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INFILTRATION TRENCH DESIGN FOR STORM WATER BMP

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Abstract: This study presents a surface-to-subsurface two-dimensional model for the design of infiltration trench basins. It assists engineers to estimate the size of an infiltration basin and to determine the subsurface geometry required for having an adequate hydraulic conductivity. The Federal Aviation Administration's method is revised to predict the detention storage volume by maximizing the volume difference between the design storm runoff and the basin infiltration rate. Diffusion of the wetting front beneath the basin is depicted by the streamline and equal-potential flow net model derived in this study with considerations of non-isotropic permeability. The required subsurface geometry are found to be related to the width of the basin, and the length of the saturation zone is proportional to the ratio of infiltration rate to coefficient of permeability.

Key Words: Sand Basin, Trench, Detention, Infiltration, Permeability.

INTRODUCTION

For a small catchment such as a parking lot, roof top, or depressed area, storm water quality and quantity control can be achieved by infiltration detention basins. Design procedures of an infiltration basin include the surface hydrologic study to size the detention storage volume, and the subsurface hydrologic study to assure the availability of an adequate groundwater conductivity. In this study, the Federal Administration of Aviation (FAA) method is revised to include the storm runoff as the inflow to the basin and the soil infiltration as the basin outflow. The basin storage volume was then achieved by maximizing the volume difference with respect to the design storm duration.

Soil infiltration rates used in the design are often determined by in-situ measurements. However, one of the major concerns in the design of an infiltration basin is whether the design infiltration rate can be sustained by the subsurface hydraulic conductivity and gradients. If the soil permeability is higher than the surface infiltration, vertical percolation of water beneath the basin can recharge aquifers directly. Otherwise, the slow groundwater motion will cause saturation to the surroundings beneath the basin. As a result, the operation of the basin is controlled by the outgoing seepage rate into the groundwater table, not the incoming infiltration rate at the surface. Under this condition, the basin designed with a high infiltration rate becomes undersized. The prolonged drain time may even cause a failure to the operation of the basin. Therefore, it is important to know if the release capacity at the basin site is a surface infiltration control or the soil permeability control. When dealing with a low permeability environment, it is necessary to select a site whose subsurface geometry can provide the required hydraulic depth above the groundwater table so that a sufficient hydraulic gradient can be developed to increase the seepage rate. In the current practice, some general guidelines on the minimum subsurface geometry are available, such as a maximum ponding depth and ponding time (Shaver 1986), and minimum of four feet between the bottom of the basin and the groundwater table (Shaver 1986)(Urbonas and Stahre 1990), they do not provide a realistic basis for estimating the subsurface geometry required to sustain an efficient infiltration operation.

In this study, a two dimensional flownet model was derived to relate the required subsurface geometry to the basin width and the ratio of infiltration rate to the coefficient of permeability. As a design tool, this model is simple enough to obtain the first estimation of the required subsurface geometry for basin site selection, and also detailed enough to provide a quantifiable flow field.

SURFACE MODEL FOR AN INFILTRATION BASIN

To predict the peak runoff from a small urban watershed, the Rational method states:

$$Q_d = CI_dA \qquad (1)$$

$$I_d = \frac{a}{(T_d+b)^n} \qquad (2)$$

in which C = runoff coefficient, A= watershed area in acres (hectare), I_d = rainfall intensity in inch/hr (mm/hr), T_d = rainfall duration in minutes, Q_d= peak runoff rate in cfs (cms) and a, b, and n= constants on the Intensity- Duration- Frequency (IDF) formula. Soil infiltration can be described by the Horton's formula which states

$$f(t) = f_c + (f_o - f_c)e^{-kt} \qquad (3)$$

in which f(t) = infiltration in inch/hr (or mm/hr) at time t after it rains, f_o = initial infiltration rate, f_c = final infiltration rate, and k = decay coefficient in 1/hr. Integration of Eq 3 yields

$$F(t) = f_c t + \frac{(f_o - f_c)}{k}(e^{-kt} - 1) \qquad (4)$$

in which F (t)= accumulative infiltration depth in inch (or mm) at time t. Urbonas and Stahre (1990) stated that the basic concept in the Federal Administration Aviation (FAA) method is to find the maximum volume difference between the inflow and outflow volumes under a series of storm events with different durations. For a specified rain storm, the detention volume is equal to:

