Compression and Consolidation of Clayey Soils, Yoshikuni & Kusakabe (eds) © *1995 Balkema, Rotterdam. ISBN 90 5410 537 2*

Changi East Reclamation Project

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ABSTRACT: This paper gives an overview of the Changi East Reclamation Project involving about 170 million m^3 of sand fill for airport expansion and other infrastructural development. The project is underlain by deep deposits of soft marine clays and involves soil improvement using prefabricated vertical drains and surcharge. The sand fill is densified by dynamic compaction. The slope stability problems of the coastal protection rock bunds as well as the problems of capping a very soft silt pond of some 20 m deep of quarry waste shall be discussed. Comparisons are made between laboratory and in-situ testing methods to determine the properties of the marine clay. Lastly some preliminary results of a vertical drain pilot test are presented.

1 INTRODUCTION

The construction of Changi Airport in the late 1970s necessitated the reclamation of about 700 ha of land. The project involved dredging and hydraulic filling of 40 million m^3 of sand by cutter suction dredgers. It was completed in 1979 (Choa, 1980). Changi Airport was completed in the early 1980s. In order to cater for future expansion of Changi Airport it is planned to reclaim a further 1500 ha of land using about 170 million m^3 of sand. Due to the enormity of this undertaking it was decided to divide the next phase of reclamation into three sub-phases namely Changi East Reclamation Phase 1A, 1B and 1C. The overall Reclamation at Changi East is shown in Fig. 1.

The Reclamation Phase 1A commenced in January 1992 and is scheduled to be completed in January 1997. This phase was undertaken first in order to form a protective arm so that the subsequent phases of reclamation can be carried out in a relatively calm marine environment. It involves substantial coastal protection works in the form of rock bunds, headlands, sheet pile and pipe pile retaining walls.

The Reclamation Phase 1B which commenced about a year later in March 1993 is scheduled to complete in March 1988. This phase covers the areas required for the future Runway 3 of Changi Airport and the associated taxiways and high speed turnoffs. It also covers the future terminal building and aircraft apron areas. Substantial soil improvement works is involved as the runway is underlain by deep deposits of soft marine clays. An enclosure of about 180 ha has been formed by sand dykes around a borrow pit where material was taken for reclamation of Changi Airport in the 1970s. This enclosure referred to as the "Silt Pond" had been filled by silt clay washings from sand quarrying operation. The up to 20 m thick layer of ultra soft silt and clay slurry within the "Silt Pond" shall be capped and improved under the Phase 1B reclamation project.

Reclamation Phase 1C is presently being designed and will probably commence sometime in 1996. This paper will deal mainly with the geotechnical problems associated with the first two phases of Changi East Reclamation Project.

2 SOIL CONDITION

Several geophysical surveys employing surface-towed boomer profiling systems were carried out. Soil borings with drilling rigs mounted on jack-up pontoons were also carried out prior to the

