DIVERSION OF WATER FROM SURFACE-WATER SOURCES THROUGH INFILTRATION GALLERIES

May, 2006
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DIVERSION OF WATER FROM SURFACE-WATER SOURCES THROUGH INFILTRATION GALLERIES

1.0 INTRODUCTION

Diversion of surface water from rivers, creeks, and streams is required to serve the needs of various intended uses, such as irrigation and domestic and municipal water supply. Open-water intake structures are often used for this purpose, but development of such structures can sometimes be problematic due to the fact that they require a sufficient depth of water available to adequately submerge the intake, they require screening to exclude aquatic organisms, and they also can create a physical obstruction that impedes navigation or recreational activities. Also, if they are seasonally installed, they require bi-annual disturbance of the stream. Infiltration galleries are a water-diversion option that can provide clean, sediment-free water without the requirement for mechanical screens or repeated entry into and disturbance of the stream corridor. While infiltration galleries can be applied to a wide range of situations, practical considerations relating to stream morphology and the geological setting often provide limitations to their use. This monograph is intended to provide guidance for the use of infiltration galleries as diversion works, and is directed primarily toward a technical audience.

2.0 TYPES OF INFILTRATION GALLERIES

Infiltration galleries that divert water from surface-water sources are usually considered as belonging to two separate types: those installed beneath the water body, and those installed adjacent to the water body. The decision of whether to place the collector adjacent to or under the surface-water body depends on several factors:

- **Yield requirements** – in general, the yield available from a gallery of a given size would be higher from a bed-mounted gallery than from a gallery installed adjacent to the water body.
- **Water Quality Considerations** – galleries installed adjacent to a water body usually deliver water that is more highly filtered, and therefore less turbid and with less bacteria, than water derived from bed-mounted galleries.
- **Construction** – it is generally more difficult to install a gallery beneath the bed of a water body than adjacent to it.
- **Maintenance** – in general, bed-mounted galleries require more because fine material carried by the stream can migrate into the filter pack. Physical maintenance of galleries installed adjacent to water bodies is usually easier due to ease of access, while backflush cleaning is more difficult.

2.1 Bed-Mounted Infiltration Galleries

Typical configurations of bed-mounted infiltration galleries are shown in plan in the following figure:

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The discussion herein will be limited to cut-and-cover types of infiltration galleries. While infiltration galleries can be created as one or more horizontal wells installed using horizontal directional drilling, such installations are beyond the scope of this document, and can be very difficult to install due to the possibility of “fraccing out” and the associated release of drilling fluid into the water body.
Most bed-mounted galleries have the axis of the collector pipe oriented perpendicular to the direction of streamflow. Bed-mounted infiltration galleries should be buried between about 1 to 1.5 m beneath the bed of the stream, and manifolded collectors should be spaced at a distance of about twice the burial depth.

### 2.2 Land-based Infiltration Galleries

A typical configuration of a land-based infiltration gallery is shown in the following figure:

![Diagram of a land-based infiltration gallery](image)

Often, one of the principle reasons for selecting an infiltration gallery as an intake option is that, at least periodically, a limited depth of water would preclude the development of an open-water type of intake. A limited depth in the water body places similar restrictions on the use of land-based infiltration galleries, because it limits the available drawdown (D-d).

Clogging of the surface-water/groundwater interface (that results from physical, chemical, and biological processes) has the potential to be a problem with any riverbank filtration system. Partial clogging during operation of a land-based infiltration gallery is likely to be unavoidable (Ref. #21). Physical clogging of the surface-water/groundwater interface results from the deposition of fine-grained suspended sediment at the interface and in the near-surface pores. The deposition and growth of microorganisms can also contribute to physical clogging. This clogging may be exacerbated during periods of low surface-water discharge, and is most apparent near the river’s edge where flow velocities are generally lower than at the center of the river. Chemical clogging can result from precipitation of dissolved surface-water constituents and may occur near the interface or anywhere along the flowpath. This is due to the change in geochemical conditions as infiltrating water enters the riverbed and aquifer. Finally, biological or microbial clogging can result from the accumulation of bacterial cells in pore spaces, the production of extra-cellular polymers, the release of gaseous byproducts from denitrifying bacteria and methanogens, and the microbially-mediated accumulation of insoluble precipitates. Biological clogging is most likely to occur near the surface-water/groundwater interface, where nutrients are most available.

While it might be possible to periodically remove material that accumulates and clogs the pores in the filter media through backflushing, such an operation in a land-based gallery would require horizontal flow within the filter media at the surface-water/groundwater interface. Achieving such a flow would be fairly difficult without significant mechanical works in addition to the collector screens (e.g. – an array of surface-wash jets located near the interface).

Because of the physical limitations associated with land-based infiltration galleries, they should probably not be considered for use except in situations where the withdrawal rate is relatively small. For this reason, they will not be discussed further herein. However, because of the potential water-quality benefits associated with riverbank filtration systems, they should definitely be considered as a diversion option in situations where the end-use is water for human consumption.
3.0 SITING CONSIDERATIONS

3.1 River Morphology
Infiltration galleries can probably be made to work at almost any location on any watercourse, but practically, their application is normally limited to locations on streams where velocities can be expected to generally be greater than about 0.3 m/s, which are typically sand and gravel-bed streams with relatively high gradients. It is also advisable to locate infiltration galleries in riffle-type reaches on meandering channels, since such locations are more likely to provide conditions conducive to the use of infiltration galleries (higher stream velocities; coarser bed material).

It is also preferable to locate an infiltration gallery in a relatively stable reach of a stream that is not going to be subject to aggradation or degradation of the bed, and is also not likely to migrate laterally. Guidance for assessing stream stability can be found in Reference #20. In situations where lateral stability is expected to be a concern, and locating the site in a more stable reach is not possible, it may be possible to locally stabilize the reach in question through implementation of bank-stabilization measures. Where feasible, bank stabilization should employ bioengineering or biotechnical approaches (see Ref. #22).

3.2 River Ice Regime
For infiltration galleries that are intended to operate year-round, care should be taken to ensure that the proposed site is not located downstream of areas that may be expected to generate significant quantities of frazil ice through the winter (eg. downstream of hydroelectric dams or rapids that remain open all winter). In such locations, there is the possibility that anchor ice may form on the river bed above the infiltration gallery, isolating the gallery from the source of water.

3.3 Regional Geology
Much of the same information used to assess the morphology of the river reach can also be used in an assessment of the regional geology (1:50,000 maps, air photo’s, etc.). In addition to this information, previously-published regional geologic reports and maps should be consulted, as well as well logs from provincial databases. This information should be compiled in a base map of the area that notes features such as springs, wells, gravel pits, surface water features, geologic outcrops and other pertinent landscape features. Using well-log information, a geological cross-section through or very near to the area of interest should also be constructed.
The regional geologic assessment may identify potential water sources that could eliminate the need for an infiltration gallery (e.g., a well or wells developed in alluvial deposits adjacent to the river, etc.).

3.4 Site Geology
The primary geologic considerations affecting infiltration galleries are the nature of the alluvial sediments that the gallery will be installed in, and the depth to bedrock.

Analysis of infiltration galleries is based on Darcy’s Law (1856), which applies the principles of fluid flow to the flow of water in a permeable media. The velocity of flow equals the product of a coefficient (hydraulic conductivity - K) times the hydraulic gradient, and the flow equals the velocity times the effective area. In general, coarser-grained soils have a higher hydraulic conductivity, meaning that a given flow can be extracted through a smaller area than for finer-grained soils. If the stream bed is composed of finer-grained soils such as clays, silts and fine sands, the required size of the gallery will likely preclude its use. Cemented sands and gravels that contain a high clay fraction can also restrict the applicability of infiltration galleries.

Shallow bedrock can limit the depth to which the collector pipes can be placed, thereby limiting the yield and increasing the chances of the installation washing out. Because of these limitations, it is important to determine the nature of the geologic conditions at the proposed diversion site through subsurface investigations (test drilling; test pits), material sampling and testing (grain-size analyses), and determination of the in-situ permeability of the material comprising the stream bed.

4.0 FIELD INVESTIGATIONS

4.1 Stream Morphology
A significant amount of the information required for assessing the condition of the stream can be gathered in the office (air photo’s; topographic maps; hydrometric data; previous studies, etc.). However, many streams have limited data associated with them, and few have the site-specific information required for assessment. A guide to field investigations relating to stream assessment can be found in Reference #11.

4.2 Site Geology
The hydraulic conductivity of the stream-bed material can be estimated from the grain-size distribution by using a number of empirical relations. These relations have been developed by various researchers, for various soil types, but they all take the same basic form (Ref. #7):

\[ K = \frac{g}{\nu} C \varphi(n) d_e^2 \]

where:

- \( K \) = hydraulic conductivity (cm/sec)
- \( \nu \) = kinematic viscosity of water (mm²/sec)
- \( g \) = gravitational acceleration (m/sec²)
- \( \varphi(n) \) = function of porosity \( n \)
- \( d_e \) = effective grain size of granular material (mm)

The following table provides the salient information for a few of the more common relations:

<table>
<thead>
<tr>
<th>Variable</th>
<th>Method</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Hazen</td>
</tr>
<tr>
<td>C</td>
<td>0.06</td>
</tr>
<tr>
<td>( \varphi(n) )</td>
<td>1+10(n-0.26)</td>
</tr>
<tr>
<td>( d_e )</td>
<td>( d_{10} )</td>
</tr>
<tr>
<td>Applicability</td>
<td>0.1&lt;( d_e &lt;3; n&lt;5 )</td>
</tr>
</tbody>
</table>
The porosity \( n \) in the foregoing relations can be approximated as a function of the uniformity coefficient \( \eta \):

\[
\frac{n}{10} = 0.255(1 + 0.83^{\eta}) \quad \text{Uniformity coefficient } \quad \eta = \frac{d_{60}}{d_{10}}
\]

where \( d_{i} \) = particle size (mm) that \( i \% \) of the material (by dry weight) is finer than.