$$V_d = \alpha CI_dAT_d - \beta A_bF(T_d) \qquad (5)$$

in which V_d = the required storage volume in cubic feet or cubic meter, the infiltrating area of the basin, A_b, α and β= unit conversion factors. The maximal value of Eq 5 is achieved by setting its first derivative with respect to T_d equal to zero, and it results:

$$\frac{dV_d}{dT_d} = \{\alpha CAa[\frac{-nT_d}{(T_d+b)^{n+1}} + \frac{1}{(T_d+b)^n}] - \beta A_b f(T_d)\} = 0 \quad when \quad T_d = T_m \qquad (6)$$

in which T_m = the design rainfall duration described by Eq 6. Solution of Eq 6 is:

$$T_m = \frac{1}{n}\left[(T_m + b) - (T_m + b)^{n+1}\frac{\beta A_b}{a\alpha CA} f(T_m)\right] \qquad (7)$$

When the value of b in Eq 7 is numerically negligible, the approximate solution of Eq 7 can be:

$$T_m = [\frac{2a\alpha CA(1-n)}{\beta A_b f(T_m)}]^{\frac{1}{n}} \qquad (8)$$

In fact, Eq 8 can also provide the first approximation to the solution of Eq 7 during the trial and error procedure. The maximum detention storage volume, V_m, is:

$$V_m = \alpha CI_mAT_m - \beta A_bF(T_m) \quad at \quad T_d = T_m \qquad (9)$$

The average infiltration rate, f_a, throughout the design storm duration is

$$f_a = \frac{F(T_m)}{T_m} \qquad (10)$$

SUBSURFACE MODEL FOR AN INFILTRATION BASIN

Groundwater movement is driven by the permeability, saturation depth, and hydraulic slope. When the surface infiltration rate is higher than the permeability, the seepage rate may still be kept up by a steeper hydraulic gradient if the subsurface geometry permits. Seepage from ditches and canals has been modeled by Zhukovsky's θ function in 1949 and further illustrated by McWhorter and Sunada in 1977. This model was derived based on the assumption that the local groundwater table did not rise and remained horizontal. As a result, the flow pattern is a downward wetting front which connects the bottom of the canal to the groundwater surface without any horizontal movement. However, due to high and concentrated seepage during the storm duration, an infiltration basin often creates a temporary water mound on the top of the groundwater table. Both vertical downward movement and horizontal dispersion need to be considered in the flow modeling. Without any specific site information, this study adopts a potential flow model around a 90-degree angle to approximate the flownet between the inflow section and outflow section. Because of symmetry to the centerline of the basin, Figure 1 shows a half of the groundwater flow field per unit width of the basin. According to the conformal mapping, the transformation function between the w-plane and the z-plane can be written as

$$w = mz^2 \qquad (11)$$

in which w = uniform flow variables on the w-plane, z = flow variables on the z-plane, and m = the scale factor between these two planes. The flownet of the uniform flow on the w-plane consists of streamlines and equipotentials which can be described by

$$w = \phi + \psi i \qquad (12)$$

And the flow variables on the z-plane is described as

$$z = x + yi \qquad (13)$$

in which ϕ= potential function, ψ= stream function, and x and y = coordinates of a point on the z-plane. Substituting Eq 13 into Eq 12 yields

$$\psi = 2mxy \qquad (14)$$

The scale factor, m, in Eq 14 can be calibrated by a known stream function in the flow field. At Point C on Section AC, as shown in Figure 1, its stream function must be equal to the accumulated infiltration volume rate per unit width of the basin. Thus, we have

$$q_o = \eta f_a B \qquad (15)$$

in which f_a = average infiltration rate over the storm duration, and q_0 = infiltration volume rate per unit width of a slender basin. Stream functions in a two dimensional flow field is to relate flow rates to the x- and y-coordinate system. At Point C, (x,y)=(B,D), its stream function is:

$$\psi_o = 2mBD \qquad (16)$$

in which ψ_o= stream function at point C, and η= dimensional unit conversion factor. Comparing Eq 15 with Eq 16, the value of m is found to be

