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CONSOLIDATION OF CLAYEY SUB-SOILS WITH INTERMEDIATE PERMEABLE LAYERS IMPROVED BY VERTICAL DRAINS WITH SMEAR EFFECT

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ABSTRACT: Design of vertical drains is usually based on Barron's theory considering the clay layer is always homogeneous. However it has often been recognized in several situations that many natural deposits have considerable in-homogeneities, such as laminations of coarser material within the clay layer. Field data from such clay layers improved by vertical drains have sometimes shown that the commonly used conventional equations should be modified. In this paper, considering horizontal and vertical inflow and outflow into and out from a small element, governing equation of consolidation has been formulated with the effects of varying permeability and compressibility with time. Two kinds of new parameters are defined as $K=(k_s/k_c)(H_s/H_c)$ and $\alpha_{98}=t_{98(2D)}/t_{98(radial)}$ considering different coefficients of permeability and sand/clay thickness. With these definitions, the importance of thin intermediate permeable layers, which were previously overlooked in designs, are discussed in this paper. The variation of c_h/c_v back calculated from the insitu tests are assessed with the above theoretical concepts considering both effects of smear and intermediate permeable layers. Practical approaches for measuring insitu permeability and thickness of intermediate permeable layers for better prediction are also included.

Keywords: Ground improvement, vertical drains, intermediate permeable layers, smear effects

INTRODUCTION

For over 50 years now, vertical drains have been installed in compressible soils to speed up consolidation. Improvement of soft ground, such as that encountered during construction of roads, railways and airports on near-shore or reclaimed shallow-water areas, is often achieved by the use of preload and vertical drains. The best-known theoretical study on this topic of vertical drains was carried out by Barron (1948). Design of vertical drains is usually based on Barron's theory although there is evidence that the predicted results of settlements are sometimes very much different with the field observations, which cannot be explained by such a simplified approach proposed for a uniform soil.

It was shown by Rowe (1968) using several field observations, that the real drainage behavior of a deposit as a whole depends on the geological details of its formation. Quite small layers, veins of silt along fissures, or organic inclusions can transform the permeability of the mass compared with that of small samples.

Permeable sand or silt layers are often found within the poor soil. Unfortunately, these natural and highly effective drainage layers are often overlooked when they are thin, especially when continuously sample borings are not made. However, even if continuous sand or silt layers are found, they may be so thin or have such a low permeability that head losses in them become excessive if the drainage path is long. Where the effectiveness of intermediate permeable or silt layers is in doubt or when such layers do not exist, positive means for accelerating drainage may be desirable.

As Johnson (1970) mentioned, vertical drains are an expense, they obviously should be installed only where preceding subsoil studies show them to be required. For safe and cost effective design of embankment on clayey subsoil, it is very important to detect intermediate permeable layers between the clayey subsoil. The existence of the intermediate permeable layers will reduce the drainage path and increase the rate of consolidation thus reduce significantly the waiting time to achieve the required settlement, and in certain case even the use of vertical drains can be omitted (Gue and Tan, 2001). According to Gue and Tan (2001), 1m center to center (*c*/*c*) spacing vertical drains will cost 300% more than 2m *c*/*c* spacing vertical drains. In view of the cost sensitive nature, it is very important to acquire sufficient information of the subsoil, then a cost effective design can be carried out.

The insitu test results of several researchers mentioned in Egashira et al., (2002) indicate the importance of the inclusion of smear effect into theoretical evaluations for non-homogeneous clayey soils, due to their findings of c_h/c_v<1.0 and c_h/c_v >1.0. Win et. al. (2001) suggested that lower c_h values could be mainly due to smear since k_h can be reduced by an order of magnitude due to smear effect around drains. However, Rowe (1968) found that c_h/c_v >3-10 in the Derwent

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reservoir due to the presence of high intermediate permeable layers in the clay. Another field observation was discussed by Calderon and Romana (1997) on the TOTAL Oil Storage Plant at Valenica Harbour. The coefficients of consolidation obtained by piezocone had been 10-1000 times greater than ones obtained in the laboratory tests and the back calculated c_h values had been 1-50 times smaller than the ones obtained by the piezocone. These two opposite observations should be considered in the analysis of non-homogeneous clayey soil.

However in all probability, it can be implied that vertical drains would be especially useful in stratified deposits. Much of the uncertainty hitherto associated with the prediction of the consolidation rates of clay strata for the purpose of vertical drain designs might be removed once attention is paid to the geological structure of the clay (i.e. especially proper consideration of intermediate permeable layers into consolidation calculations) and the appropriate testing techniques. Even though there are lot of field observations on the effects of consolidation with vertical drains due to intermediate permeable layers, so far few theoretical studies has been done considering the influence of intermediate permeable layers. In most of those cases, in which both radial and vertical effects of consolidation are considered to include the advantage of intermediate permeable layers, the combined average degree of consolidation had been calculated using its definition of Carillo (1942).

In this study, 2-dimensional numerical studies have been done, in which clay layers having several intermediate permeable layers are considered using the non-linear void ratio, permeability and effective stress relationships which take into account varying permeability and compressibility of soil with time.

