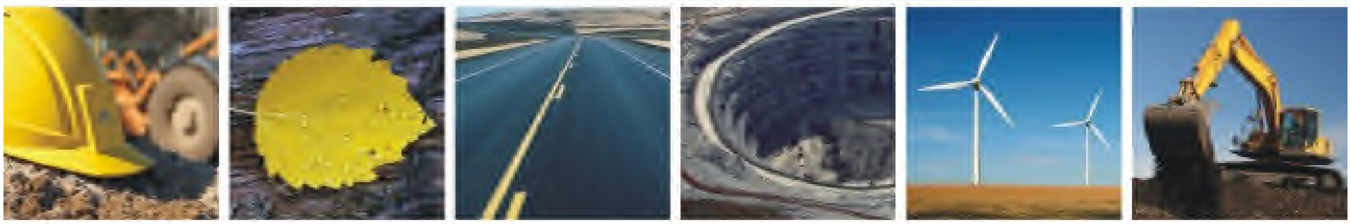
THE CITY OF CALGARY

LOW IMPACT DEVELOPMENT PROJECT
MODULE 1 – GEOTECHNICAL AND HYDROGEOLOGICAL CONSIDERATIONS



REPORT

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ACRONYMS & ABBREVIATIONS

Acronyms

AESRD	Alberta Environment and Sustainable Resources Development
ASP	Area Structure Plan
ASTM	American Society of Testing and Materials
CP	Community Plan
DSSP	Development Site Servicing Plan
LID	Low Impact Development
MDP	Master Drainage Plan
SCP	Source Control Practice
SMDP	Staged Master Drainage Plan
SWMR	Storm Water Management Report
WMP	Water Management Plan
WP	Water shed Plan
WPAC	Watershed Plan Advisory Council

Symbols

A	The area usually referred to as the cross-sectional area of flow (length by length)
i, i_v,i_h	Hydraulic gradient (length/length) with subscripts v and h identifying the vertical and horizontal directions respectively
I,P	Infiltration (I) or percolation (P) rate (typically expressed as length/time but actually represents a volumetric rate (volume/time/unit area) or as a loading rate – design infiltration rate
K, K_v, K_h	saturated hydraulic conductivity (length/time) with v and h identifying the vertical and horizontal directions respectively
K_{fs}	hydraulic conductivity measured from infiltration tests and since under field conditions where the pore space is not entirely filled with water is denoted as fs for field saturation representing saturation achieved merely for the test condition
K_{sat}	saturated hydraulic conductivity derived by multiplying the K_{fs} value by a factor of 2 but is identical in terminology in use to K above
q	Specific discharge (length/time) is a volumetric flow rate per unit time per unit area (volume/time/unit area) sometimes used interchangeable with infiltration or percolation rates or as a loading rate

Q Volumetric flow rate (volume/time) used for both estimates of well yields or capacity and flow through an aquifer

R Recharge rate (length/time) used to identify the natural recharge rate

v_l The average linear groundwater velocity (length/time)

1.0 INTRODUCTION

1.1 Background and Objectives

This document provides technical guidance on the geotechnical and hydrogeological considerations to be integrated into the development of stormwater drainage plans and the implementation of stormwater drainage works during development of residential and commercial properties within The City of Calgary (The City). These considerations are part of The City's Low Impact Development (LID) initiative. The objectives of the initiative are:

- To prevent flood damage to watersheds within The City;
- To improve watershed health;
- To prevent further stream deterioration; and
- To facilitate sustainable growth.

The integration of geotechnical and hydrogeological considerations into LIDs for stormwater management is discussed in Sections 2 and 3:

- Geotechnical Hydrogeological Investigation to Support Stormwater Management Plans; and
- Geotechnical and Hydrogeological Investigation during Implementation of Stormwater Management Features.

1.2 Low Impact Development Definition

An LID is a development (residential or commercial) that minimizes the impact of stormwater on watersheds by integration of measures to detain, retain and treat stormwater using soil infiltration and percolation to redirect a portion of the stormwater back into the hydrologic cycle.

1.3 Application and Function of Technical Protocols and Guidance Documents

This module was developed between April 1, 2011, and December 12, 2012. The information provided here has been extracted from research on geotechnical and hydrogeological investigative techniques, both current and under development, by university researchers and provincial and state regulatory agencies across North America and Australia. The protocols and supplementary guidance documents appended to this document incorporate this research.

Each of the appended technical protocols and guidance documents also illustrates the geotechnical and hydrogeological calculations needed to estimate groundwater flow rates and direction, characterize soil and hydraulic properties, and estimate infiltration and percolation rates. The protocols and guidance are in the following general format:

- Introduction;
  - A statement of the intent and format of the protocol and guidance.

- Background;
  - A summary of the current best management practices or research underway and a statement of the suggested approaches that are well suited to The City's geologic, hydrogeologic, and climatic setting.
- Method;
  - A detailed description of how to make the measurements of soil and groundwater parameters or a calculation required by the protocol.
- Worked Example;
  - A worked example of the calculations needed to estimate the soil or hydraulic properties.

The appended technical protocols and guidance documents include:

- Appendix A – Methods to Estimate Groundwater Flow Rates and Directions;
  - This appendix describes methods to estimate the hydraulic gradient and the hydraulic conductivity.
- Appendix B – Methods to Estimate Infiltration and Percolation Rates;
  - This appendix describes methods to estimate infiltration and percolation during the site assessment, stormwater management feature selection, detailed design, and post-construction care of a development and provides guidance on when and where particular methods apply.
- Appendix C – Methods to Characterize Soil Conditions;
  - This appendix describes methods to sample and measure soil properties.
- Appendix D – Characterizing Probable Hydrogeologic Consequences;
  - This appendix describes methods to evaluate the potential impact stormwater management has on local groundwater resources (wetlands and marshes).
- Appendix E – Framework to Develop a Conceptual Site Model;
  - This appendix describes the generation of a conceptual site model to show the relationship between groundwater flow, stormwater management features, and the interaction with surface water resource.
- Appendix F – Methods to Evaluate Subgrade and Slope Stability;
  - This appendix describes the methods of geotechnical investigation and soil property testing needed to support the design of stormwater management features.
- Appendix G – Methods to Evaluate the Consequence of Groundwater Mounding Beneath Infiltration Basins;
  - This appendix describes the calculations needed to estimate the height and lateral spread of a water table mound located beneath a stormwater management feature used for infiltration and the potential interaction between multiple stormwater management features. This method is also an

applicable tool to evaluate the potential impact on down-gradient infrastructure, setbacks from slopes, and inflow to sanitary sewers or other buried utility corridors.

- Appendix H – Indirect Methods for Estimating Hydraulic Conductivity and Infiltration Rates;
  - This appendix describes indirect methods useful for estimating hydraulic conductivity and infiltration rates.

For ease of reference, Table 1.1 summarizes the geotechnical and hydrogeological parameter measurement methods contained in these appendices.

Table 1.1: Index to Geotechnical and Hydrogeological Measurements and Calculations

Section 1		
Appendix No.	**Description/Title of Protocol or Guidance Document**	**Geotechnical and Hydrogeological Provided Parameters**
A	Methods to Estimate Groundwater Flow Rates and Directions	Hydraulic gradient and hydraulic conductivity
B	Methods to Estimate Infiltration and Percolation Rates	Methods to estimate surface infiltration and percolation rates
C	Methods to Characterize Soil Conditions	Soil types and texture, physical properties
D	Characterizing Probable Hydrogeologic Consequences	Estimation of the impact of infiltration surplus or loss of recharge to groundwater on down-gradient water resources
E	Framework to Develop a Conceptual Site Model	Relationship of groundwater within a planning area to surface water and off-site water resources
F	Methods to Evaluate Subgrade and Slope Stability	Soil types and texture, physical properties
G	Methods to Evaluate the Consequence of Groundwater Mounding Beneath Infiltration Basins	Calculation of the build-up of a water table mound height and lateral extent from a single infiltration and multiple infiltration sites
H	Indirect Methods for Estimating Hydraulic Conductivity and Infiltration Rates	Indirect methods useful for estimating hydraulic conductivity and infiltration rates

2.0 GEOTECHNICAL AND HYDROGEOLOGIC INVESTIGATIONS TO SUPPORT STORMWATER MANAGEMENT PLANS

Geotechnical and hydrogeological conditions need to be considered differently depending upon the level of stormwater management plan being prepared.

Sections 2.1 to 2.5 describe the geotechnical and hydrogeological factors to be considered in each type of plan and the level of detail a developer should provide. These geotechnical and hydrogeological considerations are described according to:

- The authorship and function of the plan;
- Technical requirements comprising;

- The geotechnical and hydrogeological parameters required to support the drainage plan;
- The function and need for parameters;
- The parameters to be measured; and
- The reporting requirements.

The planning levels described here are:

- Watershed Plans **(WP)** or Water Management Plans **(WMP)**;
- Master Drainage Plans **(MDP)**;
- Staged Master Drainage Plans **(SMDP)**;
- Pond Reports; and
- Stormwater Management Reports both for subdivision plans **(SWMR)** and Private Development Site Servicing Plans **(DSSP)**.

While WP or WMP and MDP planning levels provide the conceptual framework, in terms of overall distribution of soil types, groundwater flow and patterns of recharge within a watershed, an SMDP is an intermediate level planning document. These types of plans require subsurface investigation and site assessment to allow the feasibility of constructing stormwater management features to be fully appreciated. Traditionally, these plans have excluded subsurface investigation, but subsurface information will help the developer in developing more effective plans for the management of stormwater.

Pond Reports must include the detailed engineered design whether the pond is to be a dry pond, wet pond, or a wetland used to detain or retain stormwater. Pond Reports must contain the details of the subsurface investigations that characterizes the geotechnical and hydrogeologic conditions used to support the pond's design. The Pond Report requires more detailed site information, particularly of a geotechnical nature, because it needs to manage and control more intense and potentially highly variable rainfall events. Stormwater management reporting falls into two categories: those reports completed to support a subdivision development, the SWMR, and those site servicing plans completed for private development, the DSSP. The SWMR or DSSP document requires a more detailed consideration of the capacity of a proponent's plan to influence and control the impact of the stormwater management features and must correspond to those measures provided within the WP, WMP, and MDP, reports to protect watershed health and water resources sustainability. As a general guide in preparing geotechnical and hydrogeologic information for each drainage plan, Figure 2.1 shows the relevant issues at the drainage planning level and the general scope of activity recommended.

Table 2.1 summarizes these considerations for each planning level. In circumstances where the higher level planning document does not exist, the developer may need to provide this framework before proceeding at the planning level of the submission. Figure 2.2 shows where the geotechnical and hydrogeologic investigation are needed to support other elements of a development plan.

Table 2.1: Summary of the Geotechnical and Hydrogeological Requirements for Drainage Plans

Planning Level	Authorship	Geotechnical and Hydrogeological Factor	Purpose	Parameters Generated	Reporting Requirements
Watershed Plan or Water Management Plan	Province and/or City, and/or adjacent municipality	Surficial geology and soil type	To obtain subsurface information and estimates of potential infiltration and variations in infiltration within a watershed To identify geological hazards	Infiltration rate (*I*)	Map with areas of potential shallow groundwater and surface water flow directions (Appendix A)
		Topography and hydrogeology	To determine direction of groundwater flow	Hydraulic gradient (*i*) (direction and slope)	Map of surficial materials and estimate of infiltration potential (Appendix H)
			To identify wetlands and marshes	Volume and size of wetland and potential area affected by wetland infiltration	Classification of the wetland - Map of drainage courses, wetlands and marshes (Appendix B)
			To identify area of potential groundwater and surface water interaction	Tendency for vertical movement of groundwater	Map of recharge and discharge areas
			To identify areas of groundwater recharge and discharge		
		Characterization of local and regional aquifers	To evaluate potential impact on water supplies and to local and regional aquifers	Q – potential well yields and quantity of lateral groundwater flow	Cross-sections of the subsurface stratigraphy to illustrate aquifer and aquatic resources and to calculate flow rates needed to support a waterbody within the study area, see Appendices A and C for groundwater flow and soil types and Appendix D for estimates of the impact on local water resources

Table 2.1: Summary of the Geotechnical and Hydrogeological Requirements for Drainage Plans (Continued)

Planning Level	Authorship	Geotechnical and Hydrogeological Factor	Purpose	Parameters Generated*	Reporting Requirements
Master Drainage Plan	Qualified Consultant in Geotechnical Engineering and Hydrogeology	Interaction between groundwater and surface water	To preserve slope stability To protect wetland and marsh habitat	Soil lithology and direction of groundwater flow	Geological hazards – geotechnical appraisal (Appendix H). A conceptual site model (CSM) (Appendix E) Borehole logs; groundwater flow direction (Appendices A and C)
		Direction and rate of groundwater flow in the saturated and unsaturated zone	To ensure local aquifers are protected	K_n – horizontal hydraulic conductivity	Calculation methods and tabulation of results (Appendix A)
			To aid in selecting sites for stormwater management features	i_n – horizontal hydraulic gradient i_v – regional hydraulic gradient	Cross-sections and flow net (Appendix A)
		Permeability/hydraulic conductivity	To estimate the potential infiltration surplus or deficit	K_v – I,P,R vertical hydraulic conductivity, infiltration rate, percolation rate, and natural recharge rate Consequences of the infiltration surplus or deficit on the developments water resources (wetlands, watercourses, and aquifers)	Estimates of the infiltration surplus/deficit (Appendix D) Tabulation of percolation test results, estimates of the area needed to mimic the predevelopment condition. Estimates of potential for groundwater monitoring and the consequence to surface pondings, roadways, utility corridors, and nearby sideslopes (Appendix B).

Table 2.1: Summary of the Geotechnical and Hydrogeological Requirements for Drainage Plans (Continued)

Planning Level	Authorship	Geotechnical and Hydrogeological Factor	Purpose	Parameters Generated*	Reporting Requirements
Master Drainage Plan (continued)		Baseline water chemistry	To assess water quality provided to wetlands, marshes, and watersheds	Development footprint of porous and non-porous areas	Consequences of the infiltration surplus/deficit on the developments water resources chemical quality (wetlands, watercourses, and aquifers) (Appendix D)
				Chemical baseline water quality	Tabulation of water quality data
Staged Master Drainage Plan	Qualified Consultant in Geotechnical Engineering and Hydrogeology	Geotechnical setting of retention/detention features and SCPs Potential impacts to the water table and to off-site water resources	To assess subsurface conditions and soil types To assess constructability of ponds and source water control features To assess slope stability near waterbodies	Soil design parameters, soil gradation curves and soil strength properties	Geotechnical report including borehole logs, laboratory test results, and slope stability assessment input to the overall water balance for the study area (Appendix F)
			To estimate the infiltration surplus/deficit and estimate the impact on the water table due to SCPs	▪ Subsurface stratigraphic materials with the potential for developing a perched water table ▪ Surface topography post-construction ▪ Depths to the water table ▪ K_h and K_v	Height of the water table build-up and extent of spreading on the water table (Appendix G)

Table 2.1: Summary of the Geotechnical and Hydrogeological Requirements for Drainage Plans (Continued)

Planning Level	Authorship	Geotechnical and Hydrogeological Factor	Purpose	Parameters Generated*	Reporting Requirements
Pond Report	Qualified Consultant in Geotechnical Engineering and Hydrogeology	Detailed design and assessment input	To verify water retention capacity and ensure sideslope stability due to rapid changes in water level To understand the potential impact of mounding on down-gradient roadways, sanitary system utility corridors, or ponding on the ground surface	▪ Slope stability assessment and safety factors ▪ Infiltration rates and extent and spread of groundwater mound beneath infiltration basin	▪ Detailed design based upon geotechnical properties and slope stability assessment ▪ Estimates of the potential for adverse consequences of mounding or surface ponding (breakout on sideslopes, flow into sanitary system and utility corridors and beneath roadway) ▪ Advice on inspection schedule and mitigation if necessary
Stormwater Management Report	Qualified Consultant in Geotechnical Engineering and Hydrogeology	Detailed design and impact assessment	To confirm location for SCPs and optimizing the design to meet target runoff volumes, and/or other objectives	▪ Soil strength properties ▪ Slope stability and safety fact ▪ Liners design thickness and hydraulic conductivity or design specification for synthetic material ▪ Impact of water table management ▪ Subsurface drainage requirement	▪ Detailed design drawing suitable for construction ▪ Design hydraulic conductivity values of liners and drains ▪ Impact assessment on the water table or perched water table ▪ Guidance in liner protection ▪ Drain design material specification for width and depth

2.1 Watershed Plans or Water Management Plan

A WP or WMP provides a conceptual framework of the physical attributes (surficial geology, hydrology, and hydrogeology) of a watershed or sub-watershed that supports drainage servicing of a development area.

2.1.1 Authorship and Function

A WP is typically prepared by either the Bow River Council (as the WPAC for the Bow River) or local watershed group, Alberta Environment and Sustainable Resources Development (AESRD), and/or The City and adjacent municipalities. A WP provides the conceptual framework of the physical attributes of a drainage basin. A typical hydrological assessment provides an inventory of the physical framework (i.e., the hydrogeological, geological, and geomorphological setting) of a watershed. This framework is used at the subsequent MDP level to aid the municipality in identifying geotechnical and hydrogeological constraints on stormwater management that may exist from place to place within a proposed development property. Those constraints are identified by preparation of a hydrogeologic assessment describing the relationship (recharge or discharge) between shallow groundwater and surface water resources from place to place across the watershed. Apart from the geomorphological considerations, little geotechnical input is required for this level of the drainage planning.

From a geotechnical and hydrogeological perspective, the watershed plan must describe and identify the physical relationship between groundwater and surface water, the potential geologic hazards (unstable slopes), and the uses of surface and groundwater (both as a water supply and as a habitat) that can potentially be affected by developments within a watershed.

The hydrogeologic assessment is an important environmental planning tool that aids The City in deciding the merits of urban development planning relative to the potential impacts on the water resources within a watershed. These impacts can be managed through the appropriate selection of stormwater management features but to support the selection process, information on the physical framework of the watershed needs to be incorporated into the growth planning.

2.1.2 Technical Requirements

The technical requirements of the investigation of the physical framework of a watershed plan help to decide the scope of a site assessment and to identify appropriate stormwater management features at the MDP, SMDP, and SWMP planning levels.

These physical framework elements include:

- The distribution of surficial geological materials and soil types within the watershed;
- The topography and the hydrology (surface watercourses, wetlands, and marshes);
- The characteristics of regional and local aquifers; and
- The geotechnical hazard areas.

Because the WP or WMP provides a conceptual framework, these plans are developed using available published maps, plans, and files maintained by government agencies. The geotechnical and hydrogeological information obtained to support the WP or WMP is essentially a reconnaissance or desktop study but may be supported by field inspection to support some regional mapping findings. In the Calgary area, the information sources that might be referenced include but are not necessarily limited to:

- Moran, S. 1986. Geology of the Quaternary Sediments in the Calgary Urban Area, Bulletin No. 53, Alberta Research Council;
- MacMillan, R.S. 1987. Soil Survey of the Calgary Urban Permeameter, Terrain Science Department, Bulletin No. 54, Alberta Research Council; and
- Water well records from the files and water well database maintained by Alberta Environment and Water (AEW) http://www.envinfo.gov.ab.ca/Groundwater/.

A municipality with the intent of following the principles for LID needs to evaluate the distribution of surficial geologic materials and soil types within a watershed because these materials influence the amount of infiltration to the subsurface that potentially occurs within a development. Natural infiltration supplies groundwater recharge to regional and local aquifers and ultimately the base flow supplied to local surface watercourses and wetland habitats. Surplus infiltration may increase the level of the groundwater table or create perched groundwater tables and have adverse consequences that include ponding of water on the surface, inflow to buried utility corridors, affect the stability of roadways and topographic slopes. A deficit in infiltration may adversely affect the water supply available to local watercourses and wetlands, and deteriorate the viability of marsh habitats used by aquatic and terrestrial wildlife. Conversely, excess runoff threatens to cause deterioration of a watershed due to excessive erosion. Target runoff volumes to reduce the potential for excess erosion can be met using the infiltration and percolation capacity of the soil below stormwater retention and detention infrastructure used as infiltration basins.

The topography and hydrology of a watershed aids in defining the potential direction of shallow groundwater flow and locations where surface water and groundwater interact. For a municipality, the direction of groundwater flow defines where, within a development, stormwater management features are best placed to minimize impact on the water balance. Also, zones of groundwater discharge are areas where the water table is close to the ground surfaces and thus are areas where stormwater management features may be impractical due to construction difficulties. In these areas, a shallow water table that is saturated by water held in the pore spaces by tension do not provide enough capacity for infiltration basins to be effective for replacing the infiltration deficits or to aid in meeting runoff target volumes. Such areas are prone to poor soil stability without exceptional construction measures such as dewatering or shoring to support weak slopes. In such areas; however, rain gardens and absorbent landscapes that have a primary function to retain or detain stormwater are more realistic control practices than infiltration basins which have the primary function of using the subsurface capacity of the soil to meet target runoff volumes by infiltration or percolation.

Characterization of regional and local aquifers, determined from published maps and from the interpreted water well records, provides a municipality with knowledge about how the removal of infiltration water by urban development potentially affects the quantity of groundwater available to supply groundwater discharge to local streams and watercourses.

This characterization also helps to decide where the quality of stormwater that is allowed to infiltrate through the bases of bioswales, absorbent landscapes, rain gardens, or infiltration basins may be an issue for protection of an aquatic habitat.

Identifying the geological hazardous areas from a slope stability perspective is important to determine the appropriate location of stormwater control measures and eliminating areas of slope instabilities.

2.1.3 Reporting Requirement

The hydrogeological assessment provided to support the WP needs to have the following elements:

- Maps of the topography annotated to illustrate:
  - The probable direction and rate of groundwater flow (Appendix A provides advice on how these rates can be estimated at the reconnaissance or desktop level of watershed planning);
  - Watercourses, wetlands, and marshes;
  - Recharge and discharge areas; and
  - Area of potential groundwater and surface water interaction.
- Maps of the surficial materials and the soil type (and the potential infiltration rate) by soil type are available from a variety of published sources that can be adopted at this desktop study level for use by municipalities. The technical protocol in Appendix H provides estimates from monographs and charts to enable typical infiltration rates to be estimated for the soil type for surficial mapping purposes;
- Cross-sections showing the distribution and depth and potential zone of interaction between regional and local aquifers and surface waterbodies; and
- Areas of geological hazard or geotechnical instability of slopes.

2.2 Master Drainage Plan

An MDP covers a larger area within a watershed than either an SMDP or an SMWP. MDPs are prepared by the Water Resources Business Unit of The City or on behalf of The City by qualified consultants. The MDP is used to support Area Structure Plans (ASPs) and Community Plans (CPs). Consequently, it may encompass multiple development sites or subdivisions. The MDP conceptually may include one or more outfalls for release of stormwater to the off-site catchment areas, and similarly, it may contain one or more or even a combination of stormwater management practices to ensure watershed deterioration does not occur and that water is returned to an appropriate location in the hydrogeologic system. The influence of regional geologic and hydrogeologic conditions should play an important role in developing the MDP and, therefore, developing the drainage plan must encompass the findings and recommendations of the desktop hydrogeologic assessment undertaken for the WMP. From a geotechnical and hydrogeologic perspective, subsurface investigations are required to supplement the hydrogeologic assessment and to provide the most effective management options to satisfy goals of urban water management. These investigations also provide the necessary information required to minimize the adverse effects of less effective stormwater management, such as the loss of slope stability, deterioration of watercourses, and loss of sustainable recharge to aquifers and base flow to local watercourses, wetlands, and marshes.

2.2.1 Authorship and Function

The geotechnical and hydrogeological input to an MDP must be provided by a qualified professional geotechnical engineer and a hydrogeologist. The function of the geotechnical and hydrogeological information provided to an MDP is to build upon the physical inventory within the WMP by further identifying constraints within the planning area (not necessarily restricted to surveyed boundaries) that influence the type of stormwater practices and measures that will be most effective at managing stormwater. It is suggested that subsurface geotechnical and hydrogeological investigations be undertaken in phases. The first phase - the preliminary evaluation - shall be conducted at the MDP level.

2.2.2 Technical Requirements

Technically, the MDP must build upon the findings of the hydrogeologic and geotechnical assessment generated by the desktop study's review of surficial geology and soils, topography and hydrology, and the regional and local aquifers characterized as part of the hydrogeologic assessment and its inventory of physical attributes within the watershed compiled within the WMP. During the MDP preparation, it is necessary to confirm the geotechnical and hydrogeological attributes defined by the WMP to enable strategies to be identified that provide an acceptable level of service to the development. These attributes are:

- Characterizing the interaction between groundwater and surface water along watercourses and in the vicinity of wetlands and marshes to assess slope stability and aid in protecting habitats from loss of groundwater discharge/recharge or receipt of poor quality surface drainage.
- Assessing slope stability issues and the potential impact of declining water levels on aquatic habitats after the methods provided in Appendices D and F.
- Determining the direction and rate of groundwater flow (hydraulic gradient and hydraulic conductivity) to ensure recharge to the watershed and local aquifers is maintained and to aid in selecting locations for SCPs. Appendix A describes methods to determine groundwater flow rates and direction.
- Evaluating the potential hydraulic conductivity of the soil above the water table (infiltration rates and percolation rates, i.e., the vertical hydraulic conductivity). Appendix B describes methods to estimate the infiltration and percolation.
- Assessing the baseline ground water chemical quality.

At this planning level, the geotechnical and hydrogeologic investigation needed to evaluate the attributes in support of an MDP and the level of detail includes:

1. Preliminary Geotechnical Investigation of the Soil Types and Lithology;

   The geotechnical requirements for a MDP can be satisfied within the traditional geotechnical investigation required for buildings or roadways within a development. Consequently, the MDP requires no further subsurface investigations beyond that used for the assessment of building foundations.

A subsurface investigation using a portable auger-type drilling machine is required with the following guidance provided:

- Drilling a network of testholes for foundation purposes frequently involves testholes placed at approximately 200 m centres to evaluate the potential variation in soil conditions within the development. A minimum depth of 15 m is suggested but may vary, depending on the findings of the hydrogeologic assessment of the watershed. In Section 3, Table 3.1, guidance is provided on the level of geotechnical assessment needed depending upon the size of the development. The network of testholes, if candidate locations for SCPs have been identified, can be adjusted to better define soil characteristics near these features. Typically, however, SCP locations are not selected until the SMDP or Pond Reports are required.
- Visual classification of soil types (textures) from soil samples collected during drilling using the Unified Soil Classification System (USCS). A typical geotechnical investigation involves soil samples collected at a minimum of 1.5 m depth intervals or wherever a change in soil type is observed. More frequent measurements at 0.5 m depth intervals are required where moisture contents are a concern, such as the candidate locations of infiltration basins if they are decided at this level of planning. Depending upon the size of the overall development and recognizing that many SCP locations are not decided at the MDP level, the number of geotechnical testholes can comfortably be deferred until needed at the SMDP or Pond Report levels of drainage planning.
- Conducting standard penetration tests (SPTs) to identify zones of changing soil strength and consistency in terms of loose and firm soil. These tests aid in defining low permeability barriers that may inhibit successful infiltration.

Guidance on geotechnical testing methods at this preliminary level is provided in Appendix C.

The outcome of this investigation evaluates the potential for build-up of a perched water table within the unsaturated zone and characterization of the soil units and texture within the upper 15 m of the site, as well as defining characteristics of building foundation and roadway construction.

2. Characterization of the Groundwater Flow System;

   Characterizing the groundwater flow conditions involves measurement of the hydraulic conductivity and hydraulic gradient and defining the soil materials discovered during the subsurface investigation into aquifer and non-aquifer materials. This work includes:

   - Installing at a minimum of three, and preferably five, monitoring wells at the geotechnical testhole locations, and at one location, install a nested monitoring well. More may be needed in areas of complex terrain to best evaluate groundwater flow conditions, but three is the minimum number for this preliminary level of site characterization to measure the depth to the water table and the potential for upward or downward groundwater flow. Depending upon the size and sensitivity of wetlands or marshes determined by the hydrogeologic assessment, additional monitoring wells may be needed to quantify the groundwater discharge or recharge from these wetlands or marshes. Section 3, Table 3.1, provides guidance on the number of monitoring wells needed depending upon the size of the development area. Appendix A provides additional guidance on additional wells needed to assess groundwater flow pattern in complicated terrain.

- Each well should be 5l mm internal diameter (ID) and equipped with a 1.5 m screened interval to facilitate baseline water quality sampling and to undertake monitoring well response testing to determine the hydraulic conductivity. Guidance on determining groundwater flow rate, including the hydraulic gradient and measurement of the hydraulic conductivity from monitoring well response tests, is provided in Appendix A. This appendix shows a schematic of a typical monitoring well installation.
- Water levels should be measured at the wells using a water level tape. Seasonal variations in the water table depth are critical factors in evaluating the potential for water table mounding below stormwater retention facilities designed to return precipitation to the hydrogeologic cycle by infiltration. At the MDP level, it should be appreciated that as candidate locations for SCP are selected, more frequent measurements of water level (for example in response to spring recharge or rainfall events) may be needed. Monitoring wells should be constructed and protected to ensure this long-term need can be satisfied.

Information collected should be used to determine the depth to groundwater across the site. All water level data and measurements of the hydraulic conductivity should be used to evaluate the quantity of groundwater flowing into and off of the study area. Using the footprint of the development cells, if available, the potential infiltration deficit or surplus due to the development should be estimated and quantitative measures to mitigate adverse consequences of the deficit/surplus evaluated (Appendix D). Also, areas of upward or downward groundwater movement need to be identified where seasonal influences on the water levels are most prominent. Ideally, a daily set of monthly depth to water level measures can be collected and compared to monthly events of runoff so a qualitative estimation of the potential shallowest depth of the water table can be evaluated. This data is most useful if candidate locations for SCPs are available. If not yet selected, these data can be obtained during the SMDP or Pond Report planning level.

3. Pre-Development Estimates;

As described in Appendix B, there are a variety of methods to measure infiltration rates. At this stage of planning, it is considered that measuring the surface infiltration is the favoured means because it is envisioned that the natural rate of infiltration will need to be provided post-development. Section 5 identifies methods to mitigate infiltration losses due to construction activities.

To undertake surface infiltration estimates, the methods that are considered most functional include the double ring, the Guelph permeameter, or the Modified Phillips Dunne permeameter method, provided consideration is given to the specific site condition and the pros and cons of each method's application described in Appendix B. Also Consideration must be given to stripping or grading activities on the development such that infiltration rate is measured on those portions of the subgrade that will be present while site preparation activities are complete and be part of the long term performance of the development

At any development, the upper surface of the site to a minimum of 1.5 m will be disturbed. Therefore, at this planning level, percolation tests in the zone between the depth of 1.5 m and the water table shall be conducted. Testing from these depths best represents the capacity of the soil beneath surface water retention ponds, subsurface drainage galleries, porous pavements, and bio-retention facilities to replace the natural infiltration lost by the non-porous areas within the footprint of the development or to meet surface water rainfall target volumes. Guidance on conducting these percolation tests is provided in Appendix B.

2.2.3 Reporting Requirements

A geotechnical and hydrogeologic report to support the MDP shall provide:

- A CSM to illustrate the relationship between surface water retention ponds, and other stormwater management features, the groundwater flow system, and the off-site water resources. (Appendix E describes the level of detail needed in a CSM). The CSM can be used to estimate the hydrogeologic water balance for study area. This balance is a significant contribution to the overall surface water balance.
- Maps of the distribution of soil types and the soil properties measured from geotechnical testing results.
- Maps of the depth to the water table and the direction of shallow groundwater flow, including the presence of upward and downward gradients.
- An estimate of the potential adverse infiltration deficit caused by the development or the excess runoff quantity to be managed based upon comparison of the area of porous and non-porous surface materials within the development to the natural area of infiltration.
- A calculation of the potential infiltration area needed to replace the infiltration deficit as determined from the percolation test results or to provide the capacity to absorb infiltration through the SCPs. Appendix G provides advice on estimating the infiltration area and the infiltration rate that can be achieved without creating adverse effects (ponding on the surface, breakout on sideslopes, interference with road ways, or discharge into utility corridors).
- An evaluation of the baseline ground water chemical quality.
- Geotechnical construction guidance measures to maintain the pre-development soil infiltration capacity following development are discussed in greater detail in Section 3.

2.3 Staged Master Drainage Plan

An SMDP only addresses a portion of the area covered by an MDP within a watershed. It is usually prepared in support of an Outline Plan Submission and, as such, requires sufficient detail to confirm the footprint of a proposed stormwater management pond (if any are proposed). Because the SMDP is often still conceptual in nature, it may not contain the level of detail needed to support construction. However, to confirm the footprint of proposed storm water management facilities and or outline the presence of SCP's in public spaces, it should support the feasibility of construction of SCPs by evaluating the impact of an infiltration surplus/deficit on surrounding water resources and the potential for adverse impact of infiltration needed to meet runoff target volumes. The potential adverse consequences to be considered include:

- Ponding on the ground surface;

- Interference with roadways;
- Breakouts on sideslopes;
- Inflow to sanitary systems; and
- Seepage into utility corridors.

Although the MDP may provide sufficient detail for the SMDP, the SMDP is typically provided as a standalone report by a developer who wishes to proceed with a development in stages. In this circumstance, the general information provided with the MDP will need to be supplemented by the SMDP. However, the SMDP is often still a conceptual document and may not have finalized the site selections for SCPs.

2.3.1 Authorship and Function

The geotechnical and hydrogeologic input to an SMDP should be prepared by a qualified professional geotechnical engineer and a hydrogeologist. The SMDP builds on a recommendation of the MDP. It is suggested that a standalone SMDP contains all of the information identified in Section 2.2 for the MDP.

2.3.2 Technical Requirements

Because the SMDP builds upon the information provided in the MDP, the SMDP primarily focuses on the consequences or impacts of the stormwater management features on the hydrogeologic conditions. During this planning stage, the footprints of the SCP that are used to provide infiltration or percolation to the subsurface are decided as well as their best location on the site needed to meet runoff target volumes or reduce the infiltration deficit. Parameters needed during this planning event are the location of SCP sites and the infiltration rate. As discussed (volume per unit area) in Appendix B, the site loading must be less than the infiltration rate (the vertical hydraulic conductivity) for infiltration to be successful. Loading rates can be lowered by increasing the area allowed for infiltration – the basis of the SCP selection.

The SMDP must provide similar information to that of an MDP. However, the SMDP is more specific in defining the details of the size of the footprint needed to meet infiltration requirements. The following information is needed to support the SMDP:

- Maps of the depth to the water table and the direction of shallow groundwater flow, including the presence of upward and downward gradients. Appendix A describes methods to define groundwater flow direction;
- Maps of the distribution of soil types and the soil properties measured from geotechnical testing results. Appendix C provides information on the geotechnical assessments needed;
- An estimate of the potential infiltration surplus/deficit or excess of runoff caused by the development based upon the comparison of the area of porous and non-porous surface materials within the development to the natural area of infiltration. Appendix B provides advice on these estimation methods;
- A calculation of the potential infiltration area needed to replace the infiltration deficit as determined from the percolation test results or to meet runoff target volume (Appendix B);

- Geotechnical construction measures to maintain the soil infiltration capacity post construction (further details are described in Section 3); and
- A CSM, illustrating the relationship between surface water retention ponds and other stormwater management features, the groundwater flow system, and off-site water resources. Appendix E describes the use and function of a CSM.

When the SMDP is prepared to satisfy runoff volume targets and relies upon infiltration and percolation to meet these requirements, the following is required:

- Surface infiltration tests should be used to evaluate infiltration rates on those parts of the study area where: a) where minimal disturbance to the natural ground surface will take place in the development; and b) where storm water drainage takes advantage of natural surface drainage to return stormwater to the watershed. Infiltration tests using a double ring, Guelph permeameter, and the Modified Phillips Dunne permeameter method are well suited for these tests. Table 3.1 describes the number of surface infiltration tests required within a development depending upon the development area.
- Percolation tests should be conducted in areas where the development plans involve significant ground disturbance such that the pre-development soils will be removed, restructured, or replaced during the development. Under this development scenario, measurement of the surface infiltration rate on the undisturbed soil will not be an aid in designing stormwater management measures to replace the infiltration deficit or to meet runoff target volumes. Instead, percolation tests should be performed at depths between 1.5 metres below ground surface (mbgs) and the water table. Table 3.1 describes the number of surface infiltration tests required within a development depending upon the size of the development area.

  Calculation methods for surface infiltration and percolation test procedures are contained in Appendix B.
- An impact assessment that considers both:
  - the impact of the infiltration deficit on water resources down-gradient for the area of the SCPs and of the SMDP planning area; and
  - the impact of infiltrating surplus runoff volumes due to the buildup of infiltration water on the water table or of a perched water table buildup on a low permeability geologic layer/layers beneath the SCPs:
    - The assessment needs to consider the overall water balance for the SCPs; and
    - Impact of an Infiltration Deficit.

    Taking water from the site by removing the natural infiltration in areas affected by non-porous materials and taking it elsewhere for release (i.e., through storm drains) can cause the loss of groundwater that discharge to aquatic habitats (wetlands and marshes), watercourses, and local aquifers. Appendix D provides methods to estimate the consequences of the infiltration deficit and whether this loss is a concern for these surrounding water resources.
  - Impact of Infiltration and Percolation;

The net outcome of the reliance on infiltration to meet runoff volume targets is an accumulation of water on the water table or above "tight" or low permeability layers within the soil beneath the area of infiltration. This buildup is commonly referred to as a "water table mound." The consequences of the water table mound are that the level of the water table may interfere with the construction of roadways or drainage channels. A water table mound may also induce discharges into sanitary sewers or other subsurface storm water utilities or cause ponding on the ground surface. Across sloped areas, lateral spreading of a water table mound potentially creates springs on hill slopes that cause other runoff features or threatens the slope stability. Appendix G provides guidance on methods to evaluate the buildup and spread of a mound on the water table due to induced infiltration.

2.3.3 Reporting Requirements

The geotechnical and hydrogeologic reporting requirements for a stand-a-lone SMDP consist of the reporting requirements listed for the MDP in Section 2.2, but also require an assessment of impacts and advice on the mitigation of impacts. As a minimum, the impact assessment shall describe:

- The enhanced program of testing percolation infiltration rates, including the locations and the rates measured;
- The evaluation of geologic conditions to determine the depth where a water table mound might build up;
- The evaluation of the height and lateral extent of the groundwater mound built up below infiltration areas; and
- The comparison of the mound to proposed utility corridor depths and ground surface slopes.

2.4 Pond Reports

A Pond Report is supported by construction drawings that detail the size, dimension, and sideslopes for the pond. Ponds include dry ponds, wet ponds, and stormwater wetlands (and zero discharge facilities). Such ponds may either be unlined where the impact or water quality is not a concern but infiltration is needed to meet runoff target volume or may be lined when water quality impacts on wetlands are a concern.

Although Pond Reports may be contained within a SMDP report, all require a level of detail suitable for construction. Where such detail is not provided within the SMDP, an individual Pond Report is needed. The Pond Report must demonstrate how the design objectives within the MDP and SMDP are satisfied.

