PHOSPHORUS REMOVAL AND FLOW MAXIMIZATION IN THE KAMLOOPS RAPID INFILTRATION SYSTEM

by

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Abstract

A proposal concerning its rapid infiltration system at the wastewater treatment plant was submitted to and accepted by the City of Kamloops. A project was therefore undertaken to study the basins as they currently operate as well as how they have done so in the past. To date, only half of the design throughput is realized by the basins. Also to be examined was the phosphorus removal capability of the basin sand, in the hope that it would be sufficient to cease or curtail the practice of chemical phosphorus removal using alum.

The phosphorus removal capability of the sand was studied by means of a number of column tests performed using influents of varying phosphorus concentrations. The columns were packed with basin sand of varying depths. An appropriate sand depth and influent phosphorus concentration were chosen as a result of the column tests. It was concluded that a depth of 120 cm and an influent phosphorus concentration of approximately 3.5 mg/L were suitable for further testing. These subsequent experiments revealed that this influent P concentration can only be adopted if accompanied by substantial changes to the current system. These changes include either the occasional re-routing of basin effluent to spray-irrigation or the expansion of the wetlands pilot project.

To ensure the propriety of the recommended changes which resulted from the column tests, numerous analyses were performed on the RI basins. It was discovered that the basins have saturated flow; the flow was previously assumed to be unsaturated and therefore all throughput problems had always been linked to surface phenomena. This change in outlook with respect to basin operation allows for a number of alterations to be made. It is recommended that the current sand depth of 220 cm be reduced by 100 cm to 120 cm. This decrease in sand depth will decrease flow resistance by 45%; the accompanying increase in head will further increase the throughput capability. A discrepancy between in-situ permeability \((0.5 \times 10^{-5} \text{ m/s on average})\) and design permeability \((5 \times 10^{-5} \text{ m/s})\) was discovered and attributed to an error in basin construction. This information changes the view thus far maintained that the basins have never worked according to their design; this can now be altered by substituting the actual permeability for the design permeability into performance expectations. Differences in throughput between the east and west basins, which have always been evident, are attributed to a difference in surface area between the basins and to a small difference in permeability. The west basin is 21% larger than the east. Finally, it was discovered that the basins are indeed capable of removing and retaining
phosphorus from wastewater, as was seen by analyzing historical basin data. Most of this retention occurs in the organic-rich surface layer. This result further supports the cutting away of 1 m of sand, since phosphorus removal does not depend on depth. More importantly, this straightforward fact implies that the RI basins are a worthwhile process within the Kamloops treatment plant.

Included in the report are numerous recommendations which include changes to the current basins, design of future basins as well as suggestions regarding current basin operation and maintenance. No opinion by the author was given concerning alum cessation or reduction, although various options were provided. The use of fewer chemicals would be appreciated both monetarily and publicly; however this would necessitate increased attention to the RI system by City staff and would leave the basins more susceptible to operational failure, which could result in failure to comply with discharge permits. The recommendations are offered by the author, but the decisions are suitably left to the City of Kamloops.
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1.1 General and Background Information

Rapid infiltration (RI) is known as an effluent polishing step and, if incorporated, is typically the final process at a wastewater treatment plant. It consists of any number of outdoor sand basins through which wastewater is filtered and ‘polished’. The outdoor stipulation of RI basins is due to their commonly large surface area and therefore limits this treatment process to areas where land is available, often smaller towns. The city of Kamloops possesses these requirements and, having always been an innovative city, has been running pilot-scale RI basins for over ten years. As of 1985, the City of Kamloops has a wastewater treatment plant which boasts tertiary treatment due to the addition of two pilot-scale rapid infiltration basins. Kamloops, located in the Thompson-Okanagan region, where the North and South Thompson rivers meet, has a semi-arid climate. The population is 80 000 but the city remains small enough to have plenty of open land on its outskirts. The conditions and location of Kamloops therefore make it a perfect location for natural wastewater treatment processes. Figure 1-1 is a photograph of the treatment plant from above; it illustrates the large area of land available to the treatment plant, as well as depicts the desert-like surroundings of Kamloops.

Figure 1-1. View of the City of Kamloops' wastewater treatment plant located on the south shore of the Thompson River.
The City’s wastewater treatment plant is located on the south shore of the Thompson river, on the western side of Kamloops. Its treatment train consists of a recently installed grit-removal chamber, anaerobic lagoons, followed by aerated lagoons, phosphorus precipitation using alum, chlorination and finally around 70 days retention in effluent storage basins, which provides final polishing in the form of settling. This train is depicted schematically in Figure 1-2.

The grit-removal essentially catches and removes all large objects, both organic and inorganic, from the incoming stream of raw wastewater; the grit is sent to the City’s nearby landfill. There are two anaerobic cells, known as 1A and 1B, with a combined area of 4.4 ha and depths of 4.6 m. The detention time of the wastewater in these cells is approximately 4 days. The cells provide primary settling as well as the anaerobic breakdown of organics. These cells are equipped with surface aerators which are sometimes used for odour control. Following these cells are three aerobic cells, 2A, 2B and 2C, which have a total area of 11.2 ha and depths of 4 m. The detention time here is longer, about 10 days, and the treatment provided is aerobic. As required by the types of organisms, these cells are fully aerated by diffused air units. Cells P1 and P2 follow the aerobic treatment; here, aluminum sulphate is added to the wastewater to precipitate phosphorus. The alum, Al₂(SO₄)₃, is added in liquid form to these cells which have a total area of 2.75 ha and depths of 3.5 m. This method of phosphorus removal is successful, but has its drawbacks as well.
The chemicals are expensive and the resultant sludge is high in aluminum content which can render it undesirable for land application. Following the phosphorus removal stage of the treatment train comes C1 which is the chlorine addition cell. This small cell is 0.8 ha, has a depth of 3.2 m and a 4 hour detention time. Chlorine contact is therefore 4 hours and a residual of 1.0 mg/L is sought. Finally the wastewater travels to cells 3 and 4 which provide approximate detention times of 25 and 45 days respectively. Through these long retention times, dechlorination is achieved before river discharge. Both cells contain river discharge pipes. Recently, a series of two constructed wetlands cells were built which take 2C water as influent; their effluent is collected, monitored, and pumped back into 2C. This is a pilot-scale project to investigate the ability of the cattails and bullrushes to treat the wastewater, primarily nutrient removal.

As mentioned, pilot-scale rapid infiltration basins, of approximately 2.5 ha in size, were constructed in 1985. Their design had been contracted out to Stanley Associates Eng. Ltd. These basins are located on the north shore of the river, directly across from the treatment plant. Next to them, a spray-irrigation system was constructed at the same time. The wastewater which comes across the river from cell 4 is split; one portion is taken through the RI system, while the second portion is again chlorinated and then sent to the spray irrigation system. The water is applied using large monitors to City-owned farmland which surrounds the area.

The rapid infiltration basins were constructed from a series of cut and fill sections at a specified location on the north shore of the river. Stanley contracted out NOWAK Geological Services Ltd. to study the soil and hydrogeology of the region. After a number of lab and site tests, the hydraulic conductivity (k) of the sand was calculated for the various layers of sand found at the site; the most appropriate layer with the highest permeability was selected. This layer had a k of $5 \times 10^{-5}$ m/s. The flood/dry ratio was chosen to be 1.0 and the basins began functioning with 14 days flooding and 14 days drying; this number was changed a few years later to 10 days each for flooding and drying and the basins are currently on this schedule. Due to the low winter temperatures common to Kamloops, there are no drying periods during the winter. This is to mitigate problems that could be caused by ice settling on and smothering the sand. Both basins therefore remain flooded for between 3 and 4 months, depending on the temperatures for any particular year.
The original design stated that the basins were to have a combined area of 3.7 hectares. Recent measurements, however, revealed that the west basin is 1.34 ha and the east is 1.11 ha, meaning a total area of only 2.45 ha. The basin floors have a slope of 0.238%; each basin slopes toward the road which runs between the basins, under which is located the 250 mm main collection pipe. The underdrainage systems under each basin connect to this main, which discharges directly into the Thompson river. Due to the slope of the basins, the depth of sand varies between 2 m and 2.5 m; the average sand depth upon construction was therefore considered to be 2.2 m. The basins were constructed to have a water depth of 30 cm; the maximum depth is limited to about 50 cm, due to the shallow berms. The nominal design capacity of each basin was 10 000 m$^3$/day, based on a basin area between 1 and 1.2 ha and with a combined safety factor of 10.

The RI basins, however, have never run according to their design. Average throughput is between 5000 and 7000 m$^3$/day, and the west basin consistently has a higher flow. Various techniques have been implemented in efforts to mitigate the throughput problems, but with no satisfactory results. Understandably, it is difficult to solve an undefined problem; insufficient testing has been done on the basins to discover the reasons behind their low flows or the differences between east and west.

Treatment through the basins has always been acceptable with most effluent parameters having concentrations below permit levels. Perhaps the main concerns for effluent entering the river are BOD and phosphorus concentrations. The former is removed naturally through the natural processes of the treatment plant including aerobic and anaerobic basins as well as extended residence-time holding cells. Phosphorus levels have not, to date, represented a problem, due to the phosphorus precipitation with alum being a step in the treatment train. The high costs, however, of the required alum are becoming prohibitive. The City of Kamloops spends up to $500 000 on alum alone per year. Furthermore, public concerns are being raised as to the effects of aluminum concentration in water and sludge. The sludge from the phosphorus precipitation basins is, as expected, high in aluminum and has therefore limited composting prospects. Thus, for the purpose of saving money and reducing the use of chemicals at the treatment plant, the City of Kamloops is searching for an alternative method of reducing the phosphorus levels in their wastewater. This goal, coupled with the aforementioned task of understanding and ultimately correcting the current basins' flow problems, are therefore the premises of this study.
1.2 Problems and Tasks

The utilities engineer for the City of Kamloops, Mike Warren, approached the UBC Environmental Engineering Department with his following concerns regarding the City's R1 system. Alum was becoming a problem both in terms of expense and public concern. The latter entails recent and rising concern over aluminum in water. Although aluminum levels in wastewater after alum addition are low (most of the Al is tied up in the sludge), public opinion can often effect more rapid changes than science. As mentioned, the sludge from P1 and P2 is high in Al and is therefore not necessarily appropriate for composting. Mr. Warren's second main concern was over the fact that the basins realize only one half of their design flow. Whether the problem is one of design, construction or operation, or a combination, remains to date unknown. Furthermore, the observed and consistent differences between east and west performance are baffling. To investigate these concerns, a proposal was written by the author for the City of Kamloops. It was suggested that a study be done which would address the current basin problems and which would investigate possibilities of discontinuing alum addition as a treatment step. The end result would include the following:

1. Recommendations for new basins which will include:
   i. Maximum safe (ie. winter conditions with highest viscosity) achievable discharge in m$^3$/day/hectare
   ii. Best sand depth
   iii. Most appropriate media
   iv. Best flooding/drying ratio
   v. Best water depth (ie. max. head) for max. throughput
   vi. Expected treatment, especially in terms of phosphorus

2. Results of phosphorus findings through column tests. For example, the most appropriate influent phosphorus concentration. As well, the P removal capacity of the river sand. If all of the phosphorus cannot be removed, other possibilities will be suggested.

3. An idea for increasing the throughput is to cut away a portion of the sand. This requires an investigation into the current flow patterns. For instance, is the flow surface
determined? This query can be answered through an investigation of the piezometers which were placed in the basins in early 1995.

4. Recommendations on the future operational procedures will be noted. These could involve problems encountered or witnessed at the treatment plant or in the lab.

The proposal was accepted by the City of Kamloops and the author was subsequently commissioned to begin the study.
Chapter 2 - Literature Review

2.1 General
The treatment of wastewater by a rapid infiltration system is performed by biological, chemical and physical interactions within the basin sand, most of the treatment being accomplished in the near surface layer (Reed and Crites, 1984). Flow through a filter can be saturated or unsaturated, depending upon the solids accumulation at the surface, the sand itself and the underdrainage, or collection, system underneath the sand.

2.2 Treatment Through a Sand Filter
The purification of wastewater by sand works in a number of ways. One such way is surface straining, or interception, an obvious method for the capturing of particles too large for the sand’s interstices. According to Huisman and Wood, a tightly packed bed of spherical grains can capture particles which are only 15% of the sand grain’s diameter. The following illustrates the ideal case:

\[ d_0 = 0.155d \]

where \( d_0 \) = 0.155d

Figure 2-1. Huisman and Wood’s theory of particle screening.

As the filter runs and particles are captured, the screening capability is enhanced until hydraulic throughput is significantly decreased, at which point the filter must be rested.
Interception is approached in another way both by O’Melia and Stumm and by Yao et al., who write that the dominant transport mechanisms involved in RI systems are interception, sedimentation (i.e. gravity) and diffusion. The equations describing these mechanisms rely predominantly on grain size and approach velocity and can be written as follows:

For interception: \[ \eta_I = \frac{3}{2} \left( \frac{dp}{dc} \right)^2 \]  \hspace{1cm} (2-1)

where: 
- \( dp \) is the particle diameter
- \( dc \) is the collector diameter
- \( \eta_I \) is the single collector efficiency for interception

For sedimentation: \[ \eta_g = \frac{(\rho_p - \rho)gd_p^2}{18\mu v} \]  \hspace{1cm} (2-2)

where 
- \( \rho_p \) is the particle density
- \( \rho \) is water density
- \( \mu \) is absolute viscosity
- \( \eta_g \) is the single collector efficiency for gravity sedimentation

For diffusion: \[ \eta_D = 0.9 \left[ \frac{kT}{\mu d_p dc} \right]^{2/3} \]  \hspace{1cm} (2-3)

where
- \( T \) is temperature (Kelvin)
- \( k \) is Boltzmann’s constant
- \( \eta_D \) is the single collector efficiency for diffusion

Source: Yao et al., 1971

Inspection of these equations illustrates that in RI systems, the smallest particles get deposited by diffusion while large ones are deposited either by gravity or interception.
Another important mechanism is adsorption, particularly for phosphorus removal; this, however, relies on proximity, as Van der Waal's forces function only once the particle is within range. A discussion of phosphorus adsorption is contained in the following section.

2.2.1 Removal depth
Sand filters are known to be extremely efficient in the removal of bacteria and viruses from water or wastewater (Logsdon, 1991 and McConnell et al., 1984). It has been found by various studies involving sampling at many depths in a sand column or basin, that bacterial distribution throughout a sand filter is not uniform. The practical zone for bacterial reduction is the top 100 mm (Logsdon, 1991). McConnell et al. (1984) claim that 65% of virus removal occurs in the top 35 cm of a sand column; below this, there is continuous removal, but not as much as in the top layer. Lance et al. (1980) found that most of their fecal coliform removal occurred in the first 8 cm of soil; virus removal occurred predominantly in the top 20 cm.

Exact removal depths are not specified for bacteria and viruses. A significant portion of the purification in general of secondary effluent is accomplished in the upper layers of the sand beds (Aulenbach et al., 1978). Guilloteau et al. (1993) displayed removal curves of concentration versus depth which illustrated that most of the treatment of COD, SS, N and P occurred in the surface zone between 0-15 cm. They recommend, however, a deeper sand depth of 0.5 m for further removal of carbon and for nitrification. They also recommend an unspecified greater depth for bacteria and virus removal. The EPA claims, however, that nitrification occurs in the top 10 cm of soil, where the aerobic nitrifying bacteria are located. Guilloteau pinpointed the depth of biologically active substrate at 30 cm.

2.2.2 Percent removals
Purification efficiency differs from study to study, as it depends upon the type of sand, wastewater and operating parameters. Listed below in Table 2-1 is the treatment, in terms of percent removals, observed by various studies. Not all studies follow the same criteria or test for the same parameters; some entries are therefore left blank.
Table 2-1. Removal capabilities as listed by various sources.

<table>
<thead>
<tr>
<th>study</th>
<th>sand depth</th>
<th>BOD&lt;sub&gt;5&lt;/sub&gt;</th>
<th>COD</th>
<th>SS</th>
<th>N</th>
<th>P</th>
<th>colif.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nielson et al.</td>
<td>0.8 m</td>
<td>90-95</td>
<td></td>
<td></td>
<td>30-45</td>
<td>40-60</td>
<td></td>
</tr>
<tr>
<td>Oesterholt and Bult</td>
<td>2 m</td>
<td>20</td>
<td>56</td>
<td>22</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lance et al.</td>
<td>2.5 m</td>
<td></td>
<td></td>
<td></td>
<td>28.5</td>
<td>58-73</td>
<td>99</td>
</tr>
<tr>
<td>Levine</td>
<td>1 m</td>
<td>81-98</td>
<td>95-99</td>
<td>50-99</td>
<td>7-98</td>
<td>99</td>
<td></td>
</tr>
<tr>
<td>Reed and Crites</td>
<td>90-95</td>
<td></td>
<td></td>
<td></td>
<td>98</td>
<td>60</td>
<td>99</td>
</tr>
<tr>
<td>EPA</td>
<td>92</td>
<td></td>
<td></td>
<td></td>
<td>50</td>
<td>29-99</td>
<td>99</td>
</tr>
<tr>
<td>Bouwer and Rice</td>
<td>2.4 m</td>
<td></td>
<td></td>
<td></td>
<td>91</td>
<td>22</td>
<td>93</td>
</tr>
</tbody>
</table>

As is evident from Table 2-1, there is a wide variety of expectations from RI systems, depending upon the source. Some of these differences involve sand depth; particularly with nitrogen removal, the depth of sand plays the key role regarding the stage of nitrification or denitrification the wastewater will undergo.

2.2.3 Effect of loading duration on treatment capability

Within a range, Nielson et al. found that neither the hydraulic nor the organic loading had any effect on the filter’s purification ability. Oesterholt and Bult (1993) also found no direct relation between removal capacity of the filter and filtration rate. Guilloteau et al. (1993) write that the duration of flooding had no influence on the biomass content. NOWAK (1983), however, claims that lab soil column tests have shown that phosphorus removal is related to the infiltration rate, for a given phosphorus concentration. A set of tests showed that the highest soil sorption capacity was obtained under the highest application rate. Previous data by Lance (1977) and by Lance et al. (1980) disagreed with NOWAK’s findings; they said that phosphorus removal could be increased by decreasing the infiltration rate.

Reed and Crites (1984) have found that the hydraulic loading rate can affect nitrogen as well as phosphorus removals. With regard to nitrogen, they witnessed a 30% removal at a loading of 30 cm/day and a removal of 80% at 15 cm/day. Apart from nitrogen and phosphorus, however, they claim that treatment performance by a sand filter is independent of infiltration rate.
2.2.4 Nitrogen removal in sand filters

The primary removal mechanisms for nitrogen are nitrification and denitrification; these are accomplished by soil bacteria. The optimum temperature for nitrification according to the EPA is between 30 and 35°C. As the temperature decreases, so does the rate of nitrification. With regard to pH, both nitrification and denitrification rates decrease as the pH decreases. Nitrification stops completely at pH < 4.5, while denitrification is greatly reduced at pH < 5.5. For denitrification, a long detention time within the filter is required, as well as a sufficient amount of organic carbon (Reed and Crites, 1984). Carbon limitation on nitrogen removal in a sand filter can be approximated by the following equation:

\[
N = \frac{(TOC - 5)}{2}
\]

(2-4)

where: TOC is the total organic carbon, mg/L
N is the change in total nitrogen, mg/L

To maximize the nitrogen removal, the flooding period must be long enough for the sand's bacteria to deplete the oxygen, thereby producing an anaerobic environment.

2.2.5 BOD and suspended solids removal

The three removal mechanisms of BOD in a sand filter are mechanical filtration, adsorption and biological oxidation, while suspended solids (SS) are removed primarily by filtration. Most of the BOD removal occurs at or near the surface of the filter, as this is where the microbial activity is the most intense (Reed and Crites, 1984). The drying period of the basin is an important aspect of BOD removal; the restoration of aerobic conditions which occurs during the drying period is required for biological oxidation. The SS and BOD which get trapped in the soil surface are eventually degraded and consumed by soil bacteria (EPA, 1981).

2.3 Phosphorus Removal in Sand Filters

There are three forms of phosphorus (P) usually found in aqueous solutions: ortho-P, poly-P and organic phosphate. Of these, phosphorus occurs mainly as ortho-P. Since poly-P soon undergoes
hydrolysis in aqueous solutions and reverts to ortho-P and the amount of organic phosphate is very small in domestic wastewater, it is common to measure only ortho-P in a filtrate sample (Metcalf and Eddy, 1991). Measurements of ortho-P are frequently so close in value to those of total P, that only one of these need be measured (Lance, 1977).

2.3.1 Removal mechanisms

Phosphorus removal is strongly controlled by the properties of the applied wastewater and by the sand's characteristics (Aulenbach and Meisheng, 1988). The major P removal processes in natural treatment systems, such as rapid infiltration, are chemical precipitation and adsorption. Adsorption is considered by many to be the predominant phosphorus removal mechanism and is a fairly rapid reaction. The other, slower, P removal mechanism is precipitation in which P is converted into insoluble compounds by crystallization. Phosphorus adsorption is dependent upon the ionic strength of the column influent; it depends predominantly on the sulphate and bicarbonate content, as these are the species which compete with the PO$_4^{3-}$ ion for available adsorption sites (NOWAK, 1983). These P ions can be adsorbed by clay minerals as well as certain organic soil fractions in the soil matrix (Metcalf and Eddy, 1991). Aulenbach and Meisheng (1988) maintain that it is the coating on the sand and not the sand grain itself that is responsible for P adsorption; this coating may consist of complex compounds, both organic and inorganic.

Precipitation of P by metal ions is a slower process which basically takes over for the adsorption which is only responsible for initial P removal (Aulenbach and Meisheng, 1988). Some of the metal ions which are able to precipitate P include Al$^{3+}$, Fe$^{3+}$, Mg$^{2+}$ and Ca$^{2+}$. Calcium is the most important of these ions, particularly at a pH above 7.

A point of contention among many researchers is whether or not P is taken up by biological means within a filter, and if so, to what degree. Metcalf and Eddy maintain that "plants do take up some", while Sundaresan and Paramasium (1982) found that the reduced numbers of algae did not decrease the quality of effluent, which illustrated the fact that phosphorus uptake was not accomplished by algae. Lance et al. (1980) generalized this point by stating that their data showed no indication that increased organic content in the applied wastewater had any effect on P removal. An extensive
study of P removal mechanisms was performed by Aulenbach and Meisheng; they concluded that biological activity was not the predonimant factor in P removal. Sterile and non-sterile infiltration columns were set-up and the effluent P concentration monitored. Removals were slightly higher in the biologically-active column, but this was attributed to the fact that there was longer retention time in this column. For any study in which it is noted that P removal is partially due to the presence of biological life, it is difficult to pinpoint the exact cause of this increased removal. It could be due to uptake of the nutrient by organisms for growth, but it could equally be due to the filter's increased straining ability caused by the presence of organisms.

2.3.2 Equations
Reed and Crites (1984), in their Handbook of Land Treatment Systems for Industrial and Municipal Wastes, have established an empirical equation which predicts phosphorus removal in rapid infiltration systems:

\[ P_x = P_o e^{kt} \]  

(2-5)

where:  
\( P_x \) is the concentration of phosphorus at distance \( x \) from the sand surface  
\( P_o \) is the initial phosphorus concentration  
\( k \) is an instantaneous rate constant which, at neutral pH, equals 0.002 h\(^{-1}\)  
\( t \) is the detention time and is equal to \( 0.4X/I \)

where:  
\( X \) is the distance along the flow path  
\( I \) is the infiltration rate

It is suggested that if this equation provides satisfactory results, meaning that \( P_x \) is below that limit sought, then actual operating concentration will be satisfactory. Otherwise, if the resulting \( P_x \) is above the required limit, then actual phosphorus adsorption tests should be performed to see the capability of the soil. Results of these adsorption tests, however, should be multiplied by 5 to account for slow precipitation that occurs over time, and therefore is not recorded by the adsorption test itself.

In order to predict the movement of contaminants through soils, dispersion as well as velocity effects must be studied. An intensely mathematical approach which describes these factors can be found in Zarnett, 1976. Less theoretical and more applied is the
study of sorption isotherms. A sorption isotherm expresses, for a given and constant
temperature, the relationship between the amount of phosphate sorbed per unit of soil and
the concentration of phosphate in solution. The Langmuir equation is a widely used
equation which describes a soil’s ability to retain phosphorus (Zarnett, 1976):

\[ x = bx_m c / (1 + bc) \]  

(2-6)

where:  
- \( x \) is the amount of phosphorus sorbed in \( \mu g/g \) material
- \( x_m \) is the sorption maximum
- \( c \) is the residual phosphorus concentration
- \( b \) is the bonding energy index

Equation (2-6) can be rearranged and plotted in order to calculate the parameters more
easily:

\[ \frac{c}{x} = \frac{c}{x_m} + \frac{1}{b}x_m \]  

(2-7)

A plot of \( c/x \) versus \( c \) will yield a slope equal to \( 1/x_m \) and a y-intercept of \( (1/b)x_m \).

Also used is the Freundlich equation:

\[ x = kc^n \]  

(2-8)

Again, a plot of the rearranged equation (2-8):

\[ \log x = \log k + n \log c \]  

(2-9)

yields a straight line with slope \( n \) and y-intercept \( \log k \). The value for \( x \) in equations (2-6)
through (2-9), in units of \( \mu g/g \) of material is known as the soil sorption capacity (SSC) and
will be explained in the following paragraph.

2.3.3 Soil sorption capacity

A rapid infiltration basin will remove phosphorus from wastewater up to a finite point,
termed saturation. The adsorption capacity for P in soils, though finite, can be quite
large, even for sandy soils (Metcalf and Eddy, 1991). The phosphorus saturation point of
a soil can be found by calculating its soil sorption capacity (SSC) which is a ratio of the
mass of phosphorus to mass of soil, in μg/g. As phosphorus removal through a sand filter is time-dependent, a useful way to analyze P removal data is to plot a breakthrough curve; this is a curve of c/c₀ versus time, onto which is overlayed (c/c₀)crit. The first ratio represents the effluent P concentration divided by the influent P concentration; the critical ratio represents the required (for example, by permit) effluent P concentration divided by the influent P concentration. The point in time at which the curve of the former ratio crosses that of the latter is known as t_crit, and represents the point after which the filter is no longer performing satisfactorily, as the effluent P concentration in the filtrate is above permit requirements. This time value is then used to calculate the SSC; also required are values for the influent P concentration (P₀), the daily loading and the dry weight of the soil. The following equation therefore defines the soil sorption capacity:

\[
\text{SSC in } \mu g/g = \frac{(t_{\text{crit}}) \times (P_0) \times \text{(daily loading)}}{\text{soil mass}} \quad (2-10)
\]

According to NOWAK (1983), the SSC of a soil is dependent on the P concentration of the influent, as well as its ionic strength. It was discovered that changing the ionic strength of a column influent produced an increase in the soil's P sorption capacity; this increase was, however, short-lived. NOWAK also found no relationship between dosing frequency and SSC.

What happens when a basin or column of sand has reached this saturation point is also of interest. Metcalf and Eddy (1991) say that adsorbed phosphorus is held tightly and is therefore generally resistant to leaching. To study phosphorus desorption, Zarnett ran distilled water through a column close to phosphorus saturation and found that the water samples did not release phosphorus, but retained a significant quantity. These results illustrate the fact that it is not solely physical adsorption which is removing phosphorus from wastewater.

2.4 Hydraulics of RI Systems

As was previously mentioned, flow through RI systems can depend on a number of things. The suspended solids within the influent, the sand depth and characteristics and the collection system all contribute to defining the throughput.
2.4.1 Biological activity (algae) in sand filters

Sand filters, being natural systems, house significant biological life. During times of warm weather and longer hours of sunlight, algae blooms can persist. These are problematic for two reasons. The first is the fact that the cells themselves, as solids, clog the surface of the sand. Secondly, algae use CO₂ for photosynthesis which results in an increase in pH and therefore precipitation of CaCO₃. This, too, leads to clogging.

The algal distribution is predominantly in the top 1 cm of sand; below 8 cm, there is practically no trace (Bowles et al., 1983). This distribution of algae was found not to be dependent upon grain sizing or hydraulic loading. Due to the surface matting effect and quantity of algae blooms, hydraulic throughput within the basin can be greatly reduced. Filter shading is an appropriate means of improving the filter run duration by reducing the sunlight required by algae; furthermore, it does not affect filtrate quality (Sundaresan and Paramasium, 1982). Metcalf and Eddy (1991) warn that algal solids can quickly cause severe and permanent clogging of the soil surface and is known to be one of the major causes of RI system failure.

2.4.2 Saturated hydraulic permeability, k, and infiltration rate

The determination of a sand’s permeability is the first and paramount step in the design of an RI system; almost all other design parameters are influenced by this. Darcy’s equation is the best field method for the determination of the saturated hydraulic permeability, k. Other methods, using properties such as grain size, are often less accurate due to their not taking certain other, potentially important, properties into account. One such method is the Hagen-Poiseuille equation:

\[ k = N d^2 g \rho_w / \mu \]  

(2-11)

where:
- \( k \) = saturated hydraulic conductivity
- \( N \) = dimensionless shape factor
- \( d \) = mean grain diameter
- \( \rho_w \) = density of water
- \( \mu \) = viscosity (temperature dependent)
Now, both the ionic composition of the water and entrapped air can affect k, but are not revealed when using a method such as Hagen-Poiseuille. One in-situ slug test of permeability is by use of the Hvorslev method (Stanley, 1982). Data from slug tests can be analysed using the Hvorslev plot of \( \log H/H_0 \) versus time. From this, one can then calculate k from the Hvorslev eq’n:

\[
k = \frac{r_c^2 \ln(L/R)}{2LT_0}
\]  
(2-12)

where:  
- \( r_c \) = radius of casing  
- L = length of screen or open hole  
- R = radius of screen or open hole  
- \( T_0 \) = basic time lag from Hvorslev plot

The most widely used expression is that of Darcy, whose equation is as follows:

\[
q = \frac{Q}{A} = k \frac{dh}{dl}
\]  
(2-13)

where:  
- q = flux in m/s  
- Q = flow in m\(^3\)/s  
- A = cross-sectional area in m\(^2\)  
- k = saturated hydraulic conductivity  
- \( \frac{dh}{dl} \) = hydraulic gradient, ie. change in head over a distance 1

For the testing of k in a lab, common methods are the constant-head method or falling-head permeameters. Problems with lab instead of field testing include the small size and the disturbance of the sample. Under saturated conditions, k is considered a constant (EPA, 1981). In reality, it can change over time due to increased swelling of clay particles or changes in pore size distribution. Because these changes are not large, and for the sake of simplicity, most analyses consider the saturated k to be a constant.

The infiltration rate is the rate at which water enters a soil from its surface. Initial infiltration rates into dry soil are typically higher than those reached at steady-state because of more empty pore space being available. When the soil is saturated, with no surface ponding, the infiltration rate is approximately equal to the effective saturated
hydraulic conductivity. There exist in a soil two conductivity values, one being the vertical conductivity ($k_v$) and the other the horizontal conductivity ($k_h$). These two values are rarely equal due to soil stratification and particle orientation; it is most often the case that $k_h > k_v$, sometimes by a factor of over 10 (EPA, 1984). This condition is known as anisotropy (isotropy would therefore indicate that $k_h = k_v$).

2.4.3 Soil characteristics
Before a site for a rapid infiltration basin can be selected, a comprehensive study on soil structure and characteristics must be conducted. Soil structure is defined as the aggregation of soil particles into clusters, known as peds. A site must be tested to ensure that the application of wastewater will not negatively alter or destroy the soil structure. Appropriate soils for RI systems are fine-textured with strong peds (EPA, 1984). Soil colour can also indicate the suitability of a soil as potential RI media by revealing drainage characteristics. Red, yellow and yellow/brown soils imply well-aerated, non-saturated soils, while grays and blues suggest that insufficient oxygen is being received.

2.4.4 Flood/dry schedule
Basins are typically operated on a flood/dry schedule with ratios being chosen according to specific sites and hydraulic loading requirements. The purpose of the drying period is to renew the surface aerobically and to break up the surface mat of organics (the Schmutzdecke), usually by machine ripping. As stated by Reed et al. (1985), rest periods are required to allow the surface to dry and recover its original infiltration capacity. Levine (1984) includes the improvement of wastewater treatment efficiency as a reason for alternating application and drying periods.

