Technical Note

Incorporating Hydromechanical Coupling of Unsaturated Soils into the Analysis of Rainwater-Induced Groundwater Ponding

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Abstract: The moisture contents and coefficients of permeability of partially saturated soils are assumed to follow an exponential function of the pressure head. A partial differential equation is proposed here to describe the groundwater flow in a partially saturated and deformable soil mass based on Darcy's law, conservation of mass law, and elasticity theory. An accurate solution to the one-dimensional (1D) seepage equation was obtained using the Fourier integral transform. This solution was then applied to examine 1D infiltration into deformable soil that caused the groundwater table to rise. The model developed here can be applied to a 1D seepage problem in deformable soil with recharge at the surface and no flux at the base. As rainwater accumulates up from the lower boundary, the coupling effect increases. The thinner the soil layer, the faster the pressure-head profile builds up in coupled conditions ($F < 0$, swelling soils). The rise of the groundwater table is closely correlated to the coupling effect and is also affected by the soil properties, rainfall intensity, and depth of the soil layer. **DOI: 10.1061/(ASCE)GM.1943-5622.0001149.** *© 2018 American Society of Civil Engineers.*

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Introduction

Water commonly infiltrates downward through partially saturated soils, often because of precipitation. As the water infiltrates into the soil, the pore-water pressure head in the soil gradually increases. Infiltration, however, may be influenced by variations in the stress and hydraulic properties of the soil mass. Deformation, such as shrinkage and swelling of partially saturated soils in shallow layers, can occur as a result of wetting and drying cycles. There is considerable interest in water infiltration and precipitation seepage into partially saturated, deformable soils, as they may be important components of practical engineering problems. A typical example is rainfall-induced landslides in expansive soils or in poorly compacted engineering fills. However, the influence of the deformation of the soil has generally been disregarded in conventional studies of infiltration. To fill this knowledge gap, the coupling effects between deformation and seepage in partially saturated soils are presented in this paper.

Strong storms and extreme weather conditions, mainly caused by climate change, are expected to have an increasing effect on the seepage characteristics of soil and groundwater in low-lying areas and on slopes (Schnellmann et al. 2010). Infiltration from precipitation changes the extent of water absorption, resulting in soil deformation. This deformation and variations in groundwater level occur during precipitation seepage. An increase in the groundwater level following a precipitation event is a complicated process influenced by many factors, such as permeability, initial soil-water conditions, initial groundwater table, evapotranspiration, and rainfall intensity (Mansuco et al. 2012; van Gaalen et al. 2013).

The hydraulic and mechanical properties of soil mass may evolve over geological timescales or may rapidly change by engineering developments such as dams and landfills. Partially saturated soils can swell or collapse as rainfall infiltrates, depending on their clay mineral and particle composition (Ichikawa and Selvadurai 2012).

Current analytical methods provide a basis for studying the seepage mechanisms in partially saturated porous media. Infiltration into partially saturated porous media is a topic of great interest because the complicated nonlinear material response is still not fully understood (Garcia et al. 2011). The solutions to the equations that describe unsaturated infiltration can contribute to the assessment and standardization of numerical methods used to solve practical engineering problems. Various methods can be used to examine infiltration problems (Philip 1969; Warrick 1975; Batu 1983; Pullan 1990; Srivastava and Yeh 1991; Warrick and Parkin 1995; Basha 1999, 2000; Parlange et al. 1999; Tartakovsky et al. 1999). Exact solutions to 1D, 2D, and 3D infiltration problems in partially saturated soil systems were obtained based on various mathematical methods (Srivastava and Yeh 1991; Basha 1999, 2000; Chen et al. 2001a, 2001b; Zhan and Ng 2004; Huang and Wu 2012; Zhan et al. 2013; Ng et al. 2015). Using the Laplace transformation technique, Zhan et al. (2013) presented an analytical solution for precipitation seepage into an infinite unsaturated slope that could be applied to either homogenous or layered slopes. Chen et al. (2001a) employed a Fourier integral transform and a 3D linearized Richard's equation to obtain exact solutions for the distribution of the volumetric moisture in partially saturated soils. The seepage process can be modelled using these methods when the

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groundwater table is stationary. However, the above methods generally do not incorporate the effect of hydro-mechanical coupling, even though hydro-mechanical coupling in partially saturated soil is fundamental to many situations, such as infiltration in loose deposits that causes landslides and infiltration into municipal waste.