THEORETICAL BACKGROUND

In reference to Fig. 1, the vertical inflow of water into the small element during time Δt,

$$\Delta q_{z\_in} = [v_z + \frac{\partial v_z}{\partial \xi}\Delta\xi].r\theta.\Delta r.\Delta t \qquad (1)$$

The vertical outflow of water at that time interval is,

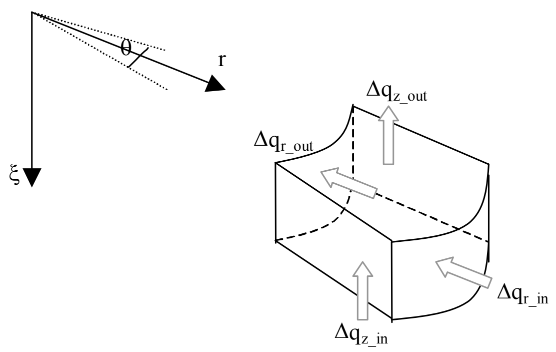

Fig. 1 Movement of water in a small clay element

$$\Delta q_{z\_out} = v_z.r\theta.\Delta r.\Delta t \qquad (2)$$

At the same time horizontal inflow of water is,

$$\Delta q_{r\_in} = [v_r + \frac{\partial v_r}{\partial r}\Delta r].(r+\Delta r)\theta.\Delta\xi.\Delta t \qquad (3)$$

and the horizontal outflow of water is,

$$\Delta q_{r\_out} = v_r r\theta.\Delta\xi.\Delta t \qquad (4)$$

Therefore the change in water volume in this small element is,

$$\Delta q = (\Delta q_{r\_in} - \Delta q_{r\_out}) + (\Delta q_{z\_in} - \Delta q_{z\_out})$$

$$\frac{\Delta(r\theta.\Delta r.\Delta\xi)}{\Delta t} = \left[\left(\frac{\partial v_r}{\partial r} + \frac{v_r}{r}\right) + \frac{\partial v_z}{\partial \xi}\right] r\theta.\Delta r.\Delta\xi$$

In reduced coordinate system, z, the law of mass conservation for the combined process of vertical and horizontal drainage of water from a soil element is therefore changed to,

$$\frac{\partial e}{\partial t} = (1+e)\left(\frac{\partial v_r}{\partial r} + \frac{v_r}{r}\right) + \frac{\partial v_z}{\partial z} \qquad (5)$$

where $\Delta\xi = (1+e)\Delta z$ and $\frac{\partial \Delta\xi}{\partial t} = \frac{\partial e}{\partial t}\Delta z$ (Imai,1995)

As the pore water moves through the pores of the soil skeleton in accordance with Darcy's law ($v = ki$) both in horizontal and vertical direction, then,

radial velocity, $v_r = \frac{k_r}{\gamma_w}\left(\frac{\partial u}{\partial r}\right)$

and vertical velocity is,

$$v_z = \frac{k_z}{\gamma_w}\left(\frac{1}{1+e}\frac{\partial u}{\partial z} - \gamma_w\right)$$

Substituting the value of v_r and v_z for those in Eq.5, a combined governing equation of consolidation is formulated as,

$$\frac{\partial e}{\partial t} = [(C_1\frac{\partial^2 u}{\partial r^2} + C_2\frac{\partial u}{\partial r}) + (C_3\frac{\partial^2 u}{\partial z^2} + C_4\frac{\partial u}{\partial z} + C_5)] \qquad (6)$$

where,

$$C_1 = \frac{(1+e)k_r}{\gamma_w}, C_2 = \frac{(1+e)}{\gamma_w}(\frac{k_r}{r} + \frac{\partial k_r}{\partial r}), C_3 = \frac{1}{\gamma_w}\frac{k_z}{(1+e)},$$

$$C_4 = \frac{1}{\gamma_w}(-\frac{k_z}{(1+e)^2}\frac{\partial e}{\partial z} + \frac{1}{1+e}\frac{\partial k_z}{\partial z}), C_5 = \frac{\partial k_z}{\partial z}$$

Considering $\Delta\sigma' = -\Delta u$, $-\Delta e = A_1.\Delta\sigma'$ and $A_1 = 0.434C_c/\sigma'$, Eq. (6) can be transformed into a normal differential equation in terms of excess pore water pressure. The following non-linear relationships are considered in the calculations to find out the change in the permeability (k) and the void ratio (e) due to an increase of effective stress σ'.

$$e = N_k + C_k \log k \tag{7}$$

$$e = \Gamma - C_c \log \sigma' \tag{8}$$

The values of N_k, C_k, Γ and C_c used in this study are given in NUMERICAL ANALYSIS Section. These values can be different in horizontal and vertical directions. However, in this analysis, same values are considered.

DEFINITION OF NEW PARAMETER *K*

It was shown by Gray (1945), that the rate of consolidation in a layered soil which consists of clay and sand is controlled by the parameter $R=(k'h)/(kh')$, a dimensionless factor that involved the permeability and thickness ratio of the two layers where k and h are the permeability and thickness of the clay, respectively; and k' and h' are the permeability and thickness of sand, respectively. However, Gray's analysis was one dimensional, which omits the effect of lateral drainage. Considering the predominant flow in the clay layers as vertical, and the horizontal flow in the sand layers towards the connecting dykes, Gibson and Shefford (1968) evaluated filter efficiency. However, the effects of sand and clay thickness and length of flow path on the overall performance of the drainage layer were not elaborated. A characteristic factor called λ of a sand layer in the layered clay-sand scheme was proposed by Tan et.al. (1992), based on the average degree of consolidation of the system. $\lambda = (k_s/k_c)(H/L)(H_s/L)$, here k_s and k_c are permeability of sand and clay respectively and H and H_s are the thickness of clay and sand layers respectively, while L is the distance from the centerline to the sand dyke (horizontal flow length).

Since the above-mentioned λ parameter is defined for layered clay-sand scheme of land reclamation which should have a permeability ratio of sand to clay with an order of at least 10^6 for fully effective drainage, this λ parameter cannot be directly applied for vertical drain applications where they meet natural sand layers which cannot be always considered as pure sand with a very high permeability. In the present study, a new parameter K is proposed as $K=(k_s/k_c)(H_s/H_c)$ for vertical drain installation in the clayey soils which have intermediate permeable layers. In this definition, k_s and k_c are permeability of sand and clay respectively and H_s and H_c are the thickness of sand and clay layers respectively.

