West Coast Civil Consultants

March 16, 2015

Martin Bollo, M.Eng, P.Eng
Civil Engineering Department
British Columbia Institute of Technology
3700 Willingdon Avenue
Burnaby BC V5G 3H2

Attention: Martin Bollo

Final Submission for the Design of a Subdivision in Maple Ridge, BC

Attached is our report for the design of a subdivision in Maple Ridge, BC. The design of the subdivision encompasses a wide range of civil engineering disciplines, including but not limited to, geotechnical, municipal, structural, and transportation. Included with the report are engineering drawings, sample calculations, and guidelines used for design.

We, the signatories, hereby mutually declare that the work depicted in this submission is completely original and completed by us independently, as well as mutually.

Sincerely,

Gurkaran Padam   Gurpal Sekhon   Raj Rattan   Thomas Pogorzelec

Enc.

Volume I Report

Volume II Appendix

Volume III Design Drawings

Cc: Civil 7090 Projects Committee

source: http://live.isitesoftware.co.nz/riley/images/Case%20Studies/Kahikatea%20Close.jpg

DESIGN OF A SUBDIVISION IN MAPLE RIDGE, BC

Prepared for: Danica Vulama, P.Eng, Design Engineer
Jayson Vadasz, AScT,
Urban Systems, Vancouver, BC

Martin Bollo, M.Eng, P.Eng, Civil Engineering
Projects Committee, Civil Engineering
School of Construction and the Environment
British Columbia Institute of Technology, Burnaby, BC

Prepared by: Gurpal Sekhon
Gurkaran Padam
Thomas Pogorzelec
Raj Rattan

West Coast Civil Consultants

Report No. CECDP-2015/06
Submitted on: March 16, 2014

Civil Engineering Capstone Design Project

Report No. CECDP- 2015/06

DESIGN OF
A SUBDIVISION IN MAPLE RIDGE, BC

by

West Coast Civil Consultants

Gurkaran Padam
Gurpal Sekhon
Raj Rattan
Thomas Pogorzelec

Department of Civil Engineering

School of Construction and the Environment

British Columbia Institute of Technology

Burnaby, BC, Canada, V5G-3H2

April 2015

DISCLAIMER

The work represented in this Client Report is the result of a student project at the British Columbia Institute of Technology. Any analysis or solution presented in this report must be reviewed by a Professional Engineer before implementation. While the students' performance in the completion of this report may have been reviewed by a faculty advisor, such review and any advice obtained therefrom does not constitute professional certification of the work. This report is made available without any representation as to its use in any particular situation and on the strict understanding that each reader accepts full liability for the application of its contents.

Volume I

Project Report

List of Volumes

Volume I: Project Report

Volume II: Appendices

Volume III: Drawings

ACKNOWLEDGEMENTS

West Coast Civil Consultants would like to acknowledge the following individuals for their help and guidance throughout our project.

Project Sponsor

- Danica Vulama, P.Eng. – Urban Systems, for sponsoring this project and for providing us project guidelines, resources, and technical assistance.
- Jayson Vadasz, AScT – Urban Systems, for sponsoring this project and for addressing design issues and construction considerations along the duration of this project.

BCIT Faculty Members

- Martin Bollo, M.Eng., P.Eng., for providing us guidance and support and for organizing the CIVL 7090 Course in a professional setting.
- Bishnu Pandey, Ph.D., for providing assistance with structural aspects of the project.
- Svetlana Brzev, PhD., P.Eng., for providing us assistance with seismic analysis.
- Paul Thurston, B.A.Sc., P.Eng., for providing guidance in the area of road design.

Industry Members

- Cormac Nolan, P.Eng., for assisting us in the area of pump station design.
- Raul Valverde, EIT, for helping us in the geotechnical aspects of this project.

EXECUTIVE SUMMARY

For this project, West Coast Civil Consultants has designed a subdivision in Maple Ridge, British Columbia. Our sponsors, Danica Vulama, P.Eng, and Jayson Vadasz AScT from Urban Systems, recommended this project as it encompasses several disciplines of civil engineering. The existing site is about 10 hectares in area and resides along 240th Street and McClure Drive in a developing Maple Ridge residential area. Challenges associated with this design included the steep gradients of the site and the protection of the onsite creek. The design of the subdivision's municipal services conforms to guidelines from *The City of Surrey 2004 Engineering Design Criteria Manual*.

Two lot layouts were produced for the initial design and the final layout produced 65 lots and 934 m of roadway. Although the alternate design produced a greater amount of lots, our choice accommodates future growth, accessibility, and a better watermain layout. Lot grading was completed using AutoCAD Civil 3D and conformed to Maple Ridge guidelines. The goals were to minimize earthworks and ensure adequate lot drainage.

The existing sanitary main that runs along 240th Street is at a higher elevation than our proposed subdivision. We designed our sanitary network to direct all sewage flow to a low point in the site to be pumped to the existing main. The partial design of a sanitary pump station was produced to handle peak sewage flows. The pump station has a circular wet well with two pump units in duty/standby configuration. It will direct the pressurized sewage to a force main and into the existing main. The existing main was then checked for flow capacity to handle the proposed pump stations sewage.

A stormwater management plan was implemented to mimic natural/pre-development site conditions for new land developments. The goal is to minimize the adverse hydrological impacts caused by land development by implementing Best Management Practices (BMPs). BMPs are often used to reduce stormwater runoff flow rates and volumes, as well as

reducing erosion, and providing settlement and contaminant control. Two structural BMPs were implemented for the subdivision: a detention facility and an erosion and settlement control plan.

As part of the stormwater management plan, the storm sewer will collected runoff and route it to a detention facility where it will be temporarily stored and released in a controlled manner back into the nearby creek. It was of high priority to protect the Kanaka Creek to the north of the site. To reduce the post-development flow to that of the pre-development flow, a detention pond was used to contain the stormwater runoff in order to slowly release it into the creek, and allow the contaminants to settle. In the stormwater pipe network, which facilitates the movement of water to the pond, lawn basins are used to prevent ponding in individual lots.

To provide clean drinking water to the residents, a water distribution network is implemented that attaches to the existing main. A 200 mm diameter watermain was sufficient to handle fire flow requirements. Hazen-William's and Bernoulli's equations were used to check key locations for adequate pressure. The existing main carries water at a 200 psi pressure, but needs to be reduced to 130 psi for residential use. To achieve the lower pressure, a system of parallel pressure reducing valves (PRV's) was designed. The PRV system includes a large PRV for peak water flows and a smaller PRV for average daily flows.

The design of the roadways provides residents convenient access to the subdivision. It is defined as a "Through Local" as classified in the City of Surrey Design Criteria Manual. Two intersections were designed to unify the grades of the crossing roadways.

The geotechnical considerations for this project involved a preliminary geotechnical report, retaining wall design, and a slope stability analysis. The preliminary geotechnical report provides site soil conditions and parameters based upon soil descriptions and published surficial geological data.

The retaining wall was designed for the 3.5 m elevation change in between lots within the subdivision. The L-shaped concrete cantilever wall was designed to resist overturning, sliding, and bearing capacity failure. Structural design of the wall included calculating shear resistance and flexural resistance of the wall's members using CSA A23.3 and ACI Code requirements. A simplified static seismic analysis was included in the design to account for seismic loading. Also, a SAP 2000 model of the wall was produced to verify results. The back drainage system of the wall uses clean, granular material as backfill, a drainage membrane, and weep holes.

Due to concerns of slope failure in the Kanaka Creek area, a slope stability analysis was performed in GeoStudio 2012. The GeoStudio model was a 31% gradient slope with a pond load on the top and a best-case and worst-case groundwater level. The soil properties of the slope were assumed as silty loam. The results of the analysis were minimal seepage from the pond and the factor of safety against failure was sufficient.

Using RS Means and estimates from local contractors, a construction cost estimate was produced for the proposed subdivision. The approximate cost will be $4.12 million, which does not include the cost of developing the structures on individual lots.

During the construction phase of this project, there will be many opportunities for the sediments that come off the site to pose harm to Kanaka Creek. Therefore, an erosion and sediment control plan is to be implemented. The erosion and sediment control (ESC) plan is divided into three stages. By installing sediment ponds during the initial stage, contaminated water is able to settle before being discharged into the creek. Other ESC measures are silt filters, silt fences, and straw wattles, which trap fine particles as water flows freely through them.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

West Coast Civil Consultants has completed the design of a subdivision located in Maple Ridge, BC. Our industry contacts for this project are Danica Vulama, P.Eng., and Jayson Vadasz, AScT, from Urban Systems. The design of this project has previously been completed by Urban Systems. For the purpose of the Capstone Design Project, we have proposed an alternate design.

The subdivision site is located at the southeast corner of 240th Street and McClure Drive. It is approximately 16 hectares in area and includes both single-family residential and future townhouse development. For this project, we have concentrated on the single-family residential development of the site, which is approximately 10 hectares of the total site area. The major design constraints of this project included the on-site creek and the existing site topography. The on-site creek posed areas of concern during the design and construction phases of this project because of the creek's environmental sensitivity. Existing topographic conditions of the site range from flat areas to slopes up to 33% grade. These design constraints were integral aspects in the design of this subdivision.

The design of the subdivision encompassed the following areas of work:

- lot layout design
- lot grading
- municipal services design
- road design
- geotechnical analysis
- structural design
- environmental protection
- storm water management
- construction estimating

One of the main reasons that we took on this project for our CIVL 7090 course is because of the wide range of civil engineering disciplines that it covers.

2.0 BACKGROUND

The subdivision was originally designed by Urban Systems in 2008, and construction was completed in 2012 for Area51 Developments Ltd. The developer saw an opportunity to develop in the increasingly accessible Maple Ridge area, as a result of the newly constructed Golden Ears Bridge. Maple Ridge is regarded as one of the fastest growing cities in British Columbia, which made this subdivision a great opportunity for the developer to take on.

The designed development is approximately 10 hectares in area, with a creek starting in the northeast region and running west towards the northwest portion of the site. The existing road, 240th Street, runs north-south on the west side of the designed development. Figure 1 below shows the location of the site.

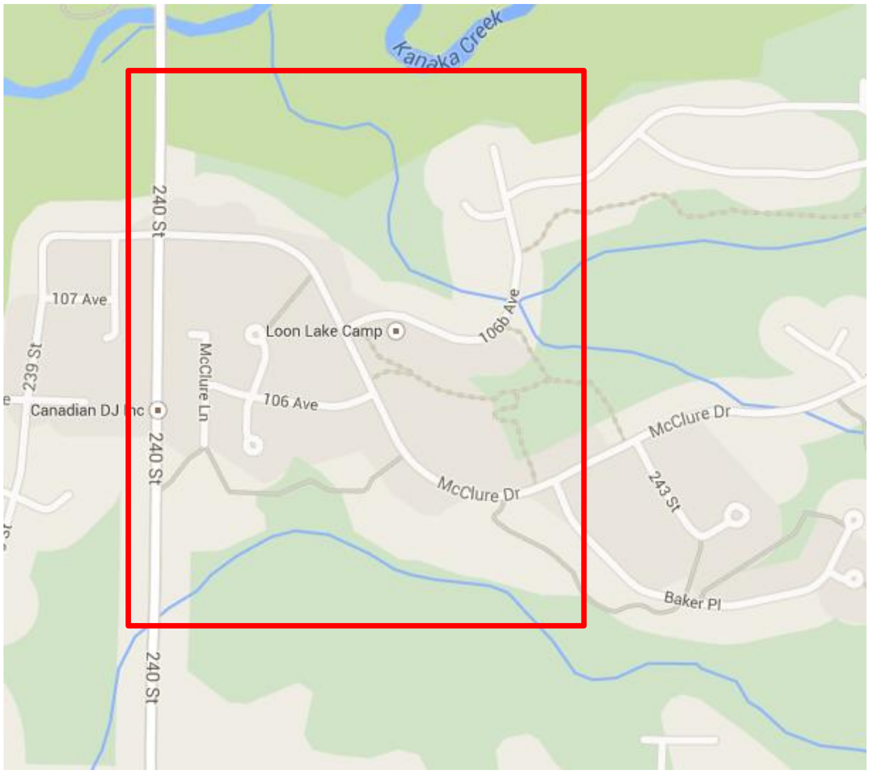


Figure 1: Site Location (Google Maps, 2015)

The existing road, 240th Street, is the primary access point for the subdivision and contains existing sanitary and water mains that have been used for tie-in points for the municipal services design.

3.0 LOT LAYOUT DESIGN

The first phase of this design project was to produce a lot layout. This phase lays out the framework that the remainder of the design work will be based upon. The lot layout design was an iterative process that started with preliminary hand sketches that received several revisions. Once the hand sketches were close to the desired layout, AutoCAD Civil 3D was used to draft two separate layouts: Option 1 and Option 2.

While designing these layouts, we considered several design constraints, including

- location of the onsite creek
- shape of the site boundary
- location of a future townhouse development
- possible future road extension
- steep terrain of the site

These constraints proved to be a great challenge in the lot layout procedure and influenced the other components of the project.

3.1 Layout Criteria and Constraints

The lot layouts were designed according to specific design criteria shown below as provided by our sponsor:

Table 1 : Lot Layout Criteria

Minimum Lot Width	15 m
Minimum Lot Depth	28 m
Minimum Building Envelope	13 m x 9 m
Front yard Setback	7.5 m

Rear yard Setback	7.5 m
Side yard Setback	1.8 m
Side yard on Flanking Street	3.6 m
Minimum Lot Area	480 m^2

The criteria listed above ensure the final designed layout meets local municipal regulations and requirements. In addition to meeting regulations, they ensure all lots are uniform throughout the subdivision.

3.2 Results

There were several constraints that governed the design of the lot layouts including the steep slopes, and roadway locations. After much analysis, two layouts were designed: Option 1 and Option 2.

3.2.1 Lot Layout - Option 1

In the first layout, Option 1, we followed a similar direction to what was implemented by Urban Systems. The design consists of a curved road going from the northwest region into the southeast. From the roadway, two cul-de-sacs protrude towards the north side of the roadway, and lots are located adjacent to the roadways as shown in the figure below.

Figure 2 : Lot Layout – Option 1

This layout has a total of 75 lots and 714 meters of proposed roadway over 11.1 hectares. The lot density is 6.76 lots/ha.

3.2.2 Lot Layout - Option 2

For the next layout, Option 2, the design consists of three roads, Road A, B, and C. Road A and Road B start from the west portion of the site and run towards the east side of the site. The east side of Road B ties into the proposed future development to the east of the site. This tie in point was one of the design constraints of this project. Lots are located adjacent to the roadways as shown in the figure below.

Figure 3 : Lot Layout – Option 2

This layout has a total of 65 lots and 934 m of proposed roadway over 11.6 hectares. The lot density is 5.60 lots/ha

Option 2 has two access points from 240th street (Roads A and B). Having two access points into the site has multiple benefits. Two of the benefits include the following:

- Greater accessibility for residents and emergency response vehicles.

- Safer design of the water main in the site due to a loop in the distribution system.

In addition to these benefits, Option 2 offers a better fit to the natural contours of the site. Road B runs along the high point of the site and acts as an internal topographic boundary within the site. Our design is located on the north side of Road B and has an overall site slope down towards the north. This simplified the design of the municipal utilities over the entire site because gravity sewers on the north side of the site are completely separate from potential future development on the south side of Road B, which slopes down to the south. Option 2 also increases the ease of future development to the south of Road B as Road B can be used as the primary access point for this future development.

3.3 Lot Layout Recommendations

After comparing the two layouts based on satisfying the design criteria and the effect of the lot layout on other design work necessary for the project, Option 2 was chosen as the final lot layout.

4.0 LOT GRADING

The purpose of lot grading is to smooth out the landscape to make it habitable for those who primarily use the landscape. It also considers that water is not retained on the properties but instead drains away from the lot and into the roadway, where it is channeled into the storm sewer.

A lot grading plan was created using AutoCAD Civil 3D. Our sponsor provided a topographic survey of the existing site, which was used as the base point for our grading design. The objective of the lot grading design was to limit steep slopes and to ensure adequate overland drainage of the lots.

The general procedure we used to design the proposed surface in AutoCAD Civil 3D was to first study the natural contours of the site, and then generate a proposed surface which complimented all other aspects of the subdivision design, including the drainage design and the vertical road layout. For the integration of the drainage design with the grading design, we ensured that no overland drainage of a lot crossed over a neighbouring property. This was done by using lawn basins at the low corners of each lot.

4.1 Lot Grading Criteria

While completing the grading design, we used the following criteria specified by our sponsor that conformed to Maple Ridge guidelines.

Table 2 : Lot Grading Criteria

Maximum Lot Gradient (Front to Back)	25%
Maximum Lot Gradient (Side to Side)	5%
Maximum Driveway Slopes	8%

The criteria listed in the table above ensures a safe and aesthetically appealing final surface for the subdivision.

4.2 Lot Grading Results and Recommendations

To meet the grading requirements in Table 2, we decided to use several earth retaining structures. The main earth retaining structure is in the center of the site. This structure is a retaining wall which varies in height ranging from 3.5 m to 1 m. The design of this structure is discussed later in the report (11.2 Retaining Wall Design). Other retaining structures include an array of 1 m high walls that act as grade breaks along the east side of the site boundary. These smaller walls were not structurally designed as they are considered to be landscaping features.

5.0 SANITARY SYSTEM

One of the major concerns in subdivision design is getting water into each of the detached dwellings. However, only a small portion of that pristine drinking water is actually consumed. A majority is used for washing purposes and is required to be removed and then treated away from the destination. Like the storm sewer, it is desired that gravity carry away this flow and it will eventually make its way into the proper treatment facility. Due to the unique topographic features of this subdivision, a sanitary pump station needed to be installed on the site. The design of the sanitary network is explained in this section.

5.1 Sanitary System Background

The use of water in households does not stay at a constant rate. In general, the early morning puts a demand on the water and sewage system, as people are usually waking up, taking a shower, then making breakfast before they head out for the day. Then throughout the day, there is a period where there is minimal water being used. As the population makes its way back home in the late evening, there is another stress point on the sewage system, as dinner is to be made, laundry is to be done, and maybe even more showers are taken. Also, let's not forget about the use of the toilet, as everyone in the household uses it. As we can see, engineers must account for the fluctuations in sewage rates that will occur in the system.

Another extreme example is from the City of Edmonton, during the 2010 Olympic Gold Medal hockey game in which Martin Bollo and team Canada faced off against the United States. Shown in the figure below is the water demand during specific periods of the game.

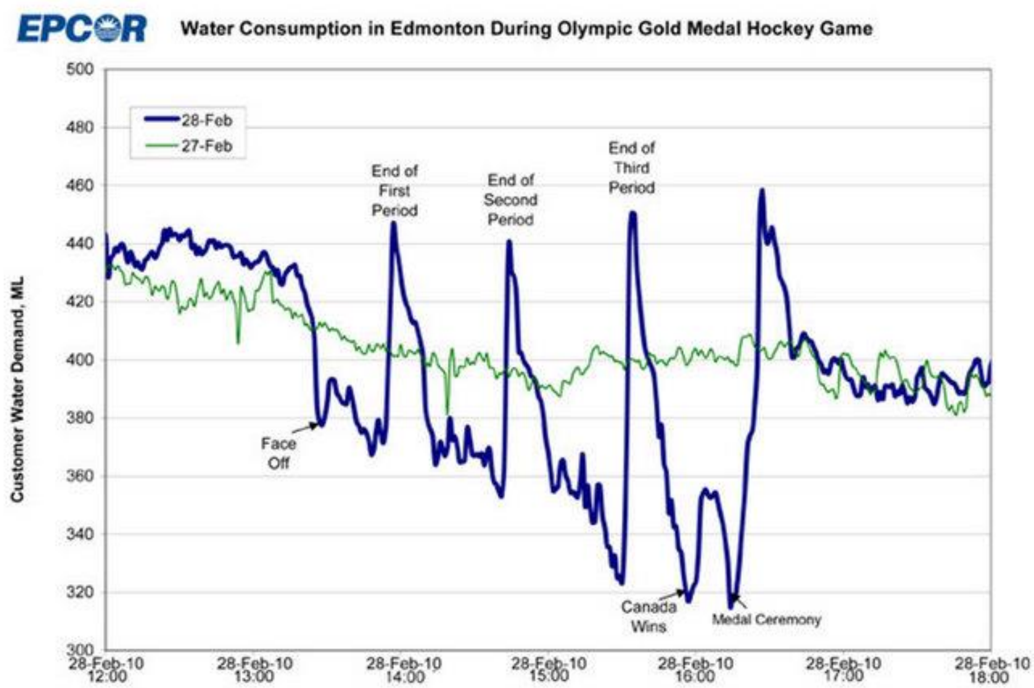


Figure 4 : City of Edmonton Gold Medal Game Water Consumption (Epcor, 2010)

It can safely be assumed that a majority of the water usage came as a result of people relieving themselves when there was a stoppage in play. With that being said, when the toilet is flushed, all of the water that gets used ends up in the sanitary system. To account for this, the City of Surrey implements a “peaking factor” that ultimately increases the flow rate in the pipes, to ensure there is no surcharging.

