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Large scaled field tests on soft Bangkok clay

Arumugam S. Balasubramaniam[1#]

Lecture

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Asian Institute of Technology
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Abstract

In this lecture the interpretations of fully instrumented tests embankments and their role in the development of appropriate ground improvement techniques for highways, motorways and airfields on soft clay deposits is illustrated through well documented case studies in Bangkok, Thailand and Muar Flat Site in Kuala Lumpur. For the Bangkok Plain and with sand backfills the performance of embankments with different schemes of vertical drains was evaluated over a period of 25 years. Aspects such as recharging effects due to the drains, inadequate measures in maintaining vacuum during vacuum applications and possible hydraulic connections with large diameter drains are discussed. For the Muar test embankments, the role of fill strength in residual soil embankment and the field deformation analysis in separating consolidation settlement from immediate settlement and creep settlements is presented. Novel interpretations of settlement from pore pressure dissipations, secondary settlement from field measurements and decay of lateral deformation rate with time were also made.

1. Introduction

In the evaluation of geotechnical parameters, traditionally laboratory tests are performed. However, when the quality of undisturbed samples as taken from boreholes or block samples is in doubt, in-situ tests are performed. These in-situ tests can be on small scale such as vane tests, cone penetration tests, pressuremeter tests and dilatometers tests. Large scale field tests are also carried out in parallel and these tests are fully instrumented. Over the last 32 years the senior author has been involved in the interpretation of the test data on several embankments in Thailand, Malaysia and in Southeast Queensland. This paper touch upon some of the lessons learnt from these studies and how they have improved our understanding on soft clay behaviour and ground improvement schemes as studied with test embankment. Major emphasis will be on the cases in Thailand and Muar site Malaysia and these experiences will be presented in chronological order and not country wise. The concept of large scale field test arise from the need that our single element laboratory scale tests are not adequate to cover all the features that we encounter in sedimentary soils with varying layer thickness and soil properties; also the small scale field tests can at time mislead the large scale behaviour covering a much larger loaded area. Thus the concept of building test embankments began.

Typical soil profile in the Bangkok Plain is shown in Figure 1. Eide (1977) reported the results of a test section on the Bangkok-Siracha Highway as measured in 1966. The ground condition along the route was considered to be very soft. Sand drains of 0.2 m diameters were installed by the displacement method. They were placed in triangular pattern at 2 m spacing. Sand drains increased the rate of settlement but not to a sufficient degree. The most negative aspect quoted by Eide (1977) was that, even though the sand drains 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 per month, which was considered to be high. Possibly due to the low factor of safety, a substantial part of the total settlement was due to continuous undrained creep without volumetric strain, when the stress states are close to failure. Further, Cox (1968), in his research report at AIT, concluded that the design and construction of road embankments over the soft deltaic clays in South East Asia is a very complex engineering problem. This is because the pore pressure response and settlement characteristics correspond to lightly over-consolidated states rather than normally consolidated states.

#Corresponding author. E-mail address: a.bala@griffith.edu.au
[1]Griffith University, Griffith School of Engineering, Gold Coast, Queensland, Australia.

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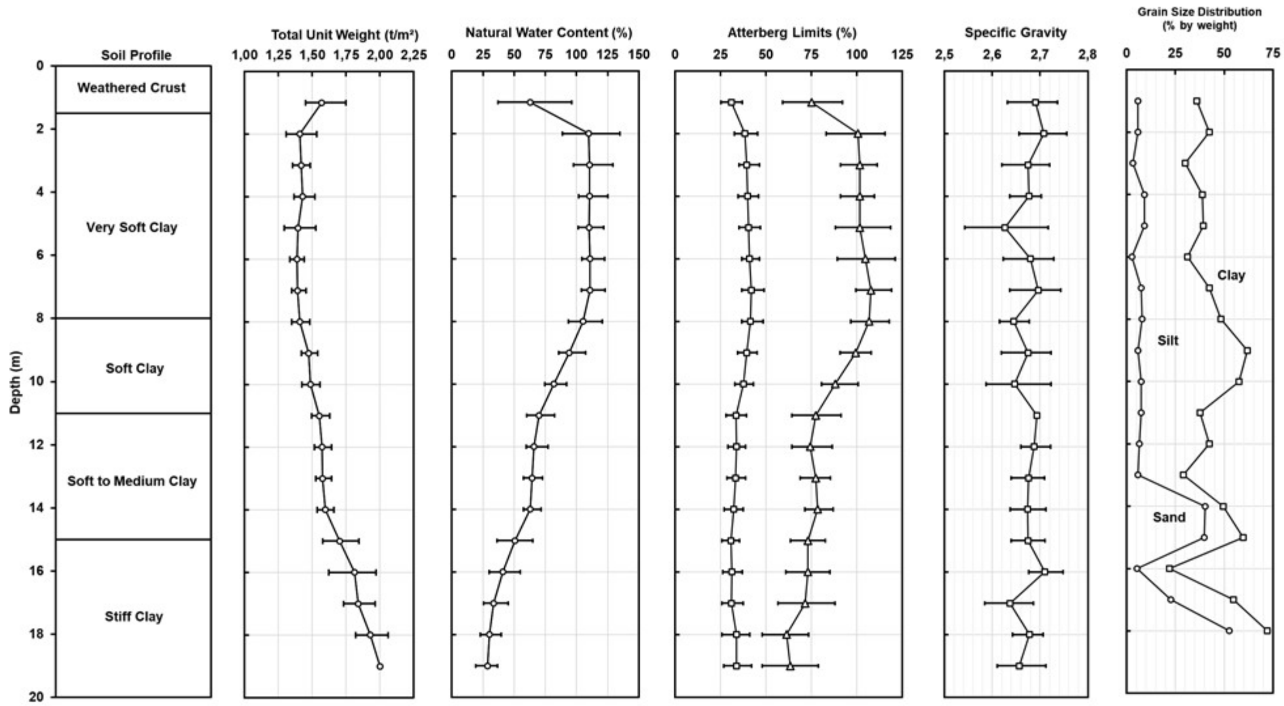


Figure 1. Typical soil profile in the Bangkok plain.

Dr. Za Chieh Moh and his colleagues were the pioneers in the study of test embankment at the Second International Airport site in Bangkok as early as 1970. Full scale test embankments were built at Nong Ngo Hao by Moh et al. (1973) to study the in-situ behaviour. Embankment I was built rapidly to failure. The failure height was only 3.4 m. Embankment II was in two sections - one was with height varying from 0.5 m to 2.9 m and the other had the 2.9 m constant height. These embankments were studied thoroughly for their behaviour by several masters and also Doctoral student at AIT.

The next major study was associated with the Royal Thai Navy Dockyard in Pomprachul, Thailand and work began on this scheme in 1975. The test site is situated at the mouth of the Chao Phraya River in Samut Prakarn province, approximately 20 km south of the Bangkok city. The embankment, which was built in two stages, was 90 m long, 33 m wide, 2.35 m high and consisted of three sections, namely a section with no drains, a section with 2.5 m spacing, and a section with drains of 1.5 m spacing (as shown in Figure 2a and Figure 2b). The soil profile is in Figure 2c. The sand drains consisted of small diameter (0.05 m) sand wicks and were installed to a depth of 17 m by the displacement method. These sand wicks were constructed at the site by pouring sand inside a permeable membrane. First a closed end steel casing 75 mm internal diameter was driven into the ground and the sand wick was lowered into the casing and the casing was subsequently withdrawn. A total of 166 piezometers were installed below the test fill area and outside of it. Surface and subsurface settlement points were installed to monitor the settlement along the centre line and the edges of the test embankment. Three hydrostatic profile gauges were installed, that is, one along each central cross-section of a test section. Also, eleven magnetic movement plates were used to monitor lateral displacements along the gauge. Three inclinometer casings were installed along the centre line of each test section.

At the airport site in Nong Ngu Hao, the most extensive sand drain studies on test embankments were performed in 1983 (see Moh et al., 1987) as part of the ground improvement scheme for the runway pavement and other sections of the taxiways and landside roads. Sand drains of minimum diameter 0.26 m were installed to a depth of 14.5 m by water jetting. The test program included three test areas, one with surcharge fill, and one with vacuum loading and a third with ground water lowering. Test Section 1 was 40 m × 40 m in plan and sand drains were installed at 2 m spacing in triangular pattern. The vacuum load was not successful as several leakages developed and finally the section was covered with plastic shield. Test Section 3 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 1; the loading was not successful. The test Section 2 was slightly larger than test Section 1 and pre-loading of 60 kN/m^2 was applied in three stages. While difficulties were encountered in maintaining the vacuum load as well as

(a)

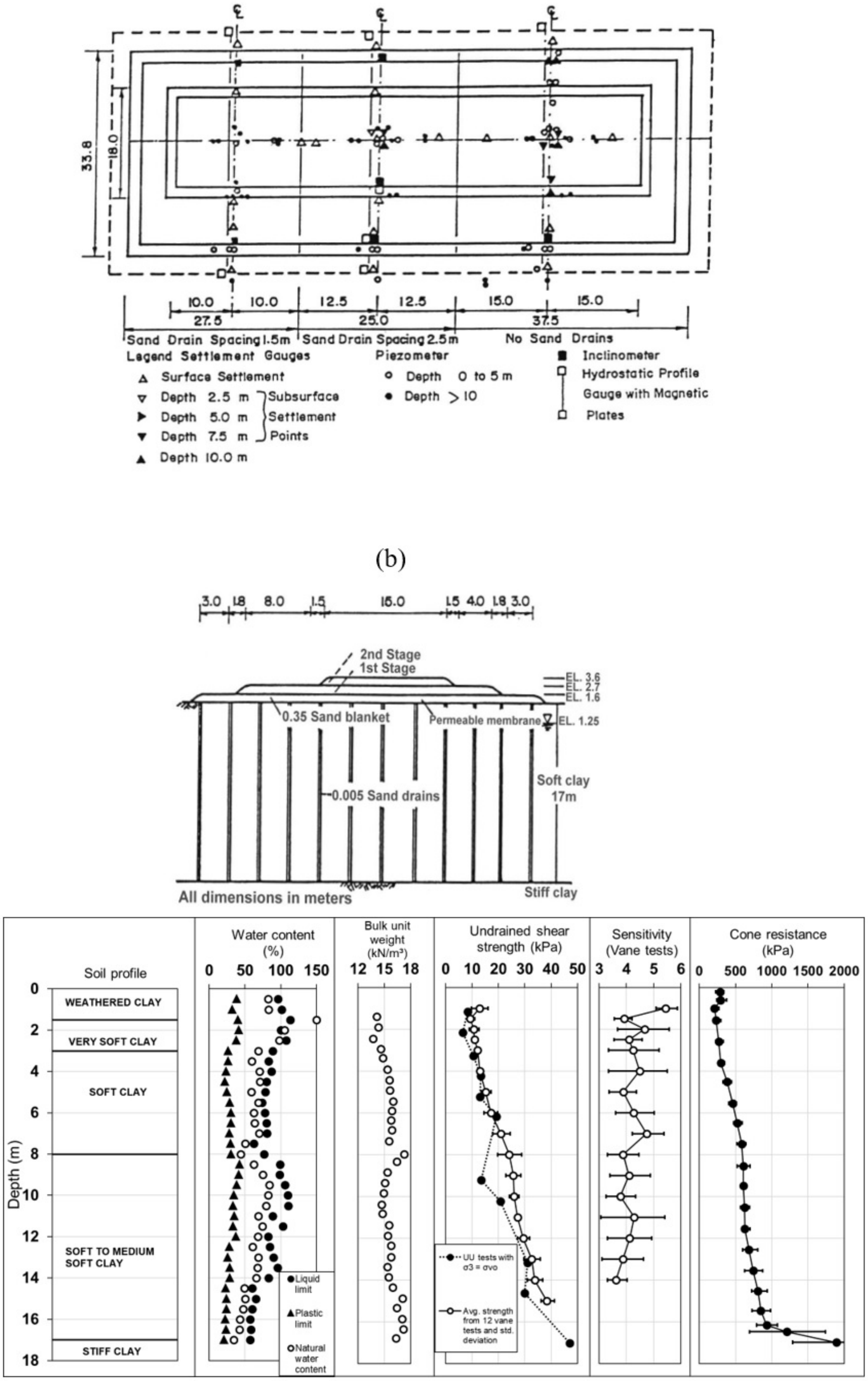


Figure 2. (a) plan of test embankment at RTN Dockyard site; (b) elevation of test embankment at RTN Dockyard site; (c) general properties of Pom Prachul Clay at RTN Dockyard site.

the ground water lowering, the embankment surcharge was found to be a reliable technique when compared to vacuum loading in accelerating the consolidation with sand drains.

A significant extent of the North-South Expressway from Bukit Kayu Hitam at the Malaysian-Thai border to Johor Baru at the southern most location passes through soft clay deposits. Fourteen soil improvement schemes were designed and constructed at Muar flat sites. These embankments were instrumented to measure the vertical settlements, lateral movements and pore pressures. Table 1 gives brief details of these embankments and also they are also shown in Figure 3. Several of these embankments were studied by the first author and his colleagues at AIT with a number of Masters Students as their thesis research. In particular the embankment built with residual soil and used as the embankment in a prediction symposium and the work of Balachandran (1990), Ratnayake (1991), Loganathan (1992) and Sivaneswaran (1993) in particular will be referenced. The work of Balachandran (1990) extends the prediction made by the first author on the embankment built to failure. Ratnayake (1991) analysed the embankment with vertical drains. Loganathan (1992) used the field deformation analysis to separate the immediate settlement from the consolidation settlement during the loading stage and to separate the consolidation settlement and the creep settlement during the performance stage. This technique was different from the Asaoka technique (Asaoka, 1978) used to estimate the consolidation settlement especially under one dimensional consolidation. When high embankments were built creep is a major factor and this makes it difficult to estimate the so-called residual settlements during the maintenance period in most road works and other projects. The work of Sivaneswaran illustrates the powerful tool of normalised settlement and normalized lateral deformation in studying the ground response under different schemes of ground improvement.

