Innovative Approach for Urban Stream Restoration

Undergraduate Thesis
CHBE 494

Kosta Sainis

Thesis Advisor
Dr. Royann Petrell

Submitted: April 7, 2006

University of British Columbia Chemical and Biological Engineering

EXECUTIVE SUMMARY

The novelty of this study is the design of a stream which is able to manage stormwater as well as provide fish habitat. A design methodology which can be used to develop a pond and stream system suitable for the University of British Columbia South Campus Neighbourhood and fish habitat was developed.

Federal and provincial guidelines recommend that post development runoff volumes roughly equal pre-development runoff volume, and that the total runoff volume is limited to 10% or less of the total rainfall volume. With the addition of the pond and stream system, post development runoff volumes differed from pre development runoff volumes by an average of 15%. Although the post development runoff volumes did not equal the predevelopment runoff volumes, the 6% decrease in runoff volume is a significant step towards achieving post development runoff volumes, which roughly equal predevelopment runoff volumes. The pond and stream system was unable to limit the total runoff volume to 10% or less of the total rainfall volume. The pond and stream system was only able to limit the total runoff volume to approximately 78% of the total rainfall volume. This guideline may have been difficult to achieve since the pond and stream system is only taking runoff from a small area of rooftops in comparison to the large area, which makes up the South Campus Neighbourhood.

The implication of this study is that an urban stream, which is able to manage stormwater as well as conform to fish habitat criteria, will flow through the University of British Columbia campus. This will restore the fish-bearing stream, which once flowed through campus. The vision is that fish will one day utilize the stream as habitat and spawning areas. Implementation of such a stream and pond system will also help the University of British Columbia to strive towards meeting federal and provincial stormwater management guidelines. On a larger scale, the research promotes sustainability through fish habitat conservation and efficient use of water resources.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF TABLES

1.0 INTRODUCTION

Several natural streams in British Columbia have vanished or have been severely altered (DFO 2005). Urbanization is one of the primary factors contributing to this decline (Hunter 1991). The demand for urban development increases with human population growth (Huth 1978). Unfortunately fish populations and stream habitat has diminished as a result (DFO 2005). As well, water table levels have dropped forcing issues with summer time irrigation and increased stormwater management costs in some areas (Finkenbine 1998). Now is the time to incorporate streams with development plans to mitigate losses, reduce costs, and help restore valuable fisheries resources for future generations.

Interestingly, a stream suitable for fish has fewer issues related to water table levels (Moyle and Chech 2004). A stream properly designed to receive stormwater directly and indirectly, may lower stormwater management costs (MWLAP 2004). To improve fish habitat in an existing stream, existing stream rehabilitation techniques can be used for planning and implementation (Johnston and Slaney 1996). Restoration of an extinct stream however, requires alternative techniques since water supply is no longer available (Johnston and Slaney 1996). Urban stream restoration is an increasingly popular method of reintroducing stream habitat (Hunter 1991). Although many methods of urban stream restoration have been developed, few have been designed to suit fish habitat or collect clean runoff as a means to sustain stream flow.

At the University of British Columbia, there is now the unique opportunity to restore a stream that once flowed through it (Figure 1). This stream is part of the South Campus neighbourhood, which is planned to be developed into a residential community in early 2006 (UBC 2004) (Figure 2). The South Campus neighbourhood makes up 22 ha of land and lies in the Northeast Sub-area (Alpin and Martin 2005) (Figure 1 and 2). This area is bounded by 16th Avenue to the north, Pacific Spirit Regional Park to the east, and Future Reserve and Bio Sciences land to the south and the west (Alpin and Martin 2005) (Figure 1 and 2). Approximately 3.4 ha of the land have been allocated to useable neighbourhood space (Alpin and Martin 2005).

High amounts of runoff volume are associated with the impermeable surfaces of urban areas (MWLAP 2004). Current stormwater runoff guidelines for new areas of development recommend that post development runoff volumes roughly equal pre-development runoff volumes (MWLAP 2004). In addition, the British Columbia Ministry of Environment states that an appropriate performance target for managing runoff volume is to limit the total runoff volume to 10% (or less) of the total rainfall volume (BC Ministry Environment 2002). Common best management practices which help to achieve these goals include: ponds, stream restoration, and infiltration basins (MWLAP 2004). Simon Fraser University is currently the leading university in British Columbia which takes into consideration these guidelines and practices. These guidelines and practices have been incorporated in their official community plan for new development since 1996 (SFU 2006). It is time that the University of British Columbia strive to meet these guidelines; innovation would be expected of a university setting.

The innovative approach that is the focus of this thesis is to examine whether a combination of common stormwater practices (i.e. ponds) and uncommon practices (designing streams suited to fish habitat) can meet the stormwater guidelines.

Figure 1. Aerial view of the proposed site for the University of British Columbia South Campus Neighbourhood. The South Campus Neighbourhood lies within the broken black bordered line. The area within the yellow bordered line is the Northeast Sub-area of the University of British Columbia. The red line is the possible location of the fish bearing stream which once passed through this area. (Alpin and Martin 2005)

Figure 2. Proposed South Campus Neighbourhood Plan. Specific development on this site is not finalized.(Alpin and Martin 2005)

2.0 THESIS OBJECTIVES

The overall objective is to develop a design methodology that can be used to design a pond and stream system suitable for South Campus and fish habitat. If the stormwater system is designed to meet recommended federal and provincial guidelines then the following guidelines must be met:

- That post development runoff volumes roughly equal pre-development runoff volumes (MWLAP 2004).

- The total runoff volume will be limited to 10% (or less) of the total rainfall volume (BC Ministry of Environment 2002).

Based on the developed design methodology, the degree in which the above guidelines can be met will be determined.

Sub-objectives include:

- Development of a stream model supported by rooftop runoff through the application fluid mechanics and consideration of fish habitat criteria.
- Design the stream taking into consideration major urban stream issues such as overflow, erosion, and periods of high and low flow.
- Determine how to manage variable rooftop runoff flow of good water quality without compromising critical stream velocities or jeopardizing fish habitat

criteria. This includes investigating the possible use of ponds to help manage variable rooftop runoff flow.

- Design of a piping distribution system for the new residential complex that will receive rooftop runoff and effectively contribute to stream flow.

The stream will be restricted to the following constraints:

- Stream width will be no more than 2-3 m wide to accommodate space issues.
- The stream morphology will be dictated by the type most closely associated with the land slope and flow conditions.
- The target fish species is cutthroat trout (*Salmo clarki*) because they are associated with the land, slope, and flow conditions present of the system. In addition, cutthroat trout utilize similar habitat to other salmonids such as coho (*Oncorhynchus kisutch*) and chum (*Oncorhynchus keta*). Thus designing the stream to suit cutthroat trout will ensure the stream meets habitat criteria of several salmonid species.
- The runoff used to develop this methology will only come from rooftops because this runoff unlike runoff from roads and parking areas is considered to be of suitable quality for fish (MWLAP 2004).

3.0 BACKGROUND AND DESIGN INFORMATION

A literature review was conducted to attain the background information needed to successfully design a stream similar to the one that once flowed through the South Campus neighbourhood. To design a stream for the target fish species, knowing the life history strategy and habitat is important (Hunter 1991). Thus, this section begins with information on the life history strategy and habitat requirements of cutthroat trout. Discussion then focuses on riffle pool and cascade pool streams which cutthroat trout often reside in. Next commonly used hydraulic equations for stream design are reviewed, followed by the major issues associated with urban streams. Finally basic design of a piping distribution system which collects rooftop runoff and supplies flow to the stream is investigated.

3.1 Cutthroat Trout Life History Strategy and Habitat Requirements

Cutthroat trout (*Salmo clarki*) belong to the family Salmoninae which includes all salmon and trout species (Moyle and Cech 2004). Cutthroat trout can be readily identified by their blunt head, small black spots on their head and body which extends below the lateral line, red to yellow streaks on the underside the jaw, faint to no red on the sides when spawning, weights between 0.3 and 0.5 kg, and an average length of 45 cm (Moyle and Cech 2004) (Figure 3).

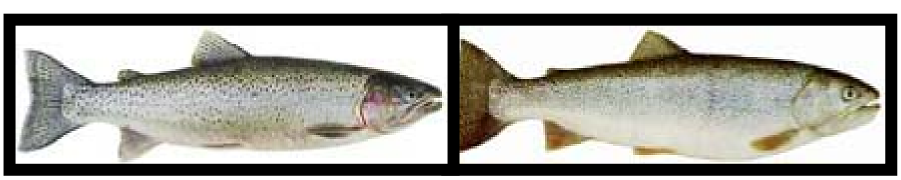


Figure 3. Adult male and female cutthroat trout (*Salmo clarki*)

Cutthroat trout have anadromous and non-anadromous forms (DFO 2005). Anadromous forms are born in freshwater systems, migrate to oceans to feed and grow, and later return to freshwater systems to spawn (Moyle and Cech 2004). Figure 4 illustrates the typical lifecycle of an anadromous salmonid.

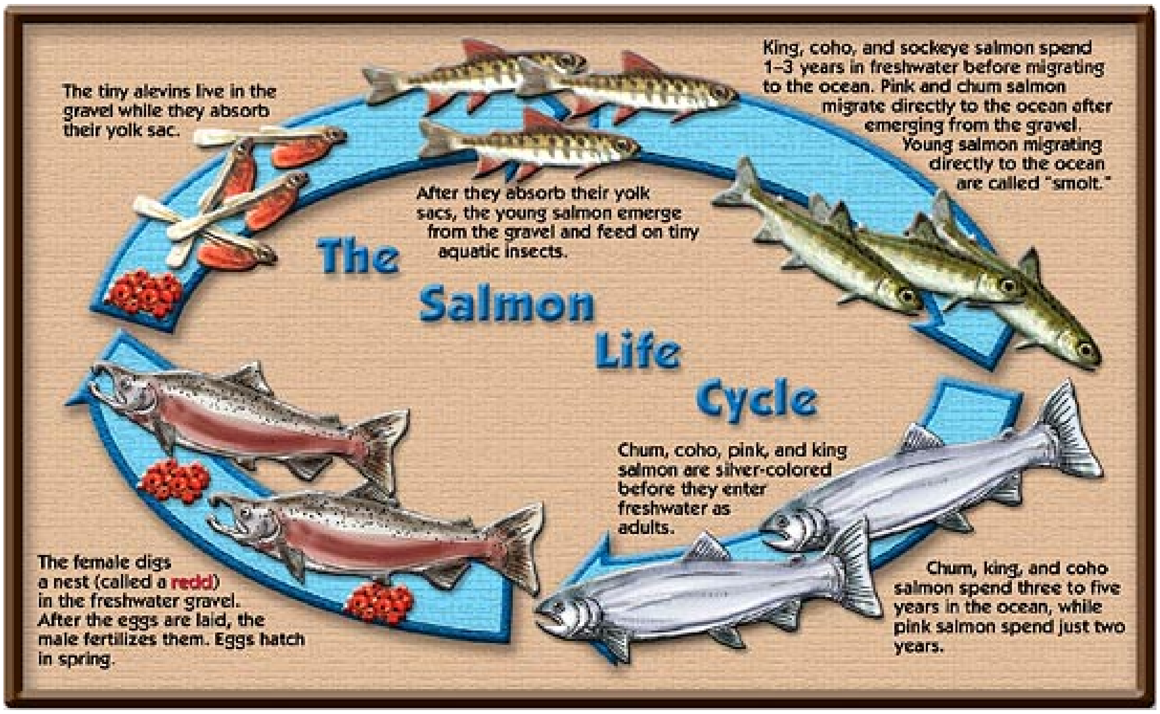


Figure 4. The typical lifecycle of an anadromous cutthroat trout (*Salmo clarki*) and other salmonid species. (USDA 2005)

Non-anadromous forms spend their entire lifecycle in freshwater systems (Moyle and Cech 2004). Cutthroat trout spawn between the months of February and May at the age of three or four (DFO 2005). This is advantageous since February through May are months of high rainfall at the University of British Columbia (Finkenbine 1998). This ensures sufficient flow will be present in streams for cutthroat trout to spawn.

Stream temperature should not exceed 24°C for optimal reproduction and growth of cutthroat trout (Moyle and Chech 2004). Cutthroat trout require a minimum stream depth of 0.06 m, velocity between 0.11 to 0.72 m/s, substrate size of 6-102 mm, and a mean redd area of 0.09-0.0 m^2 for optimal spawning conditions (Hogan and Ward 1997). The maximum swim speed and jumping height of cutthroat trout vary depending on its current life stage (Hogan and Ward 1997). Table 1, summarizes the maximum sustained, prolonged, and burst swimming speeds, as well as the maximum jumping height for cutthroat trout at various stages in its lifecycle.

Table 1. Maximum swim speeds and jump height of adult and juvenile cutthroat trout (*Salmo clarki*). (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

When designing a stream, it is important that these velocities and jumping heights are taken into consideration to ensure fish are able to readily travel through the stream (Hogan and Ward 1997). Cutthroat trout are often found in streams with riffle pool and cascade pool morphology (Hogan and Ward 1997). These streams are suited to accommodate cutthroat trout at the various stages of its lifecycle (Hogan and Ward 1997). Riffle pool and cascade pool streams are discussed in the following section.

3.2 Riffle Pool and Cascade Pool Streams

Riffle pool or cascade pool morphology streams are an ideal choice for an urban stream taking fish habitat criteria into consideration. This is because they are relatively small in size, making them ideal when space is limited (Hogan and Ward 1997). In addition, they suit a variety of fish species (Hogan and Ward 1997). Riffles and cascades are shallow areas of stream which provide aeration and increased current velocity (Hunter 1991) (Figure 5). Pools are deeper areas with slower current velocity and are often used by fish as refuge (Hunter 1991) (Figure 5).

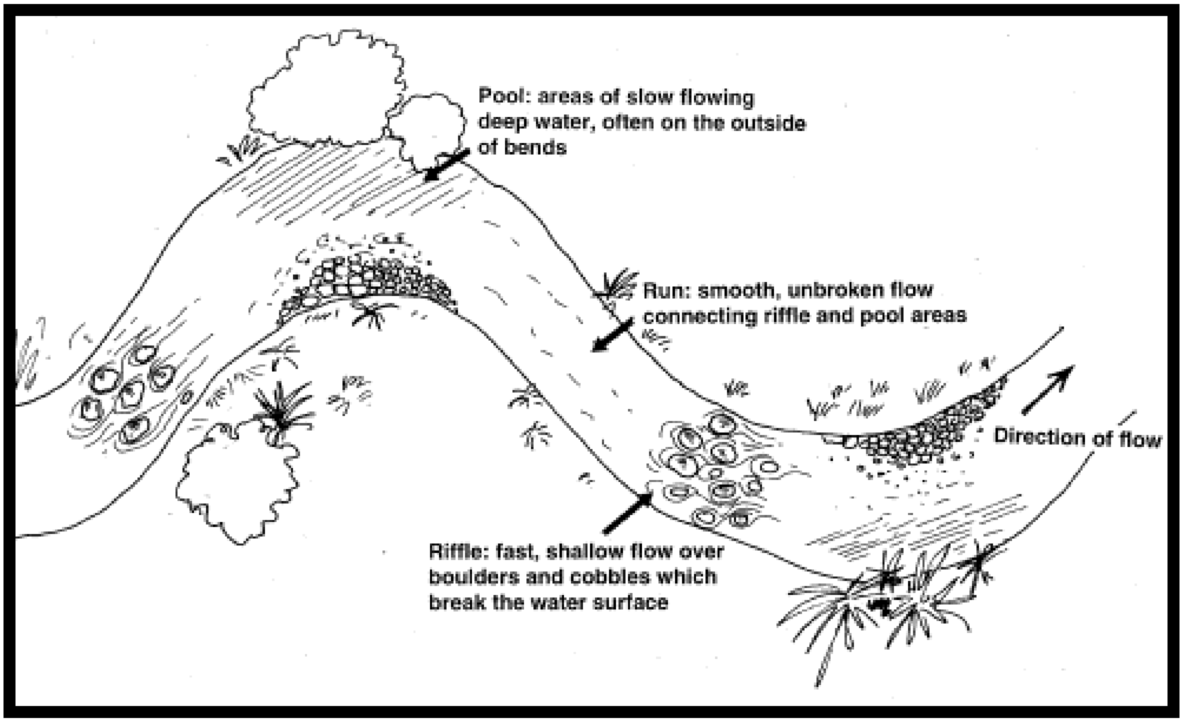


Figure 5. A typical stream with riffle pool sequence morphology.

Trout particularly like riffle and cascade pool sequences because its morphology suits various stages of their life cycle (Hunter 1991). The reduced velocities in pools provide an ideal refuge area for juvenile trout, while riffles or cascades continually carry food to them (Hunter 1991). Deeper pools also provide refuge from predators, cooler temperatures, and are essential to the over winter survival of trout (Moyle and Cech 2004).

Riffle pool and cascade pool morphologies are one of three major stream morphology types (Hogan and Ward 1997). These include step-pools, cascade and riffle pools, and riffle pools with sand beds (Hogan and Ward 1997). Channel and aquatic characteristics of each stream morphology type is outlined in Table 2.

Table 2. Typical channel and aquatic characteristics associated with the major morphological types: Step-pool, cascade-pool and riffle pool, and riffle pools with sand beds. (Hogan and Ward 1997)

General Characteristics	Small	Intermediate	Large
Channels:			
morphology	step-pool	cascade-pool, riffle-pool,	riffle-pool sand bed forms
bed sediment size	boulder, cobble	cobble, gravel	gravel, sand
Typically[a]			
bankfull width, W_b (m)	<10	10-30	>30
bedform spacing (W_b)[b]	1-4	3-5	5-7
channel gradient (%)	≥4	<4	<2
valley wall-channel relation	coupled	partially coupled	uncoupled
bank materials	boulder, bedrock	sand, gravel, cobble	alluvium
functional role of LWD	minor-moderate	major	minor
stream order (1:50,000)	1-2	3-5	>5
stream productivity	allochthonous	allochthonous and autochthonous	autochthonous
insect community	shredders, collectors	collectors, grazers	collectors
fish community	one or two species, small adults	three or four species, large adults	a diversity of species and life histories in a variety of habitats
principal fish use	residence, or juvenile rearing, restricted passage	incubation, juvenile rearing and spawning	spawning, off-channel rearing, passageway
dominant salmonid habitat	plunge pools with canopy cover, boulder clusters	abundant spawning gravel; LWD holds gravel, forms pools, and provides cover	large pools, off-channel creeks and ponds

Key stream parameters to consider when designing a stream include bankfull width, bankfull depth, dominant sediment size, and channel gradient (Hogan and Ward 1997). Bankfull width is used as the base unit when stream dimensions are being determined (Newbury *et al.* 1997). It is defined as the distance between the edges of a stream where vegetation begins to grow (Newbury *et al.* 1997) (Figure 6). Bankfull width is used to determine the spacing between cascades/riffles and pools in a cascade pool and riffle pool sequenced stream (Newbury *et al.* 1997). Optimal pool to cascade/riffle spacing is typically 6 to 8 times the bankfull width (Newbury *et al.* 1997). Bankfull depth is the water surface elevation needed to fill the channel to the point in which water does not spill into the flood plain (Newbury *et al.* 1997) (Figure 6). Bankfull depth is an important parameter used to determine how much flow a stream can effectively handle (Newbury *et al.* 1997). Dominant stream bed material size (or sediment size) is important since certain sizes of stream bed materials are ideal for salmon spawning (Moyle and Cech 2004). The ideal stream bed material for cutthroat trout is gravel with a diameter ranging from 6-102mm (Hogan and Ward 1997). Stream bed material smaller than this size, may be abrasive to fish eggs and damaging to gills (Moyle and Cech 2004). Designing a stream to maintain a particular sediment size, while discharging smaller sediment sizes, is discussed in the next section. Channel gradient is important since it plays a significant role in stream velocity and discharge rate (Johnston and Slaney 1996). When designing the stream a channel gradient of less than approximately 4% will be used to maintain cascade pool or riffle pool stream morphology. A gradient of less than 4% is typical of cascade pool streams, while a gradient of less than 2% is typical of riffle pool streams (Hogan and Ward 1997).

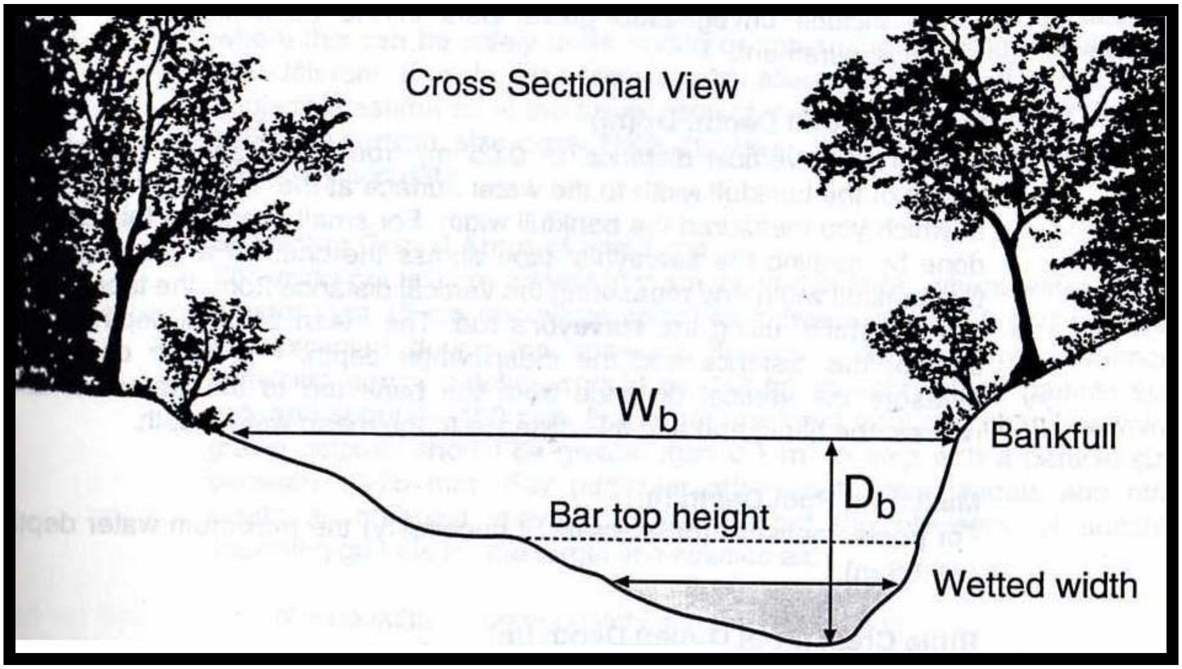
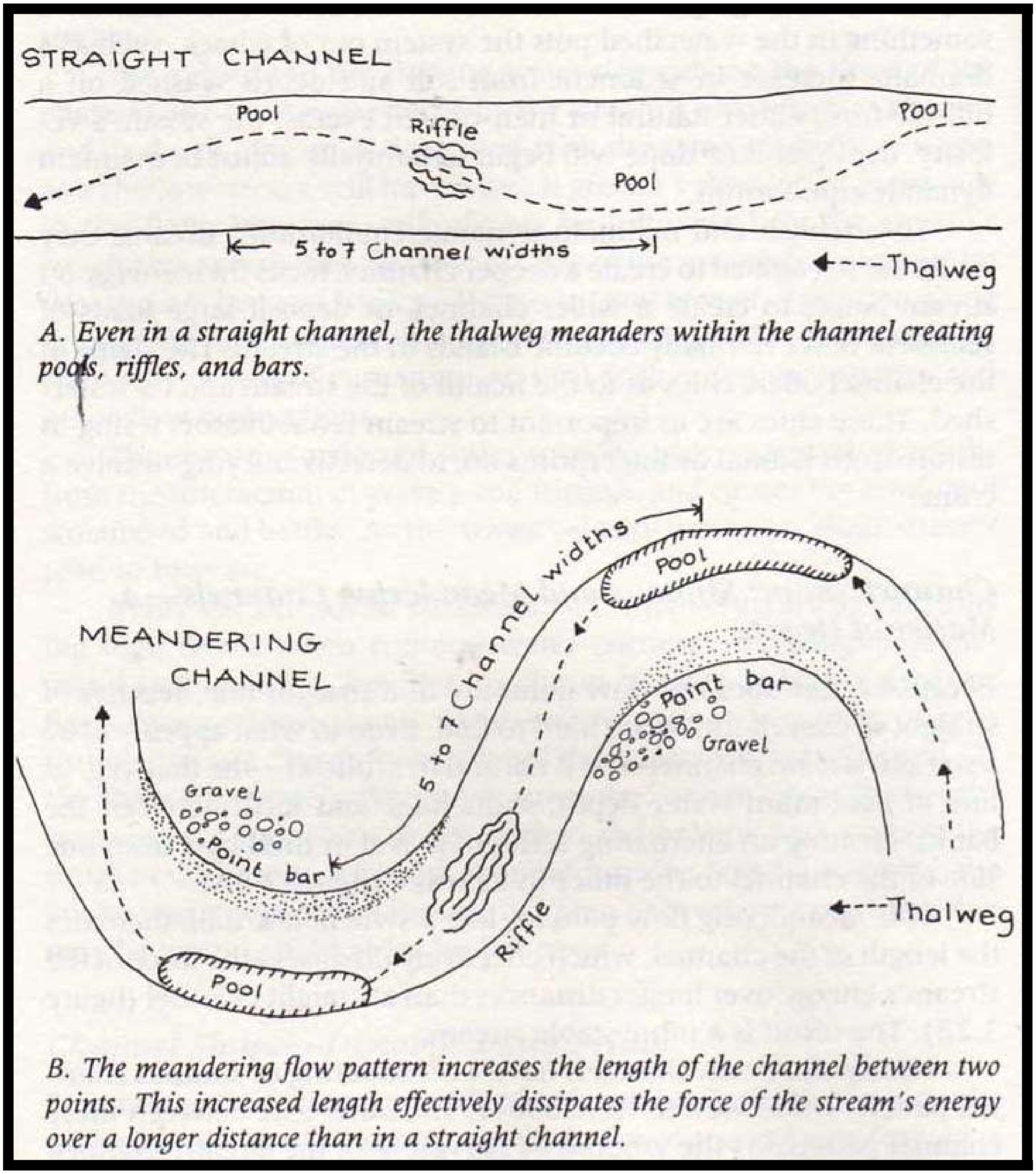


Figure 6. Cross sectional view of a stream. W_b indicates bankfull width, the distance between the edges of a stream where vegetation begins to grow (floodplain). Db indicates bankfull depth, the water surface elevation needed to fill the channel to a point in which water does not fill into the floodplain. (Johnston and Slaney 1996)

Water does not have the tendency to flow in a straight line (Hunter 1991). This is why natural streams are often meandering and sinuous (Hunter 1991) (Figure 7). Meandering streams increase the effective length of a channel and dissipates the force of the streams energy over long distances (Newbury *et al.* 1997). This increases stream stability (Newbury *et al.* 1997). In addition, strategically placing pools on the outside of stream bends will reduce the flow velocity which impacts a streams bank (Figure 5 and 7). This helps reduce erosion created by stream flow (Hunter 1991) (Figure 7). The average wavelength of meanders is 12 times the bankfull width and the average radius of curvature is 2.3 times the bankfull width (Leopold *et al.* 1964). Urban streams have the tendency to become channelized (Figure 7). This causes the stream to act as a straight and wide drainage ditch (Hunter 1991). Channelization increases stream velocity, deepens stream beds through scouring, increases erosion, increases sedimentation, and induces downstream flooding (Hunter 1991). These adverse effects of channelization create unsuitable fish habitat (Hunter 1991). To prevent channelization and to design a stream similar to a natural stream, it will be important to vary the radius of curvature, meander length, and distances between pools and cascades/riffles (Newbury *et al.* 1997). This can be done through the use of a random generator. Specific stream dimensions can be determined with the use of hydraulic equations which are discussed in the next section.



A. Even in a straight channel, the thalweg meanders within the channel creating pools, riffles, and bars.

B. The meandering flow pattern increases the length of the channel between two points. This increased length effectively dissipates the force of the stream's energy over a longer distance than in a straight channel.