EVALUATION OF SOIL PERMEABILITY

In view of the limitations of laboratory tests, many geotechnical engineers agree that the design of vertical drains should be based on permeability and consolidation characteristics obtained from appropriate insitu tests (Holtz, et.al, 1991). Determination of field permeability of sand and clay will be with great importance since these values will noticeably influence the K parameter. There are several options available for evaluating the permeability of sand in the field as discussed in this section, while laboratory tests are available for the determination of clay's permeability coefficient.

Visual Classification

The simplest approximate method for estimating the permeability of sand is by visual examination and classification, and comparison with sands of known permeability. An approximation of the permeability of sands can be obtained from Table 1.

Table 1 Approximate Coefficient of Permeability for Various Sands (NAVFAC Design Manual, 1986)

Type of Sand (Unified Classification System)	Coefficient of Permeability (x 10^{-4} cm/sec)
Sandy silt	5-20
Silty sand	20-50
Very fine sand	50-200
Fine sand	200-500
Fine to medium sand	500-1000
Medium sand	1000-1500
Medium to coarse sand	1500-2000
Coarse sand and gravel	2000-5000

Empirical relation between D_{10} and k

Hazen's formula is frequently useful for the purpose of estimating the coefficient of permeability using D_{10} results (Louden, 1952). Similar relationship has been given in NAVFAC Design Manual (1986), where the permeability of clean sand has been estimated from empirical relations between D_{10} and permeability coefficient. These results had been developed from laboratory and field pumping tests for sands in the Mississippi and Arkansas River valleys. The relationship discussed in NAVFAC Design Manual is as follows;

$$k = C(D_{10})^2$$

where,
k = coefficient of permeability (cm/sec)
C ≅100 (may vary from 40 to 150)
D_{10} = effective grain size (cm)

Empirical relations between D_{10} and k are only approximate and should be used with reservation until a correlation based on local experience is available.

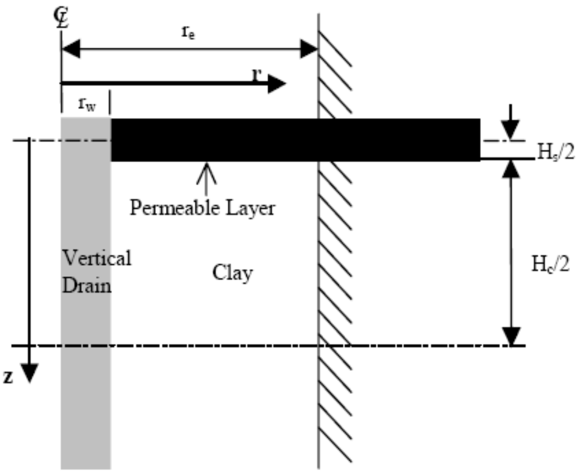

Fig. 2 Schematic Diagramof Clay with Vertical Drainsand Intermediate Permeable layers

Field pumping tests

Field pumping tests are the most reliable procedure for determining the in-situ permeability of a water-bearing formation. For large dewatering jobs, a pumping test on a well that fully penetrates the sand stratum to be dewatered is warranted. However, for small dewatering jobs, it may be more economical to select a more conservative value of *k* based on empirical relations than to make a field-pumping test.

Piezocone Dissipation Tests

The advantage of the piezocone is that it may make it possible to determine the insitu profiles of consolidation coefficients. On the basis of the interpretation of piezocone dissipation data, it is possible to calculate a coefficient of consolidation, and indirectly the coefficient of permeability although its value is not absolute but relative.

SMEAR EFFECT IN NON-HOMOGENEOUS SOIL

The smear effects caused by the installation of the drains in the clay play a significant role in the consolidation process. Permeability of the smear zone is the key parameter here and its accurate determination is essential for a successful consolidation prediction. Based on the analyses discussed by Hawlader et.al. (2002), only two values of permeability, namely coefficient of permeability of the undisturbed soil and the remolded soil should be considered to take into account of the smear effect using the linear variation of permeability in the zone. The coefficient of permeability of the undisturbed soil can be obtained from back-analysis of the case histories or the data measured in the field by use of available techniques discussed in a previous section. The coefficient of permeability near the drain can be obtained from laboratory tests using the remolded soil.

PROOF OF *K* VALUE FOR FIELD CONDITIONS

Due to the possible difficulties, which can be encountered in obtaining k_s and H_s values in practical situations, the following simplification is proposed here. In general, natural soil deposits are stratified. If the stratification is continuous, the equivalent coefficients of permeability of the whole layer for flow in the horizontal and vertical directions can be calculated.

Flow in the horizontal direction can be expressed as follows.

$$\mathrm{k}_{\mathrm{e(h)}} = \frac{1}{\mathrm{H}}(k_{h1}H_1 + k_{h2}H_2 + k_{h3}H_3 + ...) \tag{9}$$

Similarly the flow in the vertical direction can be expressed as follows.

$$\mathrm{k}_{\mathrm{e(v)}} = \frac{\mathrm{H}}{H_1/k_{v1} + H_2/k_{v2} + H_3/k_{v3} + ...} \tag{10}$$

where k_{hi}- horizontal permeability
k_{vi}- vertical permeability
H_i – layer thickness (i=1,n)
H – total thickness of the soil

These expressions can be used in the clay/sand layers as shown in Fig. 2 and are used in this research as follows.
For horizontal flow,

$$\mathrm{k}_{\mathrm{e(h)}} = \frac{1}{\mathrm{H}}(k_s H_s + k_c H_c) \tag{11}$$

and

$$\mathrm{k}_{\mathrm{e(v)}} = \frac{\mathrm{H}}{H_c/k_c + H_s/k_s} \tag{12}$$

Then the two-dimensional equivalent coefficient of permeability can be expressed as;

$$\mathrm{k}_{\mathrm{eq}} = \sqrt{\mathrm{k}_{\mathrm{e(h)}}.\mathrm{k}_{\mathrm{e(v)}}} \quad \text{(Vreedenburgh, 1936)} \tag{13}$$

