Part VI

Seepage

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Seepage Analysis

This chapter describes the methods of developing and composing the seepage function in the analysis solver, the formulation procedure and the relevant technologies. This chapter will enable the user to become familiar with the applications of the program, problem solving and assessment of the acceptability of the analysis results.

The seepage analysis is largely classified into steady state analysis and transient analysis.

Steady state analysis pertains to a time-independent solution in which internal and external boundary conditions remain constant, and inflow coincides with outflow within the analysis domain. Transient analysis on the other hand may exhibit different inflow and outflow with time even if steady state boundary conditions are imposed.

Underground water seepage takes place if an aquifer with permeable ground is present, and head difference or flux exists at the boundaries of the analysis domain.

Seepage flow takes the course of waterways connected by voids between the soil particles in the ground. The flow is governed by the Darcy's law, which states that the seepage flux passing through the volume of soil in steady state is obtained by the permeability coefficient multiplied by the hydraulic gradient and cross-sectional area. The Darcy's law originated from the state of saturation, but can be extended to unsaturated domains.

Unsaturated domains range from a completely dry state to the state close to full saturation. As the degree of saturation decreases from full saturation, air begins to fill the voids between soil particles where water is absent. When the degree of saturation is significantly low, water drops stick to the surfaces of soil particles.

As the degree of saturation decreases, the pore water pressure gradually develops tension under the influence of surface tension, which results in negative pore pressure also expressed as suction pressure. In most cases, a drop in the degree of saturation increases the suction pressure.

In transient analysis, the boundary conditions inside or outside the ground vary with time.

Transient analysis is different from steady state analysis in that boundary conditions vary with time, and the volumetric water content is required. In addition, when the groundwater level rises or recedes, the water content and porosity in unsaturated zones are required, which closely influence the velocity of the rise or drop in the water level.

When water begins to fill in a reservoir contained by an earth dam, a significant difference exists between the case of a completely dry dam and the case with a certain level of water content as far as time it takes to reach the steady state of seepage inside.

Chapter 1

Seepage Equations

1.1 Flow Rule

SoilWorks uses the Darcy's law to simulate the seepage phenomenon in saturated and unsaturated domains in the ground, expressed by the following equation:

$$q = ki \tag{1.1.1}$$

where,

q : Seepage flux (quantity of water)

k : Permeability coefficient

i : Hydraulic gradient

The Darcy's law was originally derived from saturated soils, but has become known that it could be applied to flows in unsaturated soils through a number of researches. The only difference is that the permeability coefficient of unsaturated soil is not a constant, but the flow varies with changes in water content and indirectly varies with changes in pore water pressure (or pressure water head).

The Darcy's law can be expressed as

$$v = ki \tag{1.1.2}$$

v above is known as the Darcian velocity. When water flows through permeable or porous media, the actual average velocity can be calculated by dividing the Darcian velocity by the porosity of soil. Since the porosity of soil is less than 1, the actual average velocity is always faster than the Darcian velocity.

1.2 Governing Equations

SoilWorks uses the following governing differential equation for seepage analysis:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right)+\frac{\partial}{\partial z}\left(k_z \frac{\partial H}{\partial z}\right)+Q=\frac{\partial \Theta}{\partial t} \qquad (1.2.1)$$

where,

H : Total water head

k_x : Permeability coefficient in x direction

k_y : Permeability coefficient in y direction

k_z : Permeability coefficient in z direction

Q : Influx/outflux per unit volume of soil per unit time

Θ : Volumetric water content

t : Time

The above equation shows that the change in volumetric water content equals the change in inflow and outflow flux through an infinitesimal volume at a specific location during a specific time. Simply put, the rate of flow change in the x, y, and z directions plus externally applied flux is the same as the rate of change in volumetric water content with respect to time.

The governing equation above represents the seepage equation for transient flow. Since the magnitude of flux of inflow and outflow through an infinitesimal volume is constant irrespective of time in the steady state flow, the governing equation of Eq. (1.2.2) is obtained by equating the right side of Eq. (1.2.1) to zero.

