architectural and structural design and construction all had to be completed in less than three years. Thus there was great need for flexibility, coordination and overlapping of tasks (Hamidi et al., 2010b).

The preliminary geotechnical investigation that was carried out rather sparingly indicated that the ground was very heterogeneous loose or soft soils with rapid variations of ground conditions within short distances of even 10 m. This investigation and further testing during the works indicated that more than 2,600,000 m^2 of the construction area was to be built on soil consisting of up to 9 m of loose silty sand or soft sandy silt. A schematic cross section of the site is shown in Figure 7).

As the investigation was not able to clearly define the soil profiles and consequently the ground improvement method to be applied, a dynamic reconnaissance phase was adopted by dropping a 20 ton pounder from 20 m and visual observation.

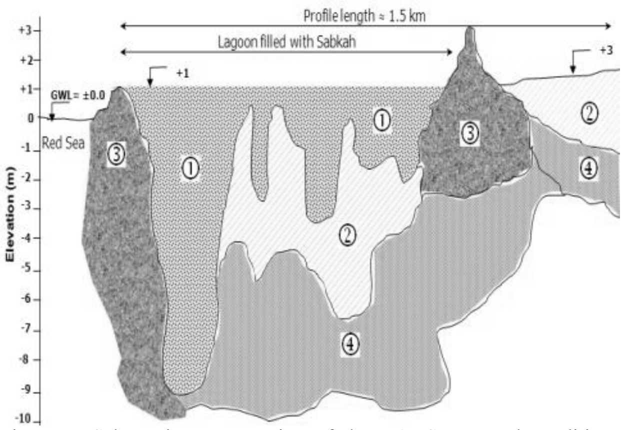

Figure 7. Schematic cross section of the KAUST ground conditions (Hamidi et al., 2010b)

Pressuremeter tests provided the parameters for the design of footings on sand or silt. As the buildings and thus footing locations were not defined at that phase, a 2 m thick sand platform was allowed on top of the silt to ensure the distribution of loads to dynamic replacement columns by arching.

The design and construct ground improvement proposal that met the project manager's technical requirements, schedule and budget was based on the below design criteria:

- Footing location: Any place within the treatment area
- Maximum footing load: 1,500 kN
- Allowable bearing capacity: 200 kPa
- Maximum total settlement: 25 mm
- Maximum differential settlement between two adjacent footings: 1/500
- Liquefaction mitigation for an earthquake with peak ground acceleration equal to 0.07g
- Level: 0.8 m below final ground level, but in any case at least 2 m above soft soil level

A pilot test was realised with pressuremeter testing and SPT (for grain size analysis) to define boundaries of application of the dynamic compaction and dynamic replacement techniques as a function of grain size, limit pressure and applied energy (see Figure 8). Furthermore, a spread sheet based on D60 rules (Menard, 1975) was prepared for the quick estimation of the bearing capacity and settlement by the site engineer.

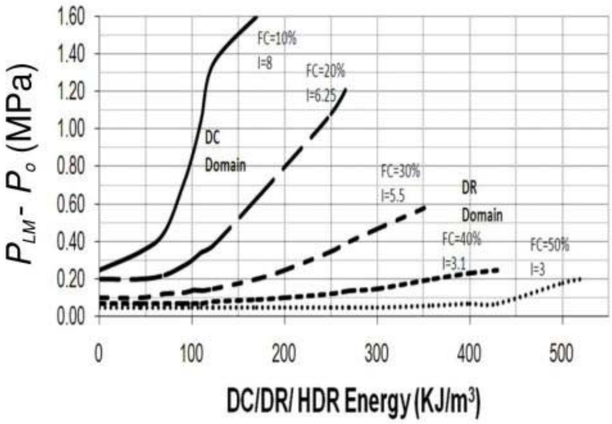

Figure 8. The relationship between net limit pressure, fines content and improvement energy (Hamidi et al., 2010b)

Of course, the boundary of application was a limit pressure of equal to or greater than 760 kPa for dynamic compaction zones and 180 kPa in between the granular columns of the dynamic compaction zones to provide sufficient lateral constraint to the columns.

Ground improvement on site was carried out by using a combination of dynamic compaction and dynamic replacement. Major changes to loads of some buildings later introduced the need to utilise dynamic surcharging as well. Dynamic compaction pounders used in this project weighed up to 21 tons.

Dynamic replacement was used in areas where the maximum depth of soft soil was 5 m. High energy dynamic replacement was used when the soft soil layer's depth was more than 5 m. In such a case, in addition to the engineered fill required for reaching final ground level, a 3 m surcharge was placed over the area for 3 weeks.

