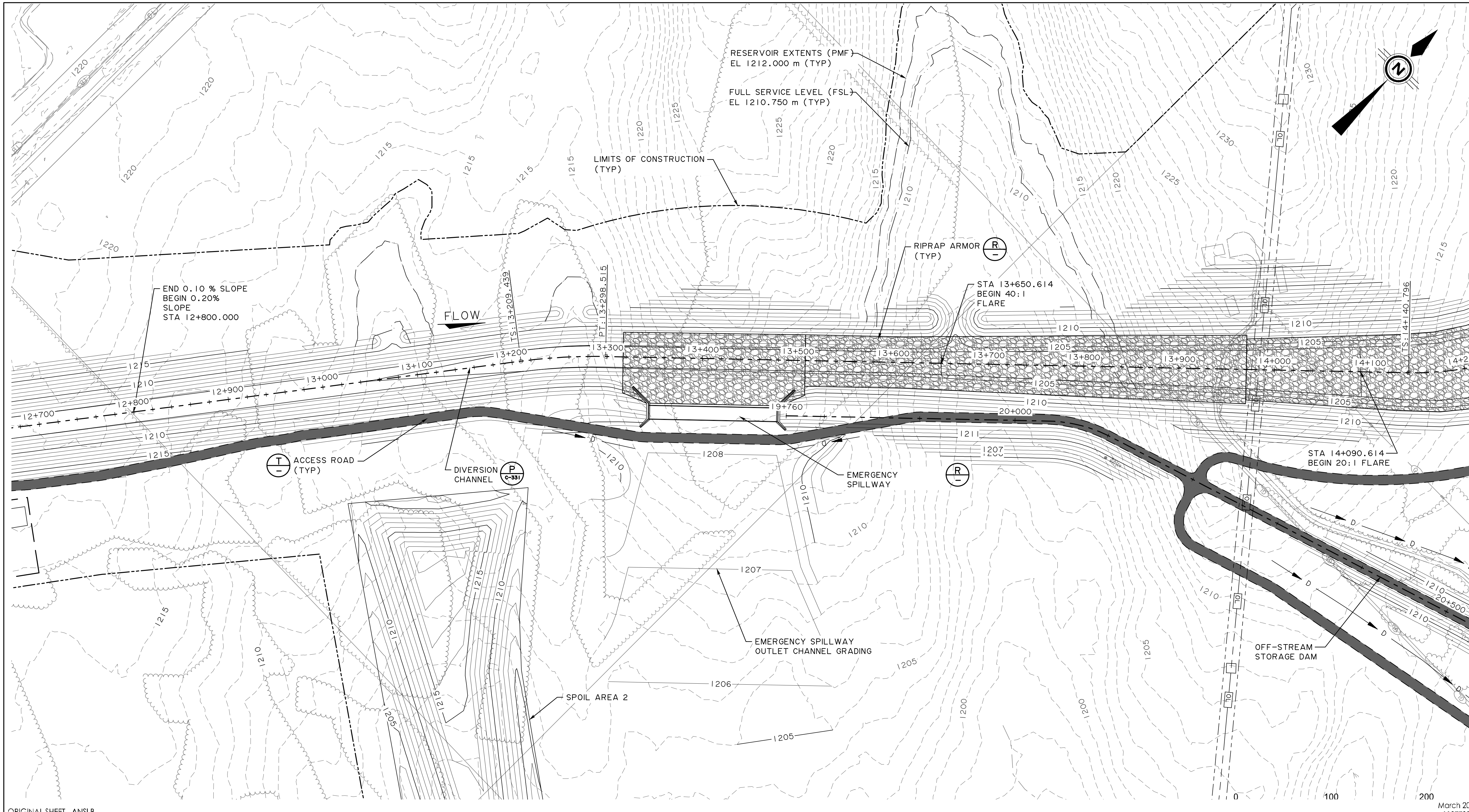

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Figure No.

20-6d

Title

Geological Profile Locations

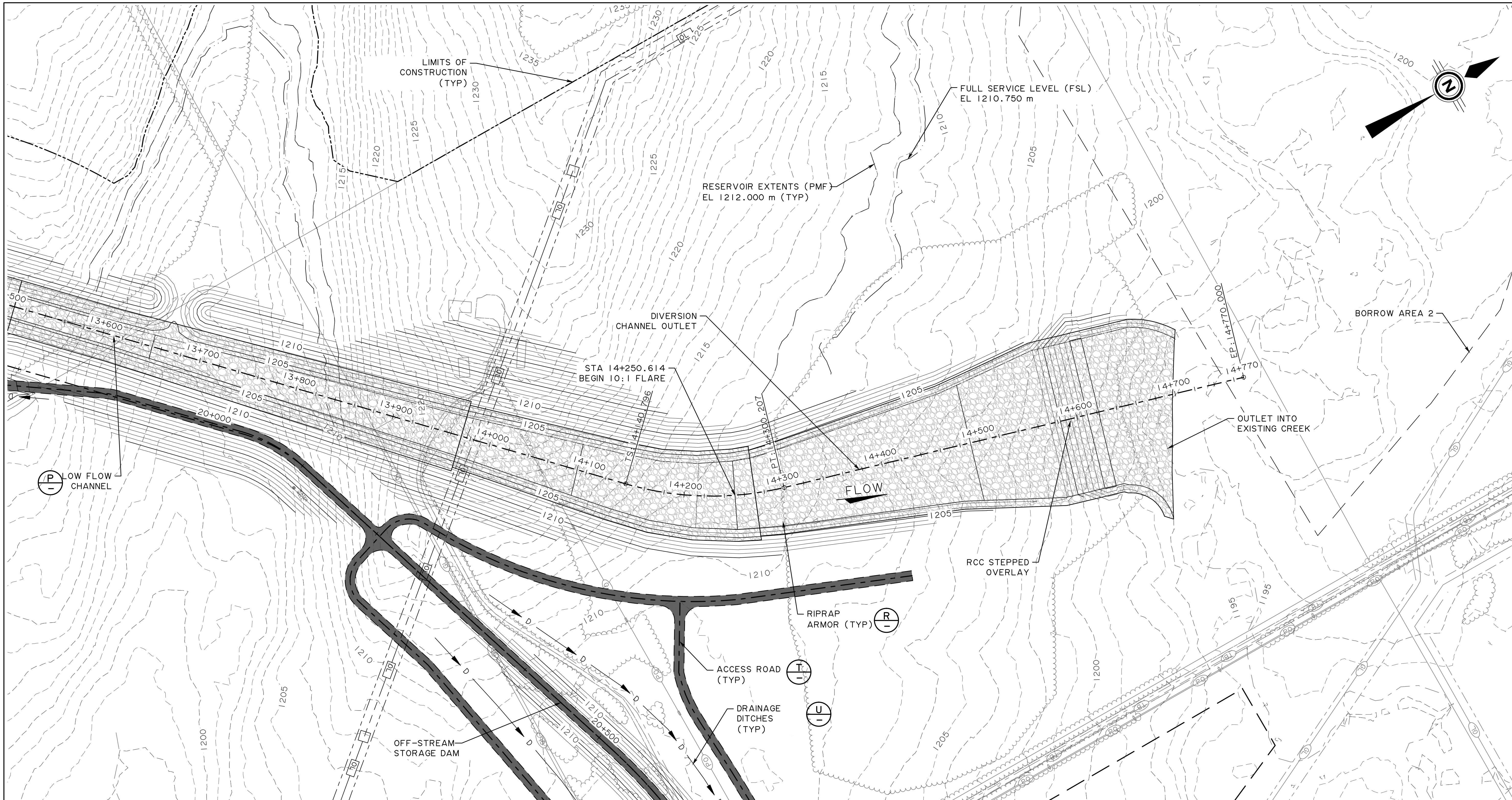

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Figure No.
20-6e

Title
Geological Profile Locations

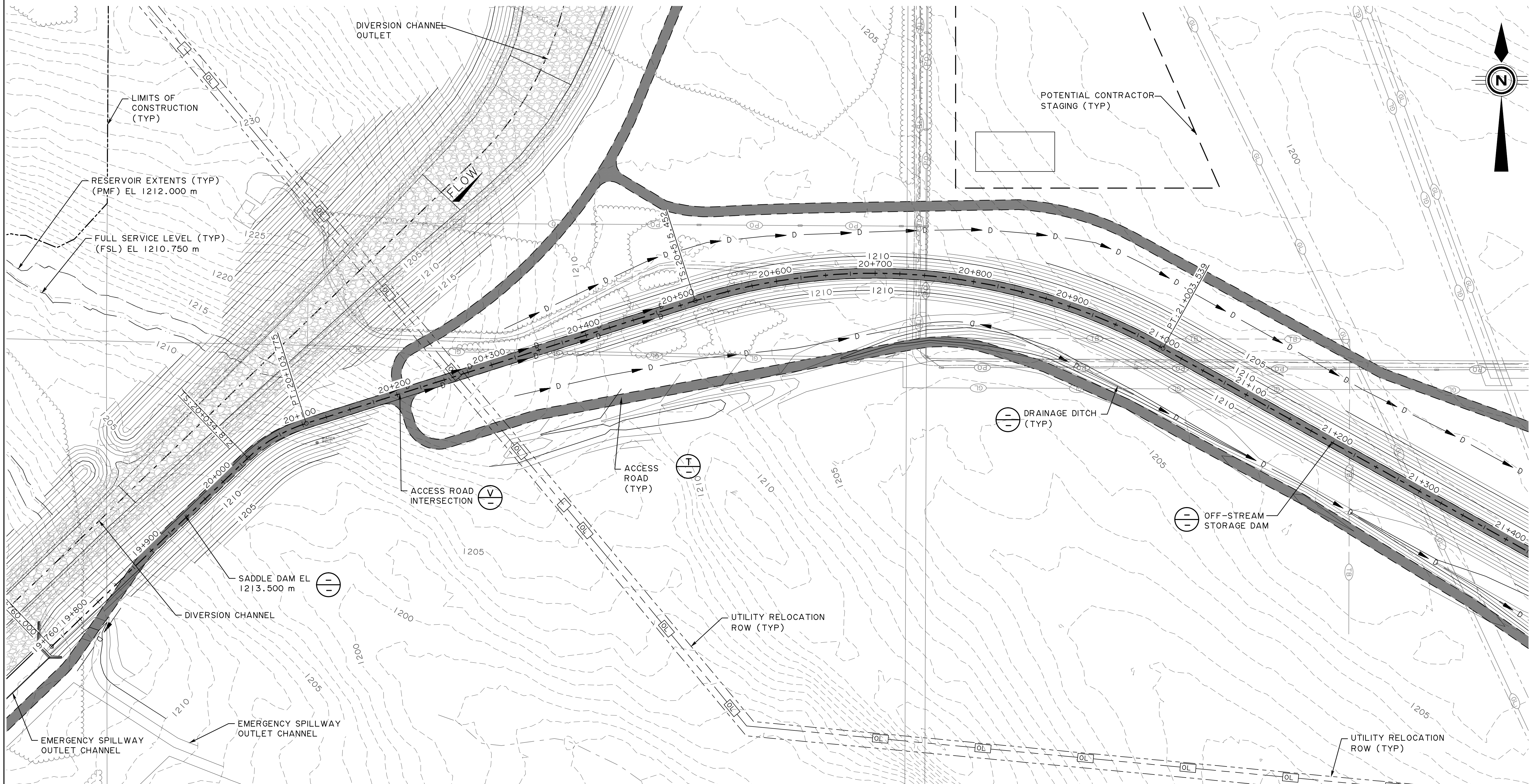

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Figure No.