2.4.1 Authorship and Function

The Pond Report is generally prepared by the developer with expertise provided by qualified professional geotechnical engineers and hydrogeologists. The function of the Pond Report is to include all details relating to the design, construction, and operation of the ponds and how the pond satisfies the objective of the MDP and SMDP. Whether the pond is used to support absorbent landscapes and rain gardens (by means of irrigation practices) or is to flow into infiltration galleries or is to be used as an exfiltration pond itself, needs to be described.

A Pond Report needs to provide sufficient detail for The City to understand how the pond, or any French toe drain berm, or drain will be constructed. Therefore, a detailed subsurface geotechnical investigation is required. Guidance on the type of testing needed to support the detailed design of ponds is provided in Appendix F. Following The City's comments on the construction details in the report, the report can be issued for construction. The hydrogeological support provides an assessment of the details of the engineering of the pond with respect to the designed infiltration rates of the ponds and drains, and the methods for verifying that these infiltration rates have been achieved post-construction. Advice on ranges in infiltration rates and ways and means of considering uncertainty in hydrogeology and water resources are provided in Section 3.0. Considering a range in values and, consequently, a "safety factor" is a prudent measure to ensure that the pond will perform as designed.

In designing stormwater ponds, two functions must be decided as part of the basis for the design:

1. Whether the pond is constructed to allow for infiltration to support runoff targets, or
2. Whether the pond is to be constructed to minimize infiltration.

Both have a design hydraulic conductivity value that must be provided post-construction.

2.4.2 Technical Requirements

The technical information provided with the pond report must include:

1. A statement of the design basis for ponds and subgrade drainage structures.
2. An analysis of the stability of the slopes in the retention ponds (assuming the ponds will be constructed from their native soil).
3. The liners' construction details of hydraulic conductivity and thickness (either as a designed hydraulic conductivity to achieve designed infiltration rates to support runoff targets, or the hydraulic conductivity required to minimize infiltration, depending upon the criteria that best satisfies the LID objective). If constructed of native soil, geotechnical testing must be provided to demonstrate that the designed hydraulic conductivity can be achieved with the native materials. Likewise, if constructed from an engineered soil, the proportion of the admixed soil (e.g., sand with native clay) to achieve the designed hydraulic conductivity needs to be supported by geotechnical testing from an accredited laboratory. If the pond liner is to be constructed of synthetic materials to minimize infiltration, then specification of the liner material must be provided and supported by the manufacturer's detailed specifications. Section 3, Table 3.1 provides advice on the types of geotechnical testing and the number of tests to be undertaken based upon the size of the pond.
4. Advice on how to protect the liner from damage.
5. The design must consider the hydraulic conductivity of the native material surrounding any subgrade drain so the impact of percolation on the groundwater can be considered. More importantly, however, the material comprising the drain must be of sufficient hydraulic conductivity to move the drainage water potentially to its outfall, and yet not be of such a contrast in grain size that pond water cannot infiltrate effectively from the drain into the native soil.

6. Analysis of the impact of percolation for ponds and subgrade drains on the water table, including the potential for impact on roadways, downstream facilities, and infiltration to sanitary sewers and utilities. Appendix G provides methods to be used for this analysis.

2.4.3 Reporting Requirements

The Pond Report needs to contain:

1. Drainage design specifications suitable for release to a contractor for construction.
2. Supporting geotechnical testing so the constructability of the ponds and their ability to be stable and to achieve hydraulic conductivity targets can be evaluated before construction approval is granted. Section 3 discusses construction control practices.
3. Advice on the monitoring and maintenance requirements for pond slopes, base liners, and fill.
4. Test results to demonstrate that the contrast in grain size between subgrade drains and the native soil will not inhibit drainage.
5. An analysis of the impact of ponds and subgrade drains on the height of the water table, adjacent roadways, or other downstream structure, including sanitary sewers and utility corridors or break through sideslopes (following the methods of Appendix G).
6. The need to protect drains from freeze-thaw activity in the winter months.

2.5 Stormwater Management Reports

As considered here, subdivision stormwater management reports fall into two categories:

- The subdivision SWMR; and
- The DSSP.

Section 2.5.1 to 2.5.2 describes the purpose, authority and function, and technical requirements to the SWMP and DSSP, respectively.

2.5.1 Subdivision Stormwater Management Report

This report is fundamentally an actual drainage report and is in support of detailed drainage design and construction drawings suitable for construction.

Within this report, linkages to absorbent landscapes, rain garden, retention areas, bioswales and permeable pavements might be provided. Further, from a geotechnical and hydrogeological perspective each of these structures should be considered to the same level of detail as provided for the Pond Report and must meet the overall objectives of the MDP and SMDP.

Hydrogeological input is needed to assess the impact of any infiltration features on subgrade drains or the water table. Also, a hydrogeological assessment will need to confirm that percolation from infiltration features and subgrade drains has no detrimental impact on adjacent waterways or other structures, including an increase in flow to the sanitary system or utility corridor and the designed SCP meets the objectives of the MDP and SMDP reports.

2.5.1.1 Authorship and Function

These reports are prepared by the developer/consultant with specialist support from a qualified professional geotechnical engineer and hydrogeologist. This report is used to ensure that where water is retained in infiltration features, the infiltrating water does not cause any adverse consequences to nearby roadways, utility corridors, or sanitary storm systems due to the buildup of the retained stormwater on the water table or on the surface of low permeability geologic layers.

2.5.1.2 Technical Requirement

The SWMR should be supported by:

- Geotechnical stability analyses of the areas where water levels are expected to rise and fall rapidly during rainfall events; and
- Hydrogeological assessments to assess the potential for mounding on the surface of the water table or impermeable subsurface layers and to assess the consequences of mounding.

2.5.1.3 Reporting Requirements

The SWMR report shall provide:

- Drainage design specifications suitable for release to a contractor for construction.
- Supporting geotechnical testing so the constructability of the ponds and their ability to be stable and to achieve hydraulic conductivity targets can be evaluated before construction approval is granted. Section 3 discusses construction control practices.
- Advice on the monitoring and maintenance requirements for pond slopes, base liners, and fill.
- Test results to demonstrate that the contrast in grain size between subgrade drains and the native soil will not inhibit drainage.
- An analysis of the impact of ponds and subgrade drains on the height of the water table, adjacent roadways, or other downstream structure, including sanitary sewers and utility corridors or break through sideslopes (following the methods of Appendix G).
- The need to protect drains from freeze-thaw activity in the winter months.
- Identify areas along on the overland drainage course where erosion and slope stability are of concern, measures to minimize slope stability should be provided (advice is provided in Appendix F or geotechnical evaluation of slope and repair and mitigation are in Section 3).
- Illustrate the consequences of the buildup of a water table mound with limits set on the timing and heights of water that can be retained within a water retention facility, bioretention area or bioswale. Advice on estimate of the buildup of a water table mound over time is provided in Appendix G.

- An inspection schedule to inspect whether infiltration occurs, rain garden, or absorbent land slopes become clogged with sediment and measures for repair of clogging are described in Section 3.

2.5.2 Development Site Servicing Plan

This report is fundamentally prepared by a developer to service a private area such as multi-family dwellings, industrial, commercial, or manufacturing area. Within the report aesthetic features might be provided to manage stormwater in a pleasing landscape that includes rain gardens, absorbent landscapes, or retention ponds. Such features need to meet the same level of detailed design as the Subdivision Storm Water Management Report and that, although privately controlled, will meet the MDP and SMDP planning objectives.

2.5.2.1 Authorship and Function

These reports must be reported by the developers/consultant with specialist input from a qualified geotechnical engineer and hydrogeologist.

2.5.2.2 Technical Requirements

The DSSP should be supported by:

- Geotechnical assessment of the stability of slopes in the area covered by the DSSP where water levels may be expected to rise and fall quickly.
- Hydrogeological assessment of the potential for adverse consequences due to infiltration through the area where water ponding is designed. These consequences include ponding with surface breakout on side slopes, interference with roadways, or inflow to utility corridor or sanitary sewer area.

2.5.2.3 Reporting Requirement

The DSSP report shall contain:

- Drainage design specifications suitable for release to a contractor for construction.
- Supporting geotechnical testing so the constructability of the ponds and their ability to be stable and to achieve hydraulic conductivity targets can be evaluated before construction approval is granted. Section 3 discusses construction control practices.
- Advice on the monitoring and maintenance requirements for pond slopes, base liners, and fill.
- Test results to demonstrate that the contrast in grain size between subgrade drains and the native soil will not inhibit drainage.
- An analysis of the impact of ponds and subgrade drains on the height of the water table, adjacent roadways, or other downstream structure, including sanitary sewers and utility corridors or break through sideslopes (following the methods of Appendix G).
- The need to protect drains from freeze-thaw activity in the winter months.

- Identify areas along on the overland drainage course where erosion and slope stability are of concern, measures to minimize slope stability should be provided (advice is provided in Appendix F or geotechnical evaluation of slope and repair and mitigation are in Section 3).
- Illustrate the consequences of the buildup of a water table mound with limits set on the timing and heights of water that can be retained within a water retention facility, bioretention area or bioswale. Advice on estimate of the buildup of a water table mound over time is provided in Appendix G.
- An inspection schedule to inspect whether infiltration occurs, rain garden, or absorbent land slopes become clogged with sediment and measures for repair of clogging are described in Section 3.

3.0 IMPLEMENTATION OF STORMWATER MANAGEMENT FEATURES FOR LOW IMPACT DEVELOPMENT

This portion of Module 1 – Geotechnical and Hydrogeological Considerations provides guidance on the type and details of geotechnical and hydrogeological investigations and techniques to be applied during the conceptual design, detailed design, construction, and post-construction stages of a SCP. Sections 3.1 to 3.4 describe these investigations and techniques according to the following topics:

- The level of effort, investigation, and technique needed from conceptual design and detailed design through to the construction and post-construction care of the stormwater management features;
- Safety factors for an engineered or natural slope within the development and to accommodate uncertainty in the infiltration capacity:
  - To convert the field-saturated hydraulic conductivity to the actual saturated hydraulic conductivity;
  - To account for clogging of infiltration beds due to fine-grained sediment; and
  - To encompass a) the natural range in hydraulic conductivity of the underlying geologic materials and b) the range of hydraulic conductivity as a consequence of construction of the infiltration beds.
- Construction inspection measures needed to confirm that the design hydraulic conductivity is maintained during and post-construction of the proposed storm water management features; and
- Evaluation of the potential design life and means to regenerate the infiltration or percolation capacity during the design life.

3.1 Level of Effort

This section recommends the investigative effort and the associated techniques for various SCPs. For convenience, the size of development is generally considered to be either:

- Less than 1 Ha;
- 1 to 10 Ha;
- 10 to 100 Ha; or
- Greater than 100 Ha.

Further, the investigative techniques will vary with the stages of a development. Subsections 3.1.1 to 3.1.4, respectively, describe the investigative techniques best applied during:

- Site assessment for conceptual planning to locate stormwater management features within the overall development;
- Stormwater management feature selection;
- Detailed design; and
- Performance verification following construction of the stormwater management features.

3.1.1 Conceptual Design Site Assessment

During the initial conceptual and planning stages of a development – the SMDP planning stage, site assessments should be undertaken (Section 2.3). The purpose of the site assessment is to help the developer decide where within the development storm water management features can be placed to create the best balance between meeting the watershed objectives and optimizing the development yield.

The site assessment will require subsurface investigations to:

- Characterize soil types and their properties and variability within the development; and
- Characterize the occurrence of groundwater, including groundwater flow rates and direction and locate areas of groundwater and surface water within the development site.

Appendix A provides advice on the instrumentation and methods to be used to determine groundwater flow rates and directions. Appendix C provides advice on the methods to use to characterize soil conditions. The variation in soil type is used to estimate the probable range and variation of infiltration rates using the methods described in Appendix H.

Tables 3.1 and 3.2 describe the number of investigative locations according to the size of the development.

Table 3.1: Number of Investigative Locations by Area of a Development Site

Development Area	Investigative Locations	Soil Samples Collected[1]	Soil Property Measurements[2]			Depth Profiles	Cross-Section
			Water Content	Atterberg Limit Testing	Grain Size Analysis (Sieve or Hydrometer)		
Less than 1 Ha	3 (but more than 10 m apart)	30	30	3	3	3	0
1 to 10 Ha	3 to 12	30 to 120	30 to 120	12	12	3 to 12	3
10 to 100 Ha	12 to 20	120 to 200	120 to 300	20	20	12 to 20	4 to 5
>100 Ha	20 minimum	200+	200+	20+	20+	20+	4 to 5

[1] Assumes one sample collected at an average of 1.5 m intervals of depth to a depth of 15 m or auger refusal whichever is shallowest.

[2] Minimum three sets of tests per soil unit.

Table 3.2: Number of Monitoring Wells and Monitoring Well Response Tests by Area of a Development Site

Development Area	Recommended Minimum Number of Wells	Recommended Monitoring Well Response Tests[1]
Less than 1 Ha	3 (but more than 10 m apart)	3
1 Ha to 10 Ha	3 to 12 (spacing of 300 m)	12
1o to 100 Ha	12 to 20 (spacing of 300 m)	20
Greater than 100 Ha	20 or more wells spaced at 300 m intervals	20+

[1] A minimum number of response tests is three tests per stratigraphic unit, although a greater number of tests is advisable where the range of values exceeds the confidence intervals described in Section 3.2.

3.1.2 Selection of Stormwater Management Features

Information collected during a site assessment defines the variation in soil and groundwater conditions within the development and helps to decide where stormwater management features may be best placed within the development. These best locations are subsequently referred to in this document as "candidate locations." However, to decide the storm water management feature best suited to the site and to confirm the specific needs for a particular SCP within a candidate location are met requires testing of the infiltration capacity.

The soil's infiltration capacity defines the infiltration area needed to manage stormwater and consequently the nature of the stormwater management feature (i.e., absorbent landscape, rain garden, bio-retention area, retention pond, or infiltration galleries, including toe drains and French drains) that best meets the overall water resources needs of the development. There are two main factors to be evaluated at candidate locations prior to selecting the stormwater management feature and proceeding to a detailed design:

- The variability of the soil (with depth and across the development area); and
- The vertical hydraulic conductivity (i.e., as discussed in Appendix B, the vertical hydraulic conductivity is the maximum infiltration rate of the soil).

The variability of the soil is assessed during the subsurface investigation and determines whether any low hydraulic conductivity stratigraphic units are present with depth below candidate locations. Low hydraulic conductivity stratigraphic units limit the infiltration capacity and may create adverse consequences to nearby roadways, utility corridors, or otherwise stable slopes. Table 3.1 provided advice on the number of investigative locations to be considered for a particular size of development.

The vertical hydraulic conductivity (i.e., the infiltration rate) is determined using the methods described in Appendix B. Because it is not yet decided whether surface infiltration features or a subsurface infiltration galley such as Toe drain or a French drain is the preferred stormwater management method, it is recommended that the infiltration capacity be tested using either the ASTM, or Argue cased borehole methods or the Guelph permeameter method. The depth to be tested must be just below the depth of the infiltration base of the storm water management feature.

To aid in selecting the best stormwater management option, infiltration or percolation tests to measure the infiltration rate should be undertaken every 400 m^2 within the candidate locations where ponds or infiltration galleries are to be used. But for rain gardens or absorbent landscapes, infiltration rates based upon a single test within the candidate location will be adequate.

3.1.3 Detailed Design

Estimates of the vertical hydraulic conductivity of the soil forming the base of the infiltration area is needed for the detailed design of stormwater management features. At the detailed design stage, a greater density of infiltration or percolation test is required than the density of tests used to select candidate locations.

All tests at the detailed design stage should be conducted on the soil surface within a stormwater management feature where infiltration is planned to take place. These tests shall be undertaken using the ASTM or Argue cased borehole methods, the Guelph permeameter or the double ring infiltrometer method described in Appendix B.

The double ring infiltrometer method provides a greater accuracy of the infiltration rate than either of the cased borehole methods but is more costly to set up and apply. Therefore, the developer and his consultant applying this method should consider the need for accuracy over the spatial coverage achievable by each type of testing method.

Further, use of the double ring infiltrometer presumes that the surface to be tested is free of stones or brittle soils that reduce the effective surface seal for the test. At those sites where this condition is not achieved, the cased hole methods are the practical alternative. The inherent assumption in this design approach is that any geologic layers that may inhibit infiltration have been considered when the preferred stormwater management feature was selected. The purpose of these surface infiltration tests is to support the sizing of the infiltration area needed to either replace the infiltration deficit or aid in meeting target runoff volumes.

A surface infiltration test should be undertaken every 100 m^2 within the area of the stormwater management feature. Test results will provide a range in values and the design value can be established using either the empirical approach or statistical approach detailed in Section 3.2. Use of these methods depends upon the size of the development with the empirical approach favoured at development sites less than 10 Ha and the statistical approach favoured on LID sites greater than 10 Ha.

3.1.4 Performance Verification

During construction, surface infiltration tests can be performed on the prepared and engineered base of the stormwater management feature using the double ring infiltrometer method. The surface to be tested is the grade of the native soil prior to placement of any protective sand or filter layer or any soil designed to support vegetative growth. Inherent in this approach is that the filter sand or growth supporting soil has a greater hydraulic conductivity than the engineered native soils on which it is placed. The number of surface tests depends upon the area of the infiltration basin. Table 3.3 provides advice on the number of tests to be considered.

Table 3.3 – Number of Double Ring Infiltrometer Tests on Prepared Sub-grades

Size of the Infiltration Base (m^2)	Number of Tests
Less than 100	1
100 to 400	2
400 to 900	4
900 to 1600	6
More than 1600	10

After construction and placement of a surface protective layer or soil supporting vegetation, the final surface should also be tested using the double ring infiltrometer method. In this case, only one test is needed per stormwater infiltration feature.

The double ring infiltrometer method is proposed here because engineering of the subgrade and preparation of the surface protective layer and vegetation supporting layer should create a uniform medium (i.e., free of stones and cobbles that may not allow the test device to be properly placed). If other quality control measures (such as the soil bulk density differ by more than 25% than additional tests) should be undertaken to confirm that the design infiltration rate has been achieved.

3.1.5 Summary of Testing Methodologies

Table 3.4 summarizes the recommended testing methodology as a function of the design phase.

Table 3.4: Summary of Testing Methodologies

Development Phase	Purpose	Recommended Method
Site Assessment	To identify candidate locations for stormwater management features	Soil type, grain size analysis, and literature values for infiltration
Selection of Preferred Option	To select the preferred option utilizing candidate locations	ASTM or Argue Cased borehole methods or the Guelph Permeameter
Detailed Design	To obtain a design infiltration rate and aid in sizing of the infiltration base of the stormwater management feature	Surface infiltration tests of the double ring method or cased borehole methods
Verification Testing during Construction and Post-Construction	To confirm construction of the design infiltration rate To confirm surface protective layer or vegetative support soils having higher hydraulic infiltration than the prepared subgrade	Surface infiltration tests using a double ring infiltrometer

3.2 Allowances for Safety Factors and Ranges in Hydraulic Conductivity Estimates

Safety factors are used in the engineering of natural slopes and in design of side wall slopes used to create stormwater retention ponds. Means to evaluate slope stability are described in Appendix F. Safety factors are also used to assess the uncertainty in the behaviour of soil on slopes, i.e., the soil stability. This uncertainty arises due to ranges in water content, soil properties, and soil structures that occur naturally from place to place across a development.

Similarly, under natural conditions, the vertical and horizontal hydraulic conductivity of the soil beneath a stormwater management feature will vary over several orders of magnitude within a development.

Appendices A and B provide guidance and methods to measure hydraulic conductivity and the infiltration capacity but not specifically on how to manage the range or "uncertainty" in the measured value.

Section 3.2.1 and 3.2.2 provide guidance on the ways and means to accommodate these infinitely uncertain elements at the detailed design stage for stormwater management factors within a development.

3.2.1 Slope Stability Factors of Safety

Development Setback Line

The City requires developers to submit Slope Stability Reports to define stable and unstable lands within the area set out in the report. The stability of the slopes generally dictates the boundaries of the development. A geotechnical assessment, including a desktop study, site reconnaissance, and a geotechnical investigation, is required to characterize the existing slope and the subsurface soil and groundwater conditions. Slope stability analyses are then required to determine the stability of the slope and to establish setback lines from the crest of the slope at which the factor of safety is 1.5 or greater.

The City requires that setback lines be established using slope stability analysis considering the worst case scenario, such as most probable adverse groundwater and loading conditions.

Since the mid-1990s, The City's guidance documentation for the establishment of setback lines has often been interpreted to allow for two setback lines: (1) a setback line for building structures based on a factor of safety of 1.5; and (2) a second setback line closer to the slope crest for the edge of the development (i.e., the back of lots including pathways) based on a factor of safety of 1.3.

However, based on experience with unstable river valley slopes and a review of slope stability policies from several other urban jurisdictions, The City is now applying the slope stability factor of safety of 1.5 as the criterion to establish the edge of development setback line.

Pond Design Slope Stability

The City requires a minimum factor of safety of 1.5 for all pond slopes. In many other jurisdictions, it is common to apply a factor of safety of 1.5 to the long-term stability conditions and to require a lesser factor of safety (as low as 1.1) to the short-term rapid drawdown condition. Therefore, the geotechnical engineer conducting the slope stability analysis needs to understand all possible operating conditions for the pond in order to understand whether the classic short-term rapid drawdown condition is actually applicable for the pond slopes. The geotechnical report shall, therefore, clearly outline which operational scenarios were considered.

3.2.2 Ranges in Hydraulic Conductivity

Ranges in hydraulic conductivity arise due to a variety of factors and among them are:

- Imprecision in the field measurements;
- Conversion from field saturated to actual hydraulic conductivity;
- Changes in the infiltration capacity due to clogging with fine-grained sediment; and

- Properties that occur naturally within the geology beneath the stormwater management feature.

However, a reliable and consistent means of estimating the hydraulic conductivity is required to support the detailed design and measure the performance of stormwater management features over the long term.

Imprecision in Field Measurements

Any two individuals, and indeed the same individual, measuring a hydraulic conductivity and using the same measurement techniques will produce a different value from test to test. Advice by the British Columbia On-Site Sewage Association (BCOSSA [2007] and Argue [2004]) suggest that at any test location used to measure the infiltration rate (the vertical hydraulic conductivity) each test be repeated four times and the second lowest value of the four tests be used as the representative value for K_{fs} (hydraulic conductivity under field saturated conditions).

Conversion from Field Saturated to Saturated Hydraulic Conductivity Value

Field measured values for the hydraulic conductivity in the zone above the water table typically rely upon a test method that requires the test zone to be "field saturated" prior to undertaking the test (e.g., Argue 2004, Elrick and Reynolds 1986, and Asleson et al. 2007). The resulting value yields a "field saturated" hydraulic conductivity value (K_{fs}), which is lower than the actual hydraulic conductivity in effect when operating as an infiltration basin. The lower value for K_{fs} is due the trapping of air in the void space which limits the ability of the soil to become fully saturated. To correct from the K_{fs} value to the saturated hydraulic conductivity K_{sat}, both Elrick and Reynolds (1986) and BCOSSA (2007) recommend multiplying the K_{fs} by two to get the K_{sat} (the design infiltration capacity). Correction for field measurements to a design value is not required for measurements of hydraulic conductivity below the water table or other naturally saturated soil.

Changes Due to Clogging

Under operating conditions, the pore spaces of the soil forming the base of the infiltration basin may become clogged by fine-grained sediments. Fine-grained and suspended sediments penetrate the soil pores with the infiltrating water. Most of this fine-grained sediment will enter the pore space of any protective sand layer or soil used to support vegetative growth above the base layer of the infiltration basin. As long as these protective layers have a vertical hydraulic infiltration more than a factor of 100 greater than the underlying soil, clogging of the protective layer will not be an issue, but from time to time, this protective material may need to be replaced (see Section 3.4) to eliminate the clogging effect.

However, Argue (2004) recommends, for those soils that may receive the stormwater, that the design value (K_{sat}) be reduced by a factor of five to accommodate clogging in the design of the infiltration basin, i.e., a value for the saturated hydraulic conductivity (K_{sat}) of 1×10^{-6} m/s should be reduced to 2×10^{-7} m/s for design of the stormwater management features.

Accommodation for a Range in Hydraulic Conductivity

The measured values for the actual hydraulic conductivity will differ markedly from place to place within an infiltration basin. Opinions on the use of a representative value for the hydraulic conductivity vary widely with some practitioners using a geometric mean value of all of the hydraulic conductivity values measured and others using the worst case or lowest value. The lowest value may be unduly conservative whereas the median value may not adequately consider the influence of soil macro pores or other geologic structures.

The guidance provided here is that for small sites, (i.e., either less than 1 Ha or between 1 Ha and 10 Ha in size) an empirical approach to providing a design value be applied whereby:

1. A minimum of four test locations be used for each stormwater management feature.
2. Four tests shall be conducted at each of the four locations and the average of the four tests be used as the K_{fs} for that particular location.
3. To obtain a design infiltration value (K_{sat}), the second lowest value of the four locations is used (e.g., for K_{fs} values of:

   1) 360 mm/day;

   2) 470 mm/day;

   3) 780 mm/day; and

   4) 190 mm/day.

   The selected representative K_{fs} is 360 mm/day (after BCOSSA 2007).
4. The K_{sat} or design infiltration rate is then estimated as 2 x K_{fs} or 720 mm/day (and if the tested soil surface is in direct contact with the stormwater) reduced by a factor of 5 to 144 mm/day to establish the K_{sat} for design purposes.

As stated by BCOSSA, this approach follows the "widely recommended approach of using a design value that is no higher than the median, but higher than the worst case measurement."

For larger sites, those greater than 10 Ha, the calculated number of infiltration tests required is more that 20 (one every100 m^2). In this case, the approach to establishing a design rate to be followed is a statistical approach based upon an appreciation of the spread or range in hydraulic conductivity values measured. The approach to be considered here is that:

1. A minimum of four tests be conducted at each of ten test locations and the average of the test values be used for the K_{fs} at that location.
2. The K_{fs} values are converted to K_{sat} values.
3. Statistically evulate the field values by:
   a) The values of all K_{fs} values are tabulated and converted to K_{sat} values (K_{sat} = 2 x K_{fs} (Tables 3.4a and 3.4b are the two hydraulic conductivity test results from two sites in Alberta investigated in 2010 by EBA as examples of the statistical test. The test method used was the Argue method).
   b) The mean and standard deviation are calculated from the log value of the K_{sat} values. Hydraulic conductivity values are not statistically normally distributed (an even spread about the mean or

average value). Instead, hydraulic conductivity is log normally distributed, i.e., the log values are normally distributed making the following statistical analysis of data reliable.

c) Using the following formula, calculate the confidence interval.

$$t_\alpha = \frac{\sqrt{n}\, D}{\sigma}$$

Where;

- t_α is the two-tailed confidence interval at a level of significance (α) available from standard statistical tables;
- n is the degree of freedom (the number of tests minus 1);
- D is the test precision (allow for 20 per cent - Mason, 1983); and
- σ is the standard deviation of the logarithmathic K_{sat} values.

As shown by the attached examples, the two t_α values are 1.38 and 0.81 for the data from the two sites in Tables 3.4a and 3.4b, respectively.

d) For those K_{sat} values producing a confidence interval of greater than 90%, the geometric mean of the K_{sat} can be used to estimate the K_{sat} (K_{sat} = 2 x K_{fs}), i.e., for Table 3.4a the value for t_a is 1.38 a value equivalent to a confidence interval (Table 3.4 c) of greater than 90%. The design K_{sat} value should be equal to the mean value for the log of -5.11 or a K_{sat} of 7.8e-06 m/s. Given the allowance for clogging of 1/5 of the K_{sat} the design rate should be 1.6e-06 m/s.

e) For those K_{sat} values producing a confidence interval of less than 90%, the K_{sat} value is calculated as the third lowest value if six tests are undertaken, the fourth lowest value if eight tests are conducted, and so on (i.e., the number of tests/two is the design value) (BCOSSA 2007). For the test results shown on Table 3.4b, there are 14 tests taken so the design K_{sat} values should be the seventh lowest value or the log value of -6.86 or equal to a K_{sat} of 1.4e-07m/s and after correcting for clogging (1/5 of K_{sat}) estimated as 7.0e-08 m/s.

Table 3.5 Example of Calculation for Vertical Hydraulic Conductivity Design Value

Site A

A)

Test Location	Field Saturated Hydraulic Conductivity (K_{fs}) - m/s	Saturated Hydraulic Conductivity (K_{sat}) - m/s	Log of the K_{sat}
1	3.00E-06	6.0E-06	-5.22
2	2.56E-06	5.1E-06	-5.29
3	5.63E-05	1.1E-04	-3.95
4	7.55E-06	1.5E-05	-4.82
5	2.90E-06	5.8E-06	-5.24
6	2.99E-06	6.0E-06	-5.22
7	3.25E-06	6.5E-06	-5.19
8	6.43E-06	1.3E-05	-4.89
9	6.67E-06	1.3E-05	-4.87
10	3.31E-06	6.6E-06	-5.18
11	6.75E-06	1.4E-05	-4.87
12	7.54E-06	1.5E-05	-4.82
13	5.78E-06	1.2E-05	-4.94
14	4.94E-06	9.9E-06	-5.01
15	2.96E-08	5.9E-08	-7.23
16	2.57E-06	5.1E-06	-5.29
17	1.00E-06	2.0E-06	-5.70
18	2.90E-06	5.8E-06	-5.24
19	3.45E-05	6.9E-05	-4.16
20	3.92E-06	7.8E-06	-5.11
		Mean	**-5.11**
		Standard Deviation	**0.63**
		Margin for Error	**0.20**
		$t_?$	**1.38**

Conclusion: t_a is greater than 90%; therefore, design value for the hydraulic conductivity is the geometric mean value (-5.11 log value) or 7.8e-06 m/s).

B) **Site B**

Test Location	Field Saturated Hydraulic Conductivity (K_{fs})	Saturated Hydraulic Conductivity (K_{sat})	Log of the K_{sat}
1	6.98E-07	1.4E-06	-5.86
2	4.00E-08	8.0E-08	-7.10
3	5.62E-08	1.1E-07	-6.95
4	6.77E-07	1.4E-06	-5.87
5	6.32E-08	1.3E-07	-6.90
6	5.65E-07	1.1E-06	-5.95
7	1.12E-09	2.2E-09	-8.65
8	3.00E-08	6.0E-08	-7.22
9	4.78E-07	9.6E-07	-6.02
10	6.98E-08	1.4E-07	-6.86
11	7.42E-09	1.5E-08	-7.83
12	1.35E-06	2.7E-06	-5.57
13	4.56E-07	9.1E-07	-6.04
14	6.39E-07	1.3E-06	-5.89
		Mean	**-6.62**
		Standard Deviation	**0.89**
		Margin for Error	**0.20**
		$t_?$	**0.81**

Conclusion: t_a is less than 90%; therefore, the design value for the hydraulic conductivity is the seventh lowest value of -7.61 (log value) or a design rate of 2.5e-08m/s.

Table 3.4c Quantiles of the t Distribution (Values of t Such That 100p% of the Distribution Is Less Than t_p)

	Degrees of Freedom	$t_{0.6}$	$t_{0.70}$	$t_{0.80}$	$t_{0.90}$	$t_{0.95}$	$t_{0.975}$	$t_{0.990}$	$t_{0.995}$	
	1	.325	.727	1.376	3.078	6.314	12.706	31.821	63.657	
	2	.289	.617	1.061	1.886	2.920	4.303	6.965	9.925	
	3	.277	.584	.978	1.638	2.353	3.182	4.541	5.841	
	4	.271	.569	.941	1.533	2.132	2.776	3.747	4.604	
	5	.267	.559	.920	1.476	2.015	2.571	3.365	4.032	
	6	.265	.553	.906	1.440	1.943	2.447	3.143	3.707	
	7	.263	.549	.896	1.415	1.895	2.365	2.998	3.499	
	8	.262	.546	.889	1.397	1.860	2.306	2.896	3.355	
	9	.261	.543	.883	1.383	1.833	2.262	2.821	3.250	
	10	.260	.542	.879	1.372	1.812	2.228	2.764	3.169	
	11	.260	.540	.876	1.363	1.796	2.201	2.718	3.106	
	12	.259	.539	.873	1.356	1.782	2.179	2.681	3.055	
from Site B →	13	.259	.538	.870	1.350	1.771	2.160	2.650	3.012	→ Value of t ?
	14	.258	.537	.868	1.345	1.761	2.145	2.624	2.977	
	15	.258	.536	.866	1.341	1.753	2.131	2.602	2.947	
	16	.258	.535	.865	1.337	1.746	2.120	2.583	2.921	
	17	.257	.534	.863	1.333	1.740	2.110	2.567	2.898	
	18	.257	.534	.862	1.330	1.734	2.101	2.552	2.878	
from Site A →	19	.257	.533	.861	1.328	1.729	2.093	2.539	2.861	→ Value of t ?
	20	.257	.533	.860	1.325	1.725	2.086	2.528	2.845	
	21	.257	.532	.859	1.323	1.721	2.080	2.518	2.831	
	22	.256	.532	.858	1.321	1.717	2.074	2.506	2.819	
	23	.256	.532	.858	1.319	1.714	2.069	2.500	2.807	
	24	.256	.531	.857	1.318	1.711	2.064	2.492	2.797	
	25	.256	.531	.856	1.316	1.708	2.060	2.485	2.787	
	26	.256	.531	.856	1.315	1.706	2.056	2.479	2.779	
	27	.256	.531	.855	1.314	1.703	2.052	2.473	2.771	
	28	.256	.530	.855	1.313	1.701	2.048	2.467	2.763	
	29	.256	.530	.854	1.311	1.699	2.045	2.462	2.756	
	30	.256	.530	.854	1.310	1.697	2.042	2.457	2.750	
	40	.255	.529	.851	1.303	1.684	2.021	2.423	2.704	
	60	.254	.527	.848	1.296	1.671	2.000	2.390	2.660	
	120	.254	.526	.845	1.289	1.658	1.980	2.358	2.617	
	∞	.253	.524	.842	1.282	1.645	1.960	2.326	2.576	

From Gilbert 1987. Statistical Methods for Environmental Pollution Monitoring.

In this way, data over a wide area can be considered relatively uniform in hydraulic performance while a site yielding values for hydraulic conductivity that has a broad range does not get assigned a design value no higher than the median value. This approach follows that of both Asleson et al. (2006) and HWC (1992) where the statistical distribution of normally distributed values was used to estimate the number of additional data needed. We are reluctant to propose more than ten tests merely for the need of satisfying the statistical principles lest we end up altering any of the inherent potential infiltration capacities.

3.3 Construction and Post-Construction Inspection Requirements

Inspection is proposed for ensuring the constructed facility meets and continues to meet the design hydraulic conductivity (i.e., infiltration rate) and thereby satisfies the LID principles.

Both surface stormwater management features such as rain gardens, absorbent landscapes, bio-retention, retention basins, and infiltration trenches (toe drain or French drain) and subsurface infiltration galleries are included in the construction and post-construction inspection needs.

3.3.1 Care and Control During Construction of the Stormwater Management Features

While the stormwater management features are constructed, the native soil used to develop the design infiltration rate will be disturbed, potentially altering the soil's drainage characteristics. During the preparation of most subgrades for industrial and residential developments in the city, standard construction practices are typically undertaken to meet specified properties and the desired engineering function.

3.3.2 Post-Construction Assessment Requirements

After construction and before the stormwater management features are put into operation, assessment of the performance of the features shall be undertaken to confirm the features all meet the infiltration requirements and to confirm that the storm water conducted to the facility does not create undue erosion or sediment deposition.

The assessment is undertaken by a combination of inspection and monitoring activities of the stormwater management feature.

Erickson et al. (2010), at the University of Minnesota, developed a four-level assessment program for evaluating the functionality and performance of stormwater treatment practices. Those four levels comprise:

- Visual inspection;
- Capacity testing;
- Synthetic runoff testing; and
- Monitoring.

These four levels of assessment have been adapted and modified here to assist The City in satisfying the LID principles. Table 3.4 summarizes these details.

The four assessment levels are proposed to represent a more intensive level of evaluation. It may not be necessary to proceed with a subsequent assessment level if a satisfactory function and performance are realized during the primary assessment level.

Level 1 – Visual Inspection

The objectives of the visual inspection of the stormwater management facilities are:

- To determine the stability of the side wall; and
- To evaluate the buildup of fine-grained sediments.

Visual inspection of each stormwater management feature within an development shall be undertaken periodically during the year. Three inspection events are recommended:

- Early spring after snow melt has begun;
- Late June to early July after the spring rains; and
- Late summer or early autumn.

This frequency may change after a couple of years when the system performance has become stable. But also, in the first several years of operations, visual inspection of the infiltration features should be made after periods of intense rainfall.

The visual inspection should:

- Include areas of water ponding;
- Identify signs of side wall or berm movement;
- Look for sediment accumulation around inlet and outlet structures (particularly around subsurface infiltration galleries) and behind areas of slow moving water (e.g., roots of vegetation and aesthetically placed trees or rocks); and
- Examine for penetration of fine-grained sediments into the upper 12 cm of soil by digging with a small hand trowel, spreading the soil to look for fines and taking a photograph of the spread soil as a permanent record. Quantification of the quantity of fine-grained sediment penetration should be undertaken using a sieve analysis. This analysis will involve extracting a 100 gm mass of soil, air drying the soil, and then passing it through a sand-sized sieve to measure the quantity of sediment finer than sand sizes. A core sample of the soil should also be taken to inspect for any fine-grained sedimentary layering in the soil.

The stormwater management feature will be considered to be performing well if there is no standing water on the surface and be sustainable if there is no evidence of sediment accumulation on the surface or in areas of reduced water velocity or penetration of the soil by fine grained sediments.

Level 2 – Capacity Testing

The objective of capacity testing is to confirm that the design infiltration capacity of the infiltration basin has been sustained under operating conditions. Capacity testing will involve testing of each infiltration basin within a development. These tests should be conducted once every five years. These levels of assessment should consist of a series of spatially distributed individual test locations across the infiltration feature and shall use a cased or open borehole method (Argue, ASTM, or Guelph permeameter) to measure the vertical hydraulic conductivity (over the infiltration area).

The performance of the infiltration feature will be considered acceptable if there is no standing water in the infiltration area three days following a storm event and if the geometric mean value of the results is within a factor of ten of the design infiltration rate.

This level of testing cannot be undertaken on subsurface infiltration galleries.

Level 3 – Synthetic Runoff Testing

The objective of synthetic runoff testing is to confirm, on a scale larger than the discrete testholes used in Level 2 that infiltration basins are performing according to the design infiltration rate. The reason for the larger area is to ensure that layer scale macro pore features such as vegetative roots, worm burrowing, and the like, which can be major contributors to the overall infiltration rates, are taken into account in the overall assessment of the infiltration capacity.

Synthetic rainfall testing involves allowing a known volume of water to be introduced to the infiltration basin and monitoring the rate of decline of the water level with time. This method is the only feasible method of evaluating the performance of a subsurface infiltration gallery.

The infiltration capacity measured for this method will be considered successful if the estimated hydraulic conductivity is within a factor of ten of the design infiltration rate, and no water is observed to build up and be retained on the surface of the infiltration bed for longer than three days after the test is initiated.