Table 1. Muar flats ground improvement schemes.

Scheme	Ground Improvement	Height
3/2, 6/6	Control	3 m, 6 m
3/1, 6/1	Electro Chemical Injection	3 m, 6 m
3/3	Sand Sandwick Method	3 m
3/4, 6/8	Preloading, Geogrid & Prefabricated Vertical Drain	3 m, 6 m
6/2	Well Point Preloading	6 m
6/3	Electro Osmosis	6 m
6/4	Prestressed Spun Piles	6 m
6/5	Sand Compaction Piles	6 m
6/7	Vacuum Preloading & Prefabricated Vertical drain	6 m
6/9	Preloading & Vertical Drain	6 m

The above experiences and the lessons learnt led to a rather cautious approach, on the final studies carried out (for a two year period from 1994 to 1995) for the construction of the runway and other structures at the Second Bangkok International Airport (SBIA) site. In this study, the use of pre-fabricated vertical drains (PVD) and surcharge was concluded as the most suitable ground improvement technique.

1.1 Vertical drain studies in Bangkok Siracha Highway (1967)

The potential use of surcharge and vertical drains as a ground improvement technique was explored as early as 1966 (see Eide (1977)) for its application in bridge approaches.

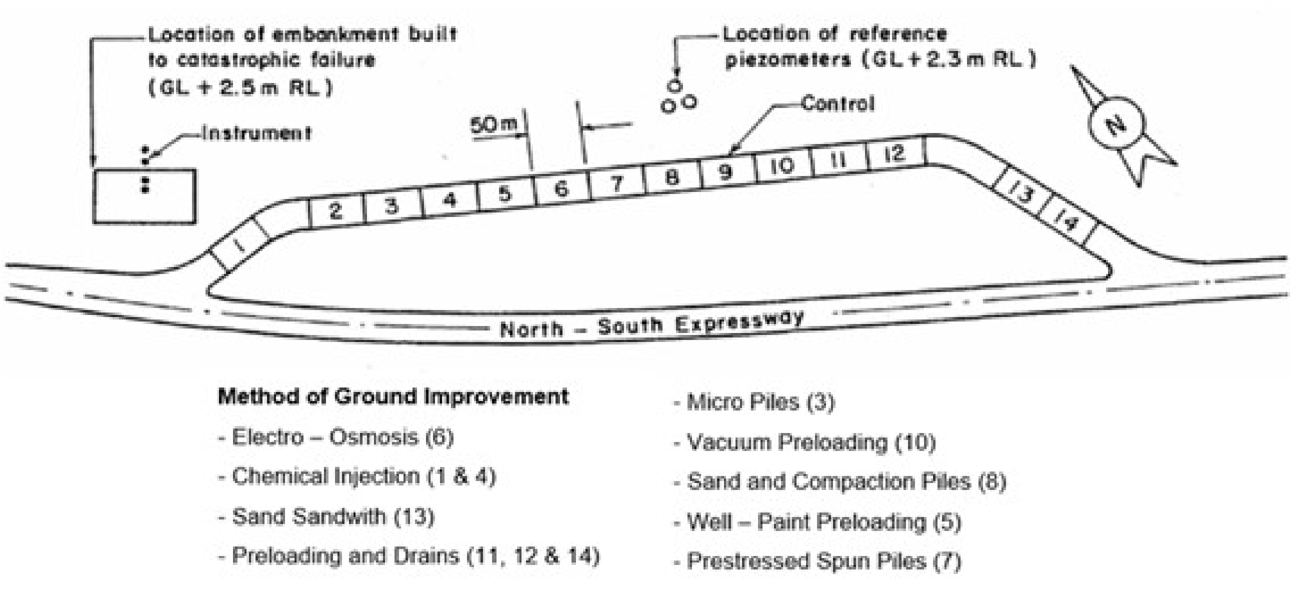


Figure 3. Locations of embankments.

A test section on the Bangkok-Siracha Highway was built and tested with 0.20 m diameter sand drains and the depth of sand drains varied in steps from 4 to 13 m. During construction, failure of the soft clay occurred in five locations and the 2.2 m high embankment was furnished with 8 m wide berms to assure stability. From the observed settlement of the test embankment, Eide (1977) concluded that the drained embankment section settled most to start with, but after one year the rate of settlement was almost the same, approximately 0.0025 m per month on both the drained and undrained sections. The practical conclusion reached by Eide (1977), was that this type of drain and spacing would not serve the purpose in bridge approaches. However, Eide (1977) agreed that the results obtained might have been affected by other factors such as slip failures, the remoulding effect from the installation of drains, the low permeability and greater secondary consolidation settlement of the soft clay.

1.2 Vertical drains study at the Dockyard site

A second attempt was made with vertical drains in 1975 to 1977, with the construction of a dockyard for the Royal Thai Navy, and a test embankment was built (see Figure 2a and Figure 2b) in order to evaluate the performance and suitability of sand drains (sand wick type of drains) as a mean to accelerate the consolidation of the soft clay. Without an understanding of the recharge effect due to the sand wicks on the piezometric drawdown, doubts were cast on the pore pressure measurements as revealed below the natural terrain in which the sand wick was installed and this led to the installation of additional piezometers and finally there were a total of 166 piezometers monitoring the pore pressures below the clay surface. It was found that, the installation of the sand wicks have recharged the area below the test embankment with and without drains, and erased the normal piezometric drawdown which normally existed in the Bangkok subsoils due to deep well pumping, as can be seen in the area which was not influenced by the test embankment location installed with sand wick drains. This recharging effect was somehow not envisaged in the original planning and design of the test embankment. However, it gave valuable insight into the extensive deep well pumping in the Bangkok plain, and the piezometric drawdown that has resulted in the Bangkok subsoils as a consequence of this ground water withdrawal.

1.2.1 Description of test embankment and soil profile

The performance of sand drains (sand wick type of drains) at the Naval Dockyard site, Pom Prachul, Thailand was investigated in 1975 by constructing an instrumented test embankment. The test site is situated at the mouth of the Chao Phraya River in Samut Prakarn province, approximately 20 km south of the Bangkok city. The embankment, which was built in two stages, was 90 m long, 33 m wide, 2.35 m high and consisted of three sections, namely a section with no drains, a section with 2.5 m spacing, and a section with drains of 1.5 m spacing (as shown in Figure 2a and Figure 2b). The soil profile is shown in Figure 2c. The sand drains consisted of small diameter (0.05 m) sand wicks and were installed to a depth of 17 m by the displacement method. These sand wicks were constructed at the site by pouring sand inside a permeable membrane. First a closed end steel casing 75 mm internal diameter was driven into the ground and the sand wick was lowered into the casing and the casing was subsequently withdrawn. A total of 166 piezometers were installed below the test fill area and outside of it. Surface and subsurface settlement points were installed to monitor the settlement along the centre line and the edges of the test embankment. Three hydrostatic profile gauges were installed, that is, one along each central cross-section of a test section. Also, eleven magnetic movement plates were used to monitor lateral displacements along the gauge. Three inclinometer casings were installed along the centre line of each test section. The typical soil profile at the site is shown in Figure 2c.

1.2.2 Pore pressure measurements below the test embankment

The loading pattern for the three test sections and the pore pressure observations are shown in Figure 4a. The piezometric drawdown is shown in Figure 4b. Before constructing the test embankment a sand blanket of 0.35 m was placed and this corresponded to a surcharge pressure of 5.5 kN/m^2. Then the first stage of loading was carried out in 25 days with a fill height of 1.1 m and a surcharge pressure of 19 kN/m^2. Thus, at the end of the first stage of loading the surcharge pressure was 24.5 kN/m^2. After a waiting period of 60 days, the embankment was raised to 2.35 m (inclusive of the 0.35 m sand blanket). The second stage loading was for a period of one month and the observations were continued for a total period of ten months.

The piezometers P41, P27 and P13 located at 7.5 m depths in the three sections indicated consistent rise during the loading phase and drop in values during the waiting period and under the time observed after the full surcharge. These piezometers measured the absolute values of the pore pressures and thus included the original ground water pressure before loading. As shown in Figure 4a, the static ground water pressure was observed from the observation well with hydrostatic assumptions and no drawdown. The dummy piezometers installed at a location far away from the test embankment and without the influence of the sand wick and the surcharge will give the static water pressure inclusive of any possible drawdown due to deep well pumping. At this site the piezometric drawdown only start at a depth below 7.5 m and as such the hydrostatic water pressure indicated by the observation well and the dummy piezometers located at 7.5 m depths are more or less coincident, indicating very small drawdown.

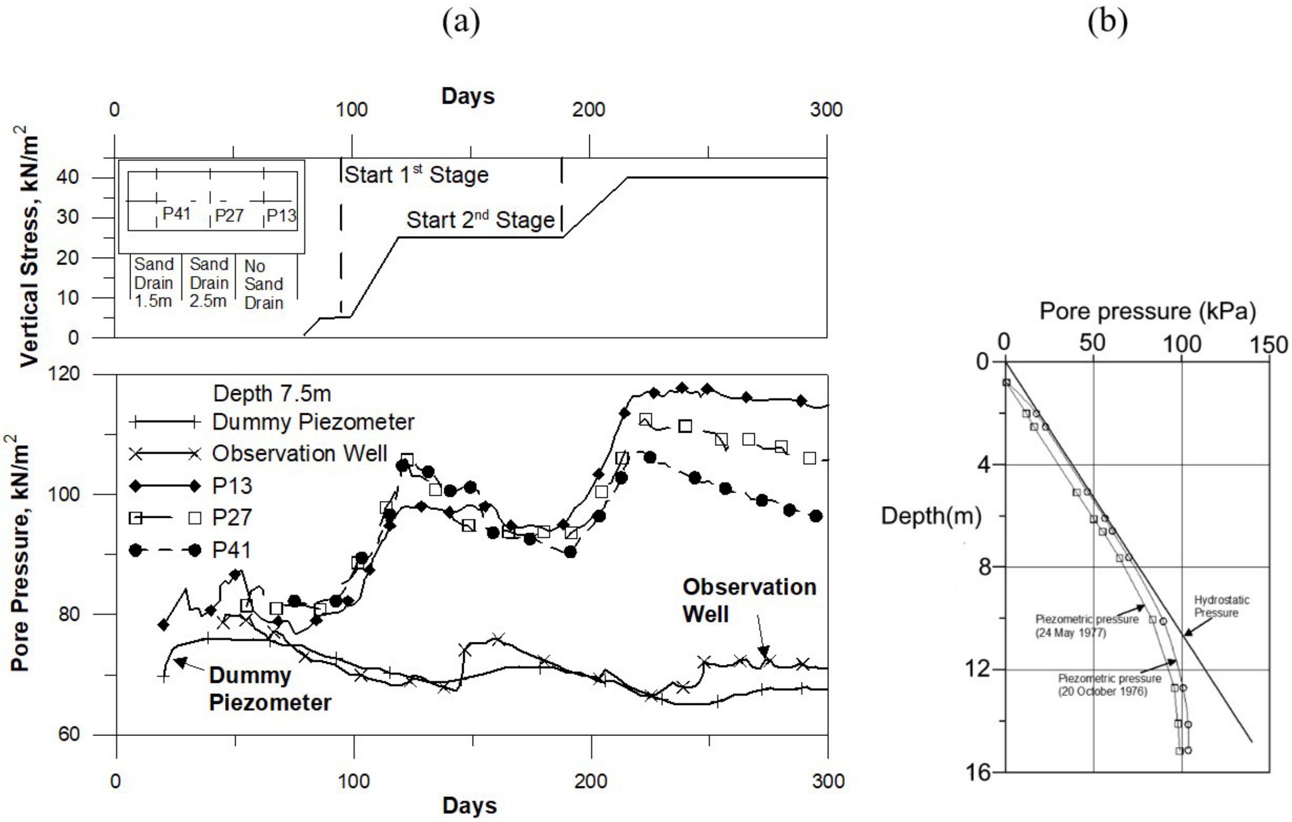


Figure 4. (a) typical piezometer readings at RTN Dockyard site; (b) variation of piezometric pressures with depth at RTN Dockyard site.

All the 166 piezometers were indicating consistent values of the pore pressures. However, the active piezometers installed at depths of 10 to 15 m indicated that the piezometric drawdown below the embankment is more or less erased by the sand drains which have recharged the drawdown area back to its hydrostatic conditions. Thus at deeper depths the absolute values of the piezometer readings under the embankments will be a sum of the static water pressure without any drawdown and the excess pore pressure induced by the surcharge loading. In order to clarify the situation, additional piezometers were installed along the centre line of the longitudinal section of the test embankment from the northern edge corresponding to the closely spaced sand drain section and also along the edge of the eastern boundary of the test embankment. Figure 5 shows the distance from the edge of the embankment in the east direction, up to which the drains have influenced in erasing the drawdown. A similar phenomenon is noted in the north direction along the centre line. In both directions the full draw down was only realized at distances of about 15 to 20 m away from the edges of the embankment. This would indicate that having the three sections side by side without any space in between them was a grave mistake in planning the overall testing program. Ideally speaking the three sections must be separated from each other with substantial allowance for the zone of influence of the drains in recharging the draw down area below the embankment. This was observed in the final planning of the test embankment with PVD at the Second Bangkok International Airport (SBIA) site in the 1994-1995 studies. The three test embankments at the SBIA site were separated from each other with substantial space between them.