20-6f

Title

Geological Profile Locations

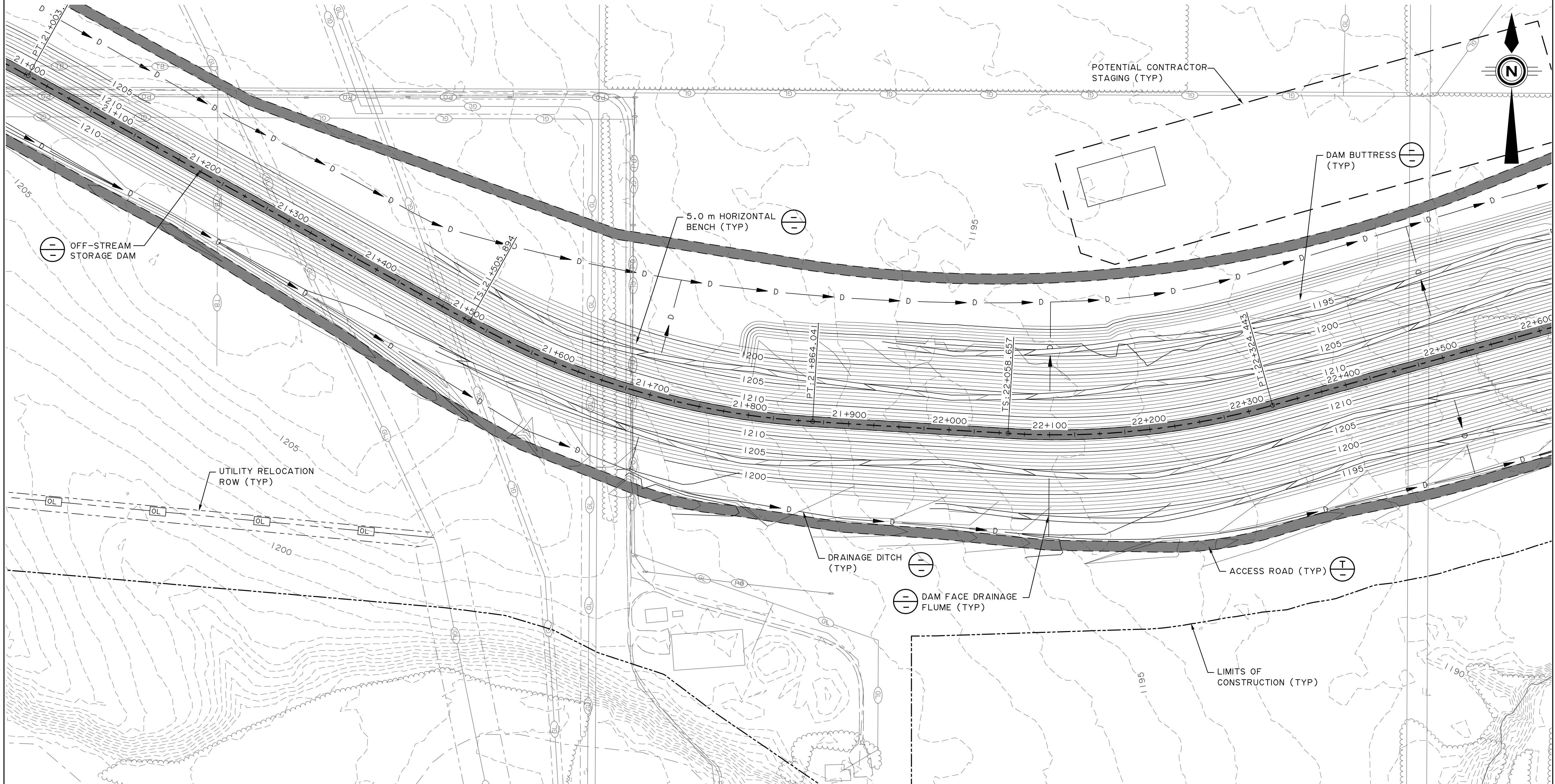

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Figure No.
20.-6g

Title
Geological Profile Locations

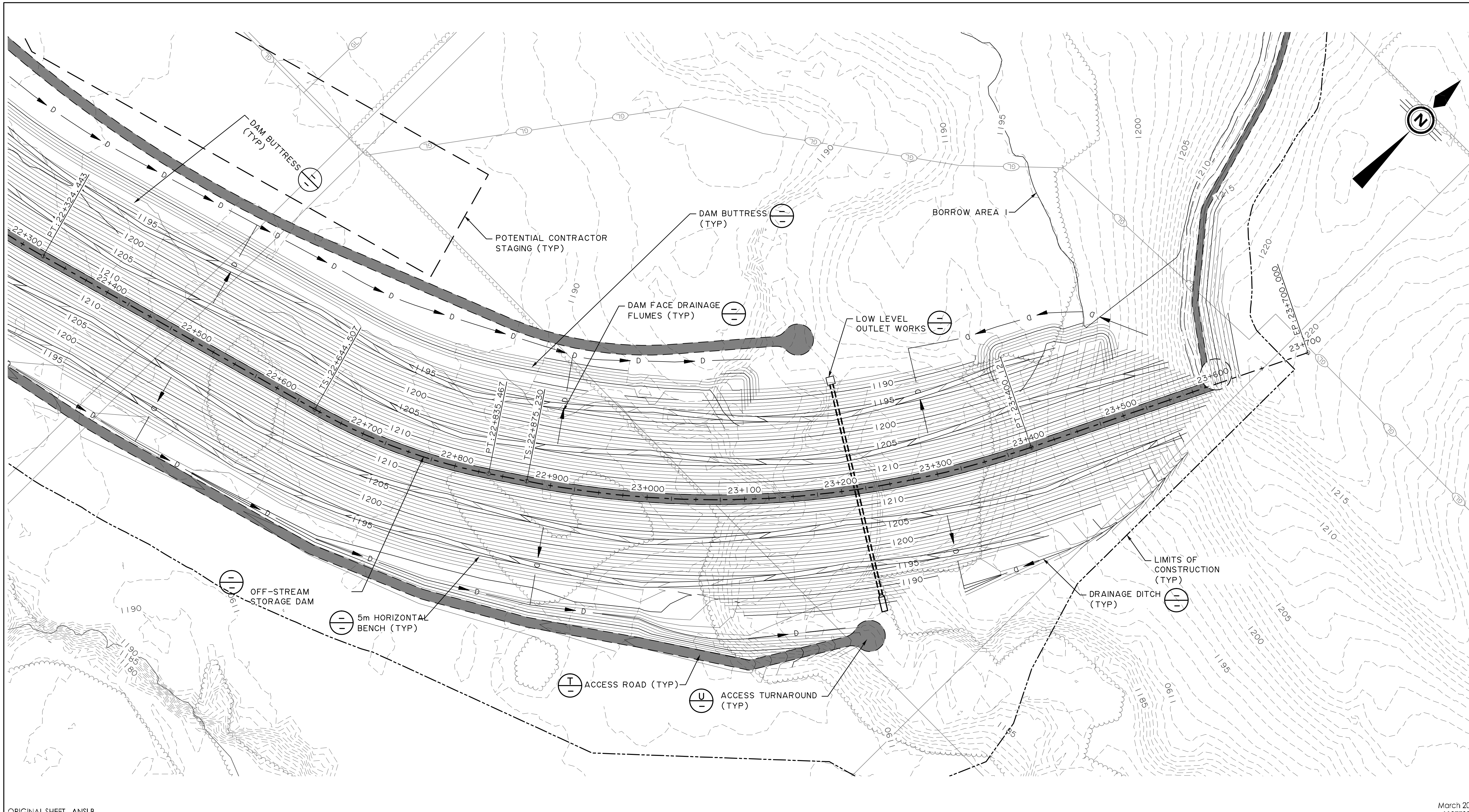

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Figure No.
20.-6h

Title
Geological Profile Locations

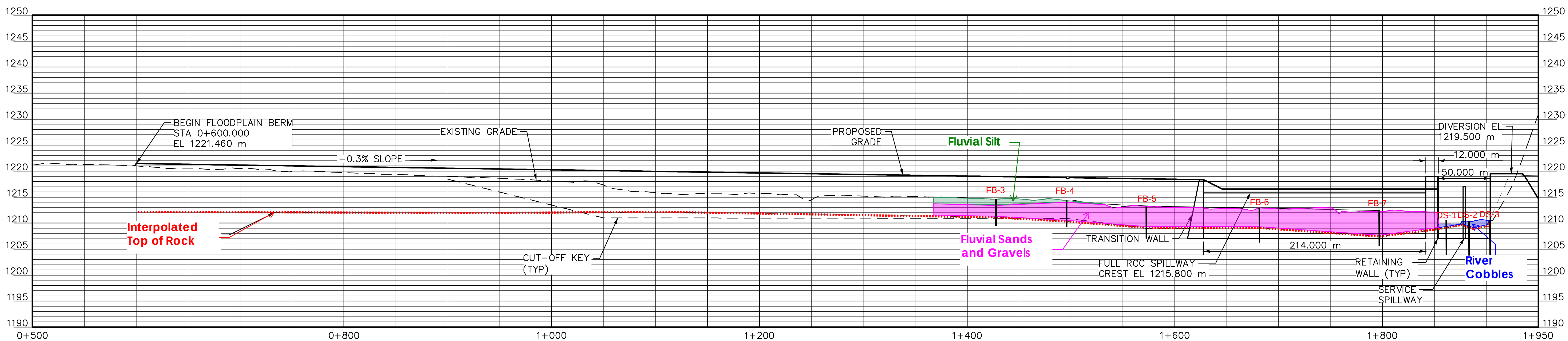

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Figure No.

20-7

Title

Geological Profile - Floodplain Berm and Diversion Structure

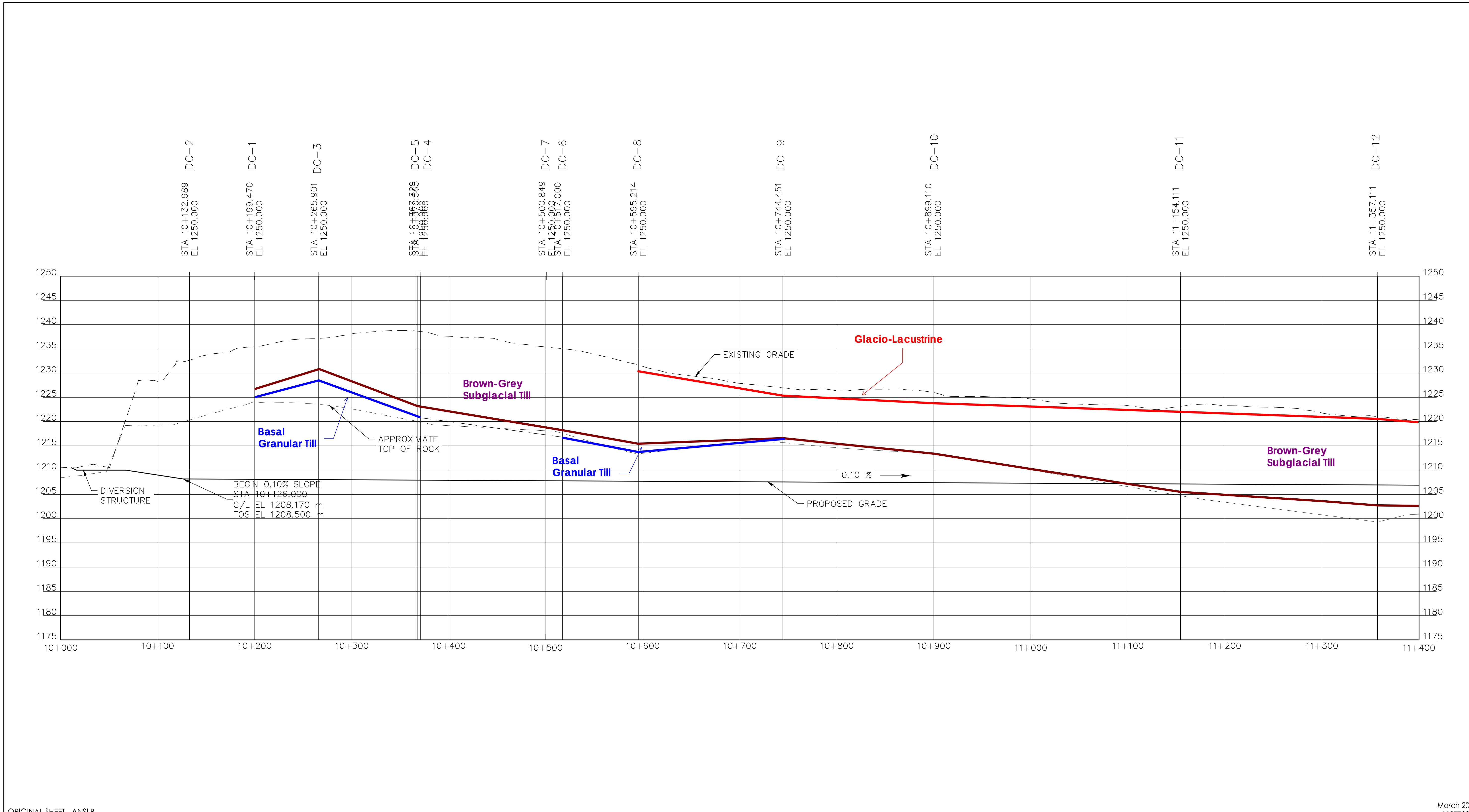

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Figure No.

20-8a

Title

Geological Profile - Diversion Channel

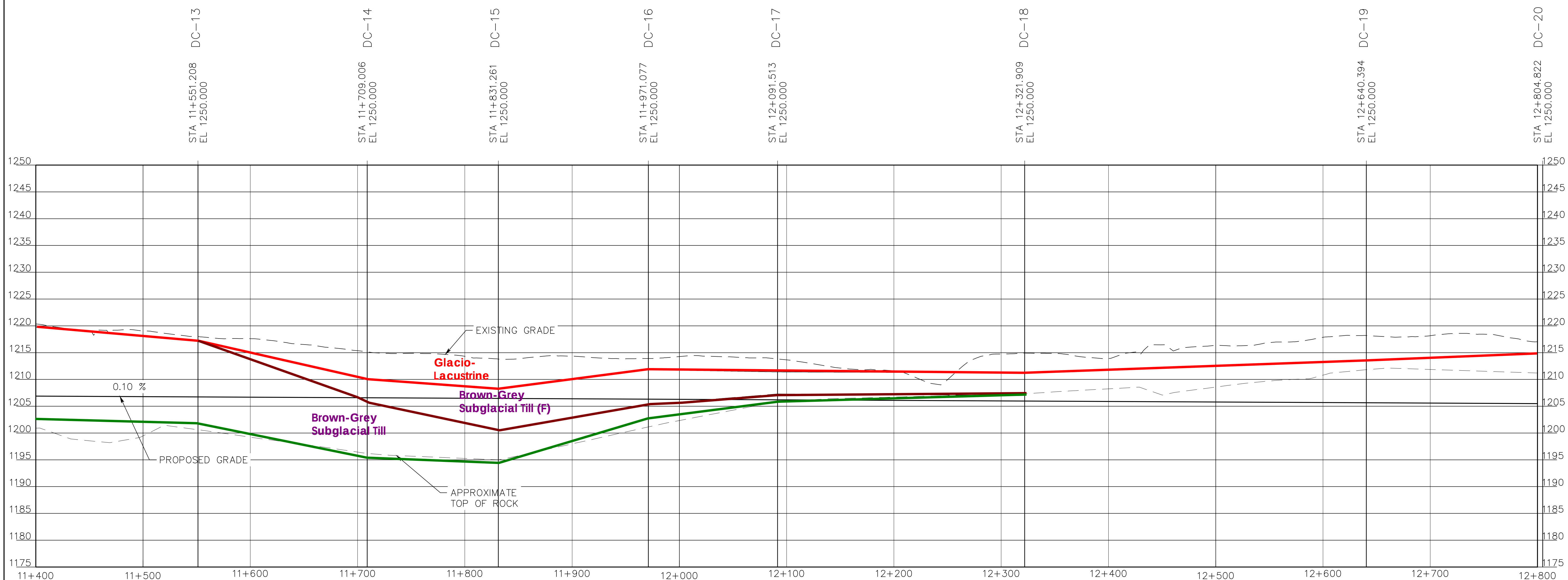

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Figure No.

20-8b

Title

Geological Profile - Diversion Channel

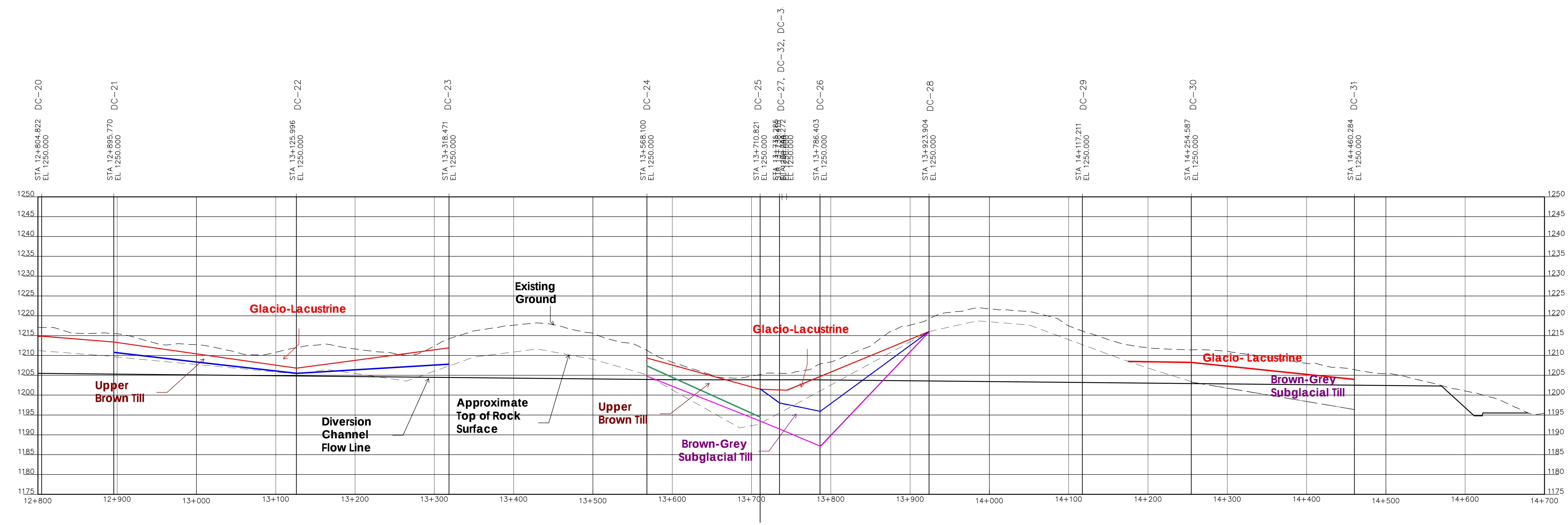

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Figure No.

20-8c

Title

Geological Profile - Diversion Channel

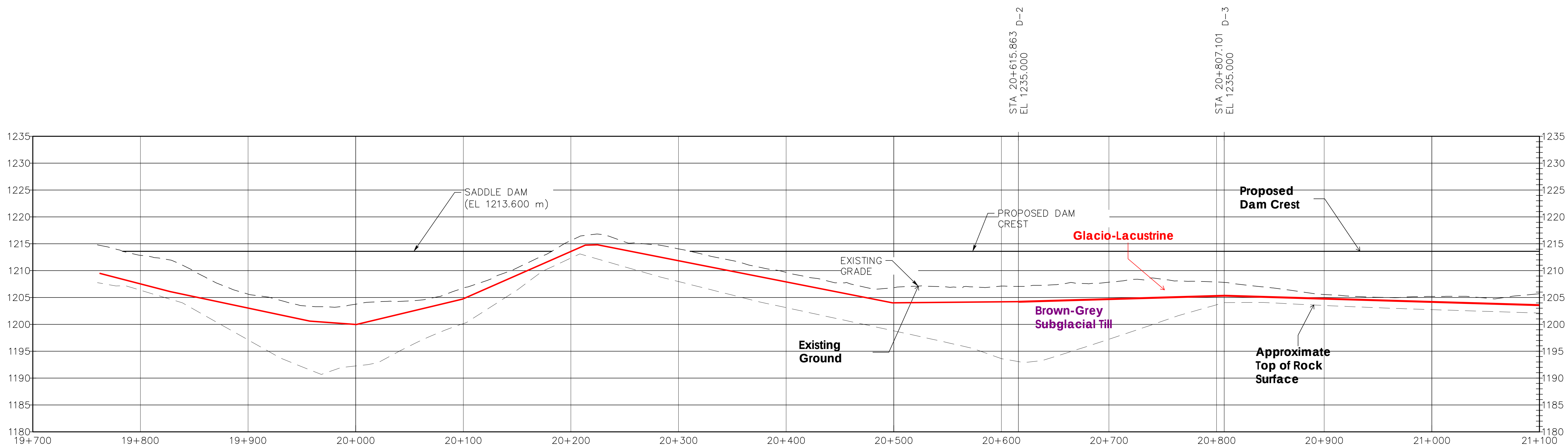

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Figure No.

