Preloading and prefabricated vertical drains design for foreshore land reclamation projects: a case study

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The Changi East Reclamation Project in Singapore comprises land reclamation in foreshore conditions. As the foreshore area was underlain by a thick layer of soft marine clay, a high magnitude of primary and secondary consolidation settlement was expected, owing to fill and future live load. An average fill thickness of 10 m was predicted to contribute to a magnitude of settlement greater than 2 m over a period of several decades. In order to minimise this expected future settlement, it was necessary to accelerate the consolidation process to complete the majority of the settlement during construction stage. A combination of prefabricated vertical drains (PVDs) with preloading ground improvement was successfully applied in this project. The technique comprises the installation of prefabricated vertical drains and the subsequent placement of surcharge to accelerate the consolidation of the underlying marine clay. This paper discusses the theories of radial drainage and the preloading technique, and considerations and design methodologies for the ground treatment of marine clay with prefabricated vertical drains in such foreshore land reclamation projects. In addition, predictions of magnitude, time rate of settlement and performance assessment of prefabricated vertical drains are also discussed. The design predictions at a case study site were compared with the field instrumentation results in order to verify the design approach used. It is found that PVDs with preloading are the most effective method for improving soft clay under land reclamation, based on the soil conditions present at the case study site.

Keywords: land reclamation; prefabricated vertical drains; preloading; soft clay; soil improvement

Le projet Changi East Reclamation à Singapour porte sur la réhabilitation de terrain dans les conditions d'estran. Comme la zone d'estran repose sur une couche épaisse d'argile marine molle, un tassement des consolidations primaire et secondaire de grande amplitude était présumé, en raison de remplissage et de charge utile future. On estimait qu'une épaisseur de remplissage de 10 m pourrait contribuer à une amplitude de tassement de plus de 2 m sur une période de plusieurs décennies. Afin de minimiser l'étendue du tassement prévue, il était nécessaire d'accélérer le processus de consolidation pour que la majorité du tassement se termine pendant la phase de construction. Ce projet combine avec succès l'utilisation de drains verticaux préfabriqués (DVP) avec amélioration des sols par préchargement. Le technique adoptée consiste à installer des drains verticaux préfabriqués puis à placer une surcharge pour accélérer la consolidation de la couche d'argile marine sous-jacente. Cet article présente une discussion sur les théories de drainage radial et la technique de préchargement, ainsi que des considérations et des méthodologies de conception géotechnique pour la consolidation de l'argile marine dans le cadre d'un tel projet de réhabilitation de terrain dans les conditions d'un estran. Il examine également les prédictions concernant l'amplitude et le taux de tassement en fonction du temps, et les performances des drains verticaux préfabriqués. Les prévisions de conception sur un site d'étude de cas sont comparées avec les résultats d'expérimentation de terrain afin de vérifier l'adéquation de l'approche technique utilisée. En conclusion, la méthode de PVD avec préchargement se montre la plus efficace pour améliorer l'argile molle dans le cadre de réhabilitation de terrain, lorsque l'on se base sur les conditions de sol présentes sur le site d'étude de cas.

Notation

C_h	coefficient of consolidation for horizontal flow
C_{kv}	permeability change index
C_v	coefficient of consolidation for vertical flow
C_{vi}	assumed average value for coefficient of consolidation for vertical flow
$C_{v\,n}$	coefficient of consolidation for vertical flow for layer n
d_e	diameter of equivalent soil cylinder
d_s	diameter of smeared zone
d_w	equivalent diameter of drain
e_0	natural void ratio
$F(n)$	drain spacing ratio function
$F_s(n)$	drain spacing ratio function with smear effect
H'_1	equivalent thickness of layer 1
H_{dri}	equivalent drainage length
H'_{Ti}	total equivalent thickness of all layers
k_h	horizontal hydraulic conductivity of soil

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k_r	coefficient of permeability of remoulded soil = well resistance coefficient
k_w	coefficient of longitudinal permeability of drain
n	drain spacing ratio
s	smear zone ratio
S_t	settlement at time t
S_{ult}	ultimate settlement
T_r	time factor for consolidation by horizontal drainage
T_v	time factor for consolidation by vertical drainage
t	time in years
U_r	average degree of consolidation with respect to radial flow
U_v	average degree of consolidation with respect to vertical flow
U_{vr}	average degree of consolidation with respect to both vertical and radial flows

