GROUND IMPROVEMENT TECHNIQUES ADOPTED IN BRISBANE AND THE GOLD COAST FOR THE CONSTRUCTION OF ROADS AND MOTORWAYS

by

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4091ENG THESIS

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Project Brief

Student Name:	**Brett Eddie**
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Course:	**Bachelor of Civil Engineering**
Date	**18 March 2003**
Title of Project:	**Ground improvement techniques adopted in Brisbane and the Gold Coast for the construction of roads and motorways.**
Supervisor:	**Professor Balasubramaniam (Bala)**

Background

Certain areas on the Gold Coast, namely the Helensvale multi-level interchange with the Gold Coast Highway and the M1, and Brisbane, namely a section of the Sunshine Coast Motorway, have experienced sub-grade strength problems and therefore required ground improvement techniques to be adopted to allow these sections of road to be serviceable.

Ground improvement techniques currently used in road construction are, surface compaction, deep compaction, prefabricated vertical drains (PVD), granular piles, lime/cement stabilization, mechanically stabilized earth (MSE) and electro-osmotic consolidation.

Objectives

1. Analyse existing laboratory and field test data to evaluate geotechnical parameters needed for the study.

2. Analyse large-scale field test data.

3. Suggest possible alternative ground improvement schemes suitable for the Brisbane and Gold Coast areas.

Brett Eddie (student) Professor Bala (supervisor)

ABSTRACT

This thesis focuses on the acquisition and interpretation of test data associated with the use of vertical drain with surcharge and stone column treatment, as possible ground improvement schemes for South East Queensland.

Three road sites were selected for study: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway. Through the analysis of field data collected from ten well-instrumented trial embankments constructed at these sites, the effectiveness of ground improvement techniques was investigated. The soil at all three sites was characterised as clay with low strength, high compressibility and high sensitivity.

It was found that the stone column treatment was ineffective in reducing settlement in the highly sensitive clay. It is suggested that disturbance coursed to the clay during column installation diminishes the effect of the columns. Subsequently, reduced column spacing had negligible effect on further reducing settlement. It was established that vertical drains installed with a surcharge were effective at significantly increasing the rate of settlement and decreasing consolidation time. Therefore vertical drain treatment is considered more appropriate for the clay of South East Queensland.

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Chapter 1

Introduction and Scope of Work

1.1 Introduction

This final year civil engineering thesis focuses on ground improvement techniques adopted in Brisbane and the Gold Coast for the construction of roads and motorways. Soft clay with a relatively high percentage of organic matter spans the South East region of Queensland. Due to its low shear strength and high compressibility, this clay requires strength improvement to accommodate road pavement loading. The selection of appropriate ground improvement techniques is important to ensure the success of road projects, namely projects undertaken by the South East Queensland Department of Main Roads.

Common ground improvement techniques adopted for the improvement of soft clays are, pre-loading with or without the use of various drains, sand compaction piles, stone columns, electro-osmosis, chemical additives via shallow or deep mixing, lime and cement piles and soil replacement with light weight fill material. The most popular of these is pre-loading, incorporating the use of prefabricated vertical drains (PVD). Stone columns have been adopted in certain instances where PVD treatment is considered less effective.

Data available from the Queensland Department of Main Roads has been used for this study. Laboratory tests and full-scaled field test data from three road sections: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway, will be analysed. These road sections encompass, vertical drain with surcharge, and stone column treatment. Investigations into the effectiveness of these ground improvement techniques, on the strength, stability and serviceability of the soft clay found in the South East Queensland, will be conducted.

This thesis is the first phrase of a detailed investigation into the clay of South East Queensland. It focuses on the acquisition and presentation of test data collected from ten trial embankments constructed at three sites. The organisation of this data into a clear and suitable form will allow further detailed analysis to be carried out.

1.2 Scope of Work

The main objectives of this thesis are:

4. Analyse existing laboratory and field test data on soft clays in Brisbane and Gold Coast areas to evaluate geotechnical parameters needed for study.

5. Analyse the large-scaled field test data obtained from well instrumented test embankments with ground improvement schemes, and present in a suitable form where subsequent detailed analysis can be carried out.

6. Suggest possible alternative ground improvement schemes suitable for the soft clays in Brisbane and Gold Coast with particular reference to the construction of roads and motorways.

1.3 Layout of Thesis

Chapter 1 contains the introduction, the scope of the work, and the layout of this thesis.

Chapter 2 is a literature review on ground improvement techniques adopted for the construction of roads and motorways.

Chapter 3 encompasses the existing soil condition investigation for three case study sites: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway, before ground improvement techniques were adopted and subsequent construction performed.

Chapter 4 collectively displays and explains the results from the individual analysis of the three case studies. Through this analysis the effectiveness of the adopted ground improvement technique is investigated.

Chapter 5 provides final conclusions regarding the effectiveness of the ground improvement techniques adopted for the three case studies. Recommendations succeeding the results obtained through analysis are provided.

1.4 Research Methodology

This thesis focuses on data obtained from the monitoring of ground improvement techniques adopted at three case study sites. This data was gathered from ten reports, listed in Appendix A. The majority of information in these reports is raw data assembled in a disorganised form, requiring extensive effort to organize it into a meaningful record, allowing time for only limited analysis.

The analysis began with determining soil characteristics: the liquid limit, plastic limit, moisture content, unit weight, undrained strength, sensitivity and pore pressure dissipations. Assessment of these characteristics indicated whether each site required ground improvement to resist the proposed design loading.

Following initial soil characteristic investigations, an analysis of the large-scaled field test data was performed. This data collected from well-instrumented test embankments allowed the plotting of: inclinometer movements, settlement, excess pore water pressure and earth pressure. These plots were critically analysed to determine the effective of adopted ground improvement techniques.

Finally, the overall effectiveness of ground improvement techniques adopted at the three sites was provided, along with possible alternative ground improvement schemes suitable for soft clays in South East Queensland.

Chapter 2

Literature Review
Ground Improvement Techniques

2.1 Introduction

Various forms of stabilisation treatments for soft clays have been utilized for many years, the oldest and most common of these being pre-loading with a surcharge. In later years vertical drains were used in conjunction with a surcharge to increase the rate of consolidation. Originally sand drains, installed via augured or jetted holes, were used as the primary vertical drain treatment, however, prefabricated vertical drains (PVD), are now very popular and are used almost universally as a ground improvement technique encompassing applied surcharge.

This review will give the greatest emphasis to stone column treatment, and preloading with vertical drains, namely PVD. The focus on these two techniques is due to the availability of data from the Queensland Department of Main Roads for the Brisbane and Gold Coast areas.

2.2 Stone Column Treatment

Stone columns have been recognised for over two decades as an acceptable means of reinforcing soft cohesive soils (Greenwood and Kirsch, 1984). Stone column construction is deemed an extension of the vibro-compaction method, used to treat non-cohesive soils, since the same vibrators are utilised. The majority of stone column development is undergone in Europe. Vibration alone of a cohesive soil does not improve the soil. Therefore it was natural and practical to induce coarse granular backfill in the form of a bore column. The granular filled columns form a composite with the soft soil, similar to piles (Holtz *et al*, 1989).

Stone columns are commonly installed in triangular or square patterns, at centre-to-centre spacings of 1.5 to 3.5m. An entire foundation area may be covered, with additional coverage around the perimeter to include stress spread with depth. Columns may also be used in clusters and rows to support footings and walls (Mitchell, 1981). Usually 15 to 35 percent, by volume, of the soft soil is replaced by stone. Columns usually range from 0.6 to 1.0 m in diameter with gravel particle sizes in the range of 20 to 75 mm. The interaction between the in-situ soil and the introduced stone creates a composite with beneficial characteristics. As the resultant composite works together to combat applied loads, greater compression and shear forces can be resisted.

If stress is applied when a granular column is constructed in soft ground, the amount of consolidation settlement decreases due to redistribution of stress in an arching effect between the original ground and the granular columns (Balasubramaniam, 1996).

Mitchell (1981) stated that despite columns of compacted sand/gravel/stone being frequently used to reduce settlements, the efficiency of compacted granular piles decreases with decreased strength of the clay. Experience from tests undertaken in Bangkok clay reported by Bergado *et al* (1992), supported Mitchell's findings. Resulting from these studies, STS Engineering (1992), chose to boycott stone columns as a satisfactory ground improvement technique for various projects. A trial embankment undertaken by the Queensland Department of Main Roads (1996), encompassed stone column treatment as the adopted ground improvement technique. It was found that the varying column spacings of 2 and 3 m, were comparable, and the overall use of the stone columns did not indicate any real gains either in terms of significantly reducing settlement or consolidation time periods. It was suggested that this poor performance might have been caused by the disturbance experienced by the sensitive soft clays during installation.

2.3 Prefabricated Vertical Drains

Most compressible soils are alluvial deposits, which are more pervious in the horizontal direction than in the vertical direction (Barron, 1948). When these soils are loaded, consolidation of the soil is increased due to the presence of horizontal. Prefabricated vertical drains, PVDs, are adopted to increase the benefit of the horizontal flow by significantly speeding up settlement and excess pore water pressure dissipation. When a soil is loaded, excess pore water pressure's travel distance is reduced via radial draining. Water captured in the drains then flows freely towards more permeable layers, reducing required settlement time.

2.3.1 Advantages of Prefabricated Vertical Drains

Atkinson and Eldred (1981) explained the advantages of using prefabricated vertical drains as follows:

1. With the increased rate of shear strength development within clay, construction plant can often be more effectively utilised due to the load being able to be applied more rapidly.
2. The time required for primary settlement is decreased, due to the increased rate of consolidation of the clay.
3. Many soft clay strata contain thin bands of silt and sand. Vertical drains relieve the excess pore pressure that spreads horizontally along these bands, thus avoiding the occurrence of instability.

2.3.2 Prefabricated Vertical Drain Components

Prefabricated vertical drains usually consist of two main components, a central core surrounded by a thin filter jacket. The central core creates a free-draining water channel which is protected by the filter jacket, stopping surrounding soil from entering the central core while allowing excess pore water free access (Atkinson and Eldred, 1981).

Functions of the central core as depicted by Rixner *et al* (1986) are to:

1. Provide an internal flow path within the drain.
2. Provide support to the filter jacket.
3. Maintain drain configuration and shape.
4. Provide resistance to longitudinal stretching as well as buckling of the drain.

Rixner *et al* (1986) stated the functions of the filter jacket as follows:

1. Form a surface, which allows a natural soil filter to develop to inhibit movement of soil particles while allowing passage of water into the drain.
2. Create the exterior surface of the internal drain flow paths.
3. Prevent closure of the internal drain flow paths under lateral soil pressure.

Carroll (1983) suggested three basic elements for geotextile filter criteria: retention ability, permeability and clogging resistance. The requirements that must be satisfied by a filter sleeve then are (Hansbo, 1979):

1. The permeability of the filter should not be considerably less than the permeability of the soil in which the drains are installed.
2. The filter should not retain fine soil particles. Otherwise the channels between the filter sleeve and the core might overtime be filled with soil and become clogged.
3. The filter should have sufficient strength to resist being completely squeezed into the central core channel by high lateral soil pressure.
4. The filter should have sufficient strength to resist installation forces.
5. The filter should be robust; to inhibit deterioration with time as this could reduce the discharge capacity of the drain.

Vreeken *et al* (1983) remarked that since the retaining capacity and the resistance of a filter are mainly dependent on the particle size and distribution and the pore size distribution of the filter, three principal filter mechanisms can be distinguished as: cake filtration, blocking filtration, and deep filtration.

Cake filtration takes place if the particles are larger than the filter pores. All particles are retained on the filter and the resistance of the filter cake rises as the thickness of the cake increases (in the case of pure infiltration and an incompressible filter cake, there is a linear relationship between the amount of retained particles and the resistance of the filter cake). Blocking filtration takes place if the particles have about the same diameter as the filter pores. The particles are retained in the filter medium and the filter resistance can rise rapidly. Deep filtration takes place if the particles are smaller than the filter pores and the particles adhere on the filter. Only a fraction of the particles are retained.

2.3.3 Prefabricated Vertical Drain Discharge Capacity

Considering that PVDs are adopted to assist in the remove of water from a soil, the most important requirement is therefore that PVDs have a high discharge capacity, to reduce resistance and improve the efficiency of the drain (Hansbo, 1981).

The discharge capacity varies due to such factors as (Holtz *et al*, 1989 and Bergado *et al*, 1996):

1. The volume of the core available for flow.
2. The effect of lateral earth pressure on that volume.
3. Possible folding, bending and crimping of the drain due to large settlements.
4. Infiltration of fine soil particles through the filter that could cause a reduction of flow capacity of the drain (siltation).
5. Biological and chemical durability of the drain system.

2.3.4 Design Factors

For the purpose of this study, the only design factor explained here is the degree of consolidation. Other design factors, such as drain spacing, equivalent diameter, well resistance and smear zone, are discussed in Appendix B.

2.3.4.1 Degree of Consolidation

The average degree of consolidation (Hansbo, 1981) due to radial drainage can be computed as:

(2.15)

where:

(2.16)

(2.17)

c_h = horizontal coefficient of consolidation
m_v = coefficient of volume compressibility
γ_w = unit weight of water
$\mu_s = F(n)$ = drain spacing factor

(2.18)

Considering the effect of well resistance and smear

(2.19)

(2.20)

where: F_s = smear factor and F_r = well resistance factor.
It is apparent from theoretical parametric studies that the drain spacing effect, $F(n)$, is always an important factor, the soil disturbance effect, F_s, can be of approximately the same or slightly more significance than $F(n)$, and the drain resistance effect, F_r , is typically of minor importance (Rixner *et al*, 1986). For both horizontal and vertical drainage occurring, the total degree of consolidation is given by Carillo (1942) as:

(2.21)

where: U_h = degree of consolidation due to horizontal or radial drainage
U_v = degree of consolidation due to vertical drainage

2.4 Concluding Remarks

In the coastal area of South East Queensland, where highly compressible soil with very low shear strength is present, ground improvement techniques are used for improving the foundations for construction of roads and motorways. These very soft clay deposits, which in some cases contain marine content, have a high organic content. These Holocene soils have very high secondary consolidation settlement and creep characteristics. Current knowledge for the design of embankments and piled foundations for the observed alluvial deposits is not adequate, and therefore extensive special laboratory and field tests must be performed. The observational approach is used as full-scaled instrumental field tests are undertaken to monitor the effectiveness of ground improvement techniques.

Chapter 3

Soil Characteristics

3.1 Introduction

Before trial embankments are constructed to investigate the effectiveness of possible ground improvement techniques, soil characteristics need to be assessed. This assessment will determine if the soil does in fact require strength and stability improvement in order to resist the design loading.

This chapter considers soil characteristics for three case studies: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway. For each study, the general soil classification, liquid limit, plastic limit, moisture content, undrained strength, soil sensitivity and pore pressure dissipation is presented, and the need for ground improvement evaluated.

3.2 Case Studies

3.2.1 Case Study 1 - Gold Coast Highway

The Gold Coast Highway is the major link between the Gold Coast and the M1, which is part of the Pacific Motorway of Queensland. Increasing traffic occupancy has made it necessary to upgrade the existing two-lane road to a four-lane facility. Adjacent to this section of highway, the land has an important environmental status, as being a wetland reserve. This section of the highway traverses a swamp with up to 13.5 m of soft clay. The lack of weathered crust is noted and high liquidity indices are indicated.

The soils and rocks along the highway belong to the Neranleigh-Fernvale Group of the Silurian Age. The rocks are mainly greywacks with some interbedded argillite. Some Quaternary alluvium is present in the valleys between the ridges. The alluvium associated with the gullies is composed of clays and silty clay overladen by thin soft organic clays.

3.2.2 Case Study 2 – Sunshine Coast Motorway

The Sunshine Coast Motorway is the main link between Brisbane and the Sunshine Coast. The current two-lane road requires rehabilitation, and provisions for a future four-lane road are to be implicated. This section of motorway, 4.7 km in length, extends north through cane farm lowlands with a minor swampy section adjacent to the Maroochy River before intersecting terrain of higher relief to the south of West Coolum Road. The route then re-enters another small swampy area and further cane farm lowlands.

The road alignment traverses predominately low-lying unconsolidated sediments of Quaternary Age viz: Coastal mangrove and tidal deposits (dark grey to black organic very soft fine grained clayey-silt mixtures which contain minor sand), Estuarine Swamp and lagoonal deposits (soft organic clay, mud, sand-clay mixtures and minor sand) and Tidal delta deposits (grey-white, fine to medium sand). The limited areas of higher relief comprise remnants of the older Triassic rock formations consisting of both volcanic and sedimentary strata.

3.2.3 Case Study 3 – Port of Brisbane Motorway

The Port of Brisbane Motorway carries large volumes of traffic everyday. It incorporates the heavily trafficked interchange between the Gateway Motorway and the Queensport Road intersection. Due to increasing demand, particularly on the Gateway Motorway, rehabilitation and upgrading works are required. This section of road generally traverses low lying flood plains associated with the Brisbane River, with large variances in clay layer thickness ranging from 2 m to over 26 m.

3.3 Index Properties and Unit Weight

Laboratory tests determined the wet density (ρ_{wet}), and dry density, (ρ_{dry}), of the soil found at the three sites, results are shown in Table 3.1.

Table 3.1: Unit weights and soil type

Case Study	Wet Density ρ_{wet} (t / m3)		Dry Density ρ_{dry} (t / m3)		Soil Classification
	Low	High	Low	High	
Gold Coast Highway	1.46	1.56	0.54	0.96	CH
Sunshine Coast Motorway	1.36	2.18	0.6	1.88	CH
Port of Brisbane Motorway	1.4	2.34	0.72	1.86	CH

Note: Soil Classification based on the Unified Soil Classification System (SAA Site Investigation Code AS 1726), CH = 'inorganic clay of high plasticity'.

The liquid limit, the plastic limit and the moisture content profiles for all the three sites are shown in Figures 3.1, 3.2 and 3.3.

Figure 3.1 shows that the moisture content at the Gold Coast Highway site ranged from 110% at the surface, to 50% at a depth of 11m. It can be seen also that the liquid limit and the plastic limit were both generally uniform with depth, with values of 60 to 70%, and 25 to 40%, respectively.

Displayed in Figure 3.2 is the moisture content at the Sunshine Coast Motorway site. It shows a range from 30% at the surface, to 120% at a depth of 3m. It can be seen also that the liquid limit and the plastic limit both varied with depth, with values from 30 to 80%, and 6.2 to 50%, respectively.