1.2.3 Measured and computed settlements at the Dockyard site

Settlement records from 47 active settlement plates have been studied and typical cases are plotted in Figure 6. In this figure the surface settlement at the centre line of the three test sections are plotted with respect to time. Also shown in this figure is the loading pattern with time in terms of the surcharge stress (vertical stress increment). These settlement records are in accordance with the pore pressure dissipation, shown in Figure 4a. The section with closer drain spacing showed higher settlements than the one with wider spacing and the one with no drain. It has already been discussed that the wider spaced drain section was interfering with the no drain section. Thus substantial lateral drainage would have taken place in the no drain section, due to the influence of the nearby adjoining drain section with wider spacing.

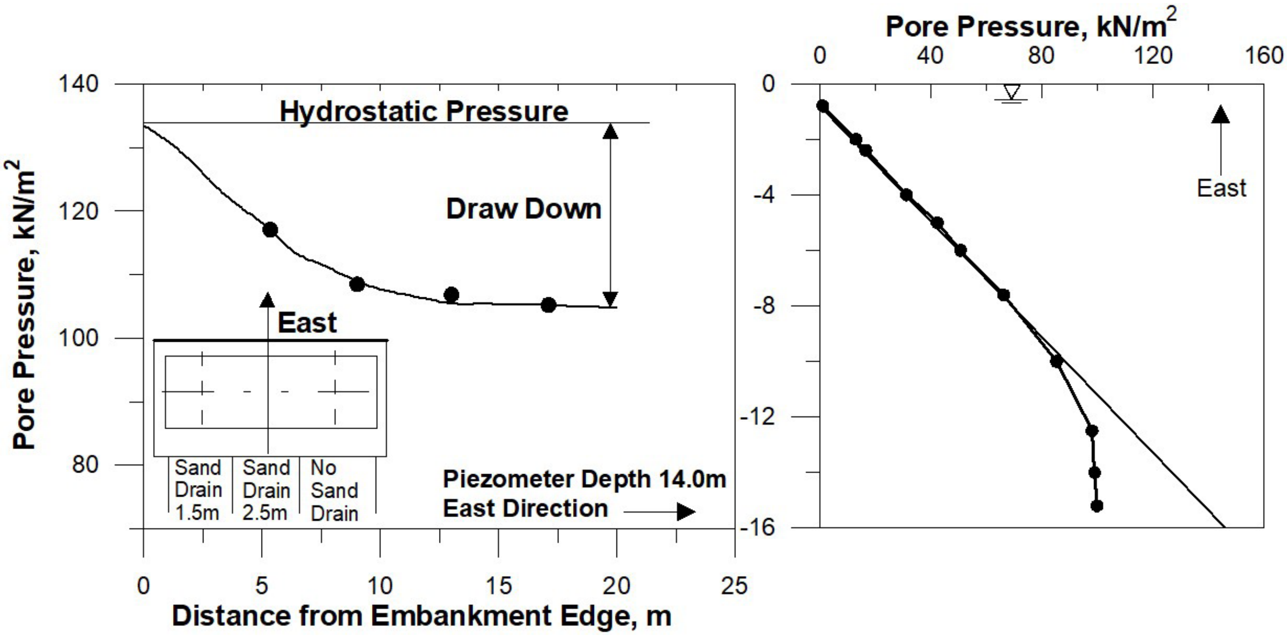


Figure 5. Piezometric pressure vs. distance from the edge of the embankment at RTN Dockyard site (East direction).

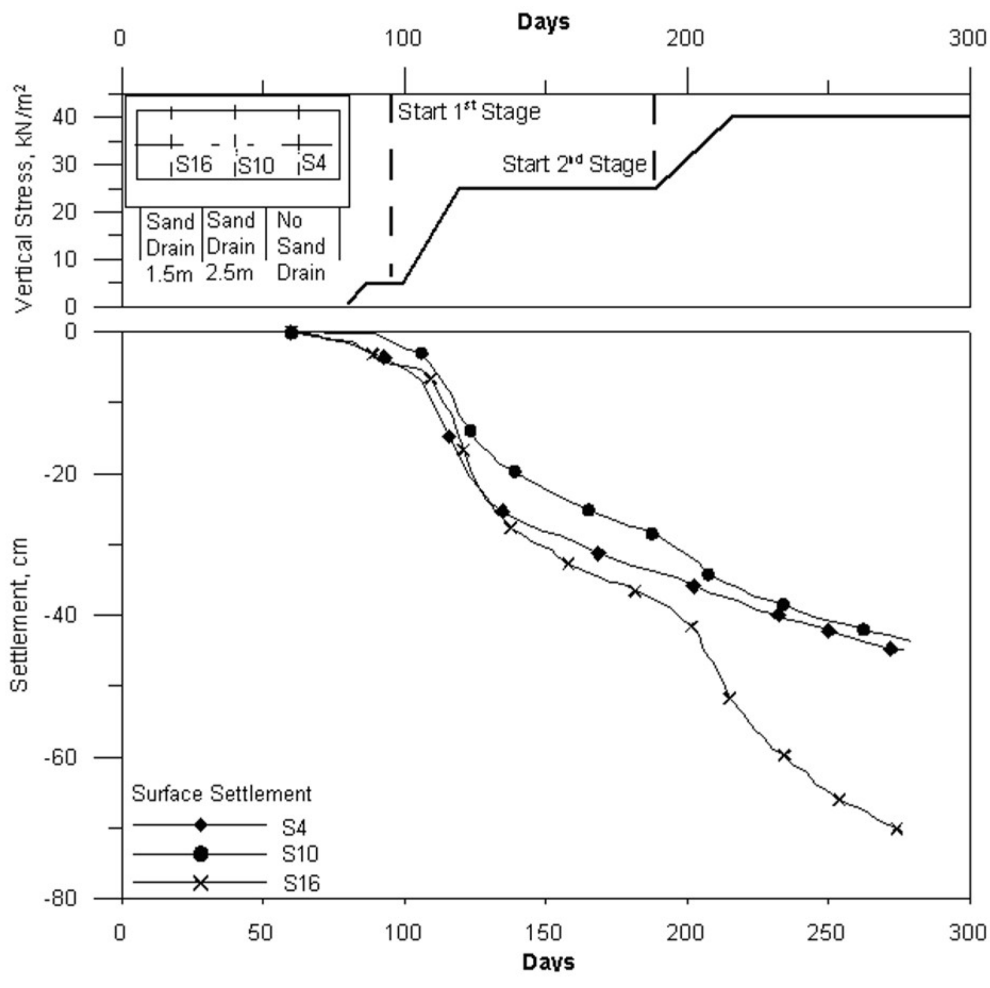


Figure 6. Typical settlement point readings at RTN Dockyard.

Using the elastic theory for increment in stresses below the test sections and the undrained modulus from CK_0U triaxial tests immediate settlements were computed and plotted in Figure 7. It can be seen that very little immediate settlement occurs below a depth of 11 m. The immediate surface settlement under the first stage loading and the second stage loading was computed to be 36 mm and 115 mm respectively. The total primary consolidation settlement under the embankment loading was calculated using the stress strain curves from Oedometer tests. Figure 8 shows the variation of the primary consolidation settlement with depth. In Figure 8, the consolidation settlements were computed from six series of consolidation tests performed with the Lever Arm Consolidometer, Anteus Consolidometer and Bishop Consolidometer. In Series SC seven tests were conducted from 1 to 16 m with 24-hour load increment duration and load increment duration of one. Similar series of tests were conducted in the Anteus Consolidometer as Test Series AC.

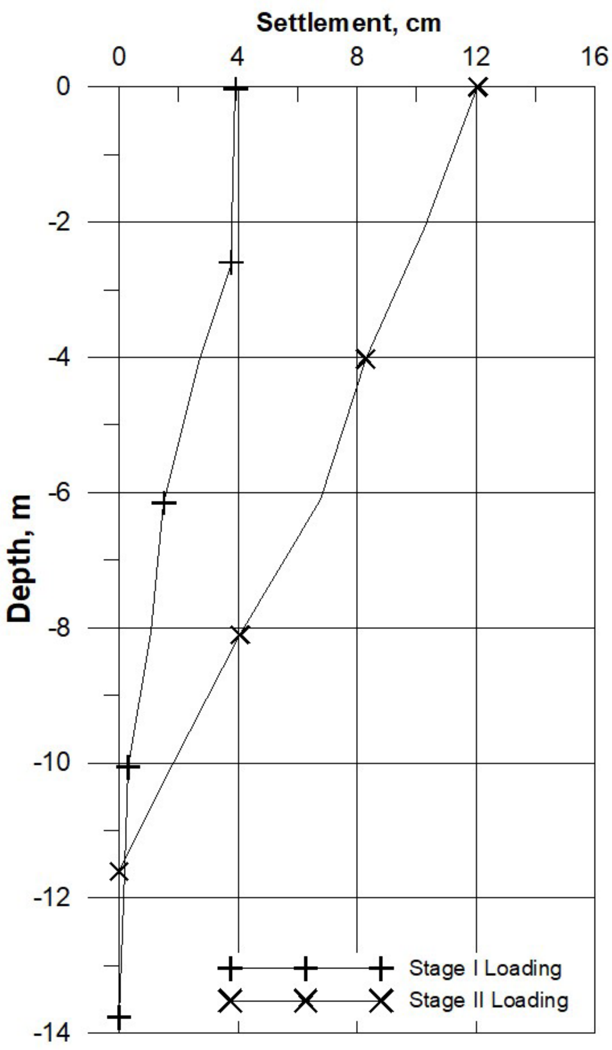


Figure 7. Immediate settlement below embankment at RTN Dockyard site.

Bishop Consolidometer was used in Test Series BC and the load increment duration and ratio were similar to the SC and AC series. In Test Series SI, small load increments were used in the Lever Arm Consolidometer to determine the accurate determination of the pre-consolidation pressure. The measured settlements were in the drained section with 1.5 m drain spacing. These data indicate that even in the 1.5 m-drained section, the primary consolidation is not yet completed.

The major lesson learnt from this trial embankment was that, the sand wick drains have recharged the zone that was originally under piezometric drawdown. Thus, for the bottom later of the soft clay the recharging would have resulted in a decrease in the effective stress and this is the reason why the settlements were smaller below the upper 5 m where there was no drawdown before. Also, the recharging zone seems to extend laterally up to a distance of about 15 m or so, as such the no-drain section would have been influenced by the drained section. It was therefore better to have had three separate sections spaced out in such a manner that there is minimum interference, and the data from each embankment truly represents a no-drain section and the sections with the wider and narrower spacing of drains.

2. Muar clay test embankments

Several lessons were learnt from the analysis of the Muar clay test embankments. The well documented publications

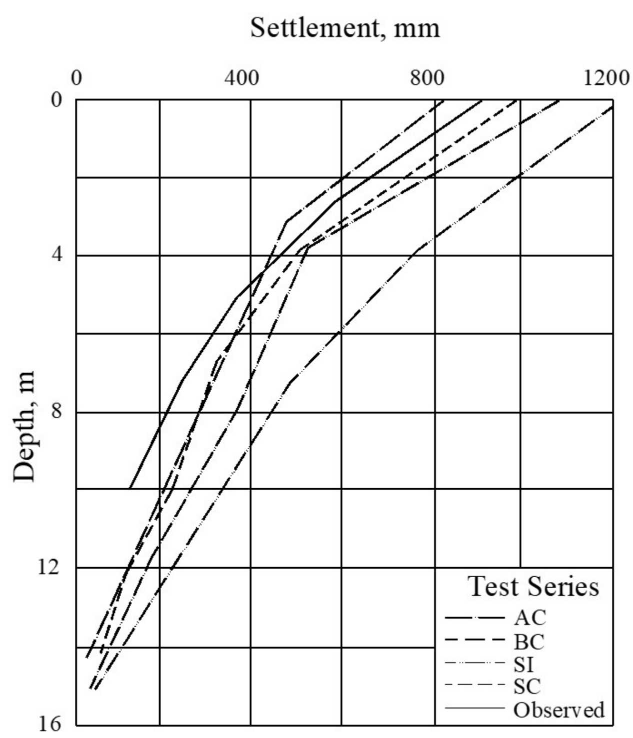


Figure 8. Consolidation settlement below embankment as calculated at RTN Dockyard site.

on the behaviour of the residual soil test embankment built to failure indicated how the performance of such a simple field problem can deviate from the known behaviour of sandfill test embankments. The inclusions of the strength of well compacted residual soil fill material, seem to offer good tensile strength characteristics and prevented the formation of tensile cracks at the base of the embankment. The performance of this embankment, especially pore pressure pattern, stability, settlement, and lateral deformation, were predicted by four predictors, namely, Prof. A. S. Balasubramaniam (Thailand), Prof. J. P. Magnan (France), Prof. A. Nakase (Japan) and Prof. H. G. Poulos (Australia). Their predictions were presented in the "International Symposium on Trial Embankments on Malaysian Muar Clays," in November 1989, held in Kuala Lumpur, Malaysia. All predictors were given the same soil properties and field instrumentation results (Brand & Premchitt, 1989). Most experts also made poor predictions of settlements pore pressures and lateral movements.

