UBC Social Ecological Economic Development Studies (SEEDS) Sustainability Program

Student Research Report

UBC Stormwater Detention: Stormwater Detention Facility at the Centre for Comparative Medicine (CCM) on UBC South Campus

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Executive Summary

Drift Consulting has been engaged by UBC Social Ecological Economic Development Studies (SEEDS) to develop a design for a multi-purpose detention facility which will address the stormwater management issues in the proximity of the UBC Centre for Comparative Medicine (CCM).

The design provides 3000 m^3 of water storage in a trench system alongside new and existing trails in the South Campus Region. The same trench system controls the outflow of flood water into the outfall at Booming Grounds Creek. The flow is controlled by a series of 30 dams positioned at 50 m intervals along the trenches. Furthermore, the retention time and flow velocity of trench system ensures that the water exiting the system has achieved primary water treatment standards.

The trench and trail system are integrated seamlessly with the surrounding environment, as well as the built infrastructure. The new trail connects two existing trials, providing a continuous connection from the Spirit Park trail network to the Botanical Garden regional trails.

This report covers a general overview of the project, the relevant regulations and standards, a detailed description of design components and detailed method for construction. A construction schedule and cost estimate are provided in addition to the design components which consists of drawings, calculations and dimensions.

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1.0 Project Overview

1.1 Overarching Objectives

The Drift Consulting's objective is to provide the client, UBC Social Ecological Economic Development Studies (SEEDS), with a final detailed design for a multi-use stormwater detention facility (the Project) adjacent to the Centre for Comparative Medicine (CCM). All design considerations will be in accordance with the sustainability goals at the University of British Columbia (UBC), including support for the Integrated Stormwater Management Plan (ISMP), Land Use Plan, Water Action Plan and more. Both components of water quantity and quality management will be addressed in proposed designs through water cycle and urban design principles.

Other objectives for the Project are listed below.

The designs will:

- take into consideration social, environmental and economic impacts to the University and users during and after construction;
- function as an integrated transition between infrastructure (neighbouring facilities) and the environment (UBC farm and forest);
- sufficiently manage 100-year storm events by minimizing flooding and relieving the sensitive coastal soils from outfall erosion; and,
- create opportunities for stormwater to be recycled, supporting the University's water independence goals.

1.2 Project Context

UBC is a post-secondary research and teaching institution located at the western tip of the Point Grey peninsula of Vancouver. The UBC Vancouver campus has grown considerably over the last 20 years with an increase of enrolled students of nearly 30% (The University of British Columbia, 2018) and the

corresponding increase of infrastructure. This renders the existing stormwater conveyance infrastructure insufficient. The erection of buildings and construction of other infrastructure has decreased the amount of rain being naturally absorbed into the ground and increased runoff, raising stormwater management issues that need to be addressed. The under-capacity conveyance network has led to issues including surface flooding, high-volume outfalls that cause erosion to the surrounding cliffs in Point Grey and quality of stormwater leaving campus.

The University supports and encourages taking a natural systems approach when developing stormwater management solutions to integrate infrastructure and the environment. Under the UBC ISMP and in accordance with the University Sustainability Initiative, opportunities to implement sustainable water management solutions will be pursued over the next decade.

Currently, the need for a multi-use stormwater detention facility directly southwest to the UBC CCM has been identified. The multi-use water detention facility will consider seasonal changes and be suitable and functional in both rainy and dry conditions. A key aspect in developing the optimal design of the detention facility is to ensure the facility can serve both as a piece of stormwater infrastructure while also improving the sense of community through urban design and landscaping.

1.3 Site Description

The CCM is located at the southern end of the UBC Campus adjacent to TRIUMF, and the National Research Council Institute for Fuel Cell Innovation. This area is prone to flooding during larger storm events. The study area includes sensitive cliffs that are part of Pacific Spirit Park, which are under the jurisdiction of Metro Vancouver. Figure 1 shows the location of the project site in relation to the UBC CCM.

Figure 1: Project Site Location

The site for the further multi-use stormwater retention facility is currently a forest region that is primarily composed of Western Red Cedar, Douglas Fir, and Western Hemlock. The soil stratigraphy can be interpreted from a geotechnical report prepared by GeoPacific Consultants Ltd. in 2006. The first foot of soil is topsoil composed of organic material mixed with silt. Below the first foot is generally glacial till until at least 30 meters. The till is composed of either a silt sand matrix with track gravel clasts and trace cobbles or a silt rich matrix with trace fine sand and gravel clasts.

1.4 Overarching Constraints

The following objectives will be met by our designs will:

- treat stormwater to standard levels of quality prior to being released;
- control the infiltration rate to avoid any seepage into the aquifer; and
- manage the high flow rates caused by the 100-year storm event.

Impacts to the local community are reduced as much as possible. Sustainability is at the forefront of the project design to keep the overall impact on the local wildlife and vegetation at minimum. Additionally, access to the roads beyond the UBC property will need to be acquired.

2.0 Design Summary

The project will consist of two primary aspects. A multi-use trail connecting the existing trails from the botanical gardens to the southern trail network in Pacific Spirit Park, and a set of water conveyance features to control the storm flows in the region. The water conveyance features will consist of a trench network complete with check dams and pipes, as well as a culvert system to convey the water under Wesbrook Mall, and into the two existing outfalls at controlled flow rates.

Figure 2: Project Overview

In Figure 2 above, the trail and trench network are shown in relation to the UBC CCM. Once implemented the project will divert and retain the storm flows from a 100-year storm event, such that the outflows do not damage the outfall locations, nor will the retained water infiltrate into the perched upper aquifer.

2.1 Technical

2.1.1 Design Criteria

The project has been designed to retain between least 2500 m^3 and 3000 m^3 of water, depending on as-built site conditions. Such a retention time will reduce the peak storm flows to the point at which they will not harm the outfall locations. In the same vein, the outflows from the system are limited to a maximum of 1.2 m^3/s. Finally, the residence time for the flows within the system are guaranteed to be less than 48 hours, which in previous studies was shown to be the minimum time required for any appreciable infiltration to begin to occur (Piteau, 2002).

2.1.2 Design Life

The project is expected to last for the foreseeable future, and with adequate maintenance the trails, dams, and pipe systems should last indefinitely, with a replacement schedule unique to each aspect of the project. For the sake of this report, and the interests of UBC, a project design life of 50 years has been assumed for all financial and maintenance activities.

2.2 Economic Impact

The economic costs of this project have been optimized by selecting the option that will meet all of the criteria set out by UBC SEEDS while keeping the cost to a minimum. The construction costs for this project have been minimized by using a standardized trail design with a minimal amount of large and

expensive mechanical systems. Limiting of expensive mechanical systems will reduce the budget for procurement, construction, and maintenance (Muga & Mihelcic, 2008).

For the duration of the project, it is important that some testing is done on the soil to ensure that the assumptions made in the slope stability design are correct. Although conservative calculations were taken for this project, a large difference between the assumed values and actual values may cause more settlement than expected. If the soil has vastly different soil parameters than assumed, significant fiscal impacts may be found for this project (Masten & Davies, 1997).

To keep low costs for this project, UBC crews will be used as much as possible to assist with the construction and do work in place of the contractor wherever possible. It is recommended that the contractor be selected by the lowest priced system. All contracting firm must have performed similar construction projects to this one before (Rosenbaum, 1942). This will ensure that UBC receives quality that matches that standards of all the other projects on UBC lands. Competitive bidding is typical for projects of this size and scope, which will have the benefits of achieving the lowest cost while maintaining a high standard of quality (Rosenbaum, 1942).

Materials were selected for the project based on the economic criteria of:

- cost of procurement;
- relative vicinity to the project in terms of delivery costs;
- cost and ease of the contractor to install;
- relative life cycle of the project; and,
- amount and cost of maintenance.

These material economic criteria are compared against the other performance requirements for the project to determine engineered materials that will minimize the economic impact while meeting all of the design requirements.

The excavated soil from this project will be used as much as possible for the fill areas that are needed to lower fill costs for this project. As determined by the Geopacific Consultants 2006 Geotechnical Report for this site, the soil is free of contaminants and will only need to be recompacted once placed in the designated fill areas. One of the secondary goals for this project is to reuse as much material as possible and to source local materials to meet environmental and budgetary goals. This project has been planned such that the cost cutting measures taken will not hinder the overall benefits of this project to the surrounding community.

2.3 Social Impact

Stakeholders and First Nations were consulted during the development of this project to notify them of the design and construction plans, discuss possible impacts, brainstorm mitigations of these impacts and incorporate their ideas into the design. The results of this stakeholder consultation process are summarized in Table *1* and Table *2* below.

Table 1: Summary of Stakeholder Consultations

Stakeholders	**Type**	**Level of Involvement**	**Consultation Format**	**Concerns**
CCM - Centre for Comparative Medicine	Local institution	Consultation	In-person bilateral meetings or committees	Construction impact - noise, dust Impact to traffic Impact to current operations Schedule estimation
NRC - Energy, Mining and Environment Research Centre	Local institution	Consultation	In-person bilateral meeting	Construction impact - noise, dust Impact to traffic Schedule estimation

Table 1 continued: Summary of Stakeholder Consultations

Stakeholders	**Type**	**Level of Involvement**	**Consultation Format**	**Concerns**
Nordion	Local institution	Consultation	In-person bilateral meetings	Construction impact - noise, dust Impact to traffic Schedule estimation
TRIUMF	Local institution	Consultation	In-person bilateral meetings	Construction impact - noise, dust Impact to traffic Schedule estimation
UBC Farm	University department	Consultation	In-person bilateral meetings or committees	Construction impact - noise, dust Impact to traffic Environmental impact Impact to current operations Schedule estimation
Wesbrook Village Community and Other Local Residents	Community residents	Consultation	An open house, consensus conference to bring concerned citizens together for an opportunity to ask questions and voice concerns	Construction impact - noise, dust Impact to traffic Safety risk Usage functionality Schedule estimation
University Hill Secondary School	Local public school	Consultation	In-person bilateral meetings to identify safety risks	Construction impact - noise, dust Impact to traffic Safety risk Usage functionality Schedule estimation
Metro Vancouver	Regional government authority	Consultation	Advisory board and consequent bilateral meetings	Identification of risks Consequences of the project
Trail Users	Users	Consultation	An open house, consensus conference to bring concerned citizens together for an opportunity to ask questions and voice concerns	Safety risk Usage functionality Schedule estimation Accessibility

Table 2: Summary of First Nations Consultation

First Nations	Type	Level of Involvement	Consultation Format	Concerns
Musqueam First Nation	First Nation	Consultation	An open house, consensus conference to bring concerned citizens together for an opportunity to ask questions and voice concerns	Construction impact - noise, dust Impact to traffic Safety risk Schedule estimation

In addition to public consultation, the final design incorporates principles outlined in the UBC Public Realm Plan, including universal physical access, embracing local vegetation and expressing the identity of the university by showcasing culture and its sustainable initiatives. Furthermore, as the project is situated at one of the main entrances to UBC, it serves as a "gateway" facility that supports positive arrival experiences for students, faculty, staff, residents and visitors.

2.4 Environmental Impact

As climate change advances and threatens more aspects of human civilization, UBC strives to be a world leading university campus in sustainable development by minimizing environmental impacts while continuing to experience economic growth. UBC has developed the UBC Land Use Plan (The University of British Columbia, 2015), the Public Realm Plan (The University of British Columbia, 2009) and the UBC Development and Building Handbook (The University of British Columbia, 2018), which all set forth design goals for projects in maximizing environmental sustainability at UBC. This project will also follow the sustainable guidelines from the three above documents and has an overall goal of being accounted as another example of environmental stewardship for UBC.

This water retention system will aim to protect and preserve the existing environment. Only necessary clearing of trees will be performed in hopes of retaining the majority of the forested area. Retaining the forested area will also aid in limiting the amount of surface runoff by storing water in the vegetation and soils.

Concrete for this project will be precast and delivered to reduce the formwork and construction materials needed on site. This will improve the safety on site as well as reduce the overall carbon emissions of this project (Dong, Jaillon, Peggy, & Poon, 2015).

The Ministry of Forests, Lands, and Natural Resource Operations require the following to be prepared prior to commencing construction projects in densely forested areas (Ministry of Forests, Lands, and Resource Operations, 2013):

- Site plan identifying the scope of the project;
- approval permits;
- development plan with site documents and design;
- issued for construction documents and specifications for the project; and
- environmental safety plan

3.0 Regulations and Standards

The following section provides an overview of regulations, standards and guidelines used in the technical design. For further explanations on the application of each standard, refer to the design components section.

Table 3: Design Standards, Regulations and Guidelines

Reference Standard/Regulation	**Usage**
City of Coquitlam Supplementary Specifications Master Municipal Construction Documents	Dam Pipe Sizing
Surrey Design Criteria	Dam Pipe Sizing
Plastic Pipe Institute Technical Design Guidelines	Dam Pipe Sizing
CSA-A23.3-14 - Design of Concrete Structures.	Concrete Dam Design
CSA-A23.4-16 - Precast Concrete - Materials and Construction.	Concrete Dam Design
CPCI Design Manual 5th Edition	Concrete Dam Design
Riprap Design and Construction Guide, 2000 (BC Ministry of Environment, Lands, and Parks)	Riprap Design
Fisheries Act	Discharge Quality
Canadian Environmental Protection Act	Discharge Quality
Water Sustainability Act	Discharge Quality
Environmental Management Act	Discharge Quality
Metro Vancouver Best Management Practices Guide for Stormwater	Discharge Quality
CSA Standard G30.18	Rebar Material
CHBDC S6-14	Rebar Coating
The Handbook of Steel Drainage and Highway Constructions	Culvert Design
MoT Supplement to TAC Geometric Design Guide	Culvert Design
MoT Section 303 Culvert Design	Culvert Design
MoT Section 201 Roadway and Drainage Excavation	Culvert Design
2016 MoT Standard Specifications for Highway Construction Volume 1	Culvert Design

Table 3 continued: Design Standards, Regulations and Guidelines

Reference Standard/Regulation	Usage
City of Coquitlam Trail Development Guidelines and Standards	Trail Design

4.0 Design Components

4.1 Hydraulic Components

4.1.1 Flow Mechanics

The trenches adjacent to the trail are designed as triangular trenches with a 4 m width and 1 m depth to satisfy slope stability requirements. To achieve storage the storage requirements, as shown in Appendix A, the roughness coefficient and the hydraulic radius will be controlled. Appendix B shows the flow depth and flow rate by variation of the hydraulic radius. This will be achieved via the seeding of natural wetland plants. Furthermore, a set of willow trees and accompanying brambles will be planted along the longitudinal axis of the channel. Additionally, a system of base flow check dams will be used to control the flow in the channel.

Trench Lining and Manning's Values

The following table of wetland plants, as described by Cronk and Fennessy (Cronk & Fennessy, 2001), provides a list of recommended wetland plants for implementation in regions within the Fraser Valley. The listed plants provide efficient uptake of nutrients and metals, while being well suited for use in a stormwater channel. The plants are expected to increase the slope stability of the channel, reduce flow rates in the channel, and increase residence time and sediment settling rates. (North Carolina State University, 2002)

Table 4: Plants for a Stormwater Channel

Latin Name	**Common Name**
Typha species	Cattails
Scripus	Bulrushes
Iris pseudacorus	Iris
Alisma species	Plantain
Phargmites australes	Common reed
Cyperus species	Sedges
Elecharis species	Sedges
Glyceria maxima	Giant mana grass

Furthermore, the UBC campus plan encourages the use of the several plant species in any new landscaping in the South or Forest Regions of campus. A table of additional potential plant species can be found in Appendix C. For this project we recommend the use of sword ferns, salals and flowering currants. These species are particularly water retentive and resistant to flood events.

Due to the predicted plant growth within the channels, it is expected that the channels will be accurately defined by three distinct phases, depending on the severity of the flood event. As the flood waters rise, the effects of the planted vegetation will be reduced. This is shown by the projected manning's values below.

Table 5: Manning's n Values

Flood Event	**Flow Depth**	**Channel Classification**	**Manning's n Value**
2-Year	15 cm	Excavated earth, winding channel, with dense weeds as high as the flow depth	0.035
10-Year	40 cm	Excavated earth, winding channel, with some weeds	0.030
100-Year	95 cm	Excavated Earth, winding channel, no vegetation	0.023

The manning's values in the table above should be confirmed experimentally prior to the final design being constructed. While these values are consistent with the literature, it is common for the roughness coefficient (n) to fluctuate by upwards of 200% depending on local conditions. As such it is imperative that in situ experiments are conducted to confirm the precise manning's value.

Flow Regimes

The flow between dams can be divided into 4 regions. Starting at the downstream side of a dam there is a supercritical region (energy-controlled depth), this region should be reduced as much as possible. The supercritical region will be followed by a hydraulic jump, this next subcritical region (momentum-controlled depth) is the section that is preferred for the design. Following the subcritical region, the manning's value becomes important (friction-controlled depth). This region is avoided as much as possible as the water depth in this region is quite low. Finally, as the flow approaches the next dam there is a stagnation zone where the velocity at the surface approaches zero (energy-controlled depth).

For the calculations demonstrating the flow patterns and fluid depths, see Appendix E. The results of those calculations are shown below. Each unique region is governed by a different flow parameter, and each region transitions into the next region due to a specific event or obstacle.

Table 6: Flow Characteristics

	Region 1	**Region 2**	**Region 3**	**Region 4**
Flow Regime	Energy	Momentum	Friction	Energy
Governing Equation	Bernoulli	Froude Number (Fr=2.115)	Manning's Equation (n=0.023)	Bernoulli
Flow Depth (m)	0.29	0.73	0.49	0.94
Cause of termination	Baffle Blocks	Roughness	Flow Stagnation	Dam

Sensitivity Analysis

The storage capacity of the system is extremely sensitive to minor change in the hydraulic radius. Determining the hydraulic radius prior to in-situ tests will be extremely uncertain, as it is dependent on the effects of the planted vegetation. As such a sensitivity analysis was conducted to determine the range of storage capacities provided by differing conditions. This simulation developed in Matlab, shown in Figure 4, assumes that region 4 dominates the channel with some variable effect from region 3, shown in Figure 3. This assumption is based on the spacing between dams being short in length, 50 m between dams.

