ASSESSMENT OF QUICKLIME PILE BEHAVIOUR

A. HOLEYMAN, Assistant Manager, Research and Geotechnics, S.A. FRANKI N.V., Liège, Assistant Professor, Free University of Bruxelles, Belgium
J.K. MITCHELL, Professor and Chairman, Department of Civil Engineering, University of California, Berkeley, CA, U.S.A.

The authors present a mathematical model that describes the physical phenomena involved in the stabilization of soil masses with the help of pure quicklime piles. The finite-difference one-dimensional formulation makes provision for the speed of the slaking of the quicklime pile, the expansion of the column, the development of lateral pressures in the soil and the subsequent consolidation of the soil in the radial direction. Physical parameters are deduced from tests published by Kuroda et al (6). The parametric analysis conducted substantiates the importance of the horizontal permeability of the soil being treated, the fineness of the lime used and its condition of placement.

INTRODUCTION

Lime treatment in one of the oldest and most widely used types of admixture stabilization for soils. Its value for improvement of road subgrades and bases has been appreciated for centuries. The value of quicklime for the rapid strengthening of soft, saturated clays has become apparent more recently. Mixed-in-place lime piles or columns (1) have been developed in Sweden and Japan for support of structures and open excavations. Yamanouchi et al. (2) have described a sandwich method for quicklime stabilization of soft clay used in embankment construction.

A special type of quicklime pile involves the placement of quicklime directly in boreholes, rather than mixing the lime with the surrounding clay. Loose lime can be compacted or, alternatively, columns can be formed by stacking discs of compacted quicklime in the hole.

Although the technique has been practised, there is little detailed information on the mechanisms of the stabilization and up to now no definite analyses of the significant phenomena. In this paper, the physics of the stabilization phenomena are described, a physical model is formulated and a mathematical solution is presented. Some computations are presented according to this method to show the influence of significant parameters.

PHYSICS

When quicklime (CaO) and moist soil are brought into contact, several processes can take place that will affect the properties of the soil :

1. Absorption of water from the soil to hydrate the quicklime and fill its voids.
2. Heating caused by the hydration of the lime.
3. Expansion of the lime upon hydration, which results in compression of the soil in the horizontal direction.
4. Hardening of the annular zone around the pile into which there has been diffusion of lime.

The process occur concurrently, and their complete analysis is complex. Therefore, some assumptions are necessary.

Considering that the migration of lime through intact clay is a very slow process (of the order of 1 cm/year), we shall not consider process 4 as playing a significant role in the stabilization, at least during the first year.

The hydration of the quicklime is expressed by the chemical reaction (3) :

$$CaO + H_2O \longrightarrow Ca(OH)_2 + 15.5 \text{ kcal.}$$

The ground temperature rises adjacent to the pile in response to the heat generation. The temperature gradient will depend on the speed of the reaction in relation to the specific heat and the thermal conductivities of the soil and the lime. This can result in increased water evaporation, provided there is some kind of ventilation. Since evaporation is possible only at the ground surface, and since the hydration of a lime pile typically takes 2 to 10 days, as will be seen later, we shall consider that the rise in temperature will have a limited impact on the overall water content of the soil to be stabilized.

Expansion accompanying quicklime hydration can be estimated. By calculating the stoichiometric quantities, we find :

$$56g\ CaO + 18g\ H_2O \longrightarrow 74g\ Ca(OH)_2 \qquad (1)$$

Solid specific gravities are 3.37 for the quicklime and 2.343 for the slaked lime, respectively. Thus, we obtain the solid volumes combination :

$$17cc\ CaO + 18cc\ H_2O \longrightarrow 31.6cc\ Ca(OH)_2 \qquad (2)$$

which means that the solid phase volume increase is about 85 %.

The relative volume of water necessary to hydrate the corresponding volume of in-situ quicklime can be derived from equation (2), once the porosity of the lime placed in the soil and the yield of the reaction are known. Assuming a compacted dry unit weight of 11280 N/m^3 and a chemical efficiency of 93 % one gets a relative volume of water needed equal to 0.30.

