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SUMMARY

This lecture summarizes the progress of the development of the different ground improvement techniques in Bangkok Plain and is reviewed in relation to the extensive deep well pumping and the resulting piezometric drawdown at deeper depths. Case records of test embankments with sand drains, sand wicks and PVD are presented to illustrate the effectiveness of different types of vertical drains with surcharge. The use of prefabricated drain seems more superior than the other types of sand drain. Construction of test embankment, criteria for PVD selection and relevant laboratory testing techniques are discussed for formulating the specification. Alternative technique, such as, the use of deep chemical mixing is reviewed in terms of the efficiency of cement, lime and flyash as additive. Vacuum pre-loading with surcharge and elctro-osmotic consolidation are also explored as possible techniques to improve the engineering properties of soft Bangkok clay.

1.0 Introduction

The city of Bangkok has experienced extensive infrastructure developments in the last two to three decades with a population of about eight to eleven Million when the outskirts of the city is also included. Tall buildings, deep basements, expressways, water pipe line networks, sewerage pipes, pipe lines for natural gas supplies are many of the construction activities that take place and there is also the desire to have a major second International Airport and MRTA systems.

The Bangkok subsoils which is a part of the larger Chao Praya Plain consists of a broad basin filled with sedimentary soil deposits which forms alternate layer of sand, gravel and clay. The profile of the surface of the bed rock is still undetermined, but its level in the Bangkok area is known to be between 550 m to 2000m. The aquifer system beneath the city area is undoubtedly very complex. Deep well pumping from the aquifer over the last fifty years or so have caused substantial piezometric draw down in the upper clay layer which is soft and highly compressible.

The soft Bangkok clay in the lower Chao Praya Plain extends to 200-250 km in the East- West direction and 250-300 km in the North - South direction. The thickness of the soft to medium stiff clay in the upper layer varies from 12 to 20 m while that of the total clay layer including the lower stiff clay is about 15 to 30 m. A typical soil profile at the Pom Prachul Dockyard site is presented in Fig.1. Thicker deposits are found close to the Gulf of Thailand and thickness decreases towards the North. Because of the soft compressible Bangkok clay, ground improvement techniques have been attempted for nearly thirty years. The lecture presented today is confined to the possible alternative methods considered for ground improvement for the soft Bangkok clay in the lower Chao Praya plain.

The design concepts found most suitable were preconsolidation with vertical drains and soil fill, deep soil improvement, piles supporting either a free spanning concrete plate or a paved embankment and finally the replacement of the natural soil with light fill materials. Of these methods, the pre-loading with vertical drains was found to be an attractive method as similar methods were used successfully in other soft clay deposits in Singapore, Taipei, and others. Alternative pre-loading techniques with the use of earthfills, water in lined ponds, vaccum beneath an impervious membrane, the lowering of ground water table and eletro-osmosis are also considered. In the current lecture the experiences with pre-loading using sand fills and vertical drains will be discussed in detail. Also presented are some data from the use of vacuum pre-loading with and without surcharge fill and some laboratory studies on the use of electro - osmosis combined with prefabricated vertical drains (PVD) as well as laboratory studies on deep chemical mixing.

2.0 Preloading With Surcharge and Vertical Drains

Attempts to use pre-load with surcharge and vertical drains date back to as early as 1967 where the technique was not explored adequately to overcome the complex soft clay conditions, which is further aggravated by the piezometric draw down due to deep well pumping.

2.1 Sand Drains used in the Bangkok -Siracha Highway

Eide (1977) reported the results of a test section on the Bangkok - Siracha Highway. The subground condition along the highway route was considered to be very soft. Sand Drains of 0.2 m diameter were installed by the displacement method. They were placed in triangular pattern at 2.0 m spacing. The observed settlement during two year period after the construction is shown in Fig. 2. Sand Drains and jute drains increased the rate of settlement but not to a sufficient degree. The most negative aspect quoted was that even though the sand drain accelerated the consolidation in the first 18 months, yet even at the end of this period the rate of settlement was still as much as 0.03 m/month which was considered high. Due to the lower factor of safety, a substantial part of the total settlement was probably due to undrained creep.

2.2 Sandwick Drains at the Naval Dockyard Site, Pom Prachul, Bangkok

The performance of sandwicks in accelerating the consolidation of soft Bangkok clay was studied in a full scale test embankment at Pom Prachul Dockyard and was reported by Balasubramaniam (1980). The test site is located at the mouth of the Chao Praya River in Samut Prakarn Province, approximately 20 km south of Bangkok. The soil profile of the site is already presented in Fig. 1. The embankment was 90 m long and 33m wide, devided into three sections, namely; a section without drain, a section with drains of 2.5 m spacing, and a section with 1.5 m spacing. The plan and elevation of the embankment is shown in Fig. 3. The sand drains consisted of small diameter (0.05m) sandwicks and were installed by the displacement method. The finished sandwicks extended to a depth of 17 m below the ground surface. The embankment was built in two stages, firstly, to a height of 1.45 m and secondly, to a final height of 2.35 m. The settlement records taken have been compiled and plotted along with lateral movements are illustrated in Fig. 4. It was observed that, most of the settlement took place in the upper 5 m of layer. Furthermore, there was no difference in the settlement behavior of the section without drain and the section with 2.5 m spacing. The section with 1.5 m spacing produces larger settlements and the settlements below 7.5 m depth were almost the same for the three sections. Thus, these observations based on the settlement records indicated that the sandwicks did not perform well especially at deeper levels. This can be expected as the embankment surcharge was only 40 kPa and this value is close to the preconsolidation pressure of the clay. One would not expect a larger effect from the vertical drains, when the surcharge load does not exceed the preconsolidation pressure.

2.3 Nong Ngo Hao Test Embankment with Sand Drains (1983)

In 1983, the most extensive sand drains studies were performed as part of the investigations related to the new airport at Nong Ngo Hao site in Bangkok. The project site is located at Nong Ngu Hao in the Samut Prakarn province, about 30 km east of the Bangkok Metropolis. The soil profile is relatively uniform consisting of a thin weathered clay crust overlying a layer of soft Bangkok clay of approximately 12 m thick. A stiff clay layer underlies the soft clay layer and extends to a depth of 20-24 m below the existing ground surface. In general, the piezometric pressures at depths greater than 6 m are measured to be less than the hydrostatic pressure. The site is covered by pond of fish farming and agricultural usage. The details of the sub-soil condition are given in Figs.5(a) to 5(i).The purpose of the trial tests in 1983 was to ensure the effectiveness of using non-displacement type of sand drains for accelerating the consolidation settlement of soft clay. Sand drains of minimum diameter of 0.26 m were installed to a depth of 14.5 m by water jetting method. The test program included three test areas , one with surcharge fill and the other two with vacuum loading and ground water lowering. Test section 1 was 40 x 40 m in plan and sand drains were installed at 2 m spacing in triangular pattern. A vacuum under pressure of 90 kPa was applied after substantial swelling of the upper 1.3 m of the sand drains with cement- bentonite slurry plug. However, the vacuum load was not successful as several leakage developed and finally the section was covered with a plastic shield. Test section III was similar to the test Section 1 except that the spacing of the drain was increased to 2.4 m. Due to similar problem as in section I, the loading phenomenon was not successful. The test section II was slightly larger than the test section I (40 x 43 m) and pre- loading of 60 kPa was applied in three stages. While difficulties were encountered in maintaining the vacuum load as well as the ground water lowering, the embankment surcharge was found a reliable technique to be used for pre-loading. The instrumentation plan of the section is shown in Fig. 6. Figures 7 and 8 present the observed settlement with time and the settlement profile across the embankment, respectively.

The field trial was not successful in the sense that (i) there was a settlement of 0.4 meter under a sand blanket of 0.7 m thick after a five-month period (Fig 8) and (ii) the settlement across the the section was remarkably asymmetric ((Fig. 8). The observations indicate some sort of hydraulic connections between the sand drains and the first sand layer located at 25 m depth with a piezometer draw down of 12 m (120 kPa). These hydraulic connections which prolong consolidation of the soft clay are probably due to significant piezometric draw down as result of excessive deep well pumping around the Bangkok area. Besides this major disadvantage, there is a tendency for the formation of shear band type of failure which is evident in the lateral deformation profile with depth.

2.4 Prefabricated Vertical Drains (PVD) at the Nong Ngu Hao Site in Bangkok

Following the 1983 study with the use of sand drain at Nong Ngu Hao site, further investigations were carried out at the same site using Prefabricated Vertical Drains (PVD). The details and findings of the study are discussed in this Section.

2.4.1 Embankment Construction, PVD Selection and Stage Loading

Three test embankments (TS1, TS2 and TS3) were 40 m x 40 m in plan dimensions with side slopes of 3:1 as shown in Fig. 9. Initially, no berm was used, However, it was installed at a later date. The embankments were instrumented with surface and deep settlement gauges, hydraulic and open standpipe piezometers and two slope indicator casings (located at the edge of the maximum full height and at the edge of the embankment). The test embankments were located in such a way that interference from adjacent test embankments and from flood control dykes is avoided.

The PVD identified from the published information and economic considerations suitable for Nong Ngu Hao clay are: Amerdrain (408), Castle Board (CS1), Colbond (CX-1000), Flodrain (FD4-EX), Geodrain (L type) and Mebra (MD 7007). The tests pertaining to the suitability of the safe installation are the puncture resistance and the burst strength . For the post installation performance, discharge capacity tests were carried out not only in the straight conditions but also in twisted and deformed states. Based on these results, Mebra, Castle Board and Flodrain were installed at the spacing of 1.0 m, 1.2 m and 1.5 m in square pattern down to a depth of 12 m in the three test embankment TS1, TS2 and TS3, respectively. A sand blanket of 1.0 m height was laid on the excavated ground (-0.3 m MSL) prior to the installation of PVD. After the PVD installation, the sand blanket was increased to 1.5 m. Then, clayey sand was used to raise the embankment to 4.2 m (i. e., 75 kPa of surcharge) in stages. During construction, Stage 1 loading was upto 18 kPa, Stage 2 was taken to 45 kPa, followed by Stage 3 to 54 kPa and Stage 4 to 75 kPa (4.2 m fill height). For TS1 embankment with 1.5 m spacing, a 5m wide and 1.5 m high berm was installed when the surcharge increased from 45 to 54 kPa. The berm width was increased to 7 m when the surcharge was raised from 54 to 75 kPa. For TS2 and TS3, a berm width of 5 m and 1.5 m high was included when the surcharge increased from 54 to 75 kPa. The waiting period was 45 days for TS1 and TS2 with 54 kPa surcharge and this was reduced to 30 days for TS3 which has the closest spacing of PVD. The design waiting period was 105 days for all the three test embankments when the surcharge increased to 54 to 75 kPa. The factor of safety without considering short term condition of machinery live load was generally in the range higher than 1.35 and the lowest value was 1.26.

The tests pertaining to the safe installation of PVD are (i) grab tensile strength, (ii) trapezoidal shear strength, (iii) puncture resistance, and (iv) burst strength. These tests are designated to determine whether the drains can withstand the extreme stresses induced during the installation procedure. For drains with the filter not fixed with the core, these tests are conducted on the geosynthetic filter. For drains with the filter fixed with the core, these tests are typically waived because these drains have already proved their survivability. In this study, it was decided to test the composite (i.e., the filter with core together). The results and the corresponding specifications are tabulated in Table 1. The basic considerations for optimal performance of the drain is that a Prefabricated Vertical Drain (PVD) must have the ability to permit the pore water from the soil to seep into the drain and should be able to transmit the collected pore water along the length of the drain upto the permeable drainage layer. These requirements can be classified into two groups; the first pertains to the permeability (k) of the geotextile, the Apparent Opening Size (AOS) and the criteria for filtration and prevention of clogging. The second group relates to the discharge capacity of the PVD both under straight and deformed conditions with additional factors of reduction due to time as well as filtration and clogging.

High permeability for geotextile is preferred but at the same time, small particles should be minimized from passing through the filter. The general guideline is $k_{geotextile} > 2 k_{soil}$. This criterion is virtually satisfied for all geotextile filter as the permeability of the clayey soil is of the order of 10^{-7} m/sec, while for the geotextile is 10^{-4}m /sec. A basic requirement for the opening size of the filter is that it should be large enough to enable the fine particles to pass through and permit the larger particles to form a natural soil filter adjacent to the filter jacket. The recommended specification for the Apparent Opening Size (AOS) is less than 90 microns. In order for the filter jacket to be effective, it must minimize the soil particles from moving through the filter of the geotextile and the flow must be stabilized. The criterion recommended is $O_{95}/ D_{85} \leq 3$. The retention ability is confirmed if the $O_{80}/D_{80} \leq 24$. A geotextile can also clog if the soil particles become trapped with the fabric structure. The clogging can be minimized if $O_{85}/D_{15} \leq 3$.

The discharge capacity is one of the most important technical characteristics of the PVD. It is defined as the rate of flow through the drain at a unit hydraulic gradient. Thus, the discharge capacity of PVD can be expressed as the product of the longitudinal permeability of the drain and its cross sectional area. In general, the discharge capacity increases non- linearily with the hydraulic gradient. The discharge capacity of the PVD under ideal conditions (straight) is generally much higher than that specified in the design. This is because substantial reduction in the discharge capacity can occur when the condition deviates from the ideal straight condition. For a given hydraulic gradient, the discharge capacity of the PVD depends primarily on the volume of the core available for the filter, the effect of lateral earth pressure on that volume, possible folding, bending and twisting of the drain due to large settlements, infiltration of the fine particles through the filter and the biological and chemical degradation.

Three factors are generally used to describe the influence of time, the deformation of PVD and the filtration and clogging effects on the ideal required discharge capacity calculated from the theory of consolidation. These factors are $F_{(t)}$ for the influence of time $F_{(c)}$ for the influence of deformation, and $F_{(fc)}$ for filtration and clogging. Thus, the actual discharge capacity is given as:

$$q_w = F_t\,(F_c)\,(F_{fc})\,q_{req} \tag{1}$$

The discharge capacity q_{req} is given by the expression:

$$q_{reqd} = \frac{\varepsilon_f x U_{10} x H \pi c_h}{4T_h} \ (m^3/year) \tag{2}$$

where ε_f is the final settlement of the clay layer taken as 25% of the total depth H of the clay layer to be improved with PVD; U_{10} is the 10% of degree of consolidation; c_h is the coefficient of consolidation for horizontal drainage. Values of flow rate, q_{req}, estimated from case histories range from 8 to 53 m^3/year.

Three different laboratory tests were performed on six PVDs to estimate the discharge capacity of the PVD in the laboratory under lateral pressures of 60,100,150 and 200 kPa. These tests also included the test carried out under ASTM testing procedure. The tests were also carried out at low lateral pressures when the PVD is surrounded by soft Bangkok clay. Figure 10 illustrates the variation of discharge capacity with lateral pressures.

The effect of lateral pressure on the discharge capacity depends on several mechanical properties of the filter and core. Generally, most specifications stipulate a discharge capacity of 500 m^3/year when tested under a lateral pressure of 200 kPa and in accordance with the relevant ASTM standard. The value is far below than the actual tested values of most PVD.

The influence factor, F_c under deformed condition is investigated from ten different PVDs in a modified triaxial apparatus when the PVD is 10% bent, 20% bent, 90^0 twisted, 180^0 twisted, 20% bent with one end clamped as well as 30^0 bent with two end clamped. These results are presented in Table 2 and the overall F_c factor estimated is about 2. Filtration tests were conducted on several PVDs and the factor F_c is estimated as 3.5. The factor F_t from the laboratory tests is about 1.25 to 1.5. The discharge capacity q_w of 500 m^3/year is estimated when $F_t = 1.25$, $F_c = 2.0$, and $F_{fc} = 3.5$ with $q_{req} = 53$ m^3/year.

It thus appears for soft Bangkok clay even after considering all the worst cases of the effect of time, deformed shape, filtration and clogging, the minimum discharge capacity is around 500 m^3/ year. Thus, the ASTM specification on discharge capacity of 500 m^3/year under a lateral pressure of 200 kPa and hydraulic gradient of unity is considered quite sufficient for practical purposes.

2.4.2 Interpretation of Data on Deformation and Pore Pressures

Figure 11 illustrates the settlement profile with depth for the test embankment TS3. It is noted that for TS3 in Fig. 11, the calculated final primary settlement profile is somewhat less than the latest measured values after about 660 days, the difference being due to the secondary consolidation settlement and the components of the cumulative settlement due to the lateral movement of the clay under undrained creep. However for TS1 and TS2, the calculated final primary settlements are still higher than the latest measured values after 660 days. In these cases (TS1 and TS2), the primary consolidation is still not yet completed as the drain spacing is high ; 1.5 m and 1.2 m respectively for TS1 and TS2. There is a consistency.The results of TS1 and TS2 are not included here but their trend is very similar to TS3.

Figure 12 illustrates the measured pore pressure profile with depth for the test embankment TS3. Here again the measured data are consistent.The measured pore pressure profile for TS3 indicated that, the excess pore pressure has fully dissipated while in TS1 and TS2, there are still some undissipated pore pressures even if the magnitude are not significantly high. The undissipated pore pressure in TS1 is higher than those in TS2, since the PVD spacing is wider in TS1 (1.5 m) than in TS2 (1.2 m). In all cases, substantial dissipation of excess pore pressure have taken place after the end of construction time and after 660 days. Similar to the settlements, the pore pressure data are only presented for TS3.

Figure 13 illustrates the degree of consolidation as estimated from the pore pressure dissipation results. In all three test embankments, the results are very consistent. The following are salient observations:

- By February 1996 (after 660 days), the consolidation is virtually completed in TS3. However, the increase in values of over 100 percent is possibly due to combined effect of errors in measurement and the assumed final pore pressure profile (Fig. 11) which is also dependent on the seasonal fluctuation of ground water table.
- In the case of TS1 and TS2, the degree of consolidation is less than 90 percent even by February 1996.
- The degree of consolidation show substantial increase after the end of construction (EOC) until June 1995 and then in February 1996 (after 660 days).
- The degree of consolidation is low at depths of 4-8 m possibly due to low c_h values at this depth.

The degree of consolidation obtained from pore pressures (Up) are consistently lower than those from settlements (Us) as indicated in Fig. 14 for all test embankments. The delay in the calculated degree of consolidation from pore pressure observation obtained herein is in accordance with the results of Mikasa's consolidation theory (Mikasa, 1963).

Further insight into the settlement characteristics can be obtained by plotting the rate of settlement (cm/month) versus inverse time (1/month) in Fig.15. The results for all the three test embankments are consistent. Over a period of 20 months or so, the rate of settlement decayed very fast with time in an exponential manner . However beyond a period of 20 to 24 months the rate seem to reach an asymptotic decay with a slope of about 25:1 for all three test embankments. This perhaps may be due to secondary consolidation effect. If the secondary consolidation settlement can be approximated as a linear function of the log time, then it can be estimated that this slope (25:1) is the product of the compressive soil thickness (H) and the secondary consolidation ratio (C_α). Therefore, for the soft soil thickness of H=14 m, the value of C_α = 0.0018 can be obtained. This value of C_α is in the range suggested by Mesri (1973), i.e., C_α = 0.01 to 0.02 for marine clays.

The rate of lateral displacement (mm/month) was also plotted with inverse time (1/month) in Figs 16. The data show that the decay of the rate of lateral movement with time indicates similar trend to the settlement, except that the rate of lateral movement as extrapolated seem to terminate abruptly after about 4 years or so. Thus, the rate of settlement at longer times indicated in Fig. 15 corresponds to a predominantly secondary consolidation rather than undrained creep.

A consolidation settlement-log time plot was made for the test embankment TS3 in Fig.17 with a view to estimate the secondary consolidation parameter, C_α . This graph and those for TS2 and TS3 indicate that the final linear portion has not been reached yet for an easy estimate of C_α . However, based on the slope of this curve of 25:1 (or C_α= 0.018), as obtained from the rate of settlement-inverse time plot, the observations seems to approach a linear section (see Fig. 17)

Figure 18 illustrates the water content - depths profile for TS3 in February 1996 (after 660 days) after pre-loading together with the mean values of the water content as determined in 1973 are also included as dotted lines. Similar water content reductions are noted for TS2 and TS3.

The back calculated values of water content from settlements are given in Fig. 19 for TS3 which is in accordance with the measured data. The following comments can be made :

- In all the test embankments, substantial reduction in water content is noted.
- For the very soft clay from 2 to 6 m, the water content reduction is consistent, i.e., higher reduction in the order of smaller spacing from TS1 to TS3. The reduction in water content in TS3 is more than 20%.
- For the medium clay at the depth of 9 m and downward, the water content reduction in all embankments are almost the same, this may be explained as the effects of sand

lenses existing in this layer that made the same consolidation process even at different drain spacing from 1.0 m to 1.5 m.

The vane strength profile as shown in Fig. 20 for TS3 illustrates a substantial strength increase after a pre-load period of 20 months.

2.4.3 Back Calculation of c_h values from Pore Pressure Measurements

From Hansbo's (1979) equation for consolidation of PVD, the following equation can be derived:

$$1-\frac{\Delta u_t}{\Delta u_0} = 1\text{-}\exp(-\alpha t) \qquad (3)$$

$$\alpha = \frac{8c_h}{D_e^2 F} \qquad (4)$$

where Δu_0 = excess pore pressure at reference time of t = 0
Δu_t = excess pore pressure at time t ;
D_e = effective diameter of unit cell of drain
F = resistance factor for the effects of spacing, smear and well resistance.