Hydraulic conductivity can also be estimated by comparing the grain-size plot of the bed material with similar plots of grain-size distributions of material having known permeability. The following chart illustrates such information:

While estimation of hydraulic conductivity from grain-size analyses is fairly straightforward, obtaining a representative sample of the stream-bed material can be more of a challenge due to the difficulty of retaining fines when sampling in a flowing stream. The most common and practical sampling method for this purpose is the freeze-core sampling method.
Reference #2 contains information on sampling methods and equipment that can be used to obtain representative samples of stream-bed material under such conditions.

While estimating hydraulic conductivity from grain-size information may be adequate for conceptual design and cost estimation, more reliable methods of estimating the in-situ permeability of the alluvium are required for final design. To estimate in-situ hydraulic conductivity for final design, it is recommended that a pump test be conducted on a test pit excavated at or near the site of the proposed infiltration gallery, or to use a constant-head injection test (CHIT) to estimate hydraulic conductivity of the alluvium at various locations and depths where the proposed infiltration gallery is to be constructed (see Reference #4 for a description of the CHIT procedure and analysis).

4.2.1 Testing Requirements – Determination of In-situ Permeability using a Pump Test

Test Pit Excavation

Equipment Needed:
1. 25' survey rod
2. 50' measuring tape
3. Survey Level
4. Tripod for Level
5. Waders
6. Pump – (with capacity to pump > design flow*)
7. Flow meter or other flow measuring device
8. 3 people minimum
9. Backhoe or Excavator
10. Stopwatch

*Data required before pump selection: Irrigation system design flow rate, Q (gpm or cfs)

Step 1: Excavation Location
Excavate a test pit with sufficient width and length to accommodate a test hole depth that is similar to the pipe invert of the design infiltration gallery. These dimensions should be sufficient to allow stable sides during the test. This test hole should be located at or near the planned gallery location.

Step 2: Log and Sample
The materials exposed by the pit should be logged, sampled, and described using recognized soil descriptions. In sampling, collect all material less than 75 mm in diameter. A grain size distribution curve should be developed so that a gravel pack envelope can be specified correctly to match the native stream gravel. Note the gradation of the alluvial material and document any differences with relation to depth.

Step 3: Test hole measurements
The object is to measure the hydraulic conductivity of the alluvium in the test section. First let the water rise to a static level in the test hole. Carefully measure the test hole top width and bottom width to pump out depths to determine the volume of the hole. Horizontal and vertical control is required. Note the following: Static water elevation in the test hole, low water elevation in the nearby creek, and creek bottom elevation. All need to be referenced together. This should be completed before the test starts.

Step 4: Timing recharge test sections
Start the pump and completely empty the test hole. Note the low water level elevation and time when the pump is shut off. The flow rate and time to empty the hole should be documented. Time the recovery for varying elevations. The increments should be 50 to 100 mm intervals. These tests should be completed at the estimated elevation of the installed pipe. Pay close attention to the first increment recovery time. This has resulted in the past as the most useful in final design of the area required for the perforated pipe.
Note: The test for each increment should be measured accurately to determine the hydraulic conductivity of the gravel alluvium more in the relationship of the pipe area and not the total area of the gravel envelope.

Step 5: Calculations (see example)
Based upon results of the test, the estimated volume, divided by recovery time, will provide the designer the estimated conductivity flow rate. This value (m³/s) can be divided by the estimated area of the test section, and provide the designer the flow rate per unit area. With this value, an estimated perforated pipe length can be determined.
Example:
The calculations shown below are for only one test section, at the lowest section of a typical test hole. It is recommended that several test sections be completed at different elevations within the area of the planned pipe below creek bed.

Given:
Dimensions; (Rectangular test hole)
Top edges of test hole ............................ 6.1 m x 7.6 m
Static water surface ................................. 5.2 m x 5.4 m.
Bottom of hole .............................. 3.20 m x 3.20 m. (0.1 m H2O depth)
Recharge test 1, water surface................. 3.40 m x 3.40 m.

Elevations & Time
Top edge of test hole.............................. 29.35 m
Static water surface (start pump to drain hole) ... 28.68 m at 13:45:00
Stop pump (test hole drained) ...................... 26.82 m at 14:30:00
Bottom of test hole (start recharge time)........... 26.82 m at 14:30:15
End of test section (end recharge time ) .......... 26.91 m at 14:32:30

Find:
1.) Recharge depth (m)
2.) Recharge time (sec)
3.) Surf. area of the bottom (m²)
4.) Surf. Area of test section 1, recharge
5.) Average end area of test section (m²)
6.) Volume of test section (m³)
7.) Recharge flow in of test section (m³/sec)
8.) hydraulic conductivity (m/s)

Solution:
1.) 26.91 m – 26.82 m = 0.09 m
2.) 14:32:30 – 14:30:15 = 2 minutes and 15 seconds or 2.25 minutes, or 135 sec.
3.) 3.20 m x 3.20 = 10.24 m²
4.) 3.40 m x 3.40 m = 11.56 m²
5.) 10.24 m² + 11.56 m² = 21.80 m² div. by 2 = 10.90 m²
6.) 10.90 m² X 0.09 m = 0.98 m³
7.) 0.98 m³ divided by 135 sec. = 0.0073 m³/sec
8.) 0.0073 m³/sec divided by 10.90 m² = 0.00067 m/s, or 0.067 cm/s.

5.0 DESIGN – Gallery in Natural Alluvium

5.1 Filter Material
The collector pipe should be surrounded by a gravel filter, the purpose of which is to prevent fines from the in-situ alluvium from migrating into the collector pipe. The grain-size distribution of the media surrounding the pipe should conform to specific requirements, and a procedure for designing such filters is articulated in Gradation Design of Sand and Gravel Filters, USDA-NRCS National Engineering Handbook, Part 633, Chapter 26 (Ref. #18). Because there is likely to be a bit of “surging” as diversion pumps are started and stopped, the filter should be designed according to the critical drain criterion in step 12 of that procedure.

5.2 Size (Diameter or Radius) of Collector Pipe
The axial velocity within the collector pipe should be low enough that there is no appreciable head loss as water flows within that pipe. In that regard, velocities should be less than about 0.9 m/s, meaning that the minimum diameter of the collector should be:

\[ D_{\text{min}} \geq 1.19 \sqrt{Q} \]

where:
\( D_{\text{min}} = \) minimum diameter of collector pipe (m)

\( Q = \) total required diversion flow, or portion thereof, that is to flow in collector pipe (m³/s)

(for example, if the total required diversion flow were 1,000 USGPM ((0.063 m³/s), the minimum size of collector pipe would be about 0.3 m (300 mm) diameter. However, if the collector system consisted of four pipes manifolded together, the minimum size of each collector pipe would be about 0.15 m (150 mm), but the header into which they are manifolded should be 0.3 m in diameter).

5.3 Length of Collector Pipe(s)
The following equation (from Ref. #6) provides a relation to determine the required length of collector pipe in a bed-mounted gallery²:

\[
L_{st} = \frac{0.366Q\log\left(\frac{1.1d}{0.5D}\right)}{0.25KH}
\]

where,…

\( K \) – hydraulic conductivity of the porous media (m/day)
\( Q \) – discharge (m³/day)
\( d \) – burial depth (m)
\( H \) – submergence of infiltration screen (m)

as in the following definition sketch,…

5.4 Check of Infiltration Area Size
The total area of the infiltration gallery will be related to the selected layout of the system. The following sketch illustrates the suggested layout for a bed-mounted infiltration gallery:

² Given the difficulty and uncertainty inherent in determining the hydraulic conductivity of the natural alluvium, the designer may wish to incorporate a “factor-of-safety”. Selection of an “appropriate” safety factor would depend on the intended purpose of the water and the consequences of not being able to divert water at the desired rate. A suggested safety factor is 1.5-2 for relatively low-risk situations, and 2-3 for higher risk installations.
According to this layout, the area of the infiltration gallery will be

\[
A = (2n + 1)/(L_s + d)d
\]

where,…

- \(A\) = area of infiltration gallery (m²)
- \(d\) = burial depth (m)
- \(L_s\) = length of each collector (m)
- \(n\) = number of collector pipes
- \(L_{st}/n\) (\(L_{st}\) = total length (m) of collectors)

Assuming that the entire area has a hydraulic conductivity equivalent to the river alluvium, the flow that would be available to the gallery would be:

\[
Q = KA(H/d)
\]

where,…

- \(Q\) = flow available for diversion (m³/s)
- \(A\) = area of infiltration gallery (m²)
- \(d\) = burial depth (m)
- \(H\) = submergence of infiltration screen (m)
- \(K\) = hydraulic conductivity of the river alluvium (m/s)

If the flow determined through this check is equal to or greater than the required diversion flow, then the size of the gallery is adequate. If this flow is less than the required diversion flow, the size of the gallery will have to be increased by adding another collector pipe, or lengthening the collector pipes, or both.

5.5 Percent Open Area in Collector Pipe

Many references suggest that entrance velocities through screen slots or perforations should be less than about 0.03 m/s (3 cm/s). The reasons for this limit are cited as being: minimizing head losses through the screen; minimizing the rate of encrustation from mineral precipitates; and minimizing the rate of corrosion. But, there appears to be little documented evidence to support this conclusion. Some research suggests that entrance velocities as high as 1 m/s can be acceptable. The same research suggests that the percent open-area of the screen should be a minimum of about 5% of the total pipe area. Nevertheless, the 3 cm/s criterion could be viewed as a conservative guideline.
Creating the required open area can be accomplished in a number of ways. Commercially-available well screen can easily meet this criterion, as can some commercially-available slotted drain pipe. In some instances, leachfield chambers for septic fields have been used. Perforated pipe can be made by cutting slots or holes in solid-wall pipe with drills, saws or torches. However, such techniques have the following limitations:

- Structural strength considerations require relatively wide spacing between slots;
- Opening sizes can be inconsistent;
- The finished product can have a very low percentage of open area;
- It is difficult to impossible to cut openings small enough to retain fine sand;
- Jagged edges around slots cut in metal pipe are more susceptible to corrosion.

