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Numerical Analysis of the Post-Filling Performance of Small Collapsing Dams

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ABSTRACT: An incremental procedure was developed for performing finite element analysis on the behavior of a small collapsing dam during transient unsaturated-saturated seepage which follows the first filling of the reservoir. A computer program, named COUPSO, has been coded based on a procedure that couples stress equilibrium and water flow using a generalized form of the theory of consolidation for unsaturated soils. In this form, a metastable-structured soil has a non-linear and stress-induced anisotropic behavior in response to a decrease in matric suction. The extended Mohr-Coulomb failure criterion for unsaturated soil defines the failure conditions in points within the small dam. The model takes into account the varying permeability of the collapsing soil when following a wetting stress path. Analyses were performed to study the post-filling performance and failure mechanisms of small dams constructed in Northeast Brazil. The results indicate that the procedure developed is a potentially useful tool.

KEYWORDS: Earth Dams, Collapsing Soil, Numerical Modelling, Post-Filling Behavior.

1 INTRODUCTION

The performance of an earth embankment depends upon its response to applied loadings and imposed hydraulic gradients. The stability analyses are more complex in cases where the soils which comprise the embankment undergo significant changes in mechanical and/or hydraulic properties as a result of a change in pore-water pressure.

The principles governing mechanical behavior of a saturated soil are well established with respect to both theoretical and practical aspects. The theories for unsaturated soils have been experimentally investigated to accommodate the key aspects involved with their mechanical behaviors. These theories are consistent with a multiphase, continuum mechanics approach and describe the mechanical behavior of an unsaturated soil as a function of two independent sets of stress variables. These are: the net normal stress (σ - u_a) and the matric suction (u_a - u_w) where σ is the total normal stress, u_a is the pore-air pressure and u_w is the pore-water pressure. In these theories, the saturated condition is a special case where the "effective stress" becomes the governing stress state variable. The collapsing behavior of soils during saturation is one of the complex aspects to be developed through application of the unsaturated soil theory. The prediction of the post-filling performance of small collapsing earth dams is one of the engineering problems depending upon these developments.

The primary objective of this study is to develop a better understanding of the mechanical behavior of small dams (height smaller than 10 meters) constructed using collapsing soils (i.e., collapsing dams). The condition of primary importance is the first reservoir filling. The behavior to be modeled must take into account the changes in both the net normal stress (σ - u_a) and matric suction (u_a - u_w) within the earth dam caused by the transient unsaturated-saturated water flow which follows the first reservoir impounding.

Immediately after the first reservoir impounding, water flows through the dam in a transient manner in accordance with the driving hydraulic gradient, the fluid characteristics, the existing boundary conditions, and the hydraulic properties of the soil. As the water flows, the soil in the dam undergoes volume changes in response to changes in total stress and matric suction. Volume changes imply changes in both the mechanical and hydraulic properties of the materials within the dam. In addition, volume changes can generate pore-water pressures and alter the transient flow regime within the dam embankment. A further advance of the water flow into the dam results in a new configuration of mechanical and hydraulic properties. This dynamic process puts in action a complex process in terms of both mechanical behavior and water flow. This is even true for an earth structure which is homogeneous. The dynamic process is transient and occurs until the establishment of steady-state conditions. Therefore, the solution to the problem requires a mathematical model coupling mechanical equilibrium and water flow in collapsing dams in order that geotechnical engineers can analyze this kind of problem. Exact solutions do not exist and numerical methods must be developed.

2 BACKGROUND

Studies have been attempted to explain the collapse of dams during first reservoir filling as a result of changes in the "effective stress" and in the stress-strain relationships of the material within the dam (Nobari and Duncan 1972, Lourens and Czapla 1987). Recently, studies have attempted to explain these phenomena by using the two independent stress state variables and a continuum mechanics approach for unsaturated soil (Miranda 1988, Lloret and Ledesna 1993). However, these studies have experienced difficulties in reproducing available experimental results in terms of the stress-strain behavior of collapsing soils during saturation. This is particularly true for the condition of collapse of the soil structure under Ko-conditions. Under these conditions, a collapsing soil undergoes an increase in mean net confining stress (Maswoswe 1985, Lawton et al. 1991). Therefore, there exists a need for a better understanding of the mechanical behavior of collapsing soils in view of current theories for unsaturated soils.

Miranda (1988) reported that failures resulting from cracks initiated by wetting-induced collapse in earth dams located in the semiarid region of Northeast Brazil are so common that these dams are publicly referred to as "Alka-Seltzer" dams. "Alka-Seltzer" dams are normally constructed with residual soil derived from gneiss. Due to local conditions, these dams often are constructed without the necessary amount of water being added and with inadequate compaction (i.e., poorly compacted). There is a need for a better understanding of the failure mechanism of "Alka-Seltzer" dams to allow the adequate design of such collapsing earth dams.

