Evaluation of Swelling of an Expansive Clay Shale from Mae Moh, Thailand

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SUMMARY The instability problems and some failures of diversion channel linings in Nothern Thailand have been caused by the presence of expansive clay shales. The clay shale found in the vicinity of the Mae Moh diversion channel is a fine-grained, fissured material. Its considerable clay fraction is responsible for adverse engineering properties such as swelling, slaking, creep and strain-weakening, all of which contribute to the progressive reduction in strength. The purpose of the investigation conducted on the Mae Moh diversion channel was to identify and classify the expansive nature of the clay shale. A discussion of various methods in predicting the swell potential of an expansive clay shale is presented, with particular reference to the oedometer based results.

1. INTRODUCTION

This paper describes about the results of some laboratory tests conducted on block samples of an expansive clay shale obtained from the right bank of the Mae Moh diversion channel in Thailand. The swelling nature of this clay shale presents slope stability problems along the diversion channel. Several types of tests were conducted to investigate the properties of the clay shale. The tests were conducted primarily using the oedometer, the triaxial apparatus, the hydraulically pressurized consolidometer and the pressure membrane apparatus. Three different types of samples, namely the undisturbed, the remoulded and the pressurized samples were used. This was done to study the change in properties with regards to the different methods of specimen preparation in the laboratory condition. Only the oedometer results are discussed within the scope of this paper.

2. PROPERTIES OF MATERIAL

2.1 Particle Size Distribution

The particle size distribution of the tested clay shale is shown in Fig. 1. The silt and clay fractions are 50% and 20% by weight, respectively. The clay size fraction is 30%, which makes the activity of the crushed (pulverized) clay in the order of 0.6, according to the definition proposed by Skempton (1953). His suggestion indicates that this clay shale was probably subjected to normal climatic weathering and deposited in fresh water with illite and kaolinite as major fine grained minerals. The subsequent sedimentary process has resulted in the genesis of clay shale.

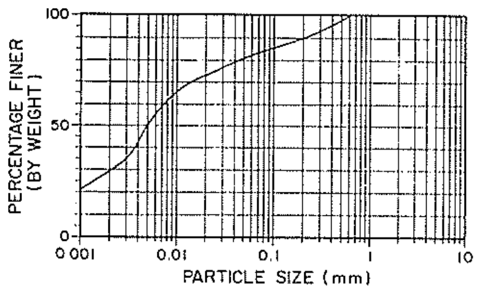

Figure 1 Particle size distribution of clay shale

2.2 Optical Microscope Test

The Becke line test showed that the fringe blur of some minerals moves out indicating that the refractive index is less than that of Canada Balsam (1.537). Birefringence was moderate and the relief was low. This observation suggests the presence of montmorillonite in the smectite group. The value between the maximum and minimum vibrational directions of montmorillonite $(n_{\mu}-n_{\alpha})$ was 0.021. The Becke Line test also showed that the fringe blur moved in indicating that the refractive index was more than that of the Canada Balsam. This further suggested the presence of illite which is the predominant mineral component of the clay shale.

According to Jiang and Peacor (1990) mixed layers of illite/smectite can occur in a variety of weathering, sedimentary, digenetic, metamorphic and hydrothermal environments. They also confirmed the prograde transition of illite to smectite during burial digenesis and very low grade metamorphism and alteration associated with hydrothermal activity. These observations suggest that this transformation may have resulted in this particular clay shale becoming more expansive with age.

3. TEST RESULTS

3.1 Free Swell Test

The free swell of this clay shale was 1.8. According to Sritharan & Rao (1988) this can be classified as a clay of medium expansivity. It is of importance to note that the free swell procedure is not designed to provide the actual magnitude of swelling under field conditions, but for the purpose of identification only. The free swell test is primarily introduced to gauge the expansivity of the clay fraction rather than providing a specific quantification.

3.2 Double Oedometer (DB) Test

The upper bound for the swelling pressure determined from the Double Oedometer test is approximately 1000 kPa (Fig. 2). The initial void ratio when both specimens were at 25 kPa loading was higher of the saturated sample (0.66) than of the dry sample (0.57). Figure 3 shows that the final compression is about 50% higher for the specimen that was loaded in the wet state than the specimen which was loaded in the dry state. The compression was found to be about the same (proportional ratio) for each stage of loading. The samples tested were almost identical in terms of initial dry density (16.5 kN/m^3), initial moisture content (14%) as well as the initial degree of saturation (62%).