Rainfall infiltration gives rise to structural changes in the soil. When rainwater infiltrates into a partially saturated soil, the porosity and degree of saturation of the soil both vary. Changes in stress conditions in the soil also lead to variations in the soil's porosity. All these processes influence the fluid seepage through the partially saturated soil. There is a growing interest in methods that couple 1D and 2D seepage with deformation in partially saturated porous media in semi-infinite or finite domains (Wu et al. 2012, 2013, 2017; Wu et al. 2016a, b; Ho and Fatahi 2015, 2016; Ho et al. 2016). The eigenfunction expansion and Laplace transformation are employed to achieve the analytical solution in the case of partially saturated soil consolidation; this solution can capture the variations in excess pore-water pressure and settlement of partially saturated soil due to loading (Ho and Fatahi 2015, 2016; Ho et al. 2016). Analytical solutions to rainfall infiltration into unsaturated soil with a stationary groundwater table are available in the literature (Wu et al. 2012; Wu et al. 2016a, b). The groundwater table may rise due to rainfall infiltration when downward water infiltration into the soil layer is obstructed by an impermeable layer. Such a problem can be solved numerically (Wu and Selvadurai 2016). However, none of the studies mentioned above have presented an analytical solution involving an impermeable layer at the base of the soil mass, with a zero-flux lower boundary. The infiltration model of the slope is characterized by a soil mass as the top layer and impermeable bedrock as the bottom layer; this stratigraphy is commonly found in mountainous regions including red-bedding regions. The impermeable bottom layer destabilizes the slope as it stops water from infiltrating into the deeper layers.

The objective of this study is to propose a conceptual model derived from practical problems and develop a method that can be used to examine the rise of the groundwater table caused by precipitation in a partially saturated, deformable porous medium. Analytical solutions for the hydro-mechanical coupling problem are obtained using a suitable set of transformations. The authors also examine the relationships between the rainfall-induced rise of the groundwater table and the coupling effect, as well as factors that control the rise of the groundwater table.

Coupled Infiltration Equations

The analysis of unsaturated seepage coupled with deformation is based on the following assumptions:

1. The partially saturated porous medium is homogeneous, isotropic, and elastic.
2. The pore water flow in the partially saturated soil follows Darcy's law.
3. The pore air pressure in the soil mass remains constant.
4. Hydraulic conductivity (Raats 1970) and moisture content are an exponential function of the pressure head.
5. The saturated coefficient of permeability is constant despite soil deformation.
6. The hysteresis of the soil–water characteristic curve (SWCC) is not incorporated.

In this study, hydraulic conductivity and moisture content both change exponentially with variations in the pore-water pressure head (Raats 1970; Chen et al. 2001a). This fact is very helpful for the development of analytical methods. Based on the exponential function, Zhan et al. (2013) employed the permeability curve and the SWCC to effectively and accurately predict the behavior of completely decomposed granite (Fredlund and Rahardjo 1993; Zhan et al. 2013). Using Assumption 4, the coupling of the 1D infiltration and the deformation of partially saturated soils can be examined, and the equations that describe the coupling can be linearized and solved.

Based on Darcy's law and mass and momentum conservation laws, the equation that governs 1D hydro-mechanical coupling in partially saturated soils, taking into account rises in the water table, can be given by (Lloret et al. 1987; Kim 2000; Chen et al. 2001a; Wu et al. 2012):

$$\frac{\partial}{\partial z}\left[k\frac{\partial}{\partial z}(h+z)\right] = \left(n\beta_w S_r \gamma_w + n\frac{\partial S_r}{\partial h}\right)\frac{\partial h}{\partial t} - S_r\alpha_c\frac{\partial \varepsilon_z}{\partial t} \tag{1}$$

$$\frac{\partial}{\partial z}\left[E\varepsilon_v - \frac{E}{F}(u_a - u_w)\right] + \left[nS_r\rho_w + (1-n)\rho_s\right]g = 0 \tag{2}$$