$$m = \frac{\eta f_a}{2D} \qquad (17)$$

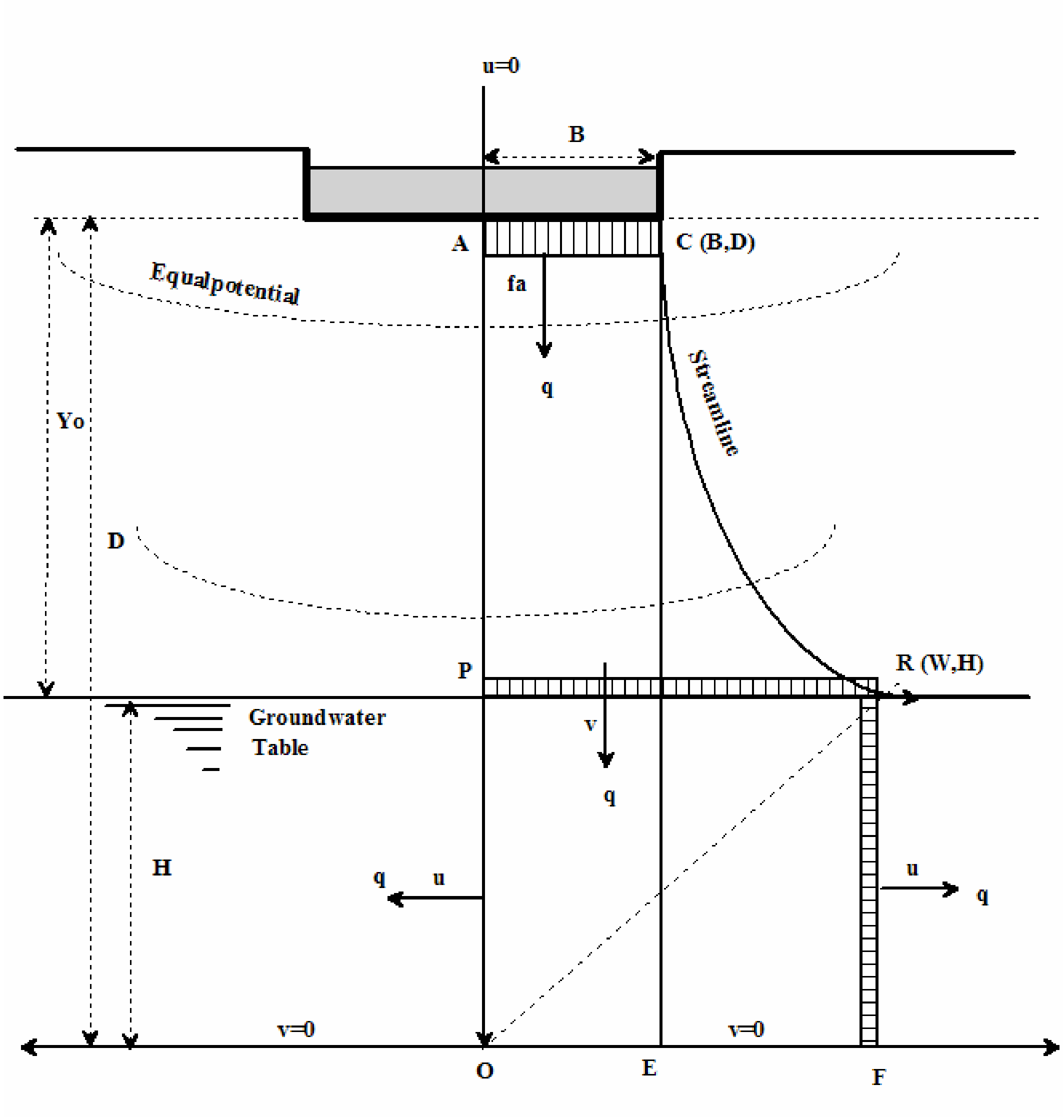

Figure 1 Potential Flow Pattern Underneath Trench

Substituting Eq 17 to Eq 14, the stream function for the flow field is

$$\psi = \frac{\eta f_a}{D}\, x\, y \qquad (18)$$

The velocity components in the flow field can be depicted by the derivatives of Eq 18 with respect to x and y.

$$u = \frac{\eta f_a}{D} x \qquad (19)$$

and

$$v = -\frac{\eta f_a}{D} y \qquad (20)$$

in which u = horizontal velocity component and v= vertical velocity component. Under a long rainfall, the effect of infiltration can produce downward percolation through the surrounding soil. It tends to raise the local groundwater table and reduces the outflow capacity from the basin. To be conservative, this model is further derived under the assumption that all flow sections can hydraulically sustain the infiltration volume provided at the inflow section. Namely, the continuity of flow is maintained.