Figure 7. A straight channeled stream typically seen in urban areas and a meandering, sinuous stream typically seen in natural environments. The meandering stream creates greater habitat diversity and is more suited to fish. (Hunter 1991)

3.3 Stream Design Hydraulic Equations

One of the most commonly used equations in stream design is the Manning equation. The Manning equation is a semi-empirical equation which allows open channel flows to be simulated (Lencastre 1987). The Manning equation typically takes the following form (Lencastre 1987):

$$Q = AR^{\frac{2}{3}}S^{\frac{1}{2}}\frac{1}{n}, \text{ (SI Units)} \qquad \text{Equation 1}$$

Where,

Q, is flowrate in m^3/s. Often this flow is taken for a given frequency of occurrence.

A, is the cross sectional area of the stream in m^2.

S, is the channel bottom slope.

R, is the hydraulic radius of the channel (m), and can be defined as the channel area divided by the wetted perimeter.

n, is roughness coefficients which vary depending on the channel conditions. These coefficients are typically unitless.

The Rational formula is often used to estimate peak discharge rates in areas less than 25km^2 (Schwab *et al.*1981). The formula takes the following form (Schwab *et al.*1981):

$$Q = ciA \qquad \text{Equation 2}$$

Where,

c, is runoff coefficient which expresses the portion of rainfall which is available as peak runoff. The runoff coefficient varies depending on the permeability of the surrounding area. Impermeable areas will have a value of 1, while highly permeable areas will approach a value close to 0. Typical values are 0.8 for developed areas and 0.3 for undeveloped areas (Schwab *et al.*1981).

i, is rainfall intensity expressed in m/s.

Q, is flowrate in m^3/s. Often this flow is taken for a given frequency of occurrence.

A, is the area of the catchment of interest in m^2.

Total flowrate (Q_{total}), can be determined from the following formula:

$$Q_{total} = Q_{rainfall} + Q_{infiltration} - Q_{evaporation} \qquad \text{Equation 3}$$

Where,

Q_{total}, is total flowrate which will contribute to stream flow.

$Q_{rainfall}$, is flow attained from rainfall or storm events. For the design of the stream, this will include rainfall collected from rooftops as well as rain which directly enters the stream.

$Q_{infiltration}$, is flow attained from ground water sources (Figure 8). During storm events, water infiltrates permeable areas and is stored (Schwab *et al.* 1981). This water is then able to seep through soil and contribute to stream flow (Schwab *et al.* 1981). The rate in which this water enters the stream depends on how much water is stored, and the current stream flow level (Schwab *et al.* 1981). Lower areas will tend to contribute flow at a slower rate since soil is generally less porous than soil at higher levels (Coduto 1999). The greater the soil porosity, the easier it is for water to flow through the soil, and the faster the rate of flow contribution (Coduto 1999). It is important to note that flow may also be lost due to infiltration (Coduto 1999).

$Q_{evaporationl}$, is flow lost due to evaporation. During winter months when temperature is relatively low, flow lost to evaporation can be considered zero.

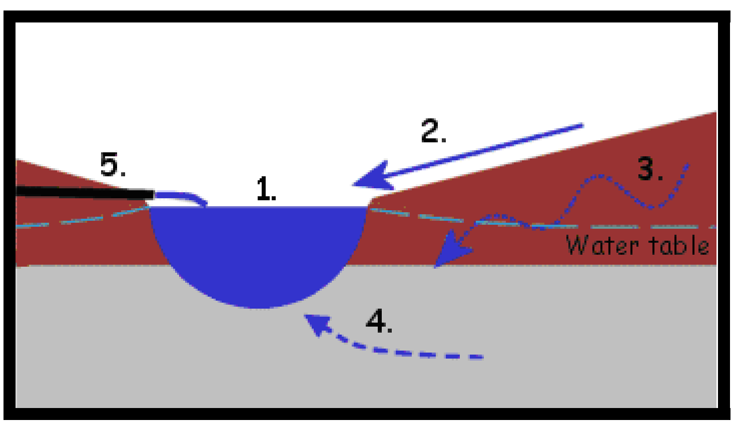


Figure 8. Routes in which water flow enters a stream. 1, represents flow from upstream tributary sources. 2, represents flow contribution from surface runoff. 3, represents flow contribution from the ground water table. This soil is considerably more porous than bedrock material, thus flow through this soil is relatively fast. 4, represents flow which percolates through less porous bedrock material. 5, represents flow collected from rooftop runoff and distributed to the stream through a piping system. (Naturegrid 2005)

Manning's roughness coefficient (n) is typically determined using the following formula (Cowan 1956):

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\, m_5 \qquad \text{Equation 4}$$

Where n values account for,

n_o , basic straight, uniform, smooth channel

n_1 , corrects for surface irregularities

n_2 , channel cross section shape and size

n_3 , obstructions

n_4, vegetation and flow conditions

m_5 , meandering channel

Values for n and m are tabulated in various publications for various channel materials. The Manning roughness coefficient for gravel ranges from 0.02-0.03 (Finnemore and Franzini 2002). There is no exact method for determining roughness coefficient values (Martin 1996). Three factors which have been determined to have the greatest influence on Manning's roughness coefficient are surface roughness, vegetation, and channel irregularity (Ven Te 1981). Surface roughness is influenced by the size and shape of stream bed materials lining a streams wetted perimeter (Ven Te 1981). Smaller stream bed materials have smaller roughness coefficients, and are thus relatively unaffected by changes in flow (Ven Te 1981). Larger sediments such as gravel and boulders will have higher roughness coefficient values (Martin 1996). The lower the flow, the greater the roughness coefficient value becomes (Martin 1996). Vegetation roughness slows down flow and may reduce capacity (Ven Te 1981). Vegetation roughness coefficients vary with vegetation type, height, distribution, and density (Martin 1996). The time of year will also vary the vegetation roughness coefficient (Martin 1996). Channel irregularities occur due to stream cross-section variation, wetted perimeter irregularities, and the size and shape along a given channel length (Martin 1996). Examples of irregularities include, sand bars, ripples, and depression (Martin 1996). Other factors which affect roughness coefficient values are changes in season, channel alignment, silting, scouring, bed load, and suspended material (Martin 1996).

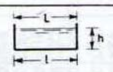
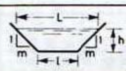
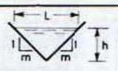
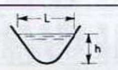
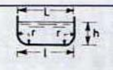
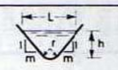
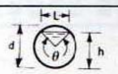
Four general approaches are used to determine Manning's roughness coefficient (Ven Te 1981). These approaches are outlined in the Masters work of Dr. Martin of the University of British Columbia. Dr. Martin summarized the approaches as follows (Martin 1996):

- Fist attempt to understand the factors which influence the values of *n*. Doing so will eliminate a significant amount of guess work.
- Next consult a table of typical *n* values for various channel shapes and sizes.
- Familiarize oneself with the appearance of typical channels with well known roughness coefficients.
- *To determine n, analytically from a known velocity distribution in the channel cross-section and on the data of either velocity or roughness measurement* (Ven Te 1981). In actuality, *n* is found through experimental measurements of the mean properties of flow. These mean properties of flow are velocity, hydraulic radius, and slope.

Several recent studies have been conducted to investigate Manning's roughness coefficient (Martin 1996). The Masters work of Dr. Martin of the University of British Columbia outlines several theories on determining roughness coefficient, *n*. To learn more on these theories it is recommended that this work is read.

Application of the Manning equation can be used to determine the cross-sectional area of a stream and thus its shape. Typical stream shapes are displayed in Table 3.

Table 3. Various stream cross sections and shapes. Application of Manning's equation to determine cross-sectional area can be used to design stream size and shape. (Lencastre 1987)

Shape	Cross-section A	Wetted perimeter P	Radius R	Top width L	Mean depth h_m	$A\sqrt{\frac{A}{L}}$
	lh	$l+2h$	$\frac{lh}{l+2h}$	l	h	$lh^{1.5}$
	$(l+mh)h$	$l+2h\sqrt{1+m^2}$	$\frac{(l+mh)h}{l+2h\sqrt{1+m^2}}$	$l+2mh$	$\frac{(l+mh)h}{l+2mh}$	$\frac{[(l+mh)h]^{1.5}}{\sqrt{l+2mh}}$
	mh^2	$2h\sqrt{1+m^2}$	$\frac{mh}{2\sqrt{1+m^2}}$	$2mh$	$^1/_2 h$	$\frac{\sqrt{2}}{2} mh^{2.5}$
	$^2/_3 \times Lh$	$L+\frac{8}{3}\frac{h^2}{L}$ †	$\frac{2L^2h}{3L^2+8h^2}$ †	$\frac{3}{2}\frac{A}{h}$	$^2/_3 h$	$^2/_9\sqrt{6}Lh^{1.5}$
	$\left(\frac{\pi}{2}-2\right) r^2+(l+2r)h$	$(\pi-2)r+l+2h$	$\frac{(\pi/2-2)r^2+(l+2r)h}{(\pi-2)r+l+2h}$	$l+2r$	$\frac{(\pi/2-2)r^2}{l+2r}+h$	$\frac{[(\pi/2-2)r^2+(l+2r)]}{\sqrt{l+2r}}$
	$\frac{L^2}{4m}-\frac{r^2}{m}(1-m\tan m)$	$\frac{L}{m}\sqrt{1+m^2}-\frac{2r}{m}(1-m\tan m)$	$\frac{A}{P}$	$2\left[m(h-r)+r\sqrt{1+m^2}\right]$	$\frac{A}{L}$	$A\sqrt{\frac{A}{L}}$
	$^1/_8(\theta-\sin\theta)d^2$	$^1/_2\theta d$	$^1/_4\left(1-\frac{\sin\theta}{\theta}\right)d$	$(\sin {}^1/_2\theta)d$ or $2\sqrt{h(d-h)}$	$^1/_8\left(\frac{\theta-\sin\theta}{\sin {}^1/_2\theta}\right)d$	$\frac{\sqrt{2}}{32}\frac{(\theta-\sin\theta)^{1.5}}{(\sin {}^1/_2\theta)^{0.5}}d^{2.5}$

When constructing a stream, it is important to try to achieve dynamic equilibrium (Hunter 1991). Dynamic equilibrium is achieved when the amount of water and sediment that exits a stream is equal to the amount that has entered it (Hunter 1991). The movement of water and sediment dictates stream morphology (Lane 1955). To maintain dynamic equilibrium, the morphology of a stream will change as conditions change (Hunter 1991). Sediment movement is particularly important when trying to achieve a stream bed suited to fish spawning (Moyle and Cech 2004). Optimal stream bed material is gravel ranging from 6 – 102mm in diameter (Hogan and Ward 1997). Substrate much smaller than gravel is undesirable since it is abrasive to fish eggs and gills (Moyle and Cech 2004). To attain an optimal stream bed for fish spawning, it is necessary to have smaller sediment discharged downstream while maintaining a gravel bed (Hunter 1991). In addition, when dimensioning an unlined channel consisting of non-cohesive materials, it is important that it is stable in relation to the hydrodynamic forces generated by the flow (Neill 1967). Lighter particles are generally dislodged from the stream bottom and transported downstream, while heavier particles remain on the stream bed (Neill 1967). The conditions in which non-cohesive materials begin to move from the stream bed or bank is referred to as critical conditions (Neill 1967). Two important parameters are critical velocity and critical shear stress (Neill 1967). The equation used to dimension streams with non-cohesive and uniform stream bed material is expressed as the following (Neill 1967):

$$\frac{U_{crit}^{2}}{\left(\gamma_s/\gamma - 1\right)d} = 2.5\times10^{-4}\left(\frac{d}{h}\right)^{-0.20}, \qquad \text{Equation 5}$$

Where,

U_{crit}, is taken as the mean velocity of flow in m/s.

d, is the diameter of stream bed material in mm.

h, is the mean depth of flow in m.

γ_s, is the specific weight of substrate in kg/m^3.

γ, is the specific weight of substrate in kg/m^3.

Equation 4 allows critical velocity to be determined. Critical velocity is the velocity which will begin to move a given substrate material on a stream bed. Figure 9, outlines the critical velocities for various sizes of stream bed material.

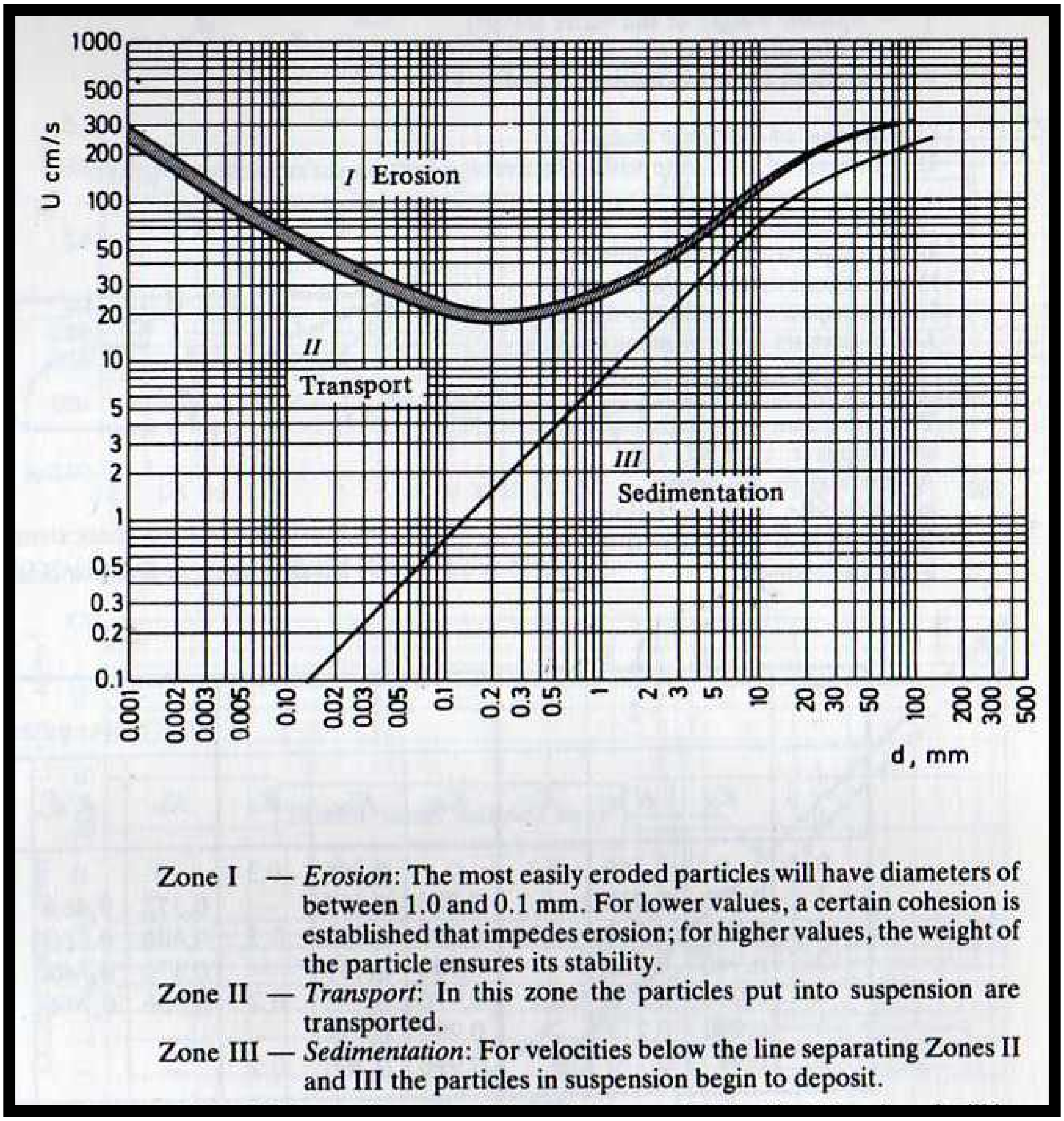


Figure 9. Critical velocity with respect to stream bed material (sediment) diameter. Depending on the velocity and the size of stream bed material, erosion, transport, or sedimentation may occur. (Lane 1955)

Critical shear stress can be used to determine stream bed and bank stability. The shear stress exerted by flow at a stream bed, for two-dimensional flow, in a rectangular channel, of undefined width, is given by the following equation (Lencastre 1987):

$$\tau_o = \gamma h i \,, \qquad \text{Equation 6}$$

Where,

τ_o, is critical shear stress in N/m^2.

γ, is specific weight of substrate in kg/m^3.

h, is mean depth of flow in m.

i, is channel slope.

The maximum shear stress at the bottom of a stream is given by (Lencastre 1987):

$$\tau_o = \gamma R i \,, \qquad \text{Equation 7}$$

Where,

τ_o, is critical shear stress on the stream bed in N/m^2.

R, Reynolds number (ratio of inertial forces over viscous forces), unitless.

h, is mean depth of flow in m.

i, is channel slope.

Figure 10, illustrates Shields curve. Shields curve describes the relationship between critical shear stress and the mean diameter of material.

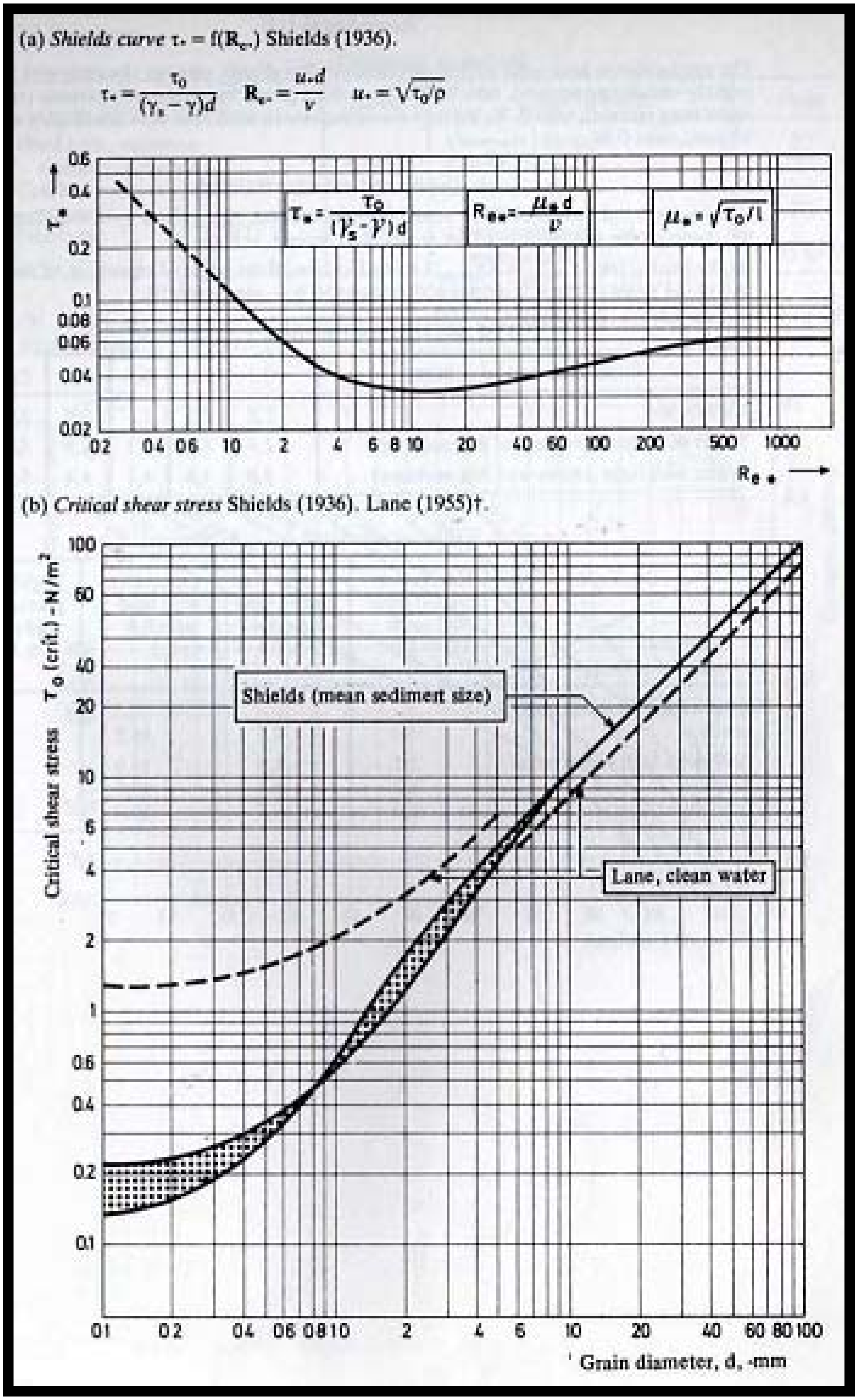


Figure 10. Critical shear stresses according to shields. (Lane 1955)

Bank stability analysis is similar to the analysis of stream bed material. The critical shear stress for rectilinear flow of a particle on a bank slope is expressed in terms of critical shear stress for a bed particle (Lane 1955). This is shown in the following equation (Lane 1955):

$$\left(\tau_o^t\right)_{crit} = K\left(\tau_o\right)_{crit}, \qquad \text{Equation 8}$$

Where,

$\left(\tau_o^t\right)$, is critical shear stress on channel side slopes in N/m^2.

τ_o, is critical shear stress in N/m^2.

K, is the Manning – Strickler coefficient of roughness in m$^{1/3}$/s.

Values for K, can be attained from Figure 11.

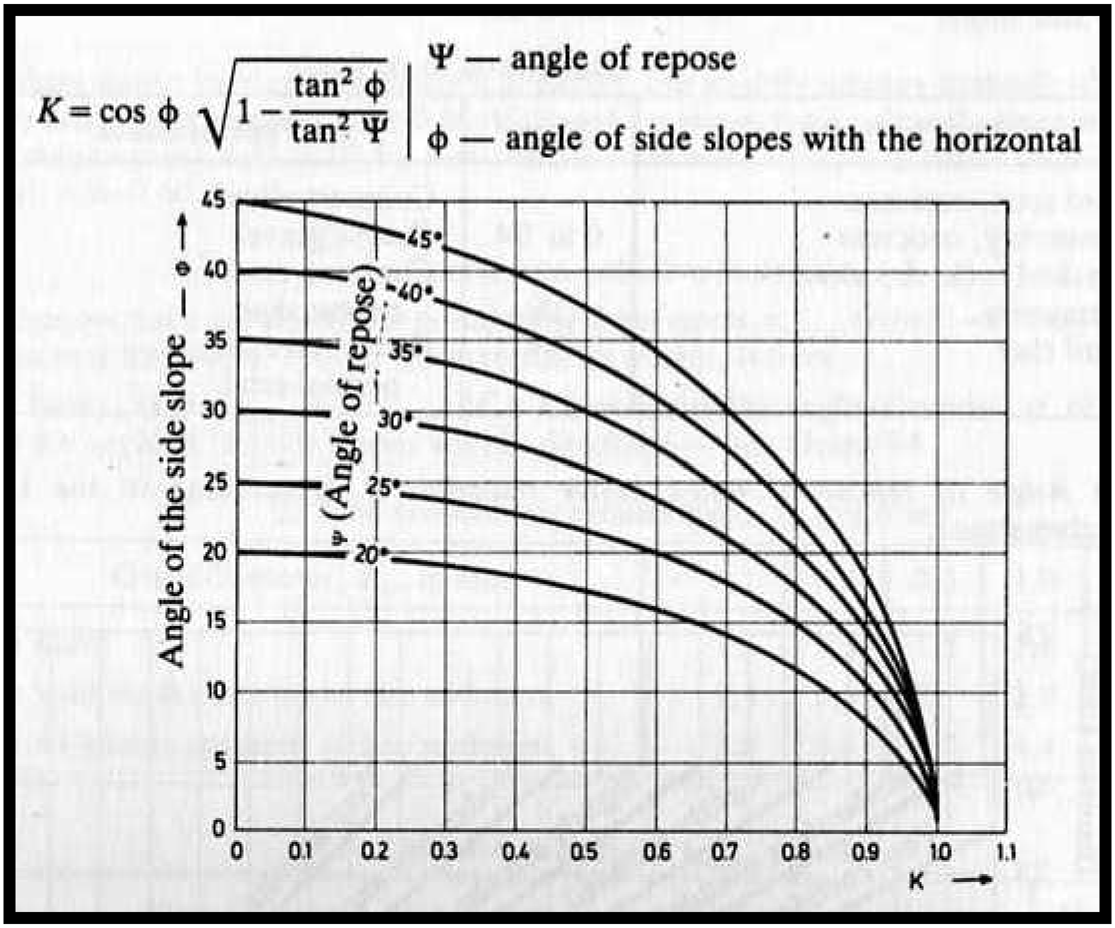


Figure 11. Critical shear stress on banks. The Angle of the slide slope with respect to the Manning – Strickler coefficient of roughness. (U.S. Soil Conservation Service 1974)

K, the Manning – Strickler coefficient of roughness, can be determined from the following formula (U.S. Soil Conservation Service 1974):

$$K = \cos\phi\sqrt{1-\frac{\tan^2\phi}{\tan^2\psi}}, \qquad \text{Equation 9}$$

Where,

ϕ, is slope angle to the horizontal.

ψ, is the angle of repose.

3.4 Major Urban Stream Issues

Several major issues are associated with urban streams. These include variable flow, flooding, maintenance of dry season flow, erosion, and space availability (Hunter 1991).

Variable Flow:

Urban streams experience greater variable flow than natural streams (Sherwood 1994). This is due to a decreased amount of permeable surfaces and water storage capacity (Sherwood 1994). During storm events, urban streams experience high rates of discharge due to the lack of available permeable surfaces (Sherwood 1994). Stormwater runs off these permeable surfaces and directly into the streams (Schwab *et al.* 1981). Natural streams do not experience these high discharge rates due to the availability of permeable area (Sherwood 1994). In addition, natural streams are able to maintain a higher level of base flow since water is able to slowly percolate into the stream from groundwater sources after a storm event (Sherwood 1994). Maintaining a particular flow velocity in a stream is difficult during highly variable flow due to varying flow constantly changing the effective area of a stream (Ven Te 1981). This often leads to erosion during low flow when non vegetated sections of the stream become exposed and susceptible to wind scour (Figure 12).

Figure 12. Area susceptible to erosion. As flow varies, area of un-vegetated stream are exposed. During extended dry periods, this area is susceptible to wind scour and erosion.

Flooding:

Flooding is of concern since urban streams flow through densely populated areas (Hunter 1991). Adverse effects of flooding include property damage and risk to human safety (Freeman 1994). The Manning equation (Equation1) is typically used to design streams which minimize flooding (Freeman 1994). Often urban streams incorporate spill zones (Freeman 1994). These are designated areas where the stream is able to overflow during periods of high flow (Freeman 1994).

Spill zones typically take the form of a field or pond which acts as an equalization basin for stream flow (Freeman 1994). Fields are orientated in a manner in which overflow can periodically flow onto the field (Freeman 1994). The slope of the field then allows flow to reenter the stream as flows decrease (Freeman 1994).

Maintaining Dry Season Flow:

Maintenance of flow during dry season can be difficult. An urban streams incorporating fish habitat should be excavated to a depth in which ground water is able to maintain a minimal level of flow (Hunter 1991). The addition of a pond which collects water during wet periods and discharges flow into a stream during dry periods may also be used. In natural riffle pool sequences, pools are often deep enough that they do not dry out during dry periods (Newbury *et al.* 1997). Pools then provide refuge for fish during periods in which shallow cascades or riffles no longer have flow (Newbury *et al.* 1994). When flow in cascades and riffles increase due to a storm event, fish are then able to continue moving through a stream (Newbury *et al.*1994). In addition, ground water levels are typically recharged during wet periods and slowly discharge into streams during dry periods allowing a base flow to be maintained.

Erosion Issues:

Erosion is a concern because it may cause streams to encroach on structures in an urban community (Donat 1995). To mitigate the effects of erosion it is important to attempt to maintain a stream in dynamic equilibrium (Hunter 1991). Bioengineering techniques are often used to achieve this (Hunter 1991). Three of the most commonly used bioengineering techniques to mitigate erosion are riparian vegetation, geotextiles, and riprap (Donat 1995).