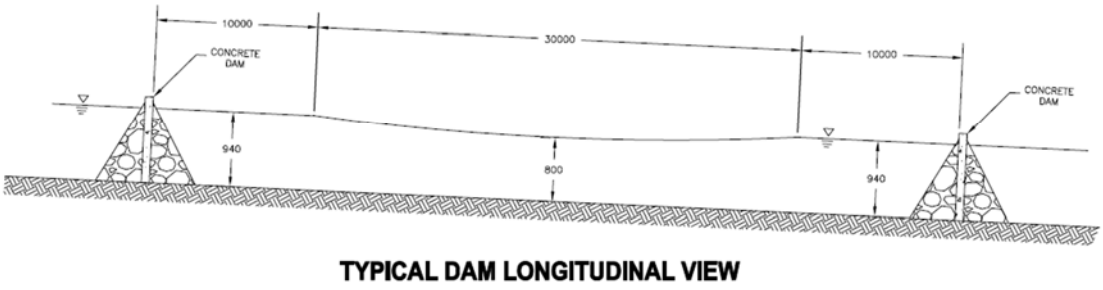


Figure 3: Dam Longitudinal View

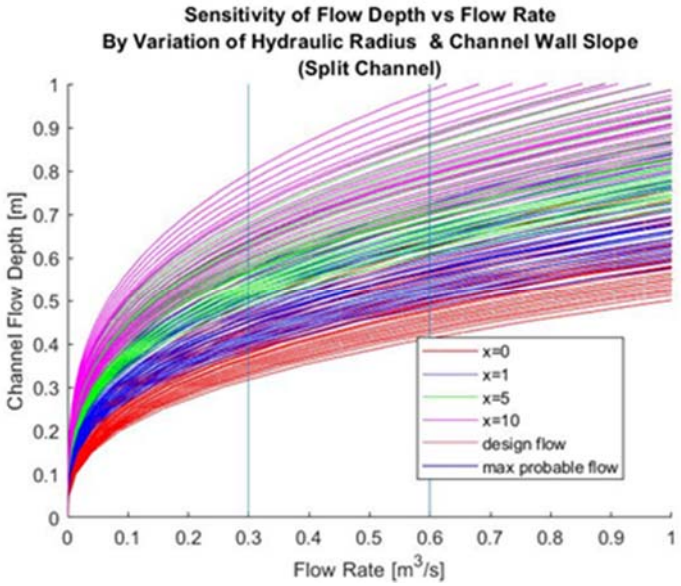


Figure 4: Sensitivity Analysis Results

In Figure 4 above, the red region represents a very conservative model, while the purple region represents the upper bounds of all feasible solutions. The model shows the field of results for a range of hydraulic radii from 0 to 10m, and for channel wall slopes ranging from 2:1 to 2.5:1. The storage capacities determined from this analysis are show in Table 7. These values have been corrected to the nearest hundred for ease of presentation. Results in green are capacities that fall within the design parameters.

Table 7: System Storage Capacity Results from Sensitivity Analysis

System Storage Capacity (Corrected)				
K\pw	0	1	5	10
2:1	2300	2400	2500	2500
2.1:1	2400	2500	2600	2600
2.2:1	2500	2600	2700	2700
2.3:1	2600	2700	2800	2900
2.4:1	2800	2800	2900	3000
2.5:1	2900	2900	3000	3100

Further experimentation should be conducted prior to the construction of the dams to determine the exact spacing required. Scale models in a sluice gate channel should be conducted to confirm the design spacing, and in-situ tests should be conducted prior to construction of the dams.

4.1.2 Water Treatment

The ISMP identifies sediments from road wear and construction sites, metals and metal dust from vehicle traffic, and hydrocarbons from asphalt and vehicles to be major sources of stormwater contamination. According to ISMP, some stormwater contaminant testing was done and results confirmed presence of sediment and the following metals: lead, zinc, iron, manganese, and various others in trace amounts. These tests did not detect any hydrocarbons, possibly due to low vehicle traffic on campus and lack of fresh paving. Continuous monitoring of certain contaminants in the stormwater might be required once the Water Act draft is formalized.

The banks of the trail can be assumed to act as grassed channels. In general, contaminant removal in grass channels is a function of length (Gibb, et al., 1999). The Metro Vancouver's Best Management Practices Guide For Stormwater (BMP) provides the following average contaminant removal efficiency of a swale in the Puget Sound area over 6 storms as shown in Table *8*.

Table 8: Average Contaminant Removal Efficiency of a Swale

Swale Length	30 m		60 m	
Parameter	mean	range	mean	range
Total Suspended Solids	60%	0%-93%	83%	69%-97%
Turbidity	65%	26%-80%	65%	42%-91%
Oil & Grease/TPH	49%	33%-65%	75%	64%-92%
Total Zinc	16%	<0%-86%	63%	38%-84%
Total Lead	15%	<0%-95%	67%	50%-90%
Total Phosphorus	45%	19%-74%	29%	<0%-58%
Nitrate+Nitrate Nitrogen	negative	-	negative	-
Fecal Coliforms	negative	-	negative	-

Claytor and Schueler (1996) also list the performance data of a 200-foot-long grass channel in the Seattle metropolitan area in Table *9*.

Table 9: Pollutant Removal Performance of a Grass Channel

Pollutant	100 Foot Biofilter	200 Foot Biofilter
Suspended Sediment	60%	83%
TPH (Hydrocarbons)	49%	75%
Total Zinc	16%	63%
Dissolved Zinc	negative	30%
Total Lead	15%	67%
Total Copper	2%	46%
Total Phosphorus	45%	29%
Bioavailable P	72%	40%
Nitrate-N	negative	negative
Bacteria	negative	negative

The performance data suggests that grass channels are suitable for managing most of the stormwater contaminants at UBC, particularly suspended sediments and most of the detected metals. Claytor and Schueler (1996) further note that "few best management practices exhibit such a great variability in pollutant removal performance as open grass channels."

The BMP imposes some design constraints on the grass channels to achieve desirable water treatment based on the water quality storm (WQS), which is defined as the rainfall volume associated with recurrence interval below 2 years in the rainfall frequency spectrum (Claytor & Schueler, 1996). These design requirements are greatly exceeded by our design and are listed below (Gibb, et al., 1999).

- Minimum residence time for the WQS should be 5 minutes (minimum of 9 minutes is recommended).

- Maximum velocity should be 0.3 m/s for the WQS and non-erosive for larger storms (should be limited to 1 m/s in the absence of relevant information).
- Maximum flow depth should be 125 mm for the WQS.

4.1.3 Dam Pipes

Design Criteria, Design Life and Loadings

The dams, as discussed in the materials section below, will be constructed of concrete, span the entirety of the trench, and stand 1.0 m tall. A pipe installed at the base of the dam near the vertex of the triangle will facilitate all flow during events lower than 100-year intensity. As a result, the pipe is designed to withstand a 0.5 m^3/s flow capacity, since there will be a trench adjacent to one side of the existing path for the first 800 m, and both sides for the last 500 m, allowing for a maximum flow of 1.0 m^3/s at the convergence with the existing stormwater infrastructure east of Wesbrook Mall. To account for surges in flow that will enter each section of the trail, two relief notches of 0.2 m width and 0.1 m height will be implemented in the dam structures. In addition, there is 0.05 m of clearance between the top of the dam and the trail surface. When the trail is used at its full capacity and filled with 3000 m^3 of water, all water is considered evenly distributed throughout the longitudinal span of the trail, and only the pipes are used to facilitate flow. The final pair of dams prior to confluence with the existing infrastructure will only have the pipe and no flow relief accommodations to restrict flow to 1.0 m^3/s. Figure 5 below shows the typical configuration of all dams and pipes.

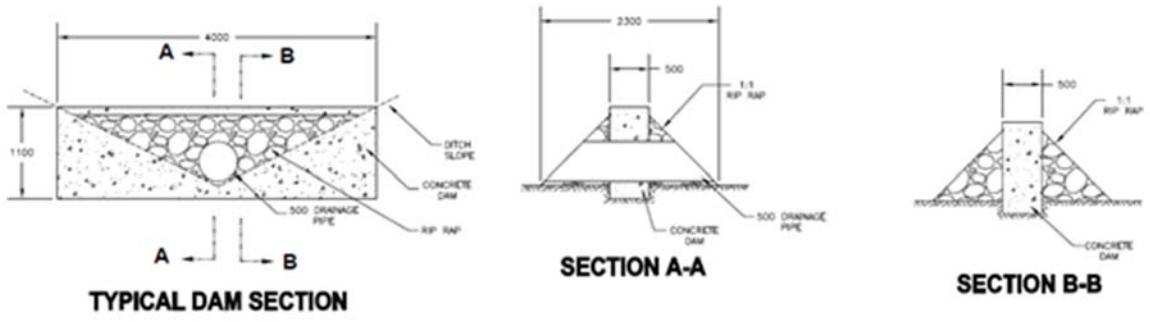


Figure 5: Typical Dam and Pipe Cross Section

Technical Considerations for Pipe Design

To span the thickness of the dam and riprap, the smooth high-density polyethylene (HDPE) pipes are 2.5 m in length. Diameter sizing was calculated using both the orifice flow and Reynolds friction methods to establish a single design diameter. The orifice flow method applies a single flow coefficient multiplier (C_o = 0.5096 for re-entrant circular orifices) to Bernoulli's energy conservation equation. The Reynolds friction method is more elaborate, considering the roughness coefficient of the pipe (0.0015 mm), frictional losses, minor losses, and Reynold's number to iterate to a solution for diameter. Refer to Appendix E for sample calculations with both methods. The diameters designed for both methods are controlled by the slope of the section immediately upstream of the design dam as the slope controls the energy head the pipe must accommodate. Diameter calculations were completed based on considering the water level immediately after the upstream dam to be at the maximum 0.95 m for the most conservative scenario as shown in Figure 6. HDPE is selected as the pipe material due to its structural abilities and common application in municipal supply and sewer lines. To provide sufficient strength, a dimension ratio of 35 (SDR 35) is selected, providing the minimum pipe thickness.

Flow Depth Ratio Considerations

In accordance to findings by the Plastic Pipe Institute (PPI), maximum flow is exhibited in HDPE pipes at 93% capacity due to reduced friction. The result is an 8% increase in flow of 0.54 m^3/s per channel and a

total flow of 1.08 m^3/s into the existing infrastructure. Because this is still within the 100-year limit of 1.2 m^3/s, design of pipes using 0.5 m^3/s is deemed acceptable.

Results of Pipe Design

The slope of each longitudinal section was determined using publicly available SRTM topographic data, accurate to one arc-second resolution. Average slope between two adjacent dams were taken for pipe sizing calculations. Table *10* below shows the dam sections, slopes and calculated pipe diameters. Refer to Figure 1 for a plan view of the trail and layout of the dams with corresponding dam indices.

Table 10: Design Pipe Diameters

Start Station (m)	End Station (m)	Dam Index	Slope	Outer Diameter (m)	Thickness (mm) SDR 35	Flow Velocity (m/s)
0	*50*	1	0%	**0.55**	16	2.4
50	*100*	2	-2%	**0.50**	14	2.9
100	*150*	3	-1%	**0.55**	16	2.4
150	*200*	4	-2%	**0.50**	14	2.9
200	*250*	5	-5%	**0.40**	11	4.5
250	*300*	6	-3%	**0.45**	13	3.5
300	*350*	7	-1%	**0.55**	16	2.4
350	*400*	8	-2%	**0.50**	14	2.9
400	*450*	9	-2%	**0.50**	14	2.9
450	*500*	10	-2%	**0.50**	14	2.9
500	*550*	11	-2%	**0.50**	14	2.9
550	*600*	12	-2%	**0.50**	14	2.9
600	*650*	13	-1%	**0.55**	16	2.4
650	*700*	14	-1%	**0.55**	16	2.4
700	*750*	15	-1%	**0.55**	16	2.4
750	*800*	16	-1%	**0.55**	16	2.4
800	*850*	17, 18	-3%	**0.45**	13	3.5
850	*900*	19, 20	-4%	**0.40**	11	4.5
900	*950*	21, 22	-3%	**0.45**	13	3.5
950	*1000*	23, 24	-2%	**0.50**	14	2.9
1000	*1050*	25, 26	-2%	**0.50**	14	2.9
1050	*1090*	27, 28	-5%	**0.40**	11	4.5

Table 10 continued: Design Pipe Diameters

Start Station (m)	End Station (m)	Dam Index	Slope	Outer Diameter (m)	Thickness (mm) SDR 35	Flow Velocity (m/s)
1090	*1150*	-	0%	**-**	-	-
1150	*1200*	-	0%	**-**	-	-
1200	*1250*	-	0%	**-**	-	-
1250	*1300*	29, 30	-1%	**0.45**	13	3.5

At station 850 m, the trail enters the region where trenches exist adjacent to both sides of the trail. Stations 1150 m, 1200 m and 1250 m have a design slope of 0% and no dams will be required to control flow. At dams 27 and 28, the trenches reach Wesbrook Mall. Water will be diverted through the existing culvert. Dams 29 and 30 are the final barriers before entry into the existing infrastructure and are constructed to the full 1 m height with only the base HDPE pipe to facilitate a final flow rate of 0.5 m^3/s per outlet back into the existing trenches. Design details pertaining to the culvert passage under Wesbrook Mall and the connection system to existing stormwater infrastructure is available in the subsequent sections.

Sizing of the check dam pipes was performed using Darcy Weisbach equations in a spreadsheet application. Standards and guidelines used include the Plastic Pipe Institutes technical design guidelines. In addition, the City of Coquitlam Supplementary Specifications (City of Coquitlam, 2016), Surrey Design Criteria (City of Surrey, 2016) and Plastic Pipe Institute Technical Design Guidelines (Plastic Pipe Institute, 2009) were referenced to verify compliance of pipe sizing.

The various HDPE pipe lengths will be ordered according to the sizing results from the manufacturer at the design SDR. Pipes have been rounded to the nearest 500 mm in diameter during design calculations for simplicity in manufacturing and availability. Because the check dams will be cast off-site, the ordered pipes will be delivered to the casting facility such that concrete can be poured with the rebar and pipe already in form corresponding to construction specifications shown in Appendix K. As a result, fully

prefabricated check dams with pipe will simply be put in place during trench construction. The dam and pipe inventory required is as follows:

- 7 dams with 2.5 m of 550 mm diameter pipe
- 11 dams with 2.5 m of 500 mm diameter pipe
- 7 dams with 2.5 m of 450 mm diameter pipe
- 5 dams with 2.5 m of 400 mm diameter pipe

4.1.4 Dam Spacing

The spacing of the dams is controlled by the flow regime between them. In order to maximize the storage volume in the event of a flood the spacing must be such that the water level flows at the subcritical level, not the supercritical level. Additionally, the spacing must be short enough that the friction regime does not negatively impact the flow levels. A section of the trail is defined by the longitudinal length spanned by two adjacent dams.

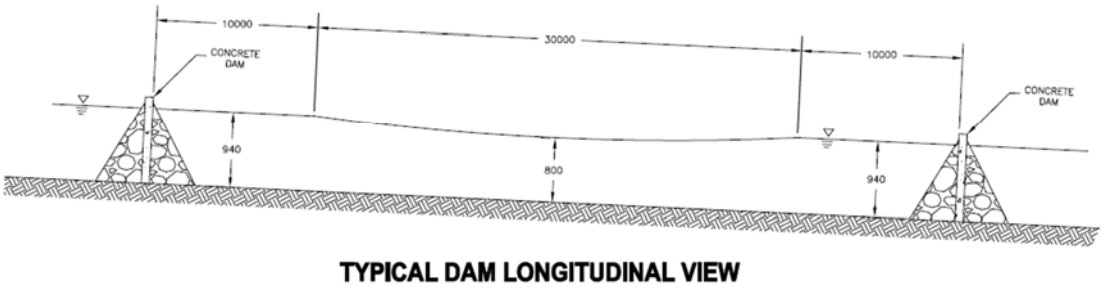


Figure 6: Dam Longitudinal View

The spacing used in this design is 50 m as shown in the longitudinal view of the trail in Figure 6 above. This spacing is expected to achieve the necessary flow regimes, as discussed in the preceding sections. By spacing the dams at 50 m intervals the dams do not negatively impact the total storage capacity of the system by virtue of their quantity, and the total cost of installation is reduced (by minimizing the total required number of dams).

4.1.5 Culverts

Two culverts will convey the flows in each trench on the sides of the trail across Wesbrook Mall. HDPE has been selected as the material for the culvert because of its high versatility, durability and elastic ability to resist seismic forces. The pipes have been sized to be 600 mm in diameter to meet flow and capacity requirements. The slope of the culvert across its entire length will be 5%. The length of the culverts will be 30 m. Riprap will be placed at the tail end of the culvert to prevent soil sloughing. Similar to the pipes for the check dams, a dimension ratio of 35 (SDR 35) has been selected. To provide additional structural support, the final wall thickness of the culverts will be 27 mm.

The trench will need to be excavated to a minimum depth of 1400 mm to ensure at least 300 mm of embedment material be placed beneath the culvert as a foundation, as well as a height of cover of 500 mm. The foundation for the sides of the culvert will require 300 mm. During excavation, if it is found that the material beneath the excavated trench will cause uneven settlement along the length of the culvert, the trench will need be to dug below grade and backfilled with gravel and compacted to ensure a uniform and firm foundation. Compaction of the below grade material will need to be completed to 100% Standard Proctor.

Embedment material will be free draining, well graded granular material. Culvert gravel will be used for bedding. Granular backfill will be used to regrade the trench to the existing elevation of the road. It is crucial that backfilling is not performed when air temperatures are below zero degree Celsius.

The structural design life of a culvert is 50 years. Testing of water hardness, pH and resistivity values need to be performed to confirm the environmental conditions. Abnormal levels of these parameters will

reduce the design life of the culverts. The larger wall thickness of the culvert will decrease the pipe's susceptibility to environmental factors and increase the pipe's durability.

4.2 Geotechnical

4.2.1 Slope Stability

As per Geopacific Consultants Ltd (Geopacific Consultants Ltd., 2006), approximately the first foot below existing grade is topsoil composed of organic material mixed with silt material. Below the topsoil there is glacial till composed of a silty sand matrix with trace gravels and cobbles. The Geotechnical Design Manual produced by the Washington State Department of Transportation (Washington State Department of Transportation, 2013) states that the glacial till in this region are generally normally consolidated and have a minimum effective friction angle of 40 degrees and a minimum cohesion of 4.79 kPa.

A slope stability analysis using the Limit Equilibrium Method was conducted in Slope/W. Two different cases were analyzed, case one had a 2m horizontal by 1m vertical slope. Case two had a 4m horizontal by 2m vertical slope. Three different phreatic surfaces were analyzed for each case. For the first scenario, the phreatic surface was at the top of the trench simulating the trench at maximum capacity. For the second scenario, the phreatic surface is located at ground level simulating saturated ground conditions with no flowing water. For the third scenario, the phreatic surface is located significantly below ground level simulating summer conditions. The results from all the analyzed scenarios can be seen in Appendix F. The worst case scenario resulted in a factor of safety of 1.977 and was the case 2 with a phreatic surface at the ground surface.

In a study completed by the US Army Corp of Engineers (US Army Corp of Engineers, 2013) on vegetation impact on soil strength, they found that having shrubs of various types increased the cohesion

forces of the root-soil composite by 29.4% to 394.6%, which will also increase the factor of safety. In addition, having small shrubs planted within the drainage channels will also increase the Manning's Coefficient of the channel and allow us to increase the spacing between the dam structures.

