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ABSTRACT

An essential feature of transient groundwater movement is the phenomenon of change in groundwater storage. Storage change is governed by three fundamental processes: change in void volume of the skeleton, change in fluid saturation, and change in fluid density. Since water expansivity is very small, it is clear that the phenomenon of change in storage in fully saturated media is dominated by matrix deformation. When the soil is partially saturated it is customarily assumed that the matrix is essentially rigid and desaturation is the only process governing change in storage. While this assumption may be reasonable at relatively low saturations, it may not be valid at high saturations or in the transition zone between the saturated and unsaturated regimes. The key to the understanding of this lies in a proper appreciation of the constitutive relationships that exist between mechanical stresses on the one hand and moisture suction on the other. This paper examines the theoretical as well as the practical consequences of the role of soil deformation in saturated-unsaturated groundwater flow.

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INTRODUCTION

In 1857 Darcy provided the equation of motion for fluid flow in a saturated porous medium, introducing the concept of hydraulic head. Half a century later, in 1907, Buckingham provided the equation of motion for an unsaturated porous medium, through the concept of capillary potential. In 1931, Richards unified the two equations of motion and brought the phenomenon of transient flow in saturated-unsaturated media within the folds of a single governing equation. Over the past decade many numerical models have been proposed (Freeze, 1971; Cooley, 1971; Neuman, 1973) for simulating transient saturated-unsaturated flow, based on Richards' approach.

The fundamental feature which distinguishes transient fluid flow from the steady state case is the phenomenon of change in storage. The principal physical phenomena which govern the change in storage in the saturated zone, on the one hand, and in the unsaturated zone on the other, are basically different; the former is dominated by change in void volume, and the latter by change in saturation. In unifying saturated-unsaturated flow, therefore, it is as essential to consider the mechanism of change in storage as it is to consider the phenomena governing fluid motion. Until recently (Narasimhan and Witherspoon, 1977) very limited attention has been given to a careful treatment of the phenomenon of groundwater storage in unifying saturated-unsaturated flow. To this extent, Richards' equation, as it is commonly used, does not completely unify saturated-unsaturated flow. One reason for this incompleteness might be that soil physicists have generally neglected deformation in the partially saturated soil while hydrogeologists and soil engineers have devoted their main attention to purely saturated flow.

The purpose of this paper is to focus attention on the importance of the storage phenomenon in unifying saturated-unsaturated flow, thereby rendering the unified treatment more complete than it is now. The present work has been motivated by two considerations. The first, philosophical one, is that conceptually a smooth transition is required as one passes from the saturated to the unsaturated region and vice versa. Indeed, under certain boundary conditions, neglecting deformation with the onset of moisture suction can cause the transient equation to break down. The second, on the practical side, is that there is no a priori justificaton to believe that a soil may not deform when undergoing desaturation. It may be that for coarse grained, relatively rigid soils such as sand, deformation can be conveniently neglected. However, in the case of fine sands, silts, clays or organic soils, significant deformation might accompany desaturation, and may not be neglected. Such deformation may be important not only from the point of view of the environment, as in the case of possible subsidence associated with heavy pumpage of unconfined aquifer systems, but also from the consideration of groundwater management, wherein one might want to separate out storages due to void volume and saturation.

STORAGE AND CHANGE IN STORAGE

The mass of fluid stored in a finite volume of a porous medium is given by

$$M_w = V_v \rho_w S \tag{1}$$

From empirical considerations it is well known that V_v, ρ_w, and S are all functions of the average fluid pressure head ψ over the volume element. Hence, $M_w = M_w(\Psi)$. The change in the quantity of fluid mass stored in the volume element, as the average pressure head changes, can be quantified by means of the parameter "fluid mass capacity" (Narasimhan, 1975; Narasimhan and Witherspoon, 1977) defined by

$$M_c = \frac{dM_w}{d\Psi} = \underbrace{\rho_w S \frac{dV_v}{d\Psi}}_{\text{deformation of skeletal matrix}} + \underbrace{V_v S \frac{d\rho_w}{d\Psi}}_{\text{expansion of fluid}} + \underbrace{V_v \rho_w \frac{dS}{d\Psi}}_{\text{desaturation of voids}} \qquad (2)$$

As noted, each term on the right hand side of (2) denotes a distinct physical phenomenon causing change in the stored fluid mass.

In shallow groundwater systems subject to small or moderate changes in Ψ, the expansivity of water, $d\rho_w/d\Psi$, is relatively very small in comparison with the deformability, or the desaturability of the voids and, hence, can be neglected with justification. Under this circumstance, therefore, when the volume element is fully saturated, the dominant mechanism controlling change in storage is $dV_v/d\Psi$. When the volume element is partially saturated, it is customary in the soil physics literature to treat the soil skeleton to be rigid, and since $d\rho_w/d\psi$ is neglected, the sole mechanism controlling change in storage is $dS/d\psi$. However, making $dS/d\Psi$ the only cause of release of water from storage in partially saturated soils has been known to lead to conceptual difficulties either when the soil is very close to full saturation or when it is fully saturated, but has negative pressure head (as is the case in the capillary fringe above the water table). Figure 1 depicts the drying curve for a fine to medium-grain sand. In this figure, Ψ_A represents the "air-entry" value. Pressure heads have to drop below the air entry value in order that desaturation can be initiated. It is obvious that in the range, $\Psi_A < \Psi < 0$, $dS/d\Psi = 0$, implying that if the soil is assumed to be rigid, the volume element has no ability to change its storage and, hence, it cannot partake in transient fluid flow. Stated differently, $dS/d\Psi = 0$ implies that the commonly used parameter soil moisture diffusivity,

$$D = K \frac{d\Psi}{d\Theta} = K \frac{d\Psi}{d(nS)} = \frac{k}{n} \frac{d\Psi}{dS} \tag{3}$$

becomes infinite, causing Richards' equation to break down. This difficulty, it should be noted, is more conceptual than real. Consequently, there is little justification to assume that porosity, n, is constant when $\Psi < 0$. Indeed, if we consider that n can continue to vary as Ψ changes from a positive value through zero to a negative value, then, for $\Psi_A < \Psi < 0$,

$$D = K \frac{d\Psi}{d\Theta} = K \frac{d\Psi}{Sdn} = \frac{K}{m_v} \tag{4}$$

where $m_v = dn/d\Psi$ and the coefficient K/m_v is usually called the coefficient of consolidation in soil mechanics literature. Comparison of (3) and (4) shows that soil moisture diffusivity, and coefficient of consolidation, are conceptually identical quantities, and differ only in the physical process by which fluid is released from storage (Narasimhan and Witherspoon, 1977).

From the foregoing, it is clear that for a proper unification of the saturated and unsaturated zones it is necessary to take into account the deformation of the porous medium in the partially saturated zone. In the light of equation 2, this amounts to the evaluation of the quantity, $dV_v/d\Psi$ when $\Psi < 0$.

DEFORMATION OF A POROUS MEDIUM

A porous medium deforms in response to changes in its skeletal stresses. Therefore, the role of Ψ, in causing the change in void volume of a volume element, depends on how change in Ψ causes the skeletal stresses to change.

In general, change in volume occurs due to three dimensional changes in skeletal stresses. A rigorous treatment of volume change, therefore, requires consideration of a three dimensional stress-strain equation in addition to the transient fluid flow equation. However, if the stresses acting on the boundary of the region of fluid flow remain constant, then one could make certain simplifying assumptions in regard to the relationship between Ψ and skeletal stresses, and conveniently avoid the effort needed for solving the stress equation. The first of these is that, since pore water pressure is hydrostatic in nature, change in Ψ affects only the normal stress components of the stress tensor. The second is, that if the lateral dimensions of the flow domain are large in comparison with the vertical (as is the case with many groundwater systems), then one could further simplify the problem by considering that change in volume is only caused by changes in vertical stresses.