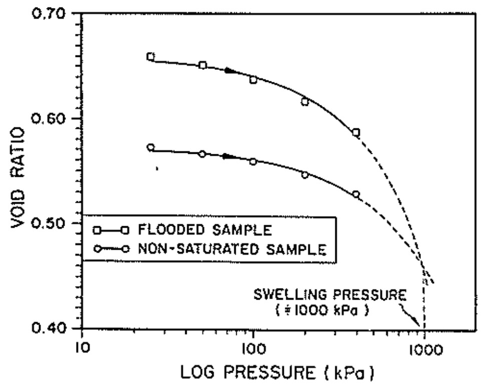

Figure 2 Variation of void ratio with pressure from double oedometer test

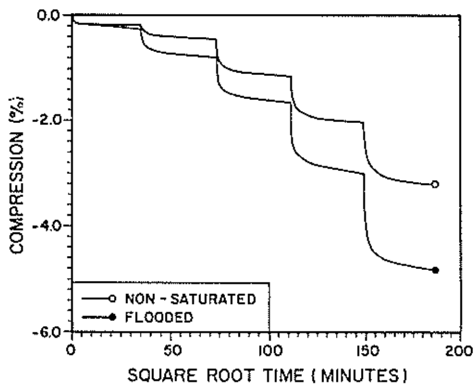

Figure 3 Variation of compression with square root time from double oedometer test

The swelling pressure from double oedometer test was found to be higher than that determined from the other oedometer methods. This is due to the fact that the pressure taken as the swelling pressure is not that which is required to bring the sample to its original height as in the swell consolidation and different pressures oedometer tests. Instead, it is the pressure required to bring the sample to its height after being compressed in the dry state by a pressure equal to the swelling pressure. Weston (1980) stated that the main drawback of this test is that the method is based on the assumption that the swell is independent of the stress or moisture path. According to his findings, the measured swell in the laboratory is about twice that found in the field.

3.3 Swell Consolidation Oedometer (SC) Test

The swelling pressure determined from this test was found to be approximately 400 kPa (Fig. 4). The maximum percentage swelling recorded when the sample was placed under a light load of 6.25 kPa was 0.48%. From the consolidation curve section it is observed that the sample is brought back to a height less than its original height under a load of 400 kPa. The initial values of void ratio, moisture content and degree of saturation are 0.58, 14% and 63%, respectively.

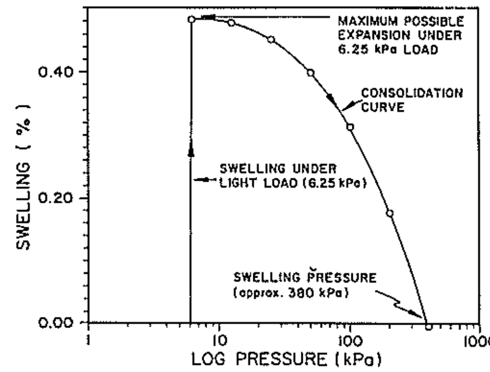

Figure 4 Variation of swelling with log pressure from swell consolidation oedometer test

This method simulates the condition in which the clay shale is allowed to absorb water and swell first before loading. This might not be a good representation of the actual conditions in the Mae Moh diversion channel before it comes into service. By swelling the clay shale before loading, it instigated a higher pressure required to compress the pre-swelled sample than the samples in the Different Pressures Oedometer Method. The higher load required to bring the specimen to its original height can be attributed to the additional energy needed to expel the absorbed water

3.4 Different Pressures Oedometer Method (DPM)

Figure 5 illustrates that the swelling pressure determined from this test is approximately 350 kPa, which is close to that determined by the swell consolidation test. The maximum swelling recorded when the samples were placed under loads of 6.25 kPa, 50 kPa and 200 kPa were 0.48%, 0.34% and 0.13%, respectively, whereas at a load of 400 kPa the sample was compressed by 0.03% of its original height. From the constructed curve joining all the maximum swell positions on the consolidation curve section, it is clear that the sample is brought back to a height less than its original height under a load of 400 kPa.

