

GEOTECHNICAL ASPECTS OF DREDGING AND RECLAMATION WORKS FOR THE PORT BOTANY EXPANSION PROJECT

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ABSTRACT

This paper contains a discussion of the geotechnical aspects of the dredging and reclamation design and construction for the Port Botany Expansion (PBE) project in Sydney, Australia. The project involved dredging approximately 11Mm³ of Botany Bay Sands to form the 63 Hectare (Ha) reclamation. Dredging and reclamation works for the project commenced in 2008 and were completed in 2010. The bulk dredging works were carried out using cutter section dredges and various reclamation techniques were used including land discharge (pipelines), spreader pontoon and aerial discharge (rainbow).

This paper examines the dredging work completed at PBE from a geotechnical viewpoint and describes issues faced during design and construction, including methods for identifying suitable source dredging material, practical dredging tolerances, design of disposal areas for unsuitable materials and stability of underwater cuts and fill slopes.

The paper also discusses the specification for ground improvement works completed for the reclamation, including the earthworks testing requirements and fill verification processes adopted. The potential for fines being included in the reclamation was a major risk to future performance of the reclamation and so the methods used to assess and mitigate this potential risk are discussed. The results obtained from different compaction techniques including dynamic compaction, vibrocompaction and impact roller compaction are also presented in the paper.

1 INTRODUCTION

The Port Botany Expansion (PBE) project is a major upgrade of Sydney's main container handling port on the eastern shore of Botany Bay, approximately 12 kilometres (km) south of the Sydney CBD. It will cater for forecast increases in container trade demand driven by the new wave of large post-panamax sized vessels following the opening of the widened Panama Canal in 2014. The total project cost including additional works by the operator is estimated at approximately \$1 billion.

The complete outer edge perimeter of 2,500 m around the PBE development is constructed from a combination of 216 precast counterfort units, revetments and three precast caisson corner and transition structures. Berthing comprises almost 2 km of deep water frontage and reclamation involves filling of 63 Ha of new land between the existing terminal and Sydney Airport's third runway. The total dredging volume comprised approximately 11Mm³ of material, of which about 8Mm³ comprised reclamation source materials.

Sydney Ports contracted a joint venture of Baulderstone (major civil contractor) and Jan De Nul (specialist dredging, reclamation and marine contractor) to design and construct the new terminal and associated works. Design of the expansion project was led by Hyder Consulting with specialist geotechnical support from Golder Associates and specialist marine engineering support from URS-Scott Wilson. The project verifier was Parsons Brinckerhoff Australia.

This paper focuses on the unique and challenging aspects of dredging and reclamation at PBE, which include:

- Substantial dredging and backfilling works required for deep foundation trenches to RL (Reduced Level) -30 mCD (Chart Datum) to remove unsuitable clay material below the wharf structures;
- Mapping out the presence of unsuitable reclamation source materials and managing dredging and reclamation processes so as to reduce the risk of these being placed in the reclamation;
- Tailoring compaction techniques and developing innovative compaction methods to improve fill compaction without adversely affecting the wall structures and
- Developing site specific compaction compliance testing targets and compaction measurement strategies to provide confidence of meeting design criteria and long term performance objectives.

2 BACKGROUND AND GEOLOGY

The foreshore around the PBE site has developed into an industrial area over the past 50 years. In the late 1970s, the existing container terminal, Brotherson Dock was constructed using a similar construction method and structure type (reinforced concrete counterforts) as that used for PBE. Backfill to the existing structures comprised hydraulically placed sand, however only surface compaction of the original container yard was completed, in contrast to deep compaction completed for PBE.

Prior to construction of Brotherson Dock, the seafloor over the PBE site area lay at approximately RL-5 mCD, with a gradually increasing depth towards Botany Bay to the south. When Sydney Airport constructed an additional runway in the 1980s, a borrow pit was dredged at the intended location of future navigational basins for an expanded Port Botany. Unfortunately this borrow pit is now located directly under the reclamation for the PBE project, requiring substantial additional backfilling of this depression. Although this feature necessitated additional backfilling for the new port facility it did provide some useful design information relating to long term stability of underwater slopes, as discussed later in this paper.

The geology of Botany Bay has been discussed extensively by other authors, including Thorne, 1985 and Albani *et al.*, 1998. A brief overview is provided below, as background to the main theme of design and construction of the dredging and reclamation at PBE and lessons learnt during the project.

The PBE site is underlain by Hawkesbury Sandstone bedrock. The depth to bedrock varies significantly across the site area, reflecting the presence of major paleochannels. The eroded bedrock surface is overlain in most areas by Quaternary sediments comprising thick, problematic fissured marine clays (Unit 3 and 4 Clay), overconsolidated botany sands including minor clay and lignite sub-units (Unit 2) and loose and sandy recent sediments (Unit 1). The upper Unit 1 and Unit 2 sands comprised the reclamation source materials for the PBE project. A schematic geotechnical cross section of the dredging and reclamation works is included as Figure 1. A graph showing results from grading tests of the reclamation source materials is included as Figure 2.