20-9a

Title

Geologic Profile - Off-Stream Storage Dam

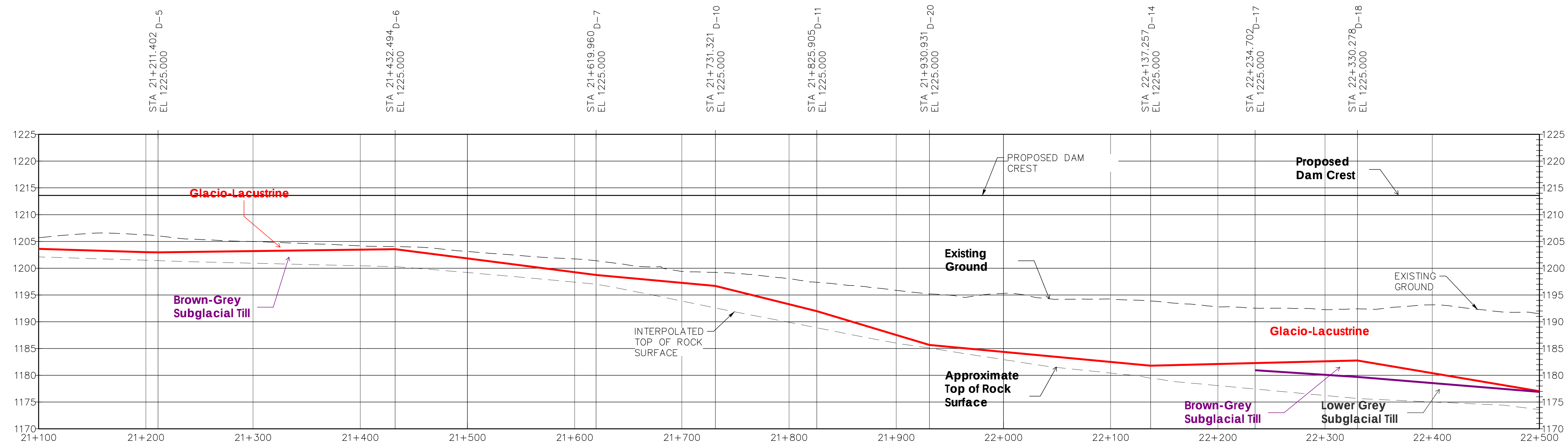

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Figure No.
20-9b

Title
Geologic Profile - Off-Stream Storage Dam

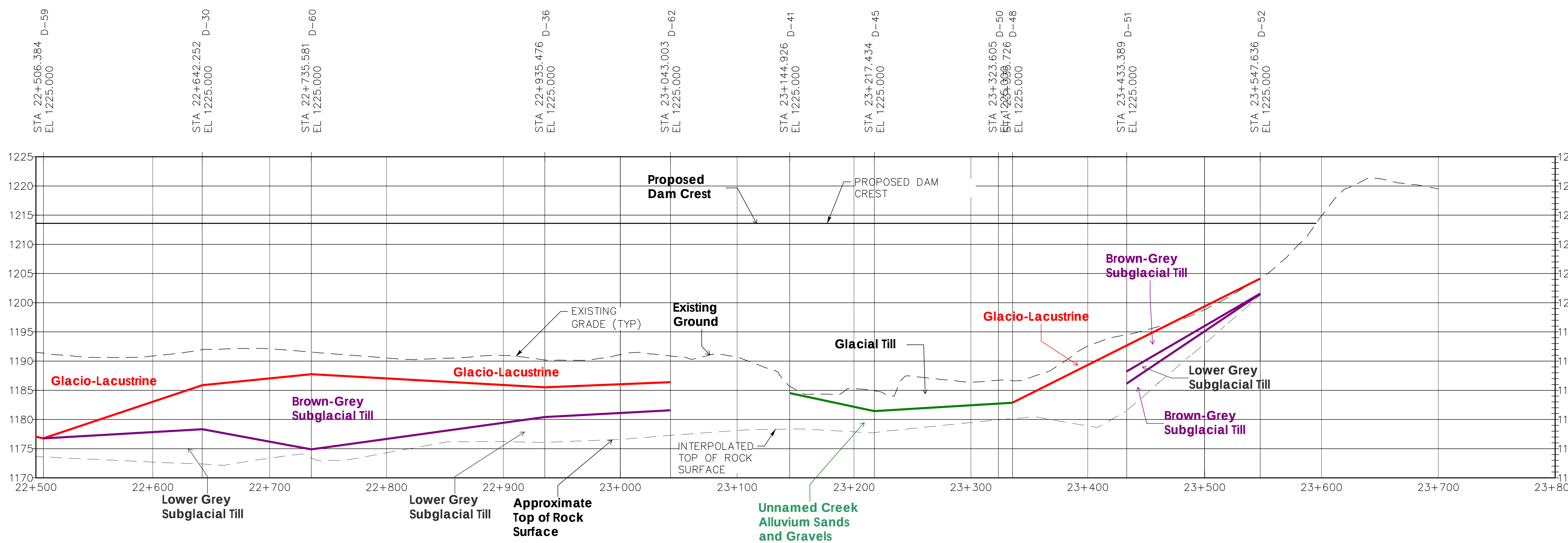

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Figure No.
20-9c

Title
Geologic Profile - Off-Stream Storage Dam

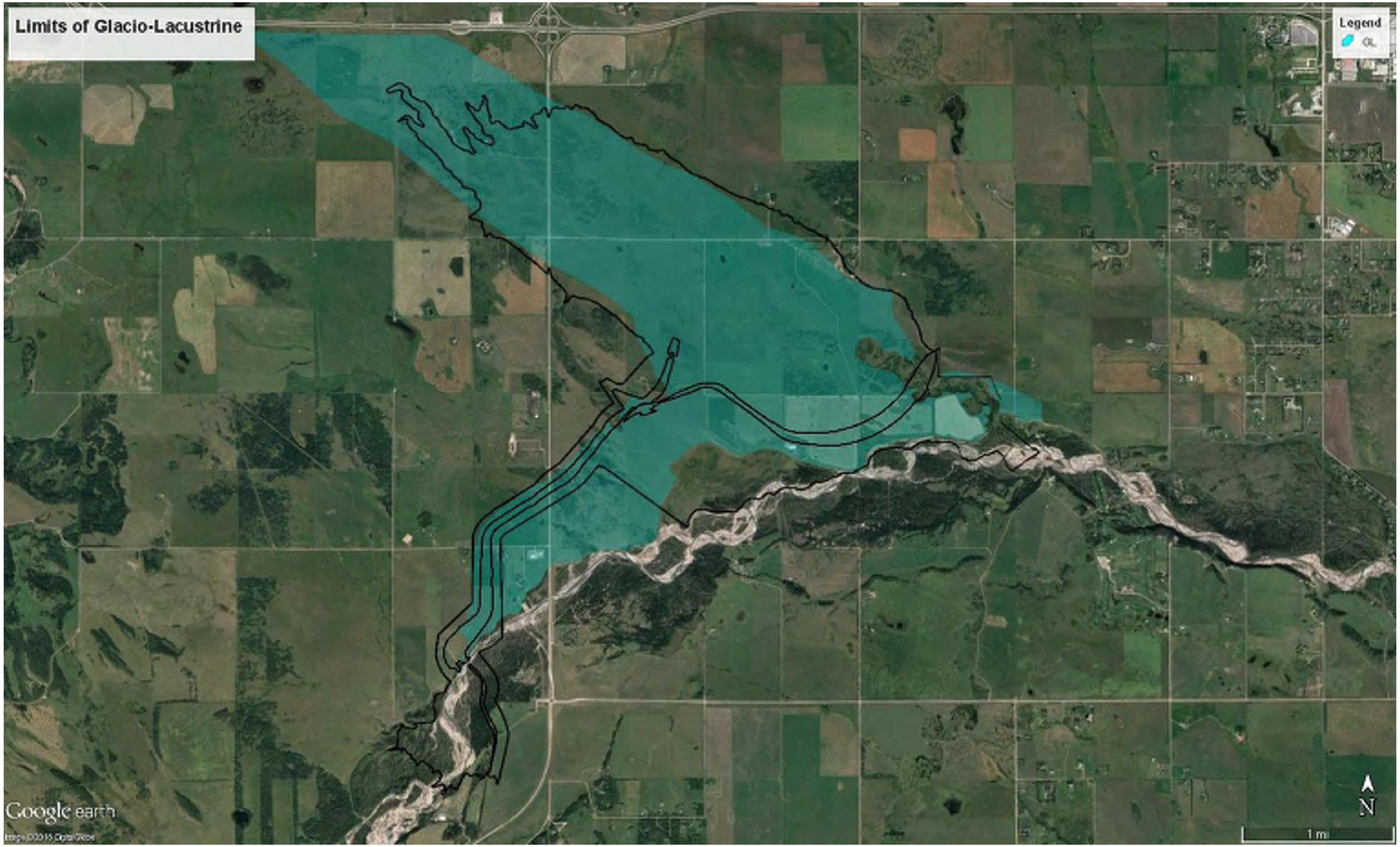

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Figure No.
20-10a

Title
Soil Extents

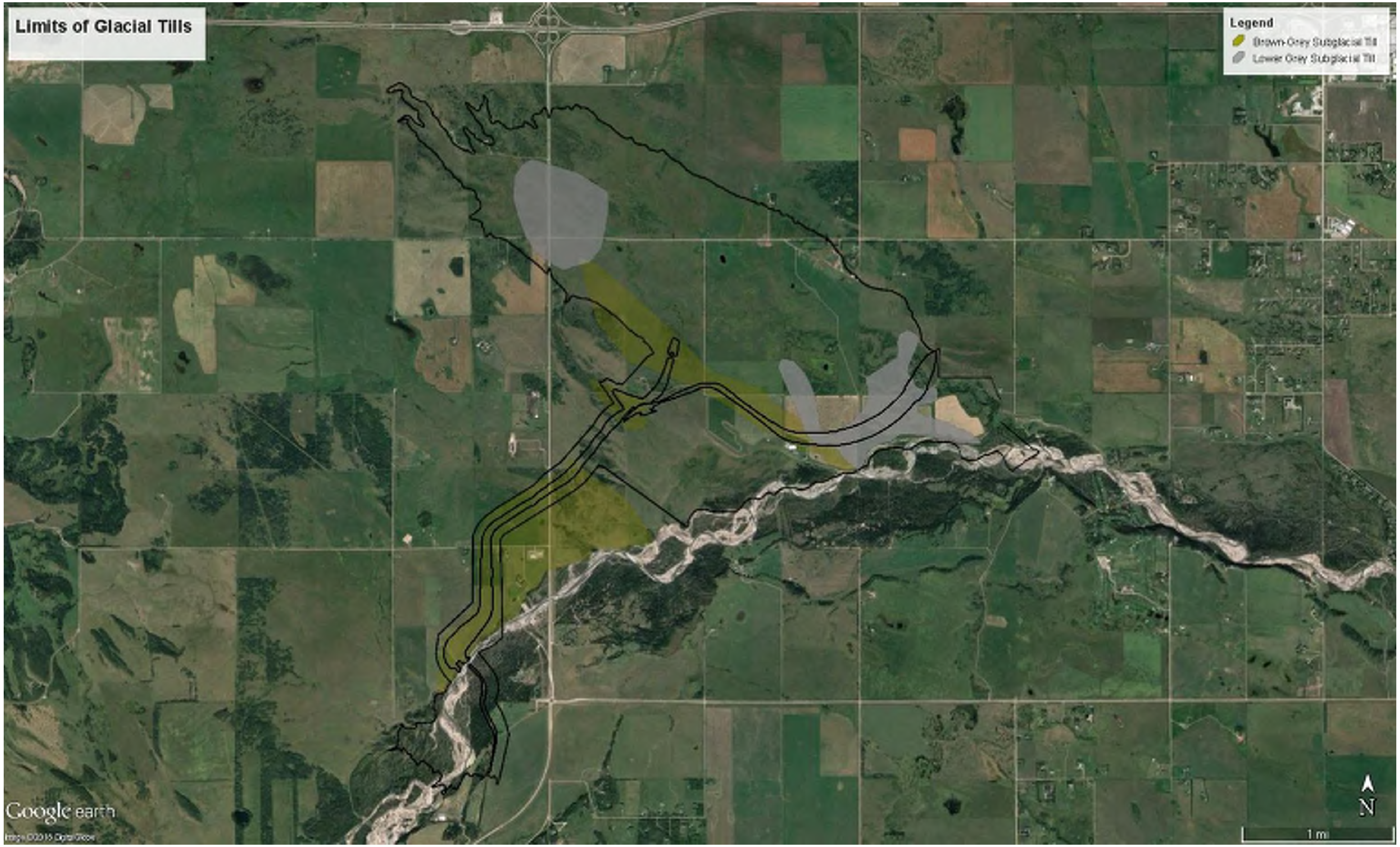

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Figure No.
20-10b

Title
Soil Extents

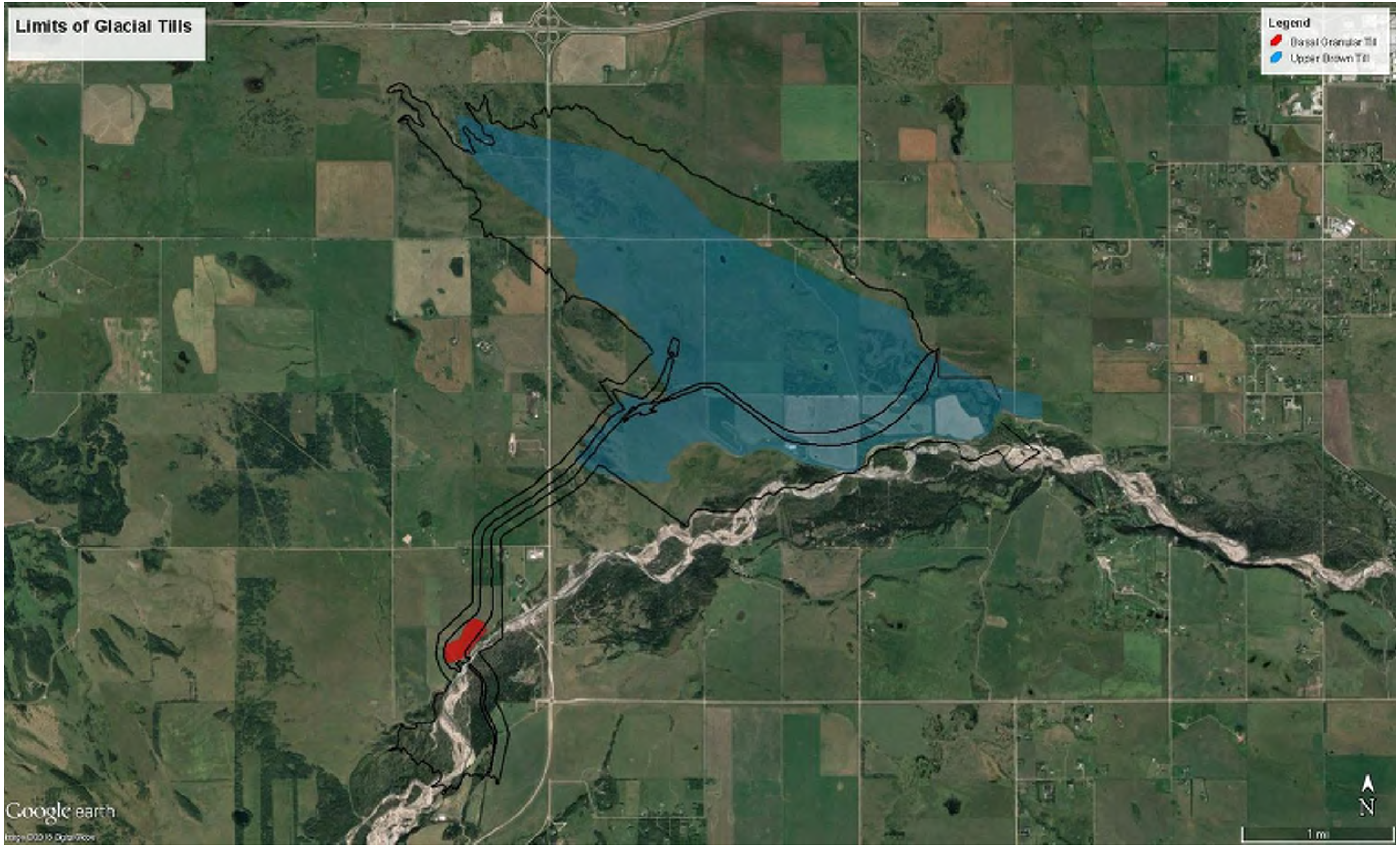

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Figure No.
20-10c

Title
Soil Extents

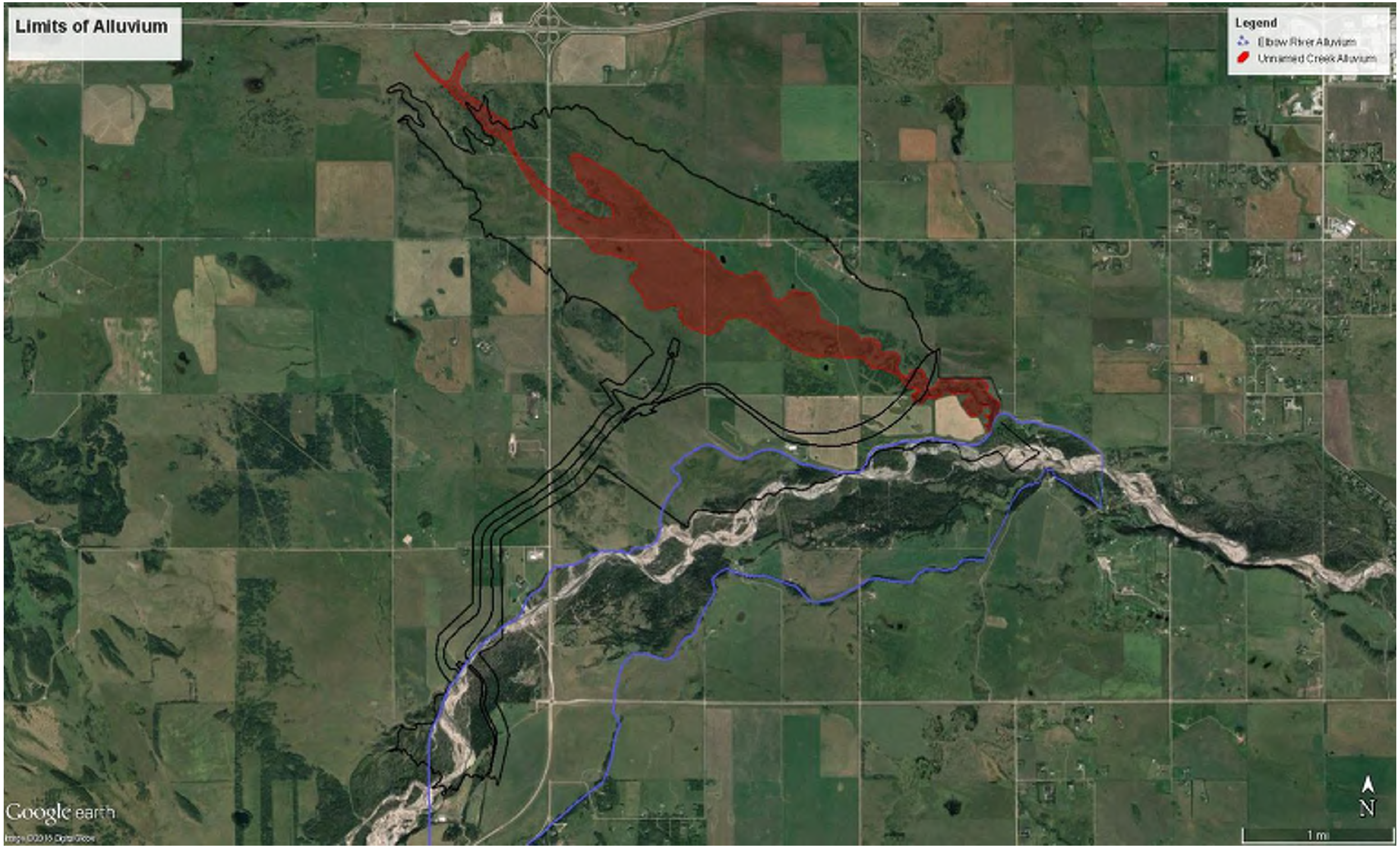

