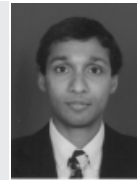**Jian Chu**
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Reclamation of a slurry pond in Singapore

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A case study for the reclamation of a slurry pond as part of an offshore reclamation project in Singapore is presented in this paper. The slurry pond covered an area of 180 ha. The slurry in the pond was recently deposited ultra-soft high-plasticity clay. The water content of the slurry was more than 120% and the undrained shear strength was less than 8 kPa. The reclamation was first carried out by spreading sand fill in thin layers 20 cm thick using a specially designed sand spreader. The filling speed was carefully controlled to allow the slurry to be consolidated before more fill could be placed. Despite the precautions a failure occurred, in the form of mud bursting. As a remedial measure, geotextile sheets were used to cover a total area of 630 000 m² before more sand fill was placed. After the completion of fill placement, fill surcharge and prefabricated vertical drains (PVDs) were used to improve and accelerate the consolidation of the slurry. As the performance of PVDs would deteriorate after they had undergone large deformation, they were installed in two passes. In the first pass PVDs were inserted with a square grid spacing of 2·0 m. After nearly 1·5 m of settlement had taken place, the second pass of PVDs with the same spacing was installed at the centre of the square grid of the PVDs installed in the first pass. After nearly 4 years of consolidation, the top of the slurry had settled more than 3 m. The undrained shear strength had also increased substantially. Therefore the use of PVDs for the improvement of the ultra-soft slurry was successful in this project.

Figure 1. Location of slurry pond and site plan of Changi East reclamation project
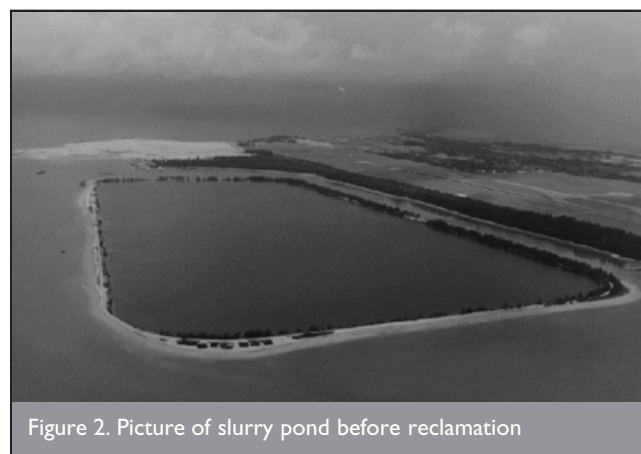
Figure 2. Picture of slurry pond before reclamation

1. INTRODUCTION

The Changi East reclamation project was carried out between 1991 and 2005 to create 2000 ha of land offshore for the extension of the Changi International Airport and other infrastructure developments in Singapore.[1,2] This project included the reclamation of a 180 ha slurry pond. The location of the slurry pond and the site plan of the Changi East reclamation project are shown in Figure 1. A picture of the slurry pond, taken before reclamation, is shown in Figure 2. The slurry pond was trapezoidal in shape, approximately 2000 m long and 750–1050 m wide, as shown in Figure 3. This slurry pond was created by dredging the original seabed to an elevation of −22 mCD (Admiralty chart datum, where mean sea level is at +1·6 mCD) between 1975 and 1978 as a
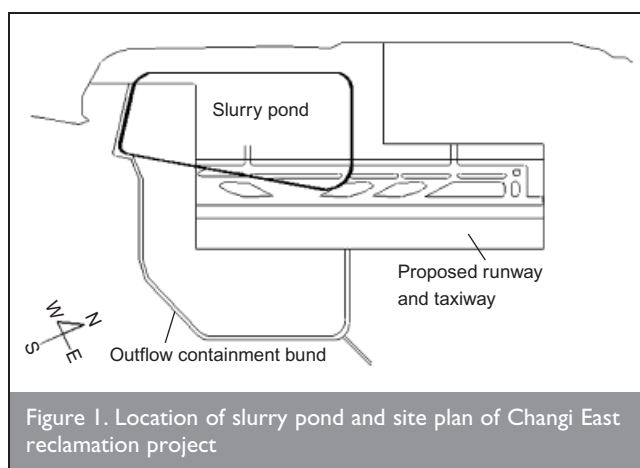
borrow pit. A containment sand bund was constructed around this borrow pit in 1986 to the crest level of about +5 mCD. Subsequently silt and clay washings from other sand-quarrying activities in the eastern part of Singapore were transported through pipelines with water and discharged into this contained area to form a pond. Therefore the slurry inside the pond consisted mainly of clay and silt.[3,4] The water level in the pond was at +3 mCD. During subsequent years suspension in the slurry had settled in the pond, and the majority of the slurry was undergoing self-weight consolidation.[4]