4.3 Dams

4.3.1 Macro Dam Design

The dams were designed as concrete gravity dams. The dams will be supported by a riprap shell, and by means of ground embedment, as shown in Figure 7. The dams are underlain by pipe system to ensure that all standing water is conveyed through the preferred flow path, and seepage around the dam is minimized. Overturning and slip failures have been accounted for, and the dams have an overall factor of safety of 5. Sample calculations can be found in Appendix E.

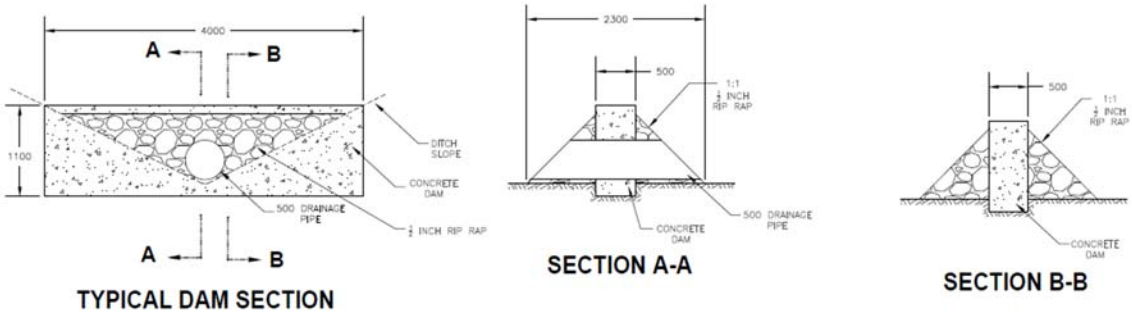


Figure 7: Concrete Gravity Dam Sections

The large factor of safety is a result of low overall forces, combined with large masses of riprap surrounding the dams. The scale of the riprap required is a function of BC rip-rap design guidelines (BC Ministry of Lands, Environment and Parks, 2000) and BC accessibility guidelines for slopes on a multi-use trail (Office of Housing and Construction Standards, 2007). The slope of the rip-rap creates a safer space than the vertical dam faces alone.

4.3.2 Concrete Dam Reinforcement

The concrete dams were modeled as vertical walls made up of vertical cantilever beams of unit width as show in Figure 8. The steel reinforcement was selected according to Clause 14 of CSA A23.3. The required amount and configuration of the vertical and horizontal distributed reinforcements are listed in *Table 11* below.

Table 11: Required Amount and Configuration of Distributed Steel Reinforcement

Distributed Reinforcement Type	**Area (mm^2/m)**	**Number of Layers**	**Bar Size**	**Spacing (mm)**	**Cover (mm)**
Vertical	750	2	15M	250	75
Horizontal	1000	2	15M	200	75

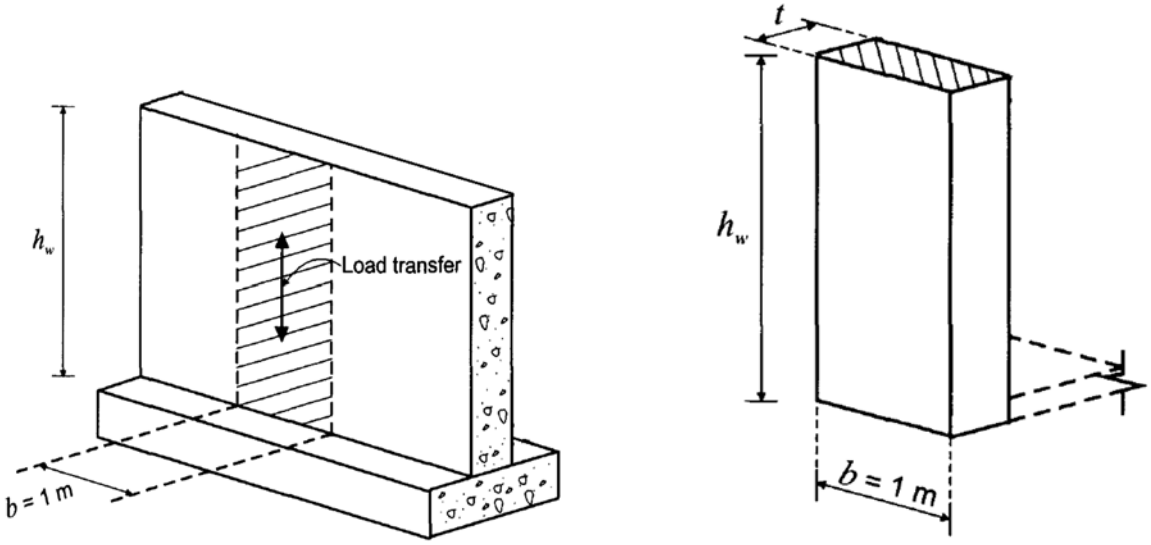


Figure 8: Illustration of the Modelling Assumptions

In addition to the distributed reinforcement, Clause 14 stipulates provision of at least 2-15M of concentrated vertical reinforcement at each end of the walls, as shown in Figure 9.

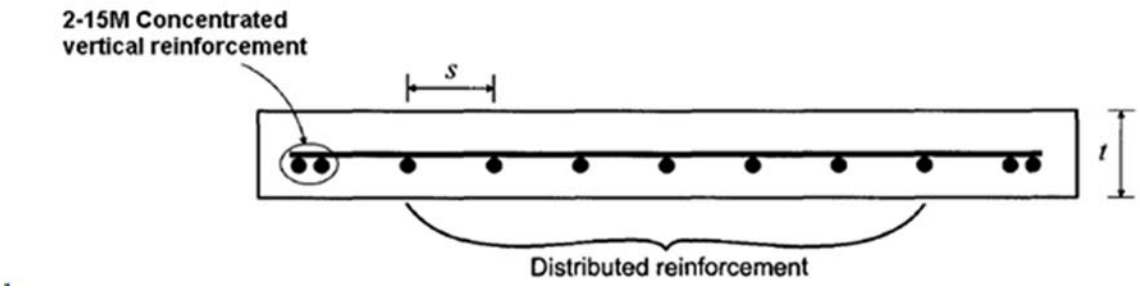


Figure 9: Location of Concentrated Vertical Reinforcement

To prevent diagonal cracking around the embedded pipes, 2-15M bars should be provided around the openings extending at least 500 mm beyond the edges of the openings. An exception is made for the bottom edge to meet the cover requirements Figure 10.

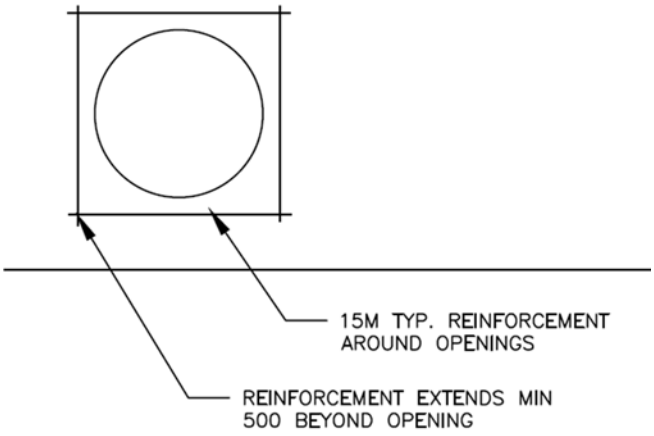


Figure 10: Rebar Around the Embedded Pipes

The selected reinforcement was found to provide adequate strength for flexure and shear demands imposed on the wall; refer to the sample calculations in Appendix I for details.

4.4 Connection to the Existing Infrastructure

The proposed trail system will connect to the existing infrastructure via link between the proposed trenches and the existing trench running along the SW Marine Drive. The link will be accomplished by

connecting the two dam pipes at the last station to a perpendicular pipe running under the proposed trail and the southern embankment Figure 11. This connection will allow the water from the proposed system to be diverted into the existing trench and conveyed downstream to the Booming Ground Creek Outfall.

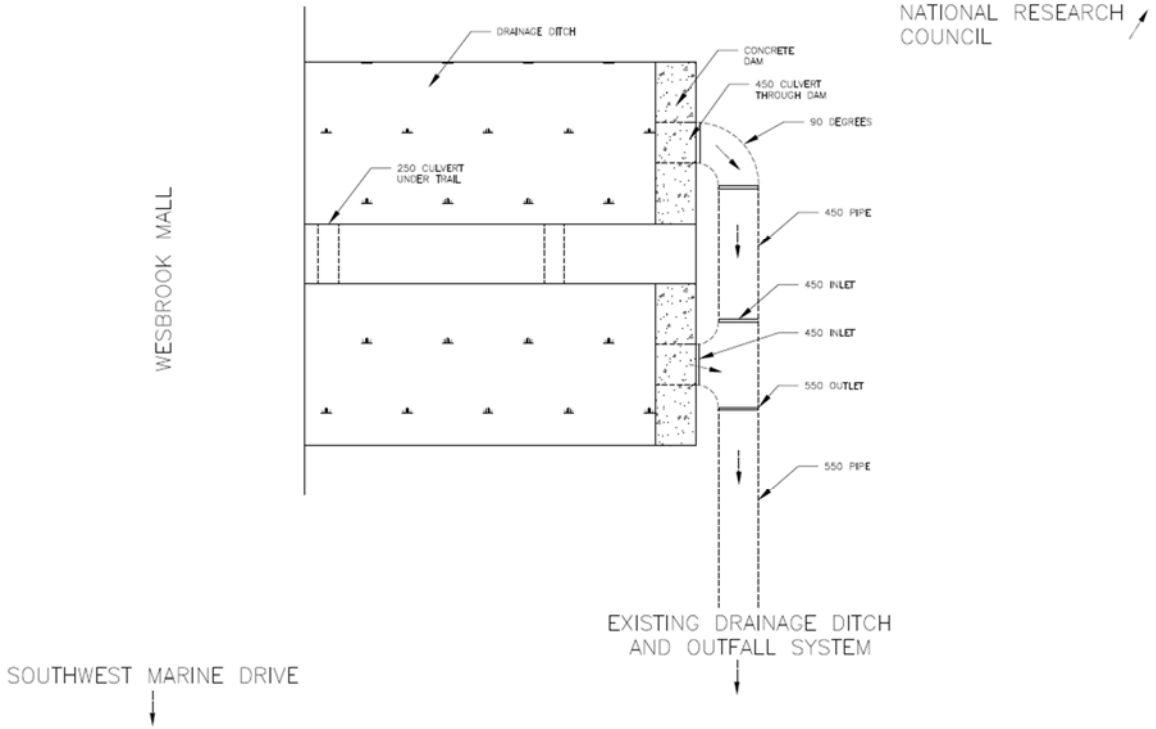


Figure 11: Layout of the Connection to the Existing Infrastructure

The pipe material (HDPE) and SDR (35) were selected to be the same as for the dam pipes. The pipes were sized using the empirically derived formulas by Swamee and Jaine for pipe flow (Potter, Wiggert, Ramadan, & Shih, 2012, p. 309), assuming the water level at the existing trench is near its capacity (0.8 m) and the flow rates of 0.5 m^3/s and 1 m^3/s before and after the lower junction, respectively. The pipe diameters were selected to be 450 mm and 550 mm upstream and downstream of the lower junction, respectively. The detailed calculations are presented in Appendix H.

4.5 Materials

Table *12* below summarizes the materials used for various construction items in our project. Relevant critical parameters are also mentioned.

Table 12: Material Selection

Items	Material / Description	Critical parameters
Pipe Culvert	High Density Polyetheylene (HDPE)	SDR 35
Pipe (Dams)	High Density Polyetheylene (HDPE)	SDR 35
Concrete	Normal Strength Concrete	50MPa, Type II
Rebar	Deformed bars (Brzev and Pao, 2011)	Grade 400R
Riprap	Clean riprap with D_{30} = ½", D_{15}> ¼"	N/A

5.0 Software Models

5.1 EPA SWMM 5

EPA SWMM 5 was used to confirm the required storage volume required for a 100-year storm. During the peak hour of the storm, the existing stormwater infrastructure at the Centre for Comparative Medicine is incapable of handling the large volumes of water. The existing stormwater infrastructure can handle a maximum of 1.2 m^3/s as stated in the UBC ISMP. The flows above the pipe capacity were calculated to determine the volume that will not be delivered through the stormwater infrastructure, but instead be surface runoff. The minimum total required storage volume was found to be 2500 m^3. A key pipe outside the Centre for Comparative Medicine (Link T6D-S27Y) was analyzed using EPA SWMM 5. The outputs from the software and calculations can be found in Appendix J.

5.2 Slope/W

Slope/W was used in the slope stability analysis using the Limit Equilibrium Method. The results can be found in Appendix F and the methodology used is discussed in Section 4.2.1.

5.3 Matlab

Matlab was used in the development of the sensitivity analysis. The code can be found in Appendix G. Manning's equation was used to model minimum flow levels, and Bernoulli's equation was used to model maximum flow levels. The transition zone between the two was approximated as a smooth line, and the resulting storage area generated.

6.0 Construction

6.1 Specifications and Standards

The following is an overview of the required specifications and standards that the project should adhere to. In the case of any discrepancies, the more stringent standards apply. Should any omissions be noted the engineer of record should be advised, and work should continue using the industry standard approach. The contractor shall take full responsibility for the ultimate serviceability of the project, and for the completed project to meet the specified design standards and intentions.

6.1.1 Dam Sections

The dam sections are constructed of precast concrete as per CSA A23.4-16 standards for precast concrete sections. The dams must be cast to match the schematics shown in the construction drawings in Figure 7. All testing shall be conducted as per A23.1-14 and A23.2-14. In the case of any discrepancies between the drawings and the standards, the standards take precedence. The full construction process shall follow the

procedures and standards as recommended by the CPCI Design Manual, 5th Edition Precast and Prestressed Concrete, or an equivalent alternative procedure.

The rebars should be prepared according to CSA Standard G30.18 (Brzev and Pao, 2011). The reinforcing bars must be of grade R (regular steel grade) and have a minimum yield strength of 400 MPa (Brzev and Pao, 2011). The ends of the strands shall be painted with organic zinc rich paint to provide corrosion resistance (CHBDC S6-14).

6.1.2 Check Dam Pipes

Pipes will be added to the check dams as part of the prefabrication process off site in accordance with the manufacturer's standards and the rebar configuration presented in Section 4.3.2. An inventory of required pipe lengths is provided in Section 4.1.3. All pipes will follow a SDR 35 rating in accordance to guidelines provided by the Plastic Pipe Institute (Plastic Pipe Institute, 2009).

6.1.3 Riprap Placement

The riprap surrounding the dam sections shall be placed as per the schematics provided, shown in Figure 7. The riprap shall be graded, placed and tested according to the Riprap Design and Construction Guide. In the event of a conflict between the construction drawings and the design guide, the more stringent criteria shall be used.

6.1.4 Trail Grading and Slopes

The trail will be constructed as per the Trail Construction Guidelines of the City of Coquitlam's Trail Development Guidelines and Standards (City of Coquitlam, 2015). No frozen trail material is to be placed, and all trail construction must occur when the air temperature is above 0 degrees Celsius. To

accommodate operations and maintenance vehicles, the base material must be compacted to 98% Modified Proctor Density.

When drainage under the trail is required, a minimum of 150 mm diameter pipe must be used. In addition, the pipe must be covered by 250 mm of trail material to protect the pipe from operations and maintenance vehicles.

6.1.5 Culvert Installation

The culvert that crosses Wesbrook Mall along Marine drive shall be constructed as per the guidelines outlined in the British Columbia Ministry of Transportation Section 303 Culvert Specifications. High Density Polyethylene (HDPE) Pipe shall be supplied in accordance with SS 318.03. It is important to note that backfilling is not allowed when the air temperature is below zero degrees Celsius. Backfill material will also not be permitted to be placed directly on frozen substrate. Embedment material shall be compacted to a minimum of 95% of the laboratory density as determined in accordance with ASTM D 698. Excavation shall conform to SS 407 Foundation Excavation.

6.2 Schedule

Construction will commence June 1, 2019 and be complete August 28, 2019 in time for the start of the 2019-2020 school year. To effectively utilize all resources, construction will be separated into three phases consisting of the left arm (west of Wesbrook Mall), the right arm (east of Wesbrook Mall) and the passthrough beneath Wesbrook Mall. The complete schedule for construction activities is available in Appendix D. The critical path is highlighted with red events. Notably, construction activities for the right arm have significant slack. Delays in the early construction of the left arm and the roadwork over Wesbrook Mall will lead to delays in project completion. Key construction milestones are summarized below in Table 13.

Table 13: Construction Milestones

Construction Milestone	Completion Date
Right Arm Trench and Trail	July 26, 2019
Left Arm Trench and Trail	August 27, 2019
Connection to Existing Stormwater Infrastructure	August 28, 2019
Wesbrook Mall Culvert Passthrough	August 29, 2019

Where possible, tasks run concurrently, subject to mandatory predecessors and resource allocation.

Project crews include:

- Two Earthworks crews with 3 workers and an excavator each,
- Site layout and survey crew with 2 workers,
- Dam placement crew with 4 workers,
- Planter,
- Roadwork crew with 4 workers,
- Concrete crew with 3 workers,
- Pipework Crew with 3 workers and,
- Compactor.

7.0 Cost Estimate

The cost estimate for this project was prepared by considering the costs for typical trail construction, along with material costs and various local labour costs. Escalation and financing costs were not accounted for in the costs of the project. As Drift has already been selected as the design consultant, further design costs were not included. Unforeseen circumstances and contractor change orders have not been directly accounted for in this cost estimate; however, there is a contingency in the budget that will account for these potential costs. The costs associated with constructing the trail are considered separately below Table *14* and are based on the trail cost analysis guidelines by Ped & Pedal (2010).

Table 14: Trail Construction Cost Estimate

Description	Unit Cost	Amount	Cost
Excavation	\$16 / m^3	6000 m^3	\$96,000
Fill	\$50 / m^3	3000 m^3	\$150,000
Clearing	-	8000 m^2	\$6,000
Permitting	-	-	\$2,500
Grading and Granular Sub Base Placement	-	-	\$133,500
Total Cost	-	-	**\$388,000**

The material costs were estimated by consulting various suppliers' data sourced online (referenced at the end of the report). The cost breakdown for the required materials is listed in Table 15 below. The transportation cost was estimated to be 10% of the total material cost.

Table 15: Material Costs

Material Description	Unit Cost	Amount	Cost
Concrete	\$300 / m^3	30 m^3	\$9,000
15M Rebar	\$1.76 / kg	4550 kg	\$8,008
HDPE Dam Pipes	\$122 / m	195 m	\$23,790
HDPE Culvert Pipes	\$200 / m	60 m	\$12,000
HDPE Discharge Connection Pipes	\$94 / m	12 m	\$1,128
Total (includes 10% transportation cost)	-	-	\$59,319

The hourly rates for various construction labour were based on the federal construction contract wage rates for the Vancouver zone (Employment and Social Development Canada, 2016) and data based on Statistics

Canada and provincial wage surveys ("Methodology", n.d.). The cost breakdowns for preparing and placing the concrete dams and other labour compensated at hourly rate are listed in Table 16 and Table 17, respectively.