5.2 Pipe Design

Sanitary sewer pipes are designed to handle the flows created by households using water as well as natural infiltration which may occur from the groundwater table. The City of Surrey Design Criteria Manual was used to ensure the design met the size requirements. For the flow requirements, the average day dry weather flow per person ("ADWF") is multiplied by the number of people that the pipe will service. However, for the design flow, the value is multiplied by a "peaking factor", calculated as

$$PF = 1 + \frac{14}{4 + \sqrt{\frac{Population}{1000}}}$$

The other component to the sanitary flow is infiltration. This is the seepage of groundwater in to the sanitary pipe network through manholes or the pipe itself. Infiltration must be accounted for because it can significantly affect the flow volumes. There are several factors that affect infiltration, including

- ground water level
- soil porosity
- pipe diameter
- workmanship
- pipe material

After determining the parameters in the sanitary system, a design flow can be calculated as

$$QQ_{design} = ADWF \ \times PF + Infiltration\ Inflow$$

Where: ADWF = Average Daily Consumption per person/day = 350L/Person/day

Infiltration Inflow = 11 200 L/Ha/day

PF = Peaking Factor

Finally, using Manning's equation, we can solve for the diameter of the pipe required to carry that flow. Manning's equation incorporates the flow, pipe roughness, diameter, and slope. However, to ensure that large objects or obstructions do not prevent flow from entering a pipe, a minimum diameter of 200 mm is required. Calculations begin at the highest elevation, towards the lowest point. Manning's equation is also used to size pipes in the storm network, which can be found in Section 6.0 of this report. The maximum allowable flow in the sanitary pipes is one-half of the pipe's full capacity.

To ensure that the system is able to cleanse itself of household waste and other intrusions that make their way into the network, a minimum velocity of 0.6 m/s is required. A maximum allowable velocity of 5 m/s is recommended, according to City of Surrey's guidelines.

For the calculations associated with the velocity of the pipe, we used Microsoft Excel's Solver function to obtain the area of water flow, which incorporates the circular geometry of the pipe. The methodology used here was to solve for area of flow based on the design flow in the pipe. If the velocity was below the minimum required, we increased the slope of the pipe.

The existing sanitary main was checked to determine if it has the capacity to service the additional sewage flow from our subdivision. The existing main was checked by using the same method explained previously for the sanitary pipes in our subdivision. The 300 mm existing main was assumed to run at a slope of 3% with an upstream serviced population of 2000 people (approx. 53 hectares of developed area). The peak design flow of the existing serviced population was then calculated and added to the peak design flow of 25 L/s from the pump station. The total design flow of the pipe was then checked for pipe flow capacity and flow velocity requirements.

5.3 Sanitary System Results and Recommendations

After conducting an analysis of the pipe network through the use of a spreadsheet the entire sanitary network is to consist of 200 mm diameter PVC SDR-35 pipe. The sanitary flow for a subdivision can be quite small, even during peak events, hence the usage of the minimum pipe size throughout the network. All flows are directed towards the north-central area of the area to be developed so it can be pumped out via the sanitary pump station.

The existing sanitary main is large enough to include sewage flow coming from the proposed subdivision pump station. The existing serviced population's peak design flow is 35.87 L/s and the pump station's peak design flow leading to the existing main is 25 L/s. Therefore, the new peak design flow of the existing main will be 60.87 L/s, which is smaller than 50% of the pipe capacity (83.3 L/s). Finally, the velocity of the sewage flow was calculated as 2.18 L/s, within the recommended range.

5.4 Sanitary Pump Station

To remove the residential sewage from the proposed subdivision site, a pump station is required. This pump station will service the entire site and pump the sewage to the existing sanitary main on 240th Street.

The pump station design was limited to the following aspects:

- Pump unit selection
- Wet well design
- Force main design

The pump station design was performed according to *City of Surrey 2004 Design Criteria Manual* guidelines and *City of Richmond Sanitary Pump Stations Design Criteria (See Volume II Appendix P and Appendix Q).* The rest of this section will detail the above aspects of the pump station design.

5.4.1 Pump Unit Selection

The pump unit selection was performed by developing a system head-flow (H-Q) curve and selecting a pump with the best efficiency along a duty point on the curve. This system curve is developed by adding the static head and dynamic head (due to head losses) which must be overcome to get the sewage from the wet well to the existing sanitary pipe. Two curves were developed, a worst-case total head and best-case total head. The two curves are shown plotted below.

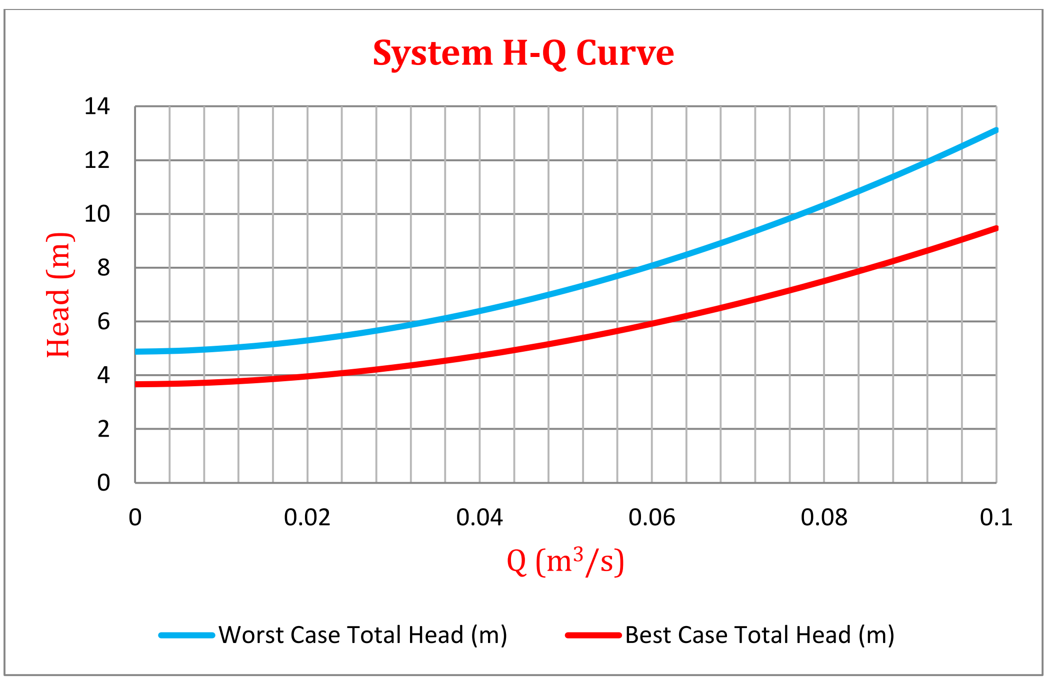


Figure 5 : System H-Q Curve

To determine the static head levels (y-axis intercept), initial wet well dimensions were assumed to determine the low water level and high water level in the pump station. The low water level is the worst possible case with no incoming sewage into the station. The high water level is the best case scenario with maximum incoming flow into the pump station for a period of 10 minutes (pump duty cycle time). The difference between these water level elevations and the elevation of the existing sanitary main gives the two possible static heads. In reality, the static head would be in between these two values.

The total head is the static head in addition to the dynamic head, which increases with flow. The dynamic head is due to the pipe energy losses in the force main. The Hazen-Williams equation was rearranged to solve for the head loss at various flow rates.

$$h_L = L(Q/0.28/C/D^{2.63})^{\frac{1}{0.54}}$$

Where:	h_L	=	head loss, m
	L	=	length of pipe, m
	Q	=	design flow, m^3/s
	C	=	roughness coefficient
	D	=	pipe diameter, m

The roughness coefficients used were 120 (worst-case total head) and 145 (best-case total head). The length of pipe was assumed as a straight line distance from the pump station to the existing main. Also, a diameter of 200 mm was assumed as the force main diameter.

Local pipe losses within the pump station fittings and valves were also accounted for but not included in the graph. The following equation was used to determine local losses.

$$hl_l = \sum \zeta \frac{v^2}{2g}$$

Where:

hL_L = local loss

ζ = local resistance factor

v = flow velocity

g = acceleration of gravity

The total local loss due to all bends, valves, and transitions in the pump station was calculated by summing all the local resistance factors for each (See Volume II Appendix O for ζ values). A flow velocity was assumed using a 100 mm diameter pipe leading from the pump to the 200 mm force main and a 25 L/s flow.

Once the system H-Q curve was generated, a pump unit closely matching the system requirements was to be selected. To accomplish this, a web-based product database, *Xylect*, was used to select a *Flygt* brand pump unit. This brand is well-known and one of the City of Surrey's approved pump manufacturers. A screenshot of the program's product search options is shown below.

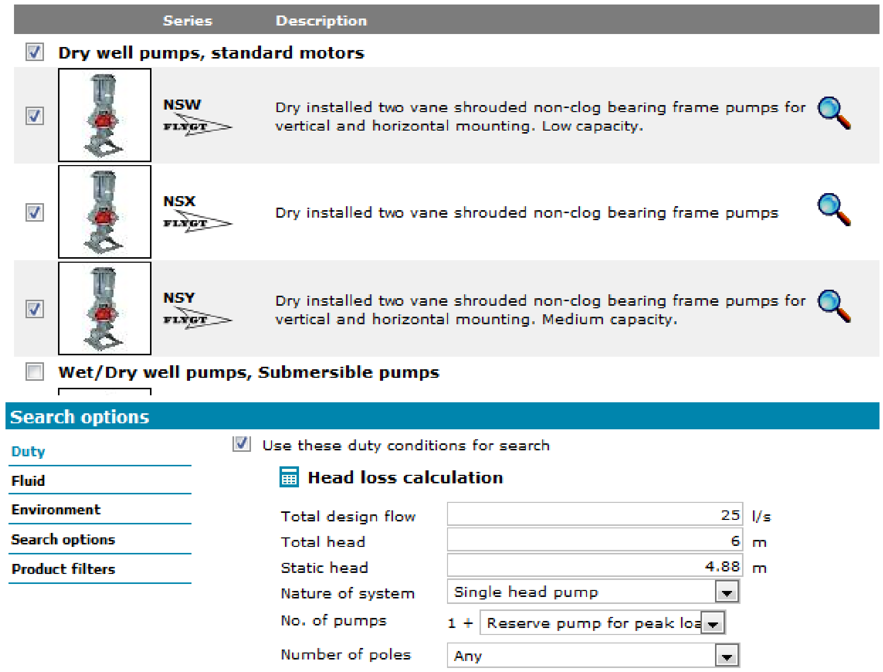


Figure 6 : *Xylect* Product Search

The duty conditions of the required pump were then input to the search options. The design flow was assumed to be 25 L/s, as it seemed like a reasonable value based upon the pump station inflows and the initial pump cycle time. Also, according to *City of Richmond Sanitary Pump Stations Design Criteria,* the inlet pipe flow capacity at 50% depth must equal the maximum pump rating for the station. From our sanitary network pipe calculations, the inlet pipe flow's 50% capacity is 30 L/s, which is close to the design flow we assumed for the pump.

The next step was to determine the total head and static head from the system curve. The worst-case curve was used, as it represents the worst possible pumping conditions. The static head from this curve is 4.88 m and the total head at 25 L/s is approximately 5.5 m. The calculated local loss of 0.5 m was added as well, to get a total head of 6 m.

The product search resulted in a number of pump products which would suit the system curve at various efficiencies. The highest efficiency pump was then selected and analyzed, a Flygt NP 3102 MT 3~ 465 (See Volume II Appendix N, Pump Specifications).

The Xylect program provided a duty analysis of the selected pump unit. The duty analysis matched the pump curve to the system curve and showed the pump efficiency and net positive suction head (NPSH) requirements. Figure 7 below is a screenshot of the pump curve intersecting the system curve.

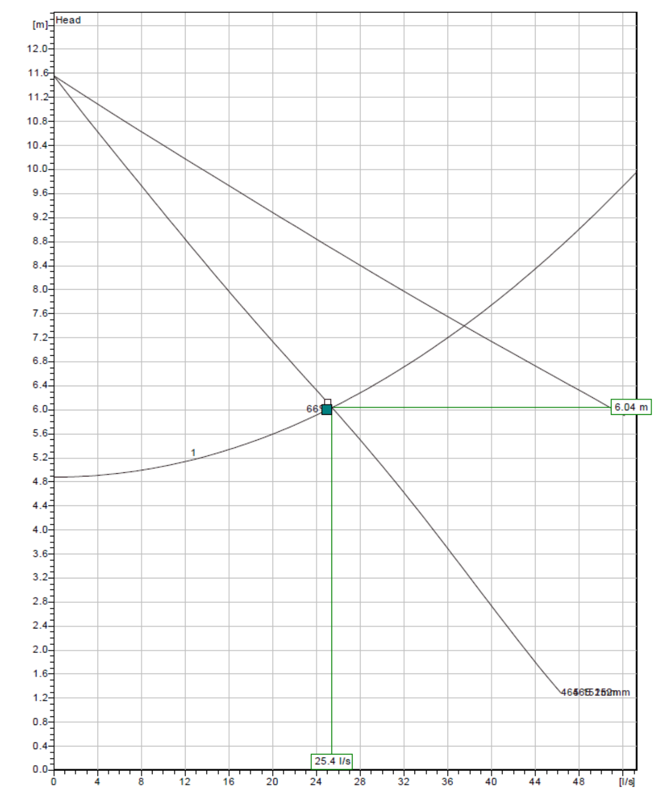


Figure 7 : Duty Analysis of NP 3102 MT 3~ 465 Pump Unit

The graph shows that the pump curve intersects the system curve at an efficiency of 66%. Also, it shows a pump curve for two pumps running at the same time, which may be required in the event of extreme flows to the pumping station.

The available NPSH in the pump station was checked to ensure it was greater than the NPSH requirements. The following equation was used to determine the NPSH available.

$$Avaliable\ NPSH = Ha + Hz - Hf - Hvp$$

Where:

Ha = atmospheric pressure (vented tank) (m)

Hz = vertical distance between surface of liquid and pump centerline (m)

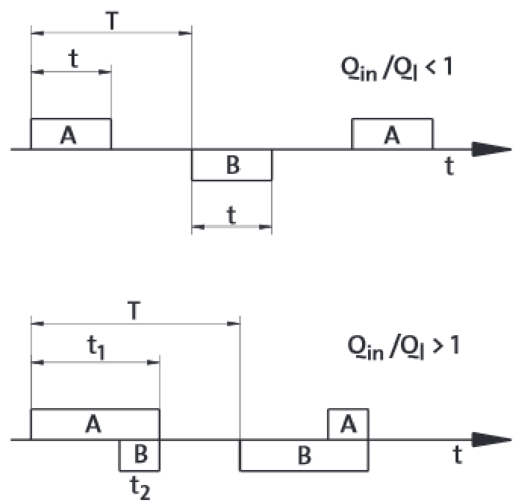
Hf = friction losses in suction piping (m)

Hvp = absolute vapor pressure of liquid at pumping temperature (m)

The NPSH available (10.6 m) was found to be greater than the NPSH required of 2.87 m, applying a safety margin of 3 m. This check ensures that cavitation will not occur in the pumping unit.

The final step of the pump unit design was to determine the duty and standby time of the pumping units. To limit fatigue of the pumping units, our pumping station will feature two identical pumps in alternating duty. The pumps will alternate position as duty (lead) and standby (lag) with each running cycle. The duty pump will handle most regular incoming flows, and the standby pump will start if the incoming flow rate Q is larger than the capacity of one pump. The following diagram shows the running cycles for both normal flow and high flow conditions.

Fig. B4



Operation time diagram of the duty and standby pumps in a pumping station for an incoming flow (Q_{in}) both smaller and larger than the capacity of one pump (Q_I).

Figure 8 : Operation Time Diagram *(Grundfos)*

The period T of the running cycles was chosen as 10 minutes. The duty time for the pump is 6 minutes running at 25 L/s. As shown in the diagram, pump A turns on and operates again 14 minutes after its last operation. If incoming flows are too high, one of the pumps will turn on and provide assistance to the duty pump during that running cycle.

5.4.2 Wet Well Design

The wet well houses the two pumping units, force main inlet piping, and the incoming sewage flow. The main types of wet wells are trench types and circular types. Trench types are appealing because of their good hydraulic environment for pump intakes, ease of cleaning, and minimum footprint size for wastewater. However, these types of wet wells are typically for higher design flows, exceeding 130 L/s, which is not an economical design for our pumping station. Instead, we chose to design a circular type wet well due to its low cost and suitability for small lift stations.

The first step in the wet well design was to determine the size of the wet well. The size of the wet well should be large enough to house the pumping units and provide adequate space for repairs. A diameter of 2 m seemed reasonable and was larger than the City of Surrey required minimum diameter of 750 mm. The wet well depth is approximately 3 m, to provide adequate storage to handle peak flows or an emergency (See 5.4.4 Pump Station Recommendations section). For practicality, the depth of the pumping station should be as shallow as possible (recommended max. depth of 4.5 m).

Another important parameter in the wet well design is the inlet pipe location. The inlet pipe should be located as close to the pump centerline as possible. This is to avoid freefall from the inlet conduit into the wet well, which releases odours and entrained air bubbles in the sewage. Air bubbles can cause loss of capacity and damage to the pumping units. Also, abrupt changes in flow direction upstream from the pump inlet can cause vortices. These abrupt changes can cause flow to become asymmetrical and overload pump shafts and bearings. The below figure depicts the ideal location of the approach pipe to reduce vortices and free fall action.

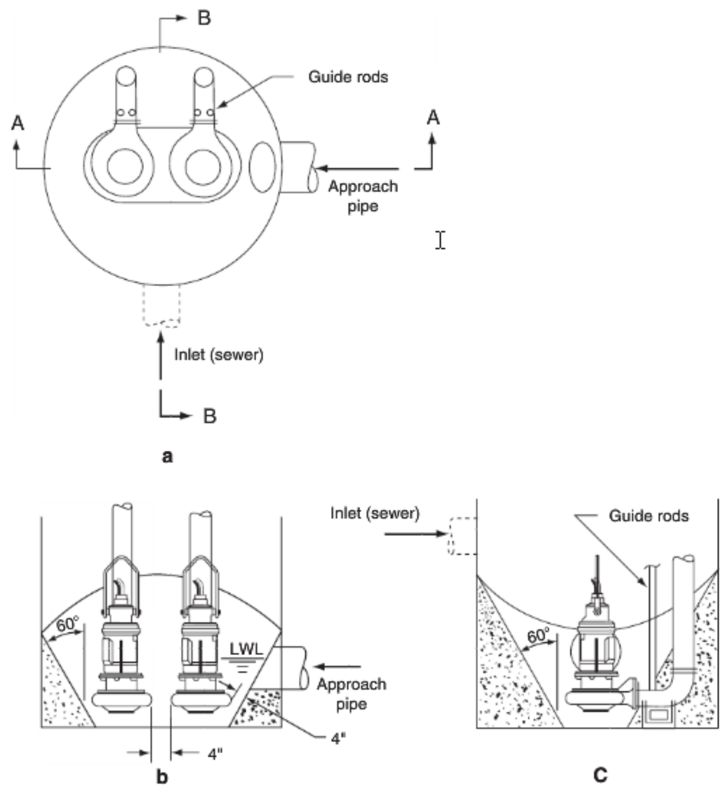


Figure 12-31. Preferred geometry for duplex submersible pumps in a small, round wet well is shown by solid lines. Unacceptable sewer inlet is shown by dashed lines. (a) Plan; (b) Section A-A; (c) Section B-B.

Figure 9 : Wet Well Inlet Location *(Jones, 2008)*

Figure 9 shows the unacceptable sewer inlet location in dashed lines. The figure also shows fillets at the bottom of the wet well, which are important for wet well sludge cleaning.

The bottom of the wet well was designed with fillets to have a minimum surface area possible at the top of the pump volute. This is to allow sludge to slide down the sides of the wet well and be drawn in to the pump through strong suction currents. The sludge is drawn into the large vortex formed beside the pump and the small surface area at the pump volute will allow it engulf the solids quickly, before it loses prime. Also, a valve can be installed in the pump to bypass some of the discharge back into the pool for a short interval of the pump cycle. This will mix the contents of the wet well and prevent accumulation of sludge. The surface area of the bottom of our pumping station is shown below with varying radii of the bottom fillets to accommodate the pump unit locations.

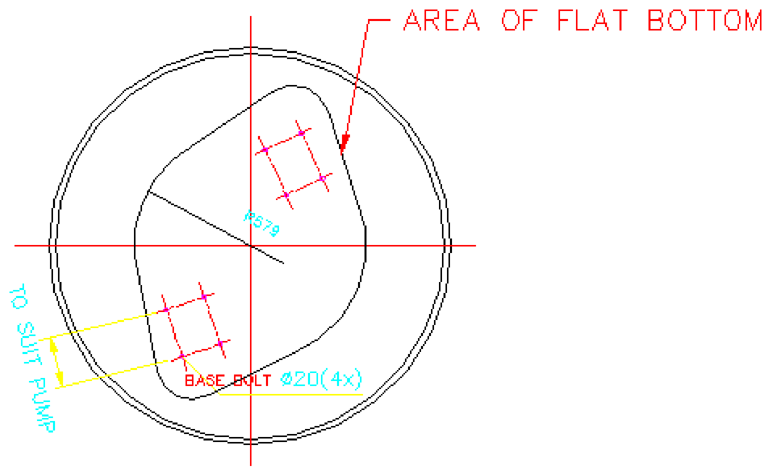


Figure 10 : Bottom of Pump Station

5.4.3 Force Main Design

The design of the force main involved determining the size of the force main and its connection to the pump outlet piping. The pump discharge diameter is 100 mm. Since the head loss in the force main would be too large with this diameter, a larger force main was required. Based on our judgement, a 200 mm diameter force main was the appropriate size to have minimal head loss in the pipe while maintaining an adequate design velocity. The recommended maximum velocity in smaller pump station force mains is 1.8 m/s to reduce the severity of water hammer upon pump activation. The practical minimum is 0.6 m/s, while a peak daily velocity of 1.1 m/s is desirable to re-suspend settled particles (Pumping Station Design, Jones, G.). With a 200 mm diameter force main and 25 L/s design flow, the force main design velocity is approximately 0.8 m/s.