### 5.6 Collapse Strength of Collector Pipe

When subjected to an external hydrostatic load (and/or internal vacuum pressure) that exceeds their buckling strength, pipes tend to fail by buckling. The buckling strength of pipe depends on the material comprising the pipe, the wall thickness and diameter, and the shape of the wall (in the case of corrugated pipe). As stated in the previous section, perforating a pipe can significantly reduce its buckling strength. Because of the wide variety of pipes and materials available, as well as the variety of perforation configurations possible, determination of buckling strength will not be addressed here. Manufacturers (or their literature) should be consulted regarding the buckling strength of a pipe or screen that is to be used in an infiltration gallery.

For design purposes, it should be assumed that the pipes comprising the infiltration gallery would be empty and the internal pressure would be atmospheric. Under such conditions, the design pressure would be equal to the combined hydrostatic pressure due to the weight of the overlying water and backfill,

\[ P = H \gamma_w + d (\gamma_s - \gamma_w) \]

where,

- \( P \) = external pressure on pipe (kPa)
- \( H, d \) = as defined in the definition sketch in section 5.4
- \( \gamma_w \) = unit weight of water (kN/m³)
- \( \gamma_s \) = saturated unit weight of alluvium (kN/m³)

### 5.7 Design of Sump

To reduce the chances of biofouling of the collector drains, the water level in the sump should not fall below the crown of the collector pipes. Assuming that the maximum pumping drawdown of the water level in the sump would coincide with the crown of the collector pipes, the bell-end of the pump intake should be set at an elevation sufficiently below that level to avoid the possibility of vortices compromising pump performance.

Vortex formation above intakes is a complicated problem in fluid mechanics that does not easily lend itself to an analytical solution. Critical submergence requirements to prevent air-drawing vortices from forming are dependent on the approach-flow patterns and other sources of vorticity in the surrounding media, and hence, a universal value of critical submergence is not meaningful. However, experimental work and observation of field installations has provided some information on which to base an empirical guideline for rationally determining an appropriate level of submergence to prevent air admittance to an inlet due to vortex formation. The following sketch illustrates the defining variables to be considered:
The submergence Froude number \((F_s)\) can be defined as,…

\[
F_s = \frac{4Q}{\pi D_b^2 \sqrt{gs}}
\]

where,…

\(Q\) = diversion flow \((m^3/s)\) \hspace{2cm} \(D_b\) = diameter of belled end of suction intake \((m)\)

\(g\) = gravitational acceleration \((m/s^2)\) \hspace{1cm} \(s\) = distance of bell end below water surface \((m)\)

To prevent vortex formation, the submergence Froude number \((F_s)\) should be less than about 0.6 for relatively quiescent flows, and less than about 0.25 for flows that already have a fair amount of vorticity associated with them (Ref. #12). Because the flow in the delivery pipe is relatively slow, the more relaxed criterion is probably adequate. If vortex formation becomes a problem regardless of whether this criterion is met or not, a vortex-suppression plate can be affixed to the pump column, as shown:

Unless otherwise directed by the pump manufacturer, the floor of the sump should be located 0.3 to 0.5\(D_b\) below the suction bell (Ref. #13).

The top of the sump should be set at a level above known or anticipated flood levels in the river to ensure that flood waters do not enter the wet well and contaminate the system.

5.8 Example

An existing irrigation intake consists of a surface-water intake located on a pilot channel in the floodplain of a river. To divert water to the existing intake site, the irrigator constructs a “push-up” dam in the main channel of the river (see following figure).
Construction of the push-up dam is required on an annual basis, or even more frequently, because the push-up dam washes out in the spring, and it has a tendency to push the main channel of the river further away during moderate-to-high flows. The irrigator would like to construct an infiltration gallery intake to avoid the ongoing requirement for entering and altering the river channel. The maximum required diversion flow is about 1,000 USGPM (0.063 m$^3$/s or 5,450 m$^3$/day).

5.8.1 Siting – River Morphology

The slope of the river is about 0.002 over the reach in question, it has a sinuosity of about 1.25, it is slightly entrenched, and it has a width/depth ratio of between about 80-100. As will be shown quantitatively in section 5.8.3, the bed material consists primarily of sand, with some gravel sizes. Based on this information, the reach of river under consideration would be classed as a C4 or C5 (more likely C5) according to the Rosgen classification system (depicted in the following figure).
Two gauging stations bound the site in question – one located about 30 km (river distance) upstream, the other located about 60 km downstream. The following chart describes some of the hydraulic and hydrologic characteristics of the river at those locations:

As can be seen from this chart, water depths at the site are likely to be very low for much of the time (<0.5 m), meaning that development of an open-water intake would be quite difficult. However, flow velocities are likely to be above 0.3 m/s for significant periods of time. As such, the site seems to be a relatively good candidate for an infiltration gallery diversion.

The site along the river in question is located a short distance upstream from the reservoir impounded by a dam that was constructed in the 1960’s. The site is far enough upstream of the reservoir that water levels in the river are not affected by the reservoir, and the upstream reaches are unregulated and largely undeveloped. As such, the site is unlikely to be subject to aggradation or degradation. This can be confirmed by noting that the rating curve for the gauging station upstream is stable (although it is located some 30 km (river distance) upstream). However, the lateral stability of the site is of some concern, as shown below:
The crossing area, indicated by the tip of the label flag for the river in the previous picture, might be an appropriate location for an infiltration gallery, but the bank would have to be stabilized to avoid further migration. A more appropriate site might be further downstream, just upstream of the bridge.

Note that this is a fairly cursory analysis of the stability of the site. Further evaluation of the vertical stability of the site should probably include development of a sediment budget for the reach in question (based on a few surveyed cross sections and a pebble count). Additional analysis of the lateral stability of the site should include a site reconnaissance to determine the composition of the material comprising the banks, and a comparative analysis of a time-series of air photographs going back to before the dam was constructed.

5.8.2 Siting – Regional Geology
The general geologic setting of the river in which the proposed infiltration gallery is to be installed is illustrated in the geologic cross-section below:

As can be seen, the river is incised within surficial deposits that overlie bedrock of the Disturbed Belt, an area of the Rocky Mountains and Foothills that has been deformed by folding and thrust faulting. The surficial deposits consist of varying thicknesses of glacial deposits composed primarily of clay till material, with intermittent sand and gravel deposits.

Within the river valley, the river meanders across a floodplain that consists of alluvial material deposited by the river itself. Across the valley, the alluvial deposits can overlie either the surficial glacial deposits or the bedrock. In general, the river bed itself generally consists of alluvial deposits of sand and gravel overlying bedrock, with the depth from the river bed to bedrock varying from less than 1 m to over 4 m. Between 70% and 90% of both banks consist of alluvial material over the 5km reach in question; that is, the entire channel cross-section is incised within alluvial material. Where the banks do not consist of alluvial material, they are either clay till, shale, or some combination of the two materials. Such situations occur where the river impinges on the valley walls. The following cross section is typical:
In plan, the surficial geology of the reach under consideration is illustrated below:

In terms of the general geology, almost anywhere along the river would be a good site for an infiltration gallery. Sites to be avoided would be those where the river impinges on the valley walls, or where the depth to bedrock is less than about 1.5 m. As can be seen in the plan above, the site under consideration appears to be as good as any other, from a general geological viewpoint.

Because the entire valley floor consists of alluvial material, it may be possible to meet the water requirements with a land-based infiltration gallery, or even a series of shallow vertical wells developed in a location remote from the river itself. In either case, a test drilling programme would be required to confirm the feasibility of such a development. For the purposes of this example, we will assume that such developments would not be feasible, and that a bed-mounted gallery is necessary to meet the flow requirements.

5.8.3 Site Geology and Design of Filter
Analysis of samples of the alluvial material at and near the site selected for the proposed infiltration gallery indicate that the material has the following grain-size distribution:
Using methods articulated in the document in Reference #16, a filter can be designed for this material, as shown below:
We will assume that the collector pipes would be buried at a depth of about 1.2 m beneath the surface of the existing river bed, and that the low-water elevation of the water surface in the river would be about 300 mm above the bed of the river. As can be seen in the annotation below the title in the plots of the grain-size distributions of the bed material, the average hydraulic conductivity of the alluvial material in the river bed has been estimated by the four equations presented in Section 4.2 as being about 0.132 cm/s (assumes a water temperature of about 8°C). This would be adequate for a conceptual design, but a final design would require field tests to determine the hydraulic conductivity of the in-situ material. Nevertheless, for the purposes of this example, we will assume that $K=0.132$ cm/s, or 114 m/day.

According to these assumptions, the defining characteristics for the design of the collector system will be:

![Defining Characteristics of Infiltration Gallery](image)

### Infiltration Gallery Sizing

<table>
<thead>
<tr>
<th># of Collector Pipes</th>
<th>Required Diameter for Velocity of 0.9 m/s $D_{\text{min}}$ (m)</th>
<th>Required Length of Individual Collector Pipe $L_s$ (m)</th>
<th>Total Required Length of Collector Pipe $L_{\text{tot}}$ (m)</th>
<th>Area of Infiltration Gallery $A$ (m²)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.300</td>
<td>45</td>
<td>45</td>
<td>222</td>
</tr>
<tr>
<td>2</td>
<td>0.250</td>
<td>24</td>
<td>48</td>
<td>182</td>
</tr>
<tr>
<td>3</td>
<td>0.200</td>
<td>18</td>
<td>54</td>
<td>185</td>
</tr>
<tr>
<td>4</td>
<td>0.150</td>
<td>15</td>
<td>60</td>
<td>195</td>
</tr>
<tr>
<td>6</td>
<td>0.150</td>
<td>10</td>
<td>60</td>
<td>189</td>
</tr>
<tr>
<td>10</td>
<td>0.100</td>
<td>7</td>
<td>70</td>
<td>217</td>
</tr>
</tbody>
</table>

As articulated in section 5.2, a size of collector pipe was chosen such that the velocity within that pipe would be less than about 0.9 m/s. If the infiltration gallery is to be made up of a manifoded series of pipes, it was felt that the required size of pipe could be reduced, since any individual pipe within the manifoded system would only carry a portion of the total discharge proportional to the number of pipes.