Table 16: Preparing and Placing Concrete Dams Costs

Activity Description	Hourly Rate per Worker	Number of Workers	Estimated Hours/Dam	Number of Dams	Cost
Pre-casting Concrete	$27.50	2	4	30	$6,600
Placing Dams	$31.00	3	4	30	$11,160
Total Cost	-	-	-	-	**$17,760**

Table 17: Hourly Labour Costs

Labour Crew	Hourly Rate/Worker	Number of Workers	Estimated Hours	Cost
Survey & Site Layout	$30.75	2	40	$2,460
Planter	$23.00	1	16	$368
Roadworks	$29.00	4	32	$3,712
Pipework	$29.00	3	72	$6,264
Compactor	$29.90	1	24	$718
Total Cost	-	-	-	$13,522

The total initial cost of the project is estimated to be $550,391, including 15% contingency. The summary is given in Table 18. The variation in value of money over time, taxes, and labourers' benefits have been excluded in preparation of this estimate.

Table 18: Total Initial Costs

Cost Description	Amount
Trail Construction	$388,000
Material Costs	$59,319
Dam Construction	$17,760
Hourly Construction Labour	$13,522
Total (includes 15% contingency)	**$550,391**

8.0 Operating and Maintenance Cost

According to other typical trails, the maintenance costs for a trail will be about $2000 per kilometer of trail per year (Knoch & Sexton, 2015). This cost will need to be increased due to maintenance of the drainage systems and clearing of any debris. However, these costs will be relatively low as the theme of the trail is to incorporate as much of the surrounding nature as possible. Therefore, the operating and maintenance costs of this trail should be around $7500 per year with a slight increase at each 10-year mark for larger maintenance to be done on the trail (Pacific Watershed Associates, 2004). Table *19* outlines the typical distribution of maintenance costs for a similar trail to this one. As shown, a majority of the costs will go towards maintenance of the vegetation and surrounding greenery.

Table 19: Typical Maintenance Budget of Trail

Maintenance Activity	Percent of Budget
Surface Clearing of Trail	10.8%
Mowing	12.0%
Vegetation Management (Leaf Clearing, Pruning, etc.)	11.2%
Keep Trail-Side Land Clear of Trash and Debris	11.5%
Whole Tree Removal	5.4%
Application of Herbicides and Pesticides	2.3%
Clearing of Drainage Channels and Culverts	5.4%

Table 19 continued: Typical Maintenance Budget of Trail

Maintenance Activity	Percent of Budget
Surface Maintenance of Parking Areas	2.7%
Litter Clear Up, Trash Cans	2.7%
Maintenance of Toilets at Trailheads	13.0%
Maintenance of Toilets Along the Trail	1.2%
Trailhead Parking Snow Removal	1.1%
Repair/Maintenance of Signs	6.3%
Recovery from Illegal Acts of Vandalism/Dumping	5.3%
Other Trail Maintenance Activities	9.1%

9.0 Conclusion

We, at Drift, feel strongly that this final detailed design will provide the greatest benefit for the surrounding community while exceeding UBC's goals for sustainability as a world-class institution. Installation of this trail highlights UBC's strong emphasis on combining technical use along with sleek aesthetics and sustainable infrastructure. Furthermore, the trail design provides an integrated solution that will be a focal point for people entering UBC in from South West Marine Drive.

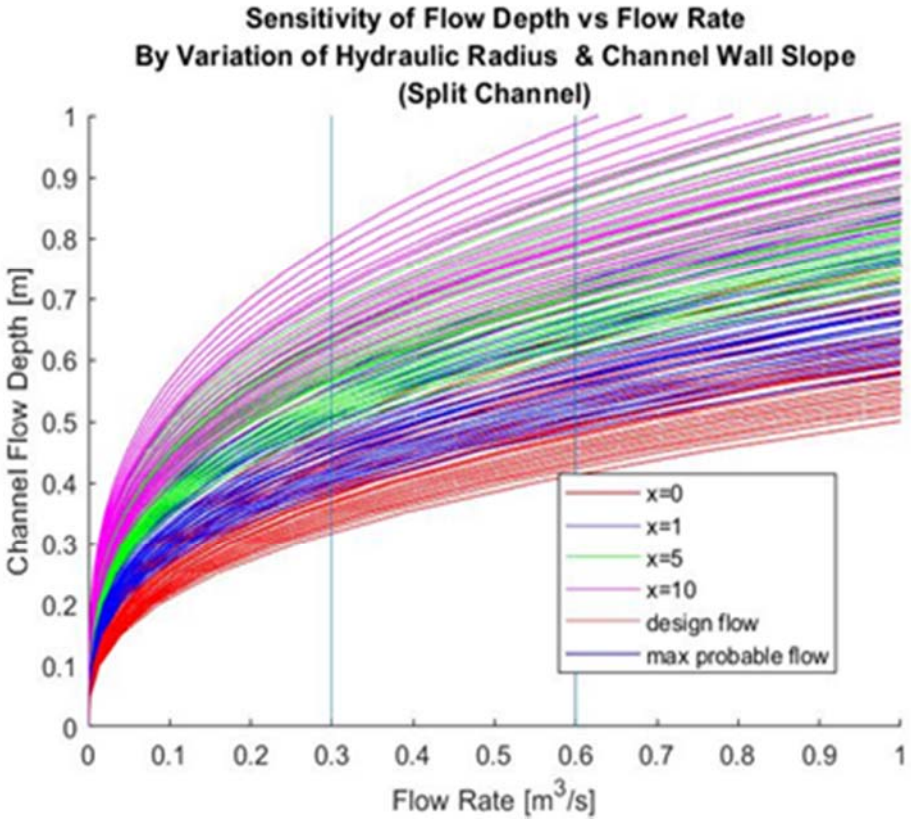
The design provides excellent water retention in the trench due to its natural flow characteristics in addition to the retention from the engineered ponding system controlled by the dam system. The natural flow path is minimally impacted, while still providing the necessary retention volume, time and flow rate. The trench also provides primary treatment to the water flowing through it. The primary treatment ensures that the water reaching vulnerable ecosystems around UBC does not damage those same ecosystems.

The design has been thoroughly deliberated and calculated to ensure design requirements have been met while adhering to all standards and regulations. Final specifications and drawings have been produced, allowing construction of the project to commence. Cost estimates and the construction schedule have also been finalized.

The design of the trench and trail system provides an integrated transition zone between the urban system of the trail and surrounding neighborhood, while the vegetated trenches blend seamlessly into the forest. Additionally, the trail system provides a connection between two existing trail systems on campus, ensuring that it will see significant use. As such, we can be confident that the trail will provide a smooth integration between the built urban infrastructure and the natural environment.

Appendix A

Flow Depth - Sensitivity Analysis


Sensitivity of Flow Depth vs Flow Rate
By Variation of Hydraulic Radius & Channel Wall Slope
(Split Channel)
Channel Flow Depth [m]
Flow Rate [m3/s]
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
x=0
x=1
x=5
x=10
design flow
max probable flow

Appendix B

Storage Capacity - Sensitivity Analysis Results

System Storage Capacity				
K\pw	0	1	5	10
2:1	2300	2400	2500	2500
2.1:1	2400	2500	2600	2600
2.2:1	2500	2600	2700	2700
2.3:1	2600	2700	2800	2900
2.4:1	2800	2800	2900	3000
2.5:1	2900	2900	3000	3100

Appendix C

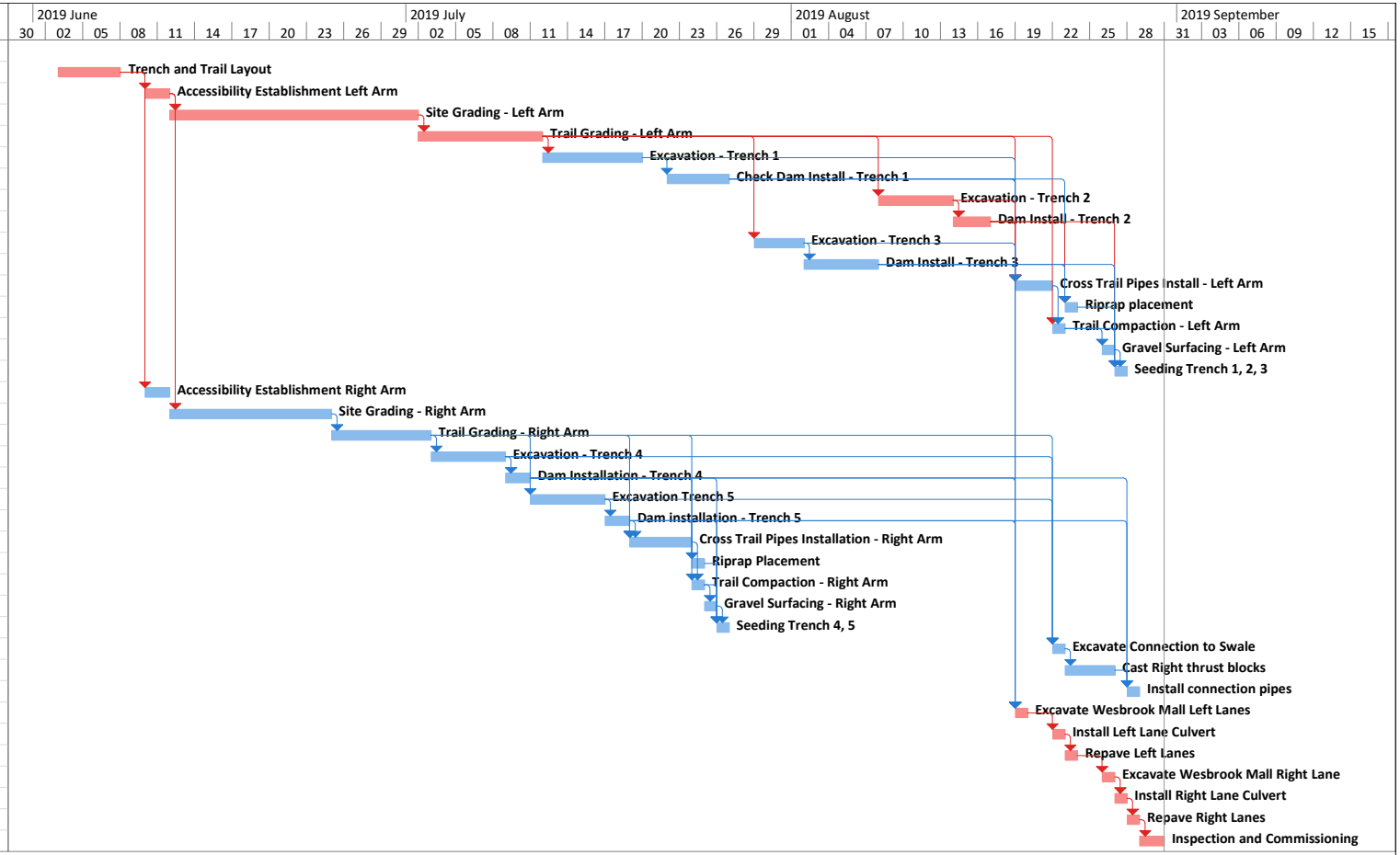
Additional Native Plant Species (Cronk & Fennessy, 2001)

Botanical Name	**Common Name**
Ribes sanguineum	Flowering Currant
Mahonia aquifolium	Oregon Grape Holly
Gaultheria shallon	Salal
Polystichum munitum	Sword Fern
Rhododendron sp.	Rhododendron
Symphoricarpos sp.	Coralberry
Vaccinium sp.	Huckleberry
Varieties	Ferns

Appendix D

Detailed Construction Schedule

ID	Task Name	Duration
1		
2	Trench and Trail Layout	5 days
3	Accessibility Establishment Left Arm	2 days
4	Site Grading - Left Arm	14 days
5	Trail Grading - Left Arm	8 days
6	Excavation - Trench 1	6 days
7	Check Dam Install - Trench 1	5 days
8	Excavation - Trench 2	4 days
9	Dam Install - Trench 2	3 days
10	Excavation - Trench 3	4 days
11	Dam Install - Trench 3	4 days
12	Cross Trail Pipes Install - Left Arm	3 days
13	Riprap placement	1 day
14	Trail Compaction - Left Arm	1 day
15	Gravel Surfacing - Left Arm	1 day
16	Seeding Trench 1, 2, 3	1 day
17	Accessibility Establishment Right Arm	2 days
18	Site Grading - Right Arm	9 days
19	Trail Grading - Right Arm	6 days
20	Excavation - Trench 4	4 days
21	Dam Installation - Trench 4	2 days
22	Excavation Trench 5	4 days
23	Dam installation - Trench 5	2 days
24	Cross Trail Pipes Installation - Right Ar	3 days
25	Riprap Placement	1 day
26	Trail Compaction - Right Arm	1 day
27	Gravel Surfacing - Right Arm	1 day
28	Seeding Trench 4, 5	1 day
29	Excavate Connection to Swale	1 day
30	Cast Right thrust blocks	2 days
31	Install connection pipes	1 day
32	Excavate Wesbrook Mall Left Lanes	1 day
33	Install Left Lane Culvert	1 day
34	Repave Left Lanes	1 day
35	Excavate Wesbrook Mall Right Lane	1 day
36	Install Right Lane Culvert	1 day
37	Repave Right Lanes	1 day
38	Inspection and Commissioning	2 days



Project: msproj11
Date: Sat 19-03-30

Task	External Tasks	Manual Task	Finish-only	Manual Progress
Split	External Milestone	Duration-only	Deadline	
Milestone	Inactive Task	Manual Summary Rollup	Critical	
Summary	Inactive Milestone	Manual Summary	Critical Split	
Project Summary	Inactive Summary	Start-only	Progress	

Appendix E

Calculations

Sizing Pipe Diameters:

Orifice Flow:

Q=CoA2gH

Where:
Q = flow
Co = Orifice coefficient
A = Orifice area
H = Hydraulic head

Reynold's Method:

Re= VD
f=1.325ln e3.7D+5.74Re0.9 2
hL=fLD+KV22g
Q=A2g(z-P-hL)

Where:
Re = Reynold's number
D = Pipe Diameter
f = friction factor
h_L = headloss
L = Pipe length
K = Minor loss coefficient
z = Upstream pressure head
P = Downstream pressure

Hydraulics:

Manning's:

$Q = \frac{R^{2/3} * {S_0}^{1/2}}{n} * A\ ,\ R = \frac{A}{p_w},\ p_w = 2\sqrt{5} * y\ , A = 2y^2$

$\therefore Q = \frac{(2y^2)^{2/3} * {S_0}^{1/2}}{(2\sqrt{5}*y)^{2/3} * n}$

Bernoulli:

$z_1 + \frac{v_1^2}{2g} + \frac{P_1}{\gamma} = z_2 + \frac{v_2^2}{2g} + \frac{P_2}{\gamma}\ , P_1 = P_2, v_1 = 0, v_2 = \frac{Q}{A}, A = 2y^2,$

$\therefore z_1 = z_2 + \frac{v_2^2}{2g}$

Momentum:

$F_r = \frac{v_1}{\sqrt{g h_m}}\ , h_m = \frac{A}{T} = \frac{2y^2}{2y} = y$

$y_2 = y_1 \left(\frac{1}{2}\right)\left(\sqrt{1 + 8{F_r}_1^2} - 1\right)$

Structural:

Forces

$F_{dam} + F_{upstream\ flow} + F_{Downstream\ fow} = 0$

$F_{dam} = Q\rho_{water}(v_2 - v_1)$

Momentum

$m_1 = \rho v_1 A_1$

Summation

$\sum F_{x-water} = -F_{dam} + p_1 A_1 - p_2 A_2 = \dot{m}(v_2 - v_1)$, $p_i A_i = \rho\gamma \left(\int_0^{h_i} (h_i - y) 2y \, dy \right)$

$F_{dam_x} = \rho\gamma \left(\frac{h_1^3}{3} - \frac{h_2^3}{3} \right) - \rho v_1 A_1 (v_2 - v_1)$

$F_{dam_y} = \rho_{dam} g V_{dam}$

$F_{riprap_y} = \rho_{riprap} g V_{riprap}$

$F_{riprap_x} = \rho_{riprap} g V_{riprap} * \mu$, $\mu = friction\ factor$

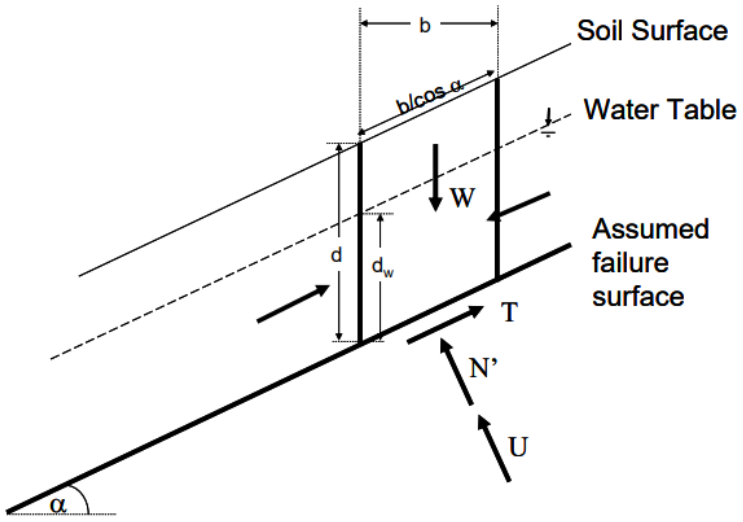
$\sum F_{x-structural} = F_{dam_x} - F_{riprap_x} - F_{active\ (earth)} = 0$

$\sum F_{y-structural} = F_{dam_y} - F_{riprap_y}$

$\sum M = F_{dam_x} * d_{y\ dam\ force\ Centroid} - F_{riprap_x} * d_{y\ riprap\ Centroid} - F_{active\ (earth)} *$
$d_{y\ active\ earth\ Centroid} - F_{dam_y} * d_{x\ dam\ force\ Centroid} - F_{riprap_y} * d_{x\ riprap\ Centroid}$

Slope Stability Analysis Calculations

Long and Shallow slopes:



All of our analysis will be performed for a 2H:1V slope:

$$\propto = \tan^{-1}\frac{1}{2} = 26.5°$$

$$FOS = \frac{\tau_f}{\tau_{mob}} = \frac{shear\ stress\ required\ for\ failure}{actual\ shear\ stress}$$

$$FOS = \frac{C' + (\gamma d + \gamma_w d_w)\,(\cos \propto)^2 \tan \emptyset'}{(\gamma d \sin \alpha \cos \alpha)}$$

