Lessons learnt from geotechnical trials at Port of Brisbane

Les enseignements tirés des essais géotechniques au Port de Brisbane

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ABSTRACT: Geotechnical risk is well accepted in the industry and various methods are employed to reduce it in any project. Geotechnical trials are useful to reduce the geotechnical risk if time permits. They provide valuable information to the designer to reduce risks and, at the same time, produce a more efficient design. Geotechnical trials were used effectively in the geotechnical design at the Port of Brisbane. The project comprised two components, viz., the construction of a 4.6km long seawall in Moreton Bay, and once it was completed, the reclamation and ground improvement in the area contained. A small scale trial on geotextiles and a large scale trial on ground improvement are described.

RÉSUMÉ : Les risques géotechniques sont bien acceptés dans l'industrie et diverses méthodes sont employées pour réduire ceux-ci. Si le temps le permet, la réalisation d'essais géotechniques est utile pour réduire le risque géotechnique. Ils fournissent des informations précieuses au bureau d'étude afin de réduire les risques et en même temps améliorer la conception de l'ouvrage. Des essais géotechniques ont été utilisés efficacement dans la conception géotechnique au port de Brisbane. Le projet comprenait deux composantes, d'une part la construction d'une digue longue de 4,6 km dans la baie de Moreton et d'autre part lorsqu'elle fut construite, le gain de terres sur la mer ainsi que l'amélioration des sols dans le polder ainsi créé. On décrit un essai à petite échelle sur des géotextiles et un essai à grande échelle sur les améliorations des sols.

KEYWORDS: Construction, Geotextile, soft clay, geotechnical trial.

1 INTRODUCTION

Geotechnical risk is a term accepted by the construction industry because of its importance in civil design and construction projects. Although the term "geotechnical risk" was not a frequently used term until recent times, risks posed by ground conditions were well known to engineers over centuries which led to the adoption of a factor of safety for geotechnical design calculations. However, apart from applying a Factor of Safety to a mobilised strength or an applied load, there was little recognition for the necessity to identify risks associated with the Factor of Safety, in particular, subsurface conditions and/or how these risks could be managed. Due recognition for the phenomenon of geotechnical risk was perhaps given after major failures during construction around the world and the reluctance of insurers to insure major projects where uncertainties and risks were high.

Although it is difficult to eliminate geotechnical risk on a project, designers and contractors strive to reduce it by adoptimg appropriate measures. Detailed site investigations, for example, reduce the risk of performance by allowing to understand the variability of the soil profile in an area or along an alignment. Installation of geotechnical instruments and frequent monitoring during construction is another method of reducing the risk of failure or underperformance. While not as popular as above, geotechnical trials could also be used to achieve the same objectives. Obviously, the cost of trials and availble time are key issues to be dealt with, in addition to the value the trials bring to the project. Sometimes the trial could be part of the final design which reduces additional costs for the trial.

It is not feasible to test and sample every inch of the ground and therefore geotechnical engineers always have to carry out their designs based on limited test data. A further issue is the behaviour of soil elements in a field test or a laboratory test may differ significantly from the actual behaviour expected under the design loading conditions. This becomes more complex in projects associated with earth fill and rock fill. While some tests can be modified to simulate some form of behaviour, generally it is impossible in situations of filling where fill materials can be large in size and therefore not feasible to scale model in laboratory tests. In such situations construction trials can provide valuable information to the designer to reduce risks and, at the same time, produce a more efficient design.

At the Port of Brisbane (PoB) in Australia, significant reclamation is carried out into the Bay and geotechnical trials were adopted to assist the design and construction. The trials varied from simple field tests to large scale trials. Lessons learnt from some of the trials are discussed in this paper.

2 GEOLOGICAL SETTING

The geomorphology and geological setting at PoB are described in detailed in Ameratunga et al. (2010). A brief description of the geological units is summarised in Table 1.

Table 1. Geological Units

Geological Units	Description
Recent	Modern dune and beach deposits and dredged fill comprising silt and clay interbedded with layers of fine to coarse grained sand.

Holocene	Normally to slightly over-consolidated marine clay (known as PoB clay), silt and sand.
Pleistocene	Older sediments comprising over-consolidated clay, sand and gravel. This layer is assessed to be incompressible for design loads at PoB.
Tertiary	Weathered basalt bedrock (Petrie Formation)

3 PORT OF BRISBANE DEVELOPMENT

The Port of Brisbane is the main port for Queensland and is located at the mouth of the Brisbane River at Fisherman Islands (see Figure 1). The Port and the surrounding areas have been progressively developed to cater for the increasing trade activities in Brisbane and Queensland. This trend is expected to continue for the next 15 years and beyond, and the Future Port Expansion (FPE) Project is expected to ensure that Port capacity is sufficient to cater for the increasing demand.