Experiments (6) show that under atmospheric conditions, an apparent volume V of quicklime and corresponding volume (0.3V) of water combine into an apparent larger volume (1.5 to 1.7 V) of slaked lime with a dry unit weight of 10000 to 8500 N/m^3. This net volume increase is of crucial importance, because the tendency for volume expansion will result in lateral compression of the soil, which will, in turn, build up pore pressures. As these pressures dissipate, more water will be expelled from the soil into the draining lime, which will then expand further, until all the quicklime has hydrated. We assume that the slaked lime pile acts as a vertical drain for analysis of the consolidaton resulting from the lateral pressure.

Based on the previous discussion, we consider that the problem can be treated as one of a cylindrical drain, the tentative expansion of which is controlled by the amount of drained water.

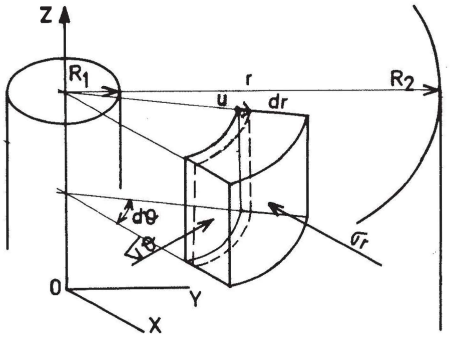

Figure 1 : Cylindrical coordinates

PHYSICAL MODEL

Stiffness of the surrounding soil

As the problem has cylindrical symmetry, we shall use the cylindrical coordinates (fig.1) r, θ and z. $\bar{r}$ and $\bar{\theta}$ are the principal directions.
Equilibrium in the radial direction gives :

$$\partial \sigma_r / \partial r + (\sigma_r - \sigma_\theta)/r = 0 \qquad (3)$$

in which σ_r and σ_θ are the principal radial and tangential stresses.
The corresponding strains ε_r and ε_θ are related to the radial displacement u as follows:

$$\begin{cases} \varepsilon_r = \dfrac{\partial u}{\partial r} \\ \varepsilon_\theta = \dfrac{u}{r} \end{cases} \qquad (4)$$

Considering the problem of an infinitely long lime pile, we shall assume a plane strain condition : $\varepsilon_z = 0$, thus the volumetric deformation ε_v is equal to :

$$\varepsilon_v = \varepsilon_r + \varepsilon_\theta \qquad (5)$$

We shall further assume that the stresses and deformations are related by the theory of elasticity, the elastic behaviour of the material being fully described by :

E : Young's modulus
ν : Poisson's ratio

$$\begin{cases} \sigma_r = (\lambda + 2G)\varepsilon_r + \lambda\, \varepsilon_\theta \\ \sigma_\theta = (\lambda + 2G)\varepsilon_\theta + \lambda\, \varepsilon_r \end{cases} \qquad (6)$$

with :

$$\begin{aligned} G &= E/2\,(1+\nu) \\ &= E\nu/(1+\nu)(1-2\nu) \end{aligned} \qquad (7)$$

With this hypothesis, the equilibrium condition (3) becomes a differential equation of the radial displacement U(r) or u (4).

$$\frac{\partial^2 u}{\partial r^2} + \frac{1}{r}\frac{\partial u}{\partial r} - \frac{u}{r^2} = 0 \qquad (8)$$

The integration constants are determined by the boundary conditions. Assuming a regular hexagonal pattern of the locations of the piles, we can consider that the mid-point between two adjacent piles will not move and will represent the radius of influence of a pile. Let R_2 be that radius and R_1 the initial radius of the lime piles. The boundary conditions become :

$$\begin{cases} U(R_2) = 0 \\ U(R_1) = U_1 \end{cases} \qquad (9)$$

in which U_1 is the radial expansion of the cavity. The solution to equation (8) can be shown to be :

$$U = U_1 \cdot \frac{r}{R_1} \cdot \frac{1 - R_2^2/r^2}{1 - R_2^2/R_1^2} \qquad (10)$$