2.4.5 Comparing SSF and RSF
There are a number of features which distinguish a rapid sand filter from a slow sand filtration (SSF) system, commonly used as a treatment system for drinking water. Rapid infiltration systems characteristically lie somewhere in between the two, but are more similar to slow sand filters. The typical differences, according to Logsdon, are listed below in Table 2-2.
Table 2-2. Common differences between slow and rapid sand filters.

<table>
<thead>
<tr>
<th>feature</th>
<th>slow sand filtration</th>
<th>rapid sand filtration</th>
</tr>
</thead>
<tbody>
<tr>
<td>application rate</td>
<td>0.1 m/h</td>
<td>10 m/h</td>
</tr>
<tr>
<td>water depth</td>
<td>1.5 m</td>
<td>1.5 m</td>
</tr>
<tr>
<td>sand depth</td>
<td>800 mm</td>
<td>800 mm</td>
</tr>
<tr>
<td>retention time in sand bed</td>
<td>3.2 hours</td>
<td>2 min</td>
</tr>
<tr>
<td>cycle length</td>
<td>1-6 months</td>
<td>1-4 days</td>
</tr>
<tr>
<td>chemicals added?</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>prechlorination?</td>
<td>no</td>
<td>yes</td>
</tr>
<tr>
<td>cleaning method</td>
<td>scraping</td>
<td>backwashing</td>
</tr>
</tbody>
</table>

The RI system in Kamloops was designed to treat 10 000 m$^3$/day on the basis of one filter working at a time. The average surface area of one filter is 1.2 ha, which translates into an expected 0.83 m/day, or 3.5 cm/hr. This design rate is considerably slower than the rate suggested even for slow sand filters. The similarity of the Kamloops system to SSF does not end with hydraulic throughput. Rapid sand filtration systems are generally backwashed when the surface organics layer has built up sufficiently to impede flow. In Kamloops, however, the basins are dried and scarified to break up the Schmutzdecke. Chemical coagulants are often added to rapid sand filter influent to speed up the process of solids removal; in slow sand filtration, however, the water has enough time within the column to be freed of its solids by mechanisms within the sand. In Kamloops, this is not specifically the case; alum is added for the removal of phosphorus approximately 50 days before the wastewater reaches the sand filter. Technically, therefore, no coagulants are added immediately prior to filtration; however, the influent to the basins is already very low in suspended solids. Table 1 also shows that prechlorination is common in RSF systems and not in SSF, the latter therefore allowing biological life to thrive. Once again in Kamloops this is not specifically the case. The basin influent is chlorinated around 50 days prior to filtration; during this time lag, a limited amount of biological life is created as the wastewater is detained in lagoons and therefore sustained within the RI system. It would seem, therefore, that the Kamloops system is a pseudo-rapid infiltration system, having some features which resemble typical RI systems, but also some which resemble slow sand filters.
2.4.6 Use of columns for testing

Pilot study results represent a relatively conservative prediction of the expected effluent from a full-scale system (Levine, 1984). To study sand filters on a small scale, columns are often used. The size and material of these columns varies from study to study, but there are a few points which should be noted. First of all, columns should be chosen such that the ratio of column diameter to particle diameter is at least 50:1 to minimize wall effects (Liljestrand and Parten 1993). Columns should be backflushed in order to remove air pockets; this allows the wastewater to gravity-drain freely. Finally, Reed and Crites (1984) state that the measured flow rate through a column is approximately five times greater than it will be through the full-sized sand basin. This must therefore be taken into account when projecting column test results to full-scale operation parameters. Metcalf and Eddy (1991) recommend that infiltration tests be performed under ‘worst-case’ conditions; this entails conditions of minimum evaporation - cold, calm and cloudy.

2.4.7 Use of piezometers

In the EPA’s Design Manual, the use of piezometers as vertical flow indicators is suggested. Piezometers are commonly used for the monitoring of groundwater, but can also be used in rapid infiltration basins. This requires the installation of a series of piezometer tubes which extend within the filter sand to different depths. Subsequent measurement and comparison of the water levels within the tubes can divulge useful information about the hydraulics of the filter. The following figure is taken from page 3-30 of the Design Manual and illustrates examples of what the piezometers can indicate with regard to groundwater. These ideas can be translated to apply to RI systems.

![Figure 2-2. Vertical flow direction as indicated by piezometers (EPA, 1981).](image-url)
In the first group above, the piezometers indicate that the ground water is going down and that there is therefore drainage. In the second, the piezometers indicate a hydrostatic pressure or that there is water coming up from a deeper strata (in RI systems, this could indicate a blockage of the underdrainage system). In the third frame, the piezometers indicate a hydrostatic pressure in a stratum and that water is being forced both up and down from the stratum. Finally, the piezometers in the fourth frame indicate that ground water is moving into the stratum and going out of the area.

2.5 Design Recommendations

2.5.1 Permeability

Metcalf and Eddy (1991) recommend that for successful RI systems, soils should have a minimum permeability \( k \) of \( 0.7 \times 10^5 \) m/s. They also warn against cut and fill construction, such as that used in Kamloops, as it can adversely affect the permeability of the surface soils. Cheung et al. (1994) add that sand with medium permeability may retard the movement of pore waters which could allow for longer times for sorption to occur; they do not, however, specify what ‘medium’ permeability may be. Stanley (1991) states the following common \( k \)-values in m/s:

- coarse sand: \( 9 \times 10^7 \) to \( 6 \times 10^3 \)
- medium sand: \( 9 \times 10^7 \) to \( 5 \times 10^4 \)
- fine sand: \( 2 \times 10^7 \) to \( 2 \times 10^4 \)

Their selected permeability value of \( 5 \times 10^5 \) m/s in the design of the Kamloops systems falls within the larger end of the medium sand bracket, or within the middle of the coarse sand values.

As a sand filter progresses in its flooding phase of a cycle, its infiltration rate slowly decreases due to clogging. Its subsurface vertical permeability at saturation, however, should remain constant (Reed and Crites, 1984). In terms of measuring the permeability, \( k \), many methods have been developed. The air-entry permeameter (AEP), developed by Bouwer, can measure \( k \) without requiring that a water table be present. As mentioned earlier, \( k_h > k_v \); a conservative approach in design would therefore be to say that \( k_h = k_v \); this also precludes the need to measure \( k_h \) in the field. If necessary, there are methods of
determining $k_h$. There exists a lot of data on measured relationships between $k_v$ and $k_h$ for specific sites. For instance, existing data on river bed sand reveal $k_h$ values of 72 m/d and 86 m/d, with $k_h/k_v$ values of 10 and 16 respectively (EPA, 1981). If confident of such data, one can estimate a relation between $k_h$ and $k_v$, but on the safer side, there are methods of actual field measurements of the horizontal conductivity. The auger hole technique combined with piezometer measurements allows $k_h$ to be found in anisotropic soils, as does the slug test method. Details of all methods can be found in the EPA’s Process Design Manual for Land Treatment of Municipal Wastewater.

2.5.2 Underdrain system and mound height analysis

The sizing of the underdrainage system is crucial to ensure that it not be responsible for any blockage in flow. Use of the Hazen-Williams nomograph can determine the flow capability given a slope, pipe diameter and material (Khan, 1987). Underdrain spacing is entirely site specific and varies according to hydraulic throughput as well as economic aspects. Reed and Crites (1984) provided some generalized numbers; underdrains are typically spaced 15 m or more apart, at depths between 2.5 and 5 m. As well, they are free flowing pipes as opposed to pressure pipes. Depending on the spacing chosen for a system, a certain amount of water accumulation, termed mounding, will result.

Mounding of water will occur when there is insufficient gradient to move the water away from beneath the basin in a lateral direction (EPA, 1981). It will also result when underdrains are present, as in the Kamloops system (Luthin, 1978). Mounding occurs in between drains, such that the maximum mound height is located at a $D/2$, if $D$ is the spacing between subsequent drains. The degree of mounding depends on the slope, the infiltration rate, the permeability of the media and the drain spacing. Temporary mounding within a drainage system is acceptable provided there is no interference with infiltration at the surface. There exists a number of different models which calculate mounding; each of them assumes saturated Darcy flow and homogeneous media. The most commonly used model is Hooghoudt’s equation:

$$S^2 = 4k (H^2 - h^2 + 2dH - 2dh) / v \quad (2-14)$$

where $S = \text{drain spacing}$
\( k = \text{permeability} \)
\( H = \text{mound height} \)
\( h = \text{drain height} \)
\( d = \text{distance between underdrain pipe and impermeable layer} \)
\( v = \text{infiltration rate} \)

For system design, use of Hooghoudt's equation can vary. One may begin with a desired \( S \) and find \( H \), or conversely, one may know the maximum desired \( H \) and calculate the required \( S \).

### 2.5.3 Soil characteristics

With regard to soil characteristics, non-uniform soils require a more complex site investigation (EPA Manual, 1984). As were found in the investigation of the Kamloops site, layers of soil with distinctly different permeabilities may be uncovered, some with a prohibitive clay content. A clay fraction of over 10% in the soil is undesirable, as the remolding of this clay during cut and fill procedures may reduce the conductivity to an unacceptable level (Reed, 1982).

A full analysis of the soil's properties must be performed. Of the hydraulic properties, it is necessary to find the infiltration rate as well as the subsurface permeability. In terms of chemical properties, the pH, CEC and P adsorption capabilities are of interest. Finally, a depth of profile is required for information about the soil's physical properties.

To test a site acceptably, thereby enabling the aforementioned properties to be discovered, a number of procedures can be followed, as suggested by the EPA's Design Manual. Test borings must be done on the actual site to be used, with one boring performed per major soil type. For systems which are less than 5 ha, between 4 and 6 shallow borings spaced over the entire site are sufficient. As well, the digging of test pits is strongly recommended for all RI site investigations. Test pits allow one to see the subsurface profile; a minimum of 2 pits is therefore recommended, even for small sites. Finally, a minimum of one basin infiltration test on each major soil type in the potential RI area should be done. The groundwater table and depth to the closest impermeable
layer may be required for later use in a mounding analysis, in order to see if underdrains are required.

2.5.4 Application rate

The most critical design aspect of an RI system is the determination of the design annual hydraulic loading rate. This rate is based directly on field and lab test results for infiltration, permeability and hydraulic conductivity. If flow maximization is the goal of an RI system, then as a general rule, one can set the unit application rate to be less than 0.5 $k_v$ to allow for the initial clogging of suspended solids (EPA, 1984).

The EPA’s Process Design Manual suggests a wide range of application rates; from its case studies, the numbers ranged from 30 to 110 m/year, with weekly loading rates ranging between 10 and 240 cm. Lower hydraulic loads, however, allow much more oxidation (Brissaud and Lesavre, 1993). Reed and Crites (1984), after their experiments with N removal, recommend a maximum application rate of 15 cm/day when an 80% reduction in nitrogen is desired.

The design hydraulic loading rate of a rapid infiltration system depends on the infiltration rate and on the treatment requirements. It can be calculated as a percentage of the test infiltration rate, which, as previously mentioned, is approximately 5 times higher than what is expected in practice. If a field $k$ was determined, then the annual hydraulic loading rate is usually limited to between 4 and 10% of $k$ (EPA, 1981). This percentage becomes the system’s safety factor.

The hydraulic loading rate can also be calculated on the basis of treatment needs. These calculated rates can then be compared and the lowest of the two selected as the design hydraulic loading rate. The application rate must be known in order to determine the required hydraulic capacity of the piping, or the discharge capacity. Reed and Crites (1984) and the EPA (1984) provide a method of determining the application rate which involves five steps:

1. Determine the time of one complete cycle (flooding plus drying days).
2. Find the number of cycles per year.
3. Divide the hydraulic loading rate, described in the previous paragraph, by the number of cycles to get a loading/cycle value.
4. Divide the loading/cycle by the application period for the application rate in m/day.
5. Multiply the area in m\(^2\) by the application rate in m/day to yield the discharge capacity.

### 2.5.5 Land requirements

Reed and Crites (1984) have provided an equation which allows for the calculation of the land requirements of an RI system which is being designed. The equation is straightforward, relying basically on balancing the units of both sides; it has been translated for use here into metric units:

\[
A = \frac{365Q}{L_w} \quad (2-15)
\]

where:

- \(A\) = area in m\(^2\)
- \(Q\) = design flow in m\(^3\)/day
- \(L_w\) = annual hydraulic loading in m/yr

The EPA’s Design Manual says that, in typical EI design, land requirements range between 3 and 23 ha.

### 2.5.6 Water depth

According to Metcalf and Eddy (1991), the accumulated water depth in an RI basin should not exceed 30-45 cm, in order to minimize compaction of the surface layer. Due to soil clogging and algae growth, the EPA (1981) suggests a depth of 30 cm.

### 2.5.7 Flood/dry schedule

Basins are typically run on a flood/dry schedule with ratios being chosen according to specific sites and hydraulic loading requirements. The purpose of the drying period is to renew the surface aerobically and to break up the surface mat of organics (the Schmutzdecke), usually by machine ripping. The duration of the drying period depends on both climatic conditions and on suspended solids loading (Brissaud and Lesavre, 1993).
There are many different recommendations with regard to both length of flooding and drying periods and as well as the ratio between the two. Table 2-3 below summarizes the recommendations proposed by various studies.

Table 2-3. Summary of flooding and drying schedules

<table>
<thead>
<tr>
<th>study</th>
<th>purpose</th>
<th>flooding period</th>
<th>drying period</th>
<th>ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Reed and Crites</td>
<td>nitrification</td>
<td>1 day</td>
<td>5-10 days</td>
<td>&lt; 1.0</td>
</tr>
<tr>
<td>Reed and Crites</td>
<td>denit/max. flow</td>
<td>1-3 days</td>
<td>4-5 days</td>
<td></td>
</tr>
<tr>
<td>NOWAK</td>
<td>inf. rate</td>
<td>14 days</td>
<td>14 days</td>
<td>1.0</td>
</tr>
<tr>
<td>Nielson et al.</td>
<td>practicality</td>
<td>7-14 days</td>
<td>7-14 days</td>
<td>1.0</td>
</tr>
<tr>
<td>Guilloteau et al.</td>
<td>O₂ recovery</td>
<td></td>
<td></td>
<td>0.5</td>
</tr>
<tr>
<td>Lance</td>
<td>inf. rate</td>
<td>9 days</td>
<td>5 days</td>
<td>1.8</td>
</tr>
<tr>
<td>Metcalf and Eddy</td>
<td>inf. rate</td>
<td>1-3 days</td>
<td>4-5 days</td>
<td>&lt; 0.5</td>
</tr>
<tr>
<td>EPA</td>
<td>inf. rate/nit.</td>
<td>1-3 days</td>
<td></td>
<td></td>
</tr>
<tr>
<td>EPA</td>
<td>max N removal</td>
<td>7-9 days</td>
<td>10-15 days</td>
<td></td>
</tr>
<tr>
<td>Levine</td>
<td>inf. rate</td>
<td>≥ 7 days</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Reed and Crites (1984) are firm in their statement that flood/dry ratios may vary from system to system, but are always less than 1.0, meaning that the drying period always exceeds the flood. Nielson et al. suggest that rest and load periods be of equal length and last between one and two weeks; their recommendation stems however from practicality and not from purification or flow considerations. Guilloteau et al. (1993), however, recommend drying for twice as long as flooding for the sake of oxygen recovery. This was discovered by monitoring the O₂ levels during filter runs; the drying period fulfills the function of air renewal in the column. NOWAK (1983) believes that it is possible to increase the phosphorus adsorption capacity of a soil by optimizing a dosing frequency; this in turn should promote the regeneration of adsorption sites. They found, however, no relationship between the dosing frequency and the soil sorption capacity. The RI systems referenced by Metcalf and Eddy (1991) had substantially longer drying than flooding periods. The authors recommended an application period of no longer than three days if the objective of the system is to maximize the infiltration rate of secondary effluent. Levine (1984) found that short drying periods, between 3 and 4 days, had a negative effect on infiltration rates. He therefore recommends a minimum of 7 days drying in order to maintain basin infiltration rates between 3 and 5 cm/h; this longer period also favours complete denitrification. Lance (1977) recommends 9 days flooding and 5 days drying.
When nitrification is the desired result, the loading cycle should consist of a short flood, about 1 day, followed by a long drying period, between 5 and 10 days. For N removal, i.e. complete denitrification, a flood/dry ratio between 0.5 and 1.0 should be tried (Reed and Crites, 1984). According to the same authors, to maximize infiltration rates using secondary effluent, there should be between a 1 and 3 day flood, followed by a 4 to 5 day dry in the summer or a 5 to 10 day dry in the winter.

2.5.8 Grain sizing and porosity

There are a number of physical characteristics of the filter sand which must be analyzed in order to choose an appropriate filter material. Effective size is the size of opening that will just pass 10% by weight of the filter material. Mean size is that opening that will pass 60% by weight of the filter material. Finally, uniformity coefficient, also known as sorting, is a ratio calculated by the mean size divided by the effective size. Coarse sand is not recommended for effective treatment. Filters containing this have been reported to be inefficient in reducing P concentrations to acceptable levels, as they often have low P sorption capacities, coupled with high infiltration rates (Cheung et al., 1994). Table 2-4 displays grain size recommendations from various sources with regard to appropriate R1 filter design.

<table>
<thead>
<tr>
<th>source</th>
<th>effective grain size $d_{10}$</th>
<th>mean grain size $d_{60}$</th>
<th>uniformity coefficient $d_{60}/d_{10}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nielson et al.</td>
<td>0.2 - 3 mm</td>
<td>1 - 2 mm</td>
<td>&lt; 3.5</td>
</tr>
<tr>
<td>Logsdon</td>
<td>0.2 - 0.5 mm</td>
<td></td>
<td>$\leq 3.0$</td>
</tr>
<tr>
<td>Guilloteau et al.</td>
<td>0.2 mm</td>
<td>1 mm</td>
<td>5</td>
</tr>
</tbody>
</table>

Latvala (1993) found that he obtained better treatment by finer (0.5-1 mm) sand than by coarser (1-2mm) sand. It is assumed that these recommended numbers are in terms of mean grain size. Reed et al. (1985) says, however, that the best soils are relatively coarse textured.
2.5.9 Operation and maintenance
As stated on page 49 of the EPA Design Manual - RI Supplement, "It is absolutely essential that particular attention be paid to the preparation of the O&M manual for the system. It is critical for the operator to understand clearly the RI concept, what controls and adjustments are available, and what the consequences of these adjustments might be." Even though RI basins are natural systems, a certain amount of maintenance is required. This, however, is often only in the sense of making and recording observations on how the filter is performing. The most important aspect to check is whether or not the filter displays its design infiltration capacity. This can be done by recording in log books the standing water depth and the amount of time it takes a basin to drain once the application ceases. These values can then be used to calculate the infiltration rate of the wastewater. If the results are lower than expected, then a number of things can be changed with regard to the filter practices. For example, the sand can be disc'd more frequently, the head can be increased or the flood/dry ratio changed, depending upon the reason for the decreased infiltration rate.

Flow control is an important aspect of the proper operation of RI systems; a full system involving time switches, float switches, tensiometers and automatic valves can be essential.

2.5.10 Climate
Rapid infiltration systems tend to operate without problems during cold weather, provided that precautions have been taken, such as thermal protection for pipes, pump stations and valves. One of the main problems that an RI system can face during a cold winter is the embedding of an ice cover on the sand surface; if this occurs, the system is essentially shut off until spring thaw. One method that is known to work against this problem is to set the surface of the sand in a series of ridges and furrows. This allows for the ice cover to settle on the surface, on the ridges, while the furrows continue to conduct water. The practice of the system in Kamloops of continuous flooding during the winter months was not come across in the literature read for this study; however, except for the aforementioned problem of occasional ice coverage of sand, the method seems to work. The system, when operating on a constant flood schedule, is no longer considered to be
an RI, but rather a seepage pond. These tend to produce percolate quality lower than that from RI systems and are therefore used at locations where water quality is not an issue.

2.5.11 Reed and Crites’ design steps
The following are the steps that should be followed when designing a rapid infiltration system as determined by Reed and Crites.

1. Determine the design infiltration rate.
2. Determine the RI hydraulic pathway (i.e., 100% vertical or some horizontal?).
3. Determine the treatment requirements.
4. Select pre-treatment (Kamloops system is already set; could, however, change the alum dose).
5. Calculate the hydraulic loading rate based on steps 1 and 3, as well as the flood/dry ratio.
6. Calculate the land requirements.
7. Design the underdrains.
8. Select a hydraulic loading cycle and number of basins.
9. Calculate the application rate and check the flood/dry ratio.
10. Determine the monitoring requirements.

2.6 Potential Problems in Design
As in the design of any system, a number of potential problems can arise when creating a rapid infiltration system. Most RI problems can be traced back to two fundamental inadequacies: improperly conducted field investigations and improper interpretation of field data by the designer. The EPA Design Manual discusses a number of these problems and Table 2-5 describes those which possibly pertain to the problems associated with Kamloops’ RI system:
Table 2-5. Probable causes of Kamloops' RI problems

<table>
<thead>
<tr>
<th>SOILS</th>
<th>DESIGN ASSUMPTIONS</th>
<th>CONSTRUCTION</th>
</tr>
</thead>
<tbody>
<tr>
<td>- layers or zones of less permeable soils not revealed during site investigation which impede water movement</td>
<td>- less than design capacity for water movement because backfill operations have reduced soil permeability</td>
<td>- excess traffic and inadvertent compaction of final basin surfaces</td>
</tr>
<tr>
<td>- field testing conducted at different location or different depth than the final system, so design may be based on inappropriate data</td>
<td></td>
<td>- failure to remove all of fine soils in surface layers or zones of unacceptable soils</td>
</tr>
</tbody>
</table>

source: EPA Design Manual - RI supplement, 1984

Table 2-5 reveals that Kamloops' RI problems may stem from three main areas, namely the sand itself, design or construction. It is impossible to propose one area before detailed analyses of the system are done.

The literature on rapid infiltration systems is plentiful and that which has been examined and cited in this Chapter has helped to formulate basic ideas and expectations of the Kamloops system which is being studied. Perhaps the most helpful suggestion by the various authors collectively, however, has been to point out the fact that each system is different and dependent upon its own characteristics, be they physical, geographical or operational. Much about the Kamloops rapid infiltration system must be determined in this study before any changes or recommendations can be made.
Chapter Three - Experimental Set-up and Procedures for Column Tests

3.1 Background
Due to the two distinct goals of this project, the field work was divided into two portions - researching the phosphorus removal potential of the basin (i.e. river) sand and studying the flow characteristics of the existing basins. To investigate the first of these goals, column tests were conducted. Wastewater, treated to various degrees (i.e. taken from different cells along the treatment train), was passed through sand columns and the resultant effluent tested for percent removals of various constituents. While the primary concern with regard to the column effluent was its phosphorus content, the removal of other constituents such as NH\textsubscript{3} and bacteria remained an important consideration, since effluent from the actual RI systems is discharged into the river and concentrations must therefore be below permit levels. Each column had a different sand depth since maximum infiltration was also a goal. It was desired to find an adequate removal depth which would also accommodate adequate flow. If it turned out, for instance, that the wastewater could be treated to practically the same degree with 60 or 120 cm of sand, then the question of maximum flow could be addressed and the shallower sand depth chosen. Such a choice would also involve various flow considerations and assumptions which will be discussed in the next section.

Not only was the depth of sand varied, but the type of influent wastewater was also changed. By securing wastewater from different locations around the treatment plant, influents of different phosphorus concentrations could be run through the columns. From the results, the soil sorption capacity (SSC) of phosphorus could be calculated and compared for the various influents, since SSC is dependent upon influent P concentration. The method of calculating a soil’s sorption capacity will be discussed in Chapter 4.

3.2 Equipment
Four cylinders were purchased for the column tests. The machining was performed in the Civil Engineering Shop; bottoms and four port holes were constructed for each column, including one near the top to handle overflow. There was also an outlet on the bottom for drainage purposes. The columns were made of clear plexi-glass, 0.64 cm thick; each was 183 cm long and 30.5 cm wide. The cylinders were mounted side by side onto a steel and wooden structure, such that their
bottom flanges were flush. Figure 3-1 illustrates the columns on their frame as they were set up at the treatment plant in Kamloops.

Figure 3-1. Set-up of columns on steel and wood frame.

It was calculated that a storage volume of 600 L would accommodate about 2 days worth of influent. This was a simple calculation which compared the theoretical differences in flow and resistance between the actual basins and the columns. The basins have an average sand depth of 2 m and water depth of 35 cm. The columns were to have an average sand depth of 90 cm and water depth of 75 cm. The columns therefore have half the depth of sand and twice the level of water which should result in approximately 4 times more throughput. If the average daily throughput of the basins is 0.23 m/day (taken from City records), then the columns can expect around 1 m/day. The surface area of the four columns is: \( \pi \times (0.152 \text{ m})^2 \times 4 \) which equals 0.29 m\(^2\). Thus, each day, an expected 0.29 m\(^3\) will travel through the columns, rendering the two day holding capacity to be approximately 600 L. Two cylindrical holding cells, 130 cm in height and 145 cm in diameter, were acquired by the City of Kamloops for storage purposes; their volume
safely exceeded that which was calculated to be required. A wooden scaffold was built by two of the staff from the City of Kamloops Public Works which connected the holding tank to the columns (see Figure 3-2). Wastewater from the tank was pressure-fed through a PVC tube, which branched into an outlet over each column.

![Figure 3-2. View from above the effluent distribution system into the columns.](image)

The columns were filled with wastewater from either cell 2C or 3 or from the wetlands, depending upon the current set of experiments. The holding tank would first be filled using a Honda 3.5 Hp pump, fitted with a 4 cm diameter hose, obtained from the Public Works yard; the water would then be retained in the tank for a pre-determined number of days. This pump was also used to empty the holding tanks into 2C (the cell containing the least-treated water) when experiments were over or, during warm and sunny weather, when too much algae had accumulated due to excessive retention time. Two Masterflex® L/S variable speed pumps were purchased through Civil Engineering for the purpose of conveying fresh water into or wastewater out of the columns.

It was originally intended that these small pumps be used for the filling of the holding tanks; this was, however, not necessary due to the availability of the Honda pump. The pumps operate by
pulsing liquid through 5 mm rubber tubing; the reversible nature of this pulsing of the pumps motor allowed for the columns either to be filled or emptied. The filtrate drained through the port at the bottom of the columns into rectangular containers, 20cm x 20cm x 70cm, which are visible in Figure 3-1. The filtrate which was not to be sampled was conveyed to a nearby manhole; from here, the wastewater was pumped into 2C when a certain level was reached.

The larger pump was driven by a gasoline engine and was therefore portable; the Masterflex® pumps, however, required electricity which was not available at the location of the experiments. A City of Kamloops electrician was therefore commissioned to connect an outlet at a nearby power box and electricity was subsequently supplied by means of an extension cord and power bar.

![Figure 3-3. View of the columns' piezometers.](image-url)
The ponded wastewater above the sand was kept at constant head to simulate the conditions at Cinnamon Ridge. This was accomplished by means of an overflow port at the top of the column; it was drilled as close to the top as possible so that maximum head could be achieved. This top port was fitted with a brass fitting to which was attached a 1 cm diameter rubber hose. Each column was machined with three port holes, other than the overflow port, 30 cm apart from one another; these can be seen in Figure 3-1. Attached to each of these ports was a 5mm diameter rubber hose which was bent upwards and secured near the top of the column, to form a type of piezometer. The filling of these tubes with filtrate during a flooding period indicated that the flow was saturated; empty tubes indicated unsaturated flow. Figure 3-3 is a photograph which gives a view of the piezometers.

3.3 Location of Columns
The columns were installed outside, at the northwest end of cell 2C. There were two reasons for this location. The first reason was simplicity; it would have been difficult to convey quantities of wastewater from various cells around the treatment plant to one indoor location. Water from cell 2C, for example, would have had to travel approximately 400 m to the columns if they had been set-up indoors; furthermore, the land is relatively flat, meaning gravity could not have helped the flow. Secondly, and more importantly, the chosen location of the columns meant exposure of the wastewater and filters to actual outdoor conditions, like those experienced by the basins on the other side of the river. Filter performance is sensitive to many parameters, such as water temperature, which dictates its viscosity and therefore its ability to flow, sunlight, which dictates algal blooms, and precipitation, which adds to the application rate.

The area chosen for the columns had a bank of approximately 1.8 m. This change in elevation was used to advantage by setting the holding tank on the bank, and the cylinders below, with the manifold extending from the tank to the tops of the cylinders.

3.4 Column Set-up
Each column was filled on the bottom with 15 cm of gravel, collected from the immediate surroundings. The purpose of this was to insure against the clogging of the drainage port on the bottom of the column by sand. Two layers of mesh, cut from common screen-door material, were placed on top of the gravel to help keep the sand from infiltrating the gravel. A layer of mesh was also placed in each of the outlet ports to insure against clogging. There were two series of
experiments; the first was run between June and September and was set-up in the following way. Three of the columns were filled with 60, 90 and 120 cm of sand (on top of the gravel) collected from the same region of river basin as that collected for the building of the rapid infiltration basins in 1985. The fourth column, column 4, was used as a blank, meaning that only tap water was run through it. In this way, the expected maximum flow rate could be recorded and compared to flow rates obtained in the other columns using the various influents. Flow rates through column 4 were obtained for sand depths of 60, 90 and 120 cm by adding sand in 30 cm increments. The second series of experiments was run between October and November and consisted of two columns which were each packed with 120 cm of “fresh” sand and no blank column.

During the summer, it became necessary to cover the columns with black plastic to discourage algae blooms, which were severely impeding flow. In retrospect, this set-up was more appropriate, since the sand of the RI basins is not exposed; the covering of the columns therefore better simulated actual basin conditions. For this reason, the covering was left on, even when algae growth had ceased.

3.5 Sand Characteristics

Before each packing, the density of the collected sand was measured and a grain-size analysis was carried out to be certain that the column sand was, for all intensive purposes, the same as that used in the basins. For comparison purposes, the density and a grain-sizing had to be done on the sand of both the east and west basins; this was due to the fact that differences in behaviour have been observed between the two basins. The density of the basin sand was measured by means of weighing a number of cores which had been obtained randomly over the east and west basins.

3.5.1 Grain-sizing

Grain-size analyses of collected samples were done by Eco-Tech Labs in Kamloops. A breakdown of size specifications can be seen in Table 3-1.
Table 3-1. Grain size classifications

<table>
<thead>
<tr>
<th>size</th>
<th>classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>4 - 64 mm</td>
<td>pebble</td>
</tr>
<tr>
<td>2 - 4 mm</td>
<td>gravel</td>
</tr>
<tr>
<td>1/16 - 2 mm</td>
<td>sand</td>
</tr>
<tr>
<td>1/256 - 1/16 mm</td>
<td>silt</td>
</tr>
<tr>
<td>&lt; 1/256 mm</td>
<td>clay</td>
</tr>
</tbody>
</table>

The first sand used for the columns was collected along the Thompson river channel near the treatment plant in early June and consisted of 94.3% sand, with 5.7% of the particles being smaller and therefore consisting of silt and/or clay. From table 3-1, this means that 94.3% of the particles were between 1/16 and 2 mm in diameter. This sand had an effective size of 0.09 mm, a mean size of 0.33 mm and a uniformity coefficient of 3.7 (see table 3-2). Recall from Chapter 2 that the literature recommends an effective size of between 0.2 and 0.5 mm, a mean size between 1 and 2 mm and a uniformity coefficient of less than 5. It seems, therefore, that the sand consisted in general of smaller particles than recommended. The sand used for the second packing in early October was collected from the same location and consisted of 95.6% sand and 4.4% smaller particles. This time, the effective size was 0.1 mm, the mean size 0.34 mm and the uniformity coefficient 3.4. The first collection of sand for the columns was done using a shovel, a garbage container and numerous trips back and forth between the columns and the channel site. The second collection in October was accomplished using a front-end loader.

Table 3-2. Grain sizing of column and basin sand.