where h is the pressure head ($h = u_w/\gamma_w$, $\gamma_w = \rho_w g$); z is the vertical direction; t is the rainfall time; ε_v ($\varepsilon_v = \varepsilon_z$ for the 1D problem) is the total volumetric strain of the soil mass, which is positive during compression and negative during swelling; k is the unsaturated hydraulic conductivity; n is the percentage of voids; S_r is the saturation; (u_a–u_w) is the matric suction; $\alpha_c = 1 - K/K_s$ is the hydro-mechanical coupling coefficient ($0 \le \alpha_c \le 1$); K is the bulk modulus of the solid skeleton, and K_s is the bulk modulus of the solid soil (Wu et al. 2012); E is the elastic modulus of the soil that accounts for variations in the net normal stresses (Wu et al. 2012); F is the elastic modulus of the soil due to variations in the matric suction, assumed to be a function of stress; β_w is the compressibility of the fluid; ρ_w is the density of water; ρ_s is the density of the soil phase; and g is the gravitational acceleration.

Eqs. (1) and (2) govern the coupling of 1D deformation and seepage in partially saturated soils and account for the rise of the groundwater table.

Based on Assumption 3, the derivative of Eq. (2) with respect to t can be written as follows:

$$\frac{\partial \varepsilon_v}{\partial t} = -\frac{\gamma_w}{F}\frac{\partial h}{\partial t} \tag{3}$$

From Eqs. (1) and (3), the following can be derived:

$$\frac{\partial}{\partial z}\left[k\frac{\partial}{\partial z}(h+z)\right] = \left(\beta_w\frac{\partial \rho}{\partial h} + n\frac{dS_r}{dh}\right)\frac{\partial h}{\partial t} + \frac{\gamma_w S_r \alpha_c}{F}\frac{\partial h}{\partial t} \tag{4}$$

Based on Assumption 4, the coefficient of permeability and the moisture content vary exponentially with the pore-water pressure head, that is, $k(h) = k_s e^{\alpha h}$ and $\theta(h) = \theta_s e^{\alpha h}$ ($h < 0, t > 0$). Here, k_s is the saturated hydraulic conductivity, θ_s is the volumetric moisture at saturation, and α is the desaturation coefficient. Because $S_r = \theta(h)/\theta_s$ when the fluid compressibility is neglected ($\beta_w = 0$), Eq. (4) can be written as follows:

$$\frac{\partial}{\partial z}\left[k\frac{\partial}{\partial z}(h+z)\right] = Me^{\alpha h}\frac{\partial h}{\partial t}\ (t > 0) \tag{5}$$

where $M = \theta_s\alpha + \gamma_w\alpha_c/F$.

Solution for Infiltration into Deformable Soils

When the dimensionless variables $Z = \alpha z$ and $T = \alpha^2 k_s t/M$ are introduced into the function $W(Z, T) = e^{\alpha h} \cdot e^{Z/2+T/4}$, Eq. (5) can be rewritten as:

$$\frac{\partial W}{\partial T} = \frac{\partial^2 W}{\partial Z^2} \tag{6}$$

The boundaries comprise a lower and an upper boundary (Fig. 1). In the literature of analytical solutions, the base boundary is usually assumed to coincide with a stationary groundwater table with the pressure head set to zero (Wu et al. 2012). However, in this study, the authors consider a zero flux at the base boundary. The hydraulic boundary condition is given by

$$k\frac{\partial h}{\partial z} + k\bigg|_{z=0} = 0 \tag{7}$$

The top boundary in Fig. 1 is controlled by constant rain intensity (q) at the ground surface, expressed as

$$k\frac{\partial h}{\partial z} + k\bigg|_{z=d} = q(z, t) \tag{8}$$

in which d is the depth in the 1D unsaturated infiltration problem.

