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Storm Water Management through Infiltration Trenches

Bhagu R. Chahar[1], Didier Graillot[2] and Shishir Gaur[3]

Abstract

With urbanization, the permeable soil surface area through which recharge by infiltration can occur is reducing. This is resulting in much less groundwater recharge and greatly increased surface runoff. Infiltration devices, which redirect runoff waters from the surface to the sub-surface environments, are commonly adopted to mitigate the negative hydrologic impacts associated with urbanization. An infiltration trench alone or in combination with other storm water management practice is a key element in present day sustainable urban drainage system. A solution for infiltration rate from an infiltration trench and consequently time required to empty the trench is presented. The solution is in form of integral of complicated functions and requires numerical computation. The solution is useful in quantifying infiltration rate and/or artificial recharge of groundwater through infiltration trenches and the drain time of trench, which is a key parameter in operation of storm water management practice. The solution has been applied on a case study area in Lyon, France. MATLAB programming has been used in the solution.

Key Words: Drainage trench; Infiltration trench; Urban drainage; Storm water; Infiltration; Groundwater; Aquifer; Seepage; Artificial recharge; Best management practice.

Introduction

About half of the world's population is living in urban areas. Land use modifications associated with urbanisation such as the removal of vegetation, replacement of previously pervious areas with impervious surfaces and drainage channel modifications invariably result in changes to the

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characteristics of the surface runoff hydrograph. The hydrologic changes that urban catchments commonly exhibit are, increased runoff peak, runoff volume and reduced time to peak (ASCE, 1975). Consequently, urban areas are more susceptible to flooding affecting all land use activities (Hammer, 1972). Urbanisation also has a profound influence on the quality of stormwater runoff (Hall 1984). Kibler and Aron (1980) reviewed basic elements in urban runoff management. The diversity of an urban catchment makes managing storm water very complicated (Jones and Macdonald, 2007). Safe disposal of stormwater through traditional sewer systems is usually very expensive (Schluter and Jefferies, 2004; Scholz, 2006). The strengths and weaknesses of state and local stormwater management programs were explored, with conclusions and recommendations to correct deficiencies by Howells and Grigg (1981). Zoppou (2001) reviewed the diversity of approaches and parameters that are considered in urban storm water models.

Stormwater management in urban areas is becoming increasingly oriented to the use of low impact development (LID), sustainable urban drainage systems (SUDS), water sensitive urban design (WSUD), best management practices (BMP) or low impact urban design and development (LIUDD) for countering the effect of urban growth, wherein the stormwater is controlled at its source through detention, retention, infiltration, storage, retardation, etc. (Charlesworth et al., 2003, Elliott and Trowsdale, 2007; Kirby, 2005; Martin et al., 2007). These methods include structural measures, such as wetlands, ponds, swales, soakaways, infiltration trenches, roof storage systems, detention/retention basins, infiltration basins, bioretention devices, vegetated filter strips, filter strips, and pervious pavements, etc. The primary objective of these measures is to replicate the pre-urbanisation runoff hydrograph. Under appropriate conditions, these structural measures have proven to be effective (Goonetillekea et al., 2005). Bioretention usage will grow as design guidance matures as a result of continued research and application (Davis et al., 2009). The application of source control options in stormwater management will improve ecological integrity of rivers and streams, reduce flooding in the city and in downstream areas, reduce sediment transport and mitigate erosion and consequently urban stormwater can become a true resource instead of a nuisance (Niemczynowicz, 1999; Braden and Johnston, 2004). Permeable pavement systems (PPS), which are sustainable and cost effective processes (Andersen et al., 1999), are suitable for a wide variety of residential, commercial and industrial applications (Scholz and Grabowiecki, 2007). The general principle of PPS is simply to collect, treat and infiltrate freely any surface runoff to support groundwater recharge. The characteristic feature of sustainable urban drainage is that aesthetics, multiple use and public acceptance of the drainage facilities play a very important role in the planning (Stahre, 2005). Martin et al. (2007) conducted a national survey in France in order to collect feedback from BMP users on their experiences and found that retention processes were used more frequently than infiltration processes (68% vs. 32%) and most of the organizations used BMPs for flood prevention (78.2%) rather than storm water pollution prevention (27.6%). As per the survey, surface detention basins are, in general, the most widely-used BMPs, followed by belowground storage tanks, surface retention ponds, roads and car parks, along with reservoir structures, swales, soakaways, infiltration trenches and lastly roof storage systems. Detention basins are a common feature of stormwater management programs in urban areas and vast literature is available for design of detention basins (Akan, 1990; Baker, 1977; Donahue et al., 1981; Froehlich, 2009; Jones and Jones, 1984; McEnroe, 1992; Mein, 1980). Barrett (2008) explored the performance and relative pollutant removal of several common best management practices using data contained in the International Stormwater BMP Database.

Infiltration supports groundwater recharge (Bouwer et al., 1999), decreases groundwater salinity, allows smaller diameters for sewers (resulting in cost reduction) and improves water quality of receiving waters. Therefore, BMPs based on infiltration are the foundation of many low impact development and green infrastructure practices. Various investigators (Emerson and Traver, 2008; Zheng et al., 2006; Guo, 1999; Guo, 2001; Guo and Hughes, 2001; Raimbault et al., 2002; Sample and Heaney, 2006) have undertaken studies on infiltration basins. Infiltration of storm water through detention and retention basins may increase the risk of groundwater contamination, especially in areas where the soil is sandy and the water table is shallow, and contaminants may not have a chance to degrade or sorb onto soil particles before reaching the saturated zone (Fischer et al., 2003; Brattebo and Booth, 2003). The ‘first flush’ is more polluted than the remainder due to the washout of deposited pollutants by rainfall (Deletic, 1998; Bertrand-Krajewski et al., 1998). This has to be considered in the management and treatment of urban stormwater runoff especially through detention/retention basins (Goonetillekea et al., 2005). Similarly, all runoff from manufacturing industrial areas should be diverted away from infiltration devices because of their relatively high concentrations of soluble toxicants (Pitt et al., 1999). All other runoff should include pretreatment using sedimentation processes before infiltration, to both minimize groundwater contamination and to prolong the life of the infiltration device (if needed). This pretreatment can take the form of grass filters, sediment sumps, wet detention ponds, etc., depending on the runoff volume to be treated and other site specific factors (Pitt et al., 1999).