Shown in Figure 3.3 is the moisture content at the Port of Brisbane Motorway site. A range from 95% at the surface, to 60% at a depth of 13m, is displayed. It can be seen also that the liquid limit and the plastic limit both varied with depth, with values from 25 to 75%, and 5.2 to 49.8%, respectively.

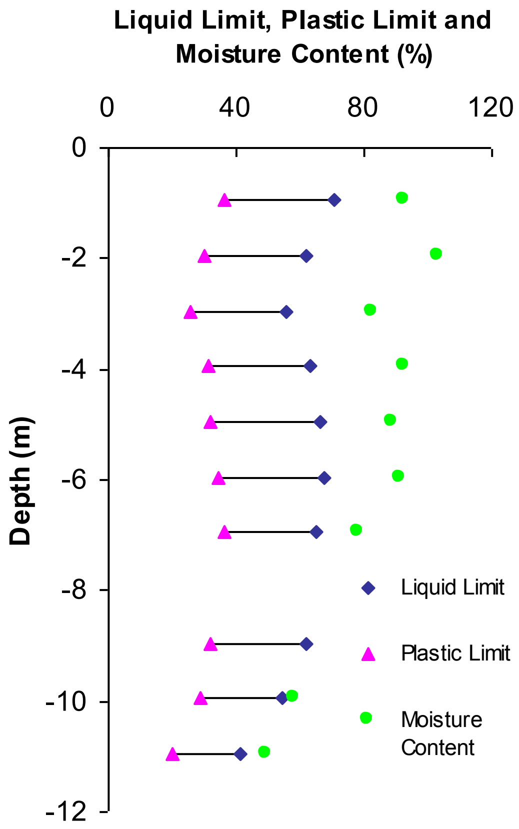

Figure 3.1: Liquid limit, moisture content and plastic limit profile (Gold Coast Highway)

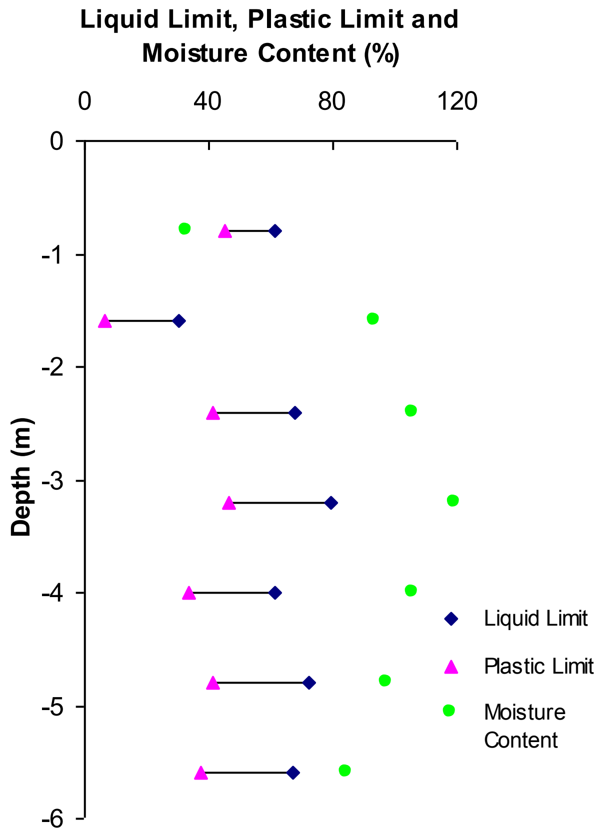

Figure 3.2: Liquid limit, moisture content and plastic limit profile (Sunshine Coast Motorway)

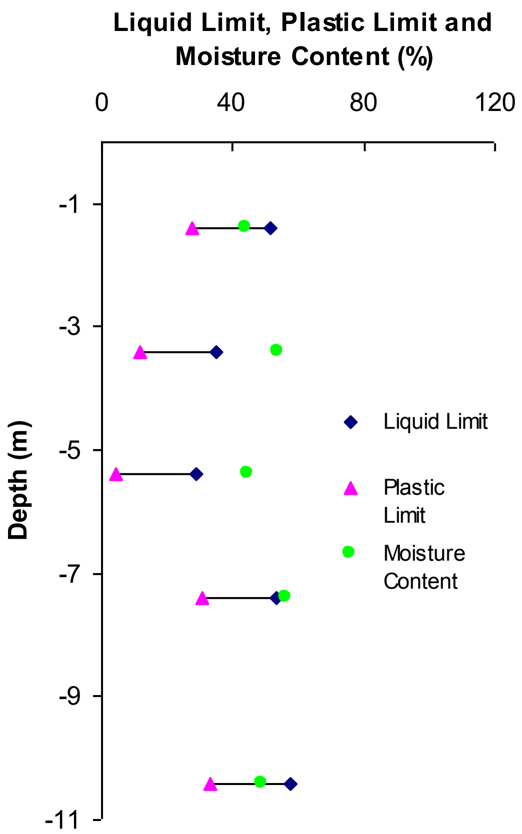

Figure 3.3: Liquid limit, moisture content and plastic limit profile (POB Motorway)

3.4 Undrained Strength

From field vane shear tests, the undrained shear strength (s_u), and the soil sensitivity (S_t), of the soft clay was determined, shown in Figures 3.4, 3.5 and 3.6. A summary of the data is displayed in Table 3.2.

Soil sensitivity (S_t), is defined as follows:

$$S_t = \frac{\text{undisturbed shear strength}}{\text{remoulded shear strength}} \tag{3.1}$$

Table 3.2: Summary of undrained strength results

Case Study	Shear Strength (kPa)			Soil Sensitivity		
	Surface	Depth 3 m	Depth 6 m	Surface	Depth 3 m	Depth 6 m
Gold Coast Highway	14	7	11	15	5.5	4
Sunshine Coast Motorway	11	12	15	4	4	3.8
Port of Brisbane Motorway	7.5	14	-	7.5	12	-

Figure 3.4 shows that the shear strength at the Gold Coast Highway site varies with depth, ranging from 8 to 20 kPa. Clays with undrained shear strength less than 20 kPa are considered very soft. It is also seen that the sensitivity of the soft clay reduces with depth. Clay with a sensitivity value between 4 and 8 is considered sensitive; therefore the top 2 m of the soft clay stratum is considered extra sensitive.

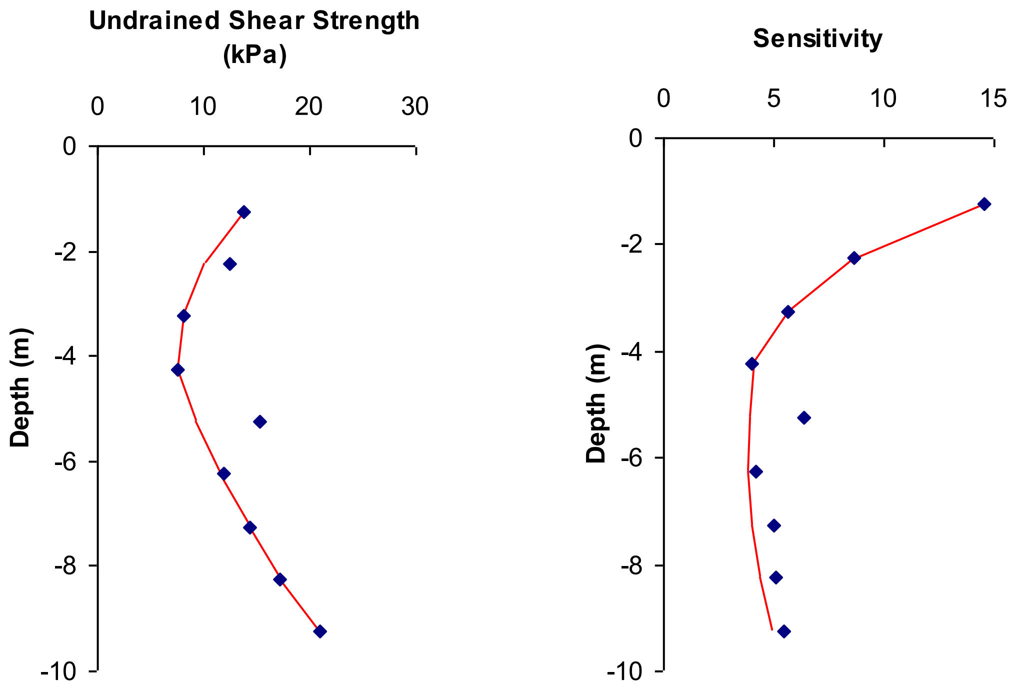

Figure 3.4: (a) Undrained strength profile **(b)** Soil sensitivity profile
(Gold Coast Highway)

Shown in Figure 3.5 is the shear strength at the Sunshine Coast Motorway site increasing slightly with depth, ranging from 8 to 15 kPa. This clay is considered to have very soft consistency. It is also shown that the sensitivity of the soft clay remained relatively constant, at a value of 4. The entire soft clay stratum is classified as extra sensitive.

The profile in Figure 3.6 shows that the shear strength at the Port of Brisbane Motorway site varied with depth, ranging from 8 to 18 kPa. This soil therefore is considered to have very soft consistency. It is seen also that the sensitivity of the soft clay varied with depth, ranging from 8 to 18. The middle 1 m of the soft clay stratum is considered extra sensitive.

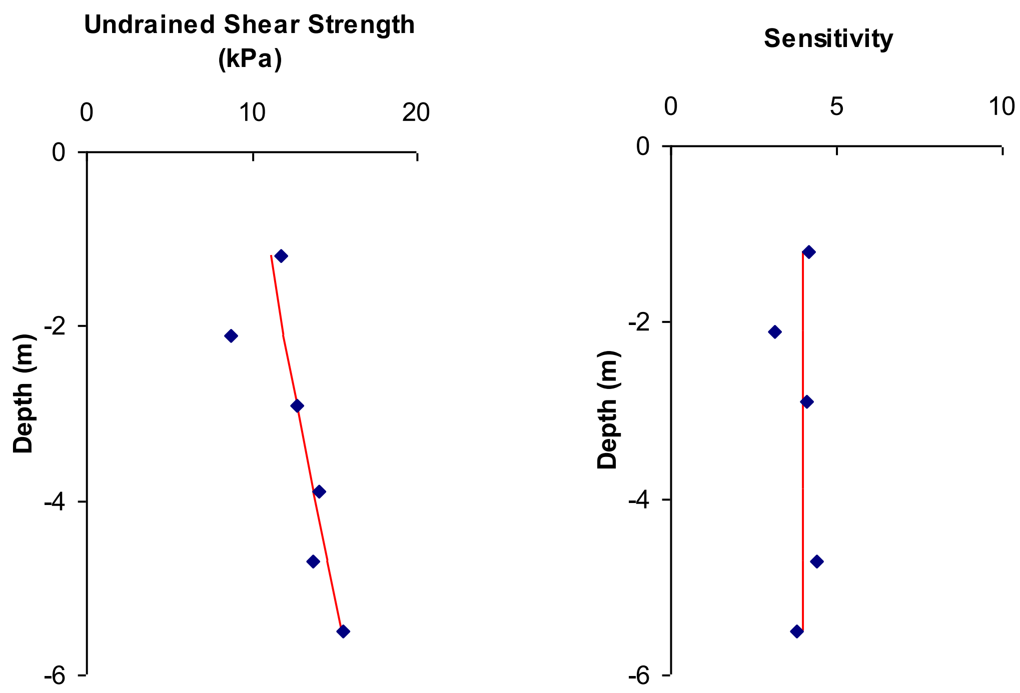

Figure 3.5: (a) Undrained strength profile **(b)** Soil sensitivity profile
(Sunshine Coast Highway)

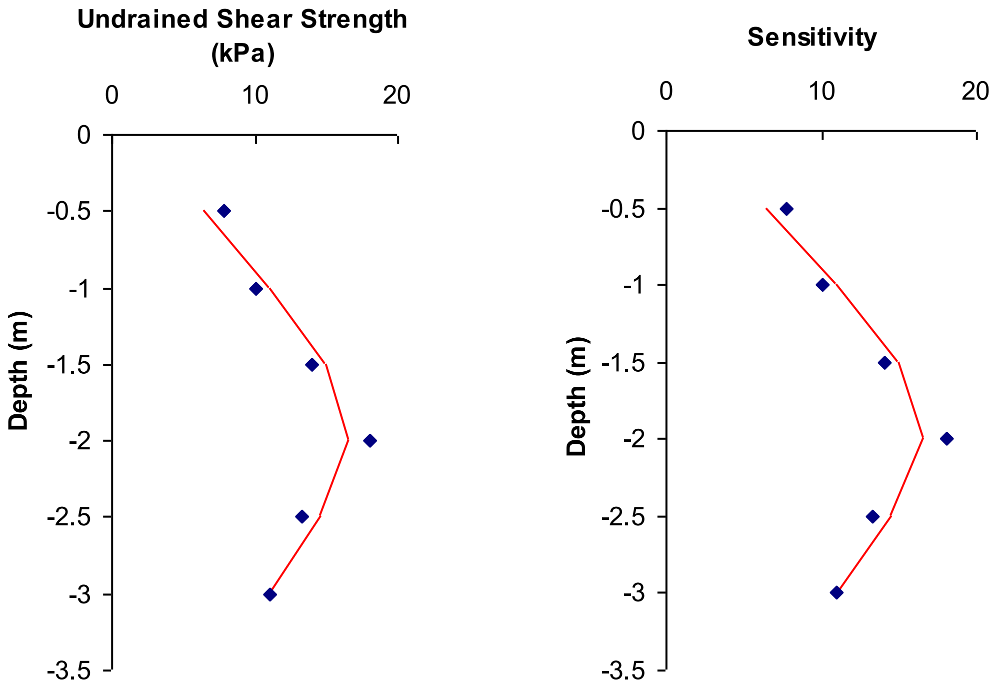

Figure 3.6: **(a)** Undrained strength profile **(b)** Soil sensitivity profile (POB Motorway)

3.5 Interpretation of Piezocone Data

Piezocone tests are often used to identify sub-surface drainage layers. The determination of the in-situ coefficient of consolidation is estimated from pore water dissipation tests. Four such tests were carried out at the Port of Brisbane Motorway site, readings shown in Figure 3.7. Five tests were carried out at the Gold Coast Highway site, readings shown in Figure 3.8. All profiles for each site showed a similar trend. Corresponding coefficient of consolidation, C_v, values are given in Table 3.3 and 3.4.

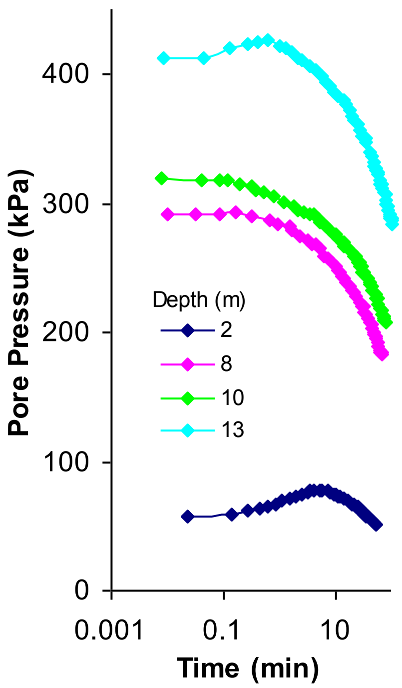

Figure 3.7: Piezocone dissipation curves (POB Motorway)

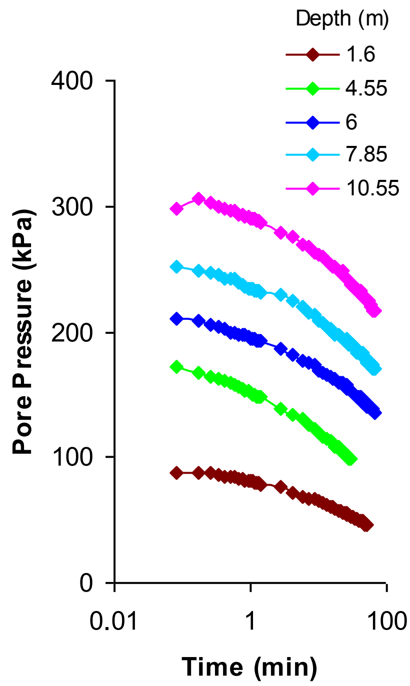

Figure 3.8: Piezocone dissipation curves (Gold Coast Highway)

Table 3.3: Coefficient of consolidation values (POB Motorway)

Depth Range (m)	Av. Pressure (kPa)	Coefficient of Consolidation, C_v (m^2 / year)
2 – 4.0	3.8	100.6

	9.4 17.9 29.2 42.9 63.6 96.4	50.3 68.0 10.6 11.9 54.1 89.0
4 – 4.4	4.9 14.7 29.4 48.9 73.4 110.3 165.3	39.0 49.6 19.5 35.6 22.0 29.6 30.2
8 – 8.4	14.6 31.1 56.0 89.1 130.5 192.5	19.9 1.6 1.5 0.7 0.3 0.2
11 – 11.4	18.2 38.4 68.7 109.2 159.7 235.3	2.0 1.5 1.7 0.8 0.3 0.3
14 – 14.4	13.1 35.4 71.4 119.4 179.0 269.1 393.1	3.7 1.1 1.0 0.8 0.4 0.3 0.3

Table 3.4: Coefficient of consolidation values (Gold Coast Highway)

Depth Range (m)	**Av. Vertical Effective Stress (kPa)**	**Coefficient of Consolidation, C_v (m^2 / year)**
0 - 3	8	9.2

	19 37 70 103	6.7 2.1 0.65 0.26
3 - 6	7 18 36 68 101	7.0 5.3 2.5 0.5 0.24
6 - 9	20 32 65 97 206 305	3.2 2.1 1.4 1.3 1.3 0.2
9 - 13.5	10 31 55 87 198 308	5.1 4.8 3.7 2.8 0.35 0.4

Note: Coefficient of consolidation calculations based on 'Terzaghi'.

3.6 Instrumentation

To aid in the selection of an appropriate ground improvement technique, trial embankments are often constructed. The design loading to support the road pavement is simulated by the trial embankment height. Instrumentation is installed within embankments to monitor lateral displacement, settlement, excess pore pressure, earth pressure, vertical stress and consolidation.