The initial condition, assuming a hydrostatic profile and boundary equations can be rewritten in the dimensionless form

$$W(Z, 0) = e^{\alpha h_i + Z/2} \tag{9a}$$

$$\frac{\partial W}{\partial Z} + \frac{W}{2}\bigg|_{Z=0} = 0 \tag{9b}$$

$$\frac{\partial W}{\partial Z} + \frac{W}{2}\bigg|_{Z=D} = -\frac{q}{k_s} e^{T/4} \tag{9c}$$

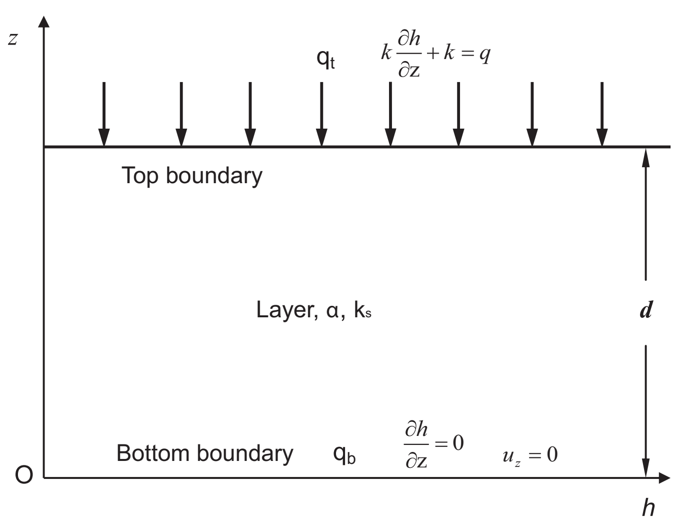

Fig. 1. Top and base flux boundaries for a soil profile with a finite thickness: the soil layer is between $z = 0$ and $z = D$, where z is the vertical coordinate

where D (αd) is the dimensionless length, and h_i is the initial pressure head when the duration (t) is zero.

Based on a Fourier integral transformation (Ozisik 1989), the exact solution to Eq. (6) can be derived as follows:

$$h(Z, T) = \frac{1}{\alpha}\ln\left(e^{-Z/2}\sum_{n=0}^{\infty}\left\{K(\beta_n, Z)\cdot e^{-(\beta_n^2+0.25)T}\left[F(\beta_n) + \int_{t'=0}^{T} e^{\beta_n^2 t'} A(\beta_n, t')dt'\right]\right\}\right) \tag{10}$$

in which

$$K(\beta_n, Z) = \sqrt{2}\left[\frac{\beta_n^2 + 0.25}{D(\beta_n^2 + 0.25) + 0.5}\right]^{1/2}\cos\beta_n Z \tag{11}$$

$$F(\beta_n) = \int_0^D K(\beta_n, z')V(z', 0)\,dz' \tag{12}$$

$$A(\beta_n, t') = \sqrt{2}\left[\frac{\beta_n^2 + 0.25}{D(\beta_n^2 + 0.25) + 0.5}\right]^{1/2} \times\left[e^{\alpha h_0}e^{t'/4} + \frac{q(Z, T)}{k_s}\cos(\beta_n D)e^{L/2+t'/4}\right] \tag{13}$$

where the eigenvalue β_n satisfies $\beta_n \tan(\beta_n D) = 0.5$.

Example

In this study, a 1D homogeneous soil layer was subjected to rainfall seepage. The flux was set to zero at the base boundary, and to a specific rate at the top boundary (Fig. 1). In Fig. 1, the displacement at the base boundary is zero, i.e., $u_z = 0$. The analysis parameters are provided in Table 1 (van Genuchten 1980; Tsai and Wang 2011; Zhan et al. 2013). In this study, the authors incorporated the effect of the dimensionless rainfall intensity (q/k_s), desaturation coefficient (α), and soil layer height on the pressure head distribution, as well as the coupling response.