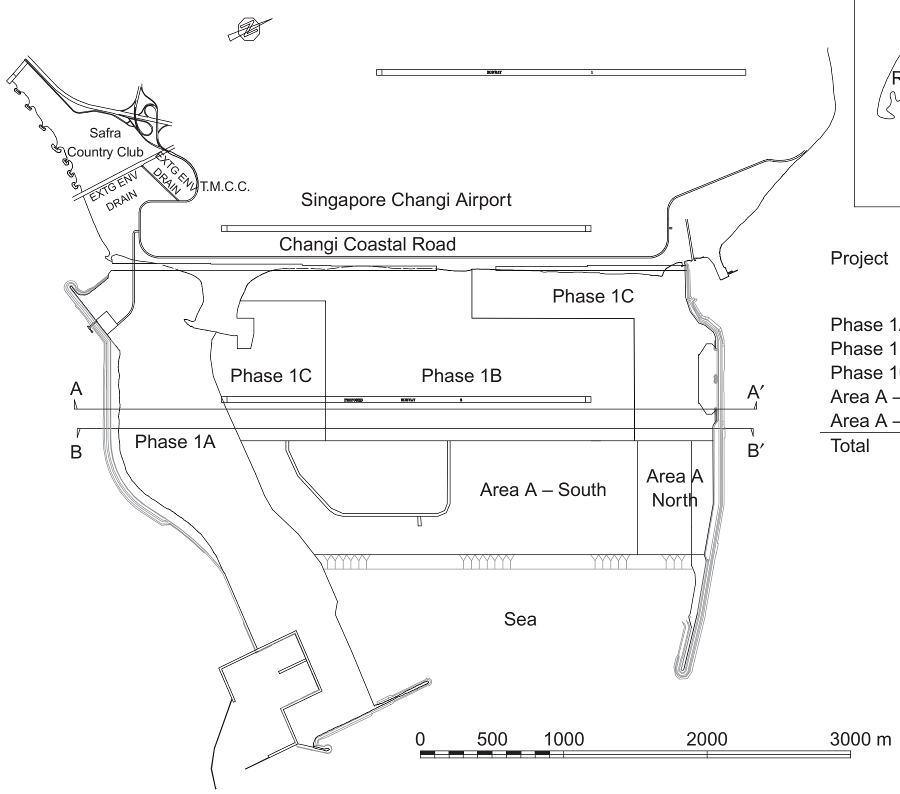
Introduction

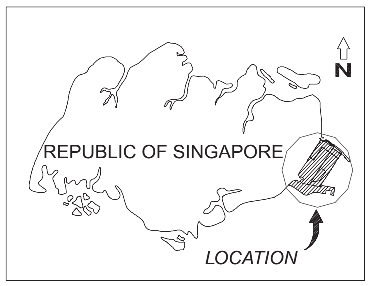
The site for the study is located in the Changi East Reclamation project in the Republic of Singapore (Fig. 1). The project comprises land reclamation and ground improvement works to allow for the future expansion of Changi International Airport, comprising the runway, taxiways, turn-offs and associated airport facilities. The area is located on the foreshore, east of Singapore, with seabed elevation varying from −2 mCD to −8 mCD. (mCD stands for metres chart datum; mean sea level is +1·6 mCD.) Vertical drains were designed for installation in the project site at the platform level of +4 mCD at 1·5 m square spacing. A surcharge placement 6 m in height (+10 mCD) was designed to be carried out. The area is underlain by a thick layer of Singapore marine clay, which is highly compressible and has low permeability. Soil improvement work is required in order to eliminate future settlement caused by fill and future loads. The marine clay characteristics are briefly described in the next section. For further details see Bo *et al.* (1998a, 2003), Arulrajah *et al.* (2004a, 2004b) and Arulrajah (2005).

Ground condition

The area is underlain by two layers of highly compressible marine clay known locally as upper and lower marine clay. These two layers are separated by a thin intermediate layer of desiccated lower marine clay. The upper marine clay is usually about 10 m thick, and the thickness of the lower marine clay can be as much as 30 m. The intermediate clay layer is usually about 1–2 m thick. The lower marine clay is underlain by an old alluvium silty sand layer. The upper marine clay has a moisture content of 60–80%, whereas that of the lower marine clay is 40–60%. The intermediate layer has a low moisture content, of 20–30%. Both the upper and lower marine clay layers are highly plastic, with plasticity indices ranging between 40% and 60%. Both layers are lightly consolidated, with compression indices ranging between 0·6 and 1. The upper marine clay layer has a low coefficient of consolidation of 0·5–0·6 m^2/year, whereas that of the lower marine clay is 0·8–1·5 m^2/year.

The permeability of Singapore marine clay ranges between 10^{-10} and 10^{-9} m/s due to vertical flow and 10^{-9} m/s due to horizontal flow. The permeability anisotropy is about 2. Permeability is generally reduced with a decrease in void ratio. The permeability change index (C_{kv}) is 0·3 of the natural void ratio e_0. Further details of the characteristics of





Project	Area: ha	Length of vertical drain: Mm
Phase 1A	501	–
Phase 1B	520	28
Phase 1C	451	49
Area A – North	91	13
Area A – South	450	50
Total	2086	140

Fig. 1. Site plan

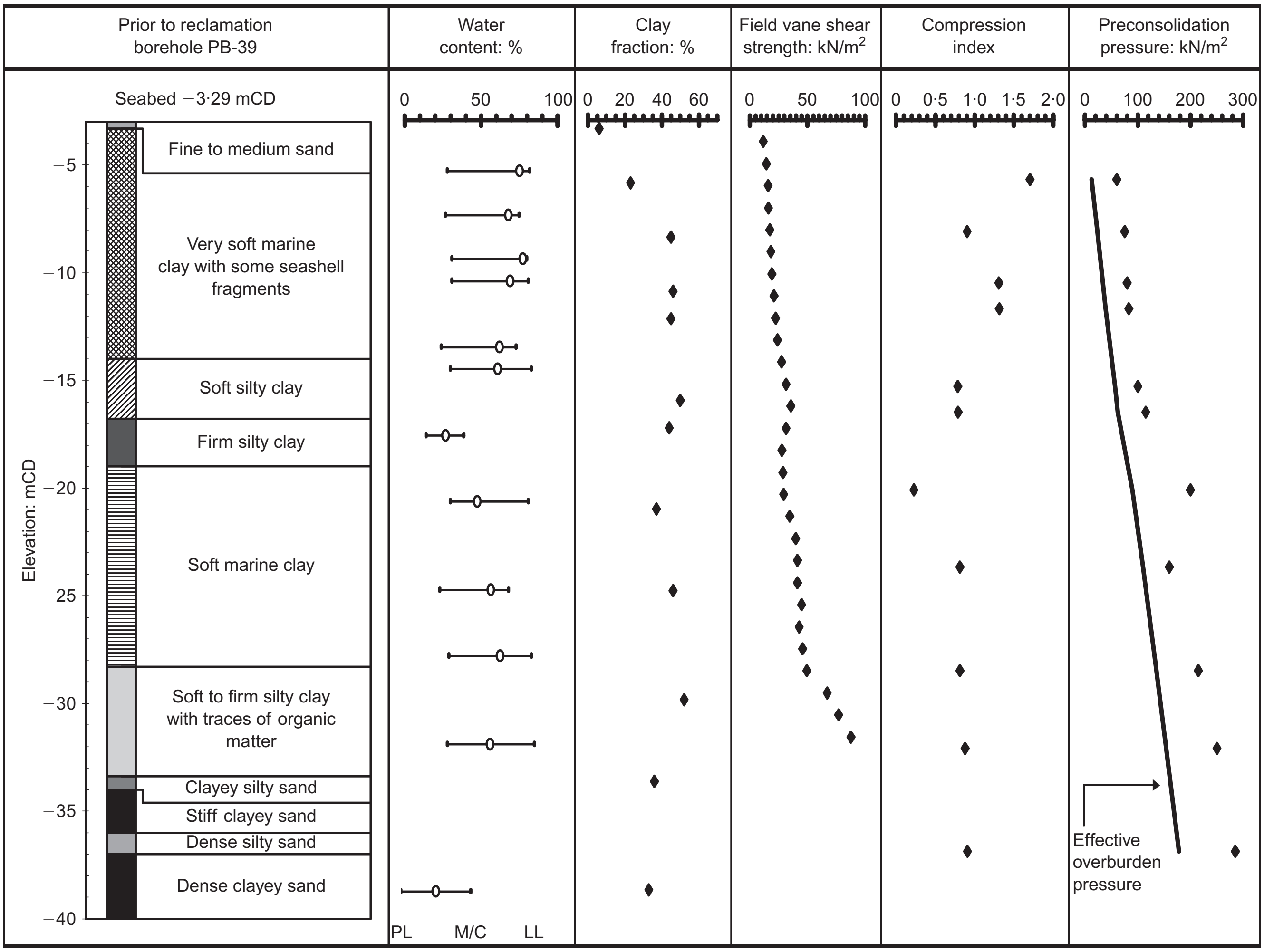


Fig. 2. Typical soil characterisation of study area

Singapore clay (consolidation, compressibility and hydraulic conductivity) can be found in Bo *et al.* (1998, 2003) and Chu *et al.* (2002). Fig. 2 presents a typical profile of geotechnical characteristics, and Fig. 3 shows the permeability profile of the case study area.

