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# Storm Water Management through Infiltration Trenches

Bhagu R. Chahar<sup>1</sup>, Didier Graillet<sup>2</sup> and Shishir Gaur<sup>3</sup>

## 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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25 characteristics of the surface runoff hydrograph. The hydrologic changes that urban catchments  
26 commonly exhibit are, increased runoff peak, runoff volume and reduced time to peak (ASCE,  
27 1975). Consequently, urban areas are more susceptible to flooding affecting all land use  
28 activities (Hammer, 1972). Urbanisation also has a profound influence on the quality of  
29 stormwater runoff (Hall 1984). Kibler and Aron (1980) reviewed basic elements in urban runoff  
30 management. The diversity of an urban catchment makes managing storm water very  
31 complicated (Jones and Macdonald, 2007). Safe disposal of stormwater through traditional sewer  
32 systems is usually very expensive (Schluter and Jefferies, 2004; Scholz, 2006). The strengths and  
33 weaknesses of state and local stormwater management programs were explored, with  
34 conclusions and recommendations to correct deficiencies by Howells and Grigg (1981). Zoppou  
35 (2001) reviewed the diversity of approaches and parameters that are considered in urban storm  
36 water models.

37 Stormwater management in urban areas is becoming increasingly oriented to the use of  
38 low impact development (LID), sustainable urban drainage systems (SUDS), water sensitive  
39 urban design (WSUD), best management practices (BMP) or low impact urban design and  
40 development (LIUDD) for countering the effect of urban growth, wherein the stormwater is  
41 controlled at its source through detention, retention, infiltration, storage, retardation, etc.  
42 (Charlesworth et al., 2003, Elliott and Trowsdale, 2007; Kirby, 2005; Martin et al., 2007). These  
43 methods include structural measures, such as wetlands, ponds, swales, soakaways, infiltration  
44 trenches, roof storage systems, detention/retention basins, infiltration basins, bioretention  
45 devices, vegetated filter strips, filter strips, and pervious pavements, etc. The primary objective  
46 of these measures is to replicate the pre-urbanisation runoff hydrograph. Under appropriate  
47 conditions, these structural measures have proven to be effective (Goonetillekea et al., 2005).  
48 Bioretention usage will grow as design guidance matures as a result of continued research and  
49 application (Davis et al., 2009). The application of source control options in stormwater  
50 management will improve ecological integrity of rivers and streams, reduce flooding in the city  
51 and in downstream areas, reduce sediment transport and mitigate erosion and consequently urban  
52 stormwater can become a true resource instead of a nuisance (Niemczynowicz, 1999; Braden and  
53 Johnston, 2004). Permeable pavement systems (PPS), which are sustainable and cost effective  
54 processes (Andersen et al., 1999), are suitable for a wide variety of residential, commercial and  
55 industrial applications (Scholz and Grabowiecki, 2007). The general principle of PPS is simply

56 to collect, treat and infiltrate freely any surface runoff to support groundwater recharge. The  
57 characteristic feature of sustainable urban drainage is that aesthetics, multiple use and public  
58 acceptance of the drainage facilities play a very important role in the planning (Stahre, 2005).  
59 Martin et al. (2007) conducted a national survey in France in order to collect feedback from  
60 BMP users on their experiences and found that retention processes were used more frequently  
61 than infiltration processes (68% vs. 32%) and most of the organizations used BMPs for flood  
62 prevention (78.2%) rather than storm water pollution prevention (27.6%). As per the survey,  
63 surface detention basins are, in general, the most widely-used BMPs, followed by belowground  
64 storage tanks, surface retention ponds, roads and car parks, along with reservoir structures,  
65 swales, soakaways, infiltration trenches and lastly roof storage systems. Detention basins are a  
66 common feature of stormwater management programs in urban areas and vast literature is  
67 available for design of detention basins (Akan, 1990; Baker, 1977; Donahue et al., 1981;  
68 Froehlich, 2009; Jones and Jones, 1984; McEnroe, 1992; Mein, 1980). Barrett (2008) explored  
69 the performance and relative pollutant removal of several common best management practices  
70 using data contained in the International Stormwater BMP Database.

71 Infiltration supports groundwater recharge (Bouwer et al., 1999), decreases groundwater  
72 salinity, allows smaller diameters for sewers (resulting in cost reduction) and improves water  
73 quality of receiving waters. Therefore, BMPs based on infiltration are the foundation of many  
74 low impact development and green infrastructure practices. Various investigators (Emerson and  
75 Traver, 2008; Zheng et al., 2006; Guo, 1999; Guo, 2001; Guo and Hughes, 2001; Raimbault et  
76 al., 2002; Sample and Heaney, 2006) have undertaken studies on infiltration basins. Infiltration  
77 of storm water through detention and retention basins may increase the risk of groundwater  
78 contamination, especially in areas where the soil is sandy and the water table is shallow, and  
79 contaminants may not have a chance to degrade or sorb onto soil particles before reaching the  
80 saturated zone (Fischer et al., 2003; Brattebo and Booth, 2003). The 'first flush' is more polluted  
81 than the remainder due to the washout of deposited pollutants by rainfall (Deletic, 1998;  
82 Bertrand-Krajewski et al., 1998). This has to be considered in the management and treatment of  
83 urban stormwater runoff especially through detention/retention basins (Goonetillekea et al.,  
84 2005). Similarly, all runoff from manufacturing industrial areas should be diverted away from  
85 infiltration devices because of their relatively high concentrations of soluble toxicants (Pitt et al.,  
86 1999). All other runoff should include pretreatment using sedimentation processes before

