ANALYSIS OF POND SEEPAGE FOR STREAM FLOW AUGMENTATION

by

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Abstract

Many coastal streams in British Columbia have problems with low flow, which can have serious impacts on aquatic life. This research was focused on examining by experimentation and hydraulic modelling, the use of pond seepage as a novel method for stream flow augmentation.

In the late summer dry period, four experiments were conducted to study the direction and magnitude of pond seepage flow under different conditions. An experimental pond (≈ 9 m^3) was excavated to the hardpan in undisturbed soil. A ditch for capturing the seepage nearly surrounded the pond at a distance of approximately 2.4 m. The ditch was effective in collecting the seepage from both the pond bottom and the banks. Water was lost more slowly in the lined pond experiment than in the unlined experiment, and the liner also affected the direction of the seepage. From the collected data, it was determined that a combination of two hydraulic models, a wetland seepage model and an embankment seepage model, could be used to predict the experimental results.

The good overall agreement between the experimental and theoretical results provided the rationale for using such models to design ponds to maintain low summer time flows in small, undisturbed streams, given that the site is similar to the experimental conditions regarding water table height. Coastal areas are the best location for using ponds as the climate conditions in these areas helps to reduce and compensate for evaporation losses.

While significantly improved stream flow over a long term can be achieved with the use of multiple ponds, there are also two important benefits that can be realized from smaller projects:

1. to allow unrestricted fish movement over a short time period (less than one month)
2. to improve the quality of habitat available to fish over the summer.

Even small increases in stream flow can improve the quality of habitat, especially in slower moving pooled areas, as well as allow the fish to migrate to other parts of the stream where better habitat might be found. Both of these benefits could be especially important to streams that current flow augmentation methods cannot help.

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1.0 Introduction

This section provides an introduction to the thesis topic along with the research objectives of this work. The organization of the remaining sections of the thesis is also explained for the reader's benefit.

1.1 Background

Watersheds in the coastal areas of British Columbia have been subjected to intense urban and rural development pressures for many decades. For example, in the Lower Fraser Valley, the great majority of pre-settlement streams have been buried, culverted and many are effectively lost, while most of the remaining streams have been altered in one or more ways, including diversion, the removal or alteration of riparian vegetation and by pollution (Lower Fraser Valley Stream Review, 1997. Many streams also have problems with low flow. The hydrology of streams in coastal areas is strongly influenced by local precipitation (Lower Fraser Valley Stream Review, 1997). These tributaries experience peak flows during the fall and winter months when precipitation is greatest, while low flows occur during the summer months when there is little precipitation and groundwater reserves are insufficient to maintain adequate base flows. Out of forty salmon bearing streams in the Fraser Delta Habitat Management Area with mean annual flows ranging from 30 to 3.0×10^4 L/s, thirty-one have problems with low flow and, in addition, nineteen also have water demand issues (Nener and Wernick, 1997). Two reasons for low flow conditions are insufficient groundwater recharge and destruction or modification of natural wetlands and ponds. As well, artificial ponds, such as water reservoirs, are deliberately constructed to limit seepage and exit flows. The causes and impacts of low flow are discussed in more detail in the literature review.

Dams at the headwater or within a stream and the construction of side channels are current methods used to augment stream flow. Both methods can be costly, cause other habitat issues, and may not be practical. Another possible method, which has so far not been thoroughly investigated, involves the management of the direction and rate of seepage from a pond. Ponds occur naturally in the areas surrounding streams and similar natural-like storage structures may be introduced to streams where the natural ponds have been removed, or existing ponds modified to increase their storage potential. The ponds would collect rainfall and surface runoff during the winter and spring, and then release the water to the stream over the summer. High stream flows could also be used to help fill the pond.

For a pond to be useful for flow augmentation, it is necessary that evaporation be kept as low as possible; the volume of water stored is adequate to last over the dry summer period, and seepage is controlled and directed into the stream at a sufficient rate. Various equations exist that can be used to model pond seepage and these are covered in more detail in the literature review. Some of the constraints of using pond seepage are the direction of the subsurface flow; soil conductivity; water table depth; stream substrate; available surface area, and pond/stream separation distance.

The concept of this thesis was to examine pond seepage, through experimentation and hydraulic modelling, to see if it could be used to achieve the goal of augmenting low stream flow over the dry summer period.

1.2 Objectives

The objectives of this research were to:

1. under varying pond conditions, experimentally collect water seeping from a pond into a nearby channel during the dry summer season.
2. determine if a hydraulic model exists that can accurately predict the experimental results.
3. apply the results to predict the physical conditions under which the seepage from a pond could contribute to a stream throughout the dry season.

The scope of this work was limited to studying an area with a high groundwater table.

1.3 Thesis Organization

The body of this thesis is organized into five major sections:

- Literature Review (2.0)
- Materials and Methods (3.0)
- Results (4.0)
- Discussion and Application (5.0)
- Conclusion and Recommendations for Future Work (6.0)

Three of these sections, Literature Review, Materials and Methods and Results, are further broken down into minor headings to help keep the information organized and easy to follow. The last major section, Conclusion and Recommendations for Future Work, is followed by a list of references which are referred to throughout the thesis, as well as six appendices. The appendices, which contain calculations and other important data, are referenced in the text at the appropriate location.

2.0 Literature Review

The literature review covers the causes and impacts of low flow, methods to determine in-stream flow requirements and current methods to augment low flow. The various hydraulic models that could be used to model pond seepage and are only mentioned briefly in the introduction are also explained in more detail.

2.1 The Causes of Low Flow

Changes to the hydrologic regime of a watershed have a direct impact on the flow patterns of a watershed's streams. Urbanization, forestry, water withdrawals, dams and global warming can all affect the hydrology of the streams and contribute to low flow.

2.1.1 Urbanization

Urbanization has been described as the land use with the greatest impact per unit area on the hydrological regime of a watershed (Nener and Wernick, 1997). As the conversion of naturally vegetated surfaces to roofs, sidewalks, streets and parking areas occurs, overland flow is accelerated and infiltration is restricted, resulting in flood discharges of greater magnitude and frequency than those that occurred before urbanization (Graf, 1977). In addition to the change in surfaces, urban development introduces another significant change in the form of the radical alteration of channel networks through the addition of numerous artificial channels (Graf, 1977). These changes cause the channel network to become more efficient in collecting water quickly and produce the flash floods common to urban areas (Graf, 1977). The greater intensity of the runoff from an urban watershed means that it has much more erosive potential, and this can cause instability of the channel downstream from the drainage area. This instability can lead to increased sedimentation and substantial changes in stream morphology (Swanson, 1987).

The changes to stream hydrology caused by urbanization often lead to the scouring of stream banks, bedload movement and the destruction of fish eggs. Flash floods can also wash out fish eggs as well as fish fry and other aquatic organisms (Graf, 1977). The changes can also cause lower groundwater tables and streams levels in dry seasons (Nener and Wernick, 1997).

Urban development is one of the main land uses affecting water quality and fish habitat in the Fraser Delta Habitat Management Area (Nener and Wernick, 1997). Water quality is affected by urbanization through the impacts of land clearing, the presence of numerous diffuse pollution sources and the disposal of solid and liquid wastes. In addition, the clearing of land, including streamside vegetation, can affect the physical aquatic habitat through the loss of pond and wetland areas, the loss of food from insects and leaf litter and the shade that riparian vegetation provides for a stream.

The population of BC is expected to increase dramatically over the next several decades and in the Lower Fraser Valley, is predicted to double by the year 2031. The anticipated population growth and urban development will have significant impacts on water quality and habitat, particularly in the Lower Fraser Valley (Nener and Wernick, 1997).

2.1.2 Forestry

Forestry is a major land use in the Fraser Basin and involves numerous activities which are potentially detrimental to water quality and biophysical fish habitat (Nener and Wernick, 1997). The relationship between forests and water is complex, depending upon the type of forest, nature of precipitation and upon geology, topography and soils. Forests play an important role in regulating stream flows and maintaining water quality (Toews and Brownlee, 1981). As a general rule, small streams (first to third order) which are heavily dependent on the riparian and terrestrial environment for their physical and biological character, will be most greatly influenced by forestry practices which alter those environments. By contrast, larger streams and rivers (fourth order or greater) are not as reliant on terrestrial environments and will be less affected by land-based forestry practices (Toews and Brownlee, 1981).

Timber harvesting can affect the hydrology of a watershed through soil compaction and vegetation removal (Toews and Brownlee, 1981). Hydrological changes in the watershed result in higher peak flows causing sedimentation of habitat and deposition of bedload (Wood, 1997). Major bedload deposition usually occurs when there is an abrupt change in stream gradient when the stream changes from a steep valley to the relatively shallow gradient of the floodplain. During the low summer flows, these deposition areas are often completely dry or have minimum surface flow which is not conducive to supporting fish habitat (Wood, 1997).

Forest harvesting, where extensive amounts of riparian vegetation are removed, can disrupt the normal thermal regime of a stream by causing increased summer and decreased winter stream temperatures and larger diurnal fluctuations, and can also exacerbate natural low flow conditions (Nener and Wernick, 1997). While the loss of riparian vegetation due to logging should be greatly reduced with the introduction of the Forest Practices Code of BC and supporting code in 1995, there are thousands of kilometers of streambanks in the Fraser basin that were cleared prior to the introduction of the Code, and problems will persist until the vegetation fully regenerates (Nener and Wernick, 1997).

2.1.3 Water Withdrawals

The diversion of water from streams and the storage of water for municipal, agricultural, flood control and hydropower uses, usually leads to altered stream flows and potential changes in the carrying capacity of streams for salmonid fisheries (Bjornn and Reiser, 1991). The Water Act requires the licensing of all surface water withdrawals in British Columbia but does not require licensing of groundwater withdrawals (Nener and Wernick, 1997). Water licenses are issued through the Water Management Branch of the Ministry of the Environment, Lands and Parks (MELP). Provisions of the Act do not recognize in-stream water requirements for supporting aquatic life and while MELP has been discussing revisions to the Water Act to address the needs of aquatic life, changes have yet to be made (Nener and Wernick, 1997). While there is some degree of co-operation developing between the Water Management Branch and the Department of Fisheries and Oceans in this regard, many streams are already over-licensed (Nener and Wernick, 1997).

Natural seasonal variations in stream flow are compounded by water withdrawals as many crops have their highest demand for irrigation water in the summer months when stream flow is at its lowest (Wallace and Pawloski, 1988). Most water licenses issued for irrigation withdrawals do not require anyone to monitor the amounts of water removed from a surface source, and the actual withdrawals may exceed permitted volumes (Nener and Wernick, 1997). In some areas of the Fraser Basin, excessive water withdrawals result in very low stream flows and may cause some streams to go dry during hot summer months. The unlicensed withdrawal of groundwater can also detract from flows in streams causing severe impacts, especially during the summer when groundwater is a stream's primary water source.

2.1.4 Water Releases

Another important issue that affects stream flow is the release of water from dams built for hydroelectric power or reservoirs. If appropriate amounts of water are not released from the dam, low flow problems can result downstream, whereas appropriate management can substantially reduce any negative impacts. In one study, rearing subyearling chinook salmon (*Oncorhynchus tshawytscha*) were captured in the Snake River and tagged with passive integrated transponders to provide an index of their survival to Lower Granite Dam, the first of eight dams encountered by seaward migrants. Water was released from the reservoirs upstream of the Lower Granite Dam to augment summer flows and decrease water temperature. The results indicated that the summer flow augmentation increased subyearling chinook survival by limiting thermally induced mortality and by reducing predation (Connor et al., 1998).

In California, efforts by a citizens group to improve the flow regime of a stream led to a successful court trial in which fish conservation played a major role. During the summer, lower reaches of the Putah creek dried up completely due to low releases from the diversion dam and a reduction of other sources of water resulting in a major die-off of fish (Moyle et al., 1998). The Putah Creek Council took the Solano County Water Agency to court and the trial resulted in a court order for sufficient flows to be released during the summer to keep even the lowermost reach of the creek a living stream (Moyle et al., 1998). The Putah Creek trial is representative in many ways of the actions being taken throughout North America to protect and restore aquatic ecosystems. In this case, as in many others, a local citizen's group was the catalyst in a successful challenge to the way water was allocated by a major water project (Moyle et al., 1998).

2.1.5 Global Warming

Global warming will not only affect temperature, but also the amounts and seasonal distribution of precipitation (Levy, 1992). In turn, these changes will directly influence the volume and timing of seasonal runoff. Watersheds where there is currently a close balance between water demand and water supply will be the most vulnerable to impacts. One scenario predicts a substantial increase in winter precipitation, coupled with possible reductions during summer months for British Columbia (Ripley, 1987). This would result in greater seasonal flow fluctuations and potentially lower summer flows in the Fraser Basin.

2.2 Determining In-stream Flow Requirements

There are several ways to determine the in-stream flow needs for fisheries' requirements as a part of an overall management plan for a stream environment (Wood, 1997). The four most commonly used methods are the average annual flow method, the watershed method, the flow characteristic method and the wetted perimeter method.

The watershed method uses basin wide information, such as the size of the watershed, to recommend stream flow requirements. The flow characteristic method incorporates a flow duration curve for the stream and the in-stream flow needs are then based on historical flow levels. The average annual flow method, also called the Montana method, identifies a percentage of the average annual flow for the in-stream flow need for fisheries. Finally, the wetted perimeter method involves a relationship between the discharge and the wetted perimeter of the stream. The recommended in-stream flow need is often identified as the point where an increase in discharge does not result in a significant increase in wetted perimeter (Wood, 1997).

For smaller projects, the Montana method is most often used and it has been field tested in the United States and Canada since the 1970's (Wood, 1997). The results of the field studies indicated that the condition of the aquatic habitat is remarkably similar for most of the streams carrying the same portion of the average flow (Tennant, 1976).

Table 1 summarizes the Montana method for estimating the in-stream flow requirements for fish. Flows less than 30% of the annual flow are considered low and summer habitat is generally poor if flows are less than 10% of the annual flow. Summer flows greater than 30% of the mean annual flow for a stream usually support healthy fish habitat (Orth and Leonard, 1990).

Table 1: Recommended In-stream Flow Requirements for Fish Based on a Percentage of the Average Annual Flow

Comparative Description of Flows	Recommended Base Flow	
	October - May	**April - September**
Flushing or Maximum	200% of the average flow	
Optimum Range	60 – 100% of the average flow	
Outstanding	40%	60%
Excellent	30%	50%
Good	20%	40%
Fair or Degrading	10%	30%
Poor or Minimum	10%	10%
Severe Degradation	10% of average flow to zero flow	

(Tennant, 1976)

2.3 Problems Caused by Low Flow

Low flow can have serious impact on aquatic life. It not only affects their biological state as a critical water quality characteristic, but also their physical habitat.

2.3.1 Water Temperature

Water temperature can be a critical water quality characteristic in many streams. Low water flow influences the maximum and minimum stream water temperature as well as the daily temperature fluctuations. The temperature of water, particularly temperature extremes, can affect the survival of certain flora and fauna residing in a body of water as the type, quantity, and well being of flora and fauna will frequently change with a change in water temperature (Brooks, 1997). In general, an increase in water temperature causes an increase in biological activity, which in turn places a greater demand on dissolved oxygen. The fact that the solubility of oxygen in water is inversely related to temperature compounds this effect (Brooks, 1997). Salmonids are coldwater fish with definite temperature requirements during rearing. Although fish may survive at temperatures near the extremes of their suitable range, growth is reduced at low temperatures because all metabolic processes are slowed, and at high temperatures because most or all food must be used for maintenance (Bjornn and Reiser, 1991). Many salmonids also change behavior with increases or decreases in temperature.

2.3.2 Contaminant Dilution

Streams with low flows are also more easily affected by contaminant inputs, as there is less water available for dilution. This can be an important factor when considering a constant pollution discharge to a stream, as the concentration levels may become toxic for a certain period over the summer. It is also an important factor in determining the toxicity of an accidental spill. The more water a stream has available for dilution, the better the chances that aquatic life will survive a contaminant input.

2.3.3 Fish Production

The carrying capacity of a stream, and hence fish production, may vary yearly if controlling habitat components such as stream flow vary widely from year to year at critical periods such as late summer (Bjornn and Reiser, 1991). A dam on the Rogue

River, Oregon, altered flows and temperatures in both summer and winter and as a result, changed the timing of salmon and steelhead (*Oncorhynchus mykiss*) fry emergence, adult migration, fish distribution in the river and adult mortality (Bjornn and Reiser, 1991). Studies have also shown that stream flow conditions cause wide variations in freshwater survival and subsequent adult returns. Low summer flows can reduce the rearing area and consequently the production of coho (*Oncorhynchus kisutch*), cutthroat (*Oncorhynchus clarki*) and steelhead smolts (Salmonid Enhancement Program, 1980). Chinook salmon may also be affected as in colder streams and more northerly rivers, juvenile chinook rear in freshwater for a full year (Salmonid Enhancement Program, 1980). Studies have shown that the abundance of adult coho salmon is a function of the number of smolts produced, which in turn is related to stream flow and other factors that regulate the production of smolts (Bjornn and Reiser, 1991).

2.3.4 Migration

Fish migrating upstream must have stream flows that provide suitable water velocities and depth for successful upstream passage (Bjornn and Reiser, 1991). A variety of techniques have been used to estimate the flows required for migrating fish. One study reported that salmon need 30-50% of the average annual flow for passage through the lower and middle reaches of a river and up to 70% for passage up headwater streams (Bjornn and Reiser, 1991). A procedure was developed for estimating minimum flows required for migrating fish on the basis of minimum depth and maximum velocity criteria (see Table 2) and measurements in critical stream reaches, usually shallow riffles. For each area measured, the minimum flow was determined as the flow that met the velocity and depth criteria on at least 25% of the total stream width and on a continuous portion equaling at least 10% of the total width. The mean selected flow from all the areas measured is recommended as the minimum flow for passage (Bjornn and Reiser, 1991). Waterfalls, debris jams and excessive water velocities may also impede migrating fish. Obstructions that are insurmountable at one time of year may be passed by migrating fish at other times when the flow has changed (Bjornn and Reiser, 1991). The primary migrations of concern are the upstream movement of adult fish to spawning habitat and the movement of juvenile fish into off-channel habitat. Most juveniles migrate to overwinter habitat in late summer to fall, when an increase in flow or decrease in temperature may provide a cue for migration (Whyte et al., 1997).

Table 2: Water Depths and Velocities that Enable Upstream Migration of Adult Salmon and Trout

Species of Fish	Minimum depth (m)	Maximum velocity (m/s)
Fall Chinook salmon	0.24	2.44
Spring Chinook salmon	0.24	2.44
Summer Chinook salmon	0.24	2.44
Chum salmon	0.18	2.44
Coho Salmon	0.18	2.44
Pink salmon	0.18	2.13
Sockeye salmon	0.18	2.13
Steelhead	0.18	2.44
Large Trout	0.18	2.44
Trout	0.12	1.22

(Bjornn and Reiser, 1991)

As the majority of migration barriers are those associated with either high water velocities or vertical drops, the swimming capabilities and lifestage of the target species can be applied to assess potential access barriers (Whyte et al. 1997) (see Table 3). Sustained speeds are the swimming velocities that can be maintained for extended periods of time. Prolonged speeds are swimming velocities that can be maintained for passage through difficult areas while burst speeds are for escape and feeding (Whyte et al., 1997).

Table 3: Swimming and Jumping Capabilities of Some Salmonids

Species and Lifestage		Maximum Swimming Speed (m/s)			Maximum Jump Height (m)
		Sustained	Prolonged	Burst	
coho/chinook:	adults	2.7	3.2	6.6	2.4
	juveniles (120 mm)		0.6		0.5
	juveniles (50 mm)		0.4		0.3
sockeye:	adults	1.0	3.1	6.3	2.1
	juveniles (130 mm)	0.5	0.7		
	juveniles (50 mm)	0.2	0.4	0.6	
chum/pink:	adults	1.0	2.3	4.6	1.5
steelhead:	adults	1.4	4.2	8.1	3.4
cutthroat/ rainbow:	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

(Whyte et al., 1997)

2.3.5 Spawning Area

Stream flow also regulates the amount of spawning area available in any stream by regulating the area covered by water and the velocities and depths of water over the gravel beds (Bjornn and Reiser, 1991). Relations between flow and amount of suitable spawning area have been assessed or predicted by methods based primarily on measurements of water depths and velocities in areas with suitable substrate (see Table 4).

Table 4: Water Depth, Velocity and Substrate Size Criteria for some Salmonids

Species	Minimum Depth (m)	Velocity (m/s)	Substrate Size (mm)
fall chinook salmon	0.24	0.30 – 0.91	13 – 102
spring chinook salmon	0.24	0.30 – 0.91	13 – 102
summer chinook salmon	0.30	0.32 – 1.09	13 – 102
chum salmon	0.18	0.46 – 1.01	13 – 102
coho salmon	0.18	0.30 – 0.91	13 – 102
pink salmon	0.15	0.21 – 1.07	13 – 102
sockeye salmon	0.15	0.21 – 1.07	13 – 102
kokanee	0.06	0.15 – 0.91	13 – 102
steelhead	0.24	0.40 – 0.91	6 – 102
rainbow trout	0.18	0.48 – 0.91	6 – 52
cutthroat trout	0.06	0.11 – 0.72	6 - 102

(Whyte et al., 1997)

2.4 Current Methods to Augment Stream Flow

There are two commonly used methods to increase the minimum summer flows for an entire stream or for a specific section of stream that has been seriously impacted by low flows. The two methods are:

1. construction of a dam
2. interception of groundwater or subsurface flow (Wood, 1997).

2.4.1 Dam Construction

The most commonly used method for flow control is to increase the storage capacity of a lake or natural impoundment in the headwaters by constructing a dam across the outlet of the lake. For example, at Cameron Lake on Vancouver Island, a low head concrete dam was installed at the outlet from the lake to augment the summer flows in the Little Qualicum River and associated spawning channels downstream (Wood, 1997).

There are many design criteria that need to be considered when constructing a small dam. For example, a dam in a lake outlet could result in higher peak lake levels and in some cases the flooding of valuable timber and property. Land may have to be purchased or long-term leases secured to compensate for land impacted by higher water levels (Wood, 1997). Dams also need to be designed for unobstructed fish passage and a fishway is often required. Topography and foundation affects the size and type of construction and most structures will require design input from a professional engineer specializing in this field (Wood, 1997).

The approximate volume of water that can be stored in a small lake or pond can be estimated by knowing the area of the lake and the average depth of water that falls within the zone of live storage. The flow that can be produced from this storage depends on the length of time over which the water will be released and can be calculated from the following equation:

$$Q = \frac{AS}{t} \qquad (1)$$

where:

Q = flow (m^3sec^{-1})

A = lake area (m)

S = available storage depth (m)

t = time of release (s)

Costs for small dams will vary widely depending on location, access and foundation materials at the outlet (Wood, 1997). Costs will increase considerably for a site where construction personnel and materials must be flown in by helicopter and will also increase with any additional equipment needed such as constant flow valves, intake screens, remote control equipment and fishways. Constant flow valves are often used on remote installations when it is desirable to release a constant flow regardless of the elevation of the water in the lake. For larger installations, when it may be necessary to vary the flow over the summer and fall months, the lake level, amount of flow released and valve opening can be monitored and regulated from a remote location using UHF radio (Wood, 1997). The cost of a dam, therefore, can range from as low as $20,000 to over $90,000.

2.4.2 Interception of Groundwater and Subsurface Flows

Another method to increase base summer flows is to increase the groundwater component and intercept subsurface flows. This work is normally carried out in an existing side-channel. While many side-channels are connected to the main channel at their upstream and downstream ends, groundwater-fed channels are connected to the main channel at the downstream end only (Wood, 1997). Interception of subsurface and groundwater flows provides a stable flow of water of consistent quality. The base flow can be increased by intercepting subsurface flow with polyethylene cutoffs, constructing deep pools, lowering the channel invert and diverting groundwater seeps into the channel (Wood, 1997). Excavated groundwater channels usually produce small discharge volumes, between 0.08-0.2 m^3s^{-1} and low water velocities of 5-15 cms^{-1} (Lister and Finnigan, 1997).

These steps are usually undertaken as a part of a project to develop an existing side channel to provided spawning, rearing and overwintering habitat for several species of fish. The cost of such a project varies depending on the size and type of work done. For work done on a side-channel adjacent to the Fording River in the Kootenays, the total cost was approximately $200,000, of which $40,000 was attributable to increasing the base flow in the side channel (Wood, 1997).

2.5 Hydraulic Pond Seepage Models

While a water budget can be used to determine the total amount of seepage from an existing pond, it cannot be used to determine the direction of seepage or for predicting seepage at the design stage. Hydraulic models are useful for predicting both the rate and direction. Three models were found that could be used to determine pond seepage and are described below.

2.5.1 Wetland Bottom Seepage Equation

Numerical models have been developed to quantify the amount of infiltration from the bottom of a wetland, based on the hydraulic conditions. The water condition of soils under a treatment wetland may range from fully saturated, forming a water mound on a shallow regional aquifer, to unsaturated (Figure 1).

