Springbank Off-Stream Storage Project Preliminary Design Report

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Project Number 110773396

December 8, 2020

Sign-off Sheet

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Revision	Date	Description
0	September 25, 2020	Original Submittal
0-A	December 8, 2020	Re-Issued Rev. 0 to fix electronic display errors. No new content or revisions incorporated.

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1.0 INTRODUCTION

1.1 BACKGROUND

Following the June 2013 flood, the Government of Alberta (GoA), established the Southern Alberta Flood Recovery Task Force (SAFRTF). The SAFRTF was tasked with reviewing options for flood mitigation throughout Southern Alberta including areas within the Bow, Elbow and Oldman River basins. Results of this review were documented in the *Southern Alberta Flood Recovery Task Force, Flood Mitigation Measures for the Bow, Elbow and Oldman River Basins* (AMEC Environmental and Infrastructure, 2014).

The SAFRTF recommended proceeding with design of the Springbank Off-stream Storage Project (SR1) to reduce the risks of flooding within the Elbow River basin. A second, independent, review of Elbow River flood mitigation options was conducted by Deltares in 2015 with the recommendations documented in a memorandum titled *Review of two flood mitigation projects: Bragg Creek / Springbank off-stream flood storage and McLean Creek flood storage* (Deltares, 2015).

Based on these findings, the GoA recommended proceeding with the design and construction of SR1 in October 2015.

1.2 PREVIOUS STUDIES

1.2.1 Conceptual Design of the Springbank Off-stream Flood Storage Site (AMEC – 2014)

The Initial Design Concept (IDC) for SR1 was presented in *Appendix G – Conceptual Design of the Springbank Off-stream Flood Storage Site* of the SAFRTF report (AMEC, 2014). Elements of the proposed IDC system were:

- Diversion Structures on the Elbow River (Gated Concrete Fishway/Sluiceway; Concrete Overflow Weir; Flood Plain Berm; Gated Diversion Outlet Structure);
- Diversion Channel leading from the Elbow River to the Off-stream Reservoir area; and
- Off-stream Storage Dam with controlled outlet.

The IDC, as originally postulated, was to mitigate flooding downstream of Glenmore Reservoir for flood events up to the 1:100 year with limited consideration given to the 2013 flood event. In addition, the plan assumed that up to 15,400 cubic decameters (dam^3) of flood storage would be available at Glenmore Reservoir to supplement SR1. The IDC also included a permanent pool within the Off-stream Reservoir for water supply augmentation.

1.2.2 Conceptual Design Update (Stantec – 2015)

Following completion of the SAFRTF study, the GoA made the following revisions to the project design criteria:

- Design Event: 2013 Flood or Equivalent Magnitude
- Permanent Pool: None (Dry Reservoir)
- Acceptable Flood Flow at Glenmore Reservoir Outlet: 170 m^3/s (from 350 m^3/s)
- Available Flood Storage at Glenmore Reservoir: 10,000 dam^3

Based on these criteria changes, Stantec reviewed potential design impacts and alternative designs which were presented in the Conceptual Design Update submitted on April 3, 2015 (Stantec, 2015a).

At the Diversion Structure, alternative sites, capacities, and configurations were considered. The recommended alternative:

- Maintained the same location as the IDC;
- Provided a revised Diversion Inlet and Channel design capacity of 600 m^3/s; and
- Replaced the concrete overflow weir with two 15.0 m wide, 4.0 m tall crest gates.

The reservoir capacity of the dam was increased from 57,000 dam^3 to 77,000 dam^3 and the location was moved further downstream to accommodate increased storage with a similar crest elevation.

1.3 TERMS OF REFERENCE

The SR1 Preliminary Design is based on the Terms of Reference (TOR) 0015997 and subsequent addendums (Government of Alberta (GoA), 2014). The TOR states the primary project objective is "to protect against a flood having a magnitude of at least the 2013 flood level."

Additional primary references and design criteria include:

- Engineering Consultant Guidelines for Highway, Bridge and Water Projects, Volume 1 – Design and Tender (AT, 2011a)
- Engineering Consultant Guidelines for Highway, Bridge and Water Projects, Volume 2 – Construction Contract Administration (AT, 2011b)
- Canadian Dam Association (CDA) Dam Safety Guidelines (CDA, 2013)

- Alberta Dam and Canal Safety Guidelines (AEP, 1999)

As necessary and appropriate, additional design criteria and reference sources are documented throughout this report and further listed in Section 15.

1.4 REPORT STRUCTURE

This Report is divided into 15 sections and supplemented with appendices. The report structure is as follows:

- Section 2 provides an overview of the project site and summary of key project components;
- Sections 3 – 7 details the methods of analysis for the key project disciplines of hydrology, hydraulics, geotechnical and structural engineering;
- Sections 8 - 11 summarize the design of each system component including the design objectives, alternatives considered, selection of preferred alternative, design methods and results, and review of construction considerations;
- Sections 12 - 13 review project costs and schedule;
- Section 14 describes the anticipated maintenance requirements; and
- Section 15 lists the relevant source documents and references.

2.0 PROJECT DESCRIPTION

SR1 is a flood diversion system comprised of a diversion structure, a diversion channel and dry storage reservoir (no permanent pool). When in operation, SR1 will divert and temporarily store excess flood water from the Elbow River and release it back into the river system in a controlled manner. SR1 will work in tandem with the Glenmore Reservoir to limit flood flows downstream of Glenmore to less than 170 m^3/s, up to SR1's design event of the 2013 flood, or equivalent.

2.1 SITE DESCRIPTION

SR1 is located in the Springbank area of Rocky View County in the Province of Alberta (Twp. 24. Rge. 04/03, W5M). Springbank is located in the northern unit of the Foothills Parkland natural sub-region of Alberta (Natural Regions Committee, 2006) and straddles the Southern Alberta Uplands and the Rocky Mountain Foothills. The climate is defined as continental with cold winters and short hot summers where July is the warmest month. The average annual temperature in the region is 3°C. The growing season runs from May to September. The highest precipitation is in June; however, high levels of insolation and strong dry westerly winds can limit moisture availability for plant growth. Cold northerly winds dominate the winter months, but southerly winds can bring moisture. The area is subject to the chinook affect, which can bring strong, warm, drying winds in winter and considerable snowpack loss through sublimation when this phenomenon occurs.

The relief within the project area is approximately 70 m with an average elevation of 1200 m. The physiography is defined as sloping lower foothills and hummocky uplands, all of which is heavily dissected by intermittent streams. Till soils dominate the landscape with significant lacustrine materials in valleys defined by outcrops of the Paskapoo, Brazeau and Coalspur bedrock formations. Quaternary soils are predominantly black chernozems, some dark grey chernozems while wetlands are mainly gleysols.

Aspen forests dominate the sub-region but are largely absent within the project footprint while stands of conifers are present in the Elbow River floodplain. Some areas of dense tall willow are present in lowlands and northerly slopes, while grasslands would dominate the natural landscape and are more common on southerly slopes.

2.1.1 The Elbow River

The Elbow River is a tributary of the Bow River in the South Saskatchewan River basin in Southern Alberta, Canada. Originating from its headwaters that border the Fisher Range in Kananaskis, and its highest point source Rae Glacier, on the eastern slopes of the Rocky Mountains, the river flows 120 km before 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 (Hudson, 1983).

As the river travels from the Rocky Mountains to the SRI project site, its bedform is largely dominated by sediment and bedrock profile. Areas of confinement and steep slopes tend to channelize and exhibit riffle-pool-run features, while flatter reaches see much bedload dropping out and form partially braided stretches of the river. Wood supply can drive drastic and sudden changes in this underfit watercourse as it loses confinement in wide flat valleys where terraces can be the only source of natural control to lateral migration, and extents of flooding.

The Elbow River's flood peaks occur in June however a less prominent freshet can occur in the months of April and May as the lowlands of the watershed warm. River ice and ice jamming has historically not been problematic on the Elbow River though limited information is available. This may be indicative of the extremely low flows and very cold temperatures that are present in the winter months. Degradation of river ice is thermally driven and precedes the June freshet.

2.1.2 Land Use in the Project Area

Most land within or near SR1 is privately owned. Public land is limited to the rights-of-way for roads and road allowances, and the bed and banks of the Elbow River and its tributaries. The privately owned land lies within land use districts identified by the Rocky View County Land Use Bylaw (Bylaw C-4841-97), which specifies the types of development allowed in each land use district and provides planning guidance for development in those areas. The land use districts within or near SR1 are:

- Ranch and farm;
- Agricultural holdings;
- Farmstead;
- Residential;
- Public services; and
- Direct control.

The privately owned land within the Project footprint is classified "ranch and farm" except for one farmstead and a small area within the Public Services District. Public service lands are owned by local organizations that use them to operate summer camps. Land ownership of most properties includes only surface rights; however, several landowners also hold mineral rights for their properties. Most mineral rights cover all mines and minerals, but some are specific for coal, petroleum, or natural gas.

2.2 PROJECT COMPONENTS

SR1 is comprised of three primary project components:

1. Diversion Structure;
2. Diversion Channel; and
3. Off-stream Storage Dam and Reservoir.

The Diversion Structure is located on the main channel and floodplain of the Elbow River in 3-10-24-4 W5M, upstream of Highway 22 and approximately 1680 m downstream of the Tsuu T'ina First Nations Reserve boundary. The Off-stream Storage Dam and Reservoir is located between Hwy 1 and Hwy 8; and predominantly east of Hwy 22. The Diversion Channel connects the Diversion Structure on the Elbow River to the Off-stream Storage Reservoir and runs in a northeasterly direction passing under Twp. Road 242 and Hwy 22 before discharging into the reservoir in 1-23-24-04 W5M. The Off-stream Storage Dam outlets to the Elbow River via an unnamed tributary stream that currently runs through the land which the reservoir will occupy. Its confluence with the Elbow River is located in 12-17-24-3 W5M.

Drawing A-110 presents the overall project layout. Oblique aerial photographs showing representations of the primary project components are presented as Figure 1 through 3. Table 1 provides a summary of the relevant design information.

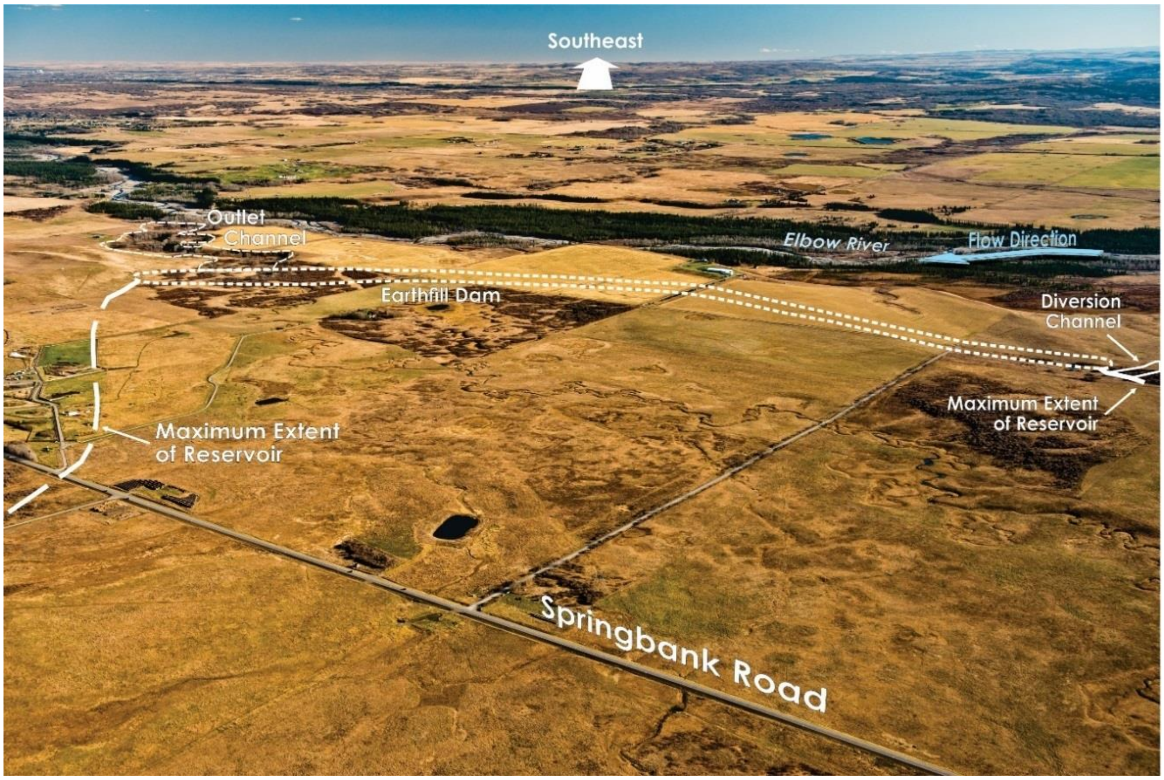


Figure 1. Looking Southeast towards Off-Stream Storage Reservoir and Dam

Figure 2. Looking South towards the Reservoir, Dam and Diversion Channel

Figure 3. Looking Northeast towards Diversion Structure and Diversion Channel

2.2.1 Diversion Structure

The Diversion Structure on the Elbow River includes five sub-components:

1. Diversion Inlet;
2. Service Spillway;
3. Auxiliary Spillway.
4. Floodplain Berm; and
5. Debris Deflection Barrier

Note that while the Diversion Inlet has been included as part of the Diversion Structure due to its integral design and operation with the Service Spillway, it is the headworks for the Diversion Channel.

2.2.1.1 Diversion Inlet

The Diversion Inlet is located at the upstream entrance to the Diversion Channel on the northwest bank of the Elbow River. The Diversion Inlet is a gated concrete structure that will control diversion of river flows into the Diversion Channel during flood events.

The concrete structure includes two 20 m wide by 4.0 m high vertical lift gates with a fixed crest (concrete sill) at Elevation 1211.5 m, approximately 1.5 m above the river bed of the Elbow River. The structure consists of an approach channel, concrete crest surmounted by gates, and a stilling basin which forms the entrance to the Diversion Channel.

2.2.1.2 Service Spillway

The Service Spillway is located adjacent to the Diversion Inlet within the main stem of the Elbow River. The Service Spillway is a gated structure designed to control the headwater elevation and limit downstream flows in the Elbow River during diversion of a flood event. This is accomplished through gate positioning to limit overtopping of the crest gates (flow in Elbow River) and raising the headwater surface above the Diversion Inlet fixed crest elevation so that excess flow passes into the Diversion Channel.

The Service Spillway is a concrete gated structure comprised of two gate bays separated by an intermediate pier. Each gate bay contains a 24 m wide by 5 m high crest gate with a sill elevation of 1210.0 m. Normal position for the crest gates is open, flush with the gate sill. Each gate passage consists of a concrete approach slab, a gate structure, a concrete stilling basin, and an outlet channel to the Elbow River.

2.2.1.3 Auxiliary Spillway

The Auxiliary Spillway is located within the right bank of the Elbow River between the Service Spillway and Floodplain Berm. The Auxiliary Spillway structure consists of a 208 m long, mass concrete "hardfill" overflow weir, approximately 8.8 m high, with a crest at EL 1215.8 m. A reinforced concrete transition wall separates the overflow weir and Floodplain Berm. The overflow weir is covered by a one-metre-high fuse plug set to activate at EL 1216.5 m. An earth embankment overlays the overflow weir to blend in with the Floodplain Berm and natural surroundings, and to allow for wildlife passage.

2.2.1.4 Floodplain Berm

The Floodplain Berm is an earthen embankment approximately 1,030 m long with a maximum height of approximately 5.5 m. The Floodplain Berm is located on the south floodplain of the Elbow River adjacent to the Auxiliary Spillway. The Floodplain Berm constrains flow within the Elbow River active channel and floodplain directing flow through the Diversion Structures. The embankment ties into natural ground on the right descending bank of the river at an elevation that prevents circumvention of the Diversion Structure.

2.2.1.5 Debris Deflection Barrier

A Debris Deflection Barrier is located within the Elbow River upstream of the Diversion Inlet. Its purpose is to reduce risks that large debris pose during a flood event to the operation of the Diversion Inlet gates and to the Diversion Channel bridge piers and other structures. The barrier consists of a 5.75 m high steel framed post and horizontal beam system bearing on a concrete foundation. The structure is 165 m long with a variable height concrete foundation wall. The concrete foundation wall forms the left bank of the Elbow River.

2.2.2 Diversion Channel

The Diversion Channel conveys flows from the Diversion Inlet to the Off-stream Storage Reservoir.

2.2.2.1 Channel

The channel bottom width is 24 m with 3-horizontal to 1-vertical (3H:1V) side slopes in earth cut and fill sections and 2H:1V side slopes in rock cut sections. A 5 m wide overburden bench is provided at the rock / soil interface. The channel slope varies from 0.1 percent to 0.2 percent. At the design capacity (600 m^3/s) and a channel slope of 0.1 percent, the required channel depth is 6.0 m. Total channel length is approximately 4700 m. A minimum of 1.9 m freeboard has been provided along the full channel length. Riprap lines the earth cut and fill sections to mitigate erosion risk.

2.2.2.2 Highway 22 and Township Road 242 Bridges

Bridges are provided at the Diversion Channel intersections with Highway 22 and Township Road 242. The Highway 22 bridge has a span length of 87.5 m and passes approximately 10.3 m (low chord) over the Diversion Channel with a width of 14.4 m. The Township Road 242 bridge span is 100.4 m and it has a width of 10.0 m as it passes approximately 12.5 m over the diversion channel. The recommended design for each includes a three-span prestressed concrete girder structure, with semi-integral abutments supported on concrete piles.

2.2.2.3 Emergency Spillway

The Emergency Spillway is located along the Diversion Channel upstream of the Storage Reservoir. The concrete unregulated overflow spillway discharges into an excavated discharge channel which directs flows over natural ground to its discharge point at the Elbow River. The spillway consists of an approach channel, a concrete lined entrance channel, a 135 m wide concrete weir and stilling basin, and an outlet channel to the tributary.

2.2.2.4 Diversion Channel Outlet

The Diversion Channel Outlet is the last section of the Diversion Channel that includes transitions designed to increase the width of flow, lower the flow depth, drop the grade of the channel, and dissipate energy before it discharges into the Off-Stream Storage Reservoir. The Outlet includes a 960 m long, riprap-lined channel with a slope of 0.2-percent that gradually expands from a bottom width of 24 m to 150 m. The channel then discharges over an approximate 7 m elevation drop into the Reservoir. The drop is facilitated by a stepped RCC grade control structure with 600 mm tall steps arranged on an average slope of 5.5H:1V. The grade control structure has an integral stilling basin at the bottom to further dissipate energy before it outlets to the Unnamed Creek at the base of the Reservoir. The Outlet structure becomes partially to fully inundated during a flood event when the Reservoir is in operation, and the channel discharges directly to the Reservoir's backwater.

2.2.3 Off-stream Storage Dam and Reservoir

The Off-stream Storage Dam system includes three sub-components:

1. Dam Embankment;
2. Low-Level Outlet Works; and
3. Reservoir

2.2.3.1 Dam Embankment

The Dam Embankment is a zoned earthen structure approximately 3300 m long with a maximum embankment height of 29 m and crest elevation of 1213.5 m.

2.2.3.2 Low-Level Outlet Works

The Low-Level Outlet Works is located approximately 200 m southwest of the Unnamed Creek near the northeast end of the Dam Embankment. The Low-Level Outlet Works is a gated concrete structure that will control discharges from the Storage Reservoir to the existing unnamed tributary to the Elbow River. The structure consists of an approach channel; intake structure with trash racks; a single, 1800 mm circular pressure conduit; a gate chamber, access shaft, and control house structure containing a guard and regulating gate in separate wet wells placed in series; a 2400 mm wide, modified "basket handle" shaped gravity conduit, a CSU Rigid Basin, and an exit channel.

2.2.3.3 Reservoir

The Reservoir is the area upstream of the Dam Embankment and downstream of the Diversion Channel. The reservoir has a capacity of 77,000 dam^3 at the full service level (FSL) 1210.75 m and 104,600 dam^3 at the Top of Dam elevation (1213.5 m). Much of the reservoir will remain as undisturbed ground with select areas to be graded for drainage, borrow and energy dissipation at the outlet of the Diversion Channel.

On the western edge of the reservoir, Highway 22 and its intersection with Springbank Road / Township Road 244 will be raised above the FSL with a minimum of 2 m of freeboard to the top of road subgrade. The proposed grade and alignment changes will result in reconstruction of approximately 1000 m of Springbank Road / Township Road 244 and 3000 m of Highway 22.

Design Summary Table

Diversion Structure	
Diversion Inlet: Gated concrete weir	
Inlet: Vertical Lift Gates (2)	20.0 m x 4.0 m each
Structure Height	13.0 m
Crest Elevation	1211.5 m
Total Crest Length	40 m
Discharge Capacity at WS EL 1215.8 m (In Elbow River)	600 m^3/s
Service Spillway: Gated concrete weir	
Service Spillway: Crest Gates (2)	24.0 m x 5.0 m each
Structure Height	12.0 m
Crest Elevation	1210.0 m
Total Crest Length	48 m
Auxiliary Spillway: Uncontrolled overflow concrete weir	
Crest Elevation	1215.8 m
Crest Length	208 m
Discharge Capacity at WS EL 1217.3 m (In Elbow River)	620 m^3/s
Floodplain Berm: Zoned earthfill	
Crest Elevation (Maximum)	1221.5 m
Crest Length	1033 m
Diversion Channel	
Channel: Lining Varies	
Length	4,700 m
Design Carrying Capacity	600 m^3/s
Bottom Width	24 m
Side Slopes	3H:1V Earth, 2H:1V Rock
Water Depth at 600 m^3/s	6.0 m
Emergency Spillway: Uncontrolled concrete weir	
Crest Elevation	1210.75 m
Crest Length	135 m
Discharge Capacity at WS EL 1212.0 m	360 m^3/s
Off-stream Storage Dam	
Dam Embankment: Zoned earthfill	
Structure Height	29 m
Crest Elevation	1213.5 m
Crest Length	3,300 m
Top Width	10 m
Maximum Base Width	275 m
Reservoir: Unimproved	
Storage Capacity at EL 1213.5 m (Top of Dam)	104,600 dam^3
Storage Capacity at EL 1210.75 m (FSL)	77,000 dam^3
Low-Level Outlet Works: Gate controlled, concrete gravity outlet	
Discharge Capacity at WS EL 1210.75 m (FSL)	27 m^3/s

2.3 HAZARD CLASSIFICATION

The Dam Safety Hazard Classification is required for selection of the appropriate design standards established in the CDA Dam Safety Guidelines (CDA, 2013) and the Alberta Dam and Canal Safety Directive (GoA, 2018). The Hazard Classification is selected based on the consequences associated with a hypothetical failure of the dam. Figure 4 lists the CDA standards-based approach for Hazard Classification. The Alberta consequence classifications closely follow the CDA Guidelines using the same class categories and similar descriptions for incremental losses.

Table 2-1: Dam Classification

Dam class	Population at risk [note 1]	Incremental losses		
		Loss of life [note 2]	Environmental and cultural values	Infrastructure and economics
Low	None	0	Minimal short-term loss No long-term loss	Low economic losses; area contains limited infrastructure or services
Significant	Temporary only	Unspecified	No significant loss or deterioration of fish or wildlife habitat Loss of marginal habitat only Restoration or compensation in kind highly possible	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes
High	Permanent	10 or fewer	Significant loss or deterioration of *important* fish or wildlife habitat Restoration or compensation in kind highly possible	High economic losses affecting infrastructure, public transportation, and commercial facilities
Very high	Permanent	100 or fewer	Significant loss or deterioration of *critical* fish or wildlife habitat Restoration or compensation in kind possible but impractical	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities for dangerous substances)
Extreme	Permanent	More than 100	Major loss of *critical* fish or wildlife habitat Restoration or compensation in kind impossible	Extreme losses affecting critical infrastructure or services (e.g., hospital, major industrial complex, major storage facilities for dangerous substances)

Figure 4. Table Excerpt from CDA Dam Safety Guidelines, 2007

A dam breach inundation study was completed and is provided in Appendix C.5. 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.

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 HYDROLOGY

This section presents the hydrologic data and analyses used for the design of the Project. The Flood of Record (FoR) (June 2013) hydrograph was used for determination of the Diversion Structure / Channel capacity and the Off-stream Storage Reservoir flood storage volume. Probabilistic discharge and volume estimates for a range of annual return intervals were developed from the historic gage record. The frequency and magnitude of expected floods inform design load cases and operations and maintenance requirements. The Inflow Design Flood for the Off-stream Storage Dam (IDF-OSSD) is the Probable Maximum Flood (PMF). Site-specific PMF values were developed from a Probable Maximum Precipitation study and a deterministic rainfall-runoff model.

The following sections provide a summary of the study methods and certain results. Detailed information regarding each analysis is provided in Appendix B.

3.1 WATERSHED CHARACTERIZATION

The hydrologic study area is comprised of the 1,212 km^2 Elbow River watershed upstream of Glenmore Dam presented in Figure 5. Except for a portion of the City of Calgary near the downstream end, the watershed is sparsely developed. The downstream half is primarily foothills parkland, agricultural and pasture land. The upstream is comprised of montane, alpine, and subalpine terrain. In its headwaters, the river is largely a steep, single-thread stream until it reaches cobble flats where the channel becomes multi-threaded and meanders as the adjacent watershed transitions from boreal forest to aspen parkland.

Peak flows in the Elbow River occur during the spring mountain snowmelt, or "freshet", which accounts for approximately 60 percent of the total annual discharge. The freshet typically lasts from May to July, with peak flow in June (accounting for approximately 25 percent of the total annual discharge). Summer rainfall may generate several flow peaks after the freshet. Historically, extreme flooding originates from runoff from the mountainous upstream half of the watershed with relatively little increase in flood discharge between the Elbow River at Bragg Creek gauge (05BJ004) and Elbow River at Sarcee Bridge gauge (05BJ010).

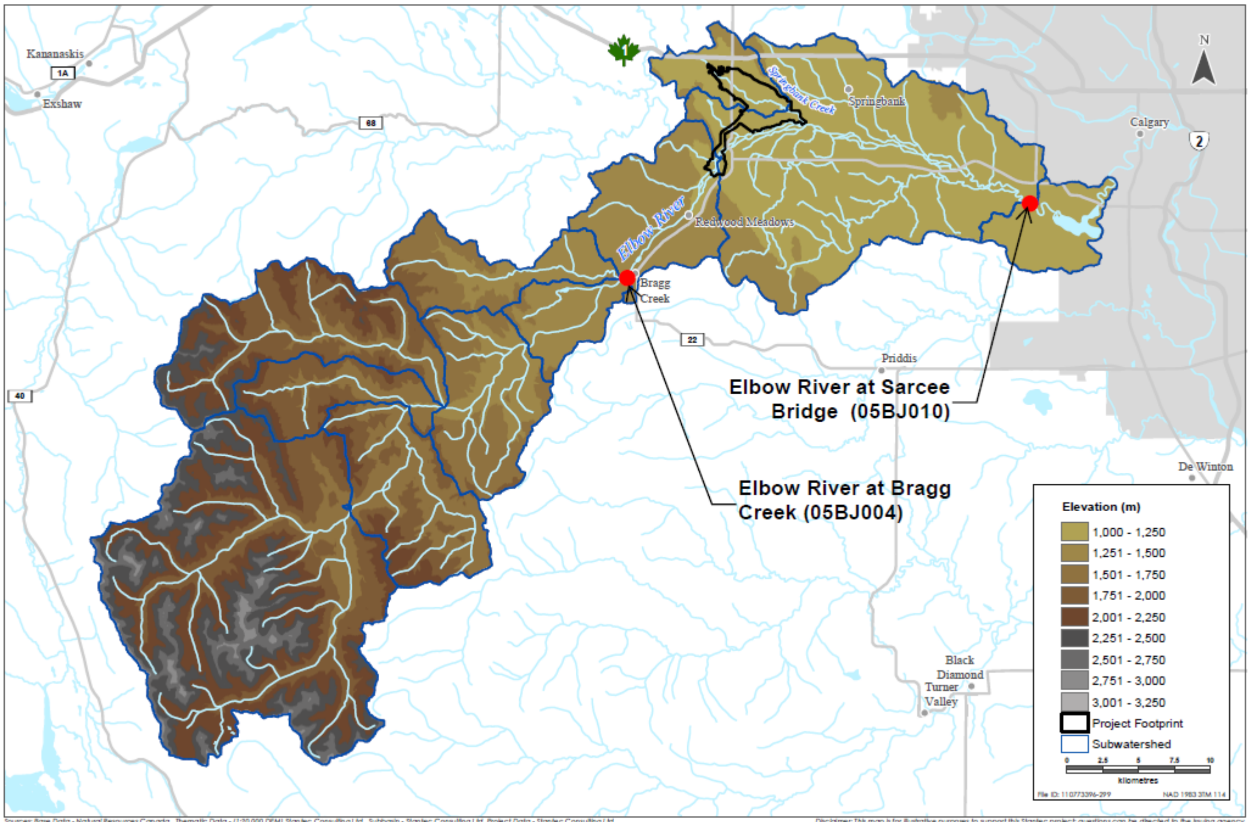


Figure 5. Hydrologic Study Area Watershed

3.2 FLOOD OF RECORD – JUNE 2013

The June 2013 flood event occurred from June 20 to 26, 2013 with the heaviest precipitation occurring from June 19 to 21. Average rainfall across the basin was approximately 200 mm with some areas receiving more than 300 mm. Due to damage during the event, official data from gauging stations at Elbow River at Bragg Creek (05BJ004) and Elbow River at Sarcee Bridge (05BJ010) are unavailable. Water Survey Canada (WSC, 2015) supplied preliminary peak instantaneous flow for the Elbow River at Bragg Creek and Sarcee Bridge as 1150 m^3/s and 1240 m^3/s, respectively. The City of Calgary provided an estimated inflow flood hydrograph into Glenmore Reservoir for the 2013 flood event based on reservoir level and outflow analysis. The estimated inflow hydrograph provided by the City is considered the Flood of Record for this project. Figure 6 presents the June 2013 flood hydrograph. The hydrograph is presented in tabular form in Appendix B.

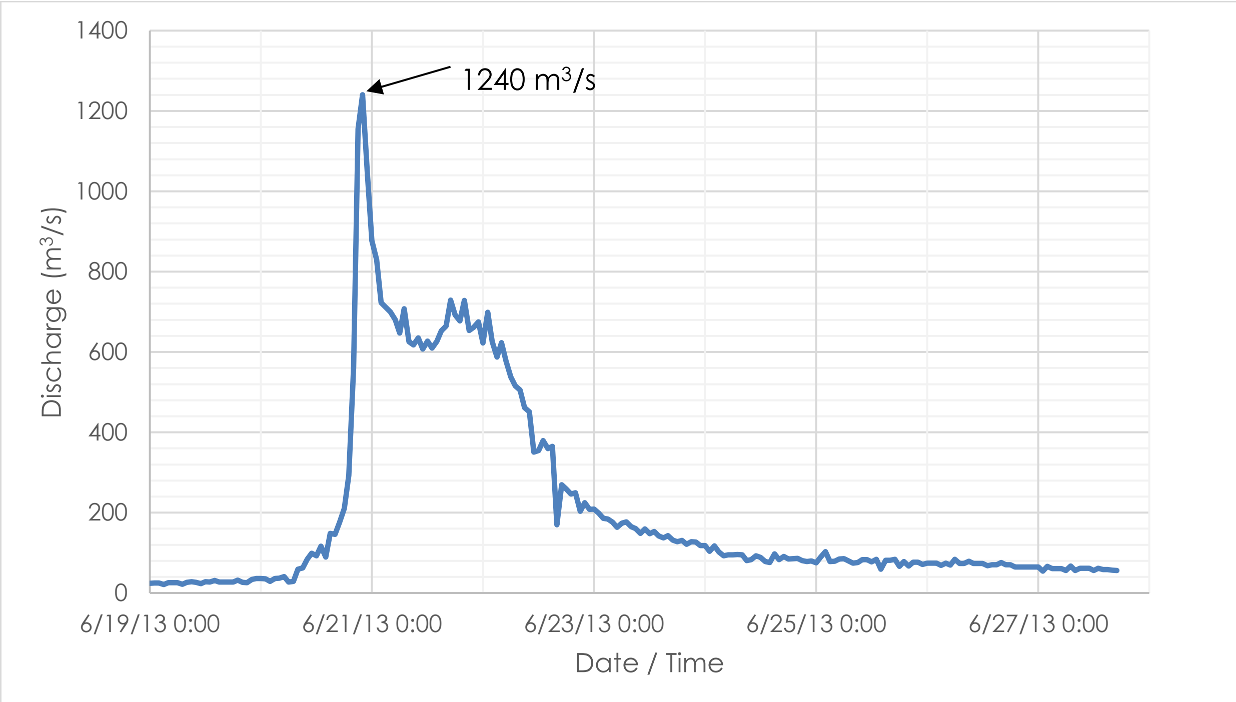


Figure 6. Design Flood Hydrograph (2013 Event Hydrograph from City of Calgary)

3.3 FLOOD FREQUENCY ANALYSIS

Flood peak discharge and volume annual exceedance probability estimates were developed for the Elbow River at the SR1 Diversion Structure. The methods and results of the probabilistic flood frequency analysis are documented in Appendix B.2. Results are presented in Section 3.5.

3.4 INFLOW DESIGN FLOOD

Per the CDA, the IDF is "the most severe inflow flood (peak, volume, shape, duration, timing) for which a dam and its associated facilities are designed" (CDA, 2013). The CDA provides guidance for selection of the IDF for use in deterministic assessments based on the Dam Hazard Classification. Per the Hazard Classification determinations in Section 2.3 and in Table 6-1 of the CDA Dam Safety Guidelines (CDA, 2013), the recommended IDF for the project components are:

- Diversion Structure (including Floodplain Berm, Auxiliary Spillway and Service Spillway):
  - IDF-DS = 1/3 between 1:1000 year and PMF

- Off-stream Storage Dam (including Diversion Inlet, Diversion Channel embankments and Emergency Spillway):
  - IDF-OSSD = PMF

The PMF is determined through deterministic methods using the Probable Maximum Precipitation (PMP) and rainfall runoff models. The PMF is then used to determine the 1:1000 year flood estimate and subsequently the value 1/3 between the 1:1000 year and PMF. These methods are described further in the following sections.

3.4.1 Probable Maximum Precipitation

Site-specific PMP values were developed for this project by Applied Weather Associates, LLC. (AWA, 2015). Detailed documentation of the analysis conducted by AWA is provided in the report entitled "Site-Specific Probable Maximum Precipitation Study for the Elbow River Basin-Springbank Off-Stream Storage Project". This report is provided in Appendix B.3.

AWA employed a storm-based approach consistent with standards established by the US National Weather Service and recommended by the CDA. This approach identifies extreme rainfall events which have occurred over a wide region in locations with similar meteorological and topographical characteristics to what could occur in the Elbow River basin. Twenty-one such storm events were identified and were categorized as either general storms (greater than 6-hour duration and greater than 500 km^2) or local storms (6-hour or less duration and less than 500 km^2). Each storm was analyzed to maximize rainfall, transposed to the study basin and adjusted for differences in climate and topography.

AWA developed 1-, 6-, 12-, 24- and 48-hour gridded PMP values for the drainage area for four storm scenarios:

1. General storm for the 1212 km^2 area upstream of Glenmore Dam;
2. General storm for the 863 km^2 area upstream of the Diversion Structure;
3. Local storms for the 863 km^2 area upstream of the Diversion Structure; and
4. Local storm for the 31 km^2 direct drainage area of the Off-stream Storage Dam.

3.4.2 Probable Maximum Flood

Stantec developed a deterministic rainfall-runoff model of the Elbow River upstream of Glenmore Dam to transform the PMP estimates into PMF hydrographs. This analysis is detailed in the report entitled "Springbank Off-Stream Reservoir Project: Probable Maximum Flood Analysis" (Stantec, 2015b) and is provided in Appendix B.4.

The hydrologic model was developed using the Hydrologic Engineering Center – Hydrologic Modeling System (HEC-HMS), version 4.0 software package. The total 1,212 km^2 drainage area upstream of Glenmore Dam was delineated and sub-divided into 11 sub-basins based primarily on topographic characteristics with consideration of vegetation, surficial geology, and land use. The HEC-HMS model was calibrated to the June 2005 and June 2013 precipitation events.

Gridded precipitation data for the four PMP scenarios discussed in Section 3.4.1 were used as inputs to the calibrated HEC-HMS model. The potential impacts of a 1:100 year antecedent rainfall event and snowmelt was considered. Resulting peak discharge and 7-day volume for each PMP simulation is summarized in Table 1 below:

Table 1. Summary of PMF Simulation Results

Description	Elbow River Discharge at SR1 Diversion (m^3/s)	Elbow River 7-Day Volume (dam^3)	Local Watershed Discharge at SR1 Dam (m^3/s)	Local Watershed 7-Day Volume (dam^3)
General storm PMP over 863 km^2 area upstream of SR1 Diversion	2,770	362,000	-	-
General storm PMP over 1,212 km^2 area upstream of Glenmore Dam	2,690	349,000	-	-
Local storm PMP over 863 km^2 area upstream of SR1 Diversion	2,640	208,000	-	-
Local storm PMP over 31 km^2 area upstream of SR1 Dam	-	-	468	8,930

The PMF discharge was selected as the maximum value resulting from the four simulations. The PMF hydrograph for the Elbow River at the Diversion Structure is presented as Figure 7 below and in tabular form in Appendix B.4.

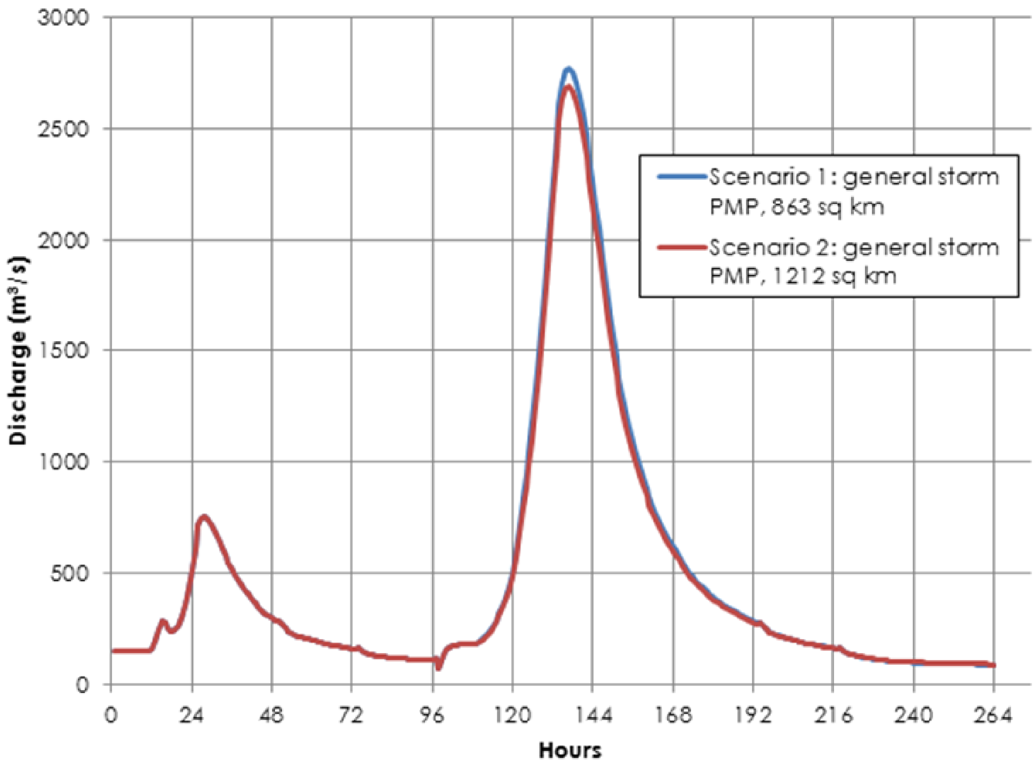


Figure 7. PMF Hydrograph

3.4.3 1000-year Flood Development

The 1:1000 year flood discharge estimate was developed using the methods outlined in the AT *Guidelines on Extreme Flood Analysis* (AT, 2004a). The 1:1000 year flood was determined using log-linear interpolation between the 1:500 year and PMF discharge, with the PMF being assigned an assumed return frequency of a 1:100,000 year event. The calculation for this interpolation is presented below:

$$Q_{1000yr} = Q_{500yr} + \left(\frac{Q_{500yr}}{Q_{PMF}}\right) \times \frac{\left[Log_{10}\left(\frac{1}{1{,}000}\right) - Log_{10}\left(\frac{1}{500}\right)\right]}{\left[Log_{10}\left(\frac{1}{100{,}000}\right) - Log_{10}\left(\frac{1}{500}\right)\right]}$$

Given a 1:500 year discharge of 1800 m³/s from the Flood Frequency Analysis (Stantec, 2017a) and PMF discharge of 2,770 m³/s, the 1:1000 year discharge is computed as 1,930 m³/s.

3.5 SUMMARY OF DESIGN DISCHARGE AND VOLUME ESTIMATES

Table 2 provides a summary of calculated design discharge and volume estimates for use in the design. These values are the result of the probabilistic flood frequency analysis and PMF analysis presented in Appendix B.

Table 2. Design Discharge and Volume Estimates

Return Period (years)	Design Designation	Instantaneous Peak Discharge (m^3/s)	7-Day Volume (dam^3)
2		70	20,000
5		140	38,100
10		200	53,100
20		330	65,600
50		530	86,600
100		765	107,000
200		1,110	132,000
	FoR (June 2013)	1,240	149,600
500		1,800	174,000
1000		1,930	
	IDF-DS (1/3 – 1:1000 and PMF)	2,210	
	IDF–OSSD (PMF)	2,770	362,000

3.6 FLOOD SIMULATION AND RESERVOIR ROUTING MODELS

Hydrologic performance of the project was evaluated using a suite of hydrologic simulation and reservoir routing models. Application of the models is documented in Sections 8 to 10. Details regarding model development and results are included in Appendix B.

4.0 HYDRAULICS

Performance of the proposed Diversion Structure and Diversion Channel was assessed using numerical and physical modeling. This section summarizes the methods of study and proposed application. Hydraulic calculations and modeling used in the design of individual hydraulic structures and other components are covered in Sections 8 to 10, as appropriate, and presented in detail in Appendices C and F.

4.1 NUMERICAL HYDRAULIC MODELING

Two-dimensional (2D) numerical modeling was developed using the RiverFlow2D Plus, version 5.1 two-dimensional finite volume river dynamics model software developed by Hydronia, LLC (Hydronia, 2017). The 2D numerical modeling supports design and assesses performance of the Diversion Structure, Diversion Channel and Diversion Channel Outlet. The models were used to determine the flow velocities (direction and magnitude) and depths for various design scenarios. Design scenarios were developed to consider certain river flows and diversion rates which would cover the range of design flood flows and operations. As described in Section 9.2, a subsequent one-dimensional, unsteady state, hydraulic model was developed to further evaluate hydraulic conditions along the length of the Diversion Channel.

For the purposes of the 2D model, the project was split into two model domains to improve model run times and facilitate efficient alternatives evaluation. The Diversion Structure 2D model domain includes portions of the Elbow River, Diversion Structure, and the Diversion Channel. The Diversion Channel Outlet domain includes portions of the Diversion Channel and Off-stream Storage Reservoir.

4.1.1 Diversion Structure Two-Dimensional Numerical Model

The results of the Diversion Structure 2D model include water surface elevations, flow rates and depth averaged velocities for a range of evaluated scenarios. These results were applied to the design of the Diversion Inlet, Service Spillway, Auxiliary Spillway, and Floodplain Berm.

4.1.1.1 Geometry Data

Four model geometries were developed for this project representing: existing conditions; proposed conditions with no flow over the Auxiliary Spillway and grading in place for fish passage; proposed conditions with no flow over the Auxiliary Spillway and fish passage grading eroded within the Service Spillway stilling basin; and proposed conditions with flow over the Auxiliary Spillway (cover soil eroded). The model domain is comprised of a triangular mesh with elevations assigned from a digital terrain model. Model mesh elements vary in size from less than 1 m to 7 m depending on the complexity of the terrain and detail of proposed project features. The existing conditions mesh is composed of approximately 310,000 elements whereas the preliminary design mesh is composed of approximately 450,000 elements.

The model domain and mesh structure are provided in Appendix C.1.

4.1.1.2 Roughness Parameters

Manning's roughness parameters in the model are spatially varied based on terrain data and aerial imagery. The roughness parameters were selected based on field reconnaissance photos and recommended literature values included in "Open-Channel Hydraulics" (Chow, 1959). Table 3 below summarizes the Manning's values used in the models.

Table 3. Roughness Parameters

Surface / Land Use Type	Manning's "n"
Open Space / Grass	0.040
Wooded Area	0.100
Wooded Island	0.070
Main Channel / Riprap	0.038
Diversion Structure Concrete	0.013
Auxiliary Spillway (Cover Soil Eroded)	0.020
Exposed Bedrock	0.025

4.1.1.3 Boundary Conditions

The Diversion Structure model domain includes approximately 3.5 km of the Elbow River extending from approximately 1.2 km downstream of the Diversion Structure (just above Highway 22) and 2.3 km upstream. In addition, the design models include approximately 4.2 km of the Diversion Channel extending from the Elbow River to just upstream of the Diversion Channel Outlet.

For each flow scenario, the downstream boundary of the Diversion Channel was modeled with a fixed water surface elevation of 1210.75 m which corresponds to the Off-stream Storage Reservoir FSL. The downstream boundary of the Elbow River at Highway 22 was established using a rating curve developed from the 1D regulatory model of the Elbow River (AEP, 1995). For each scenario, a fixed water surface elevation was set based on the river discharge. Due to the distance downstream from the Diversion Structure, the selected downstream boundary conditions were observed to have a limited effect on model results at the Diversion Structure.

The upstream boundary for each model scenario is a specified constant discharge rate. The simulation is then run until a steady-state condition is reached within the model. Table 4 summarizes the boundary conditions for the range of simulated discharge values.

Table 4. Summary of 2D Model Boundary Conditions

Elbow River Discharge (m^3/s)	Elbow River Tailwater (m)	Diversion Channel Tailwater (m)
70	1204.4	1210.75
160	1205.1	1210.75
200	1205.3	1210.75
330	1205.9	1210.75
530	1206.2	1210.75
765	1206.4	1210.75
1000	1206.4	1210.75
1240	1206.5	1210.75
1500	1206.6	1210.75
1850	1206.6	1210.75
1930	1206.6	1210.75
2210	1206.6	1210.75
2490	1206.6	1210.75
2770	1206.7	1210.75

4.1.1.4 Results

Results from the Diversion Structure 2D model are presented in Appendix C.1. Tabular results are presented for each scenario simulated and include descriptions of gate settings, discharge through each gate and spillway as well as headwater and tailwater elevations at the Diversion Structure. Profile figures are also presented in the appendix for certain scenarios through the gate structures, along the Elbow River channel and along the upstream face of the Floodplain Berm and Auxiliary Spillway. Plan view figures depicting velocity magnitude, velocity vectors and flow depth within the vicinity of the Diversion Structure are also presented for certain scenarios.

Model scenarios considered certain operating and maintenance conditions, as described below:

- Base Condition (No Diversion): These scenarios include the Diversion Inlet gates closed and the Service Spillway gates fully lowered. Appendix C.1-2 includes a tabular summary of results and water surface profiles through the gates along the Elbow River channel and along the Auxiliary Spillway and Floodplain Berm. Plan views of flow depth and velocity distribution are also presented.

- Flood Diversion Operation: These scenarios include the Diversion Inlet gates fully open and the Service Spillway gates positioned to create the required head pond elevation and diversion flows. Diversion rates vary up to 600 m^3/s. Appendix C.1-2 includes a tabular summary of results, water surface profiles and plan view results.

- Maintenance Condition: Scenarios were modeled assuming certain maintenance conditions. These conditions included one Diversion Inlet gate out of service (fully closed) with the Service Spillway gates operating to divert flow for the 1:100 and FoR; and one Service Spillway gate out of service (fully closed) and the Diversion Inlet gates fully open. The calculated water surface elevations provided in Appendix C.1-2 were utilized to evaluate hydraulic loads for certain structural load cases as defined in Section 8.0.

- Diversion Inlet and Service Spillway Gates Open: Diversion discharge rates were calculated for various Elbow River flows assuming the Diversion Inlet and Service Spillway gates are fully open. These results are provided in the tabular summary in Appendix C.1-2. These results were used in subsequent analyses to route flows through the Diversion Inlet for extreme floods, including the PMF.

4.1.2 Diversion Channel Outlet Two-Dimensional Numerical Model

The Diversion Channel Outlet 2D model is used to assess performance and design of scour protection for the proposed Diversion Channel Outlet structure. This component was modeled separately from the Diversion Structure 2D model.

4.1.2.1 Geometry Data

The Diversion Channel Outlet model includes approximately 1.4 km of the Diversion Channel and Diversion Channel Outlet upstream of the reservoir. The model also includes portions of the Off-stream Reservoir between the Diversion Channel and the Dam. Model mesh elements vary in size from less than 3 m at the Diversion Channel Outlet to 30 m in the upper areas of the reservoir. The preliminary design mesh is composed of approximately 158,000 elements.

The model domain and mesh structure are provided in Appendix C.1-1.

4.1.2.2 Roughness Parameters

Manning's roughness parameters in the model are spatially varied based on the scour protection specified in the preliminary design. The values for various scour protection measures for this model match the values listed in Table 3.

4.1.2.3 Boundary Conditions

A free flow outlet condition was specified at the downstream boundary with discharge introduced at the upstream boundary. The model simulation was run until conditions at the Diversion Channel Outlet and the immediate area downstream achieved a steady-state condition. The simulation was ended prior to the reservoir filling and producing tailwater impacts at the structure.

4.1.2.4 Results

Results from the Diversion Channel Outlet model are presented in Appendix C. Results include plan view figures of velocity and depth along the Diversion Channel and through the Diversion Channel Outlet and the immediate vicinity downstream. Application of the model results are discussed in Section 8.5.

4.2 PHYSICAL HYDRAULIC MODELING

A physical hydraulic model of the Elbow River, Diversion Structure and upstream reach of the Diversion Channel was developed for this project by the National Research Council of Canada's Ocean Coastal and River Engineering Portfolio (NRC-OCRE, 2016). Details of the analysis are provided in the report entitled "Physical Model Study of the Springbank Off-stream Storage Project Diversion Structure on the Elbow River," which is included in Appendix C of this report. Physical model testing had the following goals:

- Inform decision making on the preliminary design and test design refinements;
- Inform sediment passage performance of preliminary design;
- Inform debris passage performance of preliminary design;
- Check validity of numerical modeling results; and
- Develop operational rating curves for the Service Spillway gates.

4.2.1 Description

The physical model was constructed in NRC-OCRE's Large Area Basin (LAB), a rectangular indoor facility with interior dimension of 50 m by 30 m, capable of accommodating depths up to 1.4 m and discharges up to 1.6 m^3/s.

The physical model was constructed at an undistorted length scale of 1:16. The topographic surface of the model was constructed of concrete with a broom finish to roughen the surface. Model trees were inserted into wet concrete where appropriate to model forested regions of the floodplain. Upstream of the Diversion Structure, the model surface was lowered 0.5 m to allow for simulation of a mobile bed. Figure 8 presents a simplified schematic of the physical model layout.

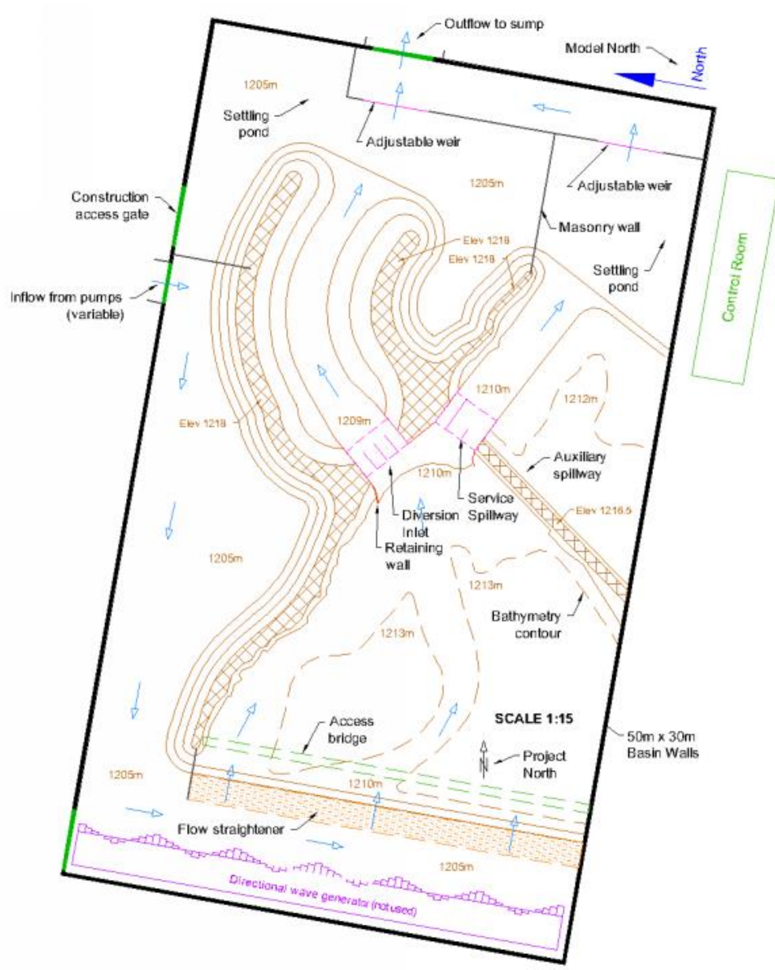


Figure 8. Simplified Physical Model Schematic (Initial Diversion Structure Configuration)

The physical model was constructed with the ability to modify gate and pier configurations. Two design iterations of the Diversion Structure were evaluated in the model. The initial Diversion Structure configuration consisted of a Diversion Inlet with four 10 m wide gates and a Sluiceway with a 10 m wide gate and Service Spillway with two 15 m wide crest gates. Review of the results from the initial configuration led to the development of a revised Diversion Structure configuration which consisted of consolidating the Diversion Inlet gates to two 20 m wide gates, eliminating the Sluiceway and widening the two Service Spillway gates to 24 m and 18 m.

Gate settings and tailwater elevations in the physical model were selected from the Diversion Structure 2D numerical model geometry and results.

Sediment transport tests were conducted using the initial Diversion Structure configuration only. Service Spillway rating curves were developed for the 20 m wide Diversion Inlet gates and the 24 m and 18 m wide Service Spillway gates. Debris passage tests were conducted for both the 4 – 10 m gate and 2 – 20 m gate configurations. Testing of two debris exclusion barrier layouts was conducted with the revised Diversion Structure configuration.

4.2.2 Results and Findings

Physical model results are included in the report provided in Appendix C. Table 5 below summarizes the test series which were evaluated with the physical model.

Table 5. Summary of Test Series for Diversion Structure Physical Model

Series	Description	Structure Configuration	Elbow River Discharges
Test Series A	Clearwater tests	Initial	60 m^3/s – 1240 m^3/s
Test Series B	Debris passage tests	Initial	760 m^3/s – 1240 m^3/s
Test Series C	Sediment passage tests	Initial	320 m^3/s – 1240 m^3/s
Test Series D	Debris passage tests	Revised	320 m^3/s – 1240 m^3/s
Test Series E	Clearwater tests	Revised	60 m^3/s – 1240 m^3/s
Test Series F	Service Spillway rating curve development	Revised	44 m^3/s – 894 m^3/s
Test Series G	Debris barrier tests (barrier layout 1)	Revised	320 m^3/s – 1240 m^3/s
Test Series H	Debris barrier tests (barrier layout 2)	Revised	320 m^3/s – 1240 m^3/s

Application of model results to the project design are provided in subsequent sections of this report.

4.3 HYDRAULIC MODELING APPLICATION

4.3.1 Comparison of Numerical and Physical Model Results

In order to provide for direct comparison of model results, an interim version of the Diversion Structure 2D model was developed to match the final revised physical model layout and gate settings (Test Series E).

Water surface elevations in the numerical model were compared against the physical model at 13 water level sensor locations. Velocities in the numerical model were compared against the physical model at six velocity sensor locations. Sensor locations for Test Series E are presented in Appendix C.4 to this report.

For most scenarios, the distribution of flow entering the Diversion Channel versus the downstream river channel when comparing the numerical and physical models match within approximately five percent. Instances with greater discrepancies in flow distribution were observed to occur for physical model test runs where the Diversion Inlet gates were closed. This is likely the result of flow seeping under and through the structures in the physical model, which was observed.

At most locations, the numerical model produces water levels higher than those recorded in the physical model. The water surface elevations downstream of the Diversion Inlet and Service Spillway varied within 150 mm between the two models (WL sensors 17, 10, 18, and 11). Upstream of the gates, the results were typically within 300 mm (WL sensors 4, 6, 7, 8 and 9). Comparison of water levels to sensors further upstream showed greater differences between the models with variations of up to 820 mm for some locations and scenarios (WL sensors 1, 2, 16, and 5). These upstream locations are closer to the physical model boundary and likely reflect the influence of boundary applied flow distribution at lower flows. During higher flows, most water levels differences at these upstream sensors fell within 150 mm.

Generally, the numerical model produced lower velocities than those observed in the physical model. The differences in simulated versus observed velocity are mostly within 0.5 m/s with better correlation at higher discharges. Observations during physical model testing suggested a strong agreement between the numerical and physical model with respect to the direction of velocity and flow distribution.

A tabular comparison of the 2D numerical model and physical model results is presented in Appendix C.4 of this report.

4.3.2 Recommended Uses

Comparison of physical and numerical model results indicate a strong correlation in the flow distribution and water surface elevations for the simulated conditions and validated the numerical model for application in design including required flow parameters and loads. The physical model results were useful in evaluating performance of the design under sediment and debris loads and observance of local hydraulic phenomena. Improvements to the hydraulic structure design and layout were facilitated.

5.0 BEDLOAD SEDIMENT TRANSPORT

This section summarizes bedload sediment transport analyses and modeling for the Elbow River and discusses the potential impacts of bedload sediment on project performance. Suspended sediment loading, including the washload or colloidal fraction, was not considered during these analyses; suspended sediment was assumed to stay suspended in the water column during operational flows and therefore transported downstream, either down the Elbow River or to the Off-Stream Storage Reservoir. Localized colloidal deposition was assumed to be insignificant during operational flow periods.

5.1.1 Field Observations and Bed Load Estimates

Stantec conducted an analysis of the bed load materials present in the project area and developed bed load rating curves for the Elbow River for use in physical and numerical sediment transport modeling. The results of that work are presented in Appendix C.3.

Four bar samples were collected from near the site of the Diversion Structure using methods outlined in Watershed Assessment of River Stability and Sediment Supply (WARSSS) (Rosgen, 2006). A composite of the four samples is presented in Table 6. The estimated median particle size (D_{50}) is 26 mm.

Table 6. Composite Grain Size Distribution of Four Bar Samples

Grain Size (mm)	% Finer
2	7.5
4	10.5
8	16.5
16	37.4
32	56.9
63	85.8
120	100

Stantec reviewed eight sediment transport models or equations for developing bed load sediment rating curves. Three were selected as most applicable to the project site based on site geology and hydrology and published literature. These selected methods for evaluation included the Meyer-Peter and Müller (1948) sediment transport equation, the Bagnold (1980) sediment transport equation, and the Wilcox and Crowe (2003) sediment transport equation. Stantec developed bed load sediment rating curves using each of these methods and a range of channel roughness values. The resulting curves along with data collected by Holliingshead (1971) on the Elbow River at Bragg Creek are presented in Figure 9 below.

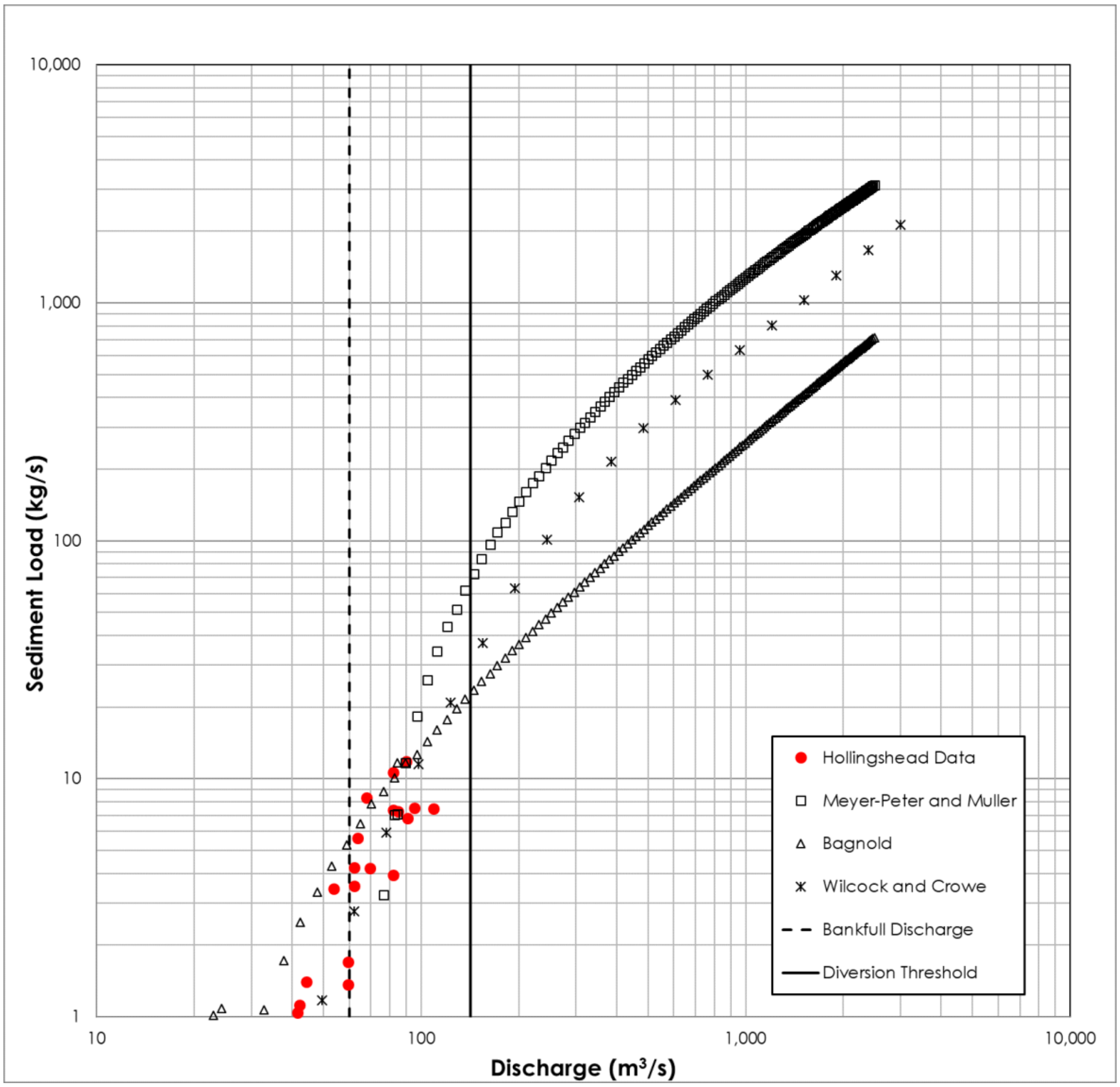


Figure 9. Elbow River Predicted Sediment Rating Curve at Diversion Structure

Given the uncertainty in bedload transport analyses at flood flows, a conservative approach is warranted for design evaluation. The Meyer-Peter and Müller equation produced the highest bedload transport rates and therefore is used in the physical modeling and numerical modeling. Based on Figure 9 the estimated loading rates utilized are 300 kg/s at 320 m^3/s, 950 kg/s at 760 m^3/s, and 1600 kg/s at 1240 m^3/s.

5.1.2 Physical Model Testing – Bed Load Transport

The Diversion Structure physical hydraulic model was used to assess performance of the structure considering sediment transport. Due to the large predicted bedload volume and equipment limitations, a long-term simulation of sediment performance in the physical hydraulic model was not possible. The short-term results however were used as a comparison to the results of the numerical bed load transport model. Bed load transport simulations are documented as Test Series C in the physical model report provided in Appendix C.4.

5.1.2.1 Sediment Material and Loading Method

Grain size distribution and bed load feed rates for the model were developed based off the information presented in Section 5.1.1. Sediment particle sizes were then scaled based on the model scale and Shields' equation for particle mobility. Due to the limitations of the model size and equipment, a truncated grain size distribution was selected for use representing the upper 50 percent of the curve.

Three motorized spreaders were used to load sediment at the upstream end of the model. The channels and forebay area upstream of the diversion structure were pre-loaded with sediment.

5.1.2.2 Bed Load Scenarios

Sediment transport simulations were completed for discharges at 320 m^3/s, 760 m^3/s and 1240 m^3/s. Sediment transport simulations were run at a constant flow rate until sediment spreaders were emptied. Sediment transport simulations proceeded from the lowest discharge to the highest discharge. Between each simulation, the facility was drained and 3D laser scanning equipment was used to measure changes in the mobile bed before and after each sediment run. The next discharge was then tested in series leaving the previous sediment in place. Model simulations were performed for 214 model scale minutes (14 full scale hours).

5.1.2.3 Results

The majority of changes to the bed were observed during the 320 m^3/s simulation, which saw deposition of sediment in the Diversion Structure forebay as a bar curving toward the Diversion Inlet. During the 760 m^3/s simulation, only minor changes to the bed geometry was observed because most of the sediment remained near the sediment loading locations indicating that the elevated head pond level created by raising the Service Spillway crest gates at this flow reduces transport capacity immediately upstream of the Diversion Structure and deposition likely would occur upstream of the model limits. The 1240 m^3/s simulation showed slight increases to the depositional bar feature in the forebay area. However, the development of scour holes were observed adjacent to the left and right abutments of the Diversion Structure. During the simulations, most of the bed sediment remained upstream of the Diversion Structure. Small quantities of bed load sediment which passed through the Diversion Inlet deposited immediately downstream of the stilling basin. The report included in Appendix C.4 includes detailed figures showing changes in bed and photos showing areas of scour and deposition. Figure 10 shows cumulative change in bed geometry after all simulations.

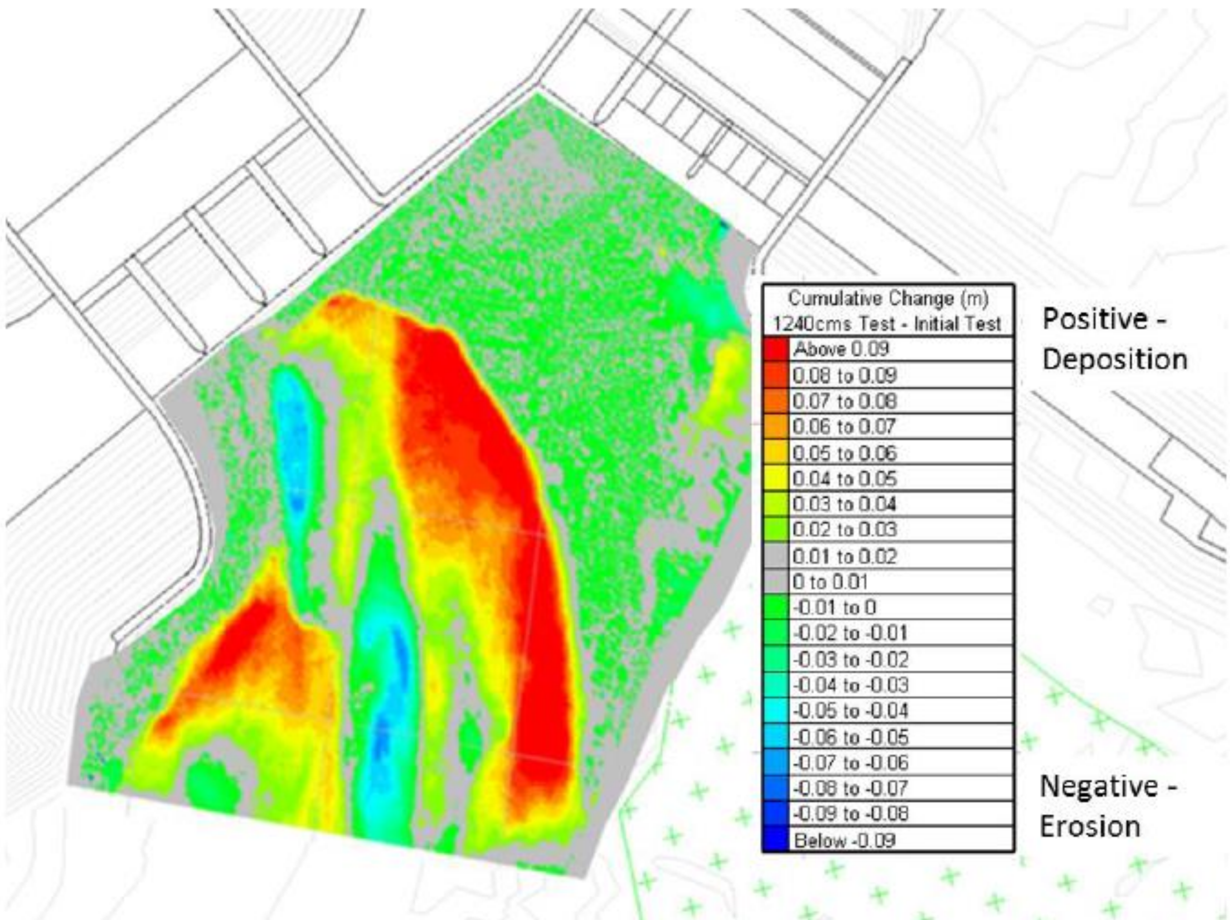


Figure 10. Physical Model Mobile Bed Cumulative Change (Model Scale)

5.1.3 Numerical Modeling – Bed Load Transport

Due to the limitations of the Physical Model, the Diversion Structure 2D sediment transport numerical model was used to assess performance of the preliminary design during extended periods of sediment loading which are anticipated during the design event.

5.1.3.1 Geometry Data

The proposed design numerical model discussed in Section 4.1.1 was simplified to improve model performance and computation time for the purpose of performing bedload transport calculations. The size of mesh elements around the Diversion Structure were increased from approximately 1 m to 4 m. In the river channel, they were increased from 3 m to 12 m and in the floodplains they were increased from 7 m to 25 m. These modifications resulted in a reduction in the number of mesh elements from 430,000 to approximately 45,000. The simplified geometry produced the same headwater elevation and a diversion discharge within three percent of the detailed model for the river flow considered for this evaluation of 765 m^3/s. This river flow represents the 1:100 year peak discharge and represents a condition where most of the river flow is diverted.

Gate settings for both the Diversion Inlet and Service Spillway were selected to replicate a diversion of 600 m^3/s.

Appendix C.3 to this report presents the layout of the Diversion Structure 2D sediment transport numerical model.

5.1.3.2 Roughness Parameters

Manning's roughness parameters in the model are spatially varied based on values presented in Table 3.

5.1.3.3 Boundary Conditions

The model boundary conditions were set as described in Section 4.1.1.3.

5.1.3.4 Sediment Transport Simulation

The numerical model simulations utilize a constant discharge rate and sediment loading. Discharge and sediment are introduced at the upstream boundary of the model and routed through the domain. Figure 11 presents the 2013 flood event hydrograph. Overlain on the hydrograph is the sediment simulation discharge (765 m^3/s) and a duration of 48 hours. This box hydrograph represents a comparable flood volume to the 2013 flood event and provides a reasonable surrogate for the event sediment loading (magnitude and duration).

Bed load sediment transport methods and inputs within the 2D hydraulic model are consistent with recommendations from Section 5.1.1. The Meyer-Peter and Muller equation was selected as the transport function. A sediment loading rate of 949 kg/s (0.36 m^3/s) was selected corresponding to the 765 m^3/s discharge and Figure 11. Sediment properties included an assumed sediment density of 2,650 kg/m^3, mean sediment diameter of 26 mm, porosity of 0.4, and a Shields stress of 0.047.

The model simulation began with sediment and discharge introduced at the upstream boundary. After 48 full scale hours, the simulation was evaluated with regards to deposition, headwater elevation and diversion rate. As observed in the Physical Model, the headpond created by gate operation significantly reduced the Elbow River transport capacity with most of the bedload sediment depositing at the upstream extents of the headpond. The model simulation was then continued for 120 full scale hours in order to further understand the effects of deposition in the headpond from long-term processes or successive flood events.

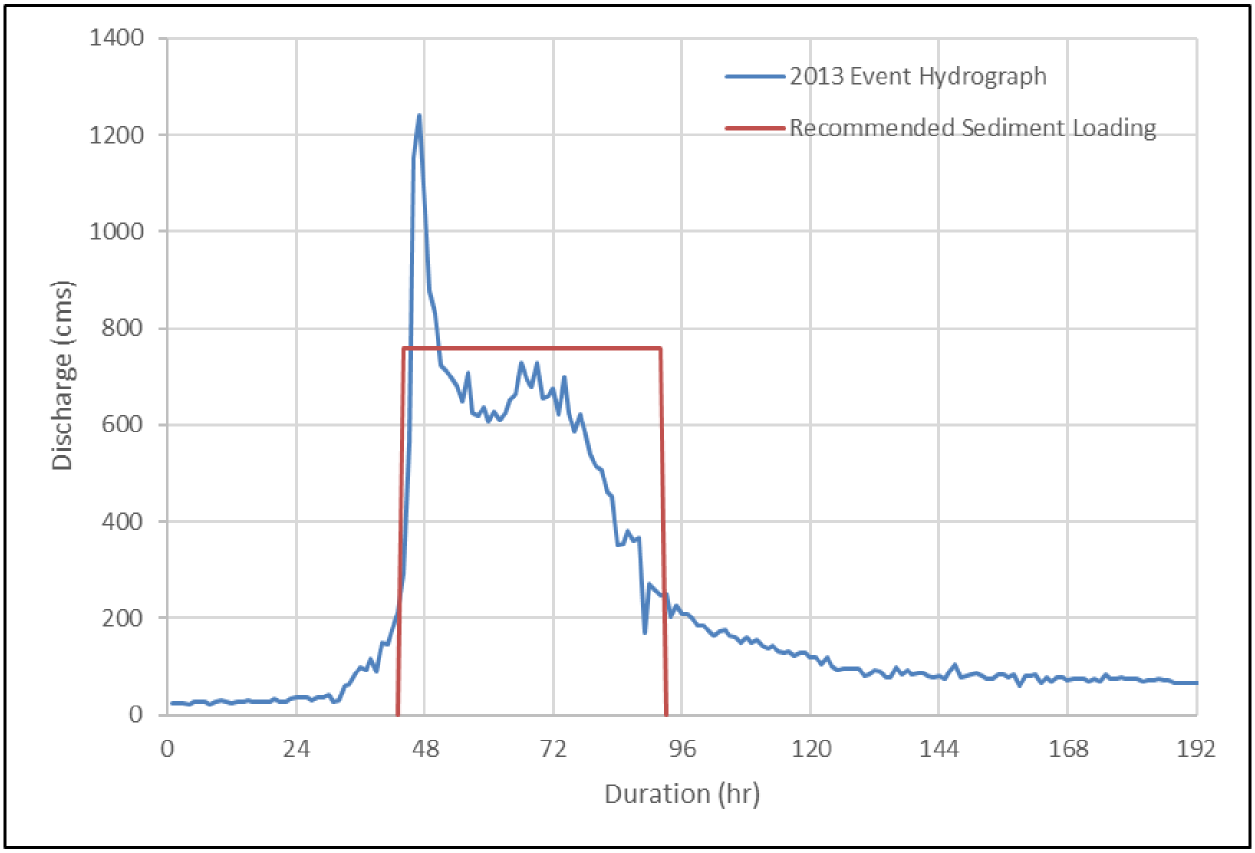


Figure 11. Sediment Loading Scenario

5.1.3.5 Results

Model results were reviewed to assess the potential impacts of bedload sediment erosion and deposition on the hydraulic performance of the Diversion Structure under a simplified flood hydrograph and extended operations. Table 7 summarizes the results of the numerical model at various time steps.

Table 7. Summary of 2D Sediment Transport Results

Scenario	Headwater Elevation (m)	Diversion Discharge (m^3/s)
Base (Clearwater Model) – See Section 4.1.1	1215.8	601
Base (Clearwater Model) - Simplified Geometry – Hour 0	1215.8	587
765 m^3/s, Sediment Transport Simulation – Hour 48	1215.8	581
765 m^3/s, Sediment Transport Simulation – Hour 120*	1215.8	575
765 m^3/s, Sediment Transport Simulation – Hour 168*	1216.0	556

*These simulations represent the impact of sediment loading durations 2.5 and 3.5 times the idealized block hydrograph presented in Figure 11.

Erosion and deposition patterns are presented at 12 hour increments in Appendix C.3. The following observations are noted:

- The simulated hydrograph indicates limited effects of bedload on the diversion capacity for a single event.
- The sediment introduced in the model initially began aggrading upstream of the structure in the low velocity zones created by diversion operations.
- Depositional patterns grew during the simulation in a downstream direction, filling in the area upstream of the Diversion Structure until it reached the Diversion Inlet. The depositional area grew further to just downstream of the Diversion Inlet.
- The sediment depositional front reached the Diversion Structure within 72 hours (Day 3)
- After 120 hours, sediment deposition in the Diversion Channel downstream of the Diversion Inlet had a maximum depth of approximately 4.0 m tapering to approximately 0.1 m of deposition after 800 m downstream.
- Sediment deposition both upstream and downstream of the Diversion Structure results in increased headwater elevations along the Auxiliary Spillway. At 120 hours, the freeboard at the Auxiliary Spillway is 0.5 m. At 168 hours, the freeboard is less than 0.4 m.
- The effects of sediment deposition on the Diversion Inlet and Diversion Channel capacity for the full simulation period are presented in Appendix C. As presented in Table 7, the diversion discharge rate declines over the simulation period by two percent after 120 hours and four percent after 168 hours. At completion of the model simulation, diversion rates remained 75 m^3/s greater than the minimum required operation discharge of 480 m^3/s.

5.1.3.6 Conclusions

Sediment transport is a complex process dependent on varying discharge, localized hydraulics, operations schemes and available supply. The presented analysis is meant to understand the expected trends related to sediment transport and its potential effects on flood operations and the project design. The simulation results indicate the following:

- Sediment deposition will occur in the headpond upstream of the Service Spillway.
- Deposition will begin at the upstream point where the water surface elevation affected by gate operation meets existing grade.
- Deposition patterns will advance downstream as the river channel aggrades and the upstream water surface gradients increase.
- Bedload transport into the Diversion Channel is not anticipated until the area upstream of the Diversion Inlet increases to the fixed weir at Elevation 1211.5 m.
- Simulation results indicate that this deposition is unlikely to occur over a single flood event.
- If it does occur, model results indicate sufficient excess capacity is available within the Diversion Channel and Diversion Inlet to achieve design diversion rates.
- Sediment deposition within the headpond could result in a modest increase in upstream water surface elevations. Freeboard provided for the Auxiliary Spillway and Floodplain Berm crest is sufficient to manage the risk of overtopping under the simulated conditions.
- Maintenance and removal of sediment upstream of the Service Spillway after an event may improve performance and reduce potential risks associated with deposition.

6.0 GEOTECHNICAL

This section provides a summary of geologic and geotechnical site conditions, field exploration activities, and characterization of soil and rock materials. Geotechnical analyses and recommendations used in the design of individual project components are addressed in Sections 8 to 10, as appropriate. A complete geotechnical report is provided in Appendix D.

6.1 REGIONAL AND SITE GEOLOGY

The Project site is located within the Western Canada Sedimentary Basin. This is a 1.4M km^2 sedimentary basin that underlies Manitoba, southern Saskatchewan, Alberta, northeastern British Columbia and the southwest corner of the Northwest Territories.

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. 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. 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.

6.1.1 Brazeau Formation

The Brazeau Formation (BZF) subcrops beneath the western portion of the SR1 Project Site. 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.

6.1.2 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.

The boundary with the underlying BZF is the Entrance Conglomerate. Jerzykiewicz (1997) identified this boundary on the Highway 22 road cutting 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.

The boundary between the CSF and the overlying Paskapoo Formation (PPF) was not identified and is inferred from AGS Mapping (AGS, 2015; 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).

6.1.3 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 LLOW and the Reservoir.

The PPF is comprised of an inter-bedded non-marine sandstone, siltstone and mudstone with minor amounts of bentonite and coal (Lyster and Andriashek 2012). 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 site 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).

6.2 FIELD EXPLORATIONS

The geotechnical field exploration program was executed through a series of mobilizations due to site access limitations, and additional data requests associated with design progression.

6.2.1 Initial Field Exploration

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;
- Twenty (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 Works locations.

The Draft Geotechnical Investigation Report (Stantec, 2016a) documented the data collected (borehole records, cone penetration testing (CPT) report, laboratory testing results, and geophysical survey reports). This report is provided in Appendix D.

6.2.2 2018 Supplemental Field Exploration

The 2018 fieldwork was completed 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 Debris Deflection Barrier and 11 boreholes and 6 Seismic Cone Penetration Test soundings within the dam footprint for multiple Low-Level Outlet Works alignment options.

The second mobilization occurred 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 LLOW alignment, and 14 test pits and trenches throughout the dam footprint.

The additional factual data collected from the 2018 exploration was incorporated into the Supplemental 2018 Geotechnical Investigation Report (Stantec, 2018). This report is provided in Appendix D.

6.2.3 Additional Supplemental Explorations

Some areas of the project site were not available for equipment access during the field explorations 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. While general type of soils to be encountered at the site are known, critical variations in the thicknesses and properties were determined to be significant. Recommendation details and proposed boring layouts are provided in the geotechnical report presented in Appendix D. Below is a summary of the supplemental exploration program.

Eleven borings are planned near the upstream toe of the Dam between Stations 21+000 and 22+500. The purpose of the borings is to confirm the depth to rock and the thickness of the glacio-lacustrine layer, and to investigate for the presence of materials different from current assumptions for the foundation soils in this area.

The planned location of the Emergency Spillway has been moved to the southwest with two locations still being evaluated. There is no site-specific subsurface data for these locations. Twelve borings are planned to determine depth to rock and to characterize the soil/rock materials.

The proposed Diversion Channel Outlet was extended and shifted during preliminary design. Therefore, there is not sufficient geotechnical data to support design of the structure. Two borings are proposed to determine foundation material characteristics and depth to rock.

A horizontal directional drill bore crossing is proposed for relocation of utilities near the Dam. Installation and alignment were not known at the time of the 2016 geotechnical exploration. Two borings are proposed, one on each side of the channel, to determine the top of rock elevation and characterize the rock strength and permeability properties.

Finally, eight additional borings are proposed to better characterize the nature and extent of available borrow soils.

6.3 SITE CHARACTERIZATION

6.3.1 Laboratory Testing

Laboratory tests 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. The test results are presented in the Geotechnical Investigation Report (Stantec, 2019).

6.3.2 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 works. The methodology, survey locations and results of the surveys can be found in the Geotechnical Investigation Report (Stantec, 2019).

6.3.3 Hydraulic Conductivity Investigation

In order to characterize the hydraulic conductivity of the soil and rock materials the following tasks were undertaken:

- Standpipe piezometers were installed in 35 locations. Thirty of these were single well installations with five (5) comprising an upper and lower nested well;
- Thirty-seven (37) single packer permeability tests were conducted in five (5) boreholes to determine the permeability of the bedrock. The equipment used for these tests 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;
- Ten (10) rising head tests were undertaken by Stantec to estimate the hydraulic conductivity adjacent to the well completion elevations; and,
- Thirty (30) Pore Pressure Dissipation Tests (PPDT) were undertaken during 15 of the CPT soundings.

6.4 GEOTECHNICAL CHARACTERIZATION

6.4.1 Alluvium

The site characterization activities indicated that two assemblages of alluvium were present within the 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 Elbow River sub-unit 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 Service Spillway. 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-shaped cobbles of Front Range and Foothills-derived lithology are extensive. The fines content was less than ten 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 approximately 350 m southeast of the active river channel.

The Unnamed Creek sub-unit comprises an assemblage of fluvial deposits associated with the Unnamed Creek. The creek is located in a sinuous-shaped, over-sized valley. The gradient of the creek 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 approximately 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. 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 clayey 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.

6.4.2 Glacigenic Units

The 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.

The site characterization activities have identified five (5) glacigenic units within the 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.

- 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 GL was encountered beneath the Dam footprint and the Diversion Channel within the exploratory holes and the geological mapping of outcrops. 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 GL thickness ranged between 0.5 and 16 m.

Index testing indicate that this unit was a medium to high plasticity clay with silt. The LL ranged between 41 and 78 percent with approximately 2/3 of the test results having a LL greater than 50. The PI ranged between 23 and 62 with most of the test results between 30 and 40. The LI was typically between 0 and 0.1. Clay was the dominant fraction typically comprising between 50 and 70 percent.

The UBT unit was encountered beneath the GL within the Dam footprint and the eastern portion of the Diversion Channel. 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. 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. 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 was typically up to 10 percent gravel, although higher contents up to 59 percent were locally encountered.

The BGST sub-unit was identified throughout the project site, in particular within the Diversion Channel. The BGST was typically encountered as a dark brown to grey, sandy, silty clay with variable gravel content. 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. 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 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 BGT was typically encountered as a brown, well-graded, sand and gravel with a variable fines content. SPT N values indicated that the density of this unit was ‘hard’ with typically +50 blows.

The LGST unit was identified beneath the Dam footprint by the boreholes and geological mapping. 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 Elevation 1187.7 and 1173.4 m. The thickness ranged between 1 and 9.3 m. 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.

6.4.3 Brazeau Formation

At the Diversion Structure and Floodplain Berm location, the top of Brazeau 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 formation by approx. 10 to 12 m with the top ranging between Elevation 1207.5 m and Elevation 1210.9 m. In the diversion channel, the top of the formation 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 becomes more undulating and ranges between Elevation 1212.2 m and Elevation 1205.4 m.

Direct shear and unconfined compressive strength (UCS) tests were undertaken on 10 bedrock samples 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 Coefficient = 0.43 (Peak), 0.25 (Residual)
- Shale/Mudstone DSSS Friction Coefficient = 0.54 (Peak), 0.47 (Residual)
- Mudstone/Claystone DSSS Friction Coefficient = 0.82 (Peak), 0.58 (Residual)
- Mudstone/Shale DSINT Friction Coefficient = 0.54 (Peak), 0.43 (Residual)
- Mudstone DSINT Friction Coefficient = 0.34 (Peak), 1.16 (Residual)

Slake durability index (SDI) testing was performed on 15 bedrock samples in the Brazeau formation. 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.

Localized artesian conditions were encountered in the Diversion Channel footprint in boreholes 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.

6.4.4 Coalspur Formation

The Coalspur formation was encountered in the eastern portion of the Diversion Channel, beneath the Emergency Spillway and on the western abutment of the Dam. The formation was encountered as a gently dipping sequence of thin to medium bedded, sandstones interbedded with thin beds of mudstone. No strength or hydraulic conductivity testing was undertaken on the formation during this stage of the investigation. Testing will be undertaken for the emergency spillway in the next phase of investigation.

6.4.5 Paskapoo Formation

The Paskapoo formation comprised an interbedded sequence of weathered clay, mudstones, sandstones and siltstones. Sandstone units were predominant within the western portion of the dam footprint, while mudstones and claystones units were predominant in the eastern portion of the dam footprint. Rotary drilling and geological mapping was undertaken to determine the presence of weak mudstone layers within the formation. The following observations were made as part of this 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;

- 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 four direct shear tests performed on mudstone samples. Residual strengths are discussed in the geotechnical report of Appendix D;

- 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 Elevation 1161. 5 m;

Based on residual strength discussions presented in the geotechnical report of Appendix D, a Ø'R = 17.5° was adopted for the mudstone units in this formation. The residual strength assessment has also identified evidence that potentially lower values could be mobilized locally. The results of packer testing and groundwater testing in this formation indicate the in-situ hydraulic conductivity ranged between 6.5×10^{-5} and 6.1×10^{-8} m/s.

6.5 EARTHWORK ASSESSMENT

An earthworks assessment has been undertaken using laboratory test data from designed excavation areas and borrow sources. The quantity of data comprised:

- 174 Atterberg Limits
- 166 Particle Size Distribution
- 2 dispersion suites (crumb test, pinhole, and double hydrometer)
- 30 Standard Proctor on GL and GT samples
- 17 consolidated undrained triaxial tests on remolded GL and GT samples
- 9 permeability tests on remolded GL and GT samples.

Results of the laboratory testing are provided in Appendix D. This section describes the classification and specification of materials for use as embankment in the project. Specific applications to each project component are discussed in Sections 8 through 10.

6.5.1 Impervious Fill Zone 1A Applications

Impervious Fill Zone 1A will be required at:

- Floodplain Berm Core
- Diversion Channel Embankments
- Off-Stream Storage Dam Core
- Low Level Outlet Works (LLOW) Backfill
- Diversion Structure Backfill

Supplemental specifications for the Impervious Fill Zone 1A will be required to meet the design intent of the project. Impervious Fill Zone 1A embankment core shall be limited to plastic glacial clay till soils compacted to a minimum of 95 percent of standard Proctor value and placed with an allowable moisture content ranging from one percent below to two percent above Proctor optimum moisture content. The recommended maximum liquid limit is 50 percent with a maximum particle size of 75 mm (3 inches). The minimum recommended plasticity index is 10.

6.5.2 Random Fill Zone 2A Applications

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

Random Fill Zone 2A will be required at:

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

The following supplemental specifications to the CWMS Random Fill Zone 2A requirements will be necessary for the subclasses of material:

Random Fill Zone 2A (1): Select soil embankment may include moderate to highly plastic glacio-lacustrine clay soils or glacial till clay 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 two percent below to two percent above Proctor optimum moisture content. Do not use moderately plastic glacial clay till until a plan has been developed to demonstrate that an adequate quantity of Zone 1A compliant soil remains available to complete Impervious Fill Zone 1A (core) placements.

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 mm (18 inches).

6.5.3 Borrow Sources

The Diversion Channel excavation will be the primary source for the project. The majority of the soil and rock excavated from the Diversion Channel can be used as Impervious Fill Zone 1A or Random Fill Zone 2A material. The majority of the fill excavated from Borrow Source 1 can be used as Impervious Fill Zone 1A.

6.5.4 Waste Material

The CWMS defines 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 not be used as engineered fill in the Floodplain Berm or the Off-stream Storage Dam and will only be placed in designated stockpiles or used as fill in the Unnamed Creek area upstream of the dam or other locations as warranted.

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.

6.6 HYDROGEOLOGY

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

6.6.1 Surficial Aquifers

The RGA 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.

6.6.2 Bedrock Aquifers

The RGA also defined two 'shallow bedrock' aquifers within the 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.

- The Dalehurst Member aquifer. This is the youngest stratigraphic member of the PPF. 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.

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.

6.6.3 Local Hydrogeology

Piezometers and groundwater monitoring wells were installed to characterize the local hydrogeology for the project site. Also, depth to water was recorded during the geotechnical investigation for each borehole. The impact of the project on the overall hydrogeology of the area is discussed in the hydrogeological assessment as part of the EIA. The data obtained from the groundwater wells installed for the hydrogeological assessment, piezometers installed during the geotechnical investigation, and depth to water from the geotechnical boreholes was used to determine local groundwater conditions at each project component to use in the geotechnical analyses. Where local groundwater impacted design, additional discussion is included in the specific component sections of this report. Potential construction issues due to local groundwater are included in the construction consideration sections of this report for each project component.

The groundwater at the Floodplain Berm is closely related to the river elevation so the design assumed saturated foundation conditions. Therefore, it was not necessary to install piezometers to monitor groundwater at the Floodplain Berm for design. The depth to water recorded in each borehole during the geotechnical investigation ranged from 2.0 m to 2.9 m with an average depth to water of 2.4 m. The Diversion Structure boreholes were advanced in the river, so no depth to water was recorded.

Near the beginning of the Diversion Channel (Station 10+000 to 10+800), the depth to water varies significantly. The average depth to water is 8.6 m with depths to water ranging from 1.2 m at DC9 to 16.8 metres at DC7. During the geotechnical investigation, artesian conditions were encountered in DC1 and DC5 in the upper 4 m of bedrock. The remainder of the diversion channel appears to have a more consistent depth to water with an average depth of 3.9 metres with depths to water ranging from 3.6 metres at DC21 to 4.4 metres at DC27.

At the beginning of the Dam (Station 20+200 to 21+600), groundwater was generally not encountered in the soil overburden. The groundwater was typically three to six metres into the bedrock. The remainder of the dam has groundwater in the soil overburden, with an average depth to water of 4.7 metres ranging from 1.1 metres at D28 to 7.8 metres at D27.

6.7 SEISMIC ASSESSMENT

A site specific Probabilistic Seismic Hazard Assessment (PSHA) is presented in Appendix D. The purpose of the PSHA was to define ground motion parameters for use in seismic design for the project. According to the CDA Dam Safety Guidelines (CDA, 2013), an Extreme hazard 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.

The 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.

Additionally, induced seismicity is common in the foothills region of Southwestern Alberta. Notable areas in which induced seismicity has been documented include the Crooked Lane Sequences (Schultz et al., 2015a) located approximately 30 km west of Fox Creek, the Brazeau River Cluster (Schultz et al., 2014) located approximately 150 km northwest of Calgary, the Rocky Mountain House Seismogenic Zone (Wetmiller, 1986) located approximately 100 km northwest of Calgary, and the Cardston Earthquake Swarm (Schultz et al., 2015b) located approximately 200 km southeast of Calgary. 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. Both natural seismicity and induced seismicity were considered for this assessment.

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.

Seismic parameters developed from the PSHA were used in the analyses for the individual components. For the Floodplain Berm and Off-stream Storage Dam, the PGA was used to determine the horizontal pseudostatic coefficient for use in the pseudostatic stability analyses. Earthquake time histories from the PSHA were used in the seismic deformation analysis conducted for the Off-stream Storage Dam. Seismic loading parameters derived from the results of the PSHA were utilized for the Diversion Structures stability analyses.

7.0 STRUCTURAL

7.1 GENERAL

This section summarizes the design standards, design parameters, loads, analysis methodology, and stability acceptance criteria needed for structural analysis and design of the hydraulic structures associated with the Springbank Off-stream Storage Project. Design objectives, design criteria, stability assessment results, and structural analyses of major components for the hydraulic structures are described in subsequent sections or detailed in Appendix E - Structural:

- Section 8 - Diversion Structure: Diversion Inlet (DI), Service Spillway (SS), Auxiliary Spillway (AS), and Debris Deflection Barrier (DDB).
- Section 9 - Diversion Channel: Emergency Spillway (EMS) and Diversion Channel Outlet Grade Control Structure (GCS)
- Section 10 - Off-stream Storage Dam: Low-Level Outlet Works (LLOW)

Hydrologic, hydraulic, geotechnical investigations and design parameters, and operation requirements are described in other sections of this report.

7.2 APPLICABLE STANDARDS

The design complies with current AT Design Standards and relevant AT Design and Construction Bulletins. By reference in AT Standards, CDA Dam Safety Guidelines including Technical Bulletin Nos. 1 through 9 provide primary guidance for design of the project including the hydraulic structures. Other recognized industry standards referenced in the AT/CDA Guidelines are used to supplement design where the AT/CDA Guidelines do not address a design aspect. Such references include the US Army Corps of Engineers (USACE) Engineering Manuals, US Bureau of Reclamation (USBR) Design Standards, and Federal Emergency Management Agency (FEMA) Best Practices. In case of conflicting criteria, AT provisions are the defaults unless a "more stringent" requirement was deemed appropriate based on CDA Guidelines or engineering judgment.

Where referenced by AT or CDA, the AT Bridge provisions and Alberta Building Code (ABC), supplemented by the National Building Code of Canada (NBCC) were used to obtain certain design loads (wind, snow, live, vehicle). AT Bridge provisions or ABC were used primarily for evaluation of individual elements such as access decks, parapet walls, stair/walkways, and other ancillary structures as well as defining the pertinent standards to be used for reinforced concrete, structural steel, mechanical, electrical, and other design codes.

The codes, guidelines and standards used on this project are enumerated in the Structural Design Reports presented in Appendix E.

7.3 CONSTRUCTION MATERIALS

The major construction materials are described below. Other materials used in permanent construction such as grout, sealant, and waterstops will be described in the specifications.

7.3.1 Concrete

The following classes of concrete will be used in the various hydraulic structures:

Structural Concrete – Class A1
30 MPa @ 28 days, (AT Civil Works Specifications)
General use reinforced concrete where thermal control and volume change are not a concern such as the LLOW intake and its retaining walls.

Structural Concrete – Class B1
30 MPa @ 90 days, (AT Civil Works Specifications)
General use reinforced concrete where thermal control and volume change need to be considered (typically thickness > 600 mm) such as DI and SS retaining wall slabs and stems, and providing an air-entrained cover for mass concrete or hardfill.

High Performance Concrete – Class HPC
45 MPa @ 28 days (AT Bridge Construction Specifications)
Reinforced concrete elements needing high strength, durability or in a corrosive environment. Typical elements include precast concrete and bridge parapets.

Foundation Concrete - Class F
15 MPa @ 28 days, (AT Civil Works Specifications)
For use in protection of newly exposed foundations and for dental concrete, mud mats, or low strength fill.

Mass Concrete – Class M
20 MPa @ 90 days, 30 MPa @ 180 days (New mixture to be specified)
Unreinforced concrete for monoliths, slabs, piers and retaining walls where thermal control and volume change need to be considered (typically thickness >1500 mm).

Roller Compacted Concrete – Class RCC
15 MPa @ 28 days. (New mixture to be specified)
For exposed RCC lifts used for overtopping protection such as for the DCO.

Hardfill (Cemented Sands and Gravels) – Class CSG
7.5 MPa @ 90 days. (New mixture to be specified)
For use in the interior of mass concrete structures, such as the AS interior concrete, where low compressive strength materials can be used but where thermal control, volume change, and permeability are the primary considerations.

7.3.2 Metals

Reinforcement - CAN/CSA-G30.18, Grade 400W steel deformed bars, Grade 400 galvanized steel deformed bars, and stainless steel deformed bars.

Structural Steel - CSA-G40.21, Grade 300W or 350W

Stainless Steel - ASTM A276

Miscellaneous Metals (stairs, ladders, handrails) - Galvanized steel

Grating - Galvanized steel – serrated bar grating

7.3.3 Earthwork Materials

Soil backfill parameters are based on terminology in AT's Civil Works Master Specification – Division 2. Refer to Appendix D for design values for various backfill soil and riprap classes. Backfill materials used for structural analyses and design for individual hydraulic structures are described in Appendix E for each hydraulic structure.

7.4 LOADS

7.4.1 Dead Loads (D)

Permanent loads on the structure include concrete mass, fixed equipment, and post-tensioned anchors if present. Unit weights for principal materials are included in Table 8.

Table 8. Dead Load Unit Weights

Material	Unit Weight	Source
Water	9.81 kN/m^3	CSA S6-14, Table 3.4
Reinforced Concrete	23.5 kN/m^3	AT WCS Design Guide. 4.2
Mass Concrete/RCC/Hardfill	22.8 kN/m^3	USBR, Design of Gravity Dams, Section III.B
Steel	77.0 kN/m^3	AT WCS Design Guide, 4.2

7.4.2 Hydrostatic Loads (H)

Both horizontal and vertical components of water load were included based on water surface elevation for the load condition considered. The water surface elevations were considered to be hydrostatic pressures without kinematic effects. Headwater was generally considered the water surface elevation at the upstream face of gate or structure. Tailwater was either maximum tailwater elevation indicated on tailwater rating curves, or a reduced tailwater elevation to account for hydraulic jump depending on load condition considered and which condition produced a more adverse effect on the structure.

7.4.3 Uplift Pressure (U)

For gravity structures and stilling basins, uplift was assumed to vary from 100 percent of the hydrostatic headwater pressure at the upstream face of structure to 100 percent of hydrostatic tailwater pressure at the downstream face of the structure with application over 100 percent of base. For retaining walls, the uplift was assumed to vary from 100 percent of the water pressure at the face of the foundation heel to 100 percent of water pressure at the face of the foundation toe applied over 100 percent of the base.

For analysis of overturning capacity and floatation for gravity structures, stilling basins and retaining walls, uplift pressure was considered to vary proportionally along the length of concrete structure/rock contact surface. For sliding and bearing capacity analysis of gravity structures and retaining walls, uplift was assumed to vary along the length of the linear sliding failure plane under consideration (horizontal concrete/rock contact, or through rock if structure contact with rock was keyed or sloped). These uplift distributions follow the guidance provided in USACE EM 1110-2-2502 (USACE, 1989) for gravity and retaining walls.

The foundation interface was assumed to have zero tensile capacity. For bases where stability calculations indicated bearing pressures less than zero, the foundation interface was assumed to crack, and 100 percent of the hydrostatic pressure was applied over the entire area of the cracked foundation, then vary linearly to 100 percent of tailwater pressure. For seismic evaluations, uplift loading remained unchanged from the pre-earthquake condition to the post-earthquake evaluation unless seismic loading resulted in a cracked foundation, in which case full hydrostatic pressure was applied to the entire area of the cracked foundation during the post-earthquake evaluation.

Where seepage reduction measures were provided, such as drains, a reduced uplift pressure was used for stability analyses for the Usual Load Condition only. For stability analyses of other load conditions, the seepage reduction measures were conservatively neglected.

7.4.4 Earth Pressure/Sediment/Siltation Loads (E)

Soil loads include both vertical and horizontal forces due to backfill, sediment, and siltation. The sediment transport analyses, described in Section 5, indicate there is little to no expected accumulation of sediment adjacent to the structures located on the Elbow River, so loads associated with Sediment/Siltation were excluded from the structural analysis.

Vertical force associated with soil mass above the structure was based on vertical projection of footing or structure below the soil. Soil mass was based on moist unit weight for material above the waterline and buoyant unit weight for material below the waterline. Vertical force associated with water above the structure was calculated separate from the soil mass.

Horizontal force associated with soil was based on at-rest condition represented by the empirical relationship:

$$K_o = 1 - \sin\theta$$

where: K_o = At-rest lateral pressure coefficient(*)
θ = Soil friction angle

**In accordance with EM 1110-2-2100 & EM 1110-2-2502 to use At-Rest Coefficient (K_o).*

7.4.5 Live Loads (O and V)

The principal live loads include Occupancy Loads (O), typically for buildings, Vehicle Loads (V), hoist/equipment loads for gates, and heavy equipment surcharge loads, particularly adjacent to retaining walls. Occupancy Loads were not used for stability of the hydraulic structures but were used in design of specific components such as personnel gratings and hand rails in accordance with ABC. Vehicle Loads were included on the access deck between gate piers at the Diversion Inlet and the access bridge at the LLOW gate structure. Hoist/Equipment Loads are associated with gate operation and gate hoist support design.

Heavy equipment surcharge was applied to retaining wall design as a separate load condition to account for future modifications such as building additions, long-term material storage, or top-of-wall modifications. This load is not applied simultaneously with Vehicle Loads.

Live Loads described in this section are considered transitory loads. Transitory loads were used for strength design of individual structure elements but were not included in stability analyses.

Table 9. Occupancy, Vehicle and Hoist Live Loads

Description	Live Load	Source
Occupancy & Access (machine rooms)	3.6 kPa	2014 ABC, Table 4.1.5.3
Vehicle (Vertical Application)	CL-625	CSA-S6-14, Section 3.8.3
Vehicle (Horizontal Application)	CL-625	CSA-S6-14, Section 3.8.3
Dynamic Load Allowance (Increase)	0.5	CSA-S6-14, Section 3.8.4.5.3
Traffic Barrier Load (TL-2 Performance)	50 kN Transverse, 20 kN Longitudinal, 10 kN Vertical	CSA-S6-14 Section 12
Vehicle Guardrails	22 kN @ 500 mm above grade	2014 ABC, 4.1.5.15
Bracing Force (Equivalent Static Force)	180 kN applied at deck surface	Minimum CL-W loading x 1.25
Crane Surcharge	CL-800	CSA-S6-14 & AT Bridge Provisions
Gate Hoist & Equipment	Refer to Appendix E - Structural for DI, SS and LLOW	Appendices E.1, E.2 and E.5
Heavy Equipment Surcharge	15.0 kPa (Equivalent 0.75 m soil)	AT WCS, Section 4.9

7.4.6 Hydrodynamic Loads (HD)

Hydrodynamic loads include wave action, sub-atmospheric pressure at the fixed crest, and hydraulic dissipater forces. For the hydraulic structures, these forces have been excluded from stability analysis since they are considered insignificant or of a localized nature.

- Wave action is not included due to the short-term duration and relatively short fetch.
- Sub-atmospheric pressure is not included since there is insufficient head to develop sub-atmospheric pressure on the fixed crest.
- Hydraulic Dissipater forces are localized forces addressed in the hydraulic design of stilling basin and chute blocks.

7.4.7 Debris and Impact Loads (I_M)

Impact loads associated with debris flows were based on geometry of the Diversion Structure conservatively assuming the Debris Deflection Barrier was not in place. Debris impact and drift loads were derived from 2D hydraulic modelling of the Diversion Structure based on various flood events.

7.4.8 Ice Loads (I)

Static Ice Load (I_s), Dynamic Ice Loads (I_d), Ice Accretion Loads (I_v) and Frost Heave were considered for design of the hydraulic structures.

Static Ice Load (I_s) is a result of rising temperatures within a confined space causing an ice sheet to expand. Static ice loading has the potential to occur at low flow conditions, particularly within the Service Spillway stilling basin. Static Ice Loads were applied in Usual Load Cases which address winter operating conditions.

Dynamic Ice Loading (I_d) is a result of moving ice floe impacting the structure. Dynamic Ice Loading was not considered as a design load case because no structures have a permanent pool.

Ice Accretion Load (I_v) occurs when ice bonds to the structure and must be broken as water level rises. Ice Accretion Load associated with water level rise was not considered for stability of hydraulic structures due to the small order of magnitude relative to hydrostatic loading and low probability of occurring simultaneous with spring and summer flooding. However, ice accretion was considered for certain individual element design such as the Debris Deflection Barrier superstructure. Ice accretion loads were developed according to the ABC.

Frost Heave. Vertical ice loading associated with "frost heave" is a realistic consideration. The structures are normally in a dewatered or low-water state with freeze/thaw action tending to open rock joints or concrete/rock interface and subject the structure to increased uplift potential. To reduce frost heave loading potential and remove this condition from the analysis, foundation interfaces were located below the identified frost depth (2 m) for the site, insulation provided between foundation and structure, or drainage provided to reduce the formation of ice in the foundation.

Table 10. Ice Loads

Ice Condition	Load	Source
Static Ice (applied to structure)	150 kN/m @ 0.3 m below WS	AT WCS Design Guide 4.5.1.1
Static Ice (applied to gates)	75 kN/m @ 0.3 m below WS	AT WCS Design Guide 4.5.1.1
Ice Accretion	Per requirements of ABC for affected structures	ABC

7.4.9 Seismic – Earthquake Loads (Q)

The seismic classification for the hydraulic structures was based on the site specific PSHA described in Section 6.7 and included in Appendix D. Since the hazard classification for this project is either High (Service Spillway, Auxiliary Spillway and Floodplain Berm) or Extreme (Diversion Inlet, Diversion Channel and Off-stream Storage Dam), the seismic parameters are based on an Earthquake Design Ground Motion (EDGM) with an Annual Exceedance Probability (AEP) of 1/10,000 resulting in Peak Ground Acceleration (PGA) of 0.26 g for horizontal application and PGA of 0.15 for vertical application.

This project site is situated in an area of low to moderate seismic activity, and CDA Guidelines Section 6.5 allow for the seismic stability analysis of concrete gravity structures to be completed using a pseudo-static approach (coefficient method) with application of seismic force in a rigid body analysis with the objective of determining sliding and overturning response of the structure. Since the pseudo-static method does not recognize the oscillatory nature of seismic loads, accepted practice is to perform the stability calculations using sustained acceleration values equivalent to 2/3 of the peak acceleration values.

When performing concrete structure stability analyses, the objective is to determine the tensile crack length induced by the inertia forces applied to the structure, so peak acceleration is used to calculate seismic coefficients. This approach assumes an instantaneous acceleration spike can induce cracking but is not sustained long enough to develop significant displacement along the crack plane. If no significant displacement occurs, the dynamic stability is maintained.

7.4.9.1 Seismic Effects on Concrete Mass

The horizontal force required to accelerate the concrete mass is calculated as:

$Q_h = k_h \times W$ where:

Q_h = Horizontal seismic load (kN)
k_h = Horizontal seismic coefficient
W = Structure mass (kg)
PGA = Peak ground acceleration= 0.26g

For Stability Analysis (Table 11): k_h = 2/3 x 0.26 = 0.17
For Member Analysis (Table 12): k_h = 1.0 x 0.26 = 0.26

The vertical force required to accelerate the concrete mass is calculated as:

$Q_v = k_v \times W$ where: Q_v = Vertical seismic load (kN)
k_v = Horizontal seismic coefficient = 0.56*k_h
W = Structure mass (kg)

For Stability Analysis (Table 11): k_v = 2/3 x (0.56*k_h) = 0.10
For Member Analysis (Table 12): k_v = 1.0 x (0.56*k_h) = 0.15

Since an earthquake produces oscillating forces, the horizontal PGA and vertical PGA cannot occur at the same time. To account for this in the stability calculations, three separate combinations of vertical and horizontal seismic combinations were considered, but only the maximum value was reported. The three combinations of vertical and horizontal seismic load are as follows:

Table 11. Stability Analysis – Seismic Coefficients

Seismic Combination	Horizontal	Vertical
100% Horiz., No Vert.	1.0*k_h = 0.17	-
100% Horiz., 30% Vert.	1.0*k_h = 0.17	0.3*k_v = 0.03
30% Horiz., 100% Vert.	0.3*k_h = 0.05	1.0*k_v = 0.10

When designing structural members for seismic conditions, the peak acceleration is not modified by the 2/3 factor. As with the stability analyses, three combinations of vertical and horizontal seismic loads were considered, but only the maximum value was used in design. The three combinations of vertical and horizontal seismic load for member design are as follows:

Table 12. Member Analysis – Seismic Coefficients

Seismic Combination	Horizontal	Vertical
100% Horiz., No Vert.	1.0*k_h = 0.26	-
100% Horiz., 30% Vert.	1.0*k_h = 0.26	0.3*k_v = 0.05
30% Horiz., 100% Vert.	0.3*k_h = 0.08	1.0*k_v = 0.15

7.4.9.2 Seismic Effects on Water

Using a pseudo-static method, hydrodynamic effects on water were approximated by using the Westergaard method to calculate the seismic water force (H_E). The calculated hydrodynamic force is additive to the hydrostatic water pressure force. The distribution is parabolic with the line of action for the force H_E at 0.4h above the base of the water column. Detailed explanation of method can be found in Section 4-7.e, EM 1110-2-2100 (USACE, 2005).

7.4.9.3 Seismic Effect on Soils

Dynamic soil pressures and associated forces were analyzed assuming non-yielding backfills and an elastic response using the Wood's method. As referenced in Section 5-5.a.1, EM 1110-2-2100, (USACE, 2005) and verified by project specific calculation (Appendix D), this method can be expected to have dynamic soil pressures greater than those predicted by the Mononobe-Okabe method for yielding backfills.

The use of Wood's method is considered reasonable and was used for analysis of gate bays that have relatively short backfills (<4 m) consisting primarily of rock fill for erosion protection. The use of Wood's method may be overly conservative for taller retaining walls with height ranging above 4 m with backfill consisting of granular fills and/or glacial till materials.

For conditions where seismic load cases control the wall design, the Mononobe-Okabe method with active soil pressure during seismic conditions was considered to assess wall stability.

7.4.10 Climatic Conditions

7.4.10.1 Snow Loads (S)

Snow loads were considered insignificant compared to hydrostatic loads and were not considered for stability of the hydraulic structures. Snow Loads were included in load combinations for certain component designs such as breastwalls, bridge decks between piers, and access platforms.

Snow Load data for this project was obtained from Ontario Climate Centre – Environment Canada.

Ground snow load, snow component (S_s) =1.7 kPa

Ground snow load, rain component (S_r) =0.1 kPa

Snow load, Importance factor (I_s) =1.25

7.4.10.2 Thermal Loads (T)

Thermal loads will be evaluated during Final Design to determine joint condition, concrete mixture, monolith sizing, and lift heights for concrete structures. Monthly temperature data for use in the evaluation was obtained from the Calgary International Airport records, which is considered representative of typical temperature ranges at the project site.

7.4.10.3 Wind (W)

Wind loads were evaluated for the Debris Deflection Barrier and the LLOW structures. Wind loading is not a critical condition for the remaining structures since water loads would govern. A wind load of 0.48 kPa was determined for use at the site based on the Alberta Building Code.

7.5 STRUCTURAL ANALYSIS METHODOLOGY

For the purposes of stability assessment, analysis and structural design, the hydraulic structures were divided into individual structures, such as the LLOW intake, monoliths for mass concrete structures, such as gate crest monoliths, or reinforced concrete monoliths, such as retaining walls, based on structure geometry, size, joint location, and loading considerations. The structures or monoliths were analyzed as either concrete gravity sections (gate structures, piers), retaining walls (wing walls, training walls, and abutment walls), or as independent reinforced concrete structures (LLOW intake). Each structure or monolith was evaluated for global stability, strength, and serviceability.

Global stability was assessed using the rigid body analysis method and application of unfactored loads. This method uses the summation of forces applied to the monolith to determine resultant location, foundation bearing pressures, sliding resistance along identified potential failure plane(s), and floatation. Analysis methodologies are detailed in the section for each hydraulic structure.

Strength evaluation of individual elements or members of structures and monoliths was used to verify member sizes based on application of factored loads. In general, structural analyses were performed manually using MathCAD or Excel spreadsheets. For more complex structures, such as the LLOW intake, a commercial 3-Dimensional Finite Element Model (FEM) was used to evaluate multiple load combinations, identify stress concentrations, and generate shear and moment values for design of individual elements. The FEM was supplemented with manual calculations to verify/validate model results and where necessary, refine the analysis of individual elements. Based on model output, a combination of manual calculation and commercial software was used for strength design. Additional elements evaluated as part of strength design included joint detailing, equipment anchorage, and embedded parts.

Serviceability includes limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR). The same manual calculations, commercial software, or 3-Dimensional FEM used for strength evaluation were used to evaluate deflection and thermal growth, while design detailing and material specification were used to mitigate cracking and AAR potential.

7.6 STABILITY ANALYSIS METHODOLOGY

Representative monoliths, retaining walls, or individual structures were analyzed for stability assessing bearing, resultant location, sliding, and floatation for load conditions applicable to the site. The stability results for these structures are described later in Sections 8, 9 and 10. The stability analysis methodology that was used in assessing these structures is described below.

7.6.1 Overturning and Bearing Stress

The Rigid Body Method (conventional gravity method) was used for the analysis of overturning and bearing stress criteria. Overturning was evaluated as a percentage of base that remains in compression and not a safety factor. This method is outlined in Section 7.2 of CDA Technical Bulletin No. 9 (CDA, 2007b) and further described in USACE EM 1110-2-2100 (USACE, 2005). It uses the vector summation of all forces, including uplift, acting on the monolith to determine the vector resultant force (V), resultant force eccentricity (e) within the base, and moment (Ve/S) based on an elastic and homogeneous rectangular beam analogy. Stresses were calculated as indicated below and stability is assured by maintaining the resultant force eccentricity within acceptance criteria limits for various loading conditions. These limits are described later in Acceptance Criteria for Stability Analyses.

$$\sigma = \frac{V}{A} \pm \frac{Ve}{S}$$

Where: σ = Applied bearing pressure at each end of base (kN/m^2)
V = Summation of forces normal to base (kN)
A = Base area in compression (m^2)
e = Eccentricity of normal load about centroid of base in compression (m)
S = Section modulus of base area in compression (m^3)

7.6.2 Sliding

The sliding factor of safety was calculated for each load case using the limit equilibrium method as outlined in Section 7.2 of CDA Technical Bulletin No. 9 (CDA, 2007b). This method reduces to the equation shown below for a single wedge system with a horizontal sliding plane, along the concrete/rock interface (CRI) or through rock/rock failure plane as identified for each hydraulic structure. For inclined sliding planes projecting from the base of shear key to bottom base slab at the toe, vertical and horizontal forces are resolved into components normal and parallel to the sliding plane. Rock mass between the inclined plane and CRI is included in the dead load summation (EM 1110-2-2100). For this project, cohesion was conservatively assumed to be zero and sliding acceptance criteria were based only on the friction angle.

$$SSF = \frac{(V tan\varphi + c\ A)}{H}$$

Where: SSF = Sliding Safety Factor
V = Summation of vertical loads including uplift (kN)
$tan\ \varphi$ = Coefficient along sliding plane being considered
c = Cohesion at concrete/rock or rock/rock interface (assumed as 0) (kN/m^3)
A = Base area in compression (m^2)
H = Summation of horizontal forces (kN)

7.6.3 Floatation

The floatation factor of safety was determined for components of the project such as stilling basins and apron slabs as outlined in Section 8.5, AT WCS (2004b). The factor of safety against floatation is defined as the ratio of resisting gravity force to driving uplift force. The possible resistance due to friction between adjacent structures or between structure and backfill was neglected unless shear provisions were provided.

$$FSF = \frac{\Sigma N}{\Sigma U}$$

Where: FSF = Factor of Safety against Floatation
ΣN = Summation of normal forces
ΣU = Summation of uplift forces

7.6.4 Acceptance Criteria for Stability Analyses

The following acceptance criteria are based on AT WCS Chapter 8 (2004b), CDA Table 6-4 (2007), and CDA Technical Bulletin No. 8, Section 6.0 (2007). The load cases to be evaluated are divided into five categories as listed in Table 13.

Table 13. Acceptance Criteria for Hydraulic Structure

Loading Combination	Position of Resultant Force (Percent of Base in Compression)[1]	Normal Compression Stress[2]	Sliding Safety Factor (Friction Only)	Floatation Safety Factor
Usual	Middle third of the base: 100% compression	<0.3 x f_c	≥1.5	≥1.5
Unusual	Middle third of the base: 100% compression	<0.5 x f_c	≥1.3	≥1.3
Extreme Flood	Within middle half of the base, and all other acceptance criteria must be met	<0.5 x f_c	≥1.1	≥1.1
Extreme Earthquake	Within the base, except where an instantaneous occurrence of resultant outside the base may be acceptable	<0.9 x f_c	Note [3]	
Post-Earthquake	Within middle half of the base	<0.5 x f_c	≥1.0	≥1.1

[1] Foundation bearing stress is compared to allowable stress determined from Geotechnical Investigation
[2] Where f_c = compressive strength of concrete
[3] Earthquake load case is used to establish post-earthquake condition of the structure

7.6.4.1 Usual Condition

Those conditions under which the structure is intended to serve during normal operations and further defined as a condition that has a high likelihood of occurring within the design life of the structure. Usual load conditions include normal pool and winter conditions. For the hydraulic structures, this includes flood events up to the 50-year frequency flood for high hazard classified structures and flood events up to the 100-year frequency flood for extreme hazard classified structures.

7.6.4.2 Unusual Condition

Those conditions that occur infrequently and may stress the structure more, under certain aspects, than normal conditions and may occur within the design life of the structure. Unusual load conditions include construction conditions, maintenance conditions, flood events between the 50-year and 1000-year frequency, infrequent earthquake events other than the MDE, and plugged drain conditions for Usual Load Cases.

7.6.4.3 Extreme – Flood

Extreme Load Conditions have a very remote likelihood of occurring within the design life of the structure. For the SR1 project, it is defined as those floods that occur from the 1000-year frequency event up to the structure's IDF.

7.6.4.4 Extreme – Earthquake

For the SR1 project, the Extreme - Earthquake load condition to be assessed is the MDE as it has a very remote likelihood of occurring within the design life of the structure. The MDE is applied to the Usual Condition load cases. The Extreme – Earthquake condition is used to establish Post-Earthquake condition of the hydraulic structure. Thus, there are no stability acceptance criteria for this condition.

7.6.4.5 Post-Earthquake

The Post-Earthquake condition assesses the stability of the hydraulic structure following the applied seismic event based on earthquake induced cracking at the foundation/structure interface and within the structure so that it can still be capable of resisting the Usual Loading.

7.7 STRENGTH EVALUATION

Strength evaluation of individual elements or members of structures and monoliths was used to verify member sizes based on application of factored loads as described in ABC with some adjustments for more severe conditions or loads not considered in the ABC.

Reinforced concrete design, except for bridge components, was performed according to Design of Concrete Structures, CSA A23.3-14 (2018) with the additional requirements of the CSA's SEED Document – Structural Design of Wastewater Treatment Plants-(2018) for revisions addressing service load conditions, water tightness, shrinkage and temperature reinforcement, and crack control. The Seed Document contains references to ACI 350M-06 for modifying CSA A23.3-14. For bridge components, such as the Diversion Inlet access bridge, reinforced concrete design was performed according to AT Bridge Design Criteria (2017) supplemented by the Canadian Highway Bridge Design Code, CSA S6-14 (2018).

Structural steel design was performed according to Design of Steel Structures, CSA S16-14 (2018), and codes for welding, materials, and other pertinent references.

7.8 SERVICEABILITY

Serviceability relates primarily to concrete durability including limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR) and other chemical attack. Shrinkage control and volume changes were addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The monolith layout and design include joint locations that define monoliths with balanced aspect ratios and placements less than 12 to 18 m in any one planar direction. Joint placements allow for "checkerboard" placements of monolith lifts to allow dissipation of heat of hydration and initial shrinkage before an adjacent monolith lift is placed. Expanded guidance related to placement sequence and joint locations will be addressed as part of constructability review during Final Design.

Member size and stiffness was controlled by deflection limits established for gate and hoist equipment and climate thermal expansion/contraction. Retaining walls must be much stiffer at gate locations to limit movement that might cause leakage or poor gate operation. Deflection of the gate itself must be controlled to maintain integrity and contact of gate seals. Hoist support deflection must be uniform to minimize secondary loads on shaft and gears, and within limits of comfort of personnel working on the hoist support platform.

Tight installation tolerances for gates, hoists and other embedded components were critical for their operation or installation. These tolerances were addressed primarily through second stage concrete placements occurring after initial concrete shrinkage has occurred. Using a second stage concrete placement also allows gate assemblies to be installed, checked and adjusted for operation before components were permanently embedded in concrete.

8.0 DIVERSION STRUCTURE

8.1 GENERAL

The Diversion Structure is located on the Elbow River approximately 1.2 km upstream of Highway 22. Structure layout and grading is presented on Drawing C-201. The Diversion Structure is comprised of four sub-components that collectively constrain Elbow River flood flows and control the discharge to the Diversion Channel and downstream Elbow River channel. These sub-components include the Diversion Inlet, Service Spillway, Auxiliary Spillway, and Floodplain Berm. The Debris Deflection Barrier, a fifth sub-component, located within the Elbow River channel limits the passage of large debris entering the Diversion Inlet.

8.1.1 Design Objectives

The Diversion Structure is designed to constrain and manage flood events up to the FoR, 2013 Storm hydrograph, for diversion to the Off-stream Storage Reservoir via the Diversion Channel. As a High Hazard structure, the Service and Auxiliary Spillways are designed to pass the IDF-DS without overtopping the Floodplain Berm.

8.1.2 Diversion Capacity

The required project diversion capacity was calculated for the FoR assuming Glenmore Dam operates with a constant release rate of 160 m^3/s and provides 10,000 dam^3 of flood storage. Based on these assumptions, the minimum required diversion capacity was computed as 480 m^3/s as illustrated in Figure 12 below. The USACE HEC-ResSim model described in Appendix B.6 was used to verify that a diversion capacity of 480 m^3/s was sufficient to meet the downstream flood control requirements for the FoR design hydrograph.

The presented minimum 480 m^3/s diversion scenario represents an idealized operations scenario where the gates operate in perfect timing and the effects of sediment and debris on diversion capacity are not considered. During design planning, it was deemed appropriate that the design capacity should be higher to accommodate potential debris and sediment impacts and to accommodate additional diversion needs in the event the Glenmore Reservoir volume is filled early during the design hydrograph. Therefore, an additional 25 percent capacity is recommended and was incorporated into the design to account for these risks and uncertainty. Thus, the recommended design diversion capacity is 600 m^3/s.

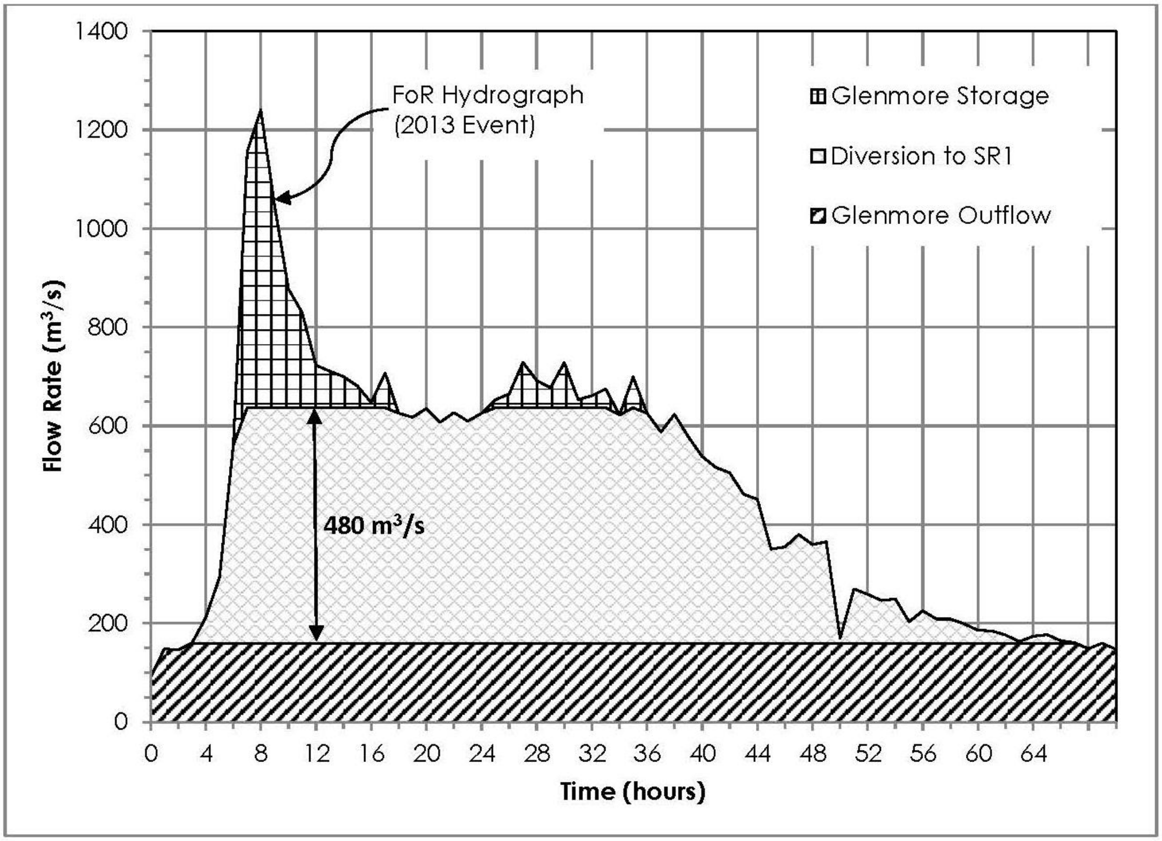


Figure 12. Minimum Diversion Capacity Hydrograph

8.1.3 Operations

The maximum discharge capacity of the low-level outlet works at the Glenmore Dam is 160 m^3/s. When the Elbow River discharge exceeds 160 m^3/s at Glenmore Dam, the reserved storage capacity of the reservoir will diminish and the combined flood operations will begin. For this reason, it is assumed that SR1 may begin operations when discharge within the Elbow River exceeds the capacity of the low-level outlet works at Glenmore Dam (160 m^3/s). Three distinct operation regimes are defined by discharge in the Elbow River. They are:

- **No Diversion (0-160 m^3/s).** Diversion of floodwaters from the Elbow River is not anticipated for discharge less than the 160 m^3/s capacity of the low-level outlet at Glenmore Reservoir. Diversion Inlet gates are closed and the Elbow River flows freely through the Service Spillway (Service Spillway gates are in the open (lowered) position).

- **Design Flood Operation (161-1600 m^3/s)**. Diversion operations commence when Elbow River discharge exceeds 160 m^3/s. The Diversion Inlet gates are opened and the Service Spillway gates are operated to control flow through the Diversion Inlet and downstream on the Elbow River. From 160 to 760 m^3/s, the Service Spillway gates are raised incrementally to maintain a constant flow rate downstream on the Elbow River of 160 m^3/s. Diversion rates may range from 0 to 600 m^3/s.

  For Elbow River discharge greater than 760 m^3/s, the Service Spillway gates are incrementally lowered to maintain a headwater elevation upstream of the spillway and a diversion rate up to but not exceeding 600 m^3/s. At 1600 m^3/s in the Elbow River, the Service Spillway gates are fully open (lowered position) and the Diversion Inlet gates will begin to close to limit flows into the Diversion Channel to no more than 600 m^3/s.

- **Structural Resilience and Dam Safety (1600 – 2770 m^3/s).** The Diversion Structure is designed to safely pass the IDF-DS through the Service Spillway and Auxiliary Spillway and to prevent flow from the IDF-OSSD from entering the Diversion Channel. The Diversion Inlet gates and structure are not designed for precise flow control. Diversion of floodwaters may be facilitated through incremental gate closure; however, gates should be positioned to control discharge into the Diversion Channel and not exceed the design capacity of 600 m^3/s.

Rating curves and operation rule curves will be provided in the Operations Manual, which will be developed as the design advances.

8.1.4 Design Progression

The Preliminary Design process began with evaluation of the Initial Design Concept (IDC) presented in the SAFRTF report (AMEC, 2014). Global changes to the Diversion Structure concept were reviewed and presented to AT in Stantec's Conceptual Design Update (2015). These alternatives considered alternate locations and diversion capacities. Following this initial concept update, geometries and debris mitigation measures were refined as part of the Physical Model Study (NRC-OCRE, 2016).

8.1.4.1 Conceptual Design Update

Stantec reviewed potential adjustments to the Diversion Structure location relative to the proposed IDC. Downstream locations were considered but dismissed due to the required storage elevation in the reservoir for the 2013 design event relative to river elevations. An alternate upstream location, approximately 400 m upstream of the IDC site, was identified with potential design, operations and construction cost impacts assessed relative to the IDC location.

The comparison between the upstream and IDC locations revolved around the benefits of increased channel elevations (at the upstream location) versus the shorter Diversion Channel (at the downstream location).

Results of the review indicated that the upstream location is approximately $5-15 Million more expensive than the IDC location and provides limited advantages to Diversion Structure operations.

Figure 13 shows the recommended general arrangement following the Conceptual Design Update in April 2015.

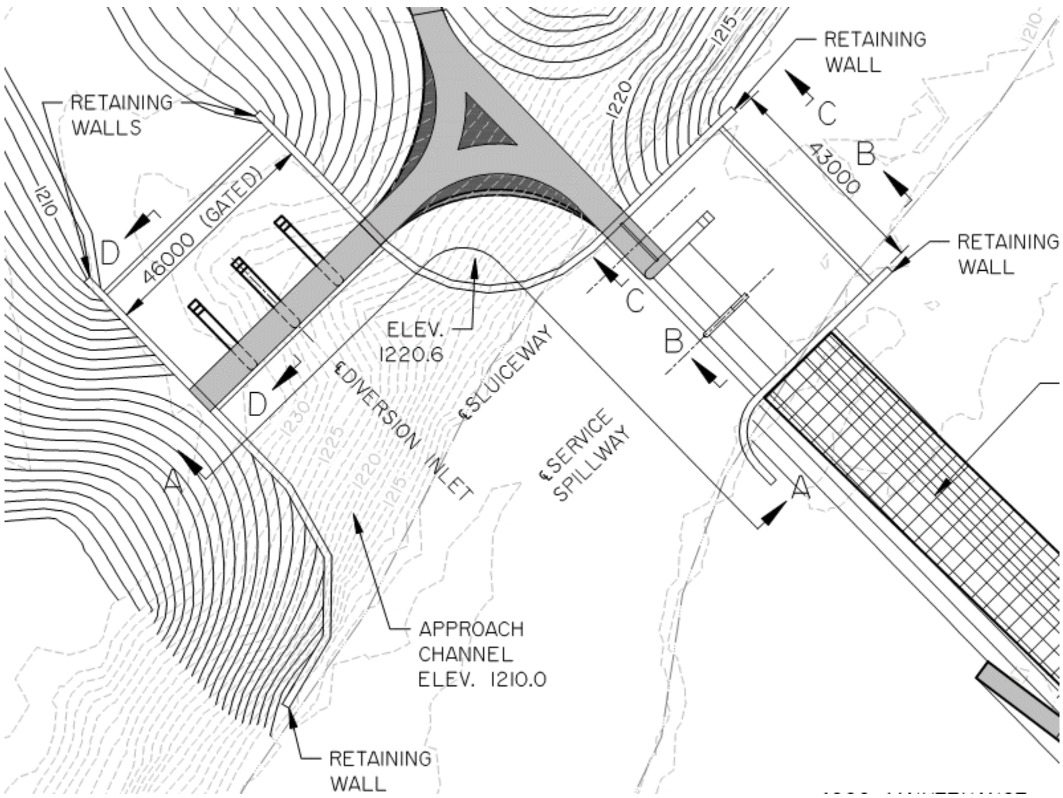


Figure 13. Conceptual Design Update (April 2015) – Diversion Inlet and Service Spillway

8.1.4.2 Physical Model Study – Large Woody Debris and Sediment

Methods and results of the numerical and physical model testing are provided in Section 3. Detailed discussions of the physical model are presented in Appendix C.

Revisions to the Diversion Inlet and Service Spillway occurred through an iterative hydraulic modeling process. Revisions to the proposed design were made to:

- Improve hydraulic conditions through the spillways;
- Improve the transport capacity of bed-material sediment and large woody debris through the Service Spillway (down river); and

- Improve the passage of large woody debris through the Diversion Inlet to reduce the risk of impacting the inlet hydraulics and gate operation.

In the design and testing process, priority was given to revisions that promoted the passage of debris through the Service Spillway and away from the Diversion Inlet. However, the results of the model testing indicate that during the evaluated diversion scenarios, flow from the main river channel (river left) was dominant to the Diversion Inlet.

8.1.4.3 Service Spillway Modifications

The following design revisions were incorporated into the Service Spillway following the conceptual design. The modifications were made based on observed performance during physical model testing.

- The structure was moved approximately 30 m upstream to reduce the distance between the crest gates and the Diversion Inlet. This change facilitated greater passage of debris down river and improved hydraulic conditions at the left approach wall.

- The right approach wall was revised to improve the hydraulics in the right gate bay. The curved approach wall from the conceptual design was replaced with a full-height semi-circular wall that reduced vertical mixing of flows and improved the transition of flows from in front of the Auxiliary Spillway into the right gate bay.

- The Sluiceway was removed from the design. The Sluiceway proved ineffective at transport of bed-material load during model tests. Further, the Sluiceway accumulated debris during some tested flows promoting jamming in front of the Diversion Inlet.

- The crest gates were widened to two 24 m gates. The increased gate bay width improved passage of debris through the Service Spillway.

- Finally, the operation of the crest gates was changed to an asymmetrical configuration, from the earlier constant gate height across bays. This approach results in the left gate bay operating at a lower level than the right bay. This improved passage of debris through the Service Spillway by concentrating flow and increasing flow depth such that debris is less prone to snagging on the gate leaf.

The revisions to the Service Spillway structure and operations improved debris passage for conditions when most of the river flow is passing down river through the Service Spillway. For flow rates with larger proportion to the Diversion Inlet, the proposed changes did not have a substantial effect. This performance can be improved by temporarily lowering the Service Spillway gates during the flood event to increase river flows through the spillway, and thus potentially moving debris through the structure. Once debris is moved, the gates can be raised and flood diversion continued.

8.1.4.4 Diversion Inlet Modifications

The following design revisions were incorporated into the Diversion Inlet following the conceptual design update. These modifications were made based on observed performance during physical model testing.

- The left approach wall was revised to improve the hydraulics in the left gate bay. The curved approach wall from the conceptual design was replaced with a full-height semi-circular wall that reduced vertical mixing of flows and improved the transition of flows from the main river channel.

- The four 10 m wide gate bays were replaced with two 20 m wide gate bays to improve the passage of debris through the structure. Due to the potential risks associated with debris affecting gate operations, it was recommended that the gate size be large enough to pass debris without consideration of a debris barrier. This size was determined during the physical model study.

- The gate type was changed from radial gates to vertical lift gates because of increased gate bay width.

- The pier nose between the gate bays was extended upstream approximately 8 m to increase the distance between the pier face and gate slots and improve debris passage. See Pier Nose 3 from the physical model study in Appendix C.

- The crest of the Diversion Inlet weir was changed from an ogee weir geometry to a broad crested weir geometry to improve upstream hydraulics.

The revisions to the Diversion Inlet substantially improved debris passage in comparison to the conceptual design. Where the initial design resulted in large debris jams forming on the face of the structure, risking interference with gate operations (closing), the revised design passed nearly all debris. Single debris elements that were caught on the pier face were generally dislodged and passed by subsequent debris. While the design includes a debris deflection system (Section 8.1.4.5), there remains residual risk that some debris will reach the Diversion Inlet gates. Given the adverse impact of debris accumulation at the gates, it was deemed appropriate that the gate width and inlet design facilitate passage of debris.

Figure 14 shows the recommended general arrangement following the physical modeling.

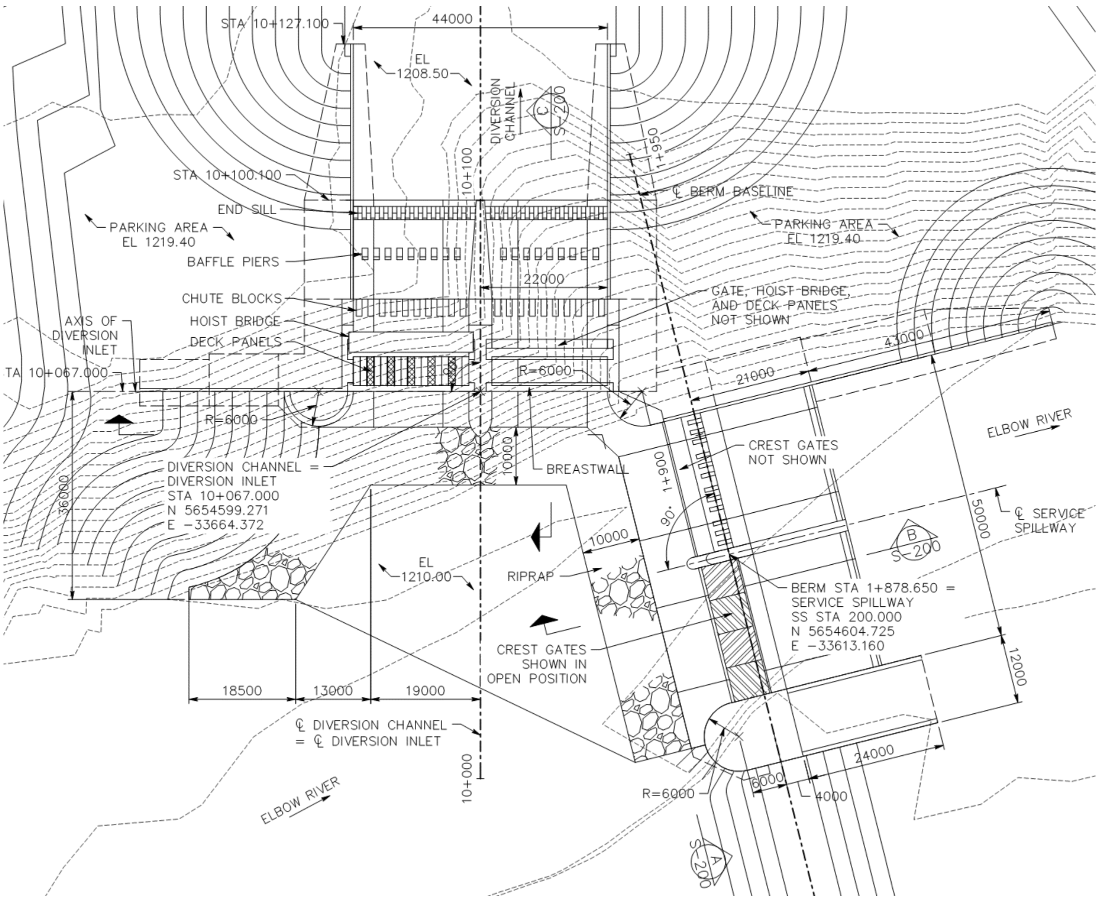


Figure 14. Physical Model Updates – Diversion Inlet and Service Spillway

8.1.5 Debris Management

8.1.5.1 Debris Management Alternatives

In addition to the design revisions discussed above, further debris management measures were evaluated for their suitability to the design. Effectiveness and performance were considered during normal operation (typical year, non-flood) and during flood operations. Three alternatives were considered including:

1. Base Condition (No Special Actions): This scenario is the base-level condition. It assumes typical yearly maintenance is performed at the structure to remove debris accumulation. Based on model testing, most debris is expected to pass through the Diversion Inlet and into the Diversion Channel during flood operations.

2. Debris Capture: This alternative seeks to collect large woody debris upstream of the Diversion Inlet using a series of vertical members spaced at even intervals. Multiple variations were evaluated in the physical model testing. Debris Barrier C provided the best debris capture performance while reducing the potential for a complete blockage of the Diversion Inlet opening. Debris Barrier C is shown in Figure 15 and discussed further in Appendix C.

Figure 15. Debris Barrier C

During a flood event, debris is expected to accumulate on the vertical members and form a jam. Some debris may pass through or around the structure, as occurred during testing. These debris elements are expected to then pass through the Diversion Inlet. As debris accumulates on the rack, flow through the barrier will diminish and it is expected that the flow pattern will shift to river right with debris potentially circumventing the right side of the rack. Based on model tests, operation of the gates (Diversion Inlet or Service Spillway) is unlikely to clear debris from the barrier. Following a flood event, significant debris removal from the river would be expected. During typical operations, debris removal from the rack is also anticipated and the presence of the racks within the river channel may pose a safety concern.

3. Debris Deflection: This alternative seeks to promote passage of debris downstream through the Service Spillway by constructing a structure comprised of horizontal members mounted to vertical supports. The barrier would be aligned generally parallel to the modeled flow vectors to facilitate the debris movement downstream. Multiple variations were evaluated in the physical model testing. Debris Barrier F provided the best debris deflection performance. Debris Barrier F is shown in Figure 16 and discussed in detail in Appendix C.

Figure 16. Debris Barrier F

During tests, Debris Barrier F retained modeled river debris with no model debris elements reaching the Diversion Inlet. Debris was observed to accumulate at the upstream end of the barrier between points A and B (see Figure 16). During even flow splits, debris from the secondary channel (river right) was observed to pass through the Service Spillway. During flow splits with the most flow towards the Diversion Inlet, debris would impinge on the barrier. Similar to the Debris Capture alternative, significant maintenance is expected to occur post-flood and position of the barrier within the river channel could present a public safety risk.

8.1.5.2 Debris Management Recommendations

The relative benefits and detriments of the three alternatives were reviewed with consideration to maintenance, operations and dam safety risks.

Based on the results of the alternatives evaluation, the Debris Deflection Barrier was selected for design advancement. Each of the alternatives evaluated have positive and negative impacts on project risks. However, it was decided that the Debris Deflection Barrier mitigated the risk of floating debris reaching the Diversion Inlet gates, while offering an opportunity to move debris downstream. Primary debris movement is facilitated through the alignment of the barrier parallel to river flow with a secondary option to temporarily lower the Service Spillway gates during a flood event to flush debris that accumulates on the barrier.

Design of the Debris Deflection Barrier is presented in Section 8.6.

8.1.6 Erosion Protection

Erosion risks were assessed for the Diversion Structure globally and at the interface of river flows and structure components. Design of erosion protection was provided to maintain system operations and structural stability.

There are several mechanisms and river processes in both flood and non-flood periods which have the potential to erode portions of the Elbow River and floodplain adjacent to the Diversion Structure. These mechanisms and the proposed mitigations are described below.

8.1.6.1 Main Channel Lateral Erosion

The main channel of the Elbow River moves laterally with typical bankfull processes of gravel and cobble bed rivers. Flood events can accelerate this gradual process. Drastic changes in meander bends have been observed in past flood events. It is assumed that this process has the potential to re-align the main channel anywhere within the Elbow River floodplain with the potential (rate) decreasing at terraces and other high ground in the floodplain.

Risk to the Diversion Structure from lateral erosion is considered low at this time. The outside meander bend on the descending right bank nearest the Floodplain Berm is approximately 150 m from the structure. The left bank adjacent to the Diversion Inlet and downstream of the Service Spillway is comprised primarily of bedrock and not susceptible to significant lateral movement.

8.1.6.2 Main Channel Scour

Scour potential within the main channel at the Diversion Inlet and the Service Spillway has been considered in the design. Results from the numerical hydraulic model of the Diversion Structure were used to assess net scour potential and perform riprap sizing calculations using methodology from USACE EM 1110-2-1601 (USACE, 1994). The controlling simulation selected for this assessment was the FoR, assuming no flood operations. This would result in an Elbow River discharge of 1240 m^3/s concentrated through the Service Spillway.

Using this basis, the net scour potential of an unprotected, representative section of the gravel and cobble bed materials is approximately 3.5 m as determined using both the Lacey and Blench computational methods. These calculations are presented in Appendix C. At the structures, this net scour potential is muted by the presence of the shallow bedrock in the area. Net scour potential of the bedrock downstream of the Service Spillway was assessed using the USBR and USACE stream power-erodibility index method. These calculations indicate that bedrock may scour down to Elevation 1207.0 m downstream of the Service Spillway, approximately 1.6 m below the top of stilling basin end sill. Calculations for this approach are presented in Appendix F. Cutoff keys for both the Service Spillway retaining walls and stilling basins are extended below this computed scour depth.

A riprap apron was designed along the upstream side of the Diversion Inlet and the Service Spillway to mitigate scour of backfill placed within the structure excavation. Proposed riprap designs are included on Drawings C-214 and C-215. Calculations are presented in Appendix F.

8.1.6.3 Channel Switch

During a flood, the Elbow River's channel processes are dominated by woody debris accumulation and sediment deposition. The subsequent erosion and avulsion can induce rapid channel planform changes and switches that can span between floodplain terraces. Such switches can occur multiple times during a single flood event. Post-flood evidence on site suggests such channel switches occurred downstream of the Diversion Structure location during the 2013 event.

A channel switch can be induced when flows overtop the banks in the upstream, from either clear-water hydraulics, or elevated water levels caused by debris jamming in the main channel. When that overland flow finds an easier, and sometimes shorter path through a low lying sub-channel and channel remnants within that floodplain, it can circumvent the main channel at a different hydraulic profile than that being experienced by the main channel. When that overland flow returns to the main channel, it generally does so at a higher elevation than the main channel and its return can induce head-cutting that progresses through the floodplain (sub-channel or channel remnant) from downstream to upstream. The extent of the head-cutting is dependent on the duration of overland flow and composition of the eroded material (vegetation, bedrock, etc.). As shear stresses from the overland flow increase, avulsions along the overland flow route can increase the flow through the sub-channel and can accelerate the channel switch process.

Figure 17 shows the terraces and sub-channels identified in the vicinity of the Diversion Structure. These sub-channels are the most likely path for a channel switch to take. The anticipated routes for channel switches within the Diversion Structure backwater is provided in red in Figure 20. The Floodplain Berm cuts off these routes and guides the overland flow to the Service Spillway and Diversion Inlet. This concentrating of flow along the toe could lead to erosion and head-cutting if not properly mitigated. The design includes features to mitigate the risk of this erosion potential head-cut.

The Floodplain Berm between approximate Station 1+075 and 1+630 has an earthen core, and the upstream face is armored with a riprap revetment featuring a self-launching apron to mitigate erosion should the channel switch up against the embankment toe. The self-launching apron was selected to reduce the excavation required to reach the required protection depth for scour. Details of the typical sections are shown on drawing C-270. The design translates the mid-channel velocities (approximately 3.5 m/s) of the Elbow River calculated in the 2D model and assumes their application in a switched channel adjacent to the Floodplain Berm. Associated calculations are presented in Appendix F.

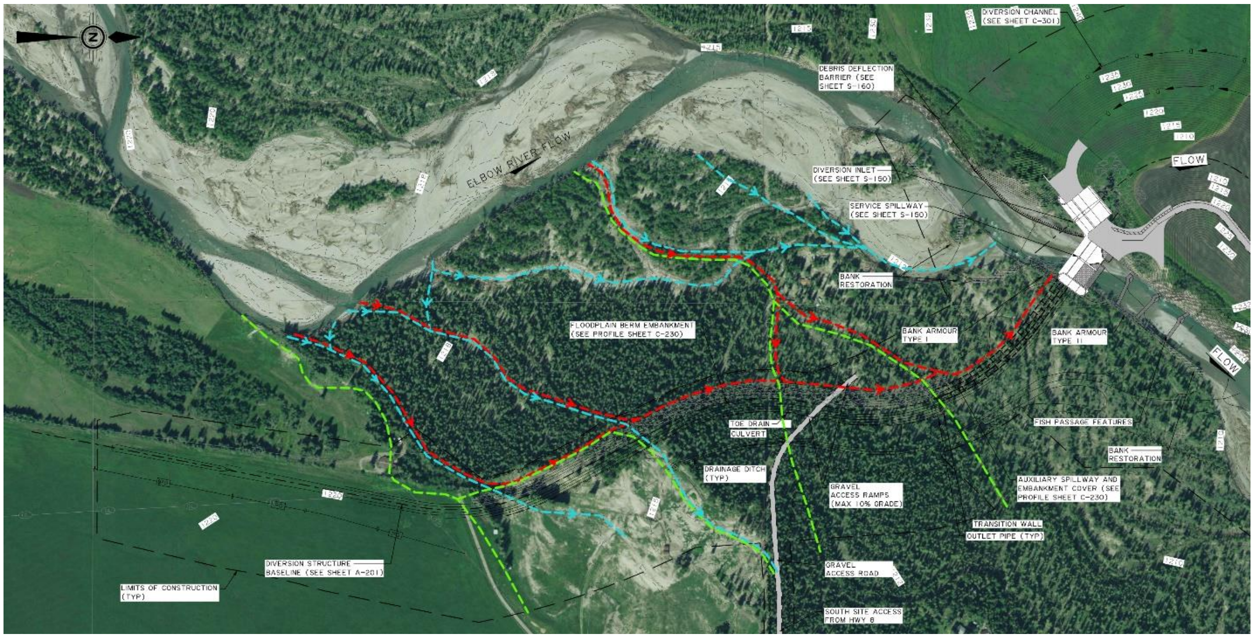


Figure 17. Floodplain Terraces in the Backwater and Potential Channel Switch Routes

The design includes additional protection against head-cut development at the location where flows against the Floodplain Berm return to the main channel. A large diameter stone sill is proposed in the right bank of the existing main channel, in the areas where the Auxiliary Spillway meets the Service Spillway. This is the location where a potential head-cut would likely initiate.

8.1.7 Fish Passage Provisions

To facilitate passage of fish through the Service Spillway, the drawings incorporate conceptual designs including grading and placed rock structures downstream of the Service Spillway gates and within the Elbow River. Analyses were undertaken to assess multiple factors that influence fish movement ability including local geomorphology, local fish species in the Elbow River, biologically sensitive periods (BSPs) for the identified fish species, Elbow River flow seasonality and annual variability, and Elbow River hydraulics including water depths and velocities. Results of these analyses are provided in a series of memoranda submitted as part of the Environmental Impact Assessment including: SR1: Fish Passage Flow Analysis (Stantec, 2016b); Fish Passage Mitigations at the SR1 Diversion Structure (Stantec, 2017b) and Hydraulic Modeling to Support Fish Passage Assessment (Stantec, 2017c).

Based on the analysis, the fish passage provisions are designed using downstream reference reaches to emulate existing downstream conditions and provide a minimum water depth of 18 cm for the Elbow River flow of 0.8 m^3/s.

The design mitigations consist of a series of three rock v-weirs downstream of the Service Spillway to stabilize the existing thalweg and limit step heights to a maximum of 20 cm. These weirs are an extension of riprap vanes that are intended to affix an existing gravel bar in place and maintain the existing river geometry under normal flow conditions. The downstream side of the v-weirs are lined with a cobble apron as protection against erosion and undermining, and to form plunge pools that act as a refuge for migrating fish. This design is hydraulically similar to the existing geometry and profile of the river with the same velocity and depth characteristics as the river upstream and downstream of the diversion structure, as demonstrated in the hydraulic modeling (Stantec, 2017c).

At the Service Spillway, a backwater is maintained by a riprap vane at the end sill of the stilling basin that constricts flow through a gap in the left bay. Boulders placed in this gap will further constrict flow, maintaining flow depths of at least 18 cm for the controlling flow scenario (0.8 m^3/s).

The rock weirs are designed to remain stable for flows up to the 1:100-year flood. During larger flood events, the fish features will likely erode with repair or replacement required after the flood.

The effect of the project on fish, including the proposed fish passage, are currently under review, as part of the Provincial and Federal review process. Comments or requirements from the regulatory bodies may result in changes to the design. Following concurrence from Canada Fisheries and Oceans and Provincial regulators on the fish mitigation measures, the conceptual design will be advanced to the Preliminary Engineering level and submitted under separate cover.

8.2 DIVERSION INLET

8.2.1 Arrangement

The Diversion Inlet is a gated concrete structure located on the left bank of the Elbow River at the entrance to the Diversion Channel. The primary elements of the Diversion Inlet include:

- Left abutment retaining walls and embankment transitions;
- Concrete monoliths with two 20 m wide gate bays with a center pier divider and fixed crest at Elevation 1211.5 m;
- Two 20 m wide by 4 m tall vertical lift gates with dual wire rope drums and hoist supported by a hoist bridge spanning the full 20 m bay width;
- Access bridge comprised of bridge deck, breastwall, and headwall which provides access to gate equipment, vehicle access across the Diversion Channel entrance, and serves as a debris and overtopping barrier during extreme flood events;
- Stilling basin concrete monoliths with chute blocks, baffle piers, and end sill to provide energy dissipation; and

- Right abutment retaining walls and embankment transitions.

Figure 18 shows an isometric view of the Diversion Inlet, a general arrangement is shown on Drawing S-150, and detailed drawings of the monoliths are depicted on Drawings S-300 to S-368.

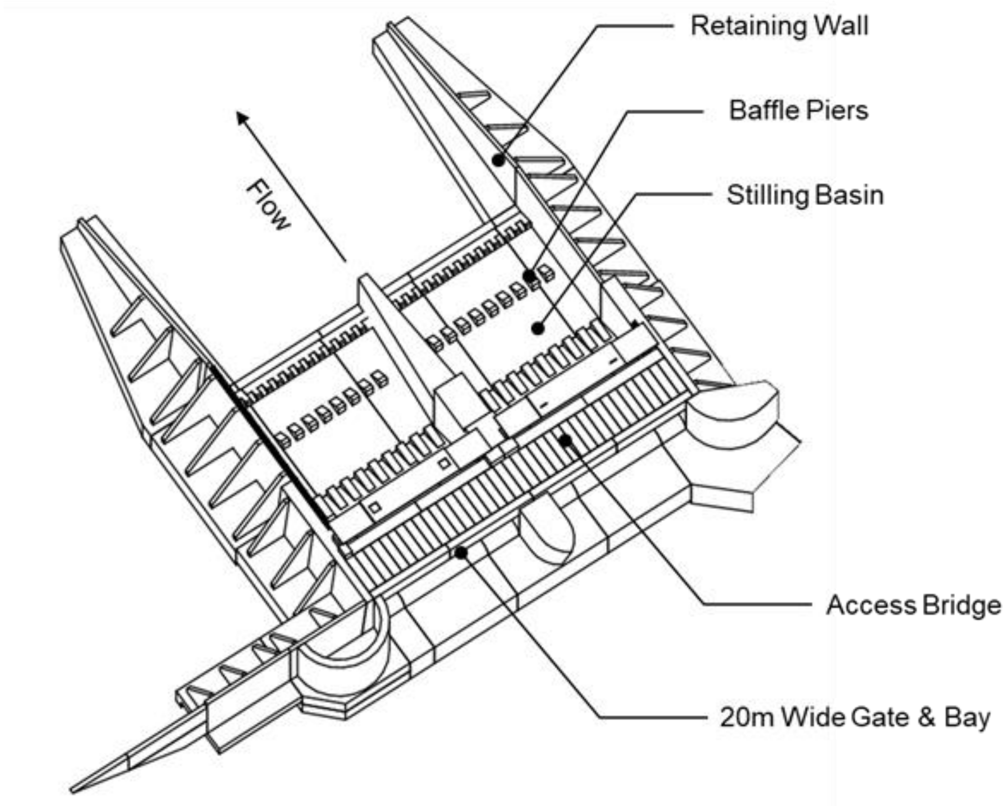


Figure 18. Diversion Inlet Structural Arrangement

8.2.2 Design Objectives

The primary objective of the Diversion Inlet is to prevent flow from entering the Diversion Channel except during flood events when the operation plan calls for diversion of flood flows. The fixed crest at Elevation 1211.5 m is designed to be above the approximate 1:2 year peak discharge water surface in the Elbow River and acts as a control weir during flood operation. The gates will normally remain in the closed position.

Since the Diversion Channel and Storage Reservoir have limited capacities, the Diversion Inlet must also be capable of restricting Diversion Channel inflow during extreme events including the IDF-OSSD, emergency conditions, and unplanned operations. To achieve this, the breastwall and headwall provide a permanent physical barrier from Elevation 1215.5 m to Elevation 1219.0 m to prevent overtopping when the gates are closed.

The Diversion Inlet abutment and training walls retain embankment fills, serve as a water barrier and prevent overtopping during flood events. For this reason, the walls were designed as concrete hydraulic structures to address stability, strength, and serviceability considerations for multiple operating conditions.

Gate operation requires hoist equipment to be positioned directly above the vertical lift gates with sufficient clearance to raise the bottom of gate above Elevation 1215.5 m. The gate hoist bridge serves as a mounting surface for the hoist equipment platform and grating. The hoist bridge is designed with sufficient stiffness to control deflection within the range of equipment tolerance and personnel comfort during hoist operations. The gate hoist bridge consists of steel plate girders spanning 20 m from pier to abutment with infill bracing to provide lateral stability and grating support. The girders have been sized for normal operating loads as well as hoist overload conditions with bottom of steel located at Elevation 1220.25 m to provide adequate clearance for the vertical lift gates in the fully raised position. Steel bar grating has been provided on the gate hoist bridge to facilitate pedestrian access and maintenance activities for hoist equipment.

To effectively operate and maintain the vertical lift gates and hoist equipment, vehicle and pedestrian access is required to the gates An access bridge spanning from breastwall to headwall consisting of precast deck panels has been included to provide a 6 m wide travel lane for vehicle access. The panels were designed for heavy equipment wheel loads, and the headwalls/breastwalls were designed to carry vehicle and crane outrigger loads anticipated during construction and maintenance activities. Steel bar grating was incorporated in the deck design to improve visibility to areas below the bridge deck, and individual panels will be removable (with appropriate equipment), if necessary, for maintenance activities.

8.2.3 Alternatives Considered

A brief description of alternatives considered for each of the primary elements of the Diversion Inlet is as follows.

8.2.3.1 Breastwall and Headwall Arrangement

Early concepts included taller gates without a breastwall/headwall to prevent overtopping during a PMF event. As described earlier, this wall was sized based on hydraulic criteria to prevent overtopping of the structure during extreme flood events. Initial revisions included a breastwall slightly downstream of the pier nose and a reduced height gate immediately downstream of the breastwall; however, physical model findings related to debris indicated that further separation was needed between the pier nose and breastwall. To address this need, the gate was moved downstream and a headwall was added to minimize the gate height and weight. The breastwall/headwall combination were designed to serve dual purposes of hydraulic control and vehicle access. The breastwall/headwall provides the structure necessary to span the 20 m bay width and reduce the size of vehicle access bridge components which can now span 4.5 m in the upstream-downstream direction.

8.2.3.2 Gate Type

Previous concepts included four 6 m high by 10 m wide radial gates. However, gate size changed when the gate bay width was increased from 10 m to 20 m in response to physical hydraulic model findings and debris considerations, and the gate height was reduced with the addition of a headwall. The combination of increased gate width and decreased gate height resulted in reevaluation of gate type selection. Vertical lift gates were selected over radial gates based on the evaluation of the following considerations:

- span-to-depth ratio;
- fabrication complexity;
- installation complexity;
- response to temperature fluctuation;
- reducing pier size; and
- ability to pass debris.

8.2.3.3 Access Bridge Deck

Precast concrete panels were incorporated into the access bridge design to simplify deck construction, facilitate removal if needed for maintenance activities, incorporate standardized Alberta Transportation components, and provide a durable low maintenance surface capable of spanning 4.5 m with heavy equipment wheel loading. Cast-in-place concrete was considered, but dismissed due to longer construction duration, higher installation cost associated with overhead formwork, and inability to remove/replace the bridge deck for maintenance. Stainless steel bar grating was considered to improve visibility to the gate approach under the access bridge and improve surface drainage. AT Bridges requires the use of stainless steel for vehicle deck grating. Consequently, it was not economical considering a 4.5 m span and heavy wheel loading. The additional framing between breastwall and headwall to reduce grating span would further increase cost and present a long-term maintenance and reliability concern. Short-span bar grating placed between precast panels was included in the design as shown on Drawing S-360 to retain some of the benefits without incurring cost or maintenance issues associated with long-span grating.

8.2.3.4 Gate Hoist Bridge

Three alternatives where considered to provide a gate hoist bridge across a gate bay including cast-in-place concrete girders, precast concrete girders, and steel plate girders. Utilizing the concrete headwall as one side of a hoist support system was considered since the headwall will be constructed to serve a hydraulic function. Although the headwall could support the upstream side of the gate hoist bridge, a second wall with comparable stiffness would be required to ensure uniform deflection below hoist equipment. This type of overhead cast-in-place construction is relatively expensive and cannot be easily removed to provide clearance for future gate removal and maintenance activities. A precast girder with prestressed/post-tensioned strands is potentially viable, but due to span and weight would face shipping limitations and require mobilization of specialty contractor(s) solely for installation of this component. Steel plate girders were selected for use because they provide a reliable support that is relatively low maintenance and comparable to the heavy steel fabrication and installation of the vertical lift gate and gate hoist platform shown. Steel girders can also be readily fabricated with provisions for field splices to meet standard shipping requirements.

8.2.4 Hydraulic Design

8.2.4.1 Physical Model Study and Numerical Modeling

The hydraulic performance of the Diversion Inlet design was evaluated using the physical hydraulic model which is discussed in Section 4.2. Abutment walls and pier shapes were adjusted in the physical model to better align flow through the structure and mitigate the occurrence of wakes due to the abutments and piers. The adjustments evaluated in the physical model were incorporated into the design.

8.2.4.2 Rating Curve

A headwater rating curve was developed for the Diversion Inlet with gates fully open. Headwater elevations ranging from 1211.5 m to 1217.0 m were calculated for a series of discharges and tailwater elevations based on methodology described in USACE EM 1110-2-1605 (USACE, 1987). The calculated rating curve is presented as Figure 19 with the related calculations presented in Appendix C.6-1. The design discharge capacity (600 m^3/s) was calculated to occur at a headwater elevation of 1215.8 m.

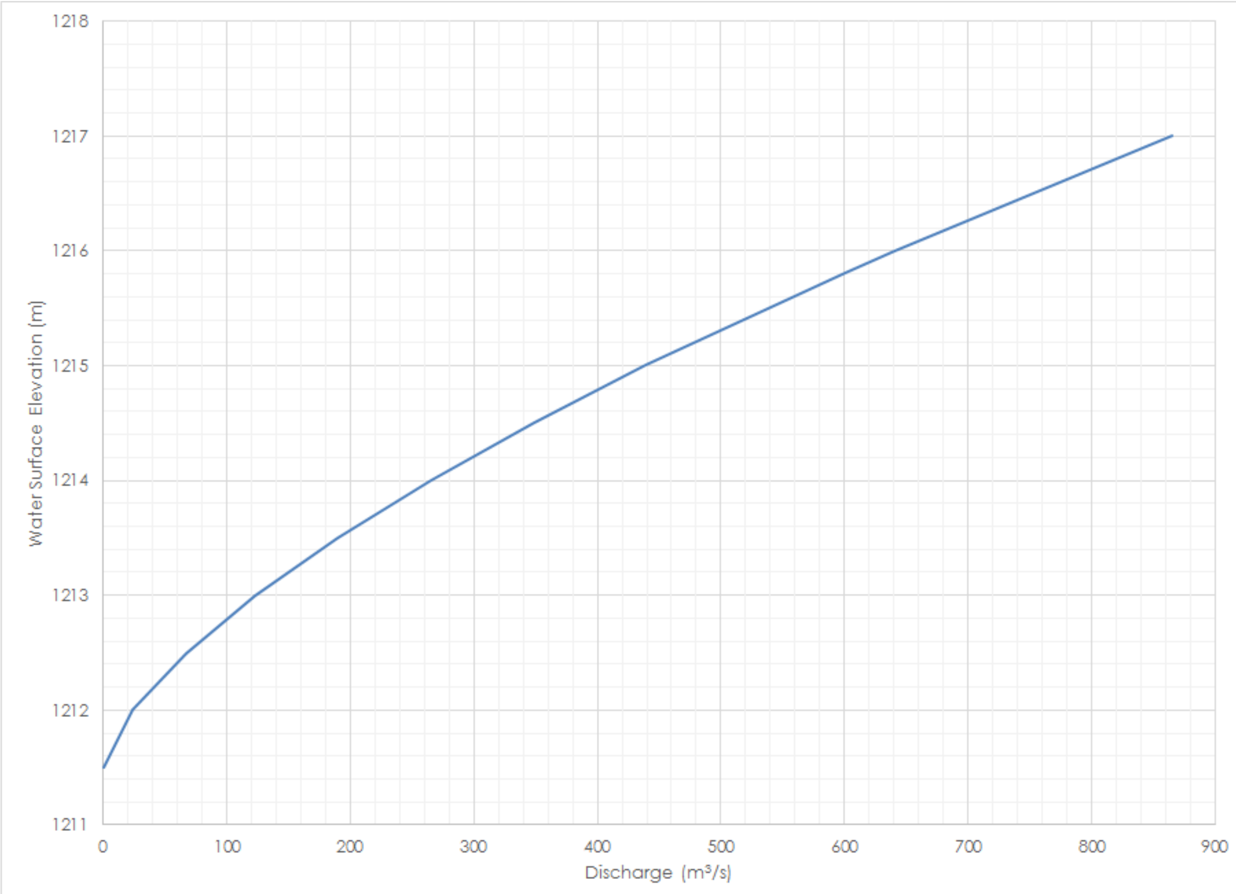


Figure 19. Diversion Inlet Rating Curve

8.2.4.3 Stilling Basin

The stilling basin for the Diversion Inlet was designed based on methodology described in USBR REC-ERC-78-8, Low Froude Number Stilling Basins, 2.5 to 4.5 (USBR, 1978) for discharges up to 870 m^3/s. Calculations are presented in Appendix C.6-2.

8.2.5 Geotechnical Considerations

8.2.5.1 Foundation

The Diversion Inlet structure will bear on bedrock, which consists primarily of shale, mudstone, claystone and sandstone. The upper portion of the bedrock is highly weathered and steeply dipped; therefore, the bearing bedrock surface will require preparation to remove any loose and/or weak material and attain a level surface. This will require placing a foundation concrete protection (mud-mat) to limit damaging the foundation after exposure and allow the construction of features such as foundation underdrains and shear keys required to achieve the stability formulated by the structural design. This structure requires a significant rock cut to reach the target elevation needed for hydraulic design, therefore selection of the concrete/rock interface elevation is controlled by stability and constructability factors more than bedrock profile.

The recommended parameters related to the allowable bedrock bearing capacity, drained cross bed shear strength and frost depth penetration, and the design basis used to derive these parameters are presented in Appendix D.

8.2.5.2 Seepage

Seepage is anticipated around and below the Diversion Inlet structure. The seepage patterns were modeled using the Geostudio SEEP/W software package and estimated bedrock permeability values. The estimated flow rate below and around the Diversion Structure is less than one liter per second. Upstream keys included in the design will control some of the seepage but will primarily function as erosion protection in case of rock scour or riprap movement along the leading edges.

Foundation pressure grouting is recommended to reduce uplift pressures and decrease the permeability of the Brazeau bedrock formation immediately below the Diversion Inlet to a uniform depth (25 m) within the bedrock. The foundation grouting design consists of a single row of pressure grouted rock core boreholes extended to a uniform depth within the bedrock, initially spaced approximately three metres apart along the upstream cutoff key of the primary Diversion Structure water control elements. Supplemental (secondary and tertiary) grouting boreholes may be added between borings where significant grout takes are observed in the primary grout holes.

8.2.5.3 Drainage

The retaining walls will be backfilled with soil material consisting of sandy lean clay with gravel (glacial till) obtained from the Diversion Channel excavation. Appendix D summarizes the recommended backfill soil parameters used in the design of the different walls. The backfill wall loading parameters were developed based on the laboratory testing performed as part of the geotechnical exploration.

The design for backfill of the retaining walls includes sub-drainage features to reduce and control hydrostatic pressures behind the walls. The recommended drainage system includes chimney and blanket drains, perforated pipes wrapped in filter fabric, collection pipes, and manholes.

8.2.6 Stability

Stability analyses were performed in accordance with the Structural Design Criteria outlined previously and included in Appendix E. The Diversion Inlet structures were evaluated for Usual, Unusual, and Extreme loading conditions under an Extreme hazard classification, which represents the potential range of conditions the structure will be exposed to during its design life. The loading conditions are described in Appendix E with a summary of stability results indicated in tables below.

8.2.6.1 Gate Structure

Stability analyses indicate a stable structure within the limits of acceptance criteria. For all loading conditions considered, floatation factors of safety were above required, 100 percent of the base was in compression, and sliding factors of safety were above required. Stability results indicated that sliding stability was the primary concern due to the low friction angle at concrete/rock interface and rock/rock bedding planes, but adequate factors of safety were achieved assuming an inclined failure extending from the upstream key to the downstream toe of the foundation. The controlling load cases were Load Case E4 (EDGM applied during 100-Year Flood), and Load Case UN2 (2013 Flood with No Diversion).

Table 14. Center Pier Monolith – Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases										
U1 Normal Operation *160 m³/s*	1212.1	1207.5	17900	1.5	3.0	1.5	37.9	155	160	100
U2 Diversion Operation *100 Yr. Flood*	1215.8	1213.1	25600	1.5	2.4	1.5	5.8	130	181	100
Unusual Load Cases										
UN1 Diversion Operation *2013 Flood*	1215.8	1213.1	19690	1.3	3.1	1.3	5.5	130	181	100
UN2 No Diversion *2013 Flood*	1216.2	1207.5	29518	1.3	2.0	1.3	3.4	135	143	100
UN3 No Diversion *1000 Yr. Flood*	1217.0	1207.5	25461	1.3	2.4	1.3	3.2	125	186	100
UN4 Construction/ Maintenance *100 Yr. Flood*	1215.8	1211.0	25585	1.3	2.3	1.3	8.2	169	136	100
Extreme – Flood										
E1 PMF without Diversion	1217.8	1207.5	26696	1.1	2.3	1.1	2.6	126	197	100
E2 PMF with Diversion	1216.6	1214.4	27560	1.1	2.2	1.1	4.6	131	184	100
Extreme – Earthquake used to determine Post-Seismic Condition										
E3 EDGM applied to U1	1212.1	1207.5	17900	1.1	2.7 (E3.3)	1.0	1.9 (E3.2)	90 (E3.2)	229 (E3.1)	100
E4 EDGM applied to U2	1215.8	1213.1	25600	1.1	2.2 (E4.3)	1.0	1.4 (E4.2)	61 (E4.2)	254 (E4.1)	100

Notes:
1. See Appendix E.1 for definition of monolith description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

Table 15. Gate Crest Monolith – Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Hydro-static Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (kPa)	
Usual Load Cases										
U1 Normal Operation	1212.1	1207.5	16523	1.5	1.7	1.5	6.1	93	72	100
U2 Diversion Operation *100 Yr. Flood*	1215.8	1213.1	23617	1.5	1.5	1.5	2.7	88	85	100
Unusual Load Cases										
UN1 Diversion Operation *2013 Flood*	1215.8	1213.1	23617	1.3	1.5	1.3	2.7	88	85	100
UN2 No Diversion *2013 Flood*	1216.2	1207.5	27247	1.3	1.3	1.3	1.7	95	45	100
UN3 No Diversion *1000 Yr. Flood*	1217.0	1207.5	23502	1.3	1.5	1.3	2.1	96	79	100
UN4 Construction/ Maintenance	1215.8	1211.0	23617	1.3	1.5	1.3	2.5	81	84	100
Extreme – Flood										
E1 PMF without Diversion	1217.8	1207.5	24642	1.1	1.5	1.1	1.9	92	80	100
E2 PMF with Diversion	1216.6	1214.1	25440	1.1	1.5	1.1	2.5	85	86	100
Extreme – Earthquake used to determine Post-Seismic Condition										
E3 EDGM applied to U1	1212.1	1207.5	16523	1.1	1.5 (E3.3)	1.0	1.33 (E3.2)	65 (E3.2)	103 (E3.1)	100
E4 EDGM applied to U2	1215.8	1213.1	23617	1.1	1.4 (E4.3)	1.0	1.01 (E4.2)	55 (E4.2)	120 (E4.1)	100

Notes:
1. See Appendix E.1 for definition of monolith description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

8.2.6.2 Stilling Basin

Stability analysis results indicate a need for anchorage of the stilling basin into the foundation bedrock. The required anchor force is within the capability of conventional passive ground anchors. The results of the stilling basin floatation analysis are presented in Table 16 below. The controlling Load Case for the stilling basin anchors is F1 (Usual - Diversion Flow).

Table 16. Stilling Basin – Floatation Analysis Summary

Load Case	Headwater/ Tailwater Elevation (m)	Vertical Force Down (kN)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Anchor Force Required
				Required	Calculated	
F1 Unusual Diversion Flow	1215.8 / 1213.1	18322	17534	1.50	1.04	36.9
F2 Unusual Const./Dewatered	1215.8 / 1211.0	10152	11654	1.30	0.87 < 1	23.1
F3 Extreme Ineffective Drain	1215.8 / 1207.5	10152	7989	1.10	1.27	0

8.2.6.3 Retaining Walls

The abutments, approach walls, and training walls are concrete gravity structures designed as either counterfort or cantilever retaining walls depending on wall height. In general, walls with stem heights more than 6.5 m require counterforts to provide adequate stiffness and lateral load path. Four representative sections were identified to capture the range of retaining wall geometry and loading conditions for the overall structure. These representative sections are indicated on Figure 20 and described as follows.

- Section DI-6: Counterfort wall serving as part of left abutment. This section was selected to determine point at which counterforts are required due to increasing differential fill heights.

- Section DI-5B (DI-1B similar, opposite): Counterfort wall serving as the part of the Diversion Inlet Gate Structure training wall. Compared to other wall sections, this section has a thicker stem to carry access bridge and gate loads, reduce deflection, and thicker footing to match the gate bay concrete profile.

- Section DI-5C (DI-1C similar, opposite): Counterfort wall integral with Stilling Basin. This is one of the tallest wall sections and is subjected to potential unbalanced water load when Diversion Inlet gates are closed and diversion flow is terminated.

- Section DI-5D (DI-1D similar, opposite): Counterfort or cantilever wall serving as a downstream training wall and slope protection. This section was selected to determine wall height where counterforts were no longer required. Section DI-5D was analyzed at 3 sections to account for the geometric variability of the monolith.

Section DI-7 shown in Figure 20 is a concrete core wall surrounded and supported by embankment fill. Structural stability analysis of this wall was not needed due to the embankment support.

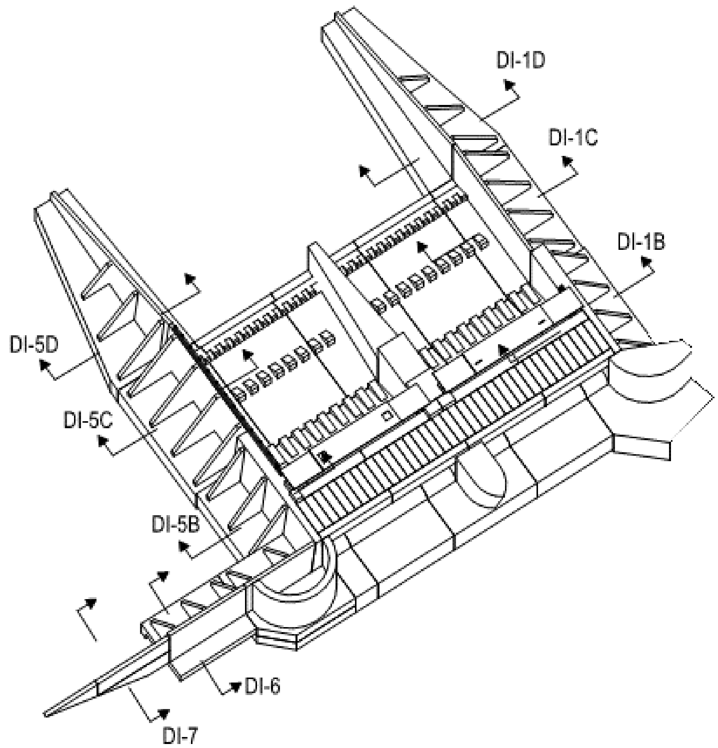


Figure 20. Diversion Inlet Retaining Wall Key Plan

Stability analyses were performed in accordance with the Structural Design Criteria outlined previously and included in Appendix E. Each of the four representative wall sections was evaluated for Usual, Unusual, and Extreme loading conditions representing the potential range of conditions the structure will be exposed to during its design life. The loading conditions are described in detail in Appendix E with a summary of stability results indicated in Table 17 below. In accordance with guidelines for hydraulic structures, at-rest soil pressures were used for all load case calculations except active soil pressures were used when considering seismic load cases.

The principal factors affecting the structural design of the walls include significant driving force associated with high groundwater conditions; poor rock quality along the foundation interface; relatively weak material (glacial till) anticipated in the backfill zone of influence; and potential for significant uplift pressure when water levels recede faster than pore pressure can dissipate. Design calculations indicate that retaining walls are most sensitive to groundwater conditions, concrete shear capacity of stem walls, and sliding stability provided by foundation shear keys.

For all loading conditions considered, floatation factors of safety are above required, 100 percent of the base is in compression, and sliding factors of safety are above required. Stability results indicate that sliding stability is the primary concern due to the low friction angle at the concrete/rock interface and rock/rock bedding planes. To achieve stability results within the limits of acceptance criteria, a shear key at the heel of footing, and a wall drain system are required. The structural shear key ensures an inclined base sliding analysis is valid, and the wall drain system significantly reduces load associated with groundwater. The controlling load case is Load Case UN2 (high groundwater due to ineffective drain).