A full-scale physical model of a modified infiltration trench was constructed by Barber et al. (2003) to test a new storm water best management practice called an ecology ditch. The ditch was constructed using compost, sand, and gravel, and a perforated drain pipe. A series of 14 tests were conducted on the physical model. For larger storms, the ecology ditch managed a peak reduction in the range of 10 to 50%. A grass swale-perforated pipe system results in a pleasant curb less design, which may replace open ditch systems in low density residential areas. Abida and Sabourin (2006) studied a grass swale underlain by a section of perforated pipe enclosed in an infiltration trench. They conducted field tests to measure the infiltration rates of typical grass swales and existing pipe trenches. The total seasonal discharge for a properly designed perforated pipe system was found to be 13 times smaller than that for a conventional stormwater system.

Martin et al. (2007) applied a multicriteria approach to evaluate different BMPs for the decision-making process. The analysis showed that for local government with primary consideration of cost minimisation, the ranking were infiltration trenches, soakaways, porous pavements, roof storage, swales, surface wet retention ponds, belowground storage tanks and dry detention basins. In case of regional planning (planning improvements), the order were infiltration trenches, surface dry detention and wet retention basins, swales and porous pavements, roof storage and soakaways, with storage tanks winding up in the lowest position. For residents association level (environmental protection), infiltration trenches, soakways, porous pavements, swales, surface dry detention and wet retention basins, roof storage and belowground storage tanks were the top to bottom ranking. Thus the infiltration trenches are placed first in all three levels. Their use remain relatively infrequent, probably due to the fact that BMP users are more inclined to choose classical stormwater source control solutions, such as basins and ponds.

Modified rational method was applied by Akan (2002) to size infiltration basins and trenches to control storm water runoff, while the same method was used by Froehlich (1994) to size small storm water pump stations. The critical storm duration producing the maximum runoff volume depends on characteristics of the catchment and rainfall-intensity-duration relation. Although the maximum inflow rate to a detention basin will result from a storm of duration equal to time-of-concentration, the maximum volume will be produced by a storm that lasts significantly longer than the time-of-concentration of the catchment. Akan (2002) presented a design aid for sizing stormwater infiltration trenches. The proposed procedure is based on the hydrological storage equation for an infiltration structure coupled with the Green and Ampt infiltration equation. For the filling process, the two equations were solved simultaneously using a numerical method. For the emptying process, the governing equations were integrated analytically resulting in an algebraic equation that can be solved for the emptying time explicitly. de Souza et al. (2002) presented an experimental study on two infiltration trenches at IPH-UFRGS, in Porto Alegre, Brazil. Both trenches were able to control excessive runoff volumes, which ultimately infiltrated into the soil. The Bouwer Model (1965) was selected to represent the hydraulic functioning of the trenches, taking into account the typical characteristics of the regional soil (with high percentage of clay).

The literature review shows that urbanization of a watershed with its associated impact on the quantity and quality of storm-water runoff has resulted in the implementation of a number of alternatives for storm-water management. Infiltration trenches are one of them. An infiltration trench is an underground-storage zone filled with clean gravel or stone (Fig 1). Infiltration trenches are constructed to temporarily store storm runoff and let it percolate into the underlying soil. Such trenches are used for small drainage areas. They are typically used for control of runoff from residential lots, commercial areas, parking lots, and open spaces like ring roads. Also, they are relatively easy to construct in the perimeters and other unutilized areas of a development site. Moreover, they can be used below the porous pavements or with grass swales (Fig 1) and combined with detention basins, etc. Furthermore, they can be provided below pavements, walkways, pedestrian or cycle tracks so no additional area is required like other storm water management practices. Infiltration trench emptying time is important to operate and manage storm water. If the time between two successive storms is less than the trench emptying time then the excess storm water should be diverted to another detention basin or to the storm sewer. Unlike detention basins (Emerson and Traver, 2008; Zheng et al., 2006; Guo, 1999; Guo, 2001; Guo and Hughes, 2001; Sample and Heaney, 2006), widely accepted design standards and procedures for infiltration trenches do not exist. The present study finds a solution for the infiltration rate from a trapezoidal trench and time required to empty the infiltration trench.

Analytical Solution

Infiltration trenches are generally long, moderately wide, and shallow in dimensions. They are filled with coarse gravel to provide storage; they collect runoff from adjacent paved areas and infiltrate the water into the aquifer beneath. The coarse gravel fill material in the trench is usually much more permeable than the underlying soil, so there is negligible resistance to flow within the trench and the perimeter of the trench is an equipotential surface. Let the aquifer be composed of multi-layer porous medium, such that upper layer has hydraulic conductivity less than the lower layers. If water table in the aquifer is lower than bottom of the top layer then the wetting front of infiltrating water from the trench will advance all around and may saturate the low permeable top layer but seepage flow in lower more pervious layers will be unsaturated. For example, when seepage from a lined canal takes place, and liner conductivity is much less than that of the underlying soil medium, the soil medium remains unsaturated (Polubarinova- Kochina ,1962). In such situations the lower unsaturated layers act as drainage layer to the top saturated layer and ultimately recharge the aquifer. The position of water table in the aquifer is governed by horizontal or vertical controls in terms of river, stream or pumping wells present within the aquifer boundary. Let a trapezoidal trench (as shown in Fig 2) of bed width b (m), depth of water y (m), and side slope m (1 Vertical : m Horizontal) is constructed in such aquifer and the saturated hydraulic conductivity of the top layer is k (m/s). Also, assume the thickness of the top layer below the bed of the trench is d (m). As the length of the trench is very large, seepage flow can be considered 2D in the vertical plane. Initially the top layer is unsaturated and seepage from the trench is unsteady but after some time the layer will get saturated and steady seepage will establish.

