UBC Social Ecological Economic Development Studies (SEEDS) Sustainability Program

Student Research Report

Replacement of the Spiral Drain at the North End of UBC Campus

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University of British Columbia

CIVL 446

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UBC NORTH CAMPUS
SPIRAL DRAIN REPLACEMENT

Final Design Report

PREPARED FOR:

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Executive Summary

Vortex Consulting has prepared a detailed design report, as requested by UBC Social, Ecological, Economic Development Studies (SEEDS), for the replacement of UBC's current North Campus Stormwater Management facility, the spiral drain. This report intends to provide UBC SEEDS with an understanding of the design components, technical analysis and design, and project costs and construction sequencing, that are required to mitigate a 1 in 200 year storm event.

The main objective of this project is to design a replacement for the spiral drain structure that can withstand the rainfall loading of a 200 year storm. The potential for significant damage due to flooding in the event of a 1 in 200 year storm is significant, both to sensitive local ecosystems (ex. the cliffs near MOA), as well as UBC assets in the area. Due to the lifespan and design limitations of the existing spiral drain, a new replacement will be required for UBC north campus. Secondary objectives such as optimizing economics, sustainability and constructability are outlined in the report.

Vortex Consulting has established the following design solution with three main components; a Detention Tank that stores storm event rainfall and releases at a controlled rate, a Baffle Drop Structure to transport and dissipate the energy of water as travels 60 meters vertically, and a Horizontal Shaft and Outfall to transport water to the outfall located on Tower Beach. With a design life of 100 years, the initial capital costs are estimated to be $9,552,226 and the project will be completed by September 20th, 2017 with a May 1st, 2017 start date.

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1.0 Introduction

The University of British Columbia is planning to replace the spiral drain, a key piece of stormwater management infrastructure that is responsible for draining the entire north campus catchment area. The structure has approximately 30 to 70 years of service life remaining, and can currently handle a 70-year storm event. A solution is needed to upgrade the capacity to handle a 200-year return storm event and to mitigate the risk posed by an unexpected failure of the existing spiral drain. Vortex Consulting is responsible for developing a replacement system, and this report outlines our detailed design. The proposal consists of a detention tank to store water during storm events, a concrete shaft containing baffles to dissipate kinetic energy, and a horizontal pipe to transport stormwater from the baffle shaft to an outfall into the Strait of Georgia.

The team of 4th year civil engineering student responsible for the entirety of this design report, all have varying degrees of expertise in differing areas of civil engineering design. Thus, each members role and given responsibilities were selected based upon their given interests and specialities. The following table summarises the roles and contribution that each member made for the development of the report.

Team Member	**Contribution and Responsibilities**
Mona Dahir	Hydrotechnical Design Loadings, SWMM Modelling, Detention Tank Design, Environmental Impact
Jas Gill	CAD Construction Drawings, Revit, Cost Estimate
Danny Hsieh	Detention Tank Design, Sketchup Model

Rachel Jackson	Baffle Drop Structure Design, Construction Work Plan and Methodology Cost Estimate, Schedule
Michael Louws	Horizontal Shaft and Outfall Structure Design, Construction Work Plan and Methodology, Geological Assessment
Chris Vibe	Detention Tank Design, Schedule, Design Improvements

Table 1. Team Responsibilities and Contributions

2.0 Project Description

The following section is intended to provide a brief project background in regards to the North campus stormwater system, in which the basis of this report was founded. This includes a site overview, project scope and objectives, and key issues and constraints facing the design.

2.1 Project Scope and Objectives

The project objective is mainly to design an energy dissipation drainage structure that can handle the 200-year design storm in a cost-effective manner and in accordance with the SEEDS program vision. The method of approach requires expertise in several knowledge areas and only the most critical design considerations are considered. It was assumed that the developed design should replace the spiral drain in a proximate area and most of the conceptual design proposals were therefore focused on managing stormwater from the existing piping network and catchments, as shown below in Figure 1.

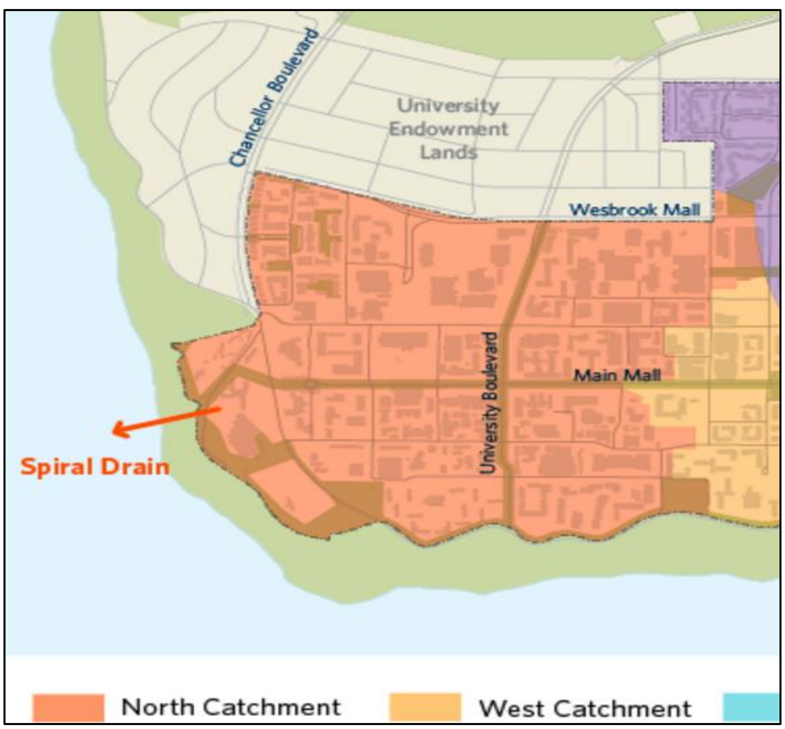

Figure 1. UBC North Campus Stormwater Drainage Map (Source: UBC ISMP)

2.2 Site Overview and Hydrological Background

The site of the replacement structure will be near the Museum of Anthropology, in close proximity to the existing spiral drain. It is proposed that the new drainage structure will connect to existing stormwater infrastructure to reduce additional costs and lessen the impact on other local infrastructure (i.e. electrical utilities, gas lines, sanitary and water lines). With respect to property lines, the construction will mainly occur in the University Endowment Lands. However, the outfall structure will exit via Wreck Beach, which is managed by Metro Vancouver.

2.3 Key Issues & Constraints

The design of the spiral drain replacement was mostly dependant on the following key issues and constraints:

- North campus catchment area and pipe network was assumed to remain the same
- Proximity of site to the cliffs and soil conditions
- Cliff instability and sensitive habitat lead to a constraint of a non-intrusive design

3.0 Design Overview and Specifications

This section of the report summarizes the replacement design solution, and provides an overview of the design components, including a general layout, and individual component function, design, and specifications.

3.1 Layout and Design Components

The design consists of three main components: the detention tank, the baffle drop structure, and the horizontal shaft and outfall pipe. The detention tank will store water during storm events. The baffle drop structure is designed to dissipate kinetic energy as water flows through the structure. The horizontal shaft will then transport stormwater from the baffle shaft to an outfall into the Strait of Georgia. An overall design layout is shown below in Figure 2.

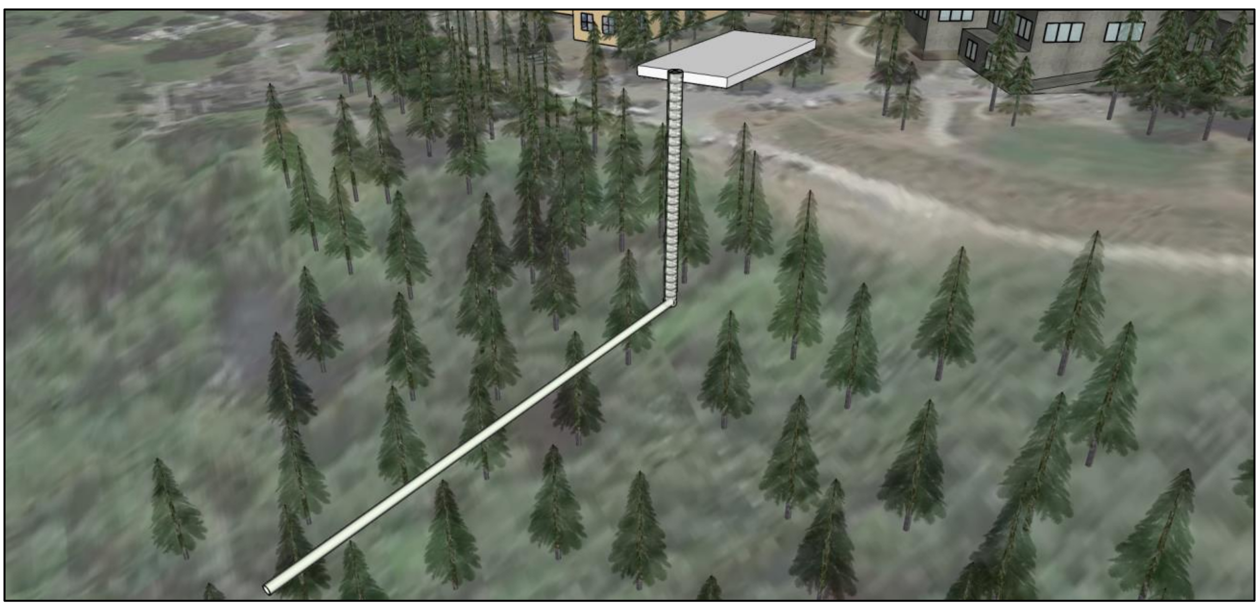

Figure 2. Overall 3D Design Layout (Source: Danny Hsieh 2017)

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3.2 Detention Tank

The detention tank is a subterranean concrete structure adjacent to the baffle structure, shown below in Figure 3. The purpose of the detention tank is to temporarily store water when the flow rate exceeds the capacity of the baffle structure. Treatment technology is still being considered, however, a document released by UBC Planning department indicates that stormwater quality is currently not an issue at UBC. The structure is separated into three main components: the top slab which carries the load of the earth above it, the walls exposed to horizontal earth pressures, and the bottom slab which carries the combined loading of the structures overtop of it as well as the volume of earth above it. All of the structures were analyzed as one way slabs. The top slab was designed to have a total depth of 600mm with compression and tension rebar. The compression rebar was 35M at 130mm spacing and the tension rebar was 35M at 150mm spacing. The wall was designed to have a total depth of 350mm with 15M tension rebar at 500mm spacing. The bottom slab is a replication of the top slab design for redundancy.

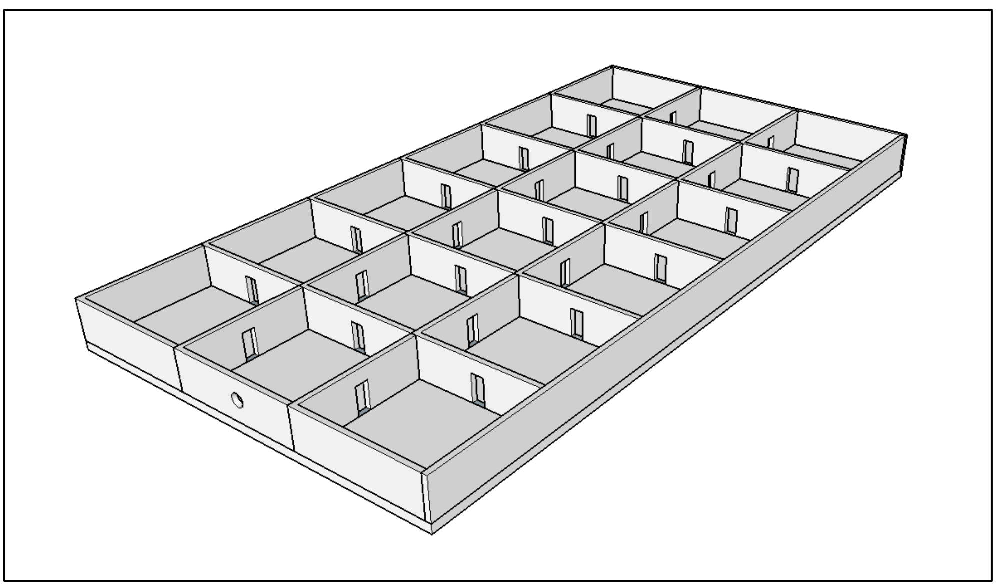

Figure 3. 3D Detention Tank Component (Source: Danny Hsieh 2017)

In reality, the boundary condition on the bottom slab allows for a thinner slab and less reinforcement. This is based on the assumption that the bottom slab would face only a crushing load due to a rigid soil support, effectively avoiding flexure. Piles will also be installed underneath the bottom slab to ensure a rigid soil boundary condition. The current piping that connects to the existing spiral drain will be rerouted to the baffle drop structure. The slope at which the new piping will be installed at will be similar to the old pipes. In addition, there will be consideration to install thrust blocks at elbows where applicable.

3.3 Baffle Drop Structure

The baffle-drop structure is a 3.7 m outer-diameter concrete shaft used to transport the stormwater 60 meters vertically from the detention tank to the horizontal outfall shaft. It contains a vertical wall dividing the structure into a wet side and dry side, two-thirds and one-third the inside diameter respectively, as shown in Figure 4. Slightly overlapped baffles, positioned on the wet side at a spacing of 1.55 meters, limit the flows' velocity and potential erosion consequences. The dry side is open, and contains windows to the wet side for potential surge mitigation and de-aeration. It also acts as an access shaft with pre-installed cast-iron ladder rungs for maintenance. The shaft will be constructed of 4 meter high pre-cast concrete segments; reinforced with 25M longitudinal and hoop rebar, encased in a permanent steel pile casing and bonded together with concrete as installed.

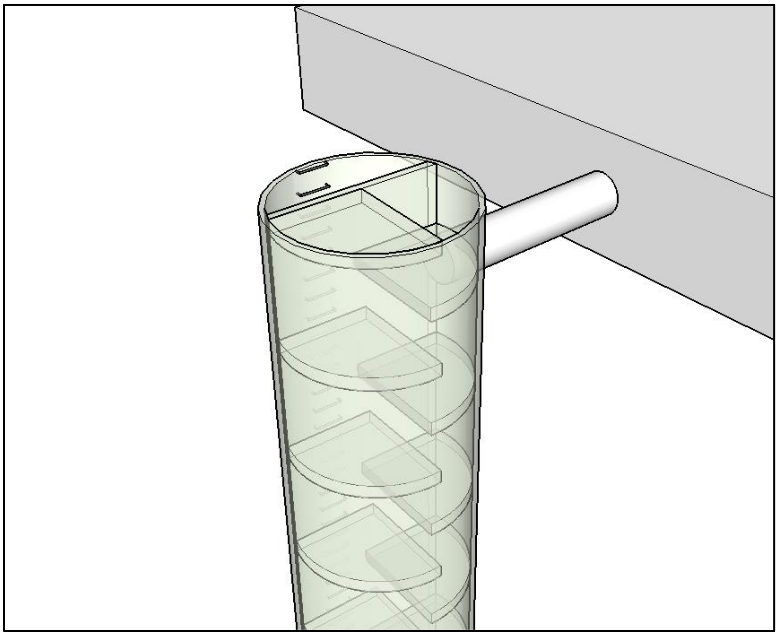

Figure 4. 3D Baffle Drop Structure Design Component (Source: Danny Hsieh 2017)

3.4 Horizontal Shaft & Outfall Pipe

The horizontal shaft design depends on soil conditions, hydraulic demand and construction methodology. The diameter of the pipe for maximum flow demand is estimated to be 1.2m. Vertical earth loads are found using the Indirect Method, and resulted in a dead load of 52kN/m. The method of construction chosen for the horizontal shaft is the Microtunnelling and Jacking Method. This results in significant axial force demand on the pipe, which is found to be 8.3kN. The pipe will be pre-cast steel reinforced concrete pipe (SRCP), which will be designed according to CSA 257.2. Design specifications for jacked SRCP were taken from DECAST Ltd. Microtunneling Pipe brochure (DECAST, 2017).

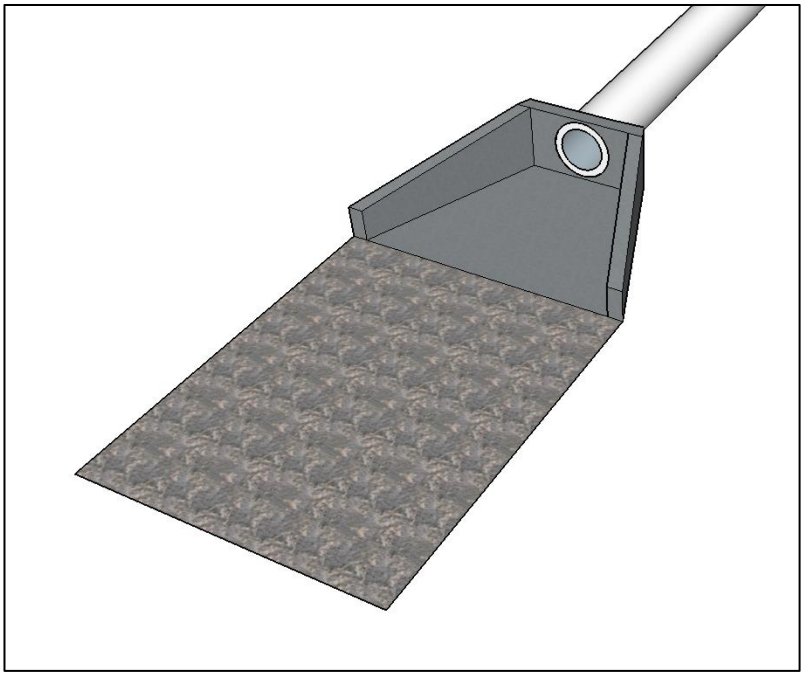

Figure 5. 3D Horizontal Shaft and Outfall Design Component (Source: Danny Hsieh 2017)

The outfall structure design is based on the outflow velocity and sea level at the outfall location. Since the water will be leaving at a relatively high velocity, a riprap apron will be constructed with a length and width of 5 and 6.5meters respectively. The outfall structure will lie slightly above the existing beach in order to have the end of the horizontal shaft ending above the highest maximum projected sea level at high tide. A cover of large aggregate and rock will be built above the outfall structure in order to protect it from weathering. The concrete of the outfall structure was designed according to CSA standards for concrete class C-1 (concrete in saline water). Detailed design parameters and calculations can be found in Appendix F.3.

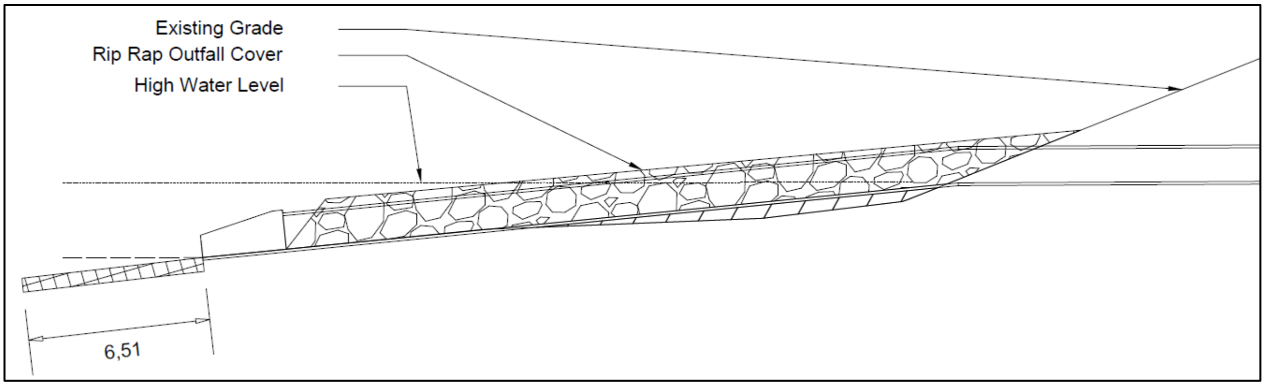

Figure 6. Cross-Section of Outfall Elevation (Source: Michael Louws, 2017)

3.5 Stormwater Re-routing

Stormwater will be re-routed to connect to the detention tank. Pipe sizes were not changed, as to save costs since not all of the existing pipes will be removed. The pipes will be spliced at certain areas and reconnected to the system when possible. In case where the pipes need to be reconstructed, the same types of pipes will be used as they are able to withstand the design flow loads. According to the Surrey Design Criteria, all of the new slopes for the pipes are above the minimum slope (0.1%) and below the maximum (15%) which would require soil anchors. Figure 7 below shows a detailed site layout and details of the pipe invert elevations and slope grades.

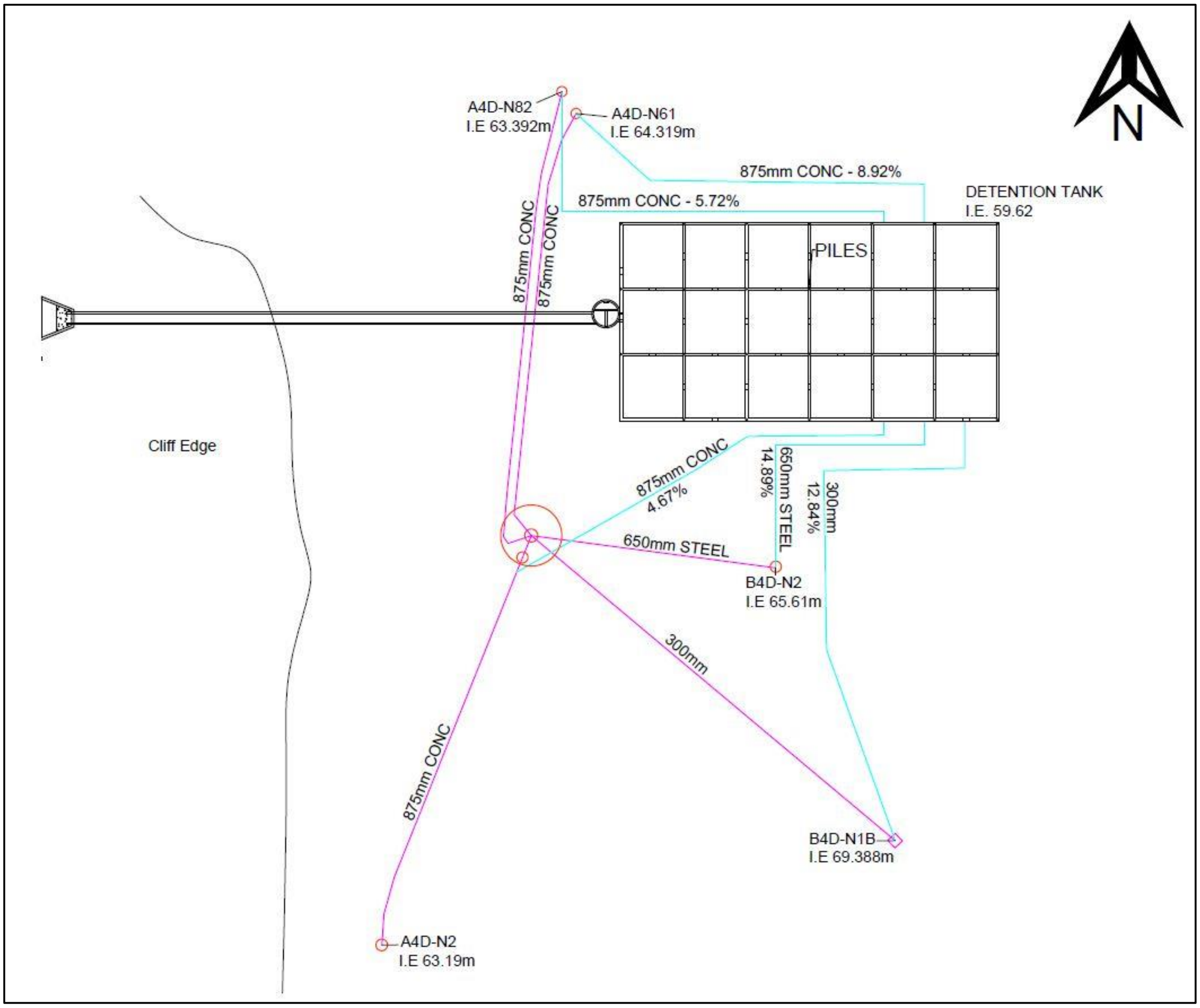

Figure 7: Stormwater Re-routing Diagram*(Source: Jas Gill 2017)*

4.0 Design Inputs

The following section summarizes the adopted design life, design loading conditions, technical design considerations, and standards used. These were developed on the basis of mitigating a 1 in 200 year storm event.

4.1 Adopted Design Life & Maintenance

The design life of the proposed energy dissipation structure is 100 years of service. Maintenance of the structure should be relatively infrequent. It is recommended, however, to closely monitor the structure in the first two years of operation for early detection any unanticipated issues. Examples include issues such as excessive soil consolidation or erosion of the riprap bedding at the outfall of the horizontal discharge pipe.

4.2 Design Loads

Based on statistical analysis for a 200 year storm event, the maximum flow demand was found to be $5.49 m^3/s$ into the system. The flow into the baffle structure was determined to be $3.98 m^3/s$ and was based off of a spreadsheet that sized both the baffle structure and the storage tank. The design life for the structures is 100 years.

System Demand	**Standard**	**Method/Software**
Hydrotechnical		
Hydrology Methods	Storm Water Management Model Reference Manual Volume I	EPA SWMM, IDF curve statistical analysis of Precipitation Data
Hydraulic analysis of shaft	Journal of Hydraulic Engineering, Baffle Drop Structure Hydraulic Analysis	
Hydraulic analysis of pipe	Federal Highways Administration, Theory and Design Calculations for Inlet	Analysis of flow in Unsubmerged inlet/outlet culvert

Table 2. Hydrotechnical System Demands and Standards

4.3 Technical Design Parameters & Standards

Upon determining the design loading flow and proposed design solution, the following technical parameters were considered in the design solution for each component. For each parameter, a standard or technical guideline was referenced.

