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Seawall construction in Moreton Bay, Brisbane

Construction d'une digue marine dans Moreton Bay, Brisbane

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ABSTRACT
The project involves the design and construction of a 4.6km long seawall east of the existing reclaimed areas at the Port of Brisbane. The construction of the seawall is the first step in allowing the Port of Brisbane Corporation (PBC) to reclaim an additional 230ha for the future expansion of the Port. The design and construction was carried out as an Alliance between the client (PBC) and a team consisting of Coffey Geosciences (Geotechnical Consultant), Leighton Contractors (Contractor), Parsons Brinckerhoff Australia (Civil Consultant) and WBM Oceanics (Hydraulic Consultant).
Significant geotechnical, environmental and construction constraints were associated with the project because of the subsurface profiles, close proximity of the Moreton Bay Marine Park, varying water depths and expected sea conditions during construction. Soft clays are encountered along the full length of the alignment with the thickness varying from 6m near the existing reclamation increasing to 30m towards the east. The consistency of the soft clay at the seabed surface is very soft to soft, with undrained shear strengths as low as 5 kPa. This paper concentrates on the design and construction of the eastern portion of the seawall.

RÉSUMÉ
Le projet comporte la conception et la construction d'une digue marine, 4,6km de longuer à l'Est des zones déja reprises sur la mer faisant partie du port de Brisbane. La construction de cette digue marine est la première étape du projet, permettant à la société du port de Brisbane (Port of Brisbane Corporation - PBC) de récupérer environ 230ha pour la future expansion du port. Construite dans Moreton Bay, la digue relie la zone récupérée sur la mer à la terre ferme, avec une forme générale de fer à cheval. La conception et la construction de cette digue ont été effectuées par une alliance entre PBC (client) et une équipe se composant de Coffey Geosciences (conseil géotechnique), Leighton (entrepreneur), Parsons Brinckerhoff Australie (conseil pour les travaux civils) et WBM Oceanics (conseil hydraulique).
Le projet a comporté un certain nombre de contraintes d'ordre géotechnique, environmentales et de construction, principalement en raison des profils de sub-surface, de la proximité immédiate du parc d'attractions marin de Moreton Bay, et des conditions de marée et de houle pendant la construction. Il existe une couche d'argile molle sur la totalité de l'alignement, d'épaisseur croissante de 6m à proximité des zones récupérées sur la mer jusqu'à 30m vers l'Est. La malléabilité de l'argile sur laquelle repose la digue varie de molle à très molle, avec une résistance au cisaillement sans drainage pouvant descendre jusqu' à 5kPa. Cet article se concentre sur la conception et la construction de la partie orientale du bund de la digue marine.

1 INTRODUCTION

The Port of Brisbane is a fast growing capital city port on Australia's east coast and is the main port for the state of Queensland. The Port is located at the mouth of the Brisbane River at Fisherman Islands.

The deep water port and surrounds have been progressively developed over the last 25 years to cater for the increasing trade activities in SE Queensland. This trend is forecast to continue over the next 25 years and beyond. Further, Port of Brisbane Corporation (PBC) currently plans to locate all its port facilities into one area thus relocating existing upriver facilities to Fisherman Islands. The Future Port Expansion (FPE) Project is expected to ensure that port capacity is sufficient to cater for the regions' increasing demand into the future.

The FPE Project involved the design and construction of a seawall 4.6km long, extending up to 1.8km into Moreton Bay (Figure 1). This construction will allow PBC to progressively reclaim and develop an additional 230ha of port land and 1800m of additional quayline, using maintenance dredging materials to fill the enclosed site.

Significant geotechnical, environmental and construction constraints were associated with the project due to the existing subsurface profiles, close proximity of the Moreton Bay Marine Park, varying water depths and expected sea conditions during construction. Geotechnical conditions at the site are highly variable with soft clays extending over 30m below the seabed on the eastern wall alignment. The seabed itself is up to 3.5m below the lowest tide level on the east wall necessitating the need for marine construction techniques. The consistency of the soft clay at the seabed surface is very soft to soft, with undrained shear strengths as low as 3 to 5kPa.

