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CASE HISTORIES OF GROUND IMPROVEMENT SCHEMES IN ROADWORKS RELEVANT TO RAILROAD EMBANKMENTS

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ABSTRACT

Australia's first trains designed 200 km/h are being delivered to Western Australia for the TransWA country rail system (Hammond, 2003). The construction of a railroad structure in soft clays in itself is a significant cost, and many design problems are caused by the requirements that the railroad embankment be visible and noise controlled. In this paper case histories of the use of ground improvement schemes to improve the behaviour of soft clays in Malaysia, Thailand (Bangkok) and Queensland are presented. Of the many possible ground improvement techniques, embankments with prefabricated vertical drains were found to be effective in improving the soft clay behaviour as revealed from the experience in Malaysia, Thailand and Queensland. Stone columns and sand compaction piles are only used in limited cases, with test embankments and further studies needed to refine the techniques and to make them more appropriate. According to the European experience (in particular those in Scandinavia) and Japan, the use of shallow and deep stabilization are emerging as the most commonly used techniques to be adopted in ground improvement works related to railroad embankments. Laboratory studies and field performance data related to the use of lime and cement stabilization are also included in this paper.

INTRODUCTION

High-speed railroads have been identified as the alternative form of transport that will be seen in the future for medium distance travel, for example, the Shinkansen Rail Link of Japan (or "bullet" train), the Channel Tunnel Railway in the United Kingdom, the France TGV Rail system, and the Inter City Express high speed train in Germany and Switzerland. The School of Engineering, Griffith University Gold Coast Campus is currently engaged in a major study related to soil property characterization in Southeast Queensland as related to the construction and maintenance of railroads, motorways and expressways. Particular attention is confined to terrains with problematic soils such as very soft clays with normal and high sensitivity, expansive and reactive clays which exhibit high swell -shrinkage characteristics, sordic and erosive soils, and acid sulphate soils. This paper is concerned with very soft marine, deltaic, alluvial and estuarine clays and in particular the difficulties encountered with embankments which support roads, expressways and motorways. The Queensland Department of Main Roads has already done excellent work in carrying out laboratory and full-scale field tests with respect to embankment design in soft clays with and without ground improvement techniques (Wijeyakulasuriya *et al* (1999)). Embankments treated with preloading and prefabricated vertical drains (PVD), or stone columns are the techniques so far tried in Southeast Queensland. Extensive studies on the use of a variety of ground improvement schemes were made in Southeast Asia and in particular in Malaysia and Thailand on the design of road embankments in soft clay deposits. In particular trial embankments were tested in the Muar clay deposit in Malaysia and these data were studied in great detail by Prof. Balasubramaniam and his team of researchers at the Asian Institute of Technology and now in Griffith University. Some aspects of this work will be briefly presented here and its potential relevance to the Queensland conditions will be discussed.

SUB-SOIL PROFILES

The subsoil profiles at the Muar flat site in Malaysia, at the second international airport site in Bangkok and in the Southern Queensland are presented in Figure 1. The moisture content in all the three locations are of comparable values, while the estuarine clays in Queensland (Figure 1(c)) seem to be more sensitive and also possibly exhibit higher secondary consolidation.

TRIAL EMBANKMENTS AT THE MUAR SITE IN MALAYSIA

Fourteen trial sections (as shown in Figure 2) were built in one stretch to explore the potential ground improvement schemes which are most useful in road embankments works. All trial sections were fully instrumented to measure vertical settlements. Lateral movements and pore water pressures were measured using settlement markers, settlement gauges, pneumatic piezometers, and inclinometers. The settlement and lateral movement characteristics of these embankments are found to be similar in Figure 3. Of these techniques, the embankments with preloading and vertical drains were analysed in great details. Some of these results will be presented here.

EMBANKMENTS WITH PRELOADING AND PVD

The embankment with vertical drains (Scheme 6/9) was analysed by Ratnayake (1993). This analysis was done in two parts, to evaluate the performance of a single drain and the composite performance. This analysis revealed for the single drain, the 90 percent consolidation time as 2500 days without the vertical drains and only 250 days with the drains. For the case with vertical drains, the consolidation settlement derived from Barron's theory for the cases of a perfect drain, a drain with smear and a drain with smear and well resistance were found to be 1.200m, 1.199m and 1.199m respectively. From the analysis of the whole embankment after 400 days, the immediate and consolidation settlements at the ground surface and along the centre were measured. The corresponding maximum lateral deformations were observed as 0.21m and 0.39m respectively compared with computed deformations of 0.15m and 1.08m while the lateral movement at the embankment toe was 0.08m at the end of the loading period and increased to 0.21 m after 400 days. The observed and predicted behaviour of this embankment for settlement is presented in Figure 4. The measured settlements were less than the predicted values when ideal drain conditions were assumed. Possibly the effect of smear and un-dissipated pore pressures similar to the well effect would have caused the actual settlement to be smaller than the predicted settlement based on ideal conditions. The embankment was treated with preloading, geogrid and vertical drains (Scheme 3/4 and 6/8). The immediate and consolidation settlements were observed as 0.21mand 0.48m at 4m, and 0.5mand 1.09 m at 6m fill heights respectively. For the scheme 6/8, after about 400 days the calculated settlement beneath the centreline was 1.55m while the maximum heave was 250 mm and the maximum lateral deformation was 380 mm. The grids had negligible effect on the vertical settlement but it reduced the lateral movements.