By means of the above stated assumptions, the seepage from the infiltration trench becomes identical to the steady seepage discharge per unit length of channel q_s (m^2/s) from a trapezoidal channel analysed by Chahar (2007). In that work, an exact analytical solution for the quantity of seepage from a trapezoidal channel underlain by a drainage layer at a shallow depth was obtained using an inverse hodograph and Schwarz-Christoffel transformation, the solution is

$$q_s = 2k(d+y)K\left(\sqrt{\gamma/\beta}\right)\Big/K\left(\sqrt{(\beta-\gamma)/\beta}\right) \tag{1}$$

where β and γ = transformation variables; and $K\left(\sqrt{\gamma/\beta}\right)$ and $K\left(\sqrt{(\beta-\gamma)/\beta}\right)$ = complete elliptical integrals of the first kind with a modulus $\left(\sqrt{\gamma/\beta}\right)$ and $\left(\sqrt{(\beta-\gamma)/\beta}\right)$, respectively (Byrd and Friedman, 1971). This involves two transformation parameters β and γ those can be determined by solving the following two equations

$$\frac{d+y}{y} = 2K\left(\sqrt{(\beta-\gamma)/\beta}\right)B(1/2,\sigma)\Bigg/\sqrt{\beta}\int_{\gamma}^{\beta}\frac{B_\tau(1/2,\sigma)\,d\tau}{\sqrt{\tau(\beta-\tau)(\tau-\gamma)}} \tag{2}$$

$$\frac{b}{y} = 2\int_{\beta}^{1}\frac{\left(B(1/2,\sigma)-B_\tau(1/2,\sigma)\right)d\tau}{\sqrt{\tau(\tau-\beta)(\tau-\gamma)}}\Bigg/\int_{\gamma}^{\beta}\frac{B_\tau(1/2,\sigma)\,d\tau}{\sqrt{\tau(\beta-\tau)(\tau-\gamma)}} \tag{3}$$

where $\pi\sigma = \cot^{-1} m$; τ = dummy variable; $B(1/2, \sigma)$ = complete Beta function (Abramowitz and Stegun, 1972); and $B_\tau(1/2, \sigma)$ = incomplete Beta function (Abramowitz and Stegun, 1972) defined as

$$B_\tau(1/2,\sigma) = 2\sqrt{\tau}\;{}_2F_1(1/2,1-\sigma;3/2;\tau) \tag{4}$$

in which ${}_2F_1$ is a Gauss-Hypergeometric series (Abramowitz and Stegun, 1972) given by

$${}_2F_1(a,b;c;\tau) = 1+\frac{a\cdot b}{c}\tau+\frac{a(a+1)\cdot b(b+1)}{c\cdot(c+1)\cdot 1\cdot 2}\tau^2+\frac{a(a+1)(a+2)b(b+1)(b+2)}{c\cdot(c+1)\cdot(c+2)\cdot 1\cdot 2\cdot 3}\tau^3+\ldots \tag{5}$$

The range of transformation parameters is $0 \le \gamma \le \beta \le 1$. The parameter γ represents the effect of the drainage layer such that $\gamma \to 0$ as $d/y \to \infty$ and $\gamma \to \beta$ as $d/y \to 0$; while the parameter β represents the effect of the water depth in the trench such that $\beta \to 0$ as $b/y \to \infty$ and $\beta \to 1$ as $b/y \to 0$. It is evident from Eqs (1) to (3) that the infiltration from a trench depends on trench dimensions, depth of water in trench, hydraulic conductivity of porous medium, and depth of drainage layer (i.e. lower unsaturated medium of higher hydraulic conductivity) and location of the ground water table.

The trenches are designed to store and to infiltrate a captured volume of runoff that is generated from its contributing area during a specific-design storm. The rational formula is used for calculating runoff from small catchments, particularly in urban areas where a large portion of the land surface is impervious. The corresponding times of concentration and thus the critical rainfall durations will also be small, typically much less than one hour. The filling process begins when the runoff first reaches the trench. The filling process can end and the emptying process can begin while the trench is still receiving runoff if the rate of infiltration from the trench exceeds the inflow rate. However, runoff rates are normally much higher than the infiltration rates during a design storm event. Therefore, it is reasonable to assume that the filling process will continue until the entire captured runoff has entered the trench. The runoff captured during the filling process is stored partly in the trench and partly within the wetted zone of the soil. The emptying process starts when the runoff into the trench ceases. The emptying time of the infiltration trench is a function of initial volume of water in trench and rate of infiltration from it. To find the time to empty the infiltration trench, let the initial water depth in the trench be y and the steady infiltration rate be q_s and then the water level in the trench will be lowered by dy in small time interval dt due to steady infiltration. Equating the volume of water in this strip to the infiltration volume in the time interval dt

$$q_s dt = \eta(b + 2my)dy \tag{6}$$

where η = porosity of refilled material in the trench. This equation can be integrated, after substituting q_s from Eq. (1), to determine the time taken in lowering the water level in trench from y_1 to y_2 as

$$\Delta t = \frac{\eta}{2k}\int_{y_2}^{y_1}\frac{(b + 2my)K\left(\sqrt{(\beta-\gamma)/\beta}\right)}{(d + y)K\left(\sqrt{\gamma/\beta}\right)}dy \tag{7}$$

Eq. (2) can be used to eliminate (d + y). Let time zero denote when the trench first starts to empty, then the total time required to empty the trench is

$$t = \frac{\eta}{4kB(1/2,\sigma)}\int_0^y \frac{\sqrt{\beta}}{K\left(\sqrt{\gamma/\beta}\right)}\int_\gamma^\beta \frac{B_\tau(1/2,\sigma)d\tau}{\sqrt{\tau(\beta-\tau)(\tau-\gamma)}}\left(\frac{b}{y} + 2m\right)dy \tag{8}$$

Simultaneous solution of Eqs. (2) and (3) for the given trench dimensions (b and σ), the depth of the unsaturated layer (d) and the depth of water in trench (y) at particular instant results in corresponding parameters β and γ. Thus integrand in Eq. (7) or Eq. (8) for any y and corresponding β and γ for fixed values of b, σ, and d can be computed. However these steps involve complicated integrals with implicit transformation variables. These integrals (complete and incomplete beta functions, complete and incomplete elliptical integrals, and remaining improper integrals) can be evaluated using numerical integration (Press et al., 1992) after converting the improper integrals into proper integrals (Chahar, 2007).

