Additional Materials for Inclusion with Water Movement and Soil Treatment Module:

1. Designing Wastewater Disposal Systems

2. Designing Large Septic Systems

and

3. Examples of Three-Step Hydrologic Analysis

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Note:

The materials included in this file consist of class notes and unpublished papers by Dr. Aziz Amoozegar of NC State University Soil Science Department. The materials are included here as an additional source of information for users of the CIDWT Practitioner Curriculum Module: Water Movement and Soil Treatment.
Section 1

DESIGNING WASTEWATER DISPOSAL SYSTEMS

Soil Assimilative Capacity in Waste Disposal -- Hydraulic Loading

The general hydrologic cycle for a liquid waste disposal system shown in Figure 1 is given by:


Figure 1. General hydrologic cycle including a wastewater disposal system.

Although soil water content changes with time, the net change is soil water content for
any waste disposal system over a one-year period (e.g., July 1 through June 30) can be assumed to be zero. Also, to increase the efficacy of any liquid land application we must eliminate both surface and subsurface lateral flow into the waste disposal area. This is true for both surface and subsurface disposal systems. Surface flow into the area (run-in) can be eliminated by creating surface drainage upslope from the waste disposal area. Subsurface lateral flow into the area can be intercepted by a subsurface drainage ditch or tile installed above the disposal area to catch the incoming lateral flow. This type of drainage is called interceptor drain. For surface disposal (e.g., spray irrigation systems) run-off from the site during and immediately after irrigation period must also be eliminated while for subsurface disposal systems (e.g., septic systems) run-off from natural rainfall is generally permitted. Lateral subsurface flow is permitted for both surface and subsurface disposal systems as long as it does not appear at the soil surface (i.e., it does not seep out) within the prescribed set backs around the waste disposal area. In general, spray irrigation systems depend on the evapotranspiration (referred to as ET) and drainage (mainly deep percolation) from the site, while subsurface systems rely on deep percolation and lateral subsurface flow from the site. Subsurface wastewater disposal systems are generally designed without considering precipitation and ET by assuming that precipitation matches ET at all times.

[NOTE: Evapotranspiration (ET) is defined as “the combined loss of water from a given area, and during a specified period of time, by evaporation from the soil surface and by transpiration from plants.” Glossary of Soil Science Terms, Soil Sci. Soc. Am., Madison, WI (can be found at http://www.soils.org/sssagloss/).

Potential Evapotranspiration (PET) is defined as the amount of evapotranspiration from a unit area of land completely covered by short grass (i.e., green crop) of uniform height and never short of water (i.e., well irrigated).]

Designing Surface Irrigation Systems

For designing surface irrigation systems we ignore the change in soil water content, and do not allow any run-off into or from the spray irrigation area, especially during and immediately after irrigation periods. We also assume lateral subsurface flow into the spray irrigation area equals the lateral subsurface outflow from the area. Based on these assumptions we rewrite the hydrologic cycle for irrigation system as

Precipitation + Liquid Wastewater Application = Evapotranspiration (ET) + Drainage.

To obtain the amount of liquid waste that can be applied to a site, a water balance must be performed over the entire year. For example, the amount of wastewater that can be applied monthly can be determined by estimating precipitation, evapotranspiration, and drainage from January 1 through December 31.

Precipitation and Evapotranspiration: Information on precipitation for a given site can be obtained from the nearest weather station. In general, instead of using average precipitation, a
five- or ten-year return frequency (i.e., the highest amount of precipitation that can be expected in a 5- or 10-year period) is used. Use the monthly average values for precipitation if no 5- or 10-year frequency rainfall data can be located. Evapotranspiration for the crop under consideration can be estimated from the potential evapotranspiration (PET) and the respective crop factor or coefficient. In general, weather stations do not measure ET or PET, but they collect data on temperature, relative humidity, daylight hours, wind speed and other weather data that can be used to estimate PET for the desired area. There are equations and models for estimating PET (ASCE REFERENCE). In addition, PET data for a given area may be obtained from literature. For example, average PET for North Carolina can be obtained from “Weather and Climate in North Carolina”, NC State University Experiment Station Bulletin 396. Information about North Carolina weather can be obtained from http://www.nc-climate.ncsu.edu. Crop factor for various crops under different conditions can be obtained from the literature. Use PET if no crop factor can be found or if there are mixed vegetation at the irrigation site.

Drainage: This parameter cannot be determined accurately for any given site. In general, the amount of potential drainage is estimated based on the saturated hydraulic conductivity ($K_{sat}$) of the least permeable layer in the upper part of the soil, and/or using the lateral flow from the site (without any ground water mounding reaching within a certain distance from the soil surface). In some cases, such as the presence of a permanent shallow ground water or when an impermeable layer is near the soil surface, drainage is assumed to be zero.

There is no hard rule as to how to estimate drainage from a site. The EPA (1981) suggested to limit the amount of drainage to 4 to 10% of the soil permeability of the least permeable layer for rapid infiltration (infiltration gallery), but the recommendation for spray irrigation systems is not clear. [NOTE: The term permeability is used in the EPA (1981) to represent saturated hydraulic conductivity ($K_{sat}$).] For spray irrigation the amount of drainage should never exceed the $K_{sat}$ value of the least permeable layer. To assure proper functioning of the system, it is best to limit the amount of drainage to a value less than the minimum measured $K_{sat}$ value of the least permeable layer at a given site. If the soil hydraulic conductivity is estimated or obtained from literature, or if the average $K_{sat}$ value of a number of measurements is used, multiply the $K_{sat}$ value by an appropriate factor selected based on the soil properties at the site (e.g., a factor of 0.04 to 0.1 may be used if $K_{sat}$ is estimated).

Water Balance Calculations

A water balance table is created by using the ET for the crop (or crops) under consideration, the amount of drainage from the site, and monthly amount of precipitation. Estimate the ET for the crop(s) under consideration from the climatic data, and use an appropriate estimated value for drainage from the site. Use PET for the area if no specific data can be obtained. Add the ET (or PET) and drainage as the total outflow from the system. Subtract the amount of monthly precipitation from the monthly total outflow, and enter the data as the permissible amount of irrigation. Enter zero if the result is negative (i.e., more precipitation than PET and drainage).
As an example, the water balance for a 12-month period for an area near Raleigh, North Carolina is presented in Table 1. Instead of ET the average monthly PET in North Carolina is used. Drainage is the estimated amount of potential vertical percolation through the least permeable layer (calculated based on hydraulic conductivity measurements and number of days in each month), and precipitation is the highest amount of 1-in-10 year value for each month.

Table 1. Example of a monthly water balance.