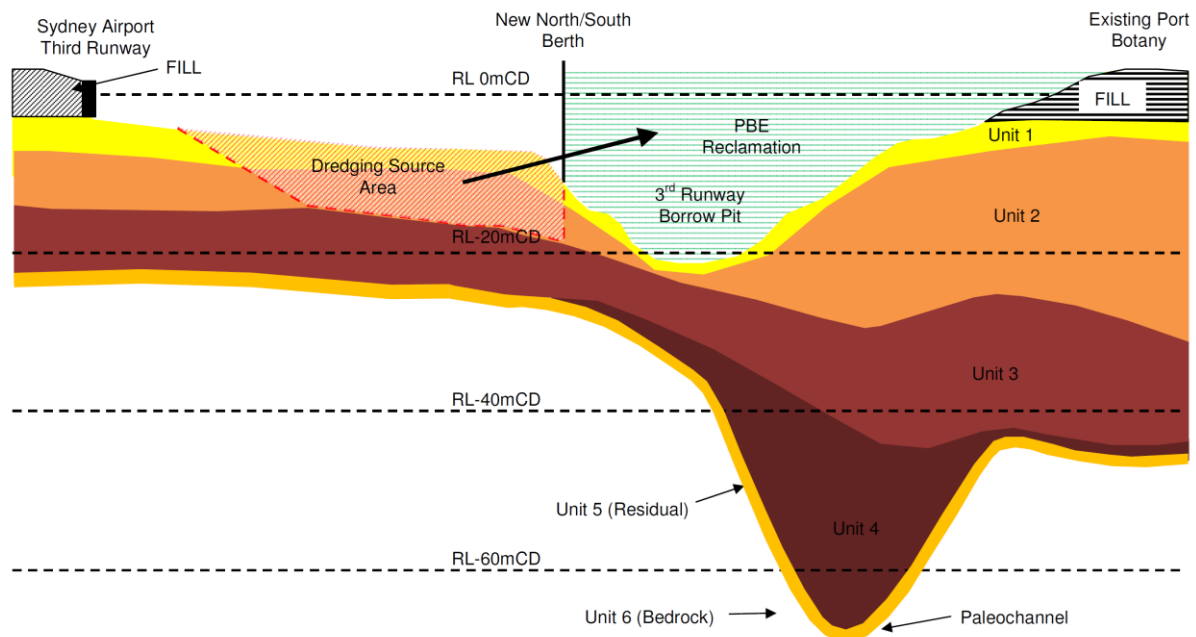


Figure 1: Schematic Geotechnical Cross Section E-W through PBE Site Area (Not to Scale)

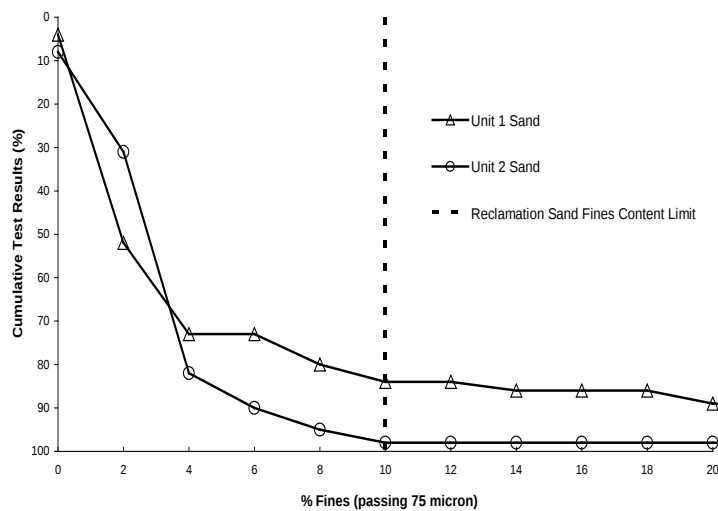


Figure 2: Summary of Grading Tests from Dredging Source Materials.

3 SCOPE AND DESIGN REQUIREMENTS

3.1 SCOPE

Dredging and reclamation for the new land within the project area was undertaken by a cutter suction dredge which collected material at seabed level and pumped it in slurry form through a floating line for discharge into the reclamation area. Most dredging was completed using a large cutter suction dredge (CSD) “Leonardo Da Vinci” and split hopper dredge “De Bougainville” (Figure 3), which can dredge, store material and dump dredged material by splitting the vessel along its length. In this way, De Bougainville was able to place unsuitable materials into a dedicated underwater dump pit.



Figure 3: Cutting Head from Cutter Section Dredge and Split Hopper Dredge – De Bougainville

Dredging was required for the main berthing basin and for deepening and widening the main Port Botany navigation channel, and to create a new tug channel to elevations indicated in Table 1, below. Additional minor dredging works were needed between the main reclamation and the land along Foreshore road to facilitate tidal flushing along the Penrhyn estuary. The scope of dredging operations also required significant environmental considerations including floating sediment curtains and compensatory seagrass planting.

In essence, the scope of dredging activities comprised:

- Dredging to required depths to provide compliant reclamation source material and facilitate navigation;
- Dredging and disposal of unsuitable seabed materials;
- Dredging of foundation trenches, removing fissured Unit 3 and 4 marine clays and backfilling the trench with clean sand and
- Trimming dredging works such as trench dredging for scour protection in front of the quay wall.

Table 1: Nominated Dredge Levels for Channels and Basins.

Facility	Dredge Area	Base Width	Minimum Depth (RL)
East/West Berth	Berthing Basin	60	-16.5 mCD
North/South Berth	Berthing Basin	60	-16.5 mCD
	Commercial Navigation Channel	172	-16.5 mCD
	Tug Manoeuvring Channel	100	-8.0 mCD
	Recreational Navigation Channel	150	-2.5 mCD
Tug Berth	Tug Channel	80	-7.0 mCD

Reclamation works included placing and compacting sand fill to the required finished surface levels which varied between RL+2.8 mCD and RL+3.7 mCD. The deepest area within the reclamation footprint was approximately RL-24 mCD at the base of the third runway borrow pit. This resulted in a maximum reclamation fill depth of about 27 m. Volume calculations to assess the amount of required fill included consideration of overfilling required to compensate for soil compaction and operational losses.

3.2 DESIGN REQUIREMENTS

The design requirements for the PBE works were set out in the Project Scope and Technical Requirements (PSTR). These were augmented by specifications and additional controls developed by the design team during detailed design. Dredging design requirements included:

- Ensuring stability of the adjacent existing berth structures, third runway and new berth / revetment structures;
- Identifying top of Unit 3 clay material which forms the natural lower boundary of suitable reclamation materials;
- Identifying unsuitable source materials (peat lenses, clay and silty seabed sediments with >10% fines) and
- Methods to manage Potential and Actual Acid Sulphate Soils.

Other dredging considerations included noise, turbidity, water quality and the need to remain within the airport flight path envelope known as the OLS (Obstacle Limitation Surface) which extended from the adjacent airport runway.

For reclamations, the overall design objectives were to provide a stable platform for future terminal facilities, to limit total and differential settlements and to have strength and stiffness characteristics that were compatible as backfill for edge structures which retain the reclamation. The distribution of fill types is shown schematically in Figure 4, below.