If single trench is insufficient for a given storm runoff contributing area, then an array of parallel trenches may be adopted. The minimum centre to centre spacing *S* (m) between two adjacent trenches (see Fig 2) can be determined using the following relation (Chahar, 2007)

$$S = \frac{(d+D)\sqrt{\beta}}{K\left(\sqrt{(\beta-\gamma)/\beta}\right)B(1/2,\sigma)}\int_0^{\gamma}\frac{\left(B(1/2,\sigma)-B_{\tau}(1/2,\sigma)\right)d\tau}{\sqrt{\tau(\beta-\tau)(\gamma-\tau)}} \tag{9}$$

wherein *D* = full depth of trench (m); and β and γ are simultaneous solution of Eqs. (2) and (3) with $y = D$. There will be interference between the infiltrations from adjacent trenches, if the spacing is kept smaller than *S*.

Generally excavating machinery digs a trench with vertical sides. If the soil can support vertical side slopes temporarily (till refilled with gravel), then rectangular trenches (rather than trapezoidal trenches) are more convenient, economical and faster to construct. For a rectangular trench, the corresponding relations are

$$\frac{d+y}{y} = \pi K\left(\sqrt{(\beta-\gamma)/\beta}\right)\Big/ \sqrt{\beta}\int_{\gamma}^{\beta}\frac{\tan^{-1}\sqrt{\tau/(1-\tau)}}{\sqrt{\tau(\beta-\tau)(\tau-\gamma)}}\,d\tau \tag{10}$$

$$\frac{b}{y} = \int_{\beta}^{1}\frac{\pi-2\tan^{-1}\sqrt{\tau/(1-\tau)}}{\sqrt{\tau(\tau-\beta)(\tau-\gamma)}}\,d\tau \Big/ \int_{\gamma}^{\beta}\frac{\tan^{-1}\sqrt{\tau/(1-\tau)}}{\sqrt{\tau(\beta-\tau)(\tau-\gamma)}}\,d\tau \tag{11}$$

$$t = \frac{\eta b}{2k}\int_0^{y}\frac{K\left(\sqrt{(\beta-\gamma)/\beta}\right)}{(d+y)K\left(\sqrt{\gamma/\beta}\right)}dy = \frac{\eta}{2\pi k}\int_0^{y}\left(\frac{b}{y}\frac{\sqrt{\beta}}{K\left(\sqrt{\gamma/\beta}\right)}\int_{\gamma}^{\beta}\frac{\tan^{-1}\sqrt{\tau/(1-\tau)}}{\sqrt{\tau(\beta-\tau)(\tau-\gamma)}}\,d\tau\right)dy \tag{12}$$

$$S = \frac{(d+D)\sqrt{\beta}}{\pi K\left(\sqrt{(\beta-\gamma)/\beta}\right)}\int_0^{\gamma}\frac{\pi-2\tan^{-1}\sqrt{\tau/(1-\tau)}}{\sqrt{\tau(\beta-\tau)(\gamma-\tau)}}\,d\tau \tag{13}$$

Many times the top soil layer may extend up to large depth ($d/D > b/D + 2m + 5$) and water table may also lie at large depth then the solution becomes independent of the location of more previous lower layer. For this case $\gamma \rightarrow 0$ since $d/y \rightarrow \infty$. The corresponding relations for trapezoidal trenches with $\gamma = 0$ are

$$\frac{b}{y} = 2\int_{\beta}^{1} \frac{\left(B(1/2,\sigma) - B_{\tau}(1/2,\sigma)\right)d\tau}{\tau\sqrt{(\tau-\beta)}} \Bigg/ \int_{0}^{\beta} \frac{B_{\tau}(1/2,\sigma)d\tau}{\tau\sqrt{(\beta-\tau)}} \tag{14}$$

$$t = \frac{\eta}{2\pi\, k\, B(1/2,\sigma)} \int_{0}^{y} \left(\sqrt{\beta}\left(2m + \frac{b}{y}\right) \int_{0}^{\beta} \frac{B_{t}(1/2,\sigma)d\tau}{\tau\sqrt{\beta-\tau}} \right) dy \tag{15}$$

$$S = \frac{2\pi D\, B(1/2,\sigma)}{\sqrt{\beta}} \Bigg/ \int_{0}^{\beta} \frac{B_{t}(1/2,\sigma)d\tau}{\tau\sqrt{\beta-\tau}} \tag{16}$$

For the similar condition, the following are the solution equations for array of rectangular trenches

$$\frac{b}{y} = \int_{\beta}^{1} \frac{\pi - 2\tan^{-1}\sqrt{\tau/(1-\tau)}}{\tau\sqrt{(\tau-\beta)}}\, d\tau \Bigg/ \int_{0}^{\beta} \frac{\tan^{-1}\sqrt{\tau/(1-\tau)}}{\tau\sqrt{(\beta-\tau)}}\, d\tau \tag{17}$$

$$t = \frac{\eta}{\pi^{2} k} \int_{0}^{y} \left(\frac{b}{y}\sqrt{\beta} \int_{0}^{\beta} \frac{\tan^{-1}\sqrt{\tau/(1-\tau)}}{\tau\sqrt{\beta-\tau}}\, d\tau \right) dy \tag{18}$$

$$S = \pi^{2} D \Bigg/ \sqrt{\beta} \int_{0}^{\beta} \frac{\tan^{-1}\sqrt{\tau/(1-\tau)}}{\tau\sqrt{\beta-\tau}}\, d\tau \tag{19}$$

Application on a Case Study

Lyon is the second largest city in France located on the Rhone River which is the third largest river in France. The population of the urban area in Lyon is 1.2 Millions and the city area is about 500 km^{2}. In the past, stormwater was managed through combined storm water sewer system in the old part of the city and through separate system in new area and at the periphery. With increase in area and population the traditional system became inefficient and uneconomical. The authorities are now looking for other best management practices, which are more efficient and more adaptable to linear drainage areas like ring roads. Thus the local authorities of Grand Lyon adopted alternative storm management practices, such as detention and infiltration basins. At present 100 devices (detention and infiltration trenches and basins) exist in the city area, which are managing 10^6 m^3 of storm water. Studies and experiments on infiltration trenches (Chocat et al., 1997; Proton, 2008) have demonstrated their performance to reduce storm water flows. Dechesne et al. (2005) studied long-term evaluation of clogging and soil pollution in four infiltration basins in Lyon. These basins are 10 to 21 years old and still have good infiltration capacities. Winiarski et al. (2006) investigated impact of stormwater on aquifer medium of Django-Reinhardt infiltration basin in Lyon. Goutaland et al. (2007) and Goutaland et al. (2008) conducted hydrogeophysical study of the same infiltration basin.