Level 4 – Monitoring

Monitoring is the most comprehensive form of assessment of stormwater management features. It will be required in the most sensitive environmental areas or where The City requests monitoring.

The objective of monitoring is to confirm, on a long-term basis, that water table mounds beneath infiltration basins will not cause an adverse impact and that no standing water accumulates within subsurface infiltration basins.

Monitoring should be carried out on a continuous basis in areas where water table mounding is a concern and wherever subsurface infiltration galleries are used to manage stormwater.

Monitoring will include:

- Measurement of water levels in monitoring well standpipes; or
- Comparisons of inlet and outlet rates for the stormwater management feature during snowmelt or rainfall events.

Monitoring will be considered a successful measurement of performance whenever the water level in monitoring well standpipes is well below the base of the infiltration feature and if standing water does not reside in the stormwater management feature for longer than three days.

Table 3.6 Inspection and Monitoring Requirements of Stormwater Management Features

	Assessment Level			
	1 Visual Inspection	**2 Capacity Testing**	**3 Synthetic Runoff Tests**	**4 Monitoring**
Timing	▪ Three times per year (particularly in the first year)	▪ Once every five years	▪ Once every five years or if capacity testing fails	▪ Ongoing
Objective	▪ To determine stability of sideslopes and degree of sedimentation observed on surface	▪ To confirm that the design infiltration capacity is maintained	▪ To confirm, on a scale larger than the cased borehole method, that infiltration capacity satisfies the design infiltration rate	▪ To confirm that water levels are not building up within subsurface infiltration galleries and a water table mound is building up on the water table
Success Factor	▪ No movement of side slopes or walls. ▪ Intake and outlet structure clean and free draining. ▪ No accumulation of surficial sediment or penetration to upper 12 cm of soil	▪ Hydraulic conductivity values on average within one order of magnitude of the design value	▪ Hydraulic conductivity within a factor of ten of the design rate and no buildup of water level above the water table	▪ No adverse consequences due to water table mounding and no standing water within the infiltration feature ▪ Installation of monitoring wells adjacent to infiltration feature and with the infiltration gallery
Factors to Consider	▪ Inspection of sand and hard dry soil for fine-grained material – photograph to record compared to visual presentation ▪ Collection of sediment in slow moving water	▪ Spatially distributed infiltration rate or percolation rate using a cased or open borehole method	▪ Measure the rate of decline of water level where a known volume of water is introduced to the infiltration basin	

3.4 Design Life and Regeneration of Infiltration Rates

Information about the design life of infiltration types, storm water management features, and mechanisms to re-generate infiltration rates has not been reported in literature. Suggestions, therefore, have been borrowed from sewage system guidelines.

The design life of infiltration basins should be considered to be no longer than 20 years without retrofit or rebuilding the structure.

The infiltration capacity of the infiltration features is reduced by clogging of the pore spaces with fine-grained sediment. Suggestions may be that back flushing of the filter bed may be practical but no evidence of this method to regenerate the infiltration capacity has been found.

Therefore, to regenerate the infiltration capacity one should consider the following actions:

1. Confirm by Testing Method D (Section 3.3) or observation of standing water (longer than three days) that the infiltration capacity is more than a factor of 10 less than its design capacity.
2. Remove the overlying media and vegetation supporting materials that overlie the native soil and either wash the removed soil to remove fine-grained material or replace.
3. Test the surface of the native soil beneath the filter sand and vegetative supporting cover materials using a double ring infiltrometer (Appendix B) to measure the actual vertical hydraulic conductivity.
4. If the vertical hydraulic conductivity of the native soil is acceptable, then replace the protective cover and vegetation supporting materials.
5. If the vertical hydraulic conductivity is less than the infiltration rate needed to manage the volume required, seek amendments to improve the vertical hydraulic conductivity. These amendments may include:
   - Drilling a network of shallow permeable circular drains to a geologic structure below the base of the infiltration basin; and
   - Constructing a series of trenches infilled with permeable material intended to achieve a vertical hydraulic conductivity that satisfies the design volume over the drainage volume over the infiltration basin.

These retrofit measures can be expensive and it is advisable the prevention of clogging should be considered for all stormwater features. This will involve use of a geosynthetic layer between the infiltration surface and the media or vegetation supporting soil placed on top of the layer. Replacement of this geosynthetic fabric and the overlying soil will be a more cost effective alternative to replacing the native soil.

/bmw

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FIGURES

Relevant Issue	Planning Levels	Scope of Activity
What is the natural setting?	Watershed Plan	Reconnassance – desktop study
What can be constructed to control stormwater within this natural setting?	Master Drainage Plan	Site Assessment – subsurface investigation
What is the consequences of construction of stormwater drainage measures in the natural setting?	Staged Master Drainage Plan	Impact assessment and evaluation of options
What do we do to preserve and sustain the stormwater control facility for the long term?	Pond Report Subdivision Stormwater Management Report Development Site Serving Plan	Detailed geotechnical design of preferred options for drainage system, slope stability and care, and maintenance over a time

LEGEND

NOTES

STATUS
ISSUED FOR USE

CLIENT
The City of Calgary

A TETRA TECH COMPANY

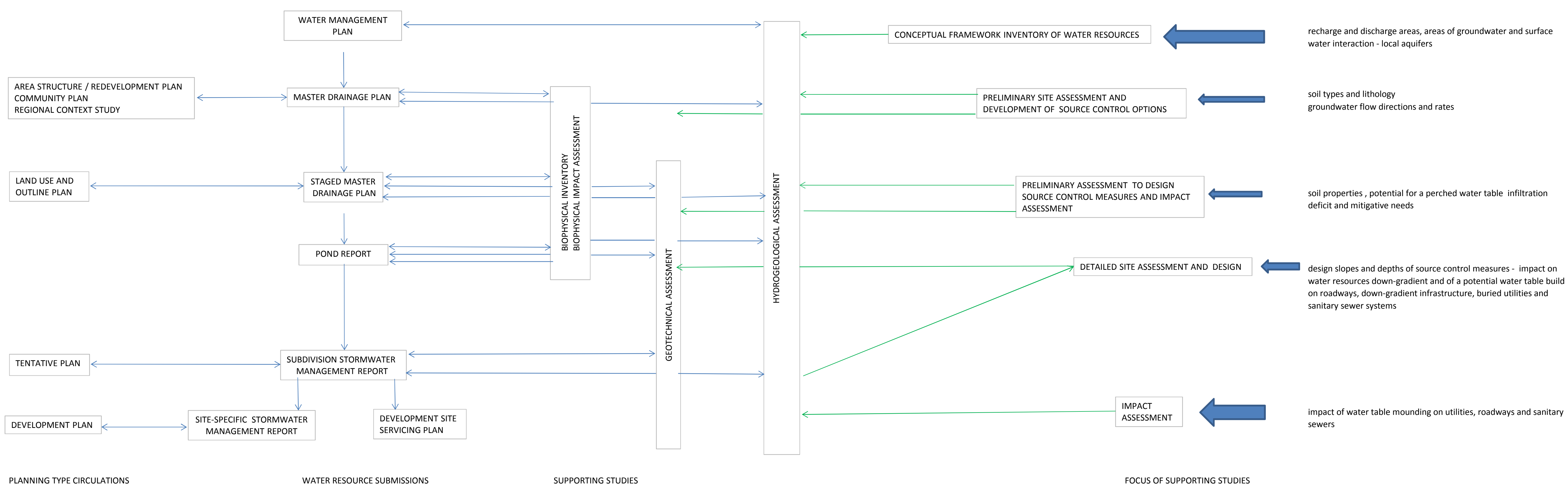
General Overview of Issues, Planning Level, and Scope of Activity

PROJECT NO.	DWN	CKD	APVD	REV
C12101310	AL	MCG	TD	0

OFFICE	DATE
EBA-Calgary	June 2013

Figure 2.1

Figure 2.2 - Stormwater Drainage Planning Levels and the Geotechnical and Hydrogeological Components



APPENDIX A

METHODS TO ESTIMATE GROUNDWATER FLOW RATES AND DIRECTIONS

CITY OF CALGARY MODULE 1 GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

TABLE OF CONTENTS

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A1.0 INTRODUCTION

This appendix provides an overview of methods to determine groundwater flow directions and rates to support Low Impact Development (LID) within The City of Calgary (The City). Developers and their consultants, submitting plans to The City for approval, will find the guidance provided here useful to ensure stormwater management features are positioned within a development in a manner consistent with LID principles.

Different stormwater drainage planning levels have different requirements for accuracy and detail in estimating groundwater flow directions and rates. For example, at the Watershed Plan (WP) planning level, the slope of the land, typically replicated by the water table slope, may be a sufficient estimate of the shallow groundwater flow direction. Therefore, the groundwater flow direction can, at the WP planning level or the Master Drainage Plan (MDP) planning level, be estimated from a desktop review of topographic maps or a site walk-over. However, for the design needs of Staged Master Drainage Plans (SMDPs), Pond Reports, Stormwater Management Reports (SWMRs), or Development Site Servicing Plans (DSSPs), site-specific data on water levels and soil hydraulic conductivity are needed that can only be determined by on-site testing.

In this document, guidance is provided on both the desktop and site-specific testing needed to obtain estimates for the hydraulic gradient and hydraulic conductivity. Table A-1 provides advice on the guidance to follow for each planning level, and an index of where the method is presented within this appendix.

Table A-1: Index to Recommended Estimation Methods

Drainage Planning	Hydraulic Gradient	Hydraulic Conductivity	Reference Section
WP	Desktop review of topographic maps	Soil type literature values	A-3.1 and Appendix H
MDP	Desktop reviews – site topographic surface and site inspection	Soil type literature values	A-3.1 and Appendix H
SMDP	In situ testing, geotechnical assessment, and water level measured at monitoring well installations	Grain-size determination or response testing of monitoring wells	A-3.1 Figures A-1, A-2, and A-3 show typical well completion details and Appendix H with method of installation described in A-3.2 and A-4.3 and illustrated on Figures A-3, A-4, and A-5
Pond report	Water levels from monitoring well installations	Monitoring well response tests	Described in Sections A-3.2 and A-4 with illustrations on Figures A-5 and A-6.
SWMRs and DSSPs	Water levels from monitoring well installation	Monitoring well response tests	Described in Section A-3.2 and A-4 with illustrations on Figures A-5 and A-6

The methods described here are minimum measures. Practicing hydrogeologists may identify additional aspects of the geology and hydrogeology to be assessed as the development proceeds or decide that other in situ testing methods will improve management of stormwater; but those alternatives must be considered on a site-by-site basis and cannot be elaborated here. If submitted as part of the development plan, the detailed methods used by these alternatives need to be provided for review by The City.

Sections A2.0 to A4.0, respectively, describe:

- The need to measure groundwater flow directions and rates;
- The current best practices used to measure the hydraulic gradients and to estimate hydraulic conductivity values; and
- The method proposed for determining hydraulic gradient and hydraulic conductivity within The City for LID projects.

A2.0 NEED TO DETERMINE GROUNDWATER DIRECTION AND RATES

Measurement of the groundwater flow direction and calculation of flow rates is needed to generate a water balance of the groundwater flow onto and off of a development area and to quantify the potential contribution of groundwater to surface water (marshes, ponds, and watercourses) within the development.

Any facility designed to retain and manage (and that potentially promotes infiltration of) stormwater disturbs the pre-development water balance. The intent of LID projects is to either minimize or mitigate this disturbance by ensuring the quantity (and quality) of groundwater discharge to local streams, watercourses, wetlands, or marshes are maintained to a level that does not affect the quality of the natural habitat.

Groundwater flow rates and shallow groundwater discharges to surface waters can be quantified using Darcy's Law, where:

– $Q = KiA$ (expressed in volume per unit time e.g., m^3/day) Equation A-1

Where:

– A, the cross-sectional flow area, is typically determined by subsurface investigations that describe the width and thickness of the geological materials where groundwater flow occurs (it is expressed as units of length by width – area, e.g., m^2).

– i, the hydraulic gradient, is determined by measuring the difference in water level from place to place across a development or towards a wetland, marsh, or watercourse. Contouring of water level elevations determines the slope of the water table surface and provides the hydraulic gradient and the direction of the hydraulic gradient. The hydraulic gradient is defined as the decline in water level elevation over the length of the flow path and is dimensionless (m/m).

– K, the hydraulic conductivity, measures the ease of groundwater passage through a subsurface geologic unit and is expressed in units of length per time, e.g., typically as either m/day or m/s.

As a side note, Darcy's Law is often expressed as:

– $q = Q/A = Ki$ ($m^3/day/m^2$ or m/day when K is expressed in m/day) Equation A-2

Where:

– q is often called specific discharge – the volume per specific square metre of the flow system or the "Darcy Flux". Often, but most frequently in studies of groundwater contamination, expression of this form of Darcy's Law is used to determine the velocity of groundwater.

$$v_l = q/n = Ki/n, \quad \text{Equation A-3}$$

Where:

– v_l is the average linear groundwater velocity m/day.

– n is the porosity (dimensionless).

– K and i are as defined previously.

In this form, the groundwater velocity (Fetter 1992, Canadian Council of Ministers of the Environment [CCME], 1994) is used to estimate the velocity of a dissolved contaminant and predict the average time of arrival of a contaminant mass at a particular point in the flow system; a requirement that is really not applicable to LID projects. Therefore, all subsequent references of groundwater quantity or flow rates used here, either refer to Q (m^3/day – volume/unit time), or q – specific discharge – volume per unit time per unit area, and shortened to m/unit time.

Temporal changes in hydraulic conductivity are not likely in a natural system. The pre-development and post-development hydraulic conductivity of the flow system may differ close to stormwater management features or other engineered structures, where the soil properties have been deliberately altered (either to enhance infiltration or inhibit infiltration as the situation allows). However; and predominantly post-development, the biggest change to the groundwater flow rate can be expected to be due to the changes in the hydraulic gradient (i).

The potential for changes in the hydraulic gradient between pre-development to post-development conditions need to be considered in all water balance calculations near wetlands, marshes, and watercourses. Of single most importance in characterizing changes in groundwater flow systems for LID is to determine the tendency for changes in the vertical hydraulic gradient.

Upward vertical hydraulic gradients provide discharge of groundwater supporting the base flow to local surface waterbodies. Changes in the quantity of groundwater discharge may disrupt watercourses and create greater potential for erosion, and hence, damage to the watershed. These impacts need to be evaluated as described in Appendix D.

Downward vertical hydraulic-gradients identify recharge areas where infiltrating rainfall or snowmelt provides a water source to local and regional flow systems, or causes groundwater recharge where none had occurred previously. Increases in this circumstance may produce a "groundwater mound" on the pre-existing water table or create perched water tables above low permeability geologic materials. Mounding is discussed further in Appendix G.

Changes in the magnitude of the hydraulic gradient may need to be mitigated or minimized for the development to proceed.

A3.0 CURRENT BEST PRACTICES

This section describes the best practices used by groundwater practitioners world-wide to estimate and measure the hydraulic gradient and the hydraulic conductivity. Equation A-1, the equation that is most useful to quantify groundwater flow rates, shows how these two unknowns, the hydraulic gradient (i) and the hydraulic conductivity (K), relate to the groundwater flow rate and quantity.

For most purposes, LID is concerned with the influence of stormwater management features on the water table or in the zone above the water table and not with respect to the more deeply seated geologic flow systems. Unless explicitly stated otherwise, all information described here should be considered as applying to the water table and the overlying unsaturated sediments.

A3.1 Measuring the Hydraulic Gradient

Table A-1 identified that measurements of the hydraulic gradient and direction of groundwater flow can be obtained either:

- By measuring the slope of the ground surface; or
- By direct measurement using monitoring wells installed specifically to measure the groundwater elevation.

The water table is traditionally taken to be a subdued reflection of the surface topography, and in a regional sense, this analogy provides a reasonable estimate of the gradient and shallow groundwater flow direction. Consequently, at the WP and the MDP planning levels, the slope of the ground surface as obtained from topographic maps or a site walkover, provides a reasonable first estimate of the hydraulic gradient.

But the water table depth converges to the ground surface in the vicinity of rivers, streams, wetlands, fens, and marshes. Similarly, groundwater flow near these features is highly influenced by seasonal climatic events. As a result, it is necessary to quantify flows and direction within 6 m to 10 m of these features by installing monitoring wells to measure water levels directly. This level of detail is required at the SMDP, Pond Report, or SWMR and DSSP levels.

Also, at the SMDP, Pond Report, and subdivision planning levels (SWMR and DSSP), it must be appreciated that the hydraulic gradient occurs in two directions, horizontally along permeable strata, and vertically between permeable geologic units isolated by lower permeability geologic strata. To determine the tendency for horizontal and vertical flow it is, therefore, necessary:

- To install horizontally spaced wells with monitoring intervals placed in the same geologic unit; and
- To install vertically spaced wells with monitoring wells installed at different depth intervals.

Differences in the direction of vertical gradients downward or upward, respectively, define, recharge, or discharge areas across a development and may need to be preserved as the development proceeds, or in some circumstances, may need to be avoided for ease of construction.

A3.2 Estimating the Hydraulic Conductivity

Table A-1 identified the methods proposed for estimating the hydraulic conductivity depending upon the drainage planning level.

To estimate the hydraulic conductivity below the water table, Table A-1 recommends using either:

- Estimates based upon the anticipated soil type or soil texture (grain size distribution); or
- In situ testing, (either pumping tests or monitoring well response tests).

Estimates for hydraulic conductivity can be based upon a soil texture as determined from the grain size distribution. These estimates are typically made during a desktop review of the development site's condition using published reports or the findings from a site inspection. This estimation method is suitable at the WP or MDP planning levels. Appendix H provides advice for using these methods. At the SMDP, Pond Report, or SWMP or DSSP levels, in-situ testing is needed.

Hydraulic conductivity values can be estimated by laboratory testing of soil cores which are described in Appendix C with the preliminary and detailed geotechnical testing procedures. These methods primarily estimate the vertical hydraulic conductivity, not the horizontal hydraulic conductivity; therefore, these methods are better suited when confirming a pond design or performance of containment features as described in Section 3.1. The method preferred for quantifying horizontal groundwater flow rates and most useful to the water balance of ponds, wetlands, marshes, and watercourses are in situ testing methods of pumping tests or monitoring well response tests.

Pumping tests are not used as frequently as monitoring well response tests, and are most applicable for testing of aquifer systems for potable water supplies. Therefore; the most applicable methods for LID purposes and the most broadly used methods in practice across North America are the single-well monitoring well response testing methods (slug tests).

Using single-well response tests to measure the hydraulic conductivity will require a minimum of three tests for each water-bearing stratigraphic unit. For example, say the stratigraphy beneath of the site (defined from the soil characterization methods of Appendix C) consists of till, sand, and weathered bedrock. It is suggested that three monitoring well response tests be conducted in each layer (i.e., a total of nine tests). Section 3.1 describes how this recommendation may be scaled for the size of the LID.

Recent advances in cone penetration testing (CPT) indicate that this type of equipment can provide in situ estimates for the hydraulic conductivity, also. This equipment, however; is not as commonly used on LID projects as traditional methods of monitoring well standpipe installation for response testing and is not discussed further in this document.

A4.0 METHODS TO MEASURE THE HYDRAULIC GRADIENT AND HYDRAULIC CONDUCTIVITY

The methods to estimate the groundwater flow direction and the rate of groundwater flow described here refer to in situ testing methods, applicable at the SMDP, Pond Report, or SWMR or DSSP planning levels.

Sections A4.1 to A4.4, respectively, describe:

- The site information needed to measure the direction and rate of groundwater flow;
- The construction of monitoring wells to measure water levels and to conduct single-well response tests;
- The method to conduct and analyze monitoring well response tests; and
- The reporting requirements.

A4.1 Site Information Requirements

The following information is the minimum information needed to ensure the measured hydraulic gradient and hydraulic conductivity values support the implementation of the desired LID features:

- Topographic maps or plans for the development area;
- Maps of the distribution of marshes, wetlands, and surface watercourses;
- As obtained from geotechnical investigation;
  - Stratigraphic units;
  - Depth to the water table; and
- Elevation of each borehole and monitoring well.

A4.2 Monitoring Well Construction

Monitoring wells are constructed in boreholes drilled to investigate the soil and groundwater conditions during the geotechnical investigation. Monitoring wells have two major physical components:

- A standpipe consisting of a length of unslotted casing (steel, or polyvinyl chloride [PVC]), and extended from the top of the well screen to the ground surface for ease of access for water level measurement or water quality sampling; and
- A well screen consisting of a length of slotted casing (either steel or PVC), and placed either at the base of the borehole or across a water-bearing geologic unit, where a water level or water quality sample is to be collected.

The two components are joined by a threaded coupling at the screen's upper end. The screen is designed to be in good hydraulic communication with the geologic material in which it is placed, and is intended to block infiltration of sediment to the monitoring well casing. The screen's length is selected to extend across a water-bearing unit but may be less than the full unit thickness providing that hydraulic communication with the geologic material being tested is not impeded. The depth of the screen for water table monitoring purposes is selected to site astride the depth of the water table discovered as the borehole is advanced. A piezometer is a type of monitoring well in which the screened area may only target a small portion of a geologic formation, but is placed at some depth below the apparent water table and in some situations is not necessarily constructed to allow for water quality sampling (e.g., a vibrating wire piezometer).

Monitoring wells constructed and placed at strategic locations are used to measure water levels across a development and, using the water levels, to determine the slope and magnitude of the hydraulic gradient.

Because the water table is a sloped surface, a minimum of three monitoring wells are required to define the direction of the slope and the hydraulic gradient. These wells should be placed at least 10 m apart to accurately measure the water table slope. However; on larger development sites, this close spacing may be prohibitively expensive and not warranted. Recommended spacing in wells is discussed in Section 3.0 and reiterated here.

Development Area	Recommended Minimum Number of Wells	Recommended Monitoring Well Response Tests[1]
Less than 1 Ha	3 (but more than 10 m apart)	3
1 Ha to 10 Ha	3 to 12 (spacing of 300 m)	12
1o to 100 Ha	12 to 20 (spacing of 300 m)	20
Greater than 100 Ha	20 or more wells spaced at 300 m intervals	20+

More wells may be required in areas of changeable terrain or where greater accuracy of the groundwater flow rates is needed (such as close to wetland, marshes, or watercourses where an adverse impact is unacceptable and that are to be protected during the development).

Multiple level wells, which are located in close proximity but with a screened section placed at different geologic horizons, are used to measure the vertical hydraulic gradient and have the same essential components as water table monitoring wells.

Typical monitoring well construction details for a water table well and a set of multiple level wells are shown on Figures A-1 and A-2, respectively. Information needed to document the well completion details is also shown on these figures.

At locations where access by wheel or tracked drilling equipment is not practical (i.e., close to wetlands), drive point wells can be installed. These wells do not require a borehole to be drilled, and are commonly steel. Figure A-3 shows a typical drive point construction.

The following six steps describe the sequence of the activities typically followed to construct and prepare a monitoring well for monitoring water levels and response testing.

Step 1: Drilling

Monitoring wells are constructed in boreholes created using auger drilling equipment or rotary drilling equipment, or are sometimes forced into place (e.g., a "drive point" well). These boreholes would be drilled as part of the geotechnical investigation conducted for SMDP, Pond Report, or SWMR and DSSP planning levels.

The depths of the boreholes should be no deeper than 2 m below the depth of the water table as observed during drilling for water table monitoring wells. For piezometers or monitoring wells placed to monitor water levels in a saturated zone or geologic horizon of interest some depth below the water table, the depth of borehole must be sufficient to intersect the horizon, but no deeper, to avoid the chance of opening a conduit for water movement between geologic units that are naturally hydraulically isolated.

Step 2: Installation of a Monitoring Well or Piezometer

Monitoring wells can be constructed using various materials. The most common construction material used is PVC pipe, generally 25 mm to 50 mm in diameter. The well screen is placed in the bottom of the borehole and a length of well casing is attached to the screen and extended to the ground surface for ease of water level measurement or water quality sampling. Screens and solid PVC sleeves are available commercially in different lengths and diameters, and can be fit together as required to suit the drilling depth. These materials are inexpensive, versatile, and are at low risk to affect groundwater quality testing.

The annular space between the wells of the borehole and the monitoring well casing within the section of the screen is backfilled with filter sand. Filter sand needs to be selected to be compatible with the slot size selected for the well screen. The filter sand is backfilled above the well screen. A good rule of thumb is the filter sand should extend above the well screen to a depth of about one-third of the length of the screen. This depth of backfill allows for settlement of the filter sand.

The screened section and its filter sand are isolated in the borehole by placement of a seal of bentonite pellets on top of the filter sand. The thickness of bentonite pellets is generally 2 m but may be greater. The remainder of the annular space to ground surface can be backfilled with a good quality material to maintain isolation of the well screen. A cement bentonite grout is preferred but in some circumstances, drill cuttings may be adequate.

Mounding of the backfill around the well casing at the ground surface helps to avoid entry of surface water to the well bore. A steel protective casing with a lock should be fitted around the well casing at the ground surface to guard against accidental damage or vandalism.

Micro or mini-piezometers are often used in proximity to sensitive surface water environments to minimize disturbance of riparian environments by tracked or wheeled equipment. The drive point monitoring well on Figure A-3 is a type of micro or mini-piezometer.

Step 3: Well Development

Well development refers to the process of removing fine-grained material from newly installed monitoring wells with either water or air in order to improve the hydraulic communication between the well and the adjacent geologic unit where the groundwater conditions are to be monitored. Well development shall proceed until the water flushed from the well is clear or at least free of large diameter particles (visible to the eye). Development time will vary with the type of geologic material adjacent to the well screen.

Step 4: Monitoring Well Recharge

After a well has been developed, the water level within the well is not reflective of the natural water table but requires a period of rest until equilibrium with the surrounding formation is reached. For wells completed in sands and gravel, equilibrium can be reached in minutes, while in fine-grained materials achieving equilibrium can take several days to months. To measure equilibrium, repeated periodic measurements of the depth to water are needed until a few millimeters of change is measured between successive measurements. The well should be allowed to recharge to its pre-development water level before hydraulic conductivity tests are carried out.

Step 5: Reference Elevation

After development of the wells, they should be surveyed to establish ground surface and top-of-the-well casing elevations; a convenient local benchmark or geologic benchmark are good reference points. Global positioning system (GPS) elevations may not be sufficiently accurate (sufficient accuracy varies from place to place but generally in the 5 mm to 10 mm is sufficient for most practical purposes).

A survey should also be conducted to establish the geo spatial northing and easting elevations of each well. GPS surveys to ± 0.5 m are often sufficient for these spatial surveys.

Step 6: Groundwater Level Measurements and Monitoring

Once the well is equilibrated, groundwater level measurements can be taken and used to calculate hydraulic gradients. Figure A-4 shows a typical hydraulic gradient calculation. Water levels are measured from a reference point at the top of the casing for consistency in measurements. Electric water level tapes are the commonly used measurement device. Groundwater levels should be recorded and tracked periodically (three to four times per year) throughout the site development. Water levels that have been monitored and recorded over time can be compared to historic and recent local precipitation records to determine whether water table fluctuations are a reflection of a change in the hydraulic regime, such as the disturbance created by a surface development, or whether they can be attributed to climatic factors.

A4.3 Monitoring Well Response Testing

A single-well response test involves instantaneously lowering or raising the level of water in a monitoring well standpipe, with a screened interval placed across the geologic unit that we desire to know the hydraulic conductivity of. Either bailers to remove water or dumping a known volume of water in the well casing are typical means used to lower or raise the water level in the well. However; neither method is instantaneous. Therefore; the slug test method is preferred (American Society for Testing and Materials [ASTM] 1996a and ASTM 1996b).

The slug test method involves dropping a slug (metal) of known volume below the water level in a well casing, and measuring the water level as it declines in the well (falling head test). After the falling head test, the slug is removed from the well and the water level is measured as it rises in the well (rising head test). Use of the slug allows both tests to be conducted with a single test setup. The water level measurements can be made manually at periodic intervals using a water level tape or with a data logger system using a downhole pressure transducer. Both falling and rising head tests are recommended because they measure two directions of flow through the well screen, outward and inward, respectively. Clogging of the well screen or the well bore may be detected if both tests are applied and a discrepancy between the two values will alter the opinion on the reliability of the hydraulic conductivity measurement.

After the water level in the well recovers to a reasonable level (i.e., the Hvorslev analysis cites a recovery of a reasonable level is 63% of its pretest level), the test can be considered sufficient to estimate the hydraulic conductivity. Methods for calculating the hydraulic conductivity vary depending upon the geologic structure. ASTM (1996c) provides these alternative calculation methods. An example using the Hvorslev method is provided on Figure A-5. A typical tabulation of the record keeping needed for a single monitoring well response test using the manual measurement method is provided in Table A-2.

Records made during the tests are the depths to water converted to a head measurement (H) above or below the static water level, or the pre-test water level, and the elapsed time (since the test started) when the water level measurement was made. The initial head at the start of the test is equal to the water level or head achieved by the addition of the slug's volume to the well. A simple conversion of the slug volume to water level for a typical PVC standpipe is provided on Figure A-5.

Figure A-6 also illustrates the water level and time information to be recorded during each type of test. However; in fine-grained materials, such tests may take several days if not months, and it may not be practical to undertake both tests. Poor well development and a well in poor hydraulic communication with the geologic materials surrounding the boreholes often results in poor estimates for the hydraulic conductivity. Tests that produce curved plots of water level versus time may need to be repeated. Usually, further development of the well improves the hydraulic communication between the well and geologic material. But in some cases, a new monitoring well installation may be needed.

Most hydrogeologists use computer aided software to calculate the results of falling or rising head test results. The manual method produced here is provided for ease of reference and for the developer or their consultant as a means of confirming the software's calculations. It is suggested that manual calculations be used to confirm the software's calculation for one test in five completed on a development site. The rationale being that such tests are often undertaken by field staff or junior professional staff, and the tendency in that circumstance is to feed the software but not necessarily to confirm the site's hydrogeologic conditions.

Table A-2: Monitoring Well Response Test Record Keeping

Date:________________ Location:__________

- Static or pre-test water level depth___________ m
- Volume of slug (V) __________ m^3 $Ho = \frac{V}{\pi r^2}$ = ______m where "r" is the radius of the well casing

Figure A-6 illustrates the convention for determining h_1, h_2, and h_3, etc., during the test.
Falling Head Test Rising Head Test

Time s = seconds m = minutes	Depth to Water (m)	Ht – Height of Water above the Pre-Test Level	Ht – Height of Water below the Pre-Test Level
t_0 0	d0	Ho	Ho
$t_{1\ =\ 15\ s}$	d1	h_1	h_1
$t_{2\ –\ 30\ s}$	d2	h_2	h_2
$t_{3\ =\ 45\ s}$	d3	h_3	h_3
t_4 = 60 s	d4	h_4	h_4
t_5 1 m 30 s	d5	h_5	h_5
t_6 = 2 m	d6	h_6	h_6
$t_{7\ =\ 3m}$	d7	h_7	h_7
t_8 = 4 m	d8	h_8	h_8
t_9 = 5 m	d9	h_9	h_9
t_{10} = 7.5 m	d10	h_{10}	h_{10}
t_{11} = 10 m	d12	h_{11}	h_{11}
$t_{12\ =\ 12.5}$	d12	h_{12}	h_{12}
$t_{13\ =\ 15}$	d13	h_{13}	h_{13}
t_{14} = 20 m	d14	h_{14}	h_{14}
t_{15} = 25 m	d15	h_{15}	h_{15}
t_{16} = 30	d15	h_{16}	h_{16}

A4.4 Reporting Requirements

To characterize the groundwater flow rate and direction across a development site and particularly in local areas where groundwater and surface water interact, the following information should be provided.

1. Complete record of the monitoring wells installed, showing ground surface elevation, top of casing elevation, drilled depth, screen depth, and well screen completion details.
2. Identification of recharge and discharge areas based upon water level measurements or the site's topography.
3. Water Table Elevation Surface Maps;

   Water table maps drawn to depict the approximate direction of groundwater flow, as well as water level elevations measured in monitoring wells or piezometers. Flow direction is expressed using equipotential lines or elevation contour lines, which are spaced according to hydraulic head differences and oriented perpendicular to the water flow direction.

4. Cross-Sections;

   Flow nets should be created from geologic information and presented in a cross-section view. A flow net is a useful tool when trying to determine which path water is taking within a hydraulic system, and what quantity of water is in movement. There are two types of aquifers a cross-sectional view will aid in illustrating – unconfined and confined aquifer. An unconfined aquifer has a non-permeable layer at the base of the groundwater body. In an unconfined aquifer, the ground-water level is referred to as the water table, and is assumed to be at standard atmospheric pressure. In a confined aquifer, both the upper and lower aquifer surfaces are limited by non-permeable material. If the aquifer is confined, the ground-water level within the aquifer is referred to as a potentiometric surface. When the potentiometric surface is above the upper level of the aquifer, the aquifer is called an artesian aquifer. Potentiometric surfaces that rise above the ground surface are called flowing artesian aquifers. Appendix E – Framework to Develop a Conceptual Site Model provides the details on characterizing aquifers.

5. Hydraulic Conductivity Estimate;

   All monitoring well response test results should be provided along with the calculations for the hydraulic conductivity. Hydraulic conductivity values should be assigned to the geologic units shown on the cross-section. To obtain a "typical value" for a particular geologic unit, the geometric mean of multiple hydraulic conductivity tests on monitoring wells should be calculated. The geometric mean is the average of the logarithmic value of the individual hydraulic conductivity values made at multiple locations in the same hydrogeologic unit (see also Section 3.2.2).

6. Groundwater Flow Rates;

   Groundwater flow across the site should be estimated from the contoured elevations using Darcy's Law, the geometric mean of the hydraulic conductivity value, and the cross-sectional area of the water-bearing formation. These calculations should be provided for all surface water areas, wetlands, or other water feature within a development.

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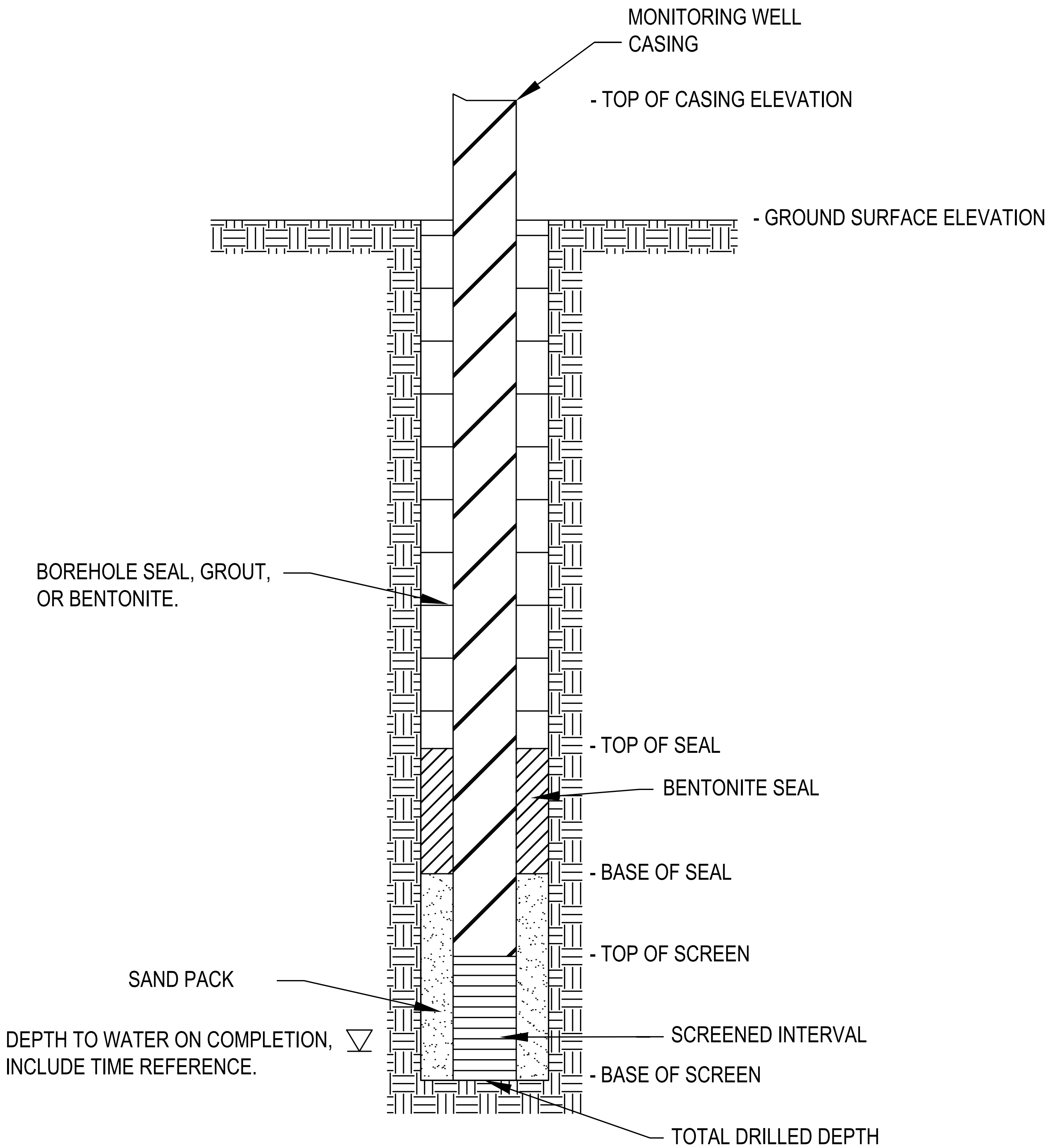
FIGURES



NOTES
DRAWING NOT TO SCALE.