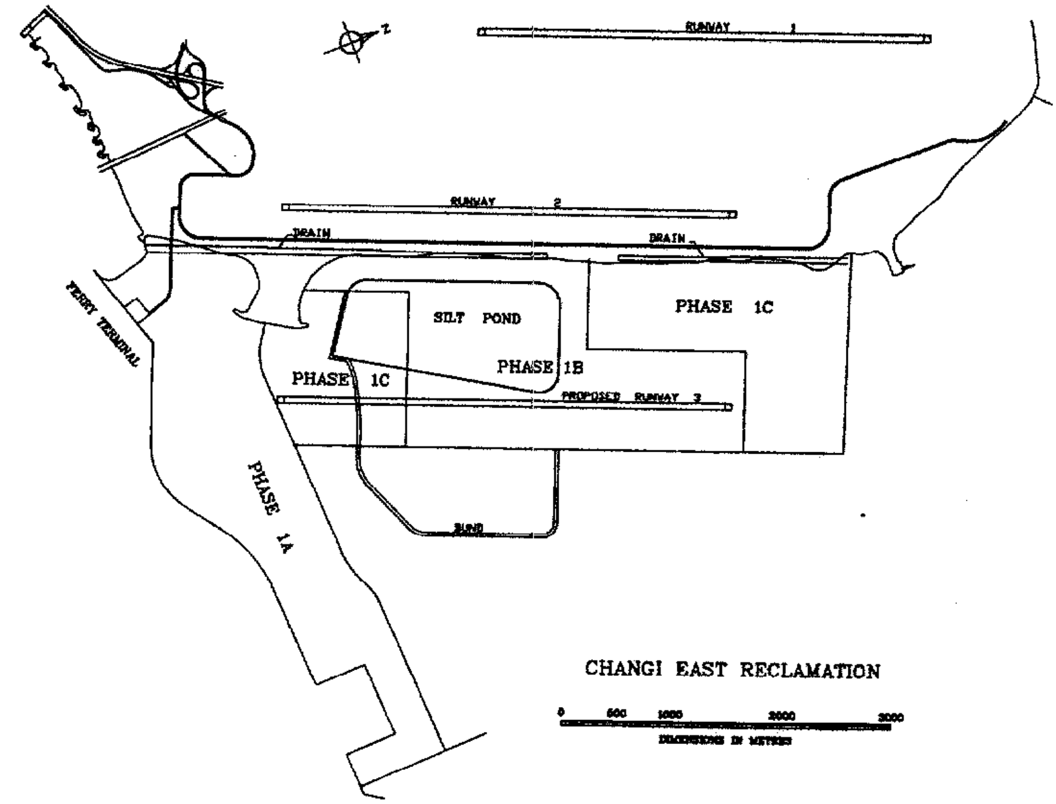

Fig. 1 Overall Site Plan

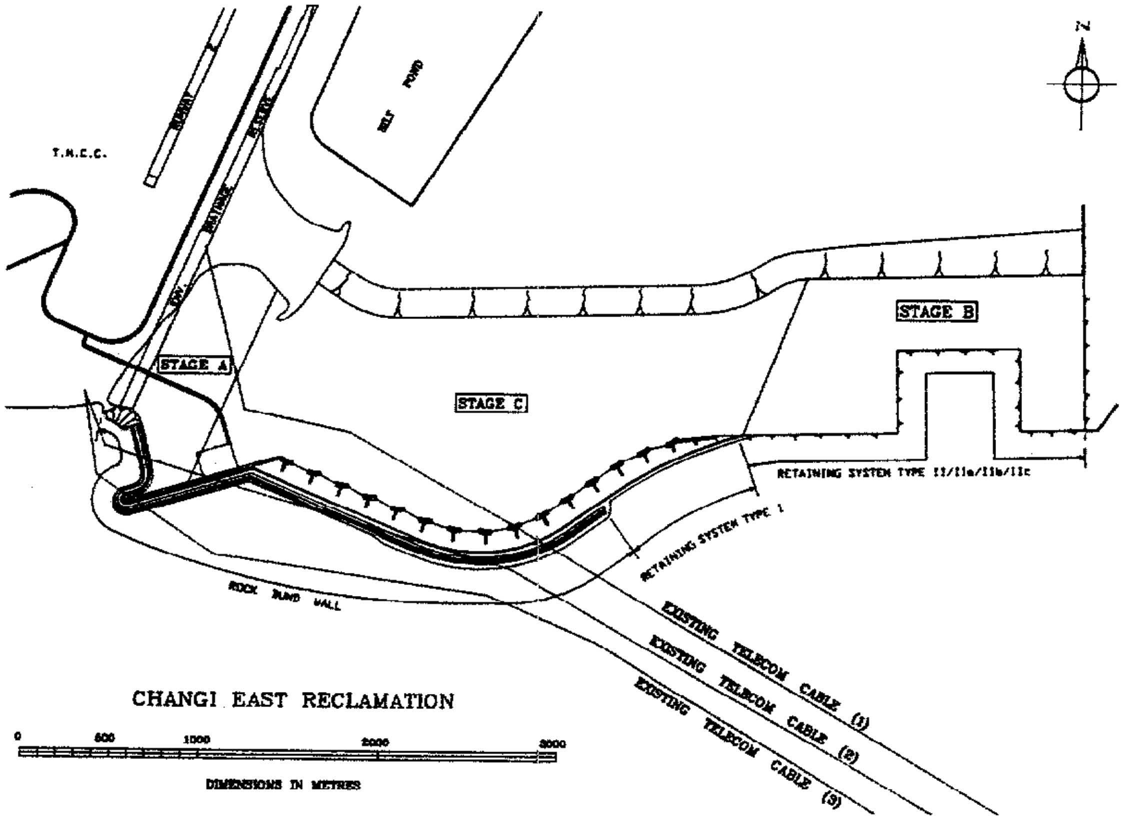

Fig. 2 Reclamation Phase 1A

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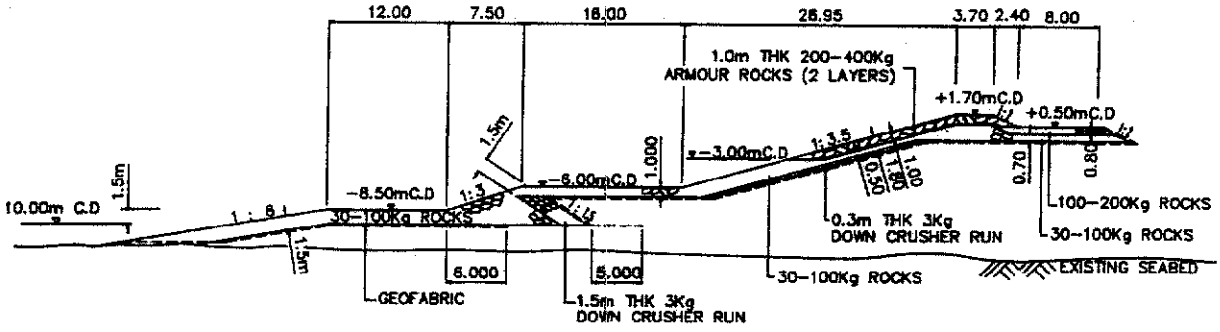


TYPICAL ROCK BUND SECTION

ig. 3 Rock Bund Wall

:clamation works. The site investigation revealed ıat the site is underlain by the Old Alluvium omprising of a cemented silty clayey sand. This eposit is in turn overlain by the Kallang Formation omprising of soft to firm silty clays and sands of ıte Pleistocene and Recent deposits which are of ıarine and estuarine origin. The infilled valleys of ıarine clays are up to 40 to 50 m deep and have ıirly steep side slopes. The marine clay deposits re of different ages generally separated by stiff :ddish silty clays and peaty clays caused by xposure of the seabed to the atmosphere during the se and fall of the sea level in the geological past. 'he marine clay properties appear to differ in the orthern half of the site from those in the southern alf. The approximate line of separation seems to e about a north-west to south-east line slightly to ıe north of the 'Silt Pond'. As the properties of the ıarine clay plays an important role in the design of ıe works it shall be discussed further in one of the ›llowing sections.

RECLAMATION PHASE 1A

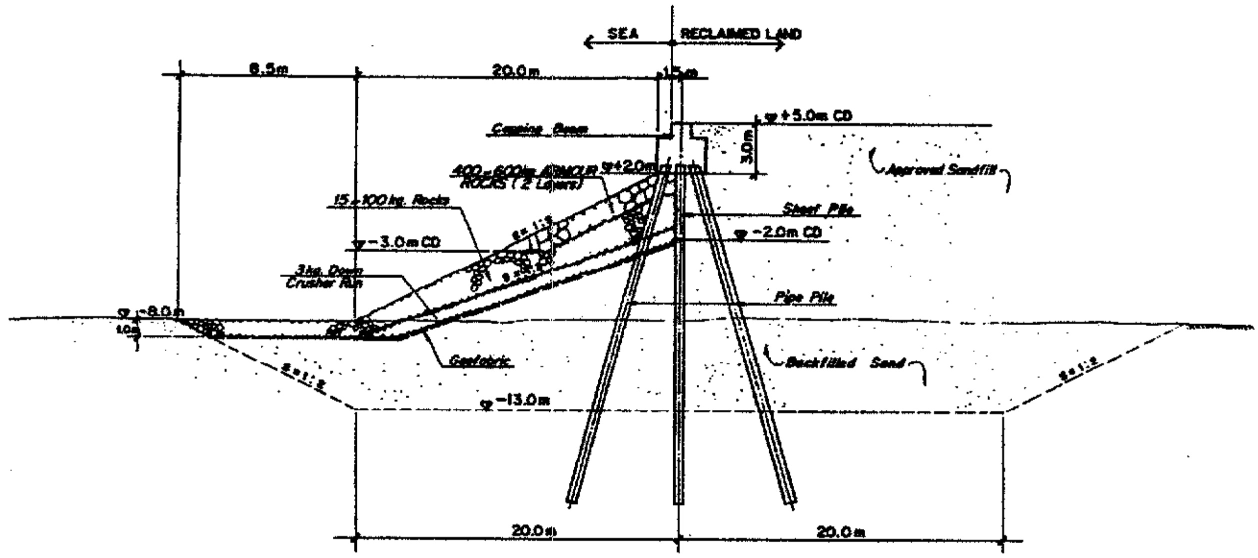
'he Changi East Reclamation Phase 1A involves ıe reclamation of about 550 ha of land with about 3 million m³ of sand (Fig. 2). The sand is 'ansported from the borrow sources about 20 to 40 m away by trailing suction hopper dredgers as 'ell as hopper barges for direct dumping at the :clamation site. Direct dumping is carried out at ıe reclamation site where the sea-bed is generally etween -5mCD and -10mCD (Chart Datum is .6m below mean sea level). The direct dumping 'as used to raise the level of reclamation to - mCD. The remaining reclamation to +5mCD is arried out by hydraulic filling using cutter suction redgers. Where the sea-bed is shallower, sand was placed hydraulically through 600 mm diameter pipe lines. Of the transported sand about 29 million m^3 is placed by direct dumping from bottom opening barges and trailing suctions hopper dredgers and 34 million m^3 by hydraulic fill using cutter suction dredgers. Three trailing suction dredgers of 4,500 to 6,000 m^3 capacity, six bottom opening barges of 1,500 to 3,000 m3 capacity, three pump dredgers and two cutter suction dredgers of 9,500 HP and 12,000 HP were employed for the reclamation works. The dumping of the sand was controlled by using a Geographical Positioning System to ensure a systematic sequence of reclamation.

The sand used for the reclamation comes primarily from two borrow sources. One is a very uniform light brown sub-angular fine to medium sand whilst the other source yielded a fairly uniform light brown sub-angular medium to coarse sand. The uniformity coefficients of the sand ranged from about 1 to 5. The fines content (i.e. less than 75 microns) is less than 5%.

3.1 Coastal protection works

A rock bund wall together with headlands and sheet pile retaining walls were used for coastal protection. Fig. 2 shows the location of the works and Figs. 3 and 4 show typical sections of these works.