Using this simplified reasoning, Terzaghi (1923) proposed the one-dimensional theory of soil consolidation, and introduced the concept of effective stress, which denotes the stresses effectively acting on the soil skeleton. Thus,

$$\sigma' = \sigma - \gamma_w \Psi \qquad (5)$$

where σ' and σ denote, respectively, effective stress and total vertical stress, and γ_w is the specific weight of water. If we assume that σ, the total stress, remains constant with time, then any change, $\Delta\Psi$, in pressure head is fully converted to an equivalent effective stress, yielding a line of unit negative slope (Fig. 2) when σ' is plotted as a function of Ψ.

Although Terzaghi's concept has proved to be valid in a multitude of field studies related to saturated soils, soil engineers have found (Skempton 1961) that (5) does not strictly hold for partially saturated soils. Experience shows that in such soils, only part of the pressure head is convertible to effective stress. To take this into account, Bishop (1955) proposed a modification to (5) by introducing a parameter χ,

$$\sigma' = \sigma - \chi\gamma_w \Psi, \quad 0<\chi<1 \qquad (6)$$

The reason as to why Ψ is not fully convertible to mechanical stresses in a partially saturated soil is not difficult to understand. As the saturation decreases, the surface area of the soil grains, over which the fluid exerts its pressure, also decreases. Hence, $\chi = \chi(S)$. It is quite well known that the desaturation process is initiated in the largest pores, and proceeds to successively smaller and smaller pores. It is also well known that specific surface (surface area per unit volume) increases rapidly as pore diameter decreases. Hence, one would expect that χ will decline gradually with S at high saturations, and more steeply at low saturations giving rise to a curve which is convex upwards as in Figure 3. Obviously the $\chi(S)$ function should vary from soil to soil, depending on the pore-size distribution. However, it is not quite clear whether χ is governed only by saturation. For example, one might suspect that at low saturations thermodynamic effects will affect the relationship between pore fluid pressure and mechanical stresses. At any rate, it is easy to see that $\chi(S)$ has to be experimentally determined for each soil. Figure 4 represents $\chi(S)$ for two partially saturated soils.

EFFECTIVE STRESS IN THE UNSATURATED ZONE

The nature of deformation, as the porous medium begins to desaturate, depends on the path of change of effective stress as Ψ decreases below zero. Since S and Ψ are mutually related, it follows that the relationship between σ' and Ψ is dependent on how χ varies with Ψ. In order to gain a feel for the role of χ in controlling effective stress in the unsaturated zone, a few calculations were made for a fine sand, assuming four different, arbitrary variations of χ in relation to S. The results are presented in Fig. 5B and 5C for a hypothetical fine sand with soil moisture characteristics as shown in Fig. 5A.

In Fig. 5B, the curve labelled 1 is probably closer to reality than the others, and indicates that effective stress may continue to increase significantly, even at low saturation. It is pertinent to point out here that finer and finer soils will have higher and higher residual saturations and, hence, $\chi(S)$ may be significant in such soils even at low saturations. The curve labelled 4 in Fig. 5C was constructed to study what the form of the $\chi(S)$ curve should be if the soil is to be treated as rigid below some saturation level.

Suppose, below some critical value of Ψ, the effective stress in the unsaturated zone stabilizes at an ultimate value. Then

$$\sigma'_{\text{ultimate}} = \sigma - \chi\gamma_w\Psi, \quad \Psi<\Psi_{\text{critical}} \tag{7}$$

Hence,

$$\chi = (\sigma-\sigma'_{\text{ultimate}})/\gamma_w\Psi, \quad \Psi<\Psi_{\text{critical}} \tag{8}$$

In other words, χ is inversely proportional to Ψ and, $(\sigma-\sigma'_{\text{ultimate}})/\gamma_w$ is the constant of proportionality.

Note that curves 2, 3, and 4 in Figures 5B and 5C all show a small interval of Ψ, over which effective stress falls and rises again. This is caused by the interplay between $\chi(S)$ on the one hand, and $S(\Psi)$ on the other. It is not clear if this could be a physical reality. If it is not, and if one should consider that σ' is monotonic with reference to Ψ, then χ might have to be treated as a function of Ψ rather than S.

THE DEFORMATION COEFFICIENT IN THE UNSATURATED ZONE

Having considered the nature of effective stress in the unsaturated zone, we now proceed to evaluate the deformation of the volume element by considering the term $\rho_w S \frac{dV_v}{d\Psi}$ in (2). Thus,

$$\rho_w S dV_v/d\Psi = \rho_w S \frac{dV_v}{d\sigma'} \frac{d\sigma'}{d\psi} \tag{9}$$

but, in view of (6),

$$\frac{d\sigma'}{d\Psi} = \frac{d}{d\psi}(-\chi\gamma_w\Psi)$$

$$= -\gamma_w\left[\chi + \psi \frac{d\chi}{d\Psi}\right] \tag{10}$$

$$= -\gamma_w \chi^*$$

In view of (10), (9) becomes

$$\rho_w S \frac{dV_v}{d\Psi} = -\rho_w S\chi^* \frac{dV_v}{d\sigma'} \tag{11}$$

To evaluate change in fluid mass in the volume element due to deformation, therefore, we need to know the relationship between the change in void volume and the change in effective stress for the material composing the volume element. In the soil mechanics literature, it is customary to present experimental data on soil deformation in the form of a plot of void ratio, e, as a function of effectve stress. Equivalently, since e is uniquely related to porosity, n, one could present the same data in the form of a plot with n as a function of σ'. Depending on how we choose to define the volume element under study we can use either the e versus σ' or the n versus σ' relationship to evaluate $dV_v/d\sigma'$. Thus, if we define the volume element to always have a fixed volume of solids, V_s, and assume the solid grains to be rigid, then, (Narasimhan and Witherspoon, 1977)

$$-\frac{dV_v}{d\sigma'} = -V_s \frac{d(V_v/V_s)}{d\sigma'} = -V_s \frac{de}{d\sigma'} = V_s a_v \quad (12)$$

where a_v is called the coefficient of compressibility. On the other hand, if we define the volume element to always have a constant bulk volume, then

$$-\frac{dV_v}{d\sigma'} = -V \frac{dn}{d\sigma'} = Vm_v \quad (13)$$

where m_v is the coefficient of volumetric compressibility. In view of (11), (12) and (13), the fluid mass capacity of the volume element (equation 2) may now be written as:

for a volume element with constant V_S:

$$M_c = V_S[\rho_w S\gamma_w a_v + eS \frac{d\rho_w}{d\Psi} + e\rho_w \frac{dS}{d\Psi}] \quad (14)$$

or, for a volume element with constant bulk volume, V:

$$M_c = V[\rho_w S\gamma_w m_v + nS \frac{d\rho_w}{d\Psi} + n\rho_w \frac{dS}{d\Psi}] \quad (15)$$

In groundwater hydrology, the coefficient S_s is generally used to denote fluid mass capacity per unit volume (specific fluid mass capacity) of a saturated soil. It is not clear from the literature whether the volume element to which S_s refers either has constant V_s, or constant V. If one assumes that S_s pertains to an element with constant bulk volume and if we let $\rho_w = 1$, then, from (15) we see that

$$S_s = \gamma_w m_v + n \frac{d\rho_w}{d\Psi} \tag{16}$$

ILLUSTRATIVE EXAMPLES

Three illustrative examples are presented below to verify the theoretical considerations presented above. All of these examples are numerical simulations carried out with an integrated finite difference computer program called TRUST (Narasimhan, 1975 ; Narasimhan, et al., 1978).

The first example relates to a column drainage experiment conducted by Liakopoulos (1965), in which a one-meter vertical column was loaded with Del Monte sand. For several hours before the experiment, water was allowed to enter the column at the top, and to drain freely at the bottom. The inflow was carefully adjusted in such a fashion that just before t = 0, the column was fully saturated, and Ψ was zero everywhere in the column as measured in several tensiometers distributed within the column. This meant that the vertical gradient in potential, $\partial\phi/\partial z = 1$. At t = 0, the inflow was cut off, and the column was allowed to drain freely. During the drainage process, Ψ was continuously monitored in the tensiometers. The problem is to mathematically simulate the observed evolution of the profile of Ψ over the column.