After completion of ground treatment in some areas, it became known that the revised master plan incorporated 20 six storey buildings. Hence, dynamic surcharging was also used to consolidate the deep soft soil layers. In this technique a combination of preloading and vibration is used to re-introduce pore pressure in the soil-water system and consequently to accelerate settlement rates. In addition to the engineered fill required for reaching final ground level a 3 m high surcharge was placed and dynamic compaction was performed on it.

Differences in ground behaviour due to pounder impact enabled the site supervisors to assess the rapidly varying ground conditions and to apply the appropriate ground improvement technique as needed. It was observed that while the first dynamic compaction pounder impact penetrated the ground by about 0.25 m, the dynamic replacement pounder penetration was substantially more and in the range of about 1 m. Also, performing dynamic compaction frequently resulted in the seepage of groundwater to the surface, but this phenomenon was rarely encountered in dynamic replacement areas. Ground rest periods in between dynamic compaction phases were 1 to 3 days, but considerably longer and from 7 to 21 days when dynamic replacement had to be performed. Also, ground heave due to pounding was not observed in dynamic compaction areas but was observable in dynamic replacement zones.

A total of 800 pressuremeter tests were performed to insure the quality.

5 NEW VERSATILE TECHNIQUES FOR DIFFICULT SOILS

The development of a new container terminal in Southeast Asia was the opportunity to make a compromise between pressuremeter and Mohr Coulomb approaches.

According the original design the soft marine clay at the seabed was to be dredged down to the depth of 30 m below sea level where the shear strength of the stiff clay exceeded 250 kPa. The excavated key was to be then backfilled with sand and compacted using vibro compaction under 3 m of additional overburden sand fill. Next, the surcharge had to be removed, a rubble mound was to be placed over the sand key, and as shown in Figure 9, finally caissons were to be sunk onto the mound.

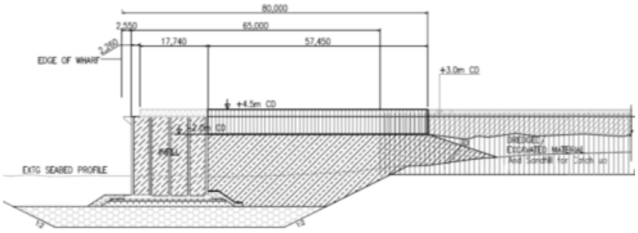

Figure 9. Cross section of container terminal based on original foundation concept

While the clay at dredge level was initially very stiff, dredging works and cutting into the clay softened the upper 1 to 1.5 m of the exposed clay surface and post dredging CPT tests performed before the removal of the overburden sand fill indicated that the clay's shear strength had dropped to about one third of its original value; i.e. to approximately 80 kPa (Hamidi et al., 2010c). Further testing at later stages by the pressuremeter test suggested that the shear strength had even further reduced at some points to as low as 16 kPa.

Dynamic replacement was used as an alternative method to treat the softened clay layer. In the proposed dynamic replacement methodology it was assumed that a 1.8 m thick granite rock fill layer would be placed over the soft clay layer. The blanket material was chosen in such a way that 30% of the stone diameters were from 150 to 200 mm and the remaining 70% were from 200 to 300 m. The rock columns were designed to be 2 m in diameter, in a 4.5 m grid and with a replacement ratio of 15%.

A pounder weighing 38.5 tons was specifically designed and fabricated for the project. This pounder was grater shaped to allow the passage of water through the pounder with the least resistance. It was also with dual side functionality; i.e. it was 1.7 m by 1.7 m on one side and used for driving rock dynamically into the clay and 2.3 m by 2.3 m on the side to dynamically compact the rock blanket. Figure 10 shows this marine pounder.

The self-bored slotted tube or *STAF* technique (Arsonnet et al., 2005) was utilised from a jack up barge, to perform the pressuremeter tests down to a depth of more than 30 m. The technique consists of sealing a casing to the sea floor and driving a BX size slotted casing with advanced drilling and by utilising an eccentric bit. The slotted casing is advanced to the required depth, the bit is removed and the pressuremeter probe is inserted to depth. After the test, the slotted casing is jacked up one meter and the next test is performed. Figure 11 shows the Staf drag bits that can either have blades or buttons.

Since the stability analysis was performed using the classical the Mohr Coulomb failure criteria, the friction angle and cohesion were necessary for the stability analysis.

Shear strength, *c*, can be estimated from the pressuremeter test by (Menard, 1965):

Figure 10. Specially designed multi-purpose marine pounder (Chu et al., 2009)

—— (5)

$P_{LM}{}^{*}$= net limit pressure and can be calculated from

$$P_{LM}{}^{*} = P_{LM} - P_o \quad (6)$$

P_o= at rest horizontal earth pressure at the test level at the time of the test.