About 930,000 m^3 of rocks were used for the 3.5 km length of rock protection works comprising of rock bund and rip rap. Staged construction was employed to ensure stability wherever these works overlay soft clays. The observational method used for the construction of these works has been described by Choa, 1994.



TYPICAL RETAINING SYSTEM SECTION

Fig. 4 Sheet Pile Retaining Wall

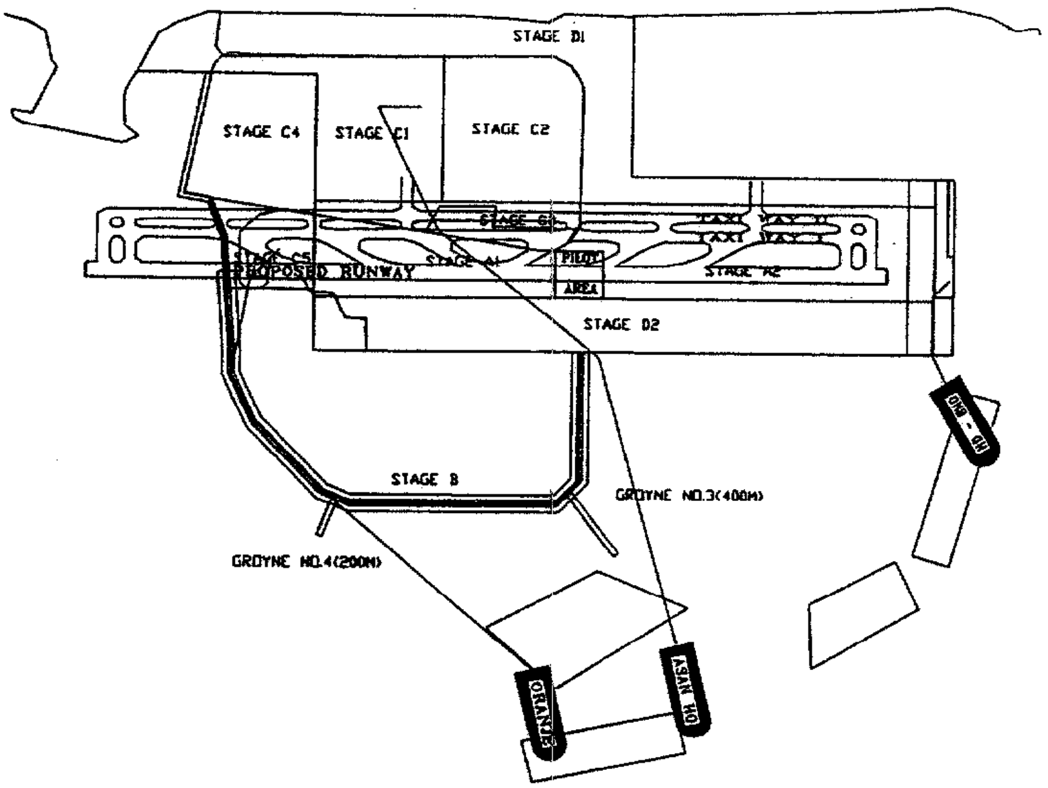


Fig. 5 Reclamation Phase 1B

The 4.5 km long retaining walls comprised about 7,500 nos of sheet and box piles of 600 mm width and 1,600 nos of 610 mm diameter pipes piles. The box piles were installed at 4.2 m spacing. The sheet piles were driven 3 m into the cemented sand layer with SPT "N' values of greater than 50 blows.

The sheet pile lengths ranged from 12 to 26 m averaging 18 m. The box pile lengths ranged from 15 to 38 m averaging 24 m and the pipe pile lengths ranged from 15 to 40 m averaging 27 m. Wherever marine clay was encountered at the sheet pile retaining sections the soft clay was dredged out to a

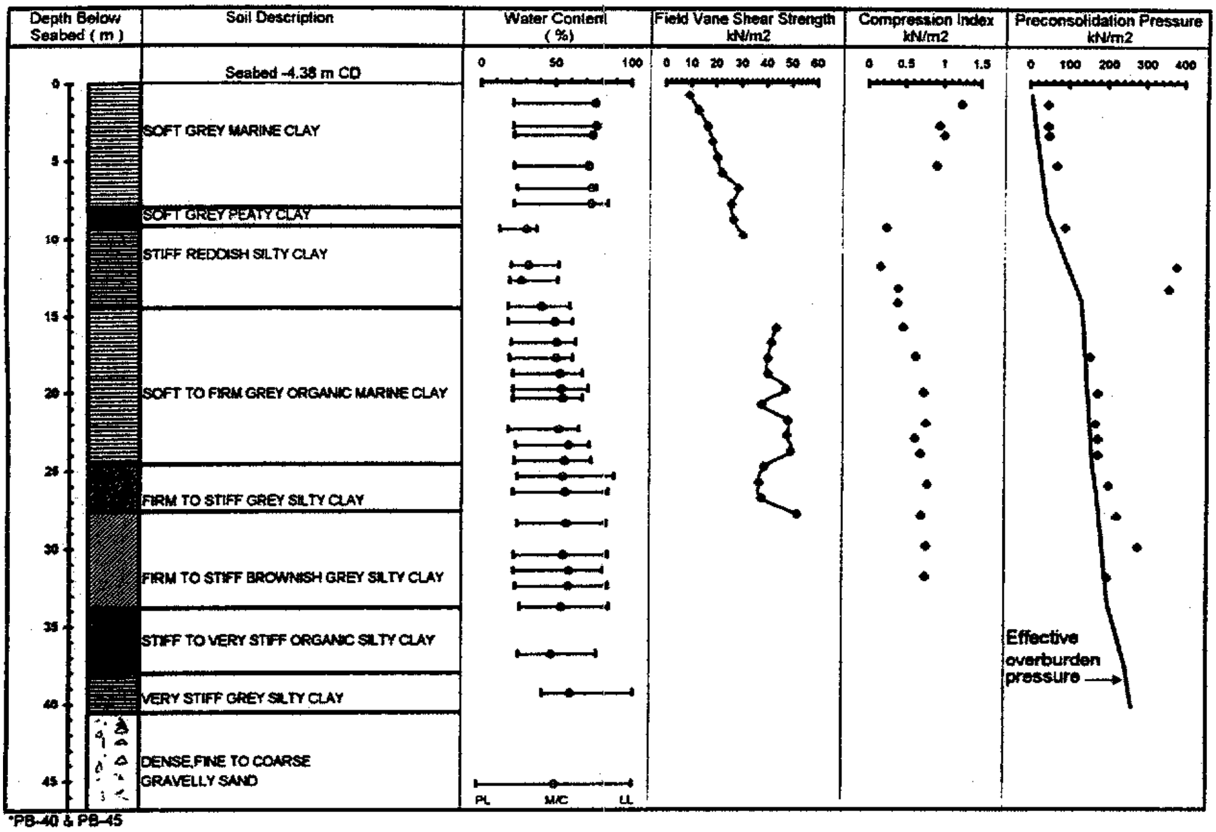


Fig. 6 Soil Profile and Properties

depth of 5 to 6 m and backfilled with sand to form a sand key. All the landside pipe piles were flushed out and ground anchors with 1,000 kN working load were installed below the toe of these piles. Inclinometers were installed in the sheet pile walls to monitor the movements during the backfilling of the wall.

