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Soil improvement works for an offshore land reclamation

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The Changi East reclamation project was carried out in five phases along the foreshore of the east coast of Singapore. The water depths in the reclaimed area ranged from 5 to 15 m. The project involved hydraulic placement of 272 million m^3 of sand onto soft seabed marine clay up to 50 m thick. A linear total of 170,000 km of prefabricated vertical drains (PVDs) were installed for accelerating the consolidation process of the underlying soft marine clay. The soil improvement works covered a total area of approximately 1200 ha. In this paper, the site conditions and the soil improvement works adopted are described. Pilot tests with full-scale field instrumentations as well as laboratory and in situ tests were carried out to verify the design, check the effectiveness of the soil improvement works using PVDs, and establish the most suitable drain spacing. Field monitoring data obtained from both the pilot tests and the reclamation works are presented and interpreted. Degree of consolidation was calculated based on both settlement and pore pressure data.

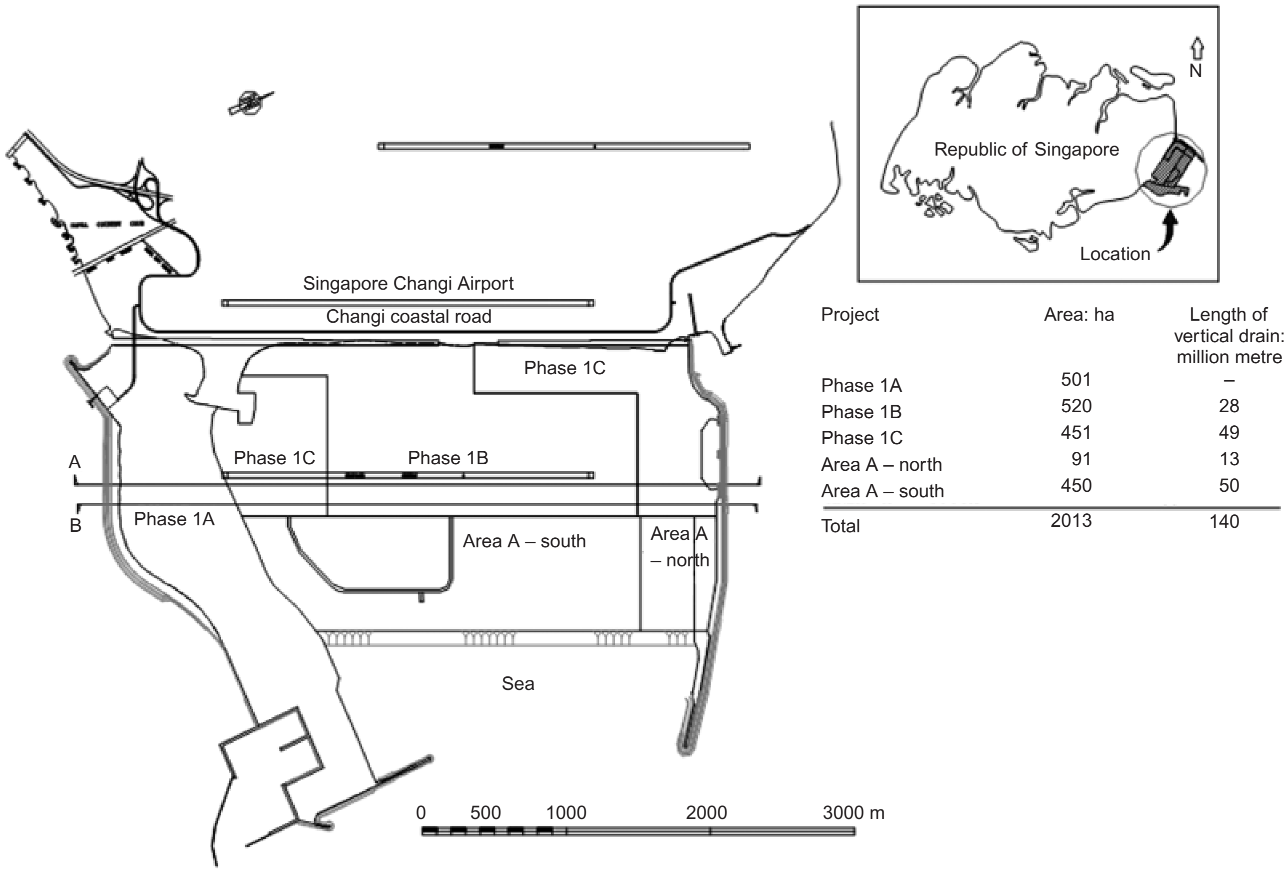
1. INTRODUCTION

The Changi East reclamation project was carried out between 1991 and 2005 in five phases to create 2000 ha of land for the expansion of the Changi International Airport and other infrastructure developments in Singapore. The location of the project and the site plan are shown in Figure 1. Phase 1 of the project comprised phases 1A, 1B and 1C, which commenced in 1991, 1993 and 1995 respectively. Each phase lasted about five years, with a couple of years' overlap between the phases. Two other phases, called area A (south) and area A (north) commenced simultaneously in 1999. Area A (north) and area A (south) were completed in March 2004 and March 2005 respectively. The project involved hydraulic placement of 272 million m^3 of sand in seawater up to 15 m deep. As the majority of the reclamation area was underlain by a highly compressible layer of Singapore marine clay up to 50 m thick, approximately 170 million linear metres of prefabricated vertical drains (PVDs) together with surcharge up to 8 m thick were used to consolidate the seabed soft clay and improve its engineering properties. The total area of the soil improvement works was approximately 1200 ha.

The hydraulically placed sand fill was generally in a loose state. Three deep compaction methods were deployed to densify the sand fill: dynamic compaction using heavy pounders, Müller resonance compaction (MRC) and vibroflotation. For dynamic compaction, heavy pounders were used in the areas where the required depth of compaction was 5–7 m. The weight of the pounders used ranged from 15 to 23 t, and drop height was from 20 to 25 m. MRC and vibroflotation methods were adopted in the areas where the thickness of compaction was 7–10 m. The equipment used for MRC was MS-100HF and MS-200H and for vibroflotation was V23, V28, V32, Keller S300 and Pennine BD400. Compaction of granular fill was carried out after the fill surcharge was removed. Densification of sand fill will not be covered in this paper owing to space limitations. For more detail, see References 1 and 2. The unit weight of the compacted sand fill was in the range 18–19 kN/m^3.

2. SITE CONDITIONS

The site conditions, as revealed by the site investigation carried out prior to reclamation, are as follows. The seabed at the reclamation area ranged from −3 to −15 mCD (Admiralty chart datum, where the mean sea level is at +1·6 mCD). In the northernmost part of the area the seabed sloped northwards from −5 to −15 mCD, and in the southern part it sloped southwards from −5 to −10 mCD. Deep hollows in the seabed, varying from −10 to −13 mCD, occurred in the eastern part of the area. A typical soil profile and the basic soil properties are shown in Figure 2. The soil profile can be divided into four layers: the upper marine clay; the intermediate layer which consisted of the stiff silty clay layer or/and the silty sand layer; the lower marine clay layer; and the old alluvium, a medium-dense to dense cemented clayey sand layer that is not shown in Figure 2. The thickness of the marine clay ranged from 5 to 55 m, and of the intermediate layer from 2 to 5 m. The large variation in the thickness of the marine clay layer was due to not only undulations in the surface of the underlying materials but also the self-weight consolidation of the clay in the area of thick deposition. As indicated in Figure 2, the upper and lower marine clay are highly compressible and high in water content. Except for the top few metres, the marine clays are generally lightly overconsolidated. The water content of the upper marine clay ranged from 50% to 85% and that of the lower marine clay from 40% to 65%. Both the upper and lower marine clays had almost 100% fines content, with 50% silt and 50% clay for the upper marine clay and 60% silt and 40% clay for the lower marine clay. The