Influence of Rainfall Intensity

The variations in the dimensionless rainfall intensity over the duration of the rainfall event for both the coupled and uncoupled analyses are shown in Fig. 2, for H = 400 cm. The parameters used were $k_s = 10^{-5}$ m/s, $\theta_s = 0.4$, $|F| = 5 \times 10^3$ kPa, and α =

Table 1. Parameters Used to Analyze 1D Groundwater Seepage

Parameter	Value
Dimensionless rainfall intensity (q/k_s)	1.0
Saturated coefficient of permeability (k_s)	10^{-5} m/s
Desaturation coefficient (α)	0.01, 0.05, and 0.1 cm^{-1}
Volumetric moisture (θ_s)	0.4
Elastic modulus with respect to suction change (F)	$\pm 5 \times 10^3$ and $\pm 10^5$ kPa
Hydro-mechanical coupling coefficient (α_c)	1

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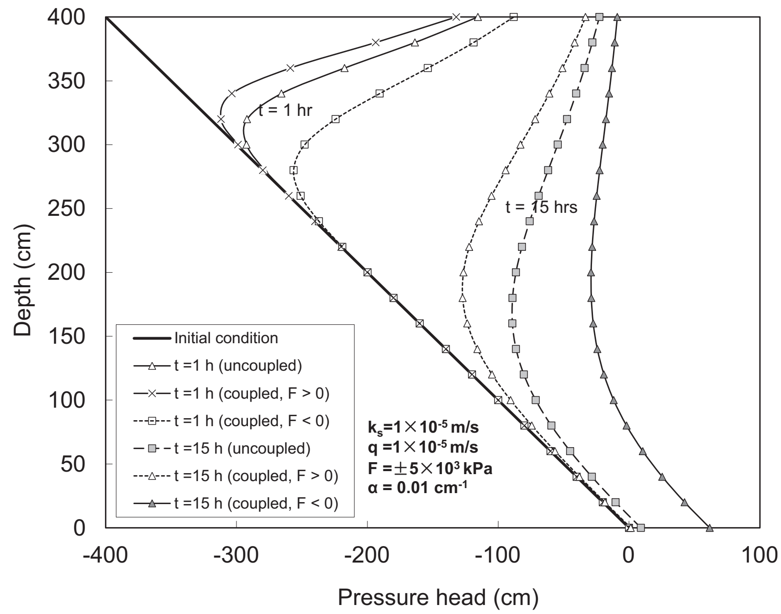

Fig. 2. Changes in the pressure-head profile over time in coupled and uncoupled states

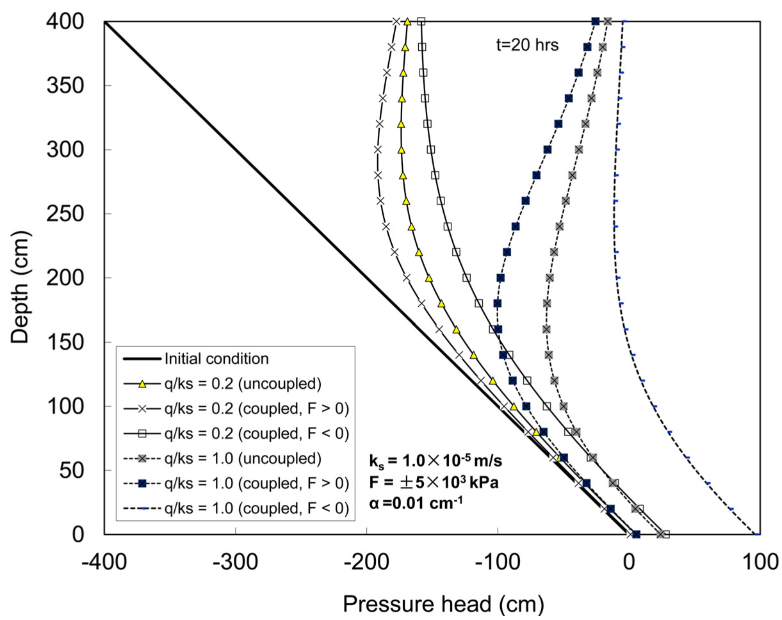

Fig. 3. The effect of dimensionless rainfall intensity on the pressure-head profile in coupled and uncoupled states