87 infiltration, to both minimize groundwater contamination and to prolong the life of the  
88 infiltration device (if needed). This pretreatment can take the form of grass filters, sediment  
89 sumps, wet detention ponds, etc., depending on the runoff volume to be treated and other site  
90 specific factors (Pitt et al., 1999).

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

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

115 Modified rational method was applied by Akan (2002) to size infiltration basins and  
116 trenches to control storm water runoff, while the same method was used by Froehlich (1994) to

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

132 The literature review shows that urbanization of a watershed with its associated impact  
133 on the quantity and quality of storm-water runoff has resulted in the implementation of a number  
134 of alternatives for storm-water management. Infiltration trenches are one of them. An infiltration  
135 trench is an underground-storage zone filled with clean gravel or stone (Fig 1). Infiltration  
136 trenches are constructed to temporarily store storm runoff and let it percolate into the underlying  
137 soil. Such trenches are used for small drainage areas. They are typically used for control of  
138 runoff from residential lots, commercial areas, parking lots, and open spaces like ring roads.  
139 Also, they are relatively easy to construct in the perimeters and other unutilized areas of a  
140 development site. Moreover, they can be used below the porous pavements or with grass swales  
141 (Fig 1) and combined with detention basins, etc. Furthermore, they can be provided below  
142 pavements, walkways, pedestrian or cycle tracks so no additional area is required like other  
143 storm water management practices. Infiltration trench emptying time is important to operate and  
144 manage storm water. If the time between two successive storms is less than the trench emptying  
145 time then the excess storm water should be diverted to another detention basin or to the storm  
146 sewer. Unlike detention basins (Emerson and Traver, 2008; Zheng et al., 2006; Guo, 1999; Guo,  
147 2001; Guo and Hughes, 2001; Sample and Heaney, 2006), widely accepted design standards and

148 procedures for infiltration trenches do not exist. The present study finds a solution for the  
149 infiltration rate from a trapezoidal trench and time required to empty the infiltration trench.

## 150 **Analytical Solution**

151 Infiltration trenches are generally long, moderately wide, and shallow in dimensions. They are  
152 filled with coarse gravel to provide storage; they collect runoff from adjacent paved areas and  
153 infiltrate the water into the aquifer beneath. The coarse gravel fill material in the trench is usually  
154 much more permeable than the underlying soil, so there is negligible resistance to flow within  
155 the trench and the perimeter of the trench is an equipotential surface. Let the aquifer be  
156 composed of multi-layer porous medium, such that upper layer has hydraulic conductivity less  
157 than the lower layers. If water table in the aquifer is lower than bottom of the top layer then the  
158 wetting front of infiltrating water from the trench will advance all around and may saturate the  
159 low permeable top layer but seepage flow in lower more pervious layers will be unsaturated. For  
160 example, when seepage from a lined canal takes place, and liner conductivity is much less than  
161 that of the underlying soil medium, the soil medium remains unsaturated (Polubarinova- Kochina  
162 ,1962). In such situations the lower unsaturated layers act as drainage layer to the top saturated  
163 layer and ultimately recharge the aquifer. The position of water table in the aquifer is governed  
164 by horizontal or vertical controls in terms of river, stream or pumping wells present within the  
165 aquifer boundary. Let a trapezoidal trench (as shown in Fig 2) of bed width  $b$  (m), depth of water  
166  $y$  (m), and side slope  $m$  (1 Vertical :  $m$  Horizontal) is constructed in such aquifer and the  
167 saturated hydraulic conductivity of the top layer is  $k$  (m/s). Also, assume the thickness of the top  
168 layer below the bed of the trench is  $d$  (m). As the length of the trench is very large, seepage flow  
169 can be considered 2D in the vertical plane. Initially the top layer is unsaturated and seepage from  
170 the trench is unsteady but after some time the layer will get saturated and steady seepage will  
171 establish.