Project	Area: ha	Length of vertical drain: million metre
Phase 1A	501	–
Phase 1B	520	28
Phase 1C	451	49
Area A – north	91	13
Area A – south	450	50
Total	2013	140

Figure 1. Location and site plan of the project

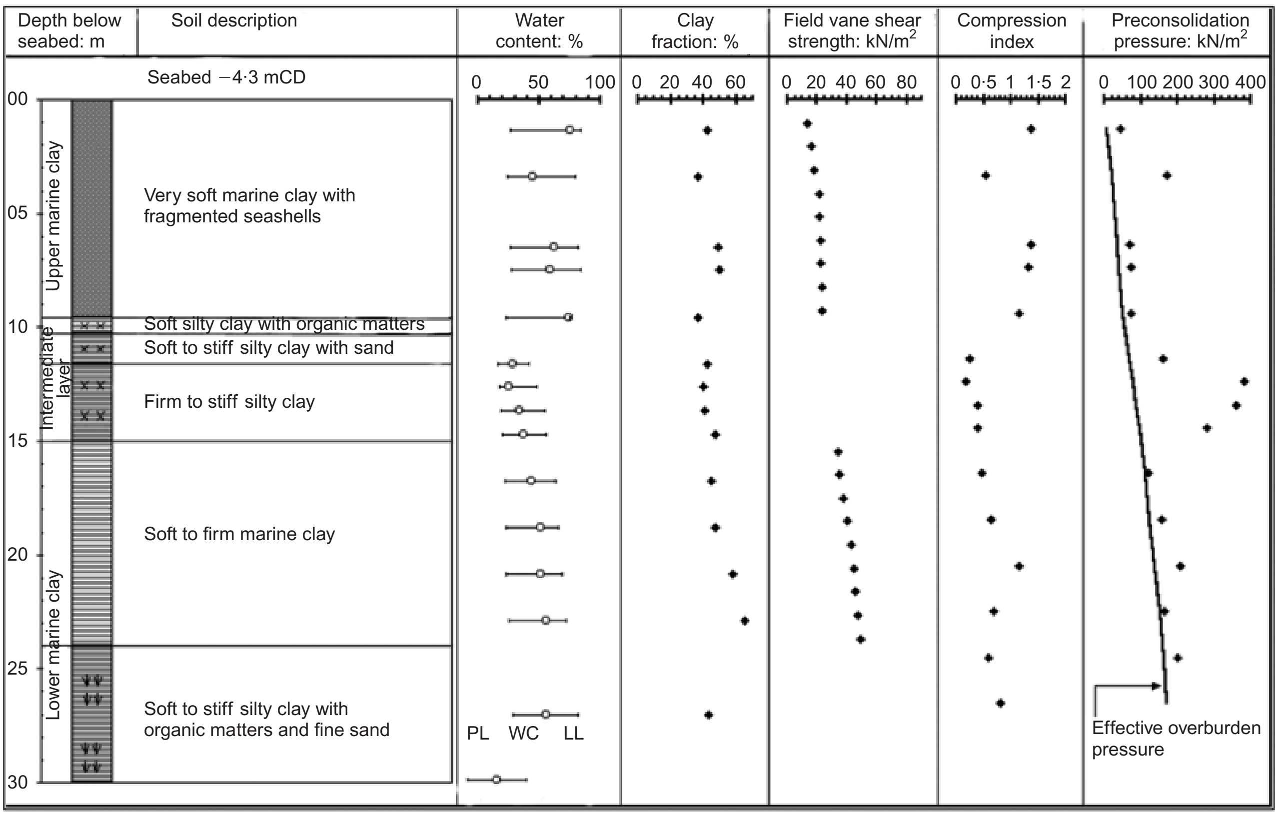


Figure 2. Typical soil profile and basic properties of Singapore marine clay: PL, plastic limit; WC, water content; LL, liquid limit

normalised undrained shear strength ratio, s_u/σ'_v, for the marine clay at normally consolidated conditions was in the range 0·25 to 0·32 based on field vane shear tests.[3] The intermediate layer was formed by desiccation of the top layer of the lower marine clay as the sea level dropped. The intermediate layer was overconsolidated, stiff and low in both compressibility and moisture content. The coefficient of consolidation in the vertical and horizontal directions of the original seabed soil, c_v and c_h respectively, as measured by laboratory and in situ tests are shown in Figure 3. The in situ tests and the Rowe cell test measured the c_h values, and the conventional oedometer test gave c_v. The coefficient of consolidation, back-calculated based on the field settlement monitoring data at the end of soil improvement using PVDs, is also shown in Figure 3. It can be seen that the back-calculated c_h values were actually lower than the c_h values measured by either in situ or laboratory tests. This was partially due to the smear effect caused by the installation of PVDs and the reduction in c_h after the soil had been consolidated into the normally consolidated (NC) state from the originally overconsolidated (OC) state, as discussed by Chu *et al.*[4] Typical values of c_h and c_v in the NC state, and other soil parameters for the Singapore marine clay at Changi, are given in Table 1. The values of c_h and c_v varied with the overconsolidation ratio (OCR). The values in the OC state were much greater than those in the NC state, as discussed in detail in Reference 4. Further description of the engineering properties of the Singapore marine clay can be found in References 5–8.

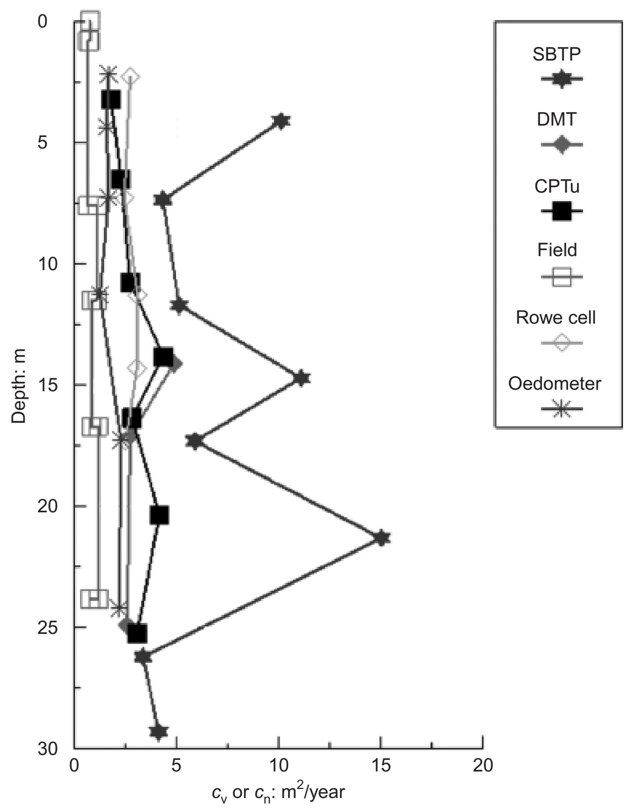


Figure 3. Comparison of coefficient of consolidation measured by different tests and back-analysis: SBPT, self-boring pressuremeter holding test; DMT, flat dilatometer holding test; CPTu, piezocone dissipation test; field, c_h back-calculated using field settlement data

