UBC Social Ecological Economic Development Studies (SEEDS) Student Report

2011 UBC South Campus Urban Stream Restoration Project

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Executive Summary

In 2009, the University of British Columbia (UBC) committed to exploring and exemplifying sustainability in its key strategic document, Place and Promise. The University Sustainability Initiative (USI) and the SEEDS program (Social Ecological Economic Development Studies) support this commitment by using the campus as a living laboratory and involving students, faculty and staff in the exploration of important questions related to sustainability on campus.

This project builds on several generations of SEEDS research that has used the campus as a living laboratory and it examines the potential of developing an urban stream in the South Campus watershed. The project looks at restoring the natural drainage environment in keeping with the UBC Comprehensive Community Plan (UBC CCP, 2000) and the Vancouver Campus Stormwater Management Strategy (Draft Report, April 2010). Both documents identify providing sustained base flows in natural watercourses as a way to demonstrate and exemplify innovative stormwater management that also provides for public amenity and rich ecological benefits.

Establishment of year-round base flow within the urban stream is key to providing a stream capable of supporting fish and fish habitat. Base flows can be established through utilizing dry weather base flow within the existing drainage system, pumping from groundwater storage aquifers, providing municipal water or using recycling treated wastewater. Each of these sources requires field verification of rate and total available quantities to guide UBC in selecting the best option for providing base flow in the USR project.

The headwater of the USR project was identified to be adjacent to the intersection of Binning Road and Gray Avenue. Base flow delivery to the stream is based on a gravity delivery system from the existing storm drainage manhole P9D-S219 at the intersection of Binning Road and Birney Avenue. A flow control weir and pipe system will be designed to direct the required minimum base flow and limit the peak flow within the urban stream.

The available land area, mean gradient and fish habitat criteria define the design constraints of the stream channel. The stream will consist of step-pool, cascade-pool and riffle-pool morphology. Depth of excavation is estimated to range from 0.5 to 1.0 metres with an overall channel width ranging from 2.0 to 4.0 meters.

This study is intended to assist UBC in developing a strategy for defining the source of year-round base flow, delivery of flow to the stream and define key channel characteristics. The implication is that further development of the USR project is required to ensure the project meets the USI objectives and stormwater management practices for the UBC campus. The next step in the South Campus Urban Stream Restoration Projection is to implement the flow monitoring program recommended by this report and a water balance investigation of the Michael Smith Park pond in the summer of 2011.

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1.0 Introduction

The intent of this report is to provide technical guidance regarding the ongoing initiative to build a constructed stream channel along the east edge of the University of British Columbia (UBC) South Campus area, adjacent to Pacific Spirit Regional Park. The 2011 phase of the project has been undertaken as an undergraduate engineering course as part of the University of British Columbia SEEDS program. The project has built on previous work completed in 2010 by UBC civil engineering student Jesse Weibe, as well as in 2006 by UBC environmental engineering student Kosta Sainis.

The objectives of the south campus stream initiative have been established within the context of the university's sustainability policy, with the intention of adding social, aesthetic and ecological value to the south campus area, as well as in keeping with the university's policy of “campus as a living lab”. The primary focus of this year's work has been with respect to the environmental and hydrotechnical issues related to the stream's design and construction. The intent has been to design the stream to be suitable for fish habitat, with cutthroat trout as the intended species.

This report contains information and technical guidance with respect to the following main topics:

- Options for water sources to supply water to the stream.
- Hydraulic considerations for the flow diversion structure.
- Recommendations for stream profile and cross section geometry
- Recommendations regarding the collection of additional information required to continue to the next phase of the project.

2.0 Background

The previous work completed in 2010 included a detailed topographic survey of the proposed stream corridor and identification of a preliminary design alignment and profile for the stream. The 2010 report concluded that construction of a stream channel was feasible along the east boundary of the UBC campus, based on an approximate channel width of two meters and a bank full depth of one half meter. The report identified determination of stream water sources and quantification of their relative flow contributions as an important next step for the project. This newest contribution to the South Campus Urban Stream Rehabilitation Project was built on recommendations from these previous reports and investigated the technical feasibility of a constructed stream in the UBC South Campus area.

The proposed headwater of the stream is near the south edge of the Wesbrook Village neighborhood, southeast of the intersection of Gray Avenue and Binning Road. The alignment of the stream is proposed to roughly parallel the existing large diameter storm sewer, which flows south to its point of discharge to a drainage ditch along the north side of Southwest Marine Drive. Total length of the proposed new stream is approximately 1,300 meters, with an overall elevation drop of roughly 30 meters.

Figure 1 - Area of interest for the proposed UBC South Campus stream project

3.0 Stream Flow Sources and Water Budget

Providing a stream with year-round flow increases the fish habitat value of the stream and creates a perennial social amenity. Urban streams provide communities with access to natural functioning watercourses, which improve the aesthetic and ecological value of the neighborhood; however, streams without sufficient year-round base flow could compromise the capacity for aquatic habitat. Low flow rates in the summer months result in increased water temperatures, which reduce pool volumes, lower dissolved oxygen levels, and may result in the stream bed drying up altogether (Tacconga and Munroe 1995).

In 2000, the University of British Columbia published the Comprehensive Community Plan, which identified providing sustained base flows in natural watercourses as one of the key concepts regarding stormwater management on campus (UBC CCP 2000). Although the proposed stream is not a natural watercourse by definition, the criteria used in the design of the stream is based on natural functioning watercourses and is an attempt to provide the same level of ecological value as a natural system.

Several key pieces of information are required to determine the minimum base flow that is required for year-round flow in the proposed stream. This information includes identifying the stream water supply sources during the dry months, determining the range of flows available from these sources, and quantifying the infiltration and evaporation losses from the stream reach.

This section of the report is intended to guide future work required to complete the design of an urban stream in the UBC South Campus area. It will outline the monitoring programs and field tests recommended to determine the appropriate flow rate required to maintain reasonable flows in the stream during the dry months of the year and will consider methods of mitigating infiltration losses, such as the use of channel liners.

3.1 Water Supply

In order for the proposed stream to convey a base flow year-round, a reliable source of water must be established during the dry months. After reviewing the existing stormwater network configuration and related infrastructure in the South Campus area, the following four options were identified as potential water supply sources:

1. Irrigation runoff from the Thunderbird Park playing fields and other contributing dry weather flow that is conveyed through the existing stormwater network
2. Pumped water from the two existing groundwater aquifer wells located in Michael Smith Park and Khorana Park
3. Dechlorinated municipal water
4. Treated wastewater discharged from the UBC "Pilot Wastewater Treatment Plant"

The following sections will outline recommended monitoring programs and field work which will aid in determining the range of flows which are available from these supply sources.

3.1.1 Existing Dry Weather Base Flow in Existing Storm System

Dry weather base flows in the South Campus catchment area were recorded in a previous flow monitoring program, which was carried out in 2000. This data was collected at a monitoring station on South West Marine Drive, located near the culvert that leads to Booming Grounds Creek at the south end of the catchment. Since this monitoring station is located close to the catchment outfall, the collected data does not indicate the flow path or volume of water that is being conveyed through the various reaches of the storm sewer. The likely source of the measured dry weather base flow is the excess irrigation of the Thunderbird Park playing fields north of 16th Avenue.

To quantify the available base flow which will be available to the proposed stream, a flow monitoring program during the dry season is recommended for the north end of the South Campus storm sewer system. Details of the flow monitoring program are as follows:

- Use of gauges to measure and record flow and temperature.
- Minimum monitoring duration of one dry season (June to October).

- Minimum of three gauges to be placed at critical locations in the north end of the South Campus storm sewer network (see Figure 2). These locations include:
    1. Diversion manhole located just south of West 16th Avenue and Wesbrook Mall
    2. Manhole located at the intersection of Birney Avenue and Binning Avenue
    3. Manhole located at the intersection of Gray Avenue and Binning Avenue

Figure 2. Proposed Flow Monitoring Locations

In addition, rainfall data that is collected by Dr. Andrew Black (UBC Faculty of Land and Food) from the UBC meteorological station must be obtained for the time period the storm water flow data is collected. This data should be used to investigate the relationship between precipitation patterns and measured storm discharge, and in particular, differentiate flows associated with rainfall from dry weather sources.

The flow monitoring program outlined above will provide a measure of the base flow available to the proposed stream from the existing stormwater system and will indicate whether base flow augmentation will be required. Flow augmentation options will be discussed in the following sections and must only be considered if sufficient base flow is not available from the existing stormwater system.

3.1.2 Michael Smith Park and Khorana Park Wells

In the case that base flow from existing sources is not sufficient to provide sustained base flow in the stream, the two groundwater aquifer wells located in Michael Smith Park and Khorana Park are potential water supply sources that could be used to augment summer season low flows. The well located in Michael Smith Park is currently being used as part of an aquifer storage and recovery system, which means excess stormwater from Michael Smith Pond is being stored in the well and then is later pumped out to top up the pond in the dry season. The well in Khorana Park is not currently being used, but with the addition of a pump, it could be used to deliver groundwater to the stream. Both the Michael Smith Park and Khorana Park aquifer wells have excellent potential for being the dry weather water source for the stream based on their location and existing infrastructure; however, more information regarding the aquifers' capacity and ability to release groundwater is required.

A pump test for the Michael Smith Park system and aquifer tests for both wells are recommended to determine the performance characteristics of each well and the hydraulic properties of the aquifer. A pump test will indicate the flow that each pump is able to discharge. The principle behind the aquifer test is that a well is pumped and the effect of this pumping on the hydraulic head, which is known as drawdown, is measured inside the well itself and any other available observation wells at specific times. Data from the aquifer tests can be used to estimate the transmissivity and storativity of the well. The transmissivity refers to the ability of the aquifer to transmit water and the storativity refers to the aquifers ability to release the water (Fortin 2003). In addition, a recovery test for each well is recommended to verify the aquifer coefficients calculated in the aquifer test. A recovery test involves shutting down a pump that has been running for a period of time, and measuring the recovery of the aquifers hydraulic head.

These tests are necessary to determine the available discharge from each well and to evaluate if pumping one well will result in a drawdown effect in the other well. As mentioned above, the well in Michael Smith Park is currently being used to top up the pond in the dry season; however, the amount of water that is being recovered from the well to accomplish this is not known. More information regarding the usage of the Michael Smith Park aquifer recovery system is required to determine the contribution that the Michael Smith Park well can make to the proposed stream.

3.1.3 Base Flow Augmentation Project Examples

There are currently at least two fish bearing streams in the Lower Mainland area that are using aquifer wells to augment base flow during the summer months, with mixed success. Below are brief summaries of the base flow augmentation projects, which were implemented in both Musqueam Creek and Elgin Creek.

Musqueam Creek

The headwater of Musqueam Creek is located near West 16th Avenue in the UBC Endowment Lands and has a total watershed area of 6.7 km^2. Musqueam Creek and its tributaries support a variety of fish species including coho and chum salmon, cutthroat trout, western brook lamprey, prickly sculpin, and threespine stickleback. In summer months, extremely low flows were common, with an average low flow of 0.2 l/s (DFO 1999). In 1996, plans were made to undertake a creek enhancement project that included drilling a well and pumping water to augment base flows.

In 2010, FSCI Biological Consultants and J. Termuende Hydrological Services prepared a report for the Musqueam Creek Preservation Society investigating the hydrology and biological productivity of the stream. In the report, the Imperial Trail Well was tested to determine its usefulness as a source of base flow augmentation. The tests found that the well had a discharge capacity of 3.15 l/s and that the pump had been operating continuously between the beginning of June and the end of October. It was observed that visible flow in the creek channel was zero, due to infiltration and evapotranspiration losses, 50 meters downstream of the Imperial Trail Well. Given the results of the tests, it was noted that the well was not likely providing any benefits to the creek and recommendations were made to critically review the use of the well.

Elgin Creek

The headwater of Elgin Creek is located in the Sunnyside Acres Urban Forest, in the City of Surrey. Elgin Creek is approximately 1.5 km long and drains into the Nicomekl River. The creek was primarily an ephemeral stream, with very little to no flow during the summer months. A local stream keepers group, Residents of Elgin Saving Creeks from Urban Effects (RESCUE), had a goal to re-introduce salmon back into the creek and requested the City of Surrey to undertake a flow augmentation study to identify methods of supplying a dry season base flow to improve the fish rearing and spawning capacity of the upper reaches of the creek. Surrey completed field tests and determined that the appropriate flow rate relative to maintaining reasonable flows in the creek during the dry months of the year was approximately 7.6 L/s (City of Surrey 2006).

Surrey finalized the Elgin Creek Base Flow Augmentation Project in 2006, by choosing to install a new pump into an existing groundwater well located at the headwaters of the creek, with a projected overall cost of $150,000 and a maintenance cost of $10,000 per year. In July, 2007, the flow augmentation system became operational, providing a flow of approximately 6.3 L/s, which was observed to be sufficient to supply the creeks entire length with a sustained base flow (The Now 2007).

No further follow-up information is currently published regarding the performance of the pump, annual operation and maintenance costs, or aquatic habitat benefits that have been realized from the flow augmentation program.

3.1.4 Municipal Water

Municipal drinking water is another possible source to augment low flows in the proposed urban stream during dry periods. Water from existing mains could be supplied to the stream headwaters with a tie-in similar to a typical residential home connection. The water main pressure must be known in order to correctly size the connection pipe to provide sufficient flow to the stream.

Municipal drinking water must be dechlorinated before being discharged to the environment. The British Columbia Ministry of Environment has set the water quality criteria for chlorine exposure to freshwater aquatic life at 2 μg/L (Singleton n.d.), and drinking water supplied to UBC by Metro Vancouver typically contains between 0.5 and 1.0 mg/L chlorine (EKON Environmental Limited 1997). Due to spatial constraints of the urban stream, the most feasible option for continuous dechlorination of water is an in-line injection system. Typically these commercially available systems are comprised of a small chemical metering pump located at least 10m upstream of the discharge point to the stream. Automated dosing systems are available to feed dechlorinating agents, such as sodium thiosuphate, into the chlorinated municipal water to adequately dechlorinate before entering the stream(EKON Environmental Limited 1997).

3.1.5 Treated Wastewater

Another possible option of water supply for the proposed stream is the treated wastewater effluent that is discharged from the UBC South Campus "Pilot Wastewater Treatment Plant". The pilot plant carries out innovative research and development work in the wastewater field, with the current research being focused on the recovery of phosphorous fertilizers from municipal wastewater. To fully evaluate this option, it is recommended that further information regarding the facility's current and projected available flow and regulations regarding wastewater discharge be collected from UBC Research Associate Fred Koch.

3.1.6 Flow Augmentation Option Evaluation

If flow augmentation is required to maintain reasonable flows in the stream during the dry season, a decision must be made in regards to selecting the source and method of delivery to the proposed stream. This decision should be based on the amount of water available from the source, initial cost, annual operation and maintenance cost, and how the method of delivery corresponds with UBC's sustainability objectives. Cost and feasibility estimates must be made before a decision can be reached, and a decision must be made before proceeding to the final design work for the South Campus Urban Stream Restoration Project.

3.2 Hydraulic Losses

To determine the appropriate flow rate that is sufficient to supply the proposed stream with a sustained flow throughout the dry season, the stream's hydraulic losses due to infiltration and evaporation need to be quantified. As mentioned in the above Musqueam Creek flow augmentation example, although a groundwater aquifer pumping system was in place, all the water recovered from the well was lost 50m from the point of discharge, due to hydraulic losses. This example illustrates the importance of quantifying the losses of a watercourse to ensure sufficient flow is provided and that resources that are required to operate the system are not wasted.

3.2.1 Infiltration Rate

The infiltration rate is the speed at which water enters the soil and is measured in millimeters per hour. In forested areas, such as the site for the proposed stream, soil is generally very porous, and therefore has a relatively high infiltration rate. To determine infiltration rates, which will be representative of the soil conditions on the stream site, it is recommended that field tests be performed in the dry season, when the infiltration rates will be the highest. Below is a brief description of two typical field procedures that can be performed to determine the infiltration rate of a site.

Double-Ring Infiltrometer Test (ASTM D3385)

The double-ring infiltrometer test involves driving two open cylinders into the ground, one inside the other, partially filling the rings with water and then maintaining the water at a constant level. The outer ring helps reduce the lateral movement of water in the soil from the inner ring. The volume of water added to the inner ring to maintain the water level is the measure of the volume of water that infiltrates the soil. The volume of water that is infiltrated into the soil in a timed interval is then converted to an incremental infiltration velocity and is plotted versus elapsed time. The maximum steady-state or average incremental infiltration velocity is equal to the infiltration rate.

Percolation Test

The percolation test methodology is based largely on the criteria for on-site sewage disposal; however, it is often used as an alternative to the double ring infiltrometer test in determining the infiltration rate of a site. The test involves the following steps (BC Ministry of Heath 1998):

1. Dig test holes 30cm in diameter and 80cm deep
2. Fill percolation test hole with water and allow the water to drain to 13 cm from the bottom of the hole
3. Refill the percolation test hole, allowing the water to again drain to 5" (13 cm) from the bottom.
4. Add enough water to the percolation test hole to raise the water level in the hole to just above 15.5 cm from the bottom of the hole.

5. When the water level reaches 15.5 cm above the bottom of the hole, start timing until the water level reaches 13 cm above the bottom of the hole. Record time.
6. Repeat procedures 4 and 5 until the last two rates of fall do not vary by more than two minutes.
7. Record all times

From this data, a percolation rate can be determined by dividing the distance (2.5 cm) by the average time taken to fall the distance. To develop a representative infiltration rate, the averaged percolation rate must be adjusted to account for the discharge from both the sides and bottom of the hole and to develop a representative infiltration rate by using Equations 1 and 2 (SEMCOG, 2008).

Equation 1. Infilration Rate

$$InfilrationRate = \frac{PercolationRate}{\mathrm{Re}\,ductionFactor}$$

Equation 2. Reduction Factor

$$\mathrm{Re}\,ductionFactor = \frac{2d_1 - \Delta d}{DIA} + 1$$

d_1 = Initial Water Depth
Δd = Final Water Level Drop
DIA = Diameter of the Percolation Hole

3.2.2 Evaporation

Evaporation is the hydrologic process by which water in its liquid phase is transformed into water vapor. The amount of water that will be lost to evaporation in the dry season is an important parameter in ensuring that there is a sufficient volume of water in the pool sections of the stream year-round. A common direct method of determining the quantity of evaporation for a given location is the Class A evaporation pan, due to the lost cost and ease of application (Stanhill 2002).

Class A Evaporation Pan

The Class A evaporation pan is a galvanized iron tank which measures 1.2 m in diameter, 0.25 m deep, and is mounted level 0.30 m above the ground surface. To estimate the evaporation the pan is filled to a depth of 0.20 m and is required to be refilled when the depth has fallen to 0.18 m. The water surface level is measured daily, and the evaporation is measured as the difference between observed levels, with adjustments made taking in account for any precipitation measured in a standard rain gauge. In most situations, measured pan evaporation is higher than the actual evaporation and must be adjusted to account for radiation and heat-exchange effects (Chiew and McMahon 1992). The adjustment factor is called the pan coefficient and typically averages between 0.7 and 0.8.

3.3 Infiltration Mitigation

One option of limiting the hydraulic losses that the stream will incur due to infiltration is to install a liner at the base of the stream channel, which will restrict water from seeping into the underlying soil layer. Although the use of a channel liner may not be compatible with the vision of a natural watercourse system, a liner could serve a vital role in maintaining pool volumes and flow depths necessary for fish habitat. Again it is important to reiterate that the proposed stream is not a natural watercourse by definition; however, one of the main design objectives of the urban stream project is to provide the same level of ecological value as a natural watercourse.

Installing a channel liner would also be in compliance with the UBC Comprehensive Community Plan, published in 2000, which states that efforts should be taken to reduce groundwater infiltration to the upper aquifer where necessary to manage the cliff instability and erosion issues that are experienced along the coast lines on campus.

3.3.1 Geosynthetic Clay Liners

Geosynthetic clay liners (GCLs) are manufactured barrier layers that contain a layer of sodium bentonite attached to geotextiles or a geomembrane. The GCLs low permeability and high strength make it an ideal choice for the containment of water in engineered/constructed wetlands and stream channels (Miller 2000). In the last decade, design engineers and environmental agencies have increasingly chosen to use GCLs over compacted clay liner systems due to the low hydraulic conductivity and the lower cost (Bouazza 2002). Other noted benefits of GCLs over compacted clay liners include (Bouazza 1997):

- Rapid and easy installation with less-skilled labour required
- Not dependant on the availability of local soils
- High resistance to the effects of freeze thaw cycles
- Can withstand large differential settlements
- High resistance to root penetration
- Easy to repair and maintain

Some of the limitations of GCLs include:

- Low shear strength of hydrated bentonite in unreinforced GCLs
- Can be punctured or experience loss of betonite during installation
- Prone to desiccation if not properly covered with a sufficient soil layer

To address the limitations mentioned above, the following design and construction practices are recommended (Miller 2002):

- For slopes greater that 10:1, a reinforced GCL is recommended. Reinforced GCLs are manufactured by needle-punching or stitch-bonding the top and bottom geotexiles together to encapsulate the sodium bentonite layer. This physical bonding of the geotextile layers increases the GCLs internal resistance to shearing and creep.
- The GCL panels are to be overlapped 150mm-300mm.
- A minimum soil layer of 300mm-600mm is to be placed on top of the GCL to provide confinement to the sodium bentonite layer. The soil layer is to be placed in a single lift using standard earthwork equipment.