Figure 1. Site locality plan

The Project involves the reclamation of an additional 230ha into the Moreton Bay for the future expansion of the Port. Currently 40% of the area to be developed is completed.

The construction of a seawall 4.6km in length over variably thick Holocene clays was the first challenge for the FPE before reclamation. The wall was constructed using rock and sand fill and completed in 2005. Once the wall was built, internal bunds were constructed to create paddocks to act as receptacles for the dredged mud. This was followed by reclamation which has been carried out using maintenance dredging materials from the navigation channels. This is followed by a sand capping to provide a platform for the earthworks machinery, especially ground improvement equipment (Ameratunga et al, 2016).

4 GEOTECHNICAL TRIALS

As mentioned above, there were two main components to the project where geotechnical analysis and advice were critical:

i. Design and construction of a seawall in the Bay
ii. Ground improvement of dredged mud up to 9m thick overlying soft to firm clay up to 40m depth.

Each of the above posed unique challenges because such construction had not taken place locally or within Australia.

4.1 *Seawall project*

The seawall project is the design and construction of the 4.6km bund to contain future reclamation. The project posed several challenges:

- water depth changing from 1m to 6m which means both land based operations (where the draft is too small for a barge) as well as marine operations required;
- seabed and shallow soils very weak, as soft as 3kPa;
- deep, soft to firm clays, as deep as 35m;
- settlement under construction significantly high, more than 1m during construction period; and
- abuts the Marine Park, a Ramsar site, and therefore cannot be disturbed.

Figure 2. FPE Seawall – soon after completion in 2004

To overcome the above issues, it was decided not to improve the ground using wick drains and/or inclusions and allow the seawall to settle with topping up acceptable to the client.

Essentially two designs were adopted for the seawall:

- North/South Bunds - Seabed shallow (within 1m below low water table); seabed clay strength 8 to 15kPa) - Rock embankment placed on a high strength geotextile laid on the seabed (see Figure 3).
- East Bund - Where the seabed was deep and the seabed strength varying from 3 to 5kPa and the clay thickness varying from 15 to 30m - Wide sand 'pancake' on a high strength geotextile (see Figure 4).

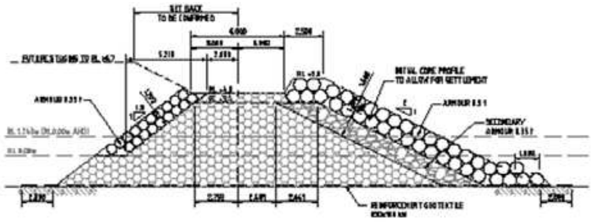

Figure 3. North/South bund cross section

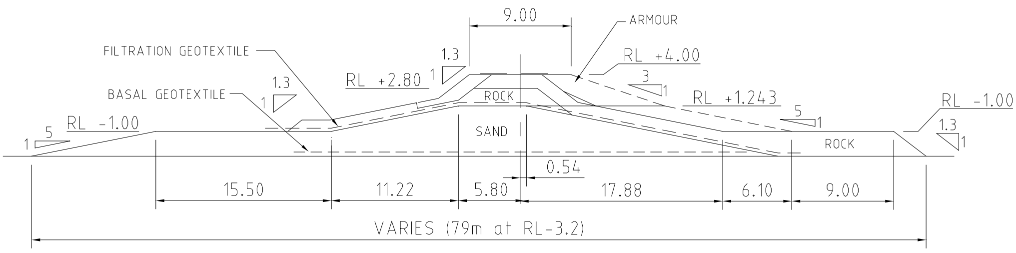

Figure 4. East bund cross section

4.1.1 *Geotextile trials*

Where the rock core is directly placed on the high strength geotextile (Figure 3), the possible damage to the geotextiles during rock placement and trafficking was identified as a significant hazard. Such damage is generally treated in the design by down-rating the basal geotextile strength. Damage factors used for sands and clays are generally recommended by manufacturers, probably based on experience, laboratory and field testing. However there was no published data available on damage due to rock placement nor due to trafficking and effects on the design.