For the trial embankments at the Gold Coast Highway site, five different types of instrumentation were installed, a diagrammatic layout is given in Appendix C. At the Sunshine Coast Motorway site, four different types of instrumentation were installed, details given in Table 3.5. For the Port of Brisbane Motorway site, three different types of instrumentation were installed, details given in Table 3.6(a) and 3.6(b). The data obtained from these instrumentations will be analysed in the following chapter, Chapter 4.

Table 3.5: Instrumentation (Sunshine Coast Motorway)

Emb.	Chainage	Offset from Centreline	Settlement Gauge	Piezometer	Inclinometer	Standpipe Piezo.
SCE1	33476	T.B.A				
	33476	0.0	1	4		1

	33481	0.0			1	
	33488	0.0		2	1	
	33460	0.0	1			
	33510	0.0	1			
SCE2	36480	0.0				
	37120	0.0		1		1
	38290	0.0				
	38300	L.S				
	M003					
	100	0.0	1			
	100	T.B.A		4	1	1
	M010					
	80	0.0				
	160	L.S 0.0				
	420	R.S				
	M013	0.0				
	315					
		0.0				

Table 3.6(a): Instrumentation (POB Motorway)

Emb.	Chainage	Offset from Centreline	Settlement Gauge	Piezometer	Standpipe Piezo.
POBE1	778	0.0			1
	794	2m R		1	
	794	1m R		1	
	794	0.0		1	
	794	1m L		1	
	794	2m L		1	
	795	0.0	1		
POBE2	1040	2m R		1	
	1040	1m R		1	
	1040	1m L		1	
	1040	2m L		1	
	1070	2m L			1
	1112	0.0	1		
POBE3	1170	1m L		1	
	1170	0.0		1	
	1170	1m R		1	
	1175	0.0	1		

Table 3.6(b): Instrumentation (POB Motorway)

Emb.	Chainage	Offset from Centreline	Settlement Gauge	Piezometer	Standpipe Piezo.
POBE4	1210	1m L		1	
	1210	2m L		1	

	1210	1m R		1	
	1210	2m R		1	
	1210	0.0	1		
	1236	0.0			1
POBE5	1319	1m R			1
	1333	0.0	1		
	1333	0.75m L		1	
	1333	0.0		1	
	1333	0.75m R		1	
	1333	1.5m R		1	

3.7 Compression Curves

The analysis of compression curves is outside the scope of this thesis, however, a typical plot shown in Figure 3.9 has been included for illustration purposes.

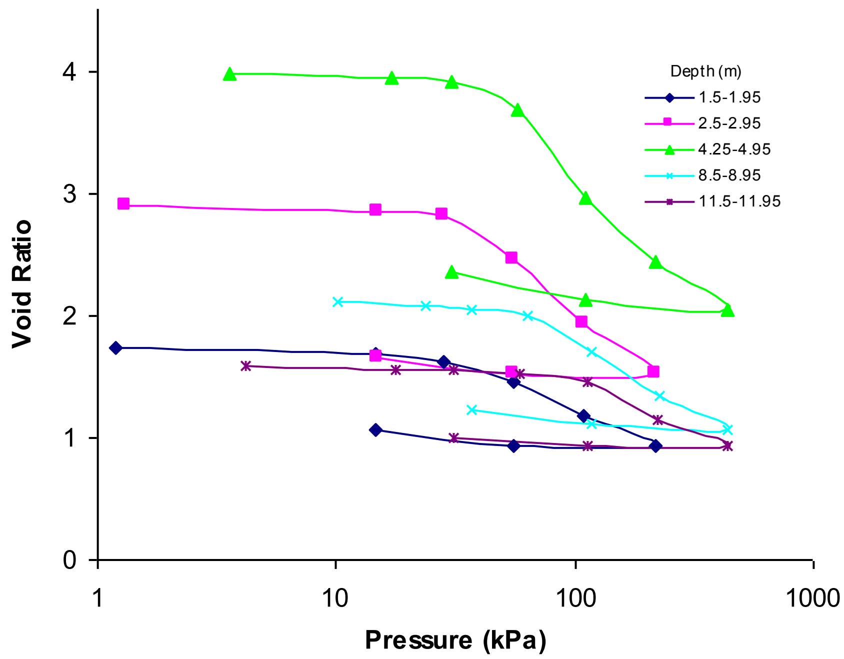

Figure 3.9: Typical compression curves

Ross Pyke is currently studying: Compressibility characteristics of South East Queensland soft clay, which includes the analysis of compression curves.

Chapter 4

Analysis and Discussions

4.1 Introduction

Trial embankments have assisted in understanding the complex behaviour between stone columns and various soil types, as well as assessing the effectiveness of vertical drains to decrease consolidation time. All data obtained provides experience that can be used for future road construction designs.

In this chapter, data collected from test embankments at three sites: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway, will be analysed. This analysis covers: inclinometer movements, settlement, degree of consolidation, earth pressure and excess pore pressure dissipations.

4.2 Case Study 1: Stone Column – Gold Coast Highway

Trial embankments were constructed primarily to investigate two objectives: investigate soft clay settlement under an imposed embankment height of 2 m, and examine strength behaviour associated with treated embankments. This leads to a more comprehensive understanding of sensitive clay behaviour.

Three trial embankments were constructed to investigate stone column treatment in sensitive clay, shown in Table 4.1.

Table 4.1: Trial embankments (Gold Coast Highway)

Trial Embankment	Maximum Construction Height (m)	Treatment	Details		
			Diameter (m)	Spacing (m)	Pattern
GCE1	2	Stone Column	1	3	square
GCE2	2	None	-		
GCE3	2	Stone Column	1	2	square

Stone columns are usually constructed from stone or crushed rock. For this site the columns were constructed from crushed rock with a minimum specific gravity of 2.7, and a maximum Los Angeles abrasion value of 35.

4.2.1 Inclinometer Monitoring

Typically, inclinometers are installed to measure horizontal sub-ground level movements. At the Gold Coast Highway site, lateral displacement was monitored at the trial embankment toe and centreline, shown in Figure 4.1 and 4.2, respectively. The maximum lateral displacement at the toe was 76.84 mm, and 47.95 mm at the centreline, seen in the sensitive upper layers.

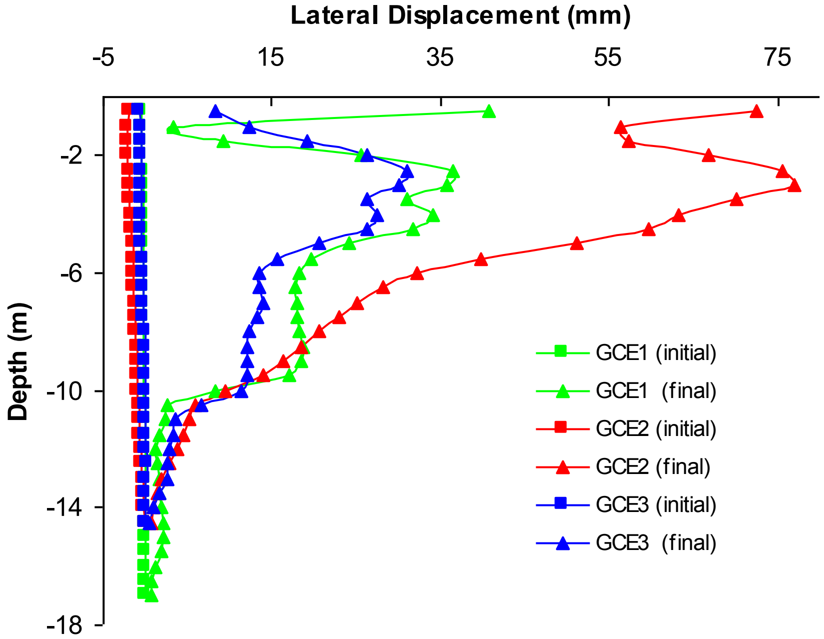

Figure 4.1: Inclinometer movements at toe (Gold Coast Highway)

It is considered that the installation of stone columns should increase the lateral stability of a soil. As seen in Figure 4.1, the installation of stone columns dramatically reduced lateral displacement, as the lateral displacement measured in the treated embankments was less than half that measured in the untreated. Seen also is that column spacing had negligible impact on reducing lateral toe displacement. As shown in Figure 4.2, column spacing does effect centreline lateral displacement. This reduction is seen to be approximately half.

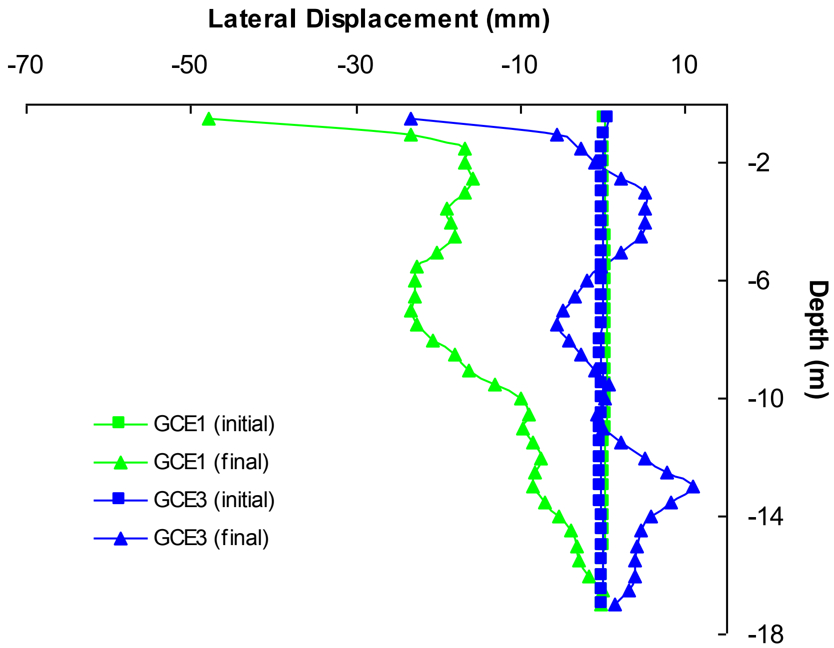

Figure 4.2: Inclinometer movements at centreline (Gold Coast Highway)

4.2.2 Settlement Monitoring

Typically, horizontal profile gauges or vertical settlement gauges are used to measure settlement. At the Gold Coast Highway site, both horizontal profile gauges and vertical settlement gauges were installed, readings shown in Figures 4.3 to 4.5.

The settlements measured are relatively similar in profile and magnitude, suggesting that the stone column treatment did not reduce settlement. It is shown that the settlement rate is initially uniform, but increases around day 48, and again around day 131, suggesting that the stone columns are having not effect. Also, the final slope seen in Figure 4.3(b), 4.4(b) and 4.5(b), suggests that the clay has not reached its maximum settlement for the 2 m high embankment loading. The abrupt profile seen in Figure 4.5(a), is due to the large disturbance coursed during installation.