Improving soft ground

Soft ground can be improved by several methods, one of the simplest of which is preloading. However, preloading generally provides an average degree of consolidation rather than a homogeneous improvement throughout the entire soil layer. In addition, there is no acceleration process involved when preloading alone is used. An alternative to preloading is to improve the soft soil with prefabricated vertical drains supplemented by preloading. This combination of vertical drains and preloading will enable prestressing and accelerating consolidation of the soft soil and ensure a more homogeneous improvement.

Other suitable methods of improving soft soil include vacuum preloading, lowering of groundwater level, and the electro-osmosis process. Details of these ground improvement methods have been discussed extensively by Bjerrum *et al.* (1967), Esrig (1968), Wade (1976), Mitchell (1981), Hausmann (1990) and Bo and Choa (2004). Each of these methods has its advantages and disadvantages, but the

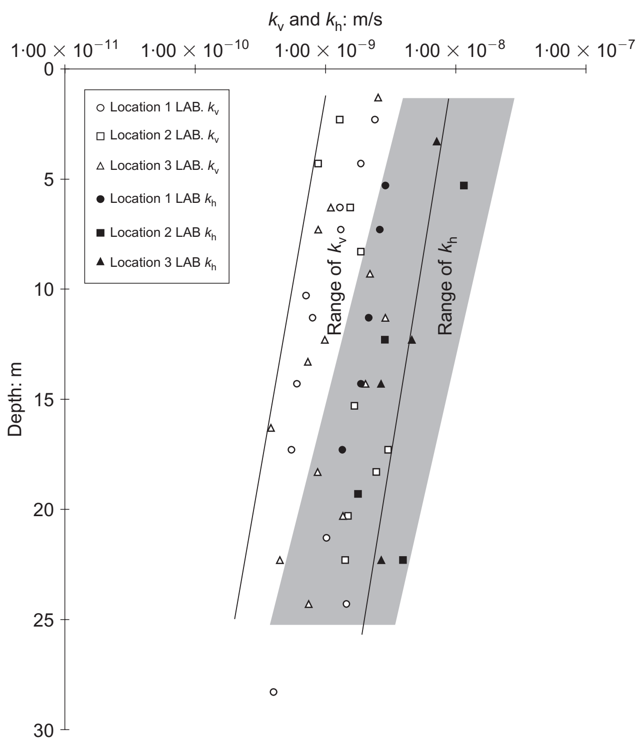


Fig. 3. Horizontal and vertical permeability profiles of the study area

method of preloading in combination with prefabricated vertical drains is well accepted by practising engineers.

Prefabricated vertical drains and preloading

The use of prefabricated vertical drains (PVDs) with preloading was considered the most feasible method, and was used in this project. The objective of the technique is to accelerate the rate of consolidation and to minimise future settlement of the treated area under future dead and live loads. The soil improvement works are carried out in such a way that a specified degree of primary consolidation is attained within the desired time frame by improving the soil drainage system. The primary use of PVDs is to accelerate consolidation to greatly decrease the duration of the consolidation process caused by an embankment built over soft soils. This will ensure that the final construction can be completed in a reasonable time and with minimal post-construction settlement. Preloading increases the effective stress and reduces the compressibility of weak ground by forcing soft soils to consolidate. By doing so, the consolidation process also improves the strength of in situ soft soils.

Design of prefabricated vertical drains

The main variables of the design process are the surcharge magnitude, the spacing of vertical drains, the duration of preloading, the degree of consolidation, and the coefficient of consolidation for horizontal flow (C_h). Conventional methods (Terzaghi, 1925; Carrillo, 1942) were used to predict the magnitude and time rate of consolidation settlement.

Design of surcharge level

The aim of the surcharge placement works is to preload the foundation soil to attain an effective stress that exceeds the pressure due to the design load. Determination of surcharge level is dependent upon the expected settlement and the future load. In a reclamation project where future loading is not required, surcharge is still required to compensate for future settlement due to the fill load. A more acceptable way of surcharging for such a project is to place a thickness equivalent to a multiplier of the predicted settlement to compensate for the shortfall resulting from the remaining degree of consolidation (Bo and Choa, 2004). Generally, 90% degree of consolidation is specified. This is because it takes far longer to complete the final 10% than the first 90%. In this case, a multiplier of 1·1 times the predicted settlement is adopted to compensate for the shortfall resulting from the 10% residual consolidation settlement. Where future loading is required, a preload magnitude equivalent to the future load and 1·1 times the expected settlement with fill and surcharge load is advised. (That is, 100% of 1 m settlement is 1·0 m whereas 90% of 1·1 m settlement is 0·99 m. Therefore this contributes the same magnitude of settlement.) In some cases, even with the closest practicable spacing of PVDs, it may be difficult to reduce the consolidation time to the desired duration. In such cases, an alternative method is to increase the preloading magnitude to 20–30% higher than the net future load and aim for a lower degree of consolidation. This concept is illustrated in Fig. 4.

It is known that improvement with PVDs can eliminate primary consolidation settlement. However, secondary compression can be minimised only by preloading the soil beyond its expected future load, because secondary compression decreases with effective stress.