0.01 cm^{-1} (van Genuchten 1980; Tsai and Wang 2011; Wu et al. 2012). The value of F can be positive or negative: a negative F means an expansive soil where a suction decrease leads to soil volume increase, while a positive F denotes a collapsible soil where a suction decrease leads to soil volume decrease (Wu and Zhang 2009). When $t = 1$ h, the wetting front moved to a depth of 120 cm in both the uncoupled analysis and coupled analysis with $F > 0$. However, with $F < 0$ the wetting front reached 180 cm in the coupled analysis. When $t = 1$ h, the difference in the pressure head for the coupled ($F < 0$) and uncoupled analyses was 27.5 cm at the upper boundary, and the maximum difference in the pressure head within the partially saturated zone was 75.1 cm at a soil depth of 340 cm. When $t = 15$ h, the difference in the pressure head at the top boundary between the coupled ($F > 0$) and uncoupled analyses was 13.7 cm, and the maximum difference in the pressure head within the soil was 61.8 cm at a depth of 160 cm. The coupling effect became more apparent with increasing time, particularly in the lower part of the soil. Compared with the case where the base boundary coincides with the stationary groundwater table (Wu et al. 2012), the case with zero flux at the base boundary led to more pronounced coupling effects in the lower part of the soil column. It was also noted that groundwater ponding occurred at the base boundary ($t =$ 15 h, Fig. 2), particularly in the case of expansive soil ($F < 0$). For a collapsible soil the soil volume decreased when wetted ($F > 0$), and groundwater ponding did not occur until very late ($t = 28$ h).

The influence of the dimensionless rainfall intensity on the pressure-head profile for the coupled and uncoupled analyses is shown in Fig. 3 for $t = 20$ h. Two values ($q/k_s = 0.2$ and $q/k_s = 1.0$)

were used to obtain the results in Fig. 3; all other parameters were the same as in Fig. 2. As the infiltration time increased and the wetting front moved downward, the pressure head increased rapidly in the shallow depths of the partially saturated soil. Rainfall intensity plays an important role in the advancement of the wetting front, and the pressure-head profile moves more quickly as the rainfall intensity increases. The coupling effect is also closely linked with the rainfall intensity. At $t = 20\,\mathrm{h}$, when $q/k_s = 0.2$, the maximum pressure-head difference between the coupled $(F < 0)$ and uncoupled analysis results was 28.73 cm in the middle of the soil profile. However, at the base boundary with $q/k_s = 1.0$ the maximum change in the pressure head between the coupled $(F < 0)$ and uncoupled analysis results was 71.94 cm. Other studies have reported that the coupling effect is more noticeable in a shallow layer of partially saturated soil (Wu et al. 2012). However, in this study, the coupling effect became more noticeable in the base layer, particularly so with increasing time. When $q/k_s = 1$ and $t = 30\,\mathrm{h}$, the difference in the maximum pressure head between the coupled $(F < 0)$ and uncoupled analyses was 11.43 cm at the top boundary, and the maximum difference in the pressure head was 71.94 cm at the base boundary. As the rainfall accumulated at the base boundary, the coupled effect became more apparent there.

Effect of the Desaturation Coefficient

Fig. 4 illustrates the effect of the desaturation coefficient ($\alpha = 0.02$ and $0.1\,\mathrm{cm}^{-1}$) on the pressure-head distribution in coupled and uncoupled states when $t = 20\,\mathrm{h}$ and $q/k_s = 1$. The desaturation coefficient for clay soils had a significant influence on the pressure-head distribution for both the coupled and uncoupled states (Wu et al. 2012). There was a marked difference in the pressure head in the base layer ($\alpha = 0.02\,\mathrm{cm}^{-1}$) between the coupled and uncoupled analyses. When α was $0.1\,\mathrm{cm}^{-1}$, the difference in the pressure head between the coupled and uncoupled analyses decreased sharply. For large α ($0.1\,\mathrm{cm}^{-1}$) no remarkable difference was observed between the coupled and uncoupled states. The difference in the pressure head increases as α decreased. The pressure-head distribution was similar for collapsible soils ($\alpha = 0.1\,\mathrm{cm}^{-1}$) with or without the coupling effect (Fig. 4).