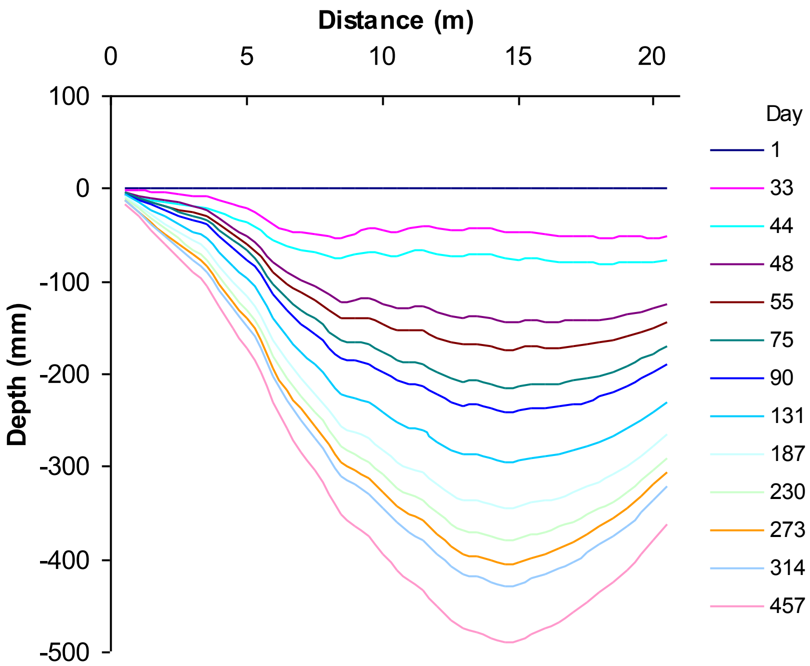

Figure 4.3(a): Measured settlement for treated GCE1

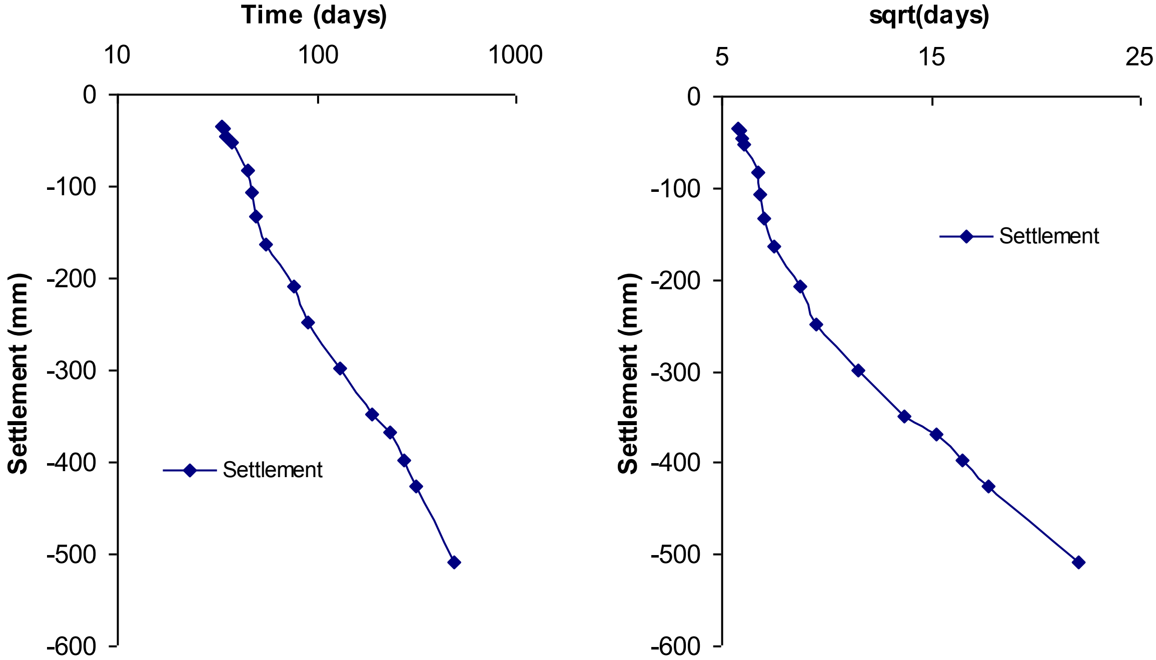

Figure 4.3(b): Measured settlement for treated GCE1

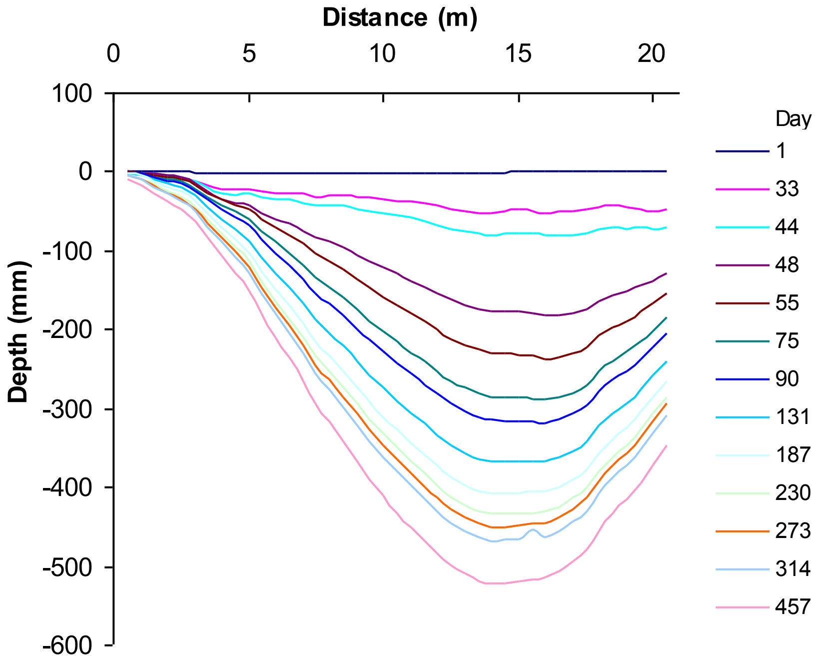

Figure 4.4(a): Measured settlement for untreated GCE2

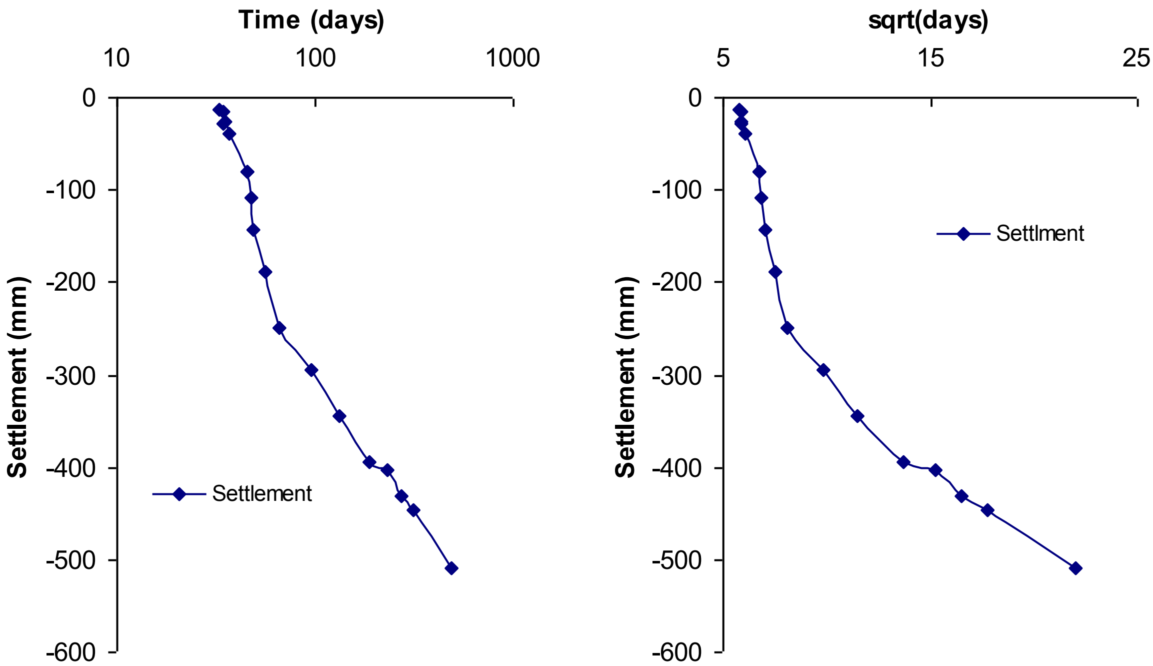

Figure 4.4(b): Measured settlement for untreated GCE2

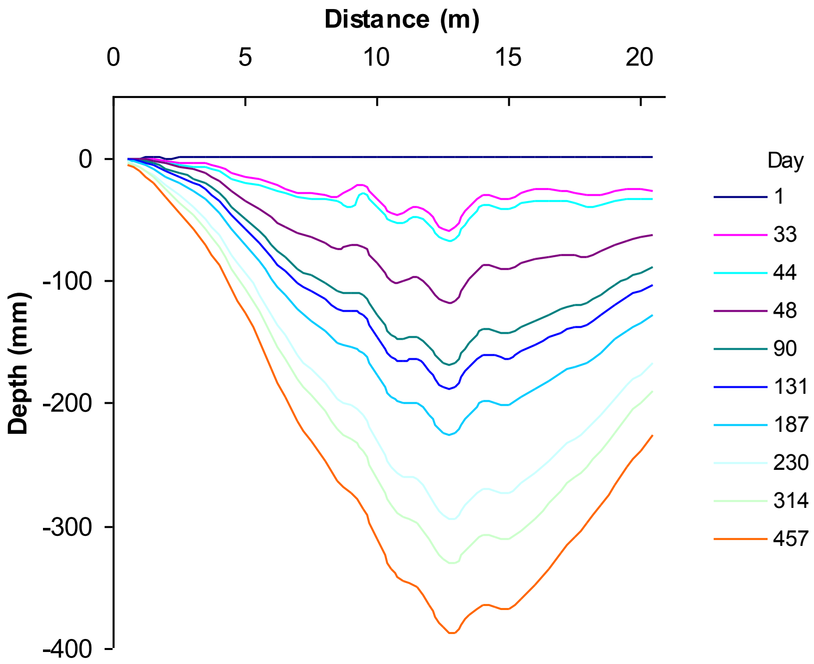

Figure 4.5(a): Measured settlement for treated GCE3

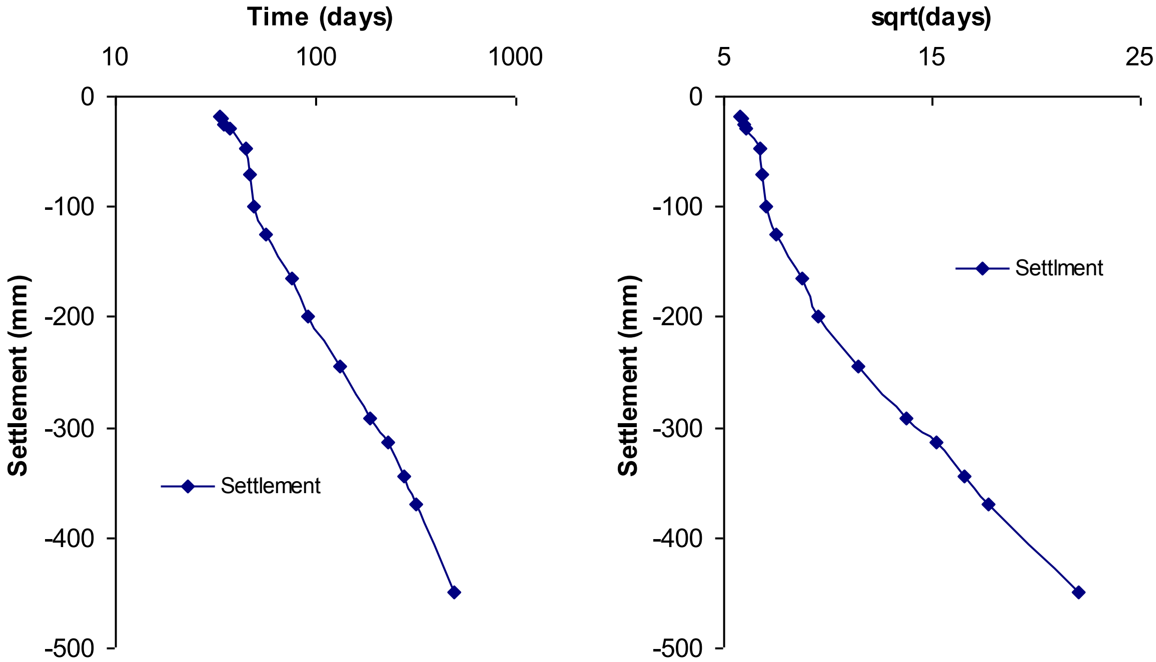

Figure 4.5(b): Measured settlement for treated GCE3

The final settlement readings obtained after 485 days of monitoring are shown in Table 4.2. A comparison between the horizontal profile gauge and vertical settlement gauge readings verifies these results. It has been suggested that some of the gauges in GCE3 were installed in the columns rather than in the soil, explaining the reading difference.

These readings indicate that stone columns had practically no impact on reducing settlement. Also, it is shown that decreased column spacing provided negligible further reductions.

Table 4.2: Summary of settlement results (Gold Coast Highway)

Trial Embankment	Maximum Horizontal Profile Gauge Reading (mm)	Maximum Vertical Settlement Gauge Reading (mm)
GC_E1	490	508
GC_E2	522	508
GC_E3	386	450

4.2.3 Excess Pore Pressure Monitoring

Piezometers were used to measured pore water pressure at the Gold Coast Highway site. As embankment height was increased, readings were taken at several day intervals, until upon reaching the full embankment height, from which readings were taken at a period of approximately one week.

Excess pore water pressure was measured at various depths, readings shown in Figure 4.6, 4.7 and 4.8. When embankment height is increased, extra load is exerted on the underlying clay. As seen in the profiles, this load courses a peak in excess pressure. This pressure dissipates over time as water is forced to escape, returning approximately to the initial values.

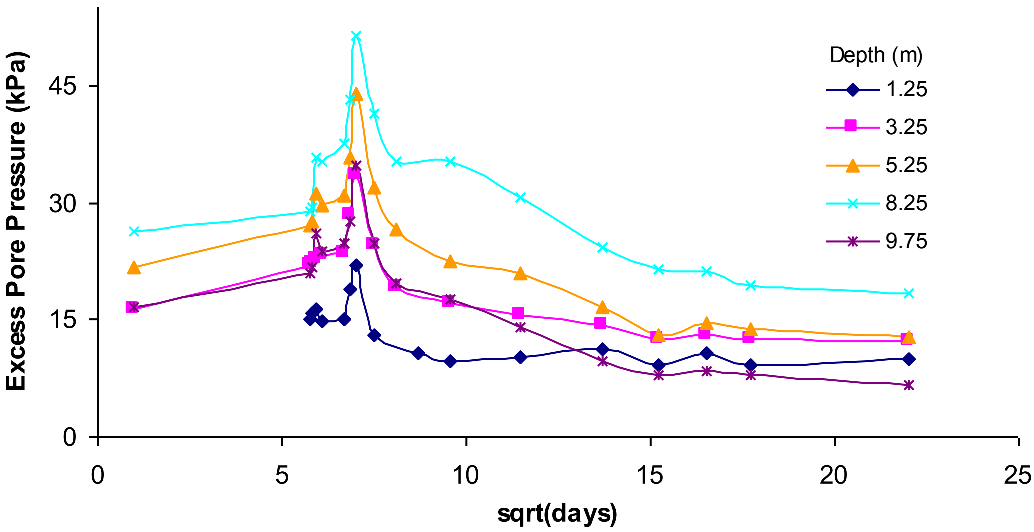

Figure 4.6: Excess pore pressure readings for treated GCE1

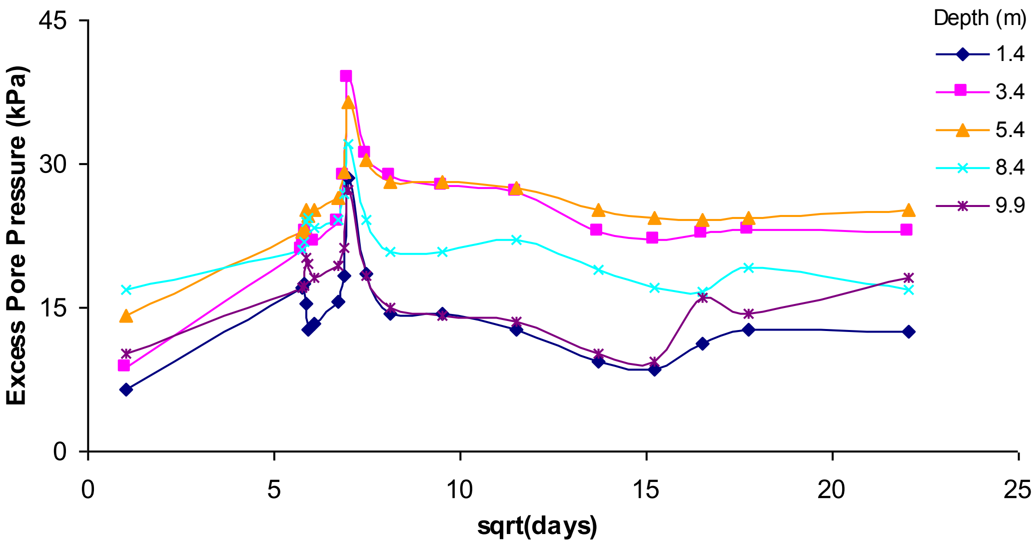

Figure 4.7: Excess pore pressure readings for untreated GCE2

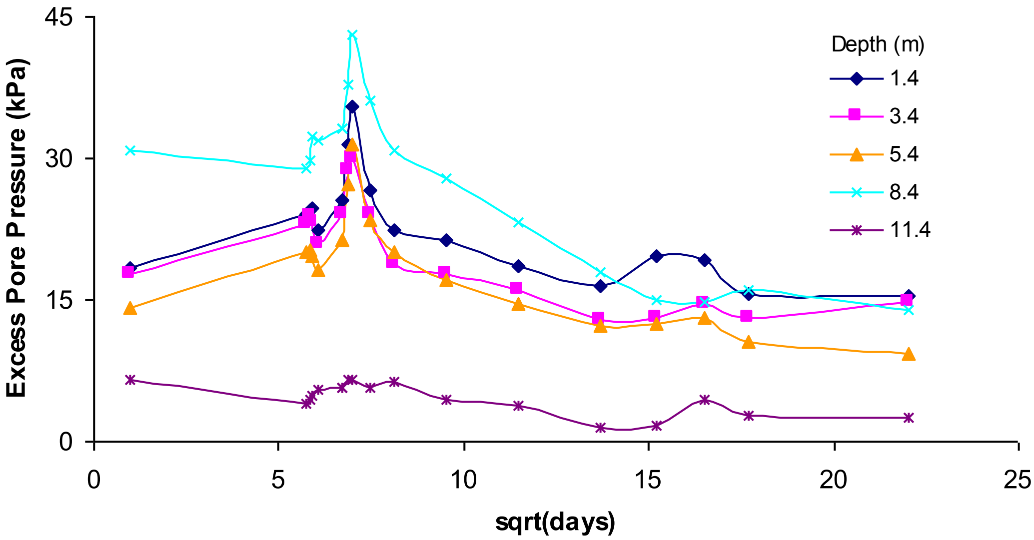

Figure 4.8: Excess pore pressure readings for treated GCE3

4.2.4 Earth Pressure Monitoring

Earth pressure is typically monitored to measure stress distribution in clay. Figure 4.9 shows the readings taken at the Gold Coast Highway site. A peak in pressure occurred when the embankment height was increased. This higher pressure held relatively steady in embankment GCE1, but steadily decreasing with time in embankment GCE3.

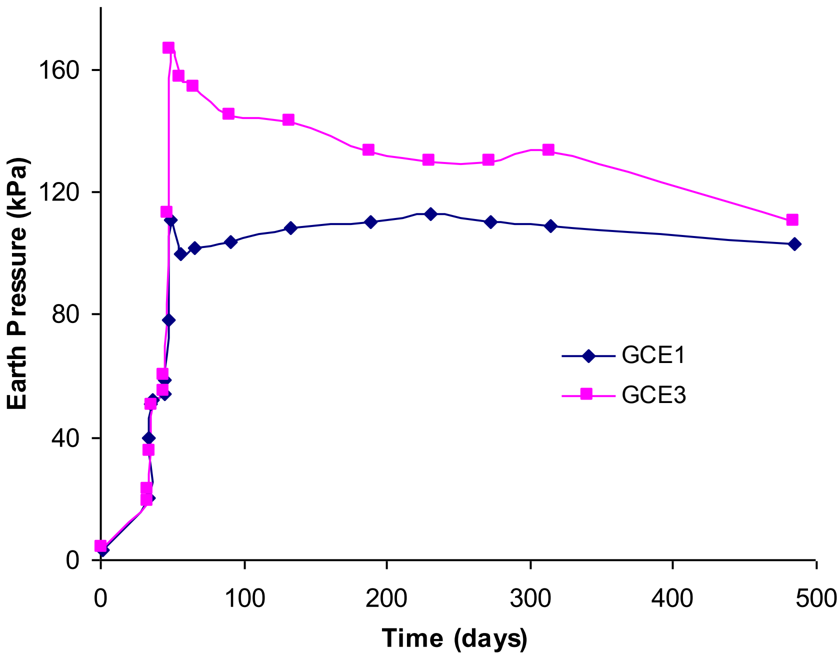

Figure 4.9: Earth pressure readings for treated GCE1 and GCE3

4.3 Case Study 2: Vertical Drain – Sunshine Coast Motorway

In this study trial embankments were designed primarily to investigate two objectives: investigate settlement of soft clay under an imposed embankment height of 3 m, and examine the strength behaviour of the soft clay present at this site.

Two trial embankments were constructed to investigate vertical drain treatment in soft clay, shown in Table 4.3.

Table 4.3: Trial embankments (Sunshine Coast Motorway)

Trial Embankment	Maximum Construction Height (m)	Treatment	Details	
			Spacing (m)	Pattern
SCE1	3	Vertical Drain	1	square
SCE2	3	None	-	

4.3.1 Settlement and Consolidation Monitoring

At the Sunshine Coast Motorway site, horizontal profile gauges and vertical settlement gauges were installed, readings shown in Figure 4.10(b), 4.10(c), 4.11(b) and 4.11(c). The corresponding embankment height profiles are given in Figure 4.10(a) and 4.11(a).

Excess pore water pressure was measured at the Sunshine Coast Motorway site, readings displayed in Figure 4.10(d) and 4.11(d). The peaks in pressure are the result of embankment height increase. Once the final embankment height is reached, the pressure dissipates over time as water is forced to escape, returning to a value approximately equal to that of the original.