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right)+\frac{\partial}{\partial z}\left(k_z \frac{\partial H}{\partial z}\right)+Q=0 \qquad (1.2.2)$$

The change in volumetric water content depends on the change in stress state and soil properties.

The stress states of saturated and unsaturated conditions are expressed in two state

variables. Those are $(\sigma - p_a)$ and $(p_a - p_w)$, where σ is the total stress, p_a is the pore air pressure, and p_w is the pore water pressure.

The seepage analysis in SoilWorks consists of the condition of constant total stress. That is, loading and unloading on soil do not take place. The pore air pressure is constant under the atmospheric pressure in the process of flow at an unsteady state since unloading on soil is absent. That is, $(\sigma - p_a)$ is constant, and the analysis has no influence over the change in volumetric water content at all. Any change in volumetric water content is thus only dependent on the change in the stress state, $(p_a - p_w)$. Since p_a is constant, the change in volumetric water content becomes a function of change in pore water pressure.

The change in volumetric water content is related to the change in pore water pressure, which depends on the soil properties, and the stress state. The relationship can be expressed as,

$$\partial\Theta = m_w \partial u_w \tag{1.2.3}$$

where,

m_w : Slope of water content curve (1/m)

u_w : Pressure head (m)

Also, the total head is expressed as the sum of pressure head and potential head as shown in Eq. (1.2.4):

$$H = \frac{p_w}{\gamma_w} + y \tag{1.2.4}$$

where,

H : Total head

p_w : Pore water pressure

γ_w : Unit weight of water

y : Elevation of surface

Eq. (1.2.4) is rewritten as,

$$u_w = (H - y) \tag{1.2.5}$$

Substituting Eq. (1.2.5) into Eq. (1.2.3), the following is obtained:

$$\partial\Theta = m_w \partial(H - y) \tag{1.2.6}$$

Substituting Eq. (1.2.6) into Eq. (1.2.1) yields the following differential equation:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial H}{\partial z}\right) + Q = m_w \frac{\partial(H - y)}{\partial t} \tag{1.2.7}$$

If the surface elevation is constant, the derivative of y with respect to time is eliminated becoming the following equation:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right) + \frac{\partial}{\partial y}\left(k_y \frac{\partial H}{\partial y}\right) + \frac{\partial}{\partial z}\left(k_z \frac{\partial H}{\partial z}\right) + Q = m_w \frac{\partial H}{\partial t} \tag{1.2.8}$$

The volumetric water content, Θ , can be defined as the ratio of the volume of water to the total volume, which can be rewritten as a function of the degree of saturation and effective porosity.

$$\begin{aligned} S_w n &= \frac{V_w}{V_v}\frac{V_v}{V} \\ &= \frac{V_w}{V} = \Theta \end{aligned} \tag{1.2.9}$$

where,

S_w : Degree of saturation

n : Effective porosity

V_v : Volume of air

The differentiation on the right side of Eq. (1.2.1) is the derivative of volumetric water content with respect to time, which cannot be handled like the derivatives of water head on the left side. Rewriting the right side as the derivative of water head,

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+Q=\frac{\partial}{\partial H}\left(S_w n\right)\frac{\partial H}{\partial t} \tag{1.2.10}$$

Rearranging the coefficient term on the right side of Eq. (1.2.10),

$$\frac{\partial}{\partial H}\left(S_w n\right)=S_w \frac{\partial n}{\partial H}+n\frac{\partial S_w}{\partial H} \tag{1.2.11}$$

The first term on the right side of Eq. (1.2.11) represents the change in pores due to the change in water head in saturated soils and the inflow/outflow caused by the change. Such changes are not considered in an unsaturated state. Therefore, only the saturated state of $S_w = 1$ is considered, it can be rewritten in terms of specific storage coefficient.