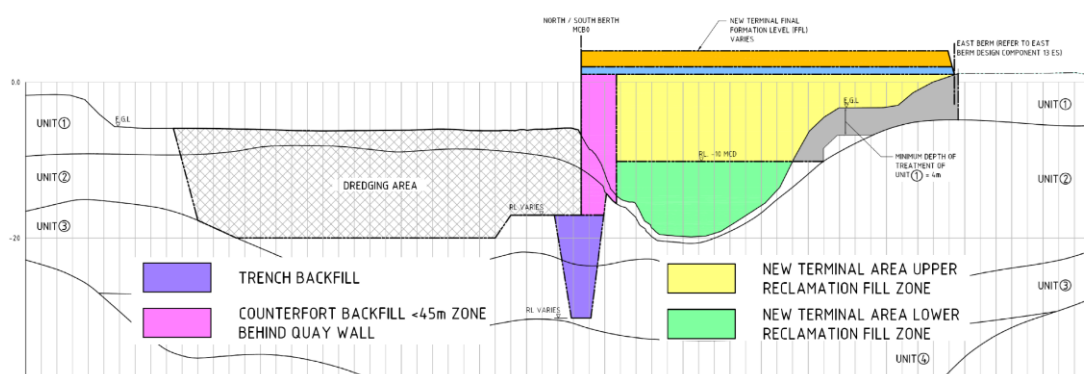


Figure 4: Schematic Plan and Section Showing Reclamation Fill Types

Required reclamation fill properties and selected performance targets nominated in the PSTR are summarised below:

- Fill to have a fines content $\leq 10\%$ and not contain concentrations of clay and silt in lenses or pockets;

- Fill to have a friction angle of $\geq 30^\circ$;
- Testing for shallow soils (Uppermost 3m below finished surface level) to include Field density testing and Density ratio, Shear box testing for friction angle, CBR;
- PSTR requirements for deep soil compaction (assessed from cone penetration test (CPT) tip resistance, q_c):
 - o $q_c \geq 5$ MPa for a minimum of 90% of test values;
 - o $q_c < 2$ MPa for no more than 4% of test values and
 - o $q_c < 0.5$ MPa for no more than 2% of test values.
- Designer specified requirements for deep soil compaction: various compaction curves (See Figure 13 for examples) and
- Vertical reclamation settlement:
 - o Settlement < 250 mm at 20 years;
 - o Settlement < 300 mm at 50 years and
 - o Settlement as a proportion of fill thickness i.e. $< 0.1\%D$ at 1 yr.

A schematic figure showing the filling sequence and corresponding settlements is shown in Figure 5, below:

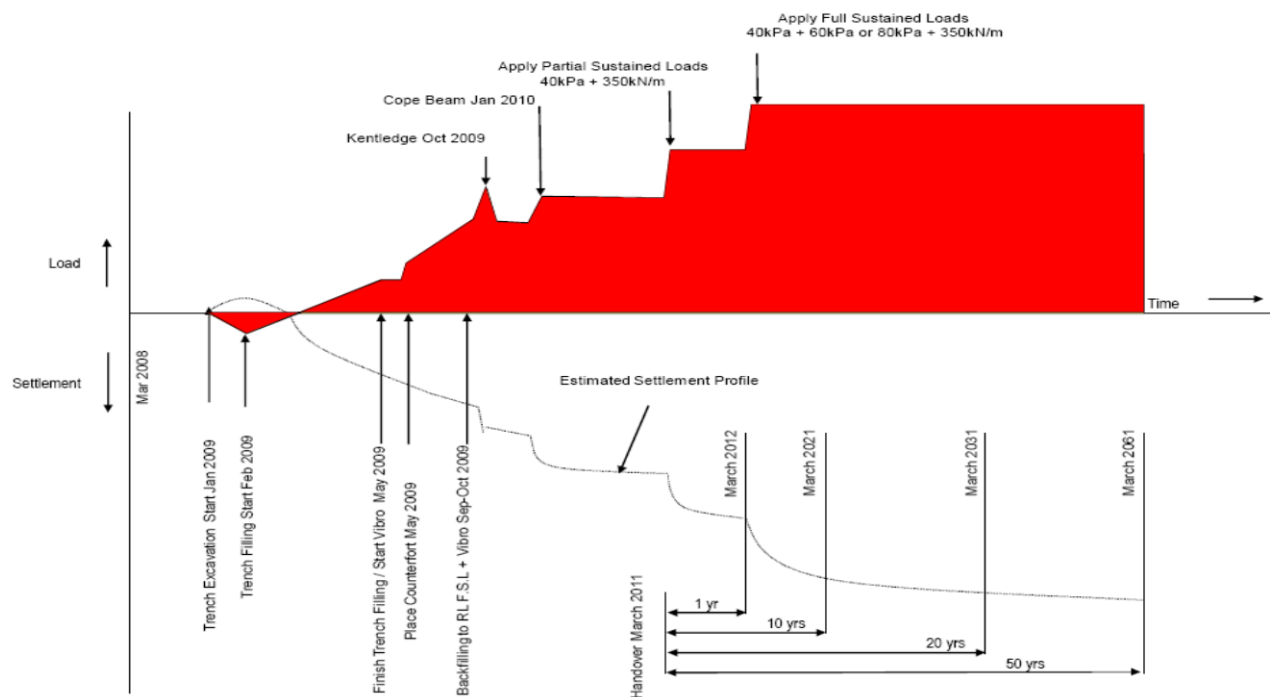


Figure 5: Schematic fill loading sequence and site response

3.3 DREDGING TOLERANCES

Vertical dredging tolerances were $+0$ mm, -400 mm close to new or existing berth or edge structures and the third runway seawall and $+0$ mm, -800 mm in other areas, where positive and negative values indicate levels above and below the design depth, respectively. The horizontal tolerance was ± 1000 mm. To enable real-time control of excavation levels during dredging, the cutting head of the cutter suction dredge was fitted with GPS equipment. In practice, these tolerances were consistently achieved, although this required skilled operation of the cutter suction machinery, taking into account the inertial effects, wind and the massive scale of equipment relative to the dimensional tolerances. In some areas over-dredging was deliberately carried out below the specified minimum depths and beyond channels and basins in order to optimise the recovery of suitable reclamation material for the new terminal area.

Levels were checked using multi-beam echo sounder survey methods, which were considered accurate to approximately 100 mm.

3.4 SETTLEMENT ASSESSMENT

Settlement studies for the reclamation fill platform considered filling, applied loads and immediate, consolidation and creep settlements at specified time intervals. These analyses took into account ground treatments required to achieve nominated density, strength and settlement requirements for the Reclamation area. Completed analyses included the following:

- One-dimensional settlement analyses using consolidation and elastic theory at inland locations below the new terminal area;
- Comparison and tie-in of above calculated settlement results with PLAXIS finite element results along the terminal edge structures;
- Contouring of total settlements and settlements within the fill at nominated post-filling time intervals and
- Assessment of treatment types needed in the reclamation fill and *in situ* seabed materials to meet the requirements of the PSTR.