An economic analysis could have been conducted to reduce the size of the force main diameter. However, we realized the larger cost comes with the pump unit and the power requirements to run a larger pump unit with increased total head. Also, a transient analysis was not performed to determine if transient conditions will affect the pump station or force main design.

5.4.4 Pump Station Results and Recommendations

The final design of the pumping station can be seen in Volume III: Drawing 40. The pump station will house two Flygt NP 3102 MT 3~ 465 pump units in a custom fiberglass wet well. The two pumps will operate in a duty-standby configuration to prevent excess wear and fatigue on the pumping units. The wet well is 2 m in diameter and approximately 3 m deep. It has filleted bottom to prevent excess sludge accumulation. The force main diameter is 200 mm, to maximize cost-savings due to material costs and pump efficiency. An in-depth analysis of force main cost savings is recommended. Also, a transient analysis of force main flow conditions could be performed to enhance the pump station design.

The City of Surrey guidelines specify an emergency storage volume corresponding to 6 hours of average daily water flow. The volume required would be 28 m^3, higher than our wet well storage volume of approximately 10 m^3. We recommend an emergency storage tank housing an emergency pump to meet the City of Surrey's requirements. This would prevent excess sewage accumulation during a power outage or extreme natural event.

Recommendations for a complete pump station design include a description of

- Station instrumentation and control logic
- Wet-well ventilation design
- Seismic design and geotechnical considerations

The station instrumentation and control logic involves determining the required equipment and electrical input required to run the pumping station. This is shown graphically through a piping and instrumentation diagram (P&ID) flow chart.

Proper ventilation is required to prevent the accumulation of noxious gases in the pump station. The ventilation could be provided through a fixed speed fan that runs continuously. The minimum ventilation rate should be at least 12 air changes or higher and ventilated air exhausted through hatch lids and in to the gravity sewer during pump operation (City of Richmond Pump Stations Design Criteria).

A geotechnical investigation is required for the pump station to determine construction issues, settlement potential, and liquefaction potential during a seismic event. To provide adequate protection from these issues, the wet well and electrical kiosks foundations should be designed to prevent overturning. Also, force main and gravity sewer connections should be design to accommodate a 300 mm of vertical ground displacement.

6.0 STORMWATER MANAGEMENT

Stormwater management involves the planning and design necessary to mimic natural/pre-development site conditions for new land developments. New developments can cause increased runoff flow rates and volumes that are caused by the addition of impervious surfaces and related ground cover change. The goal is to minimize the adverse hydrological impacts caused by land development by implementing Best Management Practices (BMPs). BMPs are often used to reduce stormwater runoff flow rates and volumes, as well as reducing erosion, and providing settlement and contaminant control.

To accomplish the aforementioned goals, the following structural BMPs were implemented for the subdivision:

- Detention facility
- Erosion and settlement control plan

The detention facility will be a permanent site feature that will hold runoff for a period of time, allowing for settling of the solid pollutants. The erosion and settlement control plan is a construction BMP to help limit erosion and sedimentation during construction activities. Further discussion into the design of the detention facility and the erosion and settlement control plan are described in sections 8.0 Detention Facility and 12.0 Environmental Considerations. Figure 11 below depicts some of the bank erosion that can occur in a creek if storm water management is not properly implemented.

Figure 11 : Stream Bank Erosion (Arlington, 2015)

Implementation of bioswales as another BMPs measure was considered, however, due to the steep grading of the lots and roadways, bioswales would be an ineffective measure of reducing stormwater runoff. Bioswales require onsite pooling to store runoff and this was not possible with slopes as steep as 9% in most lots.

7.0 STORM SEWER

Storm sewers are underground pipes used to transport stormwater from developed areas safely and conveniently into natural bodies of water, such as streams, lakes and oceans. As part of the stormwater management plan, runoff that is collected onsite will be routed to a detention facility where it will be temporarily stored and released in a controlled manner back into the nearby creek. The following is a methodology of the design for the storm sewer system for the subdivision.

7.1 Rational Method

In accordance with the City of Surrey Guidelines Section 5.3.C Rational Method, the Rational Method can be used for the design of a minor storm sewer system. This method is acceptable for catchment areas that are less than 20 Ha; the total catchment area for the subdivision was approximately 9 Ha. The Rational Method is described as follows:

$$Q_p = \frac{RIA}{360}$$

Where:	Q_p	=	peak flow, m^3/s
	R	=	runoff coefficient
	A	=	catchment area, Ha
	I	=	rainfall intensity, mm/hr

The resulting peak flow was used in conjunction with Manning's equation to size the storm sewer components for the subdivision, which is further described in Section 7.6 Sizing of the Storm Sewer Pipes. Determining the values that were used in the Rational Method are explained in the sub-sequential sections.

7.2 Sub-catchment Areas

Sub-catchment areas are hydrologic drainage subareas whose topography and drainage system elements direct surface runoff to a single discharge point. To develop sub-catchment areas for the storm sewer, a preliminary storm sewer layout had to be defined. This was determined by reviewing the lot grading topography of the subdivision, and then mapping by hand the best location for the system to route stormwater to a designated discharge point.

7.2.1 Storm Sewer Layout Design & Alternatives

Volume II Appendix H shows the two design options that were created for the storm sewer network; the preliminary storm sewer system is marked in red with accompanying flow directions. The most notable difference between the two options is that there are two proposed detention facilities for Option 2, and only one detention facility for Option 1. As well, there are fewer easements and storm sewer pipes required for Option 2.

For the storm sewer layout, Option 1 was selected for the subdivision. This design only required one detention facility for the subdivision, which would help reduce cost since detention facilities are expensive to design, construct, and maintain. According to the Greater Vancouver Sewage & Drainage District (GVS&DD), an extended detention pond can cost up to $151,000 to design and construct, along with an additional $1,100 annual maintenance cost (Gibb, et al., 1999). An estimate material cost of $11,300 was determined for the extra cost of pipe that would be necessary for Option 1, which is significantly cheaper compared to the cost of an additional detention facility.

However, Option 2 would have been selected if a single detention pond was incapable of detaining stormwater from the entire subdivision. If this detention facility was at max capacity, a second detention pond would be required in order to meet the volume capacity requirements.

In order to determine if a second detention pond was necessary, a brief runoff analysis of the catchment area contributing to the runoff being routed to the second detention facility was performed. Runoff analysis showed that the volume of generated from the contributing catchment area was fairly small and it was concluded that one detention facility was capable of handling all of the runoff generated from the subdivision.

7.2.2 Catch Basins & Lawn Basins

As per the City of Surrey Guidelines Section 5.4.F Catch Basin Spacing, the catch basins should be spaced at regular intervals along the roadway, while taking into account

- inlet capacity
- pipe sizing
- street slope
- low points on roadway
- curb returns at intersections.

The capacity of a single catch basin was calculated by the orifice equation:

$$Q = 0.5CA\sqrt{2gh}$$

Where:	Q	=	inlet capacity, m^3/s
	0.5	=	clogging factor
	C	=	orifice coefficient (0.8)

A	=	open area (0.068 m^2 for Dobney B-23 grate)
g	=	gravitational acceleration (9.81 m/s^2)
h	=	depth of ponding, m

The orifice equation was used in combination with the Rational Method to determine the capacity and spacing of the catch basins. A time of concentration 5 minutes was assumed for calculations. Additionally, as per the City of Surrey Guidelines Section 5.4.F Catch Basin Spacing, a maximum spacing permitted for each catch basin was as follows:

- 350 m^2 on road grades greater than 3%
- 500 m^2 on road grades up to 3%.

Sample calculations of the capacity and spacing of the catch basins are shown in Volume II Appendix F.

Double catch basins were used at any low points on the roadway to ensure a 100% capture of any remaining stormwater flow in the gutter. This will also help in the prevention of flooding at these low points during higher intensity storm events.

Additionally, locations for lawn basins were established for all of the lots and were placed at the lowest elevation of each lot. Any lawn basins located near retaining walls were offset about 4 m away from the wall to allow sufficient clearance.

Lastly, leads running from the catch basins and lawn basins to the storm sewer network were defined. All catch basin and lawn basin leads are to be a minimum 200 mm in diameter for single catch basins and 250 mm in diameter for double basins. Also, the slope for all of the leads are to be a minimum 1.00% for sufficient drainage.

7.3 Runoff Coefficient

The runoff coefficient, *R*, accounts for any infiltration into the ground and evapotranspiration. As stated in table 5.3(h) Runoff Coefficients in the City of Surrey Guidelines, for single-family residential homes, a runoff coefficient of 0.60 is to be used for runoff calculations. For roadways and curbs, a runoff coefficient of 0.90 was used.

7.4 Rainfall Intensity

The rainfall intensity, *I*, is the average rainfall rate for a duration equal to the time of concentration for a selected return period. An intensity duration frequency (IDF) curve was used to determine the rainfall intensity for the drainage area by using the time of concentration calculated. For this project, an IDF curve of the Surrey Kwantlen Park was used for the sizing of the storm sewer.

This IDF curve was used since there were no IDF curves for the Maple Ridge municipality that could be found and the Surrey Kwantlen Park provides a fair representation of the rainfall conditions experienced by where the subdivision is located (i.e. relatively similar elevations).

As per the City of Surrey Guidelines Section 5.4.B Level of Service, the storm sewer should be designed for a 5-year return period rainfall event.

7.5 Time of Concentration

In order to determine the rainfall intensity, the time of concentration must be defined. The time of concentration is the amount of time required for stormwater runoff to travel from the most remote point in the drainage basin to the point of analysis. The hydraulic path does not necessarily have to cover the greatest distance but rather take the longest time to reach the point of interest in the sub-catchment area.

Additionally, there are different types of flow that the runoff can make its way towards the point of interest, which are: overland flow, channel or shallow concentrated flow, and stream flow. The summation of all the flow times defines the time of concentration for the sub-catchment area.

In order to ensure uniformity in unit runoff computations for each pipe network branch, a minimum time of concentration of 10 minutes was used when necessary, as stated in the City of Surrey Guidelines Section 5.3.C.d Time of Concentration in Developed Basins.

7.5.1 Overland Flow

Overland flow is usually the first type of flow as the runoff starts from the remotest point. Typically, overland flow paths are limited to about 30 m before consolidating into a more concentrated flow. There are several types of overland flow formulas, for this project, the Airport Method by the Federal Aviation Agency was used:

$$t_o = \frac{3.26(1.1 - C)L^{0.5}}{s^{0.333}}$$

Where:	t_o	=	overland flow travel time, min
	C	=	runoff coefficient
	L	=	length of overland flow path, m
	s	=	slope of overland flow, %

7.5.2 Stream/Pipe Flow

Stream/pipe flow is usually the last and the fastest flow to occur along the hydraulic path. Manning's equation was used to define the travel time of pipe flow in catch basin and lawn basin leads, as well as for upstream pipes:

$$v = \frac{1}{n}\left(\frac{D}{4}\right)^{\frac{2}{3}} s^{1/2}$$

Where:	v	=	flow velocity, m/s
	D	=	pipe diameter, m
	s	=	channel slope, m/m
	n	=	friction factor (Manning's n)

Figure 12 on the next page shows the locations of the most remote points for each pipe branch and the assumed flow pathways which are marked in red; dashed for overland and continuous for pipe flow.

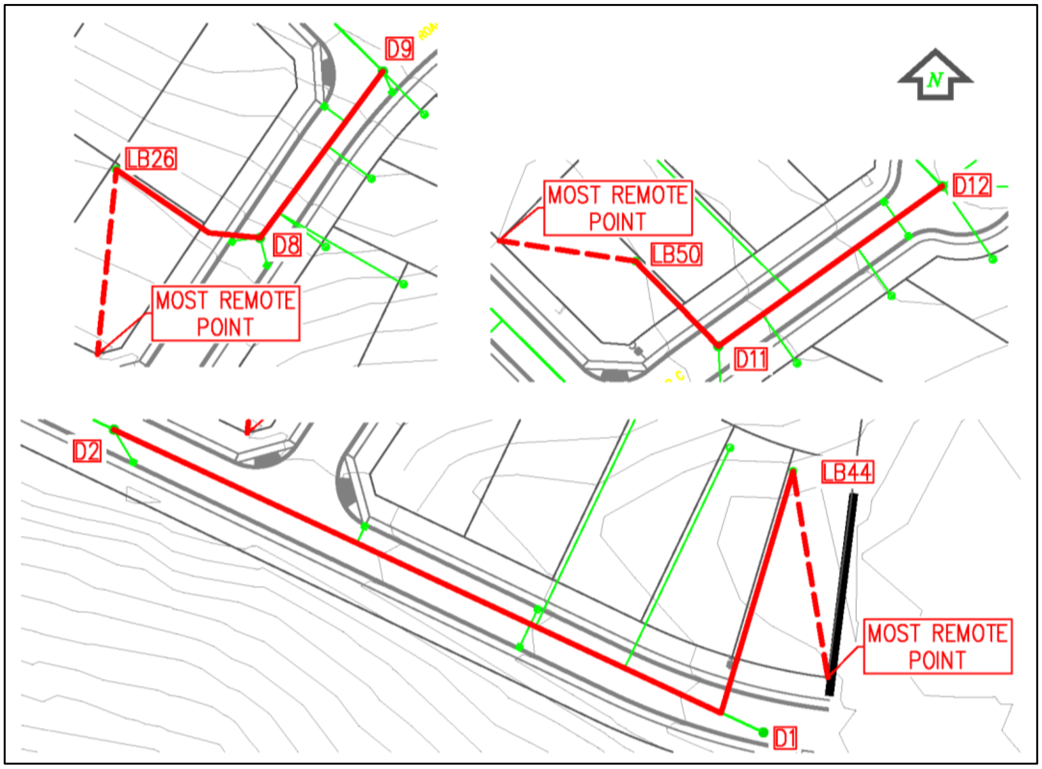


Figure 12 : Time of concentration pathways

As per City of Surrey Guidelines Section 5.4.C Sizing of Storm Sewers, a Manning's *n* value of 0.013 was used for all standard smooth wall plastic and concrete pipes. It should be noted that the time of concentration for manholes D9 and D11 was smaller than the permitted 10 minutes, therefore, a time of concentration of 10 minutes was used for design for those pipe branches.

7.6 Sizing of the Storm Sewer Pipes

The storm sewers were sized using the peak flow from the Rational Method and Manning's equation. By rearranging Manning's equation, the required pipe diameter for each storm sewer segment can be determined:

$$D_{req} = \frac{4^{\frac{5}{3}} n Q}{(\pi s^{1/2})^{\frac{3}{8}}}$$

Where:	D_{req}	=	required pipe diameter, m
	Q	=	peak discharge flow, m^3/s
	s	=	pipe slope, m/m
	n	=	friction factor (Manning's n)

The slopes for each pipe segment proved to be a challenge due to the lawn basin leads. Ideally for design, the slope of the storm sewer should be parallel to the slope of the above roadway surface, while following design criteria for the minimum and maximum pipe flow velocity. As well, the storm sewer are required to have a minimum cover of 1.0 m from the road surface. This type of design would provide the lowest cost of construction for the storm sewer system since less excavation and top soil is required.

However, this ideal design was not possible for pipes located on Road B due to the lawn basins located at the back of the lots. Since it is considered bad practice to connect neighbouring lawn basins together and forming a network on its own, the leads had to run towards the storm sewer on Road B. The issue that arises is that these leads are not running in parallel with the ground slope. Since lawn basins require a minimum 1.00% slope, the storm sewer had to be designed at a lower elevation than desired in order to accommodate for the lead connections.

As required by the City of Surrey Guidelines Section 5.4.C.d Storm Sewer Slope Requirements, the maximum and minimum flow velocities were as follows:

- minimal flow velocity of 0.60 m/s
- maximum flow velocity of 3.00 m/s

Volume II Appendix E shows a summary table of all of the calculations described in this section. It should be noted that the pipe diameter use for design was a lot bigger than what was required. This was done as a contingency factor to accommodate for any additional storm sewer tie-ins into manhole D1 from the adjacent future residential development. Refer to Volume III for accompanying storm sewer plan and profiles drawings.

8.0 DETENTION FACILITY

The typical purpose of a detention facility is to collect and convey stormwater runoff caused by new development. Detention facilities are designed to temporarily store and release runoff in a controlled manner, mimicking pre-development site conditions. Detention facilities can also provide sufficient time for suspended particulate matter to settle and can help reduce the concentration of contaminants in receiving waters. The following is a methodology of the design for the stormwater detention facility for the subdivision.

8.1 Types of Detention Facilities

There were two types of detention facilities that were considered for the subdivision: a detention pond and retention pond. The primary difference between the two is that detention ponds are required to completely empty after a storm has passed, while retention ponds maintain a constant water level year round.

A detention pond facility was selected for the subdivision. This decision was primarily based on the limited amount of available area for the pond. Retention ponds require a larger area than detention ponds due to a year round water level that they have to maintain. With the absence of a year round water level, detention ponds require less space and therefore, it was the most practical option.

We also considered an extended detention pond which is a two-bay pond. Extended detention ponds are typically designed to detain runoff from smaller, more frequent storms. They are also able to effectively control the "first flush" of storms, which is important in stormwater management since most water quality contaminants are conveyed from impervious surfaces during the initial stages of a storm event.

However, this type of a facility also required a space larger than what is available. Therefore, a conventional detention pond was selected for design.

8.2 Detention Pond Modeling

For the design of the detention pond, runoff analyses of pre-development and post-development peak flows were conducted using a hydrologic computer model. Autodesk Storm and Sanitary Analysis 2015 was used for the two models since the program had the capabilities to import storm sewer elements and topographic layouts directly from AutoCAD Civil 3D, which reduces the time required to set up the hydrologic model.

The primary purpose to develop pre-development and post-development models is to determine the required discharge flow and detention volume. This information is used for the design of the outlet control structure, as well as the design of the shape and depth of the pond. It is important to note that the City of Surrey Guidelines do not have any detention time requirements that must be met once a storm has passed. A detention pond that retains runoff for longer durations may provide sufficient time for suspended particles and contaminants to settle. However, this requires onsite testing and is beyond the scope of the project. The primary goal is to limit the downstream effects of increased runoff caused by development.

The EPA Storm Water Management Model (EPA SWMM) was the hydrologic method used to model the runoff. Additionally, the SCS Curve Number infiltration method was used to model the infiltration capacity of the soil. The following sequence of steps were performed to develop a detention pond design for the subdivision, which included

- creating a 24 hour 2-year, 5-year and 100-year design storm

- defining pre and post-development soil conditions
- determining sub-catchment average slopes and widths
- generating stage-routing curves to determine the discharge flow and detention volume
- developing a detention pond shape and contours
- designing an outlet structure and its features.

These steps are further described in detail in the subsections below. Volume II Appendix I show a summary of important detention pond data.

8.2.1 Design Storm & Capacity Requirements

As per the City of Surrey Guidelines Section 5.6.D.e Engineering Drawing Requirements, detention pond facilities are required to temporary store runoff from a post-development 24 hour 5-year storm event. As well, the facility must be able to manage a 24 hour 2-year and 100-year storm event.

One of the benefits of using the Autodesk Storm and Sanitary Analysis software is that it includes a rainfall designer which allows for the selection of any location within the USA and provides the design rainfall for a specified storm frequency. In order to use this built-in capability, the following assumptions were made:

- SCS Type 1A 24-hour storm distribution
- rainfall distribution values from the Whatcom County in Washington state

With these assumptions a rainfall model was developed that closely resembles a probable rainfall event expected in the Greater Vancouver Area. The software has a built-in corresponding total rainfall depth in its rainfall database. This was changed to match the total rainfall depth found in storm intensity tables in 5.3 (d) of the City of Surrey Guidelines for Kwantlen Park.

8.2.2 Defining Pre-Development and Post-Development Soil Conditions

With the design storms defined, the pre-development and post-development soil conditions were established. This will define how much rainfall will infiltrate into the ground during a storm event. Table 3 lists a summary of the soil conditions that were assumed.

Table 3 : Assumed Soil Conditions

	Hydrologic Soil Group	Curve Number (CN)
Pre-development	C	72 (good woods and grass combination)
Post-development	B	85 (1/8 acre lots)

A hydrologic soil group C was selected for the pre-development case due to existing site descriptions indicating a moderately high runoff potential due to slow infiltration rates caused by a clay layer underneath overburden. As well, after viewing the area from Google Maps, the existing site appeared to be mainly wooded with pockets of open grass in some areas, therefore, an assumed CN of 72 was made.

A hydrologic soil group B was used for the post-development case due to the disturbance of the surface layers of the soil from grading construction. The existing clay layer would be disturbed and mixed with the soil layers above and beneath it during this phase of construction. This will allow for better infiltration of rainfall into the soil and therefore a soil group B was appropriate. Additionally, since most of the lots were about 1/8 of an acre in size, a CN of 85 was assumed.

8.2.3 Average Slopes and Widths of Sub-catchment Areas

The Kinematic Wave time of concentration method, as is used for the EPA SWMM hydrology method, requires that an equivalent, rectangular sub-catchment be determined from the actual sub-catchment. As shown in Figure 13, the hydrologic runoff response of this equivalent rectangular sub-catchment should closely match that of the actual sub-catchment.