$$\begin{aligned}\frac{dn}{dH} &= \frac{dV_w}{dh} \\ &= \rho g\left(\alpha + nc_w\right) = S_s\end{aligned} \tag{1.2.12}$$

where,

α : Compressibility of aquifer

c_w : Compressibility of water

The second term on the right side of Eq. (1.2.11) represents the change in the degree of saturation relative to water head. Applying the relationship of Eq. (1.2.9) results in,

$$n\frac{\partial S_w}{\partial H}=\frac{\partial}{\partial H}\left(nS_w\right)=\frac{\partial \Theta}{\partial H} \tag{1.2.13}$$

Therefore, substituting Eq. (1.2.11) and Eq. (1.2.13) into Eq. (1.2.1), the following equation can be derived, expanding to consider the specific storage coefficient and water content:

$$\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+\frac{\partial}{\partial x}\left(k_x \frac{\partial H}{\partial x}\right)+Q=\left(\beta S_s+\frac{\partial \Theta}{\partial H}\right)\frac{\partial H}{\partial t} \qquad (1.2.14)$$

Finite Element Formulation

2.1 Finite Element Formulation

The finite element formulations for the planar elements used in seepage analysis are basically identical to the formulation for the plane strain elements. The difference is that the planar elements in seepage analysis use the nodal water heads as the degrees of freedom to represent the flow, whereas the plane strain elements use the nodal displacements as the degrees of freedom to represent the vertical and lateral behaviors.

The element types that SoilWorks supports for seepage analysis are 3-node, 4-node, 6-node and 8-node elements like the plane strain elements. The shape function of each element is as follows:

■ 3-Node triangular plane element

$N_1 = 1 - \xi - \eta$, $N_2 = \xi$, $N_3 = \eta$

■ 4-Node quadrilateral plane element

$N_1 = \frac{1}{4}(1-\xi)(1-\eta)$, $N_2 = \frac{1}{4}(1+\xi)(1-\eta)$, $N_3 = \frac{1}{4}(1+\xi)(1+\eta)$,

$N_4 = \frac{1}{4}(1+\xi)(1-\eta)$

■ 6-Node triangular plane element

$N_1 = (1-\xi-\eta)(1-2\xi-2\eta)$, $N_2 = \xi(2\xi-1)$, $N_3 = \eta(2\eta-1)$

$N_4 = 4\xi(1-\xi-\eta)$, $N_5 = 4\xi\eta$, $N_6 = 4\eta(1-\xi-\eta)$

■ 8-Node quadrilateral plane element

$N_1 = \frac{1}{4}(1-\xi)(1-\eta)(-1-\xi-\eta)$, $N_2 = \frac{1}{4}(1+\xi)(1-\eta)(-1+\xi-\eta)$

$N_3 = \frac{1}{4}(1+\xi)(1+\eta)(-1+\xi+\eta)$, $N_4 = \frac{1}{4}(1+\xi)(1-\eta)(-1-\xi+\eta)$

$$N_5 = \frac{1}{2}\left(1-\xi^2\right)\left(1-\eta\right) \quad , \quad N_6 = \frac{1}{2}\left(1+\xi\right)\left(1-\eta^2\right)$$

$$N_7 = \frac{1}{2}\left(1-\xi^2\right)\left(1+\eta\right) \quad , \quad N_8 = \frac{1}{2}\left(1+\xi\right)\left(1-\eta^2\right)$$

The degrees of freedom at each node represent water heads. So the rates of changes in water heads can be obtained by differentiating the shape functions.

$$\mathbf{B}_i = \begin{bmatrix} \frac{\partial N_i}{\partial x} & \frac{\partial N_i}{\partial y} \end{bmatrix}^T \tag{2.1.1}$$