It is interesting to note that the possibility of reducing the size of the collector pipes in a manifoded situation (due to the lower flow carried by each pipe) results in a longer total required length for the smaller pipes, but a variable required area for the infiltration gallery. Without a full accounting of the cost of labour and materials, it is not possible to say which the least-cost alternative would be, but it would appear as though a manifoded series of six 150 mm diameter pipes would be the optimum configuration. Note that the header pipe that each of the six collector pipes feeds into would have to be 300 mm in diameter, because it would carry the full discharge. There is not likely to be a significant cost differential between any of the alternatives, so the layout would more likely be governed by considerations relating to construction, such as ease of de-watering, site geology, etc.
5.8.5 Check of Infiltration Area Size

Based on the methods articulated in section 5.4, the following table shows the results of the check to see if the proposed area could deliver the required flow:

<table>
<thead>
<tr>
<th># of Collector Pipes</th>
<th>Required Diameter for Velocity of 0.9 m/s $D_{min}$ (m)</th>
<th>Area of Infiltration Gallery $A_i$ (m$^2$)</th>
<th>Flow Available From Infiltration Area (m$^3$/day)</th>
<th>Is Flow Available From Infiltration Area &gt; Required Flow?</th>
<th>Induced Downward Velocity at Stream Bed (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.300</td>
<td>222</td>
<td>31,635</td>
<td>YES</td>
<td>0.0003</td>
</tr>
<tr>
<td>2</td>
<td>0.250</td>
<td>182</td>
<td>25,935</td>
<td>YES</td>
<td>0.0003</td>
</tr>
<tr>
<td>3</td>
<td>0.200</td>
<td>185</td>
<td>26,363</td>
<td>YES</td>
<td>0.0003</td>
</tr>
<tr>
<td>4</td>
<td>0.150</td>
<td>195</td>
<td>27,788</td>
<td>YES</td>
<td>0.0003</td>
</tr>
<tr>
<td>6</td>
<td>0.150</td>
<td>189</td>
<td>26,933</td>
<td>YES</td>
<td>0.0003</td>
</tr>
<tr>
<td>10</td>
<td>0.100</td>
<td>217</td>
<td>30,923</td>
<td>YES</td>
<td>0.0003</td>
</tr>
</tbody>
</table>

As can be seen from this table, if it is assumed that water would flow uniformly through the entire area of the infiltration gallery (in plan), the flow that would be available through the alluvium is almost quadruple the required flow, implying that the design is fairly conservative. However, the assumption of uniform flow is fairly simplistic$^3$, and if one takes into account the potential for the reduction in system capacity that can take place as the filter media in the upper layers becomes clogged with fines or biochemical clogging, then it is not excessively conservative. It can also be seen from the above table that the induced downward velocity at the surface of the stream bed would be very small (based on the same simplistic assumption), meaning that entrainment or entrapment of fish fry, fish eggs and aquatic biota would be very unlikely.

5.8.6 Percent Open Area of Collector Pipe

To be conservative with respect to movement of filter media into the collector screens, the size of the perforations or slots in the pipe should be smaller than the $d_{15}$ of the surrounding filter media (Ref. #18). The $d_{15}$ of the filter is about 2-4 mm (0.08-0.16 in.). Commercially-available slotted PVC well casing can meet this criterion, as shown in the table below:

$^3$ It does not account for the convergence of flow in the alluvium as it approaches the collector pipes.
Using the information from the preceding table (manufacturer’s literature) for a slot width of 1.25 mm (0.05 in.), the following table illustrates what the velocity through the openings would be for the various configurations considered:

<table>
<thead>
<tr>
<th># of Collector Pipes</th>
<th>Required Diameter for Velocity of 0.9 m/s (D_{min}) (m)</th>
<th>Required Length of Individual Collector Pipe (L_s) (m)</th>
<th>Total Required Length of Collector Pipe (L_{st}) (m)</th>
<th>Area of Infiltration Gallery (A) (m²)</th>
<th>Open Area of Infiltration Pipe (A) (m²/m)</th>
<th>Velocity Through Openings (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.300</td>
<td>45</td>
<td>45</td>
<td>222</td>
<td>0.118</td>
<td>0.012</td>
</tr>
<tr>
<td>2</td>
<td>0.250</td>
<td>24</td>
<td>48</td>
<td>182</td>
<td>0.089</td>
<td>0.015</td>
</tr>
<tr>
<td>3</td>
<td>0.200</td>
<td>18</td>
<td>54</td>
<td>185</td>
<td>0.080</td>
<td>0.015</td>
</tr>
<tr>
<td>4</td>
<td>0.150</td>
<td>15</td>
<td>60</td>
<td>195</td>
<td>0.061</td>
<td>0.017</td>
</tr>
<tr>
<td>6</td>
<td>0.150</td>
<td>10</td>
<td>60</td>
<td>189</td>
<td>0.061</td>
<td>0.017</td>
</tr>
<tr>
<td>10</td>
<td>0.100</td>
<td>7</td>
<td>70</td>
<td>217</td>
<td>0.039</td>
<td>0.023</td>
</tr>
</tbody>
</table>

As can be seen, the flow velocity through the openings would be well below the 3 cm/sec limit that is generally used as a guideline.

### 5.8.7 Collapse Strength of Collector Pipe

As stated in section 5.6, it will be assumed that the pipe must be able to withstand the full hydrostatic pressure of the overlying water and backfill. Assuming that the saturated unit weight of the granular material comprising the filter and the river alluvium have an average unit weight of about 21.6 kN/m³, and that the depth of backfill is 1.2 m above the center of the pipe, and the depth of water is 2.0 m above the center of the pipe (corresponding to a flood stage in the river), the hydrostatic loading on the pipe \(P\) would be about 34 kPa (4.9 psi).

The following table, from the manufacturer’s literature, indicates what the collapse strength would be for the various pipe sizes and wall thicknesses.

**RESISTANCE TO HYDRAULIC COLLAPSE PRESSURE (RHCp) OF PVC WELL CASING**

<table>
<thead>
<tr>
<th>SDR RATED</th>
<th>SCHEDULE RATED</th>
<th>SCHEDULE RATED</th>
</tr>
</thead>
<tbody>
<tr>
<td>SDR</td>
<td>RHCP (PSI)</td>
<td>SIZE</td>
</tr>
<tr>
<td>41</td>
<td>14</td>
<td>2&quot;</td>
</tr>
<tr>
<td>32.5</td>
<td>29</td>
<td>3&quot;</td>
</tr>
<tr>
<td>27.6</td>
<td>49</td>
<td>4&quot;</td>
</tr>
<tr>
<td>26</td>
<td>50</td>
<td>4 1/2&quot;</td>
</tr>
<tr>
<td>21</td>
<td>115</td>
<td>5&quot;</td>
</tr>
<tr>
<td>17</td>
<td>224</td>
<td>6&quot;</td>
</tr>
<tr>
<td>13.5</td>
<td>471</td>
<td>8&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>10&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>14&quot;</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16&quot;</td>
</tr>
</tbody>
</table>

The collapse strength of any of the listed configurations would exceed the anticipated loading on the pipe; therefore the thinnest readily available wall thickness is selected (SDR26).

### 5.8.8 Design of Sump

The pump selected for this application is a vertical turbine pump with a 6" (150 mm) diameter pump column and a 12" (300 mm) diameter suction bell on the bowl end. The manufacturer’s literature indicates that the required clearance between the suction bell and the floor of the sump is about 8" (200 mm).
Using the more conservative criterion, where the submergence Froude number should be less than about 0.25, the required depth of submergence for the suction bell would be about 1.3 m. Therefore, the bottom of the sump should be set at an elevation about 1.5 m below the crown of the collector pipes (1.3 m to meet the submergence criteria, and 0.2 m to satisfy the manufacturer’s recommended clearance between the suction bell and the bottom of the sump). According to the hydraulic characteristics of the river, presented in Section 5.8.1, the depth of flow in the river that corresponds to a 1:50 peak flow would be about 1.8 m. So, if the top of the sump is set at a level about 2 m higher than the river bed, there should be a relatively small likelihood of it being inundated by flood waters.

6.0 DESIGN – High-Rate Infiltration Galleries

6.1 Introduction

If the natural alluvium is not sufficiently permeable to allow an infiltration gallery based on the in-situ hydraulic conductivity of the alluvium to be feasible (due to unreasonable size), it may be possible to create an infiltration gallery entirely from non-native material that has a higher permeability than the natural alluvium. However, if the natural alluvium contains enough fines to preclude development of a gallery based on the hydraulic conductivity of the natural material, it is likely that the river, at least occasionally, carries enough fine suspended sediment that a gallery constructed with non-native material will eventually become plugged with fine sediment.

Research on fish-spawning habitat has identified two mechanisms by which fine sediment intrudes into a gravel framework containing open pores; infiltration based on gravity, and infiltration of fine sediment based on interstitial flows. Intrusion of fines into an infiltration gallery would most likely be due to the latter mechanism. In such circumstances, the rate of fine-sediment intrusion increases with the concentration of suspended sediment, the size and number of open pore spaces, and the rate of interstitial flow (Ref. #2).

Fine sediment intrusion into a non-stratified deposit of coarse gravel can create a variety of vertical stratifications within that deposit, depending on the intra-gravel pore sizes and the size of the infiltrating particles. If the infiltrating particles are finer than the intra-gravel pores, the infiltrating sediment fills the pore space from the bottom up, causing no pronounced vertical variation of infilled particle sizes. If the fine sediment is a mixture of silt, sand and fine gravel, fine gravel can eventually seal the pore spaces near the surface and prevent finer sediment from infiltrating into deeper pores. Depending on how fast the near-surface pore space is sealed, there can be a gradually-upward coarsening of the infiltrated fines, or a layer below the surface that is free of in-filled fines.