CLIENT
THE CITY OF CALGARY

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A TETRA TECH COMPANY

CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM

WATER TABLE WELL COMPLETION

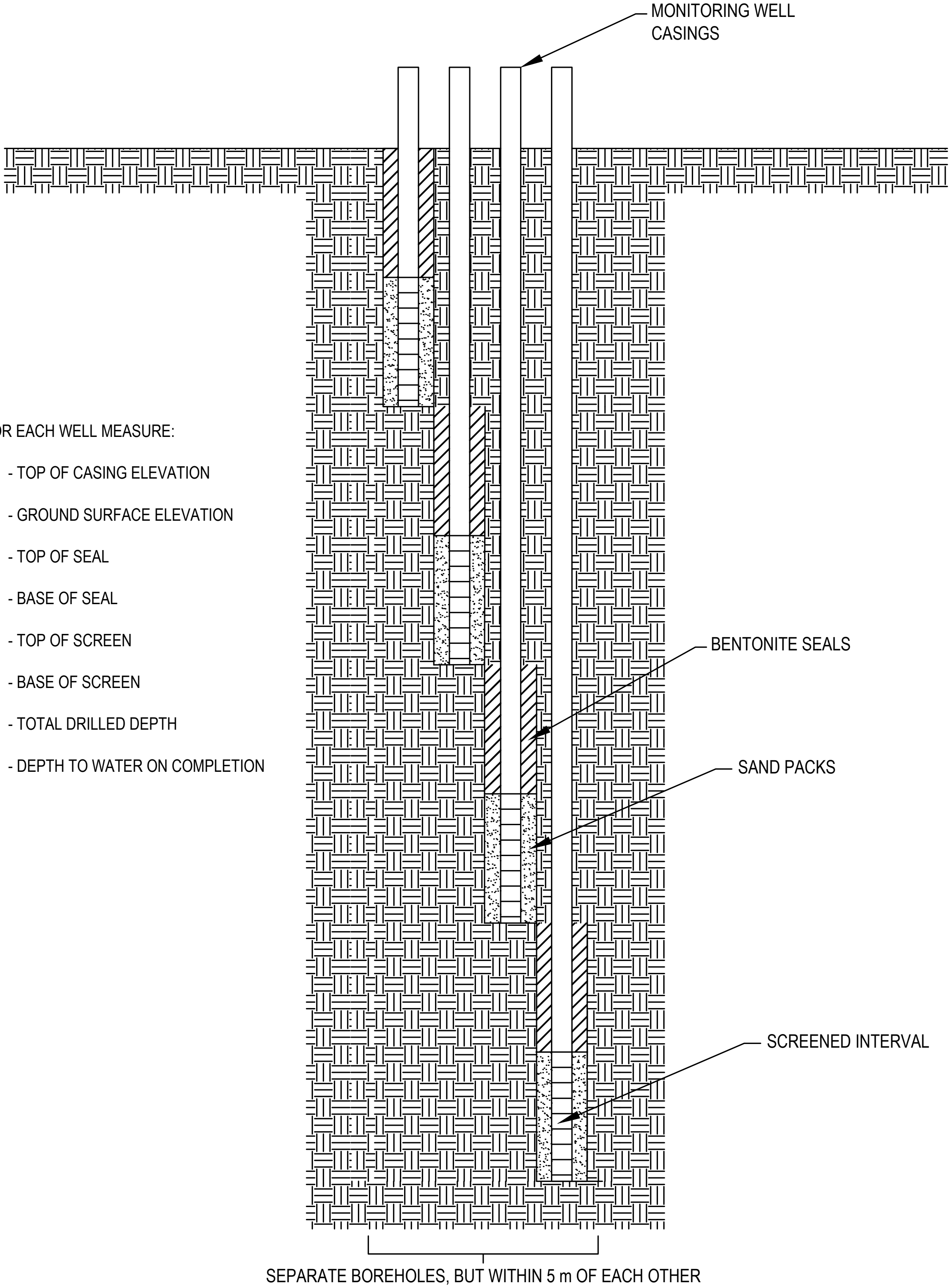
PROJECT NO.	DWN	CKD	REV
C12101310.003	MMK	TD	0

OFFICE	DATE
EBA-RIV	February 2012

Figure A-1

FOR EACH WELL MEASURE:

- TOP OF CASING ELEVATION
- GROUND SURFACE ELEVATION
- TOP OF SEAL
- BASE OF SEAL
- TOP OF SCREEN
- BASE OF SCREEN
- TOTAL DRILLED DEPTH
- DEPTH TO WATER ON COMPLETION



NOTE:
DRAWING NOT TO SCALE.

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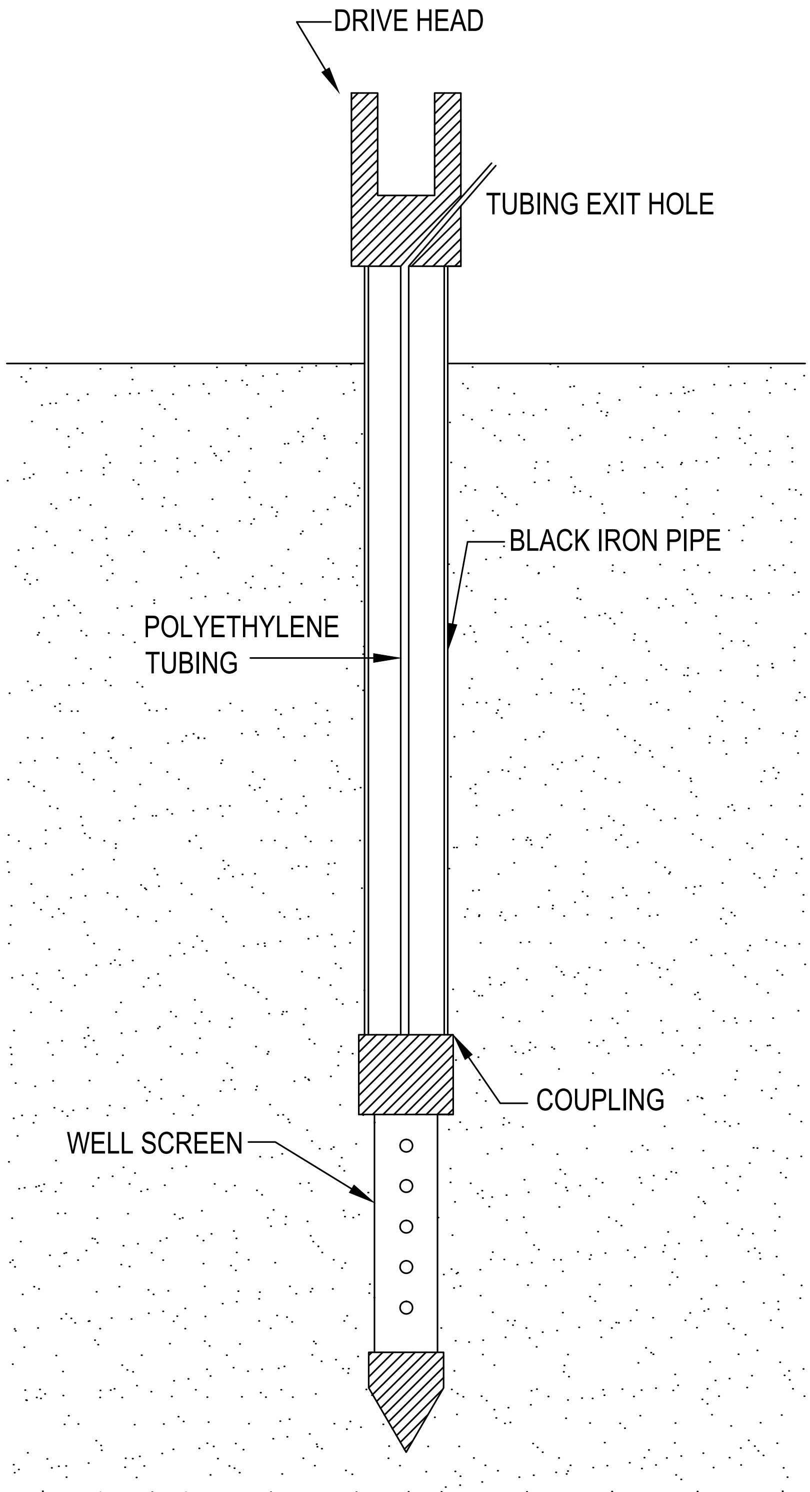
CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM

MULTIPLE LEVEL WELL COMPLETION

PROJECT NO.	DWN	CKD	REV
C12101310.003	MMK	TD	0
OFFICE	DATE		
EBA-RIV	February 2012		

Figure A-2

Q:\Riverbend\Drafting\C121\C12101310\Figures 1-6\C12101310 Figures(P2).dwg [FIGURE A-3] February 16, 2012 - 9:56:09 am (BY: MACKAY, MATT)



CLIENT

THE CITY OF CALGARY

CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM

TYPICAL DRIVE POINT CONSTRUCT

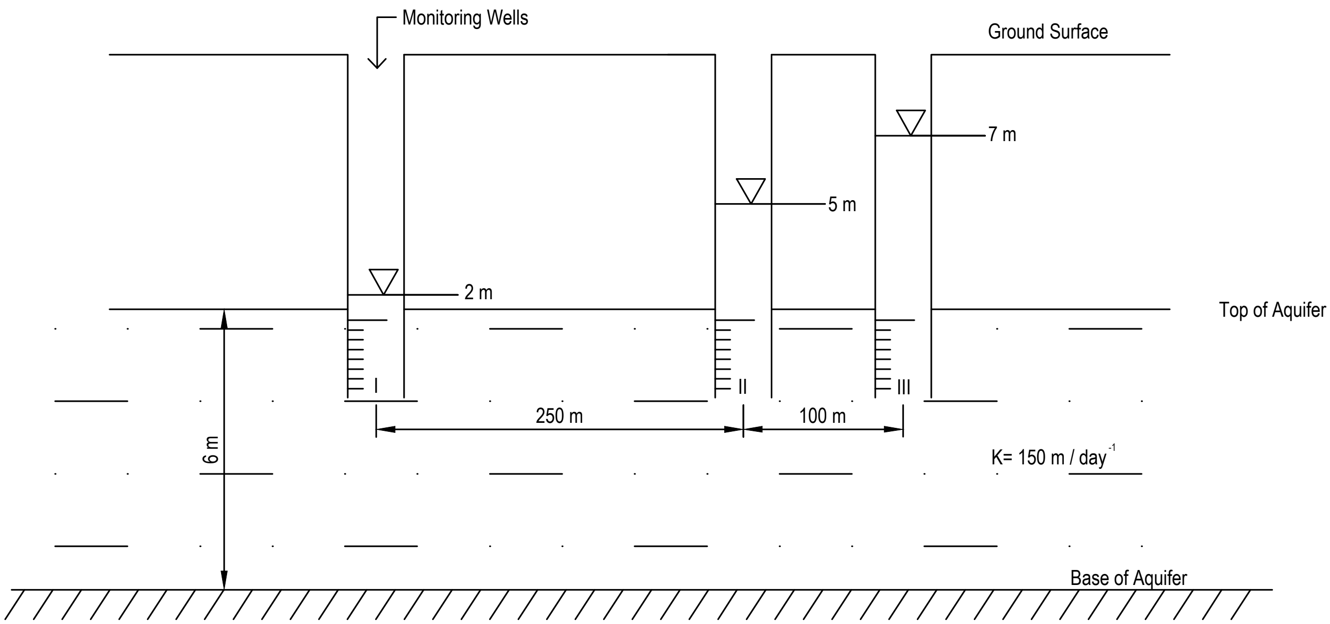
eba A TETRA TECH COMPANY

PROJECT NO.	DWN	CKD	REV
C12101310.003	MMK	TD	0

OFFICE	DATE
EBA-RIV	February 2012

Figure A-3

Horizontal Flow



$$i = \frac{dh}{dl} = \frac{(7\text{ m} - 2\text{ m})}{(250\text{ m} + 100\text{ m})} = \frac{5\text{ m}}{350\text{ m}} = 0.014 \quad ; \ k$$

$$A = 6\text{ m}^2 \text{ (unit width)}$$

$$Q = A \times \frac{dh}{dl} \times K = 6\text{ m}^2 \times 0.014 \times 150\text{ m / day}^{-1}$$

$$Q = 1.06\text{ m}^3\text{ day}^{-1}$$

After: Hydrogeology Principles and Practice,
Kevin Hiscock, 2005.Pg 33.

NOTES
DRAWING NOT TO SCALE.

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CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM

WATER LEVEL MEASUREMENT AND HYDRAULIC GRADIENT

PROJECT NO.	DWN	CKD	REV
C12101310.003	MMK	JB	0

OFFICE	DATE
EBA-RIV	February 2012

Figure A-4

Hvorslev's Method

assumes: homogeneous, isotropic, inifinite medium, soil and water are incompressible.

$$K = (r^2 \log_e(L/R)) / 2LT_0$$

$T_0 =$ $t - t_0$

Developed for a variety of well symmetries

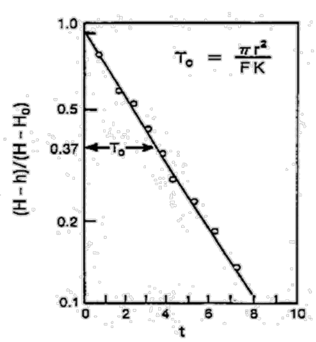
Can be applied to confined or unconfined aquifers

Technique of Analysis

1. Measure static water level from a reference point (top of casing).

2. Calculate the water level after insertion of the slug (H_o), use this as the water level for time 0.

3. Plot $\frac{h}{h_o}$ Time on semi log plot.



2) Draw straight line through data.

3) Determine T_o at $\frac{h}{h_o} = 0.37$

4) Calculate K using T_o, r, L, R

If $\frac{L_e}{R} > 8$

then use following formula:

$$K = (r^2 \log_e(L/R)) / 2LT_0$$

Variable:

$r =$	0.05
$r^2 =$	0.0025
$L =$	3
$R =$	0.05
$T_0 =$	4
$t =$	4
$t_0 =$	0

as seen on the graph below

K = 0.000426494

$r =$ radius of well casing

$R =$ radius of well screen

$L_e =$ length of well screen (m)

$T_o =$ time required for water to rise or fall

Table 1:

Conversion of Slug Volume to Head Rising (H_o)

$d =$	slug diameter
$\ell =$	slug length
$\pi (d/2)^2 * l =$	volume
V of Slug / $\pi r^2 =$	H_o

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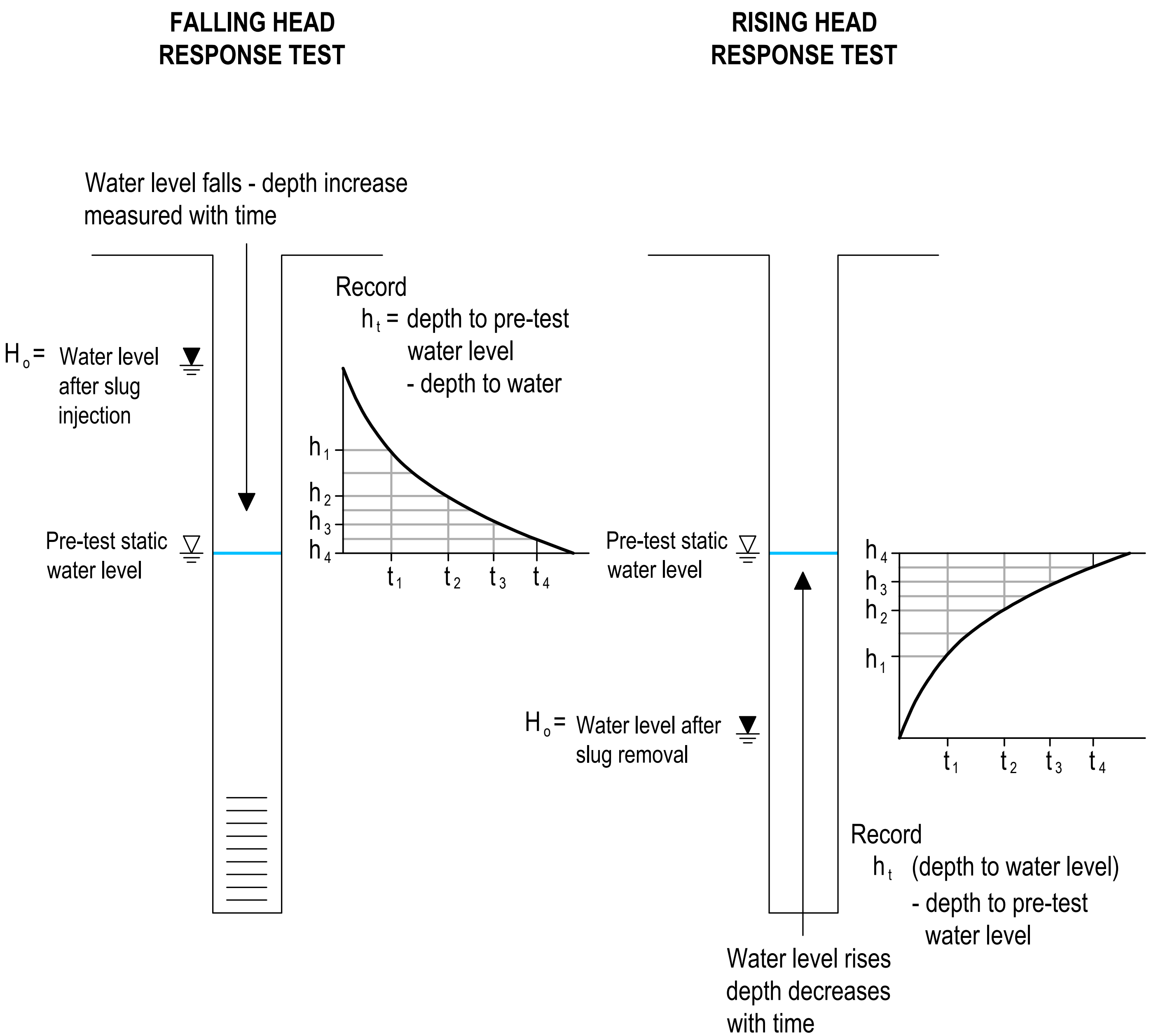
CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM

METHOD FOR ESTIMATING HYDRAULIC CONDUCTIVITY

PROJECT NO.	DWN	CKD	REV
C12101310.003	MMK	TD	0
OFFICE	DATE		
EBA-RIV	February 2012		

Figure A-5



CLIENT THE CITY OF CALGARY		**CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM** **FALLING AND RISING HEAD RESPONSE TEST**
eba A TETRA TECH COMPANY	PROJECT NO. C12101310.003 / DWN BM / CKD TD / REV 0 OFFICE EBA-RIV / DATE June 2012	**Figure A-6**

APPENDIX B

METHODS TO ESTIMATE INFILTRATION AND PERCOLATION RATES CITY OF CALGARY – MODULE 1 GEOTECHNICAL AND HYDROGEOLOGICAL CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

TABLE OF CONTENTS

FIGURES

B1.0 INTRODUCTION

This appendix provides guidance on methods to estimate infiltration and percolation rates for application to Low Impact Development (LID) projects. Developers and their consultants will find this information useful to determine the area needed within a development to either:

- Provide enhanced recharge to the groundwater to replace the recharge lost by the build-up of non-pervious areas within the development; or
- Utilize infiltration and percolation processes to help stormwater management practices meet runoff target volumes.

Water, infiltrating or percolating into soil, increases the soil's water content within the unsaturated zone. Changes in the soil's water content have no other consequence for LID except to return water to the hydrologic cycle. Therefore; methods to evaluate soil water content changes are not considered further in this document. However; when infiltration or percolation occurs, an increase in the elevation of the water table may occur. Changes in the elevation of the water table can influence the selection of a source control practice (SCP). Methods to evaluate the buildup of the water table or perched water tables in the unsaturated zone are described in Appendix G.

The need to accurately measure the rate of infiltration and percolation and to understand the variation in the rate from place to place within a development varies with the level of drainage planning being undertaken. The measurement methods described here can be categorized as either:

- Indirect methods – based upon an understanding of the soil type or differences in the soil texture (grain size distribution) and the variation thereof across the development; and
- Direct methods – based upon in situ measurement of infiltration and percolation rates using either surface or subsurface in-situ testing methods.

Table B-1 provides an index of the estimation methods to be used for each level of drainage planning and describes where those methods are contained in this appendix or other sections of this module.

As described in Section 3.0, the method selected to estimate infiltration and percolation rate should be based upon the stage of development:

- Site assessment;
- Stormwater management feature selection;
- Detailed design; and
- Performance verification.

Surface measurement of infiltration, for example, may be undertaken to test the infiltration through the base of a stormwater retention pond, bio-retention area/bioswale, French drain, or rain garden after construction to assess or verify the hydraulic performance of a constructed stormwater management feature. Subsurface infiltration or percolation rates are more frequently employed to measure the rate of percolation beneath the proposed base of a stormwater retention pond, bio-retention/bioswale, French drain, or rain garden to select a) locations for placement of these control features within a development; b) to select an appropriate design concept or c) to select a preferred storm water management feature.

Table B-1: Index to Recommended Estimation Methods for Infiltration and Percolation Rates

Drainage Planning	Surface Infiltration	Percolation	Reference Section
Watershed Plan (WP)	Desktop review of soil types	Soil type literature values	B-3.1 and Appendix H
Master Drainage Plan (MDP)	Desktop reviews of soil types or site walkover to sample soil.	Soil type literature values or soil textural analysis	B-3.1 and Appendix H
Staged Master Drainage Plan (SMDP)	Soil textural analysis grains size distribution and in situ testing using Guelph permeameter or double ring infiltrometer. American Society for Testing Materials (ASTM) D3385 and ASTM D5093-02.	Grain-size determination or in situ testing using Argue's methods or ASTM D6391-11	Description in B-3.1 and Appendix H Description in B-4.1 and B-4.2 for in situ testing methods
Pond Report	Surface double ring infiltrometer testing ASTM D3385 and ASTM D5093-02	Argue's methods or ASTM D6391-11	Description in Section B4
Stormwater Management Reports (SWMRs) and Development Site Servicing Plans (DSSPs)	Guelph permeameter or surface double ring infiltrometer testing ASTM D3385 and ASTM D5093-20.	Argue's methods or ASTM D6391-11	Description in B3.2 with examples

Sections B-2.0 to B-4.0:

- Define infiltration and percolation rates used in this module;
- Describe the current state of practice in North America and the method proposed for use within The City of Calgary (The City) for LID; and
- Summarize the analytical testing methods for the proposed methods.

B2.0 DEFINING INFILTRATION AND PERCOLATION RATES

In this module, infiltration refers to the movement of water through a soil surface, either the ground surface or the base of a stormwater management feature, such as, but not necessarily limited to, surface water retention ponds, rain gardens, French drains, absorbent landscapes, or infiltration galleries. Measurement of the surface infiltration rate is needed at the WP and MDP level to determine the baseline conditions and the quantity of infiltration potentially lost to the hydrologic cycle from non-pervious areas of the proposed development. Surface infiltration rates are needed to confirm the performance of stormwater management feature measures constructed for the SMDP, Pond Report, or subdivision reporting at the SWMR and DSSP planning levels.

Percolation refers to vertical movement of water in the unsaturated soil below the soil surface but above the water table and above the zone of 100% water saturation created by the capillary fringe.

Percolation rates are required to select locations and the appropriate stormwater management features within a development and to support a detailed design of the stormwater management features. Percolation rates are needed for the soil beneath the proposed excavation depth of the stormwater management features at the SMPD, Pond Report, and subdivision (SWMR and DSSP) planning levels.

Figure B-1 shows schematically where the two terms are most aptly applied.

The rate of infiltration and percolation are related to the soil's saturated hydraulic conductivity. The convention used here is that the infiltration or percolation rate refers to the fully saturated hydraulic conductivity and not the partially saturated or unsaturated hydraulic conductivity. Most investigators or researchers (documented in Section B-3) make achieving a state of 100% saturation a condition of in situ tests to measure infiltration and percolation rates. Section 3.2 describes the safety and uncertainty factors to be applied to the infiltration or percolation rate at locations where less than 100% saturation of the soil is expected.

The measures recommended here are minimum measures to estimate the saturated hydraulic conductivity (henceforth referred to as the hydraulic conductivity). Other practitioners may propose alternatives (including the case for unsaturated hydraulic conductivity values) as a more appropriate measure than the saturated hydraulic conductivity. If alternatives are used, the applicant is requested to provide the detailed methodology and the rationale for its application with the submission.

Most researchers use the terms infiltration and percolation interchangeably, although, in any particular packet of research, only one term or the other term is used. Asleson (Asleson et al. 2008) states that, "as a conservative estimate, the saturated hydraulic conductivity can be considered equal to the infiltration rate". Likewise; Poetter (Poetter et al. 2005) confirm that according to Darcy's Law, the hydraulic conductivity is merely a function of the soil flux multiplied by the hydraulic gradient. For a fully saturated soil, the hydraulic gradient is one. Therefore; the infiltration rate and the saturated hydraulic conductivity are equal and represent the maximum infiltration rate that can be expected.

B3.0 CURRENT STATE OF PRACTICE

Section B-3.1 describes methods to estimate infiltration and percolation rates based upon indirect methods. Section B-3.2 describes methods to estimate infiltration and percolation rates based upon direct measurement using in-situ testing methods.

B3.1 Indirect Methods to Estimate Infiltration and Percolation Rates

The most commonly used methods to make an indirect estimate of the infiltration rate (i.e., the hydraulic conductivity) of a soil are:

- Reference to published values in the hydrogeologic or soil science literature;
- Characterization using the soil texture; and
- Using the grain size distribution.

Because these sorts of estimation methods do not consider site-specific soil conditions, they should only be considered preliminary planning tools; perhaps part of the inventory of site conditions. Consequently, these methods are most useful at the WP or MDP planning levels. Appendix H describes these methods to estimate the infiltration or percolation rate and the hydraulic conductivity. In Section 3.2, uncertainty factors in converting the hydraulic conductivity to infiltration rates are described.

B3.2 Direct Method of Measuring Infiltration and Percolation Rates

A review of current practices and research being undertaken across North America and Australia identified the procedures tabulated in Table B-2 that are commonly in use to make in-situ measurements of the infiltration and percolation rate.

Table B-2 Commonly used Infiltration and Percolation Measurement Methods

Method	Description	Surface or Subsurface	Reference
Guelph Permeameter	Uses an unlined borehole and requires fitting the test results to a series of curves based upon a theoretical distribution of capillary forces in the unsaturated zone	Shallow and subsurface for any soil texture	Elrick and Reynolds, 1986 and Reynolds and Elrick, 1986
Phillip Dunne Infiltrometer	Uses a cased, snuggly fitted lined borehole only open at the lower end with the analysis requiring a set of nomographs of empirical constants to obtain percolation rates	Surface and subsurface	Philip, 1993
Modifed Phillip Dunne Infiltrometer	Uses a shallow cased borehole with a Mariot syphon to achieve a constant head analysis of results requires estimates for the wetting point suction	Shallow tests only primarily for liner and compacted soil surface	Nestingen, 2007
Double Ring Infiltrometer	Used a double ring on surface to establish vertical flow and a constant confining test section field data analyzed directly	Surface tests only but require vehicle access and a large area (minimum of 3.6 m by 3.6 m)	ASTM,1988 and 2002
Cased Borehole methods	Uses a cased borehole open across the test section at the bottom and can be used to obtain both vertical and horizontal hydraulic conductivity values field data analyzed directly	Can be used at variable depths and can be closely spaced - requires access by drilling equipment	ASTM, 2011 and Argue, 2004
Soak away pits (test pits)	Uses test pits with the geometry of the test pit well defined field data analyzed directly	Surface but extended to depths of the reach of a backhoe (5 m) and requires access by vehicles for water supply	Beardon, 2007 and IDEQ, 2011

The highlighted methods are those preferred for use within the City of Calgary for LID practices. Experienced practitioners may identify other procedures, but when other methods are applied, documentation must be provided on their applicability to the principles of the LID initiative. ASTM (ASTM 2010), in its comparison of soil infiltration methodologies, has been used as a guide in identifying methods favoured for application within The City for the LID initiative. These methods include:

- For surface infiltration estimates – the Guelph permeameter (Reynolds and Elrick 1986, and the double ring infiltrometer [ASTM 2002]); and
- For percolation estimates – the borehole casing method of ASTM (ASTM 2011) and Argue (Argue 2007).

The **Guelph permeameter** (Elrick and Reynolds 1986, and Reynolds and Elrick 1986) is relatively easy to set up and is adaptable to a wide variety of soil types. Therefore; it is best suited to aid in the design of stormwater management measures and in the selection of stormwater management features. It can also be applied to depths of up to 8 m.

The **double ring infiltrometer** (ASTM 1988 and ASTM 2002) measures the surface infiltration rate only. However; the double ring apparatus has difficulty in achieving an adequate surface seal when used in stony materials. It also requires vehicular access to bring water to the site and to carry the testing apparatus, which may be cumbersome. Because of these limitations, this method is best used on an engineered surface, such as the base of a stormwater management feature, such as a retention pond, rain garden, absorbent landscape, or infiltration gallery. It is most useful to confirm the performance of these measures following construction.

Both the **cased borehole infiltration** tests of ASTM (ASTM Method A in ASTM 2011) and Argue (Argue 2007) can be adopted for a wide variety of depths and terrain. Both methods can be used to estimate the vertical hydraulic conductivity and assume that the soil is isotropic. ASTM's Method A, however; allows a separate measurement of the horizontal saturated hydraulic conductivity to be made. In many settings in Calgary, with geologic materials that are horizontally stratified, understanding the influence of horizontal geologic structure on the percolation rate enhances the design of stormwater management features that will be cut below the existing surface. Further, the horizontal component of the hydraulic conductivity is a term used for the analysis of potential groundwater mounds built up on impermeable layers, or the water table beneath surface infiltration basins (stormwater retention ponds, rain gardens, bio-retention/bio-swale areas and infiltration galleries, for example). Therefore, both methods are useful for measuring the percolation rate below the base of a stormwater management feature built below the ground surface but, in stratified materials, the ASTM method is preferred.

B4.0 ANALYTICAL PROCEDURES AND TESTING PROTOCOLS FOR DIRECT (IN SITU) MEASUREMENT OF INFILTRATION AND PERCOLATION RATES

This section summarizes the direct testing protocols to be applied to The City's LID initiative for in situ measurement of infiltration and percolation rates. The summary provided in Sections B-4.1 to B-4.4, respectively, contains:

- The analytical expressions and field data to be collected for each in-situ testing method;
- General advice on the test setup and the number of infiltration and percolation tests within a development;
- A worked example of some recent percolation measurements; and

- Reporting requirements.

B4.1 Analytical Expressions and Field Data Collection

Section B-3.0 identified the recommended methods for in situ measurements of infiltration and percolation rates to be:

- Surface infiltration use of the Guelph permeameter or the double ring infiltrometer; and
- Percolation use of the borehole casing methods of ASTM 2011 or Argue.

Sections B-4.1.1 to B-4.1.4 present the analytical expressions used to obtain the infiltration and percolation rates by each of these methods with some limitations to the application the authors suggest. The cited references provide further details on the field equipment needed for each test and of the testing methods.

B4.1.1 Guelph Permeameter

The Guelph permeameter method (Elrick and Reynolds 1986, and Reynolds and Elrick 1986) measures both the saturated hydraulic conductivity and the matric flux potential (a function of the capillary tension forces). A detailed step by step procedure is contained in Elrick and Reyolds 1986). The method involves measuring the quantity of water (Q) needed to maintain a constant level of water (H) in a borehole. Successive changes in the level of water (H_s) and the different value of Q needed to maintain that level are compared to estimate the hydraulic conductivity and the matric flux potential.

The analytical expressions used in Reynolds and Elrick 1986 to estimate the hydraulic conductivity and the matric suction potential are based upon the following equation:

$$Q_s = \frac{2\pi H_s^2}{C} K_{fs} + \pi a^2 K_{fs} + \frac{2\pi H_s}{C} \vartheta_m \qquad \text{Equation B-1}$$

Where:

- Q_s is the steady flow rate into the test section(m^3/s);
- H_s is the steady depth of water in the test section (m);
- K_{fs} is the field saturated hydraulic conductivity (m/s);
- a is the radius of the test section (m);
- ϑ_m is the matric suction potential (m/m); and
- C is a dimensionless parameter.

Elrick and Reynolds (Elrick and Reynolds 1986) make the case that the K_{fs} value is a field saturated hydraulic conductivity and may be a factor of two less than the true saturated hydraulic conductivity (K_{sat}) due to entrapped air in the pore spaces. The case is also made that the field saturated hydraulic conductivity is a more appropriate value for unsaturated zone applications because entrapped air is always present in water retention structures that do not always contain water. This element of uncertainty in the hydraulic conductivity (and hence the infiltration rate) is discussed in Section 3.2 of the main text.

Values for C, the dimensionless parameter, are a function of the ratio of H_s/a, and obtained from graphs prepared by Elrick and Reynolds (Elrick and Reynolds 1986).

Elrick and Reynolds state that the Guelph permeameter can be used at depths up to 8 m and its practical range of quantification of the saturated hydraulic conductivity is 10^{-4} to 10^{-10} m/s.

B4.1.2 Double Ring Infiltrometer

The double ring infiltrometer (with and without the sealed inner ring) ASTM 1988 and ASTM 2002, is used commonly in Alberta to test the infiltration rate on prepared (engineered) soil surfaces. The method involves pushing two square rings of steel or other impermeable material (e.g., fibreglass) into the soil. The inner ring is recommended to be 1.52 m in diameter, and the outer ring about 3.6 m diameter across (the minimum dimensions are stated to be 0.6 m for the inner ring and separated from the outer ring by a further 0.6 m). The purpose of the outer ring is to avoid any lateral boundary effects such that water in the inner ring flows straight downward.

For the test, water is added to both rings and maintained at a constant level, with the quantity of water needed to maintain the water level in both the inner and outer ring over time recorded.

ASTM, 1988 (ASTM designation No. D3385-88) and ASTM, 2002 (ASTM designation No. D 5093-02) provides detailed descriptions of the method to be followed in completing tests using the double ring infiltrometer and the double ring infiltrometer fitted with a sealed inner ring, respectively.

The infiltration rate is estimated from the test results using the following expression:

$$I = \frac{\Delta V}{t\,A} \qquad \text{Equation B-2}$$

Where:

- I is the infiltration rate (m/s);
- ΔV is the water added over time t (m^3);
- t is the time in seconds; and
- A is the area of the inner ring (m^2).

B4.1.3 American Society for Testing and Materials - Cased Borehole Infiltration Test

ASTM, 2011 (ASTM designation No. D 6391 -11) uses a casing to stabilize the walls of a borehole for the percolation tests. The test can be conducted at any depth, and the ASTM indicates it can be used both above and below the water table (however; below the water table they recommend regular slug tests, which are more useful in materials with a saturated hydraulic conductivity greater than 10^{-5}m/s). The ASTM method (ASTM designation No. D6391-11) provides three methods for percolation testing in a cased borehole. The method recommended is Method A because it allows values for both the vertical saturated hydraulic conductivity and the horizontal saturated hydraulic conductivity to be estimated. The other two methods in D6391-11 estimate the vertical saturated hydraulic conductivity or percolation rate only. Because many of the soils in the Calgary area are sedimentary deposits that are stratified due to the mode of deposit and the saturated hydraulic conductivity values will be used to estimate mounding on impermeable layers or the surface of the water table, ASTM's Method A is the method to be used for The City's LID initiative.

ASTM 2011 provides the detailed methodology. Essentially, the test involves measuring the quantity of water used to maintain a constant head in the cased borehole – a constant head test and measuring the rate of water level decline with time – a falling head test.

ASTM's Method A is a two staged test. In the first stage, the borehole casing is placed to the bottom of the borehole and sealed from the influence of surface infiltration and the constant head test is performed. The analytical expressions to calculate the saturated vertical hydraulic conductivity are:

$$K_1 = R_T G_1 \frac{\ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)} \qquad \text{Equation B-3}$$

And $$G_1 = \left(\frac{\pi d^2}{11D}\right)\left[1 + a\left(\frac{D}{4b}\right)\right] \qquad \text{Equation B-4}$$

Where:

- R_t is 2.2902 x $(0.9842^T)/T^{0.1702}$ with T in temperature in °C;
- d is the diameter of the standpipe or casing (m);
- D is the effective diameter of the test section in the borehole (m);
- a is assigned a value of +ve 1 for the borehole casing, placed directly on an impermeable layer, 0 for an infinite depth of tested material (greater than 20 D) and –ve 1 for a permeable base at b;
- b is the thickness of the tested material (the open length of the casing) between the bottom of the casing and the impermeable layer (m);
- h_1 is the height of water at time t_1(m);
- h_2 is the height of water at time t_2 (m);
- t_1 is the time of the start of the test (seconds); and
- t_2 is the time of the end of the tests (seconds).

During the second stage, the falling head test is performed. The analytical expressions used to estimate a value for this stage of the test are:

$$K_2 = R_T G_2 \frac{\ln\left(\frac{h_1}{h_2}\right)}{(t_2 - t_1)}$$ Equation B-5

And $$G_2 = \left(\frac{d^2}{16FL}\right) G_3;$$ Equation B-6

$$G_3 = 2\ln G_4 + a \ln G_5$$ Equation B-7

$$G_4 = \frac{L}{D} + \sqrt{1 + \left(\frac{L}{D}\right)^2}$$ Equation B-8

$$G_5 = \frac{\left[\frac{4b_2}{D} + \frac{L}{D}\right] + \sqrt{1 + \left(\frac{4b_2}{D} + \frac{L}{D}\right)^2}}{\left[\frac{4b_2}{D} - \frac{L}{D}\right] + \sqrt{1 + \left(\frac{4b_2}{D} + \frac{L}{D}\right)^2}}$$ Equation B-9

Where:

All terms are as described above, but:

- b_2 is the length from the centre of the exposed or open section of the casing to the bottom of the layer being tested (m); and
- L is the length of the open section of casing (m).

K_1 is a measure of the vertical saturated hydraulic conductivity and is the average of the time weighted average of several tests using the Stage 1 testing method, and K_2 is a measure of the saturated horizontal hydraulic conductivity and is the time weighted average of several tests using the Stage 2 testing method. ASTM (ASTM 2011) suggests that the method is suitable for testing soils with a saturated hydraulic conductivity of less than 10^{-5} m/s.

B4.1.4 Argue's Cased Hole Method

Argue's method (Argue 2007) has been tested for application to Water Sensitive Urban Design in south Australia and Auckland, and is based upon the understanding that successive application of this method for water sensitive designs in fine grained soils has been achieved in Europe, Japan, and Australia. The method involves measuring water levels, over time, in a borehole lined with a perforated or slotted polyvinyl chloride (PVC) casing. Although not explicitly stated by Argue, it is presumed that the slotted section need only be placed within the soil test section. As an additional measure, the gap between the PVC liner and the borehole wall is filled with clean sand to stabilize the walls of the boring within the test section.

Similar to other infiltration testing methods, Argue's method requires both a constant head and a falling head test be undertaken. It should be considered that like the Guelph permeameter method, the test outcome is a "field saturated hydraulic conductivity". A period of pre-soaking is also recommended in an attempt to at least partially saturate the test section. Argue recommends that the pre-soaking period be 24 hours for clay soils but in sandy soils the casing should be filled and allowed to drain in rapid succession prior to the test.

The analytical expressions used by Argue to estimate the hydraulic conductivity are:

Constant head test

$$K_h = \frac{Q}{\left[\pi r^2 + 2\pi r h\right]} \text{ (m/s)} \qquad \text{Equation B-10}$$

Where:

- Q is the average flow rate as m³/s is:

$$Q = \frac{V_c}{t_c} \qquad \text{Equation B-11}$$

- V_c is the volume added to keep the water level in the casing to a set level (say the top of the casing [in m3]) and time is the time for the water to be added (seconds).
- r is the radius of the borehole (m); and
- h is the average water level maintained between additions of water (m).

For the falling head test:

$$K_h = \frac{1.15r}{(t_2 - t_1)} \log\left[\frac{h_1 + \frac{r}{2}}{h_2 + \frac{r}{2}}\right] \text{ (m/s)} \qquad \text{Equation B-12}$$

Where:

- r is the casing diameter (m); and
- t_1 and t_2 are times corresponding to the measurement of water level in the boreholes h_1 and h_2, respectively.

B4.2 General Advice on Infiltration and Percolation Test Setup and the Number of Infiltration Tests

This section provides some general advice on the set-up for infiltration and percolation tests, conducting tests under constant head and falling head test conditions, use of the test results to make infiltration estimates, and the number of tests to be applied within an SCP that uses infiltration..

