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An Approach for Prediction of Compressibility and Permeability Behaviour of Sand Bentonite Mixes

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Introduction

Seepage control is an important aspect of soil engineering. In earth dams, clay cores and cutoff trenches serve the purpose of minimising seepage losses either through or underneath the dams. Trenches and dumping yards are lined if the leachates from wastes are to be prevented from mixing with ground water. There are numerous situations which warrant seepage con-control from different requirements like construction, environmental protection, long term stability etc. Bentonite sand mixes are often used to satisfy these functional requirements. The compressibility and permeability properties of these mixes are very important input parameters to assess the suitability for such requirements. At present, there are no methods existing to help in these situations. In the present investigation, an approach is presented towards this objective.

Evaluation of the Methods

In seepage control, compressibility and shear strength are not major design considerations, but, during the service life, affect the permeability indirectly and significantly. As for example, the core and the cutoff walls in a dam should have compressibility compatible with the surrounding material to minimize the differential movements and hence stress concentrations. At the same time, it should be capable of withstanding the possible shear deformations without cracking or developing shear planes. Further, the material in the cutoff trench should not get squeezed out due to the pressures exerted from sides. These requirements point to the necessity of having a material having comparable strength and deformation properties akin to the surrounding strata, yet having the required low permeability

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values. But, low permeability and low compressibility are contradictory requirements because the plastic fines required for low permeability result in higher compressibility (D' Appolonia 1980).

Bentonite sand mixes satisfy these requirements. Permeability and compressibility of benetonite sand mixes have not been investigated intensively, particularly with regard to the role of proportioning and the resulting changes in engineering behaviour.

Background Information

Of the two engineering properties required in the present investigation, studies on permeability are less compared to compressibility. There are two different laboratory techniques that are commonly used to determine the coefficient of permeability; (1) direct measurements with an imposed hydraulic head or (2) indirect determination from consolidation test results. The coefficient of permeability can be calculated indirectly from consolidation test results by first determining the coefficient of consolidation (C_v) from curve-fitting techniques of the compression data, using either the square root of time or log of time methods (Taylor, 1948). Because of the assumptions involved with one dimensional consolidation theory, the calculated coefficient of permeability from these methods are rarely in agreement with each other or with direct measurements.

Tavenas *et al.* (1983) suggest that in the case of consolidation of clays, using permeability and compressibility of clays as independent parameters is advantageous from analysis considerations. Though the interdependence of permeability and compressibility is known qualitatively, none of the existing methods are useful to link these parameters. Of the two, permeability coefficient is difficult to estimate in a soil system. The Kozney-Carman equation for describing the permeability of coarse grained soils and silts, explains the variation of coefficient of permeability with void ratio. When this equation was applied to fine grained soils many difficulties were encountered and are explained in terms of factors like deviations from the law in the form of threshold gradient, unequal pore sizes etc. (Olson 1962).

Tavenas *et. a*l. (1983) ruleout the role of threshold gradient affecting the permeability results and show that velocity vs hydraulic gradient relationships are straight lines passing through the origin confirming the validity of the Darcy's law over a wide range of hydraulic gradients (0.1 to 50).

With regard to unequal pore sizes being responsible for the apparent discrepancies, it should be stated that Darcy's law in no way describes the state of flow in individual pores. It represents the microscopic equivalent pore corresponding to pressure in equilibrium.

From the above discussions, it is clear that it is advantageous to relate the compressibility and permeability of finegrained soils. In the present investigation, these properties for bentonite sand mixes are studied. The possibility of proportioning the mixes within the frame work of the proposed approach mentioned below, is also examined.

Proposed Approach

It is well established that the relationship void ratio, e vs $\log p$ (p the consolidation pressure) or q (the shear strength) for saturated uncemented fine grained soils is linear (Hvorslev, 1960, Balasubramaniam and Chowdary, 1978). These relationships are different for different types of soils. Wroth and Wood (1978) have shown that at liquid limit values, all the soils have the same order of equilibrium pressure (5 to 6 kPa) with a shearing resistance of about 1.7 to 2.2 kPa. This indicates that the void ratio at liquid limit ($e_L = W_L G$, W_L is the liquid limit) can be used as a normalizing parameter for the analysis of consolidation and shear strength behaviour of saturated uncemented fine grained soils and leads to equations of the form