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

177 
$$q_s = 2k(d + y)K(\sqrt{\gamma/\beta})/K(\sqrt{(\beta - \gamma)/\beta}) \quad (1)$$

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

182 
$$\frac{d + y}{y} = 2K(\sqrt{(\beta - \gamma)/\beta})B(1/2, \sigma) / \sqrt{\beta} \int_{\gamma}^{\beta} \frac{B_{\tau}(1/2, \sigma) d\tau}{\sqrt{\tau(\beta - \tau)(\tau - \gamma)}} \quad (2)$$

183 
$$\frac{b}{y} = 2 \int_{\beta}^1 \frac{(B(1/2, \sigma) - B_{\tau}(1/2, \sigma)) d\tau}{\sqrt{\tau(\tau - \beta)(\tau - \gamma)}} / \int_{\gamma}^{\beta} \frac{B_{\tau}(1/2, \sigma) d\tau}{\sqrt{\tau(\beta - \tau)(\tau - \gamma)}} \quad (3)$$

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

187 
$$B_{\tau}(1/2, \sigma) = 2\sqrt{\tau} {}_2F_1(1/2, 1 - \sigma; 3/2; \tau) \quad (4)$$

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

189 
$${}_2F_1(a, b; c; \tau) = 1 + \frac{a \cdot b}{c} \tau + \frac{a(a+1) \cdot b(b+1)}{c \cdot (c+1) \cdot 1 \cdot 2} \tau^2 + \frac{a(a+1)(a+2)b(b+1)(b+2)}{c \cdot (c+1) \cdot (c+2) \cdot 1 \cdot 2 \cdot 3} \tau^3 + \dots \quad (5)$$

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

197 The trenches are designed to store and to infiltrate a captured volume of runoff that is  
 198 generated from its contributing area during a specific-design storm. The rational formula is used  
 199 for calculating runoff from small catchments, particularly in urban areas where a large portion of



200 the land surface is impervious. The corresponding times of concentration and thus the critical  
 201 rainfall durations will also be small, typically much less than one hour. The filling process begins  
 202 when the runoff first reaches the trench. The filling process can end and the emptying process  
 203 can begin while the trench is still receiving runoff if the rate of infiltration from the trench  
 204 exceeds the inflow rate. However, runoff rates are normally much higher than the infiltration  
 205 rates during a design storm event. Therefore, it is reasonable to assume that the filling process  
 206 will continue until the entire captured runoff has entered the trench. The runoff captured during  
 207 the filling process is stored partly in the trench and partly within the wetted zone of the soil. The  
 208 emptying process starts when the runoff into the trench ceases. The emptying time of the  
 209 infiltration trench is a function of initial volume of water in trench and rate of infiltration from it.  
 210 To find the time to empty the infiltration trench, let the initial water depth in the trench be  $y$  and  
 211 the steady infiltration rate be  $q_s$  and then the water level in the trench will be lowered by  $dy$  in  
 212 small time interval  $dt$  due to steady infiltration. Equating the volume of water in this strip to the  
 213 infiltration volume in the time interval  $dt$

$$214 \quad q_s dt = \eta(b + 2my)dy \quad (6)$$

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

$$218 \quad \Delta t = \frac{\eta}{2k} \int_{y_2}^{y_1} \frac{(b + 2my)K(\sqrt{(\beta - \gamma)/\beta})}{(d + y)K(\sqrt{\gamma/\beta})} dy \quad (7)$$

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

$$221 \quad t = \frac{\eta}{4kB(1/2, \sigma)} \int_0^y \frac{\sqrt{\beta}}{K(\sqrt{\gamma/\beta})} \int_{\gamma}^{\beta} \frac{B_{\tau}(1/2, \sigma)d\tau}{\sqrt{\tau(\beta - \tau)(\tau - \gamma)}} \left( \frac{b}{y} + 2m \right) dy \quad (8)$$

222 Simultaneous solution of Eqs. (2) and (3) for the given trench dimensions ( $b$  and  $\sigma$ ), the  
 223 depth of the unsaturated layer ( $d$ ) and the depth of water in trench ( $y$ ) at particular instant results  
 224 in corresponding parameters  $\beta$  and  $\gamma$ . Thus integrand in Eq. (7) or Eq. (8) for any  $y$  and  
 225 corresponding  $\beta$  and  $\gamma$  for fixed values of  $b$ ,  $\sigma$ , and  $d$  can be computed. However these steps

226 involve complicated integrals with implicit transformation variables. These integrals (complete  
 227 and incomplete beta functions, complete and incomplete elliptical integrals, and remaining  
 228 improper integrals) can be evaluated using numerical integration (Press et al., 1992) after  
 229 converting the improper integrals into proper integrals (Chahar, 2007).

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

$$233 \quad S = \frac{(d + D)\sqrt{\beta}}{K\left(\sqrt{(\beta - \gamma)/\beta}\right)B(1/2, \sigma)} \int_0^{\gamma} \frac{(B(1/2, \sigma) - B_{\tau}(1/2, \sigma))d\tau}{\sqrt{\tau(\beta - \tau)(\gamma - \tau)}} \quad (9)$$