Design Parameter	**Standards/Guidelines**
Geotechnical	
Lateral Earth pressure	Budhu, Soil Mechanics and Foundations
Pile Capacity	Budhu, Soil Mechanics and Foundations
Vertical Earth Pressure on Tunnelled and Jacked Pipes	Ontario Concrete Pipe Association Design Manual
Materials	
Chloride exposure concrete for outfall	CSA A23.2
Concrete Reinforcement	CSA A23.1
Vertical pipe concrete	CSA A23.3-04
Detention tank concrete	CSA A23.1/23.2
Structural	
Pre-cast Reinforced Concrete Pipe	CSA A257.2
Axial Loading due to Jacking	American Concrete Pipe Association, Design Data 4: Jacked Concrete Pipe
Detention Tank Concrete Reinforcement Design Baffle Drop Shaft Concrete Reinforcement Design	Svetlana Brzev, Reinforced Concrete Structures: A Practical Approach
Construction Methods	
Microtunnelling and Jacking	Ontario Concrete Pipe Association Design Manual
Vertical drilling	Ontario Health and Safety, Construction Part IV Tunnel Shafts Caissons and Cofferdams
Hydrotechnical	
Sea level rise	NASA Sea Level Satellite Data
Outfall Scour Protection Stormwater Pipe Velocities	Auckland Council, Inlet and Outlet Design City of Surrey Design Criteria

Table 3. Technical Design Parameters and Standards

4.4 Software Package

Following the manual technical design calculations, the following software’s were utilized to model of verify design solutions.

4.4.1 EPA-SWMM

To complete an estimate of the design flow for the 1 in 200 year storm event, the US Environmental Protection Agency's Storm Water Management Model program (SWMM) was used. SWMM is a hydraulic simulation software used for planning, analysis and design related to pipe distribution systems (Storm Water, 2016). EPA-SWMM is one of the leading software's used for modeling stormwater systems. In this project, EPA-SWMM was also used to model the new stormwater infrastructure (detention tank and baffle structures) for the 200 year storm event and provide a comparison between the existing system and the proposed system that includes the new structures and stormwater routing.

4.4.2 AutoCAD, Revit and Sketchup

AutoCAD is a drafting program widely used in industry. IT was utilized to detail the overall dimensions of the design and particular components (baffles, wall thicknesses).

Google SketchUp is a 3D modelling program used to develop conceptual models. SketchUp was primarily used in the conceptual stage of the project, to give an overall visual representation in the early stages.

4.4.3 SAP2000

SAP2000 is a structural analysis program developed by CSI and was primarily used in the modelling of the detention tank. Particularly isolated cases were modelled. As such, the top slab was modelled with our design dimensions and parameters to verify the behaviour of our detention tank.

4.4.4 Plaxis 2D

Plaxis 2D is a standard software used in finite element analysis of soils. It is commonly used in foundation, pile, and tunnel design. For the purpose of our design, Plaxis 2D was used to model the stresses in the horizontal shaft. Significant limitations existed though because the only version of the software available freely limited the number of soils conditions and phases that could be used. Thus the output from this program was not included in the final report.

5.0 Technical Design and Analysis

In order to develop a design that would meet regulatory standards and requirements, technical design and analysis was completed, focusing on aspects and components of hydrotechnical, structural, geotechnical, environmental and constructability assessment. To ensure the safety and loading of the design, critical calculations relevant to the project were completed. Full detailed calculations can be found in Appendix E and F.

5.1 Hydrotechnical

5.1.1 Design Storm

To complete an estimate of the design flow for the 1 in 200 year storm event, the US Environmental Protection Agency's Storm Water Management Model program (SWMM) was used. SWMM is a hydraulic simulation software is used for planning, analysis and design related to pipe distribution systems (Storm Water, 2016). A SWMM model of UBC was provided by the client, which included a 24 hour design storm based on a 100 year return period.

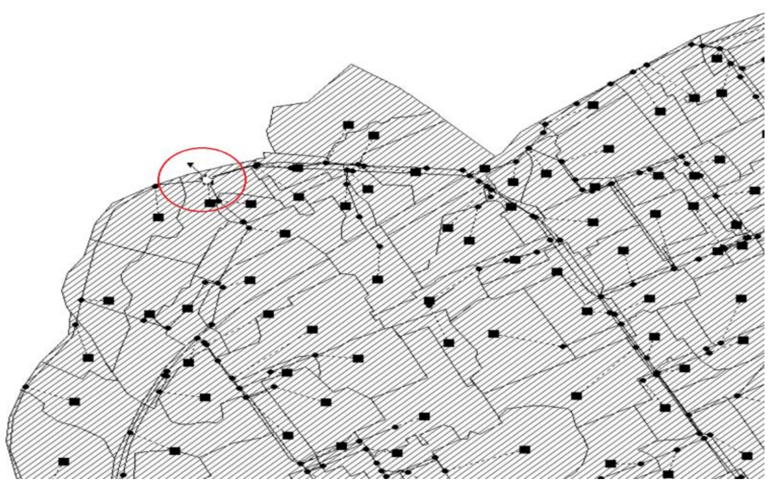

Figure 8. SWMM Model for North UBC Campus, Spiral Drain location in red circle. (Source: UBC Storm Model, 2016)

5.1.2 SWMM Modelling

For the purposes of our analysis in SWMM, the initial assumption was that the adjacent nodes in the network that exhibited flooding under 200 year storm conditions were scaled up to prevent flooding and ensure the maximum design flow was obtained at the location of the drain. Under the two hundred year flow, there is flooding in the nodes surrounding the existing structure. A comparison between existing conditions and future conditions will be completed in section 9.1.

5.1.3 System Volume and Flow Demands

Two methods were applied in order to estimate the total flow demand. The first method scaled up the 1 in 100 year storm by a factor of 1.1 to acquire the 1 in 200 year storm event. The second method considered precipitation for YVR Airport, conducting standard statistical analysis to determine a composite 200 year design storm. Full methodologies can be found in Appendix E. The flow coming through the spiral drain node for the 1 in 200 year storm is between approximately 5.49 and 5.64m^3/s (without detention tanks). Based on these maximum flow volume demands, the sizing of the baffle drop structure and the horizontal outfall shaft were determined. These calculations are summarized in Appendices E and F. The outfall pipe and baffle shaft internal diameters were found to be 1.2 and 3.7 meters respectively.

5.2 Structural

All structural components were designed in accordance to the following design guidelines and references:

- Canadian Standards Association (Design of Concrete Structures)
- National Building Code of Canada (NBCC 2010)

- Reinforced Concrete Design: A Practical Approach (Brzev)

SAP2000 was primarily used as the modelling tool for the analysis of the detention tank. Gravity loads, seismic loads, and their respective load combinations were determined according to the NBCC 2010. Any concrete structural elements such as beams, slabs, walls, columns, compression shafts were also analyzed in accordance to the CSA guidelines and Brzev's textbook. Detailed structural reinforcement calculations can be found in Appendix F, under each respective component.

5.3 Geotechnical

In an attempt to understand the underlying ground conditions and anticipated soil conditions, significant interpolation and analysis was completed utilizing the geotechnical information provided surrounding the UBC north campus cliffs.

5.3.1 Site Description

The site of the proposed Drainage structure lies in the north end of the UBC campus. Historically, the region was heavily treed, until the early 1920s when logging operations contributed to the erosion scars seen on the cliff face near the Museum of Anthropology. Construction on the north end of the ubc campus near the location of the cliffs intensified from 1945 to 1958, with a second wave of development happening in the 1960's and 1970's (Piteau, 2002).

Surface drainage from the north end of UBC campus is conveyed to the spiral drain, where it drops and is deposited into the Georgia Straight. The exposed cliffs allow for observation of soil and groundwater conditions near to the cliffs. Observation of the cliff faces near MoA indicates the presence of thick deposits of Quadra Sands (Q1) overlaying relatively fine-grained sand and silt (Q2). Within the overlaying Quadra sand, thin layers of silty sand and silt are present. The largest portion of groundwater seepage is located at the boundary between the lower sand/silt layer and the Quadra Sand.

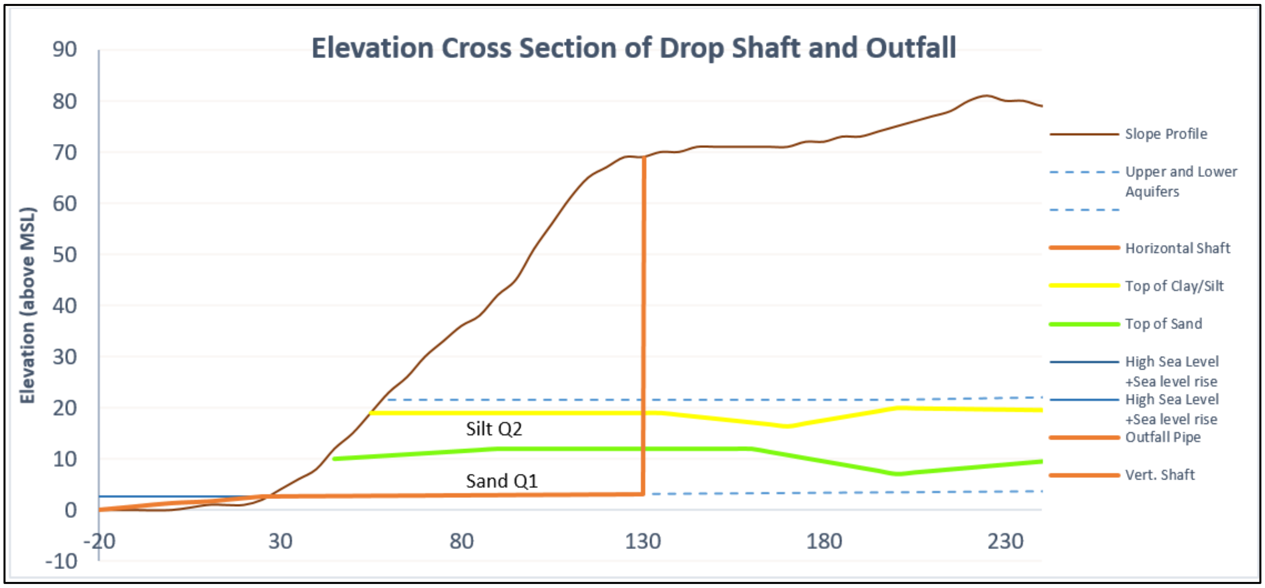

Figure 9. Elevation Cross Section (Source: Michael Louws 2017)

5.3.2 Area Geology and Subsoil Conditions

The geological history of the region is a result of the previous glaciation and erosion patterns. Sediments, both sandy, clayey, and silty, deposited from approximately 50,000 to 20,000 years ago were compressed during the last ice age. When the glaciation left the region, leaving the sands and silts to rebound, in some places by as much as 60 meters. The eroded profile of this rebounding is now visible at some of the exposed cliff faces, particularly in our location of interest.

The basic profile of the sedimentary layers has the following approximate profile:

- Silt-Clay Layer: beginning at the base of the cliffs this unit consists of layers of clay interspersed with lenses of sand and organics.
- Sand Layer: beginning above the clay layer is a younger sand layer that has a cross-bedded direction as a the result of the ancient Fraser River continuously changing direction.

- Glacial Deposit layer: this layer had a mixture of dense sands, silt and clay with occasional boulders and gravel, resulting from the end of the glaciation in the region.

Soil Unit	Density (kg/m^3)	Cohesion (kPA)	Friction angle (deg.)	Shear modulus (MPa)	Bulk modulus (kPa)
SAND top 5 m dense to very dense	2000	0	38	200	2000
SAND very dense	2080	0	44	260	2600
SILT inter-layered	1900	200 to 600	0	100 to 300	1000 to 3000

Figure 10. Soil Parameters (Source: Budhu 2017)

5.3.3 Design Groundwater level

The design groundwater level is determined by the location of the lower of two aquifers that exist in the Point Grey Peninsula. At the upper level of the upper Clay layer (Q2) occurring around 21.5 meters above MSL, seepage was observed. This seepage is due to the upper aquifer which lies above the clay layer. Some of the water percolates through the semipermeable layer of clay and then through the layer of sand-silt below, eventually reaching the lower aquifer which lies on top of the clay aquiclude below sea level. The level of the groundwater table at shore is approximately 2 meters above MSL (close to the level of high water). This lower aquifer is the groundwater level for consideration in design of the horizontal drainage pipe.

5.3.4 Slope Stability

The Piteau Associates report includes a seismic slope stability analysis conducted by TROW Consulting Engineers Ltd. It makes use of the dynamic modelling software FLAC, in order to simulate the earthquake loading conditions on the cliffs. Under this analysis, a recommended setback of 25 meters is found, up which the location of the detention tank design is based.

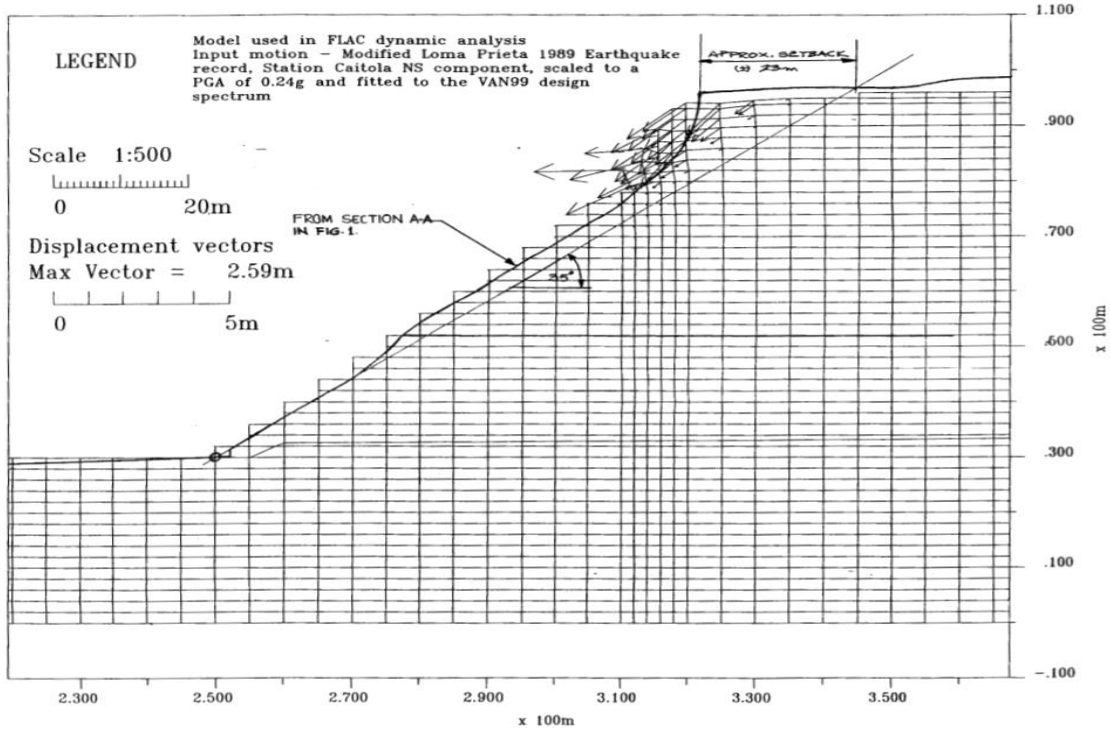

Figure 11. FLAC Model with recommended setback for all structures near cliff face (Source: Piteau Associates, 2002)

5.3.5 Recommendations for Site Investigation

While the data currently available in the Piteau Associates *Hydrogeological and Geotechnical Assessment of Northwest Area UBC Campus* is quite comprehensive, it lacks in a couple areas: soil laboratory testing, and tower beach conditions. The following are recommended as further investigation for detailed design:

- New borehole within 5 meter radius of proposed baffle drain shaft
- Detailed laboratory soil testing for determination of soil parameters for Plaxis 2D and 3D modelling
- Tower beach test hole for design of jacking structure

This knowledge will provide accurate parameters to assist in designing a system that can withstand all failure modes, and predict immediate and elastic settlements. UBC, as well as the surrounding Vancouver area, is also prone to earthquake activity. Further investigation is critical in the next stages of developing a detailed design solution, selecting the right method of construction, and managing any associated risks.

Soil calculations in this report represent the conservative calculations based on the best available geotechnical data, standards, and software. Calculations can be found in Appendix F for individual component geotechnical calculations.

5.4 Environmental Impact

There are several regulatory provisions that the project design must adhere to. It should be noted that in the regulatory provision documents there is not a typical standard for stormwater quality and discharge. Rather, the approach to stormwater management is through best management practices. The following regulations, plans, and guidelines, will be followed for the discharge of stormwater:

- 2014 UBC Draft Integrated Stormwater Management Plan
- Metro Vancouver's Integrated Liquid Waste Management and Resource Plan
- British Columbia's Water Quality Guidelines
- Canadian Environmental Protection Act
- Fisheries' Act - Wastewater System Effluent Regulations

UBC Draft Integrated Stormwater Management Plan

The 2014 UBC Draft Integrated Stormwater Management Plan (ISMP) is a document that was created to manage UBC's stormwater. It outlines key objectives concerning flooding, campus values, discharge impact, quality, and incorporating the natural hydrologic cycle. In addition, the plan goes over current stormwater practices, the changing land use, and future actions that need to be taken to effectively meet the objectives of the ISMP.

Metro Vancouver's Integrated Liquid Waste Management and Resource Plan

The Metro Vancouver's Integrated Liquid Waste Management and Resource Plan (2010) is a document that outlines a plan to:

- Protect Public Health and the Environment
- Use Liquid Waste as a Resource
- Achieve effective, affordable and collaborative management

This plan seeks to achieve the above-mentioned goals by establishing collaboration with organizations (municipalities) to implement specific actions for wastewater collection and treatment. More importantly, it also specifies strategies and actions for stormwater management. Although UBC is not a municipality, UBC will follow this plan as it has similar functions in terms of stormwater management.

British Columbia’s Water Quality Guidelines

The BC Water Quality Guidelines set certain limits for deleterious substances that may harm aquatic life, wildlife, agriculture drinking water, and recreation. In this case, the quality guidelines set for aquatic life will be observed.

Canadian Environmental Protection Act

The Canadian Environmental Protection Act is a federal document that sets standards to regulate the disposal of waste, including ocean dumping. Although the act is more focused on solid waste, the act is still relevant to the project, as it meets all applicable standards.

Fisheries Act - Wastewater System Effluent Regulations

The Fisheries Act is a federal document created for the protection of fish, fish habitats, and human health. The Act sets standards for discharge into water bodies inhabited by fish species. The project will incorporate the Act.

5.4.1 Water Quality

In UBC's ISMP, it is suggested that as the campus develops further that there will be more opportunities to implement more water quality techniques. Oil-grit separators are a big part of what may come in the future, and in the case of this design, it is suggested that they be installed in the future further upstream. In the proposed design, there are five pipes collecting the flow from the north catchment to the detention tank, which is a large volume of water. Further study needs to be completed to see if oil-grit separators are feasible at those locations as it is possible that the separator may be bypassed frequently due to large heavy flows.

5.4.2 Erosion Control

Erosion control is accounted for in the design of the outfall shaft. An armoured apron is implemented into the design of the outfall outlet in order to prevent erosion. See Appendix A for more details on erosion protection.

5.5 Constructability of Design

The following section review the constructability of the three components, baffle drop shaft, horizontal outfall, and detention tank, primarily focusses on the material selection and fabrication techniques.

5.5.1 Baffle Drop Shaft

The baffle drop shaft required high constructability considering the lateral soil pressures acting during construction and limited installation space inside the permanent steel casing. By pre-fabricating the concrete shaft segments construction and onsite construction was reduced considerably. The design also includes a “dry side” on one third of the shaft diameter to facilitate access for both construction and maintenance.

5.5.2 Horizontal Shaft

Constructability was a key consideration in the design of the horizontal outfall and shaft. Site access on both side limited the options. Since access to the site is inherently limited (only via Tower Beach), movement of materials and equipment is minimized. On the MOA side of the site, access for the horizontal shaft is only through the 65 meter vertically drilled shaft. Soft unstable slopes also imposed a possible limitation on site disturbance on the Tower Beach side. Based on these requirements, Microtunnelling was chosen as the method of construction, with the main entry point being Tower Beach, and the exit point being through the vertical shaft.

5.5.3 Detention Tank

In terms of the detention tank construction, large consideration was given to determining whether cast-in-place slabs and walls would be a better option than pre-fabricated components. Access was determined to not be an issue, and transporting large 8 meter by 3m pre-fabricated units to site provided more concern. With the limited information regarding soil conditions, soil anchors were not included in the design, however, cast in place walls opposed to pre-cast units would allow for flexibility in installation. Therefore, cast-in-place walls were determined to be the better of the two material fabrication options.

6.0 Construction Drawings and Plans

Issued for construction drawings and specifications are available for all design components, and can be found in Appendix B. The drawings were created utilizing dimensional and specifications determined from technical design and analysis.

7.0 Construction Work Plan

This section of the report is intended to reflect the schedule found in section 10. The construction work plan outlines the methodology for construction, sequencing of construction, and any anticipated risks or site issues.

7.1 Construction Methodology

The following section outlines the construction methodology for each key design component.

7.1.1 Detention Tank

At site, the groundwater table is well below the depth of excavation (5 to 10 meters respectively) thus it does not come into contact with the foundation slab or walls. It is thus recommended to use steel sheet piling as an alternative retention system in this case. By installing interlocked sheet piles in sequence to a pre-calculated design depth below the bottom of basement excavation, a temporary or permanent wall is formed. If required, ground anchors can also be added for additional lateral support. This technique is relatively cost effective and easily installed by hammering them into the till and sand layers. Caution should be made as there may be adverse effects such as noise and disturbance due to vibration to the neighbouring infrastructure. Changes to the water table depth should be monitored using a piezometer.

7.1.2 Baffle Drop Shaft

The baffle drop shaft construction, 4 meters in diameter will require immense planning and accuracy. To ensure the shaft remains stable during auguring, a permanent hollow steel casing will be oscillated or driven in prior. Due to sensitivity of adjacent cliffs, oscillating the casing in is recommended. This entails a starter casing with cutting teeth be utilized, and rotated in a single direction. Opposed to welding or splicing the casing segments, they are bolted on to resist torsional loads imposed from rotation. Auguring of the shaft will require a 4 meter diameter rotary drill rig, where special provisions may require a custom rig be fabricated. The casing will then also act as a friction pile resisting vertical and lateral loads. Baffle Segments will be lowered in by crane, placed and sealed using concrete grout. Due to shaft impeding on high water level, careful attention will be required to ensure shaft is de-watered at all times, yet does not impact groundwater levels.

7.1.3 Horizontal Shaft

Site Preparation and Layout

An area of roughly 256m^2 is suggested as a minimum staging area for the entry working space of the MTBM work site. This is based on the assumption of an entry shaft, which will not be needed for this application.

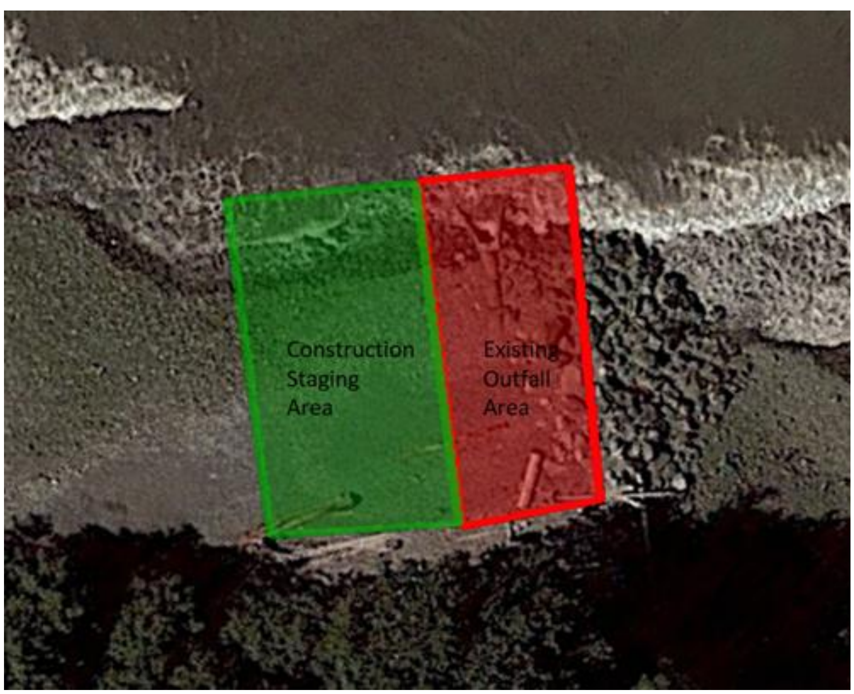

Figure 12 Construction Staging Area, Approximately 300m^2 (Source: Michael Louws, 2017)

In order to protect the work site during Microtunnelling from tidal surges and waves, a temporary cofferdam will be employed around the perimeter of the Construction Staging Area. This cofferdam structure will be constructed from driven sheet piles. Subject to further environmental assessments and application through the Department of Fisheries and Oceans, access to the staging area will be via Spanish Banks. Machinery access will be limited to during low tide. Alternatively access may occur via barge. Steps will be taken to minimize environmental impacts, by limiting the transport of machinery and construction materials over the beach.

Site staging area should be sufficient to accommodate the following equipment and materials (ASTM 36-15, 2015); Control Room, Power Source, Lubrication system, Pipe storage, Lifting Equipment, Slurry separation, Temporary storage of muck.

Shaft Construction Methods

In order to calculate design loading and consequently materials for the pipe, the method of construction is first considered. Typical stormwater drain tunnels are constructed in two categories of methods: trenchless and trenched. Due to the depth of the pipe below surface (up to 65m below grade), trenched methods commonly used in stormwater drainage systems are not possible for this project. Thus, two trenchless technologies are considered: Horizontal Directional Drilling and Microtunneling and Jacking.

A comparison of methods can be seen in the table below.

	Horizontal Directional Drilling	**Microtunnelling and Jacking**
Tolerance	+/-25mm	+/-100mm
	Pit-launched	Surface -launched
Initial cost	Lower	Higher
Diameter	<1200mm	<3400mm

Table 4. Comparison of Horizontal Drilling Methods

While the methods are quite similar in terms of cost, a couple key factors are at play here. The diameter of the pipe required to adequately meet the design flow requires an external pipe diameter greater than the maximum pipe diameter drilled with a HDD. Since the tunnel will be completed from the cliff face on Tower Beach, a high tolerance is required in order for the tunnel to meet up suitably with the vertical shaft. Additionally, the available space, while susceptible to saltwater intrusion during high tide, has a much greater accessibility than an access tunnel of greater than 60 meters depth. Thus, the choice is Microtunnelling with a Micro Tunnel Boring Machine (MTBM) as seen in figure 13 below.

Figure 13. MTBM being lowered into access shaft (Source: Microtunnelling Systems)

Because of the depth of the shaft at the location of the vertical shaft, completing the microtunnelling by beginning from the vertical shaft will not be possible. Instead tunnelling must occur from the beach side. This presents a significant divergence from the most common applications of the microtunnelling and jacking methods. Since there is no jacking wall (as there would be in an entry shaft scenario), jacking thrust forces must be transmitted through the application of a concrete and steel structure on Wreck Beach. This system is not explored in much detail in existing literature.