Table 17. Retaining Walls – Stability Analysis Summary

Load Case	Headwater (Heel) Elevation (m)	Tailwater (Toe) Elevation For Uplift (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (kPa)	
WALL BLOCK DI-6										
U1 Normal Operation	1213.1	1212.1	494	1.5	3.38	1.5	2.24	76	281	100
UN1 Equip. Surcharge	1213.1	1212.1	494	1.3	3.54	1.3	1.95	58	322	100
UN2 Ineffective Drain	1216.2	1216.2	811	1.3	2.1	1.3	1.34	26	279	100
E1 Seismic	1213.1	1212.1	494	1.1	3.05	1.0	1.78	48	276	100
WALL BLOCK DI-1B/DI-5B										
U1 Normal Operation	1212.0	1210.0	645	1.5	3.99	1.5	2.40	153	228	100
UN1 Equip. Surcharge	1212.0	1210.0	645	1.3	4.12	1.3	2.03	143	256	100
UN2 Ineffective Drain	1215.8	1215.8	1212	1.3	2.16	1.3	1.30	98	203	100
E1 Seismic	1212.0	1210.0	645	1.1	3.60	1.0	1.69	108	236	100
WALL BLOCK DI-1C/DI-5C										
U1 Normal Operation	1210.5	1207.5	529	1.5	5.00	1.5	2.34	151	263	100
UN1 Equip. Surcharge	1210.5	1207.5	529	1.3	5.21	1.3	2.02	137	298	100
UN2 Ineffective Drain	1214.5	1214.5	1171	1.3	2.31	1.3	1.30	97	226	100
E1 Seismic	1210.5	1207.5	529	1.1	4.52	1.0	1.96	123	250	100
WALL BLOCK DI-1D/DI-5D (Upstream)										
U1 Normal Operation	1210.0	1207.5	733	1.5	4.25	1.5	2.21	148	346	100
UN1 Equip. Surcharge	1210.0	1207.5	733	1.3	4.41	1.3	2.02	135	382	100
UN2 Ineffective Drain	1213.5	1213.5	1338	1.3	2.37	1.3	1.31	103	322	100
E1 Seismic	1210.0	1207.5	733	1.1	3.84	1.0	1.85	113	338	100

Table 17. Retaining Walls – Stability Analysis Summary (Continued)

Load Case	Headwater (Heel) Elevation (m)	Tailwater (Toe) Elevation For Uplift (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
WALL BLOCK DI-1D/DI-5D (Mid-section)										
U1 Normal Operation	1210.0	1207.5	507	1.5	2.97	1.5	4.09	92	239	100
UN1 Equip. Surcharge	1210.0	1207.5	507	1.3	3.12	1.3	3.26	81	273	100
UN2 Ineffective Drain	1213.5	1213.5	926	1.3	1.67	1.3	1.30	31	244	100
E1 Seismic	1210.0	1207.5	507	1.1	2.68	1.0	2.53	57	250	100
WALL BLOCK DI-1D/DI-5D (Downstream)										
U1 Normal Operation	1209.5	1207.5	265	1.5	2.17	1.5	6.92	53	96	100
UN1 Equip. Surcharge	1209.5	1207.5	265	1.3	2.35	1.3	3.93	54	115	100
UN2 Ineffective Drain	1209.5	1209.5	323	1.3	1.77	1.3	3.60	53	79	100
E1 Seismic	1209.5	1207.5	265	1.1	1.96	1.0	1.88	30	110	100

Notes:
1. See Appendix E.1 for definition of wall section description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Seismic results utilize active soil pressure coefficients for stability values reported.

8.2.7 Strength

The Diversion Inlet is designed as a mass concrete gravity structure sized primarily for stability. Most elements exceed 2 m in thickness and are surface reinforced for crack control and durability rather than strength. Each element is checked to confirm calculated stress from factored loads do not exceed member capacity. Some elements which are subjected to higher stress and controlled by strength design include:

- Headwall/breastwall which is designed as a beam spanning 20 m from abutment to pier and subjected primarily to dead load, vehicle load, and lateral hydraulic load;
- Access deck panels which span 4.5 m from breastwall to headwall and subjected primarily to vehicle and equipment loading;
- Gate hoist bridge spanning 20 m and subjected primarily to dead load and hoist load; and
- Concrete gate slots, gate hoist bridge end supports, and headwall/breastwall end supports which are subjected to concentrated bearing forces.

For each of these elements, preliminary strength calculations were performed to acquire order-of-magnitude stress and establish basis for preliminary member sizing. Strength calculations to develop reinforcement sizing and steel detailing will be performed during Final Design.

The retaining wall monoliths will be detailed during Final Design using commercially available finite element software with beam, shell, and solid elements where appropriate.

Footings were designed as a structural slab on an elastic foundation as the stability analysis concluded that the foundations are in compression based on the value of the subgrade modulus. The critical sections considered for evaluation of shear and moment are at half the footing thickness as measured from the face of the wall for the toe and at the face of the wall for the heel. In general, footing geometry was dictated by the gate bay limit of excavation, and desired hydraulic profile resulting in footing thicknesses exceeding 1.5 to 2.0 m with relatively low stress at the critical sections.

Cantilever stem walls were designed as a cantilever beam fixed at the footing interface. The critical sections considered for evaluation were at the base of stem, 1/3 of the stem height, and 2/3 of the stem height. Wall thickness increases from top to bottom with thickness ranging from 0.5 to 2.0 m, respectively. Due to increased thickness and increased load near the base of walls, shear strength becomes a controlling factor, and transverse shear reinforcement (cross ties) will be required.

Counterfort stem walls were designed as continuous beams spanning horizontally between counterforts, with only the lower portions of the stem exhibiting plate (2-way spanning) action and designed as a cantilever from the footing to a height approximately half of the counterfort spacing.

Counterfort heels were designed with a similar load path as the stem. The portion of footing closest to the stem acts as a cantilever beam, and the portion which is further from the stem by more than half of counterfort spacing, is designed as a continuous beam spanning between counterforts.

Counterforts were designed as cantilever deep beams fixed at the footing interface. The wall serves as the beam flange, and the flange width is calculated as the lesser of 12 times the thickness of the wall or half the distance between the counterforts using equation 10.3.3 of CSA 23.3 (2018). The counterfort is considered to act as the stem of tee beam and is fixed at its base. The tee beam is sized so that the neutral axis of the tee beam is located within the flange. The depth of the tee beam is the perpendicular distance between the sloping face of the counterfort and the vertical face of the retaining wall. Critical sections for evaluation of counterfort shear and moments include the foundation interface and the third points of the counterfort.

8.2.8 Serviceability

Serviceability concerns with the Diversion Inlet relate primarily to concrete durability, shrinkage control, relief of internal stresses associated with volume changes, and deflection of access deck and gate hoist bridge.

Shrinkage control and volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The preliminary design includes joint locations that define monoliths with balanced aspect ratios and placements less than 12 to 18 m in any one direction. Expanded guidance related to placement sequence and horizontal joint locations will be addressed during Final Design.

Allowance for thermal expansion/contraction is critical for gate operation. These affects are addressed primarily through second stage concrete placements occurring after initial concrete shrinkage has occurred and detailing of clearance within the gate slot.

Gate hoist bridge stiffness and member size are controlled by deflection limits established for hoist equipment. Hoist bridge deflection must be uniform to reduce secondary loads on shaft and gears, and within limits of comfort of personnel working on the gate hoist platform. Lateral deflection of the gate itself and headwall must be controlled to maintain integrity and contact of gate seals.

Serviceability criteria for the bridge deck will be in accordance with AT Bridge Standards (AT 2011a and 2011b) and sized for heavy equipment loading.

Serviceability concerns for the retaining walls relate to concrete durability, shrinkage, crack control, volume changes and wall deflections. Durability, shrinkage, and crack control are achieved primarily through reinforcement placement, high reinforcement ratios, and use of high load factors that account for both strength and serviceability in accordance with the CSA SEED (CSA, 2018) document. Volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The retaining walls include vertical joints at locations of footing geometry change, and at locations needed to maintain horizontal wall lengths less than 12 to 18 m. Expanded guidance related to placement sequence and horizontal joint locations will be addressed as part of constructability review during Final Design.

Wall deflections are controlled using counterforts to provide rigidity, by reducing wall and footing spans, and using at-rest soil pressure when sizing wall elements. Locations where wall deflection is critical includes walls serving as gate bay abutments, walls adjacent to access roads and control building foundation, and walls along the upstream face which must maintain tight joints for water retention. Wall deflections will be addressed during Final Design.

8.2.9 Construction Considerations

Construction specifications and details for the Diversion Inlet will be furthered during Final Design. The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the Diversion Inlet. The services of a specialist dewatering contractor may be needed.
- Excavation will be in competent bedrock. All soil, including alluvium, talus and other unconsolidated deposits should be removed to expose unweathered or slightly weathered bedrock. Excavation should be performed by mechanical means only; blasting will not be permitted.
- Foundation preparation will require special care in cleaning and preparation of concrete/rock interface. Care must be taken during excavation of the foundation to identify unsuitable rock conditions or weak bedding planes that could impact stability. Loose material and rock overhangs will need to be removed. Small voids will be filled with dental concrete. Once ready, foundation protection will be placed over exposed rock.
- If extensive jointing/fracturing is observed after excavation of the foundation, consolidation grouting may be required.
- Shear keys are required to maintain adequate sliding stability for gate monoliths and retaining walls. Care should be taken during excavation of the shear key trenches to identify unsuitable rock conditions or weak bedding planes that could compromise capacity of the shear key.
- Anchors, along with a foundation underdrain to relieve uplift pressures, will be required to maintain adequate factors of safety against floatation in the stilling basin. Static anchors drilled and grouted in a grid pattern prior to placement of the stilling basin concrete are proposed.
- Lift joints in the base mats and footings will be required to reduce placement thickness, control heat of hydration, reduce crack potential, and develop hydraulic profile. Changes in mix design will be required to provide lower cement ratio and larger aggregate in mass concrete placements, with higher strength and smaller aggregate mix placed as part of the reinforced "surface skin".
- Vertical joints in gate bays and stilling basins will be spaced and detailed so that "closure grouting" needed to accommodate shrinkage during initial curing is not needed.
- Horizontal joints in the retaining wall stems will be required to reduce placement height to minimize aggregate separation, improve access for adequate vibration, reduce potential for form bulging, and allow for fill placement to progress in stages with wall construction.

- Joint preparation will require attention to proper installation of water stops, shear keys, dowels, and reinforcement. Joint alignment and water-tight integrity are critical for reducing water levels on the back side of retaining walls.

- Gate slots, access bridge, and gate hoist beams at the Diversion Inlet will require combinations of concrete block outs, anchor bolts, and embedded parts in first and second stage concrete placements. Placement tolerances for some of these items are tighter than typical heavy construction tolerances due to fit and clearance requirements.

- Procurement lead-time for gate components will likely be driven by steel availability and fabrication schedules. An allowance of 12 to 18 months is recommended to account for design, shop drawing review/approval, fabrication, testing, and delivery.

- Placement of free draining backfill, filter material, and drain systems are critical for groundwater level control behind the walls. Material selection and installation methods will require appropriate quality control and monitoring.

- Fill placement and compaction methods must be reviewed and monitored to ensure wall movement does not occur during construction.

- Construction sequencing will be required to ensure the Diversion Inlet and gates are fully functional before a tie-in with the Diversion Channel is made.

8.3 SERVICE SPILLWAY

8.3.1 Arrangement

The Service Spillway is a gated concrete structure located on the main channel of the Elbow River serving as the regulating feature for river and diversion flow. The primary elements of the Service Spillway include:

- Left abutment retaining walls and embankment transitions;

- Concrete monoliths with two 24 m wide gate bays with a center pier divider and fixed crest at Elevation 1210.0 m;

- Two 24 m wide by 5 m high bottom hinged crest gates with pneumatic bladders for control;

- Stilling basin concrete monoliths with an end sill to provide energy dissipation and reduce channel erosion during gate operations; and

- Right abutment monoliths and training walls.

Figure 21 shows an isometric view of the Service Spillway, a general arrangement is shown on Drawing S-150, and detailed drawings of the monoliths are depicted on Drawings S-200 to S-245.

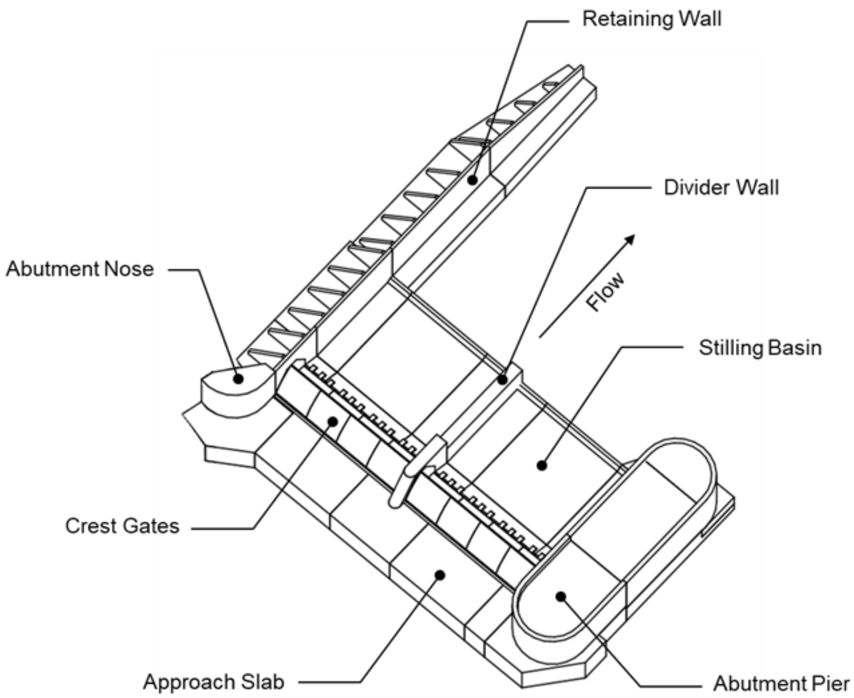


Figure 21. Service Spillway Structural Arrangement

8.3.2 Design Objectives

The primary objective of the Service Spillway is to regulate river flow and headpond elevation during flood events. Since the Diversion Inlet gates are not intended for operational control, the headpond water surface elevation immediately upstream of the Diversion Structure directly corresponds to the diversion rate. The crest gates are normally in the open (lowered) position at Elevation 1210.0 m to allow "free flow" of the Elbow River. When the operation plan calls for diversion of flood flows, the crest gates are raised to desired position to retain water and control either headpond elevation or discharge flow depending on the operation plan for a given flood scenario.

The Service Spillway left abutment serves as a retaining wall for the left embankment, primary site access road, control building, parking, and work area. The right abutment provides a physical separation between Service Spillway flow and Auxiliary Spillway flow during discharge of extreme floods and acts as a training wall for the Service Spillway stilling basin.

Since the Service Spillway is the primary water conveyance and regulating structure, steps were taken to reduce the potential for debris hang-ups and flow obstruction. For this reason, there is vehicle and pedestrian access to the left abutment, but not across the Service Spillway. Although there is no river crossing at the site, the right abutment is accessible via an access road upstream of the Auxiliary Spillway under typical river conditions.

The retaining walls support embankment fill, serve as water barriers to contain the Elbow River, and prevent overtopping during flood events. For this reason, the walls were designed as concrete hydraulic structures to address stability, strength, and serviceability considerations for multiple operating conditions.

8.3.3 Alternatives Considered

A brief description of alternatives considered for each of the primary elements is as follows.

8.3.3.1 Gate Type

Selection of gate type and size drives the Service Spillway design. Multiple gate types and configurations were considered including bottom-discharge radial gates and overflow crest gates. The bottom discharge radial gates were eliminated from consideration due to their susceptibility to debris impacts and limited hydraulic control at low flow openings.

Two types of bottom hinged crest gates were considered: steel crest gates with hydraulic operators and steel crest gates with pneumatically actuated bladders (manufactured by Obermeyer Hydro Inc.). The two crest gate types provide similar performance metrics and can accommodate the design spans without obstructions. The hydraulically operated gates are more susceptible to sediment and debris impacts during operations. The underside of the gate may accumulate sediment and the exposed actuators could be obstructed or damaged by debris. By contrast, the pneumatically operated bladder takes up the space on the underside of the gate mitigating sediment and debris impacts. The disadvantage of the pneumatically operated gates is the use of a proprietary system and reliance on a single supplier.

Ultimately, the pneumatically operated gate was selected because it provides equal operational control, the least obstructive profile, and highest operational reliability from impacts associated with sediment and debris. The gate size shown in the design conforms to the manufacturer's standard size leaf panels, is within the range of previously installed gates, and provides a symmetric gate system to improve operational reliability.

8.3.3.2 Abutment Arrangement

Hydraulic design considerations and findings of the physical model testing informed the abutment arrangement and profile. Iterations leading to arrangement shown on Drawing S-150 included straight abutment walls, elongated approach channel, angled approach walls, lateral wall, and curved walls of various heights. Ultimately physical hydraulic modeling dictated the geometry and arrangement of abutments and walls. Further information regarding the physical modeling is provided in Appendix C.

8.3.4 Hydraulic Design

8.3.4.1 Physical Model Study and Numerical Modeling

The hydraulic performance of the Service Spillway design was evaluated using the physical hydraulic model. Abutment walls and pier shapes were adjusted in the physical model to better align flow through the structure and mitigate the occurrence of wakes due to the abutments and piers. The adjustments evaluated in the physical model were incorporated into the design.

Further refinements made to the Service Spillway design after the conclusion of the Physical Model study were evaluated using the numerical model discussed in Section 4.0.

8.3.4.2 Rating Curves

A series of headwater rating curves were developed for the Service Spillway using results from the physical hydraulic model. The Service Spillway crest gates in the physical model were set at five elevations between 1211.05 m and 1215.00 m and the model was run with up to five discharges for each gate setting to develop a series of rating curves. The headwater rating curves were developed based on measurements taken from the physical model. A rating curve with the gates in the fully lowered position (1210.00 m) was developed based on results of the numerical model discussed in Section 4.0. The rating curves are presented in Figure 22. Development of the rating curves is discussed in Appendix C.6-3.

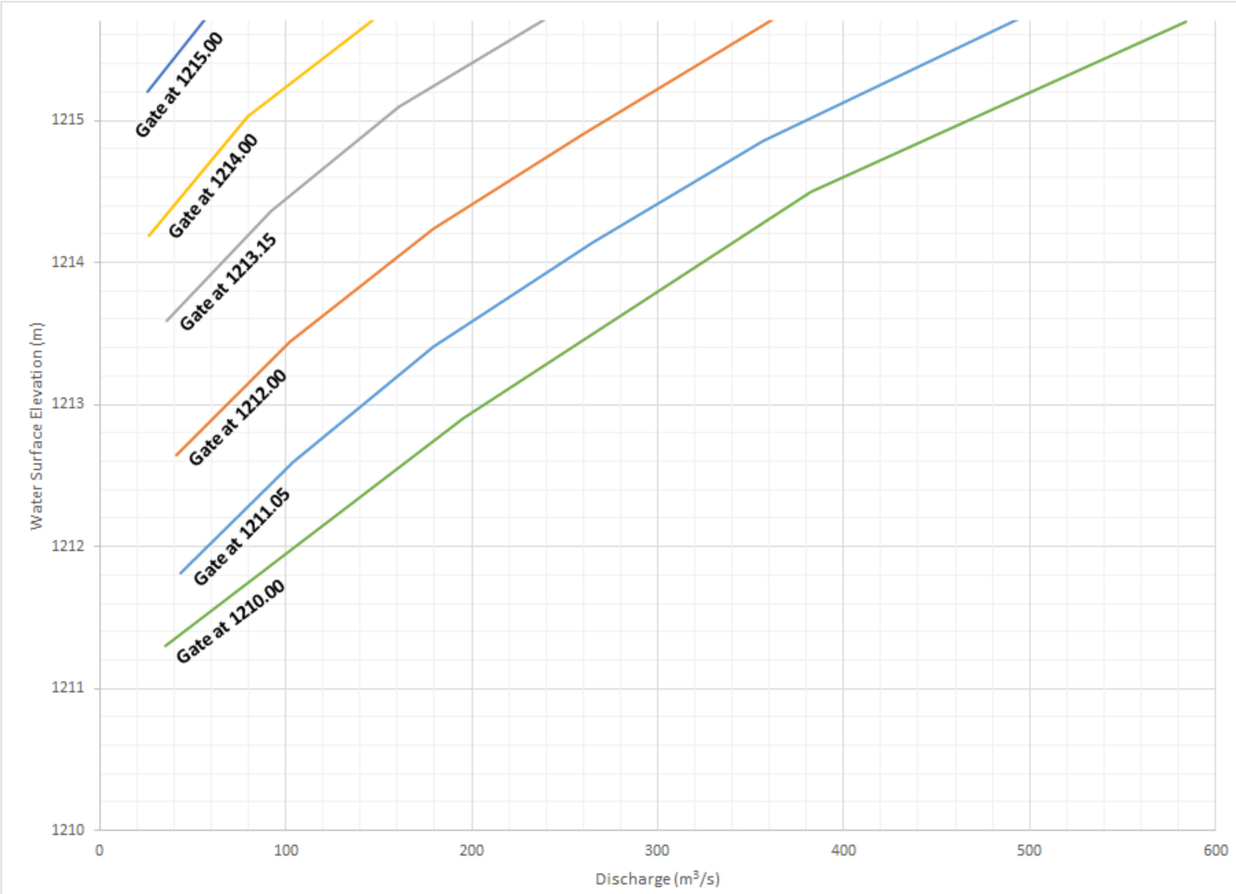


Figure 22. Service Spillway Discharge Rating Curves (Single 24 m Gate Bay)

8.3.4.3 Stilling Basin

The Service Spillway operates as a low-head hydraulic structure with minimal drop from the crest to downstream channel. With the crest gates in the open position, this design facilitates fish passage and reduces risks to navigation. Under flood operations and the gates in the raised position, the Froude Number entering the basin ranges between 3.5 and 5.4 and the stilling basin provides for energy dissipation and formation of a hydraulic jump. As flows within the river increase and the gates are lowered, the Froude Number drops below 3 and the stilling basin has a reduced effect on discharge hydraulics and the flow characteristics match the upstream and downstream river channel.

Based on these hydraulic conditions, the stilling basin for the Service Spillway was designed using the methodology described in USACE EM 1110-2-1605 (USACE, 1987) for the range of discharges up to the FoR under various gate operation scenarios. The standard design presented by USACE includes both baffle blocks within the basin and an end sill but describes scenarios where the baffle blocks may be eliminated if there are concerns related to abrasion from bedload transport and risks to navigation. As the Service Spillway's position in the Elbow River presents both of these conditions, the baffle blocks were removed from the design and the basin depth increased, so that the tailwater to conjugate depth ratio approached 1 for the gate operating conditions.

The expected basin performance was then confirmed in the Physical Model (See Appendix C.4) under the range of operation scenarios. Hydraulic calculations are presented in Appendix C.6-4.

For discharges through the Service Spillway that exceed the design operating condition, the potential for scour was assessed in the downstream river channel for flows up to 1240 m^3/s through the Service Spillway which is the point of activation of the Auxiliary Spillway. Further description of the scour analysis is provided in Section 8.1.6 with calculations presented in Appendix F.1-2. Structure foundations for the Service Spillway stilling basin and retaining walls were extended below the calculated ultimate scour depths.

8.3.5 Geotechnical Considerations

8.3.5.1 Foundation

The Service Spillway structure will bear on the same bedrock formation encountered under the Diversion Inlet structure. Therefore, the recommendations discussed above in Section 8.2.5.1 for the Diversion Inlet structure also apply for the design and construction of the Service Spillway. Unlike the Diversion Inlet which requires a deep rock excavation, the Service Spillway structure is within the overburden and highly weathered surface layer and requires excavation somewhere between Elevation 1206.5 and 1207 m to expose rock suitable for foundation construction.

The recommended parameters related to the allowable bedrock bearing capacity, drained cross bed shear strength and frost depth penetration, and the design basis used to derive these parameters are presented in Appendix D.

8.3.5.2 Seepage

Seepage is anticipated around and below the Service Spillway structure as in the case of the Diversion Inlet. Therefore, the recommendations discussed above in Section 8.2.5.2 for the Diversion Inlet structure also apply for the design and construction of the Service Spillway.

Foundation pressure grouting to reduce uplift pressures and the permeability of the Brazeau bedrock formation, as recommended above in Section 8.2.5.2, also applies to the subgrade below the Service Spillway to a uniform depth (25 m) within the bedrock.. The foundation grouting design consists of a single row of pressure grouted rock core boreholes extended to a uniform depth within the bedrock, initially spaced approximately three metres apart along the upstream cutoff key of the primary Diversion Structure water control elements. Supplemental (secondary and tertiary) grouting boreholes may be added between borings where significant grout takes are observed in the primary grout holes.

8.3.5.3 Drainage

The Service Spillway left abutment retaining walls will be backfilled with soil material consisting of sandy lean clay with gravel (glacial till) obtained from the Diversion Channel excavation. Appendix D summarizes the recommended backfill soil parameters to be used in the design of the walls. The backfill wall loading parameters were developed based on the laboratory testing performed as part of the geotechnical exploration**.**

The design for backfill of the left abutment retaining walls includes drainage features to reduce hydrostatic pressures behind the walls. The recommended drainage system includes chimney and blanket drains, perforated pipes wrapped in filter fabric, collection pipes, and manholes.

8.3.6 Stability

Stability analyses were performed in accordance with Structural Design Criteria outlined previously and included in Appendix E. The Service Spillway structures were evaluated for Usual, Unusual, and Extreme loading conditions under a High hazard classification, which represents the potential range of conditions the structure will be exposed to during its design life. The loading conditions are described in detail in Appendix E with a summary of stability results indicated in tables below.

8.3.6.1 Gate Structure

Stability analyses indicate a relatively light structure sensitive to sliding instability when crest gates are used to retain water during diversion operation. Stability calculations indicate results within the limits of acceptance criteria utilizing an inclined base analysis. For all loading conditions considered, floatation factors of safety were above required, 100 percent of the base was in compression, and sliding factors of safety were above required. Stability results indicate that sliding stability was the primary concern due to the low friction angle at concrete/rock interface and rock/rock bedding planes. To ensure an inclined failure plane utilized in the analysis was valid, the upstream shear key is designed as a structural element. The controlling load cases were Load Case E3 (EDGM applied during 50-Year Flood), and Load Case UN4 (Construction/Maintenance dewatered during 100-Year flood).

Table 18. Gate Crest Block 2A – Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Critical Depth at Gate Lip (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
					Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases											
U1 Normal Operation	1212.1	1211.8	1211.4	18920	1.5	1.6	1.5	43.8	89	34	100
U2 Diversion Operation *50 Yr. Flood*	1214.6	1211.8	1211.8	22323	1.5	1.6	1.5	2.3	60	81	100
Unusual Load Cases											
UN1 Diversion Operation *2013 Flood*	1215.8	1213.1	1215.03	25726	1.3	1.5	1.3	3.2	74	70	100
UN2 Diversion Operation *2013 Flood*	1216.1	1213.0	1215.73	25998	1.3	1.6	1.3	2.8	62	94	100
UN3 No Diversion *1000 Yr. Flood*	1217.0	1214.7	1214.67	29537	1.3	1.4	1.3	5.9	86	46	100
UN4 Construction/ Maintenance	1215.0	1212.5 bay with flow	1211.9	23003	1.3	1.5	1.3	3.2	55	75	100
Extreme – Flood											
E1 IDF-DS without Diversion	1217.3	1214.9	1214.87	30217	1.1	1.4	1.1	5.6	86	46	100
Extreme – Earthquake used to determine Post-Seismic Condition											
E2 EDGM applied to U1	1212.1	1211.8	1211.4	18920	1.1	1.5 (E2.3)	1.0	1.4 (E2.2)	77 (E2.3)	38 (E2.3)	100
E3 EDGM applied to U2	1214.6	1211.8	1211.8	19601	1.1	1.4 (E3.3)	1.0	1.0 (E.3.2)	30 (E3.2)	112 (E3.1)	100

Notes:
1. See Appendix E.2 for definition of monolith description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

Table 19. Gate Crest Block 4A – Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Critical Depth at Gate Lip (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
					Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases											
U1 Normal Operation	1212.1	1211.8	1211.4	18920	1.5	1.6	1.5	43.8	89	34	100
U2 Diversion Operation *50 Yr. Flood*	1214.6	1211.8	1213.76	22323	1.5	1.6	1.5	2.7	69	78	100
Unusual Load Cases											
UN1 Diversion Operation *2013 Flood*	1215.8	1213.1	1213.87	25726	1.3	1.5	1.3	5.5	85	54	100
UN2 Diversion Operation *2013 Flood*	1216.1	1213.0	1214.07	25998	1.3	1.5	1.3	4.9	84	58	100
UN3 No Diversion *1000 Yr. Flood*	1217.0	1214.7	1214.67	29537	1.3	1.4	1.3	5.9	86	46	100
UN4 Construction /Maintenance	1215.0	1211.9 dry bay	1213.63	23003	1.3	1.6	1.3	4.4	67	67	100
Extreme – Flood											
E1 IDF-DS without Diversion	1217.3	1214.9	1214.87	30217	1.1	1.4	1.1	5.6	86	46	100
Extreme – Earthquake used to determine Post-Seismic Condition											
E2 EDGM applied to U1	1212.1	1211.8	1211.4	18920	1.1	1.5 (E2.3)	1.0	1.4 (E2.2)	77 (E2.3)	38 (E2.3)	100
E3 EDGM applied to U2	1214.6	1211.8	1213.76	19601	1.1	1.5 (E3.3)	1.0	1.1 (E.3.2)	38 (E3.2)	109 (E3.1)	100

Notes:
1. See Appendix E.2 for definition of monolith description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

Table 20. Gate Center Pier Block 3A – Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases										
U1 Normal Operation	1212.1	1211.8	15136	1.5	1.8	1.5	40.3	78	66	100
U2 Diversion Operation *50 Yr. Flood*	1214.6	1211.8	17858	1.5	1.7	1.5	2.5	56	77	100
Unusual Load Case										
UN1 Diversion Operation *2013 Flood*	1215.8	1213.1	20580	1.3	1.6	1.3	3.0	61	97	100
UN2 Diversion Operation *2013 Flood*	1216.1	1213.0	20798	1.3	1.6	1.3	3.5	67	98	100
UN3 No Diversion *1000 Yr. Flood*	1217.0	1214.7	23629	1.3	1.5	1.3	6.2	83	64	100
UN4 Construction/ Maintenance	1215.0	1211.9	18403	1.3	1.7	1.3	3.2	55	97	100
Extreme – Flood										
E1 IDF-DS without Diversion	1217.8	1214.9	24174	1.1	1.5	1.1	4.8	78	70	100
Extreme – Earthquake used to determine Post-Seismic Condition										
E2 EDGM applied to U1	1212.1	1211.8	15136	1.1	1.6 (E2.3)	1.0	1.6 (E2.2)	48 (E2.3)	94 (E2.1)	100
E3 EDGM applied to U2	1214.6	1211.8	17858	1.1	1.6 (E3.3)	1.0	1.1 (E3.2)	34 (E3.3)	136 (E3.1)	100

Notes:
1. See Appendix E.2 for definition of monolith description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

8.3.6.2 Stilling Basin

Stability analysis results indicate a need for anchorage of the stilling basins into the bedrock below, however the required anchor force is within the capability of conventional active or passive ground anchors. The results of the stilling basin stability analysis are presented in Table 21 below. The controlling Load Case for the stilling basin anchors is F3 (Unusual - Single Bay Dewatered for Maintenance/Construction).

Table 21. Stilling Basin – Floatation Stability Summary

Load Case	Tailwater Elevation (m)	Vertical Force Down (kN)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Anchor Force Required
				Required	Calculated	
F1 Usual Normal Operation	1212.1 / 1211.8	22123.0	15123.5	1.50	1.46	2.1
F2 Unusual 2013 Flood	1215.8 / 1213.1	25470.6	19973.3	1.30	1.28	1.9
F3 Unusual Const./Dewatered	1214.6 / 1211.8	12337.5	14936	1.30	0.83	27.0
F4 Extreme Ineffective Drain	1215.8 / 1211.9	22123.00	17439.3	1.10	1.27	0.0

8.3.6.3 Right Abutment Pier

The Right Abutment Pier is subject to forces in multiple directions. Stability analysis results indicate a relatively heavy structure sensitive to seismic effects. Due to the directional forces on the structure, the sliding failure plan is considered as a horizontal failure plane, ignoring the keys and cut-off walls. The results of the Right Abutment Pier stability analysis are presented in Table 22 below. The controlling Load Case for the Abutment Pier is E3 (seismic conditions applied to Usual 2 load case).

Table 22. Right Abutment Pier – Stability Analysis Summary

Load Case	Headwater Elevation (m)	Tailwater Elevation (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (Kpa)	
Usual Load Cases										
U1 Normal Operation	1212.1	1211.8	16750	1.5	3.4	1.5	5.6	55	224	100
U2 Diversion Operation *50 Yr. Flood*	1214.6	1211.8	20817	1.5	2.8	1.5	5.6	42	225	100
Unusual Load Case										
UN1 Diversion Operation *2013 Flood*	1215.8	1213.1	24298	1.3	2.5	1.3	5.1	52	200	100
UN2 Diversion Operation *2013 Flood*	1216.1	1213.0	24013	1.3	2.6	1.3	4.7	86	175	100
UN3 No Diversion *1000 Yr. Flood*	1217.0	1214.7	26957	1.3	2.3	1.3	4.2	57	185	100
UN4 Construction/ Maintenance	1215.0	1211.9	21575	1.3	2.7	1.3	4.2	16	240	100
Extreme – Flood										
E1 IDF without Diversion	1217.8	1214.9	28858	1.1	2.1	1.1	4.3	73	158	100
Extreme – Earthquake used to determine Post-Seismic Condition										
E2 EDGM applied to U1	1212.1	1211.8	16750	1.1	2.0 (E2.3)	1.0	1.3 (E2.2)	24 (E2.1)	260 (E2.1)	100
E3 EDGM applied to U2	1214.6	1211.8	20817	1.1	1.6 (E3.3)	1.0	1.1 (E3.2)	8 (E3.2)	242 (E3.1)	100

Notes:
1. See Appendix E.1 for definition of monolith description, analysis methodology, and stability calculations.
2. Reported seismic results are controlling values for the three combinations of vertical and horizontal seismic load considered.

8.3.6.4 Left Abutment Retaining Walls

The left abutment retaining walls are concrete gravity structures designed as either counterfort or cantilever retaining walls depending on wall height. In general, walls with stem heights more than 6.5 m required counterforts to provide adequate stiffness and lateral load path. Three representative sections were identified to capture the range of wall geometry and loading conditions for the overall structure. Representative sections are indicated on Figure 23 and described as follows.

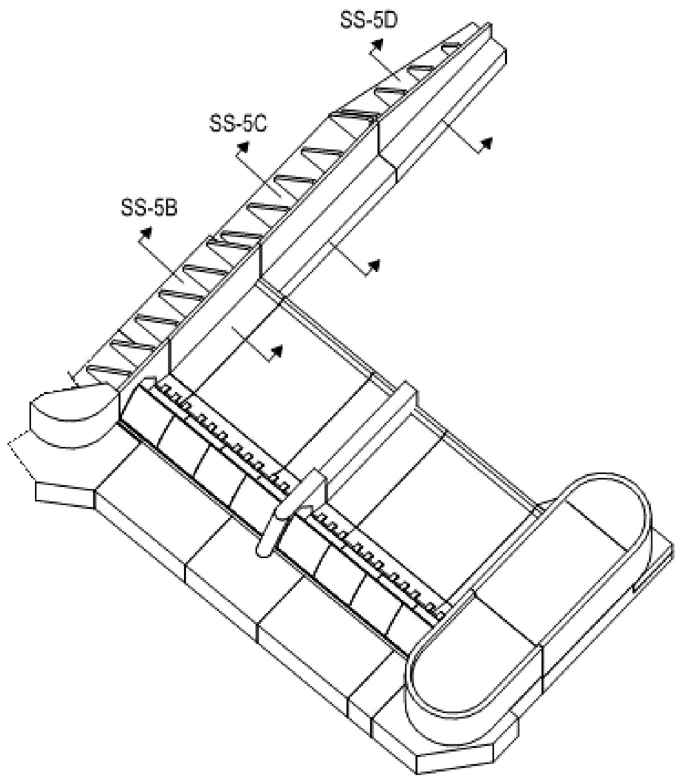


Figure 23. Service Spillway Retaining Wall Key Plan

- Section SS-5B: Counterfort wall serving as the Service Spillway gate training wall. Crest gates do not transfer load laterally, so the stem wall is similar to other wall sections, but the footing is thicker to match the stilling basin concrete profile.

- Section SS-5C: Counterfort wall downstream of the stilling basin representing the tallest wall in the Service Spillway. This section was considered one of the critical wall sections due to retained soil height, retained groundwater depth, minimal resistance on the toe, and potential for vehicle surcharge.

- Section SS-5D: Counterfort wall serving as a downstream training wall and slope protection. This location downstream of the Service Spillway is subjected to different water loading and increased potential for rock scour along the toe than the upstream Section SS-5C retaining wall. Section SS-5D was analyzed at two sections to account for the geometric variability of the monolith.

Stability analyses were performed in accordance with the Structural Design Criteria outlined previously and included in Appendix E. Each of the three representative wall sections was evaluated for Usual, Unusual, and Extreme loading conditions representing the potential range of conditions the structure will be exposed to during the design life. The loading conditions are described in detail in Appendix E with a summary of stability results indicated in Table 23 below. In accordance with guidelines for hydraulic structures, at-rest soil pressures were used for all Load Case calculations except active soil pressures were used when considering seismic load cases.

The principal factors affecting the retaining wall design include significant driving force associated with high groundwater conditions; poor rock quality along the foundation interface; relatively weak material (glacial till) anticipated in the backfill zone of influence; and potential for significant uplift pressure when water levels recede faster than pore pressure can dissipate. Preliminary design calculations indicate that retaining walls are most sensitive to groundwater conditions, concrete shear capacity of stem walls, and sliding stability provided by foundation shear keys.

For all loading conditions considered, floatation factors of safety were above required, 100 percent of the base was in compression, and sliding factors of safety were above required. Stability results indicate that sliding stability is the primary concern due to the low friction angle at the concrete/rock interface and rock/rock bedding planes. To achieve stability results within the limits of acceptance criteria, a shear key at the heel of footing, and a wall drain system were required. The structural shear key ensures an inclined base sliding analysis is valid, and the wall drain system significantly reduces load associated with groundwater. The controlling load case is Load Case UN2 (high groundwater due to ineffective drain).

Table 23. Retaining Walls – Stability Analysis Summary

Load Case	Headwater (Heel) Elevation (m)	Tailwater (Toe) Elevation For Uplift (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (kPa)	
WALL BLOCK SS-5B										
U1 Normal Operation	1213.1	1210.0	802	1.5	3.36	1.5	1.67	149	213	100
UN1 Equip. Surcharge	1213.1	1210.0	802	1.3	3.49	1.3	2.01	140	242	100
UN2 Ineffective Drain	1216.2	1216.2	1372	1.3	2.00	1.3	1.30	99	184	100
E1 Seismic	1213.1	1210.0	802	1.1	3.03	1.0	1.79	119	203	100
WALL BLOCK SS-5C										
U1 Normal Operation	1213.0	1210.0	1041	1.5	3.23	1.5	1.64	73	404	100
UN1 Equip. Surcharge	1213.0	1210.0	1041	1.3	3.35	1.3	1.52	56	444	100

Table 23. Retaining Walls – Stability Analysis Summary (Continued)

Load Case	Headwater (Heel) Elevation (m)	Tailwater (Toe) Elevation For Uplift (m)	Uplift Force (kN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (kPa)	
UN2 Ineffective Drain	1214.4	1214.4	1397	1.3	2.43	1.3	1.32	49	379	100
E1 Seismic	1213.0	1210.0	1041	1.1	2.92	1.0	1.30	43	386	100
WALL BLOCK SS-5D (Mid-section)										
U1 Normal Operation	1212.5	1210.0	862	1.5	2.78	1.5	1.92	85	272	100
UN1 Equip. Surcharge	1212.5	1210.0	862	1.3	2.90	1.3	1.73	76	304	100
UN2 Ineffective Drain	1214.4	1214.4	1202	1.3	2.02	1.3	1.31	49	258	100
E1 Seismic	1212.5	1210.0	862	1.1	2.51	1.0	1.35	55	266	100
WALL BLOCK SS-5D (Downstream)										
U1 Normal Operation	1212.5	1210.0	722	1.5	2.21	1.5	2.38	71	169	100
UN1 Equip. Surcharge	1212.5	1210.0	722	1.3	2.33	1.3	2.04	70	192	100
UN2 Ineffective Drain	1212.6	1212.6	847	1.3	1.88	1.3	1.31	69	145	100
E1 Seismic	1212.5	1210.0	722	1.1	1.99	1.0	1.28	42	172	100

Notes:
1. See Appendix E.2 for definition of wall section description, analysis methodology, and stability calculations.
2. Analysis assumes inclined sliding plane, interface friction angle Φ = 26 degrees, and no cohesion.
3. Seismic results utilize active soil pressure coefficients for stability values reported.

8.3.7 Strength

The Service Spillway is designed as a mass concrete gravity structure sized primarily for stability. Most elements exceed 2 m in thickness and are surface reinforced for crack control and durability rather than strength. Each element is checked to ensure calculated stress from factored loads do not exceed member capacity. Some elements which are subjected to higher stress and controlled by strength design include:

- Divider wall pier which is a 6 m high cantilever wall subjected to unbalanced water load and lateral seismic loading in the cross-stream direction;
- Upstream shear keys which are a structural element required for sliding stability; and
- Crest gate hinge anchor bolts.

For each of these elements, preliminary strength calculations were performed to acquire order-of-magnitude stress and establish basis for preliminary member sizing. Strength calculations to develop reinforcement sizing and steel detailing will be performed during Final Design.

The retaining wall monoliths will be detailed during Final Design using commercially available finite element software with beam, shell, and solid elements where appropriate.

Footings were designed as a structural slab on an elastic foundation as the stability analysis concluded that the foundations are in compression based on the value of the subgrade modulus. The critical sections considered for evaluation of shear and moment were at half the footing thickness as measured from the face of the wall for the toe and at the face of the wall for the heel. In general, footing geometry was dictated by the gate bay limit of excavation, and desired hydraulic profile resulting in footing thicknesses exceeding 1.5 to 2 m with relatively low stress at the critical sections.

Cantilever stem walls were designed as a cantilever beam fixed at the footing interface. The critical sections considered for evaluation were at the base of stem, 1/3 of the stem height, and 2/3 of the stem height. Wall thickness increases from top to bottom with thickness ranging from 0.5 to 2.0 m, respectively. Due to increased thickness and increased load near the base of walls, shear strength becomes a controlling factor, and transverse shear reinforcement (cross ties) will be required.

Counterfort stem walls were designed as continuous beams spanning horizontally between counterforts, with only the lower portions of the stem exhibiting plate action and designed as a cantilever from the footing to a height approximately half of the counterfort spacing.

Counterfort heels were designed with a similar load path as the stem. The portion of footing closest to the stem acts as a cantilever beam, and the portion which is further from the stem by more than half of counterfort spacing, was designed as a continuous beam spanning between counterforts.

Counterforts were designed as cantilever deep beams fixed at the footing interface. The wall serves as the beam flange, and the flange width was calculated as the lesser of 12 times the thickness of the wall or half the distance between the counterforts using equation 10.3.3 of CSA 23.3. The counterfort was considered to act as the stem of a tee beam and was fixed at its base. The tee beam was sized so that the neutral axis of the tee beam was located within the flange. The depth of the tee beam is the perpendicular distance between the sloping face of the counterfort and the vertical face of the retaining wall. Critical sections for evaluation of counterfort shear and moments include the foundation interface and the third points of the counterfort.

8.3.8 Serviceability

Serviceability concerns with the Service Spillway relate primarily to concrete durability, shrinkage control, and relief of internal stresses associated with volume changes.

Shrinkage control and volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The preliminary design includes joint locations that define monoliths with balanced aspect ratios and placements less than 12 to 18 m in any one direction. Expanded guidance related to placement sequence and horizontal joint locations will be addressed as part of constructability review during Final Design.

Allowance for thermal expansion/contraction is critical for gate operation. These affects are addressed primarily through clearance between gate and end walls with provisions for side seals and an embedded UHMW-PE (Ultra High Molecular Weight Polyethylene) wall plate to cover the full extent of gate travel.

Protection of the gate leaf is a serviceability concern due the heavy bed load of sand, gravel, and cobbles in the Elbow River. To protect the gate leaf from abrasion and premature deterioration, a surface skin of UHMW-PE is recommended on the crest gate.

Serviceability concerns for the retaining walls relate to concrete durability, shrinkage, crack control, volume changes and wall deflections. Durability, shrinkage, and crack control are achieved primarily through reinforcement placement, high reinforcement ratios, and use of high load factors that account for both strength and serviceability in accordance with the CSA SEED (CSA, 2018) document. Volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The retaining walls include vertical joints at locations of footing geometry change, and at locations needed to maintain horizontal wall lengths less than 12 to 18 m. Expanded guidance related to placement sequence and horizontal joint locations will be addressed as part of Final Design.

Wall deflections are controlled using counterforts to provide rigidity, by reducing wall and footing spans, and using at-rest soil pressure when sizing wall elements. Locations where wall deflection is critical includes walls serving as gate bay abutments, walls adjacent to access roads and control building foundation, and walls along the upstream face which must maintain tight joints for water retention. Wall deflections will be addressed during Final Design.

8.3.9 Construction Considerations

Construction specifications and details for the Service Spillway will be furthered during Final Design. The following construction considerations are noted:

- Restricted periods for disturbance within the Elbow River are May 1 to July 16 and September 16 to April 14. This means that construction of a river diversion and the coffer-works for the Service Spillway must take place between April 15 and April 31 or July 15 and September 15.

- Dewatering of excavated areas will be required to sufficiently enable construction of the Service Spillway. The services of a specialist dewatering contractor may be needed.

- Excavation will be to competent bedrock. All soil, including alluvium, talus and other unconsolidated deposits should be removed to expose unweathered or slightly weathered bedrock. Excavation should be performed by mechanical means only; blasting will not be permitted.

- Foundation preparation will require special care in cleaning and preparation of concrete/rock interface. Care must be taken during excavation of the foundation to identify unsuitable rock conditions or weak bedding planes that could impact stability. Loose material and rock overhangs will need to be removed. Small voids will be filled with dental concrete. Once ready, foundation protection will be placed over exposed rock.

- If extensive jointing/fracturing is observed after excavation of the foundation, consolidation grouting may be required.

- Shear keys are required to maintain adequate sliding stability for gate monoliths and retaining walls. Care should be taken during excavation of the shear key trenches to identify unsuitable rock conditions or weak beading planes that could compromise capacity of the shear key.

- Anchors, along with a foundation underdrain to relieve uplift pressures, will be required to maintain adequate factors of safety against floatation in the stilling basin. These are envisioned as static anchors drilled and grouted in a grid pattern prior to placement of the stilling basin concrete.

- Lift joints in the base mats and footings will be required to reduce placement thickness, control heat of hydration, reduce crack potential, and develop hydraulic profile. Changes in mix design will be required to provide lower cement ratio and larger aggregate in mass concrete placements, with higher strength and smaller aggregate mix placed as part of the reinforced "surface skin".

- Vertical joints in gate bays and stilling basins will be spaced and detailed so that "closure grouting" needed to accommodate shrinkage during initial curing is not needed.

- Horizontal joints in the retaining wall stems will be required to reduce placement height to avoid potential for aggregate separation, ensure access for adequate vibration, reduce potential for form bulging, and allow for fill placement to progress in stages with wall construction.

- Joint preparation will require special attention to ensure proper installation of water stops, shear keys, dowels, and reinforcement. Joint alignment and water-tight integrity are critical for minimizing water levels on the back side of retaining walls.

- Hinge anchors, airlines, control lines, restraining strap pockets, and wall plates for Service Spillway crest gates will need to be considered during concrete preparation and placement. Placement tolerance for some of these items are tighter than typical heavy construction tolerance due to fit and operating clearance requirements.

- Procurement lead-time for gate embedments and components will likely be driven by steel availability and fabrication schedules. An allowance of 12 to 18 months is recommended to account for design, shop drawing review/approval, fabrication, testing, and delivery.

- The Service Spillway right abutment or the divider pier may serve as a component of the water control plan during construction. These need to be functional prior to completion of the Service Spillway. Many of these details are at the discretion of the Contractor but will need to be coordinated with the Engineer to ensure appropriate loading conditions have been considered in the water control plan design.

- Placement of free draining backfill, filter material, and drain systems are critical for minimizing groundwater levels behind the walls. Material selection and installation methods will require strict quality control and monitoring.

- Fill placement and compaction methods must be reviewed and monitored to ensure wall movement does not occur during construction.

- Construction sequencing will be required to ensure the Service Spillway and crest gates are fully functional before the Elbow River is diverted back through the Service Spillway.

8.4 AUXILIARY SPILLWAY

8.4.1 Arrangement

The Auxiliary Spillway is located along the right bank of the Elbow River between the Service Spillway and Floodplain Berm. The Auxiliary Spillway consists of:

- Mass concrete "hardfill" overflow weir, 208 m long, approximately 8.8 m high, and with a crest set elevation of 1215.8 m;
- Reinforced concrete transition wall separating the overflow weir and Floodplain Berm;
- An earthen fuse plug placed on top of the overflow weir with an overflow elevation of 1216.5 m, and;
- Upstream and downstream embankments overlaying the overflow weir.

The typical section of the Auxiliary Spillway is presented in Figure 24. The general arrangement of the Auxiliary Spillway is depicted on Drawing C-213 with sections presented on Drawing C-271. Structural arrangement and details of the overflow weir and transition wall are shown on Drawings S-260 to S-279.

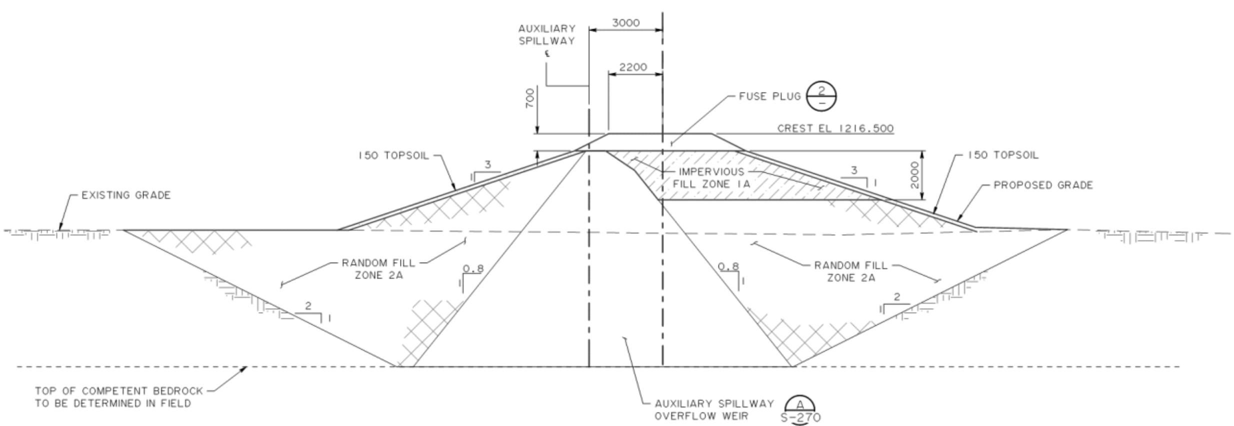


Figure 24. Auxiliary Spillway Typical Section

8.4.2 Design Objectives

The Service Spillway and Auxiliary Spillway function as the water level control structures for the Floodplain Berm embankment. The Auxiliary Spillway and Service Spillway are designed to provide the capacity needed to pass the IDF-DS with sufficient freeboard to the crest of the Floodplain Berm and to pass the IDF-OSSD with sufficient freeboard to the top of the Diversion Inlet walls. The IDF events are defined in Section 3.5.

8.4.3 Alternatives Considered

Alternatives considered for the Auxiliary Spillway were an earth embankment with an articulated concrete block overlay; an earth embankment with a Roller Compacted Concrete (RCC) overlay; and a mass concrete (RCC or cemented sands and gravels (CSG)) section with earth overlay.

The Conceptual Design Update, presented to AT in April 2015, included the use of articulated concrete block (ACB) placed along the crest and downstream slopes of the Auxiliary Spillway embankment. Hydrologic studies further advanced during preliminary design resulted in a significant increase in the PMF flow rates. These flow rates exceeded the capacity of ACB's to provide adequate armoring of the control section and slopes, and ACB's were therefore eliminated from further consideration.

The remaining two alternatives provide adequate hydraulic capacity and serviceability. However, cost comparisons indicated the RCC/CSG section would be approximately $2 million less than the RCC overlay.

When comparing RCC to CSG for the mass concrete section, CSG mass concrete with a conventional facing for forming and protection was selected for the following advantages:

- A hardfill mass concrete weir allows the design of the section based on available type of aggregates, the required maximum strength, and required modulus of elasticity of the dam body with regards to the foundation conditions.

- The traditional triangular-shaped gravity dam results in a significant change in stress distribution between full and empty reservoir conditions while the symmetrical sloped hardfill section increases the foundation contact and results in no tension stresses within the concrete mass.

- Hardfill is less expensive than RCC, in part because the locally available sands and gravels at or near the site can be used once graded instead of quarried rock for RCC aggregate.

- Hardfill has the same workability and uses the same construction processes for testing, mixing and placement as RCC, but without the need for strict temperature controls following lift placement.

8.4.4 Hydraulic Design

The Service Spillway and Auxiliary Spillway were designed to pass the IDF-DS peak discharge of 2210 m^3/s without diversion of flood flows in accordance with a High Hazard dam safety classification.

8.4.4.1 Fixed Crest Spillway

The Auxiliary Spillway fixed crest elevation was set based on the Diversion Inlet capacity rating curve and the corresponding level pool elevation that results in the design diversion rate of 600 m^3/s. This elevation is 1215.8 m.

The design of the spillway was determined based on site constraints and an iterative process that balanced height of the Floodplain Berm and Diversion Inlet to length of the overflow weir. With a crest elevation of 1215.8 m and length of 208 m, the hydraulic analysis for the IDF-DS results in a Service Spillway discharge of 1590 m^3/s and an Auxiliary Spillway discharge of 620 m^3/s with a headwater elevation of 1217.3 m. The Floodplain Berm crest adjacent to the Auxiliary Spillway was set to an elevation of 1218.3 m providing 1 metre of freeboard during the IDF-DS.

The design rating curve is presented in Figure 25. Calculation and methodologies are further discussed in Appendix C.6.

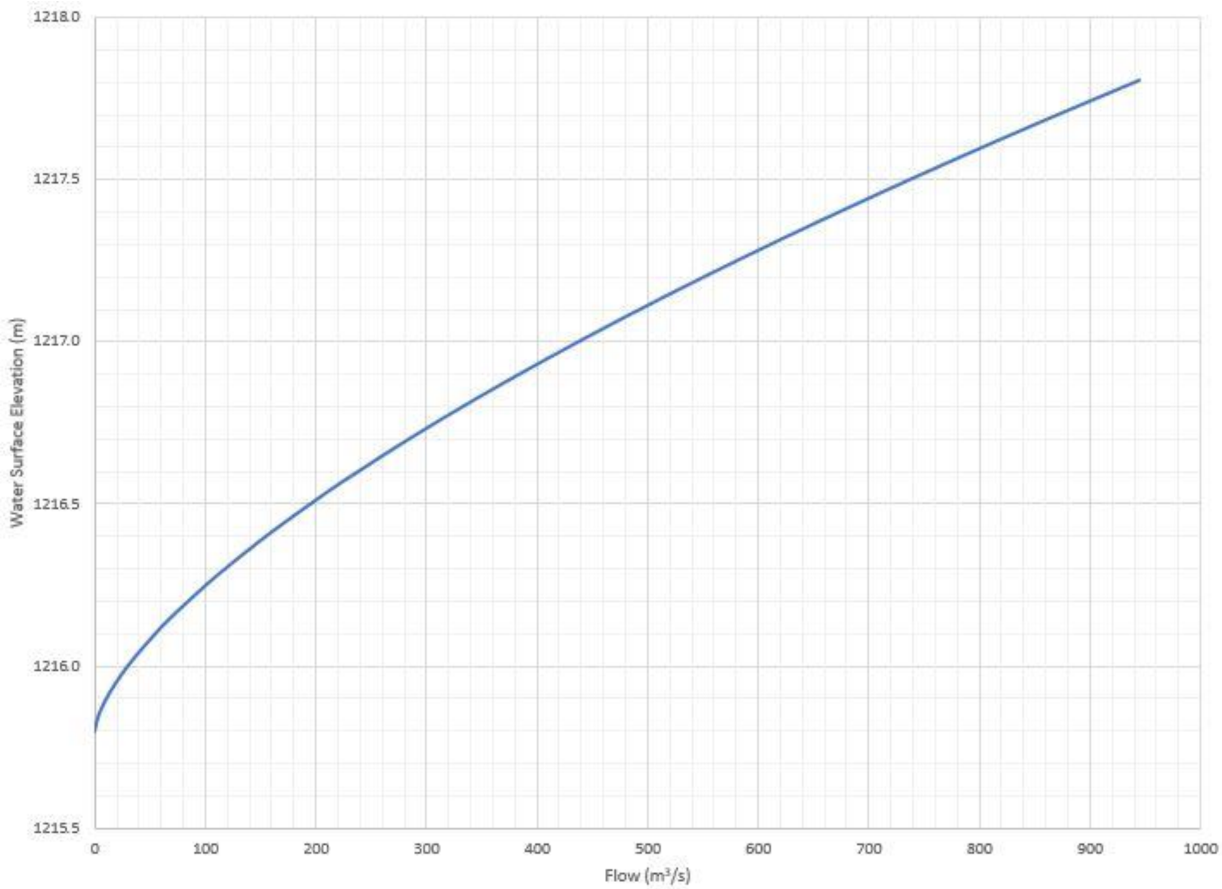


Figure 25. Auxiliary Spillway – Rating Curve

Further discussion regarding the Floodplain Berm and freeboard is provided in Section 8.5.3.

8.4.4.2 Energy Dissipation

Energy dissipation is provided downstream of the Auxiliary Spillway through erosion of the granular floodplain material and formation of a plunge pool. Plunge pool development and scour calculations were performed for the IDF-DS and are provided in Appendix F.3. Tailwater was calculated in the 2D numerical model. At the toe of the Auxiliary Spillway, the average tailwater elevation for the IDF-DS is 1214.3 m.

The calculations indicate that a plunge pool is expected to develop to approximately 4.3 m below the tailwater level. This results in a calculated plunge pool invert of 1210.0 m, approximately 3.0 m above the bedrock foundation.

8.4.4.3 Fuse Plug

Accumulation of debris on the Debris Deflection Barrier could impact the pool elevation upstream of the barrier. Modeling indicates that the pool could raise 0.3 m with 30 percent of the barrier blocked. To allow for this increased headpond elevation and mitigate for early by-passing of the flows during the flood diversion hydrograph, the design includes an erodible fuse plug. The fuse plug is designed to overflow and erode when the headpond reaches Elevation 1216.5 m.

The fuse plug was designed in accordance with USBR (1985) Hydraulic Model Studies of Fuse Plug Embankment by Pugh and USBR and USACE (2015) Best Practices in Dam and Levee Safety Risk Analysis. The design includes a series of granular zones that are stable and resist piping for hydraulic loads up to the crest elevation, but erodible once overtopping begins. Calculations for fuse plug stability and erosion are included in Appendix F.3.

8.4.5 Geotechnical Considerations

8.4.5.1 Foundation

The Auxiliary Spillway overflow weir and transition wall will bear on the same bedrock formation encountered under the Diversion Inlet and Service Spillway. Therefore, the recommendations discussed in Section 8.2.5.1 for the Diversion Inlet structure also apply for the design and construction of the Auxiliary Spillway. As with the Service Spillway, the Auxiliary Spillway structure is within the overburden and highly weathered surface layer and requires excavation likely to range from Elevation 1206.5 to 1207.0 m to expose rock suitable for foundation construction. The final excavation limits will be determined during construction based on field observations.

The recommended parameters related to the allowable bedrock bearing capacity, drained cross bed shear strength and frost depth penetration, and the design basis used to derive these parameters are presented in Appendix D - Geotechnical Assessment Report, Chapter 10.

8.4.5.2 Seepage

Seepage is anticipated around and below the Auxiliary Spillway structure as in the case of the Service Spillway. Therefore, the recommendations discussed in Section 8.2.5.2 for the Diversion Inlet structure also apply for the design and construction of the Auxiliary Spillway.

Foundation pressure grouting to reduce uplift pressures and the permeability of the Brazeau bedrock formation, as recommended in Section 8.2.5.2, may apply to the subgrade areas of the overflow weir based on the foundation conditions once exposed. If needed, the foundation grouting design will likely consist of a single row of pressure grouted rock core boreholes extended to a uniform depth within the bedrock, spaced approximately three metres apart along the upstream cutoff key of the primary Diversion Structure water control elements. Supplemental (secondary and tertiary) grouting boreholes may be added between borings where significant grout takes are observed in the primary grout holes.

8.4.5.3 Backfill and Drainage

The overflow weir will be backfilled with soil material consisting of sandy lean clay with gravel (glacial till) obtained from the Diversion Channel excavation.

Once the overflow weir and transition wall are constructed, the transition wall will be backfilled with soil material used for constructing the Floodplain Berm. The backfill wall loading parameters were developed based on the laboratory testing performed as part of the geotechnical exploration and are listed in Appendix D**.**

Other than an upstream concrete cutoff, no foundation drains are anticipated for the overflow weir or drainage measures for the transition wall backside other than the drainage zones for the Floodplain Berm.

8.4.6 Stability

8.4.6.1 Overflow Weir

The stability analyses for the overflow weir were performed according to the rigid body method using manual calculations. Results of the analyses are summarized in Table 24 and calculations are included in Appendix E.

Stability analyses indicate a relatively light structure sensitive to sliding instability. Stability calculations indicate results within the limits of acceptance criteria using a horizontal plane. For all loading conditions considered, floatation factors of safety were above required, 100 percent of the base was in compression, and sliding factors of safety were above required. Stability results indicate that sliding stability was the primary concern due to the low friction angle at concrete/rock interface and rock/rock bedding planes. The controlling load is Load Case E1-F (Inflow Design Flood).

Table 24. Overflow Weir – Stability Summary

Load Case	Headwater (Heel) Elevation (m)	Tailwater (Toe) Elevation For Uplift (m)	Uplift Force (MN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (kPa)	
U1 Normal Operation	1214.0	1211.9	0.9	1.5	3.11	1.5	9.62	159	87	100
UN1 Point of Fuse Plug Activation	1216.5	1213.1	1.2	1.3	2.38	1.3	1.67	124	86	100
UN2 1000-Year Flood	1217.0	1213.1	1.2	1.3	1.94	1.3	1.52	65	77	100
E1-F IDF 2210 m^3/s	1217.3	1213.8	1.3	1.1	1.77	1.1	1.12	72	57	100
E2-Q Normal Operation	1213.5	1211.9	0.7	1.1	4.12	1.1	1.56	226	48	100

8.4.6.2 Transition Wall

The transition wall provides a separation between the Floodplain Berm and overflow weir and consists of three sections. The top of the transition wall varies from Elevation 1212.72 m to Elevation 1218.44 m, following the surface contour of the Floodplain Berm. The cantilever walls thickness varies from 500 mm to 1700 mm, while the foundation slab ranges from 1200 mm to 1800 mm in thickness. A Load Case that represent equipment operating on top of the Floodplain Berm (UN1) was considered for the stability analysis of Section 2. Equipment loads were not considered for Section 1 and 3.

Results of the stability analyses are summarized in Table 25 and calculations are included in Appendix E.3.

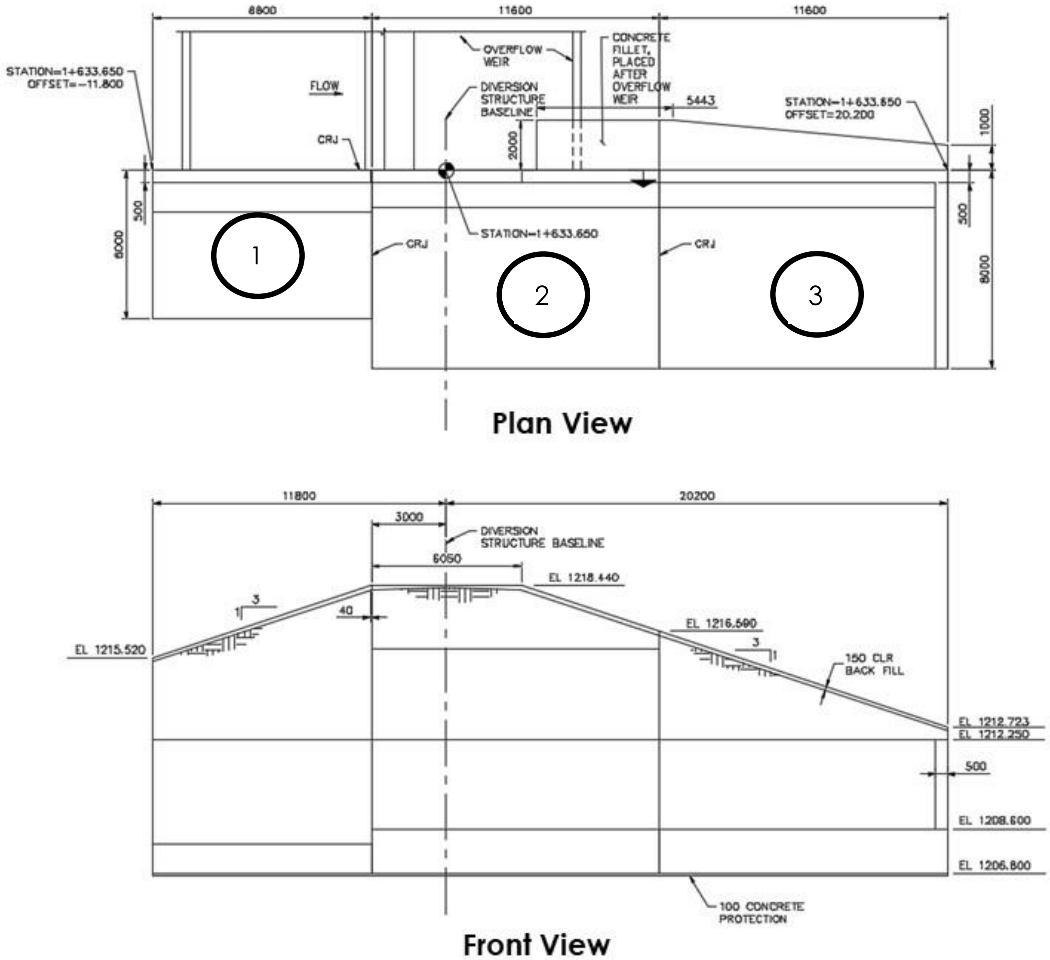


Figure 26. Transition Wall - Section Locations

Table 25. Transition Wall – Stability Summary

Load Case	Headwater (Heel) Elevation (m)	Tailwater (Toe) Elevation For Uplift (m)	Uplift Force (MN)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress		% Base in Compression
				Req	Calc	Req	Calc	Upstream (Heel) (kPa)	Downstream (Toe) (kPa)	
Section 1										
U1	1214.0	1211.9	3.4	1.5	3.17	1.5	1.80	21.2	261.3	100
UN1	1214.0	1211.9	-	-	-	-	-	-	-	-
UN2	1216.5	1213.1	4.9	1.3	2.24	1.3	1.57	3.8	225.2	100
UN3	1217.0	1213.1	5.1	1.3	2.14	1.3	1.63	8.2	211.6	100
E1-F	1217.3	1213.8	5.3	1.1	2.05	1.1	1.62	10.0	201.8	100
E2-Q	1213.5	1211.9	3.1	1.1	3.57	1.1	6.84	0.0	355.7	<100
Section 2										
U1	1214.0	1211.9	5.1	1.5	4.28	1.5	1.51	51.7	309.9	100
UN1	1214.0	1211.9	5.1	1.3	4.28	1.3	1.34	34.2	327.3	100
UN2	1216.5	1213.1	6.6	1.3	3.30	1.3	1.39	32.9	296.1	100
UN3	1217.0	1213.1	6.6	1.3	3.33	1.3	1.43	35.6	294.4	100
E1-F	1217.3	1213.8	7.3	1.1	2.92	1.1	1.37	20.9	281.0	100
E2-Q	1213.5	1211.9	5.0	1.1	4.39	1.1	1.48	0.0	395.4	<100
Section 3										
U1	1214.0	1211.9	5.69	1.5	3.06	1.5	3.27	56.3	155.9	100
UN1	1214.0	1211.9	-	-	-	-	-	-	-	-
UN2	1216.5	1213.1	7.04	1.3	2.47	1.3	2.01	35.2	152.5	100
UN3	1217.0	1213.1	7.21	1.3	2.35	1.3	1.32	42.3	147.5	100
E1-F	1217.3	1213.8	7.8	1.1	2.17	1.1	1.12	29.3	151.4	100
E2-Q	1213.5	1211.9	5.7	1.1	1.2	1.1	1.2	10.3	202.0	100

8.4.7 Strength

Strength evaluation of individual elements or members of structures and monoliths was used to verify member sizes based on application of factored loads as described in ABC with some adjustments for more severe conditions or loads not included in the ABC.

Reinforced concrete design was performed according to Design of Concrete Structures, CSA A23.3-14 with the additional requirements of the CSA's SEED Document – *Structural Design of Wastewater Treatment Plants-2018* for revisions addressing service load conditions, water tightness, shrinkage and temperature reinforcement, and crack control. The Seed Document contains references to ACI 350M-06 for modifying CSA A23.3-14.

In general, structural analysis and design was performed manually using MathCAD or Excel spreadsheets, For complex structures such as the transition wall blocks, a commercial Finite Element Model (FEM), Autodesk Robot, was used to evaluate multiple load combinations, identify stress concentrations, and generate shear and moment values for design of individual elements. The FEM was supplemented with manual calculations to verify/validate model results and, where necessary, refine the analysis of individual elements. Based on model output, a combination of manual calculation and commercial software were used for strength design. Additional elements evaluated as part of strength design included joint detailing and embedded parts.

For mass concrete structures, such as the hardfill overflow weir, thermal analyses for the construction condition and for seasonal temperature variations following construction will be performed during Final Design. These analyses are used to locate monolith joints, determine the type of joint treatment between lifts, and determine the lateral extent of mass concrete expansion and contraction due to seasonal influences.

8.4.8 Serviceability

Serviceability concerns with the Auxiliary Spillway overflow weir relate primarily to concrete durability including reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR) and other chemical attack. Because hardfill lacks long-term durability, particularly against freeze-thaw conditions, it must be protected. Various protection means include using cast-in-place concrete or precast blocks for facing, providing extra thickness beyond that needed as a sacrificial layer, and mixing grout with the hardfill mixture at the surface to increase its paste content. Currently, the overflow weir design uses cast-in-place facing concrete on both faces to protect the hardfill core. During final design, the use of concrete facing protection will be evaluated further, particularly if facing concrete is needed, since the overflow weir will be covered with soil.

Shrinkage control and volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The monolith layout and design include joint locations that define monoliths with balanced aspect ratios and placements less than 12 to 18 m in any one planar direction for mass concrete. Expanded guidance related to placement sequence and joint locations will be addressed as part of Final Design.

Serviceability concerns for the reinforced concrete transition wall relate to concrete durability, shrinkage, crack control, volume changes, and wall deflections. Durability, shrinkage, and crack control are achieved primarily through reinforcement placement, high reinforcement ratios, and use of high load factors that account for both strength and serviceability in accordance with the CSA SEED document. Volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The preliminary design includes vertical joints at locations of footing geometry change, and at locations needed to maintain horizontal wall lengths less than 12 m to 15 m. Expanded guidance related to placement sequence and horizontal joint locations will be addressed as part of constructability review during Final Design.

8.4.9 Construction Considerations

Construction specifications and details for the Auxiliary Spillway will be furthered during Final Design. The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the Auxiliary Spillway. The services of a specialist dewatering contractor may be needed.
- Excavation will be to competent bedrock. All soil, including alluvium, talus and other unconsolidated deposits should be removed to expose unweathered or slightly weathered bedrock. Excavation should be performed by mechanical means only; blasting will not be permitted.
- Foundation preparation will require special care in cleaning and preparation of concrete/rock interface. Care must be taken during excavation of the foundation to identify unsuitable rock conditions or weak bedding planes that could impact stability. Loose material and rock overhangs will need to be removed. Small voids will be filled with dental concrete.
- If extensive jointing/fracturing is observed after excavation of the foundation, consolidation grouting may be required.
- Use of a continuous hardfill batching-mixing plant or pugmill is likely. The area for Hardfill production requires approximately three to four acres to provide space for the Hardfill plant, aggregate stockpiles, cement and fly ash silos, feeding systems and material delivery and loading areas. In addition, a level area of approximately one acre should be planned for the equipment staging and maintenance area next to the production plant.
- Hardfill may be placed using either a conveyor system or an all truck transporting system. If an all truck system is used, provisions should be made to prevent truck tires from tracking soil and other deleterious materials on the fresh Hardfill.
- It is envisioned that hardfill will be spread using a dozer and compacted with a double drum or single drum, self-propelled vibratory steel drum roller. Small compaction equipment will likely be required in tight spaces such as next to forms.
- Adequate bonding between hardfill lifts requires that the overlying lift of hardfill be placed while the underlying lift is still "live" or has not become a cold joint. Where cold joints form between lifts, placement of a bonding mortar or grout will likely be required before succeeding lifts of hardfill are placed.

- For the construction of the Auxiliary Spillway, the following construction sequence shall be observed. Construction of the transition wall shall be performed first so the wall can act as a vertical form for the construction of the overflow weir and facilitate the construction of the joint between these two structures. The overflow weir shall be constructed before the construction of the Floodplain Berm embankment closure. The Floodplain Berm closure is constructed last in the construction sequence since the transition wall relies on the overflow weir for stability.

8.5 FLOODPLAIN BERM

8.5.1 Arrangement

The Floodplain Berm is an earthen embankment located on the south (river right) floodplain of the Elbow River serving to constrain flow within the active river channel and floodplain, and direct flow through the Diversion Inlet and Service Spillway. The primary elements of the Floodplain Berm include:

- A low permeability clay soil embankment, with a sand filter inclined drain and a random fill downstream shell;
- A granular toe drain with a perforated drain pipe; and
- Riprap erosion protection with a self-launching apron on the upstream face.

The Floodplain Berm is approximately 1030 m long with a maximum height of approximately 5.5 m. The Floodplain Berm has a maximum crest elevation of 1221.46 m at the right descending bank and slopes downward at a 0.3 percent slope to Elevation 1218.34 m at the Auxiliary Spillway. The general arrangement of this structure is depicted on Drawing C-201 with detailed grading plans on Drawings C-210 through C-213.

8.5.2 Design Objectives

Per the CDA Guidelines (CDA, 2013), the Diversion Structure is designed to safely pass the IDF-DS (See Section 3.5) with sufficient freeboard from overtopping.

8.5.3 Freeboard

A minimum freeboard of 1 m is provided for the IDF-DS for the length of the embankment from Station 0+900 to 1+633.15. For the upstream section of the Floodplain Berm from Station 0+600 to 0+900, the embankment height is less than 1 m in height and freeboard is reduced to 0.5 m. Figure 27 displays a profile of the Floodplain Berm and the calculated water surface elevations for a range of river discharges calculated using the numerical model described in Section 4.1.1.

As may be observed in the figure, during the IDF-DS a small amount of flow is expected to circumvent the Floodplain Berm at the upstream limits. The Floodplain Berm does, however, constrain flows within the Elbow River active channel and floodplain for events equal to or less than the 1:1000 year event.

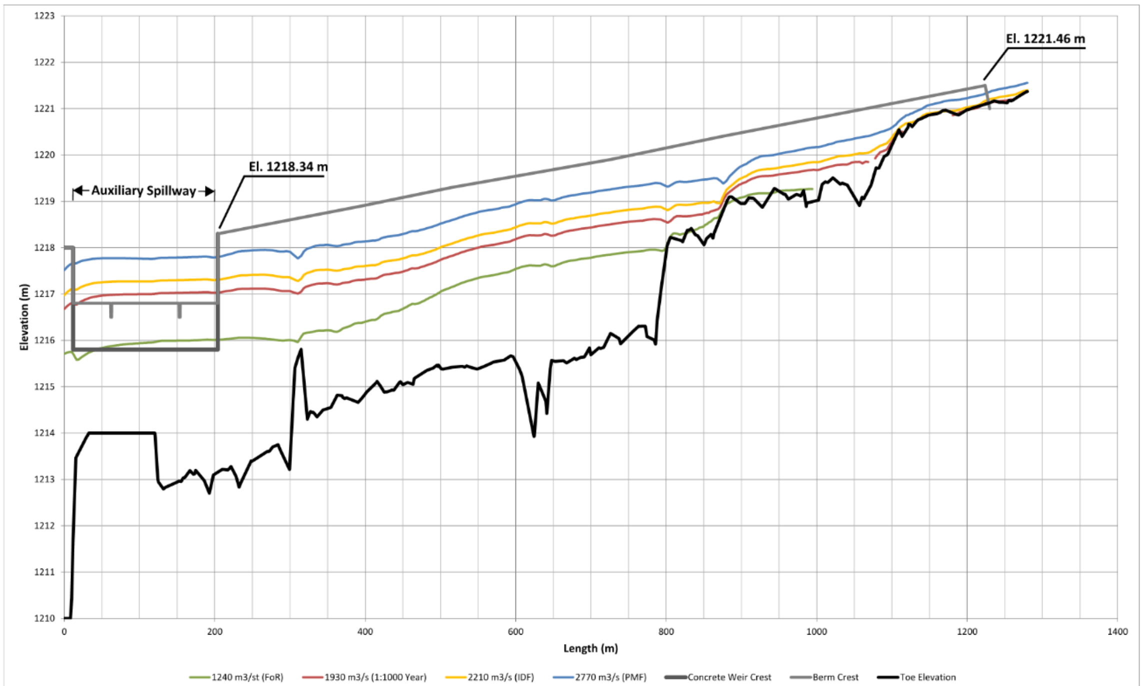


Figure 27. Floodplain Berm WSE Profiles

8.5.4 Stability and Settlement

8.5.4.1 Profiles

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 28). 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 accommodate surplus Impervious 1A or Random 2A Fill.

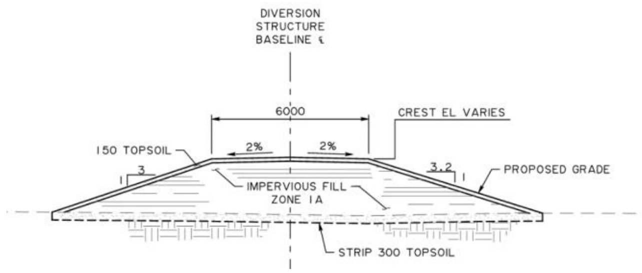


Figure 28. Floodplain Berm Typical Section A Configuration

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 29). 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.

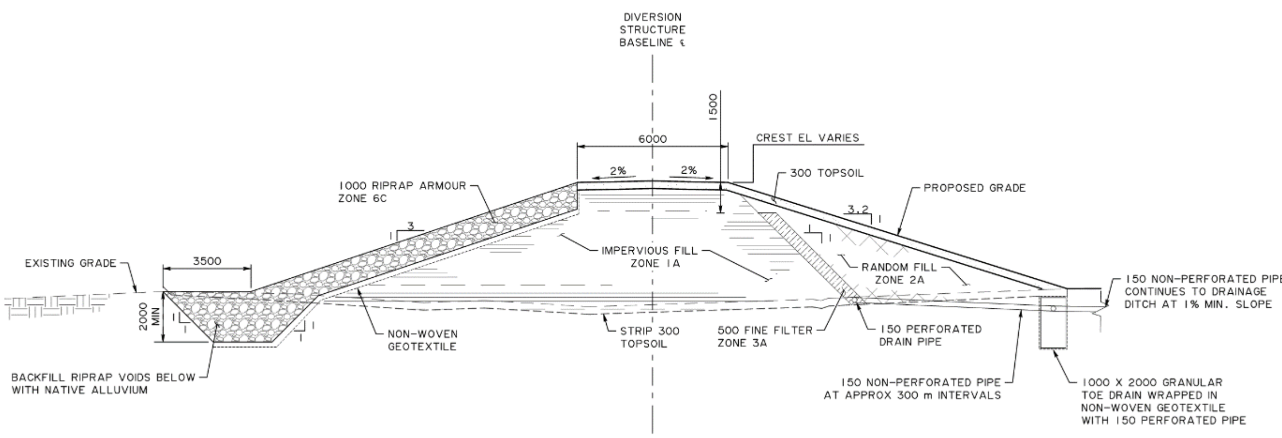


Figure 29. Floodplain Berm Typical Section B Configuration

8.5.4.2 Stability Load Cases

The load cases evaluated are described in Table 26 below.

Table 26. 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.

8.5.4.3 Stability Results

The results of the slope stability analysis for each load case for the proposed cross sections are presented in Table 27.

Table 27. 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 Appendix D.

8.5.4.4 Settlement Results

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 28.

Table 28. 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.

8.5.5 Seepage

Seepage analyses were performed for the two typical sections. The hydraulic gradients estimated in the seepage model were used to evaluate the potential for piping at the seepage exit, in the area of the downstream toe of the Floodplain Berm sections. Safety factors for piping due to heave, defined in terms of the seepage exit gradient, are described in Table 30.

Table 29. 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]	IDF-DS	FoR Operations	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 at the toe during the operations flood event.

Analysis indicated adequate FOS against piping at both sections. A detailed discussion of the seepage analyses is included in Appendix D.

8.5.6 Construction Considerations

The following items have been identified as construction considerations for the Floodplain Berm:

- Foundation preparation work should include stripping of topsoil and removal of all soft surficial deposits within the footprint of the embankment. Excavation should be performed by mechanical means only; blasting should not be permitted.

- The elevation of the water encountered in the geotechnical boreholes was similar to that of the river. The design assumed the foundation soils for the Floodplain Berm will be saturated. Saturated soils and groundwater infiltration should be anticipated when excavating for the Floodplain Berm elements. Dewatering of excavated areas will be required to sufficiently enable construction of the Floodplain Berm. The services of a specialist dewatering contractor may be needed.

8.6 DEBRIS DEFLECTION BARRIER

8.6.1 Arrangement

The Debris Deflection Barrier consists of a steel framed post and horizontal beam system bearing on a concrete foundation. The concrete foundation bears on the rock subgrade and incorporates a tension component using drilled shafts. The structure is 165 m long with a variable height concrete foundation wall surmounted by a 5.75-metre-tall frame. The concrete foundation wall forms the left bank of the Elbow River and extends from rock to the approximate 1:2 year water surface. This ranges from Elevation 1211.5 m at the downstream end to Elevation 1212.0 m at the upstream end. The horizontal members of the frame are comprised of hollow steel structural piping spaced 750 mm apart. The top of the frame is set to Elevation 1217.25 m; the Probable Maximum Flood water surface in the Elbow River assuming no diversion.

8.6.2 Design Objectives

The primary objective of the Debris Deflection Barrier is to reduce the risks that large debris pose to the operation of the Diversion Inlet gates and to the bridge piers and other structures in the Diversion Channel during a flood operations event. The alignment of the barrier, parallel to the river, promotes the passage of debris downstream and through the Service Spillway.

The structure will normally be in a dry condition except during flood events. After flood operations, the removal of debris from the barrier and river channel will be required.

8.6.3 Alternatives Considered

Section 8.1.5 addresses the various debris management alternatives considered and the basis for selection of the Debris Deflection Barrier. A brief description of alternatives considered during design advancement is provided below.

Multiple alignments of the Debris Deflection Barrier were tested within the physical model with the final iteration identified as Alignment F. After completion of the physical model testing, additional alignments were considered including those identified as Alignments G and H in Figure 30. The goal of the alternate alignments was to reduce the potential for debris hang-ups at the upstream "hinge point", which was observed during the physical model testing.

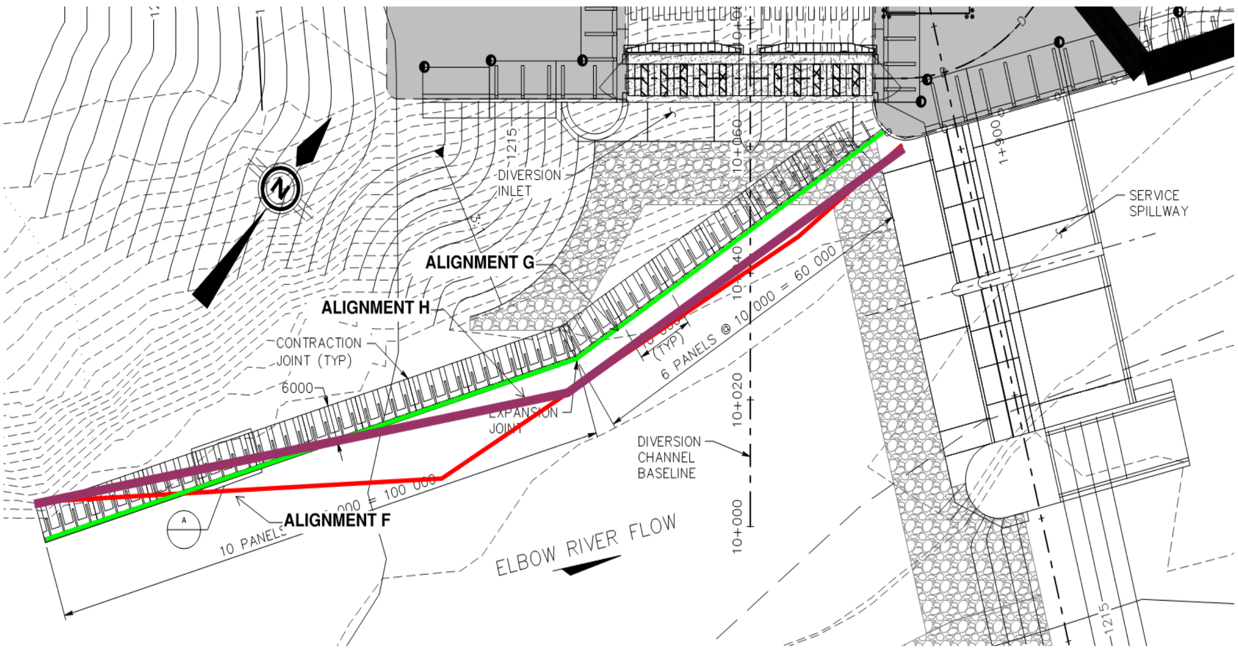


Figure 30. Debris Deflection Barrier – Alternate Alignments

Alignments G and H increased the open width of the existing Elbow River channel, but reduced the area on the backside of the barrier and in front of the Diversion Inlet. As a result of relocating the hinge point of the barrier, the upstream leg of the barrier (more adverse to flow vectors) was increased and the lower leg (more parallel to flow vectors) was decreased.

Ultimately, Alignment F was retained with minor modifications that included rounding of the angled bends and re-grading of the Elbow River channel to maintain the existing channel width to the south and east of the barrier.

8.6.4 Hydraulic Design

The physical hydraulic model of the Elbow River, Diversion Structure and Diversion Channel is described in Section 4. As part of this study, a series of debris management alternatives including debris deflection barriers were tested. The study provided insight to debris collection and passage.

To further evaluate the design of the proposed system, Stantec developed a three-dimensional computational fluid dynamics (CFD) model using the ANSYS Fluent software package.

Initial model simulations were performed using the conceptual design. The objective of these initial model simulations was to determine inputs to the structural design including:

- Hydrodynamic forces acting on the barrier;
- Water level differential for clean and partially blocked barrier; and
- Near surface velocities for application to the debris impact load calculations and debris mat drag force calculations.

Further description of the methods and results of the preliminary CFD modeling are provided in Appendix C.9.

8.6.5 Geotechnical Considerations

8.6.5.1 Foundation Characterization

The Debris Deflection Barrier will bear on bedrock, which consists primarily of shale, mudstone, claystone and sandstone. The recommended parameters related to allowable bedrock bearing capacity, drained cross bed shear strength and frost depth penetration, and the design basis used to derive these parameters are presented in Appendix D.

8.6.5.2 Foundation Design

Foundation analyses for the Debris Deflection Barrier supports were completed to size the foundation elements. The design consists of two foundation elements: caissons on river side and a spread footing on bluff side.

Drilled shaft analyses were conducted for the caissons using the SAP 2000 program with 2.5 metres center to center spacing. The analyses considered side friction, point bearing capacity, lateral capacity, and uplift due to adfreeze.

Continuous footing analyses were conducted for a tributary area corresponding to the 2.5 metres center to center spacing. The analyses considered bearing capacity and sliding resistance.

Further information regarding the methods and results of the foundation design are provided in Appendix D.

8.6.6 Stability

The Debris Deflection Barrier was analyzed for stability for loading conditions in accordance with AT WCS, CDA Guidelines and CSA standards.

8.6.6.1 Methodology

The structural analysis of the Debris Deflection Barrier was completed based on a strength evaluation and design. Since the structure is supported by a continuous footing and caissons, a rigid body method was not used in the analysis.

All forces applied to the structure were computed and analyzed depending on the load case using a three-dimensional structural analysis model. The foundation was then designed to resist the force resultant at the foundation/rock interface.

8.6.6.2 Load Conditions

The load combinations considered in the review included hydraulic and debris impacts and are described in Table 30. The different load factors are also represented.

Table 30. Load Combinations – Debris Deflection Barrier

Load Case		Load Combinations
Usual Load Cases:		
U1	Normal pool (Sunny day)	1.25D+1.5H+1.5W+1.25E
U2	1:100 Year, 760 m^3/s	1.25D+1.5H+1.5HD+1.25E+1.5IM
Unusual Load Cases:		
UN1	1:250 Year, 1240 m^3/s	1.25D+1.5H+1.5HD+1.25E+1.5IM
UN2	1:100 Year, 760 m^3/s	1.25D+1.5H+1.5HD+1.25E
Extreme Load Cases:		
E1	1:100 Year, 760 m^3/s	1.25D+1.5H+1.5HD+1.25E
E2	Sunny day with seismic	1.25D+1.5H+1.25E+1.0Q
Notes:		
D	Dead Load: Weight of concrete and water	
H	Hydrostatic Load: See each load case for headwater and tailwater conditions	
HD	Hydrodynamic Loads: Not applicable for this analysis	
E	Earth/Sediment/Silt Loads: Includes horizontal and vertical loads	
IM	Impact Load: Debris carried by flow, applied 150 mm below top of wall	
Q	Seismic Loads: Design Earthquake load – evaluation to consider simultaneous horizontal and vertical components for three combinations	
W	Wind Loads: Used for Strength Analysis only – Not Applicable to Stability	

8.6.6.3 Summary of Stability Analyses

The structural analysis resultants based on strength evaluation were used to perform a static stability assessment rather than using the rigid body method (conventional gravity method).

Table 31 shows a summary of the forces applied to the structure and the safety factors used for the foundation structure design. The summation of the vertical forces was compensated by the vertical resistance of the caissons and footing, resulting in a structure that is stable against floatation. The summation of the horizontal forces is compensated by the horizontal resistance provided by the bedrock in which the foundation will be installed in and it results in a structure that is stable for sliding.

This analysis is comparable to the stability analysis and the safety factors shown are a result of the design of the foundation to resist the forces applied to the structure. The safety factors are higher than CDA values since the design of the foundation was done using a strength evaluation method.

Table 31. Stability Summary

Load Comb.	Σ Vert. Forces (KN)	Σ Horiz. Forces (KN)	Σ Moments (KN*m)	Minimum Required Floatation FS	Floatation SF Calc.	Minimum Required Sliding FS	Sliding FS Calc.
U1	1177	148	-8	1.5	3.75	1.5	3.93
U2	1430	-77	-10	1.5	3.09	1.5	7.60
UN1	1498	150	-19	1.3	2.95	1.3	3.88
UN2	1390	-224	-12	1.3	3.17	1.3	2.60
E1	1739	-320	-15	1.1	2.54	1.1	1.83
E2	1020	191	-6.6	1.1	4.33	1.1	3.04

8.6.7 Strength

Strength evaluation of individual elements or members of the structure was used to verify member sizes based on application of factored loads as described in ABC with some adjustments for more severe conditions or loads not included in the ABC.

Reinforced concrete design of the foundation structure was performed according to Design of Concrete Structures, CSA A23.3-14 with the additional requirements of CSA's SEED Document – *Structural Design of Wastewater Treatment Plants-2018* that addresses service load conditions, water tightness, shrinkage and temperature reinforcement, and crack control. The SEED Document contains references to ACI 350M-06 for modifying CSA A23.3-14.

Structural steel design was performed according to Design of Steel Structures, CSA S16-14, and codes for welding, materials, and other pertinent references.

In general, structural analysis and design was performed using SAP2000 and Excel spreadsheets. The Three-Dimensional Finite Element Model (FEM) was used to evaluate multiple load combinations, identify stress concentrations, and generate shear and moment values for design of individual elements. The FEM was supplemented with manual calculations to verify/validate model results and where necessary, refine the analysis of individual elements. Based on model output, a combination of manual calculation and commercial software were used for strength design. Additional elements evaluated as part of strength design included joint detailing, equipment anchorage, and embedded parts.

The structural design calculations for the foundation and steel framing can be found in the Structural Design Report for the Debris Deflection Barrier, Appendix E.6.

8.6.8 Serviceability

Serviceability concerns with the Debris Deflection Barrier foundation relate primarily to concrete durability including limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction and other chemical attack. The same manual calculations, commercial software, or 3-D FEM used for strength evaluation were used to evaluate deflection and thermal growth, while design detailing and material specification were used to mitigate cracking and chemical attack.

Serviceability concerns with the Debris Deflection Barrier steel superstructure relate primarily to steel longevity, and ability to maintain and service the DDB, particularly following a flood event. Steel longevity considerations include paint or galvanizing coatings for wet and dry conditions, sealing of internal chambers to eliminate oxygen, or the use of weathering steel for the members. The maintenance and servicing of the Debris Deflection Barrier was addressed by using modular construction with standardized parts and fabrication using exposed connections for ease in replacement if damaged.

8.6.9 Construction Considerations

Construction specifications and details for the Debris Deflection Barrier will be furthered during Final Design. The following construction considerations are noted:

- Dewatering of excavated areas will be required to sufficiently enable construction of the Debris Deflection Barrier. The services of a specialist dewatering contractor may be needed.

- Excavation will be to competent bedrock. All soil, including alluvium, talus and other unconsolidated deposits should be removed to expose unweathered or slightly weathered bedrock. Excavation should be performed by mechanical means only; blasting will not be permitted.

- Foundation preparation will require special care in cleaning and preparation of concrete/rock interface. Care must be taken during excavation of the foundation to identify unsuitable rock conditions or weak bedding planes that could impact stability. Loose material and rock overhangs will need to be removed. Small voids will be filled with dental concrete.

- Shear keys are required to maintain adequate sliding stability. Care should be taken during excavation of the shear key trenches to identify unsuitable rock conditions or weak beading planes that could compromise capacity of the shear key.

- Concrete placement will require sequencing for construction of upstream intermediate walls between piles.

- Fill placement and compaction methods must be reviewed and monitored to ensure wall movement does not occur during construction.

- Construction sequencing will be required to ensure the Debris Deflection Barrier is fully functional before a tie-in with the Diversion Channel is made.

8.7 ELECTRICAL AND MECHANICAL CONTROL SYSTEMS

Electrical and Mechanical Control System information will be provided under separate cover.

9.0 DIVERSION CHANNEL

9.1 GENERAL

The Diversion Channel conveys flows northeast from the Diversion Inlet to the Off-stream Storage Reservoir. The channel alignment and grading are presented on Drawings C-310 to C-313. Channel alignment alternatives considered and basis for selection of the preferred alternative are presented in Stantec's Conceptual Design Update (Stantec, 2015a).

Diversion Channel design elements presented in this section include:

- the channel cross section, including erosion protection and stability;
- Highway 22 and Township Road 242 bridges;
- utility relocations;
- Emergency Spillway and discharge channel; and
- the Diversion Channel Outlet approach channel and grade control structure.

9.1.1 Design Objectives

The Diversion Channel is designed to convey and manage flood events up to the design diversion capacity of 600 m^3/s. (See Section 8.1.2) Erosion protection for the channel and embankment zones must be sufficient for full Project operations at completion of construction with or without full-vegetation establishment.

As part of the Extreme Hazard dam system, the IDF-OSSD governs design of the Diversion Channel including embankments and hydraulic structures.

9.2 CHANNEL

9.2.1 Arrangement

The typical channel cross section is trapezoidal with a 24 m bottom width. Side slopes are 3H:1V in soil and 2H:1V in rock. For certain sections, a 5 m wide bench is included at the soil/bedrock interface.

From Station 10+129 (Diversion Inlet) to Station 13+970, the channel invert slopes -0.1 percent. Ahead of Station 13+970, the channel invert slopes -0.2 percent. A 3.0 percent cross slope toward the channel center line is included on the -0.1 percent reach of the channel. This is to promote drainage of surface waters during periods of no diversion.

9.2.2 Freeboard

Freeboard for the Diversion Channel considered both the free-flow condition without influence of the reservoir and a backwater condition where portions of the channel function as part of the Off-stream Storage Reservoir.

For the free-flow condition, no Alberta provincial or CDA freeboard selection criteria was identified. In the absence of local criteria, a design freeboard of 1.9 metres was selected based on criteria from the USBR Design Standard No. 3 – Canals and Related Structures (1967). Appendix F.4 provides further information on the calculations.

For the backwater condition, freeboard criteria were calculated based on the CDA guidelines for dams and reservoirs. Required freeboard is 2.2 metres for the FoR and 1.5 metres for the PMF. Detailed descriptions of standards and methods for the Reservoir criteria are described further in Section 10.1.3.

Hydraulic modeling was performed to confirm the channel hydraulic capacity and demonstrate compliance with freeboard criteria. A one-dimensional hydraulic model of the Diversion Channel was developed in HEC-RAS Version 5.05. The model geometry includes the full length of the Diversion Channel, the Emergency Spillway side-channel weir and the Off-stream Storage Reservoir. Details regarding the development of the model are presented in Appendix C.7.

Water surface profiles were developed along the channel for three design conditions:

- steady flow design operation capacity (600 m^3/s) with no tailwater;
- unsteady flow routing of the FoR hydrograph with reservoir storage; and
- unsteady flow routing of the IDF-OSSD hydrograph assuming the Diversion Inlet gates remain open throughout the hydrograph and with reservoir storage.

The resultant water surface profiles were overlaid; freeboard applied; and minimum crest elevation of the channel side determined. Embankments were added on the descending right bank of the channel for areas where the local topography resulted in insufficient freeboard. The water surface profiles are presented in Appendix F.4.

9.2.3 Erosion Protection

The proposed Diversion Channel will be excavated into glacigenic soil materials comprising glacio-lacustrine units overlying glacial tills and bedrock comprising the Brazeau Formation (Station 10+000 to13+200) and the Coalspur Formation (Station 13+200 to 14+600). The bedrock units general dip to the east and comprise a regional-scale series of ridges with intervening valleys. The ridges are dominated by relatively durable sandstone, while the intervening valleys typically have a larger component of softer argillaceous units. These findings suggest that the native materials comprising the bottom and side slopes of the Diversion Channel will vary across the site from durable rock layers to more erodible fine-grained soils. Figure 31 shows the proposed Diversion Channel invert elevation. The top of bedrock is based on the geotechnical exploration (See Section 6.2) with interpolation between bore holes.

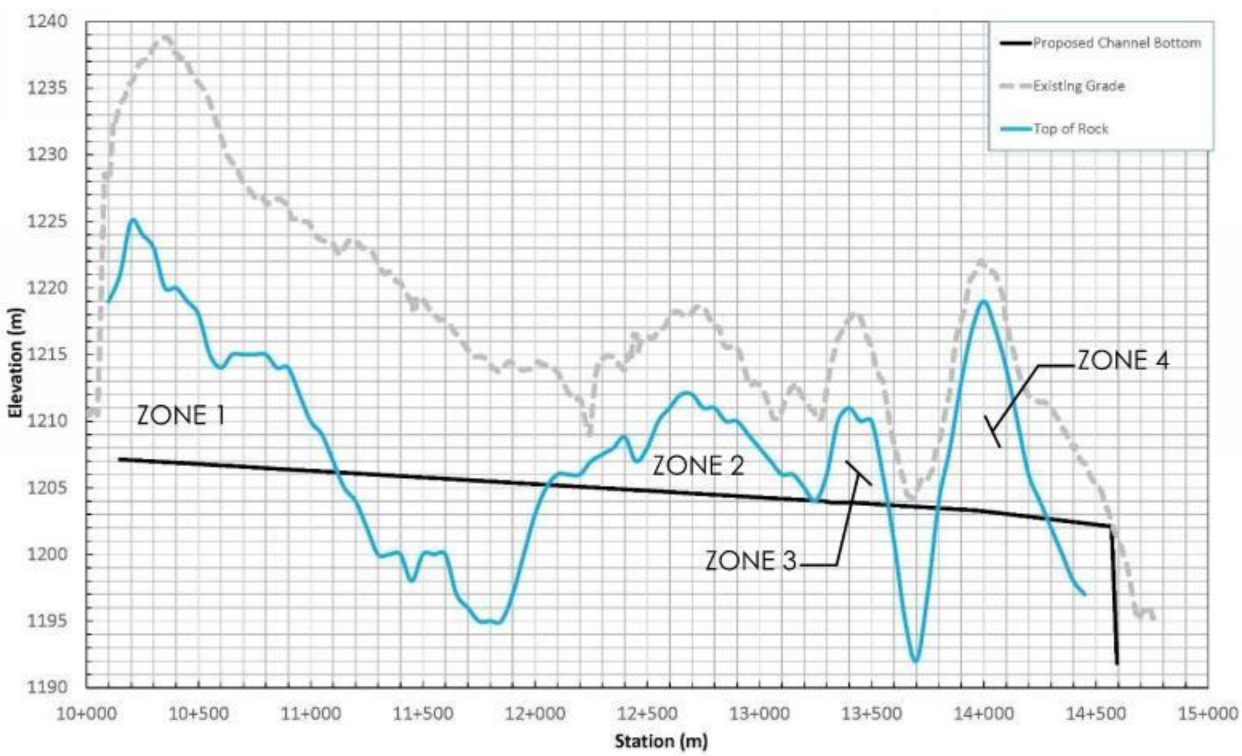


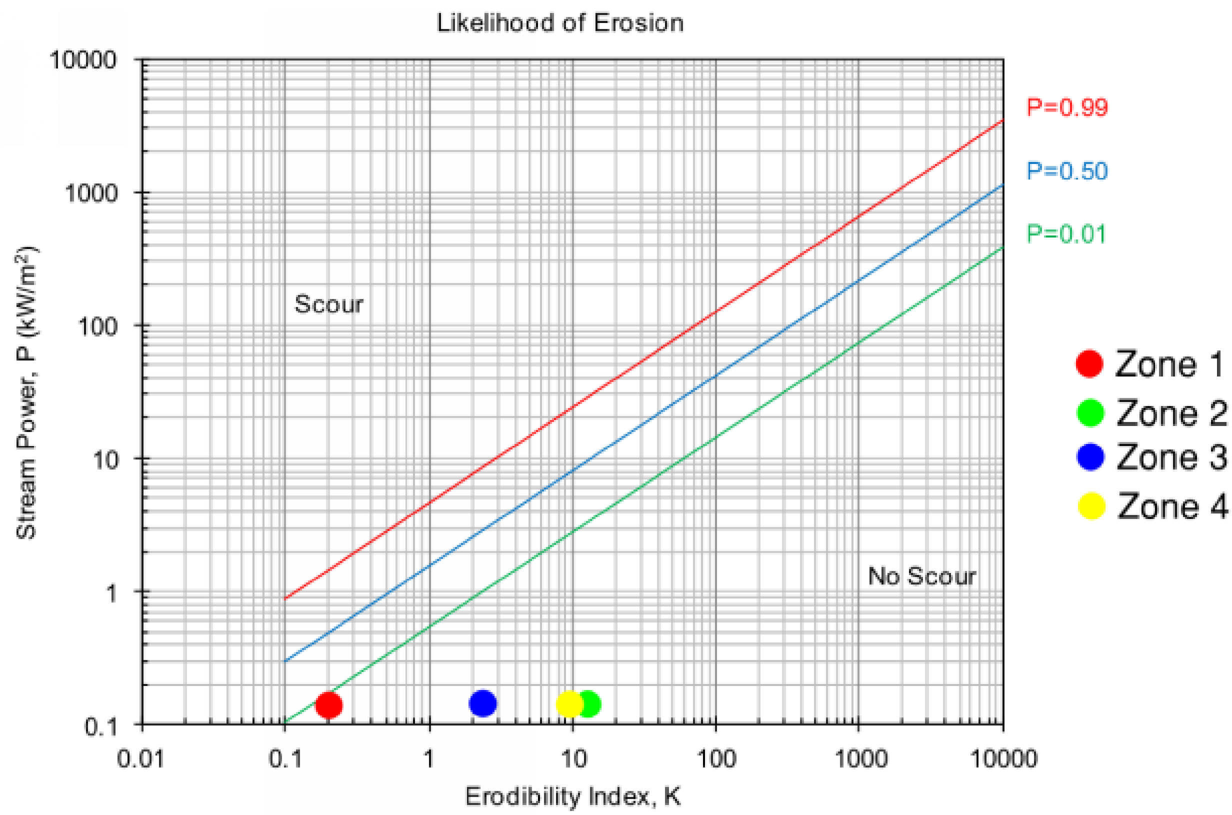
Figure 31. Diversion Channel Excavation

9.2.3.1 Bedrock Excavation Zones

Bedrock erosion potential was estimated using the Erodibility Index Method (EIM) developed by Annadale and Smith (2001). This method estimates an index value based on the rock mass characteristics and intact rock strength; which is subsequently compared against the estimated stream power for a specified design event.

For the erodibility index, four different rock zones were identified along the alignment of the channel. These zones are shown in Figure 31. Hydraulic analyses detailed in Appendix C.7 were used to calculate stream power for the design channel flow (600 m^3/s). EIM calculations are presented in Appendix F.4.

Based on the results of the EIM calculations, each of the identified rock zones demonstrate less than a 1-percent-chance of erosion for the design diversion flow rate of 600 m^3/s.



P = Probability of erosion for a given Stream Power and Erodibility Index

Figure 32. Rock Erodibility Index

The EIM does not consider the effects of weathering, freeze-thaw cycles, seepage and other long-term degradation processes on the durability of the surface. Potential ground water conditions may cause the channel to be saturated on a frequent or regular basis, which may increase the rate of weathering and reduce the channel surface erosion resistance. The geotechnical investigation indicates that Rock Zone 1 appears at a higher risk for continued long term weathering than the other zones. This may manifest as possible maintenance issues in the rock zone and this zone may lose surface erosion resistance with time.

Areas in rock cuts that experience weathering may be expected to see localized scour during operation; however, the underlain rock would remain in place. Over time, areas that weather and erode may need to be supplemented with aggregate or riprap to maintain channel grade and performance.

9.2.3.2 Soil Excavation Zones

Project design criteria require the channel to be functionally complete after two-years of construction and fully operable up to the design diversion rate at substantial completion. For this reason, the erosion resistance approach incorporates a multi-tiered strategy in soil zones.

Hydraulic analysis of the channel was used to calculate shear stress and depth average velocity for a range of flow rates including the design diversion rate (600 m^3/s) and the IDF-OSSD.

Erosion resistance for channel sections excavated through soil was evaluated based on methods published in Appendix F of Alberta Transportation's Erosion and Sediment Control Manual (AT, 2011c). The Erosion and Sediment Control Manual specifies that a grass mixture over easily eroded soils may withstand velocities up to 1.2 m/s. The HEC-RAS model, described in Appendix C.1, was then used to determine the horizontal variation in depth-averaged velocity across the channel cross section at certain channel stations for the design diversion rate. On average, portions of the channel 4 m above the channel invert are expected to have velocities less than the 1.2 m/s threshold for a channel discharge of 600 m^3/s.

For these locations, it is recommended that a temporary erosion control blanket be installed over topsoil and seeded. The erosion control blanket should have a minimum design life of five years and designed to withstand the expected shear stresses and velocities.

Riprap channel lining is recommended in the higher velocity portion of the channel below a flow depth of 4 m and for the full flow depth at critical locations along the Diversion Channel at structural risk from scour and erosion. These areas include utility crossings, bridge foundations, the Emergency Spillway and areas where embankments are needed to form the channel or provide adequate freeboard. At locations of embankment fill that retain the reservoir, the riprap design was checked for flow rates up to the IDF-OSSD, and increased if necessary to achieve a safety factor greater than 1.0.

Riprap was sized using the methods outlined in USACE EM 1110-2-1601 Hydraulic Design of Flood Control Channels (USACE, Rev 1994), with hydraulics information determined from the HEC-RAS model. These calculations are provided in Appendix F.4.

See Drawings C-310 through C-313 for the proposed channel lining along the Diversion Channel.

9.2.4 Slope Stability

The anticipated general ground conditions are presented in Figure 31. 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+200 m: excavation within GT and GL units
- Station 12+200 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

Slope stability analyses utilizing the limiting equilibrium modelling software, Slope/W (part of the Geostudio 2012® suite) were performed on a series of representative cross sections along the Diversion Channel. 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; and
- Phreatic conditions from the matching SEEP/W model were used in slope stability analysis.

The stability of the channel slopes was analyzed at seven cross sections. Detailed discussion of the inputs, methods and results are presented in Appendix D, Section 11. The Design Criteria loading condition and associated factor of safety for the Diversion Channel slope stability analyses is listed in Table 32. A brief discussion of results follows here.

Table 32. Recommended Design Load Case for the Diversion Channel

Load Case	Reference	Flow Depth in Channel	Foundation Behavior	Pore Pressures	Min Required FOS
Long Term	CFEM	None	Drained strength parameters	Measured/Inferred	1.5

9.2.4.1 Slope Stability Results

Typical channel excavations can be sorted into two groups: excavations with significant amounts of bedrock removal, and excavations primarily within soil units. As previously discussed, channel excavations into bedrock occur between Sta. 10+000 to 11+100 and 12+200 to 14+570. Excavations primarily within soil units occur between Sta. 11+100 and 12+200. Slope stability analyses were performed at seven cross sections between Sta. 10+000 and 14+570. At four cross sections (Sta. 10+150, Sta. 10+400, Sta. 11+000, and 14+000), slope stability analyses resulted in acceptable factors of safety, without any groundwater control measures. At three cross-sections (Sta. 11+400, Sta. 11+900, Sta. 12+400), groundwater control measures were required in order to achieve acceptable slope stability factors of safety. Note that sections requiring groundwater control measures are near the region where excavations are primarily within the soil units.

Two typical stability results are shown below. One shows the critical slip surface at Sta. 10+150, where stability results are acceptable, without groundwater control. The second shows the critical slip surface at Sta. 11+400, where normal conditions (i.e. no groundwater control), results in deficient factor of safety. Slope stability factor of safety at Sta. 11+400 is acceptable with groundwater control. Additional results are included in Appendix D.

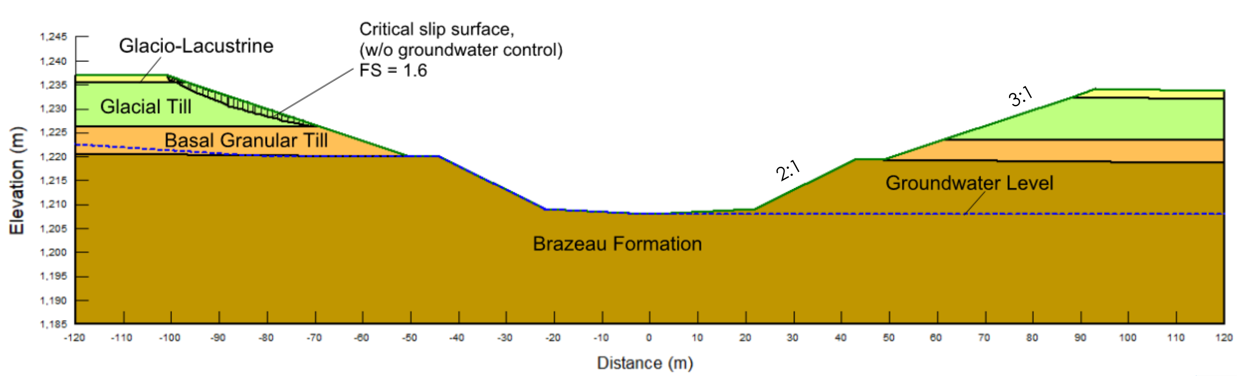


Figure 33. Stability results, without groundwater control measures, at Sta. 10+150

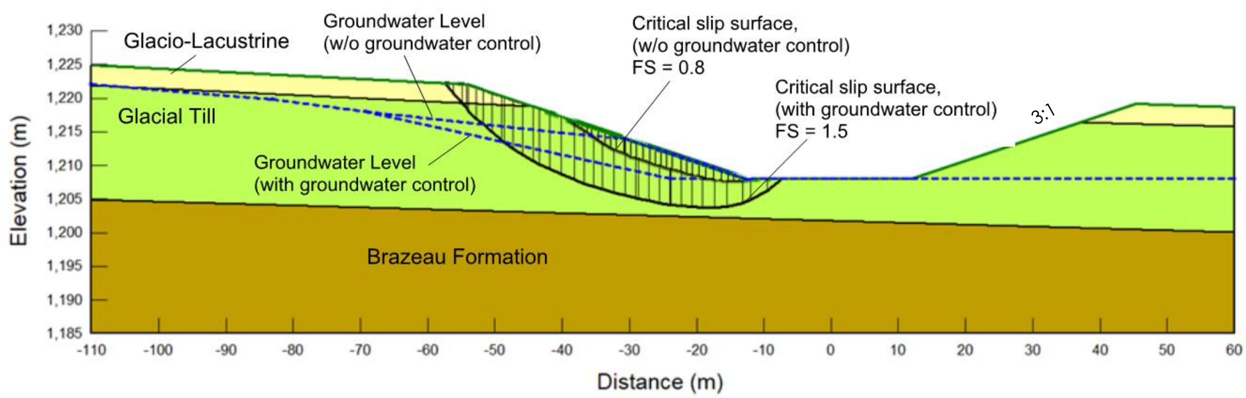


Figure 34. Stability results, with and without groundwater control measures, at Sta. 11+400

9.2.4.2 Groundwater Considerations

The slope stability analysis demonstrates that the existing and post-construction groundwater elevations will directly impact the slope stability of the Diversion Channel excavations in soil.

Groundwater control will likely 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 Brazeau formation 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, where the soil slopes are relatively tall with soil beneath the flowline of the channel are likely to require groundwater control. This may occur from Station 11+000 to 12+000; however, it may not be necessary at all locations. Groundwater control may also be required at other locations along the channel.

It is recommended that a series of piezometers be installed near the proposed upstream crest of the Diversion Channel prior to construction. These piezometers will be used to monitor the effects of the excavation on the groundwater level. Assumptions from the seepage analyses can then be verified during construction and groundwater control can be implemented in a more efficient manner.

Two potential groundwater control measures were assessed. The first involves a sand and riprap triangular drain at the base of the channel side slope. The second approach involves over-excavation of the channel side slope, installation of a 1 metre thick sand blanket and then replacement of the soil above to reestablish the channel side slope. Required extents of these water control features should be assessed on a case by case basis. Each approach produced a drop in the computed phreatic level and an increase in the slope stability factor of safety, with the larger sand blanket producing a greater improvement.

An assumed length of groundwater control, consisting of the rock toe option, is included in the Preliminary Design cost opinion for budgetary purposes and should be carried through Tender. The actual locations where groundwater control is necessary will be determined in the field during construction using observed slope conditions and piezometer data.

With the 3H:1V soil side slopes, there is increased risk that mitigation measures may be required in the future as groundwater changes, like those observed along other channels in Alberta. It is recommended that a monitoring program be in place during construction to anticipate problem areas and construct mitigation measures during the project to prevent future issues.

9.3 LOCAL DRAINAGE

The Diversion Channel will intercept storm water runoff from an 8.5 km^2 drainage area north and west of the channel alignment. Three local channels will be intercepted, with the largest entering the Diversion Channel near Station 12+200. Hydrologic analyses for these local inflows are presented in Appendix B. Hydraulic analyses and erosion calculations for the transitions into the Diversion Channel are presented in Appendix F.4. For the two smaller channels near Stations 13+075 and 13+274, runoff rates are small and the proposed grass and riprap channel lining is sufficient. For the larger channel at Station 12+200, a riprap lined transition channel is incorporated into the design.

9.4 HIGHWAY 22 AND TOWNSHIP ROAD 242 BRIDGES

The proposed Diversion Channel alignment intersects public roadways, Highway 22 and Township Road 242, requiring the construction of two bridges. Preliminary design reports for these bridges are provided as Appendix F.9. The bridges are designed with approximately 4 m of freeboard from the low chord elevation of the bridge to the calculated water surface elevation for the design channel discharge (600 m^3/s).

The proposed Highway 22 bridge will remain on the same horizontal alignment as the existing roadway. The profile grade will be raised approximately 160 mm over the channel. The bridge will carry two 3.7 m lanes of traffic, with 3.0 m shoulders for a total bridge width of 13.4 m, excluding the rails and barriers. A three-span girder arrangement will be used with the longest 30 m section centered over the channel so that hydraulic effects of the support piers are reduced.

The proposed Township Road 242 Bridge has a similar configuration and will remain on the same horizontal alignment. The proposed roadway width is 9.0 m on the bridge, excluding rails and barrier, which consists of two 3.5 m lanes and two 1.0 m shoulders. Similar to Highway 22, a 3-span arrangement with a 30 m center span is proposed.

9.5 UTILITIES

The proposed Diversion Channel alignment intersects multiple utility corridors including oil and gas product pipelines and overhead electric. These utilities will be removed and relocated prior to, or during, the construction of SR1. The following sections provide an overview of the utilities and the proposed relocation requirements.

9.5.1 Electric Transmission

The Diversion Channel alignment crosses a 138kV transmission line owned by AltaLink between Station 10+900 and 11+000. Two sets of wooden h-pole transmission towers are within the proposed excavation. The transmission towers will be relocated outside of the channel excavation and raised, as necessary to provide sufficient clearance for vehicle and construction traffic. Conceptual relocation is shown on Drawing C-142.

9.5.2 Oil and Gas Product Pipelines

The Diversion Channel crosses three oil and gas product pipeline corridors between Stations 11+000 and 12+000. Table 33 provides a summary of pipeline documentation. The proposed design, included on Drawings C-143 to C-144, shows realignment of the pipelines to provide 3 m minimum clearance between the pipe crown and channel invert. Final methods for installation are to be determined by the owner.

Table 33. Pipelines

Owner / Operator	License #	Status of Pipeline	Diversion Channel Station	Conveyed Medium	Outside Diameter	NEB Regulated
Caledonian Midstream	7850-23	Operating	11+055	High vapour pressure product	114.30 mm (4.50")	No
Alberta Ethane / Pembina Pipeline	14766-2	Operating	11+060	High vapour pressure product	168.30mm (6.63")	No
TC Energy / Nova Gas Transmission Ltd.	80096-28	Operating	11+890	Natural Gas	914 mm (35.98")	Yes
TC Energy / Foothills Pipeline Ltd	80006-3	Operating	11+910	Natural Gas	914mm (35.98")	Yes

9.5.3 Local Service Utilities

A number of local utility providers have distribution facilities within the SR1 project footprint. Local gas, electric, and communication lines that service individual properties and are impacted by the proposed construction must be abandoned or relocated. Final construction and abandonment methods are to be determined by the owner of each respective utility. Those include:

- ATCO Gas is the owner of the shallow natural gas distribution facilities within the SR1 project footprint.
- FortisAlberta Inc. (Fortis) is the owner of the electricity distribution lines within the SR1 project footprint. Fortis' infrastructure consists of overhead power lines that service individual properties and run along Springbank Road and Highway 22.
- TELUS is the owner of a majority of the telephone and internet cables within the SR1 project footprint. TELUS infrastructure consists of underground cables that service individual properties and run along Springbank Road and Highway 22.

- Shaw Communications Inc. (Shaw) owns a fiber optic cable that runs along the ditch on the east side of the Highway 22 corridor and services a property to the west.

The ultimate plans for the utility relocations are dependent on land acquisition and which customers require continued service once SR1 is in place. Where it is necessary to cross under the diversion channel, utility lines are proposed to be installed at least 3.0 m beneath the bottom of the diversion channel lining. For utilities that cross over the diversion channel, utility lines are proposed to be installed as overhead spans a minimum of 10 m above the edge of the channel.

9.6 EMERGENCY SPILLWAY

9.6.1 Design Objectives and Status

The Emergency Spillway provides a secondary emergency outlet in the event flows entering the Off-stream Storage Reservoir exceed its design storage capacity and the Diversion Inlet gates do not close. Under planned operating conditions, the Diversion Inlet gates close when the Off-stream Storage Reservoir water level reaches the FSL (Elevation 1210.75m), forcing flow to the Elbow River and bypassing the reservoir.

The design is at the conceptual level and will not be completed until further geotechnical explorations are advanced to confirm subsurface conditions. Three locations are considered, but for the purposes of this report Location No. 2 is presented on the drawings and cost opinion.

Following completion of the geotechnical program and confirmation of the preferred alternative, the design of the Emergency Spillway will be advanced to the Preliminary Engineering level and submitted under separate cover.

9.6.2 Conceptual Arrangement

The Emergency Spillway design concept is located along the Diversion Channel alignment. The Emergency Spillway consists of a 135 m-wide side channel concrete drop structure, a short riprap exit channel between retaining walls, and an excavated outlet channel, where the flow will continue to the Elbow River. The crest elevation of the drop structure overflow weir is Elevation 1210.75 m and the maximum design head of the Emergency Spillway is 1.25 m, or Elevation 1212.0 m, which correlates to the maximum design pool elevation in the Off-stream Storage Reservoir.

9.6.3 Alternatives Considered

The three alternative locations are shown in Figure 35 in plan view and Figure 36 in profile view along the Diversion Channel.

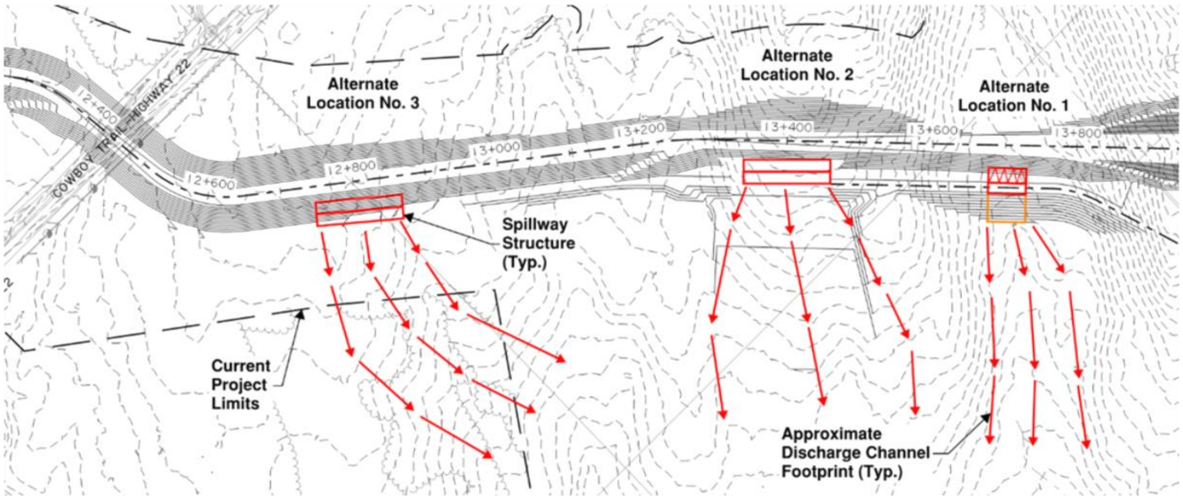


Figure 35. Three Proposed Alternate Spillway Locations

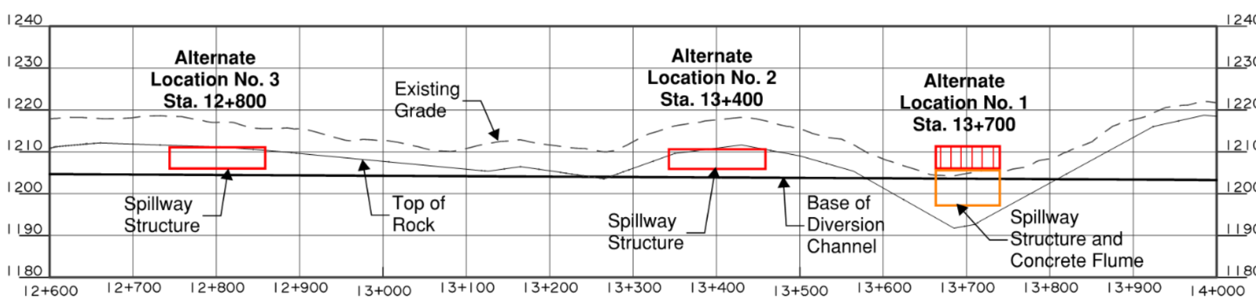


Figure 36. Profile of Diversion Channel Depicting Emergency Spillway Location Alternatives

Alternate Location No. 1 would be located primarily over existing soils and fill material that has a relatively high potential for scour and head-cutting. To reduce the overall width of the spillway structure, a labyrinth spillway was considered. A concrete chute would be required to safely transport flows away from the Emergency Spillway weir, the Diversion Channel embankment, and the Off-stream Storage Dam embankment.

Geotechnical explorations at Alternate Location Nos. 2 and 3 are proposed but have not been completed. Based on interpolation of borings drilled for the Diversion Channel, the Emergency Spillway outlet channel for these locations is anticipated to be primarily placed over existing bedrock. The bedrock is more durable than the native glacial till and clay materials, thereby being less susceptible to erosion and head cutting. Comparing these two locations, based on the information available from the geotechnical exploration, Location No. 3 appears to have more durable bedrock present than Location No. 2, and the structure and discharge channel will likely be cut deeper into the bedrock, providing side-slope armoring as well. However, the discharge channel at Location No. 3 is also at a steeper slope after the discharge channel cut daylights, resulting in higher velocities which balances head-cutting potential.

Performance of the recommended geotechnical program is required prior to selection of the preferred alternative and advancement of the design.

9.7 OUTLET

9.7.1 Design Objectives and Status

The Diversion Channel Outlet is designed to dissipate energy and expand flow prior to discharging into the Reservoir. The proposed design maintains velocities below 2 m/s within the Reservoir for a discharge of 600 m^3/s.

The design is at a conceptual design level and will not be completed until further geotechnical explorations are conducted to confirm subsurface conditions. A preferred alternative has been identified, and for the purposes of this report, is presented on the drawings and cost opinion. However, until the geotechnical exploration is completed, the design is subject to change.

Following completion of the geotechnical program and confirmation of the preferred alternative, the design of the Diversion Channel Outlet will be advanced to the Preliminary Engineering level and submitted under separate cover.

9.7.2 Conceptual Arrangement

The Diversion Channel Outlet provides a transition from the typical Diversion Channel cross section to the Off-stream Storage Reservoir. The Outlet includes a 960 m long, riprap-lined channel that gradually expands from a bottom width of 24 to 150 m, a stepped RCC grade control structure and stilling basin, and an unlined discharge channel which discharges into Unnamed Creek upstream of Range Road 41.

The riprap transition channel begins at Station 13+611 and ends at the RCC grade structure at Station 14+571. The channel expands through a series of decreasing Length:Width ratios. These are presented in Table 34.

Table 34. Diversion Channel Outlet Transition Section

Begin. Station	End Station	Begin. St. Width	End St. Width	Length	Transition Ratio
13+611	13+971	24	42	360	40:1
13+971	14+271	42	72	300	20:1
14+271	14+471	72	112	200	10:1
14+471	14+571	112	150	100	5.26:1

The RCC grade control structure drops from Elevation 1202.1 m to Elevation 1195.2 m at an average slope of 5.5H:1V over a series of 600 mm steps. The 12.5 m long stilling basin slopes at 0.5 percent downstream and 0.1 percent cross towards the center. At the end of the stilling basin, an end sill with a trapezoidal cut notch is located at the low point to facilitate drainage.

9.7.3 Alternatives Considered

Multiple alternatives were considered to meet the energy dissipation and flow expansion objectives listed above. Outlet components were varied including the approach channel slope, expansion ratio and length, channel lining materials, spillway type and stilling basin. Alternatives were compared in relation to their ability to meet project objectives, cost and susceptibility to debris impacts. The following considerations were noted:

- Approach channel slopes between 0.1 and 2 percent were reviewed. A steeper slope potentially eliminated the need for a step or drop structure; however, the channel lining requirements were prohibitive for sourcing and costs.

- Expansion lengths between 300 and 1000 m were considered. Shorter flare lengths resulted in a rapid drawdown of the hydraulic grade line which manifested as higher velocities. These velocities required larger and more expensive lining. Shorter flare lengths also result in less uniform distribution of flow across the grade control structure.

- Channel lining alternatives included riprap, RCC, soil cement, articulated concrete block (ACB), and A-Jacks. Hydraulically rougher linings, such as riprap or A-Jacks, resulted in greater energy dissipation and reduced spillway sizes. RCC or ACBs were less costly on an areal basis, but required greater coverage due to their smooth surface and larger energy dissipation structures.

The proposed design balanced the size and extent of channel lining (riprap) with the required spillway and stilling basin and provided the most economical solution.

9.7.4 Rip Rap Armoring

The approach channel riprap was sized using the methods outlined in USACE EM 1110-2-1601 (USACE, 1994) using results from the numerical model discussed in Section 4.1.2. Design calculations are provided in Appendix F.

9.7.5 Hydraulic Design

The RCC grade control structure was designed to pass a discharge of 600 m^3/s with an integral stilling basin designed to promote sub-critical flow into the Reservoir. Hydraulic calculations were performed using the methods outlined in Simplistic Design Methods for Moderate-Sloped Stepped Chutes (Hunt et al, 2014). These calculations are presented in Appendix F. Table 35 provides a summary of the results.

Table 35. RCC Grade Control Structure Hydraulic Design

Description	Unit
Design Flow	600 m^3/s
Unit Discharge	4.0 m^2/s
Design Discharge Head	1.6 m
Flow Depth at Toe	0.5 m
Max Velocity at Toe	7.7 m/s
Design Stilling Basin Length to Create Hydraulic Jump	12.5 m

10.0 OFF-STREAM STORAGE DAM

10.1 GENERAL

The proposed Off-stream Storage Dam (Dam) is a zoned earthen embankment approximately 3600 m long with a maximum height of approximately 30 m. The Dam forms a Reservoir of approximately 770 hectares at Elevation 1210.75 m (FSL). A Low-Level Outlet Works (LLOW) is provided to facilitate drainage of the reservoir and maintain flow of the existing Unnamed Creek. The Emergency Spillway discussed in Section 9.6 provides for a secondary discharge point should the Reservoir be overfilled. Drawing A-111 displays an overview of the Dam and Reservoir including the FSL and IDF-OSSD pool levels.

10.1.1 Design Objectives

To meet the project design criteria, the Reservoir must provide 70,210 dam^3 of storage volume. In order to provide for sufficient storage over the life of the project, a 10 percent increase in volume is proposed to account for potential sediment and debris accumulation. Finally, an additional 540 dam^3 was provided to account for anticipated local watershed inflow. The total cumulative design storage volume is 77,770 dam^3 which corresponds to the proposed FSL Elevation 1210.75 m. Table 36 summarizes the storage contribution and the corresponding reservoir elevation. Figure 37 provides the Stage-Volume-Area relationship for the Reservoir.

Table 36. Storage Dam Capacity Summary

Storage Source	Storage Volume (dam^3)	Cumulative Storage Volume (dam^3)	Stage Elevation (m)
2013 Storm	70,210	70,210	1209.78
Sediment (10% of 2013 Inflow)	7020	77,230	1210.66
Tributary Inflow	540	77,770	1210.75

Per the CDA Guidelines (CDA, 2013), the Dam and associated spillways are designed to safely pass the IDF for an Extreme Hazard facility (see Section 3.5) with sufficient freeboard from overtopping.

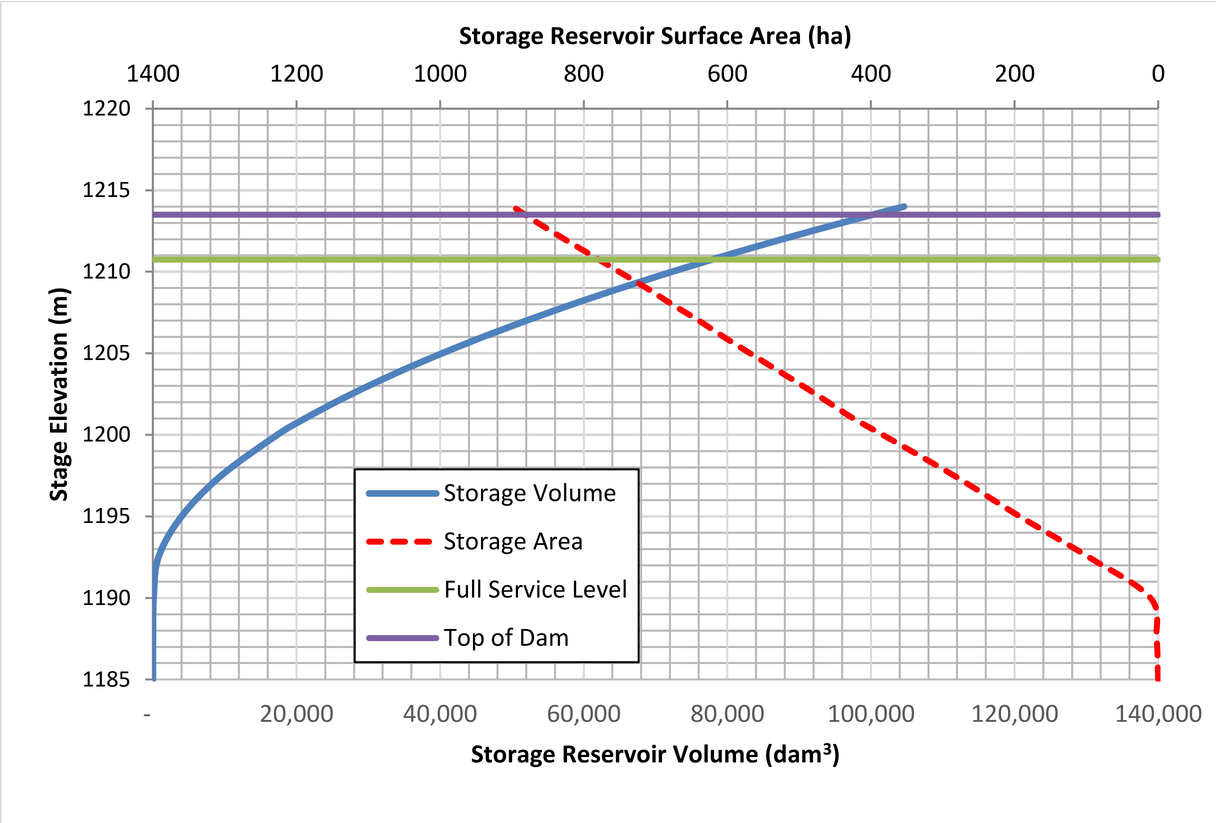


Figure 37. Reservoir Stage-Volume-Area

10.1.2 Reservoir Routing

Hydrologic and hydraulic modeling was performed to route certain design hydrographs through the Reservoir, LLOW and ES. Details regarding model development are provided in Appendix B.

Performance of the Reservoir and LLOW was assessed for a local 1:10 year, 24-hour storm over the direct drainage area to the site and assuming no diversion. This analysis is used for evaluation of the LLOW conduit for diversion of the Unnamed Creek during construction. Model development and results are presented in Appendix B. Inflow-outflow-stage hydrographs are presented as Figure 38. For the 1:10 year, 24-hour local storm, water surface levels will peak at 4.8 m above the LLOW invert with a pool area of approximately 40 hectares.

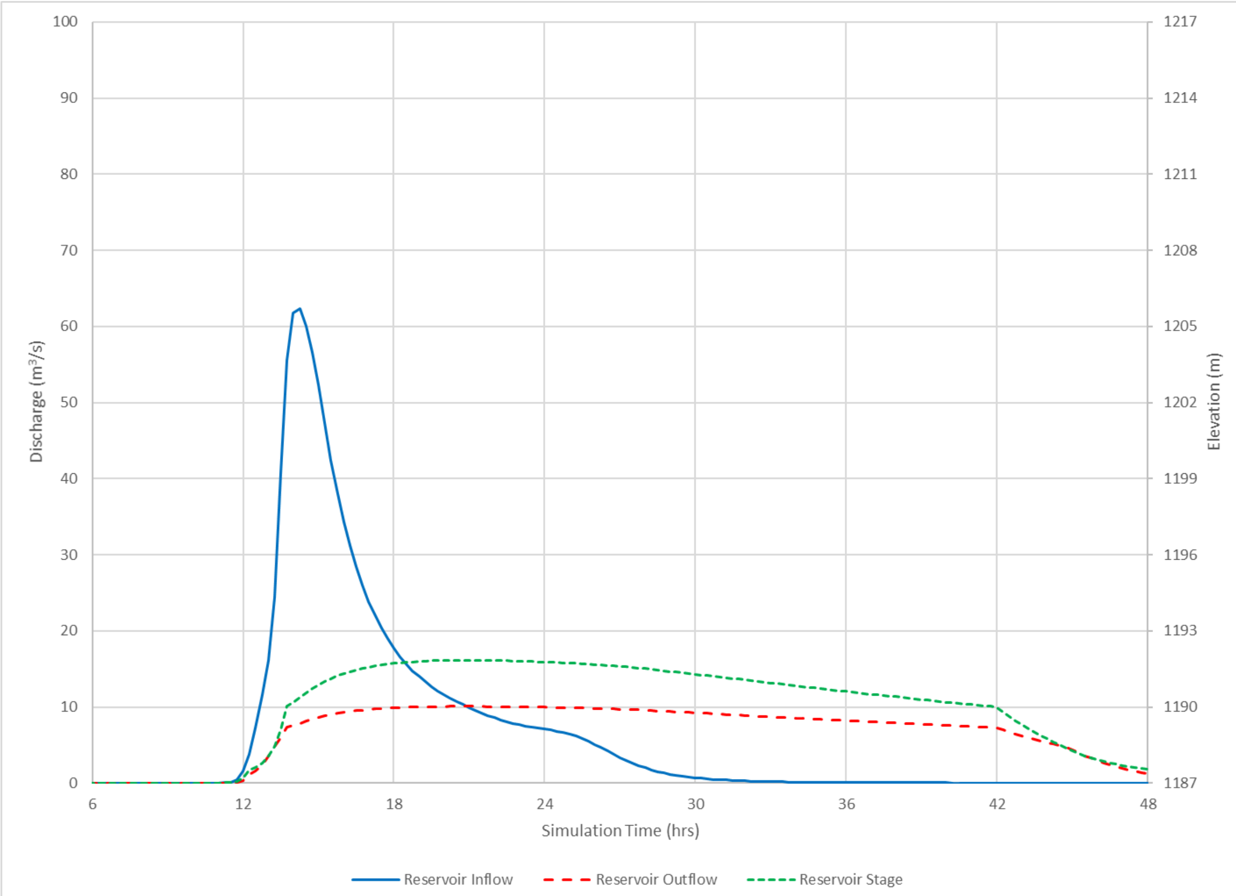


Figure 38. Reservoir Routing - 1:10 Year Local Storm

Storage capacity and FSL stage for the FoR was confirmed using the HEC-ResSIM model presented in Appendix B. Inflow from the Diversion Channel is based off the idealized operating conditions presented in Section 8.0 and illustrated in Figure 12. Model development and results are presented in Appendix B. Inflow-outflow-stage hydrographs are presented as Figure 39. The FoR simulation results in a peak reservoir elevation of 1209.8 m, approximately 0.95 m below the FSL.

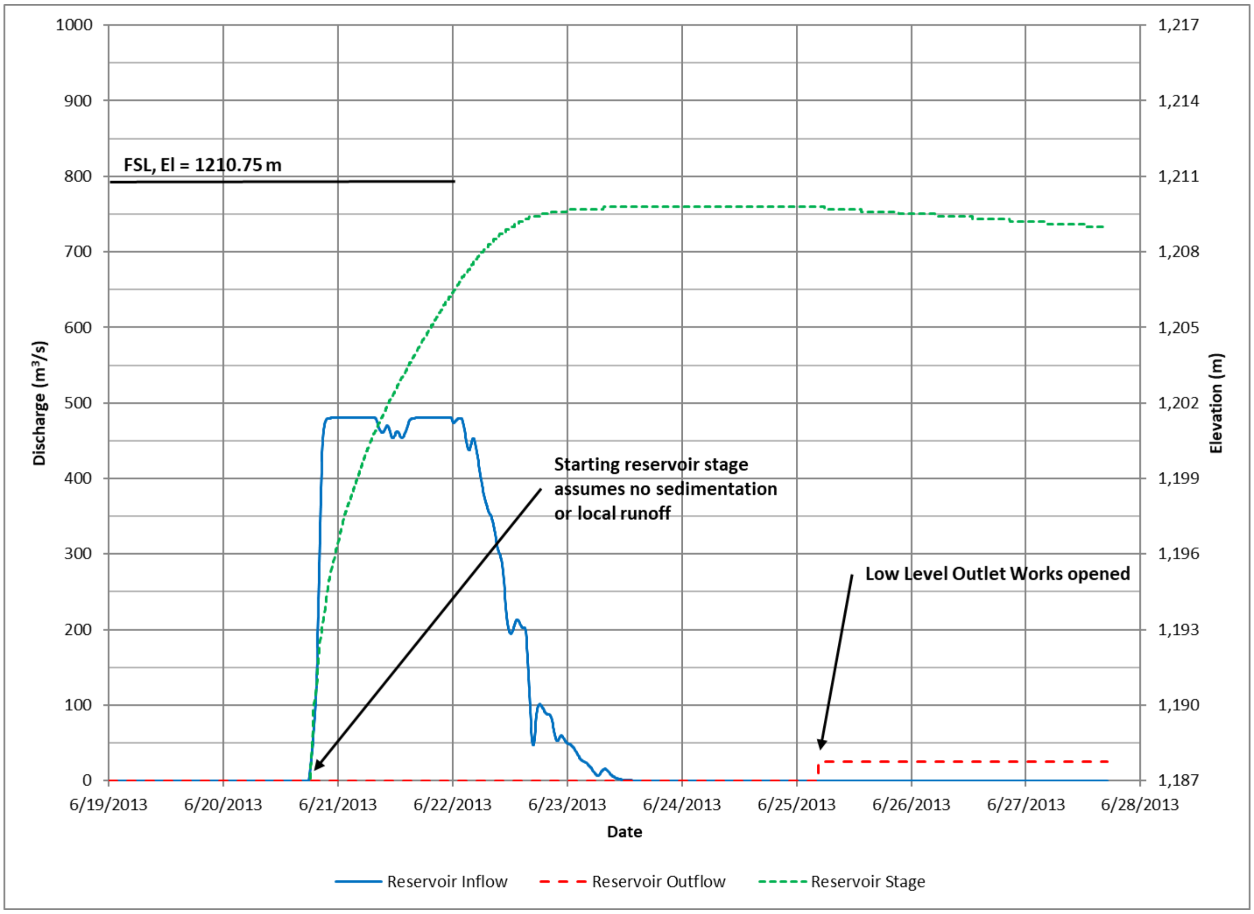


Figure 39. Reservoir Routing – Design Flood Operations (June 2013 Flood)

Proposed operations of the Project under the IDF-OSSD event were simulated in the HEC-Res Sim Model. For this scenario, operations rules follow those described in Section 8. Diversion begins under flood operation rules up to the FoR peak. As the flows in the Elbow River continue to increase, the Diversion Inlet gates are incrementally closed to limit diversion flows to 480 m^3/s. The gates are fully shut as the reservoir reaches maximum capacity. Inflow-outflow-stage hydrographs are presented as Figure 40. The IDF-OSSD operations simulation results in a peak reservoir elevation of 1210.6 m, approximately 0.15 m below the FSL.

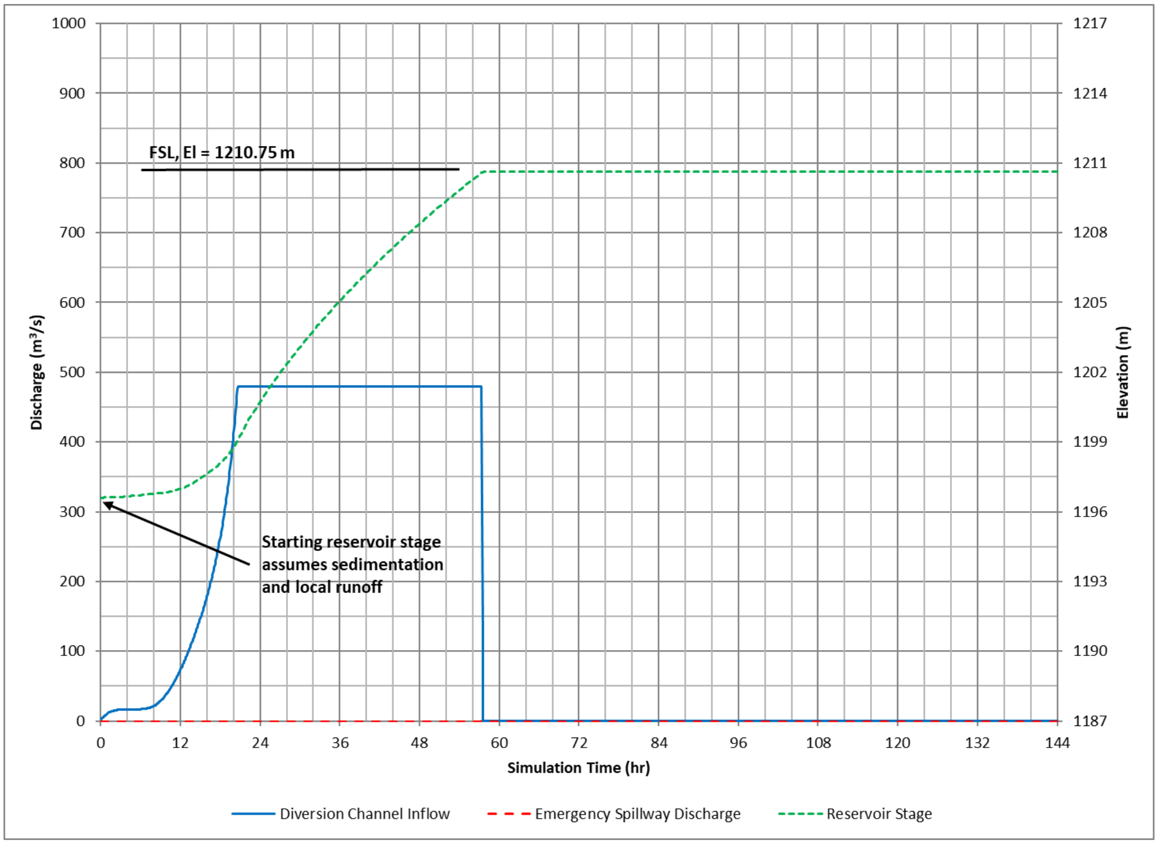


Figure 40. Reservoir Routing – Design Flood Operations (IDF-OSSD)

IDF-OSSD stage and performance of the Emergency Spillway was confirmed using the unsteady HEC-RAS model presented in Appendix B. For this scenario, inflow from the Diversion Channel is based on the Diversion Inlet gates remaining open during a PMF on the Elbow River. This scenario represents a failure condition of either operator error or gate system failure. Inflow-outflow-stage hydrographs are presented as Figure 41.

The IDF-OSSD simulation scenario without Diversion Inlet gate control results in a peak reservoir elevation of 1212.0 m. Flow through the Emergency Spillway initiates when the reservoir stage reaches the crest elevation of 1210.75 m and peaks at approximately 350 m^3/s under this scenario.

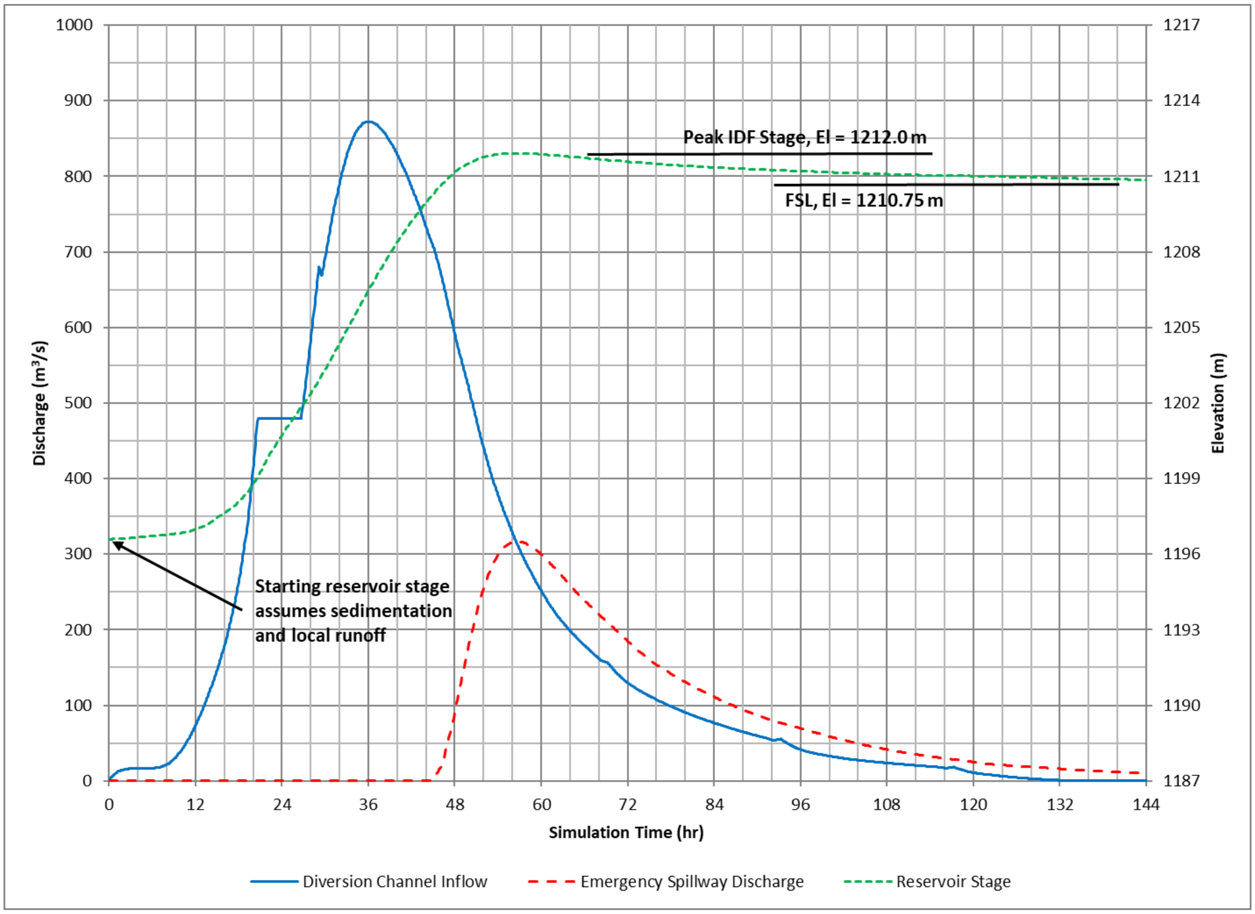


Figure 41. Reservoir Routing – PMF (Assuming Gates Remain Open)

10.1.3 Freeboard

Freeboard criteria for the Reservoir and Dam were determined based on the CDA Dam Safety Guidelines (CDA, 2013). Freeboard considered wind generated wave height, setup and runup, land-slide generated waves, hydrologic uncertainty and spillway / outlet works malfunction.

10.1.3.1 Wind Generated Waves

Two wind generated wave scenarios were evaluated. The Normal Freeboard scenario starts with the Reservoir at the FSL and considers the 95th percentile wave caused by wind with a recurrence frequency of 1:1000 years. The Minimum Freeboard scenario assumes the Reservoir is at its maximum elevation during passage of the IDF-OSSD event and the 95th percentile wave caused by wind with a recurrence frequency of 1:2 years.

Calculations for wind and wave run-up were performed using the methods outlined in USBR Technical Memorandum No. 2 (1981). Table 37 provides a summary of calculation parameters and results with detailed calculations provided in Appendix F.

Table 37. Normal and Minimum Freeboard Calculations Summary

	Normal Freeboard	Minimum Freeboard
Wind Velocity Return Interval (AEP)	1:1000	1:2
Design Wind Velocity (m/s)	29.0	24.5
Fetch Length (km)	4.80	4.80
Calculated Wave Runup (m)	2.12	1.42
Calculated Wave Setup (m)	0.13	0.04
Total Freeboard Required Above Pool Elevation (m)	2.25	1.46
Pool Elevation	1210.75	1212.00
Required Crest Height	1213.00	**1213.46**

10.1.3.2 Additional Considerations

In addition to wind generated waves, the CDA Guidelines on freeboard suggest consideration of factors related to uncertainty, malfunction and geotechnical performance including:

- Hydrologic Uncertainty: SR1 is designed to limit inflows to the Off-stream Storage Reservoir when it is full. The IDF-OSSD reservoir levels calculated are contingent upon gate failure (failure to fully close when reservoir is full). As such, the impacts of hydrologic uncertainty are appropriately addressed. No additional adjustments to freeboard are recommended for this item.

- Spillway and Outlet Works Malfunction: Flood routing calculations for the FoR and IDF-OSSD assumed the LLOW gates are closed. Malfunction of the LLOW will not affect freeboard assumptions. The Emergency Spillway is designed to pass diversion flows during the IDF-OSSD assuming mis-operation of the Diversion Inlet Gates (gates are not fully closed) and the Reservoir is full. As a passive, ungated structure, the Emergency Spillway is designed to prevent malfunction. With the debris deflection barrier in place, risk of large debris impacting the Emergency Spillways capacity is not significant. No adjustments to freeboard are recommended for this item.

- Earthquake and Landslide-Generated Waves: The reservoir area does not include landslide prone land, such as steep slope hillsides. Therefore, landslide-generated waves are not expected to occur.

- Embankment and Foundation Settlement: Settlement of the dam embankment and foundation is discussed in Section 10.3.7. Recommendations are provided to accommodate overbuild into the embankment construction to maintain minimum post-settlement freeboard. This overbuild is shown on the embankment profile on Drawings C-430 to C-432. Settlement monitoring of the embankment is recommended post-construction with additional material placed to maintain the required freeboard, if necessary.

10.2 RESERVOIR

10.2.1 Arrangement

The reservoir area at the IDF-OSSD level covers approximately 827 hectares. This area will be reserved for flood storage with uses restricted to temporary activities. Within this flood zone, existing buildings and utilities will be demolished and removed or relocated. Highway 22 and portions of Springbank Road and Township Road 244 near the Highway 22 intersection will be raised above the FSL. The remaining sections of Springbank Road and Township Road 244 will be allowed to overtop during certain flood storage events.

Proposed drainage within the Reservoir will generally follow existing patterns with the following exceptions:

- The drainage intercepted by the Diversion Channel will discharge into the existing Unnamed Creek just downstream of the Diversion Channel Outlet;

- Drainage from the upstream slope of the Dam will be collected and conveyed to the Unnamed Creek just upstream of the LLOW; and

- Approximately 470 m of the Unnamed Creek will be re-routed through the LLOW before discharging back into the creek downstream of the dam.

Mass grading within the Reservoir is not anticipated; however, certain areas may be used for borrow.

10.2.2 Transportation Infrastructure

The proposed FSL would result in existing sections of Highway 22, Springbank Road and Township Road 244 being overtopped during a flood storage event. More information can be found in Appendix F.

The vertical profile of Highway 22 will be raised up to 12 m above the existing road grade to accommodate the FSL elevation of 1210.75 m and provide at least 2 m between the road top of subgrade and FSL. The proposed cross section of Highway 22 will match the existing cross section consisting of one 3.7 m lane per direction and 3.0 m shoulders.

Approximately, 400 m of Springbank Road will be raised and reconstructed to meet the proposed elevation of Highway 22. Springbank Road is 9.0 m in width, with two 3.7 m lanes and two 0.8 m shoulders. West of Highway 22 is the existing Township Road 244. The road will be reconstructed for about 300 m to match the grade of Highway 22 at the intersection. A low point in the road is proposed approximately 240 m west of the intersection which coincides with existing roadway low point. The road is 8 m wide with a gravel finish surface as per Rocky View County Service Standards.

Refer to Appendix F for further information regarding the proposed roadway work.

10.2.3 Utility Infrastructure

10.2.3.1 Plains Midstream

Plains Midstream Canada (Plains) operates three pipelines that cross the alignment of the Off-stream Storage Dam between OSSD Stations 21+300 and 21+800. Those pipelines will need to be re-located from their current right of way as shown on Drawings C-150 and C-151. The proposed alignment for the oil pipelines moves it to the west to avoid the Off-stream Storage Dam while traversing beneath the Diversion Channel (near Station 14+000). In areas within the zone of influence of the foundation of the Off-stream Storage Dam (near OSSD Station 20+250) a minimum clearance of 30 m beneath the embankment is proposed. Final construction methods are to be determined by the owner. Below is a table of the information available for the Plains pipelines that was current as of December 08, 2016.

Table 38. Existing Pipeline Information

License #	Status of Pipeline	Conveyed Medium	Outside Diameter	H2S	NEB Regulated
26431-1	Abandoned – To Be Removed	Low Vapour pressure product	168.3 mm (6.63")	0 mol/kmol	No
5844-15	Operating – To Be Relocated	High Vapour pressure product, Crude Oil	323.90 mm (12.75")	0.32 mol/kmol	No
3084-1	Operating – To Be Relocated	Low vapour pressure product	114.3 mm (4.50")	0 mol/kmol	No

10.2.3.2 Local Utilities

A number of local utility providers have distribution facilities within the SR1 project footprint. Local gas, electric, and communication lines that service individual properties and are impacted by the proposed construction must be abandoned or relocated. Final construction and abandonment methods are to be determined by the owner of each respective utility. Those include:

- ATCO Gas is the owner of the shallow natural gas distribution facilities within the SR1 project footprint.
- FortisAlberta Inc. (Fortis) is the owner of the electricity distribution lines within the SR1 project footprint. Fortis' infrastructure consists of overhead power lines that service individual properties and run along Springbank Road and Highway 22.
- TELUS is the owner of a majority of the telephone and internet cables within the SR1 project footprint. TELUS infrastructure consists of underground cables that service individual properties and run along Springbank Road and Highway 22.
- Shaw Communications Inc. (Shaw) owns a fiber optic internet cable that runs along the ditch on the east side of the Highway 22 corridor and services a property to the west.

The ultimate plans for the utility relocations are dependent on land acquisition and which customers require continued service once SR1 is in place. In general, utilities will be relocated outside the inundation area for the reservoir and/or placed within the right of way of the revised Highway 22 corridor.

10.3 DAM

10.3.1 Arrangement

The Dam includes two zoned earthen embankments. The primary embankment located between Station 20+310 and 23+595 is approximately 3300 m long with a maximum embankment height of 29 m. A typical section of the embankment dam is presented in Figure 42. The proposed typical section consists of 3.5H:1.0V side slopes with 10-metre wide horizontal benches located every 10 vertical metres. A 6 m tall rock toe with a 10 m top width is added between Stations 21+750 and 22+750 to improve stability where foundation soils are deepest.

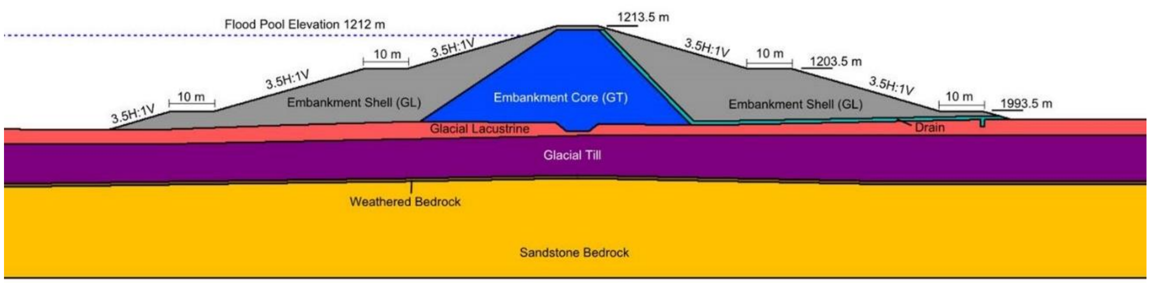


Figure 42. Typical Dam Section

The second portion of the embankment, referred to as the Saddle Dam, located between Station 19+784 and 20+182 is approximately 400 m long with a maximum embankment height of 11 m. The upstream face of the Saddle Dam forms the right descending bank of the Diversion Channel. A typical section of the Saddle Dam is presented in Figure 43. The proposed typical section consists of a 3.5H:1.0V side slope on the downstream side and a 3.0H:1.0V on the channel sides without benches.

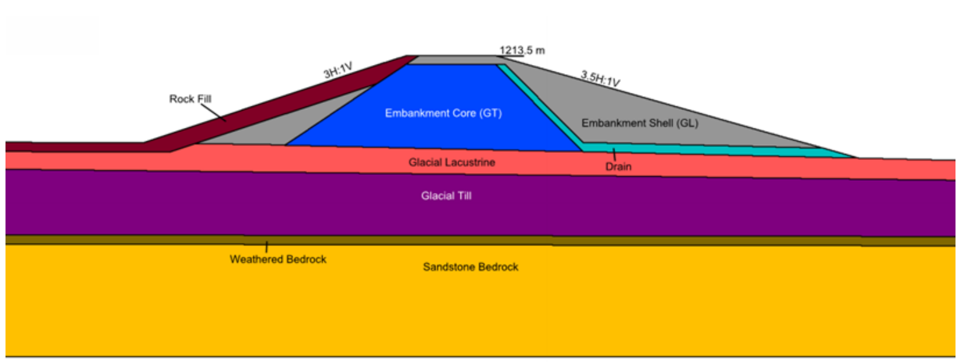


Figure 43. Typical Saddle Dam Section

The primary elements of the Dam and Saddle Dam include:

- Soil embankment with a low permeability core, granular filter/drain, and outer shell;
- Rock toes (upstream & downstream) from Station 21+750 to 22+750;
- Gravel access road on the dam crest;
- Surface drainage network including channels at the bench / slope interface, drainage flumes connecting the benches and conveyance channels at the upstream and downstream toe; and
- Subsurface drainage including a blanket drain and downstream toe drain along the length of the dam and a vertical drain located within the Unnamed Creek valley from Station 23+100 to 23+400.

The general arrangement of this structure is depicted on Drawing C-401 with detailed grading plans on Drawings C-410 through C-412.

10.3.2 Design Objectives

The Dam and its appurtenances are designed as an Extreme hazard facility in accordance with CDA Guidelines (2007) and Alberta Dam and Canal Safety Directive (2018).

10.3.3 Zoned Embankment

The interior of the dam consists of a low permeability core and exterior embankment shells. A drain is located on the downstream face of the core and along the existing ground. A vertical toe drain and key trench are also included. Between Stations 21+750 and 22+750 rock fill zones are provided at both toes and a granular layer is provided beneath the upstream embankment shell. These elements were provided due to anticipated pore pressure buildup in the foundation soils during construction. Details regarding pore pressures and impacts on construction are presented in Appendix D.

The earthwork materials described previously in Section 6.5 are applied to the zones as follows:

- Impervious Fill Zone 1A – Impervious Embankment Core and Key Trench
- Random Fill Zone 2A – Embankment Shell (Upstream and Downstream)
- Random Fill Zone 2A(3) – Rock Toes
- Fine Filter Zone 3A – Sand / Fine Filter Material

10.3.3.1 Impervious Core

The embankment core will be constructed using Impervious Fill Zone 1A. The top elevation of the low permeability core was selected considering steady or quasi-steady state reservoir levels and the potential impact of desiccation and freezing on the embankment. The minimum core elevation was set to Elevation 1212.5 m, just above the IDF-OSSD.

Some impact of desiccation and freezing is likely to extend as deep as 2 metres. The frost penetration criteria are discussed in Appendix D. Given the crest elevation of 1213.5 m, soil softening and desiccation cracks could extend into the upper section of the core. From the flood routings of the IDF-OSSD presented above, water levels above Elevation 1211.5 m would be sustained for less than 36 hours. During this period, the embankment would not saturate; however, water could flow through connected cracks in the embankment and core. Given the extreme nature of the flood, short time duration, limited head, and presence of the filter drain, progressive internal erosion is unlikely, and the proposed core elevation was deemed acceptable.

Supplemental specifications for Impervious Fill Zone 1A material were previously discussed in Section 6.5.1. Glacial till soil (GT) excavated from the Diversion Channel and Borrow Area 1 will be used as Impervious Fill Zone 1A. Most of the balance of the material excavated will be utilized to construct the embankment shells. This comprises glacial tills (GT), glacial-lacustrine (GL) and bedrock. There are significant areas of GL and localized GT which have LL > 50 percent and because they are expected to occur at elevated moisture contents will require special preparation (drying) and handling for use in the embankment.

The remolded strength and hydraulic conductivity parameters of the glacio-lacustrine soils are discussed in Appendix D.

10.3.3.2 Filter

Filter gradations were selected for the interfaces between the blanket drain and the core and foundation soils. The dam core will be composed of moderate plasticity clay (GT) and the foundation soils are comprised of low and high plasticity clay soil with USCS classifications CL and CH.

Filter design was performed in accordance with USACE procedures as published in EM 1110-2-2300 (1994). Gradations representing the dam core were obtained from testing of samples from Borrow Area 1 and the Diversion Channel. Gradations for the foundation soils were obtained from testing of samples from boreholes drilled along the dam alignment.

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. The filter gradation requirements are in Appendix D, Section 12.

Due to the very fine nature of the base soils, the filter criteria will require the filter gradation to be carefully controlled during construction. Available Filter Zone 3A sand may provide a suitable filter for the Zone 1A core material. The downstream foundation includes finer GL material and depending on tests on source material, the blanket drain may require a modified gradation specification. Filter criteria and calculations are provided in Appendix D.

The anticipated flow from the SEEP/W analysis for the typical section outside the Unnamed Creek is approximately 1×10^{-9} m^3/s. The capacity of the blanket drain was estimated to be approximately 1×10^{-7} m^3/s. The drain capacity exceeds the estimated seepage by two orders of magnitude.

10.3.3.3 Shell

The embankment shell will be constructed using Random Fill Zone 2A material as defined by the CWMS. Three subclasses of Random Fill Zone 2A are defined based on the materials to be sourced from the Diversion Channel and borrow areas.

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

Supplemental specifications to the CWMS Random Fill Zone 2A were discussed in Section 6.5. The moderately plastic glacial clay till should not be used as Random Fill 2A until specified Impervious Fill Zone 1A placements have been completed. Care should be used to avoid comingling of the three subclasses of 2A in order to meet placement and compaction requirements.

The remolded density, strength and hydraulic conductivity of these materials are discussed in Appendix D.

10.3.3.4 Slope Protection

Established turf and proposed drainage features will provide erosion protection. Maintenance to repair erosion rills will be required until grass is established.

The reservoir is a “dry” impoundment. As such there will not be a permanent pool and therefore wave wash protection was not deemed necessary. In addition, any flood pool would be a temporary condition. Erosion associated with wave action or pool drawdown may require grading maintenance or re-establishment of turf.

10.3.4 Storm Water Drainage

Drainage on the dam embankment is controlled with benching and down-slope rock-lined flumes. Benches are 10 m wide and placed at 10 m vertical offsets. A cross-slope of 2 percent concentrates flow at the bench and slope interface and a 2 percent slope along the profile directs flow to the flumes. The flumes are located at 400 m intervals and discharge into a vegetated drainage channel at the toe of slope.

The drainage feature capacity and erosion protection are designed for the 1:100 year local storm event. Calculations are provided in Appendix F.

10.3.5 Stability

Design of the exterior slopes of the dam is controlled by the presence of the GL deposits underlying the dam site. Excess pore pressures generated within these materials can be slow to dissipate, making the rate of embankment construction critical to meeting stability criteria.

Glacio-lacustrine deposits, such as those encountered at the site, have contributed to embankment foundation deformations and slope instability problems throughout Canada. The presence of these glacio-lacustrine units was accounted for within the slope stability evaluation through the use of laboratory testing based drained strength parameters and the use of excess pore pressure (B-bar & FEM) analyses. The embankment template was adjusted to provide appropriate computed factors of safety for planned embankment construction rates.

10.3.5.1 Stability Criteria

The Off-stream Storage Dam was designed according to the guidance of the Canadian Dam Association and other industry references.

Seven load cases were considered in the slope stability analyses. Each load case considered a different combination of soil strengths, pore water pressures, surcharge loads, seismic loads, and/or geometry in order to assess the performance of the dam. These load cases include:

- Non-operation long term load case, estimated long-term phreatic surfaces were used for pore water pressure input and drained (effective stress) shear strengths were used.

- The USBR operational design flood case, where drained shear strengths are used, and pore water pressures are taken from a steady-state seepage analysis at the flood pool.

- The USACE operational design flood case, where undrained shear strengths are used, and pore pressures are taken from the estimated long-term phreatic surface. A surcharge load, representing the hydrostatic force of the flood pool, is also modeled.

- The rapid drawdown analysis, where a multi-stage analysis considers drained and undrained shear strengths, using pore water pressures from phreatic surfaces of steady state seepage at flood pool and estimated long-term phreatic surfaces. The rapid drawdown methodology follows Duncan and Wright (2005).

- The pseudostatic analysis, where undrained shear strengths (reduced to account for potential strength loss due to cyclic loading) were used, and pore water pressures were represented by the estimated long-term phreatic surface. The seismic loading is applied as a horizontal force.

- The post-earthquake analysis, where pore water pressures were taken from the estimated long-term phreatic surface. Undrained shear strengths (reduced to account for potential strength loss due to cyclic loading) were used.

The Factor of Safety criteria used for the stability analyses are provided in Table 39.

Table 39. 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, pore pressures from B-bar or FEM analyses	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

10.3.5.2 Methodologies

The stability analysis was performed on six cross sections using the methodology discussed in Appendix D. The cross section location and selection process are described in Appendix D. The first five cross sections have similar foundation soils (glacio-lacustrine underlain by glacial till) with changes in foundation soil thicknesses, overall dam height and number of benches. The sixth section is located in the Unnamed Creek and has different foundation soils (glacial till underlain by fluvial creek deposits). This section is also the tallest dam section. Specific notes for each cross section are included in Table 40.

Table 40. Analyzed Cross Sections

Cross Section (Baseline Station)	Notes
Station 20+050	Represents the saddle dam from Station 19+800 and 20+200. Dam is up to 11 metres tall with foundation soils between 3 and 6 metres thick. Foundation consists of glacio-lacustrine underlain by glacial till. Weathered sandstone is present at top of bedrock.
Station 21+050	Represents the segment of dam from Station 20+300 to 21+500. Dam is between 8 and 13 metres tall with foundation soils between 3 and 6 metres thick. Foundation consists of glacio-lacustrine underlain by glacial till. Weathered sandstone is present at top of bedrock.
Station 21+750	Represents the segment of dam from Station 21+500 to 21+750, Station 22+750 to 22+980 and Station 23+400 to 23+600. Dam is between 13 and 20 metres tall with foundation soils between 8 and 14 metres thick. Foundation consists of glacio-lacustrine underlain by glacial till. Weathered sandstone is present at top of bedrock.
Station 22+500	Represents the segment of dam from Station 21+750 to 22+750. Dam is between 13 and 20 metres tall with foundation soils between 8 and 18 metres thick. Foundation consists of glacio-lacustrine underlain by glacial till. Weathered sandstone is present at top of bedrock.
Station 22+990	Represents the segment of dam adjacent to the LLOW conduit trench, from Station 22+980 to 23+060. Dam is between 20 and 24 metres tall with foundation soils between 12 and 15 metres thick. Foundation consists of glacio-lacustrine underlain by glacial till. Weathered sandstone is present at top of bedrock.
Station 23+175	Represents the segment of dam in the Unnamed Creek from Station 23+100 to 23+400. Dam is between 24 and 29 metres tall with foundation soils between 7 and 10 metres thick. Foundation consists of glacial tills underlain by fluvial deposits (sands and gravels). Weathered sandstone is present at top of bedrock.

The material parameters used in the stability analyses are discussed in Sections 5 and 6 of Appendix D.

10.3.5.3 Construction Period Stability

A large portion of the storage dam embankment foundation footprint contains a significant zone of glacial lacustrine clay soil. This soil material is well known in the Canadian Prairie to create issues with low strength and high pore pressure response during - and for a period following - embankment construction load application. Accordingly, the SR1 Storage Dam foundation soil was evaluated for likely response to construction loading.

The construction load cases were evaluated using two different approaches. The first was a traditional total stress analysis using laboratory triaxial undrained strengths. The second method was an effective stress analysis using drained strengths and estimated pore pressure increases due to loading. Additionally, two methodologies were used to estimate the pore pressure response, the Simplified B-bar Parameter method and the Finite Element Method.

Pore pressure response in the foundation soils will vary depending on the amount and rate of load placement, the thickness, strength, compressibility and permeability of the soils involved and the geometry of available drainage paths for pore pressure dissipation. The development of pore pressure predicting models is relatively involved and is covered in detail in Appendix D, Section 12. A brief description of the two methods is provided in the following sections.

10.3.5.4 B-bar Analysis (Construction Period)

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. Additionally, B-bar is generally used to predict dissipation of the excess pore pressure between the time of load application and the time of interest for the analysis.

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 (cv) was also considered. Additionally, documented case histories from dams constructed on lacustrine / alluvial soils in the Canadian Prairie region were reviewed to compare the selected SR1 pore pressure response with the excess pore pressures measured in similar conditions.

The foundation soils were divided horizontally in order to apply different B-bar values to different foundation materials. Different B-bar values were applied based on where they are located under the dam and when material placement during dam construction will end. The dam was divided vertically to split the material into year one, year two and year three construction seasons. Figure 44 shows a typical section with the soil horizon zones and associated B-bar values.

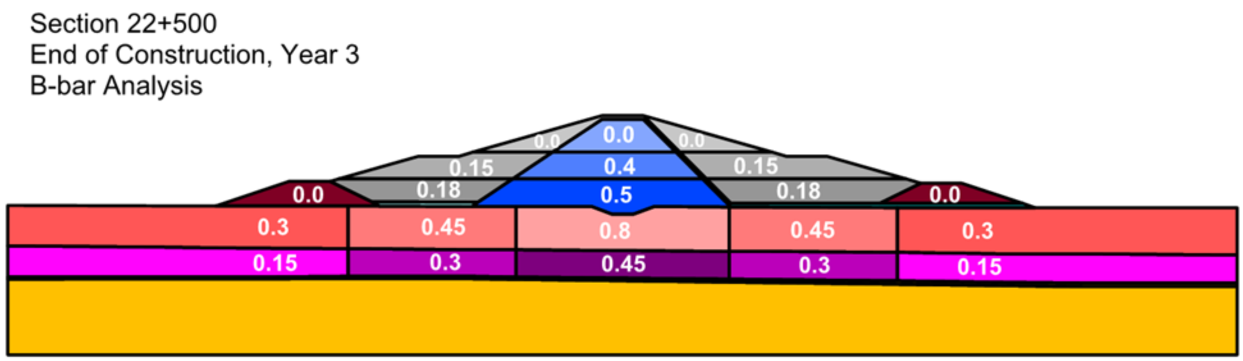


Figure 44. B-Bar Analysis Typical Section

10.3.5.5 Finite Element Analysis (Construction Period)

To provide additional information about likely pore pressure response a second method was utilized to estimate pore pressures. This method utilized a finite element model to apply and dissipate excess pore pressure over time. The Plaxis software platform was chosen to perform the computations. A typical image of a Plaxis analysis section is provided below as Figure 45. While the Plaxis software has the capability to model soil strength and compressibility, for this application the Plaxis model was used only to characterize the direction and rate of pore water pressure dissipation. The pore pressures obtained from the PLAXIS analysis were then mapped to limit equilibrium slope stability analyses, completed within GeoStudio's Slope/W. A detailed discussion of the constitutive model selection and model input parameters is included in Appendix D. To model the pore pressures, the analysis was divided into 1-month time steps and the appropriate amount of soil embankment loading was added at each time step. The FEM code then computed pore pressure buildup, redistribution and dissipation as successive time steps occurred. This computational process continued through the winter shut down without the addition of more embankment loading.

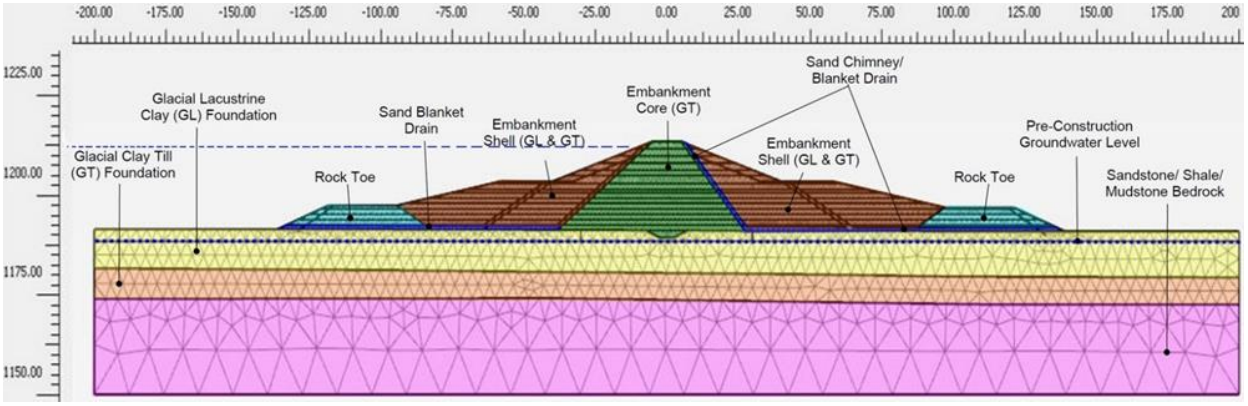


Figure 45. PLAXIS Model Configuration

10.3.5.6 Construction Period Shear Strength

To better characterize the undrained shearing response of the glacial lacustrine clay (GL) soil a program of Direct Simple Shear (DSS) testing was performed on undisturbed samples collected from the embankment footprint. The strength of the GL soil was then characterized with a strength ratio, or the S_u/σ_v' ratio developed from that testing program, in conjunction with the estimated pore pressures. Details of this laboratory testing program are provided in Appendix D. The shear strength was computed for numerous points across the GL layer for each analysis time of interest. A detailed presentation of the pore pressure response and shear strength computations is included in Appendix D.

10.3.5.7 Slope Stability Results

The results of the slope stability analysis for each load case for the proposed dam cross sections are presented in Table 41. The design of the dam was controlled by the long term, seismic post-earthquake, and flood load – USACE load cases, as well as the GL end of construction cases.

Table 41. Stability Analyses Results – Recommended Embankment Section

Load Case	Section	Factors of Safety	
		Upstream	Downstream
End of Construction – Undrained Analysis (Target FOS = 1.3)	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

Table 40. Stability Analyses Results – Recommended Embankment Section (Continued)

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.

The results show that the proposed Off-stream Storage Dam meets the required criteria for all load cases, with the exception of pseudostatic. A pseudostatic factor of safety less than 1.0 is not a failure criterion in itself, but indicates the need to perform a deformation analysis. The deformation analysis was conducted, and results show that expected deformations (up to 230 mm) are below the accepted threshold of 1 metre. Detailed discussion of the deformation analysis is included in Appendix D.

10.3.6 Seepage

The slope stability analysis requires pore water pressures for computing effective stresses, as defined for the specified load conditions. For each load case, the results of the seepage analysis from SEEP/W with the appropriate pool was used to model the pore pressures in the stability analysis. The seepage analysis required to create the appropriate models prior to running the slope stability analysis of the same six cross sections is discussed in this section. The seepage analysis was conducted following the methodology discussed in Appendix D.

The Off-stream Storage Dam will function as a "dry" dam with no permanent pool for normal conditions. However, seepage analyses were performed to determine the steady-state phreatic surface at the IDF-OSSD pool elevation (1212 m) for the USBR flood pool analysis. The headwater pool was modeled as a head boundary condition at Elevation 1212 m. The tailwater pool was modeled as a head boundary condition to replicate the fall in the groundwater from the dam location to the Elbow River water elevation. To reduce edge-boundary effects, seepage model extents were expanded at least 100 m beyond the downstream toe of the dam slope.

10.3.6.1 Seepage Exit Gradients

The exit gradients from the SEEP/W model were evaluated. Assuming steady state conditions, the critical exit gradients at the toe of the storage dam were assessed. The factor of safety against piping due to heave was then calculated. The results are presented in Table 42 below. Plots from SEEP/W presenting the results of the seepage analyses and the exit gradient calculations are included in Appendix D.

Table 42. 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 steady-state conditions, adequate FOS are likely for piping due to heave.

10.3.6.2 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 likely that hydraulic conductivity may exist between the fluvial materials and the reservoir, which could result in unacceptable factors of safety against piping. To mitigate against this, seepage control measures were evaluated. 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 beneath the dam. 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:

- A 2-metre thick seepage blanket covering the ground surface for some extent upstream of the dam.
- A key trench excavated to bedrock, replacing the fluvial soils with low-permeability compacted clay.
- Vertical drain under the downstream toe to provide seepage relief from the fluvial materials to the horizontal blanket drain
- Pressure Relief System consisting of wells or a trench drain extended through the fluvial materials 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.