Figure 3. Dimensions of slurry pond, and isolines indicating elevation of top of slurry in the pond (mCD). The water table was at +3 mCD
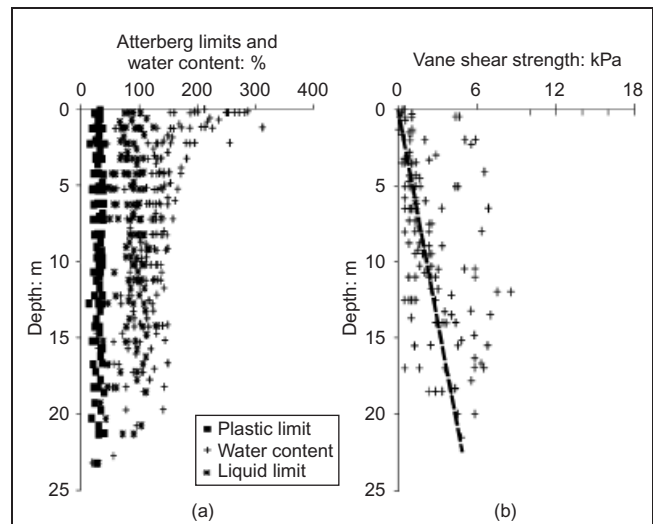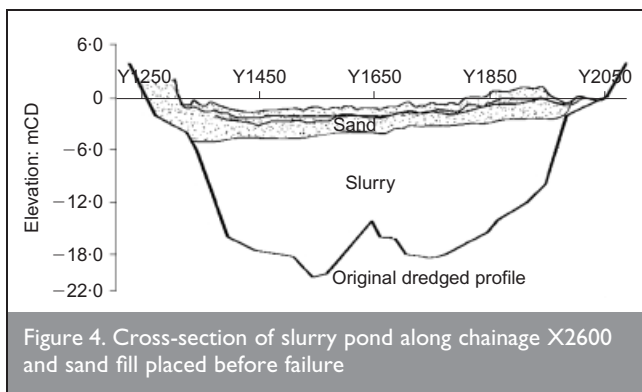

Figure 5. Variation of basic slurry properties with depth: (a) liquid limit, plastic limit and water content; (b) vane shear strength

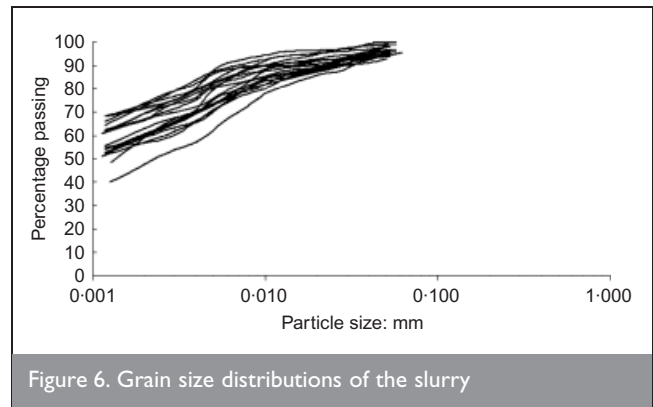Figure 6. Grain size distributions of the slurry

Property	Value
Liquid limit: %	65–115
Plastic limit: %	22–45
Water content: %	75–180
Bulk unit weight: kN/m^3	11·0–16·0
Specific gravity	2·66–2·70
Void ratio	2·0–4·5
Fines content: %	70–90

Table 1. Properties of soil in the slurry pond

2. SITE CONDITIONS AND PROPERTIES OF SLURRY

Site investigation for the slurry pond was carried out in 1992. It included 82 gamma-ray density probes and 20 boreholes with sampling and in situ vane shear tests. The sampling was carried out using twist samplers. The twist sampler provided a closing door on a twisting mechanism, and thus allowed very soft core samples to be taken continuously. A cross-section of the soil profile in the slurry pond along chainage X2600 is shown in Figure 4. The elevation at the top of the slurry varied from -1 to -5 mCD as shown by the isolines in Figure 3. There was no clear boundary between water and slurry, as the transition was rather gradual. The top of the slurry was taken as the elevation at which the density was greater than 1·1 Mg/m^3. The bottom of the slurry was in the range 0 to −22 mCD. The thickness of the slurry ranged between 3 and 20 m.

The slurry material consisted mainly of high-plasticity clay. The variation of liquid limit, plastic limit and moisture content distribution with depth is shown in Figure 5a. The water content of the slurry varied from 60% to 300%, but was mainly in the range of 120–180%. The bulk density was low, with values ranging from 1·2 to 1·4 Mg/m^3. The slurry was still undergoing self-weight consolidation. The undrained shear strength profile of the slurry as determined by field vane shear testing is shown in Figure 5b. The majority of the data points in Figure 5b show an increasing trend with depth at a rate of 0·2 kPa/m depth. Even so, the values were less than 8 kPa for soils at 20 m depth. The grain size distribution of the slurry is shown in Figure 6. It can be seen that the upper bound of D_{50} was 0·024 mm, but mostly in the range smaller than 0·001, and D_{85} was in the range 0·004–0·02 mm. The fines content was in

the range 70–93%. The other physical properties of the soil in the slurry pond are shown in Table 1.

3. RECLAMATION AND SOIL IMPROVEMENT WORKS

3.1. Sand placement by spreading

As the top surface of the slurry had little strength, direct hydraulic placement of sand fill on top of the slurry would have caused the slurry to be displaced by the penetration of the sand fill. The sand fill therefore had to be spread in thin layers using a specially designed sand spreader, as shown in Figure 7. The sand fill, with a high water to sand ratio, was pumped into the spreader using a suction dredger. The sand spreading was carried out by moving the spreader repeatedly from left to right along the x-direction (see Figure 3). Small lifts of 20 cm were