<table>
<thead>
<tr>
<th>sample</th>
<th>% sand</th>
<th>% silt/clay</th>
<th>effective size, mm</th>
<th>mean size, mm</th>
<th>uniformity coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>June packing</td>
<td>94.3</td>
<td>5.7</td>
<td>0.09</td>
<td>0.33</td>
<td>3.7</td>
</tr>
<tr>
<td>Sept. packing</td>
<td>95.6</td>
<td>4.4</td>
<td>0.10</td>
<td>0.34</td>
<td>3.4</td>
</tr>
<tr>
<td>basins -10 cm</td>
<td>97.5</td>
<td>2.5</td>
<td>0.11</td>
<td>0.32</td>
<td>2.9</td>
</tr>
<tr>
<td>- 50 cm</td>
<td>95.9</td>
<td>4.1</td>
<td>0.10</td>
<td>0.28</td>
<td>2.8</td>
</tr>
<tr>
<td>-100 cm</td>
<td>96.7</td>
<td>3.3</td>
<td>0.10</td>
<td>0.31</td>
<td>3.1</td>
</tr>
</tbody>
</table>

To compare basin sand and column sand, samples were taken from the basins in a number of different locations, at three different depths - 10, 50 and 100 cm. Holes were dug
using a shovel to the appropriate depths and samples were then taken in small plastic bags. The grain-size analysis on these basin samples averaged 96.7 % sand and 3.3 % smaller particles; these numbers were similar to those attained from the column sand. The effective particle size at all three depths was 0.1 mm, the mean size averaged 0.31 mm and the uniformity coefficient averaged 2.96; each of these numbers is similar to its column sand counterpart. The slightly higher percentage of smaller particles in the column sand taken from the river channel can be attributed to the fact that this sand had not yet been “washed”; when flooded, a number of silt particles are released. This phenomenon was witnessed by the highly turbid water which was produced during the first column flush in all four of the columns. A more detailed breakdown of particle size for both column and basin sand can be found in Appendix A.

3.5.2 Density
The bulk density of the basin sand was calculated in the following way. Six sand samples, 3 from the east basin and 3 from the west, were collected in June using metal cylindrical cores. These cores were measured and their volume recorded. The core depths ranged from 15 to 30 cm and were taken at the end of a drying phase, so the sand had been drying for 10 days. Samples in the cylinders were then weighed, as were the empty core cylinders. Density could then be found by dividing the sand mass by the core volume; it was calculated to be 1.63 g/cm$^3$. A more comprehensive core sampling was done in July; ten samples were collected from each basin, 20 samples in total, randomly spaced, at depths ranging between 10 and 100 cm. The resultant average density of both basins was calculated to be 1.49 g/cm$^3$. The average density of the east basin was 5% higher than that of the west, but this difference was not considered to be significant, since the moisture content of each core was not assumed to be exactly the same, nor could the exact amount of sand in each core be considered to be the same. The 9% difference in density between the two sets of measurements was attributed to these same differences.

With the average density of the basins, 1.56 g/cm$^3$, and the measured volume of the columns, the required mass of sand for the columns could be calculated. For example, for column 3 which was to be filled with 120 cm of sand, the following calculations were done. The volume of a 120 cm segment of columns is: \( \pi \times (15.2 \text{ cm})^2 \times 120 \text{ cm} \) which equals \( 8.7 \times 10^4 \text{ cm}^3 \). Then, mass is found by multiplying this volume by the density; the
resultant required mass was 136 kg of sand. A set of ‘bathroom’ scales was used to weigh the sand before it was packed into the columns.

3.5.3 Porosity and Void Ratio
The total porosity of the sand in the columns was found to be 30%. This was calculated by filling the columns from the bottom, using a Masterflex® pump, with tap water. The pump sucked water from a large volumetric container, thereby allowing one to observe the volume of water entering the column. The volume of water which filled the gravel was subtracted from the total volume of water which was pumped in until the water level was flush with the top of the sand. This number could then be divided by the total volume of the cylinder, sand plus air. It was discovered that the absolute porosity was 11.5 L air per 39.4 L total volume or 11.5 L air per 27.9 L sand in column 1; in column 2, there was 18 L air per 60.4 L total volume or 18 L air per 42.2 L sand. The results of columns 1 and 2 correlate well by both indicating a porosity of approximately 30%. From the experimental numbers listed above, another relation can be calculated which is known as the void ratio, e. This is defined as the void volume divided by the solid volume and therefore equals 0.42 or 42%.

3.6 Experiments
The experiments are divided into two series due to the columns being packed twice with sand. Each series was further divided into a number of sets, each using a different column influent. The first series of experiments, which were performed from June through to September of 1995, used effluent from cell 2C (see Figure 2) as column influent. In the current treatment train at the City of Kamloops plant, wastewater from 2C flows into P1; at this stage, alum is added to remove phosphorus (P) through chemical precipitation. Effluent from 2C, when used as column influent, was therefore high in P. The phosphorus removal capacity of the sand was sought. A second set of experiments used effluent from cell 3 as column influent. Wastewater treated to this degree is essentially the same as that currently entering the RI system from cell 4, the only difference being the longer retention time of cell 4 wastewater. The purpose of using cell 3 wastewater was therefore to analyse the flow possibilities, and not the phosphorus removal capabilities, through different depths of sand. Finally, water from the wetlands pilot project was used as column influent; the wetlands receive 2C water (high in P) and, during warm weather, are generally able to remove 40 to 60% of the phosphorus. The results of this third set of tests could therefore also
be seen as results using wastewater which had received half of the current alum dosage. All three of these sets were performed using the columns packed in early June with 60, 90 and 120 cm of sand. A final set of experiments performed during the first series consisted of combining the columns to obtain as it were deeper sand filters. For example, filtrate collected from the 120 cm column would be run through either to 60 or 90 cm column, to obtain filtrate which had been run through 180 or 210 cm of sand. The purpose of this was to obtain a percent removal versus sand depth relation.

Each set of experiments was conducted by flooding the three columns of different sand depths with the type of wastewater chosen for that particular set. This was done according to various flood/dry ratios, which were seasonal and chosen in accordance to the literature (see Chapter 2). Generally, one day flooding followed by 2 to 3 days drying was used. This meant a flood/dry ratio of between 0.5 and 0.7. Some of the pre-chosen schedules had to be altered during the run due to filter clogging; this will be discussed in detail in Chapter 5.

The second series of experiments were conducted during October and November of 1995 and used two columns newly packed with 120 cm of sand. Both columns were similarly packed in order to be able to compare their results. At times, some parameter would be slightly altered for one of the columns, and the differences in results of the columns were noted. In order to examine the differences made, for example, by first flushing the sand in a column with tap water to rid it of any air pockets, one of the columns was flushed from the bottom at the beginning of an application period and the other was not. Any changes in removal capabilities and in flow were sought. As well, it was desired to see any removal differences between a filter with saturated flow and one with unsaturated flow.

As previously mentioned, not only was the effluent analyzed during the experiments, but the flow monitored. This included recording flow rates periodically (typically every 30 minutes), recording water height in the piezometers and calculating the residence time of the wastewater in the column. Measuring the flow entailed the timed filling of a graduated cylinder from the bottom port of the column. The captured volume divided by the time yielded a flow rate in L/min. Recording of water level in the piezometers indicated the whether the flow was saturated or unsaturated. If this was not constant, the transition from saturated to unsaturated due to clogging could be witnessed by observing the tubes.
Sampling of the filtrate was accomplished by placing an acid-washed, 1 L bottle under the bottom port and allowing the bottle to fill to the rim. Analysis of samples for most constituents was usually performed immediately upon collection, and was always performed immediately when BOD or coliforms were being tested. When completion of lab work was impossible for the day, the samples were placed in a fridge which was kept at 4°C.

There was initially some concern over flow restriction within a column due to too small an outlet at the bottom. This would have resulted in lower flows being recorded as well as longer residence time within the sand for the wastewater, both unrepresentative of an unrestricted sand filter. It was noticed, however, that at no time was the underlying 15 cm of gravel saturated; that is, the rocks on the bottom were never “swimming” in water. This observation allowed for the conclusion of unrestricted outflow from the columns.

3.7 Lab Tests and Procedures for Column Influent and Effluent
All testing of wastewater was performed by the author in the laboratory facility at the City of Kamloops Wastewater Treatment Plant. Column influent and effluent were tested for the following: pH, alkalinity, biochemical oxygen demand (BOD), total phosphorus (P), soluble P, ammonia (NH₃), nitrate (NO₃), nitrite (NO₂), dissolved oxygen (DO), total suspended solids (TSS), total dissolved solids (TDS), fecal and total coliforms. Both the absolute values and the relative values of these parameters between column influent and effluent were of interest. The latter values are commonly reported as percent removals through the column. While percent removal is an important aspect of an RI system, the absolute value of the effluent concentration of a particular parameter remains important, as it must be kept within permit limits.

3.7.1 pH and alkalinity
Both pH and alkalinity were measured using a Hanna Instruments 8417 probe. First the pH was measured by the probe and recorded; the sample was then titrated with 0.02N H₂SO₄ until a pH of 4.5 was reached. The volume of titrant was recorded and the alkalinity of the sample was then calculated using the following equation:

\[
\text{alkalinity} = \frac{\text{titre} \times N \times 50000}{\text{sample size}} \quad (3-1)
\]
The sample size was commonly 100 mL and the normality was always 0.02; this simplified the equation to the alkalinity simply being equal to 10 times the volume of titrant. The calculated alkalinity was always reported in units of mg/L as CaCO₃.

3.7.2 BOD
The procedure for the measurement of the BOD₅ of the sample is a common one. Four containers are filled with distilled water plus a steady supply of air to keep the concentration of dissolved oxygen high. Depending on the expected resultant BOD concentration, doubles of three different aliquots of sample are decanted into BOD bottles which contain seeded water. This water contains seed which has been added in the form of a powder pillow to provide food during the 5-day incubation period. The seed is a mixed bacterial culture which has been acclimated to the organic matter expected in the wastewater; it was in the form of a Hach BOD nutrient buffer pillow and contained K₃PO₄, Na₃PO₄, CaCl₂ and H₂O. After adding the seed to the water, equilibration is allowed overnight. Two bottles are filled only with this BOD water, to serve as blanks, rendering a total of 8 bottles. One set of 4 bottles is measured immediately for its DO content (DO₁ for the samples and DOₐ₁ for the blank) and these numbers are recorded. The remaining set of bottles is then tightly capped to minimize evaporation and placed in a fridge of constant 20°C temperature for 5 days. The DO is measured at the end of this incubation period (DO₂ and DOₐ₂), and the BOD is then found using the following equation:

\[
\text{BOD} = \frac{[(\text{DO}_1 - \text{DO}_2) - (\text{DO}_{a1} - \text{DO}_{a2})]}{\text{sample size}}
\]  

where the sample size, as previously mentioned, loosely depends on the expected results.

3.7.3 Phosphorus
The measurement of the three forms of phosphorus, total, ortho and dissolved, was performed using two different methods due to periodic problems with the instruments. The two instruments used were a spectrophotometer and a Hach DR/2000. The latter is a direct reading spectrophotometer commonly used in the field because it is portable; albeit a field instrument, its results were frequently more plausible than those of the larger, more expensive but much older spectrophotometer. Of primary interest in the testing of influent and effluent for phosphorus concentrations was the percent removal. As
previously mentioned, due to the river discharge of the RI basins, absolute values are pertinent, but not until an appropriate influent has been chosen which indicated adequate removal through the columns.

The procedure for determining total P when using the Hach DR/2000 is as follows:

1. Pour 25 mL of sample into 50 mL Erlenmeyer flask.
2. Add one potassium persulfate pillow and mix with magnetic stir bar.
3. Add 2 mL of 5.25N H₂SO₄.
4. Boil gently for 30 minutes, maintaining a volume ≤ 20 mL by adding deionized water.
5. Cool to room temperature.
6. Add 2 mL of 5N NaOH; swirl to mix.
7. Pour sample into a 25 mL graduated cylinder. Fill to 25 mL with deionized water.
8. Enter method #496 onto meter; set wavelength to 890 nm.
9. Fill sample cell with 25 mL of sample.
10. Add one PhosVer3 P⁰₄³⁻ pillow and shake immediately.
11. Press “shift timer” to time a 2 minute reaction.

This procedure yields a total phosphorus concentration when an unfiltered sample is used and a dissolved phosphorus concentration when the sample used has been filtered. Following steps 9 through 11 only with an unfiltered sample yields a concentration for ortho phosphorus. The DR/2000 method does not call for the use of a standard. Periodically, however, a standard provided by Hach (usually 1 mg/L P) would be used for the purpose of testing the instrument. Results were always satisfactory, with the resultant concentration being within 2% of the expected concentration (ie. between 0.98 and 1.02 mg/L).

The second method for determining the concentration of phosphorus in the influent and effluent of the column was using a spectrophotometer. This spectrophotometer has an infrared phototube and can therefore be used at 880 nm; the light path provided is at least 2.5 cm. The method used is termed the ascorbic acid method and follows the procedure laid out in Standard Methods, pages 448-450. A number of standards, including a
phosphate solution, are required by this method and must be made up weekly. The
standards which combine to form the reagent include: 5N H₂SO₄, potassium antimonyl
tartrate solution, ammonium molybdate solution and ascorbic acid. Details on how to
make up these standards, including the phosphate standard which is later used to calibrate
the instrument, can be obtained by consulting Standard Methods. The procedure for the
determination of ortho-P is as follows:

1. Pipet 50 mL of sample into an acid-washed, 125 mL Erlenmeyer flask.
2. Add 8 mL of the reagent and mix thoroughly.
3. After 10 minutes (but before 30 minutes), measure the absorbance or
   concentration of each sample at 880 nm (depending on the mode on which the
   instrument is set).
4. If step 3 used the absorbance mode of the spectrophotometer, then a
calibration curve must be drawn, using a series of 6 standards and a distilled
   water blank.

For total or dissolved phosphorus determination, the above steps are followed after
having digested the samples (a filtered sample for dissolved P). This method was not
often used, as the spectrophotometer was plagued with problems. It tended to drift within
a few minutes of being set, and hence the results were always questionable. For this
reason, the DR/2000 was the most common method of phosphorus determination.

3.7.4 Ammonia
This, too, was measured using two different methods due to instrumentation problems.
With the spectrophotometer, tests were done in both absorbance and concentration mode
for comparison purposes. The absorbance results were plotted versus concentration using
the known concentration of the standard. Results from these graphs were similar to those
concentrations read directly off of the spectrophotometer in concentration mode;
subsequently, only the concentration mode was used.

The phenate method was used for measuring NH₃ on the spectrophotometer. Once again,
the method comes from Standard Methods and can be found there for a detailed
description of the procedure. The required reagents, made up weekly, included
hypochlorous acid, manganous sulfate solution, phenate reagent and a stock ammonia solution. The basic steps to be followed are found in Standard Methods.

The second method used to determine the concentration of ammonia in a sample was known as the salicylate method (high range test tube method) and was performed on the Hach DR/2000. The steps involved:

1. Enter 343 on the DR/2000 and a wavelength of 655 nm.
2. Obtain 2 High Range AmVer Diluent Reagent vials from storage at 4°C.
3. Add 0.1 mL of sample to one vial and 0.1 mL of ammonia free water to the second.
4. Add in succession to each vial one Ammonia Salicylate Reagent Powder Pillow and one Ammonia Cyanurate Reagent Powder Pillow; cap vials and shake to dissolve all powder.
5. Leave to react for 20 minutes and place in DR/2000 to read value in terms of N (multiply by 1.22 to get answer in terms of NH₃)

3.7.5 Nitrate and Nitrite

Nitrate and nitrite concentrations of filtered samples were measured using the Hach DR/2000. Determination of nitrate was done using the cadmium reduction method and proceeded as follows:

1. Enter 351 on machine and a wavelength of 507 nm.
2. Pour 30 mL of filtered sample into a 50 mL graduated cylinder add a NitraVer 6 Nitrate Reagent Powder Pillow.
3. Shake during a 3 minute reaction period then leave sample for 2 minutes to allow the cadmium to settle.
4. Pour 25 mL of this into a sample cell and add a NitiVer 3 Nitrite Reagent Powder Pillow; shake to dissolve the powder.
5. Leave this for a 10 minute reaction then measure blank followed by sample.

The procedure for the determination of nitrite is similar, except a 25 mL aliquot is added directly to the sample cell, with no prior cadmium reaction. The pillow is added and the sample left for a 15 minute reaction period.
3.7.6 Dissolved Oxygen and Total Dissolved Solids

The parameters of the column influent and effluent were measured using a calibrated HORIBA Water Quality Checker U-10.

3.7.7 Total Suspended Solids

The procedure for measuring total suspended solids was a straightforward and simple one. A filter is placed in a COORS USA crucible and their weight is recorded. A 100 mL sample is then filtered through a glass-fiber filter (Whatman GF/A) with a pore size of 1.2 μm. The container and filter are then placed in an oven of temperature 104°C for one hour. Matter which evaporates at this temperature is not defined as a solid. Cooling occurs in a dessication chamber. It is important that the vessel cool to room temperature, since the weighing of anything hotter could be affected by convection currents. The cooled dish and filter are then weighed. The difference in weight before and after is multiplied by 10 to yield a total suspended solids value in mg/L.

3.7.8 Total and Fecal Coliforms

The membrane-filter technique was used for the determination of total and fecal coliforms. The testing of the two types of coliforms is similar, the only differences being the type of broth (culture medium) used and the incubation temperature. All materials used for coliform testing must be kept sterile for accurate results; all glassware was stored in containers which emitted UV radiation. One hundred millilitre samples are collected in sterilized containers and testing is done soon after collection. Influent samples had often to be diluted to facilitate counting at the end of the test, whereas column effluent samples were generally left undiluted, as incubated colonies were far fewer. Details of the incubation procedure are found in Standard Methods.

Results of the column tests are presented in Chapter 5. Problems or suggestions with regard to procedures are discussed in Chapter 9.
Chapter 4 - Experimental Procedures for Understanding Flow through RI Basins

4.1 Background

To attain the second of the project's goals, namely the understanding of the current RI system in terms of flow and sand characteristics, a number of tests and measurements was performed. In the fall of 1994, groups of piezometers were placed in both basins in the hope that they may offer some insight into flow characteristics. Measurements of water level within piezometers can provide answers to a number of different questions with regard to the driving forces of the basins. For example, if the tubes appear always to be empty during a basin application period, then it can be concluded that the surface layer of the sand is dictating this unsaturated flow. If, on the other hand, the tubes contain water, then the flow is saturated. Saturated flow can indicate that either the sand itself is dictating the flow, or the underdrain system is not functioning properly. A malfunctioning underdrain systems typically means clogging or design error. The underdrain design can be checked and a mounding analysis done to see whether or not the saturated flow could be due to the bottom of the basin. Piezometer readings also allow for the calculation of the sand's permeability (or hydraulic conductivity, used synonymously here) using Darcy's equation which was presented in Chapter 2. This calculated k value can then be compared to the design k and conclusions drawn as to whether or not the system is running as it was designed to be running.

Further uses of piezometers in an RI system are as follows. Measurements in a basin can be made both near the beginning of an application period and near the end of one. Differences in head between these two periods can be compared to see whether flow changes from saturated to unsaturated or the other way around, as the case may be. As well, measurements can be made on both the east and west basins during their respective flooding periods, providing that the weather does not vary and therefore water temperature remains the same. Comparisons between these measurements may reveal the cause of some of the differences that seem to have always existed between the two basins. Comparison of piezometer readings between the seasons can illustrate the differences in flow which were calculated theoretically due to viscosity changes.

Piezometers need not only be used for their flow indication capability; they can also be useful for effluent quality experiments. The four different tube lengths within each piezometer nest in the Kamloops system allow for samples to be taken from each of a nest's four tubes and analysed to
reveal removal depth for various parameters. As well, daily sampling throughout an entire application period can reveal any percent removal trends which might occur; this can help determine whether or not the current flood/dry schedule is appropriate.

4.2 Piezometer Characteristics and Positioning
The piezometers placed in both basins were in the form of 7.5 cm diameter PVC tubes of varying lengths, with the bottom 30 cm of each tube perforated to allow for the undisturbed entrance of water. The base of each tube was surrounded by an individual base of gravel, and both the bottom and top of each tube were capped. Each basin received 12 nests consisting of 4 piezometers each, rendering a total of 48 piezometers per basin. Each of the 4 tubes within one nest was different in length in order to compare differences in water depth between the tubes. Their placement can be seen below in Figure 4-1.

Each black dot within the basins in Figure 4-1 represents a nest of piezometers, each of which contains four tubes of different lengths. Due to inaccuracy of placement, the four tube depths of each piezometer nest are not the same from nest to nest; they are all within a range of approximately 20 cm from their counterparts in other nests. Roughly, however, the depths are 100 cm, 140 cm, 170 cm and 200 cm.

4.3 Equipment
The length of each tube was measured using a tape measure which was taped on the a 2.5 m wooden stick. Measurements of water depth were made using a depth finder which consisted of a
diode attached to the end of a weighted tape measure. This instrument was lowered down the
piezometer and when the light path was altered, as from the change in medium between air and
water, an audible tone was emitted. At this point, the depth could be read from the tape measure
and recorded. Since all piezometer readings were taken at full flood, this meant the basins
contained between 35 and 45 cm of water above the filter surface. To facilitate navigation on
foot throughout the basins, hip-waders were worn at all times. Measurements of basins
dimensions were made using a tape measure attached to a wheel which had a meter-long handle
to facilitate walking. Finally, for sampling from piezometer tubes, a bailer was used for first
flushing out the tube. This consisted of a 50 cm long plastic, 3 cm diameter cylinder containing a
marble within its shaft which served as a weight to close the bottom of the tube when full of
water. This bailer was used to flush each tube three times before filling a 300 mL plastic, acid-
washed bottle with a sample.

4.4 Piezometer Measurement Procedures
In order to maximize the information offered by the piezometers, various sets of measurements
were taken from both east and west basins. For the reasons previously mentioned, this entailed
two measurements within one application period (beginning and end) per basin. Measurements of
east and west basins on successive application periods were made for the purpose of ensuring
similar weather conditions; in this way, east and west basin flows could be directly compared. As
well, measurements which were taken in July were followed up by ones in November to witness
changes in flow due to viscosity differences. Finally, measurements were made at the beginning
and end of the winter flood (the time during which both basins remain flooded) to investigate any
changes in winter flow. The first set of these measurements was taken in the middle of
November, just as ice was beginning to form on the basins and the second set was recorded in the
middle of March, just as the ice cover was breaking up.

The taking of a set of measurements consisted of using the depth finder and walking from
piezometer nest to piezometer nest, recording the depth at which water was found for each tube in
each nest. Water depth could then be calculated by subtracting the depth at which water was
found from the total length of the particular tube.

The results of the measurements could be plotted for each basin, one row per graph (each basin
contains a north and a south row of tubes). The cross-sectional graphs illustrate tube length,
water depth, basin location, sand depth, water depth and basin slope. The studying of these graphs can reveal much about the dynamics of the basin flows. Differences in hydraulic head between piezometers within one nest were used to compute the permeability of the soil using Darcy's equation. A k value was computed between each set of two successive piezometer, meaning three k values were calculated per nest. The lowest of these three values was then taken to be the controlling k for that area of basin.

4.5 Investigations into Basin Characteristics

Apart from analyzing flow, which is achieved by means of the piezometers described in Section 4.4, a number of other tasks with respect to the basins were performed. To begin with, the exact dimensions of the basins were recorded. This simple but important task had not been performed since the installation of the basins and it was unknown if they had been constructed according to design. Any differences in surface area between the east and west basins would affect flow comparisons. More detailed investigations into the basin characteristics involved studying the underdrain system, performing a mounding analysis and finally executing grain-sizing and bacteriological tests on basin samples from various depths.

4.5.1 Underdrain system

Further investigations into the state of the basins led to calculations of the underdrain capacity. Hidden problems beneath the sand with either the underdrain sizing or spacing could be affecting the basin flows. It was necessary to ensure that too small a slope or insufficient pipe volume of the underdrain system was not the flow-reducing culprit. Design plans were studied to obtain the pipe dimensions and geometry. Use of the Hazen-Williams nomograph allowed for the calculation of required pipe diameter, knowing the flow and slope of the basins. This value could be compared to the design value to ensure that the latter is greater.

4.5.2 Mounding analysis

As described in Chapter 2, mounds are created as a result of an underdrainage system. Calculations were made of the maximum height of these mounds; this height is dependent upon the underdrain spacing, the flow and the permeability of the sand. It was necessary to ensure that these mounds were not greater than the head available at that depth.
Mounding calculations were made using Hooghoudt’s equation, (2-13); this equation can be derived from simple expressions. Before doing so, however, Hooghoudt’s Equation makes a number of assumptions. The first is the assumption that the hydraulic gradient \((dh/dl)\) at any point is equal to the slope of the water table \((dy/dx)\) above that point; this is known as the Dupuis Forchheimer (D-F) assumption. It implies that water flows horizontally because all of the equipotentials are vertical planes. Secondly, it is assumed that the drains are evenly spaced at a distance \(S\). Also assumed is a homogeneous soil with hydraulic conductivity \(k\). Darcy’s law is assumed to be valid and the flow therefore laminar. Water is replenished at a rate \(v\) and finally, there is an impermeable layer below the drains. There are two separate equations which define the flux, \(q_x\). These are Darcy’s expression:

\[
q_x = k \cdot A \frac{dh}{dl} \quad (4-1)
\]

and

\[
q_x = \left(\frac{S}{2} - x\right) v \quad (4-2)
\]

where \(S\) is the drain spacing. Setting the right side of equations (4-1) and (4-2) as equal yields the following ordinary differential equation:

\[
\int k \cdot y \ dy = \int Sv/2 \ dx - \int v \ x \ dx \quad (4-3)
\]

Integrating on both sides according to limits assigns the left side limits between \((h + d)\) and \((H + d)\) while the right side spans 0 to \((S/2)\). The resultant expression defines drain spacing and is known as Hooghoudt’s equation:

\[
S^2 = 4 \ k \left(H^2 - h^2 + 2 \ d \ H - 2 \ d \ h\right) / v \quad (2-13)
\]

The equation when applied to the Kamloops system can be slightly simplified. The distance, \(d\), between the impermeable layer and the underdrain pipe is taken to be zero. This is true according to the design drawings; even if the actual basins are slightly different, \(d=0\) represents the worst-case scenario and is therefore the safest assumption.
4.5.3 Grain sizing and bacteriological examinations

For further understanding of the flow within the basins, a number of samples was taken at depths of 10 cm, 50 cm and 100 cm at several locations within each basin. These samples were then sent to Norwest Labs to test biological content and to Eco-tech Labs for a grain-sizing. It was thought that there would be an increase in hydraulic conductivity vertically down the sand column, since biological activity in rapid infiltration systems is primarily in the upper layer of sand (see Chapter 2). Organisms in the basins are primarily diatoms which are 10 µm or less; the skeletons could therefore act as silt or clay and block the passage of water. The procedure by Norwest was primarily to illustrate the expectation that biological content decreased with depth. The grain-sizing was for the purpose of supporting the first test by showing that grain-size increased with depth.

4.6 Phosphorus Removal Capability of the Basins

One of the outcomes of this study was to be design recommendations for new basins. Before doing so, proof was required from past RI basin data to substantiate the claim that these basins were actually removing a net amount of phosphorus from the influent wastewater. To investigate this claim, data from 1990 through to 1995 were examined; data prior to 1990 were not accessible. These data included basin influent P concentrations (in the form of cell 4 effluent P concentration) and basin effluent P concentrations. The difference between these numbers is the P retained in the basins. By multiplying this concentration with the total volume of both basins, a mass of P in kg could be obtained for each year’s data; these masses could then be summed over all the years. From this, the amount of P sorbed by the soil could be determined and compared to the calculated SSC of both the column soil and the basin soil, the latter having been measured by NOWAK. To investigate further the possible retention of phosphorus within the basins, samples were taken from the surface layer and analysed for soluble phosphorus as well as for carbon content. It was thought that the high organic content of this layer could retain a greater amount of P than the sand itself.

Also to be determined by this study is an appropriate sand depth for the new basins. Before recommending a depth, phosphorus removal with depth must be investigated. To accomplish this task, a number of samples was obtained from various piezometer tubes at different depths within
both east and west basins. Results from the four tube depths could be averaged and compared to see if any trends emerged with regard to removal with depth.

Finally, the practice of scarifying the basins’ surface prior to application periods was questioned. Evidence that this procedure increased throughput was not found, but before cancelling it altogether, its effect on phosphorus removal, if any, had to be investigated. To accomplish this, data were again obtained for the years 1990 to 1995; these data included the log books in which were recorded the exact dates of basin switching. If a trend could be established between cycle day (ie. temporal proximity to scarifying since this was done just prior to basin flooding) and P removal, then the practice of basin scarifying could be better understood and its necessity addressed.

Results from these basins experiments and studies are presented and discussed in Chapter 6. Recommendations regarding possible changes to the basins are discussed in Chapter 10.
Chapter 5 - Analysis of Column Test Results

5.1 Series One - June to September
The first series of column test, conducted between June and September of 1995, served a number of purposes. These included getting a general overview of the removal potential in terms of various parameters of the sand, as well as the throughput capabilities of this same sand. More specifically, the tests were performed for the purpose of being able to choose, in the end, the best type of influent and depth of sand for further tests, to be conducted in the autumn (Series II). In order to term a specific influent or sand depth "best", a number of results must be considered. The overall percent removal of important parameters must be high; effluent concentrations must always be kept below permit levels for river discharge; the time for the sand to reach its phosphorus saturation level must be maximized; and finally, the flow rate must be satisfactory. The maximization of as many of the aforementioned characteristics as possible yields the choice for "best" influent and sand depth, for the given sand of the Kamloops RI system.

5.1.1 Constituent removal for three different influents through various sand depths
Column tests during the period between June and September were performed as described in Chapter 3. The primary goal of these experiments was to gain insight into the phosphorus removal capability of sand which is similar to that which currently fills the City of Kamloops' RI basins. Also monitored was the removal capability of the sand with regard to other important parameters as well as the rate of flow of the filtrate through the columns. With these results, the depth of sand most appropriate to both satisfactory treatment and flow rate will be selected. An overview of the laboratory results of filtrate testing is listed below in Table 5-1. Complete lab results can be found in Appendix B. Only those parameters pertinent to river discharge regulations are listed, although various other parameters (mentioned previously in chapter 3) were also measured. Noticeably absent from Table 5-1 is phosphorus as a parameter. This is due to the fact that its removal is entirely time dependent, and therefore an average percent removal number would represent nothing. Studying P removal from columns is therefore dealt with in another fashion and will be discussed in detail in the following section. With regard to Table 5-1, recall that column 1 was filled with 60 cm of sand, column 2 with 90 cm and column 3 with 120 cm.
Table 5-1. Average percent removals for columns with various influent constituents.

<table>
<thead>
<tr>
<th>parameter*</th>
<th>no. of analyses</th>
<th>column influent</th>
<th>average influent conc</th>
<th>percent removals/ avg effluent values</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>col. 1</td>
</tr>
<tr>
<td>TSS</td>
<td>10</td>
<td>2C</td>
<td>35</td>
<td>94; 2</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>wetlands</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>cell 3</td>
<td>25</td>
<td>88; 3</td>
</tr>
<tr>
<td>turbidity</td>
<td>10</td>
<td>2C</td>
<td>49</td>
<td>63; 18</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>wetlands</td>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>cell 3</td>
<td>11</td>
<td>44; 6</td>
</tr>
<tr>
<td>BOD</td>
<td>10</td>
<td>2C</td>
<td>55</td>
<td>84; 9</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>wetlands</td>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>cell 3</td>
<td>17</td>
<td>42; 10</td>
</tr>
<tr>
<td>NH₃</td>
<td>10</td>
<td>2C</td>
<td>12</td>
<td>-55; 19</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>wetlands</td>
<td>2</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>cell 3</td>
<td>1</td>
<td>50; .5</td>
</tr>
<tr>
<td>total col.**</td>
<td>10</td>
<td>2C</td>
<td>30000</td>
<td>83; 5100</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>wetlands</td>
<td>600</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>cell 3</td>
<td>10</td>
<td>67; 3</td>
</tr>
<tr>
<td>fecal col.</td>
<td>10</td>
<td>2C</td>
<td>800</td>
<td>80; 160</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>wetlands</td>
<td>100</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>cell 3</td>
<td>5</td>
<td>50; 2.5</td>
</tr>
</tbody>
</table>

* - all values in every table reported as mg/L except turbidity in NTU and coliforms in #/100 mL

** - coliforms

Table 5-1 displays the averaged laboratory results. It is evident that column 1 with 60 cm of sand is a little less effective in its treatment capability, while columns 2 and 3, with 90 cm and 120 cm respectively, indicate similar capabilities. Figures 5-1, 5-2 and 5-3 allow for direct visual comparison of these removal capabilities. Figure 5-3 does not include column 1, as wetlands influent was never introduced to it. In general, the figures do not
indicate increased treatment with increased depth of sand for each constituent. Removals, surprisingly, were similar for the three different depths of sand, though 60 cm was often a little lower in its percent removals than the other two. This can be attributed to the fact that the wastewater, whether it was from 2C, wetlands or cell 3, is already well treated before being introduced to the sand filter, whatever its depth. Although some parameters were better removed by one certain depth, no patterns were established.