Riparian vegetation is vegetation adjacent to the normal high waterline in a stream and is influenced by the stream (Newbury *et al.* 1994) (Figure 13).Riparian vegetation helps to mitigate erosion through the following (Donat 1995):

- Soil reinforcement by roots increases shear strength
- Anchors topsoil into firm strata
- Dynamic forces created by wind led into slope via vegetation
- Interception by foliage
- Increased infiltration capacity due to increased ground surface roughness and permeability
- Water-uptake by roots lowers pore water pressure

Figure 13. A natural riffle pool morphology stream lined with riparian vegetation. This vegetation reduces the effects of erosion.

Geotextiles are primarily used to stabilize loose top soil until vegetation is able to take over (Donat 1995) (Figure 14). Typical geotextiles are made out of biodegradable material such as reed, flax, and synthetic cellulose fibers (Donat 1995). Advantages of geotextiles are that they are immediately effective, have standardized materials, and can be easily combined with other techniques (Donat 1995). Disadvantages include cost and limited use (Donat 1995).

Figure 14. The initial application of a geotextile along a stream bank.

Riprap utilizes permanent rock cover to stabilize stream banks and provide in-stream channel stability (Donat 1995) (Figure 15). This rock is placed on stream banks which may be susceptible to erosion (Donat 1995). The rock material typically used however, is relatively resistant to scour and erosion effects (Donat 1995). The advantage of riprap is that it is easy to apply, inexpensive, requires little maintenance, and can be used to improve existing structures (Donat 1995). Disadvantages are that construction is only possible during dormant seasons, stabilization only occurs only after the incorporation of plant rooting, and wall structures are difficult to create if needed (Donat 1995).

Figure 15. The application of rip-rap along a stream bank to help prevent erosion.

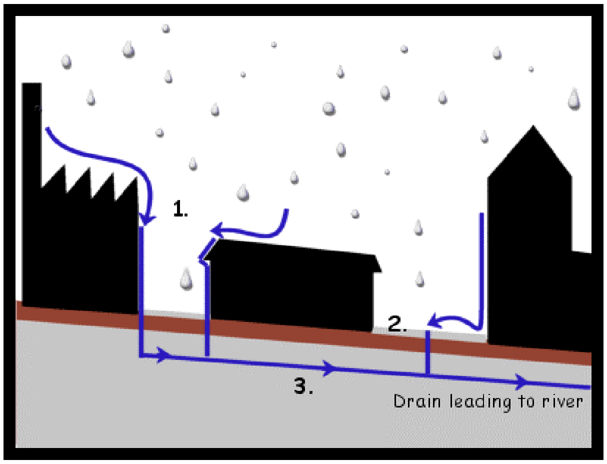
Space Availability:

An urban stream incorporating fish habitat criteria is difficult to construct due to space limitations. Urban areas are often densely populated with little green space (Huth 1978). This often causes urban streams to suffer from channelization (Hunter 1991). Most urban streams utilize large recreational fields as areas for overflow and ponds to help equalize flow (Freeman 1994). Space limitations make it difficult to incorporate these structures (Freeman 1994). To mitigate the dilemma of space availability a piping distribution system which selectively collects rooftop runoff will be investigated.

3.5 Design of Piping Distribution System

Design of a piping distribution system which selectively collects rooftop stormwater runoff and provides stream flow will help mitigate flooding and space availability issues. No work on the design specifics of a piping distribution system which receives rooftop stormwater runoff and provides flow to a stream was able to be found.

The feasibility of a piping system which collects rooftop runoff and diverts it into a stream will be investigated. How much flow can be collected from rooftop runoff can be determined using a UBC rainfall chart. The area of rooftops in which stormwater will be collected can be determined. The stream can then be designed to manage various flows from various storm events through application of Manning's equation (Equation 1) and a University of British Columbia rainfall chart. Pipe size will ensure only a certain amount of flow will be collected from rooftop runoff. This will help insure stream overflow never occurs. A simplified schematic is illustrated in Figure 16.



Runoff is first collected from rooftops during a storm event and collected in a piping distribution system which diverts flow to a stream.

Runoff transported in the piping system is then discharged into the stream contributing to flow.

Collected runoff will be discharged by the piping distribution system along various portions of the stream.

Figure 16. Simplified concept behind a piping distribution system which will fist collect stormwater rooftop runoff, transport this runoff through a pipe, and distribute it along various areas sections of a stream.

4.0 DESIGN METHODOLOGY AND SAMPLE SYSTEMS

To achieve the objectives the following four phases were completed:

- Phase 1: Site Analysis
- Phase 2: Conceptual Model Development
- Phase 3: Pond Design and Dimensioning
- Phase 4: Stream Design and Dimensioning
- Phase 5: Piping Distribution System Design

Phase 1 involved site analysis and the development of a conceptual model. This phase was used to determine ideal stream placement. To determine this, a site analysis was conducted. This analysis included a brief site description and a review of stream and pond placement considerations. If the original lost stream was not able to be restored based on these considerations, an ideal stream placement would be recommended.

Phase 2 involved the development of a conceptual model that describes the pond and stream system layout, as well as the flow inputs and outputs. Methods to determine if the federal and provincial stormwater guidelines are achieved are also outlined.

Phase 3 involved pond design and dimensioning. After Phase 1, it was found that two areas were suited for the placement of ponds. Details on the design and dimensioning of these ponds are discussed.

Phase 4 involved stream design and dimensioning. Details on the design and dimensioning of the stream and how it conforms to fish habitat are discussed.

Phase 5 involved the development of a piping distribution system. The general design of a piping distributions system which conveys stormwater into the two ponds is discussed.

PHASE 1: SITE ANALYSIS AND CONCEPTUAL MODEL DEVELOPMENT

Phase 1.1 Site Description

The South Campus Neighbourhood encompasses approximately 301 000m^2 of land and lies in the Northeast Sub-area (Alpin and Martin 2005). The area is bounded by 16th Avenue to the north, Pacific Spirit Regional Park to the east, and Future Reserve and Bio Sciences land to the south and to the west (Alpin and Martin 2005). The neighbourhood has been designed for an estimated population of 5000 people (Alpin and Martin 2005). Of the 301 000m^2 of land, approximately 240 000m^2 is allocated to residential use, 30 000m^2 to commercial use, and 34 000m^2 to open space (Alpin and Martin 2005). The topography of the land slopes towards the southeast (Appendix A). The general surficial geology of the area is described as Vashon Glacial deposits (GeoPacific Consultants Ltd 2006). The topsoil encompasses a depth from 0 to 0.6m (GeoPacific Consultants Ltd 2006). Below this topsoil layer lies till which is described as a brown, orange staining silt matrix, with trace fine sand, trace gravel clasts, and trace cobbles (GeoPacific Consultants Ltd 2006). This till layer lies approximately 3.5m below the ground surface. Below this till later is predominantly sand (GeoPacific Consultants Ltd 2006).

Due to the intrusion of current development and the inability to alter the already zoned South Campus Neighbourhood plan, it was not viable to restore the lost stream. Thus, an alternative stream path would have to be developed. A variety of optional stream locations were reviewed. Stream and pond placement considerations are outlined next in Phase 1.2.

Phase 1.2 Stream and Pond Placement Considerations

To determine the optimal stream and pond locations, several factors were considered. Key factors included:

- Natural topography of the area
- The number of roads the stream would cross
- Cost
- Space limitations

Several stream and pond locations were considered. Many of these options were not feasible due to the limitations of the considerations listed above. For instance, the natural topography of the area slopes towards the southeast (Appendix A). This made it unfeasible to place the stream flowing in a southwest direction since it is difficult for a stream to flow against the natural gradient of the land (Hunter 1991). The excavation costs to create a slope which would allow the flow to move in a southwest direction would be high (Hunter 1991). In addition, if the stream flowed in the southwest direction several roads would have to be crossed. To cross these roads culverts would have to be built. The costs associated with culvert placement are high (MELP and MF 1999). To minimize these costs the number of roads a stream crosses should be minimized (MELP and MF 1999). In addition, due to the South Campus Neighbourhood Plan, space available for pond and stream placement was limited. The area located west of the development is currently used for agricultural research (Appendix B). This makes placement of a stream through this area difficult. Within the South Campus Neighbourhood Plan, pond and stream location was limited to designated green space and greenways (Appendix C).

Phase 1.3 Chosen Pond and Stream Placement

The determined optimal stream and pond locations are outlined in Figure 17 below.

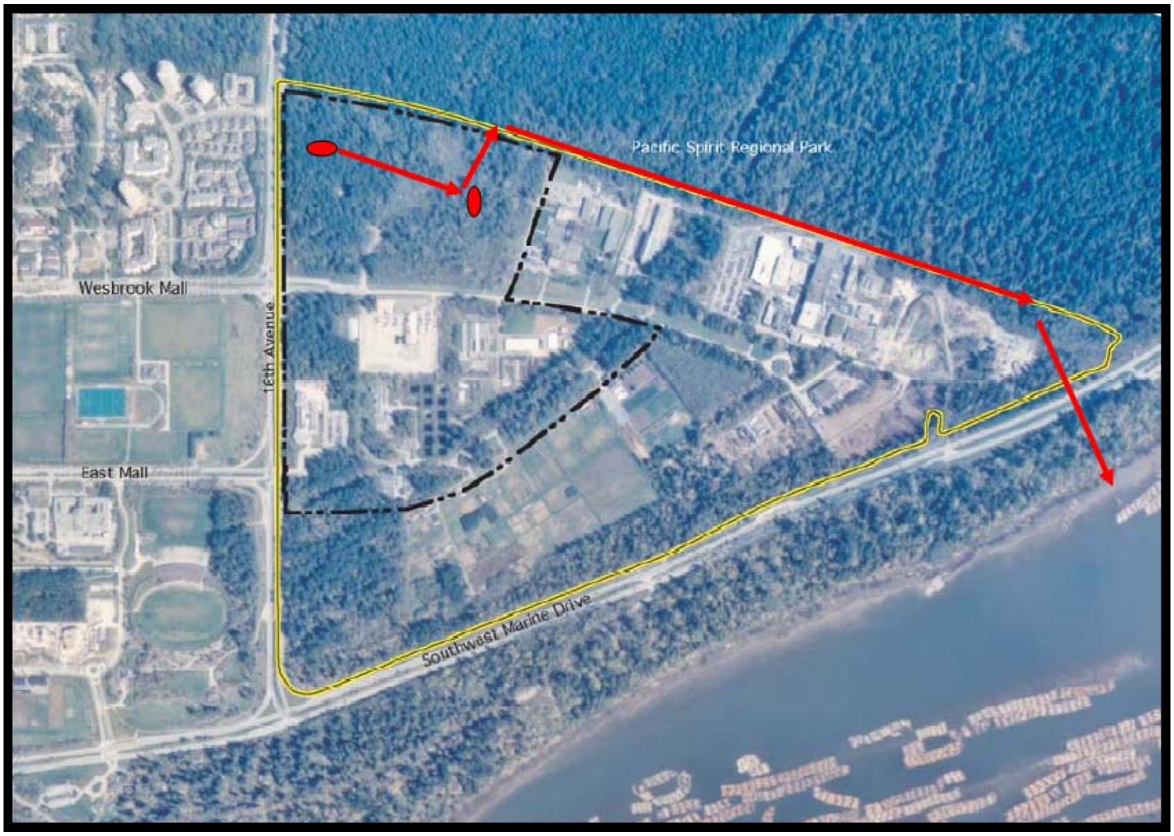


Figure 17. Recommended pond and stream placement. The red ovals indicate pond locations. The red arrows indicate the direction of stream flow. The actual flow path is not a straight line. It will meander with moderate sinuosity to simulate a natural stream.

The total distance of the ponds and stream system is approximately 2.2km.The ponds are situated in areas designated as green space. The upstream pond lies in the SC2H area of development, while the second pond lies in the SC3G area of development (Figure 18). These areas provide adequate space for pond placement. This open area will allow adequate wind to pass over the pond to allow surface disturbance and pond aeration. The uppermost portion of the stream starts from Pond 1, travels down designated greenways, collects flow from Pond 2, and proceeds towards the border of Pacific Spirit Regional

Park. The stream then flows down the edge of the park before discharging into the ocean. The described flow path is illustrated in Figure 18 below.

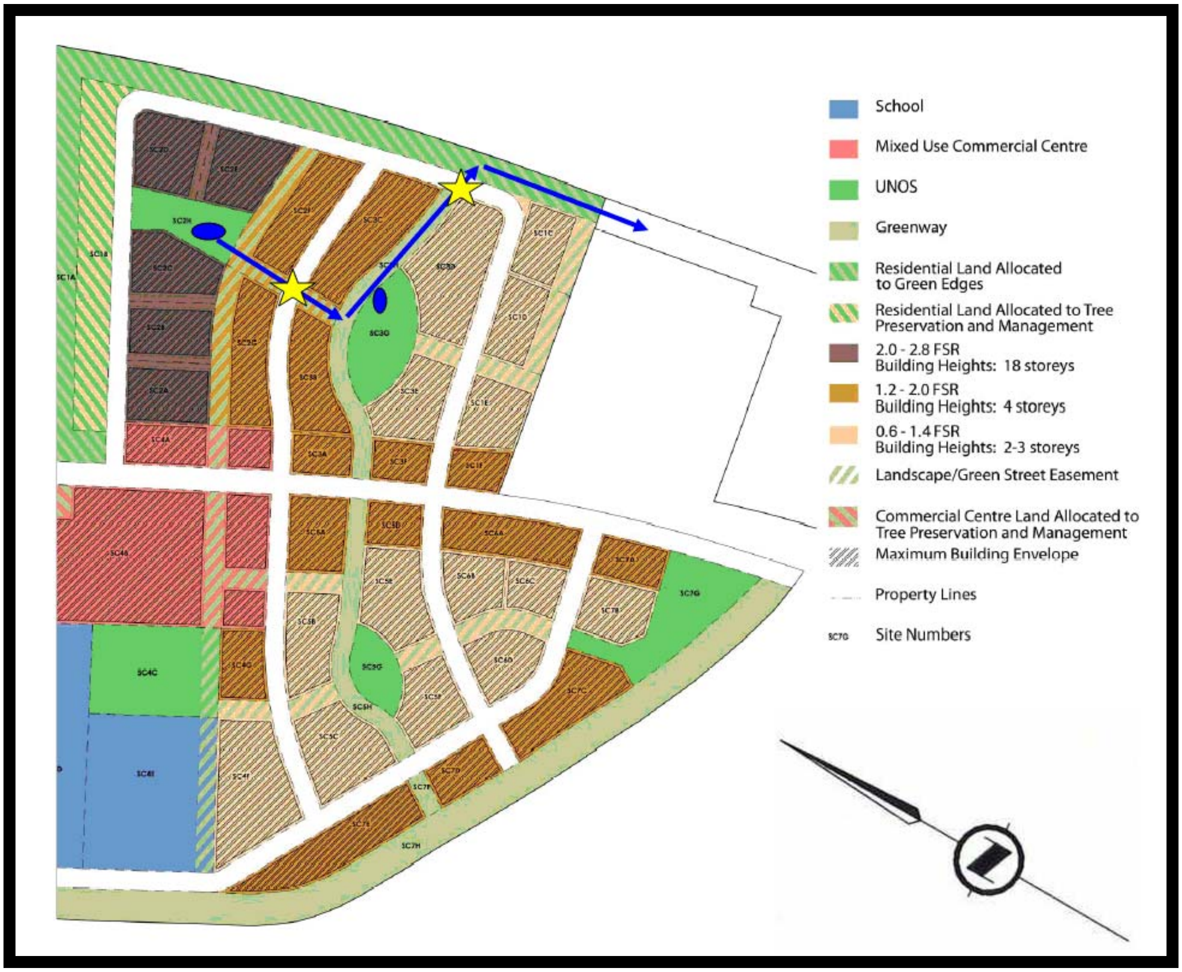


Figure 18. South Campus Land Use Plan and recommended stream and pond locations. The blue ovals indicate pond locations. Pond 1 is located in the designated greenway section SC2H. Pond 2 is located in the designated greenway section SC3G. The blue arrows indicate the recommended initial path of the stream. The two yellow stars indicate where culverts will need to be located to allow the stream to cross roads.

This path was chosen because it allows the stream to follow the natural topography of the land which slopes towards the east and to the south. In addition, the recommended path minimizes the number of roads the stream will cross. Through this path, two roads will be crossed within the South Campus Neighbourhood, thus two culverts will need to be constructed. This flow path is particularly advantageous since it flows along the edge of Pacific Spirit Regional Park. The stream is then able to collect nutrients from the surrounding park forests and carry essential nutrients to young salmon located downstream.

Phase 1.4 Location and Placement of Riffle Pool and Cascade Pool Sequences

The slope of the land is a key parameter in determining whether sections of stream would consist of cascade pool or riffle pool morphology. The upstream portion of the stream located between the two ponds has a steep slope ranging from 3.3 to 5%. This section is ideal for cascade pool sequences (Hogan and Ward 1997). The portion of the stream between Pond 2 and the border of Pacific Spirit Regional Park has a gentle slope of approximately 2%. This portion of the stream is ideal for riffle pool morphology (Hogan and Ward 1997). The portion of the stream traveling along the border of Pacific Spirit Regional Park has a slope ranging from 2 to 4.8%. This slope range is ideal for cascade pool morphology (Hogan and Ward 1997). Figure 19 illustrates the recommended location of cascade pool and riffle pool sequences.

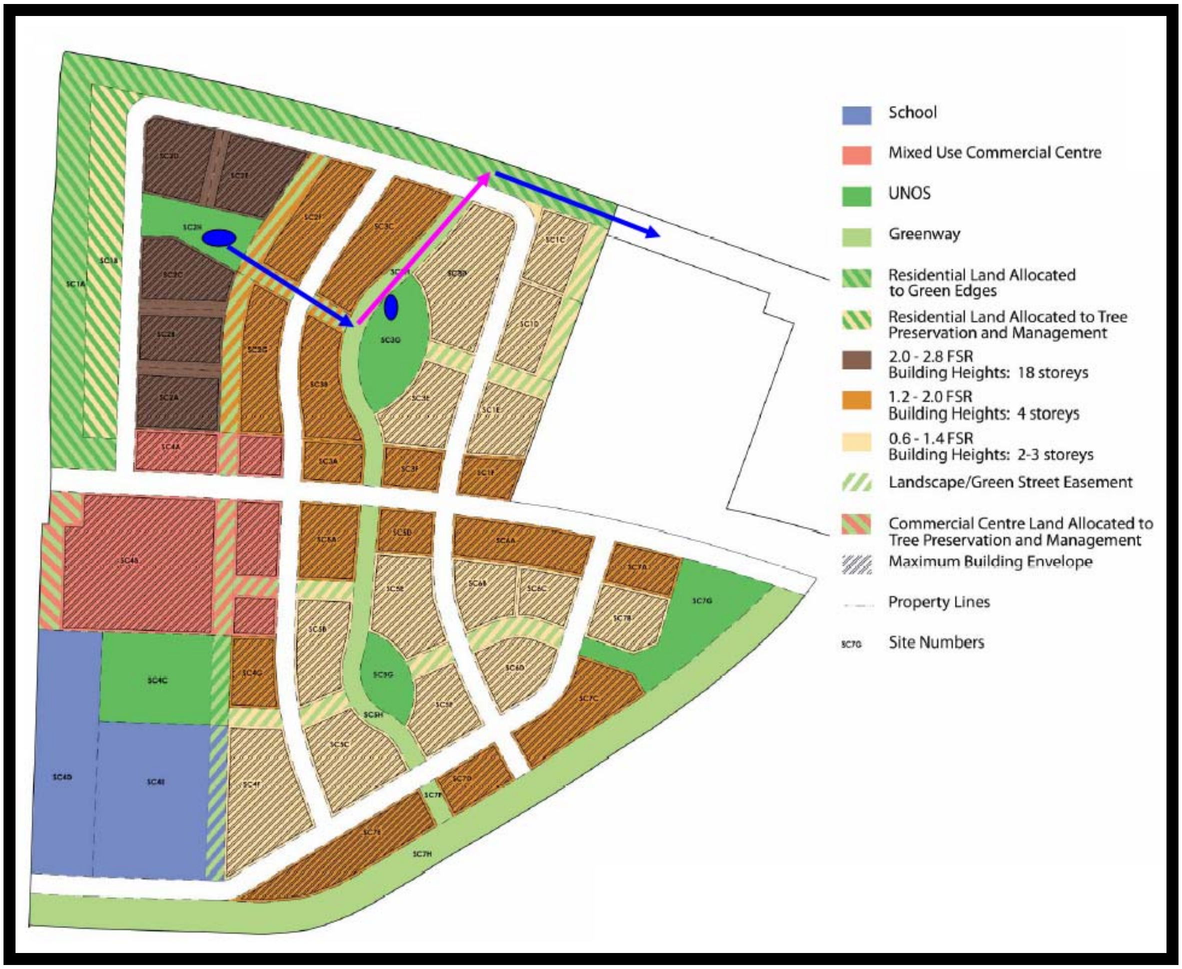


Figure 19. Recommended location of cascade pool and riffle pool sequences. The blue arrows indicate portions of stream which will have cascade pool morphology. The pink arrow indicates the portion of the stream which will have riffle pool morphology.

Based on the chosen stream and pond locations, a conceptual model was developed. This model outlines the general concept behind the stormwater management system which will be developed. The stream model will be designed to be supported by rooftop runoff through the application of fluid mechanics and fish habitat considerations. The conceptual model is displayed in the next section.

PHASE 2 CONCEPTUAL MODEL DEVELOPMENT

Based on the information attained from the completion of Phase 1, a conceptual model was developed. This model describes the pond and stream system layout and flow inputs and outputs. The model was divided into two reaches illustrated in Figure 20.

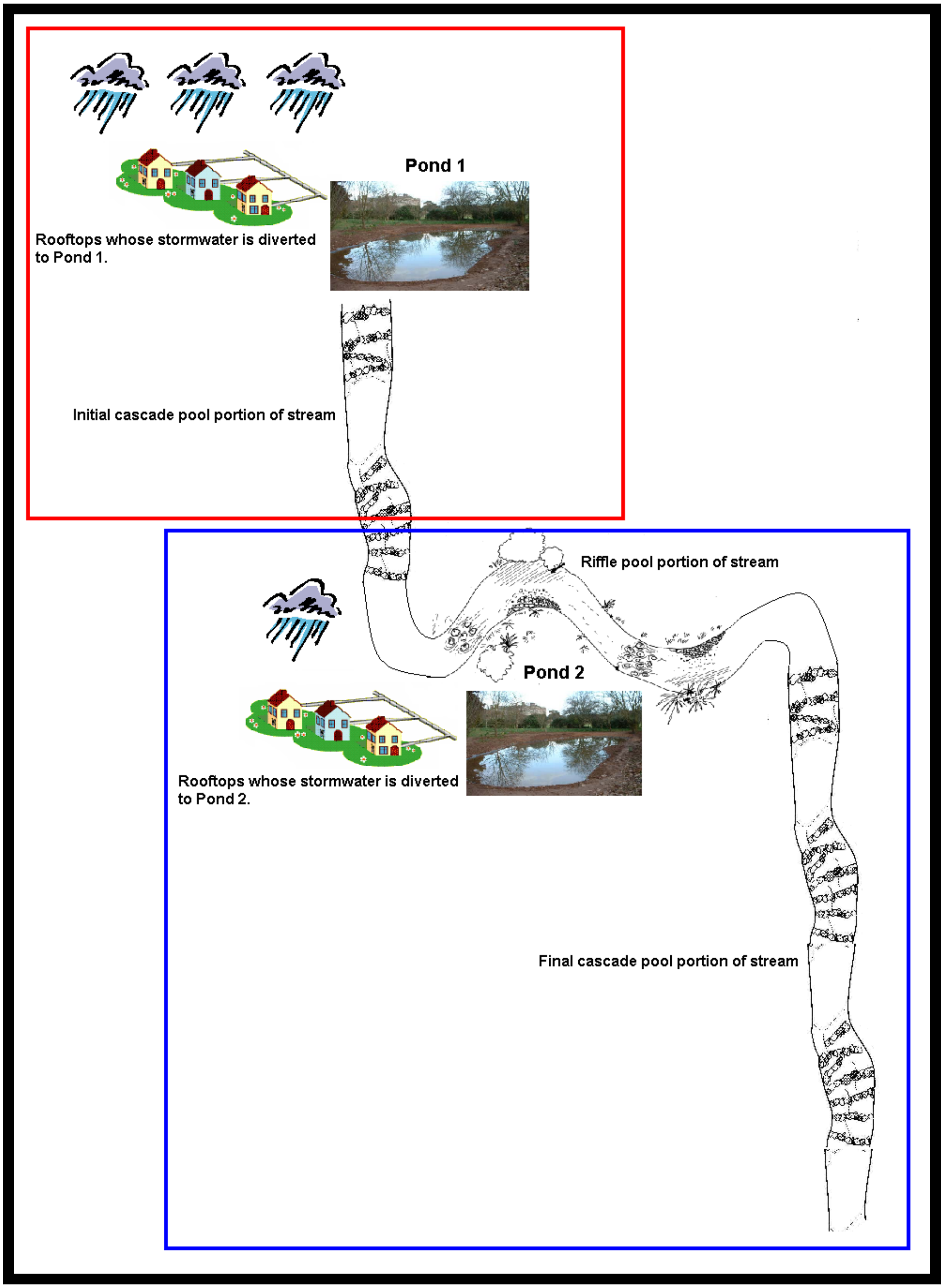
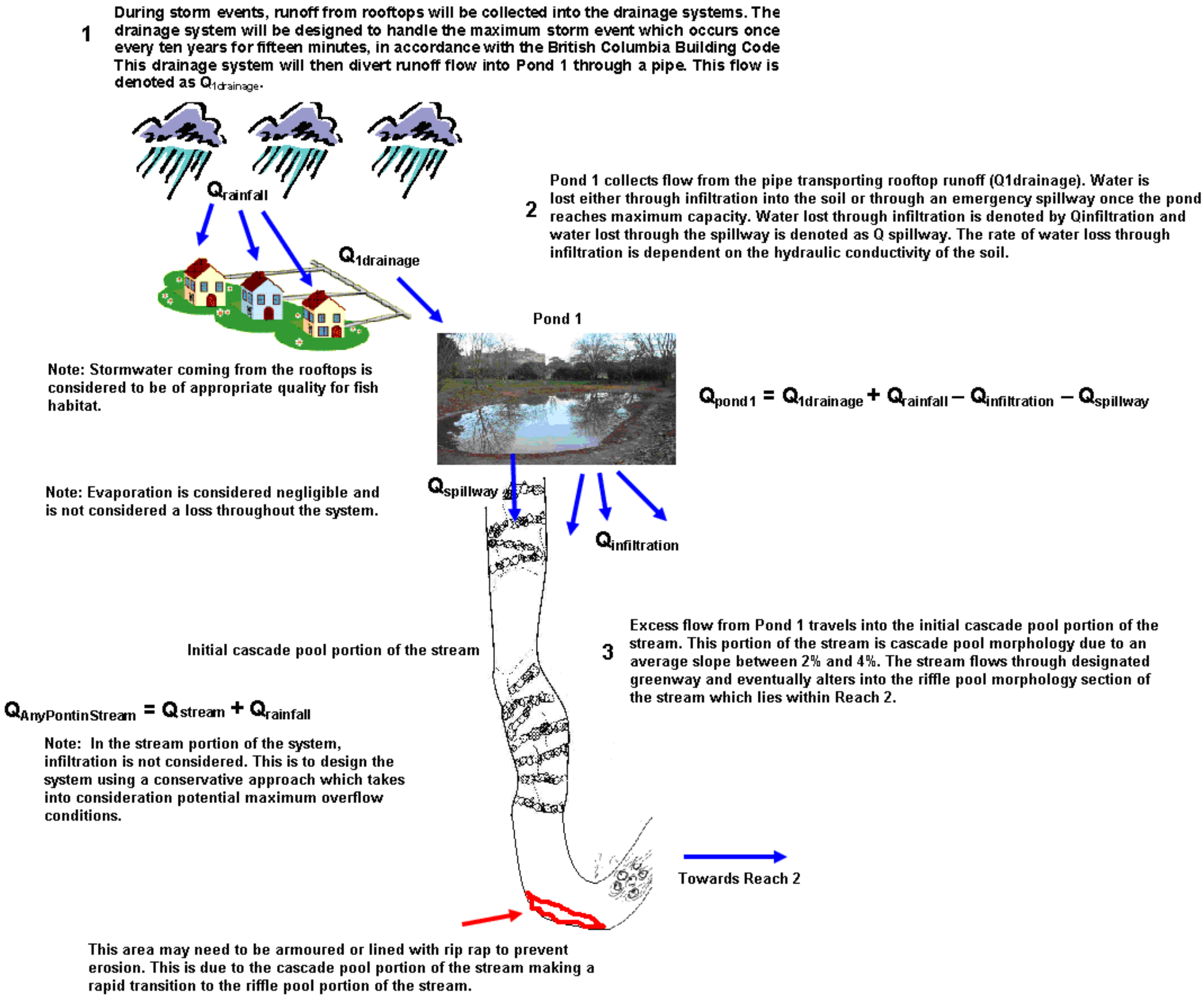


Figure 20. Conceptual ponds and stream model. The area was divided into two reaches. Reach 1 is outlined by the red box and encompasses Pond 1, the surrounding rooftops whose stormwater is diverted to Pond 1, and the initial cascade pool portion of the stream. Reach 2 is outlined by the blue box and includes Pond 2, surrounding rooftops whose stormwater is diverted to Pond 2, the riffle pool portion of the stream, and the final cascade portion of the stream.