Effect of F

The elastic modulus of the soil structure for variations in the matric suction is a key parameter when examining seepage into deformable partially saturated porous media (Wu et al. 2012). The effect of F ($F = \pm 5 \times 10^3$ and $\pm 1 \times 10^5\,\mathrm{kPa}$) on the pressure-head profile in coupled and uncoupled conditions is presented in Fig. 5, for $t = 15\,\mathrm{h}$ and $q/k_s = 1$. In the analysis, the desaturation coefficient (α) was set as $0.01\,\mathrm{cm}^{-1}$, and four values of F were used: 10^5, 5×10^3, -5×10^3, and $-10^5\,\mathrm{kPa}$. The other parameters are the same as in Fig. 2. At any given time, the difference in pressure heads between the uncoupled and coupled analyses increased as $|F|$ decreased. Indeed, when $|F| = 1 \times 10^5\,\mathrm{kPa}$, the difference between the coupled and uncoupled analyses was minimal. Thus, the effect of F on the coupling effect was significant. The greater the soil stiffness ($|F|$), the more marked the coupling effect. When $F = -5 \times 10^3\,\mathrm{kPa}$ (i.e., an expansive soil) and $t = 15\,\mathrm{h}$, the negative pore-water pressure head dissipated faster in the coupled analysis. When $F = 5 \times 10^3\,\mathrm{kPa}$ (i.e., loess-type soil) the pressure-head profile for the coupled analysis fell behind that for the uncoupled analysis. With time, the coupling effect may become more apparent in the bottom layer than in the top layer (Fig. 5).

Effect of the Soil Layer Height

The effect of the dimensionless soil layer thickness on the pressure-head profile in uncoupled and coupled analyses is presented in Fig. 6. The following parameters were used: $k_s = 10^{-5}\,\mathrm{m/s}$, $q/k_s = 1$, $\theta_s = 0.4$, $|F| = 5 \times 10^3\,\mathrm{kPa}$, $\alpha = 0.01\,\mathrm{cm}^{-1}$, $t = 10\,\mathrm{h}$. H was taken as 200 and 600 cm. To effectively study the influence of the depth of the soil layer, both the soil depth and the pressure head are dimensionless (dimensionless soil depth = depth/H; dimensionless pressure head = pressure head/P_{max}; $P_{\max}$ is the maximum absolute value of the initial pressure head). In Fig. 6, the wetting front did not reach the base boundary when $H = 6\,\mathrm{m}$ at $t = 10\,\mathrm{h}$, but did when $H = 2\,\mathrm{m}$, showing that the pressure-head profile moved faster as the depth of the soil layer decreased. In Figs. 2–6, water accumulated after a prolonged period of rainfall infiltration in swelling soils. Rainwater-induced groundwater ponding occurred earlier in expansive soils than in "collapsible" soils. Swelling soils easily became

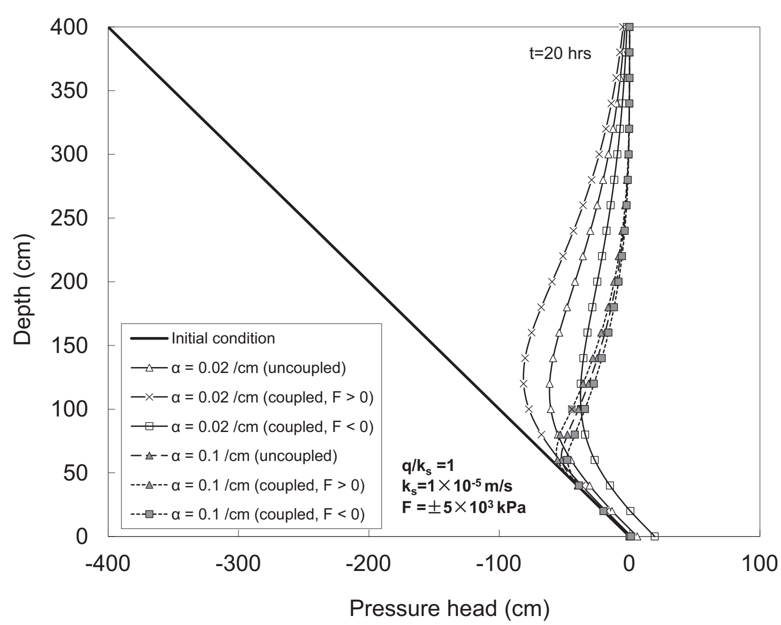

Fig. 4. The effect of α on the pressure-head profile in coupled and uncoupled states