The CRISP program as based on the critical state soil mechanics is superior in predicting the coupled behaviour of undrained and consolidation phenomenon in these embankments. The Muar clay test embankments also illustrated that the Pads available in the market for accelerating the dissipation of pore pressures are not 100 percent effective as expected by classical theories of Barron and others. Hansbo in particular considered the non-Darcian flow of water during consolidation with drains in clays and also several other authors studied the effect of smear and possible well resistance in the drains. The Muar clay test embankments also showed the defects in using sand compaction piles, piled embankments and the use of electro-osmosis.

Further studies conducted at AIT on the creep behaviour of the Muar clay test embankments in which continuous undrained creep occurred with the increase in lateral deformation indicate that undrained creep in soft clays due to high embankments is quite substantial. In places where high embankments are constructed with residual soils such undrained creep plays an important role. Loganathan (1992) used the field deformation analysis to separate the immediate settlement from the consolidation settlement during the loading stage and to separate the consolidation settlement and the creep settlement during the performance stage. This method was different from the Asoka technique used to estimate the consolidation settlement especially under one dimensional consolidation. When high embankments are built in soft clays creep is a major factor and this makes it difficult to estimate the so-called residual settlements during the maintenance period in most road works and other projects. Table 1, Figure 3 and Figure 9b contain details of the test embankments built at the Muar Plain. Details of the soil profile at the Muar test embankment site is shown in Figure 9a.

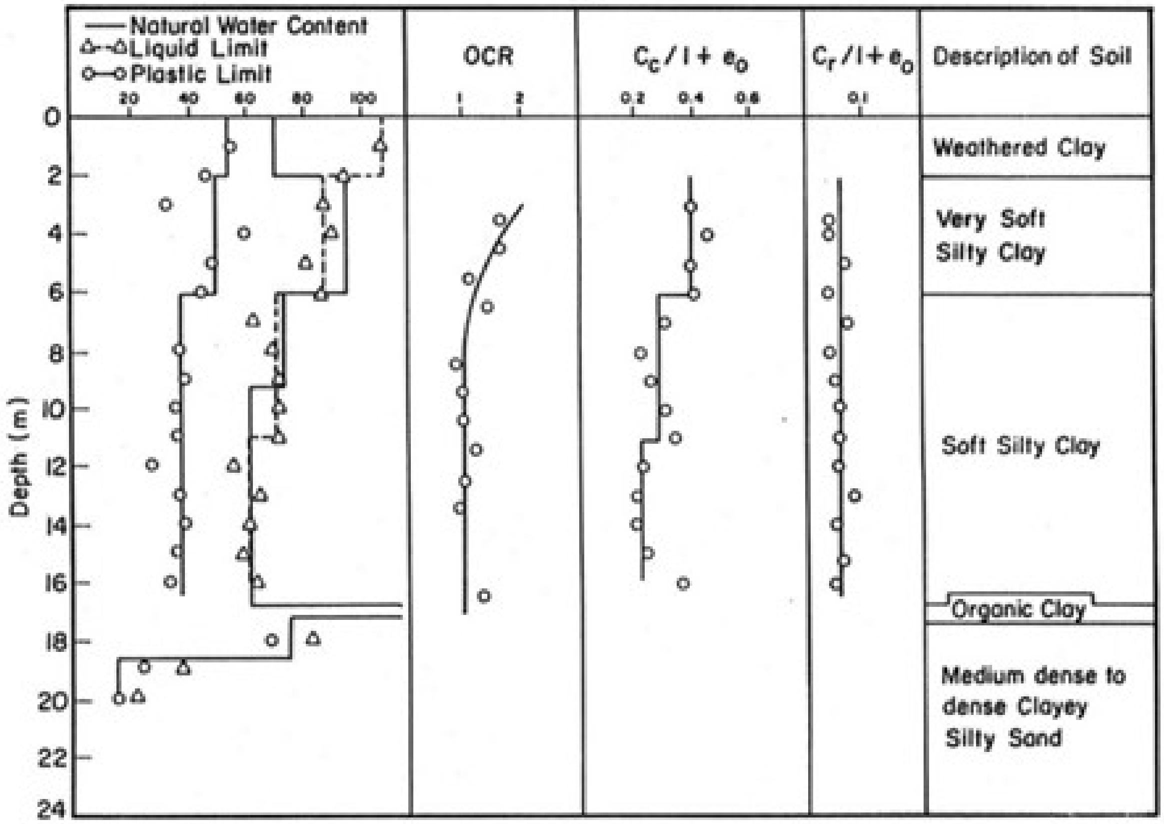
The total settlement observed beneath an embankment subjected to step loading, is basically a combination of different components, namely, immediate settlement, consolidation settlement, and creep settlement. Establishing relationships among these settlements components, upon separating them from the total settlement observed in the field, will facilitate settlement predictions from relatively simple numerical computations. The separation of settlement components provides better understanding of settlement mechanism and thus far better design of step loading. Time-dependent deformations due to undrained creep can be quite large in both normally, consolidated and highly over-consolidated clays. Creep effects are more important for horizontal than for vertical deformations (Christian & Watt, 1972). However, coupling of drained creep with the undrained one could be analytically more cumbersome and would require soil data that are difficult to obtain. A new methodology, termed as field deformation analysis (FDA), based on the changes in volume of foundation soil under embankment loading, is proposed by Loganathan (1992) to separate and quantify settlement components. Shibata (1987) observed that significant volume changes occur during embankment construction and that the behavior of the embankment deviated significantly from undrained conditions. Ting et al. (1989) and Toh et al. (1989) used a similar concept, considering volumetric deformation of embankment foundation under loading, to separate settlement components for Malaysian embankments.

The total settlement observed during loading is a combination of immediate and consolidation settlement components. Figure 10 shows the subsoil deformation pattern due to undrained deformation, which causes the immediate settlement. Since this occurs in an undrained manner, the magnitude of settlement deformation volume, designated as *AOC,* should be equal to the lateral deformation volume, designated as *APM.* Due to dissipation of excess pore pressures, the process of consolidation takes place simultaneously. Figure 10 also shows the ultimate deformation pattern of the embankment foundation at the end of loading, where the volume changes vertically *(ABC)* and laterally *(APMQA)* are due to consolidation. It should be noted that the volumes referred to here are for the unit length of the embankment.

The observed settlement volume in the field from settlement gauge readings, for half the embankment is defined as V_{vL} (volume OAB). The settlement volume V_{vL}(volume OAB) at the end of each loading stage is the resultant of the volume change due to the immediate settlement (V_{iL}= volume OAC) and the consolidation settlement (V_{cL} = volume CAB) as shown in Figure 10. Since the loading period is comparatively small the creep settlement is ignored. The lateral volume increase (= volume APM) due to undrained deformation (immediate settlement) decreases during consolidation due to the dissipation of excess pore pressures (Christian & Watt, 1972). Let α be the ratio of the lateral volume reduction to the consolidation settlement volume. Then

$$\alpha = \frac{Lateral\ Consolidation\ Volume}{Consolidation\ Settlement\ Volume\ (V_{cL})} \quad (1)$$

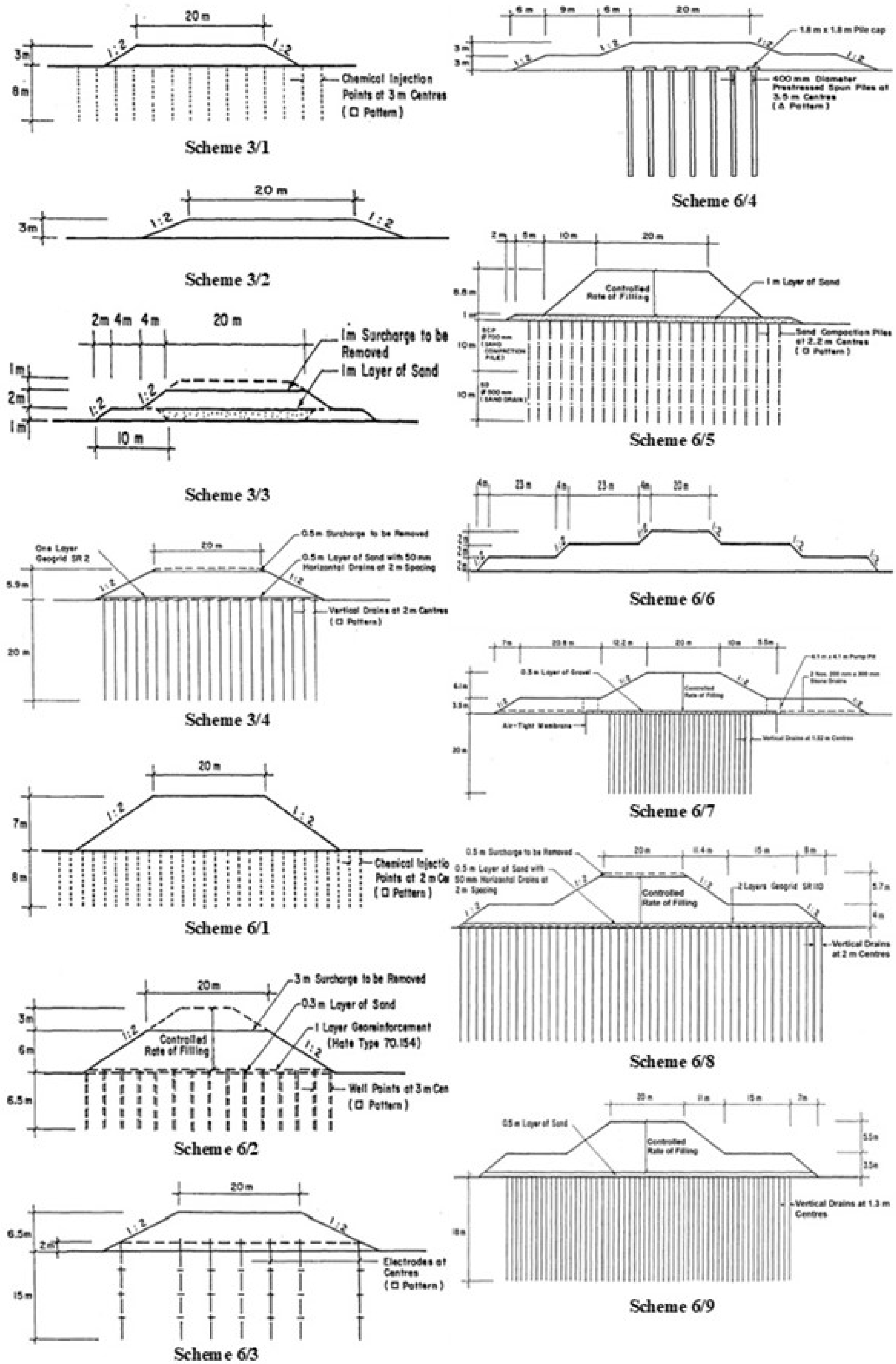
(a)



Elevation	Description	Liquid Limit w_L (%)	Plastic Limit w_P (%)	Natural Water Content w_n (%)	Plasticity Index	Grain Size (%) Clay	Silt	Sand	$\frac{C_c}{1+e_o}$	$\frac{C}{1+e_o}$	Preconsolidation Pressure P_c (kPa)
+2.5 mRL											
+0.5	Weathered Crust	108	55	70		42	57	1	.24	.04	110
-5.5	Very Soft Silty Clay with Decated Leaves and Roots	90	40	100	50	48	52	0	.48	.04	40
-15.3	Soft Silty Clay with Traces of Shell Fragments Occasionally Sand Lenses	80	30	60	50	40	60	0	.31	.04	60
-15.9	Peaty Soil										
-19.9	Sandy silt / clay with Organic Matters					22	43	35			
	Dense Medium to Coarse Sand with Gravels SPT N = 21 to 37										

Figure 9. (a) soil profile at Muar test embankment site; (b) schematic diagram of ground improvement scheme at the Muar flat site, Malaysia.