By substituting Equations 11 and 12 in Eq. 13, the following expression can be found.

$$\mathrm{k}_{\mathrm{eq}} = \sqrt{\frac{1}{\mathrm{H}}(k_s H_s + k_c H_c).\frac{\mathrm{H}}{H_c/k_c + H_s/k_s}}$$

$$\mathrm{k}_{\mathrm{eq}} = \sqrt{(k_s H_s + k_c H_c).\frac{1}{\frac{H_c}{k_c}(1+\frac{H_s/k_s}{H_c/k_c})}}$$

when H_s<<H_c and k_s>>k_c, then $\frac{H_s/k_s}{H_c/k_c} \Rightarrow 0$

therefore, $k_{eq} = \sqrt{(k_s H_s + k_c H_c).\frac{k_c}{H_c}} = k_c\sqrt{\frac{k_s}{k_c}\frac{H_s}{H_c}+1}$

$$(\frac{k_{eq}}{k_c})^2 = \frac{k_s}{k_c}\frac{H_s}{H_c}+1$$

Since $K = \frac{k_s}{k_c}\frac{H_s}{H_c}$, it can be simplified that,

$$K = (\frac{k_{eq}}{k_c})^2 - 1 \quad (14)$$

Here, k_c value can be found by use of the usual laboratory tests. To find out k_{eq}, it is needed to calculate $k_{e(h)}$ and $k_{e(v)}$. $k_{e(h)}$ can be evaluated using the insitu permeability tests discussed in a previous section and $k_{e(v)}$ value can be calculated as follows.

Using the assumption of $\frac{H_s / k_s}{H_c / k_c} \Rightarrow 0$, the equation [12] can be modified as,

$$k_{e(v)} = H\frac{k_c}{H_c} \quad (15)$$

Then k_{eq} value of Eq.13 can be obtained using $k_{e(h)}$ and $k_{e(v)}$ values thus obtained, and an approximate K-value can be determined from Eq.14. As shown in Fig. 3, it can be seen a good linear relationship between two K values defined by the two methods.

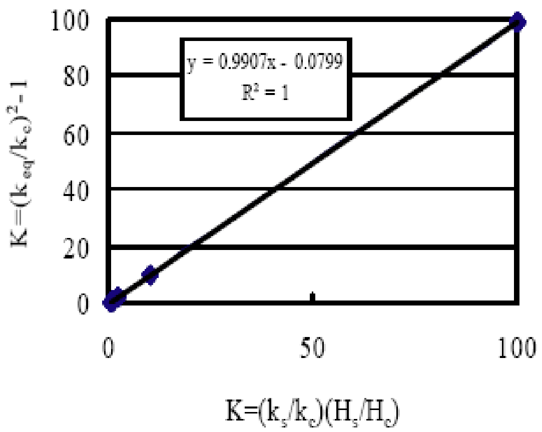

Fig. 3 Variation of $K=(k_s/k_c)(H_s/H_c)$ and $K=(k_{eq}/k_c)^2$-1

AVERAGED TREATMENT OF INTER-MEDIATE PERMEABLE LAYERS IN CLAY

In clayey soils containing intermediate permeable layers, large diameter sampling and testing combined with insitu testing is required to assess the nature and spatial distribution of intermediate permeable layers and their influence on the engineering behavior of clayey soils. Thus, for these materials, small diameter sampling and testing is of limited value although careful examination of cores can indicate the existence of laminations and thereby the need for further investigations. However, McGown and Hughes (1981) suggested that, much use could be made of small diameter samples with layered soils. They should be taken consecutively since a complete record of the soil profile can be obtained at each borehole and then comparisons can be made between the boreholes to check on the continuity and disappearance of critical layers. Sample sizes of 54-102mm dia. can be used for this purpose, depending on the availability of equipment and soil types encountered. The sampling should always be carried out using piston techniques, rather than percussion driving, in order to minimize disturbance (McGown and Hughes, 1981).

Table 2 Intensity Classification for Layered Sediments

Classification of intensity	Percentage overall thickness (McGown et. al., 1980)	Range of K value
Very low	Less than 2.5	$K < 0.05$
Low	2.5 to 5.0	$0.05 \leq K < 0.1$
Medium	5 to 10	$0.1 \leq K < 0.2$
High	10 to 20	$0.2 \leq K < 0.5$
Very high	20 to 50	$0.5 \leq K < 3.0$
Dominant	More than 50	$K \geq 3.0$

The data obtained from the preliminary investigations allow the location, quality and size of sampling and testing in the next stage of investigation to be assessed. As a result of quantification of intermediate permeable layer observations, the location of critical layers in the soil profile may be identified with more confidence. The intensity classification for layered sediments shown in Table 2 was proposed by McGown et.al. (1980), and that classification system can be compared with a similar classification using K values as described below.

With the assumptions discussed in the previous section to obtain K value for field conditions,