<table>
<thead>
<tr>
<th>Month</th>
<th>PET</th>
<th>Estimated Drainage</th>
<th>†Total Outflow</th>
<th>Precipitation</th>
<th>Allowable‡ Irrigation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Jan</td>
<td>23</td>
<td>106</td>
<td>129</td>
<td>84</td>
<td>126</td>
</tr>
<tr>
<td>Feb.</td>
<td>36</td>
<td>96</td>
<td>132</td>
<td>89</td>
<td>128</td>
</tr>
<tr>
<td>March</td>
<td>56</td>
<td>106</td>
<td>162</td>
<td>94</td>
<td>158</td>
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<tr>
<td>April</td>
<td>84</td>
<td>103</td>
<td>187</td>
<td>97</td>
<td>183</td>
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<tr>
<td>May</td>
<td>109</td>
<td>106</td>
<td>216</td>
<td>97</td>
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<td>June</td>
<td>122</td>
<td>103</td>
<td>225</td>
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<tr>
<td>July</td>
<td>119</td>
<td>106</td>
<td>226</td>
<td>150</td>
<td>220</td>
</tr>
<tr>
<td>Aug.</td>
<td>102</td>
<td>106</td>
<td>208</td>
<td>137</td>
<td>202</td>
</tr>
<tr>
<td>Sept.</td>
<td>84</td>
<td>103</td>
<td>187</td>
<td>117</td>
<td>182</td>
</tr>
<tr>
<td>Oct.</td>
<td>48</td>
<td>106</td>
<td>155</td>
<td>71</td>
<td>152</td>
</tr>
<tr>
<td>Nov.</td>
<td>30</td>
<td>103</td>
<td>133</td>
<td>76</td>
<td>130</td>
</tr>
<tr>
<td>Dec.</td>
<td>18</td>
<td>106</td>
<td>124</td>
<td>81</td>
<td>121</td>
</tr>
</tbody>
</table>

† Total Outflow = PET + Estimated Drainage
‡ Allowable Irrigation = Total Outflow - Precipitation

Based on the above calculations, the site in this example can be irrigated 12 months out of the year. However, we need to realize that, depending on the type of plant (e.g., evergreen trees vs. grass), and climatic conditions (e.g., snow on the ground), we may want to irrigate only during certain parts of the year (e.g., from March through November). The minimum amount of land required for irrigation is calculated by dividing the yearly volume of wastewater that must be disposed by the total amount of irrigation. Also, the volume of wastewater produced during the times with no irrigation (e.g., December through February) must be stored on site or managed otherwise (e.g., disposal through a public sewer system).

**Section 2**

DESIGNING LARGE SEPTIC SYSTEMS
Similar to other waste management systems, the design of a large or small septic system is an iterative and multi-stage process. The designer of the system must know the design flow rate and should follow the state and local regulations. In addition, the designer must be familiar with various systems that are permitted under the respective regulations, and be aware of the installation, operation, and maintenance requirements/costs for any system that will be selected. The design flow rate for the system (i.e., the volume of wastewater that must be disposed of daily) is based on the number of people using the dwelling or the facility under consideration. For example, in North Carolina, for a residential dwelling the daily flow rate is the number of bedrooms × 120 gal (assuming 2 persons/bedroom, and 60 gal/d water use by each person), while for a country club the daily flow rate is the number of members × 20 gal and for a school with cafeteria is the number of students × 12 gal.

Impact of Septic Systems on Soil/Site Hydrology

As discussed in Section 1, the natural input into the soil system at a given site is generally composed of precipitation plus the surface and subsurface flow into the area. The natural output from the soil system at the area is composed of evapotranspiration, run-off, subsurface lateral flow above a slowly permeable layer in the upper part of the soil profile, and deep percolation into ground water. In general, net change in soil water content over a long period (e.g., 1 year) is negligible and should not be considered in any annual hydrologic cycle analysis.

Application of wastewater to the soil through a septic system may increase the daily input of water into the soil system substantially. As for the output, only the subsurface lateral flow above a slowly permeable layer in the upper part of the soil profile and deep percolation into ground water can be impacted. In general, installation of a septic system should not increase run-off from the site, and the added wastewater to the soil may not increase evapotranspiration significantly. This can have a significant impact on the local hydrology at the site. For example, for a large septic system serving 20 4-bedroom homes with a design daily flow of 9,600 gal/d (480 gal/d per home) and a loading rate of 0.2 gal/(ft² d), based on North Carolina regulations, the amount of wastewater applied to the site annually is equivalent to 115 in or 292 cm of rainfall over more than one acre of drainfield. For a location similar to Raleigh, NC, this is approximately 2.5 times the mean annual precipitation over the area. For a 4-bedroom single family home (design flow 480 gal/d) located on 40,000 ft² (≈ one acre), and a similar design loading rate of 0.2 gal/(ft² d), on the other hand, the amount of wastewater added to the entire lot is equivalent to 7 in (17.5 cm) of rainfall/year. This represents approximately 15% of the mean annual precipitation for Raleigh, NC. As evident from these examples, installation of a large system at a site can have a significant impact on the local hydrology over and immediately around the drainfield area.
Figure 2 depicts the hydrologic cycle for a septic system. To reduce the amount of water input into the drainfield area, surface and subsurface flow of water into the drainfield area must be minimized. To minimize surface water flow into the area (run-in), water from rooftops (gutters) and impervious surface areas (e.g., parking area, paved patio, driveway) must be diverted from the drainfield area. If potential for subsurface lateral flow exists, interceptor drains must be used upslope to capture the incoming subsurface water before entering the drainfield.

General Steps in Designing a Large Septic System:

1. Determine site suitability
   a. Collect information about the site (e.g., soil map, topo map, planning map)
   b. Visit the site
   c. Conduct a preliminary site assessment (e.g., type of soil, slope, restrictions on the site, presence of wetlands and riparian zones)
   d. Conduct a detailed evaluation to determine drainfield location
2. Determine the type of system to be used and the loading rate for the drainfield
3. Delineate the location of the drainfield on the topo map of the site

4. Conduct a hydrologic evaluation of the site to determine if the loading rate is appropriate

5. Repeat the process by making appropriate adjustments if the hydrologic evaluation shows the system cannot function as proposed, or declare the site unsuitable for the type of system under consideration

Soil/Site Characterization

Although the location of the drainfield for the septic system serving a dwelling or another facility (e.g., club house, school) is often determined after selecting the location of the respective physical structures on the property, the drainfield area must meet all the necessary standards set by the respective state and/or local regulations. In general, a preliminary evaluation of a property is conducted to determine the site suitability and to delineate the potential areas for installing a septic system. The preliminary evaluation includes assessment of slope, landscape position, soil depth, texture, wetness (e.g., seasonal high water table, redoximorphic features), property lines, buried utilities, and other pertinent properties as required by the respective regulations. A more comprehensive soil/site evaluation is then needed to determine the type of septic system (e.g., low-pressure pipe distribution, sand filter pretreatment), the design loading rate, the depth and spacing between trenches, and the location of the drainfield within the suitable areas. This comprehensive evaluation may include, but is not limited to, assessing soil profile using hand-auger borings and/or observation pits, measuring or estimating depth to bedrock or other restrictive layers (e.g., a water table), and characterizing drainage outlets.

For small septic systems serving single family homes (or other facilities with a design loading rate below a threshold value set by the prevailing regulations) the detailed evaluation of the site is often limited to assessing various soil properties at a few locations by hand auger boring technique with or without requiring soil profile description using observation pit(s). In general, a septic system will be designed based on this comprehensive evaluation, and the locations of the trenches will be marked on the contour lines over the topographic map of the property. For large septic systems, on the other hand, a hydrologic analysis of the drainfield area is needed to confirm the design-loading rate and to assess the proper functioning of the system after installation and usage. The purpose of the hydrologic analysis in North Carolina (and other places) is to determine if the soil under consideration is capable of assimilating the design daily flow of wastewater and if the minimum requirement for the presence of the unsaturated soil below the bottom of the trenches can be met following the full use of the system based on the specified daily flow rate. For example, currently in North Carolina, a minimum of 2 ft (60 cm) of suitable and unsaturated soil must be present below the bottom of the trenches.