Reach 1

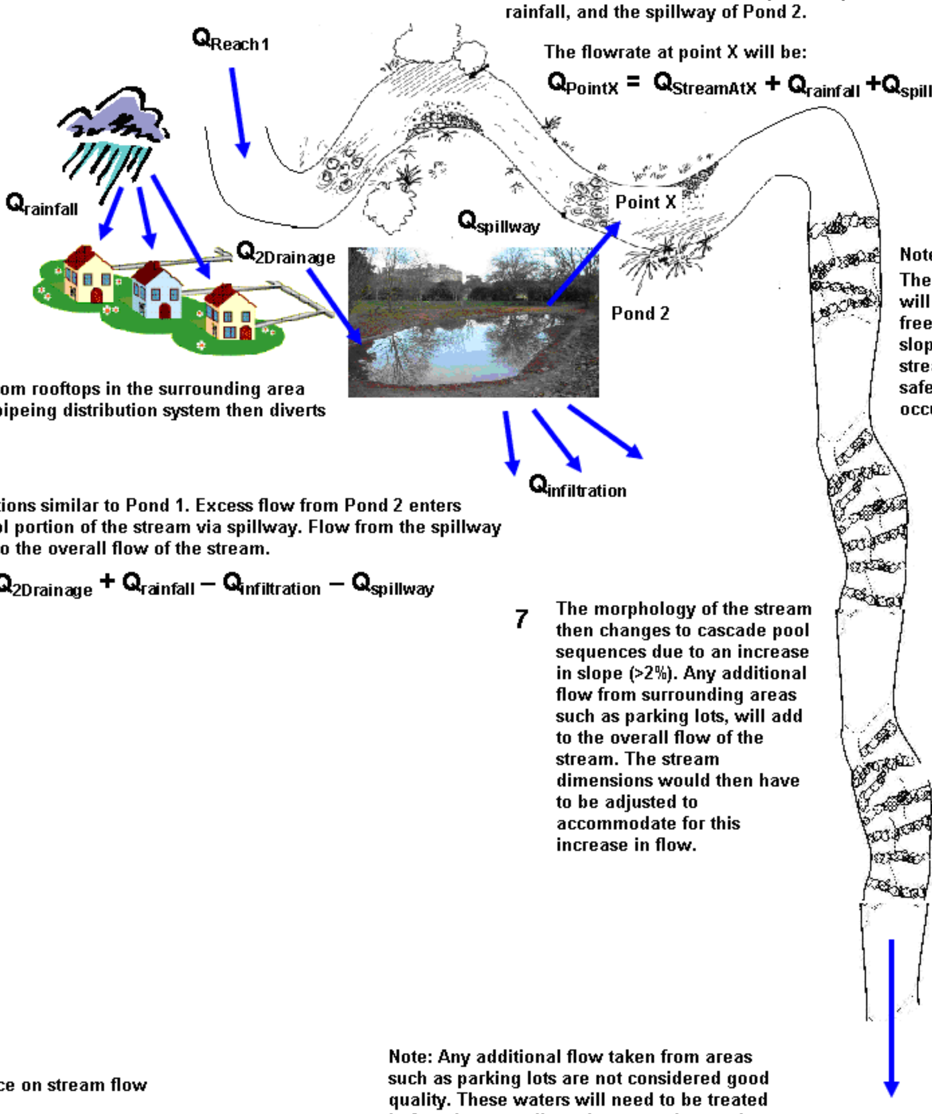

1 During storm events, runoff from rooftops will be collected into the drainage systems. The drainage system will be designed to handle the maximum storm event which occurs once every ten years for fifteen minutes, in accordance with the British Columbia Building Code This drainage system will then divert runoff flow into Pond 1 through a pipe. This flow is denoted as $Q_{1drainage}$.
$Q_{rainfall}$
$Q_{1drainage}$
2 Pond 1 collects flow from the pipe transporting rooftop runoff (Q1drainage). Water is lost either through infiltration into the soil or through an emergency spillway once the pond reaches maximum capacity. Water lost through infiltration is denoted by Qinfiltration and water lost through the spillway is denoted as Q spillway. The rate of water loss through infiltration is dependent on the hydraulic conductivity of the soil.
Pond 1
Note: Stormwater coming from the rooftops is considered to be of appropriate quality for fish habitat.
$Q_{pond1} = Q_{1drainage} + Q_{rainfall} - Q_{infiltration} - Q_{spillway}$
Note: Evaporation is considered negligible and is not considered a loss throughout the system.
$Q_{spillway}$
$Q_{infiltration}$
3 Excess flow from Pond 1 travels into the initial cascade pool portion of the stream. This portion of the stream is cascade pool morphology due to an average slope between 2% and 4%. The stream flows through designated greenway and eventually alters into the riffle pool morphology section of the stream which lies within Reach 2.
Initial cascade pool portion of the stream
$Q_{AnyPontinStream} = Q_{stream} + Q_{rainfall}$
Note: In the stream portion of the system, infiltration is not considered. This is to design the system using a conservative approach which takes into consideration potential maximum overflow conditions.
Towards Reach 2
This area may need to be armoured or lined with rip rap to prevent erosion. This is due to the cascade pool portion of the stream making a rapid transition to the riffle pool portion of the stream.

Reach 2

6 This portion of the stream has riffle pool morphology due to the slope of being less than 2%. The riffle pool portion of the stream collects flow from the upstream portions of the stream, rainfall, and the spillway of Pond 2.

The flowrate at point X will be:

$$Q_{PointX} = Q_{StreamAtX} + Q_{rainfall} + Q_{spillway}$$



Note:
The banks of the stream will have an additional freeboard height of 0.3m sloping at 3:1 towards the stream. This will act as a safety in the rare occurrence of high flow.

4 Again rain from storm events is collected from rooftops in the surrounding area and collected into the drainage system. A pipeing distribution system then diverts the flow into Pond 2 through a pipe.

5 Pond 2 functions similar to Pond 1. Excess flow from Pond 2 enters the riffle pool portion of the stream via spillway. Flow from the spillway contributes to the overall flow of the stream.

$$Q_{Pond2} = Q_{2Drainage} + Q_{rainfall} - Q_{infiltration} - Q_{spillway}$$

7 The morphology of the stream then changes to cascade pool sequences due to an increase in slope (>2%). Any additional flow from surrounding areas such as parking lots, will add to the overall flow of the stream. The stream dimensions would then have to be adjusted to accommodate for this increase in flow.

Note: Culvert design and culvert influence on stream flow will not be considered at this time.

Note: Any additional flow taken from areas such as parking lots are not considered good quality. These waters will need to be treated before they are allowed to enter the ponds and stream system.

Based on the conceptual model, the pond and stream system can be used to estimate the degree in which the following recommended federal and provincial guidelines can be met:

- Post development runoff volumes should roughly equal pre-development runoff volumes (MWLAP 2004).
- The total runoff volume will be limited to 10% (or less) of the total rainfall volume (BC Ministry of Environment 2002).

The pre-developed and post development runoff volumes can be determined through the application of the Rational Formula (Schwab *et al.*1981):

$$Q = ciA \qquad \text{Equation 2}$$

Where,

c, is runoff coefficient.

i, is rainfall intensity expressed in m/s.

Q, is flowrate in m^3/s. Often this flow is taken for a given frequency of occurrence.

A, is the area of the catchment of interest in m^2.

The runoff coefficient will be determined with the use of the table presented in Appendix D. Note that the system is designed to manage the maximum storm even which lasts for 15mintues and occurs once every 10 years. In addition, the south campus area will be considered to have a moderate slope. The runoff coefficient for post development will be 0.85 and the runoff coefficient for predevelopment will be 0.67. The catchment area of interest is the South Campus Neighbourhood which has an area of 301 000m^2.

To determine if post development runoff volumes roughly equal pre-development runoff volumes the following approach can be used:

$$Q_{PostDevelopmentRunofff} - Q_{StreamOutSC} \approx Q_{\Pr edevelopmentRunoff}$$

Where,

$Q_{PostDevelopmentRunoff}$, is the flowrate of runoff after the construction of the South Campus Neighbourhood takes place. Units: m^3/s.

$Q_{StreamOutSC}$, is the flowrate of the stream leaving the South Campus Neighbourhood in m^3/s.

$Q_{PredevelopmentRunoff}$, is the flowrate of runoff before the construction of the South Campus Neighbourhood takes place. Units: m^3/s.

It is important to note that any flow which discharges into the stream is not considered runoff since the stream is considered a natural system (MWLAP 2004). This is why $Q_{StreamOutSC}$ is subtracted from the total runoff created by the post development runoff.

The rational formula assumes that the ground is saturated and can be used to determine runoff volume. The system is designed for the maximum 15mintute storm which occurs once every 10 years. Thus, multiplying the post development runoff flowrates, stream flowrates, and predevelopment flowrates by 15 minutes (900 seconds) will give the corresponding volumes.

$$Q_{PostDevelopmentRunofff} \times 900s - Q_{StreamOutSC} \times 900s \approx Q_{\Pr edevelopmentRunoff} \times 900s$$

$$V_{PostDevelopmentRunofff} - V_{StreamOutSC} \approx V_{\Pr edevelopmentRunoff}$$

Or

$$V_{PostDevelopmentRunoffwithStream} \approx V_{\Pr edevelopmentRunoff}$$

Where,

V is volume in m^3.

A direct comparison can then be made between the post development runoff volume with the addition of the stream, and the predevelopment runoff volume.

To determine if the total runoff volume, with the addition of the pond and stream system, will less than or equal to 10% of the total rainfall volume the following approach will be used:

$$\text{Total Runoff Volume} = \text{South Campus Runoff Volume} - \text{Stream Transported Volume}$$

Or

$$V_{TotalRunoffVolume} = V_{SouthCampusRunoffVolume} - V_{Stream}$$

Also,

$$\text{Total Rainfall Volume} = ciA \times 900s$$

Where,

c, runoff coefficient will equal 1, since want total rainfall volume.

i, is the design rainfall intensity of $9.72 \times 10^{-6} m/s$.

A, is the area of the South Campus Neighbourhood which is 301 000m^2.

If,

$V_{TotalRunoffVolume} \leq V_{TotalRainFall} \times 10\%$

Then the guideline is achieved.

If,

$V_{TotalRunoffVolume} > V_{TotalRainFall} \times 10\%$

Then the guideline is not achieved.

PHASE 3: POND DESIGN AND DIMENSIONING

Key factors to consider when designing and dimensioning a pond include the following:

- Available area for pond placement and sizing.
- Soil type within the pond. Which in turn affects:
    - The infiltration rate of water within the pond
    - The depth of the pond
    - The side slopes of the pond
- Flood and Erosion Protection
    - Bank freeboard
    - Emergency spillway
- Placement of the pipe delivering rooftop runoff
- Vegetation to enhance aesthetics and bank stability

Pond 1 lies in the UNOS area SC2H and Pond 2 lies in UNOS area SC3G (Appendix E). These designated green spaces provide adequate area for Pond placement. UNOS area SC2H encompasses 4320.7m^2 of land, while UNOS area SC3G encompasses 6796.7m^2 of land (Appendix F). These areas provide adequate open space for wind to cause surface disturbance on the ponds water surfaces. This ensures adequate pond aeration (Yoo and Boyd 1994). In addition, in the rare occurrence of pond overflow, water will be able to briefly flow into this area before proceeding back into the pond or infiltrating into the ground.

Soil testing in the proposed pond placement locations has not been conducted. It will be assumed that the soil in the proposed pond locations are similar to the bore hole test findings conducted by GeoPacific Consultants Ltd on the SC3D lot of the South Campus Neighbourhood (Appendix G). The upper 3.5-4m of soil is primarily composed of silt. Beneath this silt layer is sand which has a typical hydraulic conductivity between 5 to 50m/day (Lencastre 1987). If the pond was excavated to a depth were sand was dominant, the infiltration rate would be too high and the pond would have difficulty filling (Yoo and Boyd 1994). To ensure infiltration rates are not too high, it is recommended that the pond be excavated within the silt layer of soil. The typical hydraulic conductivity of soils made up of silt is between 0.1 to 0.5 m/day (Lencastre 1987). Since the silt is described as a silt matrix and based on field observations where standing water was present, an estimated hydraulic conductivity of 0.02m/day will be used for the initial stream design. This hydraulic conductivity lies between typical values for clay and silt (Lencastre 1987). Soil testing will need to be conducted to determine the actual hydraulic conductivity of soil at the pond location.

For the pond model, the ponds will be a depth of 1.5m. This will ensure the soil within the pond is made up of silt and not sand which lies at greater depths. To create aesthetically pleasing and natural looking ponds, the ponds will be oval in shape. To ensure slope stability, slopes made predominantly of silt should have at least a slope of 5:1 (Yoo 1994). For the initial pond model the base of the ponds will be oval in shape with a width of 1.5m and length of 5m. At slopes of 5:1 and a depth of 1.5m, the surface of the ponds will then have a length of 20m and a width of 16.5m. At these dimensions the ponds will be able to hold an approximate volume of $152m^3$ and will encompass a surface area approximately $260m^2$. Appendix H outlines the calculations involved in pond design and dimensioning. Figure 20 below illustrates the pond dimensions.

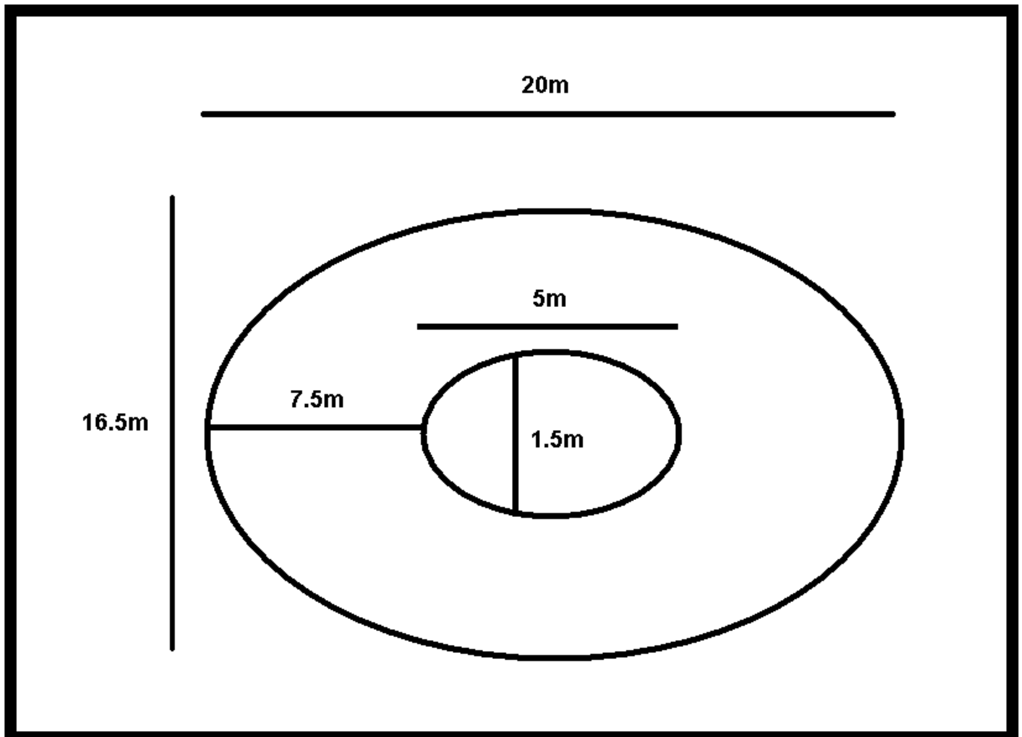


Figure 21. Pond dimensions. Aerial view of pond. The base of the ponds have a length of 5m and a width of 1.5m. The total length of the pond is 20m and the total width is 16.5m. The side slopes of the pond are 5:1. Since the sediment within the pond is primarily silt, the 5:1 side slopes will provide adequate stability.

Bank freeboard is the addition of additional height to the banks of the ponds (Yoo and Boyd 1994). This extra bank helps protect against instances of overflow. Freeboard is often made up of soil excavated out of the pond (Yoo and Boyd). It is recommended that the bank freeboard have a height of at least 0.6m (Yoo and Boyd1994). The slope of the freeboard will be towards the pond at a slope of 5:1, since a slope of 5:1 is recommended for slopes primarily composed of silt. Essentially the effective height of the pond is being extended an additional 0.6m illustrated in Figure 21 below.

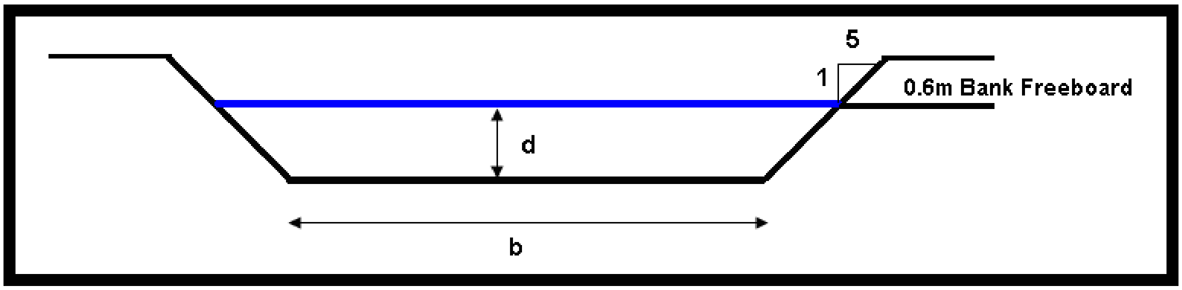


Figure 22. Pond cross section. d, is the base of the pond. d, is the depth design depth. An additional 0.6m bank freeboard is recommended. Freeboard helps mitigate against instances in which water volume exceeds the pond bankfull volume. The slope of the freeboard is 5:1.

To bypass excessive runoff after heavy rains, an emergency spillway on each pond needs to be developed (Yoo and Boyd 1994). Emergency spillways will also mitigate erosion issues during periods of high flow (Yoo and Boyd 1994). To design the spillway, recommendations were taken from the book *Hydrology and Water Supply for Pond Aquaculture* by Kyung H Yoo and Claude E Boyd. The spillway will have a control section and an exit section illustrated in Figure 22 below.

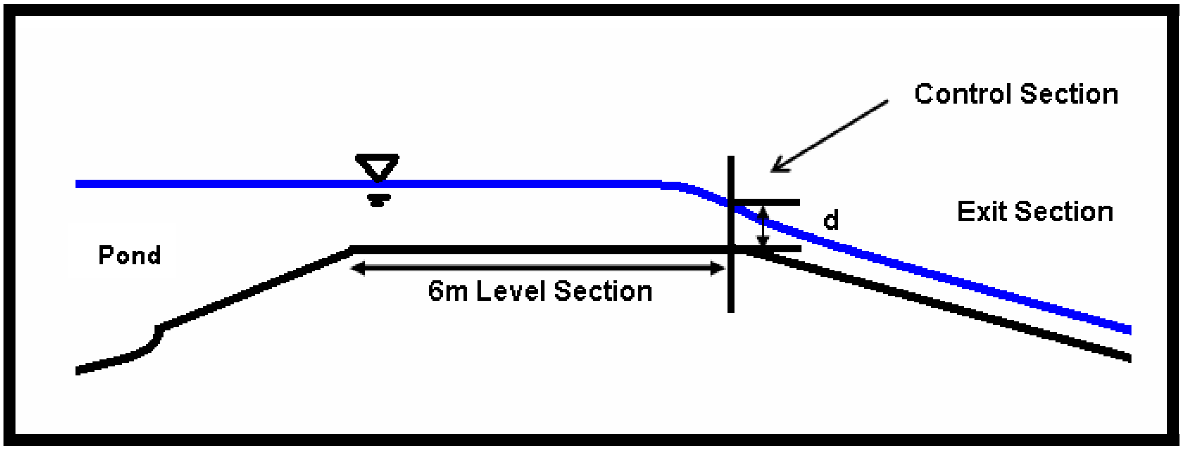


Figure 23. Emergency spillway. The emergency spillway consists of a level control section 6m in length. Excess flow will then discharge through the exit section and into the stream.

The control section of the emergency spillway will be a flat section at least 6m in length and will be lined with natural grass (Yoo 1994). The maximum depth of flow in this section will be at least 0.3m (Yoo 1994). The cross section of the spillway will be rectangular in shape, although an additional freeboard of 0.3m at a slope of 5:1 towards the control section will be added (Yoo 1994). Flow from the control section of the spillway will then discharge into the exit section. The exit section will be lined with natural grass and have a gentle slope of 5:1 until it discharges into the stream system. The width of the control section and exit section will be the same as the initial width of the most upstream portion of the stream which is determined in Phase 3.

The pipe which discharges rooftop runoff will need to be appropriately sized. At this time proper pipe sizing has not been conducted due to the unavailability of South Campus Neighbourhood rooftop data. Details on pipe sizing will be conducted in Phase 5. The depth the pipe enters the pond is dependent on a variety of factors and will be further investigated in Phase 4. The pipe should extend no further than 1/3 the length of pipe into the pond (Schwab *et al.* 1981). The area of pond directly influenced by the pipe discharge should be lined with riprap to prevent murky water created by silt disturbance (Schwab *et al.* 1981). The Vancouver Building Code designs rooftop drainage systems to be able to handle a rainfall event of 15minutes which may occur once every 10 years (BC Building Code 2000). This corresponds to a rainfall intensity of 35mm/hr (Appendix I). The pipe which collects rooftop runoff will be designed to handle a flowrate of at least 35mm/hr.

Vegetation along the banks of the ponds will increase aesthetics and bank stability (Donat 1995). It is important that this vegetation be able to survive during wet and dry periods (Adams 1997). Vegetation will also provide oxygen to the ponds which will mitigate the formation of anoxic conditions (Adams 1997). Ideal vegetation commonly seen near ponds include: Irises, Narrow Leaf Cattail, Softrush, and Stem Bullrush (Adams 1997). These are displayed in Appendix J.

PHASE 4: STREAM DESIGN AND DIMENTIONING

The novelty of this study is to design a stream suited to fish habitat requirements as well as being able to manage stormwater. The stream will manage stormwater by conveying overflow from each pond downstream towards the Pacific Ocean. To ensure the stream conforms to the target species habitat requirements, design took into consideration the following:

Fish Habitat Considerations

- Proper placement and spacing of cascade pool and riffle pool sequences.
- Randomization of cascade, riffle and pool spacing. This ensures a natural looking stream. Spacing typically varies from 6 to 8 times the bankfull width (Newbury *et al*. 1997).
- Degree of meandering (typically 8 to 12 times the bankfull width) and radius of curvature (Leopold *et al.* 1964).
- The bankfull width to cascade/riffle depth ratio should lie between 5 to 1 and 10 to 1 to maintain good habitat (Hunter 1991).
- In stream pools will hold a minimum volume of $1m^3$ (Hunter 1991). This will ensure adequate depth for fish refuge and ensure pond areas do not dry out easily during dry periods.
- Stream velocities will not exceed the critical swimming velocity of ~1m/s (Dane 1978).

- Stream velocities maintain cobble and gravel on the stream bed. Cobble and gravel are characteristic stream bed material found in natural cascade pool and riffle pool sequences (Hogan and Ward 1997). Smaller sediment which is abrasive to fish gills and eggs will be transported out of the stream system.
- If possible a summer time depth of 0.06m should be maintained (Whyte *et al.* 1991).
- Sufficient bank side vegetation for food, cover, and slope stability (Hunter 1991).

Additional Considerations

- The flowrate should increase from upstream to downstream.
- To ensure downstream portions of the stream are able to handle upstream flow contributions, the width of the stream will gradually increase from upstream to downstream.
- To mitigate the effects of periods of high flow, the stream banks will have an additional freeboard bank height of 0.3m which slopes towards the stream at a 3:1 slope.

The location of riffle and cascade pool stream morphology was discussed in Phase 1 and is illustrated in Figure 19. Typical cascades and riffles are spaced 6 to 8 times the bankfull width apart from instream ponds (Leopold *et al.* 1964). To ensure a natural looking stream, a randomization function was utilized using Microsoft Excel. The results are displayed in Appendix K. To achieve sufficiently sinuous stream, the degree of meandering typically occurs at 8 to 12 times the bankfull width with a curvature of radius at approximately 2.3 (Leopold *et al.* 1964). The degree of meandering would normally be determined with the use of a random generator to simulate a natural environment. Since space is limited it may not be feasibly to randomize meandering. Appendix L displays the results of a random generator for meandering. The degree of meandering and curvature of radius will be determined after a survey has been done along the edge of Pacific Spirit Regional Park to best determine how much space is available for meandering.

The stream was divided into fourteen sections based on the contour lines on a topography map of the south campus area. These fourteen sections were labeled A to N and are displayed in Appendix M. The slope of each section was determined (Appendix N). Areas with slopes less then 2% were designed to have riffle pool morphology, while slopes between 2% and 5% were designed to have cascade pool morphology. Riffle portions of the stream will have a stream bed primarily made up of gravel which has a diameter ranging from 2 to 64mm (Arcement and Schneider 2005). Cascade portions of the stream will have a stream bed primarily made up of cobble and gravel. Cobble varies in diameter from 64mm to 256mm (Arcement and Schnieder).

The chosen channel morphology is flat bottomed with vertical sides (Figure 23). This type of morphology will become more rounded as the stream matures. It will also promote the formation of undercut banks which are ideal for fish refuge (Appendix O). An additional freeboard bank height of 0.3m sloping towards the stream will be placed on the stream banks to mitigate any occurrence of overflow (Figure 23). The typical stream cross section is illustrated in Figure 23 below.