These processes can lead to what has been termed the “clogging paradox”, where less permeable sand sustains higher infiltration rates, for longer periods of time, than gravel, in a semi-natural system (Ref. #17). The reason is that “cake” that accumulates on the surface of a sand deposit can be more easily removed by the scouring action of river flows (or by backwashing) than material that has infiltrated deeply into the more open framework of coarse gravel. The following table and figure illustrates this situation:

<table>
<thead>
<tr>
<th>Material Characteristics</th>
<th>Sand</th>
<th>Gravel</th>
</tr>
</thead>
<tbody>
<tr>
<td>Particle Size (mm)</td>
<td>0.064-2</td>
<td>&gt;2</td>
</tr>
<tr>
<td>K (m/day)</td>
<td>1-100</td>
<td>100-1000</td>
</tr>
<tr>
<td>Porosity</td>
<td>0.3-0.4</td>
<td>0.2-0.3</td>
</tr>
<tr>
<td>Clogging characteristics</td>
<td>Surface cake</td>
<td>Deep infiltration</td>
</tr>
</tbody>
</table>
Further illustration of this phenomenon can be found from the results of a recent investigation of an existing infiltration gallery (Ref. #10). As shown in the photograph at the left, the original filter pack consisted of coarse, poorly-graded, uniform "drain rock" with a median size of about 20 mm. It was placed within natural alluvium that contained significant amounts of fine material. Soon after construction, the capacity of the infiltration gallery began to diminish.

Subsequent investigation using freeze-core sampling revealed that at least one cause of the diminished capacity was that fine material from the river and/or from the surrounding and overlying natural alluvium had migrated into the interstitial pore spaces of the filter pack throughout its entire depth, as shown in the following photographs:
6.2 Backwashing Requirement
Recognizing that some form of mechanical clogging of the filter media is probably inevitable, some means of clearing or restoring the capacity of the media would be required. Periodic scarification of the surface is one method that has been employed, but this method requires physical entry to the water body and disturbance of the stream bed, and as such, is no longer considered an acceptable practice. Another method is reversal of flow through the media, or backwashing.

Effective backwashing of the filter medium requires that there be sufficient inter-particle abrasion and flow to remove the accumulated fine material. If water alone is used for backwashing, the flow required has to be slightly more than that required to “fluidize” the filter medium. The flow velocity required for bed fluidization can be determined from the following relations:

\[ V_{mf} = \frac{\mu}{\rho d_{eq}} \sqrt{\frac{33.7^2 + 0.0408Ga}{\rho \mu}} - \frac{33.7 \mu}{\rho d_{eq}} \]

\[ Ga = d_{eq}^3 \frac{\rho (\rho s - \rho) g}{\mu^2} \]

where,…

\[ V_{mf} \] = bed fluidization velocity  \( g \) = gravitational acceleration  
\( \mu \) = absolute viscosity of water  \( \rho \) = mass density of water  
\( \rho s \) = mass density of filter particles  \( Ga \) = Galileo number  
\( d_{eq} \) = equivalent spherical diameter of filter media grains  usually \( d_{65} \) or \( d_{90} \) to ensure full fluidization of the bed

Backwashing with water alone is a relatively weak washing method, and it also requires a significant flow of water. Because infiltration galleries are usually backwashed in sections, where one part of the gallery is backwashed with water drawn from the rest of the gallery, the flow rate available for backwashing will be limited by the capacity of the gallery itself, which, naturally, diminishes somewhat between backwash cycles. For that reason, backwashing of infiltration galleries should ideally employ methods whereby the water backwash is assisted or supplemented with some other method.

An air-scour-assisted backwash method, where air and water are introduced simultaneously, is probably the most effective and efficient, because the required flow of water is well below that required for full bed fluidization, and there are high interstitial water velocities and inter-particle abrasion. The required rates of air and water flow in a simultaneous air-water backwash system can be determined from the following relation:

\[ 0.45Q_a^2 + 100 \left( \frac{V}{V_{mf}} \right) = 41.9 \]

where,…

\( Q_a \) = air flow rate (scfm/ft²)  \( V \) = superficial water velocity within porous media

and \( V/V_{mf} \) is the ratio of the superficial water velocity to the minimum fluidization velocity, based on the \( d_{60} \) grain size of the filter media.

The preceding relationships were developed for rapid-sand filters typically used in water treatment applications (Ref #1). Guidelines for the design of such filters suggest that the media for filters that are to be washed with air and water simultaneously should consist of relatively coarse, poorly-graded sand-size material (an effective size (ES) or \( d_{10} \) greater than 1 mm, and a uniformity coefficient (\( d_{60}/d_{10} \)) less than about 1.65). The hydraulic conductivity of such material is more than sufficient for most infiltration gallery applications, and the use of coarser material is not recommended, because it would require higher backwash flow rates. If the screen openings are too large to prevent the filter medium from entering the screen, coarser material can be placed in the region immediately surrounding the collector screens. In such situations, the screens should be surrounded with a gravel pack, and the gravel should be separated from the sand filter with a geogrid, woven wire screen, or woven geotextile to prevent movement of the support gravel into the filter during backwashing.
6.3 Design Process

6.3.1 River Morphology, Regional Geology & Site Geology
All of the analyses associated with evaluation of the geologic and morphologic setting of the proposed infiltration gallery would be applicable to any proposed infiltration gallery. The only difference is that the hydraulic conductivity of the natural alluvium is of lesser importance.

6.3.2 Gallery Design
The design process should start with the selection of filter media. As stated previously, the media should have an effective size (ES) of about 1 to 2 mm, and a uniformity coefficient of about 1.6-1.7. Then, rather than approaching the problem in a step-by-step manner, as was done for infiltration galleries based on the natural alluvium, the size, number and total length of collector screens, the depth of installation, and the allowable screen entrance velocity, should be varied on a trial basis until a gallery configuration is achieved that satisfies criteria previously established, as well as the ability to draw the required backwash flow for an individual screen from the portions of the gallery that are not being backwashed. A spreadsheet is an ideal tool for such purposes. As part of this process, the required percent open area of the collector pipe should be determined and a check made to ensure that screen material is available that can satisfy that requirement. The collapse strength of the selected screen should also be checked.

Note that there is no unique solution for the required rates of air and water flow to backwash the filter; a range of water flows corresponding to various air flow rates will form a solution set for a given situation. It is left to the discretion of the designer to select what they feel is a reasonable rate of water flow that can be drawn from those portions of the gallery not being backwashed. The rate of backwash water flow can be estimated as the superficial water velocity within the porous media times the surface area of the gallery.

6.3.3 Design of Backwash System
The backwash system should consist of an array of diffuser pipes, installed above each collector screen, that can distribute air and water uniformly throughout the filter media, as shown in the following sketches:
For a situation as depicted in the preceding sketches, the required number of diffuser pipes \((n_d)\) can be estimated from the following relation:

\[
n_d = \text{int}\left[\frac{10d}{d - 1.5D}\right]
\]

It follows that, if the collector screens are spaced a distance of \(2d\) apart, the required spacing of the diffuser pipes \((s_d)\) will be:

\[
s_d = \frac{2d}{n_d + 1}
\]

The validity of this design approach (including the design for the air and water diffusers that follows) was verified through testing on a physical model (Ref #15). It should be noted that, to ensure that the flow of backwash water is confined to the medium above a given collector pipe, an impermeable barrier should be included to isolate that portion of the media from those portions of the gallery that are not being subjected to backwashing (see sketch on previous page).

### 6.3.3.1 Design of Water Diffuser System

The water diffuser pipes should be the same length as the collector screens, and should have a series of holes or ports drilled in them, with the orientation of the holes being such that the backwash flow is directed vertically upward.

To ensure that the backwash flow is distributed uniformly among the holes or ports along the diffuser, conditions must be such that the effects of friction within the diffuser pipe tend to cancel out the effects of momentum of the flow discharging from the diffuser ports. For this condition to be satisfied, there is an optimal diameter of diffuser for a given flow and length of diffuser (for diffuser pipes larger than this optimal diameter, there will tend to be greater discharges near the downstream end of the diffuser, with the reverse being true for diffuser pipes smaller than the optimal diameter). From Reference #5, the optimal diameter can be determined from,…

\[
D_h = \frac{0.353}{2 - \gamma_d} f L_s
\]

where

\[
D_h = \text{optimal diffuser diameter} \quad L_s = \text{length of diffuser} \quad \gamma_d = \text{pressure recovery coefficient (~0.86)} \quad f = \text{Darcy friction coefficient}
\]
The Darcy friction coefficient, for a given flow and pipe diameter, can be approximated as:

\[ f = \frac{1.325}{\left[ \ln \left( \frac{e}{3.7D_h} + \frac{5.74}{\nu \sqrt{\frac{e}{3.7D_h}}} \right) \right]^{\frac{1}{2}}} \]

where

- \( \nu = \) kinematic viscosity of fluid flowing in pipe
- \( e = \) height of roughness element on pipe wall (~0.06 mm)

The flow in each diffuser pipe will be the total selected backwash water flow divided by the number of diffuser pipes \( (n_d) \).

Solving for the optimum diffuser diameter will, by necessity, be an iterative process. The nearest standard pipe size to this optimum diameter should be selected.