3.2 Staged construction

Stability in this project is complicated by the existence of telecommunication submarine cables laid in shallow trenches underlying the reclamation (Fig. 2). A staged construction was required over these cables. A build-up of sand fill by 0.5 m layers with rest periods between lifts was specified for filling over a width of 50 m on either side of the cable. A trial reclamation section was carried out using inclinometers and settlement gauges to monitor the movements as each stage was built up with suitable rest periods. Work was allowed to commence over the cable itself only after the satisfactory completion of the trial reclamation in soil conditions similar to those that underlie the cables.

The reclamation over these cables has been successfully executed and the placement of the rock protection works are in progress.

3.3 Sand compaction

An area of about 105 ha designated as Stage B shall be compacted by dynamic compaction. (Fig. 2). The compaction is required to densify the sand fill to a depth of 6 m to a density giving a cone penetration resistance of 10 MPa. The initial sand fill has a cone penetration resistance of between 4 to 10 MPa. The weight of hammer, height of drop, print spacing etc. is to be determined in a compaction trial. Work is expected to commence in April 1995. The area within 30 m behind the sheet pile wall is to be compacted by vibratory probes or other methods which will not damage the retaining system. Trials will also be carried to determine the method of compaction. The degree and extent of densification is similar to the dynamic compaction areas.

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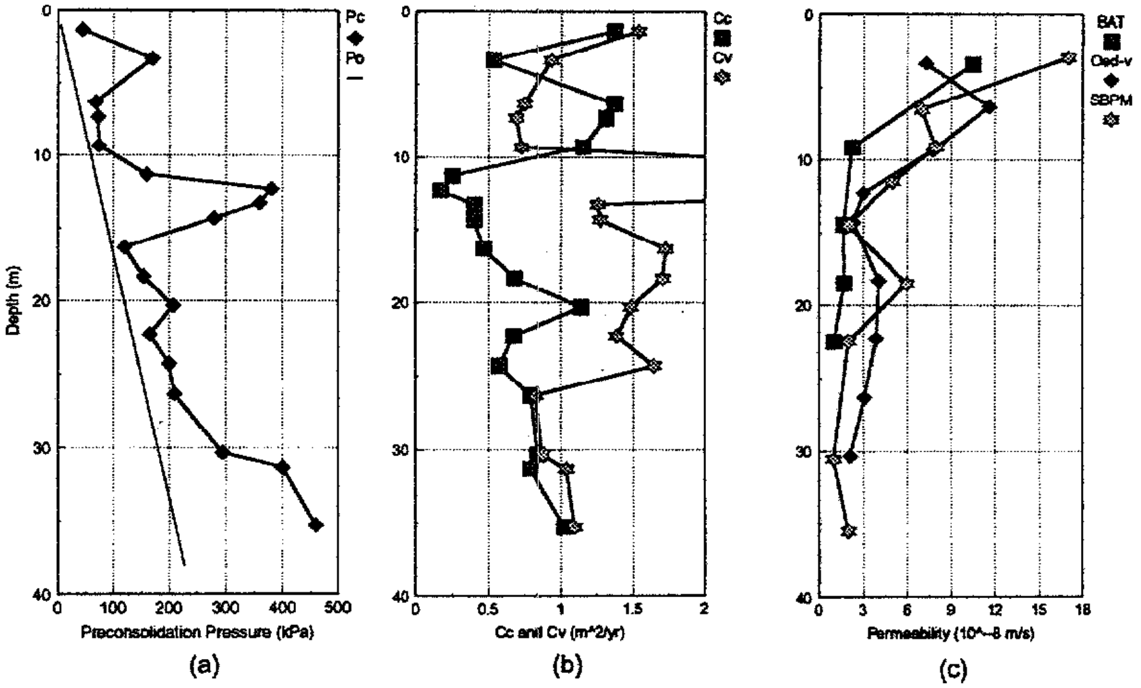

Fig. 7 Soil Properties at FT-4

4 RECLAMATION PHASE 1B

The Changi East Reclamation Phase 1B involves the reclamation of 490 ha of land with about 76 million m^3 of sand. (Fig. 5). The sand is transported from borrow sources about 30 to 45 km away by trailing suction hopper dredgers and self propelled hopper barges of 9,000 m^3 and 3,000 m^3 capacity respectively. The sand is deposited into rehandling pits adjacent to the reclamation site and repumped through 600 mm diameter pipe lines to the reclamation site by cutter suction dredgers with capacities of 8,000 HP to 20,000 HP. Sand pumps of 1,200 HP capacity as well as a suction hopper dredger of 3,000 m^3 capacity were also used to win the sand from the borrow source. The transportation of sand was also supplemented by bottom opening barges of 2,000 to 4,000 m^2 capacity. At peak a fleet of additional dredging equipment was deployed to boost the reclamation production rate.

The sea-bed is generally between -3 mCD to -5 mCD and the reclamation is initially brought to +4 mCD. At this platform level prefabricated band-shaped vertical drain are installed over the runway, taxiways and high speed turnoff areas. Surcharge is then placed hydraulically to +10.0 mCD and +8.5 mCD over the runway and taxiways/turnoffs respectively. After a surcharge period of 18 months the surcharge level is reduced to +5.0 mCD and the sand over the runway and taxiways/turnoffs is densified by dynamic compaction. The final level of the reclamation is to be between +5.0 and +5.5 mCD.

Additionally a 3.8 km long containment sand bund with a top width of 30 m is constructed to provide a spill over area for the capping of the "Silt Pond" with sand to a level of +4.0 mCD. The proposed method of filling is to spread the sand in thin layers not exceeding 10 cm per pass and for each lift to be between 0.5 to 1.0 m thick. About one to two weeks rest periods between each lift is to be provided for the underlying soil to gain sufficient strength to support the subsequent filling. Vertical drains are to be installed at +4 mCD and sand surcharge is to be placed initially to +7.5 mCD and finally to +9.0 mCD in order to improve the very soft silt and clay slurry.

4.1 Vertical drains and surcharge

The soft marine clays underlying the runway, taxiways and turnoff areas occur to depths of about 40 to 50 m below the seabed. Prefabricated band drains of about 100 mm width are to be installed at square grids of 1.5 m and 1.7 m under the runway and taxiway/turnoffs respectively. The height of

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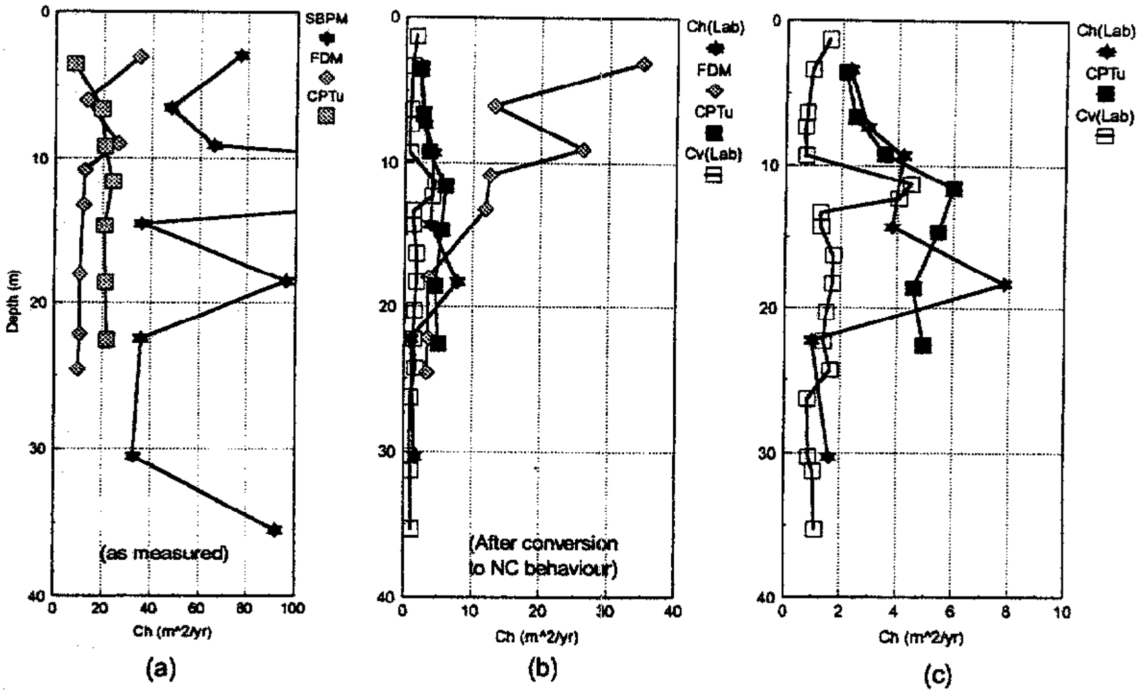