Figure 4. Cross-section of slurry pond along chainage X2600 and sand fill placed before failure
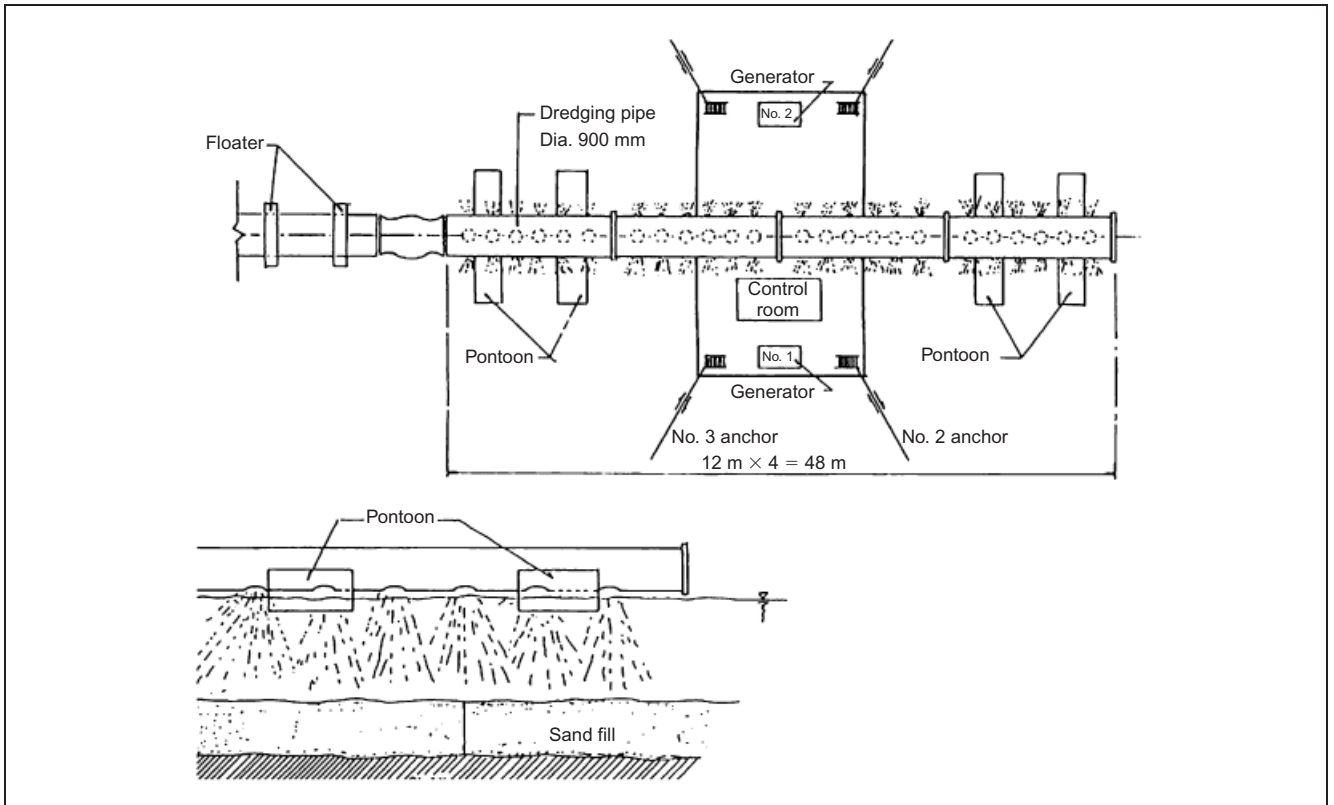
Figure 7. Sand spreading system used for slurry pond

used in the first phase of spreading to ensure stability of the fill. A waiting time was given between each 20 cm lift to allow the slurry to gain strength under the small surcharge of the sand fill. A hydrographic survey carried out along chainage X2600 when the sand layer reached 0 mCD is shown in Figure 4. When the fill reached an elevation between 0 and +2 mCD, a failure in the form of slurry bursting occurred at the location shown in Figure 8. A cross-section across the failure area along chainage Y1700 is shown in Figure 9. The boundary between the sand fill and the slurry was established by observation of the borehole samples. It can be seen from Figure 9 that, at the burst location, the sand fill was displaced to two sides and the mud shot out to an elevation of +2 mCD.

The failure was attributed to uneven settlements and uneven spreading of the sand fill. It can be seen from Figures 4 and 9
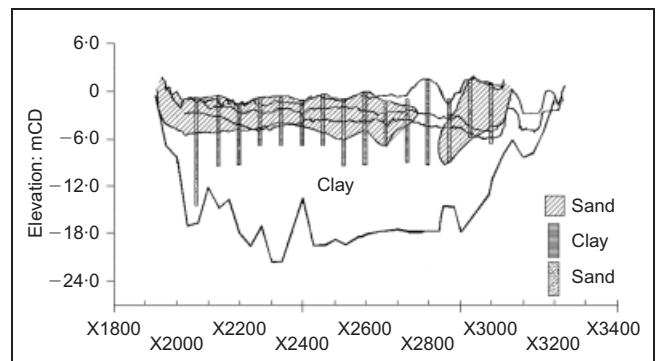

Figure 9. Cross-section of slurry pond along chainage Y1700 after failure
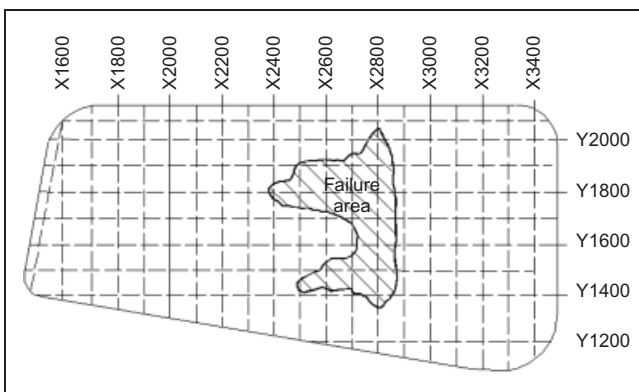
that the thickness of the slurry layer varied considerably across both the x and y directions. The amount of compressibility of the slurry layer was thus different across both the x and y directions, even when the load applied was the same. The pore water pressure dissipation rates would also be different. The profile of the sand fill before failure along X2600 is shown in Figure 4. The profile of the sand fill along Y1700 can also be seen from the stable sections in Figure 9. Although the sand fill was spread carefully, a 1 m or so difference in the sand layer thickness could have resulted. The difference in loads, coupled with the different pore pressure dissipation rates in the soil, led to upheaval of the slurry at some locations. This in turn displaced the sand above the upheaved slurry to two sides and subsequently caused the overburden pressure provided by the sand to be reduced.