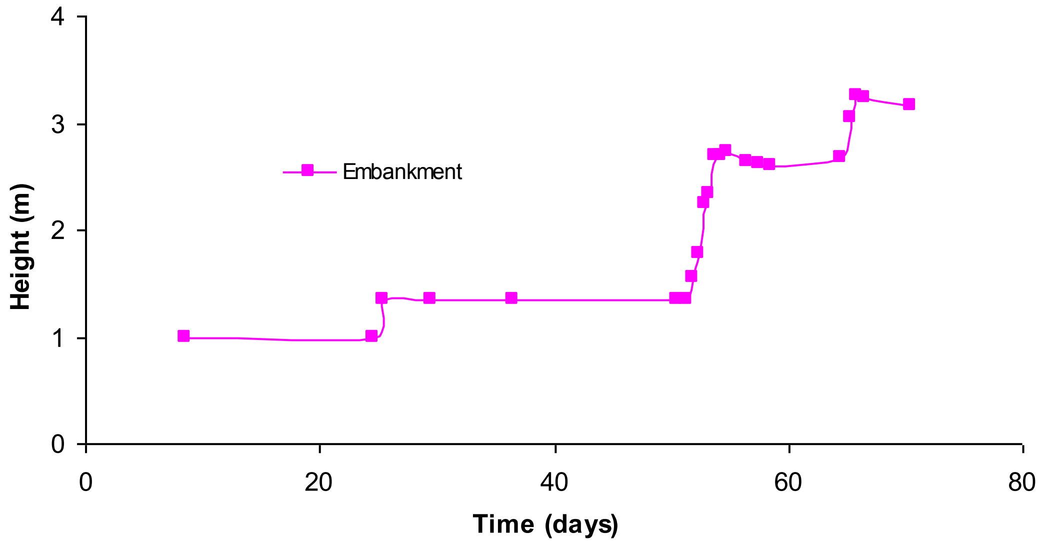

Figure 4.10(a): Embankment height profile for treated SCE1

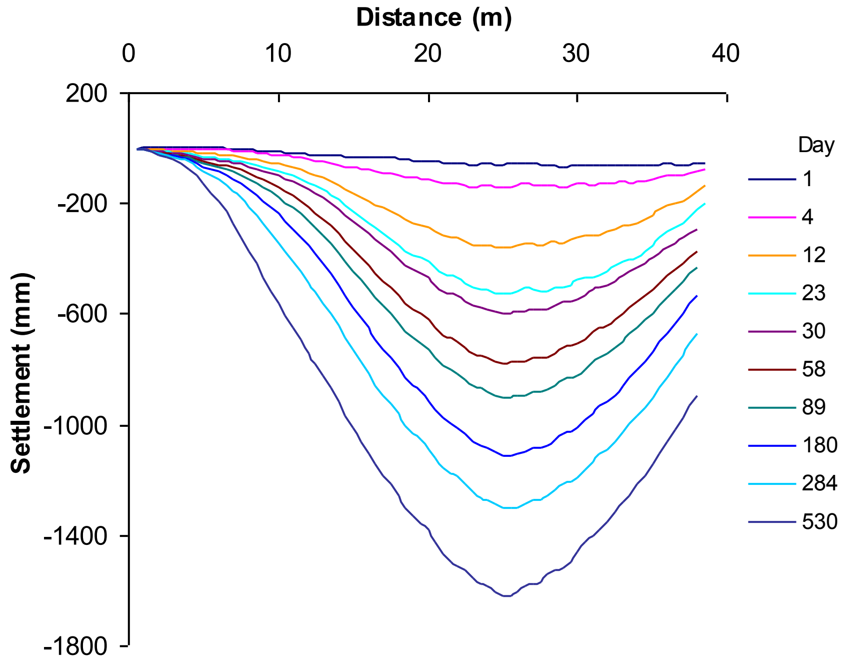

Figure 4.10(b): Measured settlement for treated SCE1

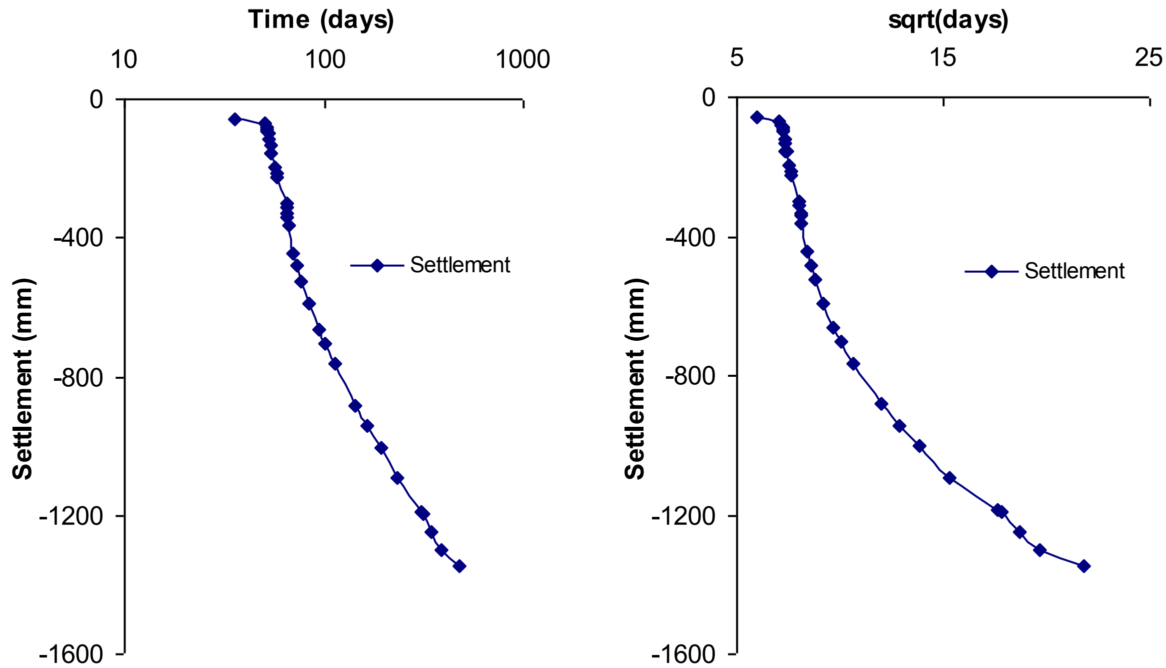

Figure 4.10(c): Measured settlement for treated SCE1

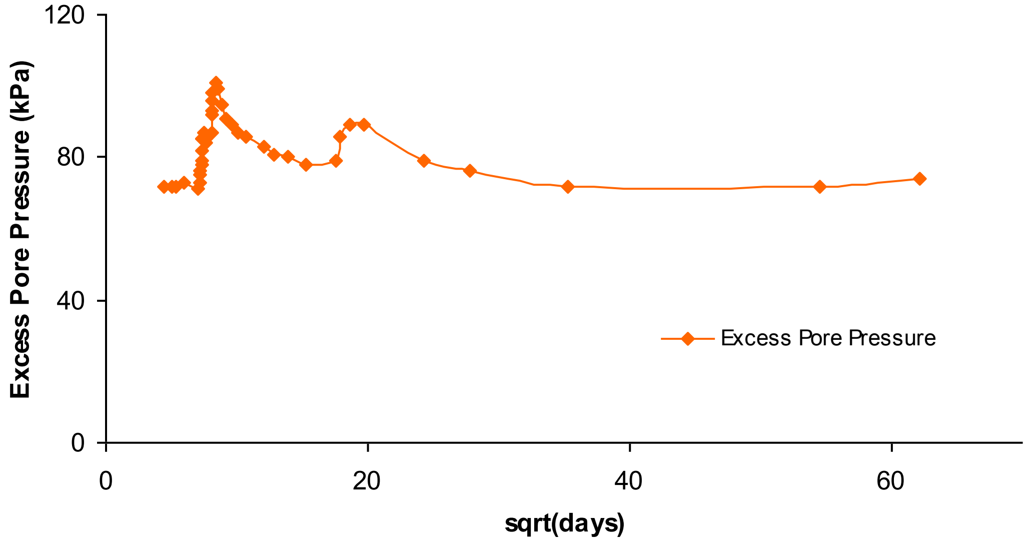

Figure 4.10(d): Excess pore pressure readings for treated SCE1

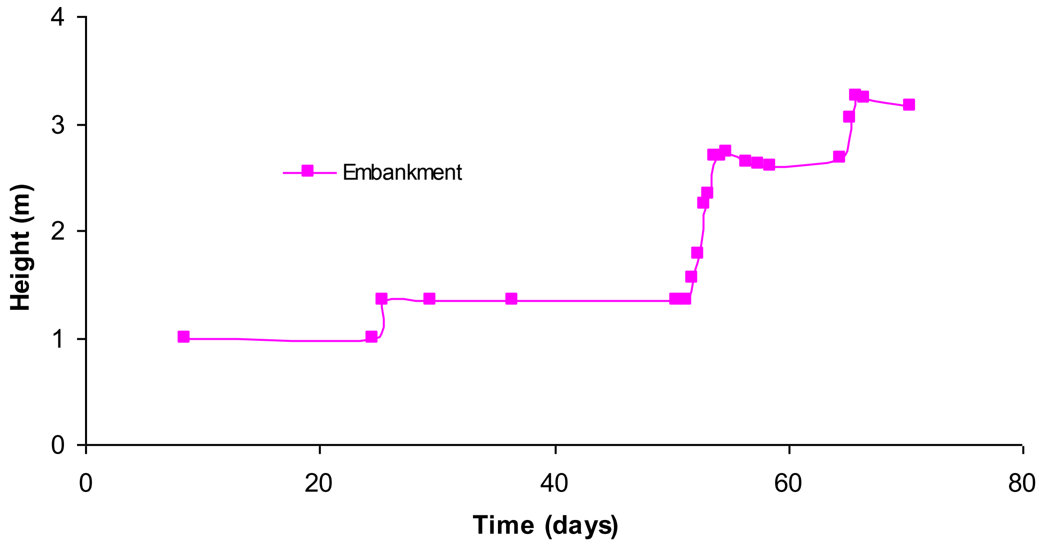

Figure 4.11(a): Embankment height profile for untreated SCE2

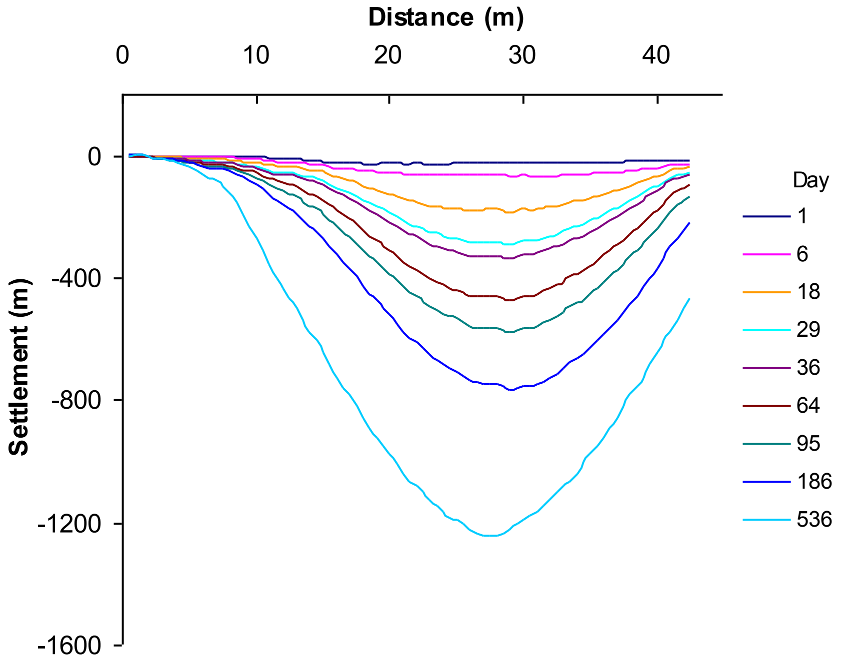

Figure 4.11(b): Measured settlement for untreated SCE2

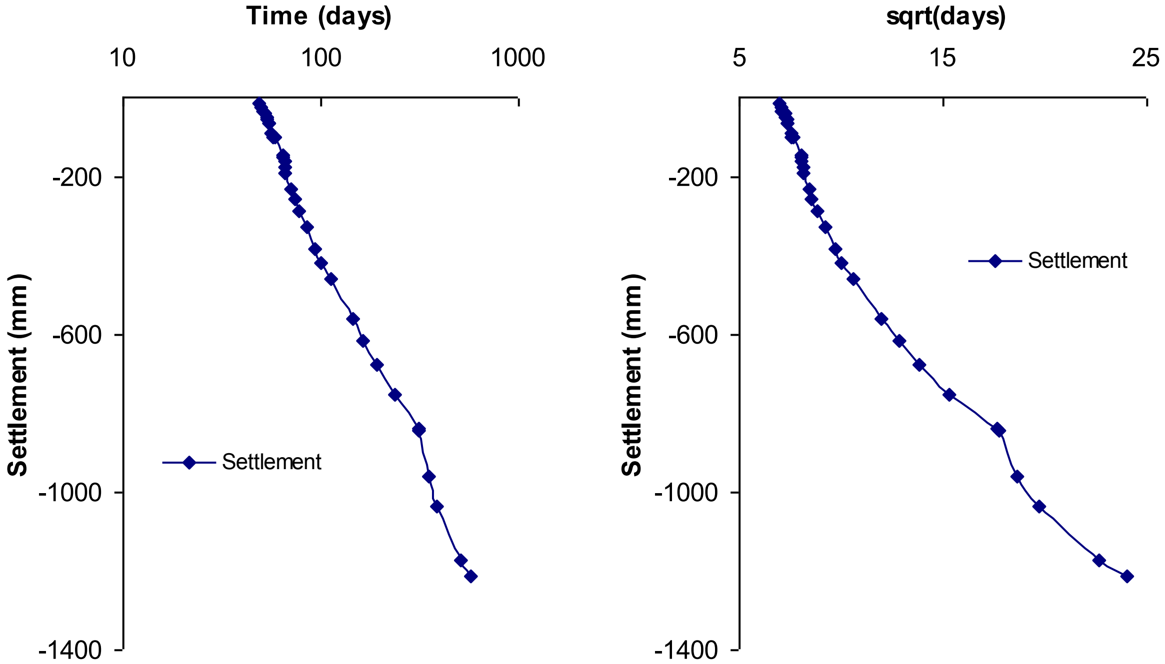

Figure 4.11(c): Measured settlement for untreated SCE2

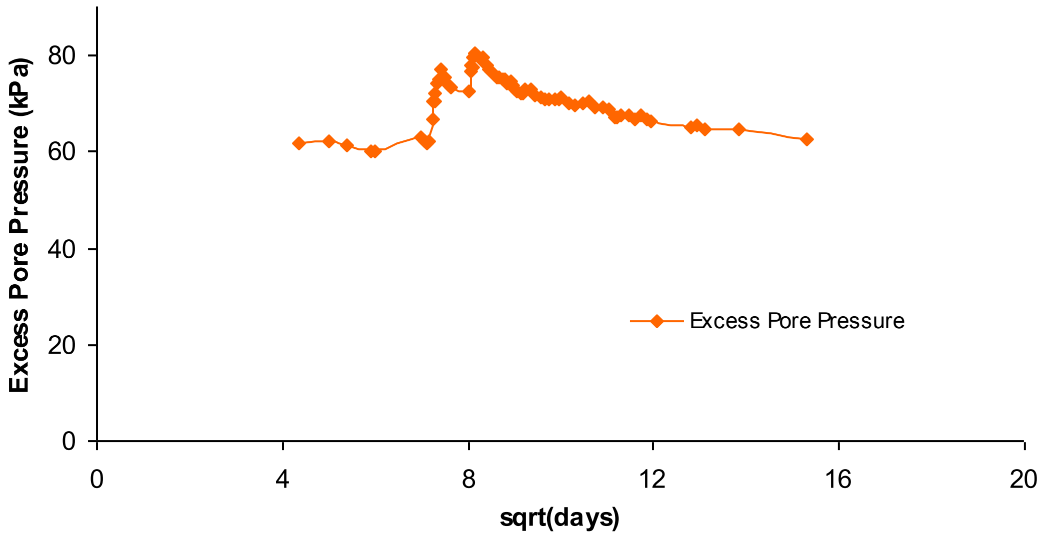

Figure 4.11(d): Excess pore pressure readings for untreated SCE2

The final settlement obtained after 536 days of monitoring is shown in Table 4.4. It is seen that the installation of vertical drains induced a settlement increase of approximately 25%. This increase in settlement suggests that the rate of settlement increased. This is verified when Figure 4.10(c) is compared with Figure 4.11(c). In approximately half the time SCE1 achieved settlement equal to the maximum settlement in SCE2.

Table 4.4: Summary of settlement results (Sunshine Coast Motorway)

Trial Embankment	Initial Gauge Reading (mm)	Final Gauge Reading (mm)	Settlement (mm)
SC_E1	54.73	1619.65	1564.92
SC_E2	24.65	1221.82	1197.17

It is suggested from the settlement results that vertical drain installation reduced consolidation time. This is confirmed by consolidation results shown in Figure 4.12. SCE1 reached an 80% degree of consolidation, while SCE2 only reached 50%. A 60% reduction in consolidation time is seen, indicating the vertical drains dramatically decreased consolidation time. Furthermore, it is suggested that it would take over 4000 days for SCE2 to reach 80% degree of consolidation, approximately 4 times that required by SCE1.

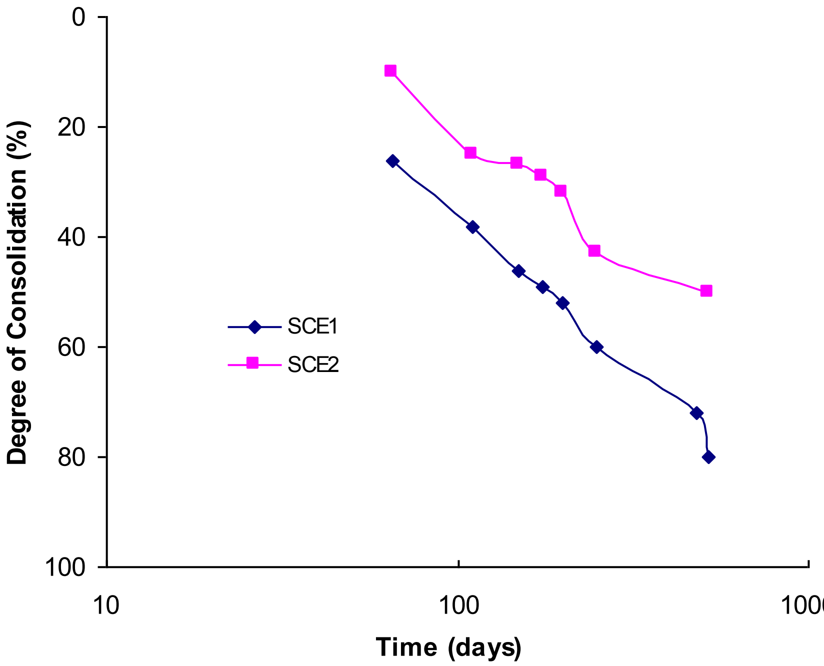

Figure 4.12: Degree of consolidation (Sunshine Coast Motorway)

4.3.2 Relationship Between Pore Pressure and Vertical Stress

Figure 4.13 shows the relationship between pore pressure and vertical stress for the Sunshine Coast Motorway site. Both profiles begin horizontal, as pore pressure doesn't increase until embankment height is raised. Once embankment height is raised a relatively uniform relationship is formed. It can be seen that a greater vertical stress was experienced in SCE2 for at the same pressure.

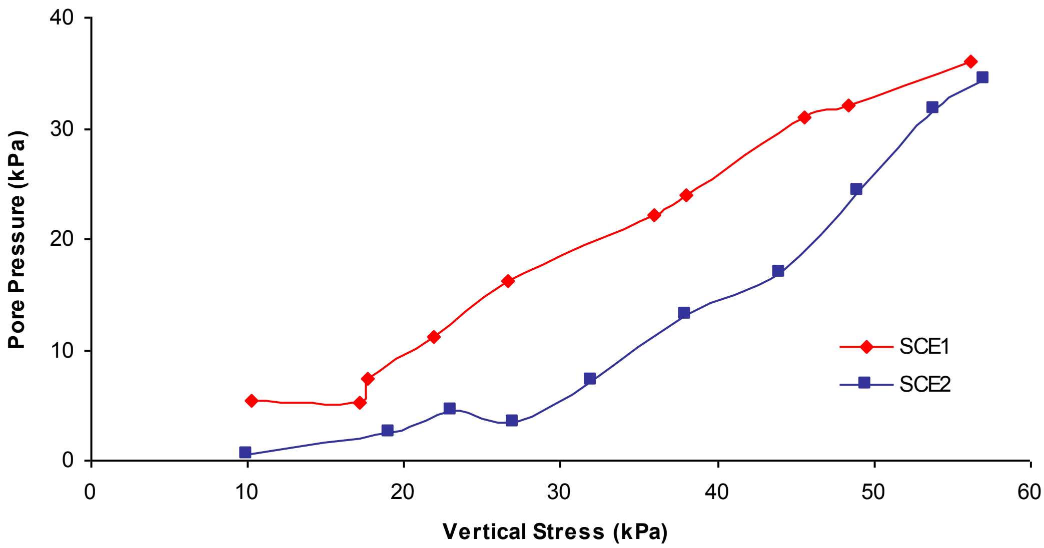

Figure 4.13: Relationship for pore pressure and vertical stress (Sunshine Coast Motorway)

4.3.3 Earth Pressure Monitoring

Earth pressure was measured to determine stress distribution at the Sunshine Coast Motorway site, readings are given in Figure 4.14 and 4.15. An increase in pressure occurs when the embankment height is raised. The readings for embankment SCE2 were terminated at day 80 because the cells were damaged.