$$K \cong (\frac{k_{eq}}{k_c})^2 - 1 \text{ where } k_{eq} \cong H.(k_c/H_c)$$

therefore, $\frac{H}{H_c} = \sqrt{K+1}$

since $H=H_s+H_c$, $\frac{H_s}{H} = 1 - \frac{1}{\sqrt{K+1}}$

Using the definition on classification by McGown et al., (1980),

$$\frac{H_s}{H}\times 100 = (1 - \frac{1}{\sqrt{K+1}})\times 100$$

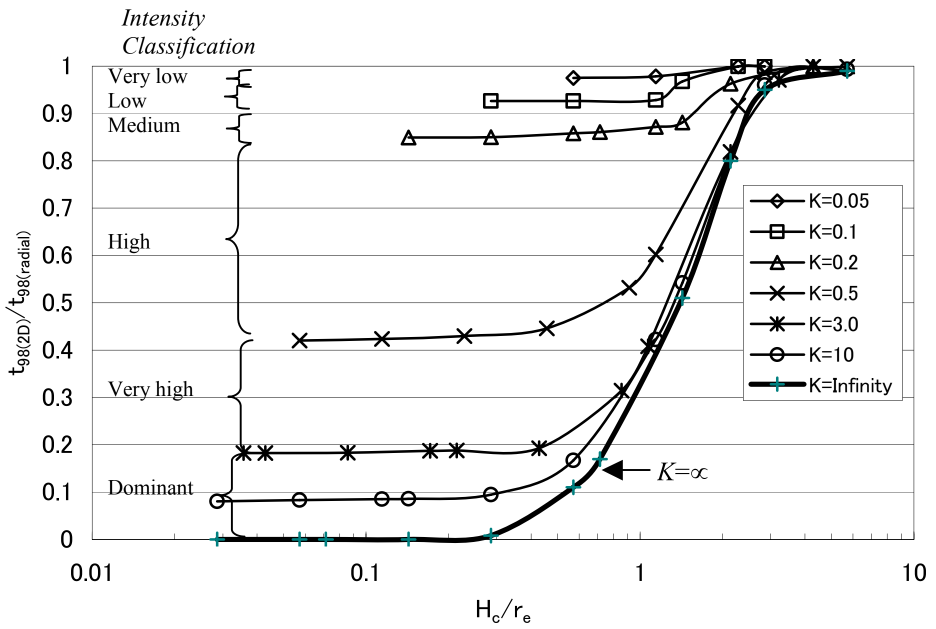

Fig. 4 Variation of K value for re=140cm and rw=20cm

This modified classification considering K value, can be compared with McGown et.al. (1980) as shown in Table 2.

Example calculations for the comparison with McGown et. al., (1980)

For *very low* intensity,

$$\frac{H_s}{H}\times 100 < 2.5 \text{ then } (1-\frac{1}{\sqrt{K+1}})\times 100 < 2.5$$

$K < 0.05$

IDEALIZATION OF CLAY/SAND SOIL FOR NUMERICAL ANALYSIS

As shown in Fig. 2, due to the symmetrical situation of clay and intermediate permeable layers, half of the effective area is considered for the analysis. H_s and H_c are the thickness of sand and clay layers respectively. A well radius (r_w) of 20cm and different effective radii (r_e) are used. It is considered that the permeability of the vertical drain as infinite and the permeability in sand and clay as k_s and k_c respectively.

Since the overall stability and deformation depend on effective stress or pore pressure of the soil elements, the average degree of consolidation at time t can be considered as a measure of overall improvement of clay layer. It is calculated by,

$$U_t = 1 - \frac{\int_A u(r,z,t)dA}{\int_A u_0 dA} \qquad (16)$$

where u_0 and u are the initial and current pore water pressures respectively, and dA is the area of the small element corresponding to a grid point. Finite difference method is used for the analysis.

Initial and Boundary Conditions

Initial condition

$t = 0 \qquad u = u_0$

in the vertical drain ,

$0 \le r \le r_w \qquad u = 0$

at the maximum radial distance,

$$r = r_e \qquad \frac{\partial u}{\partial r} = 0$$

at the sand/ clay boundary,

$z = H_s/2 \qquad u_{sand} = u_{clay}$

$$k_{sand}\frac{\partial u}{\partial z} = k_{clay}\frac{\partial u}{\partial z}$$

at the mid-depth of the clay layer considering the symmetry, when $z = H_s/2 + H_c/2$, then $\frac{\partial u}{\partial z} = 0$

To consider the effect of both radial and vertical combined consolidation, another dimensionless ratio is defined as, $\alpha_{98} = \frac{t_{98(2D)}}{t_{98(radial)}}$

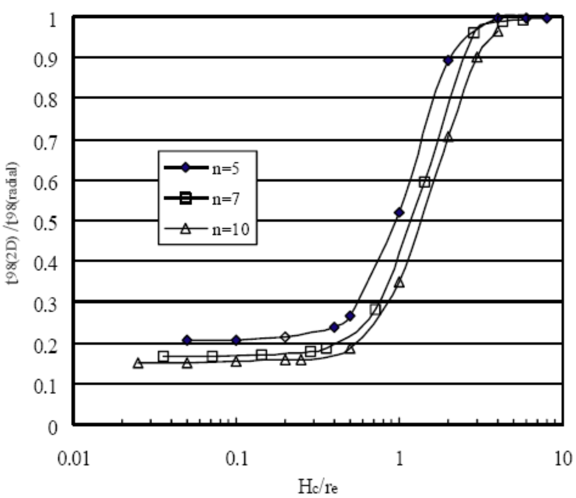

Fig.5 Variation of $\alpha_{98}=t_{98(2D)}/t_{98(radial)}$ value with n for K=2.0

where $t_{98(2D)}$ and $t_{98(radial)}$ are required times for 98% consolidation for 2D and radial only cases respectively. Using this ratio, it can be evaluated how far, the actual time needed for consolidation deviates from the conventional Barron's solution, when intermediate permeable layers are present in the clay. When this time ratio is unity, there is no effect of intermediate permeable layers, while its effect is larger for a smaller time ratio.

NUMERICAL ANALYSIS

Numerical simulations have been done assuming that the effective area assigned to each vertical drain has been transformed into circular unit cell with radius, r_e, as demonstrated in Fig. 2 taking the fixed parameters of N_k =8.64, C_k=0.72, Γ=3.0, C_c=0.70; these are typical values of Japanese alluvial clays with w_L=100%, and r_w has been fixed to 20cm of sand drains. As shown in Fig. 4, for different K values selected according to the classification proposed by McGown et. al., (1980), the ratio of α_{98} for clay thickness normalized by effective radius (H_c/r_e) is discussed for r_e=140cm. Here K=∞ presents the situation where the intermediate permeable layers have infinite permeability. In this situation, α_{98} is almost zero for very thin clay layer. It can be clearly seen that the effect of K value is significant especially when H_c/r_e ratio is less than 1.0. This indicates that the effect of K value is considerable when clay layer thickness is less than two or three times of the effective radius of the vertical drain. In actual clayey soil, there may be several intermediate permeable layers where the thickness of the clay layer (H_c) between two intermediate permeable layers is small, then this relationship may be useful to analyze actual consolidation behaviors. Using this relationship, it can be observed that lower K values result in higher ratio of α_{98} for smaller thickness of clay layers. However, for thick clay layers, this relationship is not observed. Small α_{98} values for thinner clay layer means that whenever there are closely spaced thin sand layers, radial consolidation results in a longer time required for full consolidation.