Lyon is on the banks of the Rhone River below which alluvial deposits underlie. Types of alluvial deposits in Lyon are glaciofluvial and fluvial deposits and they form good aquifers. The vadose zone overlying these aquifers plays a dominant role in recharging aquifer and in contaminant retention mechanism. Sedimentary deposits, constituting aquifers and vadose zones, are complex, three-dimensional, heterogeneous and commonly anisotropic (Goutaland et al. 2008). Infiltration trenches have been built on real-scale in completely controlled conditions adjacent to one of the main ring roads in North of Grand Lyon. The drainage basin is limited to a band along the road and is rather impervious. The contributing area is about 2.2 hectares. Interception of run-off is achieved with a pipe of 100 mm of diameter which is connected to the sewer under the road. The discharge into the trenches is regulated with a gate so that they can be completely isolated from the sewer. In this case the trenches are not connected to the sewer and are directly supplied with storm runoff. Prior to inlet into the infiltration trenches, storm water is stored in a detention basin, which is lined with an impervious geomembrane. The volume of the detention basin is 60 m^3. Considering the average of rainfall in this region, the detention basin can be filled 30 times per year. A pumping system allows controlled supply to the trenches. The water levels into the trenches are measured with submerged pressure sensors. All these equipments have been described in (Proton, 2008). The shape of the observation trench is trapezoidal and it has dimensions as follwing: depth = 1.0 m; bed width = 0.8 m; and side slopes = 0.45 (1 Vertical: 0.45 Horizontal). The length of the trench is 12 meters and the refilled material in the trench provides porosity = 0.25. The soil adjacent to trench has varying hydraulic conductivity. The top soil layer is underlain by another highly pervious layer at a depth of 10.0 m and the prevailing water table is about 18.0 m below the ground surface. Observations on water level vs. time are available at three locations (H1, H2, and H3) from 1986 to 1991 by Essai

(Proton, 2008) and 2005 by Proton (2008). The starting water depth varied from 0.62 to 0.75 and the emptying time was 60 to 150 minutes.

The time Eq. (8) involves β and γ in the integrand. For β and γ, Eqs. (2) and (3) should be solved simultaneously. However, since these equations are nonlinear and contain improper integrals, an indirect method has been used to find β and γ values. The method consists of *fsolve* function of the MATLAB (2010) program. The objective function has been constituted as

$$f(\beta,\gamma)=\left(\frac{d}{y}+1-f_1(\sigma,\beta,\gamma)\right)^2+\left(\frac{b}{y}-f_2(\sigma,\beta,\gamma)\right)^2 \tag{20}$$

where $f_1(\sigma, \beta, \gamma)$ and $f_2(\sigma, \beta, \gamma)$ are right hand sides of Eqs. (2) and (3), respectively. Since minimum of this function is zero, which can only be attained when both parts of the function reach zero values and hence satisfy Eqs. (2) and (3). After removing singularities and using Gaussian quadratures (96 points for weights and abscissa for both inner and outer integrals) for numerical integration (Abramowitz and Stegun, 2001), the function has been minimized for β and γ for a particular set of σ, b/y and d/y. To find the emptying time of the trench the above scheme has been incorporated in computation of Eq. (8) through the MATLAB (2010) programming.

An appropriate value of hydraulic conductivity was not known, so an average of observed time to drop water level from 0.6 m to 0.2 m at three locations (H1, H2, and H3) for three years (1987, 1989, and 1991) equal to 45 minutes has been used to get equivalent hydraulic conductivity, which came out to be = 1.7809×10^{-5} m/s. With k = 1.7809×10^{-5} m/s; η = 0.25; b = 0.8 m; and m = 0.45 the resulting graphs with different starting water depths (i.e. 0.9 m, 0.75 m, 0.6 m, 0.45 m, and 0.3 m) are plotted in Fig 3. The computed emptying times have been 173.7 min, 159.9 min, 145.4 min, 130.0 min, and 113.1 min, respectively. The graphs are asymptote to the time axis and thus result into large emptying time for the final small water depths in the trench. Had the time taken to empty the last one cm of water depth been not considered, the empty times would have been 104.1 min, 90.3 min, 75.8 min, 60.4 min, and 43.4 min, respectively. If multiple trenches were adopted, then Eq. (9) or (19) would yield the required spacing between trenches = 4.59 m.

For the case of rectangular trench, the corresponding graphs are shown in Fig 4. The emptying times for the rectangular ditch are 147.8 min, 140.1 min, 131.2 min, 120.9 min, and 108.3 min, respectively for the complete emptying case and 78.3 min, 70.6 min, 61.7 min, 51.3 min, and 38.7 min, respectively when the last cm of water depth is not considered. The required spacing between trenches in this case = 4.15 m. Both graphs show that substantial time is taken to drain the last 1 cm of water depth. For more effective operation of the trenches, they may be refilled with water before complete emptying. This will also establish early saturated flow in the next cycle.

Discussions

Eq. (1) assumes that groundwater flow is viscous and steady and follows the Darcy law, so the governing equation is 2D Laplace equation. It has also been assumed that soil around the trench is saturated. During the initial period, the medium is unsaturated, the flow is unsteady, and the infiltration rates are high. As the saturation of the soil around the trench increases, the infiltration rate decreases exponentially with time. It may acquire a relatively constant rate (approaching to saturated hydraulic conductivity) within 20-30 minutes (Duchene et al., 1994). The wetting front moves fast and saturated flow conditions exist within this front. If antecedent-soil moisture is present then the attainment of saturation and constant rate of infiltration are even faster. Generally the surface infiltration rate is higher than the hydraulic conductivity of aquifer material, so the slow ground water motion will cause saturation to the surrounding area of the trench. As a result, the operation of the trench is controlled by the saturated seepage rate rather than the infiltration rate at the surface. Under this condition, the trench designed with a high infiltration rate becomes undersized and hence it is important to consider saturated seepage rate (Guo, 1998). Therefore the assumption of saturated porous medium with constant hydraulic conductivity is realistic except for a limited initial phase of the operation. Effects of these assumptions are underestimation of the rate of infiltration from the trench and overestimation of the drain time of trench and thus, these assumptions are on conservative side.