PREFABRICATED VERTICAL DRAINS AT THE SECOND INTERNATIONAL AIRPORT, BANGKOK

Prefabricated vertical drains were used for the first time on three embankments at the airport site. The embankments labelled TS1, TS2 and TS3 were 40 m x 40m in plan dimensions with side slopes 3:1 (as shown in Figure 5(a) which shows the elevation of typical section with instrumentation). Initially no berm was used, however it was installed at a latter date. The PVD were selected from published information and economic considerations and the tests pertaining to the safety of the 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 three types of PVD were selected and installed at spacings of 1.5m, 1.2m and 1.0m respectively in three embankments labelled TS3, TS2 and TS1. The PVD were installed in a square pattern up to a depth of 12m. 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 1.5m., then clayey sand was used to raise the embankment to 4.2m in stages (i.e., to 75 kPa of surcharge). During construction, Stage 1 loading was up to 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). Figure Illustrate the loading pattern for embankment TS3. The factor of safety without considering short-term condition of machinery load was 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. The basic considerations for optimal performance of the drain is that a 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. 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. 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. These results are presented in Figure 5(b) to Figure 5(f). The strength increase after the primary consolidation is shown in Figure 6 and indicated the positive influence in using the preloading technique with PVD.

The major problems with the use of preloading technique in the Bangkok Plain is the scarcity of sand for surcharge and the serious difficulty in moving such a large quantity of sand fills when the traffic conditions without such frequent truck movements is already bad enough. 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 led to the proposal to explore vacuum preloading techniques. Two trials made with vacuum preloading with sand drains and PVD indicated that the vacuum is difficult to maintain over a long period of time and the method gave poor results when compared to the use of the traditional surcharge for preloading.

QUEENSLAND EXPERIENCES

Wijeyakulasuriya *et al* (1999) presented the behaviour of three trial embankments founded on sensitive soft clays along the eastern coastal belt of Queensland. In Case Study A, the embankment at Mackay was built to 5m height by stage loading. The construction period was 6 months and the period of preload was 9 months. Case study B was at the Sunshine Motorway, to assess the feasibility of 2-staged construction within an overall construction period of 300 days. Case study C was at Coombabah Creek on the Gold Coast. The behaviour of these embankments is still being studied. In Case Study C, stone columns were used. In Case Study B, the section of the motorway traverses a swamp comprising very soft organic marine silty clay ranging in thickness from 4-10 m and underlain by sandy deposits. The height of the embankment was 2.8 m and was reinforced with high strength geo-synthetic at the embankment and foundation interface.

The embankment at the Sunshine Coast had a top width of 17 m on nominally 1V:2H batters. The berms were 1m high with a width of 5m on one side and 8m on the other side. The trial embankment comprised three 20 m sections, with the end sections installed with wick drains (Section A, 1m spacing and Section C, 2m spacing) and the middle section on undisturbed virgin ground as a control section (Section B). The settlements of the three sections shown in Figure 7 indicate that the wick drains have not accelerated settlements to any great extent. However the settlements in all sections were in excess of 1.5m. These measurements are now being further analysed.

The trial embankment at Coombabah traversed a swamp with soft clay up to 13 m. Here again the trial embankment consisted of 3 sections. Section A had stone columns at 1m spacing and Section C had the stone column spacing as 3m. Section B was without any stone columns. The stone columns extended to 16 m. The height of the trial embankment was 2m with a top width of 12 m on 1V: 2 H batters. The stone columns were installed by the jetting process as used in vibro-flotation. The stone columns too were not found to be effective in reducing the settlements (as shown in Figure 8).

STONE COLUMNS

Stone columns are similar to the use of piled foundations, but the stone columns can be readily constructed in-situ using gravel material. Stone columns were used in Malaysia especially for expressway interchanges which involve high embankments. In some countries, a combination of PVD with surcharge, stone columns and sand compaction piles were tried in trial sections. These techniques are still to be tried and developed on the basis of full-scale field tests in a manner similar to the thorough check undergone for PVD and other types of drains for foul proofing.

SHALLOW AND DEEP CHEMICAL MIXING

Shallow and 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 conducted at the Asian Institute of Technology (AIT) has revealed that both lime and cement are very effective in substantially increasing the strength of soft clays and also in reducing their compressibility characteristics. Unconfined compression tests, consolidation tests and triaxial tests were performed in the soft Bangkok clay and similar studies are now in progress at the Griffith University on the marine and estuarine clays of Queensland.

The strength and compressibility characteristics of treated and untreated samples are examined in the unconfined compression tests, oedometer tests and triaxial tests (as shown in Figure 9(a) to 9(d)). Figure 9(a) illustrates the treated and untreated behaviour with various additives (fly ash, lime and cement). The cement treated samples have the highest unconfined compressive strength. In the oedometer tests, 10%

cement is found to suppress the volumetric compressibility substantially and quasi pre-consolidation pressure in the range of 2000kPa can be achieved (as shown in Figure 9(b)). The consolidation characteristics of soft clay are improved effectively by cement additives. At 10% cement content, the consolidation properties improve substantially. These results are consistent with the undrained and drained triaxial compression behaviour (as shown in Figure 9(c) and Figure 9(d)). The strength and deformation characteristics of soft Bangkok clay treated with different types of additives have been examined.

A successful case study in Bangkok with deep mixing method (DMM) is presented. Severe settlement and stability problems characterize the problems of the 55 km Bangna-Bangpakong Highway in Thailand (Bergado *et al.*, 1999). To rehabilitate this major arterial road, the deep mixing method (DMM) with soil-cement using ordinary Portland cement has been utilized for foundation improvement. Field monitoring instruments were installed in both the main road (MR) and frontage road (FR) embankments. Plan and section views of the embankment at km 29 + 992 are illustrated in Figure 10(a). In the field, the unconfined compressive strength of the cement piles was specified at 600 kPa. Analyses were performed regarding the bearing capacity, total settlements and their rates, and stability analyses. The predicted vertical and horizontal deformations were compared and generally agreed with the corresponding values observed in the field. The settlement analyses were carried out for different values of cement pile length and spacing. The Young's Modulus of cement piles was assumed to be 100 times the undrained shear strength. The permeability ratio of the surrounding soil and cement piles was taken as 20. For the portion from km 28+000 to km 30+950 at an embankment height of 2.5 m and a cement pile length of 16 m, the calculated settlements for different pile spacing are plotted in Figure 10(b), and the settlement reduction ratio was estimated as 0.46. The surface settlement records of the MR and FR are plotted in Figure 10(c). In general, the settlement magnitudes between and on the cement piles are similar, indicating equal strain.