A hydrologic analysis for a septic system must be conducted in conjunction with a detailed soil and site evaluation. It should be emphasized that, before conducting a hydrologic
analysis, the site suitability must be determined and a loading rate for applying wastewater to the drainfield must be selected through a detailed soil/site evaluation for the type of the septic system (e.g., gravity fed, low-pressure pipe, drip system) under consideration. Furthermore, a preliminary design must be drawn for the system and the locations of the trenches on the contour lines within each subfield (or the drainfield if the drainfield is not divided into zones) must be identified on the topographic map of the site. The topographic map should show the potential drainage outlets.

Type of Wastewater Dispersal Systems

In a gravity fed septic system wastewater always enters the beginning part of each trench. If the applied wastewater cannot infiltrate the soil within the first few meters of the length of the trench, it will flow by gravity along the trench until the surface area of infiltration matches the ratio of the volume of applied wastewater and the average infiltration rate of wastewater into the soil from the bottom and side walls of the trench. As the time progresses, due to the ponding of wastewater at the beginning of the trench, a biological mat (also called clogging mat or biological clogging mat) will be created at the interface of gravel and soil in the trench (see Siegrist, 1986). With time, this biological mat may eventually cover the entire length of the trench and result in a substantial reduction in the infiltration capacity of the trenches. Surfacing of wastewater over the drainfield and/or backing of sewage into the dwelling occurs when the trenches of a septic system fill up and the volume of wastewater applied daily to the trenches can no longer infiltrate the soil within each 24-h period.

Unlike gravity fed systems, wastewater can be distributed relatively uniformly over the drainfield by LPP and drip systems. In addition, LPP systems are designed such that all the wastewater applied to the trenches must infiltrate the soil within a relatively short period of time (referred to as dosing cycle) to allow adequate time for the aeration of the bottom area of the trenches between wastewater application events (referred to as resting cycle). Surfacing of effluent over the drainfield area occurs if the rate of wastewater application to the trenches exceeds the rate of infiltration of wastewater into the soil. Because of better utilization of the entire drainfield LPP or drip systems may be preferable to gravity or pressure manifold (Berkowitz, 1985) distribution systems.

Three-Step Hydrological Analysis of Septic Systems

Although wastewater application to the drainfield of gravity fed septic systems is different than pumped ones, the overall soil hydrology at the site is the same for both types of wastewater application. In order for a septic system to function properly all the wastewater applied to the drainfield of the system must infiltrate the soil and move away from the drainfield area without surfacing. This process takes place in three stages:

1. All the wastewater applied to the trenches daily must infiltrate the soil within a 24-hour time period. Otherwise, the trenches of the system will eventually fill up and wastewater will surface over the drainfield or sewage will back up to the dwelling.
2. Wastewater entering the soil through the sides and bottom of the trenches must move vertically through the least permeable layer in the unsaturated zone near the bottom of the trenches before reaching an impermeable or slowly permeable layer (e.g., bedrock, Cr horizon) or water table. If the areal application rate of wastewater to the drainfield is greater than the rate of vertical water flow through the least permeable layer below the trenches, the soil immediately under the trenches become saturated resulting in hydraulic and/or treatment failure of the system.

3. Wastewater reaching the impermeable/slowly permeable layer or ground water under the drainfield must move laterally away from the system. Ground water mounding (i.e., localized rise in the water table immediately below the drainfield) occurs when the amount of wastewater applied to the drainfield exceeds the total lateral flow from the area. As the mounding increases, the volume of water moving laterally increases due to an increase in hydraulic gradient and/or cross sectional area of the flow path. Treatment of wastewater by soil may cease if the mounding reaches within a certain distance (e.g., 60 cm) below the bottom of the trenches, or the system may fail hydraulically if the mounding reaches the trenches.

Available Models: Currently, there is no universally accepted protocol for conducting a hydrologic assessment for predicting the behavior of a septic system and its impact on soil hydrology or water table elevation. Water flow from septic systems is complex. As a result, development of a dedicated model to describe infiltration of wastewater from the trenches into the soil and movement of water through and away from the drainfield of the system may require derivation of complex mathematical equations and/or use of numerical techniques for solving a multidimensional water flow problem. In addition, any such model will require site specific field data with a high degree of reliability to account for spatial and temporal variability of various soil properties. The cost associated with the development and use of a sophisticated model, as well as the requirement of high intensity field data may make the approach impractical and/or uneconomical for routinely assessing sites and designing large septic systems. One available model that can be used for assessing water flow from septic system trenches is HYDRUS-2 (Simunek et al., 1999). This model, however, is based on theoretically characterized soil properties and requires high expertise to run. In addition, simulating wastewater flow from the trenches through saturated and unsaturated zones around the trenches of a septic system may take a long time (e.g., few days). The models that are available for analyzing ground water mounding under the drainfield area include Hantush (1965) and MODFLOW (for information see http://water.usgs.gov/nrp/gwsoftware/modflow2000/modflow2000.html). The Hantush (1967) model has been transformed into a user-friendly program (Molden et al., 1984) and is available through the Civil Engineering Department at Colorado State University. The MODFLOW model, is also available in a relatively user-friendly format called Visual MODFLOW available through Waterloo Hydrogeologic, Inc. (Waterloo, Ontario, Canada) and other private vendors. In lieu of sophisticated models, Amoozegar and Martin (1997) offered a brief description of a relatively simplified method for designing large septic systems in areas with and without shallow water table (similar to the Coastal Plain and Piedmont regions of North Carolina, respectively). Although their methodology depends on the hydrologic setting at a given site, it
consists of a comprehensive soil/site evaluation and three general steps that can be applied to assess water movement from septic systems in any region. Also, they offered this methodology for designing low pressure pipe distribution systems where all the applied wastewater infiltrates the soil within each dosing cycle. With modifications and proper assumptions, their method can be applied to gravity fed conventional or pressure manifold (Berkowitz, 1985) systems where wastewater enters the beginning of the trenches and moves along each trench by gravity.

Data Requirement: Observation pits must be dug at a few strategically selected locations for describing the soil profile and determining the properties (e.g., thickness, color) of various horizons, including the least permeable layer. If necessary, soil samples may be collected from various horizons and submitted to a laboratory for determining soil properties, such as particle size distribution and cation exchange capacity (CEC). Soil morphological properties must also be assessed by hand-auger boring technique at a number of places throughout the proposed drainfield area to determine the site suitability, the type of wastewater dispersal system, and the proper loading rate for wastewater application. The soil evaluator(s) can use the profile observed in each observation pit for calibrating his/her soil profile description by hand auger boring technique. To do this, the evaluator describes the soil profile in an auger hole dug adjacent to the wall of the pit where profile has been described, and compares this profile description to that described on the pit wall. The locations and number of auger borings required depend on the landscape, the complexity of the soils in the area, and the expertise of the soil/site evaluator. One way is to lay a 50 to 100 ft (or 15 to 30 m) grid system over the landscape and evaluate the soil profile at each grid point. In addition, deep borings by hand auger or a mechanical auger (e.g., hydraulic probe, drill rig) must be conducted at a few locations to determine depth to water table or an impermeable (or a very slowly permeable) layer.