On the other hand, when there are no closely spaced intermediate permeable layers, i.e. clay layer thickness between two intermediate permeable layers is high, consideration of radial only consolidation is reasonable and there is less effect on consolidation due to the presence of sand layers since α_{98} value becomes unity.

Figure 5 shows the variation of α_{98} value for different n (= r_e/r_w) values for the case of K=2.0. Figures 4 and 6 clearly show the importance of the spacing when H_c/r_e is less than 1.0. Therefore in order to evaluate the

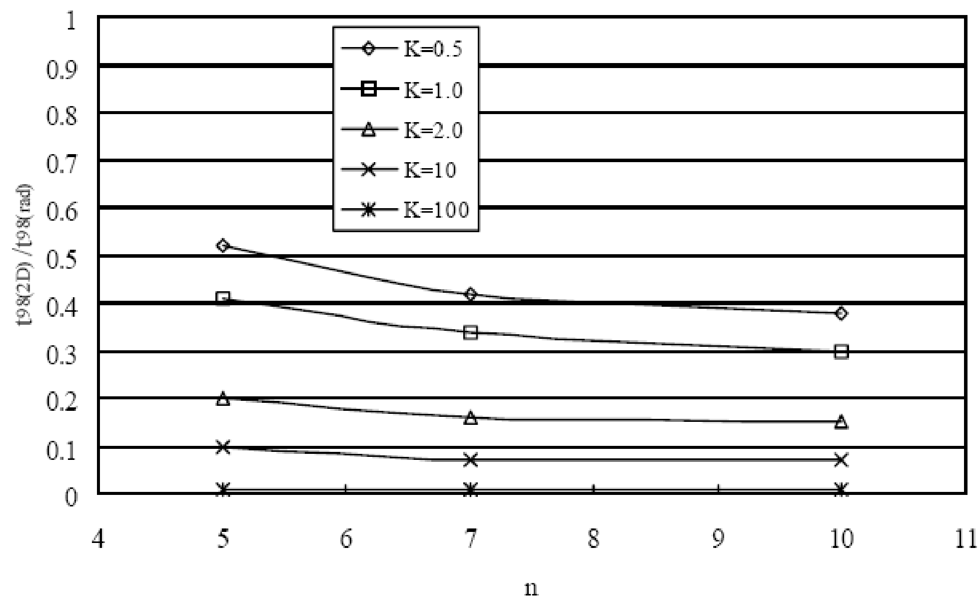

Fig. 6 Variation of K value for different n values when H_c/r_e<1.0

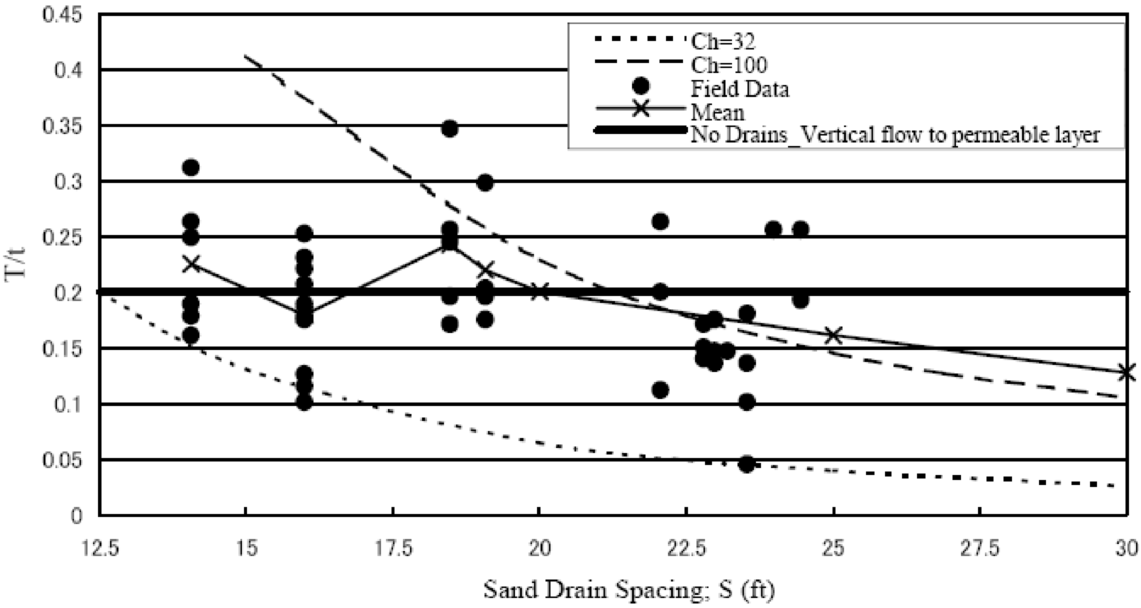

Fig. 7 Influence of drain spacing on time factor/time ratio (after Rowe, 1968)

importance, another comparison is done as shown in Fig. 4, between n values and α_{98} for different K and n values for the region H_c/r_e<1.0 .