3 THEORY

The theory for the analysis of the mechanical behavior of "Alka-Seltzer" dams during first reservoir filling is based on a general coupled solution for consolidation of unsaturated soils as presented by Fredlund and Rahardjo (1993). The problem of an "Alka-Seltzer" dam is here analyzed considering the two-dimensional plane strain condition. The equations are developed by neglecting the air phase continuity equation. It is assumed that the air pressure is always atmospheric during the transient water flow process. This assumption is based on the fact that as the water advances into the dam, there is a gradual expulsion of the pore-air through the soil voids which remain with air at atmospheric condition. The soil is considered to be an incrementally isotropic, linear and elastic material in terms of mechanical properties related to changes in net normal stresses. The soil is assumed to be an incrementally stress-induced anisotropic, linear and elastic material in terms of mechanical properties related to changes in matric suction (Pereira 1996, Pereira 1997). The soil is assumed to be anisotropic in terms of its hidraulic properties.

The solution for the plane strain case is solved using the finite element method. Galerkin's residual weighted method is used for the spatial discretization of the continuum and a finite difference scheme is used for the temporal discretization. In order to formulate the theory for the behavior of a small earth dam during its first reservoir filling, it is necessary to have:

a.) a definition of the basic equations governing the phenomena involved;

b.) the constitutive relationships for the component phases of the soil in terms of both mechanical behavior and pore-fluid (e.g., water in this case) flow properties;

c.) the initial stress state conditions in the dam, (i.e., $\sigma - u_a$ and $u_a - u_w$), at the end-of-construction phase;

d.) the essential and natural boundary conditions required for the basic equations (i.e., equilibrium and water continuity) governing the phenomena involved;

e.) the change in the stress state immediately after the first filling of the reservoir, (i.e., the effect of

the water pressure at the upstream slope of the dam);
f.) constitutive relationships for the soil in terms of both mechanical and hydraulic properties.

3.1 Basic Equations of Physics

A rigorous analysis of the mechanical behavior of an unsaturated soil requires the coupling of the following system of equations: a.) water phase continuity equation; b.) air phase continuity equation; c.) static equilibrium of the overall soil medium. These equations are here presented for a referential elemental volume.
Pereira (1996) presents the equations considering the air phase continuity equation. However, in this paper, only the static equilibrium of the soil medium and the water phase continuity equation are considered.

3.1.1 Water continuity equation

The water flow continuity equation can be expressed, according to Freeze and Cherry (1979), as follows:

$$\frac{\partial(\rho_w nS)}{\partial t}+\nabla\cdot(\rho_w \mathbf{v}_w)=0 \qquad (1)$$

where: n = porosity, S = degree of saturation, ρ_W = mass density of water, $\mathbf{v}_W$ = $v_w^x\mathbf{i}+v_w^y\mathbf{j}+v_w^z\mathbf{k}$, macroscopic (i.e., rate of flow through a unit area), velocity vector of water; ∇ = $\frac{\partial}{\partial x}\mathbf{i}+\frac{\partial}{\partial y}\mathbf{j}+\frac{\partial}{\partial z}\mathbf{k}$, the divergence operator.

3.1.2 Equilibrium of the soil element

The static equilibrium of a soil element can be expressed in a condensed form, as presented by Lloret et al. 1993, as follows:

$$\frac{\partial\sigma_{ij}}{\partial x_j}+b_i=0 \qquad (2)$$

where: σ_{ij} = total stresses, x_i = the directional system coördinates, b_i = the body forces.

3.2 Constitutive Relationships and Laws of Motion

The displacement vector, u and the water pore-pressure (i.e., u_w) are chosen as the basic variables of the flow/deformation problem. To solve the basic governing equations, a set of constitutive relationships and laws of motion are necessary.

For the soil element equilibrium equation it is necessary to define constitutive relationships linking volume/displacement changes to the stress state variables. Additionally, the shear strength constitutive relationship is necessary in problems where there is a potential risk of failure of the soil. In these cases both a failure criterion and a post-failure behavior have to be defined.

3.2.1 Soil structure constitutive relationship

An stress induced anisotropic formulation for the elasticity form of the constitutive equations for a collapsing soil has been proposed (Pereira 1996, Pereira and Fredlund 1997). By using a xyz-Cartesian system the proposed equations are as follows:

$$\varepsilon_x=\frac{(\sigma_x-u_a)}{E}-\frac{\mu}{E}(\sigma_y+\sigma_z-2u_a)+\frac{(u_a-u_w)}{H_x} \qquad (3)$$

$$\varepsilon_y=\frac{(\sigma_y-u_a)}{E}-\frac{\mu}{E}(\sigma_x+\sigma_z-2u_a)+\frac{(u_a-u_w)}{H_y} \qquad (4)$$

$$\varepsilon_z=\frac{(\sigma_z-u_a)}{E}-\frac{\mu}{E}(\sigma_x+\sigma_y-2u_a)+\frac{(u_a-u_w)}{H_z} \qquad (5)$$

where: H_i = $H/(1+H\chi_i)$, is the elasticity modulus for the soil structure in the i-direction relative to a change in matric suction, H = isotropic elasticity modulus (function of the mean net total stress) for the soil structure relative to a change in $(u_a - u_w)$, χ_i = stress induced anisotropic collapse factor in the i-direction (function of the stress ratios σ_i/σ_j and σ_i/σ_k) for the soil structure relative to a change in $(u_a$

- u_w), i, j, k = directions of a 3-orthogonal coordinate system (e.g., x, y, z).