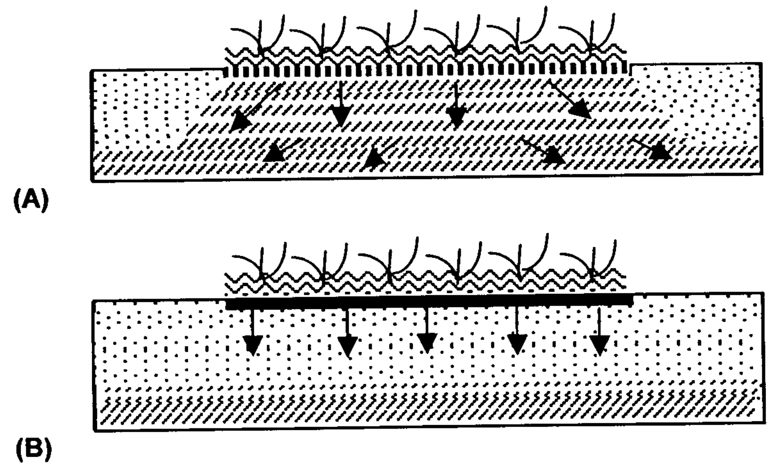

Figure 1: Groundwater-Wetland Interactions

(A) large leakage leading to groundwater mounding

(B) small leakage, unsaturated conditions beneath the clay layer

If there is enough leakage to create a saturated zone under the wetland, three-dimensional flow calculations are needed to ascertain the flow though the wetland bottom to the groundwater (Kadlec and Knight, 1995). These calculations require a substantial quantity of data on the regional water table, regional groundwater flows and hydraulic conductivity of soil layers and as a result, can be costly (Kadlec and Knight, 1995). If, however, the wetland is lined with a relatively impervious clay layer, unsaturated conditions will likely exist in the soil beneath this layer with the regional shallow aquifer some distance below, and the wetland leakage can be estimated from the following equation (Kadlec and Knight, 1995):

$$Q_{gw} = kA\left[\frac{H_w - H_{cb}}{H_{ct} - H_{cb}}\right] \quad (2)$$

where:

A = wetland bottom area (m^2)

H_w = wetland water surface elevation (m) (see Figure 3)

H_{cb} = elevation of clay bottom (m)

k = hydraulic conductivity of the clay (m/day)

H_{ct} = elevation of clay top (m)

Q_{gw} = seepage rate (m^3/day).

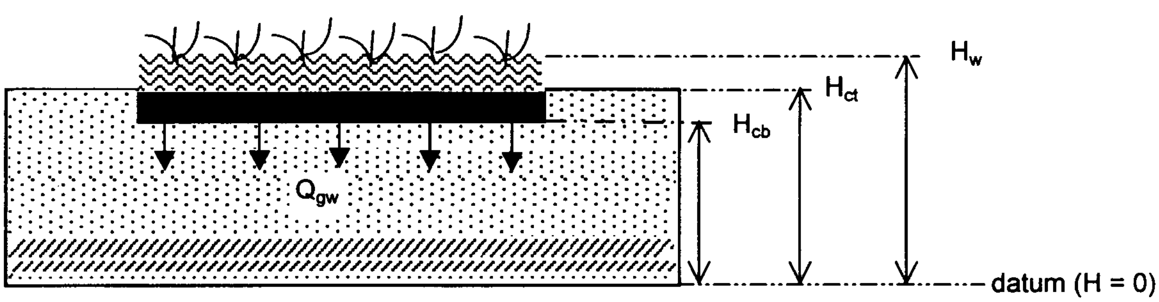

Figure 2: Wetland Bottom Seepage Equation Variables

2.5.2 Embankment Seepage Equations

Equations have been developed since the 1930's to determine the flow line and seepage of water through an embankment. The formulae vary depending on the angle of the discharge face (α) (Figure 3).

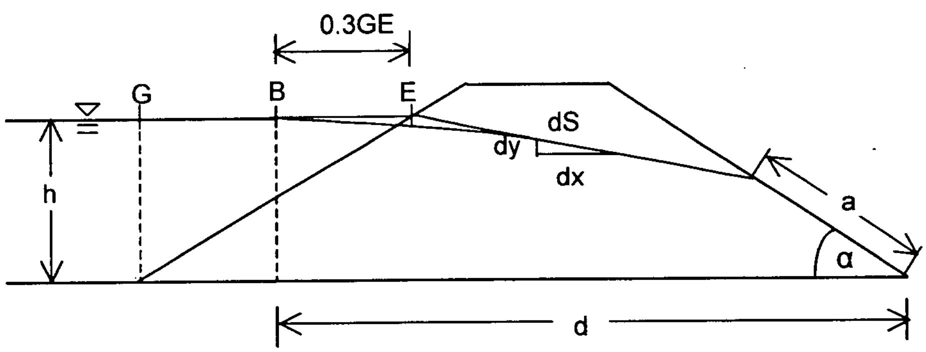

Figure 3: Dam Section

For α >30°, the following equations are used (Singh and Varshney, 1995):

$$q = \frac{k\left(a^2 \sin^2 \alpha - h^2\right)}{2(a - S)} \qquad (3) \qquad\qquad a = S - \sqrt{S^2 - \frac{h^2}{\sin^2 \alpha}} \qquad (4)$$

where:

q = discharge per unit length of the dam (m^3/m/day)

k = hydraulic conductivity (m/day)

h = distance from embankment base to the water level in the reservoir (m)

S = length of the seepage curve (m) determined from h and d (d = distance along the base from the discharge face to B) (see Figure 2)

a = distance along the discharge face from the base to the point where the seepage line meets the discharge face (m)

2.5.3 Three Dimensional Pond Seepage Equation

Fipps and Skaggs (1998 & 1990) investigated pond seepage in two and three dimensions. Using a numerical method, they solved the Richards equation for three-dimensional, combined unsaturated and saturated flow and this was used to analyze pond seepage for physical conditions characteristic of the North Carolina Barrier Islands. The numerical solutions from this analysis were then used to develop and test an approximate analytic method for calculating three-dimensional pond seepage. The analytic method was determined by dividing the flow regime into two components: radial flow near the pond and linear one-dimensional flow away from the pond (Figure 4).

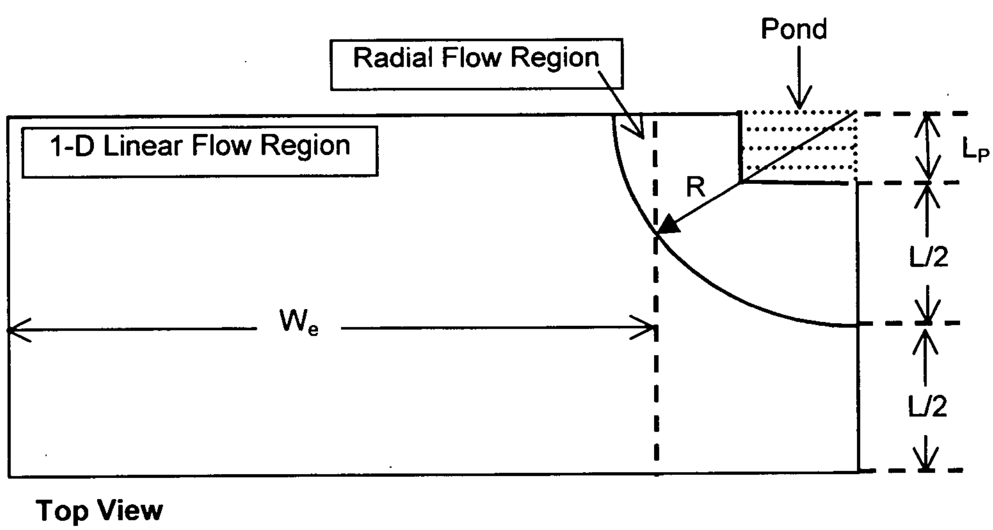

Figure 4: Division of the Solution Domain into Two Regions: Radial Flow near the pond and 1-D Flow away from the pond

Equating the flow equations for the linear and radial regions, the approximate solution for the pond seepage was (Fipps and Skaggs, 1990):

$$Q = \frac{k_s L_e}{2W_e}\left(H_P^2 - H_S^2\right) \quad (5) \qquad\qquad L_e = \frac{L + L_P}{1 + \left[2\left(L + L_P\right)/\left(W_e \pi\right)\right]\ln\left(R/L_P\right)} \quad (6)$$

where:

k_s = saturated conductivity (m/day)

W_e = effective width of the linear flow region (m)

H_P = head at the pond (m)

L = length from the pond to the no-flow boundary at y = 0 (m)

H_S = head at the sink (m)

L_P = pond length (m)

R = (L_p+L/2) = horizontal extent of the radial flow region (m)

Equation 5 is valid as long as $W_e > 0$. The predicted rates from the analytical solution agreed with those obtained from the numerical solution (Fipps and Skaggs, 1990).

3.0 Materials and Methods

This section covers the specifics of the experiment location including the soil profile and the work done in this area to determined soil type and hydraulic conductivity, as well as tests performed on a well upslope of the experiment site. The site layout is covered as are experimental design and data analysis. Details can be found in following five sections.

3.1 Site Characteristics

The experiment, which was conducted on Vancouver Island near Courtenay, B.C., started during the summer dry period (mid August) in 2000 and ended before the heavy rain started in October. This site was chosen for two reasons: from observations of the site and a pond already located there, it was known that the water table was high, and there was available space to construct the experimental pond. The observations regarding water table location were made over a five year period by the owner of the approximately 1 ha property, Dr. Royann Petrell. In the winter, the water table would be an average of 0.31 m below the surface and in the summer, between 0.9-1.5 m below the surface, depending on the location within the property.

3.1.1 Soil Profile

The soil profile of the excavated zone in the experiment area consisted of five undisturbed layers. Particle size analysis was done on soil samples from the site, and this information was used to determine the soil type of each of the layers. Samples of soil from four of the five layers from two different locations (A and B) at the experiment site were analyzed. The layer that was not analyzed was the hardpan material, as the properties of this layer are dependent on its cemented structure and this structure cannot be determined from particle size analysis. A sample of 120 grams of soil from each of the four layers analyzed was placed in glass beakers with about 250 ml of distilled water. Any organic matter was oxidized from the samples by adding 30% hydrogen pyroxide solution in stages over the course of several days. Next, the samples were dried in an oven at 105 °C for 24 hours and then gently pulverized with a mortar and pestle to ensure that the individual grains were separated but not broken.

A set of sieves was used that had the following sizes: 2 mm, 1.7 mm, 1.18 mm, 1 mm, 710 μm, 250 μm, 100 μm and 53 μm. A 40 g and a 20 g sample of the soil from each layer were shaken through the sieves. The different sample weights were used to check for any differences due to sample weight. The amount of soil trapped by each sieve and the amount that passed through the finest sieve (<53 μm) were all weighed and the results can be seen in Appendix 1. The samples from the two locations were similar and there were no major differences between the results from the two different sample weights. The soil fractions were then classified according to the particle size limit classification scheme of the Canada Soil Survey Committee (Gee and Bauder, 1986) (see Appendix 1) For the first three soil layers, the percentage of sand was very high (85, 95 and 98%), with the first layer considered a loamy sand, the second layer a medium-fine sand and the third a fine sand. The bottom soil layer did, however, have a much higher percentage of fine sediment, approximately 40%, and could therefore fall into one of three different textural classes of soil. There were two different methods that could have been used to determine which of the three classes the bottom layer belonged. The first was the hydrometer method which is done in the lab, the second a common field method which determines textural class by feel. As this layer would not be used directly in model calculations, the faster and simpler field method was used. This method involves moistening a small quantity of soil with water and kneading it to the consistency of putty to determine how well the soil forms casts or ribbons (Brady, 1990; Foth, 1990). The kind of cast or ribbon formed is related to clay content and is used to categorize soils as loams, clay loams or clays. The test indicated that the bottom layer was a sandy clay. The test procedures and results can also be found in Appendix 1. The textural classes and depths of the five soil layers can be seen in Table 5.

3.1.2 Hydraulic Conductivity

Once the textural class of each soil layer had been determined, the range of hydraulic conductivity (k) for each soil type was then found from literature (see Table 5).

Table 5: Soil Properties

Soil Textural Class	Average Layer Depth (m)	Hydraulic Conductivity (m/day)
Loamy Sand	0.26	5 - 1.51
Medium-Fine Sand	0.40	26- 4.3
Hardpan	0.12	0.12 - 0.03
Fine Sand	0.20	8 - 2.46
Sandy Clay	4.6	0.30 - 0.12

(Bouwer, 1978; Coche, 1985; Kadlec and Knight, 1995)

3.1.2.1 Inverse Auger Hole Method

On-site hydraulic conductivity testing was also done using the inverse auger hole method to help determine an overall k for the soil profile. This overall k value could then be used to help determine appropriate values for the individual layers (section 3.1.2.3). The auger hole method, where a hole is made to a depth below the water table and then pumped out, could not be used to determine k as the water table was too low.

Two different auger holes were dug to a depth of 1 m. The soil in and around the holes was then soaked for over six hours before the tests were performed. This step is necessary when the initial conditions are moist or dry. Once the soil was saturated, the holes were filled with water and the rate at which the water surface dropped measured until the rate became nearly constant. This rate was then translated into a k value using the following equation:

$$k = \frac{1.15r}{t_2 - t_1} \log \frac{y_1 + 1/2\,r}{y_2 + 1/2r} \qquad (7)$$

where:

r = borehole radius (m)

$y_{1,2}$ = height of water-level in borehole above its bottom at times t_1 and t_2 (m)

$t_{1,2}$ = time (s)

(Coche, 1985; Burke et al., 1986)

The resulting overall k values for the soil profile ranged from 1.9 – 2.2 m/day. The results from the inverse auger hole tests can be seen in Appendix 2.

3.1.2.2 Hazen Method

It is also possible to estimate the hydraulic conductivity for sand sediments from their grain size using the Hazen method. This method could only be used to determine an estimate of the individual k values for the two sand layers in the soil profile, the medium-fine sand and the fine sand. The method uses grain size distribution information obtained from sieve analysis to plot the particle distribution curve from which a value for the grain size at 10%, called the effective grain size (D_{10}), can be found. The following equation is then used to calculate k:

$$k = C(D_{10})^2 \quad (8)$$

where:

k = hydraulic conductivity (m/day)

C = Hazen coefficient

D_{10} = effective grain size (mm)

(Brassington, 1998)

This method is only capable of giving a general idea of the hydraulic conductivity of sands. Values for C can be found in literature and vary with grain size ranging, from 350 for very fine sand to 1300 for coarse sand (Brassington, 1998). Values for k of 7.1 and 5.4 m/day were calculated for the medium-fine sand layer and the fine sand layer respectively. (See Appendix 2)

3.1.2.3 Selection of k Values for Each Soil Layer

Using depth measurements from 3 different soil profiles (Appendix 2), and the average of the range of k values in Table 5 (values from literature) and the results from the Hazen method (for the two sand layers) the overall k value, K_{avg}, was calculated using the following equations (Singh and Varshney, 1995):

$$K_{avg} = \sqrt{K_h K_v} \qquad (9)$$

$$K_v = \frac{L}{\frac{L_1}{k_1} + \frac{L_2}{k_2} + \frac{L_3}{k_3} + \ldots.. + \frac{L_n}{k_n}} \qquad (10) \qquad\qquad K_h = \frac{k_1 L_1 + k_2 L_2 + k_3 L_3 + \ldots. + k_n L_n}{L} \qquad (11)$$

where:

K_v = average permeability normal to the flow (m/s)

K_h = average permeability parallel to stratification (m/s)

L_n = layer thickness (m)

L = $L_1+L_2+\ldots.L_n$ (m)

k_n = layer k value (m/s)

K_{avg} = overall average permeability (m/s)

The individual layer k values were adjusted within the ranges from Table 5 such that the resulting K_{avg} for each ditch face was close to the average value determined from the inverse auger hole method (see Appendix 2). The individual k values for the top three soil layers were then used in model calculations (Section 4.3 and Appendix 5).

3.1.3 Well Test

A 5.7 m deep well existed approximately 30 m upslope of the experiment site. Two rate-of–rise tests were done to determine the recharge rate of the groundwater. As the well penetrated to a soil depth much deeper than the soil profile of the pond, these tests were done only to gain information about groundwater movement, not to determine k values for use in model calculations. A pump was used to abruptly lower the water level in the well by a minimum of 30 cm (Bouwer, 1978) and then the rising water level was recorded every 15 minutes. The collected well data can be seen in Appendix 3. The time necessary for the water level in the well to rise 90% of the distance back to equilibrium ($t_{90\%}$), is given by (Bouwer, 1978):

$$t_{90\%} = 0.0527 \frac{r_c^2}{kL_e} \ln \frac{R_e}{r_w} \qquad (12)$$

where:

r_c = radius of the well section where the water level is rising (m)

k = hydraulic conductivity of the aquifer (m/day)

L_e = height of the perforated, screened, uncased or otherwise open section of the well through which groundwater enters (m)

R_e = effective radial distance over which the head difference is dissipated (m)

r_w = radial distance between the well center and undisturbed aquifer (m) (r_c plus the thickness of gravel envelope or developed zone outside the casing)

As the $t_{90\%}$ value was known for each well test, this equation was used to calculate a k value for the well (see Appendix 3). The calculated values ranged from 5.5×10^{-3} - 6.6×10^{-3} m/day, indicating a very slow moving aquifer.

3.1.4 Site Layout

The test pond was constructed using an excavator on an undisturbed site (Figures 5-7) that had a downward slope of approximately 1.5%. The pond was sized so as to maximize the space available for construction. The bottom measured 6.1 m by 2.5 m and followed the natural slope of the hardpan, which resulted in a slight downward slope of approximately 2% and a maximum depth of 0.5 m. The pond bottom followed the hardpan as it was assumed to be the impermeable layer. The side slopes were designed for stability and averaged 35%.

To ensure that the seepage from the pond could be collected regardless of flow direction, the pond was almost completely enclosed by a ditch, which was positioned 2.4 m away from the pond surface and dug to a depth below the pond bottom (Figure 8). The ditch did not completely surround the pond because of an obstructing culvert, through which water flows into one of the ditch faces (D) starting in mid fall. The ditch faces were dug down past the hardpan layer and sloped in such a way as to permit flow sampling (Figure 8). Ditch faces A to C were approximately 1 m wide and the bottoms of faces A and B were sandy clay, while the bottom of face C was compacted fine sand. Face D also had a sandy clay bottom but was slightly wider and deeper than the other ditches, measuring 2.8 m wide and 1.6 m deep. Pipes were used at the end of each ditch face to channel the water for collection and measurement (Figure 9), and a sump was located at the corner of A and B. A sump pump was used to empty the sump and keep the water from backing up in the ditch.

Figure 5: Site before Construction
(The site was used to store soil which are the piles seen in the picture)

Figure 6: During Construction

Figure 7: Pond and Ditch Construction Complete

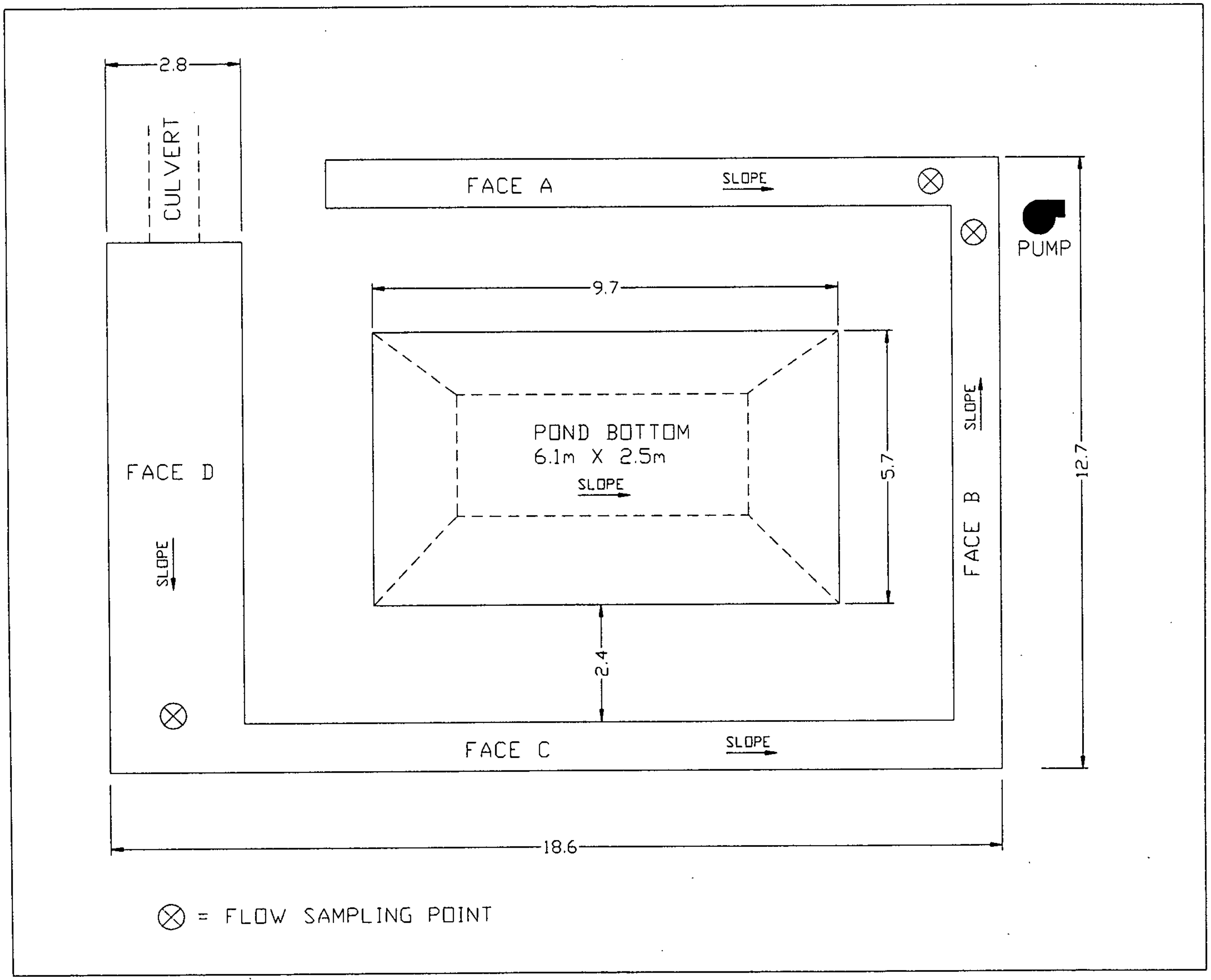

Figure 8: Pond and Ditch Layout

Figure 9: Flow Sampling Location for Ditch Face C

The ground water table was approximately 0.7 m below the pond bottom so that the overall characteristics of the site, in terms of ground water depth, were assumed to be similar to what would be found adjacent to a natural stream receiving groundwater discharges. An evaporation pan was set up at the site 150 mm above the ground on a level wooden platform. Any changes in the pan water level were multiplied by a pan coefficient of 0.7 (Yoo and Boyd, 1994) to obtain the approximate evaporation loss for the pond. The evaporation pan was monitored along with a rain gauge throughout the experiment and there was no appreciable water loss or gain.

3.2 Experimental Design

Four experiments were conducted to study direction and magnitude of seepage flow under different conditions: unlined staged, rising unlined staged, lined staged and falling stage. In the "unlined staged" experiment, the pond was filled to a depth of 45.7 cm and then the water level was dropped in stages, so that seepage data could be collected at five different stages: 45.7 cm, 38.1 cm, 30.5 cm, 22.9 cm and 15.2 cm. The pond was filled by water pumped from either the well that the rate-of-rise tests were performed on, or from another pond located on the property. At each stage, three sets of seepage flow (i.e. in the ditch faces) measurements were taken over a two to three hour period to ensure that a steady state had been reached. The pond stage was maintained over this period by water pumped out of the ditch, which was recycled into the pond. A water hose was also turned on as necessary to help maintain the water level. The flow reading for each ditch per set of measurements was taken a minimum of three times. Once the readings for the lowest stage (15.2 cm) were taken, the pond was refilled to the next stage, 22.9 cm, and the readings for the "rising unlined staged" experiment were taken in the same manner as described above for the unlined staged. This was repeated until the pond was back to the highest stage (45.7 cm). This entire process was repeated twice so that two data sets for both unlined staged and the rising unlined staged experiments were obtained. The process of dropping the pond stage and data collection for the unlined staged experiment was repeated with the pond partially lined for the "lined staged" experiment. A liner was placed in the pond that covered the entire bottom and all the sides except B to examine if flow out of the pond could be restricted and thus be sustained for a longer time period. The pond was empty for one week prior to the start of the lined experiment. Side B was chosen because it had the highest level of measurable flow and could, therefore, be measured most accurately. In the "falling stage" experiment, the pond was filled to the 45.7 cm stage and allowed to fall continuously until it reached the 15.2 cm stage. The water height was recorded at 30-minute intervals and flow measurements were taken at various stages. Because of time restrictions due to the approaching heavy rain season, this was only done once and was not repeated for the lined experiment as it was for the unlined experiments.