Modifications to the proposed design were developed to control potential under seepage. The vertical toe drain was moved towards the center of the dam and extended one metre into the fluvial materials. A relief well / French drain 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 46.

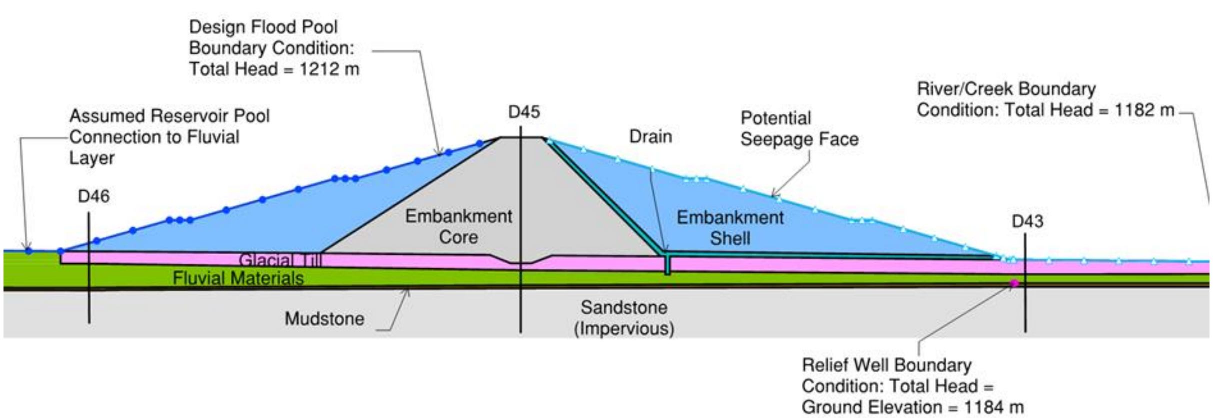


Figure 46. Unnamed Creek Foundation Treatment Seepage Model

As noted in Table 42, 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.

Piezometers should be installed in the fluvial materials and glacial till near the Unnamed Creek to monitor pore pressures during operation. During the startup test filling (and other significant filling of the reservoir), both the piezometers and downstream pressure relief system should be monitored and analyzed to determine if there is a significant direct connection to the alluvial materials from the pool, and whether the drain systems adequately control the downstream exit gradients.

Outside the limits of the Unnamed Creek, the dam cross section includes a 1.5-metre deep vertical toe drain 6 metres from the downstream toe.

10.3.7 Settlement

Settlement calculations were performed at ten embankment centerline locations between Station 20+600 and 23+400 where the embankment fill thicknesses are expected to vary from 6.6 m to over 29 m. 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. Approximately 200 mm of settlement is estimated to occur within the embankment fill at the tallest section (Station 22+600). Prorated settlement values within the embankment were used for the other dam sections.

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 proposed overfill for the 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.
- Embankment fill settlement was prorated to shorter embankment sections.
- Total settlement estimates were increased by ten percent to account for variability in foundation soil conditions and consolidation test results, and uncertainty with these types of estimates.

Recommended embankment overfill values were established based on the final settlement estimates. A generalized overfill profile for embankment design is presented in Figure 47.

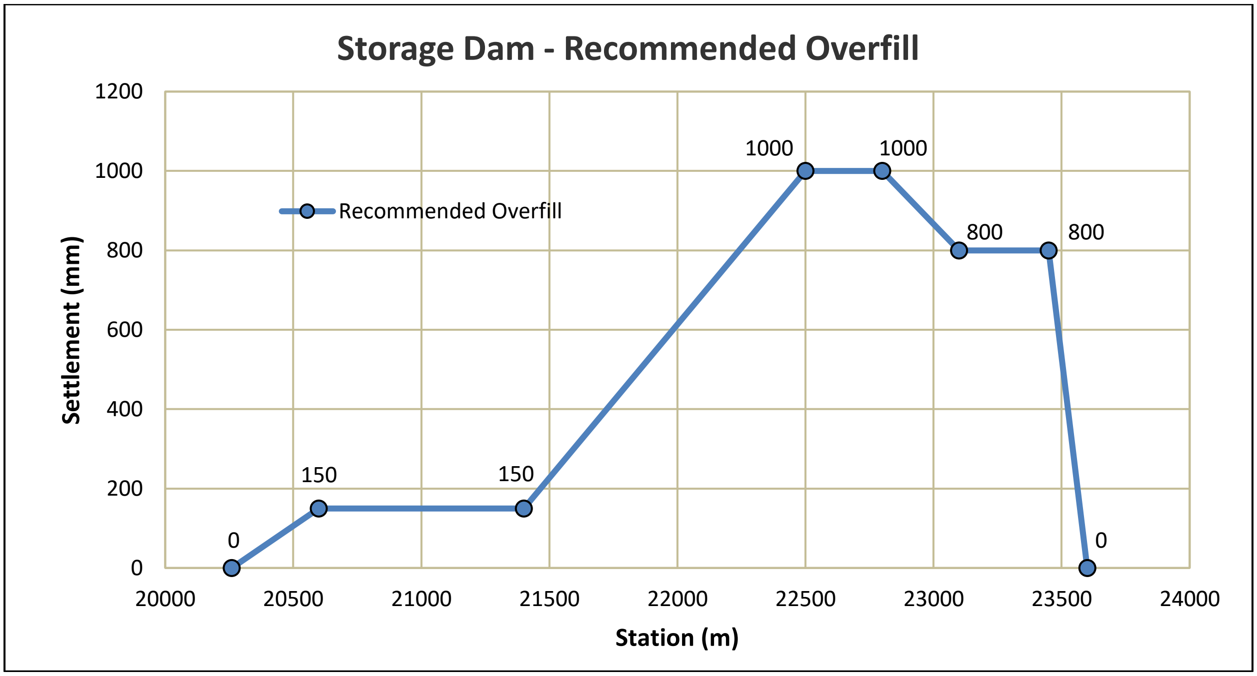


Figure 47. Recommended Storage Dam Overfill

10.3.8 Construction Considerations

The following items have been identified as construction considerations for the Dam:

- Foundation preparation will require the stripping and removal of topsoil and zones of soft and saturated subgrade and regrading existing slopes prior to earthfill placement. Slopes will be flattened to 4H:1V parallel to the dam centerline and 4H:1V perpendicular to the dam centerline.
- The piezometers and depth to water encountered in each borehole indicate that generally groundwater is sufficiently deep below the ground surface to not have a large impact on the construction of the dam. However, occasional areas where depth to water was as close as 1 m to the ground surface were encountered in the lower elevations of the dam foundation. The contractor should be prepared to control groundwater when excavating for foundation preparation, if necessary.
- The geotechnical performance of the earthfill dam should be monitored throughout construction with an instrumented dam safety management system. This should comprise of vibrating wire piezometers, standpipes, slope inclinometers / ShapeAccelArrays, sondex settlement gauges, settlement plates and laser scanning. Measured performance that does not conform to the expected behavior of the dam may require design reviews and potential modifications to the dam geometry or construction sequence.

- The rate of earthfill placement and subsequent pore pressure response in the foundation units and lower earthfill layers should be monitored throughout active and inactive construction periods. Elevated pore pressures that are slow to dissipate due to embankment construction loading were assessed in the stability analysis. Piezometers should be installed in the GL and GT foundation soils and lower portions of the embankment to monitor the increase in pore pressure in relation to the added load. If the pore pressure increases are greater than those estimated in the analysis and/or displacement is recorded in slope inclinometers, the rate of construction may be modified or other contingency measures, such as toe berms may need to be incorporated to provide adequate factors of safety against slope instability during construction.

- The dam construction sequencing should be planned to account for anticipated weather conditions. The earthfill cannot be placed and compacted when frozen or outside the permitted moisture content range. It is assumed embankment placement will occur in the warmer, dryer months (May through October). The stability analyses assumed three construction summer seasons and that pore pressures would partially dissipate during the intervening winter breaks. The earthfill around the Low-Level Outlet Works cannot be constructed until the cast-in place conduit is complete. Depending on schedule, this may require this zone of the embankment to be completed in two seasons. GL foundation soil replacement with GT soil is anticipated if this occurs, however, increased monitoring of piezometers will be required. If pore pressures do not dissipate at the rate assumed in the analyses, the rate of construction may need to be reduced.

- The dam will be constructed using material excavated to form the Diversion Channel and select Borrow Areas with the reservoir area. To maximize re-use of fill and reduce wastage, this should be undertaken with a modification to the CWMS requiring plasticity and particle size limitations. The Random Fill Zone 2 has been modified to account for the anticipated durable and non-durable rock which will be excavated from the diversion channel. Special care will be required to sort the excavated material for use in the various zones of the dam.

10.4 LOW-LEVEL OUTLET WORKS

10.4.1 General Arrangement

The Low-Level Outlet Works (LLOW) is a gated gravity drainage structure located to the southwest of the existing Unnamed Creek and constructed through the native foundation materials at Dam Station 23+022. Primary elements of the LLOW include:

- Excavated, riprap lined, approach channel, approximately 330 m long with a 0.5 percent slope, from the Unnamed Creek to the intake structure located at the embankment upstream toe.

- Reinforced concrete intake structure incorporating seven vertical 2800 mm by 2800 mm and one horizontal 2450 mm by 2450 mm trash rack panels.
- Reinforced concrete pressure conduit 1800 mm in diameter with a length of 64 m set on a 1 percent slope from the intake structure to the gate structure.
- Gate structure with a 1200-mm-wide by 1600-mm-high sluice guard gate and 1200-mm-wide by 1600-mm-high sluice regulating gate located in separate wet wells in series with electrically powered, mechanical operators.
- Reinforced concrete 2400-mm-wide by 2400-mm-high modified basket handle shaped gravity conduit with a length of 181.5 m set on a 1.8 percent slope running through the embankment dam foundation from the gate structure to the CSU rigid basin.
- Reinforced concrete, 17-m-long, CSU rigid basin located at the downstream end of the conduit and downstream toe of the embankment to provide at-grade energy dissipation of flow releases.
- Excavated, riprap lined, exit channel, approximately 765 m long with slopes varying from 0.5 to 2 percent, from the CSU rigid basin to the Unnamed Creek.

The general arrangement of the LLOW is depicted on Drawing S-170 with details for each component shown on Drawings S-400 through S-460.

10.4.2 Design Objectives

The LLOW has two primary design objectives to be satisfied:

- Pass normal stream flow from the Off-stream Storage Dam local watershed without creating a permanent pool upstream.
- Drain the Reservoir from the FSL within specified drawdown time.

The Canadian Dam Association (CDA) Guidelines (2013) and the Alberta Dam and Canal Safety Directive (2018), do not address requirements for sizing of outlet works or evacuation times for reservoirs. In the absence of Provincial and Federal governing criteria, two sets of criteria for evacuating reservoirs from the USBR and USACE were reviewed. The USBR criteria are specified in ACER Technical Memorandum No. 3 – Criteria and Guidelines for Evacuating Storage Reservoirs and Sizing Low-Level Outlet Works, (USBR, 1990). The USACE criteria are described in ER 1110-2-50, Low-Level Discharge Facilities for Drawdown of Impoundments, (USACE, 1975). The USBR criteria are the most recent and include the USACE criteria along with additional considerations; therefore, USBR criteria were followed.

The USBR criteria for determining emergency evacuation periods are based on a combination of hazard and risk classifications. The USBR defines hazard as the consequence of having an adverse event and risk as the probability of occurrence of an adverse event. Both hazard and risk are each assigned a rating of "High", "Significant", or "Low" according to a listed set of criteria. The combination of these two factors then defines the evacuation criteria.

Given the Extreme Hazard classification and infrequency of planned operations, a High-High hazard-risk combination was selected. Table 43 provides the suggested drawdown criteria.

Table 43. Reservoir Drawdown Criteria

Evacuation Stage	Days
75% Hydraulic Height	10-20
50% Hydraulic Height	30-40
25% Hydraulic Height	60-80
10% Storage Volume	40-50

10.4.3 Alternatives Considered

Alternatives for the LLOW included two alignments and two gate structure positions. The alignments included: Alignment A, located at Station 23+205 aligned within the Unnamed Creek valley; and Alignment B, located at Station 23+022 and positioned within an excavation of the soil overburden.

Foundation conditions were the primary differentiator between the two alignments. Alignment A was underlain by varying depths of fine-grained soils overlying granular deposits, while Alignment B has a relatively uniform foundation located within glacial till. Given the irregularity of the Alignment A foundation Alignment B was selected as the preferred alternative.

Two gate structure positions relative to the dam cross section were evaluated including upstream of the dam toe and mid-slope.

The mid-slope tower is recommended based on the following considerations:

- Cost: The mid-slope tower is approximately $900,000 to $1,500,000 (10%) less than the upstream toe option for either alignment.
- Accessibility / Public Safety: The mid-slope tower requires a shorter pedestrian bridge with lower fall heights.

- Visual / Aesthetic: The mid-slope tower is incorporated into the embankment and will appear better integrated into the dam, while the upstream toe location would feature more prominently on the landscape.

10.4.4 Hydraulic Design

10.4.4.1 Hydraulic Sizing

The LLOW hydraulic design is governed by the drawdown criteria established in Section 10.4.2. Based on the selected criteria, 90 percent of the reservoir volume is to be evacuated within 40 days, an average discharge rate of 20 m^3/s.

Initial drawdown scenarios were modeled to determine a preliminary size of the hydraulic control section and calculated a maximum discharge rate at the FSL of 27 m^3/s. Hydraulic design calculations were then developed for five elements of the LLOW based on a design discharge of 27 m^3/s and design head at maximum storage pool, Elevation 1210.75 m. The calculations are documented in Appendix C.6 including:

- Intake structure calculations to size the openings and trash rack spacing.
- Head loss calculation to size the circular pressure conduit, and sluice gate.
- Conduit sizing calculations to select the appropriate gravity conduit dimensions.
- Energy dissipation calculation to size the structure at the downstream end of the LLOW.

Design calculations for the intake structure, pressure conduit and gate structure were developed following guidance presented in USACE *EM 1110-2-1602 Hydraulic Design of Outlet Works* (USACE, 1980) and USBR *Design of Small Dams* (USBR, 1987). Head losses were calculated through the trash rack, inlet transitions, circular pressure conduit, and gate to select a 1200-mm-wide x 1600-mm-high sluice gate, which produces the design discharge of 27 m^3/s at the maximum storage pool elevation.

The gravity conduit section was developed based on normal depth open channel flow within a closed conduit, with maximum depth limited to 75 percent of the conduit height (USBR *Design of Small Dams*). The depth criteria was to limit the potential for the conduit to become pressurized due to the narrowing of the semicircular crown, to allow sufficient space for air to travel within the conduit for two phase flow, and to minimize undesirable flow conditions such as slug, plug and bubble flows. The slope was selected to produce supercritical flow in the conduit so that a hydraulic jump would not form, with the maximum slope limited by intake and exit channel inverts.

Design calculations for the energy dissipator were based on guidance presented in U.S. Federal Highway Administration's *HEC 14: Hydraulic Design of Energy Dissipators for Culverts and Channels* (FHWA, 2006). A CSU rigid boundary basin, streambed level dissipator was selected. Calculations were completed for a design discharge of 27 m^3/s, resulting in a structure 10.0 m wide and 17.0 m long.

10.4.4.2 Rating Curve

A discharge rating curve was developed for the LLOW based on the hydraulic design. The rating curve for the structure with the gate fully open is presented as Figure 48.

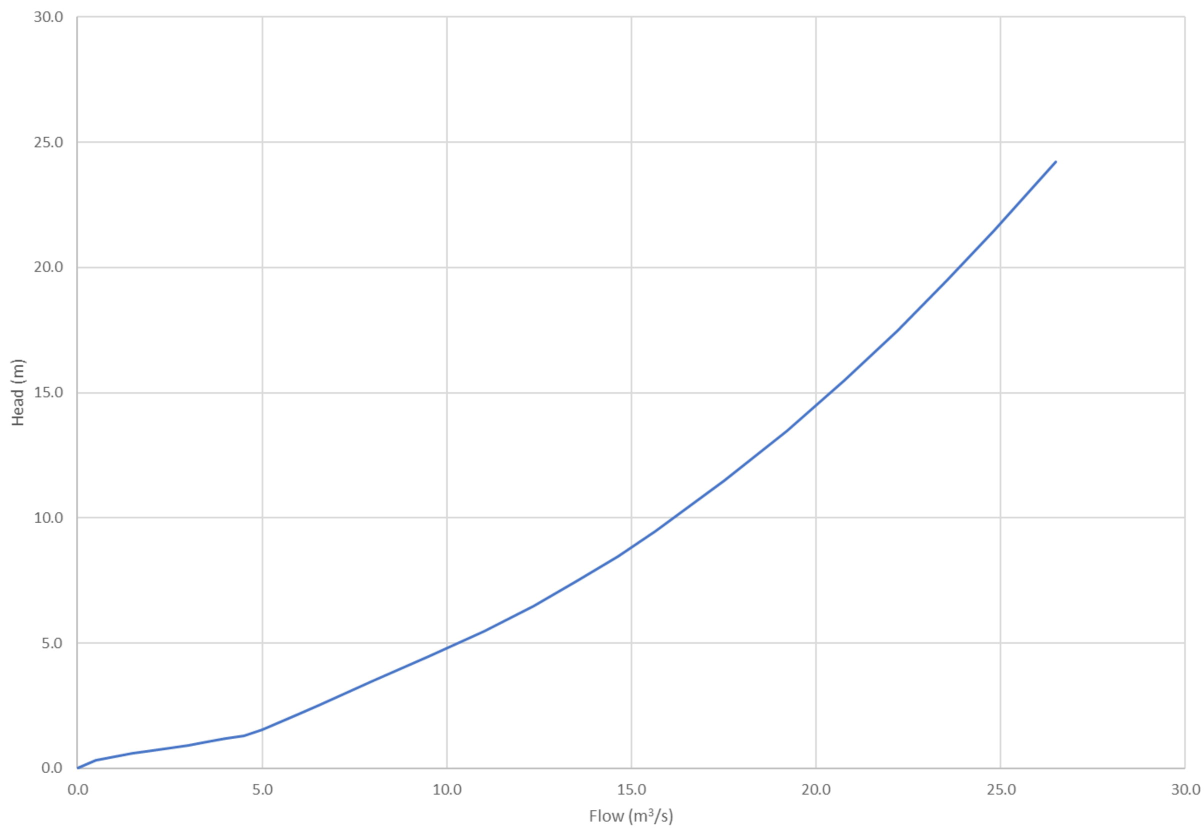


Figure 48. LLOW Rating Curve

Figure 49 presents the reservoir drawdown analysis with the gates fully open. Design levels and durations are identified to demonstrate compliance with the selected criteria.

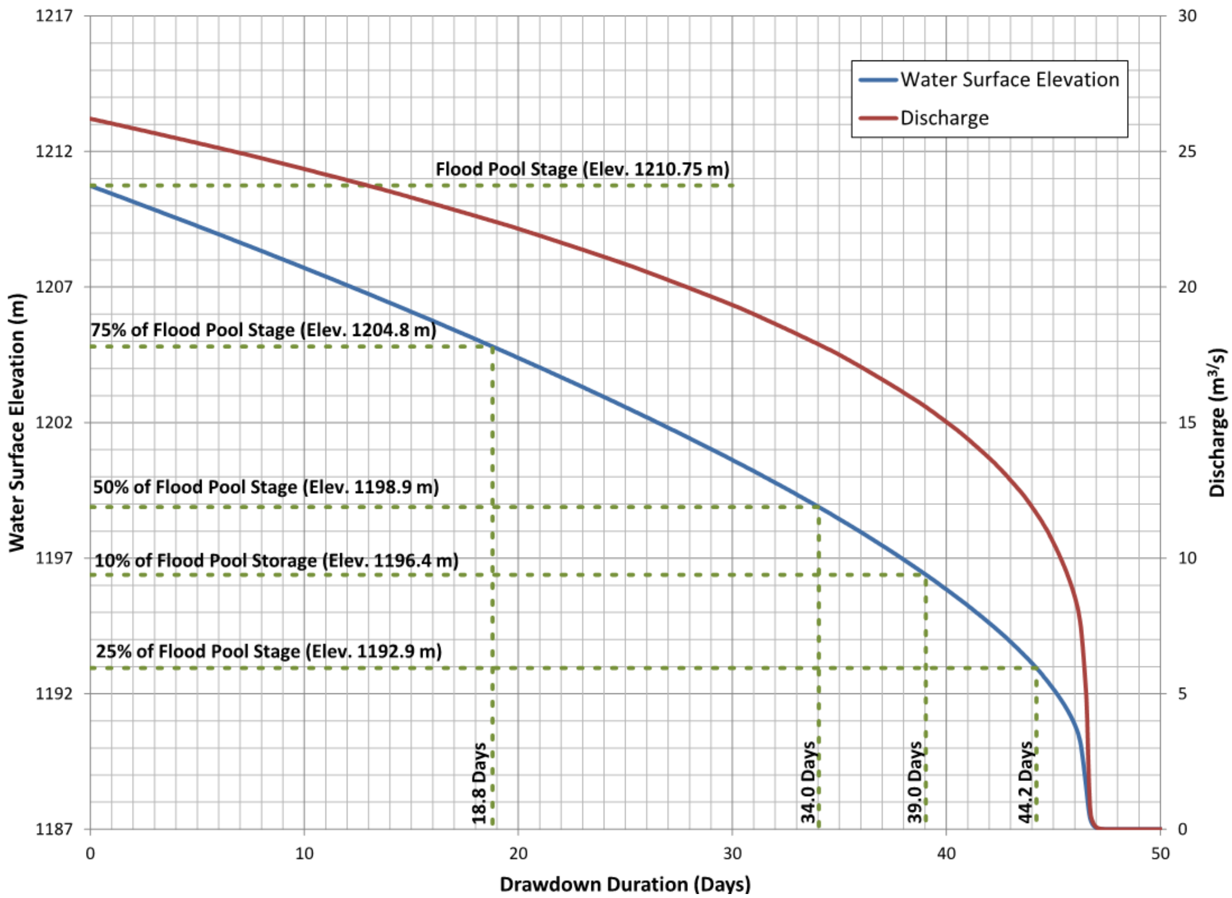


Figure 49. LLOW Drawdown from FSL

10.4.5 Approach and Exit Channels

The LLOW Approach Channel is approximately 330 m long with a 0.5 percent slope and will convey flows from the Unnamed Creek to the Intake at the embankment upstream toe. The channel is riprap lined on the bottom and for 1.5 vertical metres up the side slopes, there is a five-metre-wide bench on the right descending side of the channel at the limit of the riprap, with channel slopes above the riprap not armored. A typical Approach Channel section is shown on Figure 50.

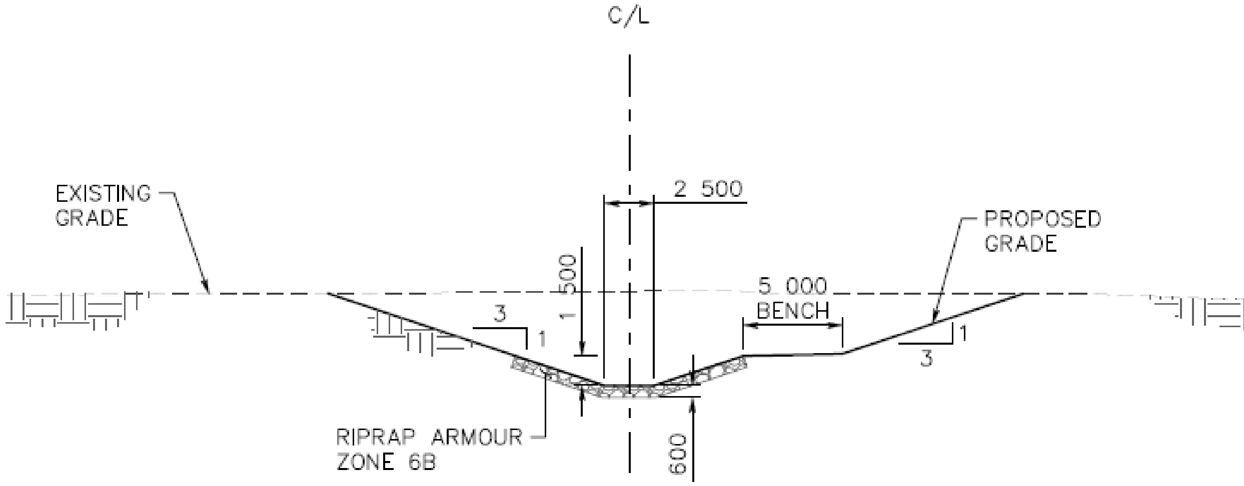


Figure 50. Approach Channel Typical Section

The Exit Channel downstream of the LLOW CSU rigid basin will convey the LLOW discharge away from the toe of the dam to the existing natural stream. The designed Exit Channel has been graded downstream of the LLOW for a length of approximately 765 m. The channel is trapezoidal with a 10 m bottom width, 3H:1V side slopes, and a bed slope that varies from 0.5 to 2 percent. The channel is riprap lined on the bottom and for three vertical metres up the side slopes. Five-metre-wide benches on both sides of the exit channel are provided for maintenance access. A typical Exit Channel section is shown on Figure 51.

At the Exit Channel terminus, a transition to the existing channel is designed. The transition includes a narrowing of the hydraulic section to match the smaller existing stream channel. A riprap lined scour pool is proposed. The pool would be excavated, lined with riprap and then backfilled with native material. During typical flows, the existing channel form will be maintained. During large releases from the LLOW, the immediate backfill within the scour pool would erode, and the formed pool would provide energy dissipation and reduce the risk of a headcut forming.

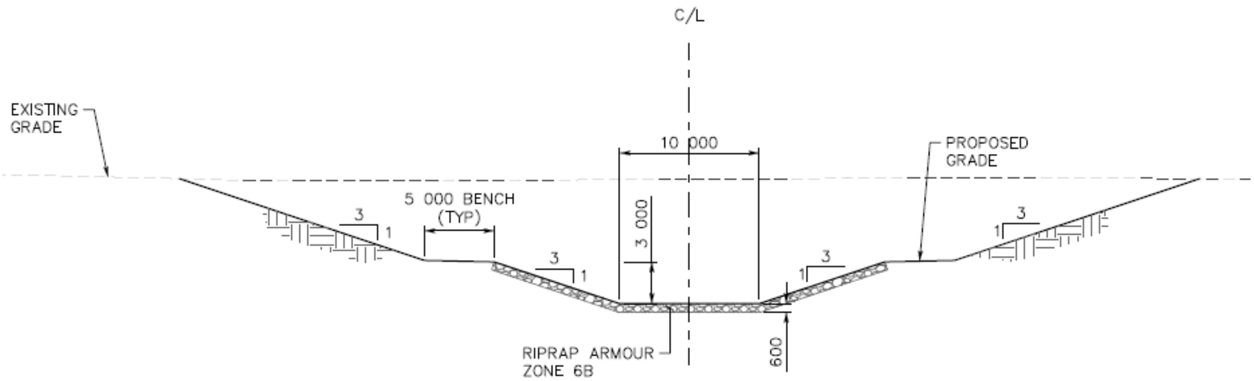


Figure 51. Exit Channel Typical Section

10.4.6 Geotechnical Considerations

10.4.6.1 Foundation

The LLOW conduit is designed to be supported on a soil bearing foundation. Recommended soil parameters to be used in the design of the LLOW were selected based on project wide laboratory testing and are presented in Appendix D. The LLOW components will be backfilled with embankment core soil material (Zone 1A) obtained from the Diversion Channel excavation or borrow areas within the reservoir pool limits. The embankment soil parameters used in the design of the LLOW structures, also developed based on laboratory testing and standard correlations, are presented in Appendix D.

During construction, the excavation and foundation preparation for the LLOW should be reviewed by the responsible geotechnical engineer prior to LLOW 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.

10.4.6.2 Settlement

Settlement analyses were performed along the LLOW alignment (Station 23+022). The dam cross section is presented in Figure 52.

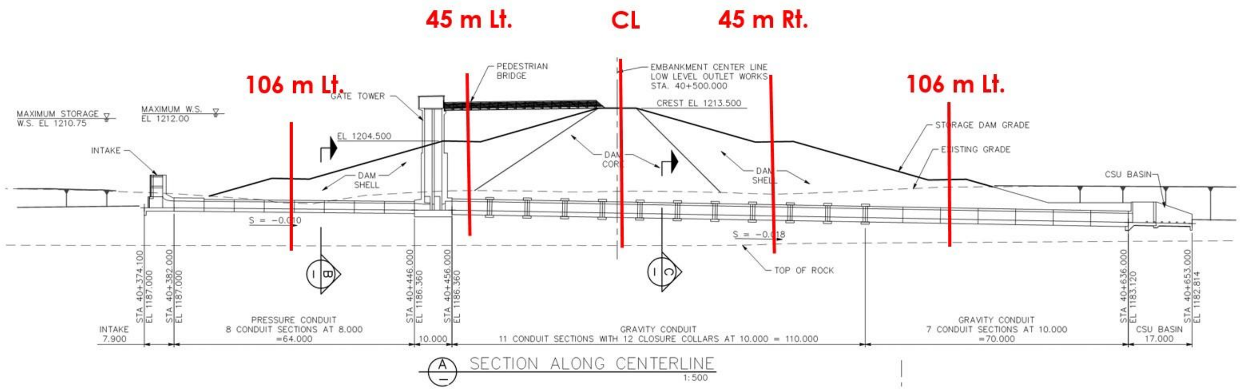


Figure 52. Storage Dam Section at Low-Level Outlet Works

Foundation settlements beneath the embankment were calculated at the LLOW location using the guidance from CFEM (2006).

Five soil columns (Figure 52) were analyzed below the storage dam crest and below the upstream and downstream slope benches. Settlement calculations and figures are included in Appendix D. The settlement below the conduit varied from near 0 mm at the ends to just over 400 mm near the center.

To accommodate settlement of the conduit, a camber of 200 mm will be incorporated into the design with a smooth form such as an arc based on approximately 50 percent of the estimated settlement values presented in Appendix D. The profile has been designed to be sloped to drain at the end of construction through the end of long-term settlements.

10.4.6.3 Lateral Earth Movement

Extension of the LLOW conduit could be caused by lateral deformation of the embankment. Two different simplified methods were used to estimate the potential lateral dam deformation and extension of the LLOW. These include methods by Walker and Duncan (1984) and the United States Department of Agriculture (USDA, 1969).

The estimated lateral bulging at the mid-height of the dam estimated using the Duncan and Walker method 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 two-thirds reduction in the mid-height lateral deformation appears to be a conservative estimate of the lateral deformation at the base of the dam. One-third 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 LLOW between the crest and the upstream inlet, and 14 joints in the LLOW between the crest and the downstream outlet. Assuming the lateral deformation is linearly distributed along the length of the LLOW, lateral extension at each joint would be approximately 60 mm. If the lateral deformation is not linearly distributed along the length of the LLOW, the lateral extension at some of the joints could be larger than 60 mm.

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 estimates joint opening due to conduit rotation and lateral strain.

The estimated required joint extensibility using the USDA design procedure is 56 mm using the best estimate of the parameters. An estimated joint extensibility of 61 mm was obtained based on the conservative estimate of undrained shear strength.

The two simplified methods used to estimate lateral extension of the LLOW and lateral dam displacement predicted lateral deformation of 60 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, Stantec considered that it is appropriate to apply a safety factor of two to the estimates. Accordingly, an estimated joint extension of 120 mm will be incorporated into the Final Design of conduit joints. A 2-D finite element model will be used to determine local deformations, rotations and joint extension at each joint during Final Design

10.4.7 Structural Design Approach

Each component of the Low-Level Outlet Works was assessed for the applicable imposed loads to which they may be subject per the governing criteria. These components include the intake, pressure conduit, gate structure, gravity conduit and CSU rigid basin. Secondary structures such as gate hoists, gate operating mechanisms, ladders, etc. will be designed to work with the supporting structure and to function as required by the ABC. Each structure was evaluated, as applicable, for global stability, strength, and serviceability.

Global stability was calculated based on a summation of forces to determine resultant location, foundation bearing pressures, and sliding resistance along the concrete/soil interface. A 3-dimensional finite element method (FEM) model was created to validate stability calculations and identify areas of stress concentration. Mathcad and Excel sheet templates were developed to assess the stability.

Strength evaluation was used to verify size of critical components such as walls, aprons, gate operating mechanism reaction ledge, training walls, and identify location of primary expansion/contraction joints. Strength evaluations also provided design values for sizing concrete reinforcement, joint doweling, gate anchorage, and gate hoist supports.

Serviceability includes limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR). The same 3-dimensional FEM used for strength evaluation was the primary method used to evaluate serviceability of the intake and gate structure. A combination of Mathcad and Excel sheet templates and commercial software were used to assess serviceability requirements of the conduits and other structures.

10.4.8 Stability

Stability analyses for the LLOW structures were performed in accordance with criteria and procedures outlined in the CDA Technical Bulletin No. 9, "Structural Considerations for Dam Safety" (CDA, 2007b), and the USACE EM 1110-2-2100 "Stability Analysis of Concrete Structures" (USACE, 2005). Each section or structure was evaluated for Usual, Unusual, Extreme, and Post-Seismic loading conditions representing potential conditions the structure will experience during its design life. Summaries of the stability calculation results are presented in the sections that follow. Refer to Appendix E for stability calculations and results.

10.4.8.1 Intake Structure

Stability analysis results for the intake structure are summarized in Table 44. As shown, the intake structure meets the criteria for sliding, resultant location, floatation, and bearing capacity for the analysis load cases. Refer to Appendix E for detailed stability analysis calculations for the Intake Structure.

Table 44. Intake Structure - Stability Analysis Results Summary

Load Case	Reservoir Elevation (m)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress (kPa)		Resultant Location	
		Req.	Calc.	Req.	Calc.	Req.	Calc. (Max)	Req.	Calc.
Usual Load Cases									
LC01- Empty Reservoir (Winter)	1187.00	1.5	8.8	1.5	>10	200	133	100%	100%
LC02 – Empty Reservoir (Summer)	1187.00	1.5	8.7	1.5	>10	200	132	100%	100%
Unusual Load Cases									
LC03 – Construction	1192.00	1.3	9.2	1.3	8.0	200	142	100%	100%
LC04 – 10-Year Flood	1192.00	1.3	2.2	1.3	>10	200	120	100%	100%
Extreme – Flood									
LC05 – 10-Year Flood + Impact	1192.00	1.1	2.2	1.1	7.0	200	131	75%	100%
LC06 – 10-Year Flood + Clogged Trashracks	1192.00	1.1	2.0	1.1	>10	200	102	75%	100%
LC07 – Maximum Pool	1213.50	1.1	1.3	1.1	9.0	200	103	75%	100%
Extreme – Earthquake									
LC08[1] - Earthquake	1187.00	*n/a*	*10.8 – 9.3*	*n/a*	*3.4 – 1.0*	*n/a*	*199-450*	*>0%*	*100% - 51%*
LC09 – Post-Earthquake	1187.00	1.1	8.7	1.1	>10	600[2]	326	100%	100%

1- *No criteria for Earthquake cases. Results are for post-earthquake estimate. A range of results is provided for 24 seismic acceleration configurations.*
2- *Ultimate bearing pressure is used.*

10.4.8.2 Intake Structure Wing Walls

Stability analysis results for the Wing Walls are summarized in **Error! Reference source not found.**. The proposed Wing Walls meet the stability criteria described in section for sliding, resultant location, floatation, and bearing capacity for the analysis load cases. Detailed stability calculations for the wing walls are provided in Appendix E.

Table 45. Intake Structure Wing Walls - Stability Analysis Results Summary

Load Case	Reservoir Elevation (m)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress (kPa)		Resultant Location	
		Req.	Calc.	Req.	Calc.	Req.	Calc. (Max)	Req.	Calc.
Usual Load Cases									
LC01- Empty Reservoir	1187.00	1.5	4.7	1.5	10.9	150	88	100%	100%
Unusual Load Cases									
LC02 – Construction	1187.00	1.3	4.7	1.3	1.7	150	136	100%	100%
LC03 – Rapid Drawdown	1189.75[3]	1.3	2.1	1.3	1.3	150	107	100%	100%
Extreme – Earthquake									
LC04[1] - Earthquake	1187.00	*n/a*	*4.6*	*n/a*	*2.1*	*n/a*	*196*	*>0%*	*65.7%*
LC05 – Post-Earthquake	1187.00	1.1	4.4	1.1	2.3	450[2]	450	100%	100%

1- *No criteria for Earthquake cases. Results are for post-earthquake estimate. Worse case result is provided for 4 seismic acceleration configurations.*
2- *Ultimate bearing pressure is used.*
3- *Top of wing wall elevation*

10.4.8.3 Gate Structure

Stability analysis results for the Gate Structure are summarized in Table 46. The Gate Structure meets the criteria for sliding, resultant location, floatation, and bearing capacity for the analysis load cases considered. Detailed stability calculations for the Gate Structure are provided in Appendix E.

Table 46. Gate Structure - Stability Analysis Results Summary

Load Case	Reservoir Elevation (m)	Floatation Safety Factor (FSF)		Sliding Safety Factor (SSF)		Foundation Bearing Stress (kPa)			Resultant Location	
							Calc. (Max)			
		Req.	Calc.	Req.	Calc.	Req.	Long Term	Short Term	Req.	Calc.
Usual Load Cases										
LC01- Empty Reservoir	1184.40	1.5	>10	1.5	>10	427	412	424	100%	100%
Unusual Load Cases										
LC02 – 10-Year Flood, Gates Open	1191.43	1.3	6.1	1.3	>10	427	368	384	100%	100%
LC03 – 10-Year Flood, Gate Closed	1191.43	1.3	5.95	1.3	>10	427	353	369	100%	100%
LC04 – Staged Construction	1184.36	1.3	>10	1.3	>10	250	203	n/a	100%	100%
LC05 – Crane Surcharge	1184.36	1.3	>10	1.3	>10	427	n/a	428	100%	100%
Extreme – Flood										
LC05 – Maximum Pool, Gates Open	1213.50	1.1	2.2	1.1	>10	427	350	359	75%	100%
LC06 – Maximum Pool, Gate Closed	1213.50	1.1	2.0	1.1	>10	427	337	345	75%	100%
Extreme – Earthquake										
LC08[1] - Earthquake	1184.36	*n/a*	*>10*	*n/a*	>10	*n/a*	*n/a*	*718*	*>0%*	*100%*
LC09 – Post-Earthquake	1184.36	1.1	>10	1.1	>10	427	n/a	398	100%	100%

1- *No criteria for Earthquake cases. Results are for post-earthquake estimate. Worst case result is provided for 24 seismic acceleration configurations.*

10.4.8.4 CSU Rigid Basin

The stability analysis results for the CSU Rigid Stilling Basin are summarized in

Table 47. Refer to Appendix E for stability calculations and results.

Table 47. CSU Rigid Basin - Stability Analysis Results Summary

Load Case	Loading Type	Sliding Safety Factor	Flotation Safety Factor	Resultant Location	Bearing Safety Factor[1]
LC01	Dry Condition	2.5	N/A	Middle Third	1.6
LC02	10-Year Flood	1.4	4.3	Middle Third	2.0
LC03	Rapid Gate Closure	1.5	2.8	Middle Third	2.3
LC04	Construction/Maintenance	1.9	N/A	Middle Third	1.7
LC05	Maximum Flow	1.4	3.3	Middle Third	2.1
LC06[2]	Earthquake	1.0 – 1.9	>10	Middle Third	2.2 - 2.8
LC07[3]	Post-Earthquake	Passes	Passes	Middle Third	Passes

1 *Bearing is compared to allowable.*
2 *Seismic load case is reported for information only. There are no acceptance criteria for this load case.*
3 *Post-Earthquake load case passes if the resultant location for all Earthquake load cases is within the middle third.*

10.4.8.5 Concrete Conduit

The stability analysis results for the Pressure Conduit are summarized in Table 48. Detailed stability calculations for the Concrete Conduit are provided in Appendix E.

Table 48. Pressure Conduit - Stability Analysis Results Summary

Load Case	Loading Type	Flotation Safety Factor
U1	Usual Load Condition 1: Empty Reservoir	N/A**
UN1	Unusual Load Condition 1: Construction	N/A**
UN2	Unusual Load Condition 2: Conduit submerged with water inside of conduit	1.9
UN3	Unusual Load Condition 3: Conduit submerged with no water inside of conduit	1.7
E1	Extreme Load Condition 1: Empty Reservoir and earthquake load	N/A**

**Groundwater elevation is assumed at bottom of conduit base.

10.4.9 Strength

Strength evaluation of individual elements or members of structures and monoliths was used to verify member sizes and steel reinforcement based on application of factored loads as described in the ABC with some adjustment for more severe conditions or loads not included in the ABC.

Reinforced concrete design was performed according to Design of Concrete Structures, CSA A23.3-14 with the additional requirements of the CSA's SEED Document – *Structural Design of Wastewater Treatment Plants-2018* for revisions addressing service load conditions, water tightness, shrinkage and temperature reinforcement, and crack control. The Seed Document contains references to ACI 350M-06 for modifying CSA A23.3-14.

Structural steel design was performed according to Design of Steel Structures, CSA S16-14, and codes for welding, materials, and other pertinent references.

In general, structural analysis and design were performed manually using MathCAD or Excel spreadsheets. For more complex structures, such as the LLOW Intake and the Gate Structure, a commercial Three-Dimensional Finite Element Model (FEM) was used to evaluate multiple load combinations, identify stress concentrations, and generate shear and moment values for the design of individual elements. The FEM was supplemented with manual calculations to verify/validate model results and where necessary, refine the analysis of individual elements. Based on model output, a combination of manual calculations and commercial software were used for strength design. Additional elements evaluated as part of strength design included joint detailing, equipment anchorage, and embedded parts.

10.4.10 Serviceability

Structural serviceability concerns with the Low-Level Outlet Works relate primarily to concrete durability including limiting deflections, reducing crack potential, providing thermal stress relief, and incorporating measures to mitigate alkali-aggregate reaction (AAR) and other chemical attack. The same manual calculations, commercial software, or 3-D FEM used for strength evaluation were used to evaluate deflection and thermal growth, while design detailing and material specification were used to mitigate cracking and AAR potential.

Shrinkage control and volume changes are addressed primarily with placement sequence, mix design, surface reinforcement, and material specifications. The structure layout and design define concrete placements with balanced aspect ratios. Joint placements allow for proper dissipation of heat of hydration and initial shrinkage before placement of adjacent concrete. Expanded guidance related to placement sequence and joint locations will be addressed as part of constructability review during Final Design.

Tight installation tolerances for gates, hoists and other embedded components are critical for their operation or installation. These tolerances are addressed primarily through second stage concrete placements occurring after initial concrete shrinkage has occurred. Using a second stage concrete placement also allows gate assemblies to be installed, checked and adjusted for operation before components are permanently being embedded in concrete.

10.4.11 Construction Considerations

Construction specifications and details for the LLOW will be furthered during Final Design. The following construction considerations are noted:

- The foundation subgrade was located in glacial till for subgrade uniformity. Foundation preparation will require care in preparation of the structure subgrade to meet bearing capacity for the tall structures and conduits. Foundation preparation details will be developed during Final Design.

- Construction sequencing of conduit placements will be required to control thermal expansion and contraction due to the concrete's heat of hydration.

- Settlement and conduit elongation are expected to occur. Conduit joint spacing, treatment and type will be developed to address estimated settlement and elongation along the conduits. Total settlement and camber will be used to design appropriate joint spacing in the conduits and to select the types of joint collars to use.

- Joint preparation will require proper installation of water stops, shear keys, dowels, and reinforcement.

- Procurement lead-time for the fabricated slide gate components will likely be driven by stainless steel availability and fabrication schedules. An allowance of 3 to 6 months is recommended to account for design, shop drawing review/approval, fabrication, testing, and delivery of embedded items. An additional three months is required for supply of the gate and hoist system.

- Fill placement and compaction methods must be properly reviewed and monitored to reduce the risk of conduit monolith and wing wall movement during construction.

- Gate slots, access bridge, and the control house at the LLOW will require combinations of concrete block outs, anchor bolts, and embedded parts in first and second stage concrete placements. Placement tolerance for some of these items are tighter than typical heavy construction tolerance due to fit and operating clearance requirements.

11.0 SITE ACCESS, PUBLIC SAFETY AND SECURITY

11.1 ACCESS

Access for the maintenance, inspection, observation, and operation of the various flood mitigation structures is provided by way of private access roadways. The roadways will be gated at their point of connection with public roadways to control their use by the general public. The primary access road will extend from the Diversion Structure, north along the Diversion Channel, to the Off-stream Storage Dam for maintenance and inspections. This roadway has gated intersections at both Highway 22 and Township Road 242. Access to the Floodplain Berm and Service Spillway south of the Elbow River is provided from an access road off Highway 8. An emergency pathway off Springbank Road provides a secondary access to the east end of the dam.

To discourage foot and vehicle traffic within the project footprint, AT specified Wildlife Friendly Barbed Wire Fence will be installed along the perimeter of the project, north of the Elbow River. Security will be enhanced around the Diversion Structure control building and gate system with 8-foot Class H chain link fencing.

11.2 PUBLIC SAFETY

Public safety for the SR1 Project Site will be accomplished through site access restriction and visible and audible warning systems. Access restriction was discussed in the previous section. Warning and hazard identification signage are discussed below. Warning sirens and lights will be addressed further during Final Design. Sirens are not required under the CDA Guidelines but are recommended to provide additional public awareness to specific hazards, such as gate activation, when sufficient physical barriers are not practical.

The following documents were referenced in the selection of sign placement, wording and sizing/visibility:

- Canadian Dam Safety Guidelines, Canadian Dam Association 2007 and 2013 (CDA, 2013).
- Water Control Structures Selected Design Guidelines, Alberta Transportation, 2004 (AT, 2004b)
- Safety Signs PDF received from Alberta Environment and Parks (AEP, 2017)
- FERC Guidelines for Public Safety at Hydropower Projects (FERC, 1992)

The SR1 project has been broken down into six areas with separate considerations and signs required at each including:

- Upstream of the Diversion Structure
- Downstream of the Diversion Structure
- Diversion Channel
- Reservoir Outlet
- Dam Structures
- Roads and access

11.2.1 Upstream of the Diversion Structure

It is assumed that during flood events of sufficient discharges to require operation of the Diversion Channel and associated infrastructure that there will be no public users on the Elbow River. Signage is designed for "typical" conditions during which time discharge is contained within the Elbow River and its associated floodplain.

To warn users on the Elbow River (kayakers, canoeists, people in rafts etc.), a series of signs are proposed to advise the public that they are approaching a dam and gate structure and then direct them to a safe portage location.

Additional signage indicating danger with instruction not to proceed further will be located downstream of the portage location. Signs will be located on both banks of the Elbow River to increase visibility.

11.2.2 Downstream of the Diversion Structure

The signs located downstream of the Diversion Structure will be similar to those located upstream but with reduced spacing as users will not be traveling as fast (against the current). Some wording will also be changed as there is a hazard from water being released from the structure with associated fluctuating water levels which is not present upstream.

Signs will be located on both banks of the Elbow River to increase visibility.

11.2.3 Diversion Channel

When the Diversion Inlet gates are closed the Diversion Channel will be dry, except for local drainage. This equates to normal site conditions. The hazard to the public from the Diversion Channel is during operation, or more specifically, due to the rapid increase in water level and flow within the channel that occurs when the gates are opened. An alarm siren will activate and draw attention to the signs indicating that a rapid increase in water level is imminent. The siren(s) will be located along the channel at road crossings, which are the most accessible locations. Additional signs and sirens can be located if walking trails or other access routes are identified during site visits or consultation with the public.

At the Emergency Spillway and outlet locations, members of the public may be located on the areas of land downstream of these structures and be affected by a sudden discharge. Sirens will be located at these two locations and be independently controlled. Signs will be located around the Emergency Spillway and Diversion Channel Outlet that will instruct the public to evacuate the area if the sirens are activated.

11.2.4 Low-Level Outlet Works

During Reservoir drawdown, discharge from the Reservoir could reach flows up to 27 m^3/s. This increase in discharge can pose a risk to persons near the Unnamed Creek and to those on the Elbow River near the confluence.

It is proposed to install a siren with warning signs located near the LLOW CSU Basin as well as signs along trails leading to the tributary to warn of rapid water level rise during drawdown.

A secondary siren, with signs, located near the confluence is proposed to warn members of the public located on the Elbow River near the confluence that flows from the tributary may fluctuate and impact the Elbow River accordingly.

11.2.5 Dam Structures

The Diversion Structure and the Off-stream Storage Dam discharge structures will be gated and fenced to deter members of the public from entering these areas but additional signage indicating "Danger" as well as "No Trespassing" are proposed to increase the visibility of the hazards.

11.2.6 Roads and Access

The Reservoir footprint impacts a number of roads. Highway 22 passes through the western end of the Reservoir but upgrades are planned to raise the road elevation above the flood waters and maintain passage during flood storage events. Signs located at the north and south sides of the Reservoir along Highway 22 will advise the public not to swim or use watercraft on the water when present.

Springbank Road and several private residence/agricultural access roads are not planned to be raised and will be submerged during flood retention operations. It is recommended that these roads have gates located outside the reservoir footprint which will be closed during diversion operations along with signs advising road users that the road is closed due to flooding and to use an alternate route.

11.3 NAVIGATION

The Elbow River is navigable and will require approval from Transport Canada under the Canadian Navigable Waters Act. Provisions for navigation are closely tied to those for public safety and site security.

The Elbow River itself boasts many natural hazards, particularly when flows reach bankfull levels, including debris jams, sweepers, rollers, as well as both physical and hydraulic traps. Navigating this river is not safe; however, the provisions for navigation at the structure should consider safety through both exclusion and safe passage measures under the assumption that the public will attempt to pass through the bays. This reach can be navigated by non-motorized vessels and common vessels are canoes and kayaks, but drift boats, rafts and inner-tubes may also attempt to navigate this reach. It is expected that this activity will continue following construction of SR1. The hazard to navigation created by SR1 are the Diversion Structure components within the 2-year (bankfull) main channel of the Elbow River.

Signing described in Section 11.2.1 includes warning signs on the river to advise boaters of the downstream hazards. Accompanying these signs, portage signs with distance markers instructing persons to use the designated portage route in the river-right floodplain that circumvents the Diversion Structure via a path on the Auxiliary Spillway.

Booms, float lines and other exclusion measures were investigated. The investigation identified two different types of booms and anchor/release mechanisms that could be considered within the context of safety, flood operations and maintenance. Stantec reviewed the hydraulic characteristics of the bankfull channel and found velocities to exceed 1.5 m/s which is the common limit for the smaller diameter boom and float lines before submergence from the hydraulic forces.

Though it is possible to procure boom and float line products that will float in velocities greater than 1.5 m/s; it is necessary to consider the frequency that the river will experience these hydraulic conditions; and if those hydraulic conditions pose a hazard with the boom line in place. Velocities in the main channel exceed 1.5 m/s nearly every year and it is assumed that boaters, especially whitewater kayakers and canoeists, will be trying to navigate the river during these conditions. Encountering any type of boom line in these conditions is hazardous and there is a high likelihood that vessels will strike the boom. Hard bodied vessels like canoes and kayaks may impinge against it as currents attempt to side-dump their vessel. Rafts and inner tubes could also impinge against it but with the added hazard of causing a tear in the raft and a less-likelihood of freeing oneself from the impingement as the soft vessel is pressed against the boom.

In considering the effectiveness of a boom as an exclusion measure, and the hazard the boom itself would impose on boaters, it is recommended that a boom line not be included as part of the navigation and public safety plan at this time.

12.0 PROJECT SCHEDULE

A draft construction schedule with an assumed start date of September 2021 was developed based on the preliminary design and is included in Appendix G.

12.1 SCHEDULE OVERVIEW

The Province has established the following schedule objectives for the Project:

- Completion of construction and commissioning for the Flood of Record operations by the first flood season (May 1 – Jul 31) following 3 years of construction.
- Provision of an interim level of flood mitigation for events up to the 1:100 year magnitude flood by the first flood season following 2 years of construction.

The schedule provided in Appendix G assumes a construction start date of May 25, 2022 which results in Project completion by January 2025 four months prior to the May 20245 flood season.

In order to meet the interim flood risk objective, the following milestones must be met:

- Diversion Structure completed including Floodplain Berm, Auxiliary Spillway, Service Spillway, Diversion Inlet and Debris Deflection Barrier.
- Diversion Channel excavated and erosion protection installed for a design discharge rate of 360 m^3/s including a minimum 6 metre bottom width from Station 10+100 to 10+900 excavated through rock, the full design section from Station 10+900 to 14+570 including riprap lining, and the Diversion Channel Outlet.
- Bridges constructed at Highway 22 and Township Road 242.
- Dam embankment constructed to EL. 1203.5 m.
- Temporary emergency spillway excavated through native material near the west abutment. Details and design of the temporary emergency spillway is not in this Preliminary Design Report because geotechnical investigations have not been performed. Once geotechnical investigations are performed, design of the temporary emergency spillway will be completed.
- LLOW constructed including approach and exit channels, intake, conduits, gate structure and CSU rigid basin. Gate installation and bridge access are not required. Temporary joint closure materials would be incorporated and replaced following embankment completion.

12.2 SCHEDULE ORGANIZATION

The schedule is sub-divided by the three major components of the Project: Diversion Structure; Diversion Channel; and Off-stream Storage Dam. These component sections are further divided into sub-components. Major project activities are itemized and provided with construction durations, and linked to other items with predecessors and successors. Earthwork (excavation and fill) for the project was broken into Zones A through O. These Zones are referenced in the schedule to assist with sub-division of project components and link connected activities. Figure 53 displays these zones.

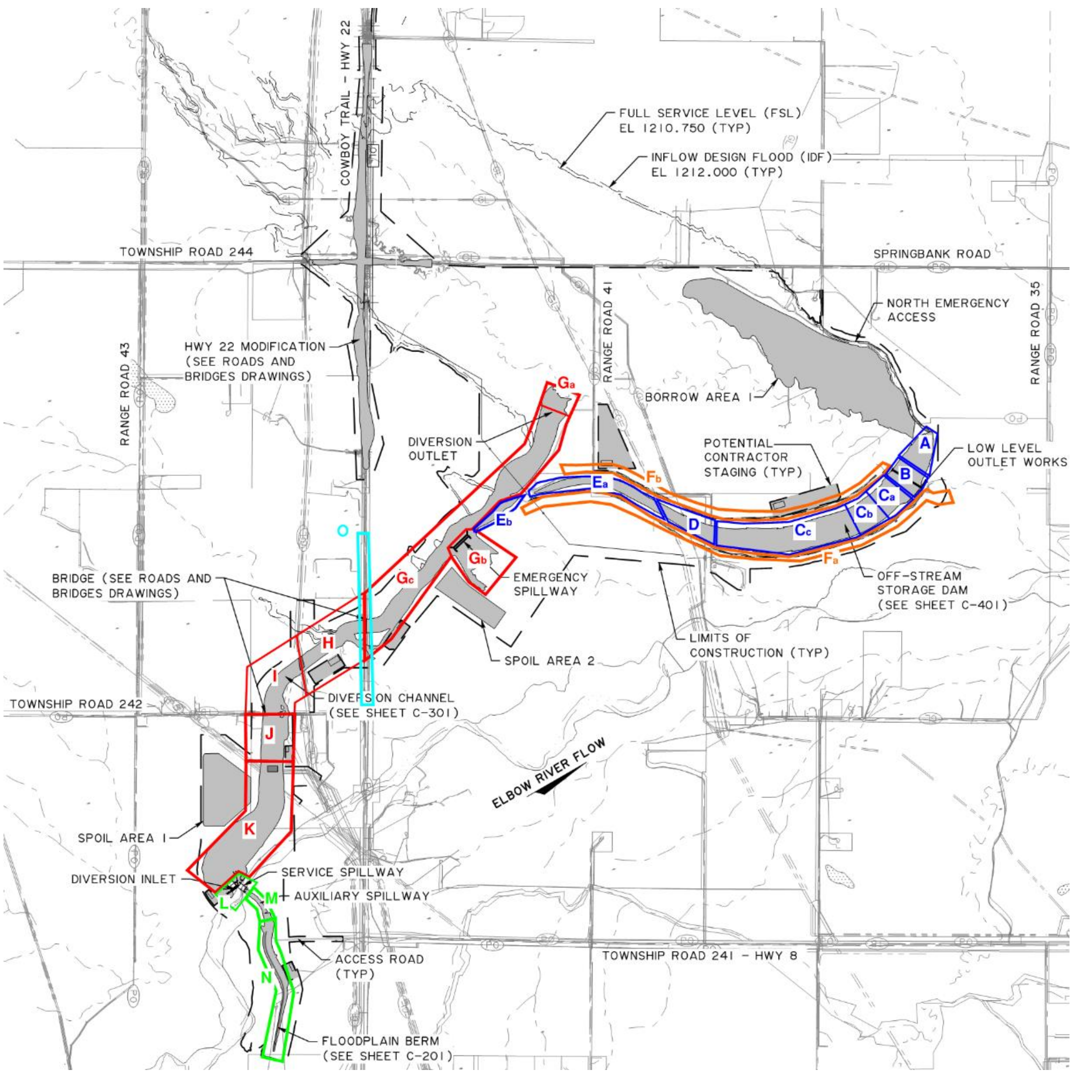


Figure 53. Earthwork Construction Zones

12.3 KEY CONSIDERATIONS

The following considerations were incorporated into the schedule development:

- Elbow River Restricted Periods: Work in the Elbow River is restricted from January 1 to April 15, May 1 to July 15, and September 15 to December 31. Given these restrictions, construction of the Diversion Structure will require diversion of the Elbow River and installation of a cofferdam or other isolation measures. Therefore, it is assumed that the Elbow River will be diverted between July 15 and September 15, and construction of the Service Spillway and Diversion Inlet will begin after diversion completion. If diversion or restoration of the Elbow River is delayed from the schedule presented, completion of the Project could be delayed by up to 1 year.

- Gate Procurement: Service Spillway gate procurement including design, fabrication, and delivery is assumed to occur over a 12-month period. Based on the assumed May 2022 start, the schedule includes less than 1 month of float for the Service Spillway gates due to the restricted periods within the Elbow River.

- Diversion Channel Excavation: Diversion Channel excavation is anticipated to occur from east to west starting in Zone G. Earthwork placement requirements for the Dam will require excavation of the channel into Zone K by the end of the first year of embankment construction.

  Separated grade crossings at Highway 22 and Township Road 242 are required to access Zones H through K. This separation will be established by either temporary structures built by the Contractor or early construction of the Highway 22 and Township Road 242 bridges. Similarly, utility relocations, as described in Section 9.5, are required to excavate portions of Zones I and J. It is assumed that these relocations will be completed by the utility owners prior to the start of channel excavation.

- Off-stream Storage Dam Embankment Placement: Embankment placement for the dam is assumed to occur during the summer and fall construction seasons (May 1 to October 31). The dam will require a minimum of three full embankment construction seasons to complete. This duration is driven by the volume of earthwork and the expected pore pressure response of the dam foundation to loading. Additional information regarding the construction loading analysis is included in Appendix D.

  Embankment placement in portions of Zones B cannot begin until construction of the LLOW is complete. Excavation of glacio-lacustrine foundation soils from beneath the LLOW in Zone C may be required to achieve the necessary embankment construction height after two seasons. This additional excavation and resulting fill are not included in this preliminary design presentation.

13.0 COST OPINION

13.1 GENERAL

An opinion of probable construction costs for preliminary design is provided in Appendix G. Quantities are derived from the design drawings included in Appendix A. Costs are presented in 2017 dollars to maintain consistency with previous estimates.

The Cost Opinion is consistent with the requirements of a Type B Estimate as defined in the Alberta Transportation Engineering Consultant Guidelines for Highway, Bridge, and Water Projects. Unit prices were developed utilizing published Alberta Transportation cumulative unit price averages from the three lowest bidders on recent tenders, weighted by bid quantity. Year 2016 (August 1, 2015 to July 31, 2016) and Year 2017 (August 1, 2016 to February 1, 2017) average tenders were evaluated to establish unit pricing for most items. Price opinions for items unavailable in the recent tenders were developed based on local construction industry experience and engineering judgement. The pay item structure is broken into major project components to delineate and define items associated with each feature. Pay items units are consistent with AT Civil Master Works Specifications, AT Standard Specifications for Highway Construction, and generally accepted industry standard methods of measurement.

13.2 METHODS AND ASSUMPTIONS

Published Alberta Transportation cumulative unit price averages have been used with modification in cases where there was material difference in the volumes or level of effort associated with that item. Adjustments have been made to recognize economy of scale, when differing. In addition, adjustments have been made to recognize the increase/decrease in effort required for similar items of work. Where published Albert Transportation unit prices did not exist, other methods of determining unit prices were obtained. This included using general engineering principles, a comparison of unit costs to current and past projects and key quantity take-offs.

13.2.1 Overhaul

The size of the project and distinct and separated areas that require excavation, or placement of materials, requires the inclusion of overhaul as an item in the cost opinion. For haul distances of 500 m or less the unit price for excavation includes hauling. Where haul distances are greater than 500 m the centroid of each the excavation and embankment areas are used to determine the additional distance (and associated cost) for the overhaul of said material.

13.2.2 Suitable Material Assumptions

The geotechnical exploration indicates that the quality and quantity of materials needed to construct this project are available on site. Due to normal variances between actual conditions obtained during construction and the data identified during geotechnical exploration both borrow and spoils areas have been included in this project. They provide the successful contractor with additional materials to use in the embankment construction, and spoil areas for materials that are either not suitable or intentionally not used by the contractor.

13.2.3 Risk Contingency

A contingency factor of 15 percent is utilized at this point in the process to reflect the level of study and knowledge that is possessed currently.

13.3 FUTURE UNCERTAINTIES

The Cost Opinion for the current submittal is not inflated to reflect construction in a future year. As such there are potential increases that occur annually. In addition, there are potential economic and environmental factors that can affect the cost of a project when built in a future year. Such factors include fluctuations in commodity price, floods, droughts, recessions, changes in environmental regulation with regard to water or power and much more. Any of these can change in any given year and can create significant changes in the cost of goods, services, and natural resources. The cost opinion does not attempt to predict any economic changes.

14.0 MAINTENANCE

This section describes the anticipated maintenance and inspection activities for the Project. Annual, periodic, special (flood) and long-term activities are addressed. A cost-opinion for these activities in 2017 dollars is presented.

14.1 ANNUAL INSPECTION

An inspection team, led by a professional engineer, should perform an annual inspection of the project facilities during periods of low river flow. The following items should be inspected to confirm that they operate correctly and do not require maintenance:

- Safety and navigation signage should be inspected to confirm that all constructed warning signs are in place, readily visible and legible.
- Hydraulic lines should be visually inspected for cracking, leaks and failures. Once they are confirmed to be intact and visibly functional they should be pressurized and observed again for leaks.
- Signage and warning measures inventory should be visibly inspected and manually operated to determine correct functionality.
- Energize and exercise all electrical components. Backup power generator should be started and used to power the operation of gates.
- Fully exercise all gates and valves. Each should be cycled through their full operating range and any deficiencies noted and corrected.
- Communication systems should be operated and observed for functionality.
- Obermeyer bladders should be visually inspected in their fully expanded state.
- Diversion Channel slopes and flowline should be walked and visibly inspected for erosion, scarping, or sloughing. Any abnormality should be identified for analysis and repaired.
- Floodplain Berm embankment slopes and crest should be walked and visibly inspected for erosion, scarping, or sloughing. RCC surfaces should be visibly inspected for spalling of concrete. Any abnormality in either the Floodplain Berm or RCC should be identified for analysis and repaired.

- Storage Dam embankment, crest, Low-Level Outlet Works and Emergency Spillway should be walked and visibly inspected for erosion, scarping, sloughing, tree or shrub growth and rodent burrows. Any abnormality should be noted and removed/repaired; or analyzed further.

14.2 ANNUAL MAINTENANCE

Regular maintenance is recommended to safeguard the investment and operation of this flood mitigation infrastructure. Regular maintenance also reduces the risk of systems not operating at their full capacity. The following annual maintenance measures are recommended:

- Mowing of Dam embankment and crest, Diversion Channel, lower reservoir, Floodplain Berm, roadway shoulders, and general access areas.

- Remove vegetation from Dam, Diversion Channel, lower reservoir, Floodplain Berm, and general access areas.

- Remove and fill any rodent burrows from the Floodplain Berm, Diversion Channel and Dam.

- Remove debris and clean Service Spillway, Diversion Inlet and Low-Level Outlet Works.

- Repair any erosion at the inlet and outlet ends of the drainage channel for the Low-Level Outlet Works.

- Repair any rutting or other loss of driving surface for the access roads and their shoulders. Regrade driving surface as necessary to maintain adequate cross-slopes for surface drainage.

- Routine maintenance (check site security for vandalism).

- Grease all mechanical connections.

- Paint miscellaneous surface when needed.

- Repair and patch miscellaneous concrete surfaces when needed.

14.3 MAJOR FLOOD EVENT STAFFING

The following personnel are anticipated to attend, operate, and inspect the facilities during a major flood event:

- One senior person-in-charge/attendant at Diversion Structure.

- One senior person-in-charge/attendant at Dam.

- One site inspector at Dam.
- One instrumentation inspector at Dam.
- One three-person maintenance crew (with excavator/picker, loader and dump truck).

14.4 POST FLOOD INSPECTION

An inspection should be performed after any flood event that requires the Diversion Structure and Diversion Channel to be operated. The following inspection items should be performed by an inspection team, led by a professional engineer with other personnel as needed:

- Typical annual inspection by engineering team immediately after flood waters recede.
- Inspect reservoir area for trapped fish and rescue as necessary.
- Inspect reservoir area for hazardous materials and debris.

14.5 POST FLOOD MAINTENANCE

The following maintenance items should be performed after a flood event passes through the Diversion Channel and impounds water against the Dam:

- Removal of debris and sediment from all project facilities (river, Diversion Channel, Diversion Structure, Floodplain Berm, Reservoir, Dam, Low-Level Outlet Works and Emergency Spillway).
- Silt and debris removal from Springbank Road.
- Riprap replacement along Diversion Inlet, Service Spillway, Floodplain Berm and Diversion Channel.
- Reseeding and mulching of Diversion Structure, Dam, Reservoir and all other eroded surfaces.
- Erosion repair in drainage channel, reservoir and downstream of the Low-Level Outlet Works.

14.6 LONG TERM MAINTENANCE

The following items require long term maintenance or replacement due to life cycle and/or normal wear and tear:

- Significant sediment and debris removal is anticipated after a moderate flood event. This maintenance is anticipated every 20 years.
- Significant stream restoration is anticipated downstream of the Low-Level Outlet Works every 20 years. Depending on repairs, this may be one-time spend.
- Replacement of Obermeyer crest gate bladders is anticipated every 35 years.
- Replacement of gate control systems, computer hardware and software is anticipated every 10 years.
- Minor concrete repairs are anticipated every 10 years.
- Minor mechanical equipment replacement is anticipated every 10 years.
- Dam instrumentation replacement is anticipated every 10 years.

14.7 OPERATIONS AND MAINTENANCE COST OPINION

Unit prices were developed utilizing published 2016 RS Means unit cost data and engineering judgement. An inflation rate of three percent has been assumed.

The pay item structure is broken into major project components to delineate and define all items associated with each feature. This methodology allows for a more thorough review of each component, versus quantifying all items globally. In addition, each component can be analyzed independently if budgetary constraints require value engineering.

A summary of the annual and periodic operation and maintenance cost items is presented in the tables below. Total life-cycle costs have not been developed. Further coordination with the GoA is required to identify estimated project life and escalation factors.

Table 49. Anticipated Maintenance Costs

Annual Maintenance Cost Items (2017 Dollars)	
Annual Engineering Inspection and Reporting	$25,000
Mowing (once per year)	$24,000
Annual Debris Cleanup - Project Wide	$89,000
Erosion Repair (Localized Riprap Replacement /Reseed Areas) - Project Wide	$67,000
Stone Access Road Resurfacing (Localized Areas)	$59,000
Stone Access Road Repair (Rutting and Erosion)	$30,000
Minor Mechanical Maintenance, Lubrication – Project Wide	$7,000

Periodic (10-Year Interval) Maintenance Cost Items (2017 Dollars)	
Concrete Surface and RRC Repair – Project Wide	$35,000
Diversion Inlet Gate Maintenance	$10,000
Obermeyer and Diversion Inlet Gates - Computer Hardware/Software Controls	$20,000

Major Flood Event Maintenance Cost Items (2017 Dollars)	
Emergency Flood Personnel Staffing and Reporting	$65,000
River Sediment and Debris Cleanup	$109,000
Diversion Channel Sediment and Debris Cleanup	$179,000
Reservoir Sediment and Debris Cleanup	$100,000
Reservoir Reseeding and Mulching (25% of 20-Year Flood Pool)	$501,000
Diversion Channel Riprap Replacement (10% Riprap Areas)	$748,000
Diversion Channel Reseeding and Mulching (10% Grass Surfaces)	$192,000
Stream Restoration Downstream of LLOW (800 metres)	$1,312,000

Obermeyer Crest Gate Replacement Costs (2017 Dollars)	
Replace Obermeyer Bladders in 2056 (35-year design life)	$2,000,000
Concrete Repair with Bladder Replacement	$20,000

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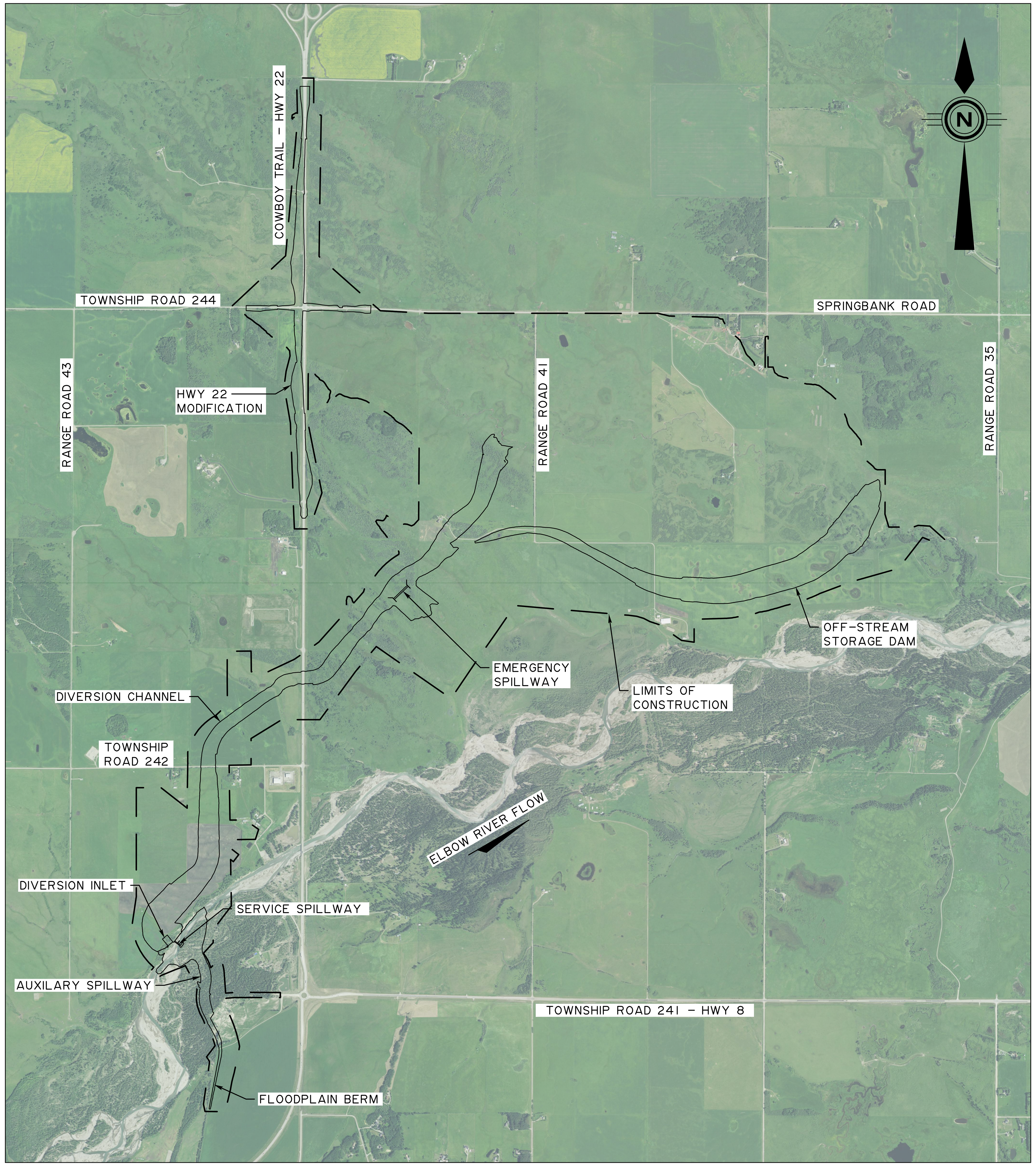
APPENDIX A
PRELIMINARY DESIGN DRAWINGS

SPRINGBANK OFF-STREAM STORAGE PROJECT (SR1)

PRELIMINARY DESIGN DRAWINGS
SEPTEMBER 25, 2020

TENDER NO.



KEY PLAN
1:20000

0 800 1600
Metres

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

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DEPARTMENT BAR CODE

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REV	DATE	REVISIONS	BY
6			
5			
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1			

PRELIMINARY DESIGN

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DESIGNER
DATE

CHECKER
DATE

Alberta
Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
COVER SHEET

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396A-100	

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TENDER No. ?

SHEET NUMBER	DRAWING NO.	DESCRIPTION
		GENERAL
*	A-100	COVER SHEET
*	A-102	INDEX OF SHEETS 1
*	A-103	INDEX OF SHEETS 2
*	A-104	GENERAL NOTES AND PROJECT LEGEND
	A-105	GENERAL NOTES 1
	A-106	GENERAL NOTES 2
	A-107	GENERAL NOTES 3
	A-108	PROJECT ABBREVIATIONS
*	A-110	PROJECT LAYOUT
*	A-111	RESERVOIR OVERVIEW
	A-113	PROJECT LAYOUT AND CONTROL
	A-120	LIMITS OF WORK AND STAGING AREAS DIVERSION STRUCTURE
	A-130	LIMITS OF WORK AND STAGING AREAS DIVERSION CHANNEL
	A-140	LIMITS OF WORK AND STAGING AREAS DIVERSION DAM
	A-201	CONTROL POINTS AND BASELINE TABLES 1
	A-301	CONTROL POINTS AND BASELINE TABLES 2
	A-401	CONTROL POINTS AND BASELINE TABLES 3
	A-501	EROSION AND SEDIMENT CONTROL DETAILS 1
	A-502	EROSION AND SEDIMENT CONTROL DETAILS 2
	A-503	EROSION AND SEDIMENT CONTROL DETAILS 3
		GEOTECHNICAL
		GENERAL
	B-110	GEOTECHNICAL SITE OVERVIEW
	B-120	GEOLOGIC SITE OVERVIEW
	B-130	OVERVIEW EXPLORATION PLAN 1
	B-131	OVERVIEW EXPLORATION PLAN 2
		DIVERSION STRUCTURE
	B-210	DIVERSION STRUCTURE EXPLORATION PLAN 1
	B-211	DIVERSION STRUCTURE EXPLORATION PLAN 2
	B-220	GEOTECHNICAL INFORMATION (GENERAL)
	B-221	GEOTECHNICAL INFORMATION (PROFILE)
	B-222	GEOTECHNICAL INFORMATION (PROFILE)
	B-280	GEOTECHNICAL INFORMATION (LOGS)
	B-281	GEOTECHNICAL INFORMATION (LOGS)
		DIVERSION CHANNEL
	B-310	DIVERSION CHANNEL EXPLORATION PLAN 1
	B-311	DIVERSION CHANNEL EXPLORATION PLAN 2
	B-312	DIVERSION CHANNEL EXPLORATION PLAN 3
	B-320	GEOTECHNICAL INFORMATION (GENERAL)
	B-321	GEOTECHNICAL INFORMATION (PROFILE)
	B-322	GEOTECHNICAL INFORMATION (PROFILE)
	B-323	GEOTECHNICAL INFORMATION (PROFILE)
	B-380	GEOTECHNICAL INFORMATION (LOGS)
	B-381	GEOTECHNICAL INFORMATION (LOGS)
	B-382	GEOTECHNICAL INFORMATION (LOGS)
	B-383	GEOTECHNICAL INFORMATION (LOGS)
		OFF-STREAM STORAGE DAM
	B-410	OFF-STREAM STORAGE DAM EXPLORATION PLAN 1
	B-411	OFF-STREAM STORAGE DAM EXPLORATION PLAN 2
	B-412	OFF-STREAM STORAGE DAM EXPLORATION PLAN 3
	B-420	GEOTECHNICAL INFORMATION (GENERAL)
	B-421	GEOTECHNICAL INFORMATION (PROFILE)
	B-422	GEOTECHNICAL INFORMATION (PROFILE)
	B-423	GEOTECHNICAL INFORMATION (PROFILE)
	B-450	GEOTECHNICAL INFORMATION (SECTION)
	B-451	GEOTECHNICAL INFORMATION (SECTION)
	B-452	GEOTECHNICAL INFORMATION (SECTION)
	B-480	GEOTECHNICAL INFORMATION (LOGS)
	B-481	GEOTECHNICAL INFORMATION (LOGS)
	B-482	GEOTECHNICAL INFORMATION (LOGS)
	B-483	GEOTECHNICAL INFORMATION (LOGS)
	B-484	GEOTECHNICAL INFORMATION (LOGS)
	B-485	GEOTECHNICAL INFORMATION (LOGS)
		STANDARD DRAWINGS AND DETAILS
	B-510	STANDARD DRAWING, PLAN (INSTRUMENTATION)
	B-511	STANDARD DRAWING, PLAN (INSTRUMENTATION)
	B-550	STANDARD DRAWING, DETAIL (INSTRUMENTATION)
	B-551	STANDARD DRAWING, DETAIL (INSTRUMENTATION)

SHEET NUMBER	DRAWING NO.	DESCRIPTION
		CIVIL
		GENERAL
	C-101	CIVIL OVERVIEW
*	C-108	DEMOLITION PLAN SHEET 1
*	C-109	DEMOLITION PLAN SHEET 2
*	C-110	DEMOLITION PLAN SHEET 3
*	C-111	DEMOLITION PLAN SHEET 3
*	C-112	DEMOLITION PLAN SHEET 4
*	C-113	DEMOLITION PLAN SHEET 6
*	C-114	DEMOLITION PLAN SHEET 7
*	C-115	DEMOLITION PLAN SHEET 8
*	C-116	DEMOLITION PLAN SHEET 9
*	C-117	DEMOLITION PLAN SHEET 10
	C-120	CLEARING, GRUBBING, AND E&S SHEET 2
	C-121	CLEARING, GRUBBING, AND E&S SHEET 2
	C-122	CLEARING, GRUBBING, AND E&S SHEET 3
	C-123	CLEARING, GRUBBING, AND E&S SHEET 4
	C-124	CLEARING, GRUBBING, AND E&S SHEET 5
	C-125	CLEARING, GRUBBING, AND E&S SHEET 6
	C-126	CLEARING, GRUBBING, AND E&S SHEET 7
	C-127	CLEARING, GRUBBING, AND E&S SHEET 8
	C-128	CLEARING, GRUBBING, AND E & S SHEET 9
	C-129	CLEARING, GRUBBING, AND E & S SHEET 10
*	C-130	TEMPORARY ELBOW RIVER DIVERSION 1
	C-131	TEMPORARY ELBOW RIVER DIVERSION 2
	C-132	TEMPORARY ELBOW RIVER DIVERSION 3
*	C-140	SITE UTILITIES OVERVIEW
*	C-142	DIVERSION CHANNEL UTILITY CROSSING 1
*	C-143	DIVERSION CHANNEL UTILITY CROSSING 2
*	C-144	DIVERSION CHANNEL UTILITY CROSSING 3
*	C-150	RESERVOIR UTILITY RELOCATION ALIGNMENT OVERVIEW
*	C-151	RESERVOIR UTILITY RELOCATION PLAN AND PROFILE
	C-160	ELECTRIC SERVICE PLAN DIVERSION STRUCTURE
	C-163	ELECTRIC SERVICE PLAN DIVERSION CHANNEL 1
	C-164	ELECTRIC SERVICE PLAN DIVERSION CHANNEL 2
	C-165	ELECTRIC SERVICE PLAN DIVERSION CHANNEL 3
	C-166	ELECTRIC SERVICE PLAN DIVERSION CHANNEL 4
	C-170	ELECTRIC SERVICE PLAN OFF-STREAM STORAGE DAM 1
	C-171	ELECTRIC SERVICE PLAN OFF-STREAM STORAGE DAM 2
	C-172	ELECTRIC SERVICE PLAN OFF-STREAM STORAGE DAM 3
	C-175	ELECTRIC SERVICE PLAN RESERVOIR 1
	C-176	ELECTRIC SERVICE PLAN RESERVOIR 2
	C-180	ELECTRIC SERVICE PLAN DETAILS 1
	C-181	ELECTRIC SERVICE PLAN DETAILS 2
	C-182	ELECTRIC SERVICE PLAN DETAILS 3
	C 190	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 1
	C-191	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 2
	C-192	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 3
	C-193	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 4
	C-194	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 5
	C-195	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 6
	C-196	SITE SECURITY, SIGNAGE, AND ACCESS PLAN 7
		DIVERSION STRUCTURE
*	C-201	OVERVIEW
	C-205	FOUNDATION AND PREPARATION PLAN 1
	C-206	FOUNDATION AND PREPARATION PLAN 2
*	C-210	FLOODPLAIN BERM AND AUX SPILLWAY PLAN 1
*	C-211	FLOODPLAIN BERM AND AUX SPILLWAY PLAN 2
*	C-212	FLOODPLAIN BERM AND AUX SPILLWAY PLAN 3
*	C-213	FLOODPLAIN BERM AND AUX SPILLWAY PLAN 4
*	C-214	ELBOW RIVER CHANNEL PLAN 1
*	C-215	ELBOW RIVER CHANNEL PLAN 2
	C-225	SOUTH SITE ACCESS AND DRIVEWAY
	C-228	FLOODPLAIN BERM ACCESS RAMPS
*	C-230	FLOODPLAIN BERM AND AUX SPILLWAY PROFILE
*	C-235	ELBOW RIVER CHANNEL PROFILE
	C-240	ACCESS ROAD PROFILES
	C-250	FLOODPLAIN BERM AND AUX SPILLWAY SECTIONS 1
	C-251	FLOODPLAIN BERM AND AUX SPILLWAY SECTIONS 2
	C-252	FLOODPLAIN BERM AND AUX SPILLWAY SECTIONS 3
*	C-270	FLOODPLAIN BERM AND AUX SPILLWAY TYPICAL SECTIONS 1
*	C-271	FLOODPLAIN BERM AND AUX SPILLWAY TYPICAL SECTIONS 2
*	C-275	ELBOW RIVER CHANNEL TYPICAL SECTIONS 1
*	C-276	ELBOW RIVER CHANNEL TYPICAL SECTIONS 2
	C-278	ACCESS ROAD TYPICAL SECTIONS
	C-280	FLOODPLAIN BERM DETAILS 1
	C-281	FLOODPLAIN BERM DETAILS 2
*	C-282	SERVICE SPILLWAY BACKFILL DETAIL
*	C-285	ELBOW RIVER CHANNEL DETAILS 1
*	C-286	ELBOW RIVER CHANNEL DETAILS 2
	C-290	CONTROL BUILDING UTILITIES 1
	C-291	CONTROL BUILDING UTILITIES 2
	C-292	CONTROL BUILDING UTILITIES 3
	C-293	CONTROL BUILDING UTILITIES 4
	C-294	CONTROL BUILDING UTILITIES 5

SHEET NUMBER	DRAWING NO.	DESCRIPTION
		DIVERSION CHANNEL
*	C-301	OVERVIEW
*	C-310	PLAN 1
*	C-311	PLAN 2
*	C-312	PLAN 3
*	C-313	PLAN 4
*	C-316	PLAN DETAIL 1 DIVERSION INLET
*	C-318	PLAN DETAIL 2 BRIDGE CROSSING TWP 242
	C-320	PLAN DETAIL - CREEK INLET AND CHANNEL FILL PLAN
*	C-322	PLAN DETAIL 3 BRIDGE CROSSING HWY 22
	C-324	PLAN DETAIL - EMERGENCY SPILLWAY
	C-326	PLAN DETAIL - GRADE CONTROL STRUCTURE
*	C-330	PROFILE 1
*	C-331	PROFILE 2
*	C-332	PROFILE 3
*	C-333	PROFILE 4
	C-336	EMERGENCY SPILLWAY CHANNEL PROFILE
	C-340	ACCESS ROAD PROFILE 1
	C-342	ACCESS ROAD PROFILE 2
	C-350	SECTIONS 1
	C-351	SECTIONS 2
	C-352	SECTIONS 3
	C-353	SECTIONS 4
	C-354	SECTIONS 5
	C-355	SECTIONS 6
	C-356	SECTIONS 7
	C-357	SECTIONS 8
	C-358	SECTIONS 9
	C-359	SECTIONS 10
	C-360	SECTIONS 11
	C-361	SECTIONS 12
	C-362	SECTIONS 13
	C-363	SECTIONS 14
	C-364	SECTIONS 15
	C-365	SECTIONS 16
*	C-370	TYPICAL SECTIONS 1
*	C-371	TYPICAL SECTIONS 2
*	C-372	TYPICAL SECTIONS 3
*	C-373	TYPICAL SECTIONS 4
	C-376	BRIDGE CROSSING SECTION 1
	C-377	BRIDGE CROSSING SECTION 2
	C-380	CHANNEL LINING DETAIL 1
	C-381	CHANNEL LINING DETAIL 2
	C-382	CHANNEL LINING DETAIL 3
	C-384	EMERGENCY SPILLWAY DETAILS
*	C-388	ACCESS ROADS TYPICAL SECTIONS
*	C-390	GROUNDWATER CONTROL DETAILS
		OFF-STREAM STORAGE DAM
*	C-401	OVERVIEW
	C-404	FOUNDATION AND PREPARATION PLAN 1
	C-405	FOUNDATION AND PREPARATION PLAN 2
	C-406	FOUNDATION AND PREPARATION PLAN 3
	C-408	FOUNDATION AND PREPARATION PLAN 4
	C-409	FOUNDATION AND PREPARATION PLAN 5
*	C-410	PLAN 1
*	C-411	PLAN 2
*	C-412	PLAN 3
	C-418	LOW LEVEL OUTLET WORKS PLAN AND PROFILE
	C-419	LOW LEVEL OUTLET WORKS APPROACH CHANNEL PLAN
*	C-420	LOW LEVEL OUTLET WORKS EXIT CHANNEL PLAN
	C-421	SITE ACCESS PLAN 1
	C-422	SITE ACCESS PLAN 2
	C-423	SITE ACCESS PLAN 3
	C-424	SITE ACCESS PLAN 4
*	C-430	PROFILE 1
*	C-431	PROFILE 2
*	C-432	PROFILE 3
*	C-435	LOW LEVEL OUTLET WORKS PROFILES
	C-440	ACCESS ROAD PROFILE 1
	C-441	ACCESS ROAD PROFILE 2
	C-442	ACCESS ROAD PROFILE 3

NOTES:
1. * DENOTES DRAWINGS INCLUDED IN PRELIMINARY DESIGN SUBMITTAL.
2. DRAWING NUMBERS SHOWN COMPRISE THE OVERALL DRAWING LIST AT THE TIME OF PRELIMINARY DESIGN SUBMITTAL. COMPLETE DRAWING ASSIGNMENTS HAVE YET TO BE ESTABLISHED FOR ELECTRICAL (E), MECHANICAL (M), EQUIPMENT (Q), AND REFERENCE (R).

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

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	-	-	-

PRELIMINARY DESIGN

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DESIGNER
DATE

CHECKER
DATE

Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
INDEX OF SHEETS 1

DATE: 2020-09-25
ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER: -
SITE: -
SHEET
CONSULTANT DRAWING: 73396A-102
TRANSPORTATION DRAWING

TENDER No. ?

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SHEET NUMBER	DRAWING NO.	DESCRIPTION
		OFF-STREAM STORAGE DAM (CONTINUED)
	C-450	SECTIONS 1
	C-451	SECTIONS 2
	C-452	SECTIONS 3
	C-453	SECTIONS 4
	C-454	SECTIONS 5
	C-455	SECTIONS 6
	C-456	SECTIONS 7
	C-457	SECTIONS 8
	C-458	SECTIONS 9
	C-459	SECTIONS 10
	C-460	SECTIONS 11
	C-461	SECTIONS 12
	C-462	SECTIONS 13
*	C-470	TYPICAL SECTIONS 1
*	C-471	TYPICAL SECTIONS 2
*	C-472	TYPICAL SECTIONS 3
*	C-475	LOW LEVEL OUTLET WORKS TYPICAL SECTION
*	C-480	EMBANKMENT SURFACE DRAINAGE DETAILS
*	C-482	PRESSURE RELIEF TRENCH DRAIN DETAILS
*	C-485	LOW LEVEL OUTLET WORKS DETAILS
*	C-488	ACCESS ROADS DETAILS
		STANDARD DRAWINGS AND DETAILS
	C-520	FENCE AND GATE DETAILS 1
	C-521	FENCE AND GATE DETAILS 2
		STRUCTURAL
		GENERAL
	S-100	STRUCTURAL GENERAL NOTES
	S-101	STANDARD STRUCTURAL DETAILS
	S-110	STRUCTURAL DESIGN DATA
	S-120	CONSTRUCTION PROCEDURE & INSTALLATION NOTES
	S-130	RIVER DIVERSION TEMPORARY STRUCTURES
*	S-150	SERVICE SPILLWAY, DIVERSION INLET AND DEBRIS DEFLECTION BARRIER GENERAL ARRANGEMENT
	S-160	EMERGENCY SPILLWAY DROP STRUCTURE GENERAL ARRANGEMENT
	S-161	DIVERSION CHANNEL OUTLET STRUCTURE GENERAL ARRANGMENT
*	S-170	LOW-LEVEL OUTLET WORKS GENERAL ARRANGEMENT
		SERVICE SPILLWAY
*	S-200	STRUCTURAL ARRANGEMENT
	S-202	FOUNDATION PREPARATION AND GROUTING
	S-204	STILLING BASING ANCHORS
	S-206	DRAINAGE
*	S-210	RIGHT RETAINING WALL BLOCK 1A CONCRETE OUTLINE 1
*	S-211	RIGHT RETAINING WALL BLOCK 1A CONCRETE OUTLINE 2
*	S-212	RIGHT RETAINING WALL BLOCKS 1B & 1C CONCRETE OUTLINE 1
*	S-213	RIGHT RETAINING WALL BLOCKS 1B & 1C CONCRETE OUTLINE 2
	S-215	RIGHT RETAINING WALL BLOCK 1A REINFORCEMENT
	S-216	RIGHT RETAINING WALL BLOCKS 1B & 1C REINFORCEMENT
*	S-220	GATE STRUCTURE BLOCKS 2A & 4A CONCRETE OUTLINE
*	S-222	GATE STRUCTURE BLOCK 3A CONCRETE OUTLINE
	S-225	GATE STRUCTURE BLOCKS 2A & 4A REINFORCEMENT
	S-226	GATE STRUCTURE BLOCK 3A REINFORCEMENT
*	S-230	STILLING BASIN BLOCKS 2B & 4B CONCRETE OUTLINE
*	S-232	STILLING BASIN BLOCK 3B CONCRETE OUTLINE
	S-234	STILLING BASIN BLOCKS 2B & 4B REINFORCEMENT
	S-235	STILLING BASIN BLOCK 3B REINFORCEMENT
*	S-240	LEFT RETAINING WALL BLOCK 5A CONCRETE OUTLINE
*	S-242	LEFT RETAINING WALL BLOCK 5B CONCRETE OUTLINE
*	S-244	LEFT RETAINING WALL BLOCKS 5C & 5D CONCRETE OUTLINE 1
*	S-245	LEFT RETAINING WALL BLOCKS 5C & 5D CONCRETE OUTLINE 2
	S-250	LEFT RETAINING WALL BLOCK 5A REINFORCEMENT
	S-251	LEFT RETAINING WALL BLOCK 5B REINFORCEMENT
	S-252	LEFT RETAINING WALL BLOCKS 5C & 5D REINFORCEMENT
		AUXILIARY SPILLWAY
*	S-260	STRUCTURAL ARRANGEMENT
	S-262	FOUNDATION PREPARATION AND GROUTING
	S-264	LEVELING CONCRETE
	S-266	DRAINAGE
*	S-270	OVERFLOW WEIR BLOCKS A1 TO A13
	S-271	OVERFLOW WEIR SECTIONS AND DETAILS
	S-272	OVERFLOW WEIR CREST OUTLINE & REINFORCEMENT
*	S-275	TRANSITION WALL CONCRETE OUTLINE 1
*	S-276	TRANSITION WALL CONCRETE OUTLINE 2
	S-278	TRANSITION WALL REINFORCEMENT

SHEET NUMBER	DRAWING NO.	DESCRIPTION
		DEBRIS DEFLECTION BARRIER
*	S-280	STRUCTURAL ARRANGEMENT
	S-281	FOUNDATION PREPARATION
	S-282	FOUNDATION PILES
	S-283	FOUNDATION CONCRETE OUTLINE PANEL A1&A2
	S-284	FOUNDATION CONCRETE OUTLINE PANELS B1-B14
	S-285	FOUNDATION OUTLINE PANEL C1
	S-286	FOUNDATION OUTLINE PANELJOINTS
	S-287	FOUNDATION REINFORCEMENT PANELS A1 & A2
	S-288	FOUNDATION REINFORCEMENT PANELS B1-B14
	S-289	FOUNDATION REINFORCEMENT PANEL C1
	S-290	STEEL FRAMING PANELS A1 & A2
	S-291	STEEL FRAMING PANELS B1-B14
	S-293	STEEL FRAMING PANEL C1
	S-294	STEEL FRAMING FABRICATION
	S-295	STEEL FRAMING CONNECTIONS AND DETAILS
		DIVERSION INLET
*	S-300	STRUCTURAL ARRANGEMENT
	S-302	FOUNDATION PREPARATION AND GROUTING
	S-304	STILLING BASIN ANCHORS
	S-306	DRAINAGE
*	S-310	RIGHT RETAINING WALL BLOCKS 1A & 1B CONCRETE OUTLINE 1
*	S-311	RIGHT RETAINING WALL BLOCKS 1A & 1B CONCRETE OUTLINE 2
*	S-312	LEFT RETAINING WALL BLOCKS 5A & 5B CONCRETE OUTLINE 1
*	S-313	LEFT RETAINING WALL BLOCKS 5A & 5B CONCRETE OUTLINE 2
*	S-314	RETAINING WALL BLOCKS 1C & 5C CONCRETE OUTLINE 1
*	S-315	RETAINING WALL BLOCKS 1C & 5C CONCRETE OUTLINE 2
*	S-316	RETAINING WALL BLOCKS 1D, 1E, 5D & 5E CONCRETE OUTLINE 1
*	S-317	RETAINING WALL BLOCKS 1D, 1E, 5D & 5E CONCRETE OUTLINE 2
	S-320	RIGHT RETAINING WALL BLOCKS 1A & 1B REINFORCEMENT
	S-321	LEFT RETAINING WALL BLOCKS 5A & 5B REINFORCEMENT
	S-322	RETAINING WALL BLOCKS 1C & 5C REINFORCEMENT
	S-323	RETAINING WALL BLOCKS 1D & 5D REINFORCEMENT
*	S-330	GATE STRUCTURE BLOCKS 2A, 2B, 3A, 3B, 4A & 4B CONCRETE OUTLINE PLAN
*	S-331	GATE STRUCTURE BLOCKS 2A & 2B AND 4A & 4B CONCRETE OUTLINE SECTIONS
*	S-332	GATE STRUCTURE BLOCKS 3A & 3B CONCRETE OUTLINE SECTIONS
	S-335	GATE STRUCTURE BLOCKS 2A & 2B AND 4A & 4B REINFORCEMENT
	S-336	GATE STRUCTURE WALL BLOCKS 3A & 3B REINFORCEMENT
*	S-340	STILLING BASIN BLOCKS 2C, 3C & 4C CONCRETE OUTLINE
*	S-342	STILLING BASIN BLOCK 3C CONCRETE OUTLINE SECTIONS
	S-345	STILLING BASIN BLOCKS 2C & 4C REINFORCEMENT
	S-346	STILLING BASIN BLOCK 3C REINFORCEMENT
*	S-350	LEFT ABUTMENT BLOCKS 6 TO 9 CONCRETE OUTLINE
	S-355	LEFT ABUTMENT BLOCK 6 REINFORCEMENT
	S-357	LEFT ABUTMENT BLOCKS 7 TO 9 REINFORCEMENT
*	S-360	GATE STRUCTURE ACCESS BRIDGE DECK
*	S-361	GATE STRUCTURE ACCESS BRIDGE DECK SECTIONS AND DETAILS
*	S-362	GATE STRUCTURE BREAST WALL SECTIONS AND DETAILS
*	S-363	GATE STRUCTURE HEAD WALL SECTIONS AND DETAILS
	S-364	GATE STRUCTURE EMBEDMENTS & MISC DETAILS 1
	S-365	GATE STRUCTURE EMBEDMENTS & MISC DETAILS 2
	S-366	GATE STRUCTURE EMBEDMENTS & MISC DETAILS 3
*	S-367	GATE STRUCTURE GATE HOIST BRIDGE FRAMING PLAN AND SECTIONS
*	S-368	GATE STRUCTURE GATE HOIST SUPPORT SECTIONS AND DETAILS
		EMERGENCY SPILLWAY
	S-370	DROP STRUCTURE STRUCTURAL ARRANGEMENT
	S-371	DROP STRUCTURE OVERFLOW WEIR
	S-372	DROP STRUCTURE OVERFLOW WEIR REINFORCEMENT 1
	S-373	DROP STRUCTURE OVERFLOW WEIR REINFORCEMENT 2
	S-375	DROP STRUCTURE RETAINING WALLS 1
	S-376	DROP STRUCTURE RETAINING WALLS 2
	S-377	DROP STRUCTURE RETAINING WALLS 3
	S-378	DROP STRUCTURE RETAINING WALLS 4
		GRADE CONTROL STRUCTURE
	S-380	STRUCTURAL ARRANGEMENT
	S-381	SECTIONS AND DETAILS
		LOW LEVEL OUTLET WORKS
*	S-400	STRUCTURAL ARRANGEMENT
	S-402	INSTRUMENTATION AND SURVEY REFERENCE PIN DETAILS
*	S-410	INTAKE STRUCTURE CONCRETE OUTLINE 1
*	S-411	INTAKE STRUCTURE CONCRETE OUTLINE 2
	S-412	INTAKE STRUCTURE REINFORCEMENT
	S-420	UPSTREAM CONDUIT CONCRETE OUTLINE
	S-421	UPSTREAM CONDUIT CONCRETE DETAILS
	S-422	UPSTREAM CONDUIT CONCRETE REINFORCEMENT

SHEET NUMBER	DRAWING NO.	DESCRIPTION
		LOW LEVEL OUTLET WORKS (CONTINUED)
*	S-430	GATE TOWER CONCRETE OUTLINE
*	S-431	GATE CHAMBER CONCRETE OUTLINE 1
*	S-432	GATE CHAMBER CONCRETE OUTLINE 2
	S-433	CONTROL HOUSE PLANS AND SECTIONS
	S-434	CONTROL HOUSE ELEVATIONS
	S-435	GATE CHAMBER REINFORCEMENT
	S-436	ACCESS TOWER REINFORCEMENT
	S-437	CONTROL HOUSE REINFORCEMENT
	S-440	DOWNSTREAM GRAVITY CONDUIT CONCRETE OUTLINE
	S-442	DOWNSTREAM GRAVITY CONDUIT CONCRETE DETAILS
	S-445	DOWNSTREAM GRAVITY CONDUIT REINFORCEMENT
*	S-450	CSU BASIN CONCRETE OUTLINE 1
*	S-451	CSU BASIN CONCRETE OUTLINE 2
	S-455	CSU RIGID BASIN REINFORCEMENT
	S-460	INTAKE STRUCTURE TRASHRACKS
	S-461	INTAKE STRUCTURE TRASHRACK LIFT
	S-463	CONTROL HOUSE MISC DETAILS
	S-465	ACCESS SHAFT GATE ACCESS LADDER
	S-466	CSU BASIN CLOSURE DOORS
	S-470	ACCESS BRIDGE
		STANDARD DRAWINGS
	S-510	STRUCTURAL HANDRAILS
	S-520	BRIDGE GUARD RAILS
	S-530	FENCING
		EQUIPMENT
		DIVERSION STRUCTURE
	Q-200	DIVERSION INLET WHEEL GATE GENERAL ARRANGMENT 1
	Q-201	DIVERSION INLET WHEEL GATE GENERAL ARRANGMENT 2
	Q-202	DIVERSION INLET WHEEL GATE EMBEDDED PARTS AND MISC. DETAILS
	Q-203	DIVERSION INLET WHEEL GATE HOIST GENERAL ARRANGEMENT
	Q-210	SERVICE SPILLWAY PNEUMATIC CREST GATE GENERAL ARRANGEMENT

NOTES:

1. * DENOTES DRAWINGS INCLUDED IN PRELIMINARY DESIGN SUBMITTAL.
2. DRAWING NUMBERS SHOWN COMPRISE THE OVERALL DRAWING LIST AT THE TIME OF PRELIMINARY DESIGN SUBMITTAL. COMPLETE DRAWING ASSIGNMENTS HAVE YET TO BE ESTABLISHED FOR ELECTRICAL (E), MECHANICAL (M), EQUIPMENT (Q), AND REFERENCE (R).

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

GENERAL
INDEX OF SHEETS 2

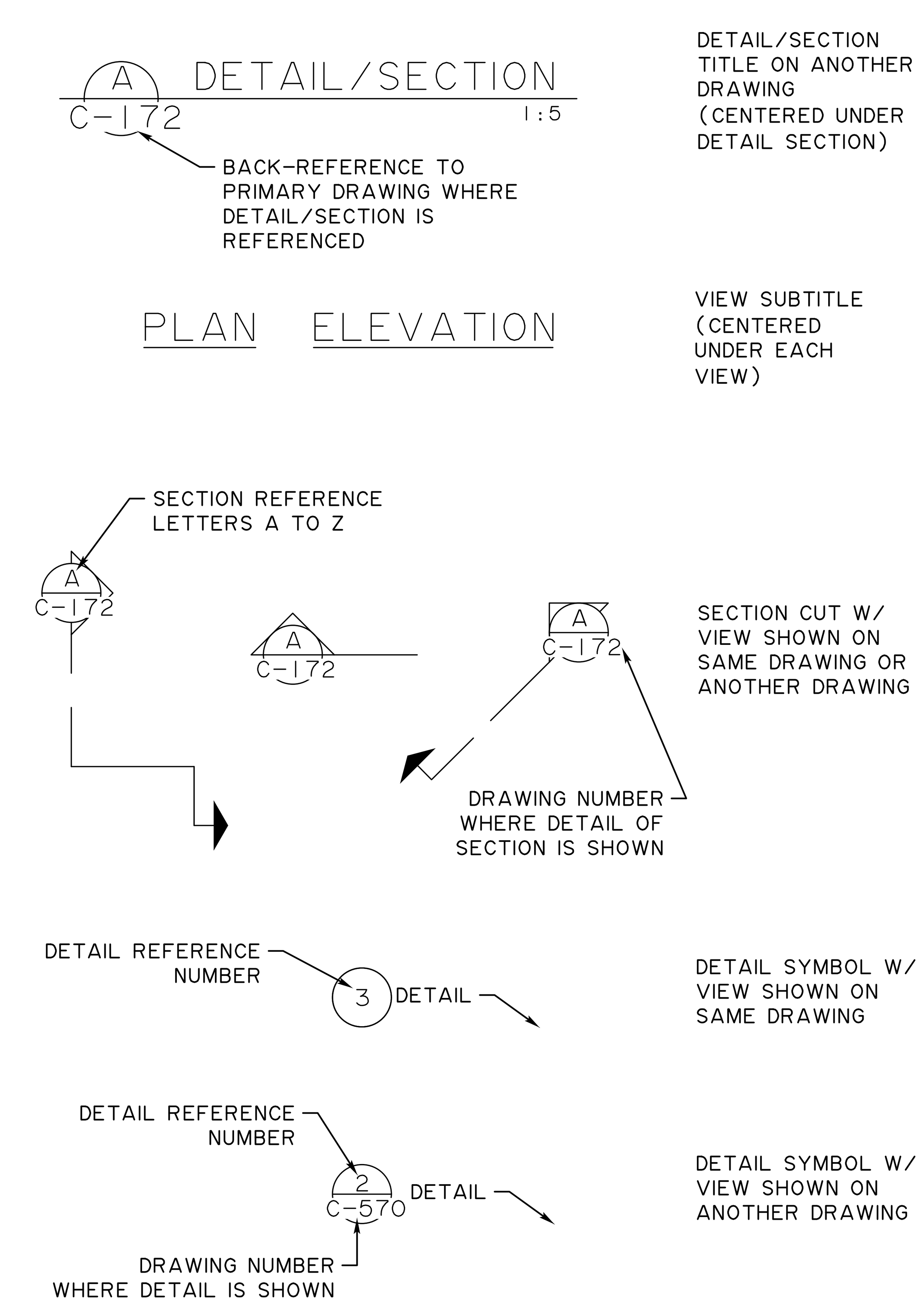
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396A-103	

PlotDate: 2017.09.22 10:07 AM Login: Sprague, Adam
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\external_files\C_WD_D_TTLB-Jan 2016.dwg TENDER No. ?

GENERAL NOTES:

1. ORIGINAL DATA COMPILED IN METRIC UNITS AND REFERENCED TO THE PROVINCE OF ALBERTA, 3 DEGREE ZONE (3TM), CENTERED AT 114 DEGREES WEST (114), NORTH AMERICAN DATUM OF 1983 (NAD 83).
2. AZIMUTHS SHOWN ARE MEASURED CLOCKWISE FROM THE NORTH.
3. UNITS FOR ALL PLAN, PROFILE AND SECTION SHEETS ARE IN METRES. UNITS FOR ALL TYPICAL SECTION AND DETAIL SHEETS ARE IN MILIMETRES.
4. ALL ELEVATIONS ARE IN METRES AND REFERENCED TO NORTH AMERICAN VERTICAL DATUM (NAVD88 2004.65).
5. DIMENSIONS AND/OR ELEVATIONS MARKED THUS (±) ARE APPROXIMATE. VERIFY ACTUAL DIMENSIONS IN THE FIELD.
6. DIMENSIONS AND/OR ELEVATIONS MARKED THUS (NTS) ARE NOT SHOWN TO SCALE. DRAWINGS ARE GENERALLY TO SCALE, BUT SHOULD NOT BE SCALED. NTS IS SHOWN ONLY WHERE DRAWING IS OBVIOUSLY OUT OF SCALE.

GENERAL CROSS-REFERENCING SYMBOLS



PROJECT LEGEND:

- EXISTING MINOR CONTOUR
- EXISTING MAJOR CONTOUR
- EXISTING WATER CENTERLINE
- EXISTING EDGE OF WATER (EOW)
- EXISTING TREELINE
- EXISTING RIGHT OF WAY (ROW)
- EXISTING EDGE OF PAVEMENT (EOP)
- EXISTING ROAD CENTERLINE
- EXISTING PROPERTY LINE
- (TB) EXISTING TELEPHONE
- (PO) EXISTING OVERHEAD ELECTRIC
- EXISTING POWER POLE
- (WM) EXISTING WATER
- EXISTING WATER FIXTURES
- WATER WELL — EXISTING WATER WELL
- EXISTING POND/LAKE
- (GL) EXISTING GAS
- (SA) EXISTING SANITARY
- INFLOW DESIGN FLOOD (IDF) EL 1212.000 (TYP)
- FULL SERVICE LEVEL (FSL) EL 1210.750 (TYP)
- LIMITS OF CONSTRUCTION
- LIMITS OF CLEARING
- LIMITS OF GROUND DISTURBANCE
- 1219 — PROPOSED MINOR CONTOUR
- 1220 — PROPOSED MAJOR CONTOUR
- PO — PROPOSED OVERHEAD ELECTRIC
- GL — PROPOSED GAS
- OL — PROPOSED OIL LINE
- ST — PROPOSED STORM
- PROPOSED RIGHT OF WAY (ROW)
- D — PROPOSED DITCH
- PROPOSED GRAVEL ROAD CENTERLINE
- x — PROPOSED BARBWIRE FENCE
- o — PROPOSED CHAINLINK FENCE
- PROPOSED GRAVEL ROAD/PAVEMENT
- PROPOSED ACCESS PATH
- PROPOSED RIPRAP
- PROPOSED TREELINE

PlotDate: 2020.06.25 12:13 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\a_general\sheet_files\100_gen\73396A-104.dwg TENDER No. ?

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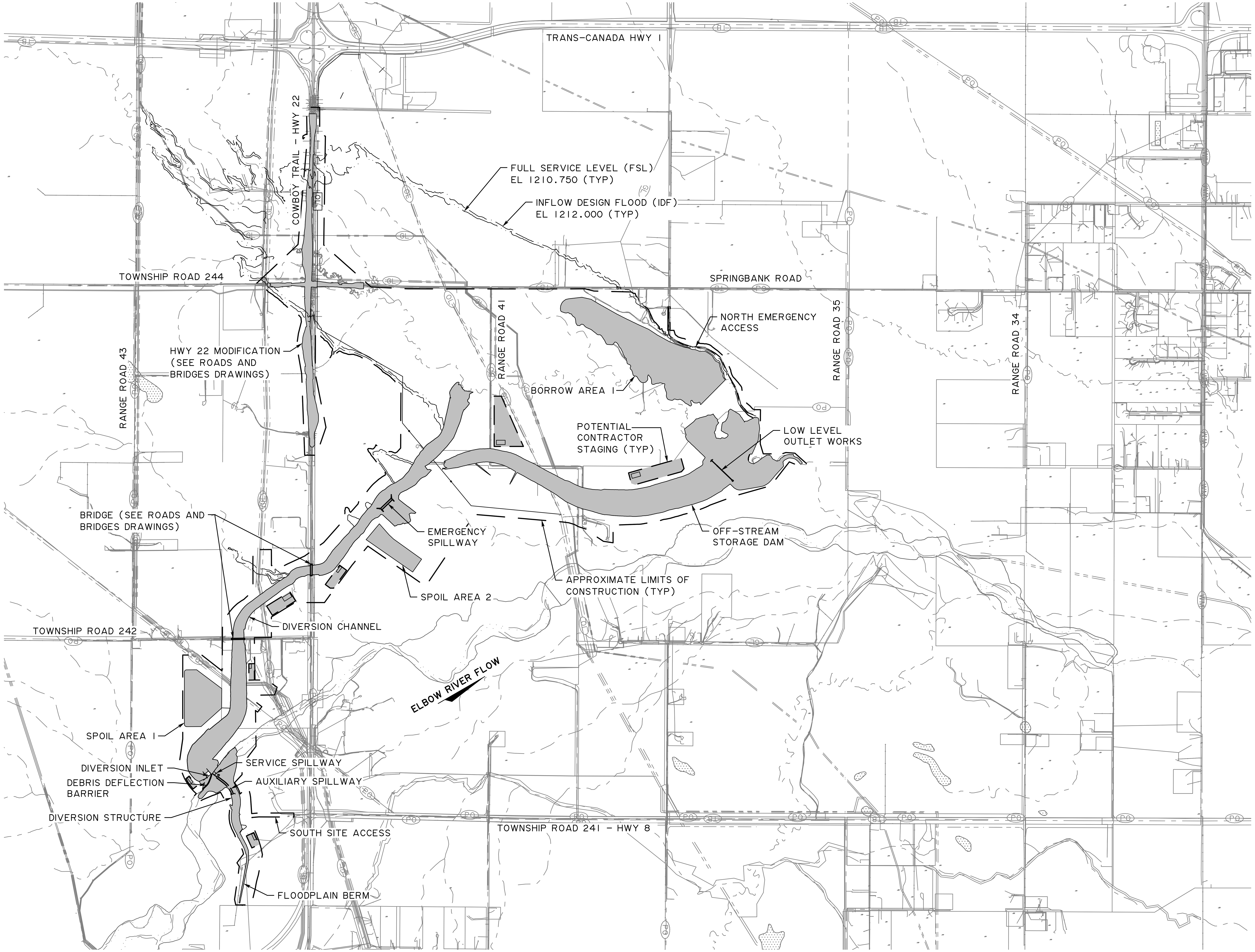
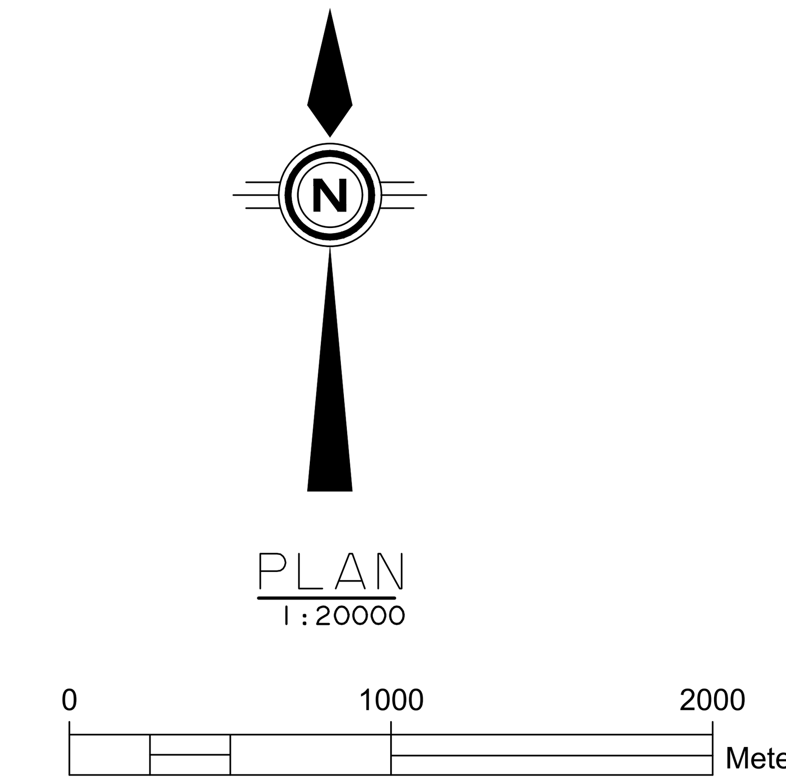
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
GENERAL NOTES AND PROJECT LEGEND

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396A-104	





NOTES:

1. LIMITS OF CONSTRUCTION ARE BASED ON DESIGN PROGRESS AS OF JUNE 30, 2020.
2. BORROW AREAS IDENTIFIED REPRESENT GENERAL LOCATIONS OF PREFERRED BORROW EXCAVATION. ADDITIONAL EXCAVATION MAY OCCUR WITHIN THE RESERVOIR FOOTPRINT.
3. SPOIL AREAS IDENTIFIED ARE DESIGNATED FOR TEMPORARY STORAGE OF TOPSOIL AND OTHER EMBANKMENT MATERIALS OUTSIDE OF THE RESERVOIR FOOTPRINT. FINAL GRADES MAY INCORPORATE THE PLACEMENT OF EXCESS MATERIALS TO BE RESTORED AND VEGETATED WITHIN THE PROJECT LIMITS. SPOIL AREAS FOR DAM CONSTRUCTION WILL OCCUR WITHIN THE RESERVOIR AREA AND ARE TO BE DETERMINED BY THE CONTRACTOR.
4. POTENTIAL CONTRACTOR STAGING AREAS ARE PRELIMINARY. FINAL LOCATION TO BE DETERMINED BY CONTRACTOR.

PlotDate: 2020.06.25 1:12 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\a_general\sheet_files\100_gen\73396A-110.dwg TENDER No. ?

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
PROJECT LAYOUT

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396A-110	

N

PLAN
1:10000

0 400 800 Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT
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GENERAL
RESERVOIR OVERVIEW

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396A-111	

PlotDate: 2019.11.26 3:27 PM Login: Bolubasz, Danielle
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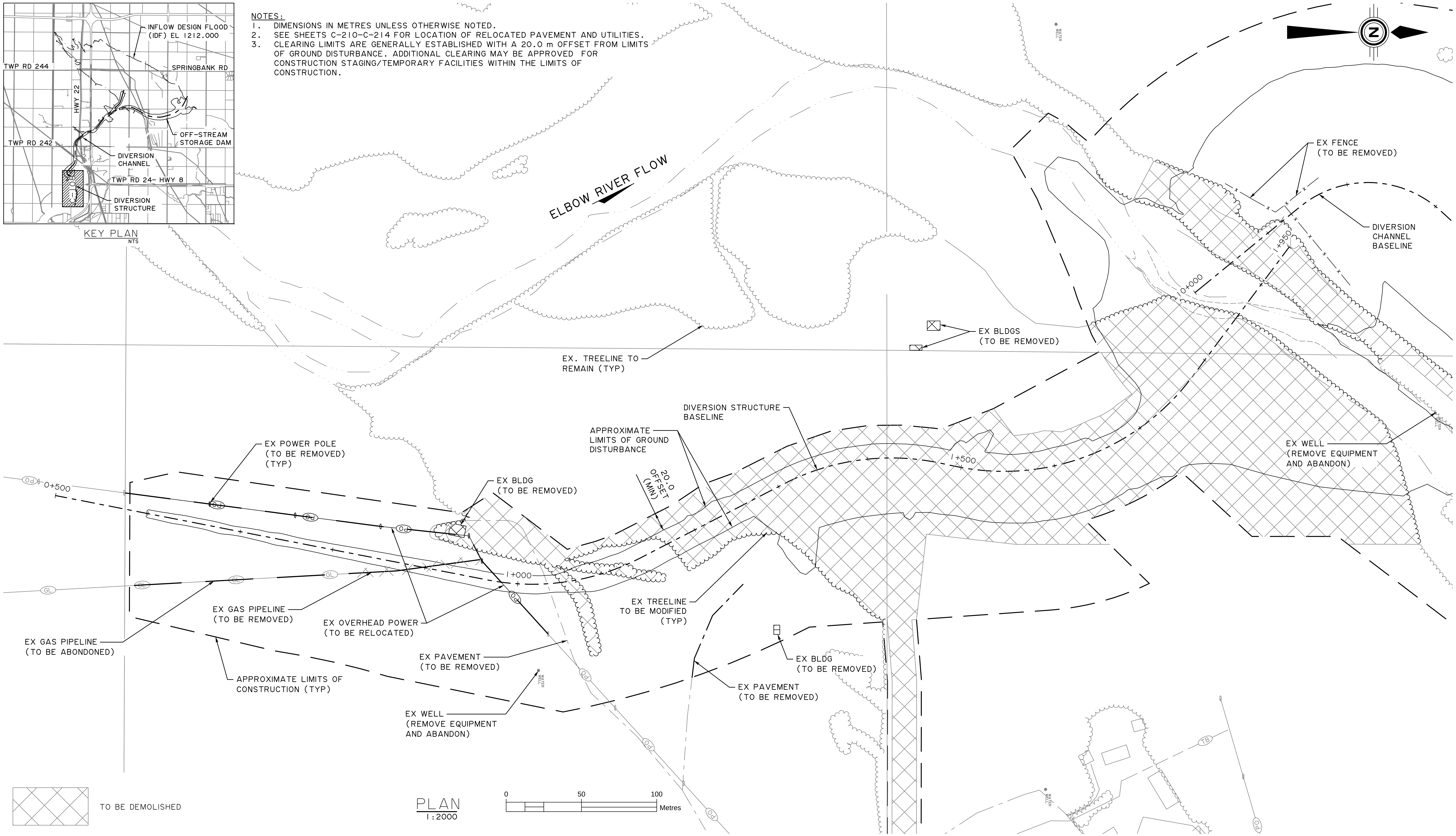
PlotDate: 2019.11.27 3:32 PM Login: Bolubasz, Danielle TENDER No. ?
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\100_gen\73396C-108.dwg

NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. SEE SHEETS C-210-C-214 FOR LOCATION OF RELOCATED PAVEMENT AND UTILITIES.
3. CLEARING LIMITS ARE GENERALLY ESTABLISHED WITH A 20.0 m OFFSET FROM LIMITS OF GROUND DISTURBANCE. ADDITIONAL CLEARING MAY BE APPROVED FOR CONSTRUCTION STAGING/TEMPORARY FACILITIES WITHIN THE LIMITS OF CONSTRUCTION.



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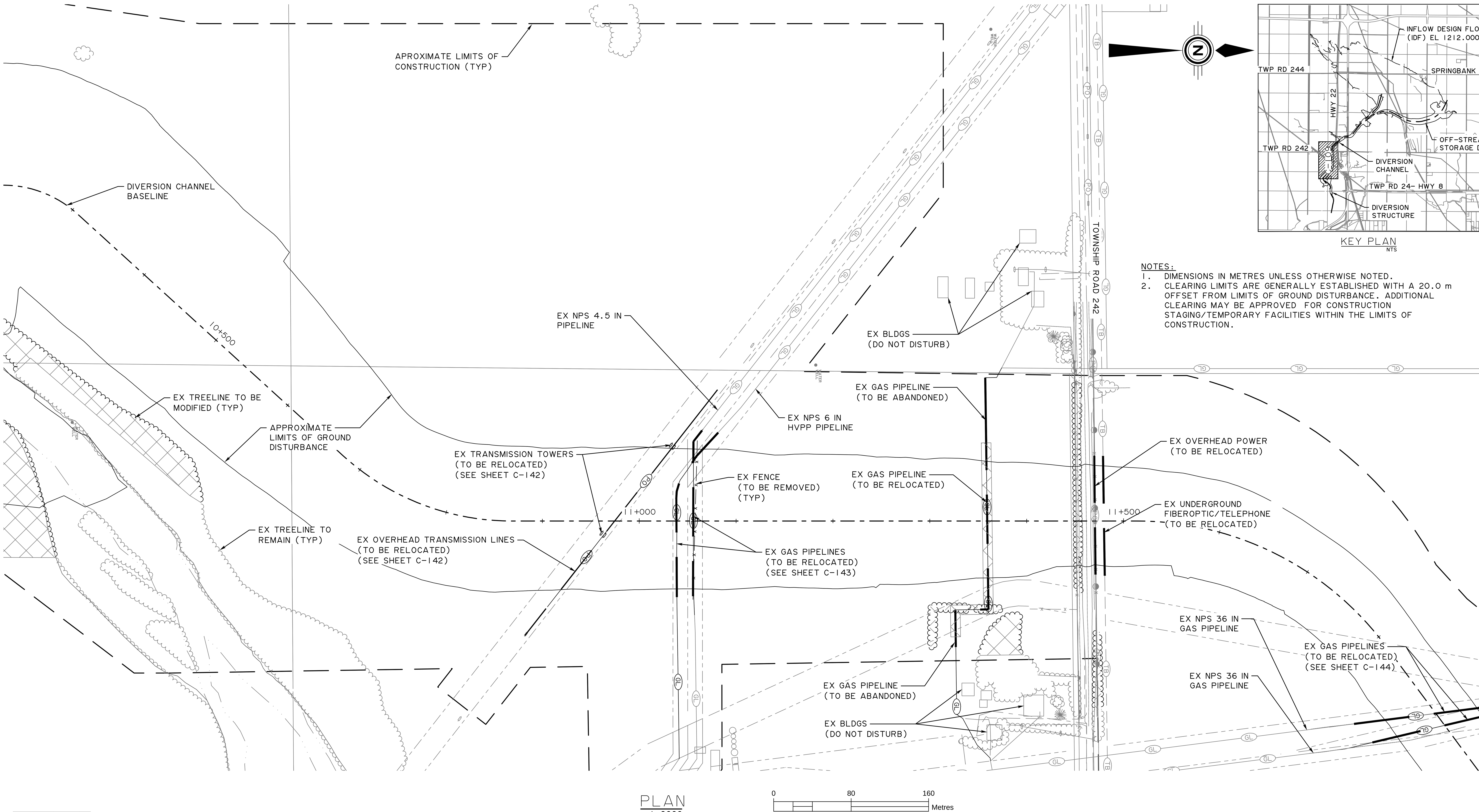
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL DEMOLITION PLAN SHEET 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-108	



NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. CLEARING LIMITS ARE GENERALLY ESTABLISHED WITH A 20.0 m OFFSET FROM LIMITS OF GROUND DISTURBANCE. ADDITIONAL CLEARING MAY BE APPROVED FOR CONSTRUCTION STAGING/TEMPORARY FACILITIES WITHIN THE LIMITS OF CONSTRUCTION.

TO BE DEMOLISHED

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

GENERAL DEMOLITION PLAN SHEET 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-109	

PlotDate: 2019.11.26 5:32 PM Login: Gindling, Kendall TENDER No. ?
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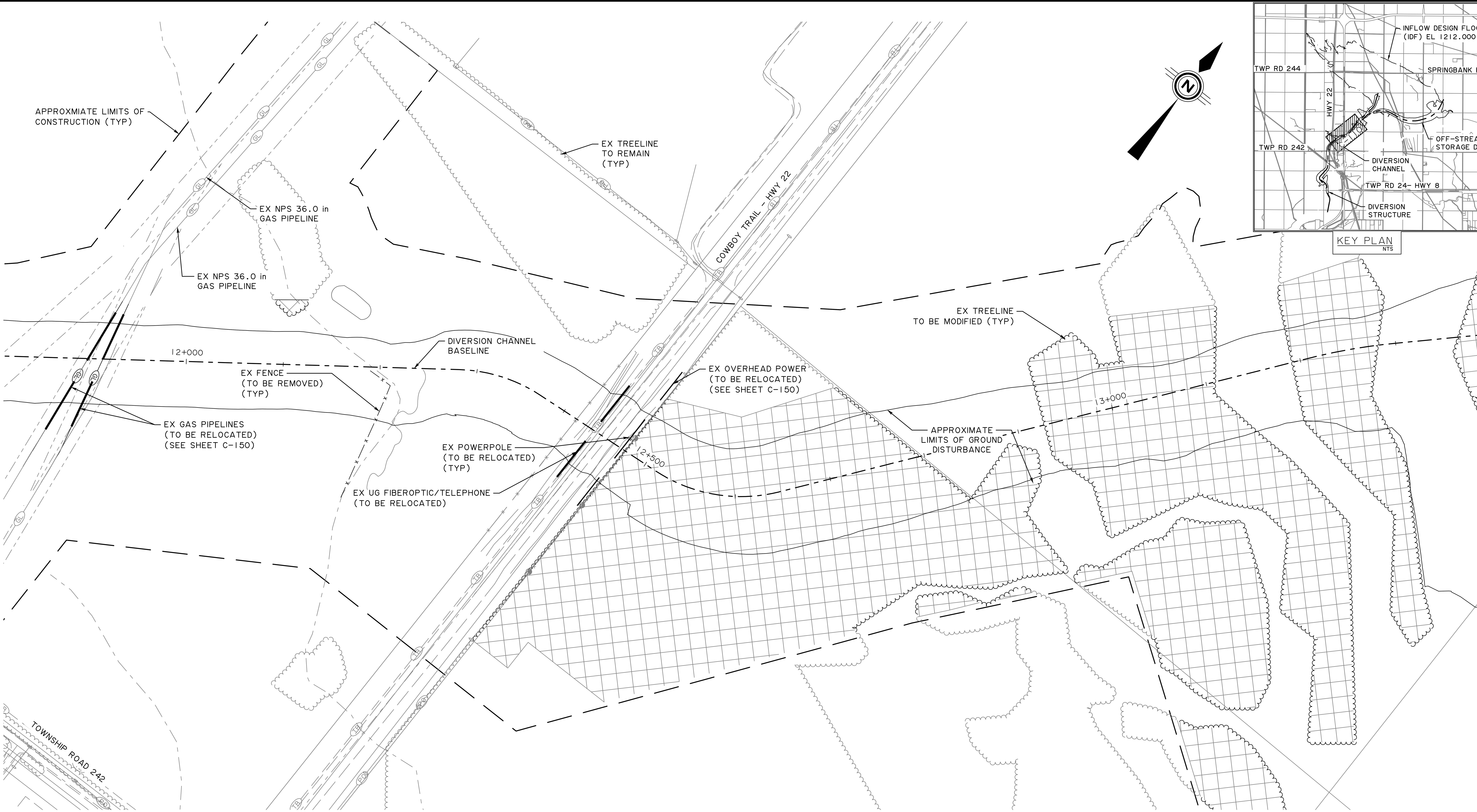

TO BE DEMOLISHED

PLAN
1:2000

0 80 160 Meters

NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. CLEARING LIMITS ARE GENERALLY ESTABLISHED WITH A 20.0 m OFFSET FROM LIMITS OF GROUND DISTURBANCE. ADDITIONAL CLEARING MAY BE APPROVED FOR CONSTRUCTION STAGING/TEMPORARY FACILITIES WITHIN THE LIMITS OF CONSTRUCTION.

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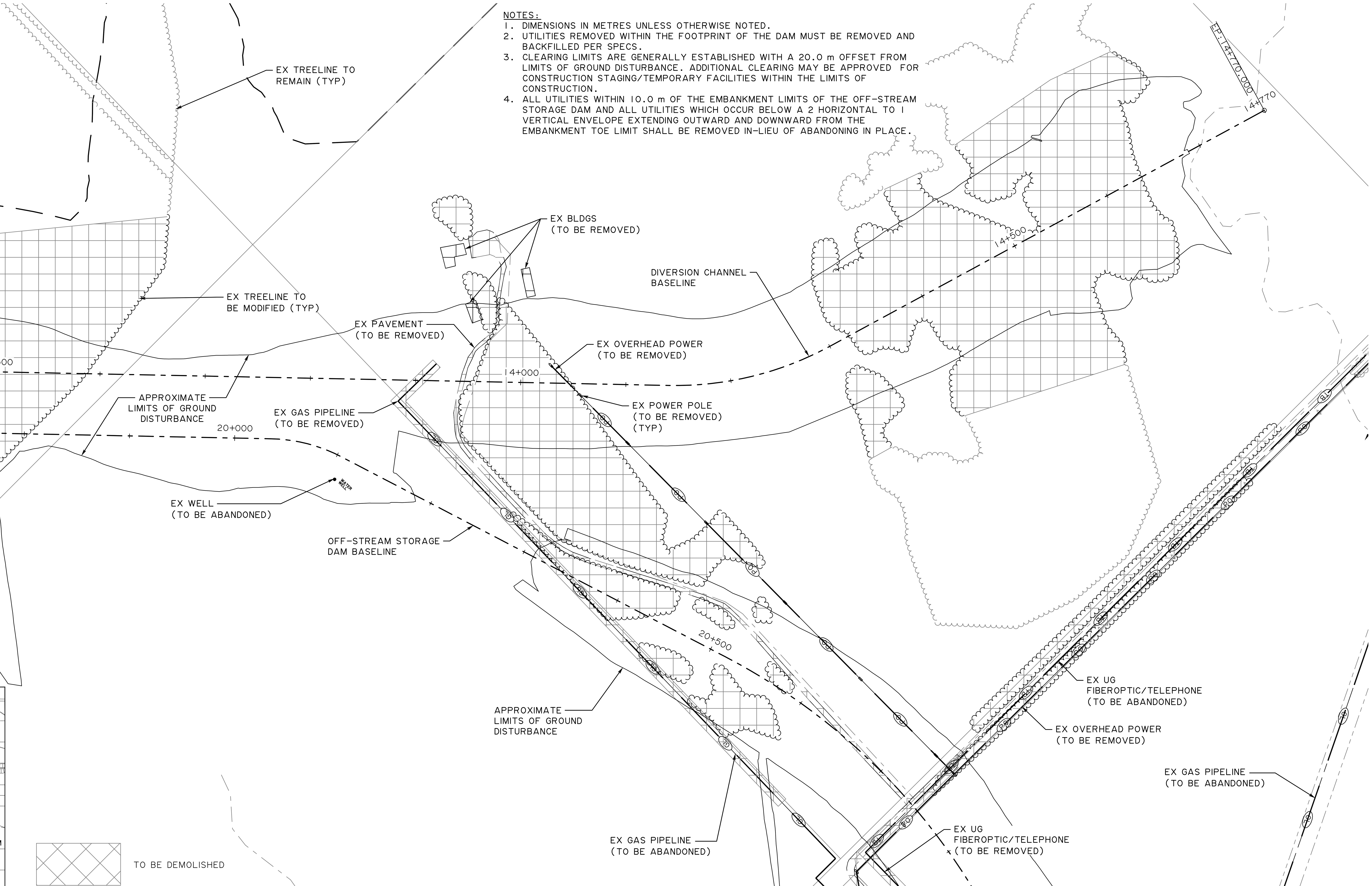
SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

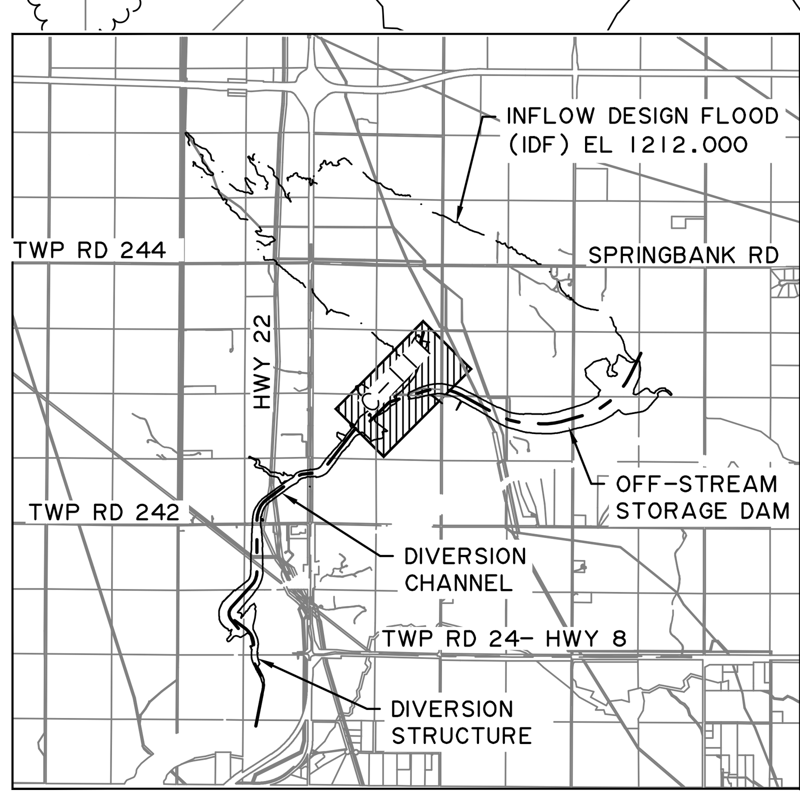
GENERAL DEMOLITION PLAN SHEET 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-110	

PlotDate: 2019.11.27 1:49 PM Login: Richardson, Jillian
AUTOCAD FILE No. \\Us0268-ppfss01\shared_projects\110773396\CAD\water\c_civil\sheet_files\100_gen\73396C-110.dwg TENDER No. ?

NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. UTILITIES REMOVED WITHIN THE FOOTPRINT OF THE DAM MUST BE REMOVED AND BACKFILLED PER SPECS.
3. CLEARING LIMITS ARE GENERALLY ESTABLISHED WITH A 20.0 m OFFSET FROM LIMITS OF GROUND DISTURBANCE. ADDITIONAL CLEARING MAY BE APPROVED FOR CONSTRUCTION STAGING/TEMPORARY FACILITIES WITHIN THE LIMITS OF CONSTRUCTION.
4. ALL UTILITIES WITHIN 10.0 m OF THE EMBANKMENT LIMITS OF THE OFF-STREAM STORAGE DAM AND ALL UTILITIES WHICH OCCUR BELOW A 2 HORIZONTAL TO 1 VERTICAL ENVELOPE EXTENDING OUTWARD AND DOWNWARD FROM THE EMBANKMENT TOE LIMIT SHALL BE REMOVED IN-LIEU OF ABANDONING IN PLACE.





KEY PLAN
NTS

TO BE DEMOLISHED

PLAN
1:2000

0 50 100 Meters

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GENERAL
DEMOLITION PLAN
SHEET 4

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-111	

PlotDate: 2019.11.26 5:48 PM Login: Gindling, Kendall TENDER No. ?
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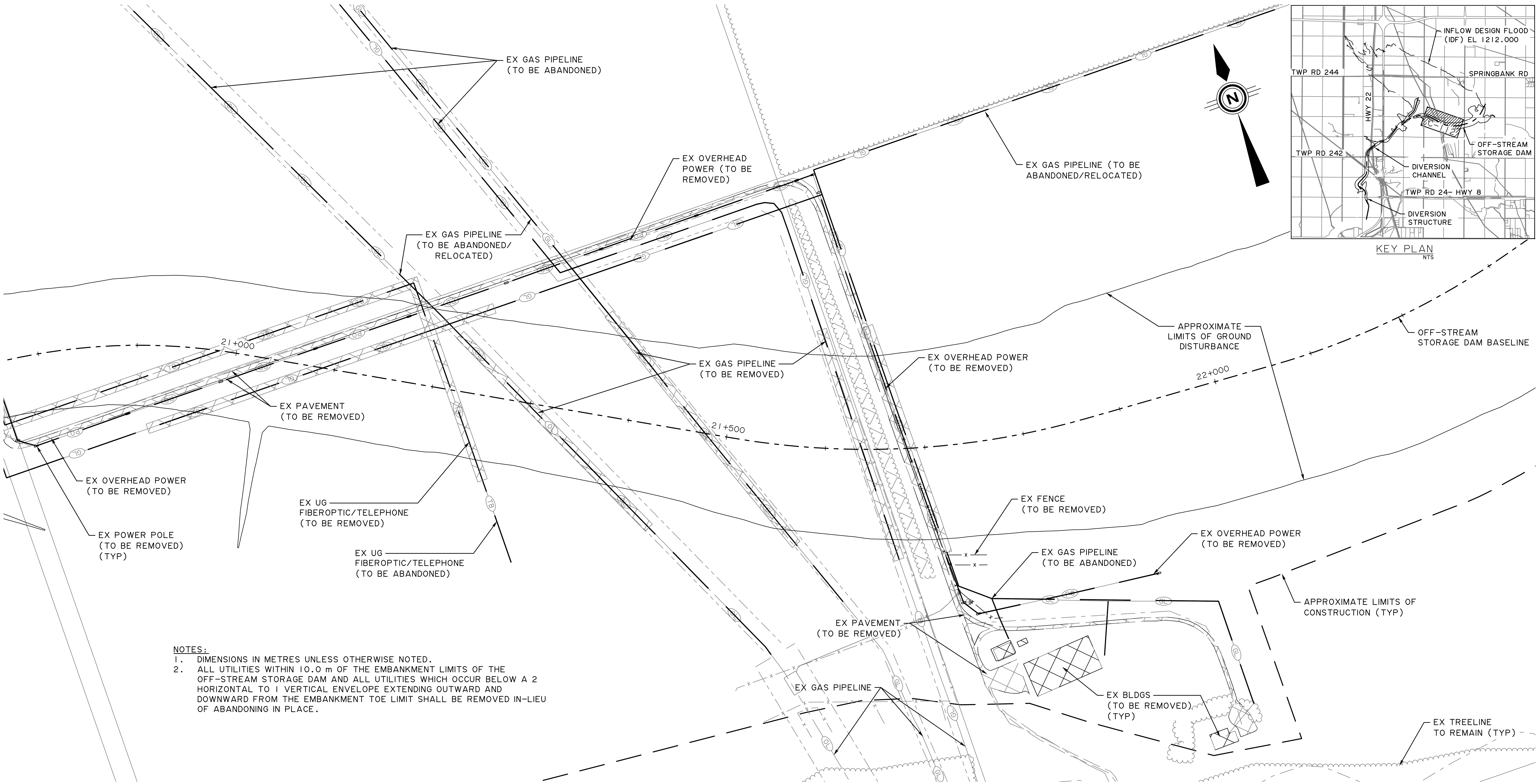

NOTES:

1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. ALL UTILITIES WITHIN 10.0 m OF THE EMBANKMENT LIMITS OF THE OFF-STREAM STORAGE DAM AND ALL UTILITIES WHICH OCCUR BELOW A 2 HORIZONTAL TO 1 VERTICAL ENVELOPE EXTENDING OUTWARD AND DOWNWARD FROM THE EMBANKMENT TOE LIMIT SHALL BE REMOVED IN-LIEU OF ABANDONING IN PLACE.

PLAN
1:2000

0 80 160 Metres

TO BE DEMOLISHED

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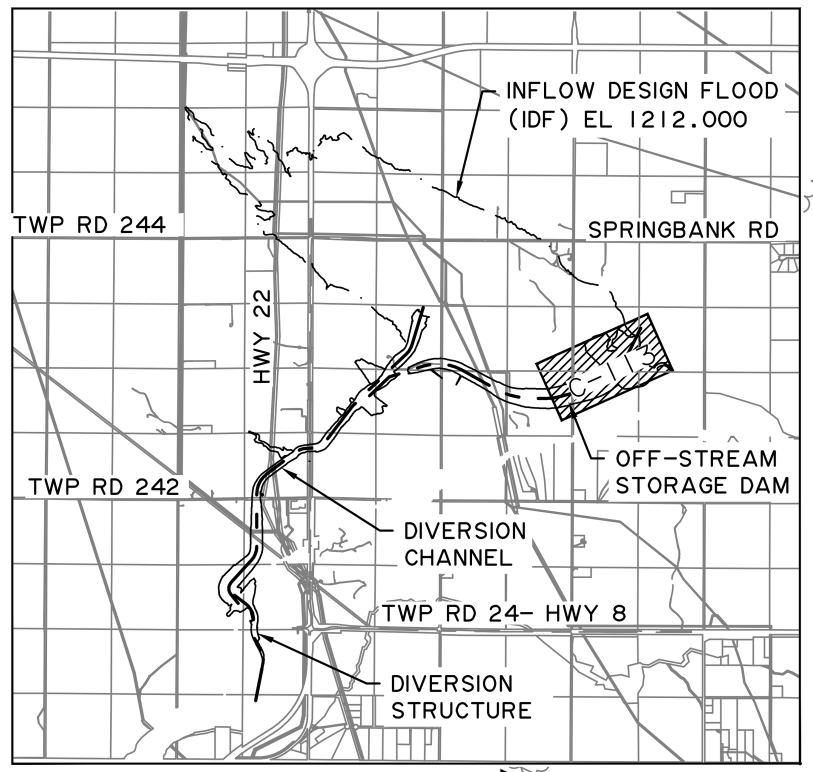
GENERAL
DEMOLITION PLAN
SHEET 5

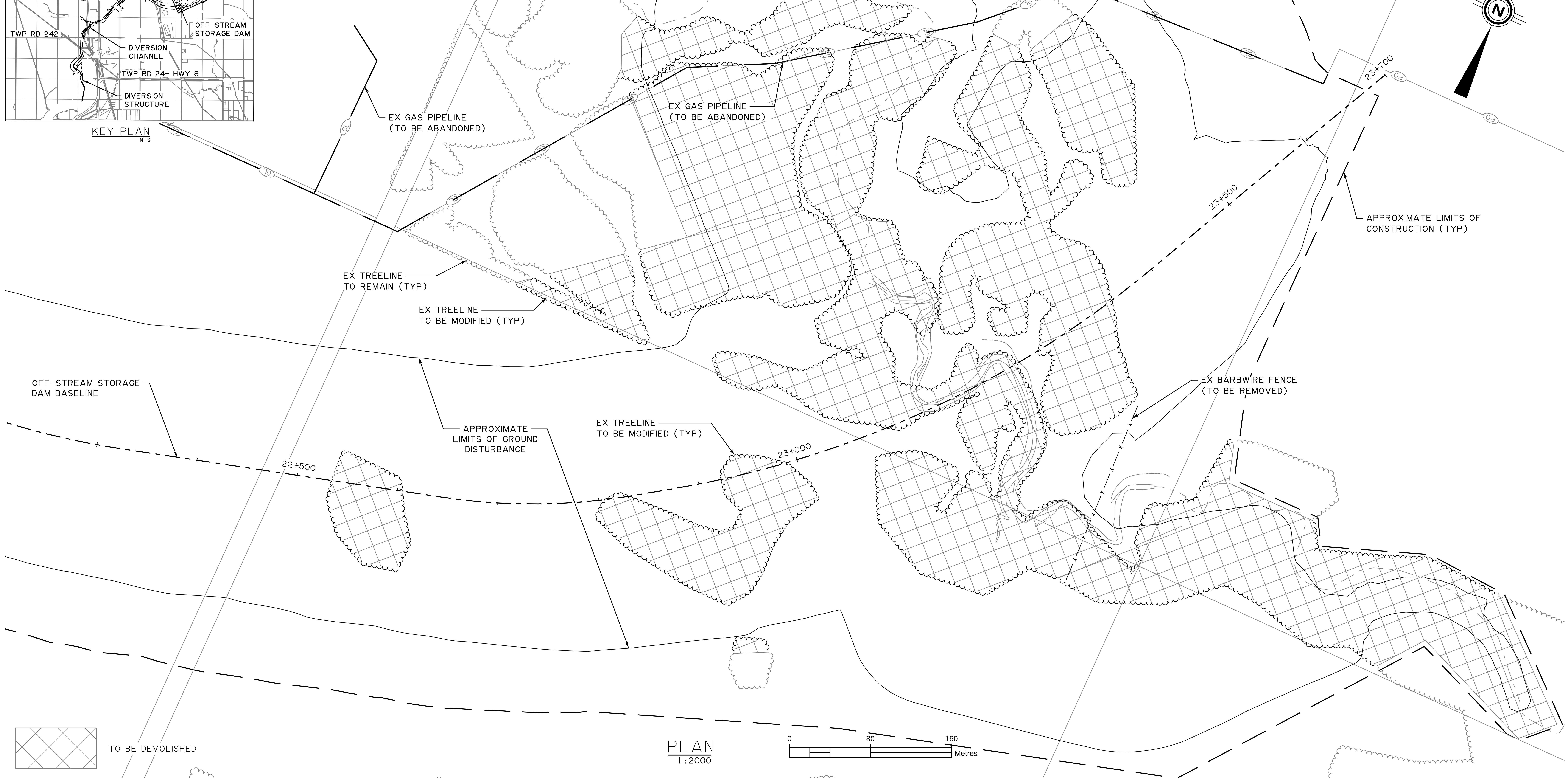
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-112	

PlotDate: 2019.11.27 2:15 PM Login: Richardson, Jillian
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\100_gen\73396C-112.dwg TENDER No. ?

NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. CLEARING LIMITS ARE GENERALLY ESTABLISHED WITH A 20.0 m OFFSET FROM LIMITS OF GROUND DISTURBANCE. ADDITIONAL CLEARING MAY BE APPROVED FOR CONSTRUCTION STAGING/TEMPORARY FACILITIES WITHIN THE LIMITS OF CONSTRUCTION.
3. ALL UTILITIES WITHIN 10.0 m OF THE EMBANKMENT LIMITS OF THE OFF-STREAM STORAGE DAM AND ALL UTILITIES WHICH OCCUR BELOW A 2 HORIZONTAL TO 1 VERTICAL ENVELOPE EXTENDING OUTWARD AND DOWNWARD FROM THE EMBANKMENT TOE LIMIT SHALL BE REMOVED IN-LIEU OF ABANDONING IN PLACE.



KEY PLAN
NTS



TO BE DEMOLISHED

PLAN
1:2000

0 80 160 Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

GENERAL DEMOLITION PLAN SHEET 6

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-113	

PlotDate: 2019.11.26 6:01 PM Login: Gindling, Kendall
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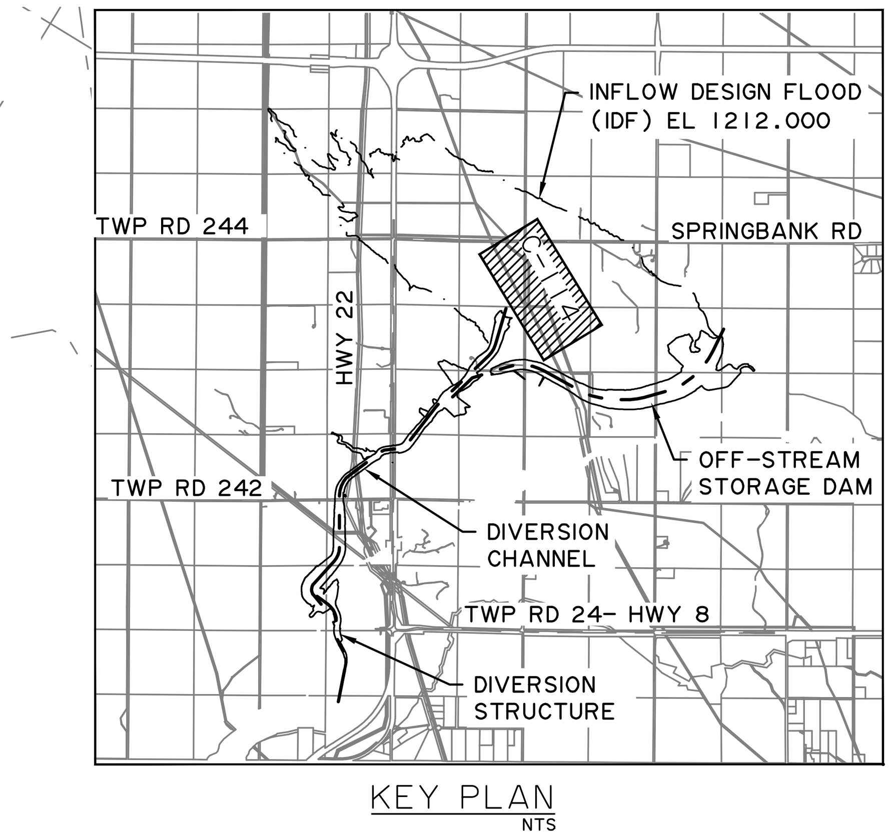
NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.



KEY PLAN
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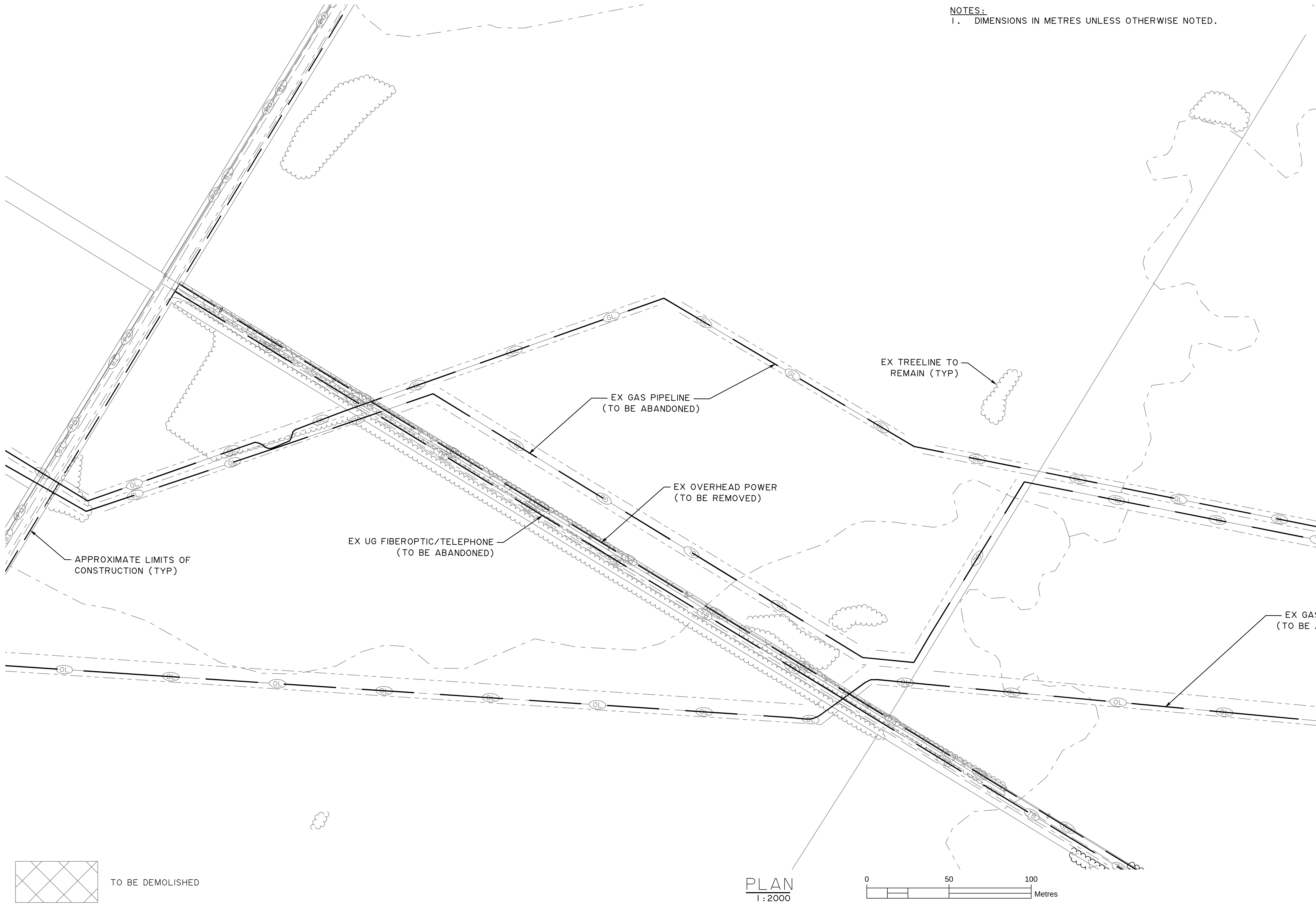

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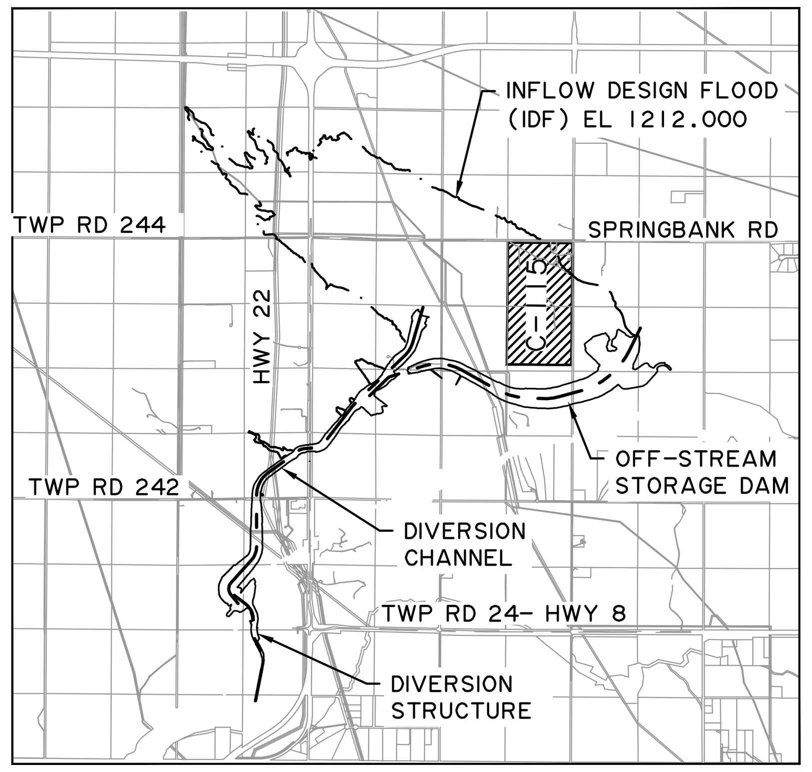
GENERAL
DEMOLITION PLAN
SHEET 7

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-114	



KEY PLAN
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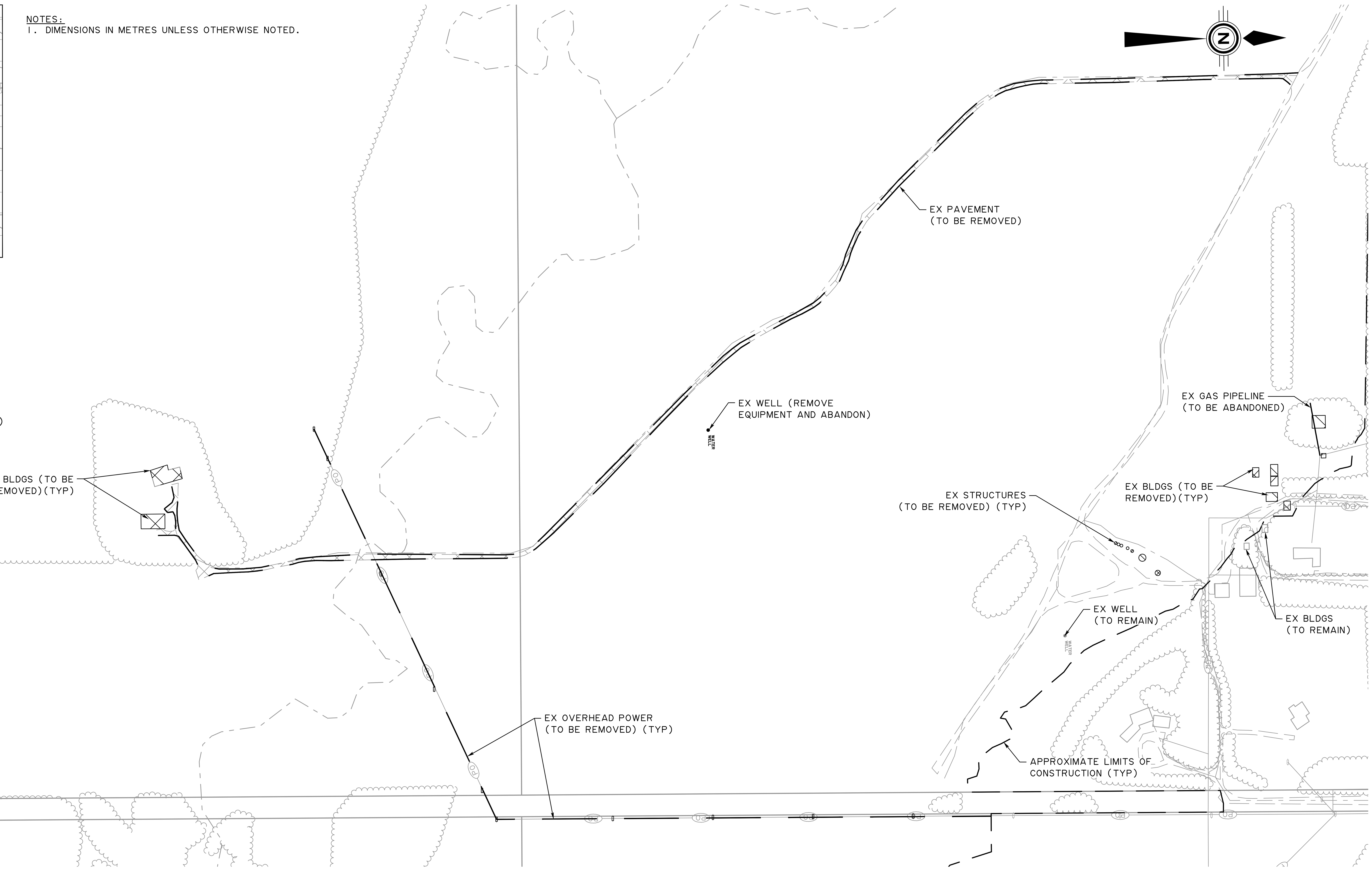
NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.



PLAN
1:2000

0 80 160 Metres

TO BE DEMOLISHED

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PRELIMINARY DESIGN

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DESIGNER
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CHECKER
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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL DEMOLITION PLAN SHEET 8

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-115	

PlotDate: 2019.11.26 6:12 PM Login: Gindling, Kendall
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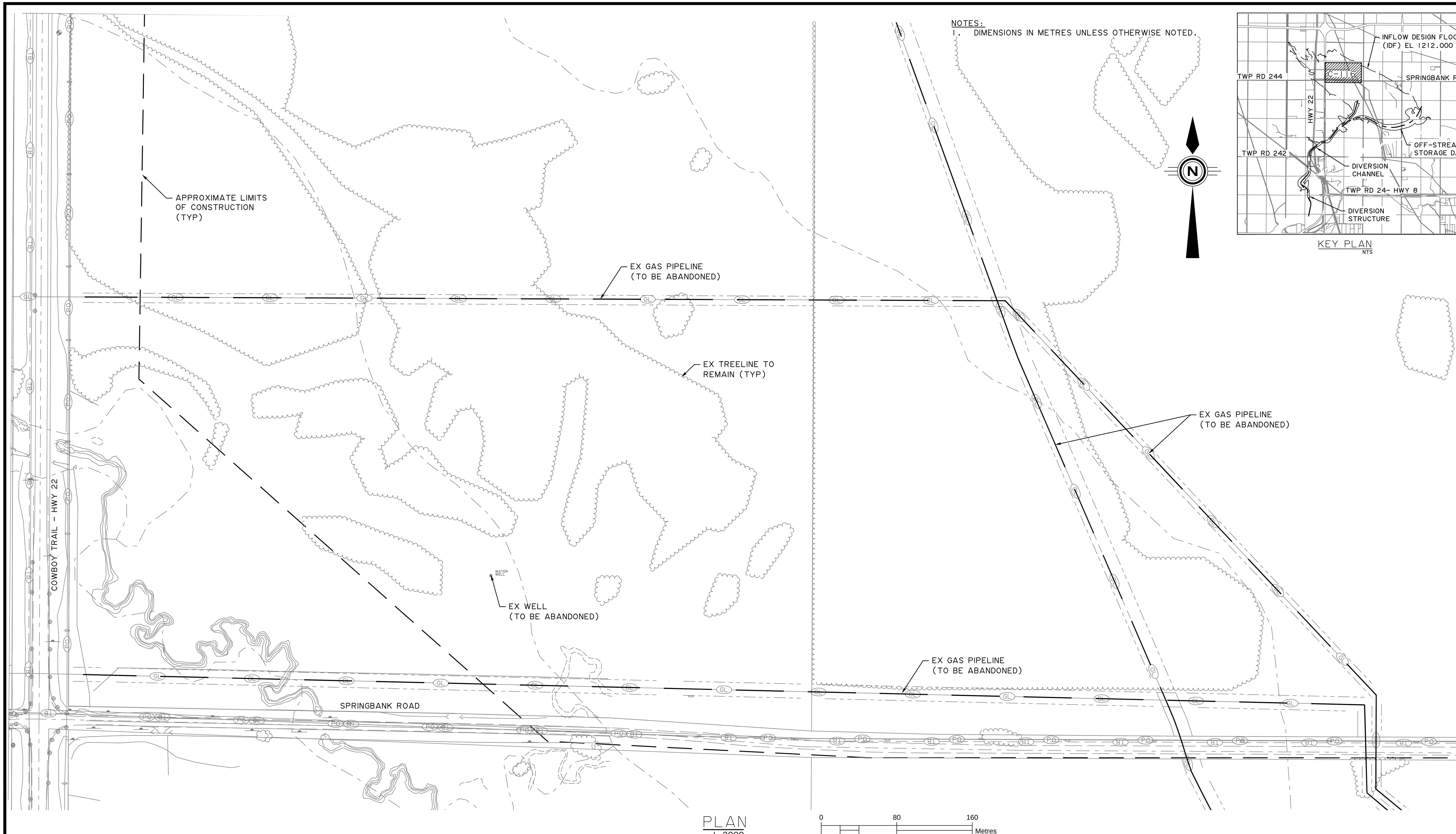

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

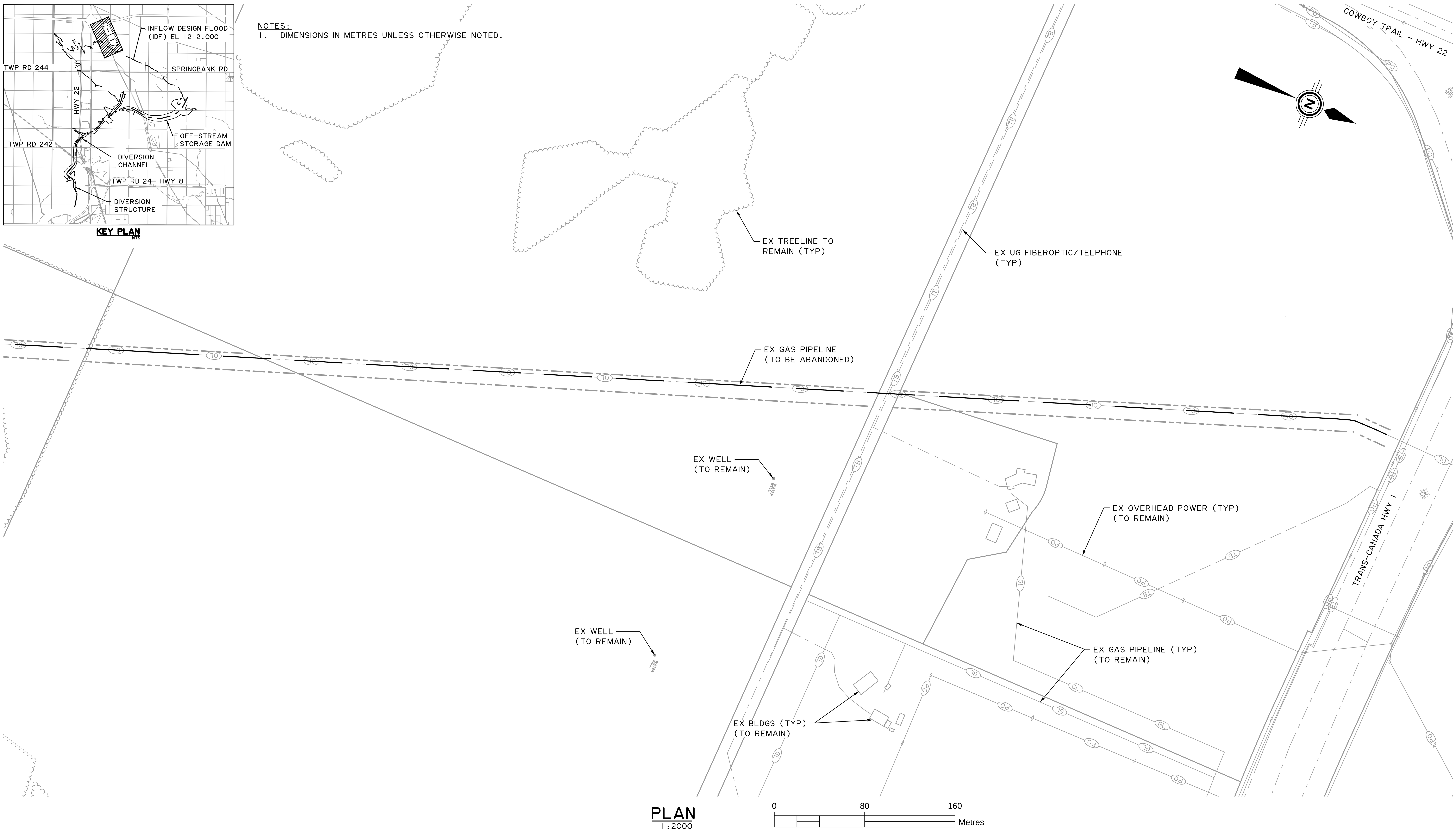
GENERAL DEMOLITION PLAN SHEET 9

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-116	

PlotDate: 2019.11.26 6:15 PM Login: Gindling, Kendall

AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\100_gen\73396C-116.dwg TENDER No. ?

NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.



PLAN
1:2000

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
DEMOLITION PLAN
SHEET 10

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-117	

PlotDate: 2019.11.26 6:20 PM Login: Gindling, Kendall
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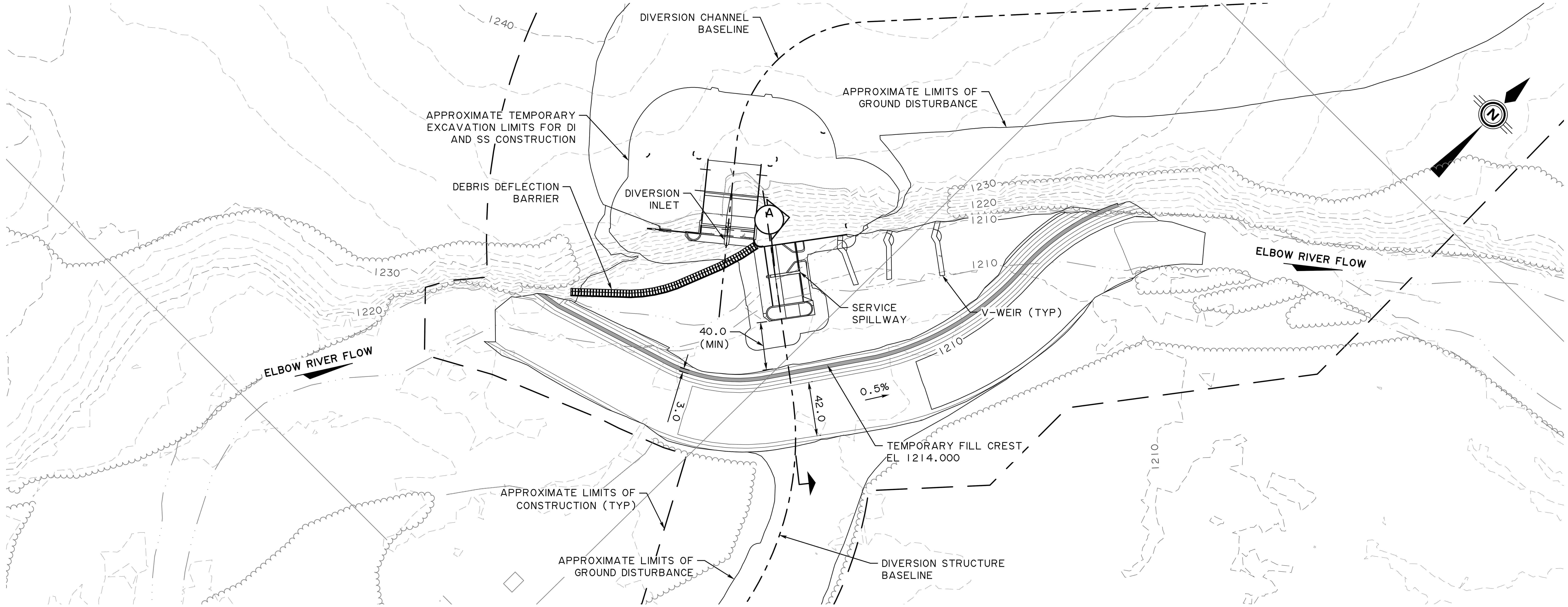

PLAN
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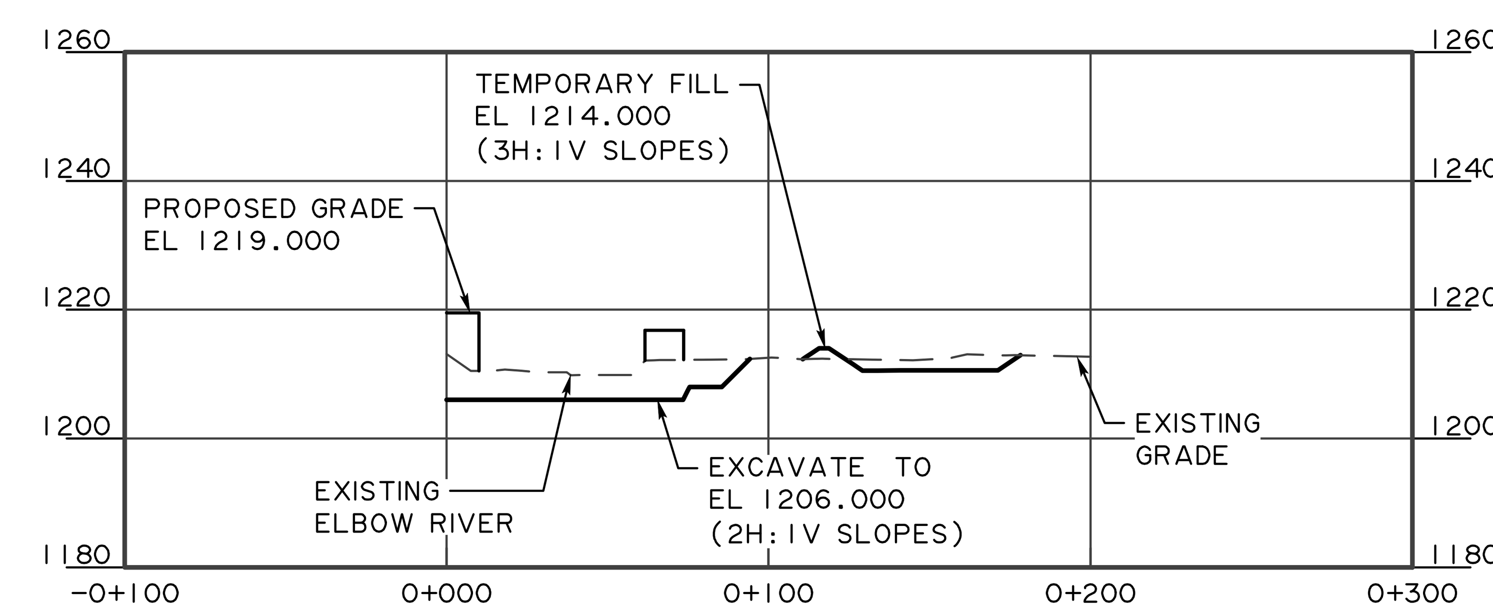

A SECTION A-A
H 1:2000 V 1:1000

NOTES:
1. 2:1 EXCAVATION SLOPE IS USED TO DETERMINE CONSTRUCTION EXCAVATION QUANTITIES. CONTRACTOR TO DETERMINE ACTUAL SLOPE TO BE USED.
2. CONTRACTOR TO COMPLY WITH ENVIRONMENTAL DOCUMENT AND BRIDGE RIVER ACCORDINGLY.
3. SERVICE SPILLWAY NOT SHOWN IN SECTION.

CONSTRUCTION SEQUENCE
PHASE 1: PLACE TEMPORARY DIVERSION FILL AND DIVERT ELBOW RIVER FLOW
PHASE 2: EXCAVATE DIVERSION INLET AND SERVICE SPILLWAY
PHASE 3: CONSTRUCT DIVERSION INLET AND SERVICE SPILLWAY
PHASE 4: REMOVE TEMPORARY DIVERSION FILL AND REROUTE ELBOW THROUGH SERVICE SPILLWAY
PHASE 5: PLACE ADDITIONAL DIVERSION FILL AS NEEDED AND CONSTRUCT AUXILIARY SPILLWAY

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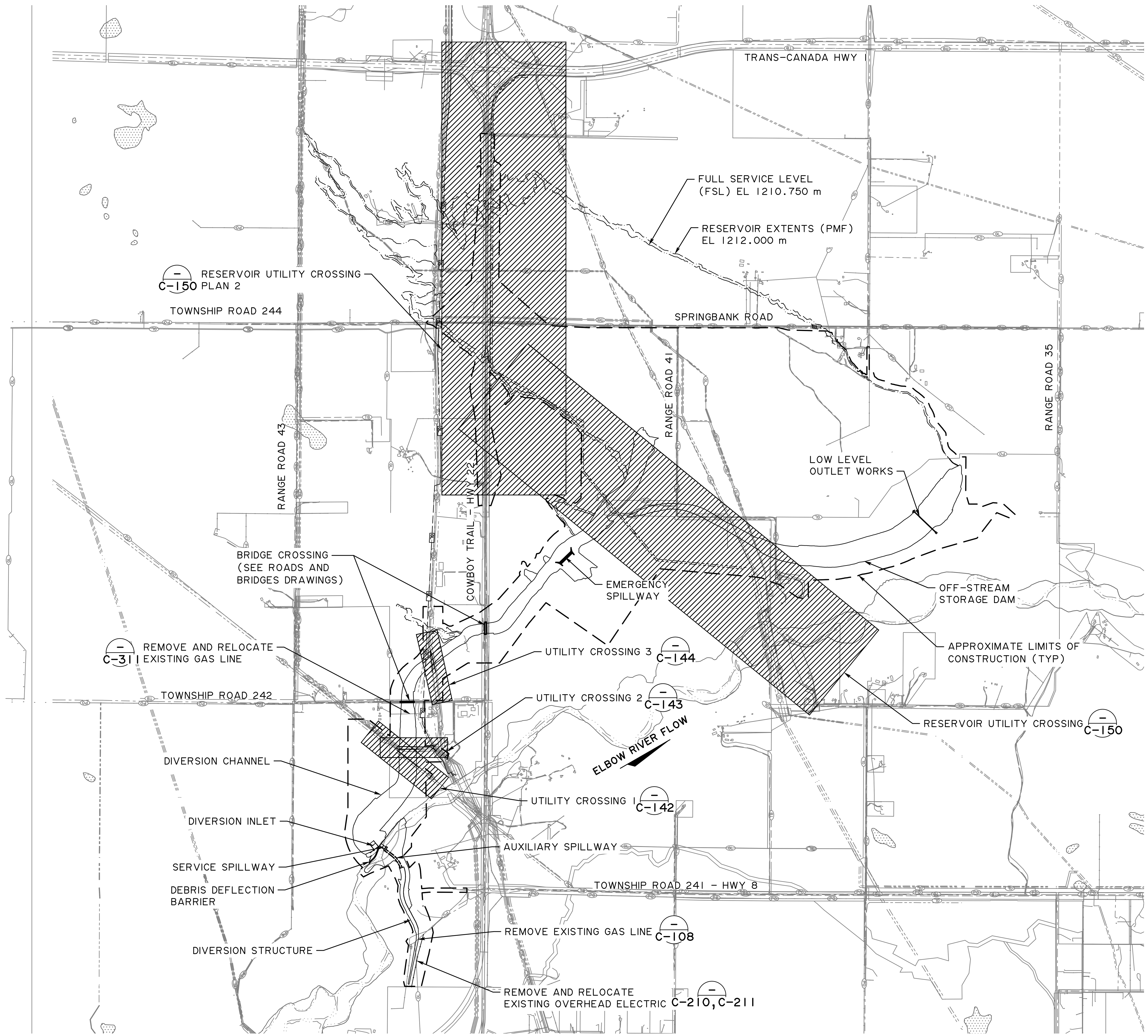
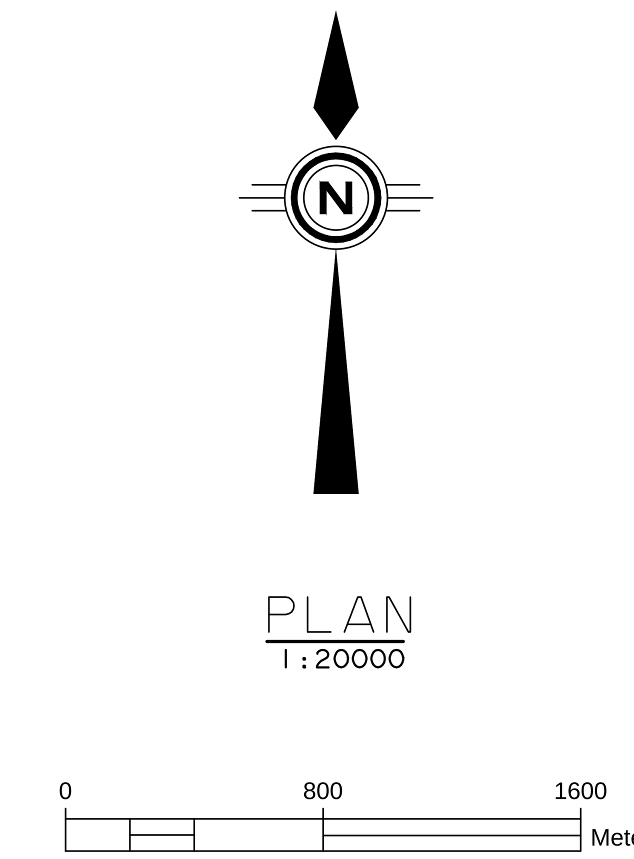
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
TEMPORARY ELBOW RIVER DIVERSION 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-130	

PlotDate: 2019.12.02 3:18 PM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\100_gen\73396C-130.dwg TENDER No. ?





PLAN
1:20000

NOTES:
1. UTILITY RELOCATION SHALL BE COORDINATED WITH ENGINEER AND OWNER WITH CONSIDERATION TO OVERALL CONSTRUCTION SEQUENCING.