From equation (3) one can get,

$$\ln\frac{\Delta u_0}{\Delta u_t} = \alpha t \qquad (5)$$

Therefore, the values of α can be obtained as the slope of the plot $\ln(\Delta u_0/\Delta u_t)$ vs. t. Having the values of α, the coefficient of horizontal consolidation c_h can be calculated from equation 5.

The back-calculated c_h values from the hydraulic piezometer are more reliable and are plotted in Fig. 21 against the increase in effective stress. From this figure, it can be seen that the c_h values decrease consistently with increase in effective stress (with the progress of consolidation) for all depths and in all three embankments. Also seen from this figure is that the weathered crust (2 m) has the highest c_h value and the weakest soil at 6 m depth has the lowest c_h value. The c_h value at 4 m and 10 m are higher than those at 6 m depth. These results are consistent with the profile of degree of consolidation as previously presented.

2.4.4 Back Calculation of c_h Values from Settlement Measurements

Equation (3) can be rewritten in terms of settlement as follows:

$$\frac{S_t}{S_f} = 1\text{-}\exp(-\alpha t) \qquad (6)$$

where S_t - consolidation settlement

S_f - final consolidation settlement
The other terms are explained before.

From equation (6), the following equation can be derived :

$$\ln \frac{S_f}{S_f - S_t} = \alpha t \tag{7}$$

Therefore, the c_h values can be obtained from the slopes α of the ln $S_f/(S_f\text{-}S_t)$ vs. t plot. The average values of c_h (for 12 m improved ground) calculated from three embankments TS1, TS2 and TS3 are presented in Fig. 22 as a function of the increased effective stress. The average c_h value at the end of construction is about 3 m^2/year and tend to reduce to less than 0.5 m^2/year at the full increase of effective stress of 75 kPa. Thus the c_h values obtained from settlement are slightly higher than those estimated from pore pressure back-calculations. This is consistent with the interpretation of degree of consolidation as given in Fig 13.

2.4.5` Finite Element Analysis

The predictions of the settlements and pore pressures during loading and the consolidation stages are made by the Finite Element Analysis using the CRISP program. The analysis can be devided into six steps, namely; calculation of incremental loads, application of the boundary conditions, assembly of the stiffness matrix; solution of the equations; calculation of strains, stresses and displacements, and output of the results. There are number of techniques available for analyzing non-linear deformation problems using Finite Element Methods. CRISP uses the incremental or tangent stiffness approach. During each increment, the stiffness properties appropriate for the current stress are used in calculation. CRISP solves the linear simultaneous stiffness equations using the frontal solution method. The frontal solution in the program uses one dimensional array for solving a symmetric stiffness matrix. Therefore, the problems of matrix behavior which only obey the Associated Flow Rule can be analyzed. In the analysis performed here, the embankment and the foundation materials were modeled as quadrilateral, linear strain elements. The analysis was carried out as a plane strain case. In the Zone improved by PVD, two columns of elements have been simulated for the soil in between two drains. The fill material is treated as a bi-linear element material. The soil properties used in the analysis are summarised in Tables 3 and 4.

Finite element analysis using the CRISP program was used to predict the settlement for three embankments and the results are plotted in Fig. 23. The predicted settlements are very consistent. If there were no PVD, the predicted settlement is about 0.75 m while with PVD, the predicted settlements are 1.25 m, 1.45 m and 1.5 m for TS1, TS2 and TS3 respectively. The predicted lateral movement and pore pressure are shown in Figs. 24 and 25 respectively for the embankment with closest drain spacing (TS3).

3.0 Electro-Osmotic Consolidation

Stress controlled and Strain controlled consolidation tests performed on soft Bangkok clay indicate that there is a sharp reduction in coefficient of consolidation when the effective stress exceeds the preconsolidation pressure. Such an effect was evident both in c_h and c_v values. It is thus not surprising that the back calculated c_h values from the full scale field test embankment reduced with increase in effective stress and the degree of consolidation. The vertical drains are only effective when the pre-loading surcharge is large enough to exceed the preconsolidation pressure and as such greater reduction in c_h values can prolong the pre-loading time especially in natural deposits of soft clays which have a structure developed over the years of deposition and subsequent aging.

Electro-Osmotic consolidation, on the other hand, creates an electro-osmotic permeability k_p which increases with the voltage gradient (See Fig. 26). Rowe's conventional consolidation cell was modified and a series of tests were conducted with different types of PVD with a view to explore whether electro osmosis combined with the use of PVD and surcharges can accelerate the degree of consolidation. The PVD used in these tests included Amerdrain 407, Mebra MD 7007, Colbond CX 1000, Castle Board, Geodrain - L type etc. However, comprehensive series of tests were only conducted with the Mebra MD 7007 drain. These laboratory tests indicate that the use of electro-osmosis does not have any effect on the c_h values which decreased with increase in effective stress. However the electro-osmotic c_h values increased with the voltage gradient.

A maximum water content reduction of 14% was noted with the use of electro-osmotic consolidation and the measured increase in undrained shear strength is of the order of 50 to 80%. These experiments are preliminary in nature and it is recommended that field trials be conducted to further evaluate the potential of the use of the electro-osmotic consolidation with PVD. The currently used PVD are not conductors of electricity and as such, in the laboratory experiments a thin strip of copper wire was wound around the core of the drains. It is certainly deserving to have a better design of PVD which is electrically conductive. Similar research can also be performed to develop geosynthetic materials which can conduct electricity.

4.0 Vacuum Pre-loading with Surcharge

One of the major problems with the use of pre-loading technique in the Bangkok plain is the scarcity of sand for surcharge fill. Also , the serious difficulty in the mobility of such large quantity of sand fill, when the traffic condition without such frequent mobility of trucks is already bad enough in the city of Bangkok and its approach roads and expressways. Additionally, the extended rainy season in Bangkok sometimes make the site inaccessible as the site is often flooded for a period of three to five months every year. These considerations have led to the proposal to explore a combined vacuum pre-load and surcharge technique. The idea is naturally to cut the fill height and to reduce the time of pre-loading. Two additional embankments were thus constructed at the Airport site in Nong Ngu Hao closer to the PVD embankments. The plan area of the test fill was the same as the PVD sections, 40 x 40 m with a platform of 0.3 m and 0.8m sand fill. The PVD spacing was 1.0 m in a triangular pattern. For embankment 1, the vacuum pre-loading with surcharge included a hyper net and the PVD was 15 m long. In the second embankment a perforated pipe was used (no hyper net) and the PVD length was 12 m. The specification of the wick drains are as follows:

Water discharge capacity	:	> $2.5 \times 10^{-5} m^3/s$ after30 days under 350 kPa lateral pressure
	:	> $4.0 \times 10^{-5} m^3/s$ at 25% vertical compression
Permeability	:	> 10^{-5} m/s
Tensile strength	:	> 2 kN
Elongation at 0.5 kN	:	> 2%, < 10%
Filter Tear Strength	:	> 200 N
Soil retention Filter	:	< 80 micron
Minimum Width	:	> 95 mm
Minimum thickness	:	> 3 mm

The drainage layer of the two full scale test embankments consists of geotextile combined with hyper net spacer. The geotextile used is a 136 g/m^2 nonwoven spun bonded polypropylene with a high modulus. The geotextile was placed directly on the top of the vertical drains which are cut off with 15 cm overlength. The perforated pipe consists of 5 rolls of the Mebra tubes which are 80 mm in diameter and weighs 297 gm/m. The perforated pipe was placed inside the drainage sand and connected to the tube which crossed at the end of each Mebra tube. The hyper net consists of a grid of HDPE-threads which crossed each other and are melted together on the intersections. The hyper net has a discharge capacity of 8×10^{-3} m^3/s. One layer of net was placed over the whole area.

At the place where the discharge pipe was connected to the drainage layer, several layers of hyper net were creating a free flow of the pore water. The pore water could travel without interruption directly from the drains into the drainage layer and was not obstructed by the drainage sand or the filters that can clog. The drainage layer also functions as separator of the HDPE liner.

On top of the drainage layer, a water and air tight liner was placed according to an approved quality control scheme.The materials of liner satisfy the following specification.

Material	:	VLDPE
Thickness	:	>1.5mm
Roll width	:	> 10 m
Tensile Strength	:	> 24 N/mm^2
Elongation	:	> 800%
Tensile Impact Strength	:	> 350kN/m^2
Tear Strength	:	> 130N
Environmental Stress Crack	:	> 2000 hrs
Carbon Black Content	:	> 2%
Thermal Stability	:	> 60 minutes

The installation was executed according to the approved quality control system. All welds were made with a hot wedge welder capable of making a double 12 mm weld. All welds were tested with an air pressure for the strength and air tightness.

In order to get an optimal result and a high rate of efficiency, several measures were taken to optimize the system. One of the most important element was the sealing of the borders of the area. Contact with permeable layers and especially with the air was avoided. The end of the liner was therefore being placed at the bottom of a trench with a 30 cm layer of sand-bentonite or cement-bentonite. Thus the border of the liners was completely sealed off from the atmosphere.

The water collection system was connected with a number of vacuum pumps, conventionally used for vacuum dewatering. A knock-out pot prevented pulsating water flow in the drainage system so that clogging of the system was avoided. For an area of about 1500 m^2, one vacuum pump with a capacity of 100 m^3/ hour was provided. The pumps were run under a pressure of 70 kPa and continued for a period of two months. The pumps were connected with a reliable diesel motor for the necessary power supply. A back-up pump was kept ready to co-opt with the failure or the malfunctioning of any of the pump installed at the site. The pumps were placed at the lowest possible level so that a maximum vacuum is realised under the liner.

As soon as the vacuum was applied and the liner was tested against the air tightness and the surcharge was placed on top of the liner. Sand fill free of large pieces of stone were placed on the top of the liner. The maximum surcharge height was 2.5 m. In order to prevent any damage to the liner, the vacuum pressure was monitored during the placing of the first layer of fill.

Summary of Actual Stage Loading and Waiting Period

For Embankment 1

Point (See Fig 28)	Types of Loading	Pressure (kPa)	Duration (days)
A-B	Sand 0.3 m Pumping	80.40	45
C-D	Sand 0.8 m + Pumping	89.40	12
E-F	Sand 1.3 m + Pumping	98.40	12
G-H	Sand 2.5 m + Pumping	120.0	60

For Embankment 2

Point (See Fig. 28)	Type of Loading	Pressure (kPa)	Duration (days)
A-B	Sand 0.8 m Pumping	89.40	45
C-D	Sand 1.3 m + Pumping	98.40	12
E-F	Sand 1.8 m + Pumping	107.40	12
G-H	Sand 2.5 m + Pumping	120	60

The surface settlement profiles of the two embankments are presented in Fig.27. It is noted that the vacuum pre-load is effective in the initial two months period in having about 0.5 m of settlement. The surcharge load applied after that date has caused an equal amount of settlement in the next 50 days, even though the pumping was continued. The major difficulty experienced with the vacuum pre-loading is the maintenance of vacuum pressure due to surcharge loading as well as the fissures and root holes in the clay. Even though a vacuum pre-loading of 75 kPa was anticipated , the actual measured values seem to indicate an efficiency of 40 to 50% equal to a surcharge pressure of 35 to 40 kPa. However, the vacuum pre-load seems more effective in the accelerated consolidation in the upper 3 to 6 m of clay layer at a much shorter time.

5.0 Deep Chemical Mixing

Deep Stabilization of soft clays with lime and cement has been the subject of research for decades in Sweden, Finland and Japan, among other countries. Extensive laboratory studies have been conducted in AIT on the effects of lime, cement and flyash on soft clay with a view to be used in deep stabilization. These tests were conducted under unconfined and confined conditions in the triaxial apparatus with and without drainage. A brief summary of the strength increase by these method would be presented here. The strength increase with flyash on soft clay as the only additive seems rather low for the construction of piles in deep stabilization. Similarly, the effect of lime is also minimum unless the lime is very pure. The cost of such pure lime is more expensive than the cost of the normal lime available in the market. Hence, on the basis of the cost of the pure lime, use of lime is not favoured. These findings have led to the use of cement in the wet process of deep chemical mixing for highways in Bangkok plain. Furthermore, research work is in progress to use chemical additives with Flyash as stabilizing agent. The results on the laboratory tests of lime and cement in Bangkok clay are presented in Fig 28 to Fig. 32

6.0 CONCLUSIONS

6.1 The full scale test embankments constructed at Nong Ngu Hao site with PVD indicated that PVD is a suitable technique for accelerating consolidation in the soft Bangkok clay. The following additional comment can be made.

6.1.1 Laboratory tests performed on PVD are of two types; i) Tests to ensure safe installation, which are grab tensile strength, trapezoidal tear strength, puncture resistance and burst strength (ii) Tests to ensure optimal discharge capacity, which are permeability of the geotextile, Apparent Opening Size and the criteria for filtration and prevention of clogging. The discharge capacity is estimated not only in straight condition but also in twisted and deformed condition of PVD.

6.1.2 A coupled -consolidation - shear deformation analysis with the finite element technique using the CRISP program incorporating Biot's theory of consolidation and Critical State Soil Mechanics model for the soft clay made good predictions of the settlements, lateral movement and pore pressures.

6.1.3 From the field measurement carried out and the analysis performed over an extended period of time from February 1994 to February 1996, the following conclusions are reached and the relevant data are presented.

a) For the test embankment TS3, the 100% primary consolidation is nearly reached over the 20 months period and the excess pore pressure have fully dissipated.

b) For all the test embankments, the degree of consolidation based on the pore pressure measurements are presented and these are the same order of magnitude as those computed from the settlement measurements.

c) The rate of settlement-inverse time plots indicate a C_α of 0.018 for secondary consolidation. This value is with in the range suggested by Mesri (1973).

d) The rate of lateral displacement versus inverse time plots indicate the possible termination of small magnitude of the lateral movement with in four year period.

e) The water content reduction from the field measurements are in agreement with the computed values from the consolidation settlements.

f) The back-calculated c_h values reduce substantially with the increase in degree of consolidation and these values are tabulated for the use in final design.

6.2 The pre-loading technique with PVD caused the effective stress to exceed the pre-consolidation pressure and thus, the field c_h and c_v values are reduced. Electro-osmotic consolidation can be considered as a possibility to be combined with the PVDs, since the electro-kinetic permeability increases with the voltage gradient by electro-osmotic consolidation.

6.3 A combined vacuum pre-load and surcharge fill can be used for the Bangkok soft clay, since the availability of sand is scarce. This technique needs further refinements.

6.4. Laboratory studies on the use of flyash, lime and cement as additives to soft Bangkok clay indicatedthat cement is the most superior additive in achieving higher strength within the optimum specified time of curing.

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TABLE 1 : Comparison of PVD Selection Criteria and Test Results

Properties	Test Designation	Proposed Values	Mebra MD-7007	Flodrain FD-4EX	Colbond CX-1000	Castle Board CS1	Geodrain L Type	Amerdrain 408
Apparent Opening Size, μm	ASTM D4751-87	< 90	75	75	>75	>75	>75	75
Grab Tensile Strength, kN	ASTM D4632-91	> 0.35	0.99	0.42	1.10	1.97	0.50	1.09
Trapezoidal Tear Strength, kN	ASTM D4533-91	> 0.10	0.34	0.15	0.12	0.60	0.05	0.66
Puncture Resistance, kN	ASTM D4833-88	> 0.10	0.273	0.085	0.250	0.383	0.081	0.247
Burst Strength, kN	ASTM D3786-80a	> 900	1382	495	1312	1382	650	1225
Discharge Capacity at 7 days, 200 kPa and Hydraulic Gradient of 1, m^3/yr	ASTM D4716-87	> 500	2340	1620	1670	1415	1685	2120
Discharge Capacity @ 200 kPa and Hydraulic Gradient of 1, m^3/yr	Modified Triaxial (Straight)	> 500	2115	690	985	1610	985	1780
Discharge Capacity @ 200 kPa and Hydraulic Gradient of 1, m^3/yr	Modified Triaxial (20% Compression, Free Bending)	> 500	1270	435	505	1305	870	1445
Discharge Capacity @ 200 kPa and Hydraulic Gradient of 1, m^3/yr	Modified Triaxial (20 % Compression, Twisted 45 Deg)	> 500	1105	500	430	1250	1085	1420
Discharge Capacity @ 200 kPa and Hydraulic Gradient of 1, m^3/yr	Modified Triaxial (20 % Compression, One-end Clamped)	> 500	570	240	400	1330	1075	1040

TABLE 2 : PVD Discharge Capacity Results Using the Modified Triaxial Cell (m^3/yr)

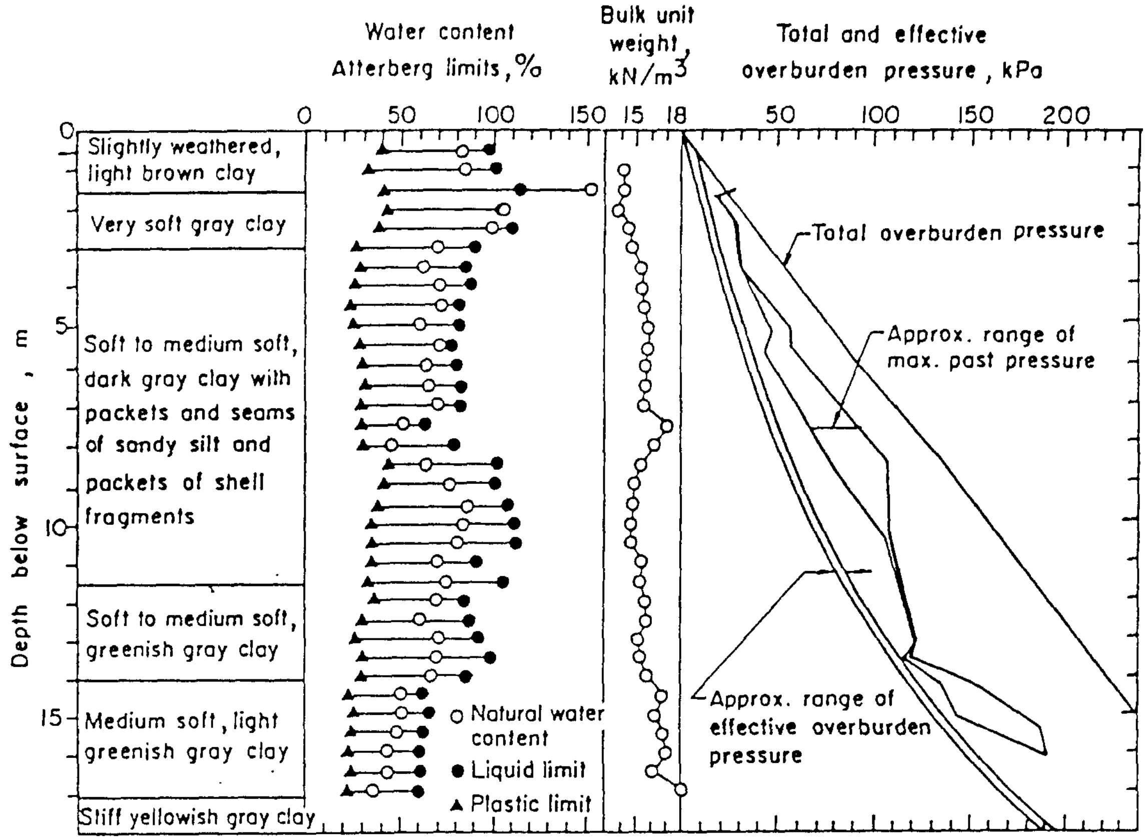
PVD Condition		Pressure (k Pa)	PVD Types					
			MD 7007	FD4-EX	AM 408	CS1	CX-1000	GD "L"
Straight		100	2,525	845	1,985	1,555	985	1,770
		200	2,115	690	1,780	1,515	815	1,375
Free Bending (%)	10	200	1,340	525	1,515	1,415	610	1,100
	15	200	1,290	445	1,480	1,330	545	900
	20	200	1,270	435	1,445	1,305	505	870
Twisted (20%)	90 deg	200	1,105	500	1,420	1,250	430	1,085
	180 deg	200	980	375	1,350	1,240	370	1,065
Clamped	15% one-end clamped	200	740	290	1,080	1,405	450	1,135
	20% one-end clamped	200	570	240	1,040	1,330	400	1,075
	30% two-ends clamped	200	520	130	630	1,270	340	1,055

Table 3: Soil Parameters Used in Settlement and Stability Analyses

Zone	Depth (m)	Z_i (m)	Γ (kN/m^3)	$\bar{\sigma}_{vo}$ (kPa)	$\bar{\sigma}_p$ (kPa)	OCR	CR	RR	c_v (m^2/yr)	S_u (kPa)
1	0.3-2.0	0.85	16.0	12.1	75	6.20	0.30	0.030	10	12.5
2	2.0-5.0	4.2	14.5	28.5	50	1.75	0.55	0.055	3	10.0
	5.0-7.0									10.5
3	7.0-9.0	8.7	14.5	48.7	65	1.35	0.045	0.045	4	14.0
	9.0-11.0									17.5
4	11.0-13.0	11.7	16.0	64.7	87	1.35	0.035	0.035	4	23.0
5	13.0-15.0	13.7	16.5	77.2	105	1.35	0.030	0.030	4	30.0