$$e/e_L = a - b \log p \text{ and}$$

$$e/e_L = a_1 - b \log q$$

Nagaraj and Srinivasa Murthy (1986), and Srinivasa Murthy, Vatsala and Nagaraj (1988) have generated equations of the above form and indicated the advantages of this approach. It is examined in the following sections whether the above considerations are helpful in understanding the permeability behaviour in relation to compressibility. Both these properties are interdependent and in turn depend on void ratio. Since, it is known that void ratio varies linearly with logarithm of consolidation pressure or shear strength it is examined whether the same mode of variation of k with respect to void ratio is valid. Since void ratio at liquid limit is the normalising parameter in earlier sections, the possibility of generating equations of the form

$$e/e_L = a - b \log p \text{ and}$$

$$e/e_L = c + d \log k \text{ is examined.}$$

Explanations for Liquid Limit as the Reference State

It is a general principle in any field to understand, interpret and generalise a particular behaviour of similar materials with respect to that behaviour at a reference state common to all these materials. One such example is the attempt of Sinha (1981) to generalise the deformation behaviour at homologous temperatures of ice and other materials of similar grain structure by normalisation with respect to their melting points. For soils, liquid limit appears to hold promise as a reference state. It has been observed by several investigators (Russel and Mickle, 1970, Wroth and Wood

1978, Whyte 1982) that the liquid limit of all fine grained soils correspond to a unique equilibrium suction/consolidation pressure of 5 to 6 *kPa* with a shearing resistance of 1.7 to 2.2 *kPa*. Nararaj *et. al.* (1990) have indicated that the above properties of soils at liquid limit are attributable to the existence of the same pattern of micro structure. These interpretations can be strengthened from permeability considerations since permeability is an indirect reflection of the microstructure of pores. Test data on permeability shown in table 1 (Shashikumar, 1989) indicate that at liquid limit water contents the permeability coefficient *k* is of the same order for all soils, though there is a seven fold variation in void ratio. Perhaps, this might be the reason why the liquid limit state can be attributed to the same level of effective stress (5 to 6 *kPa*) with shearing resistance of about 2 *kPa*. Infact Terzaghi as early as 1926 intuitively expressed that two soils of similar origin will have similar behaviour if their liquid limits are identical. These unique conditions at liquid limit reaffirm that it represents a datum state in relation to which all other states and stress conditions can be examined.

Materials Used

Commercially available bentonite clay from Kolar district in Karnataka state was used in the investigation. Medium sand passing through 425 μ and retained on 75 μ sieve is added in varying amounts to form bentonite sand mixes.

Liquid limit tests were done for pure bentonite and various bentonite sand mixes using cone penetrometer apparatus (BS 1377-1975). The properties of bentonite and bentonite sand mixes are shown in table 2. The

TABLE 1

Permeability at liquid limit water content of clays

Soil Type	Liquid limit w_L (percent)	Void ratio at liquid limit e_L	Coefficient of permeability (10^{-7}cm/sec)
Bentonite	330	9.240	1.28
Bentonite + Sand	215	5.910	2.65
Natural Marine soil	106	2.798	2.56
Air dried Marine soil	84	2.234	2.42
Oven dried Marine soil	60	1.644	2.63
Brown soil	62	1.674	2.83

TABLE 2

Details of bentonite sand mixes (by weight)

Percent bentonite	Percent sand	Liquid limit (percent)
100	—	330
59	41	195
49	51	162
36	64	117*
65	35	215*

*test mix

mixes with 36 percent and 65 percent bentonite content having 117 percent and 215 percent liquid limit values are chosen as test mixes.

Experimental Procedure

Consolidation and permeability tests were carried out using Rowe's consolidation cells supplied by Geotechnical Digital System. The advantages of this system are that:

(*i*) the loading on the soils is through hydraulic loading system making it less susceptible to vibration effects.

(*ii*) the drainage and pore water pressure measurements and control facilities are available.

The size of the sample is 76 mm in diameter and 19.4 mm in height. The mixes were initially mixed at liquid limit water contents in the form of slurry and allowed for uniform distribution of moisture. The slurry is transferred to the rings and pressures applied. The sample is allowed to consolidate under each pressure and a falling head type of permeability test is performed. The flow is assumed to be entirely vertical and the coefficient of permeability is calculated using the formula,

$$k = 2.303\,(a/A)(L/t)\,\log\,(h_0/h_1) \tag{1}$$

Where a is the area of cross section of the burette

A is the cross sectional area of sample

L is the height of sample

h_0 is the initial excess head of water applied through the burette at time $t = 0$ and h is the excess head of water after time t.

Load increment ratio of one is adopted for consolidation.

The mixes are loaded upto 800 kPa and permeability is measured at every load increment after equilibrium is achieved.