To determine the viability of using a liner at the base of the stream channel, it is recommended that an economic analysis be performed which looks at the costs associated with installing a channel liner versus the costs of supplying a higher flow from the chosen augmentation source that compensates the hydraulic losses due to infiltration.

3.4 Groundwater Monitoring

Determining the seasonal ground water fluctuations that occur on the proposed stream site will provide information that will indicate the possible interactions between the groundwater and stream. A simple and inexpensive method of monitoring the ground water level is by using an open standpipe piezometer.

3.4.1 Open Standpipe Piezometer

The ground water level can be measured by using a standpipe, which consists of an open-ended tube that is perforated near the base and is inserted into a borehole. Measurements of the water level in the standpipe are made by lowering a electrical dipmeter into the open standpipe. The dipmeter consists of a twin cable connected at the surface to a battery and a device that will detect closure of the electrical circuit. This device may consist either of a milliammeter or an oscillator, giving either a visual or audible signal when the water level is met (Geotech 2011).

3.5 Summary of Recommended Field Tests and Procedures

- Dry weather flow monitoring program
- Pump, aquifer, and recover tests performed on groundwater aquifer wells in Khorana and Michael Smith Park
- Double-ring infiltrometer or percolation test in dry season
- Class A evaporation pan procedure in dry season
- Open standpipe piezometer observations for one year

4.0 Flow Diversion

As discussed, the proposed urban stream is to be located along the eastern boundary of Pacific Spirit Park in the University of British Columbia's South Catchment Area. Based on discussions with UBC planning staff and a site reconnaissance completed on January 14, 2011, it is understood that the urban stream is proposed to commence near the intersection of Binning Road and Gray Avenue. The intent of the project is to construct an urban stream that is capable of supporting fish habitat. Based on this criterion, year round flows will be required. This section is intended to describe the options available to deliver flows to the proposed headwater location of the urban stream. Note the availability and sources of the year round flow is discussed in more detail in the *Section 3.0*. The remainder of the discussion here assumes required dry season base flow is supplied to the final diversion point.

A review of available record drawings for the south campus drainage area was completed to determine flow routing of the current system. Based on this review, it is our understanding that an existing flow diversion structure is located at the intersection of 16th Avenue and Wesbrook Mall. The flow control structure diverts base flows to the east, while allowing peak flows to discharge to the main trunk system to the south. Currently, a second flow diversion exists on Wesbrook Mall south of the existing sediment control pond. It was noted that diversion was marked on the drawings as temporary. Refer to *Figure 3* for a plan showing the approximate locations of the flow diversions.

Based on discussions with UBC planning, the intent is for base flows to be diverted at 16th Avenue and Wesbrook with flows eventually discharging into the existing pond structure at Michael Smith Park. The flow diverted to this system is regulated through a sluice gate installed in the manhole. Excess flows from the pond are either discharged to an infiltration system or into the existing gravity storm system, which drains along the eastern boundary of the catchment area. The rate and quantity of flow that could be diverted were not quantified as part of this study. Quantifying diverted flows through the summer and winter months will be a key piece of information required in order to complete the detailed design for this project. The exact arrangement and configuration of the diversion of flows into Michael Smith Pond was also not confirmed as part of this project, but should be fully investigated in the next phase of this project.

Figure 3. Existing Flow Diversion Locations

5.0 Flow Diversion Options

Based on the review of record drawings and the desired start location of the stream, three options for flow diversion were investigated. The three options reviewed were:

1. Provide a secondary outlet connection to the existing Michael Smith Pond.
2. Construct a diversion from the existing manhole at Binning Road and Gray Avenue.
3. Construct a diversion from the existing manhole and Binning Road and Birney Avenue.

5.1 Michael Smith Pond Outlet

The first option investigated was the use of Michael Smith pond as a headwater for the stream. Michael Smith pond receives flows from the local upstream area and the flow diversion at 16th Avenue as described above. Roof drainage from the adjacent buildings is also fed to the pond by a runnel system. Flows in excess of the pond high water level are diverted to a sand filter that recharges the local aquifer. Once the aquifer is fully recharged, excess flows overflow into the storm drainage system. During the summer the pond is highly dependent on the use water within the aquifer to maintain a constant water level. A pump system draws water from the aquifer to maintain the constant water level in the pond. Monitoring of flows entering the pond has not been performed; therefore, the range of flows directed to the pond could not be quantified.

Providing a secondary outlet from Michael Smith Pond would affect the hydraulics of the existing system. Impacts could include reduced water levels in the pond during the summer season. In addition, construction would require removal and replacement of existing hardscape that has been recently completed. Additionally, the pumping capacity of the supply well is not known, and may not be able to provide sufficient base flow during the dry season to both the stream and the pond. Based on the sensitive operating nature of the Michael Smith Pond, this option was not investigated further.

It should be noted that this system could potentially be a viable option for the supply of water to the proposed stream if the water balance for the system was fully evaluated. Refer to *Section 3.2.1* for detailed discussion.

5.2 Flow Diversion from Binning and Gray Avenue

The second option investigated was installing a secondary outlet in the existing manhole located at Binning Road and Gray Avenue. The existing storm drainage manhole (Q9D-S217) located at the intersection of Binning Road and Gray Avenue is directly adjacent to the proposed starting location for the urban stream. This manhole is a collection point for the upland drainage of the newly constructed buildings bounded by Wesbrook Mall, 16th Avenue and Pacific Spirit Park. It is expected that the base flow diverted at Wesbrook Mall and excess flows from Michael Smith Park eventually collect at this manhole, making this an ideal location for diverting flows to the proposed urban stream.

A review of the record drawings revealed the surface elevation for the manhole is approximately 73.86 m geodetic, while the outlet invert is 68.28 m geodetic. Based on this information it can be seen that the manhole depth is 5.58 m (18.3 ft.). Assuming a diversion pipe is installed at this invert and is placed at a grade of 0.5%, the diversion pipe would not daylight for approximately 300 m downstream. This location would roughly be adjacent to the existing TRIUMF operations. Based on this, it was deemed that diverting flows from this manhole would not meet the objective of the project, as approximately 300 m of the proposed stream length would be lost.

5.3 Flow Diversion from Binning and Birney Avenue

The final option investigated was installing a secondary outlet in the existing manhole located at Binning Road and Birney Avenue. The existing storm drainage manhole (P9D-S219) located at the intersection of Binning Road and Birney Avenue is located approximately 235 m upstream of the proposed starting location for the urban stream. This manhole is a collection point for the upland drainage of the newly constructed buildings bounded by Wesbrook Mall, 16th Avenue, Pacific Spirit Park and Birney Avenue.

A review of the record drawings revealed that the surface elevation for the manhole is approximately 78.45 m geodetic, while the outlet invert is 73.08 m geodetic. Based on this information it can be seen that the manhole depth is 5.37 m (17.6 ft.). Assuming a diversion pipe is installed at this invert and is placed at a grade of 0.50%, the diversion pipe would daylight in approximately 235 m downstream. This location would roughly be in the proposed location for the start of the stream. Based on this, it was deemed that diverting flows from this manhole would meet the objective of the project. Field investigation should be carried out during the dry summer season in 2011 to quantify the amount of base flow present in the system at this location as discussed in *Section 3.1.1*. Refer to *Figure 4* for location plan of the proposed flow diversion system.

Figure 4. Proposed Diversion Location at Binning Road and Birney Avenue

6.0 Stream Flow Requirements

As the proposed stream will be designed as if to be used as fish habitat, the habitat requirements of native fish species should be used as constraints on engineering design. Kosta Sanis, a UBC graduate student compiled relevant habitat requirements for cutthroat trout (*Salmo clarki*), the native fish species to other natural streams near the UBC campus (Sanis 2006). Table 1 outlines the maximum swim speeds and jump heights for cutthroat trout.

Table 1. Maximum swim speeds and jump height for adult and juvenile cutthroat trout (Dane 1978)

Species	Lifestage	Maximum Swim Seed (m/s)			Maximum
Cutthroat		Sustained	Prolonged	Burst	Jump Height (m)
	adults	0.9	1.8	4.3	1.5
	juveniles (125 mm)	0.4	0.7	1.1	0.6
	juveniles (50 mm)	0.1	0.3	0.4	0.3

In addition to swimming speed and jump height requirements, cutthroat trout also require a minimum stream depth of 6 cm, substrate size between 6 and 102 mm for spawning, and 1 m^3 of water maintained in pools through the dry season. Lastly, the optimal water temperature for cutthroat trout is approximately 15 degrees Celsius, with an absolute maximum temperature of 24 degrees Celsius (Sanis 2006). A complete summary of information pertaining to fish habitat requirements can be found in Sanis' report.

Based on the fish habitat criterion, flows in the dry summer months will be based on maintaining a minimum flow depth of 6 cm within the low flow channel, while during winter months the design criteria will be based on not exceeding a maximum flow velocity of 1.0 m/s (Sanis 2006). As such, the flow diversion structure should be sized to deliver an appropriate range of flow to the stream. Refer *to Section 12.0* for details on the stream hydraulics.

7.0 Diversion Structure

Diversion of flows to the future urban stream is proposed to occur at the existing storm drainage manhole located at the corner of Binning Road and Birney Avenue as described above. It is proposed the diversion of flows be accomplished by the addition of a small diameter outlet from the existing manhole. This small diameter outlet would have in invert elevation equal to the existing 675 mm diameter outlet. Summer base flows would be directed to the small diameter outlet via a weir located in the invert of the outlet pipe. The weir height would not exceed the height of the proposed diversion pipe diameter, and only needs to be tall enough to provide the required base flow. Refer to *Figure 5* for a schematic diagram of the proposed structure.

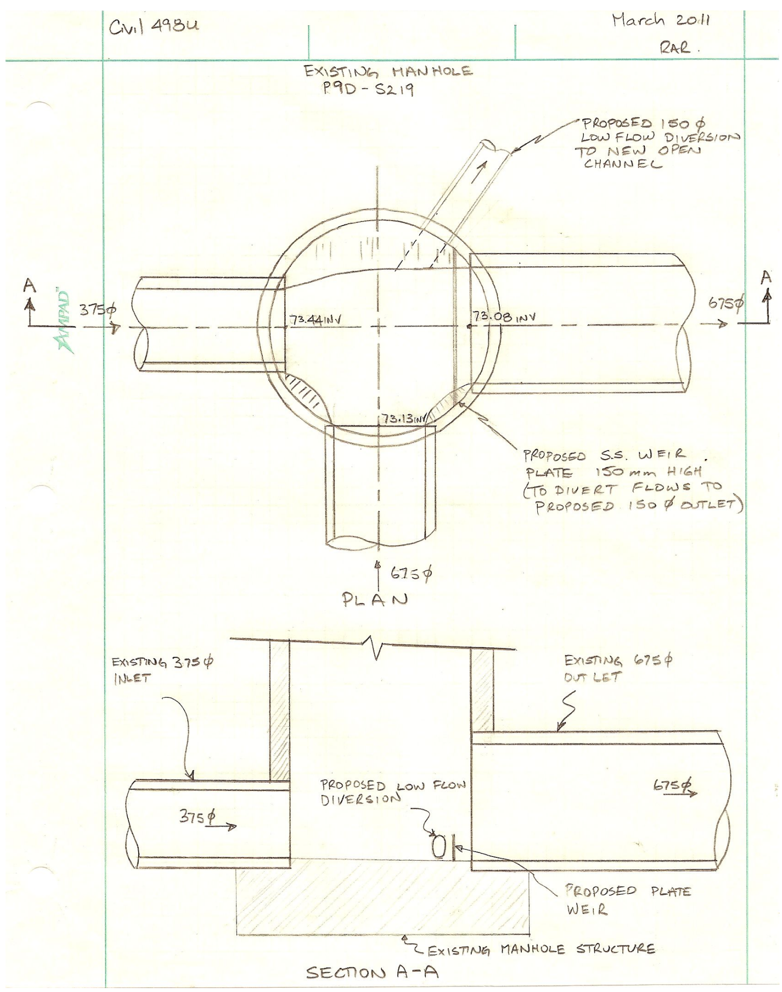


Figure 5. Schematic Plan / Profile of Diversion Manhole

7.1 Flow Diverter Design

It has been proposed that the flow diversion pipe be installed at the same invert as the existing outlet from the manhole. In order to direct base flow to the proposed diversion pipe a diversion within the manhole is required. This can be accomplished in various ways such as:

- Inflatable dams
- Orifice plates
- Sluice gates
- Weirs

Weir control was selected as they are relative easy to install, are low cost and have minimal maintenance. Sedimentation could occur behind the weir plate during low flows due to decreased velocity, however due to the benching in the manhole, it is expected that any sediment accumulation would be washed through the manhole during higher flows. Additionally, the weir could also be designed to be removable, as the weir is only required to divert flows during the dry season.

The height of the weir is dependent on the base flow requirement for the stream and the extent at which the flows can be restricted in the manhole without having serious upstream consequences. Due to the installation of a weir, the system may experience additional surcharging during extreme events. The extent of surcharging depends on the weir height selected and the peak discharge during the maximum design storm event. The effects of the weir and diversion pipe installation should be analyzed in the storm water model, which was developed for the South campus.

8.0 Diversion and Existing Pipe System Hydraulics

Hydraulically the diversion pipe will behave as a culvert. Culverts are a unique type of constriction and its entrance is a special kind of contraction (Chow 1959). The characteristics of flow in this type of system are complicated as the flow is controlled by multiple variables including:

- Inlet geometry
- Pipe slope, size and roughness
- Approach and tail water conditions

Chow further explains culvert flow can be classified into six types. The categories can be explained according to the following descriptions and *Figure 6.*

A) Outlet submerged .... Type 1
B) Outlet un-submerged
    1. Headwater greater than the critical value
        a. Culvert hydraulically long........ Type 2
        b. Culvert hydraulically short .... Type 3
    2. Headwater less than the critical value
        a. Tail water higher than the critical depth .... Type 4
        b. Tail water lower than the critical depth
            i. Slope subcritical .... Type 5
            ii. Slope supercritical .... Type 6

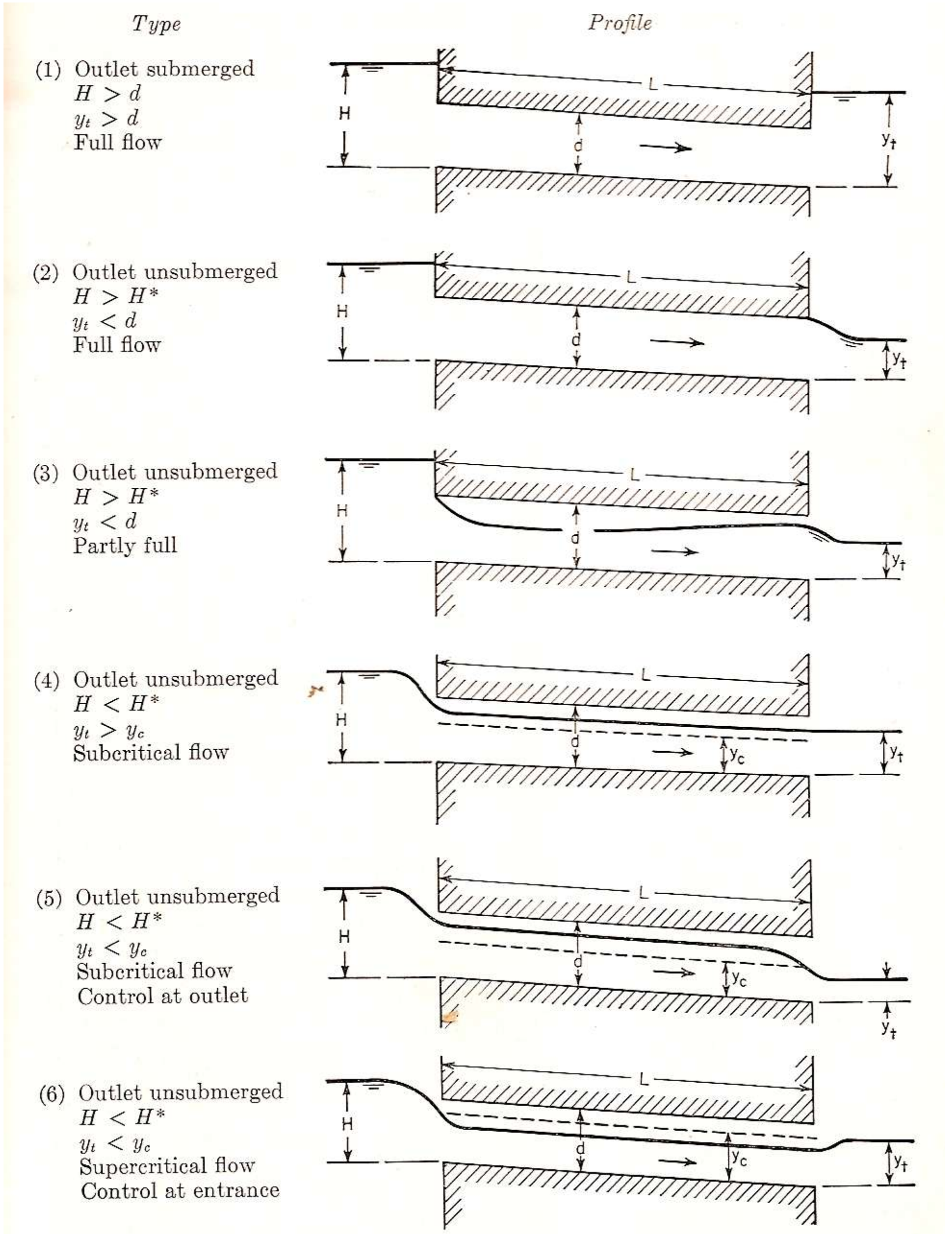


Figure 6. Types of Culvert Flow (Chow 1959)

According to Chow, a culvert will flow full when either the outlet is submerged or when the outlet is not submerged and the system is considered to be hydraulically long (Type 1 and Type 2). A hydraulically long system indicates that the culvert is sufficiently long to allow the flow to expand and fill the cross section downstream of the inlet constriction. Contrary, a hydraulically short pipe will not allow the flow to expand and thus will never flow full (Type 3). Determining if the culvert is hydraulically long or short is dependent on size, slope, entrance geometry, headwater, entrance and outlet conditions, etc. A chart has been prepared by Carter, which aids in determining between hydraulically long and short pipes, refer to Figure 7.

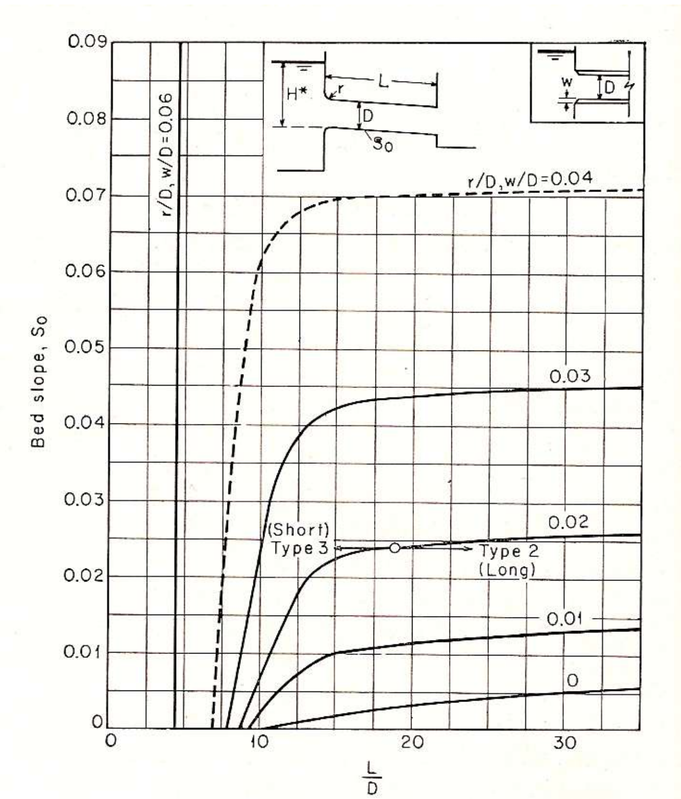


Figure 7. Criteria for hydraulically short and long box and pipe culvert (Chow 1959)

As noted in Figure 6, a value of critical headwater depth (H*) is required to further classify the culvert flow type. The critical headwater depth is related to lab testing that indicated a culvert might not be considered submerged if the headwater depth is below a critical value. For preliminary analysis H* can be assumed to equal 1.5 times the pipe diameter, but in practice is generally observed to be 1.2 times the pipe diameter.

For flow Types 4 through 6, the distinction between sub and supercritical flow within the pipe is also required. This can be determined based on the Froude Number.

Equation 1. Froude Number
$$F = \frac{V}{\sqrt{gL}} = \frac{V}{\sqrt{gD}} = \frac{V}{\sqrt{g \cdot A/B}}$$

In open channel flow the characteristic length L is equal to the hydraulic depth D, where the hydraulic depth is defined as the ratio of flow cross sectional area (A) to the width of the free surface (B). For Froude values greater than one the flow is supercritical, for values less than one the flow is subcritical.