The radial stress at the inner cavity can be obtained from equation (6) :

$$\sigma_r(R_1) = \frac{U_1}{R_1} \cdot \frac{2\,(\lambda + 2G)}{1 - R_2^2/R_1^2} = 2K_1 \cdot U_1/R_1 \qquad (11)$$

in which K_1 is defined as $K_1 = (\lambda + 2G)/(1 - R_2^2/R_1^2)$
The volumetric deformation can also be obtained using equations (5) and (4) :

$$\varepsilon_v = \frac{u}{r} + \frac{\partial u}{\partial r} = \frac{U_1}{R_1} \cdot \frac{2}{1 - R_2^2/R_1^2} \qquad (12)$$

One notices that for a given value of U_1, the volumetric strain is constant throughout the soil ring. Substituting U_1/R_1 from equation (11), the volumetric strain of the soil is :

$$\varepsilon_v = \frac{\sigma_r(R_1)}{K_1} \cdot \frac{1}{1 - R_2^2/R_1^2} \qquad (13)$$

Comparing the volume of the cavity V_c to the volume of the soil ring V, one obtains :

$$V/V_c = -(1 - R_2^2/R_1^2) \qquad (14)$$

Since the volume decrease of the soil ring is equal to the volume increase of the cavity, the volumetric deformation of the cavity ε_{vc} is given by the simple relation

$$\varepsilon_{vc} = \varepsilon_v \cdot \frac{V}{V_c} = -\sigma_r(R_1)/K_1 \qquad (15)$$

Thus K_1 is the stiffness coefficient relating the radial stress at the interface of the lime and the soil to the volumetric deformation of the cavity.

Under drained conditions, K_1 will correspond to the final stage of the equilibrium between the lime expansion and the soil reaction. If we call D the horizontal constrained modulus of the soil or the inverse of the coefficient of volume change as measured in a consolidation test (5), we find that :

$$K_1 = \frac{E\,(1-\nu)}{(1+\nu)\,(1-2\nu)} \cdot \frac{1}{1 - R_2^2/R_1^2} = \frac{D}{1 - R_2^2/R_1^2} \qquad (16)$$

Since the strength of the soil is limited, we might expect that a plastic zone develops from the inner cylinder towards the outer boundary as the radial stress $\sigma(R_1)$ increases.

If we assume the following simple failure criterion :

$$\sigma_r - \sigma_\theta = q_u \qquad (17)$$

corresponding to a purely cohesive soil, in which q_u is the unconfined compression strength, the fundamental equation (3) becomes :

$$\partial\sigma_r/\partial r + q_u/r = 0 \tag{18}$$

which leads to :

$$\sigma(r) = \sigma_{Ru} + q_u . \ln \frac{R_u}{r} \tag{19}$$

in which σ_{Ru} is the radial stress for the radial distance R_u, which defines the extent of the plastic zone.

Expressing that, at the inner boundary of the elastic zone which is a cylinder of radius R_u, the failure condition (17) is reached, and calling U_r the corresponding radial displacement, one gets the relationship between q_u and R_u :

$$\frac{q_u}{2G} = \frac{U_r}{R_u} \cdot \frac{2}{1 - R_2^2/R_u^2} \cdot \frac{R_2^2}{R_u^2} \tag{20}$$

calling $R_u/R_2 = \alpha$, and substituing for U_r/R_u, from equation (11), which becomes $\sigma_{Ru} = 2K_1.U_r/R_u$, one obtains :

$$\frac{\sigma_{R_u}}{q_u} = \frac{1 - \nu}{1 - 2\nu} \cdot \alpha^2 \tag{21}$$

The complete radial stress distribution can be worked out in the following way : by assuming a value of α , one obtains the value of σR_u through the relationship (21). The solution is obtained in the outer elastic region with the equation :

$$\frac{\sigma_r}{\sigma_{R_u}} = \frac{1 + G(1 + R_2^2/r^2)}{\lambda + G(1 + R_2^2/R_u^2)} \tag{22}$$

and in the inner plastic region using equation (19), which gives the plastic radial stress as a function of the radius of the cavity.