3.2.2 Water Phase Constitutive Relationship

The constitutive relationship for the water phase defines the water volume change in a soil element for a change in the stress state variables. This constitutive equation can be written as a linear combination of the stress state variable changes as follows (Fredlund et al. 1993):

$$\frac{dV_w}{V_o} = \frac{d(\sigma_x - u_a)}{E_w} + \frac{d(\sigma_y - u_a)}{E_w} + \frac{d(\sigma_z - u_a)}{E_w} + \frac{d(u_a - u_w)}{H_w} \quad (6)$$

where: E_w = water volumetric modulus associated with a change in $(\sigma - u_a)$; H_w = water volumetric modulus associated with a change in $(u_a - u_w)$.

3.2.3 Flow laws and hydraulic properties

Water flow through a saturated/unsaturated soil can be described by a generalized Darcy's law as follows (with z-coordinate as the vertical direction):

$$\mathbf{v}_w = -\mathbf{k}_w \nabla(u_w / \gamma_w + z) \quad (7)$$

For an unsaturated soil the coefficient of permeability can be expressed as a function either of the matric suction (Brooks and Corey 1964) or the volume-mass soil properties (Fredlund and Rahardjo 1993).

3.2.4 Shear strength behavior

The shear strength of an unsaturated soil can be formulated in terms of independent stress state variables (Fredlund et al. 1978). Using these stress variables, the shear strength equation is written as follows:

$$\tau_{ff} = c' + (\sigma_f - u_a)_f \tan\phi' + (u_a - u_w)_f \tan\phi^b \quad (8)$$

where: c' = effective cohesion, $(\sigma_f - u_a)_f$ = net normal stress state on the failure plane, u_{af} = pore-air pressure at failure, ϕ' = angle of internal friction related to the net normal stress, $(u_a - u_w)_f$ = matric suction on the failure plane at failure e ϕ^b = angle indicating the rate of increase of shear strength relative to the matric suction.

4 CONSTITUTIVE MODELLING OF THE COLLAPSING SOIL

Laboratory tests were performed to define the constitutive relationships for the collapsing soil. Soil specimens were statically compacted at dry of optimum and low dry density conditions as compared to the AASHTO compaction energy. The collapsing soil specimens were compacted at a gravimetric water content of 6.5 % (i.e., minus 4 % dry of optimum conditions) and at a dry density of 14.75 kN/m^3.(i.e. 90 %, of the maximum dry density). Table 1 shows the soil properties.

Double-oedometer tests, on the range of vertical stresses from 0 to 800 kPa, were performed on the compacted soil specimens. The soil presented low compressibility when loaded under unsaturated conditions. The soil specimens did not present any collapsing behavior when saturated under vertical stresses lower than 50 kPa. A vertical stress of 100 kPa produced soil collapse amounting to 3.0 %. The volumetric collapse reached about 7.2% and 11 % when loaded under vertical stresses of 200 kPa and 400 kPa, respectively.

Table 1 Index properties of the soil.

Soil	Residual silty sand
Grain size distribution	Sand - 52 % Silt - 35 % Clay - 13%
Atterberg limits	Liquid limit, w_l = 29 Plastic limit, w_p = 17 Plasticity index , PI = 12
Specific Gravity	G_s = 2.64
USCS	SW-SM; well-graded sand with silt

A laboratory program comprising triaxial permeameter and modified direct shear tests allowed the definition of the constitutive soil models (Pereira 1996). The soil models were defined by using best-fit analyses in the search for continuous functional relationships that could capture the essential characteristics of the behavior of the soil as observed from the available data. A superior curve-fitting was obtained by using a five parameter logistic function (Pereira 1996). Equation 9 shows this function in terms of void ratio as a function of matric suction at a given net confining stress.

$$e = e_u + (e_s - e_u) \bigg/ \left[1 + \left(\frac{(u_a - u_w)}{c}\right)^b\right]^a \tag{9}$$

where: e_u = initial void ratio of a soil specimen under a given net confining stress, e_s = void ratio at saturated conditions of a soil specimen under a given net confining stress, c = matric suction value at the inflection point (i.e., middle point of the "collapse" phase), b = slope parameter (i.e., slope of the "collapse" phase), a = the symmetry parameter which makes the logistic function asymmetric.

Equation 10 is the mathematical model for the void ratio state surface of the collapsing soil resulting from the best-fit analysis of the available data.

$$e = e_u + \frac{e_f - e_u}{\left[1 + \left(\frac{(u_a - u_w)}{c}\right)^b\right]} \tag{10}$$

where: $e_u = 0.77 - 0.007ln(\sigma_m)$, $e_f = 0.752 - 0.142 / [1+(\sigma_m/75)^{-3.5}]$; $c = c_1(\sigma_m)^2 + c_2(\sigma_m) + c_3$;, $b = b_1(\sigma_m)^{b2}$; $c_1 = 9.4x10^{-4}$; $c_2 = 7.46x10^{-2}$; $c_3 = 11.0$; $b_1 = 39.0$; $b_2 = -6.10x10^{-1}$; σ_m = net confining stress or mean stress.