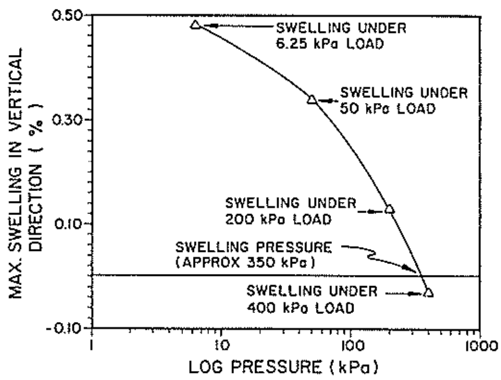

Figure 5 Results of different pressures oedometer method

This method simulates the condition in which the clay shale is loaded before it is allowed to absorb water and swell. This represents more closely the actual conditions in the Mae Moh diversion channel before it comes into service. By loading the clay shale before it is allowed to swell, a lower pressure is required to compress the samples in contrast to the case of swell consolidation oedometer test. Therefore, this test has an advantage over the swell consolidation and double oedometer tests for the determination of swelling pressure. The loading-wetting events in this method follow the same sequence as in the field, where the loads are first applied to the unsaturated soil undergoing compression before it comes in contact with water. This test is carried out on the assumption that all the samples tested are identical.

3.5 Huder Amberg Oedometer (HA) Test

The swelling pressure determined from Huder-Amberg oedometer test is approximately 350 kPa (Fig. 6). The ability of the sample to swell under a load of 400 kPa suggests that the swelling pressure might be greater than 400 kPa, but the test curves intersect at a lower pressure and that this value is considered to be the swelling pressure. This test has an advantage because it takes into consideration the disturbance of the soil and tries to normalize the differences during the introduction of a second loading cycle. Figure 6 clearly indicates that there is a certain degree of disturbance from the first and second loading. The unloading curves show that the initial disturbance was compensated for after the second loading, judging from the similarity in void ratios at the same levels of loading.

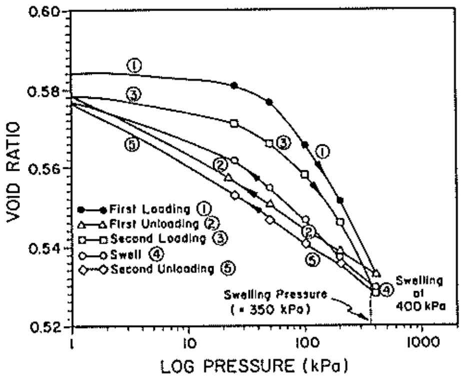

Figure 6 Variation of void ratio with pressure from Huder-Amberg oedometer test method

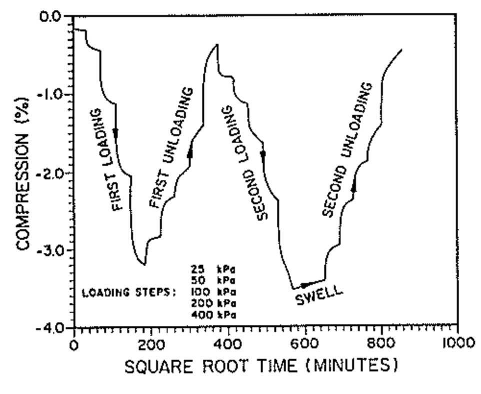

Figure 7 Variation of compression with square root time from Huder-Amberg oedometer test method

Figure 7 illustrates the compression induced by different loading and unloading conditions. The maximum swelling recorded when the sample was placed under a maximum load of 400 kPa was 0.112%. For the Huder-Amberg test, the values of the initial void ratio, moisture content and degree of saturation are 0.58, 14% and 61%, respectively. Witke (1980), in his study of tunnels situated in gypsum keuper (a swelling rock), concluded that the Huder-Amberg oedometer test results proved useful and gave reliable swelling pressure estimations. He also suggested that this test can be conducted reliably for the determination of swelling pressure. The final unloading curve in Fig. 7 can be approximated to a straight line in the semi-logarithmic plot and this result is in accordance with the results discussed by Witke (1980).