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Figure No.
20-10d

Title
Soil Extents

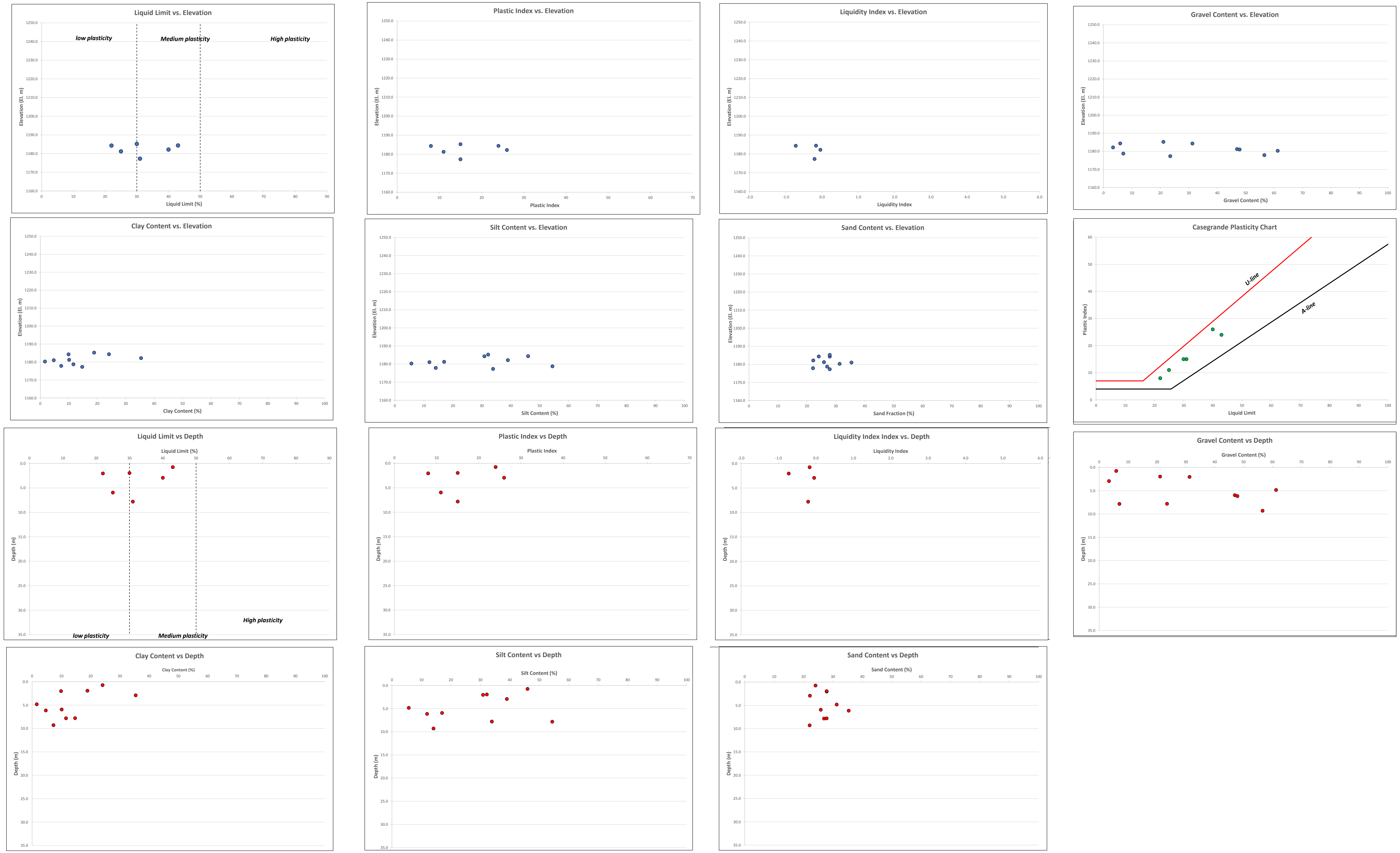

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Figure No.
20-11

Title
Alluvium - Distribution of Index Properties and Particle Sizes

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Notes

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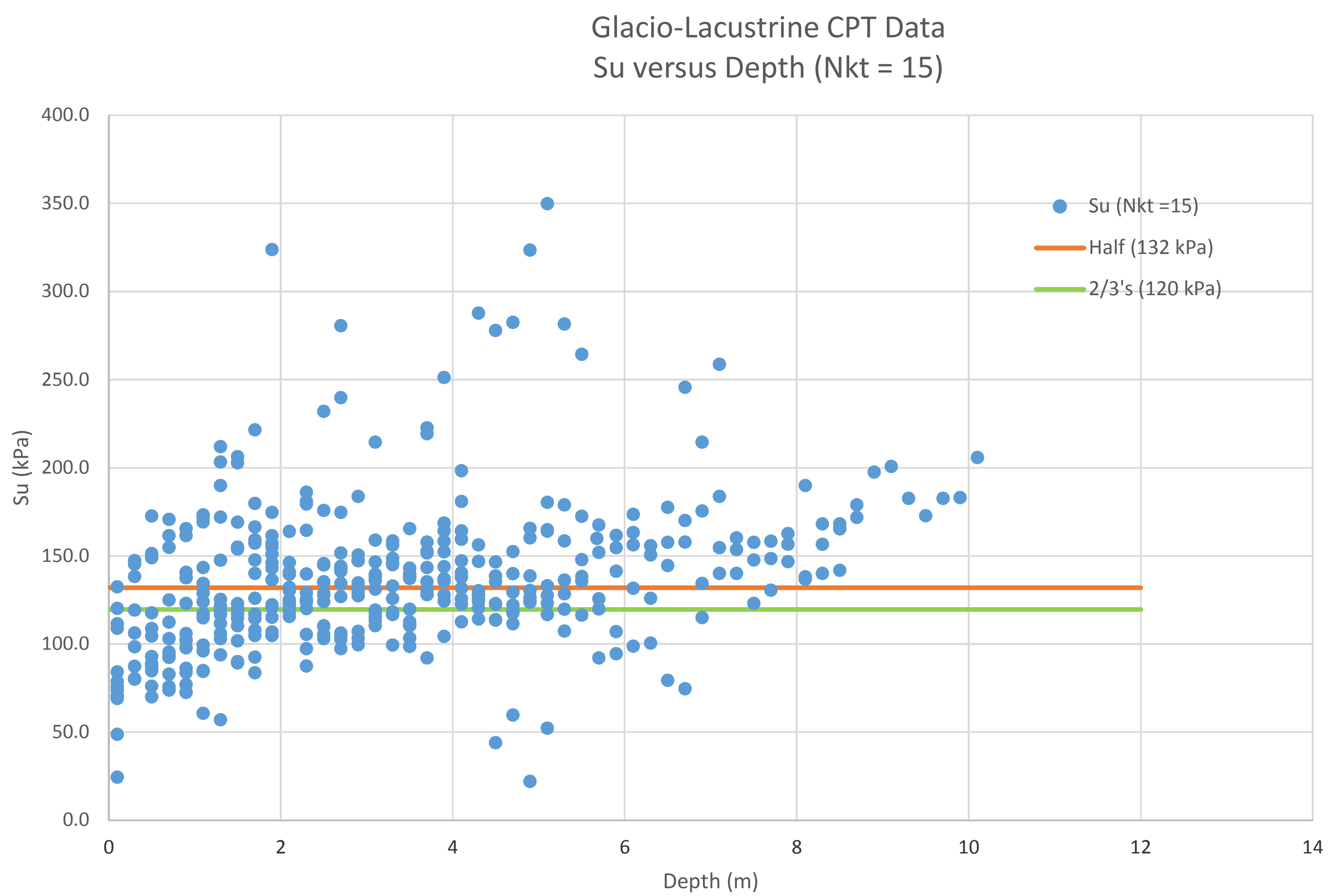
Figure No.
20-12

Title
Glacigenic - Distribution of Index Properties and Particle Sizes



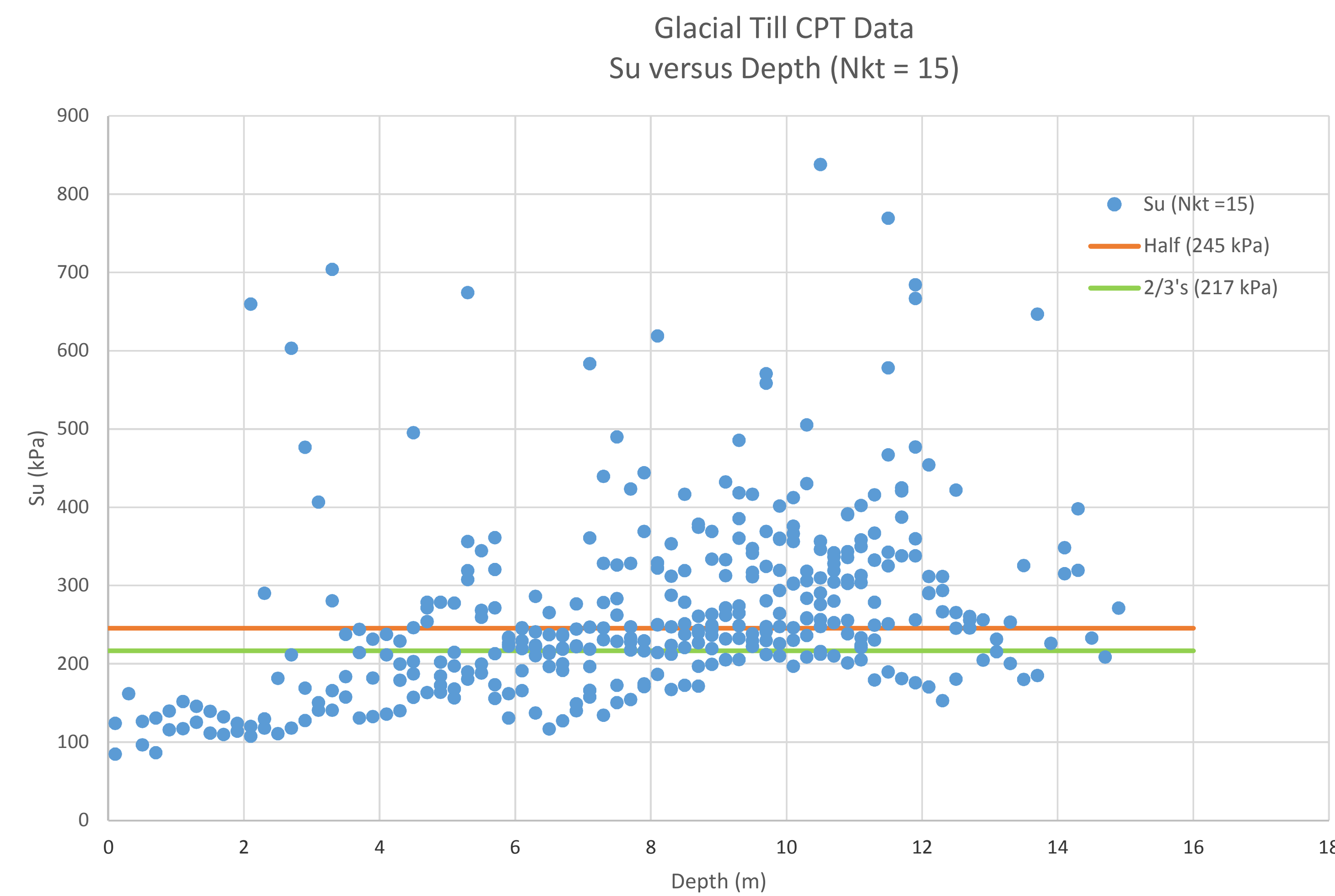


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Figure No.
20.-13

Title
CPT Profile - Undrained Strength

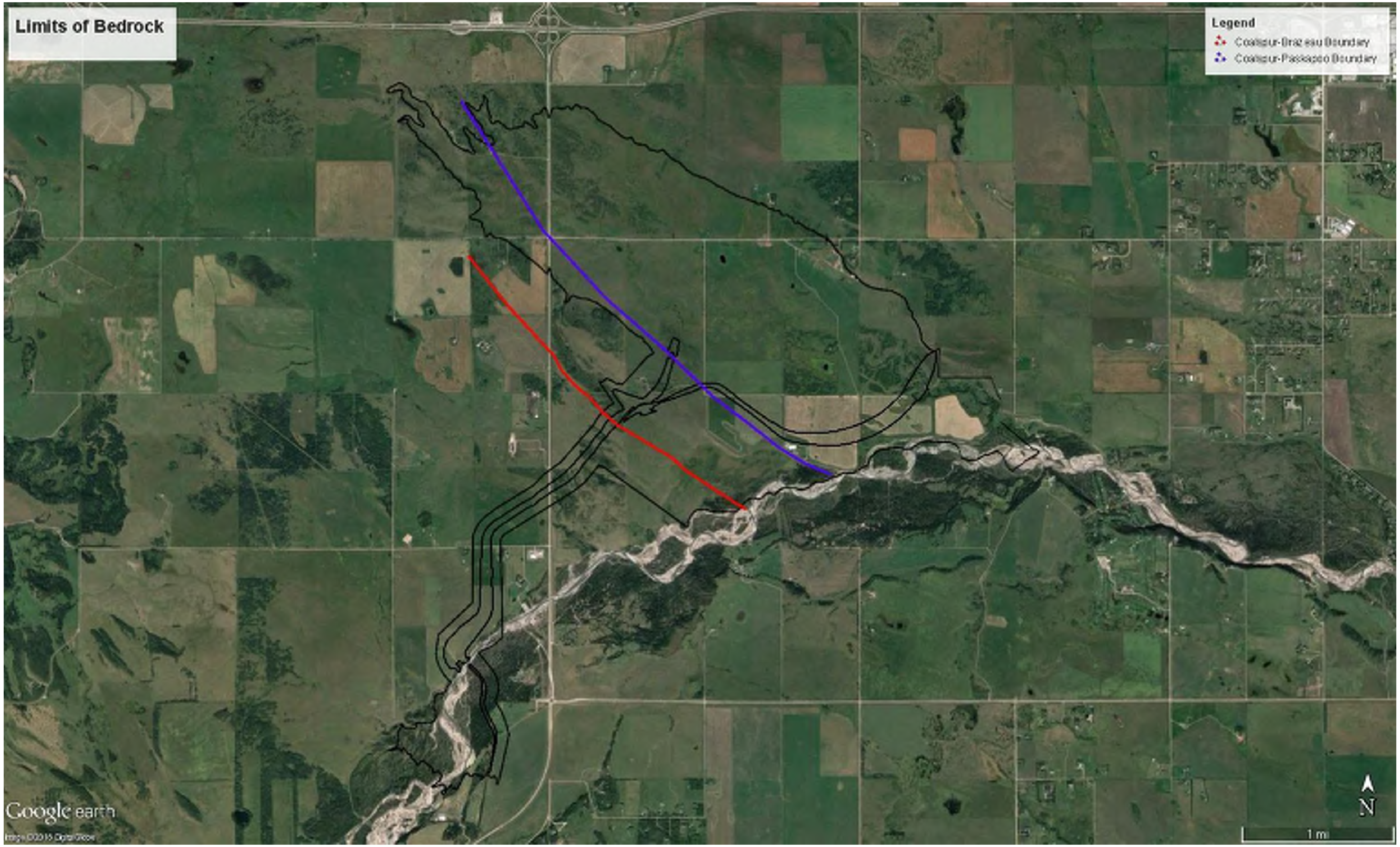

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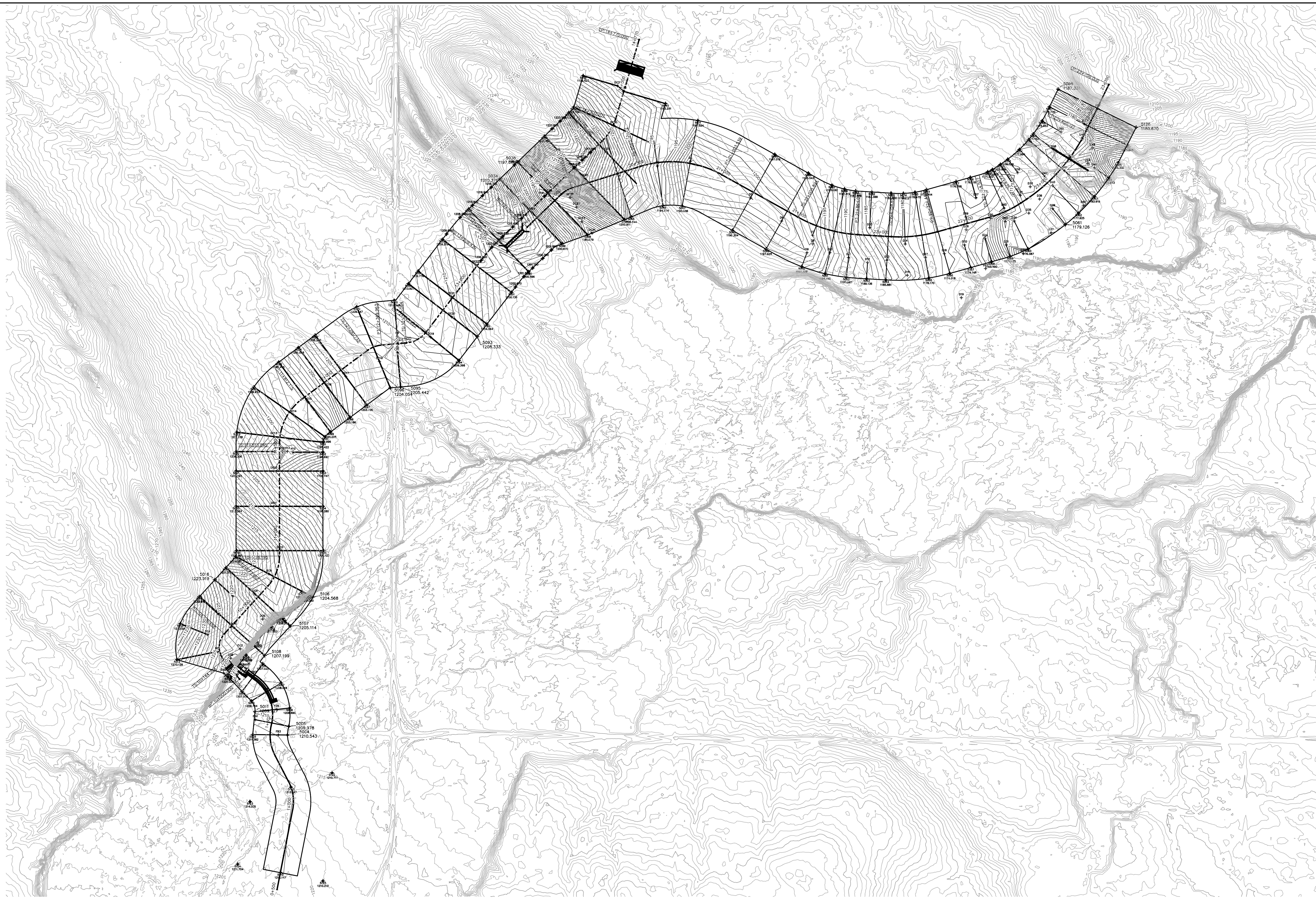
Figure No.