Figure 1 illustrates the best-fit model in terms of void ratio versus matric suction relationships under different net confining stresses. The available experimental data are also illustrated in this figure and show a comparison of the best-fit surface obtained.

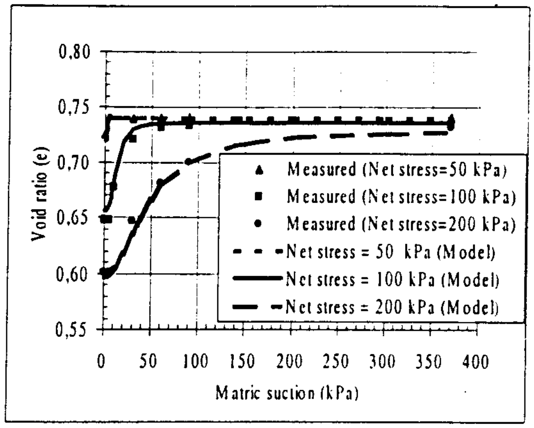

Figure 1. Void ratio best fit modeling.

The void ratio state surface (i.e., Fig. 1) allows to evaluate the soil compressibility which expresses a relationship between the soil elasticity modulus (i.e., E) and the Poisson ratio (i.e., μ). Therefore, a Poisson ratio relationship is required to completely define the mechanical behavior of the soil structure.

A Poisson ratio equal to 0.3 may reflect the as-compacted condition of a loosely compacted soil (Miranda 1988). However, the changes in the structure of a metastable structured soil upon saturation, results in changes in the Poisson ratio (Maswoswe 1985, Pereira 1996).

In this study, a first step consisted in the establishment of a relationship between the Poisson ratio and the mean normal stress of the collapsing soil at saturated conditions. Such a relationship was defined (Pereira 1996) by combining the available triaxial permeameter and double-oedometer test results with the experimental observation that: "the volumetric collapse of a soil mass is truly a function of the acting mean total stress (Lawton et al. 1991)". A given combination of mean total stress and vertical stress allowed an evaluation of the horizontal stress (i.e., σ_h) on the oedometer test. Then, for a given mean confining stress, the Poisson ratio for the saturated collapsing soil (i.e., μ_s), was evaluated as follows:

$$\mu_s = (3\sigma_m - \sigma_v) / (\sigma_v + 3\sigma_m) \tag{11}$$

where:

$$\sigma_m = (\sigma_v + 2\sigma_h)/3, \; \sigma_h = [\mu/(1-\mu)]\sigma_v$$

At unsaturated conditions it is assumed that the Poisson ratio increases with the soil collapse. This implies an increase of the Poisson ratio of the collapsing soil when the matric suction decreases (Maswoswe 1985). In this study, the same relationship used to simulate the soil collapse versus matric suction (i.e., Eq. 10) is used to simulate the change in Poisson ratio in response to a change in matric suction. This assumption implies a variation of the Poisson ratio of the collapsing soil from an initial value of 0.3 (i.e., at the as-compacted conditions) to a value calculated, by using Eq. 11, when the soil reaches saturated conditions. Therefore, the Poisson ratio for the collapsing soil under unsaturated conditions is calculated as:

$$\mu = 0.3 + (\mu_s - 0.3) \Big/ \left[1 + \left(\frac{(u_a - u_w)}{c}\right)^b\right] \quad (12)$$

where: $\mu_s = 0.092 \; Ln \; (\sigma_m) - 0.021$, is the Poisson ratio for the saturated soil (from Eq. 11).

In Eq. 12 the parameters "*b*" and "*c*" are the same as used in Eq. 10. Such a procedure maintains a proportionality between the Poisson ratio and the wetting-induced soil collapse.

The anisotropic factors χ_i's (i.e., χ_x, χ_y, and χ_z) were defined in a trial and error process using the computer program COUPSO and the available double-oedometer tests. This trial and error procedure consisted of the reproduction of the wetting-induced soil specimen collapse under Ko-conditions and under an applied vertical stress of 100 kPa. In this process, equal values were assumed for the horizontal-direction factors (i.e., χ_x and χ_z), simulating the confining oedometric conditions. The closed-form relationship χ_y equal to minus ($\chi_x + \chi_z$) defined the anisotropic factor in the vertical direction. A value of minus 1.95 was calculated for the anisotropic factors χ_x and χ_z from the above trial and error procedure (Pereira 1996, Pereira et al. 1997).

The degree of saturation state surface was also defined using a best-fit analysis (Pereira 1996). The logistic function provided the best-fit results of the available data. The mathematical model obtained is expressed as follows:

$$S = S_0 + (1 - S_0) \Big/ \left[1 + \left(\frac{(u_a - u_w)}{c}\right)^d\right] \quad (13)$$

where: $S_0 = 0.375$; $a = 0.354$; $b = 3.65*10^{-3}$; $c = 20$; $d = .977$.