(b)


20 m
1:2
3m
8m
Chemical Injection Points at 3 m Centres (☐ Pattern)
Scheme 3/1
20 m
3m
Scheme 3/2
2m 4m 4m 20 m
1m Surcharge to be Removed
1m Layer of Sand
1m
2m
10 m
Scheme 3/3
0.5m Surcharge to be Removed
One Layer Geogrid SR 2
0.5m Layer of Sand with 50 mm Horizontal Drains at 2 m Spacing
5.9m
Vertical Drains at 2m Centres (☐ Pattern)
20 m
Scheme 3/4
7m
Chemical Injection Points at 2 m Ce (☐ Pattern)
Scheme 6/1
3 m Surcharge to be Removed
0.3m Layer of Sand
1 Layer Georeinforcement (Hate Type 70.154)
Controlled Rate of Filling
6m
6.5m
Well Points at 3 m Cen (☐ Pattern)
Scheme 6/2
6.5m
2m
15m
Electrodes at Centres (☐ Pattern)
Scheme 6/3
6 m 9m 6 m 20 m
1.8 m x 1.8 m Pile cap
400 mm Diameter Prestressed Spun Piles at 3.5 m Centres (Δ Pattern)
Scheme 6/4
2m 8m 10m 20 m
1 m Layer of Sand
Controlled Rate of Filling
8.8m
Sand Compaction Piles at 2.2 m Centres (☐ Pattern)
10m
Scheme 6/5
4m 23 m 4m 23 m 4m 20 m
Scheme 6/6
7m 20.8 m 12.2 m 20 m 10m 5.5m
0.3m Layer of Gravel
Controlled Rate of Filling
Air-Tight Membrane
20 m
Scheme 6/7
0.5 m Surcharge to be Removed
0.5 m Layer of Sand with 50 mm Horizontal Drains at 2 m Spacing
20 m 11.4 m 15 m 8 m
Controlled Rate of Filling
2 Layers Geogrid SR 110
5.7m
4m
Vertical Drains at 2 m Centres
Scheme 6/8
20 m 11 m 15 m 7m
0.5 m Layer of Sand
Controlled Rate of Filling
5.5m
3.5m
Vertical Drains at 1.3 m Centres
18m
Scheme 6/9

In Figure 10, V_{hL} = the observed lateral volume change in the field from inclinometer measurements. The volume V_{hL} measured at the end of loading, is the resultant of lateral volume increase due to undrained deformation (V_{iL} due to immediate settlement) and the lateral volume reduction during consolidation αV_{cL}). If H_j and H_{j+1} correspond to the height of loading at two stages j and j+1, the α can be determined from

$$\alpha = \frac{\left[\frac{(V_{hL})_{j+1}}{H_{j+1}} - \frac{(V_{hL})_j}{H_j}\right]}{\left[\frac{(V_{vL})_j}{H_j} - \frac{(V_{vL})_{j+1}}{H_{j+1}}\right]} \tag{2}$$

Similarly at two time stages t_j and t_{j+1}, the α factor during the consolidation stage (Figure 10) can be obtained from:

$$\alpha = \frac{\left[\frac{(V_{hL})_{j+1}}{t^{\gamma}_{j+1}} - \frac{(V_{hL})_j}{t^{\gamma}_j}\right]}{\left[\frac{(V_{vC})_j}{t^{\gamma}_j} - \frac{(V_{vC})_{j+1}}{t^{\gamma}_{j+1}}\right]} \tag{3}$$

Where V_{vc} and V_{hc} correspond to the settlement volume and lateral volume during the consolidation stage.

Loganathan (1992) also defined a β factor during consolidation to describe the creep.

$$\beta = \frac{Lateral\ Creep\ Volume}{Creep\ Settlement\ Volume\ (V_{crC})} \tag{4}$$

V_{cC} during the loading stage can be calculated as

$$V_{cC} = \frac{\beta V_{vC} - V_{hC}}{\alpha + \beta} \tag{5}$$

Similarly, V_{crC} during the loading stage can be calculated from

$$V_{crC} = \frac{\alpha V_{vC} + V_{hC}}{\alpha + \beta} \tag{6}$$

Further details of these derivations can be found in Loganathan et al. (1993). In Figure 11 the creep settlement volume (V_{crC}) is denoted as volume EFAE. Similarly the lateral creep volume is represented as volume ARMSA.

The normalised settlement with maximum settlement at the surface is presented in Figures 12a and 12b. The actual settlement with time is presented in Figures 13a and 13b. The percentage contribution from each layer is also tabulated in Table 2. The empirical formulation for the 6 m and 3 m high embankments were respectively:

$$S = -0.02z^3 + 0.75z^2 - 14.11z + 102.83 \ \text{ with } r^2 = 0.98 \tag{7}$$

$$S = 0.02z^3 + 0.86z^2 - 15.02z + 103.74 \ \text{ with } r^2 = 0.97 \tag{8}$$

If the two sets of embankment data are combined then the combined relationship is

$$S = -0.02z^3 + 0.86z^2 - 14.68z + 103.38 \ \text{ with } r^2 = 0.97 \tag{9}$$

The normalised lateral deformations are presented in Figures 14a-c. Tavenas & Leroueil (1980) suggested that irrespective of the depth at which it occurs, the maximum lateral deformation developed during construction and consolidation can be expressed as a function of the settlement of the embankment. The α-values as calculated by Loganathan (1992) are given in Table 3. Also, the α-values determined for the Scheme 3/2 and 6/6 which are the control

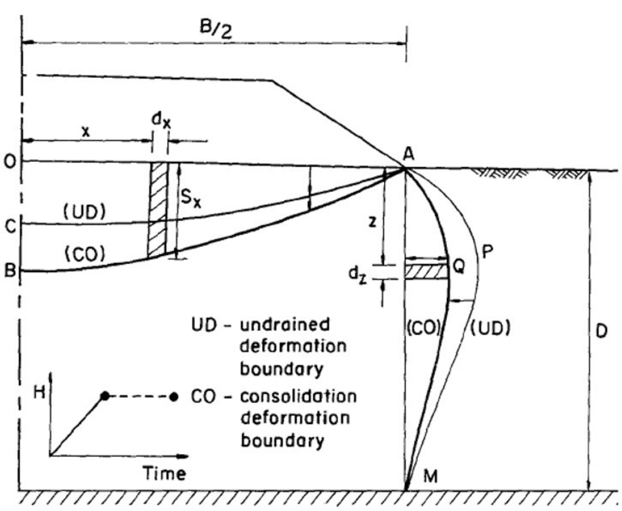


Figure 10. Deformation pattern of embankment foundation at end of loading stage.

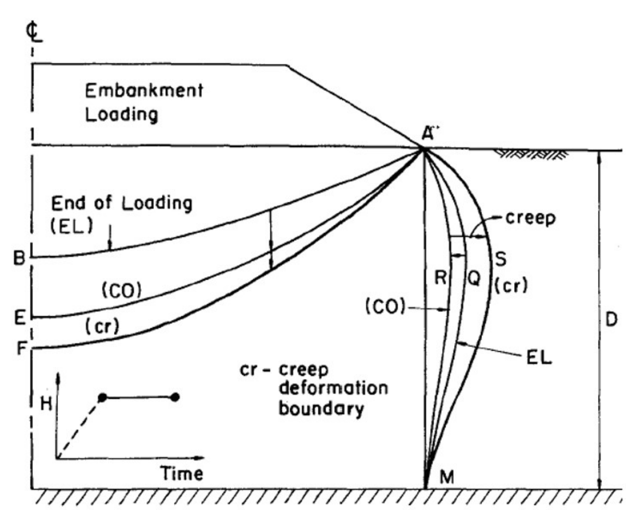


Figure 11. Deformation pattern of embankment foundation at end of consolidation stage.

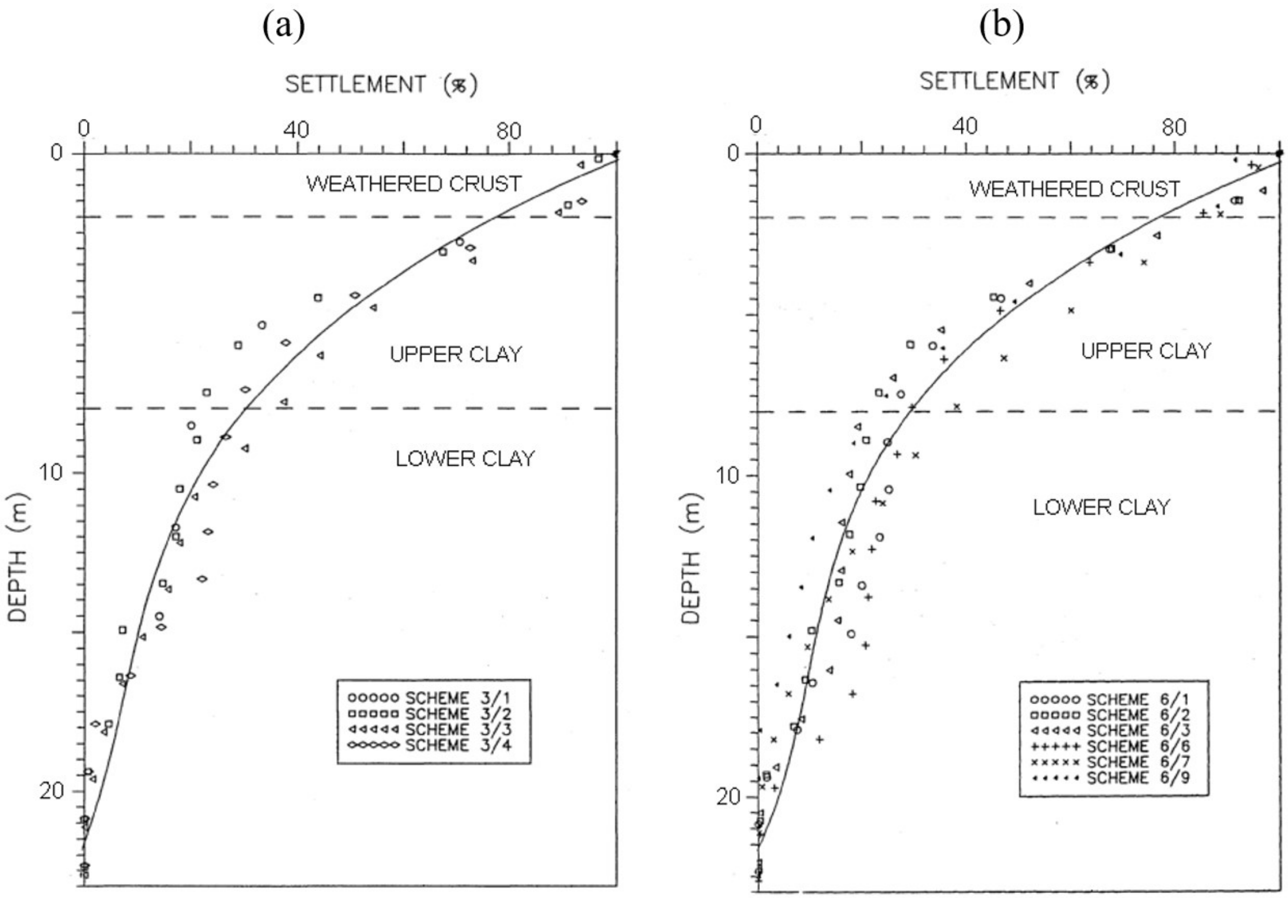


Figure 12. (a) variation of percentage settlement with depth for 3 m high dmbankments; (b) variation of percentage settlement with embankments depth for 6 m high.

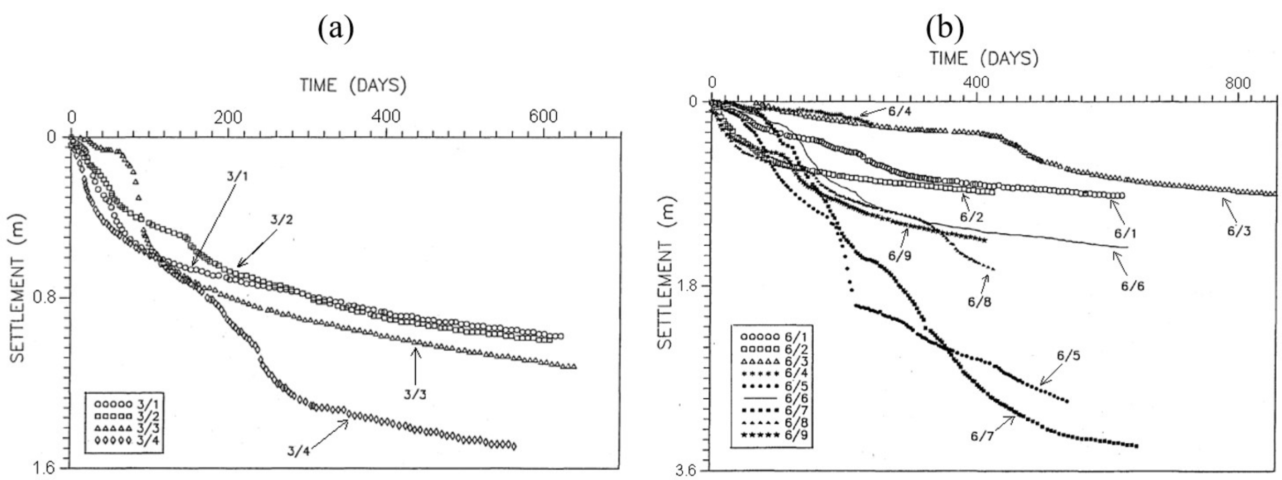


Figure 13. (a) comparison of maximum settlement profiles with time for 3 m-high embankments; (b) comparison of maximum settlement profiles with time for 6 m-high embankments.

embankments are shown in Figure 15. Then the variation of the consolidation settlement with immediate settlement for the other nine schemes is shown in Figure 16 during the loading stage. The corresponding data during the consolidation stage between the consolidation settlement and the creep settlement are given in Figure 17. Substantial creep settlement takes place as evidenced in the lateral movements as well as presented in Figure 18.

Table 2. Settlements (%) contributed by each layer for fifferently treated embankments.

Scheme	Percentage Settlement		
	Weathered Crust	Upper Clay	Lower Clay
3/1	21	57	22
3/2	20	61	19
3/3	15	54	31
3/4	12	51	37
6/1	15	58	27
6/2	14	64	22
6/3	13	68	19
6/6	16	55	29
6/7	11	51	38
6/9	14	64	22

Table 3. Ratio of Maximum lateral deformation to maximum settlement, α-value for all embankment schemes.