To conduct a hydrologic analysis saturated hydraulic conductivity ($K_{sat}$) of the soil must be determined at a minimum of three depths at a few locations within the drainfield area. The depths of interest are: (1) at a depth corresponding to the depth of the proposed trenches (or drip lines for drip systems), (2) within the least permeable layer below the bottom of the trenches, and (3) within the unsaturated (vadose) zone below the least permeable layer in the absence of any water table, and/or in the saturated zone if a water table already exists at the site. Measurements must be conducted at a number of locations (e.g., at a minimum of five locations for each acre of land) within the proposed drainfield area. To cover the entire drainfield, it is best to measure $K_{sat}$ at least in the middle and at four sides or corners of the proposed drainfield area. Depending on the soil and site properties, measurements at other depths and locations may be required.

There are various field and laboratory procedures for measuring saturated hydraulic conductivity of both the saturated (i.e., below a water table), and the unsaturated (i.e., vadose zone or above the water table). A number of these techniques are described in various publications including Amoozegar and Warrick (1986), Amoozegar and Wilson (1999), and Reynolds et al. (2002). For field method in the unsaturated zone the constant-head well permeameter technique (also known as shallow well pump-in technique, bore hole permeameter technique) is most suitable for many areas. Measurements can be conducted from the soil
surface to 4 m depth using the Compact Constant Head Permeameter (Amoozegar, 1988; Amoozegar, 2004). For the saturated zone, auger hole method (Amoozegar and Wilson, 1999; Amoozegar, 2002) and slug test (Bouwer, 1989) can be used for measuring $K_{sat}$ in the upper part of the saturated zone. Other techniques are also available for measuring transmisivity ($T = K_{sat} \times$ thickness of aquifer) of an unconfined aquifer (Freeze and Cherry, 1987) that may be present in the drainfield area. The soil/site evaluator must choose the proper technique for measuring $K_{sat}$ at the depth of interest.

Saturated hydraulic conductivity is a highly variable soil parameter with coefficient of variation ($CV = 100 \times$ standard deviation/mean) that may exceed 100% (Warrick and Nielsen, 1981). In addition, there is no universally accepted method of selecting the proper $K_{sat}$ value for designing septic systems. One recommendation is to use a prescribed percentage value of the average measured $K_{sat}$ values. As indicated earlier, EPA (1981) recommended 4 to 10% of the permeability values to be used for rapid infiltration systems. Depending on the statistical distribution of the measured $K_{sat}$ values, it may be better to use the geometric rather than the arithmetic mean of the measured values for this approach. Another option, suggested by Amoozegar and Martin (1997), is to use the lowest measured $K_{sat}$ at each depth within a site. For the later approach, the individual conducting the hydrologic analysis may also consider a safety factor (e.g., 75% of the measured value) when using this minimum value approach.

Step 1. Infiltration from Trenches

Infiltration is defined as the entry of water into soil (Glossary of Soil Science Terms, see http://www.soils.org/sssagloss/). Water flow from a trench ponded with water is considered two-dimensional. Water applied to a trench (as in a septic system) can enter the soil through the bottom and side-walls of the trench. After ponding of water (or wastewater) in a trench dug in a homogenous and isotropic soil a zone of saturation (similar to the one formed under one-dimensional flow as described by Hillel, 2004), followed by a transmission zone and wetting front will be developed. If ponding continues, the saturated zone in a homogeneous soil reaches a limit and the wetting front may disappear (Fig. 3). Most soils, however, are heterogeneous and anisotropic. Therefore, wastewater infiltration and water flow from the trenches of a septic system is not symmetrical, and may occur through macropores and special features around the trenches. Figure 4 presents photographs of soil profiles around two trenches that had received a tracer dye solution. Figure 4A shows the relatively uniform movement of water from a trench in a coarse textured soil above a clayey Bt layer, and Fig. 4B shows movement of dye solution through macropores associated with tubular root channels and animal borings, and planar voids between ped faces when the trenches are placed within the Bt horizon of a well structured soil.
As mentioned earlier, to avoid wastewater ponding in the trenches, all the wastewater applied to the trenches in a drainfield within any 24-hour period must infiltrate the soil in 24 hours. The rate of wastewater infiltration from trenches into the soil is equal to the total amount of wastewater applied to the trenches in 24 hours divided by the calculated average wetted surface area in the trenches. The rate of wastewater infiltration can also be estimated by Darcy’s law for a unit hydraulic gradient using the average wetted area in the trenches and the saturated hydraulic conductivity of the soil at the level of the bottom of the trenches. For this analysis, the volume of wastewater applied to a unit length of trench and the average surface area of infiltration need to be calculated.

The amount of wastewater applied to a unit length of a trench (e.g., ft, 1 m), hereafter referred to as \( q \), is determined by dividing the design flow \( Q \) by the total length of the trenches in the system \( L \). The surface area of infiltration for a dose of wastewater applied to a low-pressure pipe trench decreases as wastewater infiltrates the soil. To calculate the maximum wetted area for such a system, first the amount of wastewater applied to a unit length of trench of width \( w \) with no gravel or other types of aggregate (e.g., tire chips, EZ Flow systems) is expressed as the depth of wastewater in the trench \( d \). This parameter is obtained by dividing the volume of wastewater applied to a unit length of trench by the surface area of the bottom of the trench.
Figure 4. Photographs showing the dye-stained areas around one trench of a small experimental low-pressure pipe dispersal system installed above the Bt horizon at one site in Clayton, NC (A); and one trench of another system installed within the Bt horizon of a soil in Raleigh, NC (B). Note the preferential movement of the tracer dye through the macropores for system installed in the Bt horizon and the stained areas in the coarse textured materials and lack of stained areas in the underlying clayey Bt horizon for the system installed above the Bt horizon.

the unit length of trench [i.e., \( d = q/(w \times 1) \)]. [Alternatively, the amount of wastewater applied to
a trench expressed as the depth of wastewater is calculated by dividing the design flow by the total area of the bottom of the trenches for the entire system, \(d = \frac{Q}{w \times L}\). For trenches containing any type of aggregate (e.g., gravel, tire chips), the depth of water in the trench immediately after dosing \((D)\) is obtained by dividing the above value by the porosity of the aggregate (Fig. 5). For gravel aggregate the porosity can be assumed to be 33 to 38%, and for tire chips the reported porosity is approximately 60% (Amoozegar and Robarge, 1999). The average surface area of infiltration for the entire length of trenches is the sum of the bottom surface area of the trench \((w \times L)\) and one half the maximum wetted surface area on the two sidewalls of the trench \((2D \times L/2)\) (see Fig. 6).

To determine if all the wastewater applied to the trenches can infiltrate the soil before the next dose of wastewater application to the trenches, \(K_{sat}\) of the soil at the level of the bottom of the trenches must exceed the calculated average infiltration rate of wastewater from the trenches into the soil for the design loading rate. For this purpose the individual conducting the analysis must select an appropriate \(K_{sat}\) value (e.g., 20% of the average \(K_{sat}\) values, the minimum measured \(K_{sat}\) value) for the analysis.

Step 2. Vertical Percolation of Wastewater through the Least Permeable Layer Below Trenches

All the wastewater infiltrating the soil from the trenches must pass through the least permeable layer below the bottom of the trenches before reaching an impermeable layer or the water table below the trenches. For our analysis, we assume that wastewater infiltrates the soil from the trenches symmetrically and reaches the least permeable layer uniformly. That is, wastewater gets distributed uniformly through the soil in the drainfield after leaving the trenches (Fig. 7). For this, the rate of vertical flow at some distance below the trenches is the same as the daily areal loading rate for the system. The areal loading is calculated by dividing the design daily flow volume, \(Q\), by the total area of the drainfield.