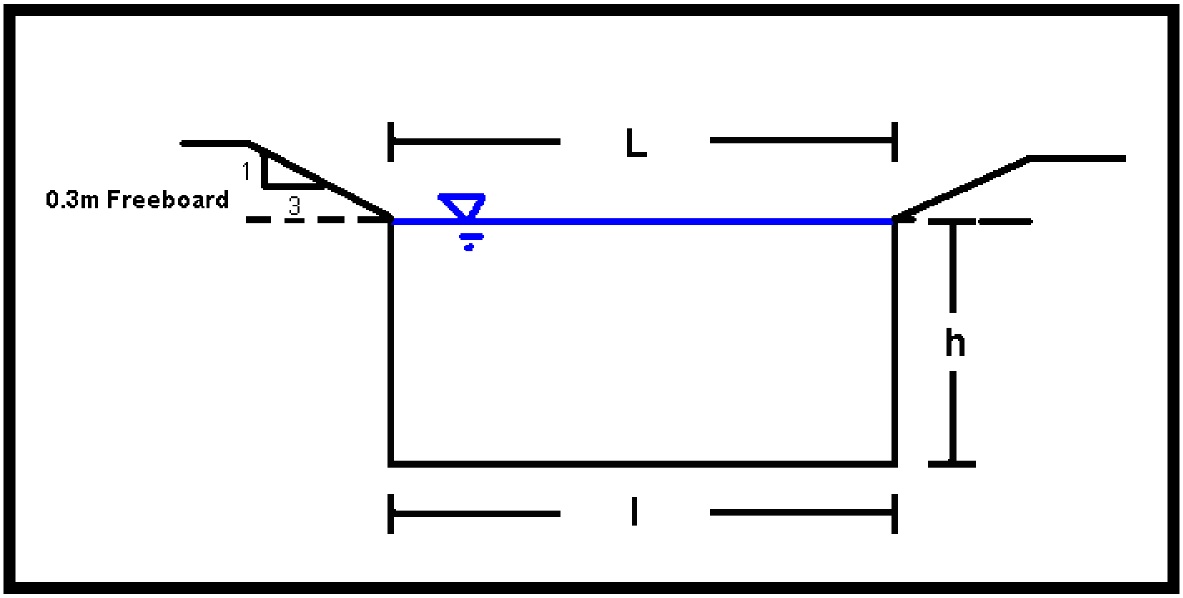


Figure 24. Cross sectional view of stream. L, is the upper stream width. l, is the stream bottom width. h, is the flow height. Above the maximum height of flow is a freeboard depth of 0.3m which slopes towards the stream at 3:1.

Since space for stream placement within the South Campus Neighbourhood is limited, the upstream portions of the stream are restricted to designated greenways and areas allocated to tree preservation. These areas range from 14m to 18m in width (Alpin and Martin 2005). It is important for the stream to lie within these widths. The most upstream portion of the stream is situated in section A (Appendix M). This section of stream was taken to have a width of 0.75m. With the additional freeboard height the stream width including freeboard is approximately 2.6m (Appendix P). This width allows the upstream portion of the stream to lie well within the greenways and areas allocated to tree preservation. The upstream width of 0.75m will also be the width of the spillway for

Pond 1 and Pond2. At the recommended width to depth ratio of 5:1, the depth of the stream in Section A was determined to be 0.15m (Appendix Q). This depth is well above the minimum required depth of 0.06m during summer periods.

In each section, the average flow rate of the stream was determined through application of the Manning's equation displayed below.

$$\boxed{Q = AR^{\frac{2}{3}} S^{\frac{1}{2}} \frac{1}{n}, \text{(SI Units)}} \qquad \text{Equation 1}$$

Where,

Q, is flowrate of the stream in m^3/s.

A, is cross sectional area of the stream in m^2.

S, is channel slope.

R, is the hydraulic radius of the channel (m), and can be defined as the channel area divided by the wetted perimeter.

n, is roughness coefficients which vary depending on the channel conditions.

The Manning's equation was not applied to the pond portions of the stream. This is because the ponds are considered deep and essentially have a velocity of zero in comparison to the cascade and riffle portions of the stream.

The Manning's coefficient was determined using the procedures outlines in Appendix R. The method outlined by Arcement and Schneider, takes into consideration channel irregularities, variation in channel cross-section, effects of obstruction, effects of vegetation, and the degree of meandering. The table below illustrates the contribution of each of these effects to the Manning's Roughness coefficient. For many of these coefficient values, a range was given. The lower and upper ranges of these values were taken to determine the lower and upper Manning's roughness coefficient values for gravel and cobble lined streams. This allowed a sensitivity analysis to be conducted to determine the effect of Manning's roughness coefficient on stream flowrate.

Table 4. Manning's Roughness Coefficients. The upper and lower Manning's coefficient values for gravel and cobble are highlighted below.

Manning's Roughness	**Gravel**		**Cobble**	
Coefficients	Low	Upper	Low	Upper
nb: base value	0.028	0.035	0.030	0.050
n1: channel irregularities	0.001	0.005	0.001	0.005
n2: variation in channel x-sec	0	0	0	0
n3 effect of obstruction	0.005	0.015	0.005	0.015
n4 effect of vegetation	0.002	0.010	0.005	0.010
m: degree of meandering	1.150	1.150	1.150	1.150
n=(nb+n1+n2+n3+n4)m	0.041	0.075	0.047	0.092

Gravel attained a lower Manning's coefficient value of 0.041 and an upper value of 0.075. Cobble had a lower Manning's coefficient value of 0.047 and an upper value of 0.092.

The average flowrate of each stream section was then determined using the upper and lower Manning's coefficients. Important parameters of each stream section were tabulated in Microsoft Excel. Figure 24 below illustrates and example of this tabulation.

Section	A	Units
Morphology	Cascade-pool	
Length	91.44	m
Slope	3.30%	
Average Velocity	0.87	m/s
Average Q	0.10	m^3/s
Area	0.11	m^2
R	0.11	m
n, low	0.04715	
Channel Width	0.75	m
Channel Height	0.15	m
Pool Dimensions		
Pool Width	0.75	m
Pool Height	0.45	m
Pool Volume	1.6	m^3

Section	A	Units
Morphology	Cascade-pool	
Length	91.44	m
Slope	3.30%	
Average Velocity	0.45	m/s
Average Q	0.05	m^3/s
Area	0.11	m^2
R	0.11	m
n, high	0.092	
Channel Width	0.75	m
Channel Height	0.15	m
Pool Dimensions		
Pool Width	0.75	m
Pool Height	0.45	m
Pool Volume	1.6	m^3

Figure 25. Sample tabulation of stream section A. The chart to the left utilizes the lower end Manning's coefficient value, while the chart to the right utilizes the upper end Manning's coefficient value. The effects of varying Manning's coefficient on flowrate could then be directly compared.

To ensure the downstream portions of stream had a higher flowrate than the upstream sections, stream depths and widths were varied accordingly with the design considerations in mind (Appendix S). Pool depth was varied to ensure a volume of at least $1m^3$. The recommended stream dimensions are outlined in Appendix T. Note that these dimensions will change depending on the current conditions.

It is important to note that any additional input of flow into the stream will increase the overall stream flow. For example, in section D the riffle pool portion of the stream will receive flow from the Pond 2 spillway during periods of pond overflow. The stream must be designed to be able to handle this additional flow. Thus the rate of flow in section D includes the additional flow from the Pond 2 spillway. This addition increases the overall dimensions of the stream. If at any section additional flow is diverted into the stream, the dimensions of the section and those downstream of it must be adjusted to be able to manage the additional flow.

Using the lower end Manning's roughness coefficient the furthest upstream flowrate is $0.10m^3/s$ with a velocity of 0.87m/s (Appendix T). The furthest downstream portion has a flowrate of $0.70m^3/s$ and a velocity of 1.30m/s (Appendix T). Using the upper end Manning's roughness coefficient, the furthest upstream flowrate is $0.05m^3/s$ with a velocity of 0.45m/s (Appendix T). The furthest downstream portion has a flowrate of $0.36m^3/s$ and a velocity of 0.66m/s (Appendix T). The streambed will be lined with gravel in the riffle pool section and predominantly cobble in the cascade-pool sections since these sediments are ideal for salmon spawning. Cobble has a diameter of 64mm to 256mm and gravel has a diameter ranging from 2mm to 64mm. Ideal substrate size for cutthroat is 6mm to 102mm (Whyte et al. 1991).

The upstream and downstream velocities determined using the lower and upper end Manning's roughness coefficients will promote the sedimentation of gravel and cobble.

Smaller streambed material will either be transported or eroded away. This is illustrated in Figure 25 below.

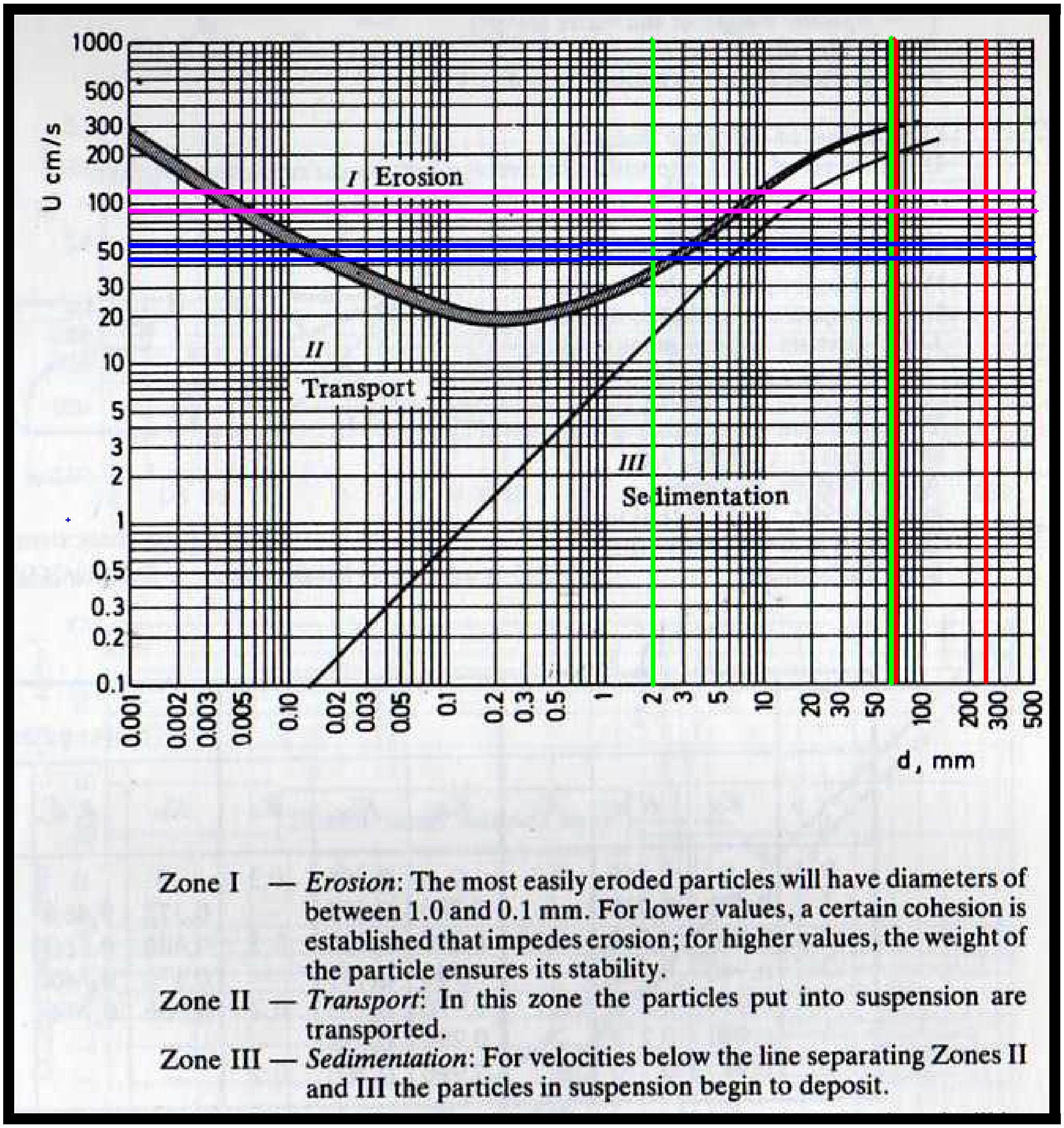


Figure 26. Critical velocity with respect to streambed material (sediment) diameter. The vertical green lines indicate the range of gravel size from 2mm to 64mm. The vertical red lines indicate the range of cobble size from 64mm to 256mm. The horizontal pink lines indicated the highest velocity of 130cm/s and the lowest velocity of 87cm/s attained using the lower end value of Manning's roughness coefficient. The horizontal blue lines indicate the highest velocity of 66cm/s and the lowest velocity of 45cm/s attained using

the upper end value of Manning's roughness coefficient. Where the horizontal lines cross with the vertical lines, indicate the transport zone the stream bed is experiencing.

Figure 26 indicates that the majority of gravel and cobble material will undergo sedimentation and remain on the streambed. Smaller diameter sediment will be transported or eroded downstream. When velocities reach greater than 16cm/s the smaller diameter gravel material erode or transport downstream. From the figure it appears that over time the stream bed will predominantly be made of sediment with a diameter greater than 10mm. This is ok since the majority of the stream is cobble pool sequence and since ideal sediment size for the target species is between 6mm and 102mm.

PHASE 5: PIPING DISTRIBUTION SYSTEM DESIGN

A detailed design of the piping distribution system was unable to be conducted. This was due to time limitations and the inability to receive information on the buildings which will be constructed on the South Campus Neighbourhood. It will be assumed that the buildings on South Campus conform to the British Columbia Building Code and are thus designed to handle a one in 10 year storm event which lasts 15minutes. This corresponds to a flowrate of 35mm/hr or 9.72×10^{-6} m/s. Focus will be placed on the important design considerations of a pipe which collects stormwater at a rate of 9.72×10^{-6} m/s and conveys flow into the ponds.

Four important design considerations which will be discussed include:

- Outlet Type
- Pipe Depth
- Pipe Size
- General Design and Layout Procedures

The two major types of outlets include gravity and pump outlets (Schwab *et al.* 1981). Pump outlets require the use of a pump to convey water through the outlet and are typically used when the water level at the outlet is greater than the bottom of the pipe outlet for an extended period of time (Schwab *et al.* 1981). Gravity outlets are more common and simply use the force of gravity to transport water (Schwab *et al.* 1981). Common design recommendations include:

- A minimum distance of 30cm between the pipe outlet and water level of the pond.
- The length of the outlet should not extend over 1/3 of the pipe length.
- The pipe should be made out of a metal material and have a minimum length of 5m.
- The addition of a flap, grid, or tide gate on the outlet to prevent rodents from entering the pipe.

(Schwab *et al.* 1981)

These design recommendations are illustrated in Appendix U.

A properly designed outlet should provide a free outlet with minimum maintenance, discharge outflow without erosion or damage to pipe, keep out rodents and small animals, prevent the end of the outlet from excessive freezing and thawing, and prevent the entrance of during periods of high flow (Schwab *et al.* 1981).

.

The depth of the pipe depends on soil permeability, outlet depth, lateral spacing, depth to the impermeable layer in the subsoil, and limitations of excavation equipment. Pipe depth is defined as the distance from the surface to the bottom of the pipe. The cover over the pipe should be at least 60cm (Schwab *et al.* 1981). This will protect the pipe from heavy surface loads. If the pipe is placed at a depth below an impermeable layer, then backfilling should be done with permeable soil. (Schwab *et al.* 1981)

The size of the pipe will vary depending the drainage coefficient (term used for velocity), drainage area, pipe roughness, and the slope of the terrain. Basic drain size can be determined using the following formula:

$$\boxed{d = 51.7 \times (D_c \times A \times n)^{0.375} \times s^{-0.1875}}$$

Where,

d, is inside drain diameter in mm

Dc, is drainage coefficient in mm/d

A, is drainage area in hectares

n, is Manning's roughness coefficient

s, is the drain slope

(Schwab *et al.* 1981)

A nomograph is often used in pipe sizing but will not be discussed.

When designing the layout of a pipe distribution system, the system needs to be coordinated with existing and proposed drains. Procedures include preliminary surveys of the area, soil investigation to determine practicality and economic feasibility, and the use of topography maps. In general, the British Columbia Building Code for pipe distributions systems and drainage will need to be followed.

5.0 RESULTS AND DISCUSSION

With the addition of the pond and stream system, post development runoff volumes differed from pre development runoff volumes by an average of 15% (Appendix V). Without the pond and stream system, the post development runoff volumes differed from the pre development runoff volumes by 21% (Appendix V). Thus, the addition of the pond and stream system will decrease the level of runoff volume by approximately 6%. These values were determined by averaging the results attained using the lower and upper Manning's coefficients. This 6% improvement corresponds to a reduced runoff volume of $134m^3$ during a 15minute, one in 10 year storm event. Although the post development runoff volumes did not equal the predevelopment runoff volumes, the 6% decrease in runoff volume is a significant step towards achieving post development runoff volumes which roughly equal predevelopment runoff volumes.

The pond and stream system was unable to limit the total runoff volume to 10% or less of the total rainfall volume. The pond and stream system was only able to limit the total runoff volume to approximately 78% of the total rainfall volume (Appendix V). This guideline may have been difficult to achieve since the pond and stream system is only taking runoff from a small area of rooftops in comparison to the large area which makes up the South Campus Neighbourhood. If only the immediate area surrounding the pond and stream system were considered, then the total runoff volume would be closer to the goal of 10% or less of the total rainfall volume.

A pond and stream model, which is supported by rooftop runoff, was able to be developed through the use of fluid mechanics and the consideration of fish habitat. The consideration of fish habitat made this study a novel approach towards better stormwater management.

The use of two ponds to collect rooftop runoff enhanced the stormwater management capabilities of the pond and stream system. This is because the ponds are able to handle a large volume of rooftop runoff before discharging flow into the stream system. It was found that Pond 1 could manage 17 150m^2 rooftop area and Pond 2 can manage 7891m^2 rooftop area (Appendix W). This was done by assuming the ponds were full from a storm event, and proceeding to fill from the occurrence of a 15minute storm event which occurs once every 10years. The flowrate into the Pond 1 spillway had to be less than the upstream portion of the stream, while the flowrate into the Pond 2 spillway had to be less than the flowrate in Section D of the stream. The corresponding area of rooftops the pond and stream system could handle were then determined (Appendix W). The actual area of the rooftops which will reside in the South Campus Neighbourhood is unknown. These areas are large considering a typical lot in the area is 5 000m^2. The addition of ponds should greatly reduce the demand on any artificial stormwater structure which may be in place. The ponds are able to hold a volume of 152m^3. At an average rainfall intensity of 0.017m/day for the month of November, it was found that each pond would take approximately 10 days of steady rain to fill (Appendix X). Any excess volume would be discharged into the spillway. The pond will also lose volume through infiltration. Infiltration is dependent on the hydraulic conductivity of a soil. If the hydraulic conductivity of the pond material is too high, the pond will be unable to maintain a base flow. The ponds were designed to have a depth of 1.5m since this depth remained in the silt matrix layer of the soil. If the depth proceeded past approximately 2-3m, then the pond would reach the sandy layers of soil and loss through seepage would be fast. Phase 3 further describes in detail the design and dimensioning of the ponds.

Stream design took into consideration major urban stream issues such as overflow, erosion, and periods of high and low flow. The design of the stream did not consider infiltration. This was to design the stream under a conservative approach to take maximum flow conditions into consideration. To mitigate rare occurrences of overflow, the stream banks were designed to have an additional freeboard height of 0.3m sloping at 3:1 towards the stream. The stream was designed to prevent erosion by designing the stream for velocities which promote the sedimentation of cobble and gravel, which are suited to the target fish species. Smaller particles which may be abrasive to fish eggs or gills would be transported down stream. In addition, vegetation should be planted on the stream banks to further protect against erosion. The stream will experience variable periods of high flow during the winter months and low flow during the summer months. To determine how variable the flow is, it will be important to excavate into the stream location to determine how much base flow may be possible through ground water seepage. The final stream design had a maximum width of 1.8 meters at the downstream portion of the stream and a depth to width ratio of 1:6 (Appendix T). This lies well within the requirement for good habitat which typically has a depth to width ratio of 1:5 to 1:10. Thus, the stream could become larger to accommodate the input of additional flow if needed. If the stream is sized to become larger, the designer needs to consider space availability and whether or not the culvert which crosses Southwest Marine Drive can handle the flow. The stream becomes wider from upstream to downstream and the flowrate through the stream increases from the upstream to downstream (Appendix T). This ensures the downstream portions of the stream are able to handle the flow contribution from the upstream portion. Any additional flow from sources not considered to have good water quality will have to be treated. The method of treatment will warrant investigation and vary depending on the type of contaminants which may be present.

Stream velocities using the lower end Manning's coefficient ranged from 0.87m/s to 1.29m/s (Appendix T). Stream velocities using the higher end Manning's coefficient ranged from 0.45m/s to 0.66m/s (Appendix T). Stream velocities at using the low end Manning's coefficient slightly exceeded the maximum swim velocity of 1m/s for cutthroat trout. It is recommended that the stream be designed to have a Manning's coefficient closer to the higher end Manning's coefficient. The higher end Manning's coefficient provided ideal stream velocities for the target species. Manning's roughness coefficient can be increased through increased stream complexity. This includes the addition of structures such as large woody debris and vegetation.

A survey along Pacific Spirit Regional Park will need to be conducted to determine how much space is available for meandering. The random generated chart in Appendix L can be used, although it is recommended that meandering be varied based on field observations due to the limited space availability. Additional details on stream design and dimensioning are discussed in the Phase 4 portion of the report.

Unfortunately due to the inability to receive sufficient information on the buildings which will be constructed on the South Campus Neighbourhood, a piping distribution system was unable to be developed at this time.

It is difficult to predict whether the stream will be able to maintain a base flow throughout the year. During the months of high precipitation (October to May), it is expected that the stream will be able to maintain a base flow. This cannot be guaranteed during the summer months. Since little precipitation is received during the summer months, it is likely that the ponds will become completely dry. This will be acceptable since it will prevent any type of mosquito problem which may occur. It is recommended that if the stream system is applied, it be allowed to function under continual monitoring to determine seepage rates into the stream and how well the system is functioning. Once the system is determined to be functioning properly then fish may be reintroduced.

6.0 CONCLUSION AND RECOMMENDATIONS

A design methodology which can be used to develop a pond and stream system suitable for South Campus and fish habitat was developed. This methodology may be used as a template whenever the restoration of a stream is being considered as an option to manage urban stormwater runoff.

With the addition of the pond and stream system, post development runoff volumes differed from pre development runoff volumes by an average of 15%. Although the post development runoff volumes did not equal the predevelopment runoff volumes, the 6% decrease in runoff volume is a significant step towards achieving post development runoff volumes which roughly equal predevelopment runoff volumes. The pond and stream system was unable to limit the total runoff volume to 10% or less of the total rainfall volume. The pond and stream system was only able to limit the total runoff volume to approximately 78% of the total rainfall volume. This guideline may have been difficult to achieve since the pond and stream system is only taking runoff from a small area of rooftops in comparison to the large area which makes up the South Campus Neighbourhood.

The pond and stream model was able to be developed through the application of fluid mechanics and the consideration of fish habitat. In addition, major urban stream issues were addressed. Unfortunately a proper piping distribution system which will discharge rooftop runoff into the ponds was able to be developed. This is something which can be developed in the future once the required information is available.

It is important to note that this study provides a methodology outlining key design considerations when attempting to design a pond and stream system which manages stormwater runoff while taking fish habitat criteria into consideration. Before the methodology is carried out, the following recommendations for future work should be considered:

- Excavation at various locations along the stream path to determine how much base flow can be achieved.
- Percolation tests need to be conducted to determine the hydraulic conductivity of the soil. More accurate seepage rates can then be attained.
- Survey of the land bordering Pacific Spirit Regional Park. This will determine the amount of available space for stream meandering.
- Once the necessary building structure information is attained, a proper piping distribution system can be constructed to convey stormwater runoff from rooftops to the ponds.
- If the stream is put in place, allow it to run for a few years. During this time periodic monitoring will allow seepage rates into the stream to be determined.
- Detailed cost analysis to determine the feasibility and possible cost savings of implementation of this methodology.
- Implementation of such a project involves the considerations of all stakeholders, which may be influence. A multidisciplinary approach is necessary to achieve success and ensure all concerns are addressed.

RESEARCH IMPLICATIONS

The possible implication of this study is that an urban stream which manages both stormwater and fish habitat criteria flow through the University of British Columbia campus, restoring the fish bearing stream which previously existed. The aim is for fish to one day utilize the stream as habitat and spawning areas. Implementation of the stream and pond system will also help the University of British Columbia to strive towards meeting federal and provincial stormwater management guidelines. On a larger scale, the research promotes sustainability through fish habitat conservation, reduced peak runoff flows, and efficient use of water resources.

ACKNOWLEDGEMENTS

I would like to thank the following people for their contributions towards this study.

Dr. Petrell – Thesis Advisor

- For her time, contribution, patience, and faith in me to undertake this project.

Brenda Sawanda – UBC Sustainability Office

- For her contribution and enthusiasm for the project.

David Grigg – Associate Director Infrastructure and Services Planner UBC

- For his time and input into the project.

Bianca Gunther – Geography

- For her contribution through her GPS and mapping abilities.

Rob Little and Carolina Silva – Economic Analysis

- For conducting cost analysis on the feasibility of the project.

REFERENCES

Arcement GJ and Schneider VR. 2005. Guide for Selecting Manning's Roughness Coefficients for Natural Channels and Flood Plains. United States Geological Survey Water-supply Paper 2339. 67 pp

Adams M. 1997. Fish Habitat Enhancement: A manual for Freshwater Estuarine and Park Habitat. Department of Fisheries and Oceans. 324 pp

Aplin and Martin Consultants LTD. 2005. A Sustainable Drainage Strategy for the South Campus Neighbourhood. 30 pp

British Columbia Building Code. 2000. City of Vancouver.
Pp

British Columbia Ministry of Environment. 2002. Stormwater Planning Guidbook: Policies for Integration of Land Use Planning and Stormwater Management Chapter 4. 17pp

Coduto D.P. 1999. Geotechnical Engineering: Principles and Practices. Prentice Hall. Upper Saddle River. 759 pp

Cowan W. L. 1956. Estimating hydraulic roughness coefficients. Agricultural Engineering, Vol 37, no. 7, pp 473-475

Dane B.G. 1978. A review and resolution of fish passage problems at culvert sites in British Columbia 4th Edition. Department of Fisheries and Environment, Pacific Region, Fisheries and Marine Service Technical Report No. 810. pp 126

Department of Fisheries and Oceans (DFO). 2005.
http://www.dfo-mpo.gc.ca/
Accessed November 2005

Donat M. 1995. Bioengineering Techniques for Streambank Restoration: A Review of Central European Practices.

Finkenbine J. 1998. The impact of urbanization on stream habitat in Lower Mainland British Columbia. University of British Columbia

Finnemore E.J. and Franzini J.B. Fluid Mechanics with Engineering Applications 10th Edition. McGraw Hill. New York. 790 pp

Freeman, G.E.1994. The Scientific Assessment and Strategy Team Contributions Assessing the 1993 Flood on the Mississippi and Missouri River Basins. Water International, 19 (1994) 177-185, International Water Resources Assoc., USA.

Hogan D.L. and Ward B.R. 1997. Watershed Geomorphology and Fish Habitat. Fish Habitat Rehabilitation Procedures. Chapter 2 pp 2-1: 2-18

Hunter C.J. 1991. Better Trout Habitat: A Guide to Stream Restoration and Management. Island Press. New York. 320 pp

Huth M.J. 1978. The urban habitat: past, present, and future. Nelson-Hall Chicago

Johnston N.T. and Slaney P.A. 1996. Fish Habitat Assessment Procedures. Watershed Restoration Technical Circular No. 8 Ministry of Environment, Lands and Parks and Ministry of Forests. pp 97

Lane EW. 1955. Design of Stable Channels. Trans. A.S.C.E., Vol. 120

Lencastre A. 1987. Handbook of Hydraulic Engineering. Ellis Horwood Limited. 540 pp

Leopold L.B., Wolman M.G., and Miller J.P. 1964. Fluvial processes in geomorphology. Freeman, San Francisco, CA. 522pp

Martin V. 1996. Velocity Structure in Gravel Rivers. University of British Columbia Graduate Thesis. pp 29-35

Ministry of Environment, Lands, and Parks and Ministry of Forests (MELP and MF). 1999. Fish Passage-Culvert Inspection Procedures. 50pp

Ministry of Water, Land, and Air Protection (MWLAP) British Columbia. 2004. Standards and Best Practices for Instream Works. 174pp

Moyle P.B., Cech JJ. 2004. Fishes: An Introduction to Ichthyology 3rd Edition. Prentice Hall Inc. 590 pp

Murray and Associates. 2005. South Campus Neighbourhood Subdivision Plan. 1pp

Naturegrid. 2005. http://www.naturegrid.org.uk/rivers/watercyclepages/riverbasin-stages.html Accessed November 2005

Neill C.R. 1967. Mean Velocity Criterion for Scour of Course Uniform Bed Material. Proc., 12th Congress of IAHR, Fort Collins.