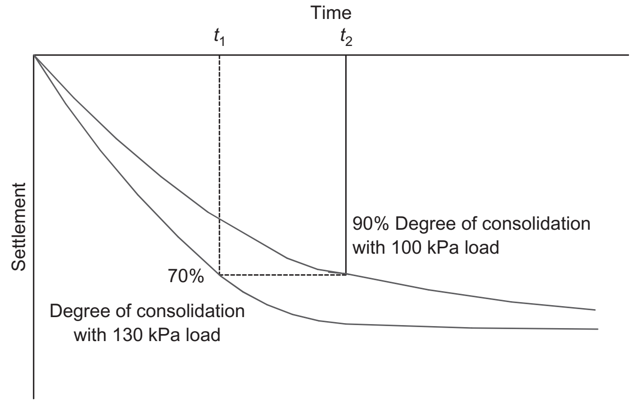


Fig. 4. Concept of effectiveness in application of surcharge

Prediction of settlement

Settlement was predicted using laboratory results from samples collected from offshore investigation boreholes. The compression parameters and stress history were carefully obtained from the results of primary consolidation tests carried out on the undisturbed piston samples. The moisture content and void ratio of soils, which are important parameters for predicting the magnitude of settlement, are obtained after the necessary salt correction. Because of the salt content in the marine deposit, both the moisture content and the void ratio determined by oven-drying in the laboratory are underestimated values (Bo *et al.*, 1998b, 2003; Bo and Choa, 2004). This shifts the curve of void ratio against log of effective stress to the left, and underestimates the yield stress. The magnitude of settlement was calculated on several subdivided layers in order to be able to predict the ultimate settlement accurately. The use of an insufficient number of subdivisions leads to underestimation of the ultimate settlement (Bo and Choa, 2004). The reduction of load due to submergence of the fill below groundwater level, and lowering of the groundwater level after stabilisation, were also taken into considered in the additional load.

Determination of coefficients of consolidation

The consolidation process with PVDs involves consolidation due to both vertical flow (C_v) and horizontal flow (C_h), although the former is relatively less important. Since the quaternary marine formation has been deposited under quiet marine conditions, there is generally no significant stratification, though lamination and foliation may be present at some locations. Hence the degree of anisotropy for hydraulic flow is considered to be low. Several in situ dissipation tests using cone penetrometer, dilatometer and self-boring pressuremeter were carried out to determine the coefficient of consolidation due to horizontal flow and the horizontal hydraulic conductivity of the marine clay. Specialised consolidation tests using a hydraulic Rowe cell have also been carried out to obtain the laboratory values. Details of the hydraulic conductivity and consolidation properties of the Singapore marine clay at Changi have been discussed by Bo *et al.* (1998b) and Chu *et al.* (2002). It has been reported that the ratio C_h/C_v for Singapore marine clay is as low as

1·5. However, it is common practice for vertical drain installation in marine clays to assume a C_h value of twice C_v (i.e. $C_h = 2C_v$), because there could have been some lamination and foliation in the marine clay formation. C_v of the marine clay was also determined from the laboratory consolidation tests. The design prediction analysis was carried out for various C_h/C_v ratios.

Equivalent thickness to calculate consolidation caused by vertical flow

As the marine clay consists of several layers, the coefficients of consolidation due to vertical flow vary along the profile. As the conventional method allows only a single value of C_v, the equivalent thickness of the marine clay was calculated applying the following equations to enable the equivalent thickness, equivalent drainage length and assumed coefficient of vertical consolidation to be used as input values (Bo *et al.*, 2003; Bo and Choa, 2004; Arulrajah, 2005).

The equivalent thickness of layer 1, H_1', is given by

$$H_1' = H_1\left(\frac{C_{vi}}{C_{v1}}\right)^{0\cdot5} \tag{1}$$

where C_{vi} is an assumed average value, and C_{vn} is C_v for layer n.

The total equivalent thickness of all layers, H_{Ti}', is given by

$$H_{Ti}' = H_1' + H_2' + H_3' + \ldots + H_n' \tag{2}$$

The equivalent drainage length, H_{dri}, for double drainage is given by

$$H_{dri} = \frac{H_i'}{2} \tag{3}$$

and for single drainage is given by

$$H_{dri} = H_i' \tag{4}$$

Vertical and radial consolidation

As the consolidation process with PVDs involves both vertical and radial flows, each process was first calculated separately. The equations applicable for the calculations of the time rate of settlement due to vertical flow are as follows.

The time factor for consolidation by vertical drainage, T_v, is given by

$$T_v = \frac{C_{vi}\,t}{H_{dri}^2} \tag{5}$$

The average degree of consolidation with respect to vertical flow, U_v, is given by

$$U_v = \frac{(4T_v/\pi)^{0\cdot5}}{\left[1 + (4T_v/\pi)^{2\cdot8}\right]^{0\cdot179}} \tag{6}$$

The solution for radial water flow towards the central drain goes back to Rendulic (1936). The result is generally expressed in terms of the average consolidation ratio for radial drainage, U_r.

The time factor for consolidation by horizontal drainage, T_r, is given by

$$T_r = \frac{C_h\,t}{d_e^2} \tag{7}$$

where d_e is the diameter of the equivalent soil cylinder.

$$d_e = \begin{cases} 1\cdot128 \times \text{spacing (square pattern)} \\ 1\cdot05 \times \text{spacing (triangular pattern)} \end{cases} \tag{8}$$

Onoue (1988) suggested the simplified formula given below for the average degree of consolidation with respect to radial flow, U_r. The equation uses the Yoshikuni and Nakanodo (1974) well resistance coefficient L, defined later.

$$U_r = 1 - \exp\frac{-8T_r}{F(n) + 0\cdot8L} \tag{9}$$

$$F(n) = \frac{n^2}{n^2-1}\log_e(n) - \frac{3n^2-1}{4n^2} \tag{10}$$

where n is the drain spacing ratio and

$$n = \frac{d_e}{d_w} \tag{11}$$

where d_w is the diameter of the vertical drain.

Carrillo (1942) derived an expression for the average degree of consolidation for combined vertical and radial water flow, U_{vr}:

$$1 - U_{vr} = (1 - U_v)(1 - U_r) \tag{12}$$

The time rate of settlement S_t for vertical drains can be calculated at any particular time for the various surcharge heights by

$$S_t = S_{ult}(U_{vr}) \tag{13}$$

Well resistance

The relevant features for the design and performance of vertical drains are their hydraulic properties: the discharge capacity and their filter permeability. If, during the consolidation period, the discharge capacity of the drain is reached, the overall consolidation process is retarded. In such cases the drains present resistance to the water flowing in them. In the calculations, the well resistance parameter L, as developed by Yoshikuni and Nakanodo (1974), has been used.

It is given by

$$L = \left(\frac{32}{\pi^2}\right)\left(\frac{k_h}{k_w}\right)\left(\frac{H_{dri}}{d_w}\right)^2 \tag{14}$$

where k_h is the horizontal hydraulic conductivity of the soil, and k_w is the coefficient of longitudinal permeability of the drain.