Detailed settlement and movement analyses were carried out in critical interface areas and structures using Settle-3D and PLAXIS software. Different design techniques were used to assess reclamation settlements in the reclamation area depending on location and stress conditions at any given point.

In parts of the main reclamation (New Terminal Area), maximum depth to rock head is approximately 70 m from the finished level of the new reclaimed platform area. Towards the edges of the reclamation area non-uniform stress changes associated with berth and revetment loadings (three dimensional effects) were significant. In berth areas, soil structure interaction effects due to the presence of the counterfort wall and rear crane beam piles (a piled raft on settlement reducing piles) were taken into account when assessing movement compliance.

The predicted settlements were based on average unit compressibility values assigned to each unit, which were largely based on calibrated back-analyses from the previous Brotherson Dock Reclamations. The settlement predictions were extended to include sensitivity analyses for key variables such as the unsuitable materials at the existing seabed. The sensitivity analyses included consideration of compressibility and consolidation characteristics, rate of loading and thickness. Some probabilistic assessment was also undertaken to investigate the spread of predicted values, which along with the sensitivity analyses provided a semi-quantitative measure of confidence in design.

Settlements occurring at seabed level during filling will be 'built-out' as filling and ground treatment takes place.

Post-filling settlements also occur in the reclamation material during the period between completion of filling and hand-over due to densification during ground treatment and due to settlements of underlying *in situ* soils. Post-densification settlements of ground surface between completion of filling and handover (1 year assumed) associated with consolidation of fill and underlying soil were assessed as approximately 10 mm to 60 mm. Maximum predicted long term reclamation settlements over 50 years post-hand-over were up to about 150 mm, which was within the specified maximum of 300 mm at that time interval. An example of settlement contouring, completed during design is shown in Figure 6.

To date, reclamation settlements are performing well against predictions, noting that at three locations, small-scale remedial preloading and monitoring were used where unexpectedly thick sediments were identified. During construction over 2,000 check analyses were done at about 250 CPT locations, with more than 99% of results satisfying all settlement criteria.

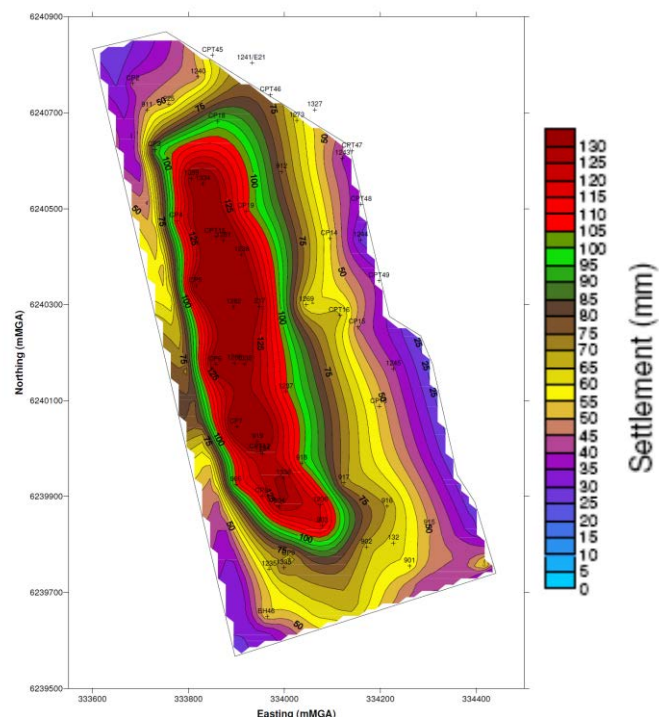


Figure 6: Predicted Settlement Contours at 50 Years

3.5 SEISMIC ASSESSMENT

An assessment of factor of safety against liquefaction in reclamation fill and sandy soils was undertaken based on fines content, material strength and design earthquake loading. Liquefaction potential of reclamation materials was undertaken for backfill using the method of Youd (2001).

The assessment was undertaken for the ultimate (1 in 1000 year) seismic event, using a Peak Ground Acceleration (PGA) value of 0.14 g, as specified in the PSTR. A limiting target Factor of Safety (FoS) against liquefaction of 1.1 was adopted.

The design showed that the PSTR nominated criteria for CPT $q_c \geq 5\text{MPa}$ criteria generally mitigated liquefaction potential and was achievable under self weight below about 12m depth of sand fill. Except near wall and crane rail structures, fill below this depth was left untreated as settlement performance was adequate provided overlying soils were treated.

4 DREDGING GEOTECHNICS

4.1 INTRODUCTION

Due to the power of modern cutter suction dredges, changes in soil types (i.e. from sand to clay) do not generally cause any noticeable change in machine operation to the dredge operator. A more marked change in material, such as from soil to rock will be noticeable. Therefore, it is important to have other controls to manage the risk of sourcing and placing unsuitable materials during the dredging and placement operations. Geotechnical aspects of the dredging design are discussed below, grouped by key design and construction activities.

4.2 VULCAN MODELLING

At the start of the PBE project the tender stage geotechnical information from the site, comprising three hundred and sixty boreholes and CPTs, was combined to produce a model for the site. The geotechnical model was generally interrogated using 2D sections, which were cut at various points along the new berths. Whilst this was useful for geotechnical modelling of stability and movement of the new structures and reclamation, a 3D model was needed to control operations. Therefore, Vulcan software was used to generate a 3D model to control the dredging operation. The Vulcan model focussed on the resources to be dredged and in particular identifying:

- The sand materials (Unit 1 and Unit 2 sands) that were suitable for use in the reclamation;
- The top of the Unit 3 Clay layer, which was the lowest point of the useable sand resources and
- Unsuitable cohesive sub-units (>0.5 m thick) within the reclamation source materials, particularly where there were larger areas of unsuitable material with the potential to be continuous between multiple investigation locations.