Table 4: Soil Parameters Used in the F.E.M. Analyses

Depth (m)	λ	κ	M	ν	k_v 10^{-4} (m/day)	k_h 10^{-4} (m/day)	e_o
0-2	0.34	0.07	1.2	0.25	25.9	25.9	2.80
2-7	0.90	0.18	0.9	0.30	5.9	10.1	5.90
7-12	0.50	0.10	1.0	0.25	2.6	5.2	4.00
12-15	0.34	0.07	1.2	0.25	1.0	2.1	3.00
15-22	0.10	0.02	1.2	0.20	0.3	0.5	1.30



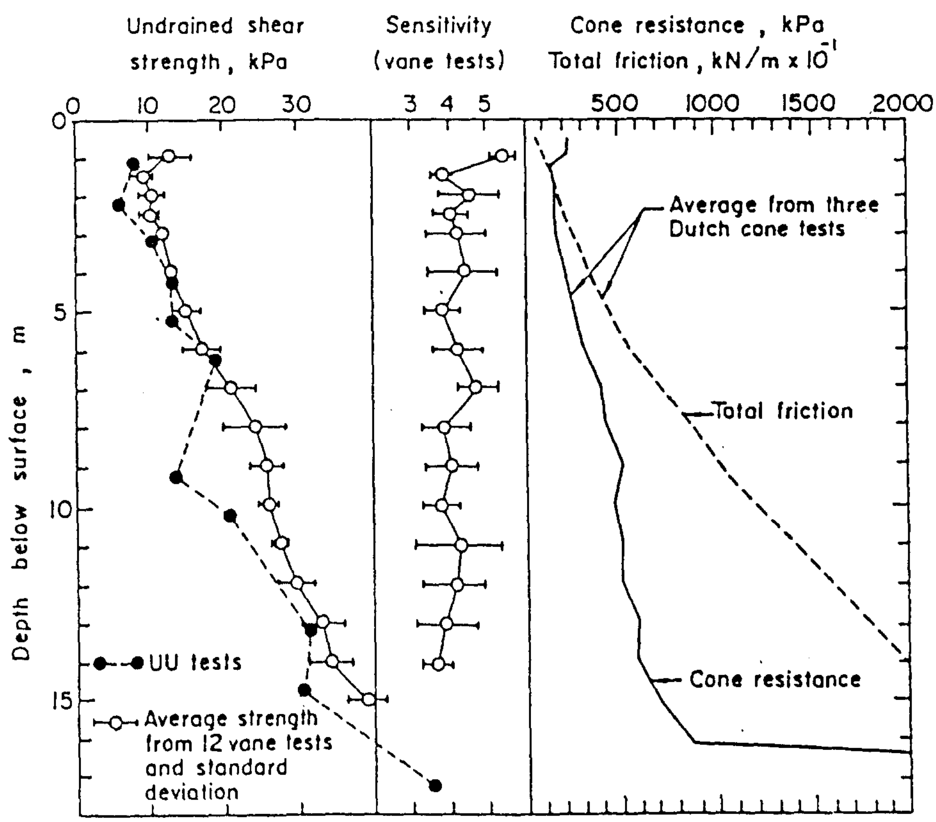


Fig. 1 Typical Soil Profile at the Pom Prachul Dockyard Site

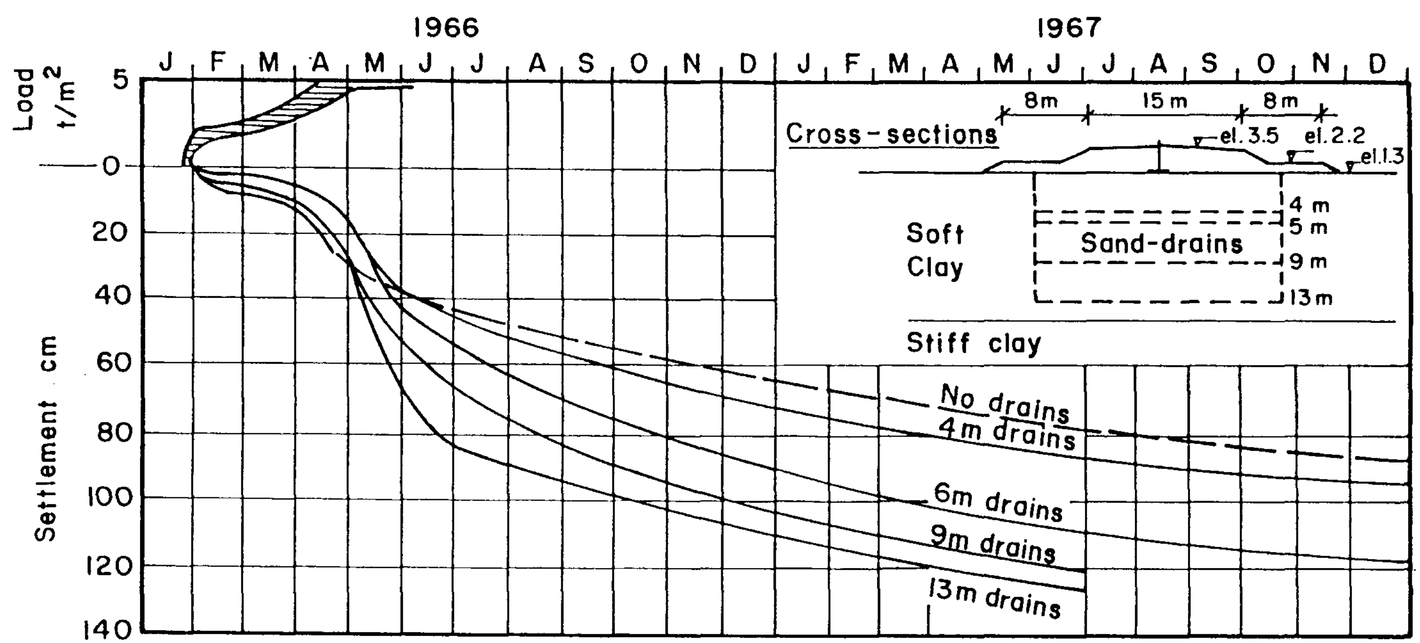


Fig. 2 **Observed Settlement on Test Section with Sand Drains (Eide,1977)**

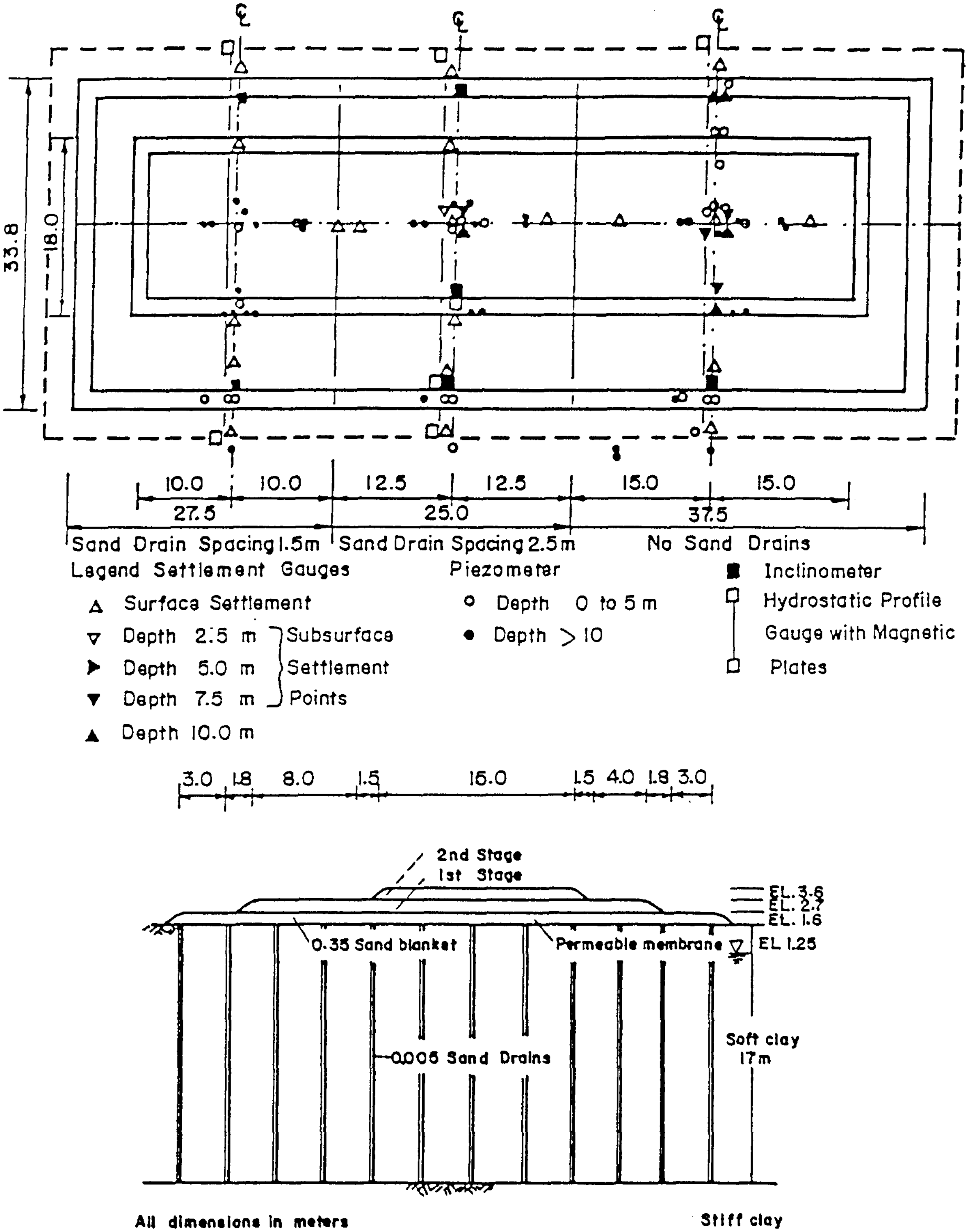


Fig. 3 **Test Embankment with Sandwicks (Balasubramaniam, 1980)**

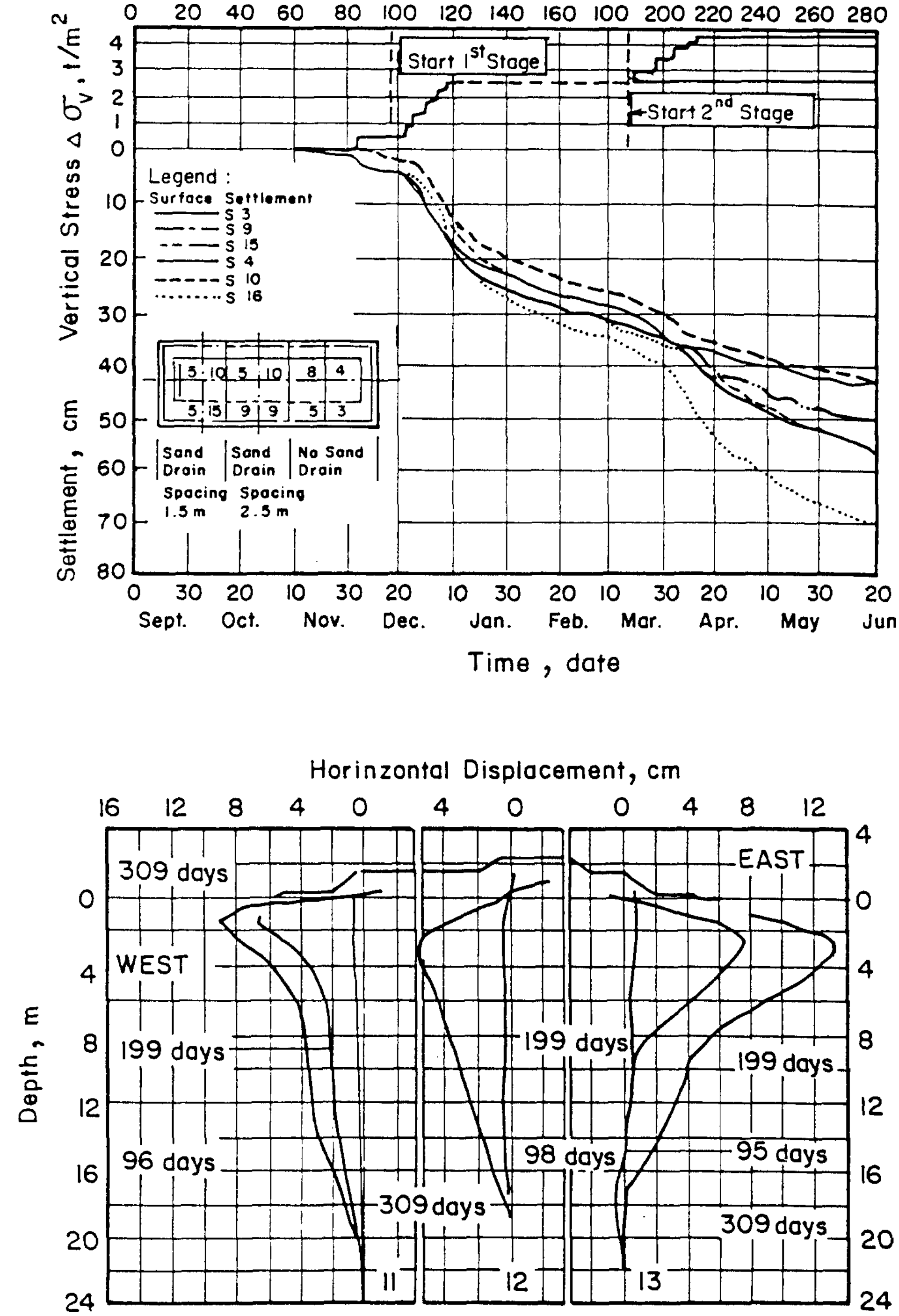


Fig. 4 **Vertical Settlement and Lateral Displacement Plots for Test Embankment with Sandwicks (Balasubramaniam, 1980)**

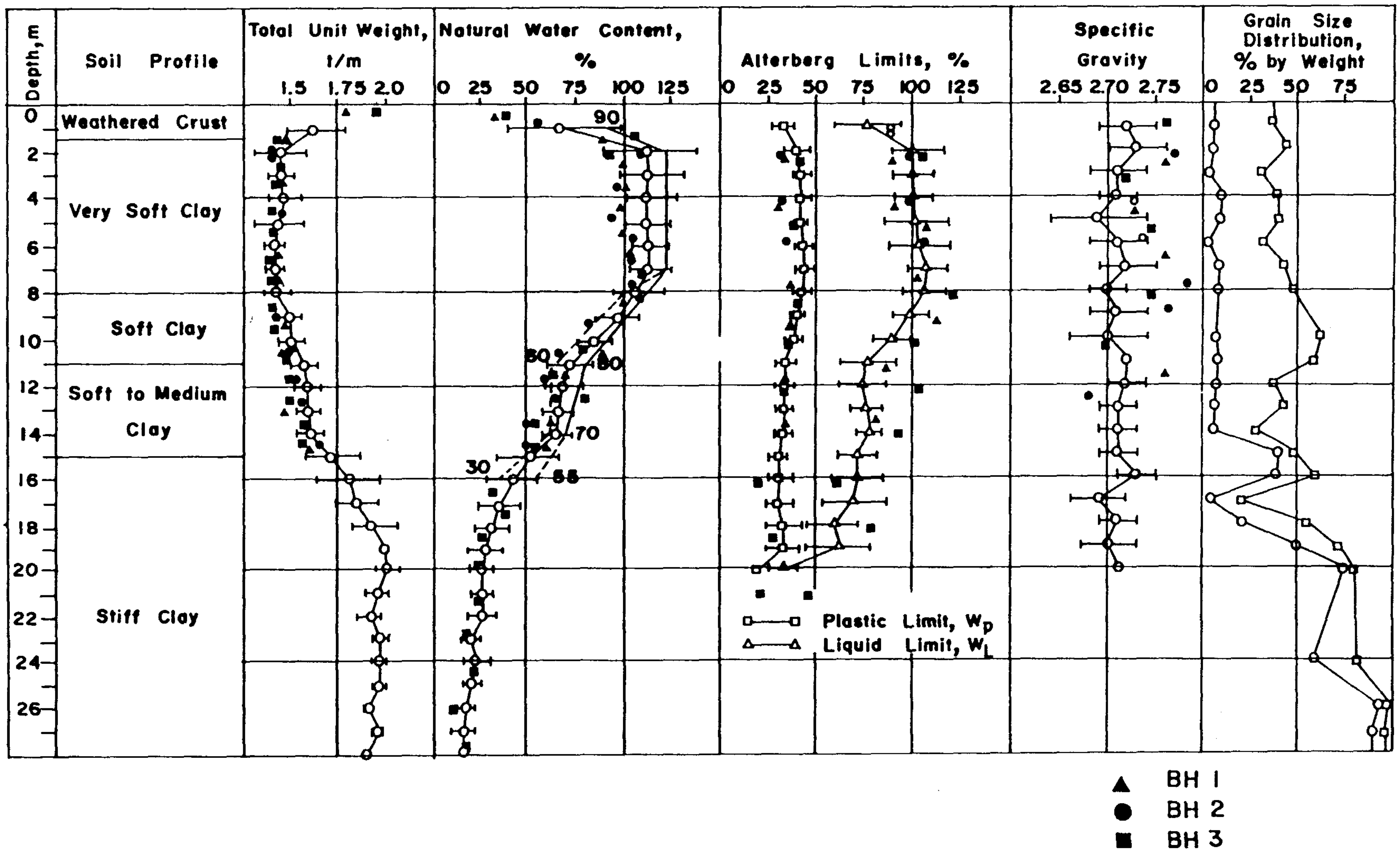


Fig. 5(a) Generalized Soil Profile and Properties for Nong Ngu Hao Test Embankment