This paper concentrates on the design and construction of the east bund of the seawall where the engineering challenges were significant.

The design and construction was carried out as an Alliance between the client (PBC) and a team consisting of Coffey Geosciences (Geotechnical Consultant), Leighton Contractors (Contractor), Parsons Brinckerhoff Australia (Civil Consultant) and WBM Oceanics (Hydraulic Consultant).

The local Port of Brisbane grid system has been adopted for purposes of this paper with the project north shown in Figure 1.

2 FIELD AND LABORATORY INVESTIGATIONS

A preliminary geotechnical investigation was carried out during the project's earlier EIS stage (URS Dames & Moore, 2001) for PBC. The investigation points were spread across an area of approximately 1.8 x 1.8km.

Figure 1 – Site Layout

The information available along the actual alignment was limited to 10 boreholes and 22 piezocones, i.e. information was available about every 0.5km. Early in the preliminary design phase, it was decided to carry out further investigations to enable realistic preliminary designs and cost estimates to be prepared by the Alliance for PBC consideration. The additional field investigations were conducted from a spud anchored barge and mainly included boreholes and piezocones, with Standard Penetration Tests (SPT) and vane shear tests conducted in the boreholes at appropriate intervals to reduce the spacing between test locations to about 200m to 250m. Conditions for undertaking the field investigations were difficult because of weather and tidal constraints. A laboratory testing program was conducted on cohesive materials to assess the physical, strength and compressibility characteristics and geophysical investigations were conducted to assess the sand quality and thickness variation across the area.

3 SITE CONDITIONS

Based on the published geology map of Brisbane (1:100,000 scale), the site is underlain by Quaternary marine deposits consisting of "fluvial lithofeldspathic sublabile sand and muddy sand". Using the information from borehole drilling and laboratory testing, the main geological formations across the project site can be summarized as Holocene deposits overlying Pleistocene deposits which in turn overlie the Petrie Formation, which consists of basalt bedrock. The Holocene alluvial deposit consists of two sub-layers:

Upper Layer: Comprises mainly sand with inter-layered soft clays and silts. The thickness generally varies across the site to a maximum of about 4m on the north bund but is relatively thin or absent along the east bund.

Lower Layer: Comprises very soft to firm compressible clay generally normally consolidated from about 3m depth below the seabed.

The thickness of the clay layer varied along the 4.6km alignment from about 6m to 30m. The majority of the east bund is underlain by about 25m to 30m of soft clay.

The Pleistocene deposit is an older alluvial deposit below the Holocene deposit and comprises mainly over consolidated, very stiff to hard clays and medium dense to dense sands. The compressibility of these materials is relatively low compared to the soft/firm clays of the Holocene deposit.

From strength and compressibility considerations, the worst conditions are along the east bund alignment, which is the subject of this paper. The upper Holocene sand layer is thin or absent and with the exception of the southern end, where the thickness is about 7m to 9m, the Holocene clay deposit is consistently greater than 25m and up to 30m thick along this bund.

4 GEOTECHNICAL MODEL

Several geotechnical models were formulated to represent the variable subsurface conditions along the alignment. The models were derived from the results of vane shear testing (which was conducted within boreholes at 1m depth intervals) and piezocone test results. Over the majority of the east bund alignment, where the clay thickness was between 25m and 30m, a single model was adopted with the shear strength being a constant 5kPa to 3m depth below the seabed increasing thereafter at a rate of 1.5kPa/m depth i.e. 0.25 x effective vertical stress. This is illustrated in Figure 2. It was also recognised that, based on the available shear strength data, lower shear strength profiles could exist locally along the alignment. The lower bound profiles were used for sensitivity analyses.