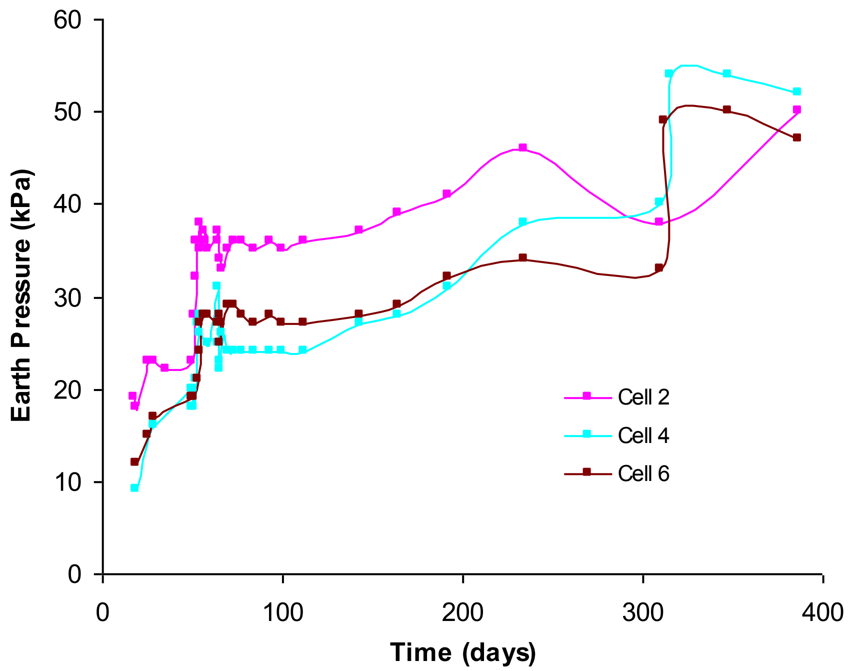

Figure 4.14: Earth pressure readings for treated SCE1

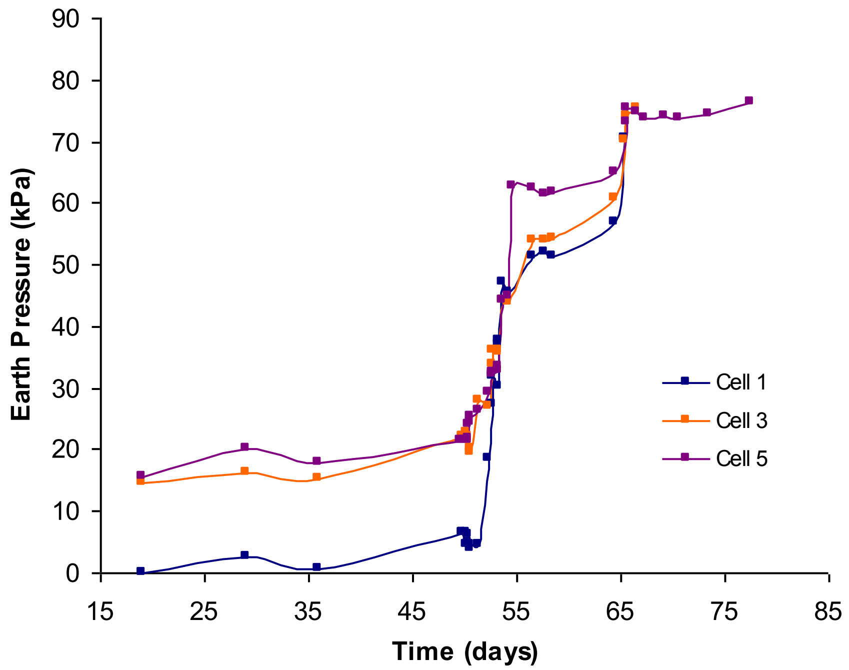

Figure 4.15: Earth pressure readings for untreated SCE2

4.3.4 Horizontal Displacement Monitoring

Horizontal displacement was monitored at embankment toe and centreline for the Sunshine Coast Motorway site, measurements shown in Figure 4.16 and 4.17. The maximum displacement at the toe was 0.73 mm, and 2.91 mm at the centreline. It is shown that vertical drains dramatically reduced toe horizontal displacement, while slightly reducing centreline displacement.

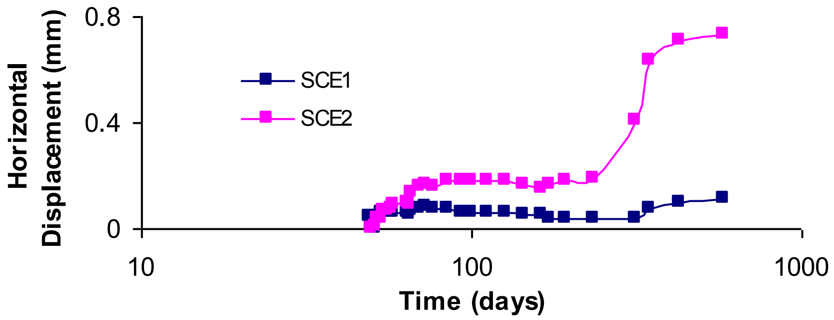

Figure 4.16: Horizontal displacement at toe (Sunshine Coast Motorway)

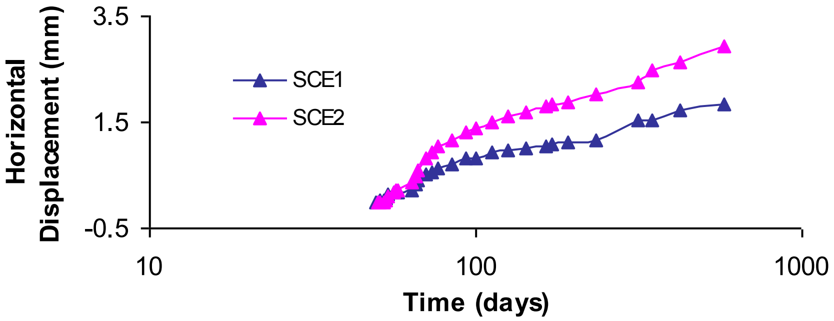

Figure 4.17: Horizontal displacement at centreline (Sunshine Coast Motorway)

4.4 Case Study 3: Vertical Drain - Port of Brisbane Motorway

Trial embankments were constructed to investigate two objectives: investigate soft clay settlement under imposed embankment height of 5 to 6 m, and examine strength behaviour of the soft clay present at this site.

Five trial embankments were constructed to investigate vertical drain treatment in soft clay, shown in Table 4.5.

Table 4.5: Trial embankments (POB Motorway)

Trial Embankment	Chainage (m)	Maximum Construction Height (m)	Treatment	Details	
				Spacing (m)	Pattern
POBE1	795	5	None	-	
POBE2	1112	5	Vertical Drain	1.5	triangular
POBE3	1175	6	Vertical Drain	1.5	triangular
POBE4	1210	6	Vertical Drain	1.5	triangular
POBE5	1333	6	None	-	

4.4.1 Settlement and Consolidation Monitoring

At the Port of Brisbane Motorway site, embankment height, settlement and excess pore water pressure was monitored, readings given in Figures 4.18 to 4.22. Initial peaks seen in the pressure plots occur when embankment height is raised. This increase in pressure dissipates over time as water is forced to escape, returning to original values.