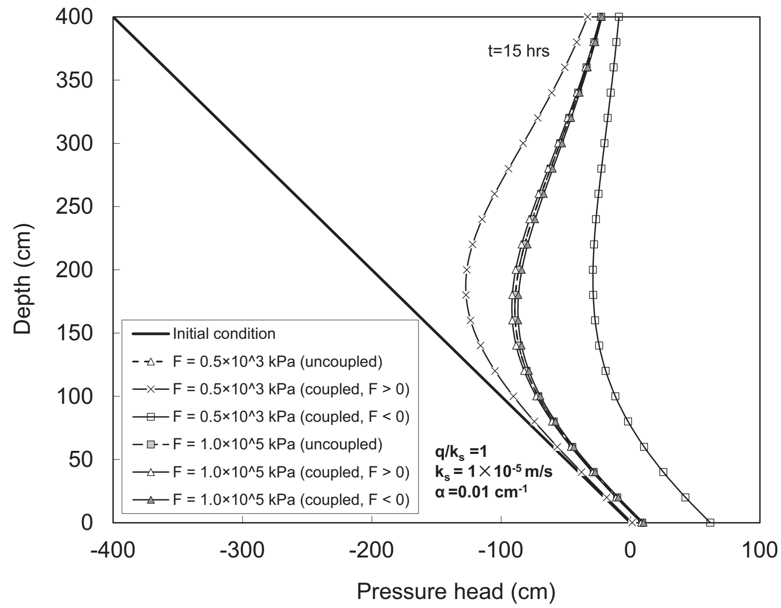

Fig. 5. The effect of F on the pressure-head profile in coupled and uncoupled states

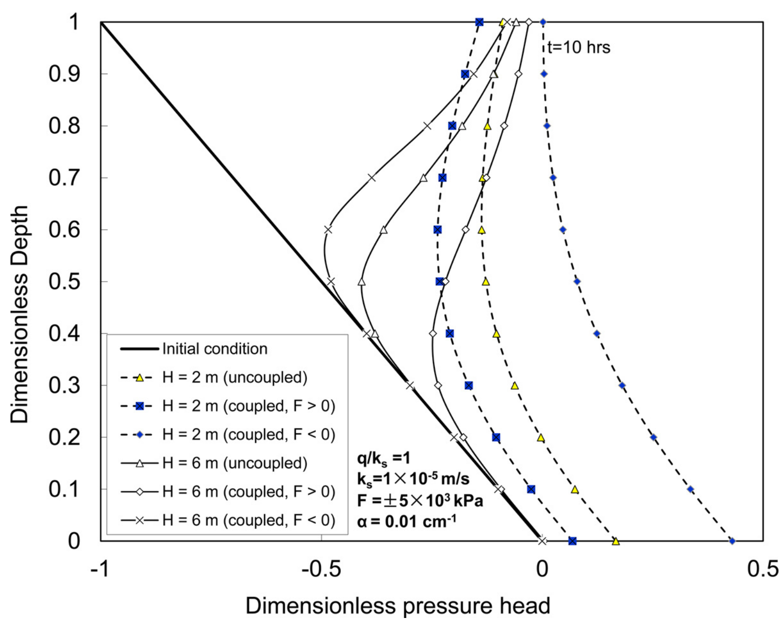

Fig. 6. The influence of the soil layer height on the pressure-head profile in coupled and uncoupled states

saturated earlier than collapsible soils in the same infiltration conditions. Therefore, ponding in expansive soils occurs earlier than in collapsible soils.

Conclusions

The authors developed an analytical solution that describes 1D transient groundwater flow in partially saturated soils using a Fourier integral transform, in which a zero-flux boundary allows the groundwater level to move. The results indicated that the lower boundary condition played an important role in the pressure-head profile and that the coupling effect was more pronounced for a zero-flux boundary than for a zero-pressure-head boundary. The zero-flux base boundary also led to groundwater ponding in the lower part of the soil mass. The ponding occurred much earlier in expansive soils than in collapsible soils. The coupling effect became more noticeable when ponding occurred. The pressure-head profile moved faster as the thickness of the soil layer decreased.

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