PlotDate: 2019.11.27 1:16 PM Login: Bolubasz, Danielle TENDER No. ?
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\100_gen\73396C-140.dwg

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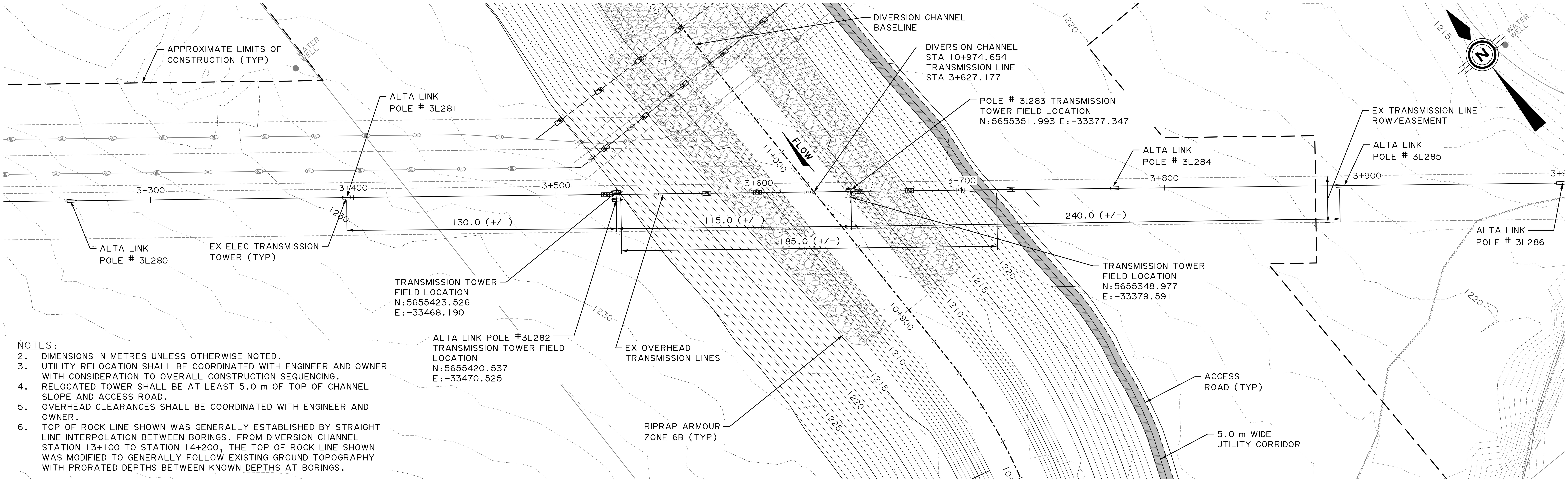
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
SITE UTILITIES OVERVIEW

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-140	

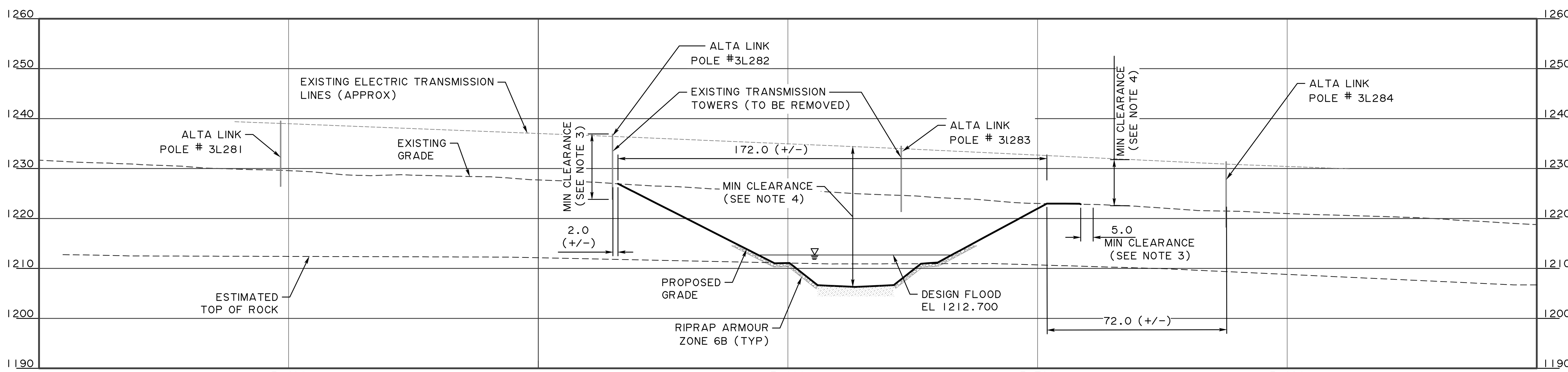


NOTES:
2. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
3. UTILITY RELOCATION SHALL BE COORDINATED WITH ENGINEER AND OWNER WITH CONSIDERATION TO OVERALL CONSTRUCTION SEQUENCING.
4. RELOCATED TOWER SHALL BE AT LEAST 5.0 m OF TOP OF CHANNEL SLOPE AND ACCESS ROAD.
5. OVERHEAD CLEARANCES SHALL BE COORDINATED WITH ENGINEER AND OWNER.
6. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM DIVERSION CHANNEL STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.

LEGEND
ALTA LINK POWERLINE

ELECTRIC TRANSMISSION LINE CROSSING PLAN
1:1000

0 40 80 Metres



ELECTRIC TRANSMISSION LINE CROSSING PROFILE
1:1000

PlotDate: 2019.12.02 3:27 PM Login: Bolubasz, Danielle TENDER No. ?
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\100_gen\73396C-142.dwg

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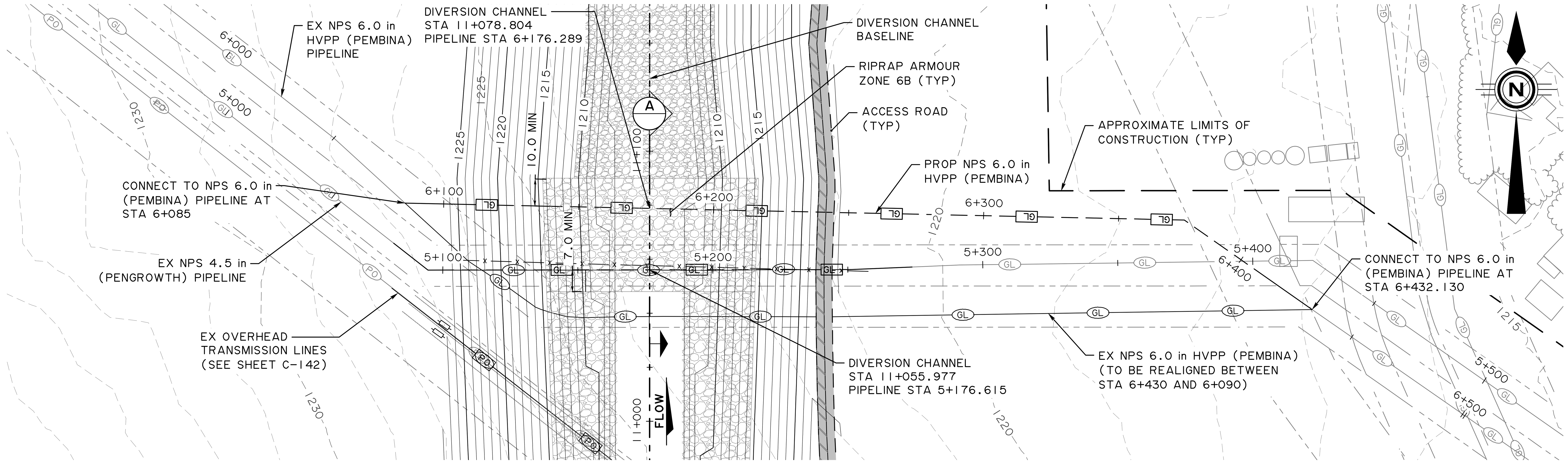
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
DIVERSION CHANNEL
UTILITY CROSSING 1

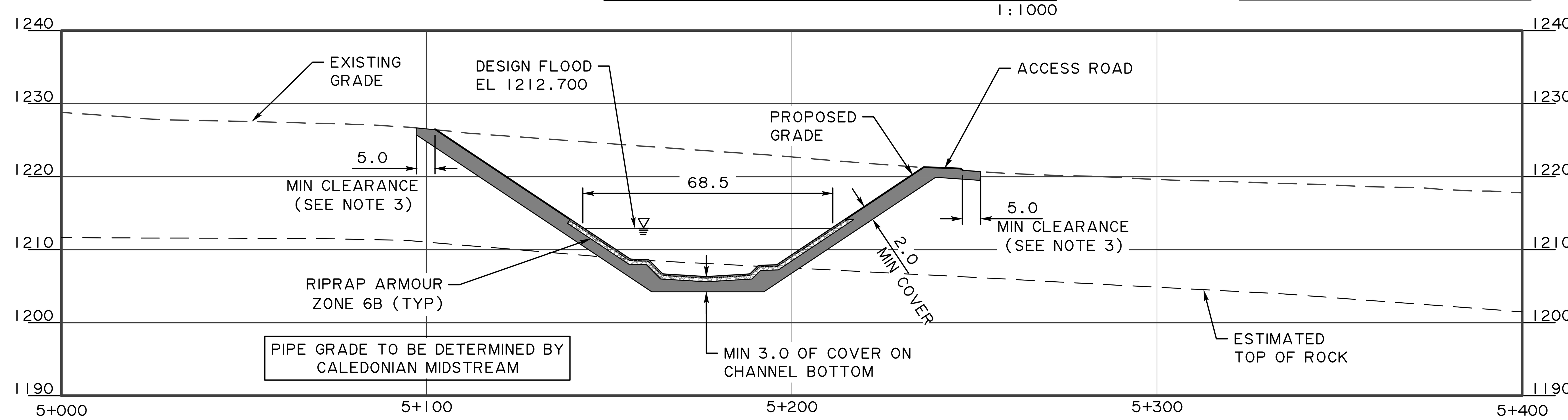
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-142	



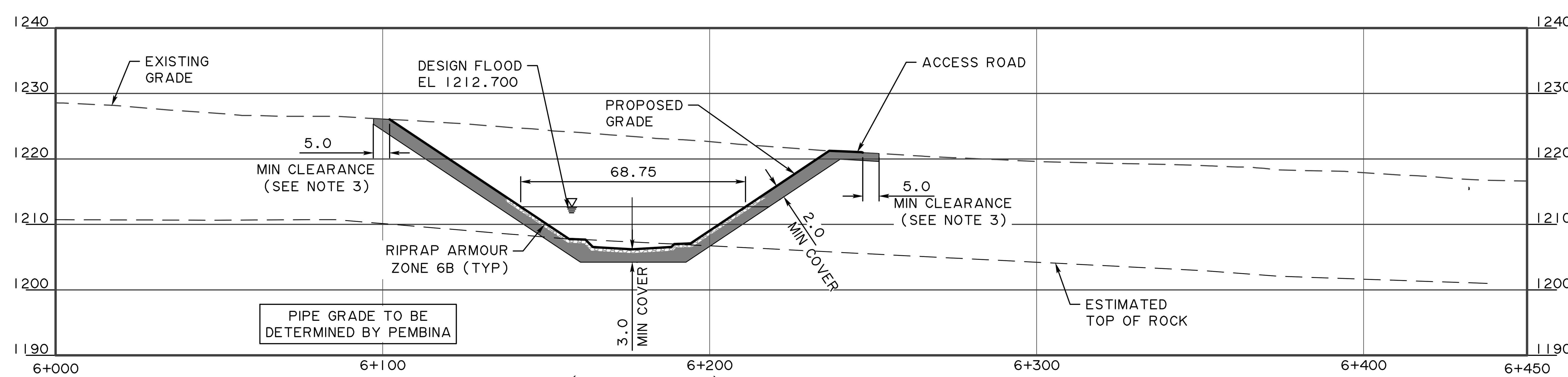
PIPELINE CROSSING PLAN
1:1000

0 40 80 Metres

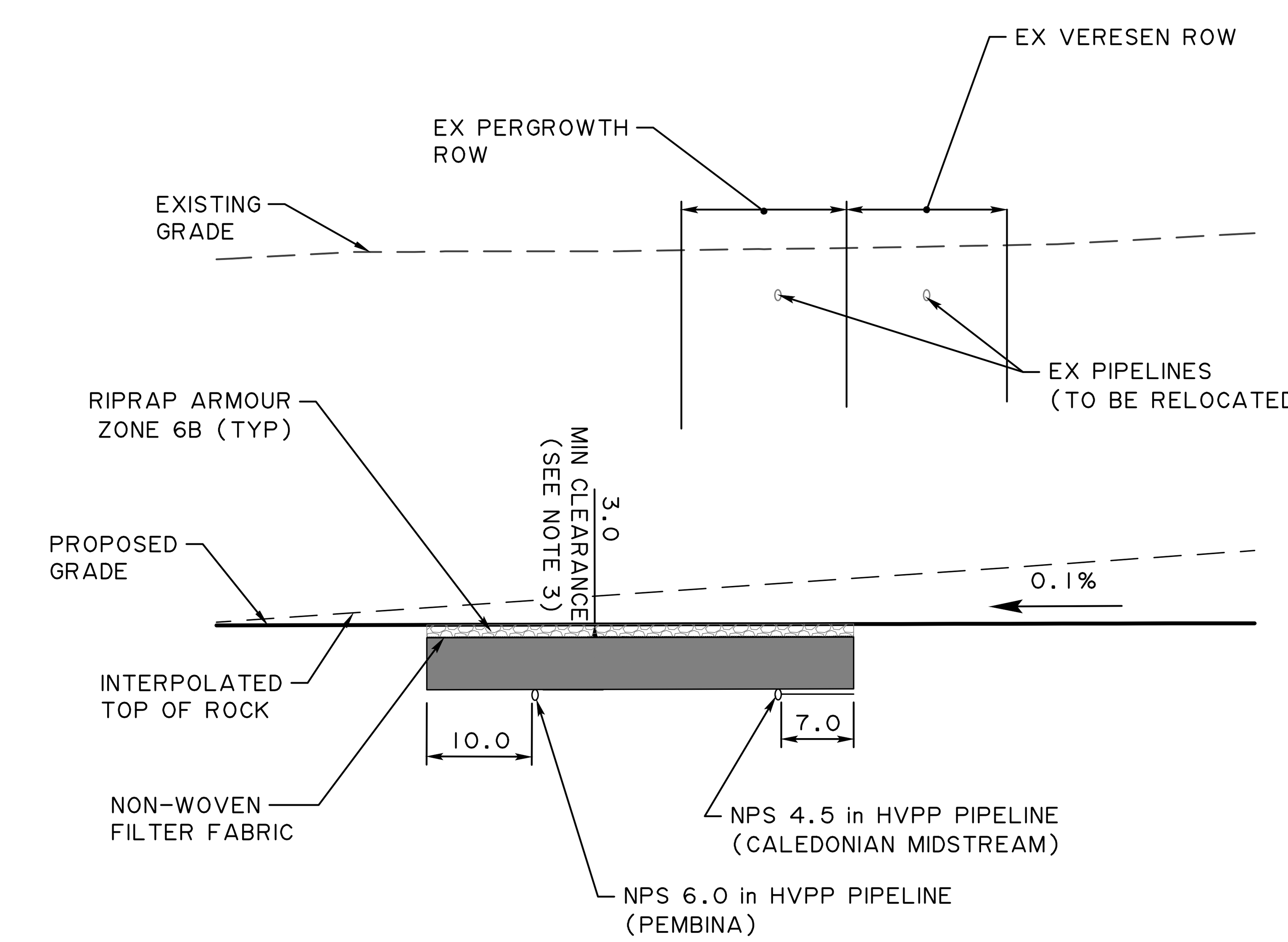
NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM DIVERSION CHANNEL STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.



PIPELINE CROSSING (CALEDONIAN MIDSTREAM) PROFILE
H 1:1000 V 1:500



PIPELINE CROSSING (PEMBINA) PROFILE
H 1:1000 V 1:500



SECTION A
H 1:500 V 1:250

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

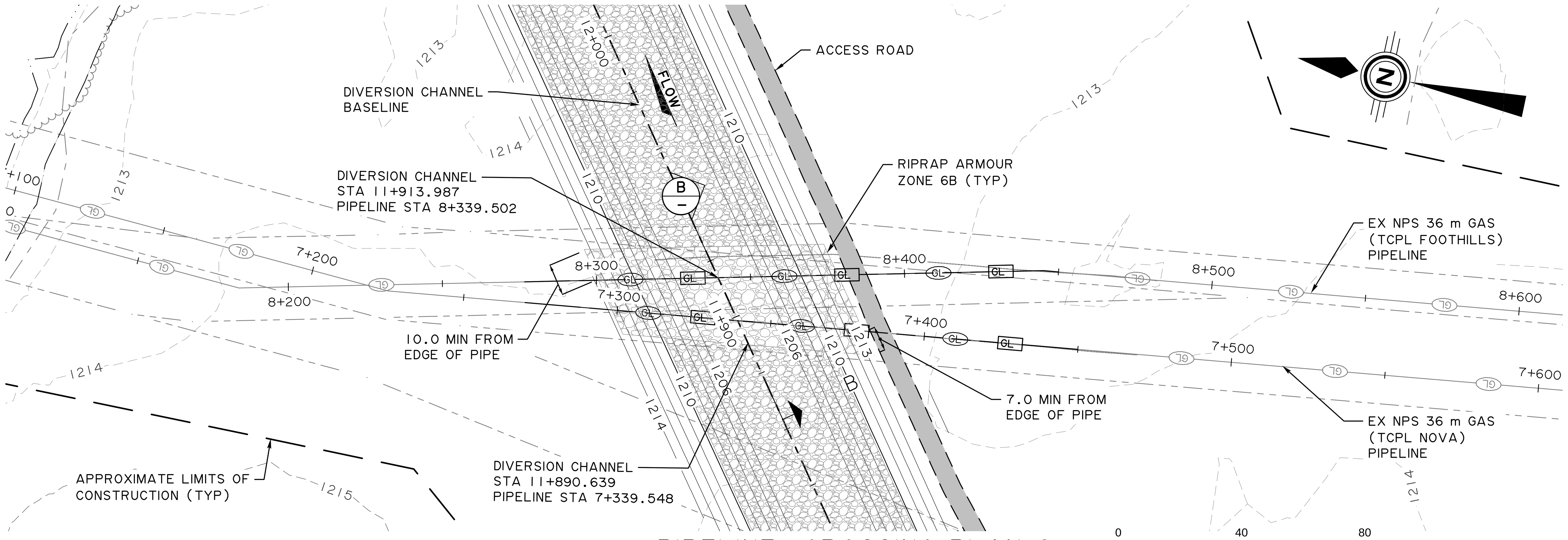
GENERAL
DIVERSION CHANNEL
UTILITY CROSSING 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-143	

DEPARTMENT BAR CODE

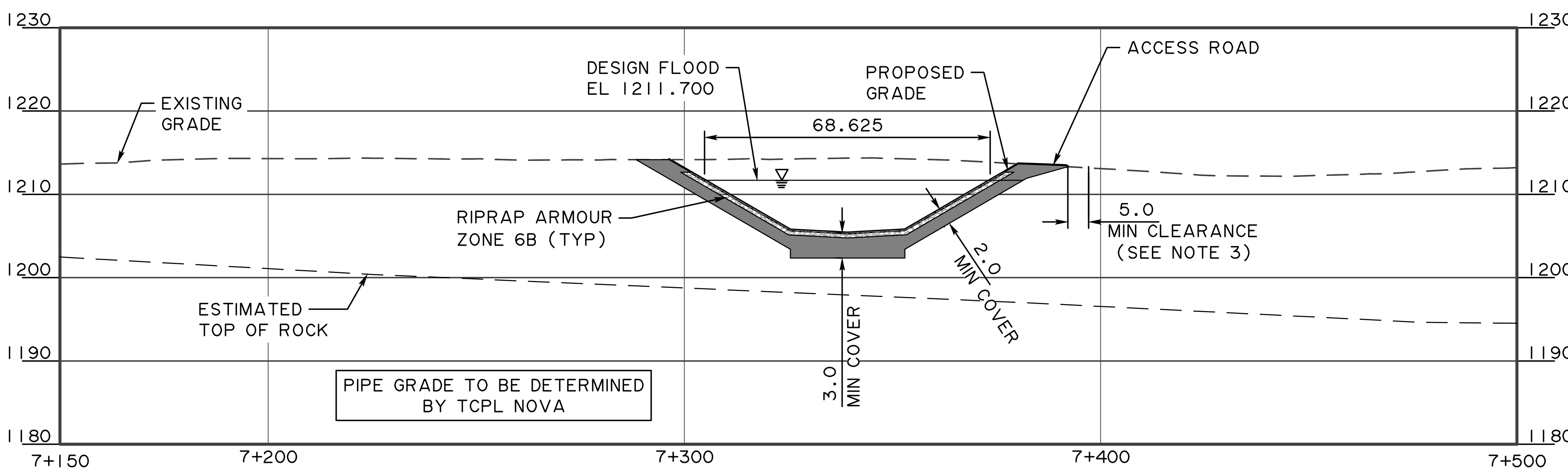
TENDER No. ?

PlotDate: 2019.12.03 9:38 AM Login: Richardson, Jillian
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\100_gen\73396C-143.dwg

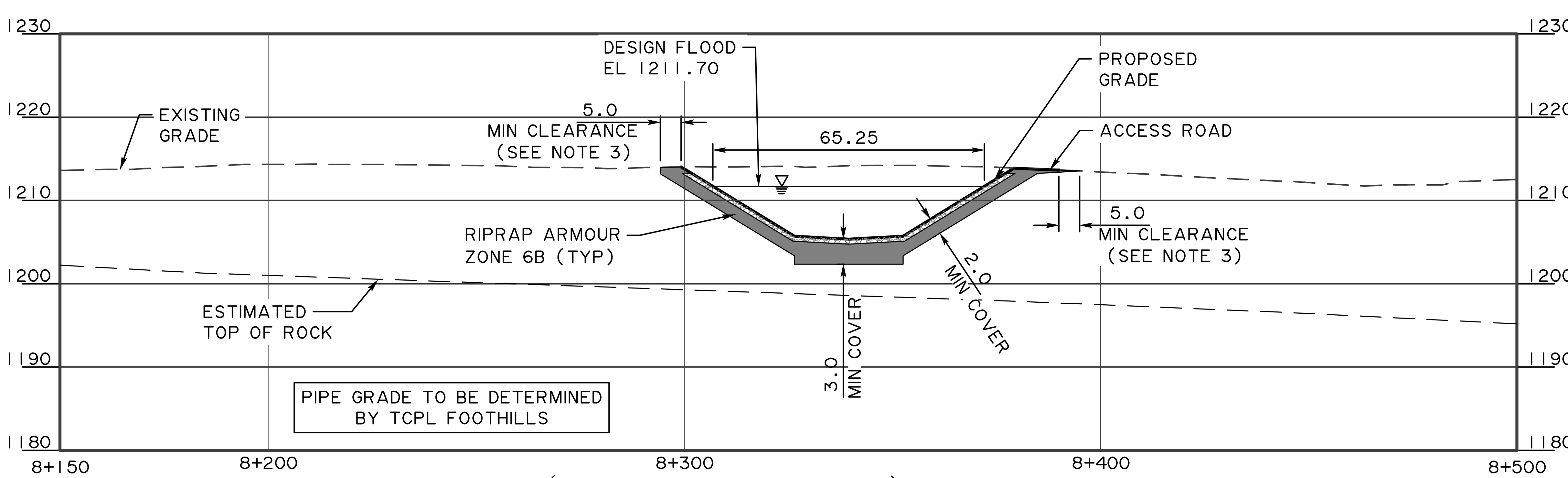


PIPELINE CROSSING PLAN 2
1:1000

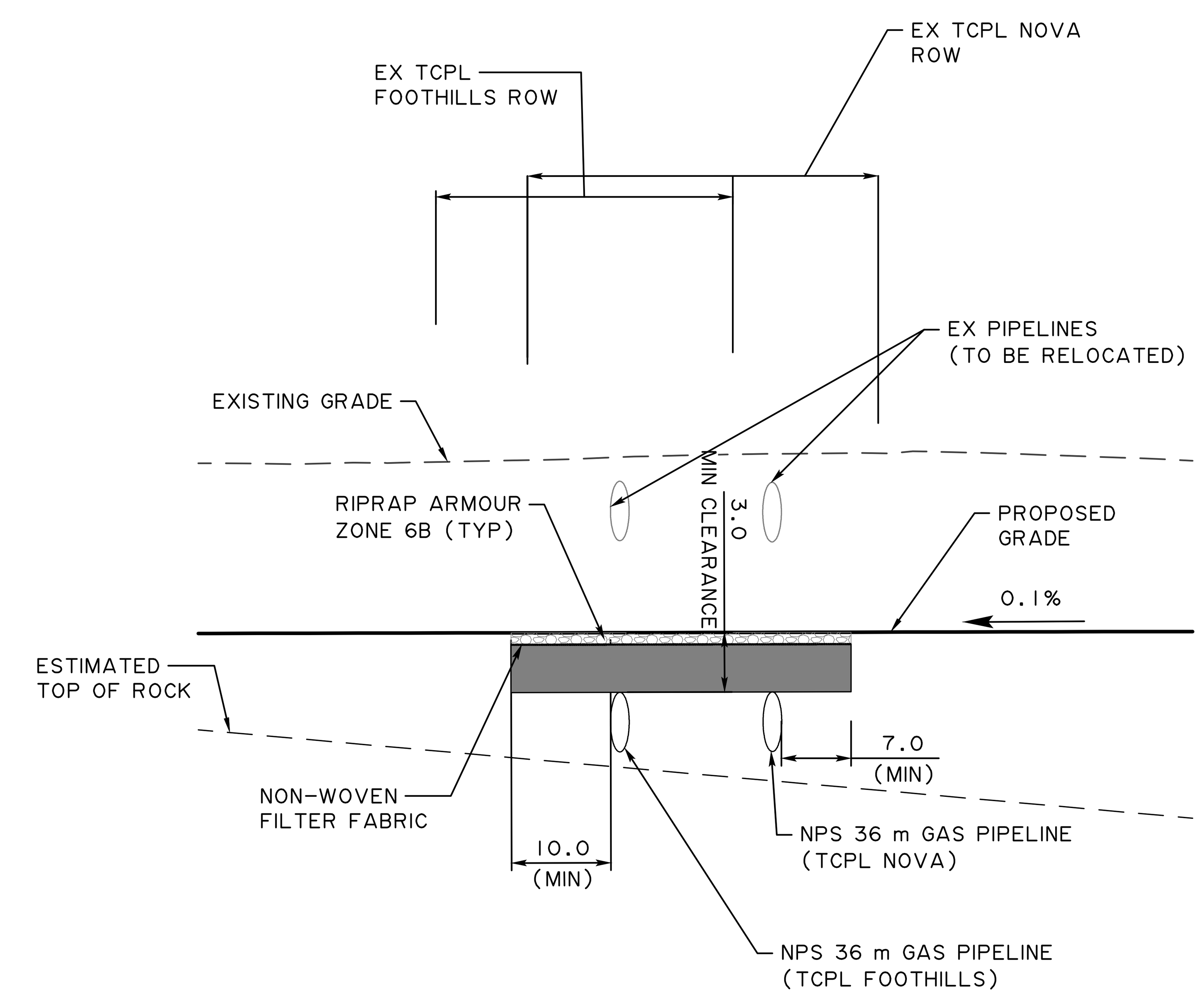
NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM DIVERSION CHANNEL STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.
3. CONSTRUCTION LOADING OVER PIPES TO BE COORDINATED WITH OWNER.



PIPELINE CROSSING (TCPL NOVA) PROFILE
H 1:1000 V 1:500



PIPELINE CROSSING (TCPL FOOTHILLS) PROFILE
H 1:1000 V 1:500



SECTION B-B
H 1:1500 V 1:250

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

GENERAL
DIVERSION CHANNEL
UTILITY CROSSING 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-144	

PlotDate: 2019.12.03 9:41 AM Login: Richardson, Jillian
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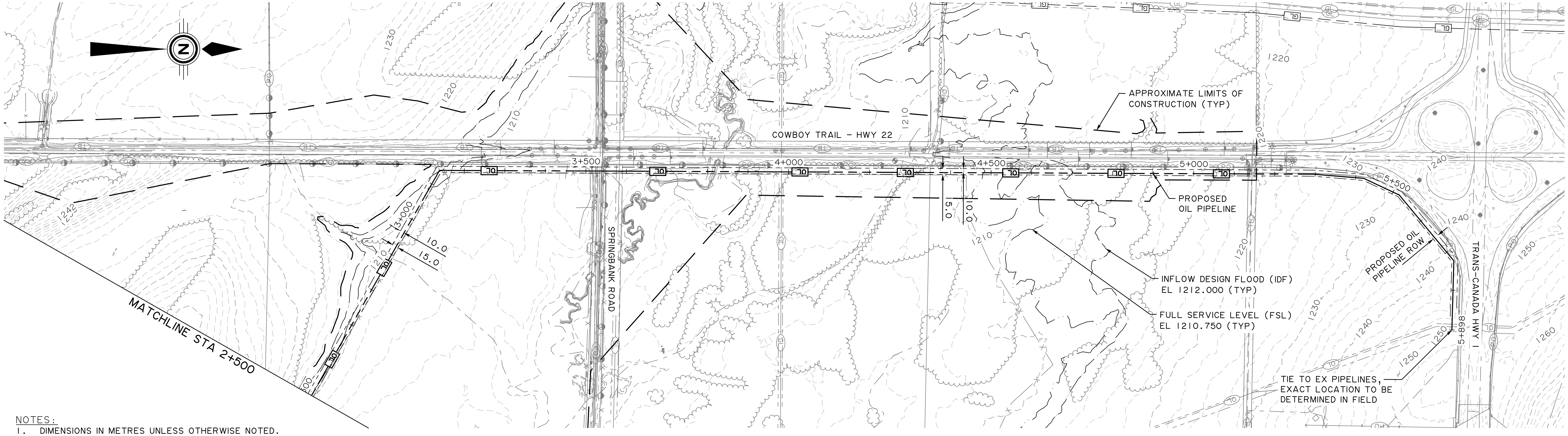
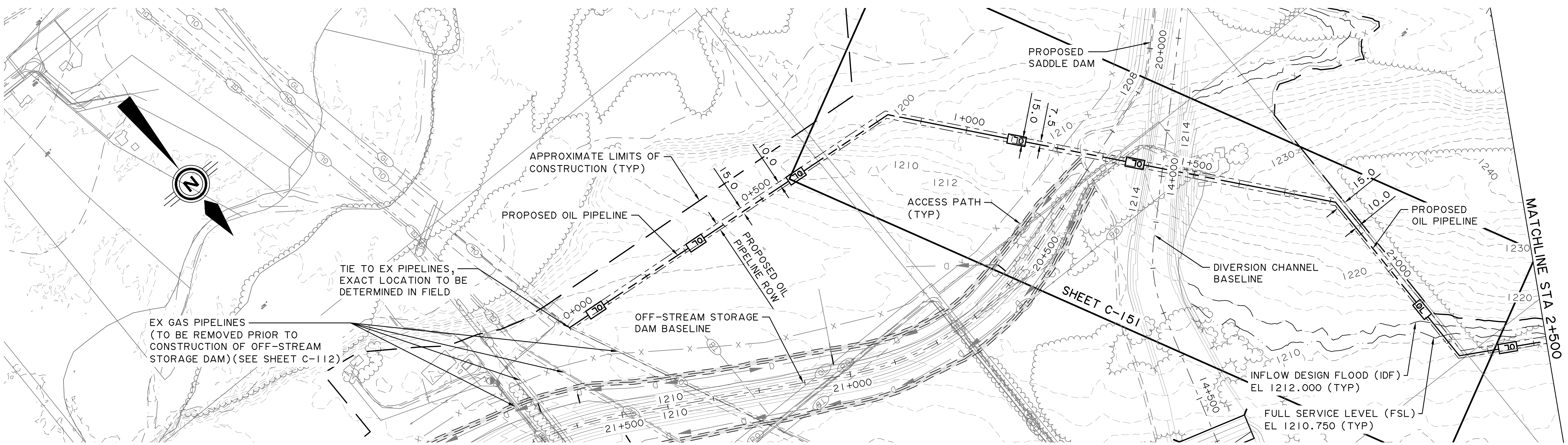


NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. SEGMENTS UNDER DAM ALIGNMENT AND DIVERSION CHANNEL SHALL BE HORIZONTAL DIRECTIONAL DRILLED (HDD).



0 200 400 Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
RESERVOIR UTILITY RELOCATION
ALIGNMENT OVERVIEW

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-150	

DEPARTMENT BAR CODE

PlotDate: 2019.12.02 3:42 PM Login: Bolubasz, Danielle
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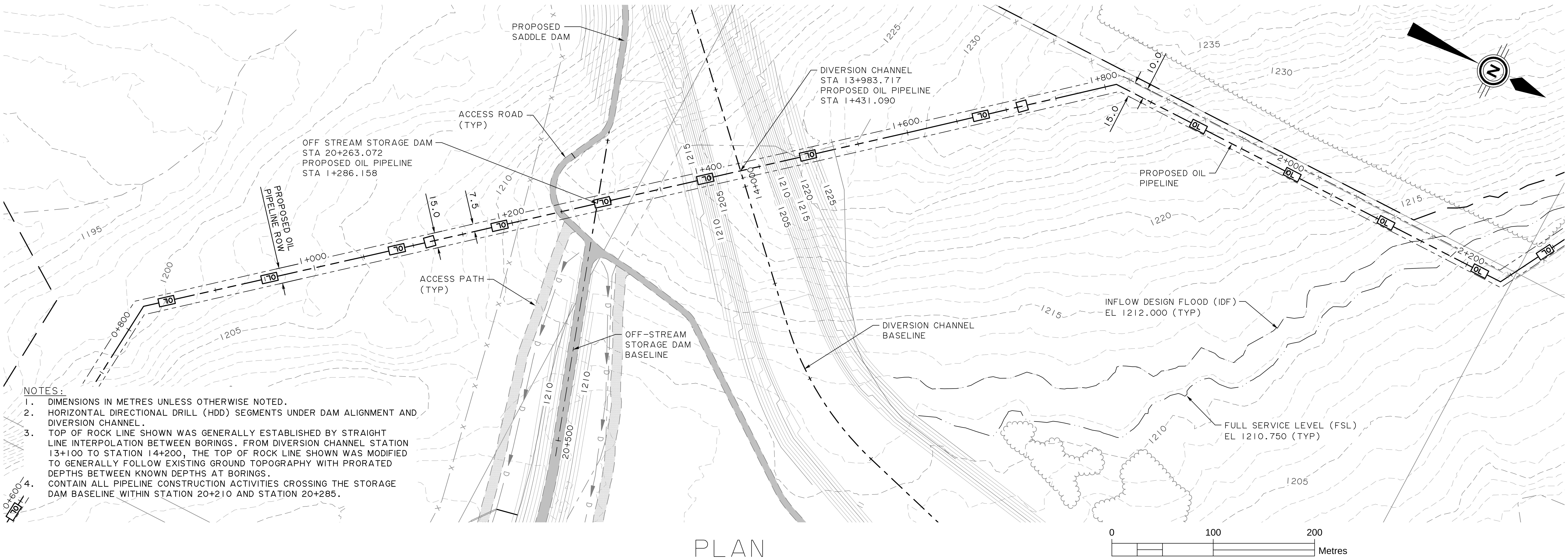
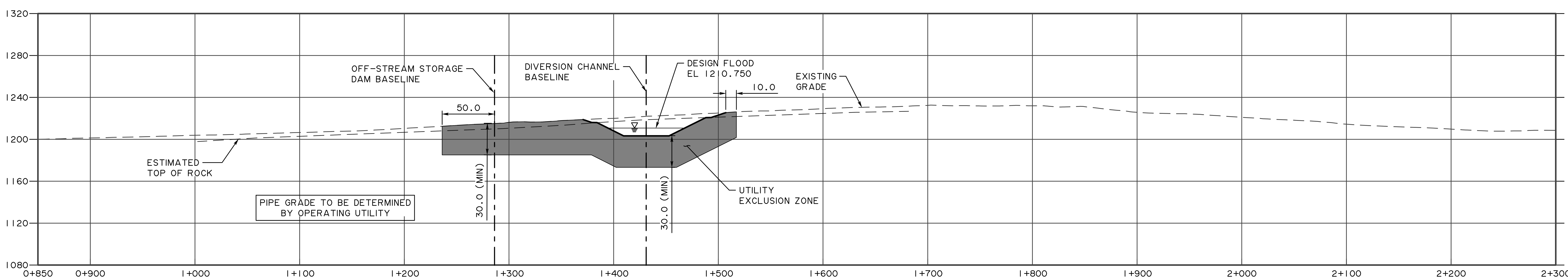


NOTES:

1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. HORIZONTAL DIRECTIONAL DRILL (HDD) SEGMENTS UNDER DAM ALIGNMENT AND DIVERSION CHANNEL.
3. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM DIVERSION CHANNEL STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.
4. CONTAIN ALL PIPELINE CONSTRUCTION ACTIVITIES CROSSING THE STORAGE DAM BASELINE WITHIN STATION 20+210 AND STATION 20+285.

PLAN
1:2000



PROFILE
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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

GENERAL
RESERVOIR UTILITY RELOCATION
PLAN AND PROFILE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-151	

PlotDate: 2019.12.03 9:44 AM Login: Richardson, Jillian
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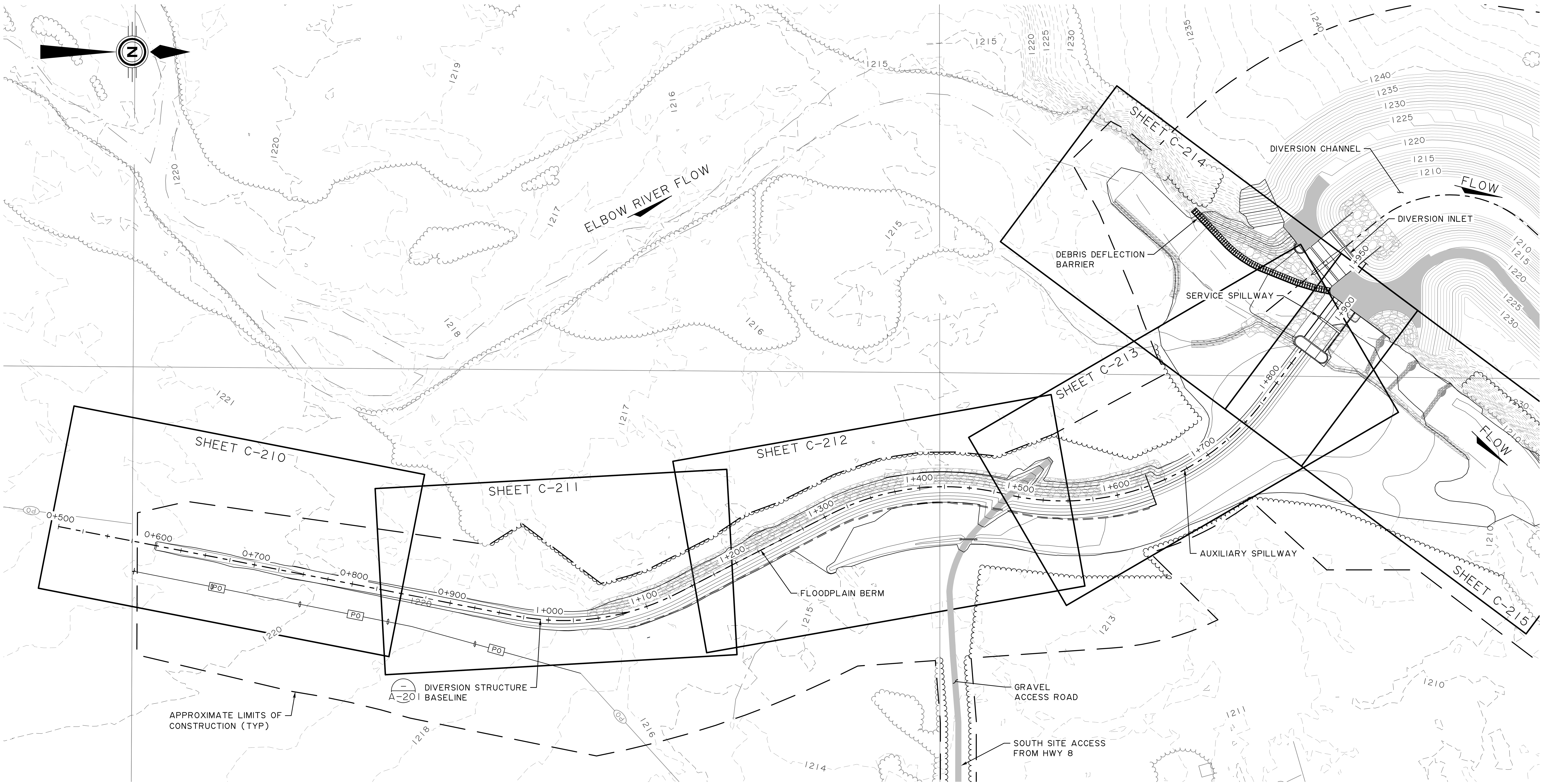

NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:2000

0 100 200 Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
OVERVIEW

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-201	

PlotDate: 2019.12.03 9:49 AM Login: Richardson, Jillian TENDER No. ?
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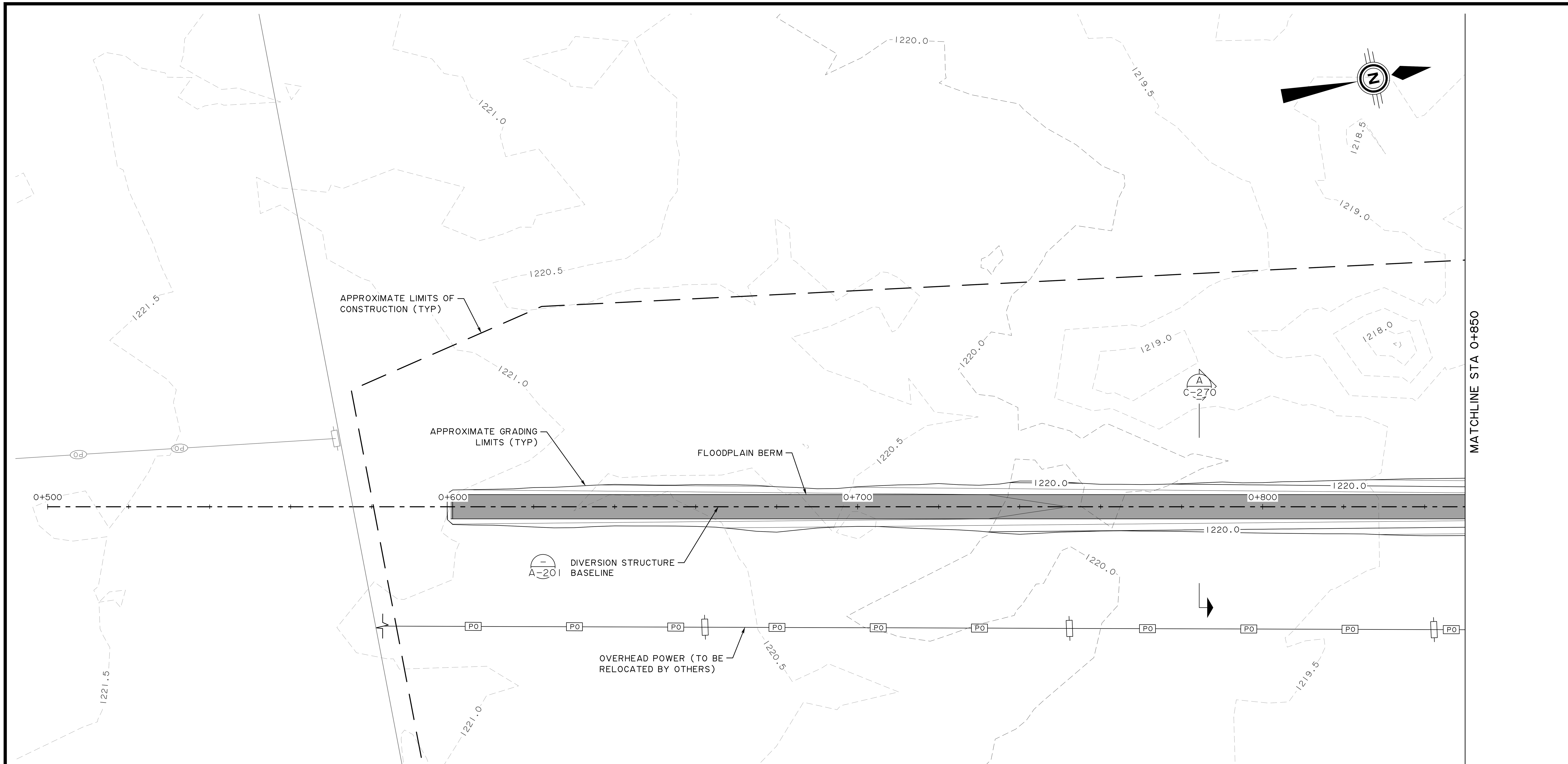

PLAN
1:500

0 20 40 Metres

NOTES:
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.

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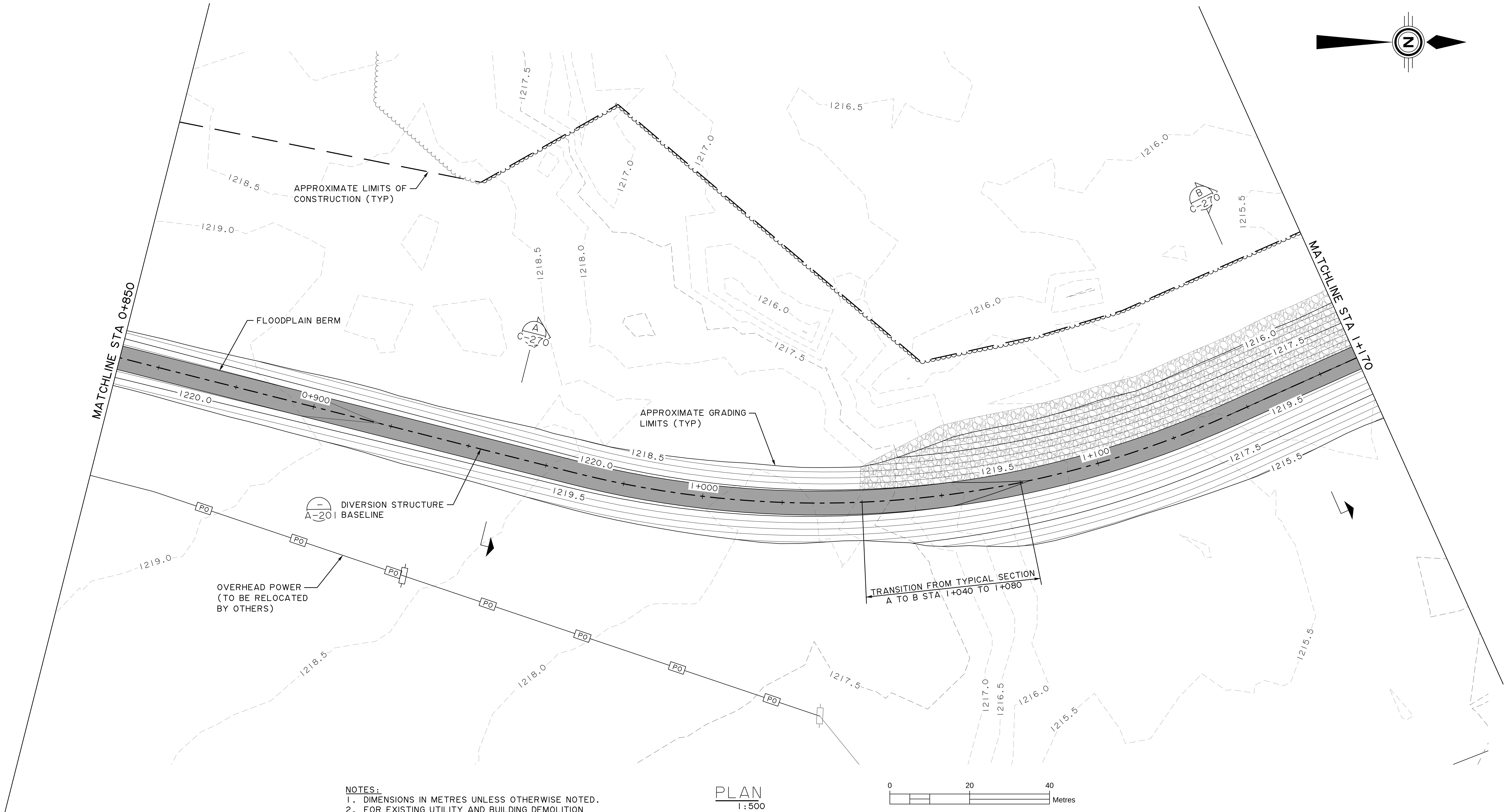
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STUCTURE
FLOODPLAIN BERM AND AUXILIARY SPILLWAY
PLAN 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-210	

PlotDate: 2019.12.02 4:05 PM Login: Richardson, Jillian
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\200_ds\73396C-210.dwg TENDER No. ?



NOTES:
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.

PlotDate: 2019.12.02 4:06 PM Login: Richardson, Jillian TENDER No. ?
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\200_ds\73396C-210.dwg

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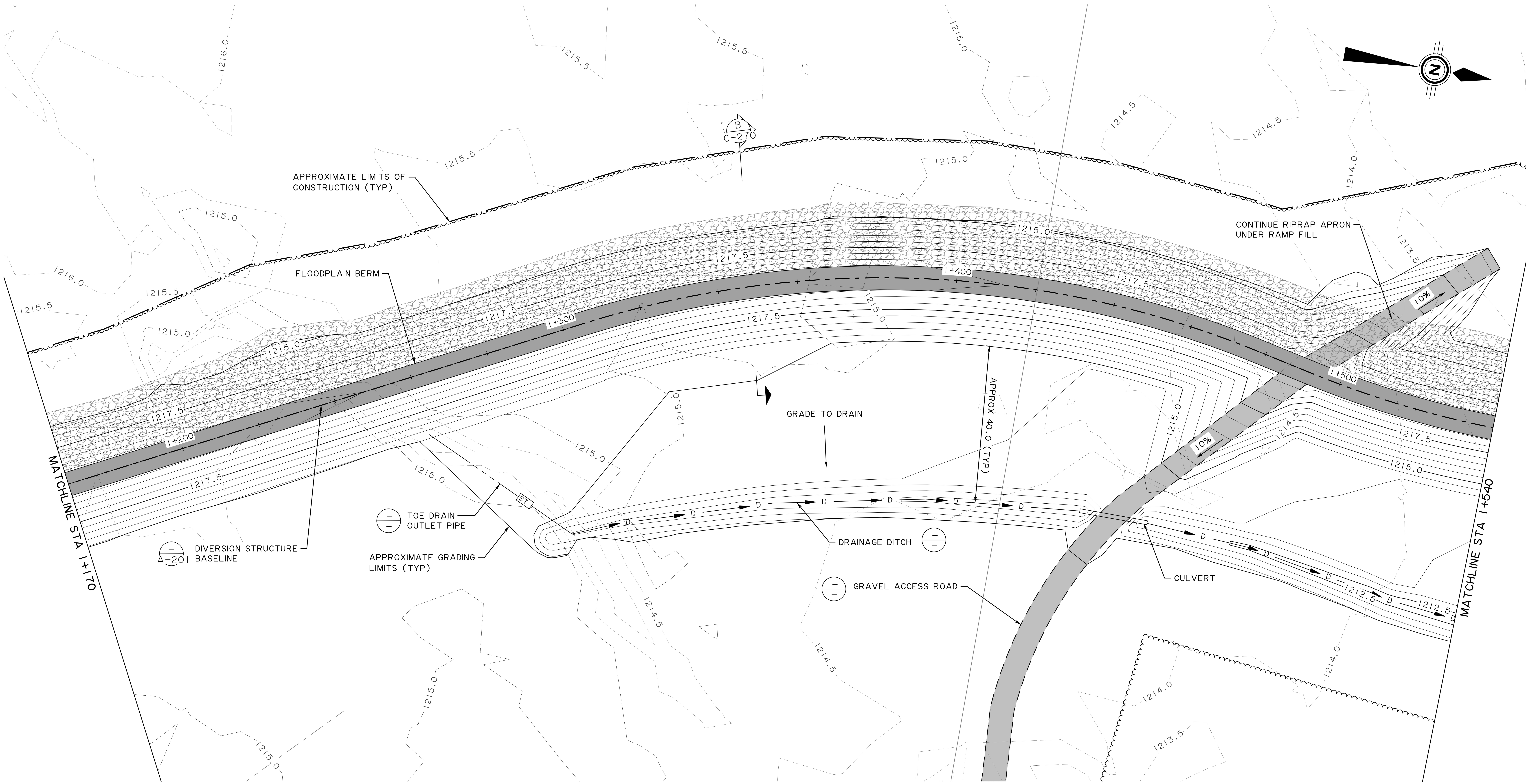
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STUCTURE
FLOODPLAIN BERM AND AUXILIARY SPILLWAY
PLAN 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-211	



NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:500

0 20 40 Metres

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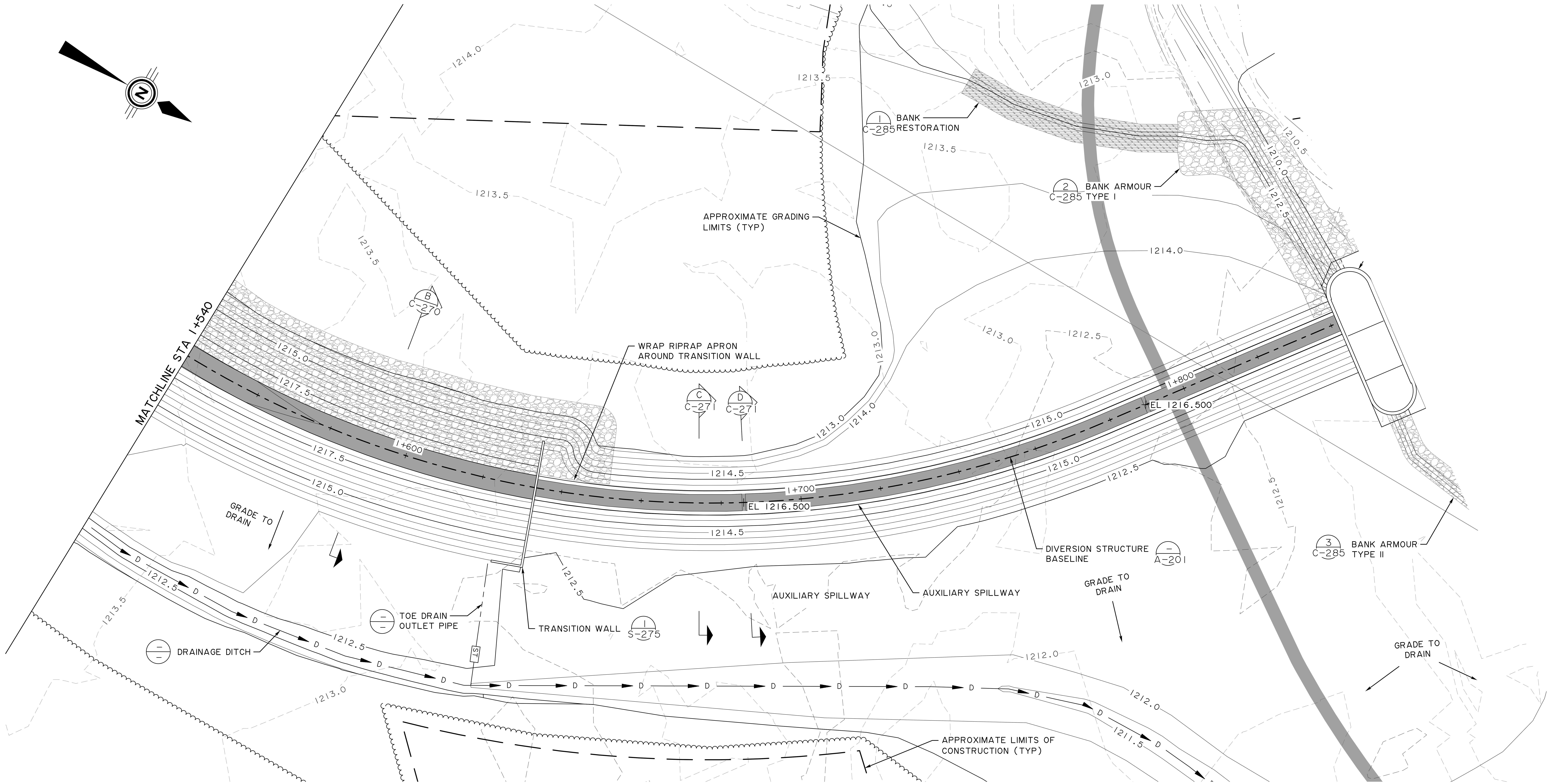
SR1

DIVERSION STUCTURE
FLOODPLAIN BERM AND AUXILIARY SPILLWAY
PLAN 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-212	

PlotDate: 2019.12.02 4:07 PM Login: Richardson, Jillian
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\200_ds\73396C-210.dwg TENDER No. ?



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:500

0 20 40 Metres

PlotDate: 2019.12.02 4:15 PM Login: Richardson, Jillian
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\200_ds\73396C-210.dwg TENDER No. ?

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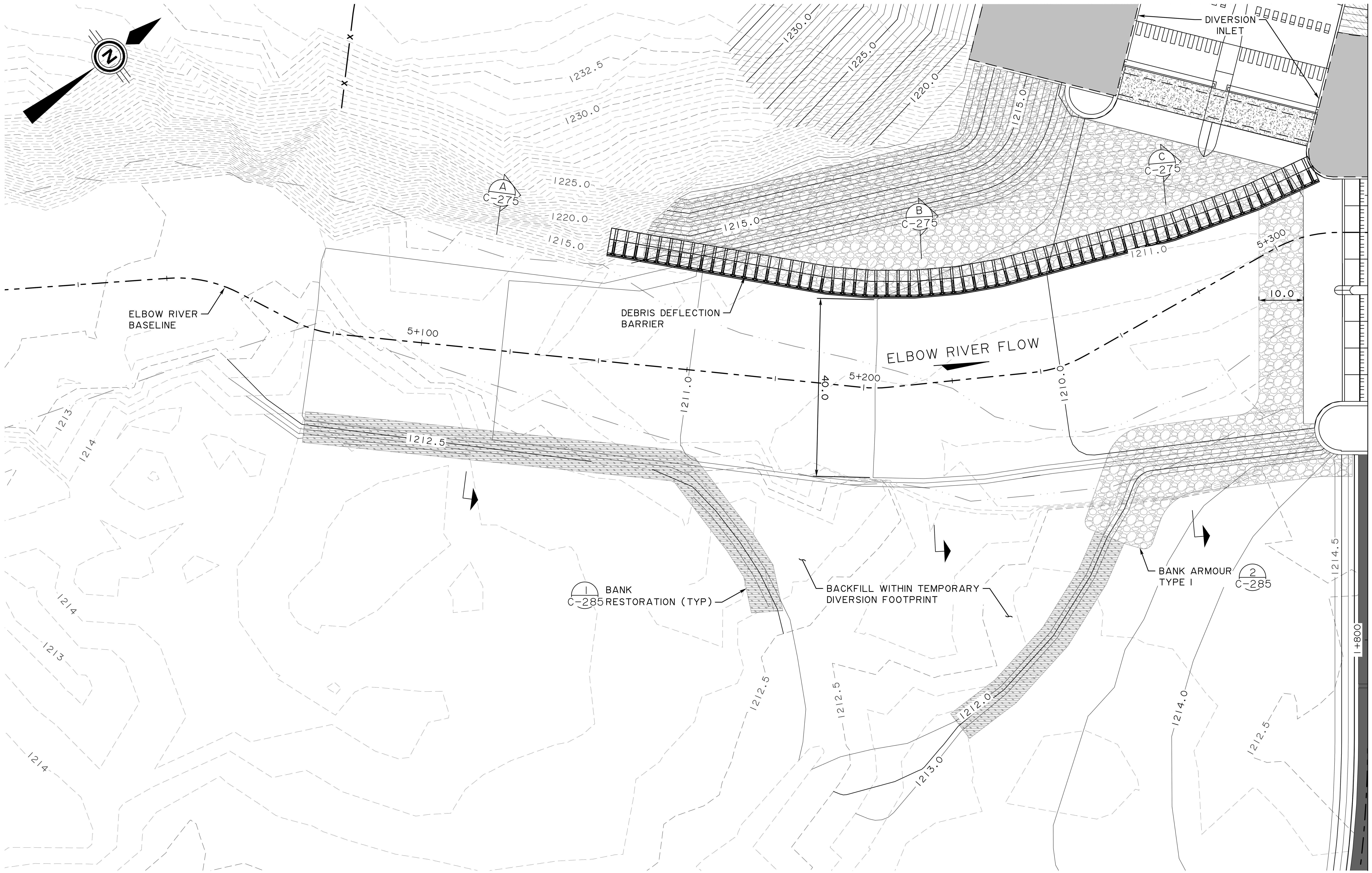
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STUCTURE
FLOODPLAIN BERM AND AUXILIARY SPILLWAY
PLAN 4

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-213	



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:500

0 20 40 Metres

PlotDate: 2019.12.02 4:34 PM Login: Richardson, Jillian TENDER No. ?
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\200_ds\73396C-214.dwg

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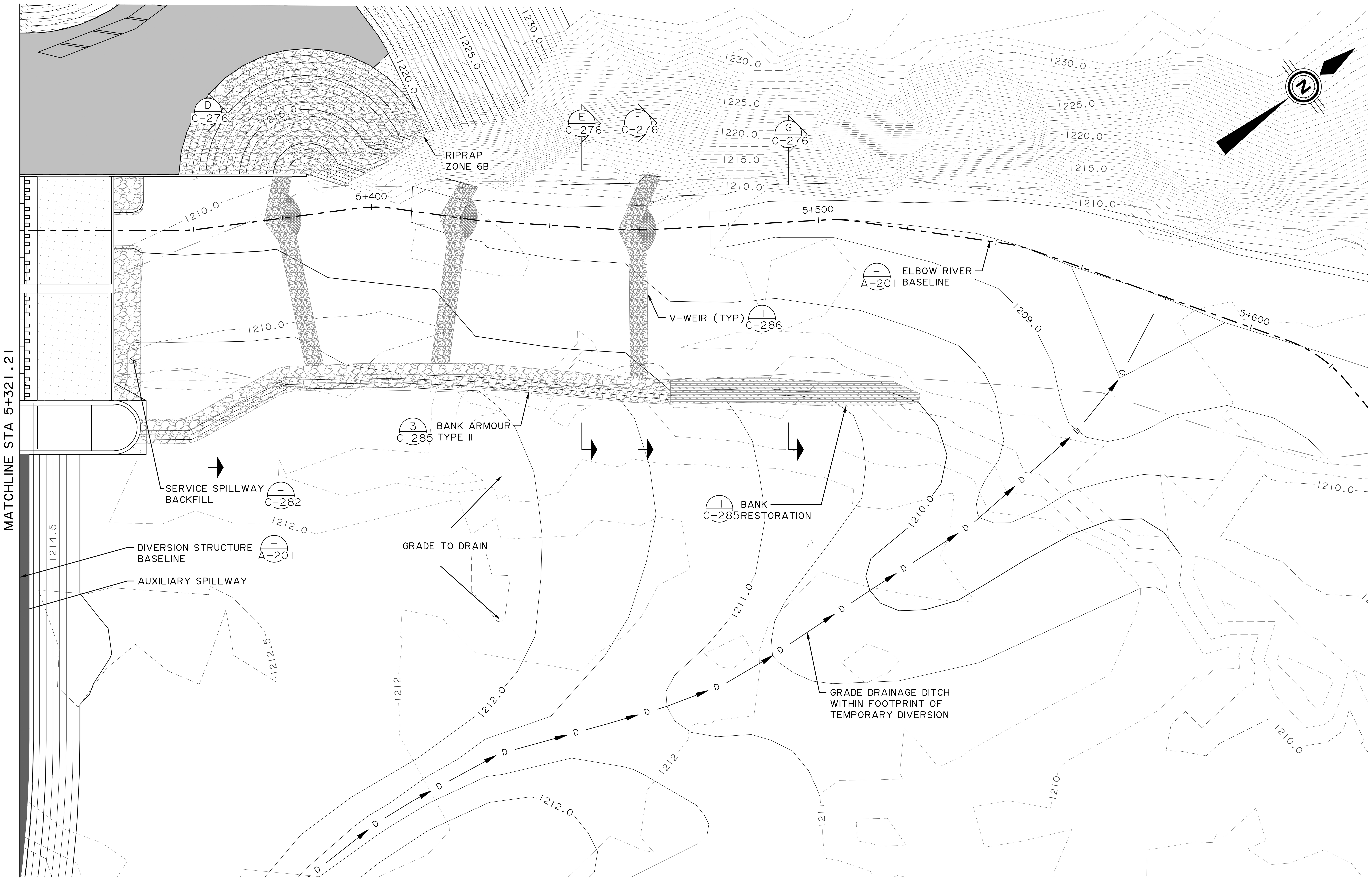
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
ELBOW RIVER
PLAN 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-214	



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:500

0 20 40 Metres

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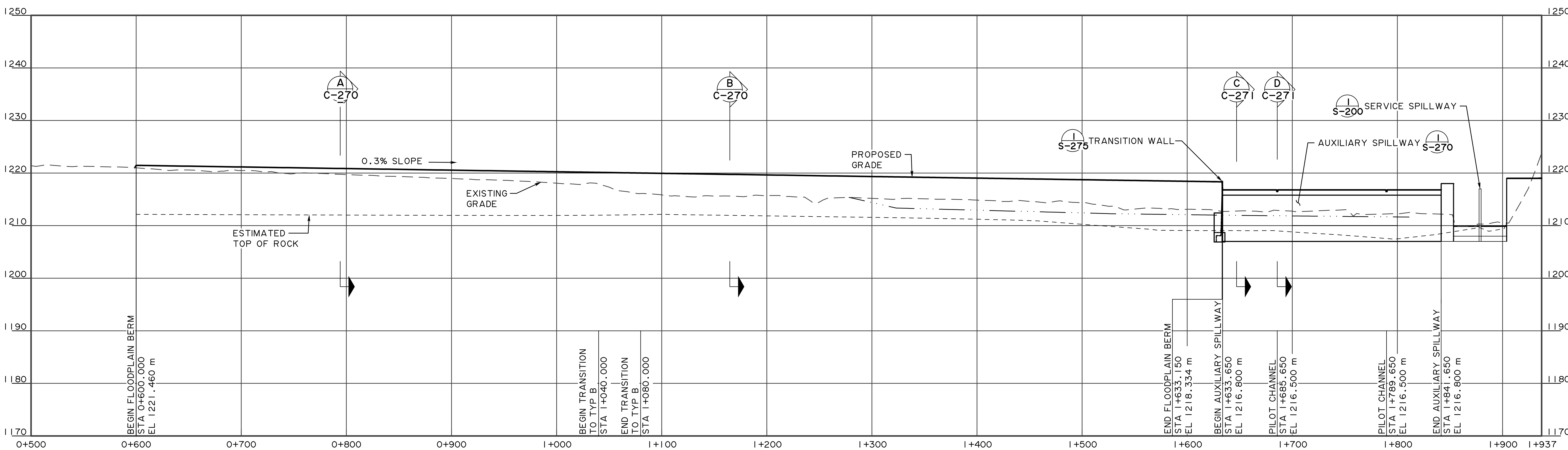
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
ELBOW RIVER
PLAN 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-215	

PlotDate: 2020.06.26 3:44 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civilsheet_files\200_ds\73396C-214.dwg TENDER No. ?



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PROFILE
H 1:2000 V 1:400

TENDER No. ?

PlotDate: 2019.11.26 4:47 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\200_ds\73396C-230.dwg

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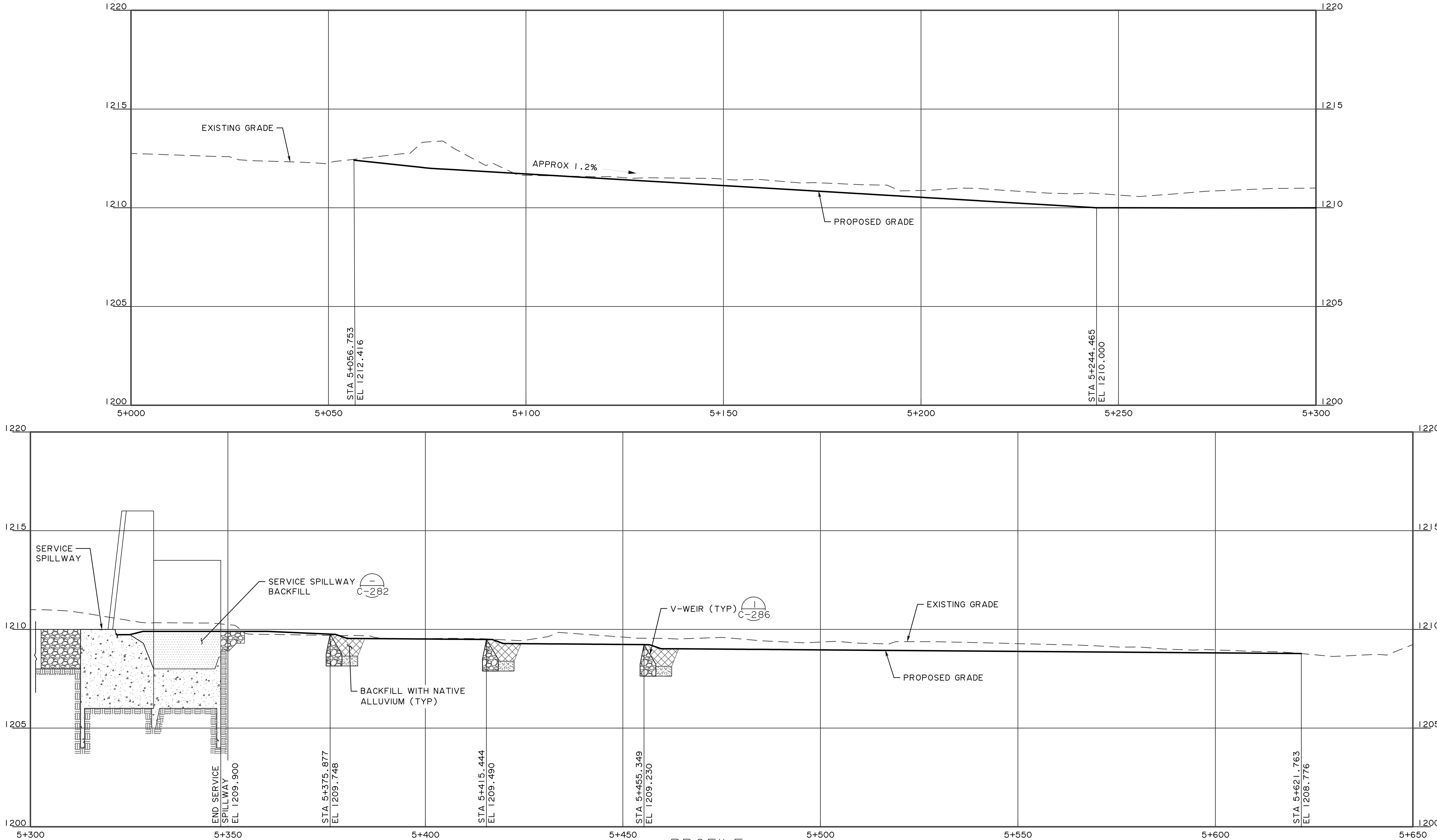
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
BASELINE
PROFILE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-230	



PROFILE
H 1:500, V 1:100

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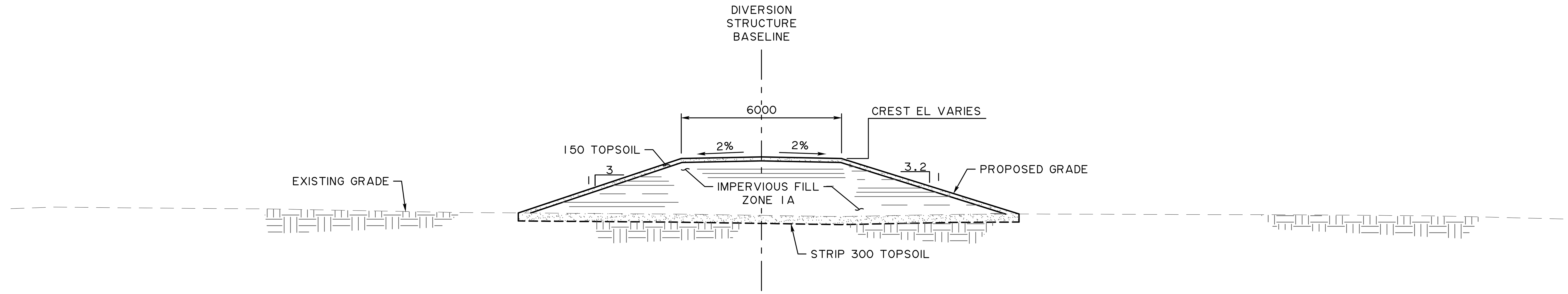
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

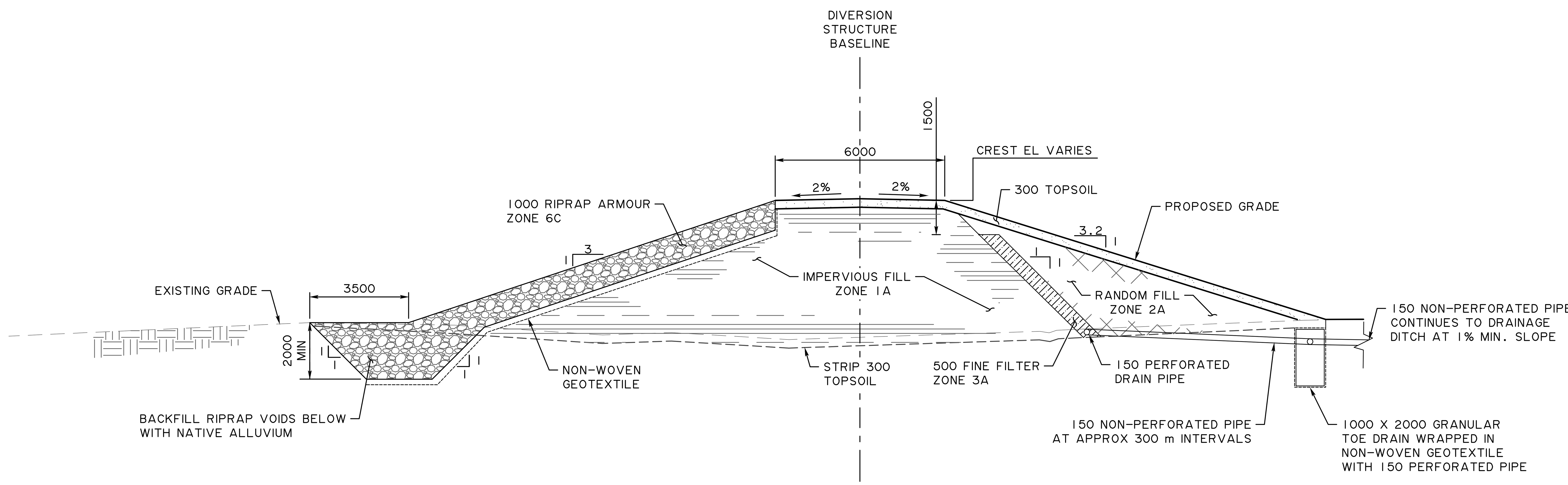
DIVERSION STRUCTURE
ELBOW RIVER
PROFILE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-	1	73396C-235	

PlotDate: 2020.06.26 5:15 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\sheet_files\200_ds\73396C-235.dwg TENDER No. ?



A TYPICAL SECTION – FLOODPLAIN BERM
C-210
NTS



B TYPICAL SECTION – FLOODPLAIN BERM
C-211
NTS

NOTES:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. SEE PLAN VIEWS C-212 AND C-213 FOR LOCATION OF DRAIN PIPE OUTLETS.

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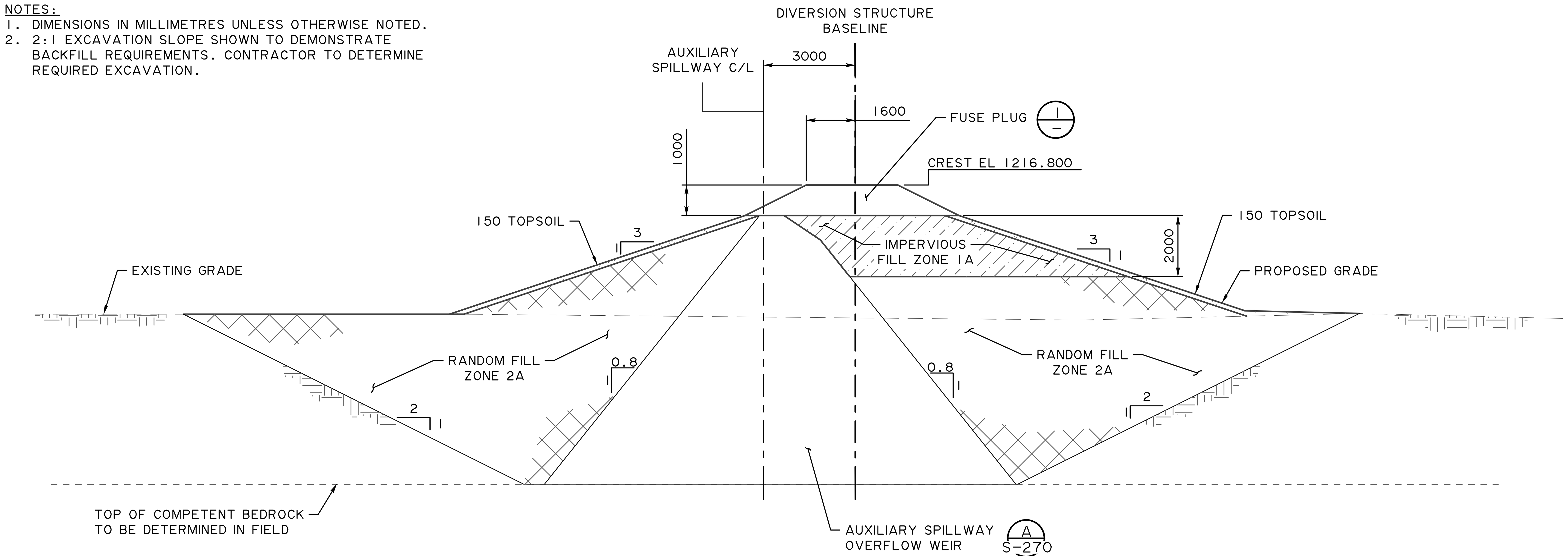
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
FLOODPLAIN BERM AND AUXILIARY SPILLWAY
TYPICAL SECTIONS 1

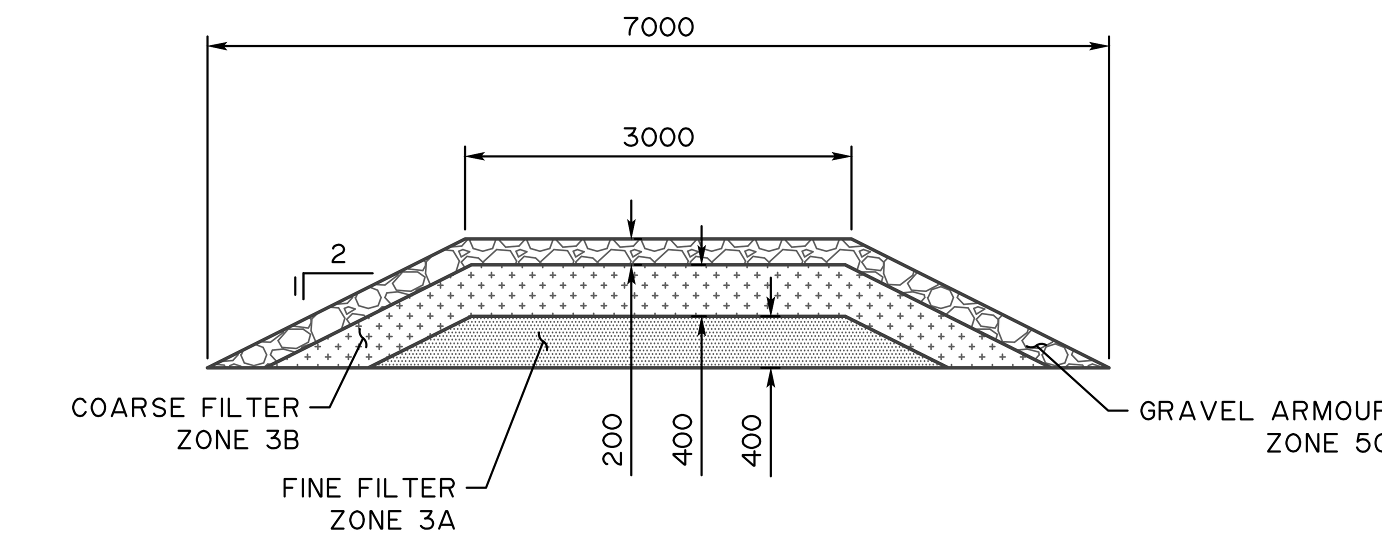
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-270	

PlotDate: 2019.12.02 4:49 PM Login: Richardson, Jillian TENDER No. ?
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\200_ds\73396C-270.dwg

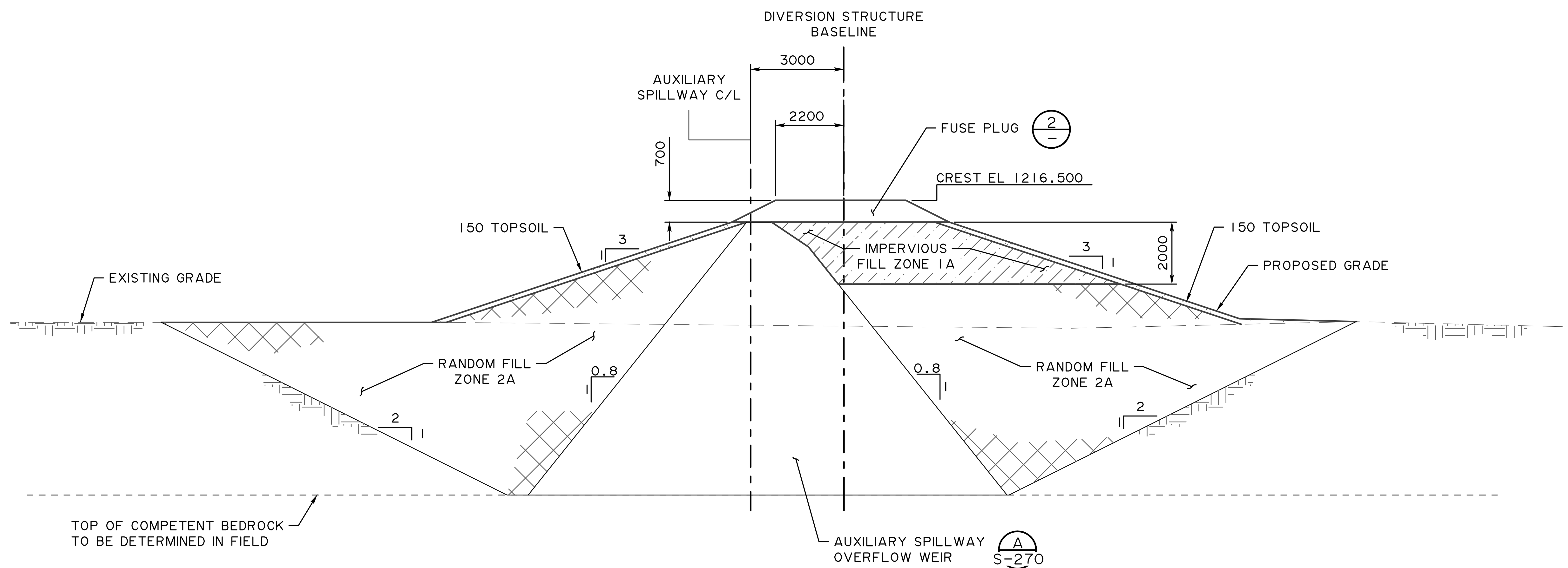
NOTES:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. 2:1 EXCAVATION SLOPE SHOWN TO DEMONSTRATE BACKFILL REQUIREMENTS. CONTRACTOR TO DETERMINE REQUIRED EXCAVATION.



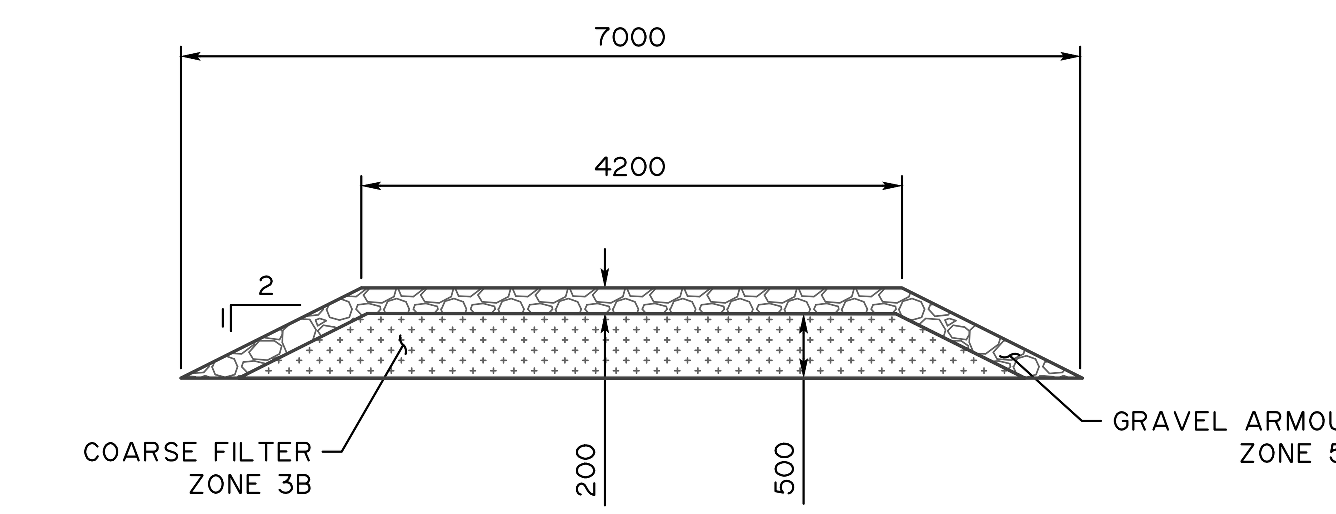
C C-213 TYPICAL SECTION – AUXILIARY SPILLWAY – FUSE PLUG
NTS



1 – DETAIL – FUSE PLUG FULL SECTION
1:50



D C-213 TYPICAL SECTION – AUXILIARY SPILLWAY – PILOT CHANNEL
NTS



2 – DETAIL – FUSE PLUG PILOT CHANNEL
1:50

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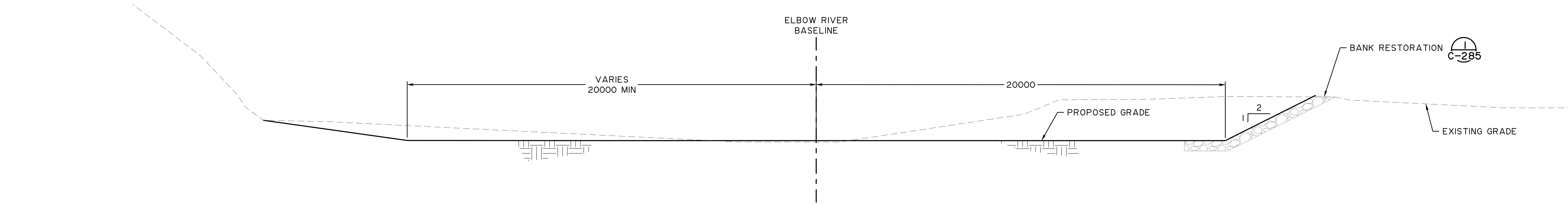
SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

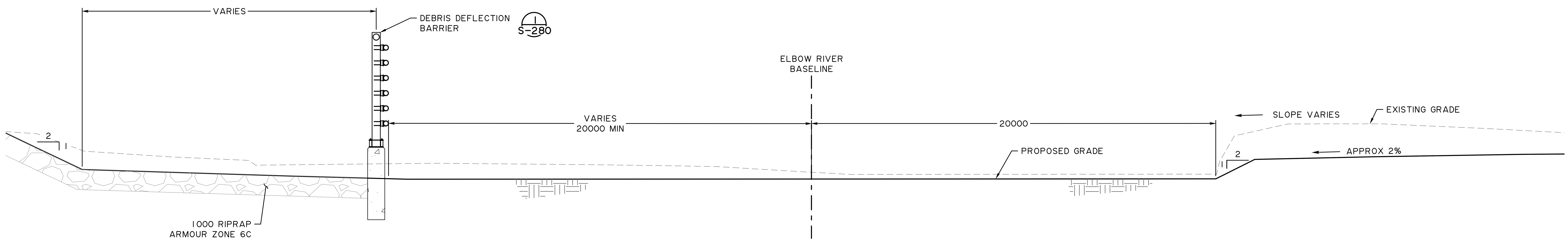
DIVERSION STRUCTURE
FLOODPLAIN BERM AND AUXILIARY SPILLWAY
TYPICAL SECTIONS 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-271	

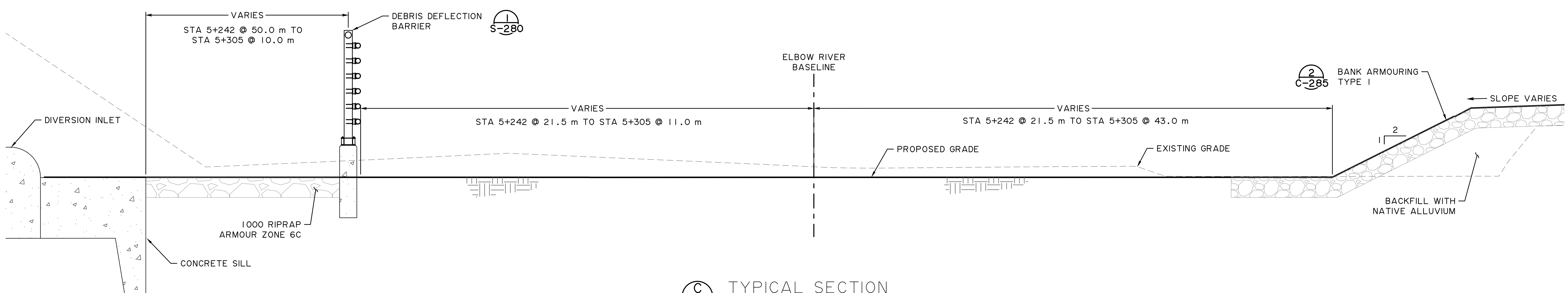
PlotDate: 2019.12.03 9:56 AM Login: Richardson, Jillian TENDER No. ?
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\200_ds\73396C-271.dwg



A TYPICAL SECTION
C-214 NTS



B TYPICAL SECTION
C-214 NTS



C TYPICAL SECTION
C-214 NTS

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PlotDate: 2019.11.26 4:58 PM Login: Bolubasz, Danielle TENDER No. ?
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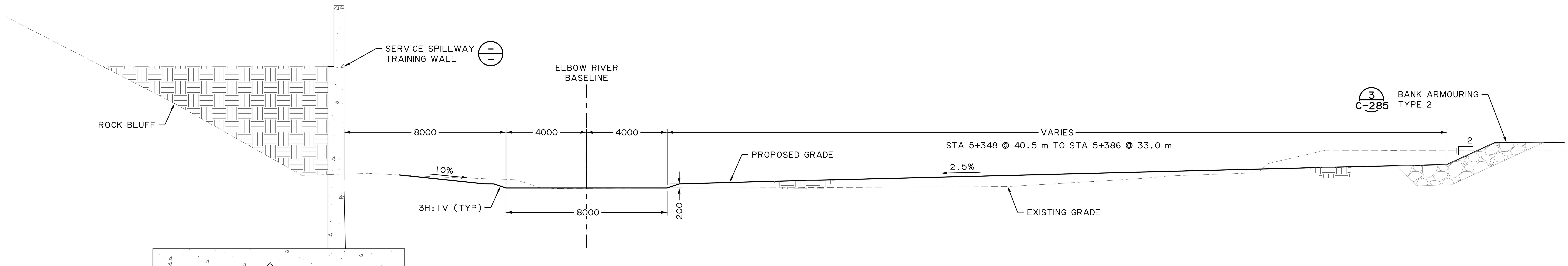
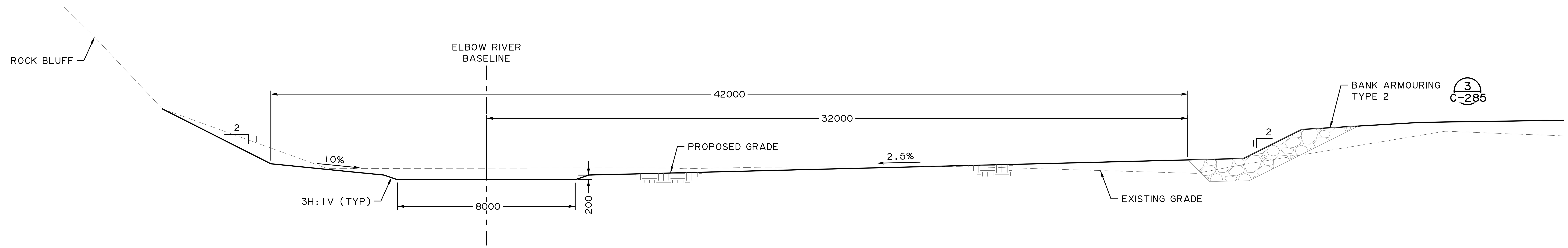
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
ELBOW RIVER
TYPICAL SECTIONS 1

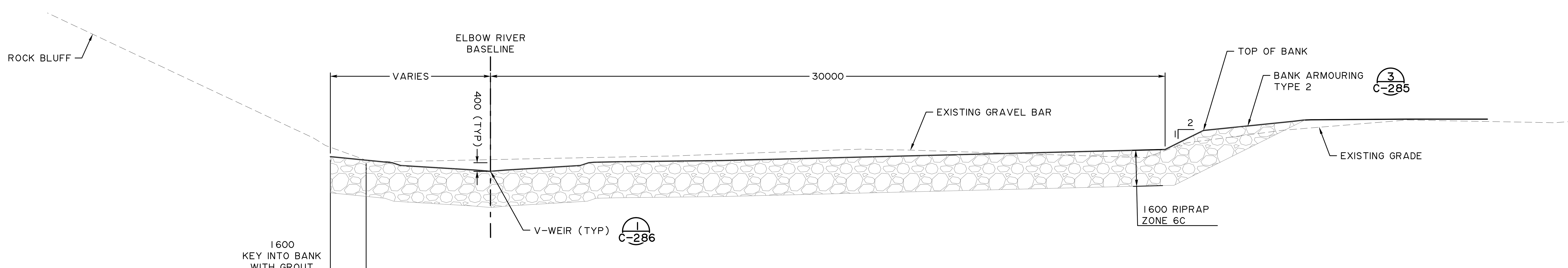
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-275	



D C-215 TYPICAL SECTION NTS



E C-215 TYPICAL SECTION NTS



F C-215 TYPICAL SECTION NTS

NOTE:
1. TOTAL 42 m CHANNEL WIDTH CONSTANT. LOW FLOW CHANNEL VARIES PER BASELINE ARRANGEMENT.

PlotDate: 2019.11.26 4:58 PM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\200_ds\73396C-275.dwg TENDER No. ?

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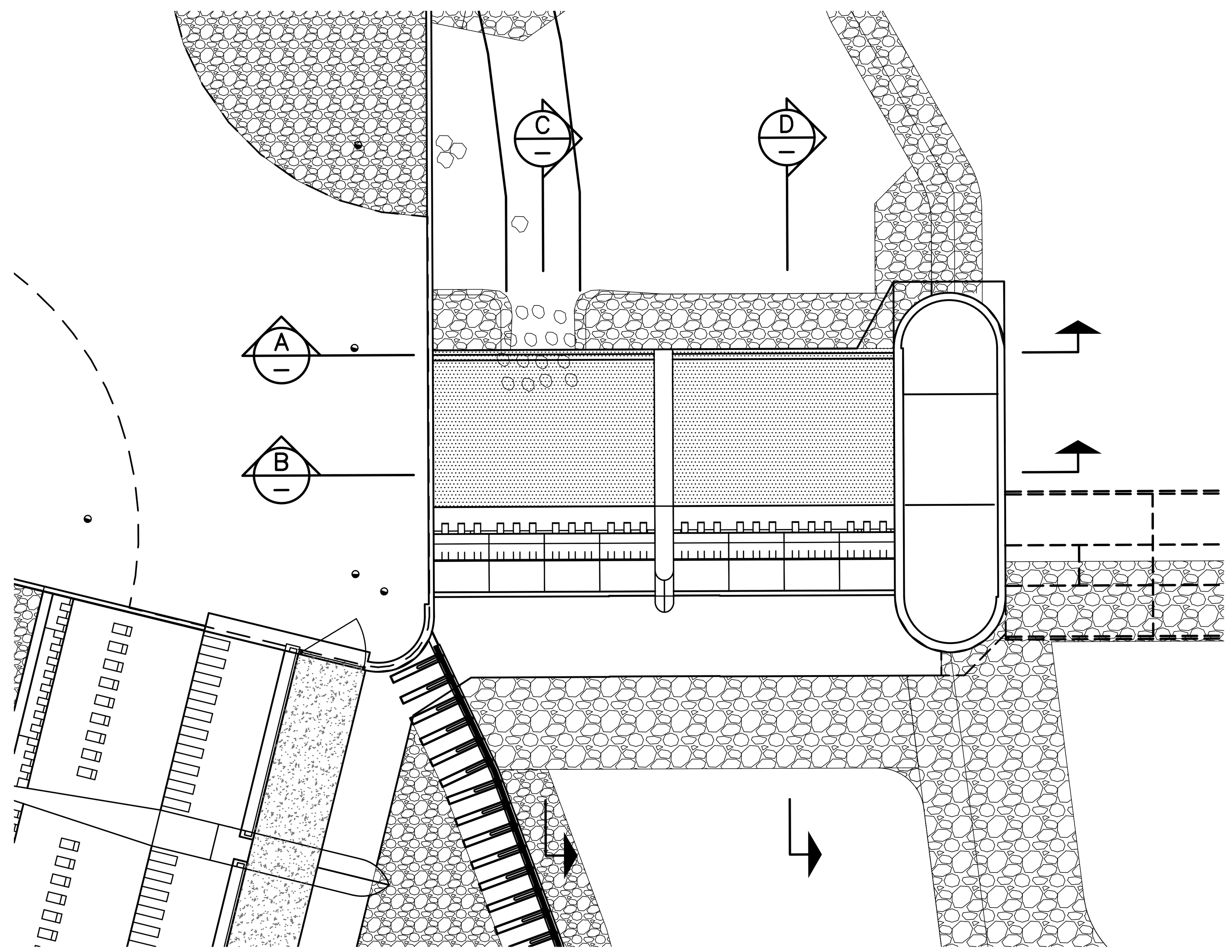
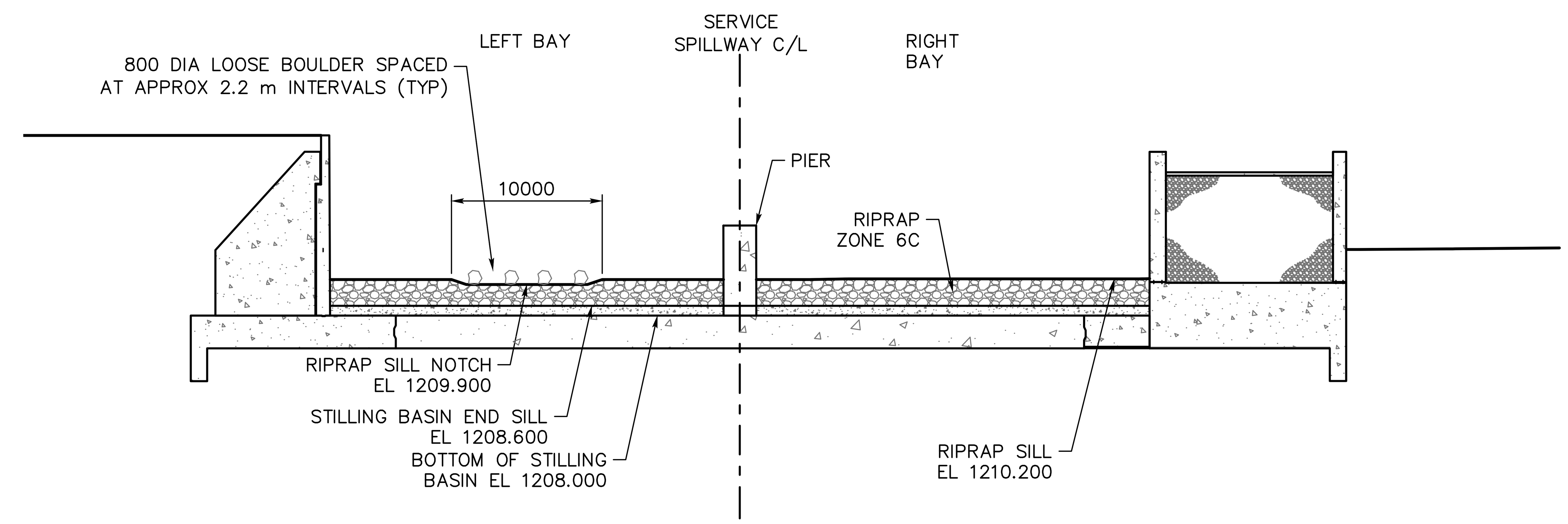
DIVERSION STRUCTURE
ELBOW RIVER
TYPICAL SECTIONS 2

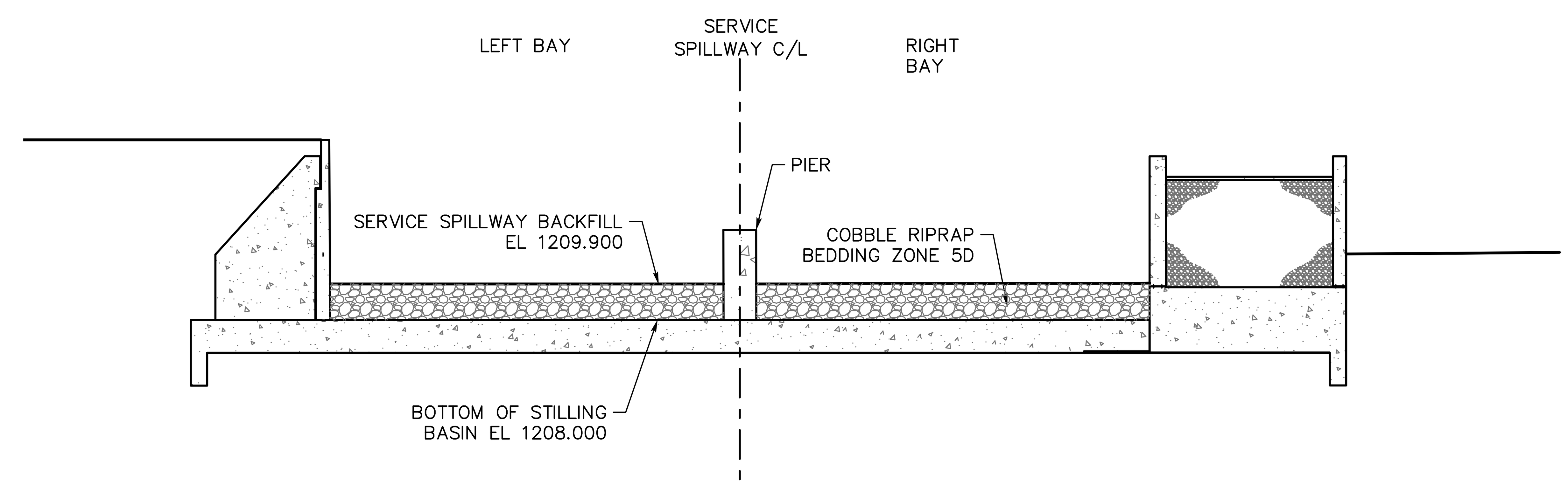
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-276	



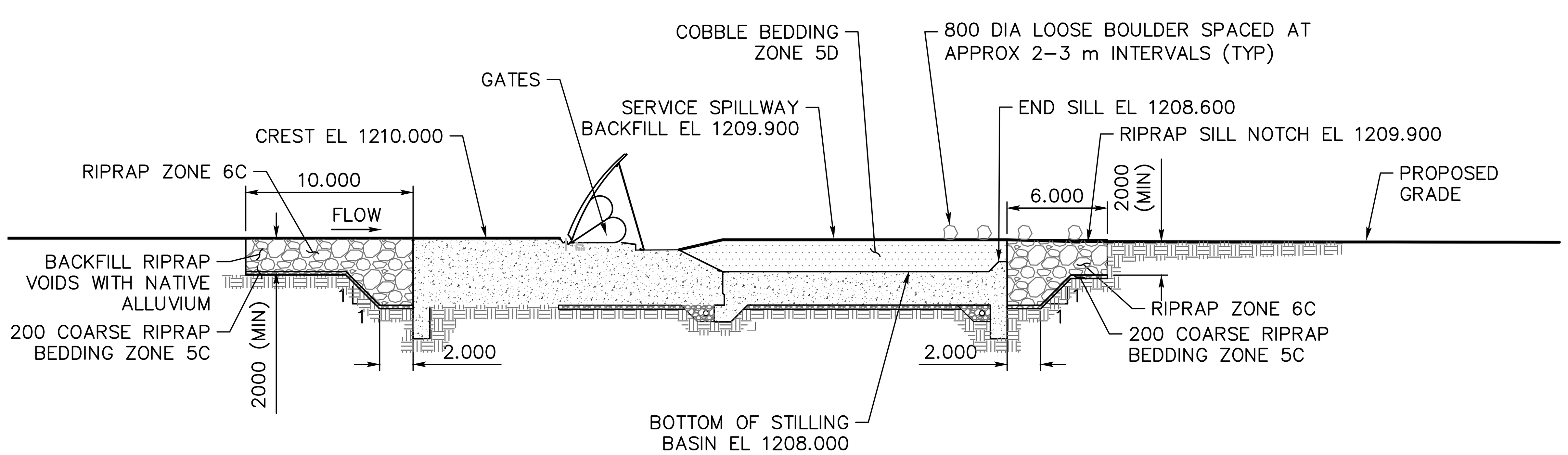
1 DETAIL
C-215
1:500



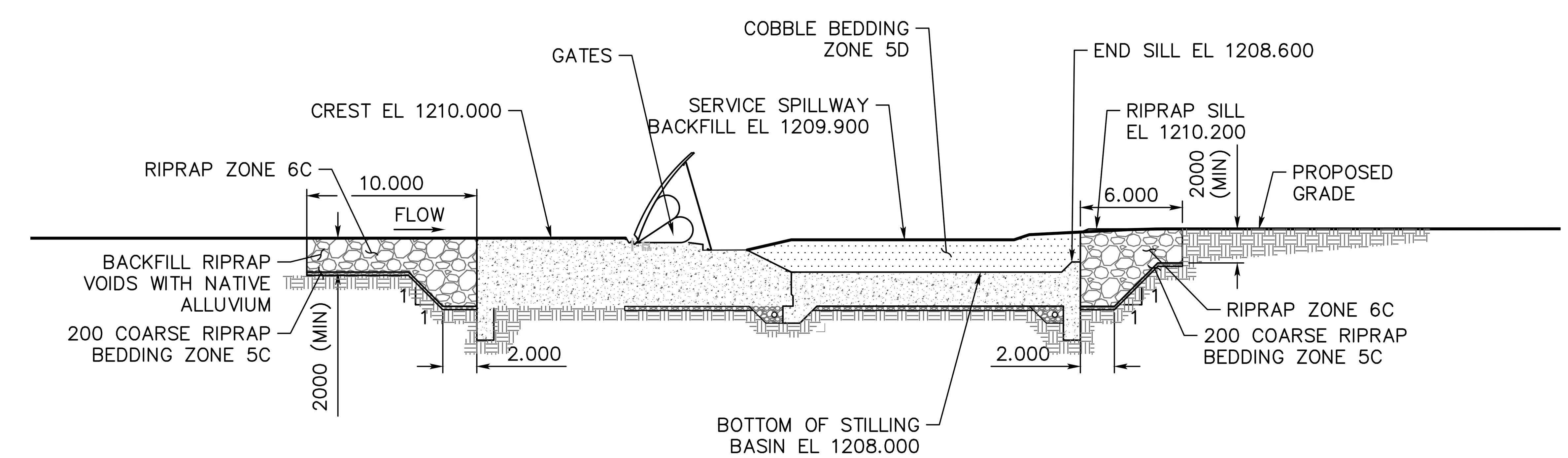
A SERVICE SPILLWAY – SILL CREST
1:250



B SERVICE SPILLWAY – STILLING BASIN
1:250



C SERVICE SPILLWAY NORTH BAY
RIGHT BAY
1:250



D SERVICE SPILLWAY SOUTH BAY
LEFT BAY
1:250

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION STRUCTURE
SERVICE SPILLWAY INFILL
DETAILS 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-282	

PlotDate: 2020.06.23 3:51 PM Login: McIlwain, Jordan
AUTOCAD FILE No. \\Ca0002-ppfss02\shared_projects\110773396\CAD\water\c_civil\sheet_files\200_ds\73396C-282_temp.dwg
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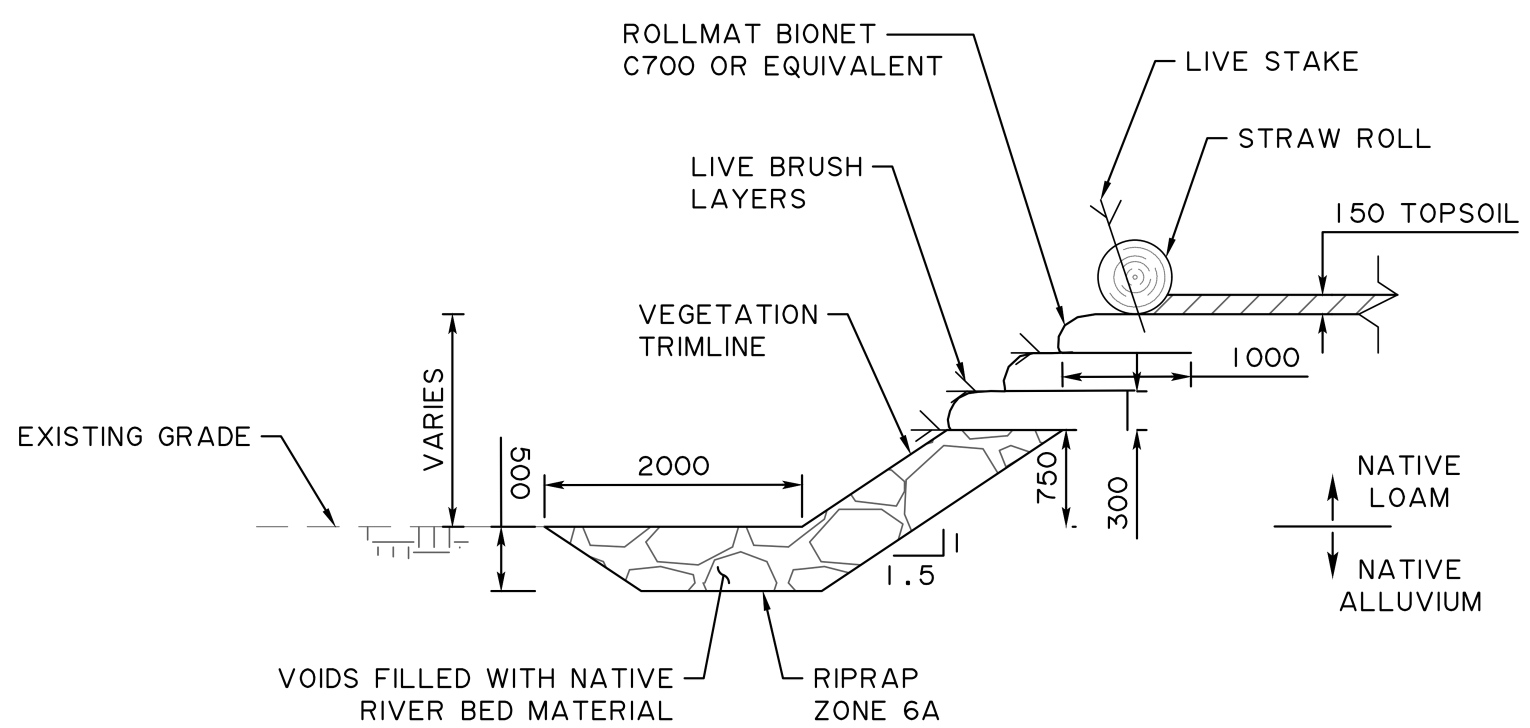
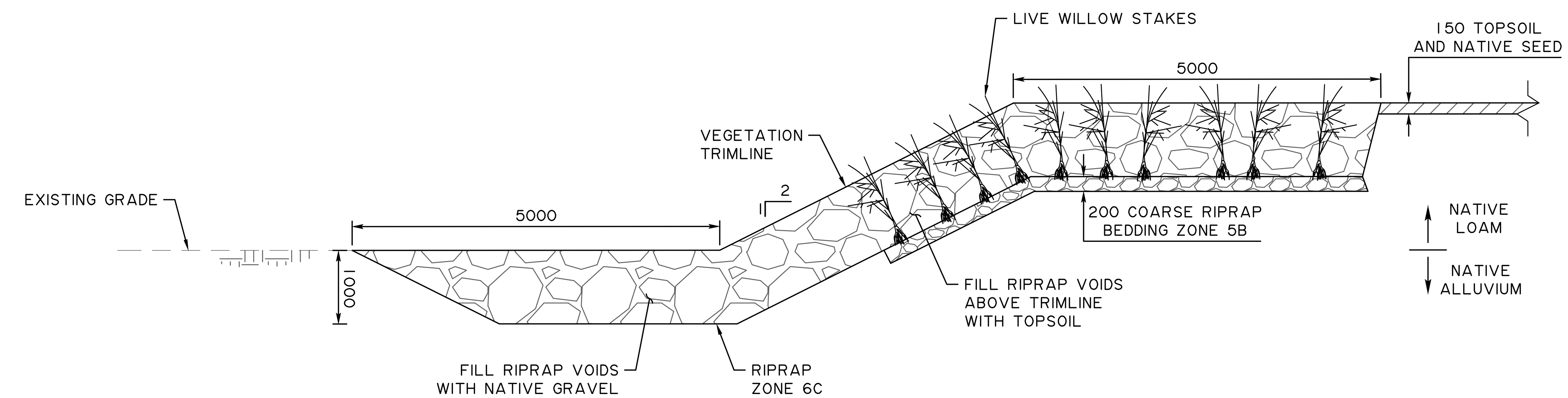
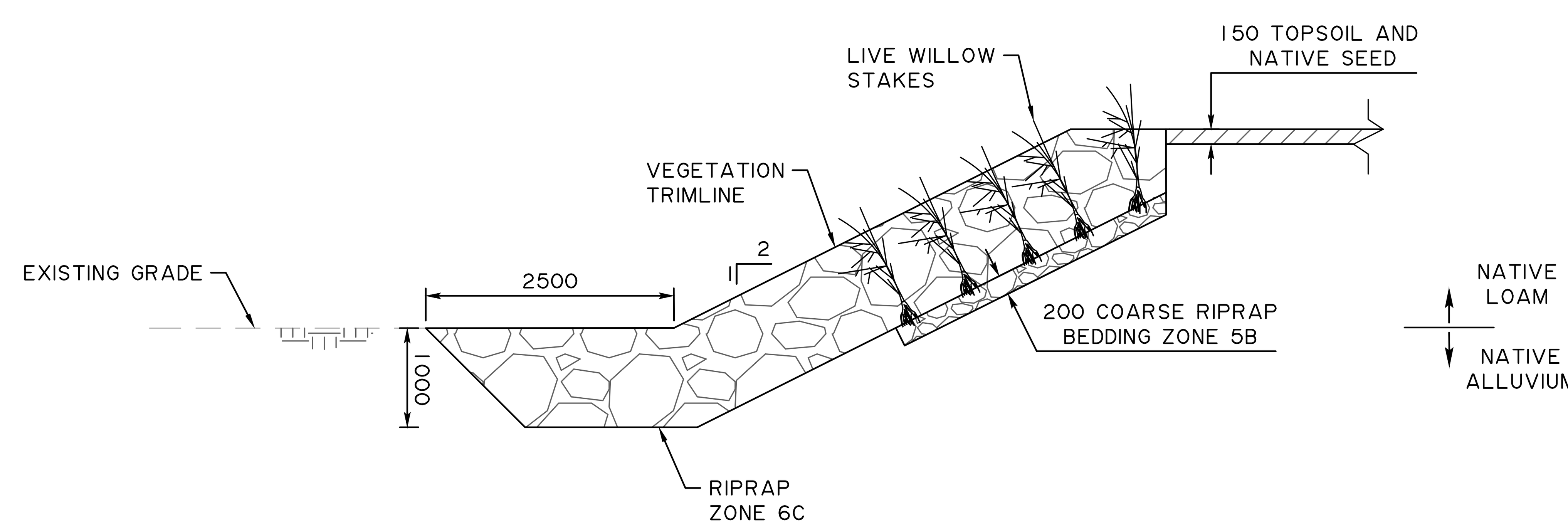


1 BANK RESTORATION
C-214 1:50



2 BANK ARMOURING TYPE I
C-214 1:50



3 BANK ARMOURING TYPE II
C-214 1:50

PlotDate: 2019.11.25 4:40 PM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civilsheet_files\200_ds\73396C-285.dwg TENDER No. ?

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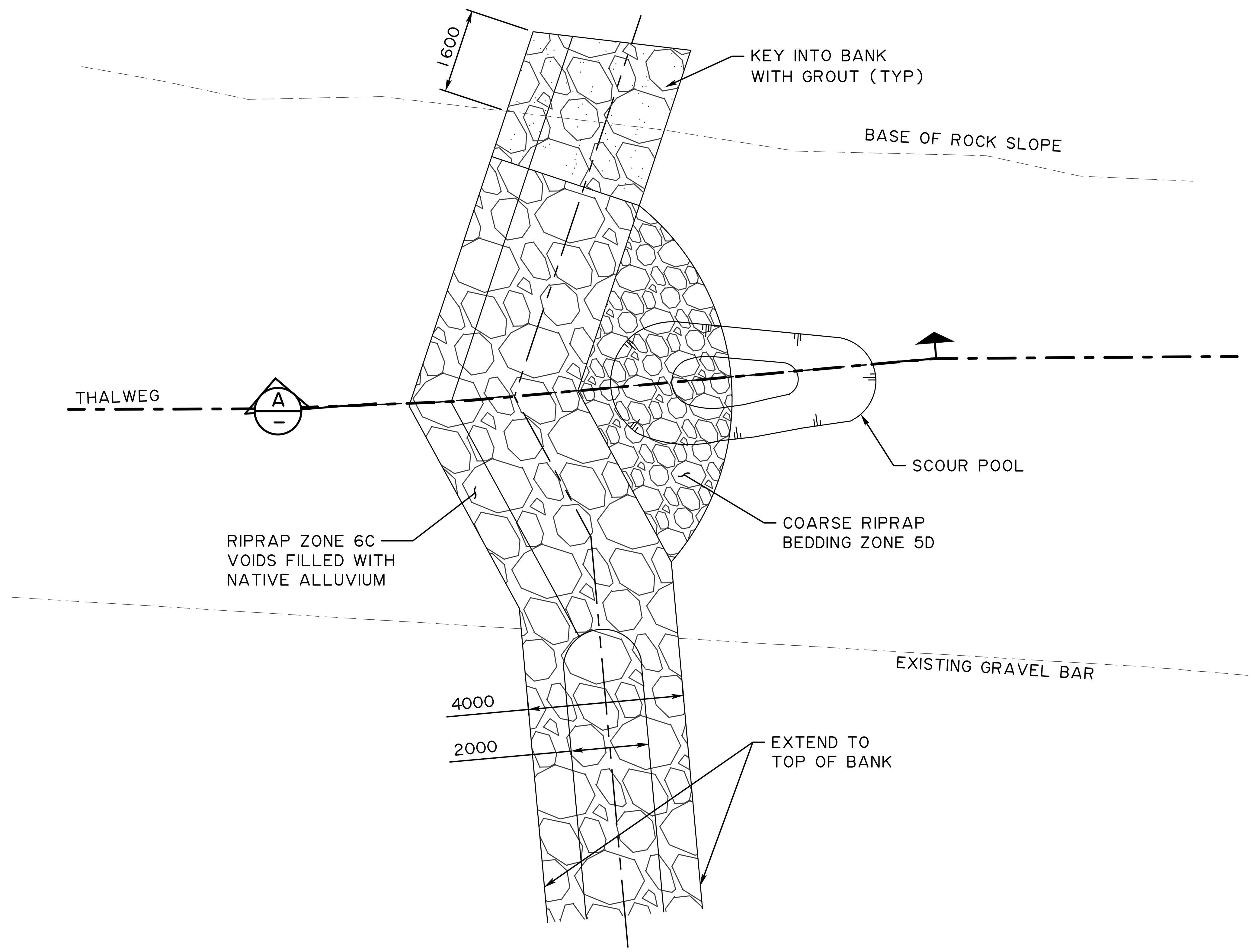
DESIGNER	CHECKER
DATE	DATE

Alberta Transportation

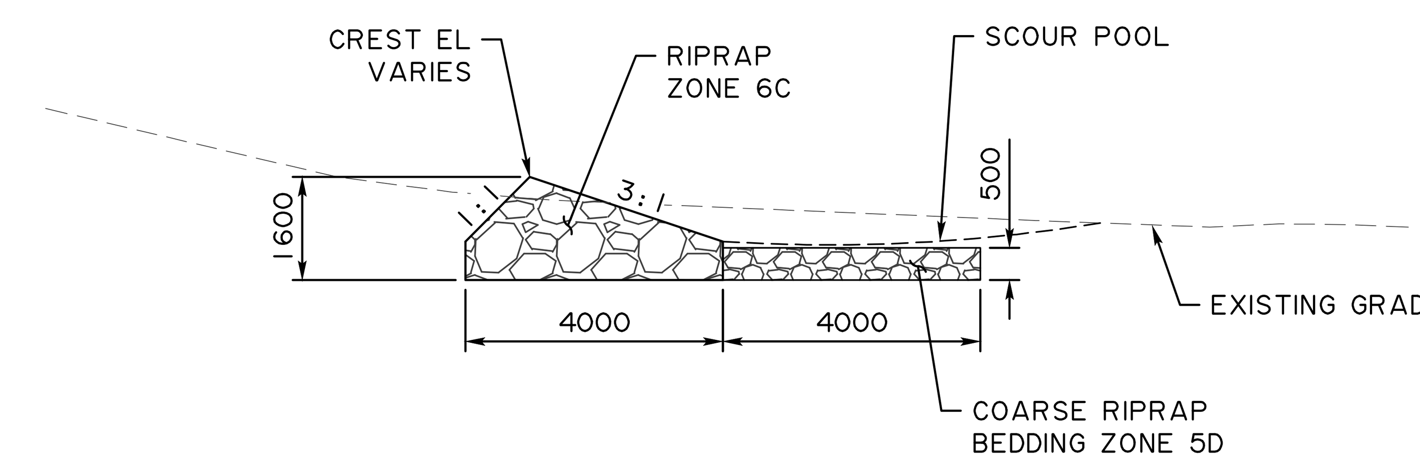
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
ELBOW RIVER
DETAILS 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-285	



1 C-215 DETAIL - V-WEIR 1:100



2 C-215 SECTION V-WEIR 1:100

PlotDate: 2019.11.25 4:43 PM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civilsheet_files\200_ds\73396C-286.dwg TENDER No. ?

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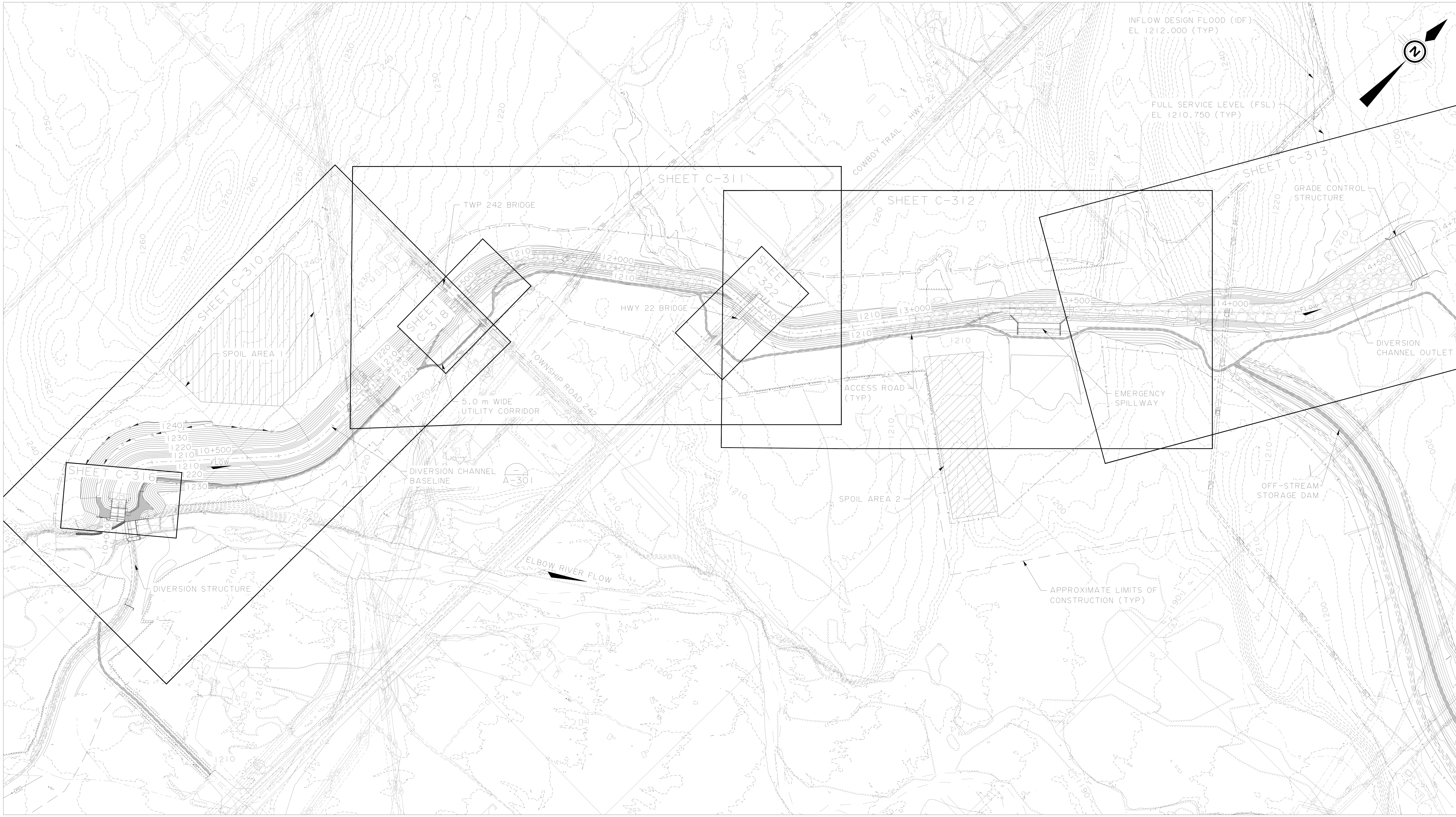
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE
ELBOW RIVER
DETAILS 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-286	



NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:6000

0 200 400 Metres

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
OVERVIEW

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-301	

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PlotDate: 2020.06.25 4:19 PM Login: Gindling, Kendall
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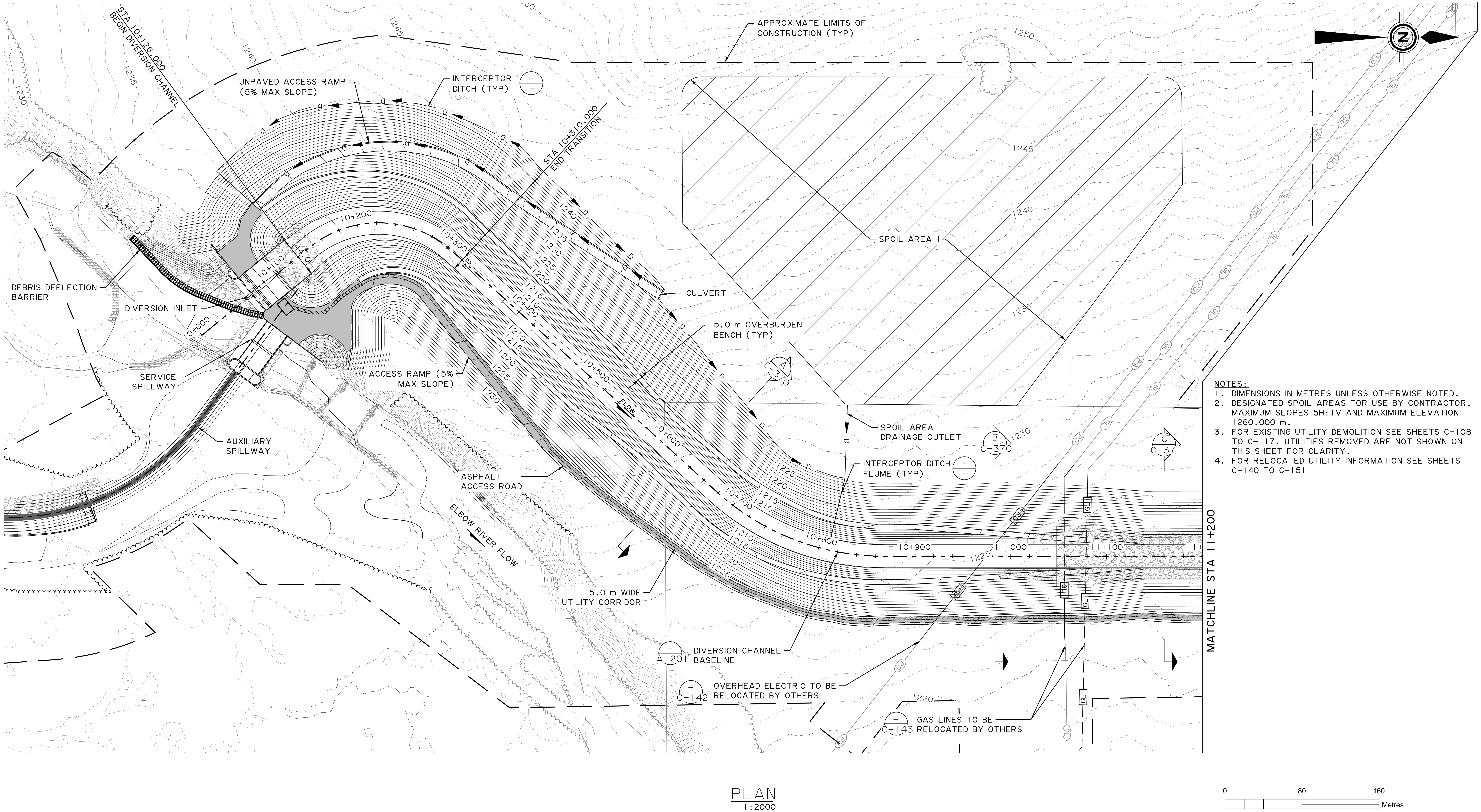
PlotDate: 2020.06.25 4:27 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\sheet_files\300_dc\73396C-310.dwg
TENDER No. ?



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

PLAN
1:2000

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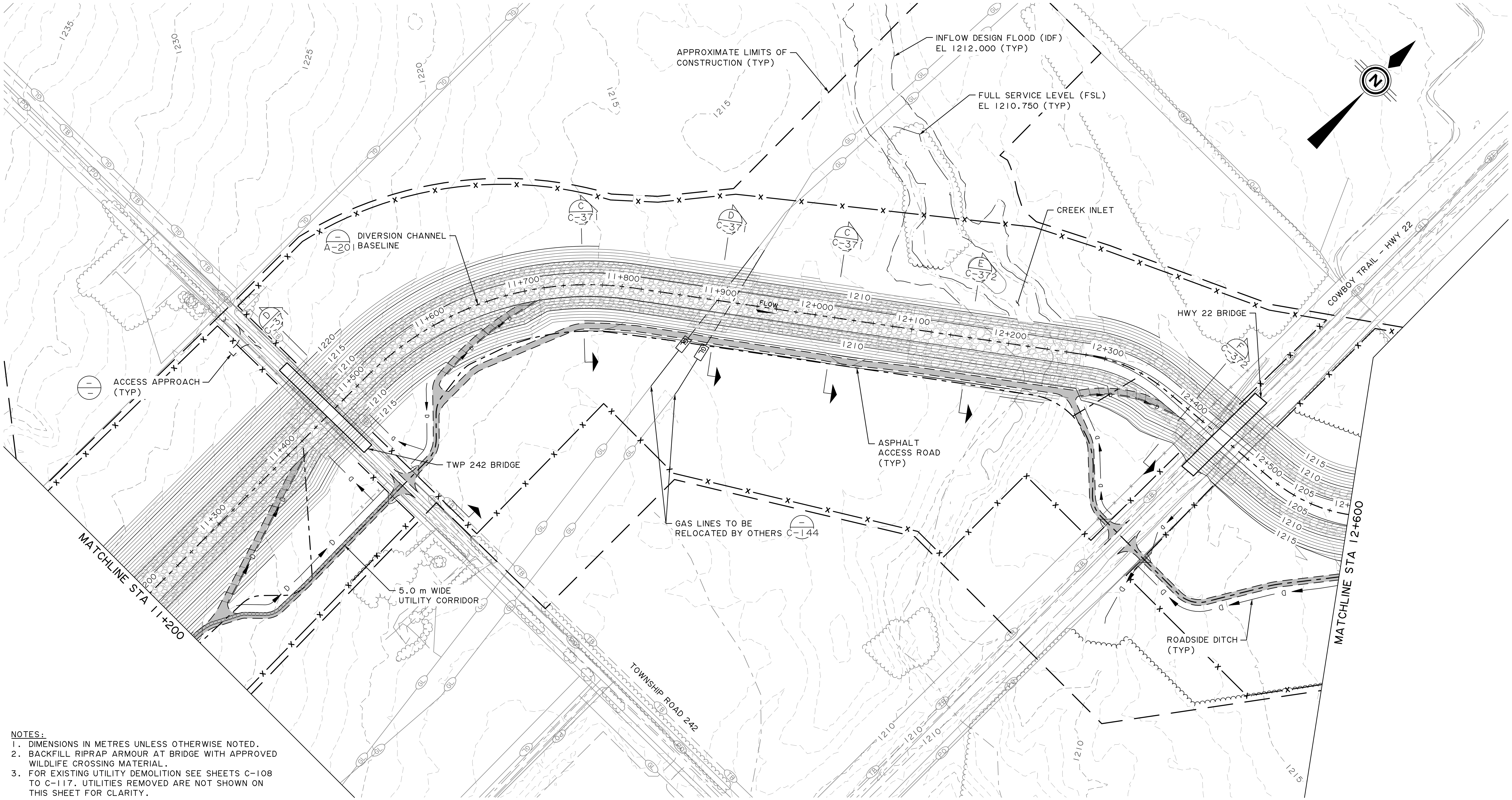
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PLAN 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-310	



PLAN
1:2000

0 80 160
Metres

NOTES:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
2. BACKFILL RIPRAP ARMOUR AT BRIDGE WITH APPROVED WILDLIFE CROSSING MATERIAL.
3. FOR EXISTING UTILITY DEMOLITION SEE SHEETS C-108 TO C-117. UTILITIES REMOVED ARE NOT SHOWN ON THIS SHEET FOR CLARITY.

TENDER No. ?

PlotDate: 2020.06.25 9:44 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\sheet_files\300_dc\73396C-311.dwg

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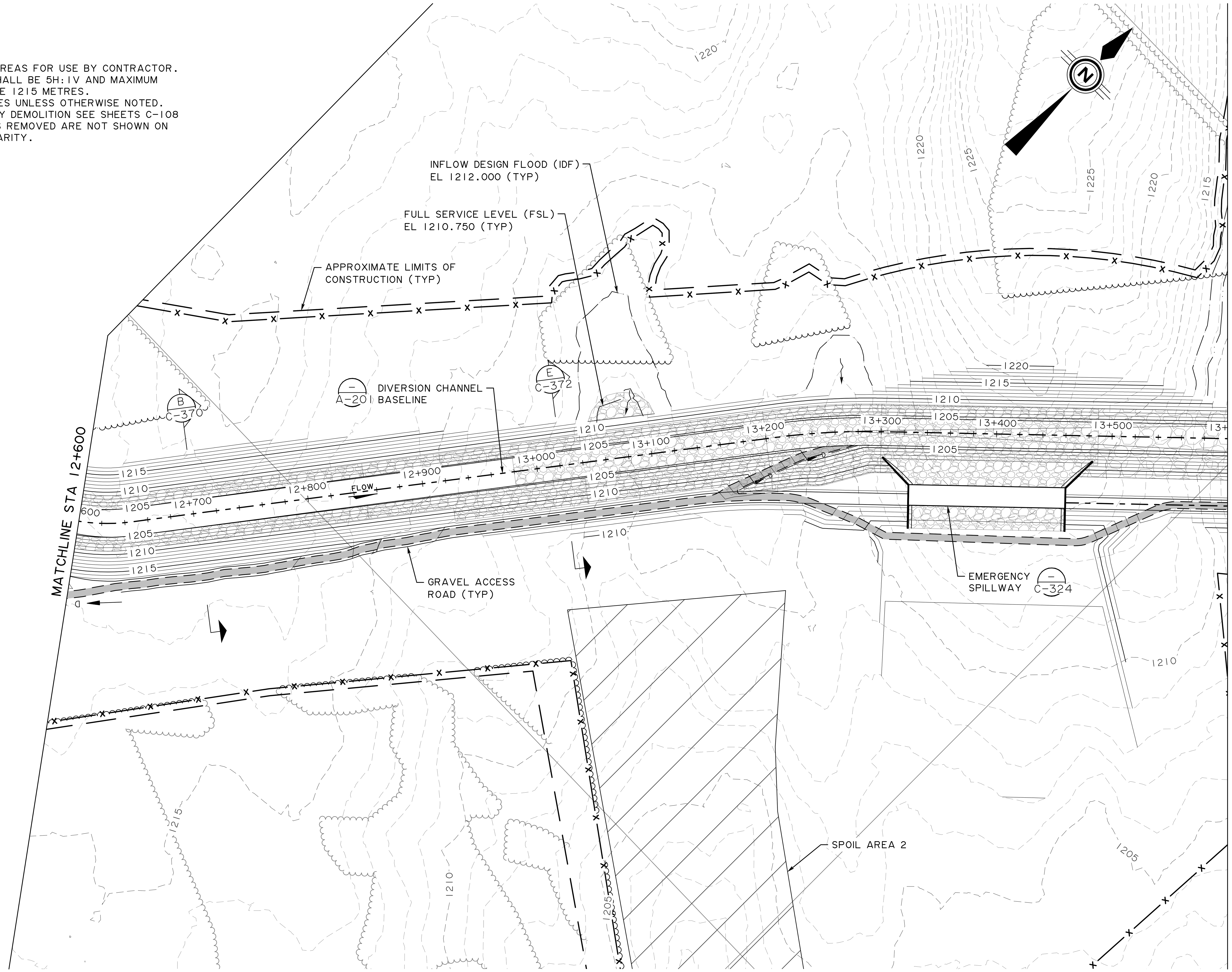
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PLAN 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-311	

NOTES:
1. DESIGNATED SPOIL AREAS FOR USE BY CONTRACTOR. MAXIMUM SLOPES SHALL BE 5H:1V AND MAXIMUM ELEVATION SHALL BE 1215 METRES.
2. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.
3. FOR EXISTING UTILITY DEMOLITION SEE SHEETS C-108 TO C-117. UTILITIES REMOVED ARE NOT SHOWN ON THIS SHEET FOR CLARITY.



PLAN
1:2000

0 80 160
Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PLAN 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-312	

PlotDate: 2020.06.25 9:50 PM Login: Gindling, Kendall
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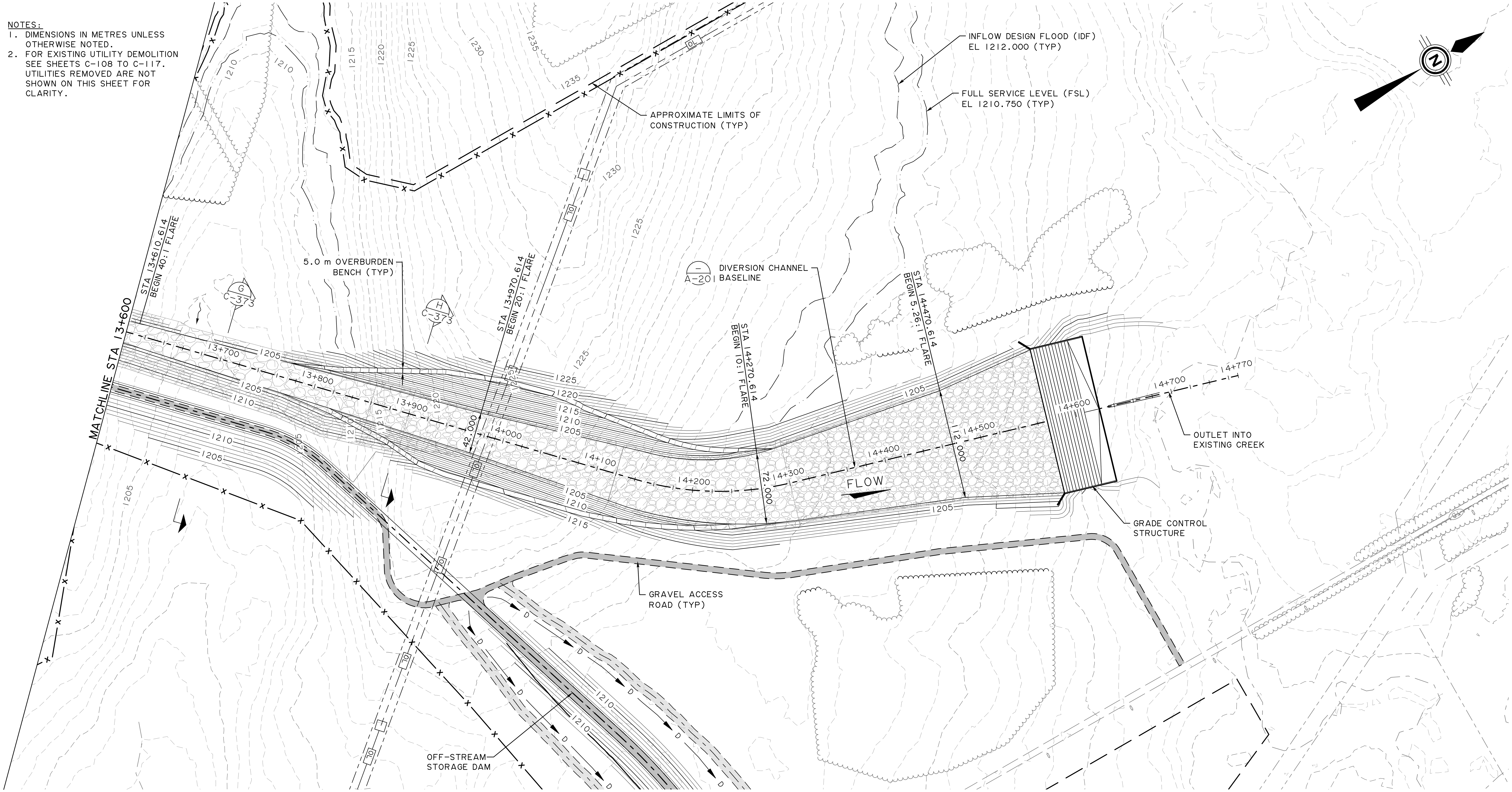
TENDER No. ?

NOTES:
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
1:2000

0 80 160 Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION CHANNEL PLAN 4

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-313	

PlotDate: 2020.06.25 9:58 PM Login: Gindling, Kendall
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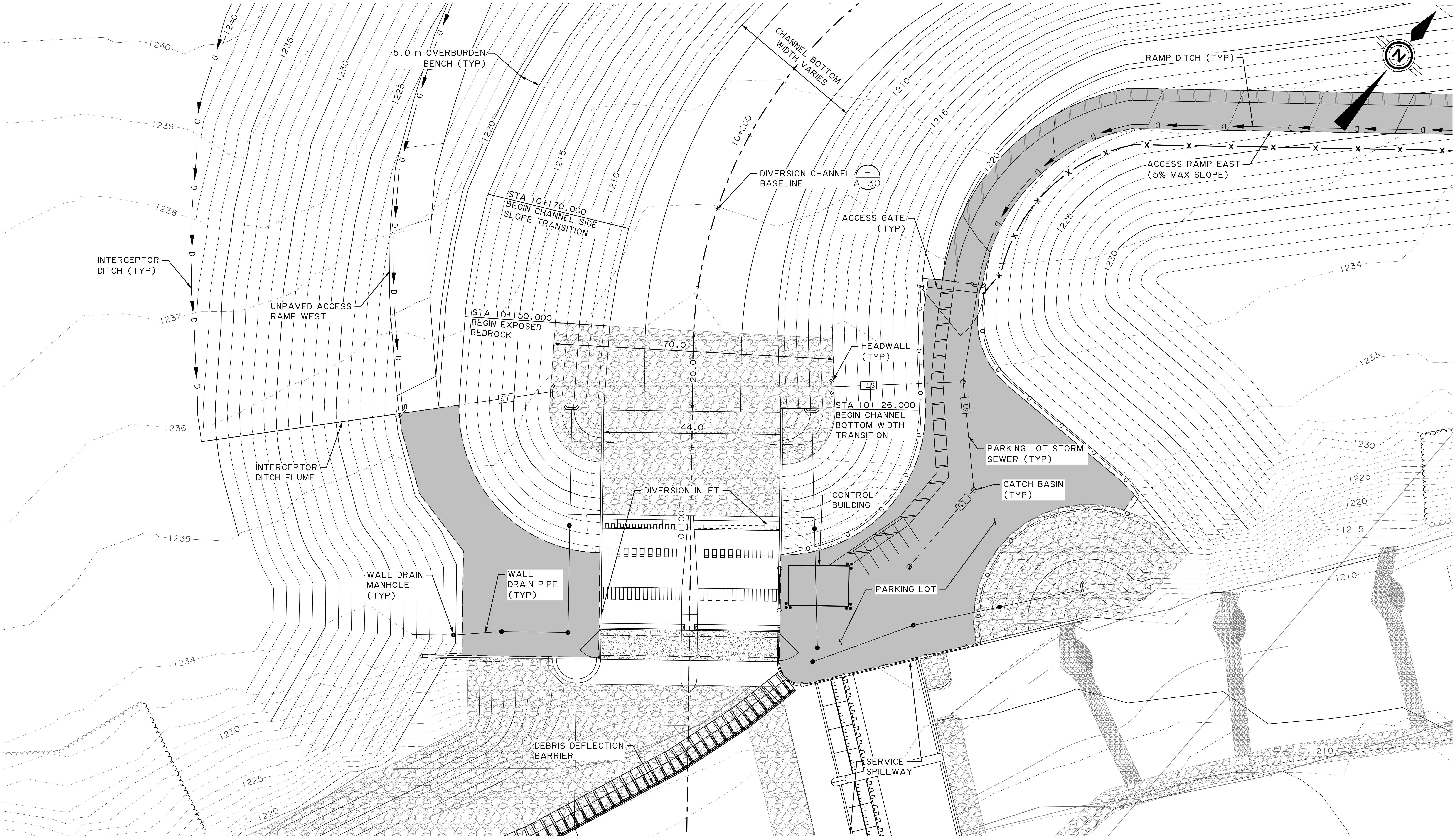

NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:500

0 20 40
Metres

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PlotDate: 2020.06.25 10:09 PM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\sheet_files\300_dc\73396C-316.dwg TENDER No. ?

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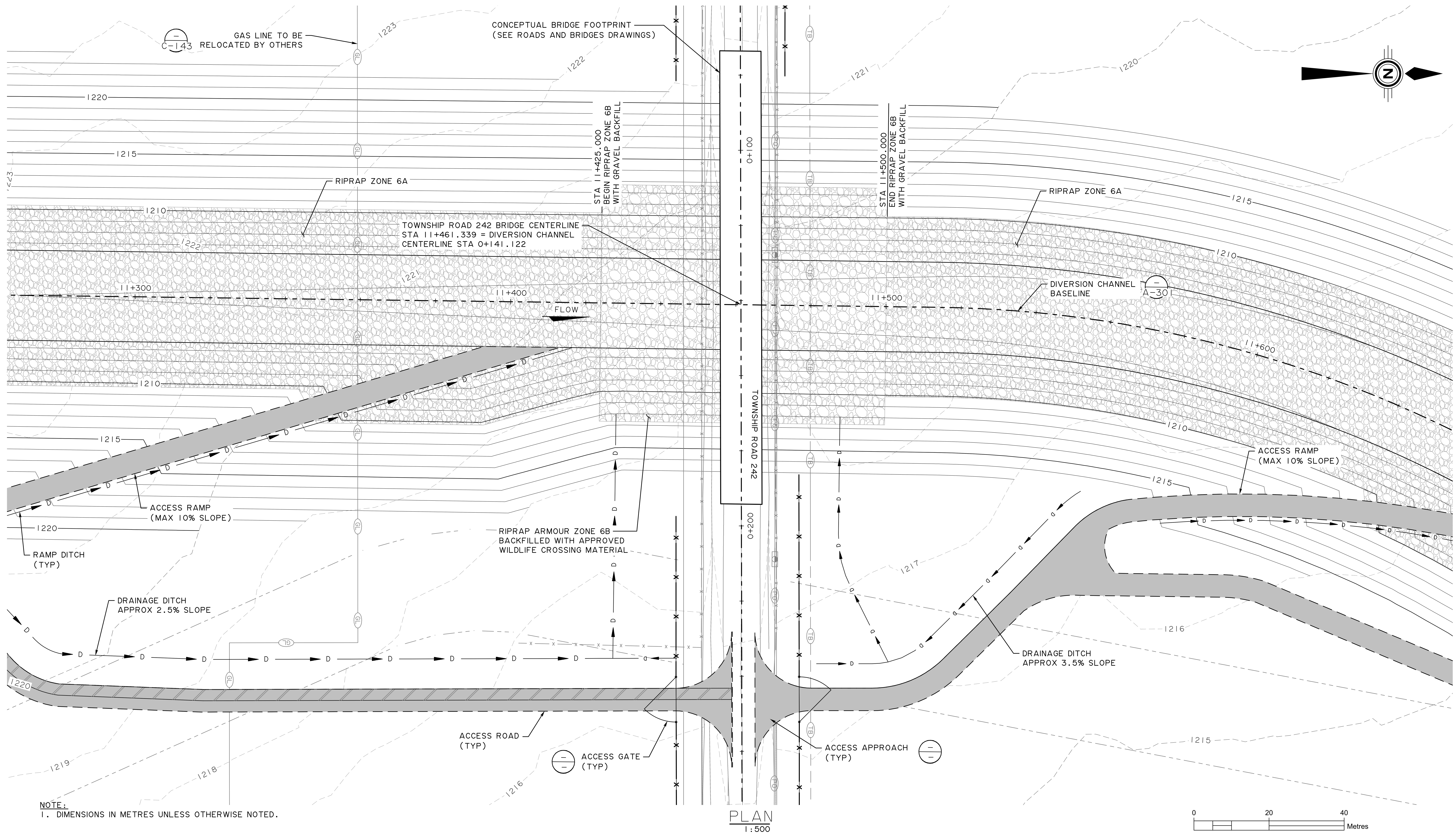
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PLAN DETAIL 1
DIVERSION INLET

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-316	



PlotDate: 2020.06.26 2:44 AM Login: Sprague, Adam

AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\sheet_files\300_dc\73396C-318.dwg

TENDER No. ?

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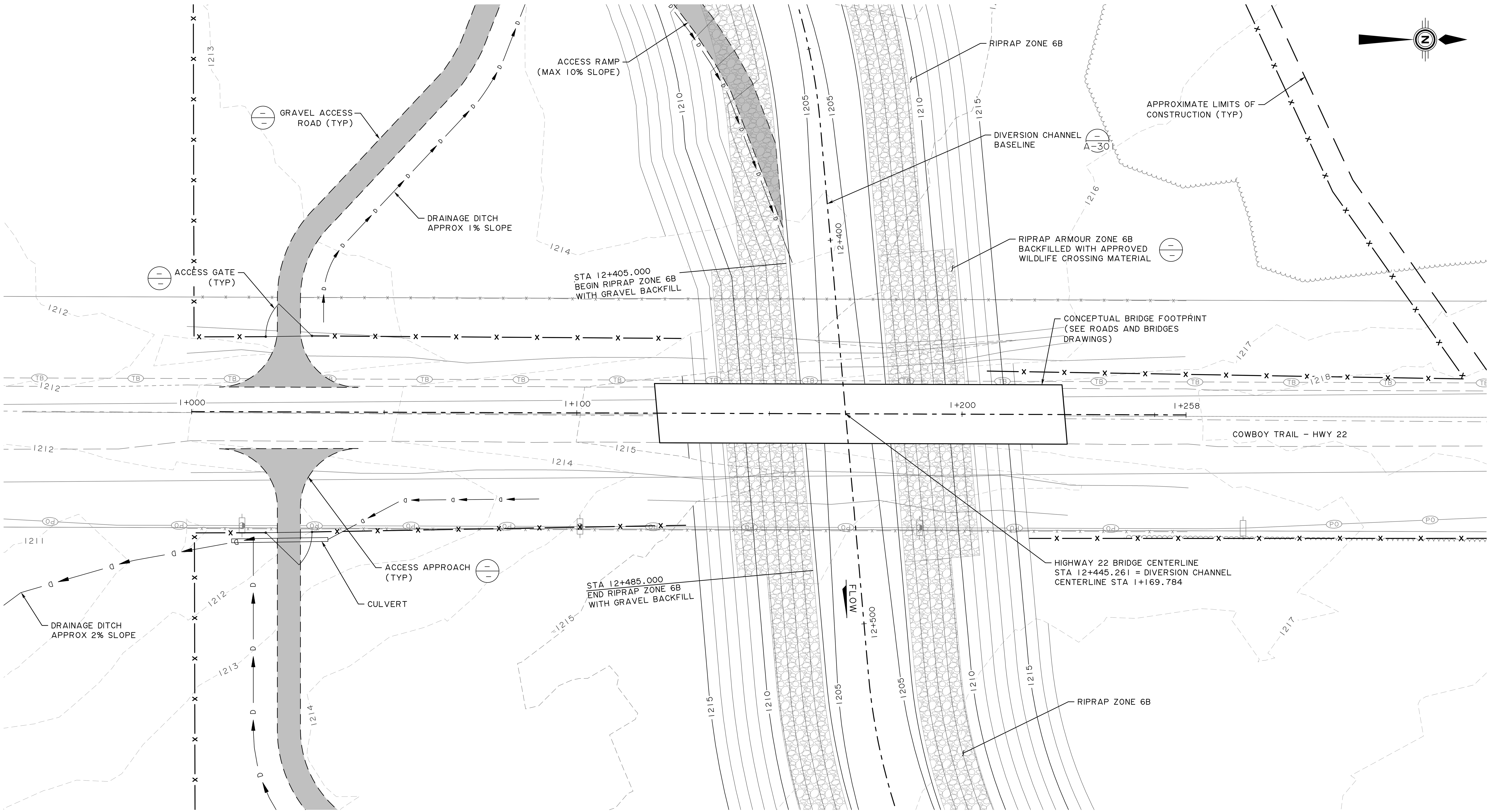
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PLAN DETAIL 2
BRIDGE CROSSING TWP 242

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-318	



NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:500

0 20 40 Metres

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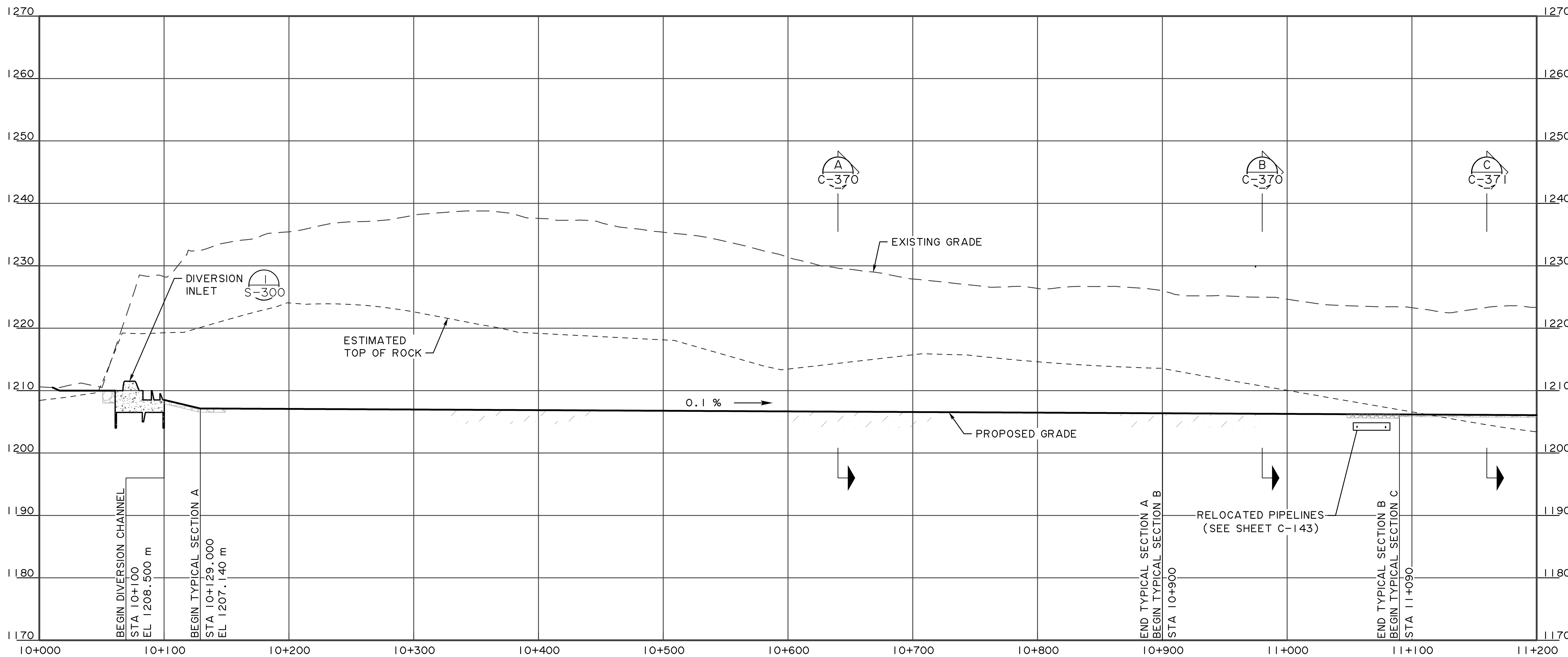
SR1

DIVERSION CHANNEL
PLAN DETAIL 3
BRIDGE CROSSING HWY 22

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-322	

PlotDate: 2020.06.26 7:02 AM Login: Gindling, Kendall
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\working\BACKUP\rev_2_drawings\c_civil\sheet_files\300_dc\73396C-322.dwg TENDER No. ?



PROFILE
H 1:2000 V 1:400

NOTES:
1. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.
2. SEE TYPICAL SECTIONS (C-370 TO C-374) FOR RIPRAP ARMOUR REQUIREMENTS.
3. FOR EXISTING UTILITY DEMOLITION, SEE SHEETS C-108 TO C-117. EXISTING UTILITIES TO BE REMOVED OR RELOCATED ARE NOT SHOWN.

PlotDate: 2019.12.03 10:31 AM Login: Sawka, Katie-Grace TENDER No. ?
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SPRINGBANK OFF-STREAM STORAGE PROJECT
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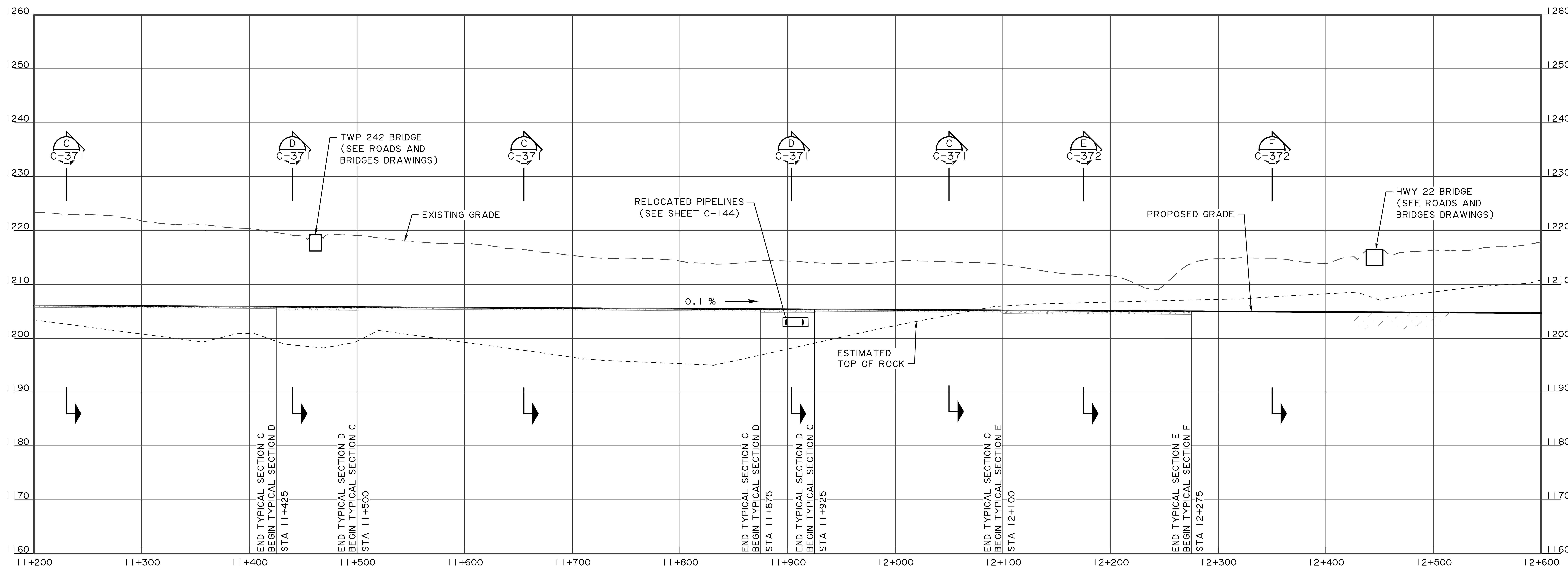
DIVERSION CHANNEL
PROFILE 1
STA 10+000 TO STA 11+200

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-330	



PROFILE
H 1:2000 V 1:400

NOTES:
1. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.
2. SEE TYPICAL SECTIONS (C-370 TO C-374) FOR RIPRAP ARMOUR REQUIREMENTS.
3. FOR EXISTING UTILITY DEMOLITION, SEE SHEETS C-108 TO C-117. EXISTING UTILITIES TO BE REMOVED OR RELOCATED ARE NOT SHOWN.

PlotDate: 2019.12.03 10:43 AM Login: Sawka, Katie-Grace TENDER No. ?
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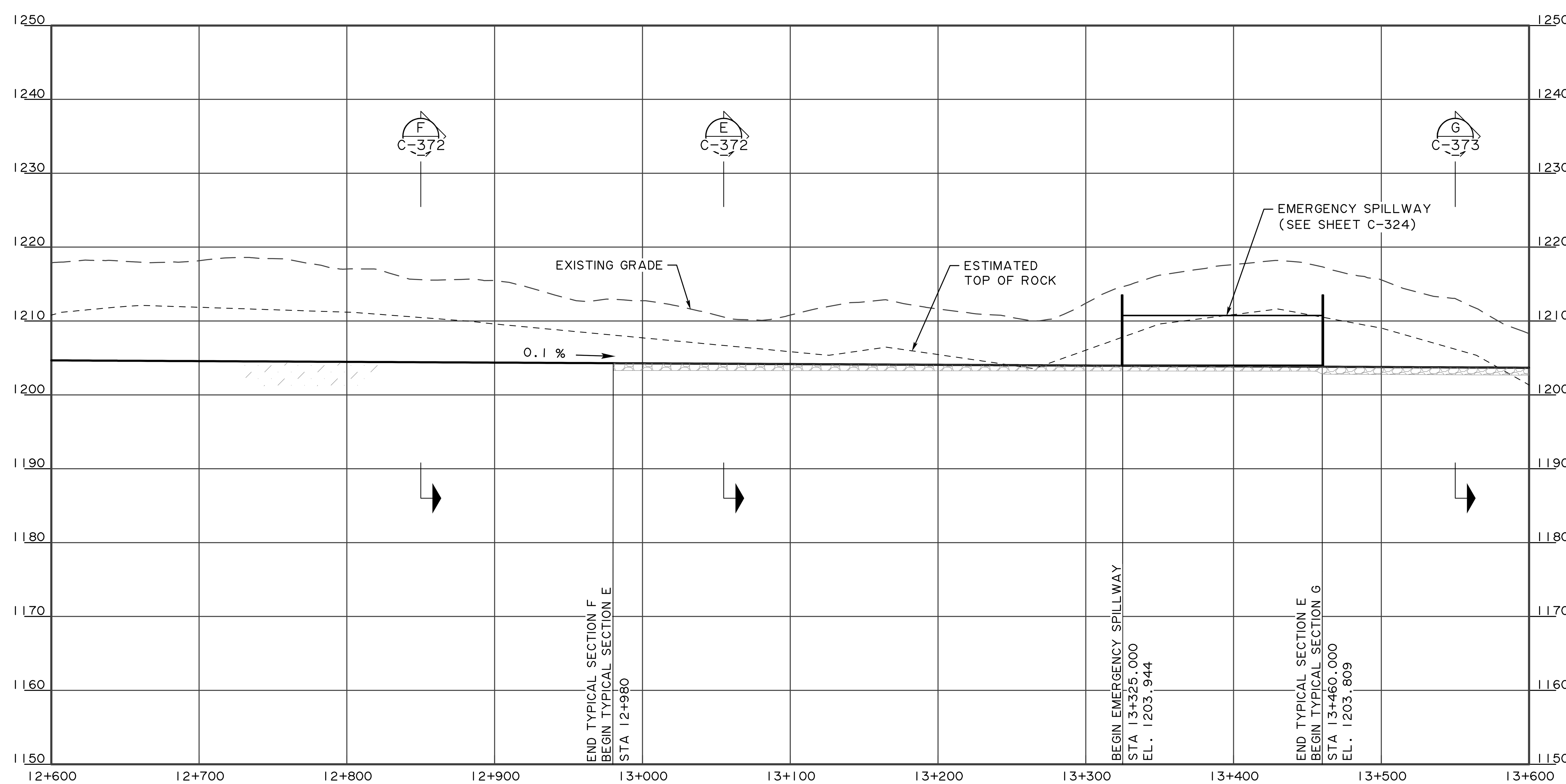
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PROFILE 2
STA 11+200 TO STA 12+600

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-331	



PROFILE
H: 1:2000 V: 1:400

NOTES:
1. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.
2. SEE TYPICAL SECTIONS (C-370 TO C-374) FOR RIPRAP ARMOUR REQUIREMENTS.
3. FOR EXISTING UTILITY DEMOLITION, SEE SHEETS C-108 TO C-117. EXISTING UTILITIES TO BE REMOVED OR RELOCATED ARE NOT SHOWN.

PlotDate: 2019.12.03 11:05 AM Login: Sawka, Katie-Grace
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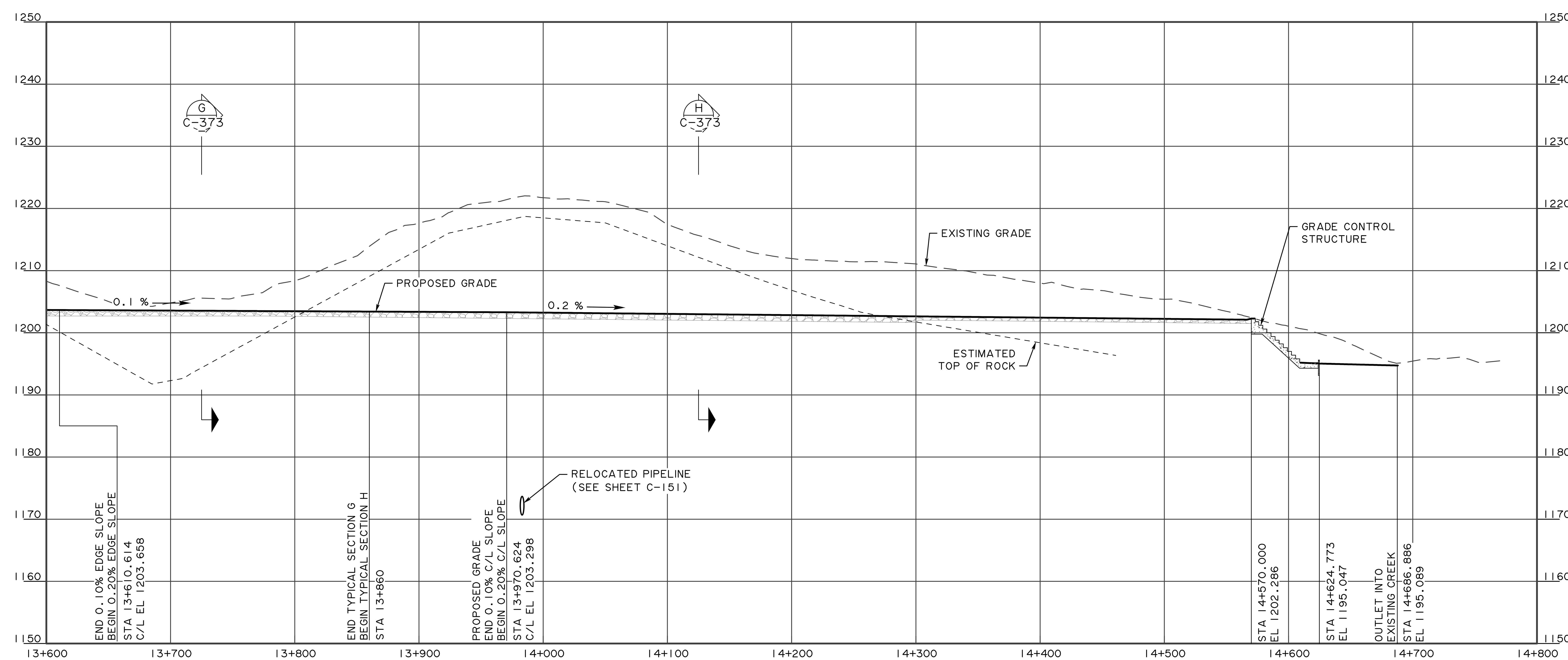
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
PROFILE 3
STA 12+600 TO STA 13+600

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-332	



PROFILE

H: 1:2000 V: 1:400

NOTES:

1. TOP OF ROCK LINE SHOWN WAS GENERALLY ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS. FROM STATION 13+100 TO STATION 14+200, THE TOP OF ROCK LINE SHOWN WAS MODIFIED TO GENERALLY FOLLOW EXISTING GROUND TOPOGRAPHY WITH PRORATED DEPTHS BETWEEN KNOWN DEPTHS AT BORINGS.
2. SEE TYPICAL SECTIONS (C-370 TO C-374) FOR RIPRAP ARMOUR REQUIREMENTS.
3. FOR EXISTING UTILITY DEMOLITION, SEE SHEETS C-108 TO C-117. EXISTING UTILITIES TO BE REMOVED OR RELOCATED ARE NOT SHOWN.

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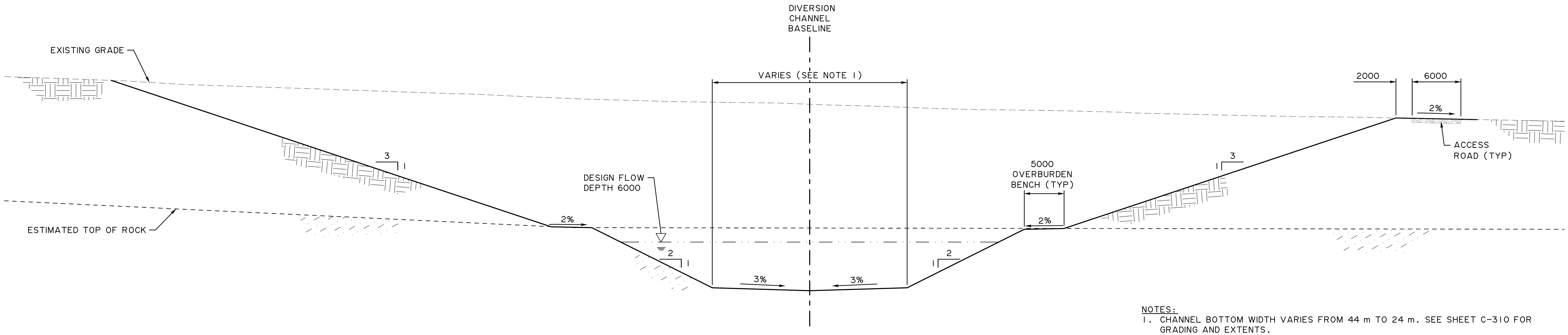
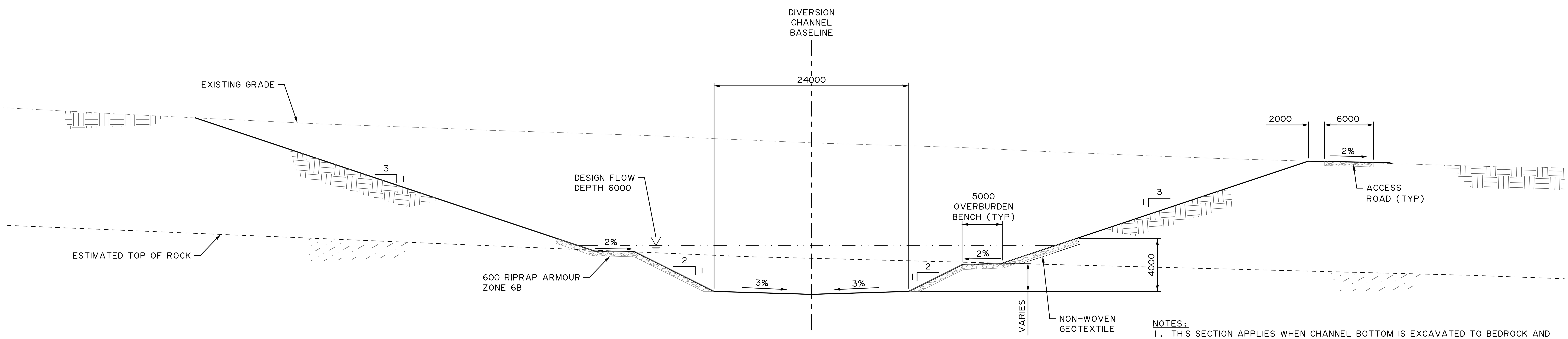
DIVERSION CHANNEL
PROFILE 4
STA 13+600 TO 14+800

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-333	



NOTES:
1. CHANNEL BOTTOM WIDTH VARIES FROM 44 m TO 24 m. SEE SHEET C-310 FOR GRADING AND EXTENTS.
2. THIS SECTION APPLIES WHEN CHANNEL IS EXCAVATED TO BEDROCK AND THE OVERBURDEN BENCH IS A MINIMUM 7 m ABOVE CHANNEL INVERT.

A / C-310 DIVERSION CHANNEL- TYPICAL SECTION
ROCK EXCAVATION NTS



NOTES:
1. THIS SECTION APPLIES WHEN CHANNEL BOTTOM IS EXCAVATED TO BEDROCK AND OVERBURDEN BENCH IS LESS THAN 4 m ABOVE CHANNEL INVERT.

B / C-310 DIVERSION CHANNEL- TYPICAL SECTION
ROCK EXCAVATION TO SOIL TRANSITION NTS

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION CHANNEL TYPICAL SECTIONS 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-370	

PlotDate: 2019.12.03 11:25 AM Login: Sawka, Katie-Grace TENDER No. ?
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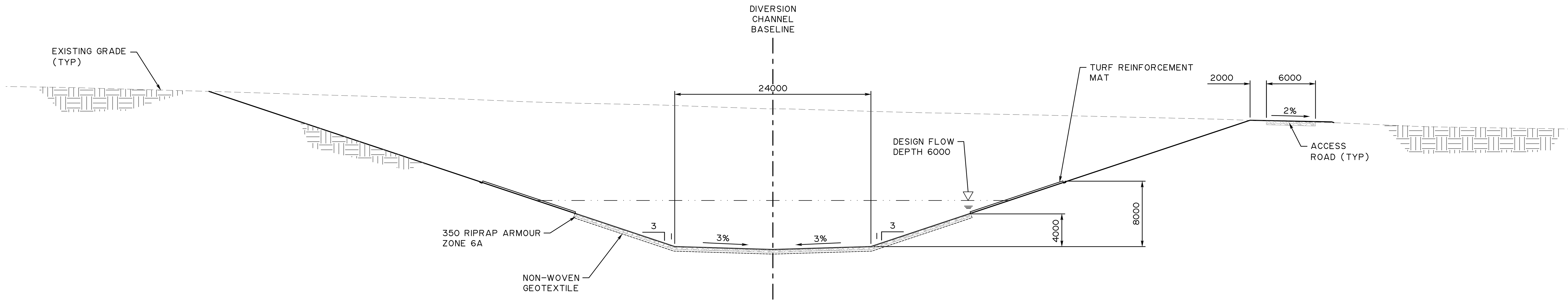
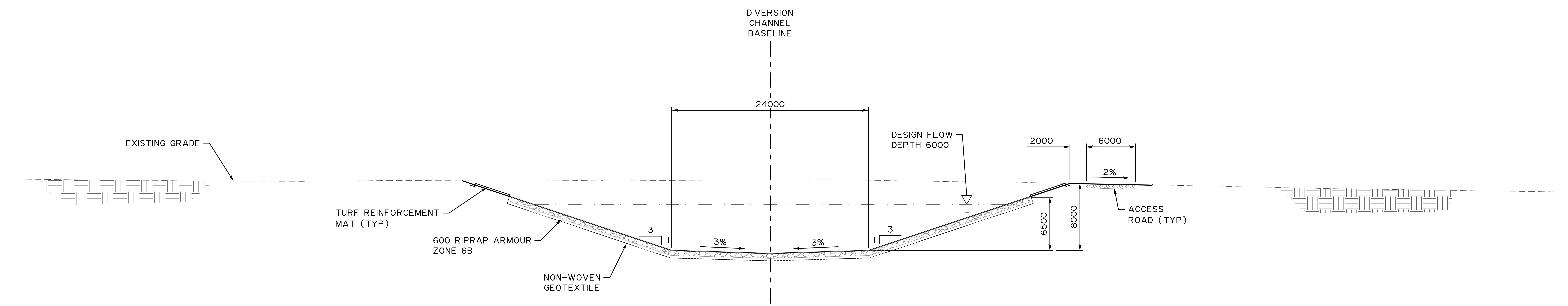


C / C-310 DIVERSION CHANNEL- TYPICAL SECTION
SOIL EXCAVATION NTS

NOTES:
1. THIS SECTION APPLIES FOR CHANNEL EXCAVATION IN SOIL UNLESS OTHERWISE NOTED.



D / C-311 DIVERSION CHANNEL- TYPICAL SECTION
SOIL EXCAVATION AT UTILITY CROSSING OR ROADWAY NTS

NOTES:
1. THIS SECTION APPLIES TO IDENTIFIED SECTIONS OF CHANNEL THAT CROSS UTILITY OR TRANSPORTATION INFRASTRUCTURE.