The percolation of water through the least permeable layer in the unsaturated zone below the bottom of the trenches is similar to vertical infiltration of water into a homogeneous soil. For infiltration into a homogeneous soil under a small ponded level of water the final infiltration rate (also referred to as final infiltration capacity) is generally set equal to the saturated hydraulic conductivity \((K_{sat})\) of that soil (see Hillel, 2004). For a very deep profile, as long as the rate of water application to the soil surface is less than the final infiltration rate, no ponding will occur. Based on this, we can assume that no ponding above the least permeable layer occurs as long as the areal application rate is less than \(K_{sat}\) of the least permeable layer. Therefore, we compare the design areal loading rate with the measured \(K_{sat}\) of the least permeable layer. As indicated earlier, the individual conducting the analysis can select a prescribed percentage of the average of the measured \(K_{sat}\) values, or choose the smallest measured \(K_{sat}\) value for comparison with the design areal application rate.
Figure 5. Schematic diagram of the cross sectional area of an empty trench (i.e., no soil or gravel in the trench) and a trench filled with gravel showing the height of the wastewater in the trench after a dose of wastewater application.

Figure 6. Schematic diagram of the cross sectional area of a trench showing the initial (maximum) and final (minimum) infiltrative surface areas for wastewater infiltration. The average of these two surface areas for the entire system is \((w + D) \times L\).
Figure 7. Schematic diagram of the cross sectional area of three trenches showing the theoretical symmetrical flow from parallel trenches and uniform flow through the least permeable layer below the trenches.

Step 3. Lateral Movement of Water from the Drainfield Area

In this step lateral movement of water from the drainfield area is investigated. The type of analysis to be performed depends on the presence or absence of a permanent water table. Presence of a permanent water table indicates little to no net loss of water from the volume of the aquifer below the drainfield. That is, in the saturated zone (aquifer) under the drainfield, either water moves very slowly due to low hydraulic gradient and/or saturated hydraulic conductivity, or ground water enters and leaves the area at about the same rate. In order for the additional water entering the aquifer from the septic system to leave the drainfield area the level of water under the drainfield must rise (i.e., ground water mounding must occur) to increase the hydraulic gradient (see Figure 8A).

In the absence of a permanent water table, a different type of analysis can be performed to determine the potential for the formation of a saturated zone below the drainfield. Note that,
Figure 8. Schematic diagram of the cross sectional area of a drainfield showing the ground water mounding in the saturated zone below the trenches (A), and formation of a saturated zone above an impermeable or slowly permeable layer in the absence of a permanent water table (B).

in general, a soil considered suitable for septic system will allow natural rainfall to percolate
through the upper part (e.g., 5 ft or 1.5 m) of the profile. The absence of a permanent saturated zone at some distance below the soil surface is an indication of the absence of an impermeable layer in that zone. In some cases, reductomorphic features may be present to show seasonal saturation and the presence of a slowly permeable layer below the drainfield. If possible, the thickness of the seasonally saturated zone above the slowly permeable layer should be determined from these features. In the absence of an aquifer below the drainfield, the wastewater from the drainfield percolates through the unsaturated zone until reaching a slowly permeable layer. In order for water to move away, a saturated zone is then formed above the impermeable or slowly permeable layer (see Fig 8B). The thickness and extent of this saturated zone depend on the slope of the impermeable or slowly permeable layer, the closeness of the natural drainage affecting the newly created saturated zone, hydraulic conductivity of the slowly permeable layer as well as hydraulic conductivity of the layer(s) immediately above slowly permeable layer.

Whether there is a permanent water table or not, water moving laterally in the unsaturated zone under the drainfield must move horizontally toward a drainage outlet or spread around and dissipate into the ground water. In this publication we refer to the cross sectional area of the flow in the saturated zone as the “window of flow.” One example of the cross sectional area and a corresponding areal view for the lateral movement of water from the drainfield of a septic system is shown in Fig. 9.

Using Darcy’s law, the amount of flow through the window of flow of width $W$ can be calculated by knowing the saturated hydraulic conductivity of the saturated zone, thickness of aquifer at the window of flow ($m$), the difference in the total hydraulic head for ground water (elevation) at the first trench and at the window of flow ($\Delta H$), and the length of flow ($L$). The cross sectional area of the flow, that is the area of window of flow is $A = m \times W$. The hydraulic gradient for the flow is $\Delta H/L$. If saturated hydraulic conductivity of the aquifer is $K_{sat}$, the total flow from under the drainfield is:

$$Q = A \times K_{sat} \times \Delta H/L$$  \[1\]

Conversely, we can determine $\Delta H$ for the design flow rate for the system under consideration. Solving for $\Delta H$ we get:

$$\Delta H = [(Q \times L)/(A \times K_{sat})]$$  \[2\]

Based on the elevation of the original water table we can determine the distance between the bottom of the first trench and the ground water mound under the system.
Figure 9. Schematic diagram of the cross sectional area (A) and plan view (B) of a hypothetical septic system.

In the above analysis it is assumed that ground water from under the drainfield moves only in one direction through a window that has a width equal to the width of the drainfield. In reality, however, water under a mound can move in all directions. Therefore, the window of flow will be larger than what is used in the above analysis. Because of the difficulties associated
with presenting lateral water flow analyses for all scenarios; it is advised that the individual conducting the analysis take into account the potential size of the window of flow through which the volume of wastewater applied to the drainfield and entering the water table can move laterally away from the system. In the absence of a drainage ditch (artificial or natural), for example, ground water under the drainfield can move in all directions. The model presented by Molden et al. (1986) and available through the Colorado State University for determining the rise in the water table under a rectangular or circular basin assumes a level impermeable layer and original water table when water moves in all directions. Other simplified models also make similar assumptions. MODFLOW model, on the other hand, can simulate other conditions, but the level of expertise needed to run the model is much higher than what is required for simplified analysis.

For cases where no water table is present, the conductivity of slowly permeable layer may be high enough to allow vertical movement of some or all of the applied wastewater to the drainfield. In some cases, a saturated zone may form above a slowly permeable layer directly under the dispersal area of the system. As the saturated zone grows, its base extends beyond the boundary of the dispersal area, resulting in a higher infiltrative surface for vertical percolation to a deep aquifer. Therefore, it is important to consider vertical movement of water through the slowly permeable layer when considering lateral flow of water above this layer.