Figure 8. Location of failure area where bursting of slurry occurred

When the pore pressure became higher than the overburden pressure a mud burst occurred.

3.2. Remedial measures using geotextile

As a remedial measure, geotextile fabric was used to cover the failure area. Geotextile has also been used for other, similar projects.[5,6] Before this, the burst mud was partially removed using a high-capacity submersible mud pump. The tensile strength of the geotextile required was estimated from the equation[7]

$$
\boldsymbol{p} - \boldsymbol{T}_g \sin\beta = \boldsymbol{c}_u N_c + \left(\gamma' D + T_g \sin\beta\right) N_q \\
+ 0 \cdot 5 \gamma_2 N_\gamma
$$

where $\boldsymbol{p}$ is the vertical stress due to sand fill; T_g is the tensile strength of the geotextile; β is the angle of curvature of the geotextile due to arching, determined based on the length and width of the geotextile sheet used and the predicted maximum settlement of the geotextile along the centreline; $\boldsymbol{c}_u$ is the undrained shear strength of the slurry; γ' is the effective unit weight of underlying soil; D is depth and N_c, N_q and N_γ are Terzaghi's bearing capacity coefficients.

Based on the calculation, a tensile strength of 150 kN/m in both the warp and weft directions was determined. Two types of woven geotextile were used. The first type, HS150/150, had a tensile strength of 150 kN/m in both directions. It was placed in a single layer to cover an area 700 m by 300 m, shown in Figure 10 as area 2 on the left. Owing to the shortage of supply of this type of geotextile, a second type, HS100/50, with tensile strengths of 100 kN/m and 50 kN/m in the warp and weft directions respectively, was also used. By overlaying two layers with different warp and weft directions, a combined 150 kN/m tensile strength was achieved. The second type of geotextile was placed in two layers over an area 700 m by 600 m, shown in Figure 10 as area 3 on the right. Therefore the total area covered by the geotextile was 700 m × 900 m, or 630 000 m².
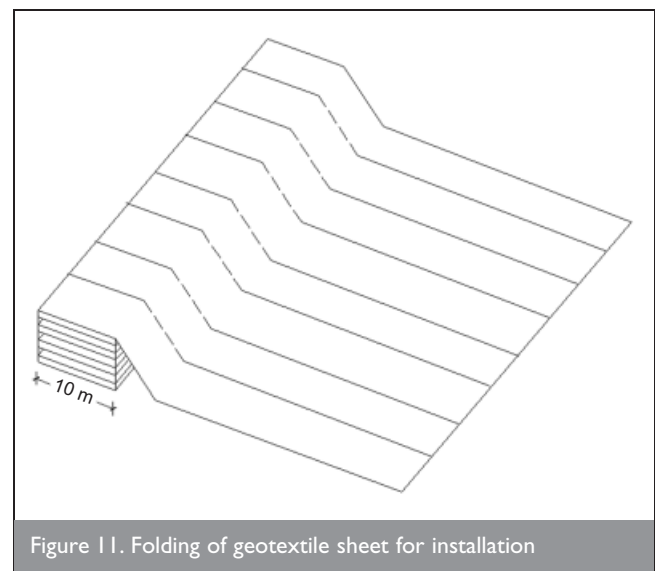
The installation was carried out in the following sequence. The geotextile as supplied was in rolls 5 m wide and 90 m long. They were sewn together using portable sewing machines to form a geotextile sheet either 700 m × 600 m or 700 m × 900 m. When the geotextile was sewn together to form one piece, it was extremely heavy and required considerable force to be pulled during installation. The total weight of the geotextile
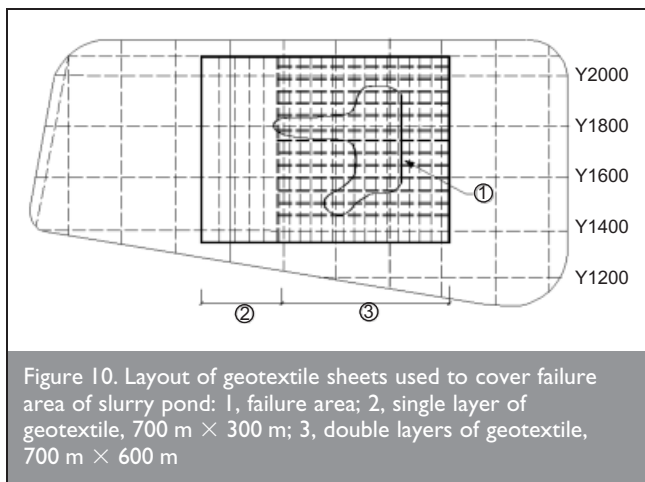
used was 320 t. To overcome this problem, the geotextile sheet was folded into zigzag strips 10 m wide, as shown in Figure 11. This made unfolding of the geotextile sheet much easier. The sheet was anchored at one end by placing sand on top of it. The geotextile was then folded back to cover the sand, which would be covered again by sand fill placed subsequently. The amount of sand used was 3–6 m³ per metre run. The other end of the geotextile sheet was connected to pipelines floating on water. The pipelines were pulled slowly towards the other side of the pond using seven bulldozers. Plastic buoys were also used for the inner area of the geotextile sheet to keep it floating on top of the water. When the geotextile sheet reached the other side of the pond, it was fixed in place by placing sand fill on top in similar fashion as for the other end. For the area where double layers of geotextile were used, the second layer was placed in the same way after the first layer had been installed. After the geotextile sheet was properly anchored at both ends, the plastic buoys were removed to allow the geotextile sheet to sink. At the interface of the two areas where the two different geotextiles were used, an overlapping length of 50 m was provided. A sand layer 50 cm thick was placed on the overlapping area for anchoring.