Fig. 8 Coefficient of Consolidation at FT-4

surcharge is 6 m and 4.5 m for the runway and taxiway/turnoffs respectively. A degree of consolidation of 90% is specified throughout the entire depth of the compressible layer overlying the cemented sand layer. A minimum surcharge period of 18 months has also been specified.

The very soft silts and clays in the "Silt Pond" are also to be improved with vertical drains at 2.0 m square grid. The surface of the very soft silts and clays are at -4 mCD and thickness is up to 20 m. A surcharge of up to +9.0 mCD will be placed after drain installation at +4.0 mCD. There is a possibility that the first round of drain installation may fail to produce the desired improvement due to the large strain envisaged as well as the very fine nature of some of the mine washings. In this event a contingency of a second round of supplemental drain installation at 2.0 m square is allowed for.

Colbond prefabricated band shaped drains are currently being installed by nine drain stitchers using static or vibratory force to advance the mandrel. The maximum depth of penetration of the mandrels range from 24 to 50 m. The current rate of installation is between 3,000 to 8,000 m per rig per 10 hour working day. The average rate is about ,000 m per rig per day. A total of 19 million metres of drains are to installed.

A strict quality control of the drains is in place where samples are taken daily for laboratory testing. The frequency of sampling is one per 20,000 m of drains installed. Tensile strength, filter permeability and discharge capacity of the drains are some of the tests being routinely carried out.

4.2 Sand compaction

The entire sand fill under the runway, taxiways, and turnoffs are to be densified. The seabed is approximately at -3 mCD to -5 mCD and the densification is to be carried out from +5 mCD. The proposed method of compaction is by the dynamic compaction method comprising of tamping the ground with drop weights. The compaction rigs are to be capable of imparting a minimum energy of 500 t-m (ton-metre) per blow (or such higher capacity rigs if the Contractor so desires). A total compactive effort (impact energy) of at least 300 t-m/m^2 and 400 t-m/m^2 is required in the areas requiring 12 MPa and 15 MPa cone penetration resistance respectively. The Contractor will be required to impart the above specified densities. If however, the above total compactive effort is insufficient to achieve the specified densities as determined by the cone penetration test and other tests then the Contractor is required at no

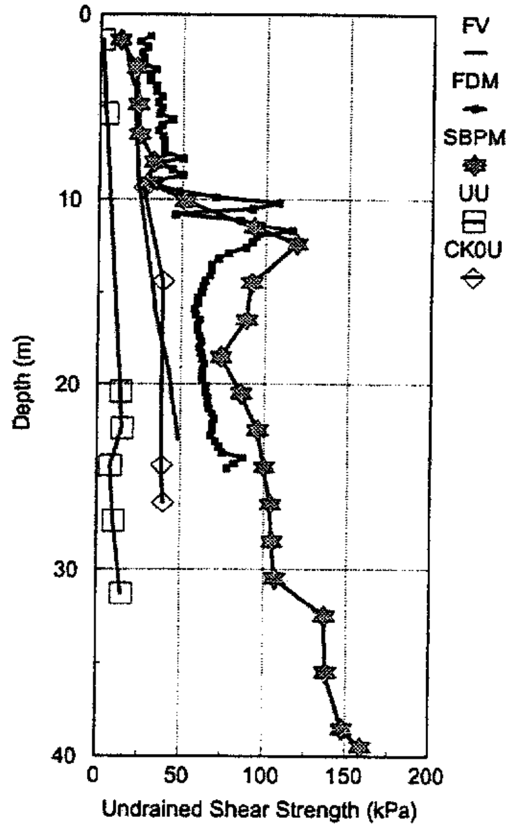


Fig. 9 Comparison of Undrained Shear Strength from In-situ and Laboratory Tests

additional cost to the Client to impart such additional compaction as is required to achieve the specified densities. The method of compaction such as pounder weight, height of drop, number of drops per location, spacing of pounding locations, number of compaction phases are to be determined in a compaction trial prior to the commencement of the main compaction works.

The acceptance criterion are to be determined by Cone Penetration Tests, Pressuremeter Tests, Standard Penetration Tests and surface settlements. The minimum cone penetration resistance after compaction is to be 12 MPa and 15 MPa under the taxiway/turnoffs and runway respectively. The equivalent acceptance criteria for pressuremeter tests is to be limit pressures of 1.5 and 2.0 MPa and pressuremeter moduli of 20 and 25 MPa. The equivalent Standard Penetration Tests is to be corrected 'N' values which indicate 65% and 73% Relative Density respectively. The settlement criteria is to be established during the compaction trial.