3.6 Variable Density Oedometer Test

The void ratios of samples compacted at different initial dry densities under specific loads is illustrated in Fig. 8. For the sample compacted at an initial dry density of 13.5 kN/m^3 (R1), the final void ratio (0.935) was lower than the initial void ratio (0.941). However, the final void ratios of the samples R2, R3, R4 and R5 indicate final void ratios greater than their initial void ratios. This is a result of the increased initial dry densities for samples R2-R5. Figure 9 shows that the swell under a load of 25 kPa is generally higher for samples compacted at a higher initial dry density. As expected, samples with lower initial dry densities are more compressive than those compacted at higher initial dry densities. The compression index, C_c, appears to decrease with the increasing initial dry density. The swelling index, C_s, of the sample R1 (lowest dry density) is quite high (0.118), while that of the other samples is rather consistent and converges to approximately 0.027. The magnitudes of C_c and C_s were based on the void ratios corresponding to the loading pressures of 200 kPa and 400 kPa for each test.

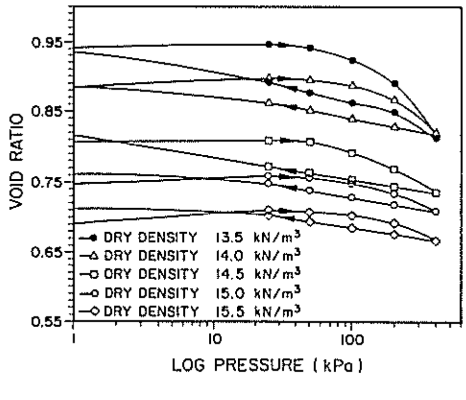

Figure 8 Results of variable density oedometer test

The test on remoulded clay shale was carried out to determine the variation of the compression index, swell index and the ratio C_s/C_c for the samples with different initial dry densities. The samples were initially saturated at a constant volume in order to overcome the swelling tendency of remoulded samples. The shape of the rebound curve is related to the unbending and slippage at particle contacts (Cepeda, 1987). The ratio C_s/C_c (0.19-0.46) shows that the material is a relatively active clay. Furthermore, the test results obtained for the Mae

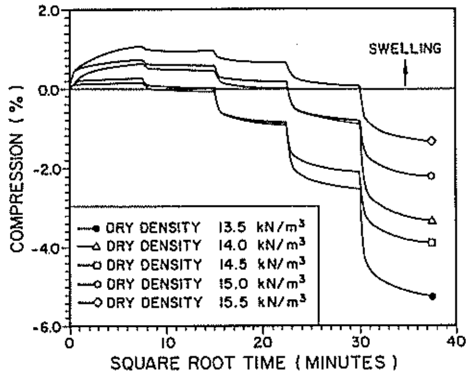

Figure 9 Variation of compression with square root time from variable density oedometer test

Moh clay shale are in accordance with the findings of Cepeda (1987) who has proposed the relationship: C_s = 0.0008 to 0.003 LL for reconstituted as well as fresh shale. This test may be used to simulate the field conditions whereby expansion could be quantified for the case when the cementation of the clay shale was suddenly destroyed, and saturation allowed under heavy load prior to subsequent unloading (Husin, 1991).

3.7 Pressure Membrane Oedometer Test

The variation of void ratios with applied pressure for samples remoulded at different air pressures is shown in Fig. 10. The initial void ratio of the sample subjected to a pressure of 300 kPa (3 bars) was greater than that of the sample subjected to a pressure of 1200 kPa (12 bars). The final void ratio of the samples ranges from 0.97 to 1.03. The range of ultimate compression was between 8.7% to 10.7% as shown in Fig. 11. The compression index, C_c, for all the specimens is approximately 0.26. The swelling index, C_s, is approximately 0.032 for the samples subjected to applied pressures of 300 kPa and 600 kPa and is 0.017 for the sample subjected to an applied pressure of 1200 kPa. The ratio C_s/C_c is found to be about 0.12 for the samples pressurized at 300 kPa and 600 kPa and 0.07 for the sample subjected to a pressure of 1200 kPa. The test showed that the compressibility may not be directly