For flow Types 1 and 2, flow within the culvert is based on pipe full flow conditions and can be calculated using the Manning Equation. For flow Type 3 the flow capacity of the culvert is determined through orifice calculations. While, for flow Types 4 through 6, the flow capacity of the culvert is based on weir hydraulics.

9.0 Diversion Pipe Flow Conveyance

The diversion pipe must be sized to divert base flow in the dry season, while limiting peak discharges to the stream in order to support the potential for fish habitat. The proposed diversion from the existing manhole located at Binning Road and Birney Avenue will be placed at a shallow grade of 0.50 %. This minimum grade will allow the pipe to daylight quickly and at the desired location for the stream headwater. Based on this slope and invert elevation of the manhole, the diversion pipe is expected to be in the range of 235 m long. Hydraulic analyses were completed for pipe sizes from 150 mm to 450 mm to evaluate the optimum diversion pipe size. The following sections detail the hydraulic calculations completed.

The first step in determining the flow capacity of the diversion pipe is to determine if the diversion pipe will act as a hydraulically long or short culvert, see Table 2 below. As can be seen, the diversion pipe will act as a hydraulically long culvert for all pipe sizes considered.

Table 2. Diversion Pipe Determination Hydraulically Long or Short

Option	Pipe Diameter (mm)	Pipe Length (m)	L/D	r/D	Hydraulically Long or Short
1	150	235	1567	0	LONG
2	200	235	1175	0	LONG
3	300	235	783	0	LONG
4	375	235	627	0	LONG

Due to the shallow slope of the pipe, flow within the pipe will be subcritical. When the headwater depth in the manhole is below the critical depth the pipe will act as a Type 4 or 5 culvert depending on the tail water depth. Due to the fact that the flow is subcritical the tail water depth will not affect the inlet condition, therefore the capacity can be based on the Manning equation. Once the headwater depth in the manhole exceeds the critical depth, the culvert will act as a Type 2 culvert and capacity is also based on the Manning equation, however the slope is based on the slope of the hydraulic grade line. Refer to *Appendix B* for sample and detailed calculations. For the design calculations a Manning coefficient of 0.013 was used for new smooth plastic pipe.

9.1 Headwater Depth Less than Critical (Type 4)

Water flowing within a pipe conduit with a free water surface is hydraulically classified as open channel flow. In addition, flow within the pipe is considered to be uniform flow when:

- Water depth is constant
- Cross sectional flow area is constant
- Energy grade line, water surface and conduit slope are parallel

Uniform flow rates in open channels can be calculated using the Manning equation as shown in Equation 2.

Equation 2 – Manning Equation $Q = \frac{1}{n} \cdot A \cdot R_h^{\frac{2}{3}} \cdot S^{\frac{1}{2}}$ **(SI Units)**

In order to determine conveyance capacity, it is important to quantify the geometry of the pipe section, refer to Figure 8. The Manning equation determines flow rates based on the Manning roughness (n), cross sectional flow area (A), wetted perimeter (WP), hydraulic radius (R_h) and the pipe slope (S), where the hydraulic radius is defined as the ratio of flow area (A) to the wetted perimeter (WP). Refer to sample calculations in *Appendix B*.

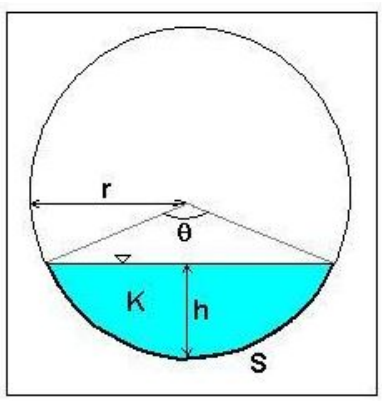


Figure 8. Partial Pipe Flow Geometry

It should be noted that, the water level in the manhole will be above the flow depth in the pipe due to entrance losses and velocity head (assuming velocity in the manhole is zero). The headwater depth in the manhole can be calculated based on the flow depth and velocity in the pipe. Refer to Equation 3 and profile in Figure 9.

Equation 3. Headwater Depth

$$H = h + \frac{kv^2}{2g} + \frac{v^2}{2g}$$

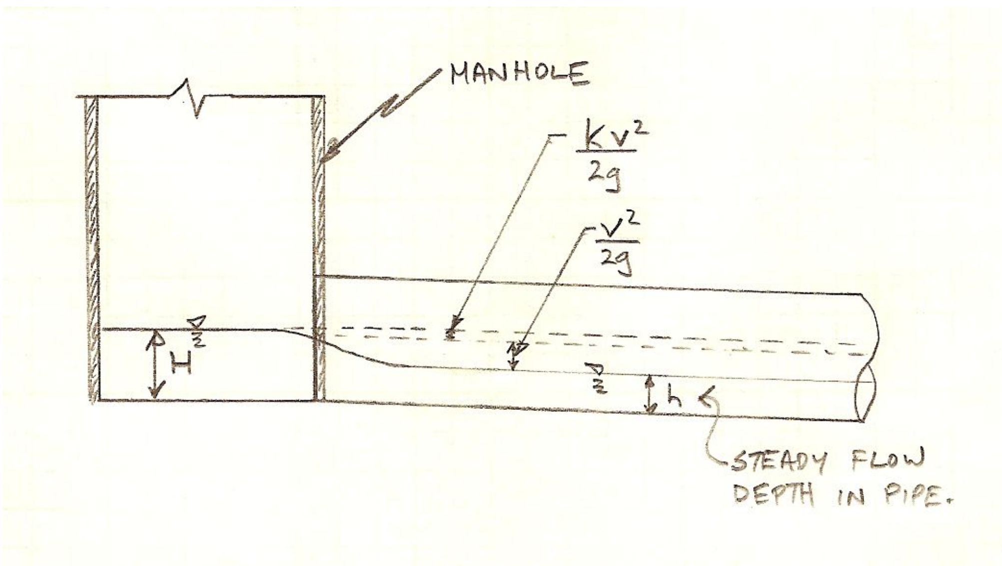


Figure 9. Headwater Depth at Manhole

9.2 Headwater Depth Greater than Critical (Type 2)

Once the flow exceeds the pipe full capacity, the water level in the manhole will rise eventually resulting in submergence of the inlet. Once this condition occurs the capacity of the pipe changes from Type 4 to Type 2 culvert flow. Under this condition pipe capacity is also based on the Manning equation, however the pipe slope used to calculate capacity is based on the slope of the hydraulic grade line. Refer to sample calculations presented in *Appendix B*.

Based on the above calculations the estimated capacity of the diversion pipe is presented in Figure 10. As can be seen once the headwater depth in the manhole exceeds the critical depth the capacity of the pipe changes dramatically. Typically, it is undesirable to operate gravity drainage systems in a surcharged state. Assuming a maximum headwater depth in the manhole equivalent to the existing outlet pipe diameter (675 mm) the maximum capacity of each diversion pipe option is summarized in *Table 2*.

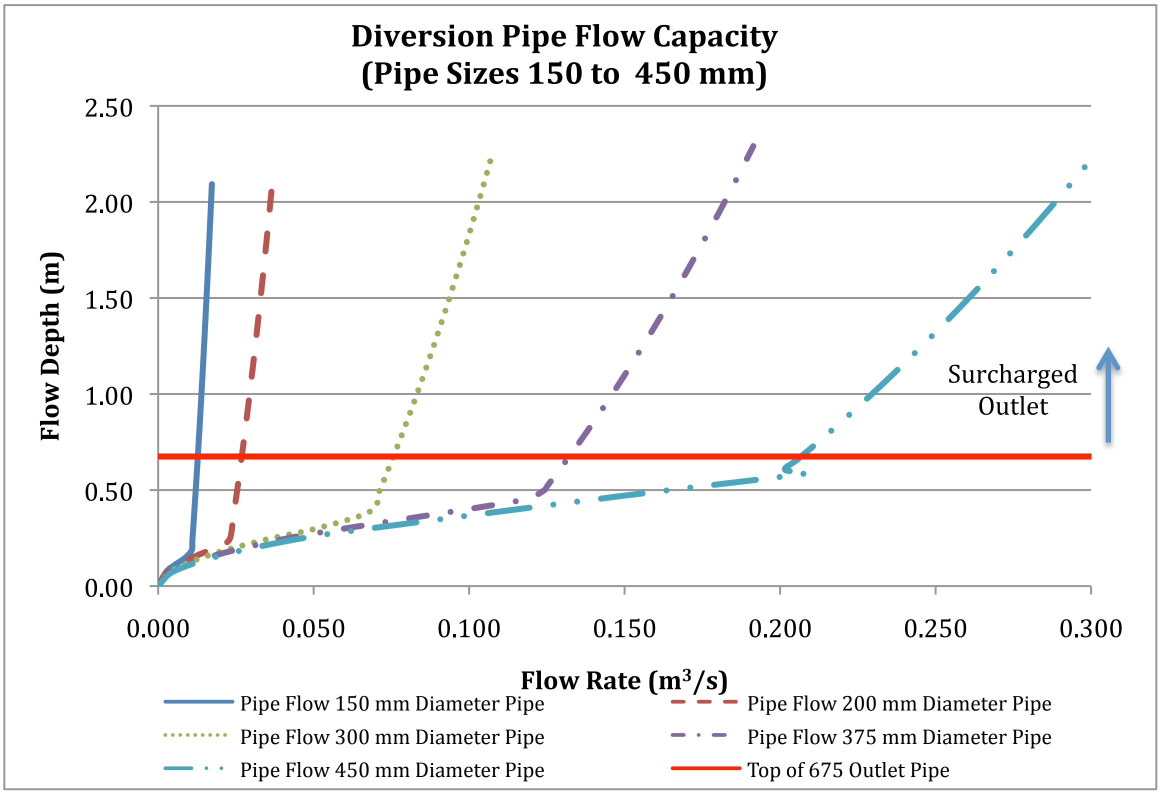


Figure 10. Diversion Pipe Capacity

Table 3. Diversion Pipe Capacity

Option	Pipe Size (mm)	Headwater Depth (m)	Flow Capacity (m^3/s)	Flow Capacity (L/s)
1	150	0.675	0.013	13
2	200	0.675	0.028	28
3	300	0.675	0.079	79
4	375	0.675	0.140	140
5	450	0.675	0.222	222

10.0 Existing Manhole Outlet Conveyance

10.1 Current Condition

The existing manhole outlet pipe is 675 mm diameter and has an approximate grade of 3.6%. Based on pipe length to diameter ratio and the chart presented in Figure 7, the pipe acts as a hydraulically long culvert. See Table 4 for summary.

Table 4. Existing Outlet Determination Hydraulically Long or Short

Pipe Diameter (mm)	Pipe Length (m)	L/D	r/D	Hydraulically Long or Short
675	60	89	0	LONG

Due to the pipe grade, flow within the pipe will be supercritical; therefore the critical flow capacity of the pipe will occur when the Froude Number is equivalent to 1. At this Froude Number the pipe flow will be supercritical, thus the existing 675 mm diameter outlet behaves as a Type 6 culvert. Flow capacity will be inlet controlled and based on weir constriction for headwater depths up to 1.5 times the pipe diameter, while above the critical headwater depth; flows will be based on orifice constriction calculations. Refer to *Appendix B* for sample and detailed calculations. The following assumptions were used in the calculations:

- Weir coefficient 0.90
- Orifice coefficient 0.72

The capacity of the existing outlet is shown in *Figure 11*. As mentioned previously, flow monitoring and design flow calculations for the system were not available at the time of this study; therefore verification if peak flows are contained within the pipe could not be completed. The capacity of the existing outlet without surcharging is approximately 1170 L/s.

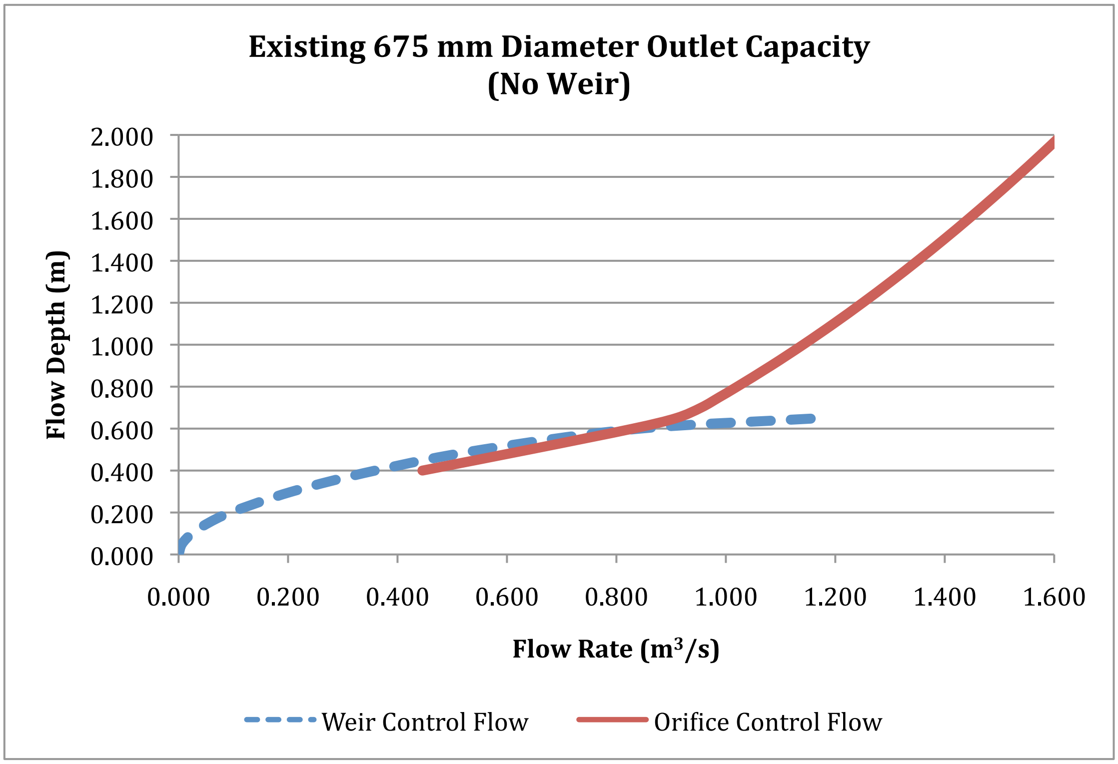


Figure 11. Existing 675 mm Dia. Outlet Capacity

10.2 Effects of Weir Installation on System Capacity

Installation of a weir within the existing 675 mm diameter invert will affect the capacity of the system at the manhole by restricting the available flow cross sectional area. The extent of the effects is directly based on the height of the weir installed. The effects of 50, 100 and 150 mm tall weirs were analyzed. *Figures 12* through *14* shows the effects of the various weir heights, while Table 5 summarizes the percent flow restriction caused by the various weirs at pipe full flow condition (675 mm). Flow calculations are based on weir and orifice flow with coefficients as follows:

- Weir coefficient 0.90
- Orifice coefficient 0.72

Table 5. Summary of Weir Effects at Pipe Full Condition

Weir Height (mm)	Flow at Pipe Full (m^3/s)	% Flow Reduction over Existing Condition
No Weir (existing condition)	1.17	0.0
50	1.11	5.4
100	1.01	13.7
150	0.90	23.1

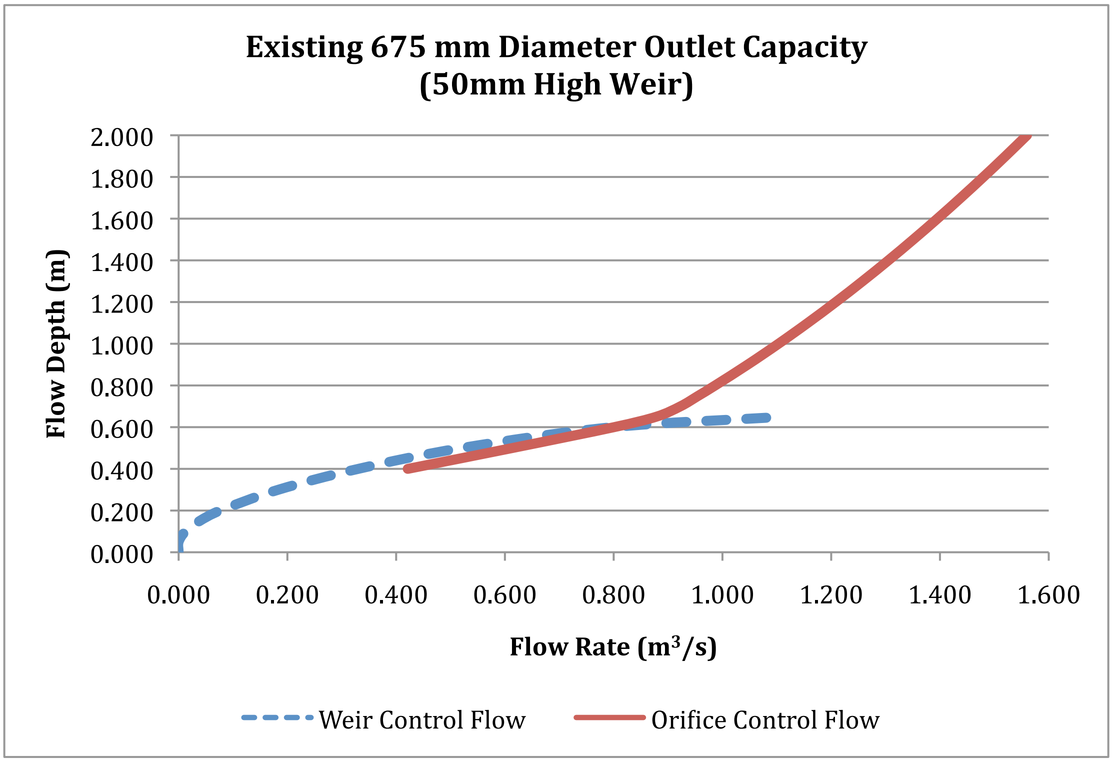


Figure 12. Effects of 50mm Weir

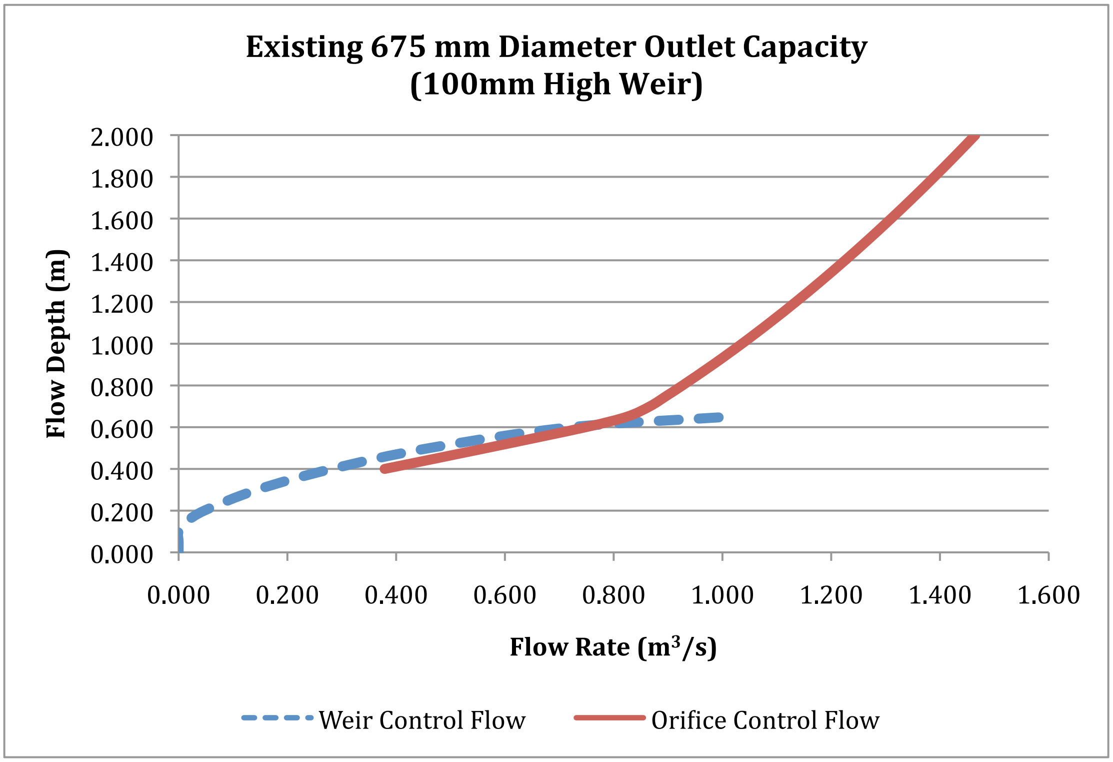


Figure 13. Effects of 100mm Weir

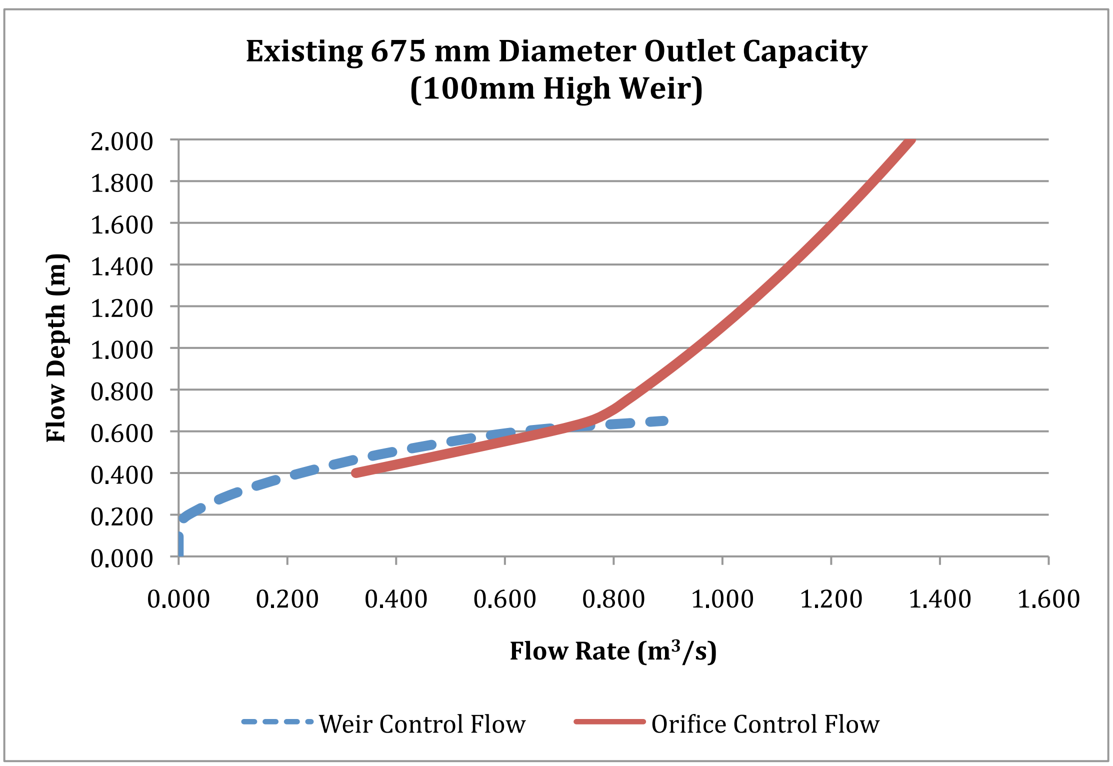


Figure 14. Effects of 150 mm Weir

Based on the above information it can be seen that the installation of a weir will impact the capacity of the existing system. The weir height chosen should be limited to the minimum required to divert base flow to the stream. This will minimize the restriction of the existing outlet.