The finite element formulation in arriving at the derivation of the governing differential equation from the application of the weighted residual of the Galerkin's approach is as follows:

$$\int_V \left(\mathbf{B}^T\mathbf{C}\mathbf{B}\right) dV\, \mathbf{H} + \int_V \left(\lambda \mathbf{N}^T\mathbf{N}\right) dV\, \mathbf{H},_t = q\int_A \left(\mathbf{N}^T\right) dA \tag{2.1.2}$$

where,

$\mathbf{B}$	: Hydraulic gradient matrix
$\mathbf{C}$	: Permeability coefficient matrix of elements
$\mathbf{H}$	: Nodal water head vector
$\mathbf{N}$	: Interpolation function vector
q	: Unit water quantity of element edge
λ	: $\left(= m_w \gamma_w\right)$ Storage term of transient seepage
$\mathbf{H},_t$	: $\left(= \frac{\partial \mathbf{H}}{\partial t}\right)$ Change in water head with time

For two-dimensional analysis, SoilWorks considers a uniform thickness for all the elements. The finite element formulation can be then expressed as,

$$b\int_A \left(\mathbf{B}^T\mathbf{C}\mathbf{B}\right) dA\mathbf{H} + b\int_A \left(\lambda \mathbf{N}^T\mathbf{N}\right) dA\mathbf{H},_t = qb\int_L \mathbf{N}^T dL \tag{2.1.3}$$

b is the element thickness. If the thickness is uniform over the entire area, the entire

area can be integrated, which then becomes integration along the length from one node to another node.

The finite element formulation can be expressed in a simplified form as,

$$\mathbf{KH} + \mathbf{MH}_{,t} = \mathbf{Q} \tag{2.1.4}$$

where,

$\mathbf{K} = b\int_A \left(\mathbf{B}^T\mathbf{CB}\right)dA$: Element characteristic matrix

$\mathbf{M} = b\int_A \left(\lambda\mathbf{N}^T\mathbf{N}\right)dA$: Mass matrix

$\mathbf{Q} = qb\int_L \mathbf{N}^T dL$: Applied flux vector

Eq (2.1.4) represents a general form of finite element formulation for transient seepage analysis. In case of steady state seepage analysis, the equation can be simplified. Since water head is not a function of time, the $\mathbf{MH}_{,t}$ term is eliminated.

$$\mathbf{KH} = \mathbf{Q} \tag{2.1.5}$$

2.2 Time Integration

The finite element solution of transient seepage analysis is a function of time as shown in the $\mathbf{H}_{,t}$ term of Eq. (2.2.1).

If the water head $\mathbf{H}_0$ at the time t_i and the water head $\mathbf{H}_1$ at the time t_{i+1} are assumed to satisfy Eq. (2.1.4) individually, the following equations are arrived:

$$\mathbf{K}\mathbf{H}_0 + \mathbf{M}\mathbf{H}_{0,t} = \mathbf{Q}_0 \tag{2.2.1a}$$

$$\mathbf{K}\mathbf{H}_1 + \mathbf{M}\mathbf{H}_{1,t} = \mathbf{Q}_1 \tag{2.2.1b}$$

Weighted averaging Eq. (2.2.1) with ω and $1-\omega$, Eq. (2.2.2) is derived.

$$\mathbf{K}\left[\omega\mathbf{H}_1 + (1-\omega)\mathbf{H}_0\right] + \mathbf{M}\left[\omega\mathbf{H}_{1,t} + (1-\omega)\mathbf{H}_{0,t}\right] = \omega\mathbf{Q}_1 + (1-\omega)\mathbf{Q}_0 \tag{2.2.2}$$

Applying Eq. (2.2.3), which expresses the change in water head with the change in time, to Eq. (2.2.2), Eq. (2.2.4) can be written, which is expressed in terms of $\mathbf{H}_0$ and $\mathbf{H}_1$.