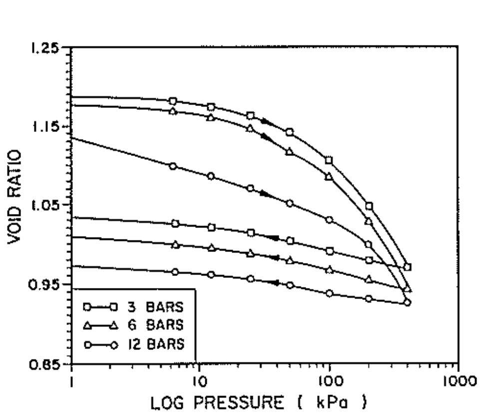

Figure 10 Results of oedometer test on samples prepared in the pressure membrane apparatus

related to the suction $(u_a - u_w)$ at moderate applied pressures. The compression indices (0.26 and 0.28) and swelling indices (0.030 and 0.034) are about the same for the samples pressurized at 300 kPa and 600 kPa, respectively. While the compression index for the sample pressurized at 1200 kPa is similar to that of the samples pressurized at lower air pressures. Its swelling index seems to be smaller (0.017). This indicates that the swelling potential decreases as the applied air pressure, hence suction head increases. The ratio of C_s/C_c ranges from 0.07 to 0.12.

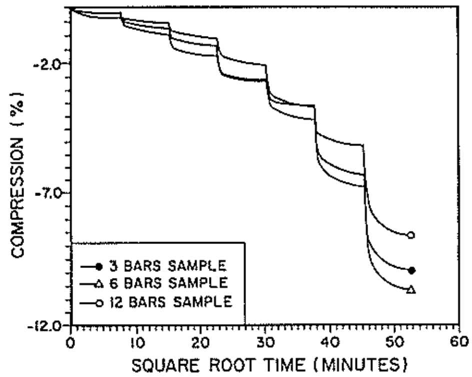

Figure 11 Variation of compression with square root time from pressure membrane test

4. RELATIONSHIP BETWEEN SWELL POTENTIAL AND SOIL PROPERTIES

A large number of undisturbed samples were collected and their free swelling behaviour was evaluated. These tests were conducted with the aid of the Electricity Generating Authority of Thailand. The Atterberg limits of the clay shale material and the particle size distribution (clay %) were considered as the basic governing properties. The plasticity index (PI) and the activity of the clay shale were determined for this wide array of undisturbed specimens. Figure 12 represents % swell against the plasticity index. Although the results indicate significant scatter, two relationships (linear and non-linear) based on regression analysis can be proposed, where % swell increases with the plasticity index. Figure 13 illustrates the variation of clay shale activity with the % clay size, according to the USBR swell classification chart. The Mae Moh clay shale indicates % swell mainly in the region 3-10%, hence its degree of expansion can be classified as medium-high.

5. CONCLUSIONS

The experimental results showed that the clay shale was highly cemented. The material contained some active and expansive minerals (smectite group) which contributed to its moderate expansive behaviour. Consequently, the construction of the Mae Moh diversion channel required careful planning, as slope instability was introduced by the swelling nature of these clay shales.

The swelling pressure was about 400 kPa for the majority of the tests conducted. The free swell type of oedometer tests have a limitation in that it allows volume changes and incorporates hysteresis into the estimation of the in situ