Fig.2 gives two examples of the radial stress distribution and comparisons with the elastic solution, thus not taking any strength criterium into consideration. It can be seen from these examples that when the value of α varies from 0.4 to 0.8, the radial stresses at the cavity, for this case where $R_1/R_2 = 0.2$, differ by the same amount ($\Delta_{p\ell} = \Delta_{c\ell}$) whether shear failure is taken into consideration or not.

If we further assume that the soil undergoes plastic deformation at zero volume change, it means that the radial displacement U_1 will be equal to that corresponding to the elastic problem. It can therefore be concluded that within the range of the plastic zone extension mentioned, the elastic stiffness coefficient K_1 is not significantly affected by the inclusion of a failure criterion in the analysis.

Expanding cylinder

Let us schematize the situation as follows : (fig.3).

Initially, the soil and the quicklime are at a zero stress condition. As a part of the available lime reacts, it expands. If we call C_1, the volumetric expansion under atmospheric conditions and μ the proportion of the quicklime which has reacted to the initial volume, we can say that :

$$\Delta V_f = C_1 \cdot \Delta\mu \tag{23}$$

if ΔV_f is the free volume expansion corresponding to the variation of μ.

In fact, due to the restraint provided by the soil, the effective volume variation will be smaller. If we call :

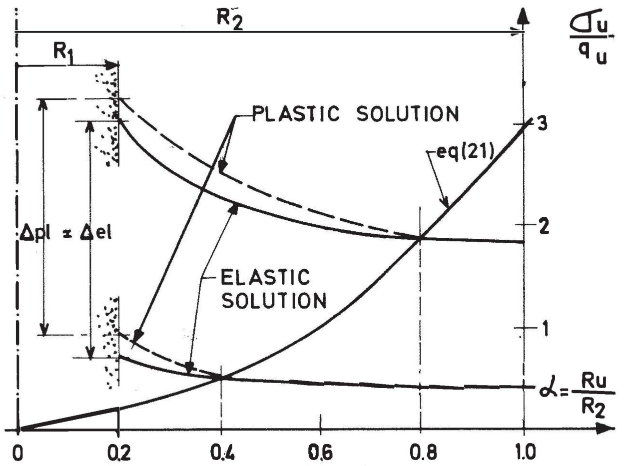

Figure 2: Comparison between elastic and elasto-plastic solutions.

K_1 : volumetric stiffness coefficient = $\frac{\Delta\sigma_r}{\Delta V/V}$ (see eq.15)

C_3 : compressibility of the quicklime = $\frac{\Delta VCaO/VCaO}{\Delta\sigma_r}$

C_5 : ratio of the compressibility of the slaked lime to the quicklime.

We find the equilibrium incremental pressure :

$$\Delta\sigma_r = \frac{C_1.\Delta\mu}{C_3\,[1 + \mu_p.C_1 + (1 - \mu)C_5] + 1/K_1} \tag{24}$$

in which μ_p is the ratio of the actual expansion to the unrestrained expansion at the same stage of the hydratior reaction.

If we assume that the increase in radial pressure at any time can be expressed by :

$$\Delta\sigma_{r,t}(\mu) = \int_0^{\mu} \frac{\Delta\sigma_r}{\Delta\mu} \cdot d\mu \tag{25}$$

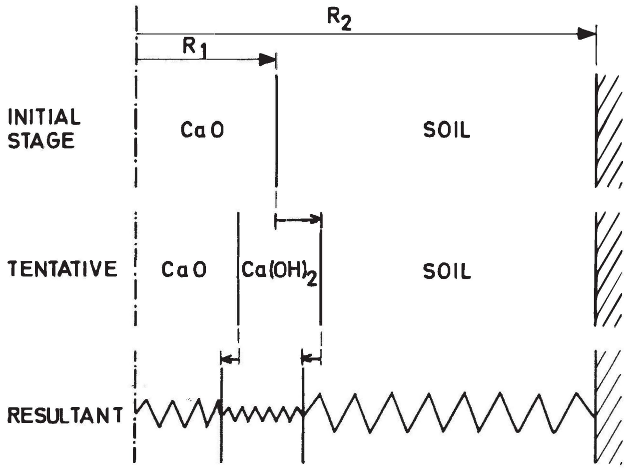

Figure 3 : Expansion of the Cylinder

we know at that time the total stress increment at the interface between the lime pile and the surrounding soil and also the resulting expansion, $\mu_p C_1$.