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION CHANNEL TYPICAL SECTIONS 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-371	

PlotDate: 2019.12.03 11:45 AM Login: Sawka, Katie-Grace TENDER No. ?
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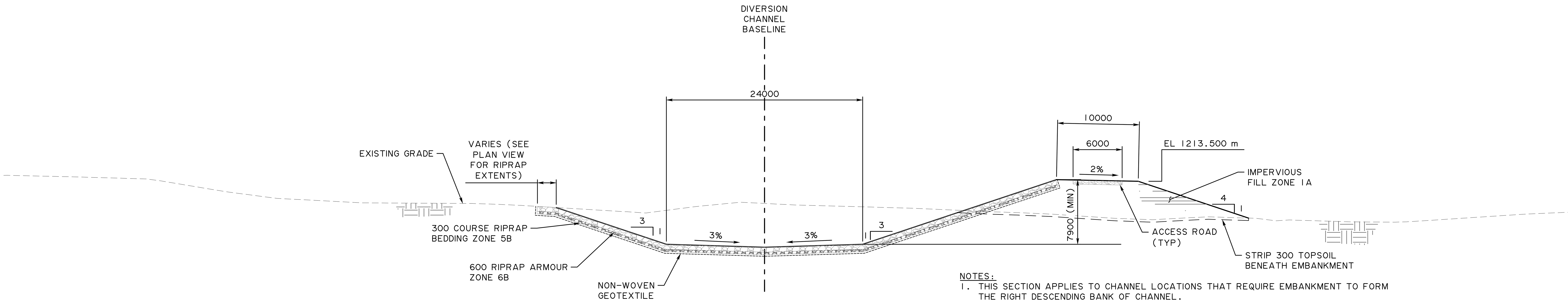
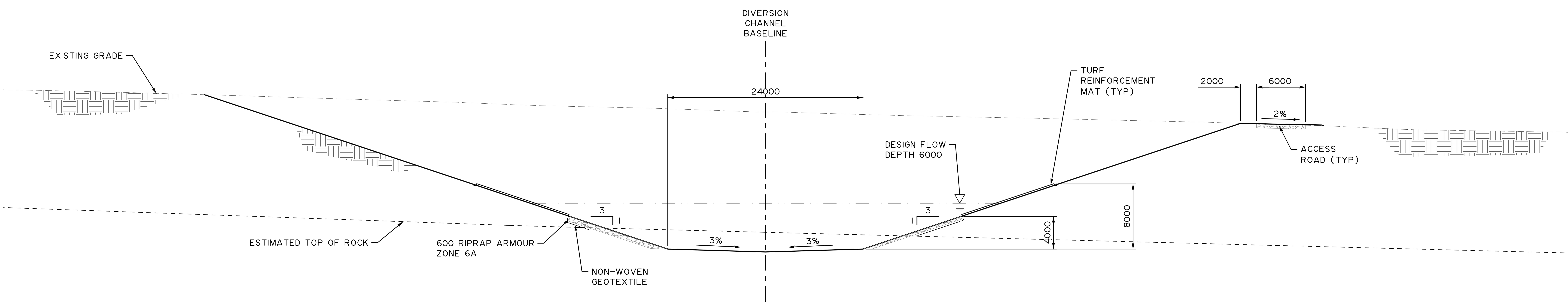


NOTES:
1. THIS SECTION APPLIES TO CHANNEL LOCATIONS THAT REQUIRE EMBANKMENT TO FORM THE RIGHT DESCENDING BANK OF CHANNEL.

E / C-311 DIVERSION CHANNEL- TYPICAL SECTION

SOIL EXCAVATION WITH EMBANKMENT — NTS



F / C-311 DIVERSION CHANNEL- TYPICAL SECTION

ROCK EXCAVATION TO SOIL TRANSITION — NTS

NOTES:
1. THIS SECTION APPLIES FOR CHANNEL EXCAVATION WITH ROCK BOTTOM AND SOIL WITHIN 4 m OF CHANNEL INVERT.

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DIVERSION CHANNEL TYPICAL SECTION 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-372	

PlotDate: 2019.12.03 12:11 PM Login: Sawka, Katie-Grace
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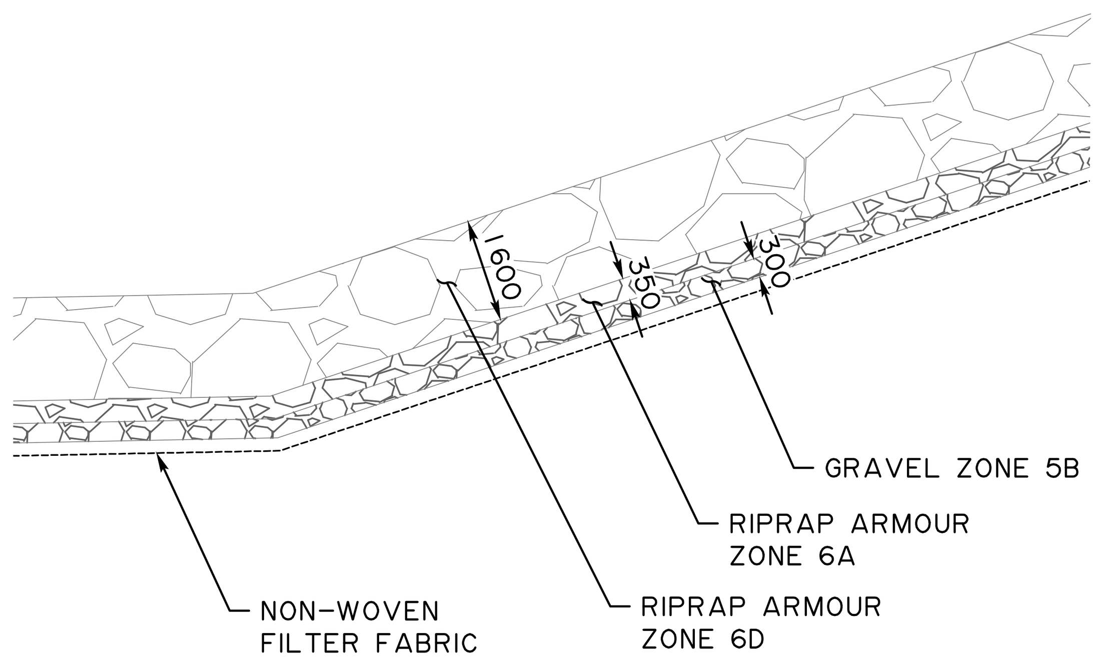
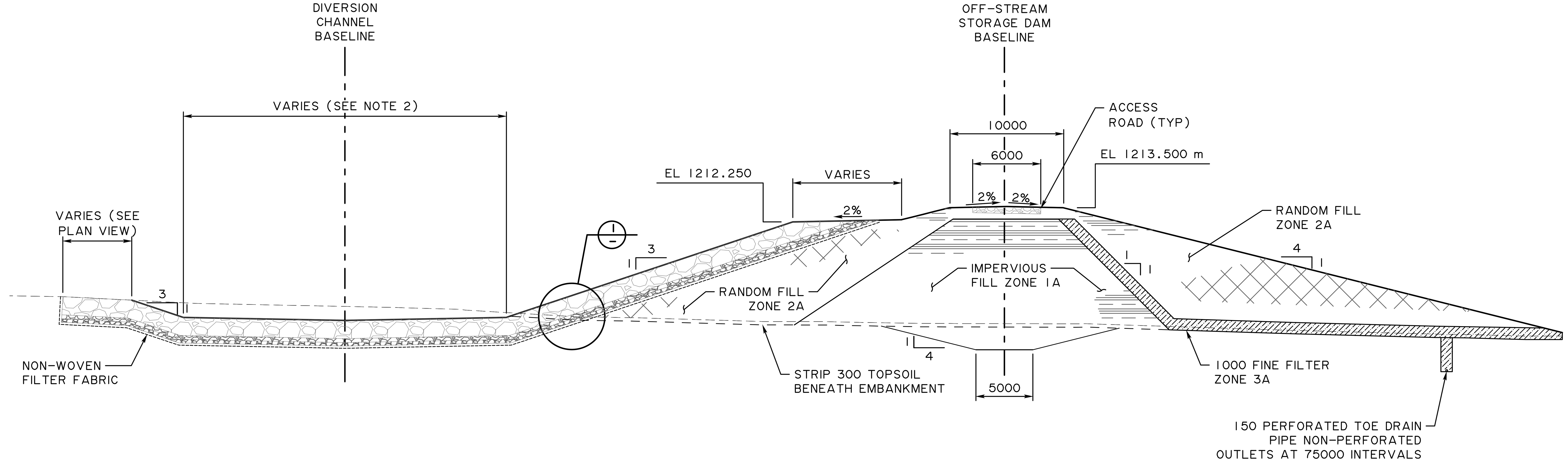
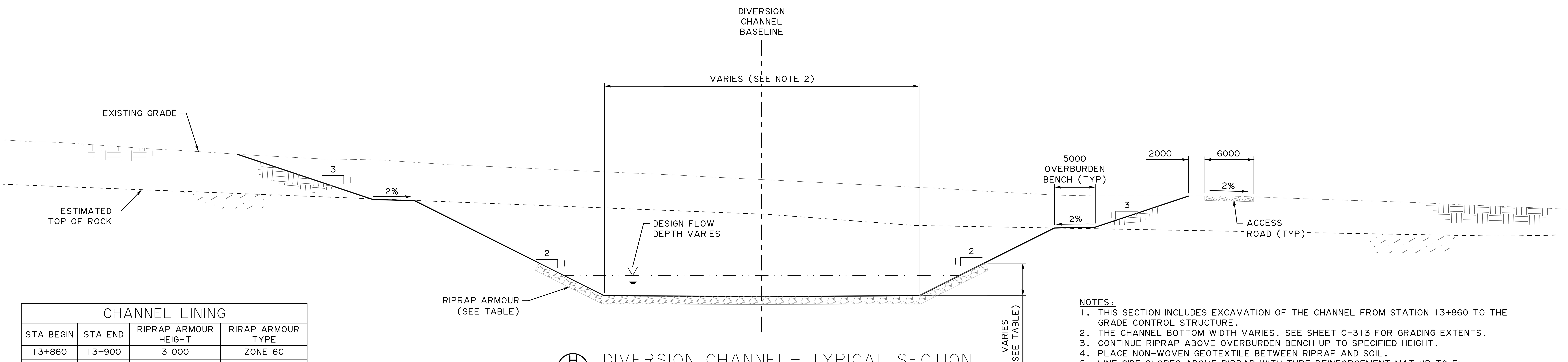


1 — ARMOURING LAYERS 1:250



G / C-313 — DIVERSION CHANNEL- TYPICAL SECTION
SOIL EXCAVATION WITH EMBANKMENT 1:250

NOTES:
1. THIS SECTION APPLIES TO THE CHANNEL FOR STA 13+488 TO 13+860.
2. BOTTOM WIDTH OF THIS SECTION VARIES. SEE SHEET C-313 FOR GRADING AND EXTENTS.
3. CHIMNEY DRAIN REQUIRED WHEN EMBANKMENT HEIGHT EXCEEDS 2 m.



H / C-313 — DIVERSION CHANNEL- TYPICAL SECTION
ROCK EXCAVATION TO SOIL TRANSITION 1:250

CHANNEL LINING

STA BEGIN	STA END	RIPRAP ARMOUR HEIGHT	RIRAP ARMOUR TYPE
13+860	13+900	3 000	ZONE 6C
13+900	14+300	2 000	ZONE 6C
14+300	14+570	1 000	ZONE 6B

NOTES:
1. THIS SECTION INCLUDES EXCAVATION OF THE CHANNEL FROM STATION 13+860 TO THE GRADE CONTROL STRUCTURE.
2. THE CHANNEL BOTTOM WIDTH VARIES. SEE SHEET C-313 FOR GRADING EXTENTS.
3. CONTINUE RIPRAP ABOVE OVERBURDEN BENCH UP TO SPECIFIED HEIGHT.
4. PLACE NON-WOVEN GEOTEXTILE BETWEEN RIPRAP AND SOIL.
5. LINE SIDE SLOPES ABOVE RIPRAP WITH TURF REINFORCEMENT MAT UP TO EL 1212.000 m. TURF REINFORCEMENT MAT IS NOT REQUIRED FOR BEDROCK SLOPES.

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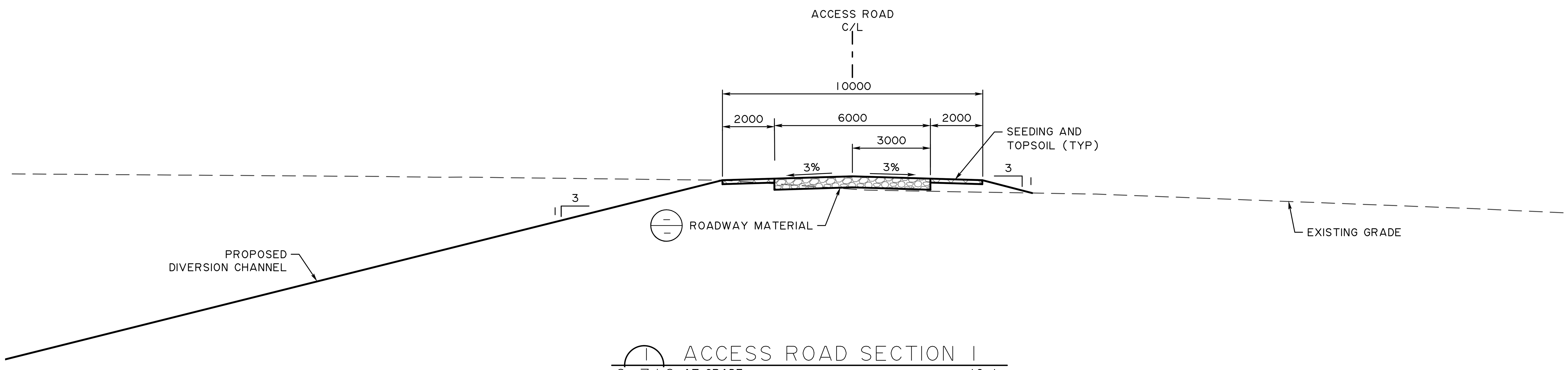
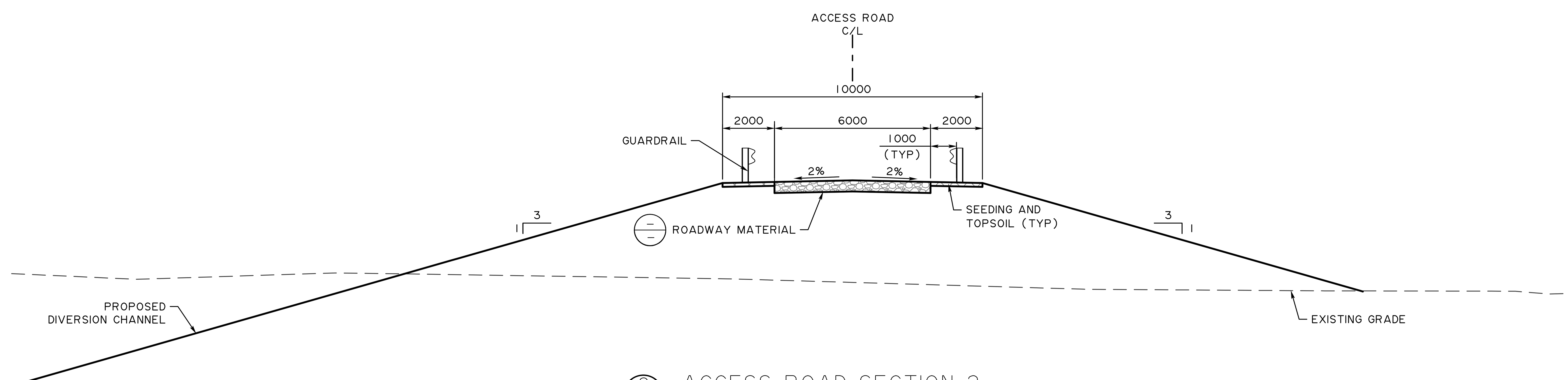
SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION CHANNEL TYPICAL SECTION 4

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-373	



1 C-310 ACCESS ROAD SECTION 1
AT GRADE 10:1



2 C-311 ACCESS ROAD SECTION 2
ON EMBANKMENT 10:1

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

PlotDate: 2019.12.03 1:04 PM Login: Sawka, Katie-Grace
AUTOCAD FILE No. \\Us0268-ppfss01\shared_projects\110773396\CAD\water\c_civil\sheet_files\300_dc\73396C-388.dwg TENDER No. ?

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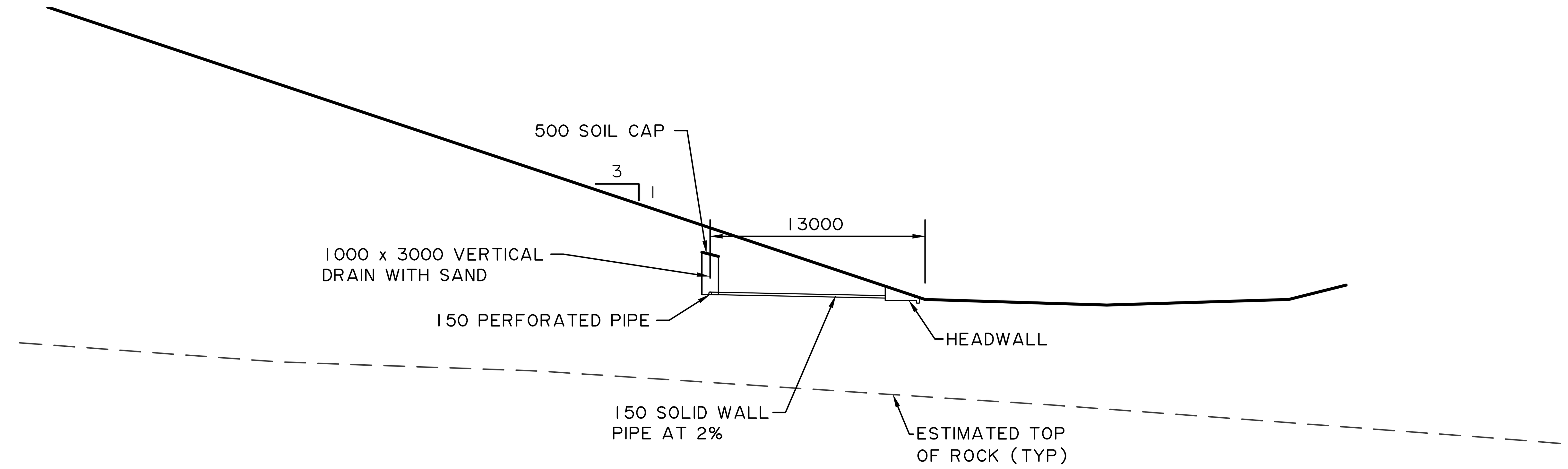
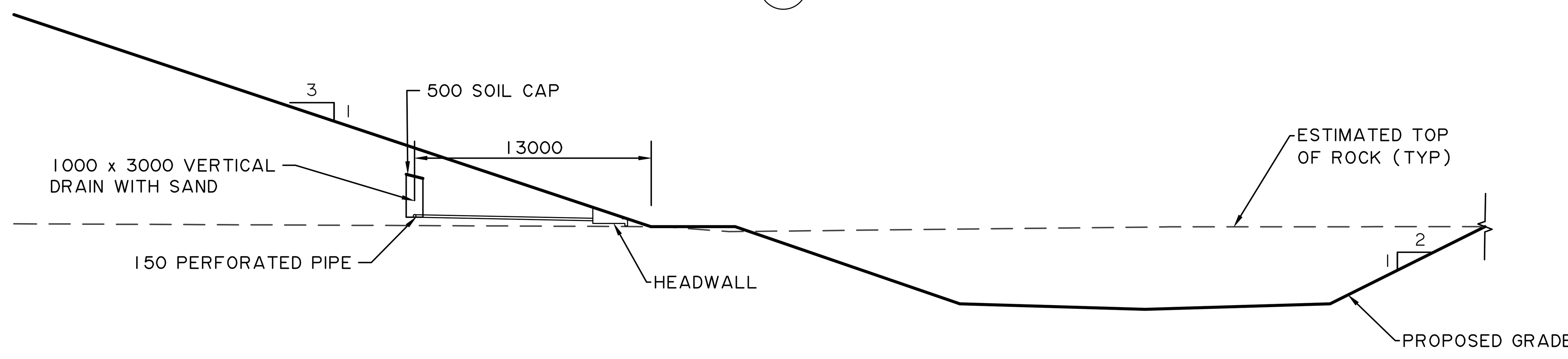
SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION CHANNEL
ACCESS ROADS
DETAILS

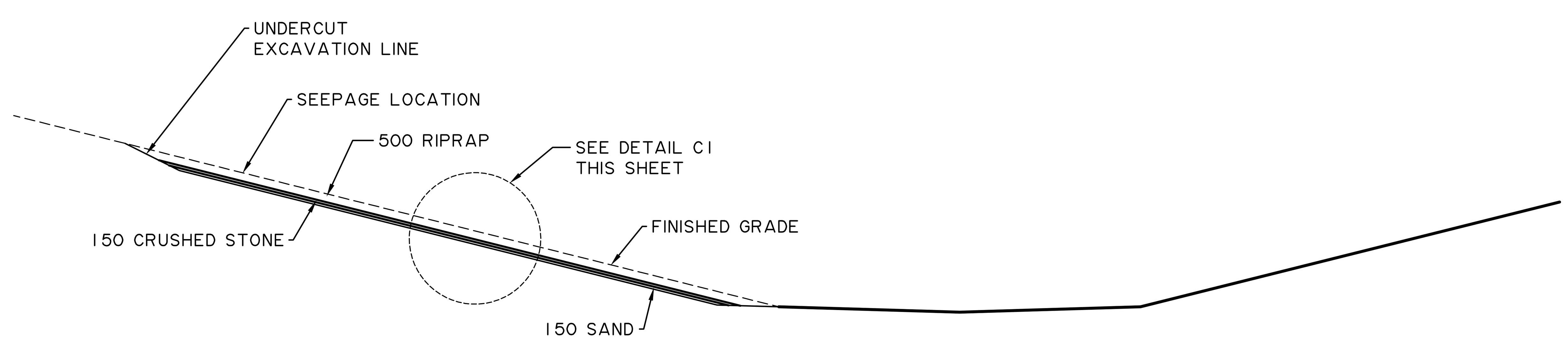
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-388	



VERTICAL DRAIN- SOIL SLOPE
NTS

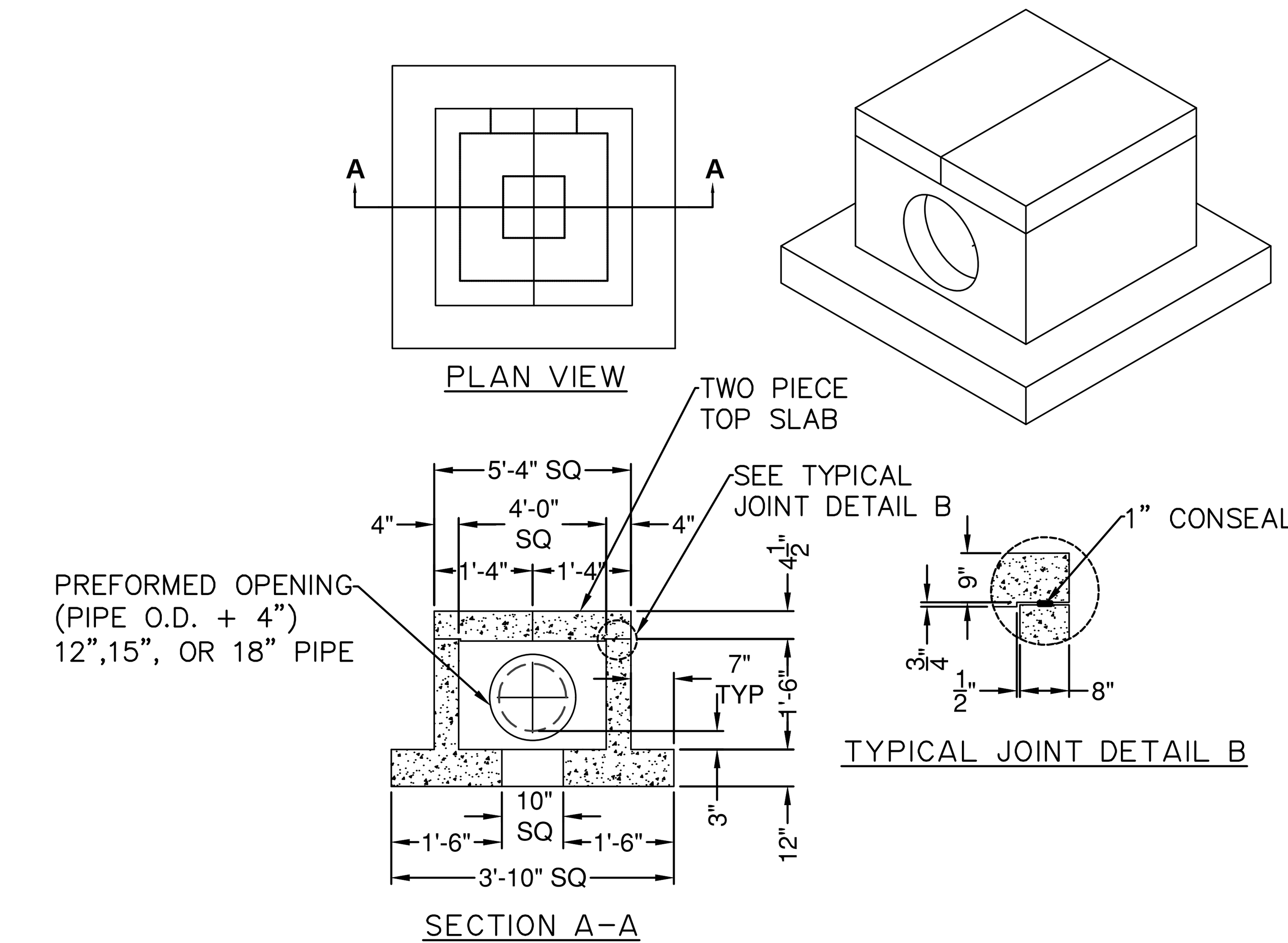


VERTICAL DRAIN-SOIL AND ROCK SLOPE
NTS

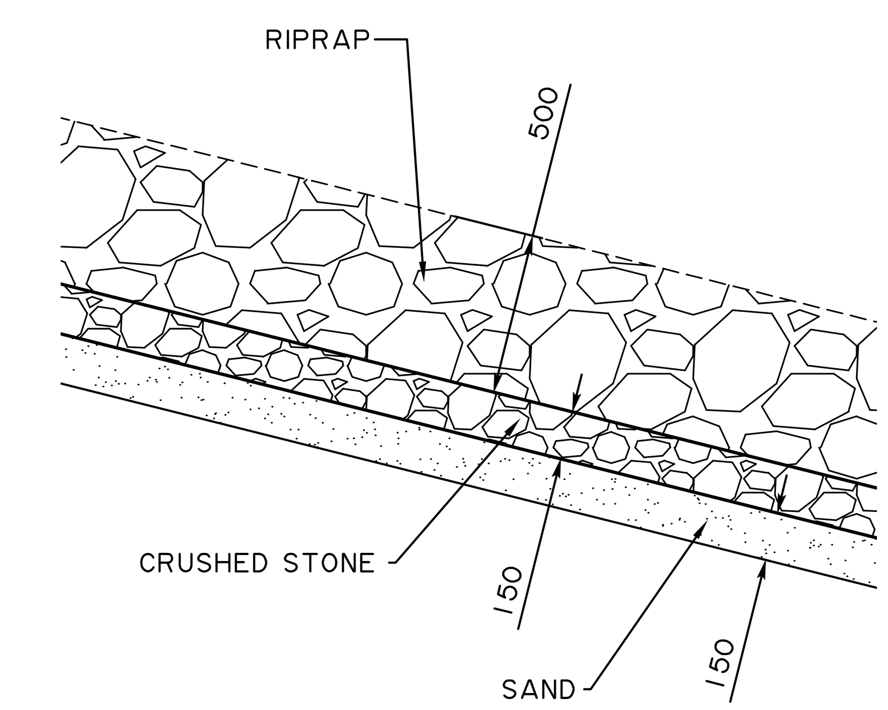


REVERSE FILTER DETAIL
NTS

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. THE VERTICAL DRAIN SHALL BE INSTALLED AS NEEDED BASED ON FIELD CONDITIONS AND AS DIRECTED BY THE GEOTECHNICAL ENGINEER. THE CONTRACTOR SHALL INSTALL OUTLET PIPES FROM THE DRAIN WHERE LOW SPOTS IN THE DRAIN OCCUR.



SPRING BOX DETAIL
NTS



DETAIL C1
NTS

GROUNDWATER CONTROL NOTES

1. DURING THE GEOTECHNICAL INVESTIGATION, ARTESIAN CONDITIONS WERE ENCOUNTERED IN CERTAIN LOCATIONS. SHOULD THE CONTRACTOR ENCOUNTER SPRINGS OR SEEPS IN THE ROCK OR SOIL EXCAVATION, SPOT TREATMENTS SHOULD BE APPLIED. THIS PLAN SET HAS TWO PROPOSED SPOT TREATMENTS FOR SPRINGS AND SEEPS INCLUDED AS DETAILS C AND D. THE SPECIFIC TREATMENT FOR EACH SPRING AND SEEP ENCOUNTERED WILL BE SPECIFIED BY THE GEOTECHNICAL ENGINEER DURING CONSTRUCTION.
2. FOR LOCALIZED SEEPS, A SPRING BOX SHOULD BE INSTALLED BY UNDERCUTTING THE LOCATION OF THE SEEP TO DETERMINE THE SOURCE, INSTALLING A SPRING BOX AND OUTLET PIPE TO THE SLOPE SURFACE, AND BACKFILLING WITH SOIL TO THE PROPOSED FINISHED GRADES.
3. FOR SEEPS AND SPRINGS WHERE THE SOURCE IS UNABLE TO BE LOCATED OR WHERE THE SOURCE IS NOT A POINT SOURCE, A REVERSE GRADED FILTER BLANKET SHOULD BE CONSTRUCTED TO INTERCEPT THE SEEP. THIS SHOULD BE INSTALLED BY UNDERCUTTING THE SLOPE AND PLACING SAND, CRUSHED STONE, AND RIPRAP TO THE PROPOSED DIVERSION CHANNEL SLOPE AS SHOWN ON THE DETAIL.
4. AT THE DIRECTION OF THE GEOTECHNICAL ENGINEER, ADDITIONAL GROUNDWATER CONTROL MEASURES MAY BE REQUIRED BASED ON AS-FOUND CONDITIONS DURING CONSTRUCTION.

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION CHANNEL
GROUNDWATER CONTROL
DETAILS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-390	

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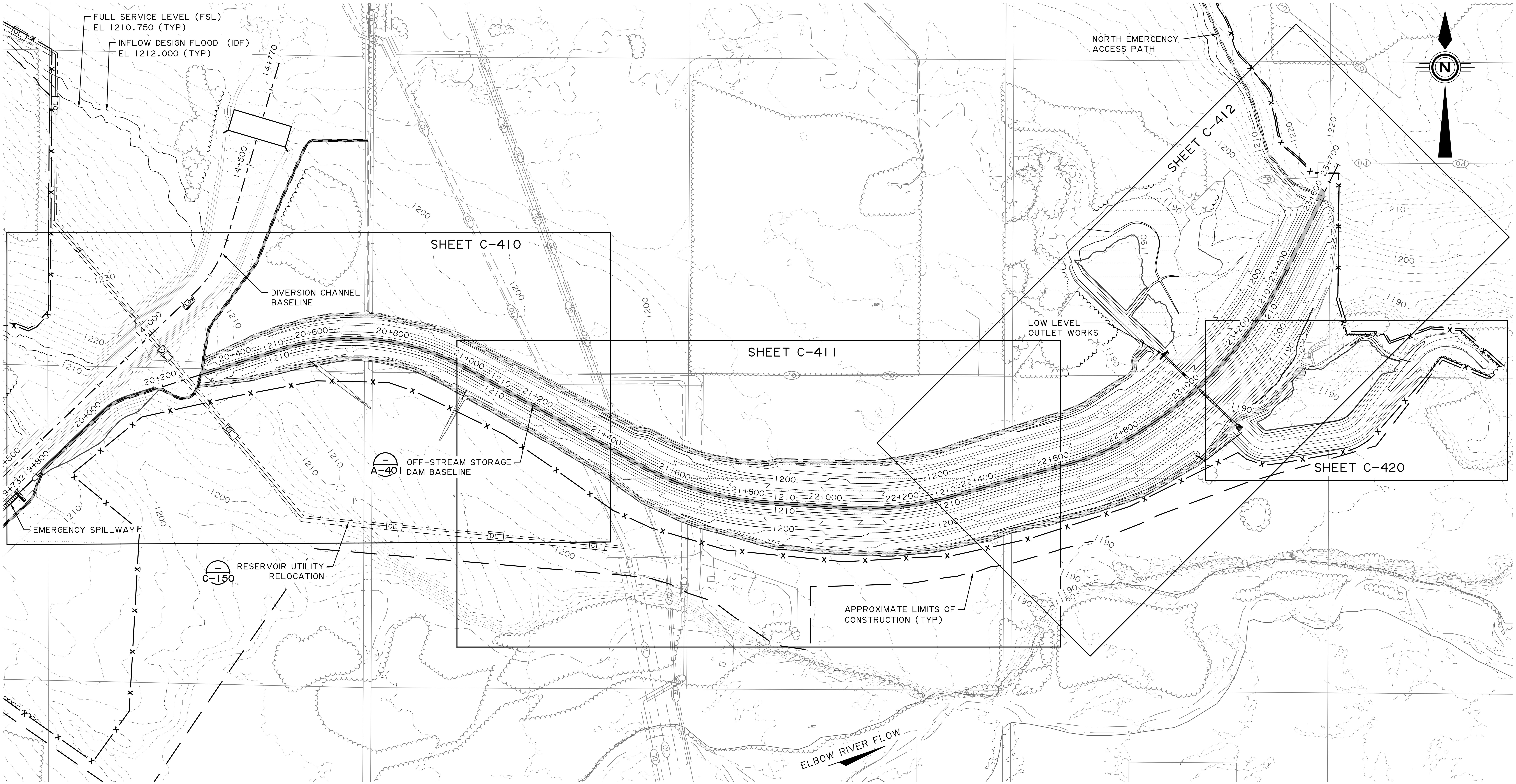

NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:5000

0 200 400
Metres

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SR1

OFF-STREAM STORAGE DAM
OVERVIEW

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-401	

PlotDate: 2019.12.02 4:28 PM Login: Bolubasz, Danielle
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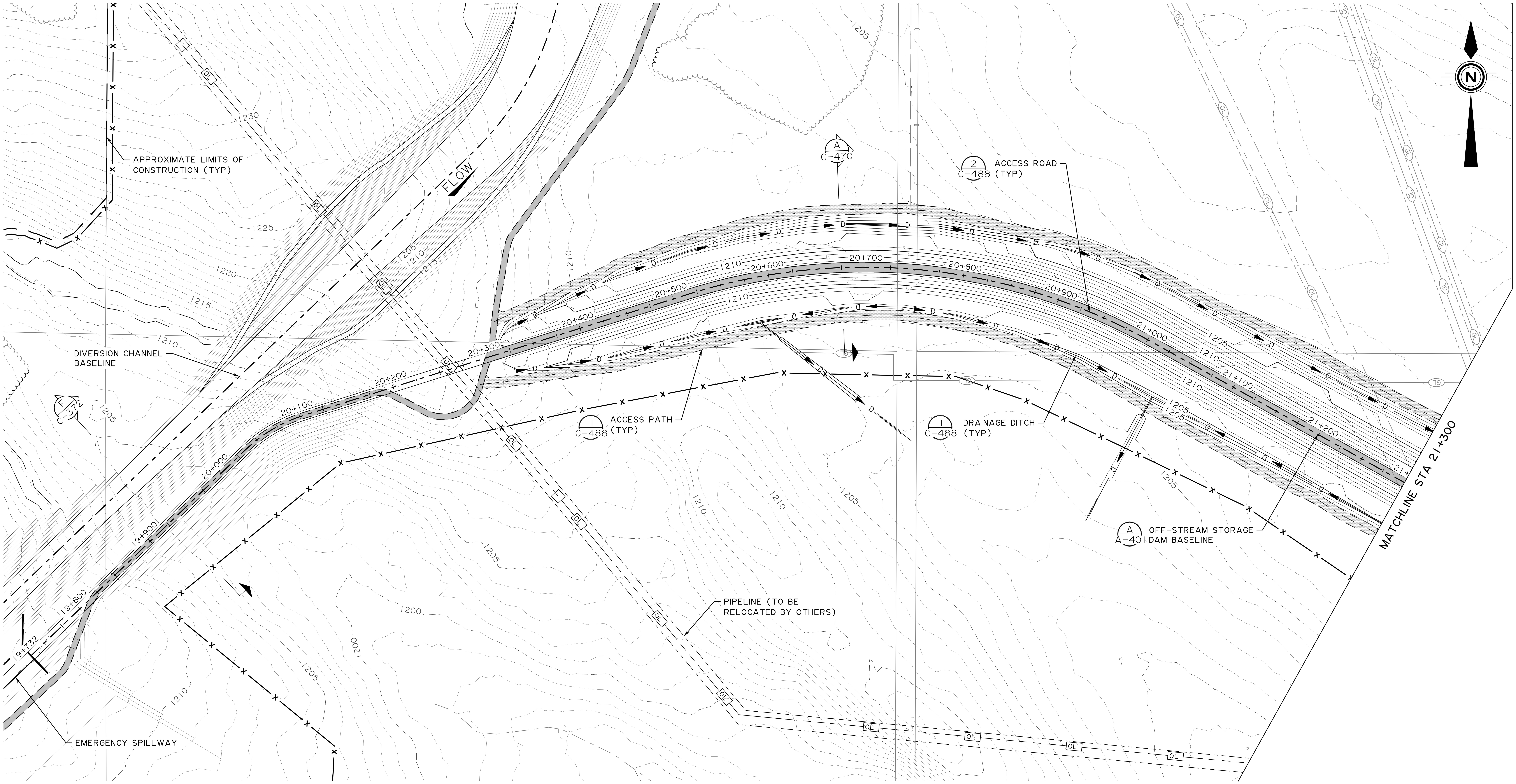

PLAN
1:2000

0 80 160 Metres

NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PlotDate: 2019.12.02 4:35 PM Login: Bolubasz, Danielle TENDER No. ?
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SR1

OFF-STREAM STORAGE DAM
PLAN 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
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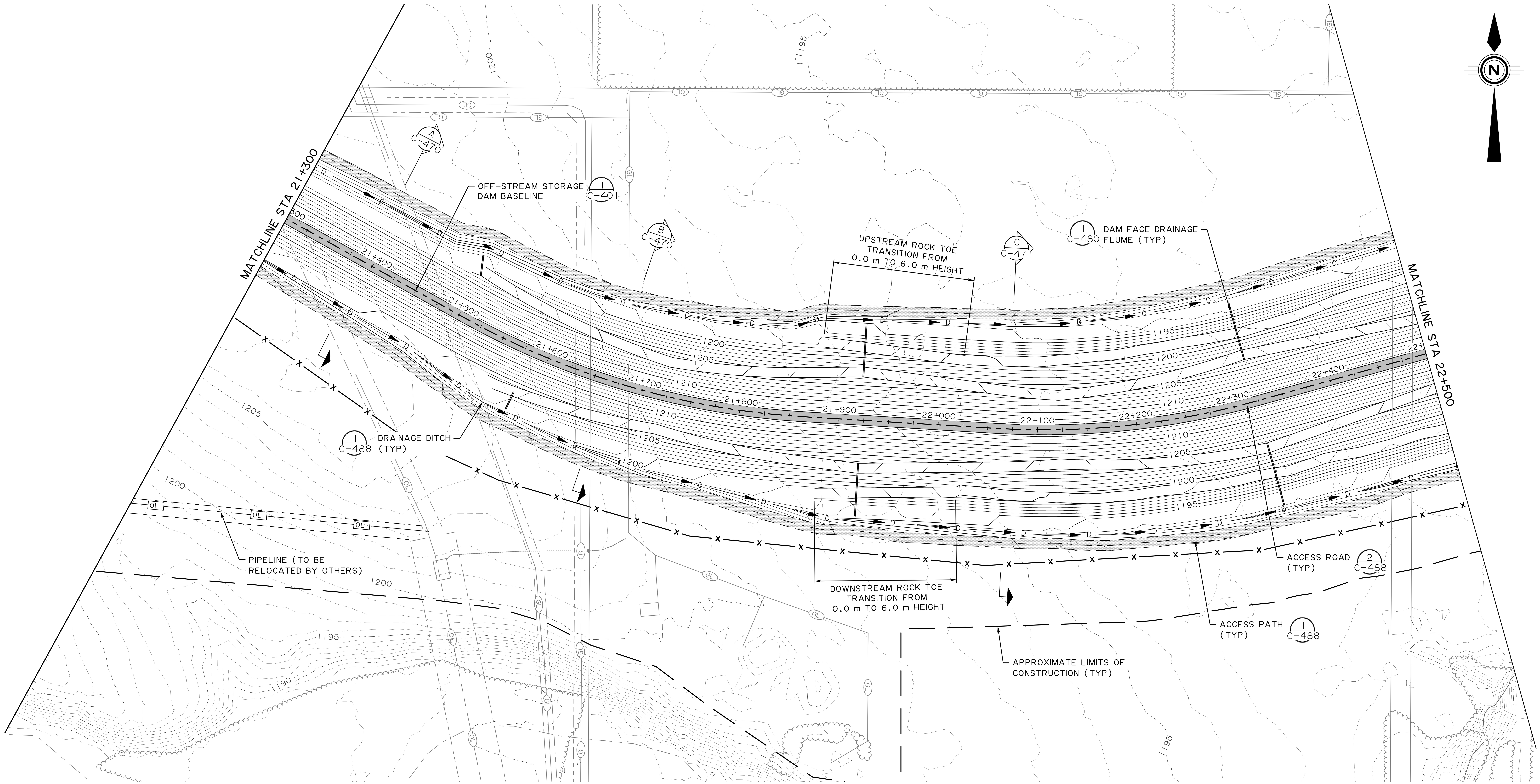

NOTES:

1. CONTOURS DO NOT REFLECT OVERBUILD FOR DAM SETTLEMENT.
2. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

PLAN
1:2000

0 80 160
Metres

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

OFF-STREAM STORAGE DAM PLAN 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-411	

PlotDate: 2019.12.02 4:36 PM Login: Bolubasz, Danielle
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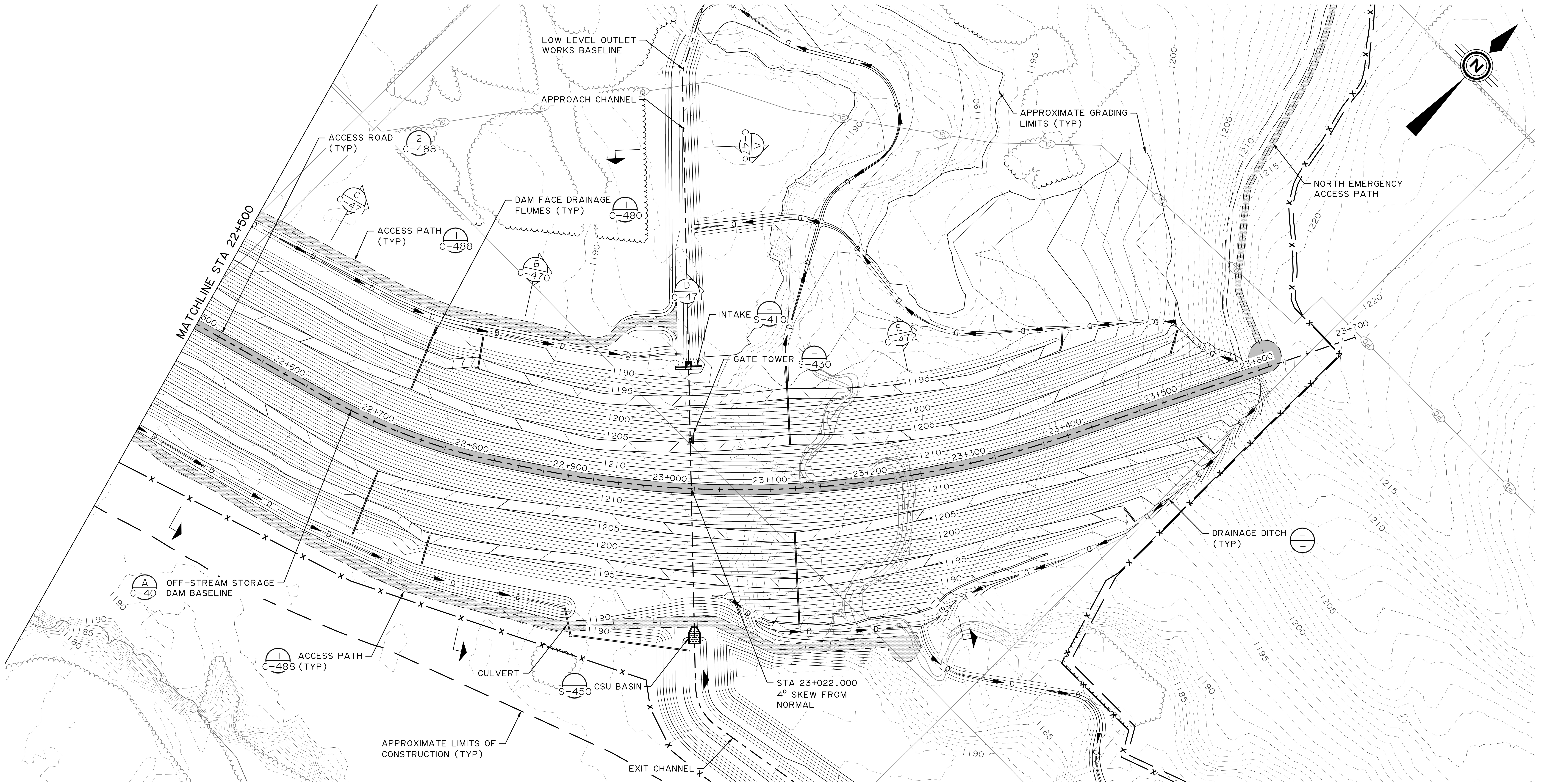

PLAN
1:2000

0 80 160 Metres

NOTE:
1. DIMENSIONS IN METRES UNLESS OTHERWISE NOTED.

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
PLAN 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-412	

PlotDate: 2019.12.02 4:37 PM Login: Bolubasz, Danielle TENDER No. ?
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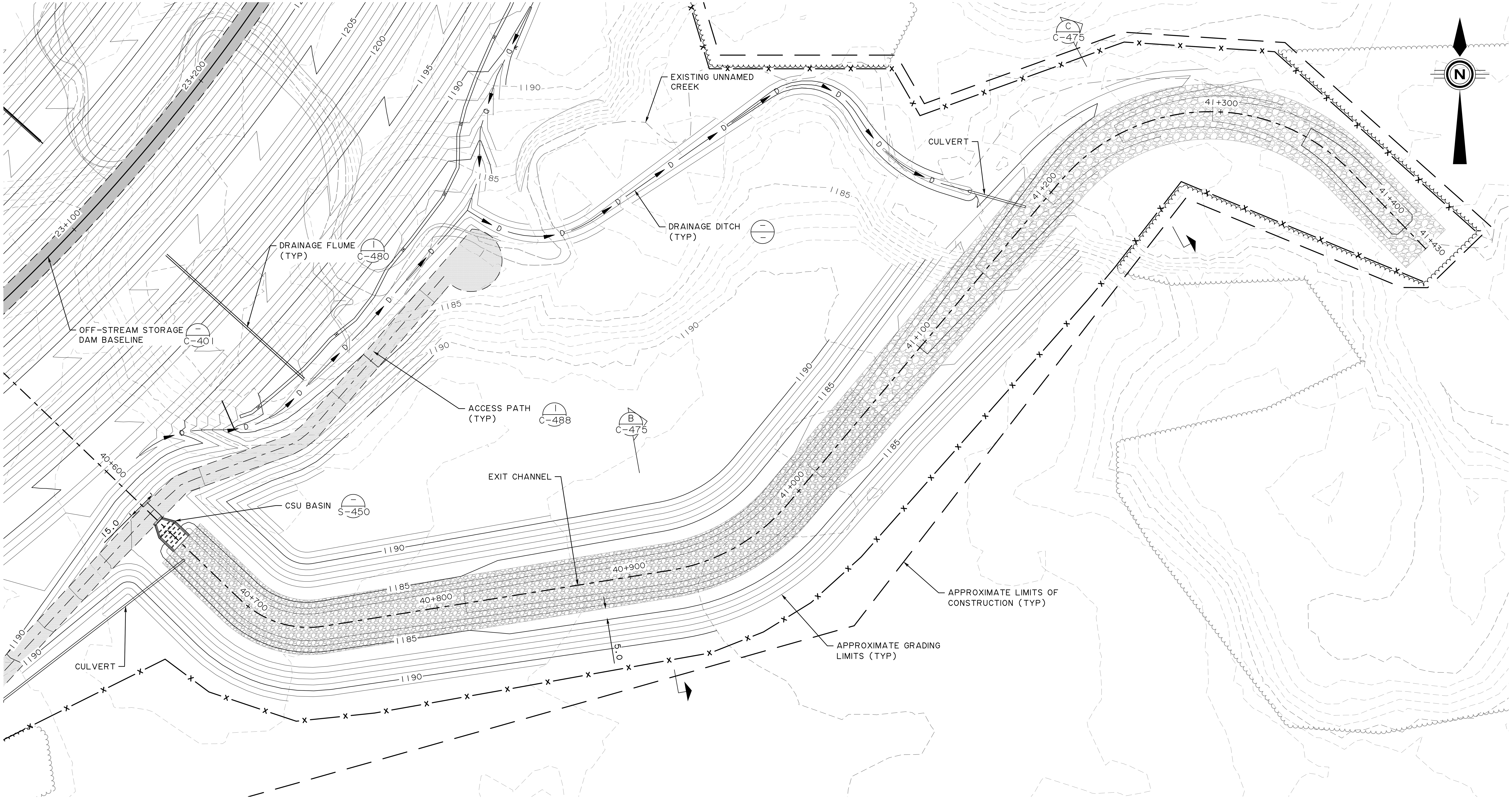
PlotDate: 2019.12.02 4:40 PM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\400_ossd\73396C-419.dwg TENDER No. ?



PLAN
1:1000

0 40 80 Meters

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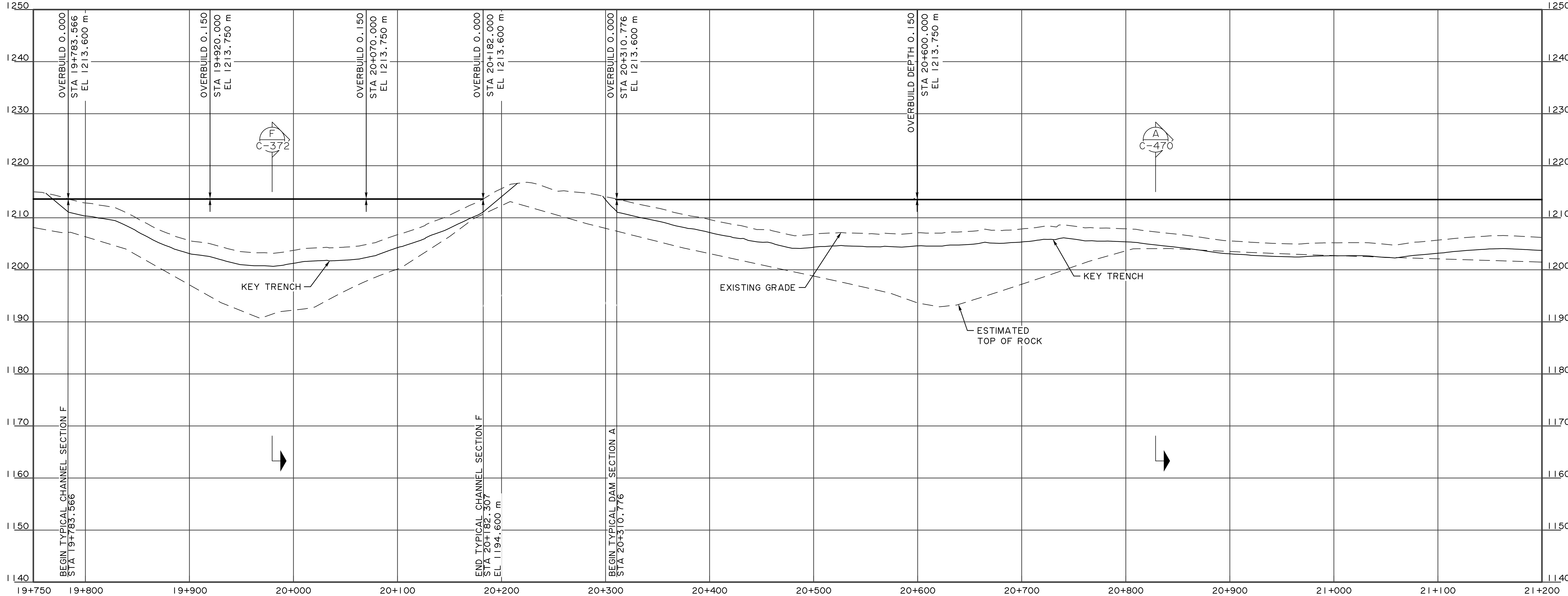
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
LOW LEVEL OUTLET WORKS
EXIT CHANNEL PLAN

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-420	



PROFILE
H: 1:2000 V: 1:400

NOTES:
1. TOP OF ROCK LINE SHOWN WAS ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS.
2. THE OFF-STREAM STORAGE DAM EMBANKMENT WILL EXPERIENCE SETTLEMENT AND INCORPORATES AN OVERBUILD CONDITION DURING CONSTRUCTION. ELEVATIONS FOR OVERBUILD AND FINAL GRADE ARE SHOWN. USE STRAIGHT LINE GRADES TO INTERPOLATE BETWEEN SPECIFICALLY IDENTIFIED OVERBUILD LOCATIONS.

PlotDate: 2019.12.03 10:06 AM Login: Bolubasz, Danielle
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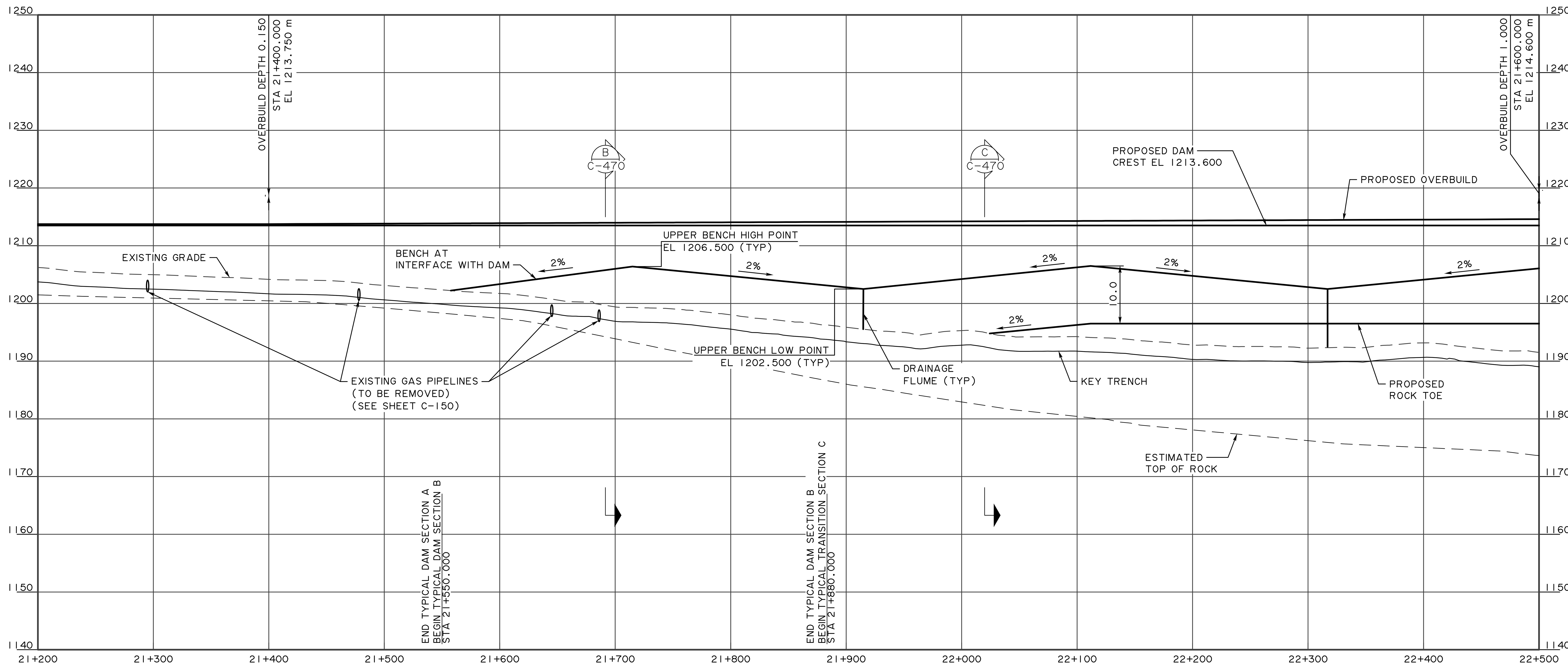
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
PROFILE 1
STA 19+750 TO 21+200

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-430	



PROFILE
H: 1:2000 V: 1:400

NOTES:
1. TOP OF ROCK LINE SHOWN WAS ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS.
2. THE OFF-STREAM STORAGE DAM EMBANKMENT WILL EXPERIENCE SETTLEMENT AND INCORPORATES AN OVERBUILD CONDITION DURING CONSTRUCTION. ELEVATIONS FOR OVERBUILD AND FINAL GRADE ARE SHOWN. USE STRAIGHT LINE GRADES TO INTERPOLATE BETWEEN SPECIFICALLY IDENTIFIED OVERBUILD LOCATIONS.

PlotDate: 2019.12.03 10:07 AM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\400_ossd\73396C-430.dwg TENDER No. ?

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

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DESIGNER
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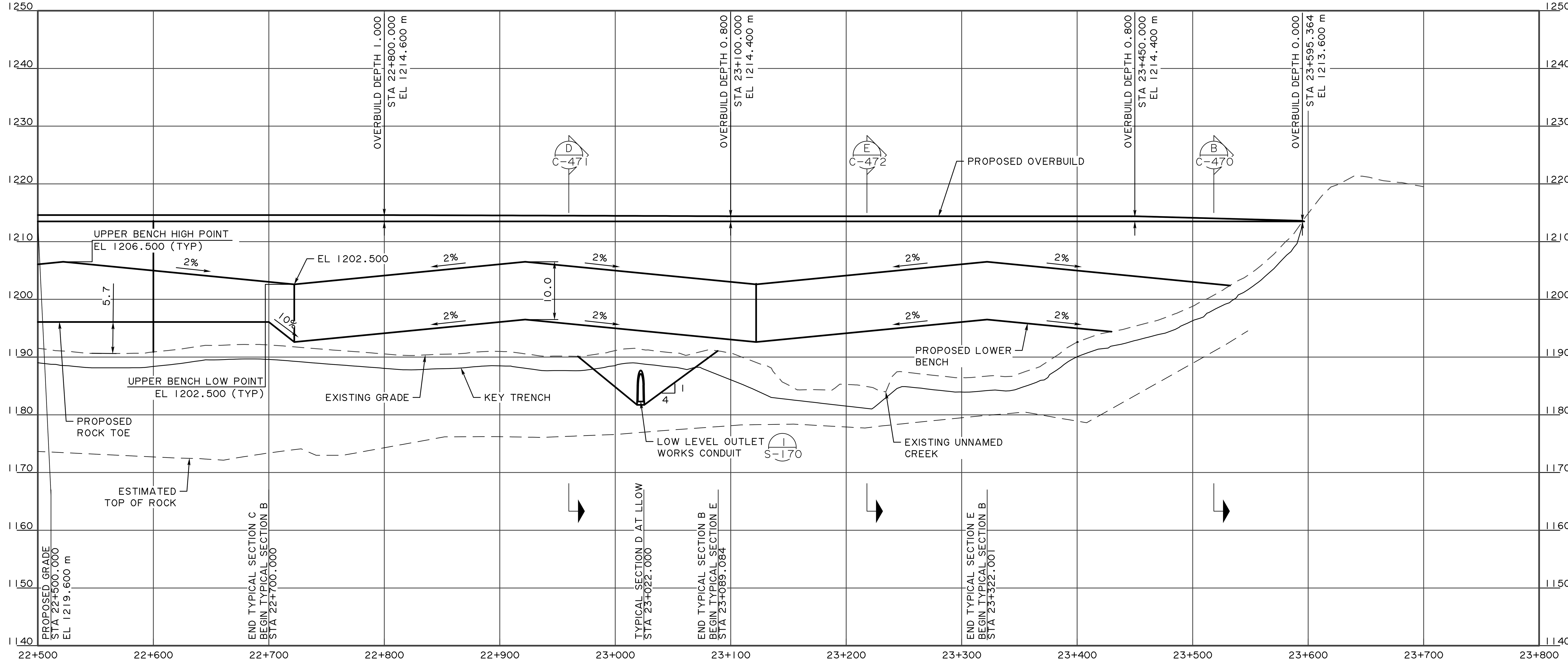
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
PROFILE 2
STA 21+200 TO 22+500

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-431	



PROFILE
H: 1:2000 V: 1:400

NOTES:
1. TOP OF ROCK LINE SHOWN WAS ESTABLISHED BY STRAIGHT LINE INTERPOLATION BETWEEN BORINGS.
2. THE OFF-STREAM STORAGE DAM EMBANKMENT WILL EXPERIENCE SETTLEMENT AND INCORPORATES AN OVERBUILD CONDITION DURING CONSTRUCTION. ELEVATIONS FOR OVERBUILD AND FINAL GRADE ARE SHOWN. USE STRAIGHT LINE GRADES TO INTERPOLATE BETWEEN SPECIFICALLY IDENTIFIED OVERBUILD LOCATIONS.
3. 4:1 EXCAVATION SLOPE SHOWN FOR LOW-LEVEL OUTLET WORKS TO DEMONSTRATE BACKFILL REQUIREMENTS. CONTRACTOR TO DETERMINE REQUIRED EXCAVATION.

PlotDate: 2019.12.03 10:07 AM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\400_ossd\73396C-430.dwg TENDER No. ?

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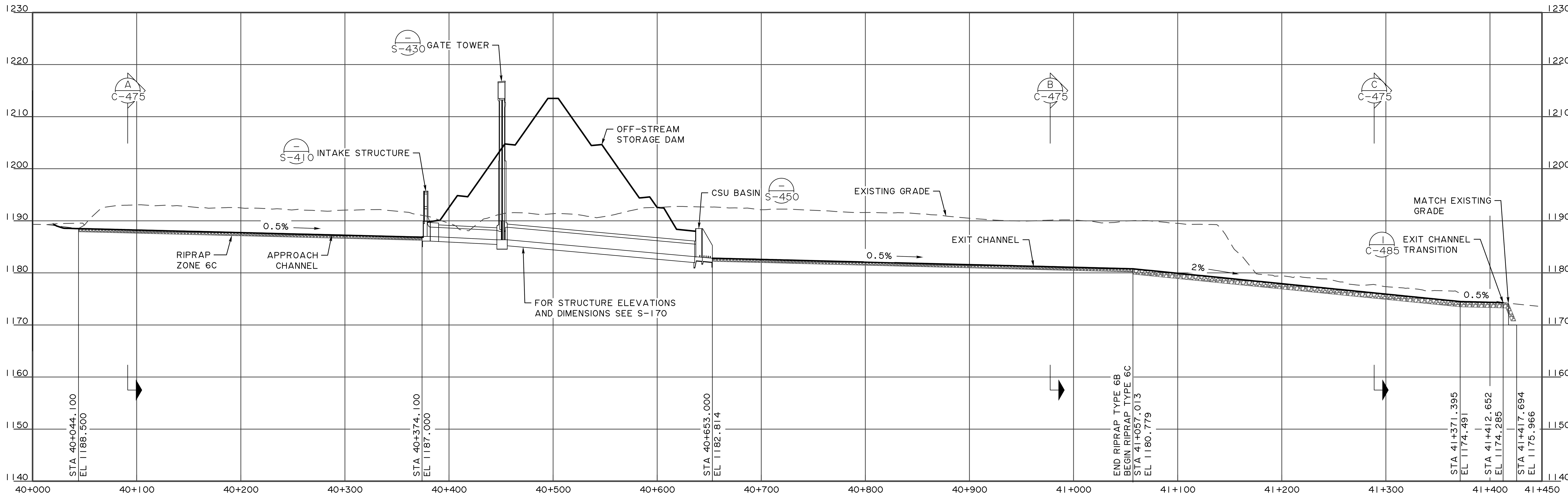
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
PROFILE 3
STA 22+500 TO 23+800

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-432	



PROFILE
H: 1:2000 V: 1:400

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
LOW LEVEL OUTLET WORKS
PROFILE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-435	

PlotDate: 2019.12.03 11:42 AM Login: Bolubasz, Danielle
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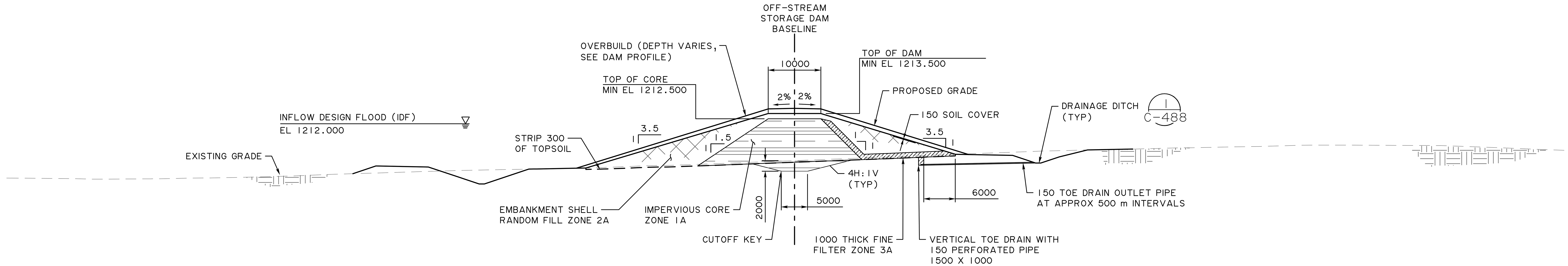
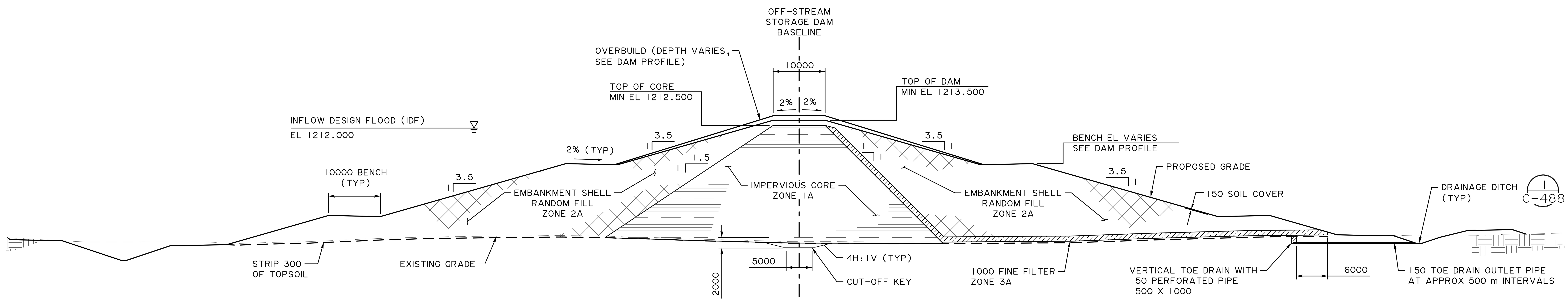


A STORAGE DAM – TYPICAL SECTION

NO BENCH NTS



B STORAGE DAM – TYPICAL SECTION

BENCHES NTS

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

TYPICAL SECTIONS 1

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-470	

PlotDate: 2019.12.02 5:07 PM Login: Bolubasz, Danielle TENDER No. ?
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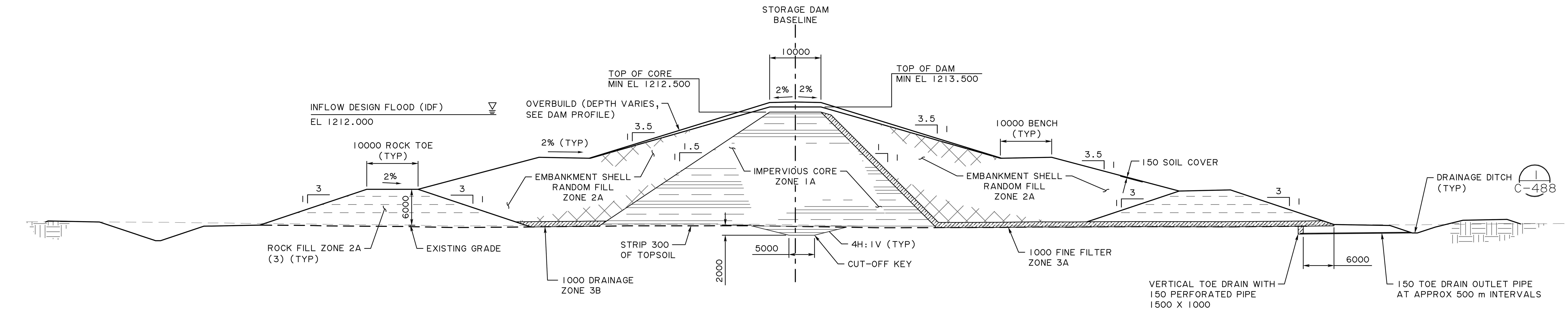
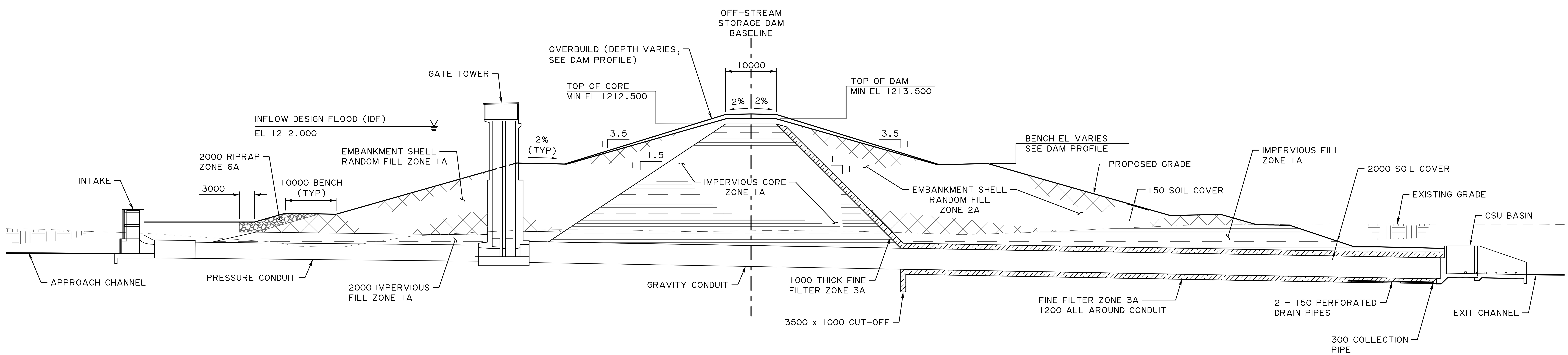


C
—
STORAGE DAM – TYPICAL SECTION
ROCK TOE NTS



D
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STORAGE DAM – TYPICAL SECTION
LOW-LEVEL OUTLET WORKS NTS

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

PlotDate: 2019.12.03 10:43 AM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CADwater\c_civilsheet_files\400_ossd\73396C-470.dwg TENDER No. ?

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Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

TYPICAL SECTIONS 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
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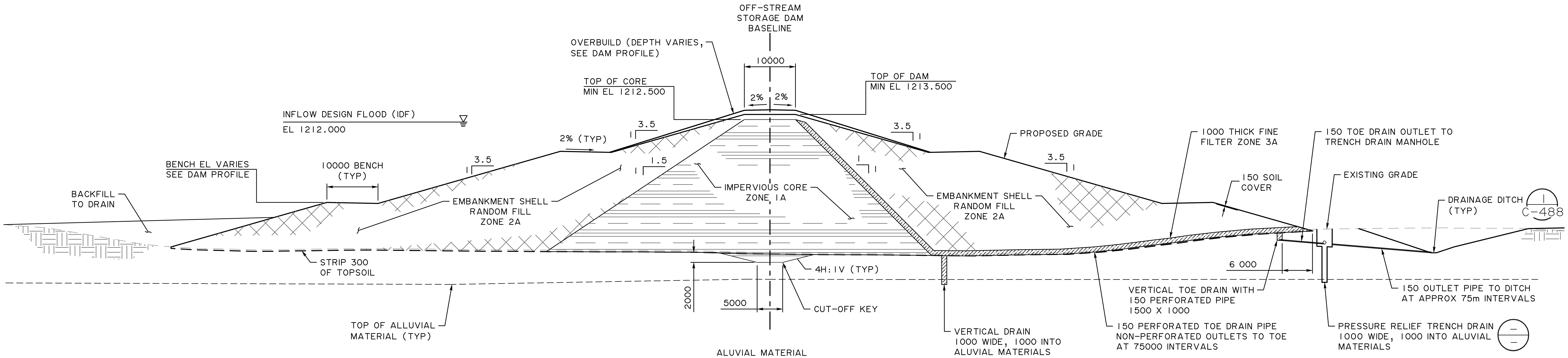

E
–
STORAGE DAM -TYPICAL SECTION
UNNAMED CREEK VALLEY
NTS

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

PlotDate: 2019.12.02 5:08 PM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\400_ossd\73396C-470.dwg TENDER No. ?

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

TYPICAL SECTIONS 3

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
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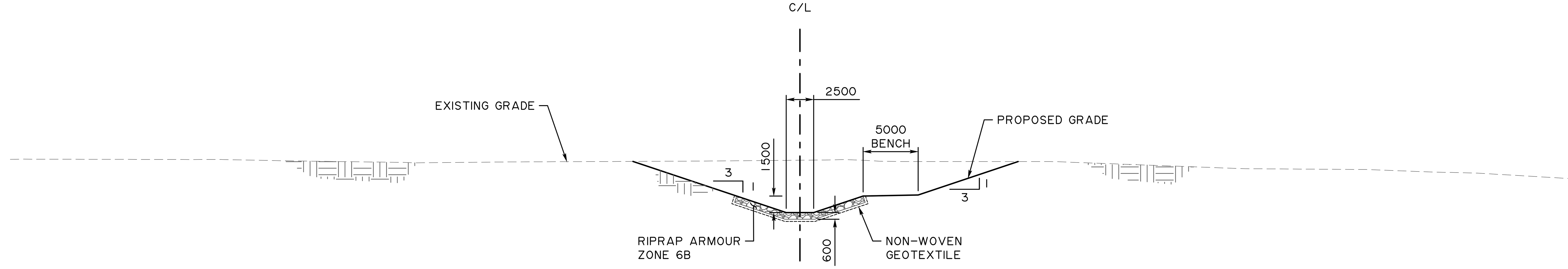
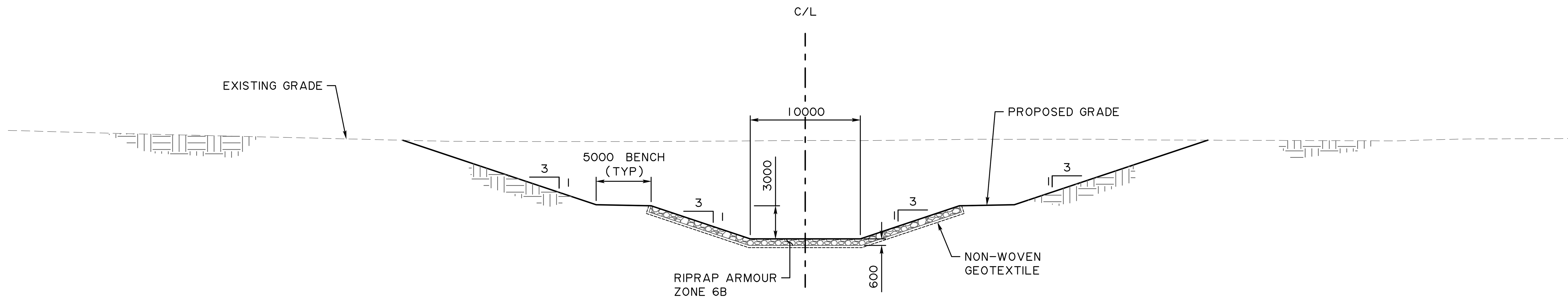
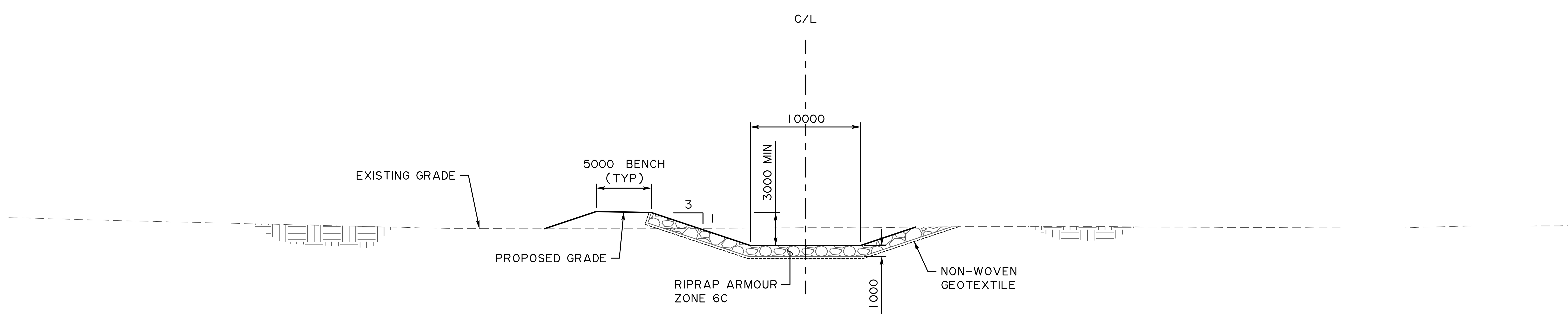


A TYPICAL SECTION - LOW-LEVEL OUTLET WORKS
INTAKE CHANNEL NTS



B TYPICAL SECTION - LOW-LEVEL OUTLET WORKS
EXIT CHANNEL NTS



C TYPICAL SECTION - LOW-LEVEL OUTLET WORKS
EXIT CHANNEL NTS

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
LOW LEVEL OUTLET WORKS
TYPICAL SECTIONS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-475	

PlotDate: 2019.12.03 8:15 AM Login: Bolubasz, Danielle
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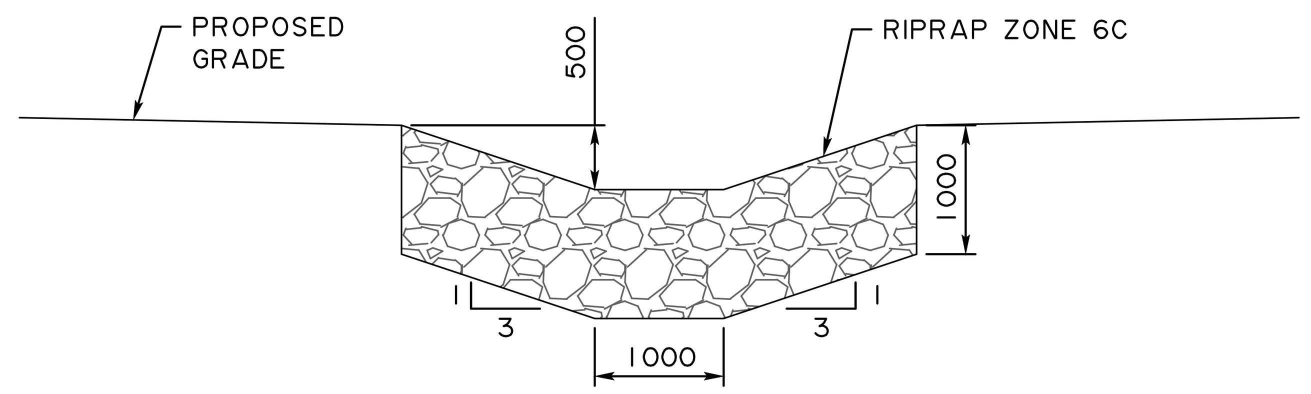
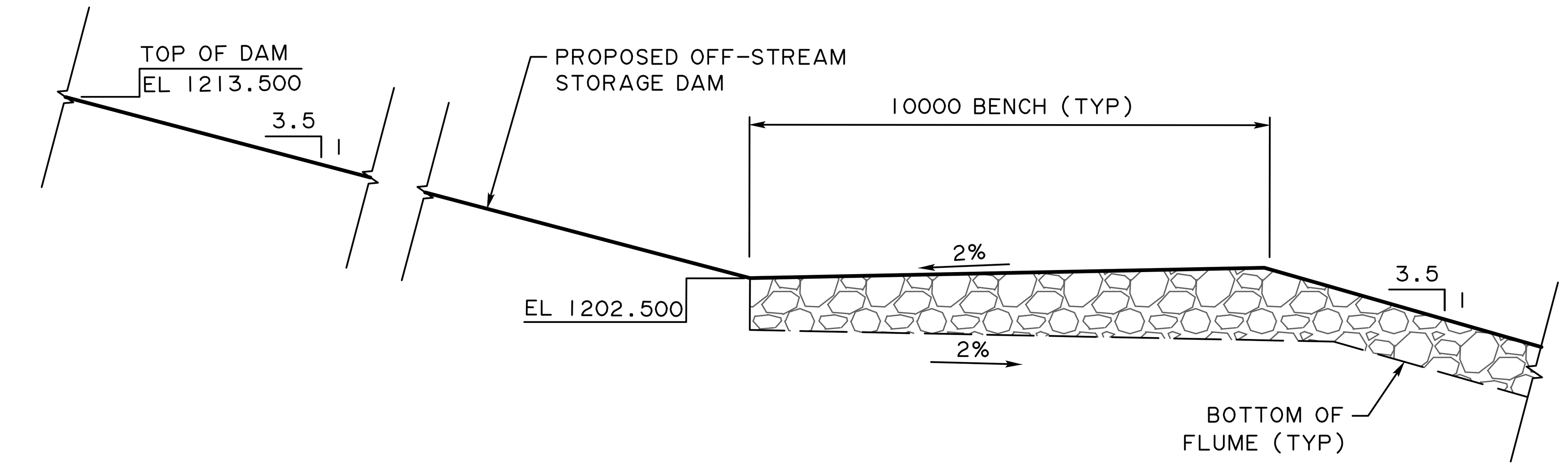


DAM DRAINAGE FLUME TYPICAL SECTION

1
C-411
NTS



DAM DRAINAGE FLUME TYPICAL PROFILE

2
C-411
NTS

PlotDate: 2019.12.03 8:20 AM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CAD\water\c_civil\sheet_files\400_ossd\73396C-480.dwg TENDER No. ?

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DESIGNER
DATE

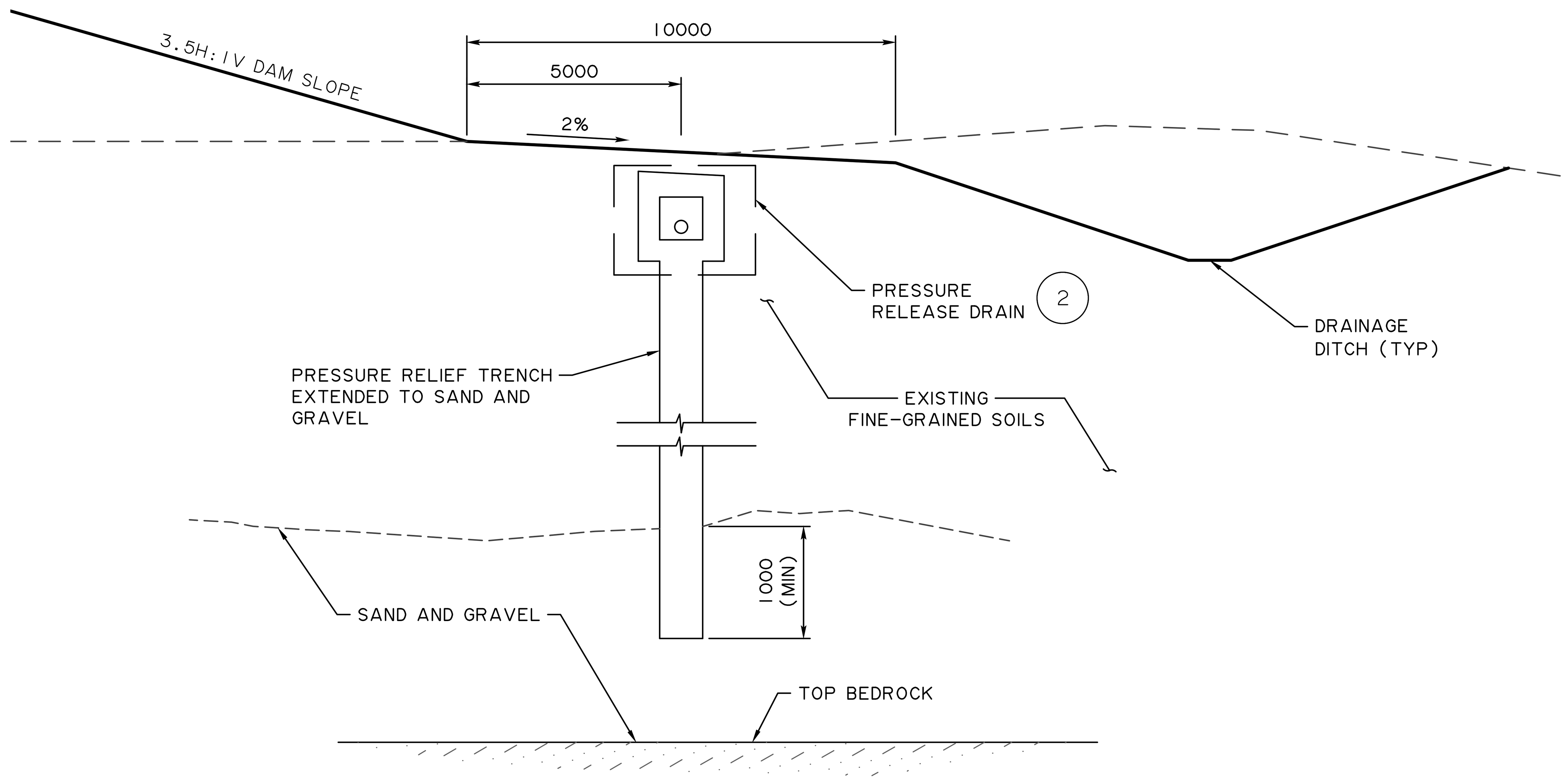
CHECKER
DATE

Alberta Transportation

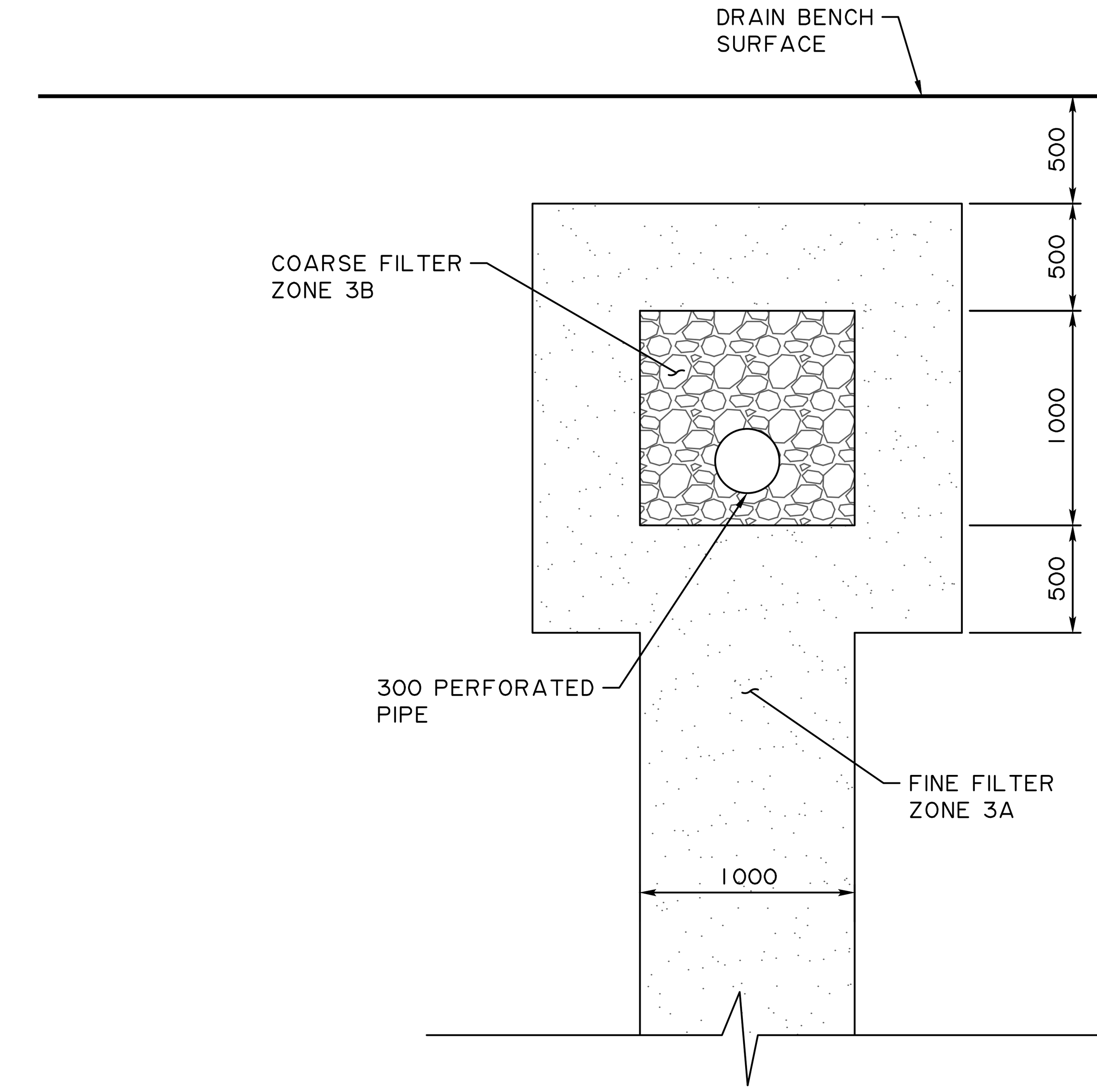
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
EMBANKMENT SURFACE DRAINAGE DETAILS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-480	



1 PRESSURE RELEASE TRENCH DRAIN
C-412 NTS



2 PRESSURE RELEASE DRAIN
C-412 NTS

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

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DATE

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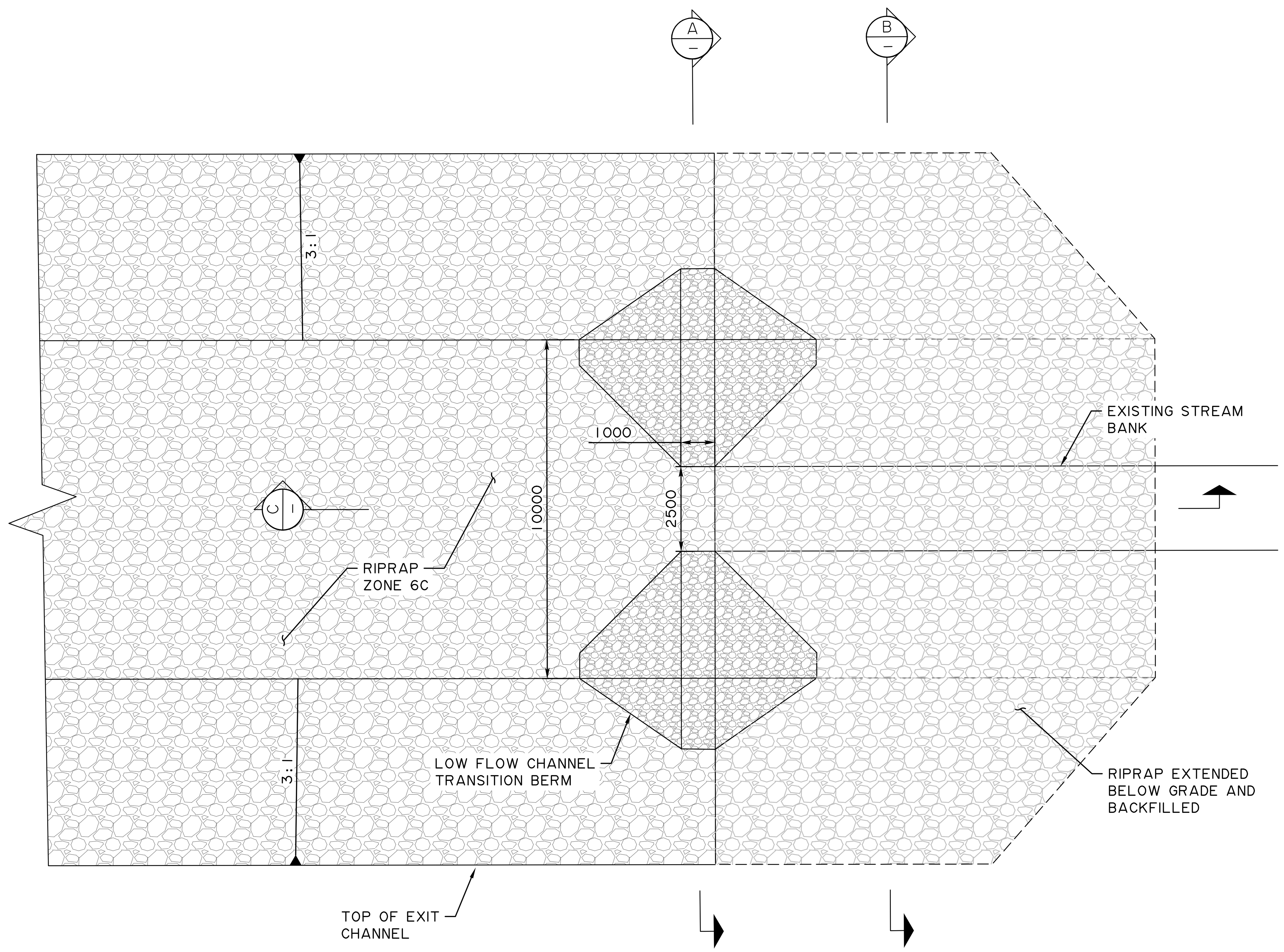
Alberta Transportation

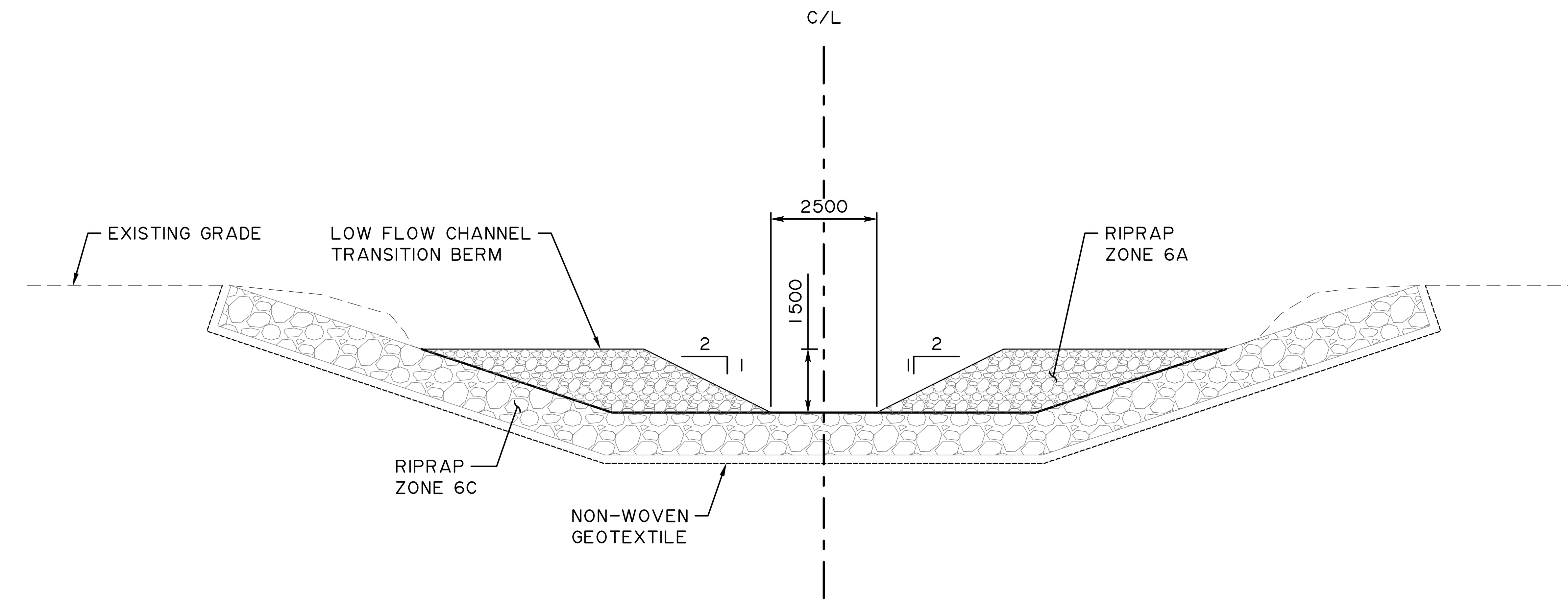
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
PRESSURE RELIEF TRENCH DRAIN
DETAILS

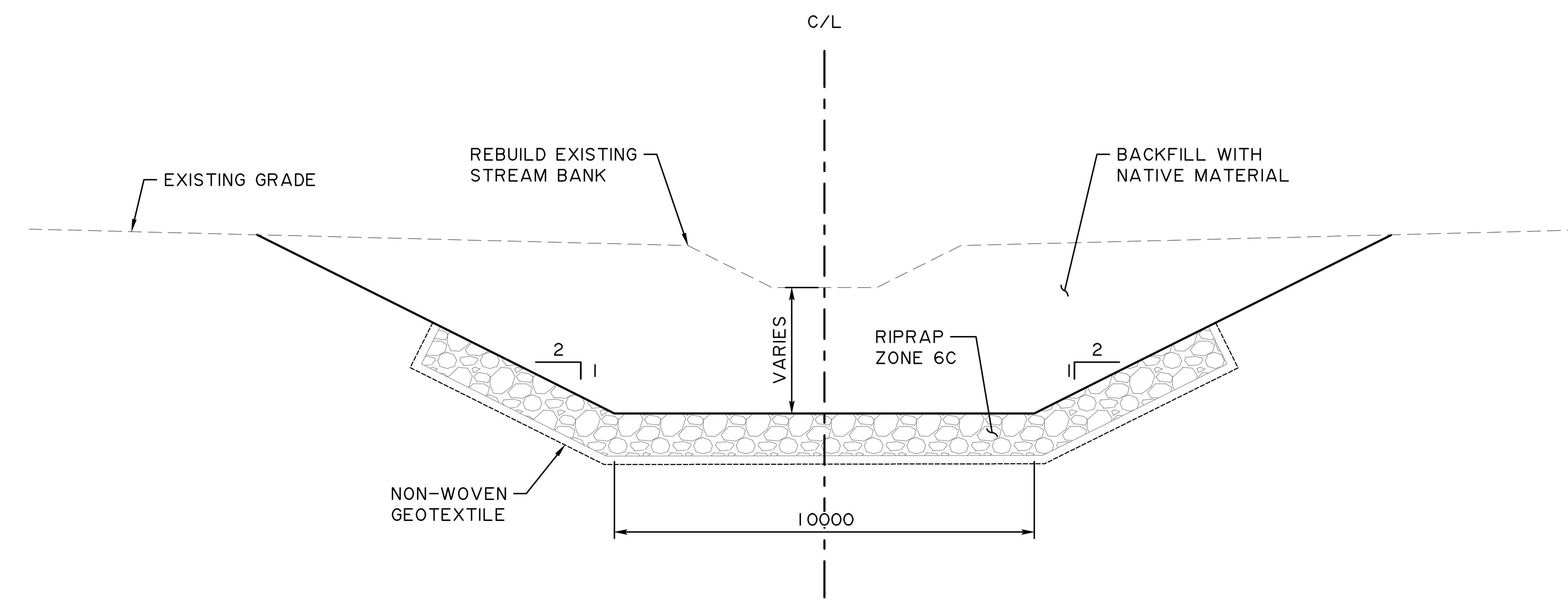
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-482	



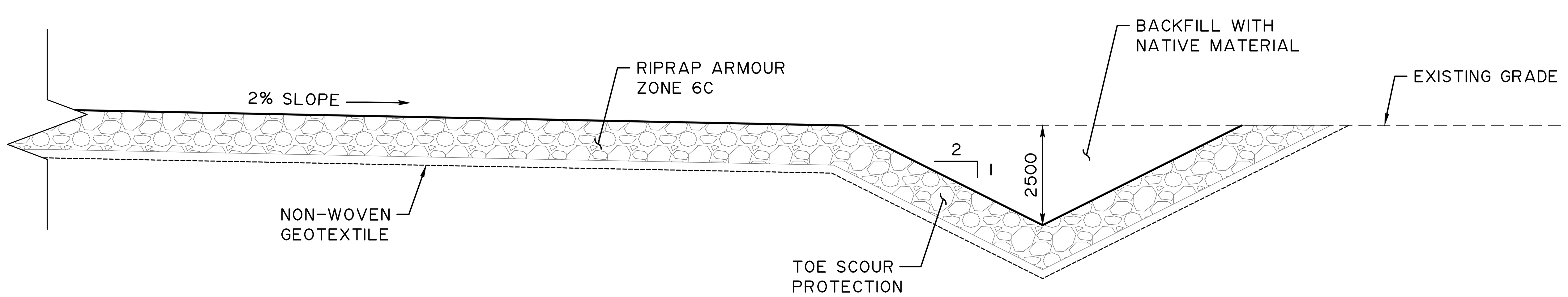
1 DETAIL - EXIT CHANNEL TRANSITION
C-435 1:100



A SECTION
EXIT CHANNEL TRANSITION 1:100



B SECTION
EXIT CHANNEL TRANSITION 1:100



C SECTION
EXIT CHANNEL TRANSITION 1:100

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
LOW LEVEL OUTLET WORKS
DETAILS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-485	

PlotDate: 2019.12.03 11:47 AM Login: Bolubasz, Danielle
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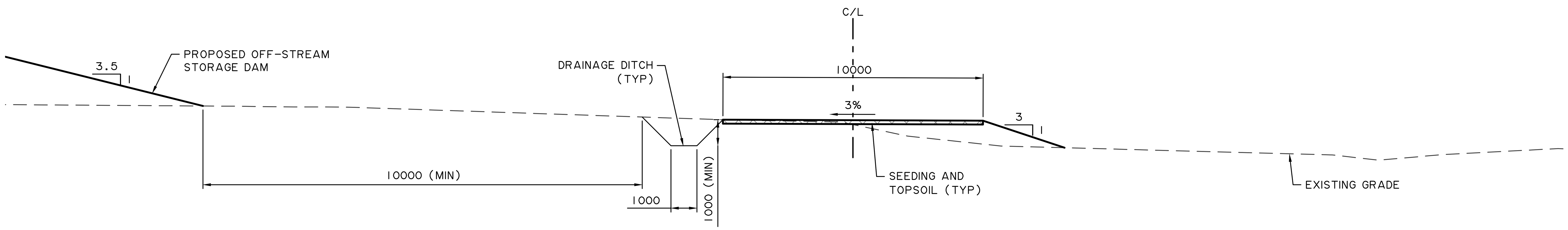
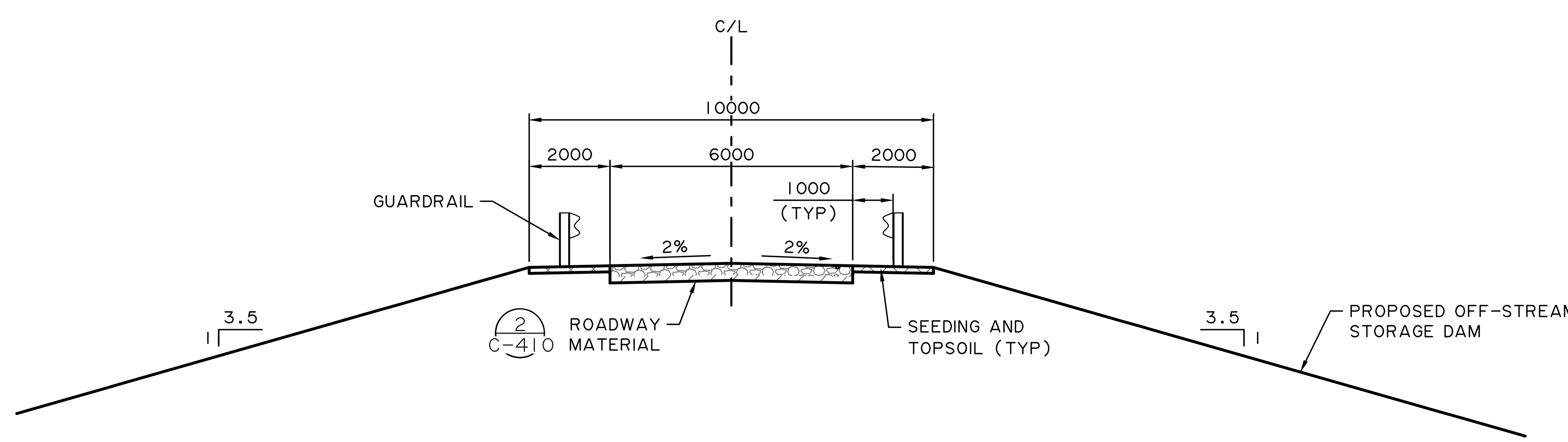


1 C-410 ACCESS PATH TYPICAL SECTION 10:1



2 C-410 ACCESS ROAD TYPICAL SECTION 10:1

NOTE:
1. DIMENSIONS IN MILLIMETRES UNLESS OTHERWISE NOTED.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF-STREAM STORAGE DAM
ACCESS ROAD
DETAILS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396C-488	

PlotDate: 2019.12.03 8:39 AM Login: Bolubasz, Danielle
AUTOCAD FILE No. U:\110773396\CADwater\c_civil\sheet_files\400_ossd\73396C-488.dwg TENDER No. ?

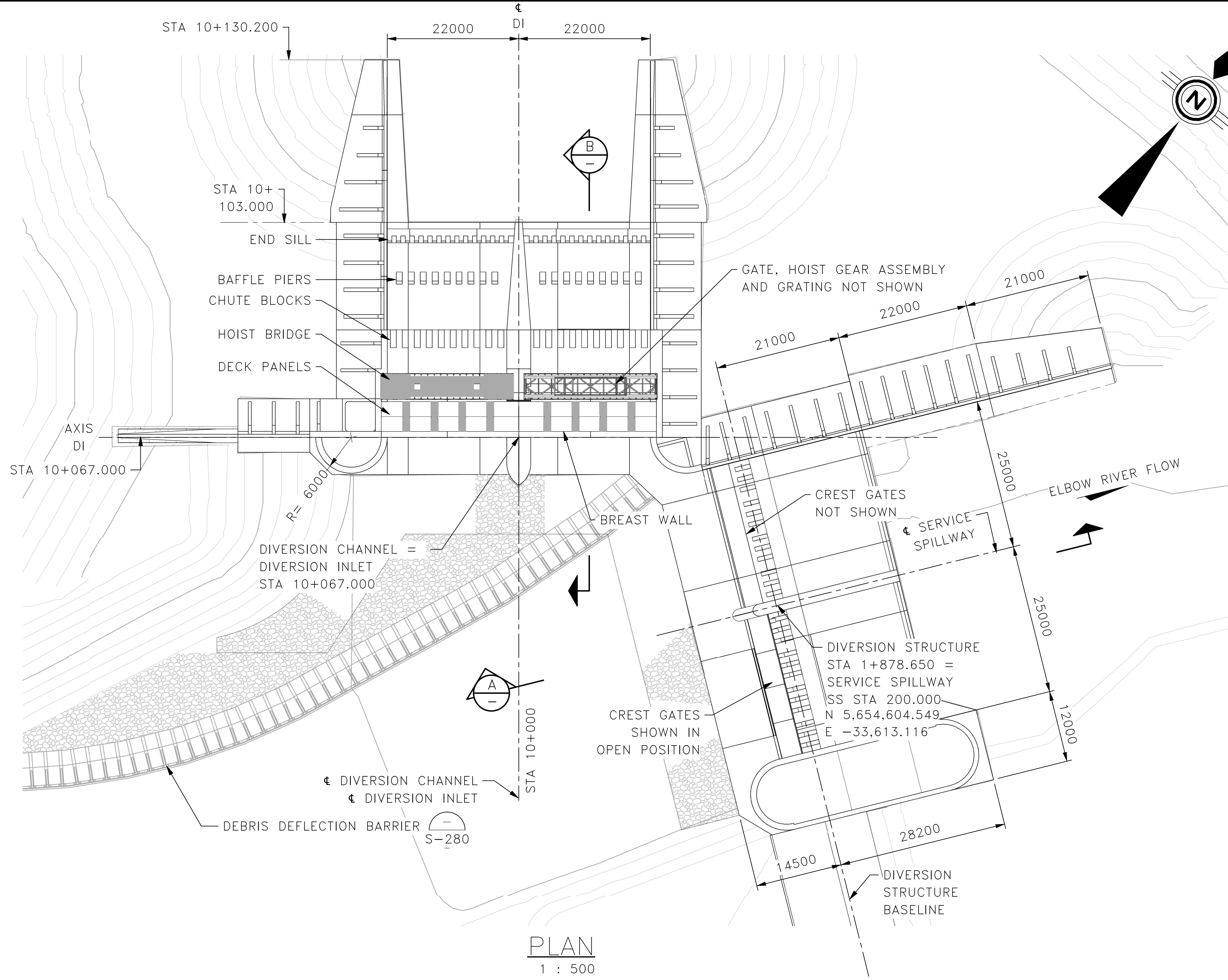
NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
3. CONTOUR INTERVALS ARE IN ONE METRE INCREMENTS.

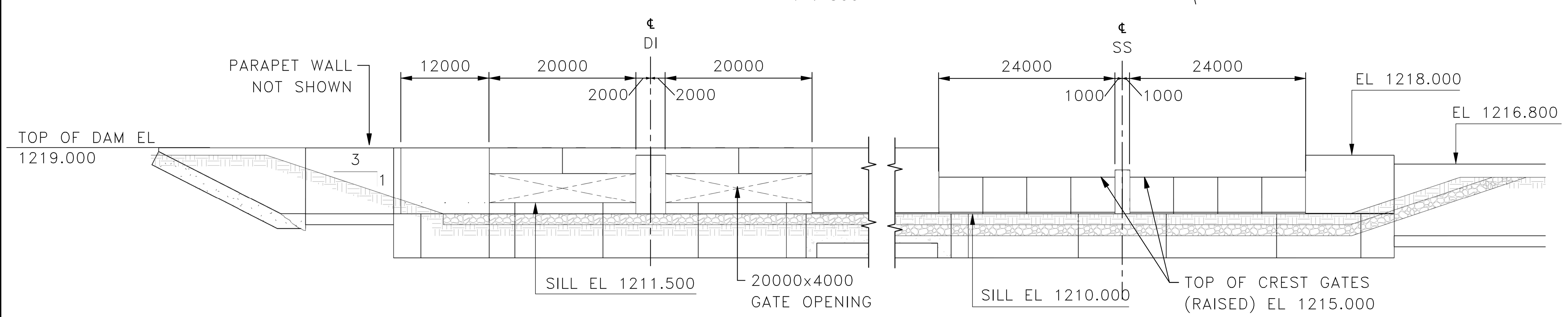
REFERENCE DRAWINGS:

SERVICE SPILLWAY PLAN - / S-200

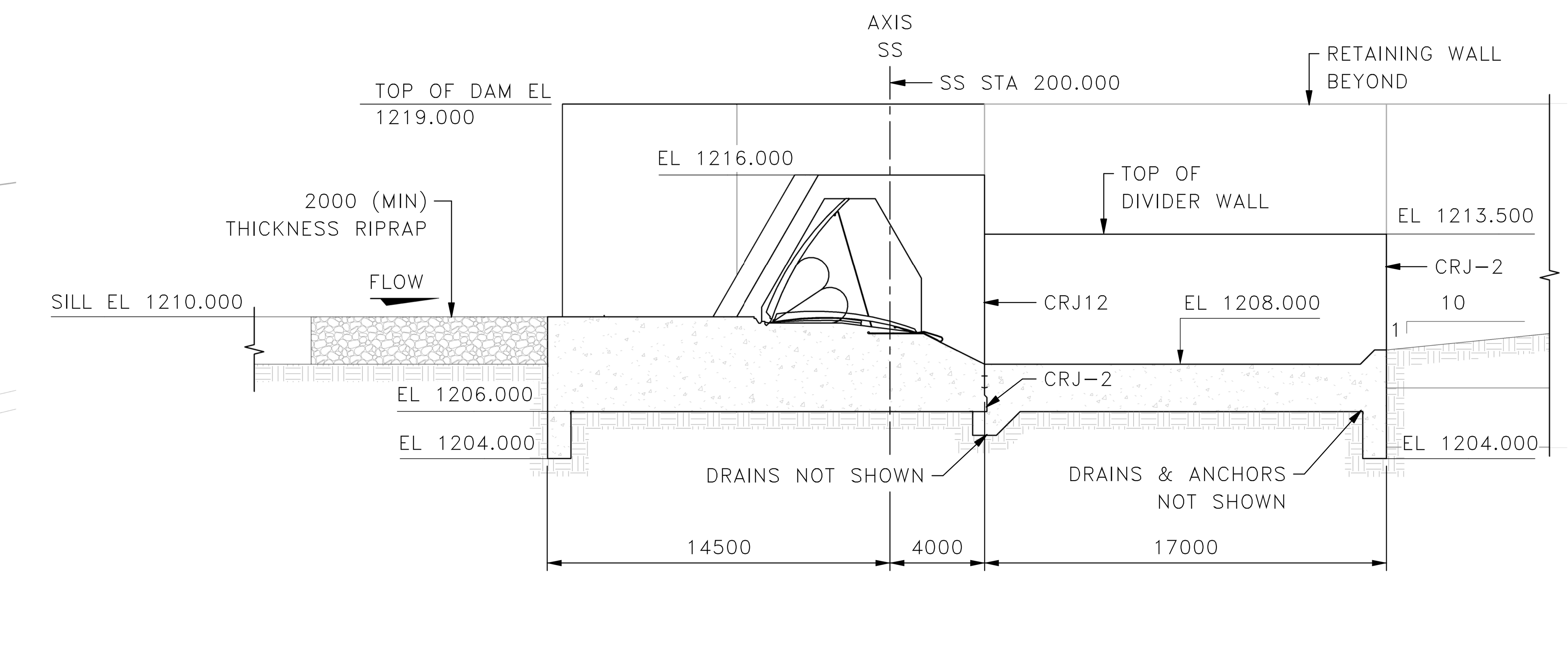
DIVERSION INLET PLAN - / S-300



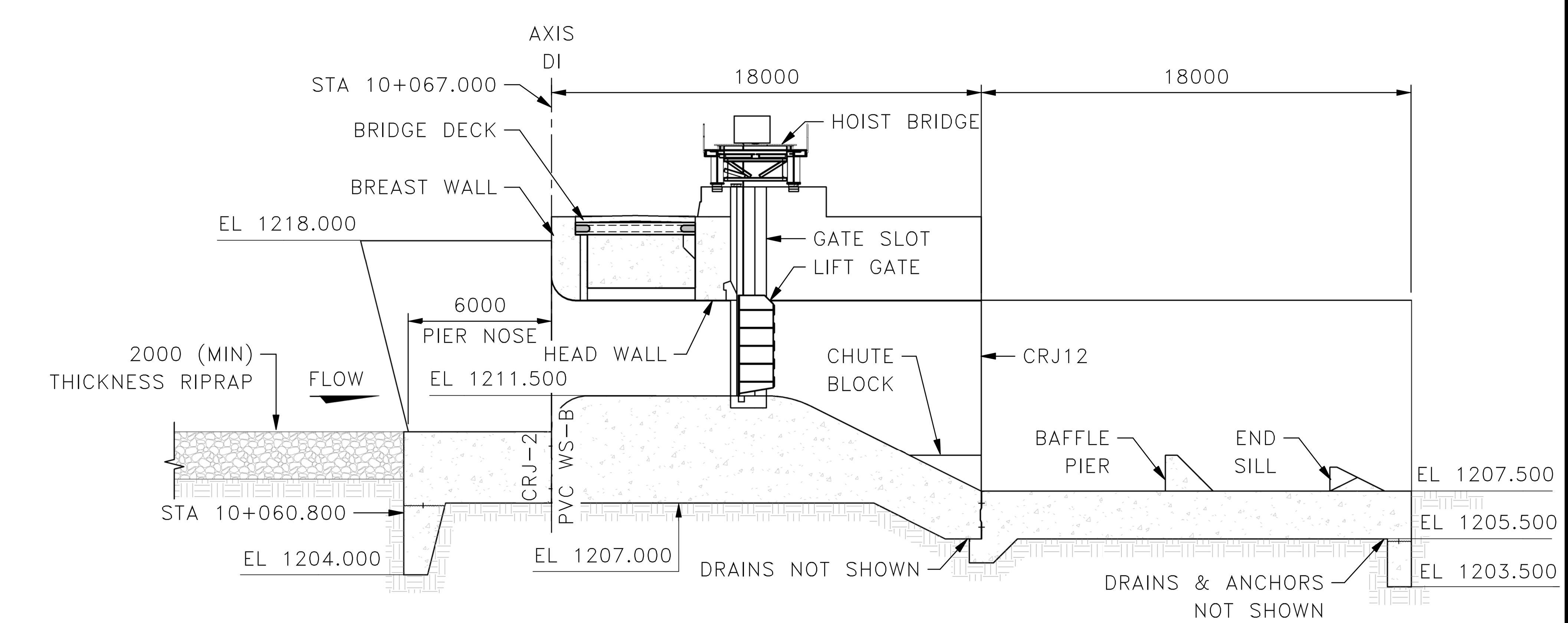
PLAN
1 : 500



ELEVATION – UPSTREAM
1 : 500



A / - SECTION – SERVICE SPILLWAY
1:200



B / - SECTION – DIVERSION INLET
1:200

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION STRUCTURE SERVICE SPILLWAY AND DIVERSION INLET
GENERAL ARRANGEMENT

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-150	

Plot Date: 12/3/2019 3:47:07 PM Login: Dosler
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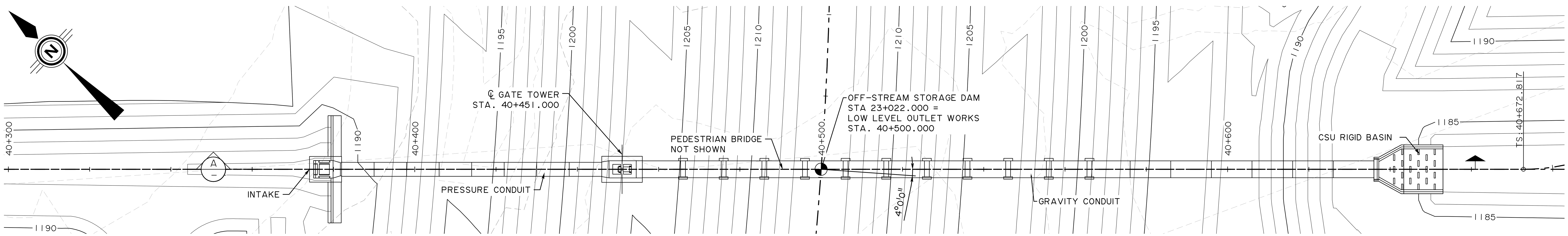
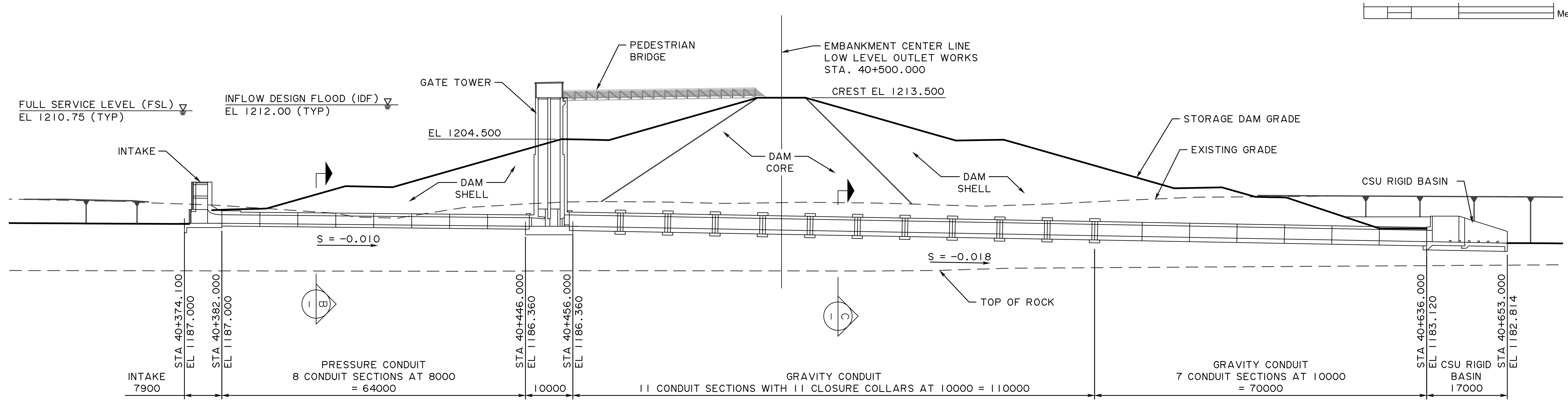
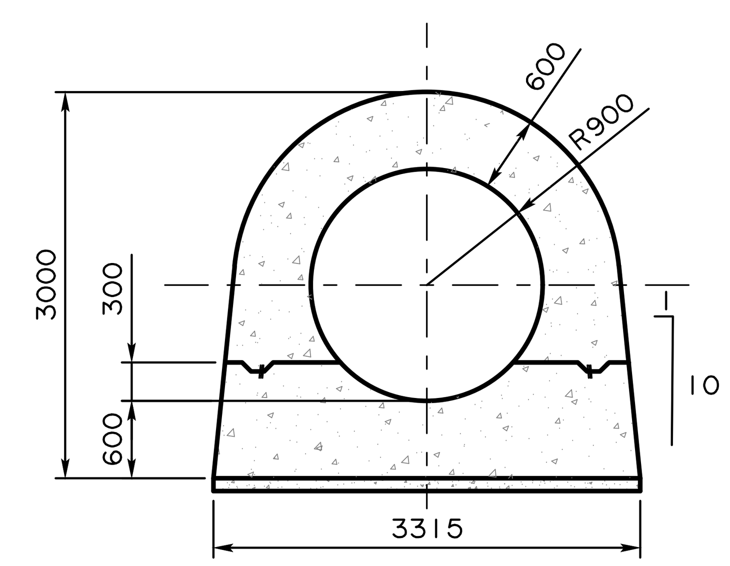


PLAN
1:500

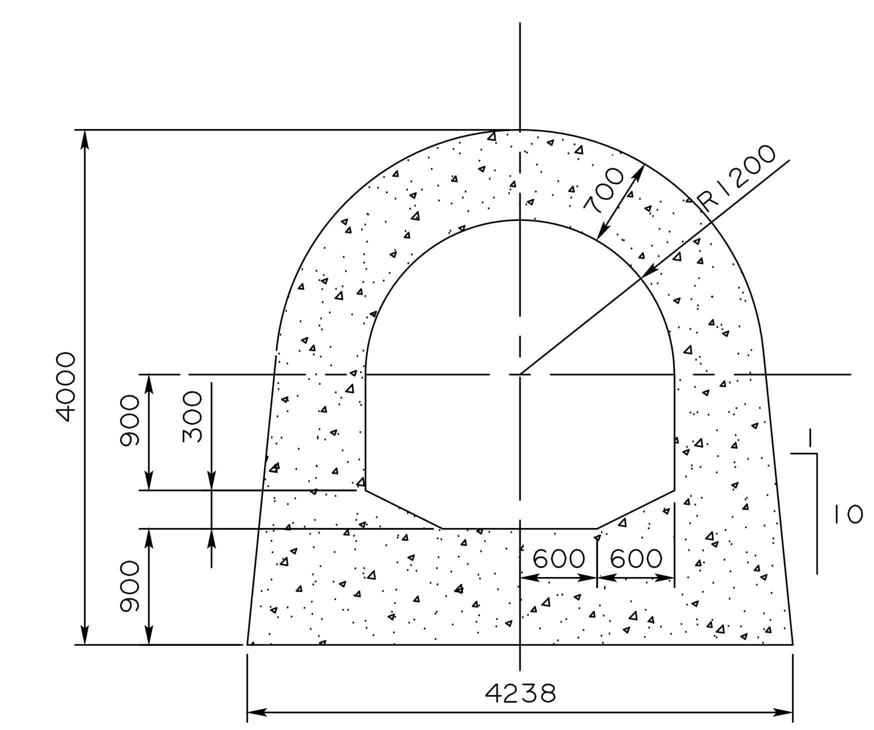
0 20 40 Meters



A SECTION ALONG CENTERLINE
1:500



B SECTION – PRESSURE CONDUIT
1:50



C SECTION – GRAVITY CONDUIT
1:50

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

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

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PRELIMINARY DESIGN

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Alberta Transportation

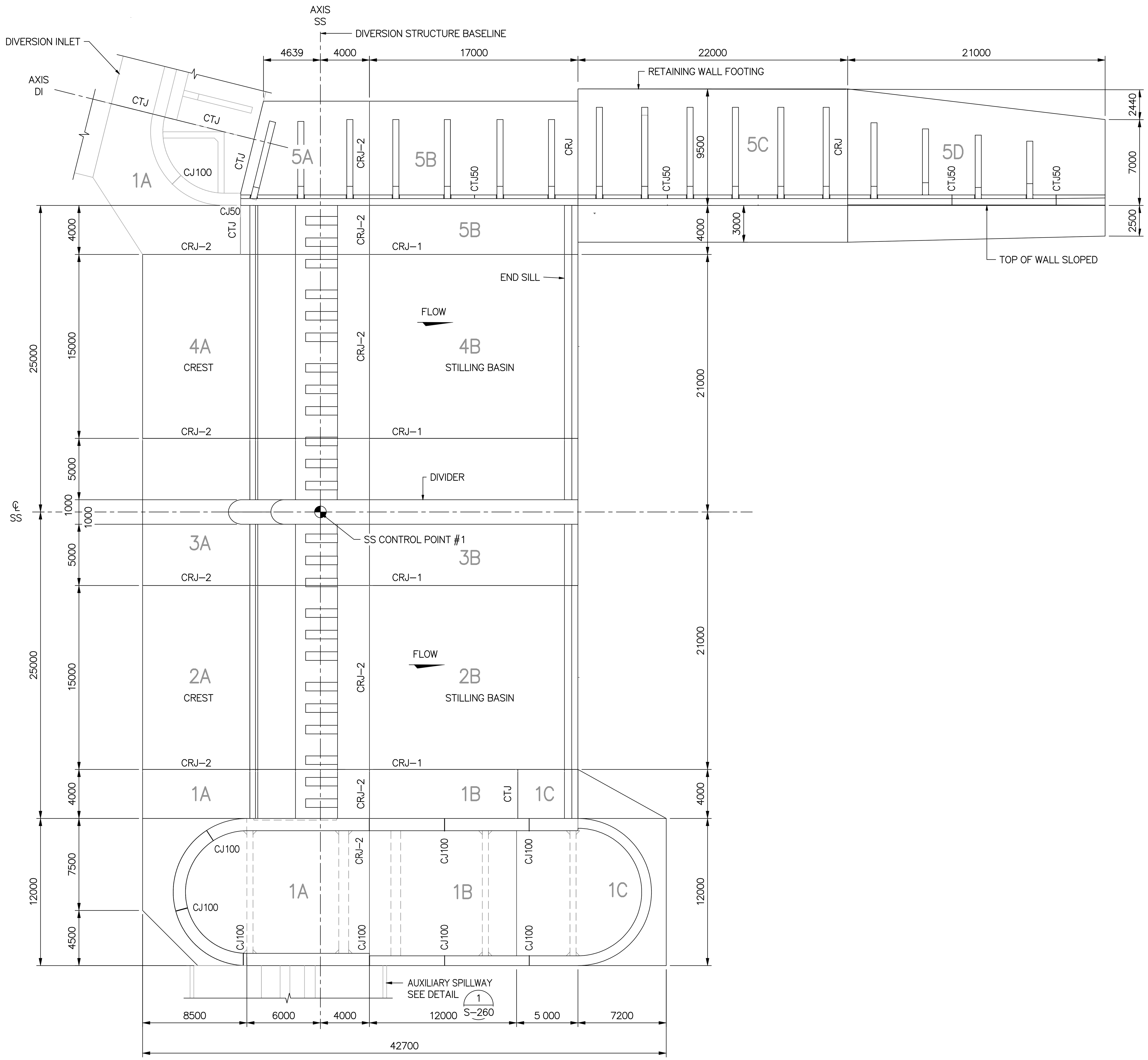
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF STREAM STORAGE DAM
LOW LEVEL OUTLET WORKS
GENERAL ARRANGEMENT

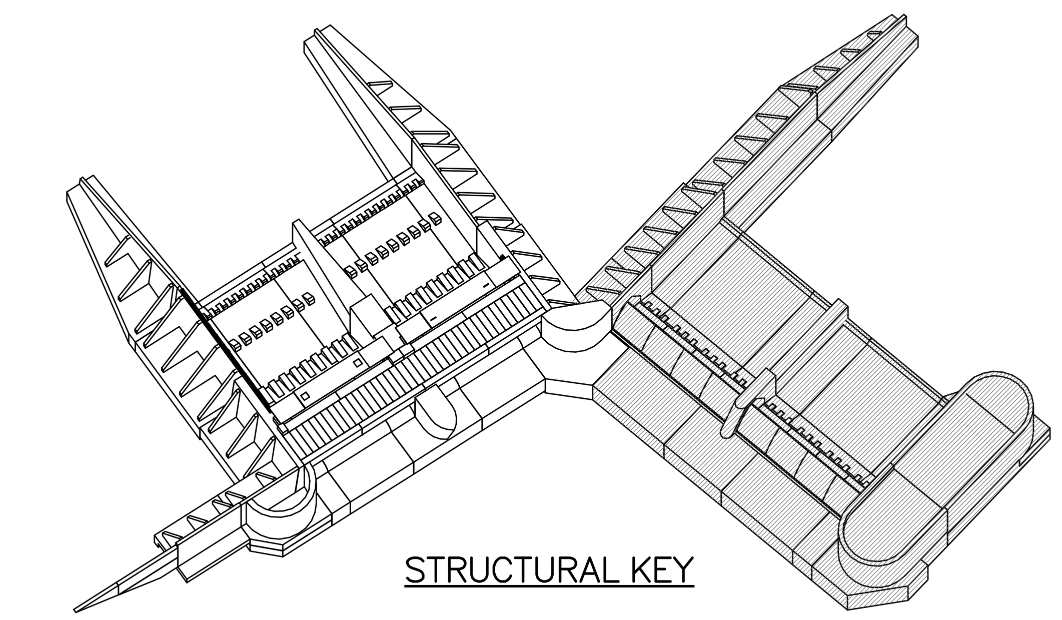
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-170	

TENDER No. ?

PlotDate: 2019.12.03 2:54 PM Login: Clinkenbeard, Adam
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\100_gen\100_gen_lexington\S-170_llow\73396S-170.dwg



PLAN
1 : 200



STRUCTURAL KEY

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
3. SERVICE SPILLWAY CONTROL POINT #1:
   NORTHING 5654604.725
   EASTING –33613.160

   DIVERSION STRUCTURE BASELINE STA 1+878.650 =
   SERVICE SPILLWAY STA 200.000

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PRELIMINARY DESIGN

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DESIGNER
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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
STRUCTURAL ARRANGEMENT

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-200	

Plot Date: 6/25/2020 2:37:56 PM Login: Dosler
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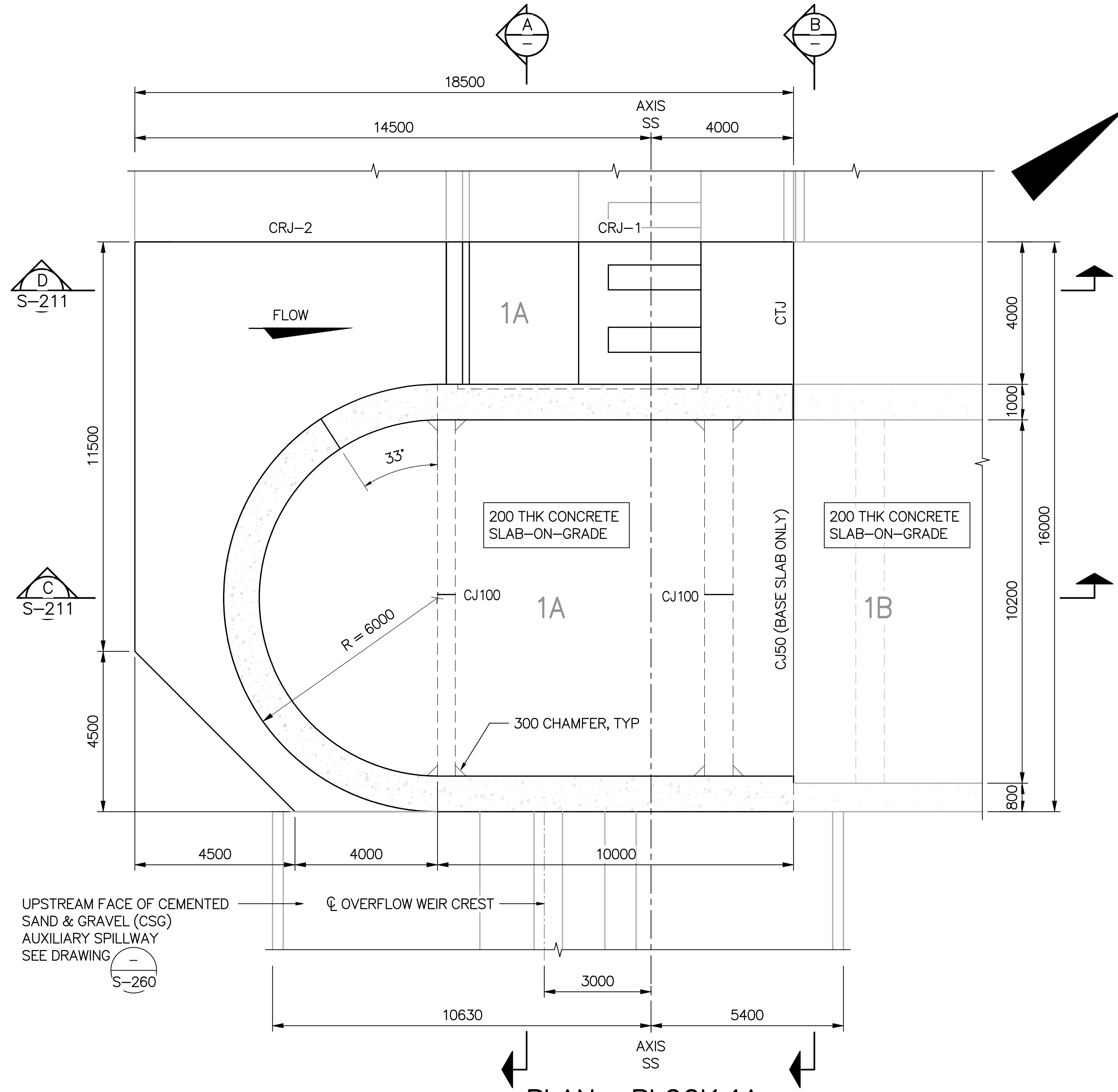
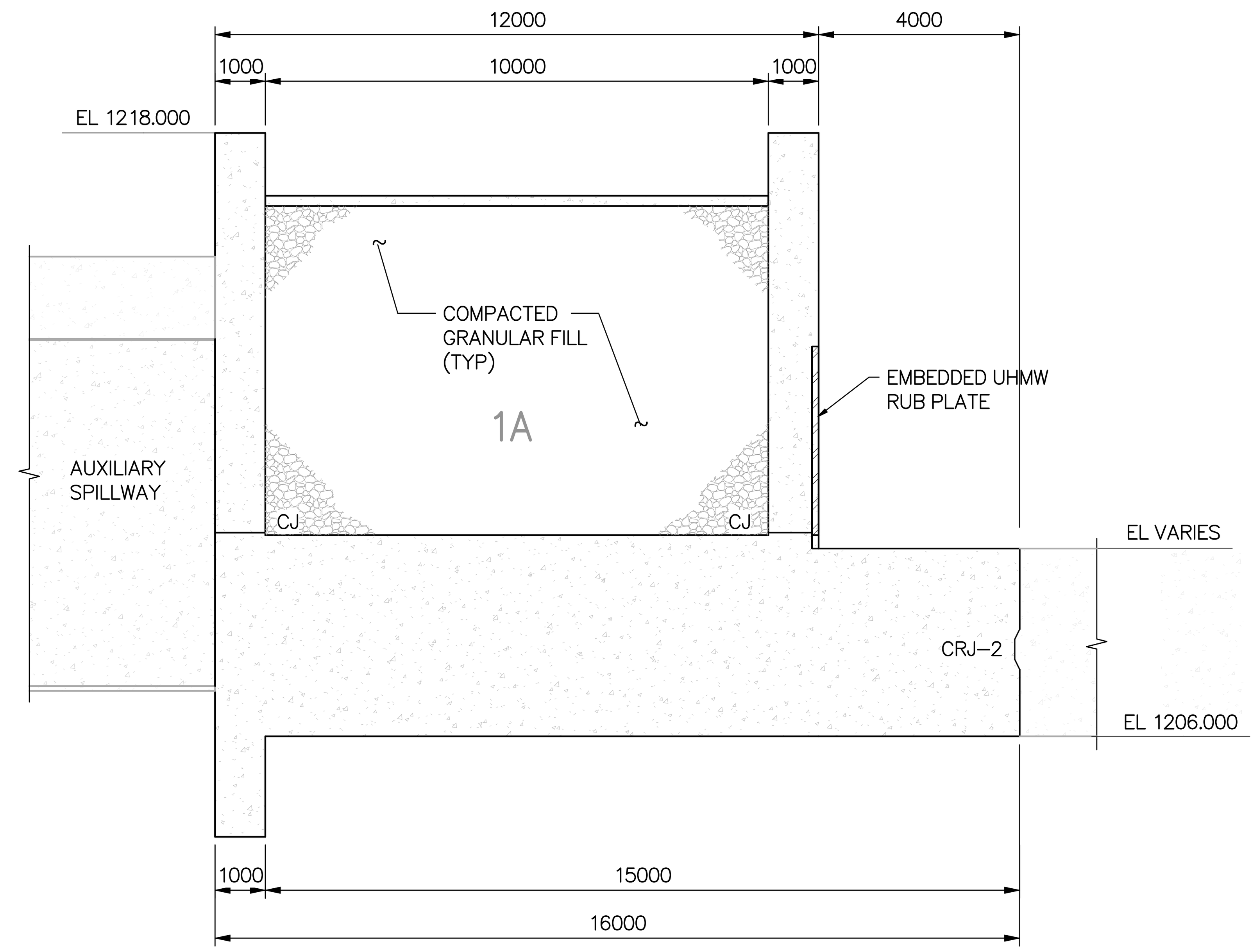


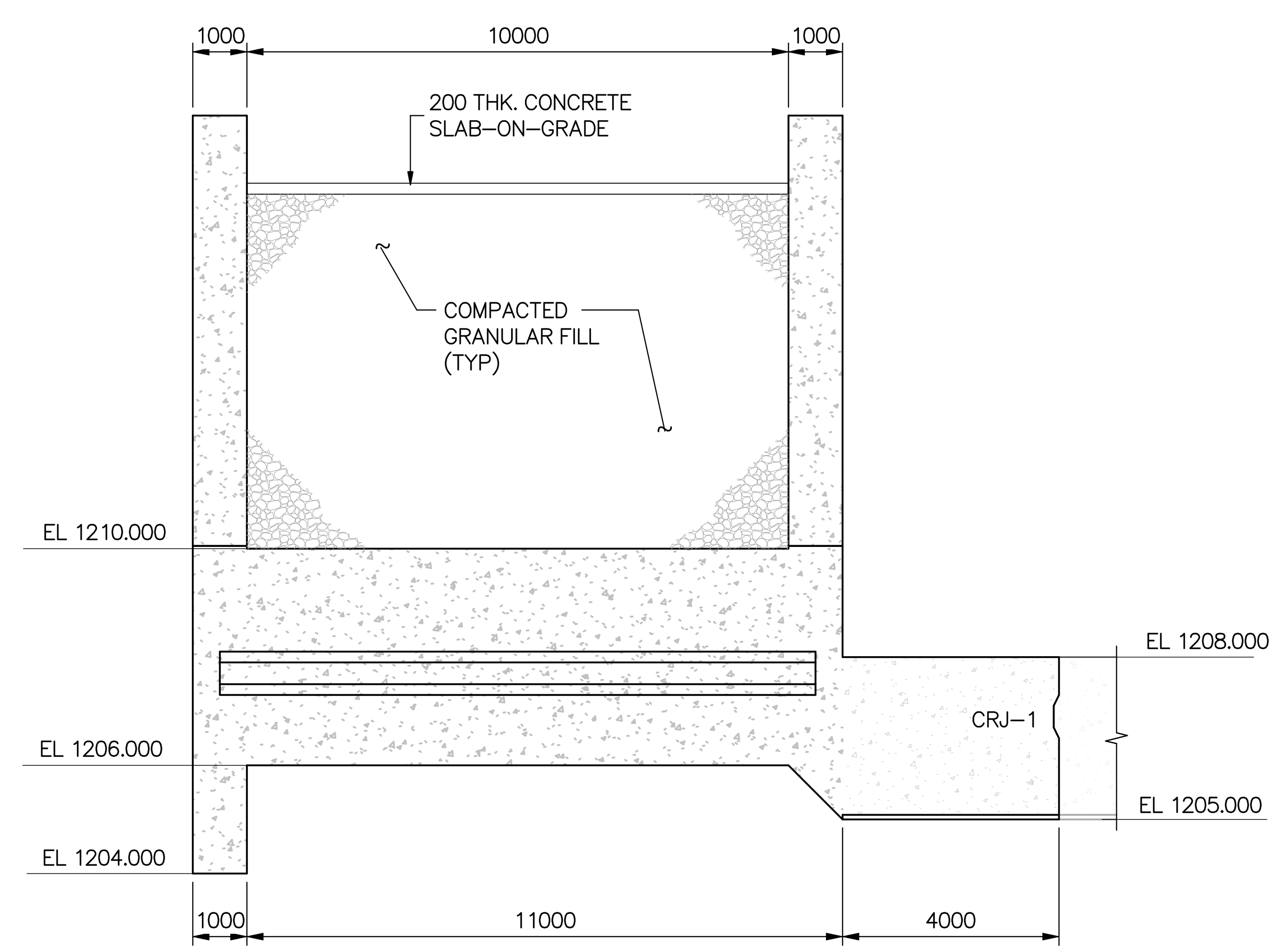
PLAN – BLOCK 1A

1 : 100



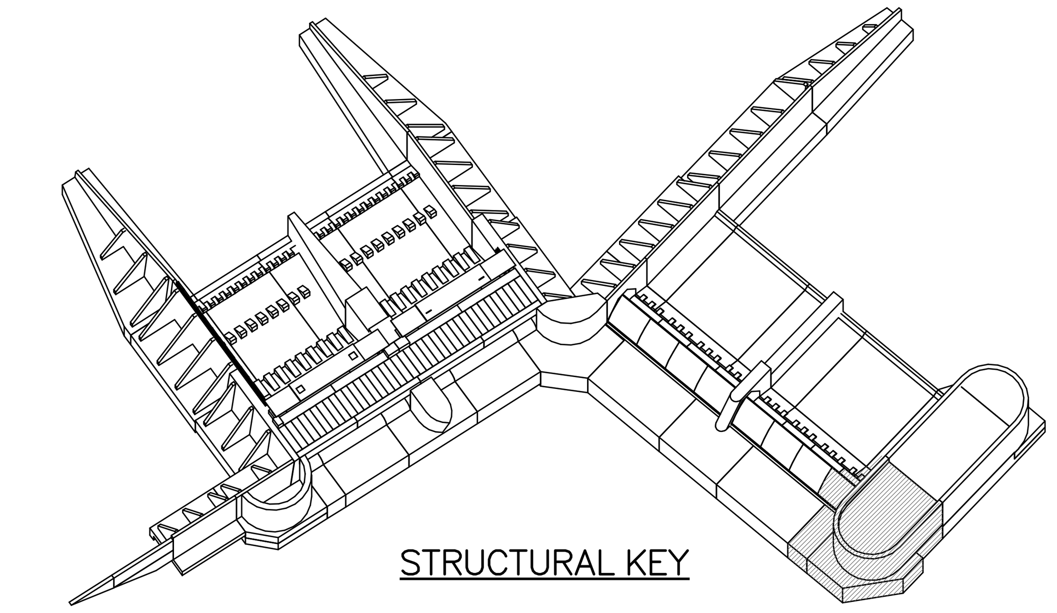
A SECTION

1 : 100

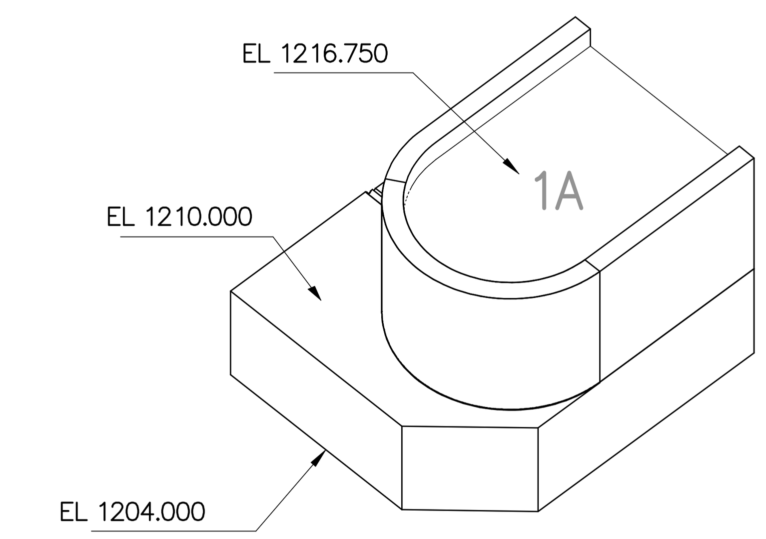


B SECTION

1 : 100



STRUCTURAL KEY



BLOCK 1A 3D VIEW

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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PRELIMINARY DESIGN

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

SERVICE SPILLWAY
RIGHT RETAINING WALL - BLOCK 1A
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-210	

Plot Date: 6/25/2020 2:37:58 PM Login: Dosler
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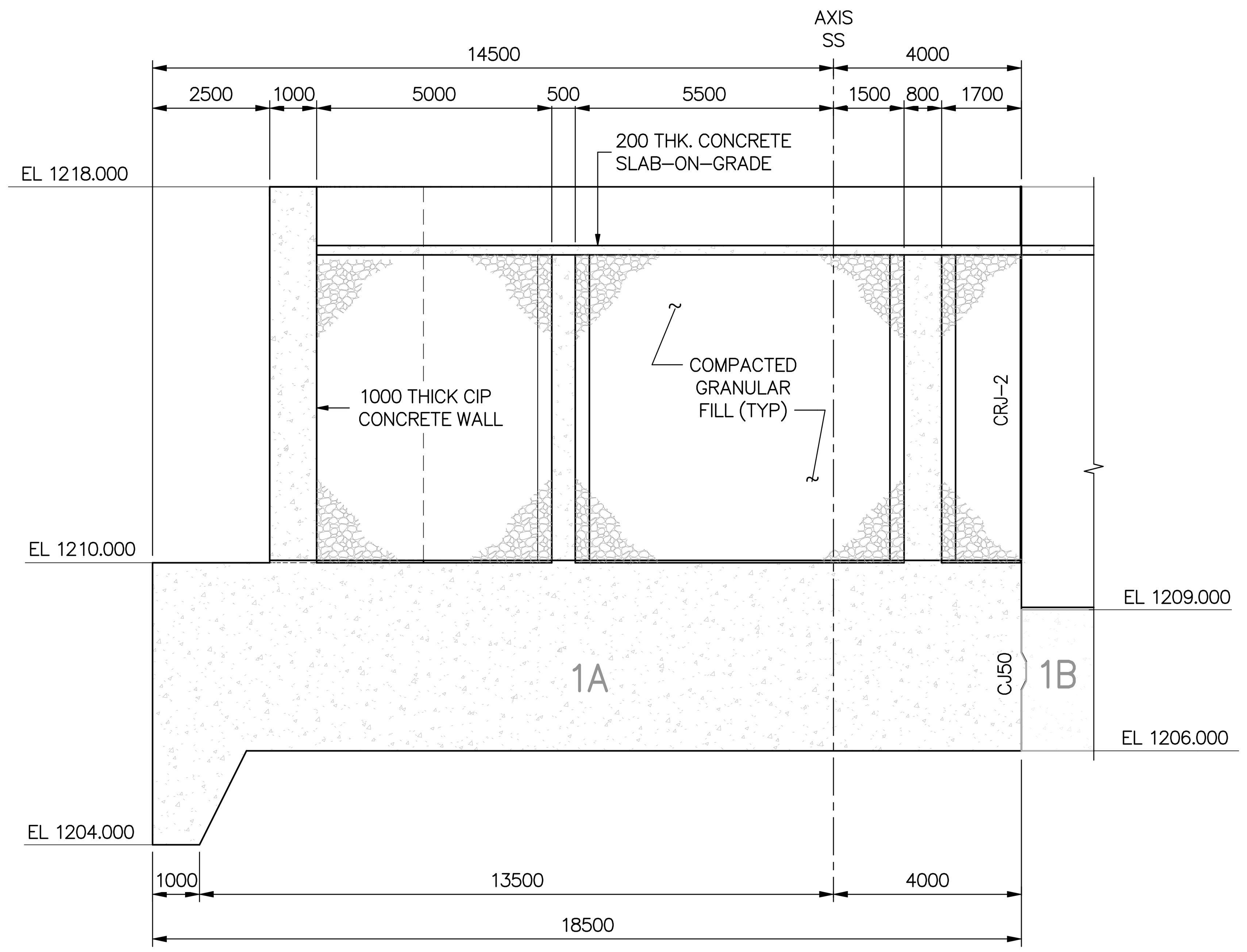
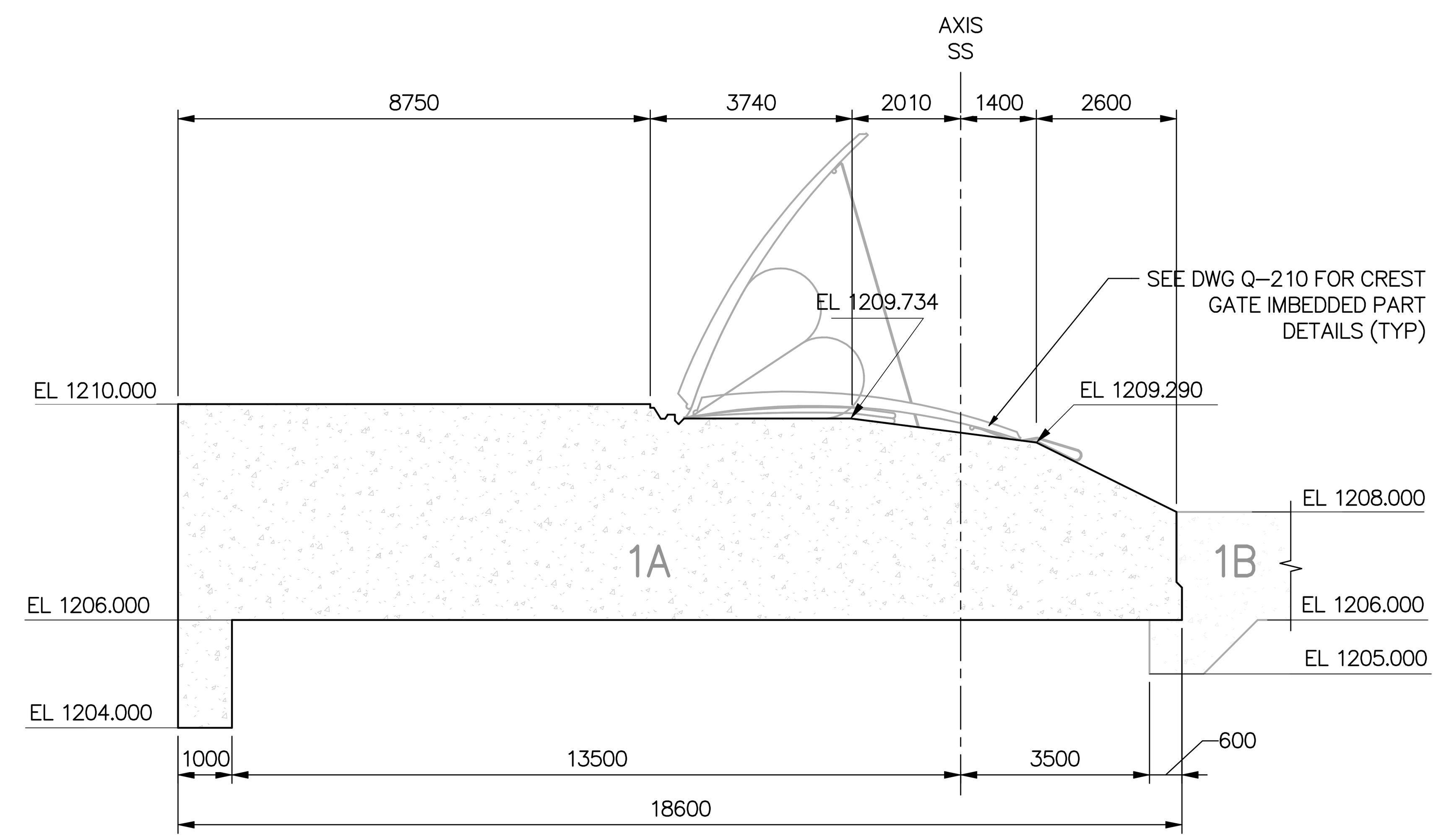


C
S-210
SECTION
1 : 100



D
S-210
SECTION
1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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PRELIMINARY DESIGN

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
RIGHT RETAINING WALL - BLOCK 1A
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-211	

Plot Date: 6/25/2020 2:37:58 PM Login: Dosler
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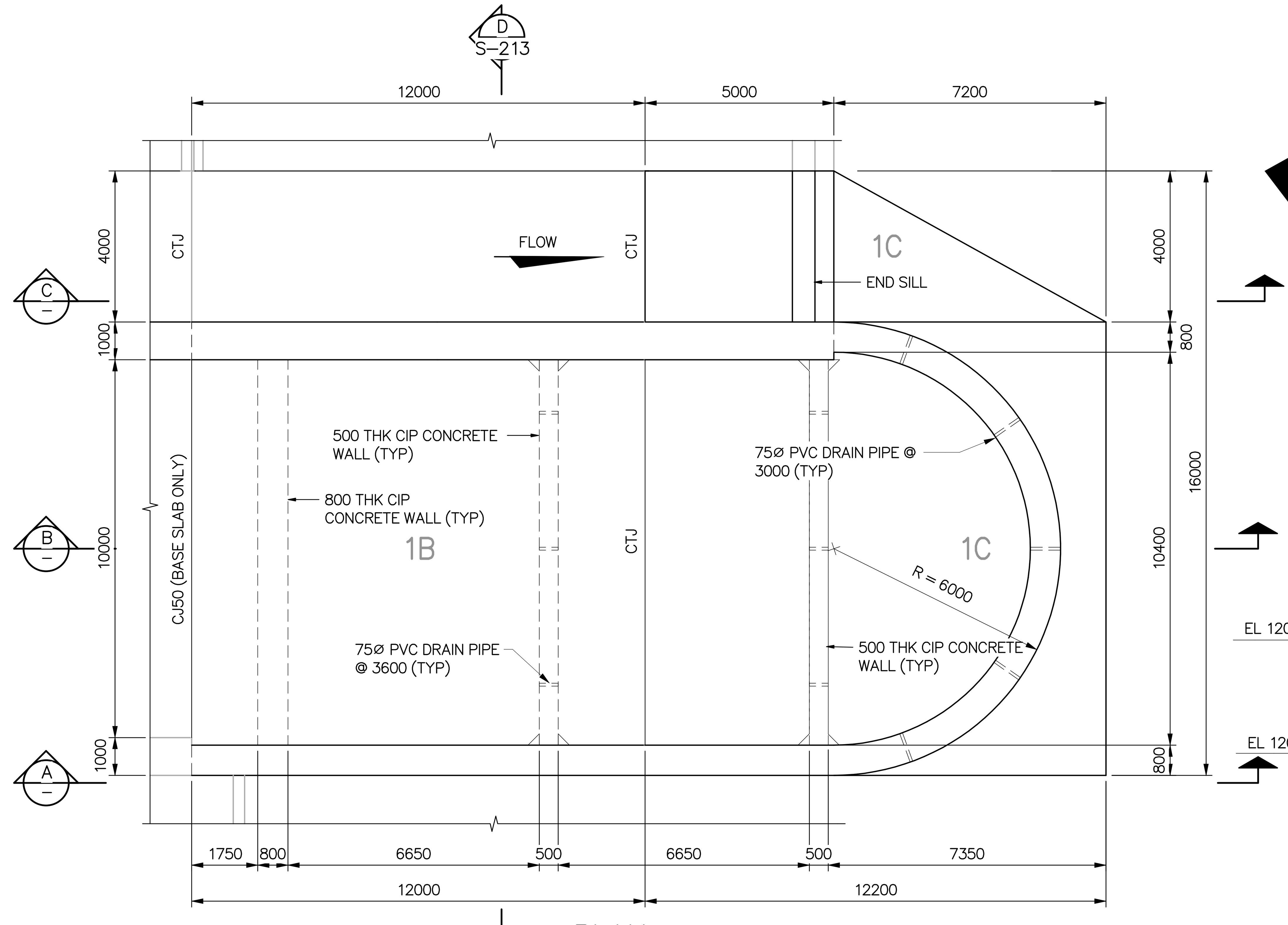
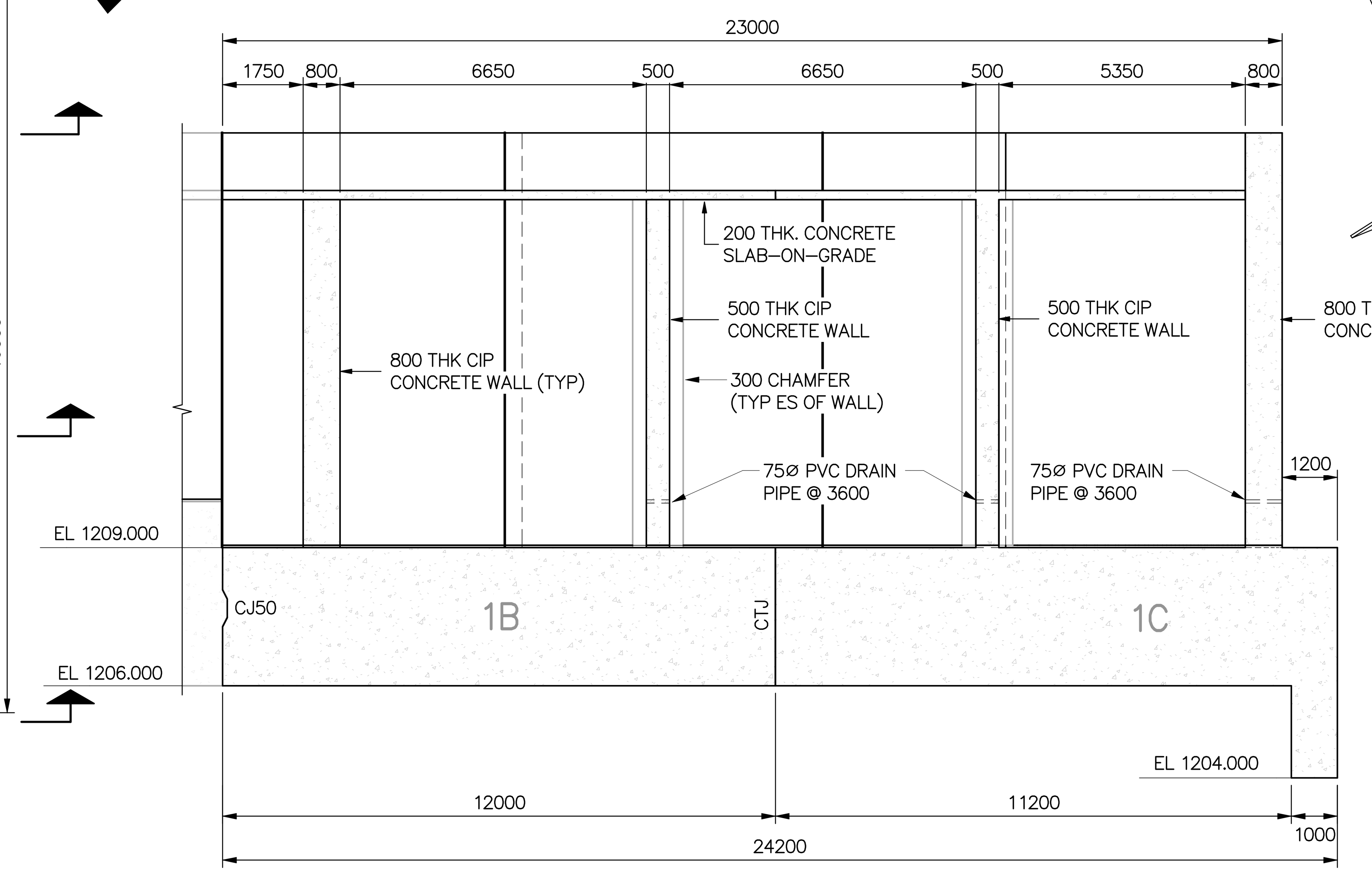


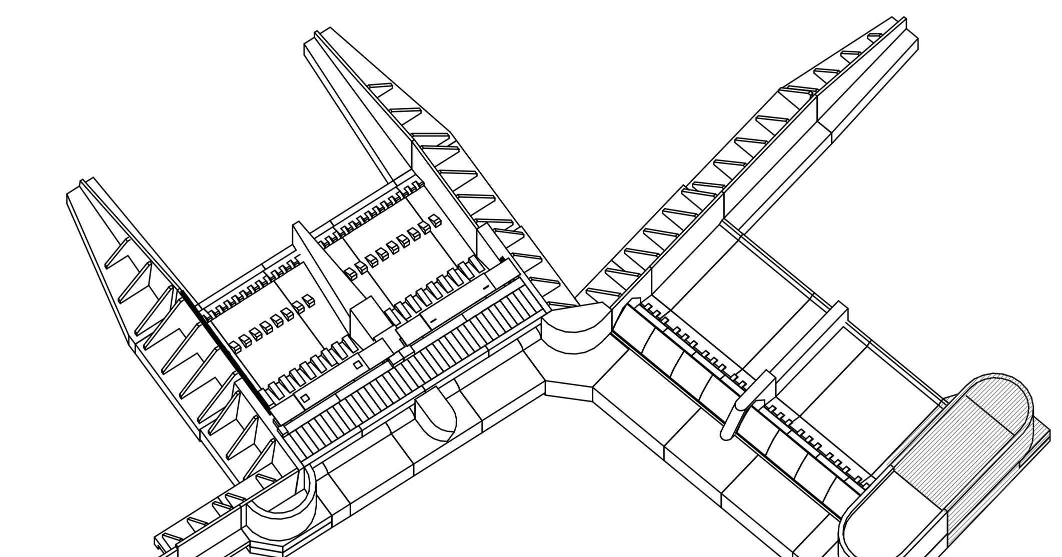
PLAN

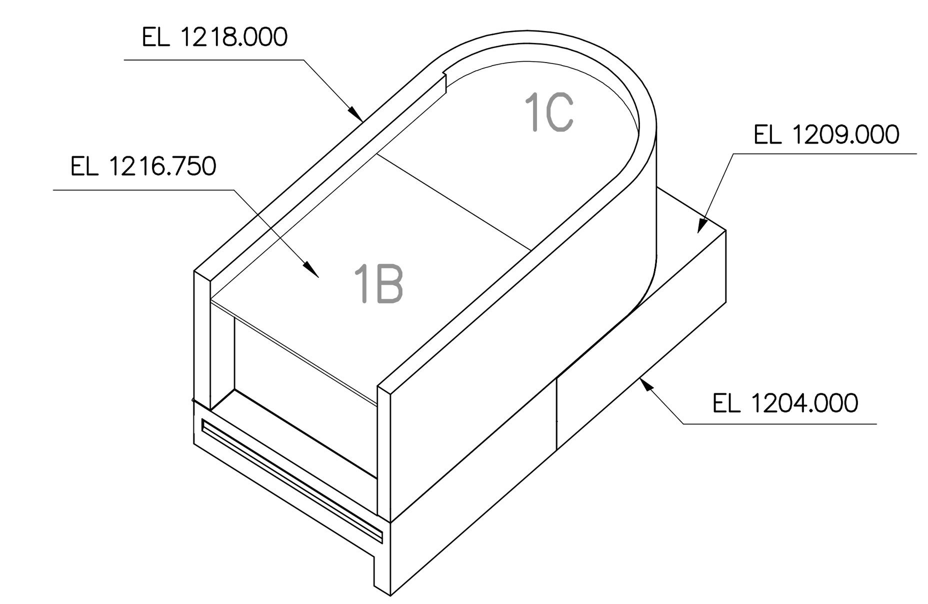
1 : 100



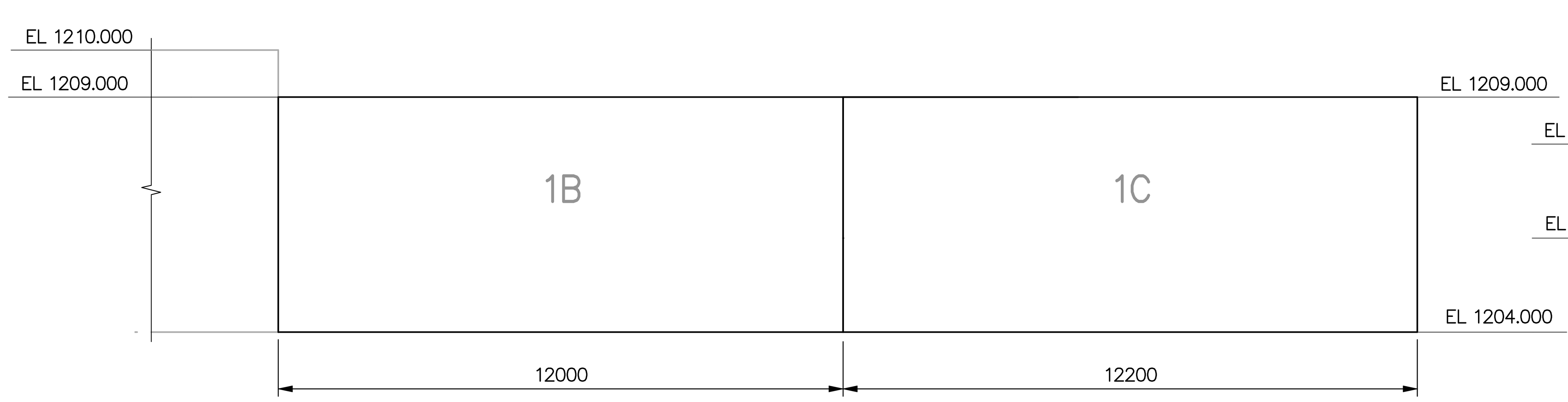
B SECTION

1 : 100



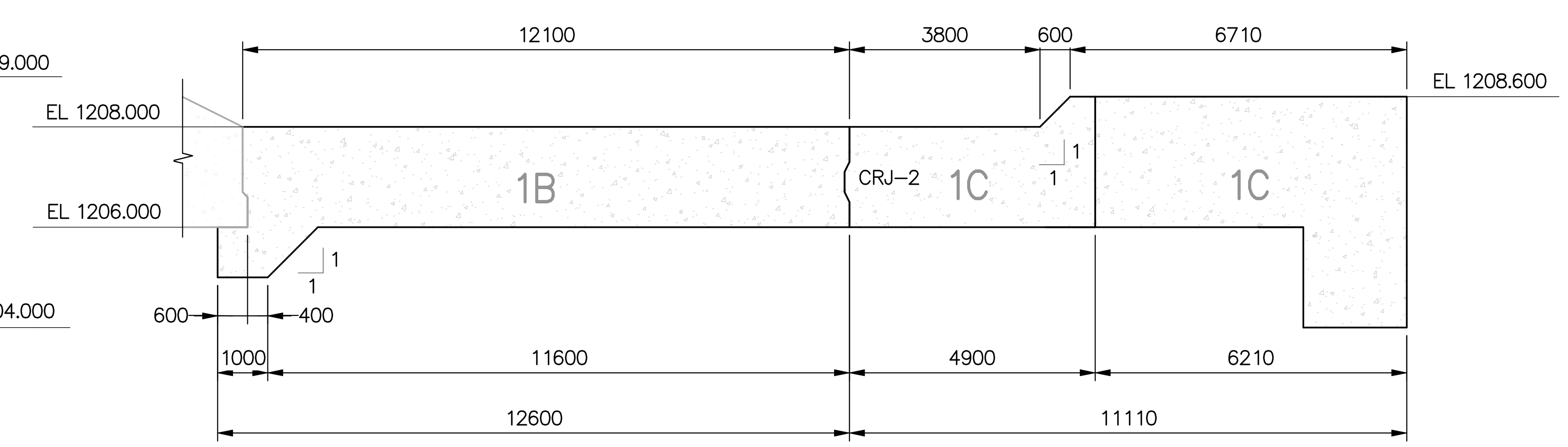


BLOCK 1B & 1C 3D VIEW



A SECTION

1 : 100



C SECTION

1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
RIGHT RETAINING WALL - BLOCKS 1B & 1C
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-212	

Plot Date: 6/25/2020 2:38:00 PM Login: Dosler
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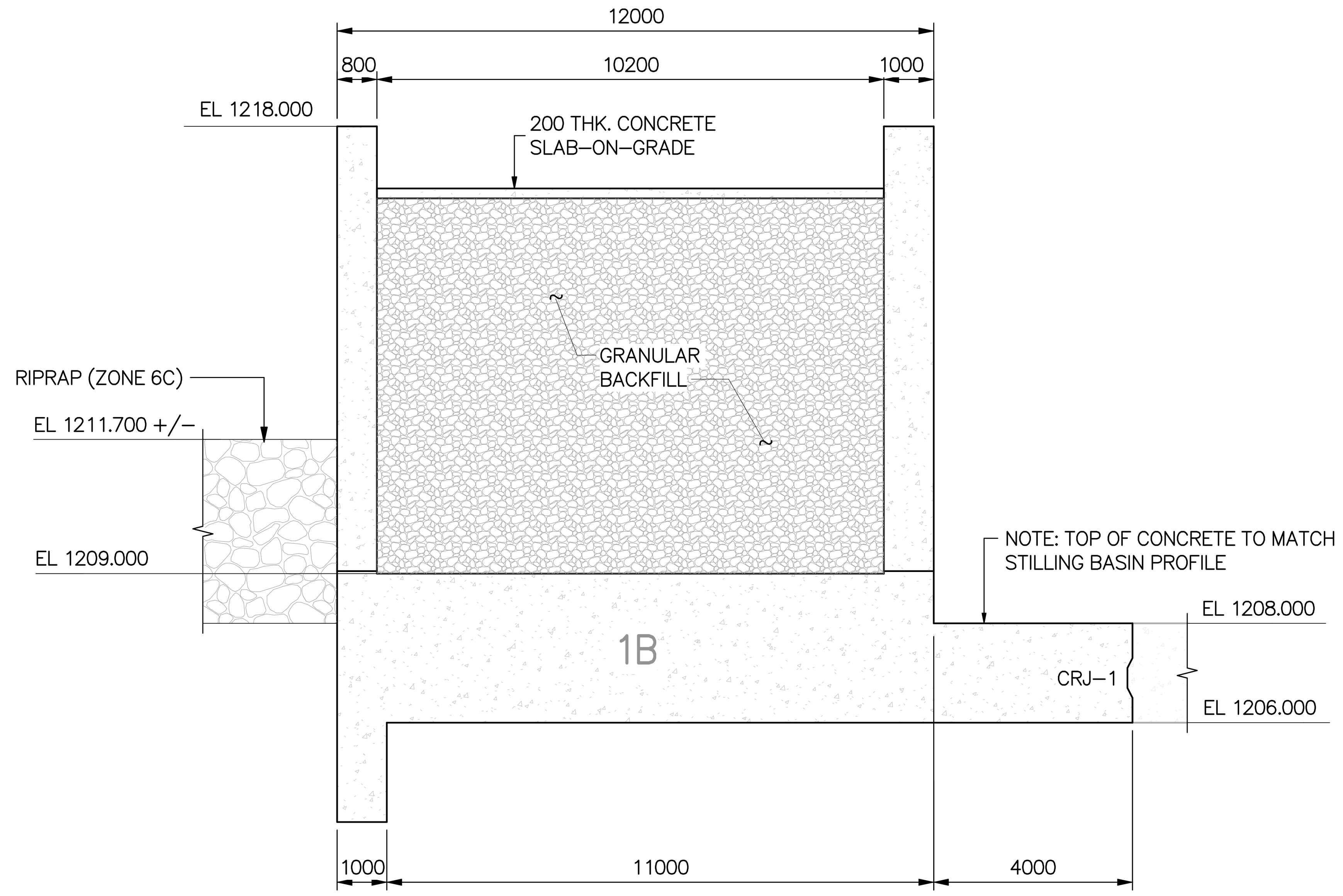

D
S–212

SECTION

1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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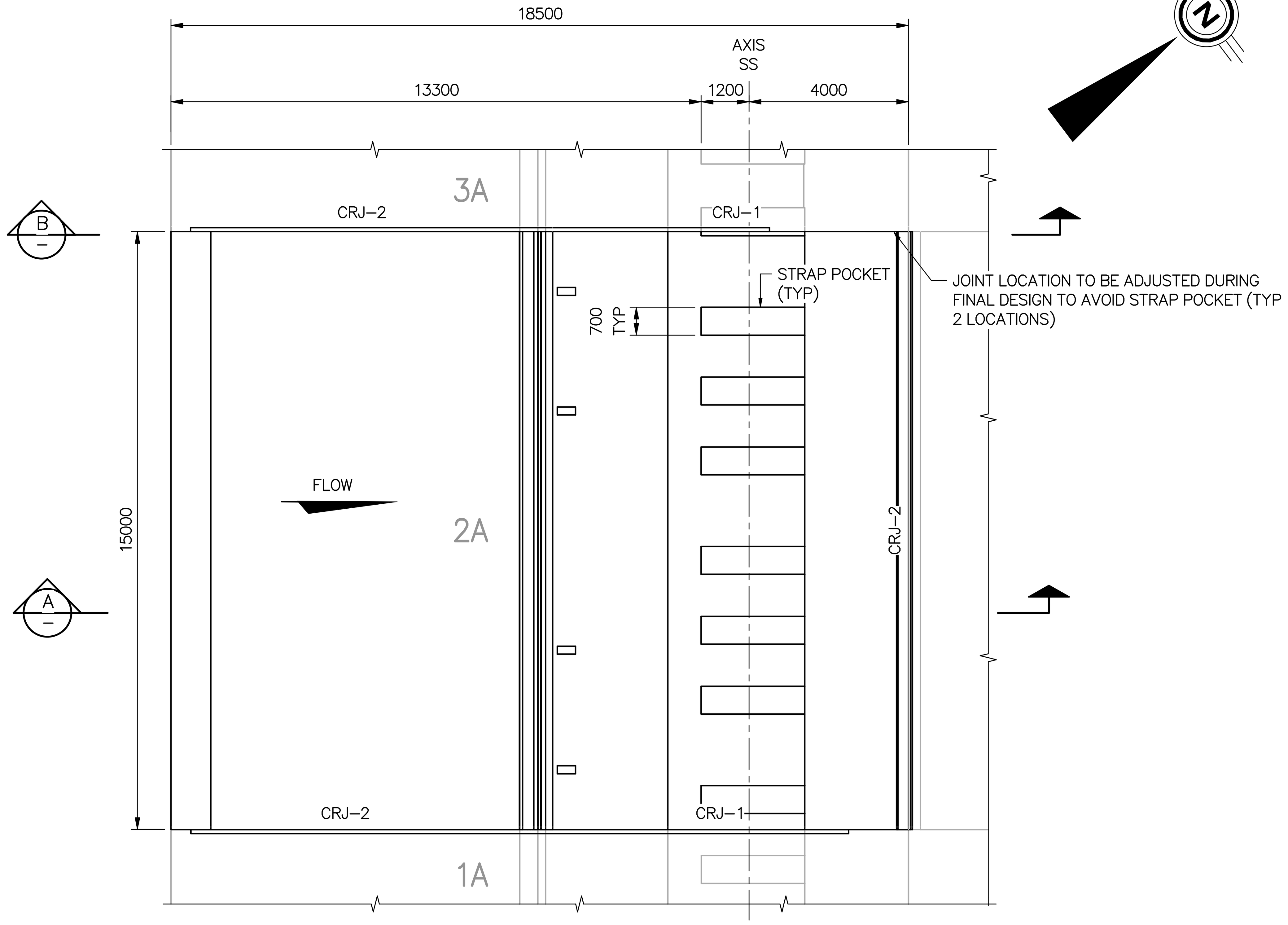
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

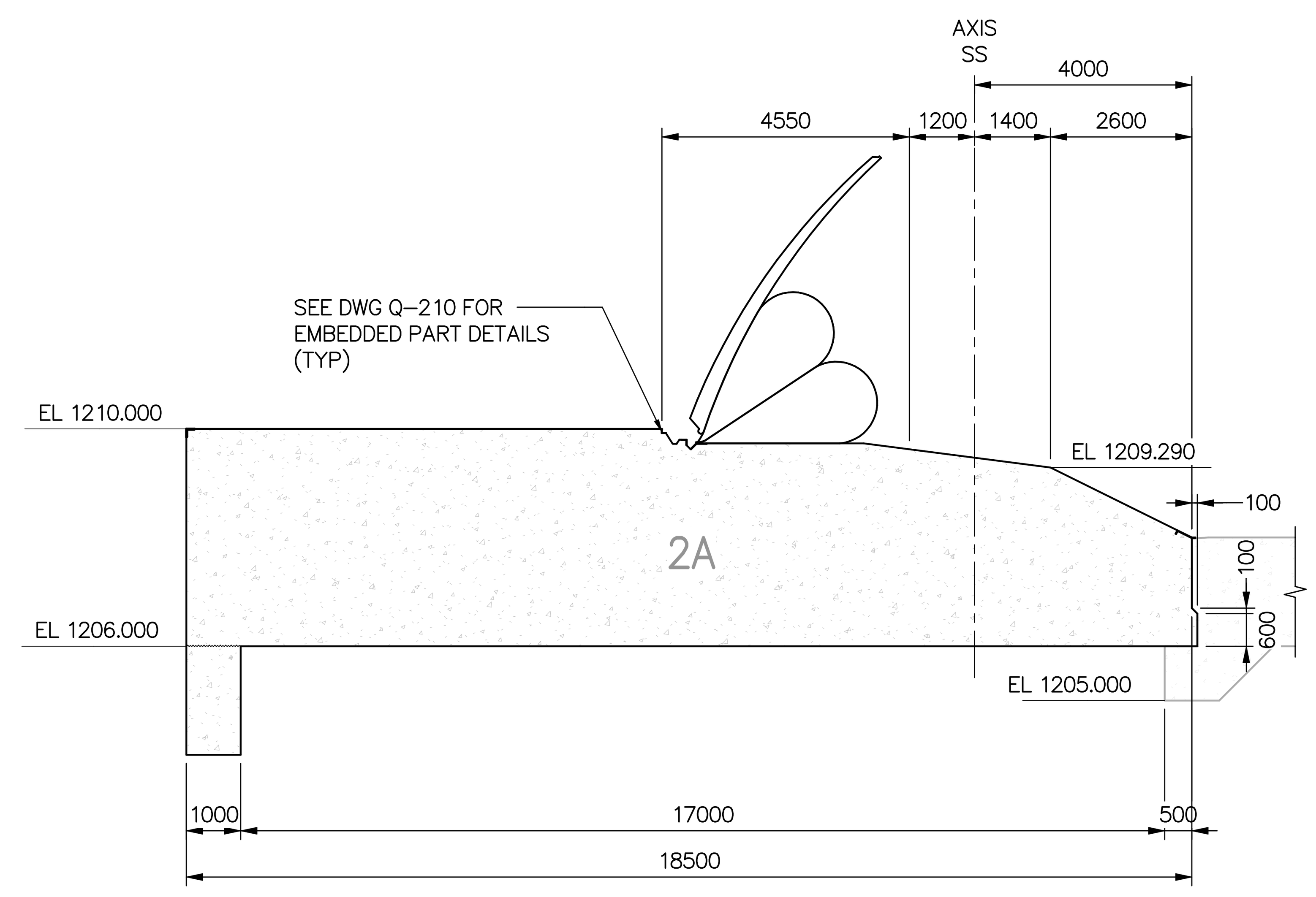
SERVICE SPILLWAY
RIGHT RETAINING WALL - BLOCKS 1B & 1C
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-213	