Figure 5-1. Percent removals by columns 1, 2 and 3 using 2C as influent.

Figure 5-2. Percent removals by columns 1, 2 and 3 using cell 3 as influent.
Figure 5-1 shows the percentage removals for the three columns with 2C as an influent. This is the least treated of the three influents, but the results showed some of the highest percentage removals. This can be partially explained by the fact that physical components, such as TSS and coliforms, are almost completely removed within the filters. This means that the influent with the highest component concentrations can achieve the greatest percent removals for those physical components. One of the difficulties with sand treatment, however, is the fact that a number of different factors influence the sand. Some of these factors are known and therefore accounted for, but some remain unknown, a fact inherent to natural treatment systems. The increase in NH$_3$ from column 1 indicates that there was insufficient time (ie. sand depth) for nitrification to occur. As ammonia is notably toxic to fish, this result caused some concern. Figure 5-2 illustrates the results when cell 3 was used as the column influent. Percent removals are lower than for those experiments using 2C as influent. This is partially due to the aforementioned reason of sand use, but is also due to the fact that cell 3 wastewater has been highly treated. Percent removals tend to be lower when influent parameter concentrations already are low. Figure 5-3 shows the percent removals for only columns with 90 and 120 cm of sand using wetlands as the influent. Here again, removals are lower than for 2C. Noticeable in all
three graphs are the high removals of both total and fecal coliforms, as well as TSS and BOD. In all experiments, the effluent concentrations of TSS averaged 1 mg/L and of BOD averaged between 2 and 8 mg/L. These numbers are well below the permit levels and are considered safe for river discharge.

From the Table 5-1's column test treatment results thus far, 120 cm of sand is the best of the three in terms of removal efficiency; the average removal of column 1 is 57%, of column 2 is 72% and of column 3 is 75%. This suggests that 60 cm of sand can be discounted as a contender due both to its low percentage removal record as well as to its unsatisfactory removal of ammonia. However, the most appropriate sand depth, one that maximizes both treatment and flow aspects, cannot be chosen without studying the dynamics of the flow. This will be discussed in detail below in part (b) of this section. However, from a purely treatment-oriented point of view, it was desired to see if the trend of higher removals with deeper sand continued to some maximum, or if not, at which point it levelled off. To accomplish this, filtrate collected from a column was filtered through a second column; the filtrate from this second column could then be said to have travelled through a sand distance equal to the sum of both column depths. This method is not considered to be exactly similar to travel through a deeper column, since the wastewater now travels through two surface layers. Table 5-2 displays the results of running cell 3 influent through various series of doubled columns. Depths of 180 cm (column 3 followed by column 1), 210 cm (column 3 followed by column 2) and 270 cm (columns 1, 2 and 3) were tried.

<table>
<thead>
<tr>
<th>parameter</th>
<th>conc. of influent</th>
<th>180 cm</th>
<th>210 cm</th>
<th>270 cm</th>
</tr>
</thead>
<tbody>
<tr>
<td>TSS</td>
<td>35</td>
<td>&gt; 99</td>
<td>&gt; 99</td>
<td>&gt; 99</td>
</tr>
<tr>
<td>turbidity</td>
<td>15</td>
<td>5</td>
<td>81</td>
<td>82</td>
</tr>
<tr>
<td>BOD</td>
<td>15</td>
<td>80</td>
<td>87</td>
<td>87</td>
</tr>
<tr>
<td>NH₃</td>
<td>4</td>
<td>66</td>
<td>63</td>
<td>66</td>
</tr>
<tr>
<td>total P</td>
<td>1.1</td>
<td>73</td>
<td>18</td>
<td>82</td>
</tr>
<tr>
<td>total col/100 mL</td>
<td>20</td>
<td>80</td>
<td>75</td>
<td>80</td>
</tr>
<tr>
<td>fecal col/100 mL</td>
<td>16</td>
<td>94</td>
<td>94</td>
<td>94</td>
</tr>
</tbody>
</table>

Table 5-2. Percent removals for combined columns.
The results of Table 5-2 do not indicate increased removal with depth in all cases. In the cases of coliforms, ammonia, TSS and BOD, removals are fairly constant. Phosphorus values are included here since this one set of experiments occurred over a relatively short period of time; while the percent removals at the different depths are interesting for comparison purposes, their absolute values should not be depended upon. One surprise in the results came with phosphorus removal at 210 cm. This disappointing result, however, is attributed to the fact that, during the time of these tests, column 1 was fairly “fresh” (it had not been frequently used), while column 2 was nearer its point of phosphorus exhaustion, a state which is explained in detail in section (c) of this chapter. In general, however, it can be seen that treatment is in fact slightly better for deeper filters. The difference, however, is not significant enough to be worth the reduction in flow which would result from the increased resistance.

5.1.2 Flow results

Flow studies of Series I experiments, being conducted during the warm summer months, were often hampered by the presence of large amounts of algae. Algae mats frequently formed on the surface of the columns, impeding the flow and rendering it unsaturated, thereby cancelling the possibility of measuring the column permeability using Darcy’s equation (2-10). These problems arose primarily when using 2C as influent, due to its high phosphorus concentration. Saturated flow, meaning flow dictated by the column sand and not the surface, was achieved for the other two influents which were used, namely those from wetlands and from cell 3. Flow rates and piezometer levels were monitored during the flooding period of each experiment. During applications, columns were always kept at constant, maximum head. For column 1, a sand depth of 60 cm allowed for a maximum water depth of 104 cm. Column 2 with 90 cm of sand and column 3 with 120 cm of sand had water depths of 73 cm and 47 cm respectively. Flow was expected to increase proportionately to increased water depth and to decreased sand depth if the sand only were dictating the hydraulics of the filter. However, due to the aforementioned problems with algae, surface clogging came into effect. This meant that decreased sand depth did not decrease the resistance as expected. Thus flow through sand columns is absolutely dependent on the influent’s suspended solids, and hence the influent’s tendency to block the filter.
Results of column flow rates are presented below in Table 5-3. Suprisingly, the flow rate through the sand when using tap water as opposed to wastewater was only higher for one of the columns. Subsequent analysis of the sand in the blank column revealed a thin silty covering located approximately 30 cm above the gravel, which could have contributed to lower flow rates. This blocking layer was also apparent by the fact that, during every application period, the bottom piezometer was always empty while the upper two always registered water levels. Therefore, the use of a blank column was not helpful in determining possible flow rates.

Table 5-3. Flow rates of columns 1, 2 and 3.

<table>
<thead>
<tr>
<th>influent</th>
<th>influent temp. in °C</th>
<th>flow range in mL/min</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>col. 1</td>
<td>col. 2</td>
</tr>
<tr>
<td>2C</td>
<td>22</td>
<td>0.77 - 0.55</td>
</tr>
<tr>
<td>wetlands</td>
<td>25</td>
<td>-</td>
</tr>
<tr>
<td>cell 3</td>
<td>23</td>
<td>0.70 - 0.35</td>
</tr>
<tr>
<td>tap (blank)</td>
<td>23</td>
<td>1.00</td>
</tr>
</tbody>
</table>

The sometimes large flow ranges of a column were due to the build up of algae on the surface of the sand during application periods. The higher flows were therefore registered near the beginning of the flood; the number would then gradually drop as the algae accumulated. Ranges in flow rates did not occur during the autumn experiments which were not hampered by solids clogging the sand's surface. The problem of clogging due to algae was solved by covering the columns in opaque plastic, where they previously had been uncovered and hence open for light penetration. It was realized that their covered state more closely emulated actual conditions of the RI basins, as there, sunlight enters only from the top. The flow rates through the various columns revealed in Table 5-3 can be used to calculate the daily throughput. This number is only appropriate for the climatic
conditions under which it was obtained, as many factors change throughout the seasons. For example, flow would no longer be impeded in the fall by algae, but it would be by increased viscosity due to falling temperatures; it is therefore a trade-off, one to which it would be difficult to assign numbers. As discussed in Chapter 2, Reed and Crites (1984) recommend that flow values from column test be divided by at least 5 when scaling up to actual systems. The number by which the column flow is divided to estimate the full-scale flow is the safety factor. Five is considered to be a low safety factor in RI design, with most engineers choosing 10 or greater (see Chapter 2). To calculate the hydraulic loading rate (h.l.r.) of the column, the surface area is required; the column radius being 14.5 cm, this is equal to 672 cm². The calculation for h.l.r. is as follows:

\[
\text{h.l.r. in m/day} = \text{(flow rate in L/min)} \times (1 \text{ m}^3/1000 \text{L}) \times (60 \text{ min/hr}) \times (24 \text{ hr/day}) \times \left(\frac{1}{672 \text{ cm}^2}\right) \times (10000 \text{ cm}^2/\text{m}^2)
\]

Simplying the above expression results in multiplying the flow rate in L/min by 21.4 to get the daily h.l.r. in m/day. Thus, the h.l.r. for the blanks would be 21.4 m/day for column 1, 19.3 m/day for column 2 and 17.1 m/day for column 3. Dividing these number by 5 results in 4.3, 3.9 and 3.4 m/day for columns 1, 2 and 3 respectively, if they were to be extended to full-scale basins. These throughputs far exceed those currently realized in Kamloops' RI basins which are less than 1 m/day. These differences in numbers are understandable when the sand's permeability is taken into account.

Results of permeability calculations for column sand are listed in Table 5-4. A sample calculation of permeability, in this case, the k for column 1 (60 cm sand) with cell 3 as influent, is as follows:

Recall: \[ q = \frac{Q}{A} = k \times \frac{\Delta h}{\Delta l} \]  

where \( A \) = surface area of the columns = \( \pi \times r^2 = \pi \times (14.5 \text{ cm})^2 = 672 \text{ cm}^2 \)

and \( Q = \) average flow = 0.5 L/min

and \( \Delta l = \) sand thickness between piezometers = 30.5 cm

and \( \Delta h = \) head difference = 30.5 cm + 16 cm - 0 cm (empty) = 46.5 cm
So, \[ k = \frac{0.5 \text{ L/min} \times 30.5 \text{ cm} \times 1000 \text{ cm}^3/L \times (1/672 \text{ cm}^2) \times (1/46.5 \text{ cm}) \times 1 \text{ min/60s}}{60} \]

= 8.3 \times 10^{-5} \text{ m/s}

Note that the difference in head (dh) is taken from some base elevation; this was taken as the base of the lower piezometer for simplicity.

Table 5-4. Column sand permeabilities calculated using Darcy's equation.

<table>
<thead>
<tr>
<th>influent</th>
<th>temperature in °C</th>
<th>permeability in m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>60 cm</td>
<td>90 cm</td>
</tr>
<tr>
<td>cell 3</td>
<td>22</td>
<td>8.3 \times 10^{-5}</td>
</tr>
<tr>
<td>wetlands</td>
<td>22</td>
<td>-</td>
</tr>
</tbody>
</table>

The fact that the calculated permeabilities in Table 5-4 are not identical, as would be the result if the Hagen-Poiseuille method had been used to calculate \( k \), illustrates the dependence of permeability on a number of factors. These include the concentration of suspended solids in the influent and the temperature of the influent. If there were no build-up of solids on the surface of any of the filters, then it would be expected that \( k \) be the same for each column, independent of influent. The permeabilities in Table 5-4 are therefore only considered to be "summer" values, and not true indications of the sand's ability to conduct water. Values for a 120 cm sand depth could not be calculated for any of the three influents, as flow through this column was always unsaturated. This was due to the fact that 2C influent was run through this column for long periods of time, thereby building up a significant algae mat. In order to simulate actual basin conditions as closely as possible, this algae mat was not removed during drying periods, but was scarified (broken up) as done in the Kamloops system. This resulted in flow impediments for successive influents, even though these were lower in phosphorus content and therefore less prone to causing algae blooms. None of the columns had saturated flow when 2C was used as influent, therefore no permeability values could be calculated. The covering of both influent storage space and columns with black plastic helped the algae problem, but it was not done soon enough to keep algae from infiltrating past the sand's immediate surface and affecting the flow. The actual RI basins had saturated flow throughout the entire
summer season, most likely due to the low influent phosphorus; without sufficient nutrients, algae cannot thrive. This will be discussed in detail in Chapter 6.

The $k$ values of Table 5-4 are higher than the design $k$ value by almost an order of magnitude, and higher than the basin in-situ $k$ by almost 2 orders of magnitude (these will be discussed in Chapters 6 and 7). However, it should be mentioned at this point that the column $k$ calculations are not dependable for design purposes, even when multiplied by a safety factor. In-situ calculations are required for more accurate results; this sentiment is supported by all the literature which discusses basin design. Calculations of column $k$ can, however, be useful for comparison purposes between the columns and influents. As well, differences in viscosity due to temperature changes will be reflected by the permeability value.

5.1.3 Time-dependent P removal: soil sorption capacity and sorption isotherms
The results of Table 5-1 can be regarded as encouraging due to the generally high percentage removals of most constituents. However, phosphorus, which is the key parameter in the column tests, undergoes time-dependent removal. Figures 5-4 and 5-5 are plots of percent removal of phosphorus and effluent P concentration versus time for different sand depths and illustrate this time-dependence. Figure 5-4 represents the removal and effluent concentration of P for the 90 cm sand column using 2C, the "strongest" influent (ie. highest in P content). The scale on the right vertical axis shows that the required effluent concentration of 1 mg/L is already exceeded after a running time of approximately 3 hours. This hints at the impropriety of using 2C as an influent, since the basins would be exhausted in terms of phosphorus after a very short length of time. Currently, the basin schedule in Kamloops of 10 days flooding followed by 10 days drying means that application occurs incessantly for 240 hours. If the columns have a flow ten times greater than the basins, then exhaustion would be reached when using 2C after only 30 hours (ten times 3 hours). This finding therefore also points to the improbability of being able to cease alum addition altogether. Potential changes to current basin practices will be discussed in detail in Chapter 10.
Figure 5-4. Effluent P concentration and percent removal vs. time for 90 cm with 2C influent.

Figure 5-5 represents the removal and effluent concentration of P for the 120 cm sand column, again with 2C as influent. This time, the effluent P concentration is exceeded after only 4 hours. As expected, exhaustion was reached later for the deeper sand column, as there are more adsorption sites; however, it is difficult to pinpoint the cause of the time difference, since influents of varying strengths were applied to each column. The cause
may also be dependent upon influent P concentration. Removal of P with depth will be studied presently in terms of sorption isotherms for the 90 and 120 cm columns. Whatever the reasons, the number of hours remains far too small to consider 2C as a potential influent. It is apparent that this sand in its natural state simply does not have the capacity to remove large amounts of phosphorus. To verify this statement numerically, a further analysis can be done which involves the soil sorption capacity of the sand.

With time, sand becomes saturated with phosphorus which means that all adsorption sites are filled. There is, therefore, a point at which the filtrate phosphorus concentration will exceed the concentration of phosphorus in the influent. This can occur if most of the P is stored in the organic material at the sand surface and is subsequently released upon degradation. A more critical interval, however, is the point at which the critical concentration ratio of effluent to influent is exceeded. This is known as acceptable breakthrough, or \((c/c_0)_{crit}\), and equals the required P concentration of the effluent \(c\) divided by the P concentration of the influent \(c_0\). In the case of the Kamloops system, and consequently the column tests, \(c\) must be less than or equal to 1.0 mg/L. The most illustrative way of presenting these data is by means of a breakthrough curve which plots \(c/c_0\) versus time, with an overlay of the acceptable breakthrough level (Figures 5-6 and 5-7). The point in time at which the curve intersects \((c/c_0)_{crit}\) is known as \(t_{crit}\). Breakthrough curves are useful for illustrating both the time-dependent removal of P and \(t_{crit}\), where breakthrough occurs.

![Figure 5-6. Phosphorus breakthrough curve for 90 cm with 2C influent.](image-url)
Figures 5-6 and 5-7 display clearly the early breakthrough times of both 90 cm and 120 cm sand columns with 2C as an influent. Because many different influents were applied to the columns intermittently, a more indicative breakthrough curve is one which represents the response of a column to all influents, no matter their P concentration. In this way, the ratio c/c₀ varies, according to variations in influent concentration. These curves are shown in Figures 5-8 and 5-9.

Figure 5-7. Phosphorus breakthrough curve for 120 cm with 2C influent.

Figure 5-8. Phosphorus breakthrough curve for 90 cm sand column.
The acceptable breakthrough values in Figures 5-8 and 5-9, termed \((c/c_0)_{\text{crit}}\) in the legend, are varied due to the changing influent P concentration. The effluent concentration requirement of 1.0 mg/L never changes, thus only the denominator of the fraction \((c/c_0)_{\text{crit}}\) alters. Consequently, an increase in the value of the lower curve indicates a decrease in \(c_0\) and a decrease in the value indicates an increase in \(c_0\). The influent values changed due to different influents being used, such as those from 2C, the wetlands and cell 3. The time to breakthrough in Figure 5-8 for the 90 cm column is distinct and occurs after approximately 2 hours. The time to breakthrough in Figure 5-9 for the 120 cm column also occurs early along the curve; the values hover around the critical values for a stretch, but on the wrong side, meaning the early time of approximately 3 hours must be recorded as breakthrough. The deeper column therefore registers a longer time until breakthrough, although the length of time remains small.

5.1.3.1 SSC

This time required to reach breakthrough can be used to calculate the soil sorption capacity (SSC) of the sand. The SSC is a ratio defined by the mass of phosphorus over soil mass, and is commonly presented in units of \(\mu g\) P / g soil. The SSC of a soil is dependent on influent P concentration (NOWAK, 1985) and is calculated by the following two steps. First of all, the time \((t)\) until breakthrough is found on \(c/c_0\) vs. \(t\) curves (Figures
5-8 and 5-9). Secondly, the daily loading is calculated and the influent phosphorus concentration \((P_0)\) must be recorded; the product of these is equal to the total P loading:

\[
P = (t) \times (P_0) \times \text{daily loading}
\]

Equation (5-1) gives a value for P in terms of mass. For a daily loading value, the flow in L/day is required; since \(t\) in equation (5-1) is in hours, the flow can be reported in terms of L/hour. When \(P_0\) varies due to different influents (as in Figures 5-8 and 5-9), the concentration used in equation (5-1) is taken to be the average influent concentration in the region of the curve around which \(t_{\text{crit}}\) was found. For example, in Figure 5-8, the influent P concentration was taken to be 5.3 mg/L since \((c/co)_{\text{crit}}\) equals 0.19, and \(c\) is always 1 mg/L.

To complete the SSC calculation, equation (5-1) must be divided by the mass of soil. In this case, the sand has an average density of 1.63 g/cm\(^3\), or 1630 kg/m\(^3\), as was calculated in Chapter 3. This number, when multiplied by the volume of sand in the column, yields the sand mass in kilograms. For the 90 cm sand column, the P loading, according to equation (5-1) is calculated as follows:

\[
(2 \text{ hours}) \times (5.3 \text{ mg/L}) \times (47 \text{ L/hr}) = 498 \text{ mg P}
\]

Now, the mass of sand in the 90 cm column is 100 kg (1630 kg/m\(^3\) multiplied by a volume of 0.0615 m\(^3\)). Therefore, the SSC for the 90 cm sand column is 4.98 µg/g. Similarly, the P loading for the 120 cm sand column is calculated as follows:

\[
(3 \text{ hours}) \times (6.7 \text{ mg/L}) \times (35 \text{ L/hr}) = 704 \text{ mg P}
\]

The sand mass in the 120 cm column is 134 kg, meaning an SSC of 5.25 µg/g. As the same sand is used in each column, it was expected that the SSC values be the same; in this case, the values differ by only 5%. The SSC values calculated by NOWAK on sand gathered from the same area were higher than those calculated in this study. Their SSC values ranged from 13.3 to 24.2 µg/g; details on calculations can be found in their report (NOWAK, 1984). However, some other tests they performed with SSC and break-through curves led them to the conclusions that SSC is P concentration dependent and that the
highest SSC was obtained under the highest application rate. These findings may explain the differences between SSC values; the average influent P concentration in the present study was significantly higher than that used in NOWAK's tests which was 2.18 mg/L. On pages 30 and 31 of NOWAK, 1984, they contend that, with regard to P removal, the rapid adsorption occurs only in the unsaturated zone and the slow precipitation occurs in the saturated zone. This, if true, means that Kamloops' removal must be due almost entirely to precipitation since the basins are predominantly saturated. But, NOWAK claims that the underdrains of RI system, installed to control groundwater mounding, mean that P removal is mostly from adsorption (unsaturated zone) since the residence time in the soil is cut down. This will be further discussed in Chapter 7.

5.1.3.2 Sorption isotherms
The exact P removal mechanism within the sand has not yet been determined. Mechanisms other than adsorption, such as precipitation, may be at work. Precipitation of P by metal ions is a slower process which basically takes over for the adsorption which is only responsible for initial P removal (Aulenbach and Meisheng, 1988). A helpful indicator of both the sand’s capacity to retain phosphorus, as well as the mechanisms at work in the retention, is by means of a sorption isotherm. Common isotherms for P retention include those defined by the Langmuir or Freundlich equations, (2-6) and (2-8) from Chapter 2 respectively (Zarnett, 1976). A caution with regard to these isotherms is that only data representative of certain concentrations will fit the curves. Recalling the Langmuir equation,

\[ x = \frac{bx_m c}{1 + bc} \]  

where \( x \) is the amount of phosphorus sorbed in \( \mu g/g \) material, \( x_m \) is the sorption maximum, \( c \) is the residual phosphorus concentration and \( b \) is the bonding energy index and its simplification in order to calculate the parameters \( x_m \) and \( b \),

\[ \frac{c}{x} = \frac{c}{x_m} + \left( \frac{1}{b} \right) x_m \]  

(2-7)
a plot of c/x versus c yields a slope equal to 1/x_m and a y-intercept of (1/b)x_m. Similarly, the Freundlich equation,

\[ x = kc^n \]  \hspace{1cm} (2-8)

where k and n are empirical constants, is simplified to:

\[ \log x = \log k + n \log c \]  \hspace{1cm} (2-9)

which, when plotted, yields a straight line with slope n and y-intercept log k. Table 5-5 displays the parameters of the Langmuir and Freundlich isotherms as calculated from plots of equations (2-7) and (2-9).

<p>| Table 5-5. Parameters of Langmuir and Freundlich isotherms. |
|---------------------------------|----------|----------|----------|--------|--------|--------|</p>
<table>
<thead>
<tr>
<th>sand depth (cm)</th>
<th>b</th>
<th>x_m (µg/g)</th>
<th>R*</th>
<th>k (mL/g)</th>
<th>n</th>
<th>R</th>
</tr>
</thead>
<tbody>
<tr>
<td>90</td>
<td>11.1</td>
<td>3.3</td>
<td>0.55</td>
<td>2.4</td>
<td>0.3</td>
<td>0.27</td>
</tr>
<tr>
<td>120</td>
<td>2.3</td>
<td>1.8</td>
<td>0.40</td>
<td>1.4</td>
<td>0.2</td>
<td>0.14</td>
</tr>
</tbody>
</table>

* Correlation coefficients

As is evident from the low R values in Table 5-5, the results are a poor fit. This could indicate that a simple isotherm is inappropriate within this range (Cheung, 1994). Isotherms are only appropriate over narrow equilibrium P concentrations. However, plots of isotherms can help to discern certain features of the P removal mechanisms within the columns. Therefore, from the results of Table 5-5, equations (2-6) and (2-7) can be reconstructed and plotted for various values of c, the effluent P concentration in mg/L. Langmuir isotherms are plotted in Figure 5-10, and Freundlich in Figure 5-11. From Figure 5-10, it can be seen that the shallower of the two sand columns has higher sorption results. This at first appears surprising, except it must be remembered that the values for P sorbed are in µg/g of soil. Since the 90 cm column obviously contains less sand than the 120 cm column, the Langmuir isotherms suggest that most of the P sorption occurs in the
upper layers of sand. This would explain the lower sorption from the deeper column; if most or all of the sorption has already occurred by, for example, 90 cm, then the extra 30 cm is doing nothing but decreasing the value of P sorbed when looking at the results on a mass of P per mass of sand basis.

Figure 5-10. Langmuir isotherms for P sorption: sand depths of 90 cm and 120 cm.

Figure 5-11. Freundlich isotherms for P sorption: 90 cm and 120 cm sand depths.
As with the Langmuir isotherms, the depiction of the Freundlich isotherms in Figure 5-11 illustrates higher sorption from the shallower column. It is evident that the data of this study fit better to the Langmuir equation than to the Freundlich, as even on a semi-log plot, the sorption maximum is not reached as quickly in Figure 5-11 as it is in Figure 5-10. Overall for both isotherms, however, there is not a good fit, as is evident from the low correlation coefficients.

5.1.4 Effect of sand depth on P removal
As stated in Chapter 2, phosphorus removal through sand is time dependent, as the precipitation process takes a while to begin. The residence time in column 2 with 90 cm of sand and a flow rate of 1.2 L per minute is 50 minutes, while that of column 3 with 120 cm of sand and a flow rate of 0.7 L/min is 2 hours. To see the effects of longer residence times on treatment, experiments were performed which involved the combining of columns to produce the effect of deeper filters. Table 5-6 presents an overview of the results of combining columns to simulate deeper sand beds, in search of better phosphorus removal.

Table 5-6. Phosphorus removal with depth.

<table>
<thead>
<tr>
<th>depth in cm</th>
<th>total P in (mg/L)</th>
<th>total P out (mg/L)</th>
<th>percent removal</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>1.25</td>
<td>1.17</td>
<td>6</td>
</tr>
<tr>
<td>120</td>
<td>1.25</td>
<td>1.04</td>
<td>17</td>
</tr>
<tr>
<td>180</td>
<td>1.25</td>
<td>0.33</td>
<td>74</td>
</tr>
<tr>
<td>210</td>
<td>1.25</td>
<td>0.92</td>
<td>26</td>
</tr>
<tr>
<td>270</td>
<td>1.25</td>
<td>0.21</td>
<td>83</td>
</tr>
</tbody>
</table>

Table 5-6 represents column tests which used cell 3 effluent as influent, which explains the low influent P concentrations. The above data are plotted in Figure 5-12 and show, with the exception of the removal at 210 cm, a trend to higher removals with increased depth. The results from this set of experiments must be viewed with caution. Of the 60 cm, 90 cm and 120 cm columns, not all had been flooded for the same number of hours, nor had each received the same influents. This means that if one column had only received half the wastewater volume, and this wastewater was of a lower influent P concentration, then it...
would have used fewer of its available phosphorus adsorption sites. It is known that the 120 cm column was flooded more frequently than the 90 cm column, which in turn had received more volume than the 60 cm column.

![Graph showing phosphorus removal with depth of sand.](image)

Figure 5-12. Phosphorus removal with depth of sand.

Figure 5-12 indicates that some P removal is occurring at all levels within the sand; it does not, however, indicate where exactly the removal occurs, and how much occurs in each section. This will be investigated and discussed in detail in Chapter 6. The literature in Chapter 2 suggests that more removal occurs within the top few feet of a sand column. The combination of columns 2 and 3, represented by 210 cm, produced unexpectedly poor results of only 26% P removal. This indicates that column 2, being the most frequently flooded of the columns, was quickly approaching phosphorus saturation and hence removed only very little P. Actually, both columns 2 and 3 were the most commonly used of the sand columns; column 3, however, takes longer to reach saturation than column 2 due to it being the deeper of the two columns by 30 cm. This means that it has more physical sites available for the adsorption of phosphorus.

5.1.5 Effect of drying period on P removal

There are conflicting reports as to the effect, if any, of basin resting time on phosphorus removal. In order to see if any differences were evident, various flood/dry schedules were used during the column tests of Series I. It was discovered that resting period did in fact
seem to have an effect, albeit small, on percent removal of phosphorus. It seems that the longer the resting period, the higher the percentage removal. Different flood/dry schedules were applied to columns 2 and 3, with the results illustrated below in Figures 5-13 and 5-14.

![Figure 5-13. Phosphorus removal in terms of drying time for 90 cm. sand column.](image)

![Figure 5-14. Phosphorus removal in terms of drying time for 120 cm. sand column.](image)

As indicated in Figures 5-13 and 5-14, there is a marked decrease in percent removal at drying times below 3.5 days. This suggests that drying times be kept above this number. Conversely, the improved performance of the filter at drying times above this number, while existant, is not drastic. Between 3.5 and 41 days, there is only an improved removal of 3% in the 90 cm column and 9% in the 120 cm column. This would suggest that such
extended drying times are not worth the decreased throughput that would result. It must however be emphasized that when trying to isolate one particular filter characteristic, such as length of drying time in this case, the results must be handled prudently. The causes, for example, for the increased removals with drying time which are witnessed in the above figures are not due only to drying time. Influent phosphorus concentration, weather conditions and level of sand saturation will all have played a role in the determination of percent removal. One must therefore try to minimize the impact of changing conditions, and be mindful when interpreting results.

Comparing Figures 5-13 and 5-14 in terms of removal with depth, it can be seen that the differences are not great between the 90 cm and 120 cm sand filters. The plot in Figure 5-14 is simply shifted up from that of Figure 5-13 by between 5 and 10%. Again, it is difficult to locate the precise causes of differing results, as so many distinct factors contribute.

5.1.6 Choice of column influent for further column tests
For the second series of experiments, the results of the Series I in terms of most appropriate sand depth and column influent were needed. The purpose of Series II was no longer to focus on depth or influent, but rather on what could be expected under various conditions. From the results presented thus far, the most appropriate depth of sand could be chosen by looking at both the percent removal capabilities, flow and the $t_{\text{crit}}$ of the breakthrough curves. The “safest” depth which retains a sufficient flow rate was deemed to be 120 cm. The 90 cm column treated the wastewater to almost the same degree as the 120 cm column, and realized higher flow, but for the purpose of the best treatment, the 120 cm depth was chosen. What remains to be selected is an appropriate influent for this depth of sand. The primary goal is to maximize phosphorus removal, but the requirement of an effluent concentration below 1 mg/L remains. Curves of effluent P concentration versus time are helpful here. As well, the concentrations of certain parameters in the filtrates must be monitored to ensure that they, too, adhere to permit levels. Table 5-7 displays the effluent concentrations of important parameters; concentrations are in terms of maximum for safety. Once again, phosphorus is not included due to its time-dependence.
The best influent for P removal translates into the longest $t_{crit}$, as determined previously in part I (c) of this chapter.

### Table 5-7. Effluent concentrations of 120 cm sand filter for various influents.

<table>
<thead>
<tr>
<th>influent</th>
<th>maximum effluent concentration*</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>TSS</td>
</tr>
<tr>
<td>2C</td>
<td>4</td>
</tr>
<tr>
<td>wetlands</td>
<td>2</td>
</tr>
<tr>
<td>cell 3</td>
<td>1</td>
</tr>
<tr>
<td>permit level</td>
<td>10</td>
</tr>
</tbody>
</table>

*all concentrations in mg/L except coliforms in #/100 mL

The choice of influent primarily relates to the possibility of ceasing or limiting the current practice of chemical phosphorus removal through the addition of alum. If 2C were deemed appropriate, then alum addition could terminate completely. If wetlands were chosen, then various possibilities arise, since this wastewater has approximately half of the phosphorus concentration of 2C wastewater. Use of this influent would therefore mean either halving the current alum dose to remove about half of the phosphorus, or extending the wetlands so it could accommodate all pre-RI influent. If cell 3 wastewater were chosen, then this study would have to recommend continuing complete chemical phosphorus removal. However, the present task is to choose an influent, and with it, perform a second series of experiments; the ramifications of the choice will be discussed in the final chapters of this study. Table 5-7 reveals that in terms of TSS, BOD and NH$_3$, a filter of 120 cm reduces their concentrations to below permit levels for all three of the influents. With regard to coliforms, removals with 2C as the column influent were unsatisfactory. It should be remembered that in any full-scale implementations resulting from the findings of this study, the wastewater, from whichever source, would be chlorinated before crossing the river to be applied over the RI basins. However, to ensure that the study’s results have a sufficient safety factor, this fact will be ignored when analyzing the results. Therefore, due to the potential coliform problem, 2C as a choice of influent will be disregarded. This
conclusion with regard to using 2C as an influent would have been drawn for other reasons as well, the most important of which is the high influent P concentration and hence short time to phosphorus breakthrough. Cell 3 as influent could unquestionably be selected, as, with the exception of fewer days retention time, it resembles the influent currently being used in the RI system and the treatment results are satisfactory. However, to choose this influent would be to relinquish any extra phosphorus removal potential that the basins may hold. Since this is not an agreeable alternative, wetlands influent is selected. Successive experiments will involve only the study of column flow and not removals, since the influent concentrations are essentially already below permit levels, with the exception of P. Thus, the influent of choice is wetlands and the depth of choice is 120 cm for Series II experiments, and potentially for full-scale use.