CONCLUSIONS

Ground improvement techniques initially developed for roads, expressways and other infra-structure developments are now widely used in railroads and in particular for embankments on soft clays supporting rail tracks. The paper summarizes successful ground improvement techniques used in Thailand, Malaysia and Queensland in Australia. Embankments with surcharge and vertical drains were found to be successful in most instances when used in road works, expressways and motorways. Shallow and deep stabilization with cement and lime additives are emerging as popular ground improvement techniques with soft clays in Japan, Scandinavia, and USA among other countries.

ACKNOWLEDGEMENTS

The authors, and in particular the third author, are most grateful to Prof. D.T. Bergado, Prof. B. Indraratne, and numerous M.Eng and DEng Students who worked with the third author at AIT. Special mention is made to Drs. N. Loganathan, P. Wedage, K. Uddin and A. H. M. Kamaruzzaman.

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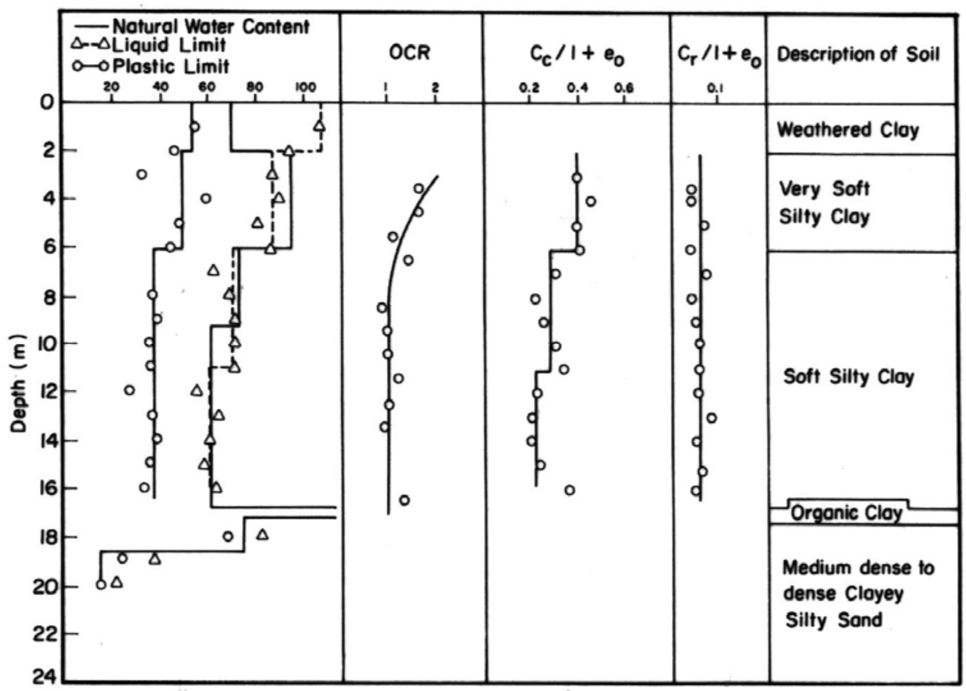

Figure 1(a) Geotechnical properties of Muar clay.

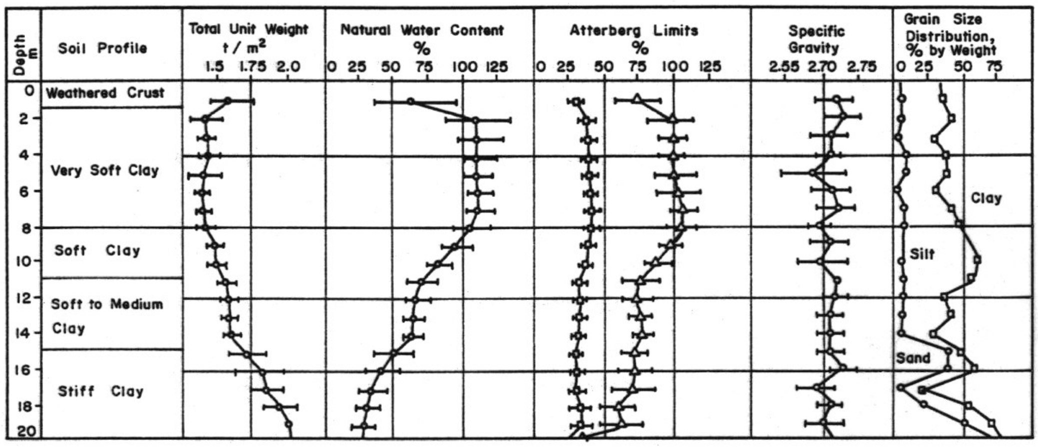

Figure 1(b): Typical soil profile in the Bangkok plain.