Concerns were raised regarding construction activities including geotextile placement, layer thickness to minimize mudwaving, turbidity conditions, toe heaving and mud waving. It was therefore agreed to construct instrumented trial embankments to obtain a better understanding and provide confidence in the design and construction phases. Several issues were important geotechnically, including construction procedures and loads for stability analyses, design parameters related to initial deformation and the ability to place geotextiles under water without folding (which would result in loss of load capacity).

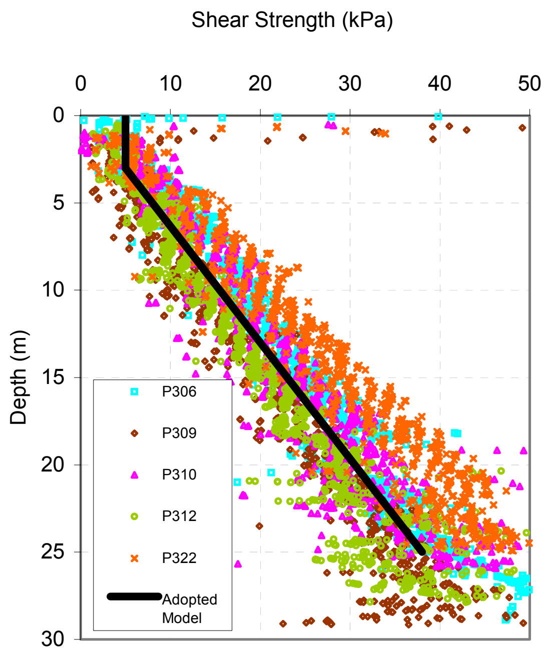

Figure 2 – Shear Strength Profiles of a Few Piezocones on the East Bund

Two trials were conducted: one inshore using landbased equipment; and the other offshore using marine based floating plant. Details of these trials are presented by Ameratunga et al (2003) and a summary is given below.

The land based trial comprised 3 x 20m sections of a multilevel rock bund up to 3m high with one of the bunds placed on a high strength geotextile whilst the others were placed directly on the seabed. The marine based trial had two separate bunds with one bund consisting of a 2m core rock layer on a geotextile and the other consisting of a 2m sand layer directly on the seabed with 1.5m of rock core on top. Ameratunga et al (2003) describe the geotechnical models adopted and analyses conducted using commercially available software PLAXIS and FLEA.

The trials provided valuable data which were used for back analyses. As a result, elastic deformation parameters for the soft clay under the east bund was modified and a value of 100 was adopted for the ratio of undrained Young's modulus and undrained shear strength. The trial results highlighted potential conservatism in the treatment of construction equipment loads and geometry in stability analyses in the direction of the alignment because of three dimensional effects.

Another concern was the expected damage to the geotextile due to rock placement and trafficking. While empirical equations were available for geofabric strength assessment due to falling rock, no literature could be found on trafficking. Several panels were constructed on land and rocks of different but known sizes were dropped at different heights. Panels were separately subjected to trafficking to assess damage. Damage on each panel was quantified and a damage factor of 1.7 was adopted to downrate the geotextile strength.

5 DESIGN AND CONSTRUCTION

The final East Bund seawall design was a sand/rock embankment. The remaining seawall sections consisted entirely of rock. A design cross section for the east bund is shown in Figure 3 and has the following key features.

1) Basal high strength geotextile (ult. capacity of 700kN/m).
2) Sand pancake wider than the basal geotextile.
3) Filtration geotextile to cover the sand and to minimise sand movements due to tides and waves.
4) Rock core bund above the sand.
5) Armour.

The seabed on the east bund alignment is about RL –3.5m (Port Datum) with a final surface level of RL 4.0m for the bund leading to an overall bund height of 7.5m. With anticipated settlement during construction alone being greater than 1m, the final bund height is closer to 9m. One of the critical issues in the design was bearing capacity, with the undrained shear strength of the soft clay at shallow depth varying from 3kPa to 5kPa. While the concepts of ground improvement using wick drains, inclusions and additives were considered, each had significant risks and/or costs associated and were discarded. A staged approach to the overall embankment construction was also not feasible due to time constraints on the construction program. Another critical issue was potential instability in the longitudinal direction for construction vehicles if a land based construction approach was adopted, which would require additional high strength geotextiles. The use of sand placed by barge in layers, on top of the basal geotextile laid in the lateral direction, was found to be the most economical and lowest risk solution.