11.0 Stream Physical Design

This section is intended to provide a conceptual physical design for the South Campus Urban Stream. Recommendations for preliminary cross section and profile geometry are provided and natural stream morphologies are applied to appropriate reaches of the stream.

11.1 Cross Section Geometry

The depth of excavation will vary with the existing topography; however, it is estimated to generally be between 0.5 and 1.0 meters. The proposed typical cross section, with a nominal depth of excavation of 0.65 meters, is shown in the figure below. Proposed bank slopes are 1.5H to 1V within the active channel, and 3H to 1V on the upper grass covered slope transitioning to natural ground.

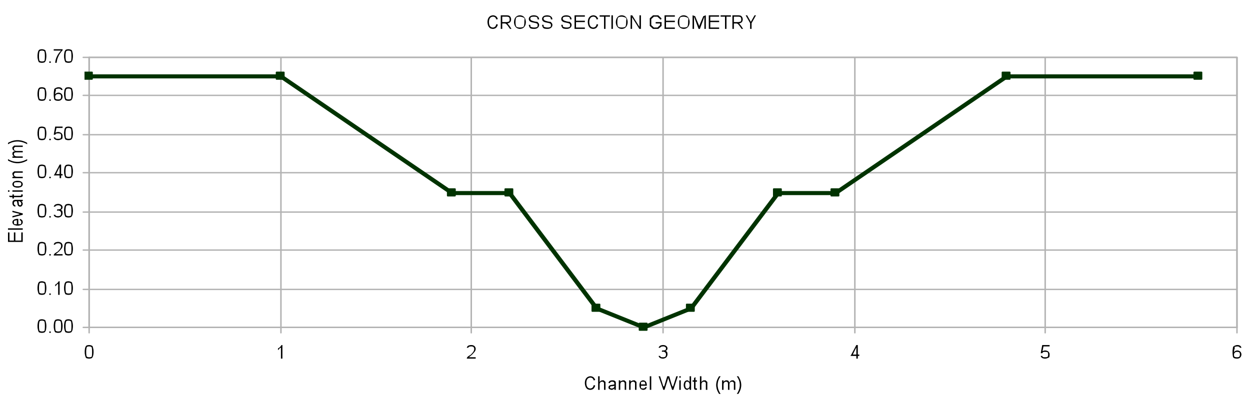


Figure 15. Preliminary cross section geometry for the proposed stream

Planting benches of nominal 0.3 meter width are included along both banks to ensure proximity of riparian vegetation for shade and insect activity, as well as to promote establishment of root structures to enhance the stability of bank material. For stepped riffle-pool and cascade-pool reaches of the creek, essentially the same cross section design may be used, with minor adjustments to the depth of excavation established in the field during construction based on proposed riffle and cascade locations.

The range of design discharge for the stream will be conveyed within the lower portion of the channel, though the area above the planting benches may become inundated during higher flow events. Based on preliminary hydraulic analysis, it appears overtopping of the planting benches may occur for flows exceeding 150 – 200 L/s. Areas at risk of such overtopping include the 1.0 % uniform slope portions of the stream, as well as immediately upstream of riffle crests through stepped riffle-pool reaches.

Local steepening of bank slopes may be required due to land tenure constraints or proximity to adjacent infrastructure, and can be accomplished with rock-stack walls, wood crib retaining structures or natural large wood debris and rootwads embedded within the bank slope. In such cases, effort should be made to avoid excessive encroachment or narrowing of the active channel. Where additional land is available nearby the proposed stream corridor, offline pool and wetland habitat should be created adjacent to the stream. In all cases effort is owed to minimizing disturbance to the margin of existing forest land along UBC's east boundary, as well as avoiding costly relocation of existing infrastructure where possible.

11.2 Stream Reaches

The BC Ministry of Environment Fish Habitat Rehabilitation Procedures document (Hogan and Ward 1997) outlines three types of fish bearing streams commonly found in British Columbia: step-pool, cascade-pool, and riffle-pool forms. Step-pool streams are found in areas with gradients exceeding 4%, cascade-pool and riffle-pool types are found on slopes of less than 4%, and riffle-pool types with sand bottoms are found in on slopes of less than 2% (Hogan and Ward 1997). Survey data was collected (Wiebe and Martens 2010) along the proposed corridor for the stream and was incorporated into the most recent UBC record drawings provided. The drawings found in *Appendix A* show the survey data, the proposed alignment for the stream, the existing ground elevations and the proposed streambed elevation along the entire channel length. *Table 6* breaks the length of the channel into reaches of roughly uniform slope based on minimizing excavation depth for the stream and indentifies the appropriate stream design type based on recommendations by the BC Ministry of Environment (Hogan and Ward 1997).

Table 6. Proposed stream reach length, slope, and construction type

Reach #	Start	End	Length	Slope	Stream Type
1	0+000	0+320	320m	3.0%	Step-pool
2	0+320	0+540	220m	2.0%	Riffle-Pool
3	0+540	0+640	100m	3.5%	Cascade-Pool
4	0+640	0+900	260m	2.5%	Cascade-Pool
5	0+900	1+080	180m	1.0%	Riffle-Pool
6	1+080	1+300	220m	2.0%	Riffle-Pool

It is important to note that due to spatial constraints, the proposed urban stream will likely have little sinuosity. For this reason, the stream morphology types chosen must adequately control flow velocity in order to prevent channelization and provide sufficient habitat for native fish species.

11.3 Step-Pool Stream Design

Due to the steep grade and close proximity to the storm water diversion, it is proposed that reach 1 not be intended for fish habitat. Instead, reach 1 will be designed to provide a buffer zone between the pipe outlet and the fish habitat reaches of the stream with the intention of improving water quality and reducing flow velocity.

The step-pool stream morphology can be applied to the design of stream reach 1. Step-pool morphology streams often are not accessible by fish due to the steep gradients (>4%) and large steps (Hogan and Ward 1997). By using a step pool design, the stream velocity will be controlled to prevent channelization and erosion in the steam. The primary bed materials should be boulders and cobbles, and the bank material should be armored with boulder to prevent erosion (Hogan and Ward 1997). The boulder and cobble bed material should exceed the normal water depth as to encourage growth of algae and moss on the surface. The materials should be sized to not be hydraulically moved by the largest flood event expected in the channel (Hogan and Ward 1997).

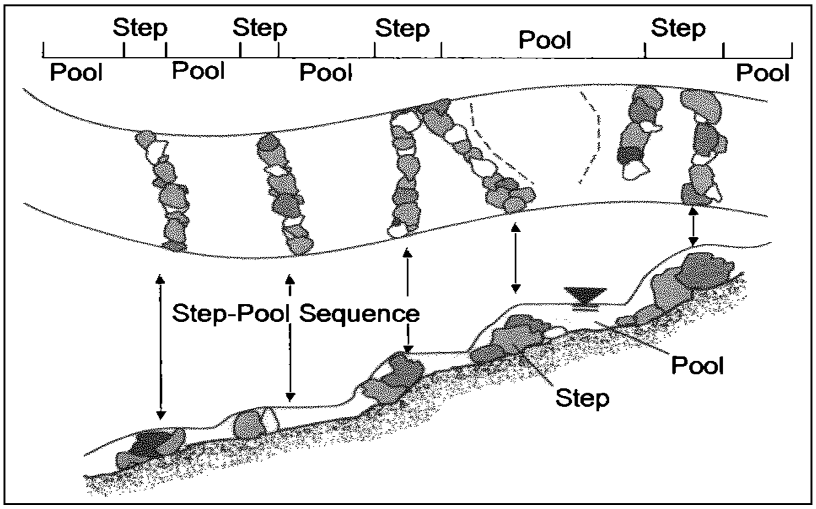


Figure 16. Step-pool stream morphology(Church 1992)

It is recommended that a precast headwall be installed at the diversion pipe outlet into the stream in order to prevent erosion around the discharge point. Additionally, a small pool can be constructed using a large riffle and a deeper excavation point near the headwaters of the stream.

11.4 Cascade-Pool Stream Design

Reaches 3 and 4 will be constructed to as a cascade-pool type stream. *Figure 17* shows an illustration of the physical appearance of a cascade-pool type stream. This stream morphology consists of cascade sections placed in between small pools (Hogan and Ward 1997). Cascades are essentially steeper pitches of the stream (2-4%) where relatively larger cobble is used to create small steps and pools down the slope. In these zones, flow energy is dissipated, water is aerated, and nutrients and sediments are transported downstream. The small steps created by the angled cobble lines provide important back eddy areas where fish are able to rest while moving upstream. The pools are to be varied in size and depth (Hogan and Ward 1997) and should contain at least 1 cubic meter of water during the dry season (Sanis 2006). In natural cascade-pool streams, large woody debris (LWD) may be present but has a minimal functional role in the stream habitat except to form steps, trap/scour sediments in certain locations, and protect banks from erosion (Hogan and Ward 1997).

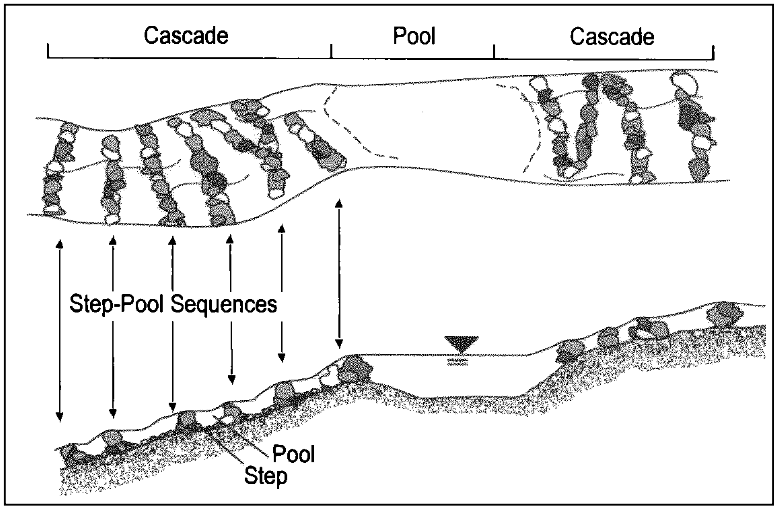


Figure 17. Cascade-pool morphology (Grant, Swanson and Wolman 1990)

The cascade sections of the stream should be constructed of stone sizes stable at bankfull flow conditions (Newbury, Gaboury and Bates 1997). The diameter of stone that is stable at these conditions can be estimated with the following equation:

Equation 4 $$\phi_s(cm) = 1500DS$$

where D is the depth of flow (m) and S is the slope (m/m) (Newbury, Gaboury and Bates 1997). The stones should be sized to be above the water surface during low flow conditions to enable growth of algae and moss on the tops in order to replicate natural cascade-pool streams (Hogan and Ward 1997). The stones should be placed in lines across the channel in areas connecting the lower gradient pools and should be footed into the bed material to improve stability (Newbury, Gaboury and Bates 1997). Pools should be at least one bankfull width in length (Hogan and Ward 1997) and should be spaced at intervals equal to 6 to 8 times the bankfull width of the channel (Newbury, Gaboury and Bates 1997).

11.5 Riffle-Pool Stream Design

Riffle-pool stream morphology is recommended for any slopes less than or equal to 2%. Riffle-pool streams have low gradients, large pools, and provide excellent spawning and rearing habitat for fish. In natural riffle-pool streams, large woody debris (LWD) does not play a significant role in fish habitat; therefore, should not serve a significant function in the proposed stream except to support the construction of riffles and pools. LWD that is placed in the stream should be oriented across and should span the entire channel (Hogan and Ward 1997).

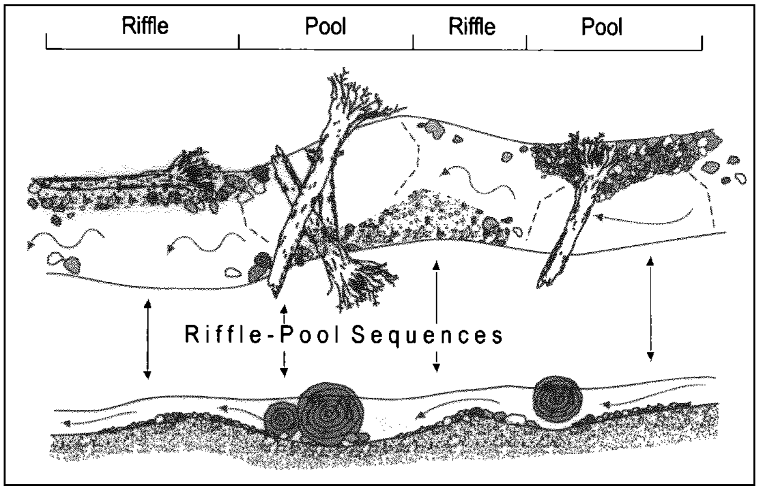


Figure 18 Riffle-pool stream morphology (Hogan and Ward 1997)

The main feature of stable riffle-pool streams is repeating riffle-bar-pool sequences. Pools, bars, and riffles should be diverse in shape and size, and the entire stream should consist of between ½ to ¾ pool environment. The dominant bed materials present in riffle-pool streams are cobble, gravel, and sand (Hogan and Ward 1997).

The construction of riffles is the critical component of the riffle-pool stream design. *Figure 19* provides an illustration and definition of important riffle design parameters. Riffle top (W_t) and base width (W_b) both are dictated by channel cross-section and should span the entire channel width. Research suggests that in order to reflect natural streams, the length of the riffles (L_r) should be between one and three times the bankfull width (Grant, Swanson and Wolman 1990, Carling and Orr 2000). S_d, the downstream riffle slope, should also reflect natural channels and research suggests that the mean downstream face slope in lower gradient channels is approximately 4%, and typically S_d is less than 10% (Carling and Orr 2000, Newbury, Gaboury and Bates 1997). The upstream face (Su) should be much steeper than the downstream face with a slope between 25% and 100% (Hey 1992, Newbury, Gaboury and Bates 1997). To further replicate natural channel morphology, the riffle v-apex slopes should be approximately 10% (Whyte 1997) or have a total v-apex depth of 0.3 to 0.6m (Newbury, Gaboury and Bates 1997).

This v-shape ensures low flows are focused toward the center of the stream to increase depth for fish passage and prevent bank scour near the riffles (Newbury, Gaboury and Bates 1997).

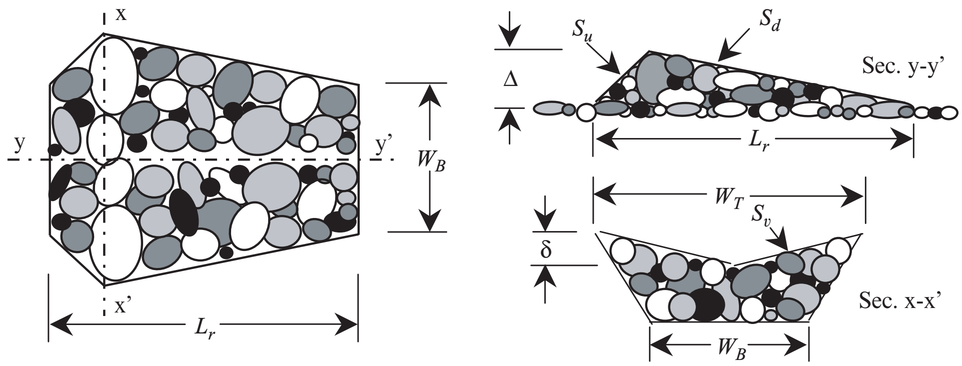


Figure 19. Template proposed for construction and design of riffles and cascades (Walker, Millar and Newbury 2004)

The most important riffle design parameter to design is the height (Δ). The riffle must be large enough to maintain pool depth for fish habitat on the upstream side, but must not negatively impact the flood conveyance capacity of the channel (Newbury, Gaboury and Bates 1997). Δ_{max} can be determined using *Equation 5* assuming no energy losses, no upstream sediment accumulation, and critical non-backwatering conditions at the crest of the riffle (Walker, Millar and Newbury 2004).

Equation 5

$$\Delta_{\max} = Y_b + \left(\frac{q^2}{2gY_b^2}\right) - \left(\frac{3q^2}{2g}\right)^{1/3}$$

In *Equation 5*, Y_b is the limiting upstream depth or the depth to the top of the bank; q is the design discharge per unit width (m^2/s); and W is the mean channel width (m). The design flood may then be used in equation 1 to ensure that the flood is contained within the acceptable floodplain (Newbury, Gaboury and Bates 1997). Stage-discharge relationships for flow over the riffles is discussed further in *Section 12.1.*

Once the riffle shape has been designed, materials must be specified and riffle stability must be analyzed. The size of material stable for riffle construction, during the maximum in channel flood stage, can be obtained using *Equation 6* (Walker, Millar and Newbury 2004).

Equation 6 $$D_{50} = \frac{10}{\chi} Y_d S_d$$

D_{50} is the characteristic riffle grain size at which 50% is finer (m); Y_d is the maximum uniform flow depth over the riffle; and χ is a correction factor for steeply sloped beds (<5%) and can be estimated with *Equation 7*:

Equation 7 $$\chi = \cos\theta(1 - \frac{\tan\theta}{\tan\phi})$$

where $\theta = \tan^{-1} S_d$ is the downstream riffle face angle (degrees); and ϕ is the natural angle of repose of the material. The typical value of ϕ for sediment diameters greater than 0.1m is 40° (Walker, Millar and Newbury 2004).

11.6 Bed Substrate

Between riffles and cascades, bed material must be sized appropriately to avoid erosion and siltation in the stream while providing sufficient habitat for fish spawning. *Figure 20* can be used in conjunction with expected peak storm flows to determine minimum stable particle size for steam bed substrate.