Smear effect

It is often assumed that the installation of a drain does not change the properties of the surrounding soil. In reality, however, drain installation disturbs the soil to a large extent, depending on the soil's sensitivity and macro-fabric (Rowe, 1972). Barron (1948) and Hansbo (1979) analysed the effect of this soil disturbance by assuming an annulus of smeared clay around the drain. Within this annulus, of diameter d_s, the remoulded soil has a lower coefficient of permeability, k_r, than the k_h of the undisturbed clay. This leads to a new boundary condition between the undisturbed zone and the smeared annulus, and this affects the solution by changing the drain factor $F(n)$ defined earlier as follows.

The drain factor with smear effect, $F_s(n)$ is given by

$$F_s(n) = \log_e\left(\frac{n}{s}\right) - 0\cdot75 + \left(\frac{k_h}{k_r}\right)\log_e(s) \tag{15}$$

where k_h/k_r is assumed to be 2; s is the smear zone ratio = d_s/d_w; and d_s is the diameter of the smeared zone.

Predictions of magnitude and time rate of settlement with PVDs

Land was reclaimed 1 m above high tide level to +4 mCD by hydraulic filling. PVDs were installed at +4 mCD level. After installation of the PVDs, surcharge was placed to +10 mCD by hydraulic filling. Fig. 5 shows the design construction sequence at the case study area (vertical drain area).

The increase in groundwater level after placement of the surcharge over the area is expected, owing to a high infiltration rate and the volume of water released from the underlying soil during consolidation.

The predictions were made in advance of the field installation of PVDs and the surcharge placement works by applying Terzaghi (1925) and Carrillo (1942). Curves were plotted for various C_h/C_v ratios. For the design predictions, the coefficient due to horizontal flow, C_h, was assumed to be twice that of C_v: that is, $C_h = 1$ and $2\ m^2/\text{year}$ for the upper and lower marine clay respectively. Figs 6 and 7 show predicted curves for settlement and degree of consolidation generated from the design of vertical drains with preloading for the vertical drain area.

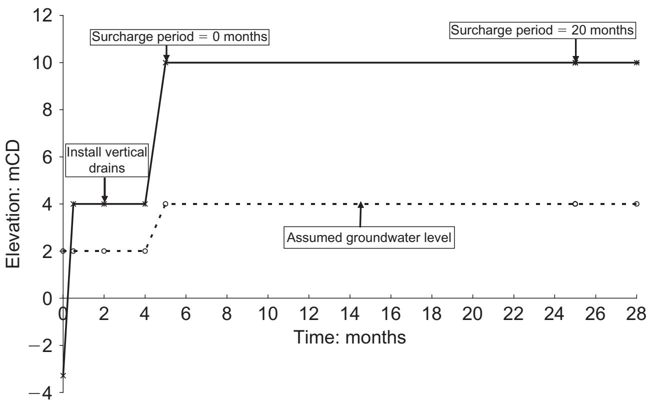


Fig. 5. Design construction sequence at vertical drain location A2S-6

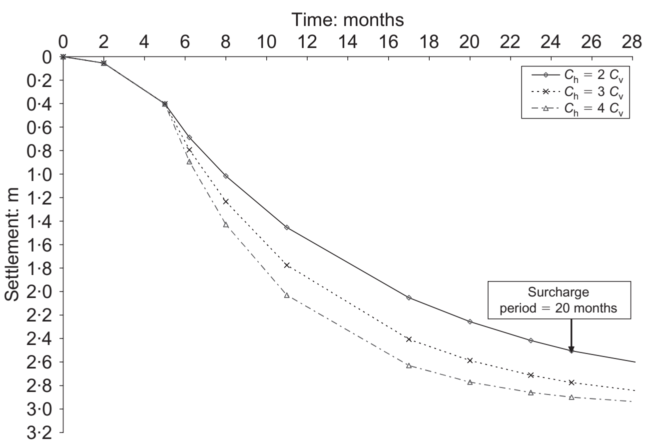


Fig. 6. Time–settlement curves predicted for different C_h for vertical drain area

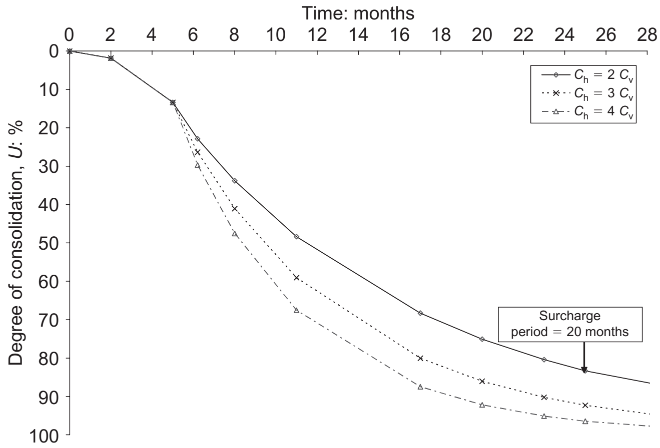


Fig. 7. Design curves for vertical drain area

Performance monitoring and assessment

In order to monitor the performance of ground improvement and to validate the efficiency of the prefabricated vertical drain system several geotechnical instruments were installed to monitor the degree of consolidation, both in the area with PVDs and in the area without PVDs, as a control area. Settlement gauges, including deep settlement gauges, were installed at the top of each sublayer, whereas piezometers were installed at the centre of each compressible layer in order to monitor the settlement and pore pressure dissipation. Details of the arrangement of the instruments are shown in Fig. 8.

Settlement and pore pressure were monitored at close intervals in the first three months, and at wider intervals during the later part of monitoring. Typical monitoring data are shown, together with the construction stages, in Fig. 9. The degree of consolidation in terms of pore pressure was also determined from the isochrones of the pore pressure dissipation data, as shown in Fig. 9.

The more accurate ultimate settlements were predicted using the field settlement results by applying the Asaoka (1978) and hyperbolic methods (Tan, 1993). Fig. 10 presents the results of the analysis by applying these methods, from which the degree of consolidation in terms of settlement was determined. Both methods can predict the ultimate settlement more accurately when the degree of consolidation becomes greater than 60%.

The Asaoka method generally plots the settlement at time t against the settlement at time $t-1$ with the same scales on the x and y axes. When the settlement at time t is equal to that at time $t-1$, it is considered that the ultimate settlement has been achieved. This can be predicted by determining the intercept of the plotted trend line with a 45° line by extrapolating.

The hyperbolic method plots the settlement against the time divided by the settlement. The reciprocal of the gradient of the linear best-match line is defined as the ultimate settlement. Details of these methods can be found in Bo and Choa (2004).