The Vulcan model generates surfaces by linear interpolation between unit boundaries identified from boreholes, which can result in apparent angularity in the modelled surfaces, rather than smooth surfaces one might expect in reality. As with most geotechnical models it was recognised that there will be variation between the modelled conditions and the actual ground conditions. This is a typical modelling issue faced on dredging operations, leading to provision of buffer layers around known unsuitable zones (typically 0.5 m) and controls being placed on the dredging process to maintain fill quality. These issues underline the importance of providing an adequate investigation scope and understanding the geological site processes and history of both source materials and unsuitable materials. At PBE, this geological knowledge was documented by previous workers (principally Thorne, 1984, Albani *et al.*, 1998 and Fell, 2006) and provided a robust geological site framework which assisted the design team to optimise the Vulcan model, target locations for additional investigation works and characterise soil behaviour within each sub-unit.

During design the Vulcan model was used to quantify dredging volumes in each unit and to assist with planning the dredging program, dump pit sizing and sizing of source dredge areas. When considering dredge volumes, bulking factor values of 1.1 and 1.75 were used for sands and overconsolidated clays, respectively.

During construction, the Vulcan model was loaded onto the dredging control system. Figure 7, below shows a diagram of a cutter suction dredge and an example screenshot from the dredge operators control panel, which shows the dredge limit as a separate boundary above unsuitable material.

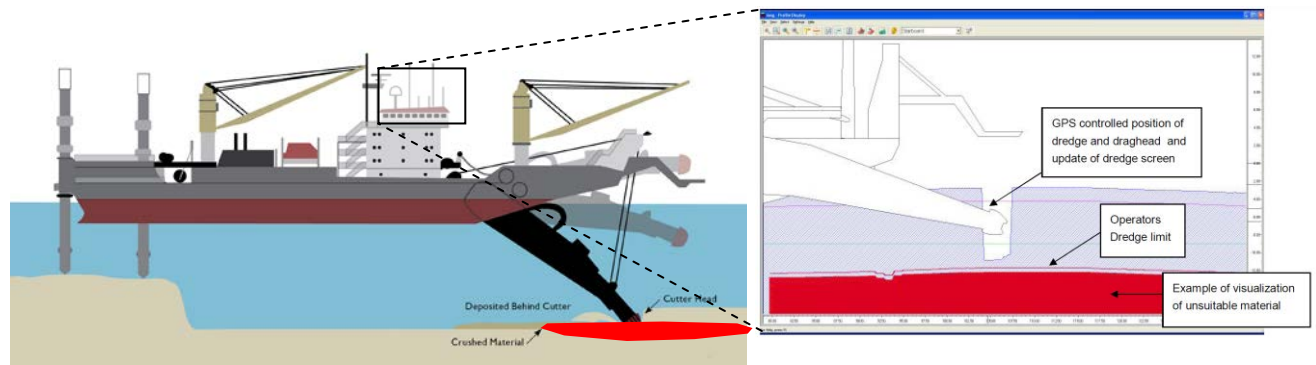


Figure 7: Diagram of Cutter Suction Dredge and Example of Operator Control Screen

4.3 TEMPORARY AND PERMANENT UNDERWATER SLOPES

During construction of reclamations and during dredging works in general, the gradients assumed for underwater slopes can have a major impact on the project, particularly in relation to the available working areas and production volumes. Published information (Look, 2007) includes recommendations for sand slopes underwater as follows:

Table 2: Underwater Slopes (After Look, 2007).

Soil	Slope in still water (v:h)	Slope in active water (v:h)
Coarse Sand	1:3	1:6
Fine Sand	1:4	1:11

A number of factors in addition to the material type impact underwater slope angles, including whether the slopes were cut into existing material or are new reclamation slopes (cut slopes generally being steeper than fill slopes) and the depth below the water surface at which the slope is located, generally shallower slopes being in a more active zone.

Table 2 is considered to be reasonable given the authors' recent experience during the Port Botany expansion works. As part of the design, the side slopes of the third airport runway borrow pit (constructed in the 1980s) were found to have overall slope gradients ranging from 1:12 to 1:4 with no major slope geometry change observed over a twenty year period. This local information was used to help justify the use of permanent unarmoured dredged slopes of 1(v):5(h). Steeper, armoured underwater slopes were constructed during the PBE project and were designed at slopes of 1(v):1.5(h) to 1(v):2.5(h), depending on location, geometry and loading requirements.

Movement of dredged areas was monitored before, during and after construction by surveys. No vertical settlement or horizontal movement limits were specified for underwater slopes in the PSTR, although it was specified that constructed dredged slopes must remain stable. Repeated surveys following initial dredging and between cleaning passes of the CSD demonstrated that trench sidewall instability was very rare. However, one risk identified during the project related to failure of rapidly placed granular reclamation fill slopes. This subject is discussed in more detail in Section 6.

4.4 AIRPORT RUNWAY STABILITY

The main dredging operation to form new channels was completed adjacent to the airport third runway. As part of the PBE design, the effect of the dredging on existing infrastructure was assessed. In addition to the geotechnical impacts of the new channels, the effect of construction of the new terminal on the waves and currents within Botany Bay were also assessed, together with an assessment of the potential impact that vessels (container ships and tugs) could have on underwater slopes, the dump pit and new and existing structures and revetments. Limit equilibrium stability assessments were completed to confirm that dredging adjacent to the existing airport third runway was acceptable (minimum global factor of safety 1.50).

4.5 SOFTENING OF CLAYS

The potential softening of Unit 3 and 4 clays was considered at the design stage, as this was considered a potential risk to the temporary stability of the counterfort trench foundation before it was backfilled with sand. Calculations for softening due to unloading caused by dredging were based on shear strength relationships presented in Thorne (1997).

Calculations indicated that softening of the Unit 3 and 4 materials in the deepest dredged areas could lead to a reduction in undrained shear strength to an average value of approximately 64 kPa, or 74% of the adopted design values. Sensitivity analyses were completed to assess the potential risk to temporary and permanent slopes, i.e. how much softening would need to occur before stability factor of safety (FoS) values were not acceptable. For the most critical permanent location it was found that average softening to approximately 35% of the adopted design value could occur before unacceptable FoS values occurred for slopes for short term (undrained) conditions.

4.6 DUMP PIT STABILITY

Extensive areas of unsuitable materials from within the dredging areas, together with the Unit 3 and 4 clays from the trench foundation of the structures were dredged during the PBE project, amounting to approximately 1Mm^3 . The PSTR required that these materials were to be disposed of in a “stable location”. An underwater bund was formed adjacent to the airport third runway to contain dredged unsuitable materials. These unsuitable materials comprised mainly stiff lumps of excavated Unit 3 and 4 clay from the trench foundation dredging. Normally consolidated clay and silt sized sediments, or “slimes”, produced from the dredging process were also disposed of in this area. The geotechnical design for the dump pit bund was completed assuming the contents of the dump pit were normally consolidated, the final dump pit design and the location of various new channels are shown in Figure 8.