Jacking Systems

Most Microtunnelling methods rely on an access tunnel in order to provide a thrust wall against which thrust forces can be dispersed. Since the access shaft will be too deep and narrow to serve as an access shaft for tunnelling and jacking system, jacking must take place at ground level, from Tower Beach.

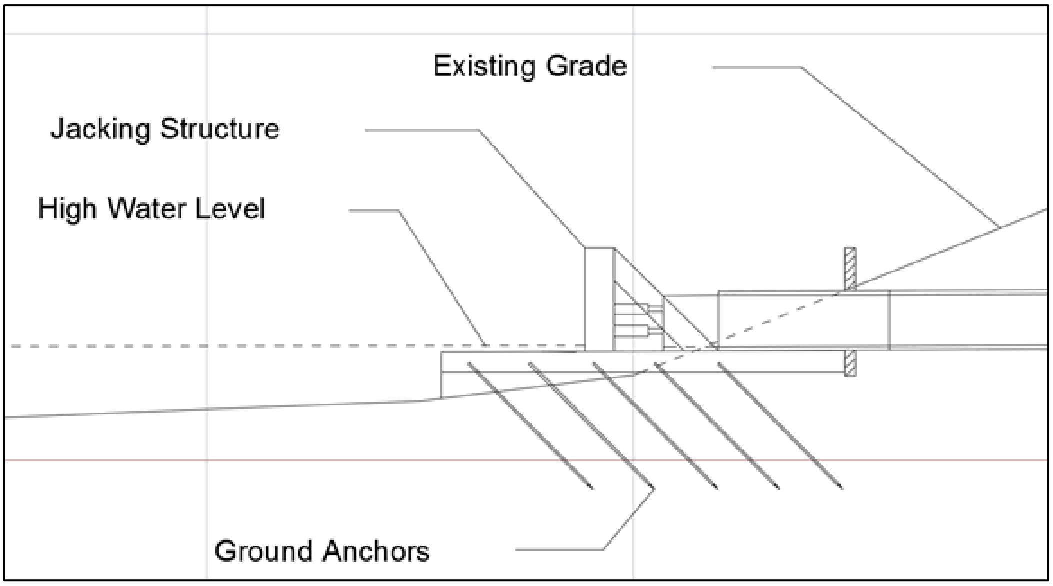

Figure 14. Ground Anchor arrangement for jacking (Source: Michael Louws, 2017)

The Mitsch and Clemence Theory were used to calculate the loading capacity of the helical anchors. Design Calculations, based on the *Performance of Helical Anchors in Sand* found the following specifications to be suitable to meet the maximum load demand required for jacking of the pipe.

Description	Multi-helical	
Diameter	0.05	[m]
Number of Anchors	12	[-]
Length	3.5	[m]
Design Load/Anchor	238	[kN]

Table 5. Ground Anchor Summary

Adequate spacing, such that overlap of the pressure bulbs is prevented from adjacent earth anchors. Not considered in this design is the possibility of compression loaded ground anchors that could be inserted towards open water, though this would depend on slope conditions of the adjacent area.

Detailed structural design of the jacking and thrust structure is required, including methods of joining helical anchors to the jacking structure.

7.2 Work Breakdown and Sequencing

Presented below is an overview of all anticipated construction activities that were utilized to complete the project schedule.

Pre-Construction Work and Project Start-up

Prior to the commencement of any construction work, the following activities must be completed:

1.) Application and approval of all required construction permits

2.) Geotechnical Survey and Borehole

3.) Procurement completed, i.e. tendering and award of construction contract to General Contractor/Subcontractors and approval of financial securities

4.) Approval of General Contractor work plan, which should include the following:

- Detailed construction methods plan and schedule
- Quality assurance and control plan
- Environmental mitigation plan
- Health and Safety Plan

5.) Purchase and order of pre-fab materials Approval of General Contractor work plan, which should include the following: Pre-cast Drop Shaft Segments, Permanent Steel Casing, Concrete Horizontal Shaft Segments

Mobilization(s) and Site Preparation

1. Topographic survey conducted to confirm existing conditions and elevations, construction limits, and grades.
2. Mobilization of equipment to site and set up of staging and material storage areas
3. Clear and grub required land.

Detention Tank:

1. Installation of Temporary Sheet Piles prior to the excavation of tank foundation
2. Drive short foundation piles prior to placement of foundation aggregates and geotextiles
3. Install formwork and rebar reinforcement for Bottom Slab, Walls, and Top slab, allowing significant time for cast in place concrete to cure
4. Inlet and Outlet connections to be installed unanimously with concrete wall installation.
5. Backfill and removing of temporary sheet piles

Baffle Drop Structure:

1. Pile driving of 4 meter diameter permanent steel casing prior to auguring or drilling baffle drop shaft and micro-tunnelling of horizontal shaft.
2. Placement of Baffle Drop segments following tie-in of horizontal shaft microtunneling, thus to remove end piece thru and up shaft
3. Backfilling with concrete grout to ensure bond between steel casing and shaft.

Horizontal Shaft and Outfall:

1. Mobilization of equipment to Tower Beach

2. Construction of Temporary cofferdam using sheet piles and de-watering prior to the set-up of Micro-tunneling equipment
3. Micro-tunneling of horizontal shaft following pile driving of Permanent steel casing.
4. Tie-in to shaft and remove of tunneling end piece up shaft prior to installation of baffle segments.
5. Installation of outfall and corrosion protection at low tide, ensuring enough curing time prior to removal of cofferdam
6. Demobilization of equipment back to site staging area

Rerouting of Stormwater

1. Excavation of trenches and installation of new pipes must be completed prior to tying in new pipes and rerouting of flow to detention tank
2. Backfill and Tie-in.

Re-storing of Site

1. Following completion of earthworks, any road curbs, paving, and landscaping can be completed

Demobilization(s)

1. Demobilization from site by removing excess materials, equipment, fencing and perimeters, etc.

7.3 Risk Management and Anticipated Site Issues

The following section summarizes the major risks and any anticipated site issues associated with the project. Given these risks, a 20% contingency has been allocated in the cost estimate. Identified early and managed properly will provide the least impact to the project.

Risk or Anticipated Site Issues	**Description**
Design Error	Error in design calculation leading to re-engineering of design and project schedule setbacks.
Unfavourable Weather	Unanticipated weather conditions during construction leading to delays.
Geotechnical Failure	High Risk cliffs with heavy vibrations due to pile driving and microtunneling.
Seismic Event	Major earthquake wit severity greater than structure was designed to withstand
Poor Soil Conditions	Design under-designed to withstand real soil conditions.
Incorrect Schedule	Projected schedule incorrectly estimated.
Inflation	Unforeseen increase in inflation over project lifetime.
Contractor Performance	Incorrect interpretation of design drawings leading to error in construction.
Poor Cost Estimate	Incorrect unit prices, productivities, crew hours, leading to project overruns.

Table 6. Risks and Anticipated Site Issues

8.0 Cost Estimate

The following section should be read while referring to the detailed Class A Cost Estimate provided in Appendix H. The expected cost to replace the current function of the spiral drain with this proposed solution is $9.55 million. Table 7, below, summarizes the estimate associated with the project, accounting for both direct and indirect costs. General cost estimating methodology is outlined below.

UBC North Campus Spiral Drain Replacement Project	
Final Cost Estimate Summary	
Permitting:	**12,000**
Design & Engineering:	**202,530**
Project Management (UBC Infrastructure):	**218,250**
Construction Contractor:	
Direct Cost	4,982,805
Mobilization/Demobilization to Site	*160,000*
Transportation of Goods	*100,000*
Site Preparation	*97,800*
Installation of Detention Tank	*1,380,945*
Installation of Baffle Drop Shaft	*1,978,800*
Installation of Horizontal Shaft & Outfall	*1,083,700*
Re-routing of Stormwater	*151,500*
Re-storing of Site	*30,060*
Indirect Cost (30% of Direct Costs)	1,494,842
Total Contractor Cost (Direct & Indirects)	**6,477,647**
Contigency (20% of Total Contractor Cost)	1,295,529
Escalation (3% Increase for 2018)	194,329
Insurance (1.5% of Total Contractor Cost)	97,165
Bonding (1% of Total Contractor Cost)	64,776
Project Cost (Intital Capital Costs):	***8,562,226***
Operation & Maintenance (Design Life):	***990,000***
Total Cost (Intital Capital Costs and O&M):	**9,552,226**

Table 7. Cost Estimate Summary

The estimate was created using the “bottom-up” technique, in a manner that a bidding general contractor would estimate the project. This technique supplies the Owner with a high level of accuracy of expected capital cost to provide to the Board of Governors for project approval. Quantity takeoffs for the estimate were based on the detailed final design calculations, analysis, and issued for construction drawings provided. The unit prices and lump sum costs were developed by referencing current Canadian material supplier pricing, manpower and labour rates, equipment rates and requirements, and reasonable crew productivities.

Indirect Contractor costs are estimated to be 30 percent of the direct costs, and include a 7% profit margin. Operational costs of the project over its 100 year design life are based on a yearly inspection and maintenance requirements. Contingency costs are estimated at 20% of total cost, given the outlined risks, additional field requirements and analysis required prior to project approval. It should be noted that taxes, PST and GST, have been excluded from this estimate.

9.0 Verification Modeling

As part of the design confirmation process the detention tank top concrete slab as well as the detention tank, baffle structure and outfall were modeled using SAP2000 and SWMM respectively.

9.1 SWMM

SWMM modeling was completed to compare existing conditions to the proposed future conditions with the baffle structure. The detention tank was modeled as a storage unit with the proposed dimensions; however the baffle structure also needed to be modeled as a storage unit. Initially, the baffle structure was intended to be modeled as a node; however there were difficulties in connected the structure to the outfall at the bottom of the structure and to the pipe connecting the detention tank to the baffle structure. For those reasons, the baffle structure was modeled as a storage unit with the proposed design dimensions.

A comparison was completed at five nodes surrounding the spiral drain and the new baffle structure / detention tank in order to see if the proposed solution was able to mitigate flooding. A large amount of flooding was mitigated; however there is still a slight amount of flooding, as seen in table 8 below.

Comparison of flooding at key nodes before and after				
	A4D-N82	A4D-N61	A4D-N1A	B4D-N2
Before (m^3)	0	1413.189	5370.453	1915.263
After (m^3)	0	0	0	144.981

Table 8: Comparison of flooding before and after implementation of design solution

According to the SWMM model, there was a 99% reduction of flooding, shrinking from close to 8500m^3 down to approximately 145m^3. It is understood that the flooding was not completely taken care of, however UBC's ISMP proposed the addition of two detention tanks (1600 and 4000 cubic meters) which would be able to handle this flooding.

SWMM was also able to show the water level in the tank over time showing a period where it reached full capacity and the period afterwards where water drained out of the tank. Figure 15 below shows how the height of the water in the tank varies over a 24 hour period.

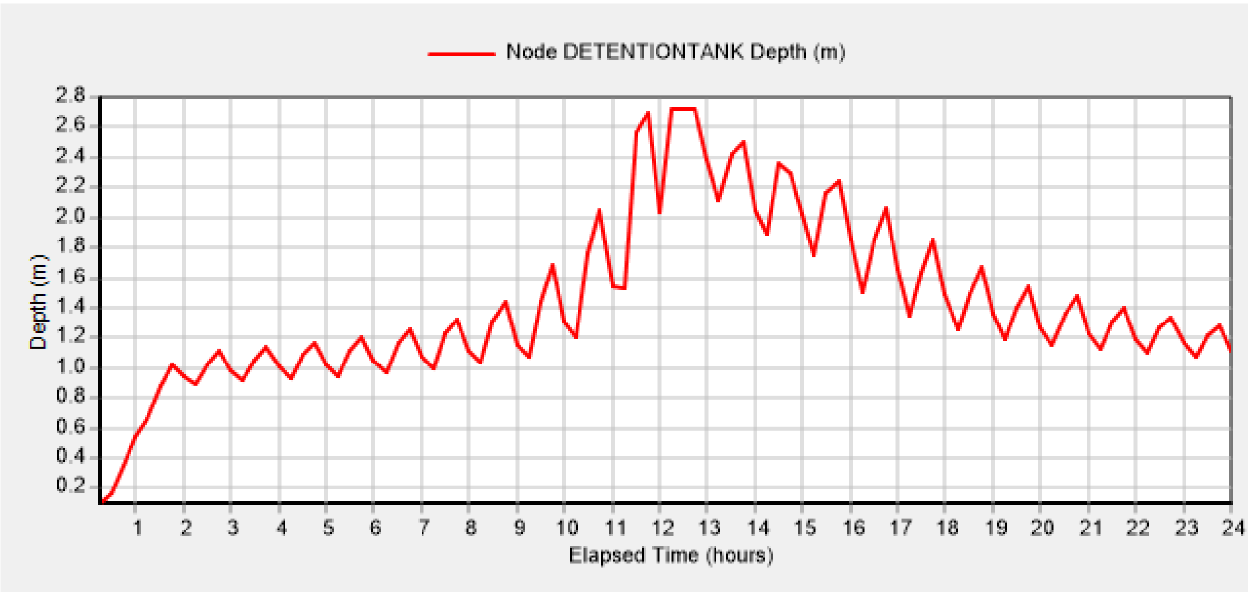

Figure 15. Height of water in Tank over 24 hour period (Source: SWMM 2017)

9.2 Concrete Slab

A flat concrete slab model was developed for the upper slab of the detention tank. The rationale behind choosing this part of the structure was that the bending moment and reinforcement design was the most complex for the one-way slab system and is exposed to high bending forces. Moreover, the two-way bending action causes complex interaction in the concrete and the finite element model is meant to verify the proposed design. The result from the modeling indicated that the proposed slab design is safe, and in fact, over designed, with more than 150% of the required steel for the tension and compression steel reinforcement.

SAP2000 Modeling Assumptions:

- Pin-pin connection about all supporting walls (Boundary Condition)
- Tension and compression reinforcement
- Soil load is reduced due to an approximate prismatic load dispersion relationship (2:1 Slope)

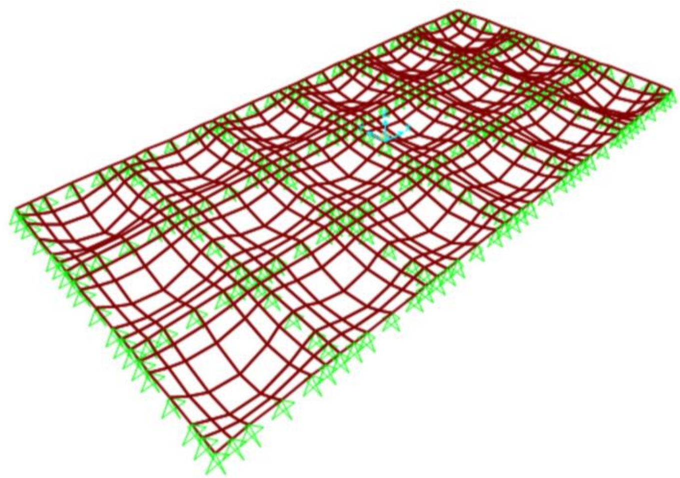

Figure 16: Flat Slab Model - Exaggerated Deformed Shape and Boundary Condition Assignment (Source: SAP2000 2017)

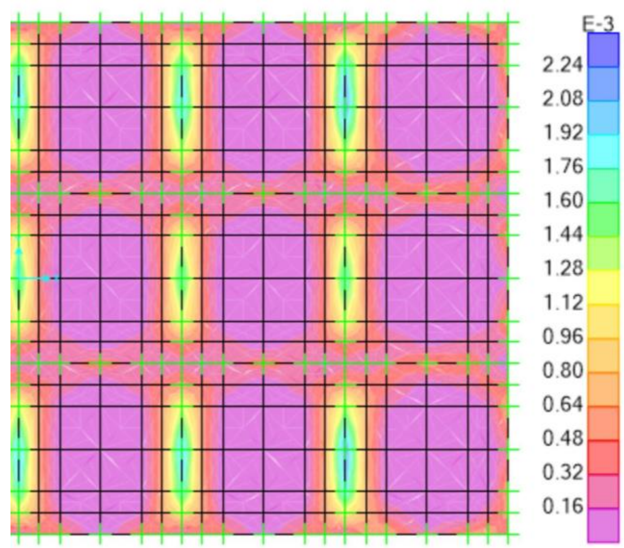

Figure 17: Flat Slab Model - Design Steel Contour for Compression Steel mm^2x10^-3 (Top Face) (Source: SAP2000 2017)

10.0 Construction Schedule

The construction schedule was developed by identifying the most critical project events and considering the relationships in sequencing. The schedule is broken down into overarching summary tasks and subtasks. Currently the project is in the middle of the pre-construction phase and is in the process of finalizing the design specification. The anticipated completion date for the project is late September, 2017. A simplified schedule is featured below. For more detail refer to the full schedule in Appendix I.

Start	End	Days	Item
15-03-17	21-04-17	33	Pre-Construction
22-04-17	22-05-17	26	Project Start
23-05-17	06-06-17	13	Project Management
03-06-17	05-07-17	28	Manufacturing & Shipping Of Pre-Fab Materials
07-06-17	29-06-17	20	Site Mob & Set-Up
30-06-17	21-08-17	45	Re-Routing Of Stormwater
29-06-17	14-08-17	40	Drop Shaft Construction
29-06-17	01-09-17	56	Horizontal Pipe And Outfall Pipe Construction
29-06-17	25-08-17	50	Detention Tank Construction
26-08-17	11-09-17	14	Re-Storing Site
11-09-17	20-09-17	9	Site Demobilization
TOTAL		334	

Figure 18: Simplified Construction Schedule (Source: Chris Vibe)

Some of the most important sequencing relationships are noted below:

- The manufacturing and shipping of the pre-fabricated materials is set begin as the work plan is approved for the general contractor as well as having established a robust health and safety plan and quality assurance/control plan. Quality assurance/control is especially important for the prefabricated components of the project.
- Drop shaft segments should be installed after the microtunneling of the horizontal shaft as equipment such as the jack hammer can only be removed while there is space in the drop shaft.
- The tie-in process for the detention tank, drop shaft, and horizontal pipe are to occur

simultaneously to ensure that the pipes are aligned within tolerances and to mobilize the specialized tie in crew only once.

- Once mobilization of equipment to the main site staging area has occurred the sites will start preparing the individual staging areas for the drop-shaft, horizontal pipe and detention tank.

11.0 Design Improvement Recommendations

There are many possible design improvements and recommendations considering the extended scope of the replacement project. Below is a table of summarising the most essential propositions:

Design Improvement/Recommendation	**Elaboration**
Water Treatment Technology	The proposed design does not incorporate basic, relatively low cost, water treatment technologies such as oil and grit interceptors. Including more filtration systems before discharge would align with sustainability goals set by SEEDS but is not a requirement.
Sustainable Material Selection	Most of the design components utilize concrete as the main structural component. Concrete is environmentally taxing and “green concrete” alternatives can be explored.
Refined Design Calculations	Geotechnical soil parameters can be updated with the geo-technical survey to be completed in mid-April. With updated parameters the conservative structural calculations can be revised with more accurate Plaxis modeling. This type of modeling would be of special interest due to the programs ability to calculate stresses in various phases of project construction.

Refined Detention Tank Calculations	The detention tank ended up being slightly undersized. There are plans to install detention tanks further upstream to deal with the minor flooding. If there were more time the tank volume would allow for an extra 150m^3 in the case that the upstream detention tanks are not installed.

Table 9. Design Improvement Recommendations

12.0 Conclusion

Vortex Consulting believes that the design outlined in this report is an innovative, economical, and constructible solution for the existing and anticipated stormwater flows from UBC's rapidly North Campus. This solution contains an elegant method of stormwater management within the "baffle-drop" structure, as well as contingency for major storm events within the detention tank element which will mitigate the risks related to a 1 in 200 year storm event. Given the construction methodology and sequencing, the anticipated finish date is September 20th, 2017. Total project direct and indirect costs are estimated to be \$9.55 million.

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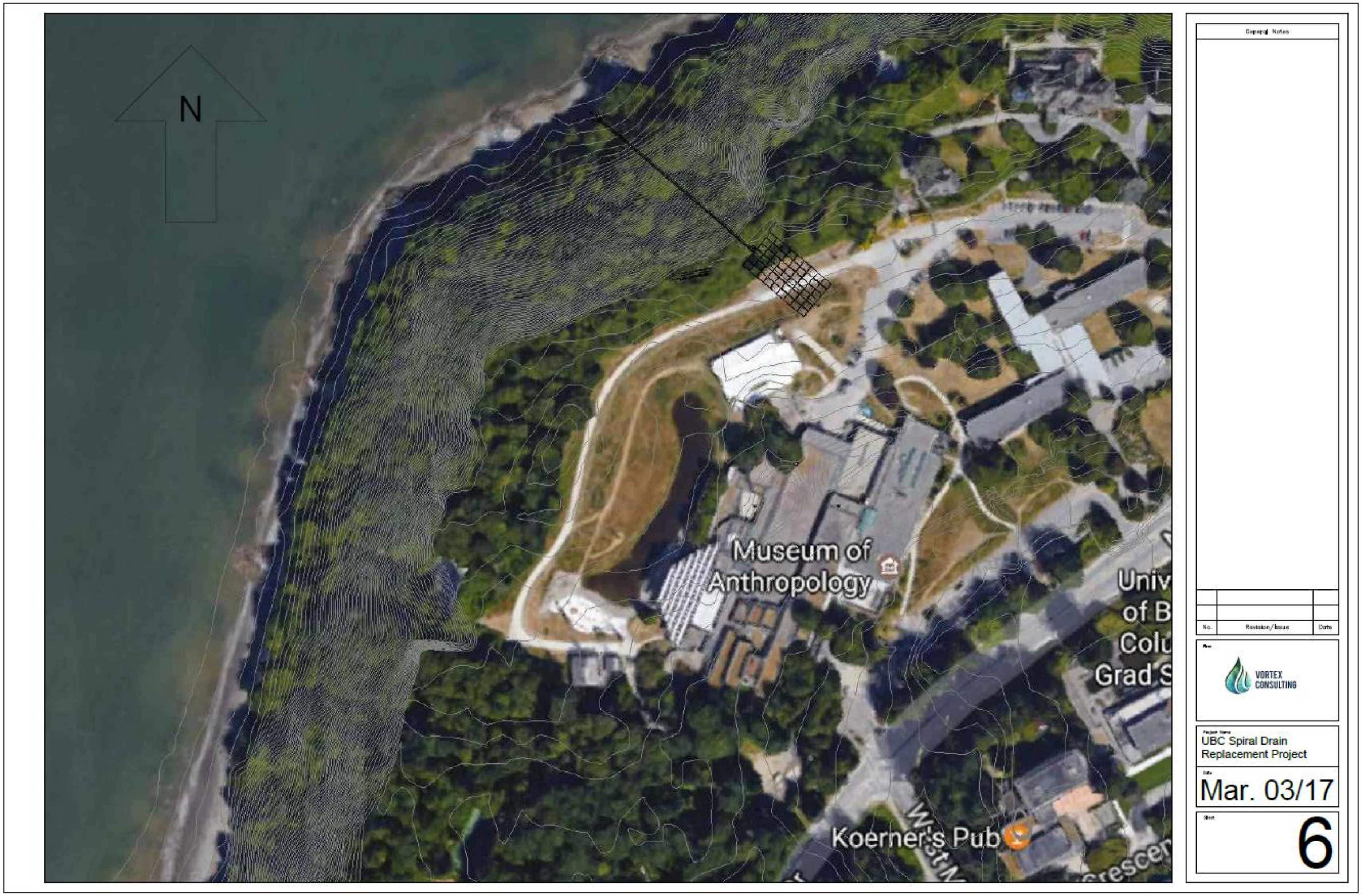
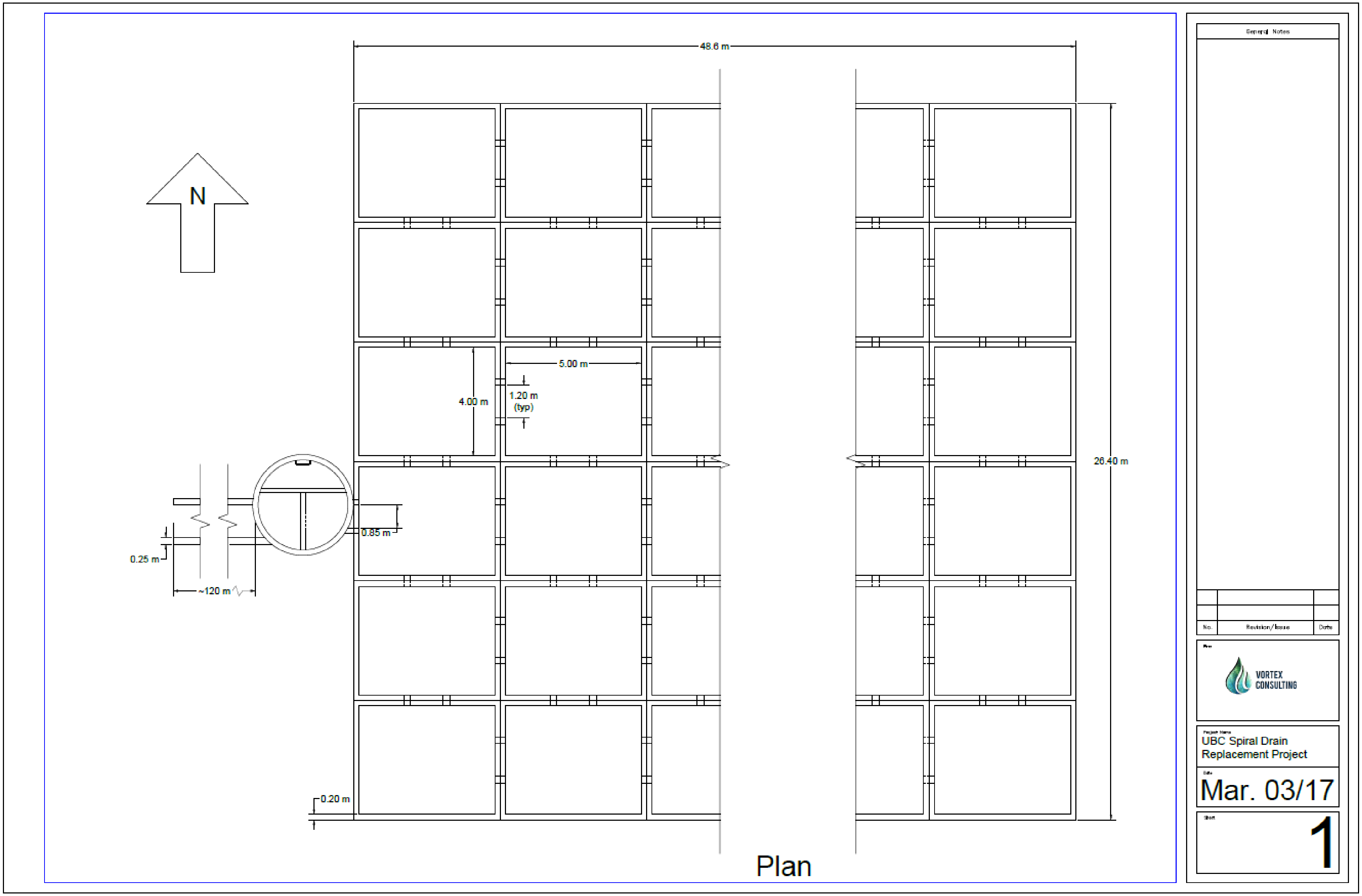
Appendix A – Site Photos