5.2. Series Two - October and November

Only wetlands effluent was used as influent to two newly-packed, 120 cm sand columns due to the findings in part I of this chapter. That is, the phosphorus levels of 2C were too high for a sand filter, but it was desired to test the filter's capability of handling P concentrations higher than those of cell 3.

5.2.1 Flow and effluent quality in general

As previously mentioned, only wetlands was used as column influent. The columns were always flooded under the same weather conditions to facilitate comparison of results. For the purpose of examining the possibility of flow-hindering air pockets within the sand, one column was first flushed from the bottom with fresh water to expel any trapped air; the second column was not touched prior to application. Upon flooding both columns with wetlands wastewater, it was discovered that the flow of the flushed column was actually lower than that of the unflushed column by between 25 and 50 percent. This could be due to the back-flushing allowing the sand to compact into a denser matrix. Flows were always saturated, as the concentration of influent suspended solids was low. Table 5-8 summarizes the flow characteristics through the 120 cm sand columns using wetlands as influent. The difference in the results of Table 5-8 between the first and second row are most likely due to temperature effects. Water of 5°C has a viscosity 14% higher than water at 10°C; colder water therefore flows less readily. This also explains the lower
permeabilities witnessed in Series II than those from Series I, when the water temperature was 22°C.

Table 5-8. Flow characteristics of 120 cm sand column with wetlands as influent.

<table>
<thead>
<tr>
<th>temp. in °C</th>
<th>average flow in L/min</th>
<th>permeability in m/s</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>flushed</td>
<td>unflushed</td>
</tr>
<tr>
<td>10</td>
<td>0.25</td>
<td>0.40</td>
</tr>
<tr>
<td>5</td>
<td>0.20</td>
<td>0.30</td>
</tr>
</tbody>
</table>

The column which was first flushed from the bottom with clear water consistently had a lower flow rate than the column which was not flushed. This could be due to the subtle and unintentional differences in the packing of the columns. The flow through both columns was always saturated because of the low influent suspended solids. Due to this, it was expected that the sand would dictate the flow.

5.2.2 Effect of flushing on P removal and flow rate

General opinion in literature has often suggested that air pockets within sand filters can affect the treatment capability of the filter. If a significant volume of air is entrapped where sand is assumed to be, then the treatment path (ie. travel distance through sand) of applied influent is shortened. In order to see the effect of this on treatment, one of the two 120 cm sand columns was first flushed from the bottom with fresh water (City water) before being flooded from the top with wastewater from wetlands. The second column was always flooded simultaneously, but without prior flushing. If substantial differences in treatment were noted, then this practice would be continued; if not, then both columns would be flooded with no prior flushing. Figures 5-15 and 5-16 below display results in terms of effluent concentration versus time. These indicate that the flushed column sustained filtrate with a lower effluent P concentration than the unflushed column for a longer period of time.
The time scale in Figures 5-15 and 5-16 (as well as in Figures 5-17 and 5-18) is different due to the fact that it was decided fairly early on that the flushed columns were not displaying largely significant differences. Since the flushing of basins would be a huge task, it was decided to carry on with just unflushed column. More indicative of what is actually occurring are breakthrough curves, plotted using the same method as that
described in Section 5.1.3. Figures 5-17 and 5-18 display the time to breakthrough for the flushed and unflushed columns, respectively.

![Phosphorus breakthrough curve for flushed column.](image1)

Figure 5-17. Phosphorus breakthrough curve for flushed column.

![Phosphorus breakthrough curve for unflushed column.](image2)

Figure 5-18. Phosphorus breakthrough curve for unflushed column.

The times to breakthrough are 15.5 hours for the flushed column and 7 hours for the unflushed, and indicate the number of hours it takes until the filtrate measures a total P concentration of more than 1.0 mg/L. The difference in breakthrough time between the columns is significant. However, Figure 5-17 illustrates a clear and single time at which the curves cross, whereas Figure 5-18 has the curves crossing back and forth a number of
times. The time of 7 hours is the first of these crosses, and must be taken, even if at a later
time the column once again produces filtrate with below the required concentration, since
at no time should this number be exceeded, even for a few hours. The time to
breakthrough can be used, as in the previous section, to calculate the SSC of these
columns. Average flows can be taken from Table 5-9 to be 0.22 L/min for the flushed
column and 0.35 L/min for the unflushed; these numbers translate to 13.2 L/hour and 21
L/hour, respectively. The average influent P concentration until breakthrough was 3.6
mg/L from Figure 5-17 and 3.3 mg/L from Figure 5-18. The mass of sand for the 120 cm
column remains at 134 kg. The SSC for the flushed column is therefore (3.6 mg/L) x
(13.2 L/hour) x (15.5 hours) / 134 kg which equals 5.5 µg/g. Similarly, the unflushed
column has an SSC of (3.3 mg/L) x (21 L/hour) x (7 hours) / 134 kg or 3.6 µg/g. The fact
that the SSC of the unflushed column is lower seems to indicate that flushing a column
prior to an application period does have some effect.

A look at the Langmuir and Freundlich isotherms may reveal further information about the
differences between these two columns. Table 5-9 below presents the parameter data,
along with the correlation coefficients. Once again, the purpose of the isotherms is only to
illustrate differences between the sorption capacities of the columns; actual sorption values
are not important.

<table>
<thead>
<tr>
<th>120 cm sand</th>
<th>Langmuir</th>
<th>Freundlich</th>
</tr>
</thead>
<tbody>
<tr>
<td>column</td>
<td>b</td>
<td>x_m</td>
</tr>
<tr>
<td>flushed</td>
<td>-3.9</td>
<td>0.6</td>
</tr>
<tr>
<td>unflushed</td>
<td>3.6</td>
<td>4.1</td>
</tr>
</tbody>
</table>

* Correlation coefficients

As is evident from the low R values in Table 5-9 the results are, once again, a poor fit with
the exception of the Langmuir for the flushed column. This better correlation can,
however, be attributed to the fact that there were fewer data points to be fitted. Possible
explanations for the poorly fitting data are the same as those mentioned in the previous
section on Series I sorption data, namely that a single sorption isotherm will fit to data within a narrow margin of equilibrium concentrations.

Figure 5-19. Langmuir isotherms for P sorption for 120 cm sand columns.

The Langmuir curve, Figure 5-19, for the flushed column is surprising in its negative beginning and its very low sorption at higher effluent concentrations. From the difference in SSC, it was expected that this column had a higher capacity to sorb than the unflushed column. The unexpected result of the plot could indicate that some other chemical species out-competed P at higher concentrations.

Figure 5-20. Freundlich isotherms for P sorption for 120 cm sand columns.
However, Figure 5-20 illustrates more expected results. Here, the unflushed curve still has a slightly higher sorption capacity at higher effluent concentrations, but not by much. The number was also influenced by the first peak on the unflushed cycle. Due once again though to the disagreement between the SSC numbers and the isotherm curves, as well as due to the very low correlation coefficients, it seems that this data does not fit and therefore can not be explained by a single isotherm plot.

5.2.3 In search of unsaturated flow for comparison purposes
A comparison which was thought to be useful was that between phosphorus removal from a column with saturated flow and from one with unsaturated flow. Thus, the two columns with 120 cm of sand were run until unsaturated flow was achieved, meaning that sufficient solids had built up on the surface to impede the flow sufficiently. At this point, the bottom of one of the columns could have been pinched enough to revert back to saturated flow; the filtrates from both columns could then have been tested for P content and compared. This point, however, was never reached; the solids never impeded the flow, which remained steady throughout the application periods, unlike during Series I. Though this would have been an interesting comparison, its conclusions could not necessarily have been implemented. The RI basins have year-round saturated flow, as indicated by the piezometers (this will be discussed in detail in Chapter 6); if, for instance, removal for unsaturated sand exceeded that for saturated sand, changing the flow characteristics of the Kamloops’ basins would not be a trivial task.

5.3 Conclusions from Column Tests
The experiments of Series I were created and executed for the purpose of obtaining a general overview of the treatment and flow capabilities of various filter depths using the sand which currently fills Kamloops’ rapid infiltration system. Closer inspection of the resultant flow data as well as treatment data, both in terms of all parameters and in terms of phosphorus specifically, allowed for the emergence of a depth and type of influent which were the most appropriate for further experimentation. It should be recalled and emphasized that of primary importance in the column tests is the investigation into the sand’s phosphorus removal capacity. The treatment of other parameters by the sand filter is also of concern due to the fact that the filtrate is river-discharged; as well, flow is important due to throughput requirements (ie. lack of storage space)
but also not the predominant component. Series II experiments built on the findings of the first series by using the selected sand depth and influent. The purpose of the second set of tests was further investigation into phosphorus removal, without being concerned with the treatment of other parameters, since adequate treatment was already ascertained from Series I. Flow continued to be monitored but primarily for comparison purposes such as differences between columns and differences due to viscosity changes.

In Series I, constituent removal revealed that 90 cm and 120 cm treated wastewater to almost the same degree. Increased sand depth (beyond 120 cm) does increase treatment but not substantially; it was therefore assumed that the reduction in flow would not be worthwhile. Flow results showed 90 cm to have the highest throughput; the 60 cm column should have had the highest, but it inadvertently contained a low permeability layer. A look at P removal with time indicated that 2C has definitely too high an influent phosphorus concentration. Cell 3 is an acceptable influent, but is similar to that currently being used therefore wetlands was chosen. Breakthrough times of under 5 hours and SSC values of approximately 5 µg/g were found for columns 2 and 3. With respect to drying period, the longer it is, the higher the P removal but this is maximized by 3.5 days; beyond this, improved treatment would be at the cost of large reductions in throughput. Isotherms indicated that 90 cm sorbs more per unit mass than 120 cm which reveals that more P is being removed in the surface layers. Opposing this, however, are the data of P removal vs. depth which showed, in general, increased P removal with increased depth. These data, however, are disputable due to the sand from various columns being at different stages of exhaustion. Series II tests revealed that flushing results in lower flow but better treatment in the form of a higher SSC. However, large-scale implementation of flushing is likely not feasible. Viewing the two series together, it can be concluded that the general treatment capability of a 120 cm sand filter is excellent, as is the flow through it. However, its phosphorus removal potential for influent of a higher phosphorus content than is currently being introduced to the RI system is poor. Even with the current basin depth of 2 m, influent with P concentrations higher than 1 mg/L would quickly exhaust the basin, meaning the running of the system would become too work-intensive. Further discussion of this conclusion will be contained in Chapter 10.
Chapter 6 - Analysis of Basin Studies

This study was required due to the questionable performance flow-wise of the RI basins to date and to investigate the phosphorus uptake capabilities of the Thompson River sand. To complement the column studies, it was necessary to undertake an in-depth study of the basins’ physical characteristics and how they currently operate. This means an understanding of flow through the analysis of piezometers, underdrain system and sand characteristics. To complete the study, investigations into past behaviour with respect to phosphorus uptake were essential.

6.1 Piezometer Graphs and Flow Indications

Measurements of each piezometer tube (48 per basin) were made during various seasonal conditions. These included a set of measurements taken in July of 1995 when the water temperature was high, 23°C, and several sets taken in the fall of 1995, with water temperatures ranging from 10°C to 2°C. A final set of measurements was taken in March of 1996 as the ice cover was breaking and the water temperature was around 1°C. These sets of measurements were important as the state of the piezometers provides the first information on the flow occurring within the basins. To date, all flow problems that the RI system encountered were blamed on various surface phenomena. The readings of the piezometers, however, showed that this was not always the case.

The piezometers are 7.5 cm diameter white PVC tubes which have the bottom nine inches drilled with holes, 1.5 cm apart, to allow for the inflow of water. The following table provides information on water at the temperatures encountered during the measurements of the tubes.

<table>
<thead>
<tr>
<th>temperature in °C</th>
<th>density in kg/m³</th>
<th>dynamic viscosity in N.s/m² x 10⁻³</th>
<th>kinematic viscosity in m²/s x 10⁻⁶</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>999.8</td>
<td>1.781</td>
<td>1.785</td>
</tr>
<tr>
<td>5</td>
<td>1000</td>
<td>1.518</td>
<td>1.519</td>
</tr>
<tr>
<td>10</td>
<td>999.7</td>
<td>1.307</td>
<td>1.306</td>
</tr>
<tr>
<td>20</td>
<td>998.2</td>
<td>1.002</td>
<td>1.003</td>
</tr>
</tbody>
</table>

Table 6-1 indicates that the viscosity between July and October temperatures changes by over 30%; this means that the water becomes about 30% "thicker" and therefore has more difficulty passing through the sand.

Before studying the graphs of the piezometers, an explanation of the hydraulics of the basin as revealed by the water level within the tubes is necessary. Each of the four tubes within one nest extends to a different depth. The difference in depth between two tubes therefore inherently has a head requirement for wastewater trying to travel between the two depths. Recall that the flow is directly proportional to the hydraulic gradient which is defined by \( \frac{dh}{dl} \), where \( l \) is the length of the sand column and \( dh \) is the difference in water height between the two piezometers being examined. By looking at the basic equation linking flow to gradient (Darcy), the larger the difference in head, the higher the flow for a constant \( k \).

### 6.1.1 Comparisons between beginning and end of application period

Measurements made in October and November from both basins consisted of two sets: one at the beginning of an application period and one at the end, just before the basin was to be shut down to begin its drying period. Results of these measurements are illustrated in Figures 6-1 through 6-6. It was thought that if at least one of the flow-controlling factors were the surface layer, that the water levels in the piezometers would decrease between the beginning and end of a flooding period due to a build-up of suspended solids on the surface. This, however, did not turn out to be the case. Apparently, the flow is controlled by the sand itself. In order to ascertain with some certainty the flow-controlling factor, the piezometer graphs must be closely analyzed. There are many possible factors which could control the flow. One of these is the drainage system; this could be volumetrically inadequate and therefore simply unable to handle the flow. A second factor could be that the basins have an inadequate slope. Finally, the surface layer or the sand itself could be controlling the flow. The former would mean clogging at the filter surface due to high influent suspended solids. The latter could mean problems with the sand gradation (i.e. silty in some areas and therefore unable to pass large volumes of water) or too high a depth of the sand. The current depth is 2.2 m.

The first set of piezometer measurements, illustrated below in Figures 6-1 and 6-2, depicts the difference in water depth between the beginning and the end of an application
period. Only the east basin's piezometers are shown here, as those of the west basin display similar characteristics with respect to changes during an application period. Before comparing Figure 6-1 to Figure 6-2, it must be noted that a striking property of basin flow is revealed by each of the graphs: the basins have saturated flow. The fact that there is water within the tubes divulges the state of the flow. This finding was surprising since all of the literature on the Kamloops RI system had assumed unsaturated flow. This was evident in the way that all measures to increase the flow involved the filter surface and not the sand itself. The piezometers, however, have revealed that it is in fact not the surface of the filter which dictates the flow; it must therefore either be the sand column or the underdrainage system which controls the wastewater flow. This will be discussed further in the following section of this chapter.

The revelation of saturated flow obtained after taking the first set of piezometer measurements could not immediately be extended to the assumption of the basins having saturated flow at all times. It was thought that the flow patterns revealed at the beginning of the application period could shift during that period. Perhaps as the flooding continued, the surface layer would build up sufficiently to cause clogging and flow reduction; this would manifest itself in the form of empty piezometer tubes when measurements were made towards the end of the application period. This, however, was not the case as is revealed in Figure 6-2. The differences in water depth within the tubes between the beginning and end of the flooding period are not great.

Figure 6-1(A) illustrates the piezometer tubes in the south row of the east basin; their water depths are typical of the beginning of an application period. Water temperature on this date was 10°C and viscosity was therefore in the mid-range. Any missing tubes in the graphs indicate that the piezometer was loose from its base and its depth was therefore impossible to determine; this occurred in 6 of the 96 tubes.
Figure 6-1(A). Piezometer measurements on the East Basin’s south row at the beginning of an application period, October 11th.

Figure 6-1(B). Piezometer measurements on the East Basin’s north row at the beginning of an application period, October 11th.

Figure 6-1(B) is similar to 6-1(A), except that it depicts the north row of piezometers. It can be seen that differences between the two rows are not great. Figures 6-2(A) and (B) reveal the state of the piezometers toward the end of the 10-day application period. Evidently, the basin remains saturated throughout the flood cycle.
In Figure 6-2(A), the water levels are only significantly different from the beginning of the cycle in the western portion of the basin, specifically in piezometers 1 through 8. Although water does remain in the tubes, the decreased level suggests that solids are building up on the surface in that region. To see if the water would continue to decrease, and the basin become less saturated, the application period would have to be longer. This idea is tested during the continuous flood between November and March and will be discussed shortly.

Figure 6-2(B). Piezometer measurements on the East basin’s north row at the end of an application period, October 19th.
Figure 6-2(B), the north row of the east basin’s tubes, indicates even less of a difference between the water levels throughout the flooding cycle than did the south row. In fact, the basin in this area retains slightly more water in its piezometers than in the beginning of the application period.

6.1.2 Differences between east and west basins

The examination of east basin graphs versus those of the west basin may be helpful in revealing distinctions between the basins. If, for example, the basins have different controlling factors, these could lead to the differences in throughput. To compare graphs directly, the water levels within the tubes must be measured under similar weather conditions (to ensure the same viscosity), similar flooding conditions (such that each basin is at maximum ponding depth of 45 cm) and finally measurements must be taken on the same day of an application period to ensure similar solids accumulation on the basin surfaces. Chosen to be studied are plots made from October measurements in the beginning of an application period with a water temperature between 5°C and 10°C. Figures 6-1 (A) and (B) represent the south and north rows respectively of the east basin measurements. Figures 6-3 (A) and (B) below represent those for the west basin.

Figure 6-3(A). Piezometer measurements on the West basin’s south row at the beginning of an application period, October 2\textsuperscript{nd}.
The orientation of west basin piezometers is different from the east, as the road between the two basins was taken to be the starting point for all measurements. Figure 6-3(A) shows the south row of tubes in the west basins as they were in the beginning of the application period on October the 2nd. A comparison between this illustration and Figure 6-1(A) indicates the south row of the east basin to be more saturated than its counterpart in the west. Recall that the west basin generally realizes a throughput which is 40% greater than that of the east basin.

![Diagram of piezometer measurements](image)

Figure 6-3(B). Piezometer measurements on the West basin's north row at the beginning of an application period, October 2nd.

As with the south rows of the basins, Figure 6-3(B) reveals the north row of the west basin to be less saturated than the north row of the east basin, Figure 6-1(B). Noticeably empty are the tubes in the second nest, numbers 5 through 8. This could indicate a build up of solids in that area or a section of coarser sand. The cause can not be established until a study of the basin permeabilities has been completed (see section 6.2). In general, then, the east basin is more saturated than the west. The fact that it is the west basin which realizes a higher throughput leads to the following assertion. The west piezometers contain less water because of areas of coarser sand and not because of more solids clogging the surface; the corollary would be that the east basin contains more pockets of finer sand or silts which can impede the flow severely. As stated previously, this must be further studied by calculating the basin permeabilities.
6.1.3 Seasonal differences in piezometer readings

Among the complaints about the RI system’s performance is the severely reduced winter throughput. However, as seen in Table 6-1, the viscosity of water at 10°C is 37% lower than at 0°C and water at 20°C is 56% lower than at 0°C. The Hagen-Poiseuille equation for permeability contains the viscosity term in the denominator; this implies that flow can increase by 37% and 78% when the temperature shifts from 0°C to 10°C and 20°C, respectively. It is therefore the author’s contention that winter reduction in flow is primarily due only to increased viscosity of the water at low temperatures. Measurements made on the east basin in June and in October can be compared to see if any changes are witnessed in the flow patterns as evidenced by the piezometers; as well, measurements made on the west basin in October at 10°C and in November at 1°C (with the beginnings of an ice cover) can be compared.

Below are Figures 6-4 (A) and (B) which represent measurements made on the east basin in June; the water temperature was 20°C. These can be compared to Figure 6-2 for evidence of the effects of viscosity.

Figure 6-4(A). Piezometer measurements on the East basin’s south row at the end of an application period, June 20th.

The south row of tubes is less saturated than the north row and suggests algae accumulation in the southwest region of the basin. Algae accumulation is not uncommon
during Kamloops' hot, dry summers. The water levels in the tubes are lower than in the October measurements of Figure 6-2(B). The north row in Figure 6-4(B) indicates more water than the south row in the piezometers, but still less than was present in Figure 6-2(B) which shows the same row in October when the water temperature was 10°C.

Figure 6-4(B). Piezometer measurements on the East basin's north row at the end of an application period, June 20th.

The difference between the level of saturation in Figure 6-2 and Figure 6-4 cannot be explained without flow data for those dates. The lower viscosity of the water in June could lead to the assumption of higher flow, but the higher numbers of algae during that time could lead to the opposite assumption. Table 6-2 below lists the flow numbers for the periods in question and compares them to changes in viscosity.

Any differences in flow between a 10°C temperature and one near freezing can be seen by contrasting Figures 6-5 and 6-6.
Figure 6-5(A). Piezometer measurements on the West basin's south row at the end of an application period on October 10th.

Figure 6-5(B). Piezometer measurements on the West basin's north row at the end of an application period on October 10th.

Figure 6-5(B) reveals a tendency towards unsaturated flow, particularly on the west end of the north row. Possibly due to differences in surface slope, more solids tend to accumulate at this corner of the basin than in other areas. However, looking at entire basin, the majority of piezometers continue to hold water which indicates that predominantly, the basin remains in its saturated state.
Figure 6-6(A). Piezometer measurements on the West basin's south row at the end of an application period, November 15th.

Figure 6-6(B). Piezometer measurements on the West basin's north row at the end of an application period, November 15th.

Figure 6-6 illustrates the state of the west basin in the middle of November when the water temperature was 1°C and an ice cover was beginning to form. Levels, especially in the north row, are low, meaning the flow is close to being unsaturated. Reasons for this could be surface clogging. It is known that increased viscosity reduces the flow as well, however this is accompanied by increased saturation levels due to a lower $k$. Once
again, a look at the flow values for these periods will help to discern the cause of the changes in basin saturation patterns.

Table 6-2. Selected flow data for east and west basins.

<table>
<thead>
<tr>
<th>basin</th>
<th>figure reference</th>
<th>average flow in m$^3$/day</th>
<th>temp. in °C</th>
<th>Δflow due to Δviscosity</th>
</tr>
</thead>
<tbody>
<tr>
<td>EAST - June 20, 1995</td>
<td>6-4</td>
<td>3152</td>
<td>20</td>
<td>1.37x**</td>
</tr>
<tr>
<td>EAST - Oct. 19, 1995</td>
<td>6-2</td>
<td>6294</td>
<td>10</td>
<td>x</td>
</tr>
<tr>
<td>WEST - Oct. 10, 1995</td>
<td>6-5</td>
<td>11219</td>
<td>10</td>
<td>1.37x</td>
</tr>
<tr>
<td>WEST - Nov. 15, 1995</td>
<td>6-6</td>
<td>7697*</td>
<td>1</td>
<td>x</td>
</tr>
</tbody>
</table>

* total average flow for both basins was 12829; this was arbitrarily divided 60/40 in favour of the west basin, since this basin consistently has higher throughput

** as discussed earlier, an increase of 10°C can increase the flow by 37%

Table 6-2 presents the flow data pertinent to the Figures 6-2, 6-4, 6-5 and 6-6. The east basin data tells of an increase in throughput from June to October, where a decrease was expected due to the increase in viscosity. This therefore suggests that the presence of algae in June was contributing to the clogging of the surface layer, thereby impeding the flow. Figure 6-4 supports this assertion by the substantially reduced water levels in the piezometers in June when compared to October, particularly in the south row; a tendency towards unsaturated flow suggests surface control. The west basin, on the other hand, shows a decrease in throughput between October and November, which was expected due to the increase in viscosity with no complications due to algae. However, increased viscosity also means a reduced $k$ which should lead to saturated conditions. Increased levels of saturation were not witnessed, therefore the difference must be attributed partially to surface effects. Visual inspection of the basins during the fall revealed very little algae accumulation on the basin surfaces, however it is expected that increases in solids other than algae were experienced. Increased solids levels were evidenced in Stanley, 1991 during the months of November and April. The difference in viscosity between 10°C and 1°C predicts a 37% difference in flow. Multiplying the November 15th flow by the factor 1.37 yields 10545 m$^3$/day, which is very close to 11219 m$^3$/day. Due
to the piezometers levels decreasing, however, these numbers must be seen as only part of the explanation; surface effects are also responsible for seasonal flow differences.

6.1.4 Differences between beginning and end of winter constant flooding period
Figures 6-6 and 6-7 provide insight into the winter flooding period of the basins. Recall that both basins remain flooded from sometime in November until March or April, depending upon the temperature. This prevents the ice cover from settling and embedding itself into the surface of the filters, thereby causing the flow to cease.

![Diagram](image)

Figure 6-7(A). Piezometer measurements on the West basin’s south row at the end of the winter application period, March 15th.

Clearly, the flow over the course of the winter flooding cycle has become unsaturated. The cause for this is straightforward as the author witnessed tremendous algae growth in both basins while measuring the piezometers on March 15th. The amount of algae in the basins was more than was ever present during the summer warmth and sunshine. This is not as unexpected as it at first may seem. In the summer, each basin is rested for 10 days after an application period of the same duration. This resting period serves to dry and at least inhibit if not kill the algae which accumulated during the flood. In the winter, however, the application period lasts well over 100 days with no intermittent resting period; this creates conditions under which algae thrive, regardless of the cold temperatures. As well, Kamloops receives many hours of sun during the winter. As can be expected from the surface being clogged by algae, throughput by March was
decreased to an average of under 5000 m³/day for both basins. Common winter throughputs (before significant algae growth causes clogging) range between 6000 and 9000 m³/day.

In Figure 6-7, the south row has slightly more water in its tubes than the north row. Levels, however, were so low that it is not thought that this is of any significance.

Sections 6.1.1 through 6.1.4 have studied various characteristics of the RI system’s flow. While different aspects were discussed, the common and most important finding was the fact that the basins have saturated flow for most of the year. This verdict dismisses the former assumption of surface control only. This conclusion has ramifications both on past and future work on the basins.

In its 1991 report to the City of Kamloops, Stanley asserted that lower flow is sometimes witnessed in November and April due to these months representing periods of increased solids entering the basins, a result of the turnover of cell 4. The flow reduction pattern could only be explained this way if the surface of the filter were dictating the flow. Typical piezometer results have indicated, however, that this is not the case and in fact the flow is dictated by the sand itself and perhaps the underdrain system (an in-depth analysis of this is presented in the following section). It seems, however, that during the winter, there is some surface interference, likely due
to the solids noted in Stanley’s 1991 report. One must be careful when calculating average monthly flow per basin in terms of $m^3$/day during the months of November and April, as it is generally during these months that the basins switch from one to two and from two to one running, respectively. Recalling that, during the winter months, both basins remain flooded with no resting periods. Flows during the winter months are therefore divided by two when flow per basin is desired. However, care must therefore be taken for the months during which the switch actually occurs, since part of the month should have its flow divided by two and the other part of the month should not. It was the author’s experience that exact dates of basin switching are often either difficult to obtain or non-existant. The findings of Stanley may be attributed partially to the use of unrepresentative numbers. It is not believed that flow changes are entirely due to increased solids, although some influence is certain. With respect to the piezometer data, the different gradients witnessed between the tubes are reflections of the permeability in that layer; however, there must be a continuity of flow from one section to the next in a vertical sense. Lateral flow is always a possibility, but would be less consequential due to the large distance even if the k’s were larger between piezometer nests. Also reflected by the piezometer data are layers within the basin, which will become evident upon evaluation of the basin permeabilities in Section 6.2.

6.2 In-situ Permeability
The hydraulic conductivity, or permeability, of each basin was calculated by using the water levels found in the measurements of the piezometers. The calculations were performed in a way similar to the calculations of sand column permeability in Chapter 5. Required therefore were a $dh$, a $dl$ and a flow, Q, for each resultant k. The flow numbers were accessed from the log book entry of the day the piezometers were measured. Once calculated, the results could be compared to the design permeability value. Before entering into the permeability results, it must be noted that all in-situ permeability values have a margin of error of 25%. This is due to the fact that the flow data for the basin effluent has a 25% margin of error. Reasons for this will be discussed in Chapter 7.

6.2.1 Calculation and comparison of east and west basin k values
The in-situ permeability, k, of the basins was calculated according to Darcy’s equation by measuring the water level within the piezometer tubes. The first attempt to find the k values of the east and west basins involved only the depth of ponded water and the water
depth within the deepest piezometer tube as \( dh \) and the basin sand depth as \( dl \). It was realized, however, that more information was available since each nest contains four piezometers, each of a different depth. This means that a permeability value could be calculated for three layers within each nest, those layers being between two piezometer tubes. Each pair of piezometers of successive depth defines the \( k \) for that layer. Since on a trial calculation these values were found to be different for each of the three layers, three values had to be calculated per nest and the lowest chosen as the controlling \( k \) for the area of the nest. As there are twelve nests per basin, twelve permeability values were averaged per basin to yield the average basin \( k \). A benefit of calculating the permeabilities in this manner is that the layer of lowest conductance is pinpointed for each nest. These layers can then be compared to see if there is one predominant driving layer of sand. The three layers 1, 2 and 3 were defined by the sand between 100 to 140 cm, 140 to 170 cm and 170 to 200 cm, respectively. Table 6-3 below displays the lowest \( k \) value for each piezometer nest and the layer in which it was found.

<table>
<thead>
<tr>
<th>nest location*</th>
<th>EAST</th>
<th></th>
<th></th>
<th></th>
<th>WEST</th>
<th></th>
<th></th>
</tr>
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<tbody>
<tr>
<td></td>
<td>( k_{\min} \times 10^5 \text{ m/s} )</td>
<td>layer</td>
<td></td>
<td>( k_{\min} \times 10^5 \text{ m/s} )</td>
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<tr>
<td>1</td>
<td>0.48</td>
<td>3</td>
<td></td>
<td>0.44</td>
<td>2</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>2</td>
<td></td>
<td>1.52</td>
<td>1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
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<td></td>
<td>0.62</td>
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<td></td>
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</tr>
<tr>
<td>4</td>
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<td>3</td>
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<td>0.98</td>
<td>2</td>
<td></td>
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</tr>
<tr>
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<td></td>
</tr>
<tr>
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<td></td>
<td>0.28</td>
<td>3</td>
<td></td>
<td></td>
</tr>
<tr>
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<td>3</td>
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</tr>
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<td>8</td>
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<tr>
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<td>2</td>
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<td>0.62</td>
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<td></td>
</tr>
<tr>
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<td>3</td>
<td></td>
<td>0.59</td>
<td>2</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

*refer to Figure 4-1 for nest geometry
As seen in Table 6-3, the average of these lowest k values for the twelve piezometer nests was calculated for each basin. As expected, due to its higher throughput history, the permeability of the west basin was found to be higher than that of the east. These numbers were averages for two days of piezometer measurements made in October; it is hoped that the proximity in date of measurement means minimum differences in viscosity. The average $k_{\text{min}}$ in the west basin was found to be $0.59 \times 10^{-5}$ m/s while the average $k_{\text{min}}$ in the east basin was $0.46 \times 10^{-5}$ m/s. The west basin therefore has a 28% higher permeability than the east basin. Average flow numbers for 1994 indicated a throughput difference of 30% between the basins (west being the one with higher throughput). The k values are therefore encouraging; they intimate that the differences which have always been present between the basins are inherent to the sand of the basins. The suggestion is therefore that, for reasons which will be discussed in Section 6.4, the east basin contains a higher percentage of silty sand, through which the wastewater has trouble flowing. Only the relative permeabilities are important when comparing the east and west basins; the aforementioned margin of error of 30% therefore is of no consequence here, as it is assumed that any bias in flow numbers is always in the same direction.

It was interesting to discover that the controlling layer in the east basin is fairly consistently layer 3, or the sand between 170 and 200 cm. Although the permeability numbers calculated within this layer differed, it was not by a substantial amount. The controlling layer within the west basin was not as evident. The lowest k values were found seven times in the second layer, between 140 and 170 cm, twice in layer 1, between 100 and 140 cm, and three times in layer 3. Once again, possible explanations for these findings will be discussed in Section 6.4.