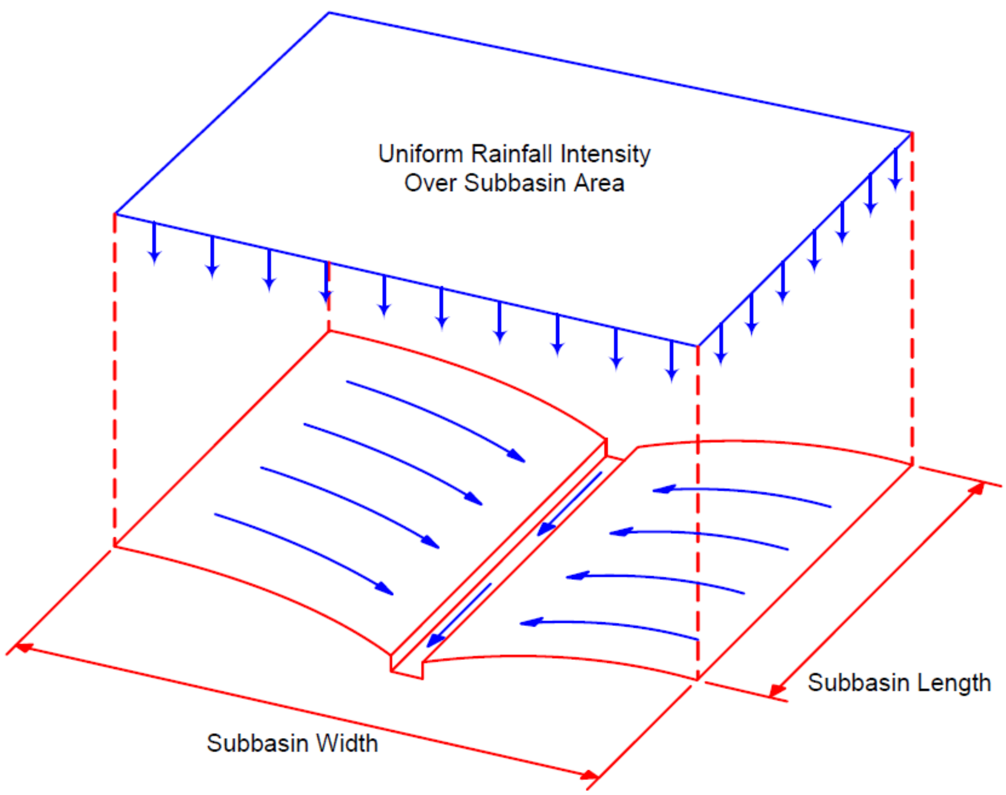


Figure 13: The constructed equivalent sub-catchment *(Autodesk, 2014)*

For the pre-development condition, the catchment area used was the tributary area for the detention basin (i.e. the post-development catchment area). Although this is a post-development area, it was used so that the pre-development and post-development catchment areas would be equal and therefore give a fair comparison of runoff. If not, the actual pre-development area would contribute negligible peak runoff at the point of interest (i.e. location of the detention pond).

The average slopes for the sub-catchments was determined using Civil 3D and taking the average slope of all of the potential runoff pathways for overland flow and shallow concentrated flow. This value, in addition to the width of the sub-catchment area was used to compute the time of concentration for the Kinetic Wave method.

8.2.4 Determining Peak Discharge Flows and Detention Volumes

The required discharge flow and detention volume for a 24 hour 5-year storm event was determined by running both the pre-development and post-development models. An inflow hydrograph for the pre-development and post-development conditions was produced. Similarly, this process was repeated for the 2-year and 100-year storm events as well. Volume II Appendix G shows the hydrographs created for the various storm events. Table 4 shows a summary of the peak flows and the required detention for each storm event.

Table 4: Summary of peak flows and required detention volume

Storm Event	Pre-development Peak Flow (m^3/s)	Post-development Peak Flow (m^3/s)	Required Detention Volume (m^3)
2-year	0.06	0.24	736
5-year	0.09	0.32	786
100-year	0.22	0.55	709

The information show in Table 4 was used to model the stage-routing time series for the detention pond. This process also includes the design of the outlet control structure, as well as the shape and depth of the pond.

8.2.5 Defining Pond Shape and Depth

AutoCAD Civil 3D was used to define the shape and depth of the pond. As per the City of Surrey Guidelines Sections 5.6.F.c, 5.6.F.d, and 5.6.F.e, the design requirements included the following:

- side slopes to have a maximum 4:1 (H:V)
- maximum live storage of 3.0 m for a 100-year storm and 1.5 m for a 5-year storm
- a minimum bottom grade of 0.7% from the inlet to the outlet structure
- minimum 500 mm freeboard depth above the 100-year storage depth.

The detention pond was shaped and configured such that it would be aesthetically pleasing to the residents living in the subdivision. Extra care was taken to make sure that the pond did not encroach neighboring properties, as well as, taking into account the edge of the nearby steep slope that the detention pond would be situated on. Due to a concern of slope stability, slope stability analysis was done for the pond and is further described in Section 11.3 Slope Stability Analysis.

This part of the design process required a bit of fine tuning in order to define a finalized pond shape and depth. We also had to consider the invert elevation of the inlet pipe, as well as the gradual slope of the pond to the outlet structure. Refer to Volume III for plan and profile drawings of the detention pond.

8.2.6 Inlet Control Structure

The purpose of the inlet control structure is to dissipate energy from runoff being discharged into the pond. This consists of riprap being placed at the inlet outfall and having a riprap pilot channel that runs to the outlet structure. A class 10 riprap will provide sufficient energy dissipation at the outfall and for the pilot channel.

8.2.7 Outlet Control Structure

A detention facility's outlet structure allows flow to discharge from the pond at a controlled rate. The control structure focuses on the management of multiple storm events, thus requiring the need for different types of outlet openings at different pond stages. As well, the structure has an outlet culvert that will route water to an open outfall, from which the runoff will flow in an open channel towards the nearby creek.

As indicated in Figure 14, there are two types of outlet openings: a control orifice and an overflow riser. The designs of these features are described in the following subsections.

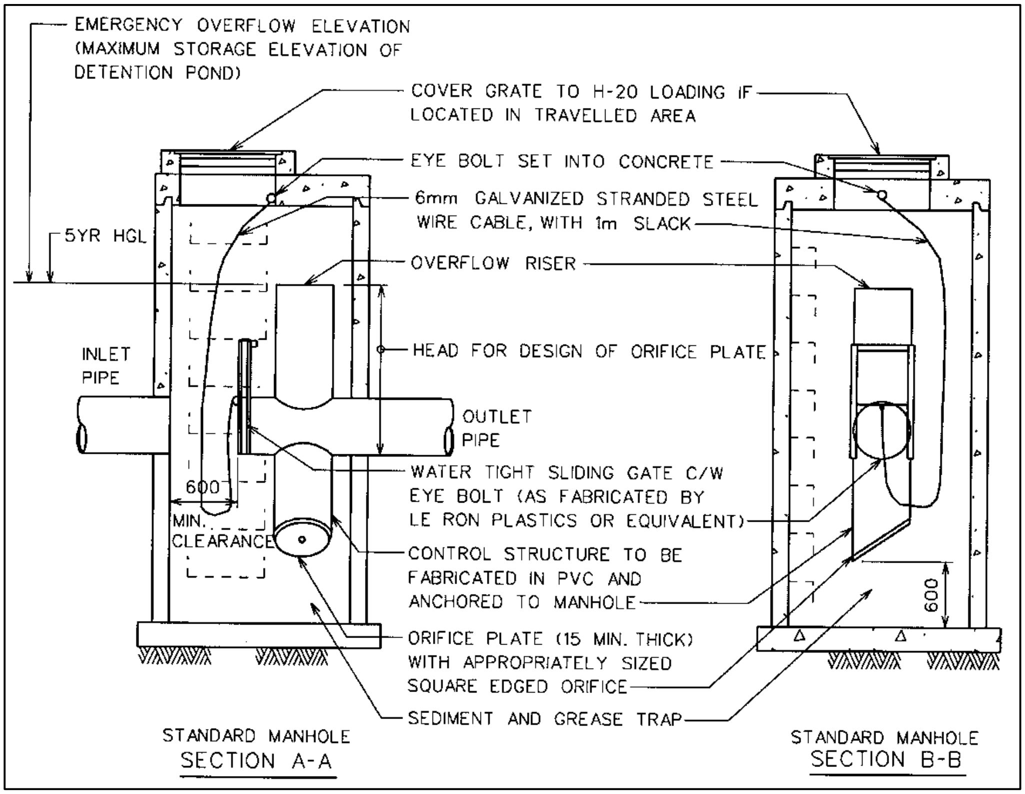


Figure 14: Outlet control structure detail *(Master Municipal Construction Documents, 2009)*

8.2.7.1 Control Orifice

A control orifice regulates the discharge flow out of the pond, mimicking pre-development flow conditions. As determined previously in section 8.2.4 Determine of Peak Discharge Flows and Detention Volume, the pre-developed peak flow was 0.09 m^3/s and was used to size the orifice using the orifice equation:

$$Q = CA\sqrt{2gh}$$

Where:	Q	=	flow rate, m^3/s
	C	=	discharge coefficient (0.612)
	A	=	area of orifice, m^2
	g	=	acceleration due to gravity, m/s^2
	h	=	head difference of the orifice, m

From the orifice equation, it was determined that an orifice diameter of 217 mm and a head height of 0.80 m would be adequate to provide an outflow of 0.09 m^3/s during peak periods. However, since orifice diameters are not available in that diameter size, an orifice diameter of 225 mm was selected for design. Therefore, with a larger orifice an outflow of 0.095 m^3/s and head depth of 0.77 m was obtained. This is still considered satisfactory since a larger diameter size will be less likely to clog from debris and the peak flow still closely resembles a pre-developed condition.

Additionally for detention requirements, a detention volume of 827 m^3 was found from analysis, and satisfying the detention criteria for a 5-year storm as described in Table 4. Table 5 is summary of values for the peak outflow, pond depth, and detention volume for the various storm events.

Table 5: Summary of peak outflows, pond depths, and detention volumes

Storm Event	Peak Outflow (m^3/s)	Pond Depth (m)	Detention Volume (m^3)
2-year	0.081	0.56	748
5-year	0.095	0.77	827
100-year	0.106	0.96	1163

8.2.7.2 Overflow Riser

The overflow riser is an additional safety measure that allows for the bypass of runoff into the outlet pipe. This commonly occurs for during severe storm events or when the control orifice is clogged by debris. As shown in Figure 14, the height of the overflow riser is consistent with the normal water depth of a 5-year storm. A normal water depth of 0.77 m was obtained from the previous calculations of the control orifice. Therefore, if a higher frequency storm would to occur, the depth of water would rise past the 0.77 m level and runoff would then flow into the overflow riser.

The size of the overflow riser was determined using the weir equation:

$$Q = C\pi D h^{1.5}$$

Where:			
	Q	=	flow rate, m^3/s
	C	=	discharge coefficient (3.3)
	D	=	diameter of weir, m
	h	=	total head on crest, m

For the design of the riser, the worst case scenario was assumed where the control orifice is clogged by debris and a 100-year storm event has occurred. Using a 100-year post-development flow rate of 0.55 m^3/s, and an assumed total head of 0.35 m, a riser diameter of 350 mm was selected. Therefore, the total depth of the pond was calculated to be 1.14 m. However, this water depth would in fact be much smaller due to the emergency spillway providing simultaneous discharge out of the pond as well.

8.2.8 Emergency Spillway

The emergency spillway is an additional safety feature that allows for the detention pond. In the event that obstruction in the outlet pipe has occurred, the emergency spillway will ensure that no flooding will take place since the spillway bypasses the outlet structure and discharges runoff directly into the open channel.

As mentioned, the emergency spillway will work in conjunction with the overflow riser. For design, it was assumed that severe clogging had occurred in the outlet pipe of the control structure and that a 100-year storm has occurred.

The design capacity of the emergency spillway was determined using the broad-crested weir equation:

$$Q = C\sqrt{2g}\left(\frac{2}{3}Lh^{1.5} + \frac{8}{15}\tan\theta\, h^{2.5}\right)$$

Where:	Q	=	flow rate, m^3/s
	C	=	discharge coefficient (0.6)
	g	=	acceleration due to gravity, m/s^2

L	=	length of weir, m
h	=	height of water over weir, m
θ	=	angle of side slopes, radians

As per the Stormwater Management Manual for Western Washington Section 3.2.1 Detention Ponds, a minimum side slope of 3:1 (H:V) was used, as well as an assumed weir length of 2 m. Using the 100-year post-development flow rate, a weir height of 257 mm was determined. Thus, a weir height of 300 mm was used in order to allow for some contingency for vegetation obstruction in the spillway.

Erosion control for the emergency spillway was checked and was not necessary since the maximum velocity through the spillway was about 0.47 m/s. A maximum velocity of 1.0 m/s for an unlined ditch is permitted under Section 5.4.G.b Ditches in the City of Surrey Guidelines.

It should be noted that for 100-year storm occurrences, and when there is no clogging in any of the outlet features, a maximum pond water depth of 0.96 m was determined. If however, only the overflow riser and emergency spillway are active (i.e. the control orifice is clogged), a maximum pond water depth of 1.01 m was found. Furthermore, the detention pond and the outlet structure meet the design criteria for as described in the City of Surrey Guidelines.

8.2.8 Outlet Culvert

The outlet culvert directs the flow from the control structure to the outfall location of the open channel. The culvert was assumed to be operating under inlet control conditions, where the culvert is capable of conveying a greater discharge than what the inlet will accept. Since there are no specific guidelines for the design of the outlet pipe, it was assumed that the size of the culvert shall be large enough for a design flow capacity of a 100-year storm event. As well, BC MoT Guidelines for culvert design were used as a reference when necessary.

The design of the culvert was done using the Autodesk Storm and Sanitary Analysis software. From the model, a 375 mm diameter reinforced concrete culvert was deemed satisfactory. Table 6 is a summary of the analysis for the culvert.

Table 6: Analysis summary of the outlet culvert

Constructed Slope	2.52%
Design Flow Capacity	0.28 m^3/s
Peak Flow during Analysis	0.278 m^3/s
Maximum Velocity	2.87 m/s
Mean Velocity	1.86 m/s

As per the City of Surrey Guidelines Section 5.4.I.a General, erosion protection is necessary for culvert discharge velocities that are greater than 1.0 m/s. Therefore, riprap was used protection at the culvert outfall. As per BC MoT 1030.04 Channel Lining, for velocity against the bank that is parallel to the flow, velocity against the bank is described as:

$$v_s = \frac{2}{3} v_m$$

Where:	v_s	=	velocity against the bank, m/s
	v_m	=	mean velocity

Using the mean velocity found from analysis, a v_s of 1.24 m/s was determined. According to the BC MoT Riprap Design Chart, a v_s that is smaller than 1.3 m/s does not require riprap. However, since the City of Surrey Guidelines state that riprap protection is necessary, a class 10 riprap was selected for erosion protection at the outfall. This riprap shall be placed in the channel bed and side slopes of the culvert outfall.

8.2.9 Open Channel Design

The open channel will route the runoff from the culvert outfall to the creek. As per the City of Surrey Guidelines section 5.4.G.b Shape, all ditches/open channels shall abide by the following design criteria:

- trapezoidal shaped
- minimum 600 mm freeboard
- maximum sides slopes of 1.5:1 (H:V)
- minimum bottom width of 0.5 m
- maximum velocity of 1.0 m/s for unlined ditches

The pathway of the open channel proved to be difficult due to the steep grade that leads to the creek. The first pathway option that was created had the channel zig-zag down the slope towards the creek. This option would reduce the slope of the channel, as well as the flow velocity running through it. However, this option was disregarded as it would be difficult to construct and could cause slope stability issues.

Therefore, as shown in Figure 15, a second pathway option was developed and used for design. As illustrated in Figure 15, the pathway runs directly down the embankment to the creek, which should have the least amount of impact on the slope's stability. However, this option would also be difficult to construct since the constructed slope would be about 24%. To provide adequate erosion protection, a concrete lined channel would be used. The lined channel will also be embedded with riprap for increased energy dissipation. Low slump concrete would be used in order for the concrete liner to be installed.

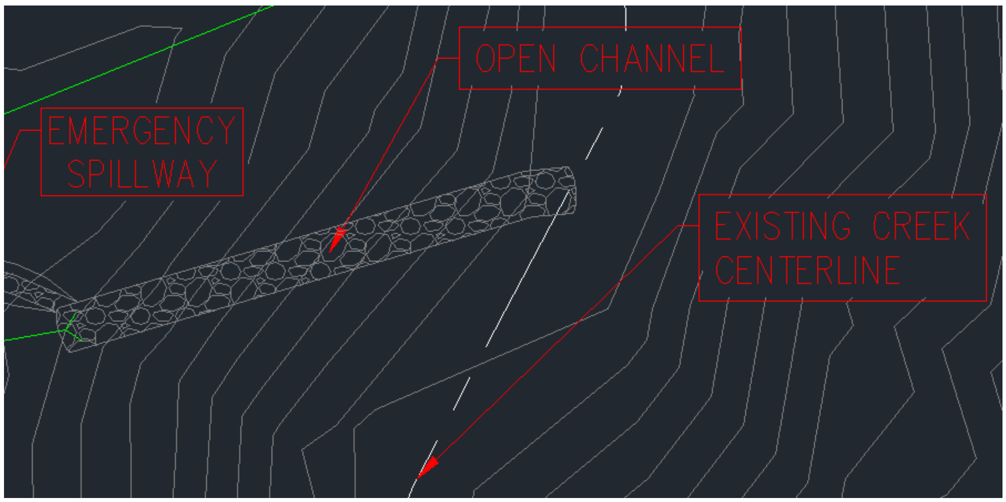


Figure 15: Open Channel Pathway

The design of the channel was completed using the Autodesk Storm and Sanitary Analysis software. Table 7 is a summary of the physical properties of the channel and the analysis of the flow.

Table 7 : Summary of Channel Properties

Physical Properties	
Height of Channel	0.70 m
Bottom Width	0.50 m
Side Slopes	1:1.5 (V:H)
Channel Length	29.58 m
Inlet Invert Elevation	12.00 m
Outlet Invert Elevation	5.00 m

Analysis Summary	
Constructed Slope	23.67%
Design Flow Capacity	6.66 m^3/s
Peak Flow during Analysis	0.454 m^3/s
Maximum Velocity	3.05 m/s
Mean Velocity	1.55 m/s

9.0 WATER DISTRIBUTION SYSTEM

The water distribution system will provide clean drinking water to the subdivision's residents, but will also be required for emergency situations. If there is a fire in or within 150 m of the development, it is required that there be sufficient pressure at each fire hydrant to be able to extinguish a large fire. The design of this system included:

- horizontal and vertical layout of the pipes
- pipe sizing calculations
- valve and hydrant location
- design of a pressure reducing valve

The rest of this section describes the above components of the water distribution system.

9.1 Pipe Layout

The horizontal layout of the watermain follows the proposed horizontal road network. This allows for the watermain to follow a "looped" layout. A "loop" is essential in terms of water distribution as it allows and in emergencies, a shutdown of parts of the system while keeping other parts in use. By using a loop and integrating valves at appropriate locations, the network can be turned off in particular areas without causing disruption to other users in the development.

9.1.1 Horizontal

In our design, the watermain follows the alignment of the roadways. However, to meet the sanitation requirements, and safety of the public, the pipe is required to be 3 m clear of the storm or sanitary utility.

9.1.2 Vertical

Unlike the storm and sanitary pipes, the water network is a pressure based system. This allows for the vertical layout to follow the profiles of the road. As long as the design criteria of a minimum slope of 0.1% and maximum of 10% are satisfied, the water can move upwards and downwards as a result of the pressure.

9.2 Flow Calculations

The first step in the design of the water distribution system is to determine the rate of water required to service the homes, as well as the fire flow requirements of the subdivision. The design flow is calculated as the greater of the maximum daily demand plus the fire flow or the peak daily demand. Usually, the peak daily demand plus the fire flow governs the design. The City of Surrey Design Manual was followed for these calculations Refer to Volume II Appendix B for flow calculations.

9.3 Pipe Sizing Calculations

To design the water distribution network in the new development, the City of Surrey Design Criteria Manual was used. This method uses hydraulic principles, including the Hazen-Williams equation to ensure there is sufficient pressure in the pipe system.

Once the water pipe network was laid out, we determined the elevations and the distances the water has to travel. From these distances and elevations, a simplified schematic plan of the distribution network was created. This schematic can be seen below.