3. SOIL IMPROVEMENT WORKS

As mentioned, approximately 170 million linear metres of PVDs were used in the project to improve the engineering properties of the seabed soils. The spacing of the PVDs was determined to achieve a 90% degree consolidation under a specified surcharge load within a given duration. Hansbo's equations[9] and a simplified soil profile were used in the design. The soil parameters were taken from both laboratory and in situ tests conducted for the specific site.[4–7] The ranges of the values are given in Table 1. Studies of the smear effect were also carried out.[7,10] Based on the studies, the permeability of the smeared soil was taken as one-half or one-third that of the undisturbed soil, and the diameter of the smear zone was four to five times the equivalent diameter of the PVD. The initial design required a drain space of 1·7 m. PVDs were installed throughout the entire compressible layer down to the hard stratum. Two types of PVD were used in the project: Colbond CX1000 and Mebra MD7007. The specifications for the PVDs are given in Table 2. The test methods that were adopted for quality control tests are described in Reference 10. The PVDs were installed when the sand fill reached a level slightly above the high tide. A fill surcharge 8·5–12 m high was then applied. The fill surcharge was chosen based on the anticipated

Parameter	Upper marine clay	Intermediate layer	Lower marine clay
Unit weight, γ: kN/m^3	14·2–15·7	18·6–19·6	15·7–16·7
Water content: %	50–85	10–40	40–66
Liquid limit: %	70–95	30–70	60–90
Plastic limit: %	20–28	18–20	20–30
Initial void ratio, e_0	1·8–2·2	0·5–0·9	1·1–1·7
Specific gravity, G_s	2·60–2·72	2·68–2·76	2·70–2·75
Compression index, C_c	0·6–1·5	0·2–0·3	0·4–1·0
Recompression index, C_r	0·1–0·2	0·02–0·15	0·05–0·2
Secondary compression index, C_α	0·012–0·025	0·004–0·023	0·012–0·023
OCR	1·5–7·0	3·0–4·0	1·8–2·0
Vertical coefficient of consolidation in NC state, c_v: m^2/year	0·5–1·7	–	0·5–2·3
Horizontal coefficient of consolidation in NC state, c_h: m^2/year	2·0–4·0	–	3·0–6·0
Coefficient of permeability: m/s	10^{-8}–10^{-9}	–	10^{-9}–10^{-10}

Table 1. Range of physical and compressibility parameters of Singapore marine clay at Changi, Singapore

Parameter	Value
Width: mm	100
Thickness: mm	3–4
Tensile strength (dry and wet) at 10% strain: kN	>1
Elongation: %	<30
Discharge capacity (straight at 350 kPa): 10^{-6} m^3/s	> 25
Discharge capacity (buckled at 350 kPa): 10^{-6} m^3/s	> 10
Pore size, O_{95}: μm	<75
Permittivity: s^{-1}	>0·005

Table 2. Specification for PVDs used for the Changi East Reclamation Project

maximum future loads to be applied. As the ground settlement was large, part of the fill used as surcharge would be sinking gradually below the water level. This led to a reduction in the surcharge load due to the submergence effect.[11] The possible variation in the surcharge load was taken into consideration in the design. Consolidation of the soft clay layer took place under the surcharge. The design specification for the pilot test site was that a degree of consolidation of 90% should be achieved in 18 months. The surcharge was removed 18 months after the required degree of consolidation had been achieved. The ground settlement was predicted based on a one-dimensional consolidation settlement calculation in which oversimplified soil profile and soil parameters were used. The predicted settlements were in the range 150–250 cm for soil deposits of different thicknesses.

3.1. Pilot test

The soil profiles at this site varied erratically. Thus it was necessary to verify the simplified design calculation, check the effectiveness of the soil improvement works using PVDs, and establish the most suitable drain spacing. For this purpose, several pilot tests were conducted at different phases during the project. The pilot tests were also used as full-scale model tests to study the consolidation process of soft soil with or without PVDs under both fill and surcharge loads; thus a better understanding of the consolidation behaviour of soft soil, and more reliable designs, could be achieved. All the pilot tests were fully instrumented. Pore water pressures, settlements and lateral displacements at the ground surface and other depths in the soil, and surcharge loads, were monitored for the entire duration of the pilot tests.

The results of the pilot test conducted during the construction of phase 1B are presented in this paper. Figure 4 shows the plan and section views of the pilot test area, which was 280 m long and 230 m wide, and divided into four separate zones. Three zones—lots 1·5, 1·7 and 2·0—were installed with drains at square grid spacings of 1·5, 1·7 and 2·0 m respectively to study the effect of drain spacing. The fourth zone, lot X, was used as a control zone with no drain installed. All zones were 50 m square. The details of the instrumentations are also shown in Figure 4. The instrument types used were surface settlement plate (SP), screw-type deep settlement gauge (DS), multilevel settlement gauge (MS), liquid settlement gauge (SG), deep reference point (DR), pneumatic piezometer (PP), open-type piezometer (OP), vibrating-wire piezometer (PZ), water standpipe (WS) and inclinometer (IN). Readings were taken once a day or once every three days for the first month, and once a week or once every two weeks for subsequent months. The instruments for the control zone, lot X, were installed offshore from a pontoon before fill was placed, whereas for the other zones they were installed when the fill reached +4·0 mCD. For lot X, the instrument clusters had to be specially protected from the disturbance imposed during fill placement and subsequent PVD installation.[12] The original seabed level was at −2·5 to −5·0 mCD. Reclamation by hydraulic pumping of sand fill was used to bring the ground surface up to +4·0 mCD. At this level, PVDs were installed. The fill surcharge was then elevated to +10·0 mCD and subsequently lowered to +5·5 mCD after a surcharge period of 18 months.

The construction sequence for the pilot area was as follows.

(*a*) Offshore soil investigation for the seabed soil was carried out from a jack-up pontoon.
(*b*) Instruments were installed for the no-drain area under marine conditions.
(*c*) The pilot area, 280 m long and 230 m wide, was reclaimed to +4·0 mCD.
(*d*) Detailed soil investigation was carried before soil improvement works.
(*e*) PVDs were installed at the required spacings.
(*f*) Instruments were installed for the three zones in which PVDs had been installed.
(*g*) Surcharge was hydraulically placed to +10 mCD and maintained for 18 months.
(*h*) Post-improvement soil investigation was carried out.
(*i*) The surcharge level was lowered to +5·5 mCD 18 months after the surcharge was placed.

The sand fill materials were dredged from the sea. The mean grain size ranged from 0·4 to 0·8 mm. It was clean sand, with fines content less than 10%. The same sand was used as surcharge. The underlying soil profile at the pilot site is shown in Figure 4. The settlements monitored at different elevations along or near the centreline of each zone are given in Figure 5. As the soil profile was very erratic (Figure 4), the soil profile for each zone is also shown in Figure 5 for easy reference. The fill elevation at each zone is shown in Figure 5. The dates at which PVDs were installed are also given in Figure 5. Applying Asaoka's method,[13] the ultimate consolidation settlements were estimated for the four zones, as shown in Figure 6 and Table 3. Asaoka's method is commonly adopted to estimate the ultimate consolidation settlement, based on curves of field-monitored settlement against time. The application of this method is explained in many papers for example Reference 14. Based on these ultimate settlement values, the degree of consolidation achieved after about 18 months of preloading was calculated as 91%, 93%, 82% and 77% for lots 1·5, 1·7, 2·0 and X respectively. Lot 1·7 achieved a slightly higher degree of consolidation than lot 1·5. This was because there were more sand lenses in lot 1·7, which accelerated consolidation in both the vertical and horizontal directions. For the same reason, a relatively high degree of consolidation was also achieved for lot 2·0 and lot X. Nevertheless, the effect of drain spacing is shown by the data. Using the slope of the Asaoka plot for lot 1·5 in Figure 6a, the equivalent c_h value was roughly back-calculated as 0·98 m^2/year. It was assumed in the calculation