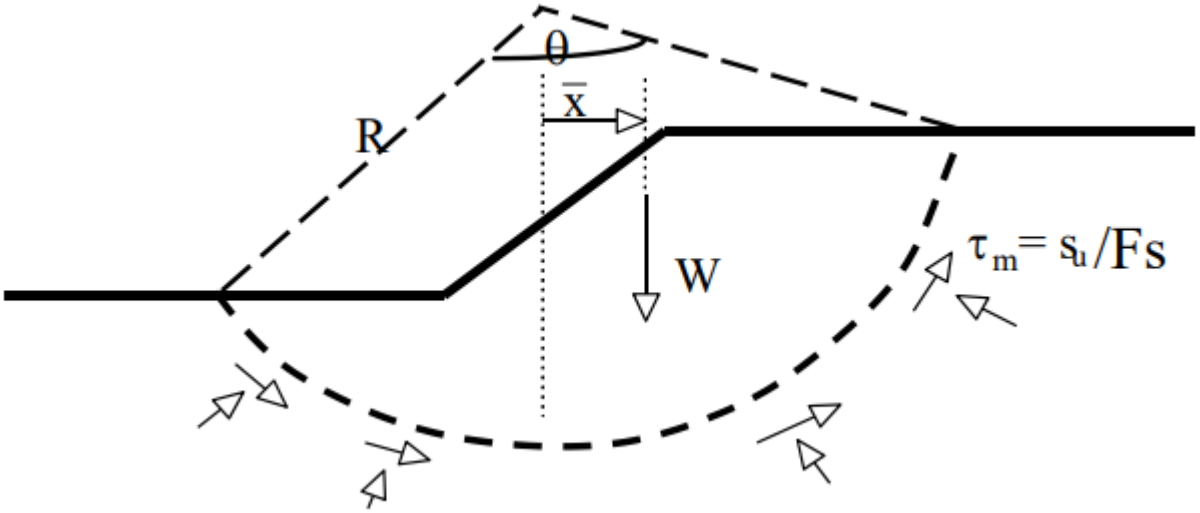
WSDOT states that cohesion of glacial till in our area range from 4.79 kPa to 47.9 kPa, has an average unit weight of 20.4 kN/m^3, and an effective internal friction angle of 40°. We will use a conservative cohesion value of 4.79 kPa for our analysis. For the worst-case scenario, the water table would be at the ground surface; therefore, d = d_w.

$$FOS = \frac{4.79\ kPa + \left(20.4\frac{kN}{m^3} d + 9.81\frac{kN}{m^3} d_w\right)(\cos 26.5°)^2 \tan 40°}{(20.4\frac{kN}{m^3} d \sin 26.5° \cos 26.5°)}$$

To find the lowest FOS we want the minimum of the function (take the derivative). However, the derivative of the first part of the function would indicate the minimum FOS occurs at -∞. The derivative of the second part of the function indicates that the minimum FOS occurs at d = 1.66697.

$$FOS = \frac{4.79\ kPa + \left(20.4\frac{kN}{m^3}(1.66697\ m) + 9.81\frac{kN}{m^3}(1.66697\ m)\right)(\cos 26.5°)^2 \tan 40°}{(20.4\frac{kN}{m^3}(1.66697\ m)\sin 26.5° \cos 26.5°)} = 11.8$$

Deep-Seated Circular Failure



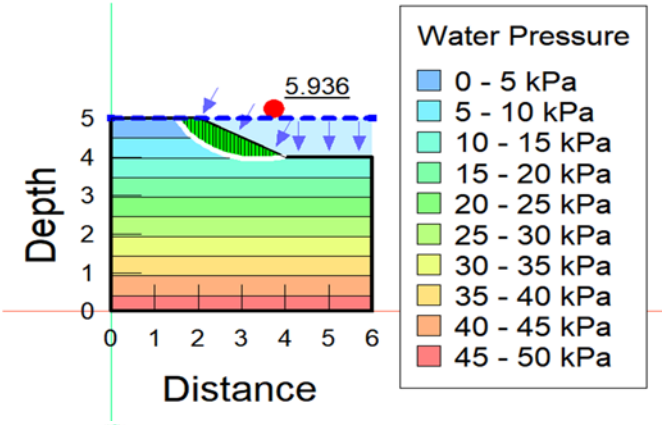
$$FOS = \frac{R^2\theta S_u}{W\bar{x}} = \frac{Resisting\ Moment}{Disturbing\ Moment}$$

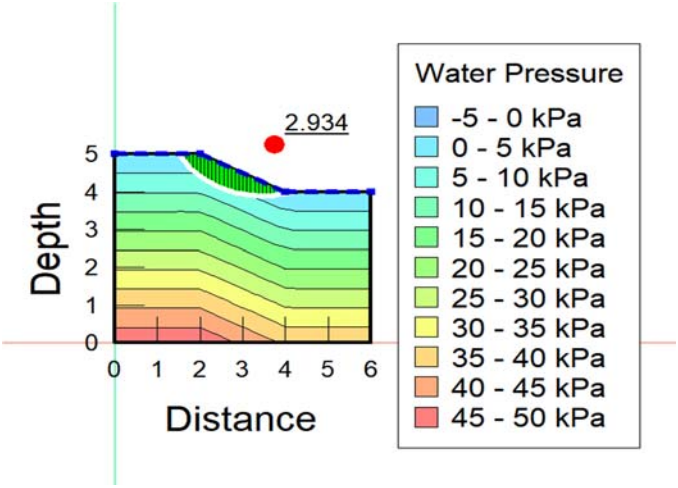
$$FOS = \frac{(1.5m)^2(96.4°) * 4.79\ kPa}{40.8kN * 1.6667} = 15.3$$

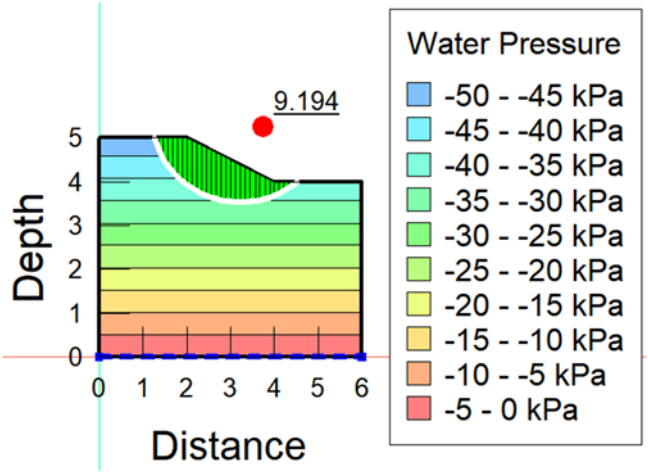
Appendix F

Geotechnical Analysis

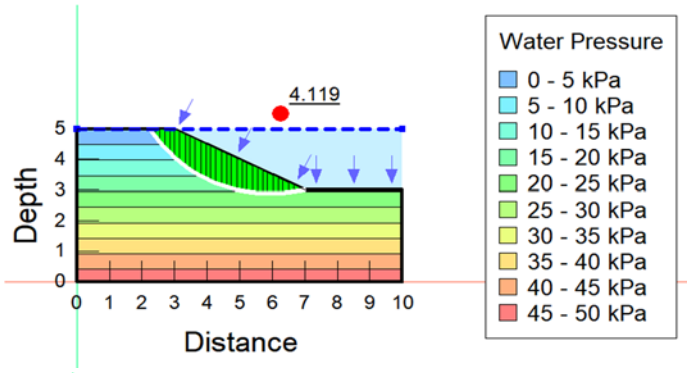
Case 1

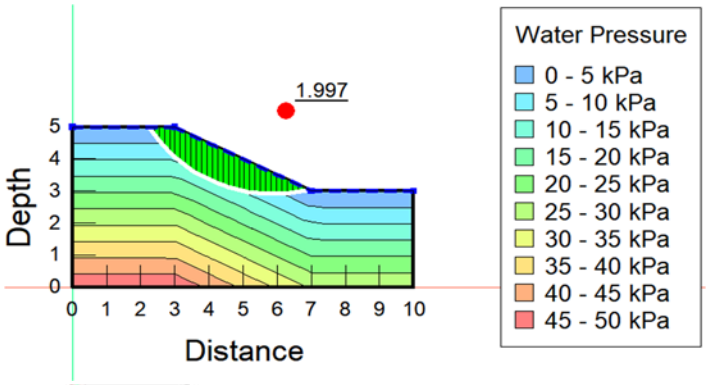

5.936
Depth
Distance
Water Pressure
0 - 5 kPa
5 - 10 kPa
10 - 15 kPa
15 - 20 kPa
20 - 25 kPa
25 - 30 kPa
30 - 35 kPa
35 - 40 kPa
40 - 45 kPa
45 - 50 kPa

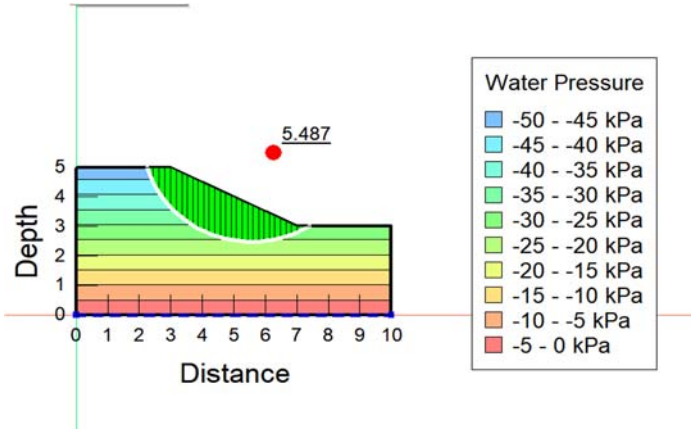

2.934
Depth
Distance
Water Pressure
-5 - 0 kPa
0 - 5 kPa
5 - 10 kPa
10 - 15 kPa
15 - 20 kPa
20 - 25 kPa
25 - 30 kPa
30 - 35 kPa
35 - 40 kPa
40 - 45 kPa
45 - 50 kPa


9.194
Depth
Distance
Water Pressure
-50 - -45 kPa
-45 - -40 kPa
-40 - -35 kPa
-35 - -30 kPa
-30 - -25 kPa
-25 - -20 kPa
-20 - -15 kPa
-15 - -10 kPa
-10 - -5 kPa
-5 - 0 kPa

Case 2


4.119
Depth
Distance
0 1 2 3 4 5 6 7 8 9 10
Water Pressure
0 - 5 kPa
5 - 10 kPa
10 - 15 kPa
15 - 20 kPa
20 - 25 kPa
25 - 30 kPa
30 - 35 kPa
35 - 40 kPa
40 - 45 kPa
45 - 50 kPa


1.997
Depth
Distance
0 1 2 3 4 5 6 7 8 9 10
Water Pressure
0 - 5 kPa
5 - 10 kPa
10 - 15 kPa
15 - 20 kPa
20 - 25 kPa
25 - 30 kPa
30 - 35 kPa
35 - 40 kPa
40 - 45 kPa
45 - 50 kPa


5.487
Depth
Distance
0 1 2 3 4 5 6 7 8 9 10
Water Pressure
-50 - -45 kPa
-45 - -40 kPa
-40 - -35 kPa
-35 - -30 kPa
-30 - -25 kPa
-25 - -20 kPa
-20 - -15 kPa
-15 - -10 kPa
-10 - -5 kPa
-5 - 0 kPa

Appendix G

MATLAB

```
syms Q y;

%Design Parameters
D=0.95; %Initial water depth
x1=0;
x2=1;
x3=5;
x4=10;
K_max=2.5; %K=side slope ratio (H:V) ie for a 2:1 slope K=2, for a 3:1 slope K=3
K_min=2.0; %Starting val for K iterations
S_min=0.01; %Starting slope for slope iterations
S_max=0.05;
Count_Dams =[4,7,9,5,2,3]; %this is an array of the number of dams for each slope from 0 to 0.05

%Primary Variables
M = zeros(30,4); %M is a matrix of minimum flow depths using manning's equation
V_lost = zeros(30,4); %V_lost is a matrix of volume lost from "ideal" (flat prism) flow conditions
T = zeros(30,4); %T is a matrix of the total volume held in each dam section
ST= zeros(30,4); %ST is the total actual storage in the system for each slope zone
FT = zeros(6,4); %FT is a summary matrix displaying the total storage in each senario

%skew Variables
skew=0.75; %This is a correction parameter for the volume lost calculations
Ts = zeros(30,4); %T is a matrix of the total volume held in each dam section
STs= zeros(30,4); %ST is the total actual storage in the system for each slope zone
FTs = zeros(6,4); %FT is a summary matrix displaying the total storage in each senario

count = 1;
count2 = 1;
```

```
figure
hold on

for K = K_min:0.1:K_max
    count2 = 1;
    for S = S_min:0.01:S_max


        l1=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x1))^(2/3)*S^0.5*(0.5*K*y^2)/0.03);
        l2=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x2))^(2/3)*S^0.5*(0.5*K*y^2)/0.03);
        l3=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x3))^(2/3)*S^0.5*(0.5*K*y^2)/0.03);
        l4=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x4))^(2/3)*S^0.5*(0.5*K*y^2)/0.03);

        set(l1,'color','r');
        set(l2,'color','b');
        set(l3,'color','g');
        set(l4,'color','m');

        s1=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+0))^(2/3)*S^0.5*(0.5*K*y^2)/0.03,y,[0,Inf]);
        s2=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+1))^(2/3)*S^0.5*(0.5*K*y^2)/0.03,y,[0,Inf]);
        s3=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+5))^(2/3)*S^0.5*(0.5*K*y^2)/0.03,y,[0,Inf]);
        s4=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+10))^(2/3)*S^0.5*(0.5*K*y^2)/0.03,y,[0,Inf]);

        M(count,1) = vpa(s1);
        M(count,2) = vpa(s2);
        M(count,3) = vpa(s3);
        M(count,4) = vpa(s4);
```

```
    for i=1:4
        V_lost(count,i)=((0.5*(D-M(count,i))*4)-((D-M(count,i))*K*(D-M(count,i))/6)*2)*30;
        T(count,i)=D^2*K*50-V_lost(count,i);
        ST(count,i)=Count_Dams(count2+1)*T(count,i);
    end


    count = count+1
    count2 = count2+1;

  end
end

hold off
ylabel('Channel Flow Depth');
xlabel('Flow Rate [m^3/s]');
ylabel('Channel Flow Depth [m]');
title({'Sensitivity of Flow Depth vs Flow Rate', 'By Variation of Hydraulic Radius  & Channel Wall Slope','(Split Channel)'});
line([0.3,0.3],[0,1]);
axis([0,1,0,1])
line([0.6,0.6],[0,1]);
legend(['x=' num2str(x1)],['x=' num2str(x2)],['x=' num2str(x3)],['x=' num2str(x4)],'design flow','max probable flow');
legend('location','best');

for i=0:5
  for j=1:4
    FT(i+1,j)= round(sum(single([433.2 ST(i*5+1,j) ST(i*5+2,j) ST(i*5+3,j) ST(i*5+4,j) ST(i*5+5,j)]))+20,-2);
```

```
    end
end


count = 1;
for K = K_min:0.1:K_max
    count2 = 1;
    for S = S_min:0.01:S_max
        for i=1:4
            V_losts(count,i)=((0.5*(D-M(count,i))*4)-((D-M(count,i))*K*(D-M(count,i))/6)*2)*30*skew;
            Ts(count,i)=D^2*K*50-V_losts(count,i);
            STs(count,i)=Count_Dams(count2+1)*Ts(count,i);
        end
        count = count+1;
        count2 = count2+1;
    end
end


for i=0:5
    for j=1:4
        FTs(i+1,j)= round(sum(single([433.2 STs(i*5+1,j) STs(i*5+2,j) STs(i*5+3,j) STs(i*5+4,j)
STs(i*5+5,j)]))+20,-2);
    end
end
```

Water Storage Volume (Sensitivity Analysis Code):

```
syms Q y;

D=0.95; %Initial water depth
x1=0;
x2=1;
x3=5;
x4=10;
K_max=2.5; %K=side slope ratio (H:V) ie for a 2:1 slope K=2, for a 3:1
slope K=3
K_min=2.0; %Starting val for K iterations
S_min=0.01; %Starting slope for slope iterations
S_max=0.05;
M = zeros(30,4); %M is a matrix of minimum flow depths using manning's
equation
V_lost = zeros(30,4); %V_lost is a matrix of volume lost from "ideal"
(flat prism) flow conditions
T = zeros(30,4); %T is a matrix of the total volume held in each dam
section
ST= zeros(30,4); %ST is the total actual storage in the system for
each slope zone
count = 1; %count variable for matrix input
count2 = 1;
Count_Dams =[4,7,9,5,2,3]; %this is an array of the number of dams for
each slope from 0 to 0.05
FT = zeros(6,4); %FT is a summary matrix displaying the total storage
in each scenario

figure
hold on

for K = K_min:0.1:K_max
    count2 = 1;
    for S = S_min:0.01:S_max



            l1=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x1))^(2/3)*S^0.5*(0.5*
            K*y^2)/0.03);

            l2=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x2))^(2/3)*S^0.5*(0.5*
            K*y^2)/0.03);

            l3=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x3))^(2/3)*S^0.5*(0.5*
            K*y^2)/0.03);

            l4=ezplot(Q==(0.5*K*y^2/((sqrt(5))*y+x4))^(2/3)*S^0.5*(0.5*
            K*y^2)/0.03);

        set(l1,'color','r');
        set(l2,'color','b');
```

```
        set(l3,'color','g');
        set(l4,'color','m');


           s1=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+0))^(2/3)*S^0.5*(0.5
           *K*y^2)/0.03,y,[0,Inf]);

           s2=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+1))^(2/3)*S^0.5*(0.5
           *K*y^2)/0.03,y,[0,Inf]);

           s3=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+5))^(2/3)*S^0.5*(0.5
           *K*y^2)/0.03,y,[0,Inf]);

           s4=vpasolve(0.3==(0.5*K*y^2/((2.236)*y+10))^(2/3)*S^0.5*(0.
           5*K*y^2)/0.03,y,[0,Inf]);

        M(count,1) = vpa(s1);
        M(count,2) = vpa(s2);
        M(count,3) = vpa(s3);
        M(count,4) = vpa(s4);

        for i=1:4
           V_lost(count,i)=((0.5*(D-M(count,i))*4)-((D-
           M(count,i))*K*(D-M(count,i))/6)*2)*30;
           T(count,i)=D^2*K*50-V_lost(count,i);
           ST(count,i)=Count_Dams(count2+1)*T(count,i);
        end


        count = count+1
        count2 = count2+1;

    end
end

hold off
ylabel('Channel Flow Depth');
xlabel('Flow Rate [m^3/s]');
ylabel('Channel Flow Depth [m]');
title({'Sensitivity of Flow Depth vs Flow Rate', 'By Variation of
Hydraulic Radius  & Channel Wall Slope','(Split Channel)'});
line([0.3,0.3],[0,1]);
axis([0,1,0,1])
line([0.6,0.6],[0,1]);
legend(['x=' num2str(x1)],['x=' num2str(x2)],['x=' num2str(x3)],['x='
num2str(x4)],'design flow','max probable flow');
legend('location','best');

for i=0:5
    for j=1:4
           FT(i+1,j)= round(sum(single([433.2 ST(i*5+1,j) ST(i*5+2,j)
           ST(i*5+3,j) ST(i*5+4,j) ST(i*5+5,j)])),-2);
```

```
    end
end
```