Table 1 presents a comparison of the design prediction results with that determined from the field instrumentation data at the vertical drain area, 20 months after surcharge. The field instrumentation results and assessment of the vertical drain area has been discussed previously by Arulrajah (2005) and Arulrajah *et al.* (2004a). The results of

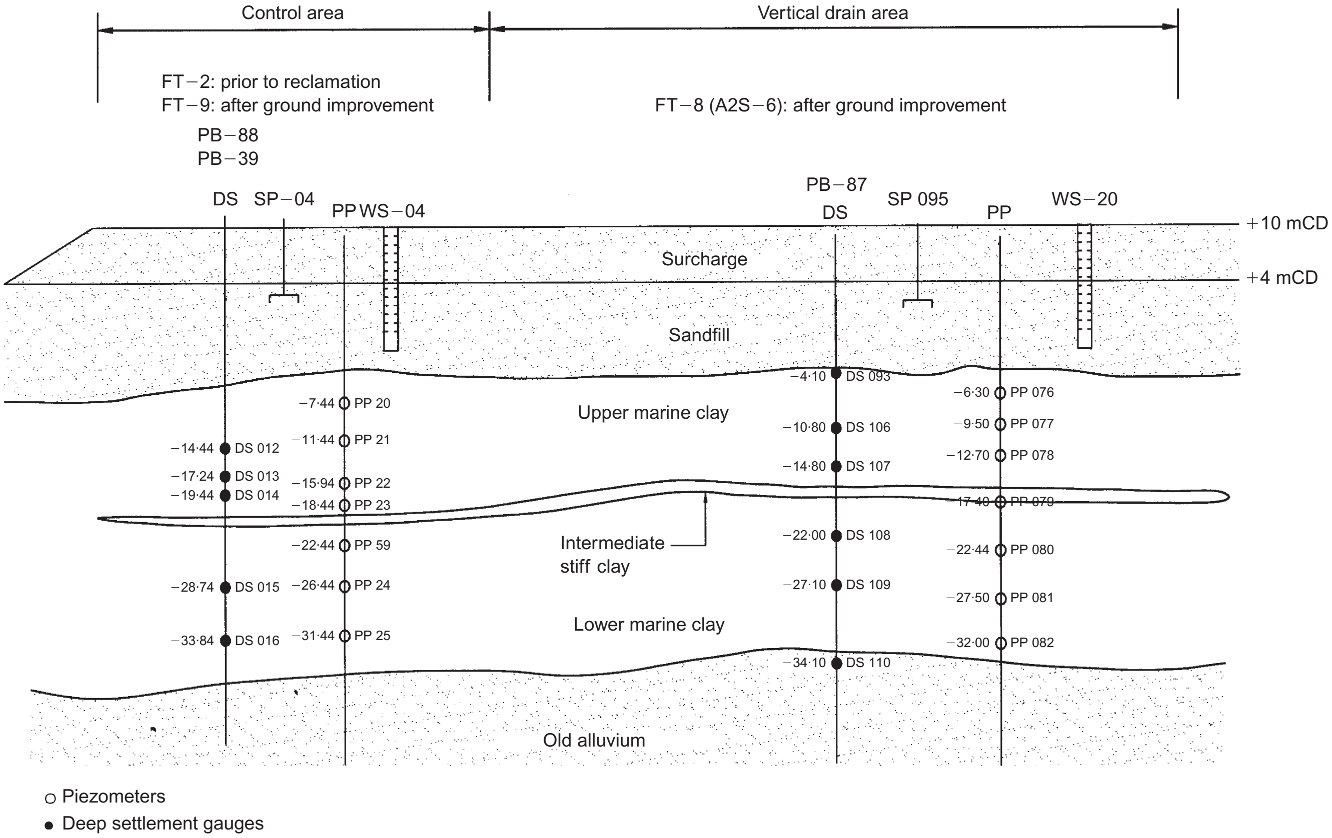


Fig. 8. Cross-sectional soil profile showing instrument elevations

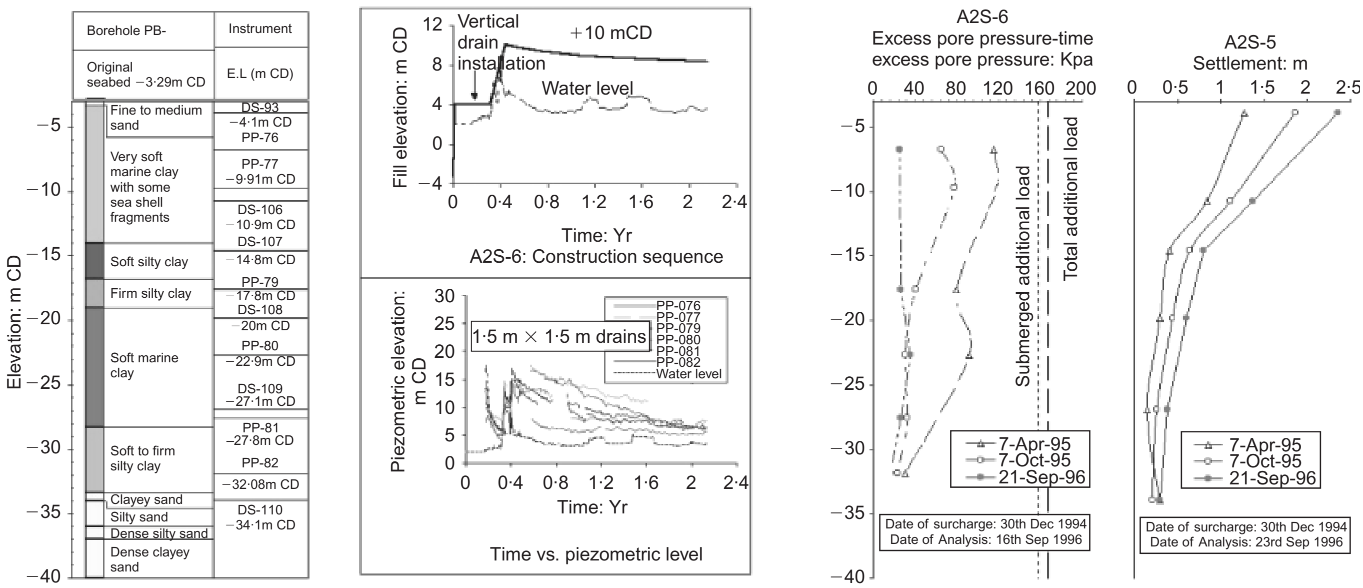


Fig. 9. Instrument monitoring data at vertical drain area: (a) construction sequence; (b) time against piezometric level; (c) excess pore pressure against time; (d) settlement

the design predictions are found to be in good agreement with that of the field instrumentation results. The degree of consolidation, based on the ultimate settlement predicted by applying the Terzaghi theory, is found to be in good agreement with that determined from the field instrumentation data.