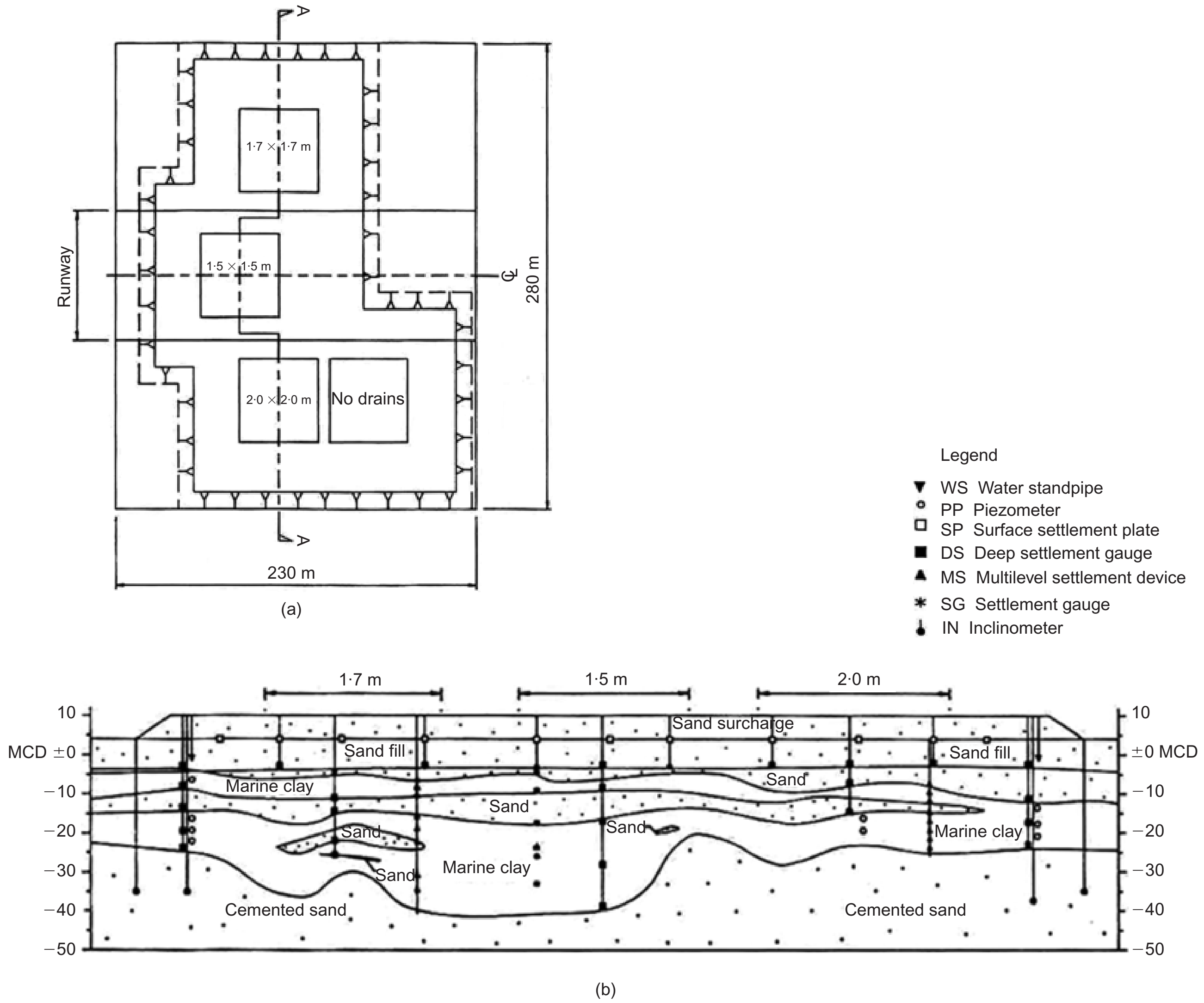


Figure 4. Pilot test at Phase 1B: (a) plan; (b) section A–A

that the diameter of the smear zone was four times that of the equivalent drain diameter, and that the permeability of the smeared soil was half that of the intact soil. This back-calculated c_h value is similar to that presented in Figure 3. As the general soil profile at the reclamation site, as shown in Figure 2, resembled more closely that for lot 1·5, the pilot test results suggest that a drain spacing of 1·5 m should be used.

Based on Figure 5, the rates of settlement after 18 months of preloading are also estimated and presented in Table 3. The values are between 0·06% and 0·08%/month, and are relatively small. The secondary compression is also calculated using a secondary compression index of 0·02 and a void ratio of 1 (see Table 1 for typical values), and shown in Table 3. A secondary compression rate of 0·005%/month is thus obtained, and this is considered insignificant. However, secondary compression may not be evaluated separately from primary consolidation, as elaborated by Leroueil.[15] Nevertheless, after the removal of surcharge, the soil became overconsolidated, and the secondary compression rate would have reduced.[16]

The pore water pressures measured at different depths together with surcharge histories are shown in Figure 7 for lot 1·5 and in Figure 8 for lot X (no drain area). These pore pressure responses are typical as described by Tavenas and Leroueil.[17] Based on Figures 7 and 8, the pore water pressure distribution profiles for lot 1·5 and lot X are also plotted in Figure 9. During the preloading, long-term piezocone (CPTu) holding tests were also conducted at locations close to the selected pore pressure transducers to verify the pore water pressures at different depths. In conducting these tests, a piezocone was pushed to a given depth and held there for over 30 h until the change in pore pressure readings became negligible. The pore pressures measured by CPTu and the pore pressures monitored from the pilot test after the same durations (17 and 14 months after surcharge for lot 1·5 and lot X respectively) are shown in Figure 9. Good agreement between the piezometer readings and the CPTu holding tests is observed for most of the points. This also confirms that the pore pressures monitored in the pilot test are reliable. The effect of PVDs can be clearly seen from a comparison of the pore water pressure profiles for the 1·5 m spacing zone and those for the no-drain zone shown in Figure 9. The average degree of consolidation for lot 1·5 and lot X can also be estimated approximately based on the pore water pressure profile using a method explained in Reference 14. Based on the pore water pressure data shown in Figure 9, the