Figure 2 shows the resulting best-fit model in terms of the degree of saturation versus matric suction relationship under different net confining stresses.

In terms of soil permeability to water versus stress state variables, Brooks and Corey's (1964) equation provided a satisfactory best-fitting relationship for the collapsing soil. Equation 14 expresses the best-fit mathematical equation for the soil permeability to water.

$$k_w = k_p \left(\frac{\psi_{cr}}{(u_a - u_w)}\right)^{\lambda} \quad (14)$$

where: $k_w \le k_s$; $k_p = -1.39\text{x}10^{-7} + 6.26\text{x}10^{-8} \; \text{Ln}(\sigma_m)$; $k_s = 1.17\text{x}10^{-6} - 1.8\text{x}10^{-7} \; \text{Ln}(\sigma_m)$, is the saturated coefficient of permeability; $\psi_{cr} = 3.0$; $\lambda = 2.10$.

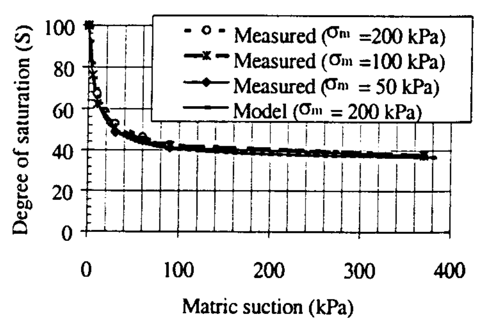

Figure 2. Degree of saturation best-fit modeling.

Figure 3 shows the best-fit results for the collapsing soil in terms of permeability to water versus matric suction relationship for different net confining stresses. The figure also shows the available data for the net confining stress of 200 kPa, in order to show the accuracy of the predicting model.

Equation 15 expresses the resulting best-fit mathematical model for the extended Mohr-Coulomb failure envelope for the collapsing soil (Pereira 1996).

$$\tau_{ff} = a_1 + b_1(\sigma - u_a) + c_1(u_a - u_w) + d_1(\sigma - u_a)(u_a - u_w)^p \quad (15)$$

where: $a_1 = -7.89$, $b_1 = 0.194$, $c_1 = 0.324$; $d_1 = 0.093$; $p = 0.043$.

Equation 15 can be seen as a phenomenological prediction of the shear strength envelope of the collapsing soil for the range of matric suction from 0 to 100 kPa. Additionally, the study assumes that at a given net normal stress, the shear strength of the collapsing soil remains constant for the range of matric suctions from 100 to 370 kPa. Such assumption is based on the fact that for matric suctions higher than the corresponding to the residual degree of saturation an unsaturated soil presents an angle ϕ^b which tends to zero (Pereira 1996). Figure 4 shows the shear strength modelling for the collapsing soil.

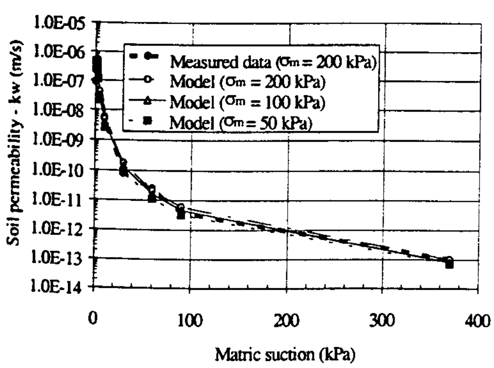

Figure 3. Permeability of the collaping soil.

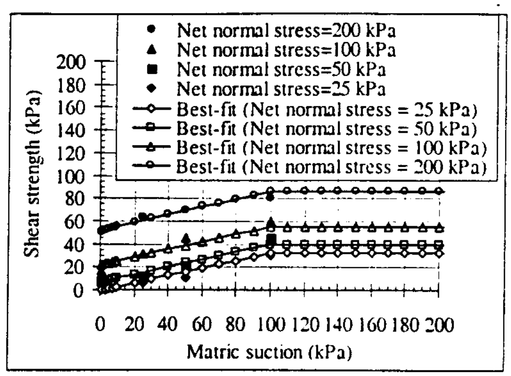

Figure 4. Shear strength envelope modelling.

5 POST-FILLING PERFORMANCE OF AN ALKA-SELTZER DAM

The computer program COUPSO (Pereira 1996), developed by using the finite element method, was utilized to analyze the post-filling performance of a small collapsing dam similar to those constructed in Northeast Brazil.

5.1 Description of the problem

Figure 5 shows a section of a small dam (i.e., h < 10.00 meters) typically constructed in northeast Brazil. Such dams are constructed as homogeneous embankments, often without internal drainage.

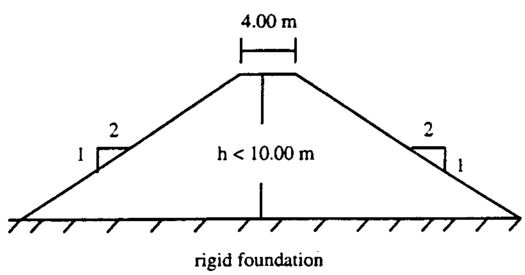

Figure 5 Typical cross section of a small dam in Northeast Brazil..