3.3. Second phase of sand spreading

After placement of the geotextile, sand fill spreading resumed, using the same sand spreader, until the ground elevation of +4 mCD was reached. In order to achieve this level, the containment bund surrounding the slurry pond was elevated to +6 mCD and the water level to + 5·5 mCD by pumping in seawater. Spreading was still carried out in stages, with the thickness of each layer controlled within 50 cm. Mini-cone penetration tests (MCPTs) were performed to determine the soil profile of the slurry pond after fill placement. The MCPT test used a 30° apex cone with a cone tip area of 2 cm². Based on the MCPT results, the soil profile along chainage Y1700 is shown in Figure 12. It can be seen that the sand fill was successfully placed on top of the slurry, except for the existence of some small sand pockets. The results of MCPT test 47 are given in Figure 13. Based on the tip resistance and friction ratio shown in Figure 13, it can be seen that at this location the sand layer ranged from zero to about 8 m deep, the clay layer started from 11 m, and a mixed clay and sand layer occurred in between. The

Figure 10. Layout of geotextile sheets used to cover failure area of slurry pond: 1, failure area; 2, single layer of geotextile, 700 m × 300 m; 3, double layers of geotextile, 700 m × 600 m

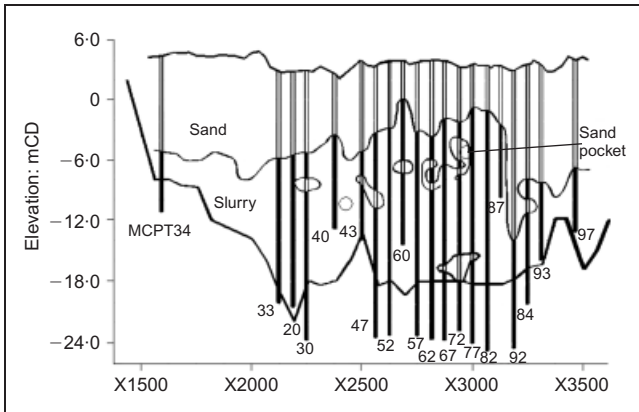Figure 11. Folding of geotextile sheet for installation

Figure 12. Soil profile along chainage Y1700 after reclamation as identified by mini-CPT tests. MCPT borehole numbers are shown. For each borehole, darkened area indicates presence of clay; the rest is sand
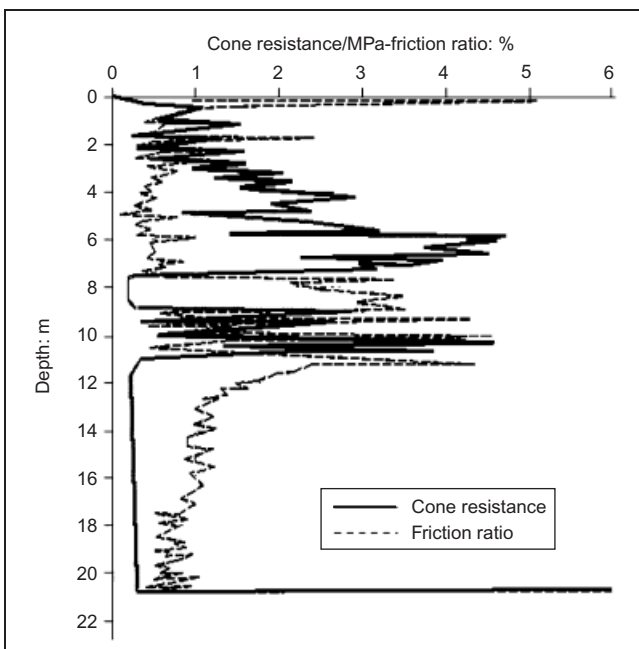
large ground settlement would cause the drains to buckle and thus affect their performance. The deterioration in the performance of the vertical drains installed in the first pass was indicated by the fact that the rate of settlement had reduced to 20 cm/month, although the pore water pressure remained very high in the soil.[4,9] The installation of PVDs itself assisted in the dissipation of the pore water pressure, as mud was seen to come out through the annulus of the mandrel.[10] This is another advantage of the two-stage PVD installation method. The final surcharge level of + 9 mCD was placed after installation of the second round of vertical drains.