POBE1

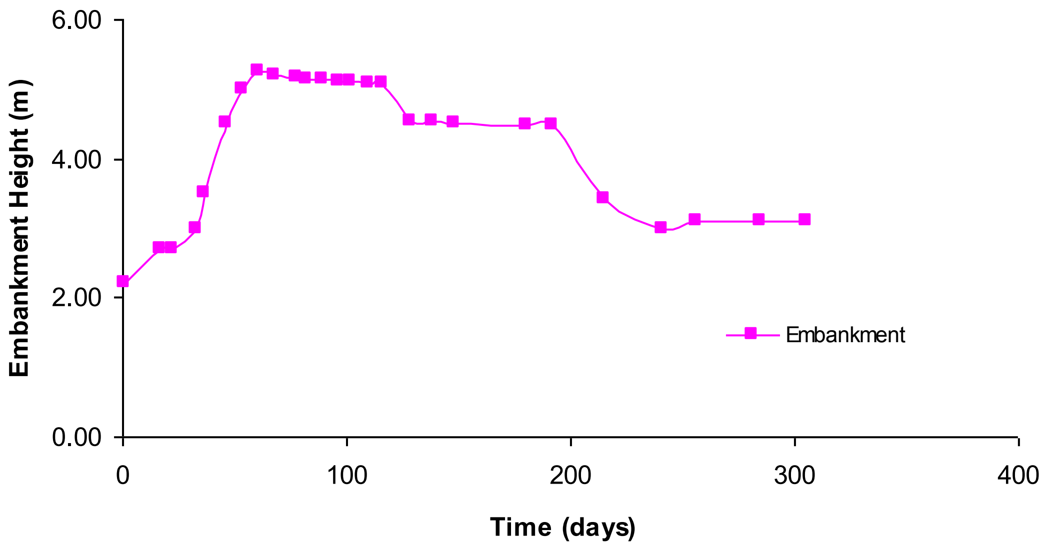

Figure 4.18(a): Embankment height profile for untreated POBE1

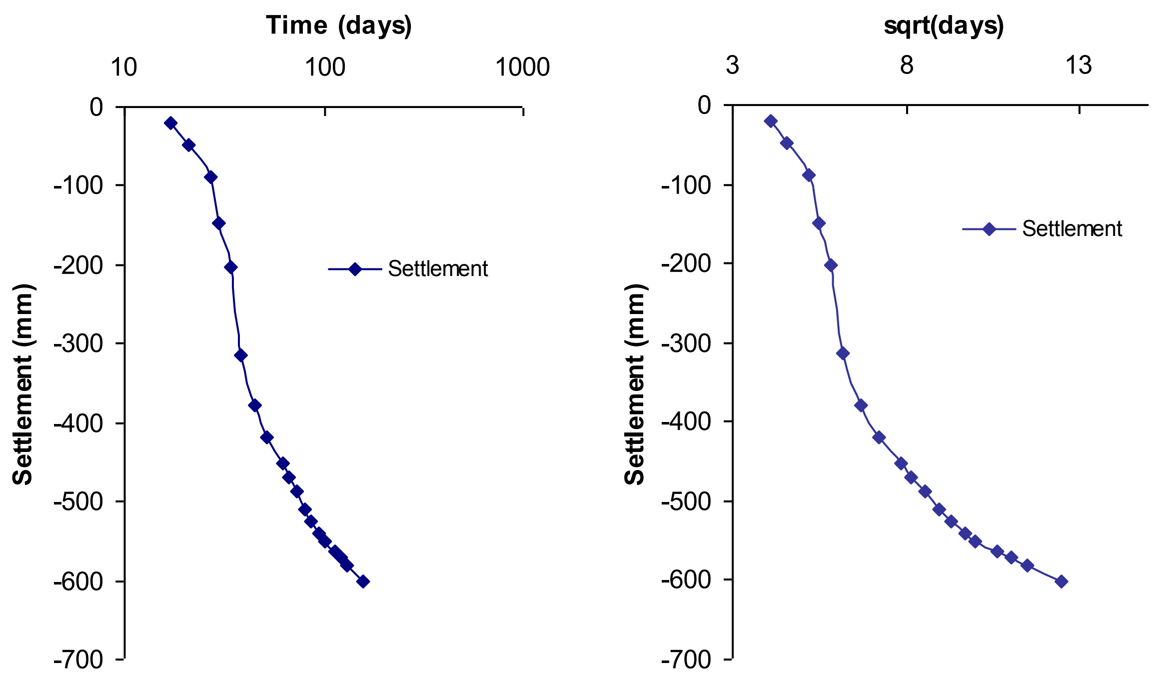

Figure 4.18(b): Measured settlement for untreated POBE1

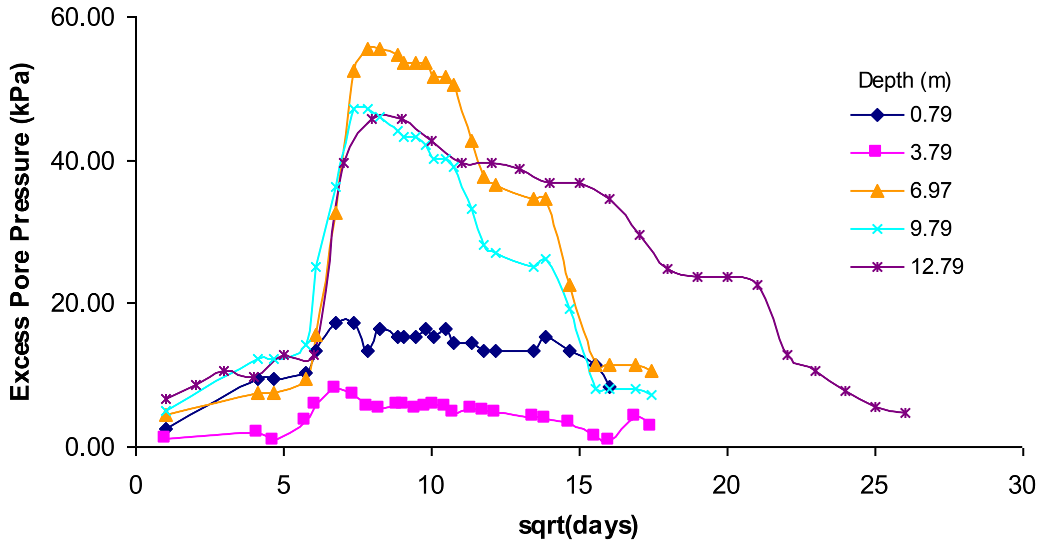

Figure 4.18(c): Excess pore pressure readings for untreated POBE1

POBE2

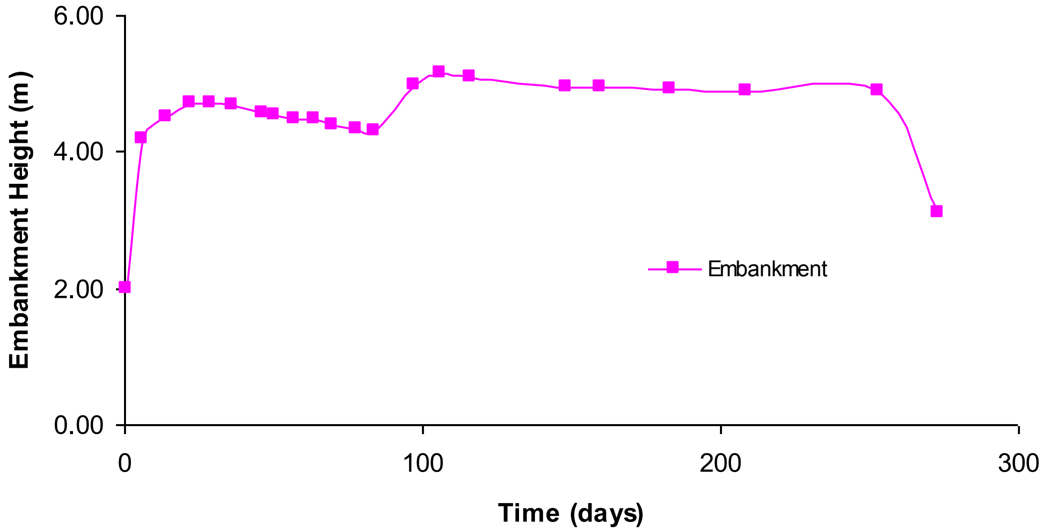

Figure 4.19(a): Embankment height profile for treated POBE2

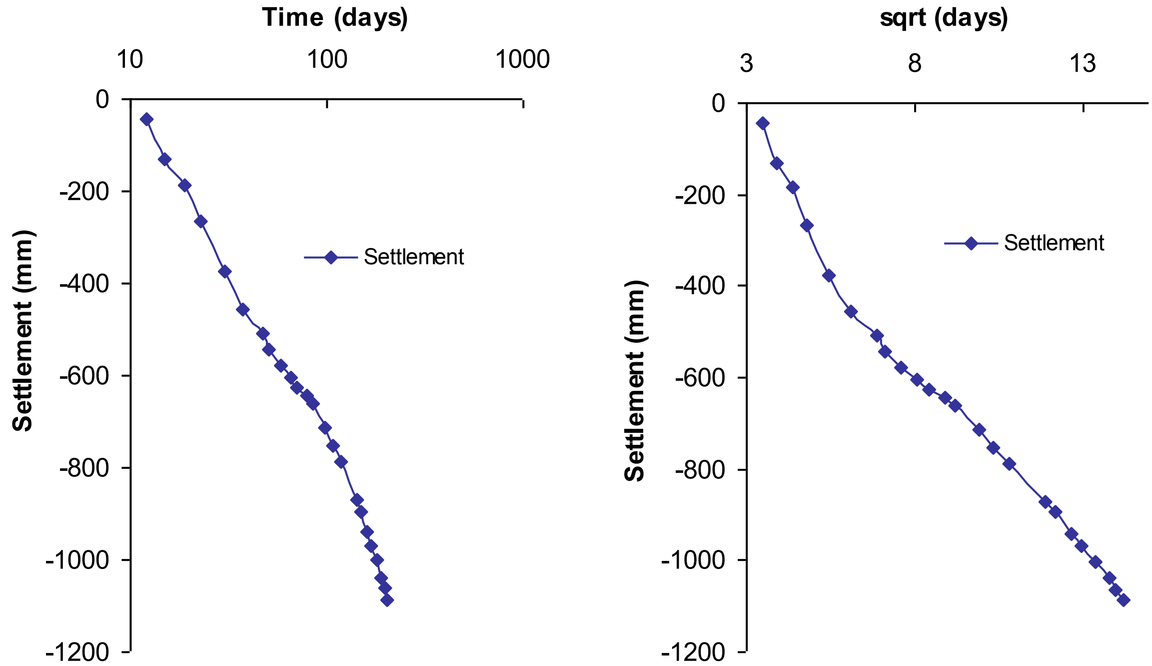

Figure 4.19(b): Measured settlement for treated POBE2

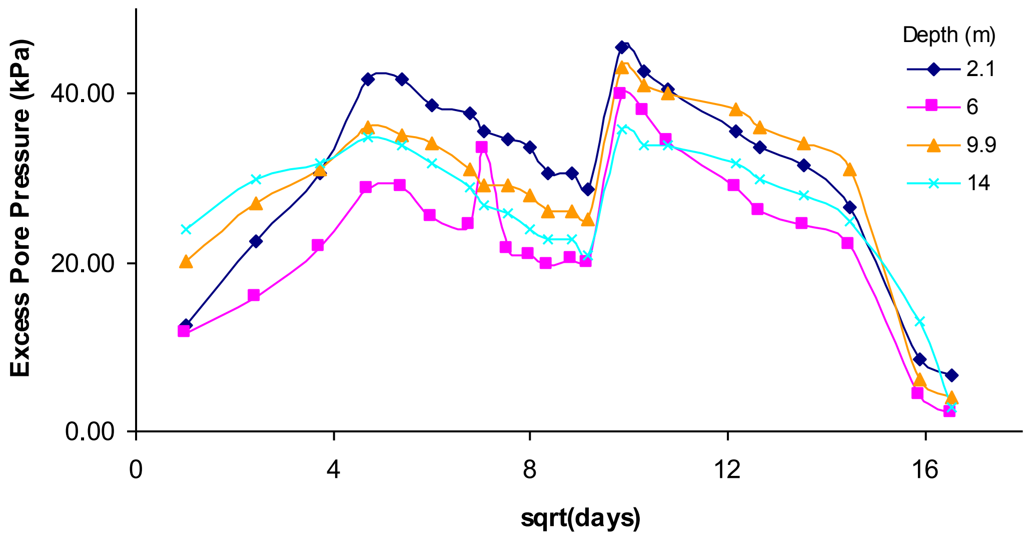

Figure 19(c): Excess pore pressure readings for treated POBE2

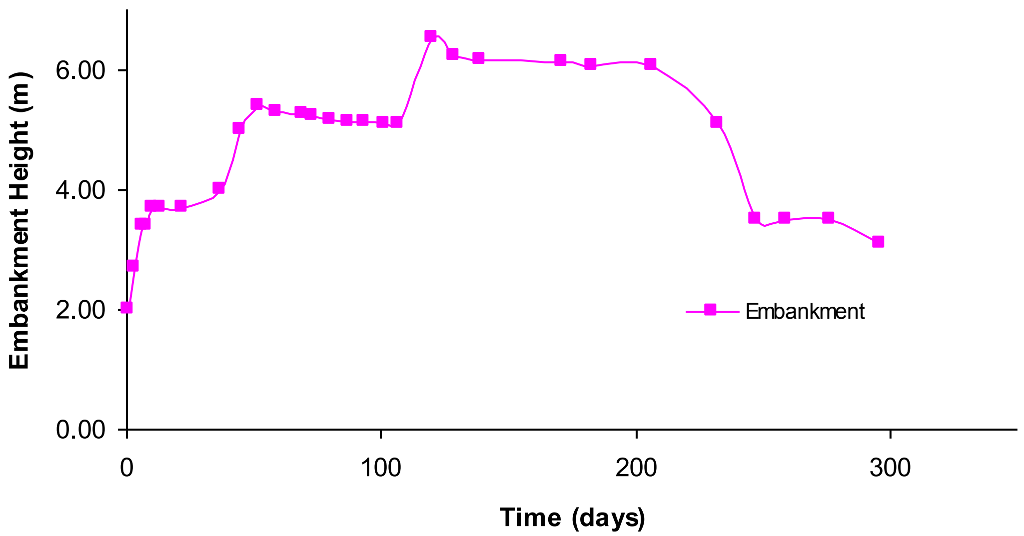

Figure 4.20(a): Embankment height profile for treated POBE3

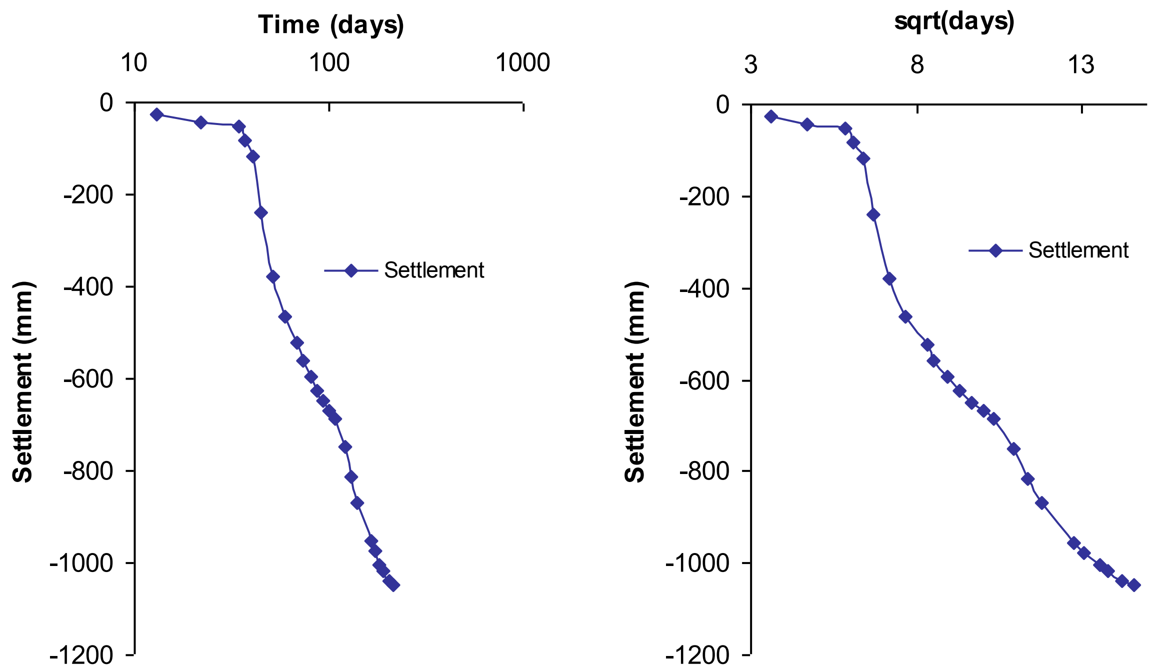

Figure 4.20(b): Measured settlement for treated POBE3

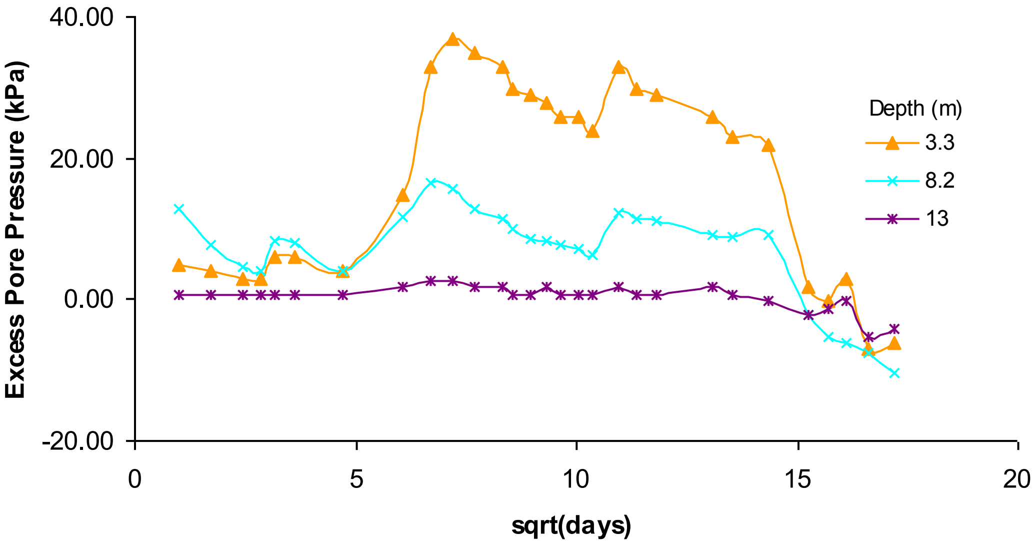

Figure 4.20(c): Excess pore pressure readings for treated POBE3

POBE4

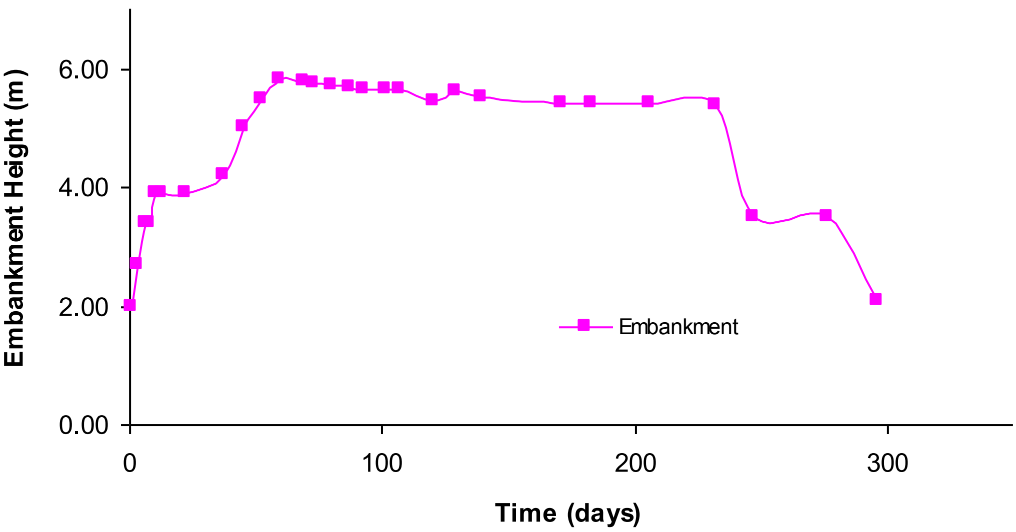

Figure 4.21(a): Embankment height profile for treated POBE4

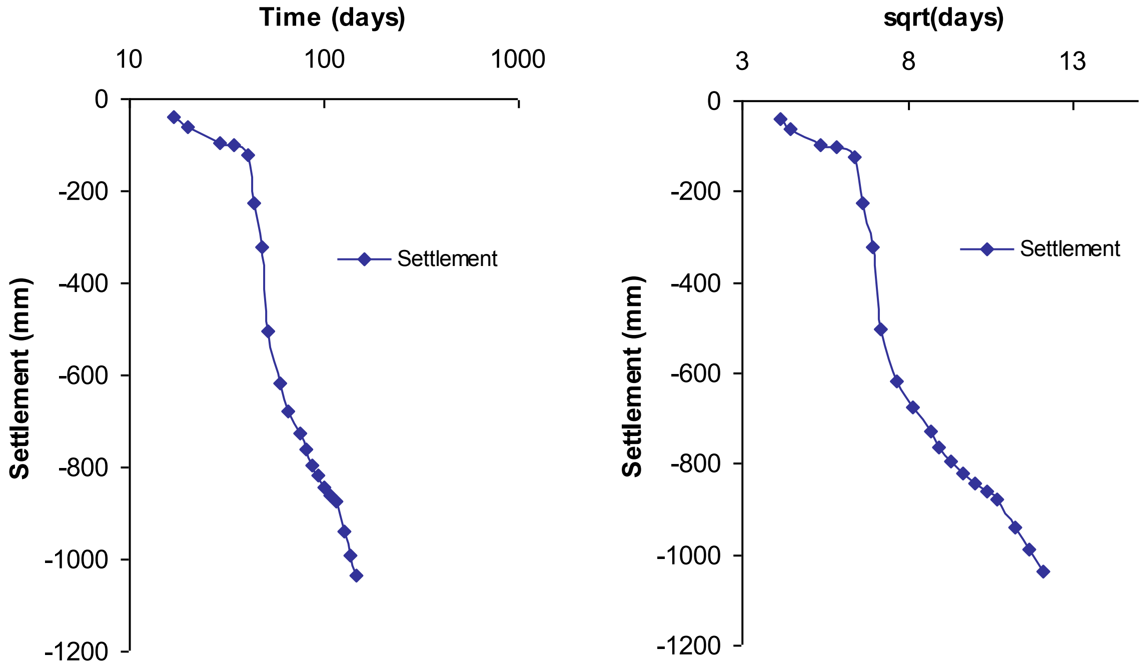

Figure 4.21(b): Measured settlement for treated POBE4

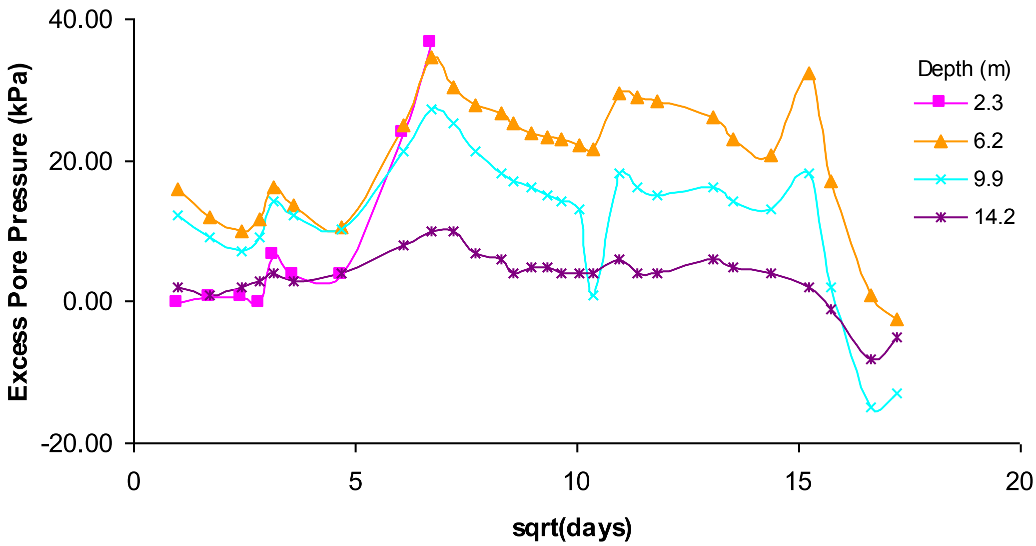

Figure 4.21(c): Excess pore pressure readings for treated POBE4

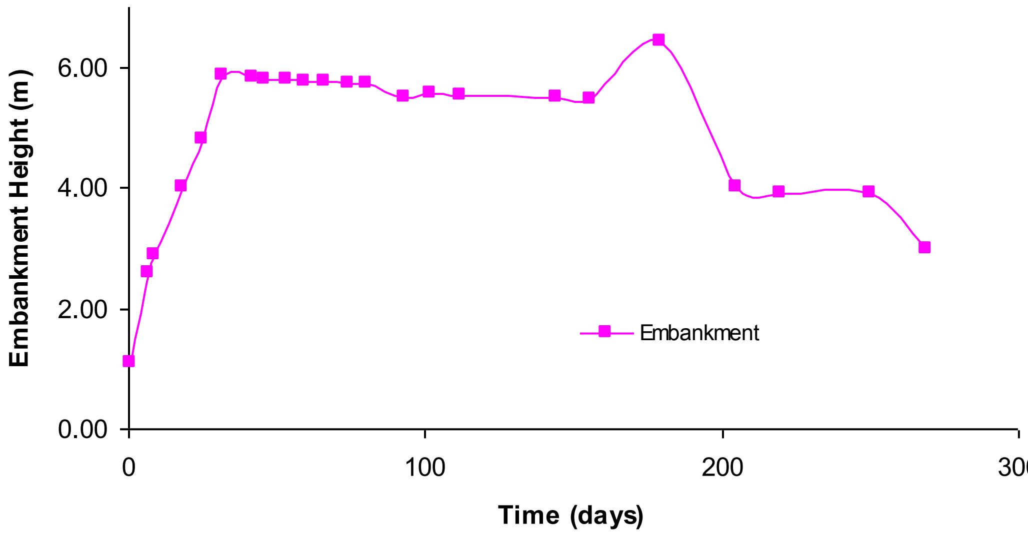

Figure 4.22(a): Embankment height profile for untreated POBE5

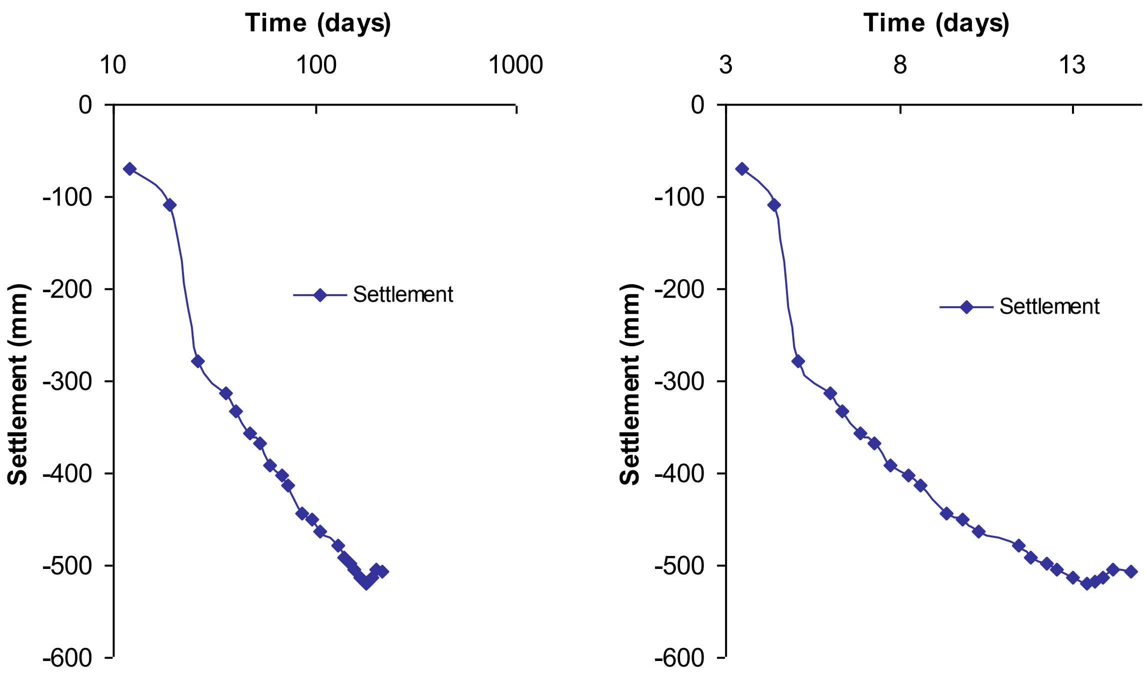

Figure 4.22(b): Measured settlement for untreated POBE5

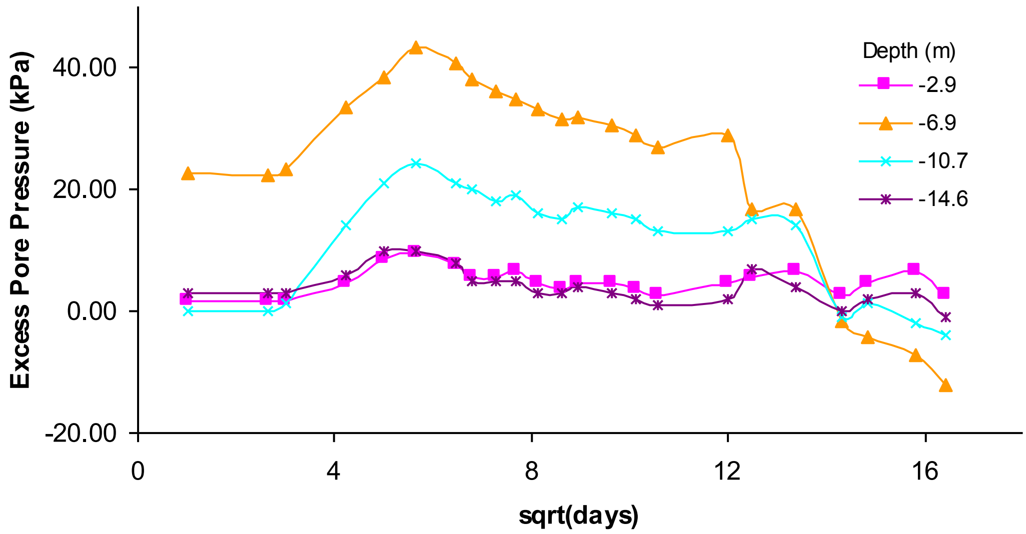

Figure 4.22(c): Excess pore pressure readings for untreated POBE5

The maximum settlement obtained after 226 days of monitoring is shown in Figure 4.23 and Table 4.6. It can be seen that vertical drain treatment significantly increased final settlement. This increase varied from 70 to 80%. It can be assumed from the settlement results that vertical drains would have increased rate of consolidation.