In 1983, the State of Ceara', the Department of Roads created a task force of civil engineers to study the consequence of frequent small dam failures on the safety of the state roads. Such dams were reported to fail in a short time after their first reservoir filling. The commission's report (DAER, 1983) stated:

a.) "during the drought of 1979 through 1983, about 20,000 small dams were built, enlarged, or rehabilitated in the State of Ceara' by the Emergency program";
b.) "the commission examined 720 dams and concluded that about 80 percent were going to fail in the next rainy season";
c.) about the quality of the construction: "The compaction of the material is, in general, very deficient, almost always without use of water";
d.) explaining why the dams were constructed with a deficiency of water: "The difficulty to provide water to satisfy the most elementary necessities of the people did not permit the use of such a precious liquid in the construction of dams".

5.2 Analysis procedure

The initial conditions for the post-filling phase of the homogeneous embankment were defined as follows:
a.) The net normal stress distributions corresponding to the end-of-construction phase and first impounding of the reservoir were calculated.
b.) The initial matric suction of the compacted soil in the entire cross-section was assumed to be equal to 370 kPa.

The post-filling performance of the small dam is simulated in a transient process where water flows through the homogeneous embankment according to a defined time discretization. In each time step the displacements, the water pore-pressures, and the stresses in the dam are evaluated.

The shear strength mobilized at internal points of the dam is calculated as follows:

$$SMOB = \left(\frac{(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma)_f} \right) \tag{16}$$

where: $SMOB$ = shear strength mobilization; $(\sigma_1 - \sigma_3)$= acting deviatoric stress; $(\sigma_1 - \sigma_3)_f$= $2*(c*\cos\phi+\sigma_3*\sin\phi)$ is the deviatoric stress at failure; $\phi = \partial\tau_{ff}/\partial\sigma = \tan^{-1}[b_1+d_1*(u_a-u_w)^n]$ is the friction angle (from Eq. 15); $c= a_1+c_1*(u_a-u_w)$ is the cohesion of the soil (from Eq. 15).

The shear strength mobilized (i.e., SMOB) is used as a criterion to assume post-failure behavior at internal points of the dam. Different post-failure behaviors are used to define the collapsing soil conditions in the saturated and unsaturated portions within the dam. A high compressibility is assumed for a soil element which reaches failure within the dam (Duncan et al. 1970). Such an assumption is consistent with the "perfectly" plastic behavior of the saturated collapsing soil when sheared under low octahedral stress. In turn, it is assumed that an unsaturated soil element at failure conditions keeps its compressibility behavior as a function of the applied stress state. Such an assumption allows an unsaturated soil element to increase its shear strength by increasing its mean net normal stress when collapsing (Maswoswe 1985). This assumption reflects a more realistic approach to the "hardening" shear strength behavior of an unsaturated collapsing soil specimen (Pereira 1996). Besides, such an assumption presents two advantages. The first one is that the concept of void ratio state surfaces remains valid for the unsaturated collapsing soil at failure. The second advantage is that numerical difficulties are avoided by maintaining the continuity of the void ratio state surface (Alonso et al. 1985).

5.3 Analysis of an "Alka-Seltzer" dam

The analysis traced the stresses, the displacements, and the pore pressures within the dam throughout construction, reservoir-filling, and transient seepage until a characterization of the dam failure had been established. Pereira (1996) presents the finite element mesh (90 elements and 407 nodes) used in the analysis. A quadrilateral Lagrangian element with 9 nodes was utilized in the finite element model.

The following three phases were considered in the analysis: the construction phase, the reservoir filling-phase, and the transient unsaturated-saturated seepage through the dam after the first reservoir filling. The analyses were performed in such a way that the final stress state conditions of one phase formed the initial stress state conditions for the subsequent phase. However, the displacements were not considered in a cumulative manner and each phase had an initial configuration based on the initial geometry of the small dam. Emphasis is given to the transient seepage phase since the construction and first reservoir-filling phases illustrate that an "Alka-Seltzer" dam presents a satisfactory structural stability for the construction and first reservoir filling phases (Pereira 1996).

After the dam has been constructed, its reservoir is filled to an elevation of 8.0 meters. In the analysis, it is assumed that the water is raised to the full supply level in a short period of time.

Figure 6 shows the shear strength mobilized within the dam after the reservoir filling phase. This figure shows that the shear strength mobilized reflects satisfactory stability of the dam.

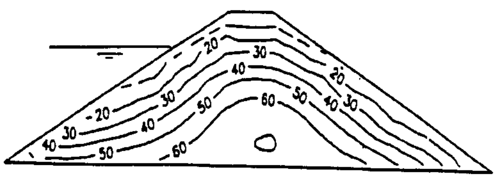

Figure 6 Shear strength mobilized (%) distribution within the dam after the reservoir-filling phase.