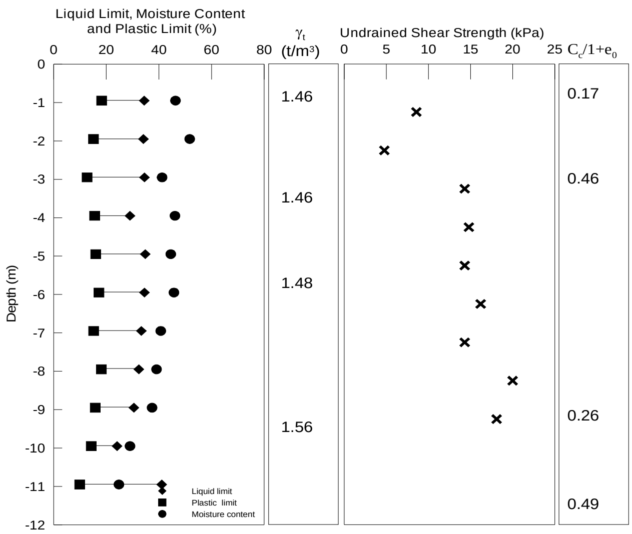

Figure 1(c) Geotechnical properties of clay in Gold Coast Highway (based on Wijeyakulasuriya *et al* (1999)).

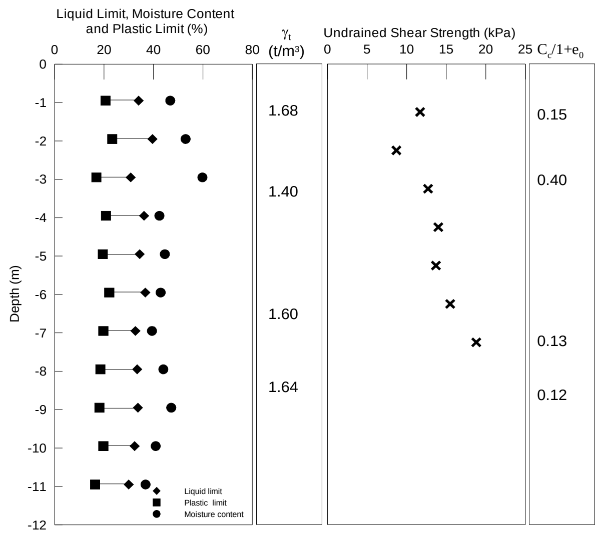

Figure 1(d) Geotechnical properties of clay in Sunshine motorway (based on Wijeyakulasuriya *et al* (1999)).

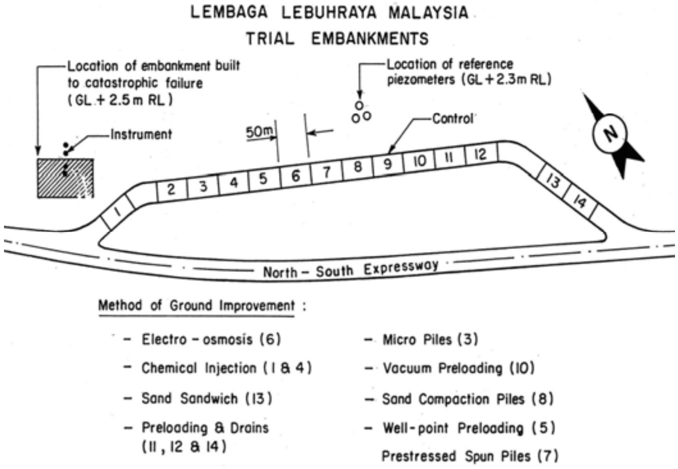

Figure 2 Layout of Muar trial embankments.

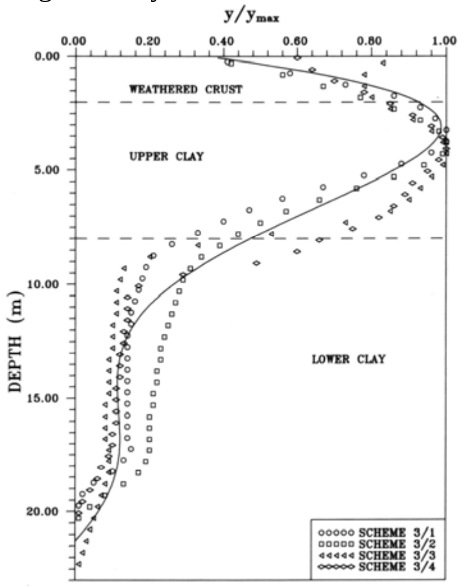

Figure 3(a) Variation of ratio of lateral deformation to maximum lateral deformation with depth for 3m high embankments.

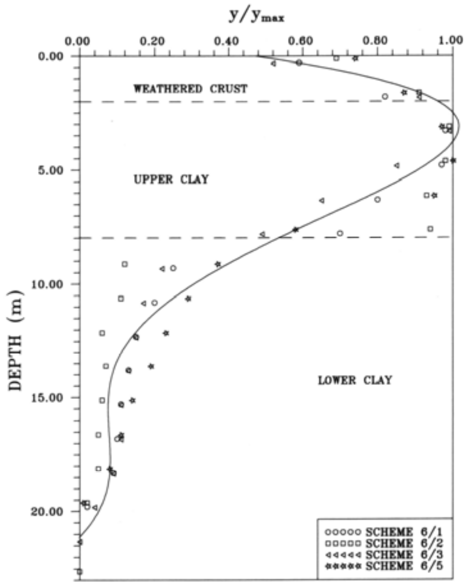

Figure 3(b) Variation of ratio of lateral deformation to maximum lateral deformation with depth for 6m high embankments.

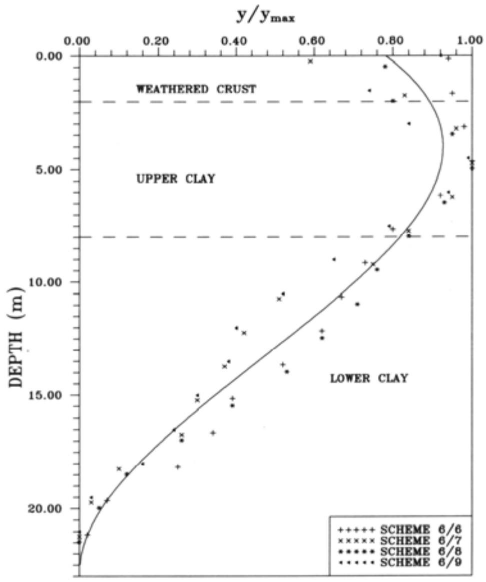

Figure 3(c) Variation of ratio of lateral deformation to maximum lateral deformation with depth for 6m high embankments.