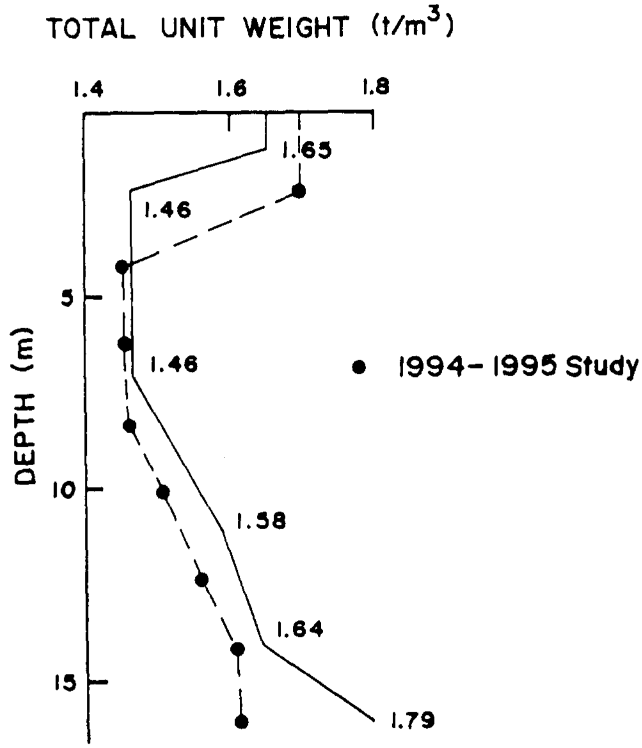


Fig. 5(b) Variation of Total Unit Weight with Depth

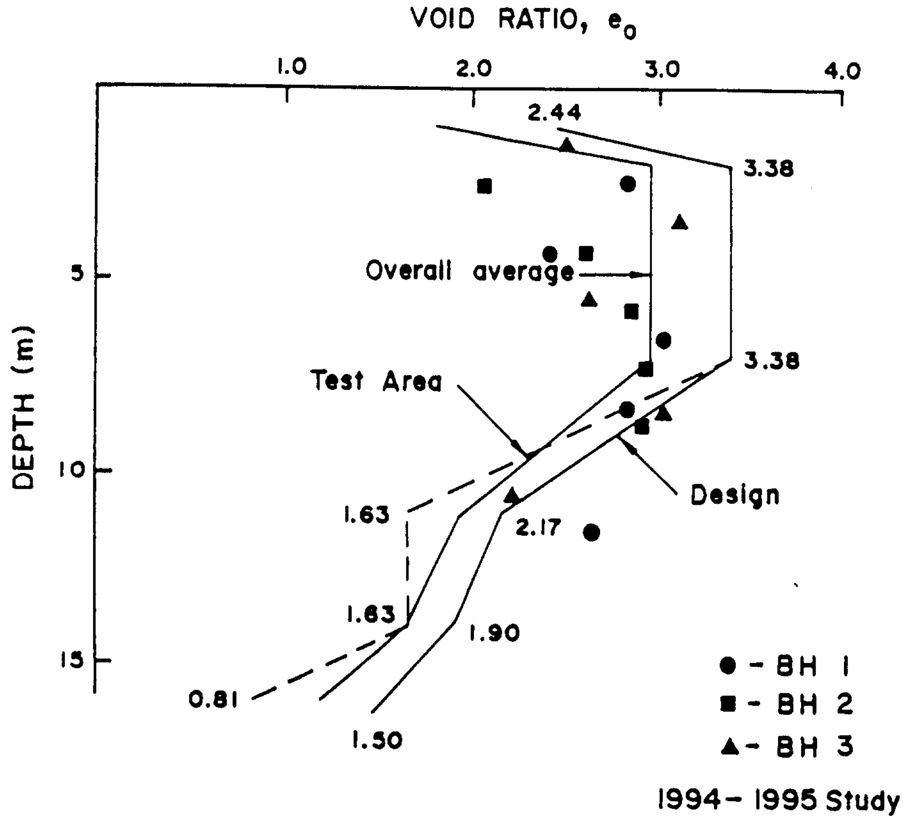


Fig. 5(c) Variation of Void Ratio with Depth

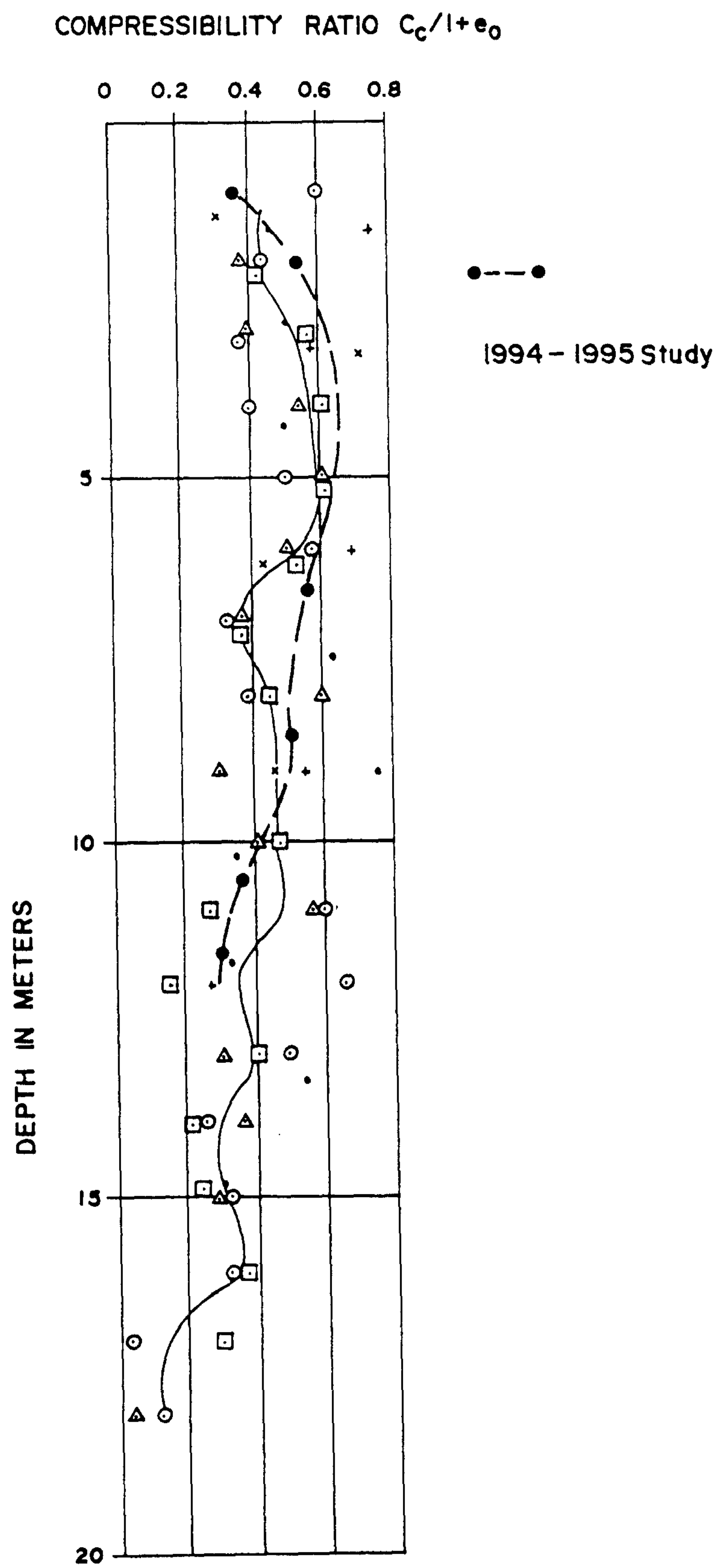
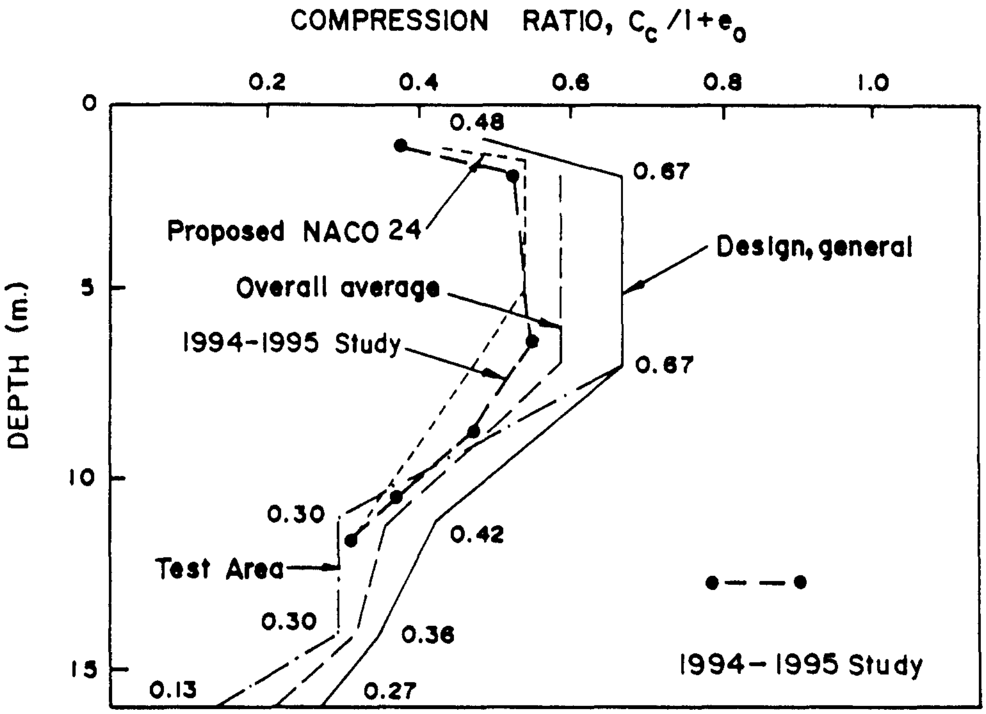
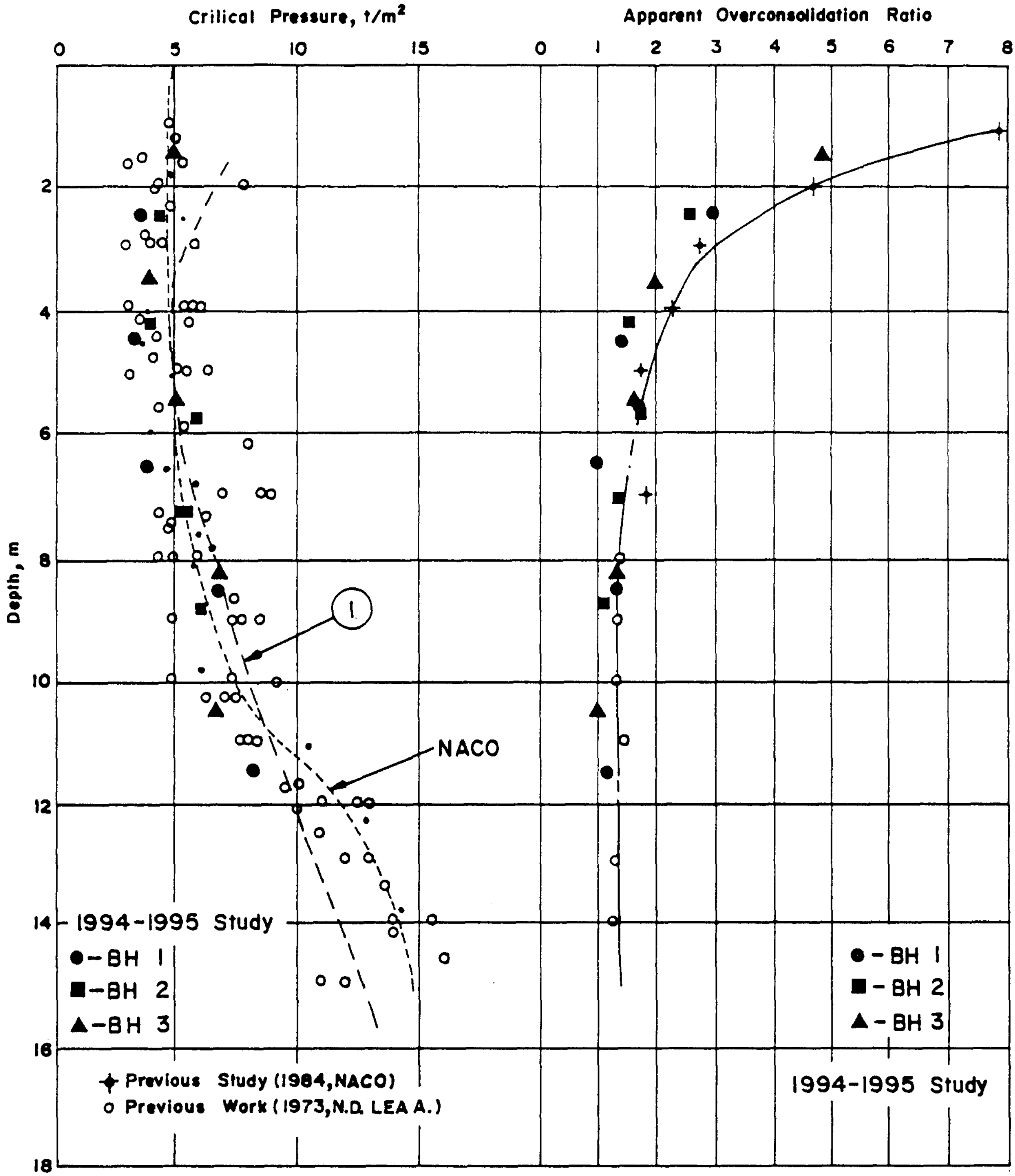


Fig 5(d) Variation of Compressibility Ratio with Depth



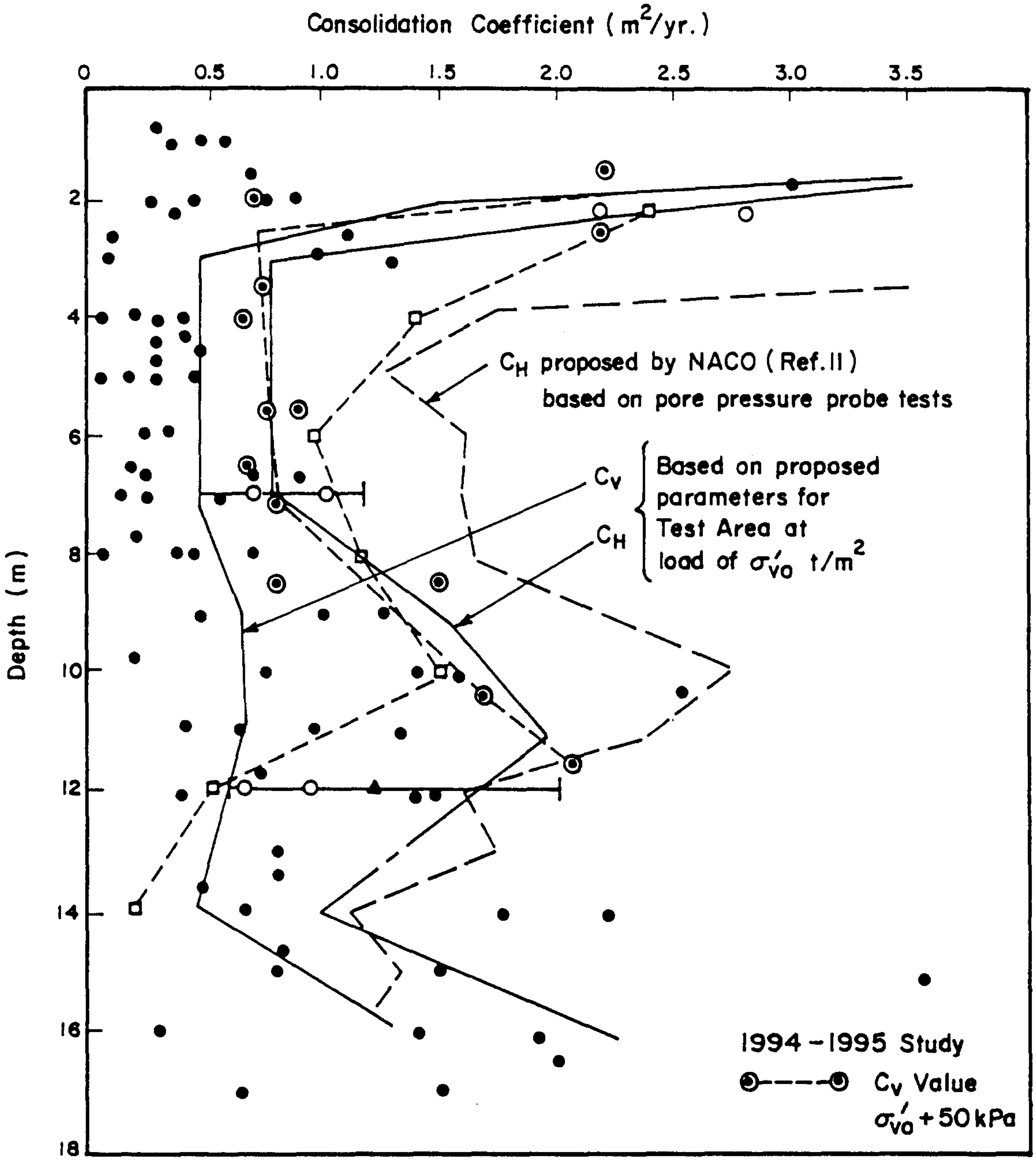
NOTE:
Design values are from proposed Correlation, $\frac{C_c}{1+e_0} = \left(\frac{w\%}{100}\right) \times 0.58$

Fig. 5(e) Variation of Compression Ratio with Depth



① This is based on correlation between OCR, (s_u/σ'_{vo}) vane and I_p according to Bjerrum (1973) and Aas et al. (1985)

Fig. 5(f) Variation of Critical Pressure and Apparent Overconsolidation Ratio with Depth



C_H	▲	Back-calculated from pore pressure dissipation , TFI
	○	Back-calculated from pore pressure dissipation , TFII
	□	Back-calculated from settlement end stage 3 , TFII
C_v	●	Directly from oedometer tests at stress level of $\sigma' \sim \sigma'_{vm} + 5\ t/m^2$

Fig. 5(g) Variation of Coefficient of Consolidation-c_v values

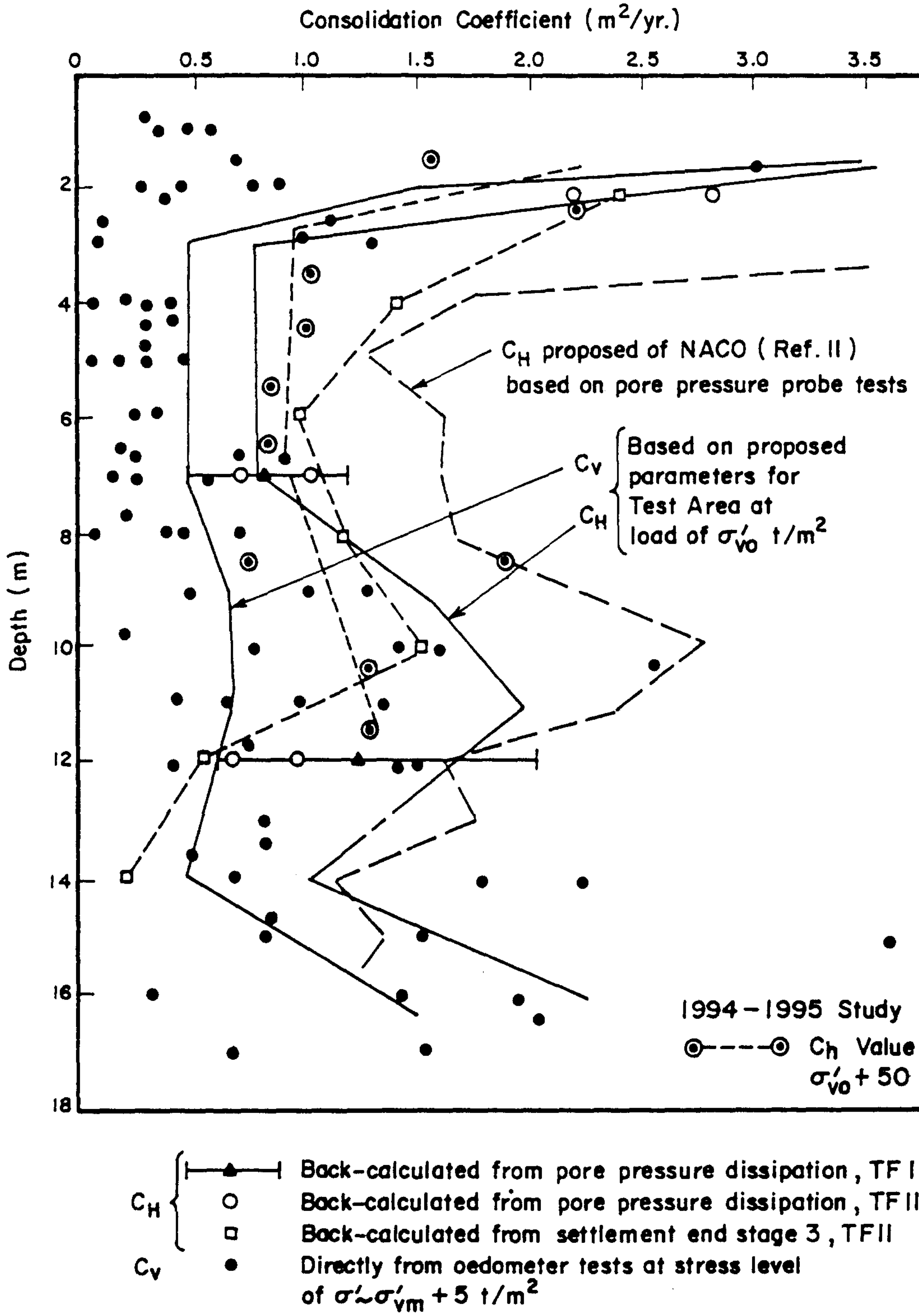


Fig. 5(h) Variation of Coefficient of Consolidation - c_h values

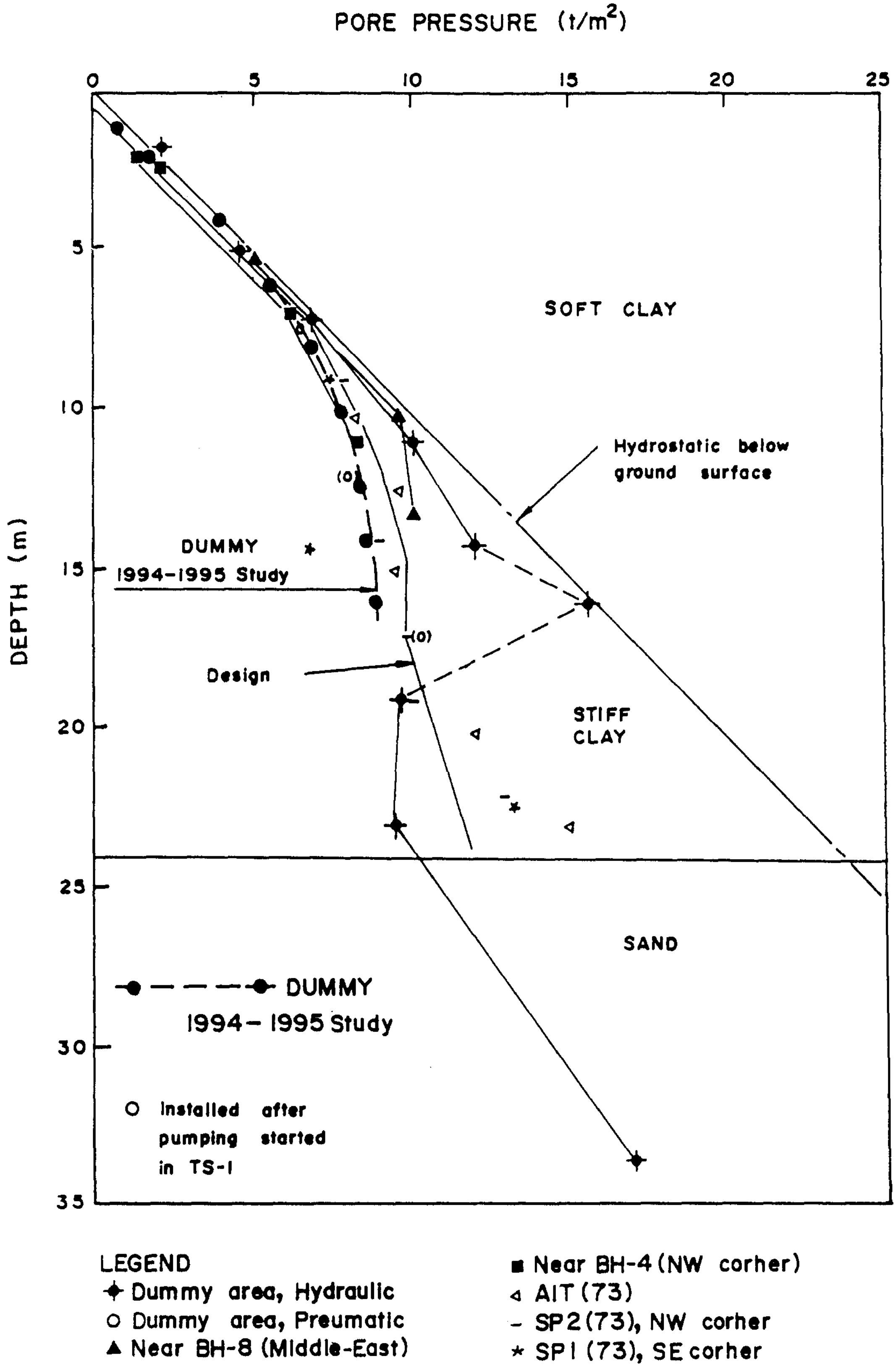


Fig 5(i) Variation of Piezometric Pressures with Depth

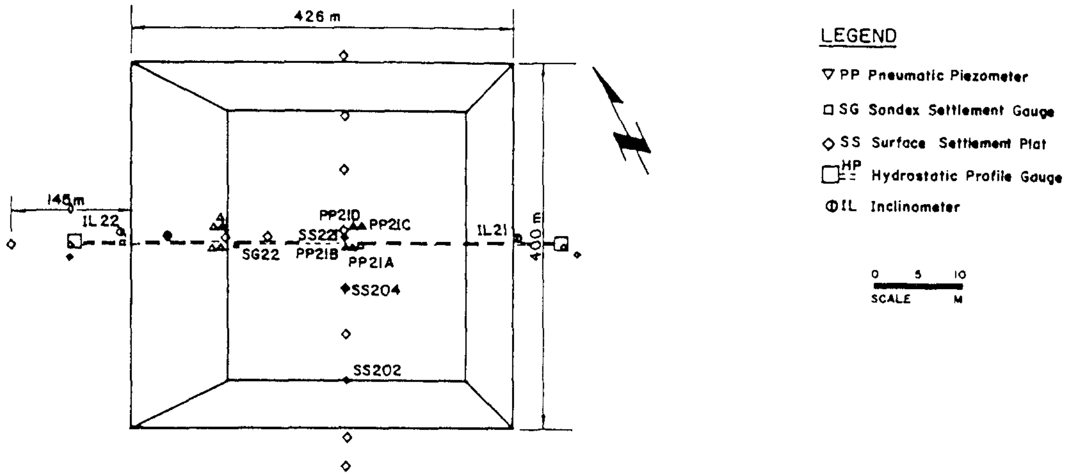


Fig. 6 Instrumentation Plan of Nong Ngu Hao Test Embankment with Sand Drains(Moh & Woo,1983)