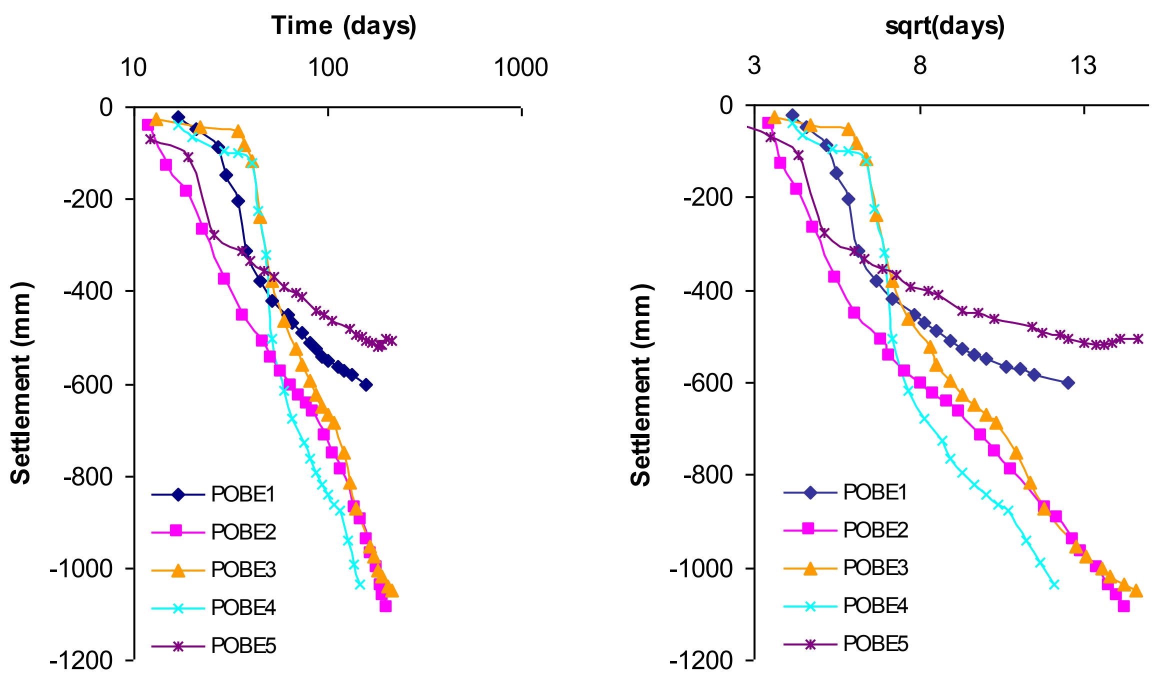

Figure 4.23: Summary of settlement results (POB Motorway)

Table 4.6: Summary of settlement results (POB Motorway)

Trial Embankment	Initial Gauge Reading (mm)	Final Gauge Reading (mm)	Settlement (mm)
POB_E1	0	601	601
POB_E2	0	1087	1087
POB_E3	0	1049	1049
POB_E4	0	1036	1036
POB_E5	0	507	507

4.5 Closing Remarks

In this chapter, data from the monitoring of ten trial embankments constructed at three separate sites: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway, was analysed. A summary of this analysis is given in Table 4.7. Conclusions and recommendations are provided in Chapter 5.

Table 4.7: Summary of all trial embankments

Case Study	Trial Embankment	Treatment Type	Installation Details	Maximum Embankment Height (m)	Maximum Settlement (mm)
1	GC_E1	Stone Column	1 m Dia. @ 3 m Spacing (square grid)	2	508
	GC_E2	None	-	2	508
	GC_E3	Stone Column	1 m Dia. @ 2 m Spacing (square grid)	2	450
2	SC_E1	Vertical Drain	1 m spacing (square grid)	3	1565
	SC_E2	None	-	3	1197
3	POB_E1	None	-	5	601
	POB_E2	Vertical Drain	1.5 m spacing (triangular grid)	5	1087
	POB_E3	Vertical Drain	1.5 m spacing (triangular grid)	6	1049
	POB_E4	Vertical Drain	1.5 m spacing (triangular grid)	6	1036
	POB_E5	None	-	6	507

Chapter 5

Conclusions and Recommendations

5.1 Conclusions

This thesis provides valuable insight into the lab and field behaviour of soft clay, especially that found in South East Queensland. The major emphasis was in the acquisition and presentation of data collected from the extensive monitoring of ten trial embankments constructed at three case study sites: Gold Coast Highway, Sunshine Coast Motorway and Port of Brisbane Motorway.

Analysis of data determined the effectiveness of two ground improvement schemes, vertical drain with surcharge, and stone column treatment. The following findings were concluded:

1. Highly sensitive clay with low strength exists at all three sites. The majority of clay tested was classified as 'inorganic clay of high plasticity'. It was determined that ground improvement was required at all sites.

2. Stone column treatment at the Gold Coast Highway site was ineffective in reducing settlement. It is considered that installation disturbance coursed to the sensitive clay diminishes the effect of the columns. Also, reduced column spacing had negligible impact on further reducing settlement, again due to installation disturbance. The construction of the 2 m high embankment had limited effect at increasing the rate of settlement. An embankment height of 3 to 4 m is suggested as more appropriate.

3. Since stone column treatment is deemed inappropriate for the Gold Coast Highway site, a suspended raft system designed to span the entire length of the swampland is a possible alternative. Financial funding and constructability are the greatest factors of influence when considering this improvement option.

5. Vertical drains were highly effective at increasing the rate of settlement and decreasing consolidation time at both the Sunshine Coast Motorway and Port of Brisbane Motorway sites. A 60% reduction in consolidation time was experienced with drains installed at 1.5 m spacing in a triangular pattern, with an applied surcharge of 3 m.

6. The construction of a 5 to 6 m high embankment at the Port of Brisbane Motorway site had significant effect on increasing the rate of settlement and subsequently reducing consolidation time. However, embankments constructed to this height must be raised in calculated increments to ensure the in-situ clay is not overloaded.

5.2 Recommendations for Future Research

Since consolidation test data is currently being analysed (Ross Pyke - Compressibility characteristics of South East Queensland soft clays), reliable laboratory parameters can be used to estimate theoretical values, and field data is arranged in a clear and logical form, a combined back analyse is possible. This is recommended to be the next phrase for the detailed investigation of clay in South East Queensland.

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APPENDIX

Appendix A: Origin of Data

All laboratory test data, small field test data and large-scaled field test data was obtained from the Queensland Department of Main Roads (QDMR), in the form of the following reports:

(1) Connell Wagner. (1999). Geotechnical review – additional geotechnical investigation requirements: Port Road for Department of Main Roads, *Technical Report D077-709604CB*, Connell Wagner.

(2) Materials and Geotechnical Services Branch. (2000). Coombabah trial embankment instrumentation – Current readings 24/08/00, Queensland Department of Main Roads.

(3) Materials and Geotechnical Services Branch. (2000). Sunshine Coast Motorway: Raw data – Sun_te01 through to Sun_te06, Queensland Department of Main Roads.

(4) Materials and Geotechnical Services Branch. (1995). Gold Coast Highway: Raw data - Results from trial embankment monitoring, Queensland Department of Main Roads.

(5) Materials and Geotechnical Services Branch. (1993). Albert Shire and Gold Coast City – Gold Coast Highway: Embankments East of Coombabah Creek (Ch. 1500m – Ch. 2500m) – Geotechnical Investigation, *Technical Report R1886*, Queensland Department of Main Roads.

(6) Materials and Geotechnical Services Branch. (1993). Gold Coast Highway (Helensvale – Southport): Helensvale interchange to Arundal Drive section – Geotechnical Investigation, *Technical Report R1828*, Queensland Department of Main Roads.

(7) Materials and Geotechnical Services Branch. (1993). Sunshine Motorway stage 2: Final report on the performance of the trial embankment – Area 2A (Ch28490-28640), *Technical Report R1862*, Queensland Department of Main Roads.

(8) Materials and Geotechnical Services Branch. (1992). Sunshine Motorway Stage 2: Interim report on the performance of the trial embankments – Area 2A (Ch28490-28640), *Technical Report R1802*, Queensland Department of Main Roads.

(9) Port of Brisbane Motorway Alliance. (2001). Ground treatment – East of the Oxbow: Concept design report, Port of Brisbane Motorway Alliance.

(10) Transport Technology. (1999). Geotechnical investigation: Port of Brisbane Road (final report) – Volume 2 Appendices A1 to A11, *Technical Report R3146*, Queensland Department of Main Roads.

Appendix B: Prefabricated Vertical Drain Design Factors

Drain Spacing

The theoretical calculation of the maximum drain spacing required to obtain a desired result comes from the long-standing assumption that each drain has an area of influence, defined as a cylindrical soil column surrounding the drain. This column is the same length as the drain with a radius of influence equal to that of the distance to which water can be pushed, or sucked, into the drain in question (Hansbo, 1979). The diameter, D_e, of the dewatered cylinder is dependent on the spacing, *S,* such that it varies from 1.05 times the spacing when the drains are placed in an equilateral triangle grid, to 1.13 times the drain spacing when they are placed in a square grid.

Atkinson and Eldred (1981) illustrated that D_e is calculated on the basis of equivalent cross-sectional areas i.e., for drains on a square grid pattern with a drain spacing of *S,*

(B1)

(B2)

For a triangular grid this becomes

(B3)

(B4)

A square pattern is easier to set out in the field, especially on sites where surveying is difficult. However, a triangular pattern is usually preferred as it provides more uniform consolidation between drains (Rixner *et al*, 1986).

Equivalent Diameter

Kjellman (1948), the first to use band-shaped drains in the form of cardboard wicks, stated that in deep drainage the effect of a drain relies on the contact area between the drain and the soil, and not directly the drain's cross-sectional area. He abandoned the circular cross-section and adopted the band-shaped drain. He found that the band-shaped drain was nearly as effective as a circular drain with the same circumference.

For analysis, a band-shaped drain can be converted into a circular cylindrical drain with the same effect on the consolidation process by (Hansbo *et al*, 1981):

(B5)

where, d_w is the equivalent diameter of a band-shaped drain with width b and thickness t.

Therefore for PVDs with a width of 95-100mm and a thickness of 3-6mm, the standard size range for band-shaped drains, the equivalent diameter will be equal to 62-67mm.

Subsequent finite element studies performed suggest that it may be more appropriate to use (Rixner *et al*, 1986):

(B6)

for conventional band-shaped drains having a ratio b/t of approximately 60 or less.

Well Resistance

Hansbo (1979) recognized that in reality a drain with infinite permeability in the longitudinal direction and having an infinite discharge capacity, does not exist. The relative influence of well resistance depends on the drain diameter and drain spacing, the n value, and on the length of the drains and the ratio k_h/q_w.

The influence of well resistance according to Hansbo (1981) is:

(B7)

where: z = distance from the drainage end of the drain
k_h = permeability coefficient of undisturbed soil in horizontal flow
k'_h = permeability coefficient in the smeared zone
l = drain length

Smear Zone

Bergado *et al* (1992) using large consolidometer apparatus found that the assumption of the diameter of the smeared zone being three times the equivalent diameter of the mandrel, to be over conservative. He suggested that the assumption of the diameter of the smear zone being twice the equivalent diameter of the mandrel would be more appropriate.

Using two different sizes of mandrel in the field, a faster settlement rate and slightly higher compression were observed in the smaller mandrel area than in the larger mandrel area, indicating a smaller smeared zone in the former than in the latter.

Hansbo (1981) represents the effect of drain spacing and smear as:

(B8)

where:

(B9)

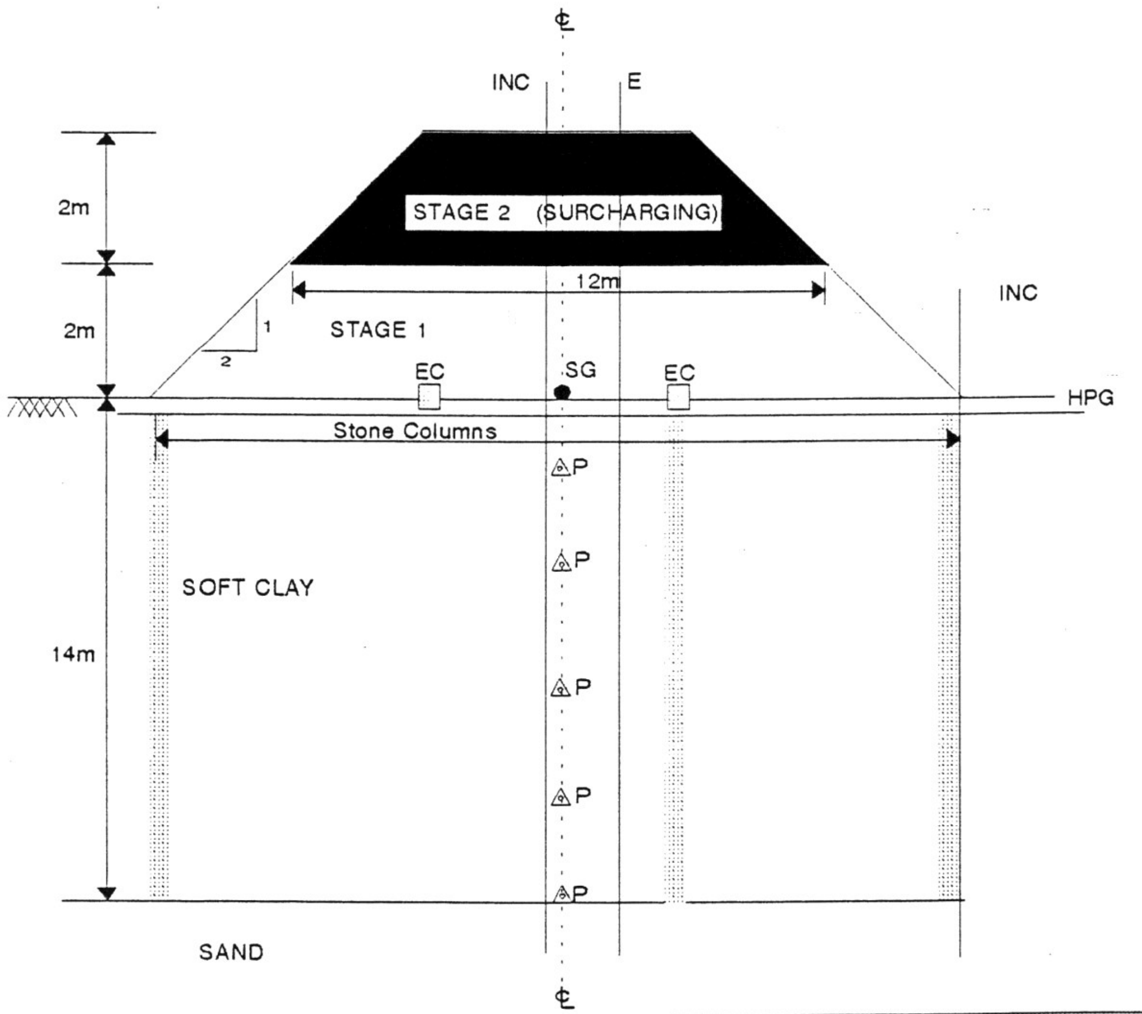
(B10)

n = spacing ratio
D_e = equivalent diameter of soil influencing each drain
d_w = equivalent diameter of drain
d_s = diameter of smeared zone

Appendix C1: Installed Instrumentation for GCE1 and GCE3

INSTRUMENTATION LAYOUT

GCE1 and GCE3



CROSS-SECTION at Y-Y & Z-Z

LEGEND
E - Extensometer
EC - Earth Pressure Cell
INC - Inclinometer
P - Piezometer
SG - Settlement Gauge
HPG - Horizontal Profile Gauge

(NOT TO SCALE)

Appendix C2: Installed Instrumentation for GCE2