Infiltration trenches are more effective where the soil has adequate hydraulic conductivity. In most alluvial deposits the soil is stratified. In many cases, highly permeable layers of sand and gravel underlie the top low permeable layer of finite depth. In those cases the high conductivity lower layer acts as a free drainage layer for the top seepage layer since all the seeping water received by this layer is insufficient to saturate it. If the stratified medium comprises more than two layers and the top saturated layer has hydraulic conductivity less than that of the next layer which is unsaturated then Eq. (1) is still valid irrespective of hydraulic conductivities and saturation conditions in the remaining lower layers. Thus the boundary condition assumed in Eq. (1) is likely to be applicable in many field problems. Efficiency of an infiltration trench decreases with increase in the depth of drainage layer (d). For drainage layer and water table both at large depth, i.e., $d/D > b/D + 2m + 5$, the special case solutions given by Eqs. (14) – (19) are applicable. If the water table and/or bedrock are at shallow depth, the infiltration trenches are ineffective and hence should not be used.

The infiltration trenches may also experience clogging problems due to settlement of fine sand particles in the interstices of soil. To minimize this, the runoff should be passed through well maintained sediment filters or detention basins prior to entry into the infiltration trench. Further, Duchene et al. (1994) observed that the impact of sediment clogging in the bottom of the trench is limited and hence the effect of clogging has not been considered in this study. The porous medium in vicinity of the trench may not be homogeneous and isotropic in true sense and hence the estimation of equivalent hydraulic conductivity of the medium may be difficult. Clogging due to migration of sediments and development of microbial growth will further change the hydraulic conductivity of the medium. As per Eq (8), the emptying time is inversely proportional to the hydraulic conductivity of the saturated porous medium of the top layer. Therefore any alteration in the conductivity value can easily be incorporated into emptying time while other parameters remain unaffected. Also, the analysis is based on the assumption of seepage flow 2D in the vertical plane, which will happen for a very large length of a trench. For finite length of a trench if its length to bed width ratio is more than 10 then the seepage flow will be 2D in the vertical plane except at the ends, therefore the present analysis is valid with negligible error for such trenches.

There is a risk of ground-water contamination if the volume of contaminants infiltrated is greater than the natural attenuation capacity of the underlying soils. This can happen if contaminants move too rapidly through the soils of high hydraulic conductivity overlying an aquifer. The type of soil underlying an infiltration trench and the distance to the water table are major determinants of the potential of ground-water contamination. A minimum of 1.25 m between the bottom of the trench and the ground water table should be insured (Guo, 1998).

Emptying time is important to operate the infiltration trench for storm water management. The captured volume of runoff is temporarily stored in the voids of the gravel and subsequently it will infiltrate into the soil adjacent to the trench and down to the aquifer. After emptying time or design storage time, the trench will be empty and ready for the next runoff. Both the captured volume and emptying time depend on the purpose of the infiltration structure and the stormwater management (Akan, 2002). The captured volume for infiltration trenches can be calculated as the volume of 12.5 mm of runoff over the impervious portion of the contributing area and the storage time for water quality infiltration basins vary from 24 h to 72 h for different agencies (Akan, 2002). The contributing drainage area to an infiltration trench is usually less than 4 ha due to storage requirements for peak-runoff control (Duchene et al., 1994). Partial storm-water control is provided for storms that produce more runoff than can be stored within the trench. An overflow for the trench is necessary to handle excess runoff that is produced from storms larger than the design storm. On large sites, other storm-water practices, such as detention basins can be used in conjunction with trenches to provide the necessary peak-runoff control. Moreover, if the time between two successive storms is less than the trench emptying time then the excess storm water should be diverted to detention basins. Infiltration trenches should be designed to drain completely within 72 h after the design event (Duchene et al., 1994). This allows the soils underlying a trench to drain and to maintain aerobic conditions, which improves the pollutant removal capability of the soil underlying the trench. Therefore to manage a stormwater generated by a particular catchment, trench dimensions can initially be fixed based on the runoff volume and the porosity of the refilled material and then the corresponding drain time can be computed. If the drain time of trench is not within desired limits, the depth and width of the trench may be adjusted to achieve it. Thus a trial-and-error method may be adopted to arrive at an appropriate design of an infiltration trench.

Conclusions

Infiltration trenches can control quality and quantity of storm water from small urban catchment. The surface hydrology of the catchment (i.e. runoff volume) determines the size of an infiltration trench while the hydraulic conductivity of the aquifer governs the emptying time of the trench. Solutions derived for the steady saturated seepage state can provide a guideline for determining the required size of trenches and their spacing/numbers. The proposed design is simple enough to obtain the first estimation of the required time to empty the trench. It overestimates emptying time and hence estimates are on the conservative side to provide a margin-of-safety. The presented result may assist a stormwater management engineer in the design of infiltration trenches.

Notation

The following symbols are used in this paper:

$B(.,.)$	complete Beta function [dimensionless];
$B_t(.,.)$	incomplete Beta function [dimensionless];
b	bed width of trench [m];
D	full depth of trench [m];
d	depth of unsaturated medium/aquifer below bed of trench [m];
$K(.)$	complete elliptical integral of the first kind [dimensionless];
k	hydraulic conductivity of top layer [m/s];
m	side slope of trench (1 Vertical : m Horizontal) [dimensionless];
q_s	seepage discharge per unit length of trench [m^2/s];
S	spacing between adjacent trenches [m];
T	top width of trench at full depth [m];
t	trench empty time [s];
y	water depth in trench [m];
β, γ	transformation variables [dimensionless];
σ	$(1/\pi)\cot^{-1} m$ [dimensionless]; and
τ	dummy variable [dimensionless].