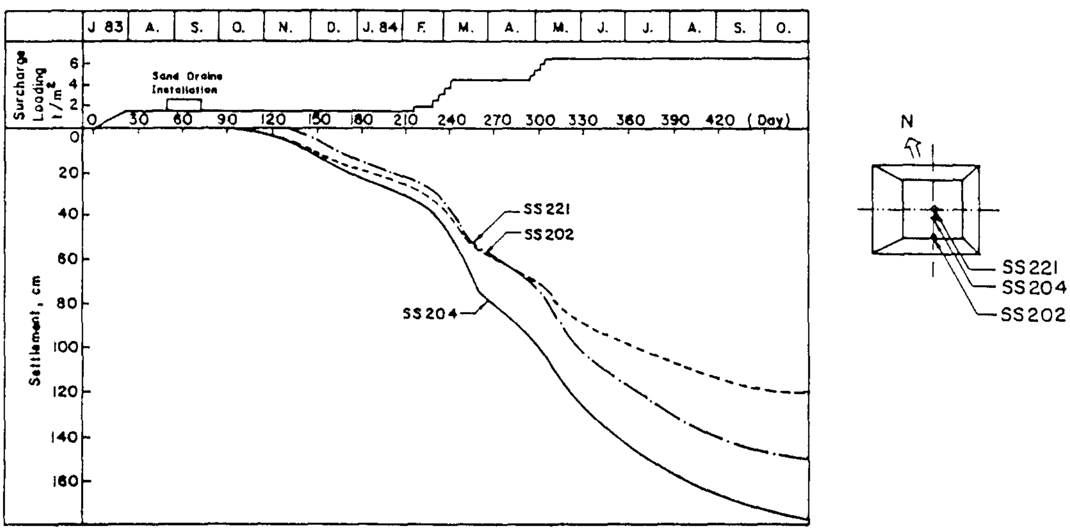


Fig. 7 Observed Settlement for Nong Ngu Hao Test Embankment with Sand Drains (Moh & Woo,1983)

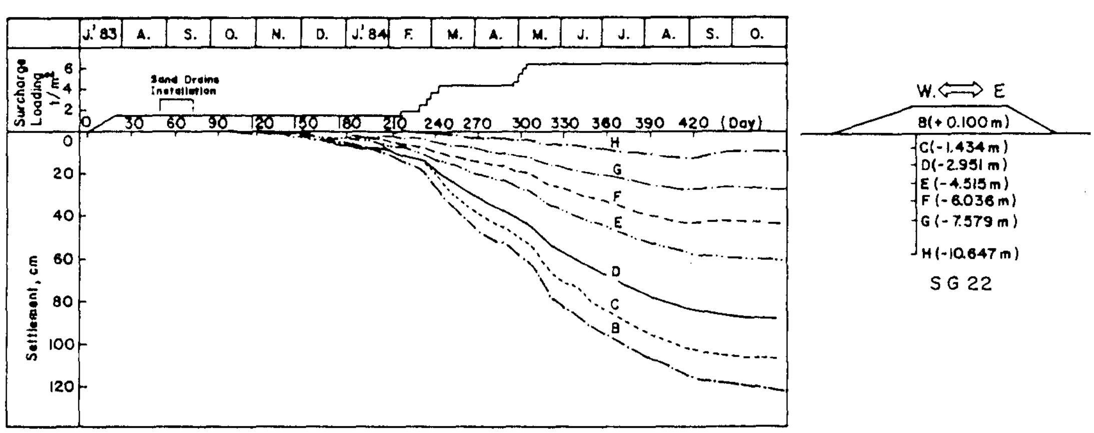


Fig. 8 Vertical Settlements of Soil Layers for Nong Ngu Hao Test Embankment with Sand Drains (Moh & Woo, 1983)

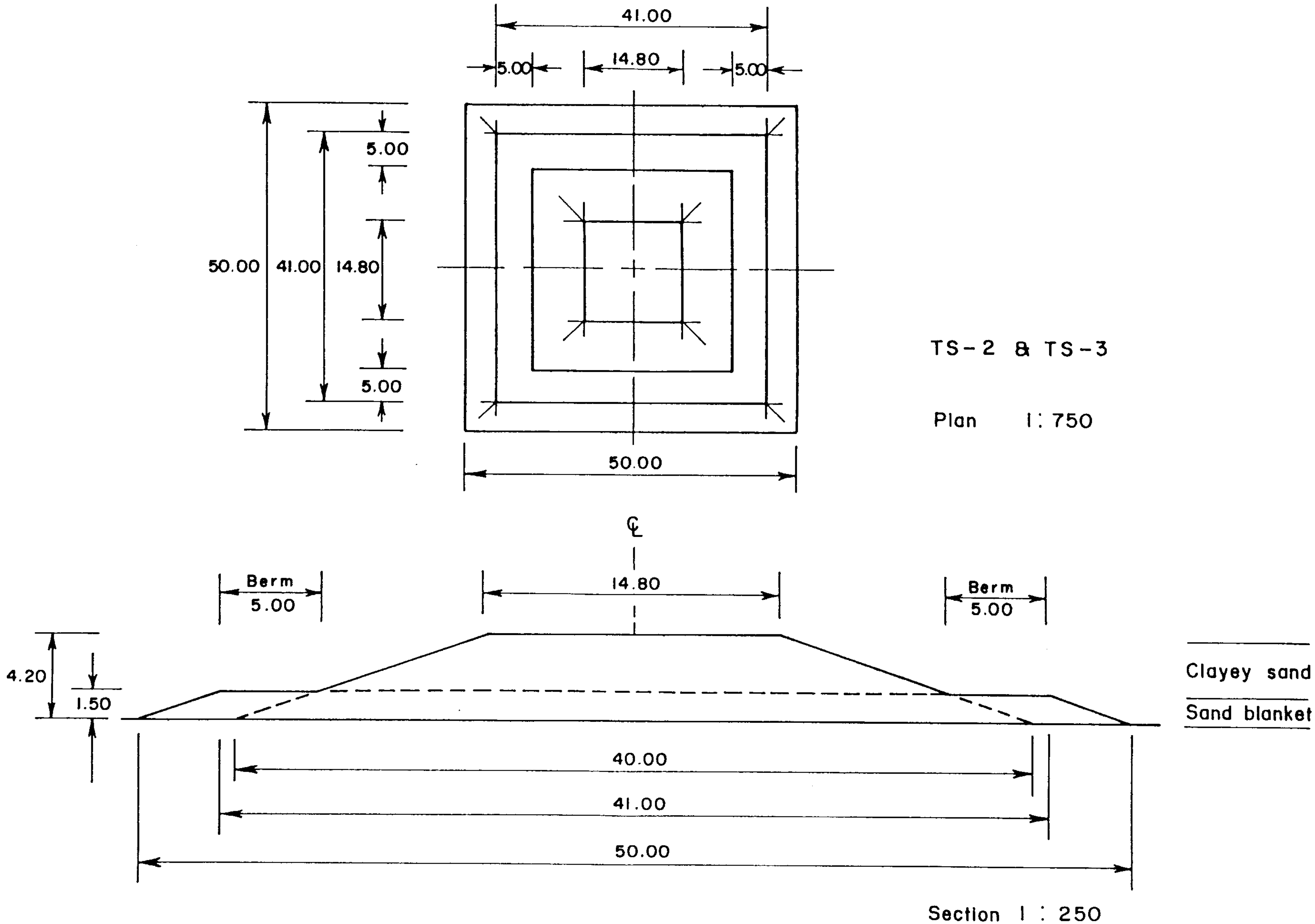
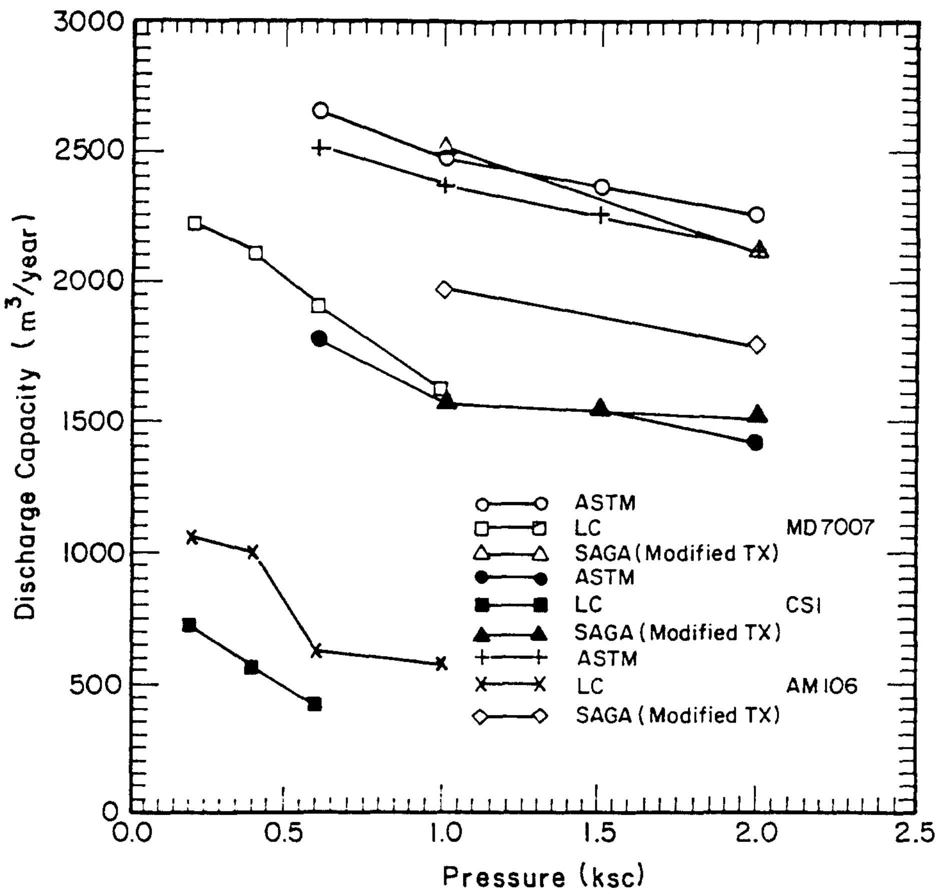