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

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

$$241 \quad \frac{d + y}{y} = \pi K\left(\sqrt{(\beta - \gamma)/\beta}\right) / \sqrt{\beta} \int_{\gamma}^{\beta} \frac{\tan^{-1} \sqrt{\tau/(1 - \tau)}}{\sqrt{\tau(\beta - \tau)(\tau - \gamma)}} d\tau \quad (10)$$

$$242 \quad \frac{b}{y} = \int_{\beta}^1 \frac{\pi - 2 \tan^{-1} \sqrt{\tau/(1 - \tau)}}{\sqrt{\tau(\tau - \beta)(\tau - \gamma)}} d\tau / \int_{\gamma}^{\beta} \frac{\tan^{-1} \sqrt{\tau/(1 - \tau)}}{\sqrt{\tau(\beta - \tau)(\tau - \gamma)}} d\tau \quad (11)$$

$$243 \quad t = \frac{\eta b \int_0^{\gamma} \frac{K\left(\sqrt{(\beta - \gamma)/\beta}\right)}{(d + y)K\left(\sqrt{\gamma/\beta}\right)} dy = \frac{\eta}{2\pi k} \int_0^{\gamma} \left( \frac{b}{y} \frac{\sqrt{\beta}}{K\left(\sqrt{\gamma/\beta}\right)} \int_{\gamma}^{\beta} \frac{\tan^{-1} \sqrt{\tau/(1 - \tau)}}{\sqrt{\tau(\beta - \tau)(\tau - \gamma)}} d\tau \right) dy \quad (12)$$

$$244 \quad S = \frac{(d + D)\sqrt{\beta}}{\pi K\left(\sqrt{(\beta - \gamma)/\beta}\right)} \int_0^{\gamma} \frac{\pi - 2 \tan^{-1} \sqrt{\tau/(1 - \tau)}}{\sqrt{\tau(\beta - \tau)(\gamma - \tau)}} d\tau \quad (13)$$

245 Many times the top soil layer may extend up to large depth ( $d/D > b/D + 2m + 5$ ) and  
 246 water table may also lie at large depth then the solution becomes independent of the location of

247 more previous lower layer. For this case  $\gamma \rightarrow 0$  since  $d/y \rightarrow \infty$ . The corresponding relations for  
 248 trapezoidal trenches with  $\gamma = 0$  are

$$249 \quad \frac{b}{y} = 2 \int_{\beta}^1 \frac{(B(1/2, \sigma) - B_{\tau}(1/2, \sigma)) d\tau}{\tau \sqrt{(\tau - \beta)}} \bigg/ \int_0^{\beta} \frac{B_{\tau}(1/2, \sigma) d\tau}{\tau \sqrt{(\beta - \tau)}} \quad (14)$$

$$250 \quad t = \frac{\eta}{2\pi k B(1/2, \sigma)} \int_0^y \left( \sqrt{\beta} \left( 2m + \frac{b}{y} \right) \int_0^{\beta} \frac{B_{\tau}(1/2, \sigma) d\tau}{\tau \sqrt{\beta - \tau}} \right) dy \quad (15)$$

$$251 \quad S = \frac{2\pi D B(1/2, \sigma)}{\sqrt{\beta}} \bigg/ \int_0^{\beta} \frac{B_{\tau}(1/2, \sigma) d\tau}{\tau \sqrt{\beta - \tau}} \quad (16)$$

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

$$254 \quad \frac{b}{y} = \int_{\beta}^1 \frac{\pi - 2 \tan^{-1} \sqrt{\tau/(1-\tau)}}{\tau \sqrt{(\tau - \beta)}} d\tau \bigg/ \int_0^{\beta} \frac{\tan^{-1} \sqrt{\tau/(1-\tau)}}{\tau \sqrt{(\beta - \tau)}} d\tau \quad (17)$$

$$255 \quad t = \frac{\eta}{\pi^2 k} \int_0^y \left( \frac{b}{y} \sqrt{\beta} \int_0^{\beta} \frac{\tan^{-1} \sqrt{\tau/(1-\tau)}}{\tau \sqrt{\beta - \tau}} d\tau \right) dy \quad (18)$$

$$256 \quad S = \pi^2 D \bigg/ \sqrt{\beta} \int_0^{\beta} \frac{\tan^{-1} \sqrt{\tau/(1-\tau)}}{\tau \sqrt{\beta - \tau}} d\tau \quad (19)$$

## 257 **Application on a Case Study**

258 Lyon is the second largest city in France located on the Rhone River which is the third largest  
 259 river in France. The population of the urban area in Lyon is 1.2 Millions and the city area is  
 260 about 500 km<sup>2</sup>. In the past, stormwater was managed through combined storm water sewer  
 261 system in the old part of the city and through separate system in new area and at the periphery.  
 262 With increase in area and population the traditional system became inefficient and  
 263 uneconomical. The authorities are now looking for other best management practices, which are  
 264 more efficient and more adaptable to linear drainage areas like ring roads. Thus the local  
 265 authorities of Grand Lyon adopted alternative storm management practices, such as detention  
 266 and infiltration basins. At present 100 devices (detention and infiltration trenches and basins)

267 exist in the city area, which are managing  $10^6 \text{ m}^3$  of storm water. Studies and experiments on  
268 infiltration trenches (Chocat et al., 1997; Proton, 2008) have demonstrated their performance to  
269 reduce storm water flows. Dechesne et al. (2005) studied long-term evaluation of clogging and  
270 soil pollution in four infiltration basins in Lyon. These basins are 10 to 21 years old and still  
271 have good infiltration capacities. Winiarski et al. (2006) investigated impact of stormwater on  
272 aquifer medium of Django-Reinhardt infiltration basin in Lyon. Goutaland et al. (2007) and  
273 Goutaland et al. (2008) conducted hydrogeophysical study of the same infiltration basin.