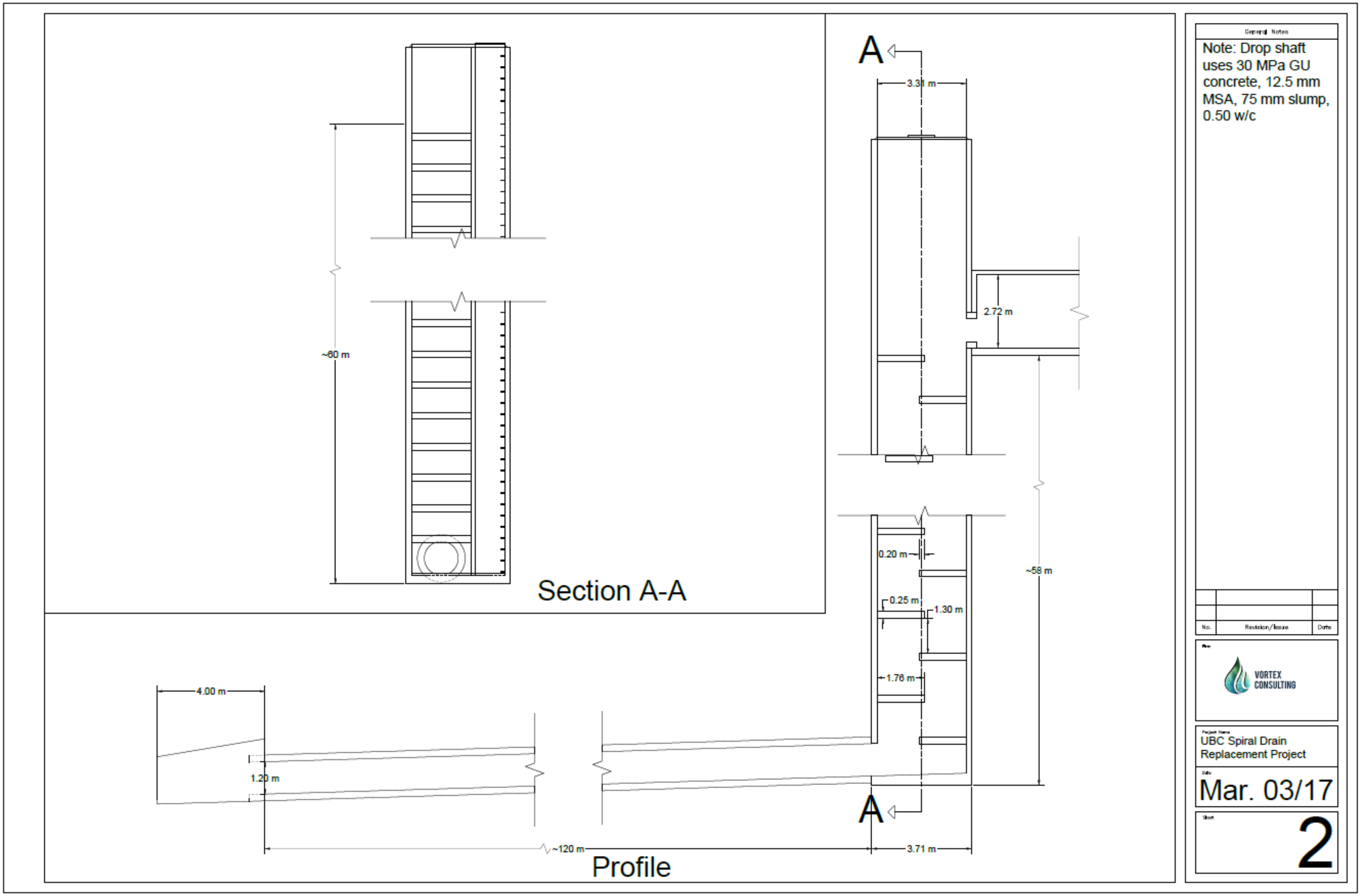

PACIFIC SPIRIT
FRAGILE AREA
PLEASE DO NOT ENTER
This slope has been bio-engineered with live wattle fences, trees, bramblies & hydo-seeding for maximum protection & biodiversity
RECONSTRUCTION WAS THE COOPERATIVE EFFORT OF:
• THE UNIVERSITY of BRITISH COLUMBIA
• GREATER VANCOUVER REGIONAL DISTRICT
• PUBLIC (through public consultation process)
RESPECT THIS AREA & STAY SAFE
PLEASE DON'T TRESPASS

Appendix B – IFC Drawings

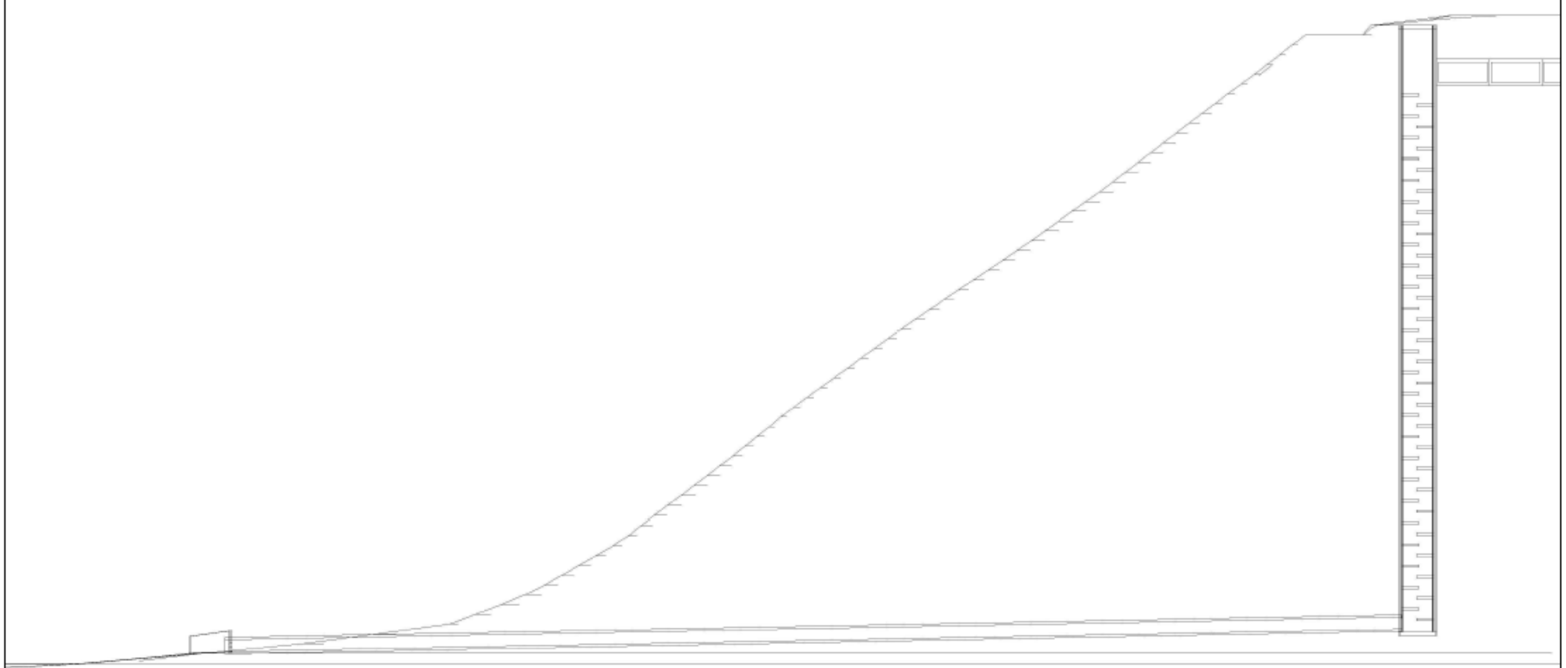


VORTEX CONSULTING


48.6 m
N
5.00 m
4.00 m
1.20 m
(typ)
26.40 m
0.85 m
0.25 m
~120 m
0.20 m
Plan
General Notes
No.
Revision/Issue
Date
VORTEX
CONSULTING
UBC Spiral Drain
Replacement Project
Mar. 03/17
1



~60 m
Section A-A
A
3.31 m
2.72 m
0.20 m
~58 m
0.25 m
1.30 m
1.76 m
4.00 m
1.20 m
~120 m
3.71 m
Profile
General Notes
Note: Drop shaft uses 30 MPa GU concrete, 12.5 mm MSA, 75 mm slump, 0.50 w/c
No.
Revision/Issue
Date
VORTEX CONSULTING
UBC Spiral Drain Replacement Project
Mar. 03/17
2



Profile (including terrain)

General Notes

Note: Vertically NTS

No.	Revision/Issue	Date

Firm

VORTEX CONSULTING

Project Name

UBC Spiral Drain Replacement Project

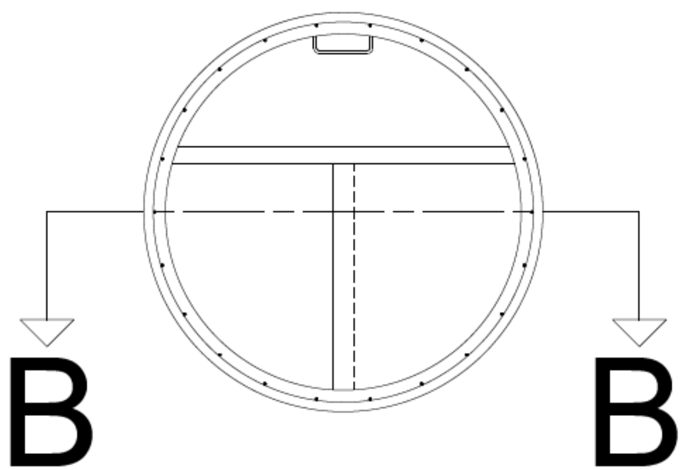
Date

Mar. 03/17

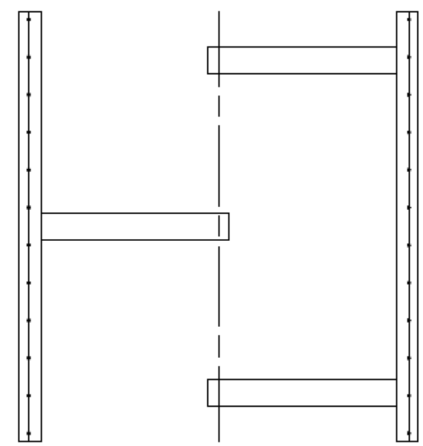
Sheet

3

Reinforcement Plan



Plan



Section B-B

General Notes

Note: Use 25M rebar

Longitudinal rebar spacing = 500 mm c/c

Hoop rebar spacing = 350 mm c/c

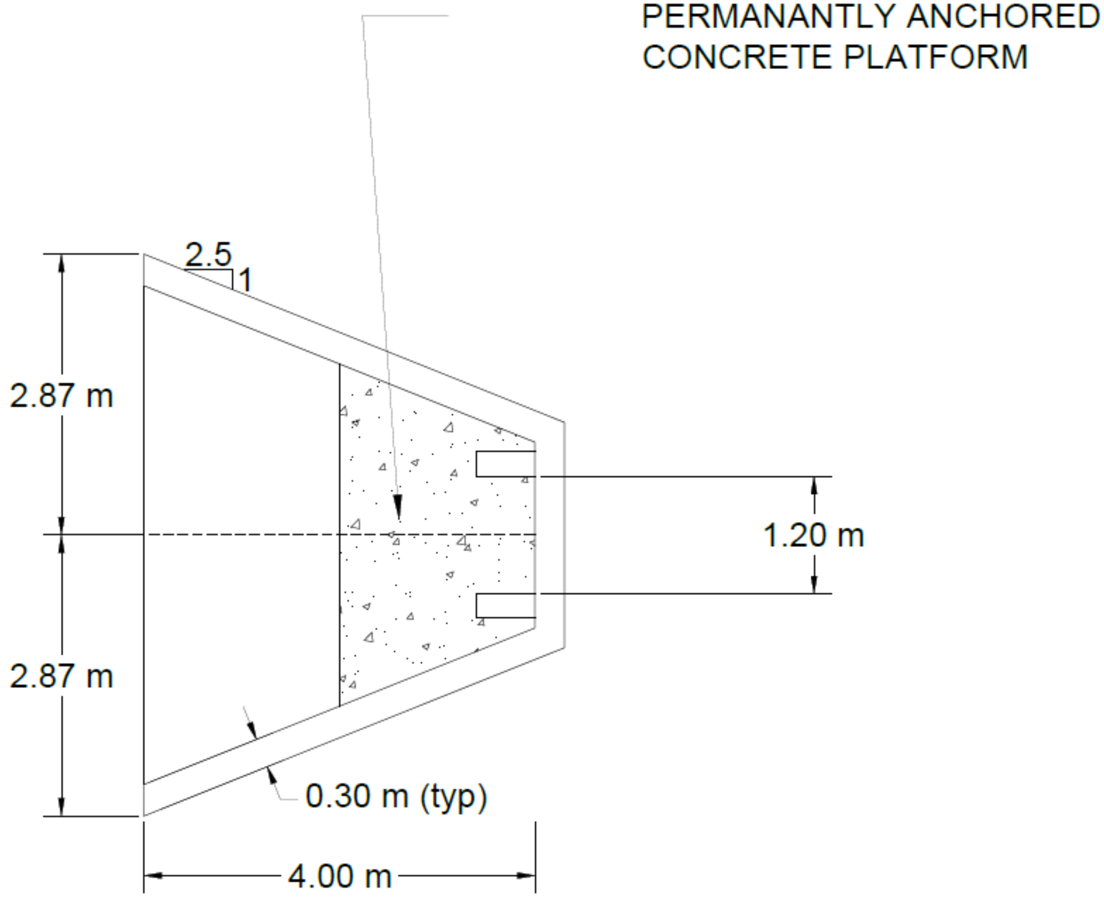
No.	Revision/Issue	Date

Firm: VORTEX CONSULTING

Project Name: UBC Spiral Drain Replacement Project

Date: Mar. 03/17

Sheet: 4



Outfall Structure Detail

General Notes

Outfall & Outfall Shaft use 35 MPa MS concrete, 12.5 mm MSA, 75 mm slump, 0.50 w/c