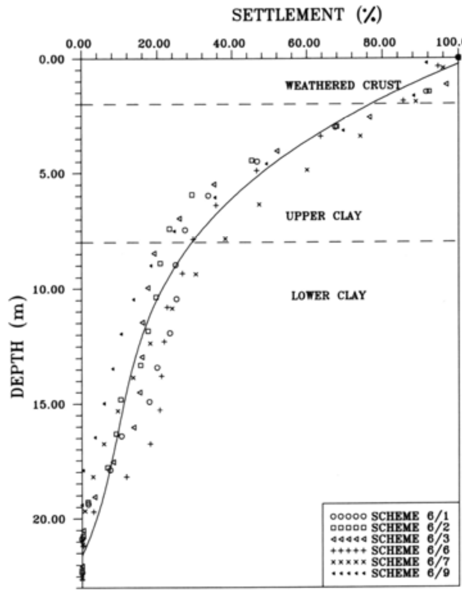

Figure 3(c) Variation of percentage settlement with depth for 6m high embankments.

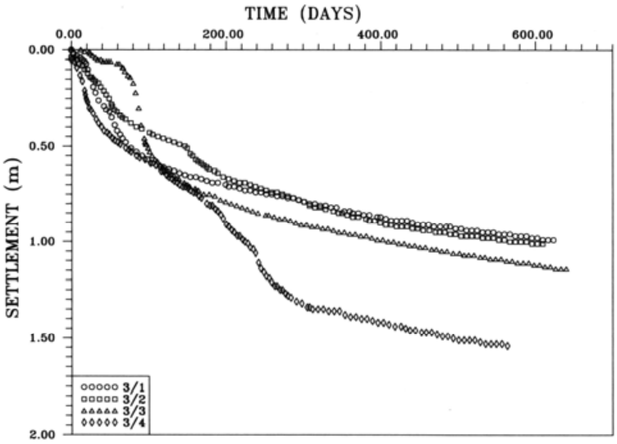

Figure 3(e) Comparison of maximum settlement profiles with time for 3m high embankments

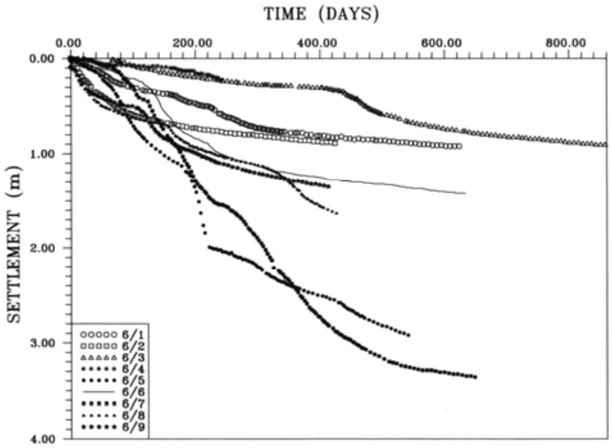

Figure 3(f) Comparison of maximum settlement profiles with time for 6m high embankments.

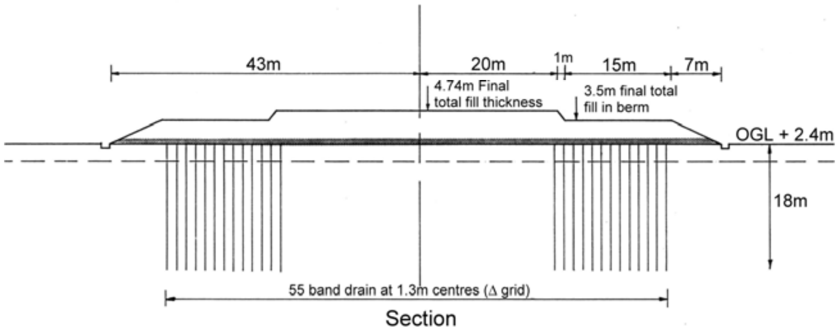

Figure 4(a) Cross section through the centre line of the embankment.

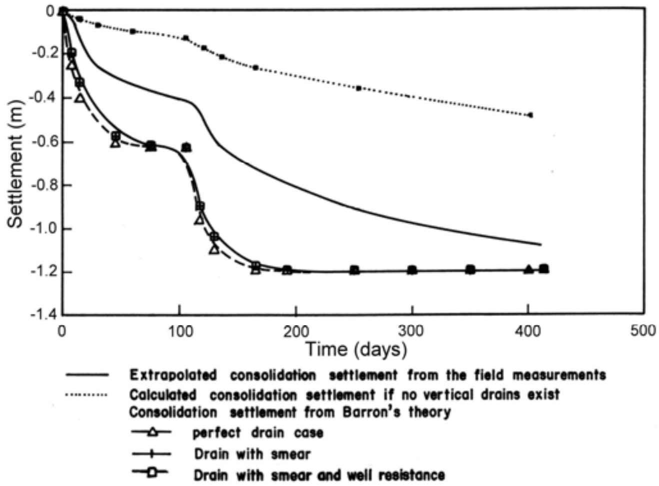

Figure 4(b) Comparison of consolidation settlement at ground surface along the centre line of the embankment with vertical drains.