1. Setup;

In preparing to measure infiltration or percolation rates in the field, the following conditions should apply:

- The test locations must correspond to the location or candidate locations of the stormwater management feature to be constructed within the LID.
- The test depths should correspond to the surface where infiltration is occurring or designed to occur, i.e.;
  - During the site assessment portion or the stormwater management feature portion of drainage planning – at the ground surface for bioswales or adsorbent landscape features that will take advantage of the natural attributes of the ground surface to manage stormwater;
  - During design or selection of stormwater management feature options – at the proposed depth of the base of stormwater retention ponds, French drains, or infiltration galleries; or
  - Post-construction to verify the performance of a prepared engineered infiltration surface.
- For cased borehole infiltration or percolation tests, PVC casings can be used to stabilize the walls of the borehole but the PVC should be open using either perforated or machine cut slots over those sections of soil to be tested.
  - The annular space between the walls of the borehole and the perforated or slotted section should be filled with filter sand to ensure stability of the borehole and prevent erosion of the walls when infiltration water is allowed to flow into the test section; and
  - The annular space should be sealed from overlying soils with bentonite to ensure that the infiltration test's waters flow into the soil for testing and not up the annular space.
- For double ring infiltrometer testing of infiltration surfaces, the two rings need to be well driven into the soil to form a good seal at the ground surface and leakage around the outside of the outer seal prevented.
- Pre-soaking for any of the four test methods should be continued for at least 24 hours for the clayey soil and for sandy soils; at least three casings full of water should be flushed through the soil before making the test measurements.

2. Constant Head Tests;

The constant head test is performed to measure the flow rate of water into the borehole that is required to maintain a constant hydraulic head over the test section (typically defined as h_0 or H_0). There will be a gradual decline over time in the amount of water necessary to maintain this constant head.

- In sandy soils, the flow rate is slow and can be maintained with a steady supply of water. The time it takes to empty a pre-determined amount of water into the borehole to maintain h_0 should be recorded. For example, if the pre-determined amount is 1 L, each time 1 L has been added to the borehole, the time should be recorded.
- In clay soils, the flow rate is very slow, and maintaining a constant head can be difficult. To achieve an acceptable "constant head" in these soils, allow the water level to fall for a period of 15 to 30 minutes

before refilling the borehole to the original water level. The time it took for the water to fall, as well as the volume of water it took to refill the borehole, should be recorded.

- Tests in sandy or clay soils should be repeated a minimum of four times.

3. Falling Head Tests;

The falling head test is conducted immediately after the constant head test has been completed. This test consists of recording the time it takes the water in the cased hole to drain without further addition of water. In some sandy or gravelly soils, the fall of water is too fast to be recorded and in these cases pressure transducers to measure the fall of water are useful. However, high variability in the measured water level during the early time portion of the test is to be expected. Likewise, in fine-grained soils, the decline in water level is too slow, making a long test prone to correction for temperature and/or barometric changes during the test – baro loggers are better suited for tests under these conditions.

4. Use of the Test Results;

- At least four infiltration or percolation tests should be conducted at each test location where sandy materials exist. Where clayey soils exist, three infiltration tests may take several days. In these settings, where the rate of decline of a falling head test is less than 5 cm in an hour, only one infiltration test is needed.
- An arithmetic average of the test results at a single location should be reported, although the standard deviation of the tests should be considered as discussed in Section 3.2 as a means of assessing the parameter uncertainty.
- For multiple test sites within an infiltration feature, the geometric mean of the test results from multiple test locations can be used to represent the overall hydraulic performance of the infiltration site. The geometric mean is the n^{th} root of the product of n tests.
- All tests recommended for infiltration and percolation rates, except the double ring infiltrometer measurement, produce a "field saturated hydraulic conductivity" (Elrick and Reynolds 1986 and ASTM 2011). Because of the entrapment of air during infiltration, the test results need to be adjusted by a factor of 2 to provide a better measure of the saturated hydraulic conductivity.
- Infiltration tests with the double ring infiltrometer provide a measure of the "infiltration velocity" (ASTM 1988 and ASTM 2002). These results need to be adjusted for the "field saturated hydraulic conductivity" as follows:

$$K_{fs} = \frac{I}{i} \qquad \text{Equation B-13}$$

Where:

- K_{fs} is the field saturated hydraulic conductivity (m/s);
- I is the infiltration rate (m/s after Equation B-2); and
- i is the hydraulic gradient measured as:

$$i = \frac{H + D}{D}$$ Equation B-14

Where:

- i is the hydraulic gradient applied to the test (m/m);
- H is the depth of water in the inner ring (m); and
- D is the driven depth of the inner ring (m).

Field saturated hydraulic conductivity values need to be corrected by a factor of two to obtain the actual saturated hydraulic conductivity as discussed in Section B 3.2.2, and for a prepared surface corrected to obtain an estimated infiltration rate by correction for the hydraulic gradient. The hydraulic gradient (i) across a prepared surface is calculated as:

$$i = \frac{H + L}{L}$$ Equation B-15

Where:

- i is the hydraulic gradient (m/m);
- H is the depth of water on the prepared surface (m); and
- L is the thickness of the prepared surface (m).

Section 3.2 provides a more comprehensive evaluation of how to incorporate the uncertainty in hydraulic conductivity values and infiltration and percolation rates into the design of stormwater management measures.

5. Number of Tests Recommended;

A minimum of one infiltration or percolation rate test is needed for each stormwater management feature measured to be built within a development for management of stormwater. The number of tests recommended depends upon the size of the infiltration feature. Recommendations are provided in Section 3.2.

B4.3 Worked Examples of Infiltration Tests

Sections B4.3.1 and B4.3.2 provide worked examples (an example created with real field results) of the cased borehole infiltration test methods by ASTM (ASTM 2011) and by Argue (Argue 2007).

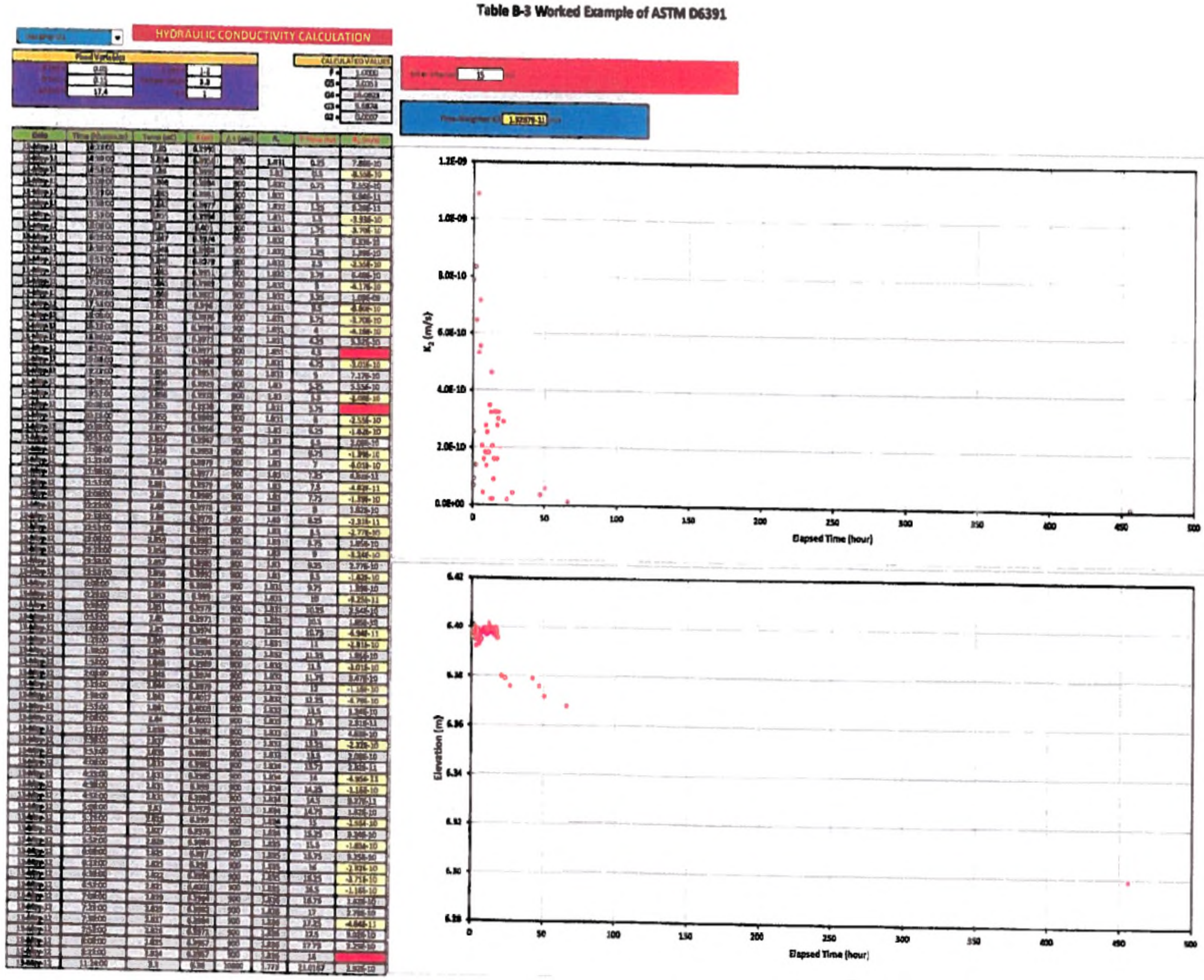
B4.3.1 American Society for Testing and Materials Method

The ASTM example is provided on the attached excel spreadsheet. The spreadsheet contains instructions on how to bring the field data into the calculation and the calculations themselves. An example of the output from the program is provided on Table B-3. This output is from a field site EBA Engineering Consultants Ltd. operating as EBA, A Tetra Tech Company's (EBA), investigated in the spring of 2012. The investigation was undertaken to confirm the saturated hydraulic conductivity of a thick zone (more than 20 m) of unsaturated material. The output consists of:

- A graph of the field saturated hydraulic conductivity estimated throughout the test;
- A graph of the head elevation to the water in the cased borehole; and
- A time weighted value of the hydraulic conductivity (in the example shown K_h = 2.4e^{-11}).

The example shown is only for the Stage 2 portion of Method A of ASTM 2011 – a Stage 1 example is available, but no field data is available to provide a sample calculation.

Table B-3 Worked Example of ASTM D6391



B4.3.2 Argue's Method Example

Argue's cased borehole method is presented here as both a calculation of the field saturated hydraulic conductivity from the constant head test and from the falling head test. The field data was obtained from a test site in Calgary investigated by EBA in the spring of 2011.

Constant Head Test Data

Trial No.	Duration (min)	Water Level Drop (m)	Average Drop (m)	Volume Required to Backfill (L)	Vc (L)	Tc (min)
1	30	0.5	0.275	0.025	0.07	120
2	30	0.2		0.015		
3	30	0.2		0.015		
4	30	0.2		0.015		

The casing diameter was three inches (0.075 m).

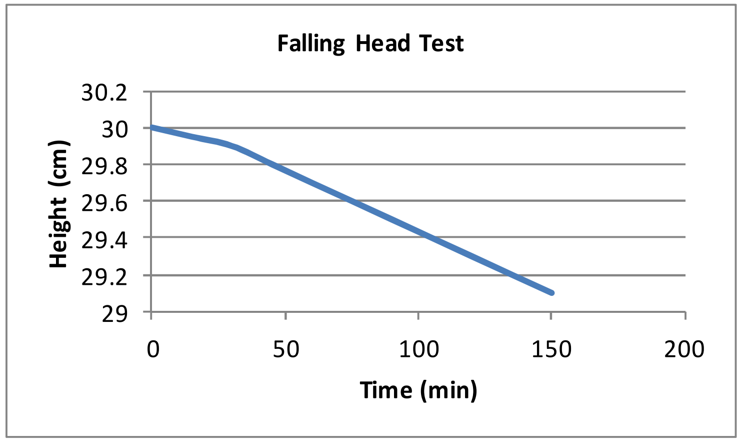
$$K_{fs} = \frac{Q}{\left(\pi r_o^2 + 2\pi r_o h\right)} \qquad \text{Equation B-16}$$

$$K_{fs} = \frac{0.07L / 120\min}{\left(\pi\, x\,(0.0375m)^2\right) + \left(2\,\pi\, x\, 0.0375m\, x\, 0.275\right)}\, x\, \frac{10^{-3}\, m}{L}\, x\, \frac{1\min}{60s}$$

$$K_{fs} = 1.4\, X\, 10^{-7}\, m\, s^{-1}$$

Falling Head Test Data

Total Time (min)	Total Depth (cm)	Water Level Drop (cm)	h(cm)
0	30	0	30
15		0.05	29.95
30		0.1	29.9
45		0.2	29.8
60		0.3	29.7
75		0.4	29.6
90		0.5	29.5
105		0.6	29.4
120		0.7	29.3
135		0.8	29.2
150		0.9	29.1



Using:

- h_1=29.8 cm;
- h_2=29.6 cm;
- t_1=45 min; and
- t_2= 75 min.

$$K_{fs} = \frac{1.15 r_o}{(t_2 - t_1)} \log\left[\frac{h_1 + \frac{r_o}{2}}{h_2 + \frac{r_o}{2}}\right] X \frac{1\min}{60s} X \frac{1m}{100cm} m s^{-1}$$ Equation B-17

$$K_{fs} = \frac{1.15 X 3.75cm}{75\min - 45\min} \log\left[\frac{29.8cm + \frac{3.75}{2}}{29.6 + \frac{3.75}{2}}\right] X \frac{1\min}{60s} X \frac{1m}{100cm}$$

$$K_{fs} = 9.6 X 10^{-8} ms^{-1}$$

The constant head and falling head tests results are in good agreement, so an arithmetic average of the two results provides an estimate for the field saturated hydraulic conductivity at this location to be:

$$(1.4 \text{ x } 10^{-7} + 9.6 \text{ x } 10^{-8})/2 \text{ or } 1.2 \text{ x } 10^{-7} \text{ ms}^{-1}$$

B4.3.3 Reporting Requirements

To report on the results of infiltration and percolation rate measurements at a development site, we provide the following recommendations:

1. A record of all boreholes drilled for infiltration tests using the cased borehole or Guelph permeameter methods should be provided. The record should include the depth of the borehole, length of the test section, and the soil conditions (colour, wetness, material type, and evidence of flora and fauna near the test location (e.g., roots and borrowings).
2. A record of the surface soil conditions for surface infiltration test locations should be provided. The record should include the soil colour, wetness, material type, and evidence of flora and fauna near the surface test location (e.g., roots and borrowings).
3. The depth and length of all test zones should be documented.
4. The location should be shown on a map and a cross-section of the location, along with the proposed location of stormwater management features and the depth of excavation of the stormwater management features.
5. All field records of water added and depth to water level measurements should be tabulated, as well as the conversion of the depth to water to height of water.
6. The calculation methods should be identified and the rationale for application of the particular test method or an alternate should be provided. All calculations should be shown for confirmation by The City, if needed.
7. The use of all infiltration rates or percolation rates and the average hydraulic conductivity values should be provided and, if calculated, the geometric mean values for the infiltration or percolation rate of the hydraulic conductivity values should be provided.
8. Weather conditions at the time of the tests, temperature, sunny or overcast, rainfall, or snow should also be recorded.

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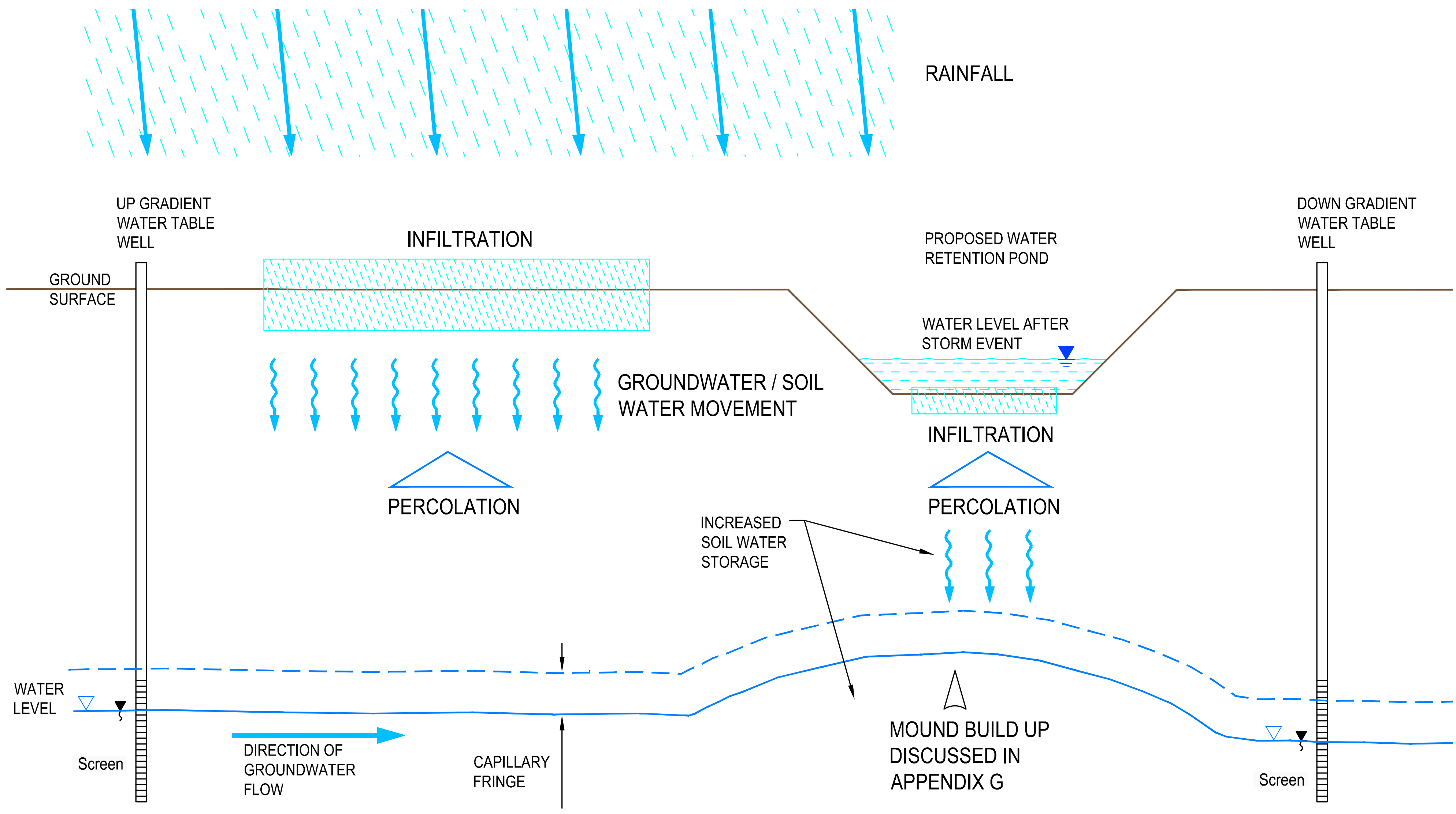
FIGURES

Q:\Riverbend\Drafting\C121\C12101310\004\Autocad\C12101310.004 Figures 1 and E-1.dwg [FIGURE B-1] June 11, 2012 - 3:35:53 pm (BY: MORGAN, BRIAN)



LEGEND

CLIENT

FIREBAG

CONVENTIONS USED TO DEFINE INFILTRATION AND PERCOLATION

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APPENDIX C

METHODS TO CHARACTERIZE SOIL CONDITIONS CITY OF CALGARY MODULE 1 GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

TABLE OF CONTENTS

TABLES

FIGURES

ATTACHMENTS

C1.0 INTRODUCTION

This appendix describes the methods to be used to characterize soil conditions for geotechnical purposes and support Low Impact Developments (LID's) within The City of Calgary (The City). Developers and their consultants, submitting plans to The City for approval, will find the guidance provided here useful to:

- Identify locations within a development best suited for constructing stormwater management features;
- Make decisions between stormwater management options;
- Support estimates for groundwater build-up beneath stormwater management facilities that use subsurface infiltration to manage stormwater; and
- Assess slope stability for areas of topographic change across the development and design sideslopes for stormwater retention ponds.

Table C-1 is an index to those Appendices of Module 1 – Geotechnical and Hydrogeologic Considerations for Low Impact Development, where the characterization of soil conditions must be considered to satisfy LID principles.

Table C-1: Index for Integration with Other Geotechnical and Hydrogeological Consideration Methods

Objective	Reference Section
Locating Stormwater Control Features	Appendix E – Framework for conceptual site model development and Appendix D – Probable hydrogeologic consequences
Selection of Stormwater Management Options	Appendix D – Probable Hydrogeologic consequences and Appendix E – Framework for conceptual site model development
Support Estimates of Groundwater Build-up beneath Stormwater Management Features	Appendix G – Groundwater mounding assessment methods
Slope Stability Assessment	Appendix F – Slope stability assessment methods

Typically, in situ soil characterization is part of a geotechnical assessment undertaken to design and assess construction requirements for roadways, utility corridors, retaining walls, and foundations for above-grade structures within a development. Guidance on these geotechnical assessment methods is not provided here. This document refers only to these methods needed to support the design of stormwater management features. The soil characterization methods described here are applicable at the Staged Master Drainage Plan (SMDP), Pond Report, Stromwater Management Report (SWMR), or Development Site Servicing Plan (DSSP) drainage planning levels.

Sections C2.0 to C4.0 describe soil characterization according to the following topics:

- The need;
- The overall step wise-process; and
- The methods to be used for LID projects within The City.

All of the characterization methods described here should only be undertaken by a qualified geotechnical engineer and are typical of the methods used in practice within the City of Calgary for other types of geotechnical investigations. Qualified geotechnical engineers may have methods other to those prescribed here. Where these other methods are applied, the engineer should provide the rationale and supporting documentation to The City to support such differences as part of the LID development approval documents.

C2.0 NEED TO CHARACTERIZE SOIL CONDITIONS

Soil conditions influence the placement of stormwater management features across the development relative to the area to be used for the development's structures, the type of stormwater management features best suited to the site position, and the design and constructability of the stormwater management features.

Characterization of the soil condition refers to:

- Evaluation of the soil types and material properties for the soil encountered beneath the proposed development;
- Understanding of the changes in soil types and material properties with depth; and
- Understanding of the differences in soil type and material properties from place to place across the development.

C3.0 OVERALL STEP-WISE PROCESS TO CHARACTERIZE SOIL CONDITIONS

In general, soil conditions are characterized using the step-wise process illustrated on Figure C-1. The first step helps to ensure the principles for LID (Section 1) are maintained. The final step is an outcome of the prior six steps and the method for that step cannot be prescribed. The purpose, methods to be followed, and outcomes of Steps 2 through 6 are described in Section C4.0.

C4.0 METHODS TO CHARACTERIZING SOIL CONDITIONS

This section describes:

- Subsurface investigation methods;
- Collection of soil samples;
- Classification of soils and measurement of soil properties;
- Creation of soil depth profile; and
- Spatial comparison.

C4.1 Subsurface Investigation Methods

Principally, subsurface investigation for LID purposes involves either drilling a network of boreholes/ testholes, or excavation of testpits at various locations with the proposed development. Skilled geotechnical and hydrogeologic practitioners can describe the number of boreholes and testpits best suited to the nature and size of a particular development. For guidance purposes, Table C-2 provides the minimum number of investigative locations that should be considered based upon the area of the development.

Table C-2: Number of Investigative Locations and Tests of Soil Conditions Recommended

Development Area	Investigation Location	Soil Samples Collected[1]	Soil Property Measurements[2]			Depth Profiles	Cross-Sections
			WC	Atterberg Limit Testing	Grain Size Analysis (Sieve or Hydrometer)		
Less than 1 Ha	3 (but more than 10 m apart)	30	30	3	3	3	0
1 to10 Ha 10 to 100 Ha	3 to12 12 to 20	30 to 120 120 to 200	30 to 120 120 to 200	12 20	12 20	3-12 12 to 20	3 4
>100 Ha	20 minimum	200+	200+	200+	20+	20+	4-7

[1] Assess one sample collected at an average depth interval of 1.5 m to a depth of 15 m.

2 Minimum three sets of tests per soil unit.

Similarly, a variety of drilling and excavation equipment is available for the subsurface investigations and skilled practitioners are adept at selecting, based upon time and schedule, the equipment best suited to satisfy the investigation needs of stormwater management features. The most commonly used equipment for geotechnical investigation is either a solid-stem auger or in areas of collapsing soils, the hollow-stem auger. Augered boreholes are well suited for the installation of monitoring wells as needed for the measurement of water levels and hydraulic conductivity testing, as described within Appendix A. Testpits are not acceptable for the installation of monitoring wells.

Details on the use of auger type drilling equipment for geotechnical investigations are available from the following sources:

- Canadian Geotechnical Society 2006 – Canadian Foundation Engineering Manual 4 Edition; and
- ASTM D5434-12 – Standard Guide for Field Logging of Subsurface Explorations of Soil and Rock.

C4.2 Field Logging and Soil Sample Collection

Soil samples should be collected during borehole drilling or testpit excavations at regular 1.5 m depth intervals from each soil type or more frequently where ever soil properties are observed to vary while drilling or excavation proceeds. This initial understanding of the soil type is based upon observations made on drill cuttings collected from the auger flights or the excavated soil and should be replaced or supplemented by classification testing (either grain size distribution or index testing or both) conducted by a certified geotechnical laboratory.

The purposes of visual observation and field textural classification for soil types are:

- To determine the overall consistency of the soil types encountered;
- To permit a preliminary geologic appraisal of the stratigraphic units to be made; and
- To record findings that are not recorded as accurately in the laboratory (e.g., soil consistency, wetness, zones of seepage along the borehole or testpit, the soil colour, and degree of weathering).

Standard Penetration Tests (SPTs) provide a quantifiable measure of relative consistency and relative density with depth. These tests provide a means of correlating stratigraphic units. The SPT testing method involves recording the number of blows needed to drive a standard-sized split-spoon sampler into the soil by a drop hammer (at a specified weight and drop height) over a standard length (0.3 m). Terms used to measure the consistency and relative density of soil using the number of blows to advance the soil sampler 0.3 m are presented on Attachment C-1.

Soil samples for laboratory testing can be collected as either disturbed samples (e.g., directly from the auger flights, the excavated soil, or using a split-spoon sampling device) or relatively undisturbed soil samples (e.g., from a soil core obtained with a Shelby tube sampler or continuous soil coring device).

The following ASTM methods should be considered when organizing and planning a field program for soil sample collection and the care of the collected samples:

- ASTM D1452-09– Standard practice for soil explorations and sampling by auger borings;
- ASTM D1586-11– Standard test method for standard penetration test (SPT) and split barrel sampling of soils;
- ASTM D2113-08– Standard practice for rock core drilling and sampling of rock for site investigation;
- ASTM D2488-09a – Standard practice for description and identification of soils (visual-manual procedure); and
- ASTM D4220-95 (2007) – Standard practices for preserving and transporting soil samples.

C4.3 Classification of Soil Types and Measurement of Soil Properties

Classification of soil types and textural analyses are essentially the same term. Defining the soil type and its textural analysis are essential components of the soil characterization for LID purposes. Both classification and textural analysis refer to:

- Determining the dominant soil particle size within a sample (clay, silt, sand, or gravel);
- Determining the secondary most prevalent soil particle size (clayey, silty , sandy, or gravelly);
- Providing a description of the remaining soil particles (some for those materials constituting less than 20% by weight of the soil sample and trace for those material constituting less than 10% by weight of the soil sample); and
- Creating a description (e.g., sand, silty some clay, and a trace of gravel).

Typical soil particle size gradation curve plots used to determine the proportion of particle sizes are provided on Attachments C-2 and C-3 for both a sieve analysis (coarse-grained soils) and a hydrometer analysis (fine-grained soils).

The shape of the gradation curve presented on a sieve and/or a hydrometer plot is used to modify the soil description by expressing the degree of grading (spread between the grain size groups, the engineering terminology or sorting, the geological terminology). A shallow grain size distribution curve indicates a well graded sample or poorly-sorted grain sizes, where as a steep grain size curve suggests a poorly graded soil or well-sorted grain sizes. These descriptions suggest the mode of sedimentary deposition of the soil and are useful to link soil types between boreholes as described in Section C4.5.

For engineering purposes, soils are also classified according to their anticipated behaviour using index testing methods (Atterberg limits to define a liquid limit and a plasticity index). Attachment C-4 contains a chart that can be used with the Atterberg limits to classify soil types. This classification system is known as the Unified Soil Classification System (USCS). Because many geotechnical firms have their own habits and standards for classifying soils, most frequently a modified USCS is used to include an intermediate (medium) plasticity classification. To assist with the interpretation of soil conditions and improve understanding of the design advice provided by the geotechnical engineer, a guidance chart such as that provided in Attachment C-4 should always be provided with the geotechnical report.

Detailed guidance on soil classification and in particular the USCS can be obtained from:

- ASTM D2487-11 – Standard practice for classification of soils for engineering purposes (USCS); and
- ASTM D4254-00 (2006) – Standard test methods for minimum index density and unit weight of soil and calculation of relative density.

For LID purposes, the most useful soil properties to be measured in the laboratory include:

- The moisture or water content – to confirm the soil relative density and degree of saturation as a means of assessing infiltration capacity and presence of perched water tables;
- The soil particle grain size distribution; and
- The relative density to identify hard and soft soils where infiltration may be resisted on lower permeability geologic layers.

For slope stability purpose, other geotechnical parameters may need to be measured, such as the sheer strength, compressive strength, and degree of soil compaction. The following procedures provide standards for measurement of these parameters:

- ASTM D698-12 – Standard test method for laboratory compaction characteristics of soil using standard effort;
- ASTM D2166-06 – Standard test method for unconfined compressive strength of cohesive soil;
- ASTM D2850-03a – Standard Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils; and

- ASTM D3038/D3080m-11 – Standard Test Method for Direct Shear Test of Soils under Consolidated Drained Conditions.

C4.4 Creation of Soil Depth Profiles

A soil depth profile shows the variation in soil type and material properties along the drilled or excavated depth of the borehole/testhole or testpit used for the subsurface investigation. These soil depth profiles are most frequently referred to as the borehole, testhole, or testpit log or record. A record must be prepared for every location investigated within a development as a permanent record of the subsurface conditions.

Each of the records must provide:

- The soil type gradation and plasticity described according to either visual classification and/or the USCS, and include colour, consistency, moisture status, and degree of weathering;
- The depth of changes in soil types or material properties of the soils and whether this change marks a distinct change in stratigraphy (including seepage or soil collapse in the borehole, testhole, or testpit);
- The depths of soil samples collected (the type of sample collected) along each borehole, testhole, or testpit;
- The depths of soil samples submitted for laboratory testing for textural analysis and for other material properties; and
- Any instrumentation (monitoring wells or slope stability indicators) constructed at the borehole location.

A typical borehole record is provided in Attachment C-5 for illustrative purposes.

C4.5 Spatial Comparison

At sites where more than three boreholes and/or testholes are drilled or more than three testpits are excavated, a cross-section should be provided. The cross-section is used to illustrate the stratigraphy and show the changes in soil types from location to location across the development. These changes are used to determine preferred locations for a stormwater management facility and to determine the depth of construction of the feature at that location. Soil properties measured on the soils can be used to estimate the probable infiltration capacity (coarse-grained soils have greater infiltration than fine-grained soils), area useful for infiltration, and any stratigraphic intervals with greater density or consistency where infiltration capacity is limited by the soils sedimentary structure.

Cross-sections must show:

- The soil types at each investigative location and changes with depth;
- The vertical and horizontal scale;
- The ground surface elevations; and
- Be accompanied by a site map showing the cross-section location.

A typical cross-section used for geotechnical purposes within a development is provided on Attachment C-6.

FIGURES

Figure C-1 - Soil Characterization Process

Step 1

Review of Development Plans and Needs with Respect to the LID Principles

Step 2

Subsurface Investigation

Step 3

Collection of Soil Samples

Step 4

Classification of Soil Types and Material Properties

Step 5

Creation of Soil Profiles

Step 6

Spatial Comparison

Step 7

Recommendations for Development

ATTACHMENT C-1

TERMS USED ON BOREHOLE LOGS

TERMS USED ON BOREHOLE LOGS

TERMS DESCRIBING CONSISTENCY OR CONDITION

COARSE GRAINED SOILS (major portion retained on 0.075mm sieve): Includes (1) clean gravels and sands, and (2) silty or clayey gravels and sands. Condition is rated according to relative density, as inferred from laboratory or in situ tests.

DESCRIPTIVE TERM	RELATIVE DENSITY	N (blows per 0.3m)
Very Loose	0 TO 20%	0 to 4
Loose	20 TO 40%	4 to 10
Compact	40 TO 75%	10 to 30
Dense	75 TO 90%	30 to 50
Very Dense	90 TO 100%	greater than 50

The number of blows, N, on a 51mm O.D. split spoon sampler of a 63.5kg weight falling 0.76m, required to drive the sampler a distance of 0.3m from 0.15m to 0.45m.

FINE GRAINED SOILS (major portion passing 0.075mm sieve): Includes (1) inorganic and organic silts and clays, (2) gravelly, sandy, or silty clays, and (3) clayey silts. Consistency is rated according to shearing strength, as estimated from laboratory or in situ tests.

DESCRIPTIVE TERM	UNCONFINED COMPRESSIVE STRENGTH (KPA)
Very Soft	Less than 25
Soft	25 to 50
Firm	50 to 100
Stiff	100 to 200
Very Stiff	200 to 400
Hard	Greater than 400

NOTE: Slickensided and fissured clays may have lower unconfined compressive strengths than shown above, because of planes of weakness or cracks in the soil.

GENERAL DESCRIPTIVE TERMS

Slickensided - having inclined planes of weakness that are slick and glossy in appearance.
Fissured - containing shrinkage cracks, frequently filled with fine sand or silt; usually more or less vertical.
Laminated - composed of thin layers of varying colour and texture.
Interbedded - composed of alternate layers of different soil types.
Calcareous - containing appreciable quantities of calcium carbonate.;
Well graded - having wide range in grain sizes and substantial amounts of intermediate particle sizes.
Poorly graded - predominantly of one grain size, or having a range of sizes with some intermediate size missing.

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ATTACHMENT C-2

SIEVE ANALYSIS REPORT

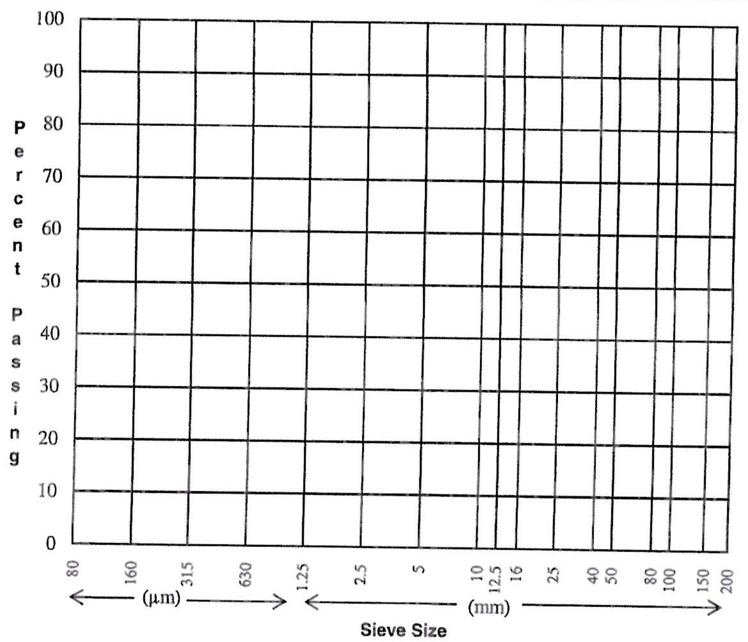
Attachment C-2

SIEVE ANALYSIS REPORT

ASTM C136, C117

Project:		Sample No.:	
Client:		Date Sampled:	
Project No.:		Sampled By:	
Attention:		Date Tested:	
Description:		Tested By:	Lab:
Source:		No. Crushed Faces:	
Location:		Moisture Content:	
Specification:			

Sieve Size	Percent Passing
200 mm	
150 mm	
100 mm	
80 mm	
50 mm	
40 mm	
25 mm	
20 mm	
16 mm	
12.5 mm	
10 mm	
5 mm	
2.5 mm	
1.25 mm	
630 µm	
315 µm	
160 µm	
80 µm	



Remarks: ______________________________

Reviewed By: ______________________ P.Eng.

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ATTACHMENT C-3

PARTICLE SIZE ANALYSIS (HYDROMETER) TEST REPORT

Attachment C-3

PARTICLE SIZE ANALYSIS (Hydrometer) TEST REPORT

ASTM D422

Project:

Client:

Project No.:

Location:

Description **:

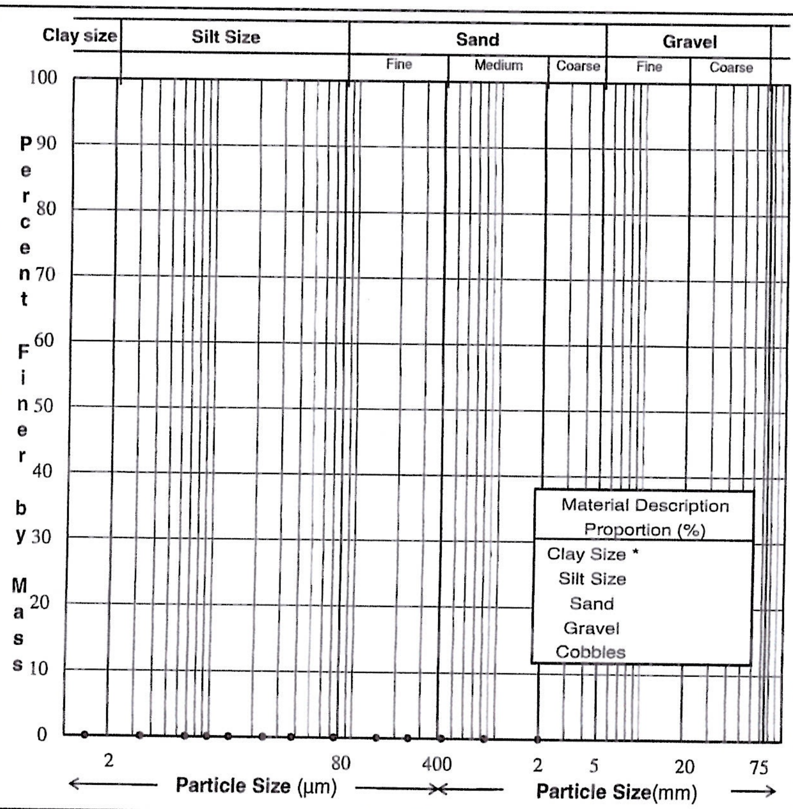
Sample No.:

Borehole/ TP:

Depth:

Date Tested

Particle Size	Percent Passing
100 mm	
75 mm	
50 mm	
38 mm	
25 mm	
19 mm	
13 mm	
10 mm	
5 mm	
2 mm	
850 µm	
425 µm	
250 µm	
150 µm	
75 µm	
38 µm	
24 µm	
14 µm	
10 µm	
7 µm	
3 µm	
1 µm	



Material Description	Proportion (%)
Clay Size *	
Silt Size	
Sand	
Gravel	
Cobbles	

Reviewed By: ______________________ P.Eng.