Results and Discussion

Fig 1(a) shows the flow curves for the three mixes studied. When they are normalized with liquid limit values, an equation of the form:

$$w/w_L = 0.537 + 0.023\ \mathrm{D} \tag{2}$$

with a correlation coefficient of 0.979 is obtained and is shown in Fig. 1(***b***).

The ramifications of the above equation are that at any level of penetration the resistance to induced shear is mobilized at the same w/w_L values irrespective of the type of mix.

Further the depth D in a cone penetration test is a measure of the shear strength of soil at that water content or normalized water content ratio for generalisation purposes.

For a particulate material, the shear strength is a function of confining pressure and in turn the permeability is affected. The leads to equations of the form

$$w/w_L = e/e_L = c - \boldsymbol{d} \log p \tag{3}$$

$$= c + \boldsymbol{d} \log k \tag{4}$$

where $e_L = w_L\ G$

Fig. 2(a) shows the e—log p plots for the mixes studied. When the relationships are normalised with respect to e_L and equation of the form (Fig. 2b)

$$e/e_L = 1.437 - 0.447 \log p \tag{5}$$

with a correlation coefficient of 0.979 is obtained.

Fig. 3a) shows the relation between void ratio and permeability. As before, when void ratios are normalized with respect to void ratio at liquid limit an equation of the form

$$e/e_L = 3.167 + 0.313 \log k \tag{6}$$

with a correlation coefficient of 0.962 is obtained and is shown in Fig 3(*b*).