$$\omega\mathbf{H}_{1,t} + (1-\omega)\mathbf{H}_{0,t} = \frac{\mathbf{H}_1 - \mathbf{H}_0}{\Delta t} \tag{2.2.3}$$

$$(\omega\Delta t\mathbf{K} + \mathbf{M})\mathbf{H}_1 = \Delta t\left\{(1-\omega)\mathbf{Q}_0 + \omega\mathbf{Q}_1\right\} + \left(\mathbf{M} - (1-\omega)\Delta t\mathbf{K}\right)\mathbf{H}_0 \tag{2.2.4}$$

where,

Δt	: Time increment,
ω	: Ratio between 0 and 1,
$\mathbf{H}_1$	: Water head at the end of time increment
$\mathbf{H}_0$	: Water head at the beginning of time increment
$\mathbf{Q}_1$	: Nodal flux at the end of time increment
$\mathbf{Q}_0$	: Nodal flux at the beginning of time increment
$\mathbf{K}$	: Element characteristic matrix
$\mathbf{M}$	: Element mass matrix

Eq. (2.2.4) can be classified into a number of different methods depending on the applied integral variable ω.

$\omega = 0.0$: Forward finite difference method
$\omega = 0.5$: Crank-Nicolson's method
$\omega = 2/3$: Galerkin's method
$\omega = 1.0$: Backward finite difference method
$\omega = 0.0$: Forward difference method
$\omega = 0.5$: Crank-Nicolson's method
$\omega = 2/3$: Galerkin's method
$\omega = 1.0$: Backward difference method

All the methods except for the forward difference method are numerically stable irrespective of time increments. The Crank-Nicolson's method is widely used in numerical analysis as it retains the advantage of second order convergence. However, oscillation may occur depending on the time increment, so the magnitude of which needs to be carefully selected. The backward difference method shows a relatively greater error compared to the Crank-Nicolson's method, but it does not oscillate and gives a stable solution. SoilWorks adopts the backward difference method using $\omega = 1.0$.

Since ω is 1.0 in the finite element formulation for the transient analysis,

$$(\Delta t\mathbf{K} + \mathbf{M})\mathbf{H}_1 = \Delta t\mathbf{Q}_1 + \mathbf{M}\mathbf{H}_0 \tag{2.2.5}$$

In order to determine the water head at the final stage of a time increment, the water head at the beginning of the time increment must be known. The initial condition must be known for transient state flow analysis.

In SoilWorks, the results of steady state flow analysis are used as the initial condition for transient state flow analysis.

2.3 Analysis Results

Using the water head at each node obtained through seepage analysis, the hydraulic gradient and the Darcian velocity are calculated for each element, which are then converted to nodal results. In addition, when the analysis considers the variation of permeability coefficient and volumetric water content by specifying the unsaturated soil properties, changes in such parameters can be also checked.

$$\begin{Bmatrix} i_x \\ i_y \end{Bmatrix} = \mathbf{TBH} \tag{2.3.1}$$

where,

i_x : Hydraulic gradient in x direction

i_y : Hydraulic gradient in y direction

$\mathbf{T}$: Interpolation matrix that interpolates the results at integration points to those at nodes

$$\begin{Bmatrix} v_x \\ v_y \end{Bmatrix} = \mathbf{TCBH} \tag{2.3.2}$$

where,

v_x : Velocity in x direction

v_y : Velocity in y direction

Chapter 3

Nonlinear Seepage

3.1 Introduction

When unsaturated soil properties are considered or when boundary conditions are nonlinear, the seepage analysis exhibits nonlinearity.

Nonlinear behaviors due to unsaturated soil properties can be caused by changes in permeability coefficient and specific storage coefficient and the coupling of the two material properties. More details are described in the Material manual.

3.2 Boundary Conditions

3.2.1 Seepage Boundary Function

The seepage boundary function assigns changes in boundary conditions with time, which can be also assigned to transient state analysis. Examples of boundary conditions in transient flow are as follows:

- ✓ Annual period of operating an irrigation canal
- ✓ Variation of water level of a reservoir
- ✓ Seasonal inflow and outflow of rainfall from ground surface
- ✓ Change in pumping rate from a well

In case of an irrigation canal used only in the summer, the water level in the canal is low when irrigation starts. As the demand increases, the water level begins to rise to the supply water level. The water slowly discharges from the canal from the time close to the end of irrigation to the winter season.