20.-14

Title

Bedrock Extents



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Notes

Top of bedrock surface created from borings and offset points. Offset point elevations used based on depth to rock of nearby borings.

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Figure No.

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Title

Top of Bedrock

Borehole ID	Station	Ground level	Layer Depth Top	Layer Depth Bottom	Layer Elevation Top	Layer Elevation Bottom	Layer Thickness	Soil Unit	Bedrock Type if Present	Fill Classification
DS6A	10,100	1233.3	0.3	2.2	1233	1231.1	1.9	Lacustrine		1A
		1233.3	2.2	7.6	1231.1	1225.7	5.4	Till		1A
DS7	10,100	1233.1	1.2	11.2	1231.9	1221.9	10	Till		1A
		1233.1	11.2	13.7	1221.9	1219.4	2.5	Gravel (Till)		2A
DS7A	10,100	1233.1	1.2	9	1231.9	1224.1	7.8	Till		1A
		1233.1	13	38	1220.1	1195.1	25	BZF	Mudstone	2A
DS8	10,100	1234.4	0.2	13.4	1234.2	1221	13.2	Till		1A
DS8A	10,100	1234.4	0.2	6	1234.2	1228.4	5.8	Till		1A
DC1	10,200	1235.8	0.2	10.8	1235.6	1225	10.6	Till		1A
DC2	10,130	1233.3	0.3	2.2	1233	1231.1	1.9	Lacustrine		1A-CH
		1233.3	2.2	9.6	1233.3	1233.3	0	Till		1A
		1233.3	9.6	12.2	1223.7	1221.1	2.6	Gravel (Till)		2A
DC2A	10,130	1233.3	2.2	9.9	1231.1	1223.4	7.7	Till		1A
		1233.3	9.9	12.4	1223.4	1220.9	2.5	Gravel (Till)		2A
		1233.3	12.4	14	1220.9	1219.3	1.6	BZF	Claystone	2A
DC3	10,270	1239.8	0.3	9.1	1239.5	1230.7	8.8	Till		1A
		1239.8	9.1	11.4	1230.7	1228.4	2.3	Gravel (Till)		2A
DC4	10,370	1238	0.2	16.5	1237.8	1221.5	16.3	Till		1A
		1238	16.5	17.2	1221.5	1220.8	0.7	Till		2A
DC4A	10,370	1238	0.2	9	1237.8	1229	8.8	Till		1A
DC5	10,370	1242.1	2.1	16.6	1240	1225.5	14.5	Till		1A
		1242.1	16.6	23.1	1225.5	1219	6.5	BZF	Claystone	2A
DC6	10,520	1230.1	0.3	3.3	1229.8	1226.8	3	Lacustrine		1A-CH
		1230.1	3.3	12.4	1226.8	1217.7	9.1	Till		1A
		1230.1	12.4	13.3	1217.7	1216.8	0.9	BZF	Mudstone	1A
		1230.1	13.3	26.2	1216.8	1203.9	12.9	BZF	Mudstone	2A
DC7	10,520	1241	0.3	5.5	1240.7	1235.5	5.2	Till		1A
		1241	5.5	6	1235.5	1235	0.5	Till		2A
		1241	6	20.7	1235	1220.3	14.7	Till		1A
		1241	20.7	23.1	1220.3	1217.9	2.4	BZF	Mudstone	2A
		1241	23.1	25.3	1217.9	1215.7	2.2	BZF	Clay Infill	2B
		1241	25.3	26.7	1215.7	1214.3	1.4	BZF	Mudstone	2B
		1241	26.7	27.2	1214.3	1213.8	0.5	BZF	Clay Infill	2B
		1241	27.2	27.6	1213.8	1213.4	0.4	BZF	Mudstone	2A
DC7A	10,520	1241	28.2	36.4	1212.8	1204.6	8.2	BZF	Siltstone	2B
DC-8	10,590	1232.4	0.3	2.1	1232.1	1230.3	1.8	Lacustrine		1A-CH
		1232.4	2.1	18.7	1230.3	1213.7	16.6	Till		1A
		1232.4	18.7	27.6	1213.7	1204.8	8.9	BZF	Claystone	2A
DC9A	10,740	1227.4	0.3	2	1227.1	1225.4	1.7	Lacustrine		1A-CH
		1227.4	2	10.8	1225.4	1216.6	8.8	Till		1A
DC10	10,900	1225.8	0.3	7	1225.5	1218.8	6.7	Till		1A
		1225.8	7	12.4	1218.8	1213.4	5.4	Till		1A
DC11	11,150	1224	2.3	18.5	1221.7	1205.5	16.2	Till		1A
DC12	11,340	1222	0.3	2.8	1221.7	1219.2	2.5	Lacustrine		1A
		1222	2.8	4.6	1219.2	1217.4	1.8	Till		1A
		1222	4.6	11.5	1217.4	1210.5	6.9	Till		1A
		1222	11.5	12.5	1210.5	1209.5	1	Clay		1A
		1222	12.5	19.2	1209.5	1202.8	6.7	Till		1A
DC13	11,550	1219.1	2	17.4	1217.1	1201.7	15.4	Till		1A
DC14	11,710	1215	0.4	5	1214.6	1210	4.6	Lacustrine		1A-CH
		1215	5	19.1	1210	1195.9	14.1	Till		2A
DC15	11,830	1213.4	0.2	5.1	1213.2	1208.3	4.9	Lacustrine		1A-CH
		1213.4	5.1	8.2	1208.3	1205.2	3.1	Till		1A
		1213.4	8.2	11.2	1205.2	1202.2	3	Till		2A
DC15A	11,830	1213.4	5.1	8.2	1208.3	1205.2	3.1	Till		1A
DC16	11,970	1213.9	2.1	5.2	1211.8	1208.7	3.1	Till		1A
		1213.9	5.2	8.6	1208.7	1205.3	3.4	Till (Silt)		2A
		1213.9	8.6	11.2	1205.3	1202.7	2.6	Till		2A
DC17	12,090	1213.4	0.3	2.1	1213.1	1211.3	1.8	Lacustrine		1A-CH
		1213.4	2.1	6	1211.3	1207.4	3.9	Till		1A
		1213.4	6	7.6	1207.4	1205.8	1.6	Till		2A

Borehole ID	Station	Ground level	Layer Depth Top	Layer Depth Bottom	Layer Elevation Top	Layer Elevation Bottom	Layer Thickness	Soil Unit	Bedrock Type if Present	Fill Classification
DC18	12,320	1213.9	0.3	2.7	1213.6	1211.2	2.4	Lacustrine		1A-CH
		1213.9	2.7	4.2	1211.2	1209.7	1.5	Till		1A
		1213.9	4.2	5.7	1209.7	1208.2	1.5	Till		2A
		1213.9	5.7	6.9	1208.2	1207	1.2	Till		1A
DC19	12,630	1217.6	0.3	3	1217.3	1214.6	2.7	Lacustrine		1A-CH
		1217.6	3	4	1214.6	1213.6	1	Till		1A-CH
		1217.6	4	5.4	1213.6	1212.2	1.4	Till (Sand)		2A
DC20	12,800	1216.3	0.3	1.5	1216	1214.8	1.2	Lacustrine		1A-CH
		1216.3	1.5	5.8	1214.8	1210.5	4.3	Till		2A
DC21	12,900	1215.8	0.4	2.4	1215.4	1213.4	2	Lacustrine		1A-CH
		1215.8	2.4	3	1213.4	1212.8	0.6	Till		2A
		1215.8	3	5.1	1212.8	1210.7	2.1	Till		1A
DC-22	13,120	1211.9	0.2	5.1	1211.7	1206.8	4.9	Lacustrine		1A-CH
		1211.9	5.1	6.4	1206.8	1205.5	1.3	Clay		1A
DC-23	13,320	1213.9	2.7	4.6	1211.2	1209.3	1.9	Till		1A
DC-24	13,570	1211.2	2.7	6.4	1208.5	1204.8	3.7	Till		1A
DC-25	13,710	1205	0.3	2	1204.7	1203	1.7	Lacustrine		1A
		1205	2	3	1203	1202	1	Lacustrine		1A-CH
		1205	3	4	1202	1201	1	Lacustrine		1A-CH
		1205	4	7.6	1201	1197.4	3.6	Till		1A
DC-27	13,830	1202.7	1.5	6.3	1201.2	1196.4	4.8	Till		1A
		1202.7	6.3	10.4	1196.4	1192.3	4.1	Till (Silt)		2A
DC-30	14,300	1211.4	0.2	3.1	1211.2	1208.3	2.9	Lacustrine		1A-CH
		1211.4	3.1	8.1	1208.3	1203.3	5	Till		1A
DC-32	14,500	1207.1	0.2	2.2	1206.9	1204.9	2	Lacustrine		1A
		1207.1	2.2	3	1204.9	1204.1	0.8	Till		1A
		1207.1	3	3.2	1204.1	1203.9	0.2	Till		2A
		1207.1	3.2	10.7	1203.9	1196.4	7.5	Till		1A
DC-33	13,820	1199.5	0.4	3.9	1199.1	1195.6	3.5	Lacustrine		1A-CH
		1199.5	3.9	7.6	1195.6	1191.9	3.7	Till		1A
		1199.5	7.6	10.9	1191.9	1188.6	3.3	Till		1A
DC-34	13,830	1198.5	0.5	4.6	1198	1193.9	4.1	Lacustrine		1A-CH
		1198.5	4.6	5.7	1193.9	1192.8	1.1	Lacustrine		1A
		1198.5	5.7	13.1	1192.8	1185.4	7.4	Till		1A
BS1		1198.9	0.4	4	1198.5	1194.9	3.6	Lacustrine		1A
		1198.9	4	12.8	1194.9	1186.1	8.8	Till		1A
BS2		1204.1	0.5	1.4	1203.6	1202.7	0.9	Lacustrine		1A
		1204.1	1.4	6.1	1202.7	1198	4.7	Lacustrine		1A
		1204.1	6.1	9.2	1198	1194.9	3.1	Till		1A
BS3		1197.4	0.3	6.1	1197.1	1191.3	5.8	Lacustrine		1A-CH
		1197.4	6.1	9.1	1191.3	1188.3	3	Till		1A
BS4		1199.4	0.4	1.5	1199	1197.9	1.1	Lacustrine		1A-CH
		1199.4	1.5	4.6	1197.9	1194.8	3.1	Lacustrine		1A-CH
		1199.4	4.6	11.6	1194.8	1187.8	7	Till		1A
BS5		1190.4	0.3	2.5	1190.1	1187.9	2.2	Lacustrine		2A
		1190.4	2.5	16.8	1187.9	1173.6	14.3	Till		1A
H1	12,460	1214.1	0.2	2.4	1213.9	1211.7	2.2	Lacustrine		1A
H2	12,460	1214.9	6.6	30.1	1208.3	1184.8	23.5	Bedrock	Mudstone	1A
H3	12,460	1215.6	0.2	3	1215.4	1212.6	2.8	Fill (Clay)		1A
		1215.6	3	6.8	1212.6	1208.8	3.8	Till (Silt)		2A
		1215.6	6.8	8.4	1208.8	1207.2	1.6	Till		1A
H4	12,460	1215.9	0.2	1.7	1215.7	1214.2	1.5	Fill (Clay)		2A
H11	11,470	1219.5	0.6	3	1218.9	1216.5	2.4	Lacustrine		1A
		1219.5	3	19.8	1216.5	1199.7	16.8	Till		1A
H12	11,470	1217.6	1	7.6	1216.6	1210	6.6	Lacustrine		1A
H13	11,470	1217.1	1.5	7.2	1215.6	1209.9	5.7	Lacustrine		1A-CH
		1217.1	7.2	9.1	1209.9	1208	1.9	Lacustrine		1A
		1217.1	9.1	15.2	1208	1201.9	6.1	Till		1A

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Notes

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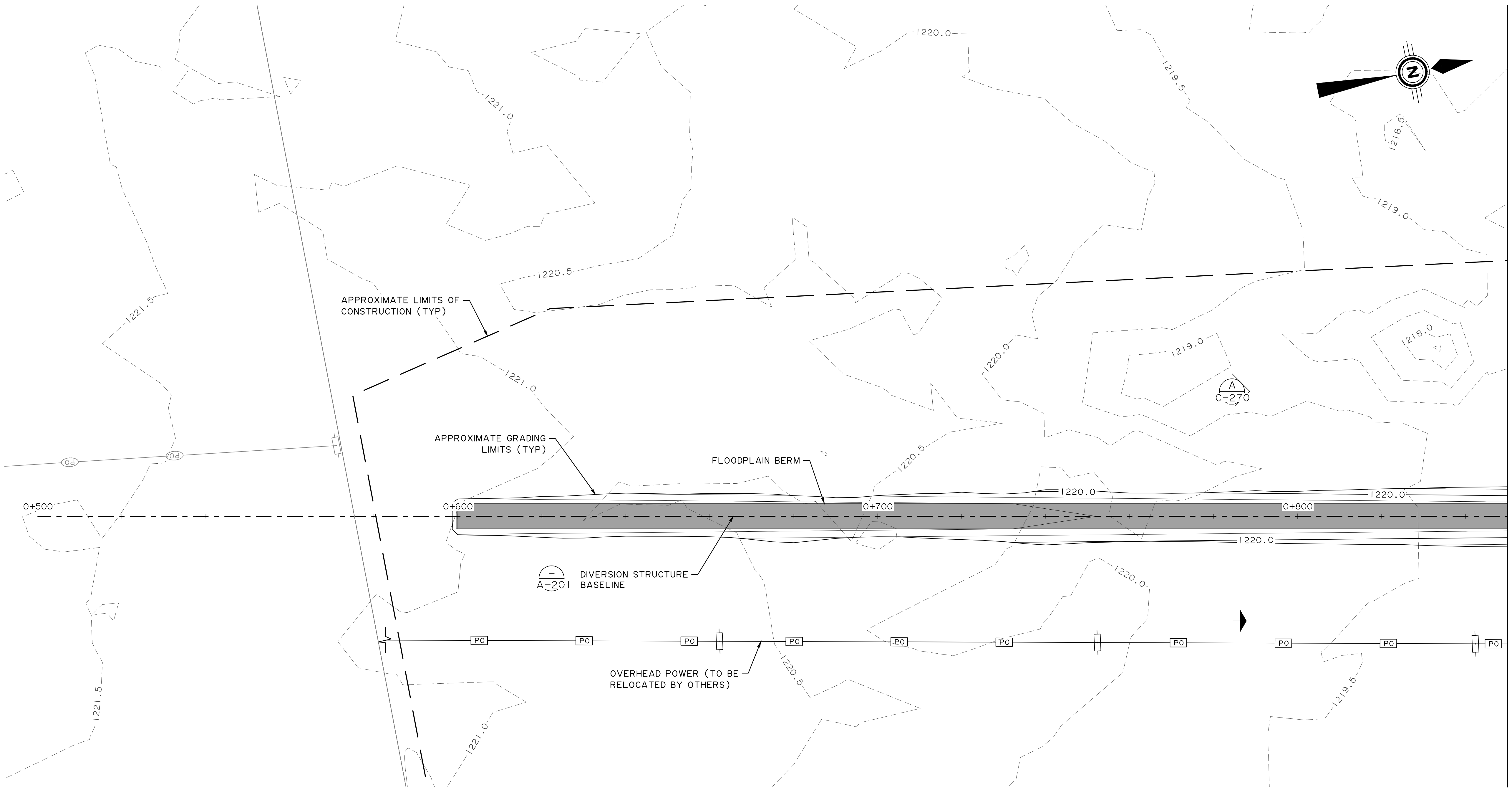
Preliminary Geotechnical Design Report

Figure No.
20.-16

Title
Earthworks Classification



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. FOR EXISTING UTILITY DEMOLITION SEE SHEETS C-108 TO C-117. UTILITIES REMOVED ARE NOT SHOWN ON THIS SHEET FOR CLARITY.