6.2.2 Comparison of winter and summer k values

To verify that the flow reduction witnessed during the winter is due to the increased viscosity of the wastewater, permeability values for the basins were calculated using data from June and data from November. Since flow is defined by the permeability multiplied by the hydraulic gradient, a change in permeability directly affects the flow.
Furthermore, permeability is inversely proportional to viscosity by definition in Hagen-Poiseuille’s relation, equation 2-11. The viscosity of water decreases by 44% between 0°C and 20°C, as seen in Table 6-1. Using equation 2-11, a viscosity decrease of 44% therefore translates into a permeability increase of approximately 70%. Using Darcy’s equation with the data from June and November, there turned out to be between a 45-50% difference between the winter and summer k values, the summer being the higher number. The permeability results provide therefore an excellent indication that the principal difference in flow between seasons is due to differences in water viscosity. The algae factor could affect the permeability results of summer readings, which would explain why the increase was not the expected 70%. Although the basins remained predominantly saturated throughout the summer, there would have been a higher number of saturated zones if not for the algae. It should be remembered that Darcy’s equation, for practical uses, is valid only for saturated flow.

6.2.3 Comparison with design permeability value
An important finding of this study is the difference between the in-situ permeability of the basins, as shown in Table 6-3 and the design permeability. The k values calculated here are an order of magnitude smaller than the design k. This difference cannot be attributed to plugging since the piezometer data displayed saturated flow throughout both basins. To discover a potential explanation for this discrepancy, an examination of Stanley’s methods for determining the permeability is required. This information is presented and discussed Chapter 7.

6.3 Underdrain Capacity and Mounding Analysis
The discovery of saturated flow within the basins was an important one. It led to the conclusion that the surface was not dictating the flow. It also led, however, to two plausible filter controllers. The flow could either be controlled by the sand itself or by the bottom of the filter. The latter could entail a number of different possibilities. The capacity of the underdrain could be insufficient, the pipes could be clogged or the spacing of the underdrainage system could be inappropriate. A study to try to determine this controlling factor was therefore necessary. It was suspected that the sand itself was responsible for filter performance, but only by the systematic dismissal of all other potential factors could this be proven.
**6.3.1 Underdrain study**

Design diagrams of the underdrainage system in the Kamloops RI beds revealed the following specifics. The system consists of 150 mm pipe at a spacing of 3.125 m, running north-south. The 450 mm collection pipe from these smaller pipes runs east-west at the south end of each basin and empties into a 500 mm collection main which runs along the centerline between the two basins. This pipe leads directly to the river, where the effluent is discharged. Since the basins are approximately 200 m in length, there are 64 pipes in each, of approximately 65 m in length. The 150 mm pipes are laid level, while the 250 mm pipe has a varying slope which averages 0.255% and leads in either basin towards the centreline between the basins. The underdrain capacity for both the 150 mm and the 450 mm pipes can be calculated using these specifics. Knowing a typical flow value, basin area, pipe spacing, slope and a friction factor appropriate for the roughness of the pipe material (PVC), a nomograph can reveal the pipe size required to handle that flow. As stated in Chapter 2, a common nomograph to use is that defined by the equation of Hazen-Williams. Their empirical formula is as follows:

\[ S = 10.675Q^{1.852}/(C^{1.852}D^{8.8704}) \]  

(6-1)

where \( S \) is the slope, \( Q \) is the flow, \( D \) is the pipe diameter and \( C \) is the Hazen-Williams coefficient, representative of pipe roughness. The \( C \)-value for PVC is between 140 and 150 (Khan, 1987). The average slope for the 150 mm pipes approaches zero, as the plans state that these were to be laid flat; the 450 mm collection pipe has an average slope of 0.255%. Calculations for each basin differed in their \( Q \) value, as the west basin consistently has a higher throughput. The \( Q \) used was not that of the entire basin, but had to be the \( Q \) per pipe, since there are 64 per basin, as previously mentioned. For the west basin therefore, using an average basin throughput of 6800 m\(^3\)/day, or 78.7 L/s for the 450 mm pipe, and dividing this by the 64 pipes, yields an average expectancy of 1.23 L/s per pipe for the 150 mm pipes. Similarly for the east basin, using an average basin flow of 5200 m\(^3\)/day, the result is 60.2 L/s for the 450 mm, and 0.94 L/s for the 150 mm. Equation (6-1) for the 450 mm pipe recommends a pipe diameter of 300 mm for the west basin and 290 mm for the east basin. The fact that the pipes are 50% larger than required means that flow is not being blocked by lack of capacity in the collection main. Again, using the middle \( C \) value of 145 and a slope of 0.0001 (arbitrarily small) for the smaller
pipes, equation (6-1) yields a D of 110 mm for the west basin and 100 mm for the east basin. Here again, the actual pipes are of a much greater size than is required by the dynamics of the flow. It can therefore be concluded that, barring any clogging, the underdrain capacity is more than sufficient for the currently experienced flows.

Now that the pipe sizing is known to be acceptable, any clogging problems must be investigated. As the only way of determining this with perfect accuracy would be to rip open both basins in order to expose the undrains, the method used here is admittedly flawed. It is, however, the best that could be achieved under the circumstances. Figure 6-8 below depicts a piece of underdrain pipe which was found during the excavation process for the installation of the piezometers. Since the filter performance before and after the piezometer placement did not alter, it has been assumed that this piece of pipe was not an integral component to the system and its removal therefore did not cause any negative effects.

![Figure 6-8. Section of underdrain pipe and filter cloth.](image)

The underdrain pipe can be described after personal inspection as follows. The pipe material is a hard, black plastic of about 1mm thickness; it seems firm and
imcompressible, although the mass of sand under which it lays is substantial and must cause at least some deformation. The pipe is covered with slots of 1.5 to 2 cm in length and 2 mm in width. They are spaced approximately 5 to 6 cm apart lengthwise and 3 cm widthwise. The pipe is covered with a filter cloth to avoid clogging. This cloth is beige in colour, made of a stretchy mesh. At first glance, it was surprising that this pipe, along with others like it, was responsible for the drainage of an entire basin. The slots did not appear to be large enough to allow for adequate infiltration. Tests on this piece of pipe were therefore performed to ensure that sufficient throughput was at least possible; they consisted of holding various parts under a direct stream of tap water. Water throughput was tested on the cloth alone, the pipe alone and on the combination of cloth and pipe. The results were surprisingly satisfactory. The cloth on its own seemed to pass the stream of water at the same rate as from the faucet. The throughput from the pipe on its own was difficult to observe, as the only available piece was severely misshapen. The combination of cloth and pipe, as is found in the basins, seemed good. It was impossible to observe the exact change in flow of the stream inside and outside the pipe, but it was certain that the rate or volume was not severely decreased. It can therefore be concluded that the underdrain system physically is not responsible for the flow limitation. It is adequate in both carrying capacity and physical attributes.

6.3.2 Mounding analysis

There remains one final area which requires examination before it can be concluded that the sand dictates the basins’ flow capabilities. It is a mounding analysis which must be performed, and is done using Hooghoudt’s equation, as presented in Chapter 2:

$$S^2 = 4k\left(\frac{H^2 - h^2 + 2dH - 2dh}{\nu}\right)$$

(2-14)

where

- \(S\) = drain spacing
- \(k\) = permeability
- \(H\) = mound height
- \(h\) = drain height
- \(d\) = distance between underdrain pipe and impermeable layer
- \(\nu\) = infiltration rate
This equation can be simplified for the Kamloops system, as the distance d is taken to be zero. It is thought that the underdrainage systems lies directly on top of the impermeable layer. Even if this is not exactly the case, taking d to be zero provides a worst-case scenario. Thus, equation (2-14) simplifies to:

\[ S^2 = 4k(H^2 - h^2) / v \quad (6-2) \]

Also known for the Kamloops RI system is h, the drain height; this is taken as the height of the pipes, 150 mm. The spacing between underdrains pipes is known from the designs to be 3.125 m. All that remains therefore is the permeability and infiltration rates for the east and west basins. The k values are taken as those averages found and presented in Table 6-3. The infiltration rates are those averages also used in the calculation of the permeabilities. A mounding height which is calculated using these numbers can be considered as a non-winter average. As has been previously explained, winter calculations differ in permeability due to the high viscosity of the water and in flow due to the conditions tending towards unsaturation by the end of the season. There are therefore two different sets of mounding height results. Once a value for H has been determined, it must be compared to the heads which are present within the filters, as indicated by the piezometer readings. The permeability numbers required by equation (6-2) are those obtained from the lower layer of the basins, as it is here that the mounding occurs. Using therefore a k of 0.82 x 10^{-5} and a flow of 6800 m^3/day for the west basin and a k of 0.48 x 10^{-5} and a flow of 5200 m^3/day for the east basin, equation (6-2) yields a non-winter maximum mounding height of 1.33 m for the west basin and 1.67 m for the east basin. Average winter flows are 5000 m^3/day, which, when roughly divided 60/40 in favour of the west basin, means an average west flow of 3000 m^3/day and an average east flow of 2000 m^3/day. Simply taking 60% of the summer k values can yield appropriate winter k values. Calculations using these winter flows and k-values yielded 1.14 m for the west basin and 1.34 m for the east basin. These mounding heights can now be compared to the available head to ensure that these are approximately equal; this will show that the basin is experiencing no significant restrictions other those caused by mounding.
The first step in determining available head is to calculate the exact vertical distances between the deepest piezometers and the underdrains. (The deepest piezometer's water levels shall be used to represent the head.) This is done by using measurements of the basin surface, including distances between piezometer nests, made recently on the basins both by Stan Reckinec from the City of Kamloops and the author. Also required is known design slope. From these numbers, basin cross-sections can be constructed. Horizontal spacing of the piezometer nests to the closest underdrain pipe can range from 0 if directly above a pipe, to a maximum of 1.56 m, half of the total space between pipes. The diagrams and data used for these calculations can be found in Appendix C.

Figure 6-9 compares the calculated head availability and the maximum mounding heights for both basins. The value for head was received by taking slope, depth of deepest piezometer and water depth in that piezometer into account.

![Figure 6-9(A). West basin's available head and mounding height for average non-winter flows.](image)

As is visible between parts (A) and (B) of Figure 6-9, in certain areas of the basins, there is some flow restriction for which mounding is not responsible. The points on the graphs at which the heads exceed the maximum mounding height indicate that here the flow is impeded. The data representative of winter head availability when compared to mounding heights are similar; most of the heads are below the calculated maximum mounding heights, particularly again in the west basin. Also of interest in the illustrations is fact that groupings between the north and south rows (ie. 1/7, 2/8) have
different heads. If on the same longitude, it would be expected that the mounding between underdrains beneath would be the same and the head requirement therefore also equal. However, the piezometer nests were not placed with the precision of being on the same longitudinal lines; they can deviate by between 2 and 5 meters. This phenomenon is not as noticeable in the east basin 6-9(B), as these nests were placed with a higher degree of accuracy.

![Piezometer location](image)

**Figure 6-9(B).** East basin’s available head and mounding height for average non-winter flows.

A significant difficulty encountered when trying to compare the mounding heights to the available head within the basins is the fact that it is impossible to determine the exact locations of the underdrain pipes beneath the sand, since this was never recorded during construction. All that is known is their spacing relative to one another, but not with respect to any known point within the basins. The point at which there is a discrepancy may not be exactly in between two underdrain pipes, where the maximum mounding occurs. Therefore, where the head exceeds the maximum mounding height, it is difficult to determine how much of this head is required by the mound and how much is required by some other blockage. A comparison of this system’s drain spacing to recommended spacing from literature reveals that Reed and Crites’ typical spacing of 15 m is close to five times greater than the 3.125 m of the Kamloops filters. This, coupled with the previous disclosure of more than adequate pipe diameter, suggests that the most of the levels witnessed in Figure 6-9 are not representative of a blocked system. Only those locations at which the head exceed the mounding are restrictions indicated.
It can now be concluded that the sand itself is the flow-determining factor. The underdrain system is adequate in carrying capacity, is not likely to have clogging problems, and the flow is assumed not to be hindered by significant mounding. It is, however, hindered in some areas by the sand as was seen in Figure 6-9. This conclusion has many ramifications for future basin design and will be discussed in detail in Chapter 7 and Chapter 10.

6.4 Differences Between East and West Basins: Possible Sources.
Since the start-up of the basins in late 1985, differences between east and west basin throughput have been apparent. A number of factors could be contributing to this, including the calculated differences in k. The fact that the west basin's permeability was found to be 28% higher than the east basin's can be considered significant. Granted, the Darcy method used for calculating k is open to error in terms of the crude piezometer measurements which are used; nonetheless, both basins were measured by the author, using the same procedures and any bias is assumed to be in the same direction.

In the east basin, a City of Kamloops employee witnessed a compact layer resembling clay which was clearly visible from the holes being dug for the piezometers. He saw this distinct layer in more than one of the pits and estimates its depth to have been around 1 m. He did not get a look at the west basin and so could not comment on any non-uniform layers there.

The areal dimensions of the basins as they currently function were found recently by the City of Kamloops to be the following. The west basin has dimensions of 198.7 m by 67.3 m, totalling 13373 m$^2$, while the east basin has dimensions of 199.6 m by 55.4 m, totalling 11058 m$^2$. This difference in surface area is both surprising, since the basins were designed to be the same size, and substantial, as the west basin is 21% bigger than the east. This size difference, while it does not account for the entire difference in flow, it certainly seems like the most obvious place to start attributing the discrepancy.

6.5 Basin P Removal
Before designing new basins, certain questions with regard to the functioning of the current basins must be addressed. The fact that a basin does in fact have the ability to retain phosphorus must be
must be ascertained. Proof of this is not as straightforward as it may seem. While the basin effluent phosphorus concentration is almost always under the limit of 1.0 mg/L, the P concentration of the basin influent is often under this concentration, and therefore the basins are not highly taxed. The 1987 Interim Report prepared by Stanley and Associated for the City of Kamloops stated the following:

"The basin continues to provide polishing of the effluent applied but the degree of phosphorus removal achieved in the filter appears to be reducing. This is as expected as the filter sand has low phosphorus adsorption capacity; little impact on phosphorus was predicted for the rapid infiltration basins."

As noted in the 1987 report, there is P breakthrough at certain times of the year, meaning the concentration of the basin’s effluent P is higher than that of the influent. This statement is backed up by the data reviewed by the author which spans the years 1991 to the end of 1995.

6.5.1 Removal with depth
Towards the end of winter as the ice was breaking over the basins, a number of samples was taken from both east and west basins. It was desired that samples be obtained from each of the piezometers within each nest, so as to ascertain phosphorus removal with depth. Sampling from each tube, however, proved to be impossible at this time, as the basins, especially the west, were almost entirely unsaturated. There was tremendous algae growth which was plugging the basins from the surface, thereby rendering the flow unsaturated and many tubes were therefore empty. Thirty samples in total were obtained, from various depths and tested in the UBC Environmental Engineering Lab for both total and ortho P. Results of total P were disappointing and not useful, as it seems that solids were contained within the samples. A table of the results can be found in Appendix D. Concentrations ranged from 1.0 mg/L up to 29 mg/L. A test solution was prepared and digested along with doubles of six different standards; this test solution was of known concentration and the results were satisfactory, meaning the sample results were to be accepted as well. Such high P concentrations in some of the samples therefore meant that solids were present and the results could not be used to indicate any P removal with depth. The results of the ortho P testing were more satisfactory, as these samples were filtered before measurement on the AAS.
While the results of the ortho P concentrations were accepted, they were nonetheless surprising. It was expected that at least some treatment would be occurring throughout the column of sand, as historical documentation has shown removal between influent and effluent concentrations. Table 6-4 below displays the average ortho-P concentrations at the various depths (representative of the surface plus the four piezometer tube depths).

<table>
<thead>
<tr>
<th>depth in cm</th>
<th>average ortho-P concentration in mg/L</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>east basin</td>
</tr>
<tr>
<td>0 (basin influent)</td>
<td>0.19</td>
</tr>
<tr>
<td>100</td>
<td>0.32</td>
</tr>
<tr>
<td>140</td>
<td>0.21</td>
</tr>
<tr>
<td>170</td>
<td>0.38</td>
</tr>
<tr>
<td>200</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Figure 6-10 below illustrates the findings displayed in Table 6-4 and indicates the unexpected results. An explanation for the apparent P uptake at certain points throughout the column, as well as the differing influent concentrations between the east and west basins, is the proliferation of algae that was apparent during time the samples were collected (early March). It should be noted that the samples, being taken toward the end of the winter flooding period, were therefore collected under conditions of unsaturated flow. Further analysis of these sample results was not thought to be worthwhile due to this fact as well as the fact that no measurement was made at the surface of the sand. Soil samples taken from the surface will be discussed in Section 6.5.3.
As stated previously, these data do not indicate any patterns of use in figuring out phosphorus removal through the column. The results of Section 6.5.3 are more revealing.

6.5.2 Effect of scarifying the surface ie. P removal in terms of cycle day

Toward the end of each drying period, the surface mat which has formed during the previous flooding period is broken up or 'scarified' by a tractor. The purpose of this is to allow for increased throughput during the subsequent flood. Due to the piezometer indication of saturated flow throughout the basins for most of the year with the exception of early spring, this study contends that scarifying the surface of the basins prior to each application period in fact may not have any effect on basin performance. Flow through the basins is dictated by the sand alone. To ensure, however, that scarifying has no hidden benefit or defect, such as affecting phosphorus removal, an analysis of phosphorus removal through the basin with respect to cycle day (ie. day 1 is the first day of flooding and hence the time closest to basin scarification) was performed. This study entailed the acquisition of daily basin data and log books. It was hoped that data dating back to the beginning of the basins in 1985 through to the present could be used, but data prior to 1991 was not to be found. Data used included reports on cell 4 effluent P concentrations and Cinnamon Ridge Effluent Disposal System (CREDS) effluent P concentrations. The former served as influent P concentrations. These data, too, were only weekly and therefore averages often had to be used. Results of phosphorus removal versus cycle day are presented below in Figure 6-11 in the form of c/c₀ versus cycle day. Recall that c/c₀,
represents the effluent total P concentration divided by the influent total P concentration.
The values plotted are averages of all values for each cycle day; the number of values involved in this averaging ranged between one and eleven.

Figure 6-11. Average basin P removal according to cycle day.

As Figure 6-11 illustrates, there is no distinct relation between phosphorus removal and cycle day, meaning no direct link to scarifying. It was thought that perhaps if much of the P uptake occurred at the surface, then the uptake would surely increase the further along the cycle, as the surface mat built up.

Stanley and Associates, in their 1987 Interim Report, suggested that the practice of discing the basins could even reduce the flow. This assertion was made under the assumption that the surface was dictating the flow, which is not the case. Stanley’s reasoning, however, was sound. Discing could lead to a loss of porosity as the pores become blocked by trapped algae which swell upon flooding.

6.5.3 Net basin P retention
Tabulated below in Table 6-5 are the average flows and phosphorus concentrations for the basins from the beginning of 1990 to the end of 1995. These concentration and flow values were obtained from data taken by the wastewater treatment plant employees and compiled by Leo Albrecht. The final column of phosphorus retained within the basins was calculated by multiplying the difference in influent and effluent phosphorus concentrations by the flow through the basins.
Table 6-5. Phosphorus retention by RI basins.

<table>
<thead>
<tr>
<th>year</th>
<th>$P_{in}$ in mg/L</th>
<th>$P_{out}$ in mg/L</th>
<th>$P_{in} - P_{out}$ in mg/L</th>
<th>flow in m$^3$</th>
<th>$P$ retained in kg</th>
</tr>
</thead>
<tbody>
<tr>
<td>1990</td>
<td>0.92</td>
<td>0.37</td>
<td>0.55</td>
<td>2 460 000</td>
<td>1353</td>
</tr>
<tr>
<td>1991</td>
<td>1.05</td>
<td>0.58</td>
<td>0.47</td>
<td>2 060 000</td>
<td>968</td>
</tr>
<tr>
<td>1992</td>
<td>0.62</td>
<td>0.45</td>
<td>0.17</td>
<td>2 660 000</td>
<td>452</td>
</tr>
<tr>
<td>1993</td>
<td>0.54</td>
<td>0.44</td>
<td>0.10</td>
<td>2 063 000</td>
<td>206</td>
</tr>
<tr>
<td>1994</td>
<td>0.53</td>
<td>0.36</td>
<td>0.17</td>
<td>2 083 000</td>
<td>354</td>
</tr>
<tr>
<td>1995</td>
<td>0.77</td>
<td>0.45</td>
<td>0.32</td>
<td>2 085 000</td>
<td>667</td>
</tr>
<tr>
<td>total</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>4000</td>
</tr>
</tbody>
</table>

Calculations of SSC for the total mass of 4000 kg of phosphorus were done knowing the surface area, depth and average density of each basin. The total combined volume of sand in the basins is 55 000 m$^3$ and the average density is 1360 kg/m$^3$. Multiplying these two numbers together yields the total mass of sand in the basins which is $75 \times 10^6$ kg. The mass of phosphorus divided by this mass of sand yields the SSC, assuming all P has actually been retained within the basin sand. The calculated SSC is 53.5 µg/g which exceeds tenfold the calculated SSC of the column sand, 5 µg/g (see Chapter 5). The range of SSC provided by NOWAK when contracted out by Stanley to do in-situ testing of the sand was between 13.3 µg/g and 24.2 µg/g; the higher end of this range is still less than half of the calculated value of 53.5 µg/g above. It should be noted that the above assumption of all P having been retained in the sand to this date is false. In 1988, the surface of each basin was scraped for the purpose of increased throughput (this practice worked for a couple of months, after which time, the basins returned to their previous average flow rates). The amount of phosphorus removed during this scraping is impossible to determine (as the volume removed was not recorded), but it has been assumed, for simplicity’s sake, to be roughly equal to the unknown amounts of phosphorus that were retained by the basins between 1986 and 1989, the years for which there is no data. Although the P retained during those 4 years was most likely more than the amount taken out in 1988, it does not shift the focus of the result, which is that the basins, thus far, should have exceeded their SSC. A final set of tests performed on the basin sand was that of soluble phosphorus concentration within the surface layer of sand.
This was to determine whether this layer, with its high organic content, could be retaining this phosphorus which as yet was unaccounted for. Samples taken from the top 3 cm of the basins were found to contain 2.5% organics and 1224 μg P / g soil. This number is almost 250 times greater than the SSC found in the deeper sand which contained fewer organics and clearly proves the basins’ capability of retaining phosphorus. The fact that almost all of the P is retained in the surface layer demonstrates that the depth of sand is not crucial to P removal and that surface organics should not be removed. Sometime in the future, however, their accumulation may begin to pose flow problems, at which time removal may become mandatory.

From the Operational Statistics data retrieved for the years 1992 through to 1995, the periods during which phosphorus was actually released (meaning c/c₀ > 1) were noted. In 1992, the average c/c₀ was greater than 1 for the months of February, April, May and June. In 1993, parts of April, May and July had releases. In 1995, the months of May and June recorded higher effluent than influent P concentrations.

6.6 Sand Characteristics
An investigation into the state of the basin sand required grain sizing with depth and a bacteriological examination to be performed.

6.6.1 Grain sizing
A group of samples was extracted from various depths within both east and west basins and analyzed in terms of grain size. The results are presented in Figure 6-12 according to the depth from which they were extracted. Interestingly, the sample depth made no significant difference to grain size distribution. This was not expected, as often in sand filters, there is an increase in grain size with depth. This is due to the fact that most of the organics are present in the surface layer and what is left behind are silicates which are smaller in size than sand.
Figure 6-12. Grain sizing results for various depths averaged over both basins.

From Figure 6-12, the screen sizes through which 10% and 60% of the sand pass can be extrapolated. The value for 10% passing is known as the effective grain size, while that for 60% is known as the mean grain size. The literature generally suggests a uniformity coefficient of below 3.5 on average when describing appropriate media for rapid sand filters. Table 6-6 below displays the uniformity coefficients for the three samples depths in both basins.

<table>
<thead>
<tr>
<th>depth in cm</th>
<th>effective grain size in mm</th>
<th>mean grain size in mm</th>
<th>uniformity coefficient</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.11</td>
<td>0.32</td>
<td>2.91</td>
</tr>
<tr>
<td>50</td>
<td>0.10</td>
<td>0.28</td>
<td>2.83</td>
</tr>
<tr>
<td>100</td>
<td>0.10</td>
<td>0.31</td>
<td>3.14</td>
</tr>
</tbody>
</table>

As depicted in Table 6-6, the uniformity coefficients for all three depths are close to one another in value and have an average of 2.96. This is within the range recommended in the literature. It is lower than the uniformity coefficients calculated in Chapter 5 for the column sand which were 3.56 on average. This, too, however, is close to the recommended value. The values for uniformity coefficient listed in the table do not reveal any patterns with respect to changes in depth, as the value decreases between 10 and 50 cm, but increases again at 100 cm.
6.6.2 Bacteriological examination

The grain sizing results from the previous section revealed that the percentage of small particles (less than 63 μm) in fact increases with depth. However, these percentages overall are so small, that it is difficult to know how much credence to give the results. Furthermore, half a meter depth readings had consistently higher numbers of small grains, but 1m in depth consistently had lower numbers of small grains than the half meter depth values (although still higher than from the surface, at 10cm). To examine further the state of the sand, a bacteriological examination was deemed to be appropriate. It was thought that this would reveal a decrease in biological activity with increased depth.

<table>
<thead>
<tr>
<th>sample depth in cm</th>
<th>standard plate count (average)</th>
<th>yeast mold count (average)</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>$7.8 \times 10^6$</td>
<td>$5.7 \times 10^4$</td>
</tr>
<tr>
<td>50</td>
<td>$5.7 \times 10^5$</td>
<td>$6.0 \times 10^3$</td>
</tr>
<tr>
<td>100</td>
<td>$7.8 \times 10^4$</td>
<td>$1.1 \times 10^4$</td>
</tr>
</tbody>
</table>

The results listed Table 6-7 indicate that, without exception, bacteria numbers decrease (as expected) tenfold between 10 and 50 cm. Between 50 and 100 cm, however, the numbers are almost the same magnitude, with those from the deeper level a little higher. This could indicate that over the life of the filter, now ten years, the surface organics have mostly accumulated within the top half meter. Numbers remain high at 1m depth, however; an explanation of this could be that the sand which first filled the basins in 1985 already contained this biological life.

This study has uncovered numerous basin characteristics, as well as answered a number of queries which had to date arisen with regard to basin operation. It is important, before this information be used to make any changes to the basins or their operation, that it be compared to design data. The permeability and the type of flow experienced within the basins, among other things, are important parameters which disagree with Stanley’s design plans. Reasons for these
discrepancies will be examined in Chapter 7. As well, a summary of the basin studies’ important findings is presented in Chapter 8.
Chapter 7. Comparison of findings to Kamloops’ RI system design specifications
and interim reports.

This study has produced some results with regard to the functioning and the characteristics of the Kamloops’ RI system which differ from design specifications and literature pertaining to this system. The examination and explanation of these differences are pertinent to the understanding of the current system, as well as to any future basin design.

7.1 Permeability

Most of the queries which result from this study revolve around the design permeability. The k value has extensive ramifications in RI design, including its use in the calculation of expected basin throughput and underdrain spacing. The k value found in this study was an order of magnitude lower than the design k. The following is a summary of the methods employed by Stanley to arrive at their design minimum permeability value of 5 x 10⁻⁵ m/s.

Stanley hired NOWAK Geological Services Ltd. to perform the required site testing and lab analyses. The latter consisted of column tests, similar to those in the present study. Prior to column testing, a site analysis had to be performed. This included slug tests and sieve analyses. The desired location for the basins was known, and it was thought that sand present in the river scar channel could be used. The characteristics of this sand, however, had to be discovered. From the Pre-Design Report (1982), a 20m depth core revealed three distinct layers which NOWAK termed Units 1, 2 and 3. The in-situ permeabilities were calculated from various tests. Fifteen in-situ slug tests were performed and the data were analyzed using the Hvorslev Plot of logH/H₀ versus time. From this, one can then calculate k from the Hvorslev eq’n:

\[ k = r c^2 \ln(L/R) / 2LT_0 \] (2-12)

The result was an average k of 5x10⁻⁴ m/s. Percolation tests were also performed on both Unit 2 and Unit 3 sand. All procedures are described in NOWAK, 1983. The permeability values from Unit 3 were always too low (between 10⁻⁶ and 10⁻⁷ m/s). Unit 2 averages were 5x10⁻⁵ m/s. Also performed on the river sand were sieve analyses; results indicated sand permeabilities ranging from 10⁻⁵ to 10⁻⁴ m/s. The chosen k value is the average of this range, 5 x 10⁻⁵ m/s. It is unclear how many sieve analyses were performed, but other sources have maintained that over 1000
measurements are required for a statistically correct k value. The following table summarizes the description of the layers found in the report.

Table 7-1. Sand layers found by NOWAK in river scar channel.

<table>
<thead>
<tr>
<th>Unit 1</th>
<th>-silty loam</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>-38 - 72% passed through 74 μm (#200) sieve</td>
</tr>
<tr>
<td></td>
<td>-permeabilities were anticipated to be very low therefore layer was disqualified</td>
</tr>
<tr>
<td>Unit 2</td>
<td>-between 1.6 and 10.8 m in depth</td>
</tr>
<tr>
<td></td>
<td>-sand plus some sandy loam</td>
</tr>
<tr>
<td></td>
<td>-1 - 9% passed through 74 μm sieve</td>
</tr>
<tr>
<td></td>
<td>-permeability $5 \times 10^{-4}$ m/s</td>
</tr>
<tr>
<td>Unit 3</td>
<td>-between 1.4 and 7.4 m in depth</td>
</tr>
<tr>
<td></td>
<td>-sandy loam, silty loam and loamy sand</td>
</tr>
<tr>
<td></td>
<td>-15 - 74% passed through 74 μm sieve</td>
</tr>
<tr>
<td></td>
<td>-permeability estimated to be between $10^{-6}$ and $10^{-7}$ m/s; confirmed by chemical tests</td>
</tr>
</tbody>
</table>

As is evident from the information provided in Table 7-1, Unit 1 and Unit 3 were inappropriate due to their lower permeabilities. Unit 2 was therefore chosen as the sand which would be used to fill the Kamloops RI basins. Attention should be drawn here to the description of this selected layer. It lies between 1.6 and 10.8 m in depth; Unit 3, however, with its substantially lower permeability, is interspersed at depths ranging between 1.4 and 7.4 m. The actual hauling of sand therefore should have been a fastidious procedure. It is unknown whether or not this was known by the drivers of the front-end loaders which performed the cutting and filling of the sand. It appears that due to the differences in layers which would not be visible to the eye, parts of Unit 3 could easily have been included in the packing of the basins. If this is the case, then the lower-than-design permeability which has been observed in the basins can be explained. As mentioned in Chapter 6, a worker witnessed a layer of clay at a depth of approximately one meter in the east basin; he was unable to comment on the west basin, as he did not see it during any digging stages. This clay layer is certain to be contributing to the low permeability values of the east basin.
Using the sand chosen from the site tests, a series of column tests was performed by NOWAK. The sand depth in the columns was 30 cm. They were packed with 55% medium sand, 44% fine sand and < 1% silt. The current study found 3.3% silt in the cores analyzed from the basins. This difference in percentage silt is small, but it can be significant with respect to differences in hydraulic conductivity. Densities were similar; NOWAK found the sand to have densities between 1.5 and 1.6 g/cm$^3$ while this study’s tests found densities between 1.4 and 1.6 g/cm$^3$.

Obviously, a number of different methods was used by NOWAK to calculate the long-term permeability of the sand to be used in the pilot-scale basins. It is therefore the contention of the author that the research into the permeability is not to blame for the discrepancy between the design and in-situ $k$ values. It is far more likely that difficulties were encountered in the accurate excavation of sand from Unit 2; it is probable that portions of Unit 3 sand were unwittingly included in the packing of the basins. Interestingly, NOWAK predicted a long-term permeability of $5 \times 10^{-6}$ m/s for Unit 3 sand. It is this value of $k$ which was also calculated by the author as the current in-situ permeability. It should be stressed that a sand filter’s flow will be affected by its lowest permeability layer. Thus, only a small portion of Unit 3 sand would have had to be mistakenly included in the basin construction for an alteration in basin permeability. Further support of this allegation comes from NOWAK itself when concerns were expressed over the choice of Unit 2 sand for the basins. Its first concern was the large excavation volume required. Secondly and more importantly, concern was expressed with respect to the availability of suitable $5 \times 10^{-5}$ m/s backfill material. Metcalf and Eddy (1991) warns against cut and fill construction, such as that used in Kamloops, as it can adversely affect the permeability of the surface soils.