The settlement and pore water pressure dissipation within the first 480 days are shown in Figure 14. The positions of the settlement and pore pressure measuring devices are shown in Figure 15. It can be seen from Figure 14b that, within the first 480 days, there was 2·7 m of ground settlement, but the pore pressure dissipations (Figure 14c) were slow. Using Figure 14c,

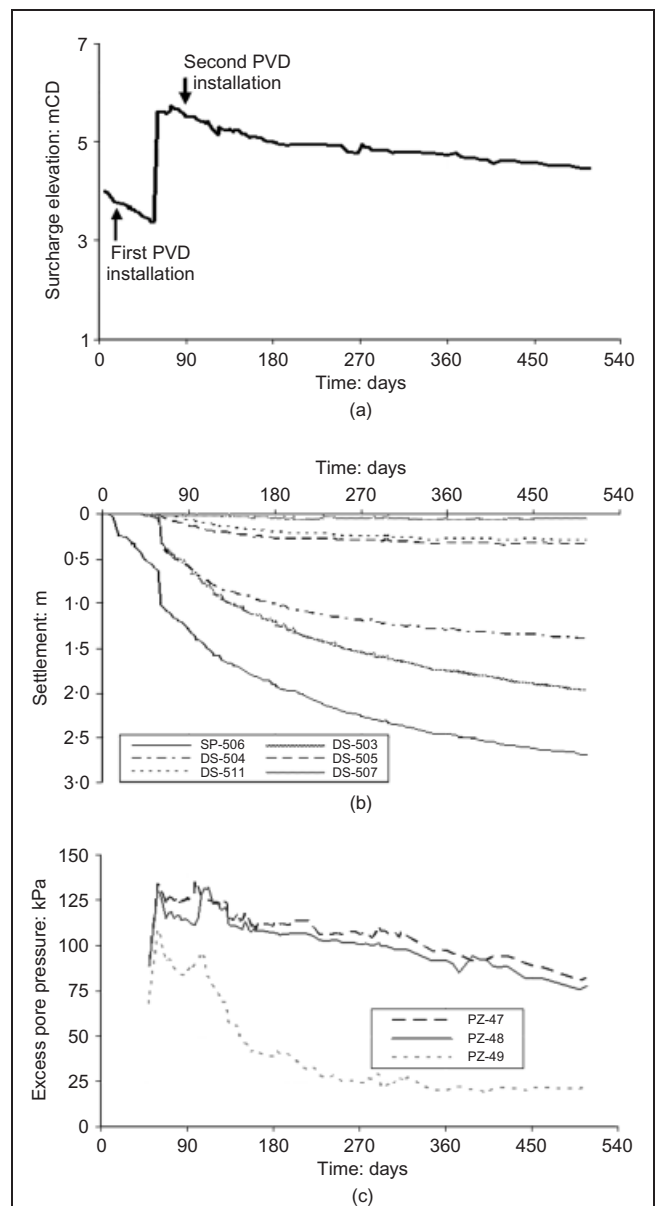Figure 13. Results of mini-CPT test MCPT47

Figure 14. Monitoring data measured against time during reclamation of slurry pond: (a) surcharge variation; (b) settlement; (c) excess pore pressure. SP, settlement plate; DS, deep settlement gauge; P2, vibrating wire piezometer

low tip resistance for the clay layer indicates that the shear strength of the clay slurry was still low after the sand fill had been placed. Therefore further soil improvement was carried out by preloading with PVDs after the sand placement.

3.4. Installation of vertical drains
Before installation of the PVDs at the +4 mCD level, the water level in the slurry pond was lowered to +3 mCD. Colbond drains CX1000 were installed with 2 m × 2 m square spacing as a first pass. The specifications for the PVDs are given in Reference 2. The procedure discussed by Chu et al.[8] was followed in selecting the type of drain and drain spacing. The surcharge in the first stage was placed to +6 mCD. The settlement of the fill was monitored using an instrumentation scheme as detailed by Bo et al.[9] After approximately 1·5 m of settlement had taken place, a second pass of vertical drains was installed with the same square grid of 2·0 m in the centre of the square grid of PVDs installed in the first round. The combined effective drain spacing is 1·4 m × 1·4 m. It was necessary to install the PVDs in two passes, as the
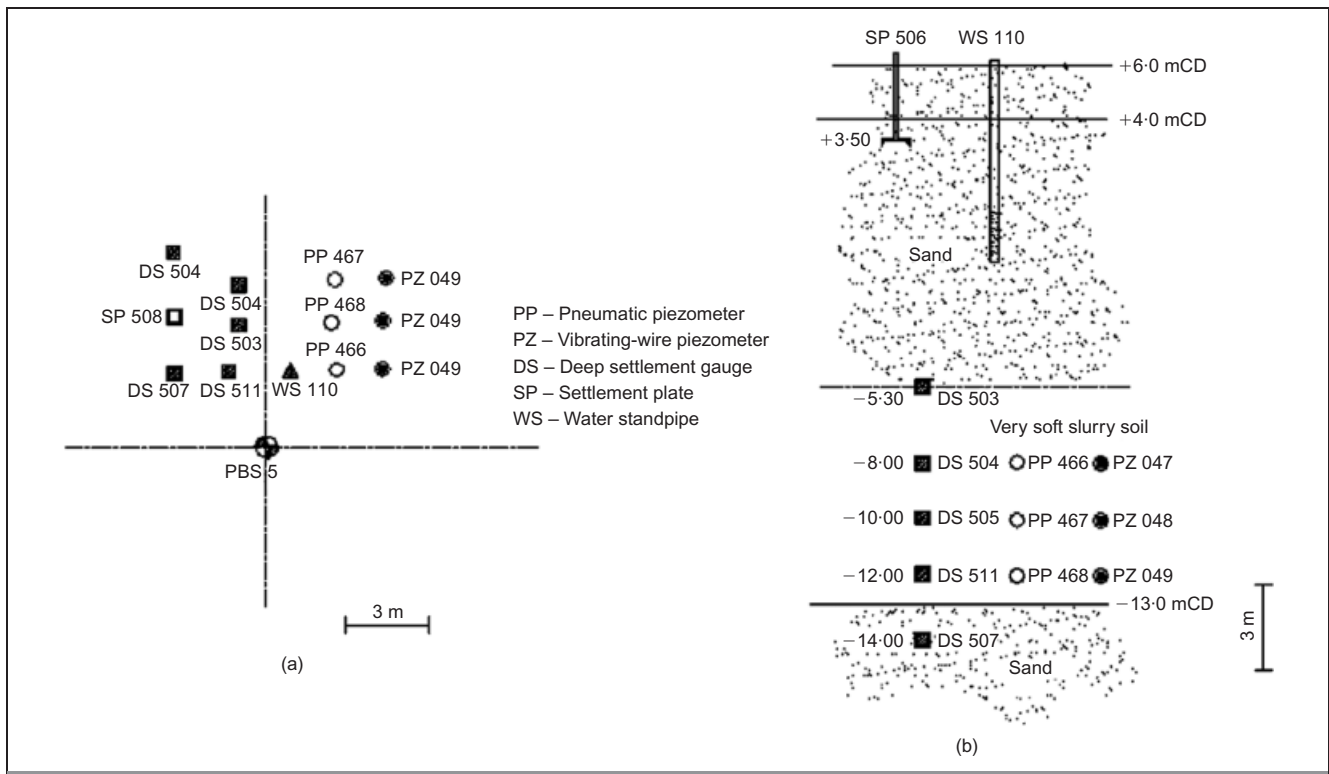
Figure 15. Arrangement of instrumentation: (a) plan; (b) elevation