274 Lyon is on the banks of the Rhone River below which alluvial deposits underlie. Types of  
275 alluvial deposits in Lyon are glaciofluvial and fluvial deposits and they form good aquifers. The  
276 vadose zone overlying these aquifers plays a dominant role in recharging aquifer and in  
277 contaminant retention mechanism. Sedimentary deposits, constituting aquifers and vadose zones,  
278 are complex, three-dimensional, heterogeneous and commonly anisotropic (Goutaland et al.  
279 2008). Infiltration trenches have been built on real-scale in completely controlled conditions  
280 adjacent to one of the main ring roads in North of Grand Lyon. The drainage basin is limited to a  
281 band along the road and is rather impervious. The contributing area is about 2.2 hectares.  
282 Interception of run-off is achieved with a pipe of 100 mm of diameter which is connected to the  
283 sewer under the road. The discharge into the trenches is regulated with a gate so that they can be  
284 completely isolated from the sewer. In this case the trenches are not connected to the sewer and  
285 are directly supplied with storm runoff. Prior to inlet into the infiltration trenches, storm water is  
286 stored in a detention basin, which is lined with an impervious geomembrane. The volume of the  
287 detention basin is  $60 \text{ m}^3$ . Considering the average of rainfall in this region, the detention basin  
288 can be filled 30 times per year. A pumping system allows controlled supply to the trenches. The  
289 water levels into the trenches are measured with submerged pressure sensors. All these  
290 equipments have been described in (Proton, 2008). The shape of the observation trench is  
291 trapezoidal and it has dimensions as following: depth = 1.0 m; bed width = 0.8 m; and side slopes  
292 = 0.45 (1 Vertical: 0.45 Horizontal). The length of the trench is 12 meters and the refilled  
293 material in the trench provides porosity = 0.25. The soil adjacent to trench has varying hydraulic  
294 conductivity. The top soil layer is underlain by another highly pervious layer at a depth of 10.0 m  
295 and the prevailing water table is about 18.0 m below the ground surface. Observations on water  
296 level vs. time are available at three locations (H1, H2, and H3) from 1986 to 1991 by Essai

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

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

$$303 \quad f(\beta, \gamma) = \left( \frac{d}{y} + 1 - f_1(\sigma, \beta, \gamma) \right)^2 + \left( \frac{b}{y} - f_2(\sigma, \beta, \gamma) \right)^2 \quad (20)$$

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

312 An appropriate value of hydraulic conductivity was not known, so an average of observed  
 313 time to drop water level from 0.6 m to 0.2 m at three locations (H1, H2, and H3) for three years  
 314 (1987, 1989, and 1991) equal to 45 minutes has been used to get equivalent hydraulic  
 315 conductivity, which came out to be  $= 1.7809 \times 10^{-5}$  m/s. With  $k = 1.7809 \times 10^{-5}$  m/s;  $\eta = 0.25$ ;  $b =$   
 316 0.8 m; and  $m = 0.45$  the resulting graphs with different starting water depths (i.e. 0.9 m, 0.75 m,  
 317 0.6 m, 0.45 m, and 0.3 m) are plotted in Fig 3. The computed emptying times have been 173.7  
 318 min, 159.9 min, 145.4 min, 130.0 min, and 113.1 min, respectively. The graphs are asymptote to  
 319 the time axis and thus result into large emptying time for the final small water depths in the  
 320 trench. Had the time taken to empty the last one cm of water depth been not considered, the  
 321 empty times would have been 104.1 min, 90.3 min, 75.8 min, 60.4 min, and 43.4 min,  
 322 respectively. If multiple trenches were adopted, then Eq. (9) or (19) would yield the required  
 323 spacing between trenches = 4.59 m.

324 For the case of rectangular trench, the corresponding graphs are shown in Fig 4. The  
325 emptying times for the rectangular ditch are 147.8 min, 140.1 min, 131.2 min, 120.9 min, and  
326 108.3 min, respectively for the complete emptying case and 78.3 min, 70.6 min, 61.7 min, 51.3  
327 min, and 38.7 min, respectively when the last cm of water depth is not considered. The required  
328 spacing between trenches in this case = 4.15 m. Both graphs show that substantial time is taken  
329 to drain the last 1 cm of water depth. For more effective operation of the trenches, they may be  
330 refilled with water before complete emptying. This will also establish early saturated flow in the  
331 next cycle.