No.	Revision/Issue	Date

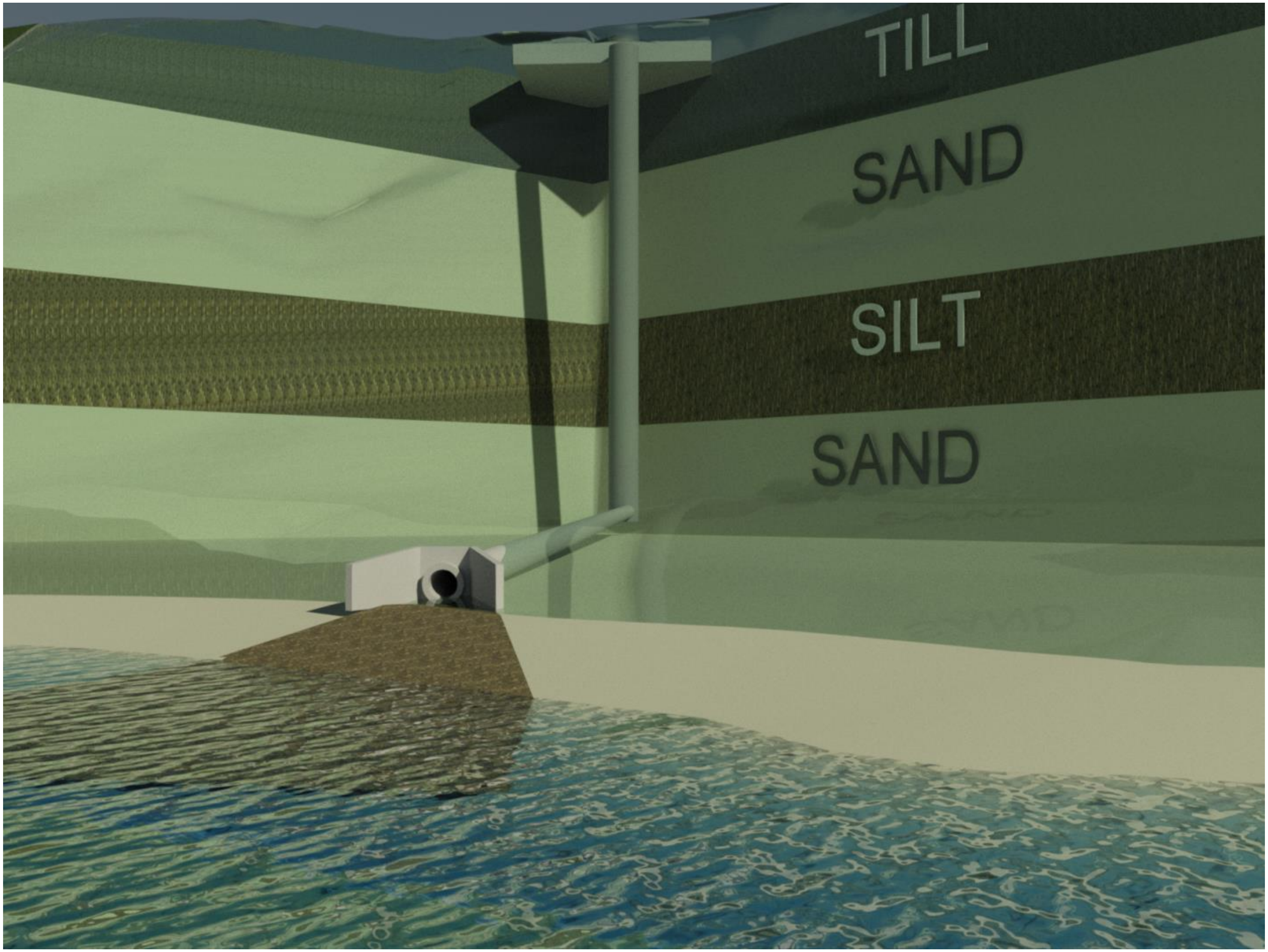
VORTEX CONSULTING

UBC Spiral Drain Replacement Project

Mar. 03/17

5

Appendix C – Design 3D Model


TILL
SAND
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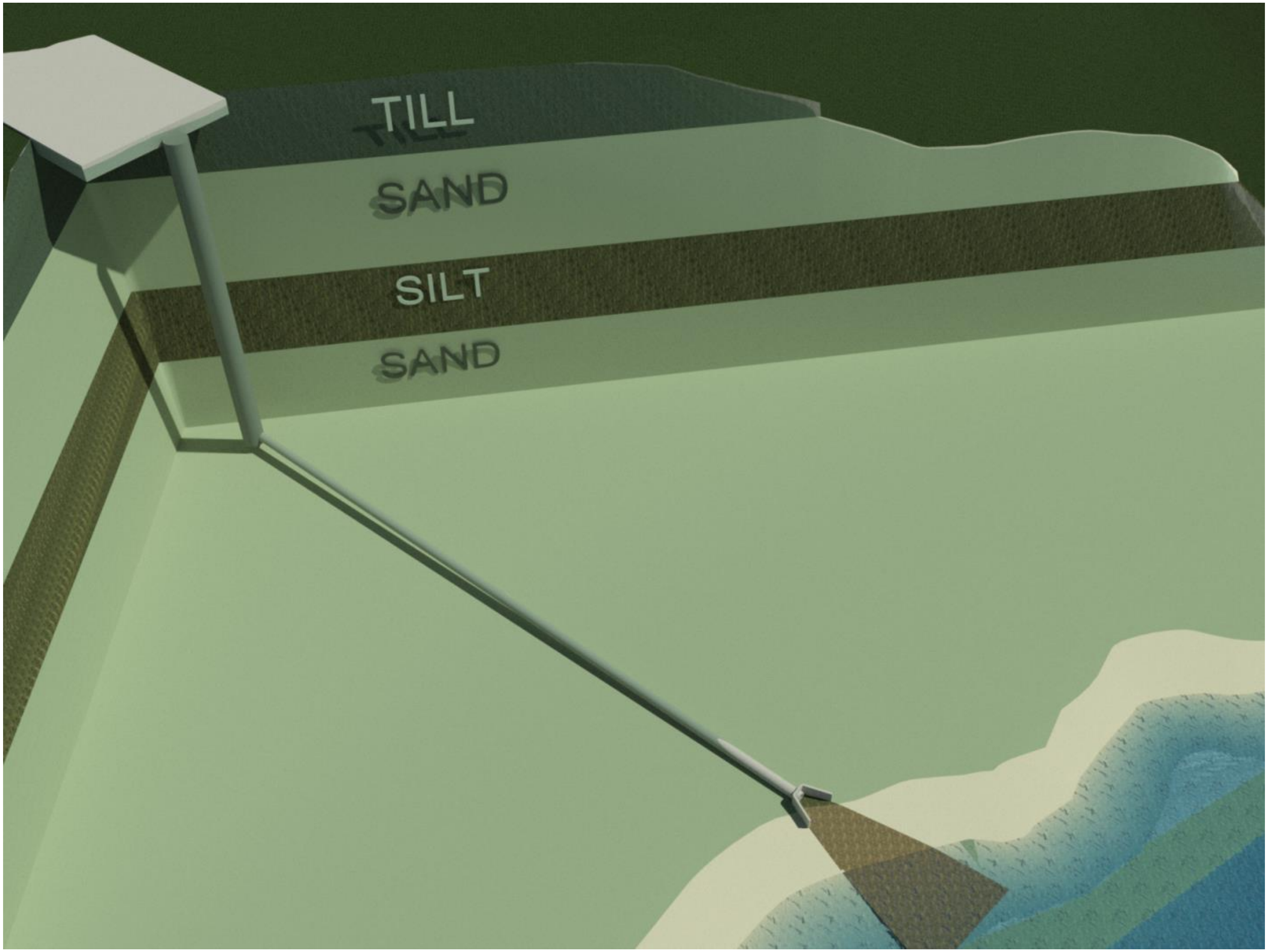
TILL
SAND
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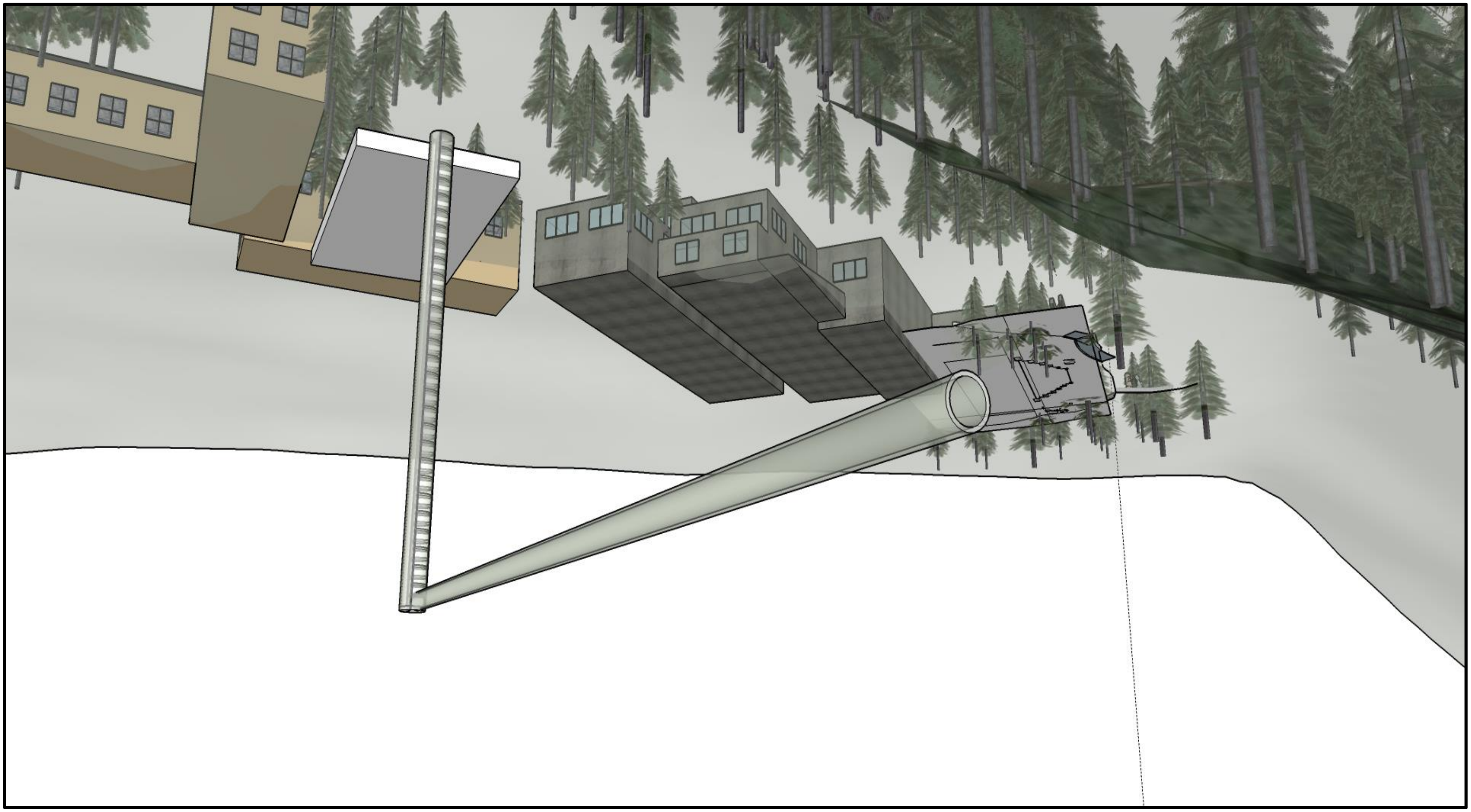
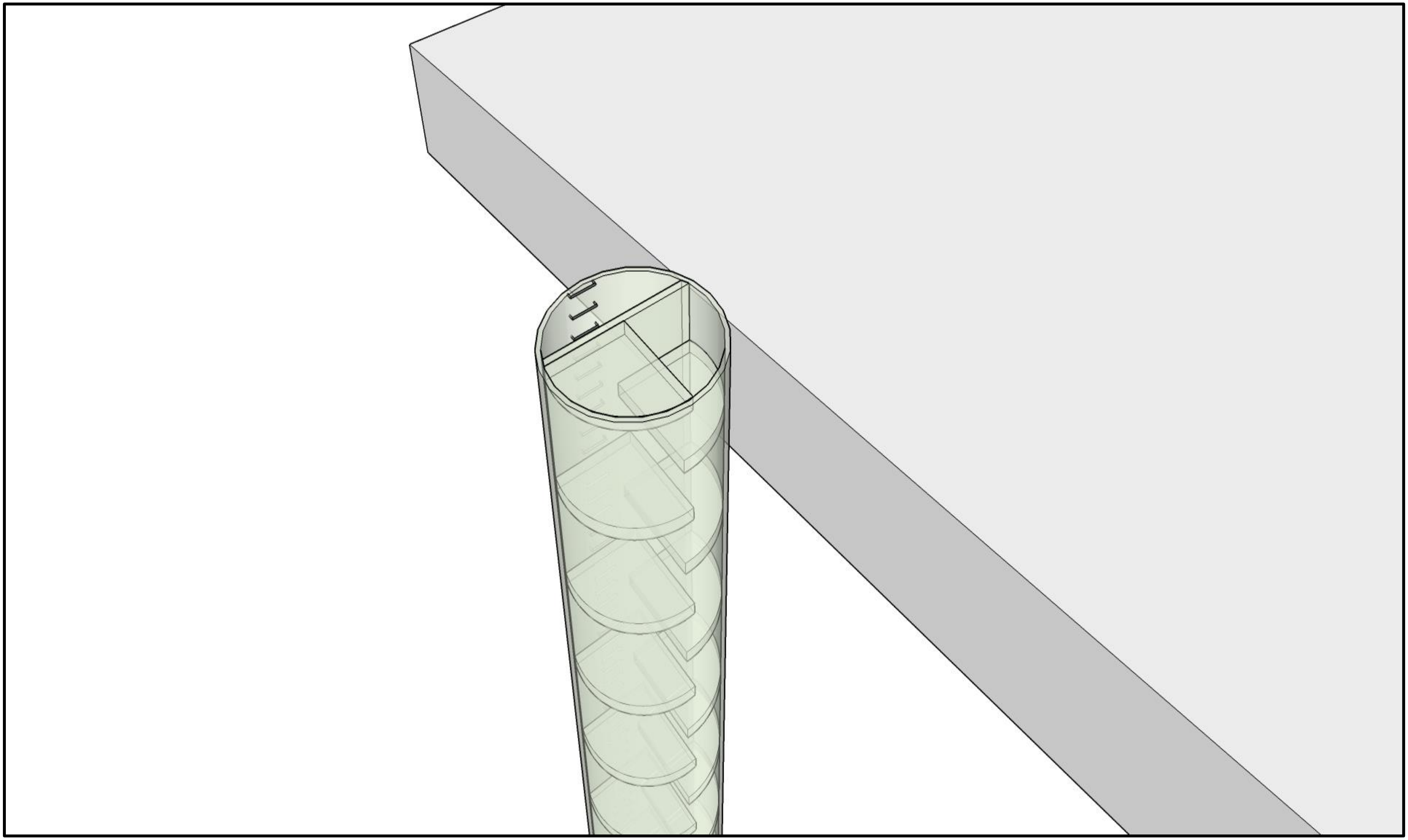
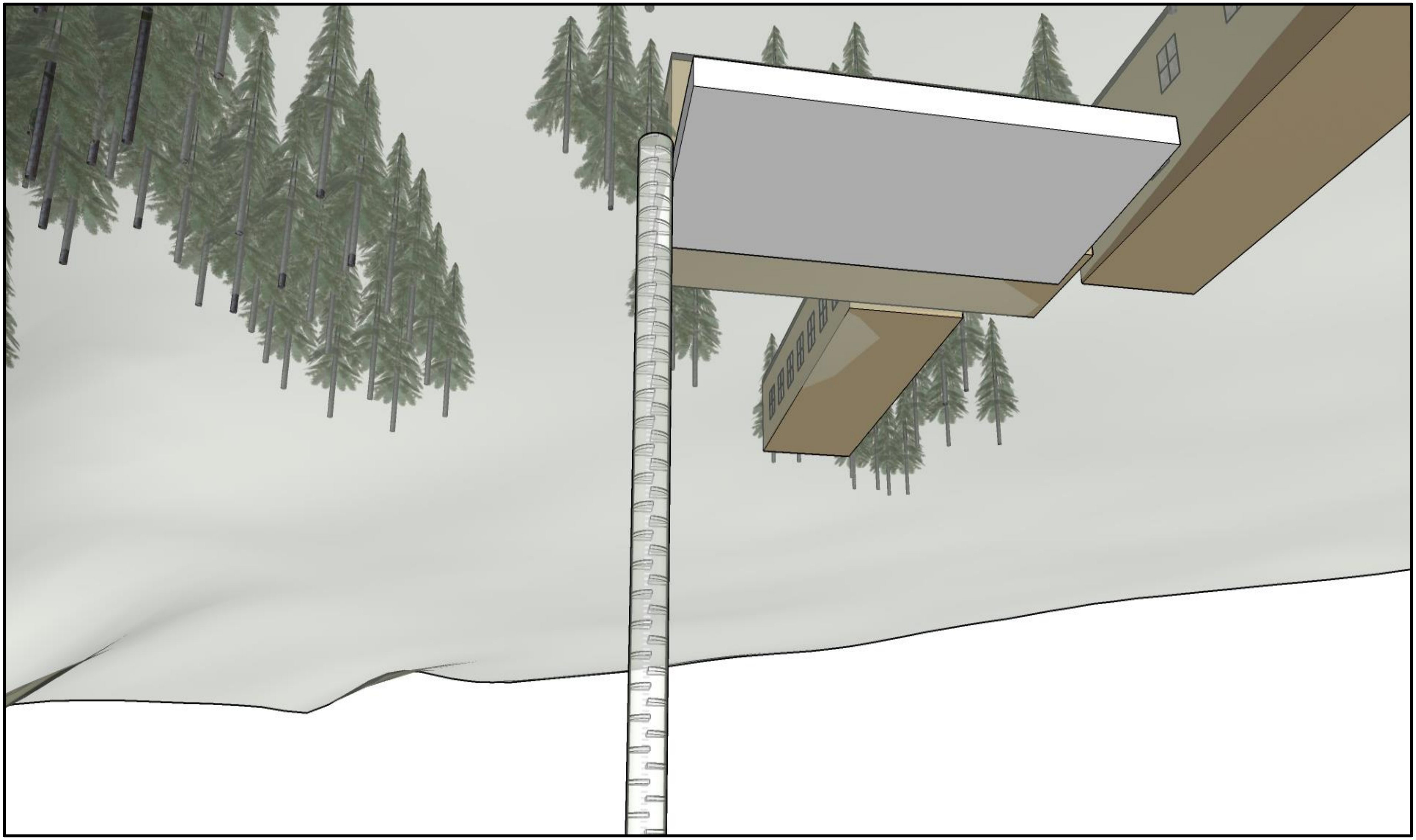
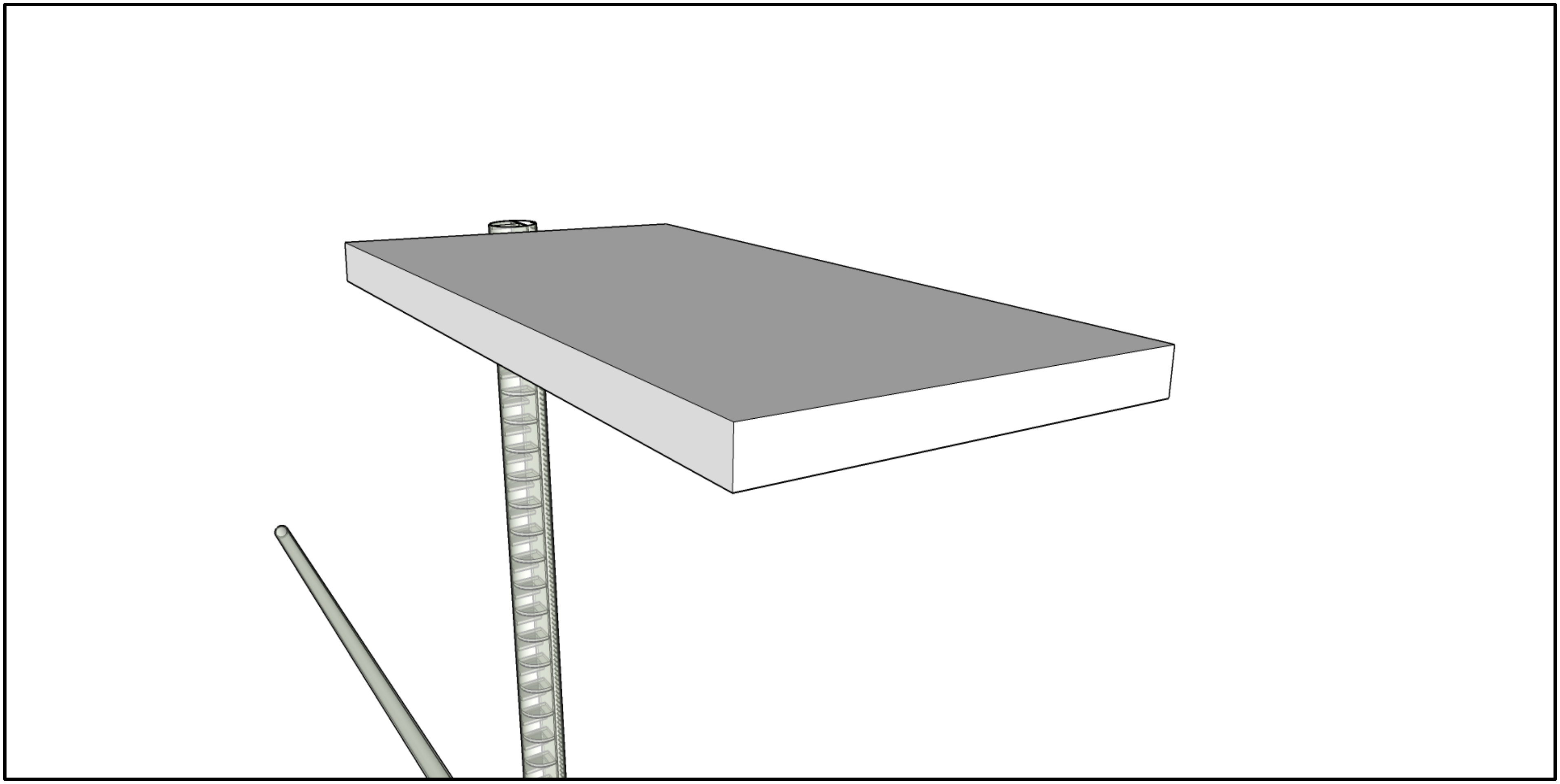
SAND









Appendix D – Material Specifications

ACI Concrete Mix Design Process

The ACI method of mix design will be followed for the majority of this process, except where more stringent CSA provisions apply. This mix assumes a final volume of 1 m3 of concrete.

Step 1: Choice of slump

As per Table 6.3.1 of ACI 211.1-91 (2002), the recommended slumps for Reinforced foundation walls and footings varies between a minimum of 25 mm and a maximum of 75 mm. In consideration of the difficult site conditions, a slump of 75 mm has been chosen.

Step 2: Choice of Maximum Size of Aggregate (MSA)

As per section 6.3.2 of ACI 211.1-91 (2002) & Section 14.2.2.1 of CSA 23.1-94, the nominal maximum size of aggregate shall not be larger than:

a) 1/5 of the narrowest dimension between the sides of forms

b) 3/4 of the minimum clear spacing between reinforcing bars

c) 1/3 of the depth of the slabs

d) The specified cover for concrete not exposed to earth or weather

e) 2/3 of the specified cover for concrete exposed to earth or weather

f) 1/2 of the specified cover for concrete exposed to chlorides

With these limitations in mind, an MSA of 12.5 mm has been chosen.

Step 3: Estimation of mixing water

As per Table 6.3.3 of ACI 211.1-91 (2002), the air content shall be 4.0% because the concrete of the drop shaft is "not exposed to freezing conditions, de-icers, or aggressive agents." From the same table, the mixing water for air entrained concrete with a slump of 75 mm and an MSA of 12.5 mm is equal to 193 kg/m3.

Step 4: Selection of water-cementitious materials (w/c) ratio

As per Table 6.3.4(b) of ACI 211.1-91 (2002), the drop shaft is in a severe exposure condition, defined as a "structure wet continuously or frequently". The maximum permissible w/c ratio for this condition is 0.50.

Step 5: Calculation of cement content

From Step 3, the water content is 193 kg/m3. From Step 4, the w/c ratio is 0.50 – therefore the cement content is simply 193/0.5, or 386 kg/m3.

Step 6: Estimation of coarse aggregate content

The fineness modulus of natural coarse aggregate is around 2.8. As per Table 6.3.6 of ACI 211.1-91 (2002), the volume of oven-dry-rodded coarse aggregate per unit volume of concrete is equal to 0.55m3/m3. The dry rodded density of natural coarse aggregate is roughly 1500 kg/m3, meaning the total coarse aggregate content is 0.55*1500 = 825 kg/m3.

Step 7: Calculation of fine aggregate content

Fine aggregate content is calculated based on the fact that the total volume of concrete is 1 m3. That is to say:

Vwater + Vcement + Vcoarse aggregate + Vfine aggregate + Vair = 1

With numbers:

193/1000 + 386/3150 + 825/2700 + Vfine aggregate + 0.04 = 1

Vfine aggregate = 0.339 m3

So the fine aggregate content is the average density of fine aggregate multiplied by the volume. That is
2680 kg/m3* 0.339 m3= 909 kg/m3

Summary

The general properties of the final concrete mix are:

Slump 75 mm

MSA 12.5 mm

Air entrainment 4.0 %

Each m3 of concrete contains:

Water 193 kg

Cement 386 kg

Coarse Aggregate 825 kg

Fine Aggregate 909 kg

Appendix E – Hydrotechnical Analysis

E.1 Hydrotechnical Design Load

Method 1:

In order to acquire the one in 200 year storm event, the precipitation data was scaled up by a factor of 1.1. This factor was acquired by calculating the difference between the 100 year and 200 year intensities in the regional IDF Curves created by Metro Vancouver. The difference is linearly correlated by a factor of roughly 1.1. The spiral drain in the model was then up scaled to an increased capacity, until the four adjacent nodes do not flood the spiral drain. The flow coming through the spiral drain node was determined to be the flow for the 1 in 200 year storm at around 5.49m^3/s (without detention tanks).

Table of 100 Year & 200 Year Precipitation Intensities from Regional IDF Curves, Metro Vancouver Climate Stations Report

Duration	100 Year	200 year	200 Yr. / 100 Yr.
5 min	86.6	96	1.109
15 min	49.7	54.4	1.095
30 min	35	38.3	1.094
1 h	24.6	26.9	1.093
2 h	17.3	18.9	1.092
6 h	9.9	10.8	1.091
12h	7	7.6	1.086
24 h	4.9	5.4	1.102

48 h	3.5	3.8	1.086
72 h	2.8	3.1	1.107
	Average Scaling Factor		**1.10**

Method 2:

In order to estimate the 200 year design storm, a number of hydrological methods were applied. Precipitation data for YVR Airport was obtained from Environment Canada via Prof. Ulrich Mayer of the UBC Geography Department of Earth and Ocean Sciences. The data includes hourly precipitation data from April 1, 1960 to July 1, 2013. Some data points throughout the data set are missing, which represent possible sources of error in statistical analysis. In order to determine the IDF curves, periods of 4, 12, 36, 120, and 240 hours were chosen. For each year, the maximum 4, 12, 36, 120, and 240 hour period was selected using Matlab. Using a Gumbel distribution and the probability density function described below

$$P(x) = \exp\left(-\,exp\left(-\frac{X - X_0}{\beta}\right)\right)$$

$$with\ P = 1 - \frac{1}{T}$$

$$where\ T\ is\ the\ probability\ of\ exceedence, in\ this\ case\ 200\ years$$

$$P = probability\ of\ exceedance$$

$$X = variable\ of\ interest$$

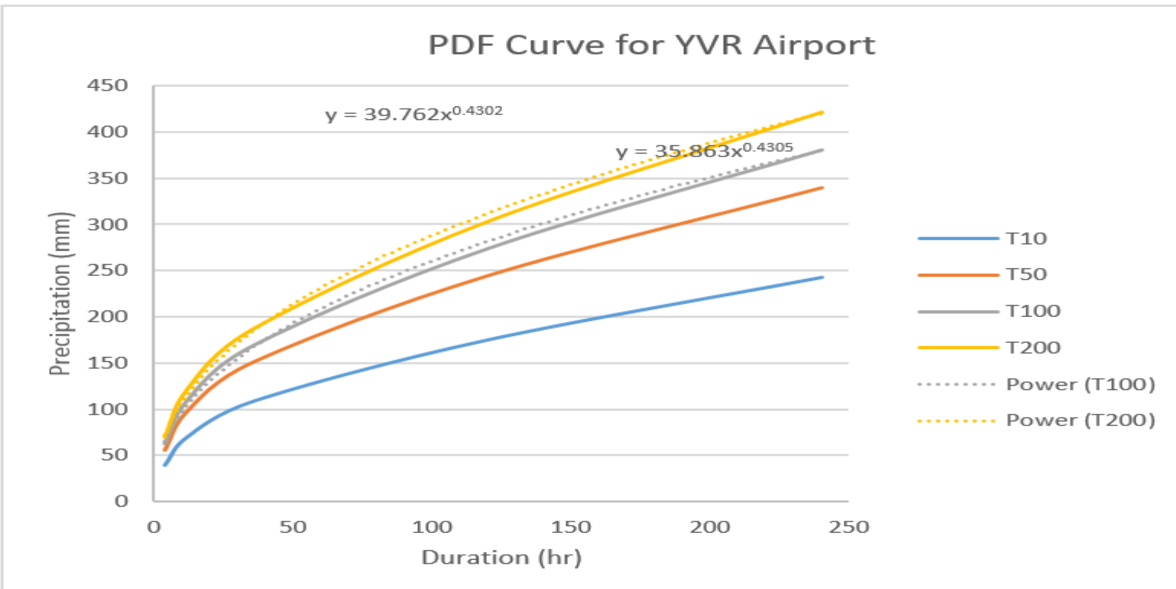
$$X_0 = location\ parameter$$

$$\beta = scale\ parameter$$

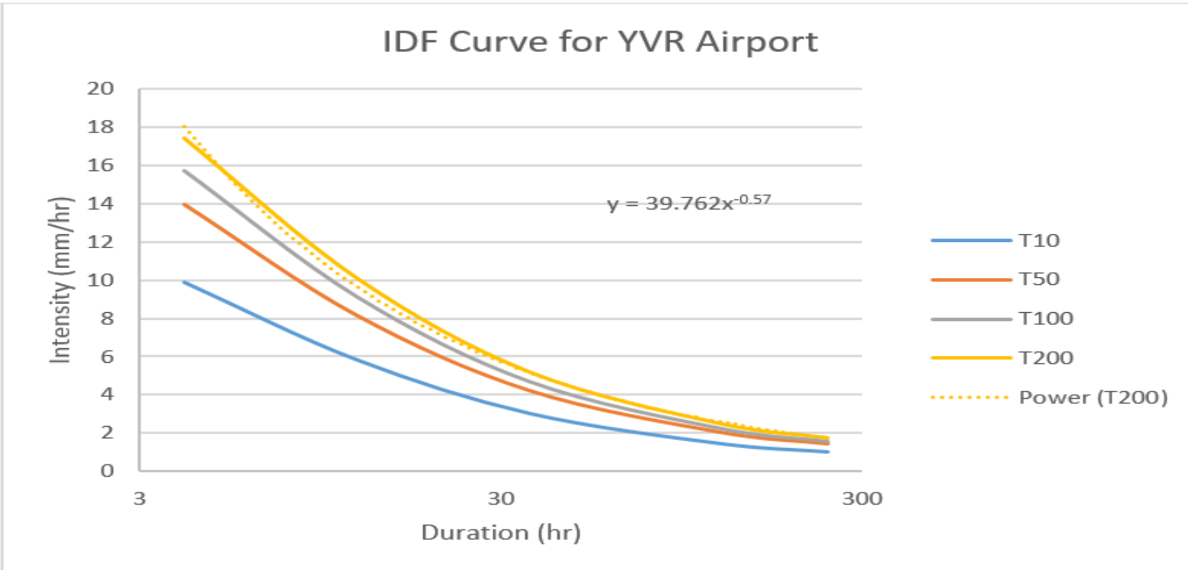
The method of maximum likelihood was used to estimate X_0 and beta iteratively, with the equations described below.

$$\beta = \frac{1}{n}\sum_{i=1}^{n}(x_i) - \frac{\sum_{i=1}^{n} x_i \times \exp\left(-\frac{x_i}{\beta}\right)}{\sum_{i=1}^{n} \exp\left(-\frac{x_i}{\beta}\right)} \text{ and } X_0 = -\beta \times \ln\left(\frac{1}{n}\sum_{i=1}^{n} exp\left|\left(-\frac{x_i}{\beta}\right)\right)$$

The value of the total precipitation can be found for each period by a separate iteration from the annual maximum values for 1960 to 2013. Each of these precipitation values is then plotted vs. the duration of storm for a particular probability, leading to a precipitation-duration-frequency curve, as seen below.

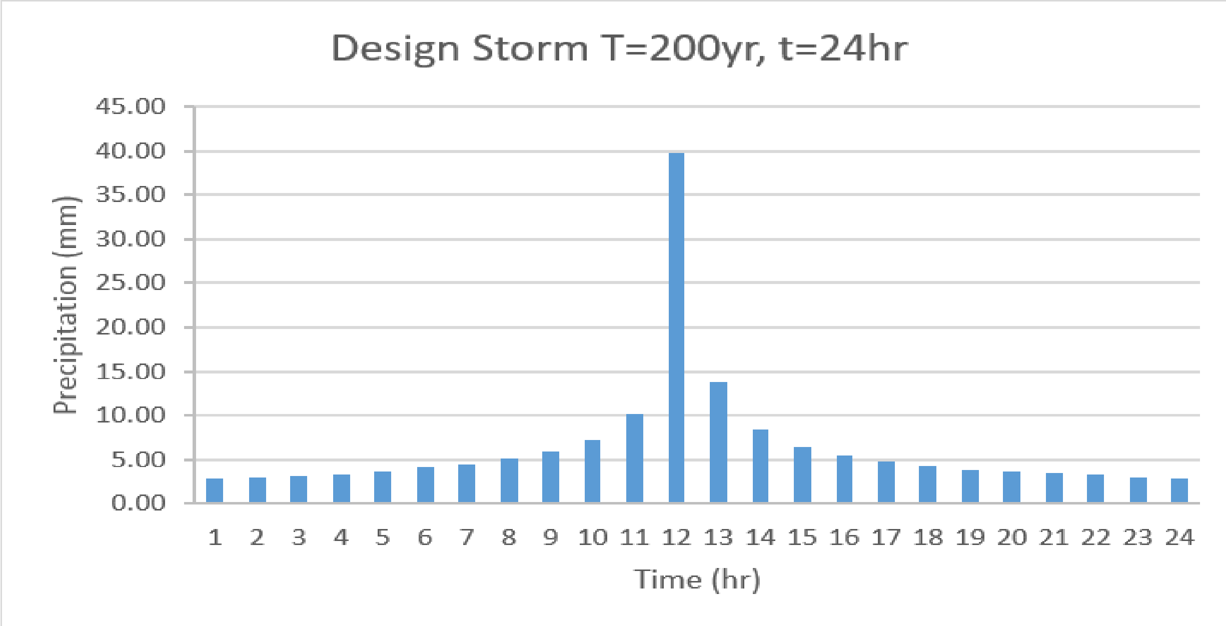


The intensity-duration-frequency curve, or IDF curve, can be determined by dividing each precipitation total by its duration. This gives an average intensity value for the given precipitation period.



In order to calculate the design storm, methodology was adopted from Field Manual for Pennsylvania Design Rainfall Intensity Charts¸ Appendix A. In this method, the PDF curve is used to estimate the composite storm over a 24 hour period. Since our data is hourly, a time step of 1 hour is used.

A power trend line was used in Excel to approximate the PDF curve for the 200 year precipitation. From this, a design storm of 24 hours was created with a time step of 1 hour, as seen in graph below.