The initial and boundary conditions of this problem are depicted in Fig. 6, while the material properties of Del Monte sand are shown in Fig. 7. Note that the air-entry value of Del Monte sand occurs at $\Theta \approx .298$ and $\Psi \approx -.2m$. The results of the numerical simulation are presented in Fig. 8.

The first simulation was carried out, assuming that the sand is rigid, and the results are presented in Fig. 8A. The simulated results show the development of much larger moisture suctions, especially during the first five minutes of the experiment, than the observed values. At the same time, the experimental data show that the maximum suction over the column was less than 0.2m of water, which is below the air-entry value, Ψ_A. Since $dS/d\Psi = 0$ for $\Psi > \Psi_A$, and since the sand was assumed rigid, release of fluid from storage had to be accounted for in the numerical model purely by the very small expansivity of water, resulting in the development of higher moisture suction in the numerical model than in the experimental data.

The second simulation was carried out using a small but reasonable deformation parameter for the sand, but assuming arbitrarily that $\chi = 1$ for $\Psi >$ air entry value and $\chi = 0$, $\Psi <$ air-entry value. The results are presented in Fig. 8B. It is seen that a reasonable agreement with the experimental results is now obtained. These results suggest that some amount of deformation, probably due to rearranging of grains, took place during the first five minutes of the experiment. Liakopoulos (1965) also carried out a finite difference simulation, and obtained reasonable comparison by assuming a finite value for diffusivity for $\Psi > \Psi_A$. This is quite consistent if one interprets the diffusivity in this case as equivalent to the coefficient of consolidation.

The second illustrative example relates to a drainage experiment conducted by Vauclin et al, (1975) on a sand box 3 m wide, 2 m tall and 0.05 m deep. The initial conditions were hydrostatic, with the lower part of the flow region saturated, the upper part partially saturated, and with the surface $\Psi = 0$ horizontally disposed at an elevation of 1.43 m above bottom (Fig. 9). At t = 0, the water level on the left hand side of the box was suddenly lowered by .63 m and maintained constant. The distribution of pore pressures and moisture contents within the box were monitored carefully during the subsequent fall of the surface $\Psi = 0$. The problem is to numerically simulate the observed evolution of Ψ over the sand box for $t > 0$. The properties of the sand used in the sand box are presented in Fig. 10. The numerically computed positions of the surface $\Psi = 0$ are compared with experimental results at four different times in Fig. 11, and the agreement is reasonable. In this problem, the release of water from storage is so much dominated by the desaturation process taking place, in the upper left hand side of the box, that a few trial simulations indicated that changing the deformation coefficient of the saturated material had little effect on the behavior of the system. Vauclin et al., (1975) also successfully carried out a finite difference simulation of the same experiment, assuming that the sand is rigid, implying that the saturated zone obeys the Laplace equation.

The last example aims at examining a possible field situation in which the deformation of the soil matrix in the unsaturated regime might be of importance. It is known that earth fissures occur in many alluvial basins of the Southwestern United States (J. T. Neal, 1978). Some of these fissures may form such typical patterns as polygons, subparallel stripes and rings around phreatophytes. The fissures may occur either in areas undergoing natural, long-term groundwater declines (playas), or in areas undergoing large scale groundwater declines due to heavy pumping. For example, ground fissures are extensively observed in the Salt River Valley and the lower Santa Cruz River basin areas of South-Central Arizona (Winikka et al., 1978) where water levels have declined from 60 to 150 meters between 1923 and 1977.

Several hypotheses have been proposed to explain the origin of earth fissures (Davis, 1978). One of these hypotheses is that the fissures are caused by horizontal movements induced by contractions associated with declines of water tables in unconfined aquifers. The credibility of this hypothesis depends on whether enough stresses can be mobilized in the flow regime above the water table which is subjected to moisture suction accompanying the desaturation process. In order to quantitatively verify the possibility that enough effective stresses could be mobilized in the unsaturated regime to cause significant contractions, numerical simulations were carried out on a hypothetical, one-dimensional drainage column as described below.

Consider a one-dimensional column (Fig. 12) of fine sand/silt material 85 m tall. The column is initially saturated up to the top and is under hydrostatic equilibrium, with a potential everywhere equal to 85 m of water. At $t = 0$, water is allowed to drain at the bottom at a constant rate of 2.746×10^{-5} m^3/sec. With the onset of drainage, desaturation is initiated at the top. The purpose of the simulation is to find out if significant volume deformations can accompany the desaturation process. The column was divided into 85 volume elements, each 1 m tall.

Two hypothetical soils were considered in the simulation with different soil moisture characteristics, but essentially the same χ versus S relationship (insets, Fig. 13). Both the soils were assumed to have the same non-linear deformation coefficient ($C_c = 0.1$), and permeability characteristic given by

$$k = {\frac{4.628}{4.628+|\Psi|}}^{1.645} \; 5.67 \times 10^{-14}$$

The relationship between effective stress and pressure head for the two soils is shown in Fig. 13.

The column was subjected to a simulated drainage over a period of ten years with simultaneous compaction. The profiles of variation in some of the interesting parameters for Soil 2, at the end of ten years, are shown in Fig. 14.

The computed deformations in the unsaturated zone (which is the prime concern of this study) are summarized in the following table.

TABLE 1. Computed Deformations in the Unsaturated Zone

	Soil assumed rigid in unsaturated zone, %	$\chi = \chi(S)$	
		Soil 1, %	Soil 2, %
Total volume strain above water table after 10 years	0.95	1.31	1.24
Volume strain in the saturated regime	0.95	0.96	0.91
Volume strain in the unsaturated regime	0	0.35	0.33

As can be seen from the table, for both soils, deformation in the unsaturated regime is over 25 percent of the total volume strain. This leads to the inference that, under certain conditions of heavy withdrawal of water from unconfined aquifers, one could expect significant volume strains to accompany the desaturation process. Such volume strains may lead (if conditions are favorable) to contractions and associated horizontal movements causing earth fissures.

CONCLUSIONS

The study shows that soil deformation is a physical phenomenon of considerable interest, and it cannot be ignored in dealing with partially saturated materials. The need for considering soil deformation, in the regime of partial saturation, arises from philosophical, as well as practical considerations. From the philosophical point of view, it has been shown that if the governing equation of saturated-unsaturated flow is to be rigorously valid in the zone of transition between the two regimes, then one must recognize the conceptual unity between the coefficient of consolidation and soil moisture diffusivity. From the practical point of view, there exists considerable quantitative justification to believe that deformation in the unsaturated zone may lead to dessication-type contraction phenomena leading to cracking and fissuring of soils.

NOTATION

D	soil moisture diffusivity	(L^2/T)
K	hydraulic conductivity	(L/T)
m_v	volumetric compressibility	(LT^2/M)
M_c	fluid mass capacity of a volume element	(M/L)
M_w	mass of water	(M)
n	porosity	
S	saturation	
S_s	coefficient of specific storage	$(1/L)$
V	bulk volume	(L^3)
V_s	volume of solids	(L^3)
V_v	volume of voids	(L^3)
z	elevation head	(L)
γ_w	specific weight of water	(M/L^2T^2)
Θ	volumetric moisture content	
ρ_w	density of water	(M/L^3)
σ	total stress	(M/LT^2)
σ'	effective stress	(M/LT^2)
ϕ	hydraulic head ($= z + \psi$)	(L)
Ψ	pressure head	(L)
Ψ_A	air entry pressure	(L)

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FIGURE CAPTIONS

Fig. 1. Saturation characteristic for a fine to medium-grained sand (Ψ_A = air entry value).

Fig. 2. Schematic relationship between σ' and Ψ when pore pressure is fully converted to effective stress.

Fig. 3. Schematic representation of the likely dependence of χ on S.

Fig. 4. χ versus S relationship for two partially saturated materials (after Bishop and Blight, 1963).

Fig. 5. Relationships of χ, S, Ψ and σ' for a hypothetical fine sand. A.) saturated characteristic. B. and C.) $(\sigma'-\sigma)$ versus Ψ for various χ versus S relations.

Fig. 6. Column drainage: initial and boundary conditions.