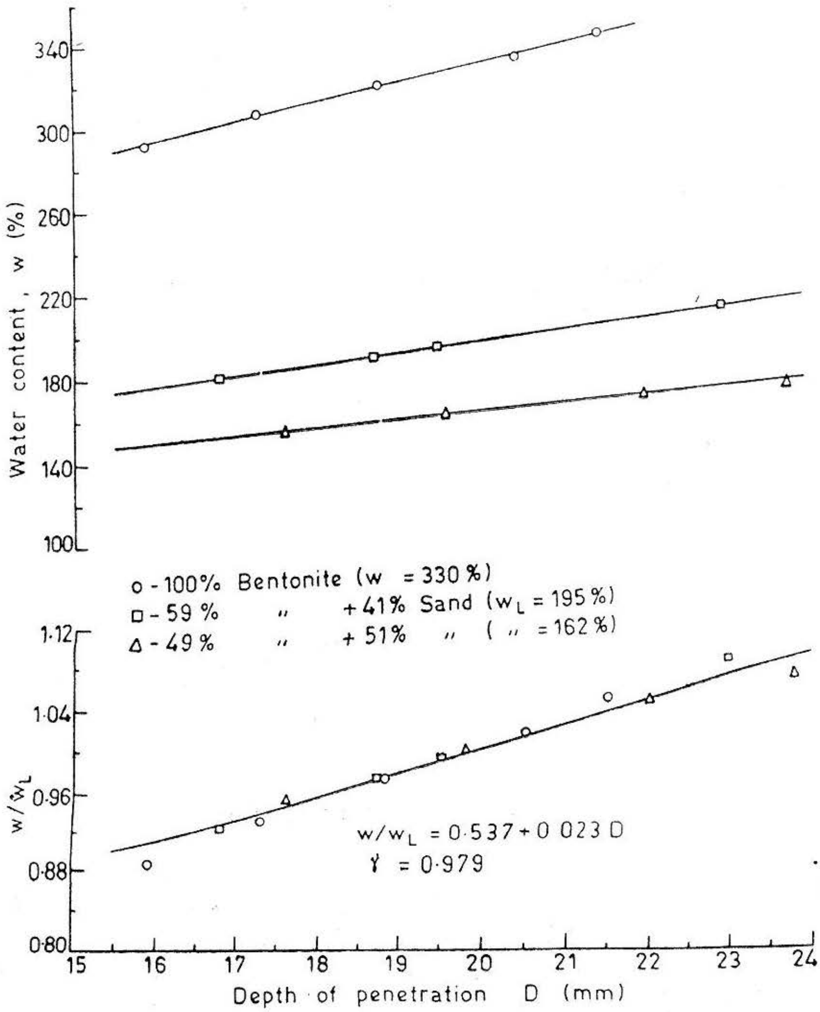

FIGURE 1(*a*) Flow lines for different mixes
FIGURE 1(*b*) Normalized flow line

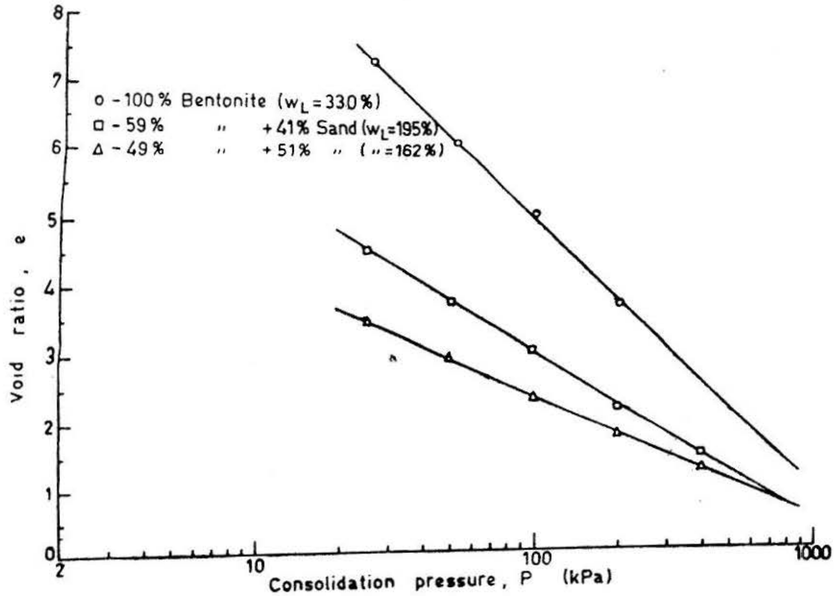

FIGURE 2(*a*) e-log p paths for the mixes studied

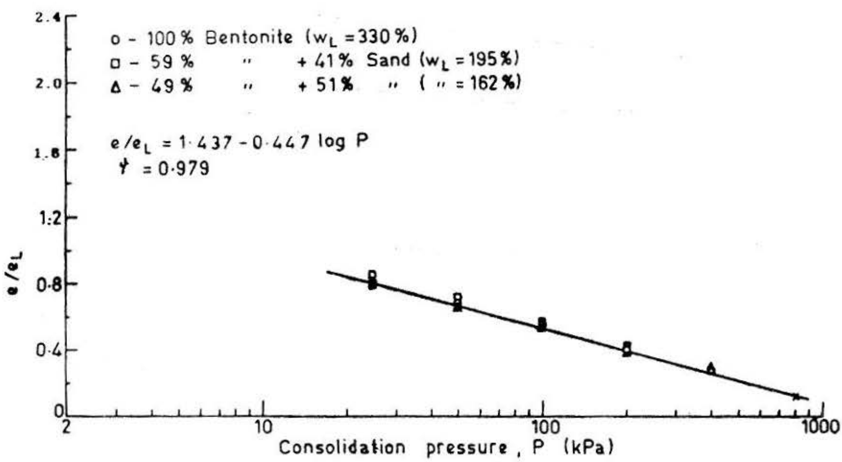

FIGURE 2(*b*) e/e_L-log p paths for the mixes studied

The mixes contain varying amounts of bentonite added to fine sand. The bentonite clay particles, due to very large specific surface, form a coating around the coarser particles, sands, thus preventing a direct contact between them. In essence, the coarse particles float in a matrix provided by the bentonite clay particles. This is in accordance with the earlier observations (Pandian and Nagaraj 1990, Pandian *et. al.* 1988. Srinivasa murthy, *et. al.* 1987) that coarser particles only dilute the physico-chemical potential of a