Figure 7 shows the mean net normal stress (i.e., σ_m - u_a) distribution within the dam immediately after the reservoir-filling phase. The small values of mean net stress (i.e., values less than 70 kPa) within the dam reflect the combined effect of the low dry density and low Poisson ratio of the collapsing soil. These values also suggest that upon saturation, higher collapse deformations should be expected at the central part of the dam due to the higher values of mean net stress there existent. Increase in self weight of the collapsing soil due to saturation contributes to further collapse deformations within the dam.

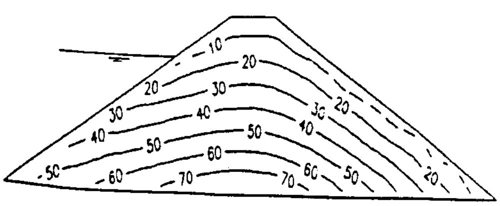

Figure 7 (σ_m - u_a) (kPa) distribution within the dam after the reservoir filling phase.

The reservoir was assumed to remain at an elevation of 8.0 meters during the transient-water-flow analysis. The small dam was considered to be anisotropic with a horizontal coefficient of permeability equal to 10 times the vertical coefficient of permeability.

The transient seepage analysis was performed in accordance with a time discretization consisting of time steps varying from 0.70 to 0.20 days depending upon the convergence requirements. The numerical analysis was carried out to simulate a period of about 150 days. This time proved to be sufficient to characterize the mechanical behavior of an "Alka-Seltzer" dam after the first reservoir filling.

Immediately after the reservoir filling, the water load on the upstream slope created a downstream and downward movement within the upstream zone of the dam. At the transient seepage phase, the displacement pattern gradually changes as a combination of the following effects.

a.) The seepage forces spreading within the saturated zone of the dam induce downstream movements within the dam.

b.) The increase in the soil self-weight due to saturation.

c.) The unsaturated soil undergoes a collapsing behavior for mean net normal stresses higher than 30 kPa.

d.) The buoyant uplift pressures acting within the saturated zone of the dam.

e.) A soil element within the dam suffers a substantial reduction of its shear strength in response to a decrease in its matric suction.

f.) Load transfer between adjacent elements of different stiffnesses during the transient seepage through the collapsing porous medium.

The above effects occur simultaneously in a coupled stress and flow analysis. Therefore, a collapsing dam presents a complex behavior during the transient water seepage. Pereira (1996) shows results in terms of pore pressures, displacements, stresses, and shear strength mobilization within the dam; corresponding to periods of 30 days, 55 days, 100 days and 145 days after the reservoir filling of an "Alka-Seltzer" dam. However, this paper concentrates only on the results at 145 days after the first reservoir filling.

5.4 Dam at 145 days after first reservoir filling

Figures 8 shows the pore-water pressure distribution within the dam 145 days after its first reservoir filling. At this stage the upstream slope of the dam has been saturated and the upper part of the phreatic line is near the downstream slope face. As previously discussed, this pattern is a consequence of the hydraulic anisotropy of the soil combined with the high hydraulic gradient which is driving the transient water flow through the dam embankment.

Figure 8 indicates a critical condition for the dam stability since the water can emerge from the top and downstream face of the dam for further advances of the transient water flow.

Miranda (1988) reports that failure of Alka-Seltzer dams is related to piping and hydraulic fracturing within the dam embankment. Such a statement was based on information from local people living in areas where an "Alka-Seltzer" dam had reached failure conditions. In summary, it had been reported that the water had emerged from the downstream slope and after a short time the entire dam was being carried out, as a mudflow, by the running water. The present research study has demonstrated that the above condition is fairly depicted in Figure 8.

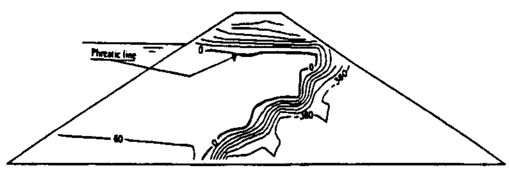

Figure 8 Pore-water pressure (kPa) distribution 145 days after first reservoir filling.

Figure 9 shows the displacement pattern within the dam 145 days after its first reservoir filling. This stage reflects the complete failure of the upstream slope of the dam embankment. At this stage, the lower central part of the dam has collapsed due to saturation and the upstream slope is sliding towards the dam reservoir. The unsaturated zone remains at high matric suctions except in a narrow zone nearby the phreatic line.

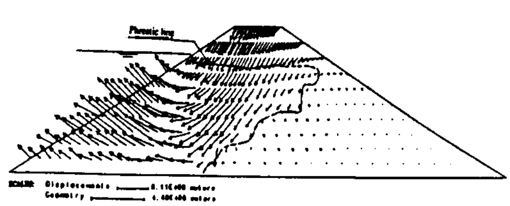

Figure 9 Displacement distribution 145 days after first reservoir filling.