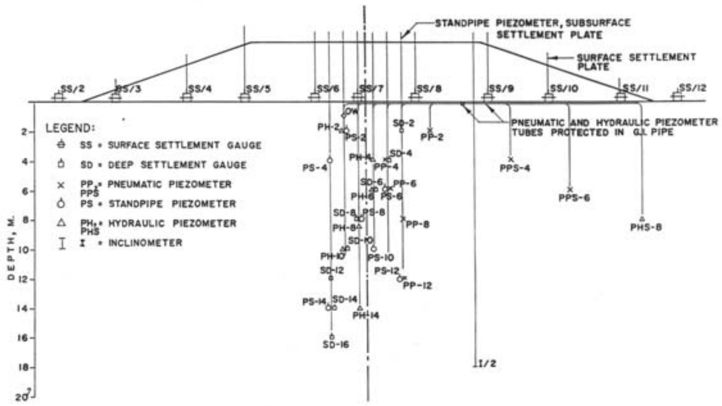

Figure 5(a) Section view of the test embankment showing the position of instruments at the Second Bangkok International Airport (SBIA).

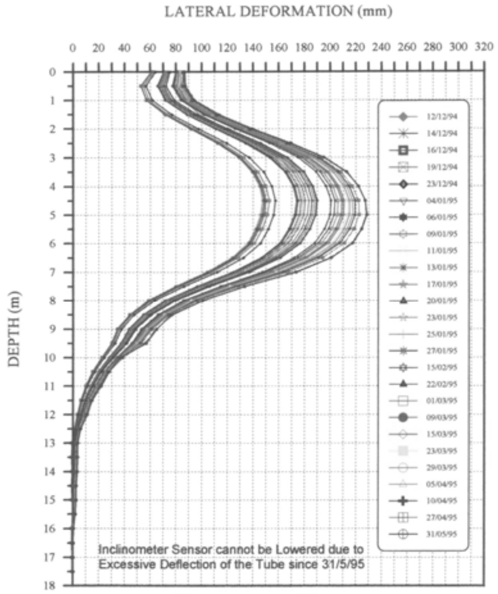

Figure 5(b) Lateral deformation with depth below embankment TS3 at the SBIA site in Bangkok

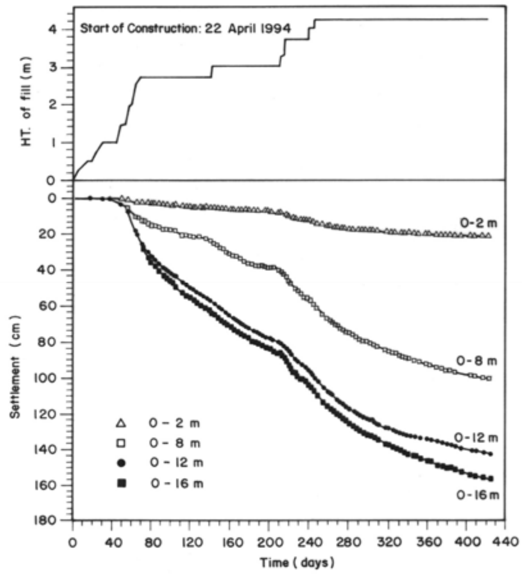

Figure 5(c) Settlement-time plot of the embankment with PVD at the SBIA site in Bangkok.

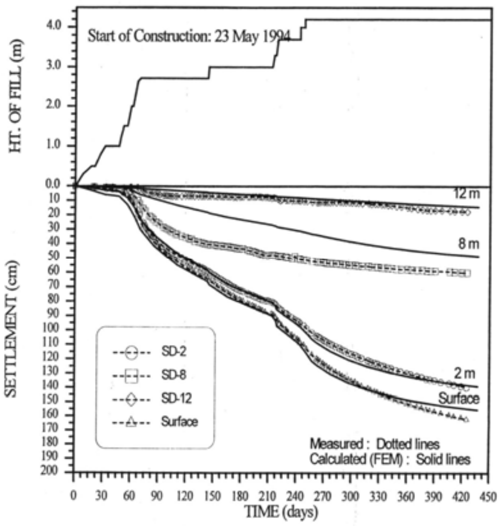

Figure 5(d) Measured and computed settlement with different depth at the SBIA site in Bangkok

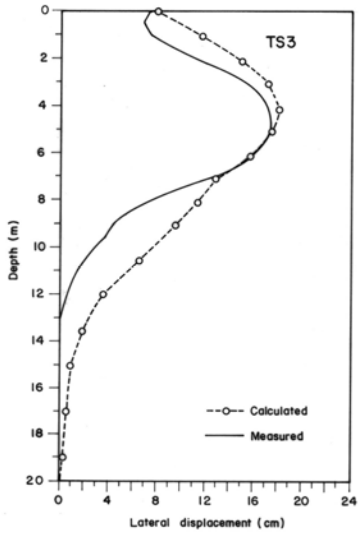

Figure 5(e) Measured and computed lateral movement of the embankment at the SBIA site in Bangkok.

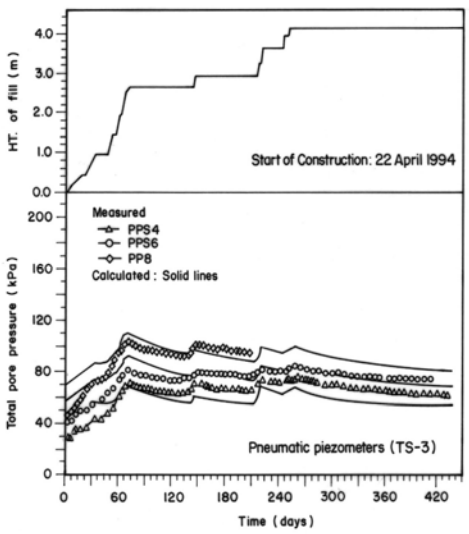

Figure 5(f) Measured and computed pore pressure dissipation of the embankment at the SBIA site in Bangkok.