Scheme	Ground Improvement	α
3/1	Electro Chemical Injection	0.200
3/2	Control	0.240
3/3	Sand Sandwich Method	0.212
3/4	Preloading, Geogrid & Prefabricated Vertical Drain	0.210
6/1	Electro Chemical Injection	0.342
6/2	Well Point Preloading	0.173
6/3	Electro Osmosis	0.194
6/6	Control	0.240
6/7	Vacuum Preloading & Prefabricated Vertical drain	0.203
6/8	Preloading, Geogrid & Prefabricated Vertical Drain	0.275
6/9	Preloading & Vertical Drain	0.167

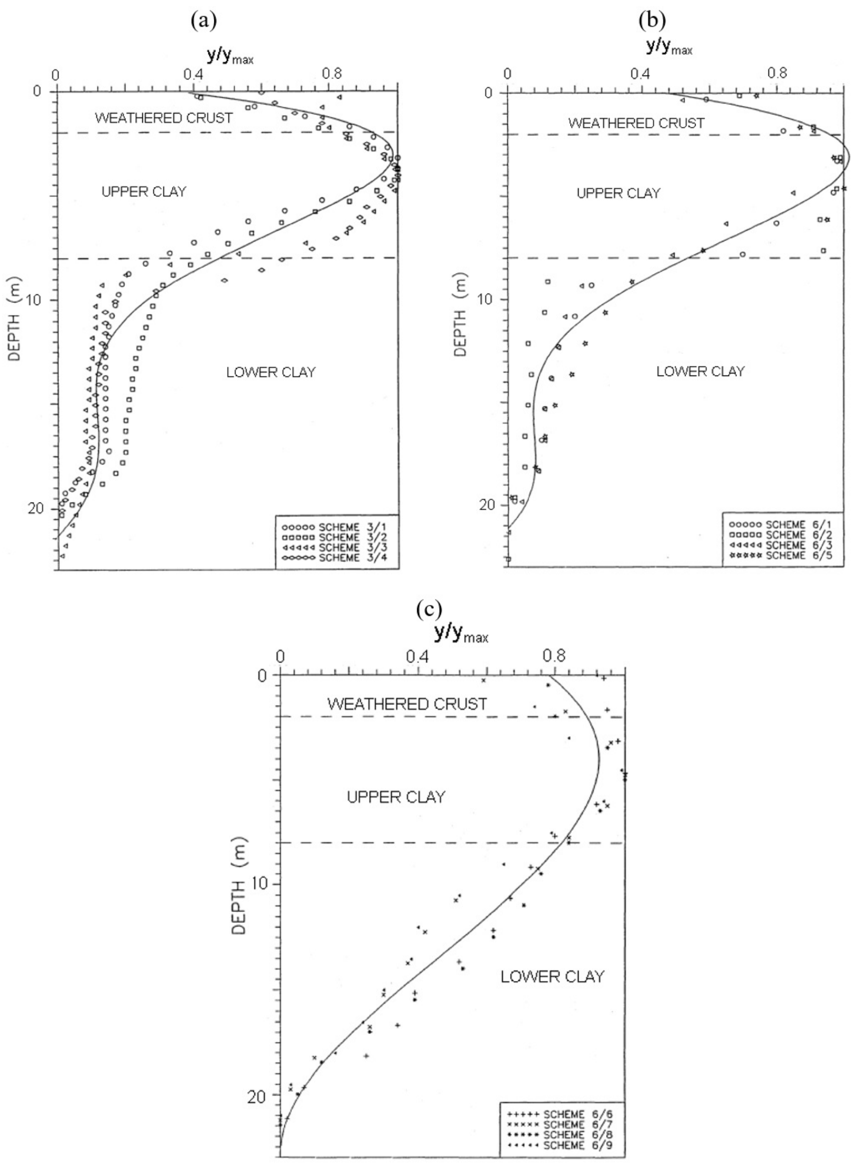


Figure 14. Variation of ratio of lateral deformation to maximum lateral deformation with depth for (a) 3 m-high embankments, (b) and (c) 6 m-high embankments.

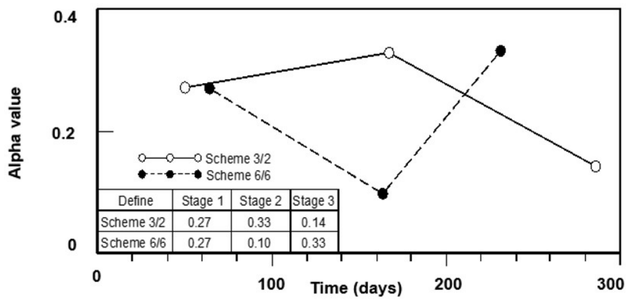


Figure 15. α-values for different stages of construction.

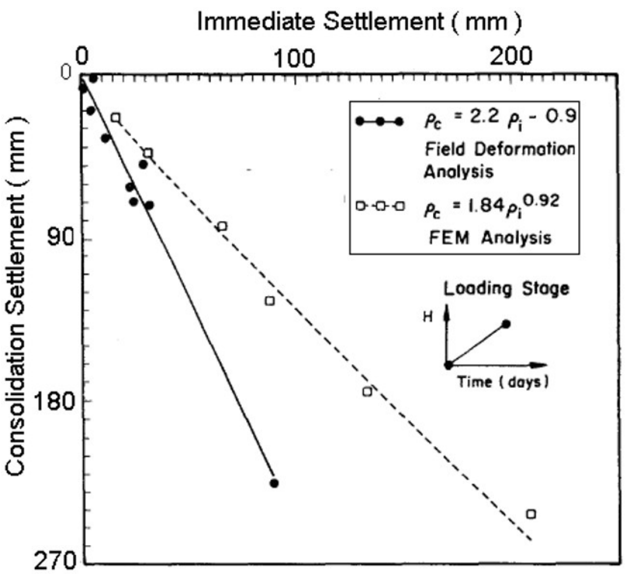


Figure 16. Variation of consolidation settlement with respect to immediate settlement.

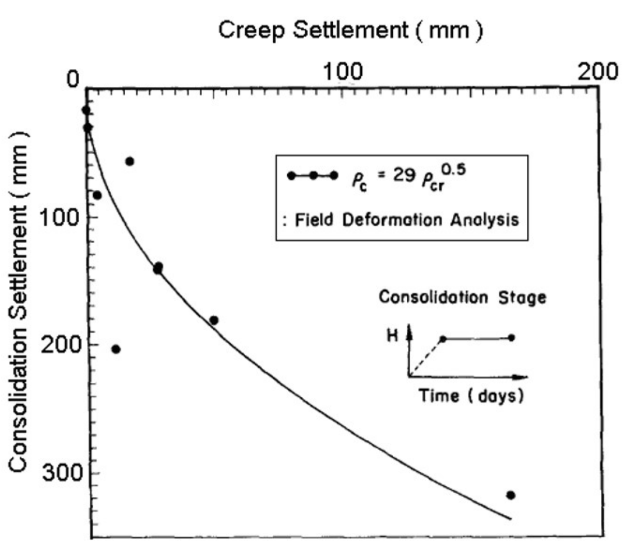


Figure 17. Variation of consolidation settlement with respect to creep settlement.

3. Test embankments at the Sbia site in Bangkok

The history of geotechnical investigations at this site included a test embankment with settlement measurement and without any form of drains and a test embankment conducted to failure, both were conducted in 1973 in the first phase of study. These results indicated that without any form of accelerated consolidation, the maximum fill height at the site can only be 3.4 m corresponding to a surcharge of 61 kPa. This is the failure height and a factor of safety of 1.5 is generally desired when PVD is used perhaps with a lower value of 1.38. The embankment raised to 2.7 m showed 0.48 m of settlement in 6 months of which 0.2 m is estimated as the immediate settlement and 0.28 m of consolidation settlement with in-situ values of coefficient of consolidation.

At the SBIA site in Nong Ngu Hao, the most extensive sand drain studies on test embankments were performed in 1983 (Moh et al., 1987) as part of the ground improvement scheme for the runway pavement and other sections of the taxiways and landside roads. Sand drains of minimum diameter 0.26 m was installed to a depth of 14.5 m by water jetting. The test program included three test areas, one with surcharge fill, the second with vacuum loading, and a third with ground water lowering. Test Section 1 was 40 m × 40 m in plan and sand drains were installed at 2 m spacing in a triangular pattern. The vacuum load was not successful as several leakages developed and finally the section was covered with a plastic shield. Test Section 3 was similar to Test Section 1, except that the spacing of the drain was increased to 2.4 m. Due to similar problem as in Section 1, the loading was not successful. The test Section 2 was slightly larger than test Section 1 and pre-loading of 60 kN/m^2 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 to be a reliable technique when compared to vacuum loading in accelerating the consolidation with sand drains. The field trial was not successful in the sense that: (i) there was a settlement of 0.4 m under a sand blanket of 0.7 m after a five-month period, and (ii) the settlement across the section was remarkably asymmetric. The observations indicate the possibility of hydraulic connections between the sand drains and the first sand layer located at 25 m depths with a piezometric drawdown of 120 kN/m^2. It appears sand wicks (as used at the Naval Dockyard site) recharged the piezometric drawdown in the clay layer; while the large diameter sand drains (as those used in the airport site in 1983) tend to form hydraulic connections with the underlying aquifer and caused additional settlements due to the piezometric drawdown.

From the previous trials, it become evident that the engineers in Bangkok were rather cautious of the potential use of vertical drains in the Bangkok plain and the client that is, the Airport Authority of Thailand, insisted that

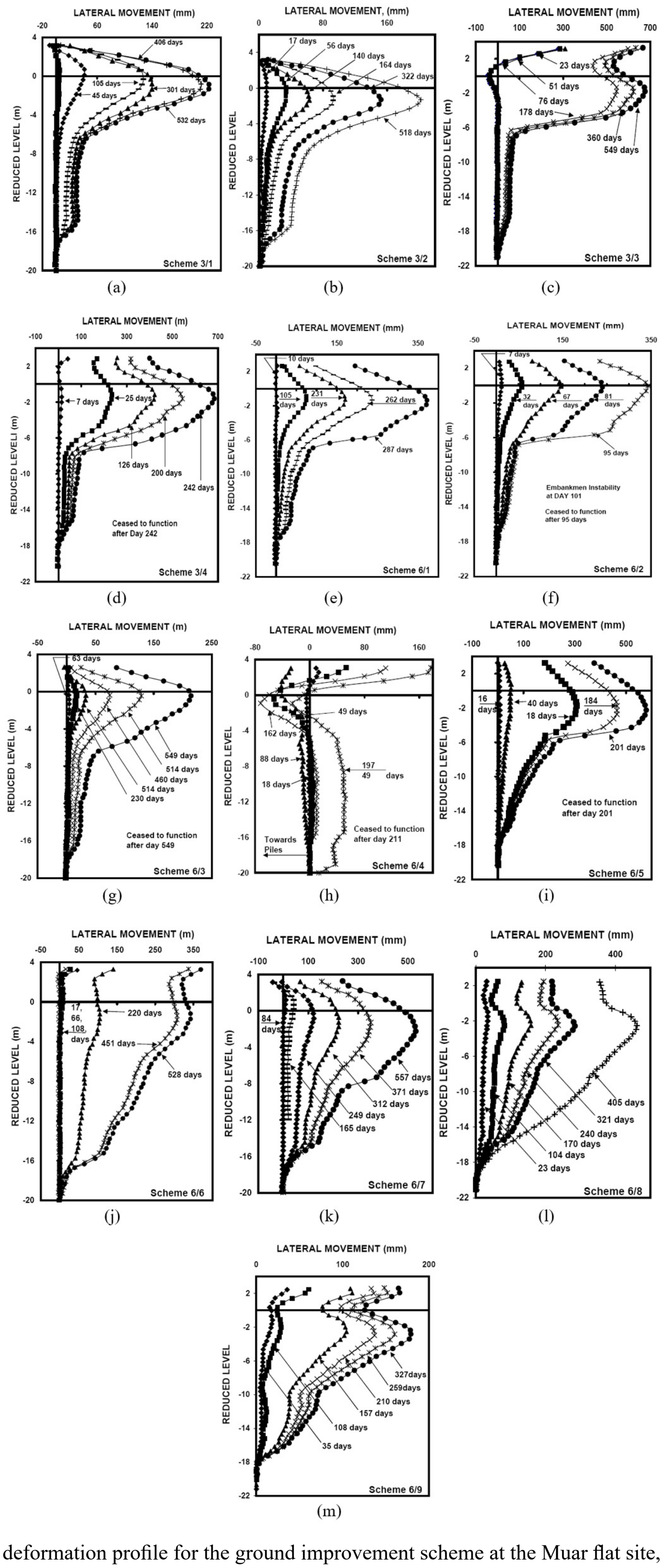


Figure 18. (a) to (m) lateral deformation profile for the ground improvement scheme at the Muar flat site, Malaysia.

the 1994-1995 study must indicate beyond all doubts that the majority of the settlement experienced in the trial embankment must be consolidation type of settlement to indicate the removal of water from the soft clay to improve its strength characteristics as well as to ensure that there is no possible hydraulic connections between the PVD used and the underlying sand strata which is experiencing substantial piezometric drawdown. These objectives are to be met by the estimation of the degree of consolidation, both from direct measurement of the settlements as well as from observation of the pore pressure dissipation in the field. Further, in-situ tests are conducted with vane apparatus to measure the in-situ strength increase with the water content reduction from the consolidation due to the use of PVD and surcharge. Additionally, the rate of settlement with time needs to be plotted to indicate that the final settlement rate is somewhat comparable to that, which one would consider acceptable at a rate similar to those experienced in secondary consolidation and not of higher values corresponding to hydraulic connections.