The internal friction angle, φ, for sands can be estimated in sands from the pressuremeter test by (Menard, 1970):

—— (7)

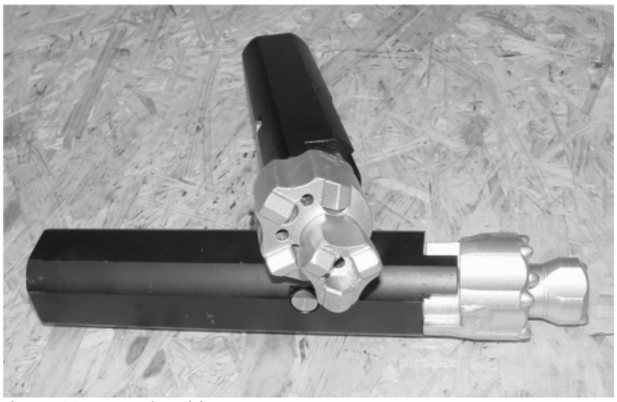

Figure 11. *STAF* drag bit

Equation 7 is not applicable to rock; hence a method was devised by Yee and Varaksin to develop an equation for rock. For this purpose, a test pit was dug out and backfilled with rock in a loose state. The internal friction angle was determined with failure loading and the limit pressure was measured. A point was set in the diagram of Figure 12, and from this point a curve was drawn parallel to Menard's limit pressure-friction angle curve (Equation 7) to develop the proposed formula of Equation 8.

—— (8)

Pressuremeter tests were carried out at 29 different locations that also included cyclic tests. As reported by Yee and Varaksin (2012) the ratios of reload to Menard modulus was in the range of 3.5 to 4.2 which agrees with the suggested value of 4 for compacted gravel and rock fill (Menard, 1975). Based on Equation 8, the internal friction angle of the rock after compaction was interpreted to be from 47 to 49°, with an average value of 48.5°, which satisfied the design requirement of 45°.

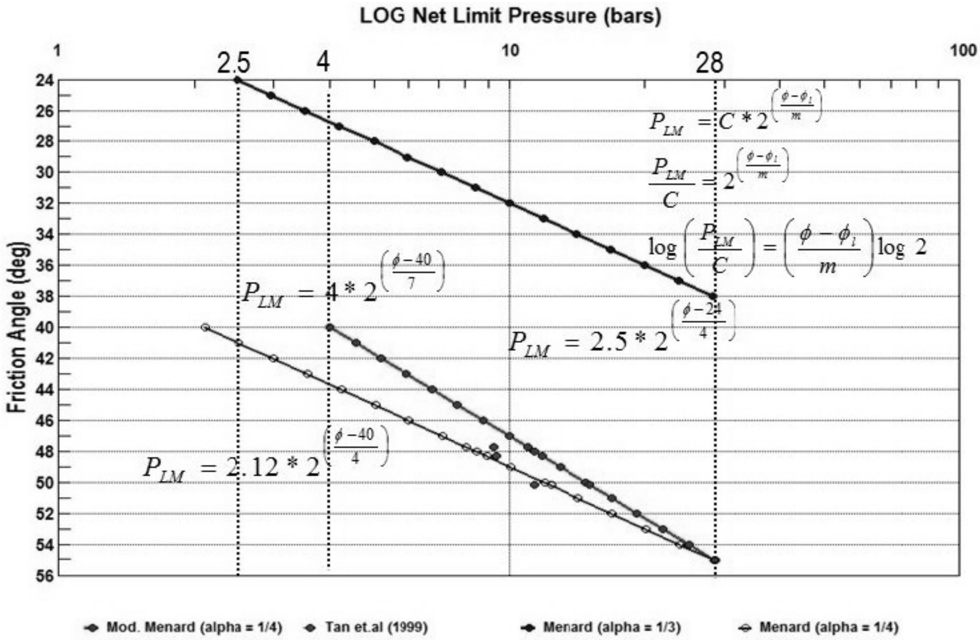

Figure 12. Developing a method for estimating rock friction angle from the limit pressure

6 CONCLUSION

The pressuremeter has not only been a tool for the design and quality control of ground improvement works, which is mostly adapted to non- cohesive soils and the only method for fills.

In the authors' opinion, the pressuremeter is the most versatile field test and proven method of analysis that can satisfy not only the geotechnical engineers' requirements, but also that of the constructors.

Specifying ground improvement acceptance criteria based on design criteria; i.e. bearing capacity, settlement, etc. is a much more realistic and smarter approach than stipulating testing values. In addition specifying calculation methods such as what has been proposed by Menard (1975) makes interpretation of data very clear, without leaving technical and contractual loose ends in a project.

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