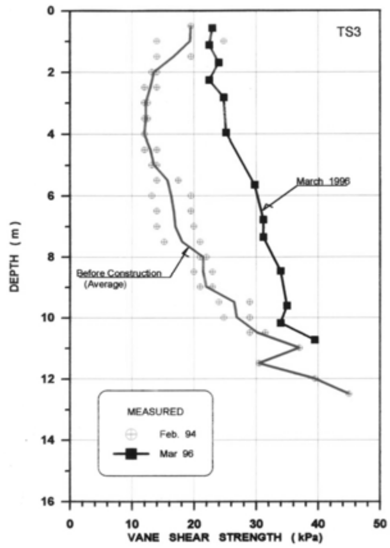

Figure 6 Improvement in field vane shear strength with consolidation settlement due to surcharge at the SBIA site in Bangkok

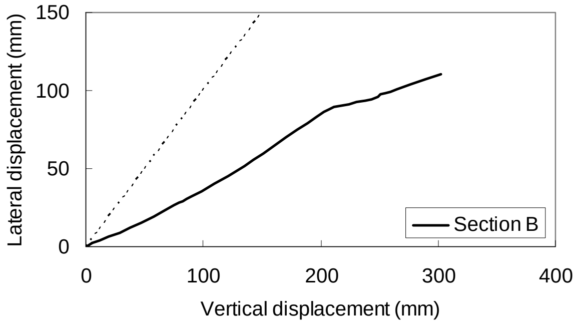

Figure 7(a) Variation in lateral displacement with settlement in Sunshine motorway trial embankment (based on Wijeyakulasuriya *et al* (1999))

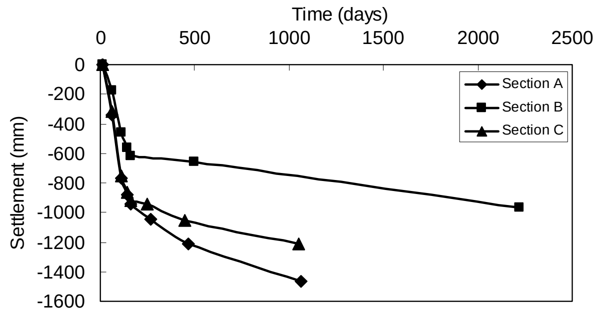

Figure 7(b) Time versus settlement curve for Sunshine motorway trial embankment (based on Wijeyakulasuriya *et al* (1999))

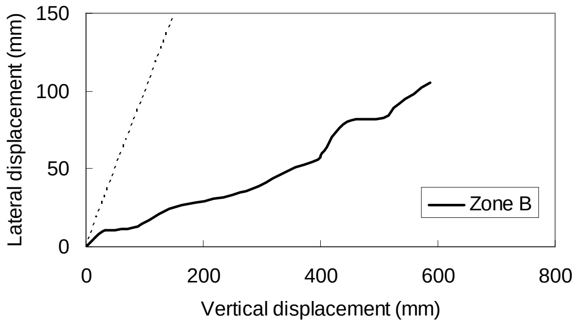

Figure 8(a) Variation in lateral displacement with settlement in Gold Coast Highway trial embankment (based on Wijeyakulasuriya *et al* (1999))

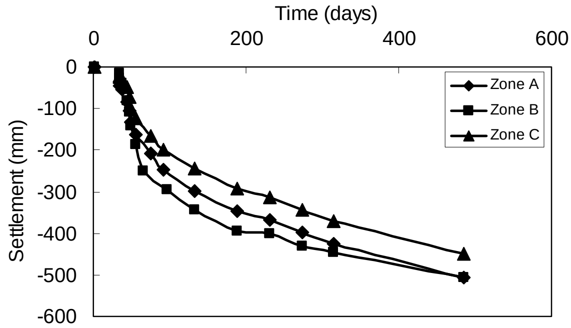

Figure 8(b) Time versus settlement curve for Gold Coast Highway trial embankment (based on Wijeyakulasuriya *et al* (1999))

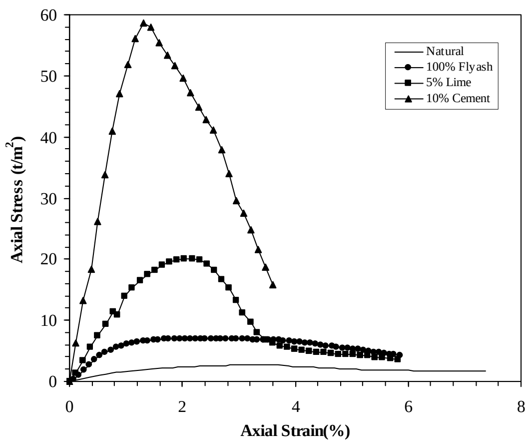

Figure 9(a) Comparison of stress-strain behaviour of treated and untreated clay

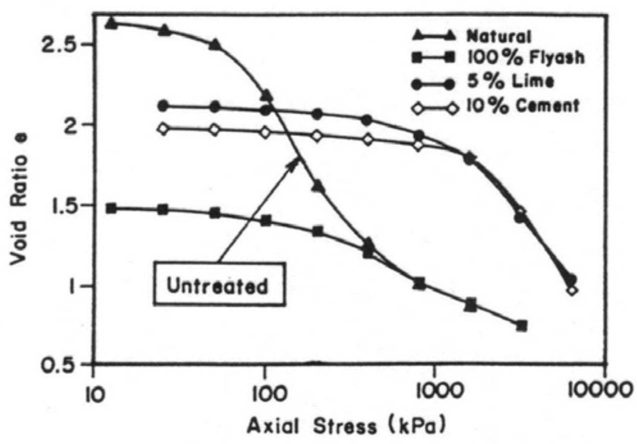

Figure 9(b) Void ratio (e) vs. $\ln \overline{\sigma}_v$ relations of treated and untreated clay in oedometer test