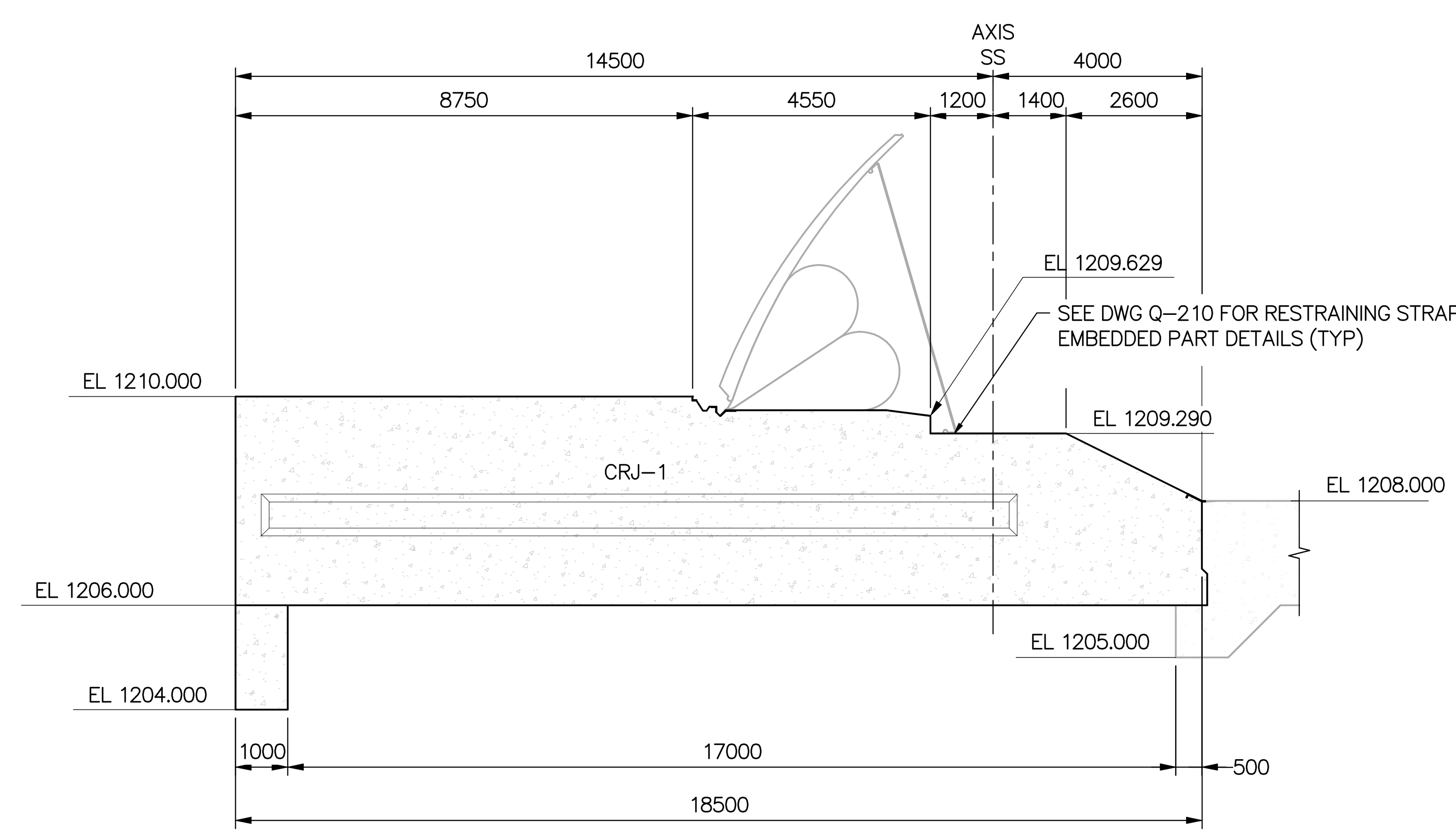
Plot Date: 6/25/2020 2:38:01 PM Login: Dosler
REVIT FILE No. BIM 360://110773396_SR1_springbank_offstream_storage/110773396_structure_SR1_200_300_pdr_reissued_2020_don.osler.rvt



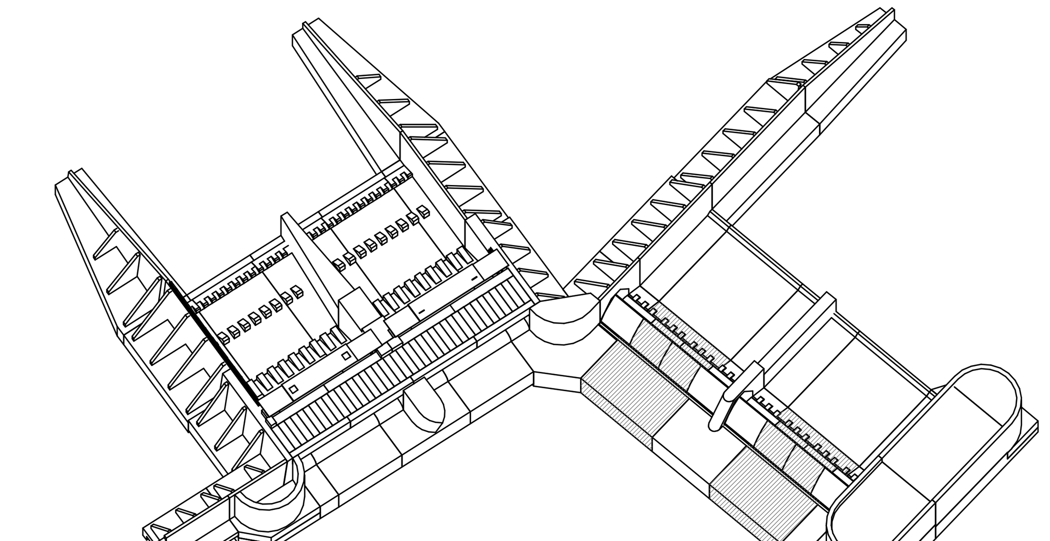
PLAN – BLOCK 2A
1 : 100



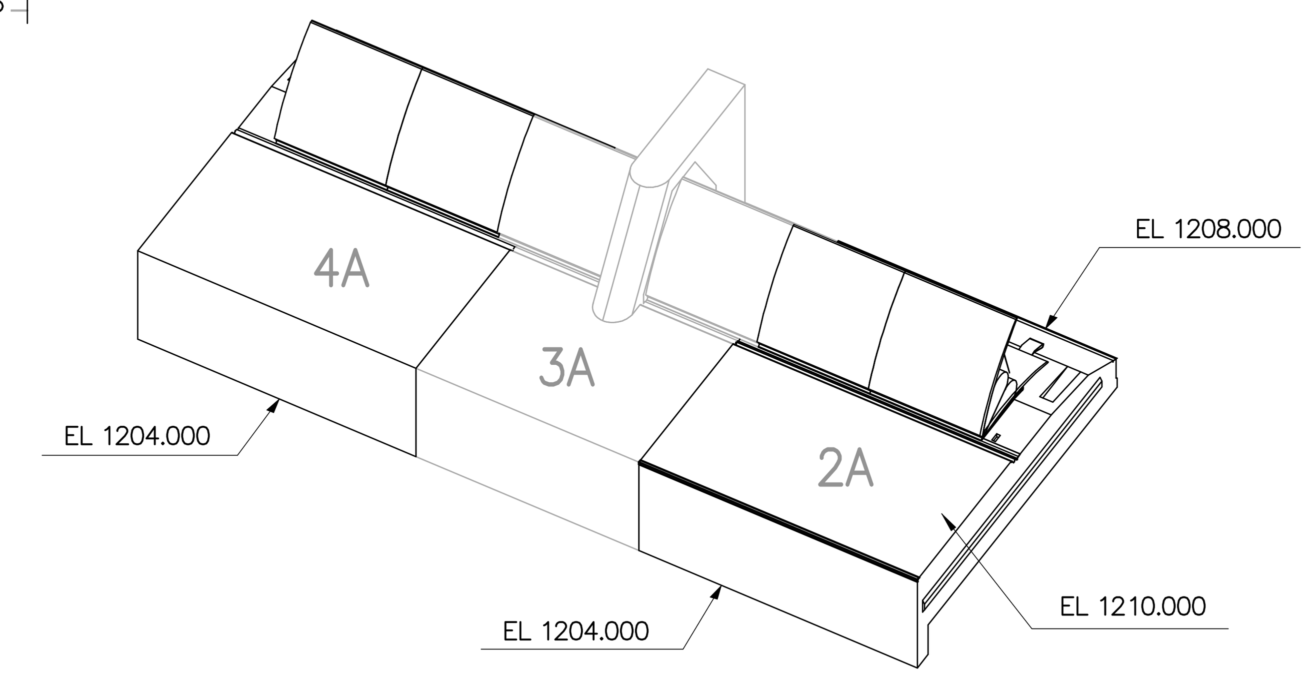
A SECTION
1 : 100



B SECTION
1 : 100



STRUCTURAL KEY



BLOCKS 2A & 4A 3D VIEW

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
GATE STRUCTURE - BLOCKS 2A & 4A
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-220	

Plot Date: 6/25/2020 2:38:02 PM Login: Dosler
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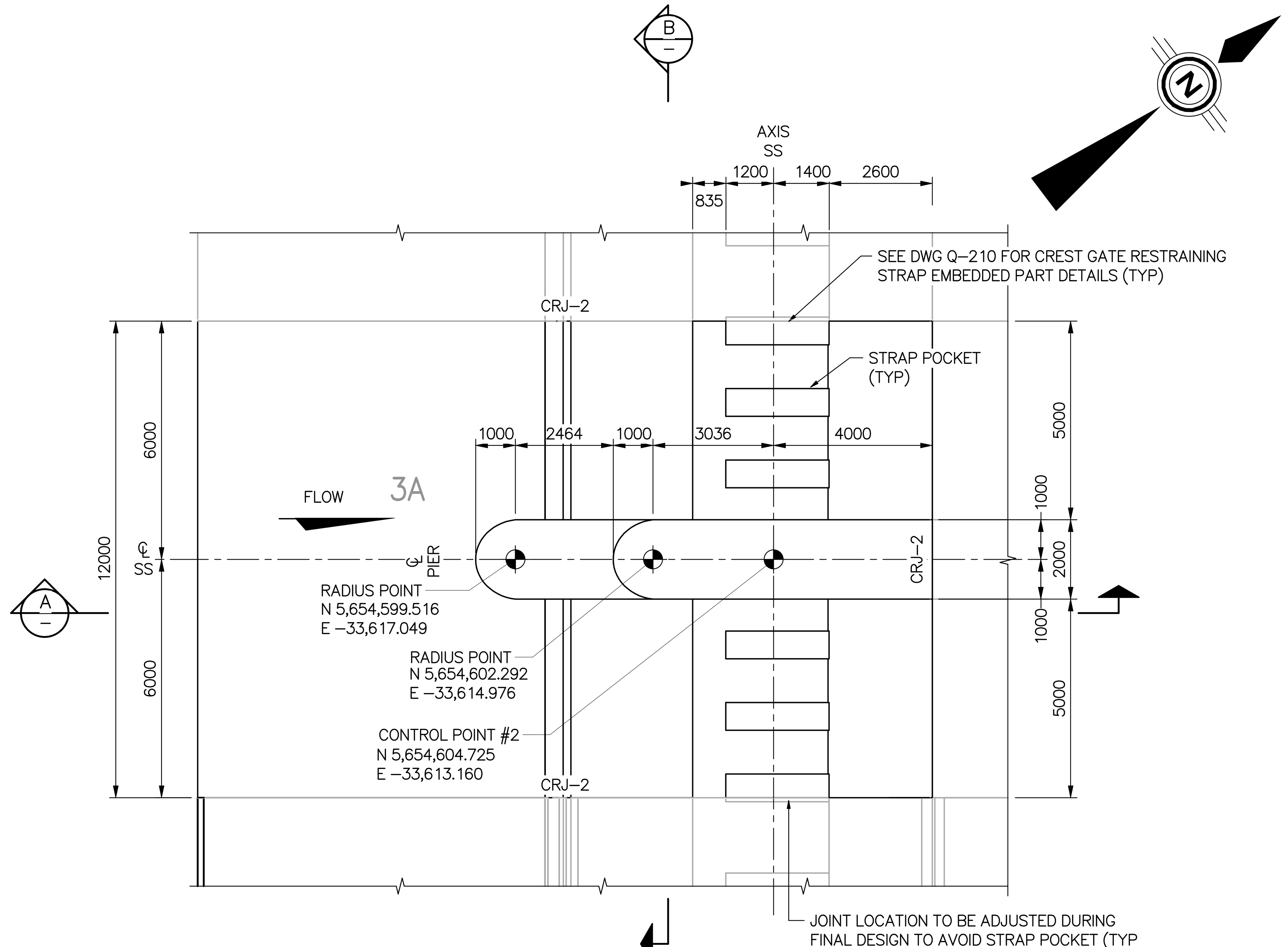

PLAN
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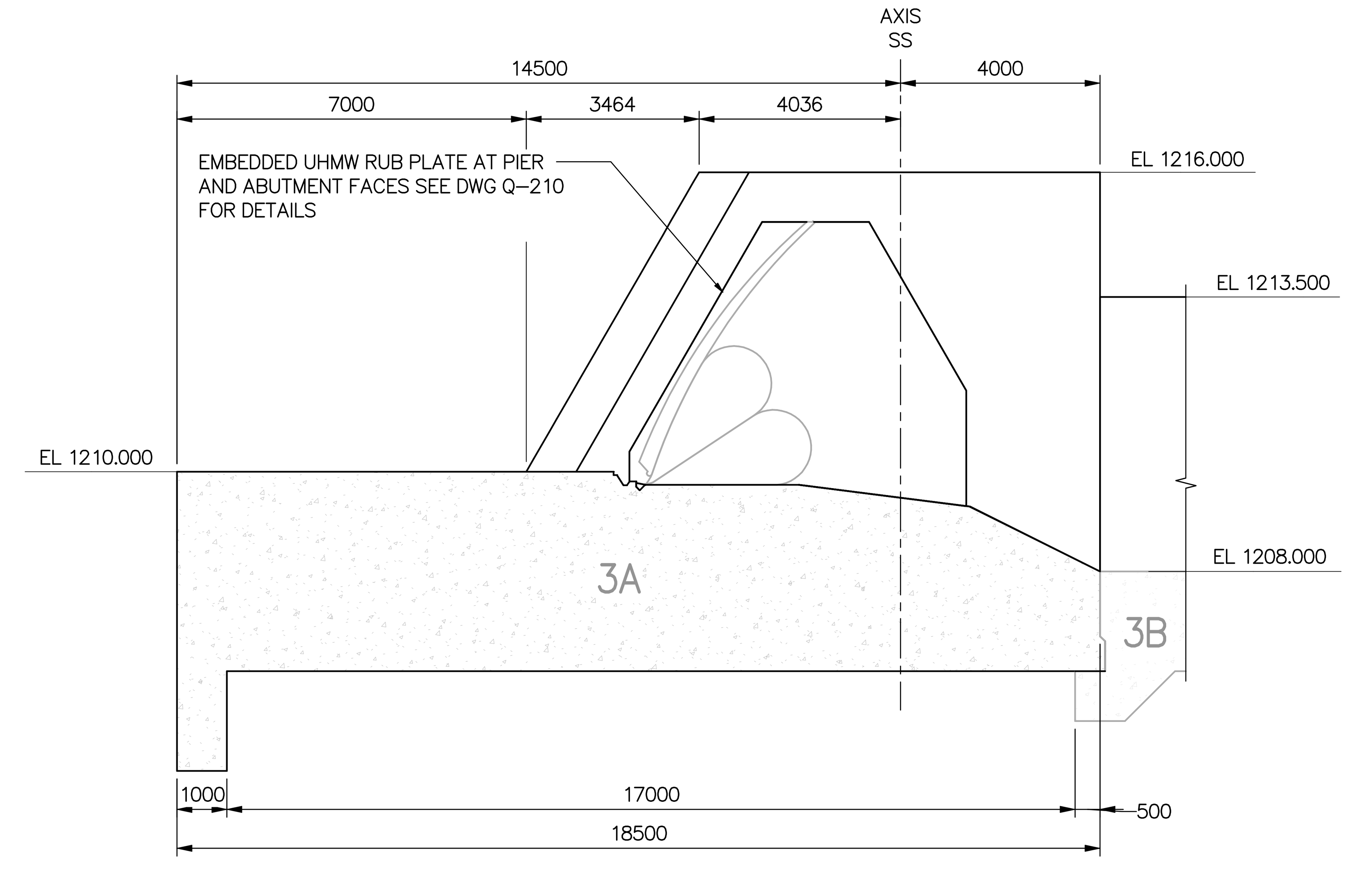
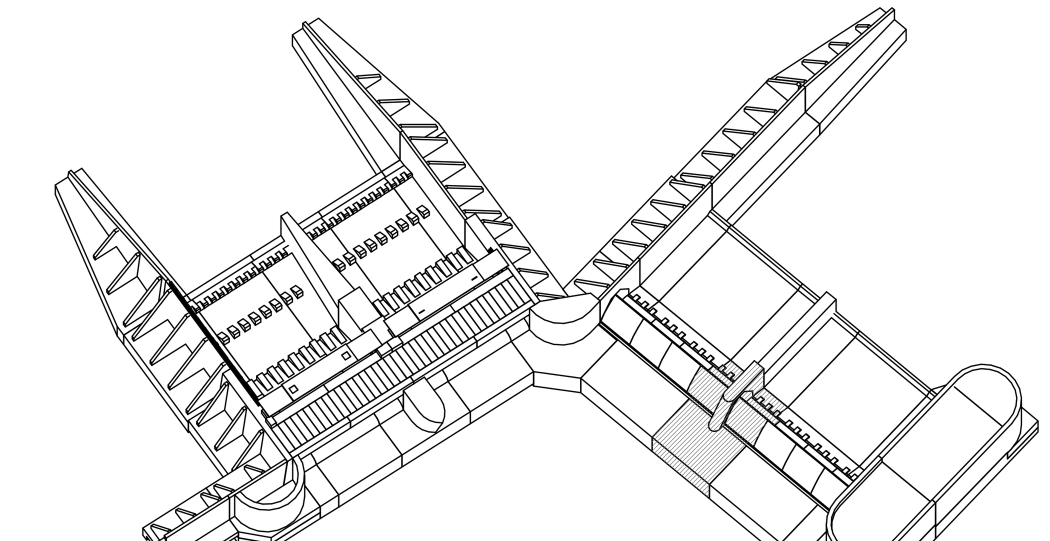


A SECTION
1 : 100



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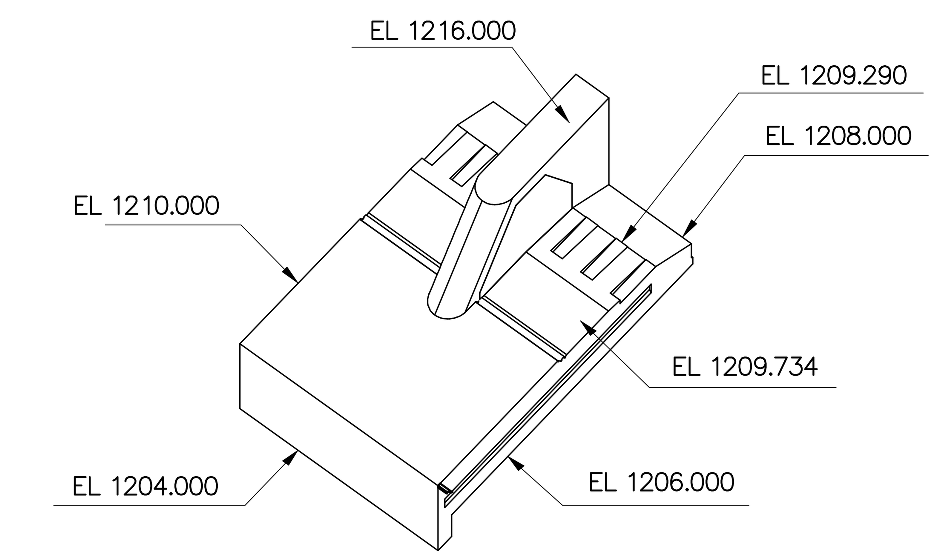
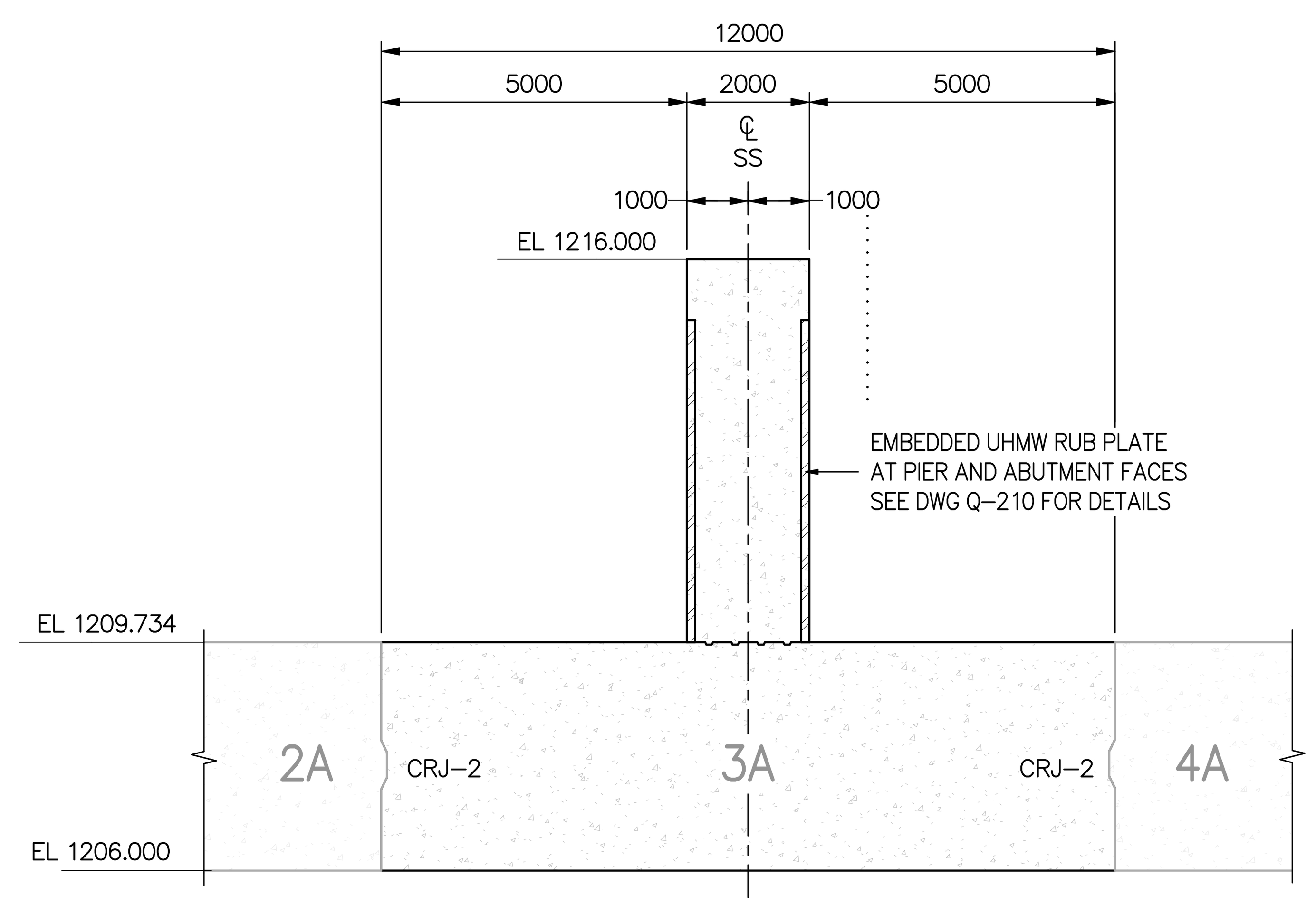


BLOCK 3A 3D VIEW



B SECTION
1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
GATE STRUCTURE - BLOCK 3A
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-222	

Plot Date: 6/25/2020 2:38:03 PM Login: Dosler
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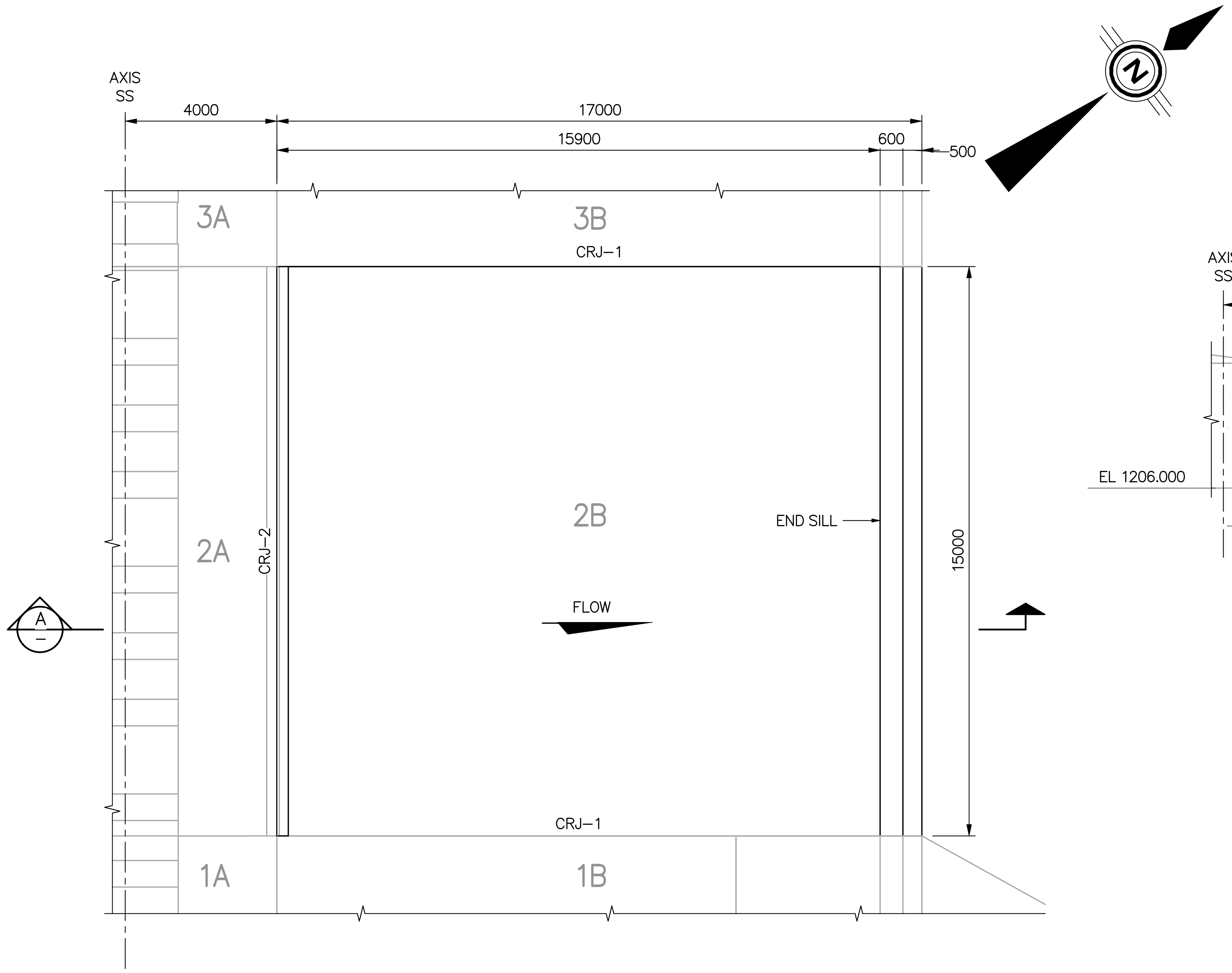
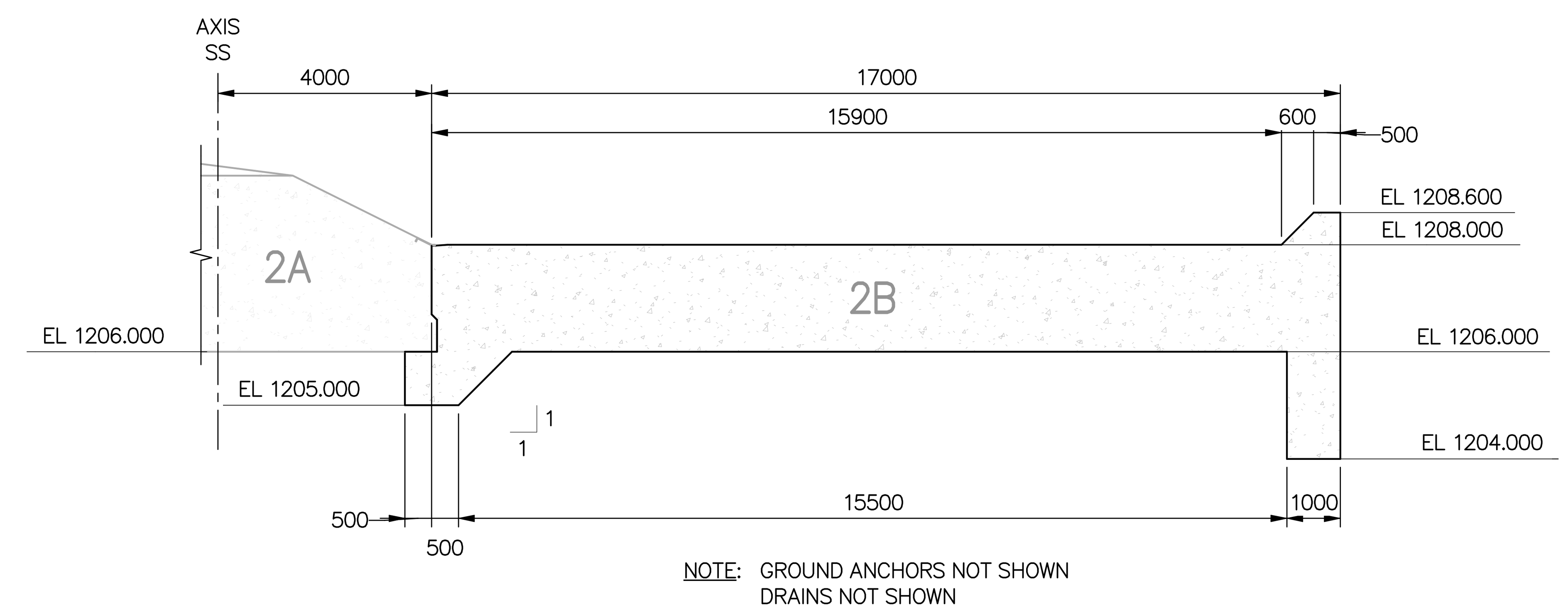


PLAN — BLOCK 2B
1 : 100



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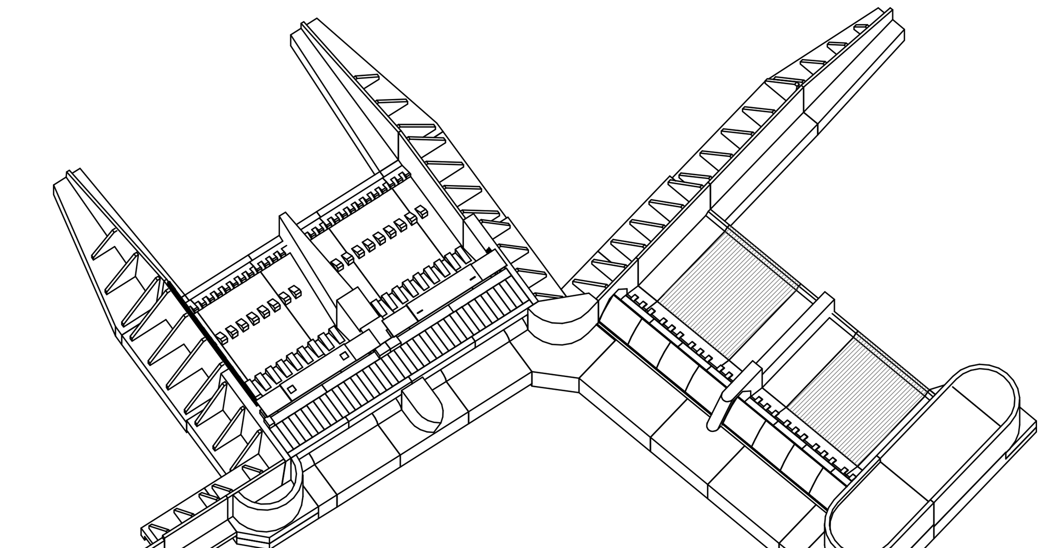
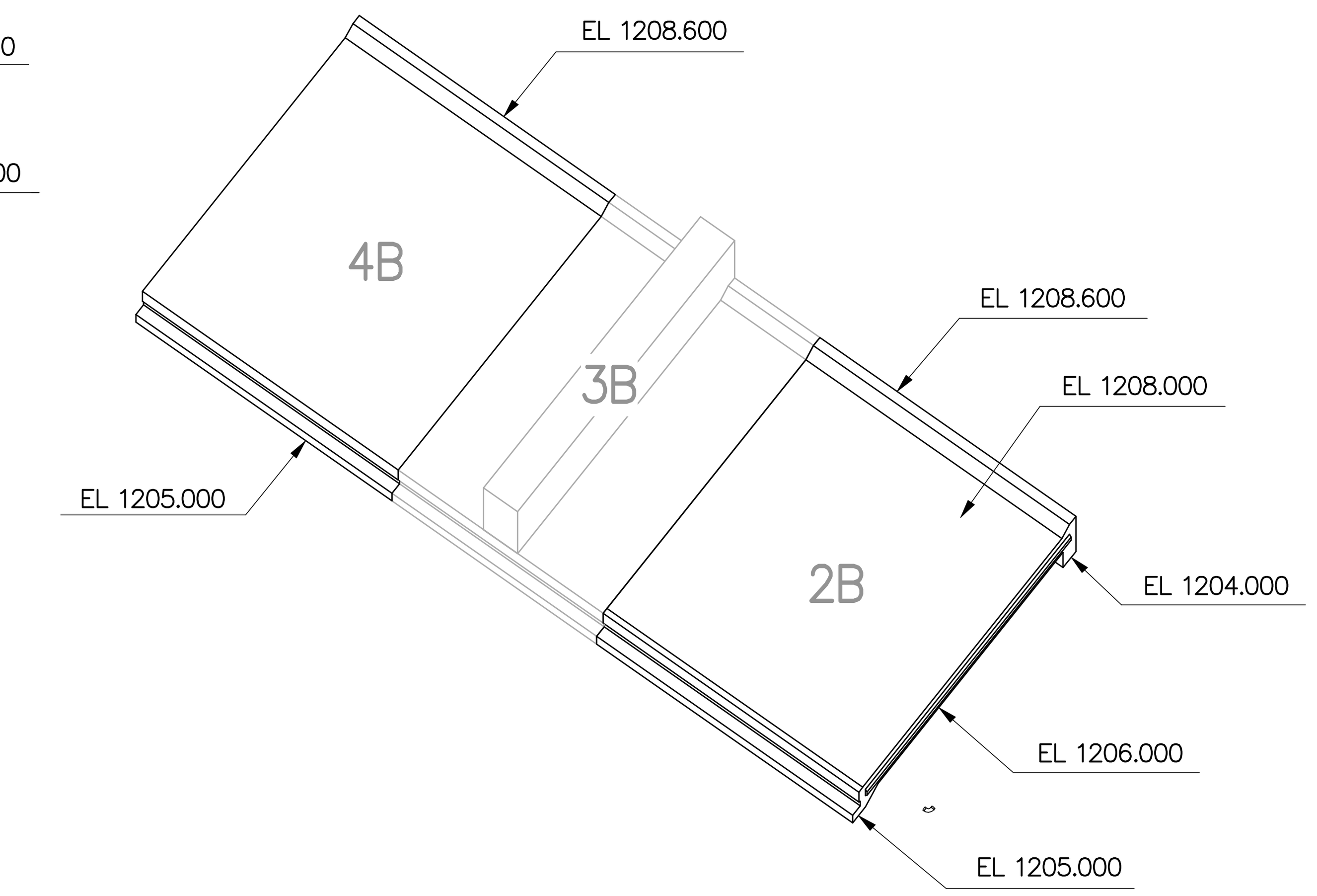
SECTION
1 : 100



STRUCTURAL KEY



BLOCKS 2B & 4B 3D VIEW

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

Plot Date: 6/25/2020 2:38:04 PM Login: Dosler
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PRELIMINARY DESIGN

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
STILLING BASIN - BLOCKS 2B & 4B
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
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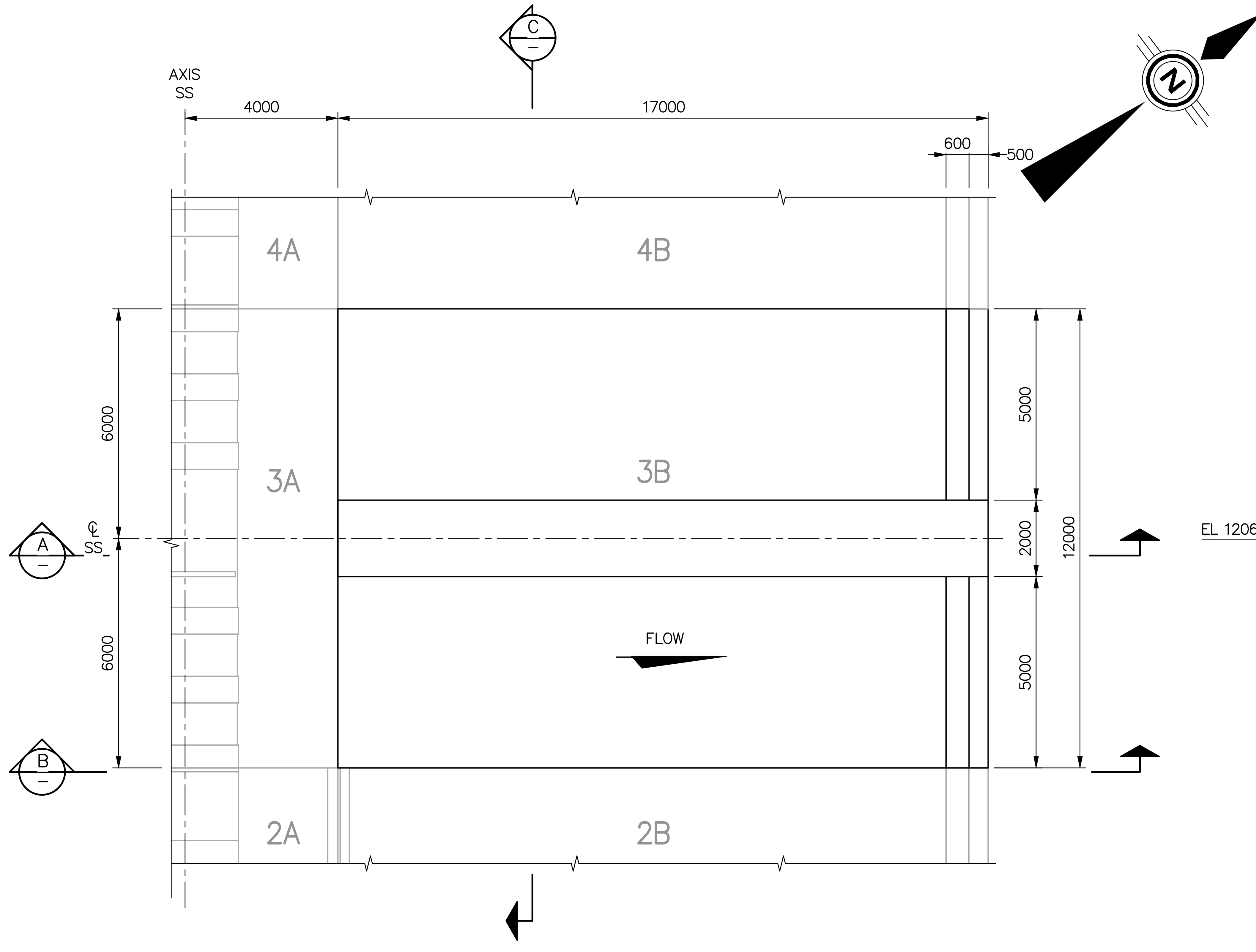

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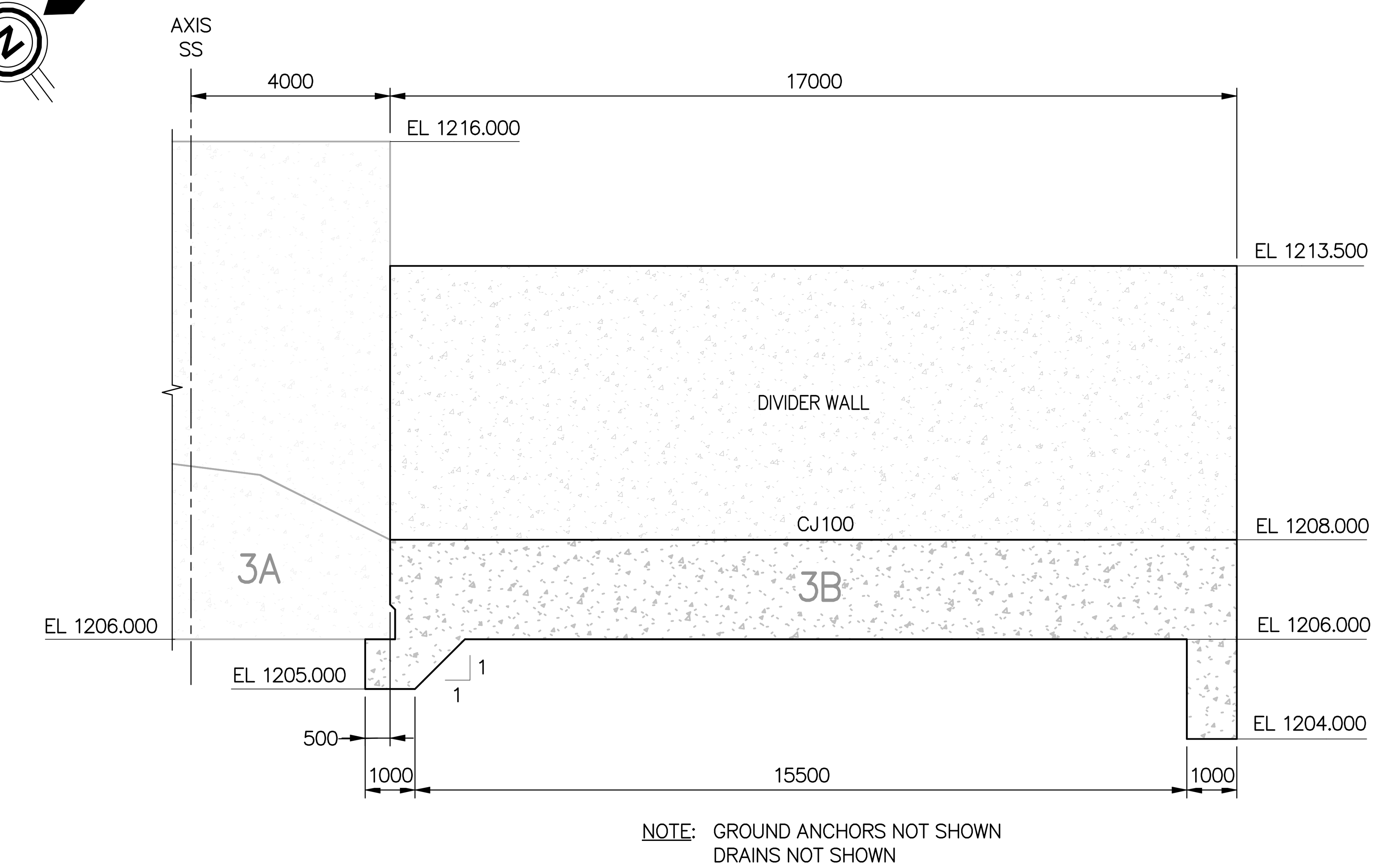
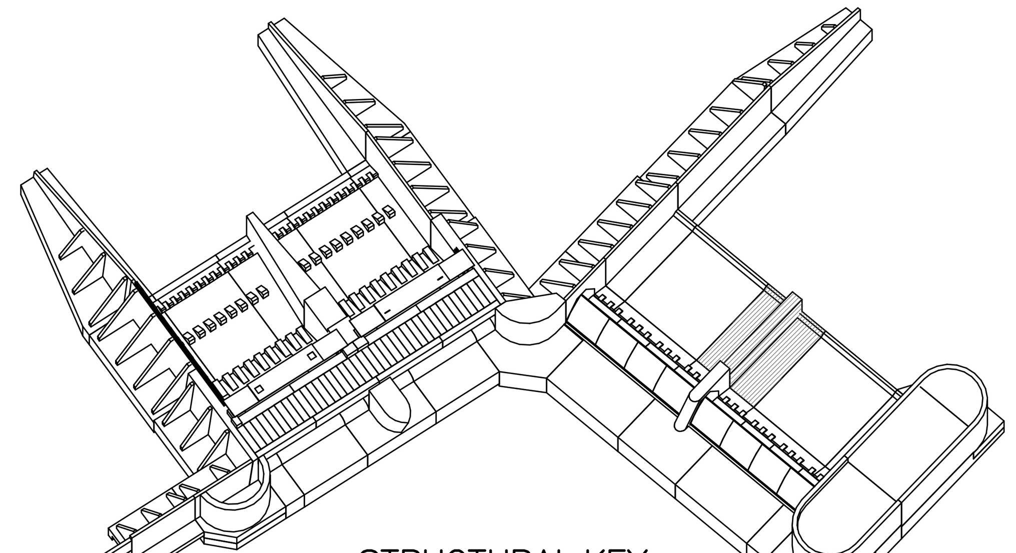
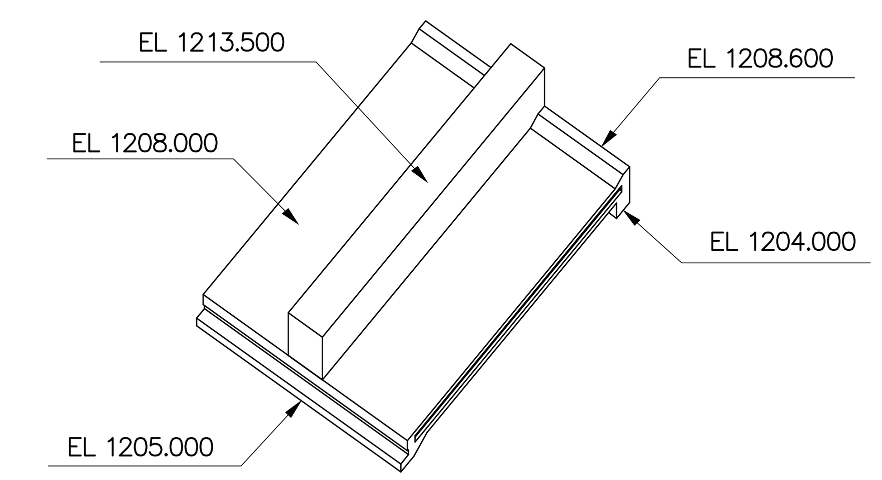


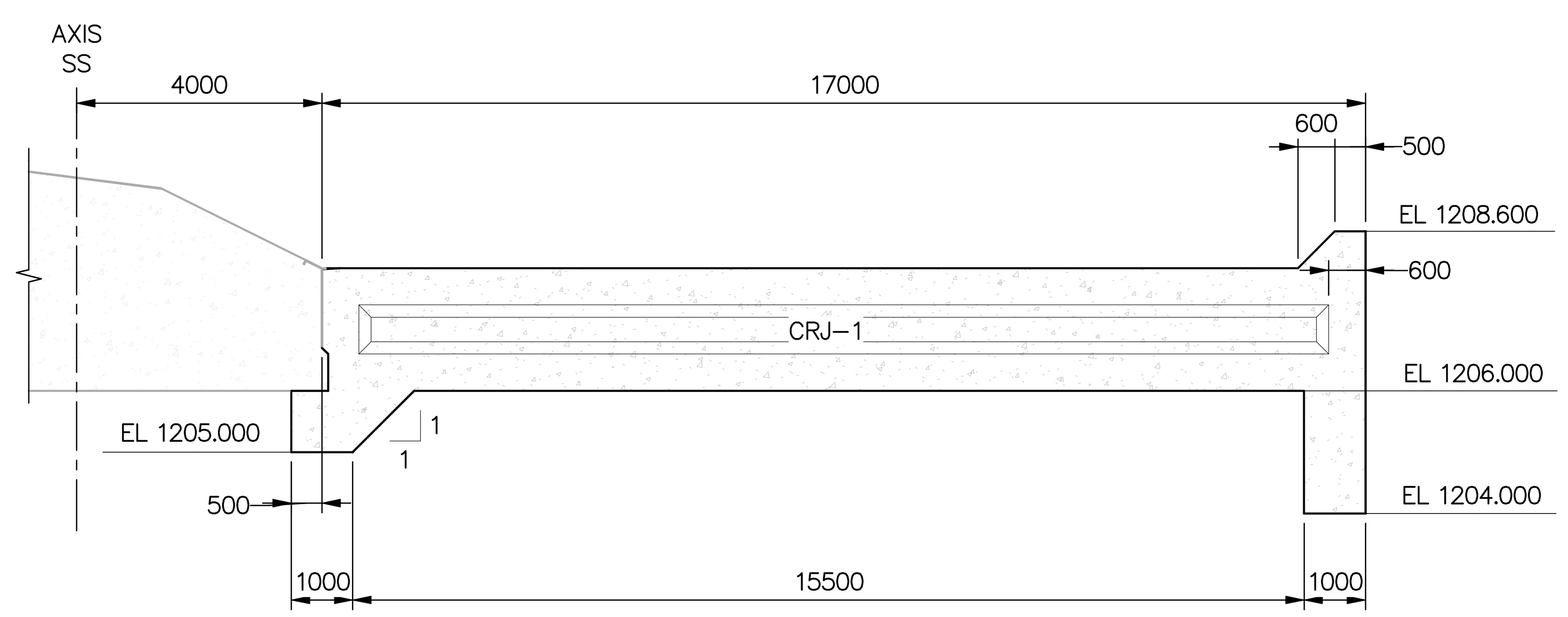
A SECTION
1 : 100



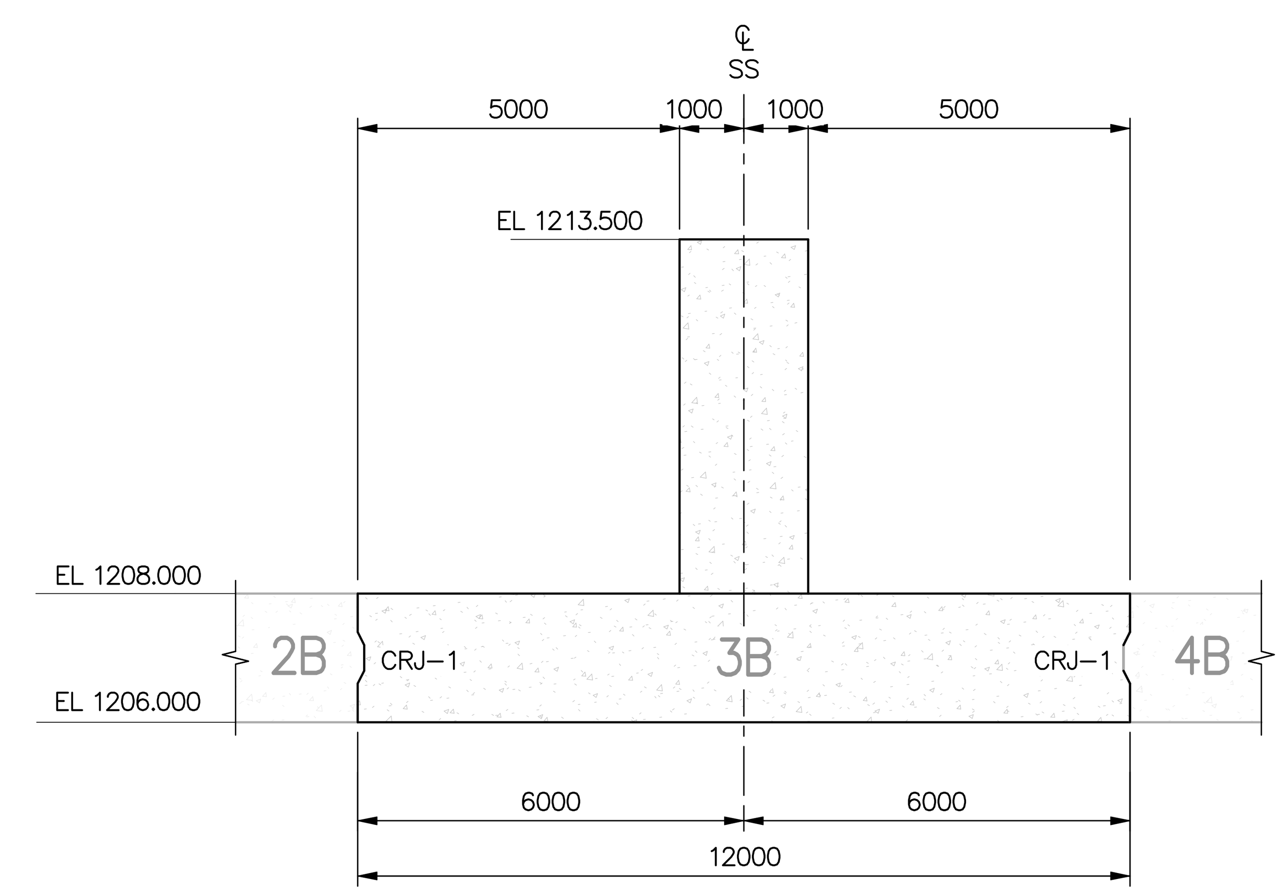
STRUCTURAL KEY



BLOCK 3B 3D VIEW



B SECTION
1 : 100



C SECTION
1 : 100

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

Plot Date: 6/25/2020 2:38:05 PM Login: Dosler
REVIT FILE No. BIM 360://110773396_SR1_springbank_offstream_storage/110773396_structure_SR1_200_300_pdr_reissued_2020_don.osler.rvt

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PRELIMINARY DESIGN
NOT FOR CONSTRUCTION

DESIGNER
DATE

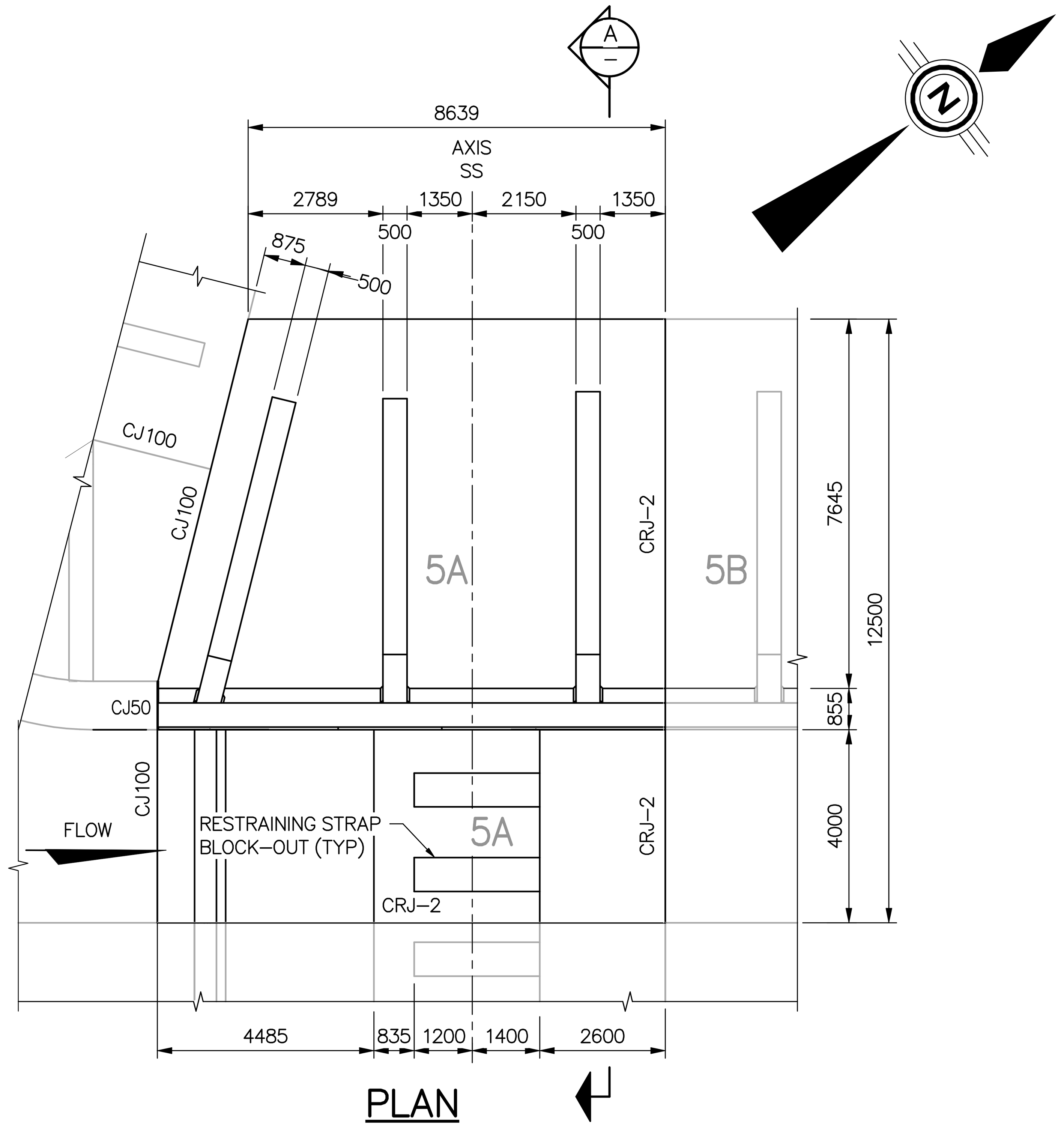
CHECKER
DATE

Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

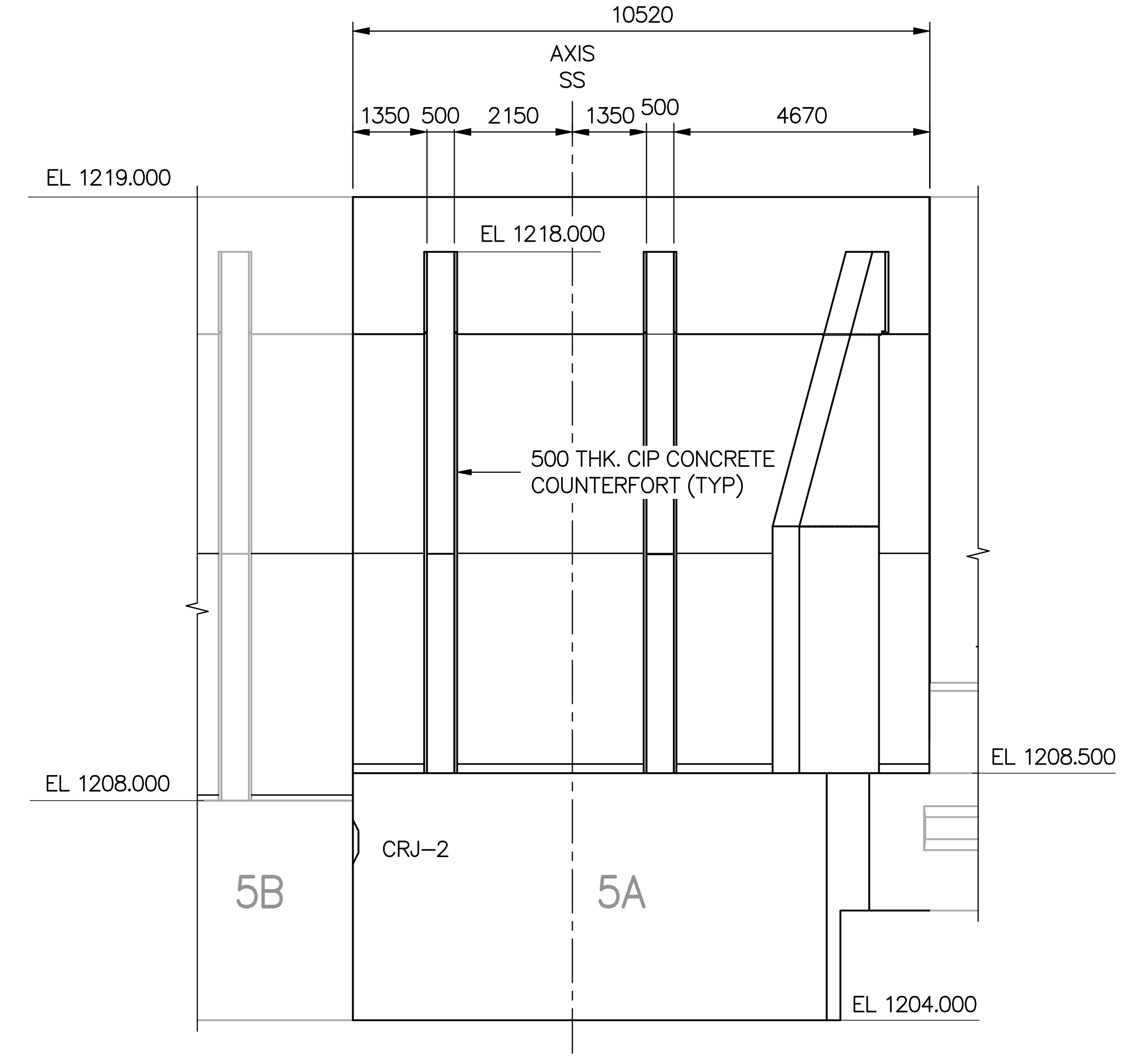
SERVICE SPILLWAY
STILLING BASIN - BLOCK 3B
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-232	



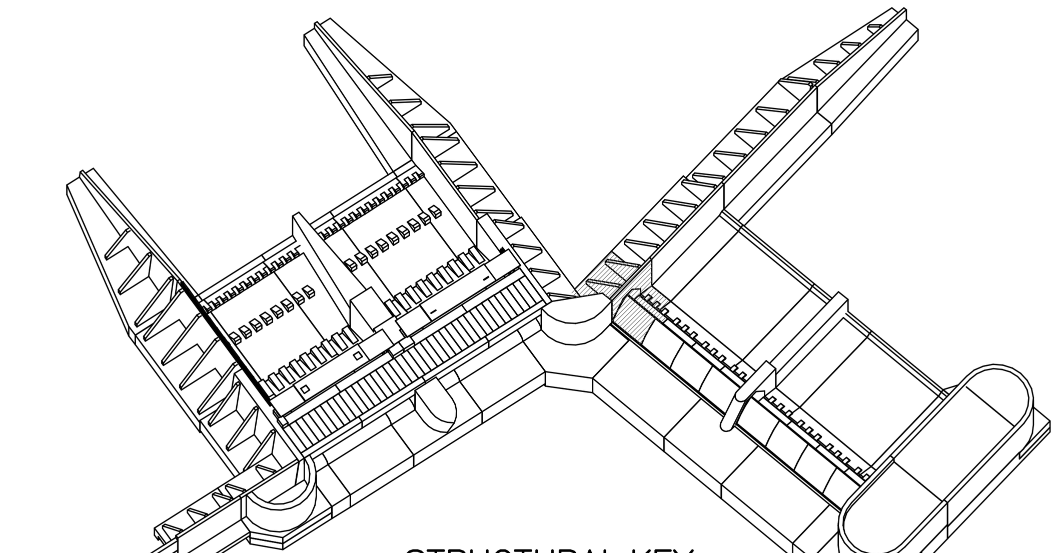
PLAN

1 : 100

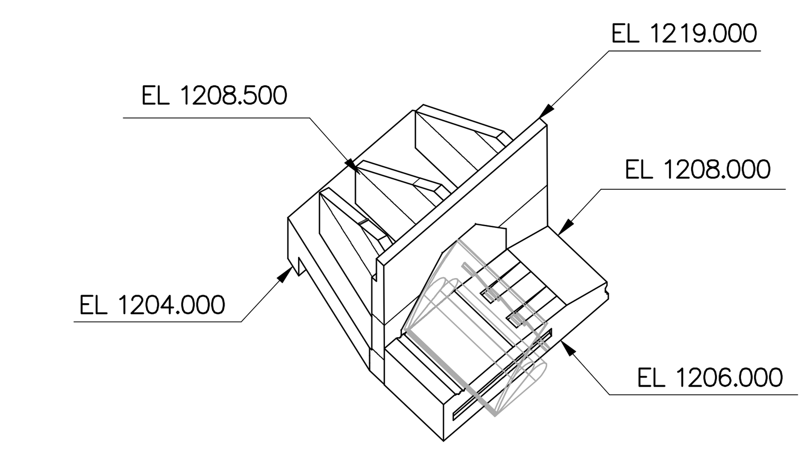


ELEVATION

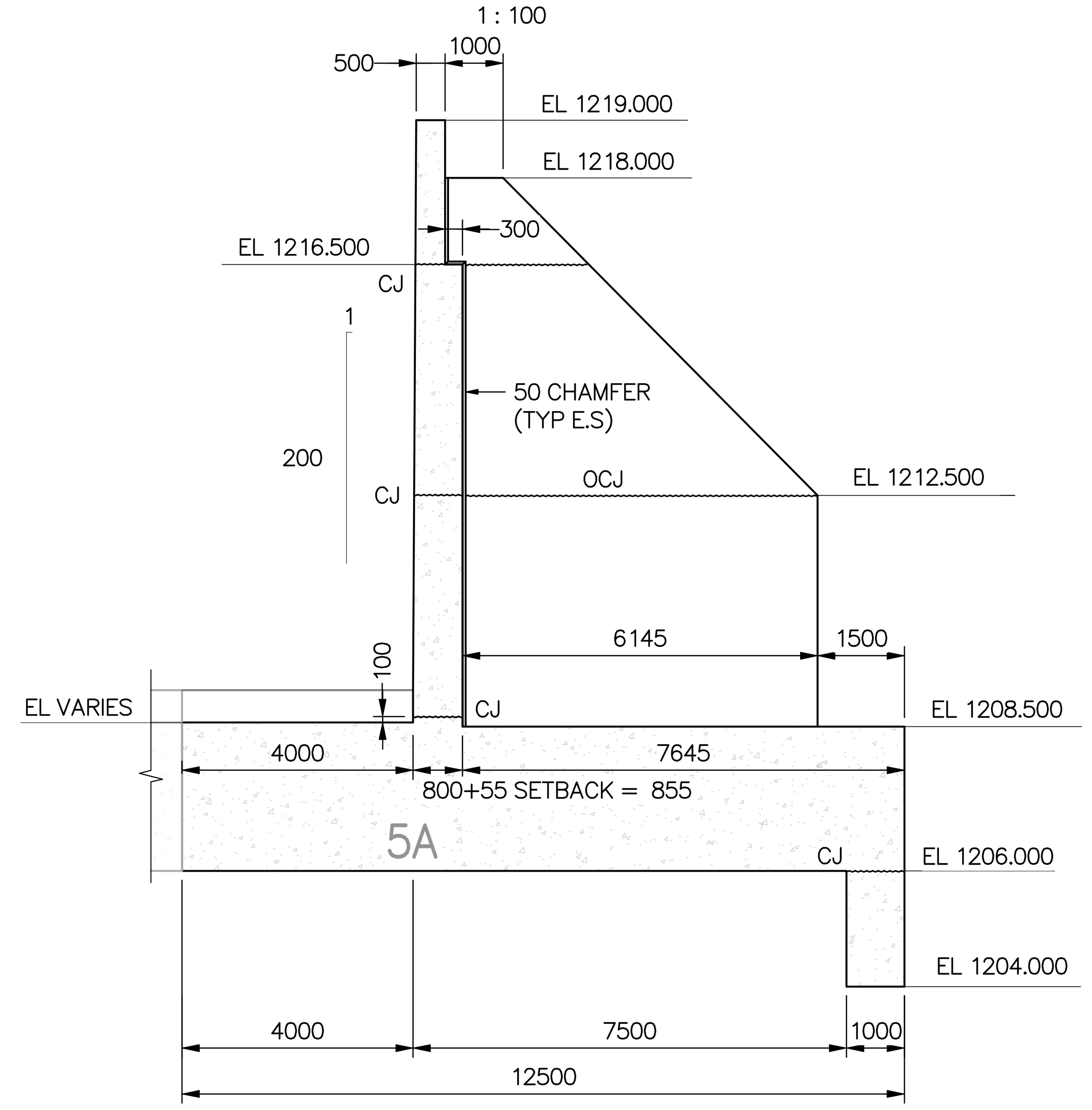
1 : 100



STRUCTURAL KEY



BLOCK 5A 3D VIEW



A
SECTION

1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.

2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
LEFT RETAINING WALL - BLOCK 5A
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-240	

Plot Date: 6/25/2020 2:38:07 PM Login: Dosler
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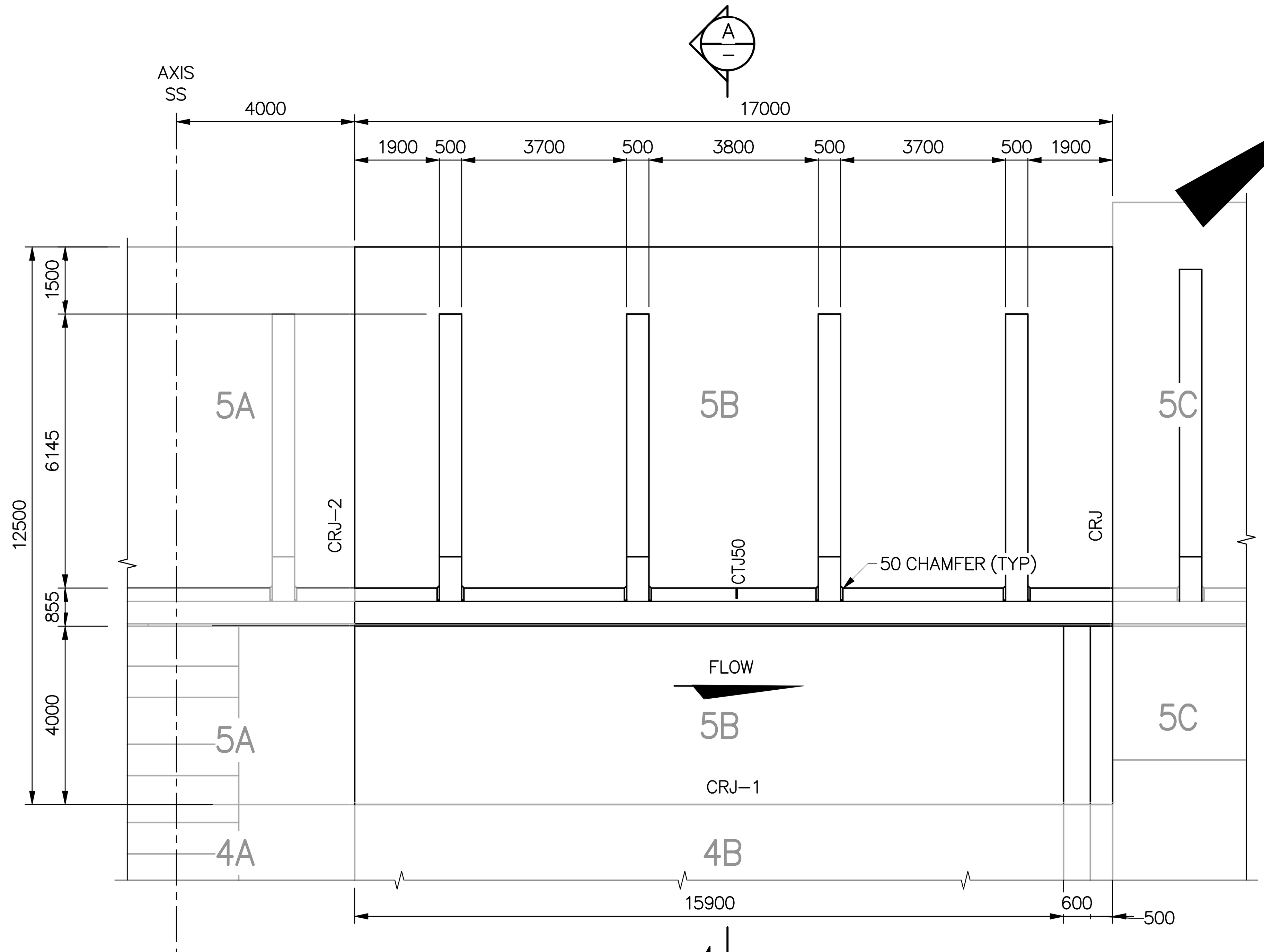
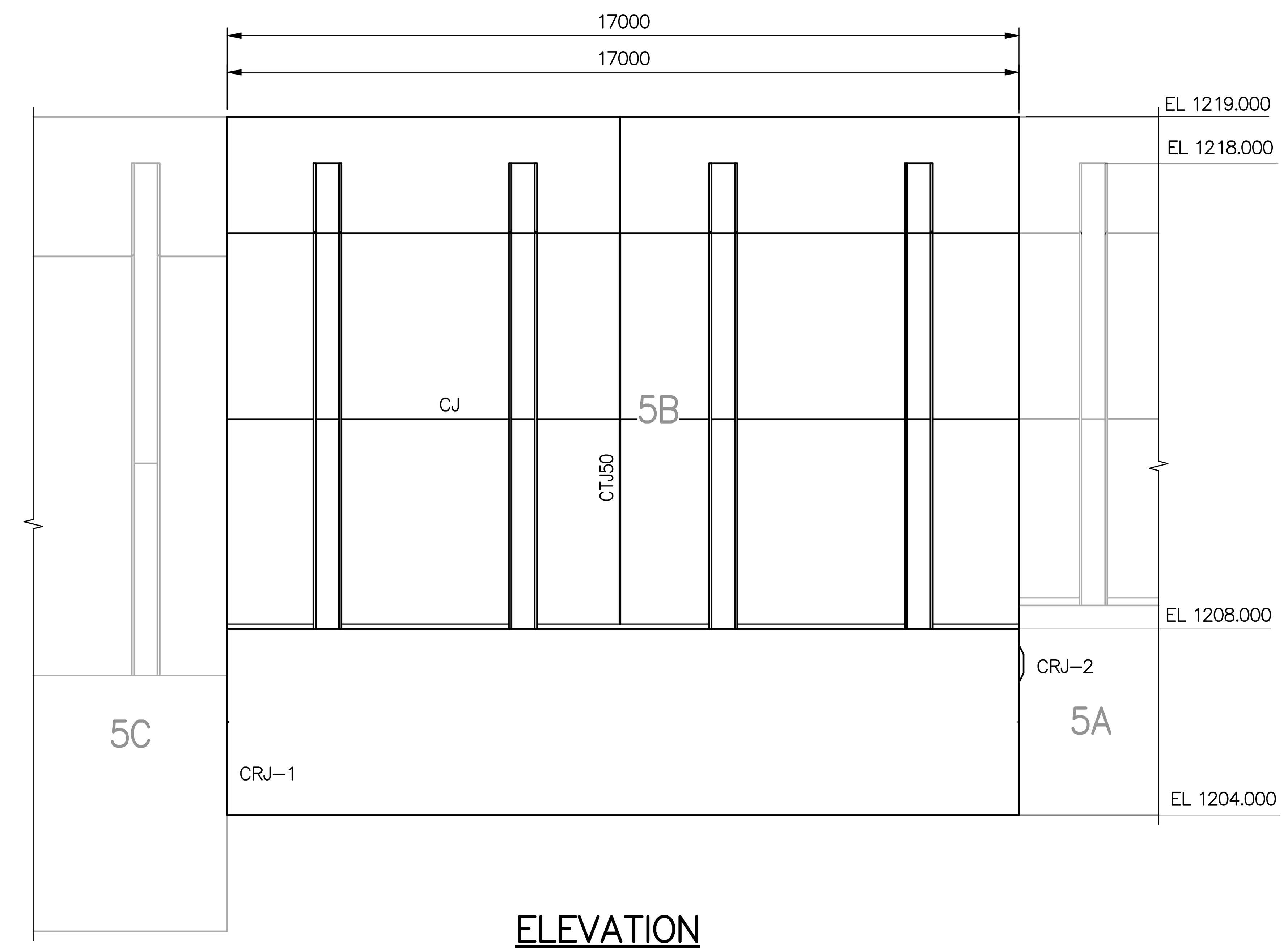
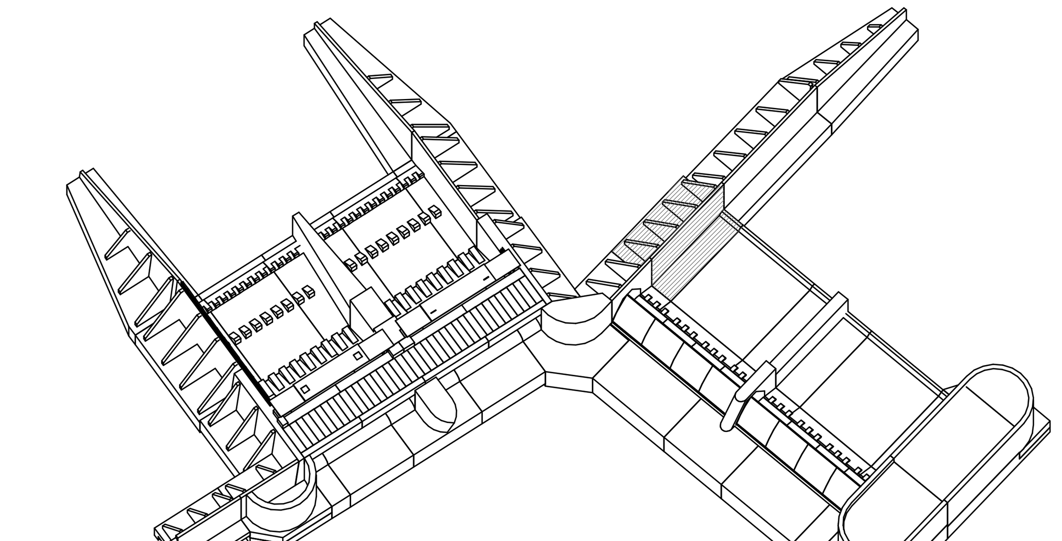


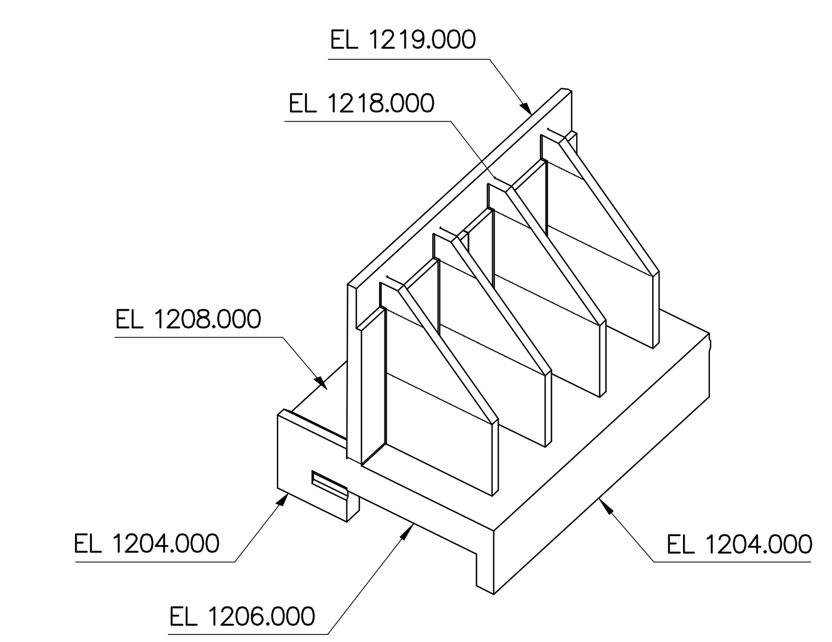
PLAN
1 : 100



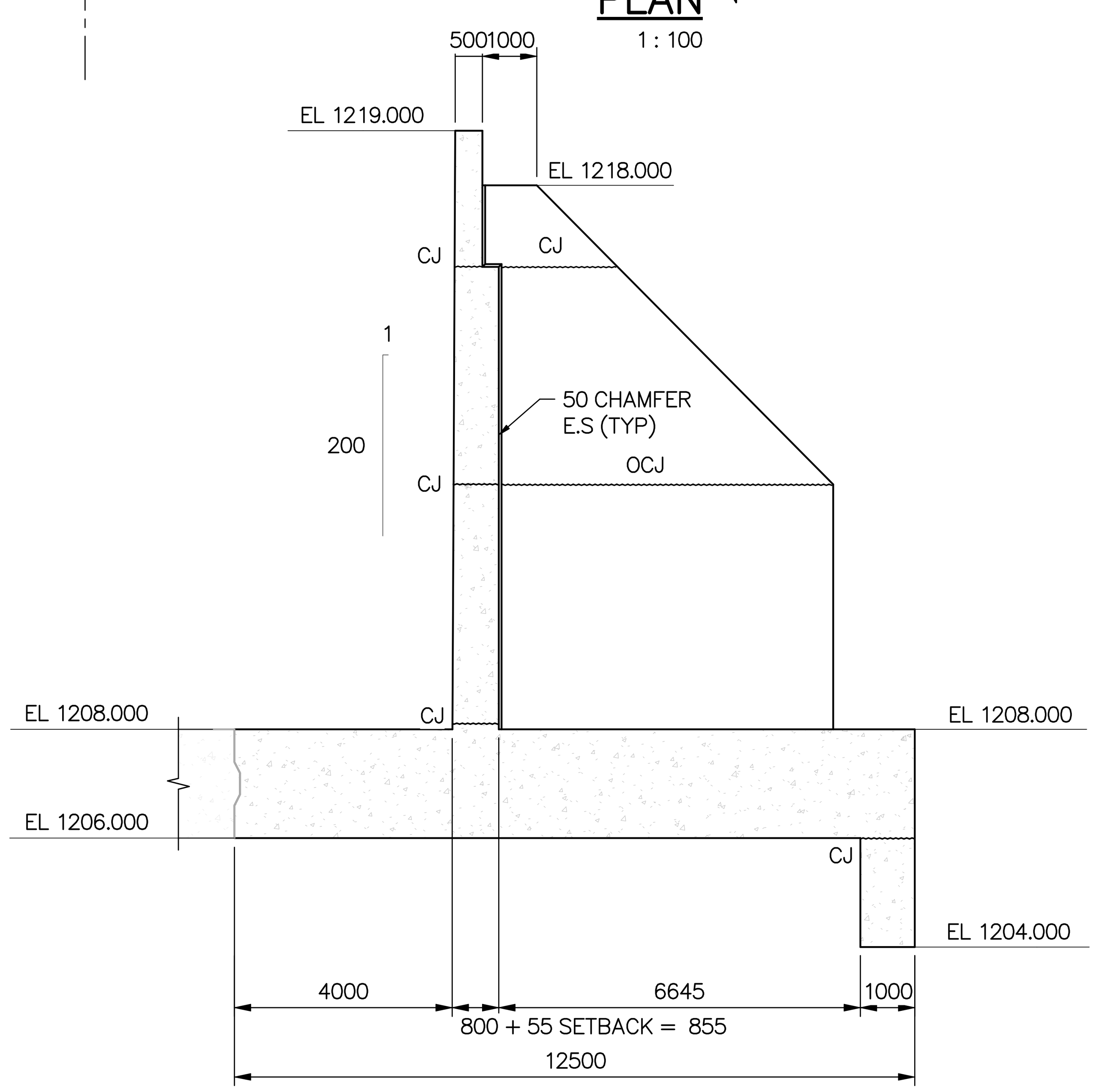
ELEVATION
1 : 100



STRUCTURAL KEY



BLOCK 5B 3D VIEW



A
–
SECTION
1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

SERVICE SPILLWAY
LEFT RETAINING WALL - BLOCK 5B
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-242	

Plot Date: 6/25/2020 2:38:08 PM Login: Dosler
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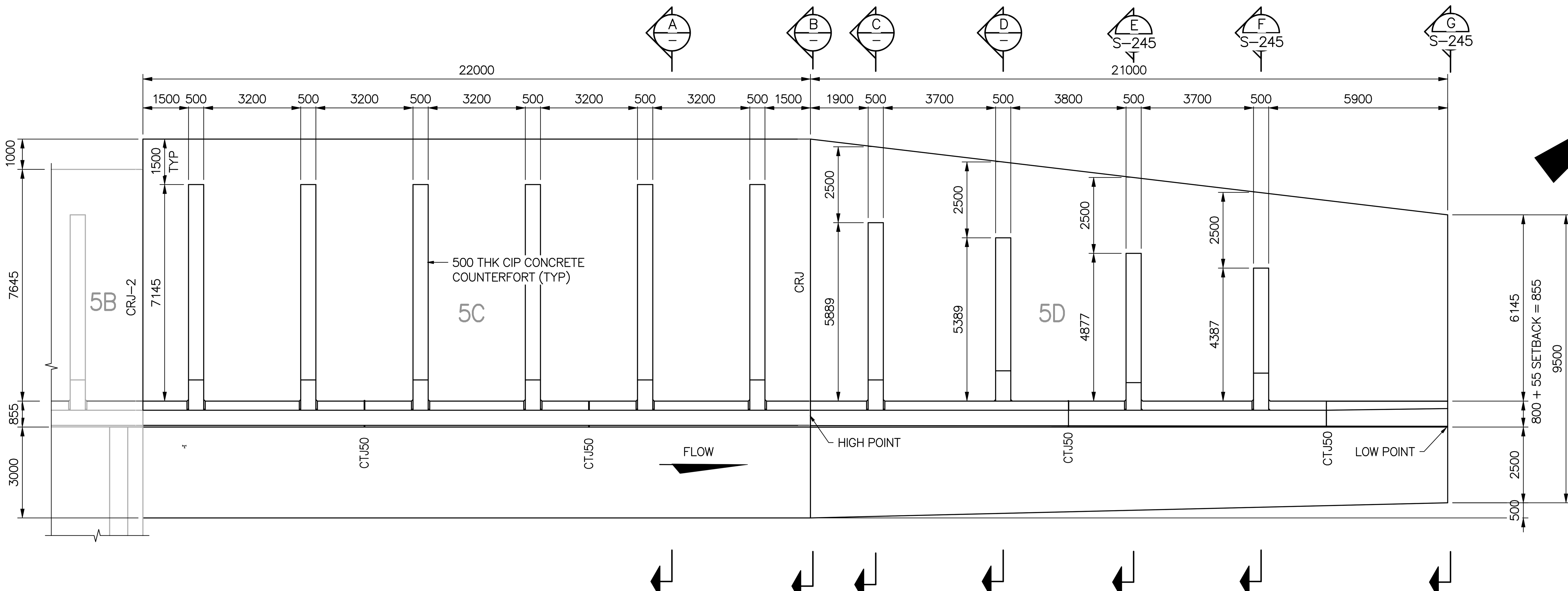
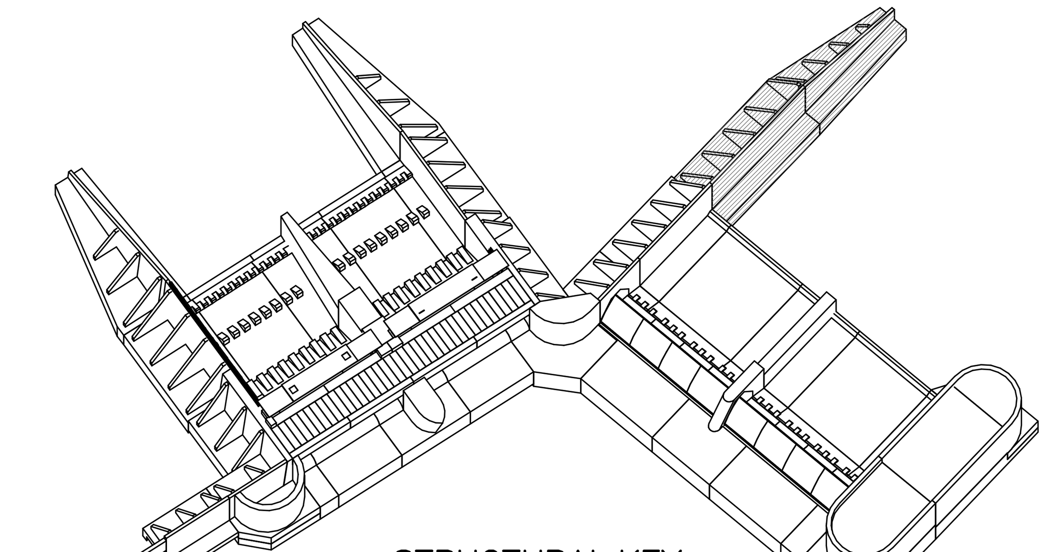


PLAN

1 : 100



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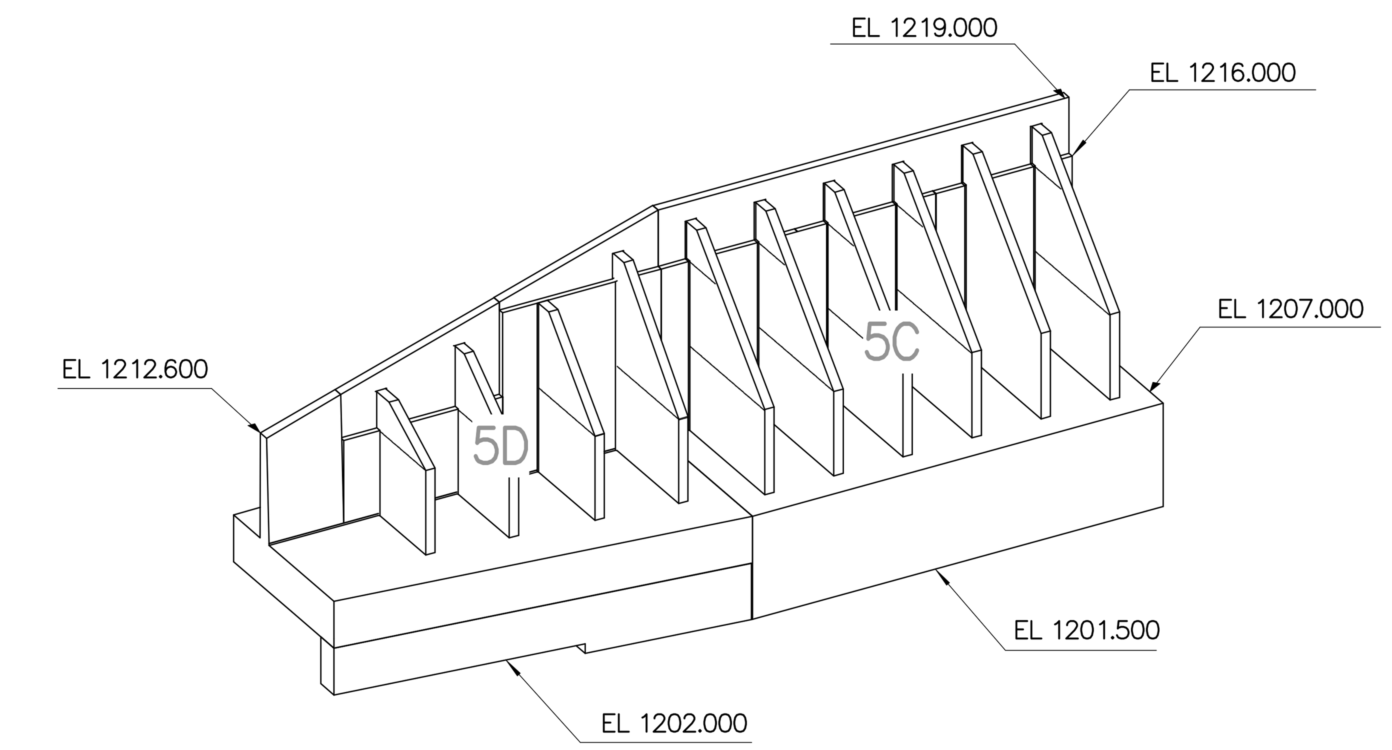
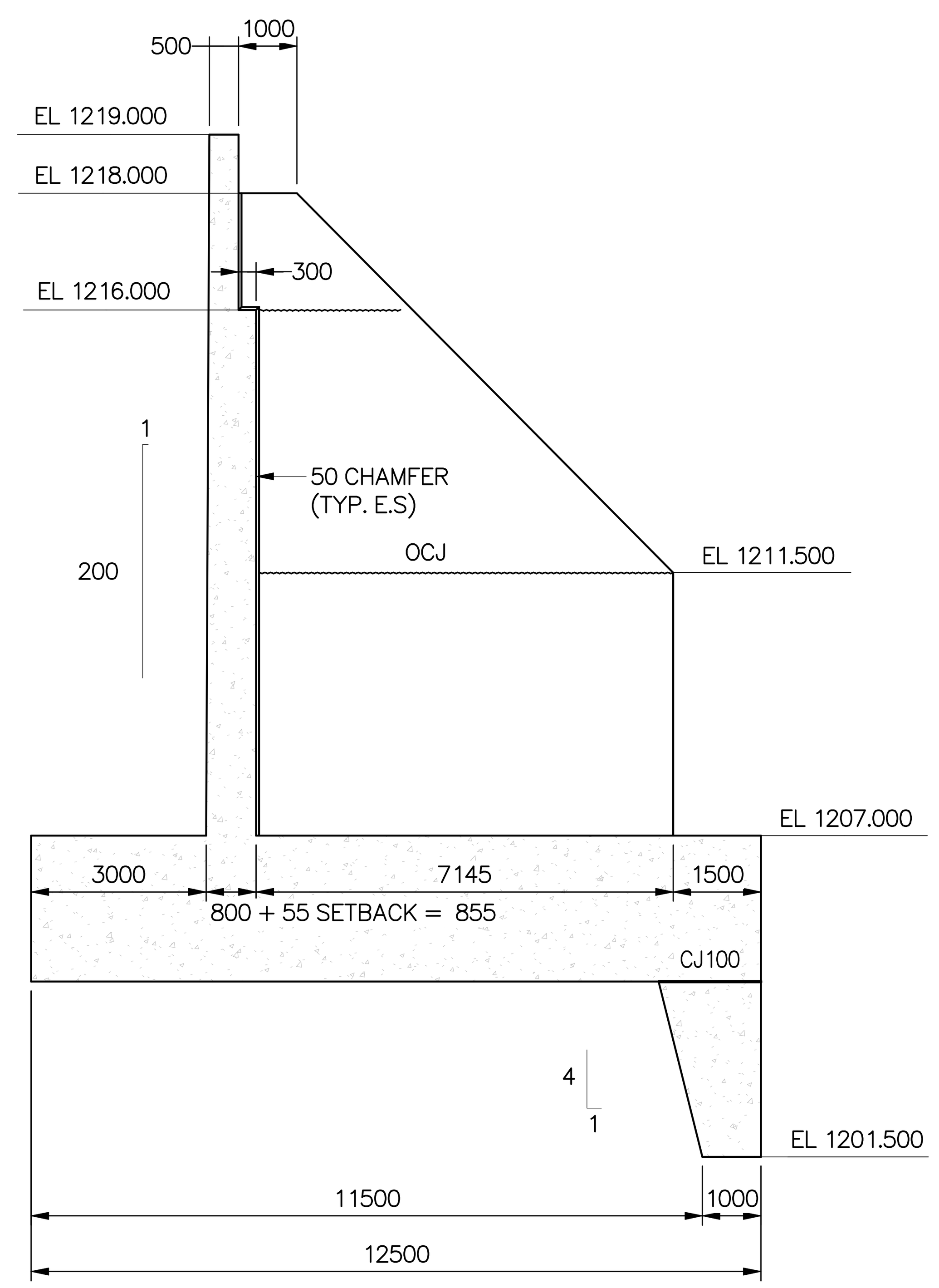
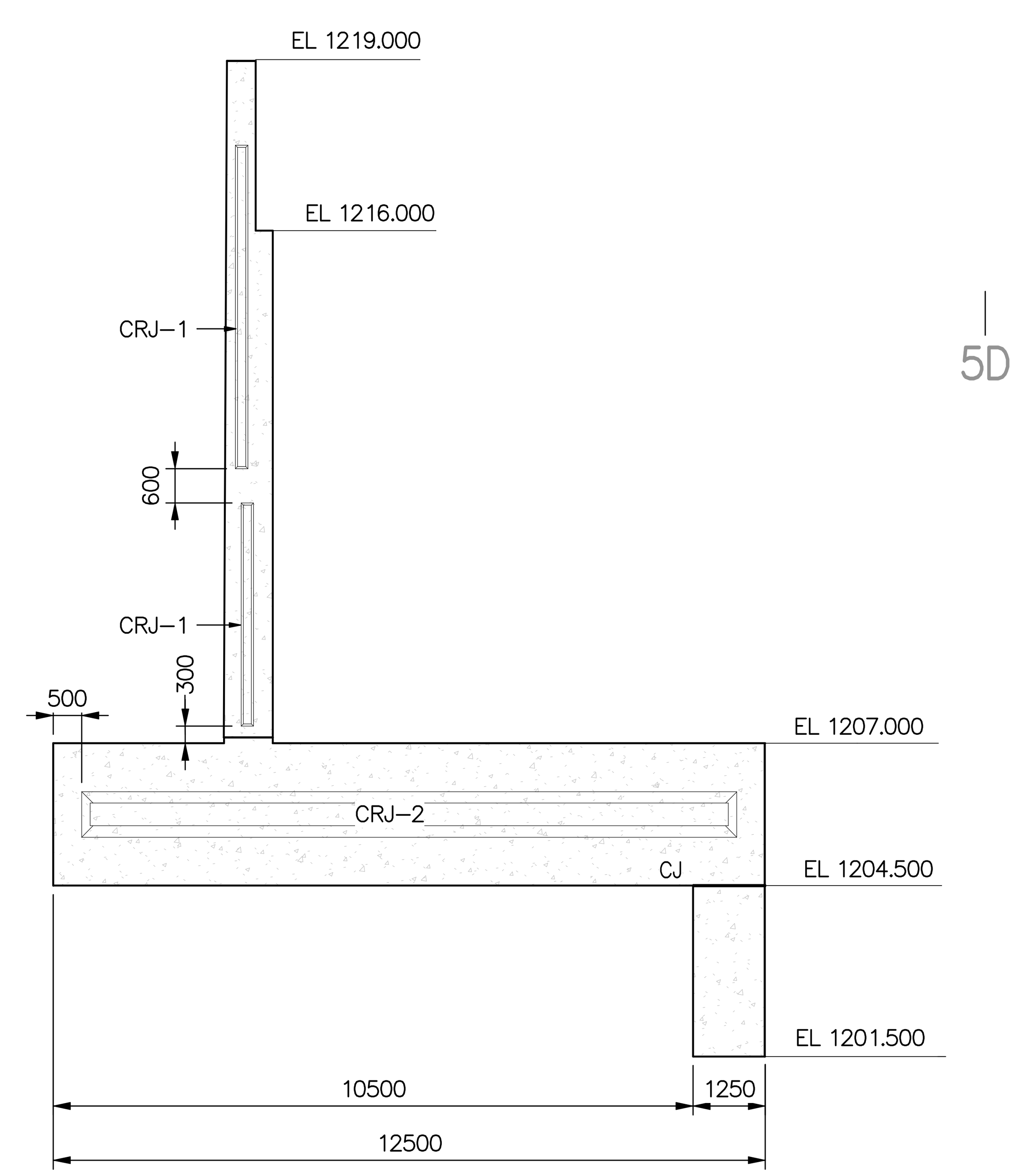


BLOCKS 5C & 5D 3D VIEW



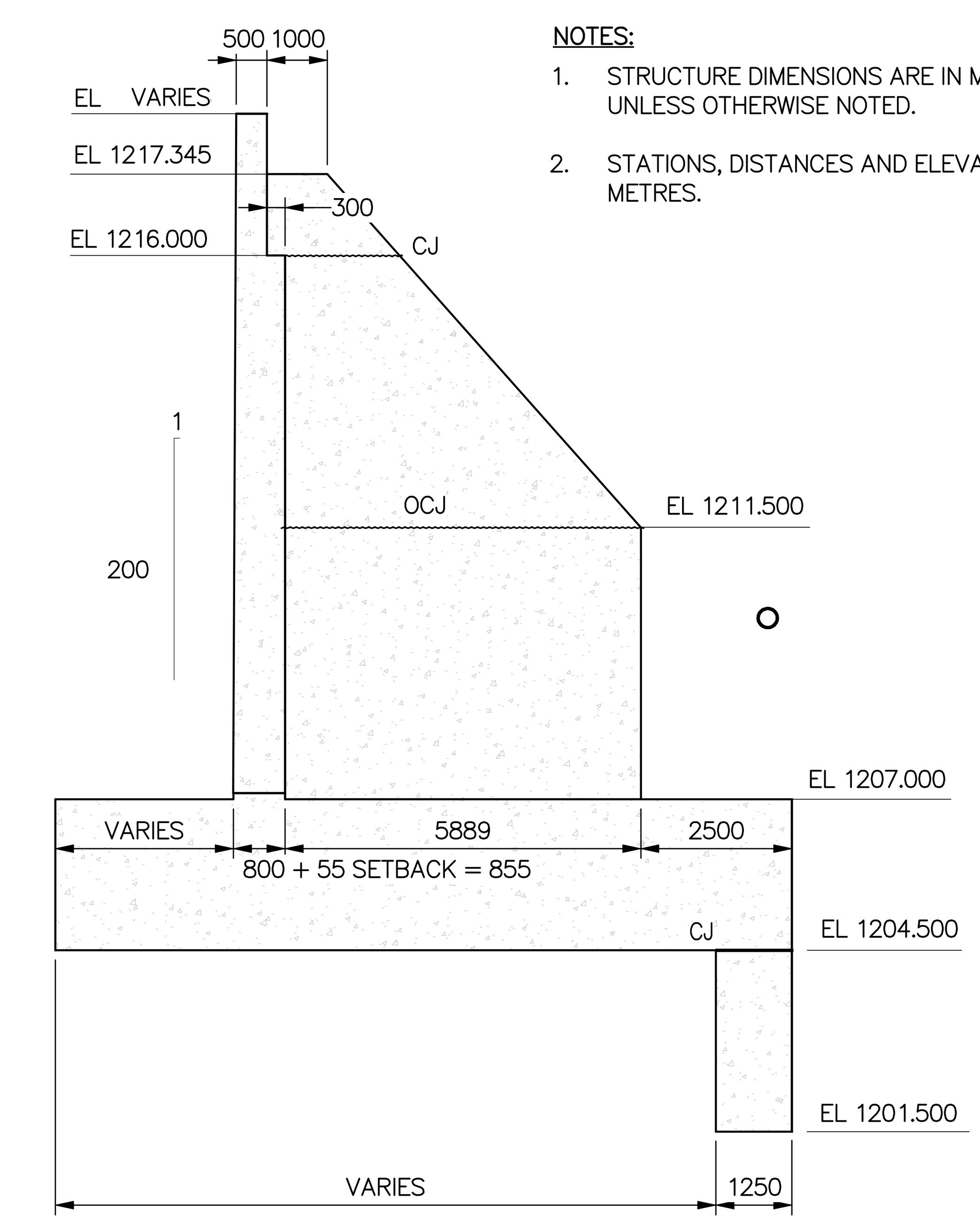
SECTION A

1 : 100



SECTION B

1 : 100

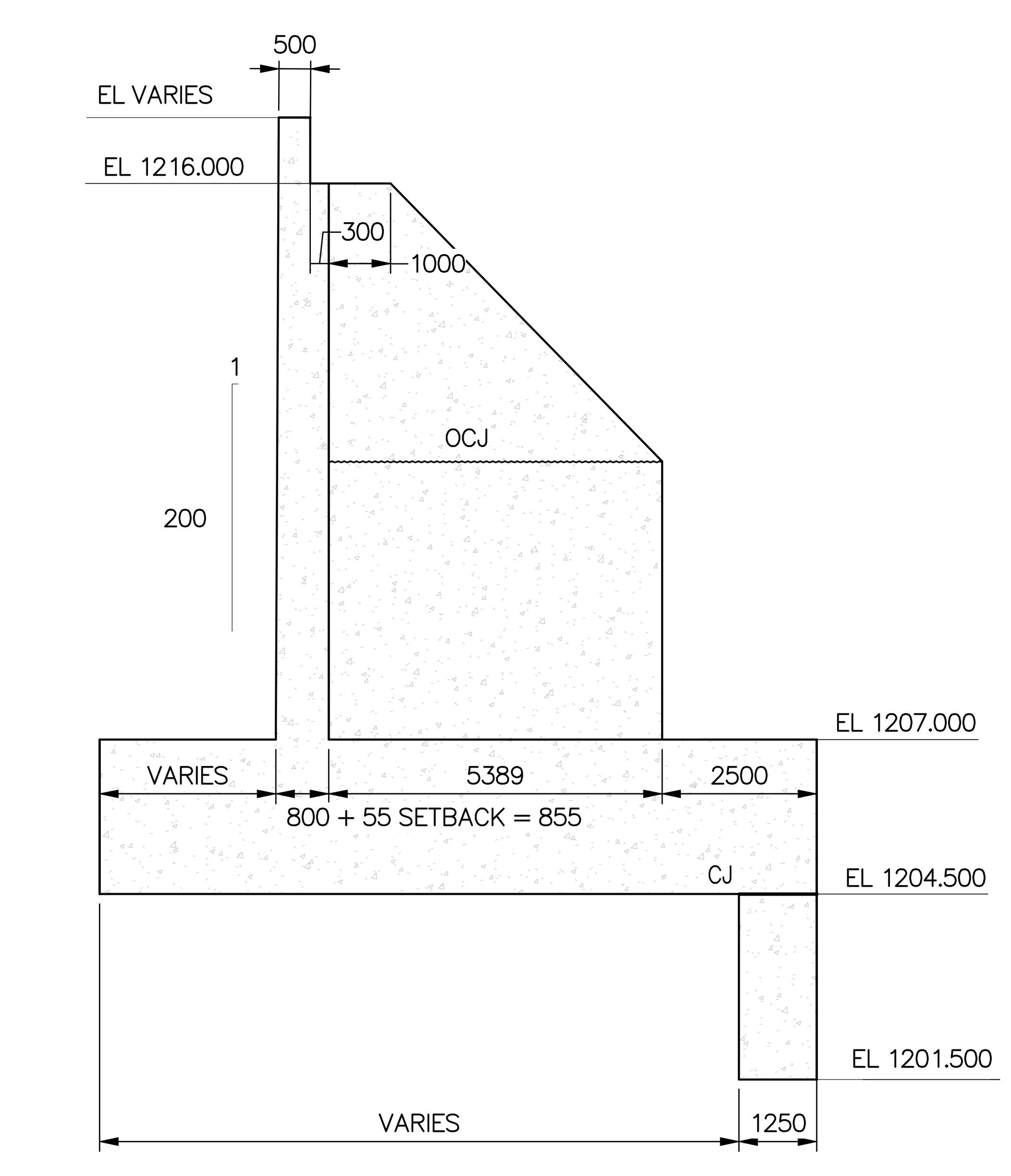


SECTION C

1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.



SECTION D

1 : 100

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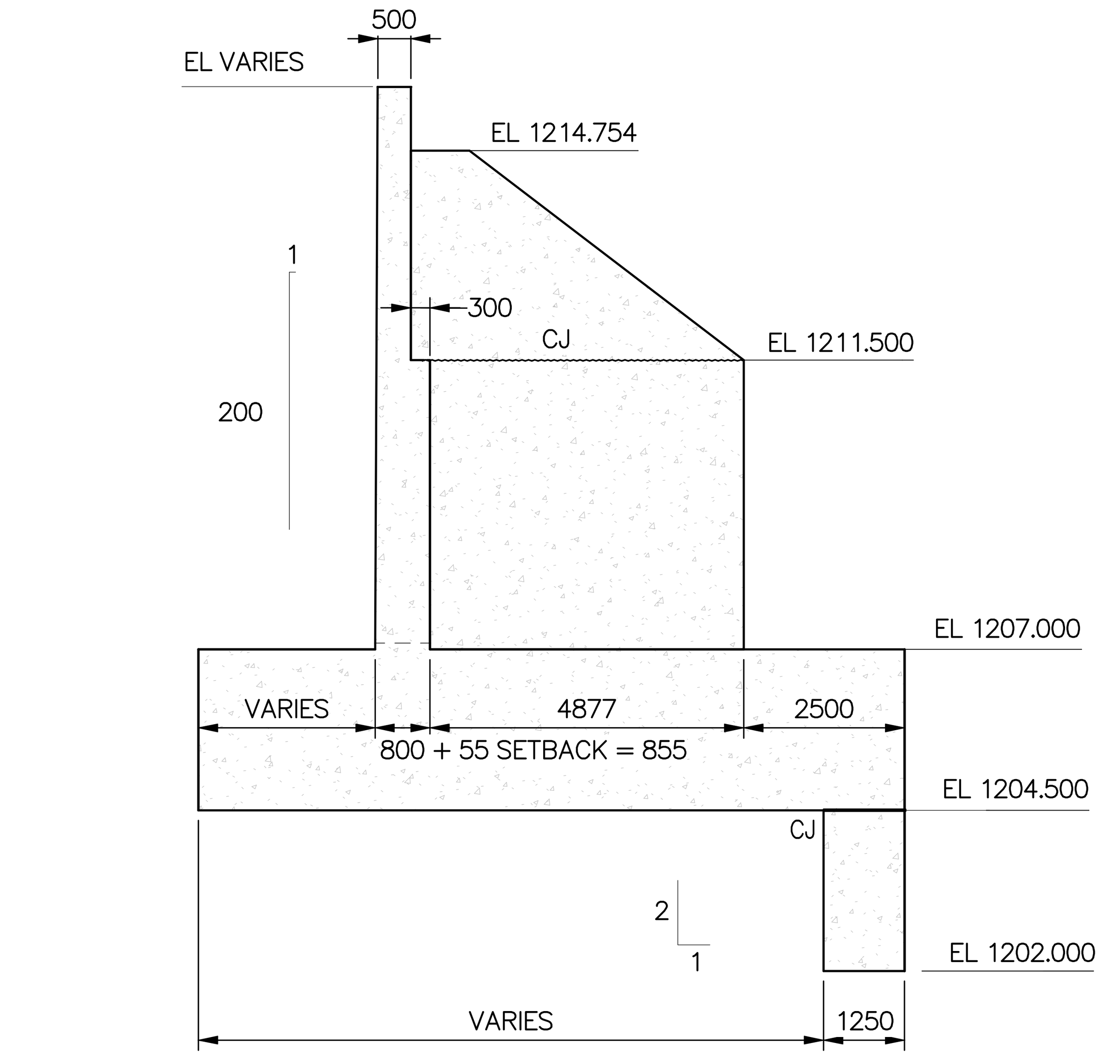
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

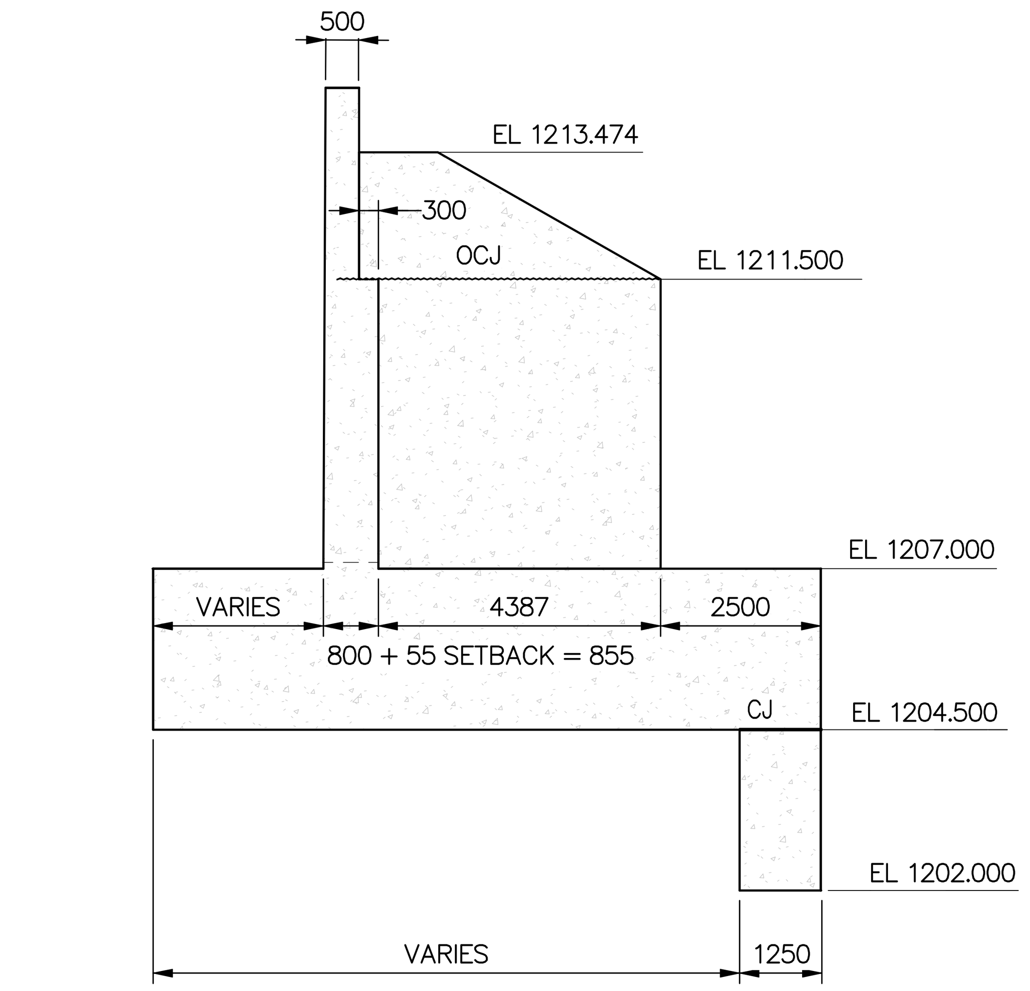
SERVICE SPILLWAY
LEFT RETAINING WALL - BLOCKS 5C & 5D
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-244	

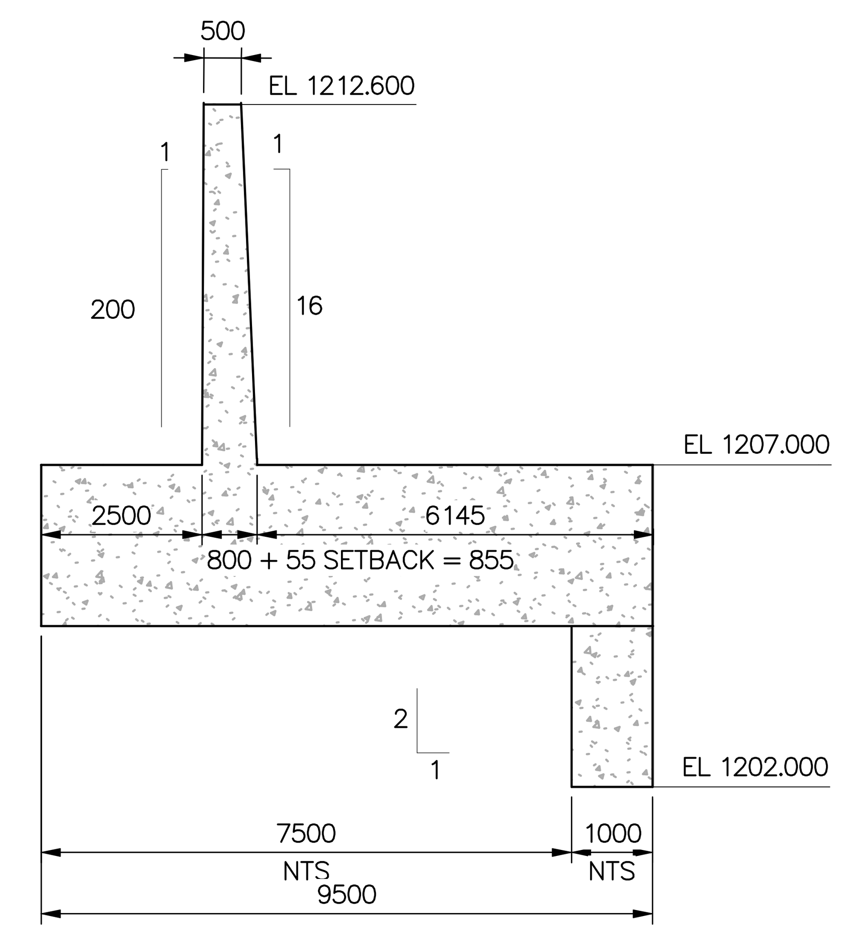
Plot Date: 6/25/2020 2:38:10 PM Login: Dosler
REVIT FILE No. BIM 360://110773396_SR1_springbank_offstream_storage/110773396_structure_SR1_200_300_pdr_reissued_2020_don.osler.rvt



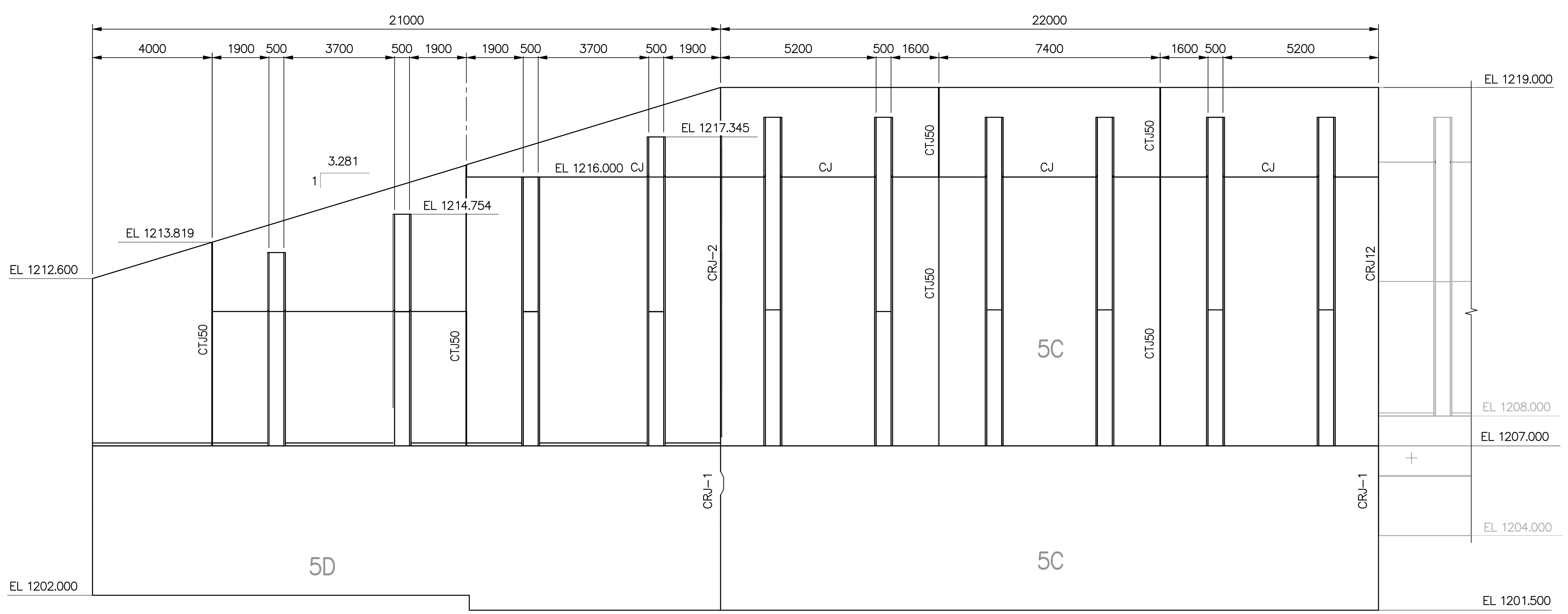
E
S–244
SECTION
1 : 100



F
S–244
SECTION
1 : 100



G
S–244
SECTION
1 : 100



ELEVATION
1 : 100

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

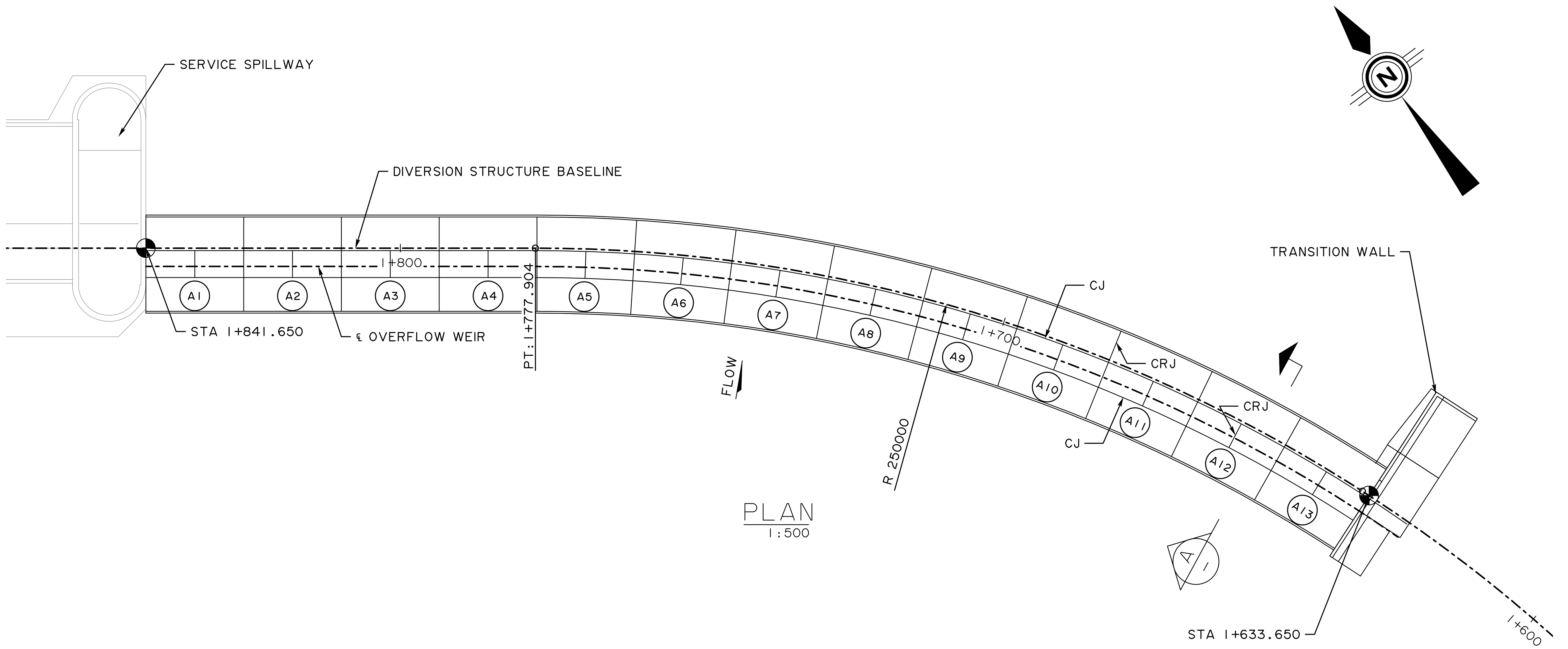
SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

SERVICE SPILLWAY
LEFT RETAINING WALL - BLOCKS 5C & 5D
CONCRETE OUTLINE - SHEET 2 OF 2

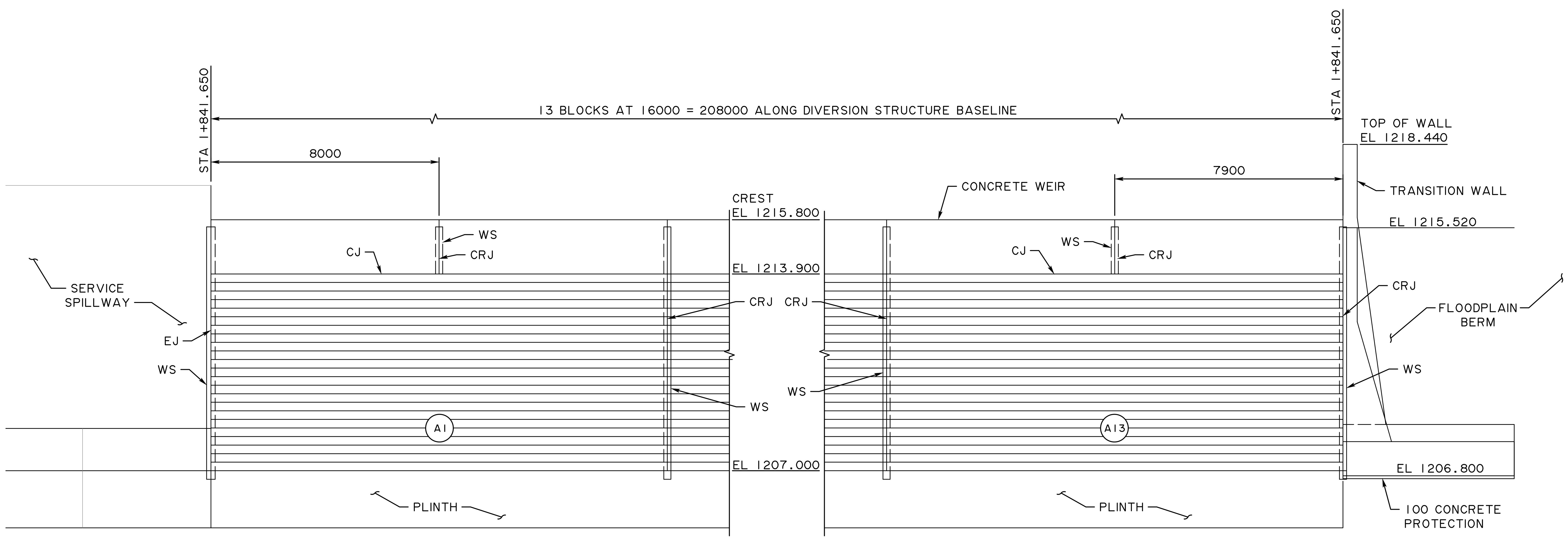
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-245	

Plot Date: 6/25/2020 2:38:11 PM Login: Dosler
REVIT FILE No. BIM 360://110773396_SR1_springbank_offstream_storage/110773396_structure_SR1_200_300_pdr_reissued_2020_don.osler.rvt

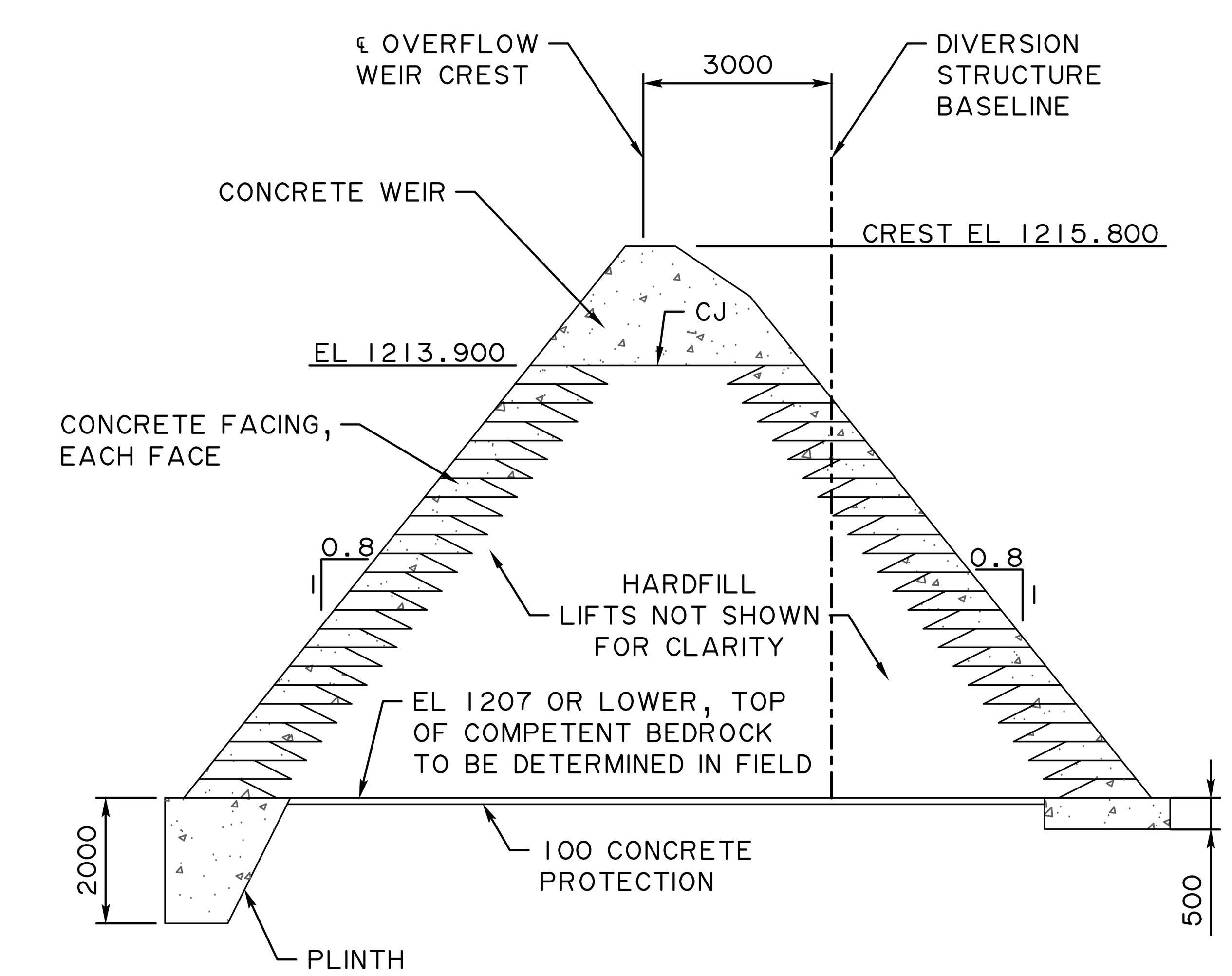
NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
3. REFER TO DRAWING C-201 FOR EXCAVATION, BACKFILL, AND FUSE PLUG FOR AUXILIARY SPILLWAY.



PLAN
1:500



ELEVATION – UPSTREAM
1:200



A SECTION
1:100

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

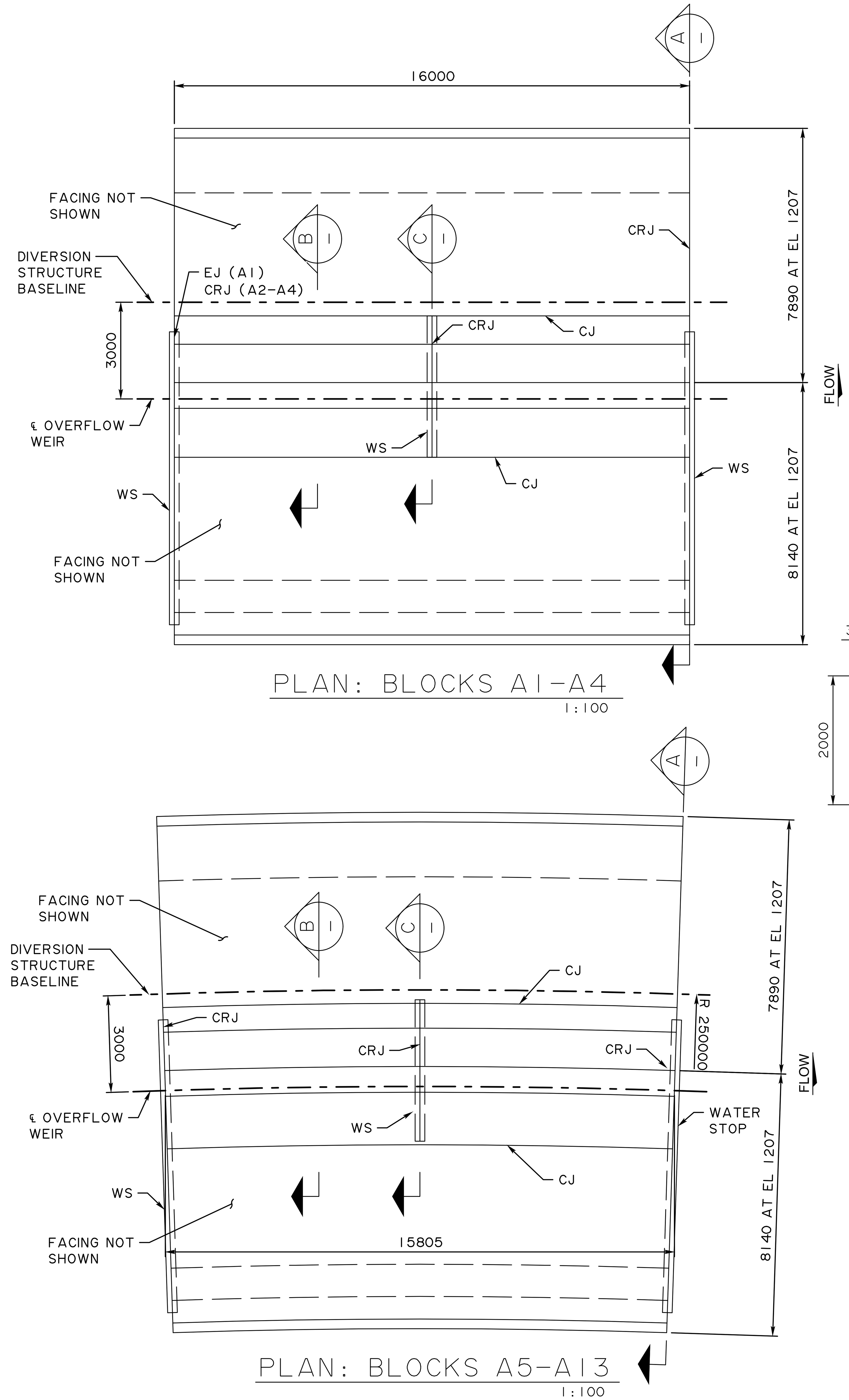
DIVERSION STRUCTURE
AUXILIARY SPILLWAY
STRUCTURAL ARRANGEMENT

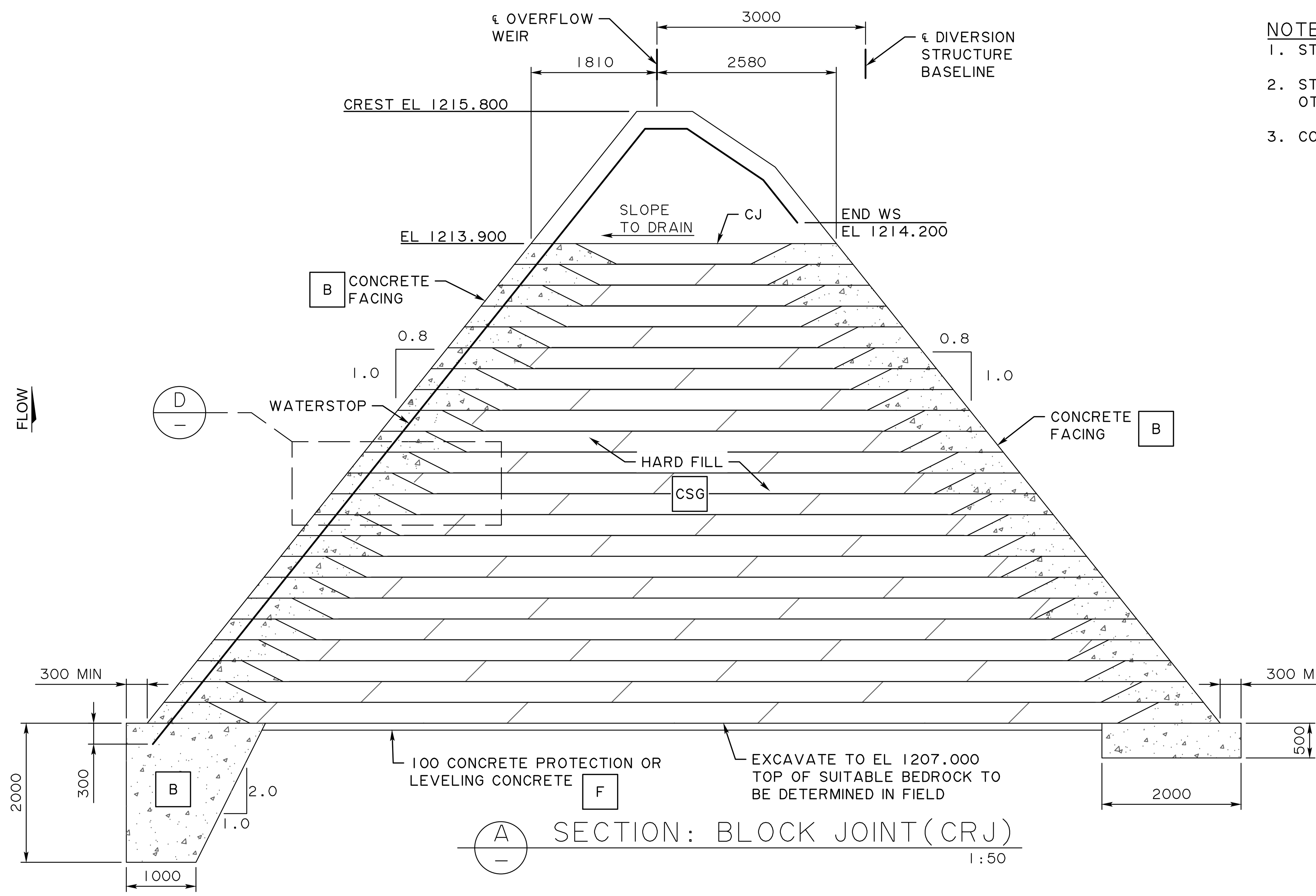
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-260	

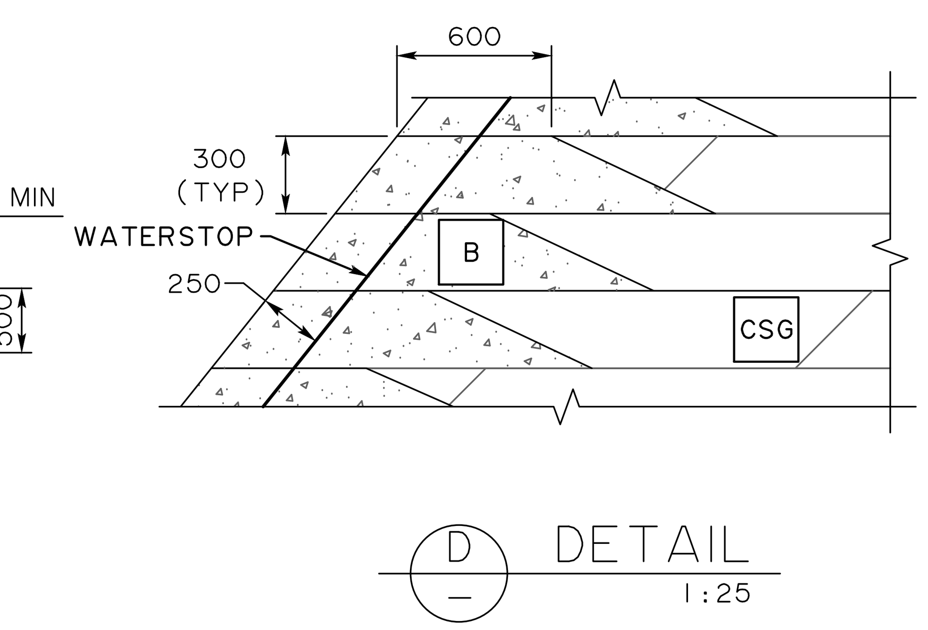
PlotDate: 2019.12.03 2:59 PM Login: Clinkenbeard, Adam TENDER No. ?
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\200_ds\200_ds_lexington\aux_spwy\73396S-260.dwg

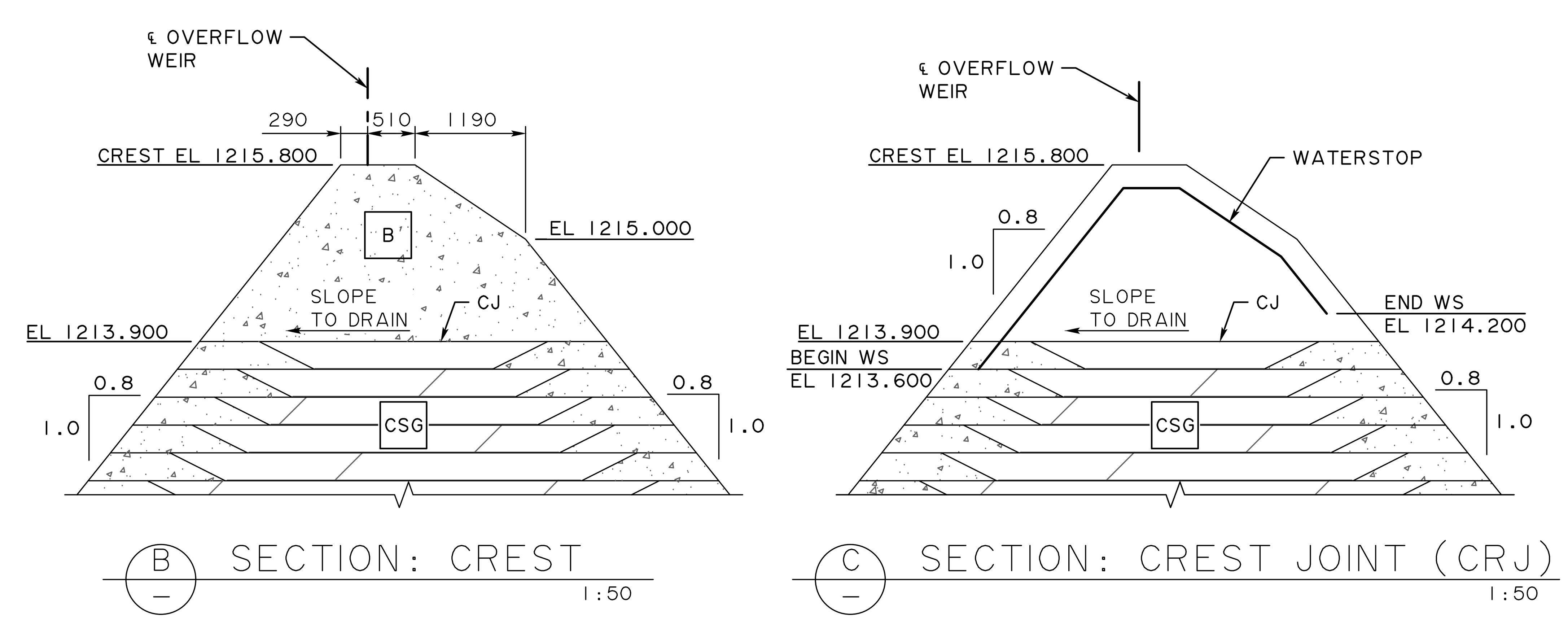
NOTES:

1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
3. CONCRETE MIX DESIGNATED BY [B].









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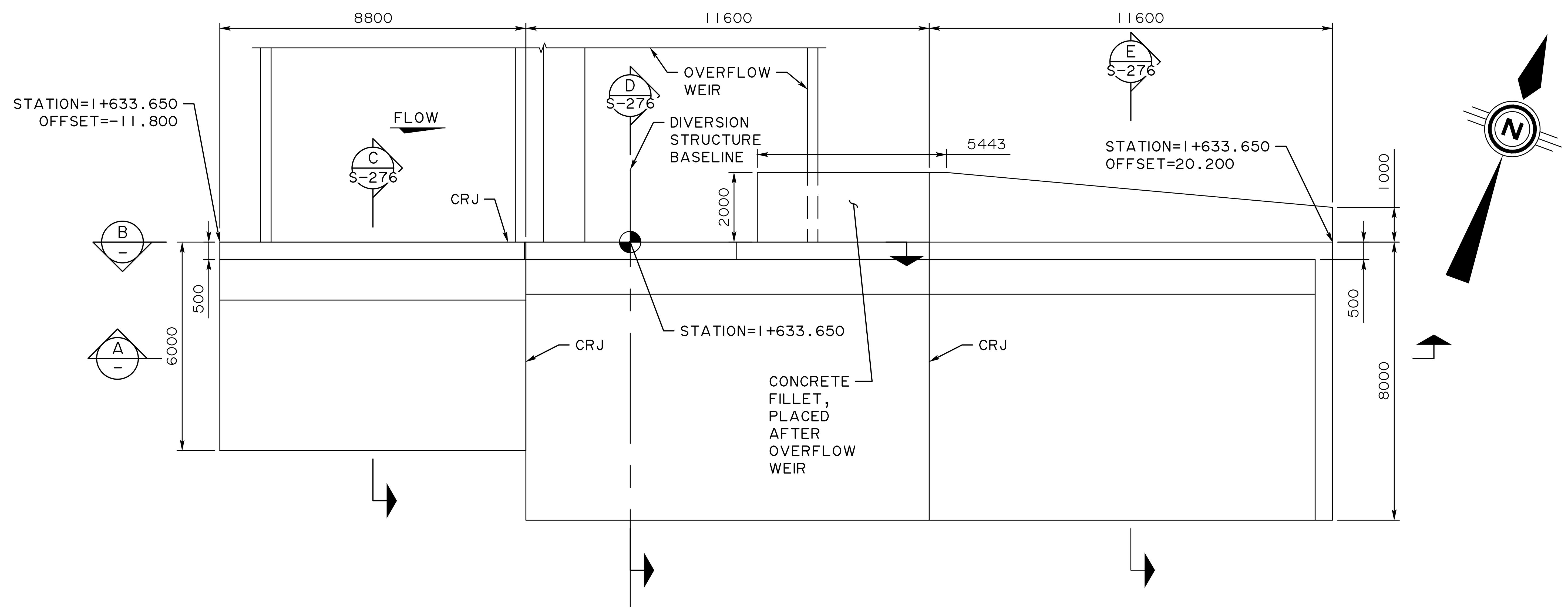
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

AUXILIARY SPILLWAY
OVERFLOW WIER
BLOCKS A1 TO A13

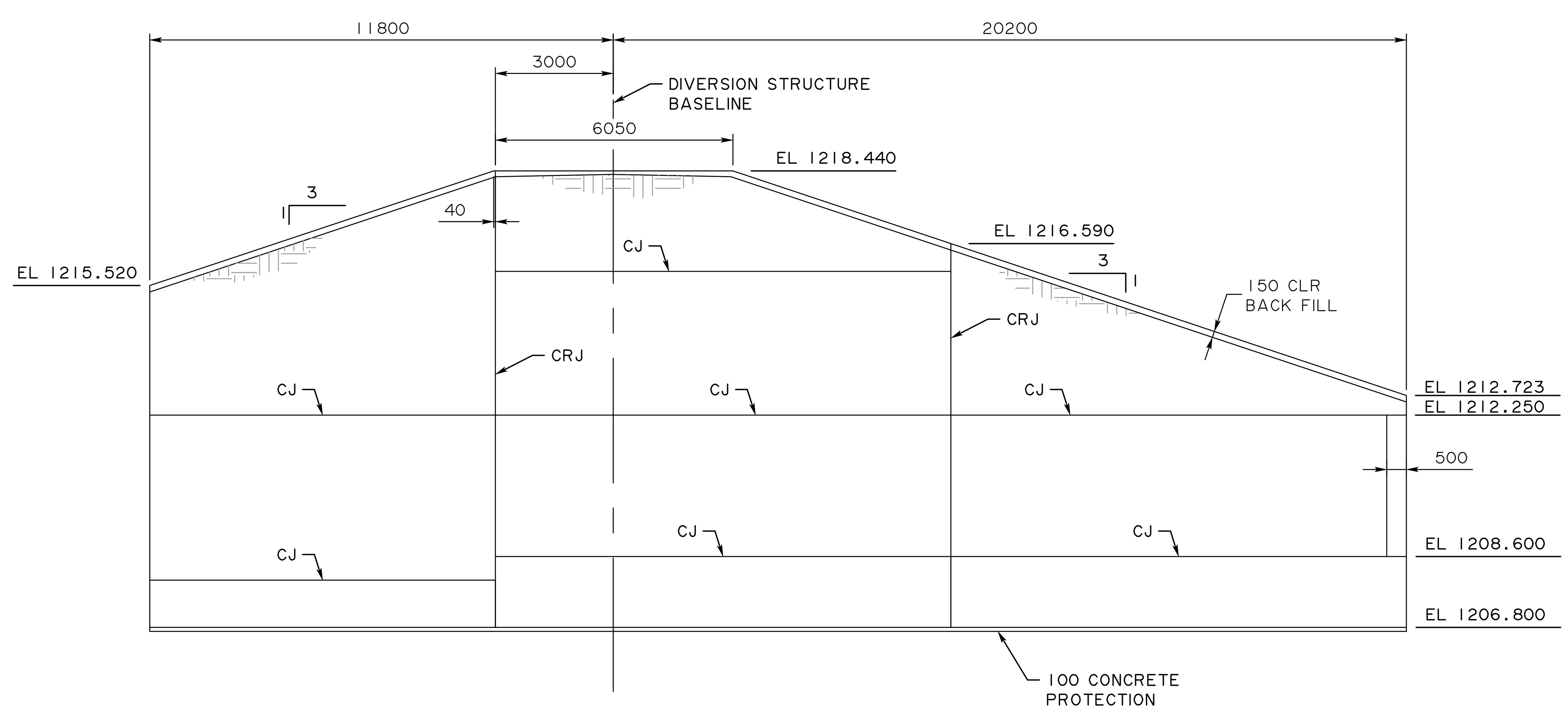
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-270	

PlotDate: 2019.12.03 2:59 PM Login: Shelton, Ben
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\200_ds\200_ds_lexington\aux_spwy\73396S-270.dwg
TENDER No. ?

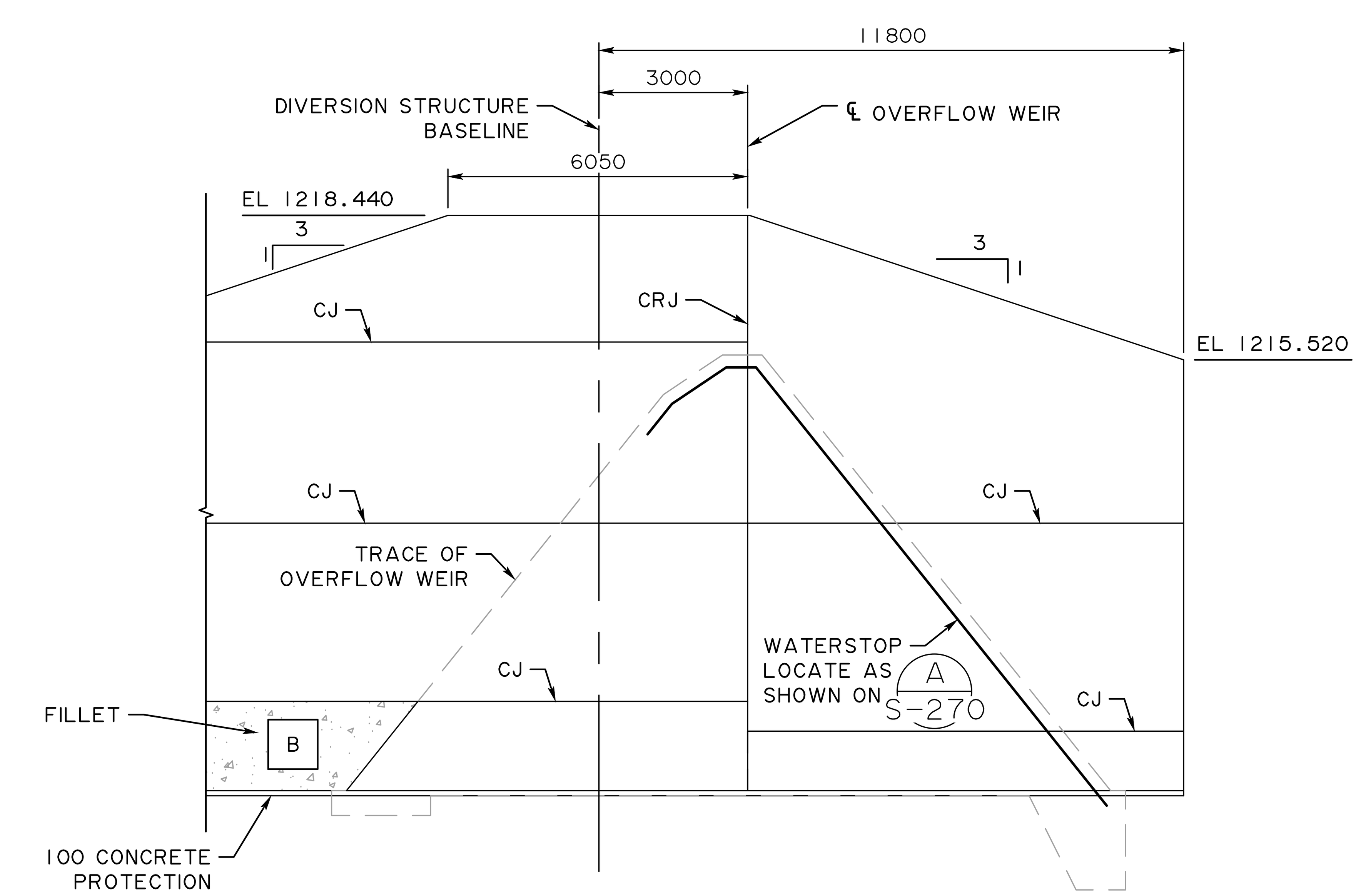
NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.



PLAN - TRANSITION WALL
1:100



A SECTION
1:100



B SECTION - BLOCK JOINT (CRJ)
1:100

NOTE:
CONSTRUCT TRANSITION WALL BEFORE OVERFLOW WEIR. DO NOT BACKFILL WALL UNTIL OVERFLOW WEIR BLOCK A13 IS COMPLETED.

PlotDate: 2019.12.03 3:01 PM Login: Shelton, Ben
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\200_ds\200_ds_lexington\aux_spwy\73396S-275-276.dwg TENDER No. ?

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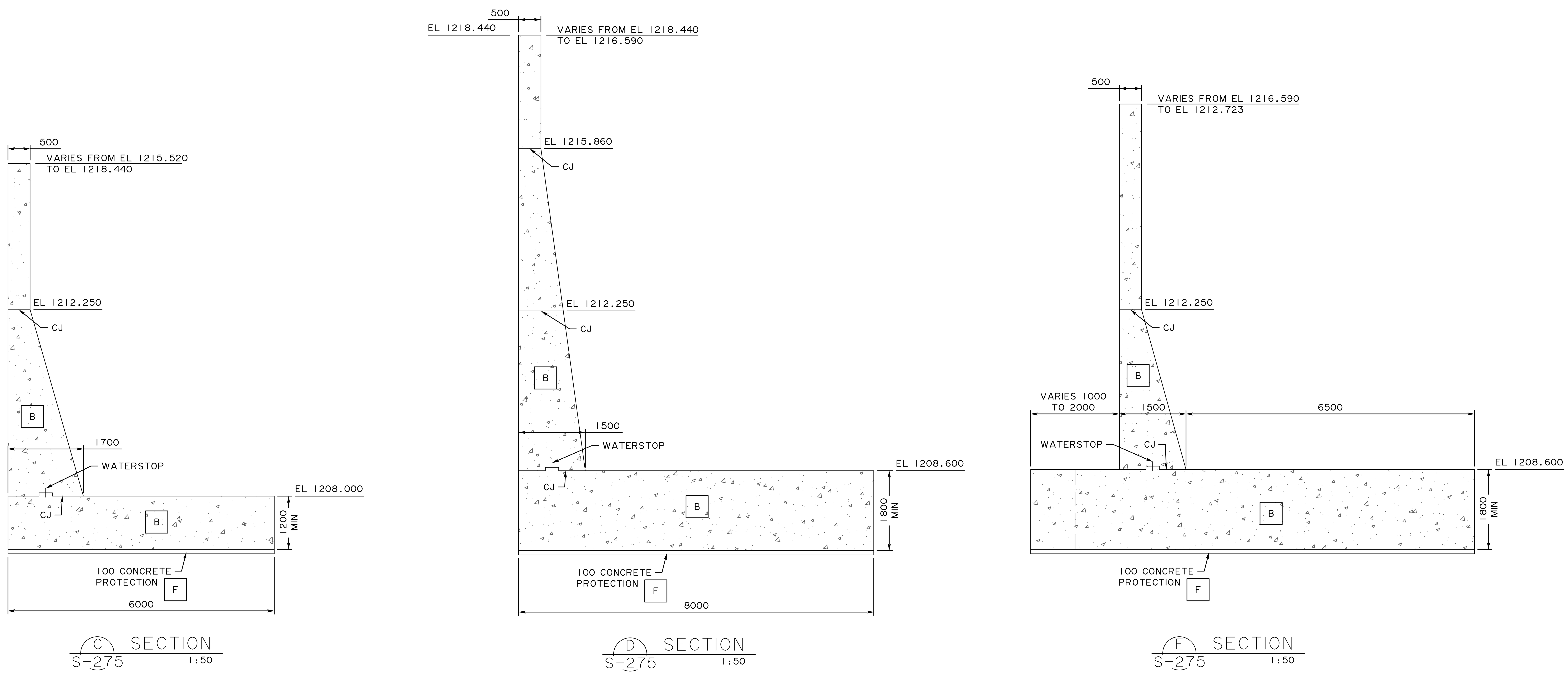
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

AUXILIARY SPILLWAY
TRANSITION WALL
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-275	

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.
3. CONCRETE MIX DESIGNATED BY B.



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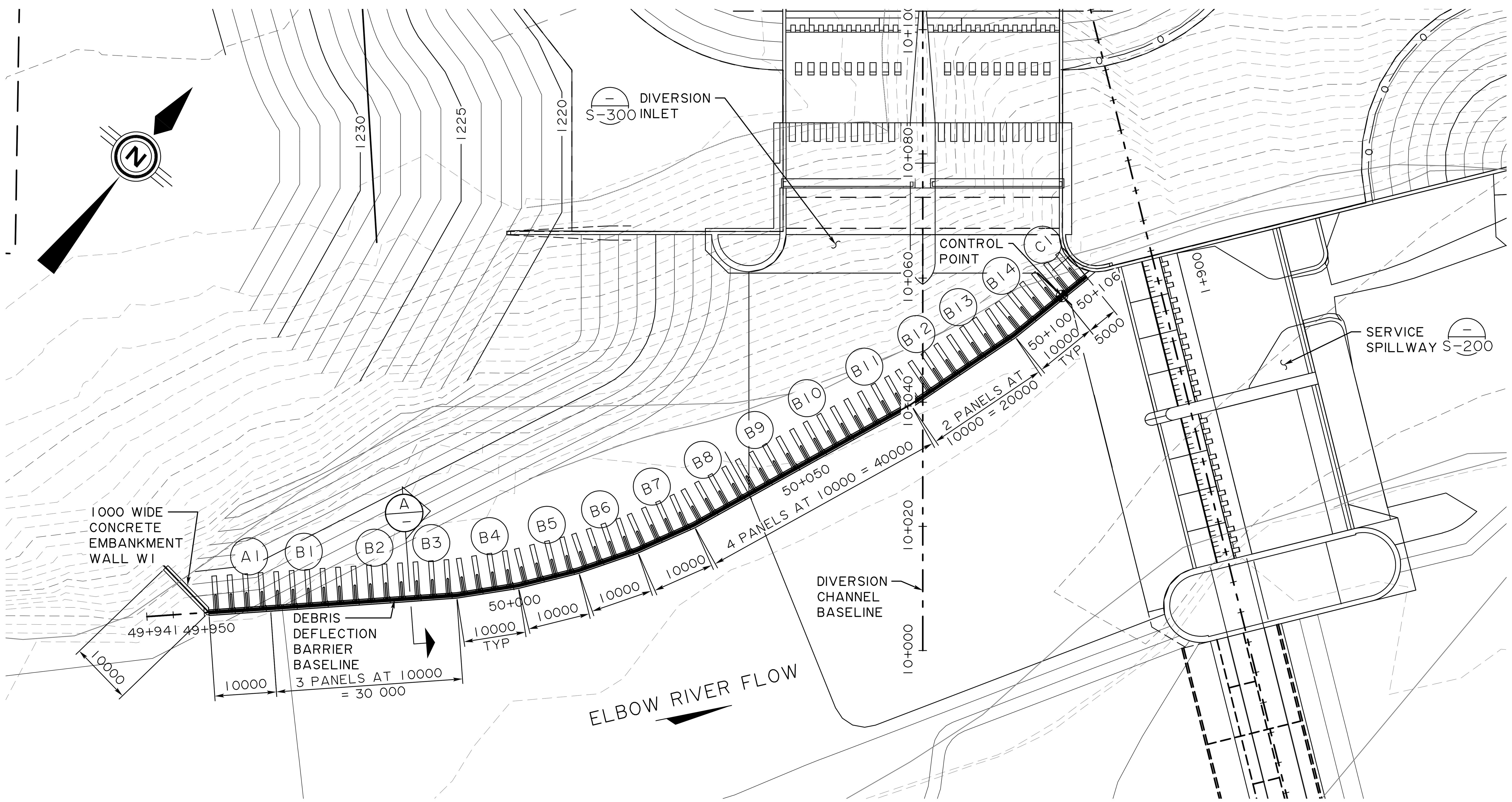
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

AUXILIARY SPILLWAY
TRANSITION WALL
CONCRETE OUTLINE - SHEET 2 OF 2

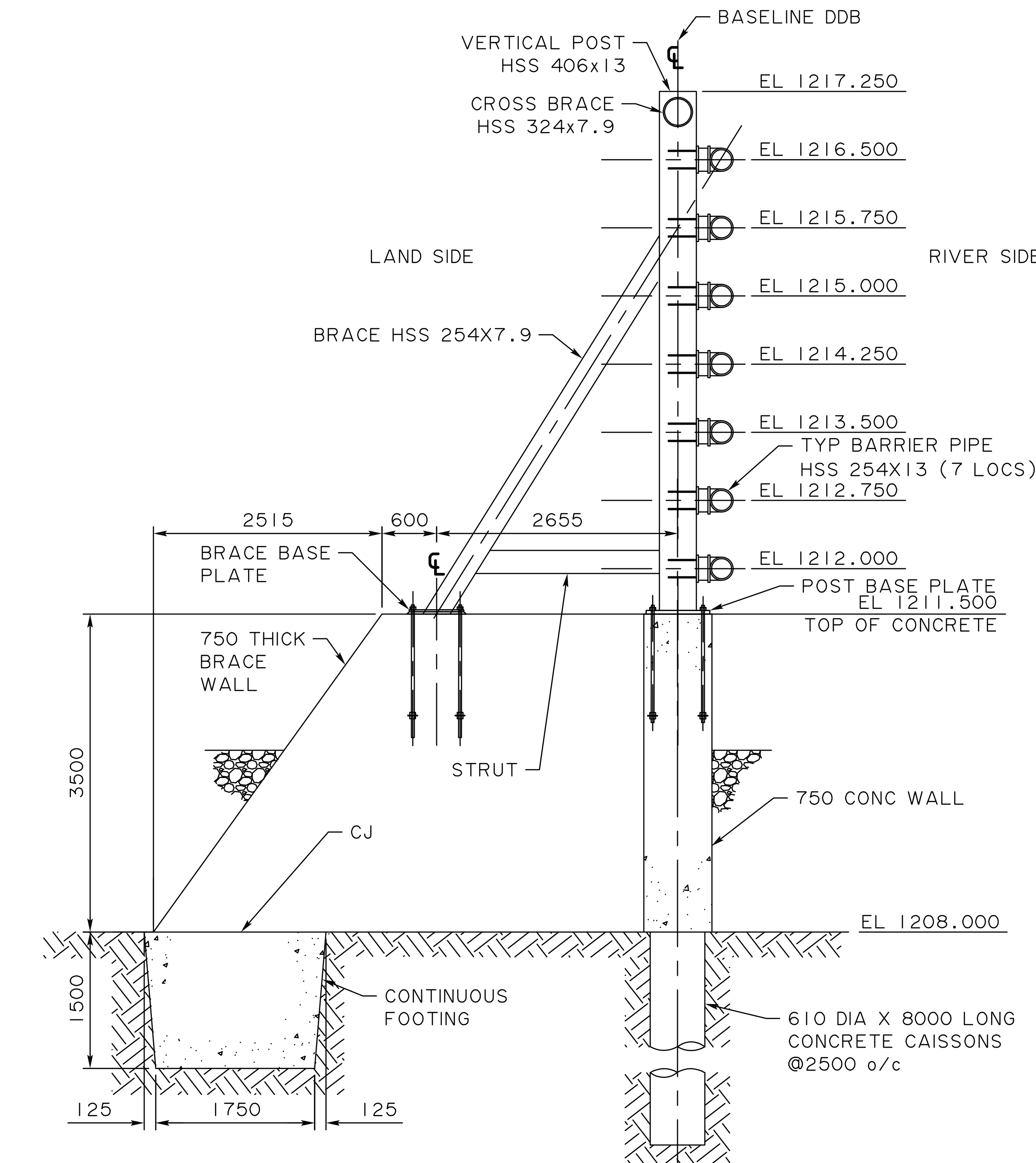
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-276	

PlotDate: 2019.12.03 11:39 AM Login: Shelton, Ben
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\200_ds\200_ds_lexington\aux_spwy\73396S-275-276.dwg TENDER No. ?