Newbury R., Gaboury M. Bates D. 1997. Restoring Habitats in Channelized or Uniform Streams Using Riffle and Pool Sequences. Fish Habitat Rehabilitation Procedures. Chapter 12 pp 1-22

Petrell R. 2005. Consultation through weekly meetings.
September 2005 to April 2006

Sherwood J.M. 1994. Estimation of peak-frequency relations, flood hydrographs, and volume – duration – frequency relations of ungaged small urban streams in Ohio. U.S. Geological Survey water – supply paper: 2431

Simon Fraser University (SFU). 2006. Watercourse and Stormwater Management Plan for Burnaby Mountain.
http://www.univercity.ca/bmcp/swmexecutivesummary.html
Accessed February 2006

Schwab G.O., Frevert R.K., Edminster T.W., Barnes K.K. 1981. Soil and Water Conservation Engineering 3rd Edition. John Wiley & Sons Inc. New York. 525 pp

University of British Columbia (UBC). 2004. South Campus Northeast Sub-Area Neighbourhood Plan. pp 53

U.S. Soil Conservation Service. 1974. Handbook of Channel Design for Soil and Water Conservation.

USDA Forest Service. 2005.
http://www.fs.fed.us/
Accessed November 2005

Ven Te C. 1981. Open-channel Hydraulics. McGraw-Hill. New York. 123 pp

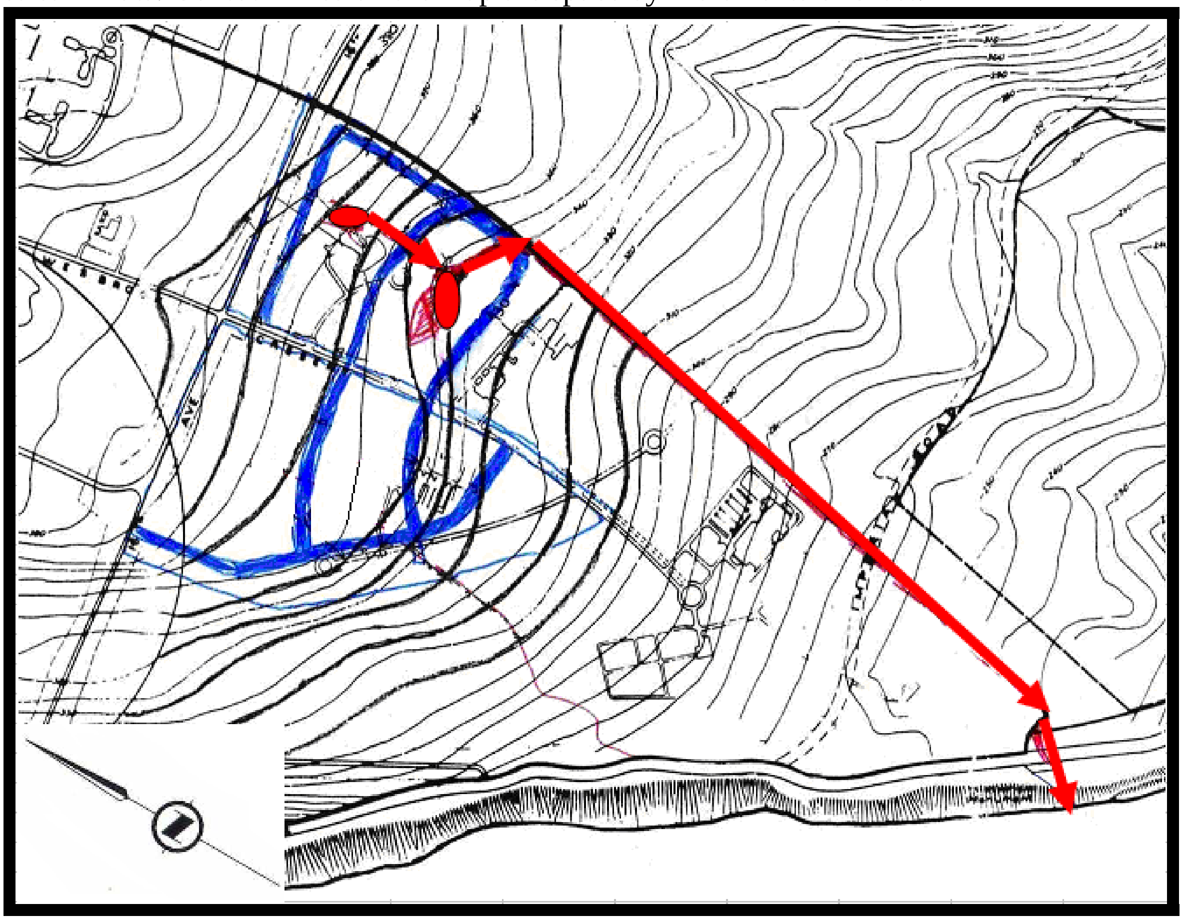
Yoo KH and Boyd CE. 1994. Hydrology and Water Supply for Pond Aquaculture. Chapman and Hall. New York. 483 pp

Ministry of Environment (MOE). 1991. Manual of Operational Hydrology in British Columbia Second Edition. Pg 149

APPENDICES

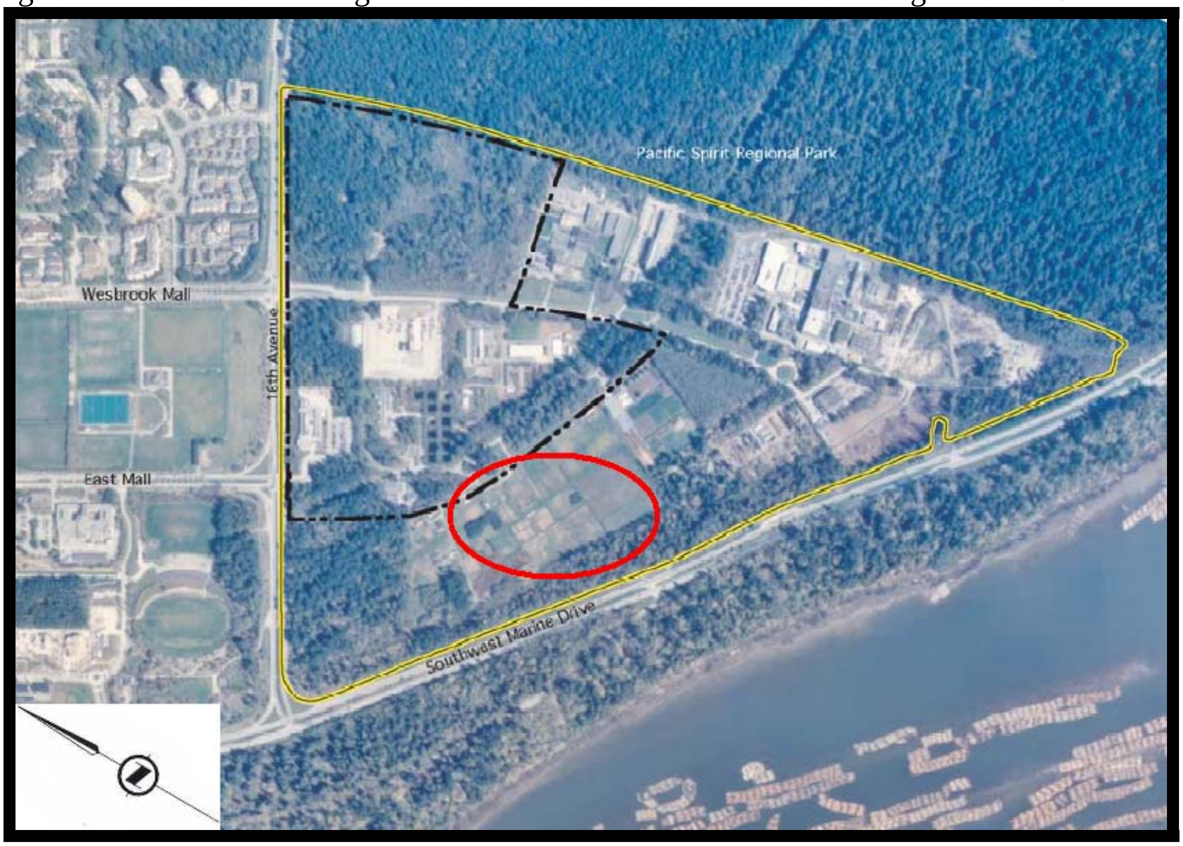
APPENDIX A: TOPOGRAPHY MAP OF SOUTH CAMPUS

The blue lines outline the roads which boarder and lie within the South Campus Neighbourhood plan. The contour lines of the map indicate that the land slopes in a southeasterly direction. This played a factor when deciding the ideal stream location. The red ovals indicate pond locations, while red arrows indicate the recommended path of stream flow. Contour lines on the map are space by increments of 5feet.



APPENDIX B: AERIAL PHOTOGRAPH OF SOUTH CAMPUS

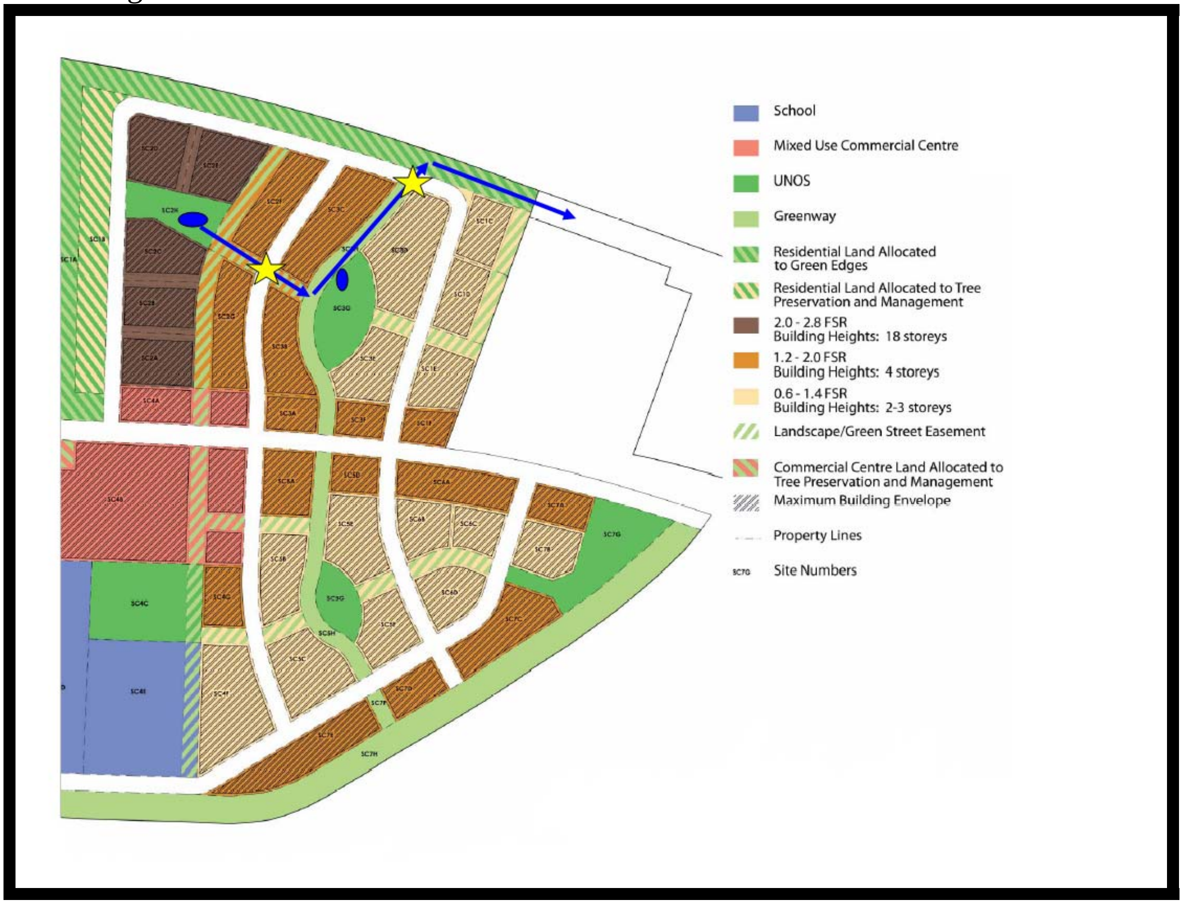
The yellow line outlines the south campus area of the University of British Columbia. The broken black line outlines the South Campus Neighbourhood. The red oval indicates the area to the west of the South Campus Neighbourhood. This area is currently used for agricultural research making it difficult to allow a stream to flow through this area.



(Alpin and Martin 2005)

APPENDIX C: SOUTH CAMPUS LAND USE PLAN

The land use plan for the South Campus Neighbourhood. The stream was limited to areas designated as greenways (including UNOS areas) and land allocated to tree preservation and management.



APPENDIX D: RATIONAL FORMULA RUNOFF COEFFICIENT, C TABLE

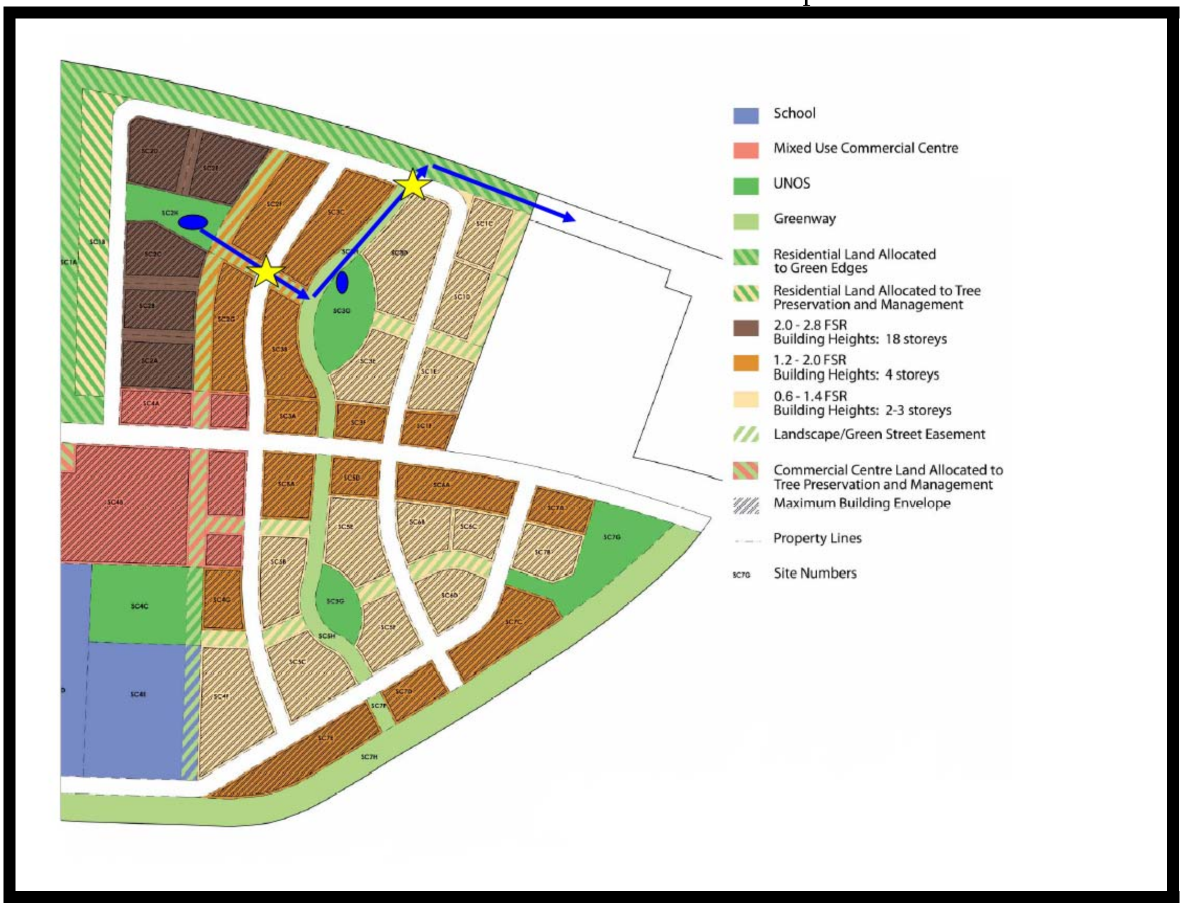
The terrain of south campus was considered moderate and the design rainfall intensity is a 15minute storm event, which occurs once every 10 years. Predevelopment is forested and will then have a runoff coefficient of 0.67. Post development is urbanized and will thus have a runoff coefficient of 0.85.

SURFACE COVER / PHYSIOGRAPHY	IMPER-MEABLE	FORESTED	AGRICUL-TURAL	RURAL	URBAN
mountain	1.00	0.90	-	-	-
steep slope	0.95	0.80	-	-	-
moderate slope	0.90	0.65	0.50	0.75	0.85
rolling terrain	0.85	0.50	0.40	0.65	0.80
flat	0.80	0.40	0.30	0.55	0.75
RI 10-25 years	+0.05	+0.02	+0.07	+0.05	+0.05
RI > 25 years	+0.10	+0.05	+0.15	+0.10	+0.10
Snowmelt	+0.10	+0.10	+0.10	+0.10	+0.10

(MOE 1991)

APPENDIX E: PONDS, STREAM, AND CULVERT PLACMENT

Pond 1 lies in the UNOS area SC2H and Pond 2 lies in UNOS area SC3G. Each pond is indicated by a blue oval. The yellow stars indicate locations where culverts are needed to allow the stream to cross roads. The arrows illustrate the flow path of the stream.



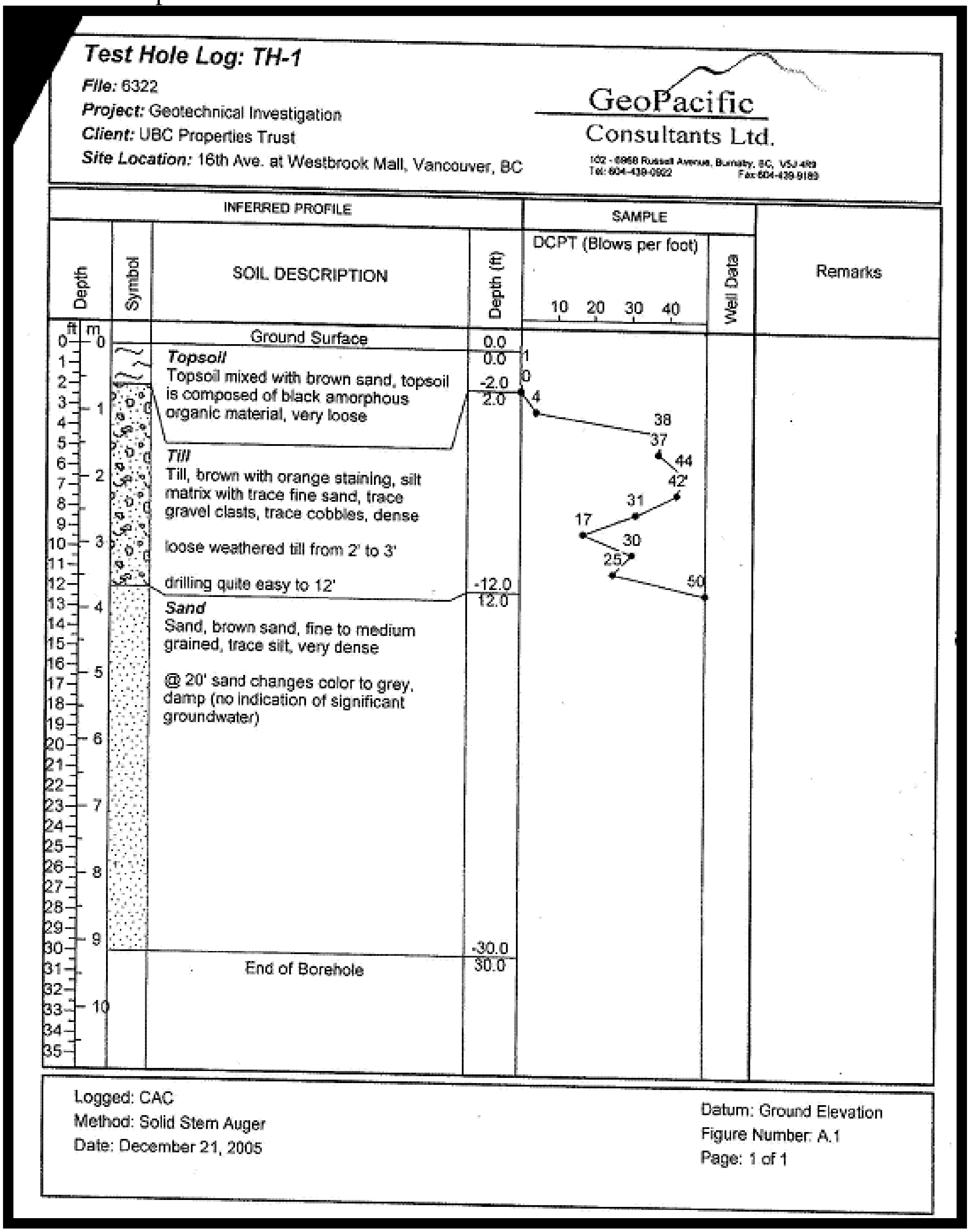
APPENDIX F: SOUTH CAMPUS NEIGHBOURHOOD SUBDIVISION PLAN

Pond 1 lies in the UNOS area SC2H and Pond 2 lies in UNOS area SC3G. These designated green spaces provide adequate area for Pond placement. UNOS area SC2H encompasses 4320.7m^2 of land, while UNOS area SC3G encompasses 6796.7m^2 of land. (Murray and Associates 2005)


PLAN SHOWING EASEMENTS OVER PORTIONS OF
DISTRICT LOT 6494
AND
DISTRICT LOT 4805
BOTH OF GROUP 1,
NEW WESTMINSTER DISTRICT
PT. DISTRICT LOT 4805 GROUP 1, N.W.D.
PT. DISTRICT LOT 4805
GROUP 1, N.W.D.
WESBROOK
16 AVENUE
SC4D
SCHOOL
SC4C
UNDS
SC4B
COMMERCIAL
SC4A
COMMERCIAL
SC1A
SC1B
SC2A
SC2B
SC2C
SC2D
SC2E
SC2F
SC2G
SC2H
UNDS
SC4F
SC4G
SC5A
SC5B
SC5C
SC5D
SC5E
SC5F
SC5G
UNDS
SC5H
GREENWAY
SC3A
SC3B
SC3C
SC3D
SC3E
SC3F
SC3G
UNDS
SC6A
SC6B
SC6C
SC6D
SC7A
SC7B
SC7C
SC7D
SC7E
SC7F
SC7G
UNDS
SC7H
SC1C
SC1D
SC1E
SC1F
REM. DISTRICT LOT 6494 GROUP 1, N.W.D.
DISTRICT LOT 140 GROUP 1, N.W.D.
BLOCK "I"

APPENDIX G: SAMPLE BOREHOLE RESULT

This is a typical borehole result done in the SC3D plot of the South Campus Neighbourhood. Till primarily composed of silt reaches depths of approximately 3.5m. Below this depth lies sand.



(GeoPacific Consultants Ltd 2005)

APPENDIX H: SAMPLE POND DIMENSION CALCULATIONS

The ponds will have the following dimensions:

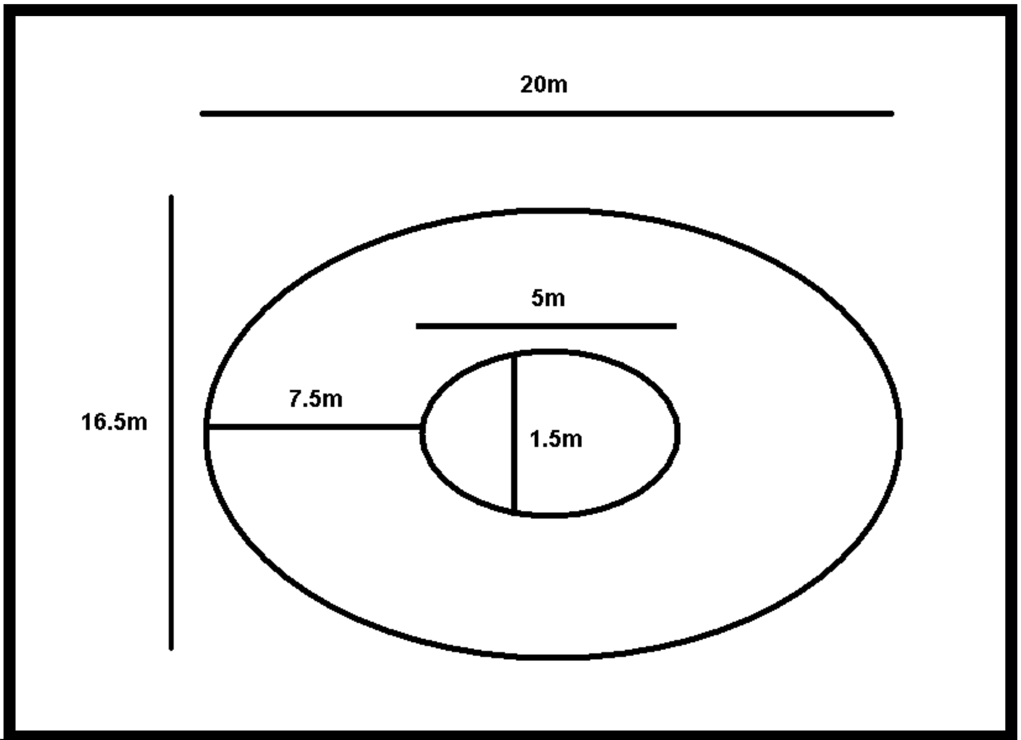


Figure 20. Pond dimensions. Aerial view of pond. The base of the ponds have a length of 5m and a width of 1.5m. The total length of the pond is 20m and the total width is 16.5m. The side slopes of the pond are 5:1. Since the sediment within the pond is primarily silt, the 5:1 side slopes will provide adequate stability.

Calculation 1: Surface area of the pond base and surface.

The surface area of the pond base and surface is oval in shape. The equation for the surface area of an oval is:

$$\text{Surface Area of Pond Surface} = \frac{L}{2} \times \frac{W}{2} \times \Pi$$

Where,

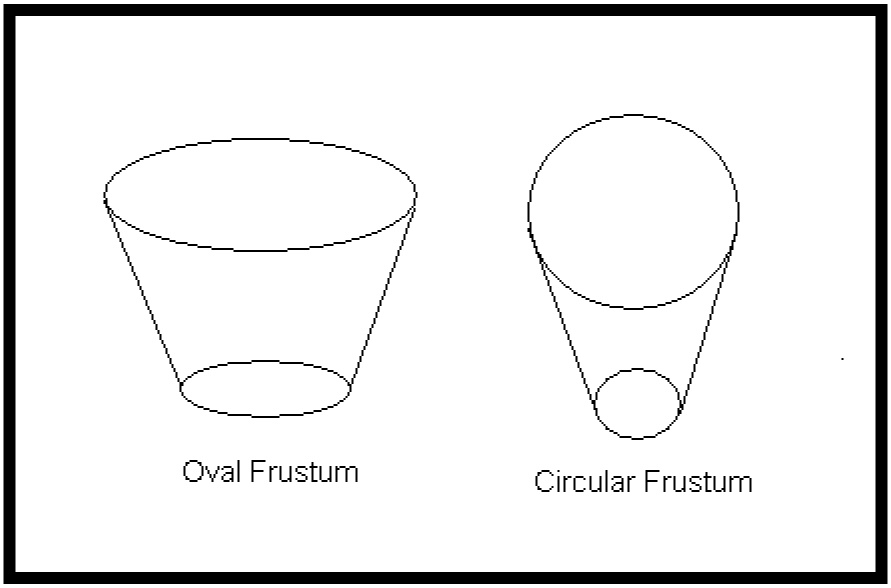
L, is the length of the pond surface or base
W, is the width of the pond surface or base
Π, is Pi

Thus,

$$\text{Surface Area of Pond Surface} = \frac{20\text{m}}{2} \times \frac{16.5m}{2} \times \Pi = 260m^2$$

$$\text{Surface Area of Pond Base} = \frac{5\text{m}}{2} \times \frac{1.5m}{2} \times \Pi = 5.9m^2$$

The ponds are essentially oval frustums. To simplify calculations, oval frustums can be looked at as horizontally compressed circular frustums. An oval and circular frustum are displayed below.