## 332 **Discussions**

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

350 Infiltration trenches are more effective where the soil has adequate hydraulic  
351 conductivity. In most alluvial deposits the soil is stratified. In many cases, highly permeable  
352 layers of sand and gravel underlie the top low permeable layer of finite depth. In those cases the  
353 high conductivity lower layer acts as a free drainage layer for the top seepage layer since all the

354 seeping water received by this layer is insufficient to saturate it. If the stratified medium  
355 comprises more than two layers and the top saturated layer has hydraulic conductivity less than  
356 that of the next layer which is unsaturated then Eq. (1) is still valid irrespective of hydraulic  
357 conductivities and saturation conditions in the remaining lower layers. Thus the boundary  
358 condition assumed in Eq. (1) is likely to be applicable in many field problems. Efficiency of an  
359 infiltration trench decreases with increase in the depth of drainage layer ( $d$ ). For drainage layer  
360 and water table both at large depth, i.e.,  $d/D > b/D + 2m + 5$ , the special case solutions given by  
361 Eqs. (14) – (19) are applicable. If the water table and/or bedrock are at shallow depth, the  
362 infiltration trenches are ineffective and hence should not be used.

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

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

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

## 409 **Conclusions**

410 Infiltration trenches can control quality and quantity of storm water from small urban catchment.  
411 The surface hydrology of the catchment (i.e. runoff volume) determines the size of an infiltration  
412 trench while the hydraulic conductivity of the aquifer governs the emptying time of the trench.  
413 Solutions derived for the steady saturated seepage state can provide a guideline for determining  
414 the required size of trenches and their spacing/numbers. The proposed design is simple enough to



415 obtain the first estimation of the required time to empty the trench. It overestimates emptying  
416 time and hence estimates are on the conservative side to provide a margin-of-safety. The  
417 presented result may assist a stormwater management engineer in the design of infiltration  
418 trenches.

## 419 **Notation**

420 *The following symbols are used in this paper:*

421	$B(., .)$	complete Beta function [dimensionless];
422	$B_i(., .)$	incomplete Beta function [dimensionless];
423	$b$	bed width of trench [m];
424	$D$	full depth of trench [m];
425	$d$	depth of unsaturated medium/aquifer below bed of trench [m];
426	$K(.)$	complete elliptical integral of the first kind [dimensionless];
427	$k$	hydraulic conductivity of top layer [m/s];
428	$m$	side slope of trench (1 Vertical : $m$ Horizontal) [dimensionless];
429	$q_s$	seepage discharge per unit length of trench [m <sup>2</sup> /s];
430	$S$	spacing between adjacent trenches [m];
431	$T$	top width of trench at full depth [m];
432	$t$	trench empty time [s];
433	$y$	water depth in trench [m];
434	$\beta, \gamma$	transformation variables [dimensionless];
435	$\sigma$	$(1/\pi)\cot^{-1} m$ [dimensionless]; and
436	$\tau$	dummy variable [dimensionless].

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577 **Figure Captions**

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

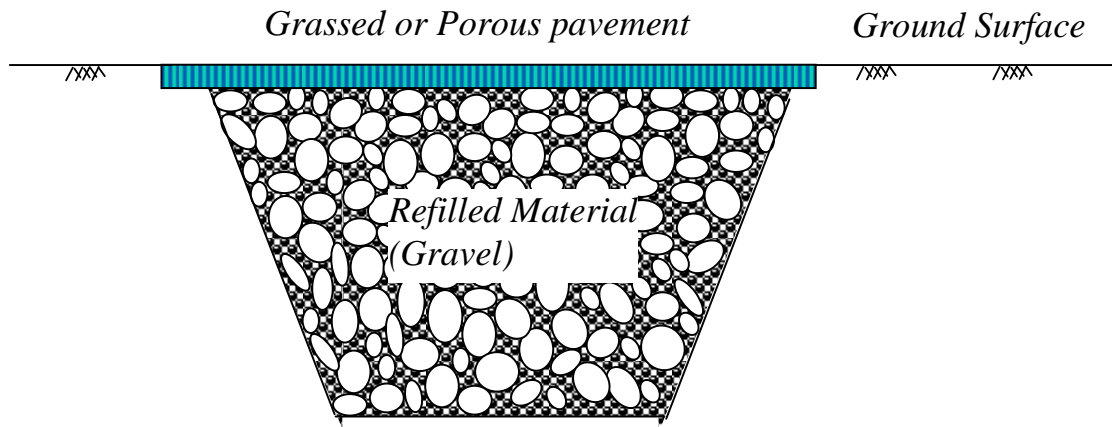
579 Fig. 2. Trapezoidal Infiltration Trenches underlain by an Unsaturated Porous Medium