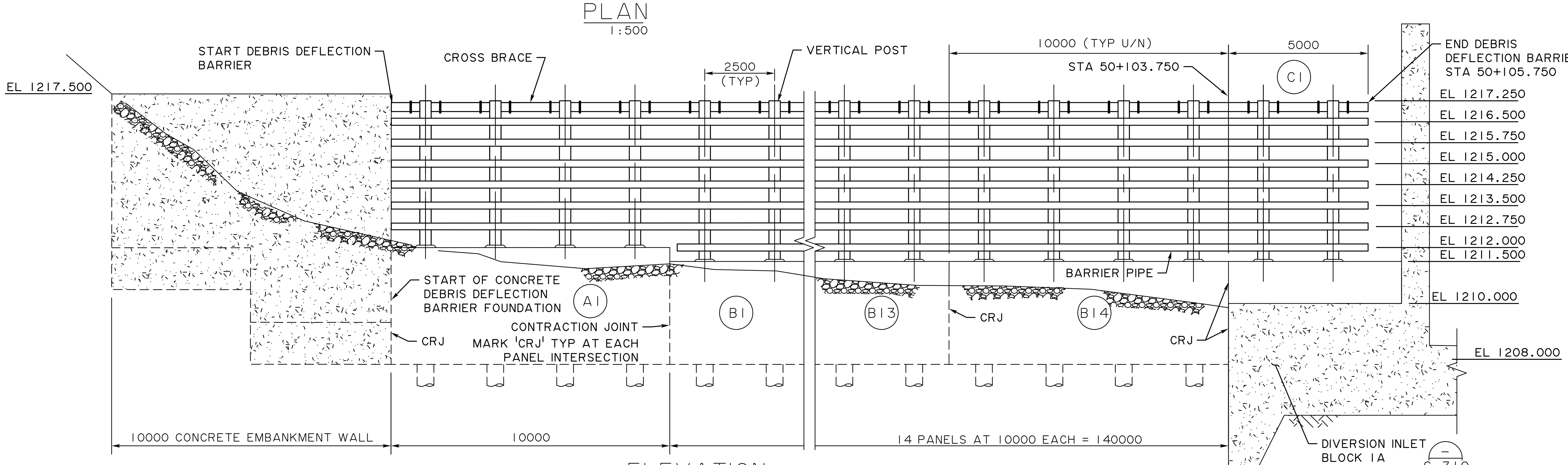
TENDER No. ?
PlotDate: 2020.06.29 3:48 PM Login: Rossnagel, Ian
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\200_ds\200_ds_winnipeg\working_pdfs\debris_barrier_20200629_S-280_subm_mod_cad&pdf\73396S-280.dwg



PLAN
1:500



A SECTION
1:50



ELEVATION
1:100

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES.
2. STATIONS AND ELEVATIONS ARE IN METRES.
3. DEBRIS DEFLECTION BARRIER CONTROL POINT
STA 50+103.750
NORTHING 5,654,605.811
EASTING -33,640.755

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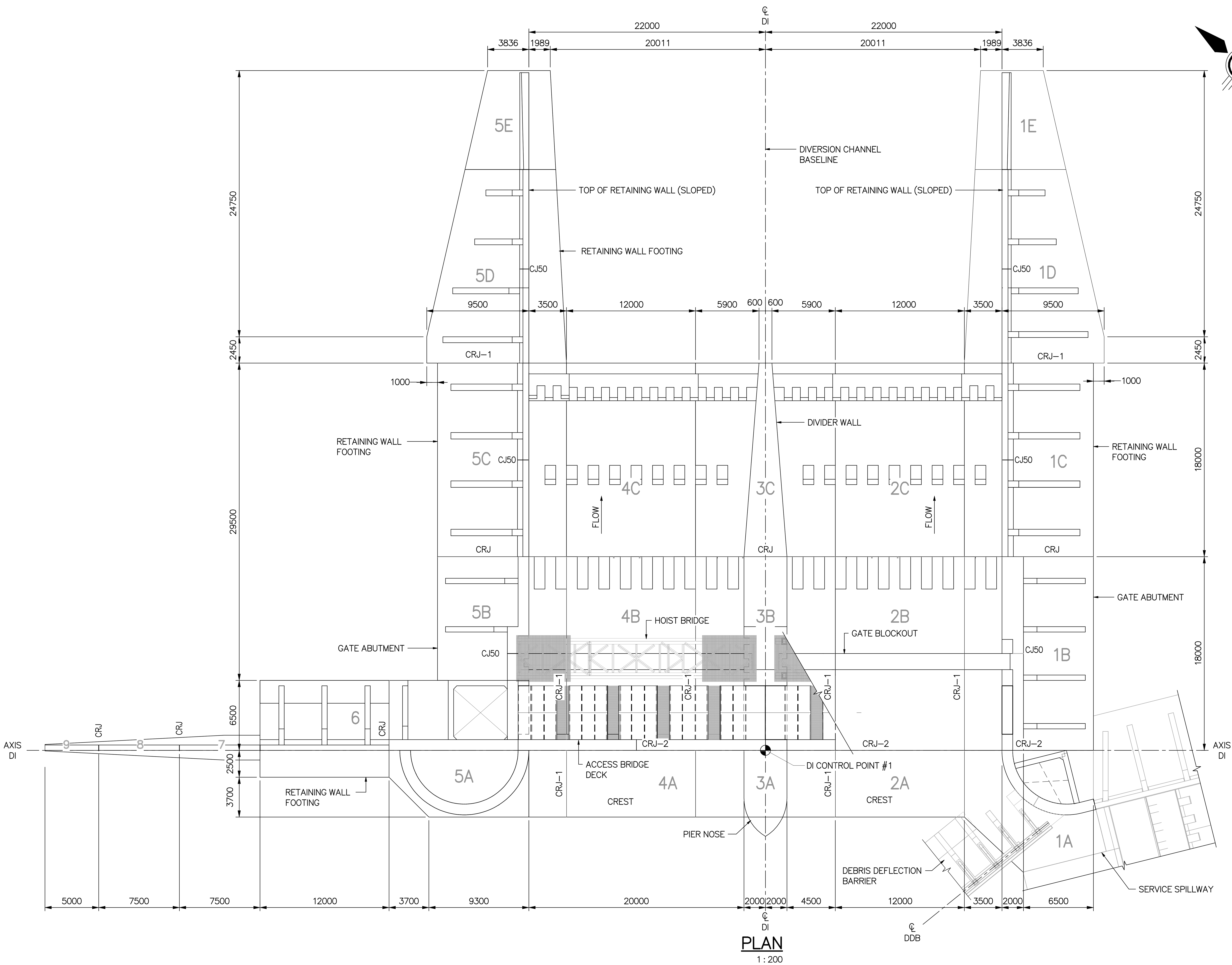
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

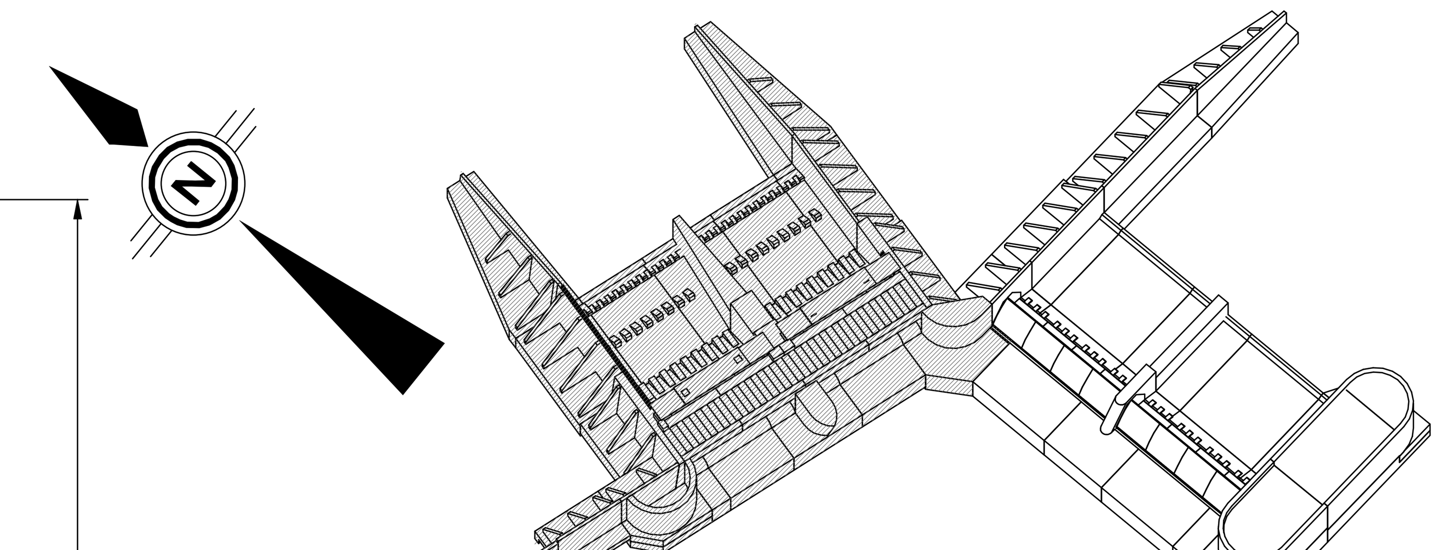
DEBRIS DEFLECTION BARRIER
STRUCTURAL ARRANGEMENT

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-280	



PLAN

1 : 200



STRUCTURAL KEY

NOTES

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
3. DIVERSION INLET CONTROL POINT #1:
   NORTHING 5654599.271
   EASTING –33664.372

   DIVERSION CHANNEL BASELINE = DIVERSION INLET STA 10+067.000

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DATE

Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
STRUCTURAL ARRANGEMENT

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-300	

Plot Date: 6/25/2020 2:38:18 PM Login: Dosler
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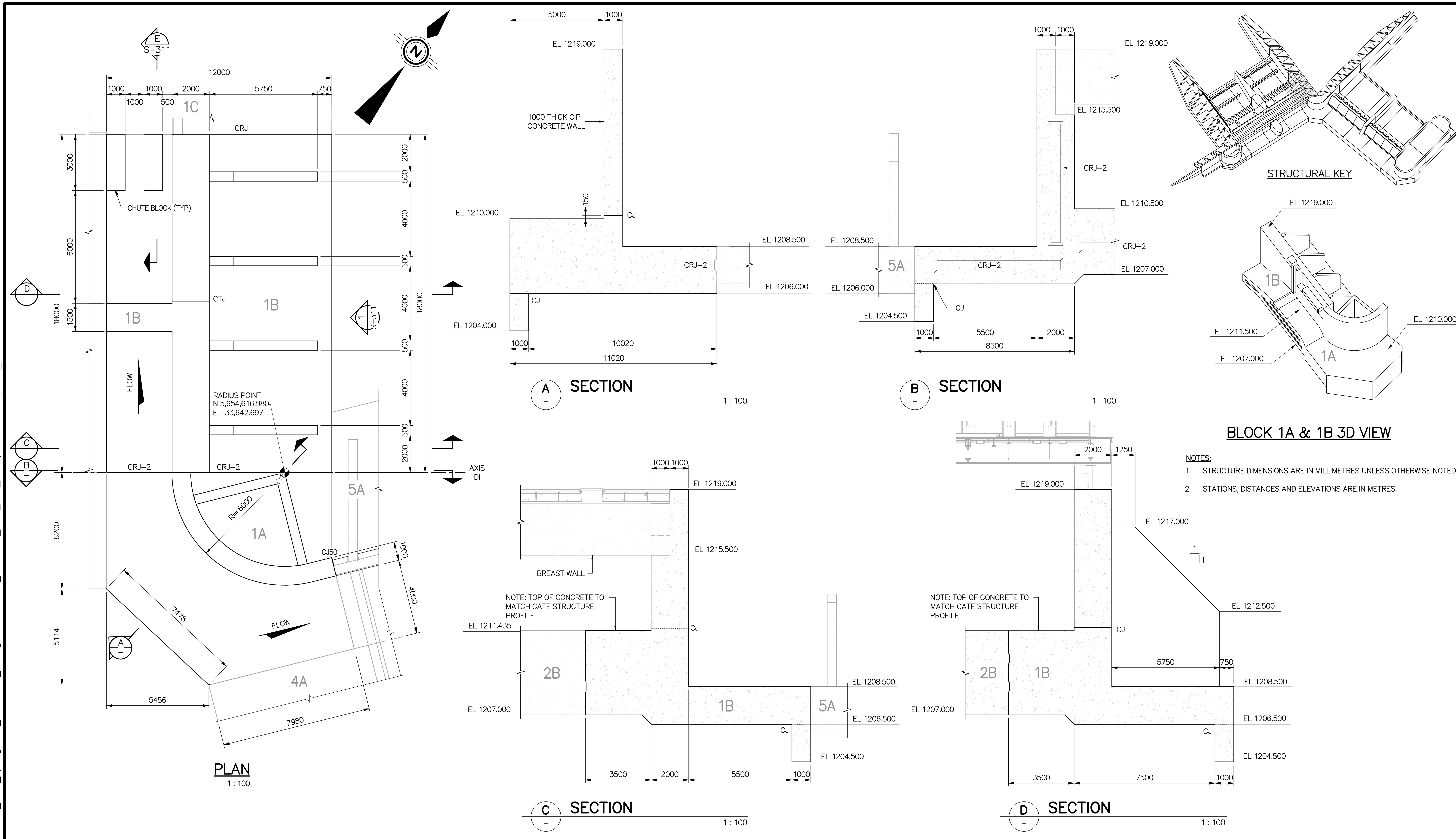

NOTES:

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
RIGHT RETAINING WALL - BLOCKS 1A & 1B
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-310	

Plot Date: 6/25/2020 2:38:21 PM Login: Dosler
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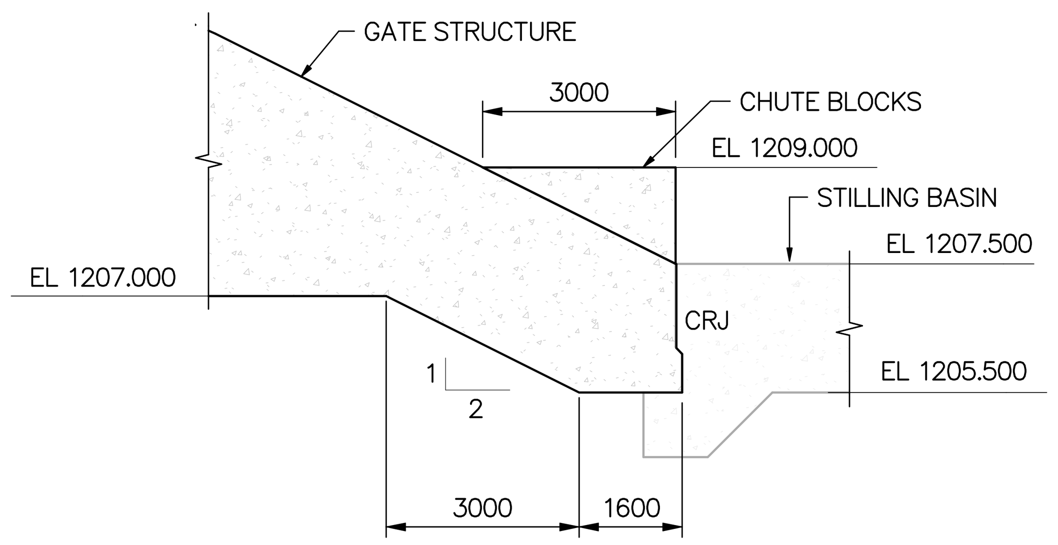
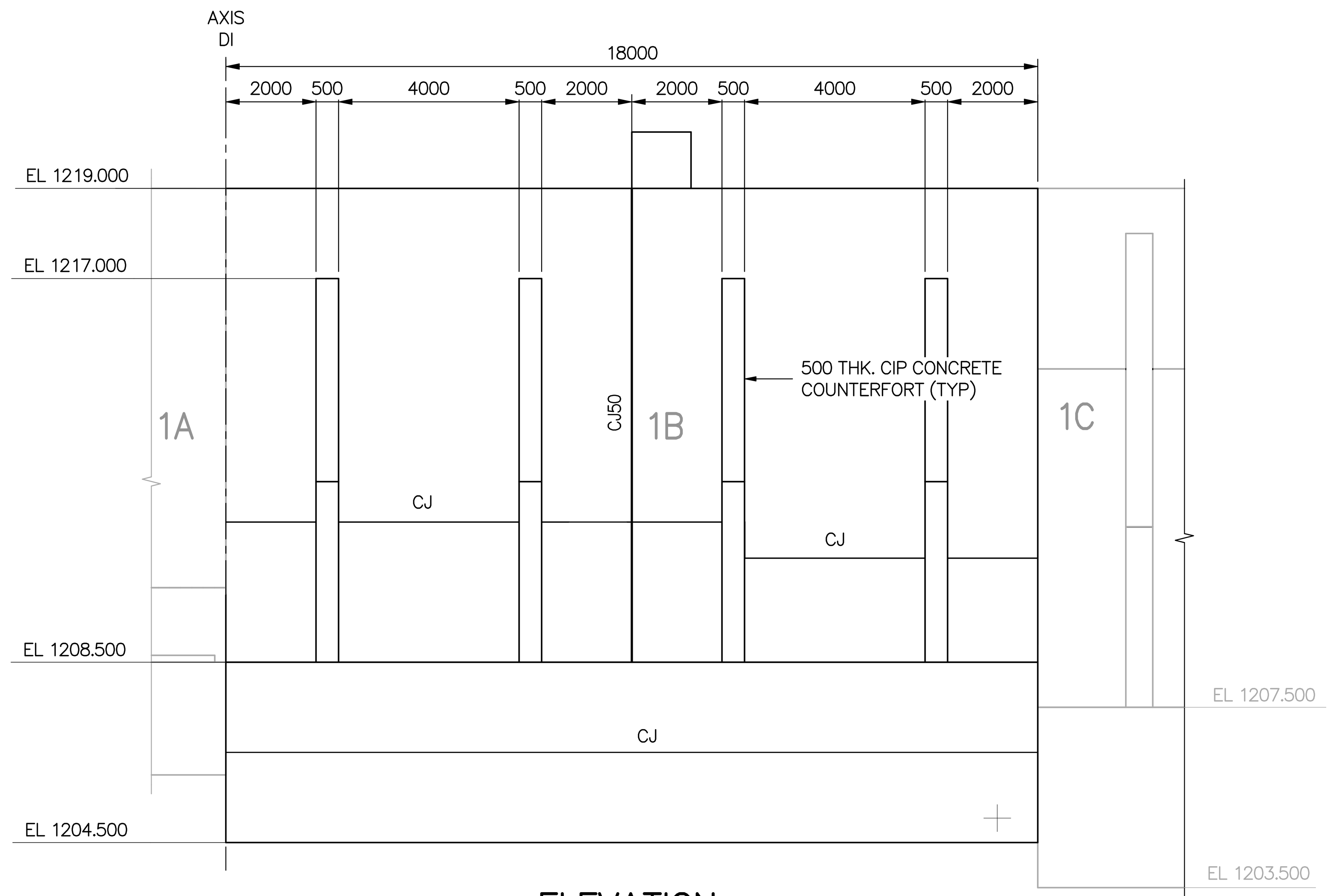


E
S-310
SECTION
1 : 100



ELEVATION
1 : 100

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
RIGHT RETAINING WALL - BLOCKS 1A & 1B
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-311	

Plot Date: 6/25/2020 2:38:22 PM Login: Dosler
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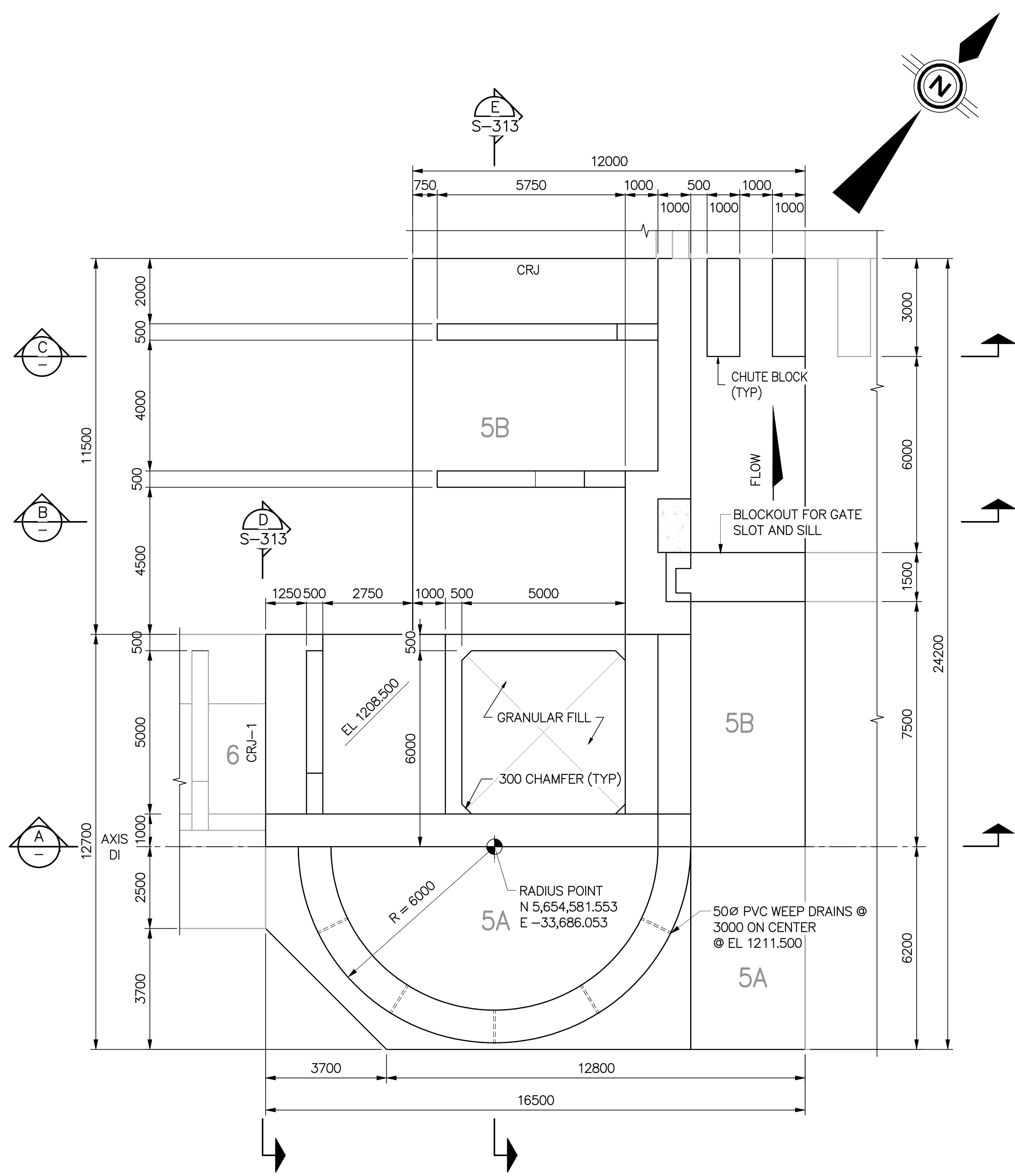
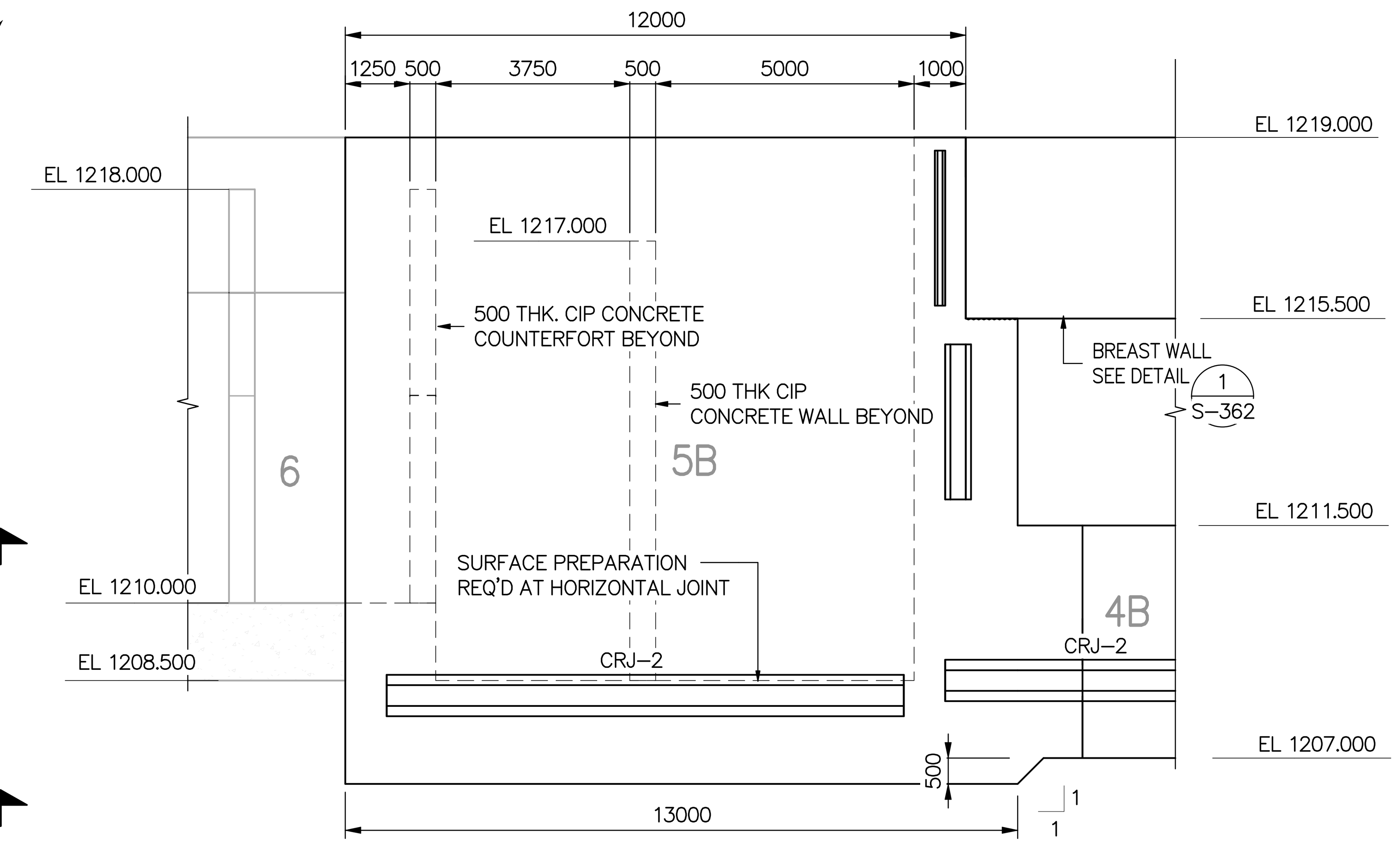


PLAN

1 : 100

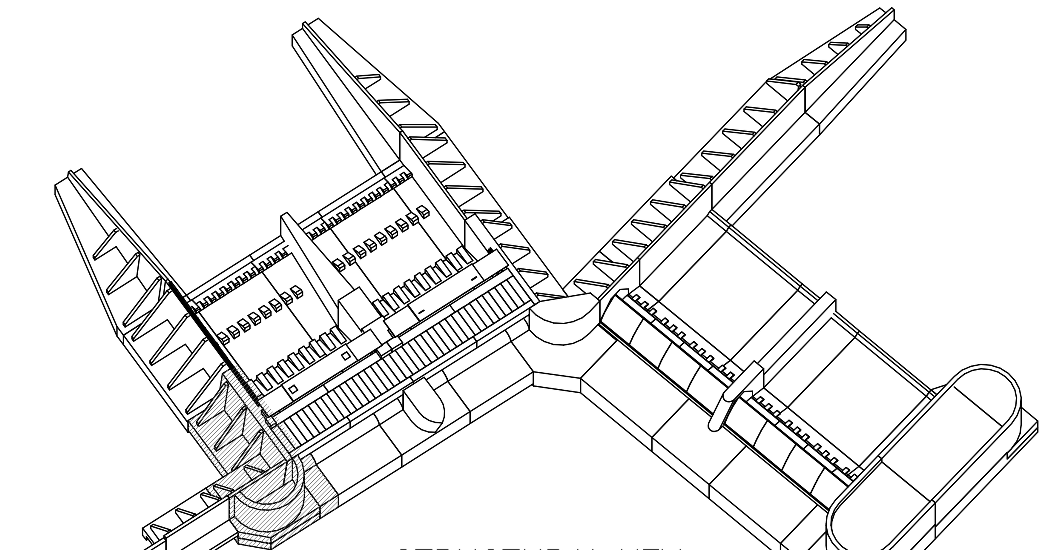


A SECTION

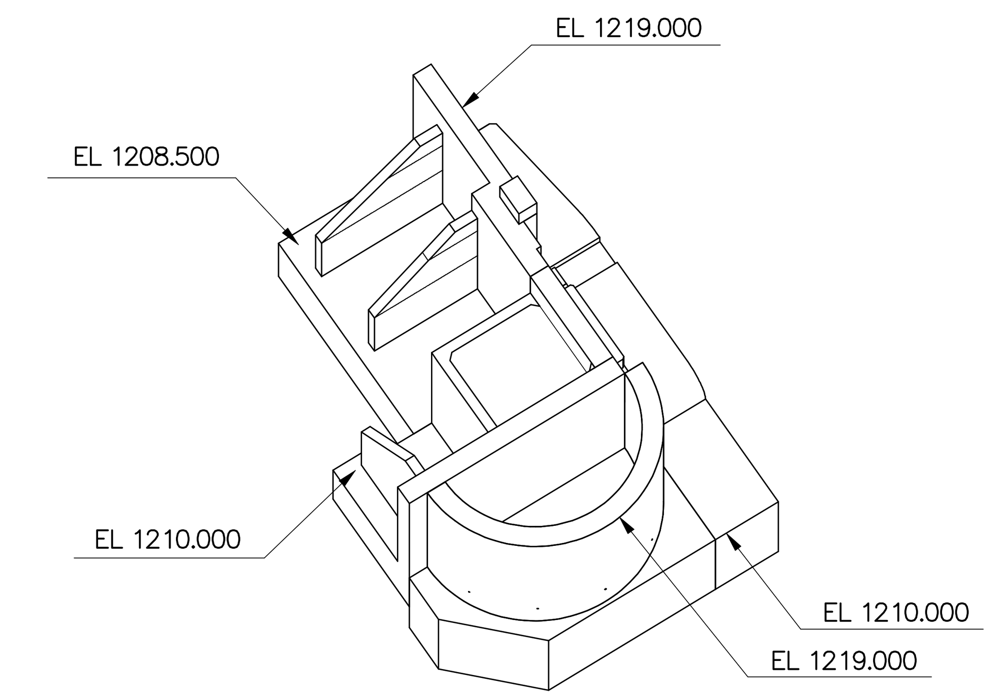
1 : 100

NOTES:

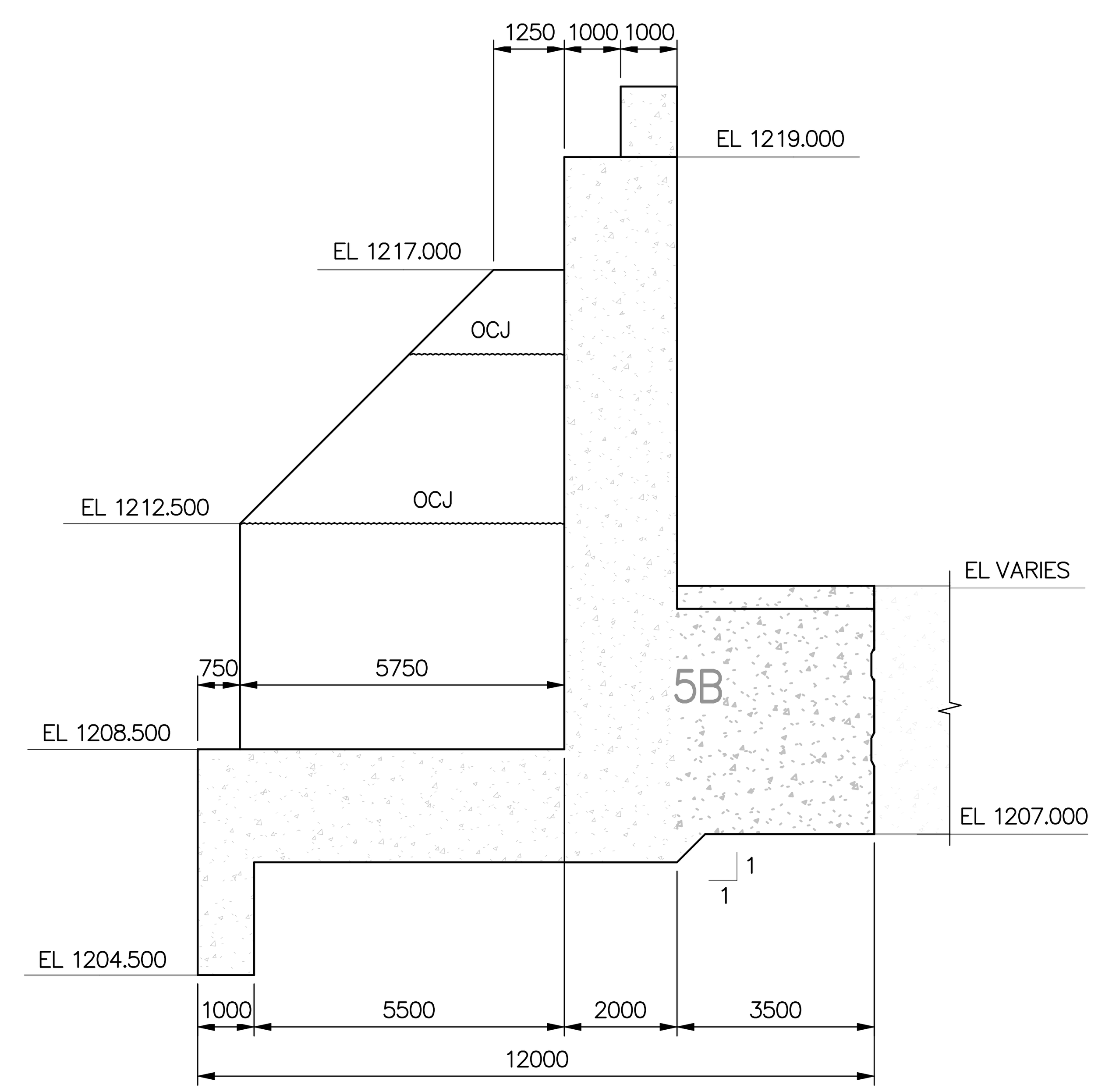
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.



STRUCTURAL KEY

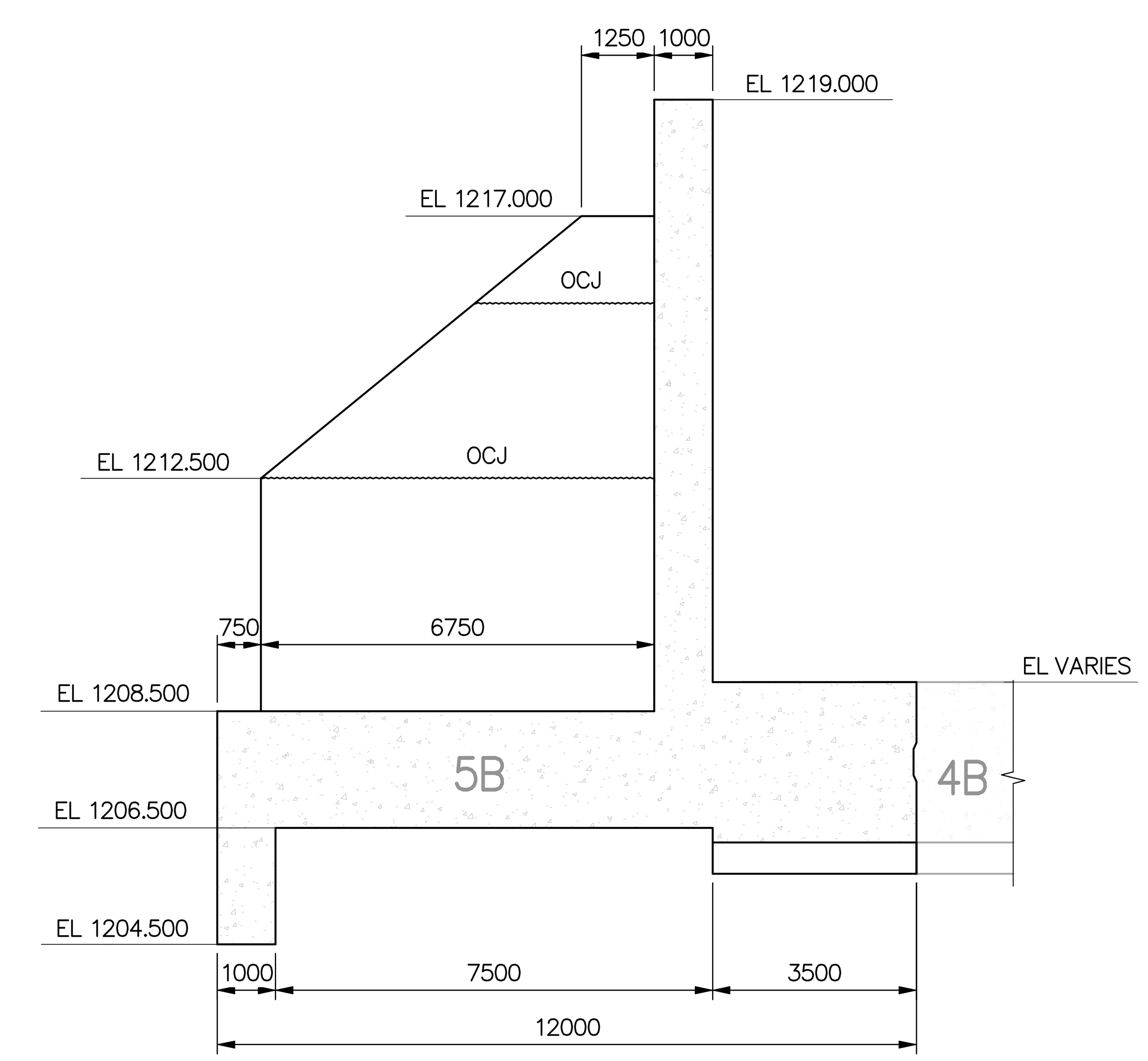


BLOCKS 5A & 5B 3D VIEW



B SECTION

1 : 100



C SECTION

1 : 100

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
LEFT RETAINING WALL - BLOCKS 5A & 5B
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-312	

Plot Date: 6/25/2020 2:38:24 PM Login: Dosler
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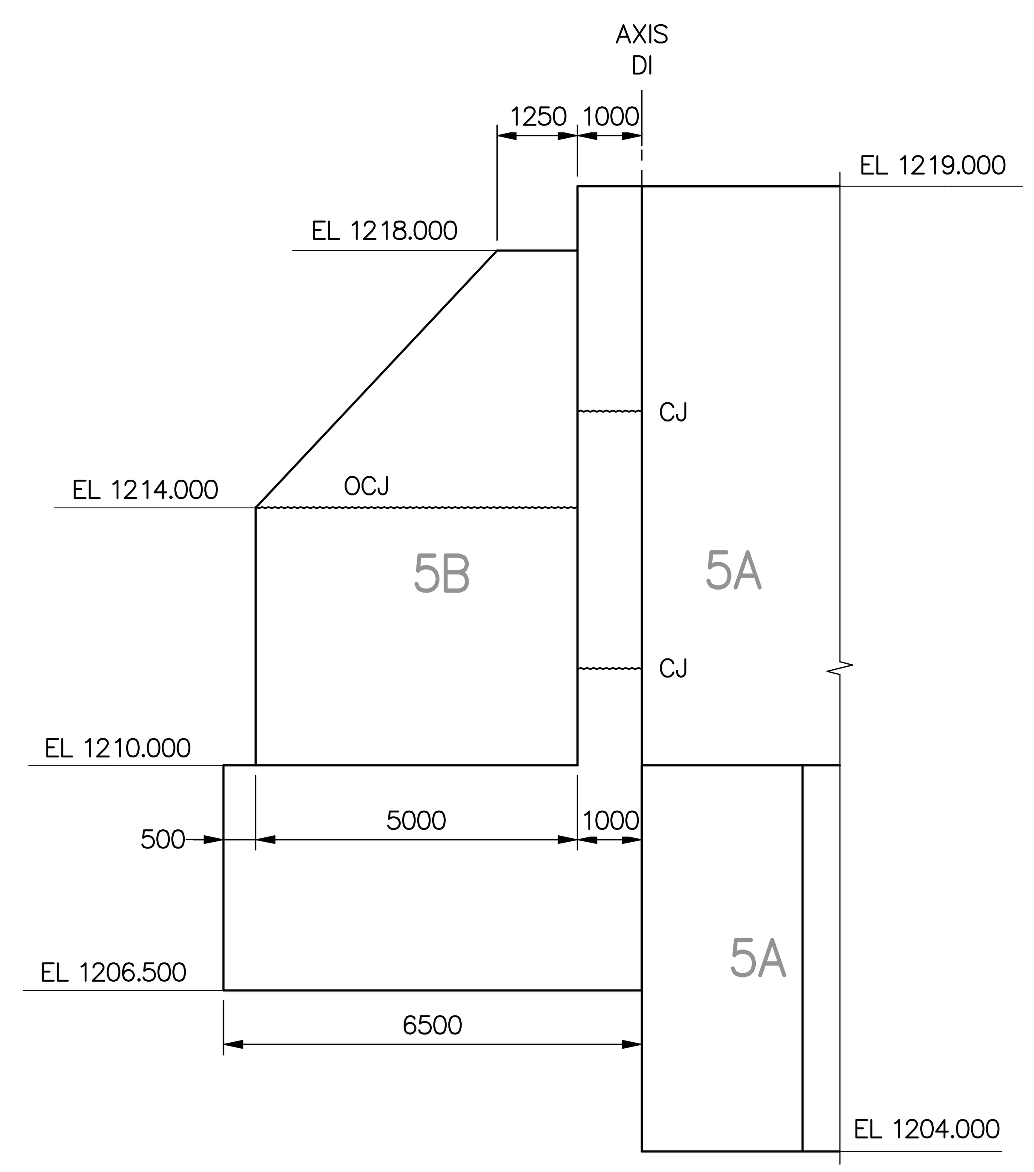
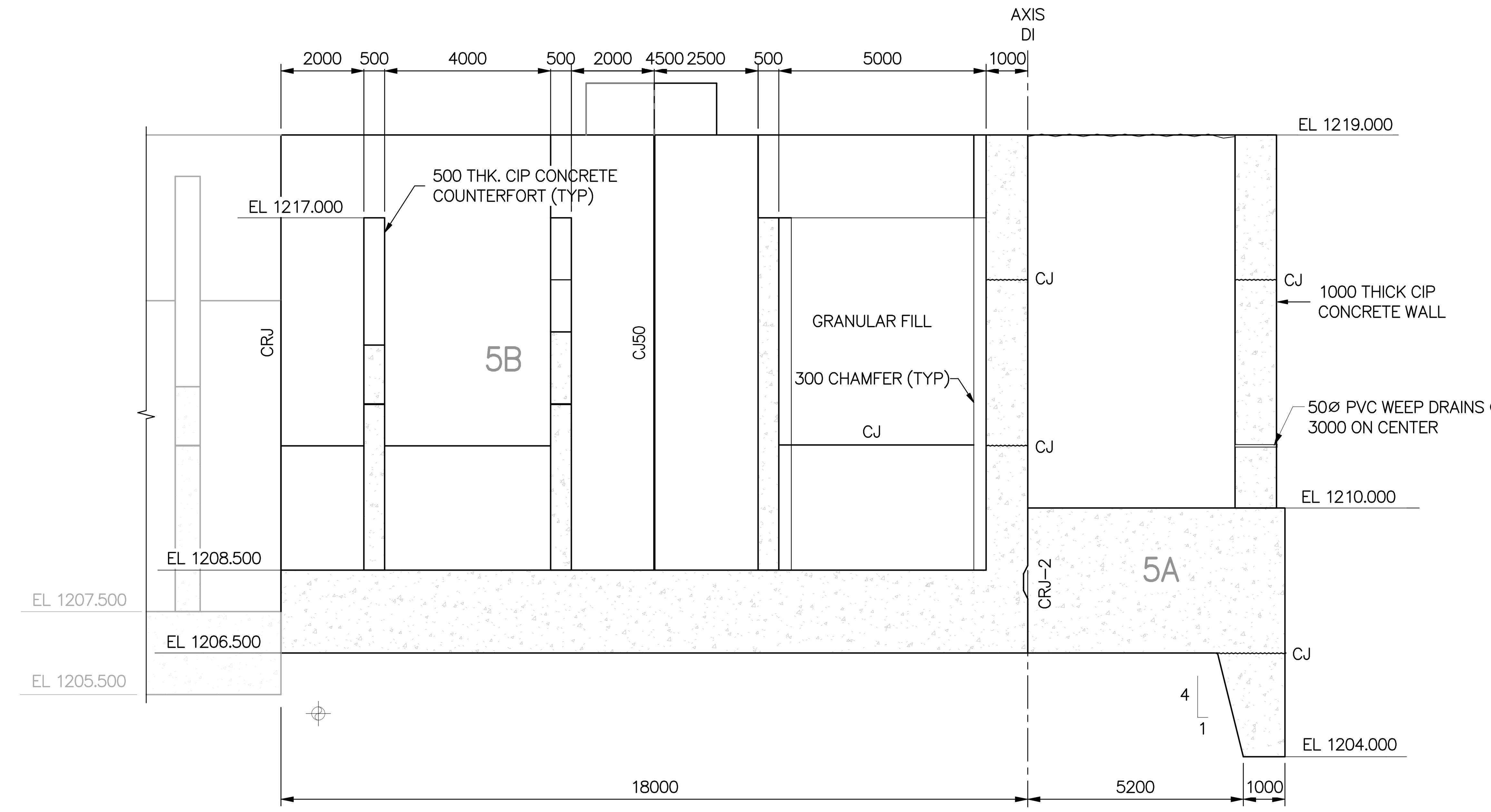


D
S-312
SECTION
1 : 100



E
S-312
SECTION
1 : 100

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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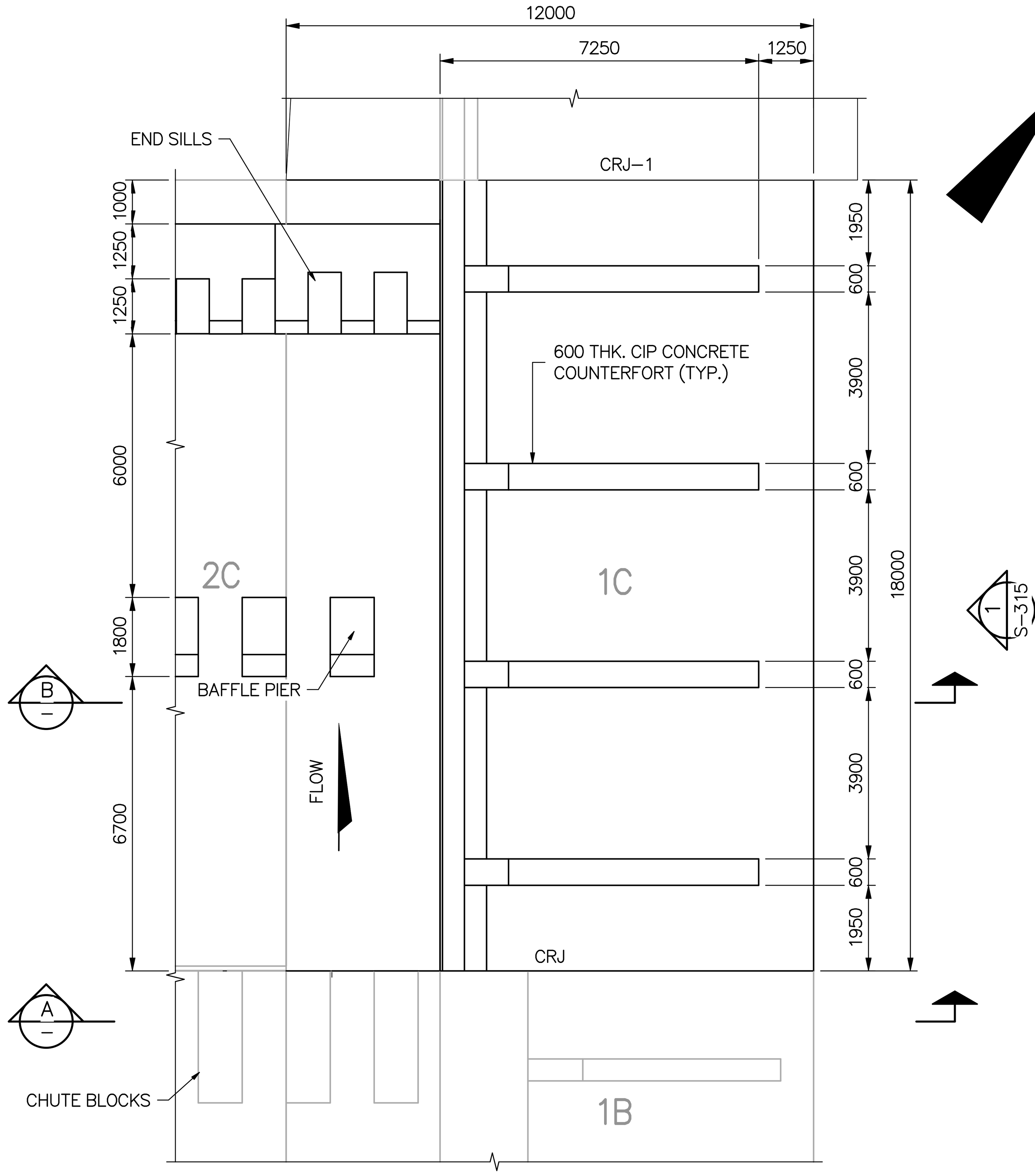
Alberta Transportation

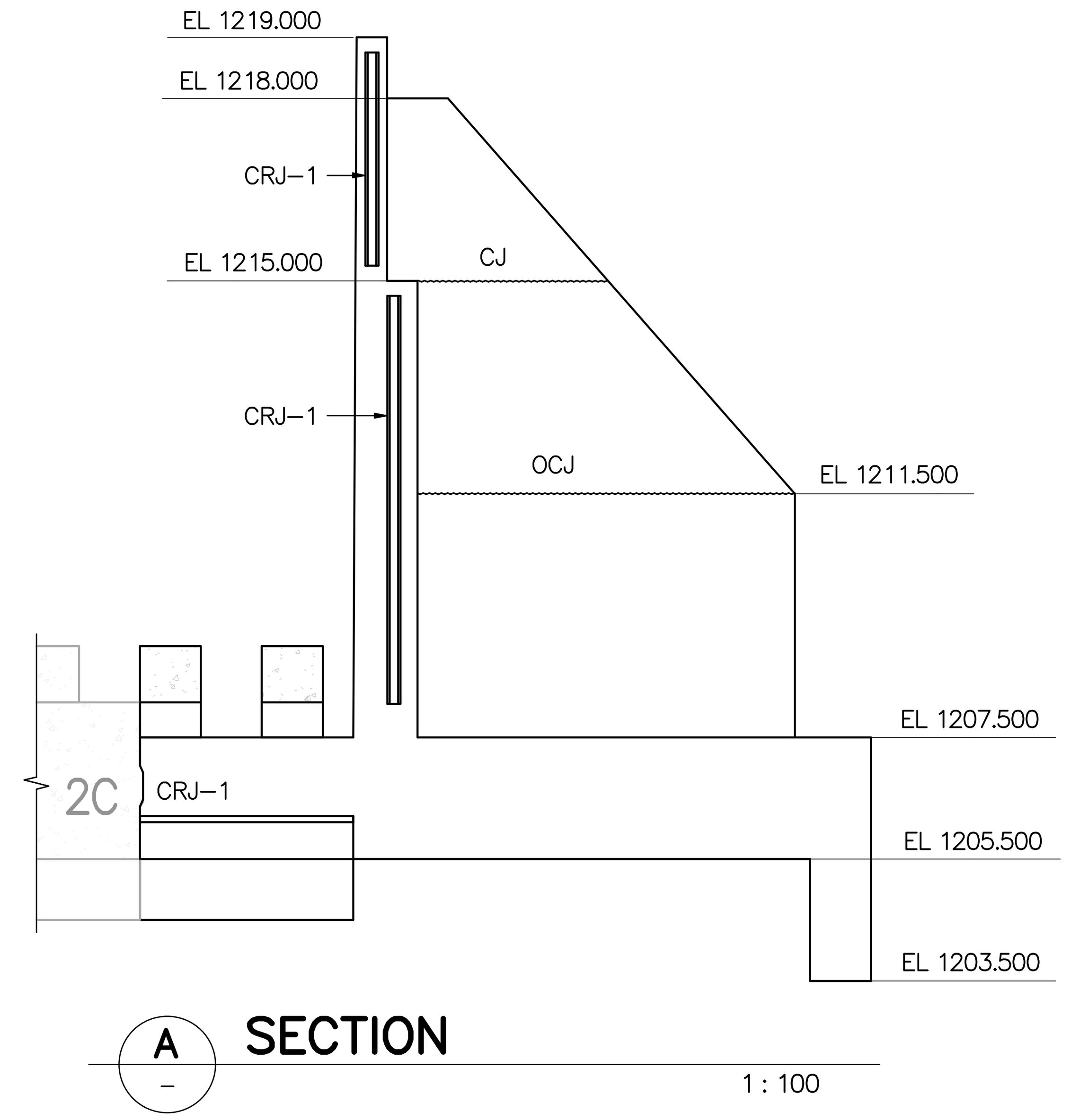
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
LEFT RETAINING WALL - BLOCKS 5A & 5B
CONCRETE OUTLINE - SHEET 2 OF 2

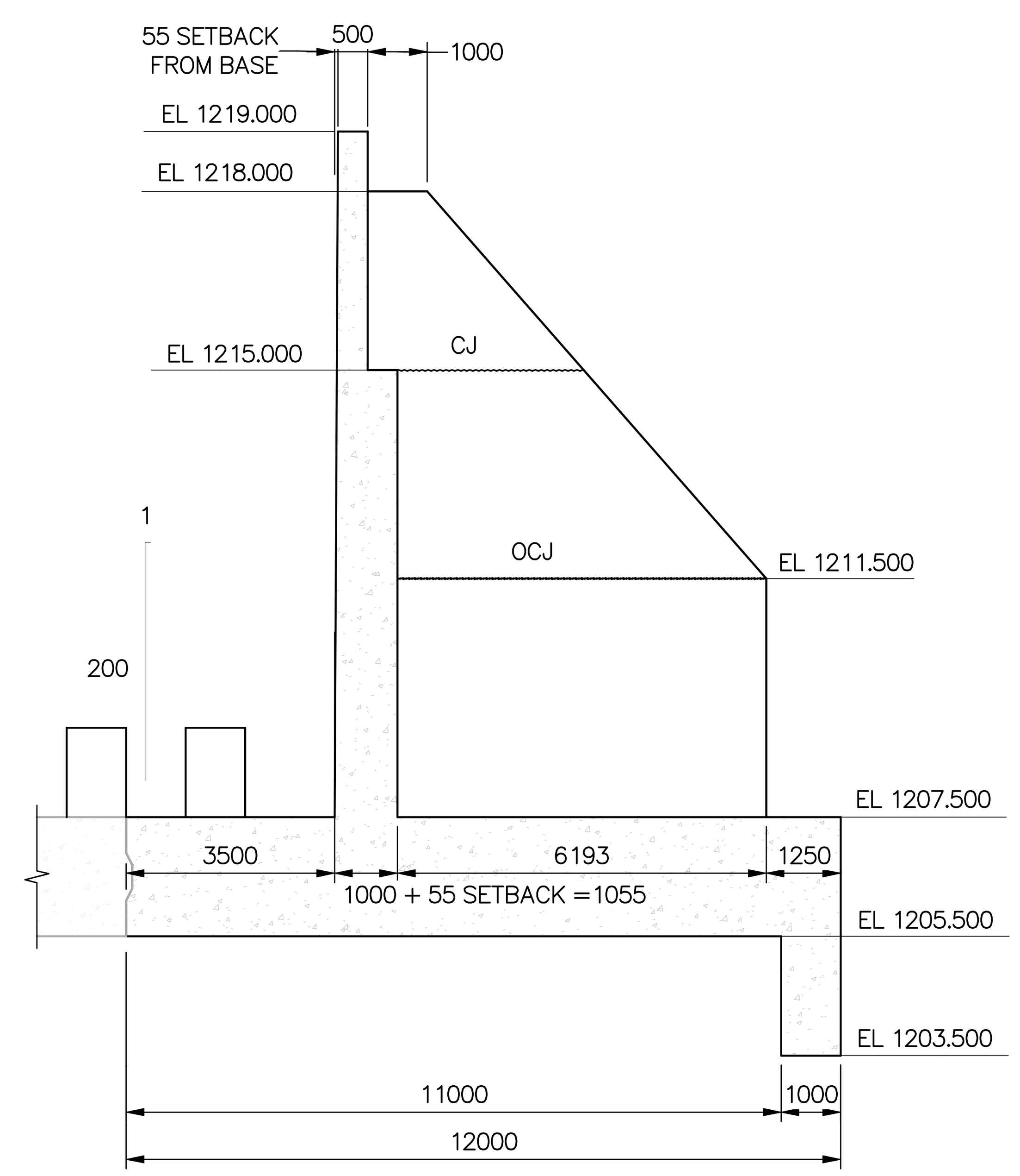
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-313	



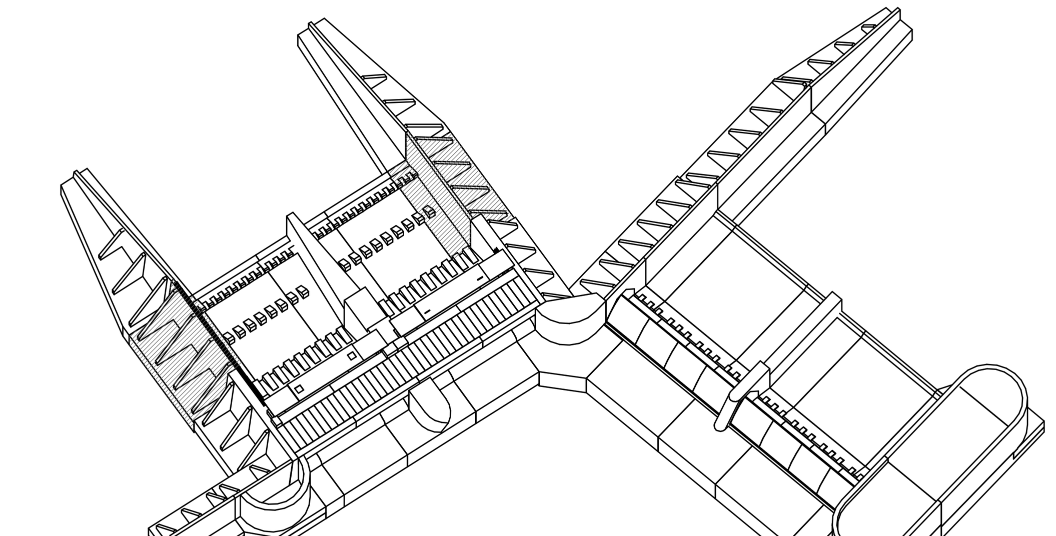
PLAN – BLOCK 1C
1 : 100



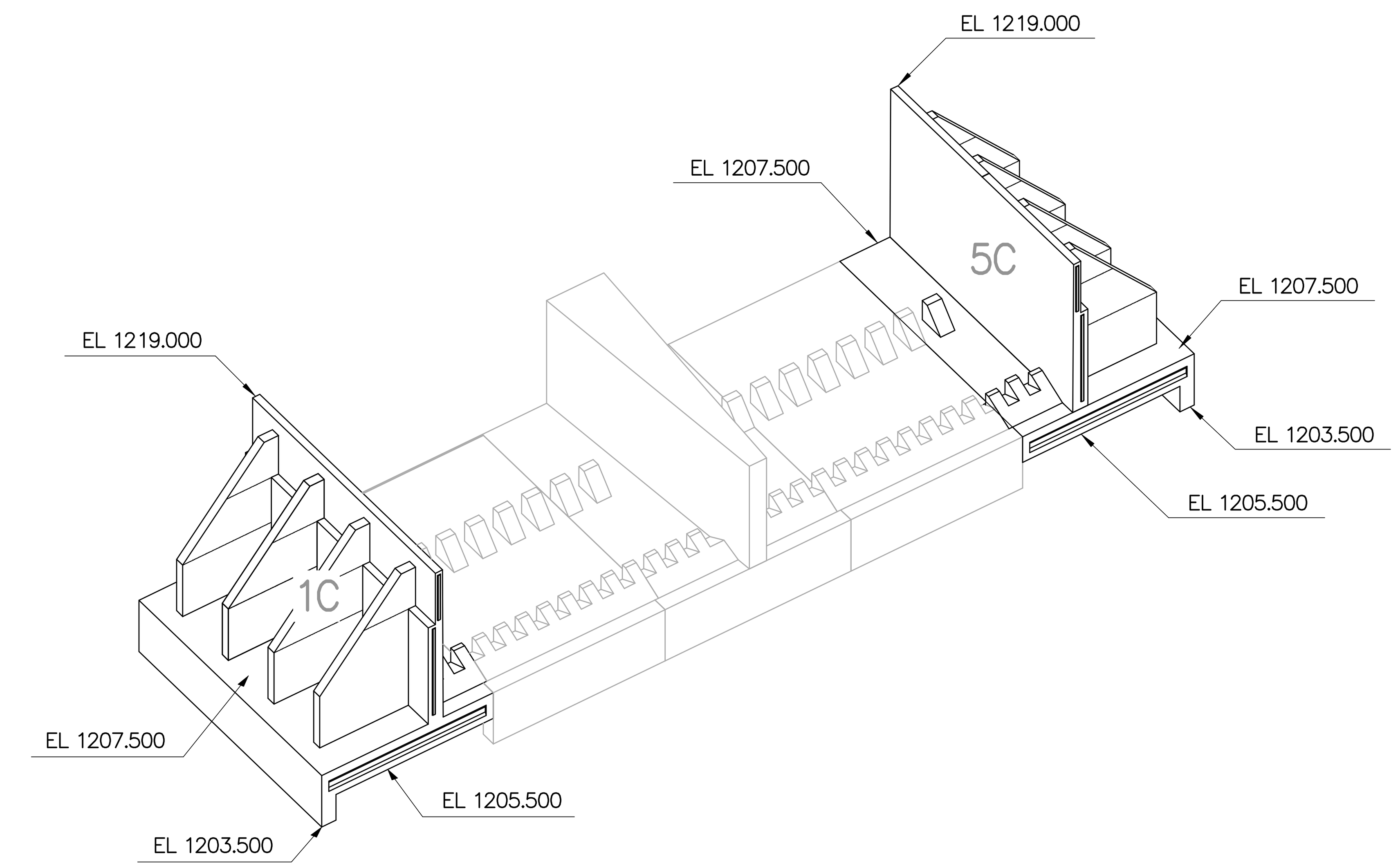
A SECTION
1 : 100



B SECTION
1 : 100



STRUCTURAL KEY



BLOCKS 1C & 5C 3DVIEW

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
RETAINING WALLS - BLOCKS 1C & 5C
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-314	

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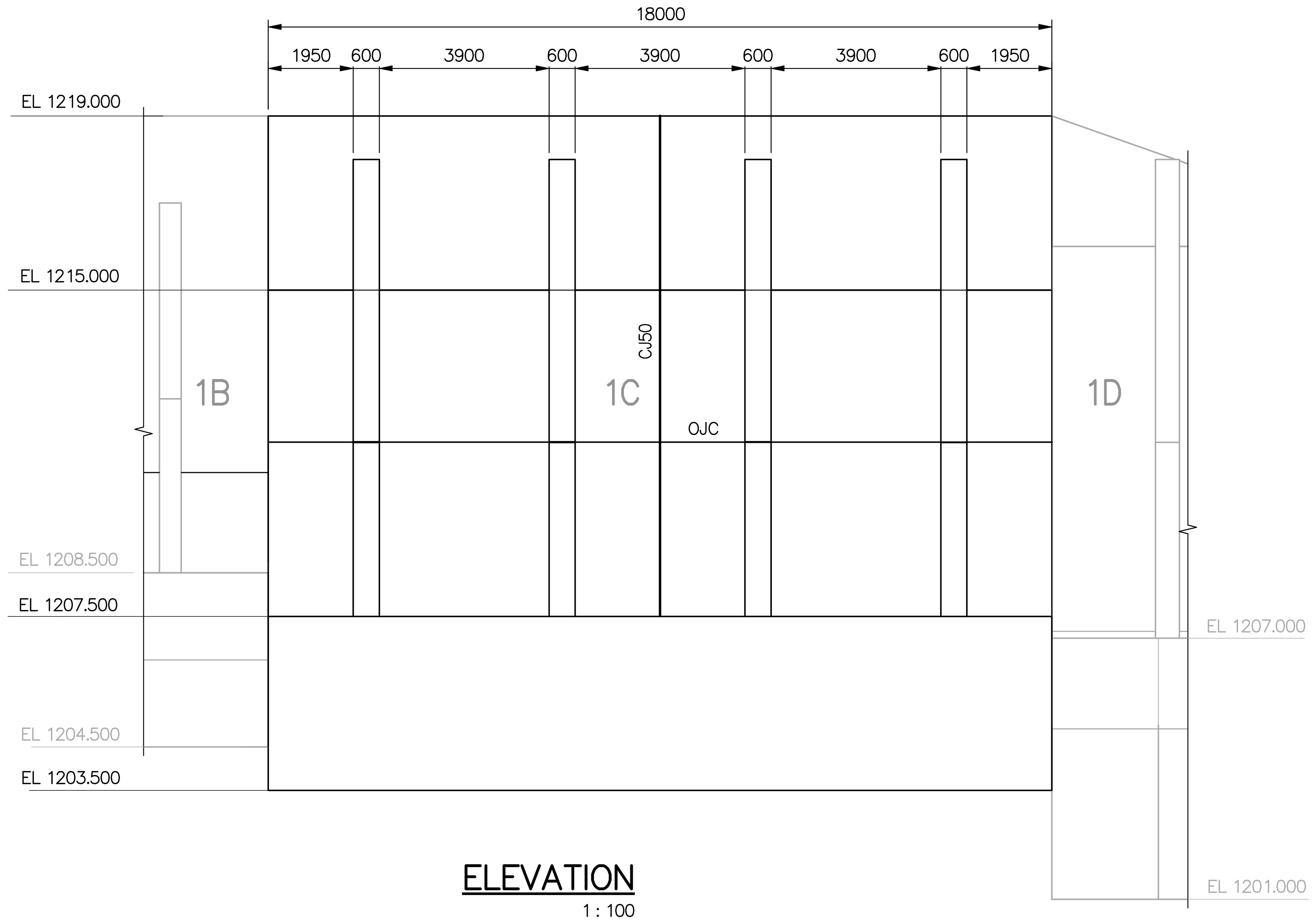

ELEVATION

1 : 100

NOTES:

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
RETAINING WALLS - BLOCKS 1C & 5C
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-315	

Plot Date: 6/25/2020 2:38:27 PM Login: Dosler
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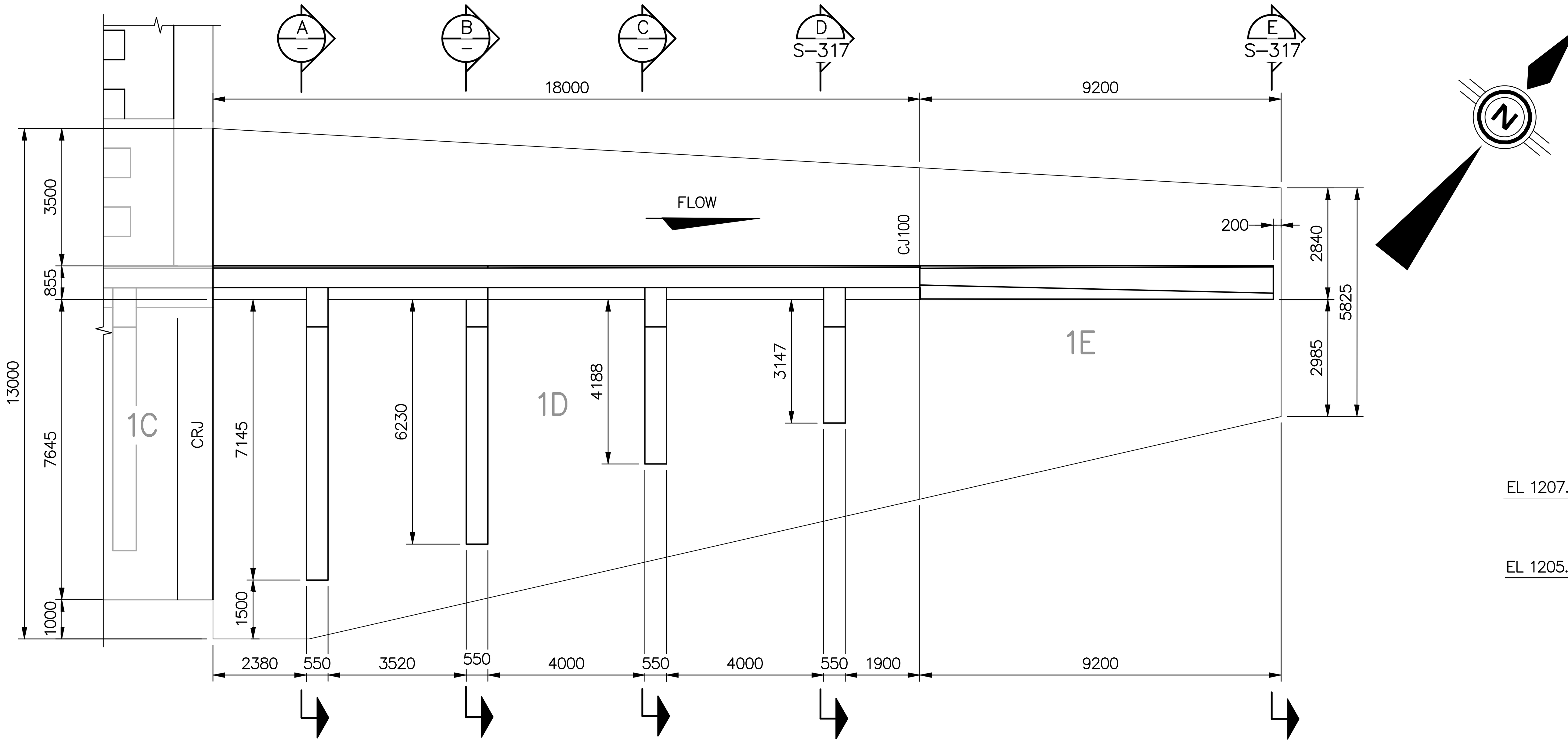
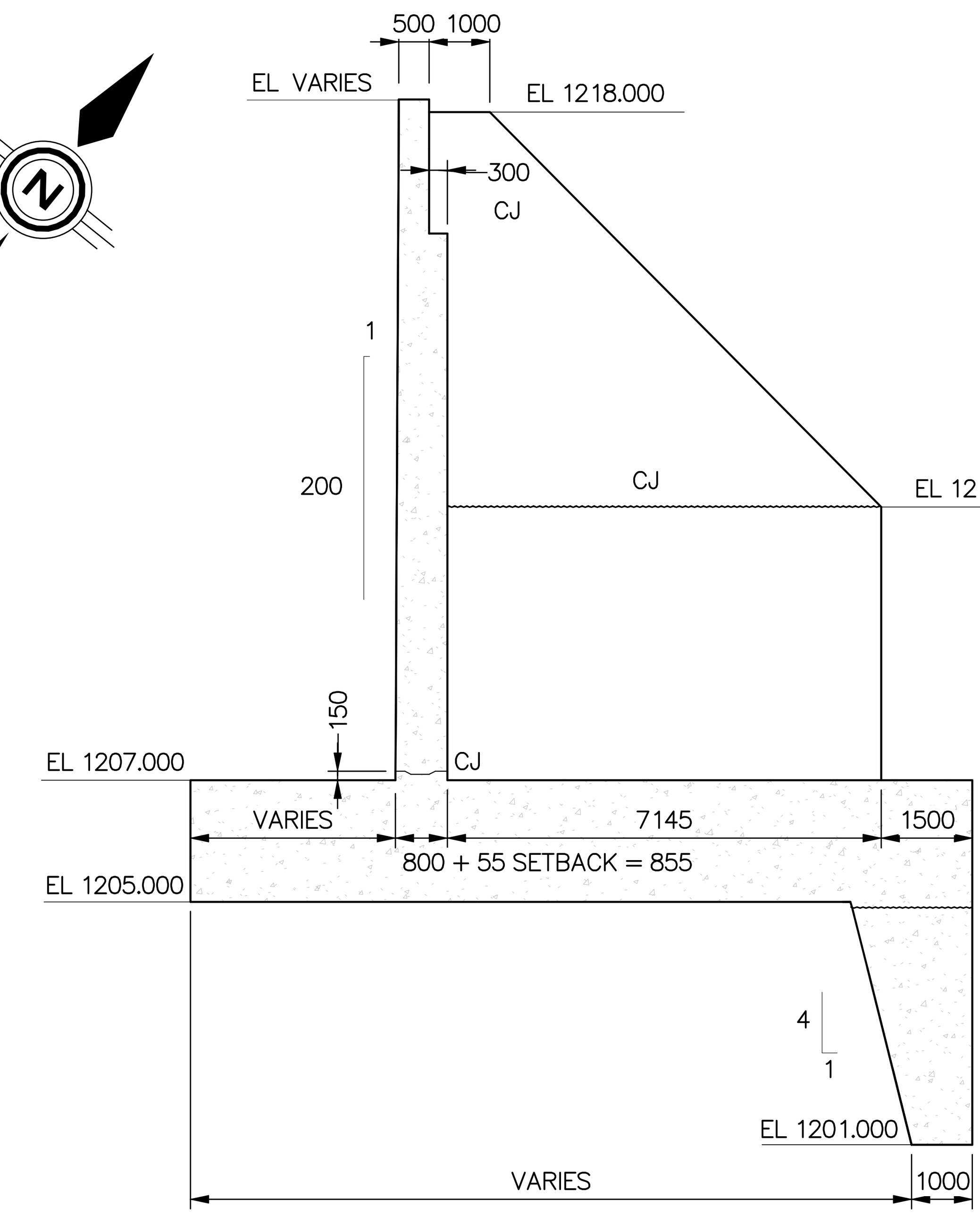


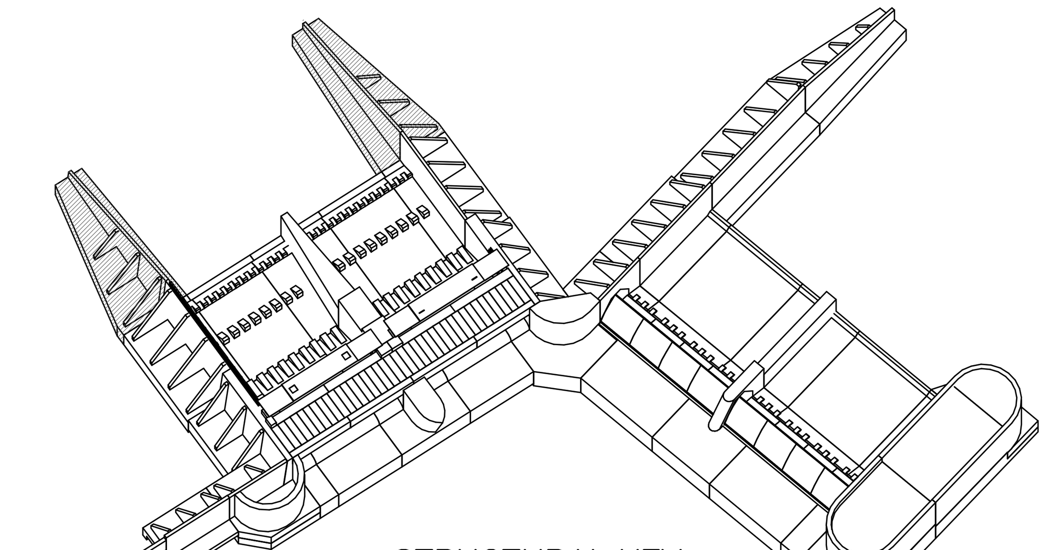
PLAN – BLOCKS 1D & 1E

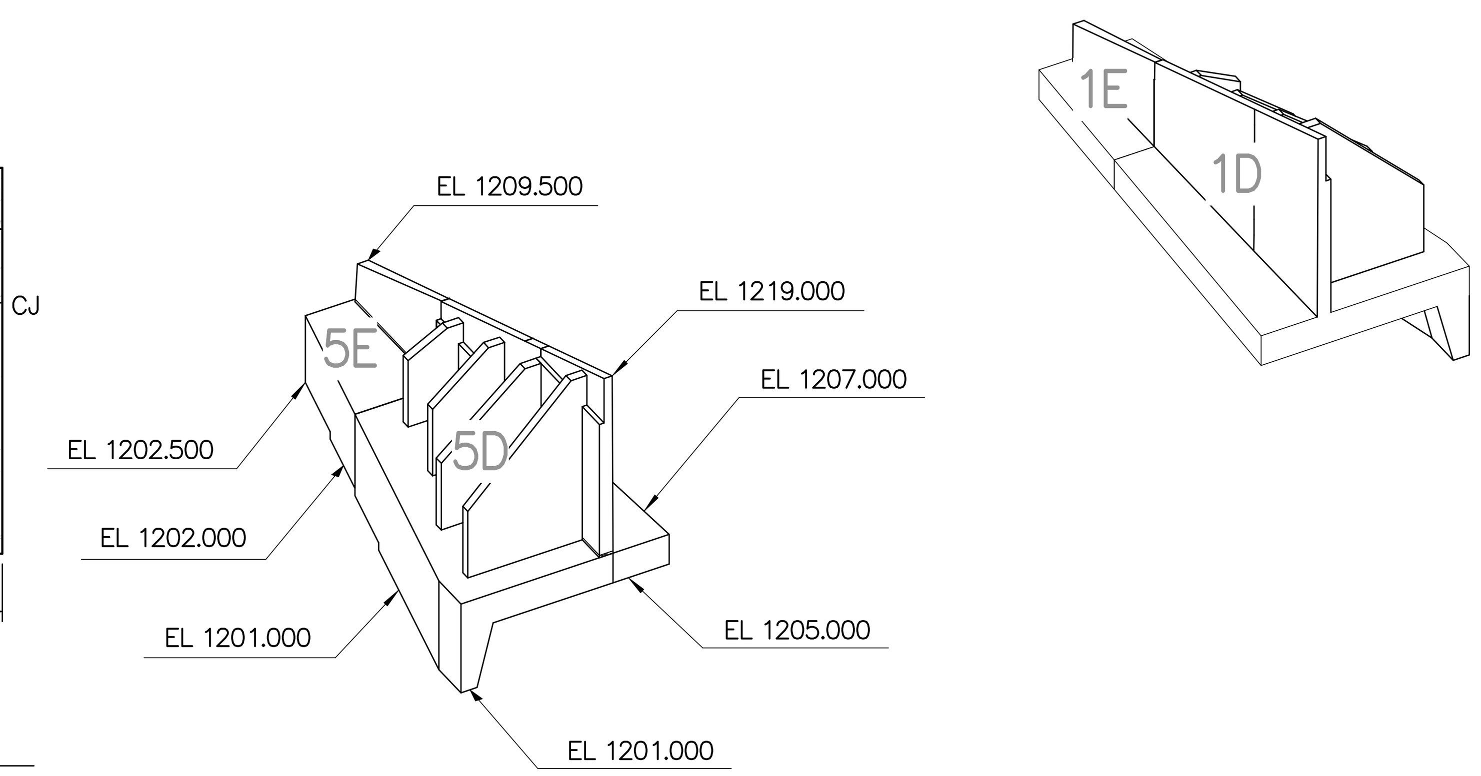
1 : 100



A SECTION

1 : 100

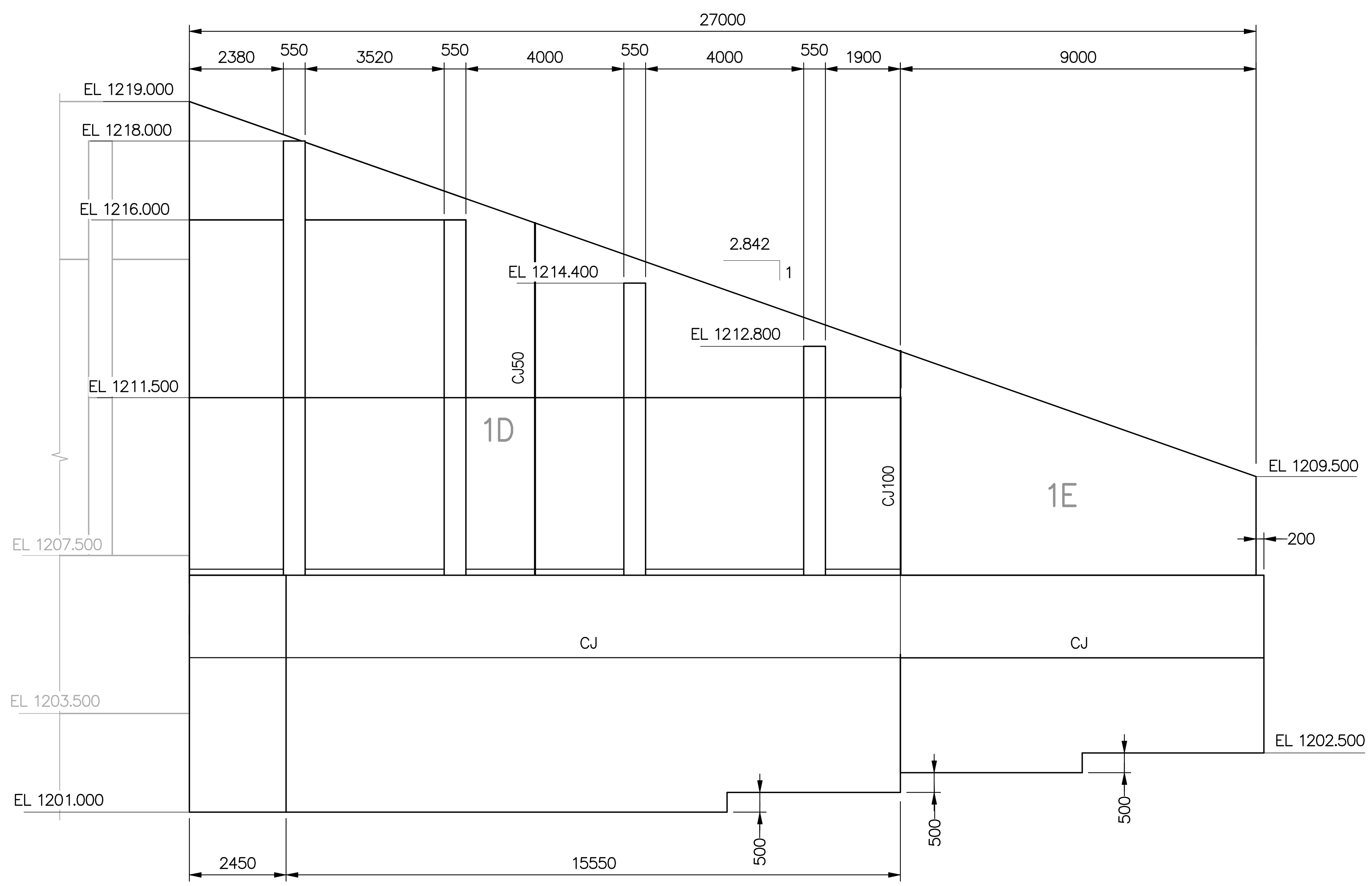




BLOCKS 1D, 1E, 5D & 5E 3D VIEW

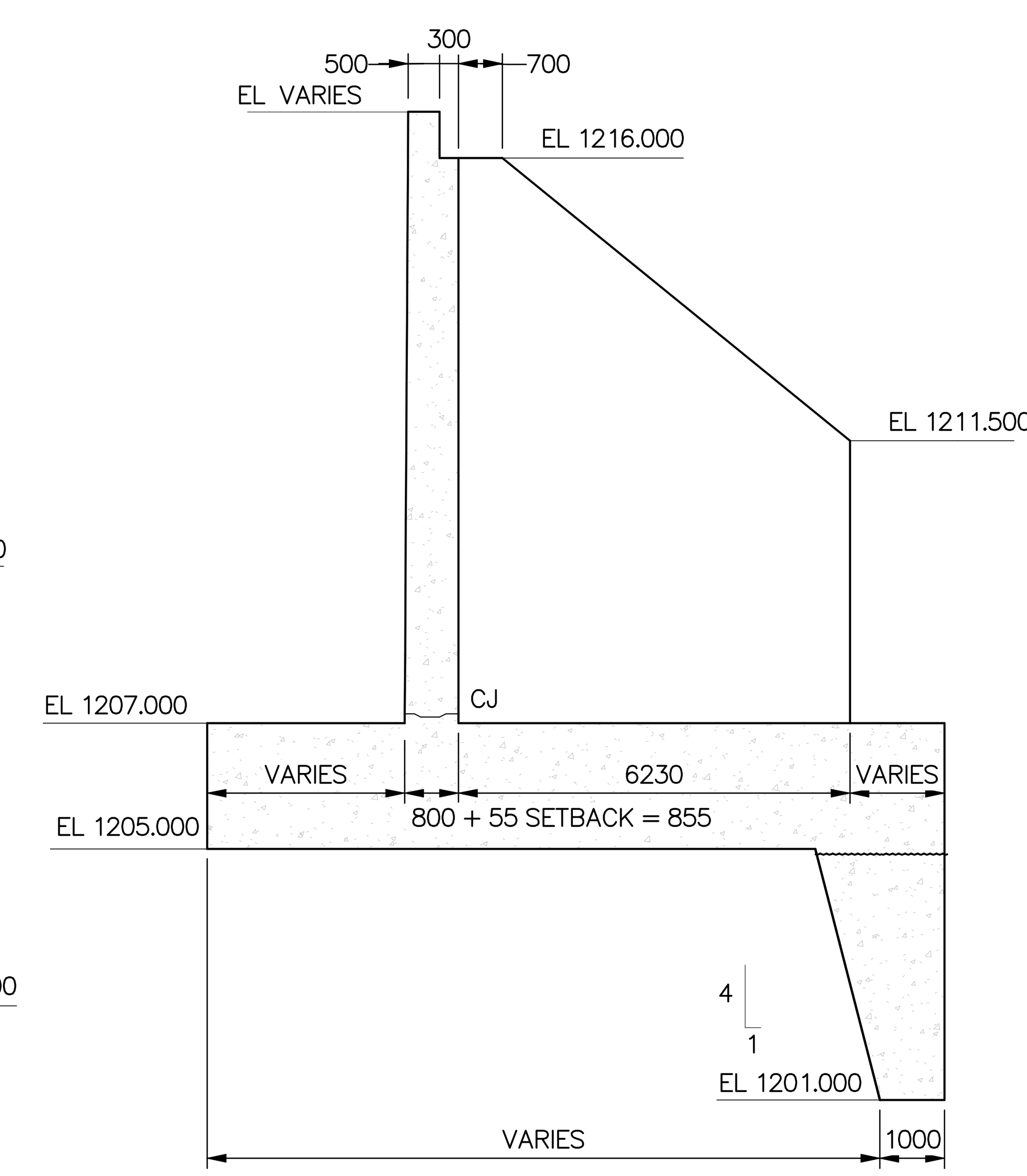
NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.



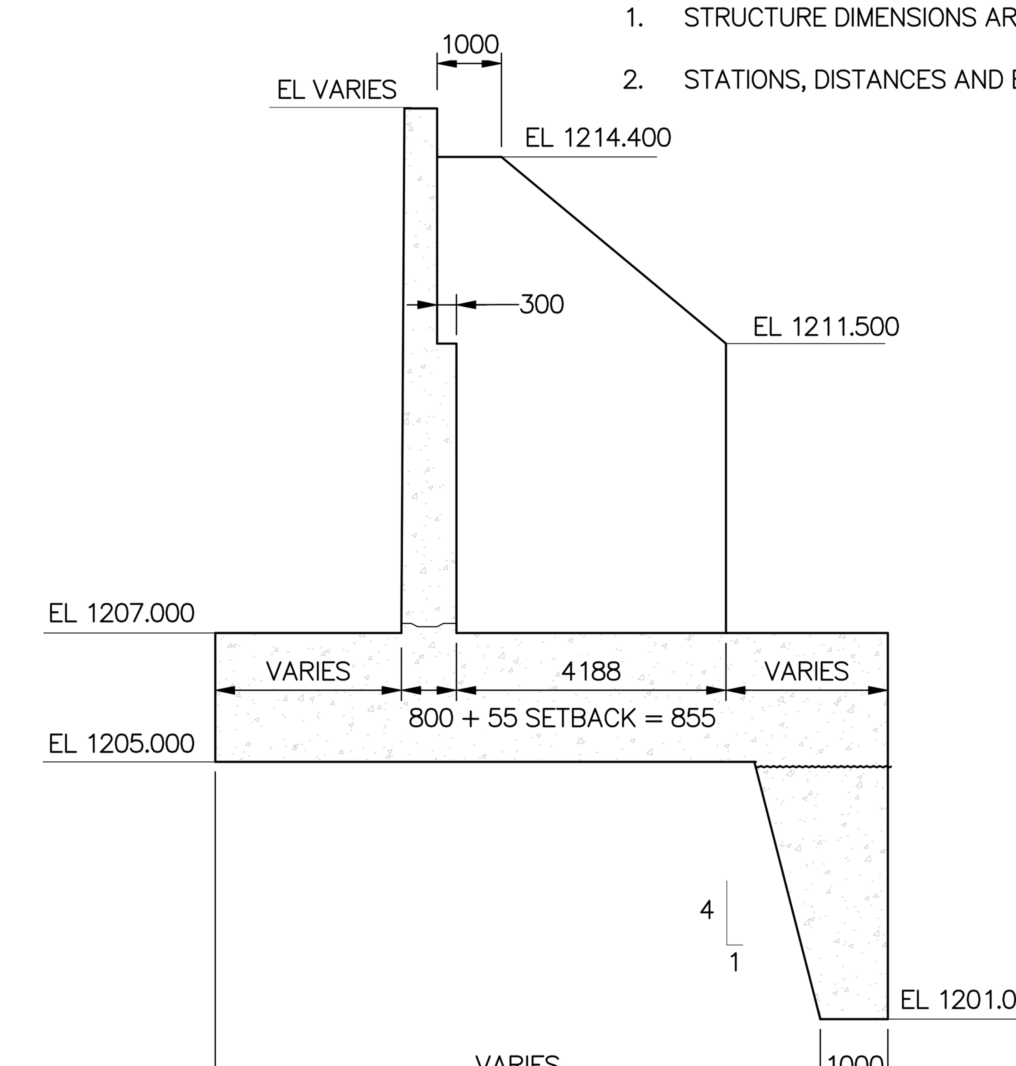
ELEVATION

1 : 100



B SECTION

1 : 100



C SECTION

1 : 100

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
RETAINING WALLS - BLOCKS 1D, 1E, 5D & 5E
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-316	

Plot Date: 6/25/2020 2:38:29 PM Login: Dosler
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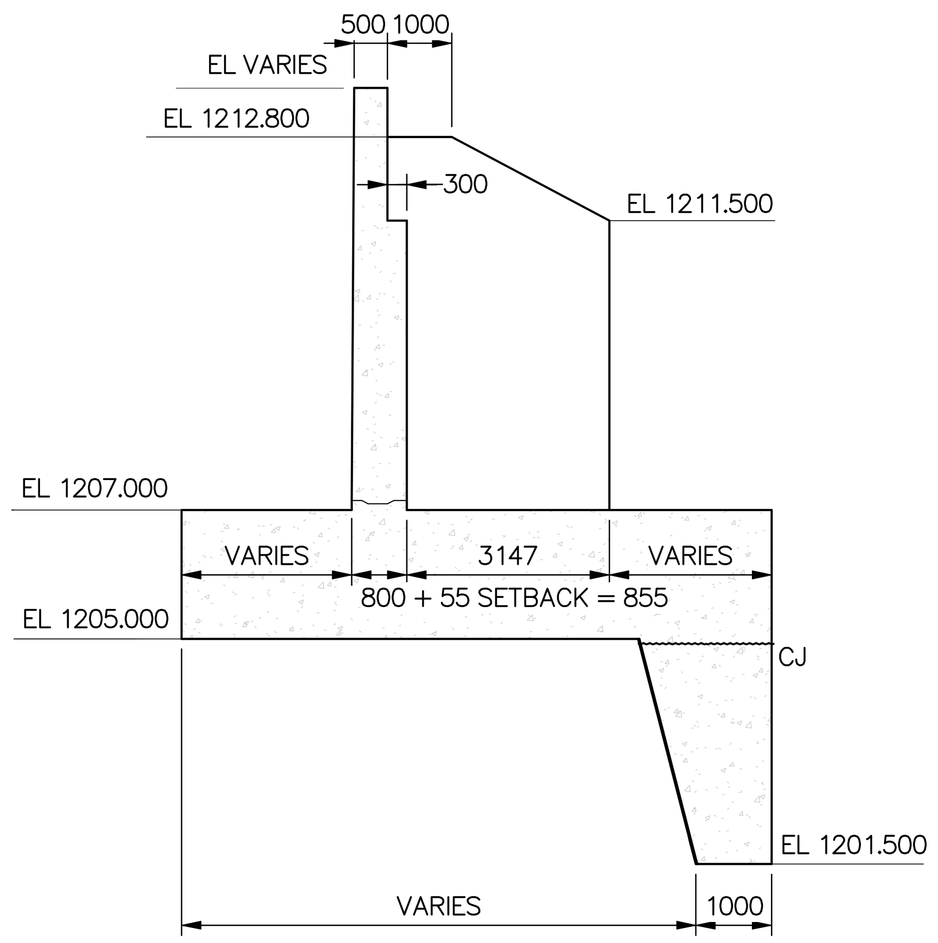

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SECTION
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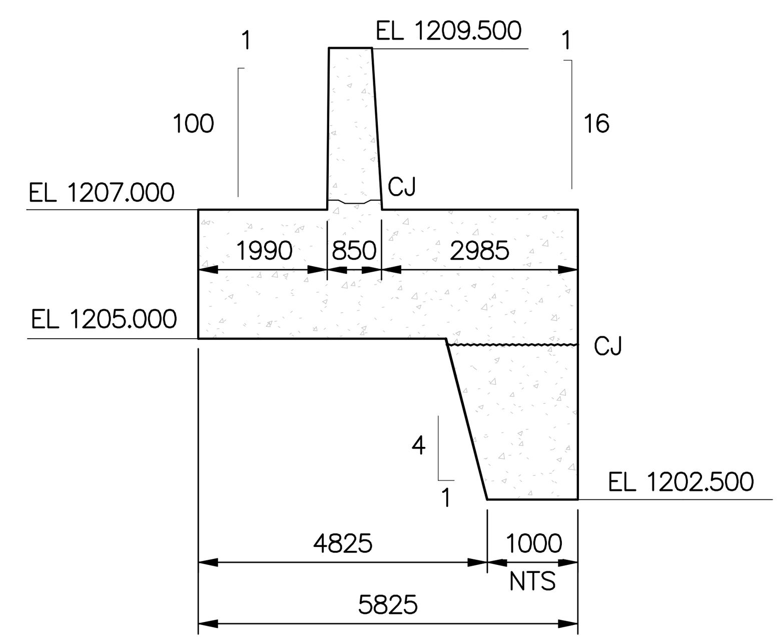

E
S-316
SECTION
1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
RETAINING WALLS - BLOCKS 1D, 1E, 5D & 5E
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-317	

Plot Date: 6/25/2020 2:38:30 PM Login: Dosler
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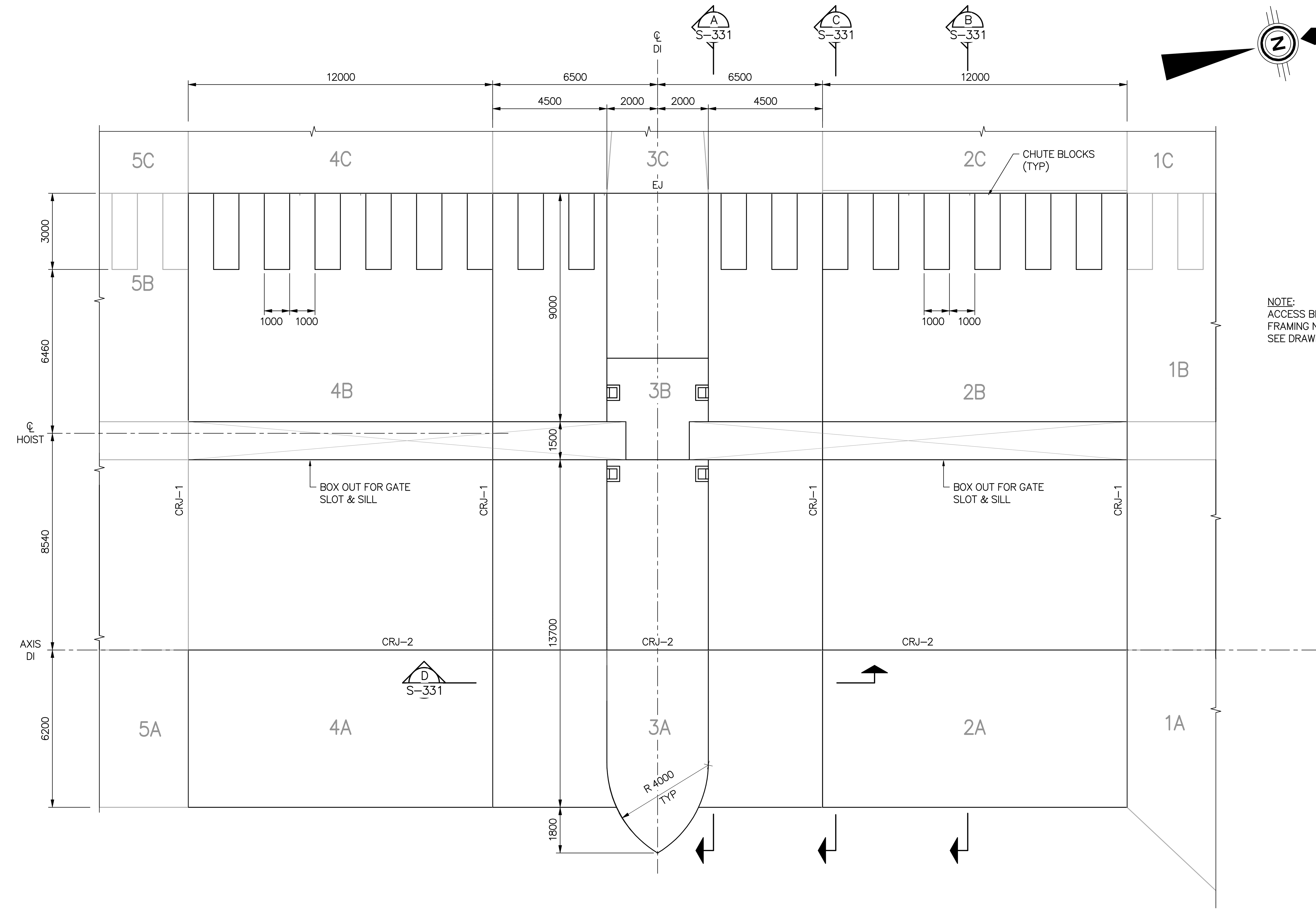
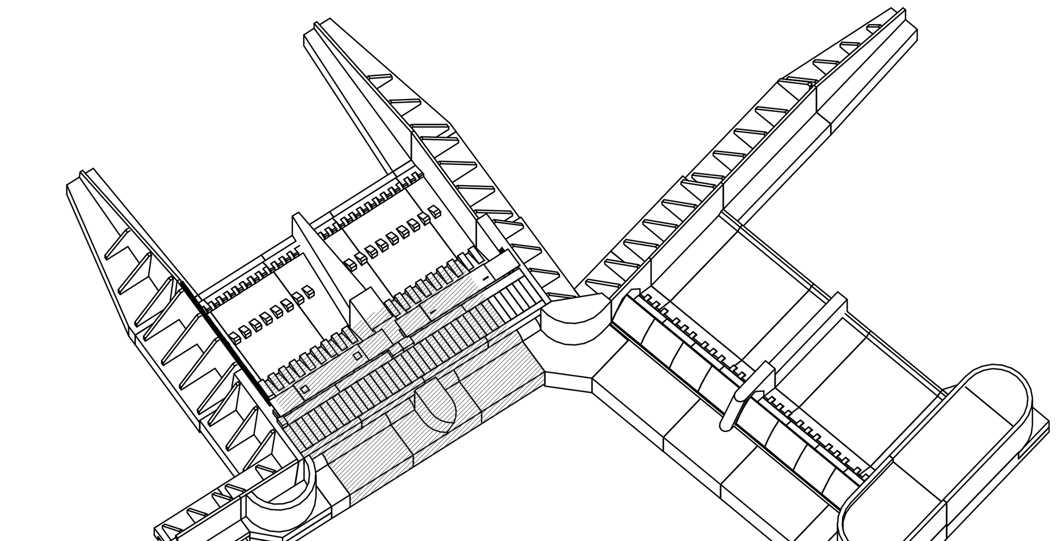


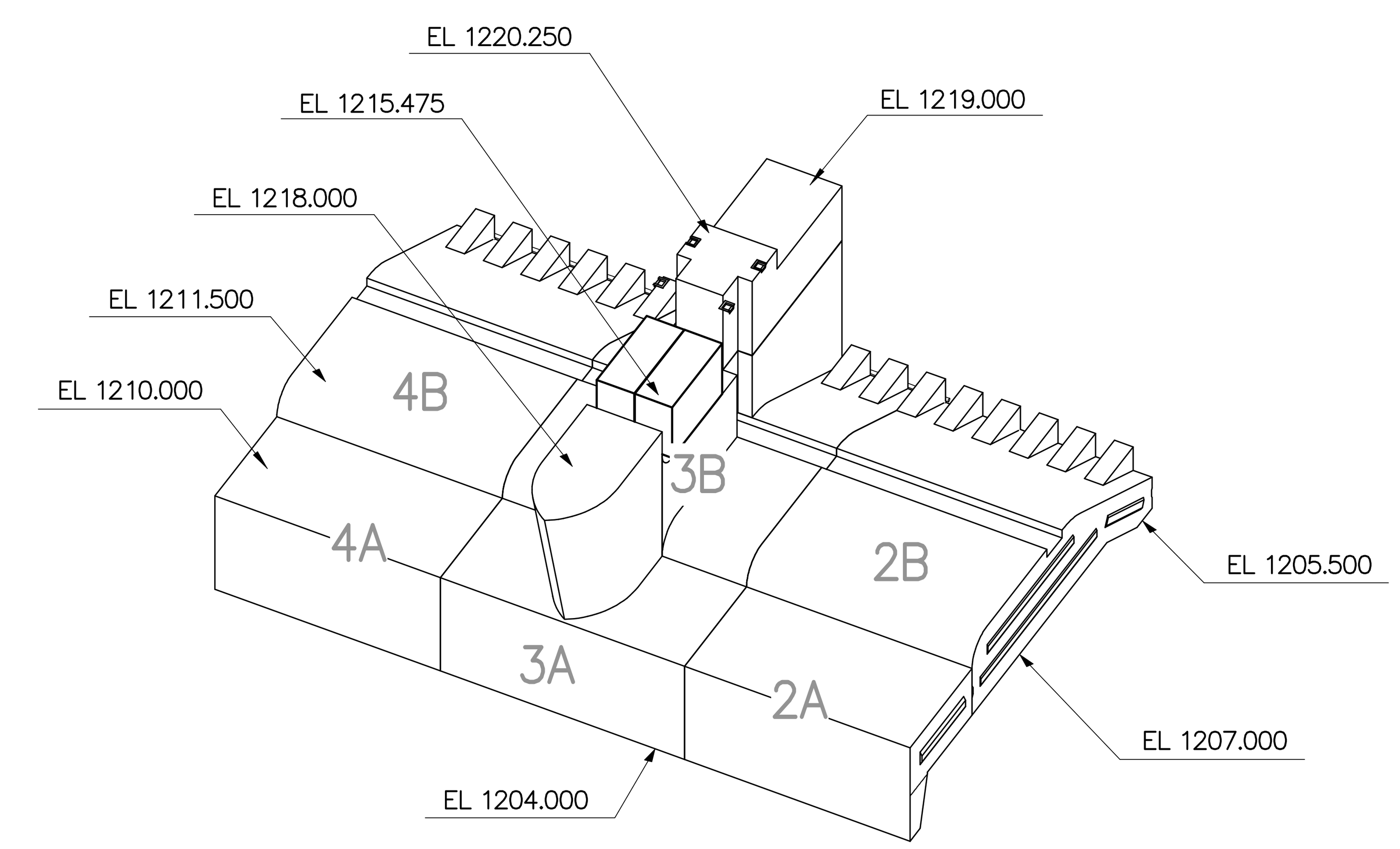
PLAN – BLOCKS 2A, 2B, 3A, 3B, 4A & 4B

1 : 100



STRUCTURAL KEY

NOTE:
ACCESS BRIDGE DECK & HOIST SUPPORT FRAMING NOT SHOWN FOR CLARITY
SEE DRAWINGS S-360 S-361 S-367



BLOCKS 2A, 2B, 3A, 3B, 4A & 4B 3D VIEW

NOTES:
1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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DESIGNER
DATE

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION INLET
GATE STRUCTURE - BLOCKS 2A, 2B, 3A, 3B, 4A & 4B
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-330	

Plot Date: 6/25/2020 2:38:31 PM Login: Dosler
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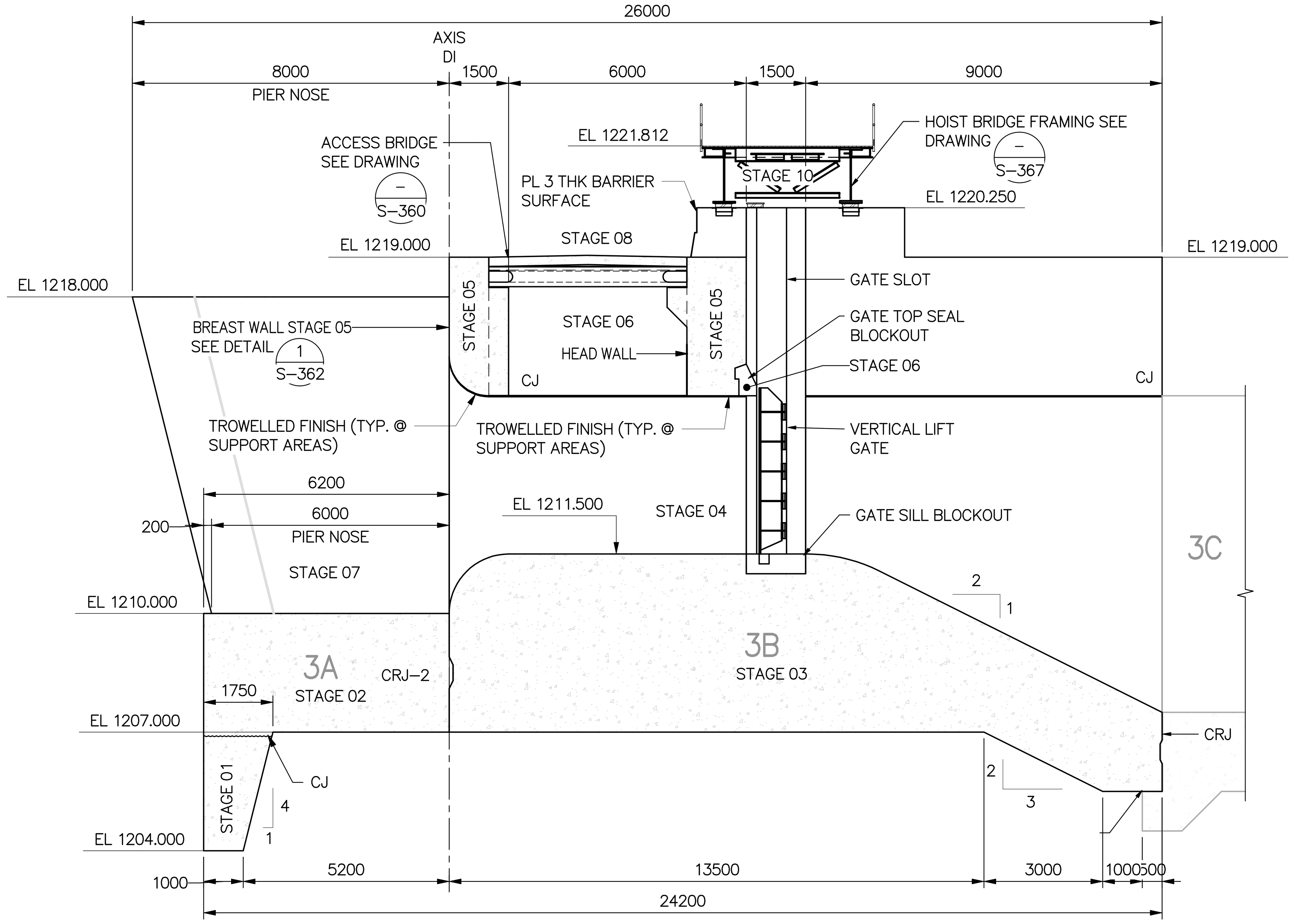
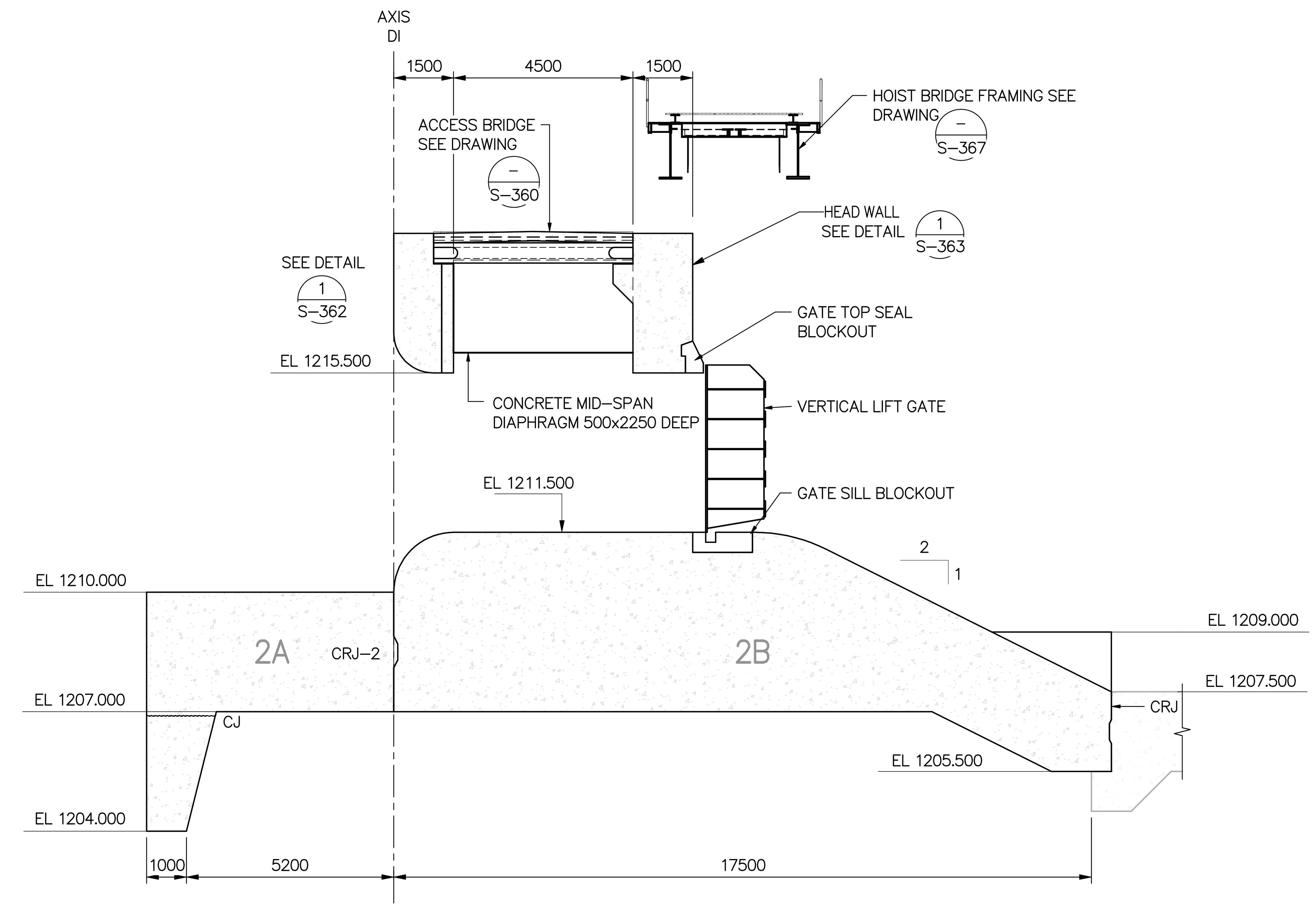


A SECTION

S-330

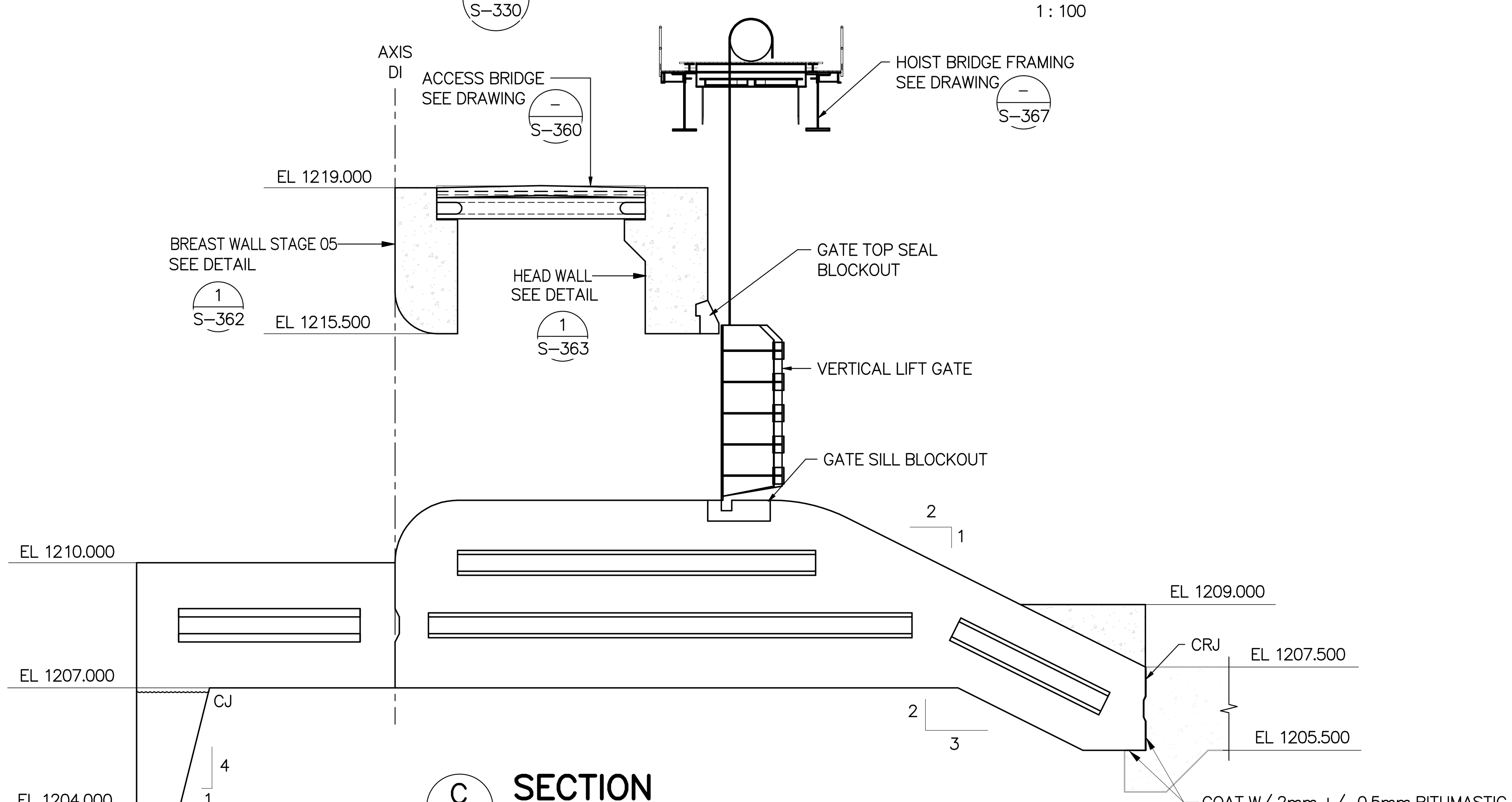
1 : 100



B SECTION

S-330

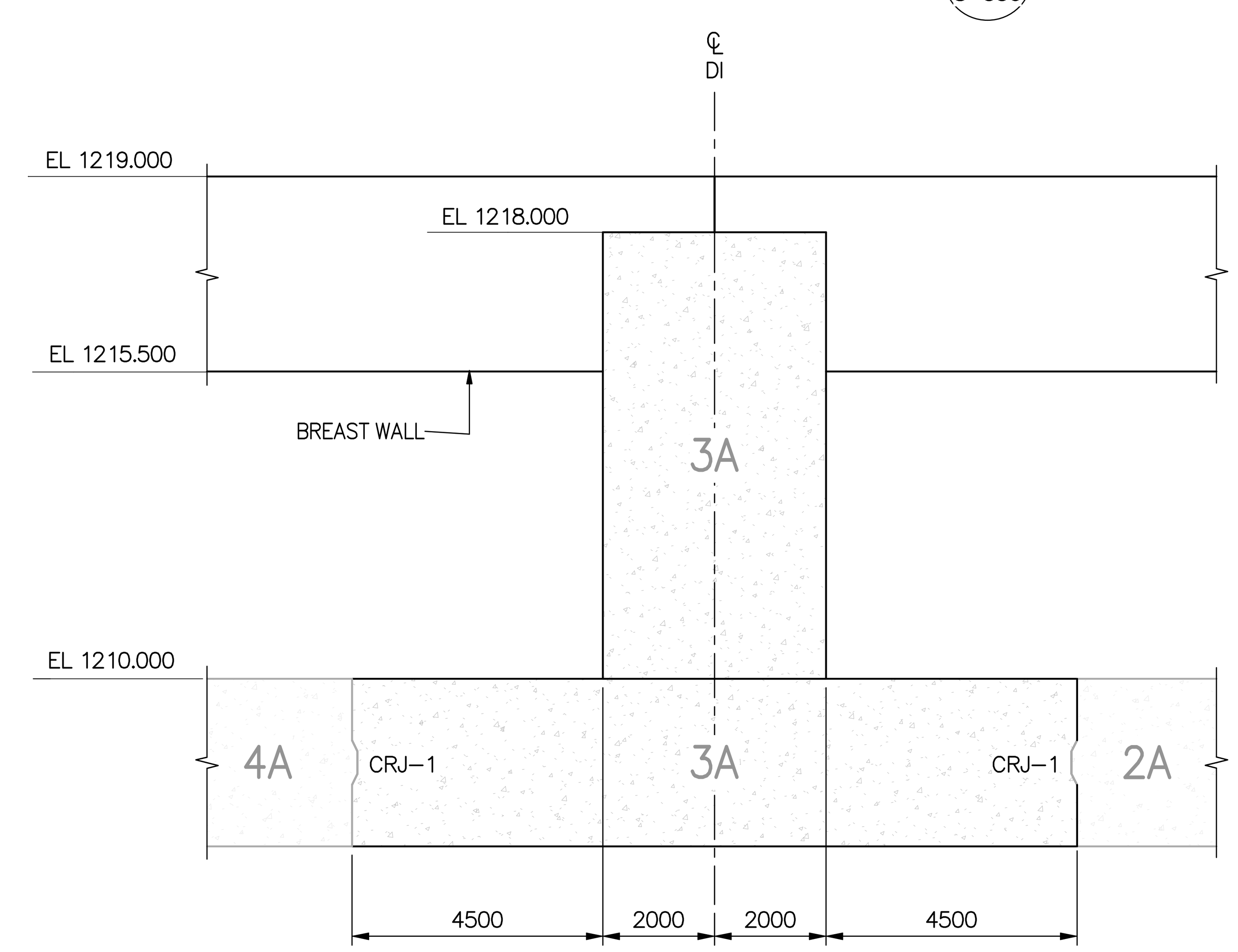
1 : 100



C SECTION

S-330

1 : 100



D SECTION

S-330

1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
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DESIGNER

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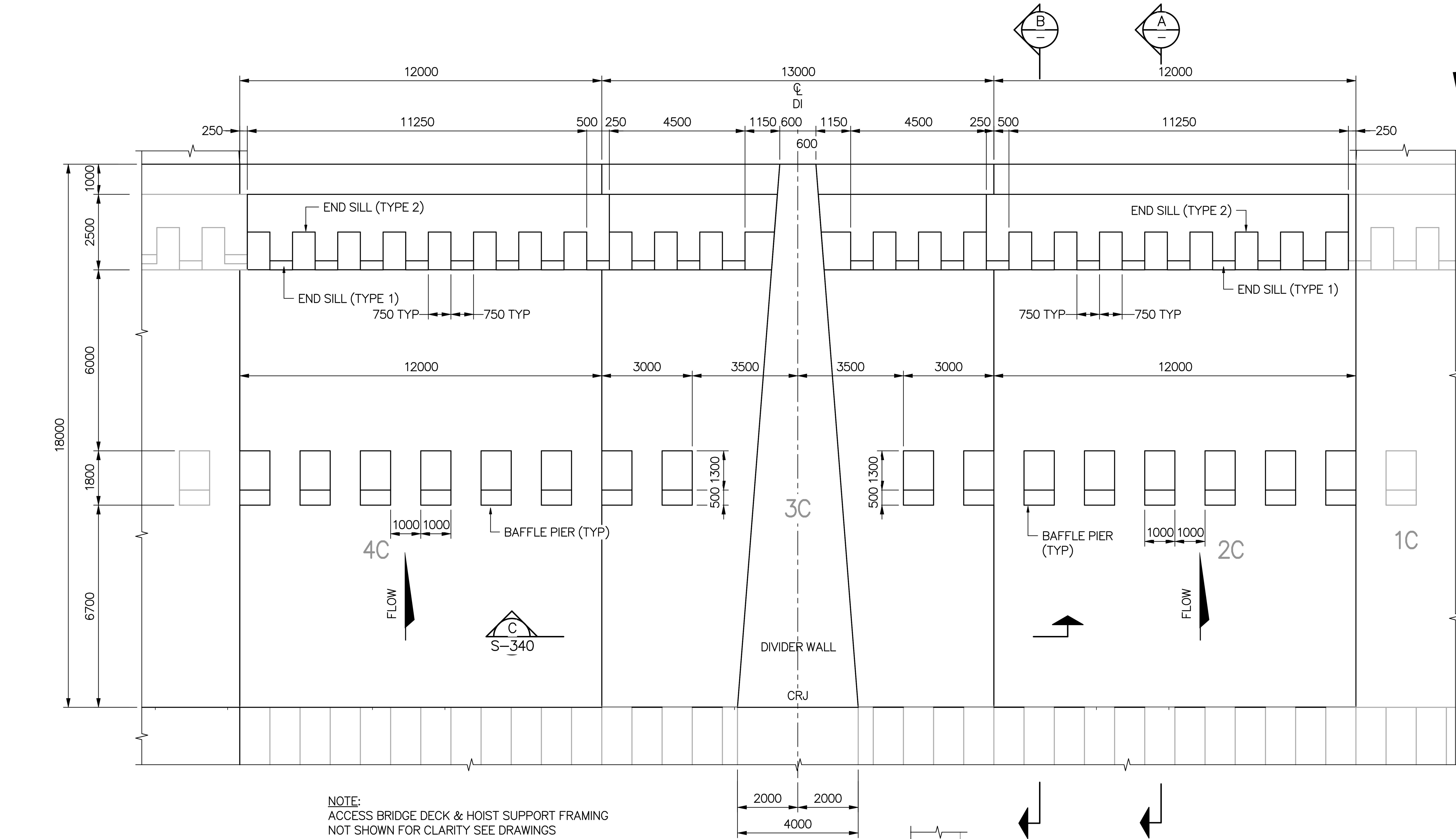
Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
GATE STRUCTURE - BLOCKS 2A, 2B, 3A, 3B, 4A & 4B
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-331	

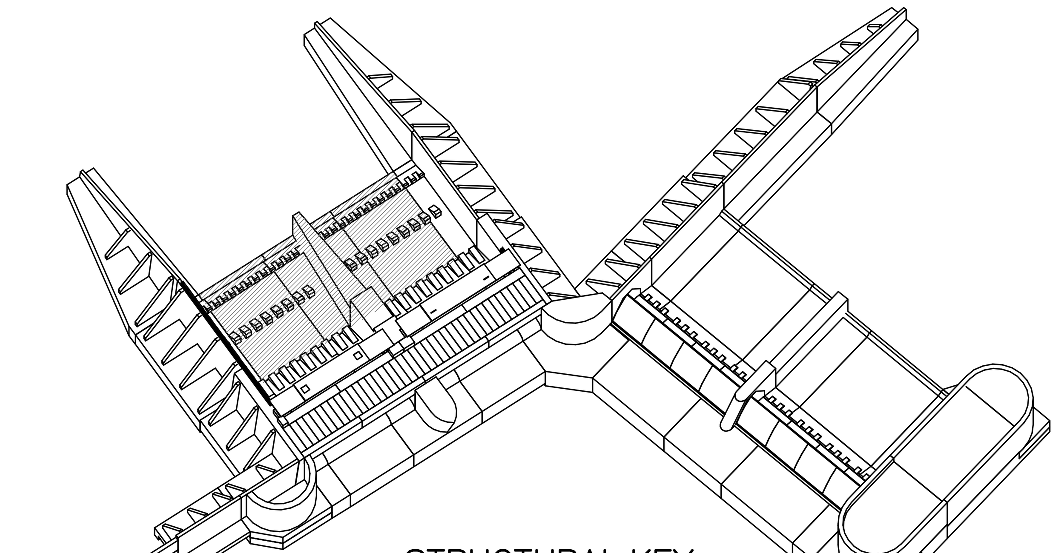


NOTE:
ACCESS BRIDGE DECK & HOIST SUPPORT FRAMING NOT SHOWN FOR CLARITY SEE DRAWINGS

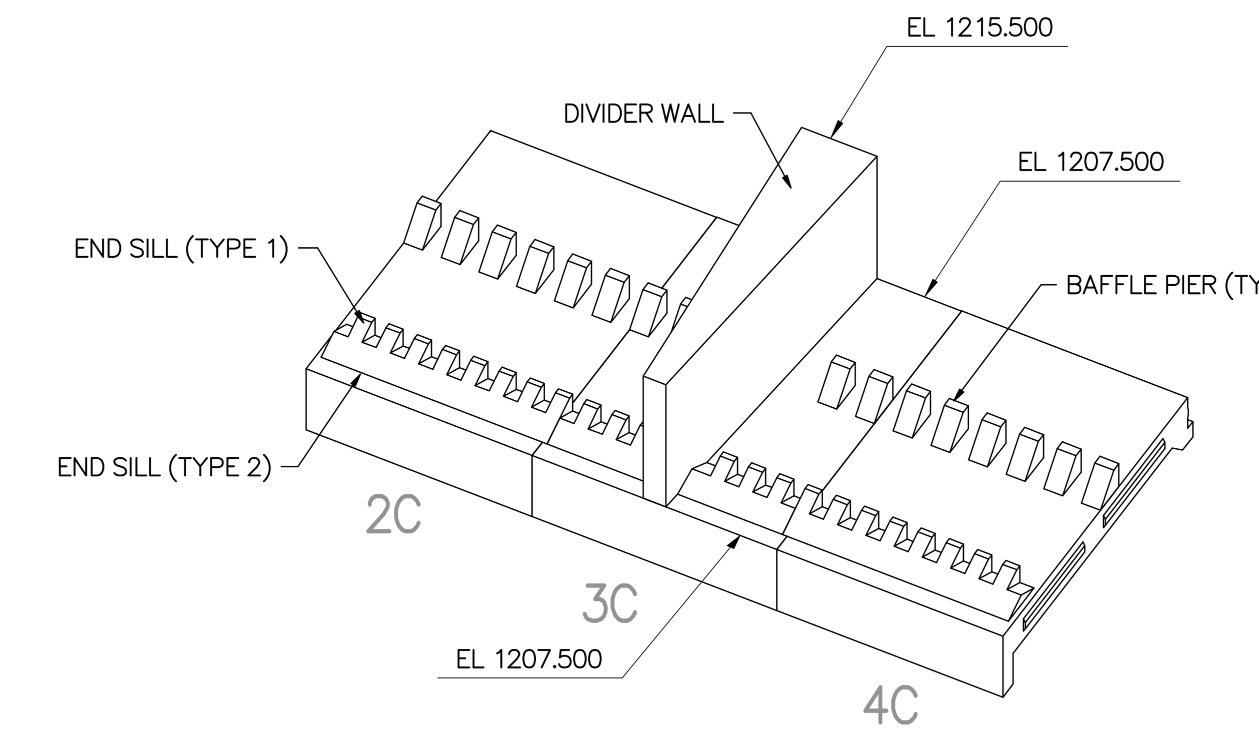
S-360 S-361 S-367

PLAN

1 : 100



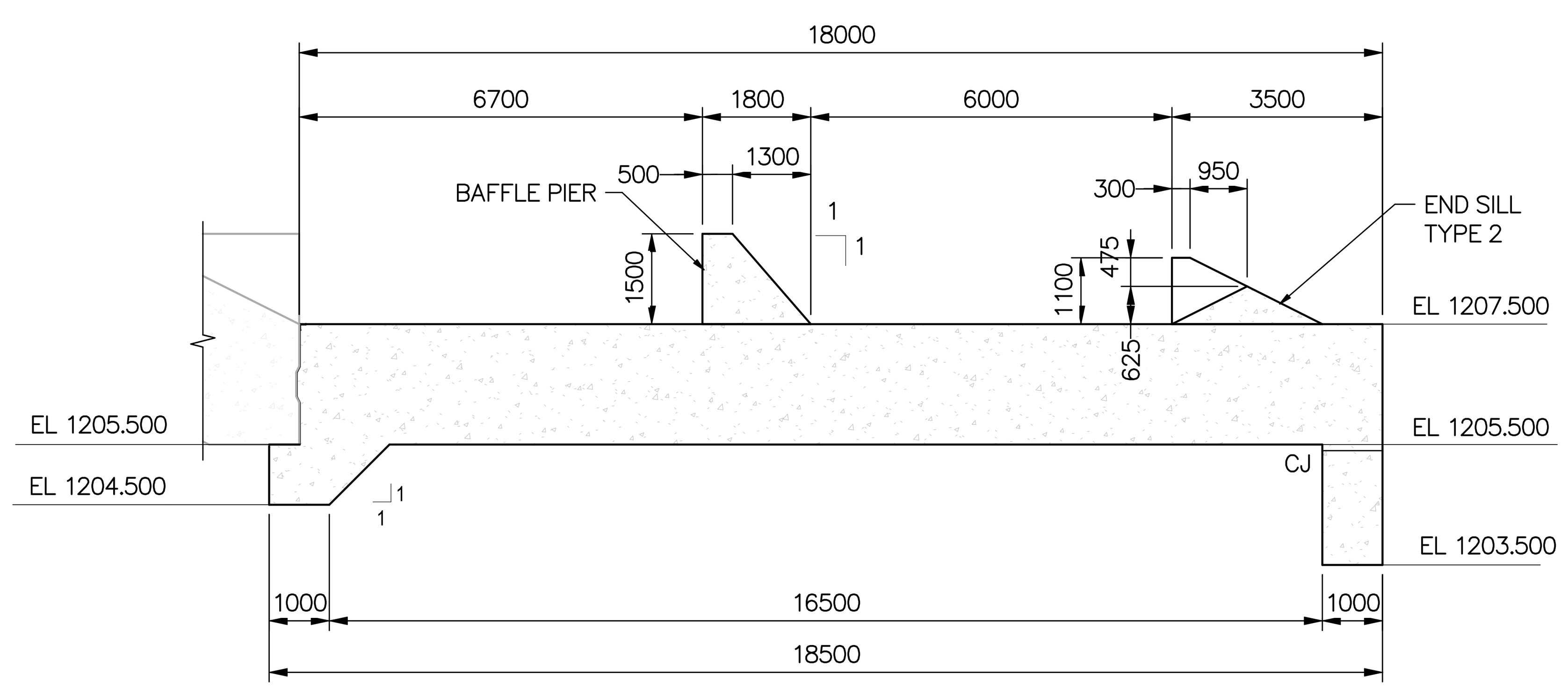
STRUCTURAL KEY



BLOCKS 2C, 3C & 4C 3D VIEW

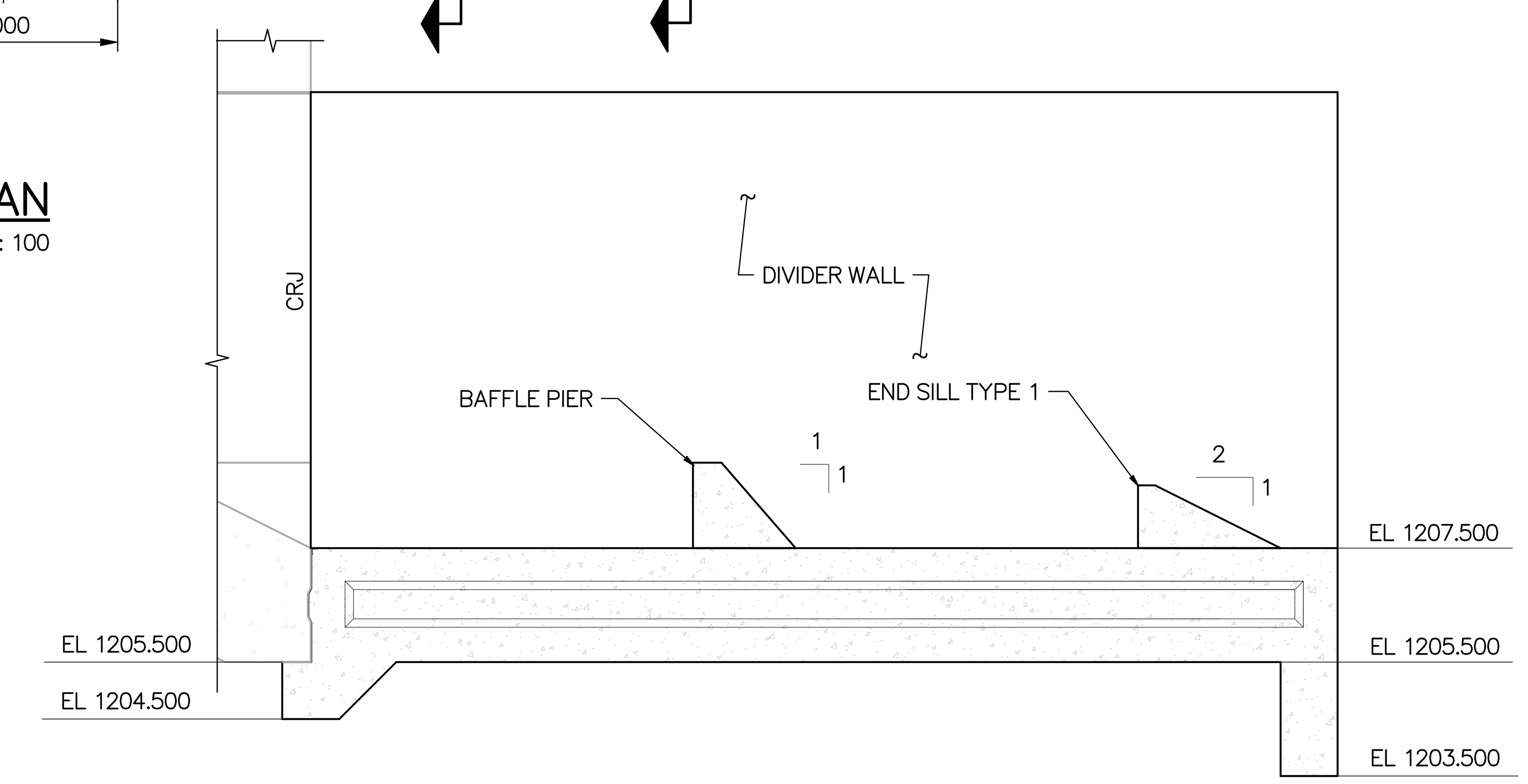
NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.



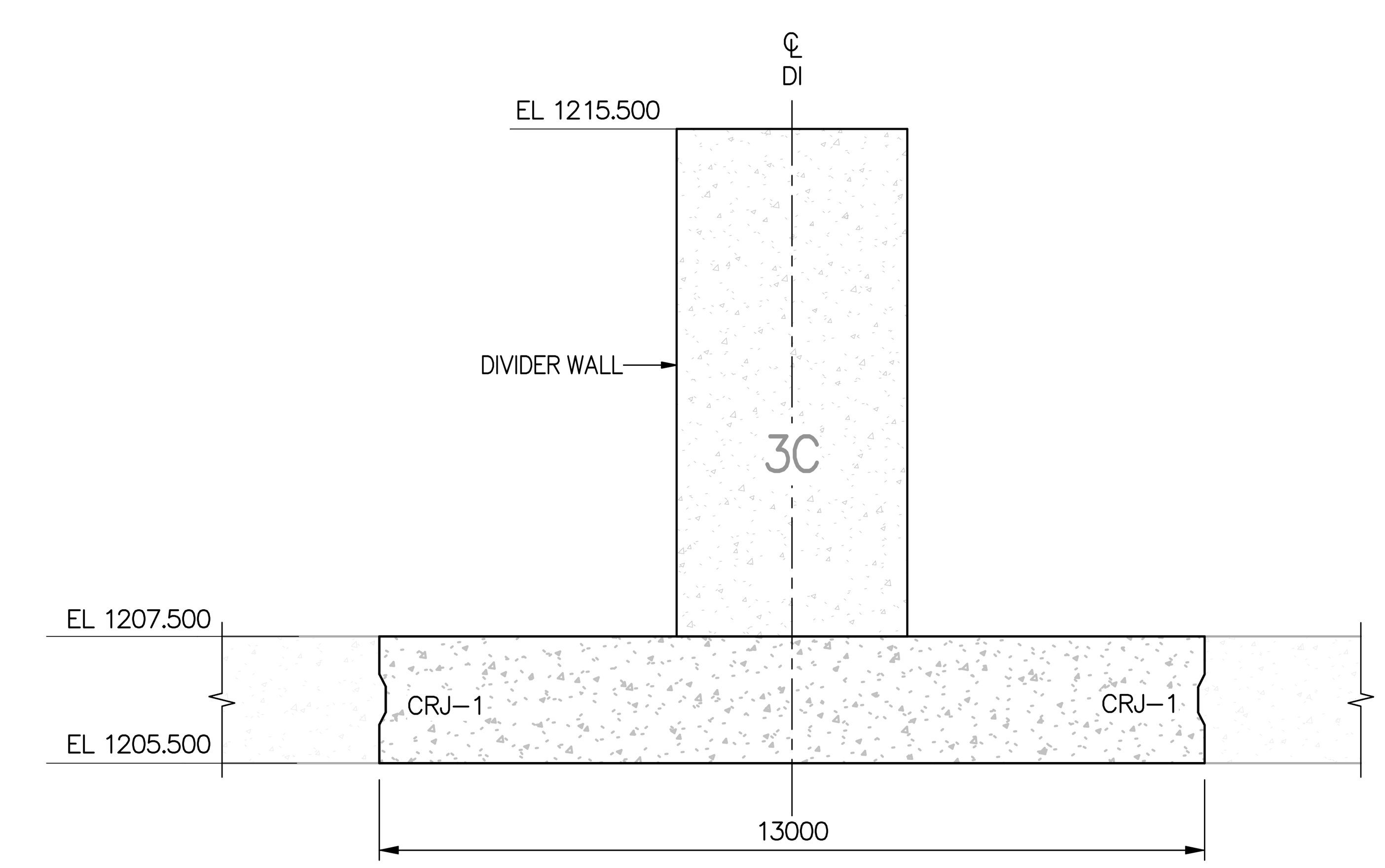
A SECTION

1 : 100



B SECTION

1 : 100



C SECTION

1 : 100

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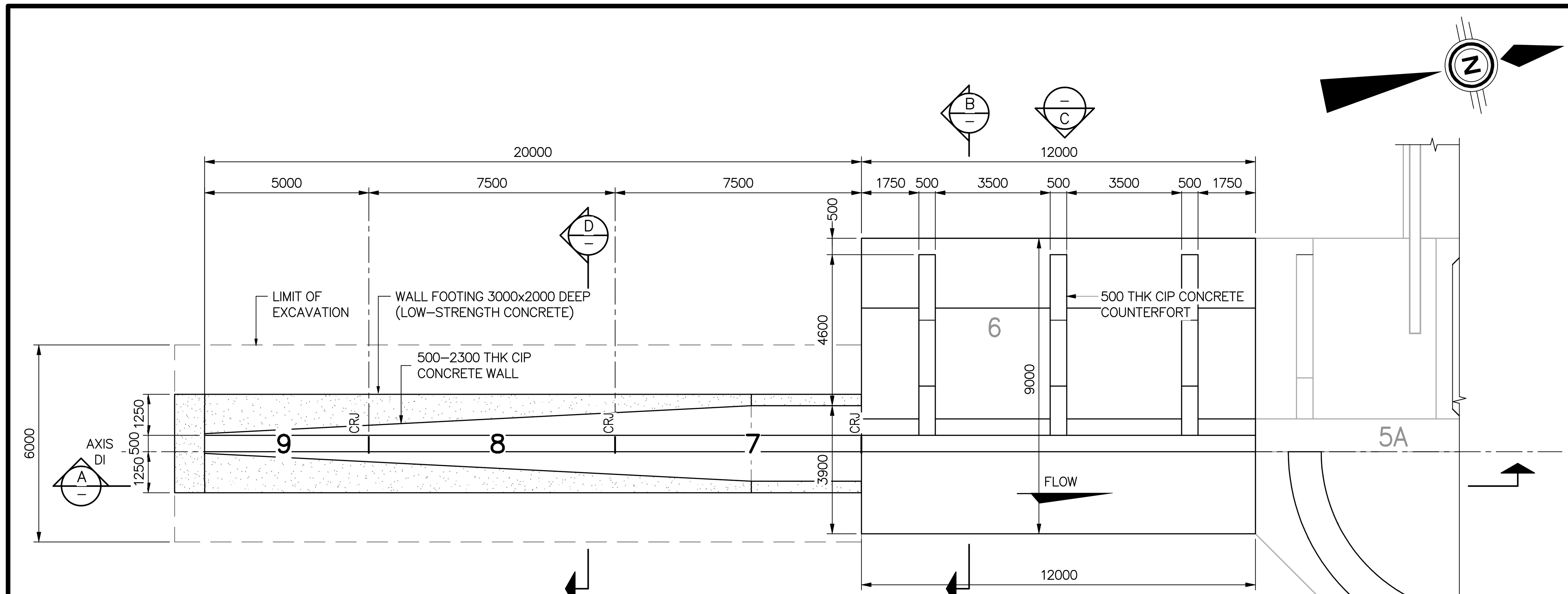
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
STILLING BASIN - BLOCKS 2C, 3C & 4C
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-340	



PLAN

1 : 100

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
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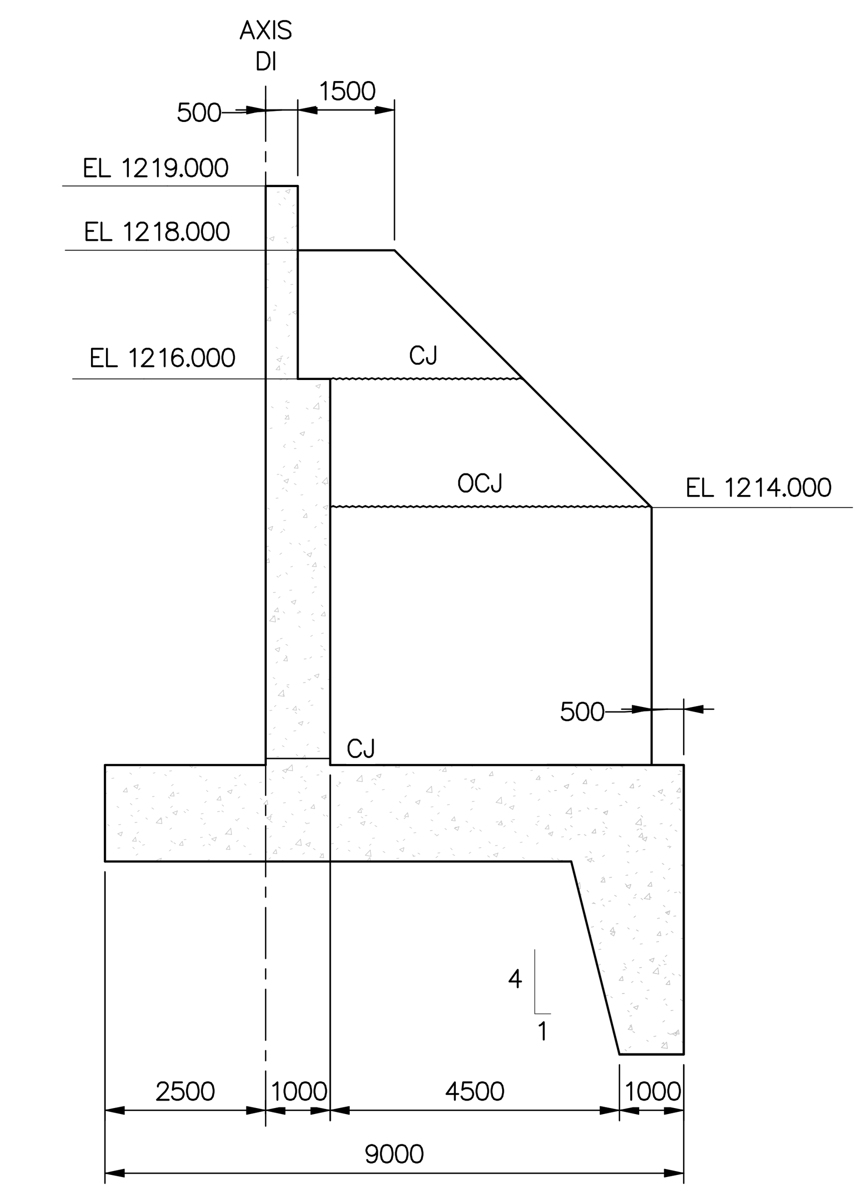
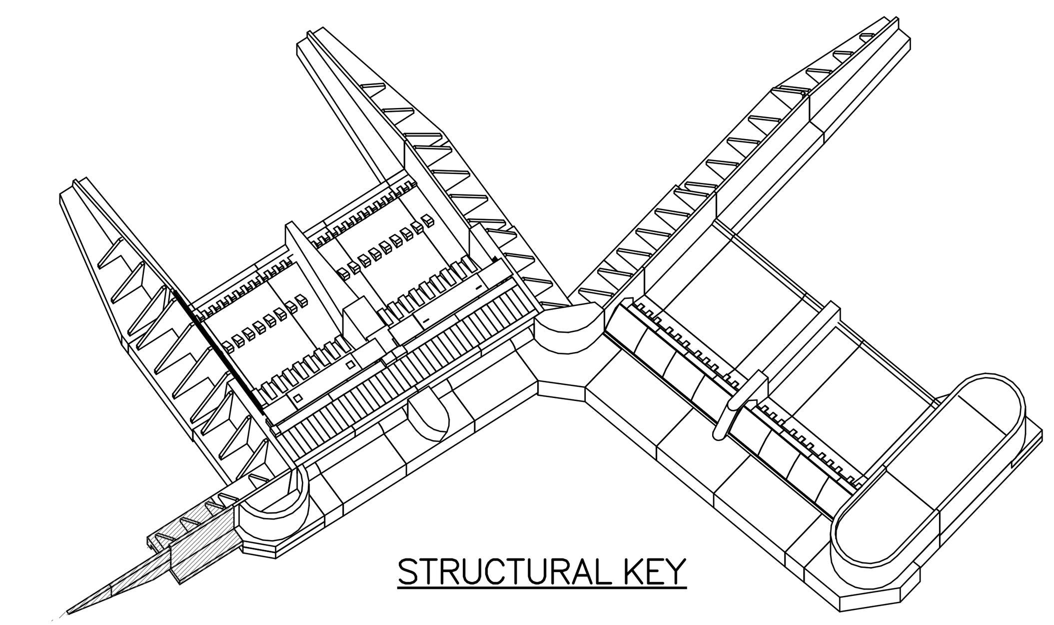
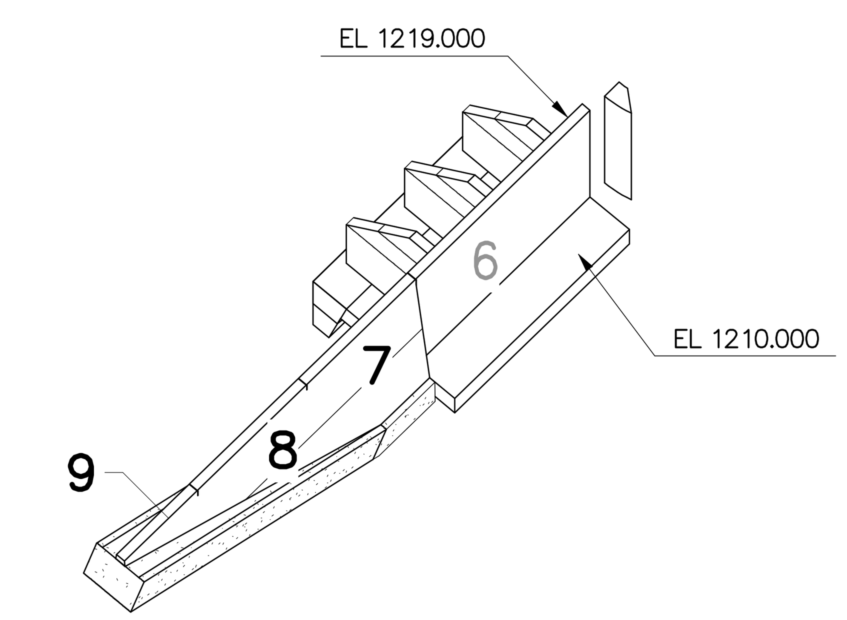


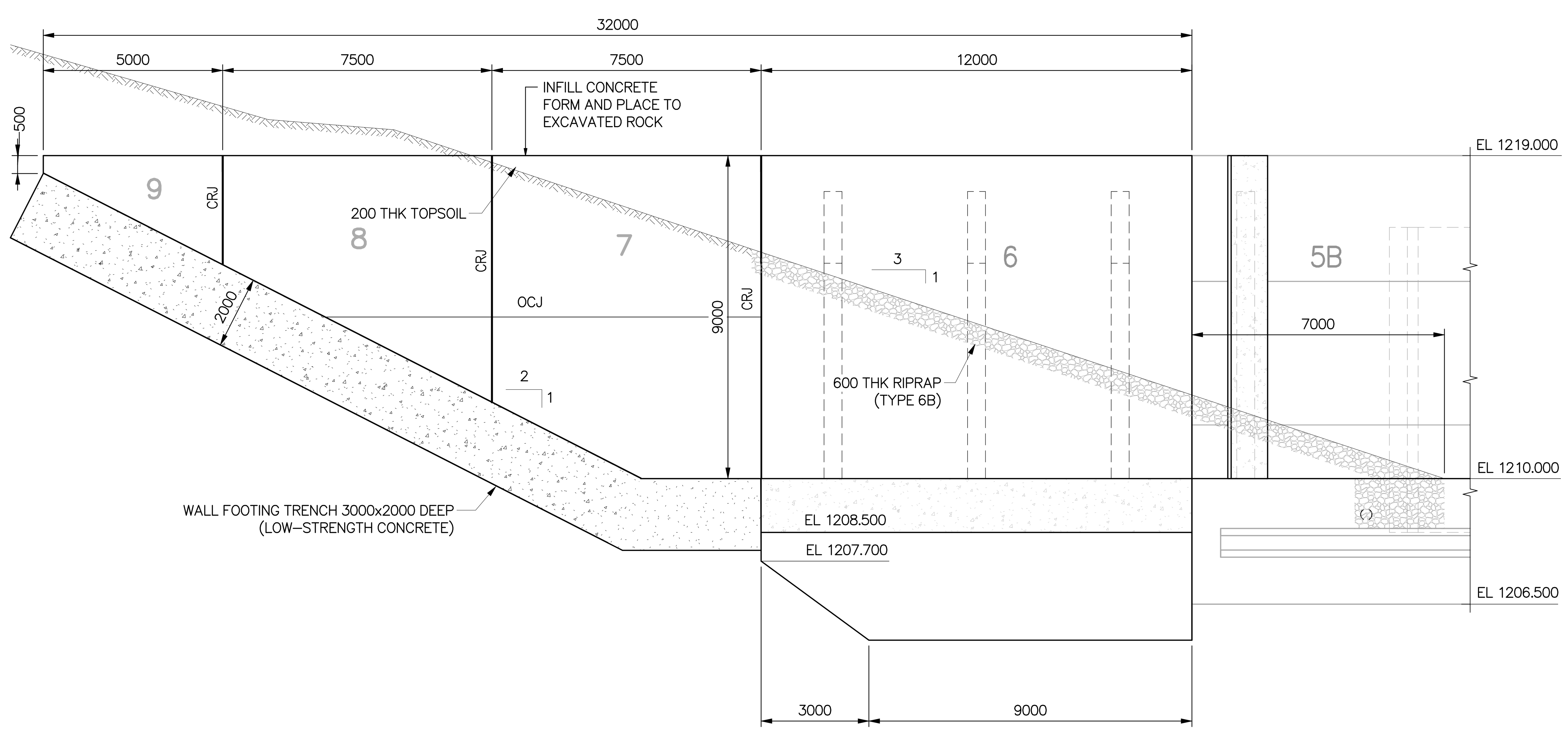
B SECTION

1 : 100



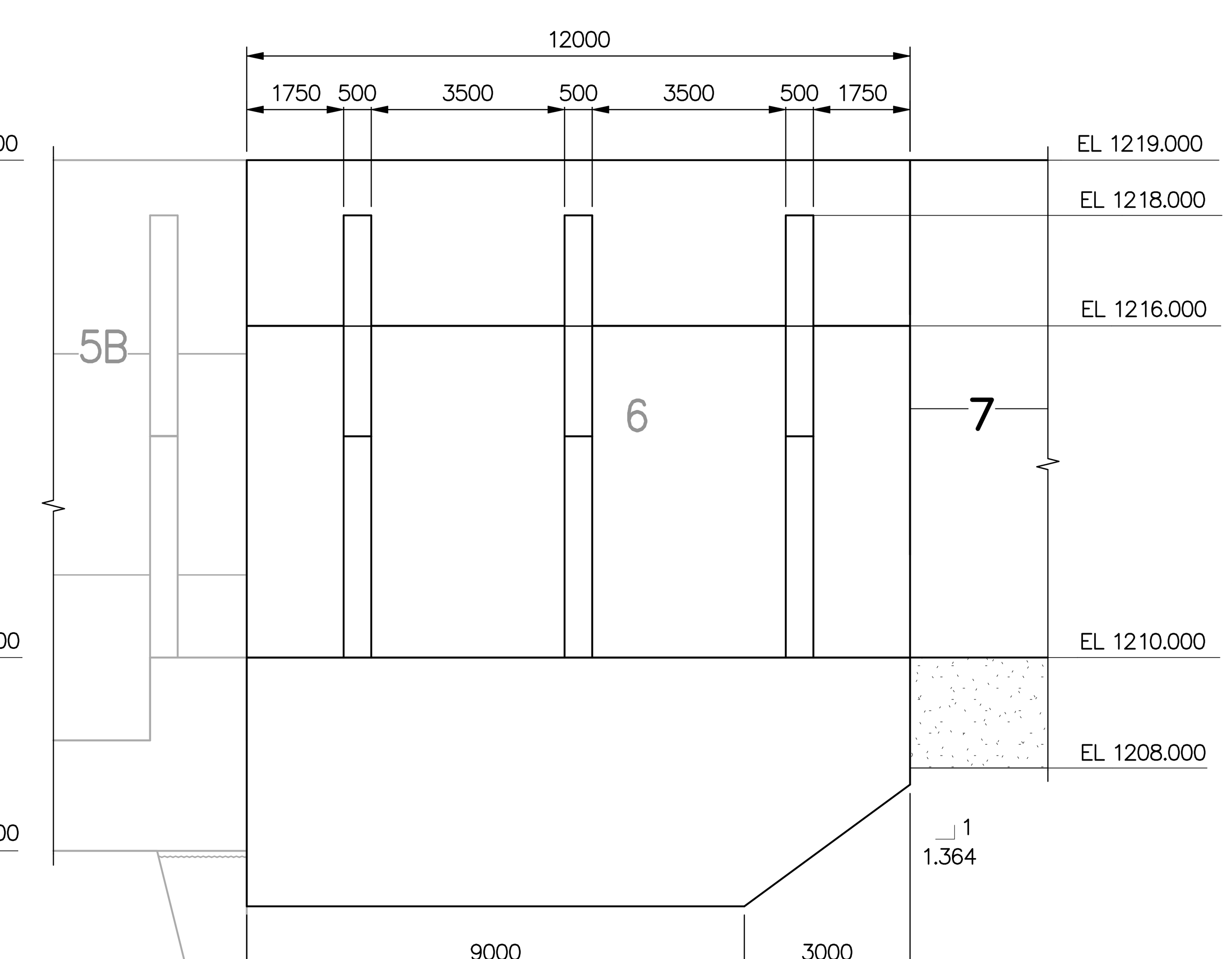


BLOCKS 6, 7 & 8 3D VIEW



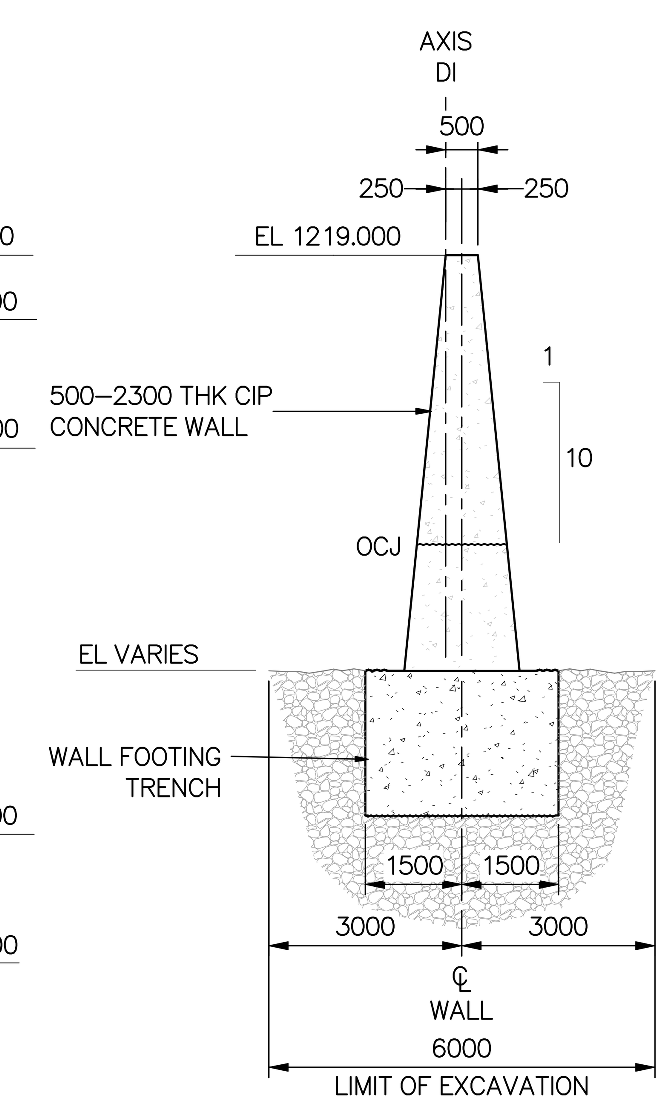
A SECTION

1 : 100



C ELEVATION

1 : 100



D SECTION

1 : 100

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
LEFT ABUTMENT - BLOCKS 6, 7, 8 & 9
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
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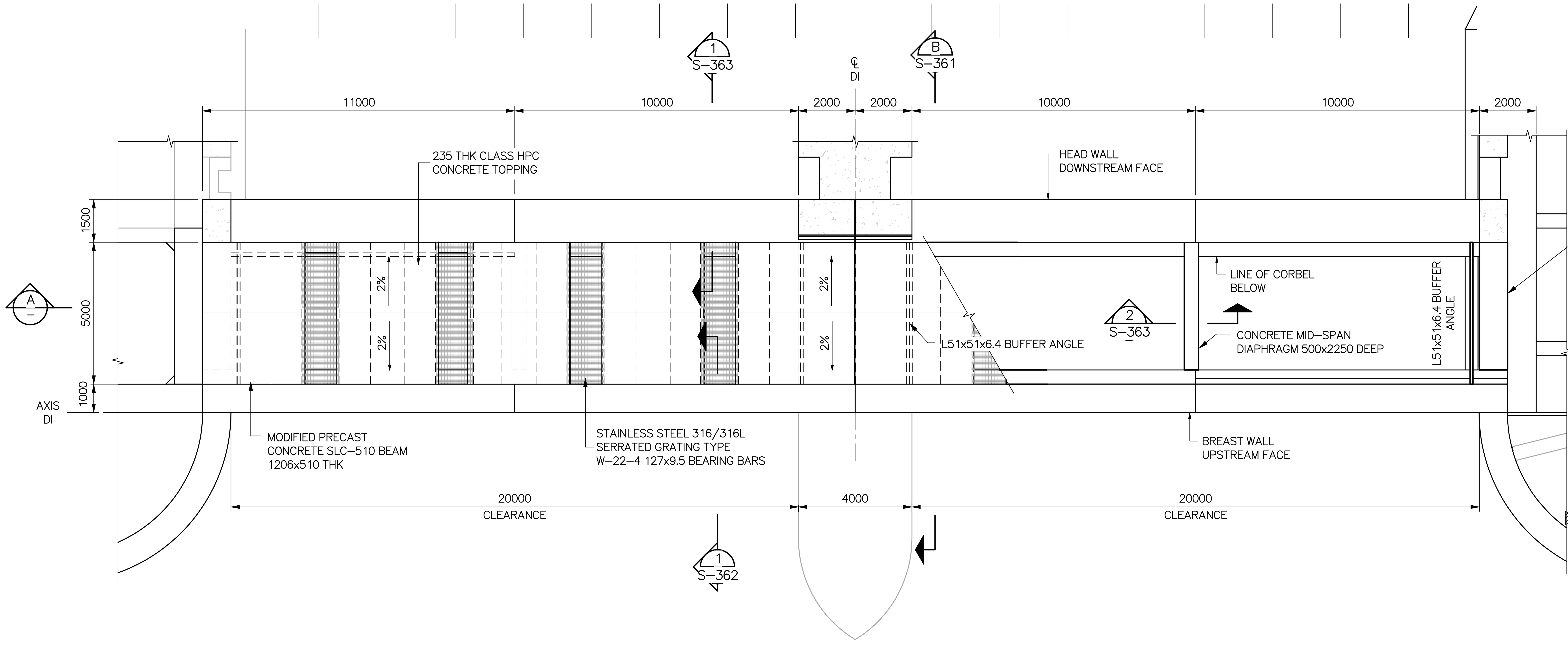
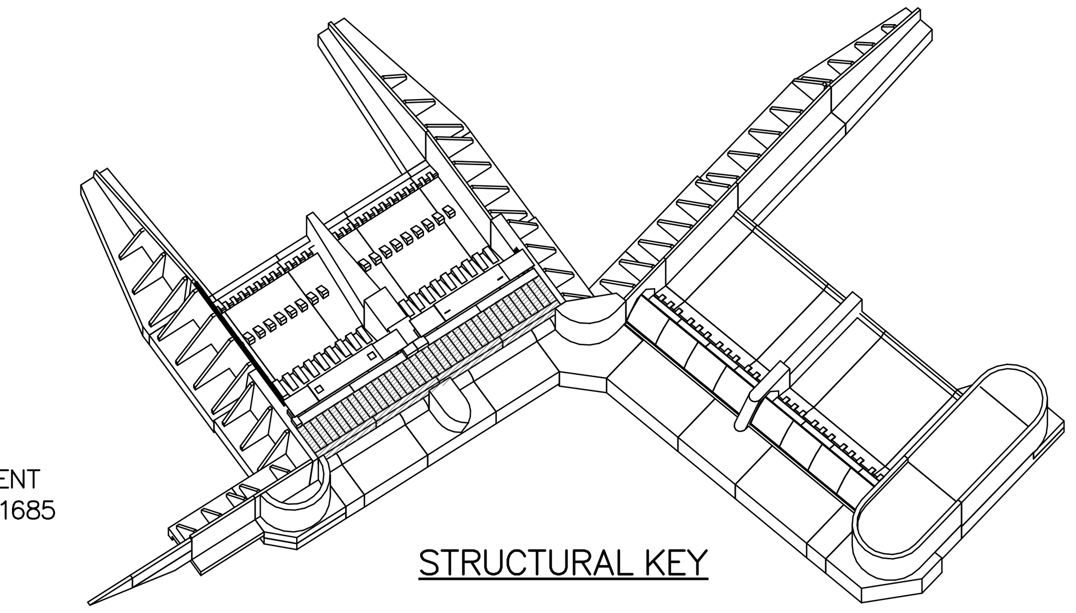
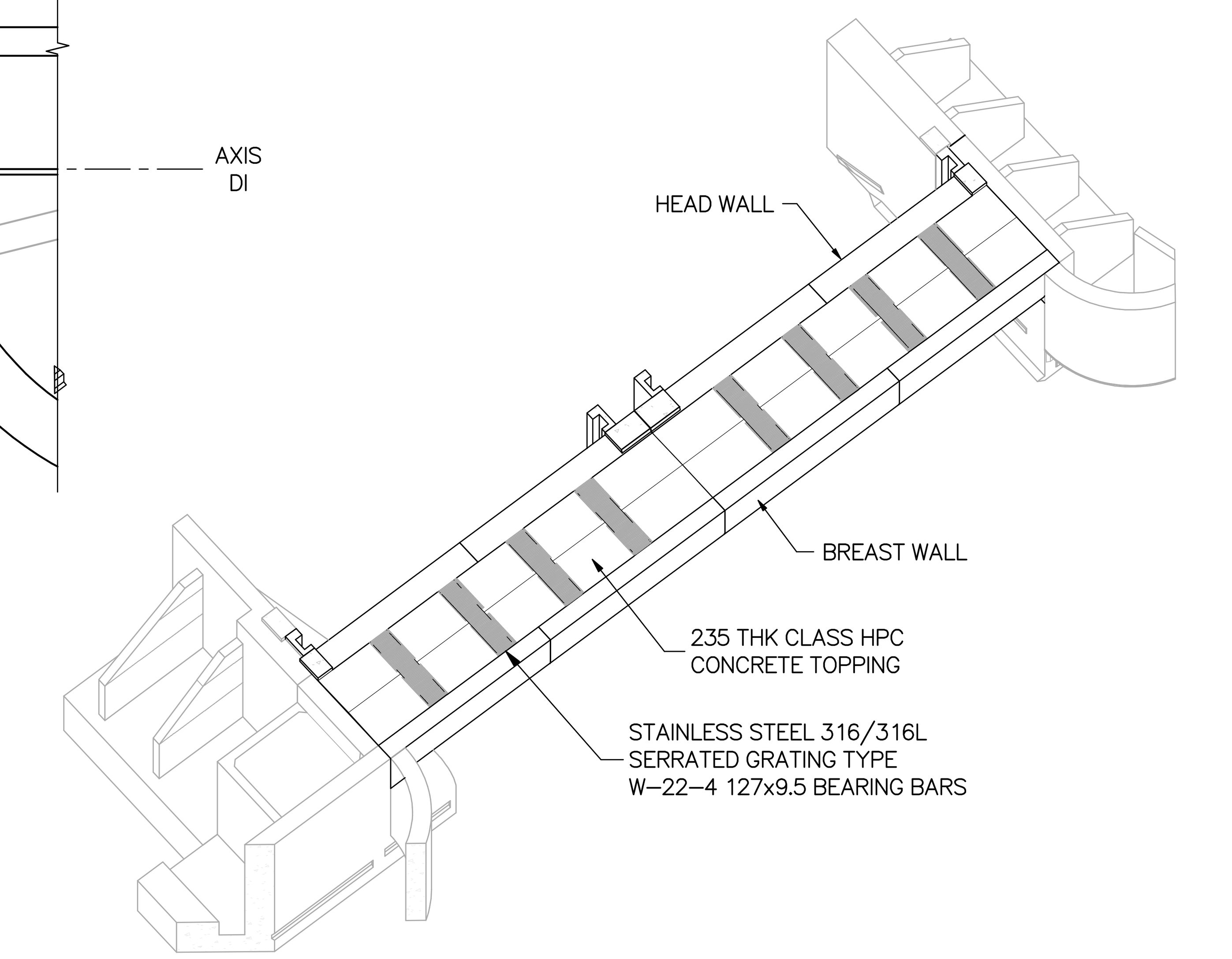


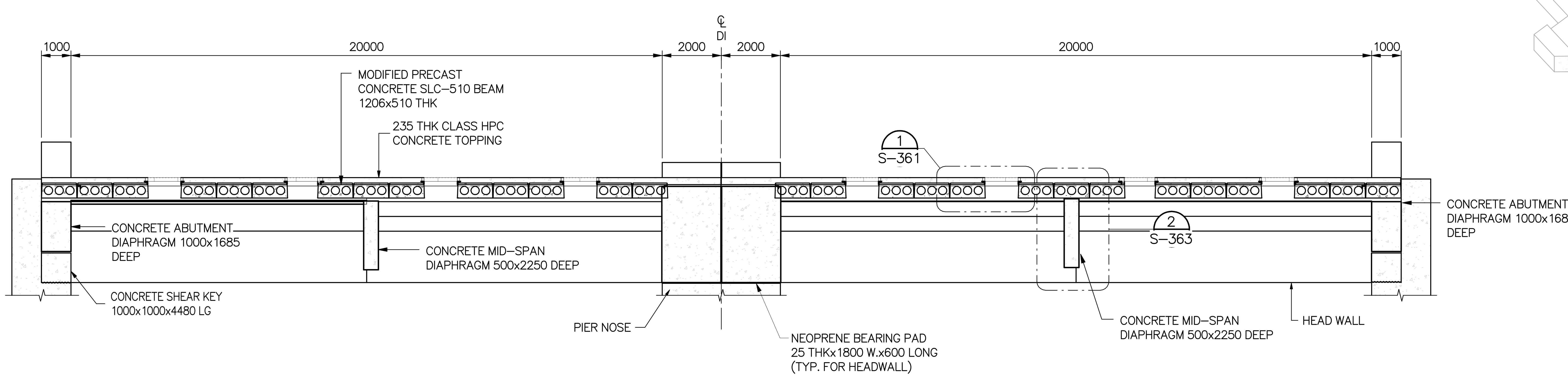
PLAN

1 : 100





ACCESS BRIDGE DECK 3D VIEW



A SECTION

1 : 100

PRECAST CONCRETE DECK SLAB MATERIALS

CONCRETE SHALL BE STANDARD WEIGHT, CONTAINING 10% SILICA FUME, AND NOT LESS THAN 5% AIR ENTRAINMENT

1. CONCRETE RELEASE STRENGTH: 35 MPa.
2. PRESTRESSING STEEL SHALL BE 15Ø, 7-WIRE LOW RELAXATION STRAND (fpu = 1860 MPa)
3. DECK CONCRETE TOPPING SHALL BE CLASS HPC, WITH CORROSION-RESISTANT REINFORCING BARS

NOTES:

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
GATE STRUCTURE - ACCESS BRIDGE DECK

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-360	

Plot Date: 6/25/2020 2:38:44 PM Login: Dosler
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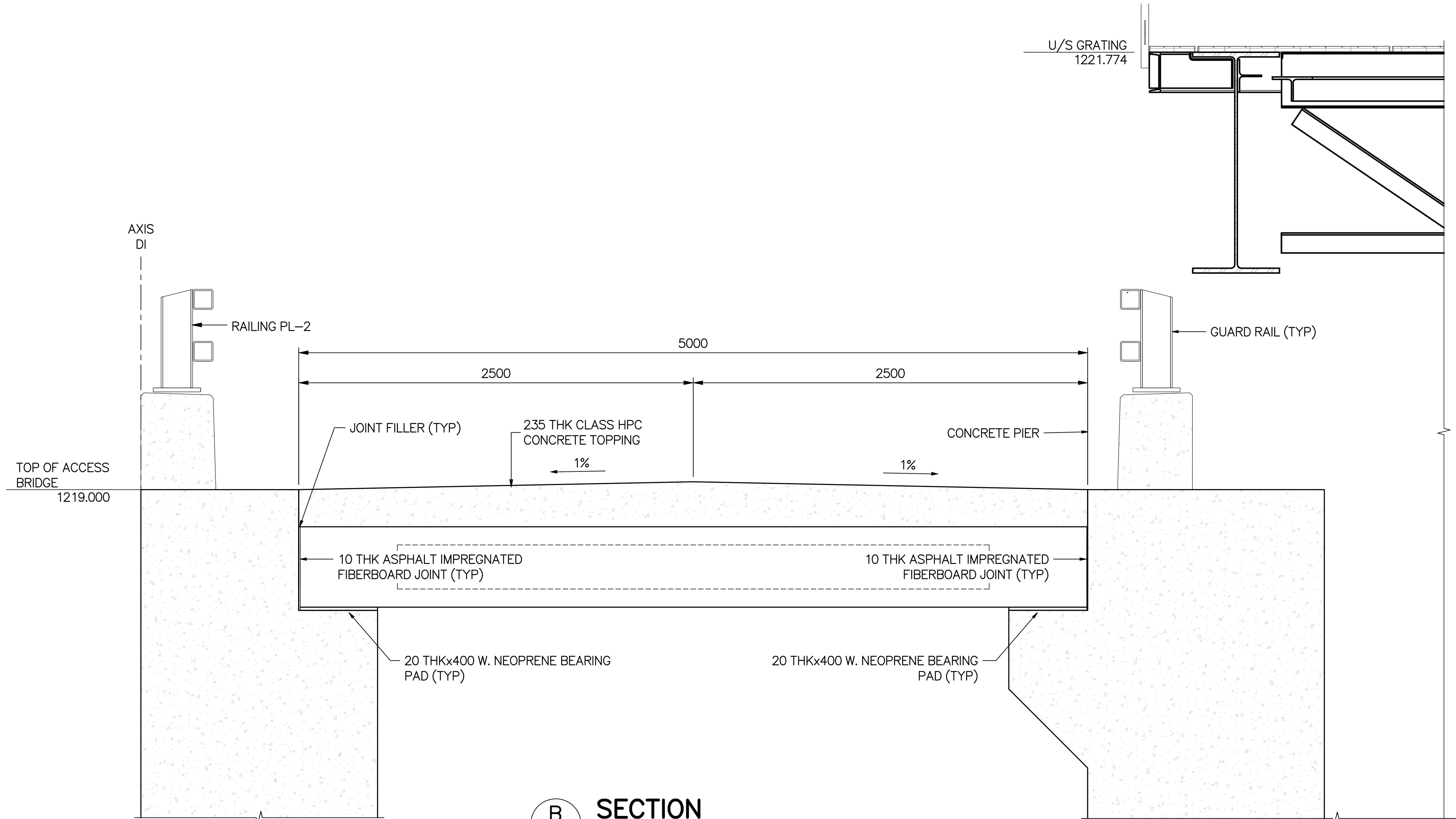
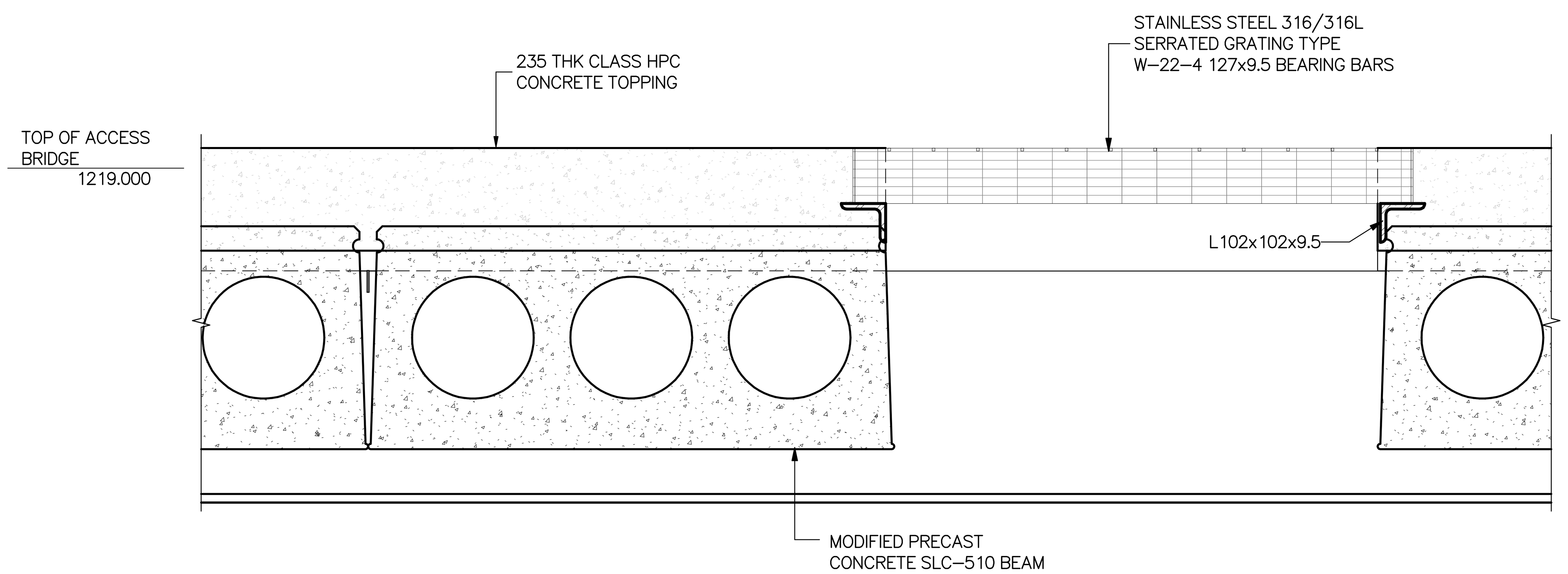


B
S–360
SECTION
1 : 20



1
S–360
DETAIL
1 : 10

NOTES:

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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
GATE STRUCTURE - ACCESS BRIDGE DECK
SECTIONS & DETAILS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-361	

Plot Date: 6/25/2020 2:38:47 PM Login: Dosler
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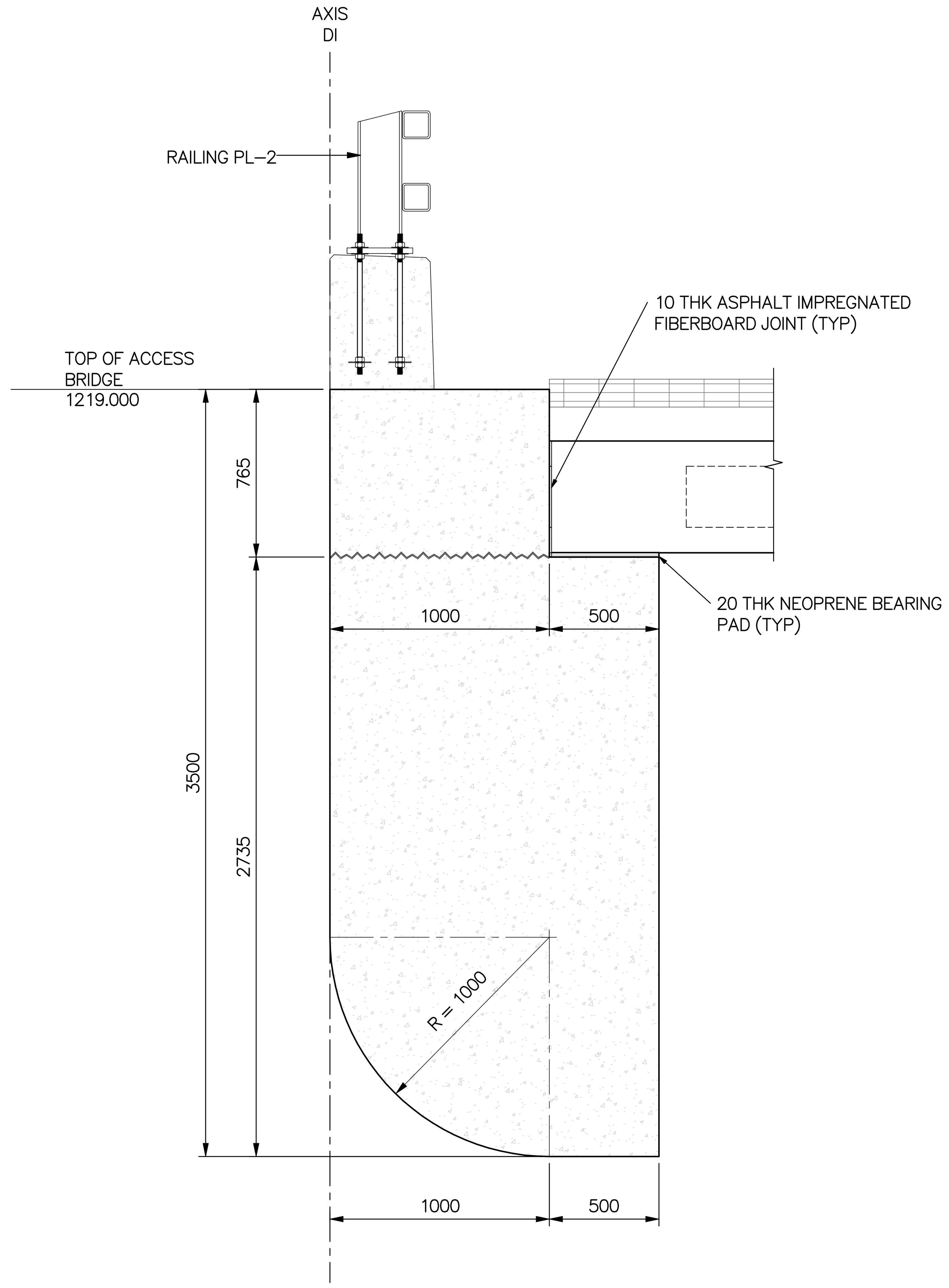
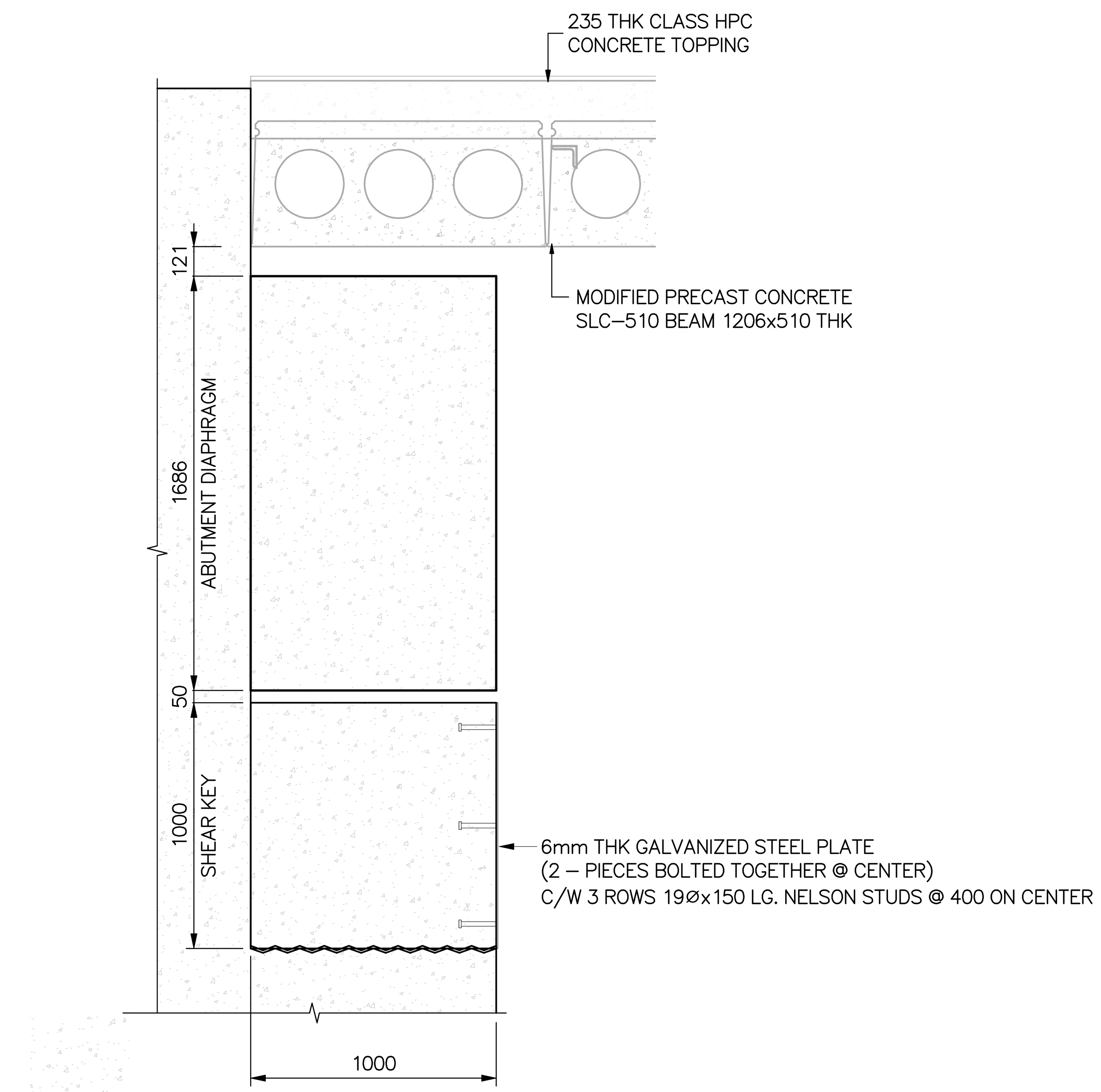


1 SECTION
S–360
1 : 20



2 SECTION – ABUTMENT DIAPHRAGM
BEAM & CONCRETE SHEAR KEY
1 : 20

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DATE	DATE

Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
GATE STRUCTURE - BREAST WALL
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-362	

Plot Date: 6/25/2020 2:38:48 PM Login: Dosler
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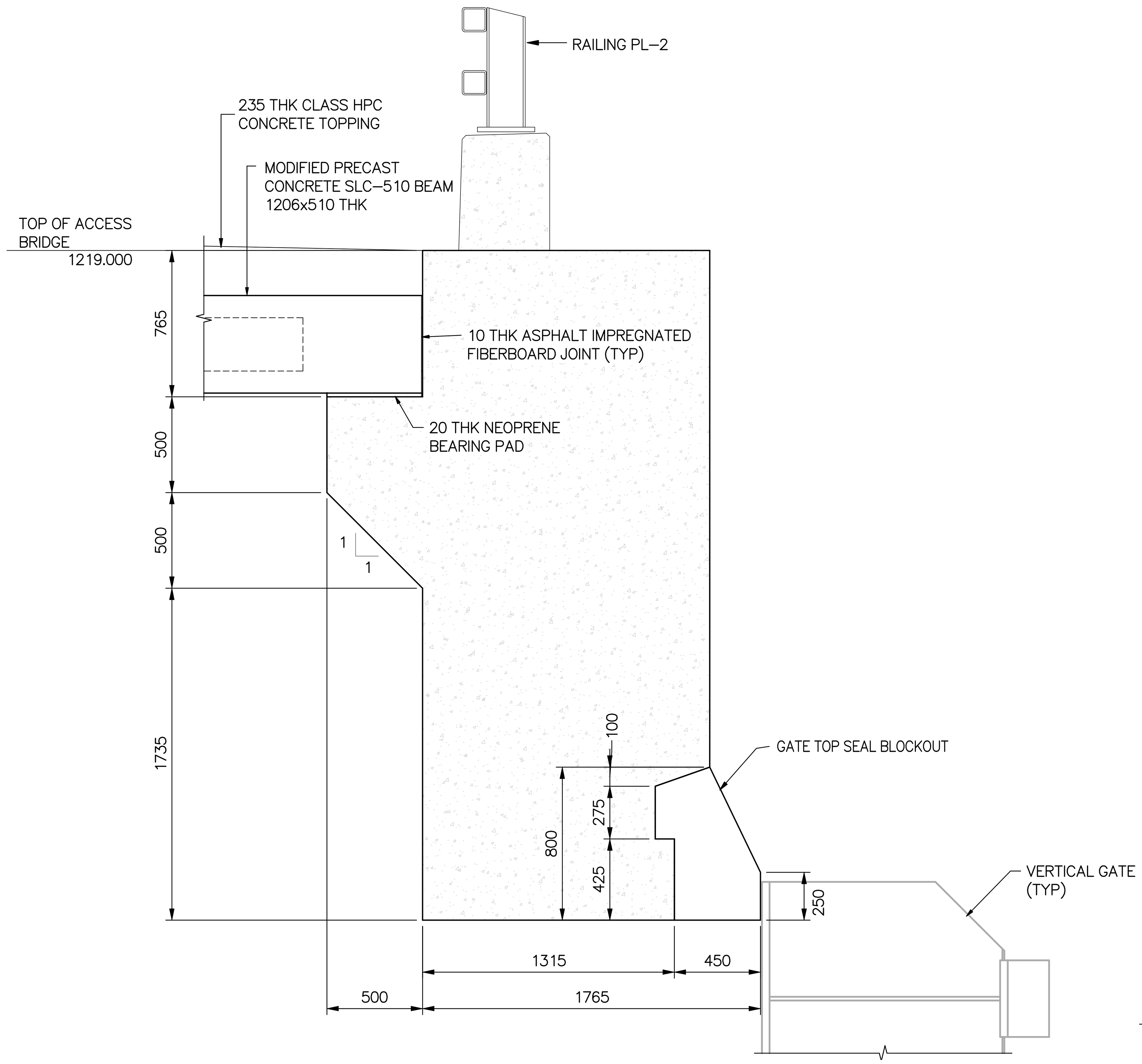
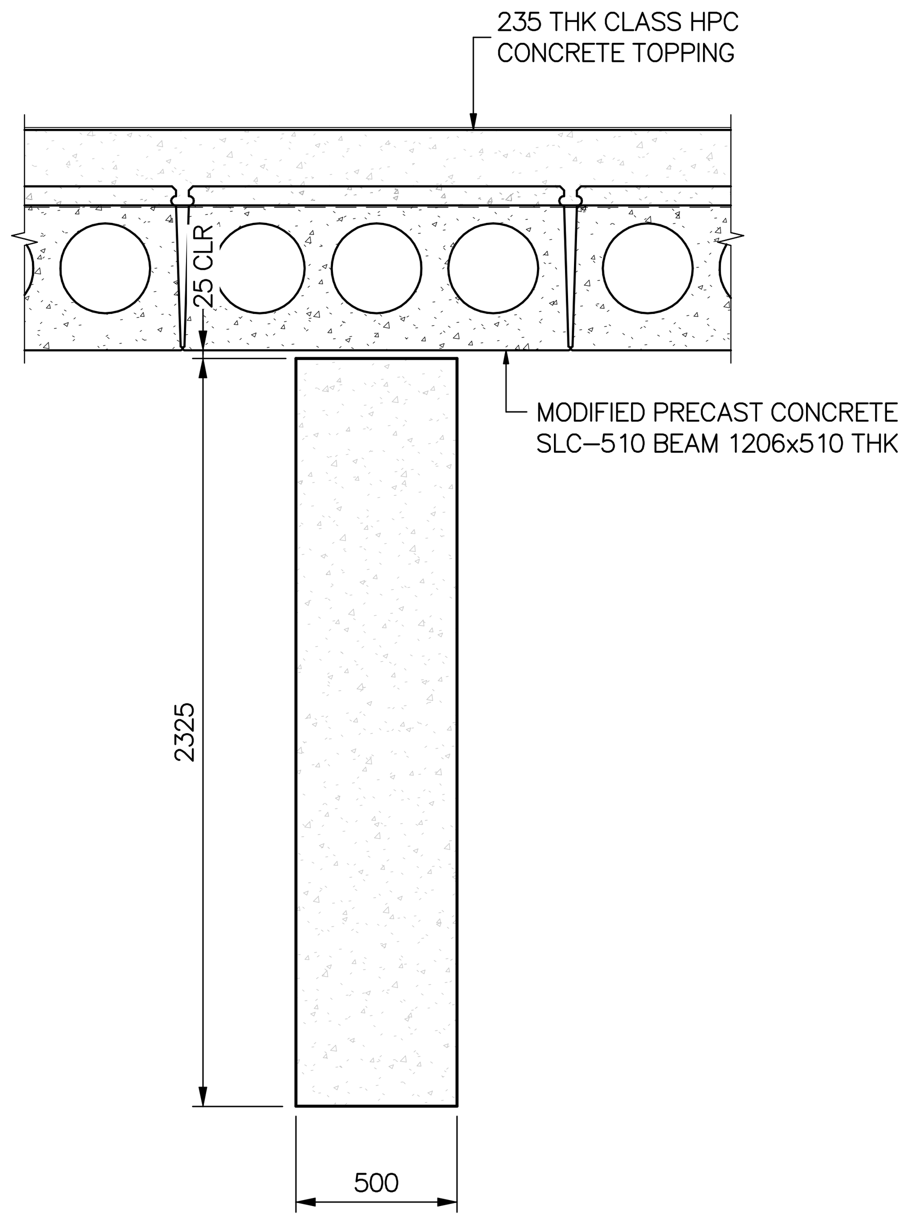


1 S–360 SECTION 1 : 20



2 S–360 SECTION – MID–SPAN DIAPHRAGM BEAM 1 : 20

NOTES:

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SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION INLET
GATE STRUCTURE - HEAD WALL
CONCRETE OUTLINE

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-363	

Plot Date: 6/25/2020 2:38:51 PM Login: Dosler
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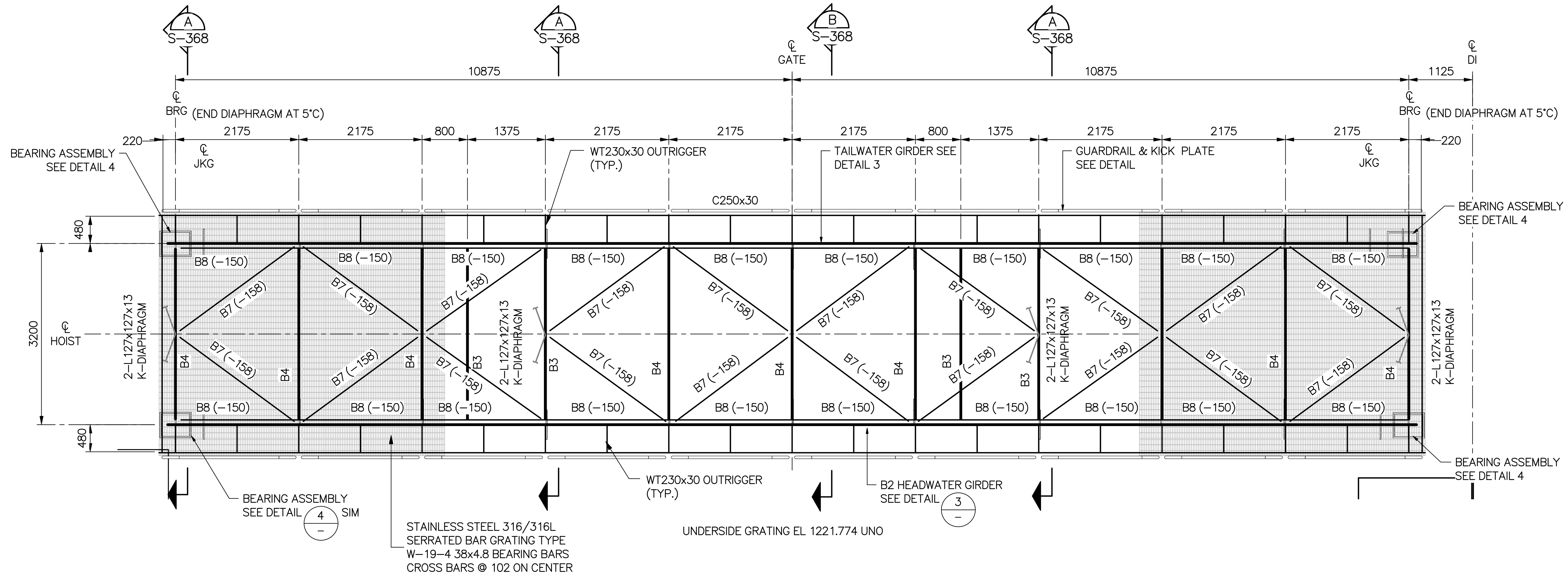
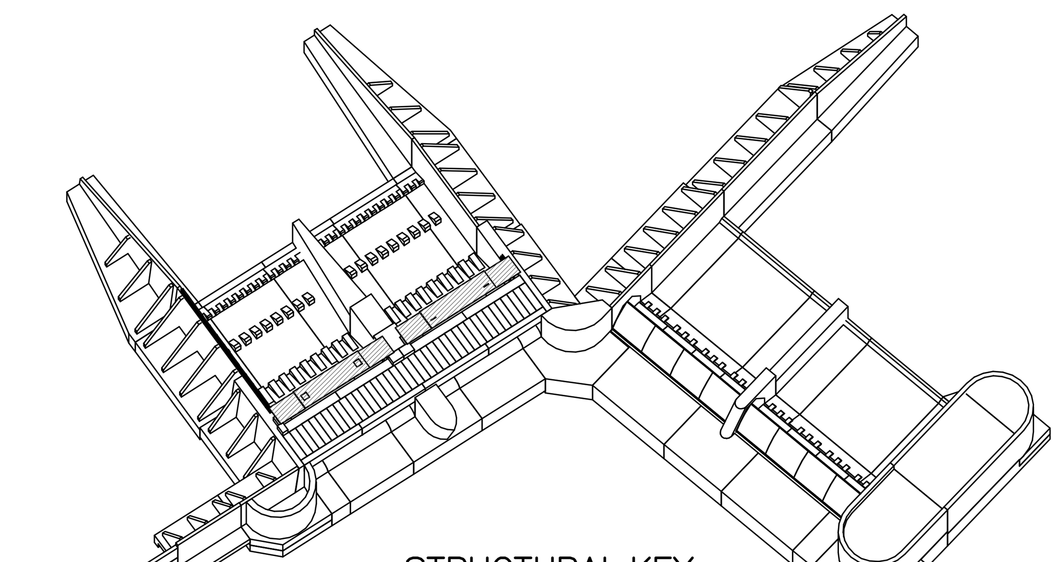
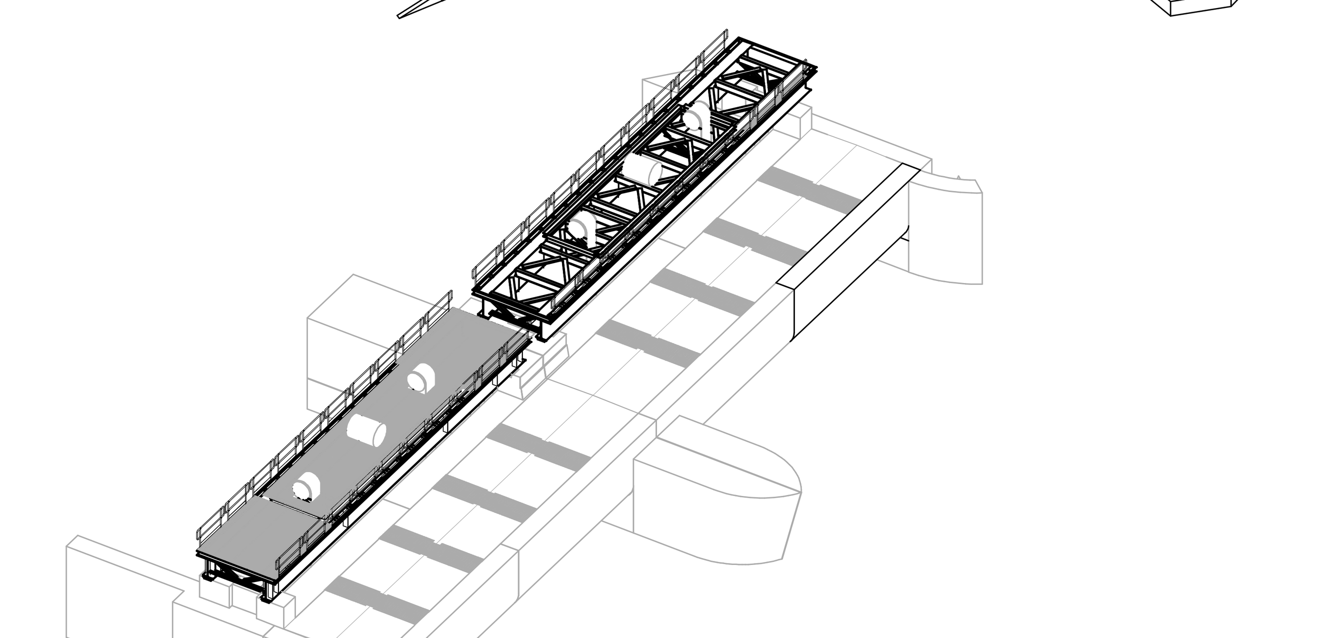


PLAN– GATE HOIST BRIDGE FRAMING

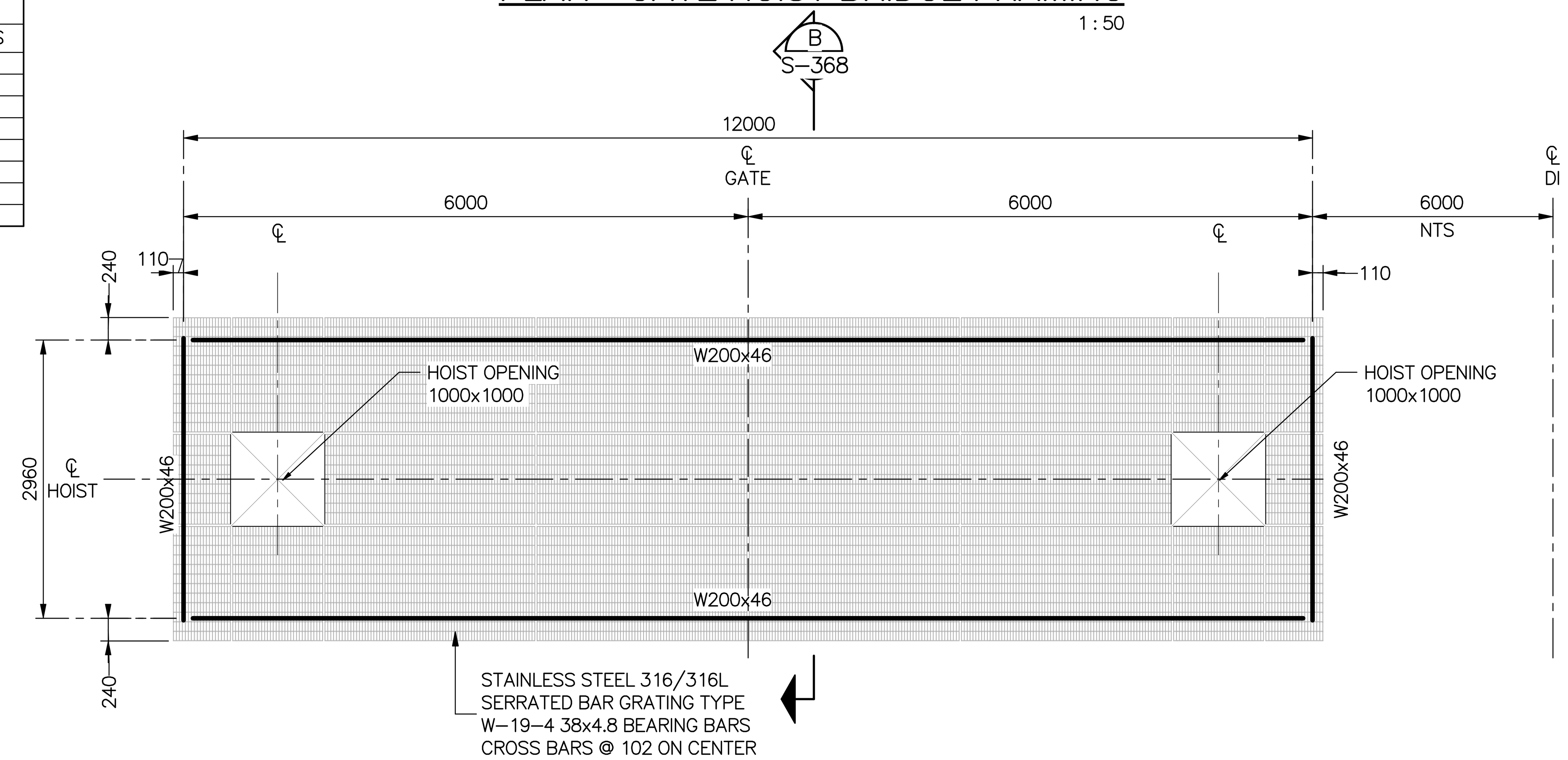
1 : 50





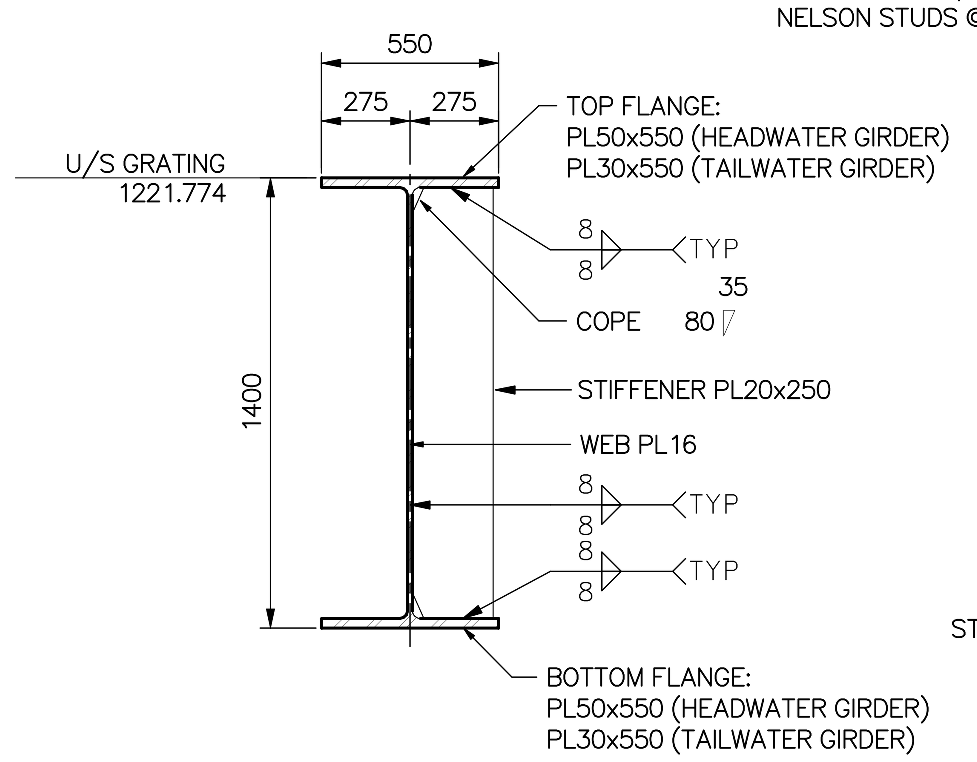
GATE HOIST BRIDGE 3D VIEW

STRUCTURAL FRAMING SCHEDULE		
MARK	DESCRIPTION	COMMENTS
B1	TAILWATER GIRDER	SEE DETAIL
B2	HEADWATER GIRDER	SEE DETAIL
B3	W460x106	
B4	W360x45	
B5	WT230x30	
B6	W200x46	
B7	WT155x33.5	
B8	WT155x33.5 FLAT	



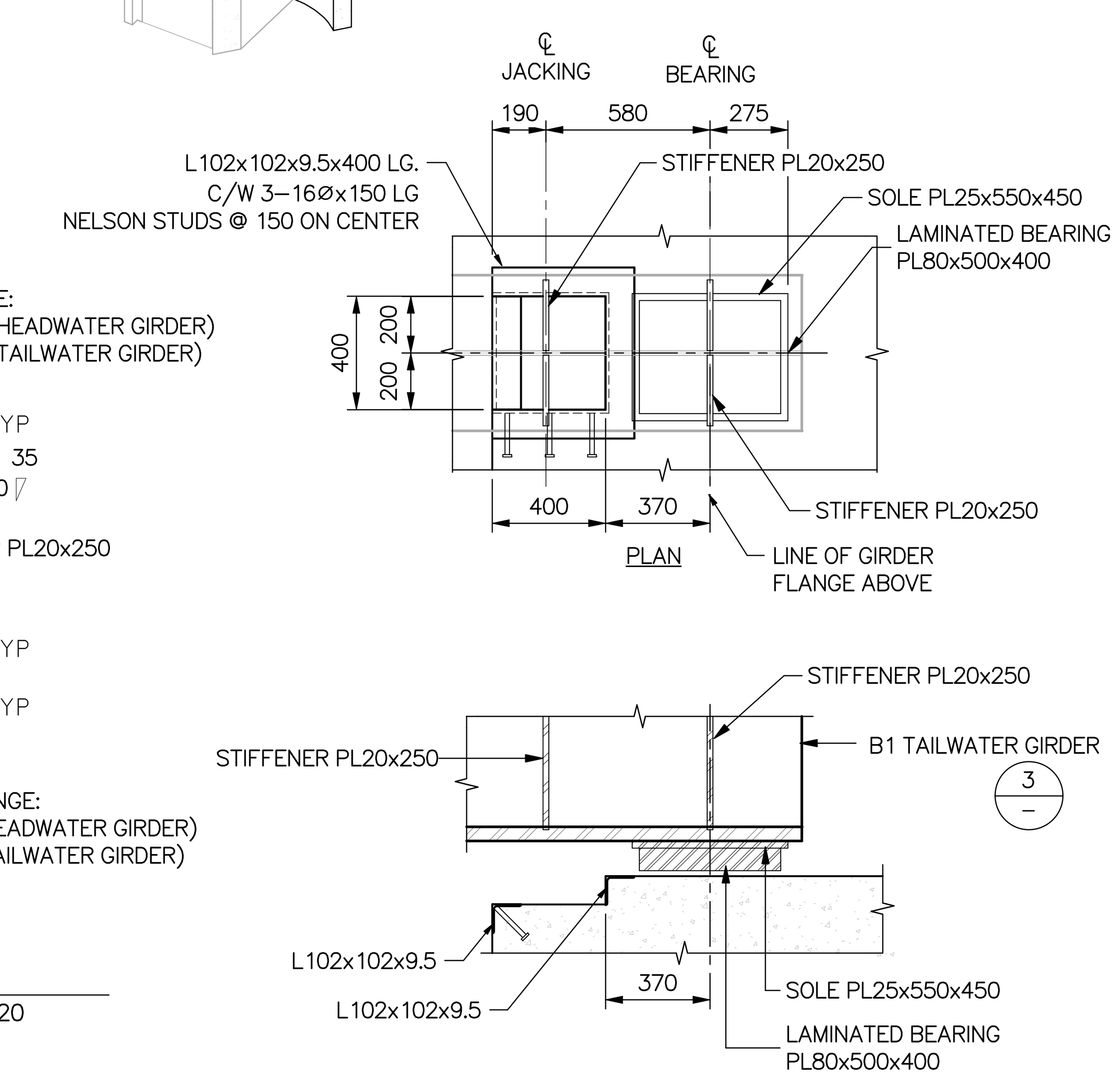
PLAN – GATE HOIST SKID FRAMING

1 : 50



3 PLATE GIRDER DETAIL

1 : 20



NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

4 BEARING ASSEMBLY & JACKING BLOCKOUT

1 : 20

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PRELIMINARY DESIGN

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT SR1

DIVERSION INLET
GATE STRUCTURE - GATE HOIST BRIDGE FRAMING
PLAN & SECTIONS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-367	

Plot Date: 6/25/2020 2:39:37 PM Login: Dosler
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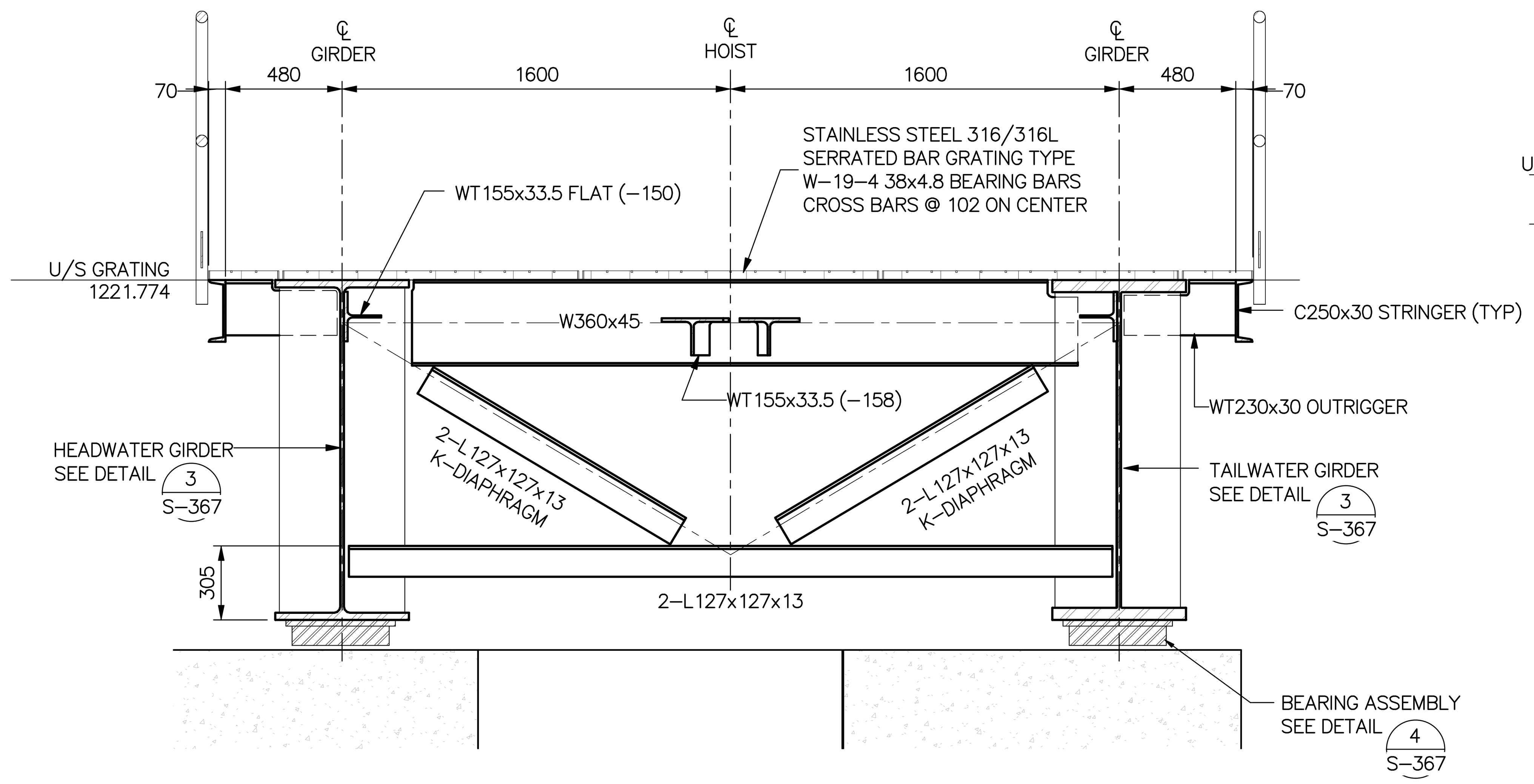
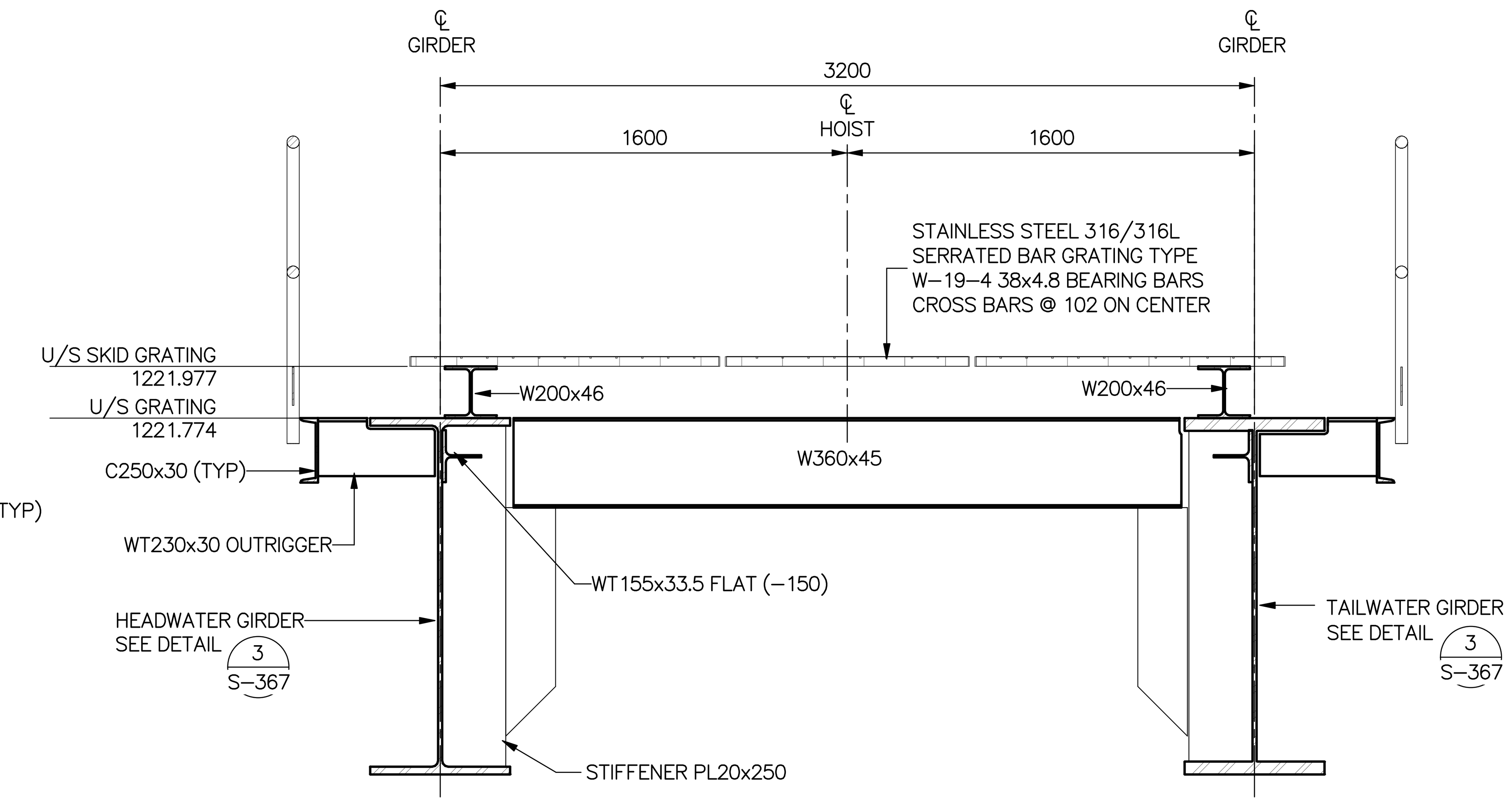


A S–367 SECTION – K–BRACING 1 : 20



B S–367 SECTION – TORSIONAL BRACING 1 : 20

NOTES:

1. STRUCTURE DIMENSIONS ARE IN MILLIMETRES UNLESS OTHERWISE NOTED.
2. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.