The required number and size of ports will similarly require an iterative solution, although, in this case, there is no unique solution. Rules-of-thumb for multi-port diffusers suggest that the ratio of the total port discharge area to the cross-sectional area of the diffuser pipe should be between \( \frac{1}{3} \) and \( \frac{1}{2} \) (Ref. #5, 8). This area ratio \( A_r \) can be defined as:

\[ A_r = \frac{n_p d_p^2}{D^2} C_d \]

where

- \( n_p = \) number of ports
- \( d_p = \) diameter of port
- \( D = \) diameter of diffuser
- \( C_d = \) discharge coefficient of port

For a given number and size of ports, the required head difference across the port to deliver the required discharge can be estimated from (Ref. #8)

\[ \Delta H_p = \frac{\left( \frac{q_p}{C_d A_p} \right)^2}{2g} \]

where

- \( \Delta H_p = \) head difference across port
- \( A_p = \) cross-sectional area of port
- \( q_p = \) port discharge \( = \frac{Q_d}{n_p} \)
- \( g = \) gravitational acceleration

The discharge coefficient in the preceding expressions depends on the geometric characteristics of the port, as well as the ratio of the velocity head in the diffuser to the total head difference across the port. For a sharp-edged port, such as that which would result from simply drilling holes in a pipe, the coefficient of discharge can be determined from,

\[ C_d = 0.63 - 0.58 \left( \frac{4Q_d}{\pi D^2} \right) \frac{\left( \frac{4Q_d}{\pi D^2} \right)^2}{2g \Delta H_p} \]

This relation was developed for situations where the port diameter was small relative to the diameter of the diffuser pipe \( (d_p < 0.1D) \) (Ref. #8).
The pressure required at the header to deliver this flow will be the sum of:

- the head difference across the port ($\Delta H_p$)
- the hydrostatic head above the diffuser (~ (H-D)), and
- the head required to induce the flow of backwash water through the overlying media at the selected velocity ($\Delta H_b$)

or, ...

$$\Delta H = \Delta H_b + \Delta H_p + (H - D)$$

The head required to induce backflow through the porous media can be estimated from Darcy's law by assuming that the length of the flow path is approximately (d-D), and remembering that the hydraulic gradient is the head differential divided by the length of flow path. Thus, the required head differential required to induce backwash through the overlying media at the selected velocity would be:

$$\Delta H_b = \frac{V(d - D)}{K}$$

where $K$ is the hydraulic conductivity of the overlying media.

### 6.3.3.2 Design of Air Diffuser System

The air diffuser pipes should be the same length as the collector screens, and should have a series of holes drilled in them at a spacing of between 50 mm and 125 mm (Ref. #9). The sketch to the right illustrates the recommended layout for the holes. Note that the holes should be oriented so that they’re on the bottom of the diffuser pipe (facing down) when it’s installed. This is to ensure that most of the water can be forced out of the pipe when the air is turned on, thereby allowing almost the full cross section to be available for conveyance of air.

For the selected air backwashing flow rate $Q_a$, the air flow in each of the diffuser pipes will be $Q_a/n_d$. The following table provides a guideline for selecting a size of diffuser pipe, depending on the air flow it is intended to carry:

<table>
<thead>
<tr>
<th>Range of Air Flow in Diffuser Pipe (L/s)</th>
<th>Recommended Sch. 40 Pipe Diameter (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>&lt;5</td>
<td>19</td>
</tr>
<tr>
<td>5-10</td>
<td>25</td>
</tr>
<tr>
<td>11-21</td>
<td>32</td>
</tr>
<tr>
<td>22-26</td>
<td>38</td>
</tr>
<tr>
<td>27-54</td>
<td>50</td>
</tr>
</tbody>
</table>

The required air flow that must be delivered through a single orifice will be the air flow in a single diffuser divided by the number of holes at the selected spacing. To deliver that air flow through a single orifice, the required pressure drop across the orifice can be determined from the following chart:
The lines on the chart extend over the acceptable range of pressure drops to achieve uniform distribution, but the shaded area indicates the most desirable region.

The required pressure in header that feeds the diffusers will be equivalent to: the hydrostatic pressure of the water body overlying the diffusers plus the pressure required to induce the water backwash flow through the overlying media; the pressure loss across the diffuser port required to deliver the desired air flow; and the pressure loss due to friction in the diffuser pipe. The following figure can be used to estimate friction losses associated with the flow of air in pipes. Because the flow of air in the diffuser diminishes with distance from the header, the friction loss in the diffuser pipe will be about $\frac{1}{3}$ of that predicted from the chart.
The required pressure at the blower that delivers air to the system will be roughly equivalent to the required pressure in the header, plus any frictional losses in the delivery line between the header and the blower.

6.4 Example

We will consider the same example as that presented in Section 5.8, with the exception that we will assume that the nature of the natural alluvium precludes the development of an infiltration gallery based on the permeability of that material.

6.4.1 River Morphology, Regional Geology & Site Geology
All of the analyses associated with evaluation of the geologic and morphologic setting of the proposed infiltration gallery would be the same as for the example in Section 5.8.

6.4.2 Selection of Filter Material
Because the natural alluvium is not suitable for development of an infiltration gallery within that material, the first step would be to select a material that could be used to create an infiltration gallery. Based on the criteria presented in Section 6.2, a material with a grain-size distribution as shown in the following chart, would be suitable:

6.4.3 Determine Size and Layout of Gallery That Would Facilitate Backwashing
A spreadsheet was used to simultaneously determine the required air and water backflush rates for this medium, as well as the required size and layout of the collector screens, based on the methodologies articulated in Sections 5.3, 5.4 and 6.2. The following figure is a screen-shot of the spreadsheet.

---

Note that if a factor-of-safety is applied in determining the required length of collector screens, it may result in a requirement for a fairly significant flow of water for backwashing.
For a given diversion discharge and burial depth, the number, size (diameter), and total length of screens, as well as the allowable screen entrance velocity, were varied until a reasonable open area for the screen, and a reasonable backwash water flow rate per screen, were obtained. A check was also included to ensure that the required driving head was less that the depth of water in the stream above the collector screens.

It can be seen that, for a layout involving five collector screens, each 5 m long (total length 25 m), the required backwash water flow rate for a single screen is well within the supply limits of the remaining four screens (800 USgpm), even if one were to assume diminished capacity due to some clogging of the filter medium. By comparison, if one were to propose a layout involving two screens, each 12.5 m long, the required backwash water flow rate for each screen would be greater than 1,000 USgpm over much of the applicable range of air and water flows. As such, it would probably not be possible to draw the required flow of backwash water from one side of the gallery if only two collector screens were used.

For the selected screen size (200 mm or 8" diameter) and entrance velocity, the percent open area for the screen should be about 73%, or in this case, about 117 in²/ft of screen. As determined in Section 5.8.7, the screen should be capable of withstanding an external pressure of about 34 kPa (4.9 psi). As can be seen from the following table, excerpted from manufacturer’s literature, 8" stainless steel well screen with a slot opening of 50-thousandths of an inch (1.27 mm) could meet these criteria.
It should also be noted that because the screen-slot width is smaller than the $d_{15}$ size of the material selected for creating the infiltration medium, there will be no need to surround the collector screens with a coarser gravel pack.

For the range of air and water flows required for backwashing, it seems reasonable to select a water flow of about 650 USgpm (0.041 m$^3$/s), for which the corresponding air flow would be about 330 cfm (155L/s). As can be seen in the spreadsheet, for the selected burial depth and screen diameter, each screen would require thirteen 5 m long diffuser pipes placed above it to distribute the flow of air and water for backwashing. In that case, the flow of air in each air diffuser would be about 25.3 cfm (11.9 L/s), and the flow of water in each water diffuser would be about 50 USgpm (0.003 m$^3$/s).

### 6.4.4 Design of Water Backwash System

As previously determined in Section 6.4.3, the required flow in each water diffuser would be about 50 USgpm (0.003 m$^3$/s). For a 5 m long diffuser, the optimum diameter has to be determined iteratively, as do the required number and size of holes or ports, and the pressure loss across the ports. A spreadsheet that utilizes macros is a tool that can be used for this purpose, and the following figure is a screen shot from such a spreadsheet:

For a 5 m long diffuser carrying a discharge of 50 USgpm, the optimum diameter is about 38 mm. If 6 mm diameter ports are used to discharge the water, the number of ports should be between about 23 and 33 to keep the area ratio between $\frac{A_p}{D_p}$ and $\frac{1}{2}$. Select the larger number of ports because the required pressure difference across the port is less than that required for the lower number of ports. The required pressure in the header would be the hydrostatic pressure of the overlying water (~1.1 m), plus the required pressure difference across the port (about 5 m), plus the pressure required to induce a backflow velocity of about 0.25 cm/s within the overlying media (the bed fluidization velocity for this material is about 2 cm/s, and the selected backflush flow corresponds to about 12.5% of that velocity). Thus, if the hydraulic conductivity of the media is 1.41 cm/s, and the length of the flow path is about 1 m, then the required head to induce this flow would be about 0.18 m, and the total pressure in the header would have to be about 6.3 m of H$_2$O, or 61.8 kPa.
Assuming that the discharge port on the pump that supplies the backwash water is connected to the header by an equivalent length of 50 m of 100 mm diameter pipe (that also comprises the header), the friction loss in the delivery line would be an additional 20 m of H2O for the 41 L/s total backwash water flow (determined from standard friction-loss calculations for water flowing in pipes). If the discharge port on the pump that supplies the backwash water is located at an elevation about 5 m higher than the diffuser pipes, then the required pressure at the pump would be 1.1 m, plus 5 m, plus 0.2 m, plus 20 m, minus 5 m, or about 21.3 m of H2O (about 209 kPa). This would probably be significantly less than the pressure that the pump would generate at that discharge at its rated speed, so a valve would be required to throttle the flow in the discharge line.

Recall that in Section 6.4.3, an observation was made that there would be no need for support gravel to surround the collector screens. However, because the openings in the backwash diffuser pipes are larger than the d15 size of the filter media, they would have to be encased in some material (e.g., stainless steel woven-wire screen) that would keep the filter media from entering the diffuser pipes.