Equations (24) and (25) have been applied to the results of experiment conducted by Kuroda et al (6) on pure lime which was allowed a set volumetric expansion : $\mu_p C_1$. When the lime is not in contact with the fixed boundary $K_1 = 0$ and $\Delta P_e = 0$ and when it is, $1/K_1 = 0$.

The resulting pressure P_e is derived from equations (24) and (25) :

$$P_e = \int_{\mu_p}^{1} \frac{C_1 / C_3}{1 + \mu_p . C_1 + (1 - \mu) C_5} \cdot d\mu$$

$$= \frac{C_1 / C_3}{C_1 \mu_p + 1 - C_5} \, \ell n \, \frac{C_1 \mu_p + 1}{C_5 + \mu_p (C_1 \mu_p + 1 - C_5)} \qquad (26)$$

The value of C_5 should be a constant for any type of lime whereas C_1 and C_3 depend on the grinding and on the compaction. C_5 was chosen on the basis of relative pressures developed at various values of $\mu_p . C_1$.

For the experiment described by Kuroda et al (6), the pressure generated after 2 hours corresponded to $C_1 = 0.7$ and $C_3 = 1 \text{ MPa}^{-1}$; whereas another type of lime, reported in the same paper, yielded the values of $C_1 = 0.5$ and $C_3 = 2 \text{ MPa}^{-1}$ (see fig.4). It can be seen that the agreement is good between the values predicted according to the theory developed herein and the measured values.

As $\Delta\sigma_{r,t}(\mu)$ is a radial stress increase due to the expansion of the lime pile, we know the effective stress state towards which the soil is moving, when it is fully drained, if the value of K_1 is that corresponding to drained conditions.

The pore pressure generation Δp_w corresponding to the radial stress increment must be known in order to analyze the tendency of the water to flow into the lime. A general expression for the pore pressure for our system is :

$$\Delta p_w = \Delta\sigma_\theta + A (\Delta\sigma_r - \Delta\sigma_\theta) \qquad (27)$$

in which A is the ratio of the pore pressure generation to the deviatoric stress increment.

For the consolidation test ($\varepsilon_\theta = 0$), A = 1 whereas for the triaxial test ($\varepsilon_\theta \neq 0$), A = 1/3 according to elastic theory. We assume that an expanding lime pile is an intermediate condition and A = 1/2 :

$$\Delta p_w = (\Delta\sigma_r + \Delta\sigma_\theta)/2 \qquad (28)$$

This hypothesis leads to the conclusion that the pore pressure generation is proportional to the volume incremental deformation, and is constant throughout the soil ring : using elastic theory, it can be shown that

$$\Delta p_w = \Delta\sigma_{R_1} / 2(1 - \nu) \qquad (29)$$

Assuming that $\nu = 0.5$ in undrained conditions, this leads to a simple form of the equilibrium condition :

$$\Delta p_w = \Delta\sigma_{R_1} = \Delta\sigma_{r,t}(\mu) \qquad (30)$$

Radial consolidation

The pore pressure which will be generated through uniform increments corresponding to the variation of the boundary conditions (eq. 30) will dissipate as water is expelled towards the pile. To examine this problem, we shall assume again that the soil is elastic, and that the vertical strain $\varepsilon_z = 0$.