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

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

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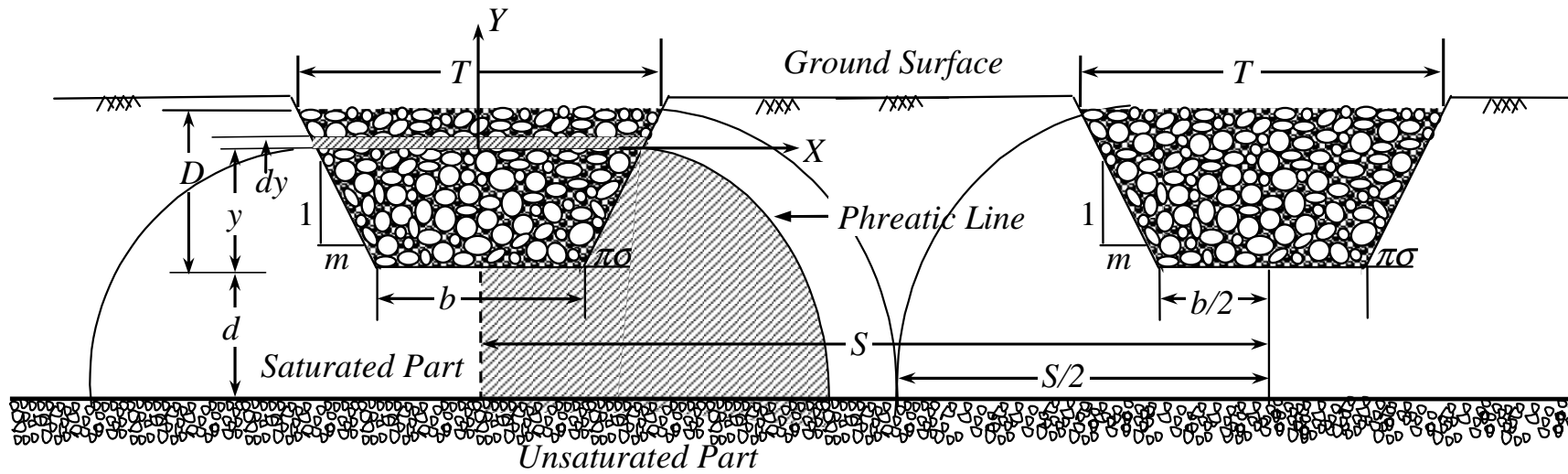
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585

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



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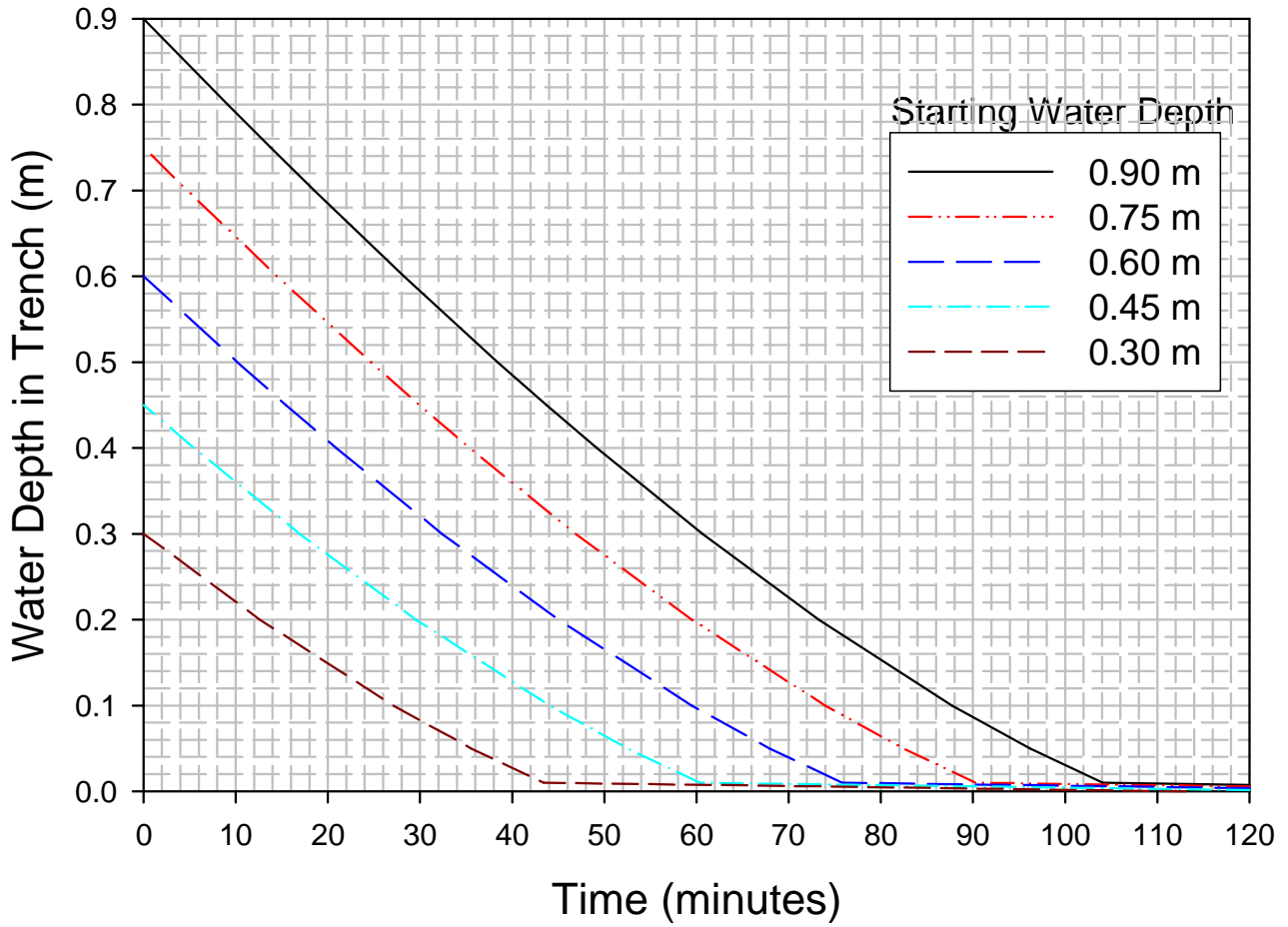