According to Fig. 6, the effects of spacing of vertical drains become less important for higher K values. This clearly confirms the observation of Rowe (1968) on Derwent Reservoir, " it would seem therefore that the sand drains made an important contribution to construction but, solely on present considerations, fewer drains could have been used." A discussion on the ratio of time factor (T) to reach 50% consolidation and the time (t) in years required to reach the same consolidation had been done on Fig. 7 given by Rowe (1968). A behavior similar to Fig. 6 can be obtained from Fig. 8, as given in Fig. 8. These results clearly show that, even for $t_{50(actual)}/t_{50(radial)}$ ratio, it decreases with n.

As shown in Fig. 9, the difference in initial void ratio, will not give a considerable change in α_{98} value for higher K values. However it gives a noticeable decrease of α_{98} value with the increase of initial void ratio.

Some insitu test results obtained from various researches done recently are shown in Fig. 10. These results show the availability of two different situations in the ground improved with vertical drains. When the presence of data in c_v/c_h >1.0 region, it indicates that the smear effect is more dominant factor while c_v/c_h <1.0 shows a higher effect of horizontal permeability mainly due to the intermediate permeable layers. This clarifies the importance of including smear effect in the model discussed in this paper.

With the following simple assumptions, the relationship between c_v/c_h and $1/d_e$ can be compared with the numerical findings.

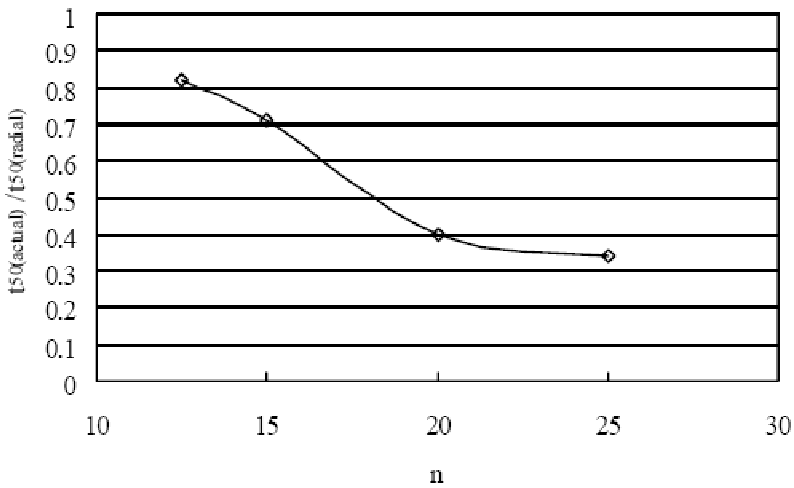

Fig. 8 Variation of $t_{50\ (actual)}/t_{50\ (radial)}$ for different n values (Derwent Reservoir), 45ft thick clay layer, r_w=1ft

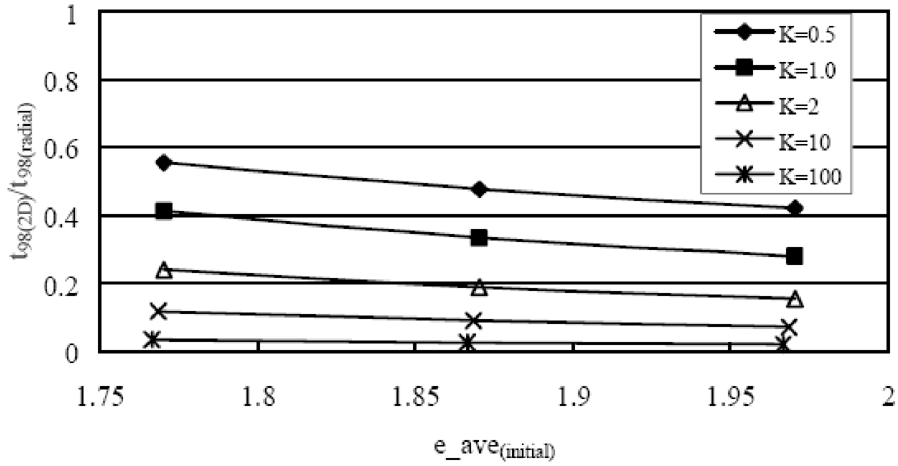

Fig. 9 Variation of *K* value for different initial void ratio when $H_c/r_e < 1.0$

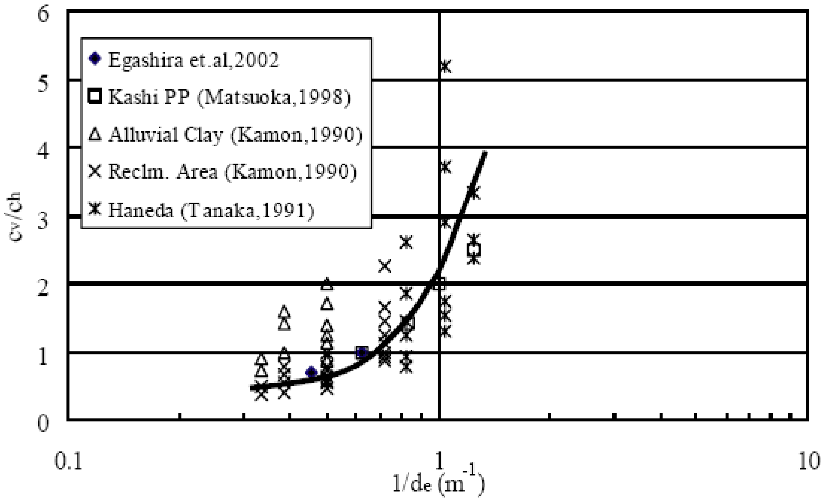

Fig. 10 Field observations for C_v/C_h values

$$\frac{c_v}{c_h}\alpha\frac{t_{98(2D)}}{t_{98(radial)}} \text{ and } \frac{1}{d_e}\alpha\frac{H_c}{r_e}$$

These assumptions are valid for particular ground condition. c_v found from oedometer tests is usually used as c_h in the Barron's solution and c_h from back analysis represent the actual ground condition. With these simplifications and $k_h/k_{actual}=c_h/c_{actual}$, the above assumptions can be made.