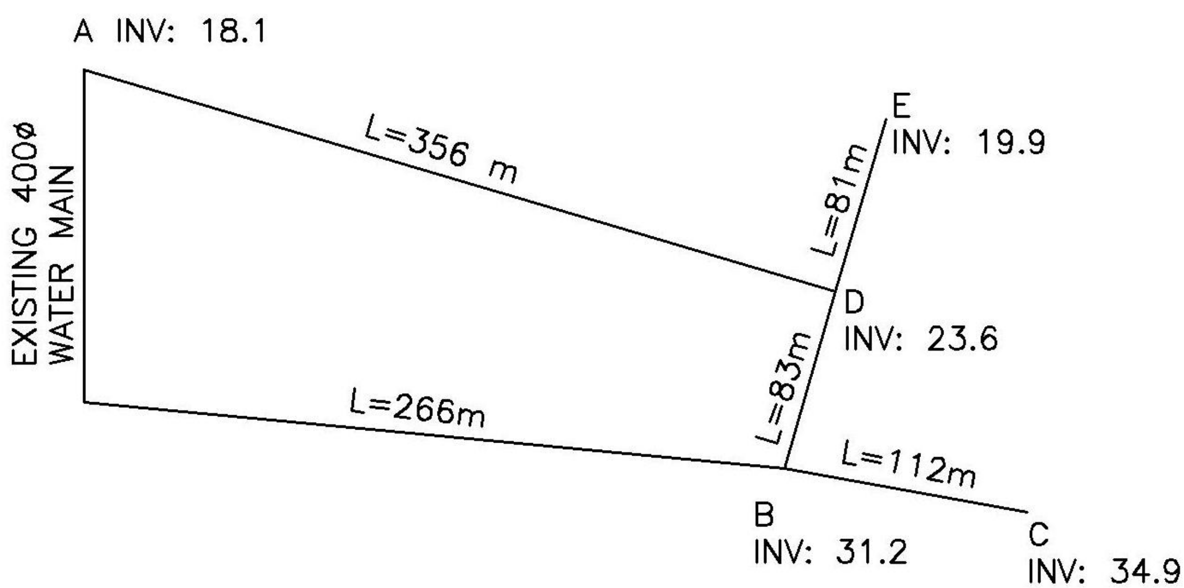


Figure 16 : Water Distribution System Schematic

To analyse this network, we used the Hazen-Williams equation and the Bernoulli Equation (assuming steady-state incompressible flow). These equations are shown below.

Hazen-Williams Equation:

$$Q = 0.28 \times C \times D^{2.63} \times \left(\frac{hl}{L}\right)^{0.54}$$

Where:

Q = Water flow (m^3/s)

C = Pipe roughness coefficient

D= Pipe diameter

hl = Head loss through pipe

L = Pipe Length

Bernoulli Equation:

$$h_1 + \frac{P_1}{\gamma} + \left(\frac{{v_1}^2}{2g}\right) = h_2 + \frac{P_2}{\gamma} + \left(\frac{{v_2}^2}{2g}\right)$$

Where:

h = Elevation head

P = Pressure Head

v = Velocity

g = Acceleration due to gravity

γ = Unit weight of water

The analysis method used to design the water main consisted of converting all of the pipes in the network to “pipes of equivalent diameter” with the Hazen-Williams equation by relating head losses. Once this was done, the Bernoulli Equation was invoked to investigate working pressures at critical points along the distribution network. These are the farthest points, because of the friction losses that occur as the water moves in the pipe, and the highest points, due to the pressure energy loss as the water is moved to higher elevations. By using this method, it ensures that the residents in the subdivision can have access to water with little to no flow interruption. The calculations for water main design can be found in Volume II Appendix B.

9.4 Pressure Reducing Valve

On 240th Street, west of the site, there is an existing 400 mm diameter water main. To get water into the subdivision development, this existing main was used. To accomplish this, a pressure reducing valve (PRV) was necessary due to the large pressure of 200 psi in the existing main. The figure below shows the configuration of a typical PRV.

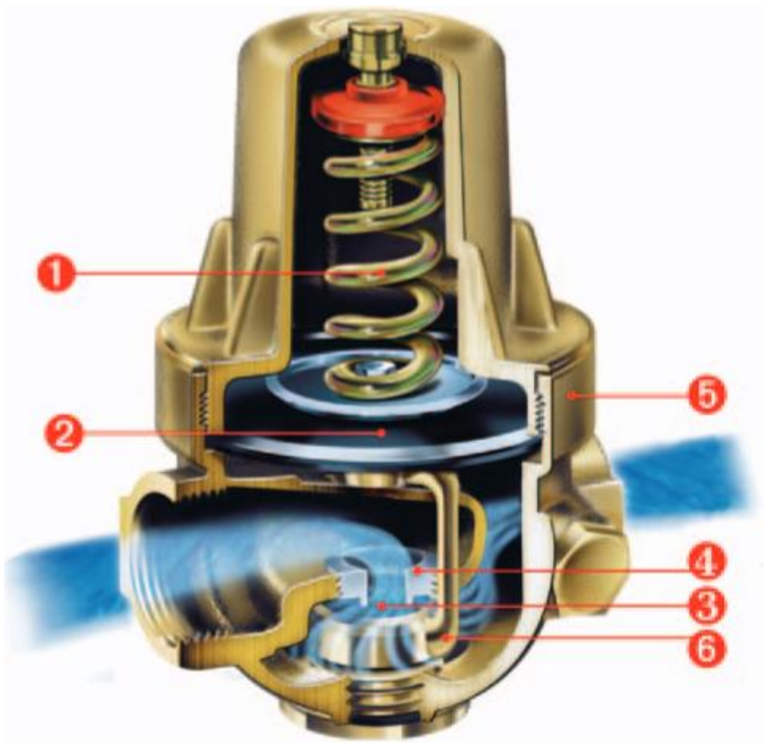


Figure 17 Typical Configuration of a PRV (Watts, 2015)

The valve is mounted inside a chamber that contains a backup valve on a bypass, in case the full-time valve requires servicing or maintenance.

9.4.1 PRV Background

To conserve the amount of water used within the subdivision, a pressure reducing valve is used to reduce the water flow rate. As more municipalities lean towards the use of water meters, there is a cost to the homeowner associated with both consumption, and disposal. If the water pressure was not reduced coming into subdivision, the high flow rate would result in a waste of the additional water coming out of the tap. Furthermore, if less water flows through the system, less energy is required to heat hot water. Also, the load on the wastewater treatment plant is reduced.

By installing a pressure reducing valve from the existing main, water is constricted inside the valve body and directed through the inner chamber. It is controlled by an adjustable spring-loaded diaphragm and disc. The PRV ensures a constant flow of water, even if the incoming flow rate fluctuates. This is assuming that the incoming flow does not fall below the valve's pre-set pressure.

9.4.2 PRV Performance

By configuring the pressure reducing valve by using two valves instead of one, it keeps a consistent supply of water going to the customers. By having two valves, it ensures that water will continuously go into the subdivision even when one requires servicing. The shutdowns to the system can be quite costly.

When a system of two PRVs in parallel are used, one should be set to be 10 psi higher than the other. This is because when low volume is required, the higher set valve operates alone, and when more volume is needed, both valves open and deliver full capacity. Therefore, in our network, to reduce the pressure from 200 psi at the existing main to 130 psi at the entrance to the subdivision, the valves should be set at 50 and 60 psi reduction.

9.5 Pipe Fittings and Valves

Other components required in the water network system are valves, which can be turned off in case a portion of the network requires servicing. Single valves are needed every 200 m along the main, two at "tee" connections, and three at "cross" connections. Fire hydrants are also required every 200 m, and cannot be more than 150 m from a lot. A major restriction is that a hydrant must branch off a loop portion of the water network. Another element which is needed for optimal flow is a "blow-off". It is installed at the high point of the network to remove air from the pipes, so that when in use, the system provides a consistent, uninterrupted flow.

9.6 Watermain Results and Recommendations

Some of the challenges from the design of the storm and sanitary pipe network design were that the existing main is higher than the land that is to be developed. However, in the design of the water main, this works as an advantage. The water, at a higher elevation has greater potential energy than at a lower elevation. Therefore, it is much easier to overcome the frictional energy losses of the pipe as the water moves to a lower elevation, and have the required pressure head available for residential and fire hydrant usage.

From the Guidelines of the City of Surrey Design Criteria manual, ductile iron pipe was selected as the pipe material with a "C" value of 100. The entire system was checked for the usage of a 200 mm diameter pipe, and through the use of Bernoulli's equation, proved to be sufficient for distribution to all the lots in the subdivision, as well as the fire flow requirements at the farthest and lowest points. The horizontal layout was able to be "looped", as pipe was laid under all of the proposed roadways. Furthermore, in the vertical profile, 1 m of clear cover is retained throughout the site, as the watermain is not required to retain a downward slope.

9.6.1 PRV Results and Recommendations

To ensure that the pressure has sufficiently been reduced coming into our subdivision, a set of Cla-Val Model 690-01 meets the recommended maximum pressure requirements (See Volume II Appendix N for Product Brochure). The main line will have an 8" valve (lag valve) to match the 200 mm pipe water main in the subdivision. It has a flow range of 145 L/sec. to 7.2 L/sec. A second smaller valve will be installed in parallel and operate continuously for normal flows. This will be a smaller 690-01 model of 3" (16 to 0.9 L/sec.) or 4" (37 to 1.9 L/sec.) and act as the lead valve. It is to be set at 10 psi higher than 8" valve to handle lower demands. Also, the system is to be installed at both tie-in points to the existing water main. The PRV system is not required to be sized for maximum flow requirements (due to fire flow) as the oversized valve will operate in a nearly closed position that causes premature wear and undesirable noise.

10.0 ROAD DESIGN

The road network is a crucial component in the subdivision design. It influences the size of the lots in the subdivision as well as the accessibility to end users and emergency services. The key goal of the road network layout was to optimize both the horizontal and vertical components in terms of earthworks movement and design restrictions.

10.1 Horizontal Road Layout

The horizontal road layout was designed using AutoCAD Civil 3D. All of the onsite roads are designed as “Through Local” roads as defined in the City of Surrey Design Manual.

While completing the design of the horizontal road layout, our main design constraint was the location of a proposed future development on the southeast corner of our site. The proposed horizontal road layout services the entire site and provides access to the future road extension. Another general rule we tried to follow was to keep our roads parallel with the existing contour lines. This limits the steepness of the roads and provides a safer driving experience.

10.2 Vertical Road Layout

When designing the vertical layout of the road, the idea is to maintain as much of the original ground profile as possible. Minimizing the quantity of earthworks lowers the overall cost of the project. By using AutoCAD Civil 3D, we were able to view the road profiles and adhere as much as possible to the original surface. However, due to the steep slopes on this site, there were certain areas that required significant cut or fill. The TAC Geometric Design Guide was followed for the vertical design of the road network.

10.3 Intersection Design

In the proposed development there are 2 onsite intersections. The design of these intersections was performed using AutoCAD Civil 3D. Key aspects of the design included maintaining cross falls as roads intersected each other and ensuring adequate drainage of the intersection. Typical intersection details shown in the City of Surrey Supplementary MMCD were followed during the design of the intersections. The following image shows a three-way intersection in Maple Ridge.

Figure 18 : Intersection in Maple Ridge (Google Maps, 2015)

The design of the intersections has to harmonize the priorities and elevations of two roadways meeting each other. In our design, we equally prioritized the needs of both roads A, B and C in terms of conforming to their elevations and how their cross slopes interact.

10.4 Road Design Results

The horizontal layout of the roadways is designed along with the lot layout in order to optimize the use of the land. In our design, the road is laid out to accommodate the utilities and allow for future development towards the east. A loop was implemented into our road network to allow for greater accessibility for residents, but more importantly emergency vehicles. The roadway on the south side of our project also allows for plenty of future development, as its entire face will still be undeveloped.

11.0 GEOTECHNICAL CONSIDERATIONS

The geotechnical considerations of the subdivision project involved determining the site soil conditions, producing a property line retaining wall design, and analysing slope stability of the creek area.

11.1 Preliminary Geotechnical Report

The scope of work includes a review of anticipated soil conditions and preliminary geotechnical recommendations for site preparation and retaining wall design.

The anticipated soil conditions are based on available published geotechnical and geological information. These soil conditions should be verified at the time of construction.

Based on published surficial geology mapping by the Geological Survey of Canada, it is anticipated that the site area is underlain by glacial and deltaic sediments. The sediments range from marine silty clays to fine sands, commonly thinly bedded and containing marine shells. The creek area is anticipated to be underlain by lowland stream channel fill and overbank sandy loam to clay loam with organic sediments up to 8-m thick.

Structural fill to restore design grades should consist of well graded sand/or gravel with less than 5% fines (passing through #200 sieve). Structural fill should be placed in 12” thick loose lifts and compacted to a minimum of 100% Standard Proctor Density.

For the retaining wall design, the following design values shall be used:

Table 8: Soil Parameters

Soil Type	Unit Weight (kN/m^3)	Hydraulic Conductivity, k (cm/s)	Internal Friction Angle, ϕ' (degrees)
Undisturbed Dense Natural Soils, sandy silt (ML)	15.0	0.001	33
Clean Granular Backfill, well-graded gravel and sand	19.0	1.0	30

These design values were obtained from tables in *Geotechnical Engineering: Principles and Practices (Coduto)* based on the soil type. Also, the internal friction angle of the backfill and natural soil was obtained from a graph (Coduto) using a conservative relative density of 75%. These design values should be verified by lab testing.

The assumed soil profile is based on a soil description from a nearby site. There is a layer of topsoil approximately 0.3 m thick, underlain by a 0.3 to 0.6 m layer of clayey silt. Below this layer, sandy silt till was found at a depth of slightly below 1 m. The groundwater table is assumed to be at least 5 m below the surface (negligible for retaining wall design). The following figure shows the assumed soil profile of the site.

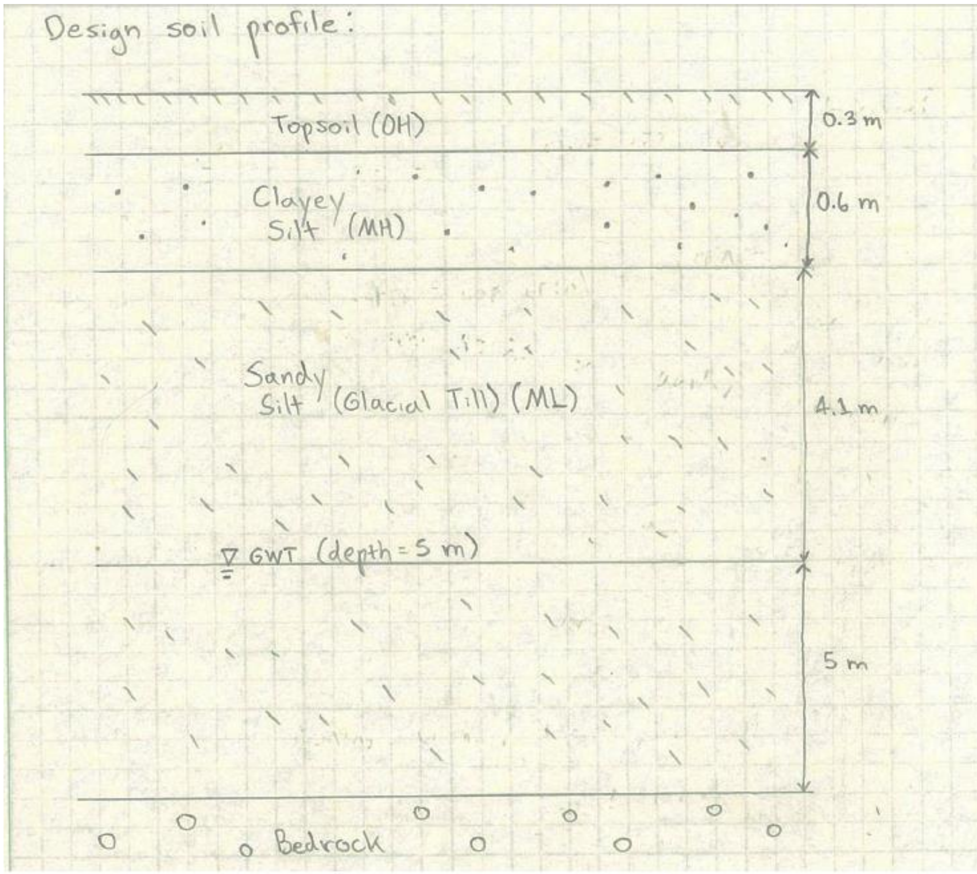


Figure 19 : Assumed Soil Profile

This soil profile is assumed to cover the entire subdivision area except for the creek setback area. Further geotechnical investigation is required upon excavation and construction, especially if the site soil conditions differ from those expected.

11.2 Retaining Wall Design

The retaining wall structure is required to accommodate the change in elevation between the rear yards of lots on Road A and Road B. The highest change in elevation is about 3.5 m, which tapers off at the existing road and towards the East.

This retaining wall sets the boundary of the property line. The following figure shows the location of the proposed retaining wall.

The selection of type of retaining wall to be designed was based on specific evaluation criteria. The table below shows the criteria and relative performance of each wall type.

Table 9: Retaining Wall Selection Criteria

Type of Wall	Footprint	Materials	Cost of Construction	Design Complexity	Aesthetics
Gravity Wall	Large, unusable area	Highest amount	Low	Simple	Unattractive
Cantilever Wall	Small, usable area above backfill	Good usage of materials, imported backfill required	Midrange due to excavation and construction	Difficult but experienced with design	Good, with masonry stem or veneer on concrete stem
Mechanically Stabilized Earth (MSE) Wall	Very small	Costly geofabrics	High	Complex and unfamiliar	Attractive with natural features

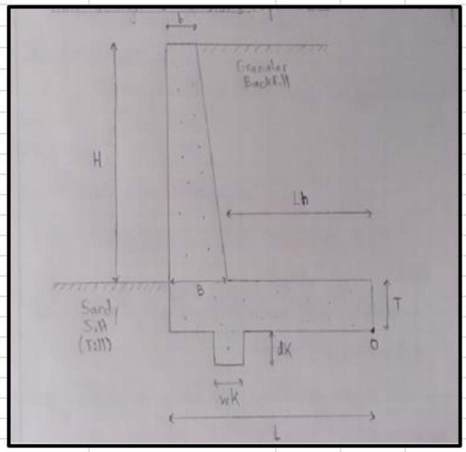
Based on these selection criteria, we decided to proceed with the design of a reinforced concrete cantilever wall. This decision was based upon the familiarity with the design procedure, aesthetic appeal, and the small footprint of the wall. A MSE wall may have been the better selection, but the complex, unfamiliar design was the main deterrent. The following photograph shows two concrete cantilever retaining walls with the right image showing a veneer finish.

Figure 20 : Photographs of Typical Concrete Retaining Walls (Engineering Tech, 2015)

To begin the retaining wall design, initial stem and footing dimensions were assumed. These were required to analyse the global and external stability of the wall from overturning, sliding, and bearing capacity failure. The design weights and soil parameters were input into an Excel Spreadsheet created to calculate the Factor of Safety against overturning, sliding and overbearing (See Volume II Appendix A). A screenshot of a section of the spreadsheet is shown below.

Retaining Wall Design Spreadsheet (External Stability)

This spreadsheet provides initial sizing of a retaining wall based on overturning and sliding failure analysis. The input sections are Design Unit Weights, Design Dimensions, and Soil Internal Angle of Friction. The structural design is to be performed seperately in the other tabs.



Design Unit Weights:		
Concrete:	24	kN/m3
Masonry:	4.4	kPa
Backfill:	19	kN/m3
Natural Soil:	15	kN/m3

Design Dimensions (See above figure):		
Stem Height (H):	3.5	m
Footing Thickness (T):	0.4	m
Stem Thickness at Top (b):	0.2	m
Stem Thickness at Base(B):	0.4	m
Footing Width (L):	2.4	m
Length of Heel (Lh):	2	m

Figure 21 : Retaining Wall Design Spreadsheet Screenshot

This allowed us to determine the optimum design dimensions of the stem, footing and shear key. It should be noted that we initially were planning to design a T-shaped cantilever wall, but our sponsor recommended an L-shaped design to keep the wall on one side of the property line.

11.2.1 Overturning Analysis

The main force acting on the back of the wall is the active soil pressure of the backfill which produces an overturning moment and sliding force. In calculating the active soil force, the effective friction angle of the backfill was used to determine the coefficient of active soil pressure. To resist overturning, the weight of the wall and weight of backfill on the heel produce a counteracting moment. The frictional force between the stem and backfill was ignored as a conservative approach. The factor of safety against overturning was calculated using the following equation:

$$FS = \frac{W_t \times d}{\left(\frac{1}{3} \times F_a \times H\right)}$$

Where:

FS = Factor of safety against overturning

W_t = Total weight

d = Resultant of total weight from front of footing

F_a = Active soil force

H = Total height of stem

The Factor of Safety against overturning was well above the recommended value of 2. Also, the passive soil force was ignored for overturning analysis (conservative assumption), as the wall may not move enough laterally to induce this force.

11.2.2 Sliding Analysis

The resistive forces acting on the wall are the passive soil pressure and shear friction between the footing and the underlying soil. These prevent sliding with the additional support of a shear key underneath the footing. The factor of safety against sliding was calculated using the following equation:

$$FS = \frac{F_p + F_{fr} + F_s}{F_a}$$

Where:

FS = Factor of safety against sliding

F_p = Passive soil force

F_{fr} = Frictional force between footing and underlying soil

F_s = Shear key force

F_a = Active soil force

The calculated factor of safety was higher than the recommended value of 1.5.

11.2.3 Bearing Capacity Analysis

The bearing capacity analysis involved determining the ultimate bearing capacity of the soil beneath the wall footing. The method of analysis was Terzaghi's Ultimate Bearing Capacity Theory. The formula to determine the ultimate bearing capacity is shown below.

$$Q_u = (c \times N_c) + (dF \times \gamma_s \times N_q) + \left(\frac{1}{2} \times \gamma_s \times L \times N_\gamma\right)$$

Where:

c = cohesion

dF = depth of footing

γ_s = unit weight of soil

L = Length of footing

N_c, N_q, N_γ = Meyerhoff Bearing Capacity factors

A conservative assumption of zero cohesion was used. Also, the depth factor was neglected as it is a shallow foundation. This inherently increases the factor of safety.