The high strength basal geotextile was a key component in the design. The geotextile, supplied in 5m wide rolls, was transported to a tarpaulin factory for stitching into panels up to 42m wide and 100m long. Several types of stitches were trialed and tested before selecting a single J Seam (i.e. one fold), with two rows of stitches 25mm apart, for the final design. The panel was folded to 2.5m width and rolled and transported to the site. The barge used for geotextile laying was fitted with equipment that could handle the roll and subsequent placement. Handling difficulties posed by the weight of rolls (6t), the length of the barge and the sea/wind conditions limited the maximum width of the geotextile to 42m, allowing little tolerance for placement. As the geotextile width did not cover the full width of the sand "pancake", the design was sensitive to the correct positioning of the geotextile. The placement of the geotextile was carried out by the barge with GPS control. An innovative method was utilised to place the geotextile using a pulley system with the geotextile being inserted into the water in front of the barge, then passing beneath the barge as the barge winched its way forward, before being placed on the seabed and weighed down with small rock core dropped from the rear of the barge.

The long-term factor of safety (FOS) adopted was 1.5. The construction FOS adopted was between 1.15 to 1.25, and sensitivity analyses were carried out on lower bound strength profiles to achieve a minimum FOS of 1.0. The construction FOS values adopted are relatively low for a project where consequences of failure were high from safety and environmental considerations and from impact to time and costs if a failure was to occur on either one of the two leading seawall construction faces. As a result, sequencing of traffic including excavators, dozers, a continuous stream of truck movements of rock core and armour, and the simultaneous progress of construction at several intermediate faces (core rock, secondary armour, primary armour) posed a significant challenge to the designers. As sequence and equipment used and construction staging differed from section to section and the possibility of encountering local weaker areas was high, it was assessed that the risk of instability during construction is high (Ameratunga et al, 2004). To manage risk, a Geotechnical Work Method Statement (GWMS) was jointly prepared by the design and construction teams. The GWMS for each section clearly illustrated step by step procedures as to how construction should proceed to maintain stability. Some of the points included where rock could be dumped, set back from construction faces, and the number and type of equipment that could be used at a particular section at any time. The information from the GWMS was disseminated via “tool box” meetings and using simple sketches (Figure 4 shows an example for South Bund).

One of the key construction issues that was sensitive to the design was the geotextile placement. With GPS control and skilled operators the as-placed location was generally within 1m. In order to verify the position conventional surveying was adopted. Another critical issue was the development of folds in the geofabric during placement. Although long steel reinforcement bars were tacked onto the panels every 10m, divers examined each and every panel and folds were measured and noted. The information on placement location and folds were fed back to the design team who checked the designs with as-constructed information. Folds of short/limited length were treated as damaged area and incorporated as a damaged factor to downrate the design strength. Where the factor of safety was compromised, the panels were stretched and/or moved by tugging from the barge. Where this was not possible the upper part of the geometry was modified to obtain a satisfactory outcome.

One of the major issues which arose during construction was associated with underfilling and overfilling of the sand “pancake’ during hydraulic placement. The sand “pancake” was surveyed soon after construction and prior to rock placement using multi-beam hydrographic surveying techniques. From this survey information, as-placed sand cross sections were created along the alignment at 5m intervals and the information was analysed to assess the departure from the standard design geometry. Where departure was outside design tolerances, sections were re-analyzed and the upper part of the geometry was changed to obtain a satisfactory construction factor of safety. Further issues that were resolved during construction with design input included: modifying the design to incorporate movement of basal geotextile due to high currents generated by passing ships; sand movement prior to covering; and inadequate lapping between geotextile panels.