Fig. 7. Moisture characteristic of Del Monte sand (based on Liakopolous, 1965).

Fig. 8. Column drainage: comparison of computed and observed pressure-head profiles.

Fig. 9. Sand box experiment: initial and boundary conditions.

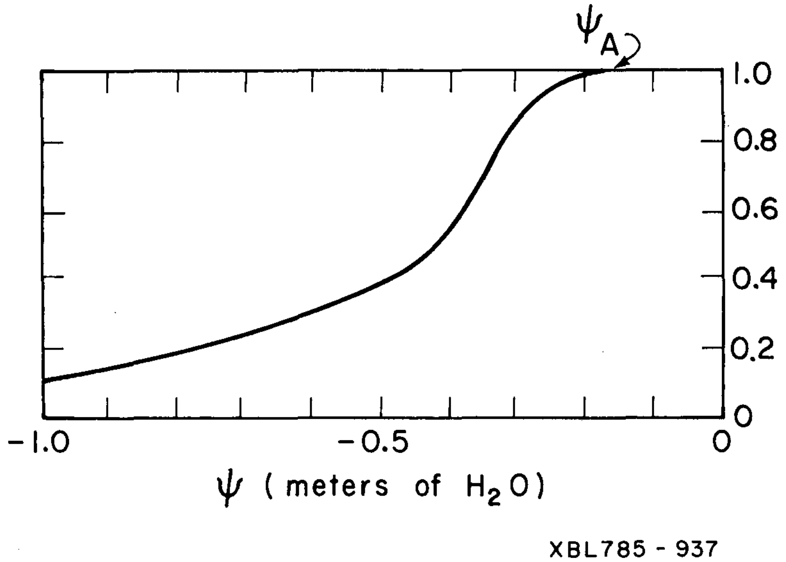
Fig. 10. Sand box experiment: material properties.

Fig. 11. Sand box experiment: observed and computed position of the surface, $\Psi = 0$.

Fig. 12. Earth fissure problem: initial and boundary conditions.

Fig. 13. Earth fissure problem: variations of effective stress with Ψ for two hypothetical soils.

Fig. 14. Earth fissure problem: profiles of S, χ, σ', and volume strain after t = 10 years.



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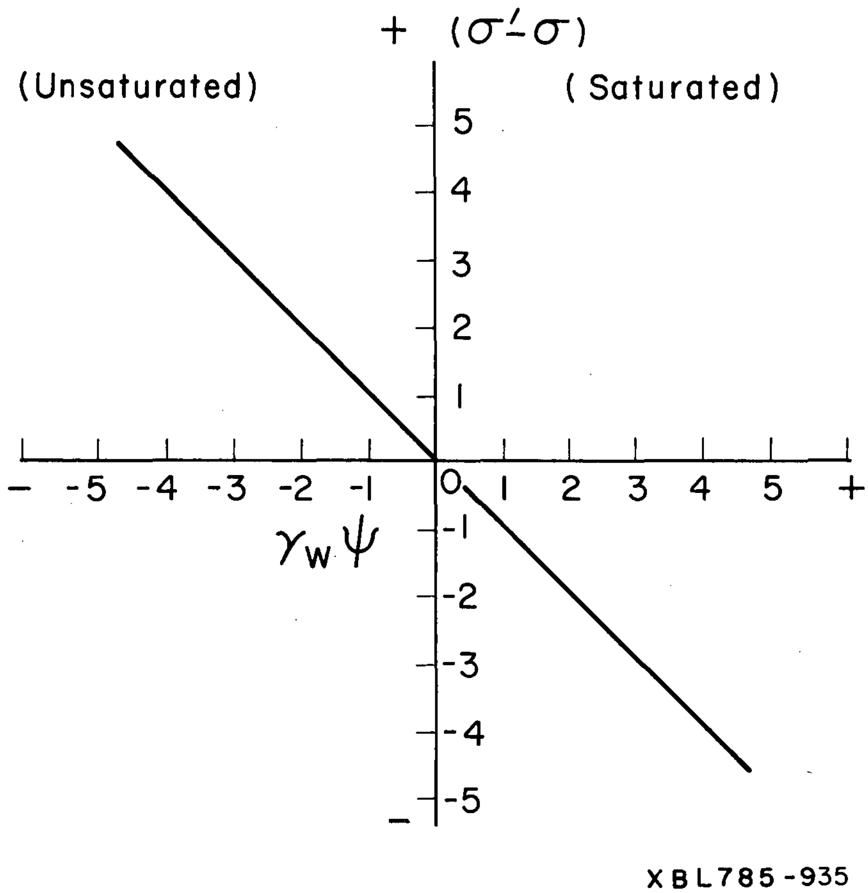


Fig. 2. Schematic relationship between σ' and Ψ when pore pressure is fully converted to effective stress.

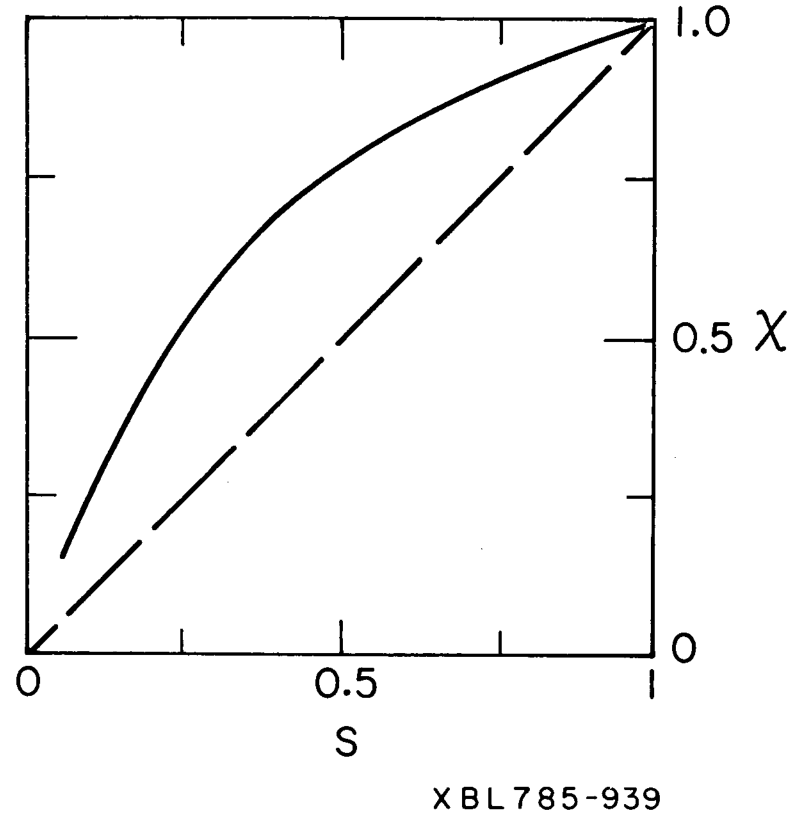


Fig. 3. Schematic representation of the likely dependence of χ on S.

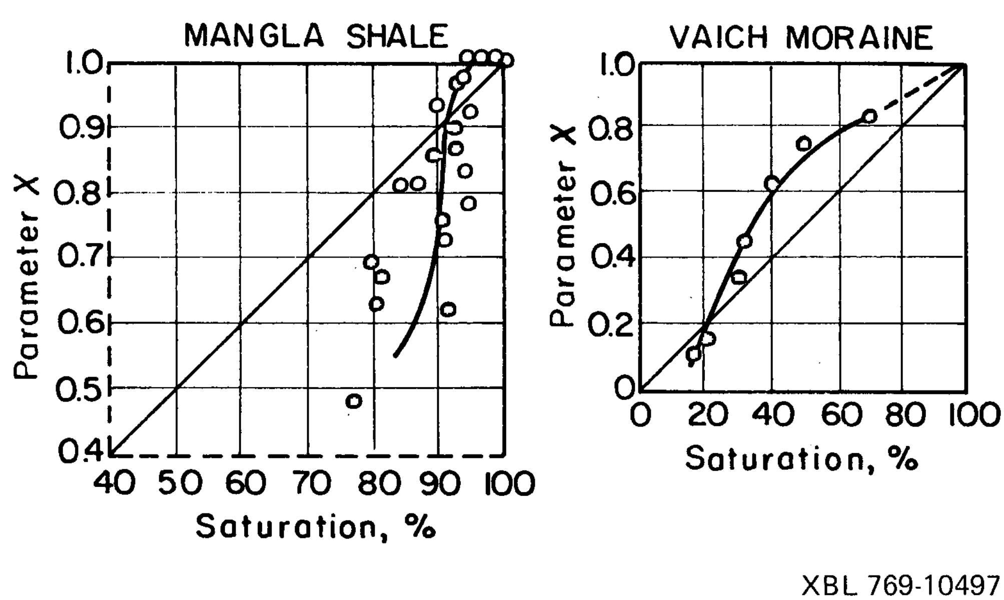


Fig. 4. χ versus S relationship for two partially saturated materials (after Bishop and Blight, 1963).