Design Storm T=200yr, t=24hr
Precipitation (mm)
45.00
40.00
35.00
30.00
25.00
20.00
15.00
10.00
5.00
0.00
1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24
Time (hr)

E.2 Hydrotechnical Design Load Dimensional Requirements

Detention Tank Size and Controlled Outlet Flow:

Summary Table	
Detention Tank Size (m3)	3000
Controlled Outfall Flow Rate into Shaft (m3/s)	3.98
Is there flooding?	No
If there is, what is the largest Flood Volume?	N/A

Time	Demand Flow Rate (m^3/s)	Demand Volume over 15 minutes (m^3)	Cumulative Demand (m^3)	Design Flow Rate (m^3/s)	Supply Volume over 15 minute without Detention Tank (m^3)	Cumulative Supply with Detention Tank (m^3)	Flooding (0 if none, -ve number if flooding) m^3	Net Storage without Tank (m^3)	Net Storage with Tank (m^3)
0:15:00	0.01	0.00	0.00	3.98	3582	3000	0.00	0.00	3000.00
0:30:00	0.04	21.98	21.98	3.98	3582	6582	0.00	0.00	3000.00
0:45:00	0.06	43.94	65.92	3.98	3582	10164	0.00	0.00	3000.00
1:00:00	0.09	65.79	131.71	3.98	3582	13746	0.00	0.00	3000.00
1:15:00	0.13	97.76	229.46	3.98	3582	17328	0.00	0.00	3000.00
1:30:00	0.24	165.65	395.11	3.98	3582	20910	0.00	0.00	3000.00
1:45:00	0.46	315.95	711.06	3.98	3582	24492	0.00	0.00	3000.00
2:00:00	0.68	513.77	1224.83	3.98	3582	28074	0.00	0.00	3000.00
2:15:00	0.83	678.56	1903.40	3.98	3582	31656	0.00	0.00	3000.00
2:30:00	0.92	787.93	2691.33	3.98	3582	35238	0.00	0.00	3000.00
2:45:00	0.99	860.13	3551.46	3.98	3582	38820	0.00	0.00	3000.00
3:00:00	1.04	915.25	4466.71	3.98	3582	42402	0.00	0.00	3000.00
3:15:00	1.07	953.14	5419.85	3.98	3582	45984	0.00	0.00	3000.00
3:30:00	1.08	970.11	6389.96	3.98	3582	49566	0.00	0.00	3000.00
3:45:00	1.08	974.58	7364.54	3.98	3582	53148	0.00	0.00	3000.00
4:00:00	1.09	975.92	8340.47	3.98	3582	56730	0.00	0.00	3000.00
4:15:00	1.10	984.01	9324.48	3.98	3582	60312	0.00	0.00	3000.00
4:30:00	1.12	1001.28	10325.75	3.98	3582	63894	0.00	0.00	3000.00
4:45:00	1.17	1030.26	11356.01	3.98	3582	67476	0.00	0.00	3000.00
5:00:00	1.22	1071.54	12427.55	3.98	3582	71058	0.00	0.00	3000.00
5:15:00	1.29	1125.74	13553.29	3.98	3582	74640	0.00	0.00	3000.00
5:30:00	1.36	1192.89	14746.18	3.98	3582	78222	0.00	0.00	3000.00
5:45:00	1.43	1258.75	16004.93	3.98	3582	81804	0.00	0.00	3000.00
6:00:00	1.49	1316.46	17321.40	3.98	3582	85386	0.00	0.00	3000.00
6:15:00	1.55	1370.25	18691.65	3.98	3582	88968	0.00	0.00	3000.00
6:30:00	1.61	1422.52	20114.17	3.98	3582	92550	0.00	0.00	3000.00
6:45:00	1.73	1500.63	21614.80	3.98	3582	96132	0.00	0.00	3000.00
7:00:00	1.87	1616.18	23230.98	3.98	3582	99714	0.00	0.00	3000.00
7:15:00	2.08	1775.22	25006.20	3.98	3582	103296	0.00	0.00	3000.00
7:30:00	2.40	2014.78	27020.98	3.98	3582	106878	0.00	0.00	3000.00
7:45:00	2.65	2271.98	29292.96	3.98	3582	110460	0.00	0.00	3000.00
8:00:00	2.82	2463.49	31756.45	3.98	3582	114042	0.00	0.00	3000.00
8:15:00	4.43	3265.30	35021.75	3.98	3582	117624	0.00	0.00	3000.00
8:30:00	5.25	4358.04	39379.80	3.98	3582	121206	-776.04	-776.04	2223.96
8:45:00	5.49	4835.44	44215.24	3.98	3582	124788	-1253.44	-2029.48	970.52
9:00:00	4.29	4403.85	48619.08	3.98	3582	128370	-821.85	-2851.33	148.67
9:15:00	3.96	3715.30	52334.39	3.98	3582	131952	-133.30	-2984.63	15.37
9:30:00	3.63	3415.08	55749.47	3.98	3582	135534	0.00	-2984.63	15.37
9:45:00	3.21	3074.22	58823.69	3.98	3582	139116	0.00	-2984.63	15.37
10:00:00	2.95	2769.26	61592.95	3.98	3582	142698	0.00	-2984.63	15.37
10:15:00	2.75	2563.21	64156.16	3.98	3582	146280	0.00	-2984.63	15.37
10:30:00	2.61	2411.02	66567.18	3.98	3582	149862	0.00	-2984.63	15.37
10:45:00	2.43	2269.94	68837.13	3.98	3582	153444	0.00	-2984.63	15.37
11:00:00	2.32	2140.22	70977.34	3.98	3582	157026	0.00	-2984.63	15.37
11:15:00	2.22	2043.47	73020.81	3.98	3582	160608	0.00	-2984.63	15.37
11:30:00	2.16	1971.93	74992.74	3.98	3582	164190	0.00	-2984.63	15.37
11:45:00	1.97	1860.18	76852.93	3.98	3582	167772	0.00	-2984.63	15.37
12:00:00	1.92	1752.20	78605.13	3.98	3582	171354	0.00	-2984.63	15.37

12:15:00	1.88	1711.08	80316.20	3.98	3582	174936	0.00	-2984.63	15.37
12:30:00	1.84	1674.54	81990.74	3.98	3582	178518	0.00	-2984.63	15.37
12:45:00	1.78	1631.17	83621.91	3.98	3582	182100	0.00	-2984.63	15.37
13:00:00	1.72	1577.75	85199.66	3.98	3582	185682	0.00	-2984.63	15.37
13:15:00	1.67	1528.85	86728.51	3.98	3582	189264	0.00	-2984.63	15.37
13:30:00	1.64	1489.52	88218.03	3.98	3582	192846	0.00	-2984.63	15.37
13:45:00	1.61	1461.23	89679.26	3.98	3582	196428	0.00	-2984.63	15.37
14:00:00	1.60	1445.60	91124.86	3.98	3582	200010	0.00	-2984.63	15.37
14:15:00	1.58	1429.50	92554.35	3.98	3582	203592	0.00	-2984.63	15.37
14:30:00	1.55	1404.91	93959.26	3.98	3582	207174	0.00	-2984.63	15.37
14:45:00	1.53	1385.05	95344.31	3.98	3582	210756	0.00	-2984.63	15.37
15:00:00	1.52	1375.25	96719.56	3.98	3582	214338	0.00	-2984.63	15.37
15:15:00	1.50	1361.80	98081.37	3.98	3582	217920	0.00	-2984.63	15.37
15:30:00	1.47	1338.96	99420.33	3.98	3582	221502	0.00	-2984.63	15.37
15:45:00	1.46	1320.08	100740.41	3.98	3582	225084	0.00	-2984.63	15.37
16:00:00	1.45	1310.79	102051.20	3.98	3582	228666	0.00	-2984.63	15.37
16:15:00	1.43	1297.83	103349.03	3.98	3582	232248	0.00	-2984.63	15.37
16:30:00	1.40	1275.43	104624.46	3.98	3582	235830	0.00	-2984.63	15.37
16:45:00	1.39	1256.64	105881.10	3.98	3582	239412	0.00	-2984.63	15.37
17:00:00	1.38	1247.21	107128.31	3.98	3582	242994	0.00	-2984.63	15.37
17:15:00	1.38	1242.33	108370.64	3.98	3582	246576	0.00	-2984.63	15.37
17:30:00	1.38	1239.71	109610.35	3.98	3582	250158	0.00	-2984.63	15.37
17:45:00	1.36	1230.21	110840.55	3.98	3582	253740	0.00	-2984.63	15.37
18:00:00	1.33	1209.79	112050.34	3.98	3582	257322	0.00	-2984.63	15.37
18:15:00	1.32	1192.03	113242.37	3.98	3582	260904	0.00	-2984.63	15.37
18:30:00	1.31	1183.11	114425.48	3.98	3582	264486	0.00	-2984.63	15.37
18:45:00	1.29	1170.60	115596.08	3.98	3582	268068	0.00	-2984.63	15.37
19:00:00	1.26	1148.64	116744.72	3.98	3582	271650	0.00	-2984.63	15.37
19:15:00	1.25	1129.81	117874.53	3.98	3582	275232	0.00	-2984.63	15.37
19:30:00	1.24	1119.98	118994.51	3.98	3582	278814	0.00	-2984.63	15.37
19:45:00	1.22	1106.94	120101.45	3.98	3582	282396	0.00	-2984.63	15.37
20:00:00	1.19	1084.79	121186.25	3.98	3582	285978	0.00	-2984.63	15.37
20:15:00	1.18	1065.59	122251.83	3.98	3582	289560	0.00	-2984.63	15.37
20:30:00	1.17	1055.30	123307.14	3.98	3582	293142	0.00	-2984.63	15.37
20:45:00	1.51	1203.15	124510.29	3.98	3582	296724	0.00	-2984.63	15.37
21:00:00	2.01	1581.58	126091.87	3.98	3582	300306	0.00	-2984.63	15.37
21:15:00	1.80	1714.42	127806.30	3.98	3582	303888	0.00	-2984.63	15.37
21:30:00	1.37	1426.89	129233.18	3.98	3582	307470	0.00	-2984.63	15.37
21:45:00	1.18	1147.25	130380.44	3.98	3582	311052	0.00	-2984.63	15.37
22:00:00	1.08	1018.72	131399.15	3.98	3582	314634	0.00	-2984.63	15.37
22:15:00	1.03	949.64	132348.79	3.98	3582	318216	0.00	-2984.63	15.37
22:30:00	0.97	896.44	133245.23	3.98	3582	321798	0.00	-2984.63	15.37
22:45:00	0.93	852.62	134097.85	3.98	3582	325380	0.00	-2984.63	15.37
23:00:00	0.91	824.90	134922.75	3.98	3582	328962	0.00	-2984.63	15.37
23:15:00	0.89	808.41	135731.16	3.98	3582	332544	0.00	-2984.63	15.37
23:30:00	0.88	797.97	136529.13	3.98	3582	336126	0.00	-2984.63	15.37
23:45:00	0.88	790.65	137319.78	3.98	3582	339708	0.00	-2984.63	15.37
0:00:00	0.87	785.38	138105.16	3.98	3582	343290	0.00	-2984.63	15.37

Detention Tank Dimensions:

Volume (m3)	Width (m)	Length (m)	Depth (m)
3009.952	25	48	2.72

Appendix F – Technical Analysis

F.1 Detention Tank

Summary

Top Slab Loading Calculations					
Assumed Pin-Pin loading situation					
L		1	m		
B		8.33333333	m		
Tributary Area		6.25	m^2		
w		109.423125	kN/m		
Max M		949.853516	kN*m		
Max V		455.929688	kN		
Tension Steeel			**Compression Steel**		
Tr	2380000	N	Tr	2380000	
Phi	0.85		Phi	0.85	
n	7		n	6	
Db	35.68248	mm	Db'	11.3	mm
Ab	1000	mm^2	Ab'	200	mm^2
As	7000	mm^2	A's	1200	mm^2
Fy	400	MPa	Fy	400	MPa
Cr	1972000	N	Cr'	408000	N
Moment, Reinforcement Ratio and Steel Area Check					
Phi	0.65				
Alpha	0.775				
Beta	0.845				
Fc'	50				
a	78.2928	mm			
b	1000	mm			
d	500	mm			
As min	720	mm (As > Asmin)			
rho	0.003091	properly reinforced			
Mr	1067.923	kN*m			
Rebar Spacing Checks					
Greater of	1.4*db	49.9554753	mm		
	30mm	30	mm		
	am	10	mm		
Tension	Less than .	Ab*(1000/As	142.8571	mm	
Compression		Ab*(1000/As'	166.6667	mm	
	Smax less	3h	1800	mm	
		500mm	500	m	
Slab Dimensions					
Tension	Spacing is 35M at 130mm				
Compression	Spacing is 35M at 150mm				
d' (Compression)	110	mm			
d (Tension)	500	mm			
Cover	40	mm			
ds	10	mm			
db	35	mm			
Total Height	585	mm			
Chosen Height	600	mm			

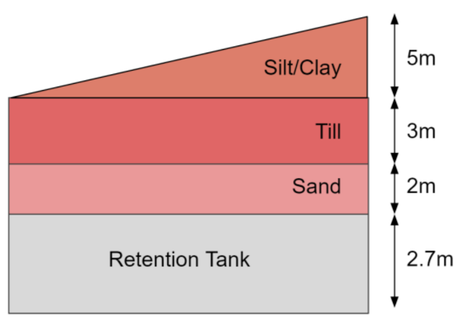
Bottom Slab Loading Calculations					
Bottom Slab	Length	Thickness/W	Height	Number	Weight (N)
Weight of Walls_L	48	0.2	2.7	7	4267468.8
Weight of Walls_B	25	0.2	2.7	4	1270080
Weight of Slab	48	25	0.6	1	16934400
Weight of Water	48	25	2.7	1	31752000
Weight of Soil					175077000
				Total Weight	229300948.8
L		48	m		
B		25	m		
Total Weight		229300.949	kN		
F/A (kN/m^2)		191.084124	kPa		
L		8.33333333	m		
B		1	m		
Tributary Area		8.33333333	m^2		
w		191.084124	kN/m		
Max M		1658.71635	kN*m		
Max V		796.18385			
Tension Steel			**Compression Steel**		
Tr	4080000	N	Tr	4080000	N
Phi	0.85		Phi	0.85	
n	8		n	7	
Db	43.7019372	mm	Db'	11.3	mm
Ab	1500	mm^2	Ab'	500	mm^2
As	12000	mm^2	A's	3500	mm^2
Fy	400	MPa	Fy	400	MPa
Cr	2890000	N	Cr'	1190000	N
Phi	0.65		d'	110	mm
Moment, Reinforicement Ratio and Steel Area Check					
Alpha	0.775				
Beta	0.845				
Fc'	50				
a	114.739454	a = (Tr-Cr') / (phi*alpha*fc*b)			
b	1000	mm			
d	500	mm			
rho	0.00781818	properly reinforced			
Asmin	1150	As > As min			
Mr	1743.30149	kN*m	Cr'*(d-d')+Cr(d-a/2)		
Rebar Spacing					
Greater of	1.4*db	61.1827121	mm		
	30mm	30	mm		
	am	10	mm		
tension steel	Less than / E	Ab*(1000/As)	125	mm	
compression steel	Less than / Equal to		428.571429		
	Smax lesser	3h			
		500mm	500		
Slab Dimensions					
Tension Steel	Spacing 10M at 200mm for top				
Compression steel	Spacing 10M at 150mm				
d' (compression)	110				
d	500				
Cover	40	mm			
ds	10	mm			
db	11.3	mm			
Total Height	561.3	mm			
Chosen Height	575	mm			

Sample Calculations:

General Structural Assumptions & Overview:

- The detention tank is split into three structural components: Top slab (vertical flexure), walls (lateral flexure), and bottom slab (crushing).
- The water in the tank will not be present during an earthquake as the probability of two disaster loading scenarios is negligible.
- There is no uplift of the tank due to the existing water table as it is below the structure.
- There is no liquefaction of the soil conditions during the earthquake (Piteau Report, 2002)

Soil Conditions:



$$Weight\ = \rho * g * depth * length * width$$

Weight on top of Top Slab							
Soil Type	**Density (kg/m^3)**	**Depth (m)**	**Pressure (Pa)**	**Width (m)**	**Length (m)**	**Volume (m^3)**	**Weight (kN)**
Silt/Clay	1600	5	78400	25	48	3000	47040
Till	1600	3	47040	25	48	3600	56448
Sand	1555	2	30478	25	48	2400	36574
						Total:	**175077**

$$Vertical\ Earth\ Pressure\ = \rho * g * depth * 0.5$$

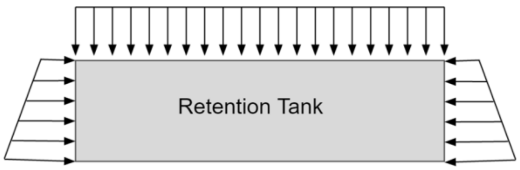
K_O $(Horizontal\ Earth\ Pressure\ Coefficient)\ = \frac{v}{1-v}$, where v is Poisson's Ratio

$$Horizontal\ Earth\ Pressure = \rho * g * depth * 0.5 * K_O$$

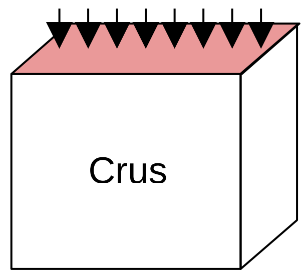
Vertical Pressure on Top Slab							
Soil Type	Density (kg/m^3)	Depth (m)	Vertical Earth Pressure (N/m^2)	Poisson	K_o = v/(1-v)	Horizontal Earth Pressure (N/m^2)	Cumulative (N/m^2)
Silt/Clay	1600	5	39200	0.35	0.5	19600	19600
Till	1600	3	23520	-	0.5	11760	31360
Sand	1555	2	15239	0.25	0.33	5080	36440
Sand	1555	2.7	20573	0.25	0.33	6858	43297

Forces:

The vertical loads are calculated based on the super positioned weight of the soil layers above the Detention tank including self-weight. The lateral loads are based on soil pressures on the walls of the Detention tank.



Crushing of Concrete for vertical walls:



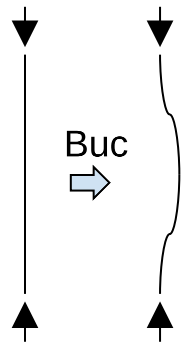
$$P = \frac{N}{A} = \frac{N}{S*T} = 1MPa \ < fc' = 50\ MPa\ OK$$

$N = 175077\ kN, Total\ vertical\ force$

$S = 436\ m, Total\ wall\ length$

$t = 350mm, Wall\ thickness$

Buckling of Walls:



$$Cr = \frac{\pi^2 * E * I}{(k * L)^2} = 160\ MN\ > 402\ kN\ (Force\ per\ metre\ of\ wall\ = \frac{N}{S})$$

$E = 33.2\ GPa\ , Young's\ Modulus$

$I = 175077\ m^4, Moment\ of\ Inertia$

$K = 1\ (dimensionless), Boundary\ Condition\ Coefficient\ (Pin - Pin = Conservative)$

$L = 2.7\ m, Wall\ Height$

Rebar Properties and Slab Properties and Dimensions:

$b = 1000\ mm, slab\ modeled\ as\ one\ meter\ section$

$h = 600\ \ mm, depth\ of\ slab$

$l_n = 8330\ mm, length\ of\ slab\ being\ model$

$\phi_c = 0.65, concrete\ strength\ reduction$

$\phi_s = 0.85, steel\ strength\ reduction$

$\alpha_1 = 0.775$

$f_c = 50\ MPa, strength\ of\ concrete$

$f_y = 400\ MPa, strength\ of\ steel$

Tension Steel

$cover = 40\ mm$

$d_s = 10\ mm, diameter\ of\ stirrups$

$d_b = 35.7\ mm, diameter\ of\ rebar$

$d \quad = 500\ mm, effective\ depth$

$n = 6, number\ of\ rebars$

$A_B \ = 1000mm^2\ , area\ of\ bar$

$As = 7000\ mm^2\ , steel\ area$

Compression Steel

$d' = 110\ mm, effective\ depth$

$d_b = 35.7\ mm, diameter\ of\ rebar$

$n = 6, number\ of\ rebar$

$A_B \ = 1000\ mm^2\ , area\ of\ bar$

$A's = 6000\ mm^2\ , steel\ area$

Rebar Area Checks

Reinforcement Ratio:

$$\rho_s = \frac{A_s}{bd}$$

$$\rho_s{}' = \frac{A_s{}'}{bd'}$$

$$\rho_{s_{total}} = \rho_s - \rho_s{}' = 0.00309$$
$$< 0.0034\ (Reinforcement\ Ratio\ for\ Concrete\ at\ 40MPa, so\ is\ under\ 50MPa\ ratio)$$

Rebar is properly reinforced

Minimum Steel Area:

$A_g = 1000 * h = 1000 * 600 = 600000mm^2, gross\ area$

$A_v = .002 * A_g = .002 * 600000 = 720mm^2\ <\ 7000mm^2, Minimum\ Steel\ Area\ is\ satisfied$

Flexural Stress Demand for Top Slab:

Loading:

$$w = \frac{NA_{trib}}{L} = 109\ kN.m\ , Distributed\ Load$$

$$M = \frac{w * L^2}{8} = 950\ kN.m, \qquad Moment\ at\ L/2\ (Pin - Pin\ assumption)$$

$$N = 175077\ kN, Total\ vertical\ force$$

$$A_{trib} = 6.25\ m^2, Tributary\ area$$

$$L = 8.33\ m, Length\ of\ longest\ slab\ span$$

Resistance:

$$C_r{}' = \phi_s * f_y{}' * A_s{}' = 408kN, compression\ strength\ of\ compression\ steel$$

$$T_r = \phi_s * f_y{}' * A_s{}' = 2380kN, tension\ strength\ of\ tension\ steel$$

$$C_r = T_r - C_r{}' = 2380kN - 408kN = 1972kN, compression\ strength\ of\ concrete$$

$$a = \frac{C_r}{\alpha_1 *\ \phi_c * f'c * b} = 78\ mm, depth\ of\ compression\ block$$

$$M_r = C_r{}'(d - d') + C_r(d - \frac{a}{2}) = 408kN * (500mm - 110mm) + 1972kN * (500 - 78/2)$$
$$= 1068000N\ =\ 1078kN$$

Rebar Spacing

$$S_{max} \leq 3h\ or\ 500mm\ =\ 500mm, maximum\ spacing$$

Tension Rebar

$$S_{max} \leq A_b * 1000/A_s = 142.9mM, maximum\ spacing$$

Spacing for tension is 35M at 130mm

Compression Rebar

$$S_{max} \leq A_b * 1000/A_s{}' = 166.67mm, maximum\ spacing$$

Spacing for tension is 35M at 150mm

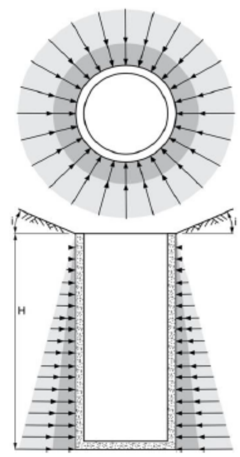
F.2 Baffle Drop Structure

Summary

Baffle Drop Shaft Design Calculations:

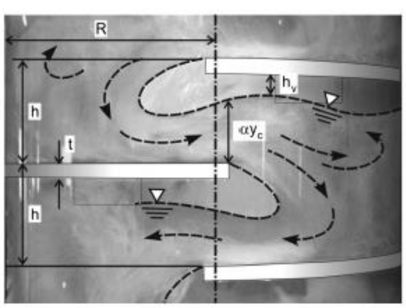
Dimension	Value	Unit
Detention Tank Outflow, Q	3.98	m3/s
Gravity	9.8	m2/s
Inside Diameter Shaft, Di	3.31	m
Shaft Wall Thickness, St	200	mm
Dividing Wall Thickness, Wt	150	mm
Outside Diameter Shaft, Do	3.86	m
Wet Side Diameter, Dw & Baffle Width, Bw	2.10	m
Dry Side Diameter, Dd	1.05	m
Baffle Spacing, Sb	1.55	m
Baffle Thickness, Bt	0.25	m
Shaft Height	60	m

Geotechnical Design	Value	Unit
Lateral Earth Pressure, P	16.13H	kN/m2
Unit Weight Sand, ys	19	kN/m3
Unit Weight Water, yw	9.8	kN/m3
Coefficient of Friction, Ks	0.33	
Soil Friction Angle, o	30	degrees
Compression Stress Induced, S	0.097H	MPa
Allowable Compression Stress in Shaft, 0.45f'c	12.6	MPa
Water Table Height, w	20	m
Compressive Strength Concrete, f'c	28	MPa
Maximum Allowable Depth, Hmax	129.9	m



Concrete "Pile" Design	Value	Unit
Base Area of Shaft, Ab	11.68	m2
Bearing Capacity Factor, Nq	40	
Overburden Pressure, po'	552	kN/m2
End Bearing Tip Resistance, Qb	12,257	kN
Dead Load Shaft, Qd	4,452	kN
Unit Weight Reinforced Concrete, yc	24	kN/m3
Volume of Concrete Shaft	185.5	m3
Required Factor of Safety, FSr	2.5	
Factor of Safety, FS	2.75	

Structural Concrete Reinforcement	Value	Unit
25M Rebar Longitudinal/Hoop , db	25	mm
25M Longitudinal/Hoop Area, Ab	500	mm2
Minimum Concrete Cover	40	mm
Minimum Shaft Thickness, tmin	130	mm
Minimum ConcreteSteel Ratio, pmin	0.03	
Minimum Rebar Volume, Vsmin	5.57	m3
Spacing of Longitudinal Rebar, Sb	500	mm
Spacing of Hoop Rebar Cage, Sh	350	mm
Design Requirements, Long Rebar	22-25M @ 500	
Design Requirements, Hoop Rebar	12-25M @ 350	



Sample Calculations

$$Inside\ Diameter\ Shaft, D_i = \frac{3}{g^{1/5}} Q^{2/5} = \frac{3}{(9.8)^{1/5}} (3.98)^{2/5} = 3.31\ meters$$

$$Outside\ Diameter\ Shaft, D_o = D_i + W_t + 2S_h = 3.31 + 0.15 + 2(0.2) = 3.86\ meters$$

$W_t = Dividing\ Wall\ Thickness$

$S_h = Shaft\ Wall\ Thickness$

$$Wet\ Side\ Diameter\ D_W\ or\ Baffle\ Width, B_w = \frac{2}{3}(D_i - W_t) = 2.11\ meters$$

$$Dry\ Side\ Diameter\ D_D = \frac{1}{3}(D_i - W_t) = 1.05\ meters$$

$$Baffle\ Spacing\ S_B = \frac{1}{0.55}\left(\frac{Q^2}{{B_w}^5 g}\right) + B_t = 1.55\ meters$$

$B_t = Baffle\ Thickness$

(a) (b) (c)

$$Minimum\ Wall\ Thickness, t_{min} = 2cover + d_b + d_h = 2(40mm) + 25mm + 25mm = 130mm$$

$d_b = 25M\ Rebar\ Reinforcement\ Diameter$

$d_h = 25M\ Hoop\ Rebar\ Reinforcement\ DiamterS$

$$Lateral\ Earth\ Pressure, P = y_s H K_s + y_w H = \left(\frac{19kN}{m3}\right)(H)(0.33) + \left(\frac{9.8kN}{m3}\right)(H) = 16.13H$$

$y_s = Unit\ Weight\ Sand\ 19kN/m3$

$y_s = Unit\ Weight\ Sand\ 19kN/m3$

$H = Shaft\ Height$

$K_s = Coefficient\ of\ Friction\ (1 - sin\ o)/(1 + sin\ (o)\ ),\ where\ o = \ 30\ degrees.$

$$Compression\ Stress\ Induced, S = \frac{PD}{2t} = \frac{(16.13H)(D)}{(1000)(2)(\frac{1}{12H})(D)} = \ 0.097H$$