The plan dimensions of the embankments were the same as the earlier study. The locations of the test embankments and the cross-section of embankment TS3 with PVD are shown in Figures 19a-c respectively. These embankments were fully instrumented to measure the surface and subsurface settlements and pore pressures, lateral movements and heave.

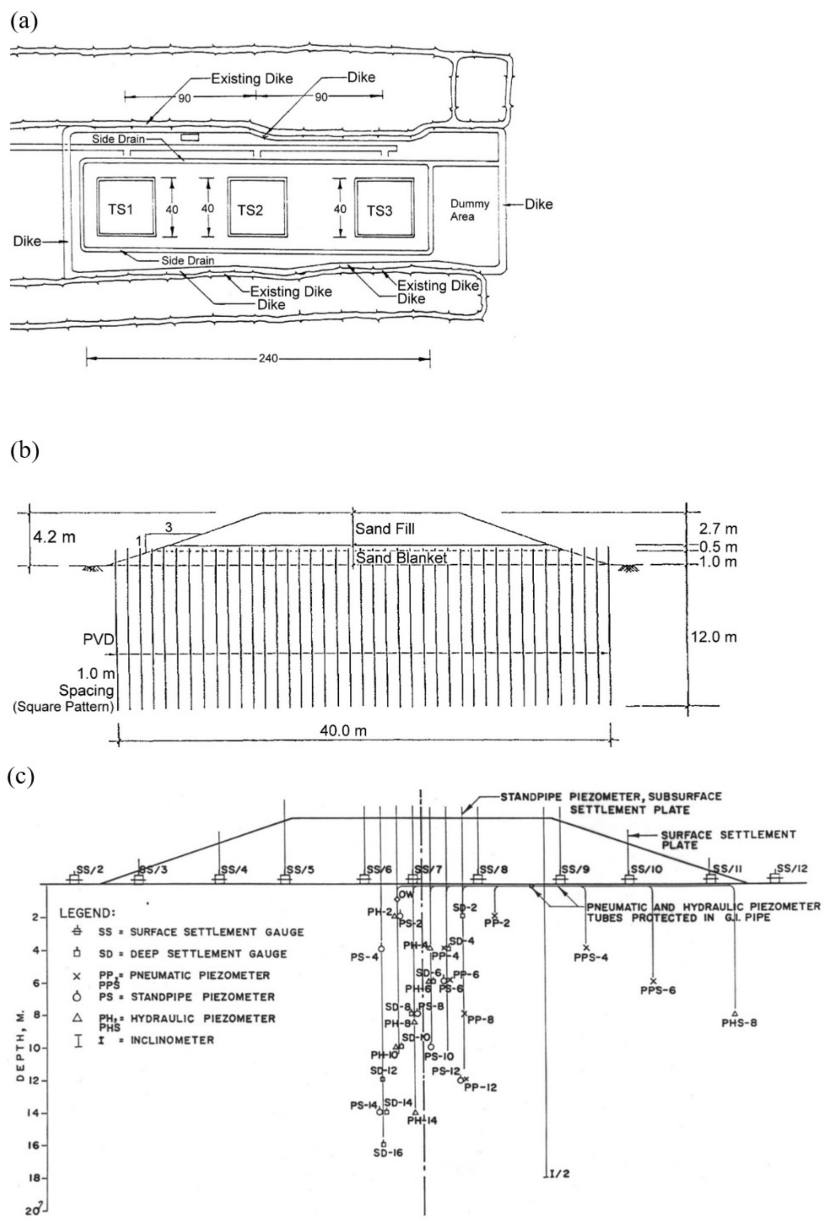


Figure 19. (a) site plan of test embankments TS1, TS2 and TS3 at SBIA site; (b) test section TS3 showing PVD at SBIA site; (c) section view of the test embankment showing the position of instruments at the Second Bangkok International Airport (SBIA).

PVD were installed to 12 m depth and the spacing was 1.5, 1.2 and 1.0 m in the three embankments TS1, TS2 and TS3, respectively.

All three test embankments performed more or less in the same manner and as such detail discussion will only be based on one (Test embankment TS 3 with PVD spacing at 1 m interval). For this embankment the settlement profile with depth, the lateral movement, and the pore pressure plots at various times are shown in Figures 20-25. In Figure 26, settlements were also independently computed from actual pore pressure dissipation. In Figure 26, the dotted curve ABC represents the actual piezometric profile with draw down as observed in September 1994 prior to the construction of the embankment. The full line curve DEF corresponds to the pore pressure profile after the full height of the embankment is reached with a surcharge of 75 kPa and prior to any pore water pressure dissipation. The end of construction pore pressure profile is also shown. Similarly the pore pressure profiles in June 95 and in February 1996 are shown. The final pore pressure after the dissipation of the excess pore pressure and the recharged hydrostatic profile is MNPQ (NPQ is the assumed final recharged pore pressure profile, where there are no data points). Settlements were directly computed from these pore pressure dissipation curves.

The degrees of consolidation computed from the pore pressure dissipations are illustrated in Figure 27 and compared with the degree of consolidation as computed from settlement measurements. In Figure 27, the ordinate axis U_p refers to the degree of consolidation as computed from the pore pressure dissipation, while the abscissa axis refers to the degree of consolidation U_s as computed from

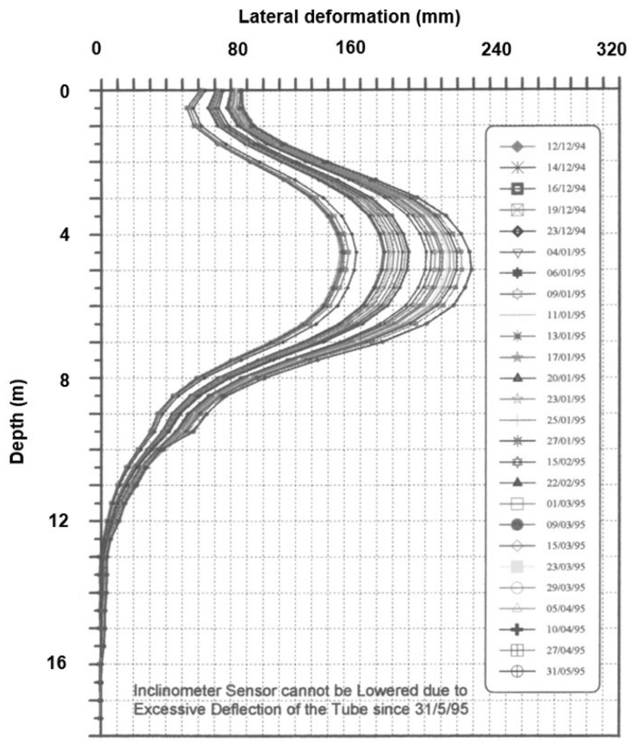


Figure 21. Lateral deformation with depth below embankment TS3 at the SBIA site in Bangkok.

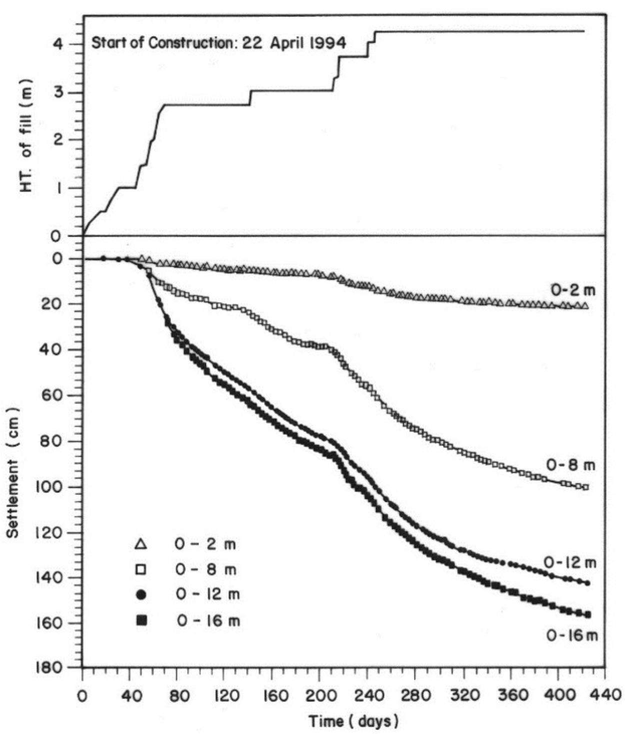


Figure 20. Settlement-time plot of the embankment with PVD at the SBIA site in Bangkok.

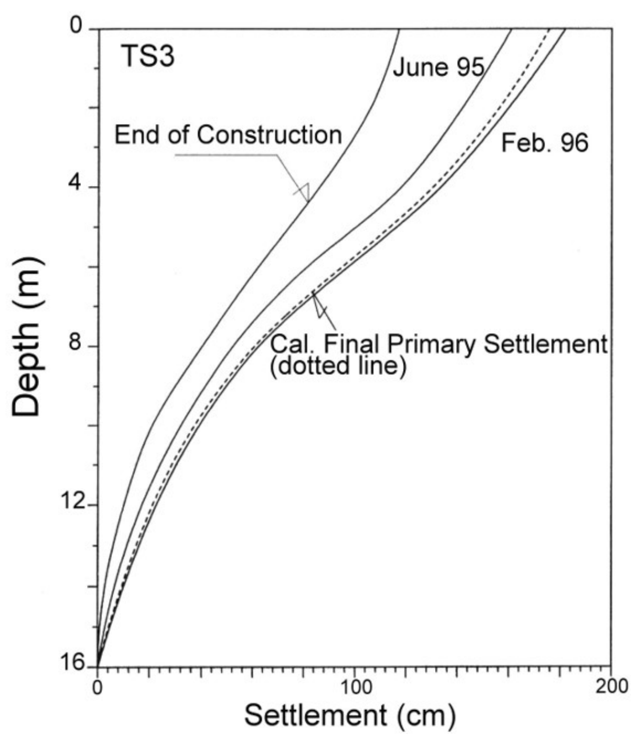


Figure 22. Settlement plot of test embankment with PVD at SBIA site.

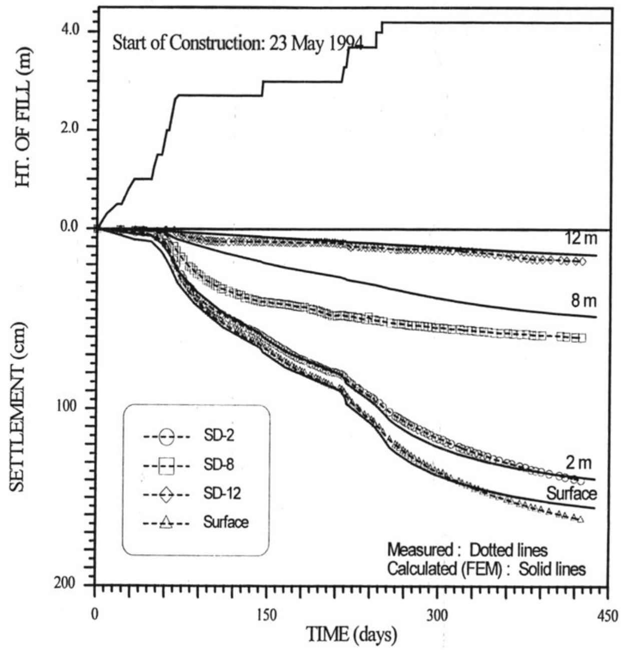


Figure 23. Measured and computed settlement with different depth at the SBIA site in Bangkok.

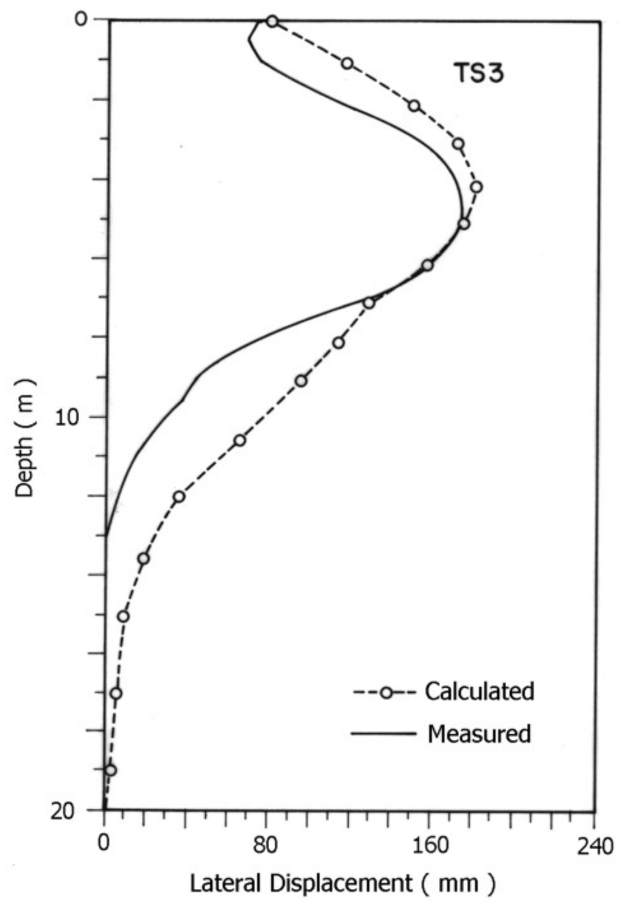


Figure 24. Measured and computed lateral movement of the embankment at the SBIA site in Bangkok.