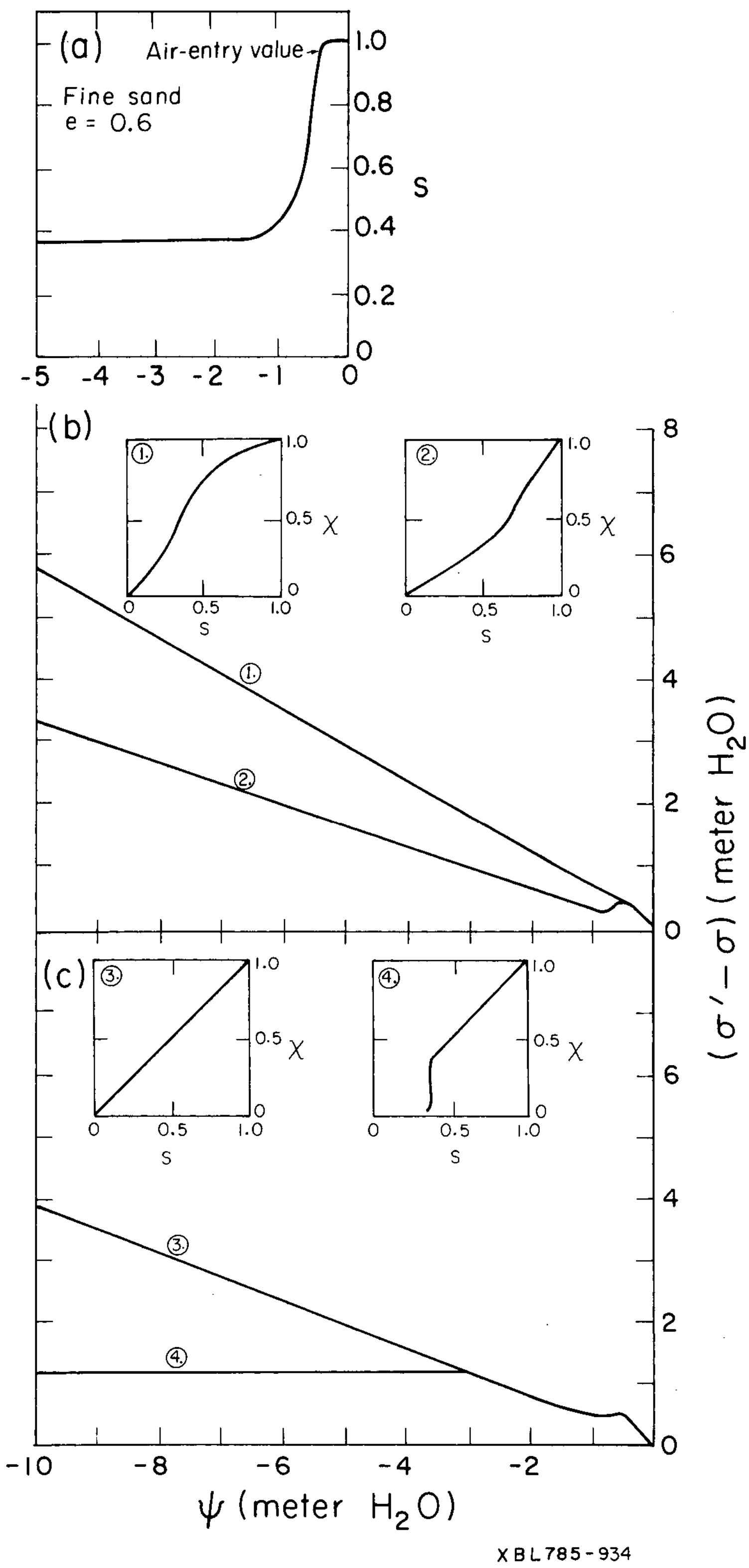


Fig. 5. Relationships of χ, S, Ψ and σ' for a hypothetical fine sand. A.) saturated characteristic. B. and C.) (σ'-σ) versus Ψ for various χ versus S relations.

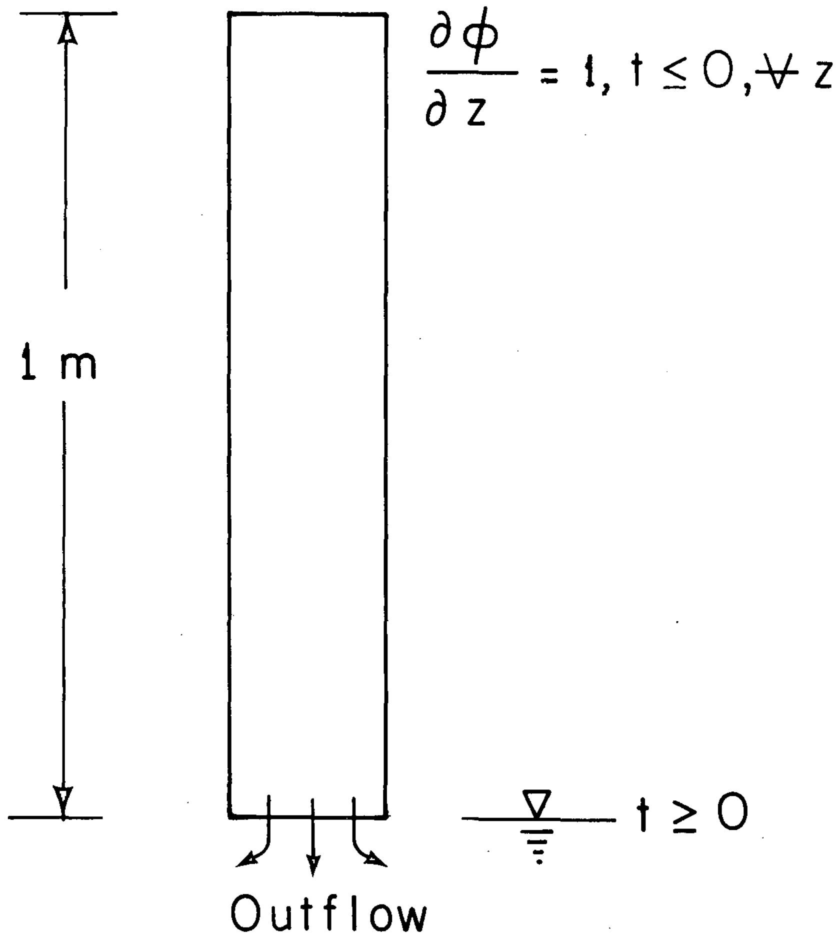
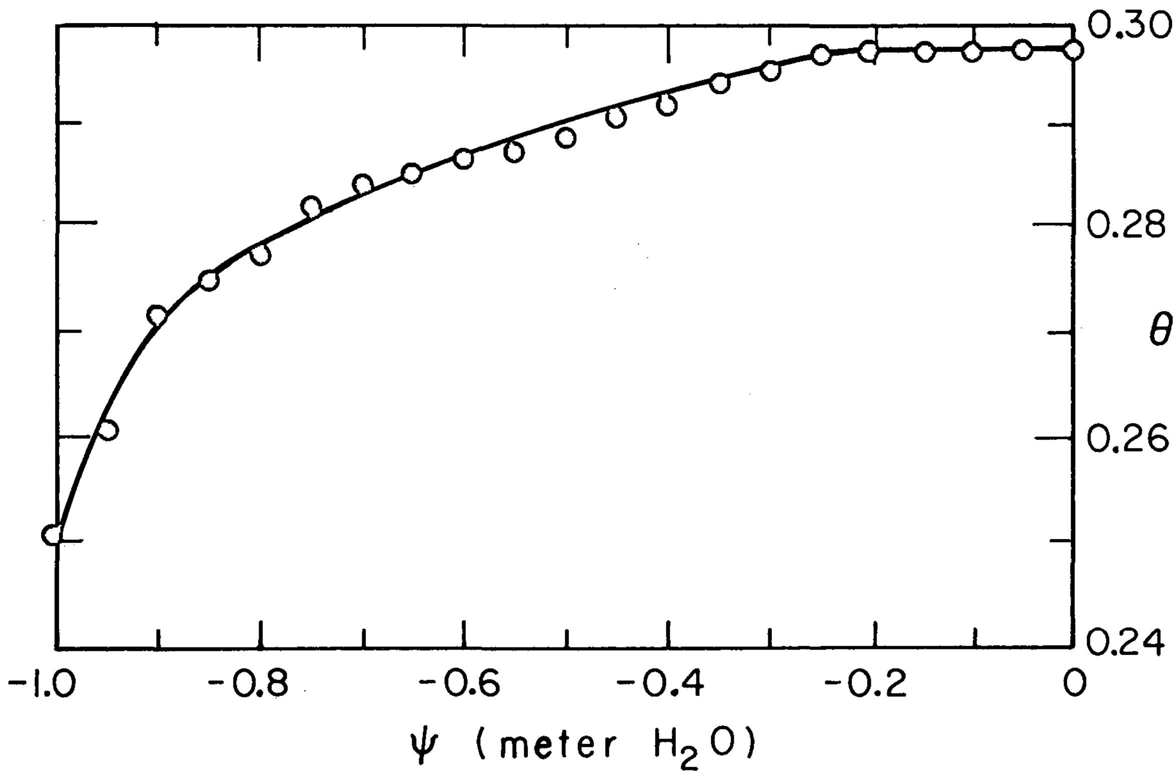