$Allowable\ Compression\ Stress, 45\%\ of\ S. \rightarrow 0.45(28MPa) = 12.6MPa = 0.097H$

$Maximum\ Allowable\ Depth\ of\ Shaft, H = \frac{12.6MPa}{0.097} = 130\ meters$ > 60 meters

$End\ Bearing\ Tip\ Resistance\ of\ Concrete\ Shaft, Qb = Nqpo'Ab$

$$= (40)(\frac{19KN}{m3})(60m) * (\frac{3.7m}{2})^2(pi) \quad = 122257\ kN$$

$N_q = Bearing\ Capacity\ Factor$, 40

$Ab = Base\ Area\ of\ Shaft$

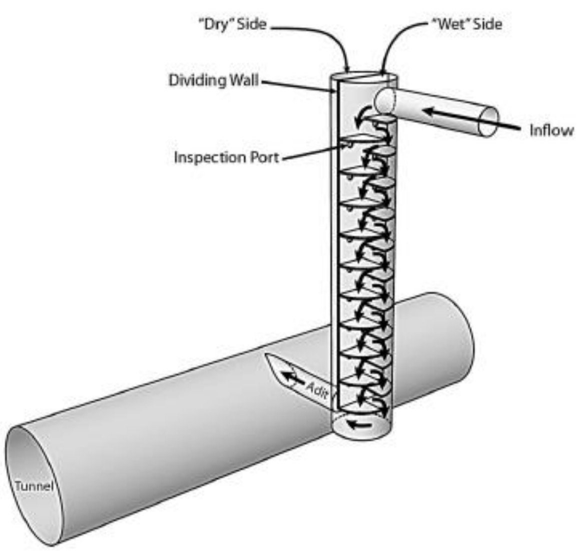
$po' = Overburden$ Pressure

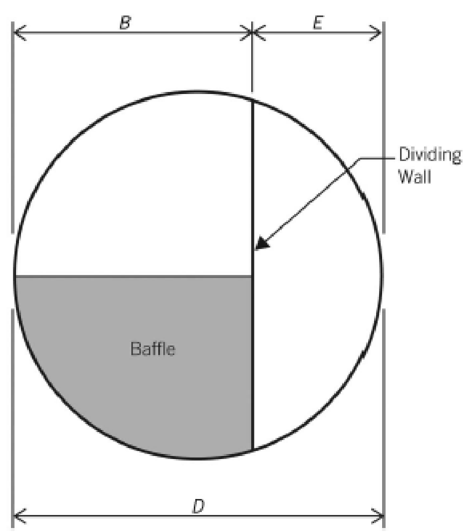
$$Dead\ Load\ of\ Concrete\ Shaft, Qd = y_cV_c = (\frac{24kN}{m3})(60m) * (\frac{3.7 - 3.3m}{2})^2(pi) = 4452\ kN$$

$y_c = Unit\ Weight\ of\ Reinforced\ Concrete, 24kN/m3$

$V_c = Volume\ of\ Concrete\ Shaft$

$$Factor\ of\ Safety, FS = \frac{Qb}{Qd} = \frac{12257\ kN}{4452\ kN} = 2.75 > FS = 2.5$$





F.3 Horizontal Shaft and Outfall

F.3.1 Summary

Soil Parameters			**Jacking and Tunnlling Loads**		
Parameter	**Quantity**	**Unit**	**Tunnel Face Pressure**		
Density	2000	[kg/m^3]	**Parameter**	**Quantity**	**Unit**
c'	0	[kPa]	σ_v	1275	[kPa]
v	0.47	[-]	K_0	0.3843	[-]
ϕ	0.66	[rad]	σ_h	490	[kPa]
Young's Modulus, E_S	200	[Mpa]	Face pressure	854	[kN]
Bulk Modulus, K	2000	[kPa]			
λ_sat	19.6	[kN/m^3]	**Tunnel Bore Stability**		
T_λ	0.3	[-]	**Parameter**	**Value**	**Unit**
			σ_T	8.8	[kPa]
Pipe Properties					
Parameter	**Quantity**	**Unit**	**Ground Closure**		
t	0.145	[m]	**Parameter**	**Value**	**Unit**
D_p	1.49	[m]	δ_v	1.30	[mm]
d	1.2	[m]	δ_h	0.75	[mm]
D_e	1.59	[m]	δ_p	0.03	[mm]
L (effective)	2.477	[m]			
Unit weight of pipe	25.0	[kN/m^3]	**Tunnel Face Load**		
W/meter	15.3	[kN/m]	**Parameter**	**Quantity**	**Unit**
Permissible Jacking Lo	6477	[kN]	45-ϕ/2	0.45	[rad]
			45+ϕ/2	1.12	[rad]
			B	0.60	[m]
Depth	66	[m]	H	63.51	[m]
H/D	44.3	[-]	K_0	0.384	[-]
Maximum Depth to in	65	[m]	K	0.238	[-]
Depth to Axis	64.255	[m]	ktan(phi)/B	0.3090	[-]
δ	0.577	[rad]	σ_v	63.5	[kPa]
			σ_h	18.8	[kPa]
			σ_P	63.5	[kPa]
			F	134	[kN/m]
			Total Front-En	2258	[kN]

Pipe Dimension Design				Jacked Pipe		
Paramete	Quantity	Unit		Parameter	Quantity	Unit
HW	0.39804	[m]		Pipe Material	Reinforced Concrete Pipe (RCP)	
Q	3.98	[m^3/s]		Pipe Internal Diameter	1.2	[m]
D	1.2	[m]		Tolerance:	(+/-)25	[mm]
A	1.13	[m^2]				
s	0.005	[%/100]		CSA/CAN A257.2-09	Reinforced circular concrete culvert, storm drain, sewer pipe and fittings	
K_u	1.811	[-]		ASCE 27-00 Standard practice for Direct Design of Precast concrete pipe for jacking in trenchless technologies		
K	0.0098	[-]				
M	2	[-]		**Earth Load**		
HW/D	0.3317			W_e	9800	lb/ft
K_u(KQ/A	0.3317			W_e	143.0	kN/m
				W_t	133.7	
				Water in pipe load		
				λ_w	9.807	[kN/m^3]
				A	1.1	[m^2]
				W_f	11.1	[kN/m]
				Live Load		
				W_L	0	[kN/m]
				Selection of Bedding		
				Grouting will be used in the annular space around the excavation of the pipe		
				since little flexural stress will be induced, a bedding factor of 3.0 is used.		
				Factor of Safety		
				F.S.	1.2	[-]
				D-Load		
				D	48.3	[kN/m]
				Axial loads due to jacking		
				ϕ	0.9	[-]
				f_c'	32	[Mpa]
				LF_J	1.2	[-]
				f	20.4	
				P	9.0	[kN]

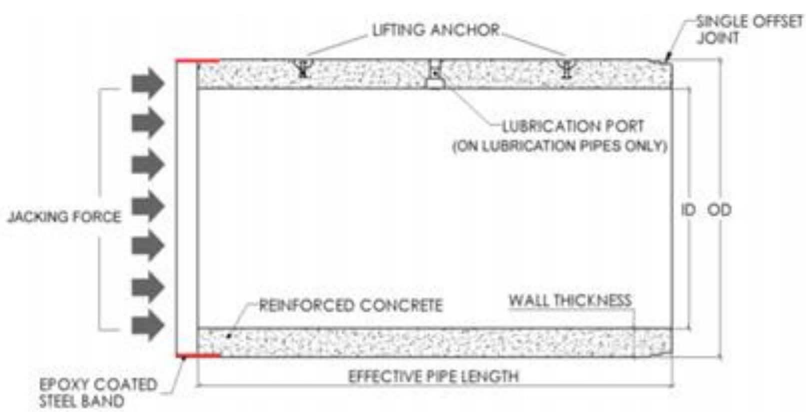


Figure 3 DECAST Ltd. Precast Steel Reinforced Concrete Pipe

Source: DECAST Ltd, 2017

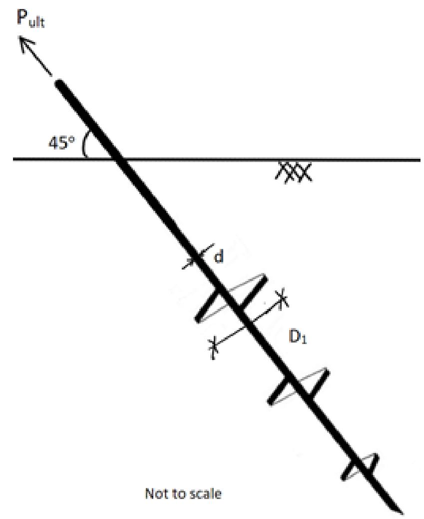


Figure 5 Tri-helical ground anchor installed at 45 degree angle

Source: Michael Louws, 2017

F.3.2 Sample Calculations

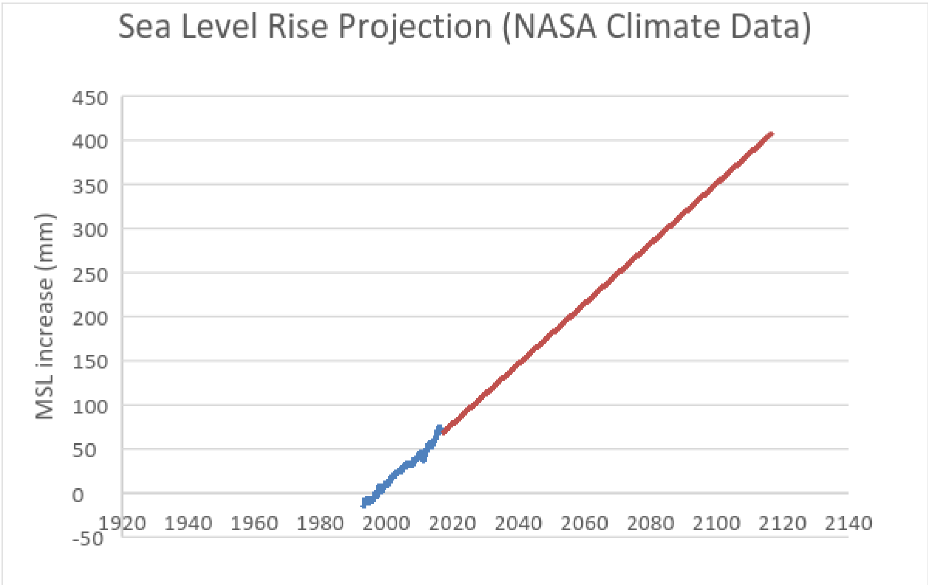
<u>Outfall Design:</u>

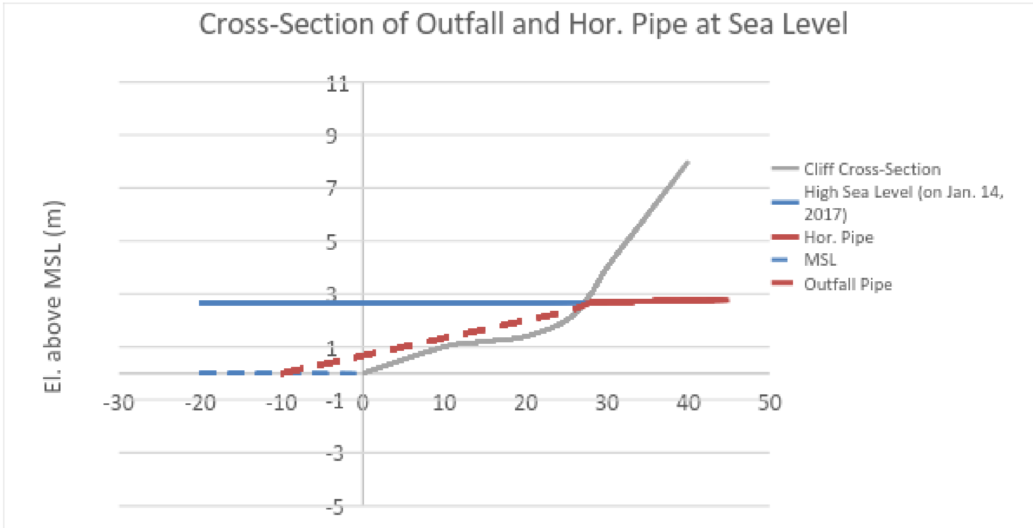
The outfall design takes into consideration several factors. Sea water presence in the region presents unique issues for material design. Tidal considerations and sea level rise are important in determining the elevation of the outfall shaft.

Tidal and Sea Level Considerations

NASA climate data was used to predict the projected 100 year sea level rise. A linear relationship between time and sea level was assumed, while recognizing that the current view of the relationship has insufficient data to know if the relationship is linear or some other correlation. The projected sea level rise in 100 years is based on the 1997 MSL. Elevation data taken from Google maps was used to plot a cross section of the cliff face nearest the location of the new shaft and outfall, and the soil profile was plotted according to the Piteau Geotechnical report. The MSL of Google earth is assumed to be approximately 1997. The projected sea level rise in 100 years is approximately 409mm.

Projected 100y Sea Level Rise	0.41	[m]
Max high tide (above MSL)	2.26	[m]





In order to ensure the bottom of the horizontal shaft is unsubmerged in saline water during high tide, the bottom elevation of the horizontal pipe must be greater than the maximum anticipated high tide during the design life of the project. Based on the calculation of expected high tide in the next 100 years, the base elevation of the horizontal pipe must be some elevation greater than 2.67m above current MSL. The plot above shows a possible outfall structure, which in this case would lie at a slight elevation above the current beach. A rock protection of the outfall is suggested for aesthetic reasons and to protect the pipe from weathering.

Concrete Design

For the concrete design of the outfall structure, saline conditions were assumed, since during high tide the majority of the structure was assumed to be submerged. CSA A23.1 standards for concrete were used in assessing the design parameters of the concrete. These parameters are summarized in Table x.x below.

CSA Exposure Class	**C-1**	
Max w/c	0.4	
Min compressive strength @ 28d	35 MPa	
Air content	5-8% (10mm agg.) or 4-7% (14-20mm agg)	
Curing Type	Curing type 2	7 d at >/= 10°C and for a time necessary to attain 70% of the specified strength. When using silica fume concrete, additional curing procedures shall be used
Chloride ion penetrability test requirements and age	<1500 Coulombs w/in 56d	

Scour Protection

An armoured apron will be provided to ensure erosion does not occur at the location of transition from pipe to beach. In the case of low tides, the apron will serve a as a flow spreading apron. A small wing wall will be located at the exit of the pipe, as seen in figure x.x.

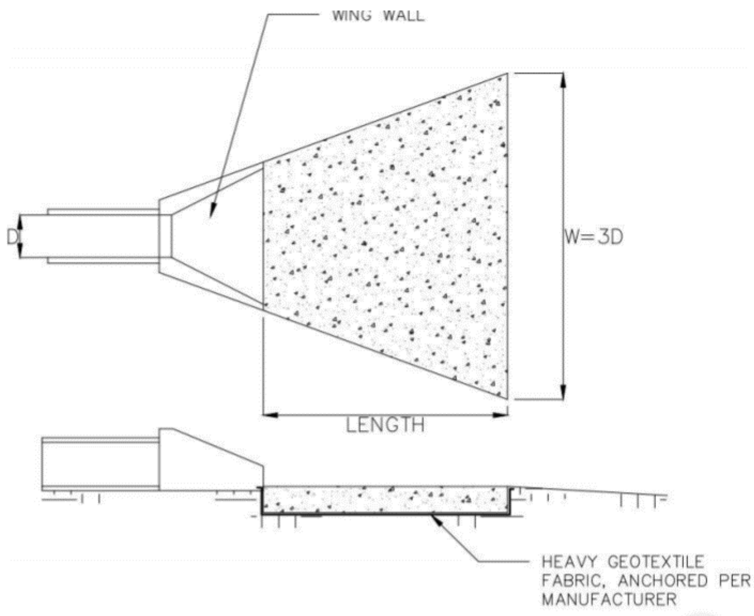


Figure 1 Riprap apron and wing wall of outfall

(Source: Auckland Council, 2013)

The dimensions of the outfall apron, and sizing of riprap are calculated below.

$W_a \geq 3D_0$ that is, the width of the apron must be greater than 3 times the internal diameter of the pipe.

$L_a = D_0(8 + 17 \times logF_0)$

where

$W_a = apron\ width, m$

$D_0 = pipe\ diameter, m$

$F_0 = Froude\ number$

$L_a = length\ of\ apron, m$

Apron Length (m)	Apron Width (m)
6.5	5

Horizontal Shaft:

The construction of the Horizontal shaft section is influenced almost entirely by the make of the local soil conditions. This section of the report will consider the soil conditions, local geology and estimation of design loads on the tunnel. Construction methodology will also be considered.

Hydraulic Design of Pipe

The hydraulic design of the pipe is based on hydraulic engineering theory for a culvert with unsubmerged inlet and outlet flow, and the assumption that submerged flow is not desired for the pipe.

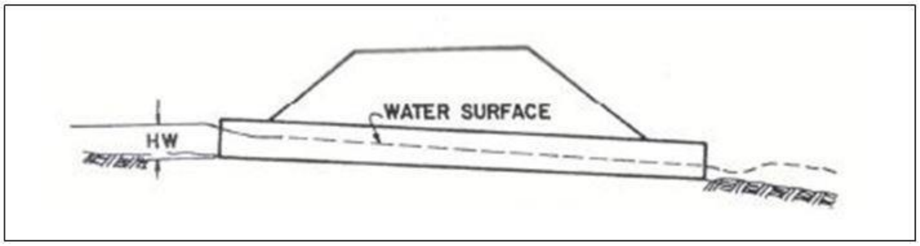


Figure 2 Unsubmerged inlet and outlet culvert

(Source: 2015, Quek Hong)

$$\frac{HW_i}{D} = K\left[\frac{K_u Q}{AD^{0.5}}\right]^M$$

Where

$HW_i = headwater\ depth\ above\ inlet\ control\ section$

$D = interior\ height\ of\ culvert\ barrel$

$Q = max\ discharge\ as\ determined\ in\ section\ XXXXXXXXXXXX$

$K_u = 1.811\ in\ SI\ units$

$A = cross - sectional\ area\ of\ pipe\ barrel$

$K, M = 0.0098, 2\ square\ edge\ with\ headwall\ entrance$

With this equation, a headwater depth was assumed, and D estimated until the equations were equal, yielding the minimum diameter required in order for the flow to meet the flow demand with the available approximate head water depth.

Construction Method:

In order to calculate design loading and consequently materials for the pipe, the method of construction is first considered. Typical stormwater drain tunnels are constructed in two categories of methods: trenchless and trenched. Due to the depth of the pipe below surface (up to 62m bel ow grade), trenched methods commonly used in stormwater drainage systems are not possible for this project. Thus, two trenchless technologies are considered: Horizontal Directional Drilling and Microtunneling and Jacking.

A comparison of methods can be seen in the table below.

	Horizontal Directional Drilling	Microtunnelling and Jacking
Tolerance	+/-25mm	+/-100mm
	Pit-launched	Surface -launched
Initial cost	Lower	Higher
Diameter	<1200mm	<3400mm

While the methods are quite similar in terms of cost, a couple key factors are at play here. The diameter of the pipe required in order to adequately meet the design flow requires an external pipe diameter greater than the maximum pipe diameter drilled with a HDD. Also, the microtunnelling method is designed to be employed from a vertical access shaft, while the HDD method is designed for use from surface elevation. Thus, the obvious choice is Microtunnelling with a Micro Tunnel Boring Machine (MTBM) as seen in figure x.x.

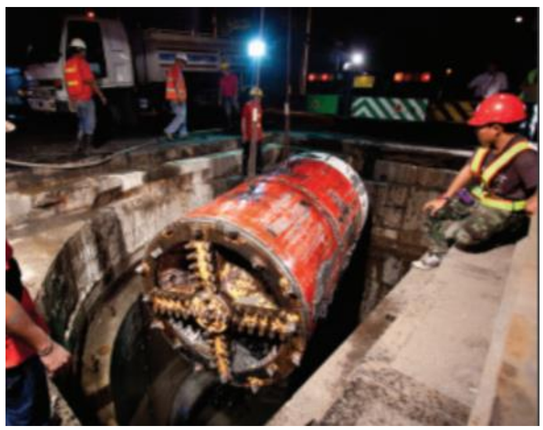


Figure 3 MTBM being lowered into access shaft

(Source: Microtunnelling Systems)

Estimation of Design Loads:

The design manual from the Ontario Concrete Pipe Association was used in determining the design loads on the horizontally drilled pipe. During micro-tunnelling there are two categories of loads: long term and short term loads. Long term loads are the earth loads, groundwater, and internal pressure loads, while short term loads are the concentric jacking force.

The Indirect Method is used to determine the Design load or D-Load of the final pipe. Based on this D-load, the CSA 257.2-09 can be used to select the design class of reinforced concrete pipe required for the project.

Earth Load:

The earth loads on the pipe were calculated using the methods described in both the *Concrete Pipe Design Manual* of the Ontario Concrete Pipe Association and the American Concrete Pipe Association manual of the same title. The required pipe strength in terms of a 0.3mm crack was found. Tables in the

ACPA manual were used to determine the values for the equation below, rather than calculate the C_t value as seen below.

$$W_E = C_t w g B_t^2 - 2cC_t B_t$$

The resultant $W_E = 133.7kN/m$

Live Load:

The live load is negligible at a depth of 62m.

Water Load in Pipe:

The maximum water load in the pipe was assumed to be if the pipe was fully submerged (though the design diameter prevents this from occurring).

$$W_f = \gamma_w A$$

The W_f was found to be 9.3kN/m.

Bedding:

The bedding will simply be grout that is placed in the annular space between the pipe and the edge of the excavation. Since the space will be filled with grout, the bedding condition is ideal, and a conservative factor of bedding of 3.0 is used.

Factor of Safety

A factor of safety of 1.2 was assumed.

D-Load:

The D-load is calculated with the following formula:

$$D - load = \frac{W_L + W_E}{B_f D} F.S.$$

The D-load is found to be 52 kN/m.

Axial Loads:

The axial loads are calculated with the ASCE Standard Practice outlined in the ACPA Design Manual. Assuming a linear distribution across the entire joint, the following equation exists:

$$f = \frac{0.85\phi f_c'}{LF_J}$$

Where

$\phi = 0.9, strength\ reduction\ factor\ for\ compressive\ axial\ thrust$

$f_c' = 35\ MPa, design\ strength\ of\ concrete$

$LF_J = 1.2, load\ factor\ for\ jacking\ thrust$

The maximum jacking force should not exceed

$$P = 0.5fA_p$$

$$A_p = contact\ area\ between\ joints, m^2$$

Calculating yields P=8.3kN.

Based on these design loads, a pipe can be selected for the project. For the purposes of cost, the most feasible option is Reinforced Concrete Pipe (RCP). CSA 257.2 could be used to determine the Pipe design specifications, but CSA standards remain unavailable to the authors of this report.

<u>Jacked Microtunnelling Design Calculations</u>

Methodology for calculating the design loads for jacking and tunnelling are summarized here. These calculations are based on the *Guide to Best Practice for the Installation of Pipe Jacks and* Microtunnels from the Pipe Jacking Association of the UK (Guide, 1995).

Soil and Pipe Parameters

Pipe design based on previous loading requirements, checking for meeting permissible jacking force.

Tunnel Face Pressure

In order to prevent movements of the drilling face, the slurry pressure in the tunnel should slightly greater than the horizontal stress.

$$K_0 = 1 - \sin(\phi)$$

$$\sigma_h' = K_0 \times \sigma_v'$$

Therefore the tunnel face loading is 854 kN. That is the slurry pressure must be kept above this value to ensure face stability.

Tunnel Face Load

In addition to the face pressure calculated above, there will also be horizontal forces related to the cutting edge of the MTBM and friction between the shield and the ground (even though lubrication will be used. Lubrication in the first few pipes is limited. This length extends 4m for the shield and 5 meters for the first two pipe sections.

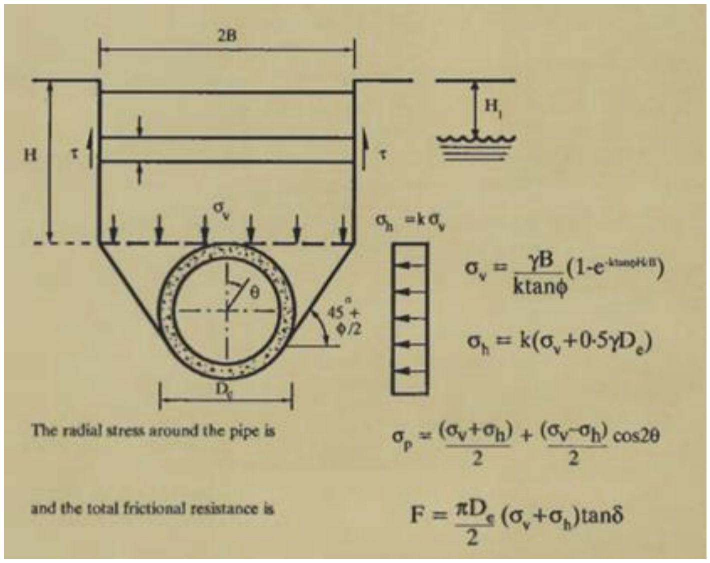


Figure xx Face Loading Diagram

Source: Design Guide, 1993

$$2B = D_e \tan\left(45° - \frac{\phi}{2}\right) + \frac{D_e}{\sin\left(45° + \frac{\phi}{2}\right)}$$

$$H = depth\ to\ axis - \frac{D_e}{2}$$

$$\frac{Ktan(\phi)}{B} = 0.309$$

$$\sigma_v = \frac{H}{0.309}(1 - e^{-0.309 \times H})$$

$$\sigma_h = K(\sigma_v + 0.5\gamma_{soil} D_e)$$

Frictional Resistance

$$F + \frac{\pi D}{2}(\sigma_v + \sigma_h)tan\delta$$

$$\delta = 0.87\phi$$

Tunnel Bore Stability

The pressure required to ensure stability of the tunnel bore is given by:

$$\sigma_T = \gamma_b D_p' T_\gamma + water\ pressure$$

$$T_\gamma = value\ from\ chart$$

Ground Closure

Ground closure is calculated based on the horizontal and vertical stresses

$$\delta_v = \frac{(1-\nu^2)D_e}{E_S}(3\sigma_v + \sigma_h)$$

$$\delta_h = \frac{(1-\nu^2)D_e}{E_s}(\sigma_v + 3\sigma_h)$$

Appendix G – Project Management Documents

G.2 Stakeholder Engagement Plan

Stakeholder	Direct Contact	Areas of Influence / Interest	Involvement	Influence	Engagement Tools	Frequency
University of British Columbia (Client)	Mr.Doug Doyle, P.Eng, Associate Director, UBC Infrastructure and Planning	Project reflects institutional values	High Project Involvement	High	Face to Face Meetings, E-mails	Bi-Weekly
Metro Vancouver	Simon So, GM, Liquid Waste Services	Planned outfalls on Metro Vancouver Property Regulatory Compliances	High Project Involvement	High	Meetings upon Request, High Level Information Notices	As requested
Musqueam Indian Band	Wayne Sparrow, Chief	Project land is owned by Musqueam	Moderate Project Involvement	Moderate	One Kickoff Meeting, and afterwards Information and Consultation Meetings upon request	As requested
Government of Canada, Department of Fisheries and Oceans	Angela Bate, Area Director, Fraser and BC Interior	Regulation Compliances	Moderate Project Involvement	Moderate	Information and Consultation Meetings upon request	As requested
B.C. Ministry of Transportation and Infrastructure	Ed Miska, Director of Highway Design	Ministry Road near Project which may be affected	Moderate Project Involvement	Moderate	Information and Consultation Meetings upon request	As requested
Pacific Spirit Park Society	Bob Meyer, Chair Board of Directors	Environmental Protection	Low Project Involvement	Low	Information and Consultation Meetings upon request	As requested
Local Residents	Sabrina Zhang, UNA, Chair of Civic Engagement	Neighbourhood Impacts	Low Project Involvement	Low	Public information Meeting, Online Survey, Public Information Notices	At project milestones (ex. Conceptual Design, Preliminary Design)

G.2 Land Use Inventory

	TYPE OF USE	HECTARES	ACRES
1	Campus area	326	805
2	Built footprint	57	141
3	Building setback allowance (15% of footprint)	9	21
4	Roads and parking	54	134
5	SUBTOTAL (LINE 1–LINES 2-4)	206	509
	Open Space/ Outdoor Research		
6	Parks	11	27
7	Plazas/ Courtyards	4	9
8	Varsity/ Recreation	23	57
9	Gardens	5	12
10	Corridors	8	20
11	Outdoor Research Space[1]	61	150
12	TOTAL OPEN SPACE/OUTDOOR RESEARCH (LINES 6–11)	111	275
	Remainder (line 5–line 12)	95	235

[1] Outdoor research space includes 24.0 ha Farm, 1.5 ha Totem Field Research, 12 ha Bioscience Reserve, 23.5 ha Botanical Gardens.