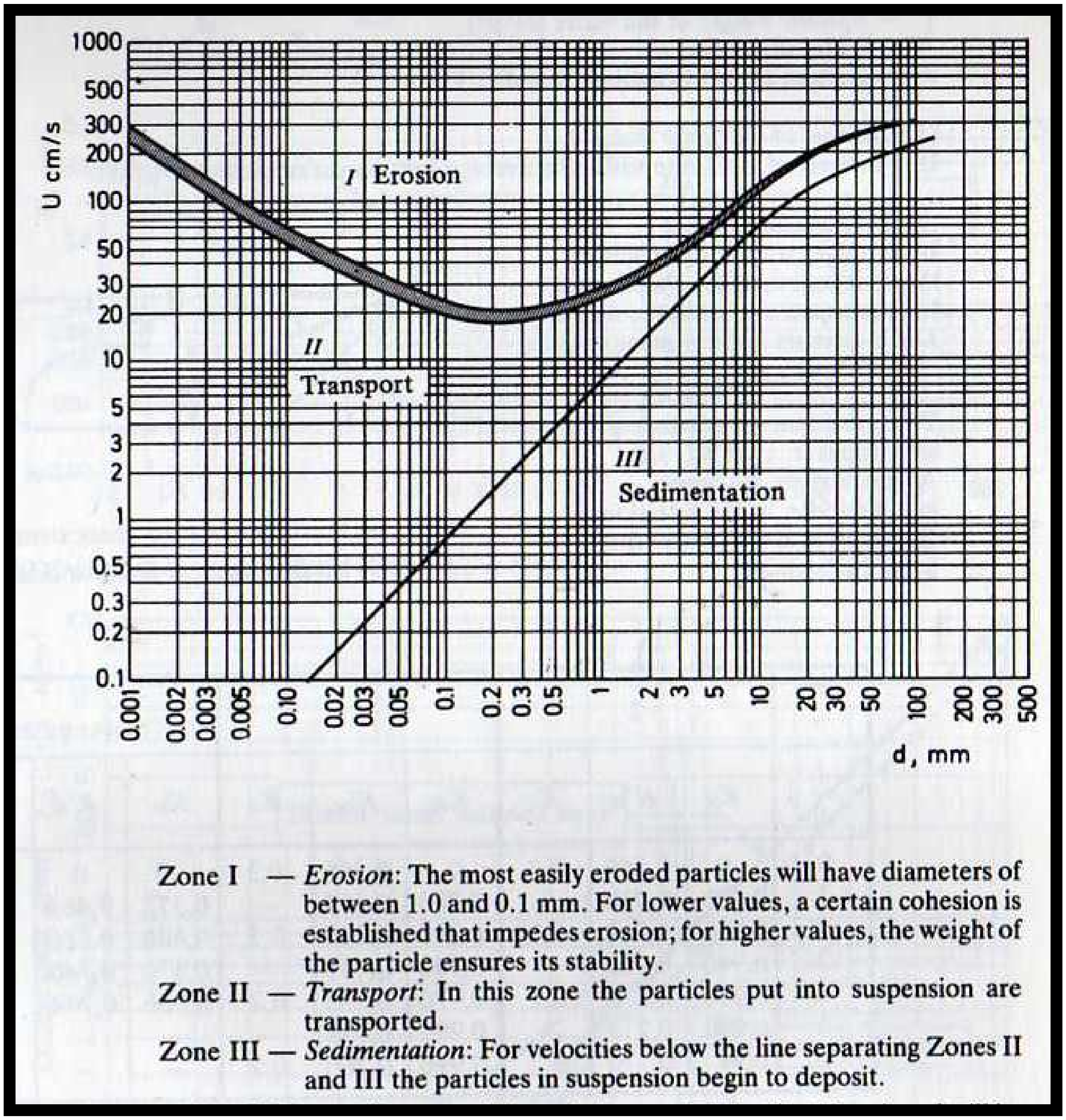


Figure 20. Critical velocity with respect to streambed material diameter (Lane 1955)

As outlined in *Section 6.0*, the maximum desired velocity for cutthroat trout habitat is 1 m/s and the ideal substrate size for spawning is 6 to 102mm. Using *Figure 20*, it can be demonstrated that substrate size greater than 15mm is appropriate for both flow and fish requirements.

12.0 Stream Hydraulics

12.1 Stream Velocity, Stage and Discharge

The relationship between volumetric discharge, mean velocity and water surface elevation (stage) for the proposed stream is illustrated by the charts below, based on Manning's equation for open channel flow. Curves have been plotted for a range of uniform channel slopes between 1.0 and 3.0%, and for Manning's n values ranging between 0.04 and 0.06 for the main portion of the channel. Overbank portions of the channel have been consistently assigned Manning's n values of 0.07.

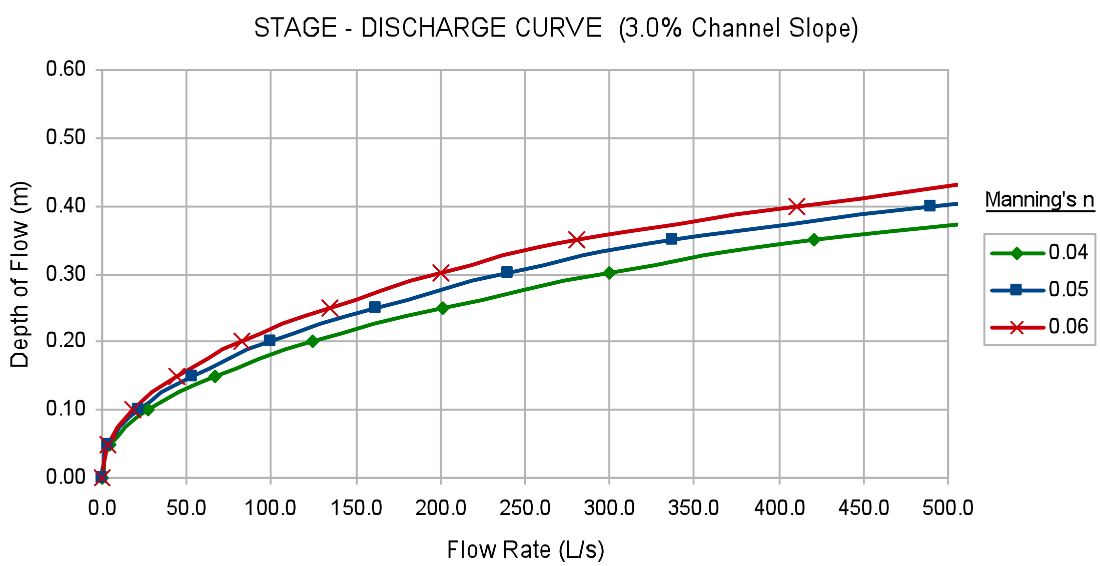


Figure 21. Stage-discharge curve for 3.0% uniform slope reaches

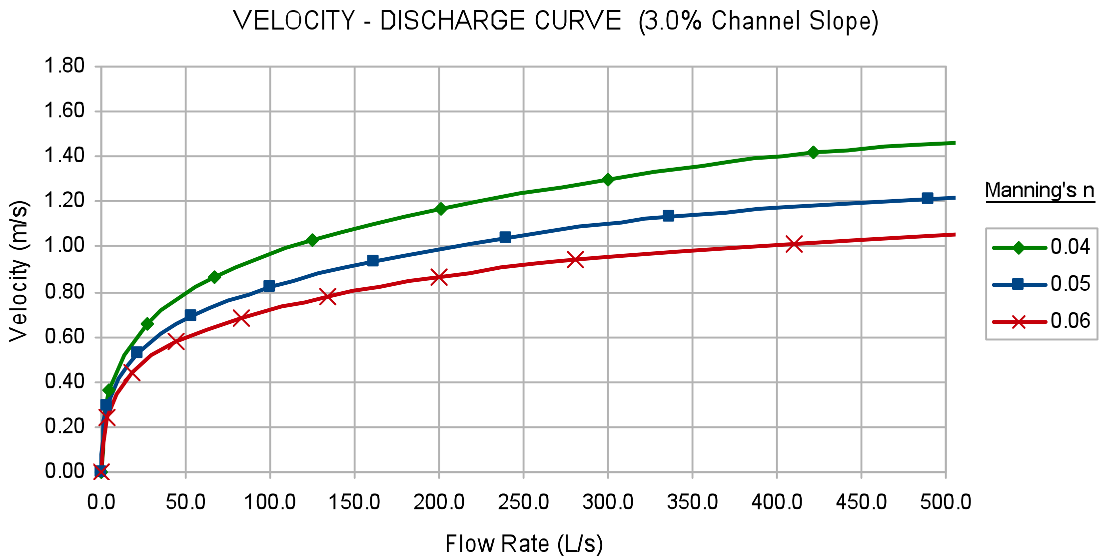


Figure 22. Velocity-discharge relationship for 3.0% uniform slope reaches

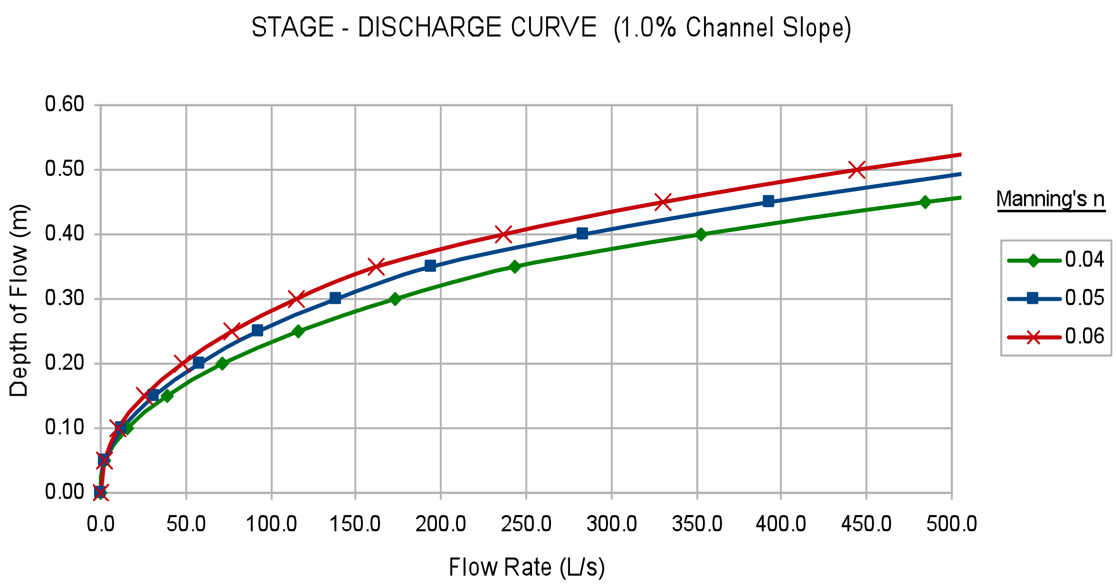


Figure 23. Stage-discharge relationships for 1.0% uniform slope reaches

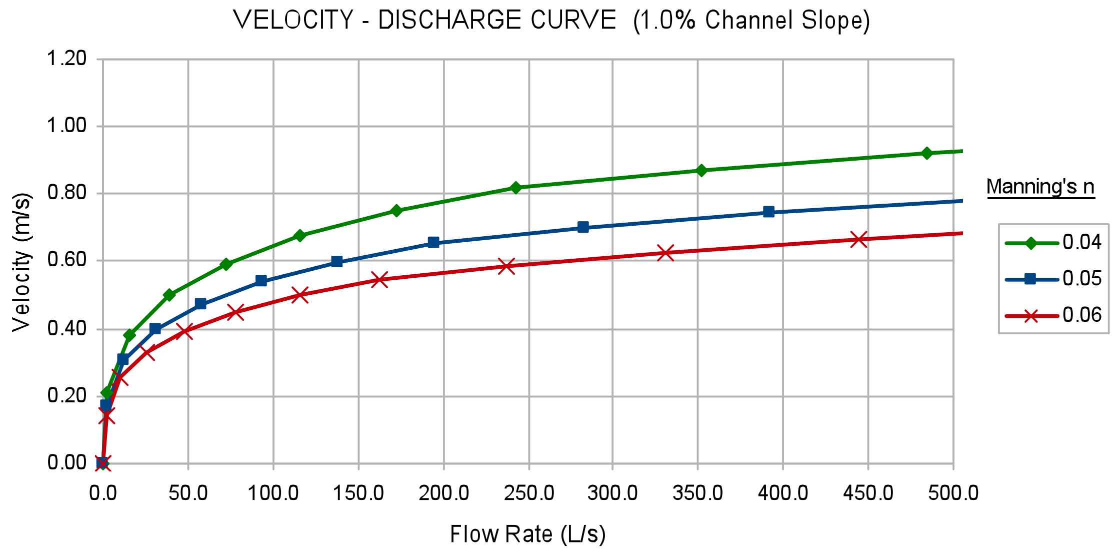


Figure 24. Stage-discharge relationship for 1.0% uniform slope reaches

Velocities shown in the charts above represent an estimate of mean channel velocity. Actual velocities will vary significantly both through the cross section as well as along the length of the stream, due to local changes in gradient, sinuosity, and habitat features. For reaches of the stream intended for fish habitat with gradients of 2.0% and steeper, a stepped riffle-pool or cascade-pool stream profile is recommended to enhance habitat values, with grade breaks reinforced with large wood debris and cobble/boulder arrangements designed to promote stability of granular substrate during high flows.

For these stepped reaches, critical flow may be expected to occur over the riffle crest, with a corresponding backwater condition in the pool habitat upstream. For preliminary design purposes, critical flow depths have been calculated based on triangular riffle geometry for depths less than 0.10 meters, and have been estimated using a rectangular channel approximation for depths greater than 0.10 meters. This approximation has been applied with the intention of providing a conservatively high estimate for the critical depth, to ensure that the estimated range of design discharge is contained within the active channel. Estimated critical depth and specific energy are plotted in the figure below for a range of flows up to 500 L/s.

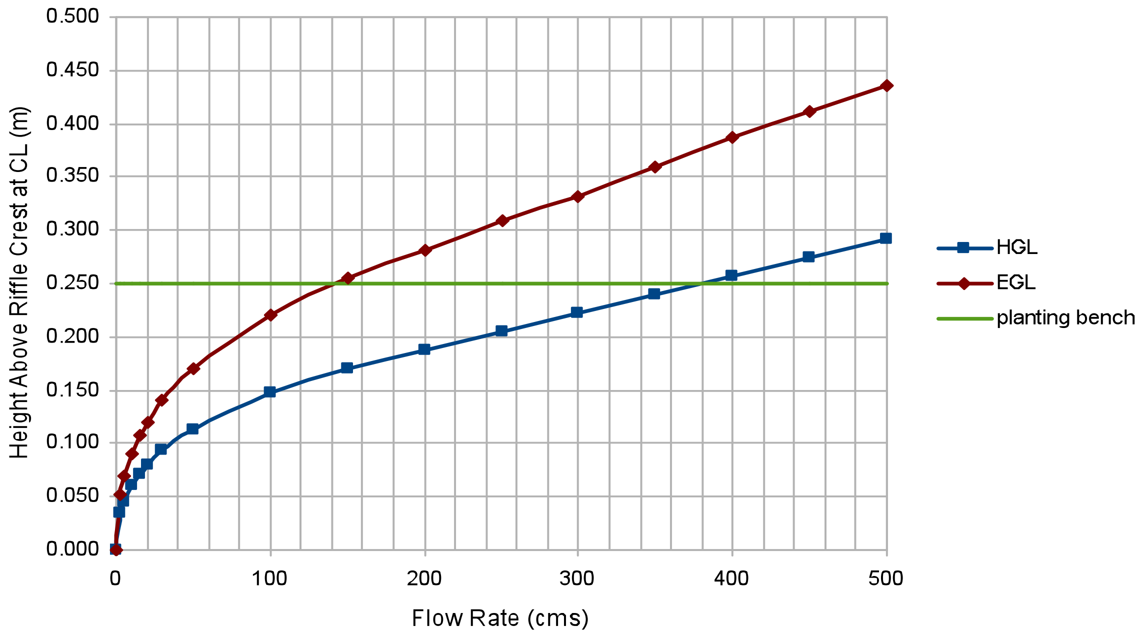
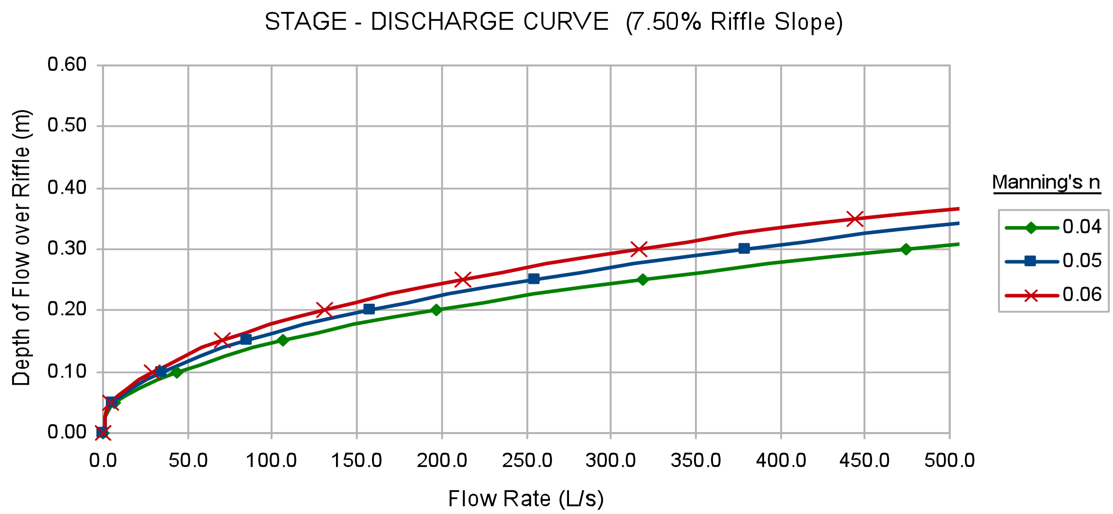
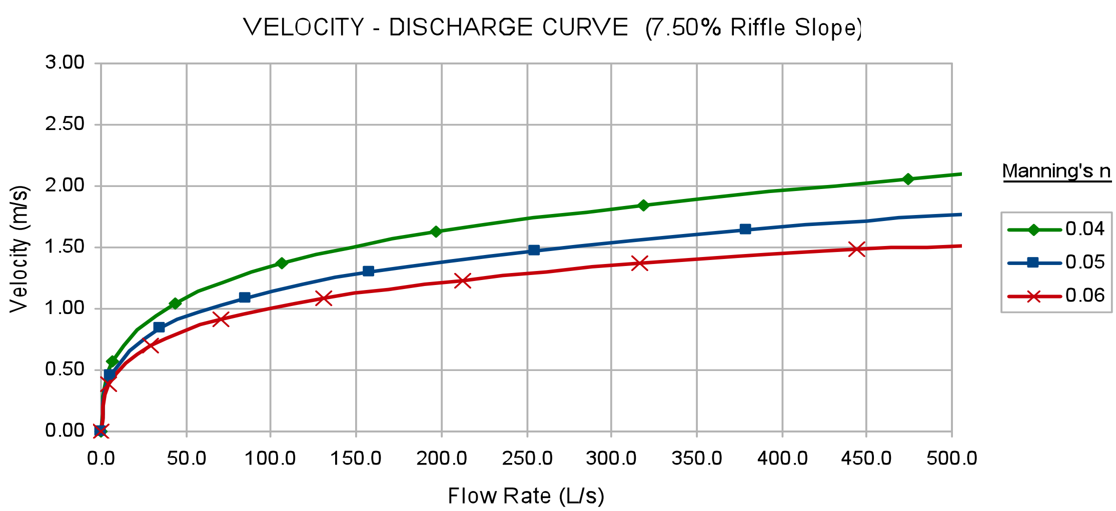


Figure 25. Estimated critical depth and specific energy over riffle crests

The downstream face of the riffle has been designed to incorporate a 0.3 metre drop over a length of 4.0 metres, for a slope of 7.50 %. Velocities through these short reaches will be significantly higher than elsewhere in the stream, and flow may be supercritical. Depending on the tailwater elevation in the pools and the roughness of the riffle face, a weak hydraulic jump may be expected to occur near the downstream end of the riffle for sufficiently high flow rates, providing beneficial aeration and energy dissipation through turbulent recirculation.

Estimated depth of flow, Froude number and flow velocity at the downstream face of the riffle are displayed in the figures below. Estimates are based on uniform flow conditions for a range of Manning n values and flow rates.