3.3 Data Analysis

Due to its irregular shape, the pond volume was determined according to the method outlined by Yoo and Boyd (1994). The surface area of the pond was measured at different stages, and then the following equation was used to determine the volume between the stages (see Appendix 4):

$$V = \frac{h}{3}\left(A_u + A_l + \sqrt{A_u A_l}\right) \qquad (13)$$

where:

V = volume between the upper and lower contour lines

A_u = area within the upper contour

A_l = area within the lower contour

h = vertical distance between contours

By adding all the resulting volumes, the total pond volume was determined. The total volume at each stage was then graphed, with the resulting stage-volume curve described by the following equation ($R^2 = 0.99$):

$y = 37.51x^2 + 2.05x$ (14) where y = volume (m^3) and x = stage (m).

For the rising unlined staged experiment, the flow from each ditch face as well as the total ditch flow was graphed for each of the five measured stages. This data was also graphed for both the lined and unlined staged experiments, as was the rate of change of the water level between the stages. The flow contribution for each ditch face was calculated for each of the five stages and compared for all three experiments. For the falling stage experiment, the changes in volume and stage over time, the ditch face flows at different stages and the total collected ditch flow compared to pond volume were all graphed. The total collected ditch flow was also graphed against pond losses.

Under the existing site conditions the equations for embankment seepage and wetland bottom seepage, Equations 2 and 3, were determined to be the most applicable for modelling the pond seepage. Equation 5, the three-dimensional pond seepage equation, assumes an infinitely large pond and as a result, does not consider seepage from the pond banks. For the small pond used in these experiments, not considering bank seepage would be a large source of error. The hydraulic conditions necessary for Equation 2 to be valid, that the underlying strata is partially dry with a shallow aquifer located some distance below, were met, as the layer of fine sand under the hardpan would ensure that unsaturated conditions existed under the impermeable layer and the water table was located a distance below the impermeable layer (0.7 m).

For the "unlined staged" experiment, both models were used to predict the pond seepage and were graphed along with the pond losses and the collected ditch flow. The same was done for the "lined staged" experiment, except that only Equation 3 was used to predict seepage out of unlined bank B as the pond was considered perfectly lined (see Appendix 5 for model calculations).

4.0 Results

This section presents the results for each of the four experiments: rising unlined staged, unlined staged, lined staged and falling stage. Several interesting observations that were noted during the experiments are also presented as well as with the modelling results for the unlined staged and lined staged experiments.

4.1 Observations

During the experiments, several interesting observations were recorded. The first concerned the portion of the ditch lengths that collected water relative to the pond length at a given stage. Water in the two side ditches, A and C, went back in the ditch a distance (d) beyond the water line of the pond water surface and the distance (d) varied with pond depth (Figure 10). Secondly, while the most flow appeared in Ditch Face B, there was very seldom if any flow in Ditch Face D. Thirdly, the ground around the pond became wet from the filling process and when the lined pond was filled, seepage could be seen under the clear plastic liner. One final observation related to the temperature difference between the pond and the ditch water. While the pond water heated up during the day, the seepage water collected in the ditches was always cooler. For example, when the pond water was 17 °C, the ditch water was 10 °C.

4.2 Rising Unlined, Unlined, and Lined Staged Seepage Flows

The flow in each ditch face in both the unlined and lined experiments contributed a different fraction of the total ditch flow and varied with stage (Table 6). For all the experiments, it can be seen that Ditch Face B had the highest percentage of flow, and that

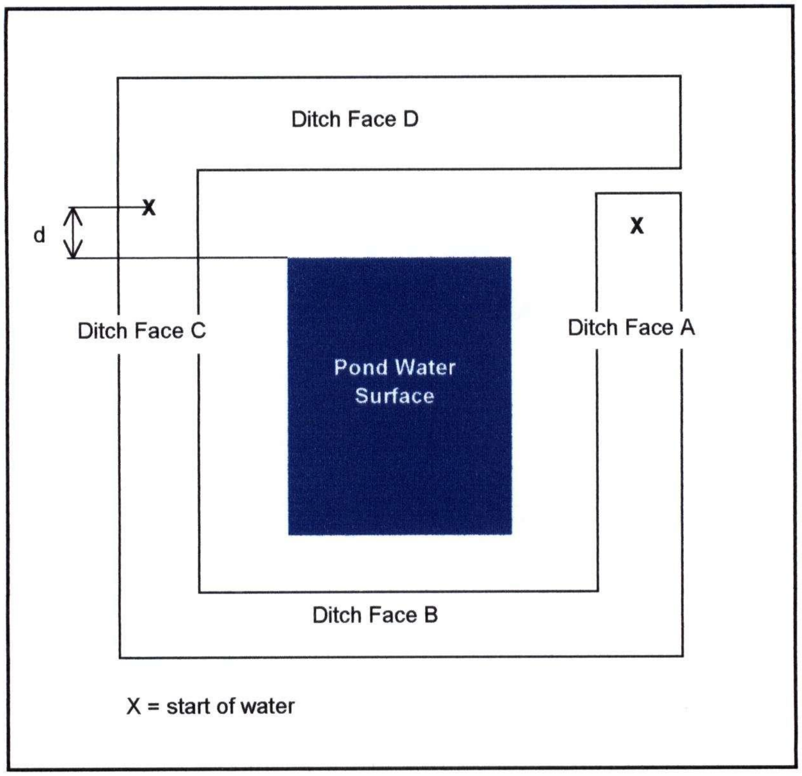

Figure 10: Start of Water Pooling in the Side Ditch Faces Relative to the Pond Edge

as the stage decreased, this percentage increased. Ditch Face C had the next highest percentage, followed by Ditch Face A. Ditch Face D always had the lowest percentage of flow. The percent values are very similar for the unlined staged and the rising unlined staged experiments. For the lined staged experiment, Ditch Face B, being the only unlined ditch face, had higher flow contributions than for the unlined staged. For Ditch Face C, the values for the lined experiment were smaller than for the unlined experiments and decreased as stage dropped, whereas for the unlined experiments the percentages increased with decreasing stage. The percentages for Ditch Face A were also smaller for the lined than the unlined, and Ditch Face D had no flow for the lined staged experiment.

Table 6: Flow Contribution from each Ditch Face for both Unlined and Lined Staged Experiments (% ± 1 Standard Deviation, n=6)

Stage	Ditch Face Flow Contribution (% of Total Ditch Flow)											
	Rising Unlined Staged Experiment				Unlined Staged Experiment				Lined Staged Experiment			
(m)	A	B	C	D	A	B	C	D	A	B	C	D
0.46	25±1.6	38±2	33±0.7	3±0.3	25±1.3	38±2.3	34±1.1	4±0.1	14±1.7	59±1.9	28±0.3	0
0.38	23±0.2	40±1.3	34±1	3±0.4	23±0.9	40±1.7	34±0.6	3±0.2	12±1.6	63±2.1	25±0.6	0
0.30	19±0.8	44±0.9	36±0.5	0.5±0.1	21±0.8	41±2.1	36±1.1	2±0.2	9±0.7	67±0.5	24±0.4	0
0.23	18±1.3	46±0.5	37±0.9	0	18±0.9	45±2	36±1.1	1.5±0.2	7±1.1	73±0.9	21±0.3	0
0.15	15±1	48±3.4	37±2.5	0	14±1.4	48±3.3	37±2	0	4±1	80±0.3	16±1.2	0

From Figures 11 and 12, the graphs of ditch flows against stage for unlined staged and lined staged, it can be seen that a linear equation best describes the relationship between the change in flow with stage while for the rising unlined staged, a polynomial equation is best (Figure 13). Lining the pond decreased the ditch face flows an average of 32% as compared to the unlined staged experiment, while the flow values for the rising unlined staged were on average 20% smaller than the values for the unlined staged. Figures 14 and 15 show the rate at which the water level in the pond dropped between the five measured stages. The rates decreased with stage interval, with the highest rate being for the drop between 0.46-0.38 m. The rates were also higher for the unlined staged experiment than for the lined staged. The change in ditch face flow between measured stages can be seen in Figure 16 for the lined staged experiment. The graph shows that the system was very responsive to changes in the pond water level with these changes being reflected in the collected ditch face flows without a long lag time.

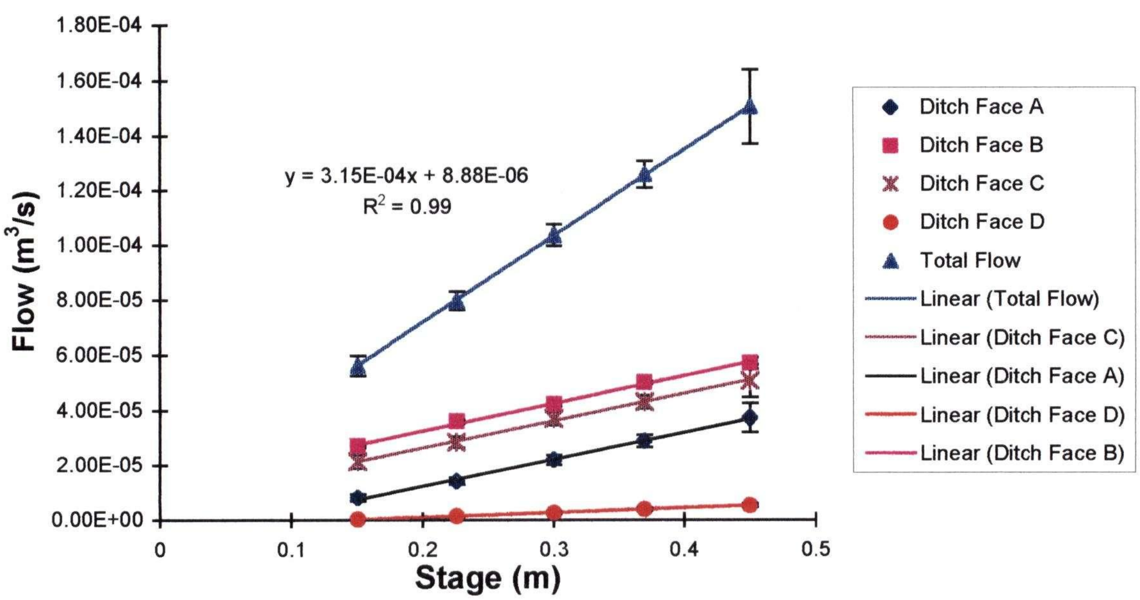

Figure 11: Ditch Face Flows and Total Ditch Flow for Unlined Staged Experiment (Error Bar = 1 Standard Deviation, n = 6)

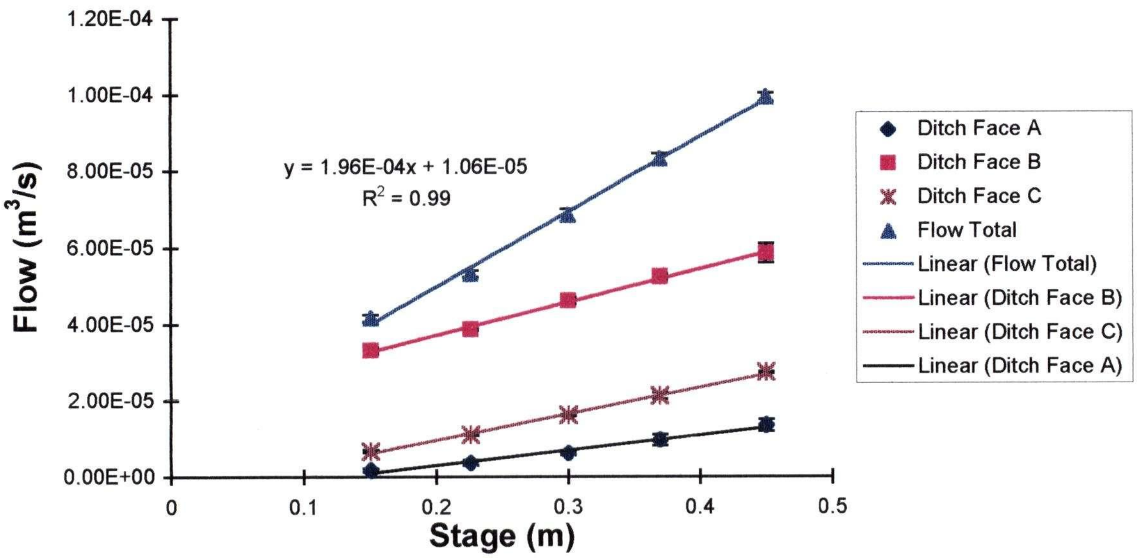

Figure 12: Ditch Face Flows and Total Ditch Flow for Lined Staged Experiment (Error Bar = 1 Standard Deviation, n = 6)

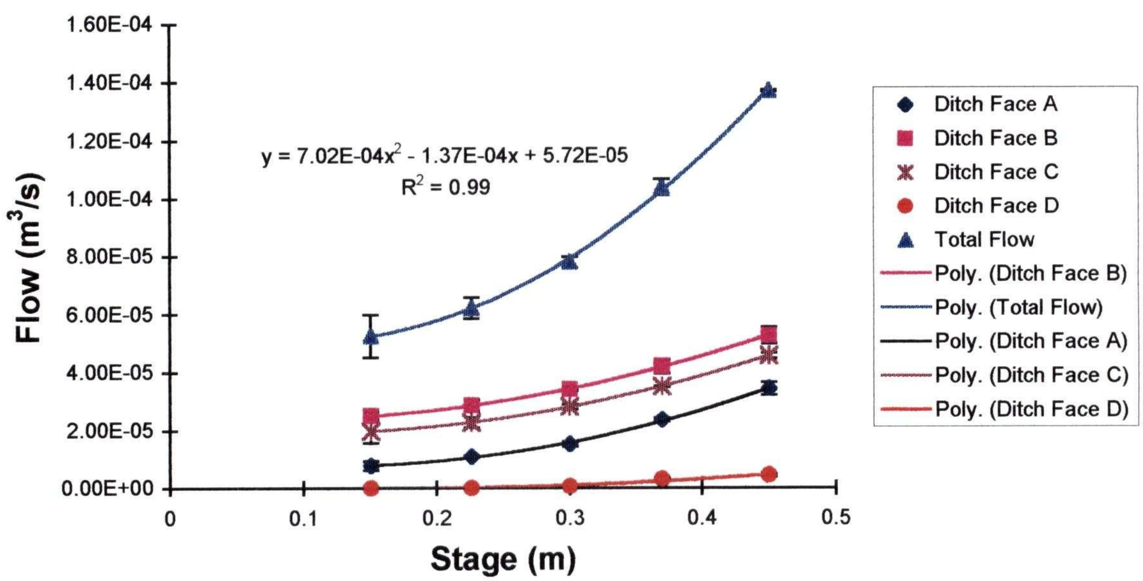

Figure 13: Ditch Face Flows and Total Ditch Flow for Rising Unlined Staged Experiment (Error Bar = 1 Standard Deviation, n = 6)

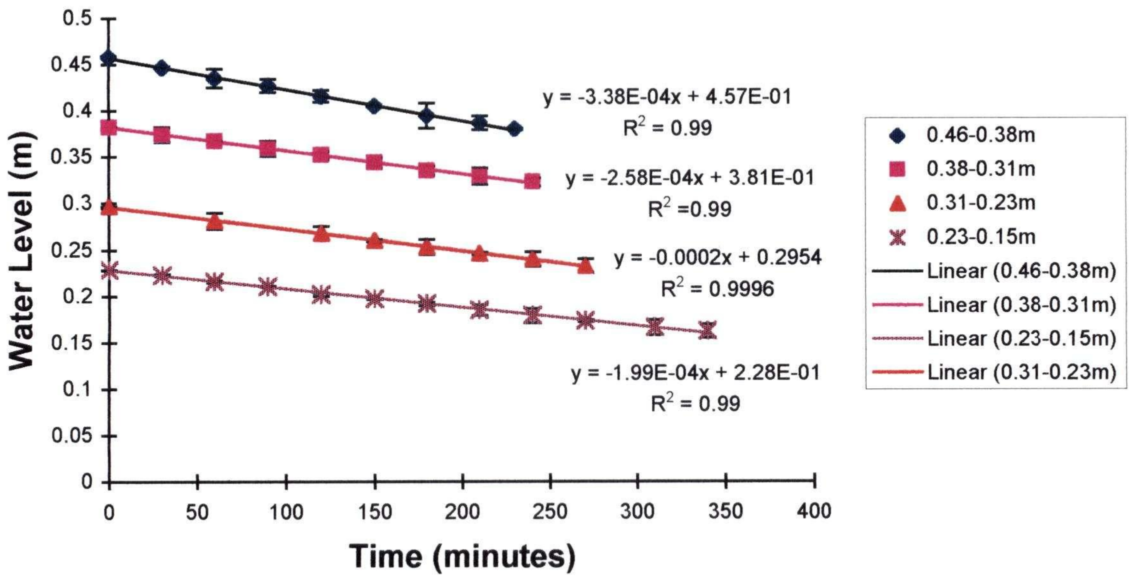

Figure 14: Change in Water Level Between Measured Stages for Unlined Staged Experiment (Error Bar = 1 Standard Deviation, n = 2)

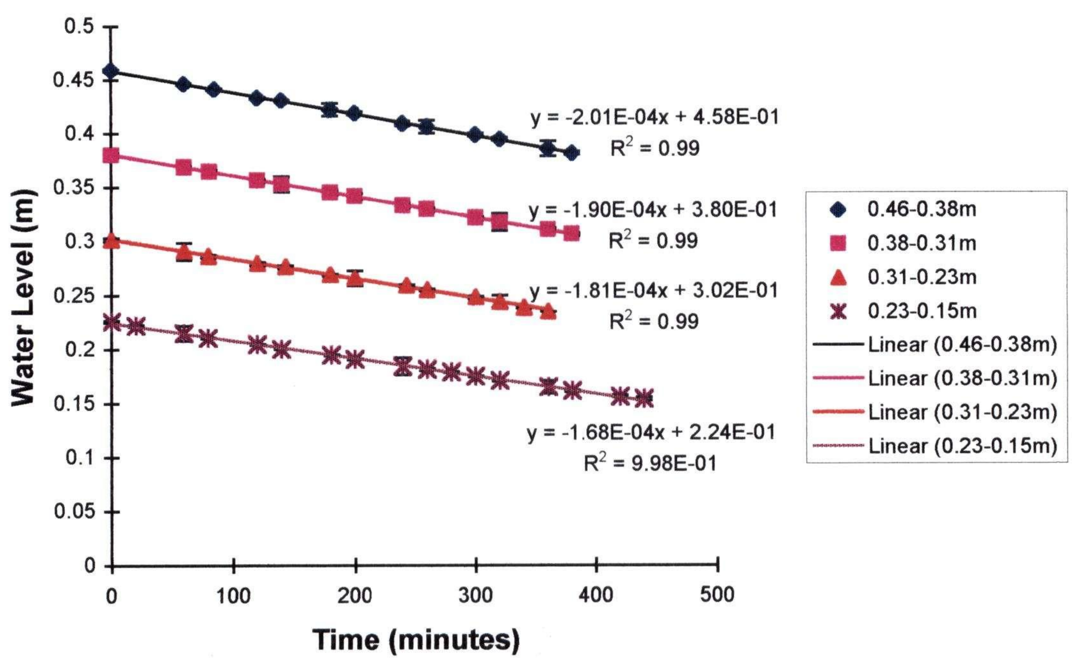

Figure 15: Change in Water Level Between Measured Stages for Lined Staged Experiment (Error Bar = 1 Standard Deviation, n = 2)

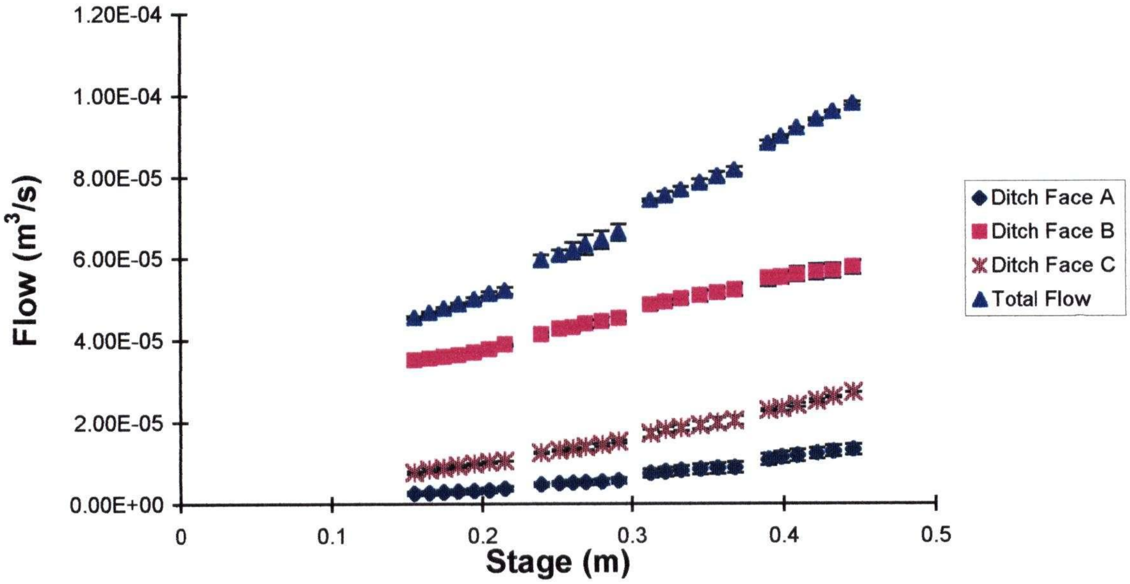

Figure 16: Ditch Face Flows Between Measured Stages for Lined Staged Experiment (Error Bar = 1 Standard Deviation, n = 2)

4.3 Modelling Results for Unlined and Lined Staged Experiments

For the unlined staged, ditch flow was 30% less than pond flow at the highest stage (and pond volume), and was 20% higher at the lowest stage (Figure 17). From Figure 17, it can be seen that the combined flow from the two models for the "unlined staged" experiment matched the pond losses. For the higher pond volumes, the flow from bank model was the most significant contributor while for the smaller volumes, the flow from the wetland model was more significant. For the "lined staged" experiment, ditch flow was 22% less than pond flow at the highest pond volume and was 9% higher at the lowest stage (Figure 18). From Figure 18, it can be seen that the flows from the bank model were much smaller than the actual pond losses.

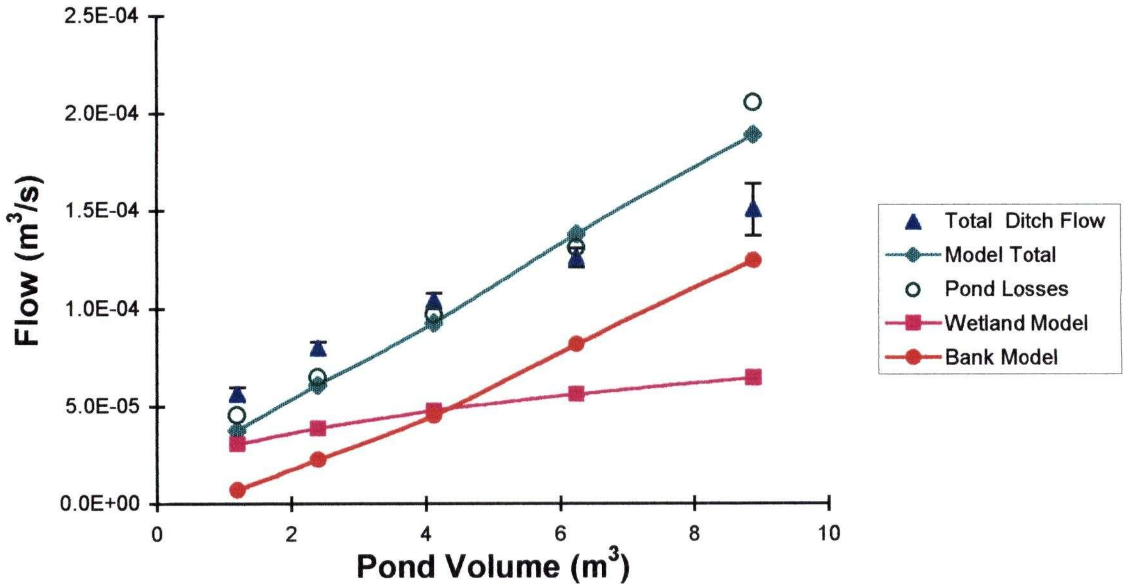

Figure 17: Comparison of Pond and Ditch Flows to Model Results for Unlined Staged Experiment (Error Bar = 1 Standard Deviation, n = 6)

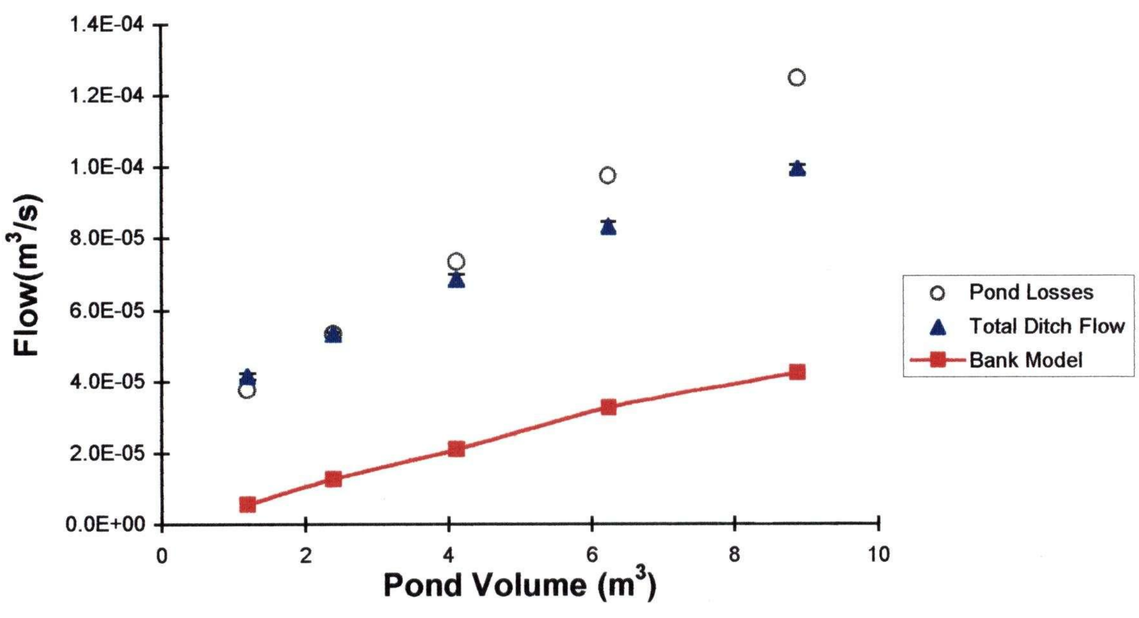

Figure 18: Comparison of Pond and Ditch Flows to Model Results for Lined Staged Experiment (Error Bar = 1 Standard Deviation, n = 6)

4.4 Falling Stage Experiment

For the "falling stage" experiment, Figure 19 shows a linear relationship between change in stage and time and a polynomial relationship between the change in pond volume with time. The non-linear relationship of the pond volume to time was expected due to the pond's shape. In Figure 20, the relationship between the pond volume and the ditch flow can best be described by a linear equation, the same as for the unlined and lined staged experiments. The amount of water collected at the lower stages was higher than the amount the pond lost (Figure 21). This trend can also be seen in Figure 17 for the lower pond volumes and again in Figure 18 for the very smallest pond volume.