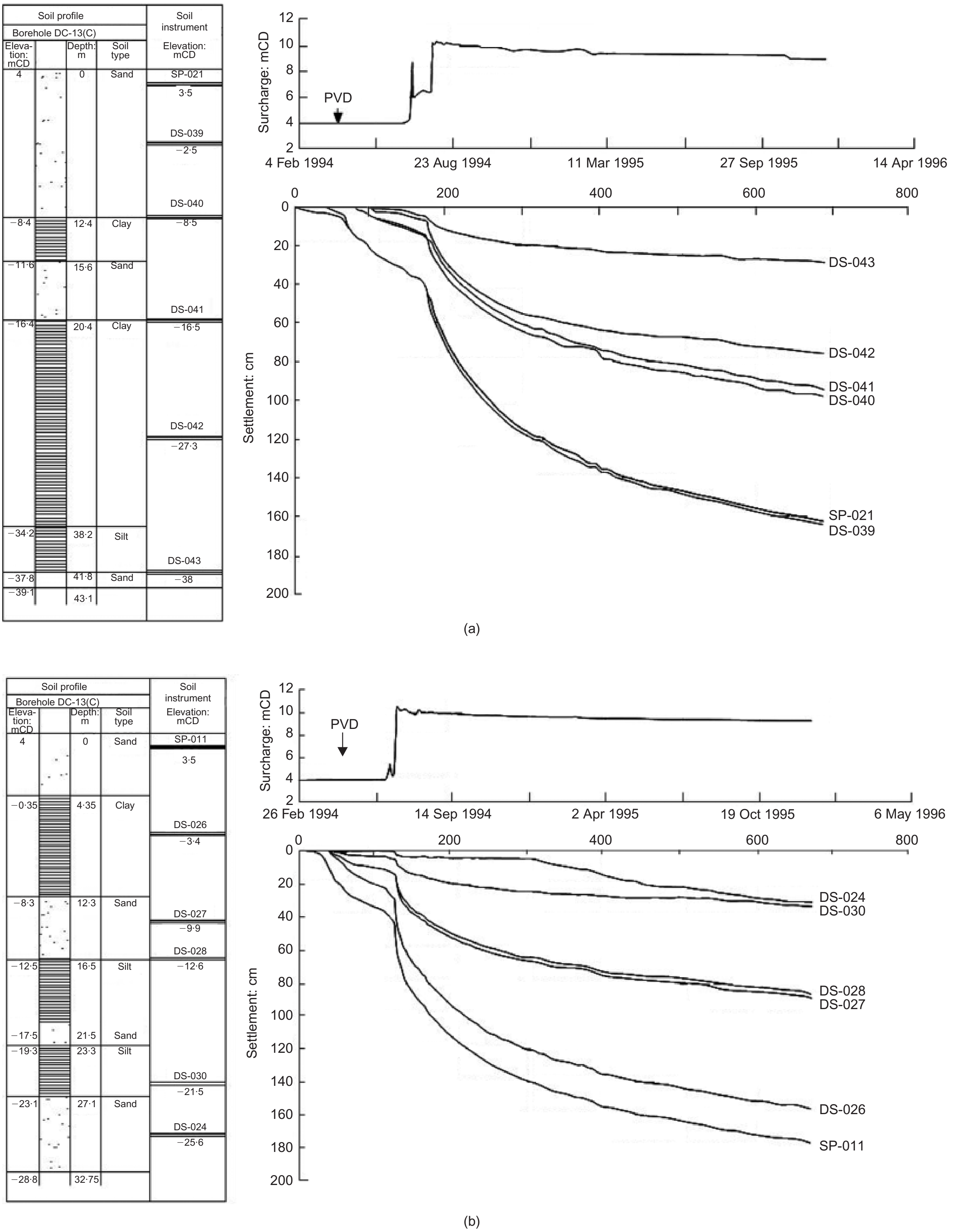
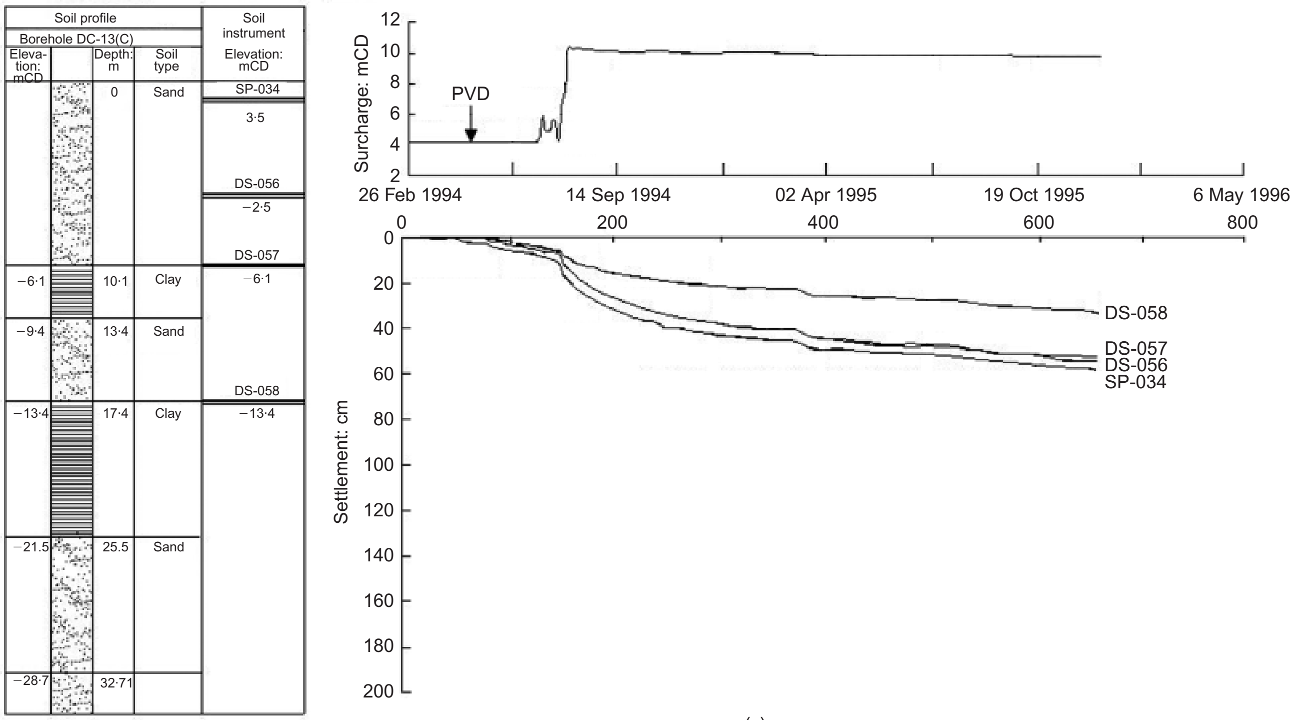
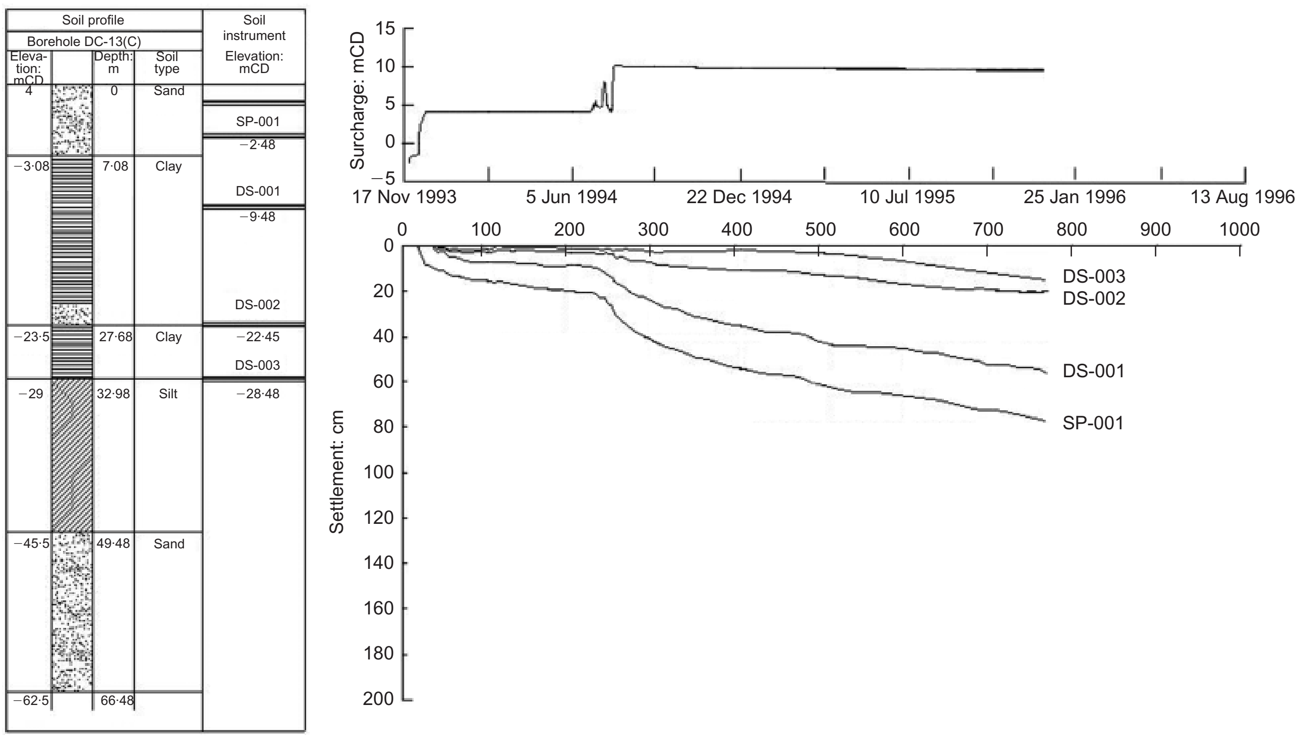