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Figure Captions

Fig. 1. Trapezoidal Infiltration Trench below a Porous Pavement or with Grass Cover

Fig. 2. Trapezoidal Infiltration Trenches underlain by an Unsaturated Porous Medium

Fig. 3. Emptying Time for different Starting Depths in a Trapezoidal Infiltration Trench

Fig. 4. Emptying Time for different Starting Depths in a Rectangular Infiltration Trench

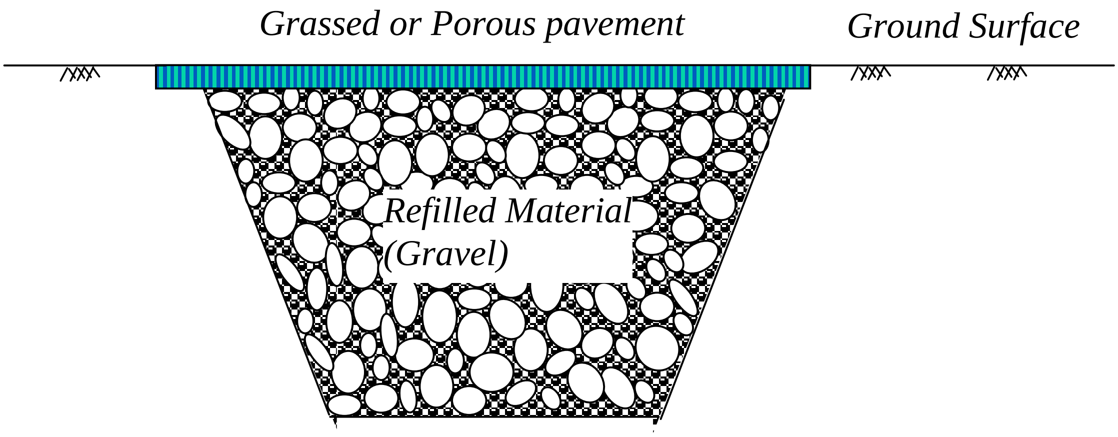

Fig. 1. Trapezoidal Infiltration Trench below a Porous Pavement or with Grass Cover

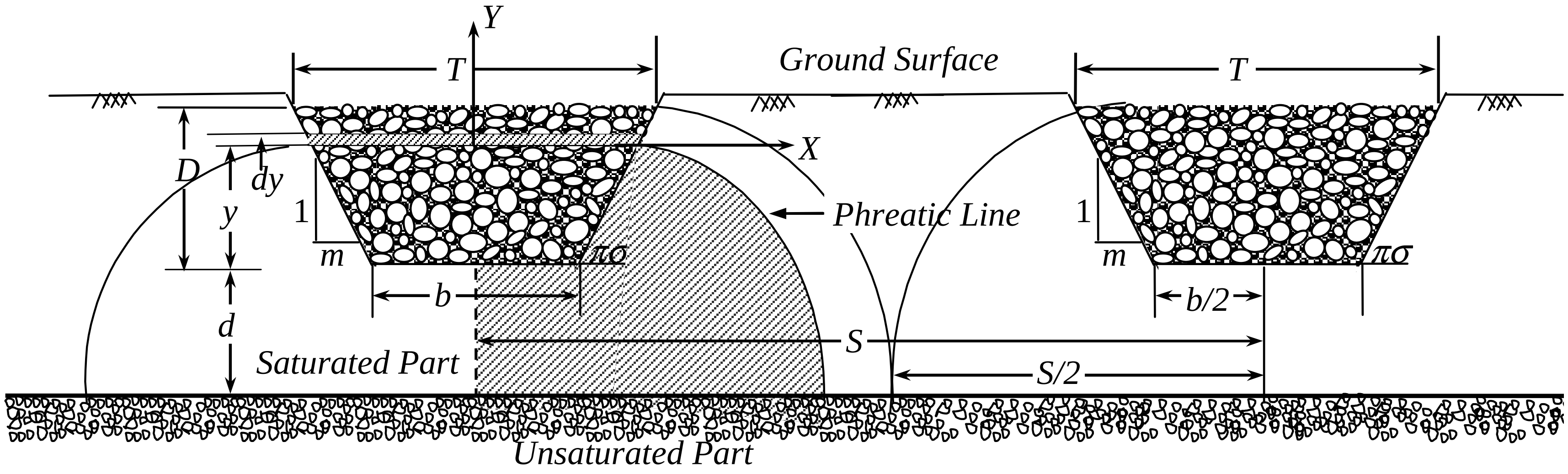

Fig. 2. Trapezoidal Infiltration Trenches underlain by an Unsaturated Porous Medium