### 6.4.5 Design of Air Backwash System

As determined in section 6.4.3, the required air flow in each diffuser would be about 25.3 cfm (11.9 L/s). According to the table in Section 6.3.3.2, the size of diffuser pipe for that air flow should be about 32 mm. Holes for diffuser ports should have a spacing of between 50 mm and 125 mm, so, for a 5 m long diffuser pipe, that would mean that there would have to be between 40 and 100 ports, meaning that the air flow through each orifice should be between 0.12 L/s and 0.30 L/s. If these values are plotted on the chart from Section 6.3.3.2,

![Air Flow Through a Single Orifice](image)

it can be seen that the required orifice size would be between about 2.5 and 4.0 mm (for a pressure drop across the orifice of about 10 mbar). We’ll select a 32 mm diameter pipe with one hundred 2.5 mm diameter holes spaced at 50 mm for the air diffusers.

As stated in Section 6.3.3.2, the required pressure in the header that feeds the diffuser pipes will be equivalent to the hydrostatic pressure exerted by the water surrounding the pipes, plus the head required to induce the selected backwash water flow through the overlying media, plus the pressure drop across the orifices, plus the friction loss in the diffuser pipes.
If the diffuser pipes are placed approximately half a screen diameter above the screen, and if the depth of water above the center of the collector screens is 1.5 m as in the previous example, then the hydrostatic pressure exerted by the water around the diffusers would be about 1.3 m of $H_2O$, or 127.4 mbar (relative to atmospheric pressure). The head required to induce the selected backwash water flow through the overlying media was calculated in section 6.5.4 to be roughly 0.2 m of $H_2O$, or 19.6 mbar. The pressure drop across the orifice was previously determined as roughly 10 mbar.

The pressure loss due to friction in the 5 m long diffuser pipes can be determined from the chart supplied in Section 6.3.3.2.

From the chart, for a flow of 11.9 L/s in a 32 mm diameter pipe, the friction loss would be about 150 mbar/100 m. Thus the pressure drop in the diffuser pipe would be $\frac{1}{2}$ of that indicated on the chart, as stated in section 6.3.3.2, or 50 mbar/100 m. For a 5 m long pipe, this would result in a pressure drop of 2.5 mbar.

Therefore, the required pressure in the header would be the sum of 127.4 mbar (hydrostatic pressure), plus 19.6 mbar (head to induce backwash water flow), plus 10 mbar (pressure drop across orifice), plus 2.5 mbar (diffuser pipe friction loss), for a total required header pressure of roughly 160 mbar.

If the entire air flow of 155 L/s (330 cfm) is delivered to the header through a 50 m length (equivalent pipe length, including losses for elbows and other fittings) of 80 mm diameter pipe (which also comprises the header), that would add roughly another 100 mbar of friction loss, as indicated on the chart above. Thus the total pressure that a blower would be required to deliver would be about 260 mbar, at an air flow rate of 155 L/s (330 cfm). Referring to manufacturer’s literature, these requirements could then be used to select a blower having the appropriate performance characteristics.
7.0 REGULATORY/ENVIRONMENTAL REQUIREMENTS

Development of infiltration galleries for diversion of water will be subject to both provincial and federal regulation, and may also be subject to municipal regulation as well. It would also be prudent for the prospective developer to check to see if the proposed development would affect any facilities owned by other agencies (railways, utilities, etc.).

7.1 Provincial Regulations

Provincial regulations that are likely to affect the development of an infiltration gallery could include the following:

- Licensing for the diversion of water;
- Approvals relating to in-stream work associated with site characterization;
- Approvals relating to the timing and nature of construction for the protection of fish and aquatic life;
- As the bed and shores of water bodies are, in most cases, designated as public land, approvals and authorizations, such as a license of occupation, may be required;
- Approvals relating to agricultural suitability for irrigation projects.

Information requirements associated with applications for provincial licensing and approval usually include:

- Plans showing legal land descriptions, water bodies with flow directions, high-and low-water levels, a layout of the proposed water distribution system, and the areas to be irrigated (if applicable);
- A written project description including: water requirements (maximum diversion rate and total annual volume), construction schedule and plans, and proposed operational details;
- A hydrologic analysis, indicating potential adverse impacts on the proposed water source, neighbouring lands and works, and the aquatic environment;
- For irrigation projects, an agricultural suitability report indicating soil and water compatibility, and water requirements for irrigation;
- Written permission for right-of-access from appropriate authorities where the proposed works would affect highways, roads, utilities or lands owned by others.

7.2 Federal Regulations

Federal regulations that are likely to affect the development of an infiltration gallery include the following:

- *Fisheries Act* – this act is administered by Fisheries and Oceans Canada (DFO) who have local offices across the country. DFO is responsible for coastal fisheries and fish habitat, as well as inland fish habitat in Canada. In inland waters, primary administration of the Fisheries Act deals with the prohibition to: harmfully alter, disrupt or destroy fish habitat; destroy fish by means other than fishing; restrict fish movement; and entrainment or impingement of fish by water management structures such as water intakes. This Act would apply to exploratory work conducted in or adjacent to a stream, as well as construction activities. For more information, contact your local DFO office or go to [http://www.dfo-mpo.gc.ca/home-acceuil_e.htm](http://www.dfo-mpo.gc.ca/home-acceuil_e.htm);

- *Navigable Waters Protection Act* – administered by Transport Canada, this act governs activities that can potentially affect the navigability of waterways. The principle objective is to protect the public right of navigation by prohibiting the building or placement of any “work” in, upon, over, under, through, or across a navigable waterway without the authorization of the Minister of Transport. The jurisdiction of the legislature begins at the high-water mark. Therefore structures that are between low- and high-water marks will require approval under the NWPA.
Species at Risk Act (SARA) - the purpose of the SARA is to: prevent wildlife species from becoming extinct or lost from the wild (extirpated); to help in the recovery of extirpated, endangered or threatened species; and to ensure that species of special concern do not become endangered or threatened. The SARA is administered primarily by Environment Canada.

Application of the SARA is not entirely clear, and there are currently no specific approvals or authorizations that have to be sought under the Act. The Act may apply on provincial or private land if “cooperative stewardship” does not succeed, or if provincial legislation does not effectively protect the species, its residence or its critical habitat. Additional information on the SARA can be found at http://www.speciesatrisk.gc.ca/default_e.cfm.

Information that must accompany applications for authorizations and approvals under these acts, where required, is similar to that required for compliance with provincial regulations.

By their nature, infiltration galleries are a relatively benign method of diverting water with respect to the public interests that these acts are intended to protect. Nevertheless, construction activities associated with the development of an infiltration gallery will, at least temporarily, be cause for referral to these agencies.

8.0 INSTALLATION/CONSTRUCTION

For bed-mounted galleries, site dewatering can be the most significant challenge facing the contractor. An adequately-sized pump, with one or two backups, is recommended. The required size of pump can be estimated from the field test for determining in-situ hydraulic conductivity of the alluvial material.

An adequately-sized trackhoe/excavator should be used for construction. A bucket width of about 1 m, and a reach of about 8 m is recommended.

Any in-stream work should be timed to coincide with low water levels in the water body in question, and should also be timed to coincide with non-critical times for spawning and migration of fish. Local provincial fish and wildlife personnel, and personnel from Fisheries and Oceans Canada, should be consulted regarding advice on such times.

In-stream sediment control techniques (cofferdams, silt fences, sensitive area isolation) should be utilized to ensure that construction impacts are minimized (see photograph). All machinery should be kept free of grease, oil and mud, and should be re-fuelled at least 30 m away from the water body.
Some gravel-bed streams will have an armour layer of larger-size material on the surface of the stream bed. If this is the case, this material should be carefully harvested and stockpiled so that it can be replaced on the surface of the completed gallery. If there is no armour layer present, it may be prudent to cover the surface of the infiltration gallery with a layer of rip rap to ensure that erosion of the river bed does not expose the collector pipes. Guidance regarding armouring can be found in Reference #20.

9.0 OPERATION/MAINTENANCE

Maintenance of infiltration galleries can be difficult, especially for bed-mounted galleries. It is therefore important to observe some operational guidelines:

- Never exceed the design pumping rate for the gallery. Higher pumping rates may cause fine material to enter the filter pack, thereby reducing its permeability. Over-pumping may also result in the migration of sand into the collector pipe and blockage of the openings in the collector pipe. Blockage of openings results in higher entrance velocities in the remaining openings, and the increased likelihood of more blockage and migration of sand into the collector.
- Do not let the collector pipes become aerated because free oxygen promotes the growth of iron bacteria and deposition of inorganic material like magnesium and calcium. These deposits and encrustations can clog the collector screens. To ensure that the gallery doesn’t become aerated, it would be prudent to install a low-level cut-out switch (set to a level about 300 mm above the crown of the collector pipes) as part of the pump control system.

Because backwashing activities can stir up sediment in the stream, they should be timed to coincide with non-critical periods for spawning and migration of fish. These non-critical periods would be the same as those identified for the construction phase.

Air and water used for backflushing should be introduced to the backflush system slowly. The pumps and blowers that deliver air and water to the backwash diffusers should be started against closed valves, and the valves should be opened slowly (over about a period of about a minute) so that the backwash flows do not “break through” the overlying filter media. If water and air cannot be introduced to the system simultaneously, the water backwash should be started first, and it should finish last. Also, in situations where the gallery is backwashed in sections, backwashing should proceed in an upstream-to-downstream direction to avoid drawing fine material that has been washed into suspension from being drawn into a newly-washed section of the gallery.

Maintenance to remove the build-up of iron bacteria and/or mineral deposits would be similar to physical, chemical and electrical treatments for wells (see Reference #6 for further discussion). However, great care has to be exercised to ensure that harmful chemicals are not released into the aquatic environment. With respect to physical activities relating to maintenance (rodding/scraping, jetting, etc.), provision for such activities should be incorporated in the physical layout of the piping the comprises the infiltration gallery by incorporating clean-out access ports and minimizing the number of bends and angles in the piping system.
REFERENCES


6) Driscoll, F. G. (ed.), *Groundwater and Wells*, 2nd Johnson Division, St. Paul, Minn.