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Fig. 2. Trapezoidal Infiltration Trenches underlain by an Unsaturated Porous Medium

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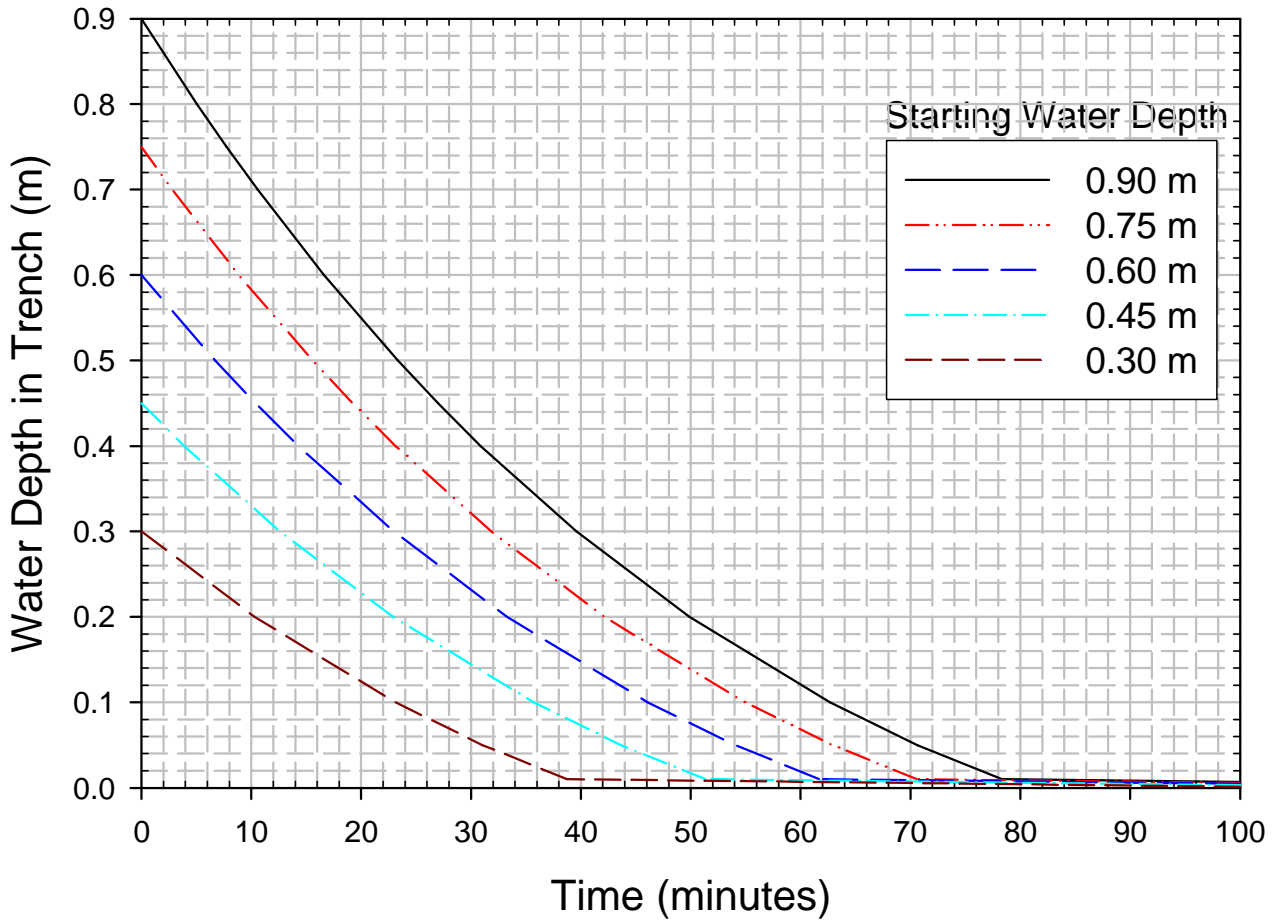


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594 **Fig. 3. Emptying Time for different Starting Depths in a Trapezoidal Infiltration Trench**

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

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