the pore pressure distribution in 480 days is plotted in Figure 16. Based in Figure 16, the average degree of consolidation can be estimated as 42%. Using this value, and taking the clay slurry layer as 7·8 m thick and double-drained, c_h can be back-calculated as 0·27 m^2/year when c_v is assumed to be 0·1 m^2/year. This value is of the same order of magnitude as that reported by Bo et al.[4] Applying Asaoka's method[11] to the settlement data in Figure 14b, the average degree of consolidation is calculated as 91%, which seems too high. Therefore Asaoka's method is not applicable in this case, where large deformation of sediment was encountered.
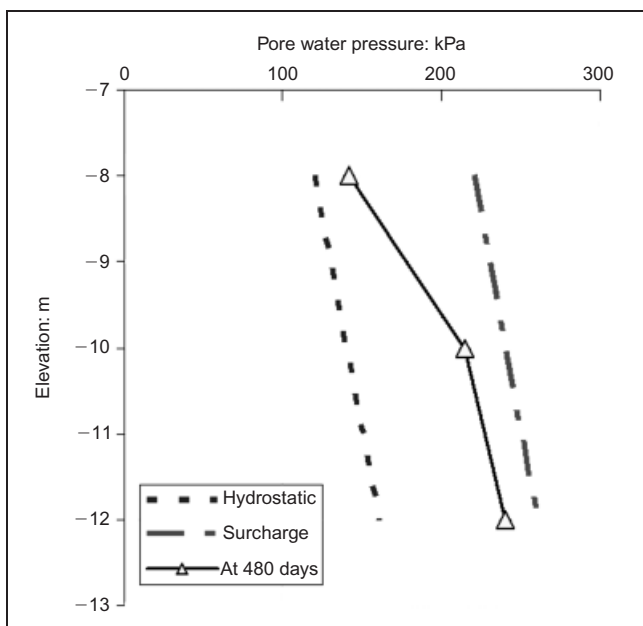

Figure 16. Pore water pressure profile 16 months after surcharge

Field vane shear (FVT) and piezocone (CPTu) tests were conducted 14 months after application of the surcharge. A comparison of the undrained shear strength profiles obtained from the FVT tests conducted before surcharge and 14 months after surcharge, as well as from the CPTu tests conducted 14 months after surcharge, is shown in Figure 17. Note that the ground had settled by more than 2 m within 14 months, as shown in Figure 14a. This explains why the starting-points of the in situ tests conducted before and after the surcharge are different. The undrained shear strength determined by CPTu was based on a cone factor of 12.[3] In Figure 17, the undrained shear strength profile estimated by assuming a uniform degree of consolidation of 90% is also plotted for comparison. It can be seen that the slurry had little shear strength initially. However, after consolidation, a substantial increase in the undrained shear strength had been achieved 14 months after the surcharge placement. As mentioned, a silty sand was present at an elevation of −12·5 mCD and a sand blanket was placed on the top surface. Therefore there was combined vertical and horizontal drainage near the top and bottom boundary. This explains why the shear strength increment was greater at both the top and the bottom, where the undrained shear strength had reached a value equivalent to 90% of pore pressure dissipation. The consolidation in the middle of the clay layer, ranging from −9·5 to −12·0 m, was contributed mainly by horizontal drainage to the PVDs, as indicated in the analysis by Chu et al.[10] The increase in the undrained shear strength in this section was thus relatively uniform, as indicated by the dashed lines. Using the undrained shear strength profile shown in Figure 17, the average degree of consolidation can be estimated as 45%. This agrees with that estimated using the field pore pressures shown in Figure 16.