PLAN
1:500

0 20 40 Metres

ORIGINAL SHEET - ANSI B

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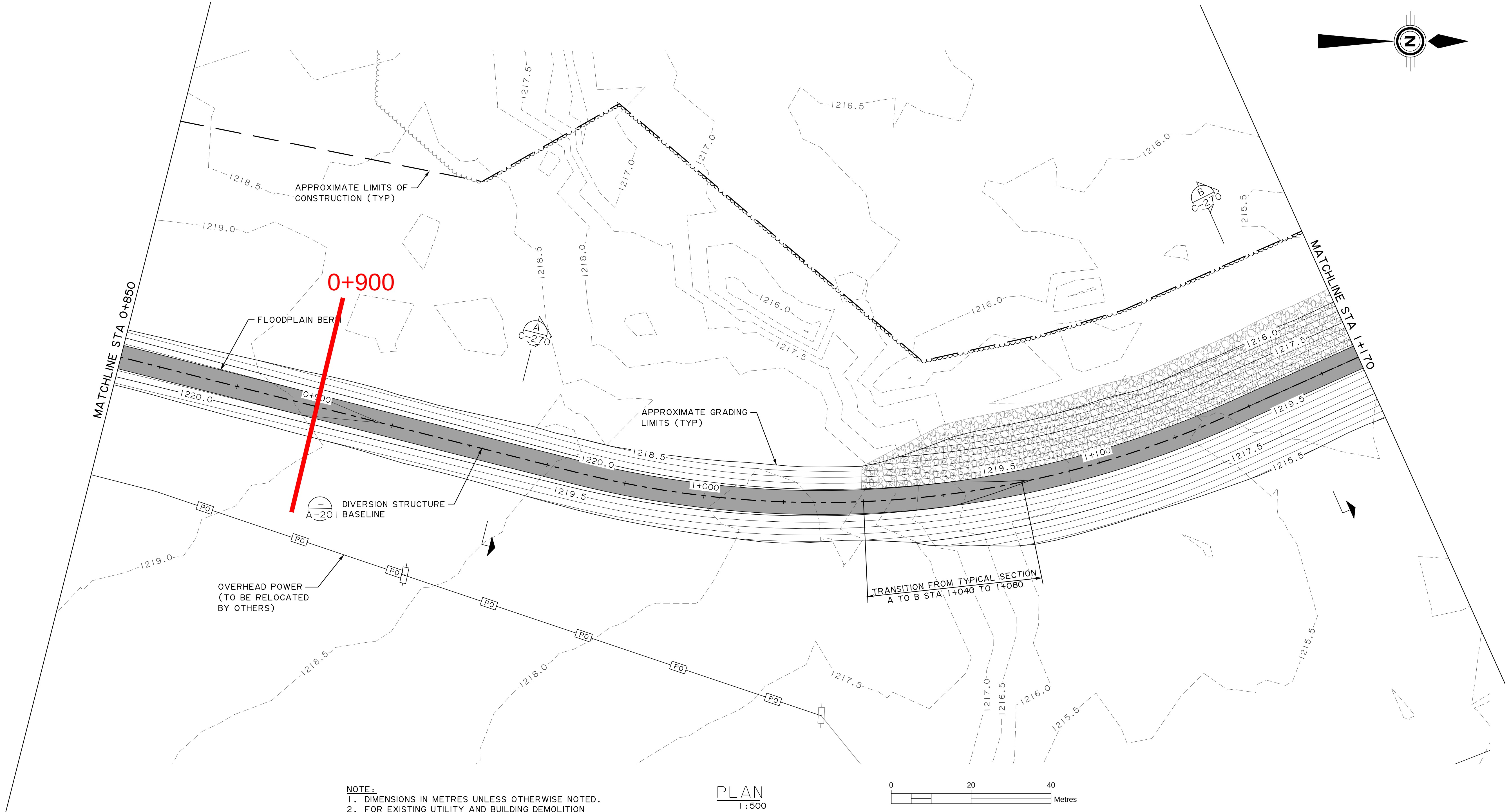
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17a

Title
Slope /W Profile Location Plan



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. FOR EXISTING UTILITY AND BUILDING DEMOLITION SEE SHEETS C-108 TO C-117. UTILITIES REMOVED ARE NOT SHOWN ON THIS SHEET FOR CLARITY.

PLAN
1:500

ORIGINAL SHEET - ANSI B

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Notes

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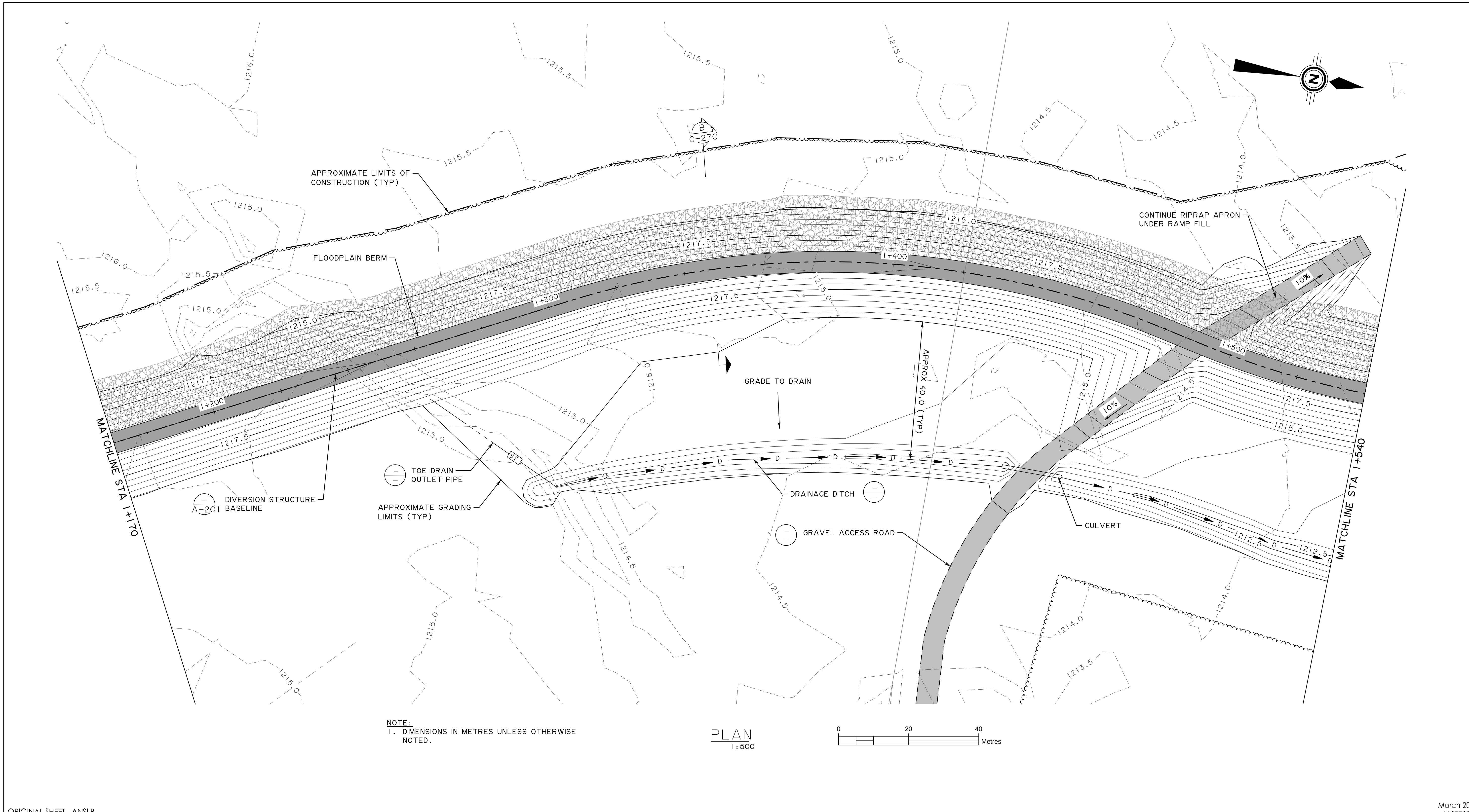
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17b

Title
Slope /W Profile Location Plan



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Notes

Client/Project

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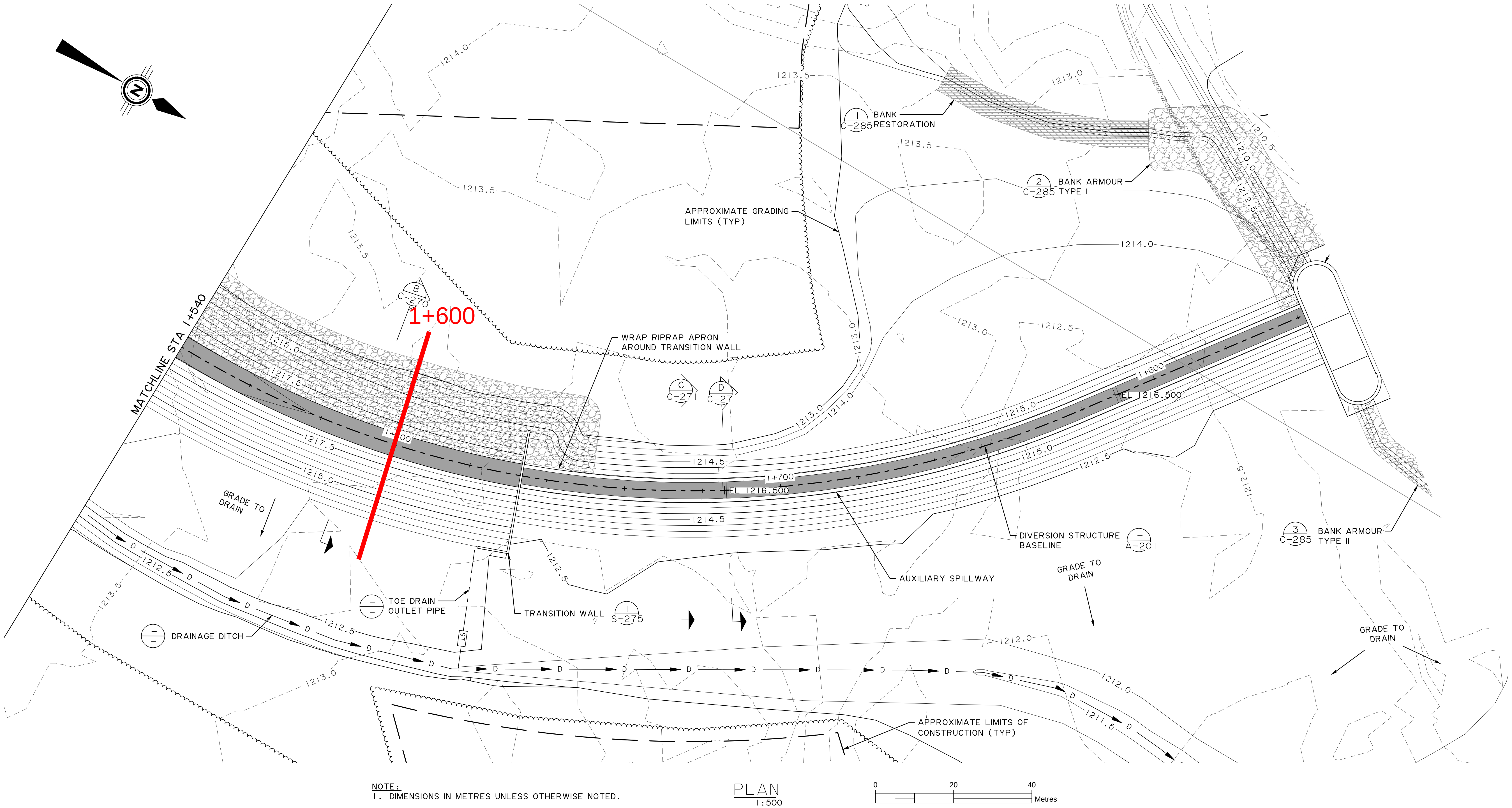
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.

20.-17c

Title

Slope /W Profile Location Plan



ORIGINAL SHEET - ANSI B

March 2017
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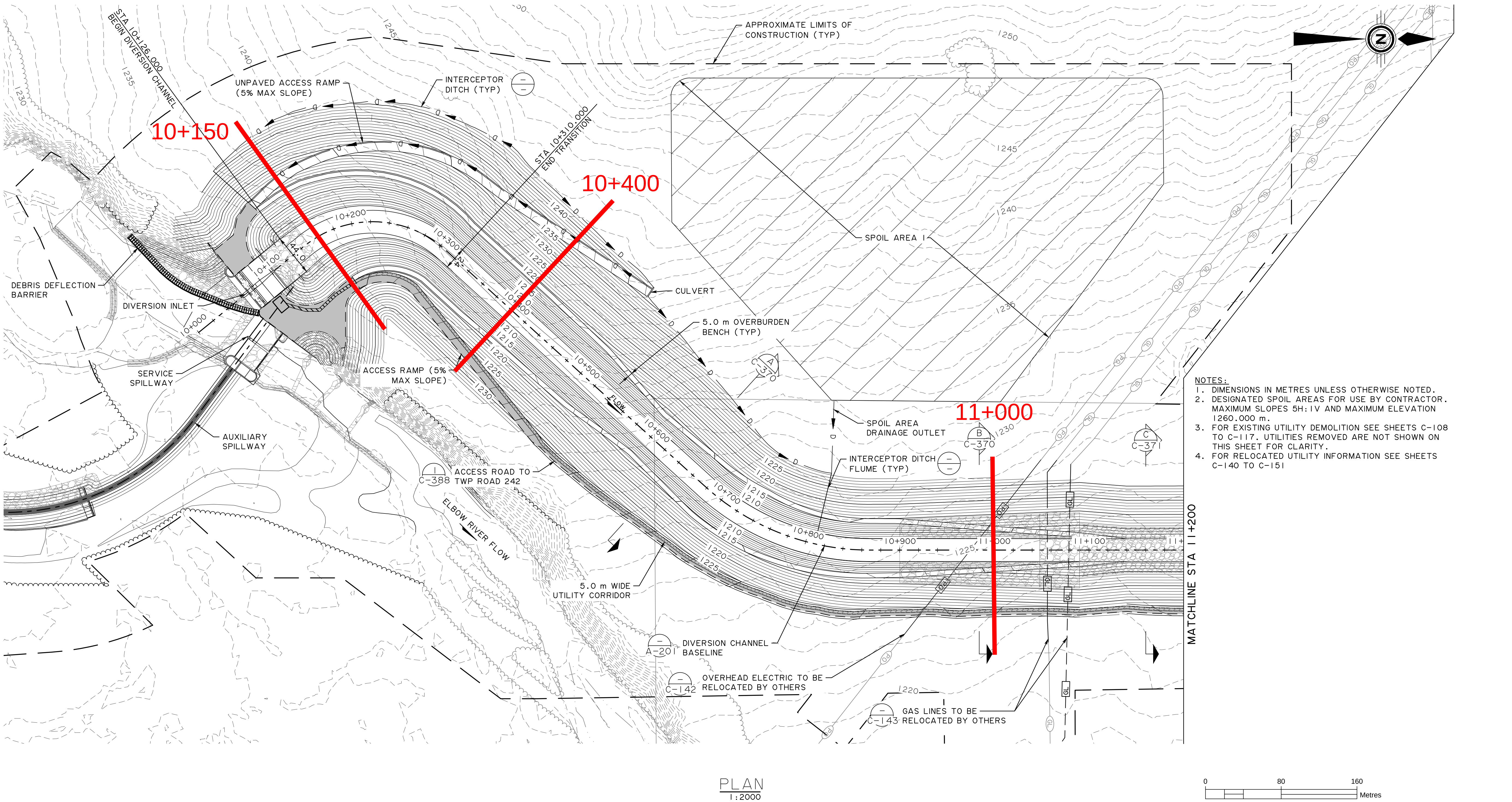
Notes

Client/Project
Alberta Transportation
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17d

Title
Slope /W Profile Location Plan



NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. DESIGNATED SPOIL AREAS FOR USE BY CONTRACTOR. MAXIMUM SLOPES 5H:1V AND MAXIMUM ELEVATION 1260.000 m.
3. FOR EXISTING UTILITY DEMOLITION SEE SHEETS C-108 TO C-117. UTILITIES REMOVED ARE NOT SHOWN ON THIS SHEET FOR CLARITY.
4. FOR RELOCATED UTILITY INFORMATION SEE SHEETS C-140 TO C-151