To determine pond volume and the wetted surface area, the oval frustum must be reformed into a circular frustum. This is done by determining the surface area of the top and bottom of the oval ponds.

To determine the corresponding circular frustum the following can be followed:

Determine the surface area of the upper and lower oval frustum. Place this area in the equation for the area of a circle.

$\text{Surface Area of Upper Frustum} = 260\text{m}^2$

$\text{Area Circle} = \Pi\text{r}^2 = 260m^2$

Solve for r,

R = 9.1m is the radius of the corresponding circle frustums upper circle.

r for the base of the pond can also be determined in a similar manner.

and will equal 1.37m

The volume of the circular frustum can then be determined.

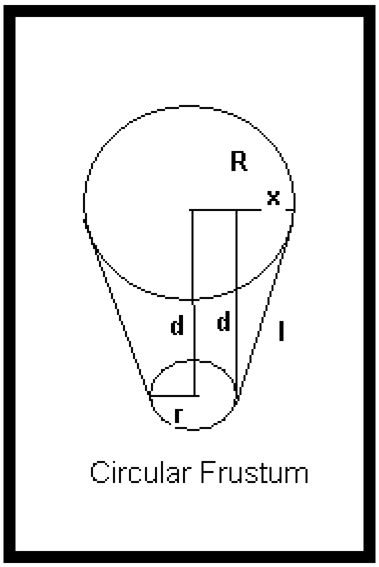
$$\text{Volume}_{\text{Circular Frustum}} = \frac{\Pi h}{3}\left(R^2 + Rr + r^2\right) = \frac{\Pi(1.5m)}{3}\left[(9.08m)^2 + (9.08m \times 1.37m) + (1.37m)^2\right] = 152m^3$$

The Area of the sides within a circular frustum is given by:

$$Area = \Pi(rl + Rl)$$

Since depth of the pond is known to be 1.5m,

l, can be determined.



Note: d is depth = 1.5m, r is the pond base radius, R is the upper pond circle radius, l is the length of the sides. X needs to be found to determine l. $x = R - r = 7.71m$

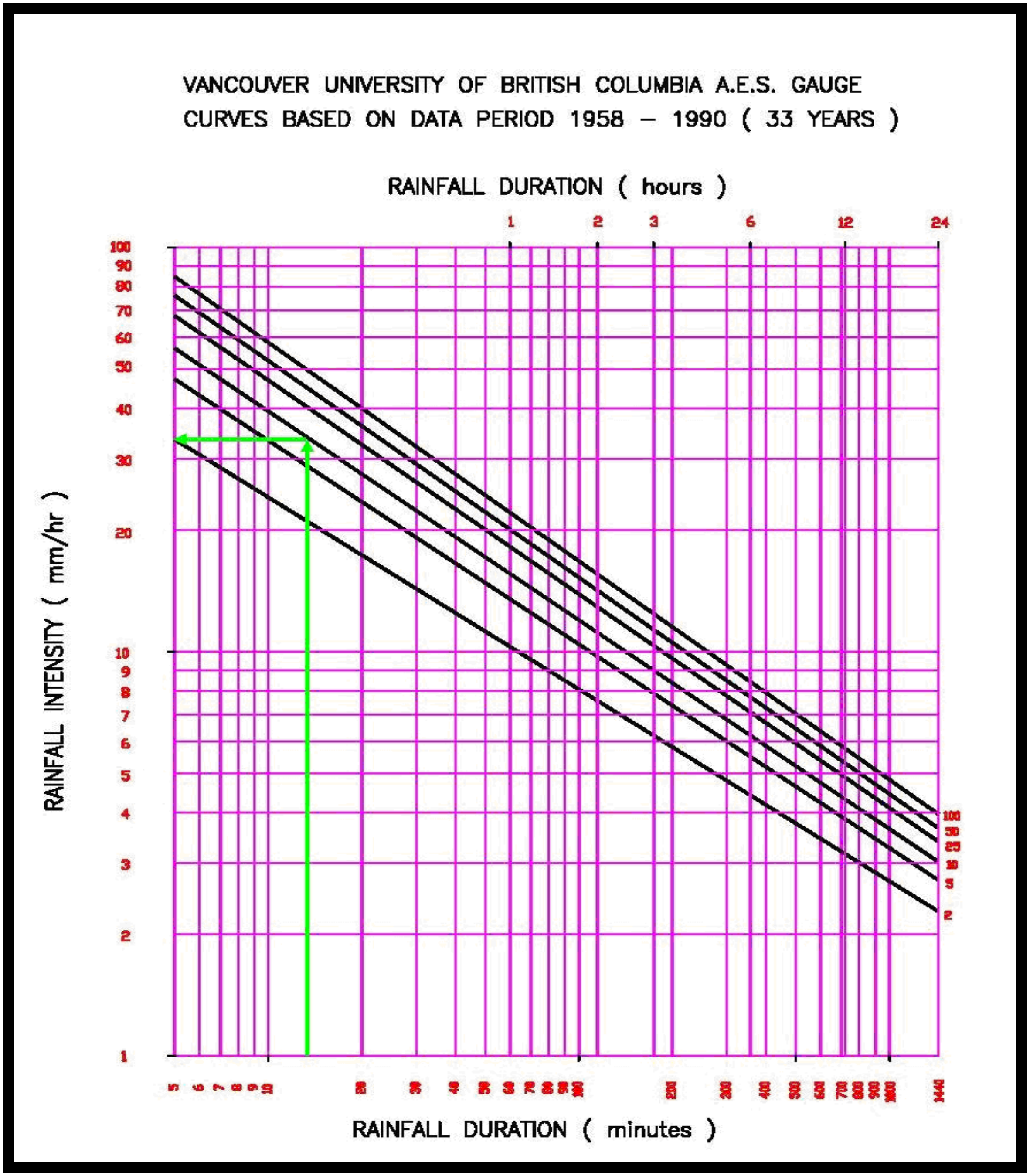
Thus, $l = \sqrt{(d^2 + x^2)} = \sqrt{(1.5m)^2 + (7.71m)^2} = 7.85m$

$$Area = \Pi(rl + Rl) = \Pi(1.37m \times 7.85m + 9.08m \times 7.85m) = 258m^2$$

Typically want to know the total surface area the rain touches, thus must add the surface area of the base to the surface area of the sides. Thus $258m^2 + 5.90m^2 = 264m^2$

APPENDIX I: UBC RAINFALL IDF CURVES

The British Columbia Building Code states that rooftop drains are designed to handle a storm event which last 15 minutes and occurs once every 10 years. This corresponds to a rainfall intensity of approximately 35mm/hr, which is the intensity the pipe, which drains into the pond, will be designed to handle. The green arrows on the graph below indicate this.



(British Columbia Ministry of Environment 2002)

APPENDIX J: TYPICAL VEGETATION FOUND NEAR PONDS

Irises, narrow leaf cattail, soft rush, and stem bulrush are commonly found along pond banks (Adams 1997). These plants are able to survive both submersed and not submersed in water, making them ideal for ponds that experience variable flow. They will also be able to withstand the variable costal climate of southern British Columbia.

Irises (*Iris versicolour)*

Narrow Leaf Cattail *(Typha angustifolia)*

Soft rush *(Juncus effuses)*

Soft Stem Bulrush *(Scirpus validus)*

APPENDIX K: RANDOMIZATION OF CASSCADE POOL & RIFFLE POOLS

The following table displays the results of the randomization trials. The results cut off when the section length is achieved. For example in section A will begin with a riffle 5.3m in length followed by a pool 6m in length.
The excel function used was: *=Int(RANDBETWEEN(60,85)/10)*Bankfull Width*

Section	A	B	C	D	E	F	G	H	I	J	K	L	M	N
Bankfull Width (m)	0.75	0.80	0.85	0.90	1.22	1.30	1.42	1.50	1.55	1.60	1.65	1.70	1.75	1.80
Section Length (m)	91	61	61	152	114	64	91	110	91	125	128	146	134	114
cascade/riffle to pool spacing	5.3	5.6	5.1	5.4	7.3	7.8	8.5	9.0	10.9	12.8	9.9	10.2	10.5	10.8
Note: 6-8 Time Bankfull Width	6.0	4.8	6.8	6.3	8.5	9.1	11.4	10.5	9.3	12.8	11.6	11.9	10.5	14.4
	4.5	4.8	6.0	6.3	9.8	7.8	9.9	12.0	10.9	12.8	11.6	10.2	12.3	14.4
	5.3	5.6	5.1	5.4	7.3	7.8	8.5	10.5	9.3	9.6	13.2	11.9	10.5	10.8
	6.0	4.8	6.0	5.4	8.5	9.1	8.5	9.0	12.4	9.6	9.9	13.6	12.3	10.8
	5.3	6.4	5.1	5.4	8.5	9.1	9.9	10.5	12.4	12.8	13.2	10.2	12.3	14.4
	4.5	4.8	5.1	5.4	8.5	7.8	9.9	10.5	9.3	12.8	9.9	10.2	10.5	10.8
	4.5	4.8	6.0	5.4	9.8		11.4	10.5	10.9	12.8	9.9	10.2	10.5	10.8
	4.5	5.6	6.0	7.2	9.8		11.4	9.0	10.9	11.2	9.9	13.6	14.0	10.8
	5.3	6.4	6.0	7.2	8.5		9.9	9.0		11.2	11.6	10.2	12.3	
	4.5	4.8	6.8	6.3	9.8			9.0		9.6	11.6	11.9	10.5	
	5.3	6.4	6.0	6.3	7.3					11.2		11.9		
	6.0			5.4	7.3							11.9		
	4.5			6.3	7.3									
	4.5			6.3										
	5.3			5.4										
	6.0			7.2										
	5.3			7.2										
				7.2										
				5.4										
				5.4										
				5.4										
				5.4										
				6.3										
				7.2										

APPENDIX L: RANDOMIZATION OF DEGREE OF MEANDERING

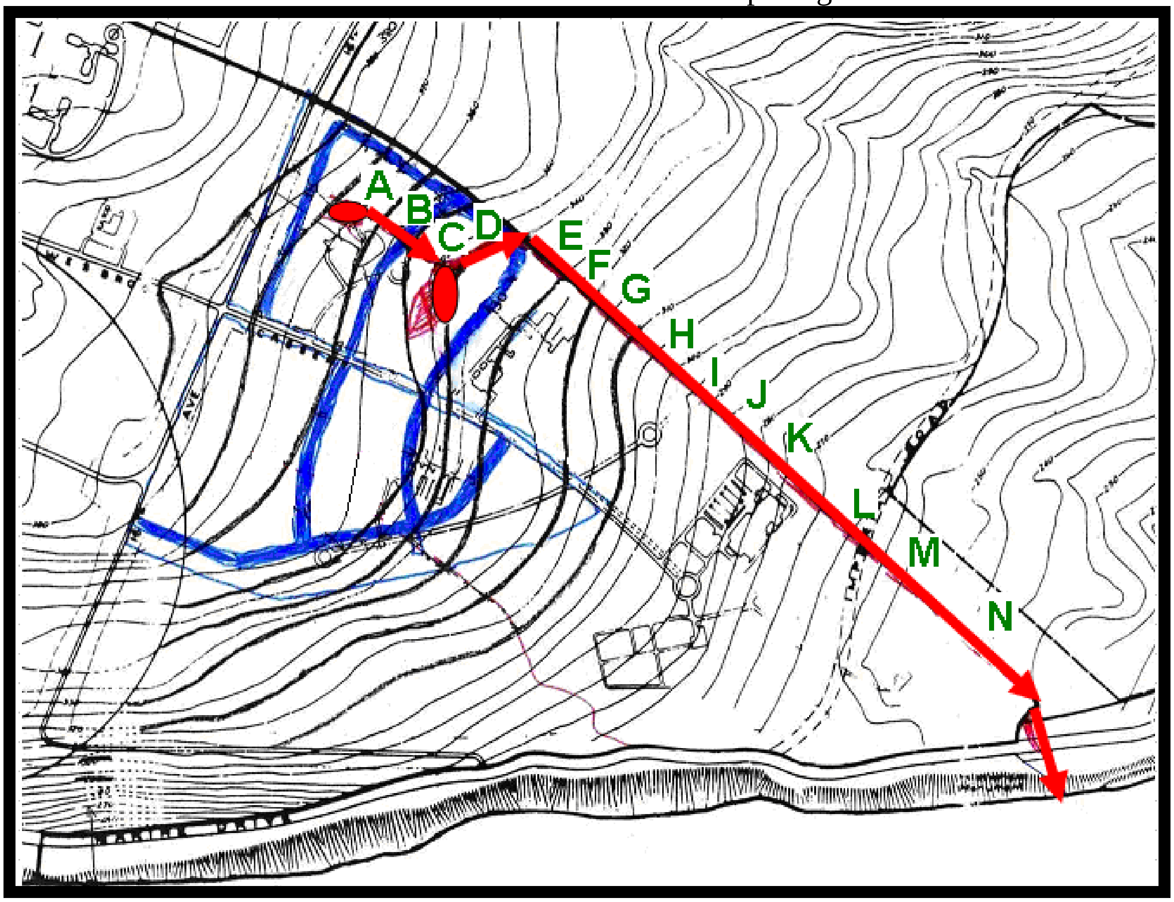
The values indicate the length of a stream in a given section before a meandering should occur. Typically cascades pool morphology streams meander at a smaller curvature of radius.

The excel function used was: *=Int(RANDBETWEEN(80,125)/10)*Bankfull Width*

Section	A	B	C	D	E	F	G	H	I	J	K	L	M	N
Bankfull Width (m)	0.75	0.80	0.85	0.90	1.22	1.30	1.42	1.50	1.55	1.60	1.65	1.70	1.75	1.80
Section Length (m)	91	61	61	152	114	64	91	110	91	125	128	146	134	114
Stream Meander Length	6.8	7.2	9.4	9.0	9.8	14.3	15.6	12.0	12.4	12.8	18.2	17.0	14.0	14.4
Note: 8-12 Time Bankfull Width	6.8	9.6	8.5	8.1	14.6	10.4	17.0	12.0	12.4	17.6	16.5	13.6	17.5	18.0
	6.0	7.2	6.8	9.0	9.8	13.0	14.2	18.0	17.1	19.2	16.5	15.3	21.0	18.0
	6.8	6.4	6.8	7.2	12.2	13.0	15.6	18.0	15.5	16.0	16.5	15.3	17.5	14.4
	7.5	8.8	7.7	9.0	14.6	14.3	17.0	16.5	15.5	19.2	13.2	15.3	17.5	21.6
	7.5	6.4	7.7	10.8	9.8	13.0	12.8	15.0	12.4	16.0	14.9	17.0	15.8	18.0
	7.5	8.0	9.4	9.0	12.2	13.0	14.2	15.0	15.5	17.6	14.9	13.6	15.8	14.4
	6.8	6.4	7.7	9.0	11.0	14.3	17.0	12.0	14.0	16.0	18.2	20.4	19.3	14.4
	8.3	9.6	9.4	10.8	11.0	10.4	15.6	15.0	12.4	14.4	16.5	20.4	19.3	18.0
	8.3	8.0	6.8	8.1	14.6	13.0	14.2	12.0	12.4	14.4	16.5	20.4	17.5	21.6
	6.8	8.0	10.2	7.2	14.6	10.4	11.4	18.0	17.1	19.2	14.9	17.0	21.0	19.8
	7.5	7.2	9.4	7.2	11.0	11.7	17.0	12.0	17.1	17.6	13.2	17.0	19.3	18.0
	7.5	6.4	9.4	9.9	13.4	13.0	12.8	15.0	17.1	19.2	16.5	13.6	17.5	16.2
	8.3	8.0	6.8	9.0	13.4	14.3	11.4	18.0	12.4	12.8	13.2	20.4	14.0	16.2
	9.0	8.8	6.8	9.0	14.6	14.3	12.8	15.0	12.4	19.2	13.2	20.4	19.3	19.8
	7.5	7.2	8.5	7.2	12.2	15.6	11.4	12.0	15.5	12.8	19.8	20.4	15.8	18.0
	9.0	8.8	7.7	9.0	14.6	11.7	11.4	13.5	18.6	14.4	18.2	17.0	21.0	16.2
	6.8	8.8	6.8	7.2	13.4	10.4	11.4	16.5	15.5	14.4	18.2	17.0	21.0	21.6
	6.0	7.2	6.8	8.1	9.8	15.6	15.6	16.5	14.0	16.0	19.8	18.7	21.0	19.8
	9.0	6.4	10.2	10.8	11.0	15.6	12.8	12.0	14.0	16.0	13.2	18.7	17.5	14.4
	6.8	8.0	7.7	8.1	13.4	11.7	11.4	15.0	12.4	16.0	16.5	20.4	15.8	19.8
	8.3	8.0	10.2	9.9	11.0	13.0	12.8	13.5	14.0	12.8	19.8	17.0	21.0	16.2
	8.3	6.4	6.8	7.2	12.2	14.3	11.4	13.5	14.0	16.0	16.5	13.6	21.0	16.2
	7.5	8.8	6.8	8.1	12.2	13.0	11.4	13.5	18.6	12.8	19.8	18.7	15.8	21.6
	6.8	6.4	6.8	9.0	13.4	10.4	15.6	18.0	12.4	16.0	14.9	17.0	17.5	21.6
	6.0	6.4	8.5	10.8	9.8	10.4	15.6	18.0	12.4	17.6	13.2	17.0	14.0	16.2
	8.3	8.8	8.5	9.0	13.4	14.3	12.8	16.5	14.0	17.6	16.5	17.0	17.5	16.2

APPENDIX M: TOPOGRAPHY MAP OF SOUTH CAMPUS AREA

The stream was divided into 14 sections based on 10feet spacing of contour lines.



APPENDIX N: SLOPE CALCULATIONS

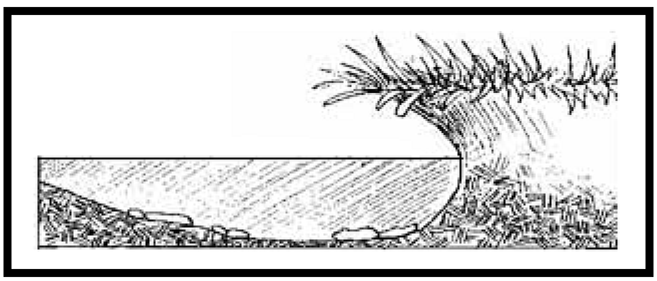
The channel slope was determined using a topography map of the south campus. The following formula was used to determine the slope of a given area:

$$\boxed{Slope = \frac{\text{Elevation Difference Between Contour Lines}}{\text{Distance Between Contour Lines}} \times 100\%}$$

For example the slope in section A will be given by,

$$\boxed{\text{Slope in Section A} = \frac{3\,\text{m}}{91\,\text{m}} \times 100\% = 3.3\%}$$

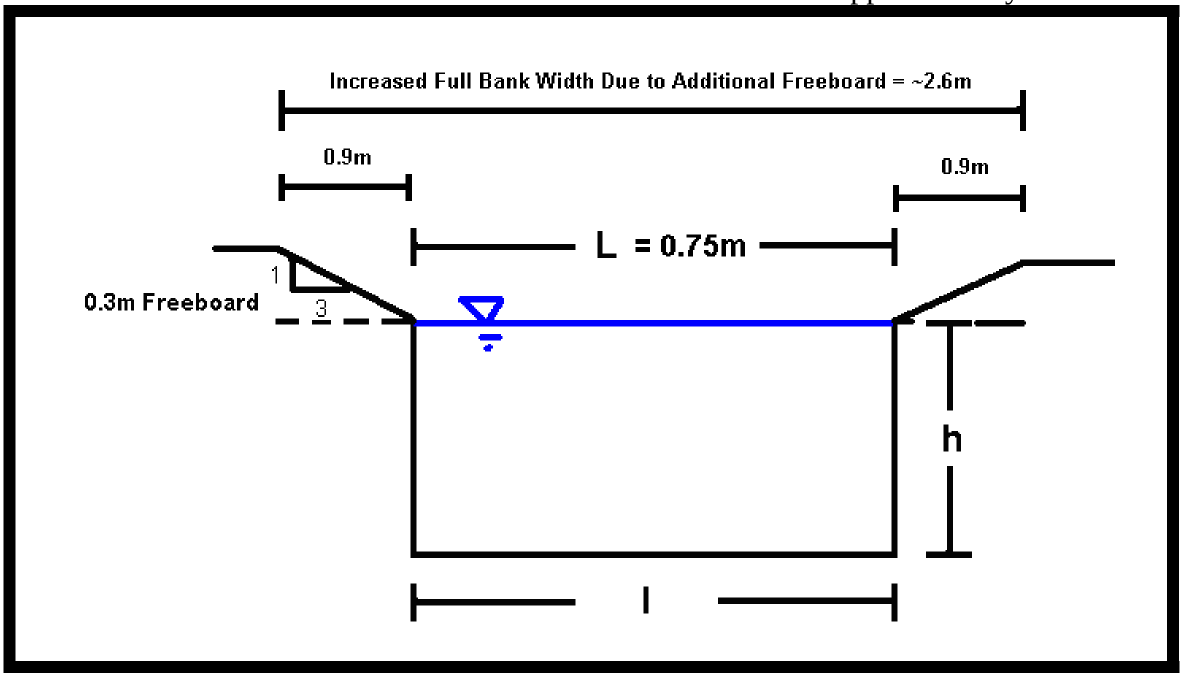
APPENDIX O: UNDERCUT STREAM BANK



Undercut stream bank, ideal for fish refuge. The undercut bank provides shade and cover for fish.

APPENDIX P: WIDTH OF STREAM WITH ADDITIONAL FREEBOARD

This illustration shows how the full bank width increases with the addition of freeboard. In this scenario, the addition of 0.3m freeboard adds 0.9m to the width of the stream on each side. This increases the total full bank width from 0.75m to approximately 2.6m.



APPENDIX Q: DEPTH TO WIDTH RATIO CALCULATION EXAMPLE

Given a width of 0.75m and a depth to width ratio of 1:5, the corresponding width is simply 0.75m/5 = 0.15m.

APPENDIX R: DETERMINATION OF MANNING'S COEFFICIENT USING METHOD OUTLINED BY Arcement and Schneider 2005.

The method outlined by Arcement and Schneider is an alteration of Cowan's (1956) procedure. The equation used to determine Manning's coefficient is:

$$n = (n_o + n_1 + n_2 + n_3 + n_4)\, m_5 \qquad \text{Equation 4}$$

Where n values account for,

n_o , basic straight, uniform, smooth channel

n_1 , corrects for surface irregularities

n_2 , channel cross section shape and size

n_3 , obstructions

n_4, vegetation and flow conditions

m_5 , meandering channel

A series of tables are given with varying values of n and m_5, depending on the channel characteristics. The following tables are taken directly from the article and illustrate what choices must be made to determine a proper Manning's coefficient. The chosen values are then entered into Equation 4 to attain a Manning's coefficient for the stream.

n_o , basic straight, uniform, smooth channel

Table 1. Base Values of Manning's *n*

		Base *n* Value	
Bed Material	**Median Size of bed material (in millimeters)**	**Straight Uniform Channel[1]**	**Smooth Channel[2]**
		Sand Channels	
Sand[3]	0.2 .3 .4 .5 .6 .8 1.0	0.012 .017 .020 .022 .023 .025 .026	-- -- -- -- -- -- --
		Stable Channels and Flood Plains	
Concrete	--	0.012-0.018	0.011
Rock Cut	--	--	.025
Firm Soil	--	0.025-0.032	.020
Coarse Sand	1-2	0.026-0.035	--
Fine Gravel	--	--	.024
Gravel	2-64	0.028-0.035	--
Coarse Gravel	--	--	.026
Cobble	64-256	0.030-0.050	--
Boulder	>256	0.040-0.070	--

[Modified from Aldridge & Garret, 1973, Table 1 --No data
1Benson & Dalrymple --No data
[2] For indicated material; Chow(1959)
[3] Only For Upper regime flow where grain roughness is predominant

n_1 , corrects for surface irregularities

Table 2 . Adjustment Values for Factors that Affect the Roughness of a Channel [modified from Aldridge and Garrett, 1973, Table 2]

Channel Conditions	*n* Value Adjustment[1]	Example
Degree of Irregularity (n_1)		
Smooth	0.000	Compares to the smoothest channel attainable in a given bed material.
Minor	0.001-0.005	Compares to carefully degraded channels in good condition but having slightly eroded or scoured side slopes.
Moderate	0.006-0.010	Compares to dredged channels having moderate to considerable bed roughness and moderately sloughed or eroded side slopes. s in rock.
Severe	0.011-0.020	Badly sloughed or scalloped banks of natural streams; badly eroded or sloughed sides of canals or drainage channels; unshaped, jagged, and irregular surfaces of channel

n_2 , channel cross section shape and size

Variation in channel cross section (n_2)

Channel Conditions	*n* Value Adjustment[1]	Example
Gradual	0.000	Size and shape of channel cross sections change gradually.
Alternating occasionally	0.001-0.005	Large and small cross sections alternate occasionally, or the main flow occasionally shifts from side to side owing to changes in cross-sectional shape.
Alternating frequently	0.010-0.015	Large and small cross sections alternate frequently, or the main flow frequently shifts from side to side owing to changes in cross-sectional shape.

n_3 , obstructions

Effect of obstruction (n_3)

Channel Conditions	*n* Value Adjustment[1]	Example
Negligible	0.000-0.004	A few scattered obstructions, which include debris deposits, stumps, exposed roots, logs, piers, or isolated boulders, that occupy less than 5 percent of the cross-sectional area.
Minor	0.005-0.015	Obstructions occupy less than 15 percent of the cross-sectional area, and the spacing between obstructions is such that the sphere of influence around one obstruction does not extend to the sphere of influence around another obstruction. Smaller adjustments are used for curved smooth-surfaced objects than are used for sharp-edged angular objects.
Appreciable	0.020-0.030	Obstructions occupy from 15 percent to 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause the effects of several obstructions to be additive, thereby blocking an equivalent part of a cross section.
Severe	0.040-0.050	Obstructions occupy more than 50 percent of the cross-sectional area, or the space between obstructions is small enough to cause turbulence across most of the cross section.

n_4, vegetation and flow conditions

Amount of vegetation (n_4)		
Channel Conditions	***n* Value Adjustment[1]**	**Example**
Small	0.002-0.010	Dense growths of flexible turf grass, such as Bermuda, or weeds growing where the average depth of flow is at least two times the height of the vegetation; supple tree seedlings such as willow, cottonwood, arrowhead, or saltcedar growing where the average depth of flow is at least three times the height of the vegetation.
Medium	0.010-0.025	Turf grass growing where the average depth of flow is from one to two times the height of the vegetation; moderately dense stemy grass, weeds, or tree seedlings growing where the average depth of flow is from two to three times the height of the vegetation; brushy, moderately dense vegetation, similar to 1-to-2-year-old willow trees in the dormant season, growing along the banks, and no significant vegetation is evident along the channel bottoms where the hydraulic radius exceeds 0.61 meters.
Large	0.025-0.050	Turf grass growing where the average depth of flow is about equal to the height of the vegetation; 8-to-10-years-old willow or cottonwood trees intergrown with some weeds and brush (none of the vegetation in foliage) where the hydraulic radius exceeds0.60 m; bushy willows about 1 year old intergrown with some weeds along side slopes (all vegetation in full foliage), and no significant vegetation exists along channel bottoms where the hydraulic radius is greater than 0.61 meters.
Very Large	0.050-0.100	Turf grass growing where the average depth of flow is less than half the height of the vegetation; bushy willow trees about 1 year old intergrown with weeds along side slopes C all vegetation in full foliage), or dense cattails growing along channel bottom; trees intergrow with weeds and brush (all vegetation in full foliage).

m_5 , meandering channel

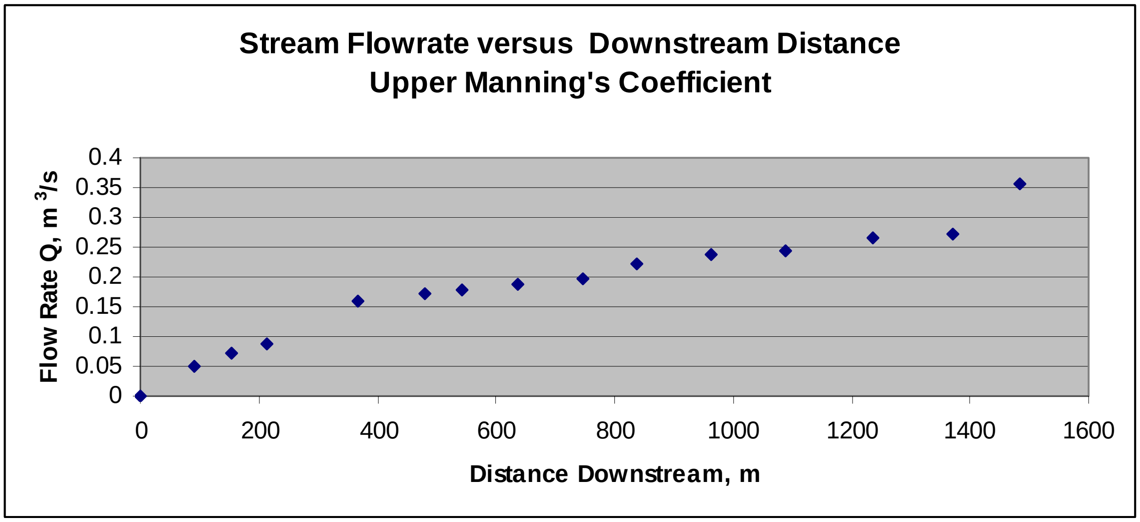
(Degree of Meandering m) [1 2] m		
Channel Conditions	***n* Value Adjustment[1]**	**Example**
Minor	1.00	Ratio of the channel length to valley length is 1.0 to 1.2.
Appreciable	1.15	Ratio of the channel length to valley length is 1.2 to 1.5.
Severe	1.30	Ratio of the channel length to valley length is greater than 1.5.