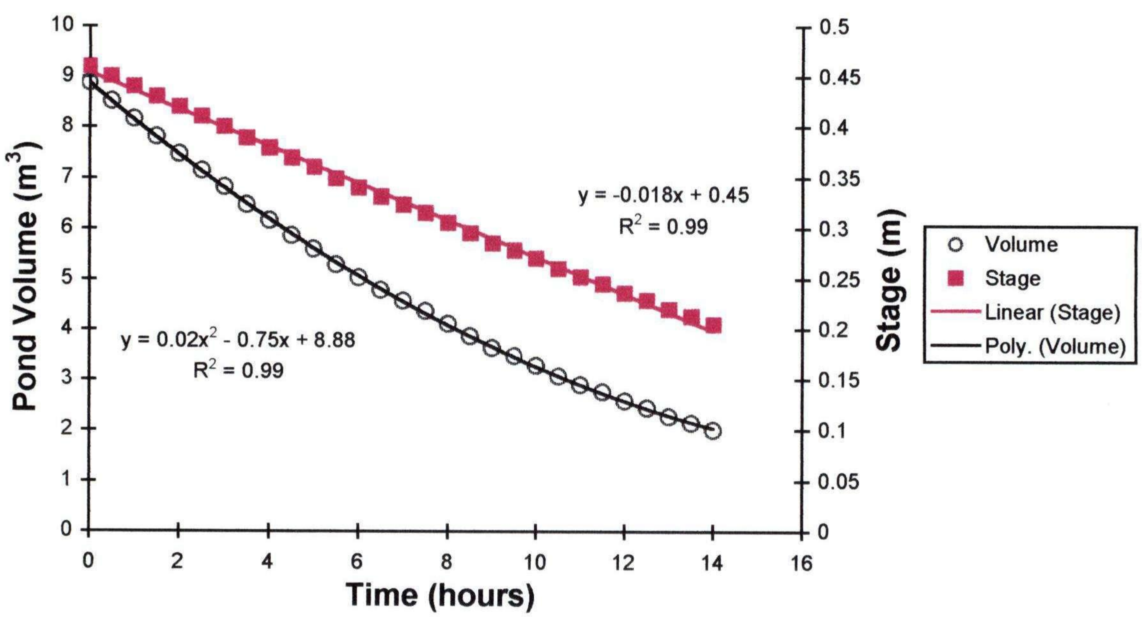

Figure 19: Change in Pond Volume and Stage over Time for Falling Stage Experiment

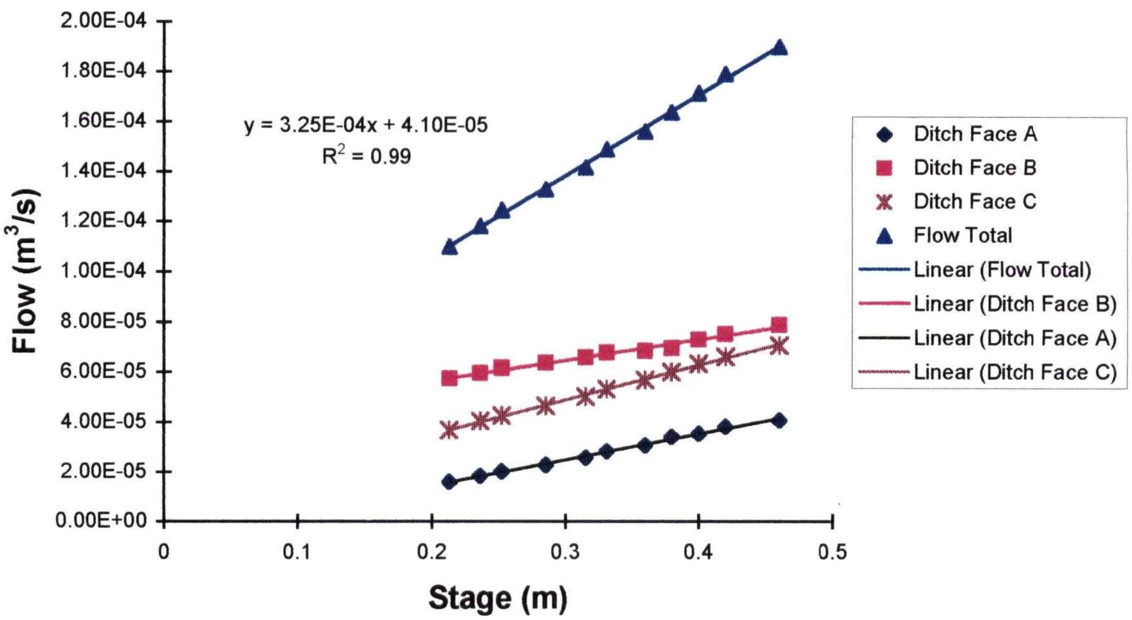

Figure 20: Ditch Face Flows and Total Ditch Flow for Falling Stage Experiment

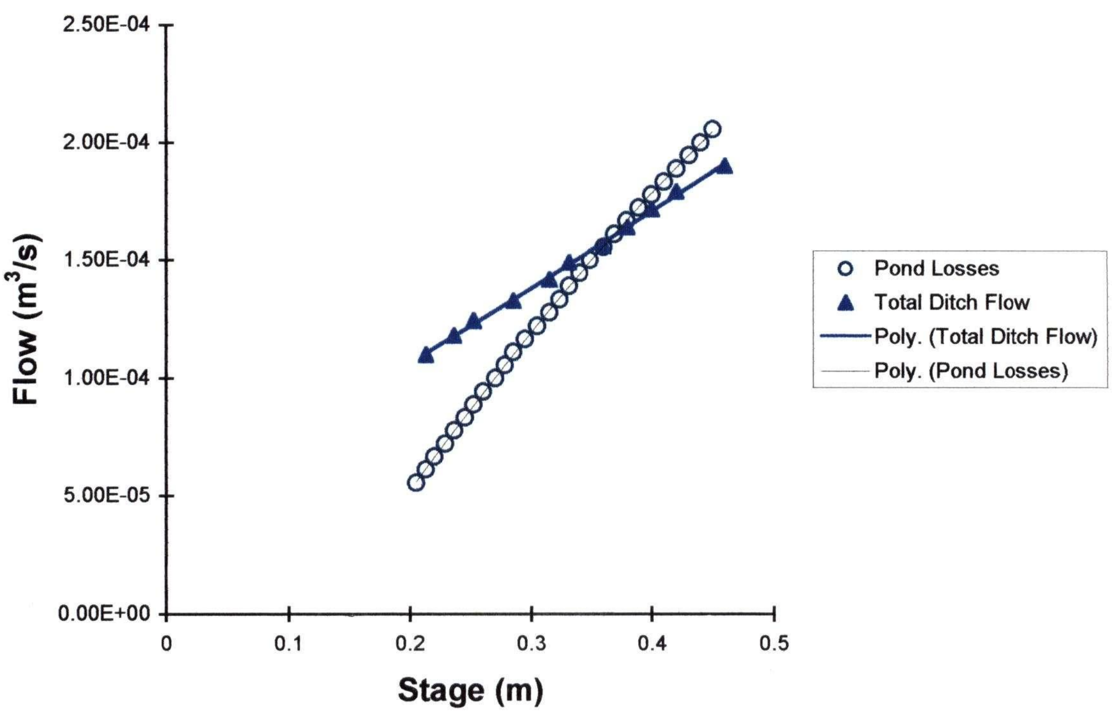

Figure 21: Comparison of Total Ditch Flow and Pond Losses for Falling Stage Experiment

5.0 Discussion and Application

The water collection system (ditch) was effective in collecting the pond seepage from both the pond bottom and the banks. In fact, the total flow collected from the ditch was 20% higher than the total pond loss at the lower stages for the unlined staged experiment, 9% higher for the lined staged experiment and 45% higher for the falling stage experiment. This over-collection was probably due to a contribution from the saturated water zone in the banks that was not considered in the water balance. At the highest stages, however, the ditch flows were smaller than pond losses. For the unlined staged experiment the ditch flow was 30% smaller than pond losses at the highest stage, 22% smaller for the lined staged experiment and 8% smaller for the falling stage experiment. This was likely due to more losses to deep seepage or water being retained to keep the banks saturated.

All of the ditch faces collected varying percentages of the pond seepage (Table 6), and this can be explained by the site slopes. The experiment location sloped in the direction of Ditch Face B, with a slight slope towards Ditch Face C, so it is reasonable that they would collect the highest percentage of flow while Ditch Face D, located at the upslope end of the pond, would have little or no flow. The land slopes also explained the increasing percentage of flow in Ditch Face B and C. As the water height in the pond fell with the resulting depths being smallest along sides A and D, bank seepage through these sides decreased and as a result, the amount of water seeping to these ditches would decrease and the main flow would be concentrated in B and C.

The ditch face flow contributions were very similar for the rising unlined staged and the unlined staged experiment. When looking at the flow vs. stage graphs for each experiment (Figures 11 and 13), however, it can be seen that while the trend for unlined staged experiment can best be described by a linear relationship, the trend for the rising unlined experiment can best be described by a polynomial equation. This difference is likely due to changes in the saturated water zone in the banks. For the unlined staged, as the pond stage falls, this water could contribute to the ditch flow. For the rising unlined staged, as the pond was being filled, the banks were becoming saturated from the seepage and this process would reduce the amount of water reaching the ditch faces. In fact the flow values for the rising unlined staged experiment were on average 20% smaller than for the unlined staged experiment.

From the changes in flow contribution (Table 6), it can be seen that the liner did affect the direction of the pond seepage compared to the unlined experiments, concentrating it to the unlined Ditch Face B. While the relationship between flow and stage (Figure 12) was best described by a linear equation, which was similar to the unlined staged, the flow values were on average 32% smaller. For the lined staged experiment, flows from the bank model were much smaller than the actual pond losses (Figure 18) indicating the pond was not perfectly sealed, which was confirmed by direct observation on site. While it may not have been perfectly lined, the lined pond did in fact lose water more slowly than the unlined pond, approximately 38% slower (Figures 14 and 15), and as a result, had smaller ditch flows.

For the unlined staged experiment, the combined flow from the two models matched the pond losses very well. The model total predicted the pond losses best for the middle pond volumes, within ±5%, while it was not as accurate for the highest or lowest volumes, predicting flows that were smaller by 9% and 20% respectively. The 20% is, however, for flows much smaller than the flow values at the higher volumes. These differences could be due to a number of reasons: k values used, measurement error or losses or gains due to bank storage. For the higher pond volumes, the flow from the bank model was the most significant contributor while for the smaller volumes, the flow from the wetland model was more significant. This agreed with site observations that as the water level dropped, more of the banks were exposed and the surrounding ground began to dry out indicating less seepage through the banks.

The good overall agreement between the experimental and theoretical results, as presented in the Results section, provides the rationale for using such models to design ponds to maintain low summer time flows, given that the site is similar to the experimental conditions regarding water table height. In using the models, however, caution must be placed on the use of k because the flow values as calculated using the two hydraulic models (Equations 2 and 3) were highly sensitive to the value of k. It is quite common for hydraulic conductivities to be quoted to the nearest order of magnitude rather than as a precise value, as this reflects the reality of spatial variations within an aquifer caused by geologic factors (Brassington, 1998). These variations in k will affect the model results, for example, an increase in k of 15% caused an average increase in flow of 5% from the bank model (Equation 3) and 15% from the wetland model

(Equation 2). An additional consideration is the importance of lining the pond; based on experimental observations the flow out of the pond should be restricted as much as possible to a small area.

If ponds are to be used to augment low stream flows, there are several factors that must be closely considered. First, it is important to have a good understanding of the groundwater hydrology of the area including the water table location, flow direction and the groundwater contribution to the stream. When a stream is in direct contact with an unconfined aquifer, the stream may recharge groundwater or receive discharge from the groundwater, depending on the relative water levels. An influent stream is one that contributes to groundwater stores; an effluent stream is one receiving groundwater. If large highly permeable aquifers are contained within a drainage area, the stream's base flow will be sustained even through prolonged droughts but if the aquifers are small and of low permeability, the base flow will decrease rapidly and may cease altogether (Bear, 1979). For a pond to be effective, the stream must remain effluent throughout the summer because if the water table drops below the stream bottom and the stream becomes influent, any water from the pond will seep down to the groundwater and not contribute to stream flow. The soil profile of the area must be studied and the necessary hydraulic conductivities determined. This process may involve extensive site testing and the use of a trial pond.

Stream morphology must also be considered. The impact that the seepage from a pond will have on a stream is highly dependent on the width, slope and velocity of a stream as well as the habitat type: pools, glides and riffles. Pools are defined as areas of slower deeper water with a concave bottom profile and a water surface gradient near 0%. Glides include all areas of fast-flowing non-turbulent water, while riffles are areas of turbulent fast-flowing water (Johnston and Slaney, 1996). Pond seepage will have a greater impact on a small, stable, slower moving stream with a gentle slope that has complex habitat, including lots of in-stream pools. The most important design consideration is the length of time the pond must last and this must be carefully determined. Rainfall data, stream flow data and the habitat requirements of the aquatic life that will be affected must all be examined.

To illustrate the use of the model, a hypothetical case will be used of a small stream that provides habitat for juvenile coho. The depth of water juvenile salmonids use depends on what is available, the amount and type of cover present and the perceived threat from predators and competitors. Juvenile coho prefer depths of at least 25 cm deep and velocities between 5-24 cm/s (Bjornn and Reiser, 1991). Assume groundwater tests have been done and the results indicate that the stream receives groundwater throughout the summer but not enough to maintain an adequate stream flows. The stream has a slope of 0.1%, width of 1 m and depths ranging from 5-15 cm in riffle/glide sections, with deeper pooled sections. Using the Manning equation, the flow rates for the different depths can be calculated. An appropriate value for the Manning roughness coefficient is determined to be 0.07 (Yang, 1997) (Lencastre, 1987). To increase the depths 5 cm in the riffle/glide sections requires varying increases in flow: to increase 20 cm to 25 cm requires an increase in flow of $9.5x10^{-3}$ m^3/s; 10 cm to 15 cm requires $7.4x10^{-3}$ m^3/s; and to increase 5 cm to 10 cm requires an additional $5.7x10^{-3}$ m^3/s (Appendix 6).

Assume the pond is filled over the winter and that the last rainfall occurs mid June, and that the dry period lasts until mid September for a total of three months. In practice, the pond should be excavated so that the bottom slope is orientated toward the stream. The pond bottom should also be higher than the stream bottom. For pond water seepage to endure throughout the dry season while still contributing reasonable flow to a stream, the banks of the pond must be carefully lined and bottom seepage contained to a small area adjacent to the bank closest to the stream. Under these conditions the wetland model, Equation (2), can be used to calculate the dimensions of the pond needed to produce a given flow (Appendix 6).

To provide a flow of $9.5x10^{-3}$ m^3/s, it was determined that 43700 m^3 of water must be stored, indicating that more than one pond will be required. Twelve ponds, with a depth of 2 m and top dimensions of 39 m by 64 m could provide the necessary flow to increase the depths from 5-20 cm to 12.5-25 cm (Appendix 6). The ponds were designed with a seepage area of 3.5% of the bottom pond area and to last close to 5 months with the flow rate dropping to $4x10^{-3}$ m^3/s at the 2.5-month mark. At this point the depth would range from 8.3 cm to 22 cm in riffle/glide sections. To provide a flow of $7.4x10^{-3}$ m^3/s, 31200 m^3 of water would need to be stored. Nine ponds with the same dimensions as before would provide the flow necessary to increase the depths from 5-20 cm to 11-24 cm at the start and 7.6-21.4 cm at the 2.5-month mark. To provide a flow of $5.7x10^{-3}$ m^3/s, it is determined that 21900 m^3 of water would need to be stored. Six ponds would provide the necessary flow to increase the depths from 5-20 cm to 9.2-22.5 cm at the start and 6.8-21 cm at the 2.5-month mark.

Obviously the construction of this number and size of ponds would be a large project that in many cases would not be economically feasible and the use of ponds, therefore, has limitations. Rather than focusing on the impact pond seepage has on the depth of riffle/glides sections, we can instead look at the affect pond seepage could have on the depth of pooled sections. Pools are areas of slower, deeper water and for the stream used for the sample calculations, must have a minimum area of 1 m^2 and a minimum residual depth of 0.2 cm to be considered a pool (Johnston and Slaney, 1996). The residual depth is the difference between the maximum pool depth and the depth at the pool outlet, and approximates the pool depth at zero flow. If 7300 m^3 of water is stored in two 39 m by 64 m ponds or eight 24 m by 38 m ponds, the depth in the flowing sections will increase from 5-20 cm to 6.5-21 cm at the start and drop to 5.7-20.4 cm at the 2.5-month mark. While the increases in the depths of the riffle/glide sections are not great, the flow has increased between 6-53% at the start and by 2-20% at the 2.5-month mark. These flow increases would translate into increased depth in pooled areas, which would in turn provide improved habitat for the juvenile coho.

Another use the ponds could have is to allow unrestricted fish movement over a shorter time period than the three months looked at in the example. Juvenile coho move into off channel areas as post-emergent fry during the early summer (Lister and Finnigan, 1997).

Ponds could be used to provide improved stream flow and allow fish to migrate to other parts of the stream where better habitat might be found during this period. As the ponds would not have to last for as long, smaller ponds could be used than the ones in the previous example.

Another benefit that may be realized with the use of seepage ponds is the reduction of stream temperatures. As noted in the Results section, the seepage flow collected in the ditches was cooler than the pond water and as many streams with low flow also have problems with high stream temperatures, the addition of cooler water to these streams would be an added advantage to using seepage ponds.

Two important factors that would affect the size of ponds that are not considered in the above examples are rainfall and evaporation. Evaporation can be a major source of water loss from ponds and this can be especially critical during dry periods. The size of the ponds should take into consideration an estimate, based on the location of the ponds, of the amount of extra water that needs to be stored to offset evaporation losses. These losses can be reduced somewhat by reducing the affect of wind velocity over the surface of the pond with the use of windbreaks. Summer rainfall would also help to offset the affects of evaporation by helping to refill the ponds. Coastal areas in British Columbia tend to have cooler, wetter summers than the hotter, drier interior regions. As a result, a pond located in a coastal area would not need to store as much water as a pond located in a drier region. Coastal areas are, therefore, a better location choice for using ponds to augment stream flow.

While the ponds in the previous examples are not small, they remain within the size guidelines for off-channel pond habitat construction, which are between 0.1-0.3 ha (Lister and Finnigan, 1997). Also, while the cost of constructing the ponds would depend upon the particular site conditions, the construction of rearing and overwintering ponds of various sizes costs between \$2.50 and \$4.00 per m^2 (Lister and Finnigan, 1997) and it is assumed that the cost to construct seepage ponds would be similar. For the eight 24 m by 38 m ponds in the example calculations, the construction cost would range from \$18, 000 to \$29, 000. The ponds would require minimal, if any, maintenance and while the pay back period for such a project would depend upon the number and value of the fish in the stream, the return on investment should be at least equal to that of rearing and overwintering ponds, particularly on streams that are used by fish species at multiple life stages (e.g. rearing and spawning) which are of the greatest value, as are streams that provide habitat to endangered and threatened species.

As mentioned previously, other factors that would affect the amount of water reaching a stream but are not included in this study include deep seepage losses, evaporation from the pond surface, rainfall and plant uptake.

The example calculations indicate that a large amount of water needs to be stored in ponds in order to produce significant and lasting increases in stream flow. This in turn demonstrates how important are natural water storage features, such as ponds and wetlands, to a stream's hydraulic regime. However small the changes to a watershed may seem, their effect can be much more significant as their combined impacts accumulate over time to cause serious harm. The importance of water storing features such as ponds needs to be recognized and these features protected whenever possible.

6.0 Conclusion and Recommendations for Future Work

Water seeping from a pond was collected into a nearby channel during the dry summer season. Four different experiments were conducted to study the direction and magnitude of seepage flow under different conditions, and the scope of this work was limited to studying a location with a high groundwater table. From the collected data, it was determined that a combination of two hydraulic models, a wetland seepage model and a bank seepage model, could be used to accurately predict the experimental results. It was also shown that a liner could reduce the seepage rates and prolong the flow period.

These results were then applied to predict the physical conditions under which the seepage from a pond could contribute to a stream throughout the dry season. For pond water seepage to endure throughout the dry season while still contributing reasonable flow to a stream, the banks of the pond must be carefully lined and bottom seepage contained to a small area adjacent to the bank closest to the stream. Under these conditions, the wetland model could be used to calculate the dimensions of the pond needed to produce a given flow. The use of ponds requires detailed knowledge of the groundwater hydrology, soil profile and rainfall patterns of the area where the stream is located. Information about the stream's morphology, flow patterns, and the habitat requirements of affected aquatic life, are also vital. Coastal areas with summer rainstorms are the best location for using ponds as the climate conditions in these areas help to reduce and compensate for evaporation losses.

While significantly improved stream flow over a long term can be achieved with the use of multiple ponds, there are also two important benefits that can be realized from smaller projects:

1. to allow unrestricted fish movement over a short time period (less than one month).
2. to improve the quality of habitat available to fish over the summer.

Even small increases in stream flow can improve the quality of habitat, especially in slower moving pooled areas, as well as allow the fish to migrate to other parts of the stream where better habitat might be found. Both of these benefits could be especially important to streams that current flow augmentation methods cannot help. The research also helped to demonstrate the importance of water storing features such as ponds and wetland to a stream's hydraulic regime and the need to protect these features whenever possible.

Future research in this area should involve an examination of the saturated water zone and other site variables, such as different water table conditions, which are not covered in this work.