System Configuration: Location, orientation, and dimensions of a drainfield on landscape have pronounced impact on the potential of ground water mounding under the system. Stacking up of trenches at sites with a slope greater than a few degrees could result in excessive wetness along the trenches at the downslope side of the drainfield due to potential lateral flow above the least permeable layer in the unsaturated zone. Where a natural (or artificial) drainage exists, every attempt should be made to make the drainfield as long as possible along the drainage system. Figure 10 presents schematic diagrams of the plan view of three equally sized drainfield along a drainage ditch. The natural flow of ground water for all cases is toward the drainage ditch. Because of the presence of drainage system on one side of the drainfield, all the applied wastewater is expected to move toward the drainage outlet with the highest level of the ground water mound at the upslope side of the drainfield. The cross sectional area for ground water flow (i.e., area of window of flow) for the system depicted in Fig. 10A is substantially smaller than the corresponding area for the system shown in Fig. 10C. In addition, because the allowable distance between the bottom of the trenches and the top of the ground water mound is the same for all systems, the hydraulic gradient for the potential mound is smaller for the system shown in Fig 10A than the other two systems. In general, the shorter the width of the drainfield, the higher is the gradient for the potential ground water mound under the system.
Figure 10. Schematic diagram of the plan view of three equally sized drainfields oriented differently along a drainage outlet.
Section 3

EXAMPLES OF THREE-STEP HYDROLOGIC ANALYSIS

The following two examples are adapted from Amoozegar and Martin (1997). One is for a septic system in the Coastal Plain region of North Carolina with relatively flat lands and seasonal high water table. The second is for the Piedmont region of North Carolina with sloping ground and deep water table. Here, the values for some of the soil and site properties reported by Amoozegar and Martin (1997) are changed, and both examples are written in a style similar to a supporting document that will be submitted for obtaining a permit for each system.

Example 1: A septic system with a daily design flow of 3185 gal (12,000 L) must be installed to serve an office complex in the lower Coastal Plain area of NC. The slope of the land at the site is <1%. After a preliminary assessment of the soil and the site a 150 by 400 ft (45 by 120 m) area was delineated as the potential drainfield and repair area for the system. A 5-ft (1.5-m) deep natural drainage ditch running in North-East direction is located 300 ft (90 m) from the edge of this area. Using a 50- by 50-ft (15- by 15-m) grid laid over the site, the soil at every grid point was described to a maximum depth of 5 ft (150 cm), and deep borings to approximately 25 ft (7.5 m) was performed at three locations. Based on these evaluations, the depth to seasonal high water table is approximately 3 ft (90 cm) and the lower boundary of the surficial aquifer is determined to be at 8 m below the land surface. Using a number of Compact Constant Head Permeameters (Amoozegar, 1992), $K_{sat}$ of the Bt horizon and the layers above the Bt horizon in the vadose zone (above the water table) was measured by the constant-head well permeameter method (Amoozegar and Wilson, 1999) at five and four locations, respectively. In addition, $K_{sat}$ of the saturated zone (below the water table) was determined at one location by pumping test and at three locations by the slug test or auger-hole method test (Amoozegar and Wilson, 1999). The results for these measurements are presented in Table 2.

Table 2. Saturated hydraulic conductivity for the vadose and saturated zones.

<table>
<thead>
<tr>
<th>LOCATION</th>
<th>NO. of MEASUREMENTS</th>
<th>MINIMUM</th>
<th>MEAN</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cm/d</td>
<td>gal/(ft$^2$ d)</td>
<td>cm/d</td>
</tr>
<tr>
<td>Vadose Zone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Horizons above Bt</td>
<td>4</td>
<td>25.0</td>
<td>6.16</td>
</tr>
<tr>
<td>Bt Horizon</td>
<td>5</td>
<td>3.9</td>
<td>0.96</td>
</tr>
<tr>
<td>Saturated Zone</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Pumping Test</td>
<td>1</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Auger-Hole Test/Slug Test</td>
<td>3</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
An LPP system with 10-in or 0.83 ft (25.4-cm) wide trenches and 5 ft (1.5 m) average spacing between them (measured from center to center) is selected for the system. The trenches will be installed above the Bt horizon. Based on the NC regulations, a long-term acceptance rate (LTAR) of 0.5 gal/(ft² d) [equivalent to approximately 20 L/(m² d) or 2 cm/d] is selected based on the type of soil found at the proposed drainfield area. According to NC regulations, the loading rate for an LPP system is based on the drainfield area. The total area required for the drainfield is, therefore, 3185/0.5 = 6,370 ft² (approximately 590 m²), and based on an average spacing between the trenches (5 ft), the total length of trenches is 6,370/5 = 1270 ft (≈390 m).

Step 1, Water Flow from Trenches: To determine the infiltration rate for the design flow we need to determine the volume of wastewater that will be applied to a unit length of trench. Assuming the average spacing between the trenches to be 5 ft (1.5 m), the volume of wastewater applied to a unit length of trench is 2.5 gal/ft (≈ 30 L/m), and the rate of wastewater applied to the trench on the trench bottom basis is 0.5 (gal/ft²/d) × 60 (in) spacing/10 (in) width of trench = 3 gal/(ft² d) [equal to 122 L/(m² d)]. Based on the above calculation, the application rate based on the trench bottom area is equivalent to 0.4 ft/d (12.2 cm/d). If we assume that the entire volume of the daily design flow (3185 gal/d or 12,040 L/d) is applied instantaneously to the trenches in one dose, and that the porosity of gravel is 35%, the maximum height of ponding in the trenches immediately after wastewater application is 0.4/0.35 = 1.14 ft (12.2/0.35 = 34.8 cm). The average surface of infiltration for one-ft length of trench is then (0.83 + 1.14) × 1 = 1.97 ft² [(25.4 + 34.8) × 100 = 5,950 cm² for 100 cm length of trench], and the infiltration rate for the design flow is 1.27 gal/(ft² d) [30 (L/d) × 1000 (cm³/L) divided by 5,950 cm² = 5 cm/d]. The calculated infiltration rate for the trenches represents approximately 20% of the minimum and 10% of the geometric mean of the $K_{sat}$ values of this horizon (see Table 2) of the soil above the Bt horizon (where trenches are located). Therefore, we can assume that all the design flow volume that would be applied to the trenches (during dosing cycle for LPP) can infiltrate the soil in a timely manner and allow adequate time with no ponding (i.e., resting cycle for LPP) within each 24-hour period.

Step 2, Vertical Movement of Applied Wastewater to Ground Water: As indicated above, the design application rate based on the drainfield area is 0.5 gal/(ft² d) or approximately 2 cm/d. We assume that wastewater infiltrating the soil from the trenches reaches the Bt horizon below the bottom of the trenches uniformly, and compare the design application rate with the minimum and geometric mean of $K_{sat}$ values of this horizon. The minimum and geometric mean values of $K_{sat}$ for the Bt horizon (see Table 2) are 1.92 and 3.4 times greater than the design application rate, respectively. Therefore, we can assume that all the applied wastewater can move vertically through the unsaturated zone and reach the water table without causing saturation below the bottom of the trenches.

Step 3, Ground Water Mounding: The lateral movement of water in the saturated zone must be determined to assess the level of rise of water table (i.e., ground water mounding) under the drainfield of the system. Based on the depths of water table (3 ft) and the impermeable boundary
below the aquifer (25 ft), the thickness of aquifer is assumed to be 22 ft. Using an average saturated hydraulic conductivity of 18 ft/d (550 cm/d), and a drainage ditch located at 300 ft (91.5 m) from the edge of drainfield, the ground water mound under the drainfield (approximately 60 ft by 105 ft) was calculated by the Hantush model presented by Molden et al. (1984). The model predicted a maximum ground water mounding of approximately 0.47 ft (14 cm), which puts the top of the water table at 2.53 ft (approximately 30 in or 80 cm) below the land surface.