For the vertical drain area, a degree of consolidation of 83·3% was obtained based on the design predicted ultimate settlement as compared with 80·1% based on that predicted applying the Asaoka method, 80·0% based on that predicted applying the hyperbolic method and 80·0% using piezometer data.

In order to be able to assess the improvement of soil in terms of effective stress gain and strength gain, post-improvement borehole and in situ tests were carried out in both the soil improvement area and the control area when the geotechnical instruments indicated that the required degree of consolidation had been achieved. Post-improvement tests were carried out more or less at the same location where, prior to reclamation, borehole and in situ tests were

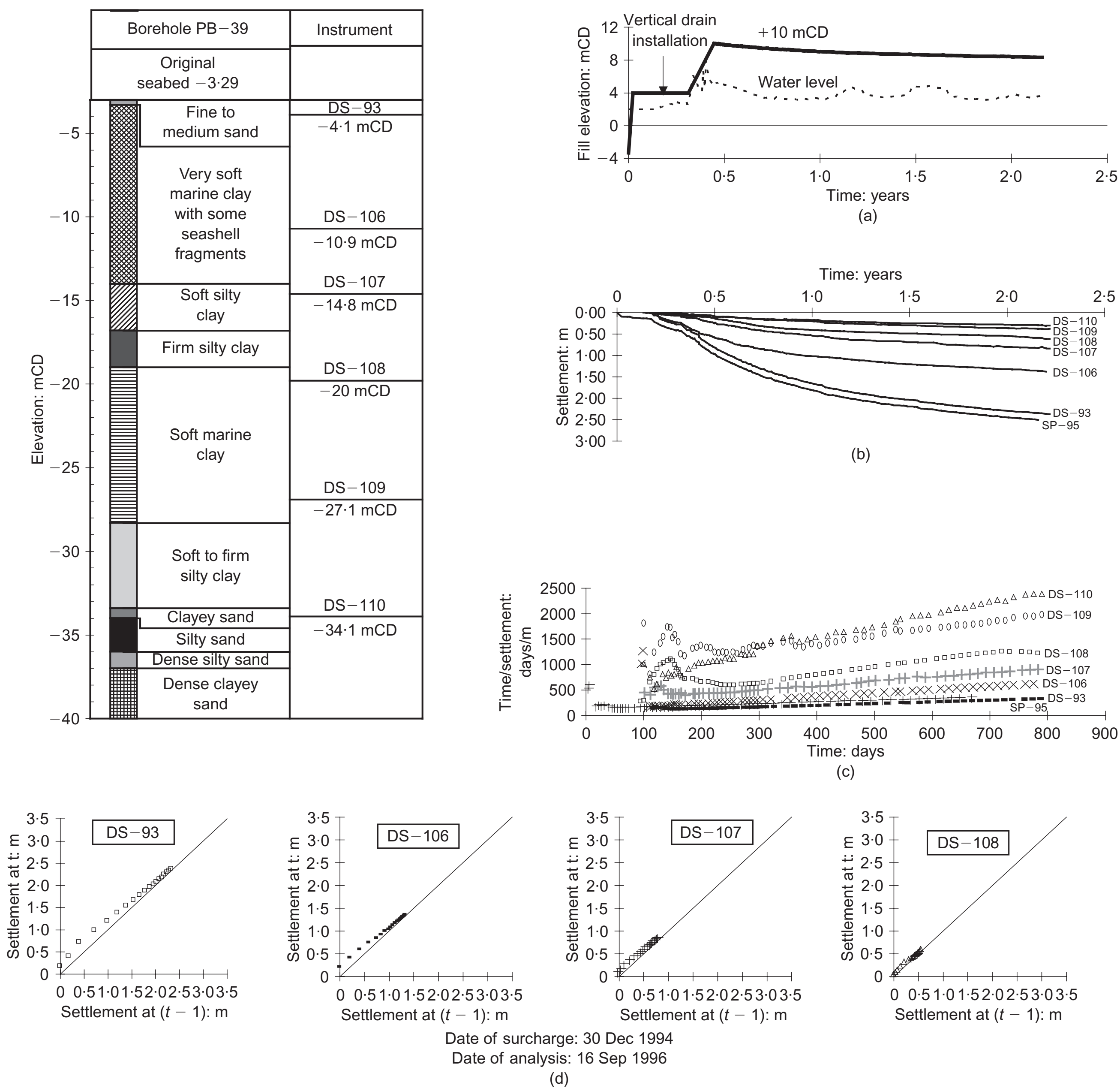


Fig. 10. Ultimate settlement prediction by hyperbolic and Asaoka methods at vertical drain location (A2S-6): (a) construction sequence; (b) settlement against time; (c) hyperbolic method; (d) Asaoka method

Table 1. Comparison between design ($C_h = 2C_v$) and Asaoka, hyperbolic and piezometer methods at vertical drain area, 20 months after surcharge

Sub-area	Comparison	Design	Asaoka	Hyperbolic	Piezometer
Vertical drain 1·5 × 1·5 m	Ultimate settlement: m	3·005	3·000	3·005	–
	Settlement to date: m	2·504	2·404	2·404	–
	Degree of consolidation, *U*: %	83·3	80·1	80·0	80·0

carried out. Note that, in the figures, PB39 is the reference borehole for the vertical drain area (A2S-6) and was bored prior to reclamation, whereas PB-87 is the reference borehole for the same area (A2S-6), bored out after reclamation and ground improvement. The locations of these boreholes are shown in Fig. 8.

Post-improvement in situ tests consist of boring and undisturbed sampling, field vane shear tests, cone penetration tests, self-boring pressuremeter tests and dilatometer tests. Collected samples were tested for physical strength, consolidation characteristics, and preconsolidation pressure measurements.