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Appendix 1

Soil Classification

Particle Size Analysis Site #1

Layer	Wt (g)		2mm	1.7mm	1.18mm	1mm	710µm	250µm	100µm	53µm	<53µm	Total	Boat Wt	Difference
A-Org	20.355	wt	0	3.084	3.672	3.312	4.52	11.144	7.246	3.91	5.857		2.815	
		wt-boat wt		0.269	0.857	0.497	1.705	8.329	4.431	1.095	3.042	20.225		0.13
											3.172			
		%		1.32	4.21	2.44	8.38	40.92	21.77	5.38	15.58			
	40.022	wt	0	3.689	5.199	3.817	5.977	18.217	11.895	5.741	7.445		2.815	
		wt-boat wt		0.874	2.384	1.002	3.162	15.402	9.08	2.926	4.63	39.46		0.562
											5.192			
		%		2.18	5.96	2.50	7.90	38.48	22.69	7.31	12.97			
		Avg	**0.00**	**1.75**	**5.08**	**2.47**	**8.14**	**39.70**	**22.23**	**6.35**	**14.28**	100		
A-S1	20.944	wt	0	2.839	3.039	2.907	3.34	11.354	12.724	3.241	3.633		2.815	
		wt-boat wt		0.024	0.224	0.092	0.525	8.539	9.909	0.426	0.818	20.557		0.387
											1.205			
		%		0.11	1.07	0.44	2.51	40.77	47.31	2.03	5.75			
	41.18	wt	0	2.903	3.069	3.077	3.885	21.065	21.117	3.509	4.165		2.815	
		wt-boat wt		0.088	0.254	0.262	1.07	18.25	18.302	0.694	1.35	40.27		0.91
											2.26			
		%		0.21	0.62	0.64	2.60	44.32	44.44	1.69	5.49			
		Avg	**0.00**	**0.16**	**0.84**	**0.54**	**2.55**	**42.54**	**45.88**	**1.86**	**5.62**	100		
A-S2	20.16	wt	0	2.947	2.914	2.964	3.058	6.216	18.216	3.267	3.04		2.815	
		wt-boat wt		0.132	0.099	0.149	0.243	3.401	15.401	0.452	0.225	20.102		0.058
											0.283			
		%		0.65	0.49	0.74	1.21	16.87	76.39	2.24	1.40			
	40.46	wt	0	3.121	3.054	2.987	3.097	10.652	32.589	4.025	3.313		2.815	
		wt-boat wt		0.306	0.239	0.172	0.282	7.837	29.774	1.21	0.498	40.318		0.142
											0.64			
		%		0.76	0.59	0.43	0.70	19.37	73.59	2.99	1.58			
		Avg	**0.00**	**0.71**	**0.54**	**0.58**	**0.95**	**18.12**	**74.99**	**2.62**	**1.49**	100		
A-Clay	20.71	wt	0	2.861	3.054	2.941	3.376	7.987	6.246	5.897	10.844		2.815	
		wt-boat wt		0.046	0.239	0.126	0.561	5.172	3.431	3.082	8.029	20.686		0.024
											8.053			
		%		0.22	1.15	0.61	2.71	24.97	16.57	14.88	38.88			
	40.95	wt	0	2.854	2.989	3.1	4.103	12.945	9.5697	8.347	18.574		2.815	
		wt-boat wt		0.039	0.174	0.285	1.288	10.13	6.7547	5.532	15.759	39.9617		0.9883
											16.7473			
		%		0.10	0.42	0.70	3.15	24.74	16.49	13.51	40.90			
		Avg	**0.00**	**0.16**	**0.79**	**0.65**	**2.93**	**24.86**	**16.53**	**14.20**	**39.89**	100		

note: any difference between initial sample weight and total weight from the sieves is added to the value for the <53 µm as it was assumed the lost material was fine dust

Particle Size Analysis Site #2

Layer	Wt (g)		2mm	1.7mm	1.18mm	1mm	710µm	250µm	100µm	53µm	<53µm	Total	Boat Wt	Difference
B-Org	20.289	wt	0	3.063	3.367	3.147	4.06	11.608	7.809	3.714	5.443		2.815	
		wt-boat wt		0.248	0.552	0.332	1.245	8.793	4.994	0.899	2.628	19.691		0.598
											3.226			
		%		1.22	2.72	1.64	6.14	43.34	24.61	4.43	15.90			
	40.992	wt	0	3.259	3.563	3.351	4.641	19.545	15.277	4.651	9.093		2.815	
		wt-boat wt		0.444	0.748	0.536	1.826	16.73	12.462	1.836	6.278	40.86		0.132
											6.41			
		%		1.08	1.82	1.31	4.45	40.81	30.40	4.48	15.64			
		Avg	**0.00**	**1.15**	**2.27**	**1.47**	**5.30**	**42.08**	**27.51**	**4.45**	**15.77**	100		
B-S1	20.48	wt	0	0	2.96	2.913	3.294	12.6	11.602	3.093	3.504		2.815	
		wt-boat wt			0.145	0.098	0.479	9.785	8.787	0.278	0.689	20.261		0.219
											0.908			
		%			0.71	0.48	2.34	47.78	42.91	1.36	4.43			
	40.43	wt	0	0	3.088	3.044	3.826	23.103	19.253	3.322	3.96		2.815	
		wt-boat wt			0.273	0.229	1.011	20.288	16.438	0.507	1.145	39.891		0.539
											1.684			
		%			0.68	0.57	2.50	50.18	40.66	1.25	4.17			
		Avg	**0.00**	**0.00**	**0.69**	**0.52**	**2.42**	**48.98**	**41.78**	**1.31**	**4.30**	100		
B-S2	20.897	wt	0	2.982	2.88	2.872	2.987	6.153	19.052	3.302	3.089		2.815	
		wt-boat wt		0.167	0.065	0.057	0.172	3.338	16.237	0.487	0.274	20.797		0.1
											0.374			
		%		0.80	0.31	0.27	0.82	15.97	77.70	2.33	1.79			
	40.15	wt	0	3.071	2.983	2.901	3.143	9.304	33.739	3.727	3.334		2.815	
		wt-boat wt		0.256	0.168	0.086	0.328	6.489	30.924	0.912	0.519	39.682		0.468
											0.987			
		%		0.64	0.42	0.21	0.82	16.16	77.02	2.27	2.46			
		Avg	**0.00**	**0.72**	**0.36**	**0.24**	**0.82**	**16.07**	**77.36**	**2.30**	**2.12**	100		
B-Clay	20.44	wt	0	0	3.008	2.865	3.17	8.549	6.547	4.879	11.014		2.815	
		wt-boat wt			0.193	0.05	0.355	5.734	3.732	2.064	8.199	20.327		0.113
											8.312			
		%			0.94	0.24	1.74	28.05	18.26	10.10	40.67			
	40.28	wt	0	0	3.156	2.924	3.809	12.986	10.036	7.688	19.268		2.815	
		wt-boat wt			0.341	0.109	0.994	10.171	7.221	4.873	16.453	40.162		0.118
											16.571			
		%			0.85	0.27	2.47	25.25	17.93	12.10	41.14			
		Avg	**0.00**	**0.00**	**0.90**	**0.26**	**2.10**	**26.65**	**18.09**	**11.10**	**40.90**	100		

note: any difference between initial sample weight and total weight from the sieves is added to the value for the <53 µm as it was assumed the lost material was fine dust

Sieve Distribution Summary

	2mm	1.7mm	1.18mm	1mm	710μm	250μm	100μm	53μm	<53μm
A-Org %	0	1.75	5.08	2.47	8.14	39.70	22.23	6.35	14.28
B-Org %	0	1.15	2.27	1.47	5.30	42.08	27.51	4.45	15.77
A-S1 %	0	0.16	0.84	0.54	2.55	42.54	45.88	1.86	5.62
B-S1 %	0	0	0.69	0.52	2.42	48.98	41.78	1.31	4.30
A-S2 %	0	0.71	0.54	0.58	0.95	18.12	74.99	2.62	1.49
B-S2 %	0	0.72	0.36	0.24	0.82	16.07	77.36	2.30	2.12
A-Clay %	0	0.16	0.79	0.65	2.93	24.86	16.53	14.20	39.89
B-Clay %	0	0	0.90	0.26	2.10	26.65	18.09	11.10	40.90

Particle Size Limit Classification According to the Canada Soil Survey Committee (Gee and Bauder, 1986)

	Very Coarse Sand 2 - 1.0 mm	Coarse Sand 1 - 0.5 mm	Med Sand 0.5 - 0.25 mm	Fine Sand 0.25 - 0.1 mm	Very Fine Sand 0.1 - 0.053 mm	Silt & Clay < 0.053 mm
A-Org %	9.31	8.14	39.70	22.23	6.35	14.28
B-Org %	4.90	5.30	42.08	27.51	4.45	15.77
A-S1 %	1.55	2.55	42.54	45.88	1.86	5.62
B-S1 %	1.21	2.42	48.98	41.78	1.31	4.30
A-S2 %	1.83	0.95	18.12	74.99	2.62	1.49
B-S2 %	1.33	0.82	16.07	77.36	2.30	2.12
A-Clay %	1.60	2.93	24.86	16.53	14.20	39.89
B-Clay %	1.15	2.10	26.65	18.09	11.10	40.90

With a sand percentage of 60% and a clay & silt percentage of 40%, the bottom layer of the soil profile could either be a sandy loam, a sandy clay loam or a sandy clay. To try and determine the textural class of the sample, the following test was done on two different samples (A&B) from the bottom layer.

Criteria Used with the Field Method of Determining Soil Textural Classes (Brady, 1990)

Criterion	Sand	Sandy Loam	Loam	Silt Loam	Clay Loam	Clay
1. Individual grains visible to eye	Yes	Yes	Some	Few	No	No
2. Stability of dry clods	Do not form	Do not form	Easily broken	Moderately easily broken	Hard and stable	Very hard and stable
3. Stability of wet clods	Unstable	Slightly stable	Moderately stable	Stable	Very stable	Very stable
4. Stability of ribbon when wet soil rubbed between thumb and fingers	Does not form	Does not form	Does not form	Broken appearance	Thin, will break	Very long and flexible

Results for samples A and B:

Criteria	Answer	Soil Textural Class
1	No	Clay Loam/Clay
2	Hard and stable	Clay Loam
3	Very stable	Clay Loam/Clay
4	Very long, flexible	Clay

While this first test ruled out the sandy loam class, it did not distinguish if the soil was a clay loam or a clay. As a result, further tests were done.

Dry Crushing Test (Coche 1985)

Step	Instructions
1.	Take a small sample of dry soil in your hand.
2.	Crush it between your fingers.
3.	If there is little resistance and the sample falls into dust, it is fine sand or fine loamy sand and there is very little clay present.
4.	If there is medium resistance, it is silty clay or sandy clay.
5.	If there is great resistance, it is clay.

Result for samples A and B: Sandy Clay

Manipulative Test (Coche, 1985)

Step	**Instructions**
1.	Take a handful of soil and wet it so that it begins to stick together but without sticking to your hand.
2.	Roll the soil sample into a ball about 3 cm in diameter and put it down. If the ball falls apart, it is sand. If it sticks together, go on to the next step.
3.	Roll the ball into a sausage shape, 6-7cm long. If it does not remain in this form, it is loamy sand. If it remains in this shape go on to the next step.
4	Continue to roll out the sausage until it reaches 15 – 16 cm long. If it does not remain in this shape it is sandy loam. If it remains in this shape, go on to the next step.
5	Try to bend the sausage into a half circle. If you cannot, it is loam. If you can, go on to the next step.
6.	Continue to bend the sausage to form a full circle. If you cannot, it is heavy loam. If you can, with slight cracks in the sausage, it is light clay. If you can, with not cracks in the sausage, it is clay.

Result for samples A and B: Light Clay

Overall Result: Sandy Clay

Appendix 2

Hydraulic Conductivity

Inverse Auger Hole Tests - to determine an overall k value for the soil profile
Hole #1
Test #1

h1 (m)	h2 (m)	delta h	h/t	t1 (s)	t2 (s)	r (m)	k (m/s)
1.016	0.99695	0.01905	0.00032	0	60	0.2286	3.2344E-05
0.99695	0.9779	0.01905	0.00032	60	120	0.2286	3.29033E-05
0.9779	0.9652	0.0127	0.00021	120	180	0.2286	2.22559E-05
0.9652	0.9525	0.0127	0.00021	180	240	0.2286	2.25193E-05
0.9525	0.94615	0.00635	0.00011	240	300	0.2286	1.13604E-05
0.94615	0.9398	0.00635	0.00011	300	360	0.2286	1.14286E-05
0.9398	0.90805	0.03175	0.00018	360	540	0.2286	1.93987E-05
0.90805	0.885825	0.022225	0.00019	540	660	0.2286	2.09114E-05
0.885825	0.8636	0.022225	0.00012	660	840	0.2286	1.42542E-05
0.8636	0.8509	0.0127	0.00021	840	900	0.2286	2.48744E-05
0.8509	0.8382	0.0127	0.00021	900	960	0.2286	2.52038E-05
0.8382	0.8255	0.0127	0.00007	960	1140	0.2286	2.14230E-05
0.822325	0.8128	0.009525	0.00016	1140	1200	0.2286	1.94502E-05
0.8128	0.802	0.0108	0.00018	1200	1260	0.2286	2.2297E-05
0.802	0.7916	0.0104	0.00017	1260	1320	0.2286	2.1721E-05
0.7916	0.7812	0.0104	0.00017	1320	1380	0.2286	2.19718E-05
0.7812	0.7708	0.0104	0.00017	1380	1440	0.2286	2.22284E-05
0.7708	0.7604	0.0104	0.00017	1440	1500	0.2286	2.24912E-05
0.7604	0.75	0.0104	0.00017	1500	1560	0.2286	2.27602E-05
0.75	0.7396	0.0104	0.00017	1560	1620	0.2286	2.30357E-05
0.7396	0.7292	0.0104	0.00017	1620	1680	0.2286	2.3318E-05
0.7292	0.7188	0.0104	0.00017	1680	1740	0.2286	2.36073E-05
0.7188	0.7084	0.0104	0.00017	1740	1800	0.2286	2.39039E-05
						Avg:	2.24E-05

Sample Calculations:

$$k = \frac{1.15r}{t_2 - t_1}\log\frac{y_1 + \frac{1}{2}r}{y_2 + \frac{1}{2}r}$$

$$k = \frac{1.15 * 0.2286}{1800 - 1740}\log\frac{0.7188 + 0.1143}{0.7084 + 0.1143}$$

$$k = 2.39E - 05$$

The average k value is calculated from k values with the constant rate (h/t)

The constant rate for test #1 is 0.00017 and starts at time t_1=1260 and runs until t_1=1740

$$k_{avg} = \frac{k_{t_1=1260} + k_{t_1=1320} + \ldots + k_{t_1=1740}}{9}$$

$$k_{avg} = 2.24E - 05$$

Inverse Auger Hole Method
Hole #1
Test #2

h1 (m)	h2 (m)	delta h	h/t	t1 (s)	t2 (s)	r (m)	k (m/s)
1.016	0.9906	0.0254	0.000423	0	60	0.2286	4.32487E-05
0.9906	0.97155	0.01905	0.00032	60	120	0.2286	3.30941E-05
0.97155	0.95885	0.0127	0.00021	120	180	0.2286	2.23869E-05
0.95885	0.942975	0.015875	0.00026	180	240	0.2286	2.83591E-05
0.942975	0.93345	0.009525	0.00016	240	300	0.2286	1.72206E-05
0.93345	0.92075	0.0127	0.00021	300	360	0.2286	2.32059E-05
0.92075	0.90805	0.0127	0.00021	360	420	0.2286	2.34924E-05
0.90805	0.89535	0.0127	0.00021	420	480	0.2286	2.37861E-05
0.89535	0.879475	0.015875	0.00026	480	540	0.2286	3.01569E-05
0.879475	0.873125	0.00635	0.00011	540	600	0.2286	1.21979E-05
0.873125	0.85725	0.015875	0.00026	600	660	0.2286	3.08412E-05
0.85725	0.84455	0.0127	0.00021	660	720	0.2286	2.5038E-05
0.84455	0.83185	0.0127	0.00021	720	780	0.2286	2.53719E-05
0.83185	0.81915	0.0127	0.00021	780	840	0.2286	2.57147E-05
0.81915	0.80645	0.0127	0.00021	840	900	0.2286	2.6067E-05
0.80645	0.79375	0.0127	0.00021	900	960	0.2286	2.64291E-05
0.79375	0.78105	0.0127	0.00021	960	1020	0.2286	2.68013E-05
0.78105	0.762	0.01905	0.00032	1020	1080	0.2286	2.57147E-05
0.762	0.7493	0.0127	0.00021	1080	1140	0.2286	2.77795E-05
0.7493	0.7366	0.0127	0.00021	1140	1200	0.2286	2.81911E-05
0.7366	0.7239	0.0127	0.00021	1200	1260	0.2286	2.8615E-05
						Avg:	2.45E-05

Inverse Auger Hole Method
Hole # 2
Test #1

h1 (m)	h2 (m)	delta h	h/t	t1 (s)	t2 (s)	r (m)	k (m/s)
1.0668	1.0287	0.0381	0.00064	0	60	0.2286	6.23945E-05
1.0287	1.00965	0.01905	0.00032	60	120	0.2286	3.19816E-05
1.00965	0.9906	0.01905	0.00032	120	180	0.2286	3.25283E-05
0.9906	0.9652	0.0254	0.00042	180	240	0.2286	4.42546E-05
0.9652	0.94615	0.01905	0.00032	240	300	0.2286	3.38797E-05
0.94615	0.9271	0.01905	0.00032	300	360	0.2286	3.44939E-05
0.9271	0.88265	0.04445	0.00037	360	480	0.2286	4.1502E-05
0.88265	0.8509	0.03175	0.00026	480	600	0.2286	3.07933E-05
0.8509	0.84	0.0109	0.00018	600	660	0.2286	2.16113E-05
0.84	0.828	0.012	0.00020	660	720	0.2286	2.40796E-05
0.828	0.804	0.024	0.00020	720	840	0.2286	2.45465E-05
0.804	0.791	0.013	0.00022	840	900	0.2286	2.71305E-05
0.791	0.766	0.025	0.00021	900	1020	0.2286	2.66435E-05
0.766	0.7305	0.0355	0.00020	1020	1200	0.2286	2.61091E-05
0.7305	0.705	0.0255	0.00021	1200	1320	0.2286	2.9161E-05
0.705	0.682	0.023	0.00019	1320	1440	0.2286	2.70913E-05
						Avg:	2.58E-05

Inverse Auger Hole Method
Hole #2
Test #2

h1 (m)	h2 (m)	delta h	h/t	t1 (s)	t2 (s)	r (m)	k (m/s)
1.0668	1.0414	0.0254	0.00042	0	60	0.2286	4.13682E-05
1.0414	1.016	0.0254	0.00042	60	120	0.2286	4.22875E-05
1.016	1.0033	0.0127	0.00021	120	180	0.2286	2.15015E-05
1.0033	0.9906	0.0127	0.00021	180	240	0.2286	2.17472E-05
0.9906	0.9779	0.0127	0.00021	240	300	0.2286	2.19986E-05
0.9779	0.9652	0.0127	0.00021	300	360	0.2286	2.22559E-05
0.9652	0.9525	0.0127	0.00021	360	420	0.2286	2.25193E-05
0.9525	0.9398	0.0127	0.00021	420	480	0.2286	2.2789E-05
0.9398	0.9017	0.0381	0.00021	480	660	0.2286	2.33506E-05
0.9017	0.889	0.0127	0.00021	660	720	0.2286	2.39357E-05
0.889	0.8763	0.0127	0.00021	720	780	0.2286	2.42406E-05
0.8763	0.8636	0.0127	0.00021	780	840	0.2286	2.45534E-05
0.8636	0.8509	0.0127	0.00021	840	900	0.2286	2.48744E-05
0.8509	0.8382	0.0127	0.00021	900	960	0.2286	2.52038E-05
0.8382	0.8255	0.0127	0.00021	960	1020	0.2286	2.55421E-05
0.8255	0.8128	0.0127	0.00021	1020	1080	0.2286	2.58897E-05
0.8128	0.7994	0.0134	0.00022	1080	1140	0.2286	2.7704E-05
0.7994	0.7765	0.0229	0.00019	1140	1260	0.2286	2.41495E-05
0.7765	0.7356	0.0409	0.00023	1260	1440	0.2286	2.98123E-05
0.7356	0.7233	0.0123	0.00021	1440	1500	0.2286	2.774E-05
0.7233	0.7112	0.0121	0.00020	1500	1560	0.2286	2.76893E-05
						Avg:	2.36E-05

$$Overall Average = \frac{2.24E^{-05} + 2.45E^{-05} + 2.58E^{-05} + 2.36E^{-05}}{4}$$

$$= 2.41E - 05$$

Hazen Method - to determine k values for the sand layers

Sample Calculation:

$k = C^*(D_{10\%})^2$

$k = 700^*(.101)^2$

$k = 7.14$

A-S1 see Chart 1

sieve (μm)	% finer than	$D_{10\%}$ (μm)	C	k (m/day)
2000	100.00	101	700	7.14
1700	99.84			
1180	98.99			
1000	98.45			
710	95.90			
250	53.36			
100	7.48			
53	5.62			

B-S1 see Chart 2

sieve (μm)	% finer than	$D_{10\%}$ (μm)	C	k (m/day)
2000	100.00	101	700	7.14
1700	100.00			
1180	99.31			
1000	98.79			
710	96.37			
250	47.39			
100	5.61			
53	4.30			

A-S2 see Chart 3

sieve (μm)	% finer than	$D_{10\%}$ (μm)	C	k (m/day)
2000	100.00	101	525	5.36
1700	99.29			
1180	98.75			
1000	98.17			
710	97.22			
250	79.10			
100	4.11			
53	1.49			

B-S2 see Chart 4

sieve (μm)	% finer than	$D_{10\%}$ (μm)	C	k (m/day)
2000	100.00	101	525	5.36
1700	99.28			
1180	98.92			
1000	98.67			
710	97.85			
250	81.79			
100	4.42			
53	2.12			