Therefore, at Section PR in Figure 1, the streamline, ψ_o, that intercepts the groundwater table at Point D, (x,y)=(W,H), must have a stream function of

$$\psi_o = \frac{\eta f_a}{D} HW \qquad (21)$$

Aided by Eq's 16, 17, and 21, the width of the wet front, W, at Section BD, is

$$W = \frac{BD}{H} \qquad (22)$$

According to the Darcy's law, the seepage rate is driven by the hydraulic gradient. As a result, the seepage rate across Section BD is

$$q_o = K_y i W \qquad (23)$$

in which i = hydraulic gradient, and K_y = the coefficient of vertical permeability. The average downward velocity, v, at Section BD is

$$v = \frac{q_o}{W} = K_y i \qquad (24)$$

The hydraulic gradient for the vertical flow through Section BD is unity. Eq 23 with i= -1 (downward) must be consistent with Eq 20 when y=H. Therefore, it is concluded

$$D = \lambda_y H \qquad (25)$$

and

$$\lambda_y = \frac{\eta f_a}{K_y} \qquad (26)$$

At the outflow section, Section DF, the seepage rate is dictated by the hydraulic gradient between Sections CE and DF. Therefore, we have

$$q_o = \psi_o = K_x \frac{(D^2 - H^2)}{2(W-B)} \qquad (27)$$

in which K_x = coefficient of horizontal permeability. The cross sectional average velocity at Section DF is

$$u = \frac{q_o}{H} = \frac{K_x}{2} \frac{H}{B} (\lambda_y + 1) \qquad (28)$$

Similarly, Eq 28 must be consistent with Eq 19 when x=W. Aided by Eq's 22 and 26, the ratio of H/B can be derived from Eq 28 as

$$\frac{H}{B} = \sqrt{\frac{2\lambda_x}{(\lambda_y + 1)}} \qquad (29)$$

and

$$\lambda_x = \frac{\eta f_a}{K_x} \qquad (30)$$

Substituting Eq 29 into Eq 25 yields

$$\frac{D}{B} = \lambda_y \sqrt{\frac{2\lambda_x}{(\lambda_y+1)}} \qquad (31)$$

With the aide of Eq's 29 and 31, the required saturation distance, Y_o, between the bottom of the basin to the groundwater table is

$$\frac{Y_o}{B} = (\lambda_y - 1)\sqrt{\frac{2\lambda_x}{(\lambda_y+1)}} \qquad (32)$$

Of course, for an isotropic case, $\lambda_x = \lambda_y$. It is noted that Eq's 29, 31, and 32 are only applicable to the cases that $\lambda_x \geq 1.0$ and $\lambda_y \geq 1.0$. In other words, the design infiltration rate is greater than or equal to the coefficients of permeability. In fact, when $\lambda_x = \lambda_y = 1$, Eq's 29 and 31 suggest that H=D, or Y_o=0. It is a case of downward recharge to the groundwater table. It is noted that Eq's 29, 31, and 32 are derived for the steady state condition, i.e. surface infiltration rate equal to subsurface seepage rate. Solutions for the isotropic condition are presented in Table 1.

NUMERICAL MODELING AND CASE STUDY

During the infiltration process, water reaches the groundwater table. Warner et el. (1984) indicated that the spread and growth of the mound of recharging water depend on the infiltration rate, dimensions of the basin, and hydraulic characteristics of the earth material. It is expected that at the beginning, the downward steep hydraulic gradients between the basin bottom and the groundwater table result in a higher seepage capacity. Under the mounding effects, the less the gradients are developed, the slower the seepage flow will become. This transient process reaches its steady state when the surface infiltration supply is equal to the subsurface conductivity and is illustrated by the following example.

In the City of Denver the 10-year storm intensity is described by Eq 2 with a=45.92, b=10.0 and n=0.786. A 2.1-acre residential subdivision in the Denver areas has a runoff coefficient of 0.65. The 10-year storm runoff from this watershed will drain into a 180-ft by 20-ft trench basin. The infiltration rates of the basin are: f_o= 6.50 inch/hr, f_c= 1.80 inch/hr, and k = 6.50 /hour. Based on the given information, the unit conversion factors and the bottom area of the basin are:

$\alpha = 60$, $\beta = \frac{1}{12}$, $\eta = \frac{1}{12*3600}$, and $A_b = \frac{100.0*36.0}{43560.0} = 0.083$ acre.