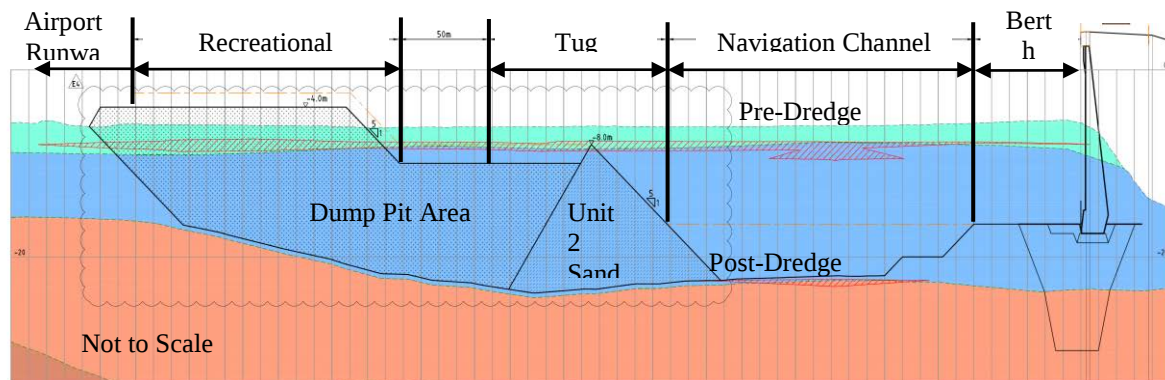


Figure 8: East-West Cross Section through Dump Pit and New Navigation Channels

4.7 SEABED SCOUR ASSESSMENT

Seabed velocities caused by tug propeller wash were considered to assess the potential impact on permanent dredge slopes. To prevent scour in critical areas, rock armour was placed on the finished seabed typically out to 20 m from berth structures and up to 33 m near the tug berth. In assessing the potential impact of tugs on underwater slopes the fact that tugs will not operate directly over the underwater slopes was taken into account.

Seabed scour calculations used to assess scour effects outside these areas were completed by URS-Scott Wilson using the PIANC WG22 method. A bed velocity of approximately 1m/s was estimated due to tug propeller wash and correlations were used to take account of diffusion associated with varying depth and distance from tug operations. Considering the actual finished seabed geometry, it was assessed as unlikely that the maximum bed velocity would exceed 0.65 m/s which was considered to present a low erosion risk to the permanent unarmoured dredged slopes, particularly at the specified gradient of 1(v) to 5(h).

4.8 ACID SULPHATE SOILS

Some soils on the PBE site were identified as potential acid sulphate soils (PASS). The soils were mostly “clean” sands, but also included some organic clays and peaty clays. The colour of the soils varied from light brown to dark grey. Whilst these soils remain saturated, they remain in their current state. However, when iron sulphides within the soil are exposed to air the resulting reaction can produce sulphuric acid, affecting the local environment and water quality.

For PBE, any PASS identified during construction through monitoring processes was disposed of within the proposed reclamation area below the water table. This, when combined with soil mixing during dredging, was considered an acceptable method to mitigate the risk of impacts from acid sulphate soil within the reclamation.

5 RECLAMATION GEOTECHNICS

5.1 FILL PLACEMENT AND COMPACTION TECHNIQUES

The general filling process involved dredging material from seabed level by a cutter suction dredge and pumping it in slurry form through a floating line for discharge by one of three methods:

- Spreader Pontoon – underwater discharge through a vertical pipe below a floating barge. Used mostly for filling deep submarine areas.
- Surface Discharge – slurry is pumped out of a land based pipe into open water or bunded settlement ponds. Dozers then move fill and discharge pipes according to progress and desired filling location.
- Aerial discharge (or ‘rainbow’) – slurry is sprayed through the air from a ship based monitor nozzle into open water or bunded settlement ponds.

Each technique was selected based on the filling application and water depth. As reclamation material was placed above water, stockpiles (with crest level being up to RL+15 mCD) were created to allow efficient placement and storage of reclamation fill without impacting the stability of the excavation for the trench foundation or the operation placing wall units. Stability analyses were undertaken for such stockpiles and controls applied to the fill rates, heights and slopes so as to control the risk of stockpile slope failure associated with build up of excess pore pressure in underlying slimes layers.

Following placement, the reclamation zones were allowed to rest for at least one week prior to compaction, allowing remnant pore pressures to dissipate and some strength gain to occur with ageing effects. Figure 9, below shows surface discharge and the commencement of compaction works.

The main methods of compaction included vibrocompaction (VC) in deep areas near wall structures, dynamic compaction (DC) across inland reclamation areas and impact roller methods in shallow fills. So as not to adversely affect wall structures, a perforated vibrating sheetpile was also trialled for use in confined areas near structures. Some notable outcomes from the compaction techniques are summarised below.



Figure 9: Reclamation activities and discharge lines

5.2 LAND DYNAMIC COMPACTION (DC)

Inland of the VC zone, compaction was undertaken using DC methods. Following trials, footprints of production impact points were completed in three to four phases, resulting in a 4 m to 5 m square grid using a pounder weight of 25 tonnes and drop height of 23 m. The DC method was shown to be highly effective in granular soils over a depth of about 8 m to 10 m, with diminishing compaction improvement thereafter. Effectiveness of the method decreases with increasing fines content of the soil. Cohesive soils having a fines content of greater than about 20% are not easily compacted or consolidated by DC. Multiple DC passes were required to satisfy stability and liquefaction requirements over revetment areas where thin cohesive layers were more persistent at the edge of the Penrhyn estuary. Figures 9 and 10 show DC in progress on the PBE site. Target compaction profiles were consistently achieved to depth of approximately 8m. Below this, the reducing effect of the DC compaction generally led to non-conformances against the design criteria, which were based on a uniform increase in compaction being achieved to RL-10mCD. However, the reduction in compaction at depth was generally offset by higher compaction at shallow depth, leading to minimal impact on overall settlement predictions.