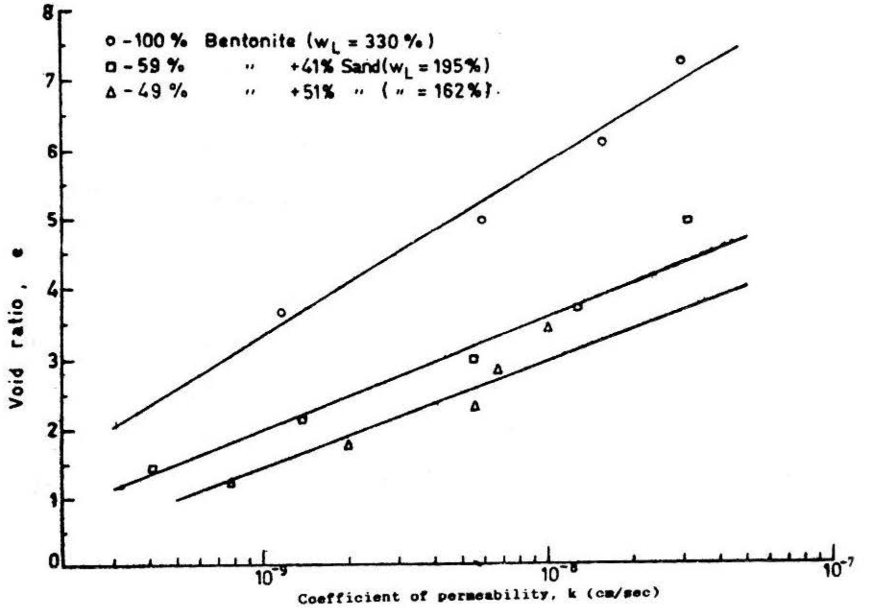

FIGURE 3(*a*) e-log K relation ships

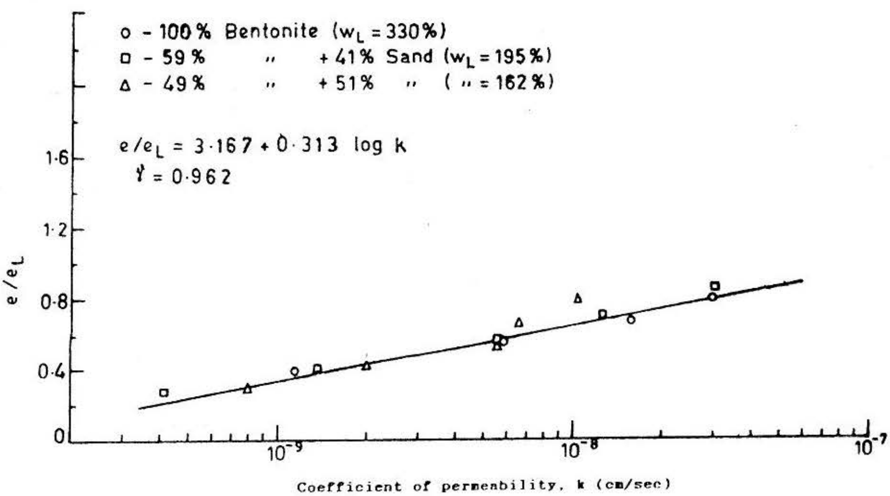

FIGURE 3(*b*) e/e_L-log K relationship

soil proportionately. Thus the physico-chemical potential of bentonite reflected by its liquid limit is reduced from 330 percent to 162 percent upon adding varying amounts of sand. Further the reduction in physico-chemical potential is manifested in the slopes of e—log p and e—log k relation too. Thus, the mixes with a higher percent of bentonite are steeper when compared to the lines of low percent bentonite content and upon normalization,

TABLE 3a

Measured Values of Coefficient of Permeability for LL-117 percent with respect to Predicted values

e	p kPa	e/e_L	measured k (10^{-8}) (cm/sec)	predicted k (10^{-8}) cm/sec)
2.327	25	0.748	4.000	1.870
1.991	50	0.639	1.280	0.840
1.525	100	0.490	0.662	0.280
1.176	200	0.378	0.400	0.123

TABLE 3b

Measured Values of Coefficient of Permeability for LL-215 percent with respect to predicted values

e	p kPa	e/e_L	measured k (10^{-8} (cm/sec)	predicted k (10^{-8} cm/sec)
4.815	25	0.813	1.710	3.010
4.105	50	0.694	1.110	1.260
3.224	100	0.545	0.422	0.419
2.925	200	0.493	0.361	0.286

with void ratio at liquid limit give rise to relations of the form (5) and (6). Tables 3(*a*) and 3(*b*) show the coefficient of permeability values predicted from equation (6) with respect to the measured values for the test mixes. From the tables it is clear that the measured and predicted values are in the same order which lends support to the predictive capability of the model.

Practical Implications

In seepage control, clay cores and cutoff walls should be compatible with the surrounding material interms of compressibility and strength.

This can be ascertained by collecting undisturbed samples and testing. The properties in both the cases should bear constant ratio with respect to liquid limit. Knowing the desired permeability from compressibility and strength considerations, the clay cores and cutoff walls can be designed (by arriving at the desired quantity of bentonite).

Further, when pure bentonite is used as a slurry, it is subjected to considerable volume changes, it may get squeezed out or peel off from the walls while shrinking. In such cases, coarse sand available in the vicinity can be added in suitable proportions. Another aspect of significance is that pure bentonite slurry can be poured into narrow trenches, when seepage control is the only criterion. However if compressibility is also the factor and if the space restrictions do not permit compaction to be carried out, sand can be added to the pure bentonite. The slurry so obtained can also be poured into the narrow trenches. This results in decrease in compressibility at the same time keeping the permeability coefficient in the same order, as bentonite particles coat the sand grains, with the result seepage control is still effected.

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