Substituting these variables into Eq's 3 and 7 yields

$$f(T_m) = 1.80 + (6.50 - 1.80)e^{-\frac{T_m}{60*6.5}}$$

$$T_m = \frac{1}{0.786}\left[(T_m+10) - (T_m+10)^{0.786+1}\frac{1/12*0.083*f(T_m)}{45.92*60.0*0.65*2.1}\right]$$

The design storm duration described by these two equations was found to be: Tm=340.0 minute. Using Eq 9, the required detention volume, V_m, for this case is 0.219 acre-ft and the total infiltration depth, $F(T_m)$, calculated by Eq 4 is 4.33 inches. The infiltration volume rate and released from the basin is:

$$Q_o = \eta f_c * (43560 * A_b) = \frac{1}{12*3600} * 1.80 * (0.083 * 43560) = 0.151 \quad cfs$$

The unit flow rate for a half of basin width is:

$$q_o = \frac{1}{12*3600} * 1.80 * 10 = 0.00042 \text{ cfs/ft}$$

If this release rate can be sustained by the subsurface conductivity, then it will take approximately 17.66 hours to drain the basin. With the consideration of the rainfall duration, this process of basin filling and recession takes a total of 23.32 hours.

Assuming that the subsurface media is isotropic, i.e.$K_x=K_y=K$. Table 1 presents a comparison among cases with different coefficients of permeability which can sustain the design infiltration volume rate. When the infiltration rate is slightly greater than the coefficient of permeability, the required saturation depth, Y_o, is shallow, but sharply increases when the infiltration to permeability ratio increases to three. For instance, Table 1 indicates that it requires a saturation depth of 24.49 ft for this trench basin in order to produce sufficient hydraulic gradients when the infiltration to permeability ratio is three.

f_a/K	K	D/B	H/B	Yo/B	D	H	Yo
	fpd				ft	ft	ft
1.10	3.27	1.13	1.02	0.10	11.26	10.24	1.02
1.50	**2.40**	**1.64**	**1.10**	0.55	**16.43**	**10.95**	**5.48**
2.00	1.80	2.31	1.15	1.15	23.09	11.55	11.55
2.50	1.44	2.99	1.20	1.79	29.88	11.95	17.93
3.00	1.20	3.67	1.22	2.45	36.74	12.25	24.49

Table 1 Subsurface Geometry Required for Example Case

A two-dimensional one-layer numerical study was also conducted for the case whose coefficient of permeability is 1.5 times the infiltration rate. The subsurface geometry has a H/B ratio = 1.10 as recommended for $\eta f_a/K = 1.5$ in Table 1. The 180-ft by 20-ft trench basin was placed in the center of a flow field of 500-ft by 60 ft which was discretized into 25 by 20 meshes. Numerical simulations using the MODFLOW model developed by McDonald and Harbaugh in 1984 were performed for the transient flows with various Y_o/B ratios at 1.0, 1.5, 2.0, and 2.25 hours. Table 2 and Figure 2 show that for a specified Y_o/B ratio, the seepage process begins with a higher capacity because of steeper gradients, and then decreases to its steady state which is defined as the infiltration volume rate is equal to the seepage rate. The computer model, MODFLOW, predicts that the steady state for this trench basin can be established after approximately 2.25-hour operation. The sustainable subsurface geometry for the prescribed H/B ratio and infiltration rate requires $Y_o/B=0.52$ at the steady state, and Eq 32 predicts this threshold ratio is 0.55. It can be seen that cases with $Y_o/B<0.55$, the steady seepage rate is less than the design infiltration rate, and with $Y_o/B>0.55$, the steady seepage rate is higher than the design infiltration rate.

Drain Time hours	Yo/B Ratio			
	1.20	1.40	1.64	2.00
1.00	0.18	0.58	1.14	2.14
1.50	0.17	0.54	1.07	2.02
2.00	0.16	0.52	1.02	1.91
2.25	0.16	0.51	**1.00**	1.87

Table 2. Seepage Flowrate to Infiltration Flowrate Ratio During the Transient Process under Various Saturation Depths.