Data source for open space components and Botanical Gardens is "Public Spaces Study" (2007) by C. Berris Associates.

Appendix H – Cost Estimate

UBC North Campus Spiral Drain Replacement Project					
Final Cost Estimate					
Permitting:					
Item	*Quanity*	*Unit*	*Unit Cost*	*Cost*	*Description*
Development Permit	1,500	m2	3	4,500	Apply to Both UBC and Metro Vancouver
Noise Bylaw Permit	1	EA	0	0	N.A. - Working during permitted hours
Water & Sewer Connection Permit	1	EA	5,000	5,000	Estimated Permit Value
Tree Removal Permit	1	EA	2,500	2,500	Estimated Permit Value
			Total:	**12,000**	
Design & Engineering:					
Item	*Quanity*	*Unit*	*Unit Cost*	*Cost*	*Description*
Design					
Pre-Construction Land Survey	1	EA	25,000	25,000	
Geotechnical Site Investigation	1	EA	20,000	20,000	Detailed geotechnical investigation,boreholes
			Subtotal:	**45,000**	
Engineering					
EIT Engineers	800	HR	130	104,000	6 EIT members, sum of total person-hours
Senior Technical Engineer	30	HR	293	8,790	Review Meetings with Dr. Li, P.Eng
CAD Drafting	100	HR	109	10,900	
QA/QC (Technologist)	240	HR	141	33,840	1 for Duration of the Project - Part Time
			Subtotal:	**157,530**	
			Total:	**202,530**	
Project Management (UBC Infrastructure):					
Item	*Quanity*	*Unit*	*Unit Cost*	*Cost*	*Description*
Project Manager	250	HR	293	73,250	1 for Duration of Project - 1/4 Time
Site Engineer	1000	HR	145	145,000	1 for Duration of Project -Full Time
			Total:	**218,250**	
Construction Contractor:					
Item	*Quanity*	*Unit*	*Unit Cost*	*Cost*	*Description*
General					
Mobilization/Demobilization to Site	2	LS	80,000	160,000	Overall Mob/demob to/from Site
			Subtotal:	**160,000**	
Transportation of Goods					
Shipping of Materials	1	LS	100,000	100,000	
			Subtotal:	**100,000**	
Site Preparation					
Survey	1	LS	75,000	75,000	
Perimeter Fencing & Signage	3,600	LM	3	10,800	Renting of Fencing
Full Depth Removal	120	M2	25	3,000	Part of Asphalt Removal of Parking Lot
Site Clearing & Grubbing	2,000	M2	2	3,000	Clearing of Trees and Bushes on site
Strip Topsoil	2,000	M2	3	6,000	Clearing of Grass and Topsoil on site
			Subtotal:	**97,800**	
Installation of Detention Tank					
Installation of Temporary Lateral Support	146	LM	675	98,550	Sheet Piles embedded 15 meters in ground
Excavation	12,240	M3	18	220,320	
Supply of 250mm Dia Concrete Piles	28	EA	2,500	70,000	4m Length Short Piles
Pile Driving of Concrete Piles	112	LM	225	25,200	
Supply and Install Geotextiles	2,000	M2	5	10,000	
Supply and Install Granular Base	200	M3	250	50,000	
Supply and Install Cast-In Place Concrete	1,825	M3	475	866,875	Formwork, pouring, finishing, Rebar Reinforcement
Install Inlet and Outlet Connections	2	EA	15,000	30,000	
Supply and Install Backfill	1,000	M3	10	10,000	
			Subtotal:	**1,380,945**	
Installation of Baffle Drop Shaft					
Vertical Drilling of 4m Diameter Shaft	65	VM	15,000	975,000	Vertcal Auger Rig.
Supply and Install Granular Base	10	M3	250	2,500	
Supply and Install Drop Shaft Segments	15	EA	36,000	540,000	4 m Segments
Supply of Permanent 4m Dia. Steel Casing	65	LM	4,800	312,000	

Pile Driving of 4m Dia. Steel Casing	65	LM	1,400	91,000	
Splicing of Steel Casing	5	EA	5,000	25,000	Every 12m Splice
Supply and Install Concrete Grout	225	M3	148	33,300	
			Subtotal:	**1,978,800**	
Installation of Horizontal Shaft & Outfall					
Site Mob of Equipment to Tower Beach	1	LS	60,000	60,000	
Install Temporary Cofferdam	1	LS	125,000	125,000	Sheet Piles, Dewatering, Stabilization, etc.
Supply 1400mm Reinforced Concrete Pipe	120	LM	1,350	162,000	2.5m segments
MTBM Drilling of Concrete Pipe Segments	120	LM	5,160	619,200	Microtunneling and Jacking of segments
Tie-In of Pre-cast Horizontal Pipe to Shaft	1	EA	17,500	17,500	
Supply and Installation of Outfall	1	EA	85,000	85,000	
Supply and Install Corrosion Protection	1	LS	15,000	15,000	
			Subtotal:	**1,083,700**	
Re-routing of Stormwater					
Supply and Install New Pipes	240	LM	600	144,000	(Excavation, Laying, Backfill)
Tie-Ins	5	EA	1,500	7,500	
			Subtotal:	**151,500**	
Re-storing of Site					
Cast-in-place Curb	50	LM	120	6,000	
Asphalt Paving - Supply and Install	120	M2	7.0	840	
Topsoil Placement	2,000	M2	7.9	15,720	
Supply and Install Landscaping	1,500	M2	5.0	7,500	
			Subtotal:	**30,060**	
Total Direct Cost				**4,982,805**	
Indirect Cost				**1,494,842**	30% of DC- Superintendent, Field Engineer, Labour
Total Cost				**6,477,647**	
Additional Costs					
Contigency	1	LS	0.2	1,295,529	20% of Total Cost
Escalation	1	LS	3.0	194,329	3% for 2018
Insurance	1	LS	1.5	97,165	1.5% of Total Cost
Bonding	1	LS	1.0	64,776	1% of Total Cost
Total Contractor Cost			**Total:**	**8,129,446**	
Total Capital Costs:				**8,562,226**	**Permits, Design and Engineering, PM, Contractor**
Operation & Maintenance:					
Item	*Quanity*	*Unit*	*Unit Cost*	*Cost*	*Description*
Yearly Inspection	100	EA	1,400	140,000	100 Year Design Life (NPV)
Yearly Maintenance	100	EA	8,500	850,000	100 Year Design Life (NPV)
			Total:	**990,000**	
Total Cost (Intital Capital Costs and O&M):				**9,552,226**	

Notes regarding Cost Estimate:

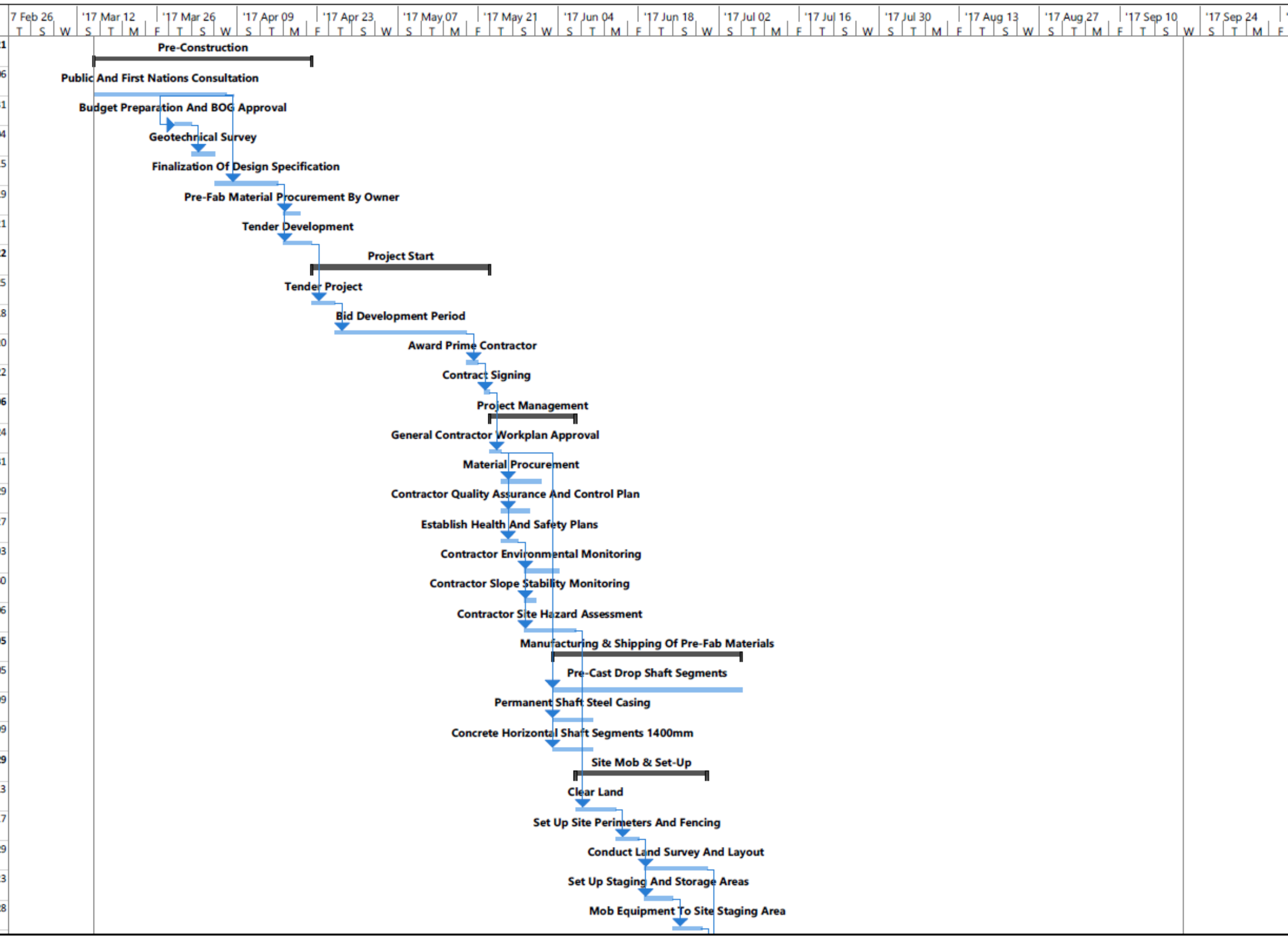
Direct Cost corresponds to the cost of materials and equipment required for construction.

A contractor profit margin of 7% is also included in the unit price.

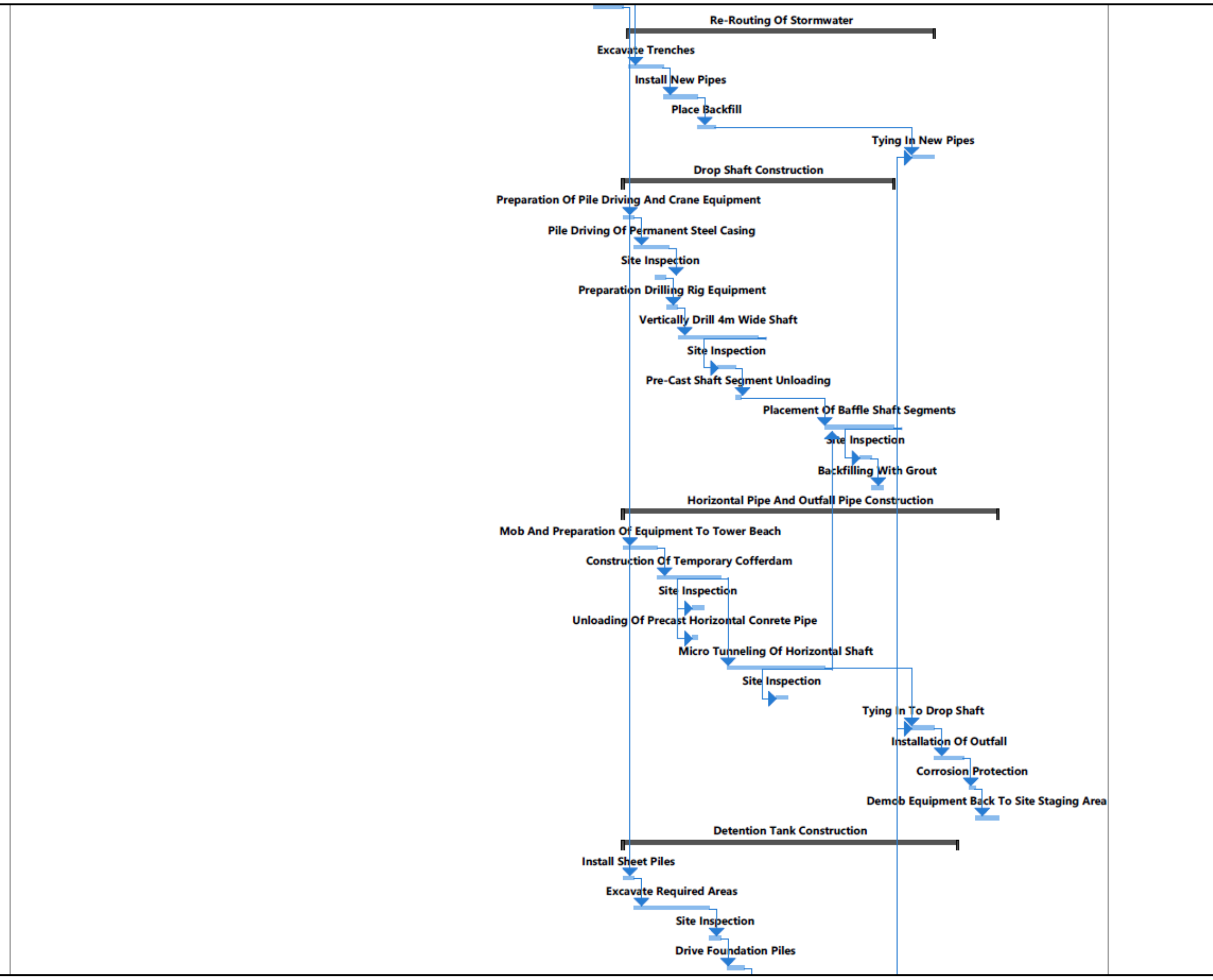
Indirect Cost corresponds to labour and consumables.

Appendix I – Construction Schedule

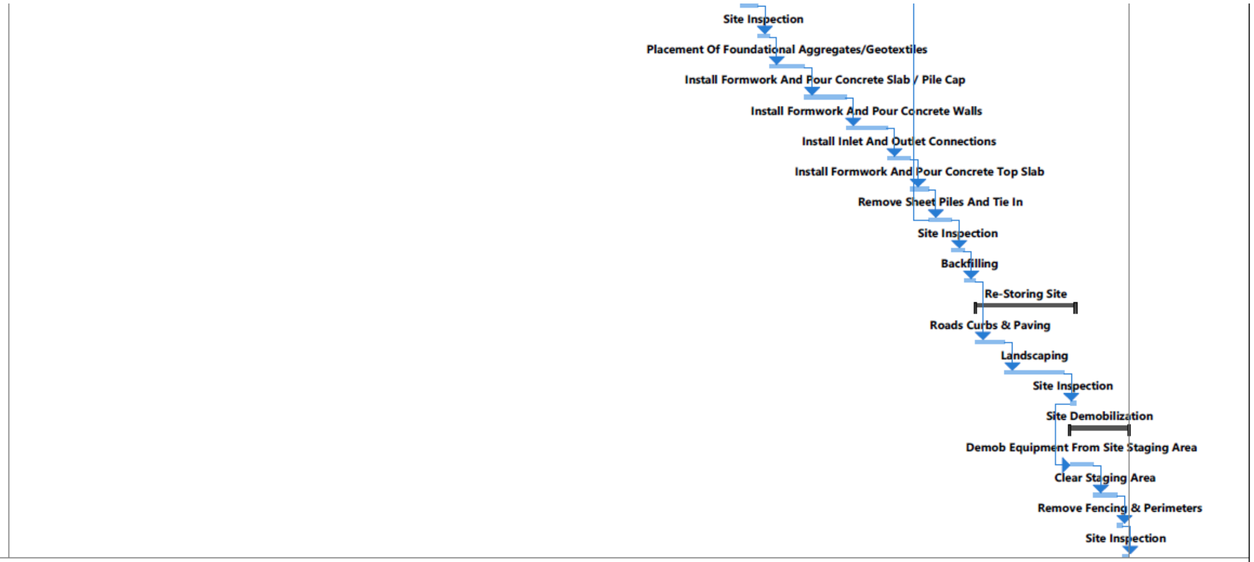
ID	Name	Duration	Start	Finish
1	**Pre-Construction**	**33 days**	**Wed 17-03-15**	**Fri 17-04-21**
2	Public And First Nations Consultation	20 days	Wed 17-03-15	Thu 17-04-06
3	Budget Preparation And BOG Approval	3 days	Wed 17-03-29	Fri 17-03-31
4	Geotechnical Survey	3 days	Sat 17-04-01	Tue 17-04-04
5	Finalization Of Design Specification	10 days	Wed 17-04-05	Sat 17-04-15
6	Pre-Fab Material Procurement By Owner	3 days	Mon 17-04-17	Wed 17-04-19
7	Tender Development	5 days	Mon 17-04-17	Fri 17-04-21
8	**Project Start**	**26 days**	**Sat 17-04-22**	**Mon 17-05-22**
9	Tender Project	3 days	Sat 17-04-22	Tue 17-04-25
10	Bid Development Period	20 days	Wed 17-04-26	Thu 17-05-18
11	Award Prime Contractor	2 days	Fri 17-05-19	Sat 17-05-20
12	Contract Signing	1 day	Mon 17-05-22	Mon 17-05-22
13	**Project Management**	**13 days**	**Tue 17-05-23**	**Tue 17-06-06**
14	General Contractor Workplan Approval	2 days	Tue 17-05-23	Wed 17-05-24
15	Material Procurement	6 days	Thu 17-05-25	Wed 17-05-31
16	Contractor Quality Assurance And Control Plan	4 days	Thu 17-05-25	Mon 17-05-29
17	Establish Health And Safety Plans	3 days	Thu 17-05-25	Sat 17-05-27
18	Contractor Environmental Monitoring	6 days	Mon 17-05-29	Sat 17-06-03
19	Contractor Slope Stability Monitoring	2 days	Mon 17-05-29	Tue 17-05-30
20	Contractor Site Hazard Assessment	8 days	Mon 17-05-29	Tue 17-06-06
21	**Manufacturing & Shipping Of Pre-Fab Materials**	**28 days**	**Sat 17-06-03**	**Wed 17-07-05**
22	Pre-Cast Drop Shaft Segments	28 days	Sat 17-06-03	Wed 17-07-05
23	Permanent Shaft Steel Casing	6 days	Sat 17-06-03	Fri 17-06-09
24	Concrete Horizontal Shaft Segments 1400mm	6 days	Sat 17-06-03	Fri 17-06-09
25	**Site Mob & Set-Up**	**20 days**	**Wed 17-06-07**	**Thu 17-06-29**
26	Clear Land	6 days	Wed 17-06-07	Tue 17-06-13
27	Set Up Site Perimeters And Fencing	4 days	Wed 17-06-14	Sat 17-06-17
28	Conduct Land Survey And Layout	10 days	Mon 17-06-19	Thu 17-06-29
29	Set Up Staging And Storage Areas	5 days	Mon 17-06-19	Fri 17-06-23
30	Mob Equipment To Site Staging Area	4 days	Sat 17-06-24	Wed 17-06-28



31	**Re-Routing Of Stormwater**	**45 days**	**Fri 17-06-30**	**Mon 17-08-21**
32	Excavate Trenches	5 days	Fri 17-06-30	Wed 17-07-05
33	Install New Pipes	5 days	Thu 17-07-06	Tue 17-07-11
34	Place Backfill	3 days	Wed 17-07-12	Fri 17-07-14
35	Tying In New Pipes	3 days	Fri 17-08-18	Mon 17-08-21
36	**Drop Shaft Construction**	**40 days**	**Thu 17-06-29**	**Mon 17-08-14**
37	Preparation Of Pile Driving And Crane Equipment	2 days	Thu 17-06-29	Fri 17-06-30
38	Pile Driving Of Permanent Steel Casing	5 days	Sat 17-07-01	Thu 17-07-06
39	Site Inspection	2 days	Tue 17-07-04	Thu 17-07-06
40	Preparation Drilling Rig Equipment	2 days	Thu 17-07-06	Sat 17-07-08
41	Vertically Drill 4m Wide Shaft	12 days	Sat 17-07-08	Sat 17-07-22
42	Site Inspection	2 days	Sat 17-07-15	Tue 17-07-18
43	Pre-Cast Shaft Segment Unloading	1 day	Tue 17-07-18	Wed 17-07-19
44	Placement Of Baffle Shaft Segments	10 days	Thu 17-08-03	Mon 17-08-14
45	Site Inspection	2 days	Wed 17-08-09	Thu 17-08-10
46	Backfilling With Grout	2 days	Fri 17-08-11	Sat 17-08-12
47	**Horizontal Pipe And Outfall Pipe Construction**	**56 days**	**Thu 17-06-29**	**Fri 17-09-01**
48	Mob And Preparation Of Equipment To Tower Beach	5 days	Thu 17-06-29	Tue 17-07-04
49	Construction Of Temporary Cofferdam	10 days	Wed 17-07-05	Sat 17-07-15
50	Site Inspection	2 days	Tue 17-07-11	Wed 17-07-12
51	Unloading Of Precast Horizontal Conrete Pipe	1 day	Tue 17-07-11	Tue 17-07-11
52	Micro Tunneling Of Horizontal Shaft	15 days	Mon 17-07-17	Wed 17-08-02
53	Site Inspection	2 days	Tue 17-07-25	Thu 17-07-27
54	Tying In To Drop Shaft	3 days	Fri 17-08-18	Mon 17-08-21
55	Installation Of Outfall	5 days	Tue 17-08-22	Sat 17-08-26
56	Corrosion Protection	1 day	Mon 17-08-28	Mon 17-08-28
57	Demob Equipment Back To Site Staging Area	4 days	Tue 17-08-29	Fri 17-09-01
58	**Detention Tank Construction**	**50 days**	**Thu 17-06-29**	**Fri 17-08-25**
59	Install Sheet Piles	2 days	Thu 17-06-29	Fri 17-06-30
60	Excavate Required Areas	11 days	Sat 17-07-01	Thu 17-07-13
61	Site Inspection	2 days	Fri 17-07-14	Sat 17-07-15
62	Drive Foundation Piles	3 days	Mon 17-07-17	Wed 17-07-19



63	Site Inspection	2 days	Thu 17-07-20	Fri 17-07-21
64	Placement Of Foundational Aggregates/Geotextiles	5 days	Sat 17-07-22	Thu 17-07-27
65	Install Formwork And Pour Concrete Slab / Pile Cap	6 days	Fri 17-07-28	Thu 17-08-03
66	Install Formwork And Pour Concrete Walls	6 days	Fri 17-08-04	Thu 17-08-10
67	Install Inlet And Outlet Connections	3 days	Fri 17-08-11	Mon 17-08-14
68	Install Formwork And Pour Concrete Top Slab	3 days	Tue 17-08-15	Thu 17-08-17
69	Remove Sheet Piles And Tie In	3 days	Fri 17-08-18	Mon 17-08-21
70	Site Inspection	2 days	Tue 17-08-22	Wed 17-08-23
71	Backfilling	2 days	Thu 17-08-24	Fri 17-08-25
72	**Re-Storing Site**	**14 days**	**Sat 17-08-26**	**Mon 17-09-11**
73	Roads Curbs & Paving	4 days	Sat 17-08-26	Wed 17-08-30
74	Landscaping	9 days	Thu 17-08-31	Sat 17-09-09
75	Site Inspection	1 day	Mon 17-09-11	Mon 17-09-11
76	**Site Demobilization**	**9 days**	**Mon 17-09-11**	**Wed 17-09-20**
77	Demob Equipment From Site Staging Area	4 days	Mon 17-09-11	Thu 17-09-14
78	Clear Staging Area	3 days	Fri 17-09-15	Mon 17-09-18
79	Remove Fencing & Perimeters	1 day	Tue 17-09-19	Tue 17-09-19
80	Site Inspection	1 day	Wed 17-09-20	Wed 17-09-20



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