Figure 11 presents a comparison of the undrained shear strengths prior to and after ground improvement based on various in situ test methods and laboratory tests in the case study area. Fig. 12 presents the degree of consolidation

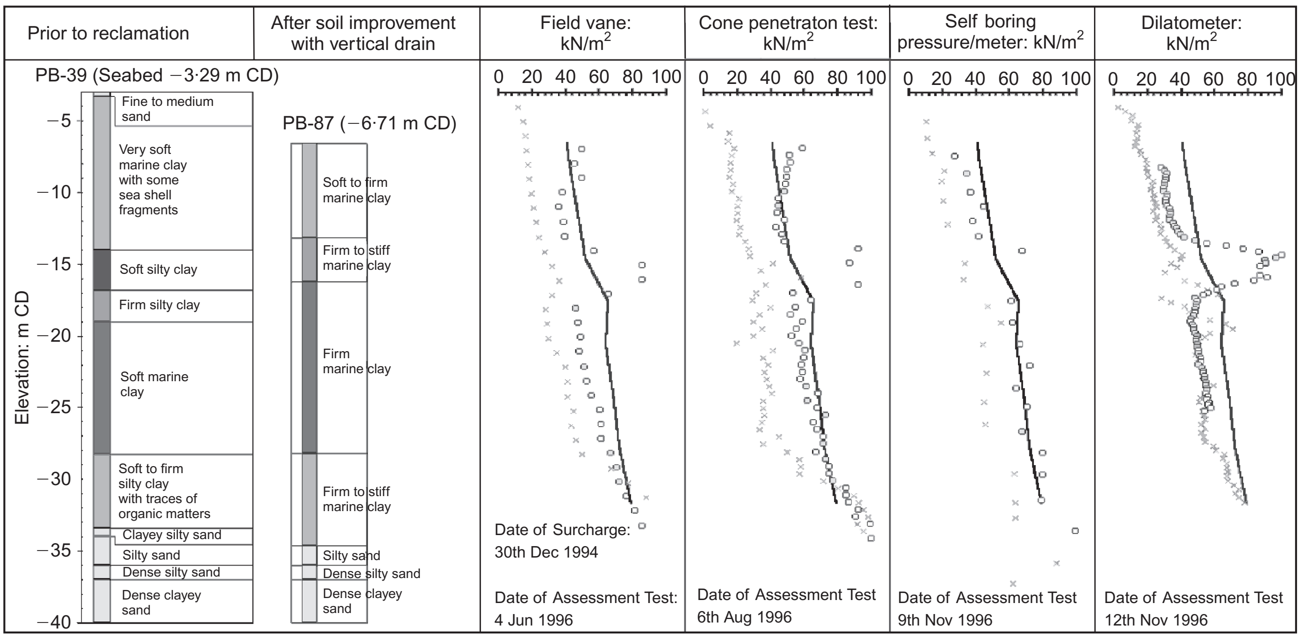


Fig. 11. Comparisons of pre- and post-improvement shear strengths from various in situ tests at vertical drain location

interpreted from instrument monitoring data and in situ tests for various sub-layers in the vertical drain area. Details of the methods for determining undrained shear strength, effective stress gain, degree of consolidation and overconsolidation ratio from laboratory and in situ tests have been discussed by Bo *et al.* (1997, 1998, 2003) and Bo and Choa (2004), and are not repeated here.

However, the preconsolidation pressure and undrained shear strength from laboratory tests were found to be underestimated, owing to limitations on the test results. It is well accepted that the determination of preconsolidation pressure is affected by the duration of loading, rate of loading, rate of strain and temperature applied during the test (Leroueil *et al.*, 1985). The effect of salt content on the e–log p' curve has also been explained in an earlier section. It has also been reported that the preconsolidation pressure determined by applying different methods gives slightly different magnitudes (Bo *et al.*, 1997). Different preconsolidation pressures also result from variation in the scale used in the e–log p' curve, even when the same method is applied (Mikasa, 1995). These effects are discussed in detail in Bo *et al.* (2003) and Bo and Choa (2004).

Conclusions

This paper discusses briefly the various theories of radial and vertical drainage, considerations and design methodologies for the ground treatment of marine clay with prefabricated vertical drains in an offshore reclamation project. The design approach used has been verified and substantiated in comparison with field instrumentation and in situ tests results at a case study site. The results of the design predictions are found to be in good agreement with

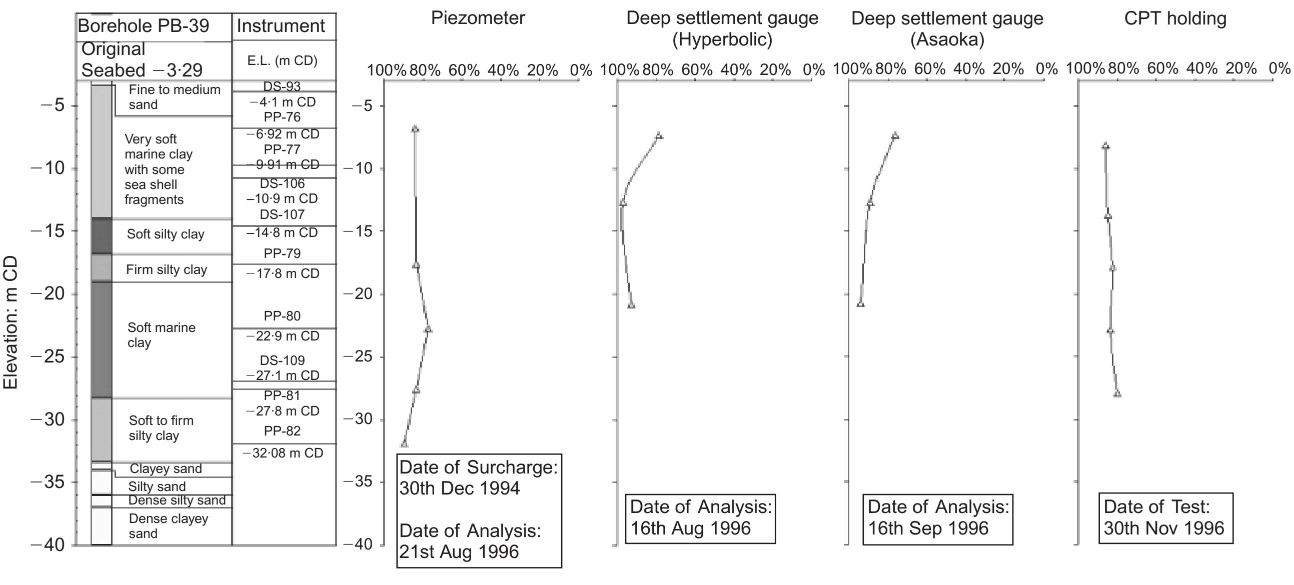


Fig. 12. Comparison of degree of consolidation from different instruments at vertical drain area

that of the field instrumentation results. The degree of consolidation obtained by the design predictions is found to be only slightly higher than that of the field instrumentation results. This indicates that the design approach used is valid for the design of prefabricated vertical drains installed in soft marine clay.

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Discussion contributions on this paper should reach the editor by 1 October 2007