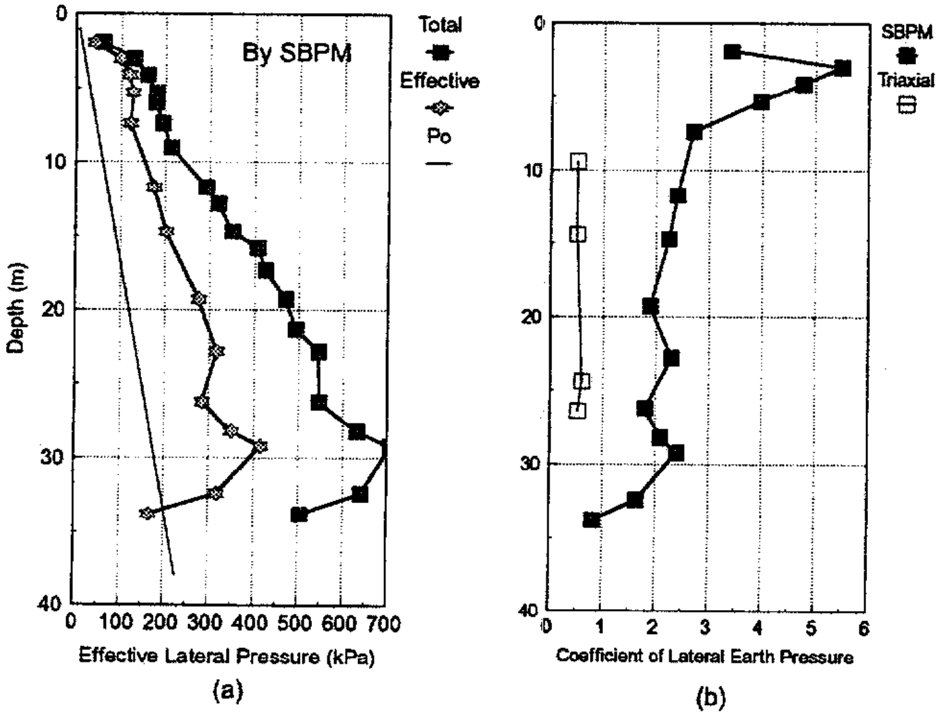
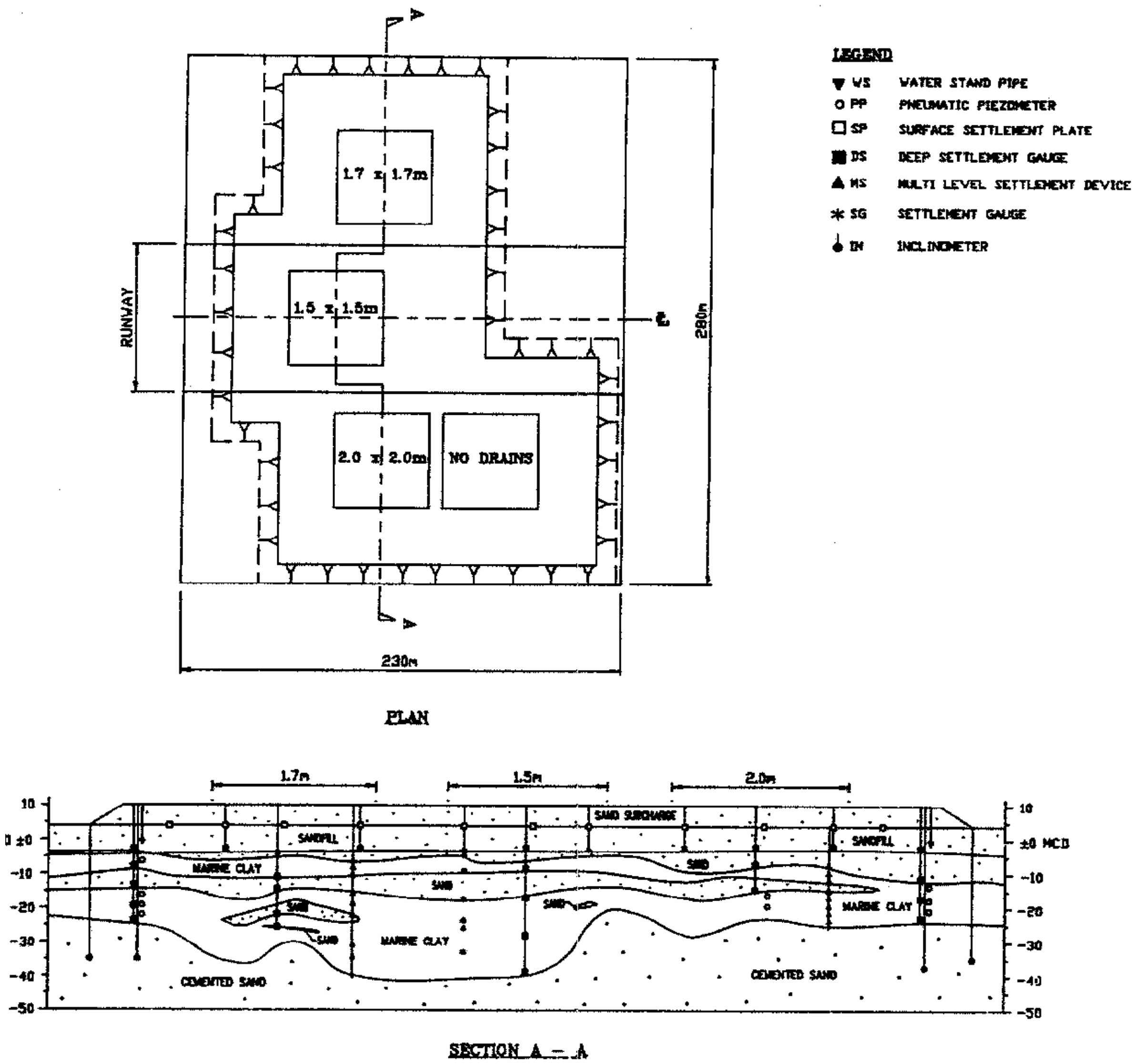


Fig. 10 Lateral Earth Pressure Measurements



ig. 11 Vertical Drains Pilot Test

PROPERTIES OF THE MARINE CLAY

ctensive in-situ and laboratory tests were nducted to investigate the consolidation and shear rength properties of marine clay. For illustrative irposes, only the soil properties measured for cation FT-4 are discussed in the following. The neral soil profile at FT-4 is shown in Fig. 6.

1 Consolidation behaviour

ie preconsolidation pressure p_c', the compression dex C_c and the vertical coefficient of nsolidation C_v were determined by conventional dometer tests (with 24 hour loading for each p). The distribution of these parameters with the pth below seabed are presented in Figs. 7(a) and 7(b). The permeability of soil can also be determined indirectly by oedometer tests, as shown in Fig. 7(c). The permeability of soil was also measured by BAT permeameter tests as well as indirectly determined by self-bored pressuremeter (SBPM) tests and the results are also presented in Fig. 7(c). It should be noted that the SBPM test measures the horizontal permeability. Nevertheless, the distributions of permeability with depth measured by the three different methods are quite close.

With the use of vertical drains, the horizontal coefficient of consolidation C_h becomes one of the most important consolidation parameters. In-situ tests, such as SBPM holding tests, cone penetrometer (CPTu) dissipation tests, as well as flat dilatometer (FDM) tests, were used to measure