Appendix H

Discharge Calculations

The system was analyzed using the empirically derived formulas by Swamee and Jain for pipe flow and the energy equation. These equations are shown below:

$$h_L = 1.07\frac{Q^2 L}{gD^5}\left\{\ln\left[\frac{e}{3.7D} + 4.62\left(\frac{\nu D}{Q}\right)^{0.9}\right]\right\}^{-2} \qquad \begin{matrix} 10^{-6} < e/D < 10^{-2} \\ 3000 < \mathrm{Re} < 3\times 10^8 \end{matrix} \qquad (7.6.29)$$

$$Q = -0.965\left(\frac{gD^5 h_L}{L}\right)^{0.5}\ln\left[\frac{e}{3.7D} + \left(\frac{3.17\nu^2 L}{gD^3 h_L}\right)^{0.5}\right] \qquad \mathrm{Re} > 2000 \qquad (7.6.30)$$

$$D = 0.66\left[e^{1.25}\left(\frac{LQ^2}{gh_L}\right)^{4.75} + \nu Q^{9.4}\left(\frac{L}{gh_L}\right)^{5.2}\right]^{0.04} \qquad \begin{matrix} 10^{-6} < e/D < 10^{-2} \\ 5000 < \mathrm{Re} < 3\times 10^8 \end{matrix} \qquad (7.6.31)$$

$$\frac{V_1^2}{2g} + \frac{p_1}{\gamma} + h_1 = \frac{V_2^2}{2g} + \frac{p_2}{\gamma} + h_2 + h_L$$

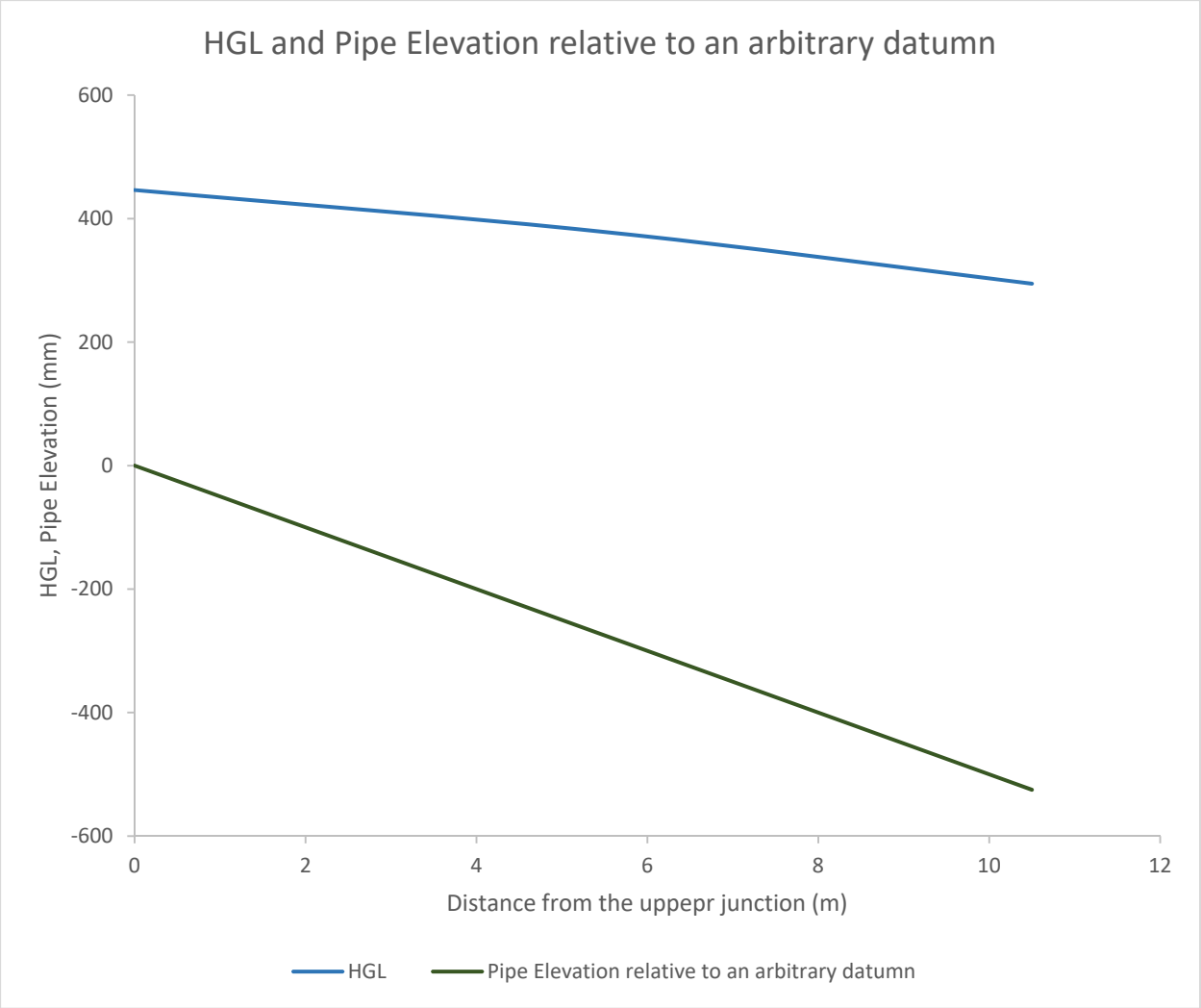
Assumptions:

- Water level at the existing trench is near its capacity (0.9m)
- Flow rates are 0.5 m^3/s and 1 m^3/s before and after the lower junction, respectively
- Minor losses are negligible
- The slope of the ground between the proposed trench and existing trench estimated to be 5 % using Google Earth
- Length between lower and upper junction = 5.5 m
- Length between lower junction and outfall = 5 m
- Roughness = 0.0015 mm, water viscosity = 10^{-6} m^2/s, g = 9.81 m/s^2

Procedure:

- Calculate the total head at the upper junction using the energy equation, H_1
- Select a trial pipe diameter downstream of the lower junction to be 600 mm and evaluate the headloss between the lower junction and outfall using equation 7.6.29
- Calculate the head at the lower junction, H2, using the energy equation and headloss found from previous step
- Find the headloss between lower and upper junction and substitute it into equation 7.6.31 to find the diameter of the pipe between lower and upper junction
- Repeat the steps by optimizing the pipe diameters until a satisfactory solution is reached

Analysis resulted in pipe diameters of 0.45 m and 0.55 m, upstream and downstream of the lower junction, respectively. HGL plot is shown below:


HGL and Pipe Elevation relative to an arbitrary datumn
600
400
200
0
-200
-400
-600
HGL, Pipe Elevation (mm)
0
2
4
6
8
10
12
Distance from the uppepr junction (m)
HGL
Pipe Elevation relative to an arbitrary datumn

Appendix I

Concrete Calculations

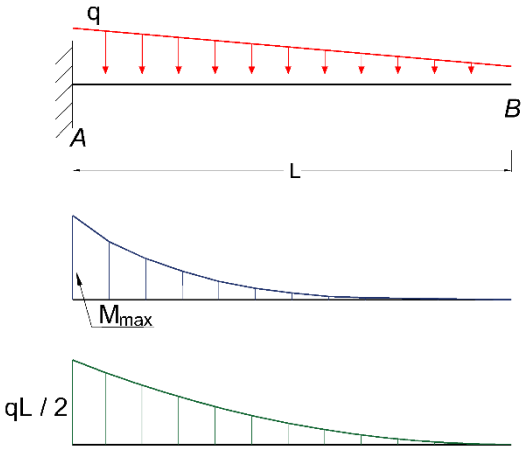
Assumptions:

- The effects of the pipe opening are negligible
- Worst case scenario, water is fully backed up on upstream side of the dam
- Soil unit weight is 24 kN/m^3
- Live load factor of 1.5 is applicable

Evaluated moment and shear demands:

$$Mmax = 1.5\left(\frac{ql^2}{6}\right) = \frac{1.5\left(\frac{24}{3} + 9.8\right)(.94)^2}{6} = 3.93\ kN\frac{m}{m}$$

$$Vmax = 1.5\left(\frac{ql}{2}\right) = \frac{1.5\left(\frac{24}{3} + 9.8\right)(.94)}{2} = 12.5\ \frac{kN}{m}$$



Required tension reinforcement for flexure:

$$d = t - cover - \frac{d_{bar}}{2} = 420\ mm$$

$$A_s = 0.0015 f_c' b\left(d - \sqrt{d^2 - \frac{3.85 M_{max}}{f_c' b}}\right)$$

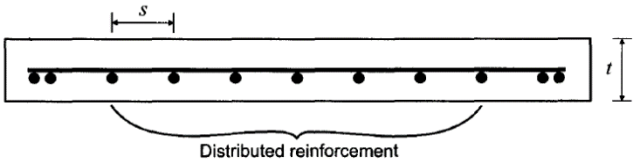
$$A_{Vmin} = 0.0015 A_g$$

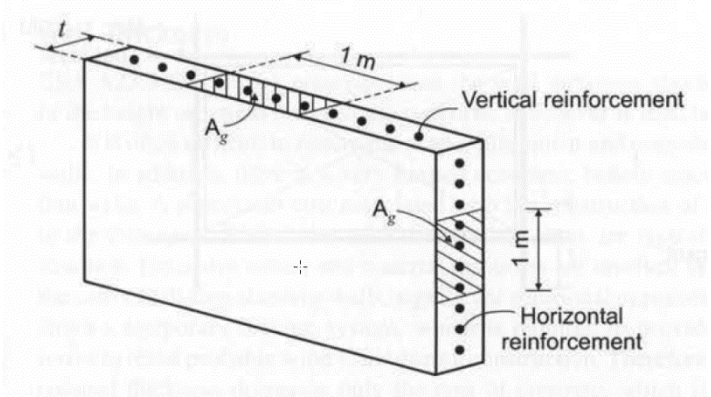
$$A_{Hmin} = 0.002 A_g$$

$$s = \frac{A_b(1000)}{A_s}$$

$$d_v = \max\{0.9d, 0.72t\} = 0.9(420) = 378\ mm$$

$$\beta = \frac{230}{1000 + d_v} = \frac{230}{1000 + 360} = 0.17$$





Balanced reinforcement ratio:

$$\rho = \frac{A_s}{bd} = \begin{cases} \frac{800}{1000 * 420} = 0.0019 < \rho_b = 0.03 \quad vertical \\ \frac{1000}{1000 * 420} = 0.0024 < \rho_b = 0.03 \; horizontal \end{cases}$$

Shear and Flexure strength vs demands:

$$Vc = \varphi_c \beta \sqrt{f_c'} b_w d_v = 0.65(0.17)\sqrt{35}(1000)(378) = 247 \frac{kN}{m} > Vmax = 12.5 \frac{kN}{m}$$

$$A_s = 0.0015(35)1000\left(420 - \sqrt{420^2 - \frac{3.85(3.93)10^6)}{35000}}\right) = 27 \; \frac{mm^2}{m} \ll A_s$$

$$= \begin{cases} 800 \frac{mm^2}{m}, \quad vertical \\ 1000 \frac{mm^2}{m}, horizontal \end{cases}$$

Appendix J

EPA SWMM 5 Outputs

Table - Link T6D-S27Y

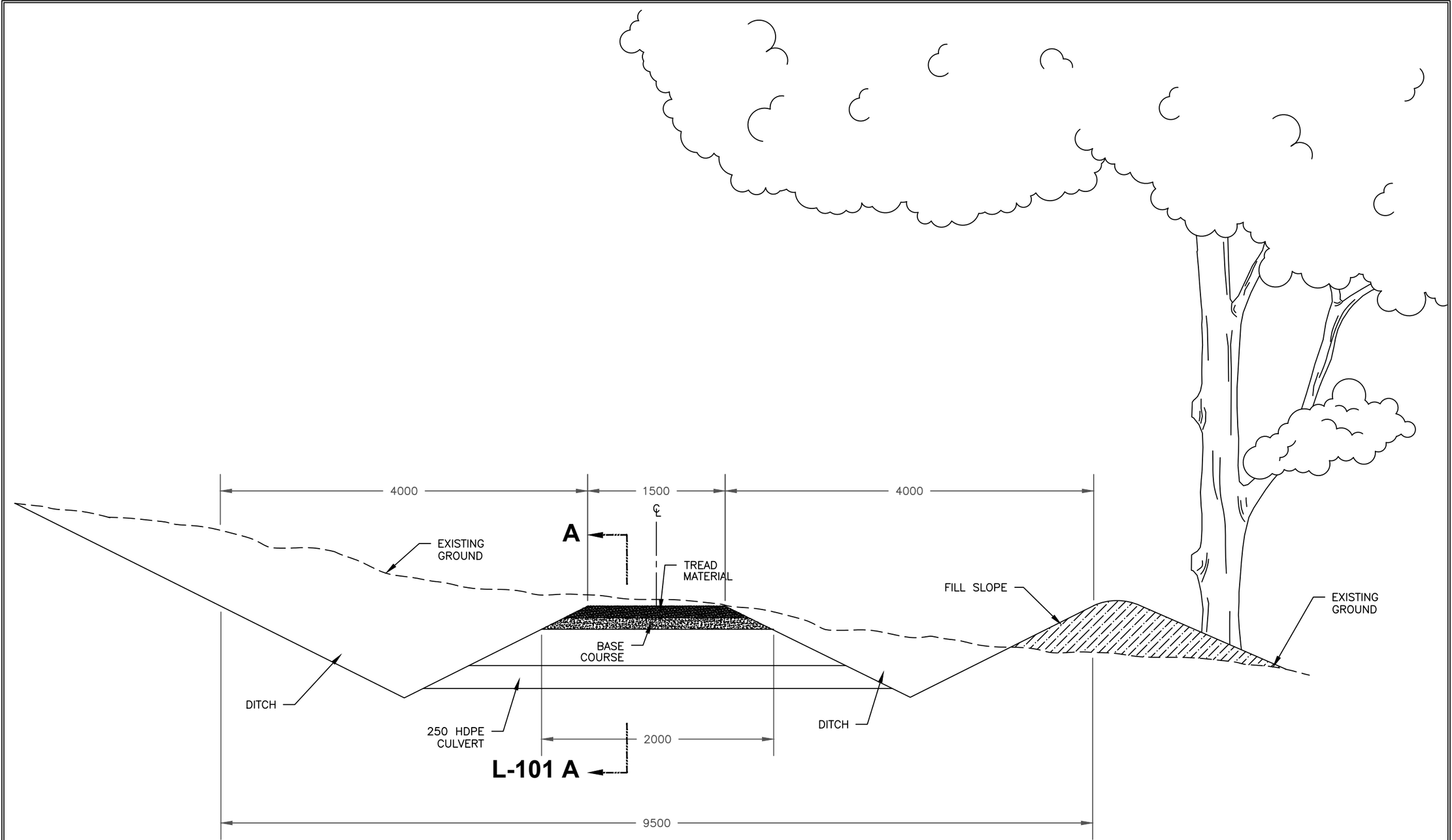
Hours	Flow (LPS)	Flow (m^3/s)	Flow above Pipe Capacity (m^3/s)	Overcapacity Volume (m^3)
8:15:00	1191.53	1.19153	-0.00847	-8
8:30:00	3000.58	3.00058	1.80058	1621
8:45:00	2038.52	2.03852	0.83852	755
9:00:00	1417.4	1.4174	0.2174	196
9:15:00	1130.2	1.1302	-0.0698	-63
			Storage Volume Required	2500

Appendix K

Full Drawing Set

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION



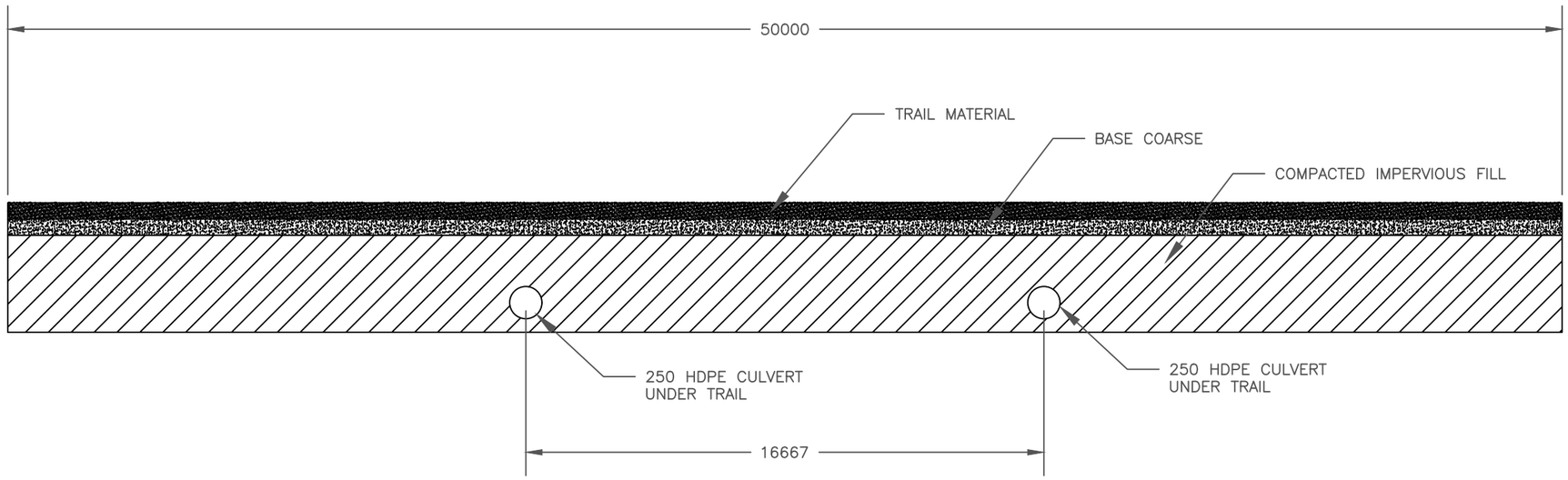
DRIFT CONSULTING **STANDARD TRAIL SECTION**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **TRAIL SECTION** SECTION **L-100-TRAIL SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-100**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