Darcy's law is applicable to describe the horizontal

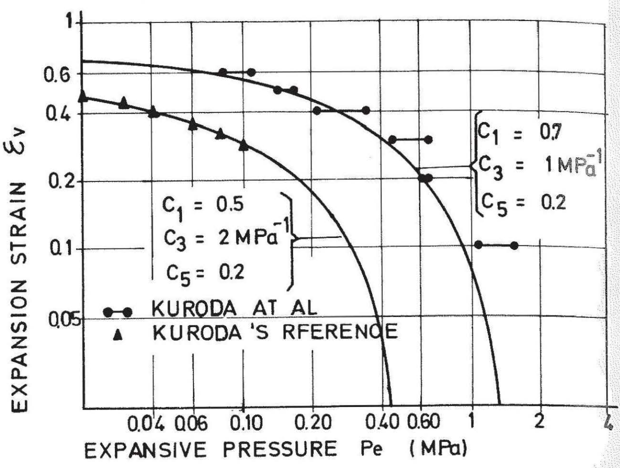

Figure 4 : Relation between the expansive pressure and the expansion strain.

velocity V_h of water upon a radial gradient $i = \partial h / \partial r$, h being the piezometric level of the water.

$$V_h = K_h . i$$

K_h = hydraulic conductivity in the horizontal direction

If the volume of water expelled during a time increment corresponds to the volume decrease of the soil element it is flowing in and out of, one obtains the equation of radial consolidation :

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} . \frac{\partial h}{\partial r} = \frac{1}{K_h} . \frac{\partial}{\partial t} (\varepsilon_\theta + \varepsilon_r) \qquad (32)$$

Since $\varepsilon_r + \varepsilon_\theta$ is constant throughout the soil ring, the equation becomes similar to that corresponding to the vertical consolidation resulting from radial drainage :

$$\frac{\partial^2 h}{\partial r^2} + \frac{1}{r} . \frac{\partial h}{\partial r} = \frac{1}{C_h} . \frac{\partial h}{\partial t} \qquad (33)$$

but in which :

$$C_h = \frac{K_h . (\lambda + G)}{\gamma} = \frac{K_h}{\gamma} \, \frac{1}{m_h [2(1 - \nu)]} \qquad (34)$$

where γ is the unit weight of water (9810 N/m^3), and m_h is the coefficient of volume change as obtained with a consolidation test on a sample loaded in the horizontal direction.

Rate of slaking

From the radial pore pressure distribution resulting from the consolidation calculation, one gets the flow rate of water Q_{H_2O} entering per unit length of lime pile.

$$Q_{H_2O} = 2\pi . R_1 . K_h . \left(\frac{\partial h}{\partial r}\right)_{r = R_1} \qquad (35)$$

If we recall that C_2 is the relative volume of water needed to react with quicklime, the volume of lime reacting during a time increment dt is :

$$\Delta V^R_{CaO} = \frac{1}{C_2} \cdot Q_{H_2O} \cdot dt$$

Therefore, equation (36) gives the upper limit value of the reaction parameter,

$$\mu = \int_0^t \Delta\mu = \int_0^t \Delta V^R_{CaO} \Big/ V_{CaO} \tag{37}$$

the lower limit being set by the speed at which the slaking can proceed. From available information, it appears that the build up of the internal pressure is not instantaneous. If we call V_{lim} the constant limiting speed of the reacting front separating slaked from quicklime in a mass of lime in excess water, we can deduce from published experiments (6,7) a consistent value of about 0.02 mm/sec.

The speed of the reacted front being limited, the allowable water at the pile interface will be expressed by :

$$V_{max} = \sqrt{1 - \mu} \cdot V_{lim} \tag{38}$$

since the radius of the cylindrical front decreases as the reaction is taking place.

This means that the hydraulic boundary condition at the drain will be h = 0 provided the gradient does not exceed a maximum value. If this value is exceeded with the normal drainage, then the condition on the maximum velocity is applied.

ALGORITHM

The algorithm to calculate the phenomena as a function of time is based on the following steps :

1. An initial pore pressure distribution is chosen :
   $p_w = 0$ for $0 < r \leq R_1$ for $t \geq 0$
   $p_w = Dp_o$ for $R_1 < r \leq R_2$ for $t = 0$

2. Consolidation according to equation (33) is calculated during a time interval Δt, using the method of finite differences where ΔR is the difference between radial nodal points

$$U_{i,t+\Delta t} = U_{i,t} + C_h\left(\frac{U_{i+1,t} - 2U_{i,t} + U_{i-1,t}}{\Delta R^2} + \frac{1}{R_i}\,\frac{U_{i+1,t} - U_{i,t}}{\Delta R}\right)\Delta t + U_{g,t}$$

in which :

$U_{i,t}$ is the pore pressure at time t at radius $R_1 + (i - 1).\Delta R$

$U_{g,t}$ is the pore pressure generated during the time Δt.