Figure 5. Settlements monitored from the pilot test for Phase 1B: (a) lot 1·5; (b) lot 1·7; (c) lot 2·0; (d) lot X. (In lot 1·5, DS-040 malfunctioned owing to an insufficient gap between the screw plate and the protective casing)

average values of degree of consolidation achieved 17 and 14 months after surcharge are roughly 80% and 38% for lot 1·5 and lot X respectively. It can be seen that the degree of consolidation estimated based on the pore water pressure is smaller. There are two reasons for this. The first is the locations where the instruments were installed. As only limited instrumentation could be used, the settlement plates were installed along the centreline, where the maximum settlements



(c)



(d)

Figure 5. (cont'd)

would occur. As a result, the settlements would be overestimated, which in turn led to an overestimation of the degree of consolidation. The piezometers, on the other hand, were installed at the centre of a square drain grid where the highest pore water pressure was normally generated. As the pore pressures near the drain would be much lower, the average excess pore water pressures across the radial distance to the drain would be smaller than that measured. Therefore

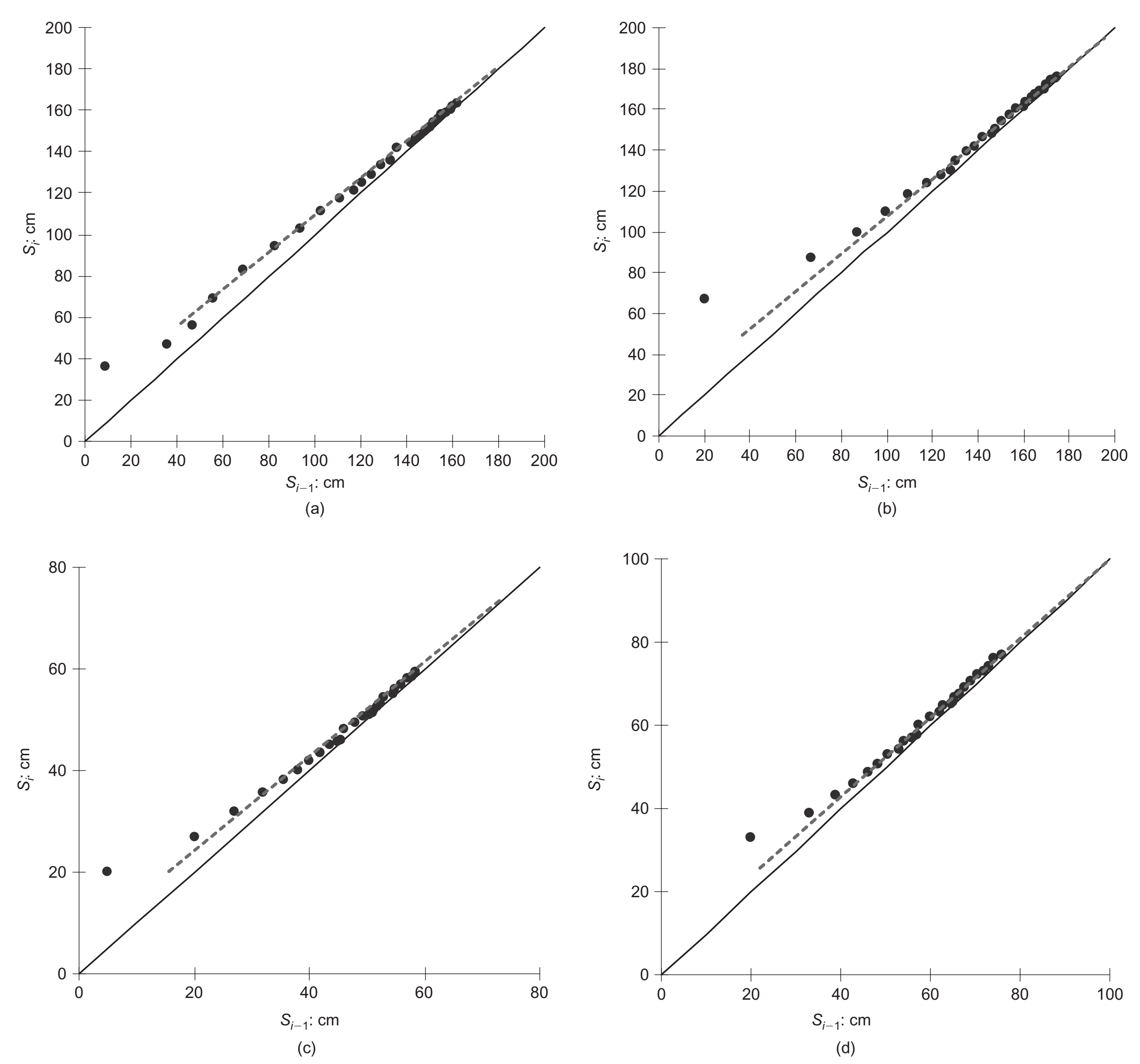


Figure 6. Asoka method applied to: (a) lot 1·5; (b) lot 1·7; (c) lot 2·0; (d) lot X

	Pilot test section			
	Lot 1·5	Lot 1·7	Lot 2·0	Lot X
Total clay thickness: m	21·0	16·75	11·4	22·0
Measured final settlement: cm	163·5	176·0	59·5	77·0
Estimated ultimate settlement: cm	180	190	73	100
Degree of consolidation: %	91	93	82	77
Estimated residual settlement: cm	16·5	14·0	13·5	23·0
Rate of settlement after 18 months' preloading: cm/month	1·7	1·3	0·7	1·5
Strain rate after 18 months' preloading: %/month	0·08	0·08	0·06	0·07
Rate of secondary compression: %/month	0·004	0·004	0·004	0·004

Table 3. Ultimate settlement and degree of consolidation estimated for the pilot test

the pore pressures measured by the piezometers are overestimated, which in turn caused underestimation of degree of consolidation. The second reason is the horizontal sand seams in the soil. These sand seams can reduce the drainage path considerably, particularly if vertical drains are not used. This is likely to be the case for lot X. As shown in Figure 4, sand seams existed in the pilot test area, although details of the sand seams under lot X were not revealed through the soil

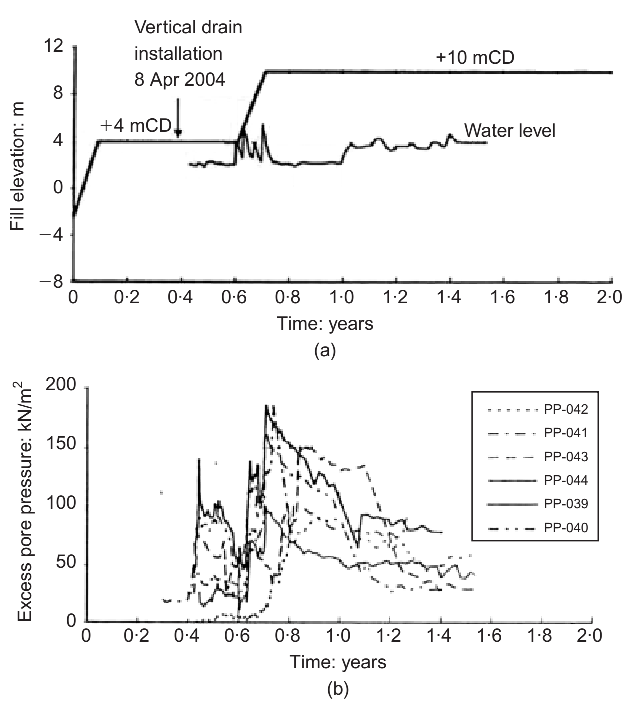


Figure 7. Variations of (a) surcharge and (b) pore water pressure with time measured in lot 1·5 (fill height not adjusted for settlement). Depth of instruments: PP-041, −6·5 mCD; PP-042, −17·0 mCD; PP-044, −21·0 mCD; PP-043, −24·7 mCD; PP-039, −28·3 mCD; PP-040, −32·0 mCD