ORIGINAL SHEET - ANSI B

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Notes

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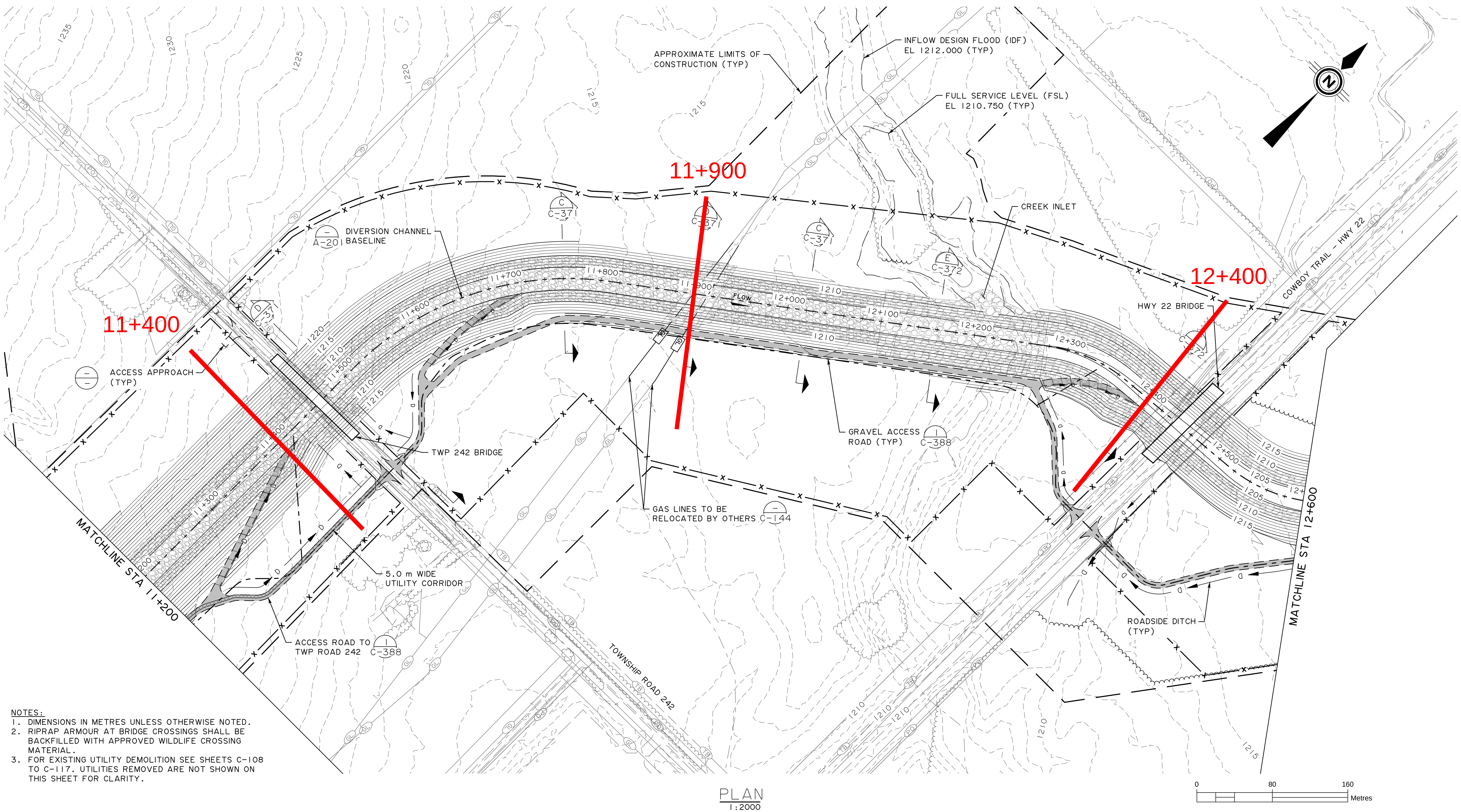
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20-17e

Title
Slope /W Profile Location Plan



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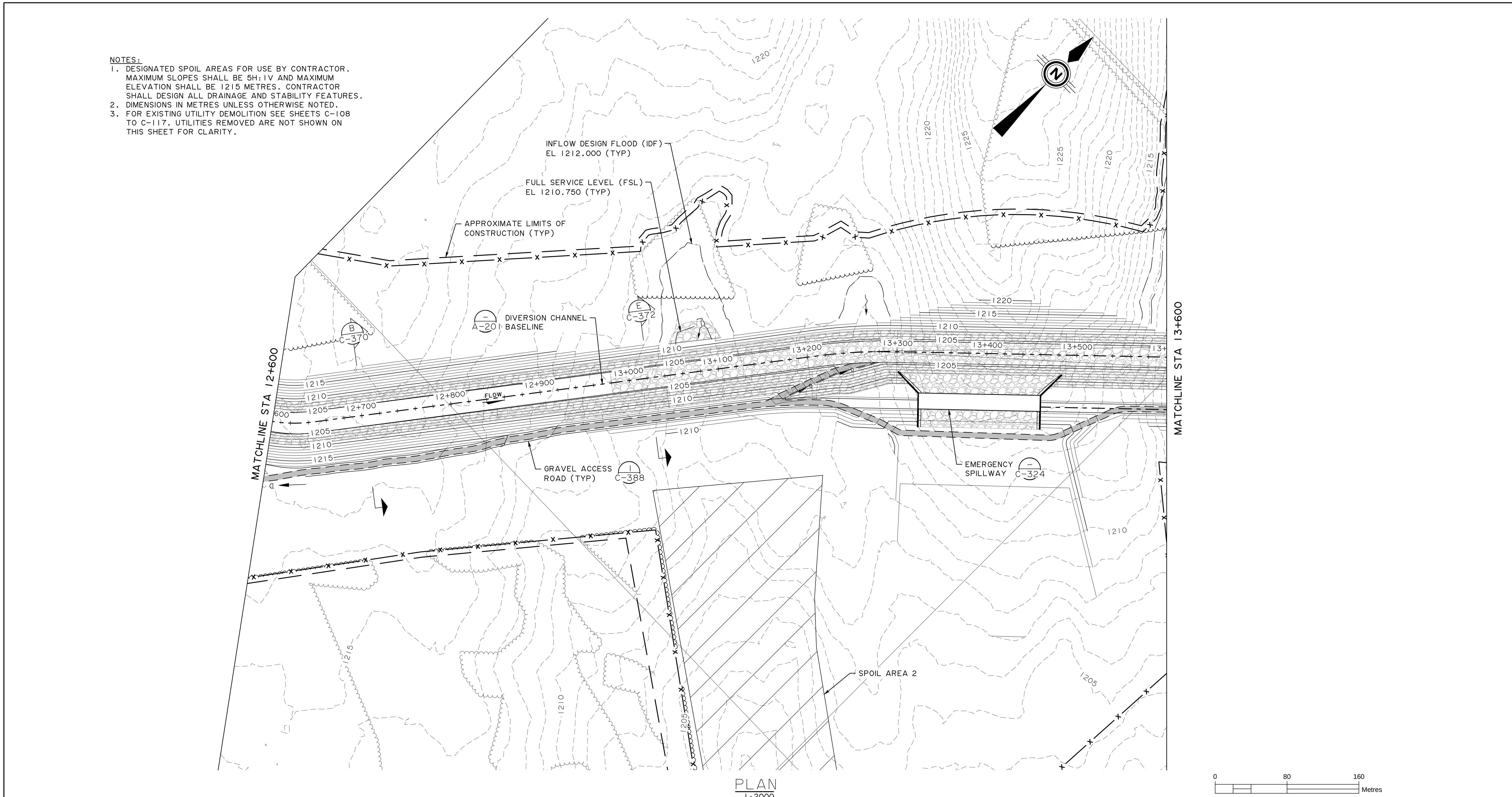
Preliminary Geotechnical Design Report

Figure No.

20.-17f

Title

Slope /W Profile Location Plan



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Notes

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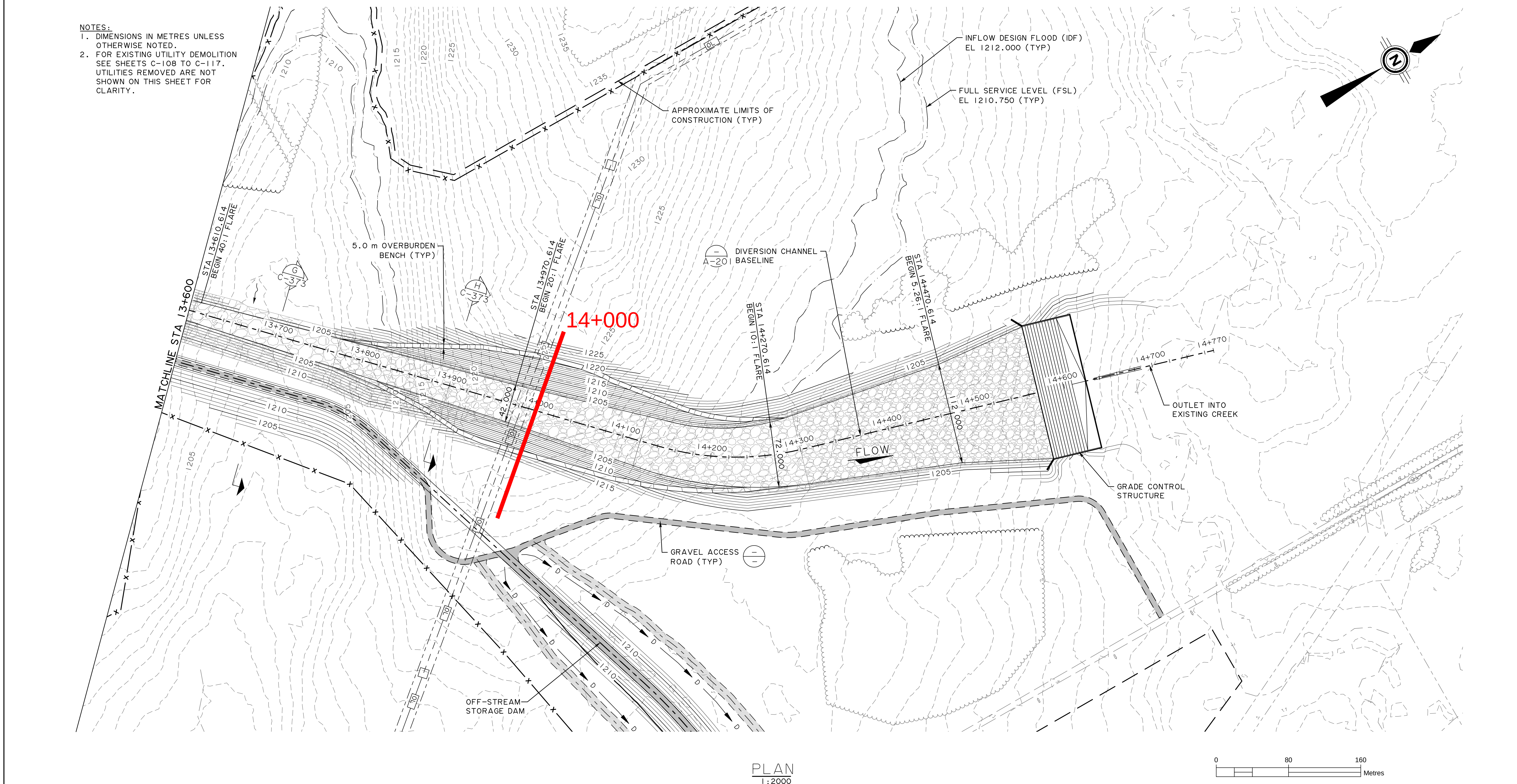
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17g

Title
Slope /W Profile Location Plan



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Notes

Client/Project

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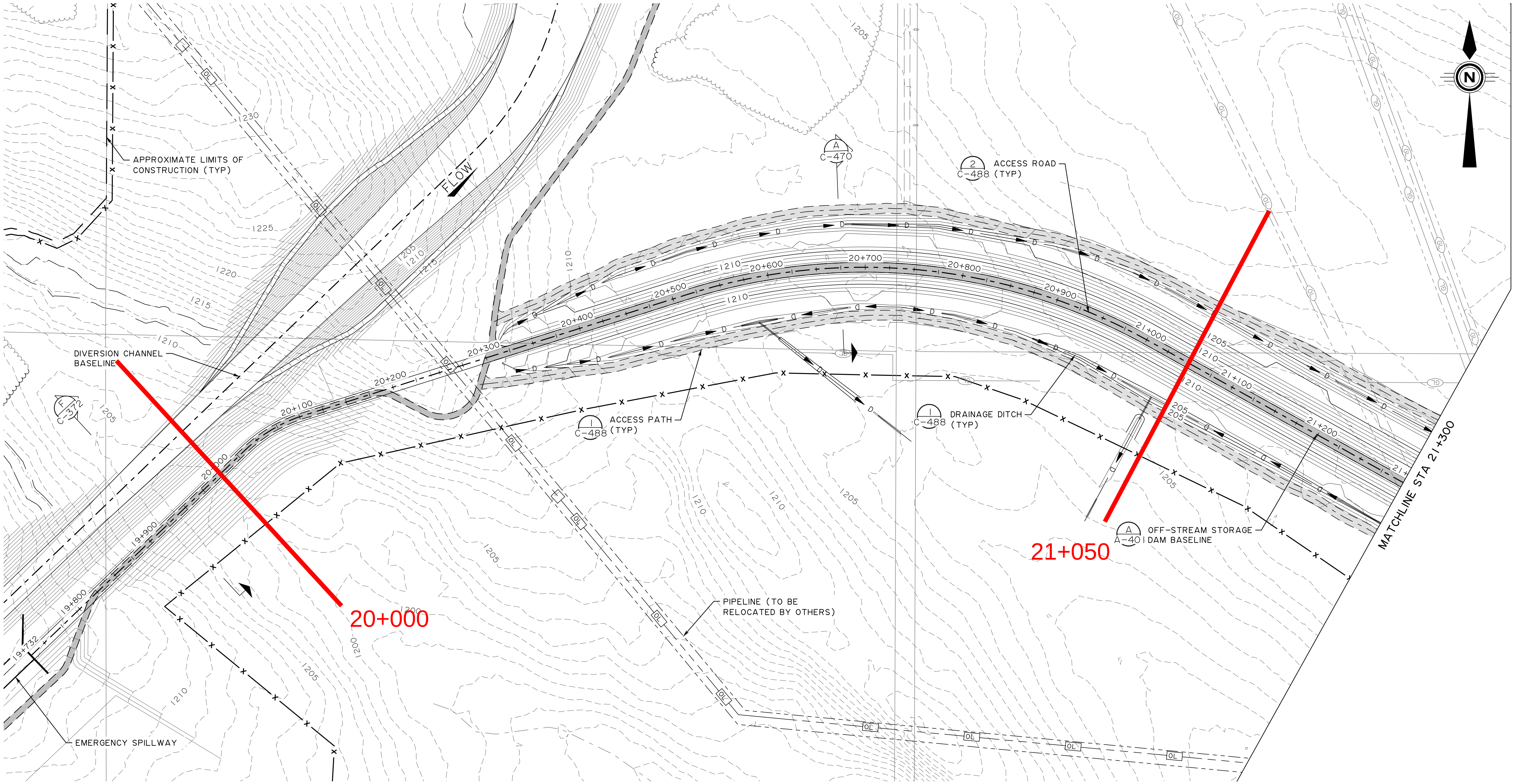
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.

20.-17h

Title

Slope /W Profile Location Plan



PLAN
1:2000

0 80 160
Metres

NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

ORIGINAL SHEET - ANSI B

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Notes

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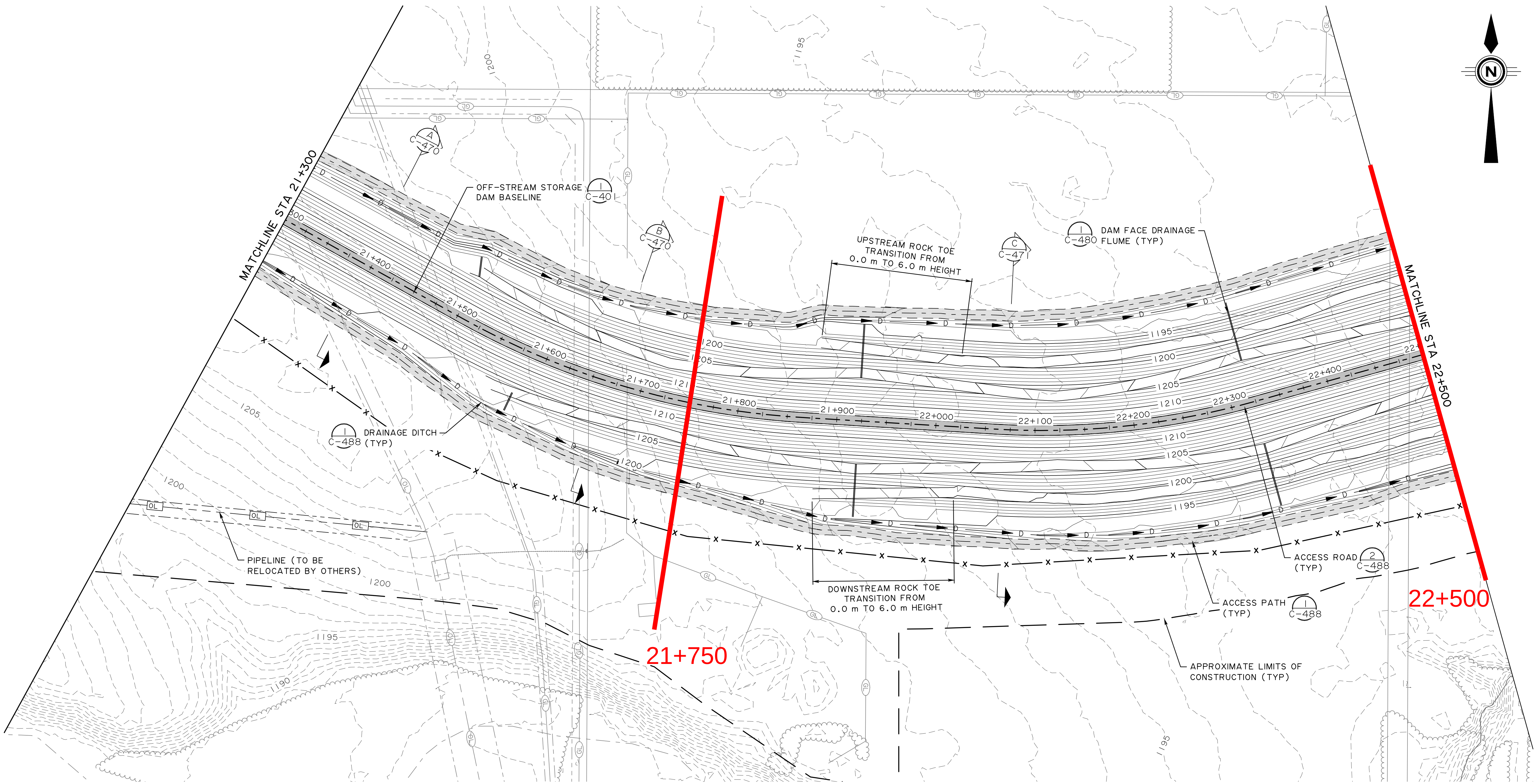
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17i