Another way to determine ground water mounding is to assume that all the applied wastewater must move toward the drainage ditch located at approximately 300 ft (91 m) from the drainfield edge. Similar to the above analysis, we assume the length of the drainfield (105 ft or 32 m) is parallel to the drainage ditch, the thickness of aquifer at the edge of the field is 22 ft (6.7 m), and the average \( K_{sat} \) for the saturated zone is 18 ft/d. The length of flow (i.e., the width of the drainfield) for calculating hydraulic gradient is 60 ft (18 m), and the cross sectional of the flow is 2,310 ft\(^2\) (214 m\(^2\)). Using Eq. [2], the rise in water table for a total flow volume of 425 ft\(^3\)/day (12,000 L/day) is then:

\[
\Delta H = \left[\frac{(425 \times 60)}{(2,310 \times 18)}\right] = 0.61 \text{ ft (18 cm)}
\]

From both calculations it is obvious that ground water mounding under the drainfield is relatively small. One way to reduce the level of ponding is by increasing the length of the drainfield parallel to the drainage ditch, which in turn will reduce the width of the drainfield (i.e., the length of flow path for calculating hydraulic gradient) and increase the cross sectional area of the flow.

For this example, in order to maintain 2 ft (60 cm) of unsaturated soil below the bottom of the trenches, an LPP system with shallow trenches (depth of trenches < 20 cm) shall be used, or fill materials must be added to the site to increase the distance between the land surface and water table.

Example 2: The septic system under consideration is for a year-round school in the Piedmont region of North Carolina. The design flow rate is 4760 gal/day (18,000 L/day). A preliminary site evaluation was conducted by a soil scientist to identify suitable areas for the drainfield and repair areas of the system. Based on this evaluation, we determined that the best method for the dispersal of the effluent at the site is a low pressure pipe (LPP) system. A comprehensive soil/site evaluation was then conducted within the suitable areas to determine soil and site properties for determining the loading rate and designing the system. Using three observation pits inside and two observation pits outside, six deep borings, and a number of hand auger borings the soil profile was described and depth to Cr was estimated throughout the drainfield area. Based on these evaluations, no restrictive layer or sign of saturation was observed within 5 ft (150 cm) depth (the maximum depth of suitable soil required by the North Carolina regulations). In addition, saturated hydraulic conductivity of each of the Bt, BC and C (saprolite) horizons was measured by the constant-head well permeameter technique (Amoozegar and...
Table 3. Arithmetic mean and range of saturated hydraulic conductivity (K$_{\text{sat}}$) values and depth intervals in which K$_{\text{sat}}$ was measured for various horizons for the system in the Piedmont region of North Carolina.

<table>
<thead>
<tr>
<th>HORIZON</th>
<th>DEPTH INTERVAL</th>
<th>cm</th>
<th>cm d$^{-1}$</th>
<th>(gal ft$^{-2}$ d$^{-1}$)</th>
<th>cm d$^{-1}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bt</td>
<td>48 to 64</td>
<td>10.9</td>
<td>2.7</td>
<td>2.8 to 21</td>
<td></td>
</tr>
<tr>
<td>BC$^\dagger$</td>
<td>117 to 220</td>
<td>2.1</td>
<td>0.5</td>
<td>0.25 to 6.1</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>230 to 307</td>
<td>12.0</td>
<td>3.0</td>
<td>1.3 to 30.2</td>
<td></td>
</tr>
</tbody>
</table>

$^\dagger$ Includes the upper and lower parts of the horizon

Table 4. General characteristics of the proposed system for Example 2.

<table>
<thead>
<tr>
<th>Characteristics</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Daily Flow</td>
<td>4,750 gal</td>
</tr>
<tr>
<td>Drainfield Dimensions</td>
<td></td>
</tr>
<tr>
<td>North Side</td>
<td>145 ft</td>
</tr>
<tr>
<td>South Side</td>
<td>145 ft</td>
</tr>
<tr>
<td>East Side</td>
<td>330 ft</td>
</tr>
<tr>
<td>West Side</td>
<td>330 ft</td>
</tr>
<tr>
<td>Total Drainfield Area</td>
<td>100,000 ft$^2$</td>
</tr>
<tr>
<td>Design Flow Rate Based on Drainfield Area</td>
<td>0.1 gal/(ft$^2$ d)</td>
</tr>
<tr>
<td>Design Flow Rate Based on Trench Bottom Area</td>
<td>0.75 gal/(ft$^2$ d)</td>
</tr>
<tr>
<td>Number of Subfields</td>
<td>7</td>
</tr>
<tr>
<td>Average Spacing Between Drain Lines</td>
<td>5 ft</td>
</tr>
<tr>
<td>Total Trench Length</td>
<td>9,500 ft</td>
</tr>
<tr>
<td>Disposal Area Based on the Loading Rate</td>
<td>47,500 ft$^2$</td>
</tr>
</tbody>
</table>
The soil in the drainfield area has a 6 to 8 in (15 to 20 cm) thick sandy loam surface layer over a 36 to 68 in (90 to 170 cm) deep clayey Bt horizon. The transitional BC horizon has a sandy clay loam to clay loam texture, and the C horizon (saprolite) has a silt loam texture. The boundaries between various soil horizons are wavy. Overall, the depth to Cr horizon within the drainfield area is greater than 108 in (270 cm). No ground water or saturated zone is present above the Cr horizon. A natural drainage ditch (with no water) is located at a distance of 100 ft on the west side of the drainfield.

Step 1, Water Flow from Trenches: First we need to determine if wastewater applied to the trenches can infiltrate the soil within each dosing cycle. As indicated earlier, according to North Carolina regulations, the design loading rate (i.e., the long-term acceptance rate or LTAR) for LPP systems is based on the drainfield area. Based on 5 ft (60 in or 150 cm) spacing between trenches, and 8-in width of trenches for the LPP system selected for this septic system, the volume of wastewater applied daily to a unit length of trench (1 ft) is 0.5 gal/ft (equivalent to 6 L/m), and the loading rate based on trench bottom basis is 0.75 gal/(ft² d) [30 L/(m² d), which is
equivalent to 3 cm/d, or 0.1 ft/d]. [NOTE: The above loading rate is calculated by multiplying the design application rate, 0.1 gal/(ft$^2$ d), by the spacing between trenches, 60 in, and dividing by the width of the trench, 8- in.] The trenches of this system will be installed in the Bt horizon. The lowest K$\text{sat}$ measured in the Bt horizon is 2.8 cm/d, which is equivalent to approximately 0.7 gal/(ft$^2$ d), and the average value is 10.9 cm/d or 2.7 gal/(ft$^2$ d), as reported in Table 4.

Assuming that wastewater is applied to the trenches of this LPP system uniformly, and that the gravel porosity is 35%, the maximum depth of wastewater in the trenches immediately after one dose of wastewater application to an empty trench is $0.1/0.35 = 0.29$ ft. The average surface of infiltration for one daily application of wastewater to one-ft length of an 8-in (0.67 ft) wide trench is then $(0.67 + 0.29) \times 1 = 0.96$ ft$^2$ (approximately 2,900 cm$^2$ for one m length of trench). Based on the average infiltrative surface, the calculated average infiltration rate for one daily application to one-ft length of trench is $0.5/0.96 = 0.52$ gal/(ft$^2$ d) (equivalent to 6,000/2,900 = 2.1 cm/d). Assuming a unit hydraulic gradient and using Darcy’s law for water flow into the soil, the calculated average potential infiltration rate is $2.7$ gal/(ft$^2$ d) [10.9 cm/d], and the average potential minimum infiltration rate is $0.7$ gal/(ft$^2$ d) [2.8 cm/d]. The calculated infiltration rate based on daily wastewater application [0.52 gal/(ft$^2$ d)] is 75% of the infiltration rate obtained using the minimum measured K$\text{sat}$ value and 21% of the infiltration rate using the average K$\text{sat}$ value for the Bt horizon.