On the eastern deeper alignment (Figure 4), the sand pancake was placed on the high strength geotextile using marine methods, therefore the risk of damage was not an issue. However, of greatest concern was the potential for damage of the filtration fabric (which covers the sand pancake and on which rock is placed by land based methods), because of the potential consequences of sand loss through tears in the fabric resulting from any rock punctures and subsequent tidal action could potentially compromise the integrity of the Seawall.

To overcome the uncertainties in design and construction related to geofabrics, damage trials were conducted on selected geotextiles, using typical rock core/armour to be used, to assess whether significant damage would occur during the placement of the rock and construction trafficking above and what allowance should be made in the design for these effects.

Basal High Strength Geotextiles

The geotextiles trialled were woven type with ultimate tensile strength 200kN/m and 800kN/m. The trials were conducted in one of the reclamation paddocks filled with dredged mud capped off with a 2m thick sand base. Dynamic Cone Penetrometer testing conducted to assess the strength variation of the base generally indicated medium dense conditions.

The geotextile was supplied 4m wide, which was stitched together to form a panel of about 12m x 12m. The panel was divided into 4 equal cells. The trials were conducted using maximum 300mm rock core with typically 1.5m drop height but varying the number of drops. A worst case scenario of a 3m drop was also assessed.

On completion of each trial, bulk of the rock core was removed by excavator bucket with the rock immediately above the geotextile removed carefully by hand to assess, measure and photograph the damage prior to quantifying the damage. To assess the effect of construction vehicle movement, rock core was placed over the geofabric to form an access track wide enough for a 45T excavator to travel. The length of the access track was about 5m and the height was 1.0m. This track was then subjected to 16 passes of the excavator moving parallel to the weft direction. The number of passes used was excessive compared to actual conditions during construction.

For the basal geotextile, the damage factor was calculated as an inverse ratio of the width of damaged section over the total width of the panel or cell. Random parallel lines were drawn and the assessment for each line was assessed.

The results indicated that:

- Damage factor varied between 1.2 and 1.8.
- 800kN/m geotextile showed better resistance than the 200kN/m. Latter was significantly damaged by the tracking trial.
- Tracking damage was more significant than damage created by rock core drops.

Based on the test results it was decided not to use the 200kN/m geotextile but use 400kN/m or higher. Further, a constant damage factor of 1.7 was used for all grades of geotextile between 400kN/m and 850kN/m used on the project.

Filtration Geotextiles

Main concern was the effect of rock placement and trafficking on the filtration geotextile covering the cohesionless white sand. Accordingly, damage trials carried out on the filtration geotextile were more extensive. The geotextile trialled was a 1200g/m^2 nonwoven staple fibre material.

As the rock was to be placed by pushing using land based techniques and not dropped, only trafficking trials were conducted. The geotextile was firmly anchored to an area of moist, loose to medium dense white sand and 0.3T armour rock placed (by excavator) over the geotextile to varying heights. The rock surface was divided into 4 sections, each approximately 4m square, so that several trials could be conducted.

A series of trials were conducted by changing the material cover between 0.3m and 1.3m, using different materials (sand, rock core and armour), and the number of passes by a 30T excavator (6 to 12 passes). In general no punctures were detected except when 12 passes over 350mm armour rock was trafficked where one (75mm) tear and six 20-30mm tears were found. There were numerous indentations which were not recorded as damage. The presence of indentations indicated the geotextile could withstand significant high strain without rupture.

Additional trials (three) using a 45T excavator indicated the damage to be higher than for a 30T excavator discussed above.

The trials concluded that the damage on filtration geotextiles due to construction trafficking is minimal if 1200g/m^2 geotextile was used and the excavator weight was limited to 30T. More details are presented in Ameratunga et al 2006.

4.2 *Reclamation and ground improvement within FPE*

The next stage of the development at the PoB was the reclamation of the land bounded by the seawall using dredged materials from maintenance dredging from the navigation channels. The geotechnical conditions become worse because of the high compressibility and low shear strength of the dredged mud up to 10m thick. This means the depth to the base of compressible clay varied from about 20m to in excess of 40m. The ground improvement posed several issues and concerns to the client and a preliminary study concluded that wick drains and surcharging would be the most economical method although equipment had to be modified or brought from overseas to cater for the deep soft soils. There was also the issue of the boundary with the Ramsar Marine Park and surcharging at the boundary was not an option because of the sensitivity previously discussed.