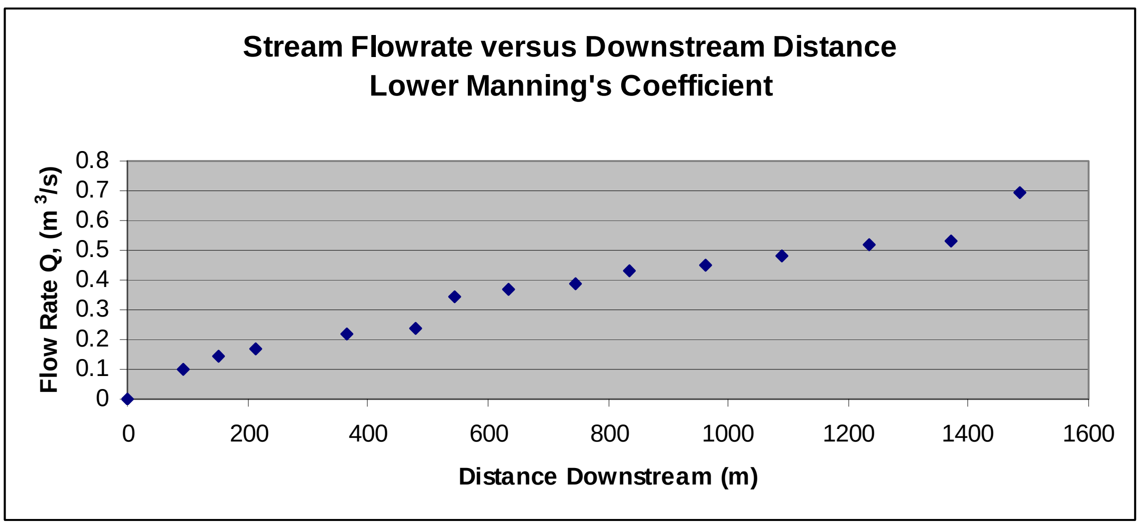
[1] Adjustments for degree of irregularity, variation in cross section, effect of obstructions, and vegetation are added to the base *n* value (Table 1) before multiplying by the adjustment for meander.

[2] Adjustment values apply to flow confined in channel and do not apply where downvalley flow crosses meanders.

APPENDIX S: STREAM FLOWRATE VERSUS DISTANCE DOWNSTREAM

These charts illustrate an increase in flowrate the further downstream one proceeds.





APPENDIX T: STREAM DIMENSIONING DATA

The following tables illustrate the tabulated and calculated parameters of each stream section. It was designed with the following design considerations in mind:

- Proper placement and spacing of cascade pool and riffle pool sequences.
- Randomization of cascade, riffle and pool spacing. This ensures a natural looking stream. Spacing typically varies from 6 to 8 times the bankfull width.
- Degree of meandering (typically 8 to 12 times the bankfull width) and radius of curvature.
- The bankfull width to cascade/riffle depth ratio should lie between 5 to 1 and 10 to 1 to maintain good habitat.
- In stream pools will hold a minimum volume of $1m^3$ (Hunter 1991). This will ensure adequate depth for fish refuge and ensure pond areas do not dry out easily during dry periods.
- Stream velocities will not exceed the critical swimming velocity of ~1m/s (Dane 1978).
- Stream velocities maintain cobble and gravel on the streambed. Cobble and gravel are characteristic streambed material found in natural cascade pool and riffle pool sequences (Hogan and Ward 1997). Smaller sediment, which is abrasive to fish, gills and eggs will be transported out of the stream system.
- The flow rate must increase from the upstream to downstream to allow the stream to downstream portions of the stream to be able to handle upstream portions.
- Sufficient bank side vegetation for food, cover, and slope stability.

Section	A	Units
Morphology	Cascade-pool	
Length	91.44	m
Slope	3.30%	
Average Velocity	0.87	m/s
Average Q	0.10	m^3/s
Area	0.11	m^2
R	0.11	m
n, low	0.04715	
Channel Width	0.75	m
Channel Height	0.15	m
Pool Dimensions		
Pool Width	0.75	m
Pool Height	0.45	m
Pool Volume	1.6	m^3

Section	A	Units
Morphology	Cascade-pool	
Length	91.44	m
Slope	3.30%	
Average Velocity	0.45	m/s
Average Q	0.05	m^3/s
Area	0.11	m^2
R	0.11	m
n, high	0.092	
Channel Width	0.75	m
Channel Height	0.15	m
Pool Dimensions		
Pool Width	0.75	m
Pool Height	0.45	m
Pool Volume	1.6	m^3

Section	B	Units
Morphology	Cascade-pool	
Length	60.96	m
Slope	5.00%	
Average Velocity	1.12	m/s
Average Q	0.14	m^3/s
Area	0.128	m^2
R	0.114	m
n, low	0.04715	
Channel Width	0.80	m
Channel Height	0.16	m
Pool Dimensions		
Pool Width	0.80	m
Pool Height	0.48	m
Pool Volume	1.8	m^3

Section	B	Units
Morphology	Cascade-pool	
Length	60.96	m
Slope	5.00%	
Average Velocity	0.57	m/s
Average Q	0.07	m^3/s
Area	0.13	m^2
R	0.11	m
n, high	0.092	
Channel Width	0.80	m
Channel Height	0.16	m
Pool Dimensions		
Pool Width	0.80	m
Pool Height	0.48	m
Pool Volume	1.8	m^3

Section	C	Units
Morphology	Cascade-pool	
Length	60.96	m
Slope	5.00%	
Average Velocity	1.16	m/s
Average Q	0.17	m^3/s
Area	0.14	m^2
R	0.12	m
n, low	0.04715	
Channel Width	0.85	m
Channel Height	0.17	m
Pool Dimensions		
Pool Width	0.85	m
Pool Height	0.51	m
Pool Volume	2.0	m^3

Section	C	Units
Morphology	Cascade-pool	
Length	60.96	m
Slope	5.00%	
Average Velocity	0.60	m/s
Average Q	0.09	m^3/s
Area	0.14	m^2
R	0.12	m
n, high	0.092	
Channel Width	0.85	m
Channel Height	0.17	m
Pool Dimensions		
Pool Width	0.85	m
Pool Height	0.51	m
Pool Volume	2.0	m^3

Section	D	Units
Morphology	Riffle-pool	
Length	152.4	m
Slope	2.00%	
Average Velocity	1.36	m/s
Average Q	0.22	m^3/s
Area	0.16	m^2
R	0.13	m
n, low	0.0414	
Channel Width	0.90	m
Channel Height	0.18	m
Pool Dimensions		
Pool Width	0.90	m
Pool Height	0.50	m
Pool Volume	2.8	m^3

Section	D	Units
Morphology	Riffle-pool	
Length	152.4	m
Slope	2.00%	
Average Velocity	0.98	m/s
Average Q	0.16	m^3/s
Area	0.16	m^2
R	0.13	m
n, high	0.07475	
Channel Width	0.90	m
Channel Height	0.18	m
Pool Dimensions		
Pool Width	0.90	m
Pool Height	0.22	m
Pool Volume	2.9	m^3

Section	E	Units
Morphology	Cascade-pool	
Length	114.3	m
Slope	2.70%	
Average Velocity	0.99	m/s
Average Q	0.24	m^3/s
Area	0.24	m^2
R	0.15	m
n, low	0.04715	
Channel Width	1.22	m
Channel Height	0.20	m
Pool Dimensions		
Pool Width	1.22	m
Pool Height	0.60	m
Pool Volume	3.4	m^3

Section	E	Units
Morphology	Cascade-pool	
Length	114.3	m
Slope	2.70%	
Average Velocity	0.56	m/s
Average Q	0.17	m^3/s
Area	0.305	m^2
R	0.177	m
n, high	0.092	
Channel Width	1.22	m
Channel Height	0.25	m
Pool Dimensions		
Pool Width	1.22	m
Pool Height	0.75	m
Pool Volume	4.3	m^3

Section	F	Units
Morphology	Cascade-pool	
Length	64.01	m
Slope	4.80%	
Average Velocity	1.33	m/s
Average Q	0.35	m^3/s
Area	0.26	m^2
R	0.15	m
n, low	0.04715	
Channel Width	1.30	m
Channel Height	0.20	m
Pool Dimensions		
Pool Width	1.30	m
Pool Height	0.60	m
Pool Volume	3.6	m^3

Section	F	Units
Morphology	Cascade-pool	
Length	64.01	m
Slope	4.80%	
Average Velocity	0.68	m/s
Average Q	0.18	m^3/s
Area	0.260	m^2
R	0.153	m
n, high	0.092	
Channel Width	1.3	m
Channel Height	0.2	m
Pool Dimensions		
Pool Width	1.3	m
Pool Height	0.60	m
Pool Volume	3.6	m^3

Section	G	Units
Morphology	Cascade-pool	
Length	91.44	m
Slope	3.30%	
Average Velocity	1.18	m/s
Average Q	0.37	m^3/s
Area	0.31	m^2
R	0.17	m
n, low	0.04715	
Channel Width	1.42	m
Channel Height	0.22	m
Pool Dimensions		
Pool Width	1.42	m
Pool Height	0.67	m
Pool Volume	4.4	m^3

Section	G	Units
Morphology	Cascade-pool	
Length	91.44	m
Slope	3.30%	
Average Velocity	0.60	m/s
Average Q	0.19	m^3/s
Area	0.312	m^2
R	0.168	m
n, high	0.092	
Channel Width	1.42	m
Channel Height	0.22	m
Pool Dimensions		
Pool Width	1.42	m
Pool Height	0.66	m
Pool Volume	4.4	m^3

Section	H	Units
Morphology	Cascade-pool	
Length	109.7	m
Slope	2.80%	
Average Velocity	1.12	m/s
Average Q	0.39	m^3/s
Area	0.35	m^2
R	0.18	m
n, low	0.04715	
Channel Width	1.50	m
Channel Height	0.23	m
Pool Dimensions		
Pool Width	1.50	m
Pool Height	0.70	m
Pool Volume	4.9	m^3

Section	H	Units
Morphology	Cascade-pool	
Length	109.7	m
Slope	2.80%	
Average Velocity	0.57	m/s
Average Q	0.20	m^3/s
Area	0.345	m^2
R	0.176	m
n, high	0.092	
Channel Width	1.5	m
Channel Height	0.23	m
Pool Dimensions		
Pool Width	1.5	m
Pool Height	0.69	m
Pool Volume	4.8	m^3

Section	I	Units
Morphology	Cascade-pool	
Length	91.4	m
Slope	3.30%	
Average Velocity	1.22	m/s
Average Q	0.43	m^3/s
Area	0.36	m^2
R	0.18	m
n, low	0.04715	
Channel Width	1.55	m
Channel Height	0.23	m
Pool Dimensions		
Pool Width	1.55	m
Pool Height	0.69	m
Pool Volume	4.8	m^3

Section	I	Units
Morphology	Cascade-pool	
Length	91.4	m
Slope	3.30%	
Average Velocity	0.62	m/s
Average Q	0.22	m^3/s
Area	0.357	m^2
R	0.177	m
n, high	0.092	
Channel Width	1.55	m
Channel Height	0.23	m
Pool Dimensions		
Pool Width	1.55	m
Pool Height	0.69	m
Pool Volume	5.0	m^3

Section	J	Units
Morphology	Cascade-pool	
Length	125	m
Slope	2.40%	
Average Velocity	1.10	m/s
Average Q	0.45	m^3/s
Area	0.41	m^2
R	0.19	m
n, low	0.04715	
Channel Width	1.60	m
Channel Height	0.26	m
Pool Dimensions		
Pool Width	1.60	m
Pool Height	0.77	m
Pool Volume	5.7	m^3

Section	J	Units
Morphology	Cascade-pool	
Length	125	m
Slope	2.40%	
Average Velocity	0.57	m/s
Average Q	0.24	m^3/s
Area	0.416	m^2
R	0.196	m
n, high	0.092	
Channel Width	1.6	m
Channel Height	0.26	m
Pool Dimensions		
Pool Width	1.6	m
Pool Height	0.78	m
Pool Volume	5.8	m^3

Section	k	Units
Morphology	Cascade-pool	
Length	128	m
Slope	2.40%	
Average Velocity	1.12	m/s
Average Q	0.48	m^3/s
Area	0.43	m^2
R	0.20	m
n, low	0.04715	
Channel Width	1.65	m
Channel Height	0.26	m
Pool Dimensions		
Pool Width	1.65	m
Pool Height	0.78	m
Pool Volume	6.0	m^3

Section	k	Units
Morphology	Cascade-pool	
Length	128	m
Slope	2.40%	
Average Velocity	0.57	m/s
Average Q	0.25	m^3/s
Area	0.429	m^2
R	0.198	m
n, high	0.092	
Channel Width	1.65	m
Channel Height	0.26	m
Pool Dimensions		
Pool Width	1.65	m
Pool Height	0.78	m
Pool Volume	6.0	m^3

Section	L	Units
Morphology	Cascade-pool	
Length	146.3	m
Slope	2.10%	
Average Velocity	1.09	m/s
Average Q	0.52	m^3/s
Area	0.48	m^2
R	0.21	m
n, low	0.04715	
Channel Width	1.70	m
Channel Height	0.28	m
Pool Dimensions		
Pool Width	1.70	m
Pool Height	0.84	m
Pool Volume	6.7	m^3

Section	L	Units
Morphology	Cascade-pool	
Length	146.3	m
Slope	2.10%	
Average Velocity	0.56	m/s
Average Q	0.27	m^3/s
Area	0.476	m^2
R	0.211	m
n, high	0.092	
Channel Width	1.7	m
Channel Height	0.28	m
Pool Dimensions		
Pool Width	1.7	m
Pool Height	0.84	m
Pool Volume	6.7	m^3

Section	M	Units
Morphology	Cascade-pool	
Length	134.1	m
Slope	2.30%	
Average Velocity	1.12	m/s
Average Q	0.53	m^3/s
Area	0.47	m^2
R	0.21	m
n, low	0.04715	
Channel Width	1.75	m
Channel Height	0.27	m
Pool Dimensions		
Pool Width	1.75	m
Pool Height	0.81	m
Pool Volume	6.6	m^3

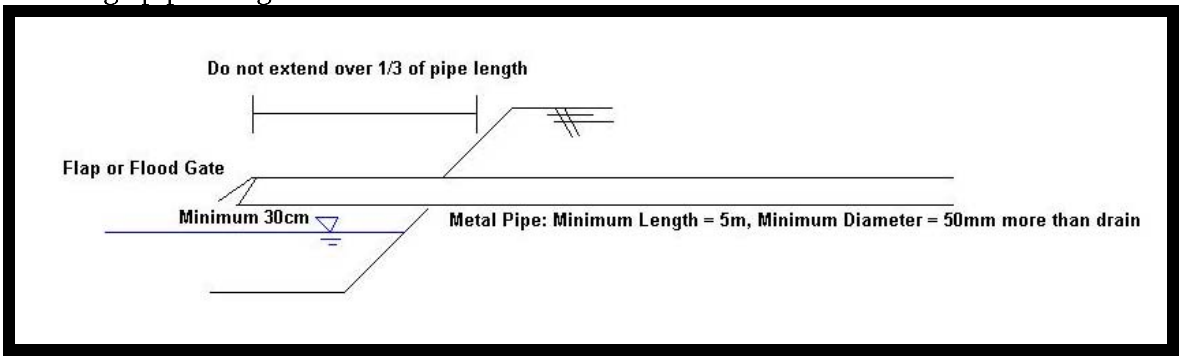
Section	M	Units
Morphology	Cascade-pool	
Length	134.1	m
Slope	2.30%	
Average Velocity	0.58	m/s
Average Q	0.27	m^3/s
Area	0.473	m^2
R	0.206	m
n, high	0.092	
Channel Width	1.75	m
Channel Height	0.27	m
Pool Dimensions		
Pool Width	1.75	m
Pool Height	0.81	m
Pool Volume	6.6	m^3

Section	N	Units
Morphology	Cascade-pool	
Length	114.3	m
Slope	2.70%	
Average Velocity	1.29	m/s
Average Q	0.70	m^3/s
Area	0.54	m^2
R	0.23	m
n, low	0.04715	
Channel Width	1.80	m
Channel Height	0.30	m
Pool Dimensions		
Pool Width	1.80	m
Pool Height	0.90	m
Pool Volume	7.6	m^3

Section	N	Units
Morphology	Cascade-pool	
Length	114.3	m
Slope	2.70%	
Average Velocity	0.66	m/s
Average Q	0.36	m^3/s
Area	0.540	m^2
R	0.225	m
n, high	0.092	
Channel Width	1.80	m
Channel Height	0.30	m
Pool Dimensions		
Pool Width	1.8	m
Pool Height	0.90	m
Pool Volume	7.6	m^3

APPENDIX U: PIPE DISCHARGING INTO POND CROSS SECTION

Discharge pipe design and recommendations.



APPENDIX V: GUIDLINE CONFORMATION CALCULATIONS

To determine if post development runoff volumes roughly equal pre-development runoff volumes the following approach will be used:

$$Q_{PostDevelopmentRunofff} - Q_{StreamOutSC} \approx Q_{PredevelopmentRunoff}$$

Where,

$Q_{PostDevelopmentRunoff}$, is the flowrate of runoff after the construction of the South Campus Neighbourhood takes place in m^3/s.

$Q_{StreamOutSC}$, is the flowrate of the stream leaving the South Campus Neighbourhood in m^3/s.

$Q_{PredevelopmentRunoff}$, is the flowrate of runoff before the construction of the South Campus Neighbourhood takes place in m^3/s.

It is important to note that any flow which discharges through the stream is not considered runoff since the stream is considered a natural system (MWLAP 2004). This is why $Q_{StreamOutSC}$ removes the total runoff created by the post development runoff.

The rational formula assumes that the ground is saturated and can then be used to determine runoff volume. The system is designed for the maximum 15mintute storm that occurs once every 10 years ($9.72 \times 10^{-6} m/s$). Thus, multiplying the post development, stream, and predevelopment flowrates by 15 minutes (900 seconds) will give the corresponding volumes.

$$Q_{PostDevelopmentRunofff} \times 900s - Q_{StreamOutSC} \times 900s \approx Q_{PredevelopmentRunoff} \times 900s$$

$$c_{PostDevelopment} \times i \times A_{SouthCampusNeighbourhodd} \times 900s - Q_{StreamOutSC} \times 900s \approx c_{Predevelopment} i A_{SouthCampusNeighbourhood} \times 900s$$

$$(0.85 \times (9.72 \times 10^{-6} m/s) \times 301000 m^2 \times 900s) - (0.17 m^3/s \times 900s) \approx (0.67 \times (9.72 \times 10^{-6} m/s) \times 301000 m^2 \times 900s)$$

Thus,

$$V_{PostDevelopmentRunoffwithStream} \approx V_{PredevelopmentRunoff}$$

$$2152 m^3 = 1764 m^3$$

%Difference= $\left| \dfrac{V_{PostDevelopmentRunoffwithStream} - V_{PredevelopmentRunoff}}{V_{PostDevelopmentRunoffwithStream}} \right| \times 100\% = \left| \dfrac{2152 m^3 - 1764 m^3}{2152 m^3} \right| = 18\%$

If the stream is not included then,

$$Q_{PostDevelopmentRunofff} \times 900s \approx Q_{\Pr edevelopmentRunoff} \times 900s$$

$$c_{PostDevelopment} \times i \times A_{SouthCampusNeighbourhodd} \times 900s \approx c_{\Pr edevelopment} iA_{SouthCampusNeighbourhood} \times 900s$$

$$(0.85 \times (9.72 \times 10^{-6} m/s) \times 301000 m^2 \times 900s) \approx (0.67 \times (9.72 \times 10^{-6} m/s) \times 301000 m^2 \times 900s)$$

Thus,

$$V_{PostDevelopmentRunoffWithOutStream} \approx V_{\Pr edevelopmentRunoff}$$

$$2238m^3 \neq 1764m^3$$

$$\%\text{ Difference} = \left| \frac{V_{PostDevelopmentRunoffwithStream} - V_{\Pr edevelopmentRunoff}}{V_{PostDevelopmentRunoffwithStream}} \right| \times 100\% = \left| \frac{2238m^3 - 1764m^3}{2238m^3} \right| \times 100\% = 21\%$$

Percent runoff improvement with the addition of the stream system is then 21%-18%, which equals **3% improvement in runoff.**

The post development volume runoff with the stream system differs from the predevelopment runoff volume by 18%.

Note: $Q_{streamout}$ is taken at the upstream portion of Section E, since this is the best representative flowrate out of the South Campus Neighbourhood. 0.17m^3/s is the flowrate of Section E at the high Manning's coefficient.

Using the low Manning's coefficient, $Q_{streamout}$ is 0.24m^3/s. Applying the same method above at this flowrate:

$$V_{PostDevelopmentRunoffwithStream} \approx V_{\Pr edevelopmentRunoff}$$

$$2022m^3 \neq 1764m^3$$

% Difference = 12.8%

Thus, 21%-12.8% = **8.2% runoff improvement**

Average % Difference = (18%+12.8%)/2=15.3%

Average Improvement = (3%+8.2%)/2 = 6%

Average Reduced Runoff Volume = %Improvement

$$\text{Average Reduced Runoff Volume} = \%\text{Improvement} \times \mathrm{V}_{\text{PostDevelopmentNoStream}} = 0.06 \times 2238m^3 = 134m^3$$

To determine if the total runoff volume, with the addition of the stream and pond system, will be less than or equal to 10% of the total rainfall volume, the approach will be used:

$$\text{Total Runoff Volume} = \text{South Campus Runoff Volume - Stream Volume}$$

Or

$$\mathrm{V}_{\text{TotalRunoffVolume}} = \mathrm{V}_{\text{SouthCampusRunoffVolume}} - \mathrm{V}_{\text{Stream}}$$

$$\mathrm{V}_{\text{TotalRunoffVolume}} = (c \times i \times A \times 900s) - (\text{Velocity at Lower or Higher Manning's Coefficient in Section E} \times 900s)$$

At Low Manning’s coefficient,

$$\mathrm{V}_{\text{TotalRunoffVolume}} = (0.85 \times (9.72 \times 10^{-6} m/s) \times 301000m^2 \times 900s) - (0.24m^3/s \times 900s)$$

$$\mathrm{V}_{\text{TotalRunoffVolume}} = 2022\mathrm{m}^3$$

At High Manning’s coefficient,

$$\mathrm{V}_{\text{TotalRunoffVolume}} = (0.85 \times (9.72 \times 10^{-6} m/s) \times 301000m^2 \times 900s) - (0.17m^2/s \times 900s)$$

$$\mathrm{V}_{\text{TotalRunoffVolume}} = 2085\mathrm{m}^3$$

Also,

$$\text{Total Rainfall Volume} = \text{ciA} \times 900\text{s}$$

Where,

c, runoff coefficient will equal 1 since want total rainfall volume.

i, is the design rainfall intensity of $9.72 \times 10^{-6}\, m/s$.

A, is the area of the South Campus Neighbourhood which is 301 000m^2.

$$\text{Total Rainfall Volume} = (9.72 \times 10^{-6}\,\text{m/s}) \times 301\,000\text{m}^2 \times 900\text{s}$$

$$\text{Total Rainfall Volume} = 2633\text{m}^3$$

$$10\%\ \text{of Total Rainfall Volume} = 0.10 \times 2238\text{m}^3 \approx 263m^3$$

At Low Manning's Coefficient:

$$V_{\text{TotalRunoffVolume}} = 2022\text{m}^3 \;>>\; 10\%\ \text{of Total Rainfall Volume} \approx 263m^3$$

Thus, guideline could not be met.

At High Manning's Coefficient:

$$V_{\text{TotalRunoffVolume}} = 2085\text{m}^3 >> 10\%\ \text{of Total Rainfall Volume} \approx 263m^3$$

Thus, guideline could not be met.

Actual Percent of Rainfall Volume:

At Low Manning's Coefficient:

$$\text{Actual}\,\%\ \text{of Rainfall Volume} = \frac{V_{\text{TotalRunoffVolume}}}{V_{\text{TotalRainfallVolume}}} \times 100\% = \frac{2022m^3}{2633m^3} \times 100\% \approx 77\%$$

At High Manning's Coefficient:

$$\text{Actual}\,\%\ \text{of Rainfall Volume} = \frac{V_{\text{TotalRunoffVolume}}}{V_{\text{TotalRainfallVolume}}} \times 100\% = \frac{2085m^3}{2633m^3} \times 100\% \approx 79\%$$

Average % of Rainfall Volume = (77%+79%)/2 = 78%

Note:

$$V_{TotalRunoffVolume} \leq V_{TotalRainFall} \times 10\%$$

Then the guideline is achieved.

If,

$$V_{TotalRunoffVolume} > V_{TotalRainFall} \times 10\%$$

Then the guideline is not achieved.

APPENDIX W: POND ROOFTOP DATA AND CALCULATIONS

Governing Pond Q Formula		
Q = Qpond + Qrooftops		
Q1	0.170	m^3/s
Q2	0.080	m^3/s

POND 1			
Qpond = A_{pond}*c*(i_{Rain}-$i_{SoilPermeability}$)			
Apond	460.0	m^2	
c	1		
i	7.17E-06	m/s	
Qpond	0.003	m^3/s	

POND 2		
Qpond = A_{pond}*c*(i_{Rain}-$i_{SoilPermeability}$)		
Apond	460.0	m^2
c	1	
i	7.17E-06	m/s
Qpond	0.003	m^3/s

Rooftop Area 1		
Qrooftop = ciA		
Arooftop	17150	m^2
c	1	
i	9.72E-06	m/s
Qrooftop	0.167	m^3/s

Rooftop Area 2		
Qrooftop = ciA		
Arooftop	7891	m^2
c	1	
i	9.72E-06	m/s
Qrooftop	0.077	m^3/s

Note: The flowrate into the Pond 1 spillway had to be less than the upstream portion of the stream, while the flowrate into the Pond 2 spillway had to be less than the flowrate in Section D of the stream. The rooftop areas were then determined using the solver function in excel and setting Q1 and Q2 lower than the section of stream they spill into.

APPENDIX X: TIME IT TAKES FOR POND FILLING

The red line is the volume of the pond. Rainfall intensity is the average rainfall that occurs over the month of November, 0.017m/day.