Fig. 6. Column drainage: initial and boundary conditions.



XBL 785-941

Fig. 7. Moisture characteristic of Del Monte sand (based on Liakopolous, 1965).

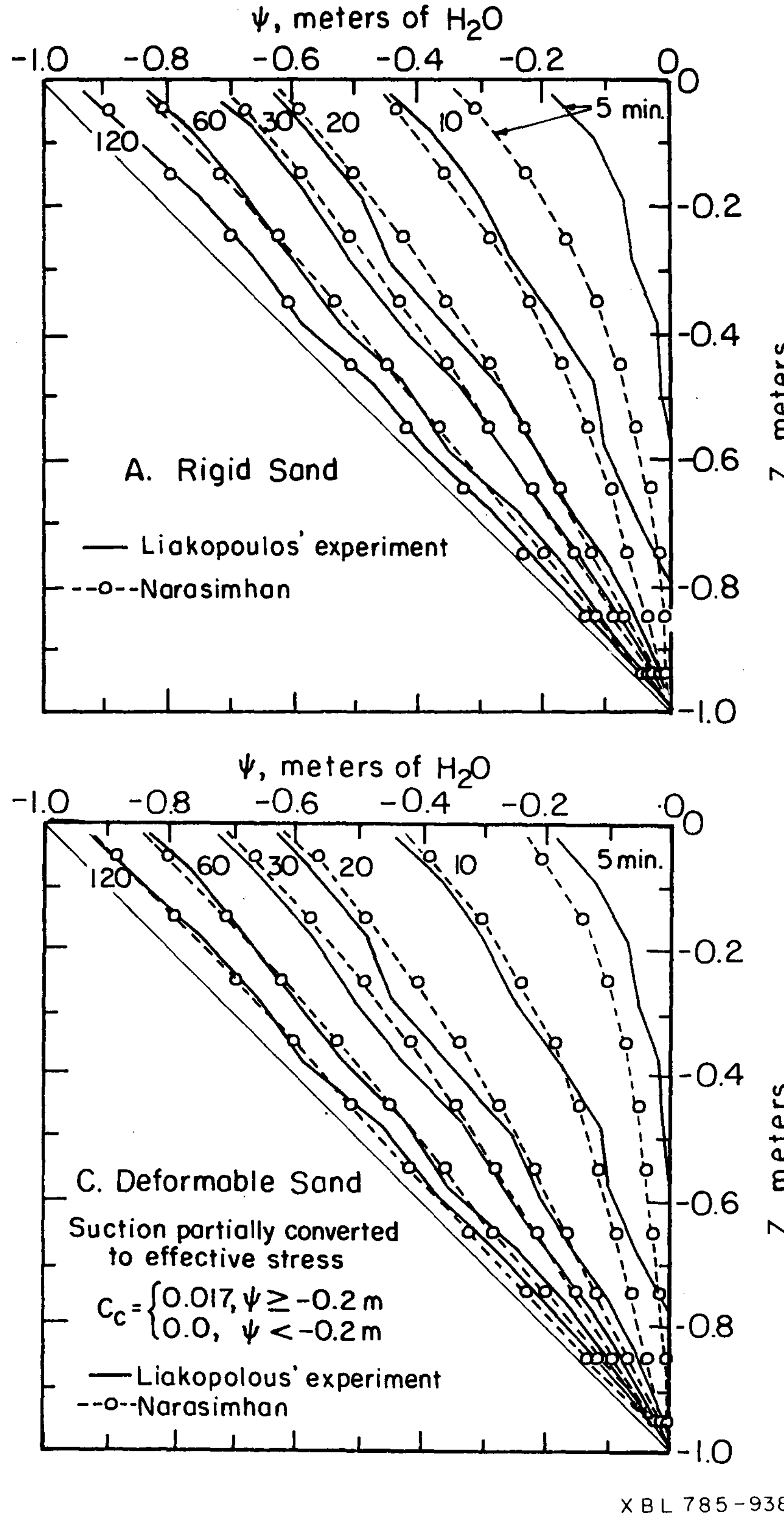
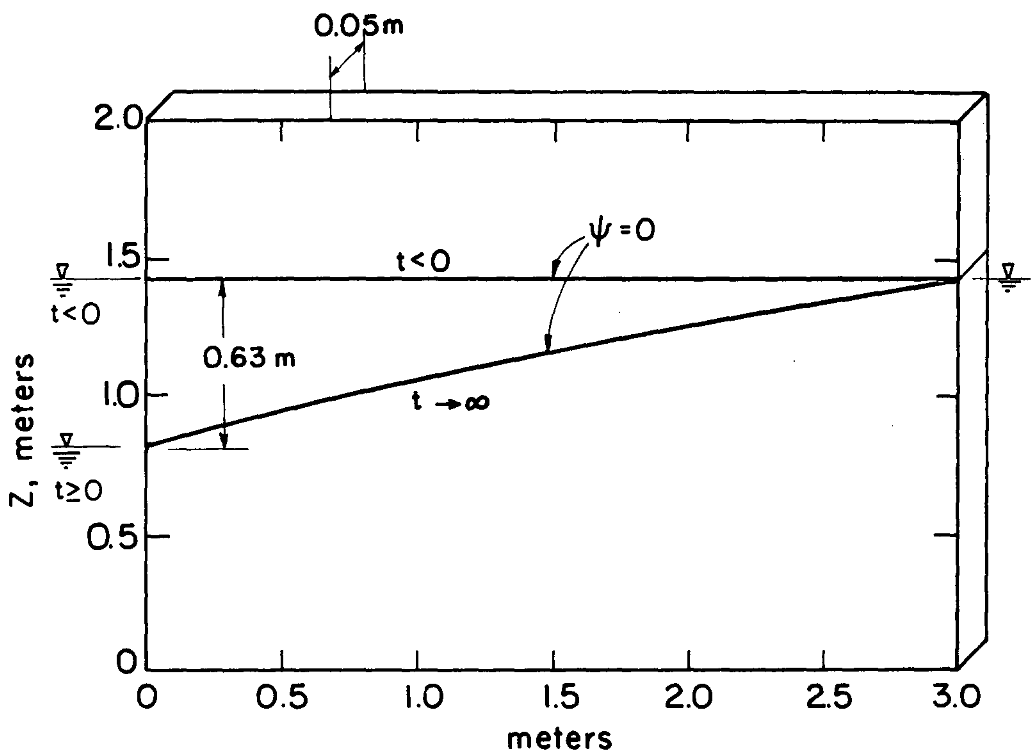


Fig. 8. Column drainage: comparison of computed and observed pressure-head profiles.