The ultimate bearing capacity was reduced by a factor of safety of 3, recommended by the *Canadian Foundation Engineering Manual* (CFEM), to obtain the allowable bearing capacity. This value was then compared to the maximum soil bearing pressure, which occurs at the front of the wall.

This theory is based on the assumption of even loading of the footing on the soil. This assumption is valid for the wall, as the resultant of the vertical load acts near the centerline of the footing. However, the horizontal load of the active soil force on the stem was not accounted for, which produces an angled resultant of load on the footing. See Section 11.2.9 Retaining Wall Recommendations for corrective measures.

11.2.4 Seismic Analysis

A simplified seismic analysis was used to determine the factor of safety against overturning, sliding, and bearing capacity with the addition of an equivalent static force. The equivalent static force due to a seismic event was calculated using the following formula (Seed and Whitman, 1970):

$$F_e = \frac{3}{8} \times \left(\frac{a_{max}}{g}\right) \times \gamma \times H^2$$

Where:

F_e = Equivalent static seismic force

a_{max} = Peak ground acceleration at the ground surface

g = Acceleration due to gravity

γ = Unit weight of backfill

H = Height of the wall

The peak ground acceleration used for the calculation was 0.48g (according to Maple Ridge Building Bylaw No. 6925 - 2012). The equivalent static force was assumed to act at a distance of 0.6H above the top of the footing.

Since this load is a short-term loading that may never occur during the life of the wall, a one-third increase in the passive resistance and bearing pressure was allowed for the earthquake analysis. Also, for the sliding and overturning analysis, the FS was lowered to 1.1 and 1.2, respectively (R.W. Day, 1999). For the structural reinforcement design, the earthquake force was assume to be distributed as a reverse triangular load acting on the wall stem (See Volume II Appendix A).

11.2.5 Structural Design

The structural design of the wall was to ensure the internal stability of the wall's structural members. The maximum shear and bending moment values in the stem and footing due to static earth pressures and earthquake loading were obtained. Then, appropriate member thickness and reinforcement requirements were developed. An Excel worksheet and MathCAD file were used in combination to perform the necessary calculations. The structural design of the retaining wall was performed according to CSA A23.3 and ACI code equations. The following sections further detail the footing and stem structural design methodology.

11.2.5.1 Footing Design

The wall footing reinforcement was designed to resist flexure due to the vertical load of the backfill acting on the heel and the underlying soil pressure. This was a non-conservative approach, as the underlying soil pressure reduces the bending moment and shear acting on the heel. Also, the bending moment from the stem was not assumed to transfer to the footing (allowed rotation) and the seismic force was not assumed to act on the footing.

The maximum moment and shear in the footing were obtained by treating it as a cantilever beam beyond the stem. The below figure shows the loads on the footing used to find the bending moment and shear at Location 1 (critical location).

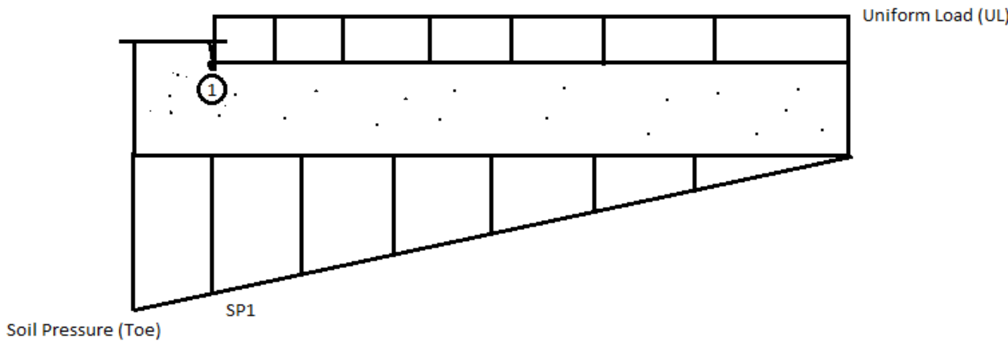


Figure 22 : Footing Loads and Critical Design Location

The uniform load was calculated by adding the weight of the backfill and self-weight of the footing, while subtracting the soil pressure at the heel. This was to simplify the soil pressure at the bottom of the footing to a triangular load, for ease of calculations. A dead load factor of 1.2 was then applied to the calculated maximum moment and shear.

A shear resistance check was performed to determine the adequacy of the preliminary footing thickness of 400 mm. The following equation was used for this check:

$$V_r = \varphi_c \times \lambda \times \beta \times \sqrt{f'_c} \times b_w \times dv \quad (CSA\ A23.3.\ Eq.\ 11.6)$$

Where:

λ = factor to account for concrete density (λ = 1 for normal-density concrete)

φ_c =concrete material resistance factor

f'_c = specified compressive strength of concrete

β = factor accounting for shear resistance of cracked concrete

β = 230/ (1000 + dv), (CSA A23.3 Cl. 11.3.6.3)

bw = 1000 mm (acting on 1 m of wall length)

dv = effective shear depth taken as greater of: 0.9d or 0.72h

The flexural resistance of the footing was designed to resist the maximum moment using 20M steel reinforcing bars. The MathCAD printout of the calculation can be viewed in Volume II Appendix A.

The procedure involved first determining the required area of steel reinforcement using the following equation (direct procedure).

$$Req'd \quad A_s = 0.0015 \times f'_c \times b \times (d - \sqrt{\left(d^2 - \left(\frac{3.85 \times M_r}{f'_c \times b}\right)\right)} \; (mm^2)$$

Where:

$f'c$ = specified compressive strength of concrete

b = 1000 mm (1 m of wall length)

d = effective depth of reinforcement

Mr = required moment of resistance (N*mm)

The effective depth was calculated using a clear cover of 75 mm as the wall is cast against and permanently exposed to earth (CSA A23.1-04 Table 17). The minimum and maximum reinforcing steel requirements were also confirmed to be satisfied. The minimum steel requirement is:

$$As_{min} = 0.2\sqrt{\frac{f'_c}{f_y}} \times bh \quad CSA\ A23.3\ Eq. 10.4$$

$$As_{min} = 0.002 \times bh \quad (For\ 25\ MPa\ concrete\ and\ Grade\ 400\ steel)$$

The maximum steel reinforcement requirement to ensure a properly reinforced section was also checked.

$$\rho = As/bd > \rho_b$$

$$Therefore,\ As_{max} = \rho_b \times b \times h\ (A23.3\ Eq.\ 10.5)$$

Where:

ρ = Reinforcement ratio

ρ_b = *Balanced reinforcement ratio*

Finally, a check was performed to ensure that the steel yields, by calculating the strain in the steel.

For the footing, the cracking control requirements governed the required spacing of the 20M reinforcing bars. The equation to calculate the crack control parameter, z, is shown below.

$$z = f_s \times \sqrt[3]{(d_c \times A)} \quad \left(\frac{N}{mm}\right) \quad (CSA\ A23.3\ Eq. 10.6)$$

Where:

f_s = stress in steel at maximum service load

d_c = distance from the extreme tension fibre to the centre of the closest longitudinal bar

A = effective tension area of concrete surrounding flexural reinforcement

The value of f_s was calculated by determining the service level of tension in the steel and the corresponding M_r, at various bar spacings. This was an iterative guess-and-check procedure to calculate a z value less than the CSA upper limit of 25,000 N/mm for exterior exposure (A23.3 Cl.10.6.1).

The development length was also checked to determine if the footing reinforcement has adequate space beneath the stem to develop its moment resisting capacity (See 11.2.5.2 Stem Design for development length check equation).

To complete the footing reinforcement design, the transverse reinforcement was specified based on the absolute minimum of shrinkage and temperature steel required for structural slabs (ACI Section 7.12). The required area of steel was calculated using the formula:

$$As_{req'd} = 0.0018 \times L \times h$$

Where:

L= footing length (transverse direction)

h = footing thickness

This transverse reinforcement is typically used to provide support for the placement of the main reinforcing bars in the footing.

11.2.5.2 Stem Design

The flexural reinforcement of the stem was design to withstand the static earth pressure behind the wall and the equivalent static seismic load of an earthquake event. These loads were converted to triangular distributions acting on the stem. The below figure shows the loads used to calculate moment and shear along the length of the stem.

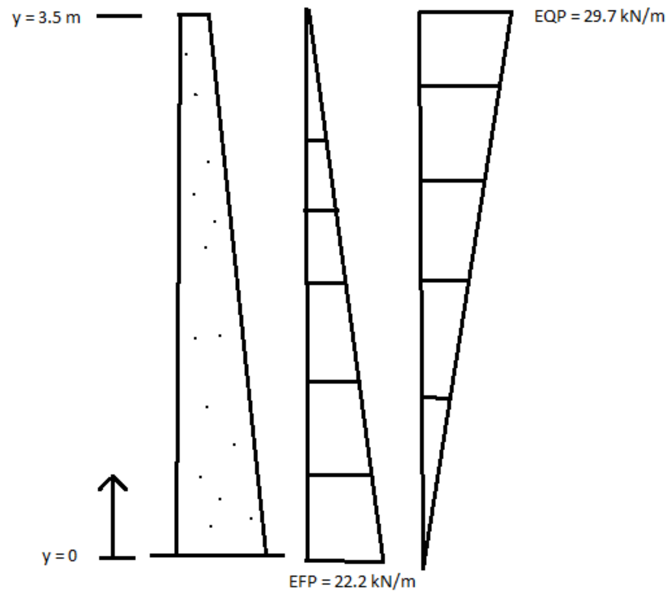


Figure 23 : Static and Seismic Loads acting on Stem

The equivalent fluid pressure (EFP) is the static active pressure of the backfill soil. The equivalent earthquake pressure (EQP) is the static seismic force (See Section 11.2.4 Seismic Analysis) distributed as a reverse triangular load. This is a good assumption because during a seismic event, the soil behind the wall will mobilize as an active wedge in the shape of the reverse triangular load.

The factored shear and moment values were then tabulated along the long of the stem at intervals of 0.1 m. Formulae from cantilever beam diagrams were used for this calculation. The highest moment and shear occur at y = 0, at the base of the stem.

The shear resistance of the concrete was checked at the base of the stem. The preliminary stem base thickness of 400 mm was sufficient as determined by the same method as described in the Footing Design section.

For flexural resistance, 2 layers of 20 M bars provide flexural strength at the base (using MathCAD sheet). However, the factored moment significantly reduces towards the top of the stem. Therefore, a cut-off point was determined for the 2 layers of 20M at y=1.3 m, where the factored moment can be resisted by 1 layer of 20M. The following graph shows the location of the theoretical cut-off point.

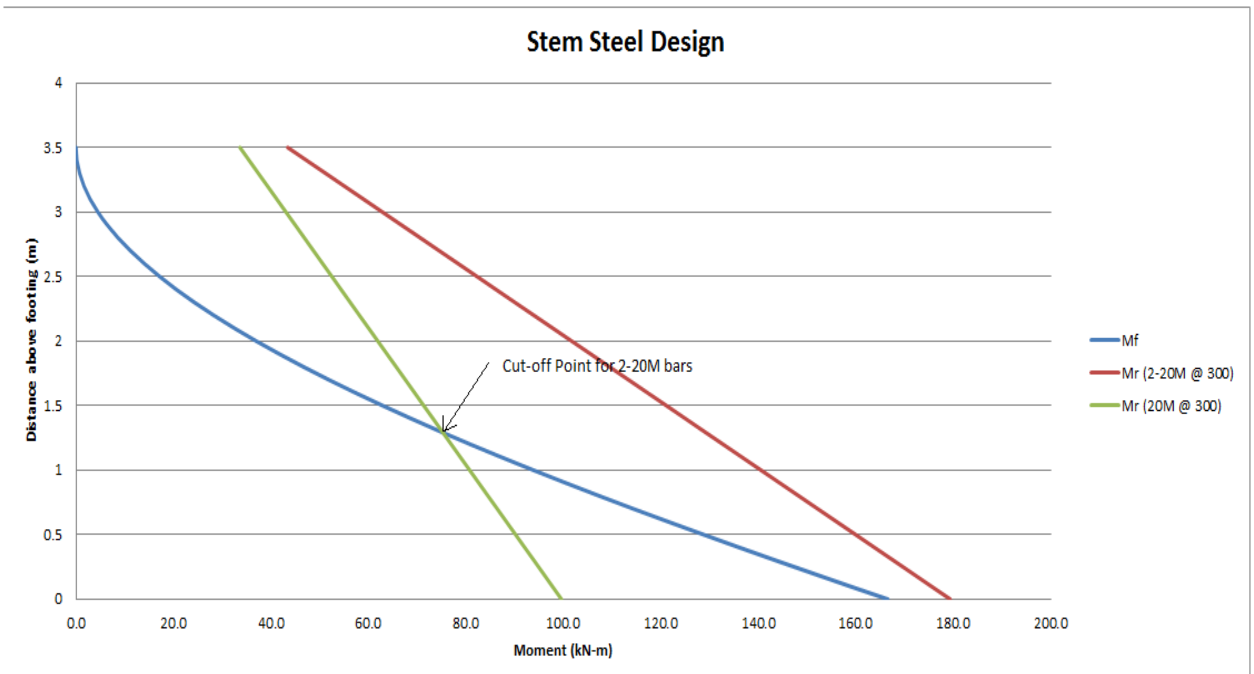


Figure 24 : Stem Steel Design Graph

The green line shows Mr (20M @ 300) which is greater than Mf up to y=1.3 m. Beyond that, 2-20M bars are required. This configuration is more economical and sustainable, as it reduces the amount of reinforcing steel along the length of the wall.

The cracking control requirements were checked at the base of the stem and the bar cut-off point using the service level force in the steel. The service level force was determined by neglecting the moment caused by the EQP.

The stem reinforcement's development length (along with footing reinforcement) was checked according to the following ACI equation.

$$L_d = K_d \times \left(\frac{\Psi_t \Psi_e \Psi_s \lambda}{\frac{c + K_{tr}}{d_b}} \right) \times K_{er} \times d_b \qquad ACI\ Equation\ 12-1$$

Where:

L_d = development length

K_d = function of various combinations of f_y and f'_c

Ψ_t = reinforcement location factor

Ψ_e = coating factor reflecting effect of epoxy

Ψ_s = reinforcement size factor

λ = light-weight aggregate concrete factor

c = smaller of cover of bar or one-half bar spacing

K_{tr} = transverse reinforcement index (K_{tr} = 0, conservative)

d_b = bar diameter

K_{er} = excess reinforcement factor

The value of (c+Ktr)/db should be checked and a value of 2.5 is the maximum permissible to be used in the equation. The excess reinforcement factor was calculated by determining the ratio between the area of steel required and the area of steel provided at the base of the stem. The development length exceeded the footing thickness, so the stem's main reinforcement must extend in to the shear key at the base of the footing.

The stem face steel was designed in accordance with the ACI Code, Section 14.3. This included horizontal and vertical reinforcement. The code specifies that walls more than 10 in. thick must have reinforcement for each direction in each face of the wall. Also, the maximum spacing should not exceed three times wall thickness or 18 in. for both horizontal and vertical bars. The rear face of the wall only required additional horizontal steel, as the main flexural reinforcement is sufficient for vertical reinforcement.

11.2.5.3 Dowel Design

Additional steel is required to provide shear-friction development at the interface between the stem and footing. It is defined as a short bar that connects two separately cast sections of concrete.

The stem and footing are cast at different times and it is common to provide shear friction at this interface. A depressed key formed by a 2 x 6 plank is common practice. This provides an additional mechanical factor of safety, but is questionable, as considerable slip is required to develop the key for lateral force transfer.

For the dowel design, the shear-friction design method of ACI Code Section 11.7 was used to design for the transfer of horizontal force between the stem and footing. This approach assumes the force is transferred through friction that develops on the contact surface between the two elements. The magnitude of force transferred depends upon the surface of the interface as well as the amount of shear-friction reinforcing crossing the surface. The following equation was used to determine the nominal shear strength of the dowels.

$$V_n = A_{vf} \times f_y \times \mu \quad (ACI\ Eq. 11 - 25)$$

Where:

A_{vf} = area of shear-friction reinforcement

μ = coefficient of friction in accordance with ACI Code, Section 11.7.4.3

The coefficient of friction can be taken as 1.0 for normal-weight concrete placed against hardened concrete. The hardened concrete should be intentionally roughened to an amplitude of ¼ in. which can be accomplished by raking of fresh concrete.

The shear force applied at the cracked plane, Vu, is equal to the maximum shear force at the base of the stem. The following equation relates V_u to V_n.

$$Vu = \varphi \times V_n$$

Solving for A_{vf} yields the formula.

$$Req'd \quad A_{vf} = \frac{V_u}{\varphi A_{vf} f_y \mu}$$

This equation allowed us to calculate the required area of dowel reinforcement per meter. For the wall, 10 M dowels at each face were deemed sufficient.

The development length of the dowels must also be checked, as they are tension members. The dowels extend equally into both the stem and footing.

11.2.6 Retaining Wall Drainage

The retaining wall has a back drainage system to prevent the buildup of excess groundwater and subsequent pore water pressure. Imported clean granular material (no silt or clay) is the recommended backfill material for the wall. This is due to its performance in the following areas:

- Predictable behavior
- Drainage system
- Frost action

The imported clean granular has predictable behavior in terms of the soil pressure and no expansive soil-related pressures. Also, it is highly permeable which maximizes the efficiency of the drainage system at the heel of the wall. This will also protect the wall from frost action as it drains effectively.

To drain the backfill effectively, weep holes are typically provided in retaining structures to prevent the backfill soils from becoming saturated. The ideal performance of weep holes is to provide free flow while preventing clogging or loss of backfill through the holes. Prevention of clogging requires a graded filter with layers of coarser grain size from the backfill soil to the gravel at the weep hole entrance. Also, filter cloths can be used to encase the gravel at the weep hole entrance to prevent passage of the backfill soil. This is the optimum strategy and will be used for our retaining wall. The image below shows the back drainage system of the wall.

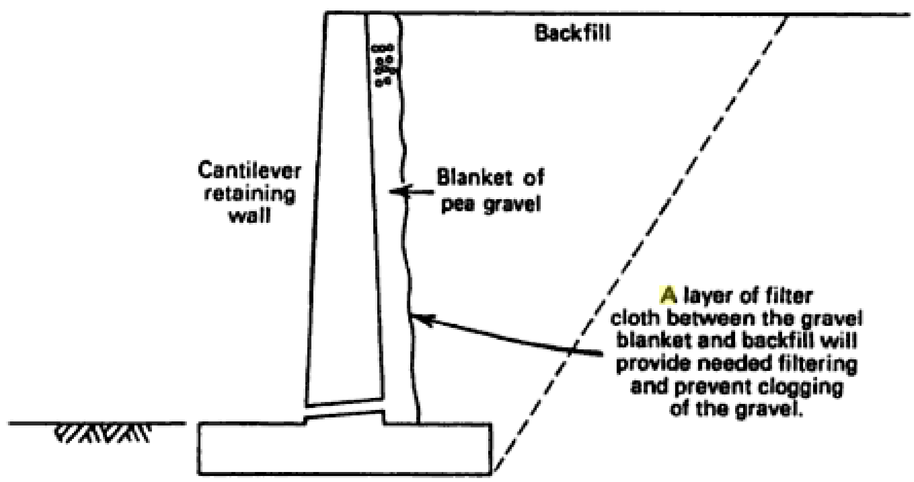


Figure 25 : Retaining Wall Drainage System (Ahlvin, 1988)

In this type of system, a layer of gravel is placed along the back of the wall, and separated from the backfill by a filter cloth. This type of system is ideal for the subdivision site as heavy rains and lawn watering are expected. For our retaining wall, we will use a prefabricated drainage membrane which acts in the same method as the gravel blanket and filter cloth. The Enkadrain 3000R Series is the specified membrane, which is a geocomposite drain designed for foundation walls. A photograph of a typical Enkadrain membrane is shown below.

Figure 26 : Enkadrain Subsurface Drainage Composite (Colbond, 2011)

The 3000R Series are designed to relieve hydrostatic pressures acting against below grade structures. This Enkadrain membrane exceeds 40% post-industrial recycled content and can contribute up to 2 LEED points when used with other recycled content building materials.

The spacing and size of the weep holes in the base of the wall were based upon typical industry practices. The weep hole spacing is based upon the principle of uniform drainage of the wall. For the wall, 4 in. holes will be spaced at 6 ft. spacing (maximum of 10 ft. recommended)

Without adequate drainage, the wall system will be exposed to greater lateral pressures than expected and may experience excess displacements.

11.2.7 SAP Model

The retaining wall was modeled using the structural analysis program, SAP 2000 version 17. The model of the wall provides verification of the moment and shear values provided by the Excel and hand calculations. Also, it provides approximate deflection values due to dead and seismic loading.

The below figure shows the wall stem and footing modeled as a 3D area model.