3. $U_{g,t}$ is computed in the following way :

$$U_{g,t+\Delta t} = \frac{C_1.\Delta\mu}{C_3[1 + \mu_{p,t}.C_1 + (1 - \mu_t)C_5] + 1/K_1}$$

in which $\Delta\mu = \Delta Q/C_2.V_c$

where ΔQ is the smaller of the two expressions :

$$\Delta Q \leq \begin{cases} 2\pi R_1.\dfrac{K_h}{\gamma}.\left(\dfrac{U_{2,t} - U_{1,t}}{\Delta R}\right)\Delta t \\ 2\pi R_1 \sqrt{1 - \mu_t}\, V_{lim}.\Delta t \end{cases}$$

the initial values of μ_t and $\mu_{p,t}$ are 0.

$$\mu_{t+\Delta\tau} = \mu_t + \Delta\mu$$

$$\Delta\sigma_{r,t} = \sum_{t=0}^{t} U_{g,t}$$

$$\mu_{p,t} = \Delta\sigma_{r,t}/(K_1.C_1)$$

4. Consolidation with pore pressure generation takes place until $\mu = 1$. When this value is obtained, all the lime has reacted and , $U_{g,t} = 0$
   $\Delta\mu = 0$

   The final resultant expansion is given by $\mu_p.C_1$ and the radial consolidation continues thanks to vertical drainage through the lime column.

CALCULATIONS

A set of examples has been calculated in order to investigate the realism of the model chosen and to evaluate the influence of the various parameters involved in the analysis.

This parametric study included four types of soil combining the following properties :

$E = .4$ MPa and $E = 4$ MPa
$K_h = 10^{-7}$ m/s and $K_h = 10^{-9}$ m/s

Poisson's ratio was kept constant : $\nu = 0.4$.

The corresponding values of the coefficient of radial consolidation were :

C_h = 7.3 10^{-6}, 7.3 10^{-5}, 7.3 10^{-8} and 7.3 10^{-7} m^2s^{-1}.

The two lime types taken into consideration had the following characteristics :

Lime 1	Lime 2
$C_1 = 0.7$	$C_1 = 0.5$
$C_2 = 0.3$	$C_2 = 0.3$
$C_3 = 1$ MPa^{-1}	$C_3 = 2$ MPa^{-1}
$C_5 = 0.2$	$C_5 = 0.2$

The limiting speeds of the reaction front were taken as 2×10^{-5} and 2×10^{-4} m/s.

The initial pore pressures Dp_o considered in the soil were .1 and .1x10^{-3} MPa.

Four sets of pile radius and radius of influence were investigated : R_1, R_2 = 0.2, 0.6/ 0.2, 0.8/ 0.2,1.60/ 0.1,0.8 m.

Fig.5 shows the results of a sample calculation, performed with a radius increment of 5 cm and a time step of 51.5 s, corresponding to the following case :

$R_1 = 0.2$	$V_{lim} = 2\ 10^{-5}$ m/s	$E = .4$ MPa
$R_2 = 0.8$		
$D_{po} = .1\ 10^{-3}$ MPa	Lime type 1	$K_h = 10^{-7}$m/s

The horizontal lines correspond to $\Delta\sigma_{r,t}$ or the undrained pore pressure, whereas the curved lines correspond to the pore pressure as a function of the radial distance.
It can be seen on fig.5 that the lateral contact pressure generation amounts to .5 MPa and that two days are necessary for the lime to react completely.

The general trends revealed by the results of this parametric study are as follows :

- the lateral contact pressure increase corresponding to the full reaction of a type of lime depends mainly on the modulus of elasticity of the soil, but only marginally on the value of R_2/R_1, provided it is larger than 3.