Figure 11 shows two different *K* values discussed for different normalization conditions. *K*=0.5 and *K*=10 cases are normalized by both commonly used Barron's equations and modified Barron's equations for smear. These results are compared with the two cases considered previously, without smear too.

Whenever smear is considered, it can be clearly seen that α_{98} becomes more than unity if it is normalized by Barron's solution. This implies that when thickness of the clay layer is higher, despite the presence of intermediate permeable layers, it is more important to consider the smear effect. On the other hand, when the clay layer thickness is smaller, consideration of both cases are important. Another remarkable observation is the variation of α_{98} values when H_c/r_e<1.0. By comparison of the results of same *K* values with and without smear, it can be concluded that the effect of smear is higher in lower *K* values compared to higher *K* values. This clearly indicates that higher effect of intermediate permeable layers (due to higher *K* values) will drastically decrease the influence of smear.

Both field observations and these numerical results confirm the need for treating the soil considering its geological features, especially the presence of intermediate permeable layers to obtain the optimum solution with reference to time and cost.

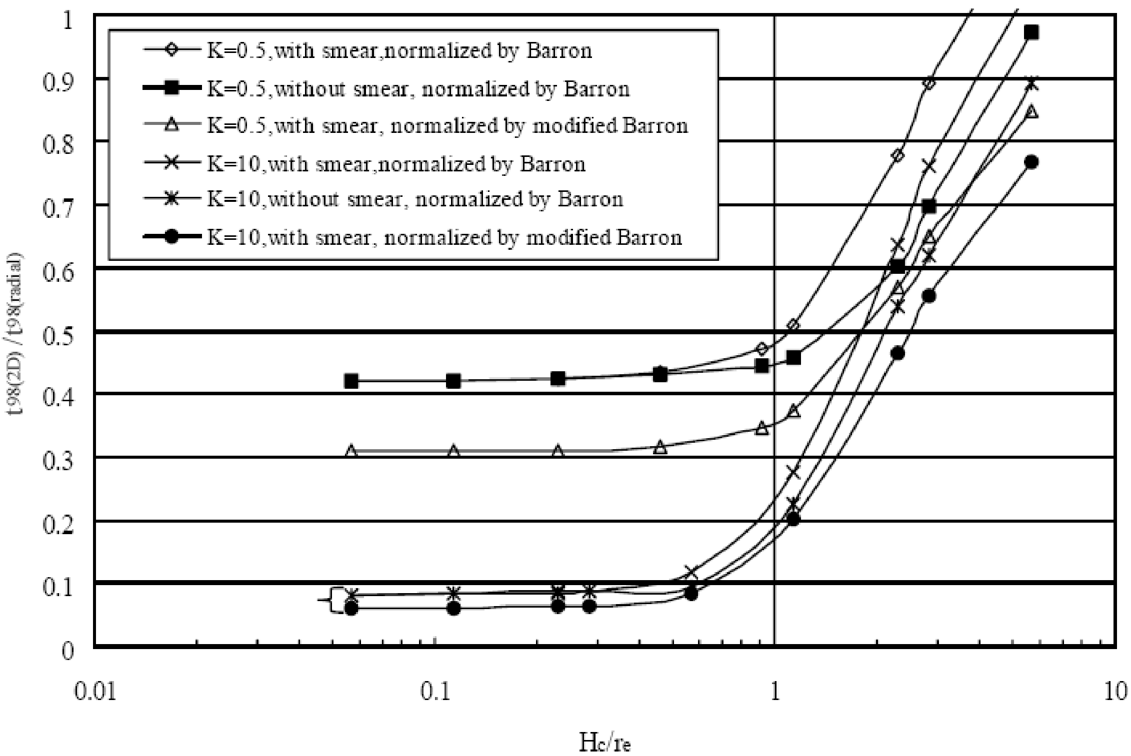

Fig.11 Consideration of *K* value with and without smear effect

CONCLUSIONS

When $H_c/r_e<1.0$ the effect of intermediate permeable layers is very high and when $H_c/r_e>1.0$ effect of intermediate permeable layers is negligible and Barron's solution may serve the need. This means that the drainage path in vertical direction is effective when the effective radial distance is higher than the thickness of clay layers in between two sand layers. Whenever several intermediate permeable layers are present in a particular area to be improved, the spacings will play a key role. (Spacing = H_s+H_c ; for an optimum solution $H_c < r_e$)

When n is greater, the effects of intermediate permeable layers are high. This does not mean that it is faster than low n cases, however compared with Barron's solution, intermediate permeable layers give a greater contribution to make it faster than the results obtained from Barron's solution.

It is clear from this study that by reducing the number of vertical drains with appropriate consideration of the effects of intermediate permeable layers, the cost for the vertical drains can be optimized. In some cases, even omission of vertical drains may successfully serve the need.

When the smear effect is considered in the analysis, the effect of smear will be higher in lower K values compared to higher K values. This means, the effect of smear may be compensated due to the presence of closely spaced high permeable layers in the clayey sub soil.

One major drawback of the above conclusions is, practically the intermediate permeable layers should be continuous or peripheral drains should be considered. Another important requirement is, intermediate permeable layers should not be in random directions in a horizontal plane. Since it will not give the optimum solution.

Real drainage behavior of any deposit as a whole depends on the geological details of its formation. Performance will entirely depend on the continuity and the spacing of the intermediate permeable layers.

Simple laboratory test will not represent the actual drainage or permeability condition of the field, so it is required to couple both laboratory tests and insitu tests to evaluate the actual drainage conditions. In addition it can be emphasized that the choosing of appropriate high quality sampling and testing techniques in soils containing pronounced laminations like intermediate permeable layers during the preliminary site investigation process will save the cost and time, if those features are properly considered in the design process.

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