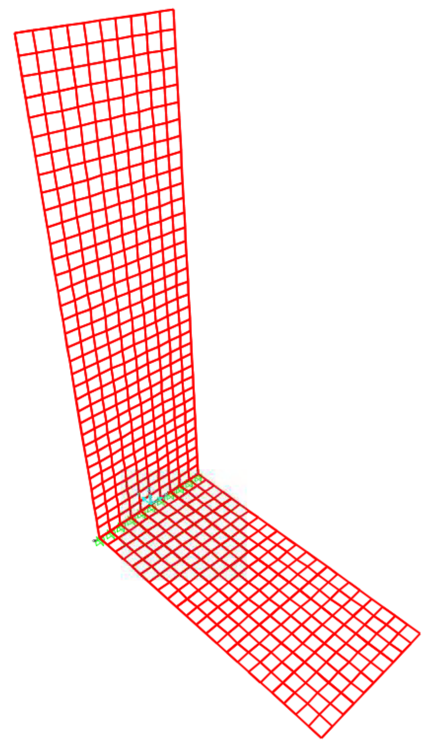


Figure 27 : 3D Model of Retaining Wall using Area Sections

The area blocks are 0.1 m x 0.1 m and defined as concrete shell sections. The shell sections are 400 mm thick and have typical concrete material values of:

- E=24 GPa
- Weight = 24 kN/m3

- f'c = 25 MPa

The stem is fixed to the footing at the base. Also, soil area springs were assigned to the bottom of the footing and the bottom 0.3 m of the stem. These area springs have a stiffness which is reflective of the typical value of modulus of subgrade reaction (k_s = 36,000 kN/m^3).

The load cases acting on the wall are a combined dead load and seismic load. The dead load acting on the wall is the weight of the backfill on top of the footing and the triangular active soil pressure on the stem. These loads were applied as area pressures on the shell sections. For simplicity, the triangular pressure was converted to varying levels of pressure at 0.25 m increments as it reduces down the stem height.

The seismic load was applied to the wall through the seismic load case. This was a response spectrum load case type and uses a standard – acceleration loading. A damping ratio of 0.1 was used to reflect the damping efficiency of a structure in soil. The peak ground acceleration, PGA =0.48g of Maple Ridge was used, as well as the NBCC 2010 Langley (close to Maple Ridge) values for various fundamental periods. The ground acceleration will cause inertial forces due to the self-weight of the structure, as well as the weight of the active wedge of soil behind it.

To model the weight of the active wedge during an EQ event, the following equation was used.

$$Wactivewedge = \frac{1}{2}\left(k_a^{\frac{1}{2}}\right)(H^2 \times \gamma_b)$$

Where:

K_a = active earth pressure coefficient

H= Height of stem

γ_b = unit weight of backfill

This soil weight was assumed to act at 2/3 H from the bottom of the stem. It was assumed to be a joint mass at the center of the wall model at this location. However, it was also distributed as area masses along the length of the stem afterwards to obtain better results.

The results of the SAP model analysis can be seen in the Results and Recommendations section compared to the Excel calculations.

11.2.8 Retaining Wall Results

The results of the retaining wall design are the following:

- selected wall type
- sizing of wall members
- reinforcement detailing
- drainage specifications
- SAP model

The final designed product can be seen in Volume III.

An L-shaped concrete cantilever retaining wall was selected with a stem height of 3.5 m, tapered from 0.4 m at its base to 0.2 m at the top. The footing length is 2.4 m with a thickness of 0.4 m. A shear key was designed beneath the footing with a depth and thickness of 0.2 m.

The main flexural reinforcement was designed for both the footing and stem. The footing reinforcement is 20M @ 225 spacing. The stem reinforcement is 2-20M @ 300 spacing up to 1.3 m above the base of the stem. Then, the reinforcement is cut-off for one of the 20M bars due to the reduced moment above 1.3 m. The dowel reinforcement at the shear-friction interface between the footing and stem is 10M dowels @ 400 spacing at each face.

The drainage of the wall is provided through clean, granular backfill placed behind the wall. Groundwater passes easily through this material to the back of the wall, where an Enkadrain 3615R membrane or equivalent is to be placed. The membrane prevents migration of fines from the backfill material to the weep holes. Free flow of water pass through the membrane to the 4 in. diameter weep holes spaced at 6 ft. o.c., to the other side of the wall.

The SAP results for the retaining wall model compared to hand calculation results are tabulated below.

Table 10 : SAP Force and Deflection Results Comparison

Forces/Deflections	SAP Results (Base of Stem)	Hand Calculations (Base of Stem)	Percent Difference (%)
Dead Load Vf (kN)	110	38.8	N/A
Seismic Load Vf (kN)	357	52.0	N/A
Total Load Vf (kN)	467	90.8	N/A
Dead Load Mf(kN-m)	41.5	45.3	8.4
Seismic Load Mf(kN-m)	152.6	121.3	25.8
Total Load Mf(kN-m)	194.1	166.6	16.5
Total Deflection (mm) - Top of Wall	10.8	N/A	N/A

The main concerning issue from these results is the very high difference in calculated stem shear vs. the SAP results for stem shear. The SAP results are much higher, which may be due to a modeling error. Shear failure is undesirable and occurs quickly, so these results should be closely examined. The SAP moment results are relatively close to the hand calculations, which verify the validity of the SAP model. The total deflection of 10.8 mm at the top of the stem is smaller than expected due to a seismic event.

11.2.9 Recommendations

The recommendations for the retaining wall design include further design analysis, investigation of other design options, and aesthetic considerations.

Further analysis of the design should be performed to determine the adequacy of the retaining wall as an earth retaining structure. The bearing capacity analysis did not include the effect of inclined loading on the ground and eccentric loading. This could impact the required length of footing and the footing embedment depth. However, due to time constraints this check could not be performed, as it would impact the whole completed design.

The battered (sloped) stem design is optimum for cost savings in total volume of concrete required. It also provides a slope for the back drainage system to direct the water to the weep holes. However, a uniform thickness of stem would improve cost savings in formwork and constructability. A cost savings analysis is recommended to determine the feasibility of using a uniform stem over a battered stem.

The investigation of an alternative design, a mechanically stabilized earth wall, as a suitable structure could also be performed. A preliminary MSE wall design to determine the cost savings due to materials and constructability is recommended.

Aesthetic considerations are recommended to improve the quality and durability of the proposed retaining wall. The retaining wall could be finished with veneer, which is costly but can add to the appeal of the properties. Alternatively, an integrally colored concrete mix can enhance the look of the wall, at a reduced cost. These aesthetic considerations will improve the subdivision properties, as a grey concrete wall may seem dull to potential homebuyers.

11.3 Slope Stability Analysis

The slope stability analysis was performed to determine the probability of slope failure at the Kanaka Creek area near our subdivision site. This is the critical area of concern within our subdivision, as it has the highest gradient and there is a proposed detention pond to be located at the top of the slope. The primary analysis tool was GeoStudio 2012 Student version. GeoStudio 2012 was used to obtain the following results:

- Slope Stability Factor of Safety
- Pond Seepage Rate

The slope stability analysis results were obtained by combining Sigma/W results with Slope/W results. The seepage analysis results were calculated through Seep/W. Although interrelated, the seepage analysis results could not be combined with Sigma/W or Slope/W to provide more accurate slope stability analysis results. The rest of this section will describe the methods used to obtain our results.

11.3.1 Slope Stability Model

To run a slope stability analysis, a 2D GeoStudio model was created to simulate the slope conditions. First, the geometry of the slope was obtained from our existing ground contour map of the creek area. The flat area at the top of the slope extends ten meters beyond the edge of the slope. The gradient of the slope was approximated as 31%. Also, the water depth in the creek was approximated as 2 m.

Based on geological maps of the area, we assumed the soil to be sandy/clay loam with organic sediments. The soil parameters of the creek area are shown in the table below.

Table 11: Assumed Soil Parameters for Kanaka Creek

Friction Angle:	33 degrees
Cohesion:	30 kPa
Unit Weight:	15 kN/m^3
Modulus of Elasticity:	10,000 kPa
Poisson’s Ratio:	0.3
Hydraulic Conductivity:	1 x 10^{-5} cm/sec.

These soil parameters are based upon common values for sandy loam soils. Further geotechnical investigation is required for verification. The cohesion value was assumed as 30 kPa to account for the effect of mechanical cohesion due to vegetation.

To begin the slope stability analysis, Sigma/W was used to include the effect of the pond water load at the top of the slope. The load applied at the top corresponds to a water depth of 2 m in the pond. The following figure shows the load applied with the exaggerated soil deformation results.

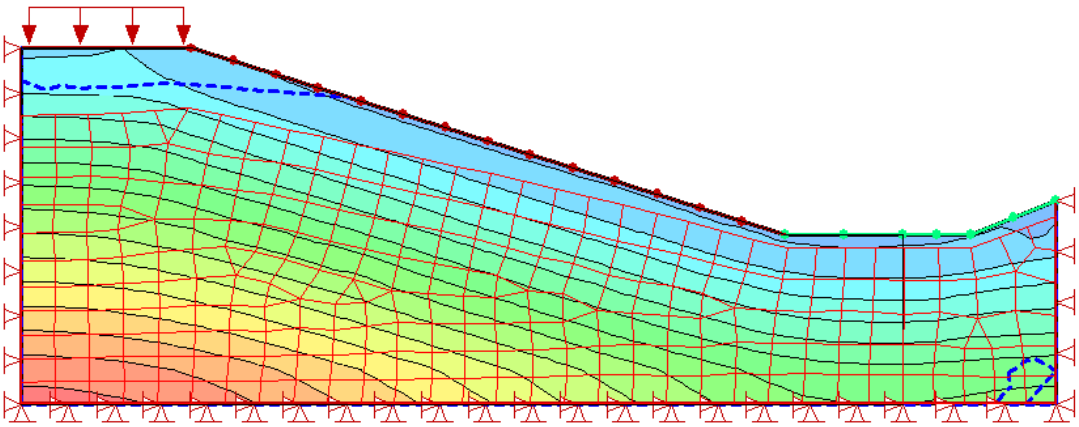


Figure 28 : Load Applied to Top of Slope

The boundaries of the model were set to various fixed configurations to limit the deformation. The load applied is 19.6 kN/m^2 over a horizontal distance of 10 m. Also, the entire model was set to one material: Loam, with the soil parameters in the previous table.

Once the Sigma/W model was set, a slope stability analysis was performed with worst-case and best-case groundwater table location. The best-case would be a nearly horizontal water table at the assumed creek water level. This configuration is shown in the figure below.

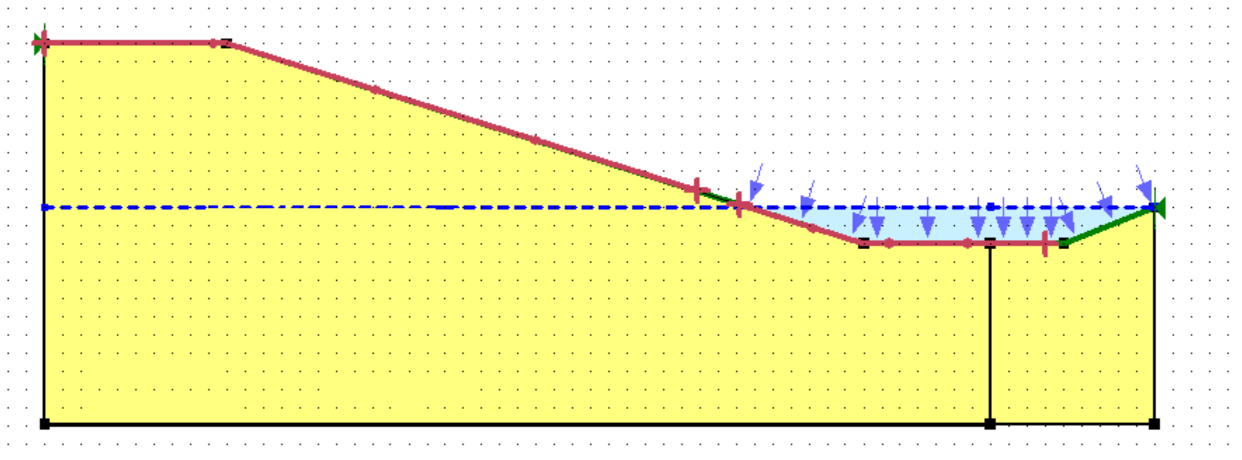


Figure 29 : Best-Case Water Table Level

The factor of safety of the slope with this water level is an upper-bound for the actual factor of safety. To develop a lower-bound value, a worst-case water level was assumed. The worst-case configuration is shown in the image below.

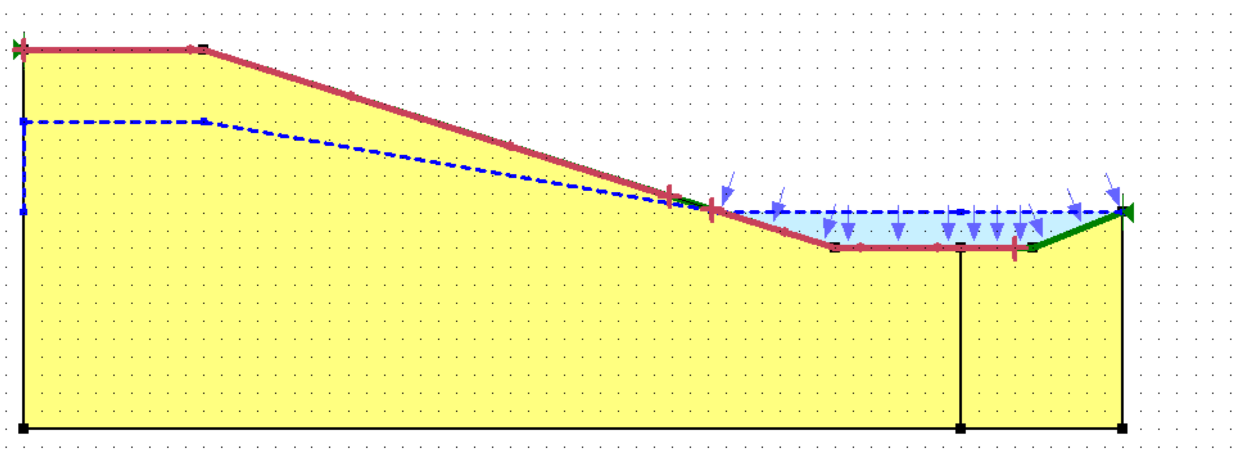


Figure 30 : Worst-Case Water Table Level

The worst-case water table would occur due to heavy rains or excess seepage from the pond. It would be drawn down to the creek water elevation as it goes further down the slope. The slope factor of safety is smaller as it decreases cohesion in the soil and increases weight and pore water pressure within the soil media.

11.3.2 Seepage Model

The seepage model provides an initial estimate of the seepage rate of the water from the pond to the groundwater table and to the surface of the slope. To produce this model, two soil layers were assumed, a soil layer directly below the pond with a hydraulic conductivity of 1 x 10^{-6} cm/s (as recommended by City of Surrey guidelines) and the loam layer throughout the rest of the model. The figure below shows the seepage model with its two layers of varying hydraulic conductivity.

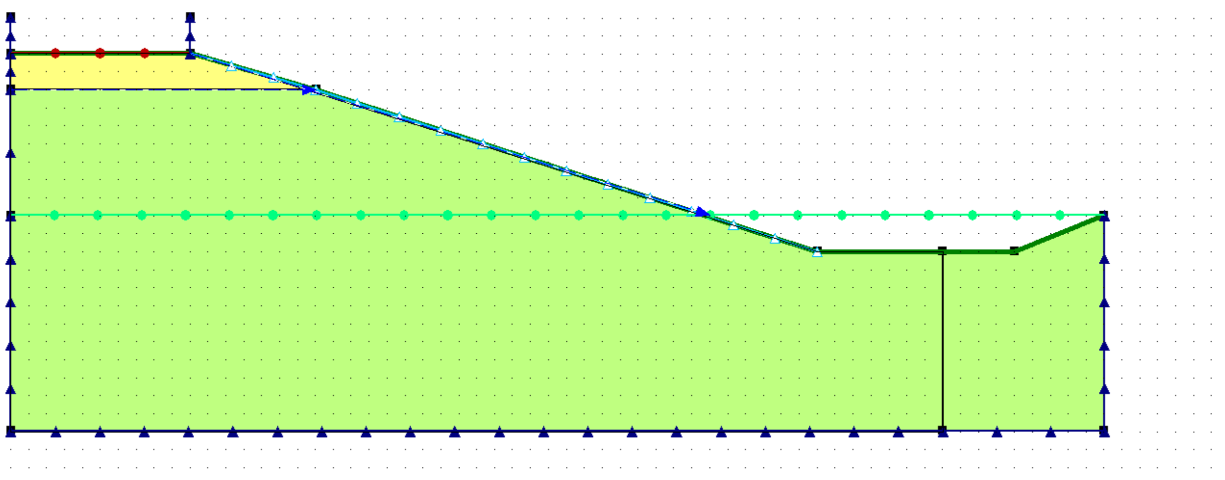


Figure 31 : Seepage Model

The seepage model includes flux lines at the interface of the two soils and at the slope surface. The flux lines provide the seepage results which are discussed further in this section's Results and Recommendations.

11.3.3 Slope Stability Analysis Results and Recommendations

The results of the slope stability analysis are values for the factor of safety for the best and worst-case of water table level in the slope. The FS results are 2.80 for the best case is and 2.31 for the worst case. The recommended slope FS for a slope where low probability of life loss due to a failure is 1.2 (Foundation Engineering Handbook). So, the slope FS is well above the recommended value based upon the assumptions made. Further slope stability techniques are not recommended at this time.

The seepage analysis results provide an estimation of the water seepage through the slope. The rate at which water seeps into the slope is critical and can reduce the FS of the slope significantly. Over an area of approximately 10 m^2, the seepage through the pond to the underlying loam is about $1.22 * 10^{-3}$ L/s. This corresponds to a seepage velocity of about $1.22 * 10^{-5}$ m/s, which is fairly reasonable and won't cause excess erosion. The seepage through the face of the slope is even smaller at 1.02×10^{-5} L/s over a 1 m section of the entire slope surface. These values are very small and could be absorbed by the vegetation easily. The below image shows seepage analysis results with flux labels.

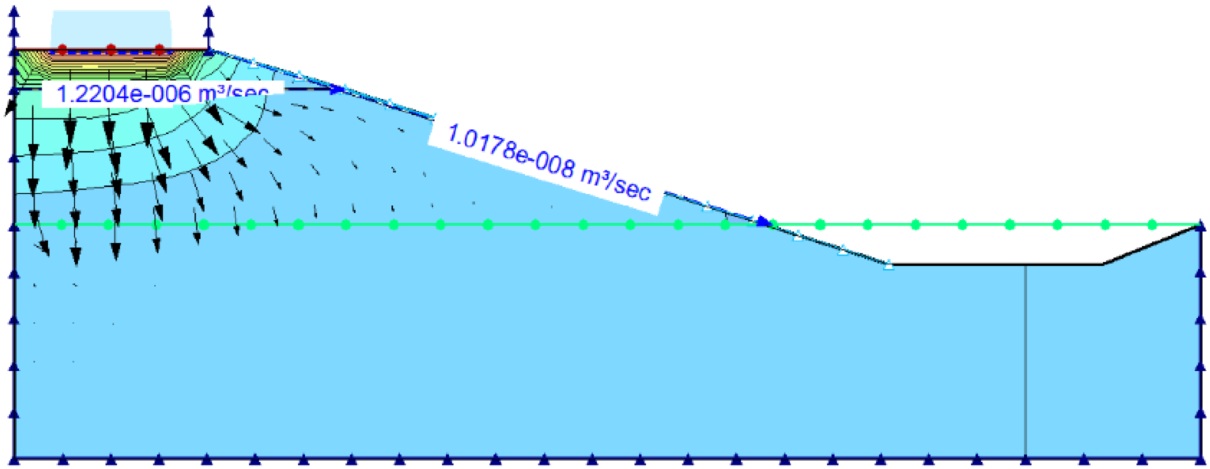


Figure 32 : Slope Seepage Results

The seepage from the pond is assumed to affect the slope only from a distance of 10 m away. A further site condition investigation should be performed upon construction of the pond, to assess the slope conditions and seepage rates. Also, the possibility of a landslide due to an earthquake should be investigated, as earthquakes are frequently responsible for most slope failures.

12.0 CONSTRUCTION COST ESTIMATE

When designing a subdivision, one of the largest constraints is the cost of construction. Every cubic meter of earth moved, and every manhole installed contributes to the overall cost of the project. The overall goal of the developer is to maximize the amount of profit gained from engaging in this type of venture.

All of the infrastructure including roadways, and utilities are to be turned over to the City of Maple Ridge one year after construction has been completed. This is the warranty period to ensure that construction was completed correctly, and gives adequate time to detect any faults or errors.

12.1 Cost Estimate

From the use of both RS Means, and estimates from local contractors, our group was able to determine an anticipated cost for the construction of the new subdivision. These costs were broken down into unit prices for various activities that take place throughout the project.

Our estimate covers costs up to the capping of services up to the property line of each individual lot. The subdivision at this point will have all roadworks and utilities in place, and the large retaining wall beginning on the west end of the site.

The builder can take responsibility for construction of the home at this point, as well as construction costs associated with the structure. The builder is also responsible for constructing the retaining wall(s) in the lots on the east side of the subdivision, as they are relatively small and easily constructible.

After conducting a cost estimate for this site, it was determined that it would cost about $4.12 million to construct the subdivision. For a breakdown of all the costs, please see Volume II Appendix J.

13.0 ENVIRONMENTAL CONSIDERATIONS

From case studies of previous construction projects, the poor management of soils onsite often results in significant pollution of the stormwater that enters into the municipality's drainage system. The eroded soils that enter into the drainage system can cause extensive damage to natural habitats and considerable maintenance issues.

Although erosion is a natural occurrence, the single most significant source of soil pollution in water courses is from construction sites in urban environments. The release of the soil into the drainage system has impacts on the main conveyance system and the receiving environment such as creeks and streams. During rain events, the accumulated sediment impedes the drainage systems capacity to carry storm water. As a result, public and private property become in danger of damages. As well, there were additional costs to the municipality for the removal of the sediments from pipes, catch basins, and drainage control structures.