An instrumentation plan was established to assist the designers and constructors to monitor adverse behaviour and to implement remedial measures if required. The instrumentation plan included 30 piezometers at different depths, 89 settlement plates, 66 surface settlement monitoring points and 20 inclinometers. The instruments were read at frequency intervals established by the designers and managed in a database. Unusual behaviour was investigated, and where required design and/or construction sequencing changed. For example, high lateral movement monitored in one inclinometer led to a change in the construction sequence and placement of a toe berm prior to lifting the bund further.

CONCLUSIONS

The following conclusions are based on the investigations, trials, design and construction of the seawall project:

- High strength geotextiles provide a sound base when operating on very weak soils. In the absence of literature on damage to geotextiles due rock placement, field trials provide valuable information for the designers.
- Back analyses using monitored data from field trials provide valuable data to enable design modifications.
- When construction factors of safety are relatively low and consequences of failure are high, appropriate construction methods and controls could be used to manage risks.
- Instrumentation can be effectively used to monitor adverse behaviour and to implement remedial measures if required.

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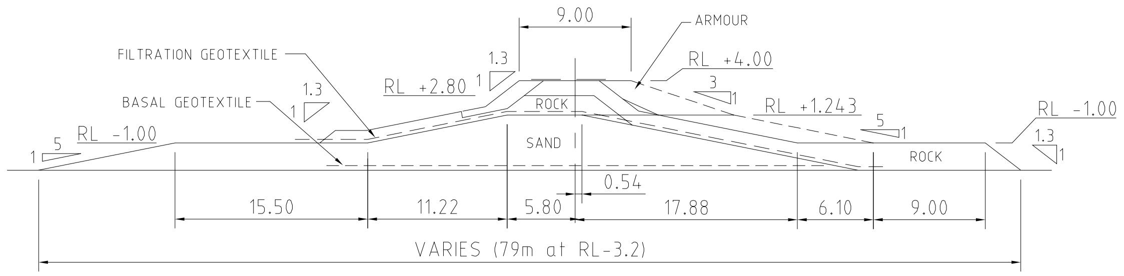

Figure 3 – Typical Cross Section of the East Bund

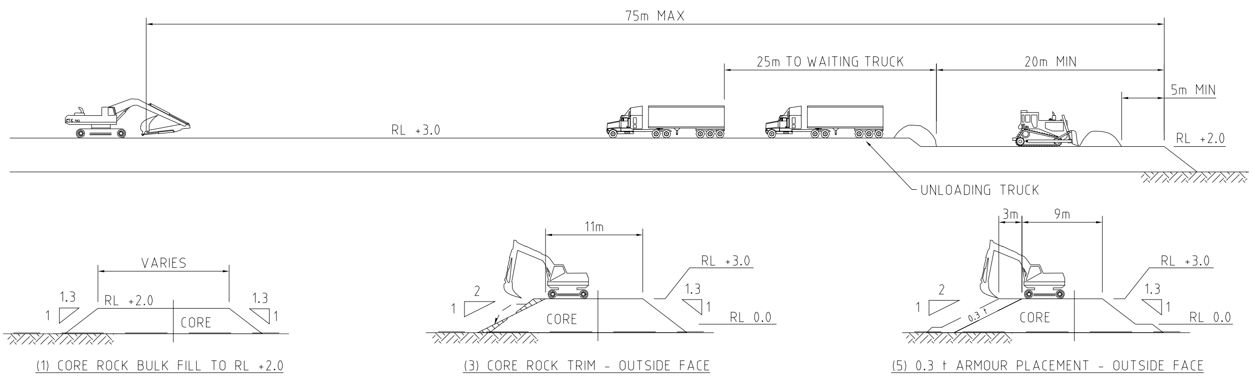

Figure 4 Sketch Illustrating a GWMS using South Bund as an Example