## GLOSSARY

<table>
<thead>
<tr>
<th>Term</th>
<th>Definition</th>
</tr>
</thead>
<tbody>
<tr>
<td>Aggradation</td>
<td>The geologic process by which stream beds, floodplains, and the bottoms of other water bodies are raised in elevation by the deposition of material eroded and transported from other areas (opposite of degradation).</td>
</tr>
<tr>
<td>Alluvial</td>
<td>Related to deposits of sediments left by rivers.</td>
</tr>
<tr>
<td>Alluvium</td>
<td>A general term for clay, silt, sand, gravel, or similar unconsolidated detrital material, deposited during comparatively recent geologic time by a stream or other body of running water, as a sorted or semisorted sediment in the bed of the stream or on its floodplain or delta.</td>
</tr>
<tr>
<td>Anchor Ice</td>
<td>Submerged frazil ice attached or anchored to the river bottom, irrespective of its formation.</td>
</tr>
<tr>
<td>Backwash</td>
<td>The process of cleaning particulate matter out of filter media by reversing the flow of water through the filter.</td>
</tr>
<tr>
<td>Biofouling</td>
<td>The impairment or degradation of something, such as mechanical equipment, as a result of the growth or activity of living organisms. The undesirable deposition of biofilms that exceeds a given threshold of interference.</td>
</tr>
<tr>
<td>Biofilm</td>
<td>A film of microorganisms attached to a surface, such as that on a trickling filter, rotating biological contactor, a well screen, or rocks in natural streams.</td>
</tr>
<tr>
<td>Buckling</td>
<td>A sudden out-of-plane deformation of slender members or thin-walled structures under compressive loading.</td>
</tr>
<tr>
<td>Colluvium</td>
<td>A general term applied to any unconsolidated sediment deposited by rain-wash, sheet-wash, slope failure, or slow continuous down-slope creep, usually collecting at the base of slopes or hillsides.</td>
</tr>
<tr>
<td>Crossing</td>
<td>A shallow area between two consecutive bends in a river or stream.</td>
</tr>
<tr>
<td>Degradation</td>
<td>A gradual wearing down or wasting, as of stream beds and banks, by the action of water, frost etc.; lowering of a river bed by erosion.</td>
</tr>
<tr>
<td>Diffuser</td>
<td>A device used to evenly distribute something (light, sound waves, fluids, etc.). In the context of infiltration galleries, diffusers are required to evenly distribute air and water for backwashing.</td>
</tr>
<tr>
<td>Diversion</td>
<td>The direction of water in a stream away from its natural course (i.e., as in a diversion that removes water from a stream for human use).</td>
</tr>
<tr>
<td>Dynamic Viscosity</td>
<td>The force, in newtons, required to move a fluid layer of one square meter area and a thickness of one meter with a velocity of one meter per second.</td>
</tr>
<tr>
<td>Empirical</td>
<td>Determined from experiment and observation rather than theory.</td>
</tr>
<tr>
<td>Fracking</td>
<td>Colloquial term in the petroleum industry for creating fractures in rock near the bottom of a well. It usually requires pumping a fluid into the well at high pressure. In the context of horizontal directional drilling beneath a water body, fraccing out would be related to the concept of the relatively high-pressure drilling fluid fracturing the overlying alluvium and creating pathways through which the drilling fluid could escape into the overlying water body.</td>
</tr>
<tr>
<td>Frazil Ice</td>
<td>Fine spicules, plates, or discoids of ice suspended in water. In rivers and lakes, frazil ice is formed in supercooled, turbulent water.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
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<tr>
<td>Geogrid</td>
<td>Net-shaped synthetic polymer-coated fibers.</td>
</tr>
<tr>
<td>Geotextile</td>
<td>A fabric or synthetic material placed between the soil and a pipe, gabion, or retaining wall: to enhance water movement and retard soil movement, and as a blanket to add reinforcement and separation.</td>
</tr>
<tr>
<td>Hydraulic conductivity</td>
<td>A measure of the ability of a fluid to flow through a porous medium determined by the size and shape of the pore spaces in the medium and their degree of interconnection, and also by the viscosity of the fluid. Hydraulic conductivity can be expressed as the volume of fluid that will move in unit time under a unit hydraulic gradient through a unit area measured at right angles to the direction of flow.</td>
</tr>
<tr>
<td>Hydraulic Gradient</td>
<td>The difference in hydraulic head between two measuring points within a porous medium, divided by the distance along the flow path between the two points.</td>
</tr>
<tr>
<td>Hydraulic Head</td>
<td>The pressure exerted by a fluid at a given depth beneath a surface. It is proportional to the height of the fluid's surface above the area where the pressure is measured.</td>
</tr>
<tr>
<td>Hydrometric</td>
<td>Of or pertaining to measurement of the velocity, discharge, etc., of running water, and/or its temporal distribution.</td>
</tr>
<tr>
<td>Hydrostatic</td>
<td>Of or relating to fluids at rest or the forces exerted by such fluids.</td>
</tr>
<tr>
<td>Infiltration</td>
<td>The movement of water or solutions into or through a rock or soil through its openings or pore spaces or fractures within the soil or rock.</td>
</tr>
<tr>
<td>In situ</td>
<td>In its natural or original position or place; in geology, of a rock, soil, or fossil, when in the situation in which it was originally formed or deposited.</td>
</tr>
<tr>
<td>Kinematic Viscosity</td>
<td>A coefficient defined as the ratio of the dynamic viscosity of a fluid to its density.</td>
</tr>
<tr>
<td>Morphology</td>
<td>Branch of geomorphology concerned with the study of river and stream beds and their ongoing changes due to erosion and sedimentation.</td>
</tr>
<tr>
<td>Newton</td>
<td>The newton (N) is the basic metric unit of force, named in honour of Sir Isaac Newton (1642-1727), who developed the laws of motion and gravitation. One newton is a mass of 1 kilogram times an acceleration of 1 meter per second (sec) per second (1 N = 1 kg m/sec²).</td>
</tr>
<tr>
<td>Permeability</td>
<td>A measure of the rate at which a substrate can pass water, the rate depending on substrate composition and compaction; the apparent velocity per unit of hydraulic gradient.</td>
</tr>
<tr>
<td>Pools and Riffles</td>
<td>The naturally undulating profile of most streams, formed by coarse materials that accumulate on stream beds at intervals. Upstream from the accumulations, a shallow pool is impounded. Downstream from the crest of the accumulation, a local increase in slope causes the flow to accelerate, forming a riffle or rapids.</td>
</tr>
<tr>
<td>Porosity</td>
<td>A measure of the water-bearing capacity of subsurface rock. The ratio of pore volume to total volume of a filter medium expressed as a percent.</td>
</tr>
<tr>
<td>Siting</td>
<td>The process of choosing a location for a facility.</td>
</tr>
<tr>
<td>Surficial Deposits</td>
<td>Includes all sediments above the bedrock.</td>
</tr>
<tr>
<td>Term</td>
<td>Definition</td>
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<td>---------------------------------------------------------------------------</td>
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<tr>
<td>Thalweg</td>
<td>The line connecting the lowest points along a stream bed or valley; <em>longitudinal profile</em></td>
</tr>
<tr>
<td>Till</td>
<td>A sediment deposited directly by a glacier. The deposit is unsorted and consists of any grain size ranging from clay to boulders</td>
</tr>
<tr>
<td>Vortex</td>
<td>A mass of fluid, especially of a liquid, having a whirling or circular motion tending to form a cavity or vacuum in the center of the circle, and to draw in towards the center bodies subject to its action; the form assumed by a fluid in such motion; a whirlpool; an eddy.</td>
</tr>
<tr>
<td>Vorticity</td>
<td>A vector measure of local rotation in a fluid flow.</td>
</tr>
</tbody>
</table>
LIST OF SYMBOLS

$A$ - area of infiltration gallery
$A_p$ - cross-sectional area of diffuser port
$A_r$ - ratio of the cross-sectional area of flow emerging from diffuser port to cross-sectional area of diffuser pipe
$C_d$ - discharge coefficient of port
$d$ - burial depth of collector screens
$d_e$ - effective grain size of granular material
$d_{eq}$ - equivalent spherical diameter of filter media grains
$d_p$ - diameter of diffuser port
$D_b$ - diameter of belled-end of suction intake on pump
$D_h$ - optimal diffuser diameter
$D_{min}$ - minimum diameter of collector pipe
$D$ - diameter of diffuser or collector screen
$e$ - height of roughness element on pipe wall
$f$ - Darcy friction coefficient
$F_s$ - submergence Froude number
$g$ - gravitational acceleration
$Ga$ - Galileo number
$H$ - submergence of collector screen
$\Delta H$ - total head difference between water backwash header and stream water surface
$\Delta H_p$ - head difference across diffuser port
$\Delta H_d$ - head difference required to induce backwash water flow through overlying media at selected superficial velocity
$K$ - hydraulic conductivity

$L_s$ - length of individual collector screen and/or diffuser
$L_{st}$ - total length of collector screen and/or diffuser
$n$ - porosity of porous media; number of collector pipes
$n_d$ - number of diffuser pipes
$n_p$ - number of ports in diffuser
$P$ - external pressure on pipe
$q_p$ - discharge through diffuser port
$Q$ - total required diversion flow, or portion thereof, that is to flow in collector pipe
$Q_a$ - air flow rate
$Q_d$ - flow in diffuser pipe
$\Re$ - Reynolds number
$s$ - distance of belled-end of pump intake below water surface
$s_d$ - spacing of diffuser pipes
$V$ - superficial water velocity within porous media
$V_{mf}$ - bed fluidization velocity
$\gamma_d$ - pressure recovery coefficient
$\gamma_s$ - saturated unit weight of alluvium
$\gamma_w$ - unit weight of water
$\eta$ - uniformity coefficient
$\varphi(n)$ - function of porosity $n$
$\mu$ - absolute viscosity of water
$\nu$ - kinematic viscosity of water
$\rho$ - mass density of water
$\rho_s$ - mass density of filter particles