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ATTACHMENT C-4

MODIFIED UNIFIED SOIL CLASSIFICATION

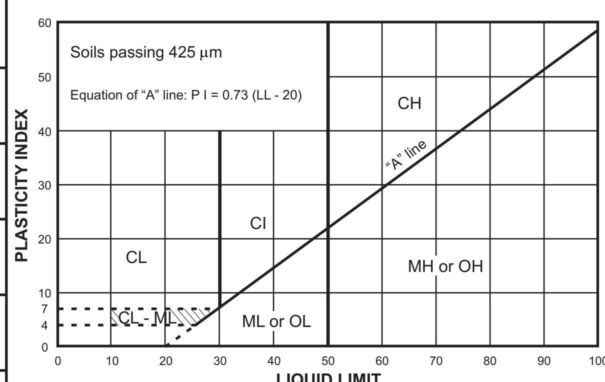
MODIFIED UNIFIED SOIL CLASSIFICATION

MAJOR DIVISION			GROUP SYMBOL	TYPICAL DESCRIPTION	LABORATORY CLASSIFICATION CRITERIA	
COARSE-GRAINED SOILS More than 50% retained on 75 µm sieve*	GRAVELS 50% or more of coarse fraction retained on 4.75 mm sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines	$C_U = D_{60}/D_{10}$ Greater than 4; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines	Not meeting both criteria for GW	
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			GC	Clayey gravels, gravel-sand-clay mixtures	Atterberg limits plot above "A" line or plasticity index greater than 7	
	SANDS More than 50% of coarse fraction passes 4.75 mm sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands, little or no fines	$C_U = D_{60}/D_{10}$ Greater than 6; $C_C = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between 1 and 3	
			SP	Poorly graded sands and gravelly sands, little or no fines	Not meeting both criteria for SW	
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures	Atterberg limits plot below "A" line or plasticity index less than 4	Atterberg limits plotting in hatched area are borderline classifications requiring use of dual symbols
			SC	Clayey sands, sand-clay mixtures	Atterberg limits plot above "A" line or plasticity index greater than 7	
FINE-GRAINED SOILS (by behavior) 50% or more passes 75 µm sieve*	SILTS Below "A" line on plasticity chart; negligible organic content	Liquid limit <50	ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands of slight plasticity		
		Liquid limit >50	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts		
	CLAYS Above "A" line on plasticity chart negligible organic content	Liquid limit <30	CL	Inorganic clays of low plasticity, gravelly clays, sandy clays, silty clays, lean clays		
		Liquid limit 30-50	CI	Inorganic clays of medium plasticity, silty clays		
		Liquid limit >50	CH	Inorganic clays of high plasticity, fat clays		
	ORGANIC SILTS AND CLAYS	Liquid limit <50	OL	Organic silts and organic silty clays of low plasticity		
		Liquid limit >50	OH	Organic clays of medium to high plasticity		
HIGHLY ORGANIC SOILS			PT	Peat and other highly organic soils		

Classification on basis of percentage of fines:
Less than 5% Pass 75 µm sieve: GW, GP, SW, SP
More than 12% Pass 75 µm sieve: GM, GC, SM, SC
5% to 12% Pass 75 µm sieve: Borderline Classification requiring use of dual symbols

For classification of fine-grained soils and fine fraction of coarse-grained soils.

PLASTICITY CHART



*Based on the material passing the 75 mm sieve
Reference: ASTM Designation D2487, for identification procedure see D2488. USC as modified by PFRA

SOIL COMPONENTS

FRACTION	SIEVE SIZE		DEFINING RANGES OF PERCENTAGE BY MASS OF MINOR COMPONENTS	
	PASSING	RETAINED	PERCENTAGE	DESCRIPTOR
GRAVEL coarse	75 mm	19 mm	>35 %	"and"
GRAVEL fine	19 mm	4.75 mm	21 to 35 %	"y-adjective"
SAND coarse	4.75 mm	2.00 mm	10 to 20 %	"some"
SAND medium	2.00 mm	425 µm	>0 to 10 %	"trace"
SAND fine	425 µm	75 µm		
SILT (non plastic) or CLAY (plastic)	75 µm		**as above but by behavior**	

OVERSIZE MATERIAL

Rounded or subrounded

COBBLES	75 mm to 300 mm
BOULDERS	> 300 mm

Not rounded

ROCK FRAGMENTS	>75 mm
ROCKS	> 0.76 cubic metre in volume

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ATTACHMENT C-5

SAMPLE BOREHOLE LOG

Attachment C-5

RMH SITE INVESTIGATION	SECURE ENERGY	PROJECT NO. - BOREHOLE NO.
SW 04-40-08 W5M	SOLID STEM AUGER	C22501059-BH09
ROCKY MOUNTAIN HOUSE, ALBERTA		

SAMPLE TYPE: DISTURBED, NO RECOVERY, SPT, A-CASING, SHELBY TUBE, CORE
BACKFILL TYPE: BENTONITE, PEA GRAVEL, SLOUGH, GROUT, DRILL CUTTINGS, SAND

Depth (m)	SOIL DESCRIPTION	SAMPLE NUMBER	SPT (N)	MOISTURE CONTENT
0	TOPSOIL - clayey, silty, some sand, moist, black.			
	SAND - silty, trace clay, medium grained, wet, loose, brown.	B1		21.9
1	CLAY (TILL) - silty, sandy, trace gravel, low to medium plastic, wet, stiff, brown, trace oxides, trace coal specks.	B2		24.7
	Material Proportion at B2 Clay 19%, Silt 46%, Sand 30%, Gravel 5%	S1	11	
2		B3		10.8
	SAND - silty, trace clay, uniformly graded, medium grained, damp, loose to compact, brown.			
3	BEDROCK (SANDSTONE) - highly weathered, very weak to weak, brown.	B4		6.8
		S2	100/100mm	
	- brown and grey mottled together	B5		
4	- grey	B6		
		S3	100/125mm	
5		B7		16.2
6		S4	100/125mm	
		B8		
7				
8	COAL Seam - black	S5	65/100mm	
	- siltstone, weak brownish grey	B9		
9	- sandstone, grey	S6	100/100mm	
	END OF HOLE AT 9.38 m. Standpipe installed to 8.84 m. Groundwater at 1.44 m upon completion. Groundwater at 2.37 m on March 22, 2012.			

Groundwater level marker: Mar. 22/12

Chart headings: ◇ SOLUBLE SULPHATE (%) ◇ 0.4 0.8 1.2 1.6; PLASTIC M.C. LIQUID 20 40 60 80; ■ STANDARD PENETRATION (N) ■ 20 40 60 80; ◆ UNCONFINED (kPa) ◆ 50 100 150 200; ▲ POCKET PEN. (kPa) ▲ 100 200 300 400; Slotted Piezometer; Depth (ft)

Chart annotations: /100, /125, /125, /100, /100

Depth (ft): 0, 5, 10, 15, 20, 25, 30, 35, 40, 45, 46

LOGGED BY: SM	COMPLETION DEPTH: 9.32 m
REVIEWED BY: JK	COMPLETE: 12/03/2012
DRAWING NO:	Page 1 of 1

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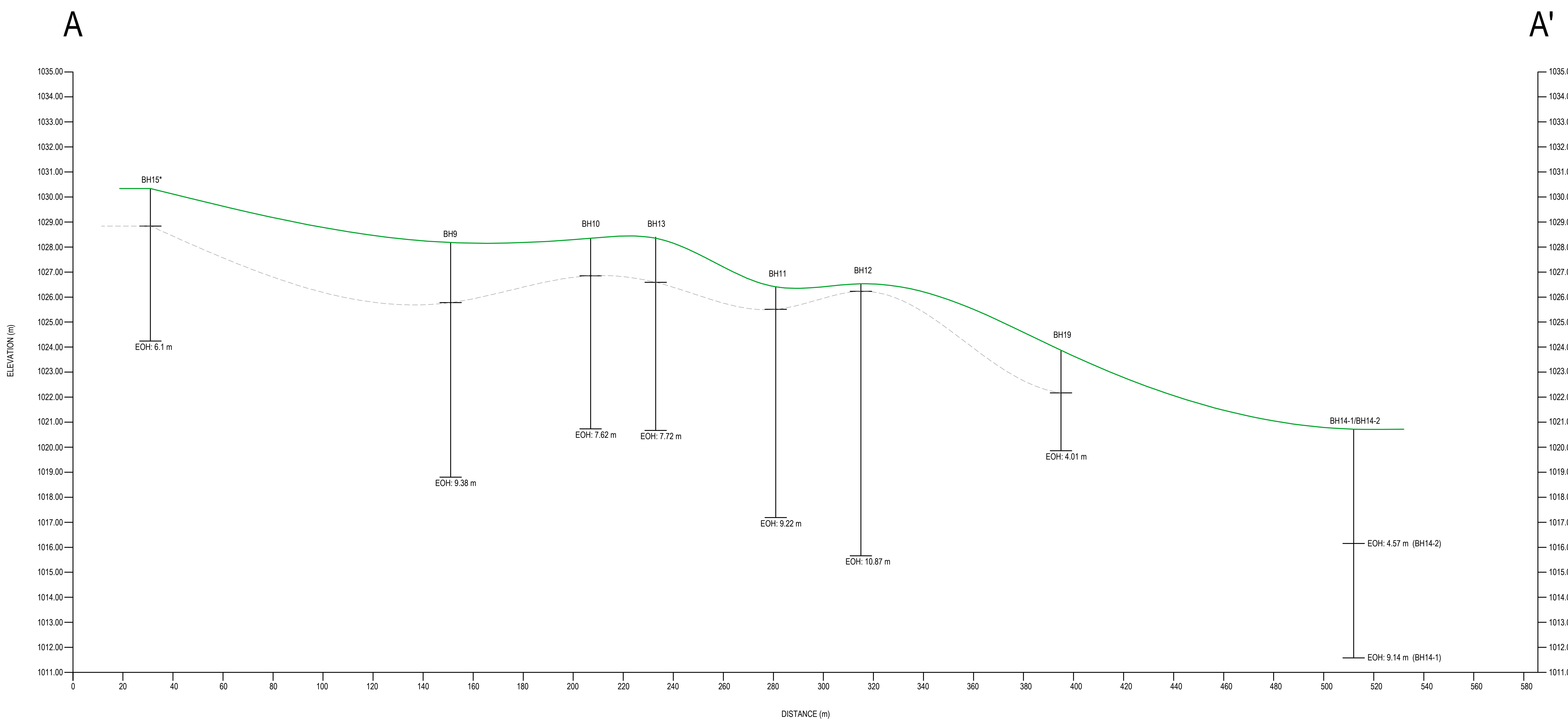
GEOTECHNICAL C22501059.GPJ EBA.GDT 13/4/12

ATTACHMENT C-6

EXAMPLE OF A CROSS-SECTION

Attachment C-6



LEGEND

- INFERRED BEDROCK SURFACE
- SURFACE ELEVATION (TAKEN FROM SURVEY POINTS AND TOPOGRAPHIC SURVEY DATA)

NOTES
BOREHOLE SPACING IS APPROXIMATE.
BEDROCK SURFACE IS INFERRED BETWEEN KNOWN BEDROCK INTERSECTIONS.
* BH15 ELEVATION TAKEN FROM HANDHELD GPS DATA IN REFERENCE TO SITE TOPOGRAPHIC SURVEY DATA PROVIDED BY SECURE ON FIGURE 2.

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ROCKY PROPOSED FULL SERVICE TERMINAL SITE

BEDROCK CROSS - SECTION A-A'

PROJECT NO.	DWN	CKD	REV
C22501059.003	MMK	JB	0
OFFICE	DATE		
EBA-RIV	April 2012		

Figure 5

APPENDIX D

CHARACTERIZING PROBABLE HYDROGEOLOGIC CONSEQUENCES CITY OF CALGARY MODULE 1 GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

TABLE OF CONTENTS

TABLES

D1.0 INTRODUCTION

This appendix describes the method to evaluate the probable hydrogeologic consequences of a proposed Low Impact Development (LID) within The City of Calgary (The City). Developers and their consultants will find this transparent step-wise procedure useful:

- To clarify the components of a development's stormwater management plan that potentially influence the pattern and quantity of groundwater flow across a development; and
- To identify the need to mitigate adverse influences on the groundwater flow system.

Assessing probable hydrogeologic consequences is an important environmental planning tool most useful at the conceptual design stage. For LID principles this protocol should be applied at the Master Drainage Plan (MDP) planning level or the Staged Master Drainage Plan (SMDP) planning level.

Sections D2.0 to D4.0 describe:

- The concerns with altering groundwater conditions with LID implementation;
- The current state of practice; and
- The method for application within The City.

D2.0 CONCERNS ABOUT ALTERING GROUNDWATER CONDITIONS WITH LID IMPLEMENTATION

Any development that manages stormwater influences the patterns and quantity of groundwater flow. For example, paved or other non-pervious areas of a development will create a recharge deficit that potentially lowers the local water table. Groundwater flow directed to these areas of lower water table may extend beyond the boundaries of a development and potentially affect water resources on adjacent properties.

Further, infiltration features used to either replace infiltration deficits or to help meet runoff target volumes potentially have hydrogeologic consequences related to:

- Ponding of water on the ground surface;
- Interference with roadways;
- Inflow to subsurface utility corridors; and
- Creation of seepage zones along topographic slopes that may create instability.

Both of these outcomes affect the principles of the LID initiative – to maintain watershed health and to ensure sustainable development within The City.

D3.0 CURRENT STATE OF PRACTICE

Environmental Impact Assessments (EIA's), used as part of the planning process for industrial and municipal developments across the province, include the evaluation of potential impacts on surface and groundwater resources. Application of the EIA process (Canadian Environmental Assessment Agency [CEAA] 2003) to groundwater resources at the conceptual design stage helps to identify and ensure that the potential hydrogeologic consequences receive consideration before stormwater management plans are incorporated into a development.

The step-wise process used on many industrial projects within the province is shown in Table D-1A. A key focus of the assessment process is identifying and characterizing the residual effects: those impacts that cannot be mitigated and may require ongoing monitoring and/or the development of a contingency plan. The method used to characterize residual effects is tabulated in Table D-1B.

D4.0 METHODS TO EVALUATE PROBABLE HYDROGEOLOGIC CONSEQUENCES

Although most frequently applied to consider human health and ecosystem sustainability, the process outlined in Section D3.0 can also be applied to water resources to help ensure LID principles are maintained.

Table D-2A provides a three-step process for assessing, mitigating, and monitoring groundwater conditions and preventing adverse consequences from occurring due to an LID development. Table D-2B provides a means for characterizing residual effects that will aid regulatory agencies in making decisions on the sustainability of an LID development.

REFERENCES

Canadian Environmental Assessment Agency. 2003. Environmental Assessment: A Critical Tool for Sustainable Development Canadian Environmental Assessment Agency Website: http://www.ceaa-acee.gc.ca/017/0004/development2001_e.htm.

TABLES

Table D-1A - Impact Assessment Method and Description of Terms

Step		Description	Hydrologic and Hydrogeologic Considerations
1	Determine Valued Eco System Components (VECS).	A VEC is "any part of the environment that is considered important by the proponent, public, scientists, and government involved in the assessment process. Importance may be determined on the basis of cultural values or scientific concern" (CEAA 1999).	Wetland, marshes, and watercourses within and adjoining the project site.
2	Identify project activities that may interact with VECS.	Identifies the components of a project that have the potential to interact with biophysical and water resources.	Interaction between surface water and groundwater during construction and post-construction operations and maintenance.
3	Identify potential impacts on valued ecosystem components.	Understanding of the potential adverse effects of the project during construction, operation, and maintenance or decommissioning and abandonment phases of a project.	Characterize changes in the level of the groundwater due to infiltration features used for stormwater management or the change expected beneath impervious areas of the development throughout the development (construction, operation, and maintenance).
4	Identify mitigation measures that may be undertaken to reduce impacts.	Measures taken within the project to mitigate adverse impacts.	Ensure target runoff volumes protect watercourses from erosion and improve sustainability of groundwater resources.
5	Determine and characterize residual environmental effects.	Effects that are likely to be in place throughout the project even after mitigation.	Permanent changes in the distribution of wetlands and marshes, the pattern of groundwater flow and the quantity discharged to local watercourses or supplied to wetland and marshes.
6	Describe significance of any residual environmental effects.	Characterize according to direction, magnitude, geographic extent, frequency, duration, and reversibility as outlined in Table D-1B.	Characterize both positive and negative changes in groundwater flow patterns created by any features used to manage stormwater.
7	Determine cumulative effects of the project.	Cumulative effects consider the actions with other past, present and future effects of the current project with those on the adjacent properties.	Consider the overall influence of development with changes created within surrounding developments.
8	Develop monitoring measures.	Follow up programs to ensure any assumptions are considered within the plan.	Monitoring of water levels.
9	Identify any knowledge deficiencies.	Information that must be collected during the project to ensure adverse environmental impacts are not realized.	Confirmed using monitoring of sites and contingency planning.

Table D-1B - Residual Impacts Rating Criteria

Criteria	Rating Term	Definition
Determine valued eco system components (VECS)	Positive	Beneficial change.
	Neutral	No change.
	Negative	Adverse change.
Geographic extent	Local	Effect is limited to the footprint of the project site.
	Regional	Effect occurs within the subdrainage basin of local watercourses.
	Beyond regional	Effect extends beyond the local watershed or sub-basin.
Duration	Short-term	Effects last less than two years.
	Medium-term	Effects last from 3 to 10 years.
	Long-term	Effect lasts longer than 10 years.
Frequency	Once	Effect occurs during construction, operations, or maintenance.
	Intermittent	Effect occurs seasonally.
	Continuous	Effect occurs continuously during either construction or during operations or both.
Reversibility	Reversible	Effect is reversed after the activity ceases.
	Partially reversible	Effect is partially reversed after activity ceases.
	Non-reversible	Effect will not be reversed if activity ceases.
Magnitude	Negligible	No measurable impact.
	Low	Potential impact may result in a decline in water flow in watercourses or declines in water levels within wetlands or marshes or an increase in erosion of watercourses.
	Moderate	Potential impact may result in a decline below baseline levels for water flow in water level or declines in water levels within wetlands or marshes or increase in erosion of watercourses.
	High	Potential impact threatens sustainability of water resources.

Table D-2A - Assessment of Probable Hydrogeologic Consequences

Step		Hydrologic and Hydrogeologic Considerations	Description of Methods to Be Used
1	Determine Valued Eco System Components (VECS)	Wetland, marshes, and watercourses within and adjoining the project site.	Use site plans and create a Conceptual Site Model (Appendix E).
		Interaction between surface water and groundwater during construction and post-construction operations and maintenance.	Use Conceptual site model (Appendix E) to illustrate changes in groundwater levels, and discharge or recharge to local wetlands, marshes, and watercourses.
		Characterize changes in the level of the groundwater due to infiltration features used for stormwater management or the change expected beneath impervious areas of the development throughout the development (construction, operation, and maintenance).	Use groundwater mounding evaluation (Appendix G).
2	Mitigative Action	Ensure target runoff volumes protect water courses from erosion and improve sustainability of groundwater resources.	Use Table D-2B to characterize residual effects.
		Permanent changes in the distribution of wetlands and marshes, the pattern of groundwater flow, and the quantity discharged to local water courses or supplied to wetland and marshes.	Assess impact of water level changes on water resources using groundwater mounding methods of Appendix G.
		Characterize both positive and negative changes in groundwater flow patterns created by any features used to manage stormwater.	
		Consider the overall influence of development with changes created within surrounding developments.	Evaluate other developments to ensure no compounding influence results.
3	Monitoring and Contingency Planning	Monitoring of water levels during construction and post construction of stormwater management features.	Based upon appraisal of uncertainty or range in parameters use guidance in Section 4 to develop monitoring plan and to collect more site information use and guidance in Appendix A to establish monitoring well network and guidance on infiltration rates to estimate percolation to the water table.
		Confirm probable consequence using water level monitoring of stormwater management sites and develop contingency plans.	Create a contingency plan.

Table D-2B - Rating Criteria for Probable Hydrogeologic Consequences of Residual Effects

Criteria	Rating Term	Definition
Determine valued eco system components (VECS).	Positive	Beneficial if water level rises but will not impact buried utilities, seepage to slopes, ponding on surface, or interfere with roadways.
	Neutral	No change.
	Negative	Adverse impact if water level rises but flows into buried utilities, creates seepage zones on slopes, ponds on the ground surface, or interferes with roadways.
Geographic extent	Local	Effect is limited to the footprint of the stormwater control/ infiltration feature.
	Regional	Effect occurs beyond the stormwater control/infiltration feature but within the development footprint.
	Beyond regional	Effect extends beyond the development footprint.
Duration	Short-term	Effects *last less than two years.*
	Medium-term	Effects last from three to ten years.
	Long-term	Effect lasts longer than ten years.
Frequency	Once	Effect occurs during construction, operations, or maintenance.
	Intermittent	Effect occurs seasonally.
	Continuous	Effect occurs continuously during either construction or during operations or both.
Reversibility	Reversible	Effect is reversed after the activity ceases.
	Partially reversible	Effect is partially reversed after activity ceases.
	Non-reversible	Effect will not be reversed if activity ceases.
Magnitude	Negligible	No measurable impact.
	Low	Potential impact may result in a decline in water flow in water courses or declines in water levels within wetlands or marshes or an increase in erosion of watercourses.
	Moderate	Potential impact may result in a decline below baseline levels for water flow in water level or declines in water levels within wetlands or marshes or increase in erosion of watercourses.
	High	Potential impact threatens sustainability of water resources.

APPENDIX E

FRAMEWORK TO DEVELOP A CONCEPTUAL SITE MODEL CITY OF CALGARY – MODULE 1 GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

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FIGURES

E1.0 CONCEPTUAL SITE MODEL PREPARATION

This appendix provides guidance on the preparation of a Conceptual Site Model (CSM). A CSM is a simplified graphical and textual description of the circulation of groundwater and of the interaction between groundwater and surface water within a proposed development. The CSM provides a vision of how stormwater management features within a development will interact with hydrogeologic systems and allows pre-development and post-development conditions to be compared visually and to be quantified so decisions on the selected stormwater management features can be made. Preparation of a CSM is inherent in all hydrogeological assessments. Despite the nearly synonymous association of hydrogeologic assessments and a CSM, no standard specification exists for creation of a CSM.

This document provides a framework for generating a CSM that will aid developers and their consultants in planning and processing hydrogeologic information for Low Impact Development LID projects within The City of Calgary (The City).

A CSM should be provided at every planning level, although the degree of sophistication and detail of the graphic and textual description and the level of quantification possible increases from the Watershed Planning (WP) and Master Drainage Plan (MDP) planning levels to the Staged Master Drainage Plan (SMDP), to the Pond Report, to the Stormwater Management Report (SWMR), and to the Development Site Servicing Plan (DSSP) planning levels.

Sections E2.0 to E4.0:

- Describe the purposes of preparing a CSM;
- Discuss the elements and steps to be followed to prepare a CSM; and
- Present an example of a CSM and the level of detail that should be included within a CSM at the various drainage planning levels.

E2.0 PURPOSES OF A CONCEPTUAL SITE MODEL

The purposes of a CSM are:

- To integrate pertinent technical information from various sources (i.e., meteorological, geomorphological, geological, hydrological, and hydrogeological) with the site development plans to illustrate the impact on groundwater flow and the interaction between groundwater and surface water that the development might generate;
- To support the locations selected for stormwater management features to manage stormwater;
- To identify the information needed to aid in quantifying hydrogeologic processes (i.e., the hydrogeologic water balance, recharge and discharge rates, and flow onto and off of a development site); and
- To evaluate the potential threat of adverse impacts to off-site water resources posed by a development and the opportunities to mitigate that threat by returning stormwater to the subsurface.

E3.0 ELEMENTS AND STEPS TO PREPARE A CONCEPTUAL SITE MODEL

Both the American Society for Testing and Materials (ASTM 1995) and Brassington and Younger (Brassington and Younger 2009) provide guidance on the generation of a CSM. The ASTM document focuses on creating a CSM for the purposes of a risk assessment of a contaminated site and is not provided specifically for LID purposes. However; the ASTM documents provide guidance on how conceptual linkages between groundwater and surface water, flow pathways, and impact or watershed health can be incorporated into an LID initiative. Similarly, Brassington and Younger provide a step-wise process to follow to use a CSM, and to create a numerical model of a groundwater flow system, but not specifically for LID purposes. However; those concepts have been modified for use by The City to apply to an LID initiative.

In principle, a CSM must describe:

- The physical framework of the development site, including the ground surface topography, locations of wetlands, marshes, ponds, and streams, stratigraphic units that underlie the site – to a minimum depth of the upper 20 m, and categorization of geologic units as aquifers and aquitards;
- The patterns of groundwater movement within the development area;
- Areas of transfer or interaction of groundwater and surface water resources; and
- Proposed stormwater management features and their geotechnical constraints and probable hydrogeologic consequences on on-site and off-site surface water and groundwater resources.

The above information is the minimum information required for WP and MDP planning purposes. At more detailed planning levels (SMDP, Pond Report, or SWMR and DSSP planning levels), quantification of groundwater flow, infiltration, or percolation from the surface, and the build-up of groundwater mounds on the water table or impermeable layers should also be illustrated within the CSM.

Table E-1 identifies the steps to be followed to prepare this illustration and how the relevant information is to be obtained within the various drainage planning levels. Also shown in Table E-1 are the planning levels where quantification of the hydrogeologic processes is needed to support stormwater management feature selection and design.

E4.0 EXAMPLE OF A CONCEPTUAL SITE MODEL AND THE CHANGES WITHIN EACH PLANNING LEVEL

Figure E-1 illustrates the composition of a CSM to be created throughout the various drainage planning levels of the LID initiative.

REFERENCES CITED

ASTM, 1995. Standard Guide for Developing Conceptual Site Models for Contaminated Sites, American Society for Testing and Materials Designation No. E, 1689-95.

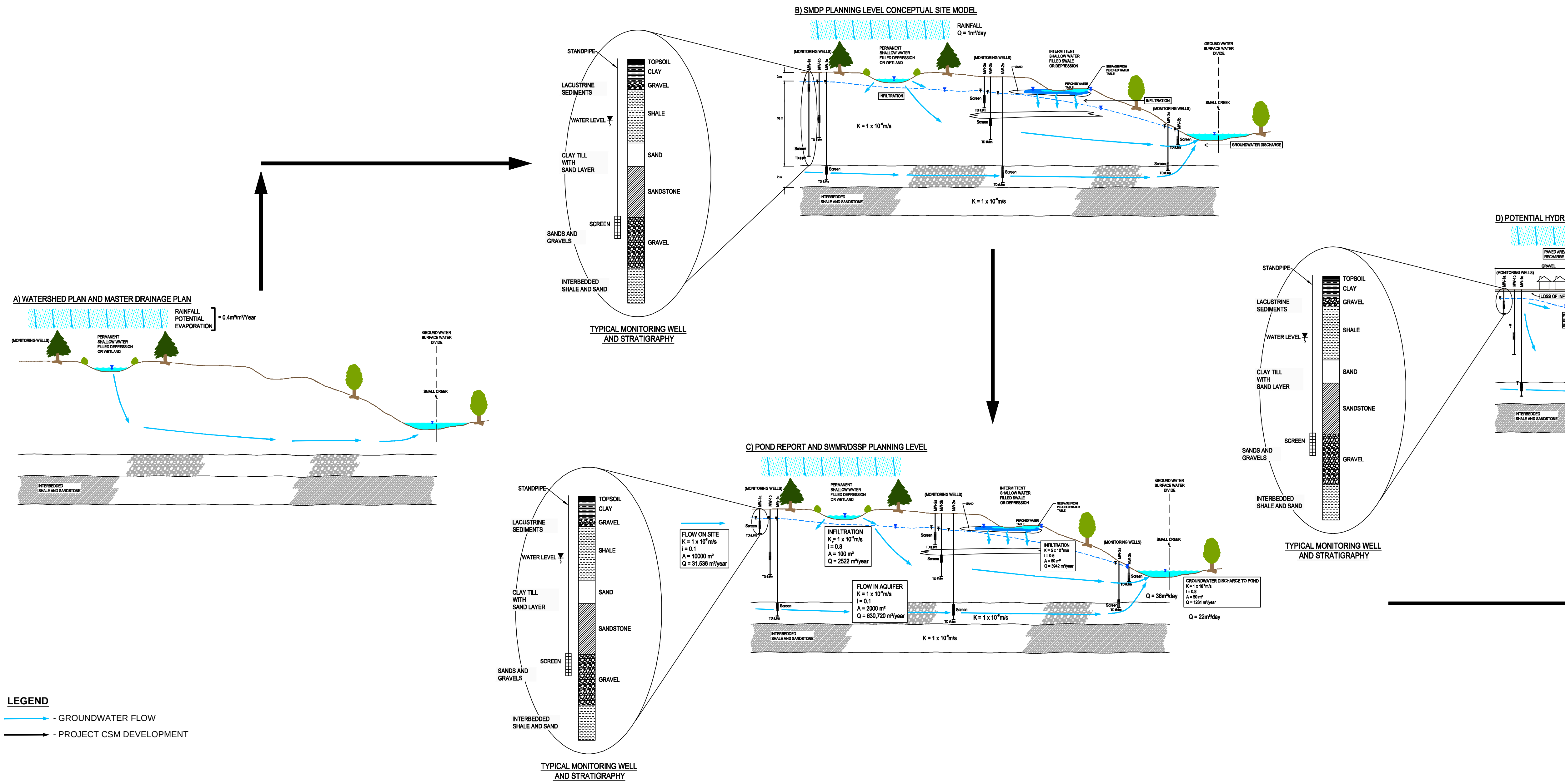
Brassington, F.C., and Younger, P.L. 2010. A Proposed Framework for Hydrogeologic Conceptual Modelling, Water and Environment Journal, 24, pages 262-273.

TABLES

Table E1 - Steps to Prepare a CSM

CSM Element	Step	Activity	Primary Sources of Information or Activity by Planning Level						Reporting Requirements	Calculation Needs
			WP	MDP	SMDP	Pond Reports	SWMR	DSSP		
Preparation	Define Objectives	Describe development needs and vision of the completed development.								
Physical Framework	Define Topography and Surface Water Features and Potential Hazardous Slopes		Walk over survey – Site Reconnaissance	Walk over survey – Site Reconnaissance	Walk over survey – Site Reconnaissance	Walk over survey – Site Reconnaissance	Walk over survey – Site Reconnaissance	Walk over survey – Site Reconnaissance	Site plan with elevation contours, water courses and wetlands laid out and mark out areas of potential slope stability concerns	
	Geology	Map and categorize stratigraphic units.	Published geologic maps and plans Water well records from files of Alberta Environment – Site Reconnaissance	Published geologic maps and plans Water well records from files of Alberta Environment – Site Reconnaissance	Borehole records prepared following geotechnical investigation protocol Appendix C	Borehole records prepared following geotechnical investigation protocol Appendix C	Borehole records prepared following geotechnical investigation protocol Appendix C	Borehole records prepared following geotechnical investigation protocol Appendix C	Stratigraphic sections of major geologic units with aquifer and aquitard units labelled (Figure E-1a)	
	Aquifer Framework	Review of water well yields and geologic units.	Published geologic maps and plans Water well records from files of Alberta Environment	Published geologic maps and plans Water well records from files of Alberta Environment	Borehole records prepared following geotechnical investigation protocol Appendix C	Borehole records prepared following geotechnical investigation protocol Appendix C	Borehole records prepared following geotechnical investigation protocol Appendix C	Borehole records prepared following geotechnical investigation protocol Appendix C	Characterize potential well yields and aquifer potential – borehole records and stratigraphic sections prepared (Figure E-1 b)	
Patterns of Groundwater Movement	Direction of Groundwater Flow	Map direction of vertical and horizontal groundwater movement.	Topographic elevations as described in Appendix A – Desktop Reviews	Topographic elevations as described in Appendix A – desktop reviews	Installation of nested monitoring wells and water table wells as described in Appendix A	Installation of nested monitoring wells and water table wells as described in Appendix A – hydraulic conductivity estimated from monitoring well response tests - Appendix A	Installation of nested monitoring wells and water table wells as described in Appendix A – hydraulic conductivity estimated from monitoring well response tests - Appendix A	Installation of nested monitoring wells and water table wells as described in Appendix A – hydraulic conductivity estimated from monitoring well response tests - Appendix A	Borehole records and monitoring well completion details, water level monitoring data and plotted on a cross section of the site (Figure E-1 B)	Estimation of hydraulic conductivity values including geometric mean values and groundwater flow rates – Appendix A
Groundwater Surface Water Interaction	Determine Recharge or Discharge Potential	Measure vertical gradients and determine infiltration and percolation rates.	Use topographic gradient and slope position to define recharge or discharge rates and use Appendix H – indirect measurements of infiltration or percolation rates	Use topographic gradient and slope position to define recharge or discharge rates and use Appendix H – indirect measurements of infiltration or percolation rates	Measurement of hydraulic gradient close to wet lands may need drive point wells described in Appendix A	Direct measurement of infiltration rate or percolation rate needed for proposed pond locations following in situ testing methods of Appendix B	Direct measurement of infiltration rate or percolation rate needed for proposed pond locations following in situ testing methods of Appendix B	Direct measurement of infiltration rate or percolation rate needed for proposed pond locations following in situ testing methods of Appendix B	Tabulation of infiltration rates	Estimate potential loading rates (infiltration losses estimate)
Geotechnical Constraints and Probable Hydrogeologic Consequences	Incorporate Development Plan	Superimpose proposed development and changes in terrain on the pre-development site condition.	Not typically useful at this planning level	Not typically useful at this planning level	Estimate potential for groundwater mound build up using methods of Appendix g and estimate slope stability for design of ponds Appendix F	Estimate potential for groundwater mound build up using methods of Appendix g and estimate slope stability for design of ponds Appendix F	Estimate potential for groundwater mound build up using methods of Appendix G and estimate slope stability for design of ponds Appendix F	Estimate potential for groundwater mound build up using methods of Appendix G and estimate slope stability for design of ponds Appendix F	Parameters used and assumptions made in assessing potential for groundwater mounding – slope stability safety factors	Assessment of issues with groundwater mound build up as potential for ponding on surface, breakout on side slopes, interference with roadways or flow into buried utility corridors and slope design to stabilize pond slopes

FIGURES



CLIENT

THE CITY OF CALGARY

eba
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THE CITY OF CALGARY LID PROJECT

CSM BY PLANNING LEVEL

PROJECT NO.	DWN	CKD	REV
C12101310.004	MMK	TD	0

OFFICE	DATE
EBA-RIV	July 2012

Figure E-1

APPENDIX F

METHODS TO EVALUATE SUBGRADE AND SLOPE STABILITY CITY OF CALGARY MODULE 1 GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENT

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FIGURES

F1.0 INTRODUCTION

This appendix provides guidance on identification and evaluation of geotechnical stability issues in support of Low Impact Development (LID) projects within The City of Calgary (The City). These geotechnical issues are considered for all development; however, the developer must give further consideration to changes in the hydrogeologic regime associated with the LID features.

During the planning and development submission process, The City requires Geotechnical Evaluation Reports, Slope Stability Reports for slopes across a property greater than 15% and to establish development setbacks, and Deep Fill Reports for all plans with areas that will have a fill thickness greater than 2 m. All of the geotechnical evaluations should be carried out by qualified geotechnical engineering professionals. These reports are reviewed by The City as part of development applications.

F2.0 GEOTECHNICAL STABILITY ISSUES

Geotechnical stability issues proximate to an LID feature that should be considered include the following:

- Stability of natural slopes;
- Stability of earth fill slopes, including those constructed during site grading, and roadway embankment slopes;
- The potential effects of a shallow groundwater table on roadway subgrade strength and stiffness and on the potential for frost heaving and thaw softening;
- The potential effects of a shallower groundwater table on foundations for buildings, including the requirement for weeping tile, the swelling of high plastic (expansive) foundation soil, and settlement of deep fill; and
- The stability of cut and/or earth fill slopes for ponds and swales based on: the design and configuration of the pond/swale; the soil properties determined by the subsurface investigation; the porewater pressures associated with the post-construction condition, the normal operating conditions, and the rapid drawdown condition.

F3.0 CURRENT STATE OF PRACTICE

The current state of practice is to consider both the current groundwater table and the probable long-term, post-development groundwater table in the evaluation, assessment, and design for each of the above-listed geotechnical stability issues. Methods to measure the depth to groundwater and the groundwater flow rates and directions are considered in Appendix A. Methods to estimate the additional recharge from LID features is considered in Appendix B. Methods to evaluate the probable hydrogeologic consequences, to develop a conceptual site model, and to evaluate the probable hydrogeologic consequences of groundwater mounding beneath and proximate to infiltration basins are presented in Appendices D, E, and G, respectively.

The current state of practice for slope stability assessments is included within the next subsection (slope stability assessment methods).

The potential effects of a shallow groundwater table on a road subgrade are usually considered by conducting saturated California Bearing Ratio laboratory tests followed by analysis of the pavement structure and loading conditions.

The potential effects of a shallow groundwater table on winter frost heaving and subsequent spring thaw settlement/softening on a road subgrade are usually evaluated by consideration of the soil grain size distribution, the depth to the water table, and the seasonal frost penetration depth. When the frost heave and thaw settlement/softening are considered unacceptable, these effects may be mitigated by a thicker clean granular structure, the inclusion of subdrains, and/or the inclusion of rigid board insulation.

The potential effects of a shallower groundwater table on the swelling of high plastic (expansive) foundation soil are initially evaluated using a suite of geotechnical laboratory tests. Because it is difficult to determine the rate and magnitude of soil swelling, the foundation system is selected to either accommodate (pile foundations and a structural floor slab) and/or tolerate the maximum anticipated movements.

F4.0 SLOPE STABILITY ASSESSMENT METHODS

F4.1 Desktop Study and/or Site Reconnaissance Review

The desktop study and site reconnaissance review assessment methods are suitable for the Watershed Plan (WP), Water Management Plan (WMP), and Master Drainage Plan (MDP) planning levels.

The desktop study is conducted by reviewing: published geology and hydrogeology maps; previous geotechnical reports that may be available in the vicinity of the slopes; topographic maps that show the slope heights, angles, and morphology; and both current and historical aerial photographs (preferably stereo photo pairs). This desktop review is used to identify historic and recent slope instability, drainage patterns and discharge locations from the uplands, locations of groundwater discharge, and a reasonable indication of the slope instability failure modes relative to the surficial geology.

Site reconnaissance consists of a site walkover by an experienced geotechnical engineer or engineering geologist, who will observe, photograph, and map the locations of slumping ground, tension cracks, evidence of groundwater seepage discharge, erosion, changes in vegetation, and evidence of the geologic sequence where exposed. For a large development, this ground reconnaissance may be supplemented by a flyover (helicopter or small fixed-wing aircraft).