However, PoB was concerned about wick drain underperformance. There had been several documented cases of underperformance of wick drains in South East Queensland, although the reason for the poor performance was not evident.

There could be several reasons and subsoil characteristics most likely would be one of them. In addition, there were many unknowns, especially relating to the installation technique and the potential smearing of the wicks.

As it ultimately needs to treat a reclaimed area in excess of 230ha, PoB needed some certainty in relation to the performance and timelines for consolidation and therefore decided to conduct large scale wick drain and vacuum trials to assess the effectiveness and performance of the various techniques before embarking on a full scale treatment of the reclamation site. This would enable optimised designs to be used in future treatment of the large areas of the reclamation site. This is probably the largest ground improvement trial ever conducted in Australia.

Apart from gathering significant knowledge on the settlement characteristics of dredged mud and soft clay, the aim at the commencement of the trial was to ascertain:

- the effectiveness of PVDs in varying site conditions;
- performance of PVDs at different spacings;
- PVD performance compared to costs;
- consolidation times and surcharge loadings using both PVDs only and under vacuum consolidation; and
- trialist's performance with respect to design and construction.

The trials which commenced late in 2006 came to an end in December 2008, providing valuable learnings into wick and vacuum treatment of reclaimed land. Boyle et al (2007) describe the details of the steps taken to set up the process and the procedures adopted to capture the performance and their assessment. The total trial area was 11.6ha but the client selected the area within the development so that the expense is covered by the land development costs. As there was a likelihood of lower than expected performance which could delay the use of the land, the client selected locations which are not critical for development. A brief summary of the results and lessons learnt is discussed in this paper and more details are found in Boyle et al 2009.

As part of the trials, two areas were subjected to vacuum trials, which were conducted simultaneously with the rest of the wick drain trials. The results of the vacuum consolidation trial are discussed by Berthier et al (2009).

Various authors involved in the project have mentioned the learnings of the trials and given below are a summary of learnings to the industry at the time of completion of the trials. Some of the differences in each site are presented in Table 2. The learnings could be summarised as follows:

- Wick drains could be used effectively in dredged mud as thick as 10m.
- Wick drains could be used effectively in PoB clays, as deep as 35m.
- Wick drains trialled were suitable for the mud and soft clays present at the PoB. There was no indication of blocking, kinking or breaking due to high settlements, which were of the order of 1.5m to 3m. (Prior to the field trials, large scale laboratory tests were conducted on the proposed wick drain types at the University of Wollongong to confirm no adverse behaviour).
- Criterion of 150mm over 20 years is too stringent for the thick clay layers.
- The MD88 series appeared to perform marginally better in terms of the degree of consolidation achieved compared to the MD7007 and MCD34 drains.
- For wick drains with pore sizes from 75µm to 150µm showed no difference in performance.
- Wick drain spacing adopted, ranging from 1m to 1.4m, has performed satisfactorily although greater smear effect evident for 1m spacing. At spacing less than 1m, the drain effectiveness is likely to further decrease due to the possible overlapping of smear zones.
- Vacuum surcharge preloading is a useful method for accelerating radial consolidation and controlling lateral displacement of the soft clay.

Table 2 - Information relevant to each trial site

	Wick Drain areas 1 & 2	Wick Drain Area 3	Vacuum Area
Dredged mud thickness	0.5m – 5m	0.5m – 6m	2.5m – 7.6m
Lower Holocene clay thickness	4m – 20m	5m – 23m	8.8m – 24.4m
Design Load	50kPa – 60kPa	15kPa	15kPa
Design residual settlement in 20 years	250mm	250mm	250mm
Wick Type	MD7007, MD88H MD88HD	MCD34, MD88, FD767	MCD34
Wick grid	Triangular	Square	Square
Wick spacing	1, 1.25 & 1.4m	1.1, 1.2 & 1.3m	1.2m
Filter Pore size	75µm - 150µm	80µm	80µm
Preload Time	6 - 12 months	12 months	9 months

5 CONCLUSIONS

The paper presents two geotechnical trials conducted at the PoB. The first trial on geotextiles, high strength and filtration, provided valuable information to the design and construction and allowed the safe completion of the project. The major trial on wick drains provided, first of all, confidence that wick drains could be effectively used in the PoB environment where the area to be improved would be underlain by up to 10m of dredged mud over deep soft to firm clay. Further lessons were learnt on wick drain performance, spacing and settlement criteria to be adopted for the development. Vacuum consolidation was found to provide an efficient way of improving the ground.

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