Although the calculated infiltration rate using volume of wastewater applied to trenches is less than the minimum K$\text{sat}$ value measured at one location within the drainfield, ponding in the trenches at this location will occur for a relatively long time within each dosing cycle (e.g., 18 hours for every 24-hour dosing cycle). To reduce the ponding period, and allow a margin of safety, one recommendation is to increase the width of trenches from 8 to 12 in (20 to 30 cm). The loading rate calculated based on the trench bottom area for 12-in wide trenches is 0.5 gal/(ft$^2$ d) [equivalent to approximately 0.8 in/d or 2 cm/d]. The maximum height of ponding immediately after a daily application of wastewater is $0.8/0.35 = 2.3$ in (0.19 ft or 5.8 cm), and the average surface area of infiltration for a unit length of trench (1 ft) is $1.19$ ft$^2$ (3,570 cm$^2$ for 1-m length of trench). Based on this value, the average infiltration rate for daily dose of wastewater is $0.5/1.19 = 0.42$ gal/(ft$^2$ d)] [equivalent to 6000/3570 = 1.7 cm/d]. This value represents 60% of the lowest and 15.5% of the average for measured K$\text{sat}$ values of the Bt. As it can be seen, by increasing the width of the trenches, the rate of infiltration for a daily application of wastewater is lowered to a more acceptable level, and we can assume that all the wastewater that will be applied to the trenches can infiltrate the soil within each 24-hour period.

Step 2, Vertical Movement of Applied Wastewater to Ground Water: For this part of analysis we assume that the volume of wastewater infiltrating the soil from the trenches moves vertically in the unsaturated zone below the trenches, and reaches uniformly over the least permeable layer (i.e., BC horizon). The lowest K$\text{sat}$ value in the BC (0.25 cm/d) is for location A5 in the North side of the drainfield (see Fig. 11). The second lowest value is 1 cm/d (not shown in Table 4) for location P3 on the East side of the drainfield. The areal application for the system, 0.1 gal/(ft$^2$ d), or 0.4 cm/d, exceeds the measured K$\text{sat}$ value at location A5 only. To alleviate the potential
problem of saturation above the BC horizon, the effective application rate can be reduced by increasing the distance between the trenches in the North side of the drainfield (e.g., from 150 cm to 300 cm). Another alternative is to adjust the application rate at this area by increasing the spacing between the orifices, and/or varying the size of the orifices on the lateral line. By adjusting the areal application at one area within the drainfield we can assume that all the wastewater applied to the drainfield moves vertically down through the BC horizon without causing any saturation in the zone above the BC (the least permeable layer) horizon.

Step 3, Ground Water Mounding: The depth to Cr horizon varies across the landscape and the hydraulic conductivity of it cannot be measured because it is difficult to bore a hole through the Cr materials at this site using a hand auger. Also, because of the presence of potential fractures, boring a hole through the Cr horizon using a mechanical device (e.g., a motorized screw auger) for the purpose of measuring $K_{sat}$ may yield an unrepresentative value for the vertical hydraulic conductivity of the materials at this site. However, based on the type and morphology of the soil (e.g., relatively thick Bt horizon), and lack of evidence (redoximorphic features) indicating prolonged saturation above the Cr horizon, it does not appears that the Cr horizon at the site impedes water movement.

For determining lateral flow in the potential saturated zone above the Cr we need to determine the maximum allowable depth to the top of mound under the system. For 18-in deep trenches and the North Carolina requirement of 2 ft of unsaturated zone below the bottom of the trenches, the top of any perched water table must be at least 42 in below the land surface at the site. Because of the complexity of the landscape and the position of the drainfield it is difficult to determine how water that may perch above the Cr horizon moves away from the drainfield area. Obviously, since no ground water is present under the proposed system, there is a potential for water to initially move from all sides, but eventually water must run down along the slope of the Cr horizon. For our analysis we assume that the Cr horizon runs parallel to the land surface at the site, and conduct an analysis for the water flow in the west direction. Based on measured elevation and depth to Cr (assuming to be 92 in or 230 cm), the thickness of potential saturated zone at the edge of the drainfield along the west side (parallel to creek) is 50 in (92 - 42 in), and the width of the drainfield where water can move laterally is 330 ft. Based on these assumptions, the minimum cross sectional area for lateral water flow in the potential saturated zone is 1,375 ft$^2$ (128 m$^2$). The slope of the land in E-W direction varies between 3 and 9%, and we assume an average slope of 6% for our calculations. The saturated hydraulic conductivity of the C horizon at location P3 on the west side of the drainfield (see Fig. 11) is 30 cm/d or 1 ft/d. Using Darcy’s law, the volume of water that can potentially move laterally away from the system is 82.5 ($= 1 \times 0.06 \times 1,375$) ft$^3$/d (or approximately 2,300 L/d or 620 gal/d). This amount represents only 13% of the design flow from the system. Obviously, the lateral flow to one direction alone is not adequate for all the applied wastewater to move away from the drainfield area.

Based on the above analysis, we must declare the site unsuitable because all the applied wastewater cannot move away from the system under the assumed conditions. However, as we discussed earlier, based on the morphological evaluation of the soil solum (materials above the C
horizon), the Cr horizon at this site is permeable, and water in the saturated zone that may form above this layer can move in all directions. We continue our analysis by assuming that the ground water in the area is rather deep (as is the case for many areas in the Piedmont region), and that the Cr and its underlying strata are not impermeable.

If wastewater that moves vertically down under the drainfield does not move through the Cr, a zone of saturation will form and water starts to move laterally from the area. As water moves laterally, the surface of infiltration through the Cr horizon increases. The area directly under the drainfield is 47,500 ft$^2$ (equivalent to approximately 4,420 m$^2$) rectangle with dimensions of approximately 330 by 145 ft. Assuming the saturated hydraulic conductivity of the Cr horizon to be 0.1 cm/day (36 cm or approximately 14 inches/year), the surface area for vertical infiltration for the entire daily design flow of 4,750 gal (18,000 L) based on a unit hydraulic gradient (using Darcy’s law) will be 190,000 ft$^2$ or 18,000 m$^2$. This area is equivalent to a rectangle with dimensions of 300 by 630 ft (approximately 90 by 200 m). That is, the base of the saturated zone must expand by 80 to 150 ft in each direction in order for all the applied wastewater to move vertically down through the Cr to recharge the ground water. Obviously, part of the water reaching the saturated zone above the Cr horizon will move laterally. In addition, formation of a saturated zone above the Cr increases the hydraulic gradient forcing more water to move vertically as compared to a unit hydraulic gradient. Therefore, the size of the base of the saturated zone above the Cr will be less than the estimated value based on a unit hydraulic gradient and total recharge of ground water.

In general, the degree of saturation above the Cr horizon depends on the rate of vertical water flow through and lateral water flow above the Cr horizon. If possible, the loading rate and/or the configuration of the drainfield can be modified if we determine that the top of the saturated zone that may form under the drainfield reaches within 60 cm of the bottom of the trenches. If no modification in the design can change the system performance, and our analyses show that the top of the saturated zone reaches within the prescribed distance below the bottom of the trenches (e.g., 2 ft in North Carolina), then the site must be declared unsuitable for the septic system under consideration.