The much delayed dissipation of pore pressure in the slurry is typical of consolidation of ultra-soft soil, as the slurry may
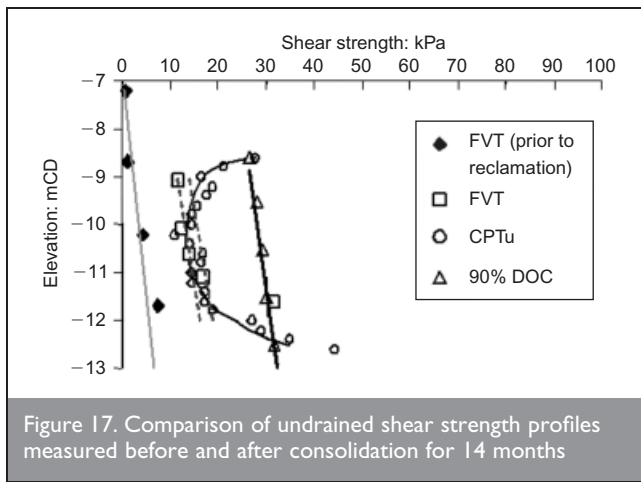
Figure 17. Comparison of undrained shear strength profiles measured before and after consolidation for 14 months

have to undergo sedimentation and self-weight consolidation stages before it can be transformed into 'soil'.[10,12] Excessive buckling of the PVDs, possible clogging of the filter of the PVDs, the Mandel–Cryer effect and non-uniform consolidation of soil around the PVDs are possible other reasons.[10,12]

Owing to the slow dissipation in pore pressure, it took several years to complete the primary consolidation despite the use of PVDs at an effective spacing of 1·4 m × 1·4 m, achieved by combining both passes of PVD installation. The complete loading history and surface settlement curves at the same location as shown in Figure 14 are presented in Figure 18. As can be seen in Figure 18a, one more surcharge increment of 5·5 m was applied after 2·3 years. The additional settlement induced by this surcharge increment was only about 0·5 m, which was much smaller than the nearly 3 m of settlement developed in the preceding stage. This indicated that the soil had been much improved over the preceding 2·3 years. The total settlement 4 years after preloading was 3·5 m, and the degree of consolidation achieved was 85%. Ground settlements
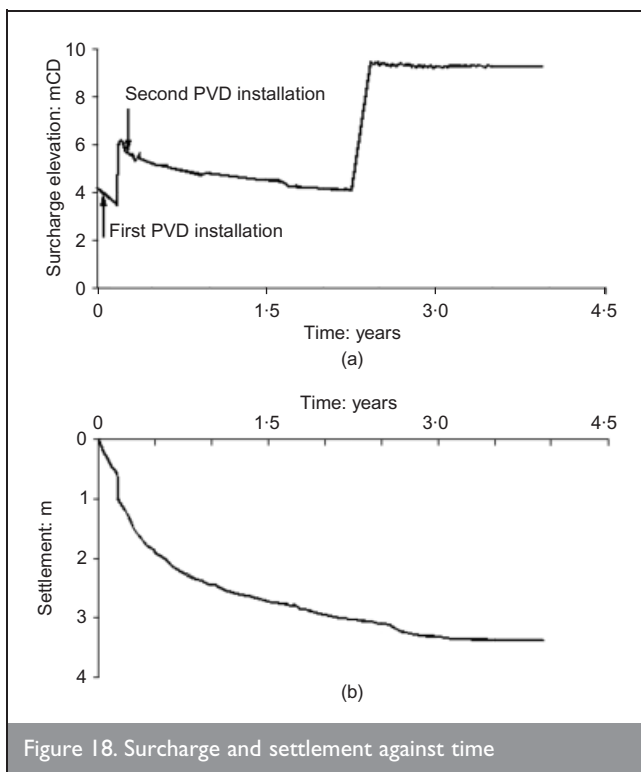

Figure 18. Surcharge and settlement against time

at other locations were also monitored, and settlements greater than 3·5 m were observed.[9] It was planned to remove the surcharge only once the land was used in the future. So far, only the eastern boundary of the slurry pond (see Figure 1), where the slurry layer was thin, has been used. The surcharge in most of the areas has not yet been removed. The long-term settlement of the slurry would be an interesting topic to be reported separately.

4. CONCLUSIONS

A case study for the reclamation of a slurry pond as part of the offshore Changi East reclamation project in Singapore is presented in this paper. As the slurry was ultra-soft, direct placement of sand fill onto the slurry caused the mud to burst, even though the fill was placed in thin layers using a specially designed sand spreader. As a remedial measure, geotextile sheets were used to cover a total area of 630 000 m² (700 m × 900 m) before further fill placement. The slurry was treated using two passes of PVDs and fill surcharge. The following conclusions can be drawn from this case study.

(a) For reclamation over ultra-soft, high water content slurry, direct placement of fill can be difficult. An attempt was made to place the fill in successive thin, 20 cm layers using a specially designed sand spreader. However, this was not entirely successful as a failure in the form of a mud burst occurred around the centre of the pond. Uneven settlement and uneven spreading of the sand fill were considered to be the main reasons for the failure. The uneven settlement was due mainly to the differing compressibility of the slurry layer with varying thickness across the cross-section. As a remedial measure, geotextile was used as a reinforcement to cover the failed area of 630 000 m² before fill placement.

(b) When PVDs are used for the improvement of ultra-soft, high water content soil, their performance may deteriorate after they have experienced large deformation. This problem was overcome in this project by installing PVDs in two passes, one at the start and another after some settlement had occurred. The second pass of PVDs was installed in the centre of the grid of the PVDs installed in the first round. This method was proven to be successful. In 4 years, the top of the slurry had settled by more than 3 m. The undrained shear strength of the slurry soil had also increased substantially.

(c) Even with the use of PVDs at a close spacing, the consolidation of ultra-soft soil is a slow process, and the dissipation of pore water pressure often lags behind the settlement. In this case, the degree of consolidation can be considerably overestimated using the settlement data. It is therefore necessary to estimate the degree of consolidation using pore water pressure or undrained shear strength profiles, as suggested in this paper.

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