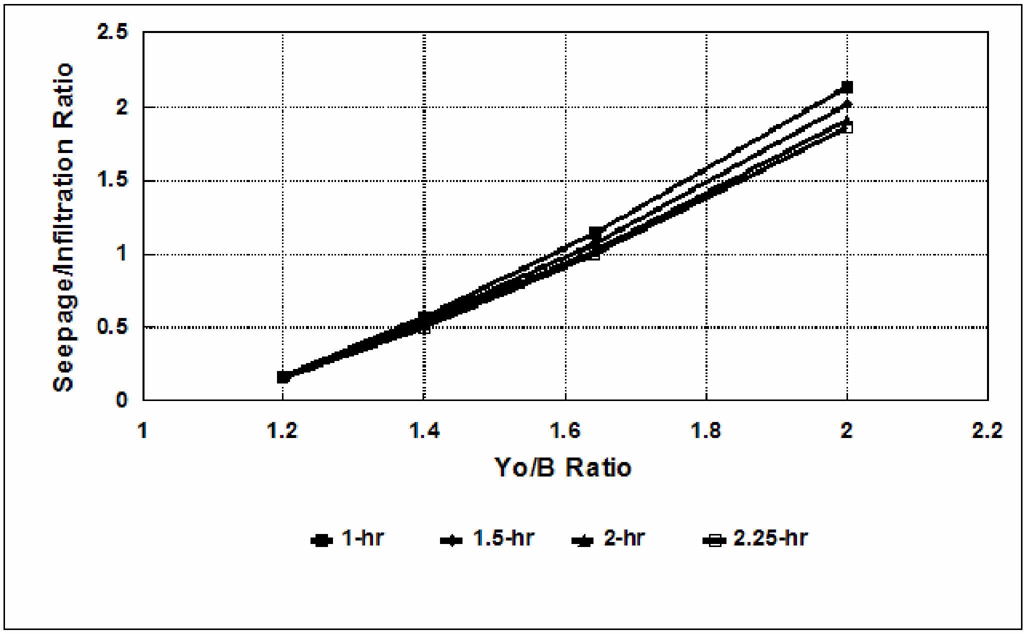

Figure 2 Transient Process for Seepage to Infiltrating Rate Ratios

To investigate the applicability of this two-dimensional approach to a three dimensional operation, this 20-ft by 180-ft trench basin is converted to a 60-ft by 60-ft square basin with the same area. The performance of such a 60-ft x 60-ft square basin was then studied by the MODFLOW model. Numerical solutions indicate that the steady flow for this situation can be established after approximately 24-hour operation with the same threshold Y_o/B ratio of 0.55 as predicted by Eq 32. However, the steady seepage rate for this square basin is equal to the unit release around the basin bottom perimeter, which is only a half of the unit discharge per linear foot of the basin determined by Eq 15. This difference is caused by the fact that the two-dimensional approach confines the unit discharge on a vertical plane without considering lateral expansion effects. It results in an overestimation of the required subsurface saturation depth for a three-dimensional infiltration operation.

CONCLUSIONS

(1) The release capacity of an infiltration system is dictated by either surface infiltration rate or subsurface hydraulic conductivity, whichever is smaller. This study presents a surface-to-subsurface two-dimensional hydrologic model for designing trench infiltration basins. The FAA method is revised to identify the design storm duration and to predict the required storm water storage volume. Solutions derived for the steady seepage patterns under the basin can provide a guideline for determining the required subsurface geometry for the steady state flow condition.

(2) When the coefficient of permeability is higher than the design infiltration rate, it is a case of direct recharge to aquifers. Otherwise, a saturation depth is required to increase the hydraulic conductivity. As described in Eq 32, such a saturation depth begins with zero when the ratio of infiltration to permeability is one, and then increases with respect to the increase of the ratio. When the ratio is greater than three, the required saturation depth drastically increases.

(3) It is understood that groundwater flows are viscous in nature. The subsurface flow model derived in this study is for an ideal potential flow. Although this model is calibrated by the boundary conditions specified by the design infiltration rates at the bottom of the basin, Darcy's law of flow through a porous media, continuity principle in terms of the Laplace equation of stream function, some discrepancies from the viscous flow patterns are expected. Care shall be taken when applying this steady state solution to such a transient process as the infiltration/seepage flow system. The two-dimensional flow model developed in this study is recommended for trench basins. It

tends to overestimate the required saturated depths for square basins. However, it can serve as a preliminary design tool for basin site selections and comparisons during the alternative study. Of course, a more precise and detailed study shall be performed afterwards.

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