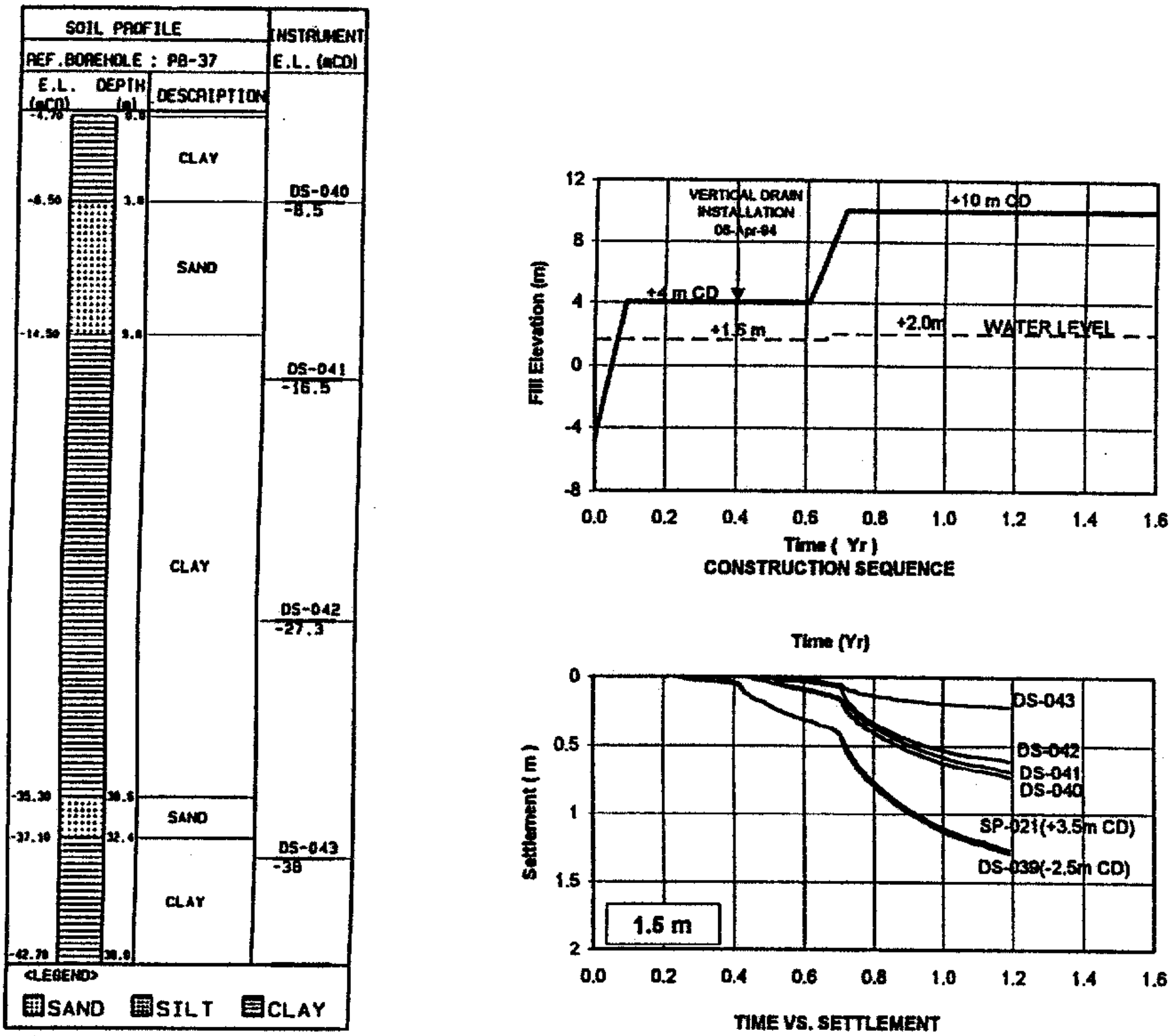


Fig. 12 Drains at 1.5 x 1.5 m square grid

the C_h values of the soil. The distribution curves of C_h versus depth measured by these three methods are presented in Fig. 8(a). It can be seen that although the measurements by FDM and CPTu tests are relatively close, the values measured by the SBPM tests can be many times larger. It should be pointed out that the C_h values measured by the above three methods are more relevant to the recompression index. Different correction methods have been suggested to convert the measured value to the C_h value for the normally consolidated region. Furthermore, the methods are not yet well established and there are various uncertainties involved in each of the tests.

Laboratory tests with a 75 mm diameter Rowe cell and a 76 mm diameter hydraulic consolidometer were also performed to measure the C_h values of soils. Generally the C_h values measured for the overconsolidation region was found to be much greater than the C_v values measured by the conventional oedometer tests. The C_h profiles measured by the laboratory and in-situ tests over the normally consolidated range are compared in Figs. 8(b) and 8(c). For the FDM and CPTu tests, the converted values are used for plotting Figs. 8(b) and 8(c). It can be seen that although the C_h values measured by the FDM tests for the upper marine clay are still much larger, the C_h measured by the CPTu tests show a good agreement with the laboratory measurements. The C_h values measured are generally a number of times larger than the C_v values.

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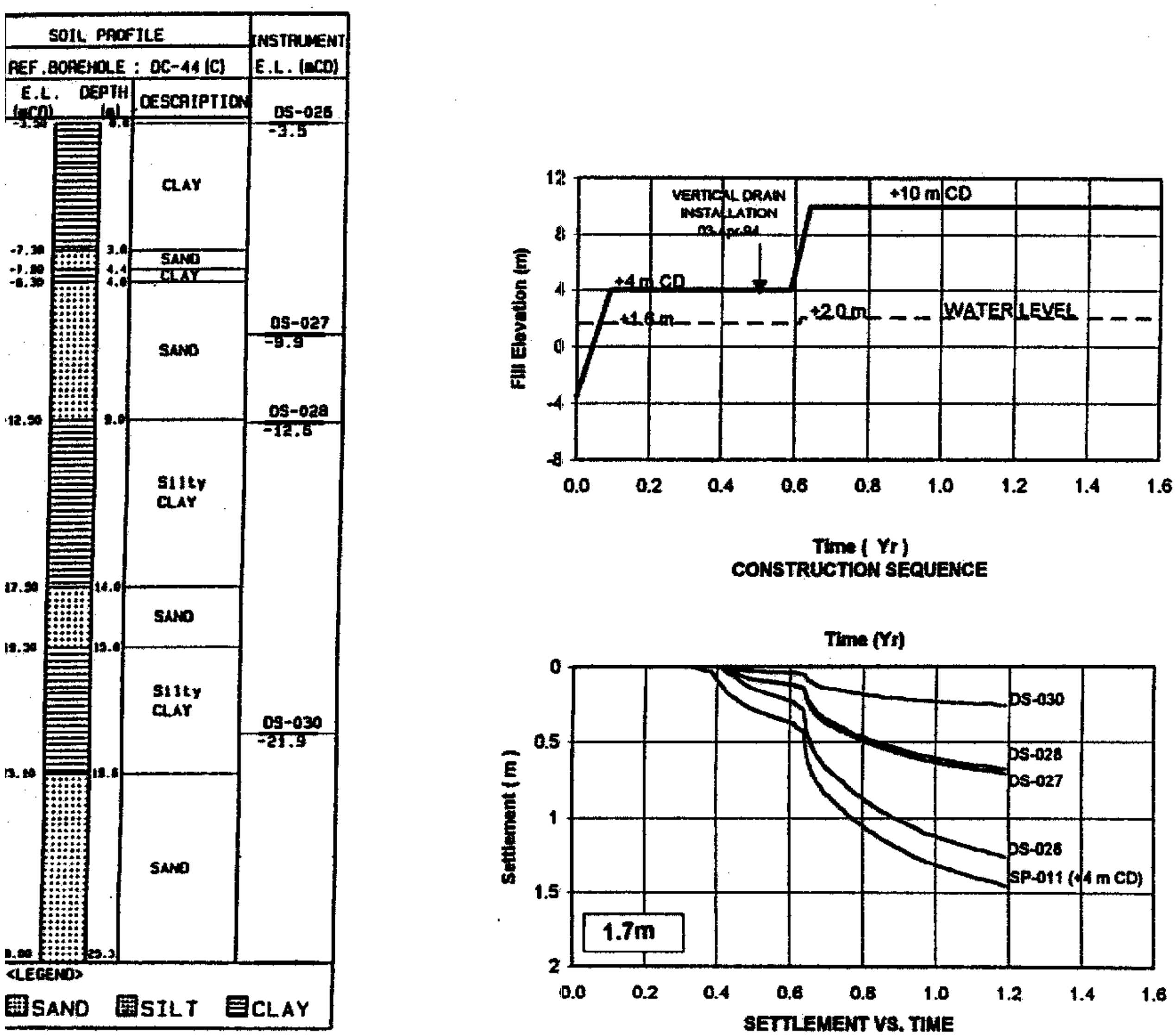

ig. 13 Drains at 1.7 x 1.7 m square grid

2 The undrained shear strength measurement

) measure the undrained shear strength of marine ay, different triaxial tests as well as direct simple ear tests were performed. The triaxial tests clude unconsolidated undrained (UU) tests,)tropic consolidated undrained compression :IUC) tests, K_0 consolidated and undrained mpression (CK_0UC) tests, and constant axial ess (σ'_v = const.) tests.