Chart 1: Particle Size Distribution Curve for A-S1

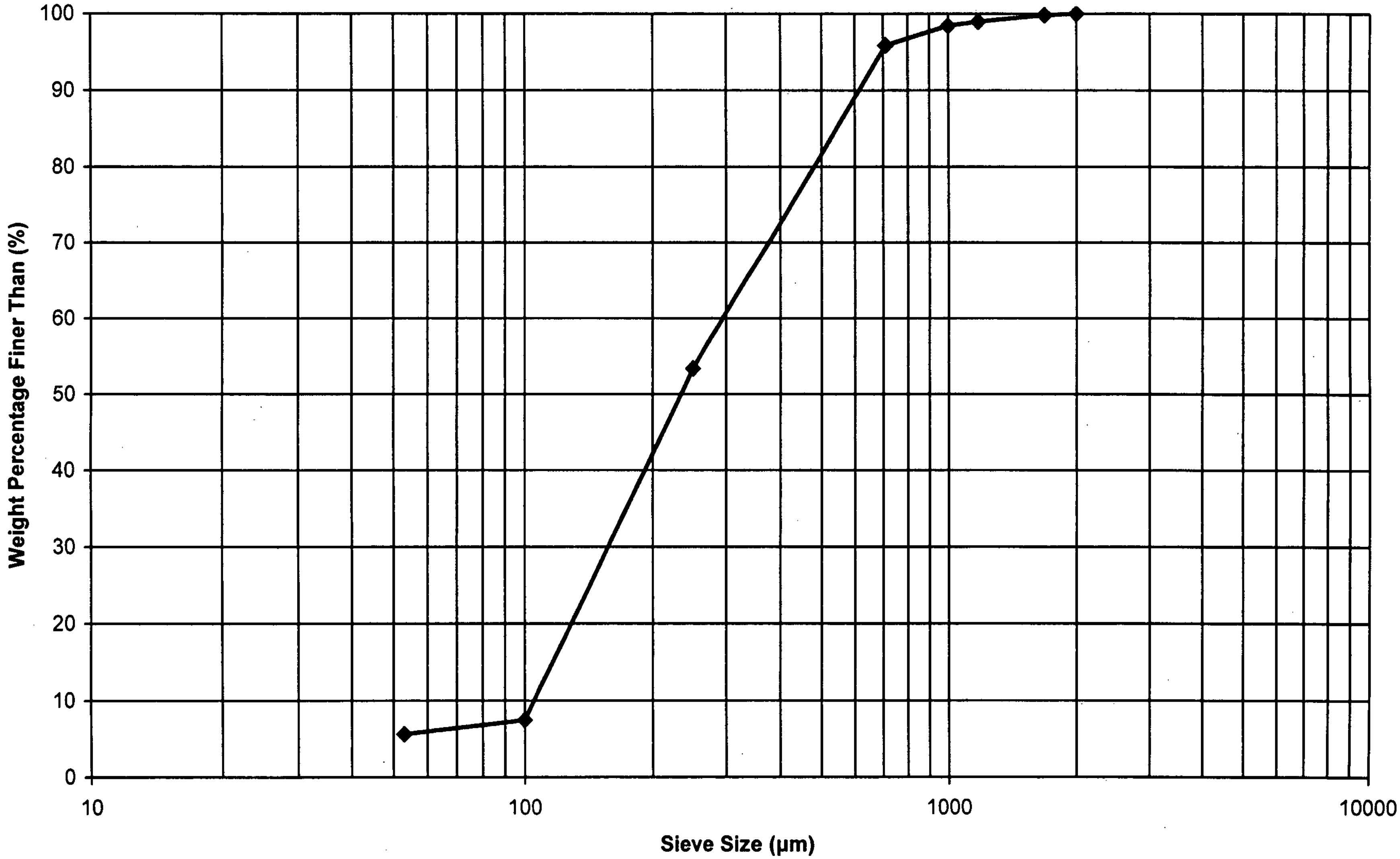


Chart 2: Particle Size Distribution Curve For B-S1

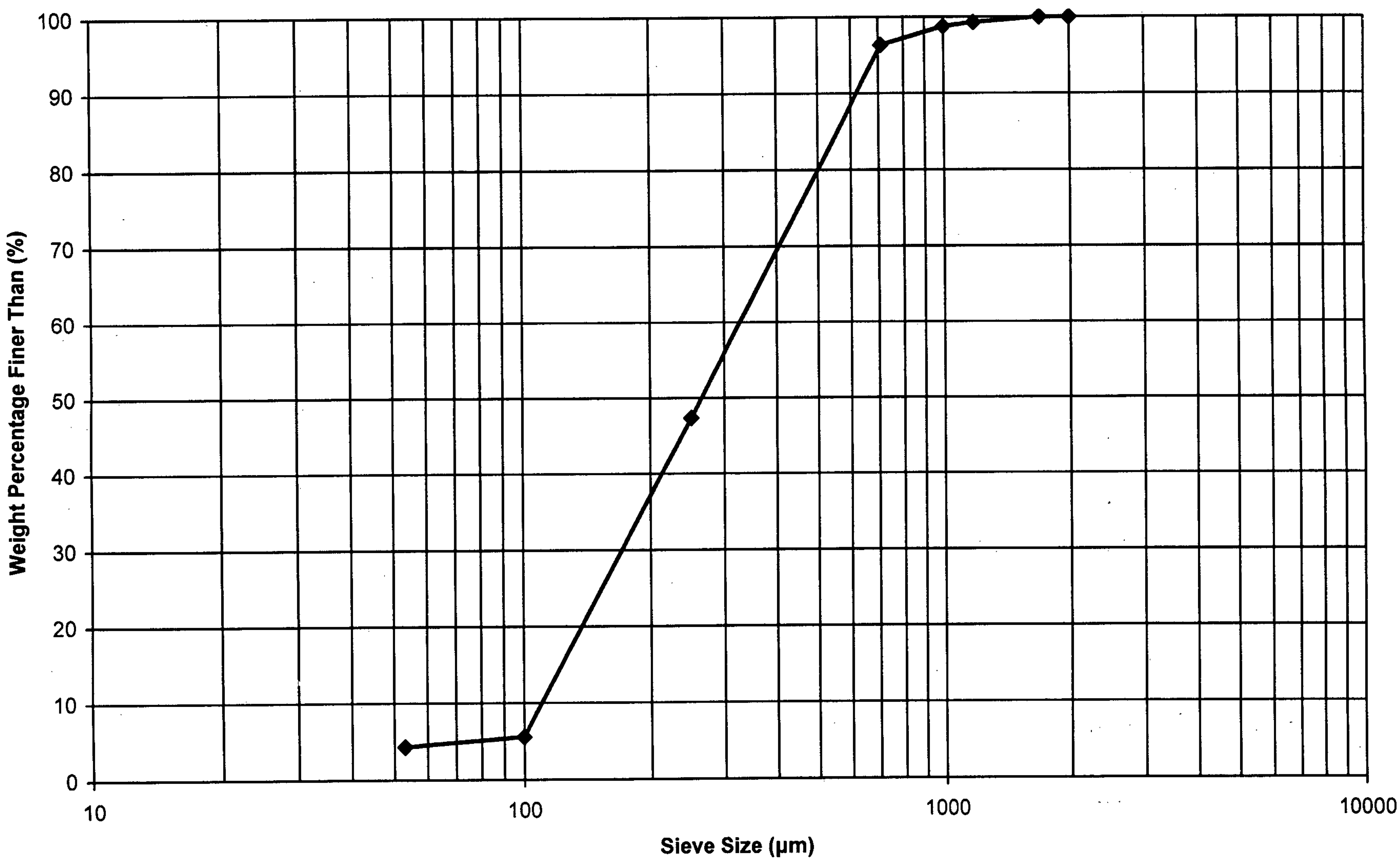


Chart 3: Particle Size Distribution Curve for A-S2

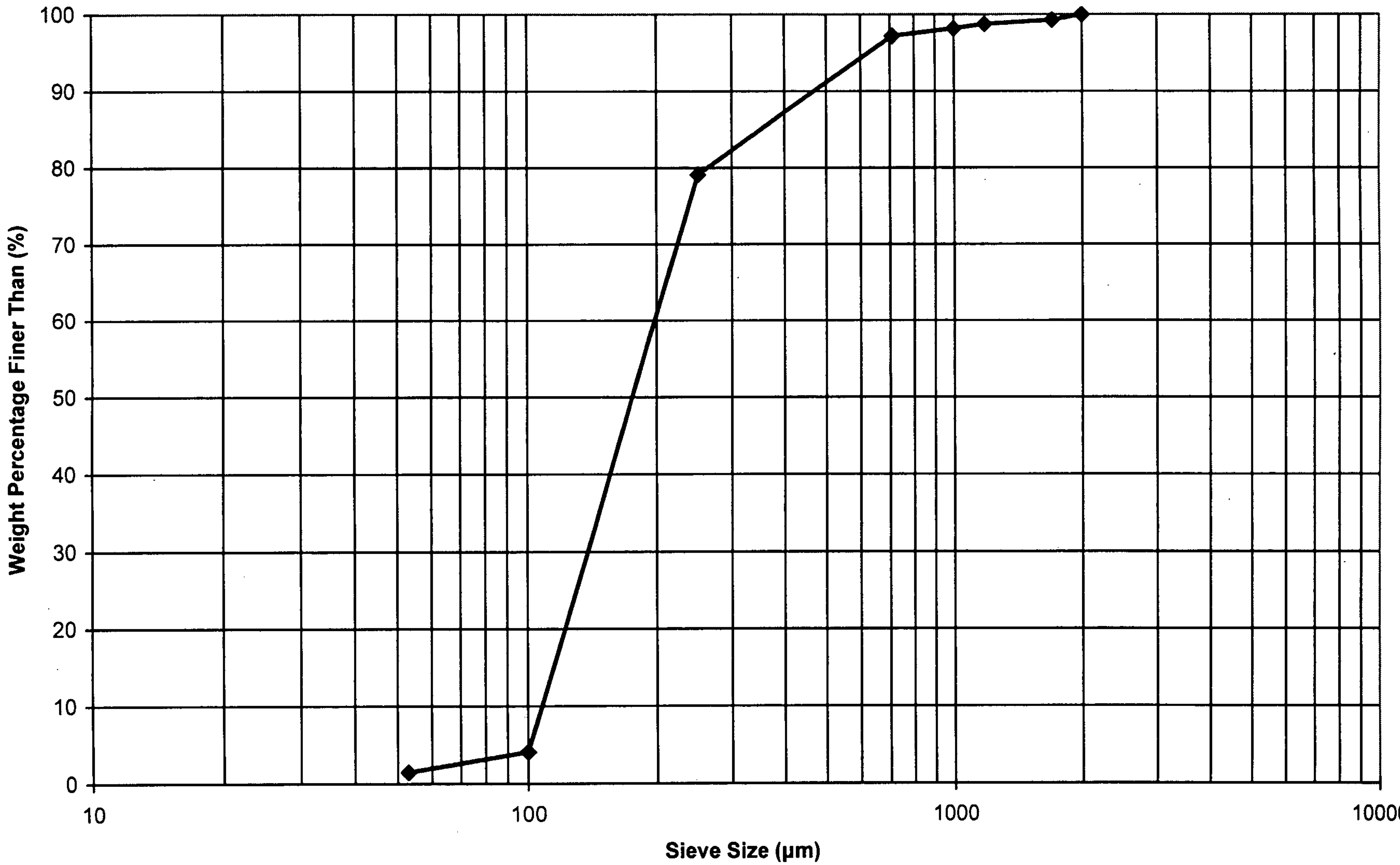
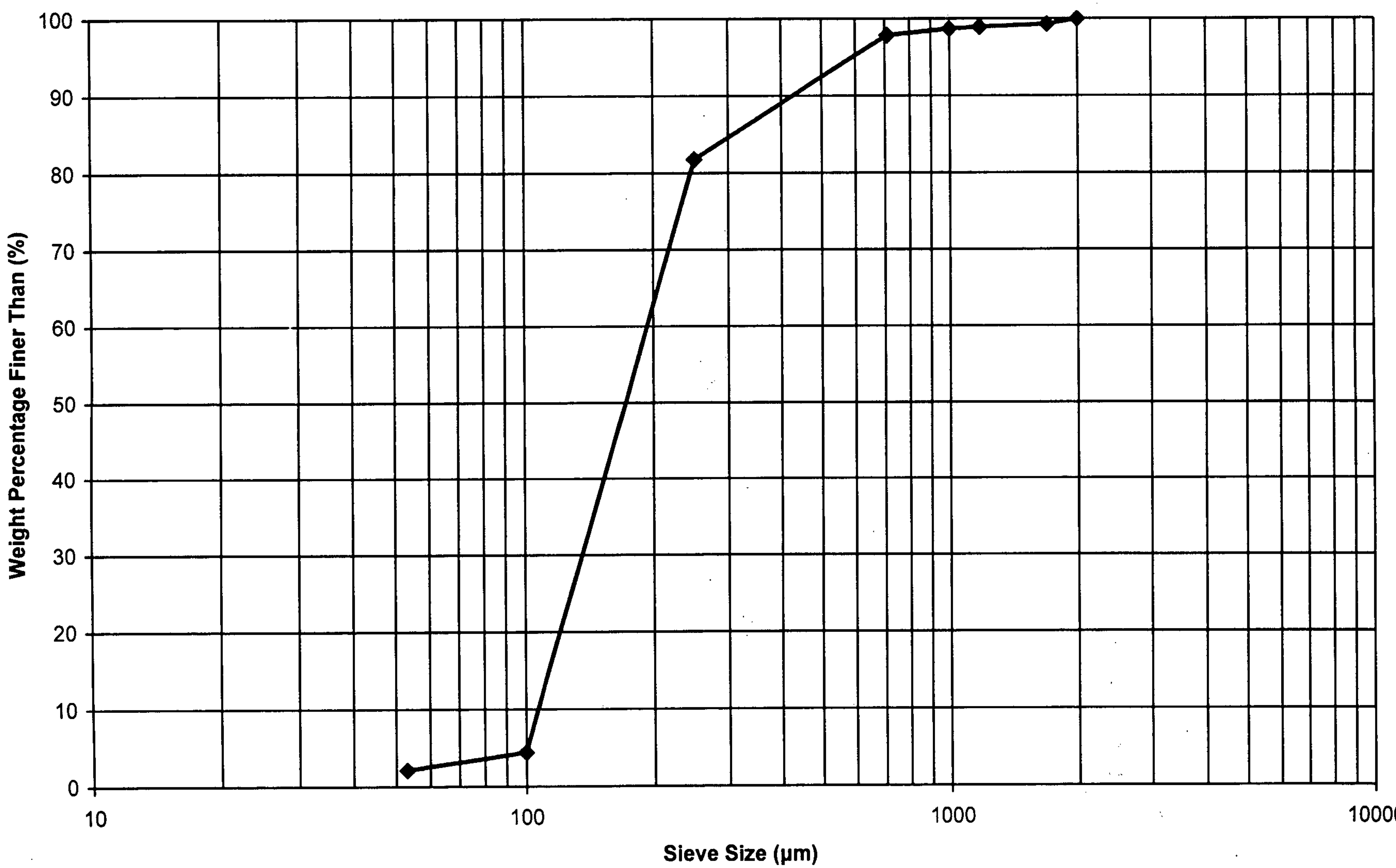


Chart 4: Particle Size Distriubtion Curve for B-S2



Ditch Face Depths for A, B and C and the Calculation of Average k for the soil profile using selected k values for each layer

Ditch A

layer	Depth 1	Depth 2	Depth 3	Avg (cm)	chosen k	kv	Kv	kh	Kh	Kav (m/day)	K avg (m/s)
org	26	26	25	25.7	3.26	0.078732	0.530355	0.836733	8.158707	2.08014708	2.41E-05
sand1	46	40	43	43.0	15	0.028667		6.45			
hp	9	10	12	10.3	0.075	1.377778		0.00775			
sand2	19	17	18	18.0	5.6	0.032143		1.008			
clay	4	5	5.5	4.8	0.12	0.402778		0.0058			
total	104	98	103.5	101.8							

Ditch B

layer	Depth 1	Depth 2	Depth 3	Avg (cm)	chosen k	kv	Kv	kh	Kh	Kav (m/day)	K avg (m/s)
org	28	29	25	27.3	3.26	0.083845	0.53144	0.891067	7.967523	2.05773231	2.38E-05
sand1	41	42	49	44.0	15	0.029333		6.6			
hp	10	11	12	11.0	0.075	1.466667		0.00825			
sand2	24	20	22	22.0	5.6	0.039286		1.232			
clay	5	5	6	5.3	0.12	0.444444		0.0064			
total	108	107	114	109.7							

Ditch C

layer	Depth 1	Depth 2	Depth 3	Avg (cm)	chosen k	kv	Kv	kh	Kh	Kav (m/day)	K avg (m/s)
org	29	29	23	27.0	3.26	0.082822	0.557937	0.8802	7.884942	2.09745089	2.43E-05
sand1	40	38	37	38.3	15	0.025556		5.75			
hp	10	16	10	12.0	0.075	1.6		0.009			
sand2	22	24	20	22.0	5.6	0.039286		1.232			
clay	0	1	0.5	0.5	0.12	0.041667		0.0006			
total	101	108	91	99.8							

The profile of Ditch D was disturbed as it was an existing drainage ditch so was not used

** k values for each layer (k chosen) were chosen from Table 5 (range of values from literature) and the results from the Hazen method (for sand layers) so that the calculated Kavg values would be in the range found from the inverse auger hole method**

Appendix 3

Rate of Rise Well Tests

Rate-of-Rise Well Tests - to study Groundwater Movement

Test #1

time(min)	ht(m)	delta ht	ht below start
0	3.81		0.3302
15	3.8354	0.0254	0.3048
30	3.85445	0.01905	0.28575
45	3.8735	0.01905	0.2667
60	3.89255	0.01905	0.24765
75	3.9116	0.01905	0.2286
90	3.9243	0.0127	0.2159
105	3.937	0.0127	0.2032
135	3.9497	0.0127	0.1905
150	3.9624	0.0127	0.1778
165	3.9751	0.0127	0.1651
180	3.98145	0.00635	0.15875
195	3.9878	0.00635	0.1524
210	3.99415	0.00635	0.14605
225	4.0005	0.00635	0.1397
240	4.00685	0.00635	0.13335
255	4.0132	0.00635	0.127
270	4.01955	0.00635	0.12065
285	4.0259	0.00635	0.1143
300	4.029075	0.003175	0.111125
315	4.03225	0.003175	0.10795
330	4.035425	0.003175	0.104775
345	4.0386	0.003175	0.1016
1081	4.1148	0.0762	0.0254
1501	4.13385	0.01905	0.00635

Test #2

time (min)	ht (m)	delta ht	ht below start
0	3.883		0.311
15	3.908	0.044	0.286
30	3.927	0.015	0.267
45	3.942	0.014	0.252
75	3.956	0.026	0.238
90	3.982	0.009	0.212
105	3.991	0.009	0.203
120	4.00	0.009	0.194
135	4.009	0.009	0.185
150	4.018	0.007	0.176
165	4.025	0.005	0.169
180	4.03	0.005	0.164
195	4.035	0.005	0.159
210	4.04	0.006	0.154
225	4.046	0.006	0.148
240	4.052	0.005	0.142
255	4.057	0.003	0.137
270	4.06	0.005	0.134
285	4.065	0.004	0.129
300	4.069	0.003	0.125
315	4.072	0.004	0.122
330	4.076	0.003	0.118
345	4.079	0.003	0.115
360	4.082	0.003	0.112
375	4.085	0.003	0.109
390	4.088	0.002	0.106
405	4.09	0.003	0.104
420	4.093	0.002	0.101
435	4.095	0.003	0.099
450	4.098	0.003	0.096
465	4.101	0.004	0.093
480	4.105	0.002	0.089
495	4.107	0.002	0.089
510	4.108	0.001	0.087
525	4.11	0.002	0.086
1160	4.15	0.04	0.084
2780	4.179	0.029	0.044

Well size

length: 1.5 m
width: 1.4 m
depth: 5.7 m

Chart 5: Rate of Rise Well Test #1

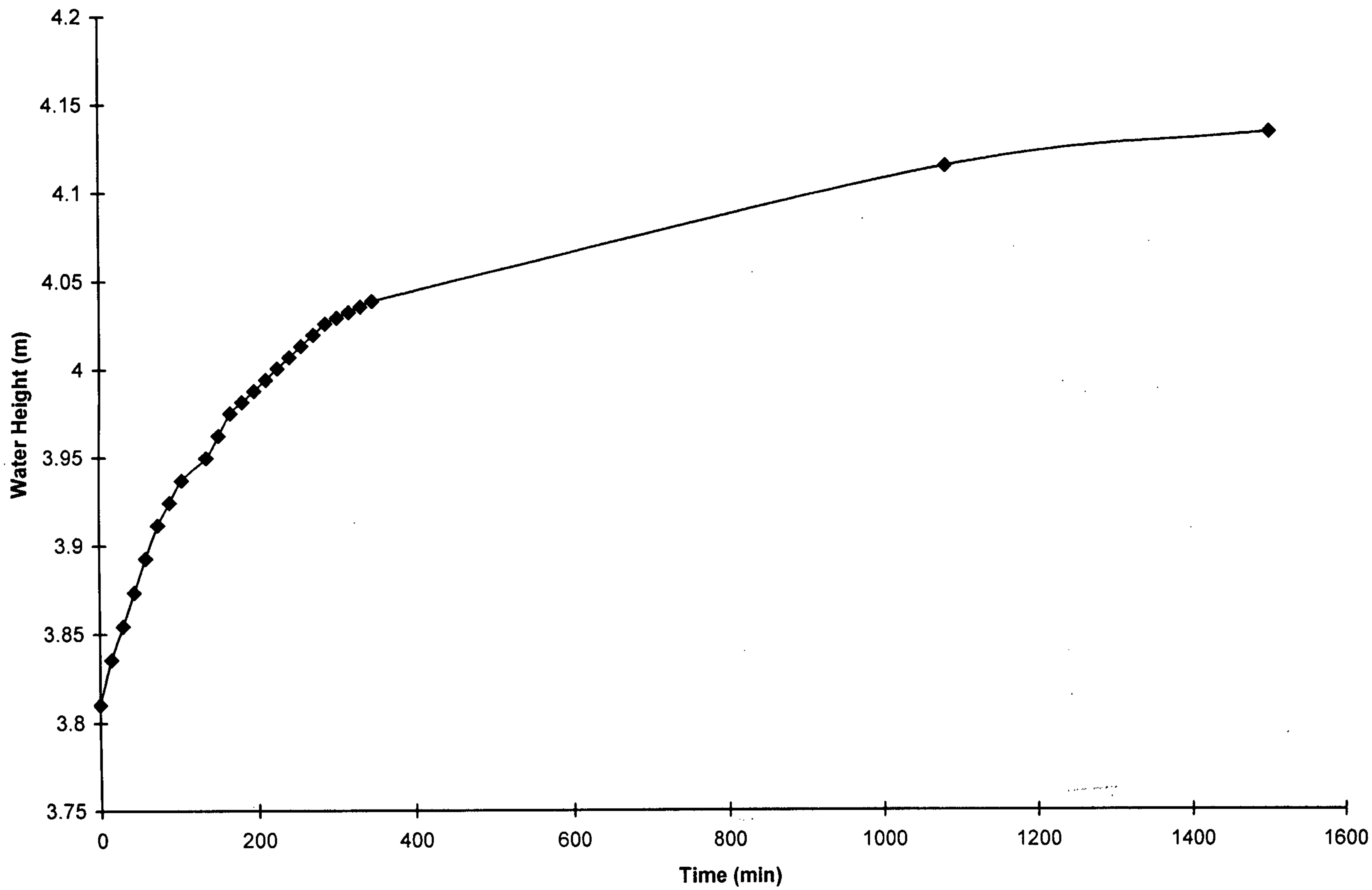
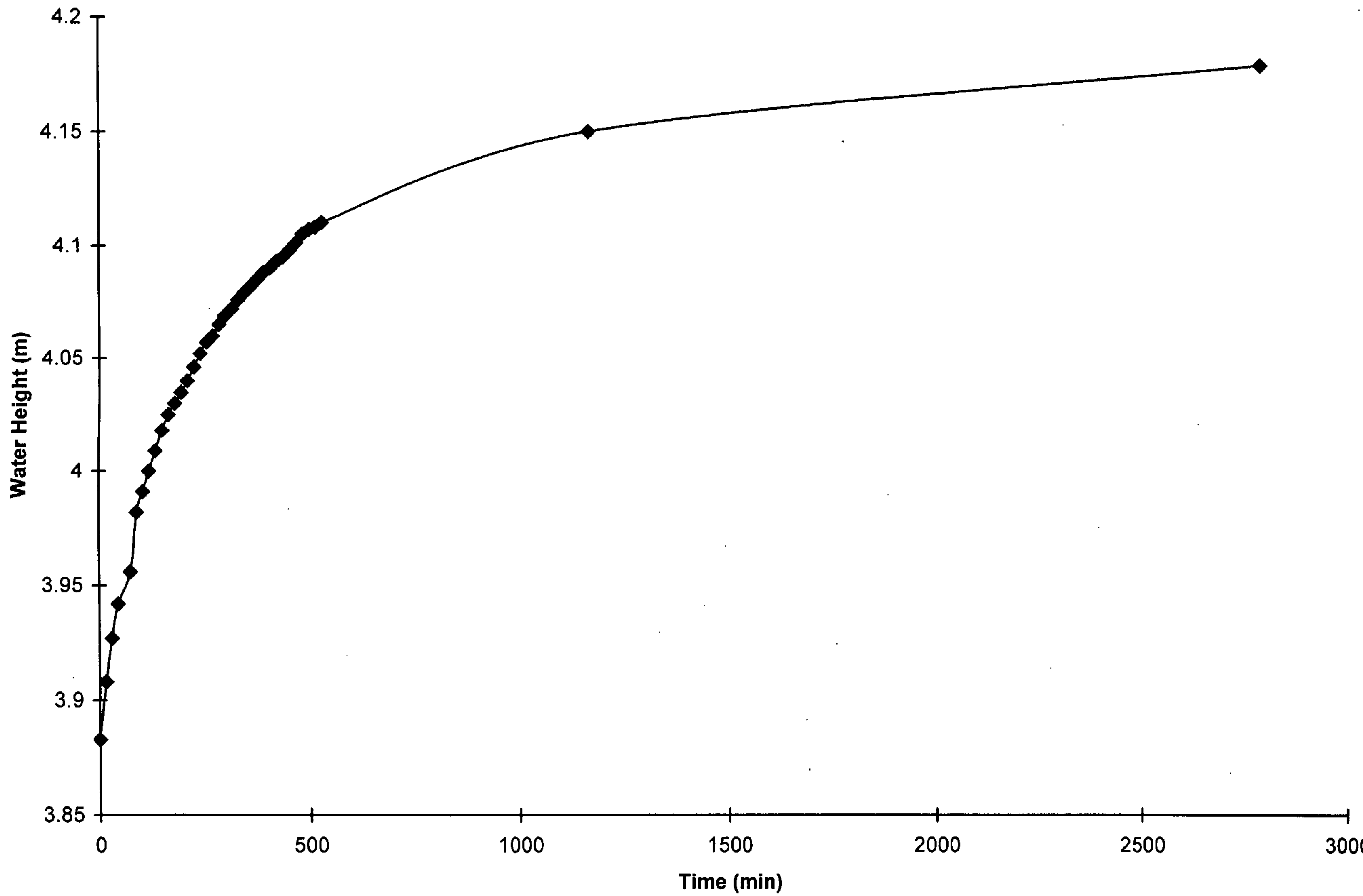


Chart 6: Rate of Rise Well Test #2



Calculation of Aquifer k from Rate-of-Rise Tests

Test #1 k calculation

initial h (m)	h_0 (m)	total drop (m)	90% drop (m)	h 90% (m)	(from chart #5) t 90% (min)
4.171	3.81	0.361	0.3249	4.135	1240
Le (m)	rw (m)	ln(Re/rw)	rc (m)	k (m/s)	k (m/day)
4.914	1.0287	1.050996	0.7112	7.66E-08	6.62E-03

Test #2 k calculation

initial h (m)	h_0 (m)	total drop (m)	90% drop (m)	h 90% (m)	(from chart #6) t 90% (min)
4.194	3.883	0.311	0.2799	4.163	1500
Le (m)	rw (m)	ln(Re/rw)	rc (m)	k (m/s)	k (m/day)
4.914	1.0287	1.050996	0.7112	6.33E-08	5.47E-03