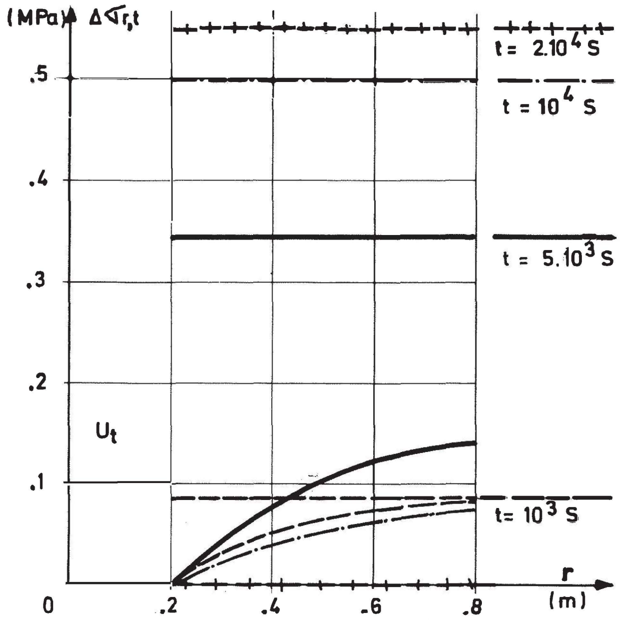

Figure 5 : Pore pressure distribution

- this lateral pressure build-up increases with the free volumetric expansion coefficient C_1 but decreases with increasing compressibility $C_3.C_5$ of the slaked lime. This means that the density of the quick lime in situ is very important as a higher compaction will increase C_1 and decrease C_3.

- the time required for the complete slaking of the lime is not dependent on the initial pore pressure, provided $K_h = 10^{-7}$ m/s. For $K_h = 10^{-9}$ m/s, the corresponding time is larger. It is greatly reduced when the limit speed of the reaction front is increased, especially when the soil possesses a large permeability. This means that if the lime is more finely ground, the time required for the total reaction will decrease, unless the soil is very impervious.

- when the initial pore pressure equals 0.1 MPa, the time required for the complete slaking to occur does not depend on C_h, unless it is smaller than 10^{-6} m^2s^{-1}. For an initial pore pressure equal to 0.1 kPa, this time increases when the value of the coefficient of consolidation decreases. This means that an initial pore pressure as induced by a vertical load, for example, will speed up the process.
- when $K_h = 10^{-7}$ m/s, the consolidation proceeds faster than the build-up of the pore pressure. The maximum pore pressure at the radius of influence is equal to 0.3 to 0.4 the undrained pore pressure increase. When $K_h = 10^{-9}$ m/s, the consolidation is slower than the build-up of the pore pressure, which means that for most cases, the maximum pore pressure obtained at the radius of influence will be equal to the undrained pore pressure increase. Therefore, to avoid an excessive build-up of pore pressure, it is necessary for the soil to possess a sufficient horizontal permeability.

CONCLUSIONS

A physical model representing the expansion of a slaking lime pile in an elastic soil ring has been presented.

The main parameters to describe the expansion behaviour of a lime pile are the coefficient of free volumetric expansion and the compressibility of the slaked lime. This model has been successfully applied in a particular case to reproduce the results of Laboratory tests done in Japan.

Another parameter of importance introduced in this analysis is the speed of the reaction front. It increases with increasing fineness of the quicklime.

A parametric study has been conducted using different pile diameters and spacings, different types of soils and different types of lime. The main conclusions to be drawn from the results are that :

- the permeability of the soil to be treated has to be sufficient, ($K_h > 10^{-7}$m/s) otherwise, quite high pore pressures are developed, especially if the grinding of the lime is fine.
- the final expansion of the lime does not depend very much on the pile spacing, which means that the relative volume reduction (or soil improvement), is uniform and inversely proportional to the influence area of a pile.
- In most of the cases examined, the lime has completely reacted after two days.
- the conditions of compaction of the lime in-situ are of utmost importance.

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