STAGE - DISCHARGE CURVE (7.50% Riffle Slope)
Depth of Flow over Riffle (m)
0.60
0.50
0.40
0.30
0.20
0.10
0.00
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
400.0
450.0
500.0
Flow Rate (L/s)
Manning's n
0.04
0.05
0.06


VELOCITY - DISCHARGE CURVE (7.50% Riffle Slope)
Velocity (m/s)
3.00
2.50
2.00
1.50
1.00
0.50
0.00
0.0
50.0
100.0
150.0
200.0
250.0
300.0
350.0
400.0
450.0
500.0
Flow Rate (L/s)
Manning's n
0.04
0.05
0.06

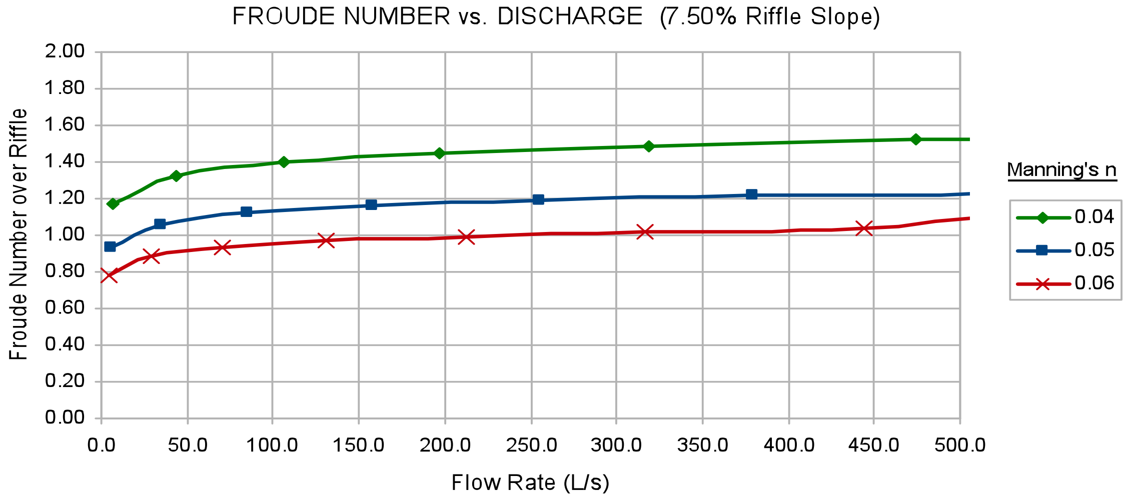


Figure 26. Stage-velocity-discharge relationship and Froude numbers for downstream face of riffles

13.0 Construction and Monitoring

It is important to note that construction for stream rehabilitation projects do not require detailed drawings and field verification. Typical stream rehabilitation projects in British Columbia have used approximate plan and profile drawings combined with surveying to direct machine operators and volunteers on where and how to construct stream features. *Figure 27* shows an example construction template provided to machine operators for construction of riffles. In the case of this project, riffles will be small and constructed of smaller materials, so construction will likely be performed by volunteers or paid workers by hand and similar drawings and pictures of natural riffles can be used to direct the constructors. *Figure 28* is an example of construction drawings used in previous stream restoration projects conducted in British Columbia and is an example of the required final design products for the UBC South Campus project.

1. PLAN: build riffle crest across the stream with large diameter boulders; back up with next largest stone downstream.
2. PROFILE: construct downstream face of riffle at a shallow re-entry slope that mimics local natural riffles (5:1 to 20:1).
3. SECTION: V-shape the crest and face downwards to the centre of the riffle (0.3 to 0.6 m).
4. SURFACE: place large rocks randomly on the downstream face 20 to 30cm apart to dissipate energy and create low flow fish passage channels.
5. BANKS: rip-rap both banks with embedded boulders and cobbles to the floodplain level.

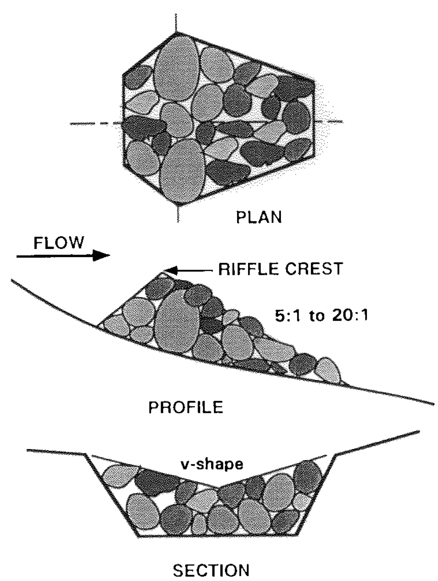


Figure 27. Example of construction drawings for machine operators to construct riffles (Newbury Hydraulics 1996)

It is recommended that the condition of the stream be monitored on a periodic basis following construction. Because the stream is intended to be a natural system, the condition may change over time with flood events of varying magnitude. Flow rate should be measured and recorded at the diversion outlet, at a convenient location mid-stream, and at the downstream outlet of the stream at Marine Drive to ensure adequate flow is available for aquatic life during all seasons of the year. Additionally, riffles and cascades should be visually inspected to ensure stability is maintained. It is important to monitor the stream conditions in the years following construction in order to mitigate erosion and channelization if it occurs.

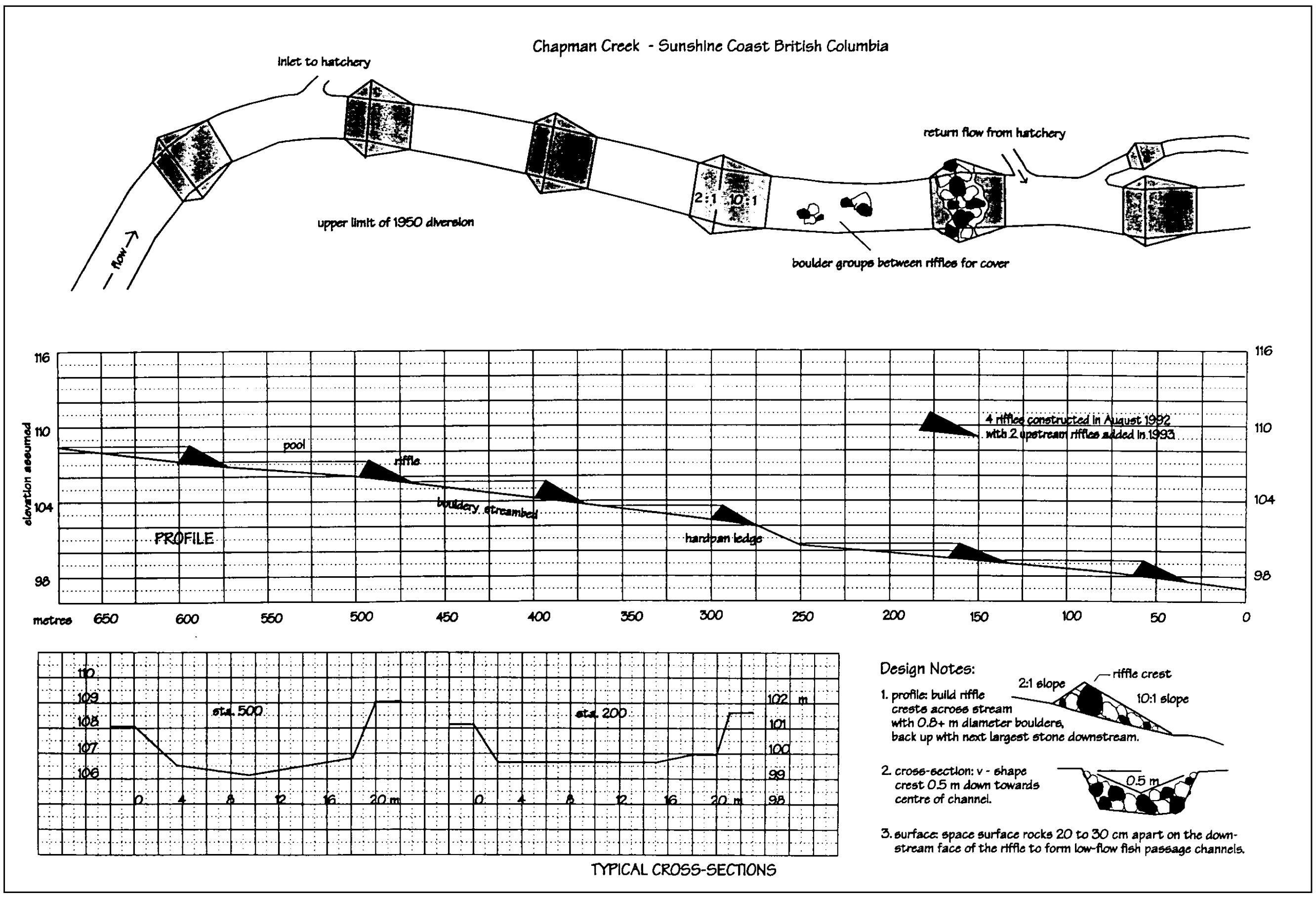


Figure 28. Condensed plan and profile drawings for the Chapman Creek Restoration Project (Newbury, Gaboury and Bates 1997)

14.0 Conclusions and Recommendations

The 2011 contribution to the UBC South Campus Urban Stream Restoration Project provided options for water supply to the stream, proposed a diversion location and structure, recommended a stream geometry, studied the hydraulic behavior of the stream, and recommended the work and data collection required to continue to the final stages of design. The South Campus Urban Stream Restoration Project now requires further data collection and analysis prior to the completion of a final detailed design. Stormwater system base flows must be measured and flow augmentation options, if required, must be compared and investigated to determine the most technically, socially, and economically feasible option for the UBC community. Public input and discussion between involved parties from UBC must also be considered in the decision making process in order to derive a solution that is truly in the best interests of the community as a whole.

With the information and guidance provided by this report, UBC will be able to continue building the design details for a constructed stream in the South Campus area. With the construction of the stream, the goals of the university's sustainability policy will be further realized in campus development and the social, aesthetic, and ecological value of the South Campus will be increased, continuing to make the UBC campus a model for sustainable development.

The following is a list of recommendations for further data to be collected and work to be completed in order to move the South Campus Urban Stream Restoration Project to the final design stage. The recommendations and next steps provided here are listed in the chronological order that they must be completed with a suggested timeline for completion.

Summer 2011:

- Complete base flow measurement for the existing South Campus stormwater drainage system for one dry season (*Section XX*)
- Measure soil permeability along proposed stream alignment to determine expected infiltration losses (*Section XX*)
- Complete water balance to determine if base flow augmentation is required (*Section XX*)
- Perform analysis to confirm the water balance of the existing Michael Smith Pond system (*Section XX*)

Fall 2011

- Analysis of existing manhole structure and developing a work plan/detailed design for the diversion piping and weir installation (*Section XX*)
- If required, perform field tests to collect information relating to flow augmentation options (*Section xx*)
- Public input meetings consultation with stakeholders

2012

- Student group to complete detailed design, including hydraulic modeling of existing stormwater system to confirm design calculations and system behavior

15.0 Works Cited

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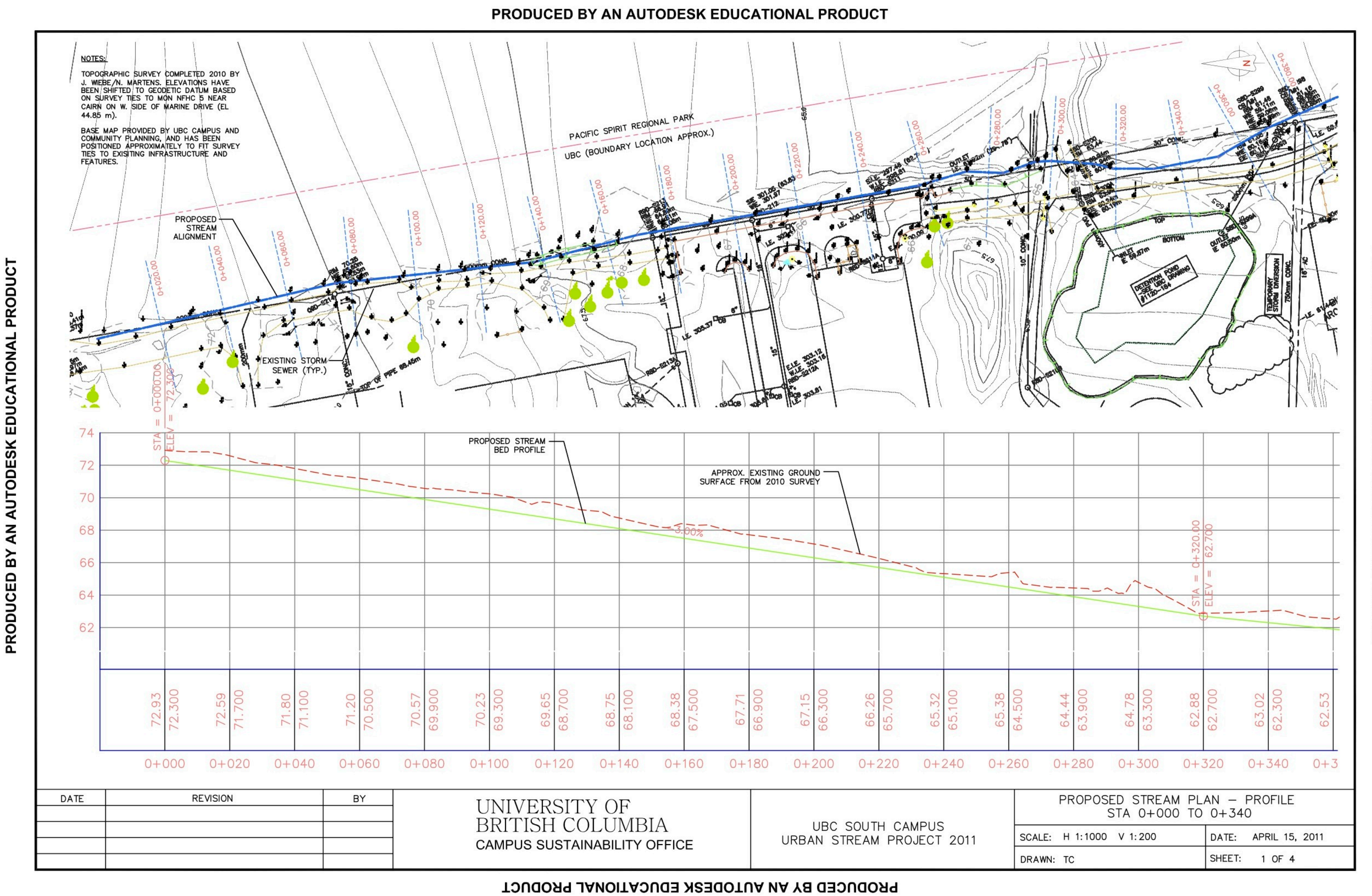
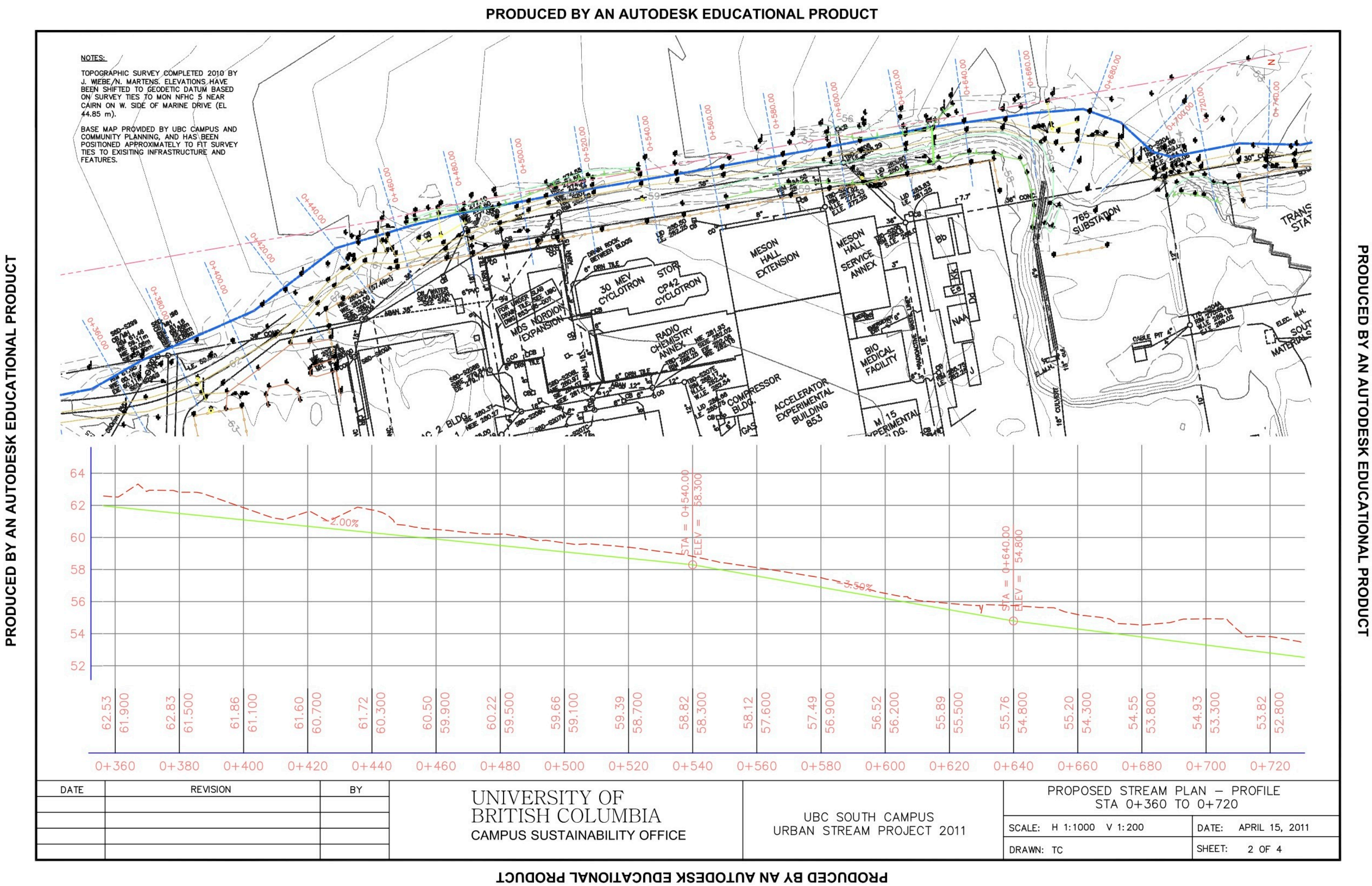
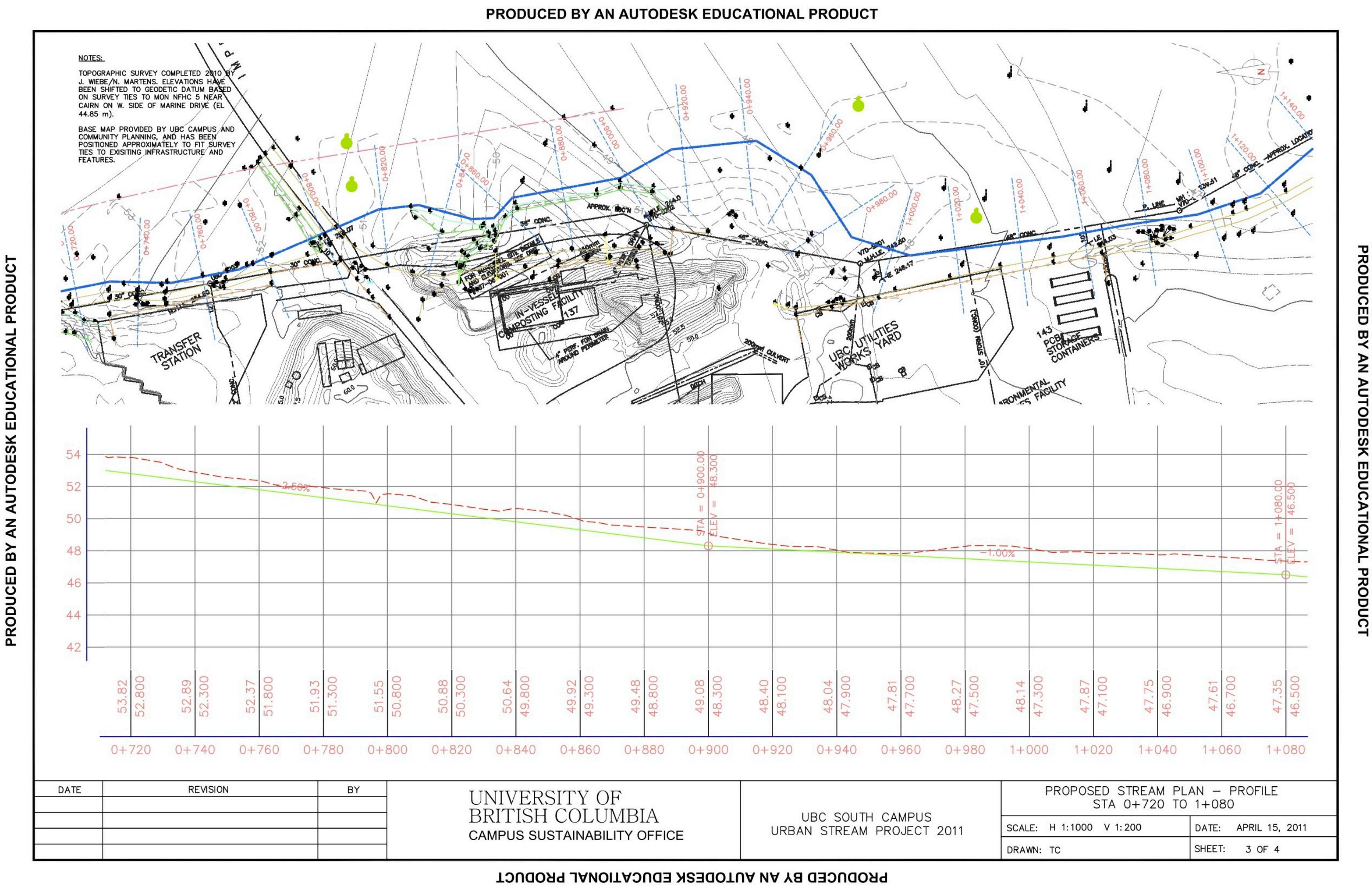
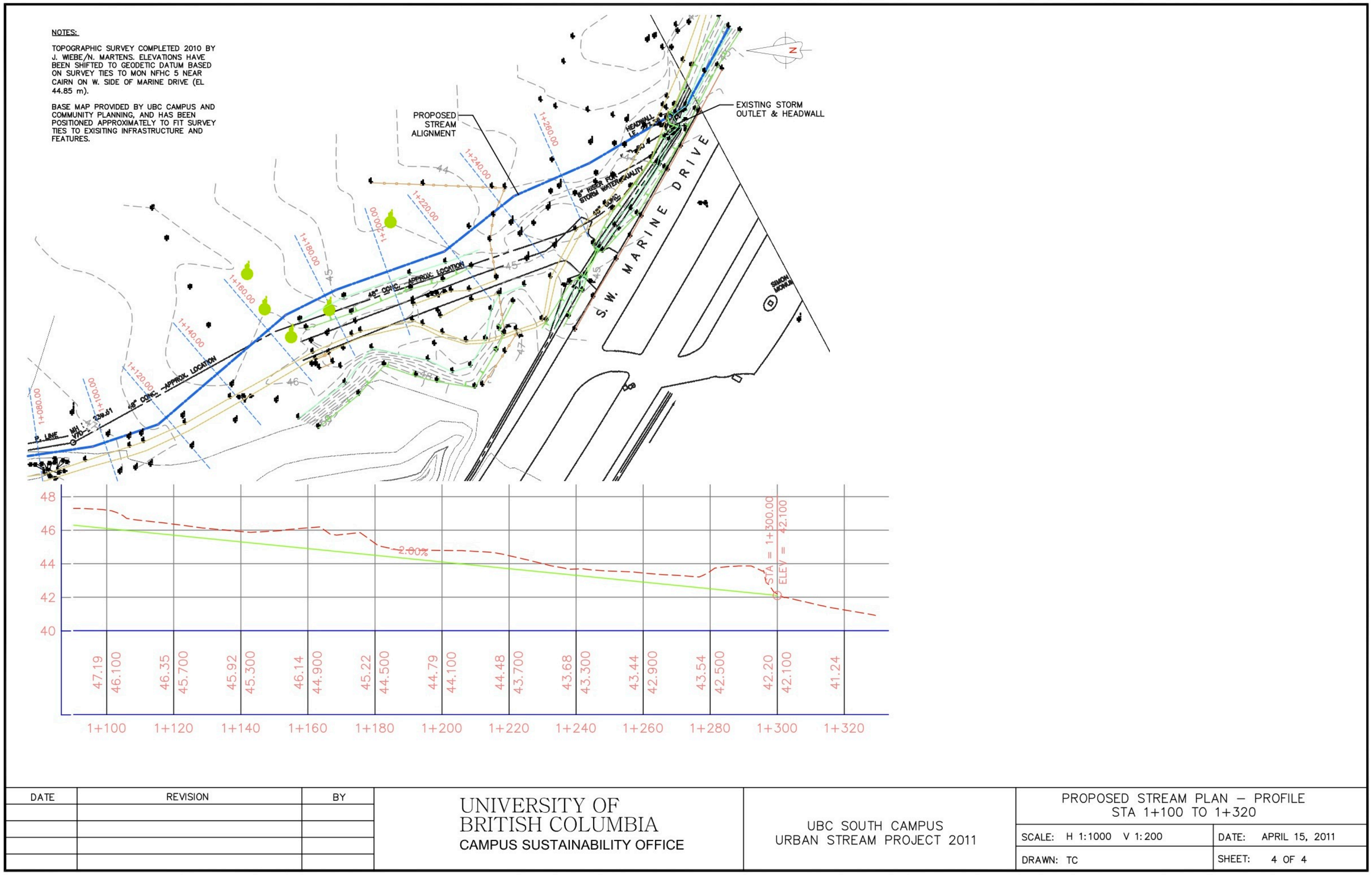
Appendix A: Plan and Profile Drawings




PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
NOTES:
TOPOGRAPHIC SURVEY COMPLETED 2010 BY J. WIEBE/N. MARTENS. ELEVATIONS HAVE BEEN SHIFTED TO GEODETIC DATUM BASED ON SURVEY TIES TO MON NFHC 5 NEAR CAIRN ON W. SIDE OF MARINE DRIVE (EL 44.85 m).
BASE MAP PROVIDED BY UBC CAMPUS AND COMMUNITY PLANNING, AND HAS BEEN POSITIONED APPROXIMATELY TO FIT SURVEY TIES TO EXISITING INFRASTRUCTURE AND FEATURES.
MESON HALL EXTENSION
MESON HALL SERVICE ANNEX
30 MEV CYCLOTRON
CP42 CYCLOTRON
RADIO CHEMISTRY ANNEX
BIO MEDICAL FACILITY
COMPRESSOR BLDG.
ACCELERATOR EXPERIMENTAL BUILDING 853
765 SUBSTATION
MDS NORDION EXPANSION
STA = 0+540.00
ELEV = 58.300
STA = 0+640.00
ELEV = 54.800
2.00%
3.50%
DATE
REVISION
BY
UNIVERSITY OF BRITISH COLUMBIA
CAMPUS SUSTAINABILITY OFFICE
UBC SOUTH CAMPUS URBAN STREAM PROJECT 2011
PROPOSED STREAM PLAN – PROFILE STA 0+360 TO 0+720
SCALE: H 1:1000 V 1:200
DATE: APRIL 15, 2011
DRAWN: TC
SHEET: 2 OF 4


PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
NOTES:
TOPOGRAPHIC SURVEY COMPLETED 2010 BY J. WIEBE/N. MARTENS. ELEVATIONS HAVE BEEN SHIFTED TO GEODETIC DATUM BASED ON SURVEY TIES TO MON NFHC 5 NEAR CAIRN ON W. SIDE OF MARINE DRIVE (EL 44.85 m).
BASE MAP PROVIDED BY UBC CAMPUS AND COMMUNITY PLANNING, AND HAS BEEN POSITIONED APPROXIMATELY TO FIT SURVEY TIES TO EXISITING INFRASTRUCTURE AND FEATURES.
TRANSFER STATION
IN-VESSEL COMPOSTING FACILITY
UBC UTILITIES WORKS YARD
PCB STORAGE CONTAINERS
STA = 0+900.00
ELEV = 48.300
STA = 1+080.00
ELEV = 46.500
-2.56%
-1.00%
54
52
50
48
46
44
42
53.82 52.800
52.89 52.300
52.37 51.800
51.93 51.300
51.55 50.800
50.88 50.300
50.64 49.800
49.92 49.300
49.48 48.800
49.08 48.300
48.40 48.100
48.04 47.900
47.81 47.700
48.27 47.500
48.14 47.300
47.87 47.100
47.75 46.900
47.61 46.700
47.35 46.500
0+720 0+740 0+760 0+780 0+800 0+820 0+840 0+860 0+880 0+900 0+920 0+940 0+960 0+980 1+000 1+020 1+040 1+060 1+080
DATE
REVISION
BY
UNIVERSITY OF BRITISH COLUMBIA
CAMPUS SUSTAINABILITY OFFICE
UBC SOUTH CAMPUS URBAN STREAM PROJECT 2011
PROPOSED STREAM PLAN – PROFILE STA 0+720 TO 1+080
SCALE: H 1:1000 V 1:200
DATE: APRIL 15, 2011
DRAWN: TC
SHEET: 3 OF 4
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT
PRODUCED BY AN AUTODESK EDUCATIONAL PRODUCT

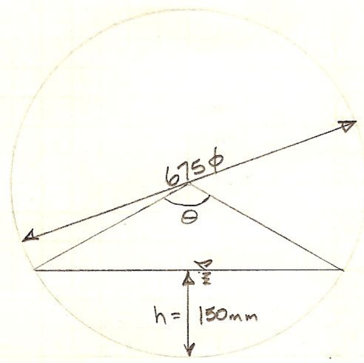

NOTES:
TOPOGRAPHIC SURVEY COMPLETED 2010 BY J. WIEBE/N. MARTENS. ELEVATIONS HAVE BEEN SHIFTED TO GEODETIC DATUM BASED ON SURVEY TIES TO MON NFHC 5 NEAR CAIRN ON W. SIDE OF MARINE DRIVE (EL 44.85 m).
BASE MAP PROVIDED BY UBC CAMPUS AND COMMUNITY PLANNING, AND HAS BEEN POSITIONED APPROXIMATELY TO FIT SURVEY TIES TO EXISITING INFRASTRUCTURE AND FEATURES.
PROPOSED STREAM ALIGNMENT
EXISTING STORM OUTLET & HEADWALL
S. W. MARINE DRIVE
2.00%
STA = 1+300.00
ELEV = 42.100
47.19 46.100
46.35 45.700
45.92 45.300
46.14 44.900
45.22 44.500
44.79 44.100
44.48 43.700
43.68 43.300
43.44 42.900
43.54 42.500
42.20 42.100
41.24
1+100 1+120 1+140 1+160 1+180 1+200 1+220 1+240 1+260 1+280 1+300 1+320
DATE
REVISION
BY
UNIVERSITY OF BRITISH COLUMBIA
CAMPUS SUSTAINABILITY OFFICE
UBC SOUTH CAMPUS URBAN STREAM PROJECT 2011
PROPOSED STREAM PLAN – PROFILE STA 1+100 TO 1+320
SCALE: H 1:1000 V 1:200
DATE: APRIL 15, 2011
DRAWN: TC
SHEET: 4 OF 4

Appendix B: Sample Calculations

Civl 498U | Sample Calculations

Partial Pipe Flow:

pipe length ~ 69 m
U/S invert = 68.28 m
D/S invert = 66.97 m
pipe slope = 1.9 %
mannings n = 0.013



$$\theta = 2\,\mathrm{Acos}\left(\frac{r-h}{r}\right) = 2\,\mathrm{Acos}\left(\frac{337.5-150}{337.5}\right) = 112.5°$$

$$\mathrm{rad} = \frac{\pi}{180}(112.5)° = 1.96\ \mathrm{RAD}$$

$$WP = r*\theta = 337.5 \cdot 1.96\ \mathrm{RAD} = 661.5\ mm = 0.66\ m$$

$$A = \frac{r^2(\theta - \sin\theta)}{2} = \frac{(0.3375)^2(1.96 - \sin 1.96)}{2} = 0.059\ m^2$$

$$R_H = \frac{A}{WP} = \frac{0.059 m^2}{0.66 m} = 0.089 m$$

$$V = \frac{1}{n} R_H^{2/3} S^{1/2} = \frac{1}{0.013}(0.089)^{2/3}(0.019)^{1/2} = 2.11\ m/s.$$

$$Q = AV = 0.059\ m^2\,(2.11\ m/s) = 0.125\ m^3/s = 125\ L/s.$$

Head in Manhole:

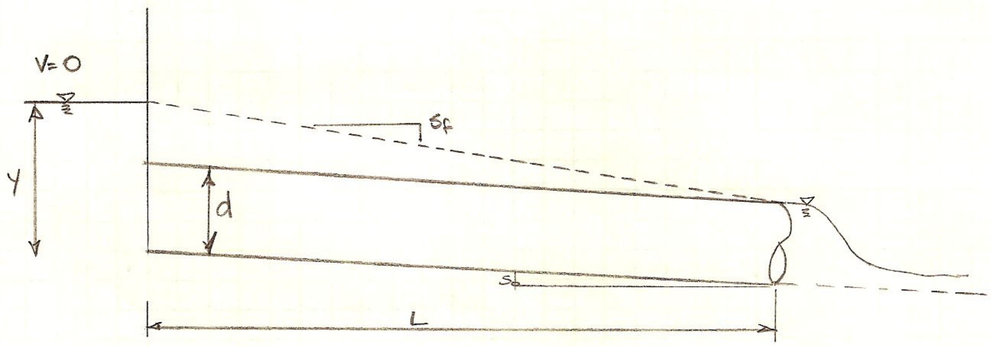
$$\text{Flowdepth}\ (h) + \frac{V^2}{2g} + \frac{kV^2}{2g}$$

entrance loss

$$0.150 + \frac{(2.11)^2}{2(9.81)} + \frac{0.9\,(2.11)^2}{2(9.81)} = 0.581\ m$$

Civil 498U | Sample Calculations

Full Pipe Flow (Inlet Submerged)



$$Q = \frac{1}{n} \cdot A \cdot R_h^{2/3} S_f^{1/2} \qquad n = 0.013$$

$$A = \frac{\pi D^2}{4} = \frac{\pi (0.150)^2}{4} = 0.01767\ m^2$$

$$R_h = \frac{A}{WP}$$

$$WP = \pi D = 0.471\ m$$

$$R_h = \frac{0.01767}{0.471} = 0.0375$$

$$S_f = \frac{Y-d}{L} = \frac{2.0-0.15}{235} = 0.00787$$

$$Q = \frac{1}{0.013}(0.01767)(0.0375)^{2/3}(0.00787)^{1/2} = \underline{\underline{0.014\ m^3/s}}$$

Civil 498 u | Sample Calculations

Weir Flow (Type 5) $h < h^*$

$$Fr = \frac{V}{\sqrt{g \cdot A/B}}$$

Critical Q occurs @ $Fr = 1$ ∴

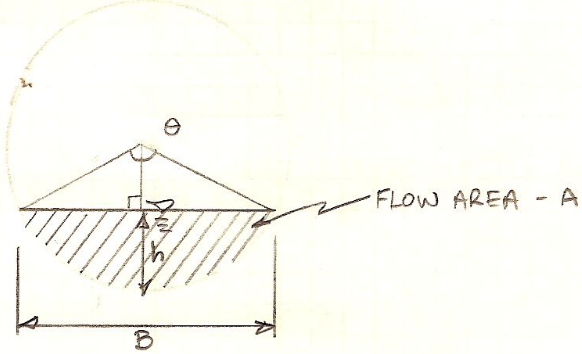
$$1 = \frac{V}{\sqrt{g \cdot A/B}} \qquad V = Q/A$$

$$1 = \frac{Q}{A\sqrt{g \cdot A/B}} \qquad Q = A\sqrt{g \cdot A/B}$$

to account for entrance losses multiply by contraction coefficient C_D

$$\therefore \quad Q = C_D \cdot A\sqrt{g \cdot A/B}$$

where : C_D = contraction coefficient (0.9)
A = flow cross sectional area
g = gravitational constant
B = free surface width.



$$\theta = 2\,\mathrm{Acos}\left(\frac{r-h}{r}\right)$$

$$B = \left[\tan(\theta/2)\,(r-h)\right](2)$$

Civil 498U | Sample Calculations

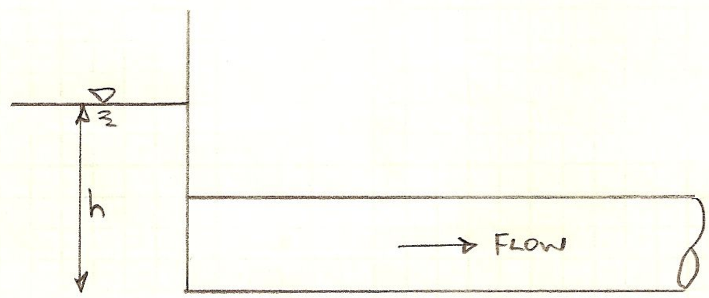
Orifice Flow :

$Q = AV \cdot K$

K = orifice coefficient (0,72)
A = cross sectional area
V = velocity

$\frac{V^2}{2g} = h \qquad V = \sqrt{2gh}$

$Q = K \cdot A \cdot \sqrt{2gh}$



$A = \frac{\pi D^2}{4}$

Civil 498U

Hydraulic Analysis
150 mm dia. Diversion Pipe

March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Invert Elevation (m)	71.905
Pipe Length (m)	235

Low Flow Outlet Diameter (m)	0.15
Pipe Roughness (n)	0.013
Pipe Slope (m/m)	0.005

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Hydraulic Head (m)	HGL Slope (m)	Theta (Rad)	Flow Area (m^2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Velocity (m/s)	Mannings Discharge (m^3/s)	Froude Number	Head in Manhole (m)
Partial Pipe Flow	73.08	0	1.175	0.005	0.000	0.000	0.000	0.000	0.000	0.000	0.00	0.00
	73.13	0.05	1.175	0.005	2.462	0.005	0.185	0.028	0.501	0.003	0.71	0.07
	73.155	0.075	1.175	0.005	3.142	0.009	0.236	0.038	0.609	0.005	0.71	0.11
	73.18	0.1	1.175	0.005	3.821	0.013	0.287	0.044	0.675	0.008	0.68	0.14
	73.23	0.15	1.175	0.005	6.283	0.018	0.471	0.038	0.609	0.011	0.50	0.19
Pressure Pipe Flow	73.28	0.2	1.225	0.005	-	0.018	0.471	0.038	0.622	0.011	0.51	0.24
	73.38	0.3	1.325	0.006	-	0.018	0.471	0.038	0.647	0.011	0.53	0.34
	73.48	0.4	1.425	0.006	-	0.018	0.471	0.038	0.671	0.012	0.55	0.44
	73.58	0.5	1.525	0.006	-	0.018	0.471	0.038	0.694	0.012	0.57	0.55
	73.68	0.6	1.625	0.007	-	0.018	0.471	0.038	0.717	0.013	0.59	0.65
	73.78	0.7	1.725	0.007	-	0.018	0.471	0.038	0.738	0.013	0.61	0.75
	73.88	0.8	1.825	0.008	-	0.018	0.471	0.038	0.759	0.013	0.63	0.86
	73.98	0.9	1.925	0.008	-	0.018	0.471	0.038	0.780	0.014	0.64	0.96
	74.08	1	2.025	0.009	-	0.018	0.471	0.038	0.800	0.014	0.66	1.06
	74.18	1.1	2.125	0.009	-	0.018	0.471	0.038	0.820	0.014	0.68	1.17
	74.28	1.2	2.225	0.009	-	0.018	0.471	0.038	0.839	0.015	0.69	1.27
	74.38	1.3	2.325	0.010	-	0.018	0.471	0.038	0.857	0.015	0.71	1.37
	74.48	1.4	2.425	0.010	-	0.018	0.471	0.038	0.875	0.015	0.72	1.47
	74.58	1.5	2.525	0.011	-	0.018	0.471	0.038	0.893	0.016	0.74	1.58
	74.68	1.6	2.625	0.011	-	0.018	0.471	0.038	0.911	0.016	0.75	1.68
	74.78	1.7	2.725	0.012	-	0.018	0.471	0.038	0.928	0.016	0.77	1.78
	74.88	1.8	2.825	0.012	-	0.018	0.471	0.038	0.945	0.017	0.78	1.89
	74.98	1.9	2.925	0.012	-	0.018	0.471	0.038	0.961	0.017	0.79	1.99
	75.08	2	3.025	0.013	-	0.018	0.471	0.038	0.978	0.017	0.81	2.09

Civil 498U

Hydraulic Analysis
200 mm dia. Diversion Pipe

March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Invert Elevation (m)	71.905
Pipe Length (m)	235

Low Flow Outlet Diameter (m)	0.2
Pipe Roughness (n)	0.013
Pipe Slope (m/m)	0.005

Calculations:

									Mannings				
	Water Surface Elevation (m)	**Flow Depth (m)**	**Hydraulic Head (m)**	**HGL Slope (m)**	**Theta (Rad)**	**Flow Area (m^2)**	**Wetted Perimeter (m)**	**Hydraulic Radius (m)**	**Velocity (m/s)**	**Discharge (m^3/s)**	**Froude Number**	**Head in Manhole (m)**
Pipe Flow	73.08	0	1.175	0.005	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	73.13	0.05	1.175	0.005	2.094	0.006	0.209	0.029	0.517	0.003	0.739	0.08
	73.18	0.1	1.175	0.005	3.142	0.016	0.314	0.050	0.738	0.012	0.745	0.15
	73.23	0.15	1.175	0.005	4.189	0.025	0.419	0.060	0.837	0.021	0.690	0.22
	73.28	0.2	1.175	0.005	6.283	0.031	0.628	0.050	0.738	0.023	0.527	0.25
Pressure Pipe Flow	73.38	0.3	1.275	0.005	-	0.031	0.628	0.050	0.769	0.024	0.549	0.36
	73.48	0.4	1.375	0.006	-	0.031	0.628	0.050	0.799	0.025	0.570	0.46
	73.58	0.5	1.475	0.006	-	0.031	0.628	0.050	0.827	0.026	0.590	0.57
	73.68	0.6	1.575	0.007	-	0.031	0.628	0.050	0.855	0.027	0.610	0.67
	73.78	0.7	1.675	0.007	-	0.031	0.628	0.050	0.881	0.028	0.629	0.78
	73.88	0.8	1.775	0.008	-	0.031	0.628	0.050	0.907	0.029	0.648	0.88
	73.98	0.9	1.875	0.008	-	0.031	0.628	0.050	0.933	0.029	0.666	0.98
	74.08	1	1.975	0.008	-	0.031	0.628	0.050	0.957	0.030	0.683	1.09
	74.18	1.1	2.075	0.009	-	0.031	0.628	0.050	0.981	0.031	0.700	1.19
	74.28	1.2	2.175	0.009	-	0.031	0.628	0.050	1.004	0.032	0.717	1.30
	74.38	1.3	2.275	0.010	-	0.031	0.628	0.050	1.027	0.032	0.733	1.40
	74.48	1.4	2.375	0.010	-	0.031	0.628	0.050	1.050	0.033	0.749	1.51
	74.58	1.5	2.475	0.011	-	0.031	0.628	0.050	1.071	0.034	0.765	1.61
	74.68	1.6	2.575	0.011	-	0.031	0.628	0.050	1.093	0.034	0.780	1.72
	74.78	1.7	2.675	0.011	-	0.031	0.628	0.050	1.114	0.035	0.795	1.82
	74.88	1.8	2.775	0.012	-	0.031	0.628	0.050	1.134	0.036	0.810	1.92
	74.98	1.9	2.875	0.012	-	0.031	0.628	0.050	1.155	0.036	0.824	2.03
	75.08	2	2.975	0.013	-	0.031	0.628	0.050	1.175	0.037	0.839	2.13