Sample Calculations

$$totaldrop = initialh - h_0$$

$$totaldrop = 4.194 - 3.883 = 0.311$$

$$h_{90\%} = h_0 + 0.9 totaldrop$$

$$h_{90\%} = 3.833 + 0.9(0.311) = 4.163$$

$$k = 0.0527 \frac{r_c^2}{t_{90\%} L_e} \ln \frac{R_e}{r_w}$$

$$k = 0.0527 \frac{(0.7112)^2}{(1500 \times 60)4.914}(1.051)$$

$$k = 6.33E-08$$

Appendix 4

Pond Volume

Calculation of Pond Surface Area for Volume Calculations
43.5 cm stage

Non Uniform Width

interval d (ft)	width (m)	res width	delta d (m)	avg w	area (m^2)
0	3.6	0.12			
1	4	0.52	0.3048	0.32	0.097536
2	4.2	0.72	0.3048	0.62	0.188976
3	4.34	0.86	0.3048	0.79	0.240792
4	4.27	0.79	0.3048	0.825	0.25146
5	4.34	0.86	0.3048	0.825	0.25146
6	4.36	0.88	0.3048	0.87	0.265176
7	4.38	0.9	0.3048	0.89	0.271272
8	4.35	0.87	0.3048	0.885	0.269748
9	4.34	0.86	0.3048	0.865	0.263652
10	4.32	0.84	0.3048	0.85	0.25908
11	4.3	0.82	0.3048	0.83	0.252984
12	4.25	0.77	0.3048	0.795	0.242316
13	4.21	0.73	0.3048	0.75	0.2286
14	4.2	0.72	0.3048	0.725	0.22098
15	4.16	0.68	0.3048	0.7	0.21336
16	4.17	0.69	0.3048	0.685	0.208788
17	4.15	0.67	0.3048	0.68	0.207264
18	4.12	0.64	0.3048	0.655	0.199644
19	4.05	0.57	0.3048	0.605	0.184404
20	4.03	0.55	0.3048	0.56	0.170688
21	3.84	0.36	0.3048	0.455	0.138684
22	3.76	0.28	0.3048	0.32	0.097536
23	3.72	0.24	0.3048	0.26	0.079248
24	3.48	0	0.3048	0.12	0.036576
				sum	4.840224

Non Uniform Length

interval d (ft)	length (m)	res length	delta d (m)	avg L	area (m^2)
0	7.16	0			
1	7.3	0.14	0.3048	0.07	0.021336
2	7.77	0.61	0.3048	0.375	0.1143
3	7.98	0.82	0.3048	0.715	0.217932
4	8	0.84	0.3048	0.83	0.252984
5	8.1	0.94	0.3048	0.89	0.271272
6	8.1	0.94	0.3048	0.94	0.286512
7	8.09	0.93	0.3048	0.935	0.284988
8	8.08	0.92	0.3048	0.925	0.28194
9	7.96	0.8	0.3048	0.86	0.262128
10	7.73	0.57	0.3048	0.685	0.208788
11	7.54	0.38	0.305	0.475	0.144875
11.417	7.3152	0.1552	0.1271	0.2676	0.034012
				sum	2.381067

Uniform and Total Area

width (m)	length (m)	u. area	n.u. area	**total area**
3.48	7.16	24.9168	7.221291	**32.13809**

Sample Calculations: see next page

Sample Calculations:

$$residual\ width = width - uniform\ width$$

$$residual\ width = 3.6 - 3.48 = 0.12$$

$$average\,w = \frac{reswidth_n + reswidth_{n+1}}{2}$$

$$avgw = \frac{0.12 + 0.52}{2} = 0.32$$

$$area = \Delta d * avgw$$

$$area = 0.3048 * 0.32 = 0.0975$$

$$total\ area = nonuniform\ area + uniform\ area$$

$$total\ area = 7.22 + 24.92 = 32.14$$

Calculation of Pond Surface Area for Volume Calculations
35.1 cm stage

Non Uniform Width

interval d (ft)	width (m)	res width	delta d (m)	avg w	area (m^2)
0	3.4	0.2			
1	3.8	0.6	0.3048	0.4	0.12192
2	4.1	0.9	0.3048	0.75	0.2286
3	3.95	0.75	0.3048	0.825	0.25146
4	3.94	0.74	0.3048	0.745	0.227076
5	3.94	0.74	0.3048	0.74	0.225552
6	3.93	0.73	0.3048	0.735	0.224028
7	3.93	0.73	0.3048	0.73	0.222504
8	3.92	0.72	0.3048	0.725	0.22098
9	3.91	0.71	0.3048	0.715	0.217932
10	3.89	0.69	0.3048	0.7	0.21336
11	3.91	0.71	0.3048	0.7	0.21336
12	3.87	0.67	0.3048	0.69	0.210312
13	3.84	0.64	0.3048	0.655	0.199644
14	3.78	0.58	0.3048	0.61	0.185928
15	3.72	0.52	0.3048	0.55	0.16764
16	3.72	0.52	0.3048	0.52	0.158496
17	3.71	0.51	0.3048	0.515	0.156972
18	3.64	0.44	0.3048	0.475	0.14478
19	3.6	0.4	0.3048	0.42	0.128016
20	3.49	0.29	0.3048	0.345	0.105156
21	3.5	0.3	0.3048	0.295	0.089916
22	3.24	0.04	0.3048	0.17	0.051816
23	3.2	0	0.3048	0.02	0.006096
				sum	3.971544

Non Uniform Length

interval d (ft)	length (m)	res length	delta d (m)	avg L	area (m^2)
0	6.94	0			
1	7.35	0.41	0.3048	0.205	0.062484
2.5	7.48	0.54	0.4572	0.475	0.21717
3	7.5	0.56	0.1524	0.55	0.08382
4	7.58	0.64	0.3048	0.6	0.18288
5	7.6	0.66	0.3048	0.65	0.19812
6	7.61	0.67	0.3048	0.665	0.202692
7	7.53	0.59	0.3048	0.63	0.192024
8	7.33	0.39	0.3048	0.49	0.149352
9	7.24	0.3	0.3048	0.345	0.105156
10	7.17	0.23	0.3048	0.265	0.080772
10.5	7.06	0.12	0.1524	0.175	0.02667
				sum	1.50114

Uniform and Total Area

width (m)	length (m)	u. area	n.u. area	**total area**
3.2	6.94	22.208	5.472684	**27.68068**

Calculation of Pond Surface Area for Volume Calculations
25.4 cm stage

Non Uniform Width

interval d (ft)	width (m)	res width	delta d (m)	avg w	area (m^2)
0	2.9	0.17			
1	3.4	0.67	0.3048	0.42	0.128016
2	3.34	0.61	0.3048	0.64	0.195072
3	3.5	0.77	0.3048	0.69	0.210312
4	3.47	0.74	0.3048	0.755	0.230124
5	3.48	0.75	0.3048	0.745	0.227076
6	3.43	0.7	0.3048	0.725	0.22098
7	3.43	0.7	0.3048	0.7	0.21336
8	3.4	0.67	0.3048	0.685	0.208788
9	3.38	0.65	0.3048	0.66	0.201168
10	3.41	0.68	0.3048	0.665	0.202692
11	3.41	0.68	0.3048	0.68	0.207264
12	3.34	0.61	0.3048	0.645	0.196596
13	3.31	0.58	0.3048	0.595	0.181356
14	3.3	0.57	0.3048	0.575	0.17526
15	3.23	0.5	0.3048	0.535	0.163068
16	3.2	0.47	0.3048	0.485	0.147828
17	3.15	0.42	0.3048	0.445	0.135636
18	3.06	0.33	0.3048	0.375	0.1143
19	3	0.27	0.3048	0.3	0.09144
20	2.87	0.14	0.3048	0.205	0.062484
21	2.9	0.17	0.3048	0.155	0.047244
21.3	2.73	0	0.3048	0.085	0.025908
				sum	3.585972

Non Uniform Length

interval d (ft)	length (m)	res length	delta d (m)	avg L	area (m^2)
0	6.43	0			
1	6.65	0.22	0.3048	0.11	0.033528
2	6.83	0.4	0.3048	0.31	0.094488
3	6.95	0.52	0.3048	0.46	0.140208
4	6.91	0.48	0.3048	0.5	0.1524
5	6.97	0.54	0.3048	0.51	0.155448
6	6.92	0.49	0.3048	0.515	0.156972
7	6.8	0.37	0.3048	0.43	0.131064
8	6.7	0.27	0.3048	0.32	0.097536
9	6.5	0.07	0.3048	0.17	0.051816
				sum	1.01346

Uniform and Total Area

width (m)	length (m)	u. area	n.u. area	**total area**
2.73	6.43	17.5539	4.599432	**22.153332**

Calculation of Pond Surface Area for Volume Calculations

15.1 cm stage

Non Uniform Width

interval d (ft)	width (m)	res width	delta d (m)	avg w	area (m^2)
0	2.6				
1	2.77	0.86	0.3048	0.43	0.131064
2	2.83	0.92	0.3048	0.89	0.271272
3	2.88	0.97	0.3048	0.945	0.288036
4	3	1.09	0.3048	1.03	0.313944
5	2.91	1	0.3048	1.045	0.318516
6	2.84	0.93	0.3048	0.965	0.294132
7	2.85	0.94	0.3048	0.935	0.284988
8	2.81	0.9	0.3048	0.92	0.280416
9	2.83	0.92	0.3048	0.91	0.277368
10	2.83	0.92	0.3048	0.92	0.280416
11	2.78	0.87	0.3048	0.895	0.272796
12	2.75	0.84	0.3048	0.855	0.260604
13	2.73	0.82	0.3048	0.83	0.252984
14	2.67	0.76	0.3048	0.79	0.240792
15	2.5	0.59	0.3048	0.675	0.20574
16	2.31	0.4	0.3048	0.495	0.150876
17	1.91	0	0.3048	0.2	0.06096
18	2.09	0.18	0.3048	0.09	0.027432
19	2.15	0.24	0.3048	0.21	0.064008
				sum	4.276344

Non Uniform Length

interval d (ft)	length (m)	res length	delta d (m)	avg L	area (m^2)
0	6.06	0			
1	6.3	0.24	0.3048	0.12	0.036576
2	6.31	0.25	0.3048	0.245	0.074676
3	6.31	0.25	0.3048	0.25	0.0762
4	6.31	0.25	0.3048	0.25	0.0762
5	6.18	0.12	0.3048	0.185	0.056388
6	6.1	0.04	0.3048	0.08	0.024384
				sum	0.344424

Uniform and Total Area

width (m)	length (m)	u. area	n.u. area	**total area**
1.91	6.06	11.5746	4.620768	**16.19537**

Volume Calculations

Head (m)	Area (m^2)	Volume per contour	Total Volume (m^3)
0.435	32.1381	2.5101	7.927
0.351	27.6808	2.4120	5.417
0.254	22.1533	1.9670	3.005
0.151	16.1954	1.0376	1.038
0	0	0	0

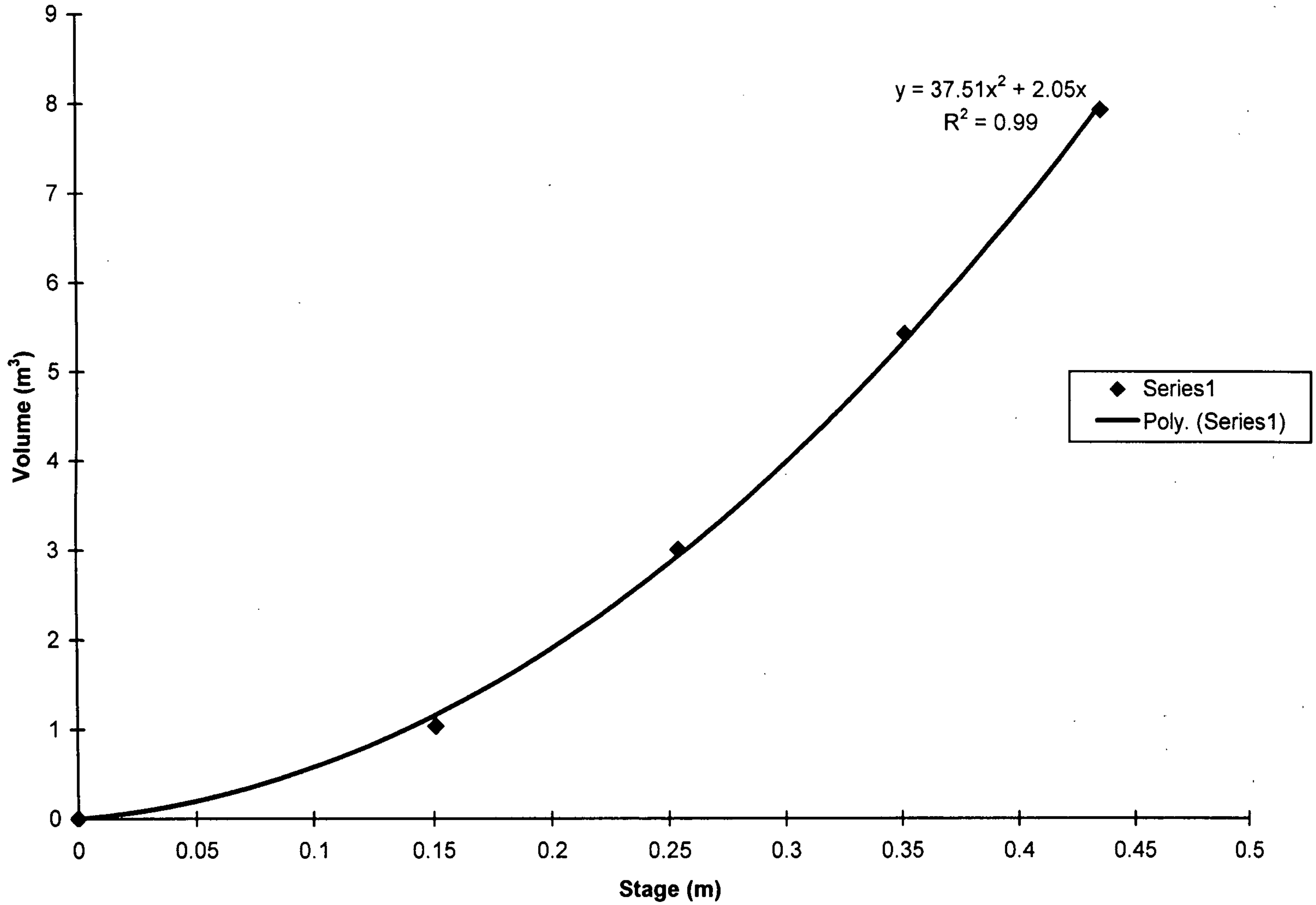
Sample Calculations:

$$V = \frac{h}{3}\left(A_l + A_u + \sqrt{A_l A_u}\right)$$

$$V = \frac{(0.435 - 0.351)}{3}\left(32.14 + 27.68 + \sqrt{32.14 * 27.68}\right)$$

$$V = 2.51 m^3$$

Volume vs. Stage



Appendix 5

Model Calculations

Calculation of k values for water depths used in the bank model that involve the top soil layer

Ditch A

h=44.925	k	kv	Kv	kh	Kh	K avg (m/day)	K avg (m/s)
1.93	3.26	0.0059	12.995	0.06275	14.497	13.72532984	1.59E-04
43.00	15	0.02867		6.45			

Ditch B

h=46	k	kv	Kv	kh	Kh	K avg (m/day)	K avg (m/s)
2.00	3.26	0.00613	12.969	0.0652	14.49	13.70838806	1.59E-04
44.00	15	0.02933		6.6			

Ditch C

h=44.925	k	kv	Kv	kh	Kh	K avg (m/day)	K avg (m/s)
6.59	3.26	0.02022	9.8142	0.21489	13.277	11.41524201	1.32E-04
38.33	15	0.02556		5.75			
h=42.775	k	kv	Kv	kh	Kh	K avg (m/day)	K avg (m/s)
4.44	3.26	0.01362	10.917	0.1448	13.781	12.26593328	1.42E-04
38.33	15	0.02556		5.75			
h=40.625	k	kv	Kv	kh	Kh	K avg (m/day)	K avg (m/s)
2.29	3.26	0.00703	12.467	0.07471	14.338	13.3698595	1.55E-04
38.33	15	0.02556		5.75			
h=38.475	k	kv	Kv	kh	Kh	K avg (m/day)	K avg (m/s)
0.14	3.26	0.00043	14.804	0.00462	14.957	14.88004191	1.72E-04
38.33	15	0.02556		5.75			

Bank Seepage Calculations for Unlined Pond

Ditch B

h (m)	dtop (m)	slope	GE	BE	d (m)	a	k (m/day)	q (m^3/m/day)	L (m)	q (m^3/s)
0.46	2.36	0.307	1.498371	0.449511	2.809511	0.046761	13.71	0.641016975	5	3.70959E-05
0.38	2.36	0.307	1.237785	0.371336	2.731336	0.032884	15	0.493261198	4.4	2.51198E-05
0.3	2.36	0.307	0.977199	0.29316	2.65316	0.021134	15	0.317007003	3.9	1.43093E-05
0.23	2.36	0.307	0.749186	0.224756	2.584756	0.012766	15	0.191491852	3.4	7.53556E-06
0.15	2.36	0.307	0.488599	0.14658	2.50658	0.005605	15	0.084078308	2.8	2.72476E-06

Ditch D

h (m)	dtop (m)	slope	GE	BE	d (m)	a	k (m/day)	q (m^3/m/day)	L (m)	q (m^3/s)
0.33	2.64	0.383	0.861619	0.258486	2.898486	0.023406	15	0.351097244	2.5	1.01591E-05
0.25	2.64	0.383	0.652742	0.195822	2.835822	0.013748	15	0.206220052	2.5	5.96702E-06
0.17	2.64	0.383	0.443864	0.133159	2.773159	0.006507	15	0.097608327	2.5	2.82432E-06
0.1	2.64	0.383	0.261097	0.078329	2.718329	0.002298	15	0.034476439	2.5	9.97582E-07
0.0188	2.64	0.383	0.049086	0.014726	2.654726	8.32E-05	15	0.001248136	2.5	3.6115E-08

Sample Calculations

$$GE = \frac{h}{slope} = \frac{0.46}{0.307} = 1.498$$

$$BE = 0.3GE = 0.3 * 1.498 = 0.45$$

$$d = dtop + BE = 2.36 + 0.45 = 2.81$$

$$a = \sqrt{d^2 + h^2} - h = \sqrt{2.81^2 + 0.46^2} - 0.446 = 0.047$$

$$q = ka = 12.34 * 0.047 = 0.58 m^3 day^{-1} m^{-1}$$

$$q(m^3 s^{-1}) = \frac{qL}{86400} = \frac{0.58 * 5.7}{86400}$$

$$q = 3.81E-05 m^3 s^{-1}$$

Ditch A

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
0-1	0.46	0.4385	0.44925	2.84	0.42	1.069643	0.320893	3.160893	0.039707	13.73	0.544994877	1	6.30781E-06
	0.38	0.3585	0.36925	2.84	0.42	0.879167	0.26375	3.10375	0.027359	15	0.410390156	1	4.74989E-06
	0.3	0.2785	0.28925	2.84	0.42	0.68869	0.206607	3.046607	0.017125	15	0.256877563	1	2.97312E-06
	0.23	0.2085	0.21925	2.84	0.42	0.522024	0.156607	2.996607	0.010013	15	0.150189859	1	1.73831E-06
	0.15	0.1285	0.13925	2.84	0.42	0.331548	0.099464	2.939464	0.004121	15	0.061808762	1	7.15379E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
1-2	0.4385	0.417	0.42775	2.84	0.42	1.018452	0.305536	3.145536	0.036189	15	0.542828572	1	6.28274E-06
	0.3585	0.337	0.34775	2.84	0.42	0.827976	0.248393	3.088393	0.024396	15	0.365934165	1	4.23535E-06
	0.2785	0.257	0.26775	2.84	0.42	0.6375	0.19125	3.03125	0.014753	15	0.221291044	1	2.56124E-06
	0.2085	0.187	0.19775	2.84	0.42	0.470833	0.14125	2.98125	0.008189	15	0.122836927	1	1.42172E-06
	0.1285	0.107	0.11775	2.84	0.42	0.280357	0.084107	2.924107	0.002962	15	0.044434866	1	5.14292E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
2-3	0.417	0.3955	0.40625	2.84	0.42	0.967262	0.290179	3.130179	0.032816	15	0.492233908	1	5.69715E-06
	0.337	0.3155	0.32625	2.84	0.42	0.776786	0.233036	3.073036	0.021587	15	0.323806906	1	3.74777E-06
	0.257	0.2355	0.24625	2.84	0.42	0.58631	0.175893	3.015893	0.012546	15	0.188185347	1	2.17807E-06
	0.187	0.1655	0.17625	2.84	0.42	0.419643	0.125893	2.965893	0.00654	15	0.098105004	1	1.13547E-06
	0.107	0.0855	0.09625	2.84	0.42	0.229167	0.06875	2.90875	0.00199	15	0.029850219	1	3.45489E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
3-4	0.3955	0.374	0.38475	2.84	0.42	0.916071	0.274821	3.114821	0.029591	15	0.443862227	1	5.13729E-06
	0.3155	0.294	0.30475	2.84	0.42	0.725595	0.217679	3.057679	0.018937	15	0.284048402	1	3.2876E-06
	0.2355	0.214	0.22475	2.84	0.42	0.535119	0.160536	3.000536	0.010507	15	0.157602826	1	1.82411E-06
	0.1655	0.144	0.15475	2.84	0.42	0.368452	0.110536	2.950536	0.005069	15	0.076038468	1	8.80075E-07
	0.0855	0.064	0.07475	2.84	0.42	0.177976	0.053393	2.893393	0.001207	15	0.018101469	1	2.09508E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
4-5	0.374	0.3525	0.36325	2.84	0.42	0.864881	0.259464	3.099464	0.026517	15	0.397751859	1	4.60361E-06
	0.294	0.2725	0.28325	2.84	0.42	0.674405	0.202321	3.042321	0.016447	15	0.246699303	1	2.85532E-06
	0.214	0.1925	0.20325	2.84	0.42	0.483929	0.145179	2.985179	0.008639	15	0.129586459	1	1.49984E-06
	0.144	0.1225	0.13325	2.84	0.42	0.317262	0.095179	2.935179	0.003779	15	0.056682317	1	6.56045E-07
	0.064	0.0425	0.05325	2.84	0.42	0.126786	0.038036	2.878036	0.000616	15	0.009235856	1	1.06896E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
5-6.1	0.3525	0.3288	0.34065	2.84	0.42	0.811071	0.243321	3.083321	0.023451	15	0.351762889	2.5	1.01783E-05
	0.2725	0.2488	0.26065	2.84	0.42	0.620595	0.186179	3.026179	0.014005	15	0.210082048	2	4.86301E-06
	0.1925	0.1688	0.18065	2.84	0.42	0.430119	0.129036	2.969036	0.006863	15	0.102950959	1.7	2.02566E-06
	0.1225	0.0988	0.11065	2.84	0.42	0.263452	0.079036	2.919036	0.002621	15	0.039307804	1.4	6.36932E-07
	0.0425	0.0188	0.03065	2.84	0.42	0.072976	0.021893	2.861893	0.000205	15	0.003077276	1.1	3.91783E-08

h (m)	Total q (m^3/s)
0.46	3.82069E-05
0.38	2.37389E-05
0.3	1.3062E-05
0.23	6.46856E-06
0.15	1.93074E-06

Note: Due to the slope of the pond bottom and the resulting changes in water depth, the length of ditches A & C is divided into 1m sections and the average depth of the sections used in flow calculations