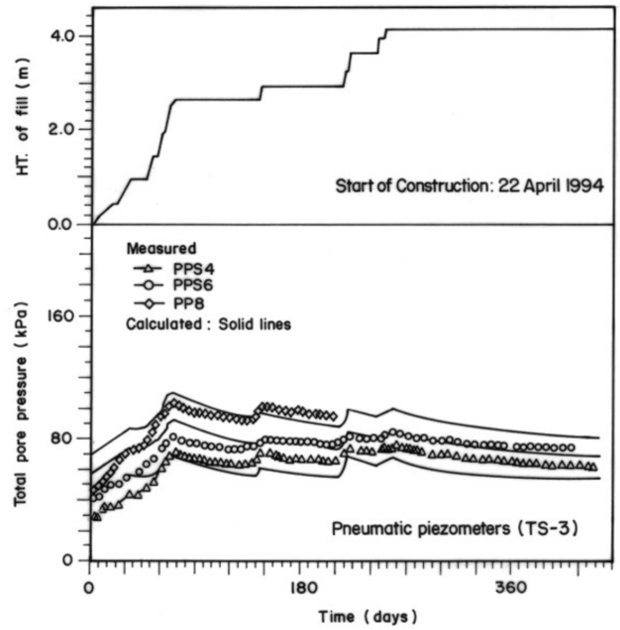


Figure 25. Measured and computed pore pressure dissipation of the embankment at the SBIA site in Bangkok.

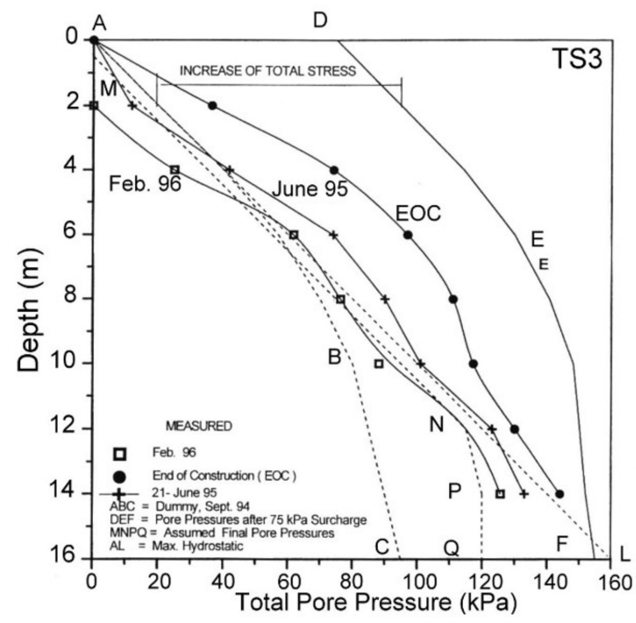


Figure 26. Pore pressure profile of test embankment with PVD at SBIA site.

the settlement measurements. With due allowance for a small percentage of secondary settlement and creep, the two degree of consolidations seem to agree well as they are close to the 45 degree line. The settlement due to the lateral movements was less than 10% as estimated by the method of Loganathan et al. (1993). The immediate settlement computed from the lateral movements as adopted by a method in which the balancing of volume (Loganathan et al., 1993) was within ten percent of the measured vertical settlement. The rate of settlement and the rate of lateral movements were plotted in Figures 28 and 29, and are found to decay with time. Also the settlement log time plots in Figure 30 for the three test embankments were found to be approaching a constant slope. An attempt was made to define the 100% primary consolidation time using Casagrande type of settlement versus log-time plots. The points P and Q (shown in Figure 30) correspond to the 100% primary consolidation for the Test Section TS3 and TS1 respectively. The data for TS2 is not shown, as it will crowd closely with the data from the other two embankments. The final portions of the test data for the two embankments seem to approach the secondary consolidation part as computed from the Casagrande settlement versus log-time plots. These results further confirmed that the PVD did not cause any hydraulic connection with the lower aquifers and the measured final settlement is of the same order as the secondary settlement.

Finally, the increase in the shear strength with time after consolidation in the field is measured with the vane apparatus as plotted in Figure 31.

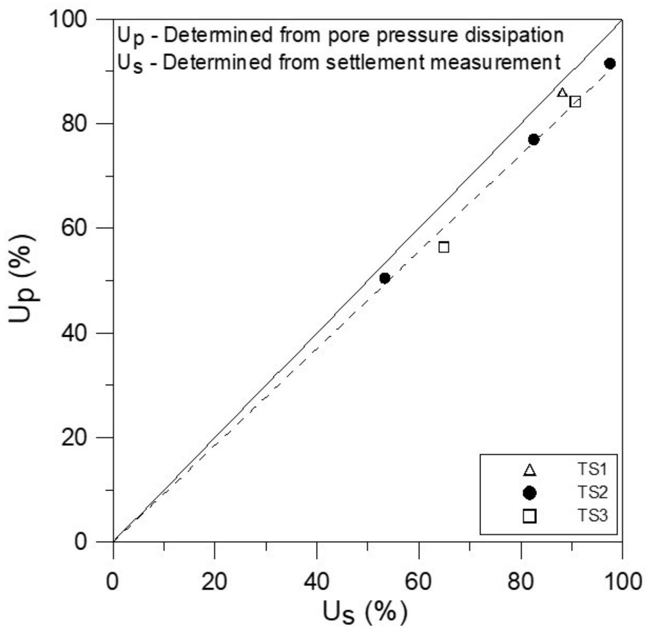


Figure 27. Degree of consolidation computed from pore pressure dissipation and settlement measurements at SBIA site.

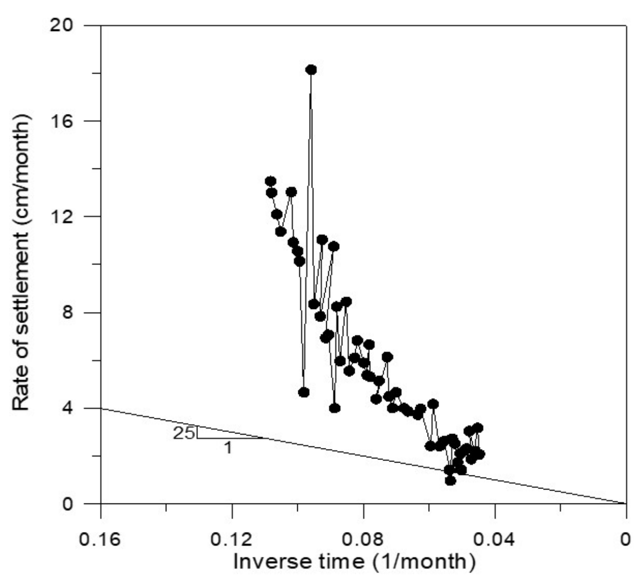


Figure 28. Rate of settlement versus inverse time plot at SBIA site.

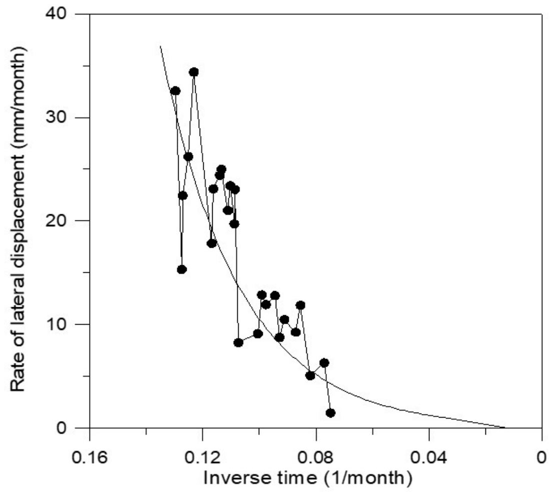


Figure 29. Rate of lateral movement versus inverse time plot at SBIA site.

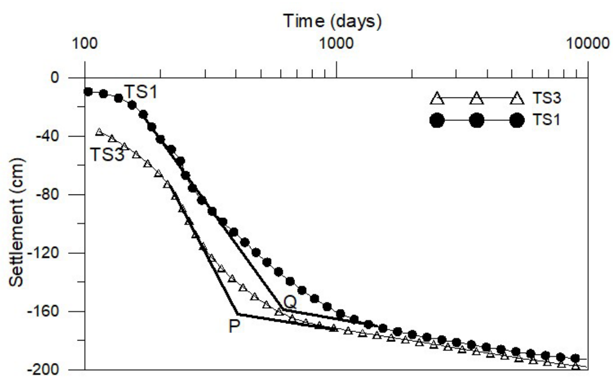


Figure 30. Settlements versus log-time plot for embankments TS1 and TS3 at SBIA site.

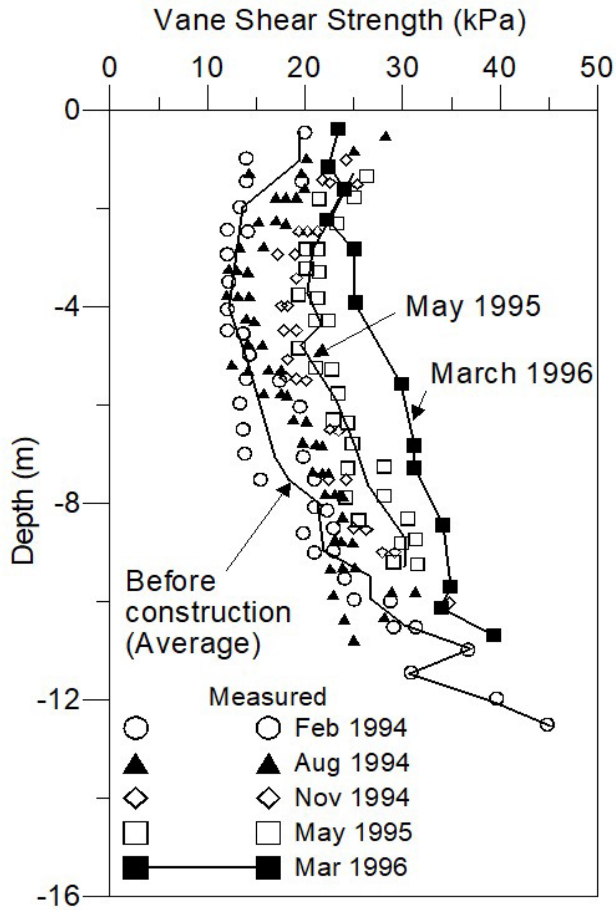


Figure 31. Field vane shear strength as measured in embankment TS3 at SBIA site.

4. Concluding remarks

This lecture is devoted to a study of the role of test embankments as a site investigation, design and construction control aspects related to ground improvement works in soft clays. Test embankments in Bangkok, Thailand and Muar site in Malaysia are used as case studies. The work is more on practical aspects related to the stability and deformation characteristics. Even though research work on this subject has been there now for more than five decades, yet the Case A type of Prediction of the stability and settlement characteristics is still a challenging task.

From the test embankment studies carried out in Bangkok with and without ground improvement, the major experience is with embankments of sand. Without any ground improvement, the failure height of these embankments is very modest and is in the range of 2.5 to 3.5 m. Vane strengths were found to be adequate to determine the stability of these embankments with a total stress analysis and the Bjerrum's correction factor as based on plasticity index is found to be essential. The Authors have no experience in using the subsequent correction as proposed by Aas.

Data from three tests embankments fully instrumented revealed that when vertical drains are used in the Bangkok Plain the piezometric drawdown which naturally exist in the Plain due to deep well pumping is virtually erased in the upper clay layer. Also the presence of sand and silt seams tend to assist lateral drainage and therefore test embankments with and without drains be separated substantially not to have such interference effects. The possibility of such effects also remains in the soft clays of Southeast Queensland and elsewhere. The presence of sand and silt seams and the existence of piezometric drawdown had made it difficult for the vacuum drainage to be implemented successfully. Recent modifications and improvements in the sealing methods together with the use of Bentonite type of cut off walls were not included in the studies made here. Test embankments built on soft clay with pre-fabricated vertical drains have performed successfully in accelerating the consolidation settlement when the PVD spacing is properly designed taking care of smear effect and well resistance as proposed by Hansbo and others. For the soft Bangkok clay this spacing is about 1.5 m. The immediate settlement observed was generally of the order of ten percent. The Asaoka method and the Field Deformation Analysis were also performed to confirm the magnitude of consolidation settlement and immediate settlement respectively. Settlement computations from pore pressure dissipation and direct settlement measurements are found to agree well and the magnitude of long term secondary settlement is also computed from the field data.

The test embankment studies at the Muar site indicated the importance of the fill strength in the stability characteristics when well compacted residual soil is used. Also the Field Deformation Analysis was successful in separating the consolidation settlement, the immediate settlement and the long term creep settlements. The normalised settlement profile and the normalised lateral movement profiles for several ground improvement schemes were found to be similar in shape. The use of sand compaction piles and pre-stressed piles were found to be successful in minimising the lateral deformation at the toe of the embankments.

In all the test embankment studies, prediction of the observed behaviour was found to be possible with the use of CRISP computer program and soft clay models of the type developed at Cambridge University.

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Declaration of interest

The author declares that are no conflicting interests that could inappropriately bias this paper.

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