Title
Geological Profile - Storage Dam



NOTES:
1. CONTOURS DO NOT REFLECT OVERBUILD FOR DAM SETTLEMENT.
2. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:2000

0 80 160 Metres

ORIGINAL SHEET - ANSI B

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Notes

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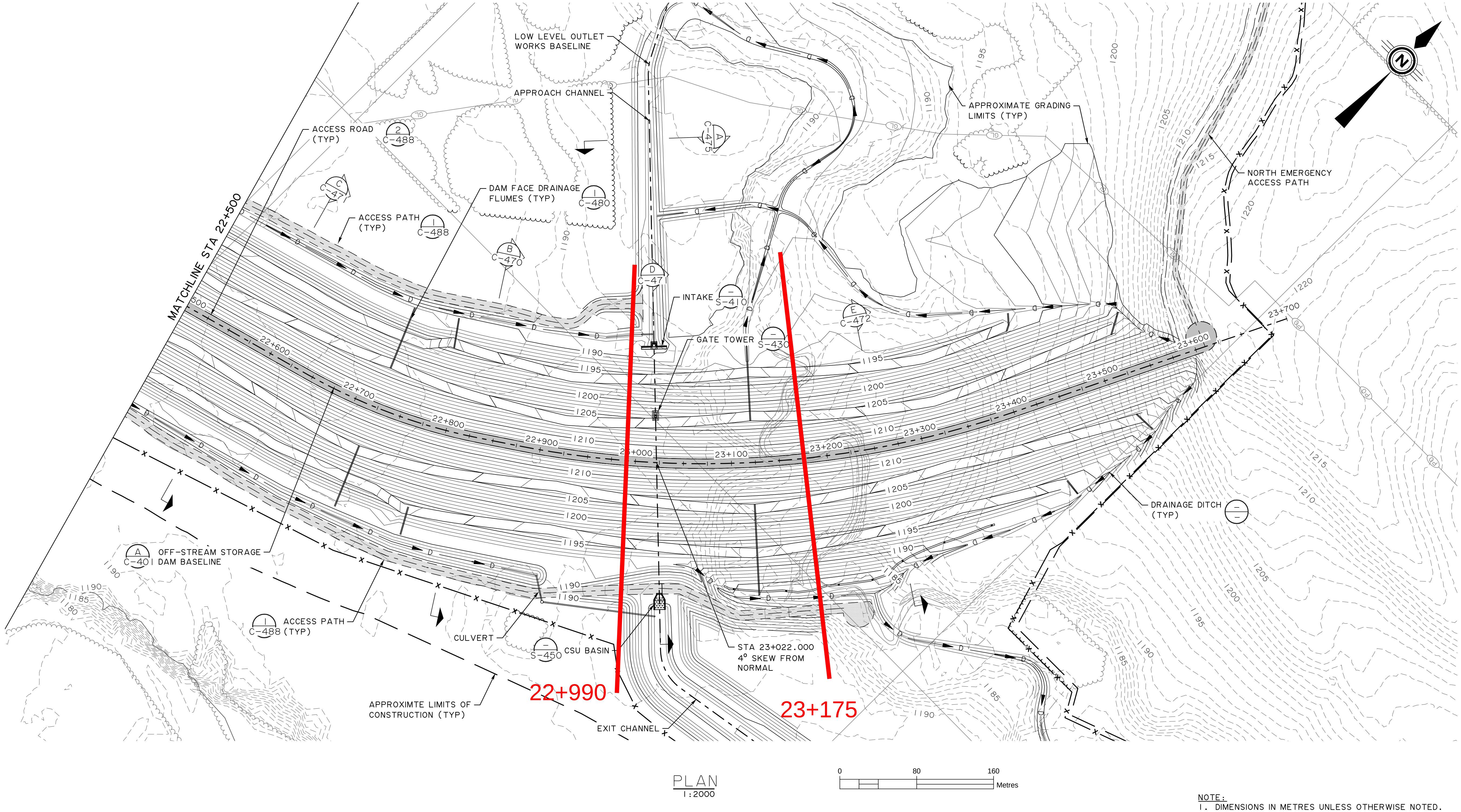
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17j

Title
Geological Profile - Storage Dam



ORIGINAL SHEET - ANSI B

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Notes

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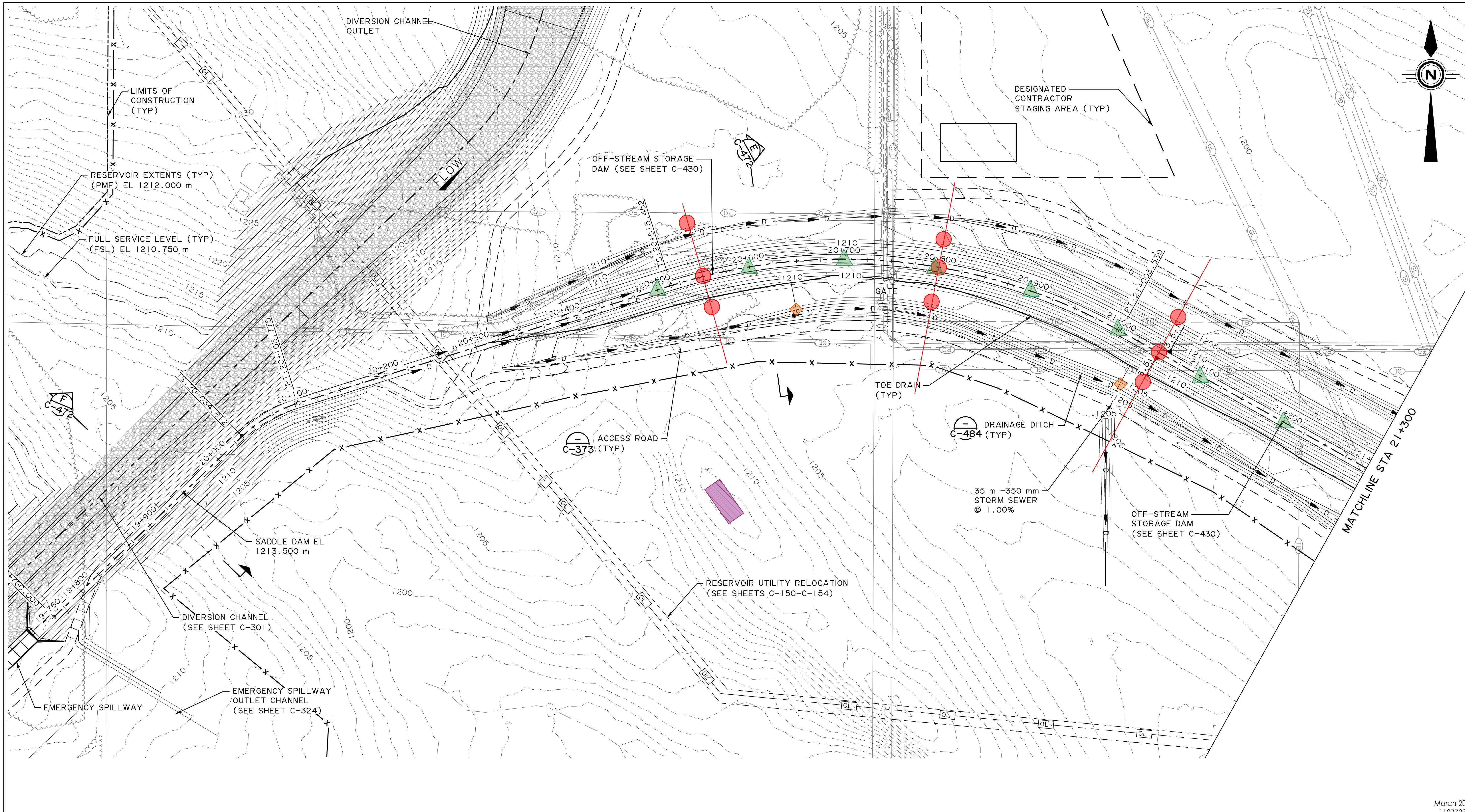
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-17k

Title
Geological Profile - Storage Dam



![Stantec logo]

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Legend

- Nested Vibrating Wire Piezometers
- Slope Inclinometer with Sondex Settlement System
- Settlement Monument
- Laser Scanner Location
- Weir

Notes

See Section 13.10 for a discussion of the instrumentation monitoring plan.

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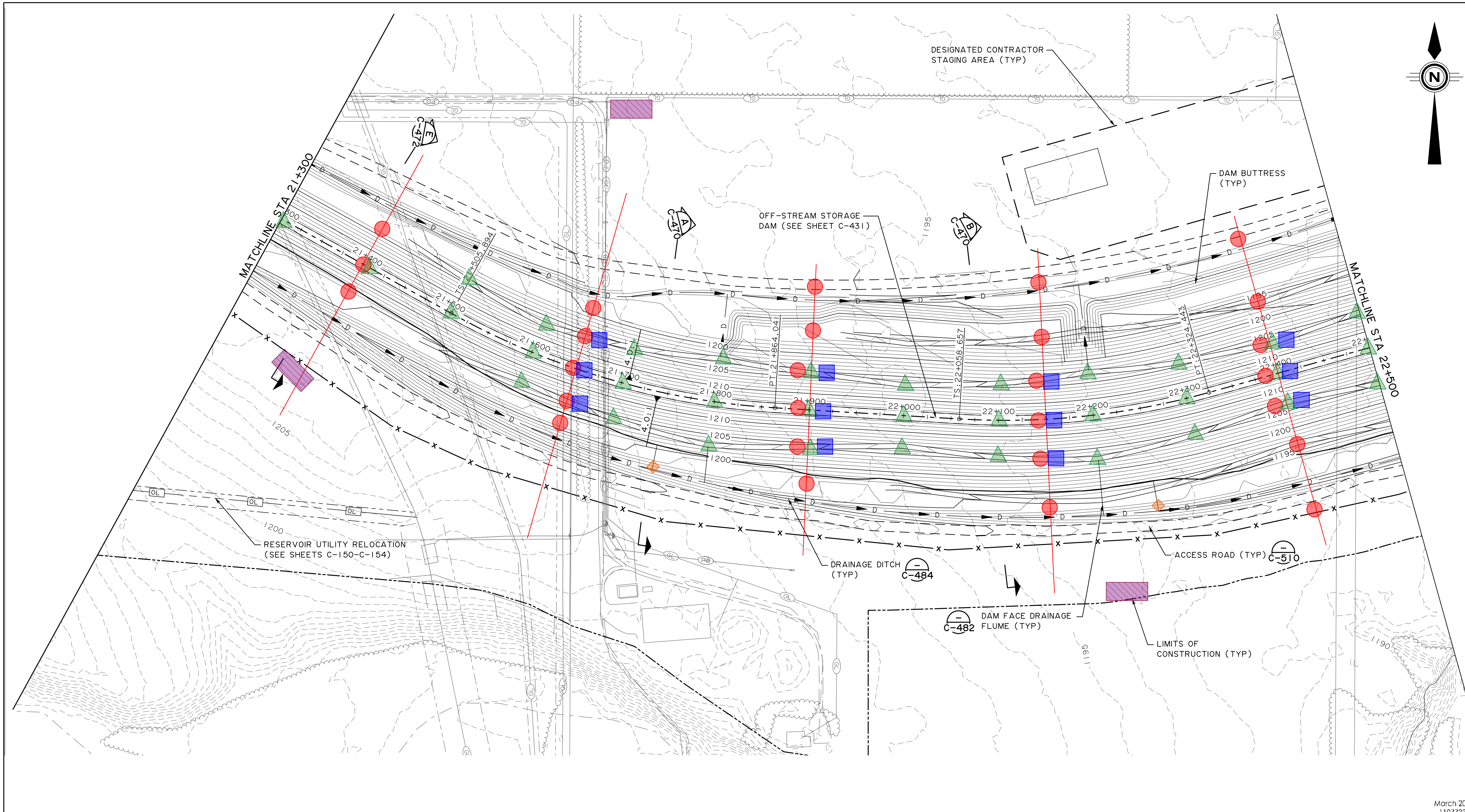
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-18a

Title
Instrumentation Monitoring Plan (IMP)



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Legend

- Nested Vibrating Wire Piezometers
- Slope Inclinometer with Sondex Settlement System
- Settlement Monument
- Laser Scanner Location
- Weir

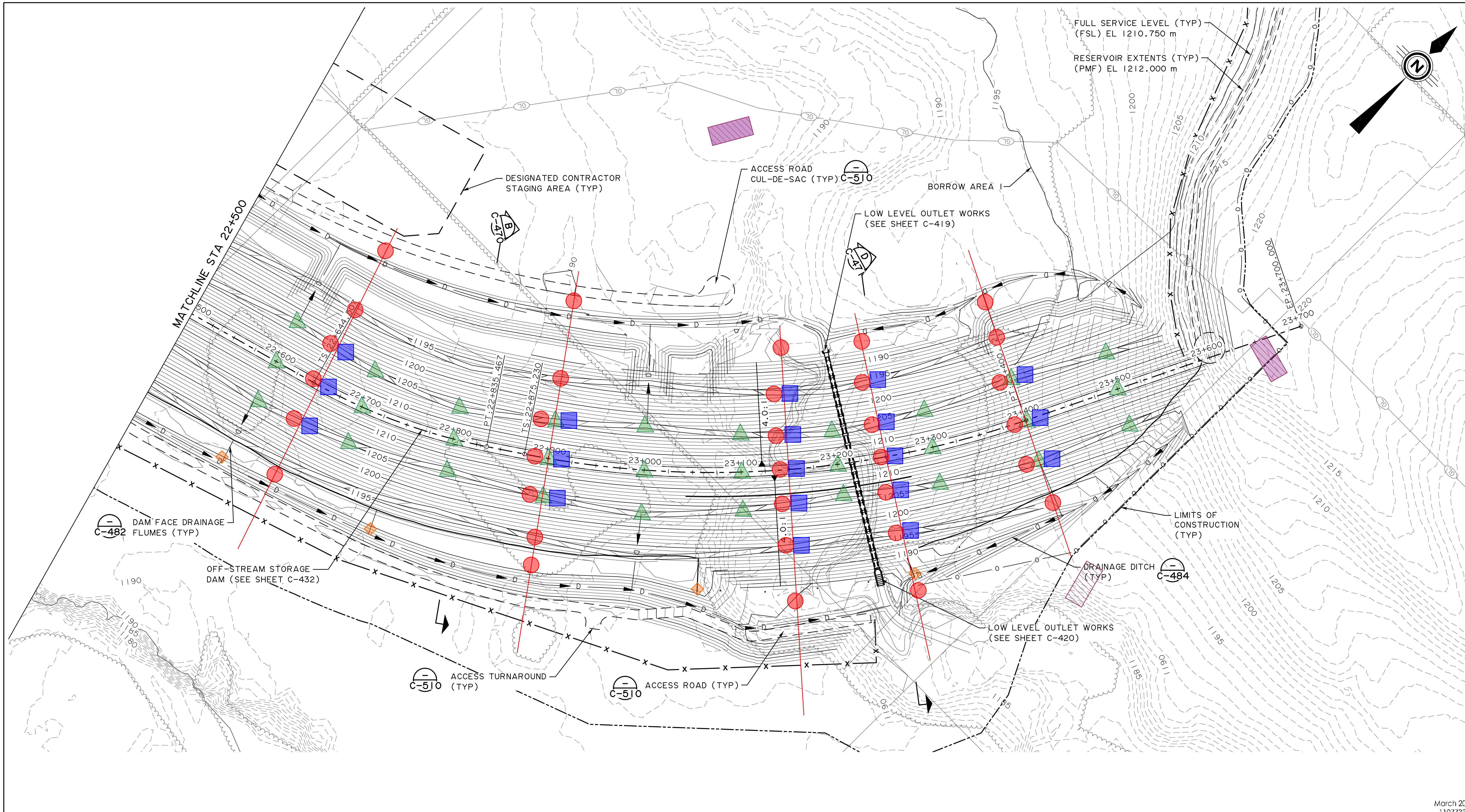
Notes

See Section 13.10 for a discussion of the instrumentation monitoring plan.

Client/Project
Alberta Transportation
Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.
20.-18b

Title
Instrumentation Monitoring Plan (IMP)



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Legend

- Nested Vibrating Wire Piezometers
- Slope Inclinometer with Sondex Settlement System
- Settlement Monument
- Laser Scanner Location
- Weir

Notes

See Section 13.10 for a discussion of the instrumentation monitoring plan.

Client/Project

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Springbank Off-Stream Storage Project (SR1)
Preliminary Geotechnical Design Report

Figure No.

20.-18c

Title

Instrumentation Monitoring Plan (IMP)