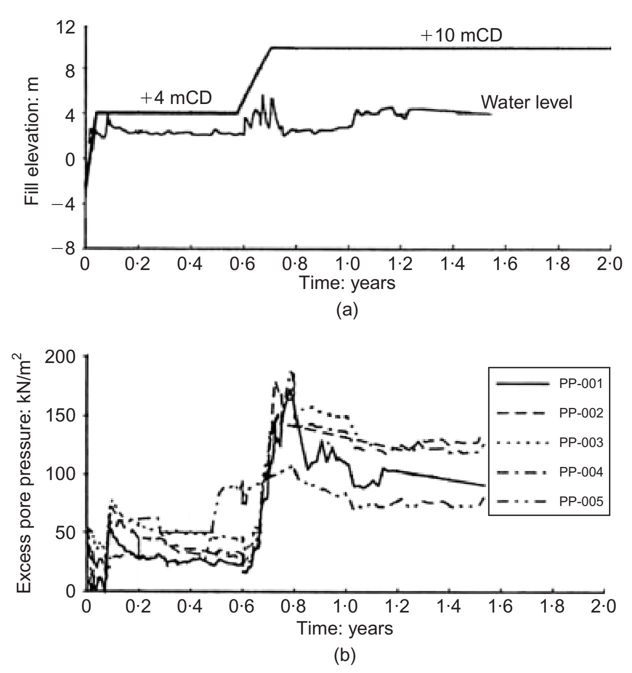


Figure 8. Variations of (a) surcharge and (b) pore water pressure with time measured in lot X. Depth of instruments: PP-001, −6·48 mCD; PP-002, −12·38 mCD; PP-003, −14·98 mCD; PP-004, −19·48 mCD; PP-005, −24·48 mCD

investigation. However, the thin sand seams did not affect the pore pressure measurements very much, unless the pore pressure transducers happened to be installed at the sand seam level. This explains why the degree of consolidation calculated

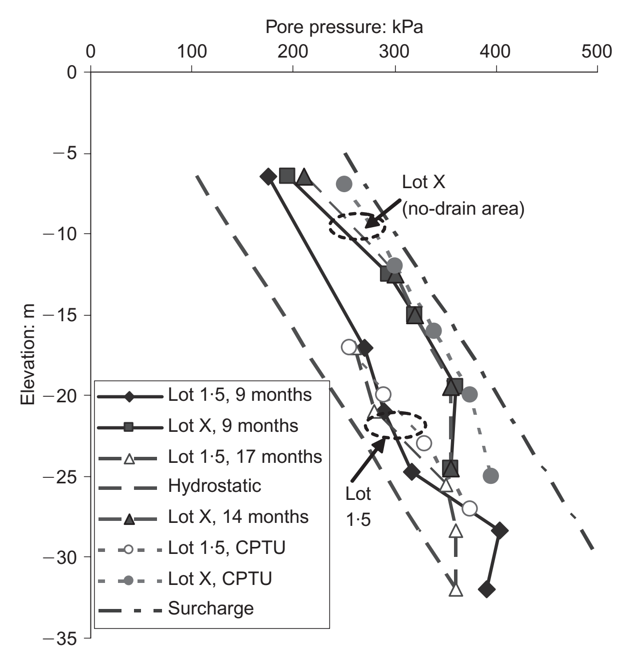


Figure 9. Pore pressure distribution profiles

by settlement is much higher than that calculated by pore pressure for lot X.

3.2. Field monitoring during soil improvement

In addition to the pilot tests, field instrumentation and monitoring were also carried out during the reclamation and soil improvement works.[18,19] Curves of the variation of settlement and pore water pressure with time at a location with PVDs installed at 1·5 m square grid spacing are shown in Figure 10. The soil profile at this location is also shown in Figure 10. Using Asaoka's method, the ultimate ground settlement was estimated as 275 cm, as shown in Figure 11. Using the data in Figure 10, the settlement and pore water pressure distribution profiles with depth are also plotted in Figure 12. The average degree of consolidation achieved at the end of preloading was 90% based on the settlement data and 87% based on the pore water pressure data. Again, the pore water pressure was measured at the centre of the square drain grid and thus represented the worst case. The settlement profiles shown in Figure 12b indicate that there were settlement developments at all depths over the whole duration of preloading. However, the majority of the settlement was concentrated in the top 15 m depth.

The effect of soil improvement in phase 1B can be shown by the comparison of soil parameters shown in Figure 13. The data in Figure 13a do not indicate a clear change in the moisture content. This is common for soft clay with its moisture below the liquid limit (see Figure 2), as discussed in Reference 19. Therefore the variation of moisture content may not be a good indicator for evaluating the effectiveness of soil improvement. Figure 13b shows a more than twofold increase in the undrained shear strength as measured by field vane shear tests using the uncorrected data. Corresponding to the increase in undrained shear strength, there should be an increase in the preconsolidation stress. However, the increase in preconsolidation stress was only marginal, as shown in Figure 13c. This was due mainly to the uncertainties involved in the