Figure 10: Land Vibrocompaction (left) and Dynamic Compaction (right)

5.3 LAND VIBROCOMPACTION (VC)

During design it was important to tailor the structural wall design for site specific earth pressure envelopes, as this provided a significant opportunity to achieve savings on wall thickness, materials and costs. Rather than designing for simplified earth pressure envelopes, soil structure interaction analysis was carried out and found to be complex and construction stage dependent. Models were studied in depth to capitalise on design efficiencies and reduce wall weight. The challenge was to tailor compaction methods so as not to exceed these wall pressure envelopes.

Compaction within a zone extending 45 m inland of wall structures was undertaken using VC methods from a working platform at approximately RL+2.5 mCD to RL+3.0 mCD. A typical triangular grid spacing of 3.5 m to 4.5 m was adopted using a Vibroflotation AG Germany 175kW probe, which has an eccentric force in the range 230 kN to 470 kN at a frequency of 60 Hz. Figure 10 shows VC underway.

A series of trials were undertaken to confirm compaction induced earth pressure effects and tailor the compaction method near the wall structures to achieve the necessary density gain for stiffness, stability and liquefaction, without causing excessive lateral earth pressures. The outcome of the trials was that reduced energy compaction was needed behind wall structures. VC was used with a minimum probe offset of 2.5 m and two different probe power settings and lift rates. A perforated vibrating sheetpile was also used depending on proximity to wall structures. Figures 11 and 12 show near wall compaction tools. Target compaction profiles were consistently achieved, although some non-conformances occurred near the top and bottom of the fill profile due to relatively low confinement at shallow test depths and due to elevated fines content in some seabed zones.

5.4 IMPACT ROLLING

High energy impact rolling was employed in areas requiring shallow treatment such as upper compaction layers above high tide level (Figure 11). Compaction was undertaken using an eccentric roller, which generally used twenty to thirty passes to achieve the required compaction targets, the process being controlled using “real time” settlement monitoring.

During early compaction works, efficiency was observed to be low due to loose sand building up in front of rollers, which tended to cushion each drum blow. The solution was to install a crushed sandstone capping layer to improve the energy delivery to the subgrade.

Subsequent performance was satisfactory over about 3m depth (down to about low tide level), with some detectable densification occurring in some areas to about 5 m depth. Impact roller compaction was adversely affected by damping due to cohesive layers occurring at some locations near seabed level which absorbed impact energy and locally prevented effective compaction at depth.

5.5 TRENCH FOUNDATIONS AND MARINE VIBROCOMPACTION

Prior to backfilling of marine trenches below berth structures, basal trench rock fill was used to reduce the risks associated with contamination of sand fill by softened or remoulded clay at the base of the trench.

Marine compaction in sand trench backfill was undertaken using barge mounted vibratory equipment of the same type as used on land (Figure 12). Compaction was undertaken on a 3.6 m triangular grid, using 3 m to 4 m of sand overburden as confinement to achieve compaction at final stripping level. Stripping was carefully carried out over the wall foundation pad using a suction device on the screeding frame to minimise loosening and disturbance of the compacted sand. Above the compacted sand trench, a 1 m thick crushed igneous gravel

(20 to 80 mm) foundation pad was placed immediately below the wall structures using a custom built underwater rock screeding apparatus to tight tolerances under 20 m of water.



Figure 11: Trial Sheetpile compaction (left) and Dynamic Impact Rolling (right)

A review of marine CPTs undertaken in compacted trench material showed generally good compaction with only minor variability in material type, comprising thin cohesive zones occurring sporadically through the trench profiles. These were inferred to comprise re-worked overconsolidated peaty sub-units that were dredged from within the Unit 2 Sands. The discontinuous and thin nature of these very stiff clay materials (typically about 0.1m thick) implied that they occur as discrete "clay balls", similar to those observed scattered at ground surface, rather than continuous layers.

Other, less common features observed in some marine CPTs included low compaction profiles, inferred to be where CPT probes follow a preferential "chimney" of loose material at the precise centreline of a VC probe location and 'wavy' compaction profiles exhibiting cyclic high and low soil density in regular 1 m layers, inferred to be where CPT probes are located close to, but not within VC probe footprint locations.

Overall, the level of compaction required for target strength and stiffness was achieved, noting that the main impact of zones of reduced compaction had greater influence on the potential movement of the structures than their stability. A robust screening framework was established to check berth structure stability and movements including PLAXIS check analyses. Ultimately, these showed that future movements were compliant with the post-handover PSTR requirements.



Figure 12: Marine VC (left) and near-wall VC (right)

5.6 COMPACTION TESTING

To monitor reclamation fill quality and satisfy future performance, a comprehensive set of earthworks construction quality tests were undertaken. For deep fill, this included over 3,000 CPT profiles to enable verification against the PSTR acceptance criteria and designer-specified strength and stiffness requirements. A Geotechnical Construction Compliance Testing Plan (GCCTP) was prepared to specify the test types and test criteria and provided the principle link between PBE geotechnical design and construction activities.