-situ tests, such as the field vane (FV) tests, the)M tests, and the SBPM tests were also used to easure the undrained shear strength of clay. A mparison of the different in-situ tests with the)oratory tests is presented in Fig. 9. It can be seen at the test data of CK_0U tests coincides with that of FV tests, but is much lower than that of SBPM and FDM tests. The difference may be due to the sample disturbances. However, considering the fact that a CK_0U test consolidates the specimen to more or less the same in-situ stress state before shearing, the sample disturbances should be abated to some extent. The difference in the stress condition and the failure mode may be one of the main reasons. A SBPM or a FDM test imposes a plane strain condition while a triaxial test an axisymmetric condition. Past studies with true triaxial and plane strain cells have established that the shear strength determined for a plane strain condition can be quite different from that for an axisymmetric condition. This factor therefore should be taken into account in the analysis.

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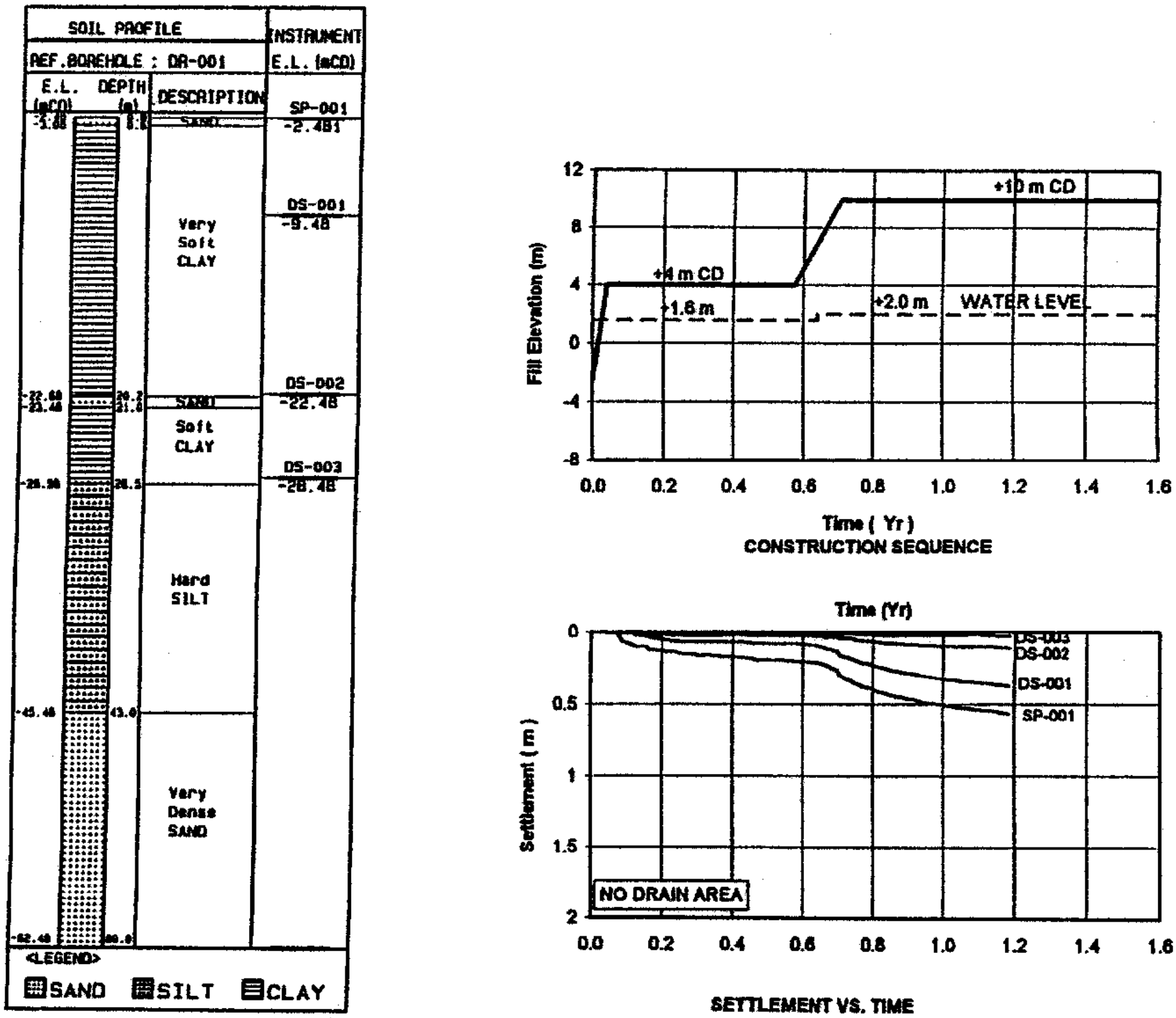

Fig. 14 No Drain Area

The consistently higher measurements by the SBPM tests than by the FDM tests for both the consolidation parameters (Fig. 8a) and undrained shear strength probably indicate that the stiffness of the soil may have been over-estimated. This speculation is supported by the lateral stress measurements presented in the next section. Some further laboratory testing with a new plane strain cell and with a torsional hollow cylinder machine have been planned to verify the measurements made by SBPM tests.

The UU tests, on the other hand, considerably underestimate the undrained shear strength of soil, as reported by many others.

5.3 In-situ lateral stress measurement

The use of SBPM has enabled the in-situ lateral stress to be measured. The total and effective in-situ lateral stresses measured by SBPM tests at FT-4 are plotted in Fig. 10(a) together with the in-situ vertical effective stress. Triaxial K_o consolidation tests with the control of zero lateral strain of the specimen were also used to measure the K_o values. The K_o values estimated by both methods are compared in Fig. 10(b). It can be seen that the K_o values measured by SBPM tests appear to be extremely high compared to the laboratory K_o values.

5 VERTICAL DRAINS PILOT TEST

\ pilot test is being carried out to study the drain at square grid spacings of 1.5 m, 1.7 m and 2.0 m. A control zone without drains was also designated. Fig. 11 shows the plan and section of the pilot test area which is situated just beyond the north east corner of the 'Silt Pond'. Soil instrumentation are also shown on Fig. 11. The results of the 1.5 m, .7 m and no drain areas are shown in Figs. 12, 13 nd 14. The preliminary back analyses of the pilot est results after only about one year since the ertical drain installation and placement of the urcharge indicate that between 70% to 80% of onsolidation has taken place in all three drain ones. The back analyses are based on settlement ieasurements and on the ultimate settlements redicted by Asaoka's method, the hyperbolic iethod and on conventional one dimensional onsolidation analyses using laboratory test results. ienerally the lowest degree of consolidation was btained based on the laboratory results. The egree of consolidation in the no drain area is stimated to be around 25%. As expected the pore ressure measurements were very difficult to iterpret.

CONCLUSION

oth the reclamation projects Phase 1A and Phase 3 are progressing extremely well. The sub-vision of the reclamation into the sub-phases of pproximately equal size and the timing of the clamation has therefore proven to be a reasonably od decision.

ie observational method used for controlling the iged construction of the reclamation and coastal otection works has been very successful with no lures encountered.

e pilot test for the prefabricated vertical drains licate that the choice of the 1.5 m and 1.7 m iare grid spacing for the treatment of the iway, taxiways and turnoffs is adequate.

e "Silt Pond" is currently being capped by eading sand in small lifts approximately 10 to cm per pass with suitable rest periods after eral passes. Preliminary indications is that the d loss is less than 30% including some lement. As at February 1995 a net thickness over 1.25 m has been placed over the very soft silts and clays. There is every indication that the "Silt Pond" can be successfully capped by the proposed sand spreading method.

ACKNOWLEDGMENT

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