The desktop study is typically limited to natural slopes along river valleys, creeks, coulees, and ravines.

The site reconnaissance should be conducted for both natural and man-made (cut and/or fill) slopes.

F4.2 Slope Stability Analysis

This slope stability assessment method is suitable for the Staged Master Drainage Plan (SMDP), Pond Report, and Stormwater Management Report (SWMR) planning levels.

F4.2.1 Soil, Groundwater, Design, and Operational Information Required

Geotechnical investigations are conducted to characterize the subsurface soil stratigraphy and the hydrogeologic conditions. The subsurface stratigraphy and hydrogeologic conditions of significant natural slopes (river valley, coulee, ravine, etc.) are typically more complex than subsurface conditions for man-made cut and/or fill slopes, and generally require a more detailed geotechnical investigation and follow-up monitoring.

The City guidelines for Geotechnical Reports, Geotechnical Reports for Slope Stability, and Geotechnical Reports for Deep Fills are currently under review and revision. Updated guidelines for these geotechnical reports will be ready for review by the development industry during the third quarter of 2013.

Appendix C describes site investigation and laboratory testing methods to characterize soil conditions. Characterization of the soil stratigraphy is accomplished by visual observations of the soil returns (auger cuttings, split spoon samples, Shelby tube samples, rock core samples), by in situ testing (Standard Penetration Testing, cone penetration testing, etc.), and by laboratory testing of selected soil samples (index testing, shear strength testing, etc.)

Characterization of the hydrogeologic regime is accomplished by observation of soil stratigraphy (water-bearing or potential water-bearing layers and intermediate low permeability layers), soil moisture content changes, zones of seepage into the borehole, etc. Based on this characterization, the geotechnical engineer will determine at which depths piezometers should be installed to measure the groundwater tables (piezometric heads) and porewater pressures. Depending on the anticipated pore pressure response (related to the hydraulic conductivity of the soil stratum) discrete zone standpipe piezometers, pneumatic piezometers, or vibrating wire piezometers may be selected. The number of groundwater/pore pressure monitoring events required following the installation of the piezometers will be highly dependent on the complexity of the subsurface stratigraphy and hydrogeologic regime, and how the geologic regime responds to seasonal recharge, flow, and discharge.

Based on the results of the subsurface investigation, the soil and groundwater/piezometric parameters required for the slope stability analysis are as follows:

- The slope profile (angle, height, etc.) of the natural slope or the man-made cut and/or fill slope;
- The soil stratigraphic sequence (stratigraphy);
- Soil strength and unit weight parameters of each stratigraphic layer, including any compacted fill that is to be placed during construction of the ponds and swales;
- Piezometric (groundwater head) surfaces and pore pressure (B-Bar) response parameters; and
- The operational water levels (including normal, high, and low) for the pond operations.

It should be noted that the soil strength parameters selected for the slope stability analysis are generally based several of the following: laboratory shear strength testing, published literature for similar site projects in the region and having similar geology, published correlations with soil index properties, and the experience of the geotechnical engineer.

It should be further noted that for natural slopes, such as river valleys, coulees, and ravines, the geotechnical engineer will frequently carry out a parametric back-analysis of the existing stable or marginally unstable slopes and revise the strength parameters for one or more of the stratigraphic soil layers.

F4.2.2 Slope Stability Analysis Methods

Slope stability analyses are almost always conducted using commercially available software. The most common software used by geotechnical consultants in Alberta is SLOPE/W 2007 (most current Version being 7.19), developed and supported by GEO-SLOPE International Ltd.

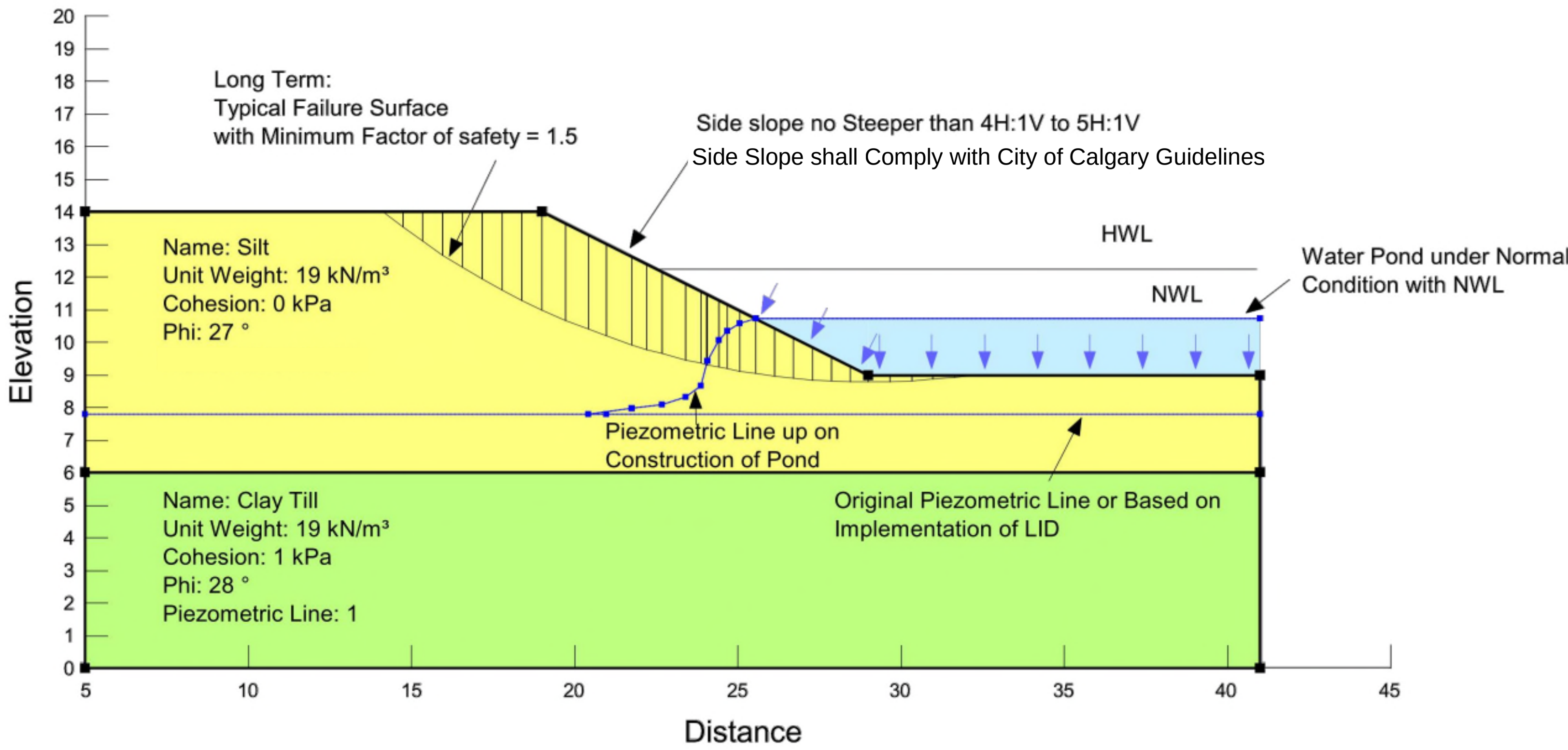
Worked examples of slope stability analyses for a pond slope are provided on Figures F-1 and F-2.

FIGURES

Appendix F: Worked Example of Slope Stability Assessment

Sample Slope Stability Analysis for Side Slope of Water Pond under Normal Condition

Methods: Morgenstern-Price, Spencer and Generalized Limit Equilibrium (GLE)

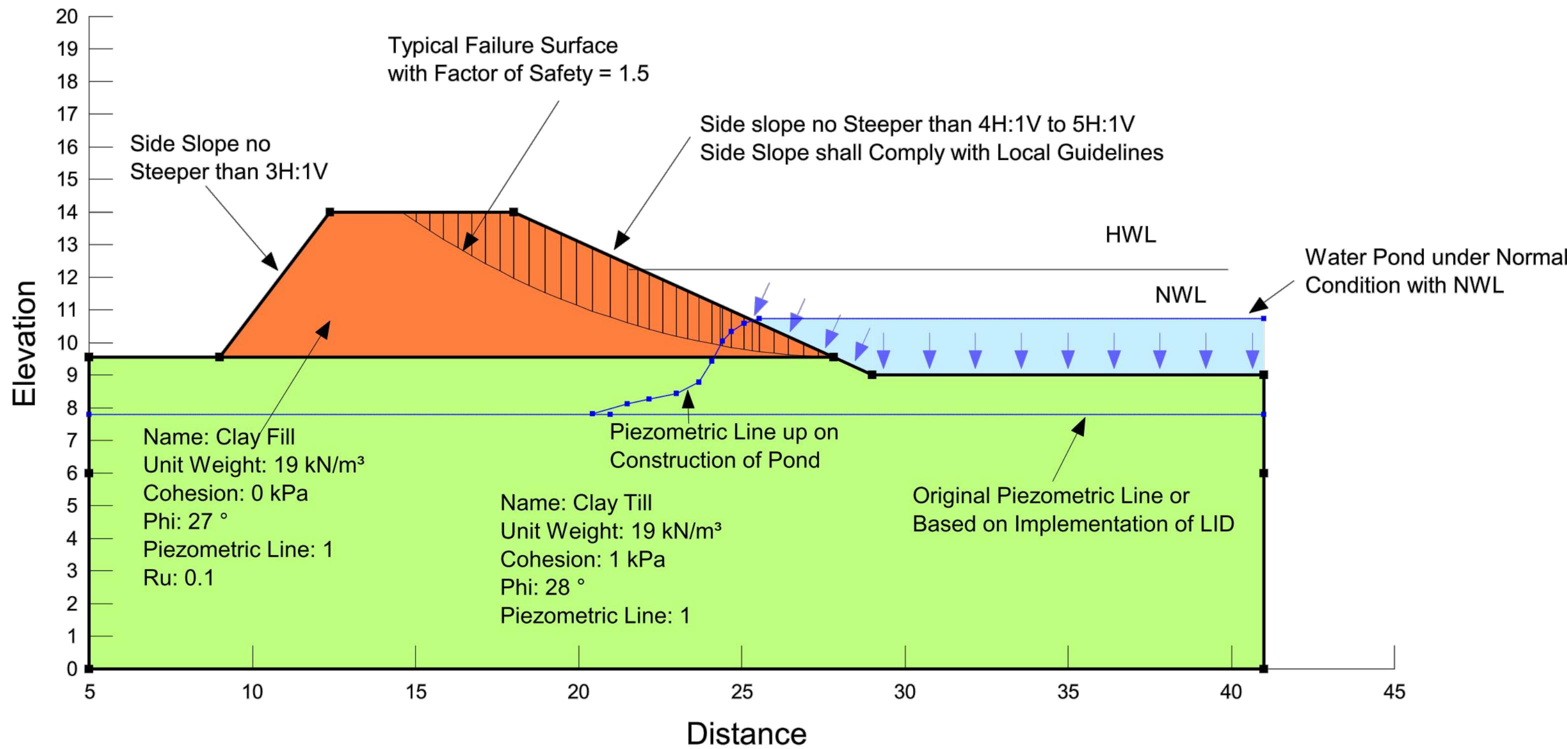


CLIENT: THE CITY OF CALGARY	CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM — SLOPE STABILITY ANALYSIS EXAMPLE 1				
eba A TETRA TECH COMPANY	PROJECT NO. C12101310.003	DWN MMK	CKD TD	REV 0	Figure F-1
	OFFICE EBA-RIV	DATE December 2012			

Appendix F: Worked Example of Slope Stability Assessment

Sample Slope Stability Analysis for side Slope of Water Pond Constructed above Ground

Methods: Morgenstern-Price, Spencer and Generalized Limit Equilibrium (GLE)



CLIENT THE CITY OF CALGARY	CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM SLOPE STABILITY ANALYSIS EXAMPLE 2			
eba A TETRA TECH COMPANY	PROJECT NO. C12101310.003	DWN MMK	CKD TD	REV 0
	OFFICE EBA-RIV	DATE December 2012		

Figure F-2

APPENDIX G

METHODS TO EVALUATE THE CONSEQUENCE OF GROUNDWATER MOUNDING BENEATH INFILTRATION BASINS

CITY OF CALGARY – MODULE 1 GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

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G1.0 INTRODUCTION TO GROUNDWATER MOUNDING

This appendix provides an overview of methods to estimate the height and lateral extent of groundwater mounds built up beneath infiltration features proposed to manage stormwater using Low Impact Development (LID) principles. The information presented can be used by developers and their consultants to assess the potential for adverse consequences of infiltration features both qualitatively and quantitatively and hence aid in the optimal placement of stormwater control features within a development.

Guidance is provided here:

- To use analytical methods to assess potential for groundwater mounding beneath single infiltration features; and
- To use either analytical methods or numerical modelling methods for multiple infiltration features within a development depending upon whether infiltration features are commonly constructed (similar geometry and uniformly spaced) or are dimensioned differently and situated non-uniformly throughout the development respectively.

Within any development, evaluation of the potential adverse consequences of groundwater mounding potentially influences the placement of infiltration features relative to roadways, utility corridors, and areas of changes in topographic slope. Therefore; evaluation of groundwater mounding is most useful during the Staged Master Drainage Plan (SMDP), Pond Report, or subdivision (Stormwater Management Report [SWMR] and Development Site Servicing Plan [DSSP]) planning levels.

Sections G2.0 to G4.0 describe the methods to evaluate groundwater mounding according to three topics:

- Concerns about groundwater mounding with respect to LID projects;
- Current state of practice to evaluate the height and lateral extent of groundwater mounds; and
- Methods to evaluate the consequences of groundwater mounding.

G2.0 CONCERNS ABOUT GROUNDWATER MOUNDING

Groundwater mounding refers to the buildup (increase in elevation) of the water table or the creation of a perched water table on the surface of a low permeable/impermeable geologic material as a consequence of surface infiltration. Deliberately induced surface infiltration has two purposes:

- It replaces an infiltration deficit created by loss of groundwater recharge through non-porous facilities (building and pavements) within a residential or commercial development; and
- It helps a development meet runoff volume targets by returning some of the stormwater captured within the development to the subsurface through stormwater retention ponds, surface drainage courses, or infiltration galleries (such as a toe drain or a French drain).

Mounding of the water table, however; can have some undesired consequences:

- Ponding of stormwater at the ground surface;
- Interference with roadways or other infrastructure down-gradient from the infiltration feature;
- Induced inflow to subsurface utilities (including flow into sanitary and storm sewers);
- Reduced slope stability – if the lateral extent of the mound causes new seepage zones on steep slopes; and
- Changed surface water runoff volumes in areas of surface water and groundwater interaction or the creation of new watercourses in areas of higher volumes of seepage to sideslopes.

An evaluation of these potential consequences (which are a function of the height and lateral extent of the groundwater mound) and their impact on the proposed servicing strategies for the future development must be part of the submission requirements for an SMDP, SWMR, or Pond Report. These methodologies are consistent within the assessment of the impact of seepage from septic fields used for wastewater systems recommended by Safety Code Council, 2009 for Alberta Private Sewage System Standard of Practice. The principal difference being that, for stormwater management facilities, infiltration is a short-term event (a few days or a week) while the duration of infiltration events for septic fields is for many years at, a more or less, a constant rate of loading. Nevertheless, facilities for stormwater infiltration should be evaluated in a similar manner to ensure that nearby surface and subsurface infrastructure are sustainable for the longer term.

G3.0 CURRENT STATE OF PRACTICE OF GROUNDWATER MOUND EVALUATION

Stormwater source control measures, such as bio-retention areas, bio-swales, absorbent landscapes, and stormwater basins (comprising both cluster and high-density wastewater soil-absorption systems) are increasingly used in land use planning to support LID principles. Stormwater source control measures are incorporated into a development for two purposes:

- To reduce the net loss of infiltration, where an infiltration deficit created by non-porous areas of development potentially threaten the viability of down-gradient water resources; and
- To manage the surplus stormwater needed to satisfy target runoff volumes where watershed deterioration is a concern.

To ensure that a proposed site has the capacity to accommodate precipitation in excess of natural infiltration, hydrogeologic assessments are required, particularly at sites where the natural infiltration volume is directed to smaller engineered infiltration feature/features. Practitioners and stakeholders must understand the issues to conduct proper investigations and evaluations before attempting to evaluate or undertake stormwater management measures.

Recharge facilities and their effects on the underlying water table have been studied for decades (Carleton 2010, Hantush 1967, Glover 1960, Warner et al. 1989, and Zomorodi 2005). In 2005, the International Groundwater Modelling Center of Golden, Colorado, commissioned by the National Decentralized Water Resources Capacity Development Project of Washington University, St. Louis MO, published a guidance document entitled "Guidance in Evaluation of Potential Groundwater Mounding Associated with Cluster and High Density Wastewater Soil Absorption Systems". This document, Poetter et al. (2005), discusses the application of a variety of analytical techniques to evaluate groundwater infiltration through single infiltration features and for clustered and high density water infiltration systems (multiple infiltration features) that are placed uniformly and of comparable dimensions. Poetter et al. (2005) also evaluates the use of numerical codes (e.g., Modflow) as an alternate means to evaluate groundwater infiltration system design (both single and multiple infiltration features within a development).

Numerical solution presents a greater level of predictive effort than analytical solutions. Numerical solution evaluations are recommended for sites where numerous infiltration facilities are located and placed non-uniformly within a development.

G4.0 METHODS TO EVALUATE THE CONSEQUENCES OF GROUNDWATER MOUNDING

Sections G4.1 to G4.4 describe:

- The site-specific information required before proceeding to evaluate the consequences of groundwater mounding;
- The procedures to be followed to evaluate groundwater mounding beneath a single infiltration feature;
- The procedures to be followed to evaluate groundwater mounding beneath multiple infiltration features; and
- The practices to be followed to report on the results of the mounding evaluation.

G-4.1 Site-Specific Information Requirements

This section describes the site-specific information required before proceeding to evaluate groundwater mounding. Infiltration and percolation rates are both used in this description and both refer to the process of allowing water to enter the subsurface and vertically migrate to the water table or accumulate on the surface of a lower permeability/an impermeable horizon. The information is described in a procedural format:

1. Use the site plan to show a location of the proposed infiltration feature/features within the development.
2. From the conceptual site model, show the current and planned ground surface elevation across the infiltration area, the site stratigraphic units, position of the current or pre-development water table, and the direction of groundwater flow. Appendix E provides guidance on the creation of a conceptual site model.
3. Use borehole records to identify the depth of impermeable barriers within the subsurface where a perched water table may build up and water level measurements to determine the depth to the water

table. Appendices A and C provide guidance on borehole record keeping and monitoring well completion records.

4. Use these depths to calculate a theoretical H_{max} or z (depending upon convention used in the applicable analytical solution to calculate the groundwater mound height). These two terms define the maximum height of mound allowable before water breakout occurs at ground surface. H_{max} or z should be either the depth to an impermeable layer or the water table, whichever is shallower. H_{max} or z should be adjusted for the base elevation of an infiltration basin, rain garden, absorbent landscape, or infiltration gallery (H_{max} or z is the depth to the impermeable barrier or water table minus the depth of the base of the infiltration feature).
5. Estimate the total design infiltration volume required. That volume is either the volume needed to make up the infiltration deficit or the volume needed to support the runoff target volumes. This volume is expressed as Q in units of m^3/day.
6. Summarize the vertical hydraulic conductivity extracted either from:
   - Tabulated values in Appendix H for preliminary site assessments and from detailed design purposes that include site-specific tests; or
   - Surface infiltration tests as described in Appendix B for site assessment and verification testing purposes; or
   - Percolation test results (as described in Appendix B for stormwater management feature selection and detailed design purposes). Plot the range and calculate a mean value for the vertical hydraulic conductivity.
7. Convert the vertical hydraulic conductivity to a design infiltration or percolation rate by assigning a safety factor of 1/5 of the mean value for the vertical hydraulic conductivity. This factor is recommended to allow for clogging (after Auger 2004, City of Calgary [The City] Source Control Handbook 2007). This value is subsequently termed q' (m/day) the volumetric loading rate per unit area per day. This value is equivalent to the K_{sat} value reported in Section 3.2 and is maximum rate of infiltration or percolation allowable because it is derived from the vertical saturated hydraulic conductivity as measured from the in situ test results. The q' terminology is continued here to differentiate between an infiltration rate and the soil property of saturated hydraulic conductivity. In some circumstances, a developer may define q' differently. In some cases, a developer may decide to define his loading rate (q') by dividing the volume of infiltration to be managed (Q) by the area of the infiltration basin (A). However; the loading rate desired from this type of calculation should in no manner be inferred to represent the capacity of the receiving soil to manage the desired infiltration volume because it ignores the soil property of hydraulic conductivities in the calculation. However; using this manner of calculation, the volumetric loading rate (q') desired must be less than the vertical saturated hydraulic conductivity; otherwise ponding on the infiltration surface occurs. It is, therefore; advisable to assign a safety factor to ensure that the ponding on the ground surface does not occur. The safety factor shall consider the full range of the hydraulic conductivity measured in the percolation test. A prudent safety factor would be to reduce the q' to the lowest value of the vertical

hydraulic conductivity measured from the infiltration or percolation rate tests rather than the mean value. Further discussion of safety factors is provided in Section 3.2 under the analysis of uncertainty.

8. Calculate the infiltration area needed by:

$$A = \frac{Q}{q'}$$

   This is area of the base area of the infiltration feature where infiltration is allowed to occur and not the ground surface area of the infiltration facility (i.e., exclude the area between successive infiltration trenches or galleries). This is the minimum infiltration area required.

9. Calculate the horizontal saturated hydraulic conductivity using the results of the monitoring well response tests (K_h) – this value is not needed in all estimates for groundwater mounding only for those that involve the buildup of a pre-existing water table but a vertical saturated hydraulic conductivity will need to be estimated for any geologic layer than might inhibit infiltration. This geologic unit might be below the depth of in situ testing of the vertical hydraulic conductivity used to estimate the infiltration rate (ground surface) or percolation rate (1.5 m below the proposed excavation depth of subsurface infiltration feature). An estimate can be made for the vertical hydraulic conductivity of these geologic barriers by assuming the layer is 100 times less permeability than the horizontal hydraulic conductivity or by using measurements made from soil cores.

10. Apply the methods described in Sections G4.2 and G4.3 to estimate the magnitude of water table mounding that will occur using various dimensions (length and width) of infiltration facilities that will best accommodate the available land and avoid unintended consequences:

    - Break-out of water by ponding at the ground surface;
    - Interference with roadways or other infrastructure;
    - Inflow to subsurface utilities, including sanitary and storm water sewers; and
    - Seepage zones on the sides of nearby slopes or planned landscape features; a change to surface water runoff patterns.

G4.2 Analytical Solution for Single Infiltration Basins

Two analytical solutions have been extracted from the technical literature to assess the consequences of a water table mound beneath an infiltration facility.

These solutions are:

- The buildup of a mound on the surface of a low permeability/impermeable layer developed by Khan et al. 1976 (as described in Poetter et al. 2005); and
- The buildup of a water table on a pre-existing water table by Carleton et al. (2010) and by Poetter et al. (2005).

G4.2.1 Khan et al. 1976 for Estimating Water Table Buildup on an Impermeable Layer

The Khan et al. (1976) solution is cited by Poetter et al. (2005) as being the most useful in assessing the height and potential gradient of infiltration water on a sideslope due to a buildup on an impermeable layer in the unsaturated zone. As stated by Poetter et al., "while this solution does not specifically address unsaturated-flow physics, it is a good tool for engineering applications."

Figure G-1 illustrates the conceptual model upon which the Khan solution is based. A worked example of this solution is provided in Appendix G-A with the definitions of all of the parameters used by the analytical solution. The relevant equations to address particular consequences of buildup of a water table mound on an impermeable layer are as follows:

- To estimate the lateral extent of a mound:

$$L = W \frac{q'}{K_2}. \qquad \text{Equation G-1}$$

Where:

- L is the lateral extent of the mound from the centre of the infiltration feature (m);
- W is the width of the infiltration feature (m);
- q' is the design infiltration rate (m/s) or the K_{sat} ; and
- K_2 is the saturated hydraulic conductivity of the impermeable layer (m/s).

This equation is most useful to determine the potential for breakout of infiltration water on a sideslope by comparing L to the ground surface topography.

- To estimate the height of a water table mound beyond the infiltration area:

$$H = \left(\frac{K_2}{K_1}\right)^{1/2} \bullet (L - x) \qquad \text{Equation G-2}$$

Where:

- H is the height of water build up on an impermeable layer (m);
- K_1 is the hydraulic conductivity of the soil through which infiltration occurs (m/s);
- K_2 is the hydraulic conductivity of the impermeable layer (m/s);
- L is the lateral extent of the potential groundwater mound (m); and
- x is some distance from the mound where we wish to know the potential mound height (m).

This equation is particularly useful if we want to know the impact on buried utilities or road beds located some lateral distance from the mound.

- To estimate the height of a mound at any particular distance from the centre of an infiltration basin:

$$H = W\left[\frac{K_2}{K_1}\left(\frac{q'}{K_2} - 1\right)\left(\frac{q'}{K_2} - \frac{x^2}{W^2}\right)\right]^{1/2}$$ Equation G-3

Where all parameters have been previously defined.

- To estimate the maximum height of the water table:

$$H_{\max} = W\left[\frac{q'}{K_1}\left(\frac{q'}{K_2} - 1\right)\right]^{1/2}$$ Equation G-4

Where all parameters are as defined previously.

G4.2.2 Estimating Groundwater Table Buildup

Two analytical solutions are provided to estimate the height and lateral extent of a ground water mound build up on an existing water table. These solutions are extracted from Carleton (2010) and Poetter et al. (2005).

Carleton (2010) conducted a study to implement the New Jersey Department of Environmental Protection (NJDEP) stormwater management rules. The Carleton study evaluated a number of variables, including hydrogeologic characteristics and infiltration structure design to better understand which factors most affect the magnitude and extent of mounding.

The study provides a quantitative method for estimating the height and lateral extent of a groundwater mound beneath an infiltration feature using the equations reproduced here on Figure G-2. By comparing the height of the mound on the water table with distance from the infiltration feature to existing topographic elevations, the potential for breakout on nearby slopes or interference with nearby utility corridors can be evaluated. This solution is also time dependent and consequently can be used to evaluate short-term infiltration events like the stormwater management events anticipated from the LID principles.

User required input values (aquifer thickness, horizontal saturated hydraulic conductivity, specific yield, basin size, recharge rate and duration, and distances away from the centre of the infiltration basin for which groundwater mound is required) are relatively easy to measure in the field using the methods described in Appendix A or can be estimated from the technical literature (Appendix H).

Carleton (2010) presents a spreadsheet developed by Dr. Arthur Baehr (U.S. Geological Survey) based upon the Hantush (1967) equations to calculate the magnitude and extent of groundwater mounds. The spreadsheet is easily obtained from the USGS website at http://pubs.usgs.gov/sir/2010/5102/. Appendix G-B contains a copy of that spreadsheet for ease of reference.

Poetter et al. (2005) provide a second means for estimating the height and lateral extent of a water table mound based upon the Hantush solution. The equation used by this solution to estimate the height of a water table mound is illustrated below.

$$z_{\max} = \sqrt{h_i^2 + \frac{q' h_{avg} t}{2S_y}\left[4S^*\left(\frac{l}{\sqrt{\frac{4K_h h_{avg} t}{S_y}}}, \frac{w}{\sqrt{\frac{4K_h h_{avg} t}{Sy}}}\right)\right]} - h_i$$

Equation G-5

Where:

- Z_{max} is (h_{avg}-h_i);
- q' is the design infiltration rate per unit area of the infiltration feature – m/s assumed to be less than K_{sat};
- h_i is the initial saturated thickness (m);
- h_{avg} is the iterated heads at a location away from the centre of infiltration and time of interest; 0.5(h_i(o)+h(t));
- K_h is the horizontal saturated hydraulic conductivity (m/s) or K_{sat};
- l is ½ overall infiltration area length (m);
- w is ½ the overall infiltration area width (m); and
- t is the time since infiltration began (s).

$$S^* = \int_0^1 erf\left(\frac{\alpha}{\sqrt{\tau}}\right) erf\left(\frac{\beta}{\sqrt{\tau}}\right)$$

Equation G-5.1

$$\alpha = \frac{l+x}{D}; \beta = \frac{w+y}{D}$$

x = 0 for z_{max} and y= 0 for z_{max}

Equation G-5.2

$$D = \sqrt{\frac{4K_h h_{avg} t}{S_y}}$$

Equation G-5.3

To estimate the height of the water table buildup at any point (x,y) from the centre of the infiltration basin, the following equation (Equation G-6) is used by Poetter et al. (2005).

$$z(x,y) = \sqrt{h_i^2 + \frac{q' h_{avg} t}{2S_y}\left[S^*\left(\frac{l+w}{D}, \frac{w+y}{D}\right) + S^*\left(\frac{l+x}{D}, \frac{w-y}{D}\right) + S^*\left(\frac{l-x}{D}, \frac{w+y}{D}\right), S^*\left(\frac{l-x}{D}, \frac{w-y}{D}\right)\right]} - h_i$$

Equation G-6

Where all parameters were defined previously.

The solutions by Poetter et al. require an iterative solution due to the need to resolve the exponential integral S*. Poetter et al. provide a spreadsheet with their publication to enable calculation of the z (max) height for the water table mound at any distance away from the centre of the infiltration. A copy of the spreadsheet is appended (Appendix G-C) for ease of reference. The spreadsheet and the full publication by Poetter et al (2005) can be obtained at http://www.ndwrcorp.org/documents/WU-HT0245_ES.pdf.

G4.3 Multiple Infiltration Features

Two methods are suggested to evaluate the use of multiple infiltration basins to manage stormwater for LID purposes. The first is an extension of the application of the Hantush solutions (Hantush 1967) provided within Poetter et al. (2005). Poetter et al. (2005) provide a spreadsheet based upon evenly spaced and uniformly constructed infiltration basins within an area used to dispose of wastewater. The spreadsheet can be obtained from http://www.ndwrcorp.org/documents/WU-HT0245_ES.pdf. Appendix G-C is a copy of the spreadsheet developed by Poetter et al. (2005) for ease of reference.

The second method to apply to multiple infiltration basins is based upon a numerical solution of the groundwater flow equations. A wide variety of such software exists, one of the most popular being the more recent modularization of groundwater flow components by Harbaugh et al. (2000). That numerical solution based upon MODFLOW or Visual Modflow is widely used within Alberta.

Numerical solutions to evaluate groundwater mounding should be considered in the following circumstances:

- The geologic and hydrogeologic conditions are complicated by either multiple geologic materials across a development or a variety of opportunities to create perched water table conditions; or
- A combination of multiple infiltration features (bio-retention areas, bio-swales, stormwater retention basins, and infiltration galleries, for example) is used within a development to manage stormwater.

Appendix F – Framework to Develop a Conceptual Site Model should be applied to all sites where numerical analysis of the flow systems is considered. Considering the variety of numerical software packages potentially applicable to the LID principles of maintaining infiltration and maintaining runoff target volumes, the guidance developed by the American Society of Testing and Material should be used to select the modelling code, establish numerical boundaries, and confirming the reliability of the model's outcome. The following guidance should be applied to any numerical modelling within The City:

- D6170 Standard Guide for Selecting a Ground-Water Modelling Code.
- D5447 Standard Guide for Application of a Ground-Water Flow Model to a Site-Specific Problem.
- D5490 Standard Guide for Comparing Groundwater Flow Model Simulations to Site-Specific Information.
- D5609 Standard Guide for Defining Boundary Conditions in Ground-Water Flow Modelling.
- D5610 Standard Guide for Defining Initial Conditions in Ground-Water Flow Modelling.
- D5611 Standard Guide for Conducting a Sensitivity Analysis for a Ground-Water Flow Model Application.
- D5981 Standard Guide for Calibrating a Ground-Water Flow Model Application.

- D5718 Standard Guide for Documenting a Ground-Water Flow Model Application.

The above list is written in the order the guidance should be applied to a numerical model for LID purposes.

G4.4 Results Reporting and Presentation

Reporting on the results of a water table mounding evaluation should include:

- A summary of the geotechnical and hydrogeologic information used;
- A statement as to the mean values used for the hydraulic conductivity and the range in values;
- A cross-section illustrating the depths to the impermeable layer or the water table;
- Assigned values for the horizontal and vertical hydraulic conductivity;
- The loading rate – total volumetric flow and the flow per unit area (the design infiltration rate);
- Calculated values for the height of the mound and the extent of the mound and measures taken to avoid ponding on the ground surface or seepage along adjacent slopes;
- Clarification as to the potential for intersection with subsurface utilities and roadways; and
- Setback requirements to avoid interferences.

REFERENCES

Carleton, G. B. 2010. Simulation of Groundwater Mounding Beneath Hypothetical Stormwater Infiltration Basins. United States Geological Survey Scientific Investigative Report 2010-S002 Prepared in Co-operation with the New Jersey Department of Environmental Protest.

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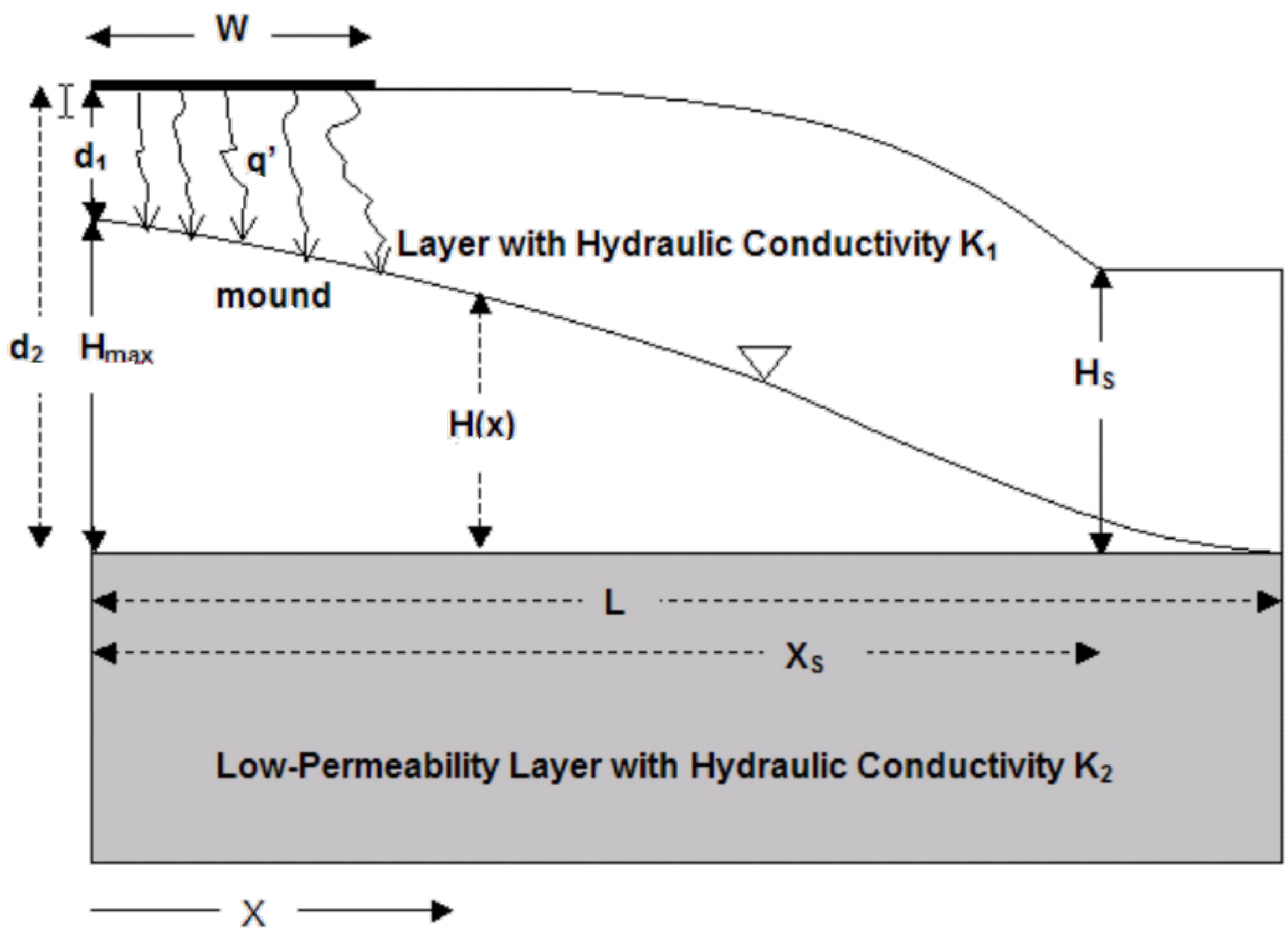
FIGURES



NOTE: d_2 - vertical distance from infiltration basin center to top of low-k layer.
d_1 - vertical distance to top of mound.

REFERENCE - (Poetter et al, 2005)

CLIENT	CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM				
THE CITY OF CALGARY	CONCEPTUAL MODEL FOR THE KHAN ANALYTICAL SOLUTION				
eba A TETRA TECH COMPANY	PROJECT NO. C12101310	DWN MMK	CKD NB	REV 0	Figure G-1
	OFFICE EBA-RIV	DATE September 2012			

$$h^2 - h_i^2 = (w/2k)(vt)\{S*(\frac{l+x}{\sqrt{4vt}}, \frac{a+y}{\sqrt{4vt}}) + S*(\frac{l+x}{\sqrt{4vt}}, \frac{a-y}{\sqrt{4vt}}) +$$

$$S*(\frac{l-x}{\sqrt{4vt}}, \frac{a+y}{\sqrt{4vt}}) + S*(\frac{l-x}{\sqrt{4vt}}, \frac{a-y}{\sqrt{4vt}})\}$$

$$\text{where } S*(\alpha,\beta) = \int_0^1 erf(\frac{\alpha}{\sqrt{\tau}})\, erf(\frac{\beta}{\sqrt{\tau}})\, d\tau \quad ,$$

where

h = head at a given time after recharge begins;
h_i = initial head (height of the water table above the base of the aquifer);
w = recharge (infiltration) rate;
K = horizontal hydraulic conductivity;
v = diffusivity, where v = Kb/Sy;
$\underline{b}$ = average aquifer thickness;
S_y = specific yield;
t = time elapsed since recharge began;
l = half-length of the recharge basin;
a = half-width of the recharge basin;
x = distance from the center of the recharge basin in the x direction;
y = distance from the center of the recharge basin in the y direction;

$$\alpha = \frac{l+x}{\sqrt{4vt}} \text{ or } \frac{l-x}{\sqrt{4vt}};$$

$$\beta = \frac{a+y}{\sqrt{4vt}} \text{ or } \frac{a-y}{\sqrt{4vt}};$$

τ = dummy variable of integration; and
erf = error function.

The integral in the above equation cannot be solved explicitly and is solved using iterative numerical methods.