As briefly mentioned in Chapter 6, the flow numbers required to calculate the permeabilities in this study contained a margin of error of approximately 25%. The rapid infiltration basins do not currently contain flow meters for either incoming or outgoing flow. The flow is measured by means of meters which measure the volume of wastewater which leaves cell 4 from the south side of the river. When spray irrigation is not functioning, RI flow is taken to be 100% of the amount which leaves cell 4. When spray irrigation is functioning, it is metered and the RI flow numbers are therefore obtained by means of subtraction. There are numerous inaccuracies involved with this method of flow measurement. It does not account for evaporation, which can be substantial during the hot, dry summers; as well, calculations made from the cell 4 meters are questionable. There are three pumps at the cell 4 outflow, each of which has a flow meter attached; these flow
meters function by means of time logging, meaning that from day to day, the numbers of hours on the dial may change from, for example, 456 to 464; this will indicate that the pump has been running for 8 hours. This number is then multiplied by the pump capacity, in terms of volume capability per hour. This capacity number should stem from a system curve, which takes into account the actual travel path between cell 4 and the north side storage container. However, the capacity number currently being used stems from the design pump curve, a number which is higher in value due to head requirements not being taken into account. Significant errors occur when all three of the pumps and hence meters are being used, which is the case some of the time. Otherwise, when just one or two pumps are operating, the values used are close to those dictated by the system curve. Actual numbers and further explanation of the flow calculation procedure can be found in Appendix E.

Even with these potential problems regarding flow numbers, it is not thought that the k values which resulted in Chapter 6 are unrepresentative. The flow numbers which were utilized, if not correct in absolute value, were always within the average ranges of throughput which were realized and recorded when the RI flow meters were operational.

7.2 Throughput, Safety Factor and Hydraulic Loading

Unit 2 sand was chosen of the three possibilities described in Table 7-1 which meant a long-term permeability of $5 \times 10^{-5}$ m/s was to be used in the basin design. The measured infiltration rate or permeability is typically multiplied by a chosen safety factor to obtain the hydraulic loading. According to the EPA's design manual, a value between 4 and 10% of this k should be chosen for the hydraulic loading rate. The inverse of whatever percentage is chosen then becomes the safety factor. For instance if 5% is chosen, this corresponds to $1/20$, meaning a safety factor of 20. Therefore, the EPA recommends safety factors ranging between 10 and 25, depending upon the individual situation and characteristics of the basins being designed. Cited from other literature in Chapter 2, safety factors for RI systems are commonly 20 for secondary effluent. The quality of the effluent to be entering the RI basins in Kamloops, however, was so high that Stanley selected a safety factor of only 10. Also noted by sources is the fact that in colder climates, higher safety factors are recommended.

Due to the fact that the basins remain permanently flooded during 4 months of the year (during winter), the safety factor is not straightforward. Stanley reports a safety factor of 5 during
application periods but leaves out the resting periods; literature, however, tends to incorporate resting periods. Therefore, the safety factor between March and November is 10 since one basin runs half the time and is rested during the other half. In the winter, then, when both basins are flooded continuously, a safety factor of 5 is in effect per basin, but it remains 10 overall since now there are two flooded basins. In a performance study of the basins after a number of years of operation, Stanley suggests that future basin design include a higher safety factor than 10.

As mentioned, Stanley’s design hydraulic loading uses a safety factor of 10 or, as previously explained, 10%. The following procedure describes how its design expectation of 10 000 m$^3$/day per basin evolved; this information was presented in Woods (1994) but some changes have been made to the 1994 analysis due to differences in data and ideas. First of all, the basins were expected each to be between 1 and 1.2 ha in surface area. The permeability tests on the sand to be used in design, Unit 2 sand, revealed that the minimum k would have a value of 5 x 10$^{-5}$ m/s; in terms of meters per day, this number is multiplied by 3600 s/hr and by 24 h/day to give 4.32 m/day. This translates into the following expected throughputs:

(i) 1.0 ha: 4.32 m/day x 10 000 m$^2$ x 1/5* = 8640 m$^3$/day

(ii) 1.2 ha: 4.32 m/day x 12000 m$^2$ x 1/5 = 10368 m$^3$/day

*the safety factor here is 5 since it is the throughput of a single basin being calculated

Thus, Stanley decided on the rough average of 10 000 m$^3$/day per basin. It is unclear from the design reports what the expectation for winter flows would be. Nowhere is 20 000 m$^3$/day mentioned for the time during which both basins are flooded. Perhaps the expectation remained at 10 000 m$^3$/day to be conservative with regard to changes in flow due to increased water viscosity in colder temperatures. From this number of 10 000 m$^3$/day, the hydraulic loading rate in m/yr can be calculated. The design intention was a total basin area of 2.4 ha and 10 000 m$^3$/day which means 3 650 000 m$^3$/yr (this assumes their assumption of 10 000 m$^3$/day for both basins together during winter flow). This number divided by the total area results in 152 m/yr.

A second way of arriving at a yearly throughput is as follows. Their k value of 5 x 10$^{-5}$ m/s translates into 1577 m/yr when multiplied by 3600 s/hr, 24 h/day and 365 d/yr. Now multiplying this by their safety factor of 10 yields an expected hydraulic loading rate of 158 m/yr. This number closely corresponds to the 152 m/yr which was the result of the previous calculation.
using 10,000 m$^3$/day. Regardless of the source for the number, the chosen hydraulic loading of approximately 150 m/yr is higher than any of the U.S. examples in the EPA’s manual, which are between 24 and 122 m/yr for secondary effluent as RI influent. It is unknown why the Kamloops system would expect any higher loadings, especially as it must endure the viscosity effects of a colder climate.

Discrepancies within historical flow numbers should be mentioned here; there are occasional disagreements between numbers from different reports which are meant to represent the same entry. For example, the number in cubic meters for total RI throughput in 1994 is less than the throughput number which is said to represent only January through October of 1994. Disagreements such as these consequently indicate that it is not certain which numbers used here are truly representative. However, due to the fact that there is only one source for RI data, this problem cannot be solved. It is thought, nonetheless, that any error in numbers is comparatively small. All flow data pertinent to the present analysis can be found in Appendix F. Using historical flow data, the average hydraulic loading which has been realized by the basins between the years of 1986 and 1995 is 83.3 m/yr. This is a poor comparison to the design hydraulic loading of over 150 m/yr. Back-calculating a safety factor using the actual hydraulic loading of 83.3 m/yr and the design hydraulic permeability yields a value of 18.9:

\[
(4.32 \text{ m/d})/(83.3 \text{ m/yr} \times 1 \text{ yr} / 365 \text{ days}) = 18.9 \text{ or 5 %}
\]

This result indicates that for the basins’ performance to date, Stanley would have needed a safety factor twice as high as that which it used, if its design k had been realized. The actual safety factor in use using this study’s calculated average k of $0.53 \times 10^{-5}$ m/s (average of .46 and .59) can also be calculated; the average k translates into 0.46 m/day. Using the average yearly loading of 83.3 m/yr, the safety factor is therefore:

\[
0.46 \text{ m/day} / (83.3 \text{ m/yr} \times 365 \text{ days/yr}) = 2.0 \text{ or 50 %}
\]

This exceedingly low safety factor of 2 illustrates the fact that if Stanley’s design k had fit the actual basin k, their use of a lower safety factor would have been substantiated. Again, it should be stressed that none of the designers’ methodology is being questioned by this study; it is the practicality of having been able to obtain Unit 2 sand only during the construction process which is debatable.
7.3 Dosing Frequency

The flood/dry cycle of the basins has changed since their start-up in 1985, although the ratio has always been 1.0. Stanley's design recommended a 14-day flood followed by a 14-day drying period. The throughput being realized by the basins after a few years of operation, however, was below expectations and this cycle was shortened upon the recommendation of the 1987 interim report to a 10-day flood followed by a 10-day drying period. It was anticipated that this change would increase the throughput since surface clogging was thought to be the cause of the reduced throughput. However, the findings of this study, as presented in Chapter 6, show that it is not the surface which controls the flow through the RI basins. It appears that the Stanley's two choices of flood/dry times were related to promoting the regenerating of adsorption sites, as recommended by NOWAK. Many of the works cited in Chapter 2, such as Lance (1977) and Metcalf and Eddy (1991), however, recommend shorter application periods than drying periods, meaning ratios of less than 1. Stanley's design cycle and current cycle both have ratios of 1.0.

The findings of this study with respect to application periods deal only with drying time. As discussed in Chapter 5, there is a marked decrease in percent removal of phosphorus at drying times below 3.5 days. This suggests that drying times be kept above this number. However, the improved performance of the filter at drying times above this number, while existant, is not drastic. Between 3.5 and 41 days, there was only a small improvement in removal which leads to the conclusion that extended drying times are not worth the decreased throughput that would result. NOWAK, however, found no relationship between SSC and dosing frequency. Also with respect to flood/dry cycle, it was found in Chapter 6 that the changes in flow encountered between the beginning and end of an application period were not drastic. Only during the summer did the flow become less saturated, due to the accumulation of algae at the surface. There was also algae accumulation on the surface towards the end of the winter flood, due to its extended length of over 100 days.

7.4 Flow Characteristics

It is evident from the 1987 and 1991 studies performed by Stanley on the Kamloops RI system that an assumption of surface flow control was made. Suggestions to increase the throughput inevitably involved changes to the surface. Furthermore, the current practice of surface scarification exists after each basin had been rested, before a new application period begins. This
is for the purpose of breaking up the 'flow-impeding' surface mat. However, the present study uncovered the important fact that the basins have saturated flow. This inherently indicates that the flow in controlled elsewhere. The choices of controlling factor were the sand itself or the underdrainage system. The latter has been discounted as a possibility after calculations of mounding heights and underdrain capacity in Chapter 6 revealed that no problems were to be found. The conclusion is therefore that the sand itself dictates the flow within the RI basins and the surface does not play a significant role. This behaviour is only altered towards the end of the winter flooding period, at which time the algae accumulation is significant enough to begin clogging the filters at the surface. Consequently, the validity of the scarification process being performed year-round is questioned.

7.5 SSC
A difference in results of the river sand's sorption capacity has also arisen. The calculated SSC's of this study were significantly lower than those from NOWAK's column tests (NOWAK, 1983), performed in 1982. It should be noted that the influent phosphorus concentration of this study was also significantly higher. NOWAK's average influent P concentration for their column tests was 2.18 mg/L, while a rough average for the present study was 5 mg/L. NOWAK's SSC values ranged from 13.3 to 24.2 µg/g, whereas the author's results ranged from 5.3 to 9.2 µg/g. Results from surface samples, containing 2.5% organics, revealed an uptake of 1224 µg/g. Sampling of this type was not an option for NOWAK due to the time required for a filter's surface to build up significant organic content. Various other tests with SSC and break-through curves led NOWAK to the conclusion that SSC is dependent on P concentration; also realized was the fact that the highest SSC was obtained under the highest application rate.

On pages 30 and 31 of NOWAK's report, it is contended that, with regard to P removal mechanisms, rapid adsorption occurs only in the unsaturated zone and slow precipitation occurs in the saturated zone. This, if true, means that most P removal realized by the Kamloops system must be due to precipitation since the basins are saturated (during periods of no significant solids accumulation). However, NOWAK asserts in its report that the underdrains of CREDS, installed to control groundwater mounding, mean that P removal is mostly from adsorption (unsaturated zone) since the residence time in the soil is cut down. This study has found that by far the most
significant removal possibility within a sand filter lies within the build-up of organics at the surface.

The differences between this study's findings and the basin designs have been investigated. The insight gained into the workings of the Kamloops RI system can be used to alter previously held misconceptions and to improve the system performance. The cumulative results from the column test and basin studies, which are summarized in Chapter 8, can be combined to formulate a number of conclusions and recommendations which involve the future of the Kamloops RI system. These can be found in Chapters 9 and 10, respectively.
Chapter 8. Summary of Results

This study was proposed to the City of Kamloops for the purpose of determining characteristics of the current rapid infiltration system, as well as solving its operational problems. It is known that the basins are not and in fact never have been working according to their design specifications; their throughput has been consistently disappointing. Investigations into the flow characteristics as well as the physical properties of the basins sand were therefore undertaken. As well, there was an interest in the sorptive capacity of the filter sand with regard to phosphorus. The current practice of phosphorus removal using alum is undesirable due both to expense and the resultant aluminum content of the sludge. To investigate the SSC of the sand, column tests were designed and executed. The results from both areas of the investigation are summarized to provide insight into the basins as they currently operate, as well as to form recommendations with respect to future basin design and use.

8.1 Column Tests
The two series of column tests were described in detail in Chapter 3 and Chapter 5. The principal goal of these tests was to formulate an idea of the sorptive capacity of the basin sand for phosphorus. Important aspects which were also continuously monitored were flow and overall treatment of wastewater. The investigation was, however, complicated by the use of actual wastewater from the treatment plant. From the description of the treatment train at the City of Kamloops’ plant in Chapter 1, it is evident that many different influent sources were available to the columns. As a result, three influents of varying phosphorus concentrations were used. The average influent P from 2C was 6 mg/L; the average from wetlands was 3.5 mg/L and the average from cell 3 was 1 mg/L. These influents were applied to columns containing sand depths of 60 cm, 90 cm and 120 cm.

8.1.1 Series I
Series I of the column tests produced a number of results regarding overall treatment, flow and phosphorus removal. With respect to overall treatment of the three different influents, the 90 cm and 120 cm columns were similar in performance; the 60 cm column produced poorer results. The efforts to obtain deeper filters by combining columns were rewarded with only slightly higher removals. The reduction in flow which results from depths greater than 120 cm is not worth the insignificant increase in treatment. Flow
testing was at first hampered by algae accumulation on the surfaces of the filters. This problem, however, was solved by the covering of the columns with an opaque plastic which discontinued sun penetration along the column. This state was considered to be more closely simulating the actual state of the basins. With no algae hindrances and therefore saturated flow through the columns, permeabilities could be measured using Darcy’s equation. These averaged $2 \times 10^{-4}$ m/s. This is an order of magnitude higher than the design $k$ and two orders of magnitude higher than the in-situ basin $k$ calculated in Chapter 6. Literature recommends that column test permeabilities not be used for RI basin design due to their high and consequently misleading values. The calculated column $k$ was therefore best used for purposes of comparison of different influents. Influent from 2C was consistently higher in suspended solids and therefore the columns experienced lower flows.

It was quickly discovered during the first series of column tests that the basin sand was unable to remove sufficient phosphorus from 2C influent. The river discharge permit requires an effluent $P$ concentration of no greater than 1.0 mg/L. As such, cell 3 influent was satisfactory since its concentration upon entering the filter was already within permit levels. This, too, however, was disregarded as a potential influent, as it closely resembled that which currently enters the RI basins from cell 4. Cell 3 wastewater has already been treated with alum, while the column tests were aimed at stretching the sand’s capacity to remove $P$. The column influent chosen as a result of Series I was therefore wetlands. This influent had potential in that it represented a wastewater which had approximately half of its raw influent $P$ concentration removed. This was due to the uptake of nutrients by the cattails and bullrushes. However, it could also represent wastewater which had received half of the current alum dosage. This would allow for more freedom on the part of the City of Kamloops in future decisions. Also chosen as a result of Series I column testing was a sand depth of 120 cm. This depth produced longer breakthrough times than 90 cm. It also appeared to be the safest depth with regard to coliform removal. Furthermore, it was always kept in mind that the results of this project could be implemented by the City of Kamloops. A new depth of 120 cm for the RI basins would imply a removal of 100 cm of sand from the current basins. This already represents a substantial change.
Also investigated during the first series of column tests was the soil sorption capacity. This averaged 5 µg/g, a value substantially lower than the values produced by NOWAK (1983) which ranged between 13.3 and 24.2 µg/g. The column test data were a poor fit to both Langmuir and Freundlich isotherms; this was evident by the low correlation coefficients, or R values. An interesting characteristic however emerged from the isotherm data; it was discovered that the 90 cm column sorbed more than the 120 cm column. Because a SSC value is in terms of mass of P divided by mass of soil, these results were interpreted to mean that more sorption occurs in the surface layers. The only difference between the two depths in terms of SSC should be the fact that there is a larger mass of soil in the deeper column, therefore producing a seemingly lower SSC value. These results seemed to contradict the P removal with depth results which illustrated, with one exception, higher P removal with higher depth. These results, however, are disputable due to the varying degrees of exhaustion of sand from different columns. More credence is therefore given to the isotherm results.

A final important discovery from the series of column tests was an optimum basin drying time of 3.5 days. Below this, P removal is decreased; above this time, removal increases slightly but not significantly enough to validate a decrease in throughput.

8.1.2 Series II
The second series of column tests was executed during colder weather and therefore produced lower permeability values. Two 120 cm columns, with wetlands applied as influent, were directly compared in terms of flow and treatment capability. One column was flushed from the bottom to expel any air pockets prior to application and the other was not. The k value of the flushed column was slightly lower than that of the unflushed column. This can be explained by the settling of the sand in the flushed column relative to the unflushed column. The time to breakthrough in terms of phosphorus was higher for the flushed column, a value of 5.5 hours compared to 3.6 hours for the unflushed column. Again, these data had a poor fit to both Langmuir and Freundlich isotherms. The flooded column therefore experienced somewhat lower flow but a slightly higher SSC. Flushing of the filters prior to application is therefore not deemed to be advantageous. Interpretation of the results is a moot point, however, since flushing of the Kamloops system would likely not be necessary since the basins appear to be saturated.
The column tests produced a number of findings. Basin sand, such as that contained in the current RI basins in Kamloops, does not have a great phosphorus uptake capability. Due to this result, it has been concluded that the complete termination of alum addition is not possible. The addition of approximately half the current alum dose produces an effluent which parallels wetlands effluent. The application of this to the basins would produce satisfactory effluent in terms of phosphorus for only a limited amount of time. Variations on a system of this type will be presented and discussed in Chapter 10. Also concluded from the column tests was that a sand depth of 120 cm produced both appropriate flow and treatment results. This depth must be increased for full-scale implementation due to mound requirements. Before a depth can be recommended for the actual basins, results of the basin flow characteristics must be presented to ensure that the cutting away of sand will not compress the surface of the filter, thereby decreasing the flow. An optimum drying time for the columns was determined and can be implemented on the Kamloops system. Stanley's design specifications do not indicate their reasoning behind their choice of a 1.0 flood/dry ratio, nor their choice of 14 days per half cycle, or subsequent shift to 10 days per half cycle. Literature suggests a ratio of less than one, and often recommends time periods of less than 10 days. Overall, many of the findings of the column test, before being implemented, must be complemented by the findings of the basin testing, to gain insight into the current workings of the Kamloops system.

8.2 RI Basin Studies
The many tests performed on all aspects of the RI basins produced some very useful results. These results provide insight into the current operation of the basins, as well as lay the foundation for possible changes to the system by providing evidence of certain basin characteristics. The most important discovery about the Kamloops system is the fact that it experiences saturated flow during most of the year with the exception of the end of the winter flooding period. This fact opens the door to the opportunity of implementing changes to the basins in an effort to increase throughput, by cutting away sand. The discovery also negates certain assertions and assumptions which have been made to date with respect to the basins. Prior to this study, unsaturated flow through the basins has been presumed, but not verified. Problems in basin throughput have therefore been blamed on surface phenomena and not on the sand column itself. Prior to each application period, the basins are scarified to allow for increased throughput through the surface layer. This does not seem necessary under the conditions. However, there is some surface
control during the summer and winter and during these times, the practice should be continued. Furthermore, ideas of increasing the flow by cutting out of a portion of sand were deemed inappropriate, as surface compaction and further flow reduction could ensue. This fear, too, now appears to be unfounded. A second discovery made by this study, which is related to the discovery of saturated flow, is the fact that it is the sand itself which controls the flow. Calculations of mounding height and underdrain capacity have indicated no problems with the underdrainage system but did indicate blockages in certain areas of the basins where the piezometer levels exceeded the maximum mounding height. This must be kept in mind when redesigning basins with less sand depth.

The in-situ permeability of the basins was calculated and led to two conclusions. The first was the disclosure that the in-situ permeability, which averages \( 0.5 \times 10^{-5} \) m/s, is an order of magnitude smaller than the design permeability. It is thought that the sand which was studied and chosen for the basins in pre-design testing was in fact of the specified permeability. However, it is suggested that an error was made during basin construction; the chosen layer of sand was difficult to distinguish from other layers and therefore a challenge to obtain. Indeed, this sentiment was expressed by NOWAK (1983); stated in NOWAK’s report to Stanley was a concern regarding the “availability of suitable \( 5 \times 10^{-5} \) m/s backfill material”. If it was in fact an error in basin packing from the point of construction, then the expectation of a throughput of 10,000 m³/day by a basin has never been possible and it is consequently likely that the basins have been working according to design, but with an adjusted permeability. A second discovery produced by the permeability calculations of this study is a discrepancy between east and west basin permeabilities. The average \( k \) in the west basin of \( 0.59 \times 10^{-5} \) m/s and that in the east of \( 0.46 \times 10^{-5} \) m/s signifies a difference between the sand of the two basins. The clayey layer which was witnessed in the east basin could represent the problem; otherwise, without actually digging through two meters of sand at various points in each basin, nothing can be proven. The difference between the basin dimensions is significant and affects basin throughput. The west basin is 21% larger in surface area than the east basin, according to measurements made in October of 1995. With regard to flow, the west has on average a 25% higher throughput than the east basin. The combination of the difference in size and the difference in permeability is a distinct explanation of the variation in performance between the east and the west basins.
Expanding upon the discovery in Chapter 5 of an optimum drying time of approximately 4 days (in reality the number is 3.5, but this would not be an appropriate cycle length for City of Kamloops staff), testing of the basins with regard to cycle time was required. Before the flood/dry ratio and number of days can be shifted, it must be proven that no detrimental effects will ensue. The removal of phosphorus within the RI basins was therefore studied from historical data with respect to cycle day. No correlation between P removal and cycle day was found. This test served a second purpose of disclosing the effect of scarifying, if any, on P removal. Since the basins are scarified prior to each application period, it might have been expected that P removal would increase with respect to increasing cycle day, meaning that more P is removed once organics begin to accumulate. No such relation was found. The recommendation can therefore be made that the current drying period of the basins be altered, and scarification can be stopped at certain times of the year since it has proven to have no effect on either flow or phosphorus removal during those times.

A final and important result from the basin analyses was the overall phosphorus removal capability of the basins. Before proposing changes in influent phosphorus concentration to the basins, it must be proven that the basins are able to retain an amount of phosphorus. From five years of basin data, it was discovered in Chapter 6 that the basins were indeed retaining significant amounts of phosphorus. At first glimpse, it was difficult to accept this result since, if the phosphorus were evenly distributed throughout the basin, the SSC of the sand would have had to exceed 50 µg/g. However, testing of surface samples of basin sand showed that the organic-rich surface layer is able to retain soluble phosphorus at concentrations in excess of 1220 µg/g.

In summary, it is felt that this study has provided sufficient insight into the current RI system in Kamloops, as well as new information from the column tests, to formulate a number of recommendations involving the design of new basins as well as the current basin operations and maintenance.
Chapter 9. Conclusions

9.1 General
This study was instigated by the City of Kamloops due to its desire to investigate and ultimately increase the throughput capability of its RI system as well as its desire to discontinue the use of alum for chemical phosphorus removal. Furthermore, the design of new basins was requested, depending upon the experimental results. Experiments and results concerning these two areas have now been analyzed, presented and discussed; from this information, conclusions can be drawn. As with any study, more conclusions are drawn than were originally intended, since comments can always be made regarding the methodology used in various experiments. Furthermore, in the case of this project, aspects of current basin practices and basin design have become contentious issues and must therefore be resolved. Thus, while the focus of this chapter is to present the answers to the two predominant problems which were investigated, a number of side issues are also addressed.

9.2 Phosphorus Removal Capacity of the Basin Sand
The sand which fills the RI basins, the same as that which filled the columns used for testing, does not have a great capacity for phosphorus retention unless enriched by organics. Whether the actual SSC of this sand is the 5 μg/g calculated in this study, or the average of 20 μg/g calculated by NOWAK in the basins' pre-design work, is of no consequence. Removal of phosphorus will only last for a certain length of time, the stronger the influent, the shorter this time. For example, it was seen in Chapter 5 that the sand is unable to handle influent phosphorus concentrations of approximately 6 mg/L for more than about one hour; after this time, the effluent concentration of phosphorus will exceed the required 1 mg/L and could continue to increase to a point at which the basin effluent P exceeds the influent P. (For this to occur, P would have to be stored in the organic material at the sand surface and subsequently be released during degradation.) Transferring the column test results to full-scale basin use multiplies the time by ten and yields therefore 10 hours; this is still not long enough for a full application cycle. This ‘failure’ of the sand, however, does not imply that the continuation of alum addition at the current dosage is necessary. A number of options exists which could reduce the dosage and will be presented in Section 9.4.
9.3 Basin Flow Patterns

The Kamloops RI basins experience saturated flow. It has been shown that the sand itself controls the flow and the possibility of increased throughput by reduction of sand depth therefore exists. Previous assumptions of surface-controlled flow are no longer viable and consequently, neither are the ideas for increased throughput which resulted. Previously, it was feared that surface compression and with it flow reduction would result from increasing the height of the water table. With the proof of sand-controlled flow, however, it is expected that an increase in head will be accompanied by an increase in basin throughput. A further conclusion is, with the exception of the end of the winter flooding period, scarification of the basin's surface is possibly not required. During times of increased algae, such as July, it is thought that the basins can be scarified if deemed necessary. This can be determined either by visual inspection or by piezometer measurements; if the water levels are low, the basins are becoming unsaturated, meaning the surface is becoming clogged.

The in-situ permeability differs from the design permeability by an order of magnitude. This is thought to be due to an error in construction and, unless complete re-testing of the site and repacking of the basins is thought appropriate, it is a situation which must be accepted. The state of these basins, however, need not imply that future basins have this lower permeability. If more care is taken to assure that the site-testing results can be properly translated into construction, then it is assumed that basins with the design k will result.

The east and west basins experience different throughput due to slight differences in permeability and to a large difference in surface area. The result of a different k for each basin and the sighting of a clayey layer suggest that, arbitrarily, the basins were packed with slightly different sand. This is not surprising, due to the often changing sand layers (as described in Chapter 7) found at the cut-and-fill site. The 21% larger west basin, however, likely plays a larger role in explaining their differences. More surface area clearly means more space through which wastewater can percolate. This situation, as with the one previously described, must either be accepted or will entail substantial work to rectify. Enlarging the east basin would also mean amending the underdrain system. The use of this exercise is questionable, since there is no distinct benefit from having basins with equal throughput. The degree of treatment received by the wastewater from each basin is similar and treatment is the primary reason for the existence of the RI system.
9.4 Implications for Future Basin Use

The combination of results from the column and basin tests has allowed for a number of conclusions with respect to the current basins design and operation. Ramifications of the conclusion drawn include possible changes in basin influent, sand depth and maintenance.

9.4.1 Possible influent P concentrations and sources

It is now known that the basins cannot handle wastewater with no prior phosphorus removal; Kamloops wastewater in this state has an average phosphorus concentration of 6 mg/L. However, the basins could manage a higher influent P concentration than that which currently enters the basins, a concentration of typically less than 1 mg/L P. However, as the breakthrough curves in Chapter 5 indicated, the sand quickly becomes saturated and begins to release phosphorus into the effluent. At this point of breakthrough, when effluent concentrations begin to exceed the required 1 mg/L, significant changes have to be made to the current system since river discharge is no longer an option. These potential changes are discussed in Section 9.4.2. Discussed here are the potential sources of basin influent which are available.

There exist two different sources of wastewater for the basins, sources out of which the phosphorus concentration can be controlled and therefore made to fall into any desired range. The first source continues to be P1 and therefore the removal of phosphorus by flocculation with alum. The alum dosage, however, does not have to be as high as the current dose, which aims to remove as much P as possible. The chemical equation for the reactions which occur upon the addition of alum is as follows:

\[
\text{Al}_2\left(\text{SO}_4\right)_3.14\text{H}_2\text{O} + 3\text{Ca(HCO}_3\text{)}_2 \rightarrow 2\text{Al(OH)}_3 + 3\text{CaSO}_4 + 14\text{H}_2\text{O} + 6\text{CO}_2 \quad (9-1)
\]

The current alum dosage at the Kamloops treatment plant averages between 75 and 80 mg/L. Benefield et al. (1982) state that "...most water treatment plants utilizing alum operate between a pH of 6.0 and 7.5 and with alum dosages of 5 to 50 mg/L". The Kamloops dosage is therefore at the high end of the range. This current dosage could, however, be reduced in order to produce an effluent from P1 which has a higher P concentration than at present. A rough average concentration of cell 3 P (P1 effluent flows into cell 3 after chlorination) is 1 mg/L. The decision regarding the exact amount
of alum which should be used can use the column tests as information. Results from 120 cm columns showed that for an influent P concentration of between 3 and 4 mg/L the time to breakthrough is roughly 15 hours; this concentration would represent prior use of approximately half of the current alum dosage. It also implies that after 15 hours, the effluent P concentration may begin to exceed 1.0 mg/L. These calculations scaled up to the basin flow rate means an approximate time to breakthrough of 150 hours, or 6.25 days. If a breakthrough time of longer than 6 days is desired, the influent concentration must be less than 3 or 4 mg/L. Jar tests with alum and 2C wastewater are necessary for providing indications of alum dosage versus effluent phosphorus concentration. This source, therefore, allows for the alum dosage to be cut down by a desired amount, resulting in the possible savings of a substantial amount of money.

There exists a second source of basin influent which allows for the complete discontinuation of alum. It has been witnessed that phosphorus concentration is reduced by running the wastewater through wetlands prior to RI filtering; this influent was used in the column tests. The wetlands pilot-project is currently situated on the south shore of the river, in between 2C and cell 3; at its present size, approximately half of the wastewater’s phosphorus is removed. It must be noted that towards the end of fall and during the winter, treatment by cattails and bullrushes is severely reduced. It is presently being decided by the City of Kamloops, due to positive results during 1995, if the system should be expanded. This could occur either on land west of cell 4 or on the north shore by the RI basins. As was previously stated, the current wetlands can reduce the phosphorus concentration of 2C water to between 3 and 4 mg/L, which means a breakthrough time of 150 hours in the basins. Although it remains to be tested, it is assumed that an expanded wetlands systems would continue to remove phosphorus and therefore provide a wastewater with a lower P concentration for the RI basins. This in turn would mean a longer breakthrough time. Removal by wetlands could theoretically reach a point at which the influent P to the basins would be low enough that breakthrough would not occur, as is the current situation. This potential combination of wetlands and RI basins therefore could represent a solution to the City’s desire to terminate alum addition. Use of the chemical may, however, be required during the winter.
9.4.2 Changes required to the RI system for different influents

The aforementioned problem of effluent exceeding 1.0 mg/L after a certain number of hours cannot be helped if the basins are to be flooded with higher influent phosphorus concentrations. However, what happens after the filtration process could potentially be changed. As has been mentioned, close to the RI basins on the north side of the river is a spray irrigation system. This system takes some of the stored effluent, re-chlorinates it, and distributes it onto adjacent farmland. This process occurs between April and October, roughly. Since the purpose is irrigation, concerns with regard to the quality of wastewater include metals, bacteria and virus content. Phosphorus, conversely, as a nutrient, is a welcome constituent in land-applied water. An idea is therefore to re-route the basin effluent to spray irrigation instead of river discharge when the sand is approaching breakthrough. From the time that the effluent P concentration exceeds the discharge permit of 1.0 mg/L, it is expected to increase to a point above the influent P concentration. After this 'purging', effluent concentrations should once again drop and, once below the permit level, the effluent can be re-routed back to river discharge. This alternative would require substantial changes to the collection system, including the pumping of all basin effluent back to a holding tank for spray-irrigation water, as well as increased monitoring work for the City of Kamloops staff to ensure timely re-routing of effluent. As well, due to the inclusion of spray irrigation in the process, it could only be implemented during the summer. The continuation of alum addition to remove P would be required during the winter.

It is thought that with regard to phosphorus removal, the most appropriate solution is the combination of wetlands and RI basins, both in terms of long-term feasibility and practicality. This entails the expansion of wetlands to an area which is able to remove P to a level which will not strain the RI system. The combination provides the best treatment, the cessation of alum (and therefore current and future chemical costs) and makes use of available land.