Fig. 9 Plan and Section of the Test Embankment with PVD



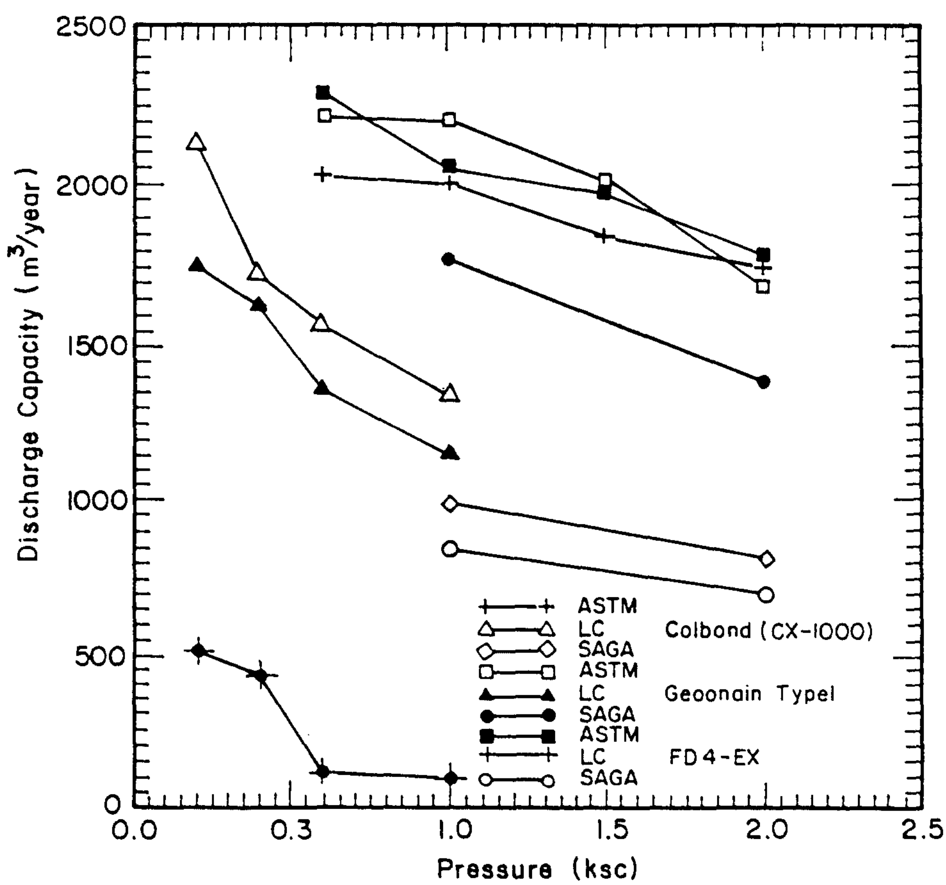


Fig 10 Variation of Discharge Capacity with Lateral Pressures

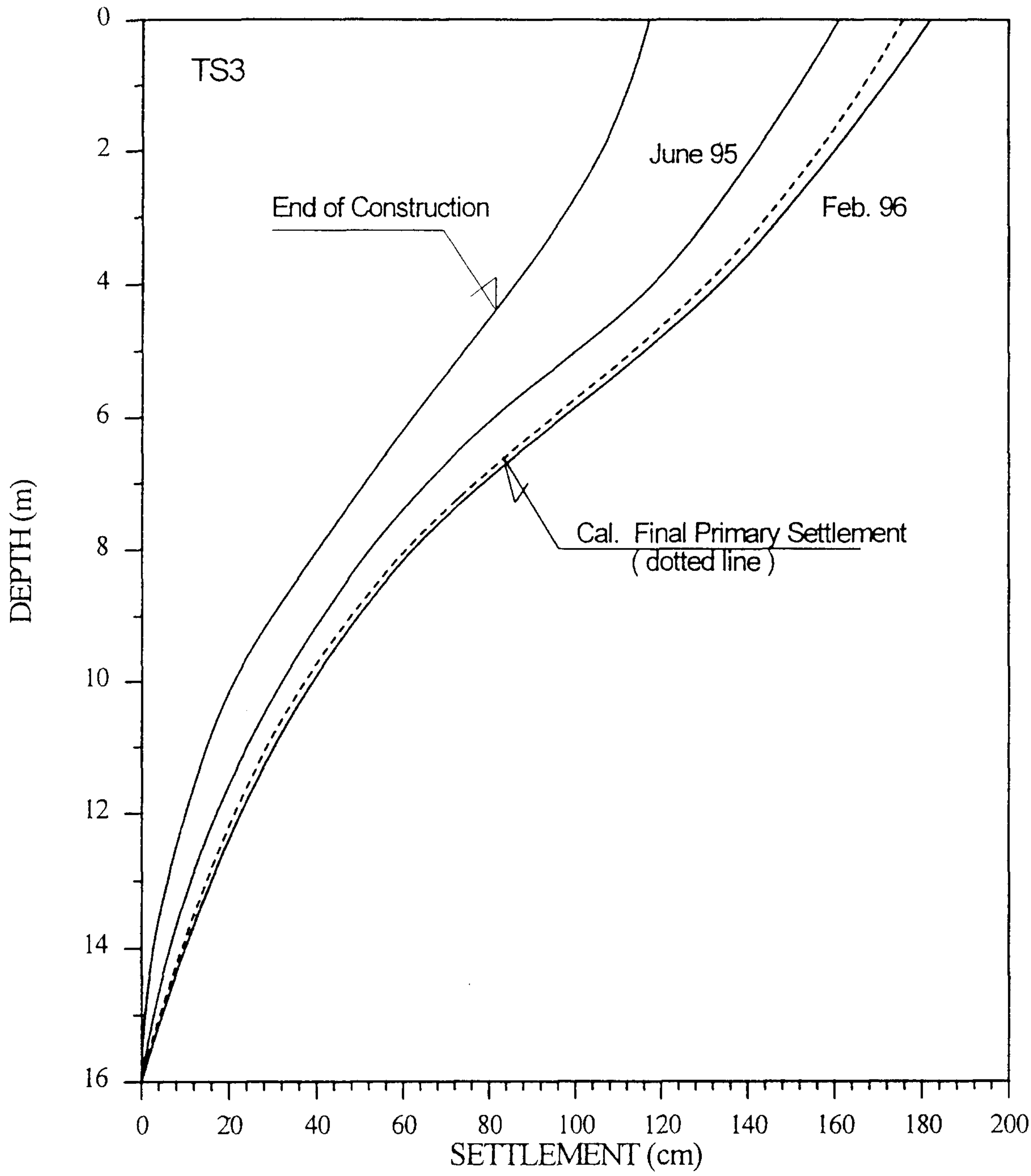


Fig. 11 Settlement Profile - TS3

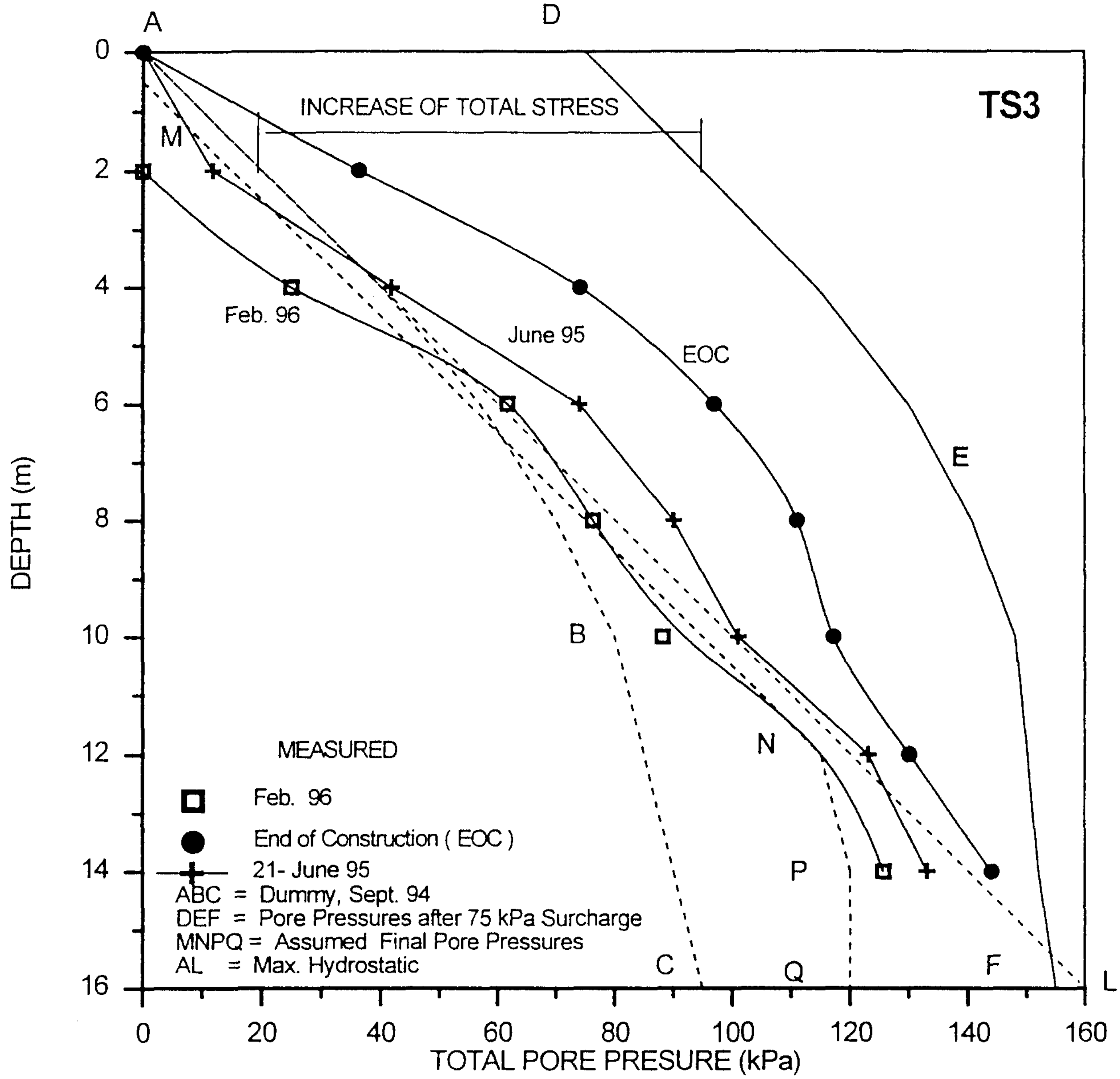


Fig. 12 Pore Pressure Profile -TS3

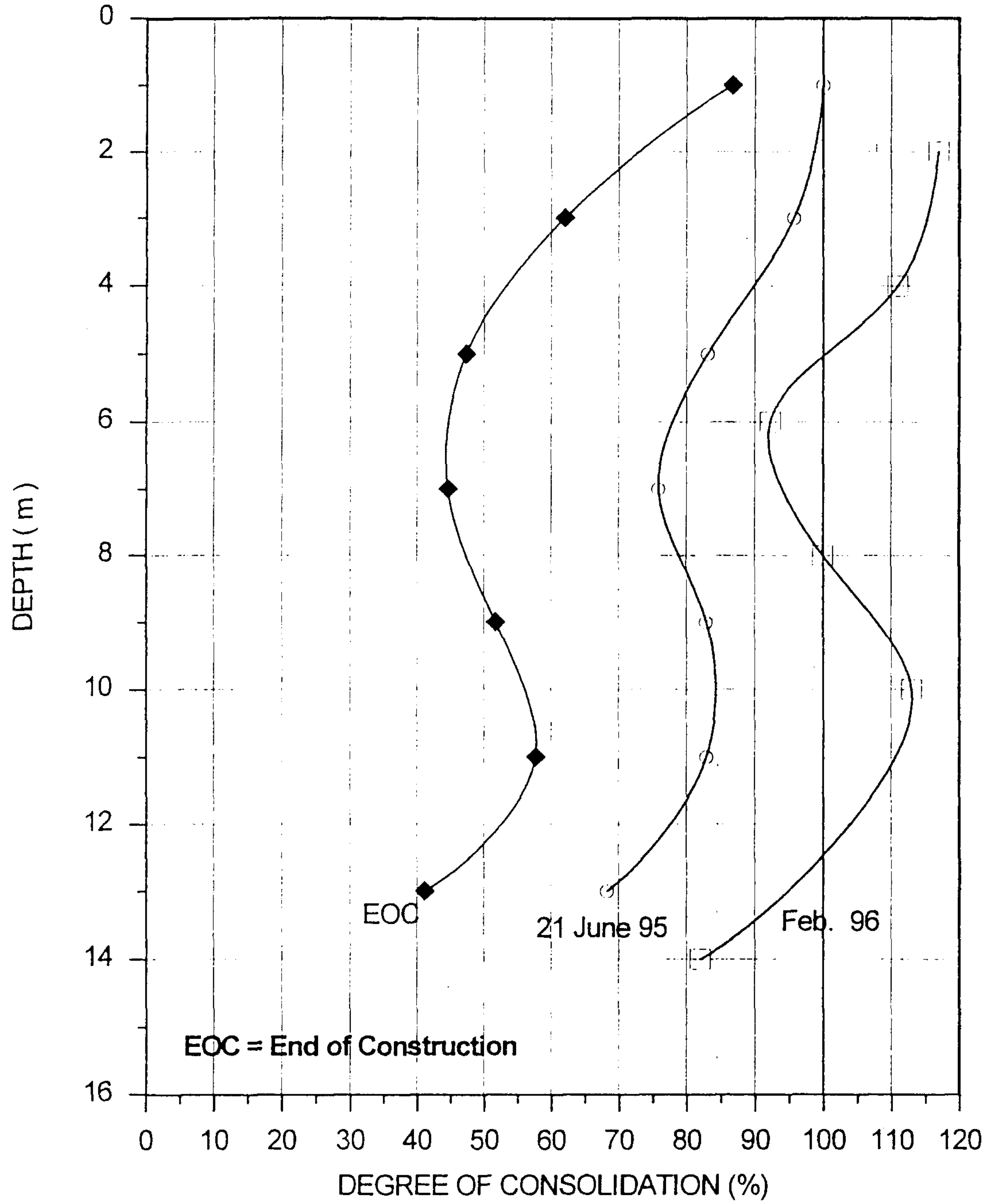


Fig. 13 Degree of Consolidation from Measured Pore Pressure - TS3

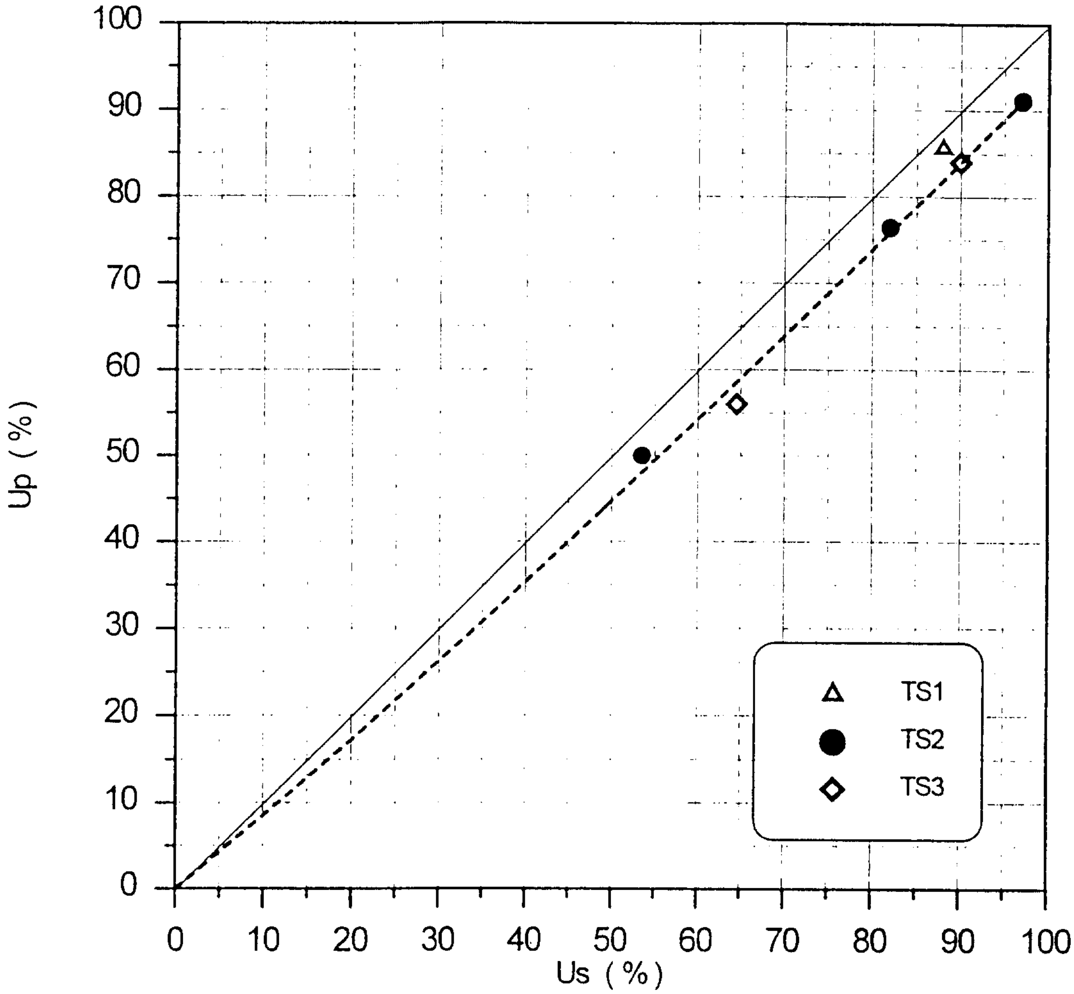


Fig. 14 Relation of Degree of Consolidation from Settlement (Us and Up)