XBL 756-1654

Fig. 9. Sand box experiment: initial and boundary conditions.

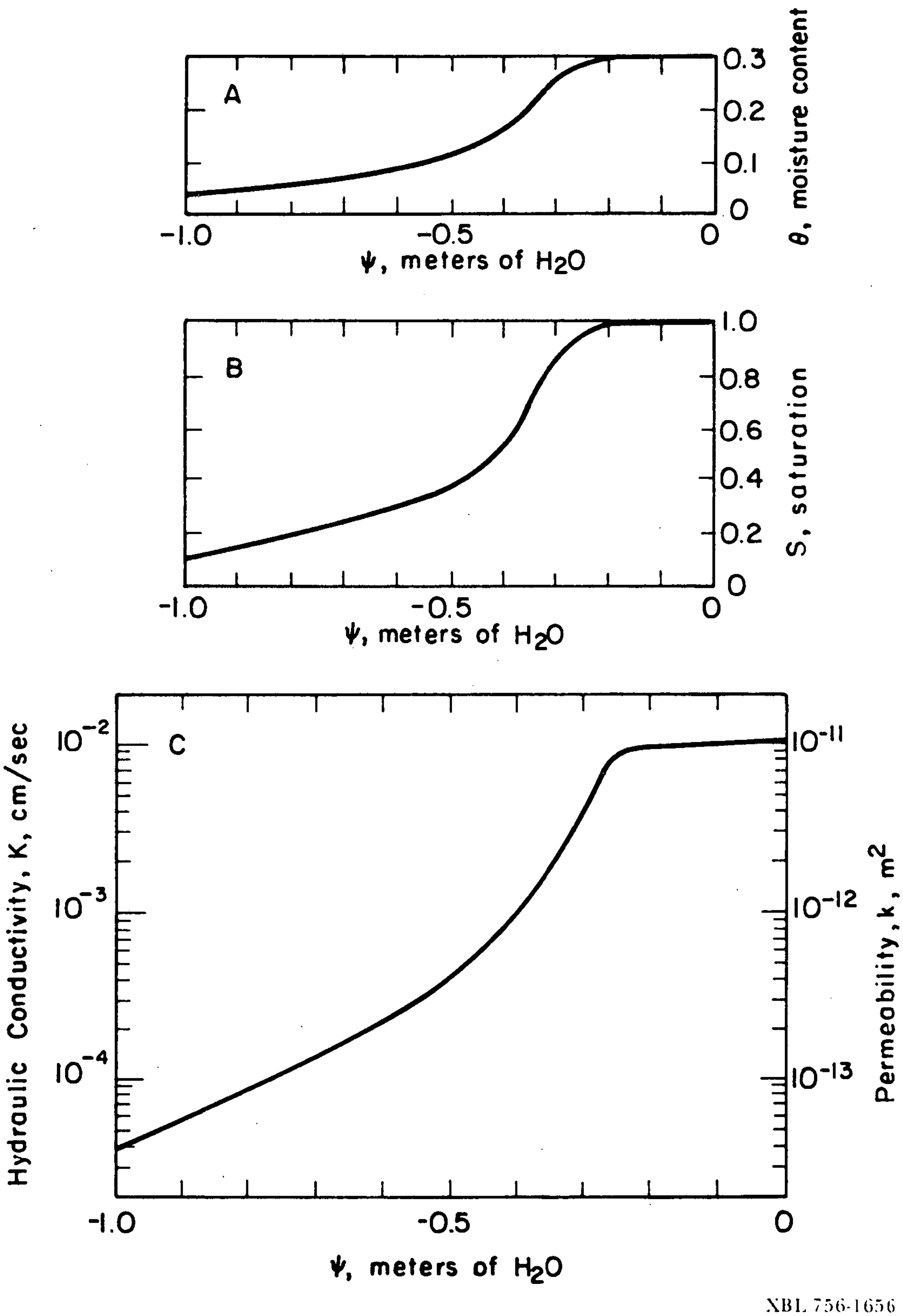


Fig. 10. Sand box experiment: material properties.

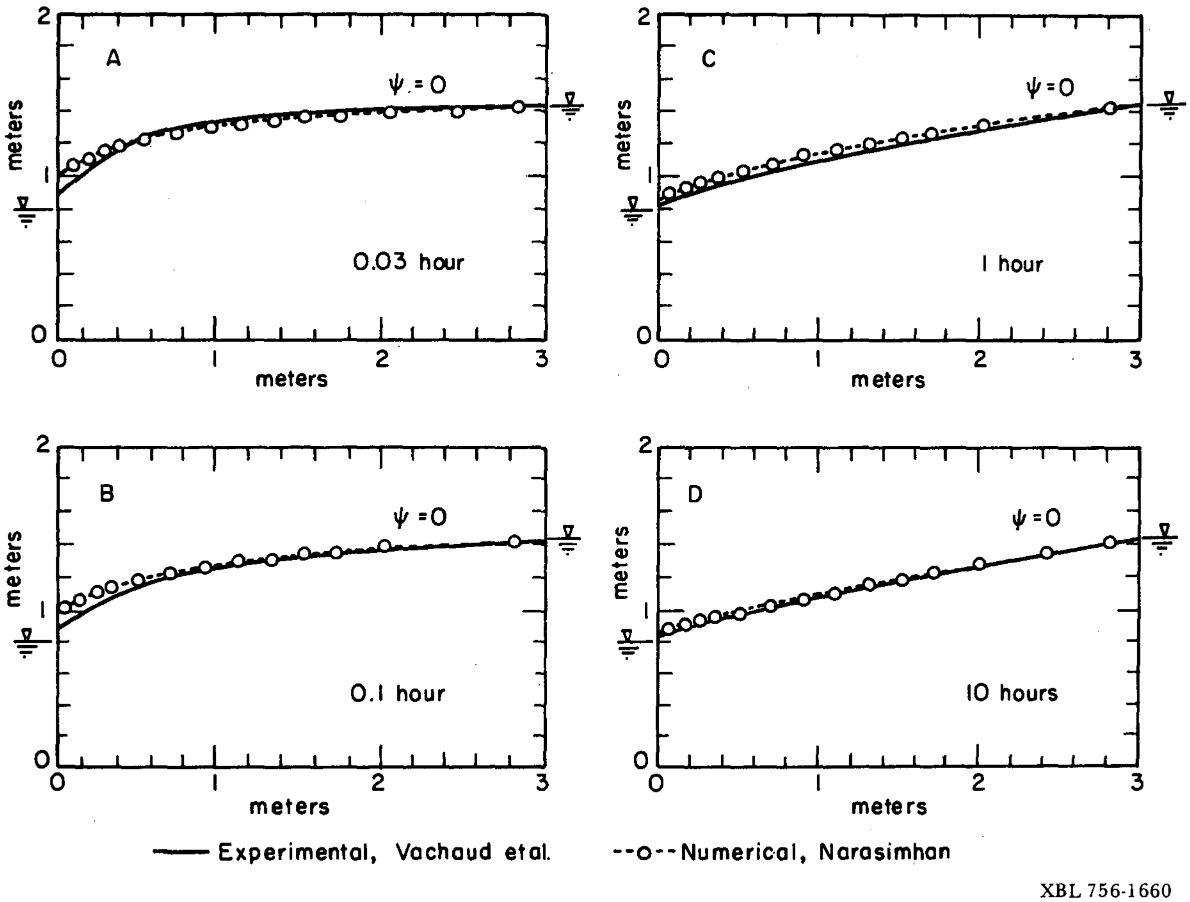
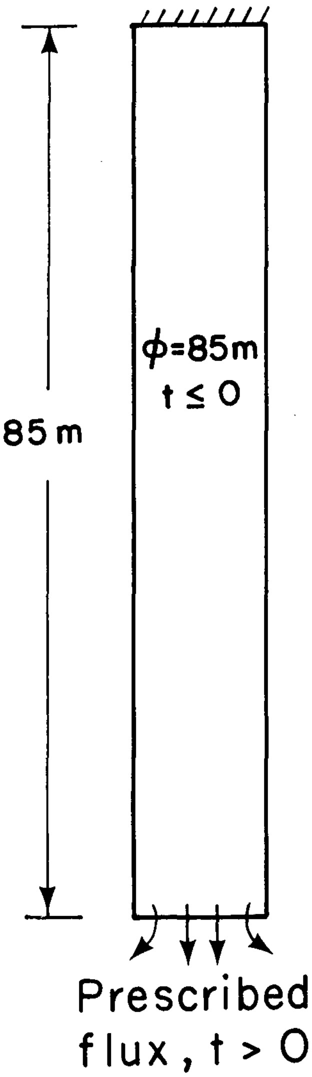