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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

DIVERSION INLET
GATE STRUCTURE - GATE HOIST SUPPORT
SECTIONS & DETAILS

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECT NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-368	

Plot Date: 6/25/2020 2:39:39 PM Login: Dosler
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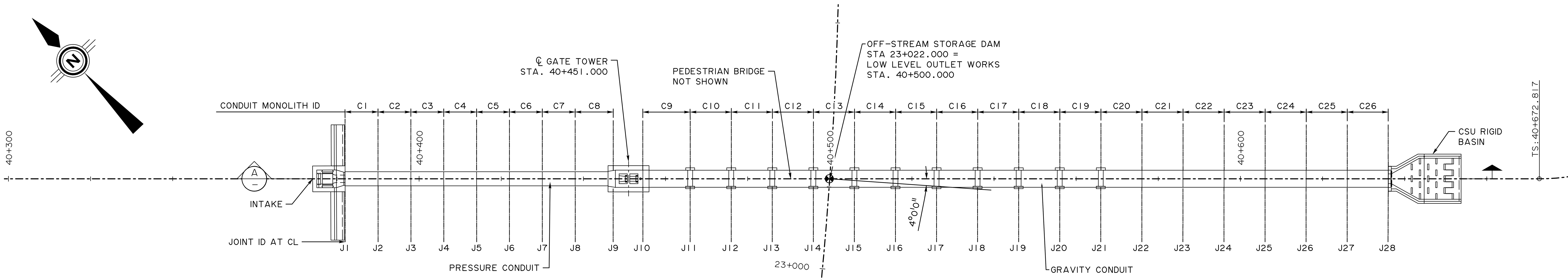

PLAN
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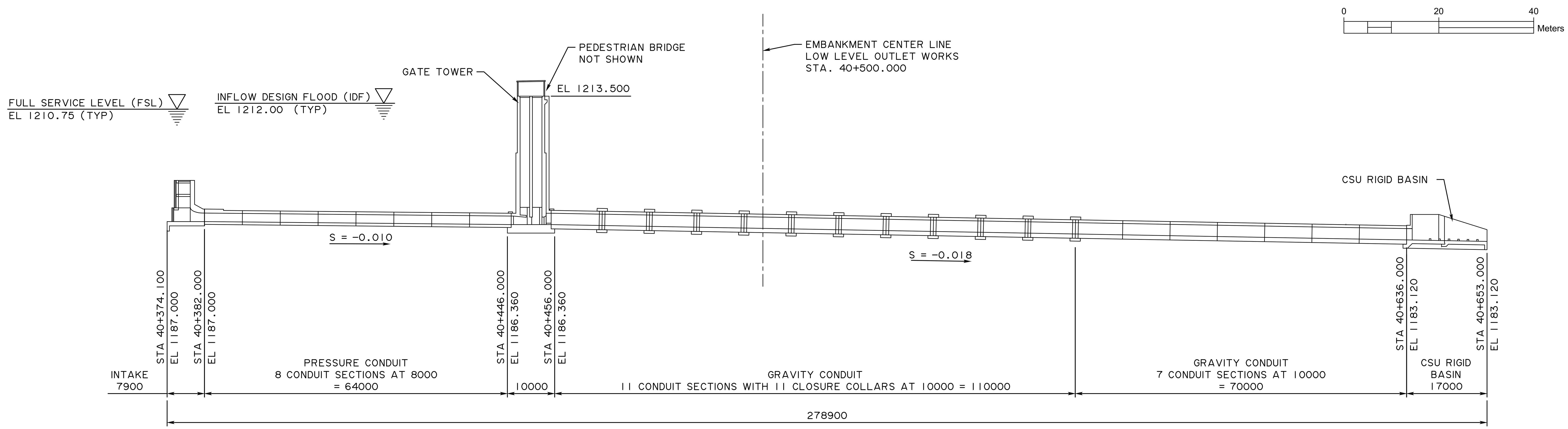

A SECTION ALONG CENTERLINE
1:500

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

PlotDate: 2019.12.03 2:56 PM Login: Shelton, Ben
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\400_ossd\73396S-400.dwg TENDER No. ?

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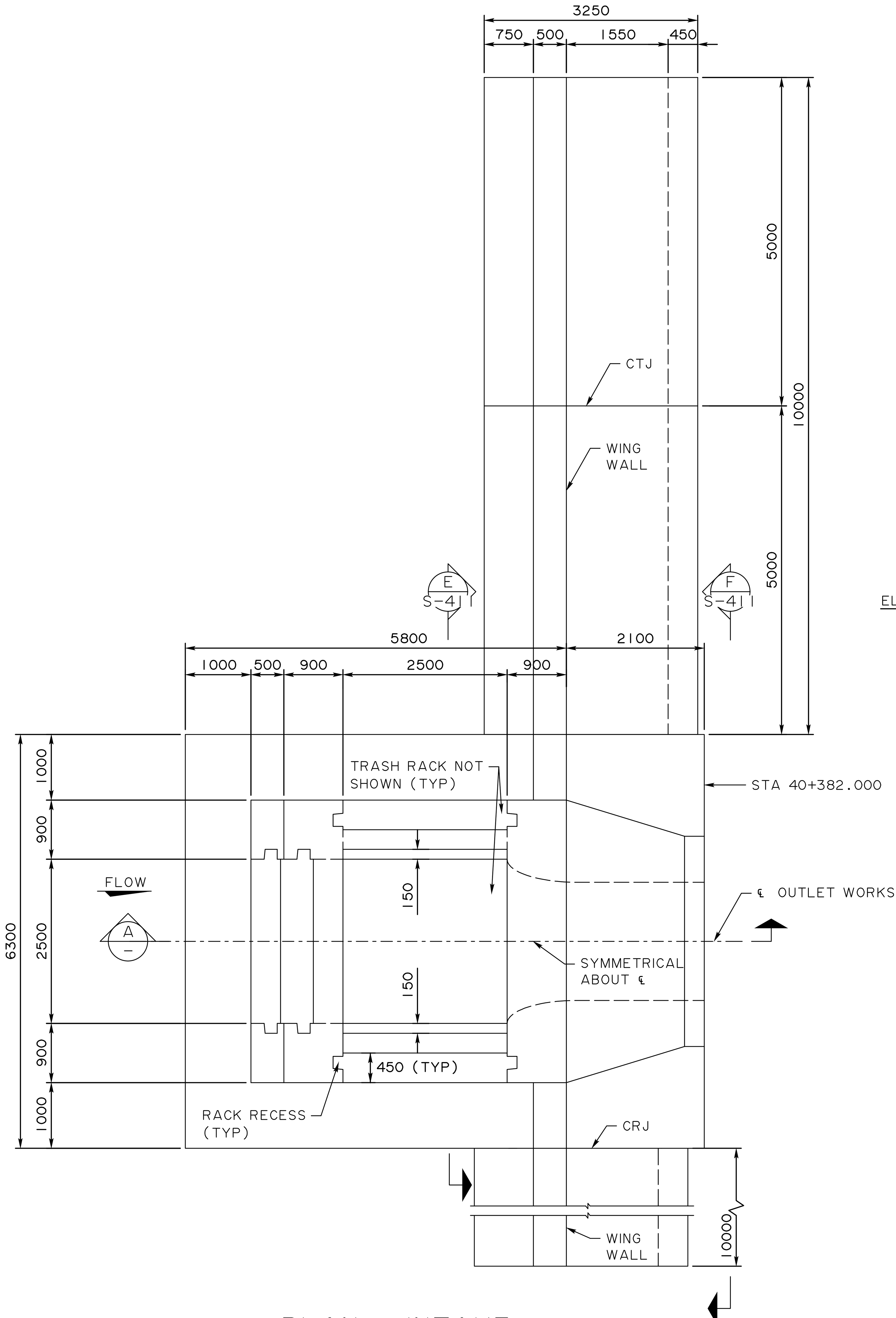
Alberta Transportation

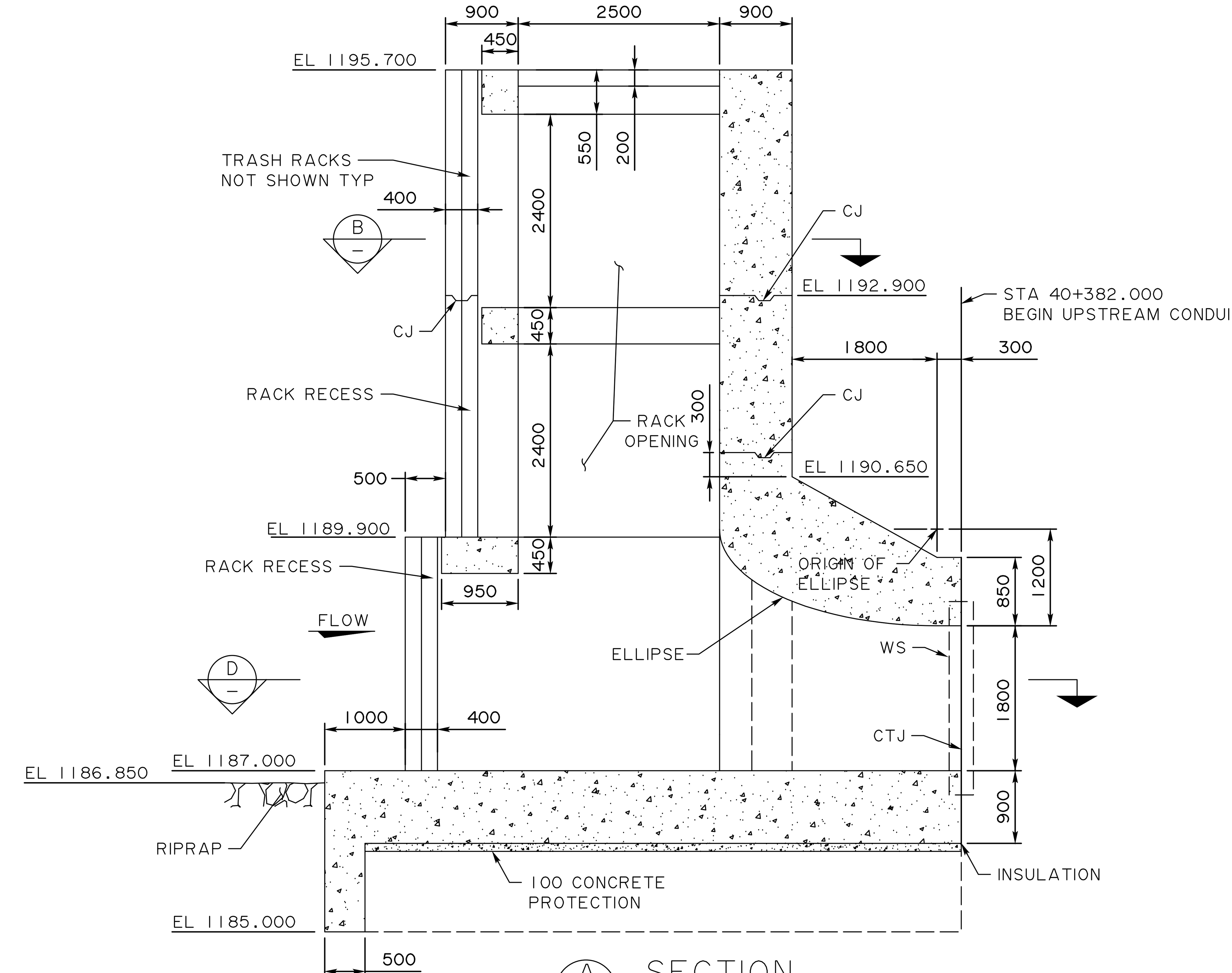
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

OFF STREAM STORAGE DAM
LOW LEVEL OUTLET WORKS
STRUCTURAL ARRANGEMENT

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-400	



PLAN - INTAKE
1:50



SECTION A
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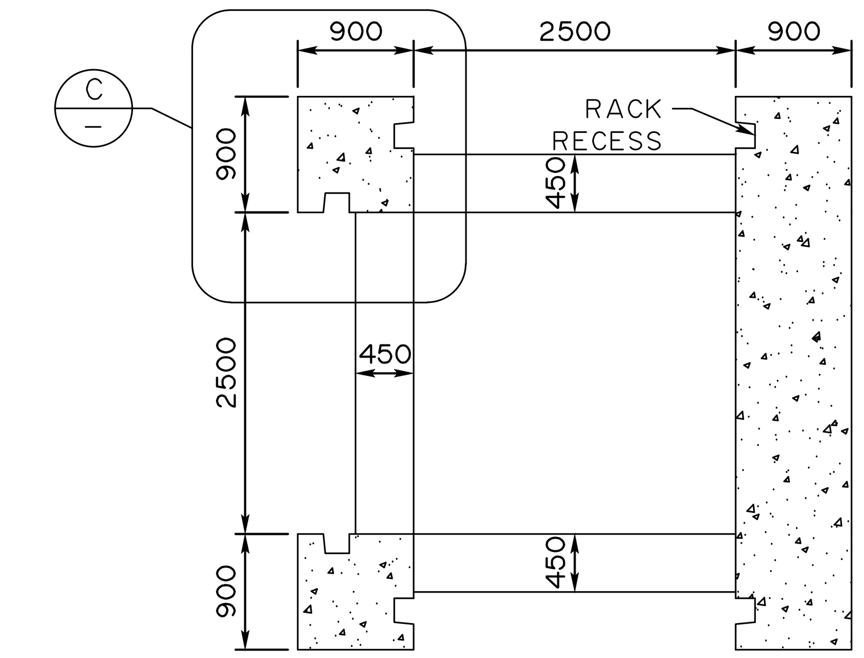
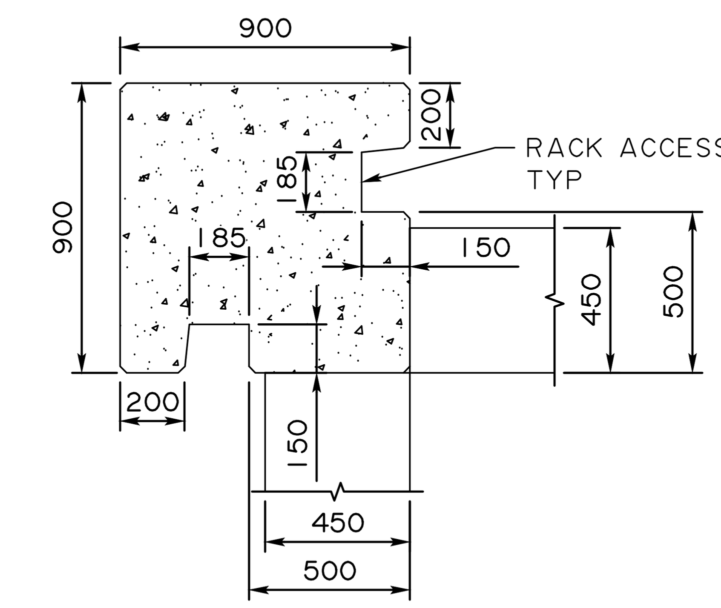
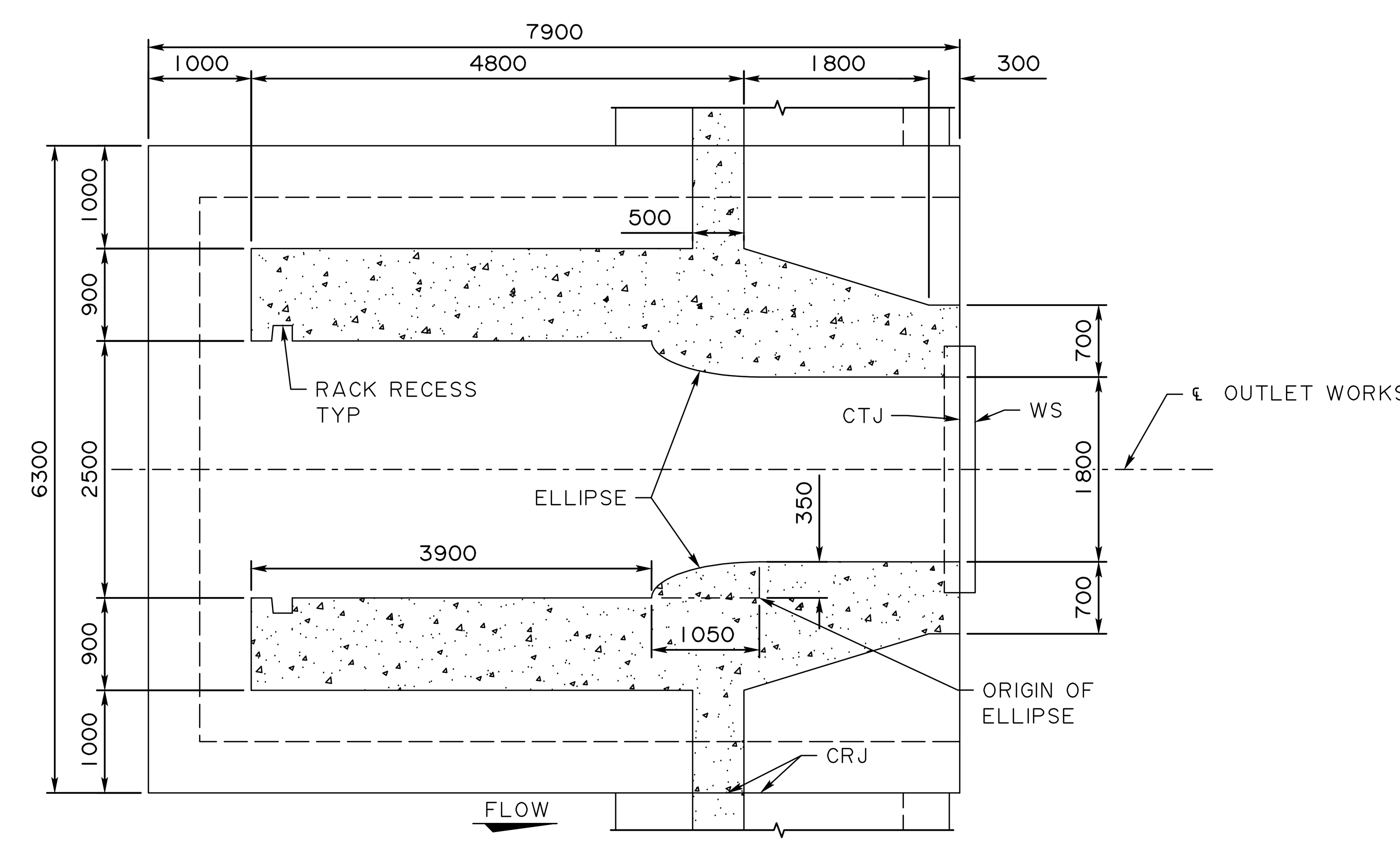


SECTION B
1:50



DETAIL C
1:20



SECTION D
1:50

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

TENDER No. ?

PlotDate: 2019.12.03 3:07 PM Login: Shelton, Ben
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\400_ossd\73396S-410_intake.dwg

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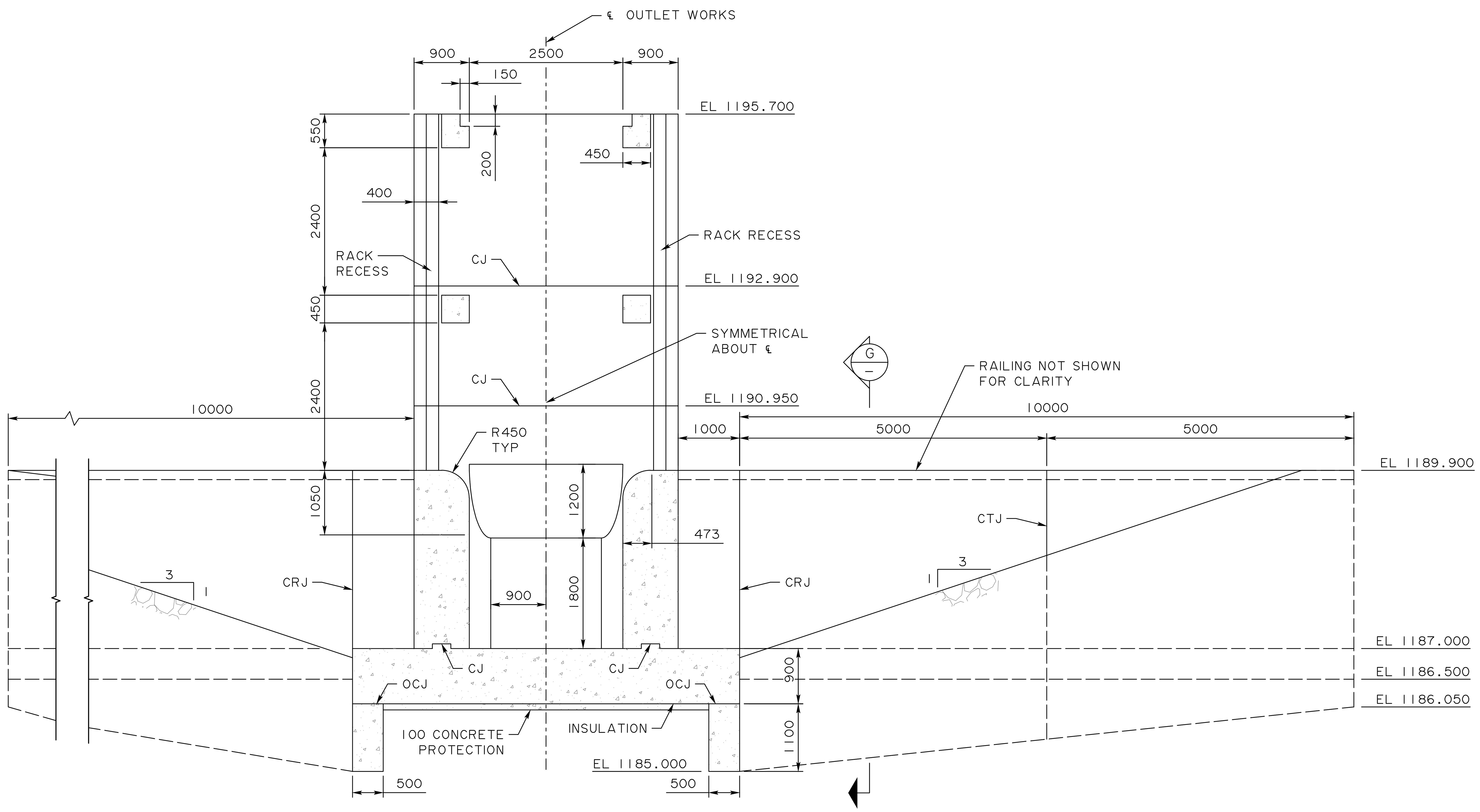
Alberta Transportation

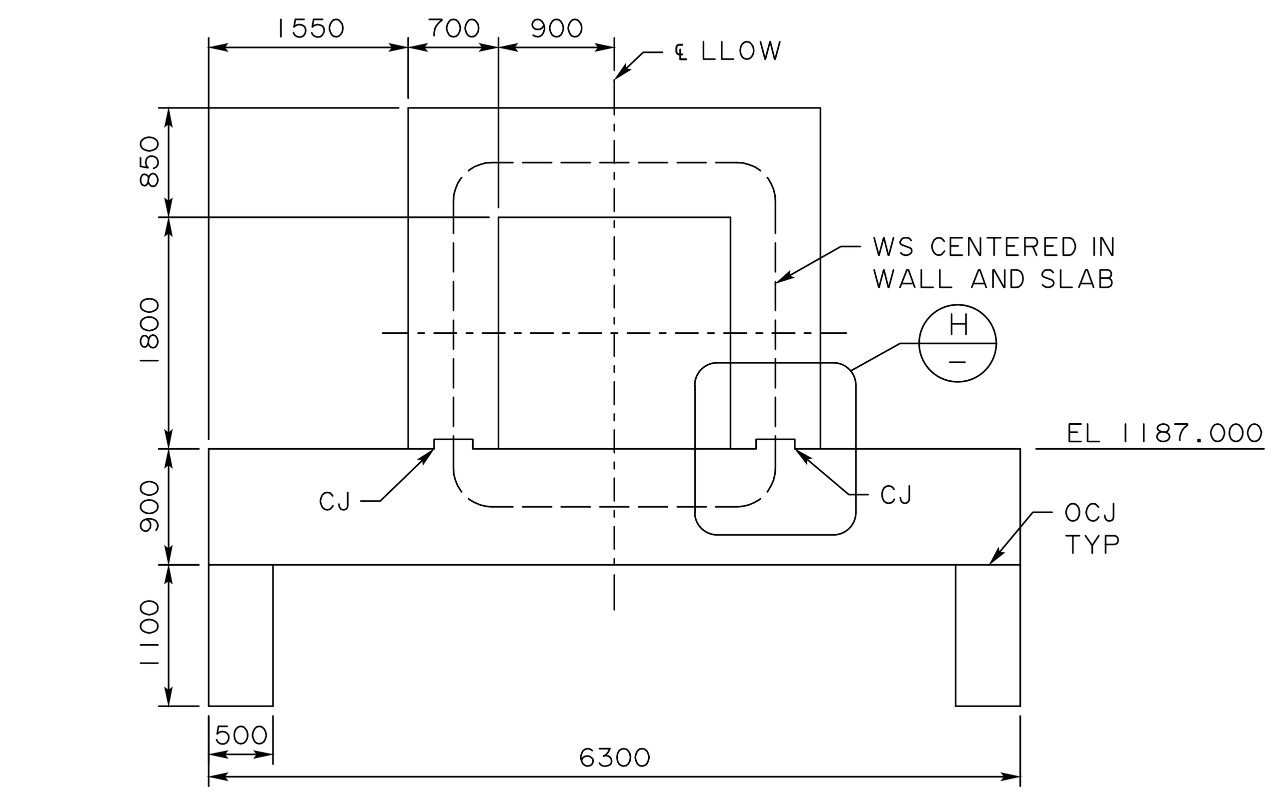
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
INTAKE STRUCTURE
CONCRETE OUTLINE - SHEET 1 OF 2

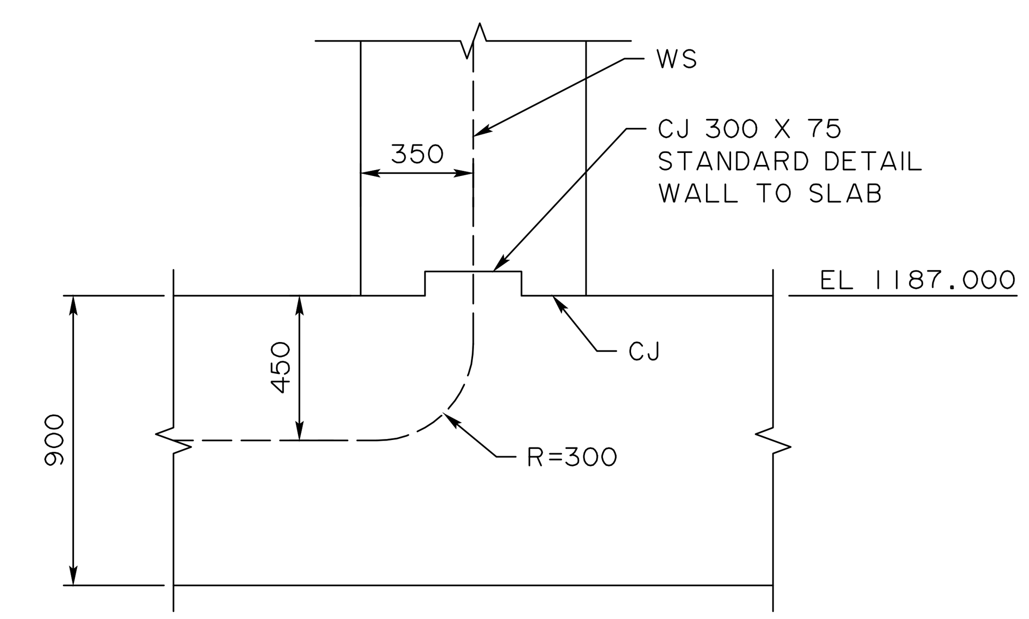
DATE: 2020-09-25
ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER: -
SITE: -
SHEET
CONSULTANT DRAWING: 73396S-410
TRANSPORTATION DRAWING



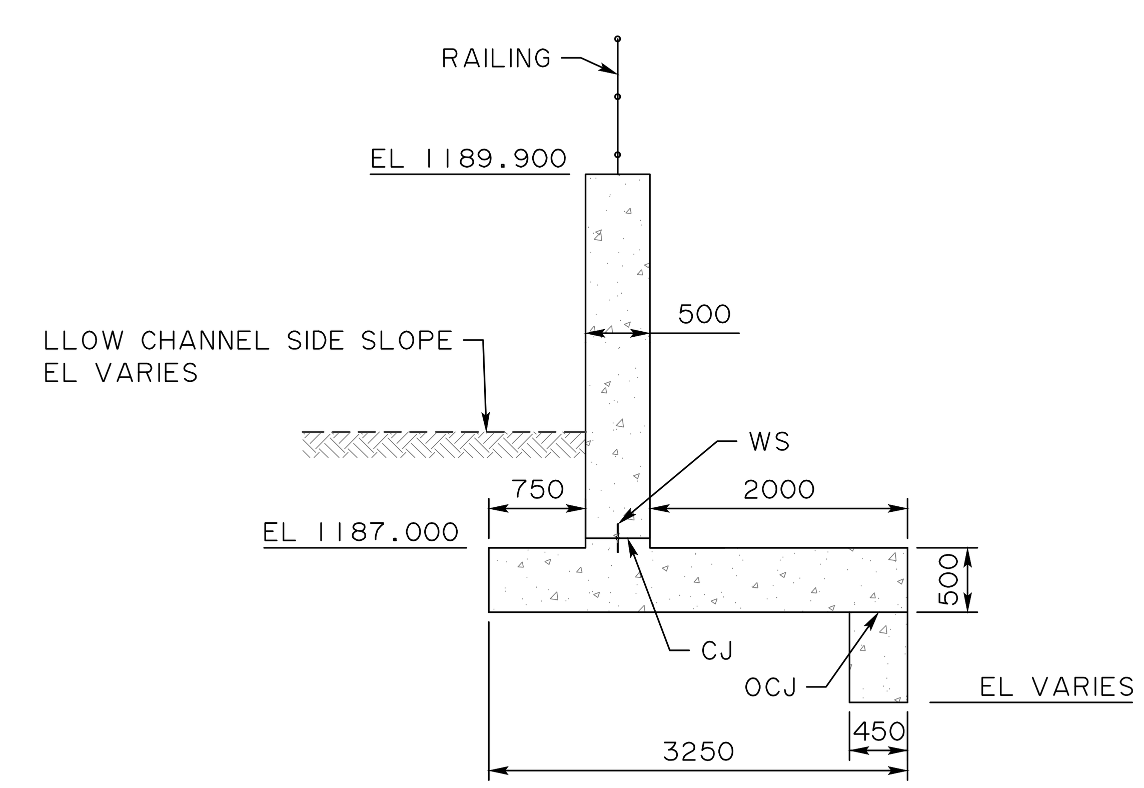
E S-410 SECTION 1:50



F S-410 SECTION - AT CONTROL JOINT 1:50



H DETAIL 1:20



G SECTION - WING WALL 1:50

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

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Alberta Transportation

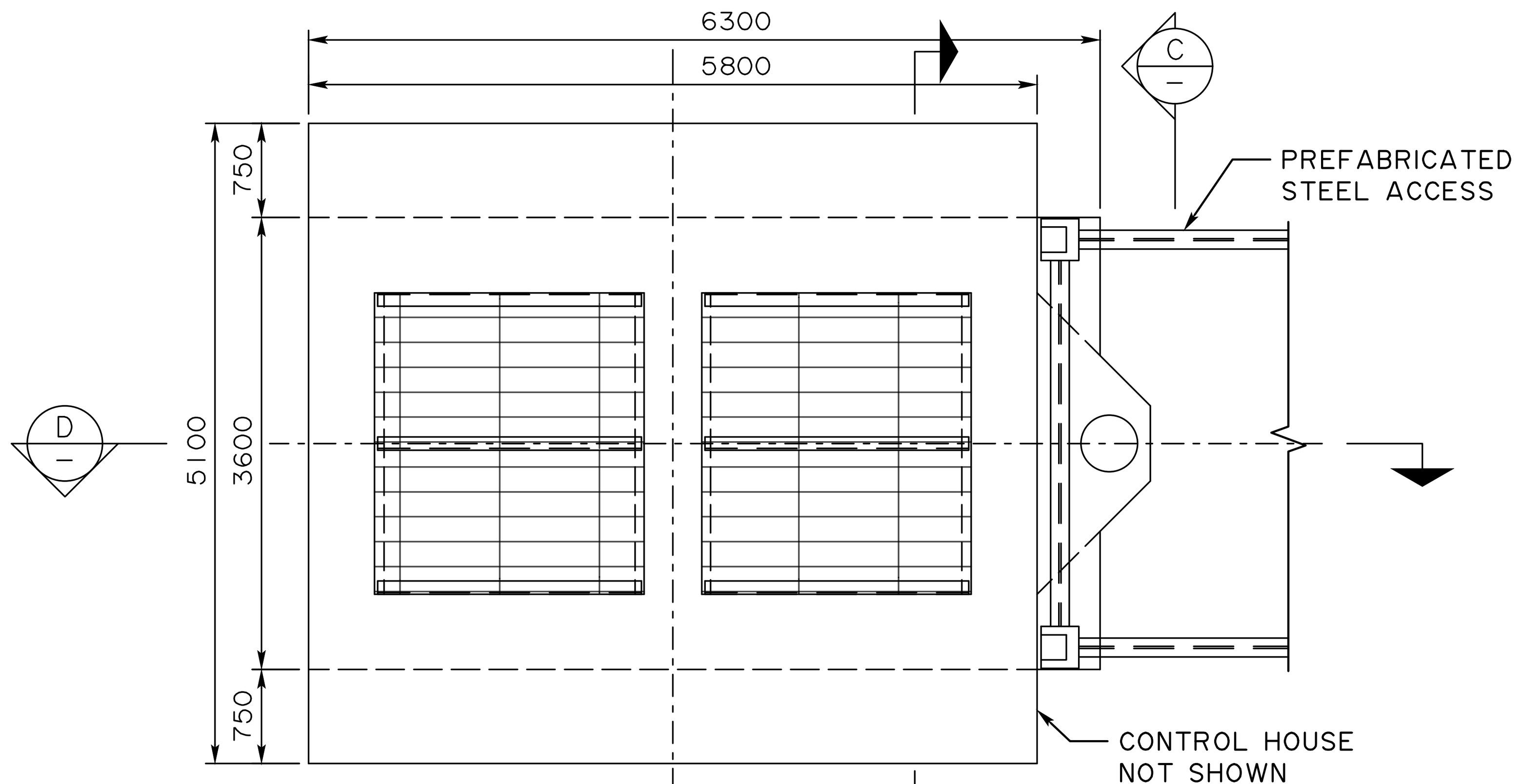
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
INTAKE STRUCTURE
CONCRETE OUTLINE - SHEET 2 OF 2

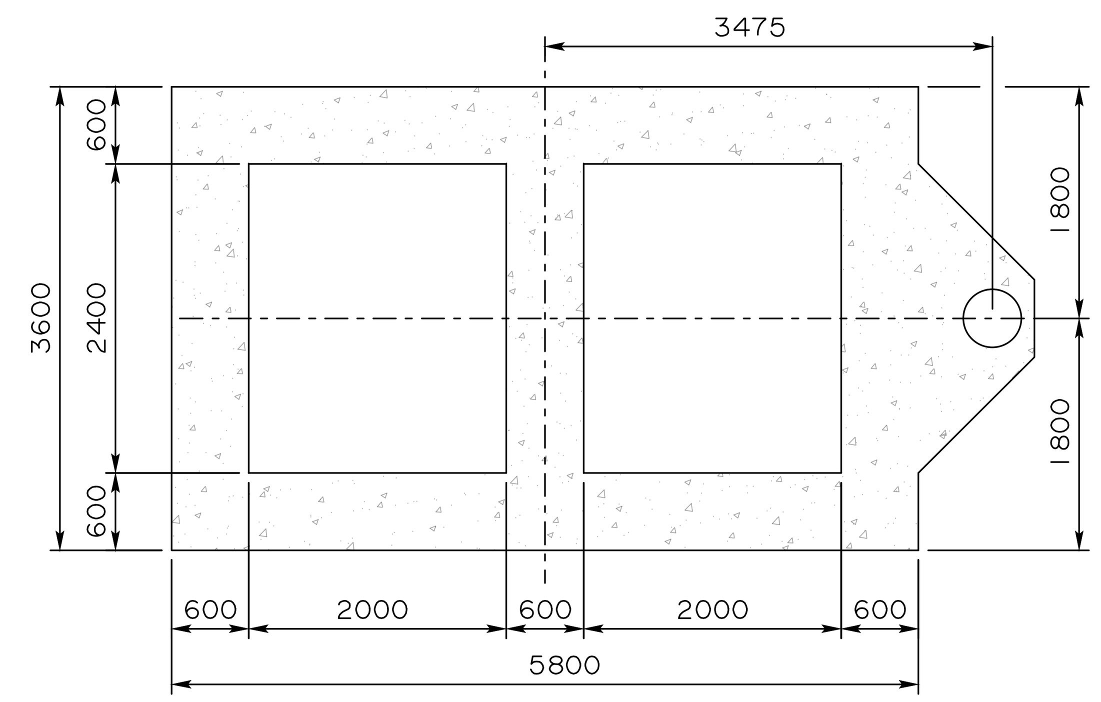
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-411	

NOTE:
1. ACCESS LADDERS AND LANDINGS NOT SHOWN.

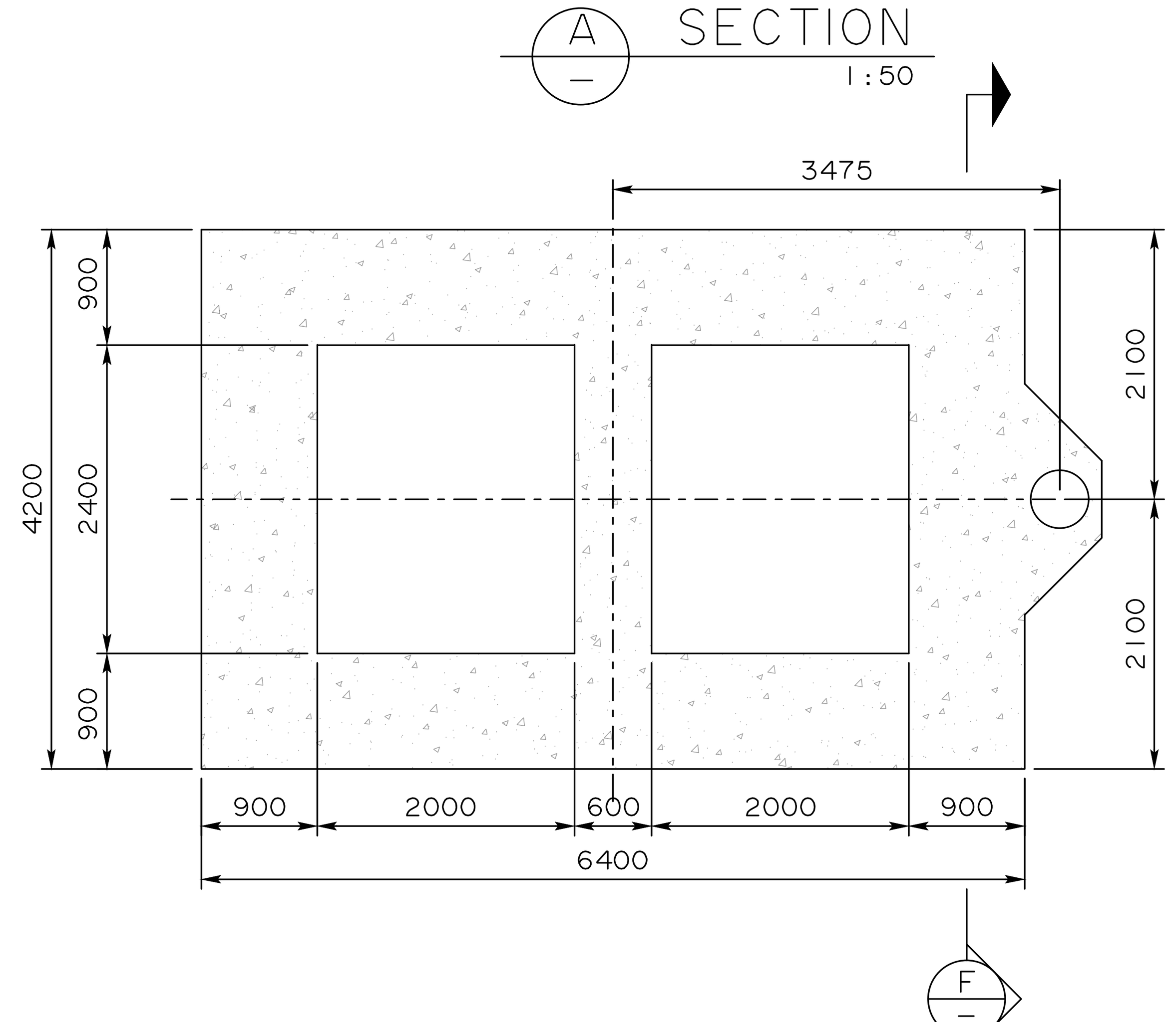
NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.



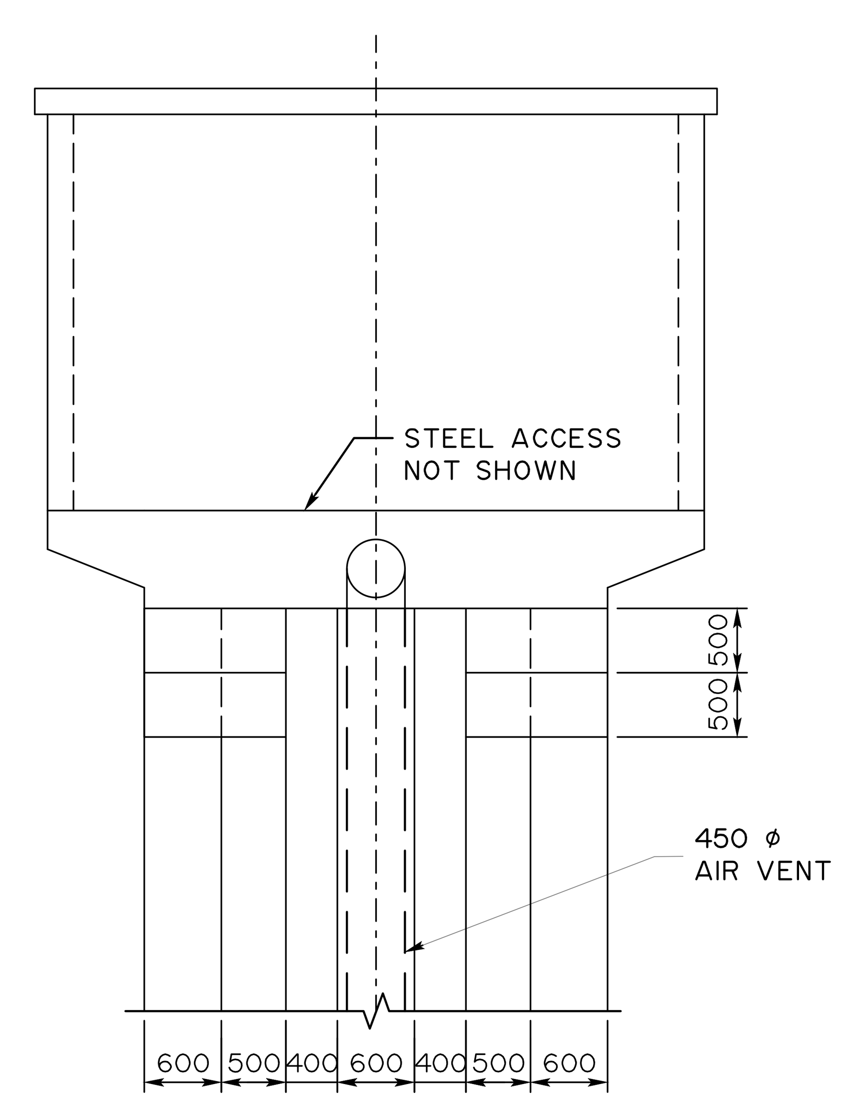
PLAN
AT EL 1213.500 1:50



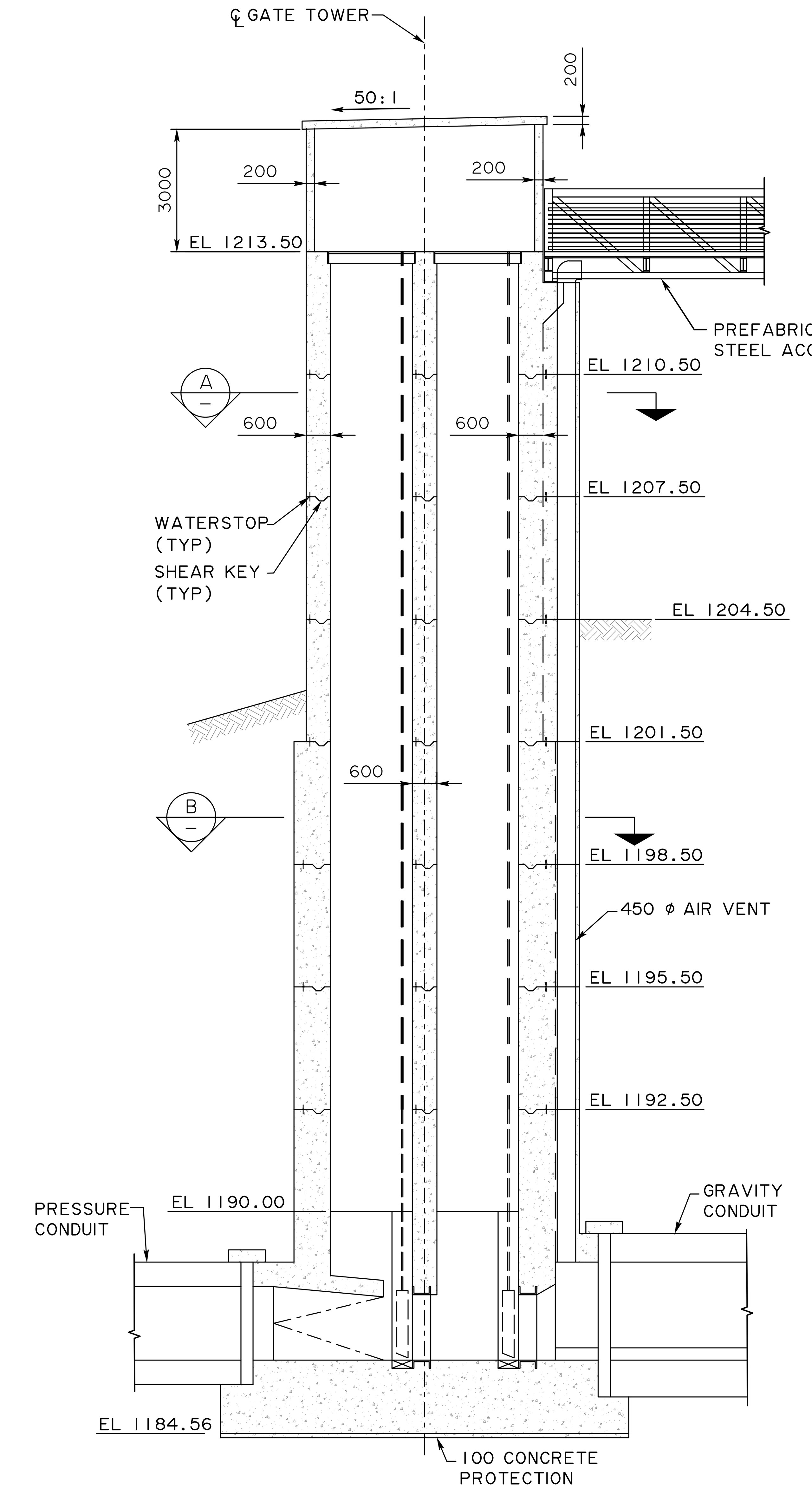
A SECTION
1:50



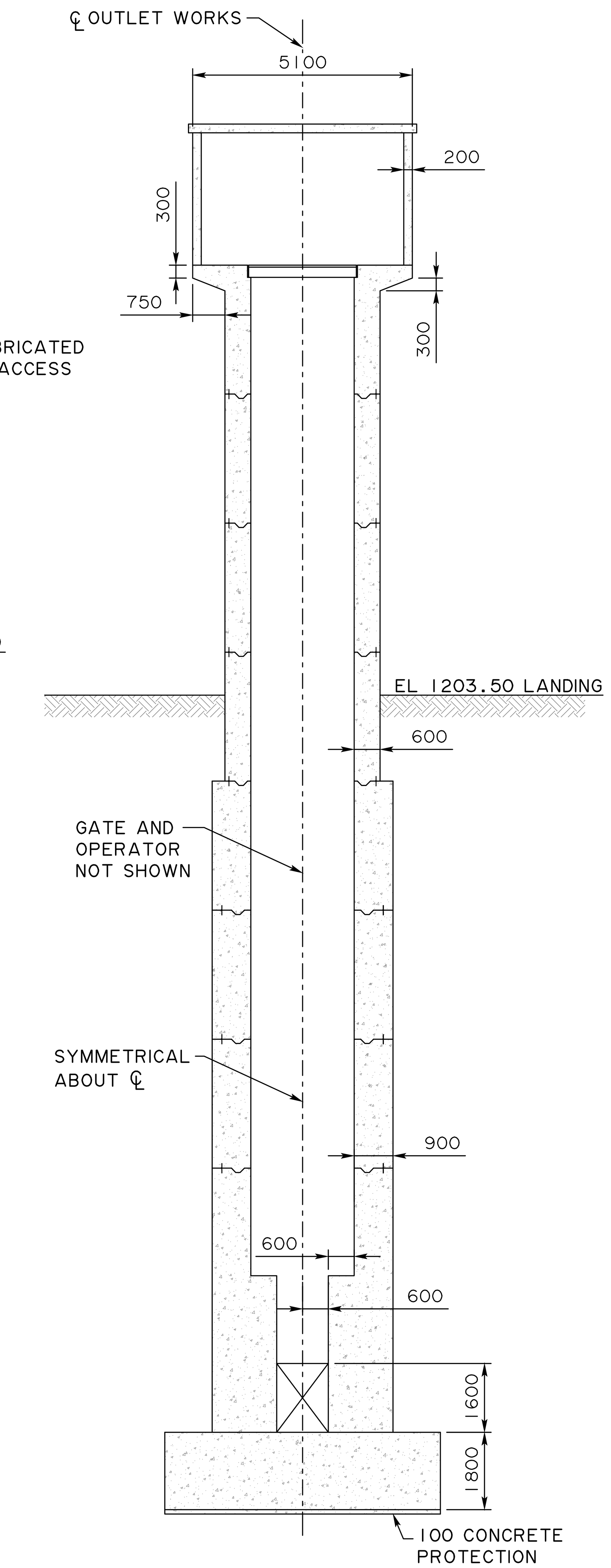
B SECTION
1:50



C SECTION
1:50



D SECTION
1:50



E SECTION
1:50

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PlotDate: 2019.12.03 3:12 PM Login: Shelton, Ben
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\400_ossd\73396S-430_gate_shaft_house.dwg TENDER No. ?

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Alberta Transportation

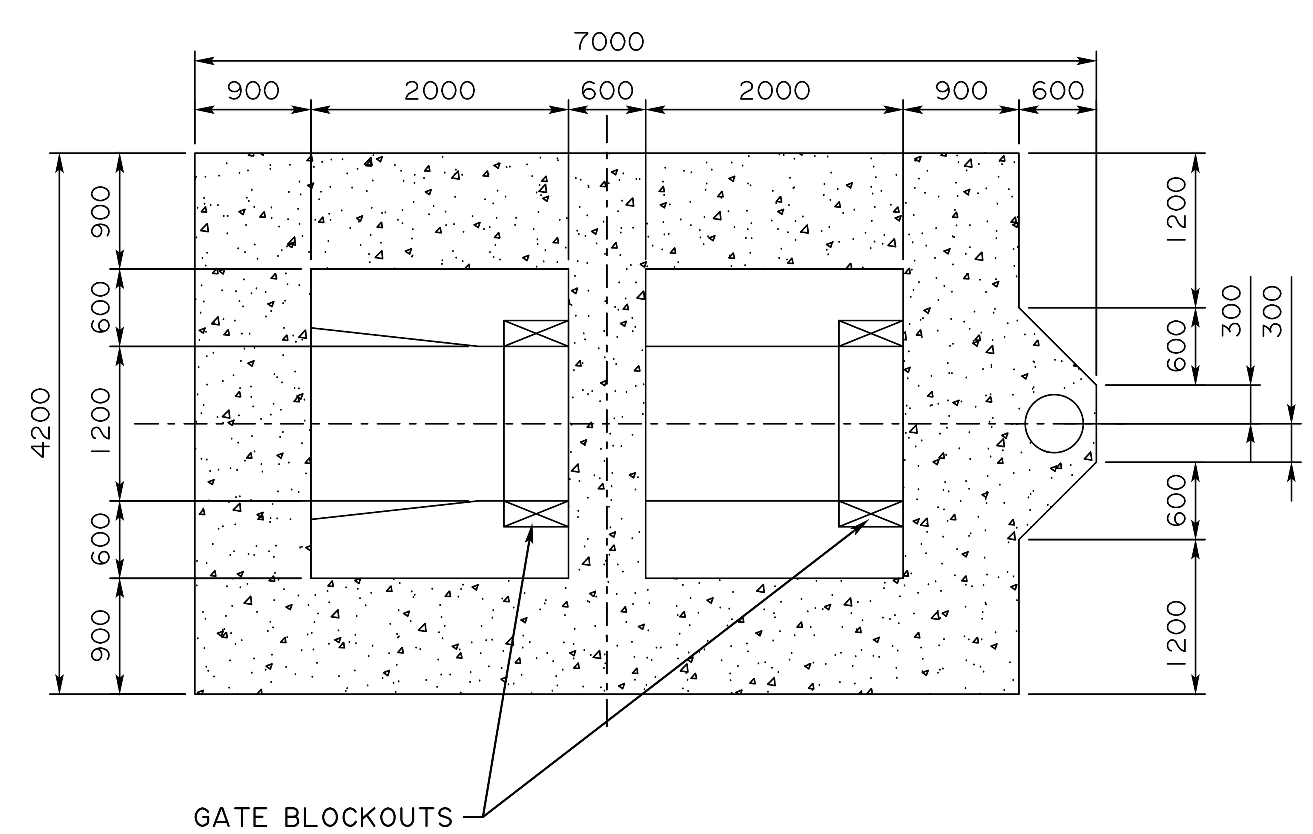
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
GATE TOWER
CONCRETE OUTLINE

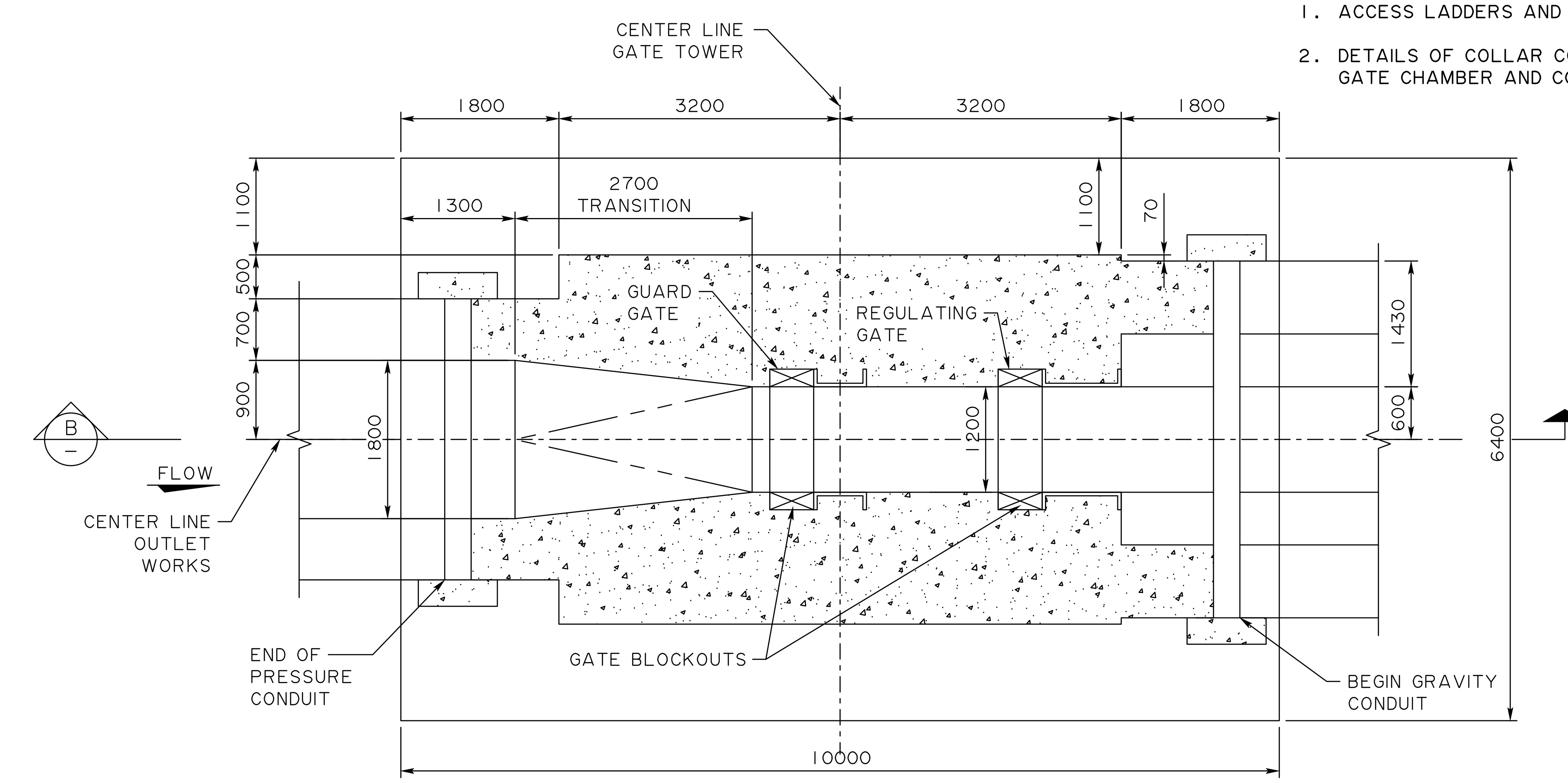
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-430	

PlotDate: 2019.12.03 3:13 PM Login: Shelton, Ben TENDER No. ?
AUTOCAD FILE No. U:\110773396\CAD\water\s_structural\sheet_files\400_ossd\73396S-431_gate_chamber.dwg

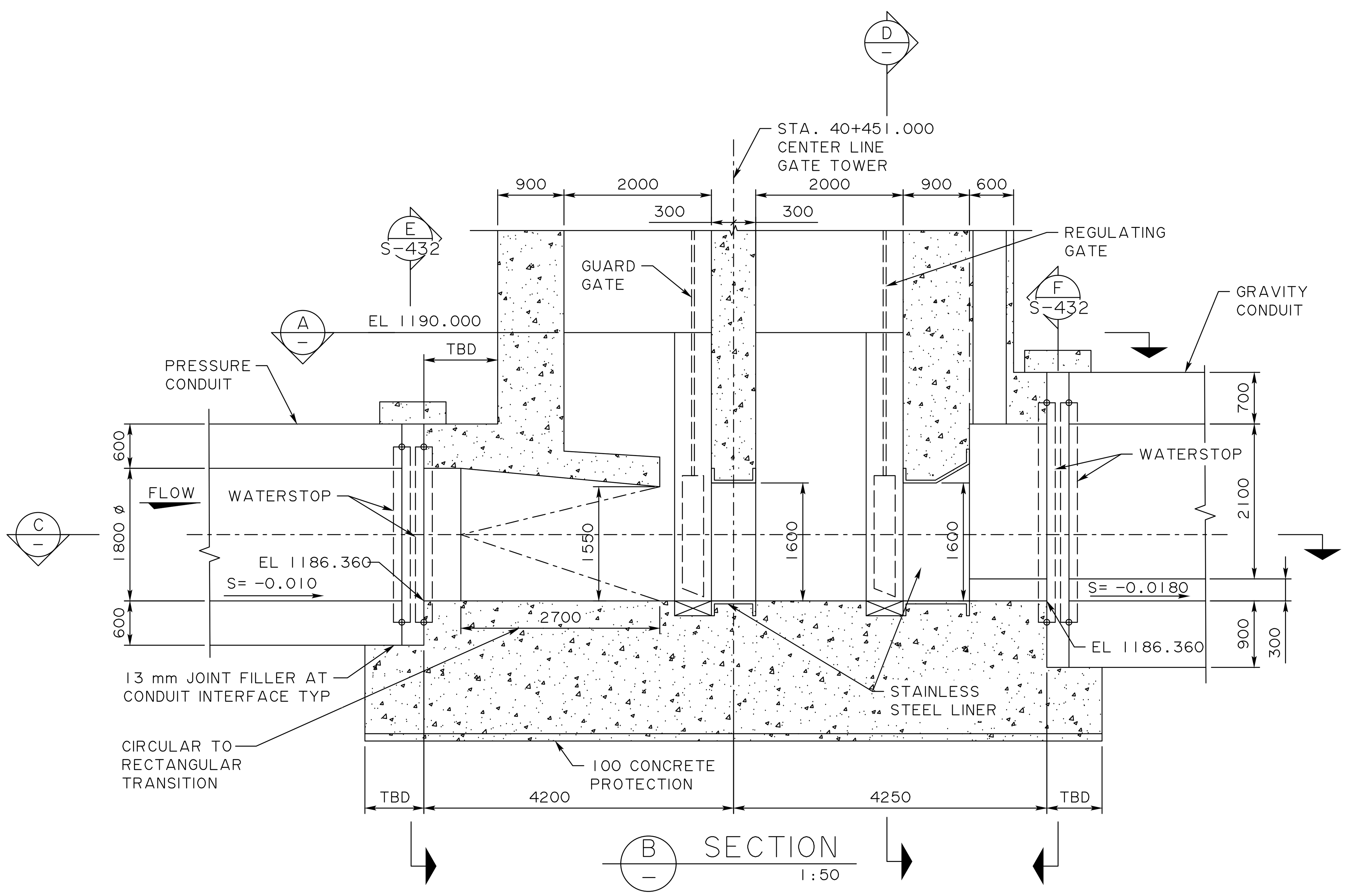
NOTES:
1. ACCESS LADDERS AND LANDINGS NOT SHOWN.
2. DETAILS OF COLLAR CONNECTIONS BETWEEN GATE CHAMBER AND CONDUITS TBD.



A SECTIONAL PLAN
AT EL 1190.000 1:50



C SECTION
1:50



B SECTION
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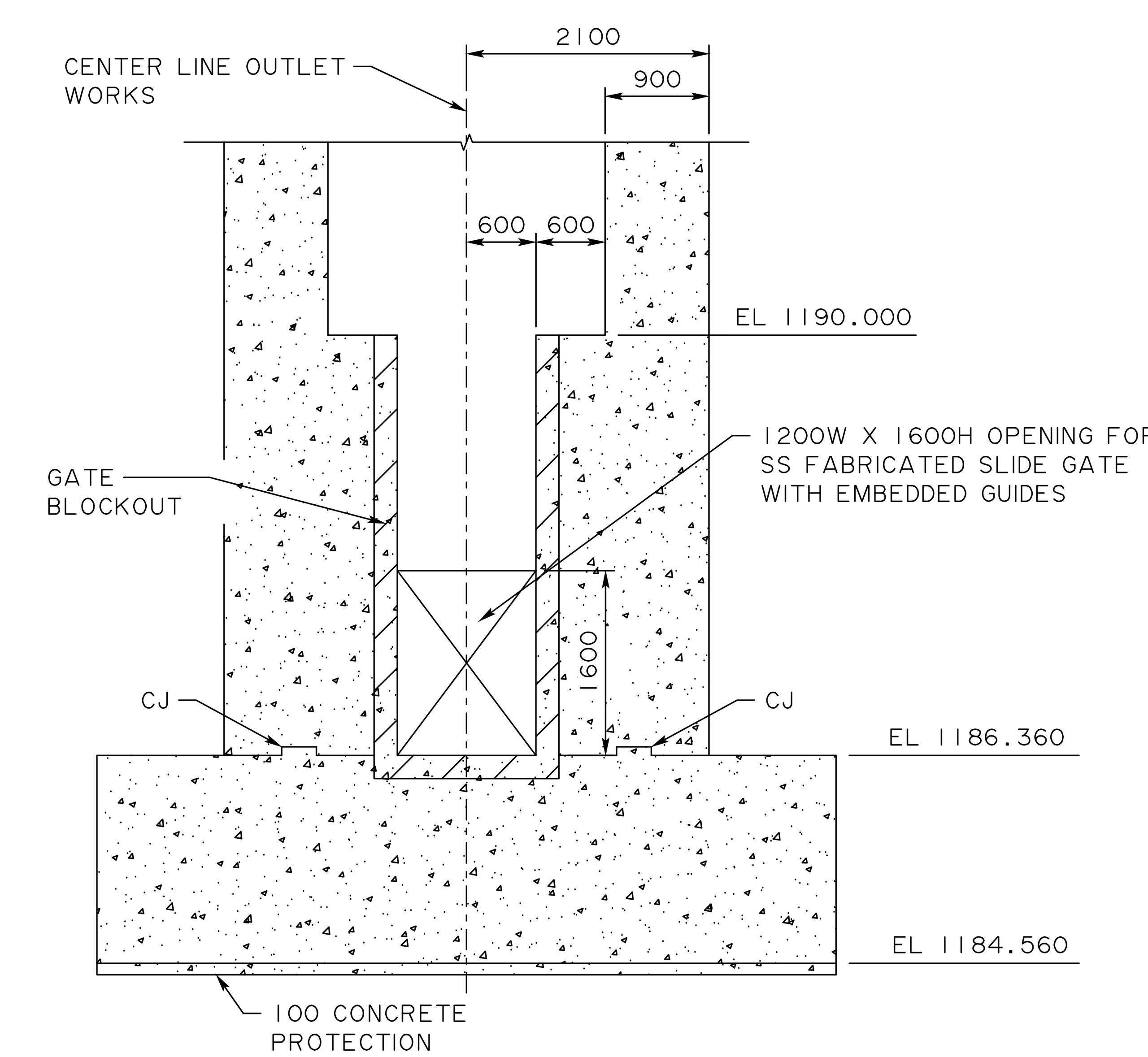

D SECTION
1:50

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
2. STRUCTURE DIMENSIONS IN MILLIMETRES UNLESS NOTED OTHERWISE.

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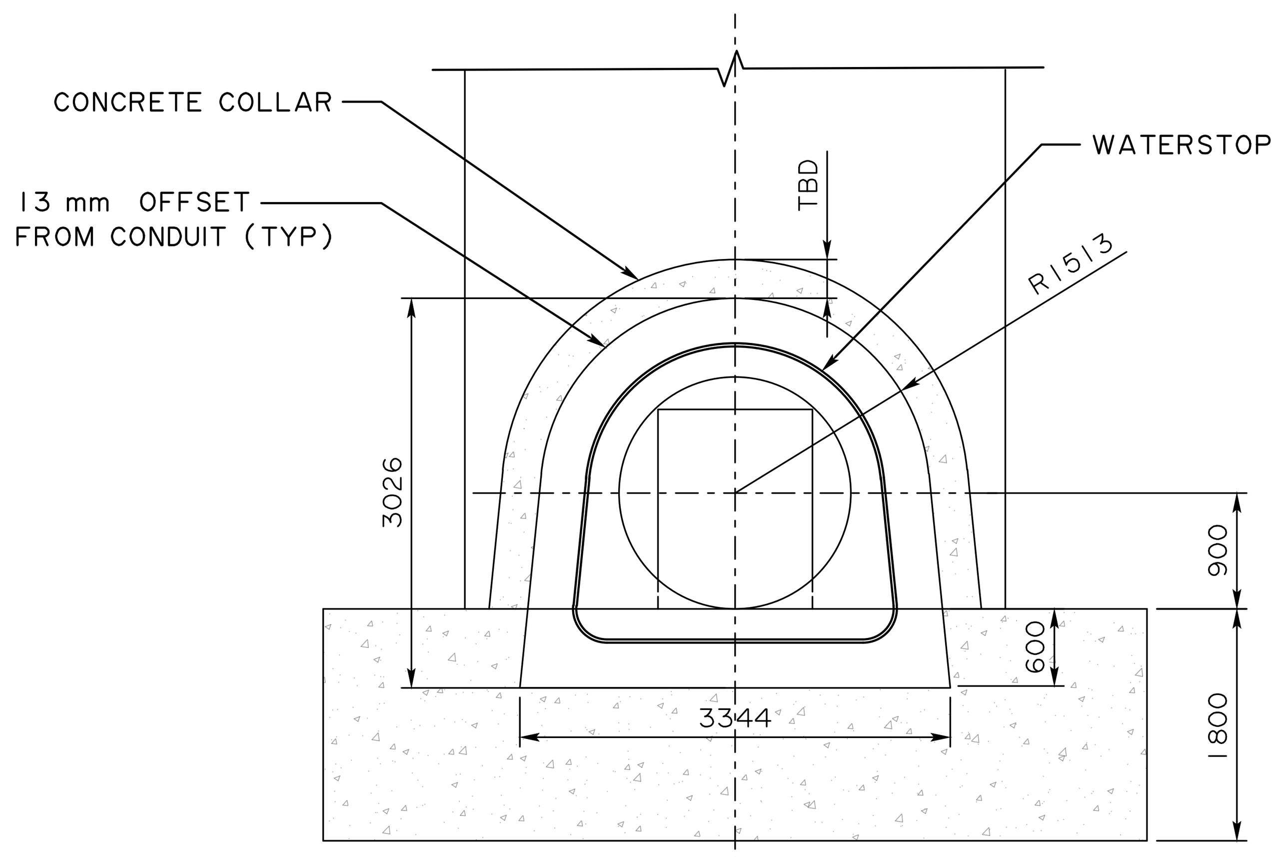
Alberta Transportation

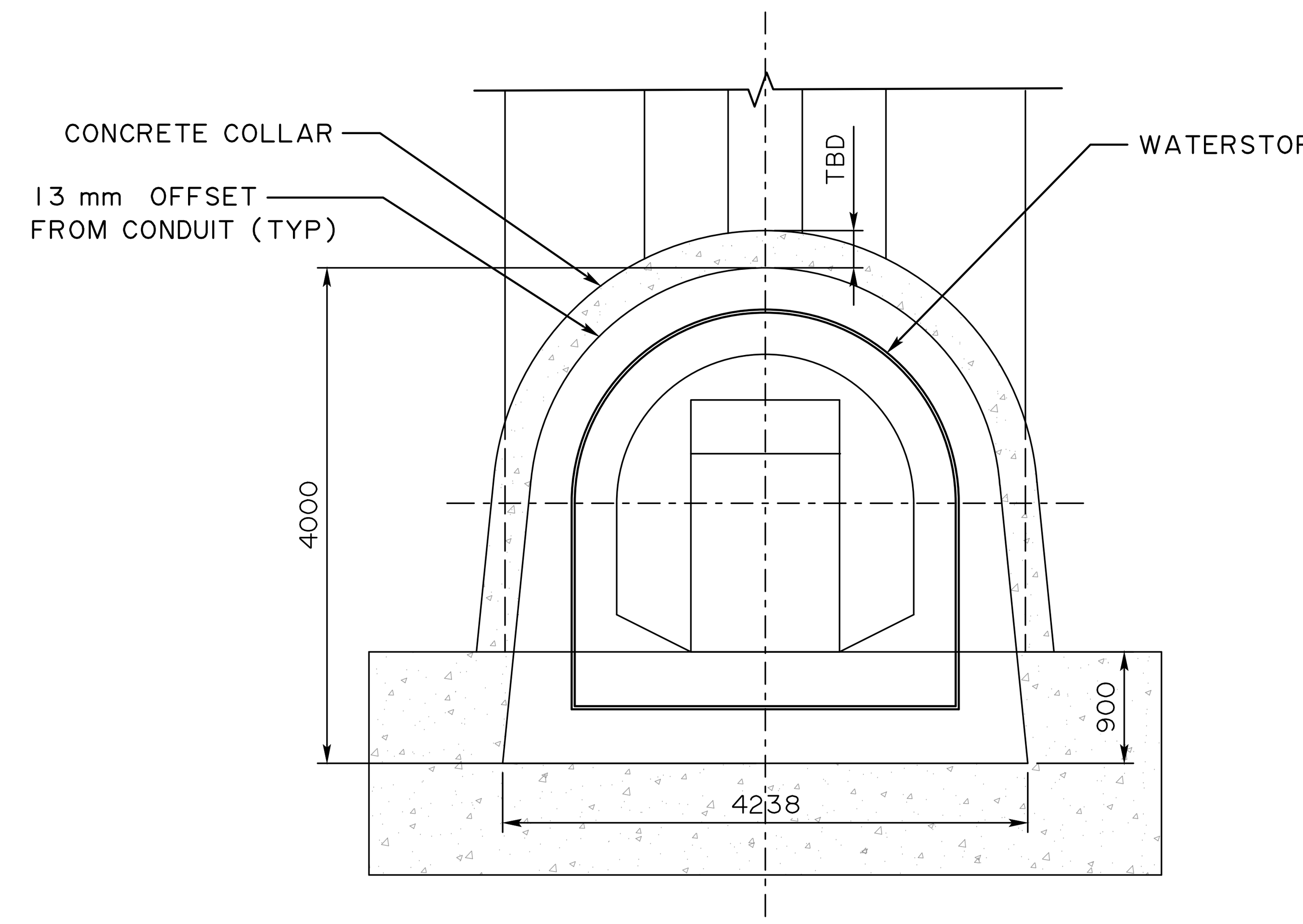
SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
GATE CHAMBER
CONCRETE OUTLINE - SHEET 1 OF 2

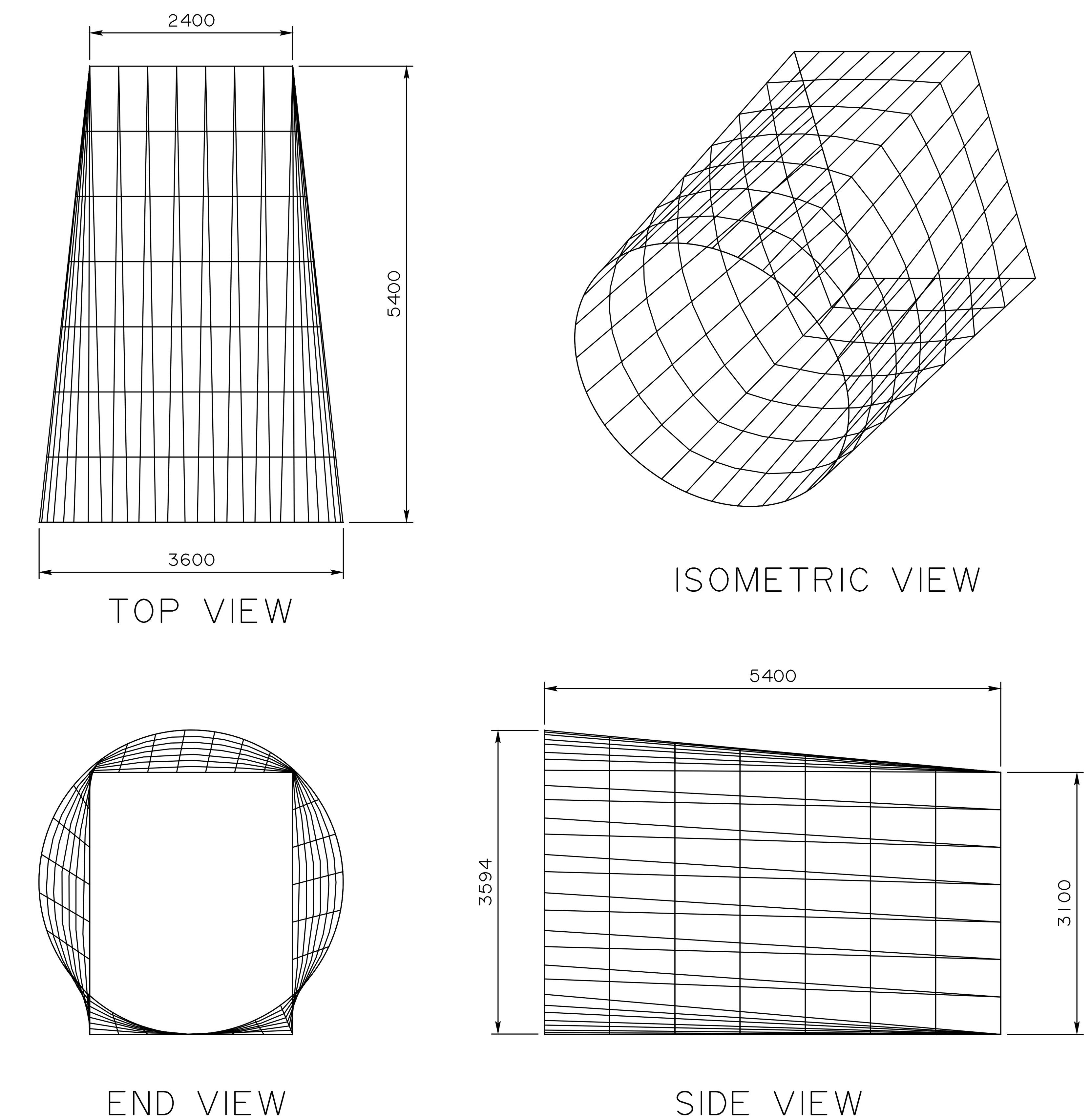
DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-431	



E SECTION 1:50
S-431



F SECTION 1:50
S-431



DETAIL
CONDUIT TRANSITION NTS

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
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Alberta Transportation

SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
GATE CHAMBER
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-432	

PlotDate: 2019.12.03 3:14 PM Login: Shelton, Ben TENDER No. ?
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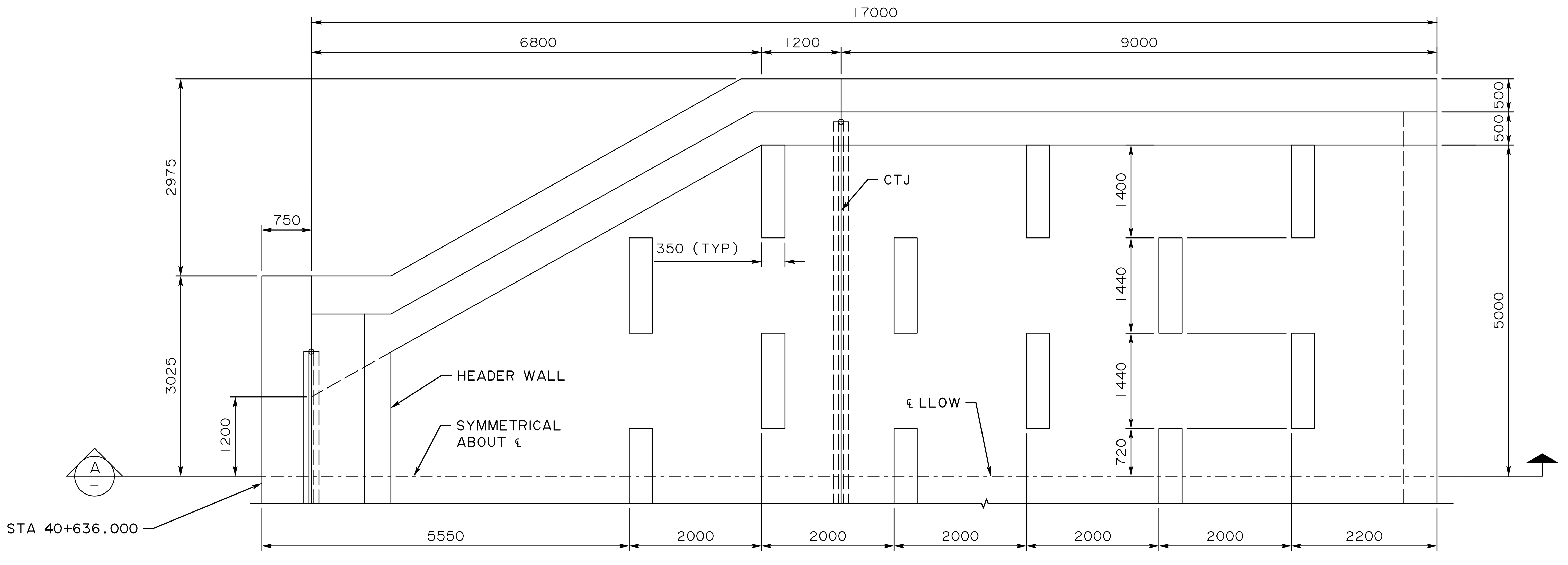
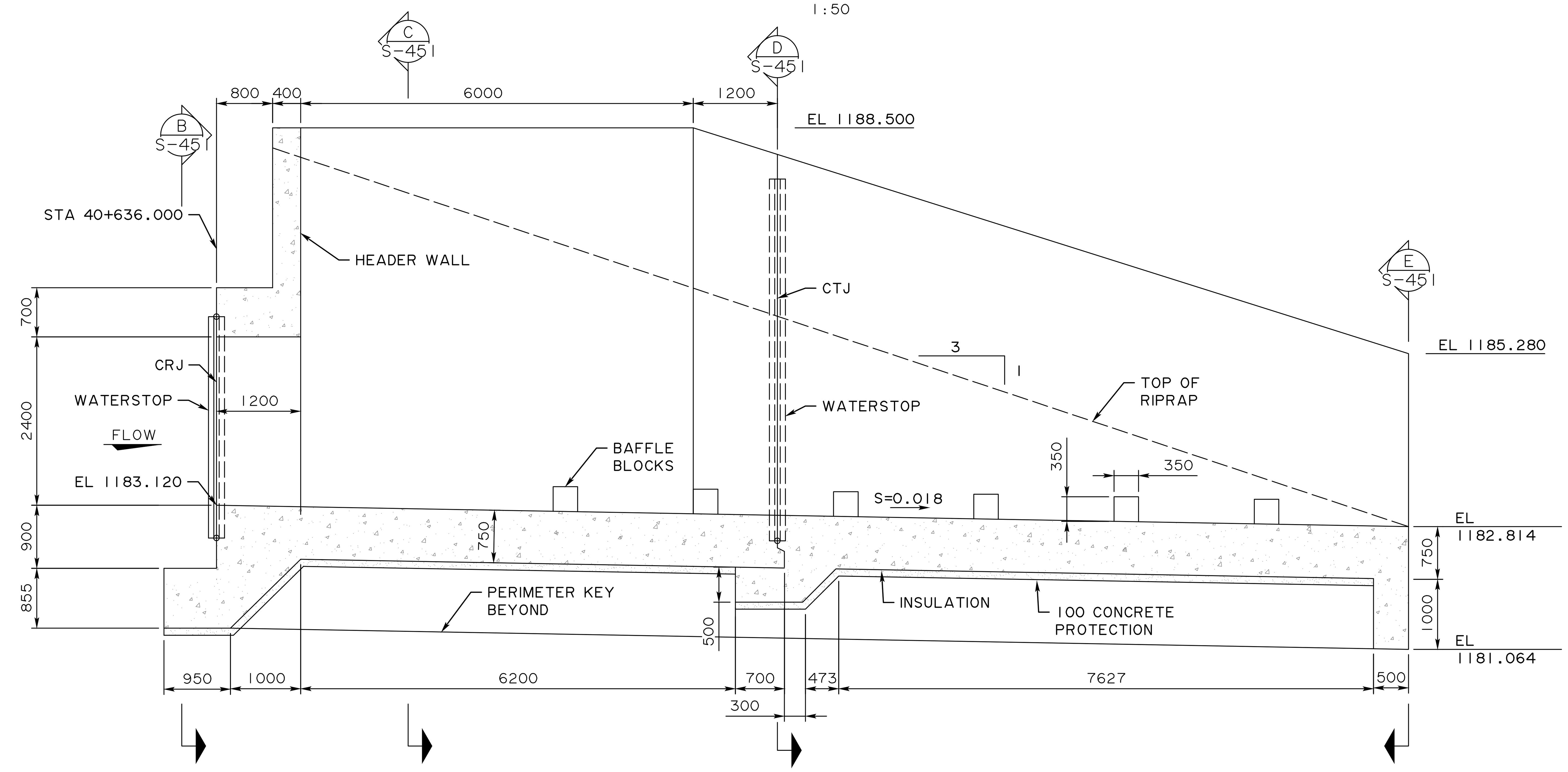


HALF PLAN
1:50



A SECTION
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NOTES:
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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
CSU RIGID BASIN
CONCRETE OUTLINE - SHEET 1 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-450	

PlotDate: 2019.12.03 3:15 PM Login: Shelton, Ben TENDER No. ?
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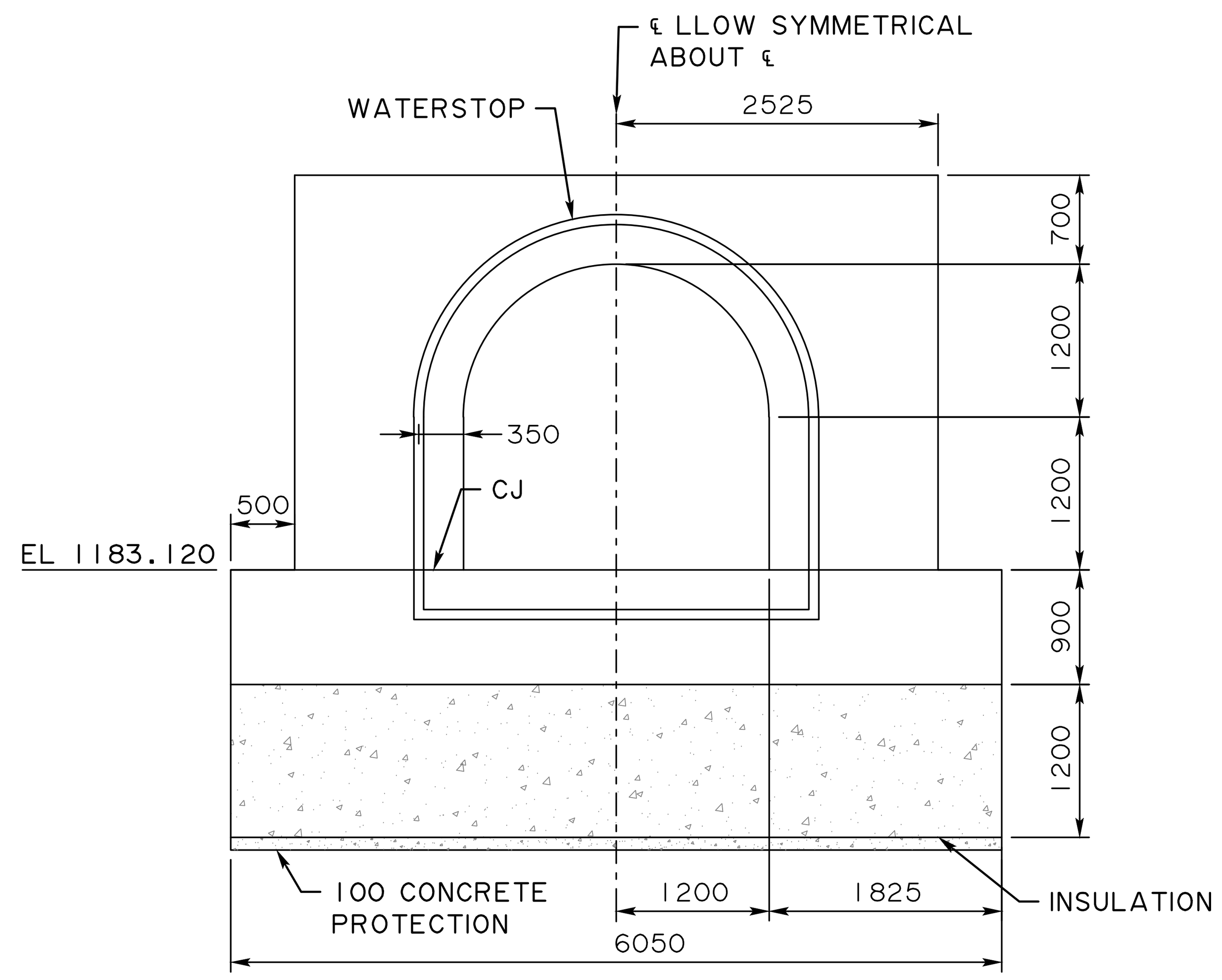
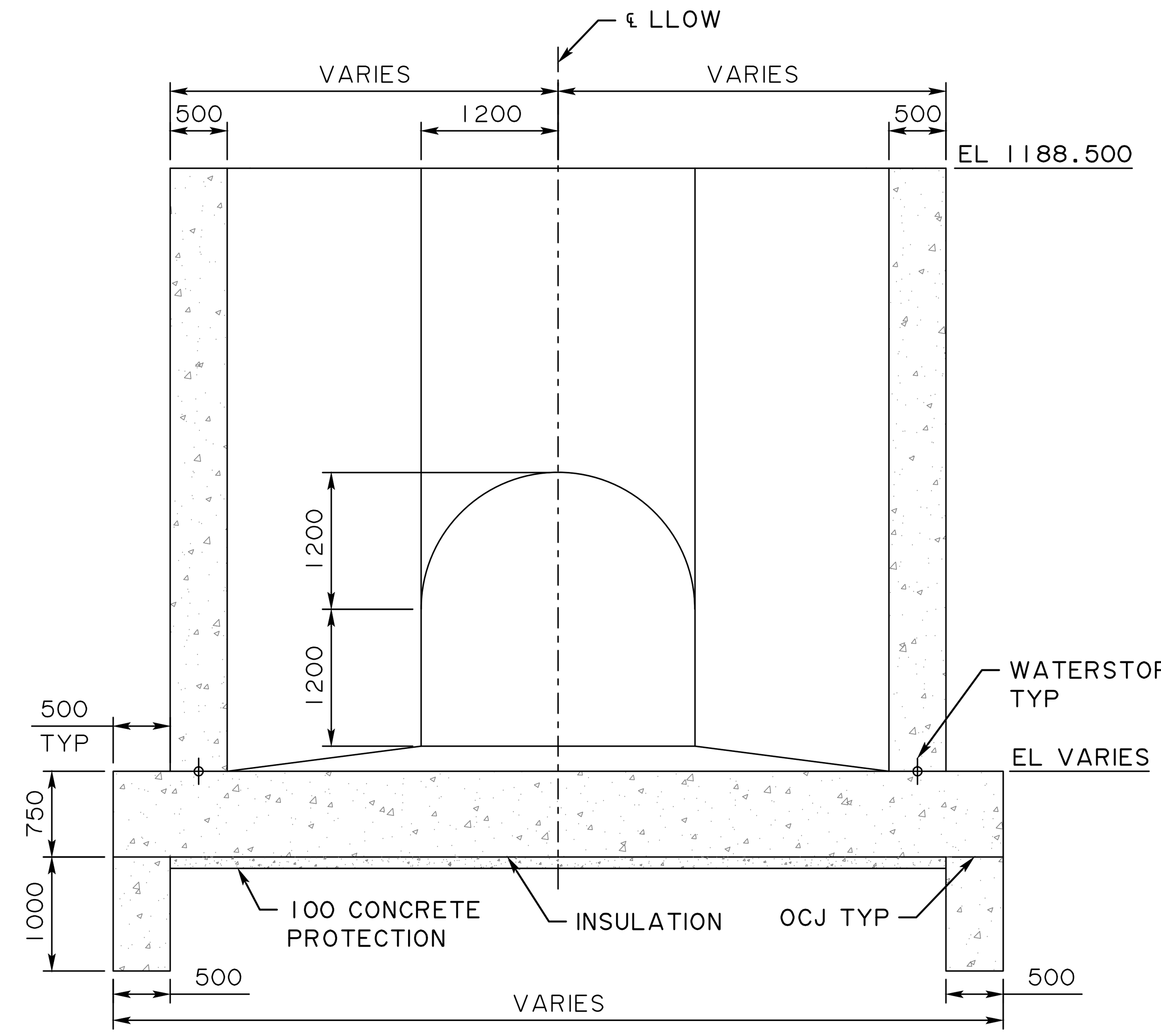
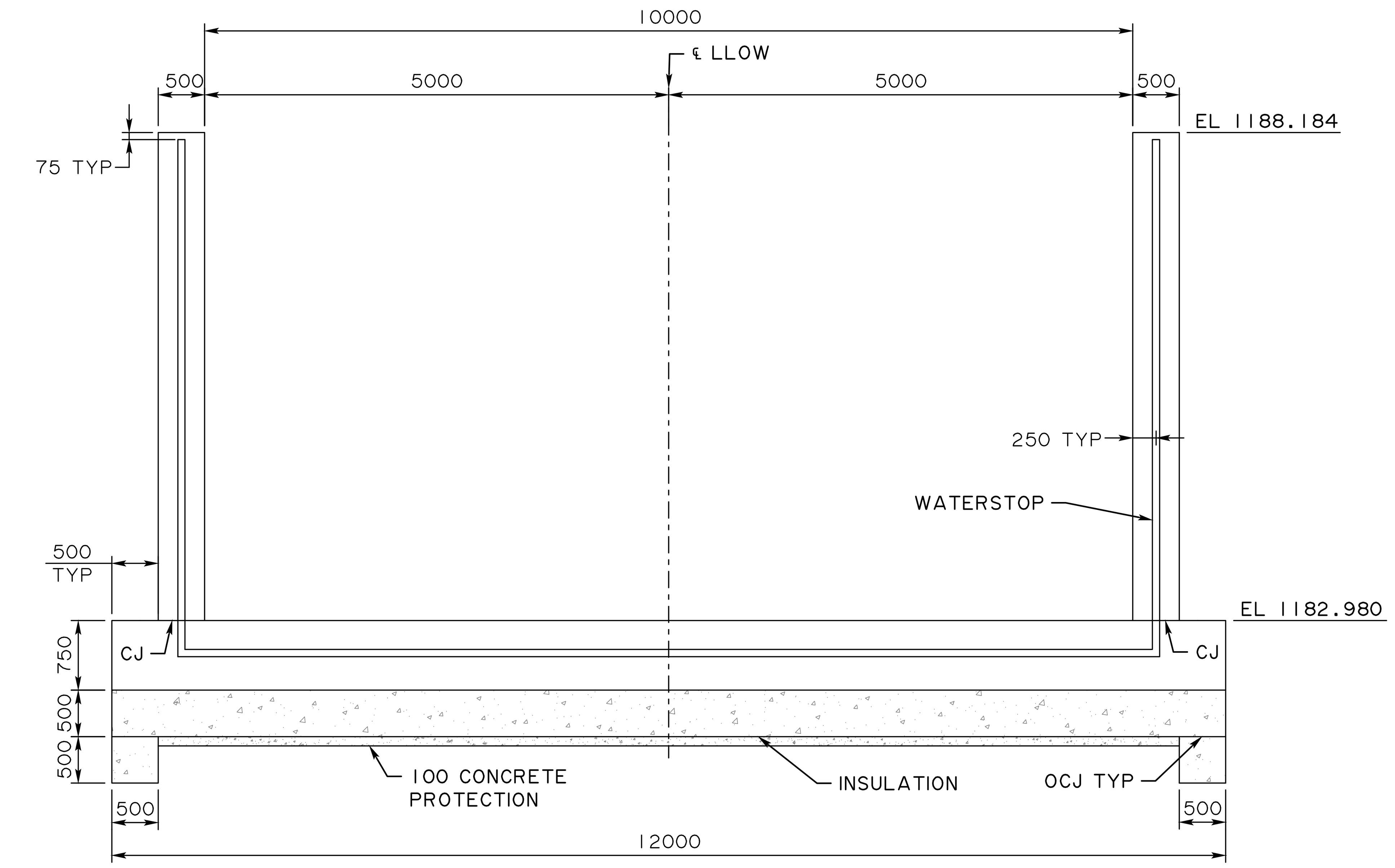


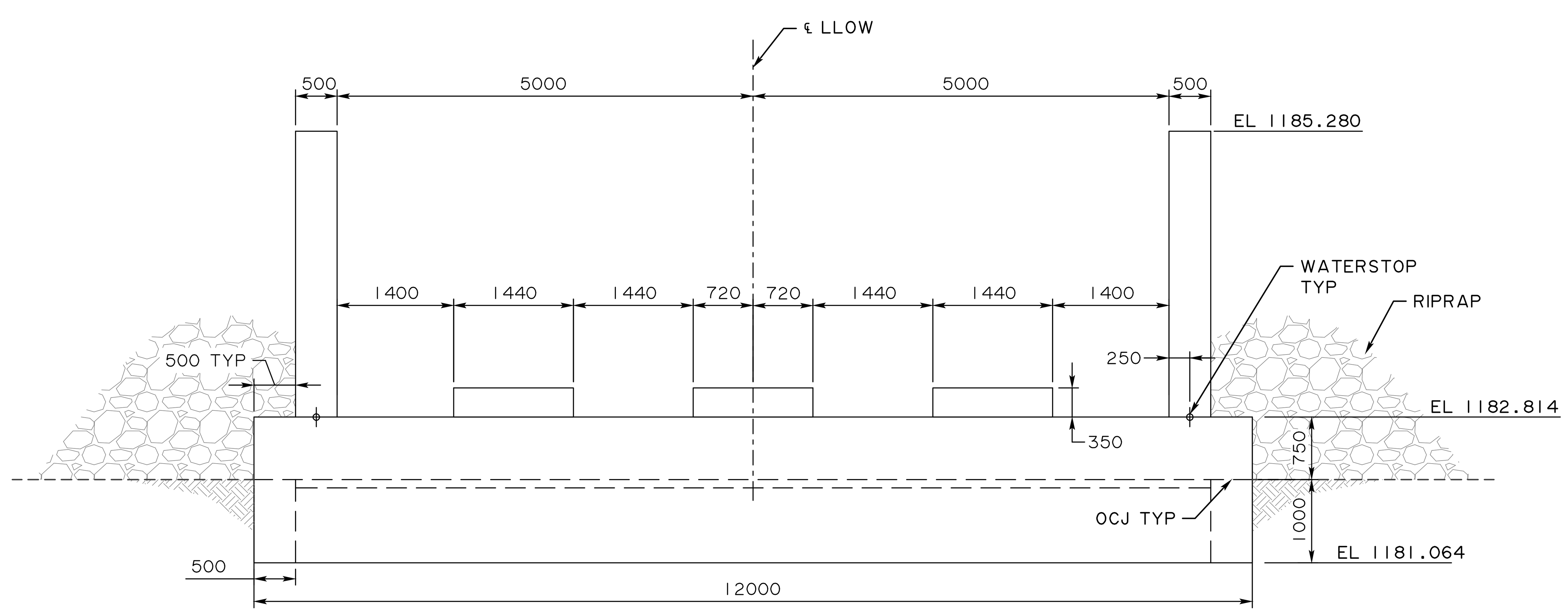
B SECTION AT JOINT
S-450 1:50



C SECTION
S-450 1:50



D SECTION AT JOINT
S-450 1:50



E SECTION
S-450 1:50

NOTES:
1. STATIONS, DISTANCES AND ELEVATIONS ARE IN METRES.
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SPRINGBANK OFF-STREAM STORAGE PROJECT
SR1

LOW LEVEL OUTLET WORKS
CSU RIGID BASIN
CONCRETE OUTLINE - SHEET 2 OF 2

DATE	ALBERTA ENVIRONMENT AND PARKS CAPITAL PROJECTS NUMBER	SITE	SHEET	CONSULTANT DRAWING	TRANSPORTATION DRAWING
2020-09-25	-	-		73396S-451	

APPENDIX B
HYDROLOGY

<PROVIDED UNDER SEPARATE COVER>

APPENDIX C
HYDRAULICS

<PROVIDED UNDER SEPARATE COVER>

APPENDIX D
GEOTECHNICAL

<PROVIDED UNDER SEPARATE COVER>

APPENDIX E
STRUCTURAL

<PROVIDED UNDER SEPARATE COVER>

APPENDIX F
CIVIL

<PROVIDED UNDER SEPARATE COVER>

APPENDIX G
CONSTRUCTION

APPENDIX G.1
SPECIFICATIONS

Springbank Off-Stream Storage Project
Technical Specifications - Outline

Civil Water Works Master Contract Documents and Technical Specifications - Outline

Prepared for:
Alberta Transportation
3rd Floor – Twin Atria Building
4999 – 98 Avenue
Edmonton, AB T6B 2X3

Prepared by:
Stantec Consulting Services Ltd
Calgary, AB

Project Number 110773396

November 27, 2019

Division 1- General Requirements

Division 2 - Sitework

Division 3 - Concrete

Division 5 - Metals

Division 11 - Equipment

CONTRACT DOCUMENTS AND TECHNICAL SPECIFICATIONS

Alberta Transportation Edmonton, Alberta

Division 13 - Special Construction

Division 25 – Integrated Automation

Division 26 - Electrical

Division 27 - Communication

Division 28 – Electronic Safety and Security

Division 31 - Earthwork

Division 35 – Waterway and Marine Construction

APPENDIX G.2
COST OPINION

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Springbank Off-Stream Storage Project (SR1)
Civil Works Cost Opinion - Type B
Revised December 3, 2019

	Item	Unit	Quantity	Unit Price	Estimated Cost (2017 CAD)
1	**General**				
2	Mob./Demobilization	lump sum	5% of Const. Cost	$ 11,490,000.00	$ 11,490,000
3	Project Advertising Signs	ea.	4	$ 3,500.00	$ 14,000
4	Maintenance Existing and Temporary Roads	lump sum		$ 500,000.00	$ 500,000
5	Care of Water	lump sum		$ 2,500,000.00	$ 2,500,000
6					
7	**Removals**				
8	Clearing & Timber Salvage	hectares	53	$ 11,000.00	$ 583,000
9	Existing Fence - Remove and Dispose	km	3.125	$ 3,000.00	$ 9,375
10					
11	**Demolition:**				
12	Remove Existing Buildings	ea.	26	$ 50,000.00	$ 1,300,000
13	Abandon Water Wells	ea.	3	$ 4,500.00	$ 13,500
14	Asphalt surface (Driveways) - Remove & Dispose	m^2	20,500	$ 7.00	$ 143,500
15					
16	**Reinstate disrupted services to residents**				
17	Reinstate Existing Gas Service	ea.	5	$ 8,000.00	$ 40,000
18	Reinstate Electrical Service	ea.	5	$ 17,500.00	$ 87,500
19	Reinstate Telecommunication Service	ea.	5	$ 17,500.00	$ 87,500
20					
21	**Landscaping**				
22	Drill Seeding	hectares	953	$ 1,260.00	$ 1,200,780
23	Hydroseeding	hectares	0	$ 8,000.00	$ -
24					
25	**Roadway Crossings**				
26	Highway 22 Bridge Crossing		See Separate Breakout		$ 4,768,000
27	Township Road 242 Bridge Crossing		See Separate Breakout		$ 3,708,400
28					
29	**Highway 22 and Springbank Road Modifications**				
30	Grade and Resurface Hwy 22 and Springbank Rd.		See Separate Breakout		$ 12,244,340
31					
32	**Site Access Roads: Diversion Structure**				
33	Prepare Subgrade Surface (First Layer)	m^2	18,785	$ 1.00	$ 18,785
34	Asphalt pathway Mix L1 S&I with base gravel	m^2	10,109	$ 35.00	$ 353,815
35	Zone 4A - Base Gravel (2-25 GBC) - 200 mm depth	m^3	2,022	$ 56.00	$ 113,221
36	Zone 4A - Base Gravel (2-25 GBC) - 75 mm depth	m^3	1,409	$ 56.00	$ 78,897
37	Supply of Aggregate - No Option	t	3,311	$ 0.60	$ 1,987
38	High Tension Cable Barrier - Supply and Install	m	205	$ 82.50	$ 16,913
39	Crash Attenuators - TL-3	ea.	2	$ 4,950.00	$ 9,900
40					
41	**Site Access Roads: Diversion Channel**				
42	Prepare Subgrade Surface (First Layer)	m^2	23,386	$ 1.00	$ 23,386
43	Zone 4A - Base Gravel (2-25 GBC) - 75 mm depth	m^3	1,754	$ 56.00	$ 98,221
44	Supply of Aggregate - No Option	t	4,122	$ 0.60	$ 2,473
45					
46	**Site Access Roads: Off-Stream Storage Dam**				
47	Prepare Subgrade Surface (First Layer)	m^2	99,625	$ 1.00	$ 99,625
48	Zone 4A - Base Gravel (2-25 GBC) - 75 mm depth	m^3	7,472	$ 56.00	$ 418,425
49	Supply of Aggregate - No Option	t	17,559	$ 0.60	$ 10,535
50	High Tension Cable Barrier - Supply and Install	m	6,570	$ 82.50	$ 542,025
51					
52	**Site Security:**				
53	New Fence - Supply & Install - Class B (wildlife friendly barbwire)	km	28.60	$ 12,100.00	$ 346,060
54	New Fence - Supply & Install - Class H (Chain-link)	m	316	$ 48.50	$ 15,326
55	Vehicle Access Control Gate	ea.	8	$ 5,500.00	$ 44,000
56	Supply of Signs, Aluminum	m^2	92	$ 250.00	$ 23,040
57	Supply and Install Post (100mm X 150mm)	ea.	64	$ 220.00	$ 14,080
58					
59	**General Subtotal**				**$ 40,920,609**

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Springbank Off-Stream Storage Project (SR1)
Civil Works Cost Opinion - Type B
Revised December 3, 2019

	Item	Unit	Quantity	Unit Price	Estimated Cost (2017 CAD)
60	**Diversion Structure**				
61	**Service Spillway (SS)**				
62	Structural Concrete - Class A	m^3	0	$ 1,340.00	$ -
63	Structural Concrete - Class B	m^3	3,027	$ 1,340.00	$ 4,056,180
64	Mass Concrete (Class M)	m^3	9,442	$ 890.00	$ 8,403,380
65	Foundation Concrete - Class F	m^3	315	$ 623.00	$ 196,245
66	Metal Railings	m	65	$ 450.00	$ 29,250
67	Gate/Bladder Systems - Crest Gates - Supply	lump sum		$ 4,000,000.00	$ 4,000,000
68	Gate/Bladder Systems - Crest Gates - Installation	lump sum		$ 40,000.00	$ 40,000
69	Controls/Instrumentation	lump sum		$ 400,000.00	$ 400,000
70					
71	**Diversion Inlet (DI)**				
72	Structural Concrete (Class A)	m^3	575	$ 1,340.00	$ 770,500
73	Structural Concrete (Class B)	m^3	3,815	$ 1,340.00	$ 5,112,100
74	Mass Concrete (Class M)	m^3	8,770	$ 890.00	$ 7,805,300
75	Foundation Concrete (Class F)	m^3	453	$ 890.00	$ 403,170
76	High Performance (Class HPC)	m^3	116.0	$ 2,080.00	$ 241,280
77	Structural Metal Framing Hoist Bridge Support Steel	kg	44,230	$ 21.45	$ 948,633
78	Parapet Railing	m	176	$ 450.00	$ 79,200
79	Gate/Hoist Systems - Fixed Wheel Lift Gates - Supply	lump sum		$ 3,300,000.00	$ 3,300,000
80	Gate/Hoist Systems - Fixed Wheel Lift Gates - Installation	lump sum		$ 600,000.00	$ 600,000
81	Controls/Instrumentation	lump sum		$ 330,000.00	$ 330,000
82					
83	**Control Building**				
84	Electrical Service 3 Phase, 400 Amp	lump sum		$ 100,000.00	$ 100,000
85	Control Building Structure	lump sum		$ 400,000.00	$ 400,000
86					
87	**SS & DI Excavation, Backfill and Apron**				
88	Topsoil and Subsoil Stripping	m^3	10,676	$ 3.00	$ 32,028
89	Topsoil Placement	m^3	5,338	$ 3.50	$ 18,683
90	Common Excavation	m^3	243,271	$ 5.50	$ 1,337,991
91	Overhaul of Common Excavation	m^3*km	10,041	$ 0.50	$ 5,021
92	Rock Excavation	m^3	130,905	$ 8.75	$ 1,145,419
93	Zone 2A - Random Fill (From Excavation)	m^3	130,275	$ 2.25	$ 293,119
94	Foundation Grouting	Grout Hole	120	$ 10,000.00	$ 1,200,000
95	Foundation Treatment	m^2	6055	$ 200.00	$ 1,211,000
96	Structure Foundation Drains	m	206	$ 550.00	$ 113,300
97	Wall Drains	m	215	$ 1,800.00	$ 387,000
98					
99	**Portage**				
100	Portage Route Pathway	m^2	1,860	$ 35.00	$ 65,100
101					
102	**Fish Passage**				
103	Zone 5B Course Rip Rap Bedding Gravel	m^3	16	$ 75.00	$ 1,230
104	Riprap Zone 6A	m^3	16	$ 165.00	$ 2,706
105	Riprap Zone 6C	m^3	2,122	$ 165.00	$ 350,130
106					
107	**Bank Armoring/Rip Rap Revetment**				
108	Rip Rap Zone 6A	m^3	774	$ 165.00	$ 127,710
109	Riprap Zone 6B	m^3	716	$ 165.00	$ 118,140
110	Riprap Zone 6C	m^3	5,888	$ 165.00	$ 971,520
111	Topsoil Placement	m^3	468	$ 3.50	$ 1,638
112	Landscaping (Willow cuttings or potted stock)	m^2	2,220	$ 6.00	$ 13,320
113					

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Springbank Off-Stream Storage Project (SR1)
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	Item	Unit	Quantity	Unit Price	Estimated Cost (2017 CAD)
114	**Floodplain Berm**				
115	Topsoil and Subsoil Stripping	m^3	13,945	$ 3.00	$ 41,835
116	Topsoil Placement	m^3	6,997	$ 3.50	$ 24,490
117	Common Excavation	m^3	3,661	$ 5.50	$ 20,136
118	Overhaul of Common Excavation	m^3*km	0	$ 0.85	$ -
119	Zone 1A - Impervious Fill	m^3	41,018	$ 1.50	$ 61,527
120	Zone 2A - Random Fill	m^3	10,271	$ 2.25	$ 23,110
121	Fine Filter - Zone 3A	m^3	1,603	$ 55.00	$ 88,165
122	Riprap Zone 6C	m^3	11,851	$ 165.00	$ 1,955,415
123	Non-Woven Geotextile	m^2	9,834	$ 3.50	$ 34,419
124					
125	**Vertical Toe Drain**				
126	Vertical Toe Drain (Sand) -Fine Filter - Zone 3A	m^3	1,100	$ 55.00	$ 60,500
127	Perforated Pipe - Supply and Install (150 mm)	m	550	$ 150.00	$ 82,500
128	Non-Perforated Pipe - Supply and Install (150 mm)	m	445	$ 140.00	$ 62,300
129					
130	**Auxiliary Spillway**				
131	Common Excavation	m^3	24,673	$ 5.50	$ 135,702
132	Topsoil and Subsoil Stripping	m^3	1,974	$ 3.50	$ 6,909
133	Topsoil Placement	m^3	822	$ 3.50	$ 2,877
134	Rock Excavation	m^3	6,694	$ 8.75	$ 58,573
135	Zone 1A - Impervious Fill	m^3	2,917	$ 1.50	$ 4,376
136	Random Fill from Common Excavation	m^3	28,543	$ 2.25	$ 64,222
137	Structural Concrete (Class B)	m^3	656	$ 1,340.00	$ 879,040
138	Hardfill Concrete (Class CSG)	m^3	11261	$ 225.00	$ 2,533,752
139	Facing Concrete (Class B)	m^3	3869	$ 350.00	$ 1,354,080
140	Pads Concrete (Class B)	m^3	0	$ 890.00	$ -
141	Crest Concrete (Class M)	m^3	1088	$ 890.00	$ 968,178
142	Leveling Concrete (Class F)	m^3	274	$ 265.00	$ 72,504
143					
143	**Auxiliary Spillway Fuse Plug**				
144	Zone 3A - Fine Filter	m^3	296	$ 55.00	$ 16,280
145	Zone 3B - Coase Filter	m^3	445	$ 60.00	$ 26,700
146	Zone - 5C Gravel Armour Zone	m^3	290	$ 75.00	$ 21,750
147					
148	**Debris Deflection Barrier**				
149	Common Excavation	m^3	7,508	$ 5.50	$ 41,291
150	Foundation Treatment	m^2	990	$ 200.00	$ 198,000
151	Structural Concrete (Class A)	m^3	1,359	$ 1,340.82	$ 1,822,697
152	Structural Steel Fabrication	kg	226,871	$ 21.45	$ 4,866,383
153	Steel Erection	days	17	$ 9,035.00	$ 153,595
154	Random Fill from Common Excavation	m^3	3,465	$ 1.50	$ 5,198
155	Caissons	each	66	$ 2,000.00	$ 132,000
156	**Diversion Structure Subtotal**				**$ 60,428,792**

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Springbank Off-Stream Storage Project (SR1)
Civil Works Cost Opinion - Type B
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	Item	Unit	Quantity	Unit Price	Estimated Cost (2017 CAD)
157	**Diversion Channel**				
158	**Emergency Spillway (EMS)**				
159	Structural Concrete (Class A)	m^3	859	$ 1,340.00	$ 1,151,060
160	Structural Concrete (Class B)	m^3	3,977	$ 1,340.00	$ 5,329,180
161	Metal Railings	m	140	$ 450.00	$ 63,000
162	Foundation Treatment	m^2	3,321	$ 200.00	$ 664,200
163	Structure Foundation Drains	m	135	$ 550.00	$ 74,250
164					
165	**Diversion Channel**				
166	Topsoil and Subsoil Stripping	m^3	176,483	$ 3.00	$ 529,449
167	Topsoil Placement	m^3	88,242	$ 3.50	$ 308,845
168	Common Excavation	m^3	3,948,858	$ 5.50	$ 21,718,719
169	Overhaul of Common Excavation	m^3*km	19,744,260	$ 0.85	$ 16,782,621
170	Rock Excavation	m^3	1,129,174	$ 8.75	$ 9,880,273
171					
172	**Diversion Channel Embankment Fill Sections**				
173	Zone 1A - Impervious Fill	m^3	21,641	$ 1.50	$ 32,462
174	Zone 2A - Random Fill	m^3	51,541	$ 2.25	$ 115,967
175					
176	**Diversion Channel Erosion Control**				
177	Zone 5B Course Rip Rap Bedding Gravel	m^3	16,935	$ 75.00	$ 1,270,125
178	Riprap Zone 6A	m^3	23,836	$ 165.00	$ 3,932,940
179	Riprap Zone 6B	m^3	66,136	$ 165.00	$ 10,912,440
180	Riprap Zone 6C	m^3	28,715	$ 165.00	$ 4,737,975
181	Riprap Zone 6D	m^3	37,848	$ 165.00	$ 6,244,920
182	Turf Reinforcement Mats	m^2	54,367	$ 17.00	$ 924,239
183	Non-Woven Geotextile	m^2	230,699	$ 3.50	$ 807,447
184					
185	**Seepage Control**				
186	Vertical Toe Drain (Sand) - Fine Filter - Zone 3A	m^3	8,487	$ 55.00	$ 466,785
187	150mm Perforated Pipe	m	2,829	$ 150.00	$ 424,350
188	150mm Pipe	m	1,226	$ 140.00	$ 171,640
189	Headwall	ea.	96	$ 300.00	$ 28,800
190					
191	**Diversion Channel Outlet (RCC Grade Control Structure)**				
192	RCC Stepped Overlay	m^3	10,542	$ 265.00	$ 2,793,630
193	Fine Filter - Zone 3A	m^3	6,594	$ 55.00	$ 362,670
194	Structural Concrete (Class A)	m^3	536	$ 1,340.00	$ 718,240
195					
196	**Storm Water Control**				
197	Access Road Culverts (Supply and Install)	m	53	$ 660.00	$ 34,980
198					
199	**Diversion Channel Subtotal**				**$ 90,481,206**

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Springbank Off-Stream Storage Project (SR1)
Civil Works Cost Opinion - Type B
Revised December 3, 2019

	Item	Unit	Quantity	Unit Price	Estimated Cost (2017 CAD)
200	**Off-Stream Storage Dam**				
201	**Dam Embankment**				
202	Topsoil and Subsoil Stripping	m^3	252,670	$ 3.00	$ 758,011
203	Topsoil Placement	m^3	107,934	$ 3.50	$ 377,770
204	Common Excavation	m^3	471,793	$ 5.50	$ 2,594,862
205	Overhaul of Common Excavation	m^3*km	0	$ 0.85	$ -
206	Zone 1A - Impervious Fill	m^3	1,748,035	$ 1.50	$ 2,622,053
207	Zone 2A - Random Fill	m^3	2,841,518	$ 2.25	$ 6,393,416
208	Toe Buttress - Random Fill Zone 2A(3)	m^3	222,884	$ 2.25	$ 501,489
209	Drainage Zone - Zone 3B	m^3	18,858	$ 24.00	$ 452,592
210	Fine Filter - Zone 3A	m^3	240,119	$ 55.00	$ 13,206,545
211	Dam Face Drainage Flumes (Riprap Zone 6B)	m^3	547	$ 165.00	$ 90,255
212	Non-Woven Geotextile	m^2	811	$ 3.50	$ 2,839
213					
214	**Geotechnical Instruments**				
215	Instrumentation	lump sum		$ 1,500,000.00	$ 1,500,000
216					
217	**Vertical Toe Drain**				
218	Vertical Toe Drain Fine Filter - Zone 3A	m^3	7,229	$ 55.00	$ 397,595
219	Relief Wells 1.0 m by 3.0 m Depth	ea.	6	$ 450.00	$ 2,700
220	Perforated Pipe - Supply and Install (150 mm)	m	3,086	$ 150.00	$ 462,900
221	Non-Perforated Pipe - Supply and Install (150 mm)	m	196	$ 140.00	$ 27,440
222					
223	**Borrow**				
224	Borrow Area Excavation	m^3	0	$ 5.50	$ -
225	Overhaul of Borrow Area Excavation	m^3*km	0	$ 0.50	$ -
226	Topsoil and Subsoil Stripping - Borrow Pit	m^3	0	$ 3.00	$ -
227	Topsoil Placement	m^3	0	$ 3.50	$ -
228	Drill Seeding	hectares	0	$ 1,260.00	$ -
229					
230	**Low-Level Outlet Works (LLOW)**				
231	Structural Concrete (Class A)	m^3	3,827	$ 1,340.00	$ 5,128,180
232	Foundation Concrete (Class F)	m^3	131	$ 265.00	$ 34,715
233	Structural Metal Framing - Steel Trash Racks	kg	8,800	$ 21.45	$ 188,740
234	Air Vent Piping and Accessories	lump sum		$ 10,000.00	$ 10,000
235	Ladders, Platform Grating, Support Beams	lump sum		$ 40,000.00	$ 40,000
236	Gate/Hoist Systems - Heavy Duty Sluice Gate	ea.	2	$ 97,000.00	$ 194,000
237	Pedestrian Bridge	lump sum		$ 121,600.00	$ 121,600
238	Controls/Instrumentation	lump sum		$ 9,700.00	$ 9,700
239	Electrical Service 3 Phase, 400 Amp	lump sum		$ 100,000.00	$ 100,000
240	Crane/Hoist system (2 ton capacity)	lump sum		$ 72,463	$ 72,463
241					
242					
243	**LLOW Inlet/Outlet Drainage Channel**				
244	Riprap Zone 6B	m^3	15331	$ 165.00	$ 2,529,615
245	Non-Woven Geotextile	m^2	25552	$ 3.50	$ 89,432
246	Drainage Culverts (Supply and Install)	m	30	$ 660.00	$ 19,800
247					
248	**Off-Stream Storage Dam Subtotal**				**$ 37,928,710**

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Springbank Off-Stream Storage Project (SR1)
Civil Works Cost Opinion - Type B
Revised December 3, 2019

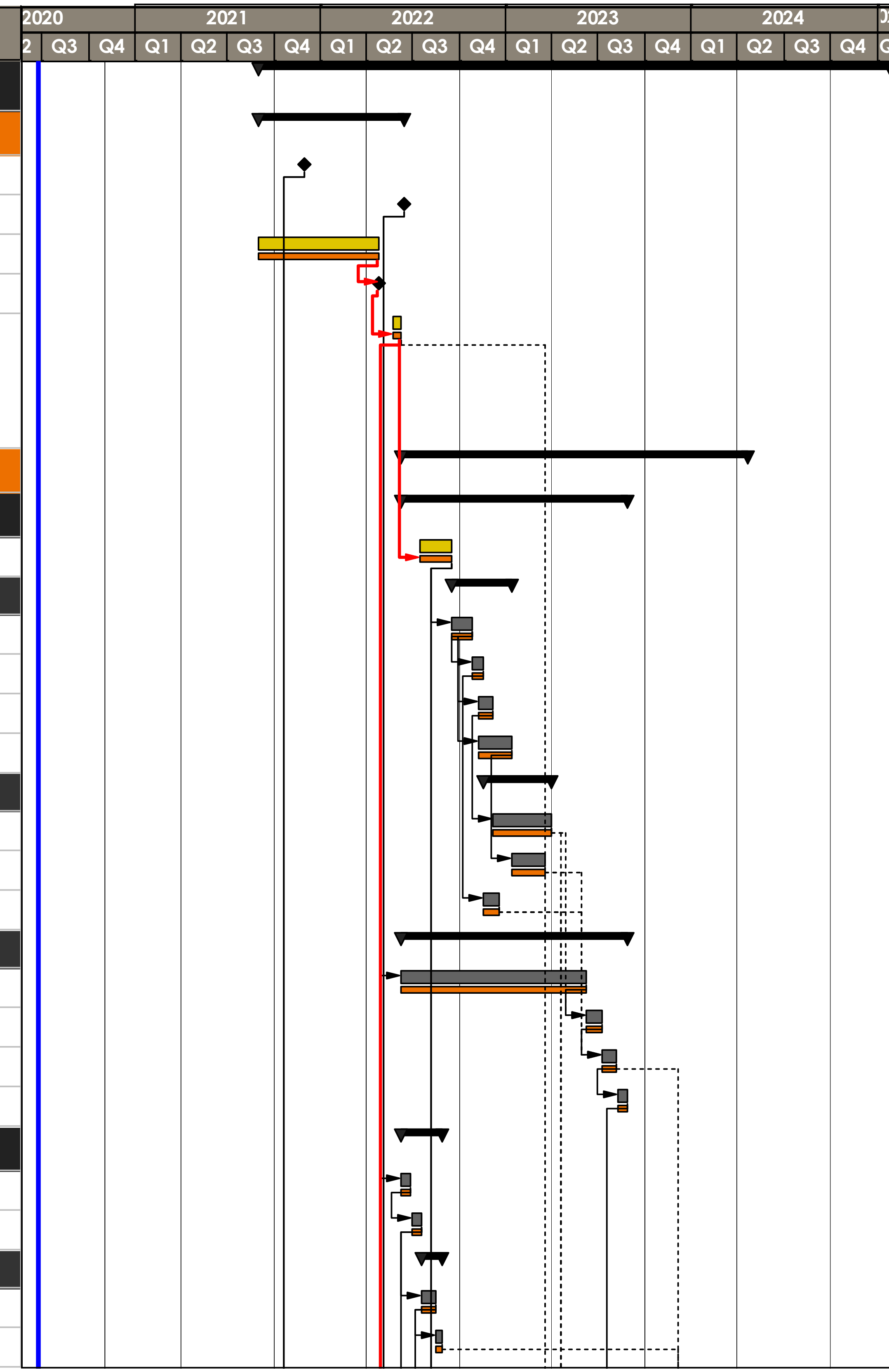
	Item	Unit	Quantity	Unit Price	Estimated Cost (2017 CAD)
249	**Totals**				
250	General - Subtotal				$ 40,920,609
251	Diversion Structure - Subtotal				$ 60,428,792
252	Diversion Channel - Subtotal				$ 90,481,206
253	Off-Stream Storage Dam - Subtotal				$ 37,928,710
254	**Construction Subtotal**				**$ 229,759,317**
255	Construction Contingencies (%)	15%			$ 34,464,000
256	**Construction and Contingecy Total**				**$ 264,223,317**
257					
258	**Utility Relocations (Mobilization and Contingency - Not Included)**				
259	**Shallow Utility Relocations**				
260	FORTIS - Salvage and Reinstate Utilities				$ 1,907,450
261	SHAW - Salvage and Reinstate Utilities				$ 401,200
262	TELUS - Salvage and Reinstate Utilities				$ 601,200
263	ATCO - Salvage and Reinstate Utilities				$ 351,150
264	**Subtotal - Shallow Utilities**				**$ 3,261,000**
265					
266	**Major Utility Relocations**				
267	TransCanada Pipelines Ltd.				$ 3,030,000
268	Pengrowth Energy Corporation				$ 718,750
269	Veresen Inc				$ 722,500
270	Plains Midstream				$ 7,672,500
271	Altalink				$ 300,000
272	**Subtotal - Major Utilities**				**$ 12,443,750**
273	**Utility Relocations Total**				**$ 15,704,750**
274					
275	**Engineering, Permitting and Administration (Mobilization and Contingency - Not Included)**				
276	Engineering/Environmental Fees				$ 60,700,000
277					
278					
279					
280	**Engineering, Permitting and Administration Total**				**$ 60,700,000**
281					
282	**Total Project Cost Opinion**				**$ 340,628,067**

APPENDIX G.3
SCHEDULE

Springbank Off-Stream Storage Project

Stantec

Activity ID	Activity Name	Remaining Duration	Start	Finish	Predecessors	Successors
Springbank Off-Stream Storage Project		1244	31-Aug-21	26-Jan-25		
Preconstruction		286	31-Aug-21	13-Jun-22		
A1000	Regulatory Approval	0	01-Dec-21*			A3130, A3140
A1010	Land Acquisition	0	13-Jun-22*			A4220, A4320
A1020	Tender Period	237	31-Aug-21*	24-Apr-22		A1030
A1030	Project Award	0		25-Apr-22	A1020	A1040
A1040	Mobilization	10	25-May-22	08-Jun-22	A1030	A2060, A2050, A3150, A3160, A3210, A4010, A4120, A4202, A2090, A2000, A2080, A4080, A2010
Diversion Structure		682	09-Jun-22	20-Apr-24		
Service Spillway		445	09-Jun-22	27-Aug-23		
A4080	Temporary River Diversion	63	15-Jul-22*	15-Sep-22	A1040	A4090, A4190, A4322
Foundation		120	16-Sep-22	13-Jan-23		
A4090	Excavation - Zone L	42	16-Sep-22	27-Oct-22	A4080	A4092, A4094, A4096
A4092	Foundation Treatment	20	28-Oct-22	16-Nov-22	A4090	A4104
A4094	Foundation Grouting	30	07-Nov-22	06-Dec-22	A4090	A4100
A4096	Install Stilling Basin Anchors	68	07-Nov-22	13-Jan-23	A4090	A4102
Structure		136	17-Nov-22	01-Apr-23		
A4100	Crest Monoliths (1A to 5A)	116	07-Dec-22	01-Apr-23	A4094	A4130, A4250
A4102	Stilling Basin Monoliths (1B to 5B)	63	14-Jan-23	17-Mar-23	A4096	A4250, A4140
A4104	Retaining Walls (5C & 5D)	32	17-Nov-22	18-Dec-22	A4092	A4140, A4250
Gates -Obermeyer		445	09-Jun-22	27-Aug-23		
A4120	Equipment Procurement	365	09-Jun-22	08-Jun-23	A1040	A4130
A4130	Gate Installation	30	09-Jun-23	08-Jul-23	A4100, A4120	A4140
A4140	Commissioning	30	09-Jul-23	07-Aug-23	A4130, A4102, A4104	A4150, A4330
A4150	Restore River Flow	20	08-Aug-23*	27-Aug-23	A4140	A4160
Floodplain Berm		82	09-Jun-22	29-Aug-22		
A4010	Demolition of Buidlings / Utilities	20	09-Jun-22	28-Jun-22	A1040	A4020
A4020	Stripping and Foundation Preparation	21	29-Jun-22	19-Jul-22	A4010	A4050, A4172
Embankment		41	20-Jul-22	29-Aug-22		
A4050	Zone N	27	20-Jul-22*	15-Aug-22	A4020	A4060, A4070
A4060	Rock Slope Protection	14	16-Aug-22	29-Aug-22	A4050	A4330

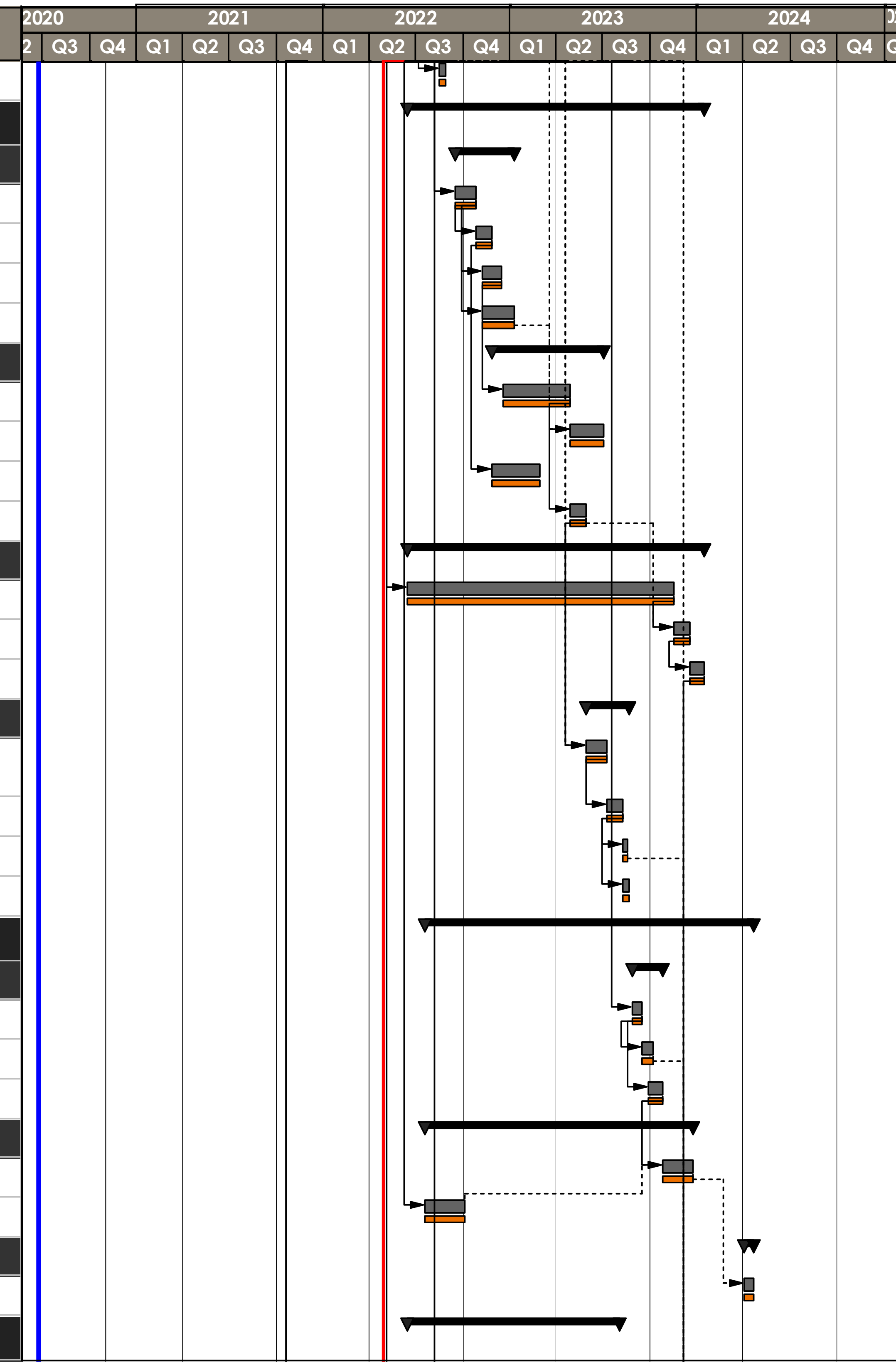


Milestone · Summary · Remaining Level of Effort-Activity · Actual Level of Effort-Activity · Remaining Work · Duration Complete · Primary Baseline · Critical Remaining Work

Date Updated: 24-Jun-20

Springbank Off-Stream Storage Project

Activity ID	Activity Name	Remaining Duration	Start	Finish	Predecessors	Successors
A4070	Vegetation	14	16-Aug-22	29-Aug-22	A4050	
Diversion Inlet		583	13-Jun-22	16-Jan-24		
Foundation		116	16-Sep-22	09-Jan-23		
A4190	Excavation - Zone K	42	16-Sep-22	27-Oct-22	A4080	A4192, A4194, A4196
A4192	Foundation Treatment	30	28-Oct-22	26-Nov-22	A4190	A4204
A4194	Foundation Grouting	40	07-Nov-22	16-Dec-22	A4190	A4200
A4196	Install Stilling Basin Anchors	64	07-Nov-22	09-Jan-23	A4190	A4202
Structure		220	27-Nov-22	04-Jul-23		
A4200	Crest Monoliths (1A&B to 5A&B)	133	17-Dec-22	28-Apr-23	A4194	A4202, A4210
A4202	Stilling Basin Monoliths (1C to 5C)	67	29-Apr-23	04-Jul-23	A1040, A4200, A4196	
A4204	Retaining Walls (1D & E, 5D & E, 6 to 8)	95	27-Nov-22	01-Mar-23	A4192	
A4210	Superstructure CIP Concrete	30	29-Apr-23	28-May-23	A4200	A4230, A4250
Gates		583	13-Jun-22	16-Jan-24		
A4220	Equipment Procurement	523	13-Jun-22	17-Nov-23	A1010	A4230
A4230	Hoist Bridge & Gate Installation	30	18-Nov-23	17-Dec-23	A4220, A4210	A4240
A4240	Commissioning	30	18-Dec-23	16-Jan-24	A4230	A4330
Control Building		86	29-May-23	22-Aug-23		
A4250	Install Drains and Backfill Retaining Walls	42	29-May-23	09-Jul-23	A4102, A4100, A4210, A4104	A4260
A4260	Building	30	10-Jul-23	08-Aug-23	A4250	A4280, A4270
A4270	Commissioning	10	09-Aug-23	18-Aug-23	A4260	A4330
A4280	Paving	14	09-Aug-23	22-Aug-23	A4260	
Auxiliary Spillway		641	20-Jul-22	20-Apr-24		
Foundation		60	28-Aug-23	26-Oct-23		
A4160	Foundation Excavation	20	28-Aug-23	16-Sep-23	A4150	A4162, A4164
A4162	Foundation Treatment	20	17-Sep-23	06-Oct-23	A4160	A4330
A4164	Foundation Grouting	30	27-Sep-23	26-Oct-23	A4160	A4170
Structure		524	20-Jul-22	25-Dec-23		
A4170	Hardfill Dam & Facing Concrete	60	27-Oct-23	25-Dec-23	A4172, A4164	A4180
A4172	Retaining Wall CIP Concrete	77	20-Jul-22	04-Oct-22	A4020	A4170
Embankment and Fuse Plug		20	01-Apr-24	20-Apr-24		
A4180	Zone M	20	01-Apr-24*	20-Apr-24	A4170	
Debris Deflection Barrier		416	13-Jun-22	02-Aug-23		



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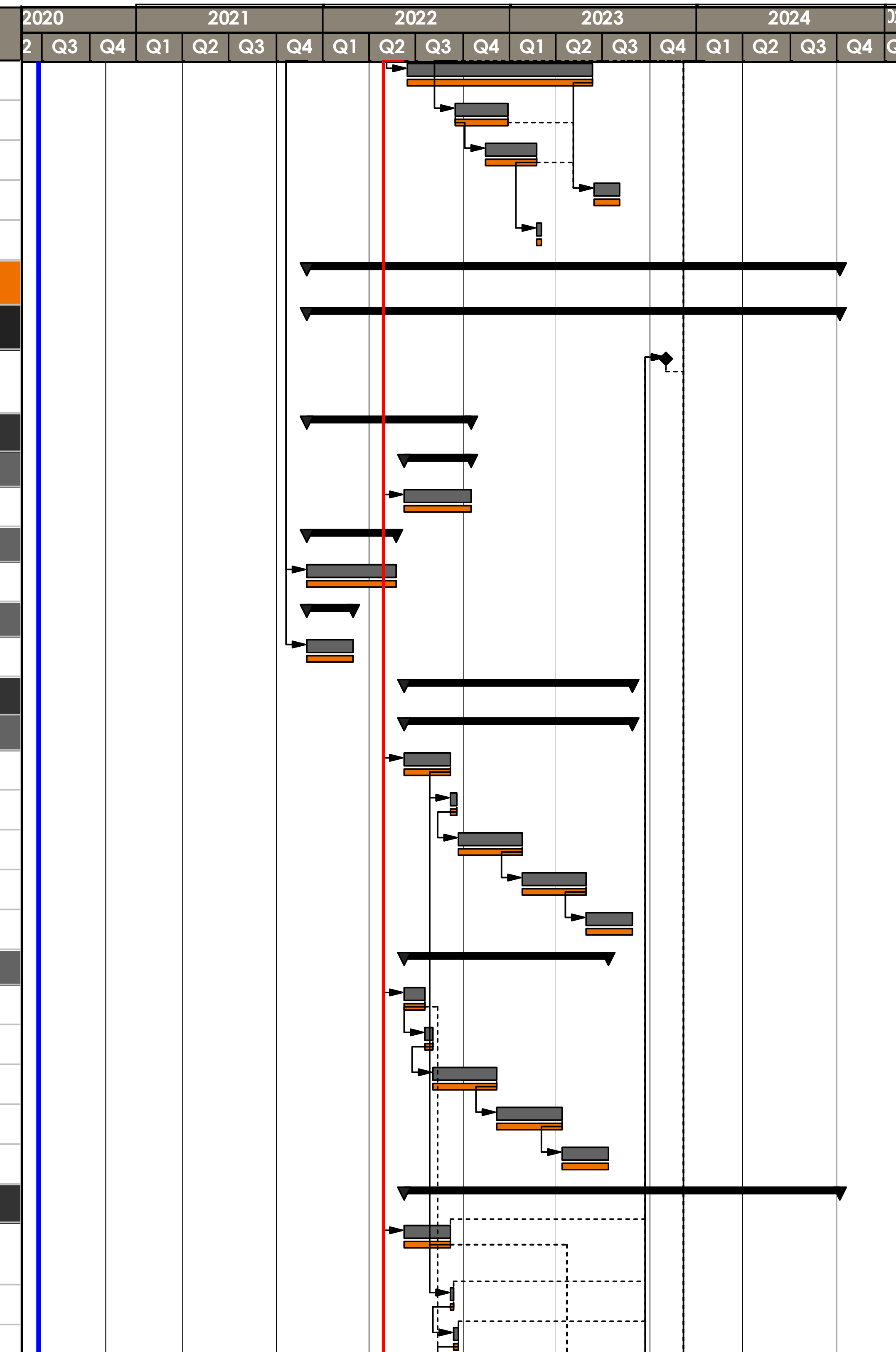
Activity ID	Activity Name	Remaining Duration	Start	Finish	Predecessors	Successors
A4320	Steel Fabrication	365	13-Jun-22	12-Jun-23	A1010	A4326
A4322	Drilled Caissons	102	16-Sep-22	26-Dec-22	A4080	A4324, A4326
A4324	Concrete Footing and Walls	100	15-Nov-22	22-Feb-23	A4322	A4326, A4328
A4326	Steel Erection	51	13-Jun-23	02-Aug-23	A4320, A4324, A4322	
A4328	Excavation and Backfilling Behind Walls	10	23-Feb-23	04-Mar-23	A4324	
Diversion Channel		1040	01-Dec-21	05-Oct-24		
Channel		1040	01-Dec-21	05-Oct-24		
A4310	Iterim Risk Mitigation Milestone	0		31-Oct-23	A3020, A3060, A3070, A3080, A3090, A3110	A4330
Utilities		221	01-Dec-21	17-Oct-22		
Trans-Alta		90	09-Jun-22	17-Oct-22		
A3150	Relocate Existing Power Poles	90	09-Jun-22	17-Oct-22	A1040	
TC Energy		120	01-Dec-21	23-May-22		
A3130	Install New Lines and Protection	120	01-Dec-21	23-May-22	A1000	
Caledonian Midstream/Pembina		60	01-Dec-21	28-Feb-22		
A3140	Install New Lines and Protection	60	01-Dec-21	28-Feb-22	A1000	
Transportation		447	09-Jun-22	29-Aug-23		
Highway 22 Bridge		447	09-Jun-22	29-Aug-23		
A3160	Temporary Detour Road and Bridge	90	09-Jun-22	06-Sep-22	A1040	A3170, A3070
A3170	Piling	14	07-Sep-22	20-Sep-22	A3160	A3180
A3180	Substructure	126	21-Sep-22	24-Jan-23	A3170	A3190
A3190	Superstructure	126	25-Jan-23	30-May-23	A3180	A3200
A3200	Finishes and Circulation Open	91	31-May-23*	29-Aug-23	A3190	
Township Road 242 Bridge		398	09-Jun-22	11-Jul-23		
A3210	Temporary Detour	41	09-Jun-22	19-Jul-22	A1040	A3220, A3090
A3220	Piling	14	20-Jul-22	02-Aug-22	A3210	A3230
A3230	Substructure	126	03-Aug-22	06-Dec-22	A3220	A3240
A3240	Superstructure	126	07-Dec-22	11-Apr-23	A3230	A3250
A3250	Finishes and Circulation Open	91	12-Apr-23*	11-Jul-23	A3240	
Excavation		850	09-Jun-22	05-Oct-24		
A3060	Zone G	90	09-Jun-22	07-Sep-22	A2090	A3070, A3000, A3030, A4310
A3070	Zone H	5	07-Sep-22	13-Sep-22	A3060, A3160	A3080, A4310
A3080	Zone I	10	14-Sep-22	23-Sep-22	A3070	A3090, A4310



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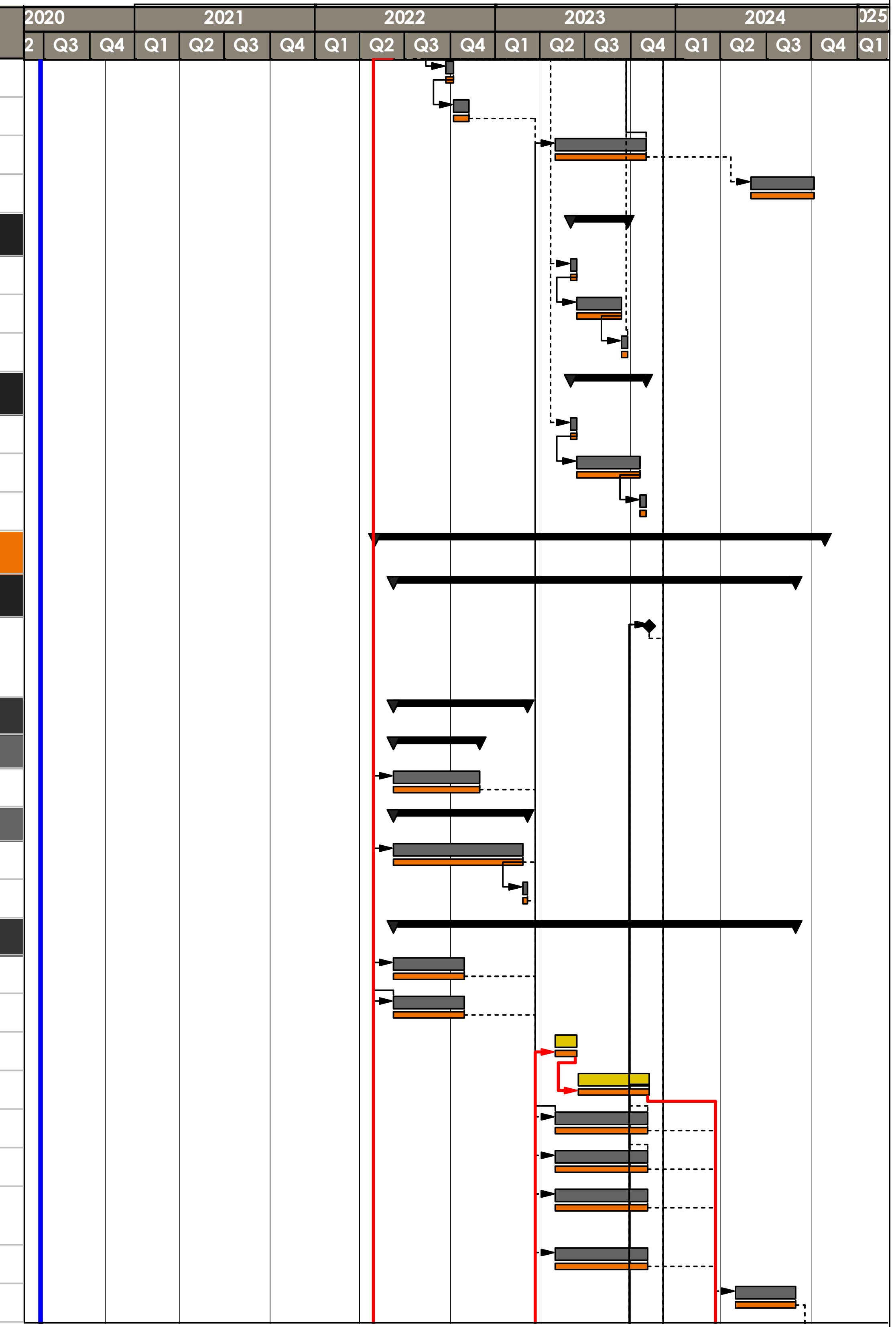
A3090	Zone J	10	23-Sep-22	07-Oct-22	A3080, A3210	A3100, A4310
A3100	Zone K (Year 1)	20	07-Oct-22	07-Nov-22	A3090	A3110
A3110	Zone K (Year 2)	184	01-May-23	31-Oct-23	A3100, A2120	A3120, A4310
A3120	Zone K (Year 3)	127	01-Jun-24*	05-Oct-24	A3110	
Diversion Channel Outlet Structure		118	01-Jun-23	26-Sep-23		
A3000	Foundation Preparation	10	01-Jun-23*	14-Jun-23	A3060	A3010
A3010	RCC Stepped Structure	90	15-Jun-23	12-Sep-23	A3000	A3020
A3020	Excavation / Stabilization of Discharge Channel	14	13-Sep-23	26-Sep-23	A3010	A4310
Emergency Spillway		155	01-Jun-23	02-Nov-23		
A3030	Foundation Preparation	13	01-Jun-23*	13-Jun-23	A3060	A3040
A3040	CIP Structure	90	14-Jun-23	19-Oct-23	A3030	A3050
A3050	Excavation / Stabilization of Discharge Channel	14	20-Oct-23	02-Nov-23	A3040	
Off-Stream Storage Dam and Reservoir		910	02-May-22	28-Oct-24		
Storage Dam Embankment		812	09-Jun-22	29-Aug-24		
A4300	Interim Risk Mitigation Milestone	0		06-Nov-23	A2110, A2120, A2130, A4290, A2020, A2030, A2035	A4330
Utilities		271	09-Jun-22	06-Mar-23		
Shallow Utilities		120	09-Jun-22	30-Nov-22		
A2050	Removal of Local Service Lines	120	09-Jun-22	30-Nov-22	A1040	A2140
Plains Midstream		271	09-Jun-22	06-Mar-23		
A2060	Installation of New Gas Lines	180	09-Jun-22	24-Feb-23	A1040	A2140, A2290
A2290	Removal of Existing Gas Lines	10	25-Feb-23	06-Mar-23	A2060	A2140
Embankment		812	09-Jun-22	29-Aug-24		
A2080	Zone A1	144	09-Jun-22*	30-Oct-22	A1040	A2130, A2140, A2150
A2090	Zone C1	144	09-Jun-22*	30-Oct-22	A1040	A2120, A3060
A2100	Zone B1	46	01-May-23*	15-Jun-23	A2020	A2110
A2110	Zone B2	144	16-Jun-23*	06-Nov-23	A2100	A2170, A4300
A2120	Zone C2	188	01-May-23*	04-Nov-23	A2090	A3110, A2180, A4300
A2130	Zone A2	188	01-May-23*	04-Nov-23	A2080	A2160, A4300
A2140	Zone D2	188	01-May-23*	04-Nov-23	A2080, A2050, A2060, A2290	A2190
A2150	Zone E2	188	01-May-23*	04-Nov-23	A2080	A2200
A2160	Zone A3	120	01-May-24*	29-Aug-24	A2130	A4340



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Springbank Off-Stream Storage Project

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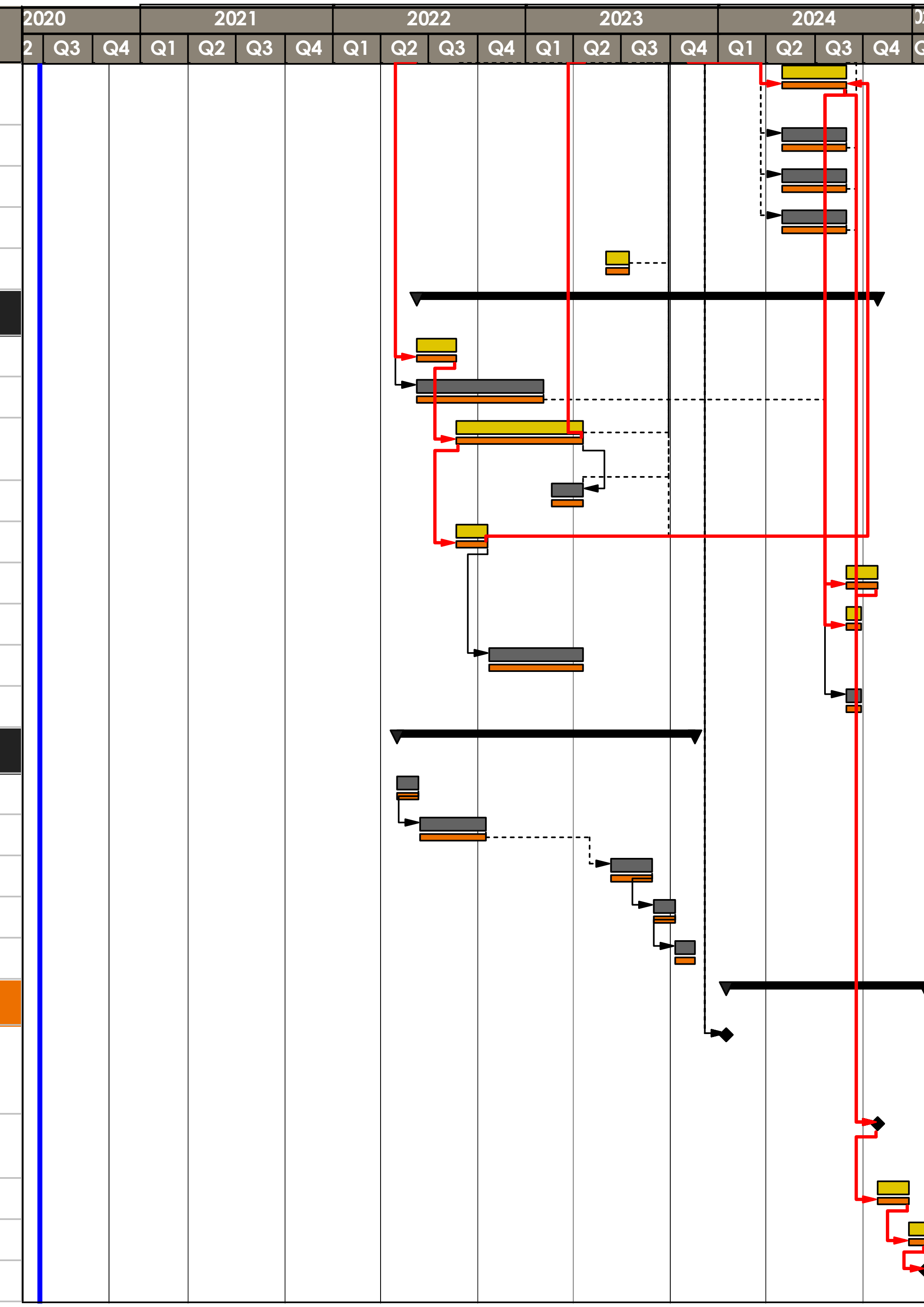
A2170	Zone B3	120	01-May-24*	29-Aug-24	A2110, A2035	A2070, A2280, A2040, A4340
A2180	Zone C3	120	01-May-24*	29-Aug-24	A2120	A4340
A2190	Zone D2	120	01-May-24*	29-Aug-24	A2140	A4340
A2200	Zone E2	120	01-May-24*	29-Aug-24	A2150	A4340
A4290	Temporary Emergency Spillway	45	01-Jun-23*	15-Jul-23		A4300
Low Level Outlet Works		872	09-Jun-22	28-Oct-24		
A2000	Excavation and Foundation Preparation	75	09-Jun-22	22-Aug-22	A1040	A2020
A2010	Equipment Procurement	240	09-Jun-22	03-Feb-23	A1040	A2040
A2020	Conduit	240	23-Aug-22	19-Apr-23	A2000	A2030, A2100, A2035, A4300
A2030	Intake/Terminal Structure	60	19-Feb-23	19-Apr-23	A2020	A4300
A2035	Gate Chamber	60	23-Aug-22	21-Oct-22	A2020	A2170, A2270, A4300
A2040	Gate Commissioning	60	29-Aug-24	28-Oct-24	A2010, A2170	A4340
A2070	Conduit - Joint Closure	30	29-Aug-24	28-Sep-24*	A2170	
A2270	Access Tower and Gate House	180	22-Oct-22	19-Apr-23	A2035	
A2280	Bridge	30	29-Aug-24	28-Sep-24	A2170	
Highway 22 Grade Change		394	02-May-22	17-Nov-23		
A2210	Culvert Installation	29	02-May-22*	10-Jun-22		A2220
A2220	Interchange Grading - Hwy 22 & Springbank Rd West	88	13-Jun-22	17-Oct-22	A2210	A2230
A2230	Hwy 22 Road Construction and Circulation Open	56	12-Jun-23*	29-Aug-23	A2220	A2240
A2240	Interchange Grading - Springbank Road East	28	30-Aug-23	09-Oct-23	A2230	A2250
A2250	Springbank Road Construction and Circulation Open	29	10-Oct-23	17-Nov-23	A2240	
Project Closeout		375	16-Jan-24	26-Jan-25		
A4330	Interim Risk Reduction Milestone	0		16-Jan-24	A4300, A4310, A4140, A4162, A4240, A4270, A4060	
A4340	Substantial Completion	0		28-Oct-24	A2040, A2160, A2180, A2190, A2170, A2200	A4350
A4350	Punch List and Closeout	60	28-Oct-24	27-Dec-24	A4340	A4360
A4360	Demobilization	30	27-Dec-24	26-Jan-25	A4350	A4370
A4370	Project Completion	0		26-Jan-25	A4360	



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