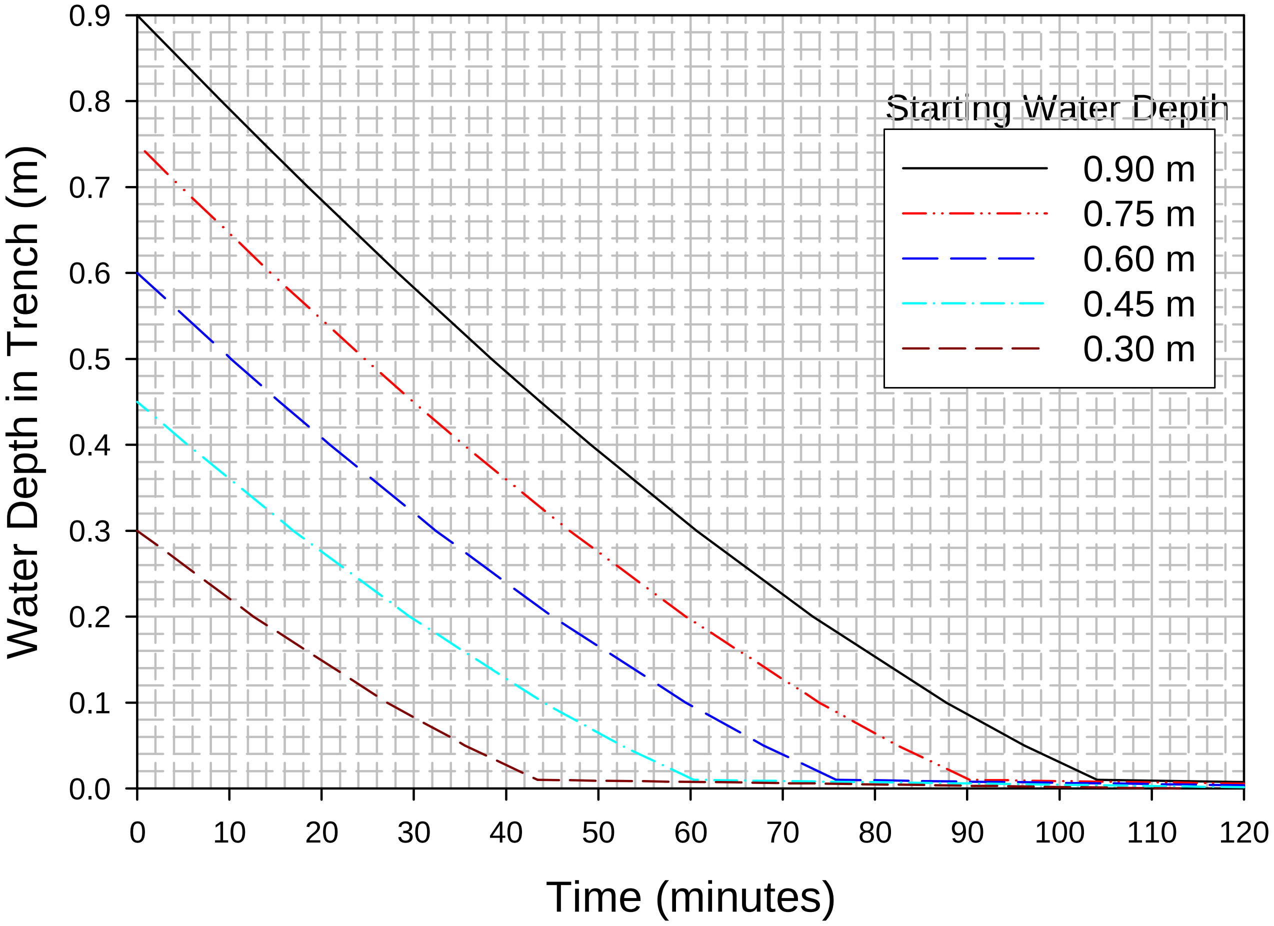

Fig. 3. Emptying Time for different Starting Depths in a Trapezoidal Infiltration Trench

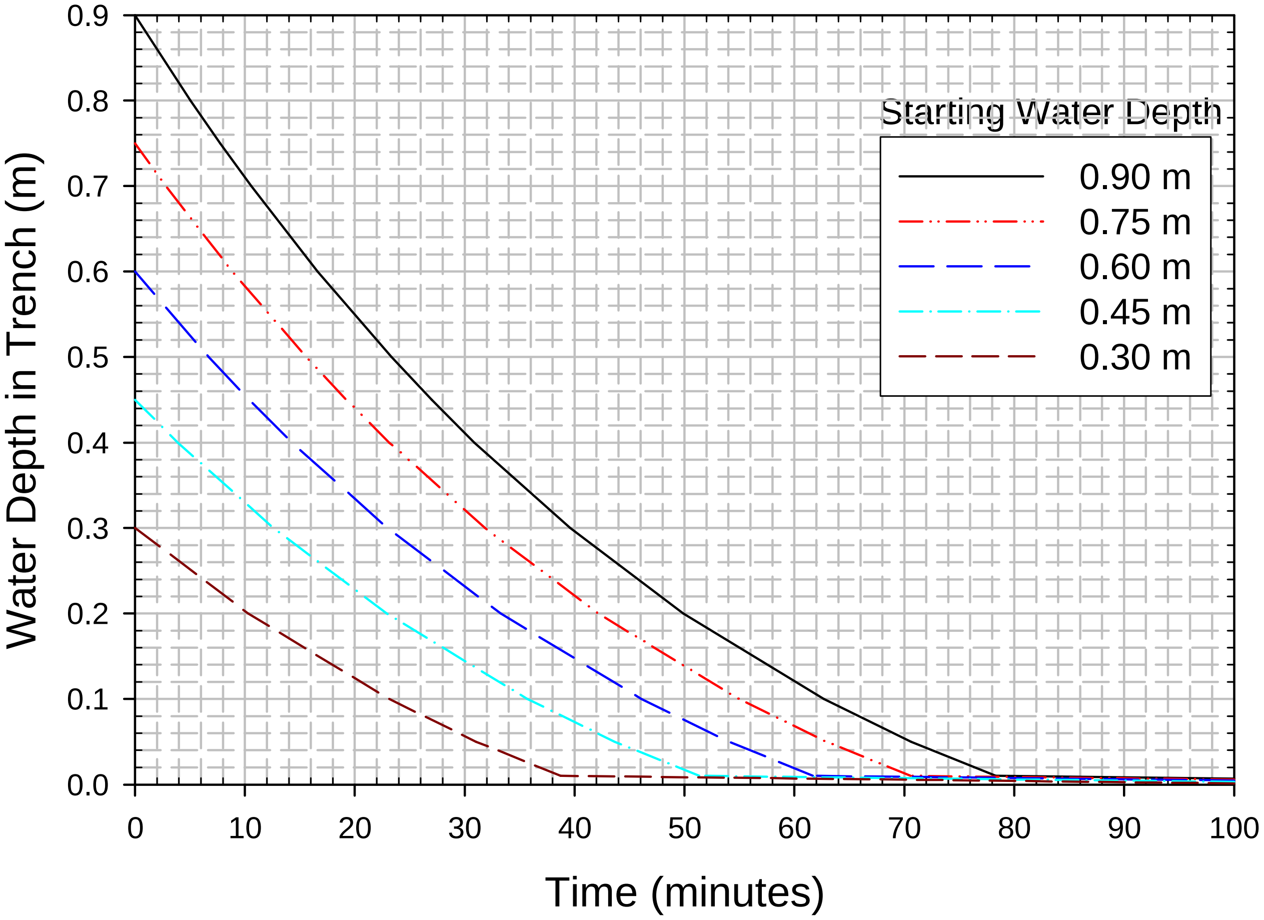

Fig. 4. Emptying Time for different Starting Depths in a Rectangular Infiltration Trench