Once the sediments reaches the receiving environment, it has extensive impacts on the aquatic organisms that inhabit the streams. The soils are able to carry other particles, such as oils, hydrocarbons, heavy metals, and pesticides into the streams. In addition, this results in higher mortality rates for animals, and lower growth rates for plant life. Salmon eggs are especially vulnerable during the spawning months from autumn to spring.

13.1 Creek Protection

The most precious natural resource in the vicinity our subdivision is the Kanaka Creek. It is home to several different species of wildlife and spawning grounds for salmon. The name comes from the term Kanakas, given to Hawaiian labourers who settled near the area in 1880 (Metro Vancouver, 2015). There are several parks in the nearby area around the creek, and it is a major tributary of the Fraser River. As required by the City of Maple Ridge, 15 m of buffer zone is required from the edge of the water line to the construction area. The image below shows a portion of the Kanaka Creek.

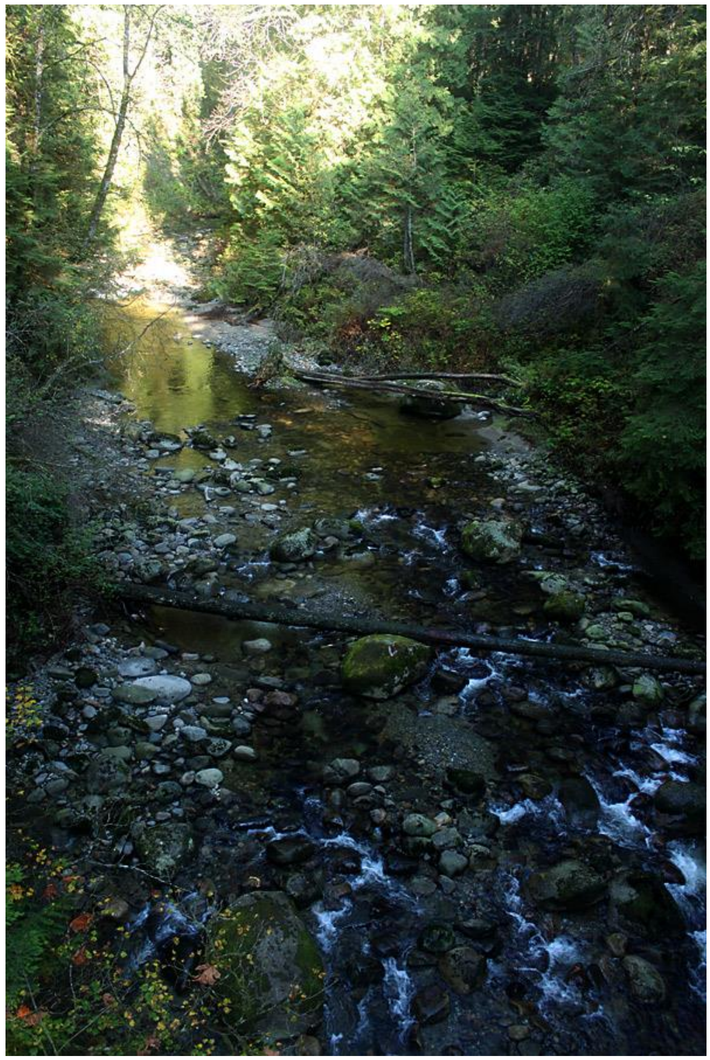


Figure 33 : Portion of Kanaka Creek (Vancouver Trails, 2015)

The Kanaka Creek extends eastward up to 256 Street in the Maple Ridge area. From there it receives flows from many other tributaries. It is also a regional park that many families visit throughout the year.

13.2 Erosion and Sediment Control

For development projects greater than or equal to 2000 m^2, it is required by most municipalities to produce an erosion and sediment control plan (ESC). This plan establishes the framework to ensure that the mandatory standards are applied to the construction site management and planning.

Along with the use of best management practices (BMPs), the ESC is used to reduce, and if possible, eliminate the export of sediment-polluted water diverted to the stormwater network. The Maple Ridge ESC by-law specifies that a site cannot discharge water greater than 75 mg/L of Total Suspended Solids (TSS). However, if the ESC is to work properly, it needs to be appropriately designed, implemented, inspected and maintained.

The objective of the erosion and sediment control plan is to ultimately keep silts and fine particulate soils from entering the stormwater network. In addition to the ESC, the following measures are to be taken during construction:

- Control access to silt by preventing vehicle access on exposed soils
- Construct gravel access pads for vehicle access
- Provide perimeter control methods to prevent migration of soils
- Provide stormwater treatment
- Cover exposed soils with hay to prevent erosion from rainstorms
- Keep roads clean at all times
- Install silt filters in catch basins
- Cover soil stockpiles with tarp or plastic
- Keep soil and sediment off of paved surfaces

For our project, the City of Maple Ridge requires a three-stage ESC plan. This plan provides the contractor with the technical details and designs that need to be followed to ensure an environmentally sound construction working area. The stages in the ESC plan are

- Clearing, road stripping, gravelling and rough grading
- Utility and roadworks installation
- Final grading stage through to substantial completion

Each of these stages plays a crucial role in the containment of sediments within the construction site. The detailed ESC plan can be seen in Volume III.

13.2.1 Clearing, road stripping, gravelling and rough grading

The first stage, consisting of the initial tree falling and stump removal, incorporates the removal of the topsoil from the road areas to allow for equipment transport. Before trucks can leave the site with material, they must go through a wheel wash to ensure they do not track soil onto nearby roadways. As well, stormwater sediment ponds are placed onsite to settle the sediments before the water can be released to the creek. Rough grading also takes place in this stage, allowing a buffer zone between the required elevations, as well as interceptor ditches that divert to the sediment tanks. After this activity, hay needs to be spread over each graded section. Another criteria is that existing catch basins be installed with silt filters and cleaned regularly to ensure the system does not get contaminated.

13.2.1.1 Silt Filters

A relatively simple method for keeping sediment out of catch basins are the silt filters. However, the contractor must consistently monitor the amount of sediment in the filter and clean it out once it has become more than half full. Figure 13 below depicts a silt filter installed under the grate of a catch basin.

Figure 34 : Typical Silt Filter in Catch Basin (BMP Supplies, 2011)

The simple nature of the filter design allows for easy access and periodic cleaning. A similar mechanism is used for placement in lawn basins.

13.2.1.2 Silt Fence

Another simple measure for containing sediments within the site is the silt fence. It consists of a geotextile filter fabric attached to lumber posts that are hammered into the existing ground. To complete the installation, native backfill is used to secure the fabric in the ground, ensuring that water cannot escape through the bottom of the fabric. Every 3 m of fence can service approximately 100 m^2. Therefore, in the areas leading down from slopes, the silt fence will be used in rows.

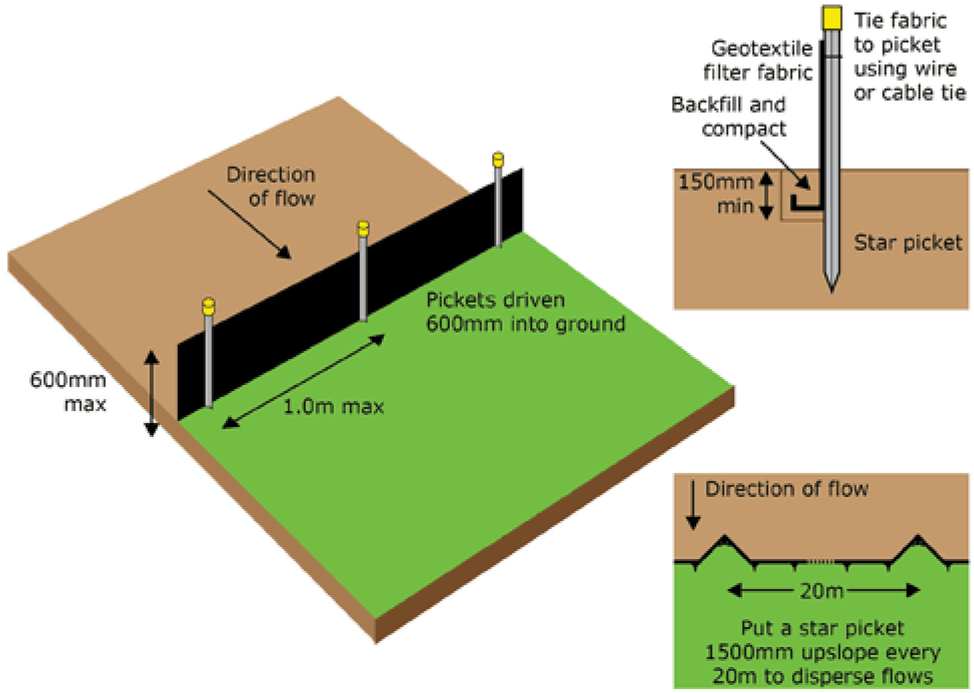


Figure 35 : Typical Fence Installation (Melbourne Water, 2015)

Figure 14 above shows how a standard silt fence is to be installed. The fence and picket are commonly sold as a single entity and are relatively cost-effective.

13.2.2 Utility and roadworks installation

Once the initial grading is completed, the roadworks and utilities are to be installed at their appropriate elevations. Also, as the new catch/lawn basins are installed, silt control filters are to be placed inside them until construction is completed. During this stage, a single or series of sediment control ponds are dug out to contain the release of excess soils. When the utility services are complete, the outlets from the pond(s) are to be capped off or removed completely before moving on the final phase.

13.2.3 Final Grading Stage Through to Substantial Completion

In the final stage of the ESC, hay is to be hand laid over all exposed areas, and the installation of temporary slope sediment barriers. The common structure used for slope barriers are straw wattles, which is straw wrapped in a containing mesh that traps sediment as water flows through it. The figure below shows how a typical straw wattle should be installed.

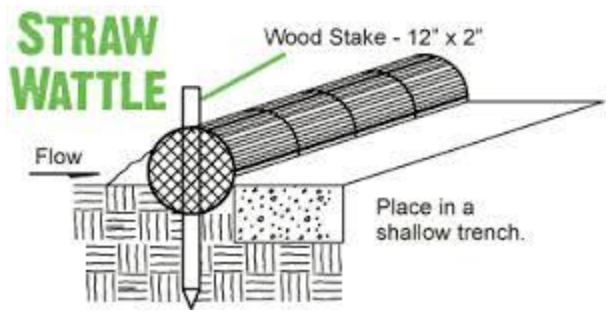


Figure 36: Straw Wattle (City of Lincoln, 2015)

As contaminated water flows through the wattle, the flow is slowed down through the loss of energy. These items may need to be periodically replaced if too much sediment gets into the fibres and prevents water flow.

Furthermore, maintenance of the structures and protection measures put in place in stage 1 of the ESC plan are to be in operating condition at all times. After our designed scope of work is completed, the home builder is to follow similar BMPs during the construction of residential homes on the site.

13.3 Recycled Materials

To reduce the overall environmental impact in construction, supplementary cementing materials (SCMs) are to be used in the design of the concrete for the retaining wall and pump station. The concrete mix should incorporate the use of

- fly ash
- slag content
- silica fume

By using these SCMs, the use of freshly produced cement is reduced. Freshly made cement has a large environmental footprint, as the heating process required to produce clinkers is an energy-intensive process. In the concrete mix, the use of recycled aggregates will also be specified to reduce the amount of quarried rock used.

Furthermore, the drainage membrane used for the retaining wall will also consist of recycled materials. The membrane consists of 40% post-industrial recycled content, while providing greater performance than a standard filter cloth would. Dependant on its application, the use of this membrane can also contribute to LEED certification.

14.0 CONCLUSION

This subdivision design project resulted in an overall design which was satisfactory and incorporates future development. The rest of this section details concluding remarks and results about each section of the report.

The lot layout design challenged us to come up with an innovative solution to provide convenient access to residents and potential for future development. Two options were produced and Option 2 was the selection we made. The selected lot layout was then enhanced to meet municipal requirements. The lot grading of the site was an important aspect which used AutoCAD Civil 3D to come up with suitable lot gradients. The goal of this task was to reduce earthworks and provide adequate overland lot drainage.

The sanitary system involved design of a sanitary pipe network and a pump station to overcome the topographic conditions of the site. The sanitary pipe network handles the incoming flows from each lot using 200 mm diameter PVC pipes. All flows were directed to a pump station with a custom fiberglass wet well housing two Flygt pump units in duty/standby configuration. The pump units were selected based upon the system requirements matching the pump curve at the highest efficiency. The 200 mm diameter force main was designed to maximize cost savings due to pump efficiency.

The stormwater management plan addresses the issues relating to increased runoff generated by new developments. Best Management Practices are used to help deal with the increased runoff generated, as well as reducing the effects of erosion, and providing settlement and contaminant control. A detention pond and an erosion and settlement control plan were implemented. The storm sewers were designed as such to direct runoff to the detention pond so that the subdivision could mimic pre-development site conditions.

The road design involved developing a horizontal and vertical road layout to minimize earthworks and to provide a safe driving experience. TAC geometric guides and MMCD guidelines were followed for the road geometry and the intersection design.

Geotechnical considerations of the subdivision project included a preliminary geotechnical report, retaining wall design, and a slope stability analysis. The preliminary geotechnical report assumed soil site conditions and design parameters. The 3.5 m high concrete cantilever retaining wall was designed to resist active soil pressures and seismic loading. The wall's members were designed to resist shear and flexural failure according to CSA A23.3 and ACI Code requirements. A SAP analysis was included to verify the loads acting on the wall. The slope stability analysis provided a factor of safety against slope failure for an area near Kanaka Creek. Assumptions regarding soil properties and water table levels were made, so the results need to be verified to provide further confidence.

A cost estimate was produced through RS Means and contractor data to get an idea of the construction cost of the proposed subdivision. A cost of $4.12 million is required for the development of the site.

Environmental considerations for the site included development of an erosion and sediment control (ESC) plan. The plan is divided into three stages of the subdivision development. It prevents excess sediment deposition to the creek through implementing silt control measures such as silt fences and sediment control ponds. Also, we recommended the use of recycled materials in the concrete structures within the subdivision.

15.0 EPILOGUE

Although in terms of number of credits the capstone design project is worth more than most courses, it was the one that felt the least like a course. Generally, when taking a class, the student expects a certain level of structure and to be consistently within a classroom setting. This course was the complete opposite, besides the weekly formal presentations and meetings, the onus was all on the group. There were times when work from other classes became a priority for few days, but in the back our minds, we were always left to thinking when we were going to get back to working on the capstone project. But when we come to think of it, this project gave us the closest thing we were going to get that simulates the practices of the "real world". In the working environment, there are deadlines that need to be met, and help along the way, but the motivation needs to come from within to achieve any true success.

As engineering students, we are constantly required to solve math and physics problems on a regular basis. However, without a clear, concise report, or drawings that are easy to interpret, the theory and logic behind the reasoning and rationale has no purpose. When in the working world, the only way to get the message across to the client is by way of these documents. At the end of the day, their decision doesn't rest upon whether the math is correct, but on the recommendations that you make. Like the consumer within us all, we don't care so much how one came to a decision, but more about how much it costs, or what it looks like.

This project would not be possible without the coordination between team members. In our particular project, there were several areas which required a team member to finish some tasks before another member could even get started on theirs. By co-ordinating effectively and managing our time according to others' schedules, we were able to complete our tasks efficiently. As motivation, we wanted to end our BCIT education with a successful project and leave something great behind.

REFERENCES

Ahlvin, R. G., & Smoots, V. A. (1988). *Construction Guide for Soils and Foundations.* Canada: Wiley-Interscience.

Autodesk. (2014). Storm and Sanitary Analysis.

BC MoT. (2007, June). Supplement to TAC Geometric Design Guide. Coquitlam, BC, Canada.

Bearing Capacity Technical Guidance. (n.d.). Retrieved February 2015, from GEOTECHNICAL INFO .COM: http://www.geotechnicalinfo.com/bearing_capacity_technical_guidance.html

BMP Supplies. (2015). *Silt Sacs*. Retrieved from http://www.bmpsupplies.com/products/catch-basin/silt-sacs/

Brzev, S., & Pao, J. (2012). *Reinforced Concrete Design: A Practical Approach Second Edition.* Upper Saddle River: Pearson Learning Solutions.

Caduto, D. P., Yeung, M.-c. R., & Kitch, W. A. (2008). *Geotechnical Engineering: Principles & Practices (2nd Edition).* Upper Saddle River: Prentice Hall.

City of Lincoln. (2015). *Don't Lose Ground*. Retrieved from Watershed Management: https://www.lincoln.ne.gov/city/pworks/watrshed/require/erosion/dlgbroch/

CLA-VAL. (2009). *Pressure Reducing Valve.* Retrieved February 2015, from CLA-VAL: http://stocktonvalve.com/pdfs/ClaVal/E-90-01.pdf

Colbond. (2011). Retrieved from Drainage: http://www.colbond-usa.com/drainage-building-menu.html

Day, R. W. (1999). *Geotechnical and Foundation Engineering Design and Construction.* Chicago: McGraw-Hill Companies.

Day, R. W. (2012). *Geotechnical Earthquake Engineering Handbook Second Editon.* Chicago: McGraw-Hill Companies.

District of Maple Ridge. (2012). *Building Bylaw No. 6925 - 2012.* Retrieved February 2015, from www.mapleridge.ca: http://www.mapleridge.ca/DocumentCenter/View/321

Engineering Tech Associates, P.A. (2013). *Retaining Wall Design*. Retrieved from http://www.engineeringtechpa.com/retaining-wall-design/

Epcor. (2010, March 8). *What If Everybody in Canada Flushed At Once?* Retrieved from Pat's Papers: http://www.patspapers.com/blog/item/what_if_everybody_flushed_at_once_Edmonton_water_gold_medal_hockey_game/

Fang, H.-Y. (1991). *Foundation Engineering Handbook Second Edition.* Retrieved March 2015, from google.ca: https://books.google.ca/books?id=X8hEt3l1SPQC&pg=PA380&lpg=PA380&dq=recommended+factor+of+safety+for+slope+stability&source=bl&ots=4I8hO9sKxf&sig=xYrEGQ_wJTED6tkCsWPeUam17Mg&hl=en&sa=X&ei=h1L_VPrxB8y6ogSyjoDACw&ved=0CCoQ6AEwBA#v=onepage&q=recommended%20fa

Flygt. (n.d.). *Product Selection and Technical Data*. Retrieved February 2015, from Flygt: http://www.flygt.com/en-us/Pumping/EngineeringExpertise/ProductSelection/Pages/Product%20selection%20and%20technical%20data.aspx

Geological Survey of Canada. (1980). Retrieved 2015 January, from Surficial Geology Map of New Westminster: http://ftp2.cits.rncan.gc.ca/pub/geott/ess_pubs/108/108874/gscmap-a_1484a_e_1980_mn01.pdf

Gibb, A., Kelly, H., Schueler, T., Horner, R., Simmler, J., & Knutson, J. (1999, October). Stormwater Best Management Practices Guide - Volume 1 Part I. Vancouver, BC, Canada.

Global Plastic Sheeting. (n.d.). *Enkamat & Enkadrain-: Only Drainage On Market that qualifies for Leeds Points-recycled poly drainage fabric* . Retrieved March 2015, from globalplasticsheeting.com: http://www.globalplasticsheeting.com/enkamat-enkadrain

Grundfos. (n.d.). *The Sewage Pumping Handbook.* Retrieved February 2015, from www.grundfos.com: http://www.grundfos.com/content/dam/Global%20Site/Industries%20%26%20solutions/waterutility/pdf/sewage-handbook.pdf

Jones, G. M. (2008). *Pumping Station Design: Revised 3rd Edition.* Oxford: Butterworth-Heinemann.

Limbrunner, G. F., & Aghayere, O. A. (2007). *Reinforced Concrete Design Sixth Edition.* Upper Saddle River: Pearson Education.

Master Municipal Construction Documents. (2009). *MMCD - Platinum Edition.*

Melbourne Water. (2015). *Sediment management measures*. Retrieved from http://www.melbournewater.com.au/Planning-and-building/Standards-and-specifications/site-management/Stormwater_management_during_construction/Pages/Sediment-management-measures-6.13.2.aspx

Methods, H., & Durrans, S. R. (2003). *Stormwater Conveyance Modeling and Design* (1st ed.). (D. Klotz, A. Strafaci, & C. Totz, Eds.) Waterbury, CT, USA: Haestad Press.

Properties of Soils. (n.d.). Retrieved March 2015, from Sefindia.org: http://www.sefindia.org/forum/files/appc_soil_properties_718.pdf

Understanding Net Positive Suction Head. (n.d.). Retrieved February 2015, from www.pumpschool.com: http://www.pumpschool.com/applications/NPSH.pdf

Vancouver Trails. (2015). *Kanaka Creek Falls*. Retrieved from Vancouver Trails: http://www.vancouvertrails.com/trails/kanaka-creek-cliff-falls/

Virginia, A. (2015). *Environment*. Retrieved March 11, 2015, from http://environment.arlingtonva.us/stormwater-watersheds/

Washington State Department of Ecology. (2012, August). Stormwater Management Manual for Western Washington. Seattle, Washington, USA.

Water Pressure Reducing Valves. (2015). Retrieved February 2015, from Watts: http://www.watts.com/pages/learnAbout/reducingValves.asp?catId=64#endconnections

Watts Industries. (2015). *Watts, the inventor of the regulator*. Retrieved from http://w4-web125.nordnet.fr/rdpgb/guid_app_rdp.htm#definition