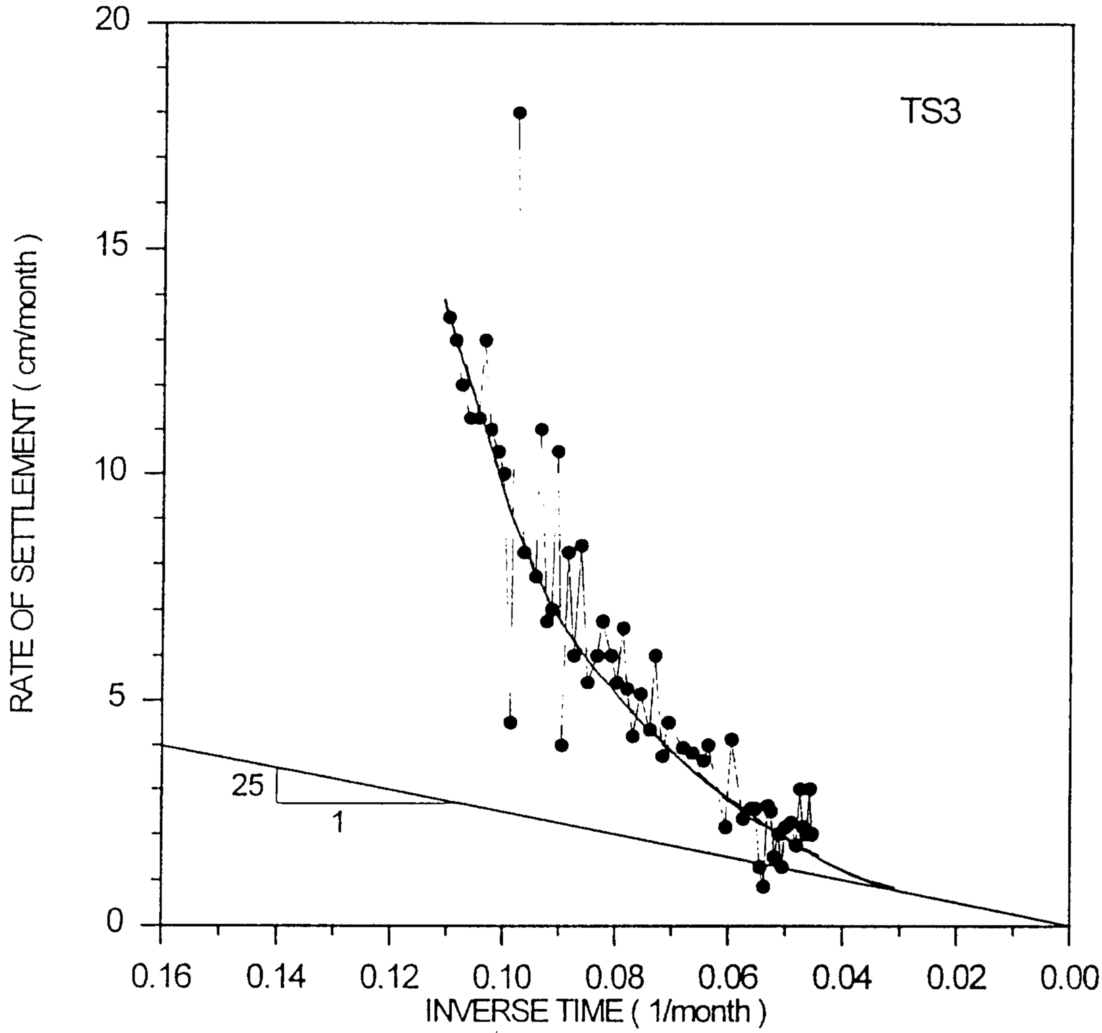


Fig. 15 **Rate of Settlement versus Inverse Time Plot - TS3**

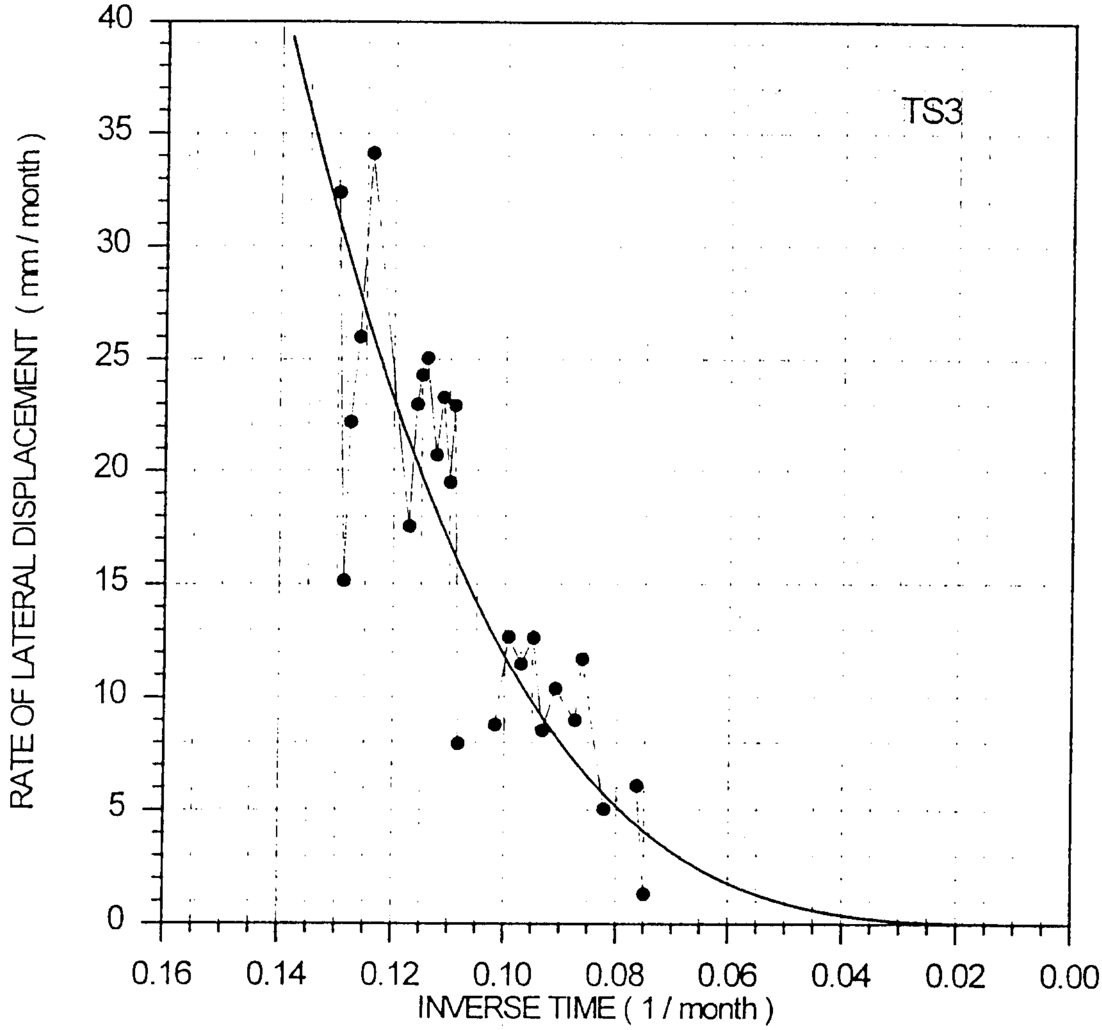


Fig. 16 Rate of Lateral Displacement versus Inverse Time Plot - TS3

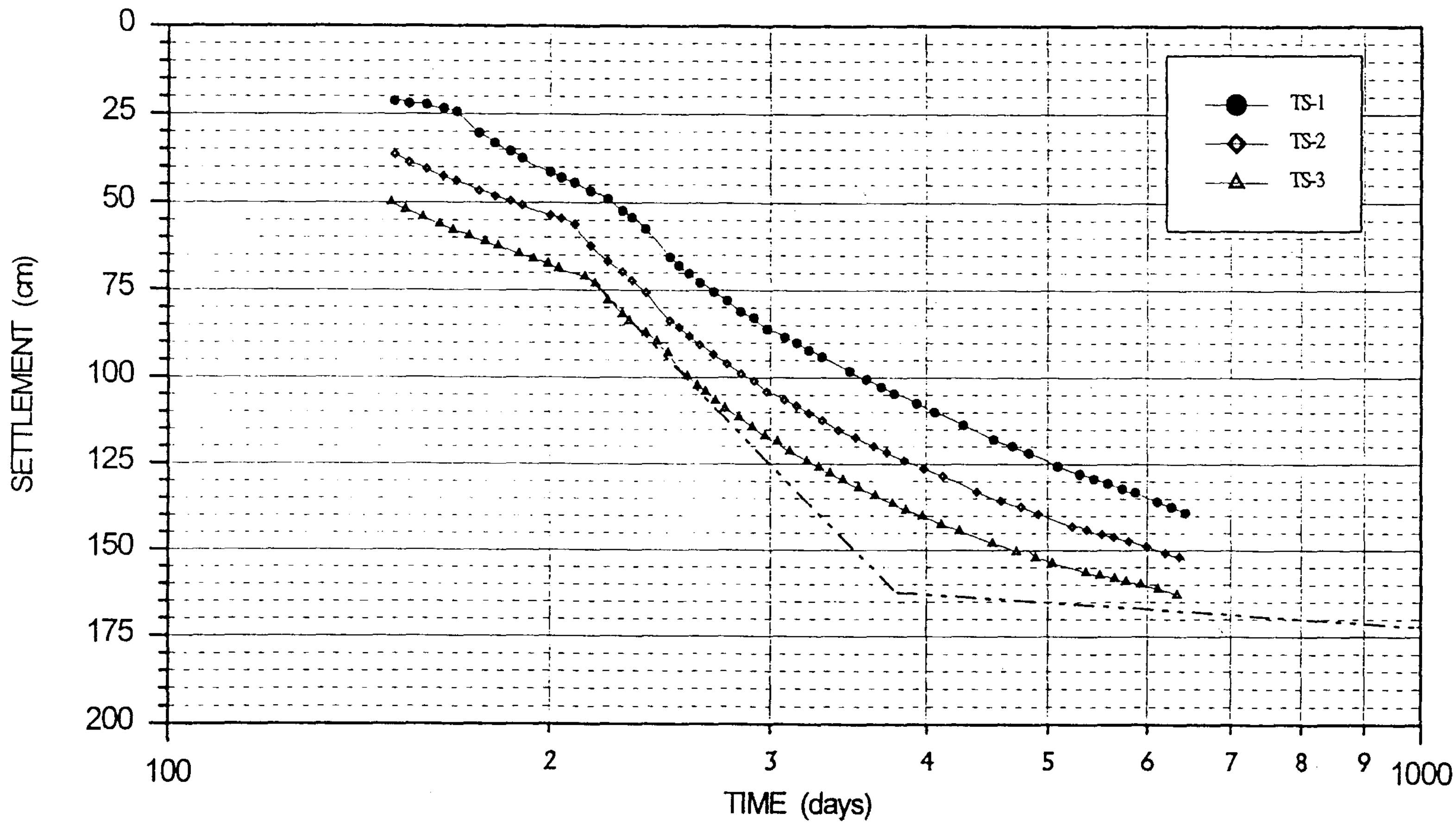


Fig. 17 Consolidation Settlement versus Log Time Plots

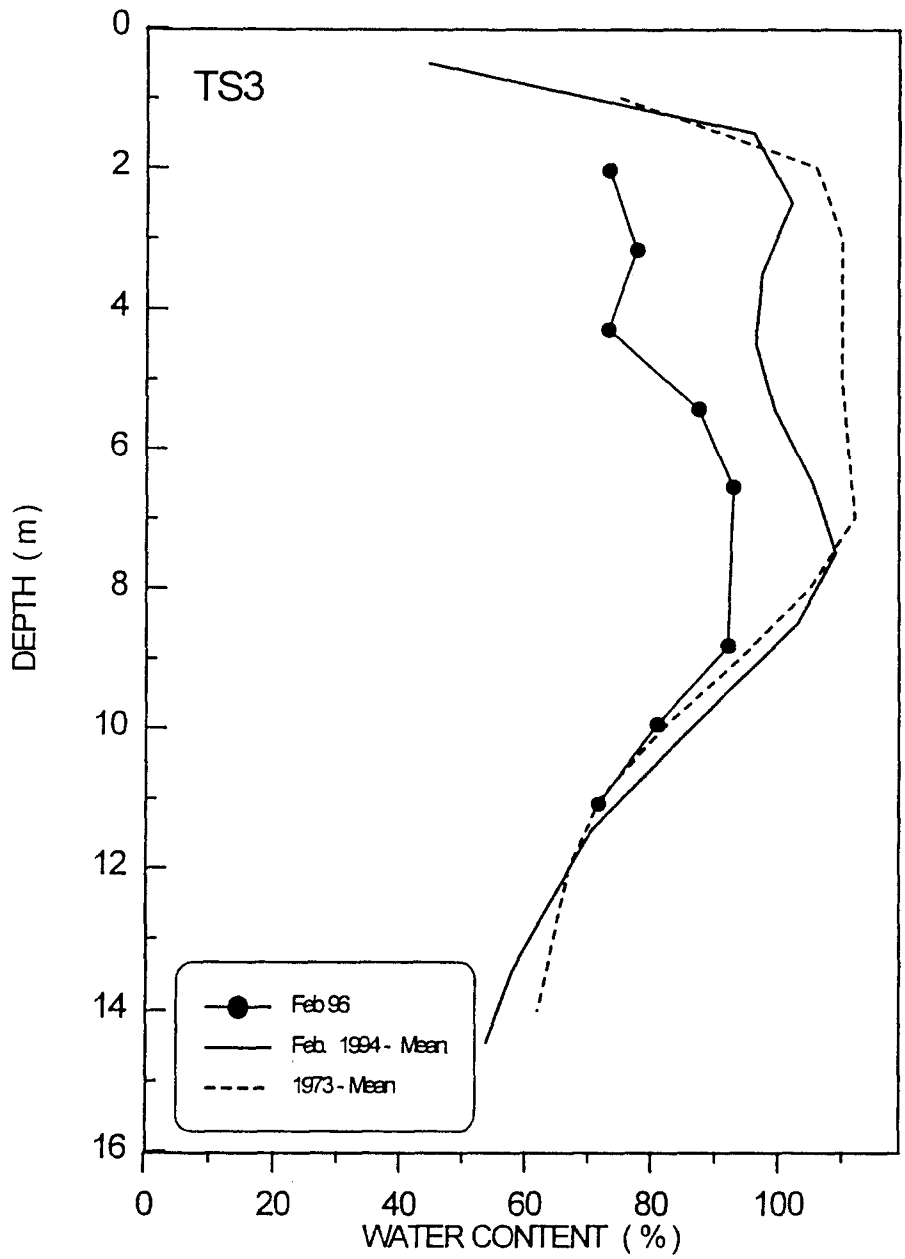


Fig. 18 Water Contents Before and After Preloading with PVD - TS3

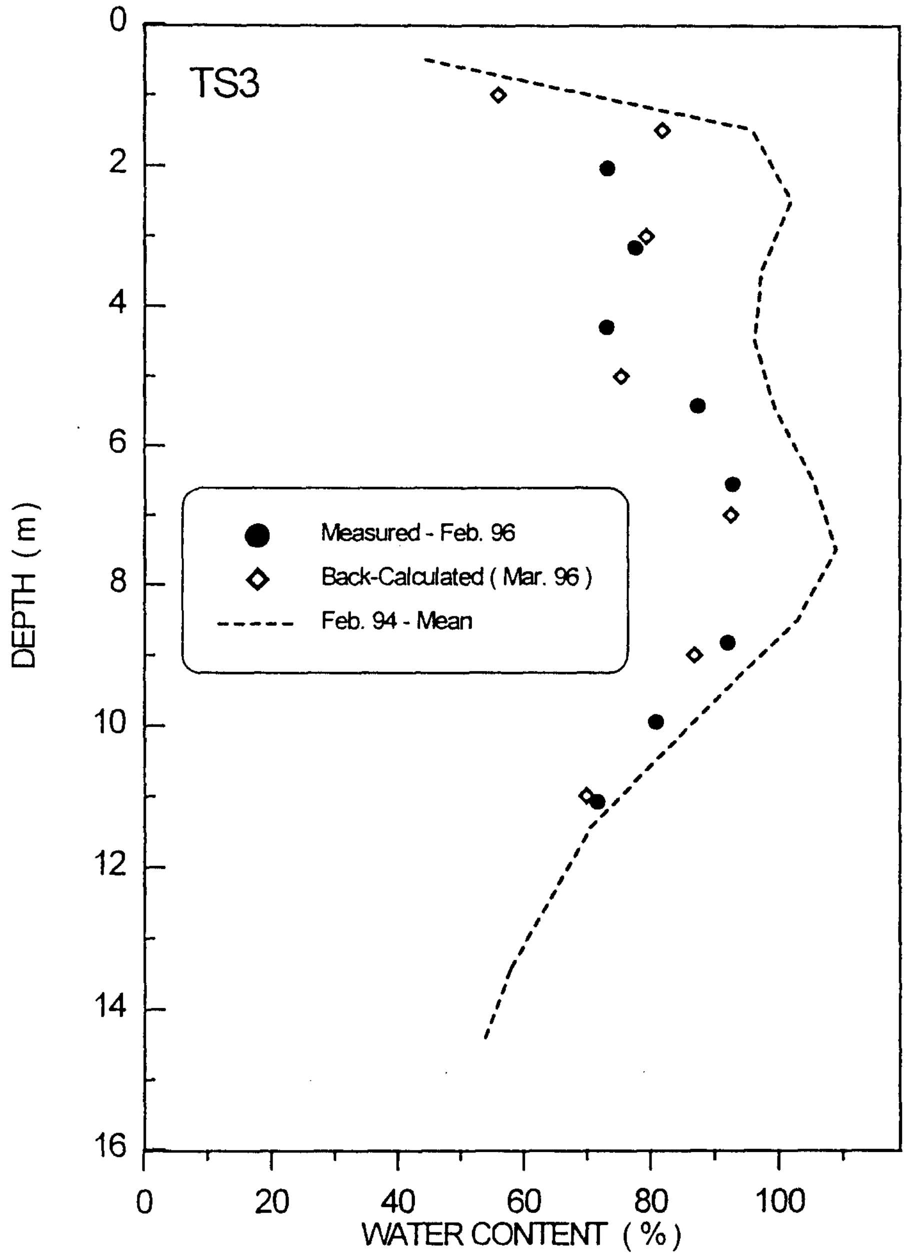


Fig. 19 Back- Calculated Water Contents from Settlements - TS3

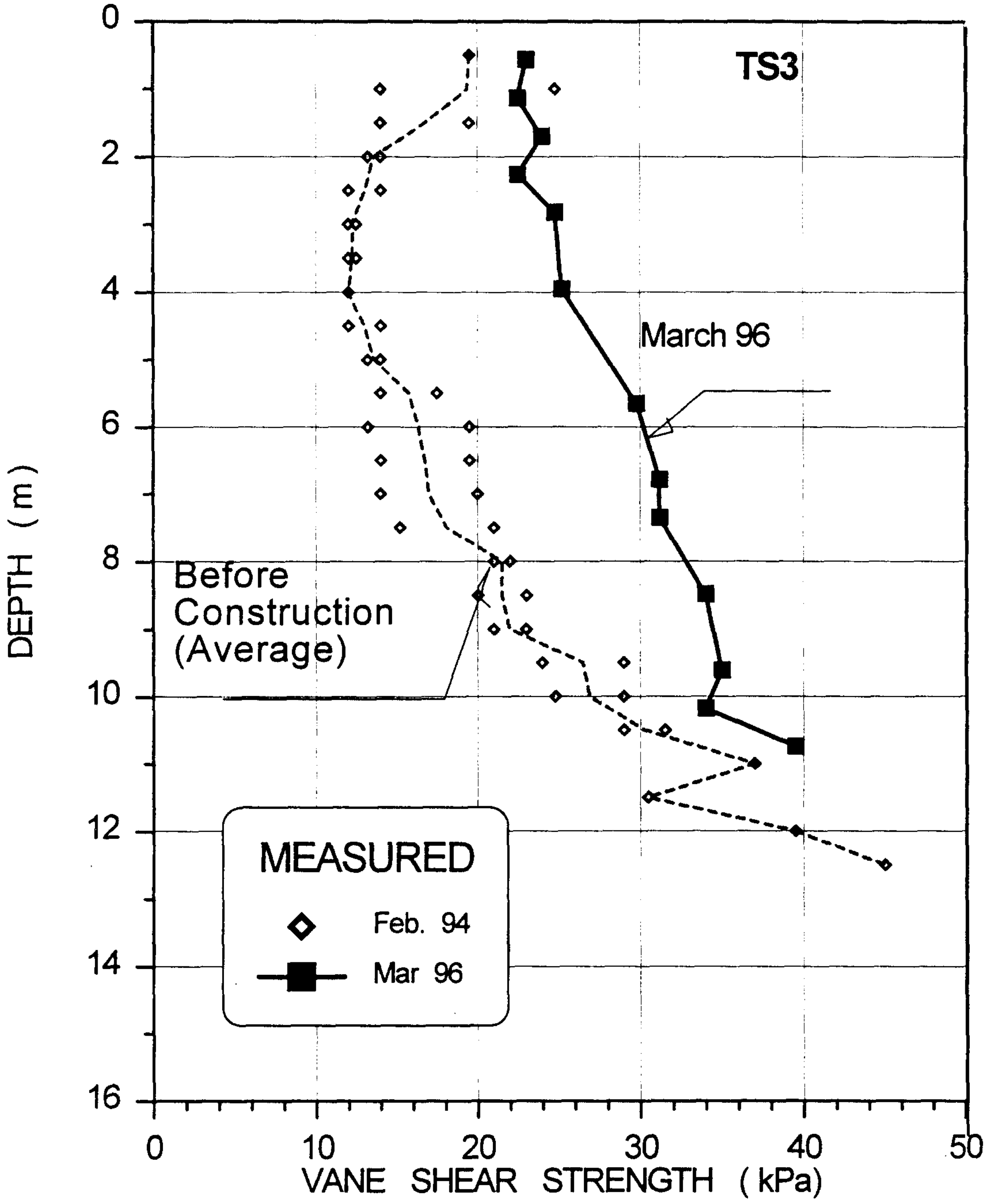


Fig. 20 Field Vane Shear Strength Measured in Embankment-TS3

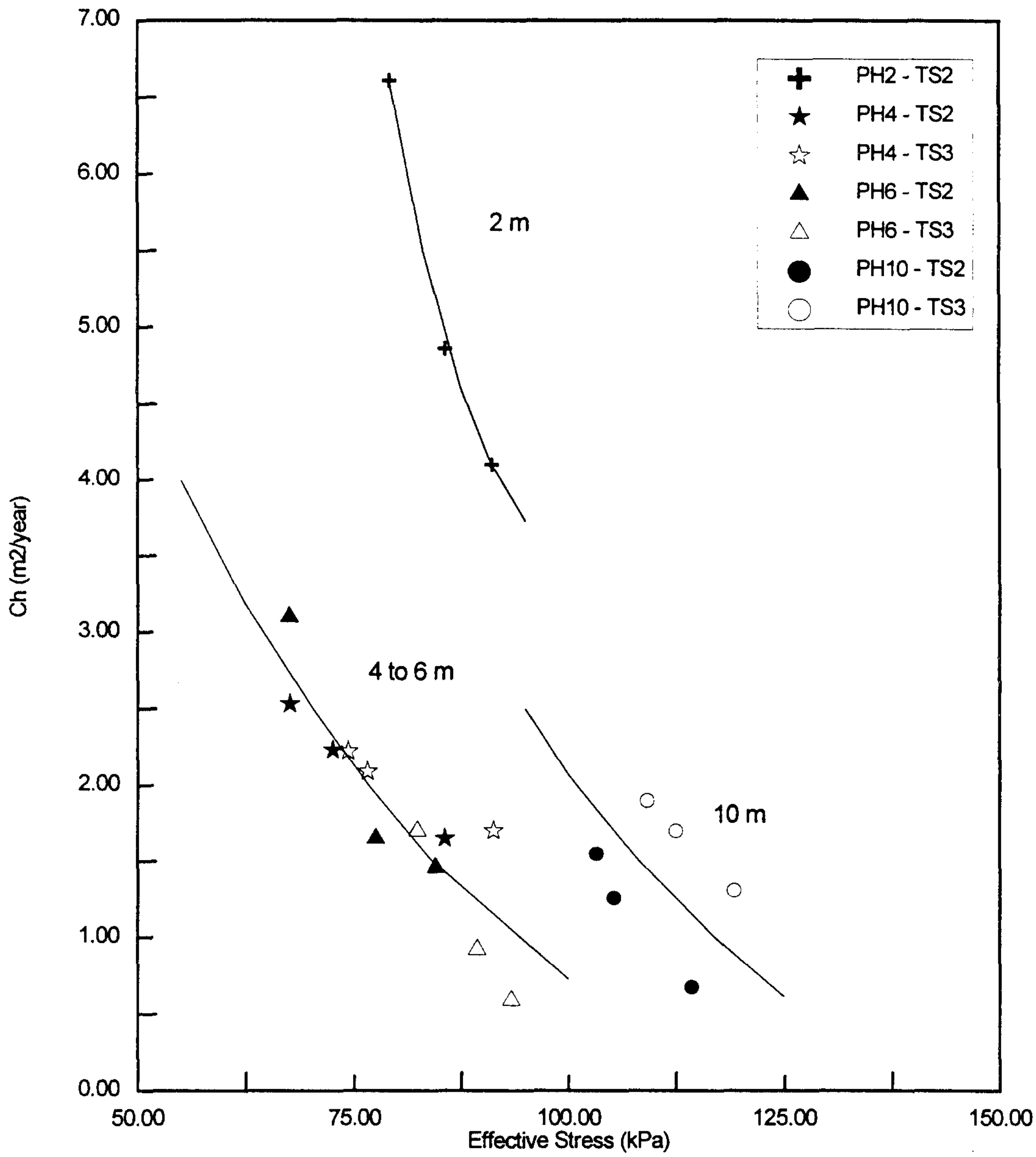


Fig. 21 Plot of Back-Calculated c_h Values from Pore Pressure versus Effective Stress

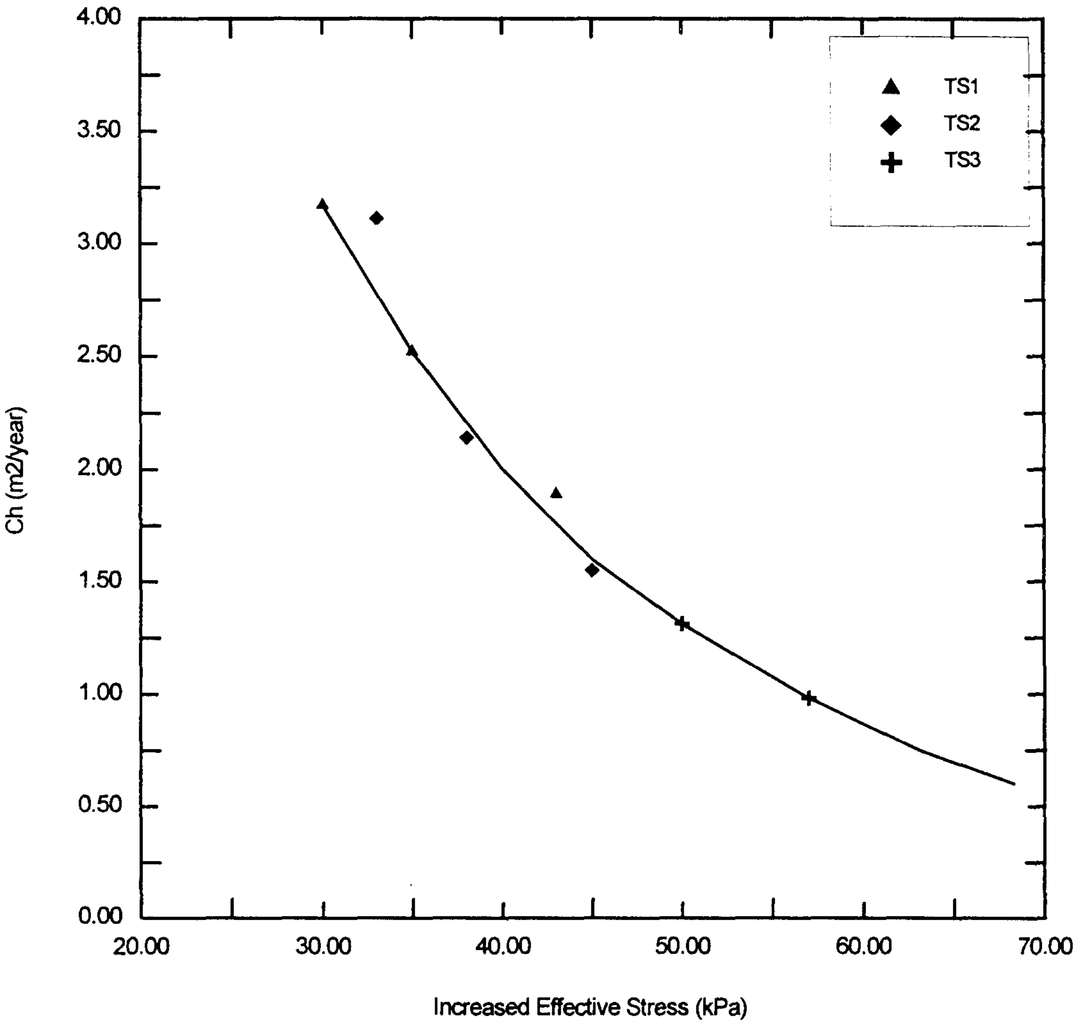


Fig. 22 Back- Calculated Average c_h Values from Measured Settlement versus Increased Effective Stress

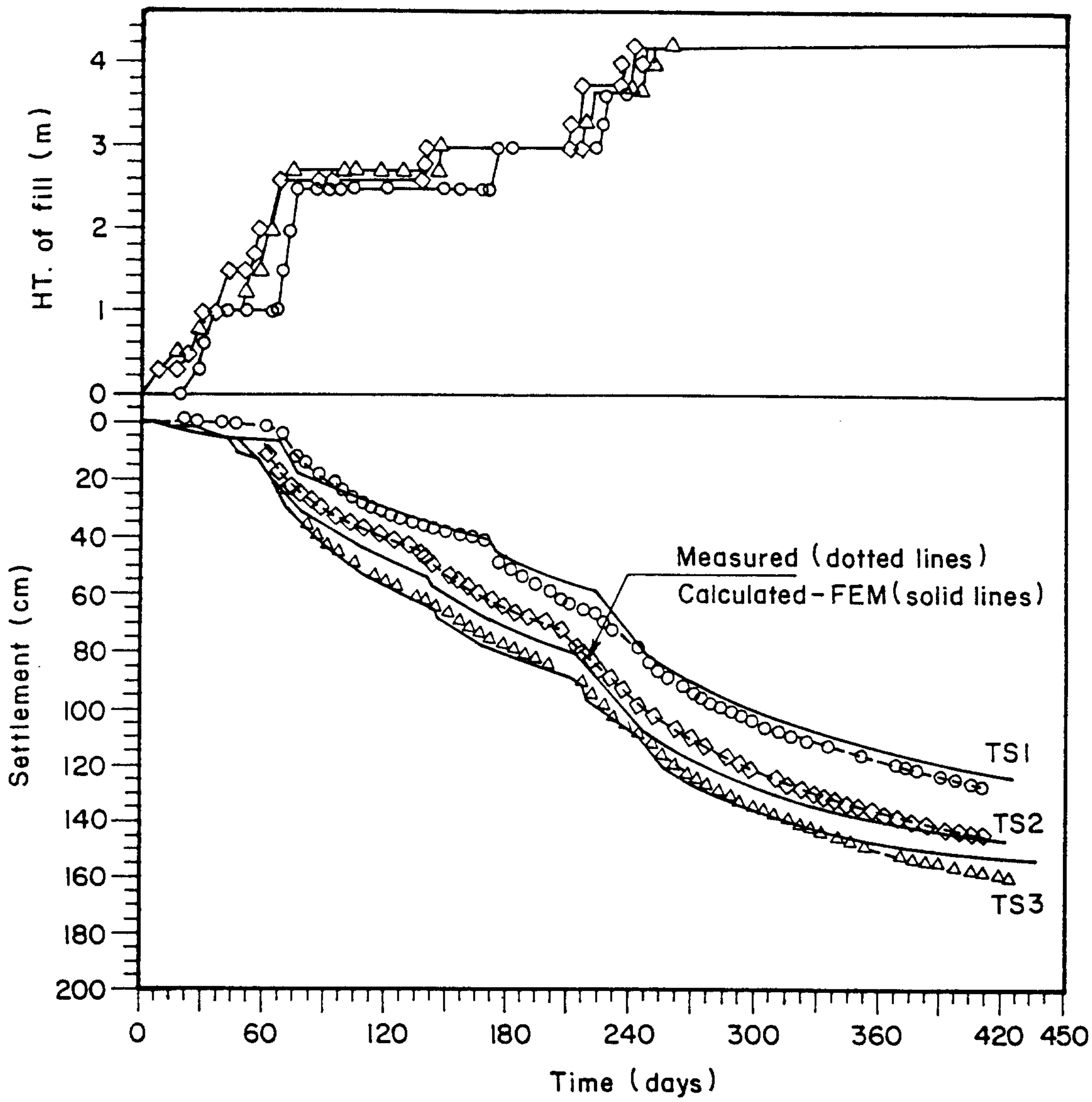


Fig. 23 Comparison of Computed (FEM) and Measured Settlement with Time of EmbankmentsTS1, TS2 and TS3 with PVD