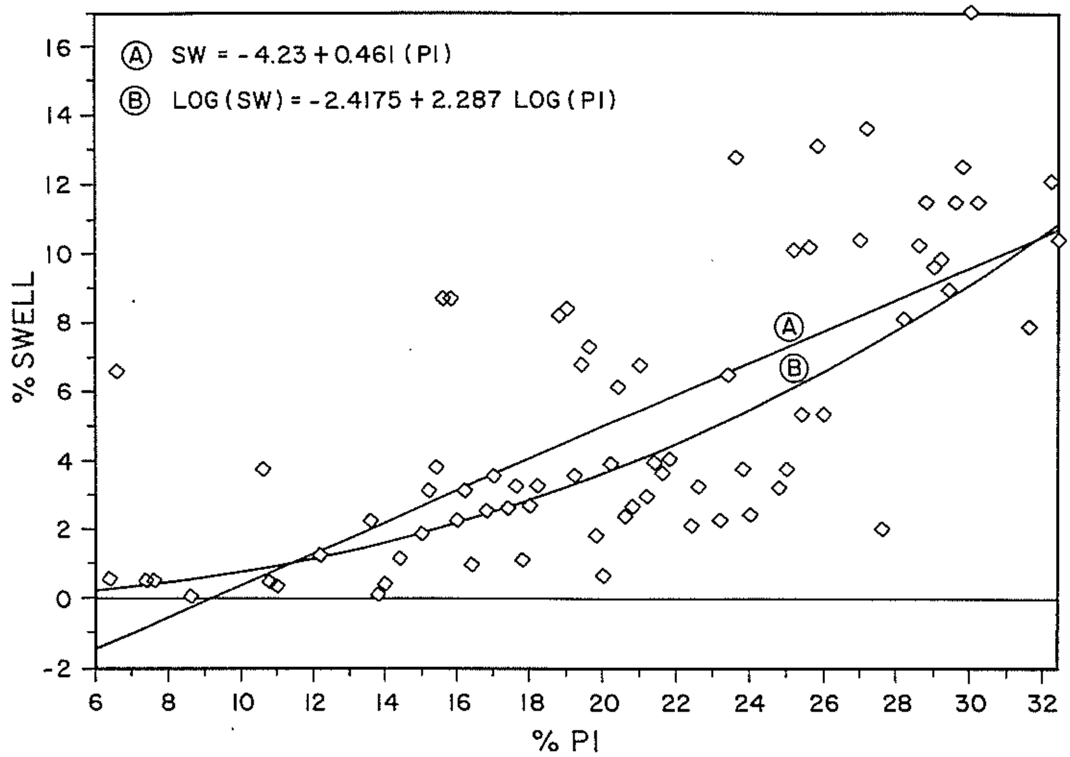

Figure 12 Variation of percentage swell with plasticity index

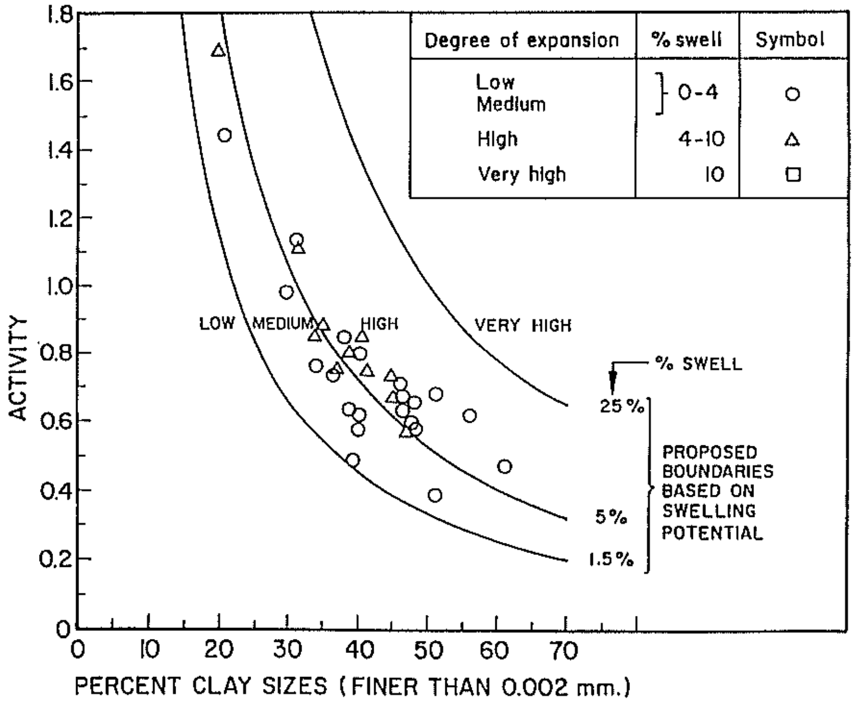

Figure 13 Variation of clay shale activity with percentage of clay size

stress state. The Huder-Amberg test was found to be the best test for measuring the swelling pressure. A rather high expansion was recorded when the sample was tested in its unconfined state. Although not discussed in detail in this paper, a low expansion was observed (less than 1%) when the sample was placed under a light load of 6.25 kPa and allowed to swell only in the vertical direction. Moreover, differential swelling was also observed in the sub-parallel directions. Therefore, the swelling of the clay shale was possibly due to the mineral orientation as well as the horizontally bedded fissures (Husin, 1991).

In the field conditions, the problem of the variation in soil properties (vertically as well as horizontally) exists, particularly at the layer boundaries. The studies of a particular swelling soil which is interbeded between non swelling soils such as sand or silt proves to be even more complicated when an attempt is made to relate the laboratory results to the field conditions. The presence of the non-swelling soil layers will reduce or even arrest the swelling of the composite layer.

5. ACKNOWLEDGEMENTS

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