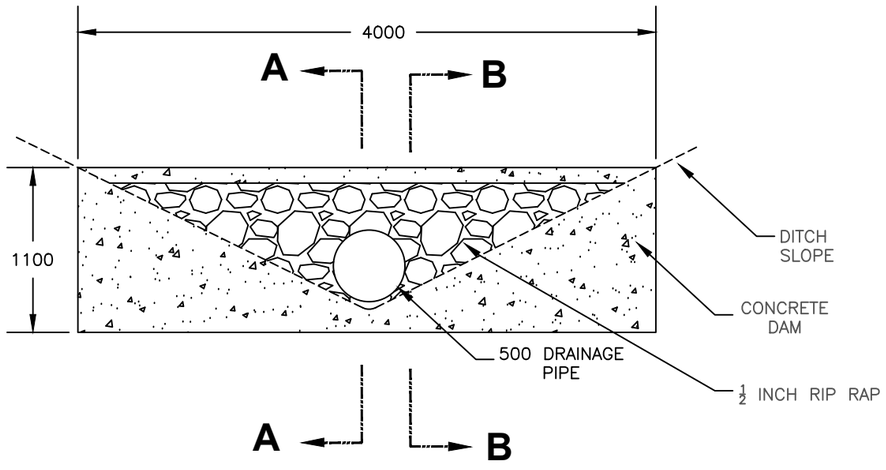


L-100 SECTION A-A

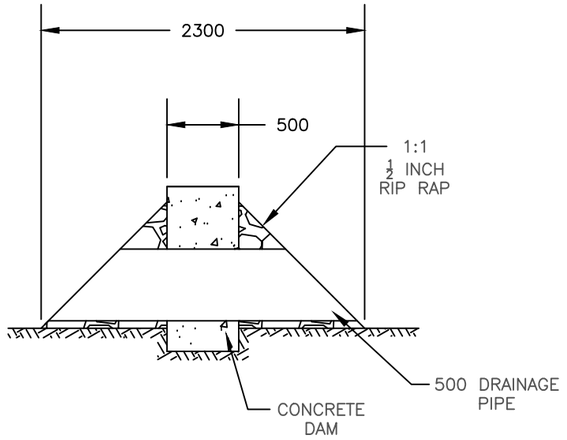
DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **TYPICAL TRAIL SECTON** SECTION **L-101-TRAIL SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-101**

PRODUCED BY AN AUTODESK STUDENT VERSION

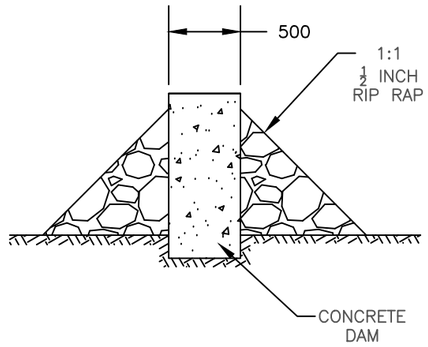
PRODUCED BY AN AUTODESK STUDENT VERSION



TYPICAL DAM SECTION



SECTION A-A



SECTION B-B

TYPICAL DAM LONGITUDINAL VIEW

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION** SECTION **L-200-DAM SECTION**	REVISION DATE **11/20/2018** **1:100**	DRAWING NO. **L-200**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

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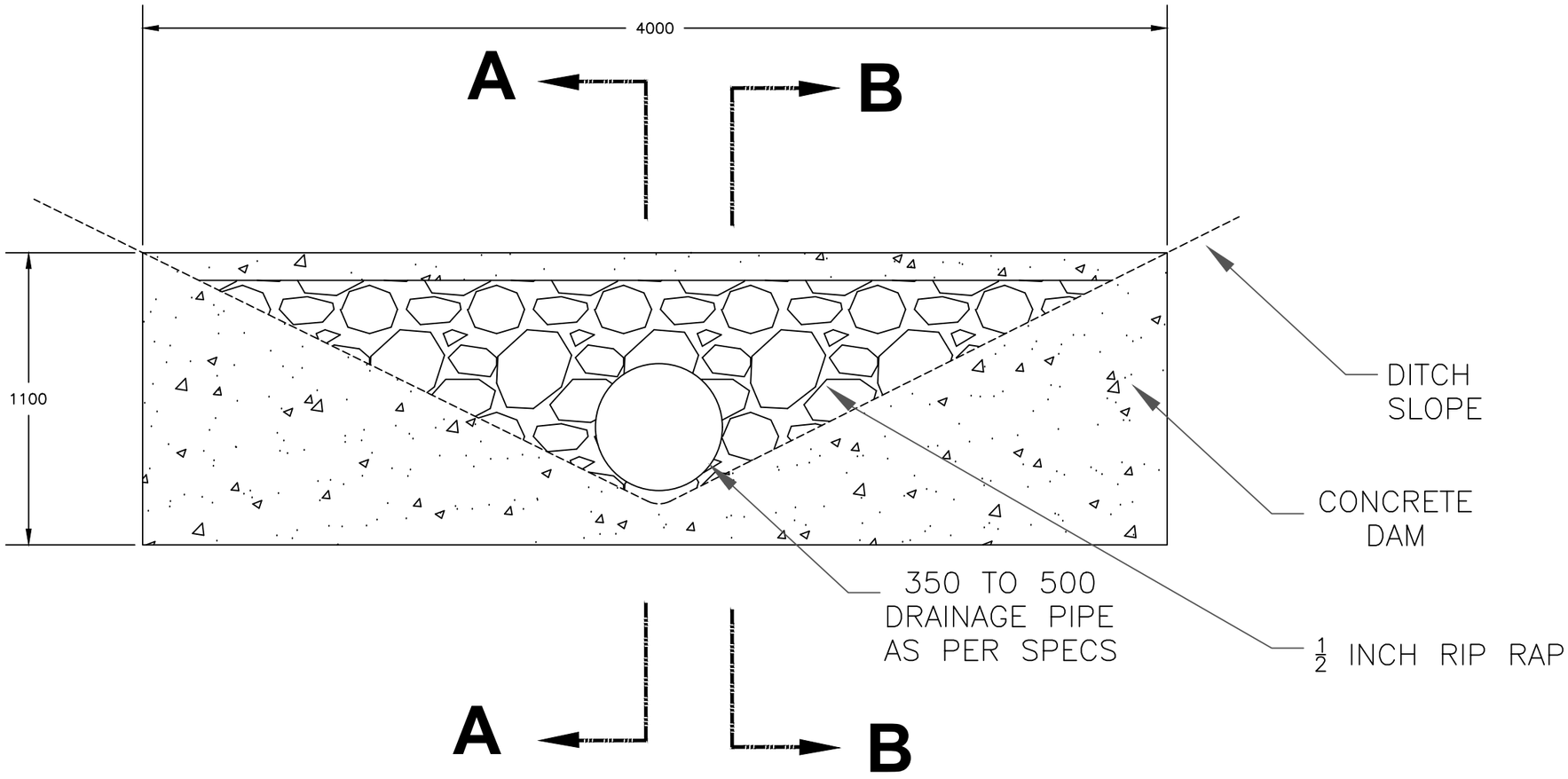
PRODUCED BY AN AUTODESK STUDENT VERSION



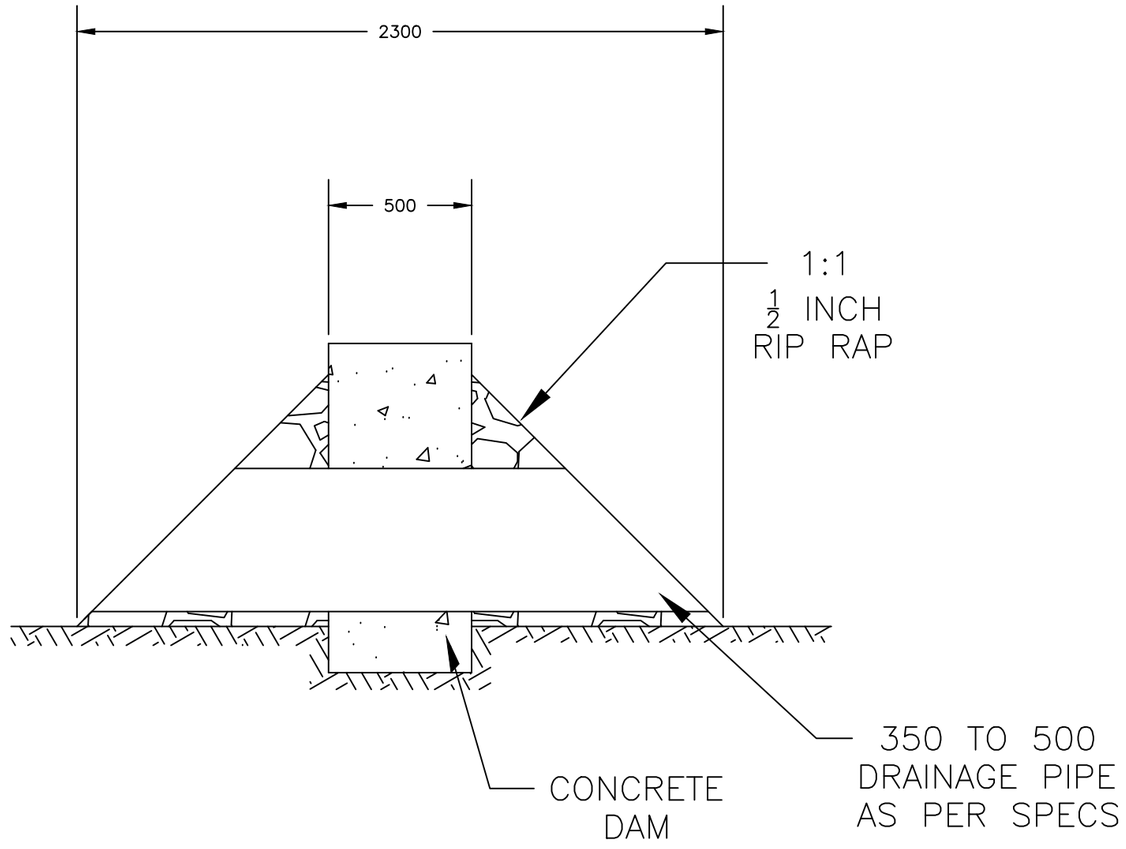
TYPICAL DAM SECTION

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **TYPICAL DAM SECTION** SECTION **L-200-TYPICAL DAM SECTION**	REVISION DATE **02/02/2019** **1:50**	DRAWING NO. **L-201**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION



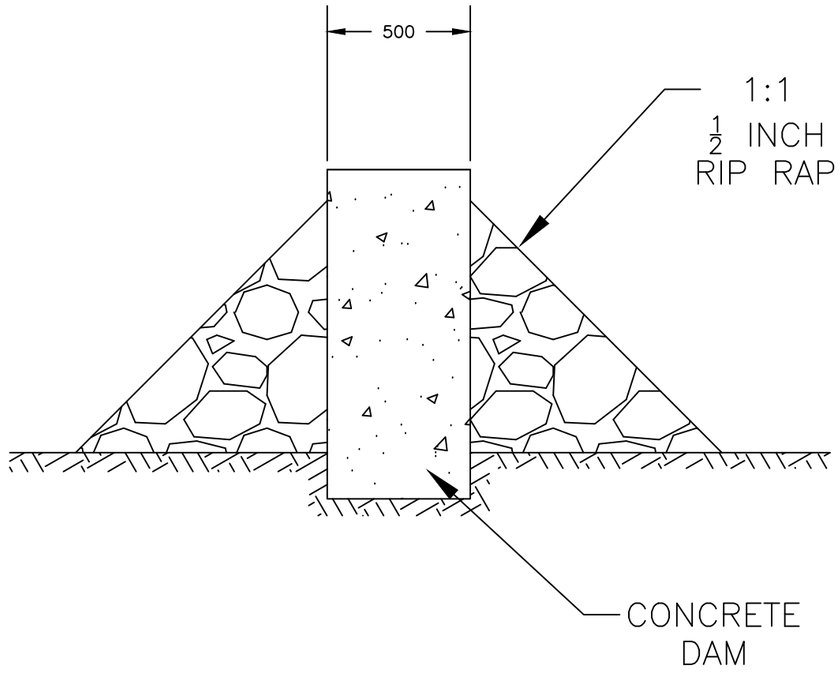
SECTION A-A

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **TYPICAL DAM SECTION A-A** SECTION **L-200-TYPICAL DAM SECTION**	REVISION DATE **02/02/2019** **1:50**	DRAWING NO. **L-202**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

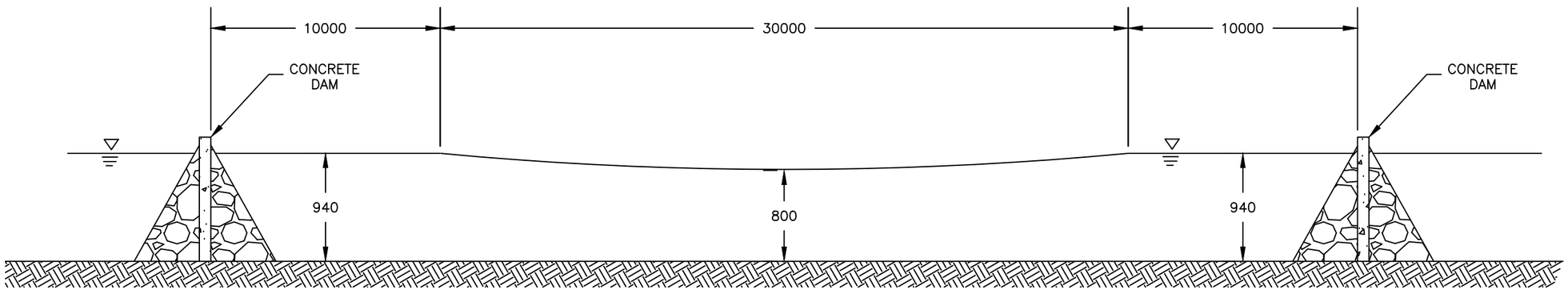


SECTION B-B

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **TYPICAL DAM SECTION B-B** SECTION **L-200-TYPICAL DAM SECTION**	REVISION DATE **02/02/2019** **1:50**	DRAWING NO. **L-203**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

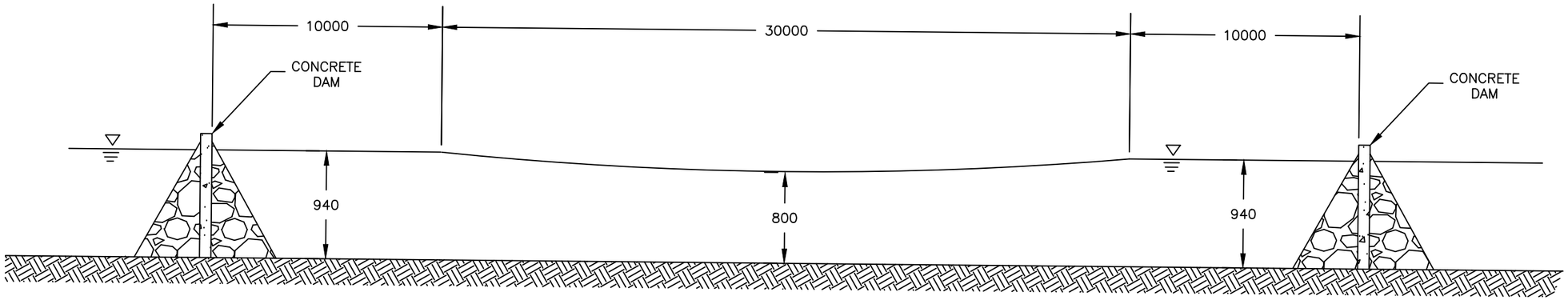


DAM SECTION SLOPED 0 DEGREES

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION SLOPED 0 DEGREES** SECTION **L-200-DAM SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-204**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

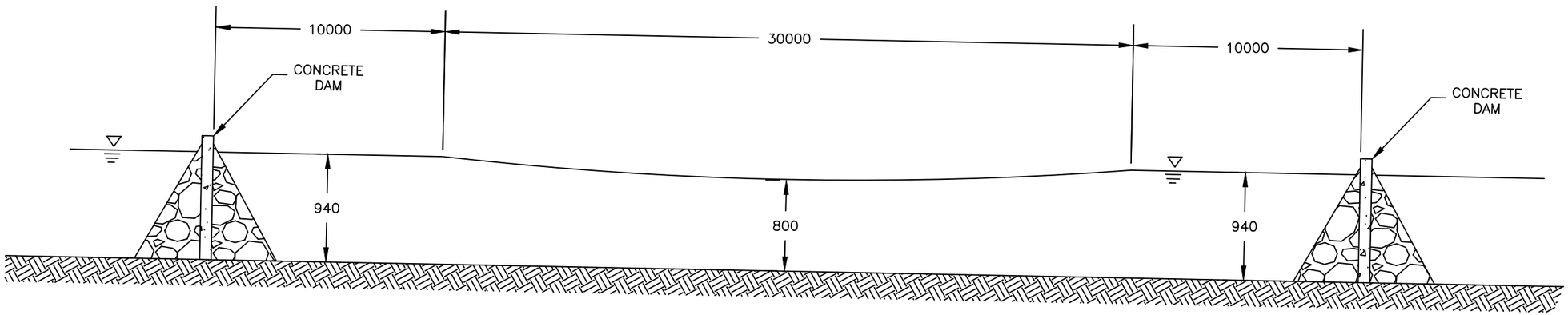


DAM SECTION SLOPED 1 DEGREE

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION SLOPED 0 DEGREES** SECTION **L-200-DAM SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-205**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

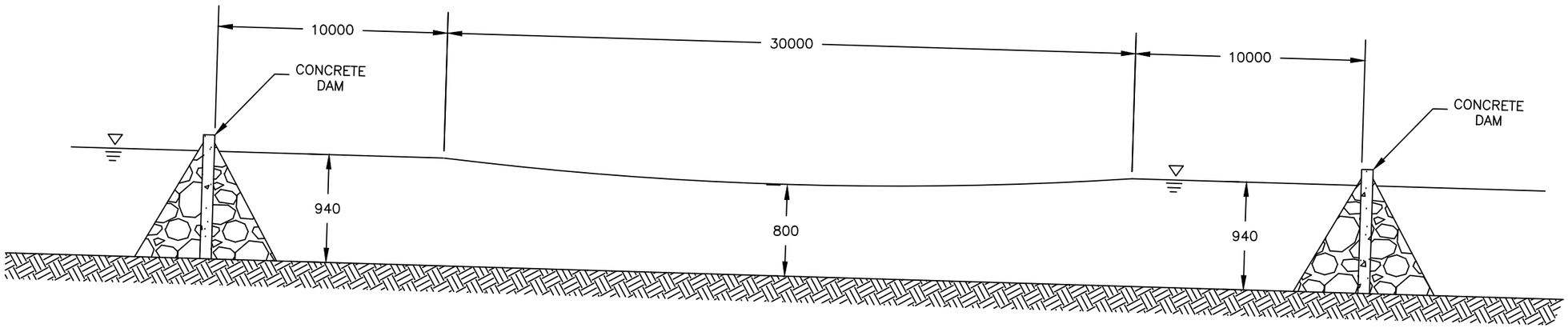


DAM SECTION SLOPED 2 DEGREES

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION SLOPED 0 DEGREES** SECTION **L-200-DAM SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-206**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION

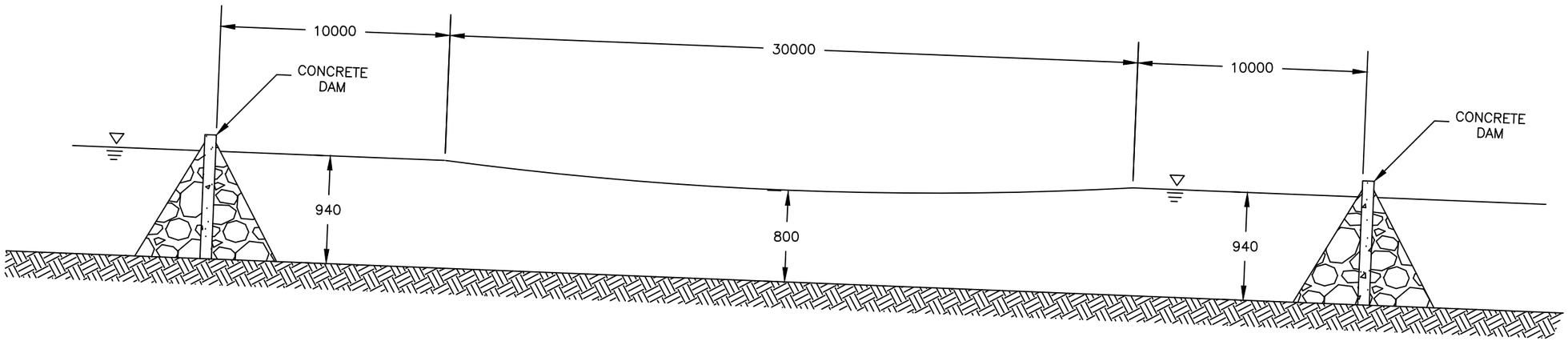


DAM SECTION SLOPED 3 DEGREES

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION SLOPED 0 DEGREES** SECTION **L-200-DAM SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-207**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION



DAM SECTION SLOPED 4 DEGREES

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION SLOPED 0 DEGREES** SECTION **L-200-DAM SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-208**

PRODUCED BY AN AUTODESK STUDENT VERSION

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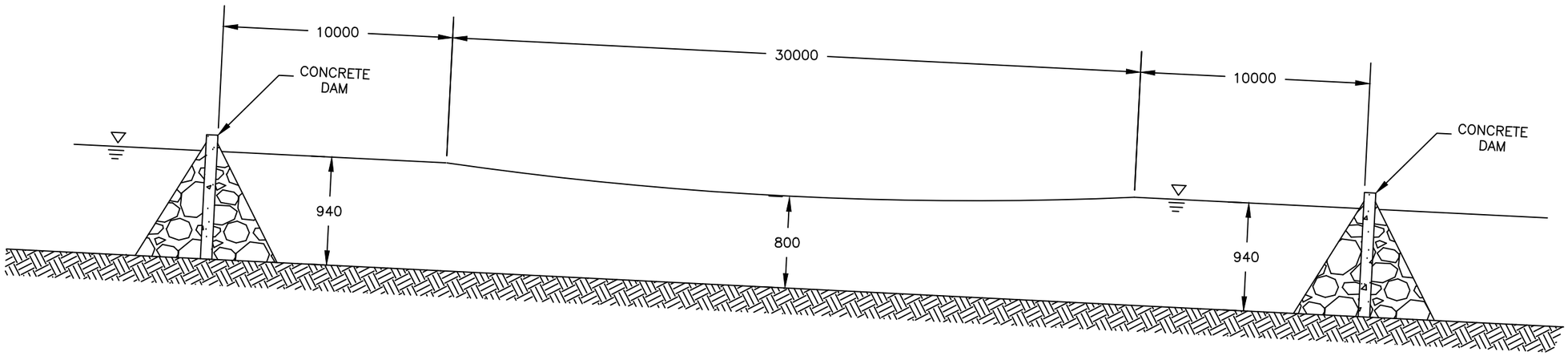
PRODUCED BY AN AUTODESK STUDENT VERSION

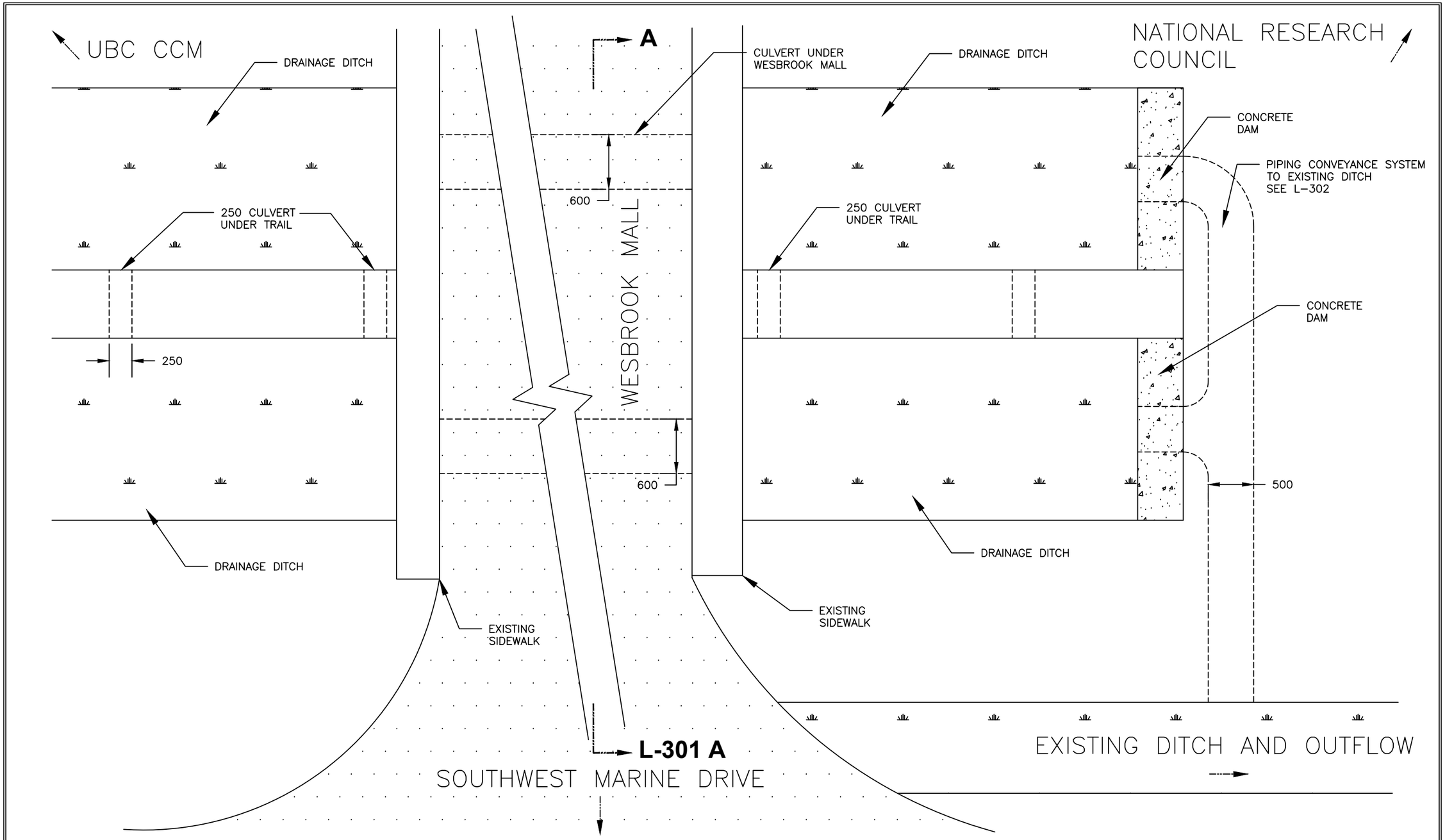


DAM SECTION SLOPED 5 DEGREES

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM SECTION SLOPED 0 DEGREES** SECTION **L-200-DAM SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-209**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION



DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **KEY COMPONENTS SITE PLAN** SECTION **L-300-SITE PLAN**	REVISION DATE **02/02/2019** **1:200**	DRAWING NO. **L-300**

PRODUCED BY AN AUTODESK STUDENT VERSION
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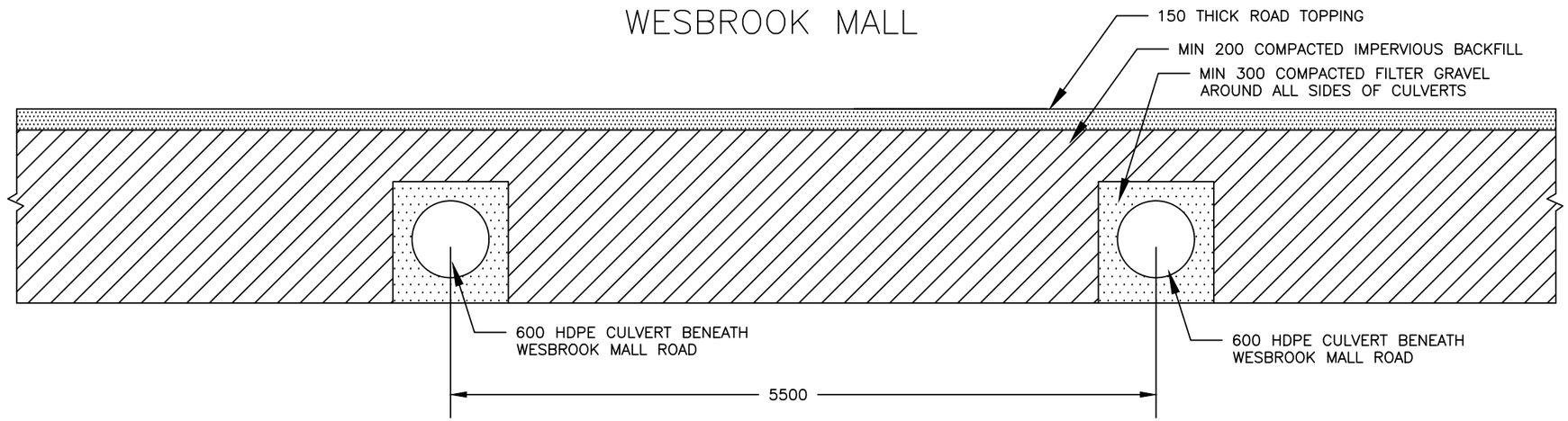
WESBROOK MALL



L-300 SECTION A-A

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **WESBROOK MALL CULVERT SECTION** SECTION **L-101-WESBROOK MALL SECTION**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-301**

PRODUCED BY AN AUTODESK STUDENT VERSION

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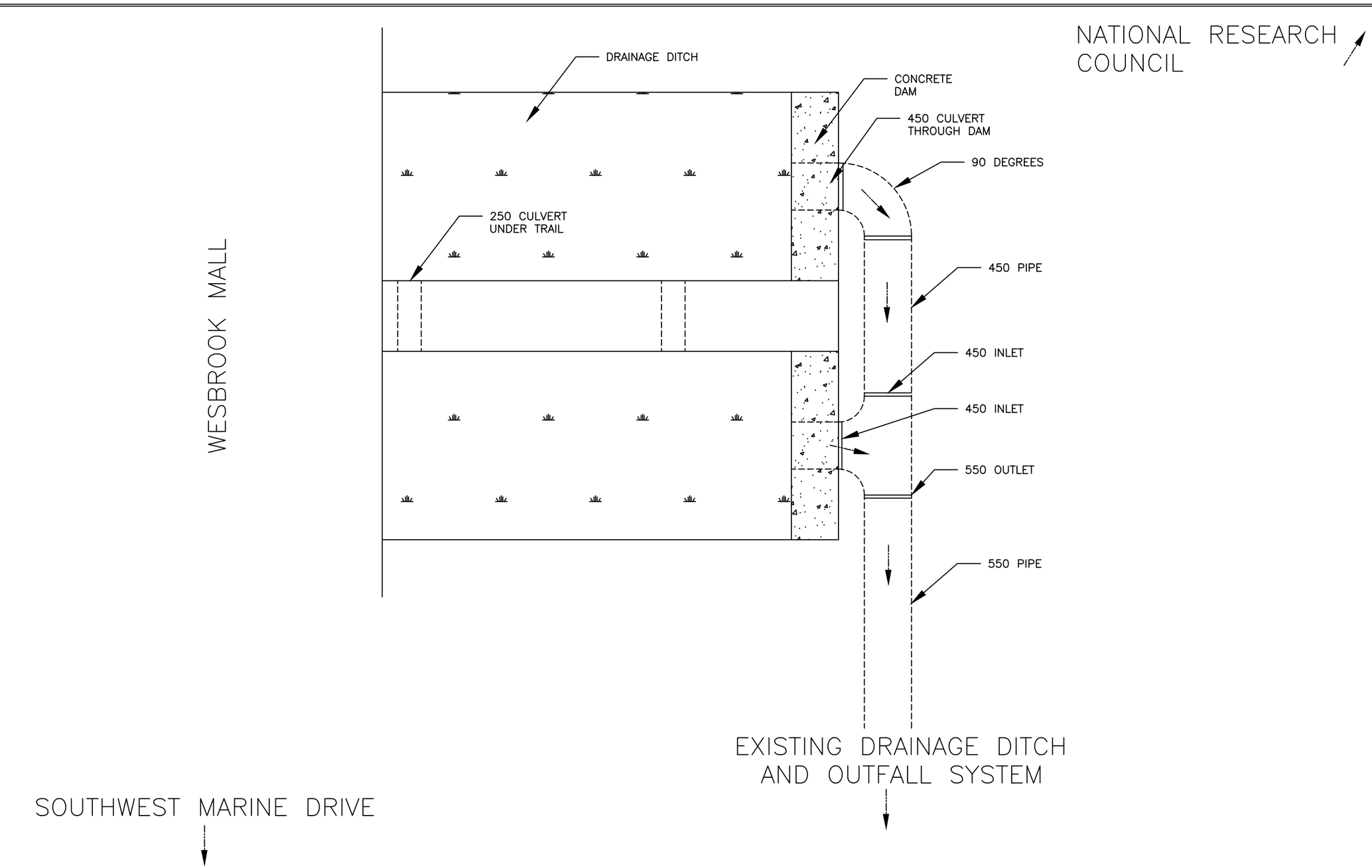
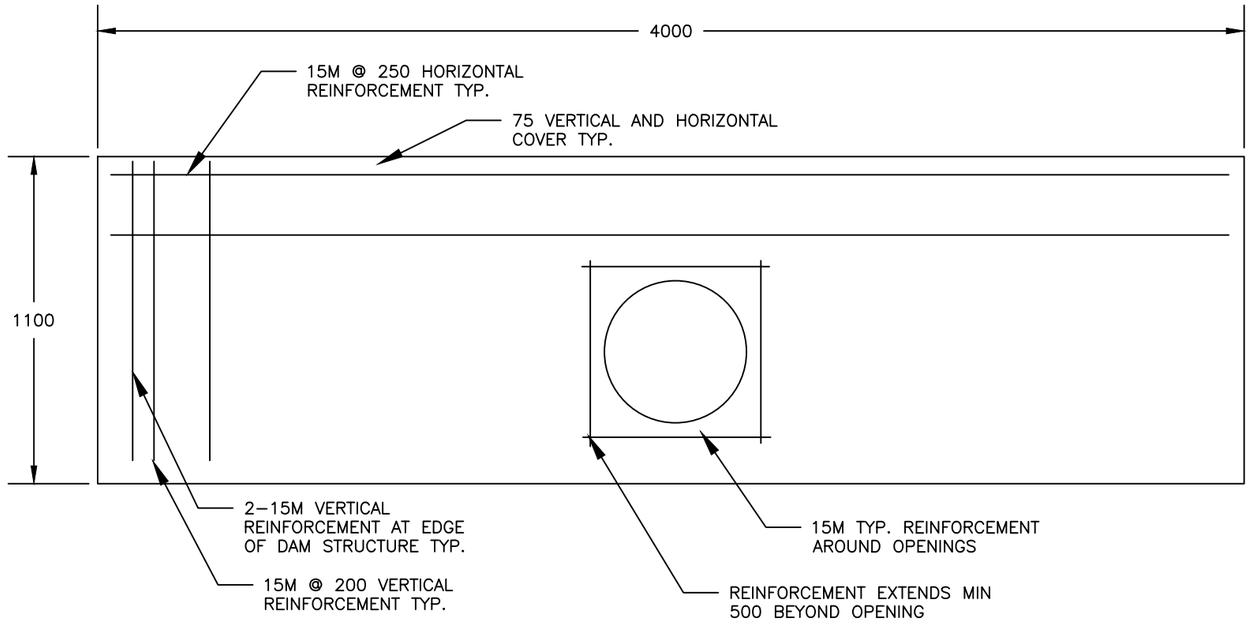
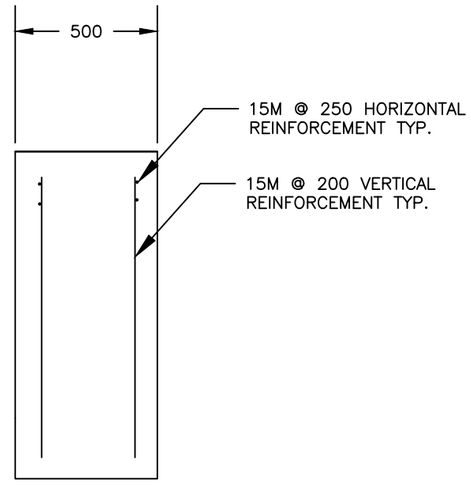


DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **OUTFALL CONNECTION DETAIL** SECTION **L-302-OUTFALL CONNECTION DETAIL**	REVISION DATE **02/02/2019** **1:100**	DRAWING NO. **L-302**

PRODUCED BY AN AUTODESK STUDENT VERSION

PRODUCED BY AN AUTODESK STUDENT VERSION





TYPICAL DAM SECTIONS

DRIFT CONSULTING **STANDARD TRAIL DESIGN**	PROJECT NAME & LOCATION **PROJECT II - MULTI USE STORM-WATER DETENTION CENTER** **UNIVERSITY OF BRITISH COLUMBIA CCM**	DRAWING NAME **DAM STRUCTURAL LAYOUT** SECTION **L-100-DAM STRUCTURAL LAYOUT**	REVISION DATE **02/02/2019** **1:50**	DRAWING NO. **S-100**

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