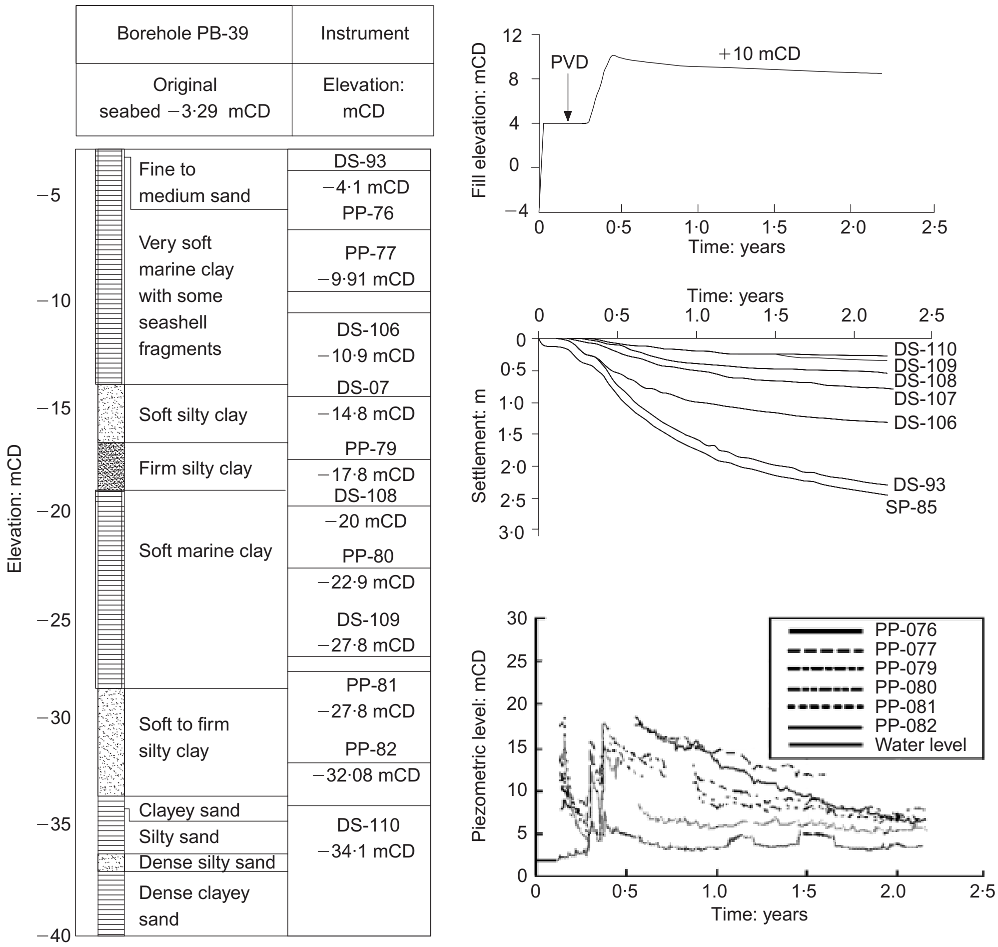


Figure 10. Settlement and pore pressure monitored during reclamation

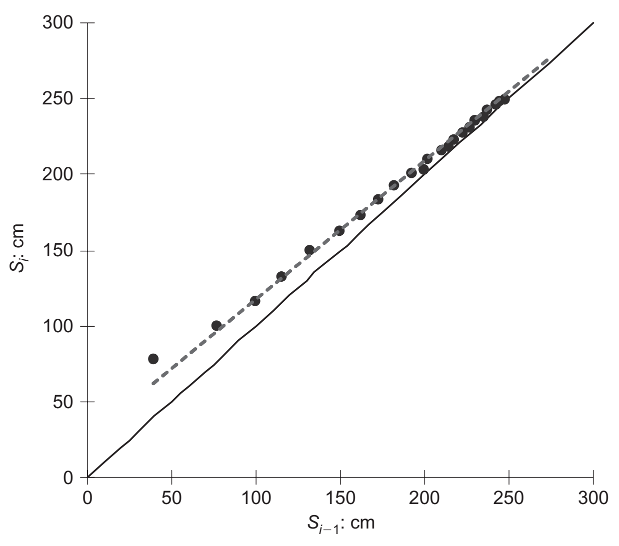


Figure 11. Asoka plot for the ground settlement measured in Figure 10

preconsolidation stress determination using one-dimensional consolidation tests, such as sample disturbances.

4. CONCLUSIONS

The multi-phase Changi East reclamation project was undertaken to reclaim about 2000 ha land offshore along the east coast of Singapore by depositing sand fill onto a thick layer of soft seabed marine clay. Soil improvement works were carried out to consolidate the soft marine clay using PVDs and surcharge preloading. Full-scale pilot tests were carried out on site with full instrumentation to verify the design, and to check the effect of drain spacing on the soil improvement results. The degree of consolidation was assessed using both settlement and pore water pressure data. The following conclusions can be drawn from the study.

(*a*) The pilot test has demonstrated that, under fill 10 m thick, a degree of consolidation of 90% could be achieved within 18 months when PVDs with a 1·5 m square grid spacing were used.

(*b*) Even when PVDs at this close spacing of 1·5 m are used, the rate of consolidation can still be considerably affected by horizontal sand seams. Therefore a detailed site investigation is still necessary for the reclamation project.

(*c*) The average degree of consolidation should be estimated using both settlement and pore pressure data. The degree of consolidation estimated using settlement is normally higher than that using pore water pressure. This can be partially explained by the fact that instruments are often installed at locations where the highest settlement and pore pressure values are measured. This leads to an overestimation of the degree of consolidation when settlement data are used, and

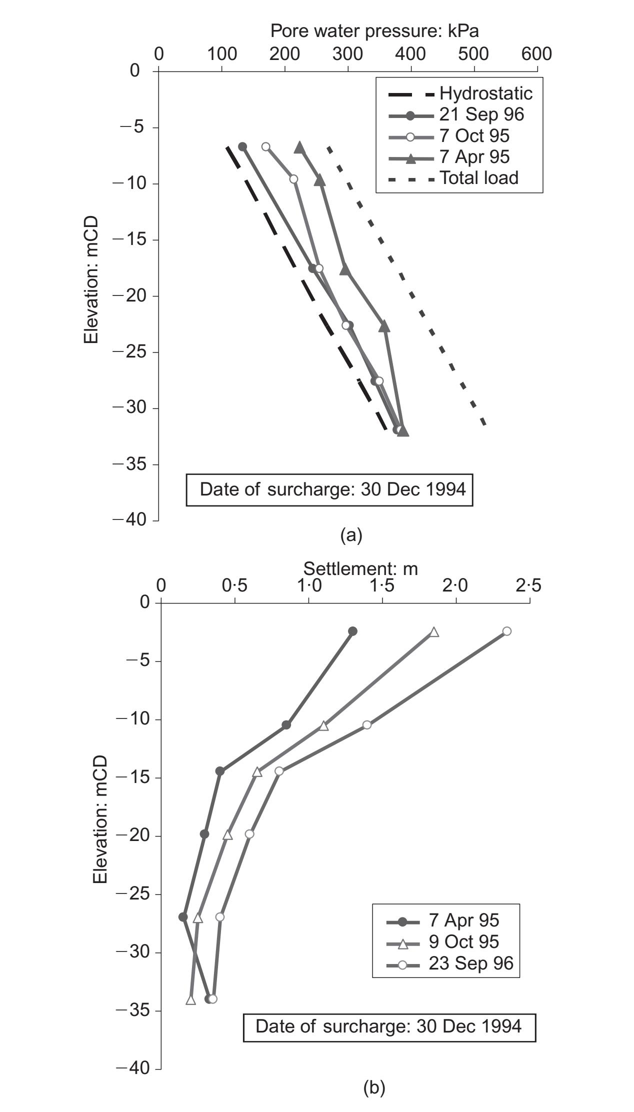


Figure 12. (a) Excess pore water pressures and (b) settlements measured at different elevations and durations in Phase 1B

an underestimation of the degree of consolidation when pore pressure data are used.

(*d*) Long-term CPTU holding tests are an effective way to verify the monitored field pore pressure data.

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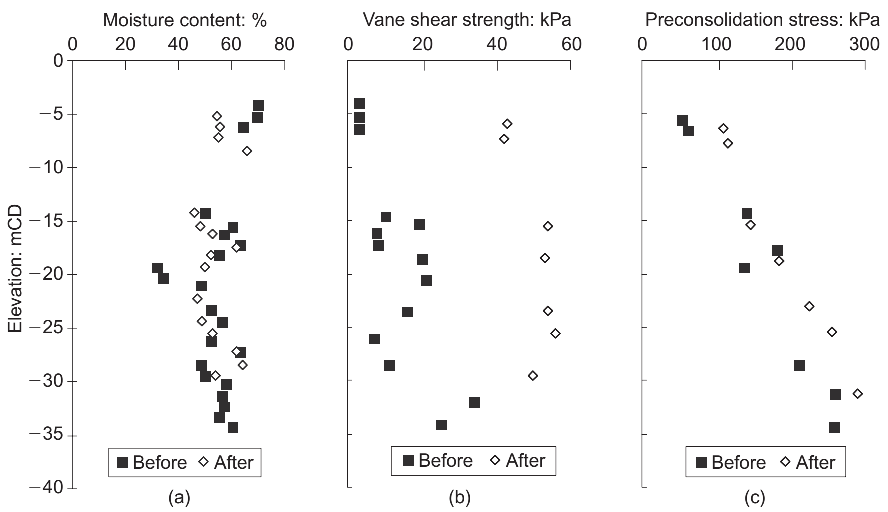

Figure 13. Comparison of soil parameters measured before and after soil improvement: (a) moisture content; (b) field vane shear strength; (c) preconsolidation stress

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