The boundary function that describes the conditions of transient flow in a canal is shown in Figure 3.2.1.

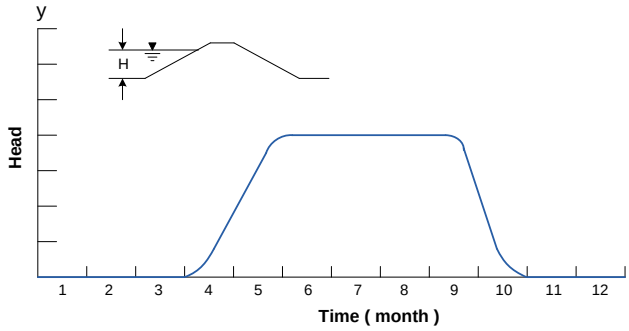

Figure 3.2.1 Boundary function of an irritation canal

3.2.2 Seepage Surface Boundary

Boundary conditions change with the process of flow in some types of seepage analysis. For example, seepage takes place at the downstream face of a homogeneous dam. However, the location of phreatic surface intersecting with the downstream face of the dam is unknown. An iterative analysis process is thus required to assess modified boundary conditions as was done to solve nonlinear finite element equations.

Every node along the boundary to which unknown modified boundary is assigned will be shown as a node to be rechecked for boundary through the user's option ("Re-check boundary based on seepage conditions"). The water head is calculated at every node. If the calculated water head is greater than the potential water head even at one single node of the checked nodes (i.e., if the pore water pressure is greater than zero), the checked nodes are modified.

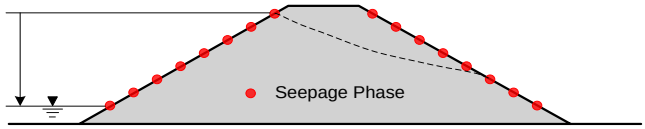

Figure 3.2.2 Typical zones in which nodes are specified for boundary rechecks

There are two ways of specifying the criteria for modifying the nodes to be rechecked. One is based on the maximum pressure and the other by the minimum height for setting the potential seepage face. The minimum height method pertains to selecting the node having the lowest height among the nodes to be rechecked and then setting the water head as the elevation (z coordinate) corresponding to the node of the lowest height for the subsequent iterative calculation. The maximum pressure method pertains to selecting the node having the greatest pressure head among the nodes to be rechecked and then setting the pressure head as the elevation (z coordinate) corresponding to the node of the greatest pressure head for the subsequent iterative calculation .

There is no specific rule as to which method should be chosen. It is recommended that both analysis methods be performed to find an appropriate boundary condition for seepage face based on the expected flow pattern.

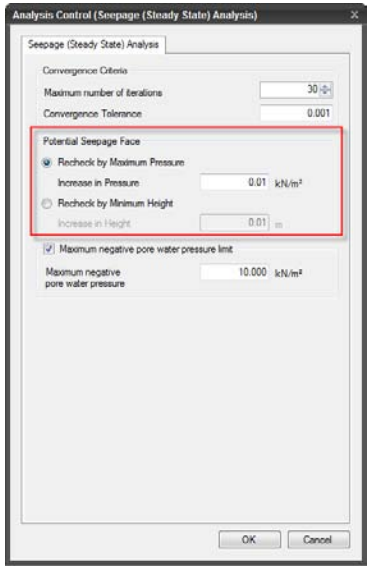

Figure 3.2.3 Methods for specifying potential seepage face

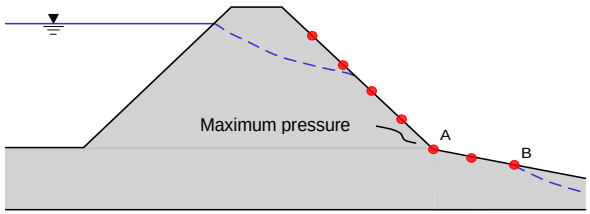

Figure 3.2.4 Possible case for boundary recheck by maximum pressure

Figure 3.2.4 illustrates an example of homogeneous flow with a constant slope. In this case, the phreatic surface must develop starting from the lowest point of the slope. As such, the nodes along the slope surface of the downstream must be rechecked on the basis of the minimum height. In case of a sudden drop in water level, both the slopes of the downstream and the upstream must be rechecked on the basis of the minimum height.