Civil 498U

Hydraulic Analysis
300 mm dia. Diversion Pipe

March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Invert Elevation (m)	71.91
Pipe Length (m)	235

Low Flow Outlet Diameter (m)	0.3
Pipe Roughness (n)	0.013
Pipe Slope (m/m)	0.005

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Hydraulic Head (m)	HGL Slope (m)	Theta (Rad)	Flow Area (m^2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Velocity (m/s)	Mannings Discharge (m^3/s)	Froude Number	Head in Manhole (m)
Pipe Flow	73.08	0	1.175	0.005	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	73.13	0.05	1.175	0.005	1.682	0.008	0.252	0.031	0.533	0.004	0.761	0.08
	73.18	0.1	1.175	0.005	2.462	0.021	0.369	0.056	0.795	0.016	0.802	0.16
	73.23	0.15	1.175	0.005	3.142	0.035	0.471	0.075	0.967	0.034	0.797	0.24
	73.28	0.2	1.075	0.005	3.821	0.050	0.573	0.087	1.024	0.051	0.731	0.30
	73.38	0.3	1.175	0.005	6.283	0.071	0.942	0.075	0.967	0.068	0.564	0.39
Pressure Pipe Flow	73.48	0.4	1.275	0.005		0.071	0.942	0.075	1.008	0.071	0.587	0.50
	73.58	0.5	1.375	0.006		0.071	0.942	0.075	1.046	0.074	0.610	0.61
	73.68	0.6	1.475	0.006		0.071	0.942	0.075	1.084	0.077	0.632	0.71
	73.78	0.7	1.575	0.007		0.071	0.942	0.075	1.120	0.079	0.653	0.82
	73.88	0.8	1.675	0.007		0.071	0.942	0.075	1.155	0.082	0.673	0.93
	73.98	0.9	1.775	0.008		0.071	0.942	0.075	1.189	0.084	0.693	1.04
	74.08	1	1.875	0.008		0.071	0.942	0.075	1.222	0.086	0.712	1.14
	74.18	1.1	1.975	0.008		0.071	0.942	0.075	1.254	0.089	0.731	1.25
	74.28	1.2	2.075	0.009		0.071	0.942	0.075	1.286	0.091	0.749	1.36
	74.38	1.3	2.175	0.009		0.071	0.942	0.075	1.316	0.093	0.767	1.47
	74.48	1.4	2.275	0.010		0.071	0.942	0.075	1.346	0.095	0.785	1.58
	74.58	1.5	2.375	0.010		0.071	0.942	0.075	1.375	0.097	0.802	1.68
	74.68	1.6	2.475	0.011		0.071	0.942	0.075	1.404	0.099	0.818	1.79
	74.78	1.7	2.575	0.011		0.071	0.942	0.075	1.432	0.101	0.835	1.90
	74.88	1.8	2.675	0.011		0.071	0.942	0.075	1.460	0.103	0.851	2.01
	74.98	1.9	2.775	0.012		0.071	0.942	0.075	1.487	0.105	0.867	2.11
	75.08	2	2.875	0.012		0.071	0.942	0.075	1.513	0.107	0.882	2.22

Civil 498U

Hydraulic Analysis
375 mm dia. Diversion Pipe

March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Invert Elevation (m)	71.905
Pipe Length (m)	235
Low Flow Outlet Diameter (m)	0.375
Pipe Roughness (n)	0.013
Pipe Slope (m/m)	0.005

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Hydraulic Head (m)	HGL Slope (m)	Theta (Rad)	Flow Area (m^2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Mannings Velocity (m/s)	Mannings Discharge (m^3/s)	Froude Number	Head in Manhole (m)
Pipe Flow	73.08	0	1.175	0.005	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	73.13	0.05	1.175	0.005	1.495	0.009	0.280	0.031	0.539	0.005	0.770	0.08
	73.18	0.1	1.175	0.005	2.171	0.024	0.407	0.058	0.816	0.019	0.824	0.16
	73.23	0.15	1.175	0.005	2.739	0.041	0.514	0.080	1.013	0.042	0.835	0.25
	73.28	0.2	1	0.004	3.275	0.060	0.614	0.098	1.063	0.064	0.759	0.31
	73.38	0.3	1.1	0.005	4.429	0.095	0.830	0.114	1.238	0.117	0.722	0.45
	73.455	0.375	1.175	0.005	6.283	0.110	1.178	0.094	1.123	0.124	0.585	0.50
Pressure Pipe Flow	73.48	0.4	1.2	0.005	-	0.110	1.178	0.094	1.134	0.125	0.591	0.52
	73.58	0.5	1.3	0.006	-	0.110	1.178	0.094	1.181	0.130	0.616	0.64
	73.68	0.6	1.4	0.006	-	0.110	1.178	0.094	1.225	0.135	0.639	0.75
	73.78	0.7	1.5	0.006	-	0.110	1.178	0.094	1.268	0.140	0.661	0.86
	73.88	0.8	1.6	0.007	-	0.110	1.178	0.094	1.310	0.145	0.683	0.97
	73.98	0.9	1.7	0.007	-	0.110	1.178	0.094	1.350	0.149	0.704	1.08
	74.08	1	1.8	0.008	-	0.110	1.178	0.094	1.389	0.153	0.724	1.19
	74.18	1.1	1.9	0.008	-	0.110	1.178	0.094	1.427	0.158	0.744	1.30
	74.28	1.2	2	0.009	-	0.110	1.178	0.094	1.464	0.162	0.764	1.41
	74.38	1.3	2.1	0.009	-	0.110	1.178	0.094	1.501	0.166	0.782	1.52
	74.48	1.4	2.2	0.009	-	0.110	1.178	0.094	1.536	0.170	0.801	1.63
	74.58	1.5	2.3	0.010	-	0.110	1.178	0.094	1.570	0.173	0.819	1.74
	74.68	1.6	2.4	0.010	-	0.110	1.178	0.094	1.604	0.177	0.836	1.85
	74.78	1.7	2.5	0.011	-	0.110	1.178	0.094	1.637	0.181	0.854	1.96
	74.88	1.8	2.6	0.011	-	0.110	1.178	0.094	1.670	0.184	0.871	2.07
	74.98	1.9	2.7	0.011	-	0.110	1.178	0.094	1.702	0.188	0.887	2.18
	75.08	2	2.8	0.012	-	0.110	1.178	0.094	1.733	0.191	0.903	2.29

Civil 498U

Hydraulic Analysis
450 mm dia. Diversion Pipe

March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Invert Elevation (m)	71.905
Pipe Length (m)	235

Low Flow Outlet Diameter (m)	0.45
Pipe Roughness (n)	0.013
Pipe Slope (m/m)	0.005

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Hydraulic Head (m)	HGL Slope (m)	Theta (Rad)	Flow Area (m^2)	Wetted Perimeter (m)	Hydraulic Radius (m)	Velocity (m/s)	Mannings Discharge (m^3/s)	Froude Number	Head in Manhole (m)
Pipe Flow	73.08	0	1.175	0.005	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.00
	73.13	0.05	1.175	0.005	1.359	0.010	0.306	0.032	0.543	0.005	0.776	0.08
	73.18	0.1	1.175	0.005	1.964	0.026	0.442	0.060	0.830	0.022	0.838	0.17
	73.23	0.15	1.175	0.005	2.462	0.046	0.554	0.084	1.041	0.048	0.859	0.26
	73.28	0.2	0.925	0.004	2.919	0.068	0.657	0.104	1.067	0.073	0.762	0.31
	73.38	0.3	1.025	0.004	3.821	0.113	0.860	0.131	1.310	0.148	0.764	0.47
	73.455	0.375	1.1	0.005	4.601	0.142	1.035	0.137	1.397	0.198	0.728	0.56
	73.48	0.4	1.125	0.005	4.924	0.149	1.108	0.135	1.400	0.209	0.706	0.59
	73.53	0.45	1.175	0.005	6.283	0.159	1.414	0.113	1.268	0.202	0.603	0.61
Pressure Pipe Flow	73.58	0.5	1.225	0.005	-	0.159	1.414	0.113	1.294	0.206	0.616	0.66
	73.68	0.6	1.325	0.006	-	0.159	1.414	0.113	1.346	0.214	0.641	0.78
	73.78	0.7	1.425	0.006	-	0.159	1.414	0.113	1.396	0.222	0.664	0.89
	73.88	0.8	1.525	0.006	-	0.159	1.414	0.113	1.444	0.230	0.687	1.00
	73.98	0.9	1.625	0.007	-	0.159	1.414	0.113	1.491	0.237	0.709	1.12
	74.08	1	1.725	0.007	-	0.159	1.414	0.113	1.536	0.244	0.731	1.23
	74.18	1.1	1.825	0.008	-	0.159	1.414	0.113	1.580	0.251	0.752	1.34
	74.28	1.2	1.925	0.008	-	0.159	1.414	0.113	1.622	0.258	0.772	1.45
	74.38	1.3	2.025	0.009	-	0.159	1.414	0.113	1.664	0.265	0.792	1.57
	74.48	1.4	2.125	0.009	-	0.159	1.414	0.113	1.705	0.271	0.811	1.68
	74.58	1.5	2.225	0.009	-	0.159	1.414	0.113	1.744	0.277	0.830	1.79
	74.68	1.6	2.325	0.010	-	0.159	1.414	0.113	1.783	0.284	0.849	1.91
	74.78	1.7	2.425	0.010	-	0.159	1.414	0.113	1.821	0.290	0.867	2.02
	74.88	1.8	2.525	0.011	-	0.159	1.414	0.113	1.858	0.296	0.884	2.13
	74.98	1.9	2.625	0.011	-	0.159	1.414	0.113	1.895	0.301	0.902	2.25
	75.08	2	2.725	0.012	-	0.159	1.414	0.113	1.930	0.307	0.919	2.36

Civil 498U | Existing Outlet Capacity Analysis (No Weir) | March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Manhole Invert Elevation (m)	70.91
Pipe Length (m)	60.00
Outlet Diameter (m)	0.675
Weir Loss Coefficient	0.90
Orifice Loss Coefficient	0.72

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Theta (RAD)	Flow Area - A (m^2)	Free Surface Width - B (m)	Ratio A/B	WEIR Discharge (m^3/s)	WEIR Velocity (m/s)	Orifice Discharge (m^3/s)	Orifice Velocity (m/s)
Weir Flow	73.08	0.000	0.00	0.000	0.000	0.00	0.000	0.000	-	-
	73.13	0.050	1.10	0.012	0.354	0.03	0.006	0.519	-	-
	73.18	0.100	1.58	0.033	0.480	0.07	0.024	0.740	-	-
	73.23	0.150	1.96	0.059	0.561	0.11	0.054	0.916	-	-
	73.28	0.200	2.30	0.089	0.616	0.14	0.095	1.070	-	-
	73.38	0.300	2.92	0.154	0.671	0.23	0.207	1.349	-	-
	73.48	0.400	3.51	0.221	0.663	0.33	0.359	1.627	0.445	2.02
	73.58	0.500	4.15	0.284	0.592	0.48	0.555	1.954	0.641	2.26
	73.68	0.600	4.92	0.336	0.424	0.79	0.843	2.509	0.830	2.47
	73.73	0.650	5.51	0.354	0.255	1.39	1.174	3.320	0.909	2.57
Orifice Flow	73.78	0.700	-	0.358	-	-	-	-	0.955	2.67
	73.83	0.750	-	0.358	-	-	-	-	0.988	2.76
	73.88	0.800	-	0.358	-	-	-	-	1.021	2.85
	73.98	0.900	-	0.358	-	-	-	-	1.083	3.03
	74.08	1.000	-	0.358	-	-	-	-	1.141	3.19
	74.18	1.100	-	0.358	-	-	-	-	1.197	3.34
	74.28	1.200	-	0.358	-	-	-	-	1.250	3.49
	74.38	1.300	-	0.358	-	-	-	-	1.301	3.64
	74.48	1.400	-	0.358	-	-	-	-	1.350	3.77
	74.58	1.500	-	0.358	-	-	-	-	1.398	3.91
	74.68	1.600	-	0.358	-	-	-	-	1.444	4.03
	74.78	1.700	-	0.358	-	-	-	-	1.488	4.16
	74.88	1.800	-	0.358	-	-	-	-	1.531	4.28
	74.98	1.900	-	0.358	-	-	-	-	1.573	4.40
	75.08	2.000	-	0.358	-	-	-	-	1.614	4.51

Civil 498U | Existing Outlet Capacity Analysis (50mm High Weir) | March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Manhole Invert Elevation (m)	70.91
Pipe Length (m)	60.00
Outlet Diameter (m)	0.675
Weir Loss Coefficient	0.90
Weir Cross Sectional Area (m^2)	0.012
Orifice Loss Coefficient	0.72

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Theta (RAD)	Flow Area - A (m^2)	Free Surface Width - B (m)	Ratio A/B	WEIR Discharge (m^3/s)	WEIR Velocity (m/s)	Orifice Discharge (m^3/s)	Orifice Velocity (m/s)
Weir Flow	73.08	0.000	0.00	0.000	0.000	0.00	0.000	0.000	-	-
	73.13	0.050	1.10	0.000	0.354	0.00	0.000	0.000	-	-
	73.18	0.100	1.58	0.021	0.480	0.04	0.012	0.591	-	-
	73.23	0.150	1.96	0.047	0.561	0.08	0.039	0.818	-	-
	73.28	0.200	2.30	0.077	0.616	0.12	0.076	0.995	-	-
	73.38	0.300	2.92	0.142	0.671	0.21	0.184	1.296	-	-
	73.48	0.400	3.51	0.209	0.663	0.31	0.330	1.582	0.421	2.02
	73.58	0.500	4.15	0.272	0.592	0.46	0.521	1.912	0.614	2.26
	73.68	0.600	4.92	0.324	0.424	0.76	0.799	2.464	0.801	2.47
	73.73	0.650	5.51	0.342	0.255	1.34	1.115	3.263	0.878	2.57
Orifice Flow	73.78	0.700	-	0.346	-	-	-	-	0.923	2.67
	73.83	0.750	-	0.346	-	-	-	-	0.955	2.76
	73.88	0.800	-	0.346	-	-	-	-	0.987	2.85
	73.98	0.900	-	0.346	-	-	-	-	1.046	3.03
	74.08	1.000	-	0.346	-	-	-	-	1.103	3.19
	74.18	1.100	-	0.346	-	-	-	-	1.157	3.34
	74.28	1.200	-	0.346	-	-	-	-	1.208	3.49
	74.38	1.300	-	0.346	-	-	-	-	1.258	3.64
	74.48	1.400	-	0.346	-	-	-	-	1.305	3.77
	74.58	1.500	-	0.346	-	-	-	-	1.351	3.91
	74.68	1.600	-	0.346	-	-	-	-	1.395	4.03
	74.78	1.700	-	0.346	-	-	-	-	1.438	4.16
	74.88	1.800	-	0.346	-	-	-	-	1.480	4.28
	74.98	1.900	-	0.346	-	-	-	-	1.520	4.40
	75.08	2.000	-	0.346	-	-	-	-	1.560	4.51

Civil 498U | Existing Outlet Capacity Analysis (100mm High Weir) | March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Manhole Invert Elevation (m)	70.91
Pipe Length (m)	60.00
Outlet Diameter (m)	0.675
Weir Loss Coefficient	0.90
Weir Cross Sectional Area (m^2)	0.033
Orifice Loss Coefficient	0.72

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Theta (RAD)	Flow Area - A (m^2)	Free Surface Width - B (m)	Ratio A/B	WEIR Discharge (m^3/s)	WEIR Velocity (m/s)	Orifice Discharge (m^3/s)	Orifice Velocity (m/s)
Weir Flow	73.08	0.000	0.00	0.000	0.000	0.00	0.000	0.000	-	-
	73.13	0.050	1.10	0.000	0.354	0.00	0.000	0.000	-	-
	73.18	0.100	1.58	0.000	0.480	0.00	0.000	0.000	-	-
	73.23	0.150	1.96	0.026	0.561	0.05	0.016	0.609	-	-
	73.28	0.200	2.30	0.056	0.616	0.09	0.047	0.847	-	-
	73.38	0.300	2.92	0.121	0.671	0.18	0.144	1.195	-	-
	73.48	0.400	3.51	0.188	0.663	0.28	0.282	1.500	0.379	2.02
	73.58	0.500	4.15	0.251	0.592	0.42	0.461	1.837	0.566	2.26
	73.68	0.600	4.92	0.303	0.424	0.71	0.722	2.382	0.749	2.47
	73.73	0.650	5.51	0.321	0.255	1.26	1.013	3.161	0.824	2.57
Orifice Flow	73.78	0.700	-	0.325	-	-	-	-	0.867	2.67
	73.83	0.750	-	0.325	-	-	-	-	0.897	2.76
	73.88	0.800	-	0.325	-	-	-	-	0.926	2.85
	73.98	0.900	-	0.325	-	-	-	-	0.983	3.03
	74.08	1.000	-	0.325	-	-	-	-	1.036	3.19
	74.18	1.100	-	0.325	-	-	-	-	1.086	3.34
	74.28	1.200	-	0.325	-	-	-	-	1.135	3.49
	74.38	1.300	-	0.325	-	-	-	-	1.181	3.64
	74.48	1.400	-	0.325	-	-	-	-	1.226	3.77
	74.58	1.500	-	0.325	-	-	-	-	1.269	3.91
	74.68	1.600	-	0.325	-	-	-	-	1.310	4.03
	74.78	1.700	-	0.325	-	-	-	-	1.351	4.16
	74.88	1.800	-	0.325	-	-	-	-	1.390	4.28
	74.98	1.900	-	0.325	-	-	-	-	1.428	4.40
	75.08	2.000	-	0.325	-	-	-	-	1.465	4.51

Civil 498U | Existing Outlet Capacity Analysis (150mm High Weir) | March 2011

Input Data:

Invert Elevation (m)	73.08
Manhole Rim Elevation (m)	78.45
D/S Manhole Invert Elevation (m)	70.91
Pipe Length (m)	60.00
Outlet Diameter (m)	0.675
Weir Loss Coefficient	0.90
Weir Cross Sectional Area (m^2)	0.059
Orifice Loss Coefficient	0.72

Calculations:

	Water Surface Elevation (m)	Flow Depth (m)	Theta (RAD)	Flow Area - A (m^2)	Free Surface Width - B (m)	Ratio A/B	WEIR Discharge (m^3/s)	WEIR Velocity (m/s)	Orifice Discharge (m^3/s)	Orifice Velocity (m/s)
Weir Flow	73.08	0.000	0.00	0.000	0.000	0.00	0.000	0.000	-	-
	73.13	0.050	1.10	0.000	0.354	0.00	0.000	0.000	-	-
	73.18	0.100	1.58	0.000	0.480	0.00	0.000	0.000	-	-
	73.23	0.150	1.96	0.000	0.561	0.00	0.000	0.000	-	-
	73.28	0.200	2.30	0.030	0.616	0.05	0.018	0.617	-	-
	73.38	0.300	2.92	0.094	0.671	0.14	0.100	1.058	-	-
	73.48	0.400	3.51	0.162	0.663	0.24	0.225	1.392	0.326	2.02
	73.58	0.500	4.15	0.225	0.592	0.38	0.391	1.738	0.507	2.26
	73.68	0.600	4.92	0.277	0.424	0.65	0.631	2.277	0.684	2.47
	73.73	0.650	5.51	0.294	0.255	1.15	0.892	3.029	0.757	2.57
Orifice Flow	73.78	0.700	-	0.299	-	-	-	-	0.797	2.67
	73.83	0.750	-	0.299	-	-	-	-	0.825	2.76
	73.88	0.800	-	0.299	-	-	-	-	0.852	2.85
	73.98	0.900	-	0.299	-	-	-	-	0.904	3.03
	74.08	1.000	-	0.299	-	-	-	-	0.952	3.19
	74.18	1.100	-	0.299	-	-	-	-	0.999	3.34
	74.28	1.200	-	0.299	-	-	-	-	1.043	3.49
	74.38	1.300	-	0.299	-	-	-	-	1.086	3.64
	74.48	1.400	-	0.299	-	-	-	-	1.127	3.77
	74.58	1.500	-	0.299	-	-	-	-	1.166	3.91
	74.68	1.600	-	0.299	-	-	-	-	1.205	4.03
	74.78	1.700	-	0.299	-	-	-	-	1.242	4.16
	74.88	1.800	-	0.299	-	-	-	-	1.278	4.28
	74.98	1.900	-	0.299	-	-	-	-	1.313	4.40
	75.08	2.000	-	0.299	-	-	-	-	1.347	4.51