A final result with respect to phosphorus removal by the sand which can be implemented involves changing the current flood/dry ratio of 10 days on and 10 days off. Four days drying was optimum for P removal in the columns and might be tried in the basins. It is not completely clear as to why the longer drying period increased (slightly) the P
removal. Guilloteau et al. (1993) stated that longer drying periods allow the media to recover as much treatment capacity as possible. Increased oxygen renewal is required for biological uptake of P. Since longer drying times did in fact have some effect in the Kamloops system, it is further indication that biological uptake is one of the removal mechanisms at work. Since literature often recommends a ratio of less than or equal to one for throughput maximization, a flooding period of 4 days can also be tried. It must be admitted that none of the literature has strong opinions about flood/dry periods. Some authors admit that certain ratios were implemented for the sake of staff convenience. It is therefore the opinion of this author that, if possible, different ratios and lengths of time be tried, and results recorded, to see if any changes in P removal are evident. If not, the schedule most appropriate for City staff should be used.

9.4.3 Measures for increasing throughput
The basins have never reached their design throughput due to a different permeability than was expected. The conductance of the sand cannot be changed, without refilling the basins with different sand. Therefore, throughput cannot be increased this way. Another method, however, which will increase the throughput is by reducing the depth of the sand, implying a reduction in the resistance faced by the water. The fact that the basins have saturated flow indicates that it is influenced by the depth of sand and the head. A new depth of 120 cm, as tested by column experiments, has been selected, implying a removal of around 1 m of sand. It was seen in Chapter 5 that treatment of wastewater through 120 cm columns was more than satisfactory; in fact, wastewater from 2C, which represented a ‘worst-case’ due to its lack of settling time and chlorination, was even treated to a high level.

It should be mentioned here that the exact current basin depth is unknown. The design depth was between 2.0 and 2.2 m, but this too was never measured exactly after construction. As well, the ripping of the basins which once occurred entailed the removal of the surface layer. A more realistic current depth is therefore between 1.7 and 1.9 m; a depth of 1.8 m shall therefore be used in calculations. The removal of 60 cm of sand from the basins will also result in the potential for 60 cm more head. Currently, the ponding depth is between 30 and 55 cm; the new depth would therefore be between 90 and 115 cm. There are two methods of estimating the resultant flow from these ‘new’
There are two methods of estimating the resultant flow from these ‘new’ basins. This expected flow should be calculated both in terms of maximum and minimum. The former is important for calculations of underdrain capacity to ensure that no impediments are encountered from the increase. The latter is required as a design flow which incorporates a safety factor, meaning the minimum flow which can confidently be expected. The first method of calculation is to use results from the column tests. Columns with a sand depth of 120 cm had a maximum head of only 47 cm, due to the size of the columns. Beginning the calculations with maximum expected flow, the average flow of wetlands influent through the 120 cm column at 22°C was 0.6 L/min. This equals 864 L/day; recall, however, that literature suggests a minimum division by between 5 and 10 as a loading safety factor when translating from column flow to full-scale flow. Thus, it will be said that an average flow of 86.4 L/day was realized. To scale up to basin size from column size, this number must be multiplied by the ratio of surface areas. Using the west basin’s surface area (recall that maximum flow is being sought, therefore the largest of the two basins is used) of 13 373 m² and the column surface area of 0.067 m², the following throughput is calculated:

\[ 86.4 \text{ L/day} \times \left( \frac{13 373 \text{ m}^2}{0.067 \text{ m}^2} \right) = 17 265 \text{ m}^3/\text{day} \]

This throughput is an extreme jump from the current realization of less than 7000 m³/day in the west basin, particularly considering the fact that the head is only 47 cm. The same calculations for ‘low’ flow, meaning higher water viscosity, results in an expectation of 8630 m³/day, still a high number. Before commenting on these results, another method can be employed to predict the flow.

The second method entails the use of the current average throughput of 83.3 m/yr and multiplying it by the ratio of increased head divided by reduced resistance which results from changing the basin depth. Changing the depth from 180 cm to 120 cm reduces the resistance by 33%. To keep calculations similar to the first method of using column flow, a head of 47 cm will be assumed. This means an increase in head from 30 cm to 47 cm, or an increase of 57%. This means an overall potential flow increase of 2.3 times the current flow. This implies a new yearly throughput of 195 m. Multiplying this number by the total surface area of the basins, 2.44 ha, yields a yearly flow of 4 760 000 m³/year. Dividing this number by 365 for an average daily flow yields 13000 m³/day per basin.
The flow of 17 265 m$^3$/day is over 30% greater than the flow obtained using the second method. Because literature often stresses the use of full-scale testing instead of lab-scale (column) testing for good data results, it is felt that the second method produces more prudent results. Because rough yearly averages have been used, it is likely that the results will not be exactly as expected. For example, it is thought that the summer flow per basin will exceed 13000 m$^3$/day, while winter flow will probably be much less than twice this number (to take into account the continuous operation that occurs during winter); current winter flows for both basins typically do not exceed summer flows per single basin.

The underdrain capacity must be addressed under the assumption of increased flow. Following the methodology used in Chapter 6 and again using equation 6-1, the required capacities of underdrain pipes for the expected increased flow were found. A flow of 13000 m$^3$/day translates into 150 L/s for the 450 mm pipe and 2.35 L/s for the 150 mm pipes. The results of using Equation 6-1 stipulate a required pipe diameter of 350 mm for the higher flow and 130 mm for the lower flow. Again, the existing pipes have diameters larger than are required and it is therefore concluded that the underdrain has sufficient capacity to handle the increased flow. It should be mentioned that no problems with underdrain pipe sizing were expected to be found, since the new flow is the same as Stanley's original intended flow and it is presumed that underdrain calculations were performed by the designer prior to construction.

The maximum mounding heights expected under the new flow regime must also be examined. Using equation (6-2), the k values from the lower basin layer, and a flow of 13 000 m$^3$/day, maximum mounding heights of 1.61 m and 2.31 m were calculated for the west and east basins respectively. These findings provide information into the required new ponding depth over the basins with a sand depth of 1.2 m. A ponding depth of 110 cm is required in the east basin, whereas only 41 cm is required in the west basin. The large difference in numbers between the basins is due to a greater difference in the k of the lower layers in each basin than the overall average basin k values. The cutting away of basin sand may expose or get rid of the clay layer in the east basin which has been seen. This could reduce the difference in flow between the basins, although it has already been mentioned that this current difference between basin throughputs is not an important one.
9.5 Methodology

Conclusions can be drawn not only from the experimental data which resulted from this study, but from the methods used to obtain this data, as well as the methods currently being used to run the RI basins.

9.5.1 Potential changes for future studies

As with any set of experiments performed for the first time, a number of things are learned with respect to methodology along the way and it is often realized in hindsight that certain steps would have been better in different order. In the column tests, the importance of treating all columns the same if comparison between them is desired has now been realized. This means that each must be flooded with the same influent and for the same number of hours. This not having always been the case for this study meant that the results from one set of experiments, the combining of columns to study P removal at depths greater than 120 cm, were not of use. Closer attention should also have been paid to the flood/dry ratios, although these are not of great significance. This study was able to produce an appropriate drying time, but had to rely on literature for a ratio and flooding time.

It has been realized that close attention must be paid to a system which is being simulated. It was found in this study that columns should have been covered immediately; if they had been, algae would not have interfered to the degree of leaving the columns with unsaturated flow. That the columns required covering should have been realized from the beginning by the fact that the basins do not have transparent sides and are rarely bothered to the point of unsaturated flow by algae. It was discovered in the ‘blank’ column that there was a flow impediment in the form of a clay layer. It is unsure whether or not this could have been avoided during the packing process, since particles tend to redistribute naturally upon settling and flushing with water.

It was found that the value for permeability found by means of column tests is not a good indication of the in-situ permeability and should not be used in design. As well, the permeability found using Hagen-Poiseuille gives, at least for the sand used in this study, a
falsely high value. The best results were obtained using Darcy’s relation for saturated flow.

9.5.2 Potential changes to current basin operation and management

During the course of this study, some setbacks occurred as a result of the current operation and management of the RI basins. As well, many of the numbers which have resulted from this study have a margin of error due to questionable values which had to be used. Inconsistent flow data were found when studying historical reports on basin performance. These should be checked before being reported. The numbers used as flow numbers through the basins had a large margin of error since there is no meter at the outflow of the basins. It is recommended that an outflow meter be placed at the discharge point of the RI system. The numbers currently available for the effluent volume stem from a design curve for cell 4 outflow pumps which produces false values. A system curve for the three pumps must be created and referenced if an outflow meter is not to be implemented. Further problems with the RI basins occurred when comparing piezometer data, either between basins, or between different seasons. The level of ponded water in the RI basins changes daily, due to it being manually controlled. An automated head level controller would be an asset to the system both for the sake of the City staff, forever having to fiddle with the inflow, as well as for the sake of consistent data. Throughput varies with varying head and direct comparisons are therefore difficult when this water level is always changing. The installation of a flow meter would also facilitate future studies on the RI system. One final comment with regard to current practices involves lab techniques. At times there were problems with reported values which did not make sense, such as a reportedly higher ortho-P value than total P value from the same sample. It is understood, however, that these problems have already been addressed since the author finished working in the lab.

This study has elucidated a number of characteristics of the Kamloops RI system. The potential to cease alum addition is existent, but not by means of filtering alone. It is thought that the potential to increase the flow while retaining the high treatment standards of the basins is also existent, by cutting out some of the basin sand. This can increase the throughput to a point which approaches the original design throughput, a number which was never realized due to differences between the design and in-situ permeability. It is thought that RI basins provide an excellent ‘polishing’ step to
the already highly-treated wastewater at the Kamloops Wastewater Treatment Plant.
Chapter 10. Recommendations

A number of recommendations was desired from the outset of this project and listed in Chapter 1. All the recommendations to be made from this study were discussed in Chapter 9 and will be summarized here.

1. The installation of RI outflow meters and an automated head level controller would be great assets to the system. Accurate data would consequently be available for future studies of the filters. Furthermore, without the flow monitoring of basin effluent, it will be impossible to determine the effect of any changes made to the system.

2. It is recommended that the basins be given a new sand depth of 120 cm. This depth continues to provide excellent treatment, while allowing for increased throughput. The maximum achievable discharge at this new depth is calculated to be 13000 m$^3$ per day for the Kamloops RI system.

3. It is recommended that the media remain Thompson River scar channel sand. This is the sand which is available and therefore inexpensive. More importantly, this is the sand on which all the testing has been done and therefore all conclusions are drawn explicitly assuming use of this sand.

4. It is recommended that the flood/dry ratio remain at 1.0, but the lengths shorten to 4 days from 10 days. This is subject to change if no beneficial changes are found.

5. The potential for a higher depth of ponded water above the sand results from removing sand. Calculations from Chapter 9 revealed that a water depth of 1.1 m is required by the mounds for proper drainage. It is therefore recommended that this number first be implemented. If the expected flow is realized, higher head can be experimented with, but the capacity of the underdrain system must be viewed with concern for throughputs higher than that suggested by this study.

6. The potential for alum reduction or termination can be realized by combining wetlands treatment with RI filtering. The degree of reduction, however, depends on decisions to be made in the near future by the City of Kamloops, which involve wetlands and/or RI expansion.
7. Scarification of the basins is only necessary after the winter flooding period has ended and during certain periods to be determined by City staff. If meter readings have indicated that basin effluent flows are lower than usual, it is recommended that two steps be followed. Visual inspection while a basin is drying can indicate significant or higher than usual accumulation of algae. The second and more important step is to measure the water levels in piezometers. If these are low, then scarification is suggested.

A number of recommendations have been made. It would not be appropriate for all the suggested changes to be made solely on word of this report; it is hoped, however, that the findings of this study aid in the many decisions to be made.
REFERENCES


BIBLIOGRAPHY


APPENDIX A. SAND CHARACTERISTICS - DENSITY AND GRAIN SIZING

Grain Sizing
The following tables represent the raw data received from Eco-tech from their grain-sizing analysis.

<table>
<thead>
<tr>
<th>mesh number</th>
<th>screen in mm</th>
<th>+ fraction in g</th>
<th>passing (%)</th>
<th>cumulative (%)</th>
</tr>
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<td>3.85</td>
<td>99.72</td>
<td>.28</td>
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<td>16</td>
<td>1</td>
<td>6.11</td>
<td>99.27</td>
<td>.72</td>
</tr>
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<td>.5</td>
<td>115.82</td>
<td>90.85</td>
<td>9.15</td>
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<td>655.87</td>
<td>43.06</td>
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<td>314.87</td>
<td>20.12</td>
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<td>.063</td>
<td>198.19</td>
<td>5.68</td>
<td>94.32</td>
</tr>
<tr>
<td>-250</td>
<td>-</td>
<td>77.99</td>
<td>-</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Table A2. Sample Two (representative of sand used in October)

<table>
<thead>
<tr>
<th>mesh number</th>
<th>screen in mm</th>
<th>+ fraction in g</th>
<th>passing (%)</th>
<th>cumulative (%)</th>
</tr>
</thead>
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<td>1.07</td>
</tr>
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<td>107.48</td>
<td>90.58</td>
<td>9.41</td>
</tr>
<tr>
<td>60</td>
<td>.25</td>
<td>618.85</td>
<td>42.51</td>
<td>57.49</td>
</tr>
<tr>
<td>100</td>
<td>.15</td>
<td>304.18</td>
<td>18.89</td>
<td>81.11</td>
</tr>
<tr>
<td>250</td>
<td>.063</td>
<td>187.04</td>
<td>4.36</td>
<td>95.64</td>
</tr>
<tr>
<td>-250</td>
<td>-</td>
<td>56.09</td>
<td>-</td>
<td>100.00</td>
</tr>
</tbody>
</table>

Samples taken from the west basin at depths down to 1 meter were also sent to Ecotech for a grain-size analysis with the following results:

1A (0.1m) - 3.6% < 63 micrometers
1B (0.5m) - 4.2% < 63
1C (1.0m) - 3.66% < 63
2A (0.1m) - 2.29% < 63
2B (0.5m) - 5.13% < 63
3A (0.1m) - 1.65% < 63
3B (0.5m) - 3.05% < 63 micrometers
3C (1.0m) - 2.88% < 63 micrometers

Density Measurement
The following illustrates the methods used to calculate the density from a number of core samples extracted from each basin.

core 1: 788.0g (total wt) - 9.8g (bag) - 232.1g (cylinder) = 546.1g
core 2: ..... = 483.1g
core 3: \( \ldots \) = 536.6g

Then, density = mass / volume, where volume of each cylinder core was 320 cubic cm (h: 7.65 cm, r: 3.65 cm). Therefore, the following densities were calculated:

- core 1: 1.70 g/cm\(^3\)
- core 2: 1.51 g/cm\(^3\)
- core 3: 1.68 g/cm\(^3\)

The bulk densities were measured to be: 1630 kg/m\(^3\) (average of three cores taken from east basin in June); cores were taken between 15 and 30 cm depth.

A more representative sampling was made on July 14th, during which 7 cores were collected from the west basin and 8 from the east. The distribution of samples was as follows: (N↑)

- west
  - G
  - F
  - E
  - D
  - C
  - B
  - A

- east
  - M
  - N
  - O
  - K
  - L
  - H
  - I
  - J

The weights of the sample bags were recorded empty and full, with the following density results:

<table>
<thead>
<tr>
<th>sample</th>
<th>total weight</th>
<th>total - bag weight</th>
<th>density in g/cm(^3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A west</td>
<td>454.7</td>
<td>449.5</td>
<td>1.40</td>
</tr>
<tr>
<td>B west</td>
<td>474.4</td>
<td>467.9</td>
<td>1.46</td>
</tr>
<tr>
<td>C west</td>
<td>493.9</td>
<td>487.2</td>
<td>1.52</td>
</tr>
<tr>
<td>D west</td>
<td>485.4</td>
<td>478.7</td>
<td>1.50</td>
</tr>
<tr>
<td>E west</td>
<td>466.2</td>
<td>459.6</td>
<td>1.44</td>
</tr>
<tr>
<td>F west</td>
<td>504.1</td>
<td>491.2</td>
<td>1.53</td>
</tr>
<tr>
<td>G west</td>
<td>461.4</td>
<td>454.8</td>
<td>1.42</td>
</tr>
<tr>
<td>H east</td>
<td>499.1</td>
<td>496.2</td>
<td>1.55</td>
</tr>
<tr>
<td>I east</td>
<td>507.1</td>
<td>502.1</td>
<td>1.57</td>
</tr>
<tr>
<td>J east</td>
<td>503.5</td>
<td>498.3</td>
<td>1.56</td>
</tr>
<tr>
<td>K east</td>
<td>469.4</td>
<td>463.8</td>
<td>1.45</td>
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<tr>
<td>L east</td>
<td>463.3</td>
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<td>1.43</td>
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<tr>
<td>M east</td>
<td>480.3</td>
<td>474.7</td>
<td>1.48</td>
</tr>
<tr>
<td>N east</td>
<td>505.3</td>
<td>498.7</td>
<td>1.56</td>
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<tr>
<td>O east</td>
<td>481.2</td>
<td>474.6</td>
<td>1.48</td>
</tr>
</tbody>
</table>

* - volume of cylindrical core was 320.2 cm\(^3\)

It should be noted that average east density is greater than the average west density (1.51 g/cm\(^3\) compared to 1.47 g/cm\(^3\)).
More cores were taken and their density measured on October 24th from the west basin; these were taken at depths of 10cm, 50cm and 1m. The following were the results:

- average density at 10cm: 1353 kg/m³
- average density at 50cm: 1388 kg/m³
- average density at 1m: 1337 kg/m³

overall average : 1363 kg/m³
APPENDIX B. LAB ANALYSES

--- | --- | --- | --- | --- | --- | --- | --- | --- | ---
14-Jun | 2C | M & M | 18.9 | 10 | 249 | 26 | 5.3 | tmtc | 1000
15-Jun | 2C | G.B. | 19.8 | 21 | 188 | 58 | 16.0 | 65 | 2000 | <1
15-Jun | P2 | G.B. | 22.0 | 22 | 210 | 18 | 8.0 | --- | --- | ---
19-Jun | tank-5 days | G.B. | 21.8 | 28 | 221 | 25 | 3.2 | 63 | 2000 | 600
22-Jun | tank-2 days | G.B. | 21.8 | 38 | 210 | 20 | 4.8 | 55 | 1800 | 660
24-Jun | tank-3 days | G.B. | 21.8 | <1 | 176 | 12 | 4.4 | 5 | 100 | 9
26-Jun | tank-4 days | G.B. | 26.5 | 44 | 241 | 47 | 4.1 | 55 | 1700 | 600
29-Jun | tank 2 | G.B. | 26.5 | 1 | 128 | 20 | 2.8 | 8 | 120 | 11
30-Jun | 2C | M & M | 19.6 | 2 | 234 | 12 | 2.3 | 7 | 40 | <1
4-Jul | tank | G.B. | 22.0 | 21 | 207 | 50 | 4.9 | 60 | 30000 | 1000
4-Jul | column 2 | G.B. | 21.9 | 1 | 152 | 19 | 4.0 | 7 | 300 | 60
4-Jul | column 3 | G.B. | 21.7 | 1 | 158 | 15 | 3.3 | 3 | 500 | 20
6-Jul | tank | G.B. | 20.7 | 83 | 240 | 46 | 2.6 | 56 | 4500 | 600
6-Jul | column 3 | G.B. | 19.8 | 4 | 257 | 11 | 2.5 | 6 | 200 | 15
6-Jul | tank | G.B. | 22.8 | 83 | 255 | 49 | 2.7 | 52 | 500 | 100
6-Jul | column 3 | G.B. | 22.9 | 2 | 234 | 12 | 2.3 | 7 | 40 | <1
7-Jul | column 3 | G.B. | 19.6 | <1 | 283 | 6 | 4.5 | 5 | 30 | 2
11-Jul | tank | G.B. | 22.2 | 49 | 297 | 47 | 2.7 | 56 | 300 | 10
11-Jul | column 1 | G.B. | 22.2 | 3 | 266 | 17 | 2.7 | 9 | 50 | 2
18-Jul | wetlands eff. | G.B. | 24.4 | 2 | 314 | 8 | 2.3 | 10 | 600 | 100
18-Jul | col2wetlands | G.B. | 23.8 | 1 | 281 | 7 | 2.4 | 2 | 20 | 2
18-Jul | col2wetlands | G.B. | 26.4 | <1 | 318 | 5 | 1.2 | 2 | 20 | <1
19-Jul | wetlands eff. | G.B. | 27.2 | 9 | 312 | 7 | 2.0 | 9 | 450 | 150
19-Jul | col3wetlands | G.B. | 28.1 | 2 | 284 | 3 | 2.2 | 2 | 15 | 21
19-Jul | col3wetlands | G.B. | 1 | 3 | --- | --- | --- | --- | --- | ---
20-Jul | col3wetlands | G.B. | 1 | 20 | 12 | --- | --- | --- | --- | ---
27-Jul | cell 3 influent | G.B. | 23.0 | 14 | 316 | 9 | 2.4 | 15 | <1 | <1
27-Jul | col3cell3 | G.B. | 20.3 | <1 | 255 | 4 | 2.6 | 4 | 1 | <1
27-Jul | col3cell3 | G.B. | 21.8 | <1 | 269 | 4 | 2.7 | 4 | 5 | 2
28-Jul | cell 3 influent | G.B. | 20.4 | 25 | 288 | 11 | 3.2 | 17 | 10 | 5
28-Jul | col2cell3 | G.B. | 20.6 | <1 | 260 | 4 | 2.8 | 4 | 2 | 1
31-Jul | cell 3 influent | G.B. | 20.7 | 35 | 244 | 15 | 6.0 | 15 | 20 | 6
31-Jul | col3cell3 | G.B. | 20.9 | <1 | 522 | 3 | 3.7 | 6 | 4 | 1
31-Jul | col(1+3)cell3 | G.B. | 20.7 | <1 | 236 | 14 | 3.6 | 3 | 4 | 1
1-Aug | col3cell3 | G.B. | 21.4 | <1 | 501 | 5 | 3.6 | --- | --- | ---
1-Aug | col(2+3)cell3 | G.B. | 21.5 | <1 | 221 | 3 | 3.3 | 2 | 5 | 1
1-Aug | col(1+2+3)c3 | G.B. | 21.6 | <1 | 203 | 3 | 3.1 | 2 | 4 | <1
2-Aug | cell 3 influent | G.B. | 20.7 | 42 | 285 | 18 | 5.9 | 12 | 15 | 2
2-Aug | column 1 | G.B. | 20.7 | 5 | 222 | 10 | 4.3 | 7 | 5 | <1
<table>
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<th>P</th>
<th>ortho P</th>
<th>diss. P</th>
<th>ammonia-N</th>
<th>nitrate</th>
<th>nitrite</th>
<th>pH</th>
<th>alkalinity</th>
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APPENDIX C. MOUNDING ANALYSIS

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<td>11</td>
<td>11</td>
<td>21</td>
<td>21</td>
<td>11</td>
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<td>58</td>
<td>179</td>
</tr>
<tr>
<td>12</td>
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<td>131</td>
<td>12</td>
<td>12</td>
<td>26</td>
<td>113</td>
</tr>
</tbody>
</table>
APPENDIX D. LAB RESULTS FROM PIEZOMETER SAMPLING

Listed below are the results from the UBC lab of total and ortho P concentrations. Note with respect to sample labelling that E represents east basin and W represents west basin. The number following the E or W indicated the piezometers nest number and the letter following this represents the piezometer tube within that nest. The order of depth from shallowest to deepest in the west basin is A,C,D,B and in the east basin is D,B,A,C.

Total P results from March 14th samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Total P concentration in mg/L</th>
</tr>
</thead>
<tbody>
<tr>
<td>East basin influent</td>
<td>1.00</td>
</tr>
<tr>
<td>E4A</td>
<td>7.95</td>
</tr>
<tr>
<td>E4B</td>
<td>14.95</td>
</tr>
<tr>
<td>E4C</td>
<td>12.93</td>
</tr>
<tr>
<td>E5B</td>
<td>13.24</td>
</tr>
<tr>
<td>E5C</td>
<td>12.70</td>
</tr>
<tr>
<td>E6A</td>
<td>0.93</td>
</tr>
<tr>
<td>E6B</td>
<td>2.46</td>
</tr>
<tr>
<td>E6C</td>
<td>1.33</td>
</tr>
<tr>
<td>E7B</td>
<td>4.10</td>
</tr>
<tr>
<td>E7C</td>
<td>3.36</td>
</tr>
<tr>
<td>E7D</td>
<td>21.44</td>
</tr>
<tr>
<td>E8A</td>
<td>6.72</td>
</tr>
<tr>
<td>E10A</td>
<td>5.98</td>
</tr>
<tr>
<td>E10B</td>
<td>5.35</td>
</tr>
<tr>
<td>E10C</td>
<td>9.61</td>
</tr>
<tr>
<td>E10D</td>
<td>14.03</td>
</tr>
<tr>
<td>E11D</td>
<td>1.09</td>
</tr>
<tr>
<td>West basin influent</td>
<td>2.60</td>
</tr>
<tr>
<td>W1A</td>
<td>14.63</td>
</tr>
<tr>
<td>W2B</td>
<td>6.39</td>
</tr>
<tr>
<td>W5B</td>
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<td>W6A</td>
<td>29.29</td>
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<tr>
<td>W6D</td>
<td>19.71</td>
</tr>
<tr>
<td>W7B</td>
<td>5.24</td>
</tr>
<tr>
<td>W9A</td>
<td>9.65</td>
</tr>
<tr>
<td>W12A</td>
<td>15.17</td>
</tr>
</tbody>
</table>

Ortho P results from March 14th samples.

<table>
<thead>
<tr>
<th>Sample</th>
<th>Ortho P concentration in mg/L</th>
</tr>
</thead>
<tbody>
<tr>
<td>East basin influent</td>
<td>0.19</td>
</tr>
<tr>
<td>E4A</td>
<td>0.30</td>
</tr>
<tr>
<td>Location</td>
<td>Value</td>
</tr>
<tr>
<td>----------</td>
<td>-------</td>
</tr>
<tr>
<td>E4B</td>
<td>0.16</td>
</tr>
<tr>
<td>E4C</td>
<td>0.28</td>
</tr>
<tr>
<td>E5B</td>
<td>0.25</td>
</tr>
<tr>
<td>E5C</td>
<td>0.28</td>
</tr>
<tr>
<td>E6A</td>
<td>0.19</td>
</tr>
<tr>
<td>E6B</td>
<td>0.19</td>
</tr>
<tr>
<td>E6C</td>
<td>0.28</td>
</tr>
<tr>
<td>E7B</td>
<td>0.31</td>
</tr>
<tr>
<td>E7C</td>
<td>0.34</td>
</tr>
<tr>
<td>E7D</td>
<td>0.40</td>
</tr>
<tr>
<td>E8A</td>
<td>0.84</td>
</tr>
<tr>
<td>E10A</td>
<td>0.19</td>
</tr>
<tr>
<td>E10B</td>
<td>0.16</td>
</tr>
<tr>
<td>E10C</td>
<td>0.31</td>
</tr>
<tr>
<td>E10D</td>
<td>0.36</td>
</tr>
<tr>
<td>E11D</td>
<td>0.19</td>
</tr>
<tr>
<td>West Basin Influent</td>
<td>0.27</td>
</tr>
<tr>
<td>W1A</td>
<td>0.40</td>
</tr>
<tr>
<td>W2B</td>
<td>0.34</td>
</tr>
<tr>
<td>W5B</td>
<td>0.34</td>
</tr>
<tr>
<td>W6A</td>
<td>0.95</td>
</tr>
<tr>
<td>W6D</td>
<td>0.11</td>
</tr>
<tr>
<td>W7B</td>
<td>0.54</td>
</tr>
<tr>
<td>W9A</td>
<td>0.74</td>
</tr>
<tr>
<td>W12A</td>
<td>0.57</td>
</tr>
</tbody>
</table>
APPENDIX E. BASIN FLOW DATA

First of all, list the major problems with doing comparisons with any sort of certainty:

1. Weather differs from year to year, and therefore so does temperature.
2. Basin head varies daily and is never really consistent.
3. Flooding periods vary; there seem to be frequent shutdowns.

Each month and year vary so much from one another, that it is impossible to blame any of these variations on any one thing, due to the three reasons listed above.

Daily flows have also been obtained for this time frame. I thought it would be easier to compare numbers directly between June and October of 1994 and 1995. Here again, however, the comparisons don't mean much. One more problem is the fact that for the same weeks, different basins were being flooded between the years (ie. June 10/94 west, June10/95 east). Average differences indicate that more 1995 flow numbers were lower than 1994. This could mean that either the calculation method underestimates the flow, or the meter is wrong.

Mike Warren provided both the design and system curves for the pumps which pump from cell 4 across the river to the holding tank at Cinnamon Ridge. One thing which needs to be clarified is which side of the river has the higher head (ie. do the south side pumps have to pump to higher head as well, or is it a declining slope as the river is crossed?). A design sheet offers the following insight:

"...head of 0.15m is required for the present design flow of 16,000 m$^3$/day between Cell 4 and the basin. Hence, the minimum operational level for Cell 4 is 341.15 metres.......The flow rates delivered from the pumps will vary significantly with the static head difference between the balancing tank and the pumphouse basin."

The following numbers were extrapolated from the provided pump curves:

<table>
<thead>
<tr>
<th></th>
<th>system curve</th>
<th>design curve</th>
<th>used by Leo</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 pump</td>
<td>220</td>
<td>237</td>
<td>208</td>
</tr>
<tr>
<td>2 pumps</td>
<td>328</td>
<td>360</td>
<td>333</td>
</tr>
<tr>
<td>3 pumps</td>
<td>380</td>
<td>423</td>
<td>468</td>
</tr>
</tbody>
</table>

* all values are in L/s
Leo Albrecht uses the numbers in the "used by Leo" column of Table B1, claiming that a pump efficiency of 80% has already been factored in. Therefore, it looks like all numbers recorded while all three pumps were running are about 20% over. Those recorded with only one or two pumps running seem appropriate. All three pumps have hour-meters, however Mike and Al maintain that only two of the pumps are measured time-wise. The June, 1995 to October, 1995 numbers from R.I. and spray irrigation can be recalculated by using the pump hours. The flow through R.I. can then be calculated with more accuracy than is currently done. One must assume, generally, an error window in the flow numbers of about 25%. This means that any calculated k values for the basins can only at best be within 25% of the true values.
APPENDIX F. BASIN FLOW DATA

The following table displays flow numbers for the RI basins between the years 1986 and 1995. Recall that basin start-up occurred in late 1985, thus this Table F-1 represents complete basin data. The hydraulic loading is the flow number divided by the total surface area of both basins; this value is 24 431 m$^2$ (13 373 m$^2$ for the west and 11 058 m$^2$ for the east).

<table>
<thead>
<tr>
<th>year</th>
<th>total flow through RI in m$^3$</th>
<th>hydraulic loading in m/yr</th>
</tr>
</thead>
<tbody>
<tr>
<td>1986</td>
<td>1 977 202</td>
<td>80.9</td>
</tr>
<tr>
<td>1987</td>
<td>1 922 487</td>
<td>78.7</td>
</tr>
<tr>
<td>1988</td>
<td>1 642 434</td>
<td>67.2</td>
</tr>
<tr>
<td>1989</td>
<td>2 189 913</td>
<td>89.6</td>
</tr>
<tr>
<td>1990</td>
<td>2 460 576</td>
<td>100.7</td>
</tr>
<tr>
<td>1991</td>
<td>2 059 758</td>
<td>84.3</td>
</tr>
<tr>
<td>1992</td>
<td>2 659 957</td>
<td>108.9</td>
</tr>
<tr>
<td>1993</td>
<td>1 940 074</td>
<td>79.4</td>
</tr>
<tr>
<td>1994</td>
<td>2 082 975</td>
<td>85.2</td>
</tr>
<tr>
<td>1995</td>
<td>1 415 852</td>
<td>58.0</td>
</tr>
<tr>
<td>average</td>
<td>2 034 858</td>
<td>83.3</td>
</tr>
</tbody>
</table>

While the numbers in Table F-1 represent the total flow through the basins per year, this does not indicate flow for 365 days of the year. During some portion of many of the years, including one witnessed by the author during the summer of 1995, the basins were not running due to a problem. The problems varied, as did the times of shut-down. Therefore, while the number in the above table do represent the throughput, they are not necessarily representative of basin capability.