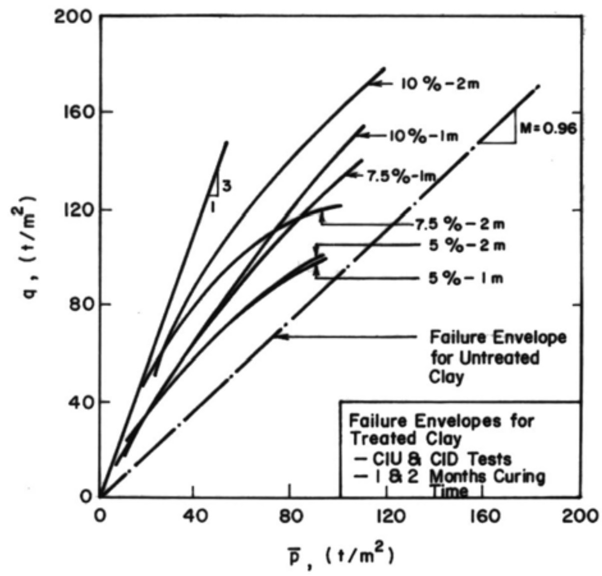

Figure 9(c) Failure envelopes for lime treated clay from CIU tests.

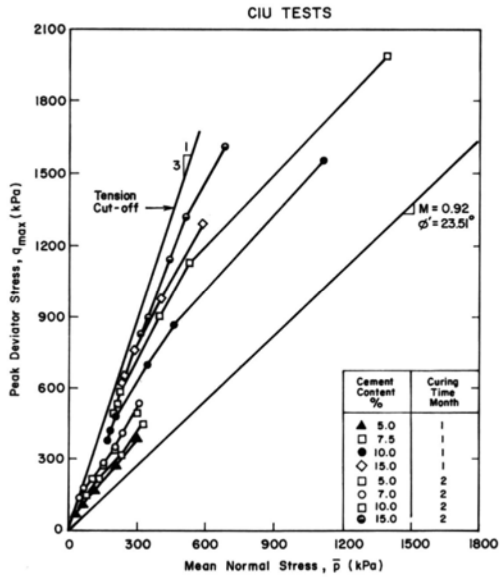

Figure 9(d) Failure envelopes for cement treated and untreated clay from CIU tests.

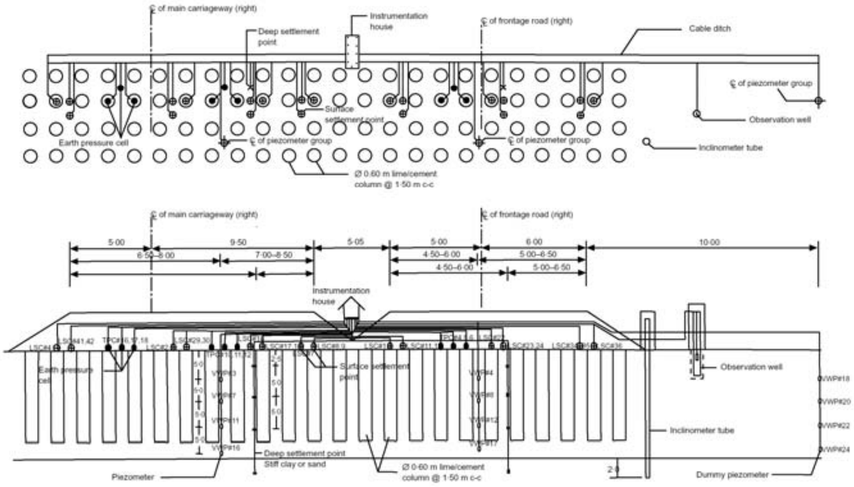

Figure 10(a) Plan and cross-section of instrumentation at km 29+992 (not to scale) (in m) (Bergado *et al*., 1999)

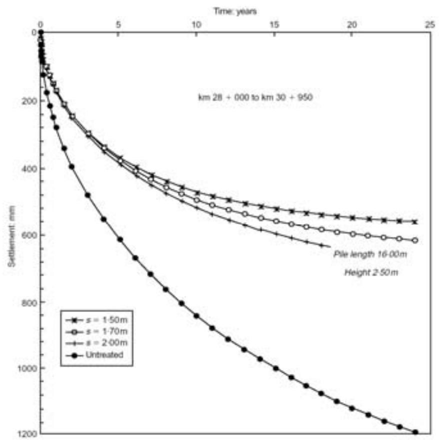

Figure 10(b) Settlement-time relationship for pile tip at 16 m and embankment height 2.5 m with various columns spacing (Bergado *et al*., 1999).

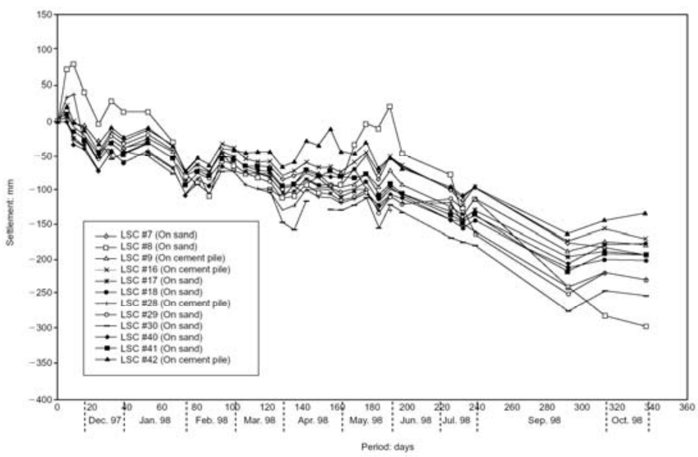

Figure 10(c) Settlement cell graph at station 29+992 (MR3) (Bergado *et al*., 1999)