REFERENCE -(Carleton 2010)

CLIENT: THE CITY OF CALGARY	CALGARY'S LOW IMPACT DEVELOPMENT PROGRAM	
	HANTUSH EQUATIONS AS PROVIDED IN CARLETON 2010	

eba A TETRA TECH COMPANY

PROJECT NO.	DWN	CKD	REV	
C12101310	MMK	NB	0	Figure G-2
OFFICE: EBA-RIV	DATE: September 2012			

APPENDIX G-A

WORKED EXAMPLE OF THE CALCULATION OF KHAN ET AL., 1978

Appendix G-A
Worked Example of the Calculation of Khan et al, 1976

based upon the Khan et al (1976) analytical solution for groundwater mounding on a low impermeability surface

where the parameters to be calculated are — assigned values

H_{max}	is the maximum height of the mound above an low permeability/ impermeable base (m)	calculated
W	is the width of the infiltration feature basin (m)	
L (m)	is the lateral distance from the centre of the basin	

where the input parameters are:

$q^{'}$	is the design infiltration rate (m/day)[1]	3.6E-08
K_2	is the hydraulic conductivity of the impermeable layer (m/sec)	2.00E-07
K_1	is the hydraulic conductivity of the unsaturated materials above the impermeable layer (m/sec)	1.00E-08

1 the design infiltration rate is 3.11 mm/day

Example Calculations

$$H_{\max} = W\left[\frac{q}{K_1}\left(\frac{q}{K_2}-1\right)\right]^{1/2}$$

Maximum height of mound

width of the infiltration feature	5	7.5	10	12.5	15	20	25	40	55	70	95
Hmax (m)	3	5	7	9	10	14	17	27	38	48	65

Length of mound from centre of infiltration basin

$$L = W\frac{q}{K_2}$$

width of infiltration feature	5	7.5	10	12.5	15	20	25	40	55	70	95
exent - distance from the centre of the basin (m)	18	27	36	45	54	72	90	144	198	252	342

Change in mound height with distance from the infiltration basin

$$H = W\left[\frac{K_2}{K_1}\left(\frac{q'}{K_2}-1\right)\left(\frac{q'}{K_2}-\frac{x^2}{W^2}\right)\right]^{1/2}$$

distance from centre (x)	mound height (width of 10 m) (m)
0	6.84
1	6.83
2	6.80
4	6.69
6	6.49
8	6.20
10	5.81
12	5.30
14	4.62
16	3.68
18	2.16
18.5	1.52
18.8	1.04

calculated values (yellow)

parameters to be input (green)

APPENDIX G-B

CARLETON, 2010, EXAMPLE SOLUTION SINGLE INFILTRATION FEATURE

This spreadsheet will calculate the height of a groundwater mound beneath a stormwater infiltration basin. More information can be found in the U.S. Geological Survey Scientific Investigations Report 2010-5102 "Simulation of groundwater mounding beneath hypothetical stormwater infiltration basins".

The user must specify infiltration rate (R), specific yield (Sy), horizontal hydraulic conductivity (Kh), basin dimensions (x, y), duration of infiltration period (t), and the initial thickness of the saturated zone (hi(0), height of the water table if the bottom of the aquifer is the datum). For a square basin the half width equals the half length (x = y). For a rectangular basin, if the user wants the water-table changes perpendicular to the long side, specify x as the short dimension and y as the long dimension. Conversely, if the user wants the values perpendicular to the short side, specify y as the short dimension, x as the long dimension. All distances are from the center of the basin. Users can change the distances from the center of the basin at which water-table aquifer thickness are calculated.
Cells highlighted in yellow are values that can be changed by the user. Cells highlighted in red are output values based on user-specified inputs. The user MUST click the blue "Re-Calculate Now" button each time ANY of the user-specified inputs are changed otherwise necessary iterations to converge on the correct solution will not be done and values shown will be incorrect. Use consistent units for all input values (for example, feet and days)

use consistent units (e.g. feet & days **or** inches & hours)

Input Values

Value	Symbol	Description
0.1000	*R*	**Recharge (infiltration) rate (feet/day)**
0.080	Sy	**Specific yield, Sy (dimensionless, between 0 and 1)**
4.00	K	**Horizontal hydraulic conductivity, Kh (feet/day)***
40.000	x	**1/2 length of basin (x direction, in feet)**
35.000	y	**1/2 width of basin (y direction, in feet)**
2.000	t	**duration of infiltration period (days)**
10.000	hi(0)	**initial thickness of saturated zone (feet)**

Conversion Table

inch/hour	feet/day
0.67	1.33
2.00	4.00

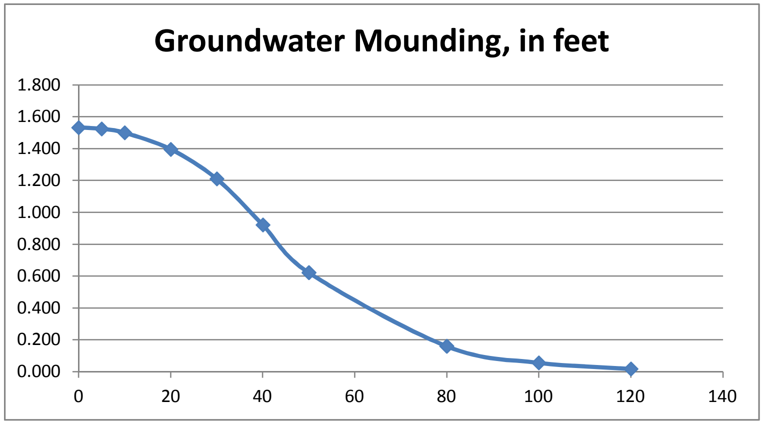
hours	days
36	1.50

In the report accompanying this spreadsheet (USGS SIR 2010-5102), vertical soil permeability (ft/d) is assumed to be one-tenth horizontal hydraulic conductivity (ft/d).

11.532	h(max)	**maximum thickness of saturated zone (beneath center of basin at end of infiltration period)**
1.532	Δh(max)	**maximum groundwater mounding (beneath center of basin at end of infiltration period)**

Ground-water Mounding, in feet	Distance from center of basin in x direction, in feet
1.532	0
1.524	5
1.499	10
1.395	20
1.210	30
0.921	40
0.621	50
0.159	80
0.055	100
0.017	120

Re-Calculate Now



Disclaimer

APPENDIX G-C

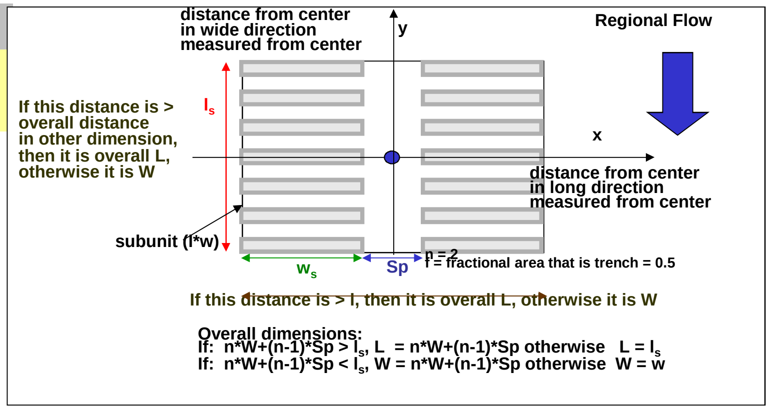
POETTER ET AL. EXAMPLE OF THE MOUNDING SOLUTION FOR WATER TABLE – MOUNDING OF MULTIPLE INFILTRATION FEATURES

Appendix G-C: Poetter et al. Example of the Mounding Solution for Water Table - Mounding of Multiple Infiltration Features

Water Table Mounding Calculated Based on Hantush 1967, WRR

Enter data in green cells as per their yellow labels, other values will be computed from those entries.

Results are highlighted in pink.



Zmax Beneath Center of Entire Drain Field (L*W)

Meters and Days	Length of Drain Field Subunit	Width of Drain Field Subunit		Separation between Drain Field Subunits	Fraction of Drain Field Subunit that is Trench Area	Horizontal Hydraulic Conductivity	Specific Yield use 0.001 to approximate steady state at 10 years	time use 10 years to approximate steady state
	l_s	w_s		Sp	f_A	K_h	Sy	time
	m	m		m		m/day	none	days
	20	15		2	0.5	10	0.001	3650
Number of subunits, n	L	W	q effective in subunit ls x ws	q in trenches	q' effective on LxW	Q	Zmax 12 iterations	Initial Saturated Thickness
	m	m	m/day	m/day	m/day	liters/day	m	m
2	35	22	0.0083	0.0130	0.0065	5000	0.119	5
2	35	22	0.0083	0.0130	0.0065	5000	0.063	10
2	35	22	0.0083	0.0130	0.0065	5000	0.043	15
2	35	22	0.0083	0.0130	0.0065	5000	0.033	20
2	35	22	0.0083	0.0130	0.0065	5000	0.027	25

copy an entire row from above and insert copied cells above this line
to evaluate various loading rates and numbers of subunits

NOTE: if a2+b2>0.04, solution is inaccurate

alpha	beta	a2+b2	W part1	W(a2+b2)	S*	z1	hiter	alpha
0.000647704	0.000407128	5.85274E-07	13.773971	13.77397136	5.11906E-06	0.120	5.059945541	0.000643856
0.000457996	0.000287883	2.92637E-07	14.467118	14.46711824	2.67589E-06	0.063	10.03161116	0.000457274
0.000373952	0.000235056	1.95091E-07	14.872583	14.87258325	1.82931E-06	0.043	15.02164725	0.000373683
0.000323852	0.000203564	1.46318E-07	15.160265	15.16026528	1.39613E-06	0.033	20.01653137	0.000323718
0.000289662	0.000182073	1.17055E-07	15.383409	15.3834088	1.13189E-06	0.027	25.01340641	0.000289584

Water Table Rise on Side Slope

Uses Subunit Geometry and Material Properties from Zmax Table

Number of subunits, n	L	W	q effective in subunit ls x ws	q in trenches	q' effective on LxW	Q l/day	Zsx 12 iterations	Distance from Center of Drain Field in Long Dimension (x in figure)	Distance from Center of Drain Field in Wide Dimension (y in figure)	Initial Saturated Thickness
	m	m	m/day	m/day	m/day	liters/day	m	m	m	m
2	32	20	0.0083	0.0156	0.0078	5000	0.117	10	0	5
2	32	20	0.0083	0.0156	0.0078	5000	0.058	20	0	10
2	32	20	0.0083	0.0156	0.0078	5000	0.038	30	0	15
2	32	20	0.0083	0.0156	0.0078	5000	0.028	40	0	20
2	32	20	0.0083	0.0156	0.0078	5000	0.022	50	0	25

copy an entire row from above and insert copied cells above this line
to evaluate various loading rates and numbers of subunits at various distances x,y from the center of the drain field

NOTE: if a2+b2>0.04, solution is inaccur

alpha1	alpha2	beta1	beta2	a2+b2	W part1	W(a2+b2)
0.000962303	0.0002221	0.000370117	0.000370117	1.06E-06	13.17718863	13.17718863
0.000942163	-0.0001047	0.000261712	0.000261712	9.56E-07	13.28312195	13.28312195
0.00098296	-0.0002992	0.000213687	0.000213687	1.01E-06	13.22649434	13.22649434
0.001036326	-0.0004441	0.000185058	0.000185058	1.11E-06	13.13554228	13.13554228
0.00109244	-0.0005628	0.000165521	0.000165521	1.22E-06	13.03877244	13.03877244

APPENDIX H

INDIRECT METHODS FOR ESTIMATING HYDRAULIC CONDUCTIVITY AND INFILTRATION RATES
CITY OF CALGARY – MODULE I GEOTECHNICAL AND HYDROGEOLOGIC CONSIDERATIONS FOR LOW IMPACT DEVELOPMENTS

TABLE OF CONTENTS

TABLES

FIGURES

H1.0 INTRODUCTION

This appendix provides an overview of methods to indirectly estimate the saturated hydraulic conductivity of soil. Indirect measurement refers to methods that do not involve in-situ testing (direct measurement) and instead rely upon an understanding of the soil type and the grain size distribution of available samples of the soil to make estimates for the saturated hydraulic conductivity.

The methods described here are useful at the early planning stages of a Low Impact Development (LID) project when an inventory of environmental conditions is being prepared such as during the Watershed Plan (WP) or Master Drainage Plan (MDP) planning levels. At this early planning stage, an indirect estimate of the hydraulic conductivity may be needed:

- To estimate groundwater flow rates for preliminary water balance calculations on local watersheds, wetlands, or other surface waterbodies; and
- To assess the feasibility of using stormwater management features that rely upon infiltration and percolation processes to replace the infiltration lost from non-pervious surface materials or to help meet surface runoff target volumes.

For all other planning stages, direct in situ measurements of the saturated hydraulic conductivity (Appendices A and B) are preferred.

In this appendix, references to hydraulic conductivity refer to the saturated hydraulic conductivity. For soil above the water table or a perched water table, the soil hydraulic conductivity is a measure of the maximum infiltration rate through the soil. Section H2.2.3 within this appendix and Section 3.2 of the main text discuss how the saturated hydraulic conductivity may be converted to an infiltration rate and how infiltration rates, through soils that are less than 100% saturated, can be accommodated in design calculations.

H2.0 INDIRECT METHODS FOR ESTIMATING THE HYDRAULIC CONDUCTIVITY

Sections H2.1 to H2.3, respectively, describe three indirect means for estimating the saturated hydraulic conductivity. These methods include use of values obtained from:

- The published technical literature;
- The soil texture; and
- The grain size distribution.

The published literature value returned here do not compare well to the values calculated by the USDA in section H2.2. This disagreement in part arises from the material tested in a geologic setting as opposed to a disturbed setting say for agricultured soils. The developer using the data shall therefore endeavor to confirm the stats (disturbed or undisturbed) of the material for which an estimated infiltration rate is required.

H2.1 Published Literature Values

Practically all text books used to instruct hydrogeology at the university and college level graduate or undergraduate programs (e.g., Domenico and Schwartz 1990, Fetter 1992, and Freeze and Cherry 1979), within their introductory chapters, publish a range in saturated hydraulic conductivity that might be expected in typical soil types. As an example, Table H-1 summarizes one such tabulation taken from Domenico and Schwartz, 1990. However; most hydrogeologic text books will contain a similar tabulation but with different ranges reflecting the authors' personal experience. Values quoted from such literature sources should also reference the source.

Table H-1: Representative Values of Hydraulic Conductivity

Material	Range in Hydraulic Conductivity Value (m/s)
Gravel	3×10^{-4} to 3×10^{-2}
Coarse Sand	9×10^{-7} to 6×10^{-3}
Medium Sand	9×10^{-7} to 5×10^{-4}
Fine Sand	2×10^{-7} to 2×10^{-4}
Silt, Loess	1×10^{-9} to 2×10^{-5}
Till	1×10^{-12} to 2×10^{-6}
Clay	1×10^{-11} to 5×10^{-9}

(After Domenico and Schwartz, 1990 page 65)

As this tabulation illustrates, the saturated hydraulic conductivity for the typical soil underlying an LID can have a broad range. In particular, the range in saturated hydraulic conductivity for till, the most common surficial material in the Calgary urban area, cover six orders of magnitude. This range in values might, at a glance, suggest that using literature values as the basis for estimating the saturated hydraulic conductivity and use in water balance estimates or assessing the feasibility of infiltration in the design of source control measures does not make sense. However; many of the professionals in the Calgary urban area have local knowledge that can drastically reduce this range in values, such that for preliminary water budget purposes, or for assessing the feasibility of applying infiltration to manage stormwater, a useful estimate can be provided.

When using published values or typical values from a local professional, the following guidelines should be followed:

- The value selected or range selected should be identified and the rationale for selecting the value or range should be provided; and
- The consequences of the soil having a value outside of the applied value or range should be stated.

H2.2 Hydraulic Conductivity as a Function of Soil Texture

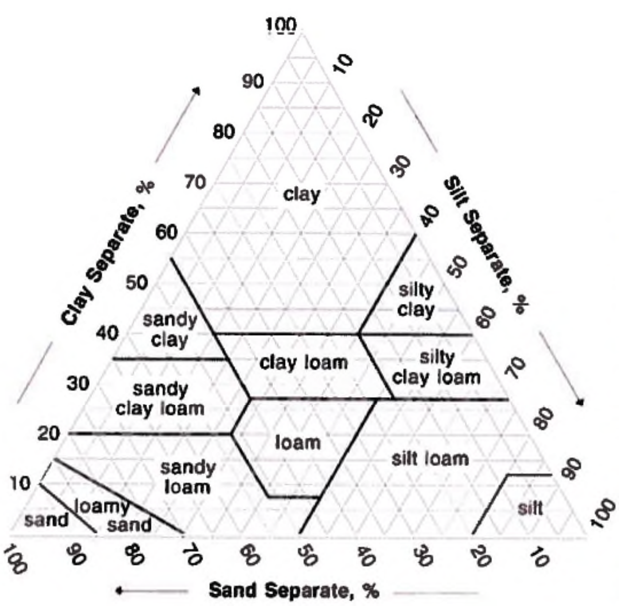
Sections H2.2.1 to H2.2.3 describe two methods for estimating the hydraulic conductivity for a soil and a method for estimating the unsaturated hydraulic conductivity by understanding the soil texture (relative percentages of sand, silt, and clay).

H2.2.1 United States Department of Agriculture Soil Textural Analysis for Estimating Hydraulic Conductivity

The United States Department of Agriculture (USDA) Soil Texture Triangle, shown on Figure H-1, is a tool used for the classification of soils based on textural properties. The only difference between the USDA and the Canadian soil textural system is that the Canadian system relies only upon the sand and clay percentages to classify a soil. Each of the 12 soil classes demonstrate varying physical properties based on texture, including infiltration rate. This is the typical rate at which water passes through the soil profile during saturated conditions. The minimum infiltration rate of various soil textural classes is tabulated in Table H-2. These values directly correspond to the saturated hydraulic conductivity of the soil texture, as published by the USDA. It is unclear why these authors refer to these values as the minimum rate of infiltration because the values tabulated do not directly convert to the corresponding saturated hydraulic conductivity and therefore these data should be used cautiously. This version of the soil textural triangle is available from the internet at http://www.pedosphere.ca/resources/texture/triangle-us.cfm?228,227.

By clicking upon the textural class, the software provides a value for all of the properties listed on the figure.

Figure H-1: USDA Soil Texture Triangle



Percent Sand	17.63
Percent Clay	12.77
Percent Silt	69.61
Texture (U.S. System)	Silt loam
Bulk density *g/cm³*	1.43
Saturated hydraulic conductivity *cm/hr*	2.4
Saturation *cm³ water/ cm³ soil*	0.46
Field capacity *cm³ water/ cm³ soil*	0.29
Wilting point *cm³ water/ cm³ soil*	0.1
Plant Available water *cm³ water/ cm³ soil*	0.19
inches water/foot soil	2.24

Table H-2: Minimum Infiltration Rate of USDA Soil Classes

USDA Soil Class	Minimum Infiltration Rate (Mm/hr.)	Saturated Hydraulic Conductivity (m/sec)
Clay	0.508	$1.67e^{-7}$
Sandy Clay	1.270	$3.33e^{-7}$
Silty Clay	1.016	$2.50e^{-7}$
Clay Loam	2.286	$6.39e^{-7}$
Silty Clay Loam	1.524	$4.17e^{-7}$
Sandy Clay Loam	4.25	$1.19e^{-6}$
Loam	13.208	$3.61e^{-6}$
Silt Loam	1.458	$1.89e^{-6}$
Sandy Loam	25.908	$7.22e^{-6}$
Loamy Sand	61.214	$1.69e^{-5}$
Sand	210.058	$5.83e^{-5}$

Source: US EPA, 2004.

This means of obtaining a hydraulic conductivity value is published within The City of Calgary Water Balance Spread Sheet (WBSS) – WBSS (Westhoff 2011).

H2.2.2 Use of SPAW

Saxton and Rawls (Saxton and Rawls 2006) developed a method based upon the soil texture and organic matter to estimate the hydraulic conductivity of a soil. That method contained within the Soil, Plant Air, and Water (SPAW) software is also provided within The City of Calgary's WBSS model software. An excerpt provided from WBSS in Figure H-2 shows the interface used by SPAW to provide the saturated hydraulic conductivity value. Typical values for the hydraulic conductivity produced by SPAW are tabulated in Table H-3.

Figure H-2: SPAW Interface

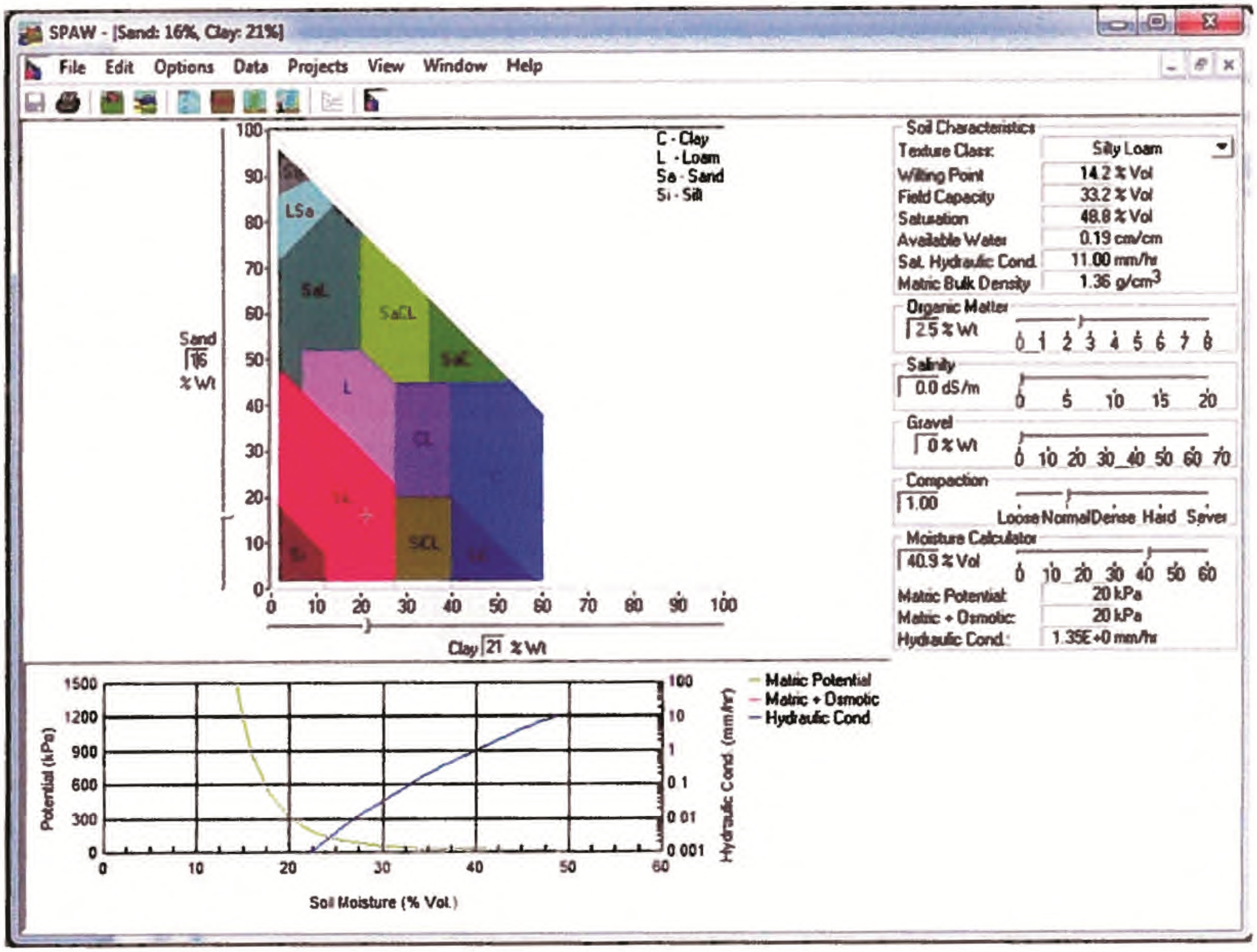


Table H-3: Hydraulic Conductivity Estimated for Selected Soil Texture Using SPAW

Texture Class	Sand (%wt)	Clay (%wt)	Hydraulic Conductivity (m/s)[1]
Sa	88	5	3.0×10^{-5}
LSa	80	5	$2.7.\times 10^{-5}$
SaL	65	10	1.4×10^{-5}
L	40	20	4.3×10^{-6}
SiL	20	15	4.5×10^{-6}
Si	10	5	6.1×10^{-6}
SaCL	60	25	3.1×10^{-6}
CL	30	35	1.2×10^{-6}
SiCL	10	35	1.6×10^{-6}
SiC	10	45	1.0×10^{-6}
SaC	50	40	3.9×10^{-7}
C	25	50	3.1×10^{-7}

Notes:

[1]Sa: sand, L: loam, Si: silt, and C: clay –after WBSS, Westhoff 2011.

H2.2.3 Estimation of the Partially Saturated Hydraulic Conductivity

Many of the source control features used to manage stormwater for LID properties may dry out from time to time. The rate of infiltration, therefore; will be governed by the moisture content of the soil. The saturated hydraulic conductivity can be used with the water content at saturation and the water content at the time of measurement to predict a plausible hydraulic conductivity/infiltration rate using the Brooks-Corey and van Genuchten model:

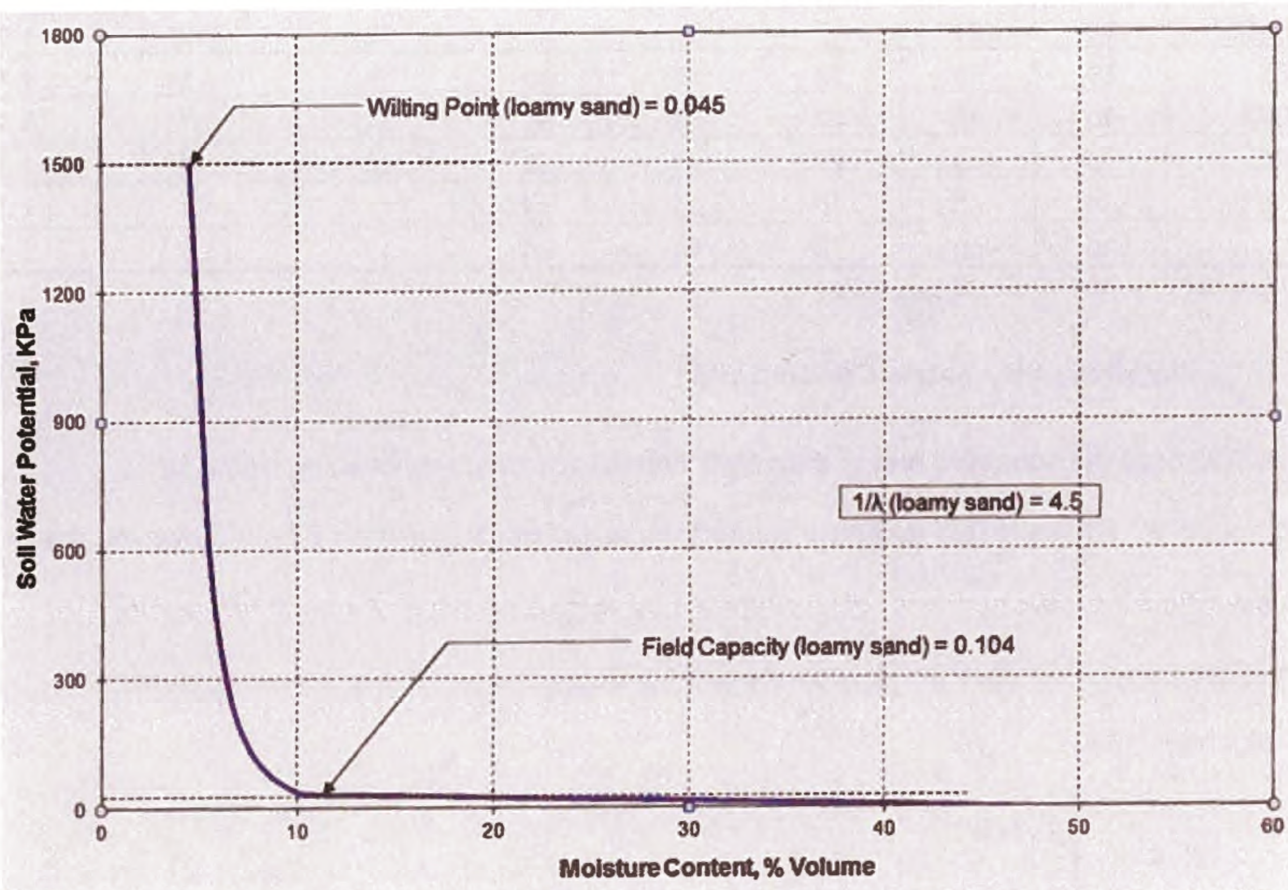
$$K = K_s \left(\frac{\theta}{\theta_s} \right)^{\left(3+2/\lambda\right)} \qquad \text{Equation H-1}$$

Where:

- K is the hydraulic conductivity for the soil at that water content (m/s);
- K_s is the hydraulic conductivity at saturation (m/s);
- θ_s is the water content at saturation;
- θ is the water content at which hydraulic conductivity is measured; and
- $2/\lambda$ is the inverse slope of the logarithmic tension moisture curve.

The inverse slope of the logarithmic tension moisture curve is available for many soil types in the literature. An example of a moisture curve as provided in the WBSS (Westhoff 2011) is provided on Figure H-3.

Figure H-3: Computation Example of Inverse Slope Logarithmic Tension Moisture Curve



The inverse slope of the moisture curve is calculated at set points on the curve of the wilting point at 1,500 kpa and field capacity 33 kpa. For the example shown:

$$\frac{1}{\lambda}=\frac{\ln 1500-\ln 33}{\ln\theta_{33}-\ln\theta_{1500}}=\frac{\ln 1500-\ln 33}{\ln 0.104-\ln 0.045}=4.5 \qquad \text{Equation H-2}$$

H2.3 Hydraulic Conductivity Predicted from Grain Size Analyses

Grain size analyses are a commonly used tool to predict the saturated hydraulic conductivity of a soil. A variety of formulas have been developed for the estimation of hydraulic conductivity using grain size distributions. Most are based upon laboratory controlled experiments. Relevant soil properties and controlling factors of hydraulic conductivity are the sorting of the soil grains, the pore size distribution, grain and pore shape, tortuosity, specific surface, and porosity (Dietrich and Vienken 2011).

The following summary presents the most commonly used formulae as published by Freeze and Cherry, (Freeze and Cherry 1979), and Sevee (Sevee 1991).

Hazen Equation

The saturated hydraulic conductivity for a soil can be estimated using Hazen's formula, which is based upon the effective particle size of a soil as follows:

$$K = C d_{10}^{2} \, x \frac{1m}{100cm} \qquad \text{Equation H-3}$$

Where:

- K = saturated hydraulic conductivity in m/sec;
- d10 = particle size (in mm), below which 10% by weight of the cumulative sample has finer particles, and 90% by weight has coarser particles; and
- C = Dimensionless factor which combines with a shape and porosity factor (for K in cm/sec and d10 in mm, C is equal to 1.0).

This formula was originally used to predict the saturated hydraulic conductivity of uniformly graded sands. Uniformly graded soils have grain size curves that are oriented vertically rather than spread across a range of particle sizes. As Freeze and Cherry suggest, however, "it can also offer a rough estimate of hydraulic conductivity for most soils in the fine sand to gravel range." For application to SCP, it can be used effectively at the design stage to predict the composition of sand most useful to achieve the infiltration rate needed by the source control practice.

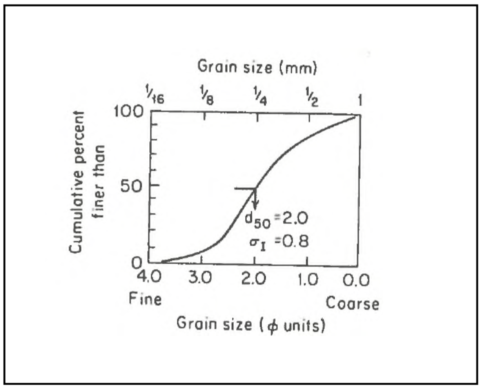
However; the textural determination of hydraulic conductivity can be more accurately measured if the spread or degree of sorting of the soil is considered. To consider the spread of the soil particles over a range in grain sizes, Masch and Denny (1966) recommended plotting the grain size curve and using Krumbein's φ units, where φ = -log2(d). Using this type of grain size distribution curve, the representative grains size, d50 can be found, as well as the measure of spread, using the inclusive standard deviation, σI as follows:

$$\sigma\ I = \frac{d_{16} - d_{84}}{4} + \frac{d_{5} - d_{95}}{6.6} \qquad \text{Equation H-4}$$

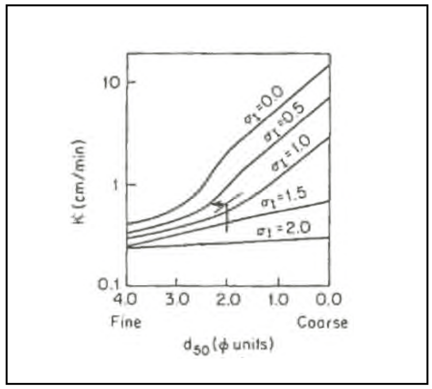
Where:

- dx = particle size (in mm), below which x% by weight of the cumulative sample has finer particles, and (100-x)% by weight has coarser particles.

- Figure H-4: Representative Grain Size Figure H5: Estimate of Hydraulic Conductivity



Source: Freeze and Cherry (1979)



Source: Freeze and Cherry (1979)

For example, Figure H-4 shows d_{50} = 2.0. Applying Equation H-4 to the values obtained from the curve of Figure H-4, σI = 0.8. D_{50} and σ_I can then be fit to the appropriate curve in Figure H-5. These curves, developed experimentally in a laboratory using prepared samples of unconsolidated sand, are used to predict the saturated hydraulic conductivity value. For example, the values observed on Figure H-4 show a saturated hydraulic conductivity value of approximately 0.7 cm/min (or 1.17 x 10^{-4} m/sec).

Kozeny-Carmen Equation

If the porous medium is a non-uniform soil, than it is necessary to use some representative grain size, d_m and the coefficient C would become dependent on the shape and packing of the soil grains. The resulting prediction of hydraulic conductivity is found using the Kozeny-Carmen equation (Freeze and Cherry 1979, after Bear 1972):

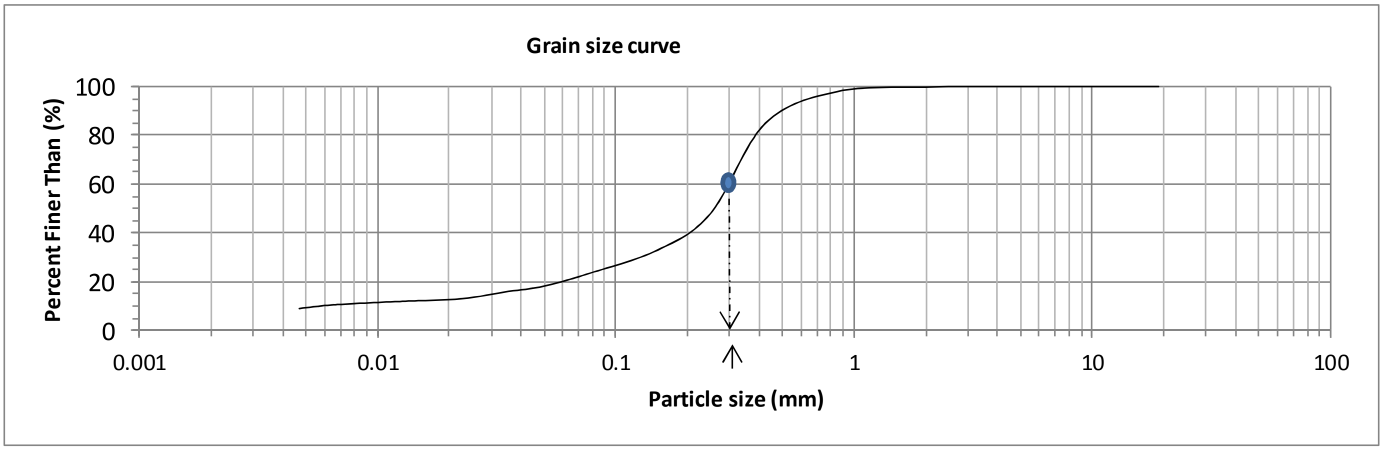
$$K = \left(\frac{\rho g}{\mu}\right)\left[\frac{n^3}{(1-n)^2}\right]\left(\frac{d_m^2}{180}\right) \qquad \text{Equation H-5}$$

Where:

- n is the porosity;
- dm is the mean particle diameter (in mm);
- µ is the viscosity of water (0.001308 kg/m sec at 10°C);
- □□is the density of water (1000 kg/m3); and
- g is the gravitational constant (9.81m sec^{-2}).

This example demonstrates how to estimate the saturated hydraulic conductivity of a silty sand aquifer using Equation H-5 – the Kozeny-Carmen equation. The grain size distribution is shown in Figure H-6 and a groundwater temperature of 10°C is assumed.

Figure H-6: Grain Size Curve for the Kozeny Carmen Equation Example



Where:

- n = 0.35;
- g = 9.81 msec-2;
- ρ = 1000 kgm-3 (density of water);
- μ = 0.001308 kgm-1 sec-1 (kinematic viscosity of water at 10°C; and
- dm = 0.00026 m (from Figure H-6, approximately d_{48}).

$$K = \left(\frac{1000\,kg/m^3 \; x 9.81 m/\sec^2}{0.001308 kg/m\sec} \right) \left[\frac{0.35^3}{(1-0.35)^2} \right] \left(\frac{0.00026 m^2}{180} \right) \qquad \text{Equation H-6}$$

$$K = 2.86 X\, 10^{-4}\, ms^{-1}$$

The saturated hydraulic conductivity, according to the Kozeny-Carmen equation with the grain size distribution shown on Figure H-6 is estimated at 2.86 x 10^{-4} $msec^{-1}$.

Fair Hatch Equation

There are similar formulas of this type to predict the saturated hydraulic conductivity, and most treat the porosity term as identical to the central element of Equation H-6; however; the grain size term can take on many different forms. For example, the Fair-Hatch equation uses grain size data from the entire grain size curve, which is useful for sandy soils with minor amounts of clay and silt. This method assumes that the shapes of the grain size curve as well as the shape of the grains both play a role in determining saturated hydraulic conductivity. The equation is as follows:

$$K = \left(\frac{\rho g}{\mu}\right)\left[\frac{n^3}{(1-n)^2}\right]\left(\frac{1}{m\left(\frac{\theta}{100}\sum\frac{P}{d_m}\right)^2}\right) \qquad \text{Equation H-7}$$

Where:

- m is a packing factor found experimentally to be about 5;
- Ө is the sand shape factor varying from 6.0 for spherical grains to 7.7 for angular grains;
- P is the % of sand held between adjacent sieves;
- dm is the geometric mean of the rated sizes of adjacent sieves;
- n is the porosity;
- µ is the kinematic viscosity of water (0.001308 kg/m sec at 10°C);
- ρ is the density of water (1000 kg/m3); and
- g is the gravitational acceleration constant (9.81m sec-2).

A worked example of this equation is not provided.

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