Figure 10 presents the distribution of shear strength mobilized within the collapsing dam 145 days after its first reservoir filling. At this stage, the failed zone within the dam has been spread throughout the saturated zone. Such a spreading is a combined effect of the reduction of matric suction and the decrease of minor principal stress within the saturated zone. An area of the middle and saturated part of the dam retained values of shear strength mobilized less than 100 percent. Such an area has its movement pattern governed by the surrounding failed zone. The unsaturated zone suffered an increase in shear strength mobilized within the central part of the dam. This increase reflects the increase of major net normal stresses in that zone due to the load transfer previously discussed. The downstream slope of the dam suffered minor changes of shear strength mobilized due to the high values of matric suction existing there. There is a sharp decrease in shear strength mobilized from the phreatic line towards the downstream zone of the dam. In terms of stability, the current stage has demonstrated the failure of an "Alka-Seltzer" dam during its first reservoir filling. The failure mechanism was triggered by the low shear strength and the high compressibility of the collapsing soil at saturated conditions. The upstream slope has reached failure conditions and presented a pronounced sliding towards the dam reservoir before steady state conditions had been reached.

The analysis indicates positive effective stresses throughout the saturated part of the dam, except for a narrow zone near the upstream face. Therefore, there is no risks of hydraulic failure in the cross section direction of an "Alka-Seltzer" dam.

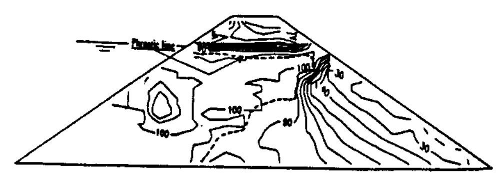

Figure 10 Percent of shear strength mobilized distribution 145 days after first reservoir filling.

Figure 11 shows the minor stress distribution within the dam at this stage. This figure illustrates that in response to the upstream slope sliding of the dam, tensile stresses at the upper part od the dam within both saturated and unsaturated zones occur. This might result in vertical cracking of the top downstream part of the dam.

Figure 12 shows the mean stress distribution within the dam at the current stage. This distribution presents an increase of mean net normal stress within the central part of the dam. Such an increase reflects the load transference from the highly compressible and saturated zone to the rigid unsaturated zone. The general pattern predicts additional soil collapse within the central part of the dam, especially the lower half, for subsequent advances of the transient water flow into the dam.

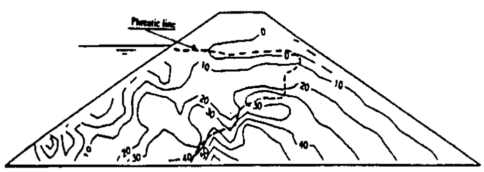

Figure 11 σ_3 (kPa) distribution 145 days after first reservoir filling.

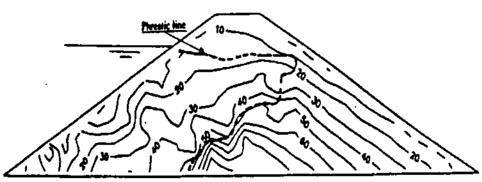

Figure 12 σ_m (kPa) distribution 145 days after first reservoir filling.

6 CONCLUSIONS

The following conclusions can be drawn from the present research study:

1. Fredlund's and Rahardjo's (1993) theory for consolidation of unsaturated soils in its more generalized form (i.e., introducing an stress-induced anisotropic behavior of an unsaturated soil element in response to a change in its matric suction), can be used to simulate the stress-strain behavior of a collapsing soil during saturation. The mechanical behavior of a stable soil is a special case of this more generalized theory .

2. COUPSO is sufficiently versatile to allow the incorporation of the most relevant theoretical aspects involved in the consolidation of both saturated and unsaturated soils. As a unique characteristic, it presents the stress-induced anisotropic soil behavior to allow for the simulation of the stress-strain behavior of a metastable-structured soil during saturation.

3. Results obtained from COUPSO demonstrate the structural instability of an "Alka-Seltzer" dam during transient seepage flow following the first reservoir filling of the reservoir. The analysis has indicated a progressive failure associated with a sliding of the upstream slope of the dam in response to the water advance into the embankment. The results reflect the combined effect of the decrease in shear strength and the collapsing behavior of the metastable-structured soil in response to a decrease in matric suction.

4.) These analysis procedures may be used to predict pore-water pressures, stresses and movements in small collapsing dams during any stage after the beginning of construction of the dam. Perhaps the greatest value of these procedures is in connection with instrumentation studies.

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REFERENCES

Alonso , E. E., Battle, F., Gens. and A., Hight, D. W. (1985). Special Soil Problems - General Report, Proc. 9th ECSMFE, Dublin, pp. 1087-1146.

Brooks R. H. and Corey, A. T. (1964). Hydraulic Properties of Porous Media. Colorado State Univ. Hydrol. Paper, No.3, 27 p.

Duncan, J. M. and Chang, C. (1970). Nonlinear Analysis of Stress and Strain in Soils. Journal of Soil Mech & Fdns. Div., ASCE, vol. SM5, pp. 1629-1653.

Fredlund, D. G. and Morgenstern, N. R. (1977). Stress state Variables for Unsaturated Soils. ASCE J. Geotech. Eng. Div., vol. 103, GT5, pp. 447-466.

Fredlund, D. G., Morgenstern, N. R. and. Widger, R. A. (1978). Shear Strength of Unsaturated