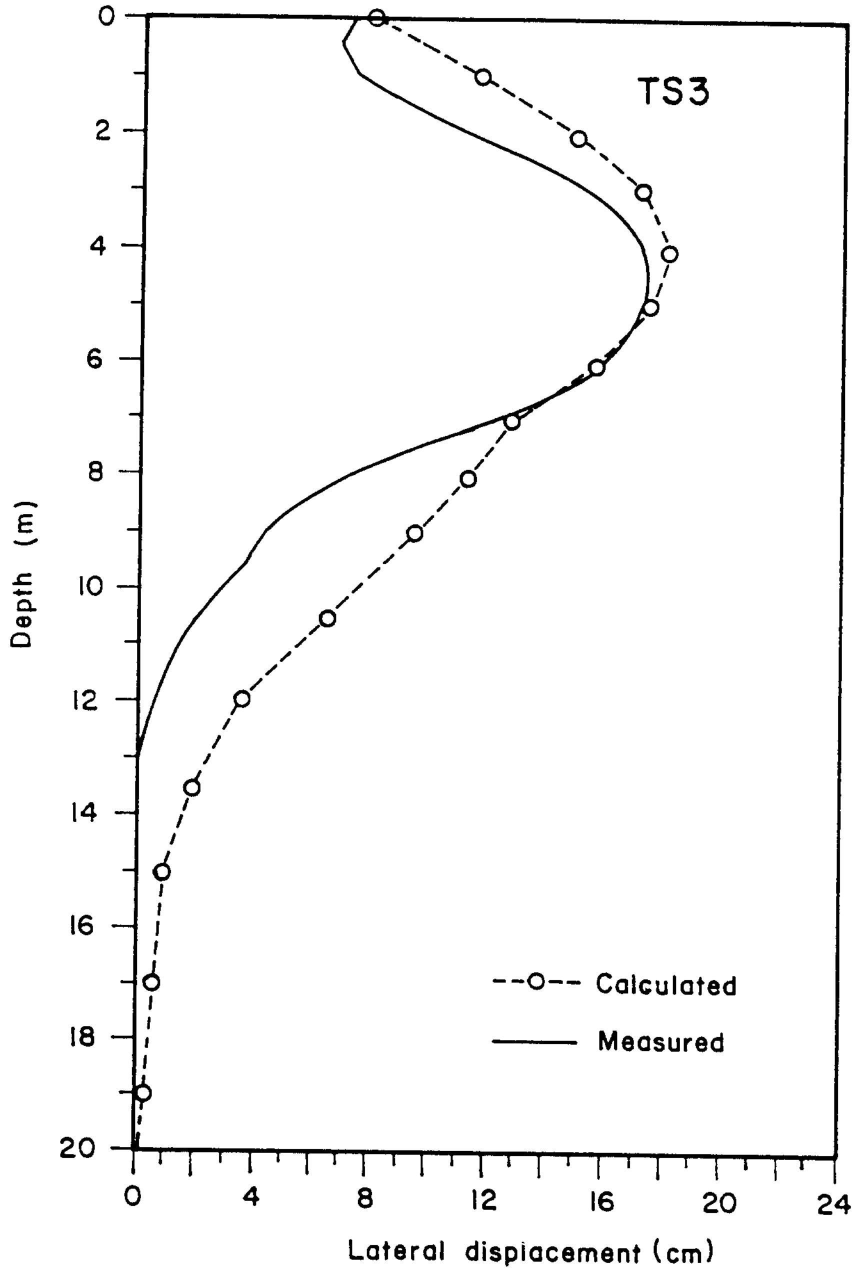


Fig. 24 **Comparison of Computed (FEM) and Measured Lateral Deformations at the End of Construction for Embankment-TS3**

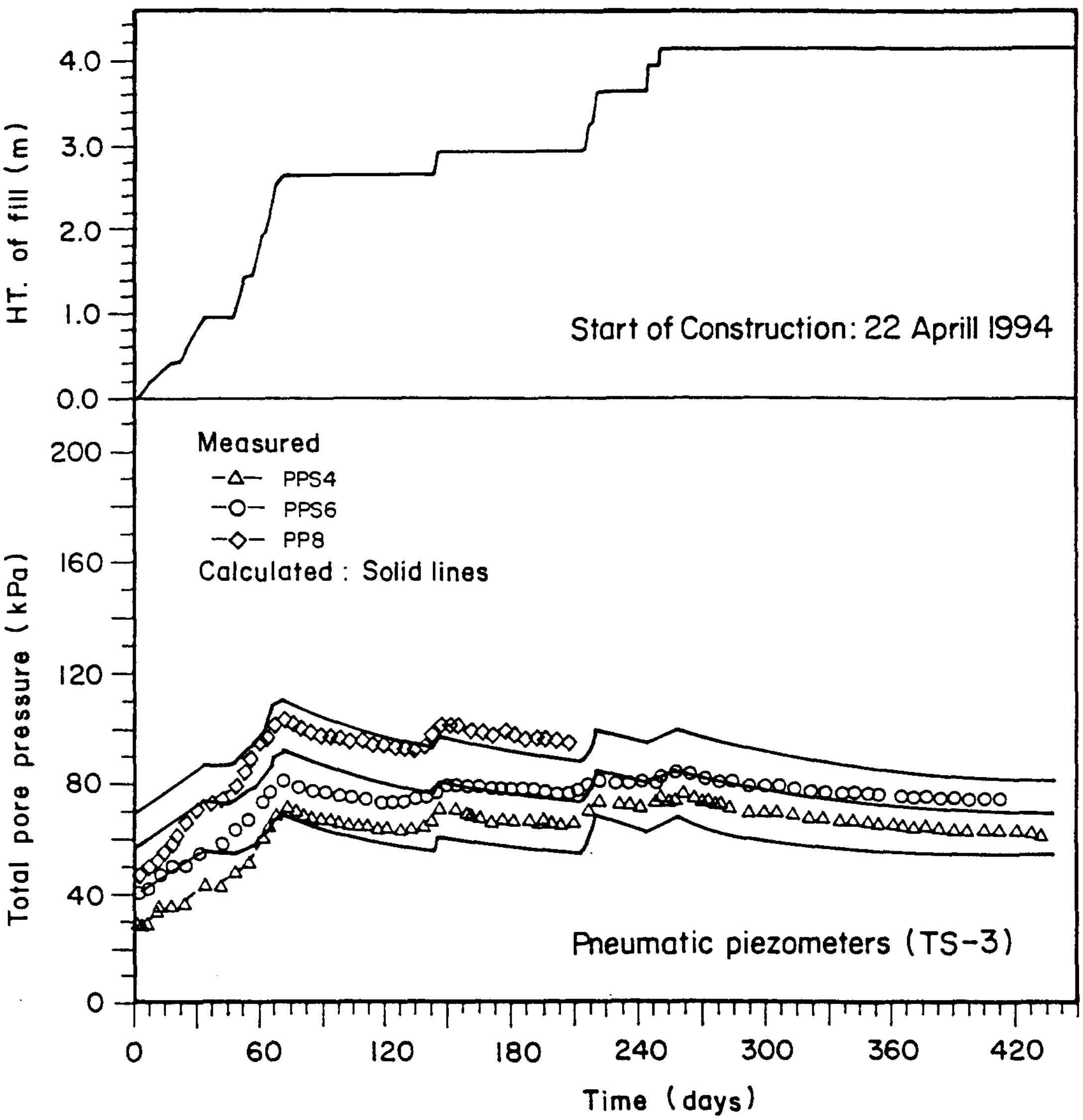


Fig. 25 Computed Pore Pressures from FEM and Measured Values-TS3

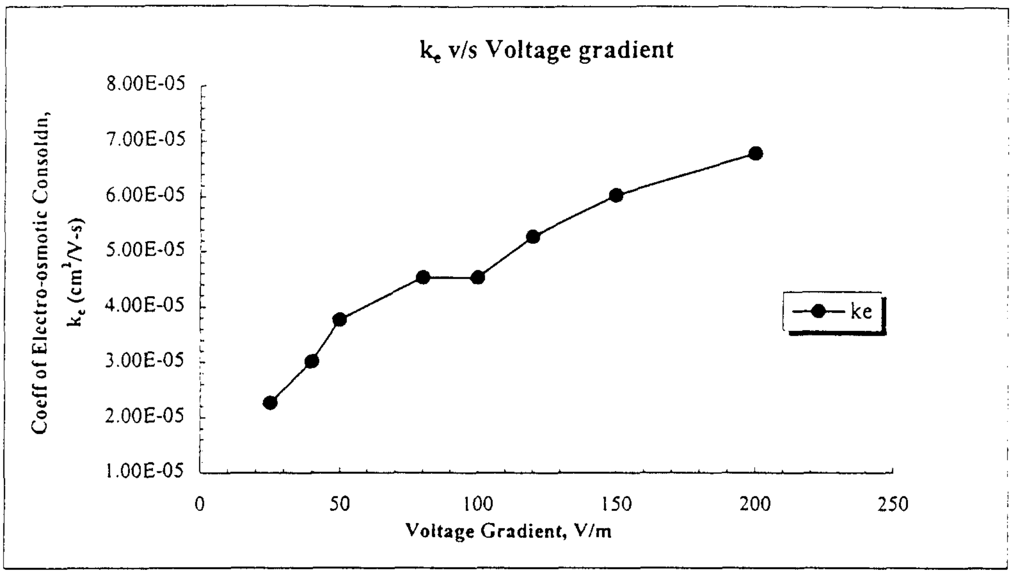


Fig. 26 Coefficient of Electro-Osmotic Consolidation - Voltage Gradient Plot

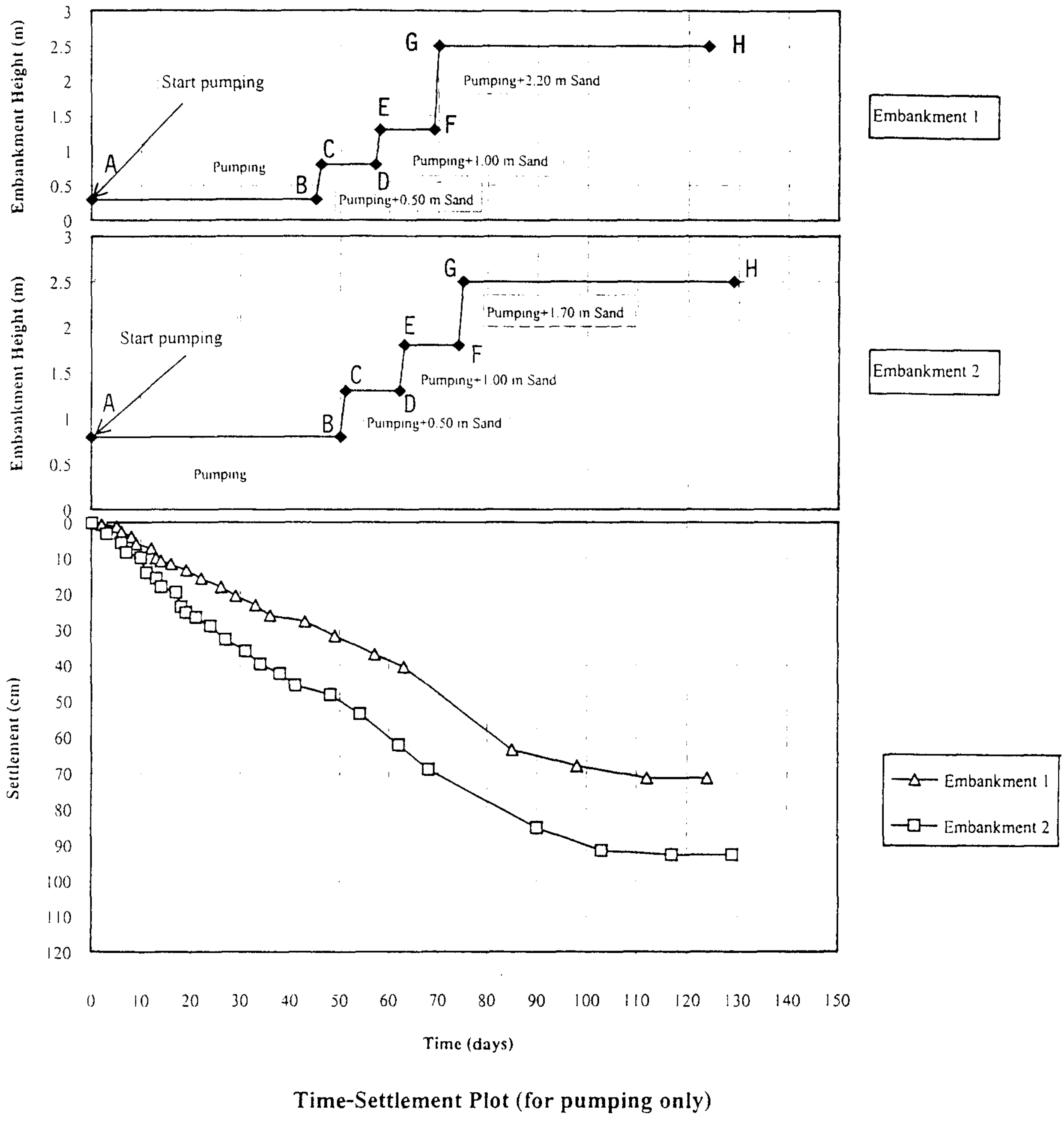


Fig. 27 Time-Settlement Plot (TV1 and TV2) with Loading Schedule

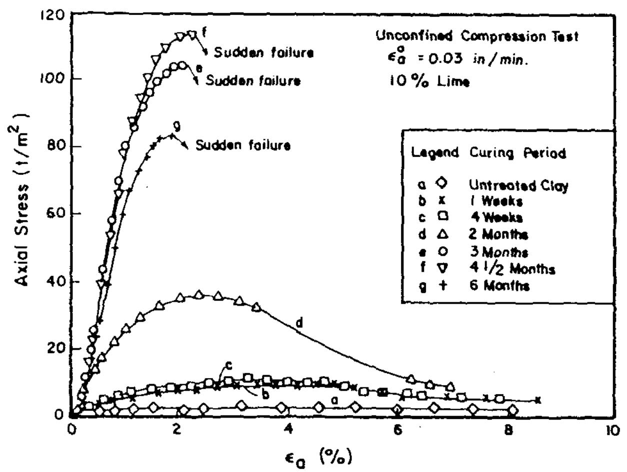


Fig. 28 **Typical Stress-Strain Curves from Unconfined Compression Tests (Lime -Treated Clay)**

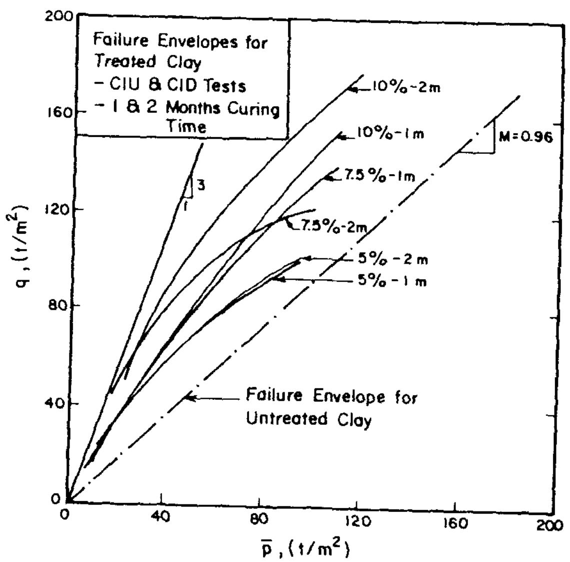


Fig. 29 **Failure Envelopes in (q, $\bar{p}$) Plot for Lime-Treated Clay**

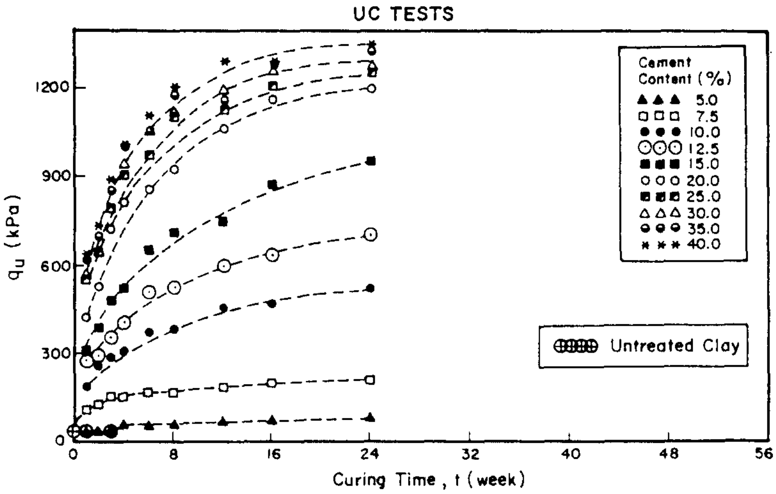


Fig.30 Unconfined Compressive Strength with Curing Time (Cement -Treated Clay)

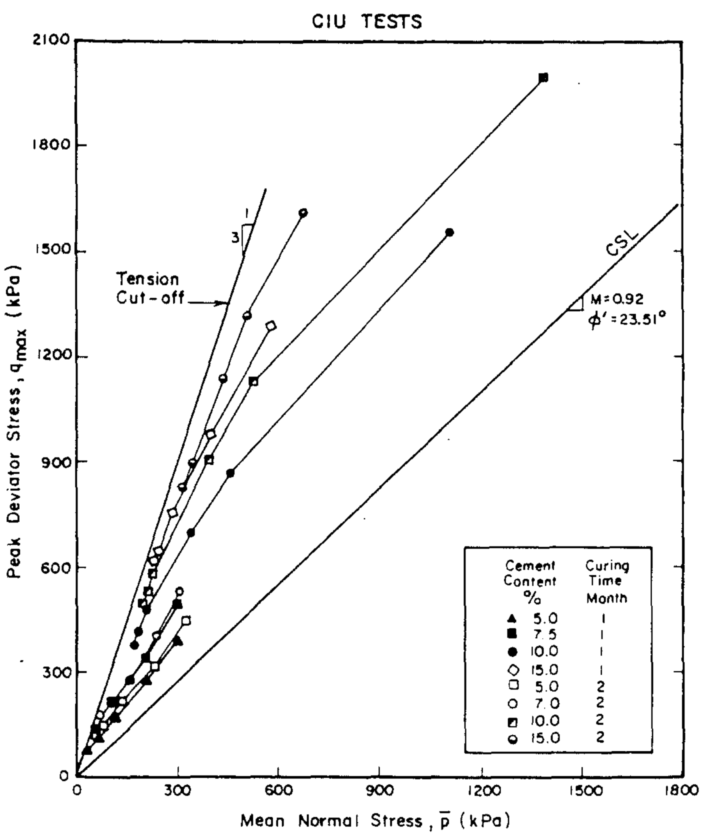


Fig. 31 Failure Envelopes for Cement- Treated and Untreated Clays from CIU Tests

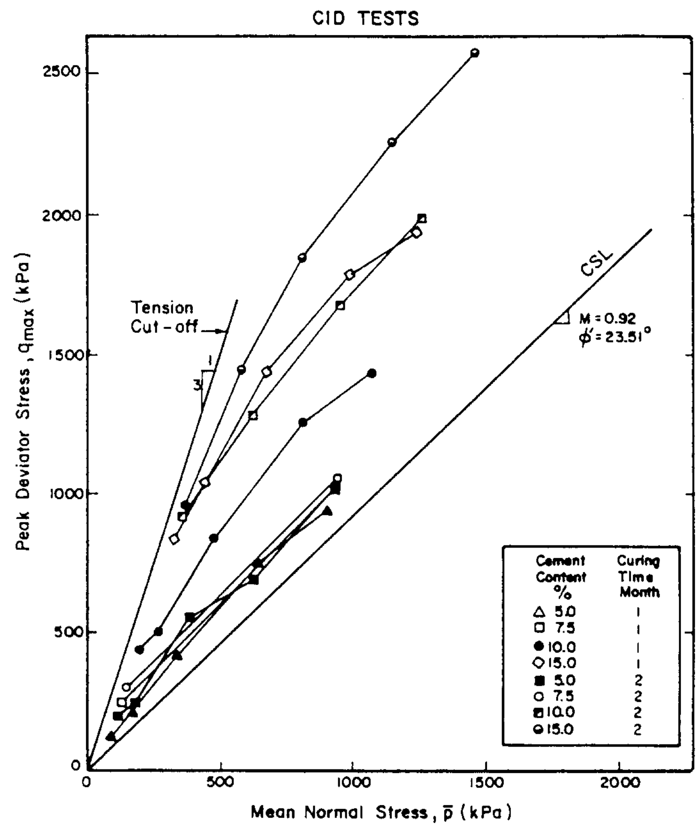


Fig. 32 Failure Envelopes for Cement -Treated Clays from CID Tests