Ditch C

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
0-1	0.46	0.4385	0.44925	2.413	0.34	1.321324	0.396397	2.809397	0.044616	11.42	0.509305175	1	5.89474E-06
	0.38	0.3585	0.36925	2.413	0.34	1.086029	0.325809	2.738809	0.030974	15	0.464611935	1	5.37745E-06
	0.3	0.2785	0.28925	2.413	0.34	0.850735	0.255221	2.668221	0.01954	15	0.293106828	1	3.39244E-06
	0.23	0.2085	0.21925	2.413	0.34	0.644853	0.193456	2.606456	0.011506	15	0.172597254	1	1.99765E-06
	0.15	0.1285	0.13925	2.413	0.34	0.409559	0.122868	2.535868	0.004775	15	0.071632166	1	8.29076E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
1-2	0.4385	0.417	0.42775	2.413	0.34	1.258088	0.377426	2.790426	0.040744	12.27	0.499759377	1	5.78425E-06
	0.3585	0.337	0.34775	2.413	0.34	1.022794	0.306838	2.719838	0.027676	15	0.41514362	1	4.8049E-06
	0.2785	0.257	0.26775	2.413	0.34	0.7875	0.23625	2.64925	0.01687	15	0.253047762	1	2.92879E-06
	0.2085	0.187	0.19775	2.413	0.34	0.581618	0.174485	2.587485	0.009432	15	0.141479524	1	1.63749E-06
	0.1285	0.107	0.11775	2.413	0.34	0.346324	0.103897	2.516897	0.003441	15	0.051616697	1	5.97415E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
2-3	0.417	0.3955	0.40625	2.413	0.34	1.194853	0.358456	2.771456	0.037021	13.37	0.494961388	1	5.72872E-06
	0.337	0.3155	0.32625	2.413	0.34	0.959559	0.287868	2.700868	0.024542	15	0.368123364	1	4.26069E-06
	0.257	0.2355	0.24625	2.413	0.34	0.724265	0.217279	2.630279	0.014377	15	0.215661857	1	2.49609E-06
	0.187	0.1655	0.17625	2.413	0.34	0.518382	0.155515	2.568515	0.00755	15	0.113249721	1	1.31076E-06
	0.107	0.0855	0.09625	2.413	0.34	0.283088	0.084926	2.497926	0.002317	15	0.034756176	1	4.02271E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
3-4	0.3955	0.374	0.38475	2.413	0.34	1.131618	0.339485	2.752485	0.033451	14.88	0.497748953	1	5.76098E-06
	0.3155	0.294	0.30475	2.413	0.34	0.896324	0.268897	2.681897	0.021574	15	0.323609608	1	3.74548E-06
	0.2355	0.214	0.22475	2.413	0.34	0.661029	0.198309	2.611309	0.012068	15	0.181013258	1	2.09506E-06
	0.1655	0.144	0.15475	2.413	0.34	0.455147	0.136544	2.549544	0.005865	15	0.087977296	1	1.01826E-06
	0.0855	0.064	0.07475	2.413	0.34	0.219853	0.065956	2.478956	0.001408	15	0.021126433	1	2.44519E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
4-5	0.374	0.3525	0.36325	2.413	0.34	1.068382	0.320515	2.733515	0.030038	15	0.450563876	1	5.21486E-06
	0.294	0.2725	0.28325	2.413	0.34	0.833088	0.249926	2.662926	0.018777	15	0.281662287	1	3.25998E-06
	0.214	0.1925	0.20325	2.413	0.34	0.597794	0.179338	2.592338	0.009945	15	0.149167712	1	1.72648E-06
	0.144	0.1225	0.13325	2.413	0.34	0.391912	0.117574	2.530574	0.004382	15	0.065733389	1	7.60803E-07
	0.064	0.0425	0.05325	2.413	0.34	0.156618	0.046985	2.459985	0.00072	15	0.010805059	1	1.25059E-07

length	h1 (m)	h2 (m)	havg	dt	slope	GE	BE	d (m)	a	k	q (m^3/m/day)	L (m)	q (m^3/s)
5-6.1	0.3525	0.3288	0.34065	2.413	0.34	1.001912	0.300574	2.713574	0.026623	15	0.399342435	2.5	1.1555E-05
	0.2725	0.2488	0.26065	2.413	0.34	0.766618	0.229985	2.642985	0.016027	15	0.240402983	2	5.56488E-06
	0.1925	0.1688	0.18065	2.413	0.34	0.531324	0.159397	2.572397	0.007919	15	0.118788592	1.7	2.33728E-06
	0.1225	0.0988	0.11065	2.413	0.34	0.325441	0.097632	2.510632	0.003046	15	0.045696218	1.4	7.40448E-07
	0.0425	0.0188	0.03065	2.413	0.34	0.090147	0.027044	2.440044	0.000241	15	0.003609254	1.1	4.59511E-08

h (m)	Total q (m^3/s)
0.46	3.99386E-05
0.38	2.70134E-05
0.3	1.49761E-05
0.23	7.46542E-06
0.15	2.24429E-06

Bank Seepage Calculations for Lined Pond

Ditch B

h (m)	dtop	slope	GE	BE	d (m)	a	k (m/day)	q (m^3/m/day)	L (m)	q (m^3/s)
0.46	2.36	0.307	1.498371	0.449511	2.809511	0.046761	13.70839	0.641016975	5	4.22893E-05
0.38	2.36	0.307	1.237785	0.371336	2.731336	0.032884	15	0.493261198	4.4	3.25415E-05
0.3	2.36	0.307	0.977199	0.29316	2.65316	0.021134	15	0.317007003	3.9	2.09137E-05
0.23	2.36	0.307	0.749186	0.224756	2.584756	0.012766	15	0.191491852	3.4	1.26331E-05
0.15	2.36	0.307	0.488599	0.14658	2.50658	0.005605	15	0.084078308	2.8	5.54683E-06

Wetland Bottom Seepage Calculations

k (m/day)	A (m^2)	Hw (m)	Hct (m)	Hcb (m)	Q (m^3/d)	Q m^3/s
0.075	15.48384	0.457	0	-0.12	5.58386	6.4628E-05
0.075	15.48384	0.381	0	-0.12	4.848377	5.61155E-05
0.075	15.48384	0.31	0	-0.12	4.112895	4.7603E-05
0.075	15.48384	0.225	0	-0.12	3.338703	3.86424E-05
0.075	15.48384	0.152	0	-0.12	2.632253	3.04659E-05

Sample Calculations

$$Q = kA\left[\frac{H_w - H_{cb}}{H_{ct} - H_{cb}}\right]$$

$$Q = 0.075 * 15.48 * \left[\frac{0.46 + 0.12}{0 + 0.12}\right]$$

$$Q = 5.58\,\frac{m^3}{day}$$

A = pond bottom area = 15.483 (from field measurments)
The value for k was determined in Appendix 2, pg 79

Model Totals

Unlined Pond

h (m)	Bank Total (m^3/s)	Wetland Total (m^3/s)	Model Total (m^3/s)
0.46	0.0001254	6.4964E-05	0.000190028
0.38	8.18391E-05	5.60035E-05	0.000137955
0.3	4.51718E-05	4.70429E-05	9.27748E-05
0.23	2.24671E-05	3.92024E-05	6.11095E-05
0.15	6.93591E-06	3.02419E-05	3.74018E-05

Lined Pond (assumed perfectly lined pond)

h (m)	Bank Total (m^3/s)	Wetland Total (m^3/s)	Model Total (m^3/s)
0.46	0	3.80785E-05	3.80785E-05
0.38	0	2.82027E-05	2.82027E-05
0.3	0	1.81252E-05	1.81252E-05
0.23	0	1.09487E-05	1.09487E-05
0.15	0	4.80726E-06	4.80726E-06

Appendix 6

Application Calculations

Required flow to produce changes in riffle/glide sections

n	slope	w (m)	d1 (m)	d2 (m)	R1 (m)	R2 (m)	v1 (m/s)	v2 (m/s)	q1 (m^3/s)	q2 (m^3/s)	difference	%
0.07	0.001	1	0.25	0.3	0.166667	0.1875	0.136815	0.147991	0.034204	0.044397	0.01019	29.80
0.07	0.001	1	0.2	0.25	0.142857	0.166667	0.123453	0.136815	0.024691	0.034204	0.00951	38.53
0.07	0.001	1	0.15	0.2	0.115385	0.142857	0.10707	0.123453	0.01606	0.024691	0.00863	53.74
0.07	0.001	1	0.1	0.15	0.083333	0.115385	0.086188	0.10707	0.008619	0.016060	0.00744	86.34
0.07	0.001	1	0.05	0.1	0.045455	0.083333	0.057538	0.086188	0.002877	0.008619	0.00574	199.59

Sample Calculations

$$R = \frac{wd}{w+2d}$$

$$R = \frac{1*0.25}{1+2*0.25} = 0.167$$

$$V = \frac{1}{n} R^{\frac{2}{3}} S^{\frac{1}{2}}$$

$$V = \frac{1}{0.07} 0.167^{\frac{2}{3}} 0.001^{\frac{1}{2}} = 0.137$$

$$Q = V * A = 0.137 * 1 * 0.25 = 0.034$$

Results from Numerous Modelling Trials of Different Pond Volumes

Pond Volume (m^3)	Area (m^2)	Top Dimensions (m)	depth (m)	seepage area (m^2)	s. area as a % of bottom	start flow m^3/hour	mid flow m^3/hour	mid time months	end time months	end time days
8.88	39.8	4.8*8.3	0.46	0.1548	1	0.0023	0.0016	3.93	7.85	243.4
8.88	39.8	4.8*8.3	0.46	0.2322	1.5	0.0035	0.0023	2.62	5.23	162.2
8.88	39.8	4.8*8.3	0.46	0.3096	2	0.0047	0.0031	1.96	3.93	121.7
8.88	39.8	4.8*8.3	0.46	0.774	5	0.01	0.01	0.79	1.57	48.7
8.88	39.8	4.8*8.3	0.46	1.548	10	0.02	0.02	0.38	0.77	23.8
8.88	39.8	4.8*8.3	0.46	15.48	100	0.23	0.16	0.04	0.08	2.4
39.55	103.65	8.6*12.1	1	0.1548	1	0.005	0.003	9.92	19.83	614.7
39.55	103.65	8.6*12.1	1	0.3096	2	0.009	0.005	4.96	9.92	307.4
39.55	103.65	8.6*12.1	1	0.4644	3	0.01	0.01	3.31	6.61	204.9
39.55	103.65	8.6*12.1	1	0.6192	4	0.02	0.01	2.48	4.96	153.7
39.55	103.65	8.6*12.1	1	0.774	5	0.02	0.01	1.98	3.97	122.9
39.55	103.65	8.6*12.1	1	1.548	10	0.05	0.03	0.99	1.98	61.4
126.08	230.4	12*19.2	1	0.1548	0.25	0.005	0.003	32.69	65.37	2026.5
126.08	230.4	12*19.2	1	0.61	1	0.02	0.01	8.29	16.58	514.0
126.08	230.4	12*19.2	1	1.548	2.54	0.05	0.03	3.27	6.54	202.7
126.08	230.4	12*19.2	1	1.83	3	0.05	0.03	2.76	5.53	171.3
126.08	230.4	12*19.2	1	2.44	4	0.07	0.04	2.07	4.15	128.5
126.08	230.4	12*19.2	1	6.1	10	0.18	0.11	0.83	1.66	51.4
126.08	230.4	12*19.2	1	30.5	50	0.89	0.13	0.17	0.33	10.3
126.08	230.4	12*19.2	1	61	100	1.78	0.11	0.08	0.17	5.1
275.3	352	15.5*22.7	1.5	6.1	10	0.26	0.15	1.29	2.57	79.7
275.3	352	15.5*22.7	1.5	3.05	5	0.13	0.07	2.58	5.14	159.3
644	707	20.5*34.5	1.5	24	10	1.01	0.49	0.86	1.72	53.3
644	707	20.5*34.5	1.5	12	5	0.51	0.24	1.72	3.44	106.6
2307	2148	36*61	1.5	125	10	5.27	2.16	0.67	1.33	41.2
2307	2148	36*61	1.5	62.5	5	2.64	1.08	1.33	2.66	82.5
3466	2496	39*64	2	125	10	6.90	2.61	0.81	1.61	49.8
3466	2496	39*64	2	125	5	3.45	1.30	1.61	3.22	99.7
3466	2496	39*64	2	125	2.5	1.73	0.65	3.22	6.43	199.4
3466	2496	39*64	2	125	3.5	2.76	1.05	2.48	4.96	153.8

Seepage Calculation for a Sample Pond

stage (m)	Pond V m^3	k (m/day)	Seep. Area (m^2)	Hw (m)	Hhpt (m)	Hhpb (m)	Q (m^3/d)	Q (m^3/hour)	t (hours)	V lost (m^3)	V2 (m^3)	stage 2 (m)
0.46	8.877844	0.075	15.48	0.46	0	-0.12	5.6115	0.233813	1	0.233813	8.644031	0.453561
0.453561	8.644031	0.075	15.48	0.453561	0	-0.12	5.549201	0.231217	1	0.231217	8.412814	0.447107
0.447107	8.412814	0.075	15.48	0.447107	0	-0.12	5.486762	0.228615	1	0.228615	8.184199	0.440639
0.440639	8.184199	0.075	15.48	0.440639	0	-0.12	5.424179	0.226007	1	0.226007	7.958192	0.434155
0.434155	7.958192	0.075	15.48	0.434155	0	-0.12	5.361448	0.223394	1	0.223394	7.734798	0.427655
0.427655	7.734798	0.075	15.48	0.427655	0	-0.12	5.298563	0.220773	1	0.220773	7.514025	0.421139
0.421139	7.514025	0.075	15.48	0.421139	0	-0.12	5.235521	0.218147	1	0.218147	7.295878	0.414606
0.414606	7.295878	0.075	15.48	0.414606	0	-0.12	5.172316	0.215513	1	0.215513	7.080365	0.408056
0.408056	7.080365	0.075	15.48	0.408056	0	-0.12	5.108942	0.212873	1	0.212873	6.867492	0.401488
0.401488	6.867492	0.075	15.48	0.401488	0	-0.12	5.045395	0.210225	1	0.210225	6.657267	0.394901
0.394901	6.657267	0.075	15.48	0.394901	0	-0.12	4.981668	0.20757	1	0.20757	6.449698	0.388295
0.388295	6.449698	0.075	15.48	0.388295	0	-0.12	4.917756	0.204906	1	0.204906	6.244791	0.381669
0.381669	6.244791	0.075	15.48	0.381669	0	-0.12	4.85365	0.202235	1	0.202235	6.042556	0.375023
0.375023	6.042556	0.075	15.48	0.375023	0	-0.12	4.789345	0.199556	1	0.199556	5.843	0.368355
0.368355	5.843	0.075	15.48	0.368355	0	-0.12	4.724834	0.196868	1	0.196868	5.646132	0.361665
0.361665	5.646132	0.075	15.48	0.361665	0	-0.12	4.660108	0.194171	1	0.194171	5.451961	0.354952
0.354952	5.451961	0.075	15.48	0.354952	0	-0.12	4.595159	0.191465	1	0.191465	5.260496	0.348215
0.348215	5.260496	0.075	15.48	0.348215	0	-0.12	4.529978	0.188749	1	0.188749	5.071747	0.341453
0.341453	5.071747	0.075	15.48	0.341453	0	-0.12	4.464557	0.186023	1	0.186023	4.885724	0.334665
0.334665	4.885724	0.075	15.48	0.334665	0	-0.12	4.398884	0.183287	1	0.183287	4.702437	0.32785
0.32785	4.702437	0.075	15.48	0.32785	0	-0.12	4.33295	0.18054	1	0.18054	4.521897	0.321007
0.321007	4.521897	0.075	15.48	0.321007	0	-0.12	4.266744	0.177781	1	0.177781	4.344116	0.314135
0.314135	4.344116	0.075	15.48	0.314135	0	-0.12	4.200253	0.175011	1	0.175011	4.169106	0.307231
0.307231	4.169106	0.075	15.48	0.307231	0	-0.12	4.133464	0.172228	1	0.172228	3.996878	0.300296
0.300296	3.996878	0.075	15.48	0.300296	0	-0.12	4.066363	0.169432	1	0.169432	3.827446	0.293327
0.293327	3.827446	0.075	15.48	0.293327	0	-0.12	3.998935	0.166622	1	0.166622	3.660824	0.286322
0.286322	3.660824	0.075	15.48	0.286322	0	-0.12	3.931165	0.163799	1	0.163799	3.497025	0.27928
0.27928	3.497025	0.075	15.48	0.27928	0	-0.12	3.863034	0.16096	1	0.16096	3.336066	0.272199
0.272199	3.336066	0.075	15.48	0.272199	0	-0.12	3.794524	0.158105	1	0.158105	3.17796	0.265076
0.265076	3.17796	0.075	15.48	0.265076	0	-0.12	3.725613	0.155234	1	0.155234	3.022727	0.25791
0.25791	3.022727	0.075	15.48	0.25791	0	-0.12	3.65628	0.152345	1	0.152345	2.870382	0.250698
0.250698	2.870382	0.075	15.48	0.250698	0	-0.12	3.586501	0.149438	1	0.149438	2.720944	0.243436
0.243436	2.720944	0.075	15.48	0.243436	0	-0.12	3.516247	0.14651	1	0.14651	2.574434	0.236123
0.236123	2.574434	0.075	15.48	0.236123	0	-0.12	3.445491	0.143562	1	0.143562	2.430872	0.228754
0.228754	2.430872	0.075	15.48	0.228754	0	-0.12	3.374198	0.140592	1	0.140592	2.29028	0.221326
0.221326	2.29028	0.075	15.48	0.221326	0	-0.12	3.302334	0.137597	1	0.137597	2.152683	0.213835
0.213835	2.152683	0.075	15.48	0.213835	0	-0.12	3.229857	0.134577	1	0.134577	2.018105	0.206276

0.206276	2.018105	0.075	15.48	0.206276	0	-0.12	3.156724	0.13153	1	0.13153	1.886575	0.198644
0.198644	1.886575	0.075	15.48	0.198644	0	-0.12	3.082884	0.128453	1	0.128453	1.758122	0.190933
0.190933	1.758122	0.075	15.48	0.190933	0	-0.12	3.008279	0.125345	1	0.125345	1.632777	0.183137
0.183137	1.632777	0.075	15.48	0.183137	0	-0.12	2.932847	0.122202	1	0.122202	1.510575	0.175247
0.175247	1.510575	0.075	15.48	0.175247	0	-0.12	2.856512	0.119021	1	0.119021	1.391553	0.167255
0.167255	1.391553	0.075	15.48	0.167255	0	-0.12	2.77919	0.1158	1	0.1158	1.275754	0.159151
0.159151	1.275754	0.075	15.48	0.159151	0	-0.12	2.700784	0.112533	1	0.112533	1.163221	0.150923
0.150923	1.163221	0.075	15.48	0.150923	0	-0.12	2.621176	0.109216	1	0.109216	1.054006	0.142556
0.142556	1.054006	0.075	15.48	0.142556	0	-0.12	2.540231	0.105843	1	0.105843	0.948163	0.134035
0.134035	0.948163	0.075	15.48	0.134035	0	-0.12	2.457786	0.102408	1	0.102408	0.845755	0.125338
0.125338	0.845755	0.075	15.48	0.125338	0	-0.12	2.373641	0.098902	1	0.098902	0.746853	0.11644
0.11644	0.746853	0.075	15.48	0.11644	0	-0.12	2.287553	0.095315	1	0.095315	0.651538	0.107309
0.107309	0.651538	0.075	15.48	0.107309	0	-0.12	2.199212	0.091634	1	0.091634	0.559905	0.097904
0.097904	0.559905	0.075	15.48	0.097904	0	-0.12	2.108224	0.087843	1	0.087843	0.472062	0.088172
0.088172	0.472062	0.075	15.48	0.088172	0	-0.12	2.014064	0.083919	1	0.083919	0.388143	0.078038
0.078038	0.388143	0.075	15.48	0.078038	0	-0.12	1.916015	0.079834	1	0.079834	0.308309	0.067395
0.067395	0.308309	0.075	15.48	0.067395	0	-0.12	1.81305	0.075544	1	0.075544	0.232765	0.056083
0.056083	0.232765	0.075	15.48	0.056083	0	-0.12	1.703604	0.070984	1	0.070984	0.161781	0.043833
0.043833	0.161781	0.075	15.48	0.043833	0	-0.12	1.585081	0.066045	1	0.066045	0.095736	0.030133
0.030133	0.095736	0.075	15.48	0.030133	0	-0.12	1.452541	0.060523	1	0.060523	0.035214	0.013743
0.013743	0.035214	0.075	15.48	0.013743	0	-0.12	1.293962	0.053915	1	0.053915	-0.0187	-0.0116

Sample Calculations

$$V_{pond} = 37.51(stage)^2 + 2.05(stage) = 37.51(0.46) + 2.05(0.46) = 8.88$$

$$Q = kA\left[\frac{H_w - H_{cb}}{H_{ct} - H_{cb}}\right] = 0.075*15.48*\left[\frac{0.46+0.12}{0+0.12}\right] = 5.6$$

$$V_{lost} = Q*time = \frac{5.6}{24}(1) = 0.234$$

$$V_2 = V_{pond} - V_{lost} = 8.88 - 0.234 = 8.64$$

$$stage_2 = \frac{-2.05\sqrt{2.05^2 - 4(37.51)(-V_2)}}{2(37.51)} = \frac{-2.05\sqrt{2.05^2 + 4(37.51)(8.64)}}{2(37.51)} = 0.454$$

Increase in depth of Riffle/Glide sections from different amounts of pond seepage

12 ponds = 43728 m^3

start flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.0092	0.0339	37.26	0.249	0.165999	0.136449	0.0339	0.0485
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.0092	0.0253	57.28	0.204	0.144634	0.124475	0.0253	0.0535
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.0092	0.0178	106.74	0.161	0.121499	0.11082	0.0178	0.0605
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.0092	0.0121	319.79	0.125	0.09968	0.09712	0.0121	0.0745

mid flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.0035	0.0282	14.18	0.219	0.152295	0.128833	0.0282	0.019
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.0035	0.0196	21.79	0.171	0.127422	0.114393	0.0196	0.021
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.0035	0.0121	40.61	0.125	0.099808	0.097203	0.0121	0.0247
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.0035	0.0064	121.66	0.083	0.071184	0.077593	0.0064	0.033

9 ponds = 32796 m^3

start flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.0069	0.0316	27.95	0.237	0.160557	0.133451	0.0316	0.0365
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.0069	0.0230	42.96	0.191	0.137944	0.120606	0.0230	0.0405
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.0069	0.0155	80.06	0.147	0.113302	0.105778	0.0155	0.0465
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.0069	0.0098	239.84	0.109	0.089154	0.090156	0.0098	0.0585

mid flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.002625	0.0273	10.63	0.214	0.14986	0.127456	0.0273	0.014
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.002625	0.0187	16.34	0.166	0.124625	0.112712	0.0187	0.016
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.002625	0.0112	30.46	0.119	0.095796	0.09458	0.0112	0.0185
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.002625	0.0055	91.24	0.076	0.065595	0.073477	0.0055	0.0255

6 ponds = 21864 m^3

start flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.0046	0.0293	18.63	0.225	0.154934	0.130317	0.0293	0.025
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.0046	0.0207	28.64	0.178	0.130996	0.116522	0.0207	0.028
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.0046	0.0132	53.37	0.132	0.10443	0.100181	0.0132	0.032
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.0046	0.0075	159.89	0.092	0.077346	0.082009	0.0075	0.042

mid flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.00175	0.0264	7.09	0.210	0.147639	0.126193	0.0264	0.009
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.00175	0.0178	10.90	0.161	0.121499	0.11082	0.0178	0.011
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.00175	0.0104	20.30	0.113	0.09217	0.092178	0.0104	0.013
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.00175	0.0046	60.83	0.068	0.059471	0.068829	0.0046	0.018

3 ponds = 10932 m^3

start flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.0023	0.0270	9.32	0.213	0.149123	0.127037	0.0270	0.0125
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.0023	0.0184	14.32	0.164	0.123494	0.11203	0.0184	0.014
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.0023	0.0109	26.69	0.117	0.094485	0.093715	0.0109	0.0165
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.0023	0.0052	79.95	0.073	0.063319	0.071767	0.0052	0.0225

mid flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.000875	0.0256	3.54	0.205	0.14539	0.124908	0.0256	0.005
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.000875	0.0169	5.45	0.155	0.118554	0.109021	0.0169	0.0054
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.000875	0.0095	10.15	0.107	0.087799	0.08924	0.0095	0.0065
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.000875	0.0038	30.41	0.060	0.053172	0.063879	0.0038	0.0095

2 ponds = 7288 m^3

start flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.001533333	0.0262	6.21	0.209	0.147142	0.12591	0.0263	0.0085
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.001533333	0.0176	9.55	0.160	0.120925	0.11047	0.0176	0.0095
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.001533333	0.0102	17.79	0.112	0.091169	0.09151	0.0102	0.0115
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.001533333	0.0044	53.30	0.065	0.057522	0.067317	0.0044	0.015

mid flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.000583333	0.0253	2.36	0.204	0.144634	0.124475	0.0253	0.0035
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.000583333	0.0166	3.63	0.154	0.117445	0.10834	0.0166	0.0035
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.000583333	0.0092	6.77	0.105	0.086435	0.088314	0.0092	0.0045
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.000583333	0.0035	20.28	0.057	0.050764	0.061935	0.0035	0.0065

1 pond = 3644 m^3

start flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.000766667	0.0255	3.11	0.205	0.145138	0.124764	0.0255	0.0045
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.000766667	0.0168	4.77	0.155	0.118029	0.1087	0.0168	0.0045
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.000766667	0.0094	8.90	0.106	0.087118	0.088778	0.0094	0.0055
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.000766667	0.0036	26.65	0.058	0.051971	0.062914	0.0036	0.008

mid flow

n	slope	w (m)	d (m)	R (m)	v (m/s)	q (m^3/s)	q pond (m^3/s)	q total (m^3/s)	q increase (%)	dnew (m)	Rnew	Vnew	Qnew	d increase (m)
0.07	0.001	1	0.2	0.142857	0.123453	0.024691	0.000291667	0.0250	1.18	0.202	0.143621	0.123893	0.0250	0.0015
0.07	0.001	1	0.15	0.115385	0.10707	0.01606	0.000291667	0.0164	1.82	0.152	0.116564	0.107798	0.0164	0.002
0.07	0.001	1	0.1	0.083333	0.086188	0.008619	0.000291667	0.0089	3.38	0.102	0.084718	0.08714	0.0089	0.002
0.07	0.001	1	0.05	0.045455	0.057538	0.002877	0.000291667	0.0032	10.14	0.053	0.04792	0.0596	0.0032	0.003