Figure 3.2.4 shows an example of flow with a changing slope at the downstream of a reservoir. Since the upper node A must be modified prior to the lower node B, the nodes along the downstream face must be rechecked by the maximum pressure.

3.2.3 Flux - Water Head Conversion Boundary for Rainfall Analysis (if surface flux > coefficient of permeability, total head = potential head)

In case of rainfall on the ground surface, the rainfall intensity can be used to define the ground surface boundary condition in seepage flow modeling. An inflow in the magnitude of the rainfall intensity is assigned to the ground surface boundary condition. If the ground surface layer is capable of absorbing the rainfall more than the rainfall intensity, all the rainfall infiltrates through the ground layer and the analysis becomes analogous to the actual behavior. However, if the absorption capacity of the ground layer is less than the rainfall intensity, only the amount of rainfall that can infiltrate through the ground layer permeates through the soil and the remainder flows out on the ground surface. If the rainfall intensity is specified as the ground surface boundary condition in partial seepage, the analysis results will be quite different from the actual behavior.

If the rainfall intensity is greater than the absorption capacity of the ground surface, the ground surface boundary exhibits the phenomenon of a water level under the state of saturation during the rainfall. Thus, the boundary condition needs to be changed to a water level condition.

In SoilWorks, as shown in Figure 3.2.5, if the option “If Surface flux > Coefficient of permeability, Total head = Potential head” is selected, the boundary condition of the ground surface is automatically changed from the rainfall intensity inflow condition to the water level condition provided that the rainfall intensity, q is greater than the absorption capacity of the ground, K_{sat} (defined initial coefficient of permeability). This option is useful when a rainfall analysis is performed.

This option can be used for defining surface flux not just in values but also as a function.

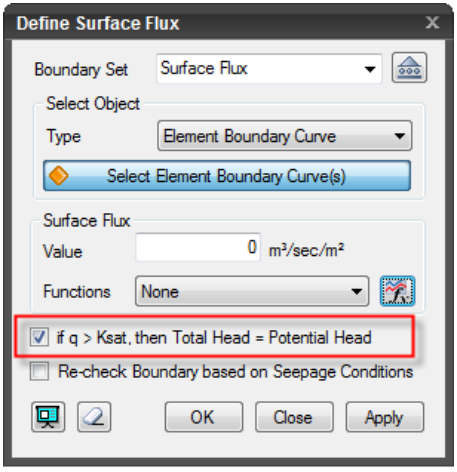

Figure 3.2.5 Change of rainfall intensity boundary condition for ground absorption capability

Note: Caution should be exercised for the following cases in rainfall analysis.
- When impervious pavement such as asphalt, concrete or epoxy is exposed to rainfall, the rainfall boundary must not be used.

- If the water level rises above the ground surface due to a flood, it cannot be accurately simulated.

3.2.4 Flux - Water Head Conversion Boundary for Analysis of Change in Water Level (if total water head < potential water head, flux = 0 is considered)

In order to simulate a drop in the water level of a reservoir or a dam, the pore water pressure needs to be dissipated in the parts where the water level drops under the boundary condition. In SoilWorks, the option “If Total water head < Potential water head, Flux = 0” allows the user to easily model such a situation.

If the water level suddenly changes, suction pressure is generated and as a result the seepage flow can overflow the dam. This option prevents such a phenomenon from

occurring.

This option can be effectively used when there are seasonal variations in rainfall or the water level changes periodically. It is possible to use this option with time varying functions.

If the water flows in the ground due to changes in the water level, this option must be checked on. However, if the water flows out, this option must not be used.