Fig. 11. Sand box experiment: observed and computed position of the surface, $\Psi = 0$.



XBL785-936

Fig. 12. Earth fissure problem: initial and boundary conditions.

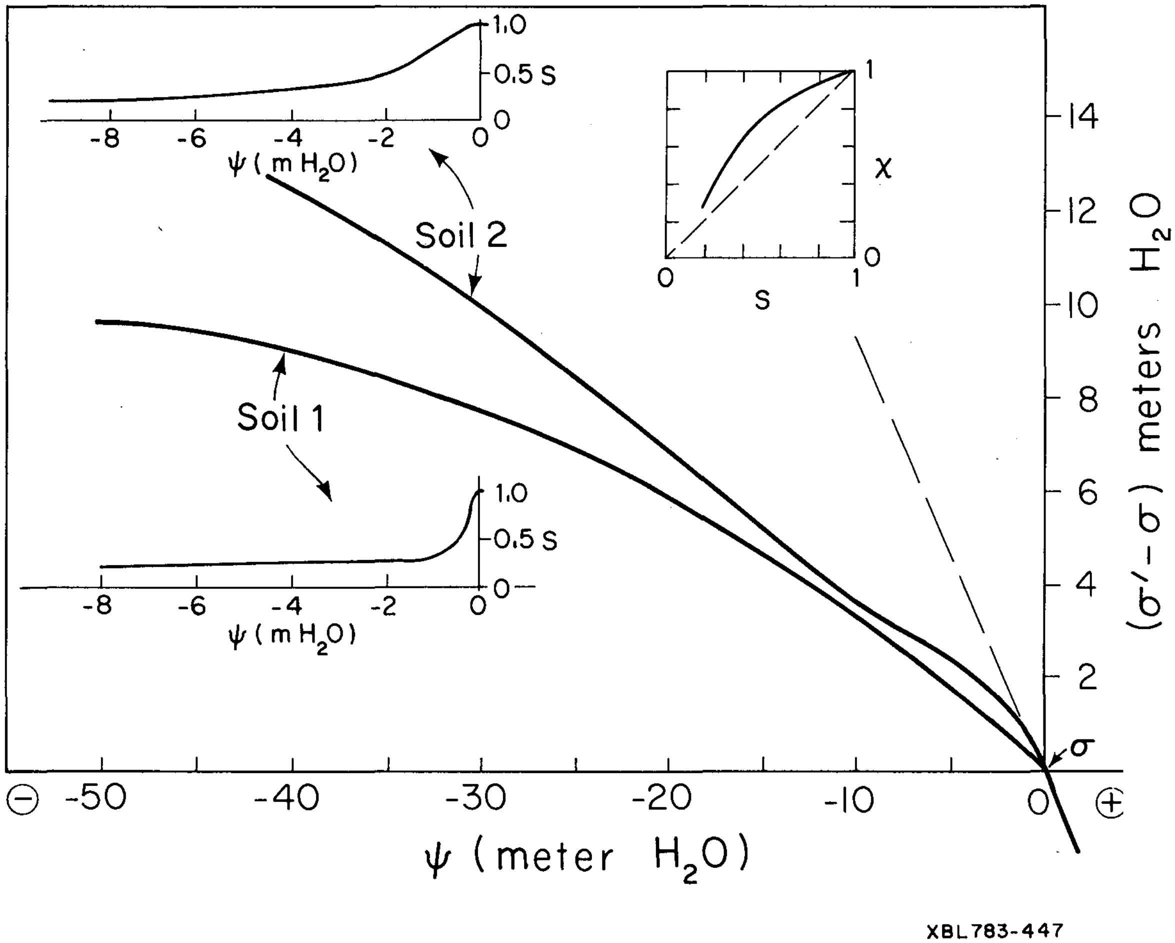


Fig. 13. Earth fissure problem: variations of effective stress with Ψ for two hypothetical soils.

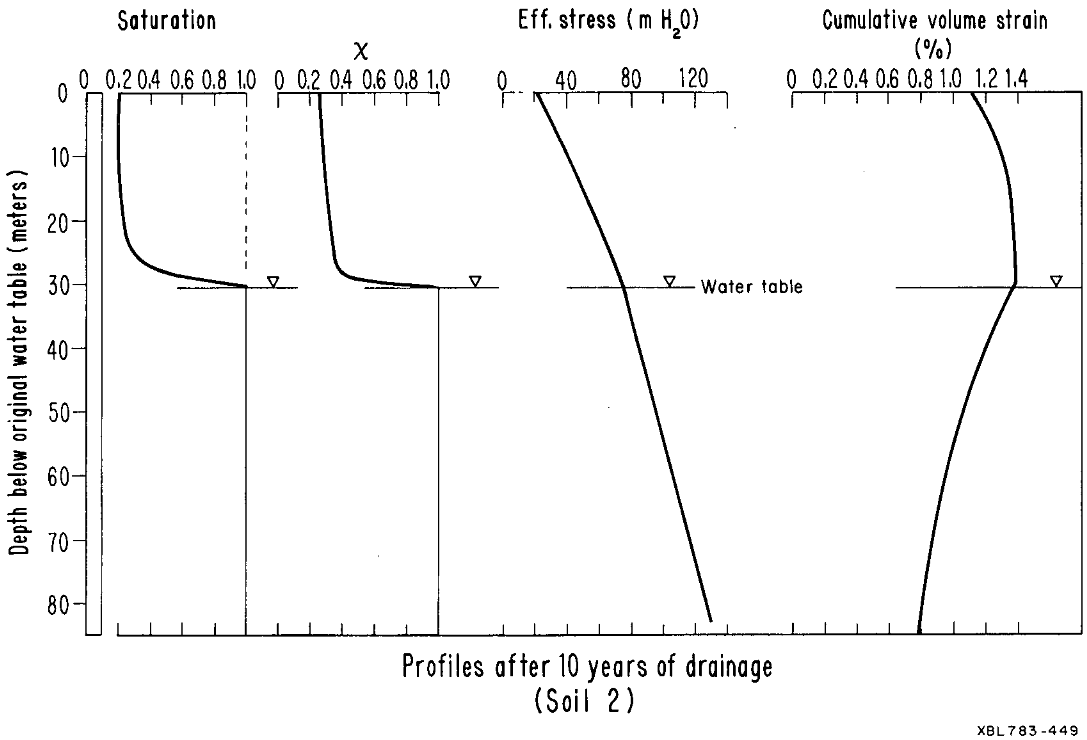


Fig. 14. Earth fissure problem: profiles of S, χ , σ', and volume strain after t = 10 years.

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