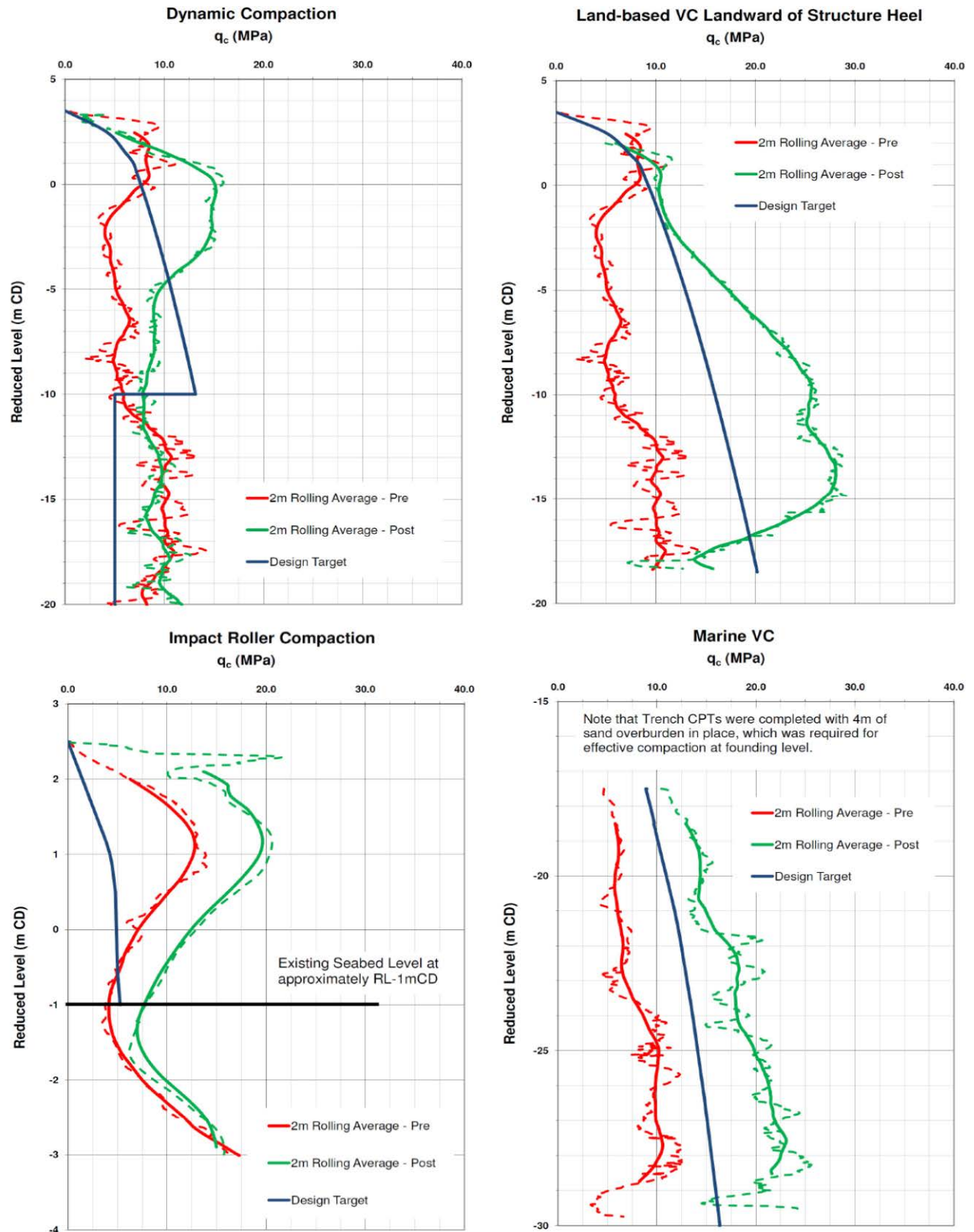


Figure 13: Target CPT profiles and Corresponding Fill Compaction Zones

As filling and early works got underway, progressive CPT testing was carried out at targeted locations to confirm that the thickness of unsuitable materials in seabed and fill was within allowable limits and to confirm the predictive model for consolidating seabed layers. Predictive CPT profiles were generated for six critical locations where pre-tender boreholes included *in situ* materials at the seabed that could produce non-compliant settlements or CPT profiles. Predictive CPT models were produced to show how known unsuitable materials

and fines would compress and strengthen under burial. These compared well with the progressive CPTs and hence validated the design parameters and predicted strength and density gain.

Example CPT profiles for fill 'before-and-after' compaction are shown on Figure 13 for each compaction method completed on the PBE project.

5.7 GEOTECHNICAL MONITORING

To provide confidence that the on-going structural behaviour was consistent with design predictions, monitoring of the reclamations and structures was carried out as an integral part of the construction process. A separate Geotechnical Instrumentation and Movement Monitoring Plan (GIMMP) was developed to address monitoring requirements and geotechnical performance of the proposed works.

At different construction and operational stages, predictions for soil and structural movements and earth pressures were generated at numerous key locations within the reclamations and along berth and revetment edge structures. Instrumentation included inclinometers, extensometers, survey pins, piezometers, earth pressure cells and vibration sensors. Trigger levels for movements and earth pressures were then developed using a 'traffic light system' to establish actions required if movements and/or earth pressures approached or became higher than the predicted or prescribed design values. To avoid missing critical feedback, a data management system was developed to handle information from the 1000 monitoring points on the site.

6 CONSTRUCTION OBSERVATIONS

During construction of PBE the design team had a strong site presence and had input throughout all stages of construction. This allowed the design assumptions to be compared with issues that arose during construction. In this section of the paper, some key construction-stage observations are described which may be of use to maritime designers.

6.1 EXPLORATORY LOCATION SPACING

When generating a geotechnical model for any job there will always remain uncertainty, regardless of how much information exists for the site. For PBE the existing site investigation information gave a reasonable coverage of the site area, nominally the dredging and reclamation area (approximately 200 Ha) were covered by 360 No boreholes, equivalent to a grid spacing of approximately 75 m. Whilst this was considered to be appropriate grid spacing for this project, some additional, targeted investigations were required to facilitate detailed design. For instance, a program of seabed probing was undertaken to better delineate cohesive materials which can significantly influence reclamation settlements.

6.2 COHESIVE MATERIALS AT / BELOW EXISTING SEABED

Material type was inferred from CPTs using a material index classification based on I_c values derived using Lunne *et al.*, 1997. Site-specific adjustments to these correlations were made by drilling boreholes adjacent to critical CPTs and undertaking grading tests. As results became available, an apparent reduction in the I_c index occurred between pre and post compaction CPT pairs at essentially the same location. This was unexpected, given that the grading of the material should not change significantly during VC. The observed over-prediction of I_c index in pre-compaction CPTs may be due to the sensitivity of the material index correlation in very loose sand at low stresses and/or remnant excess pore pressure in recently placed or compacted sand fill.

Higher fines contents were observed in down-slope areas or corners of temporary submerged bunds used to control reclamation zoning. Critical areas where more slimes occurred were investigated and borehole grading tests showed that the average fines content was less than 10%, although some isolated layers of thin, normally consolidated clay (slimes) were observed. VC is limited to soils having fines content under 12 to 15% and efficiency is known to reduce above a fines content of 10%. In the minority of cases where slimes-affected fill was observed in CPT profiles, approximately 5-10% of any given CPT profile contained sufficiently elevated fines content to reduce compaction efficiency.

On PBE, diver probing was used to target areas having been identified on the basis of pre-tender boreholes or seafloor bathymetry as having a greater risk of having unacceptably thick layers of cohesive sediments present at the existing seabed. Recovering such materials can be difficult, although piston sampling techniques were successfully used to sample and map the thickness of existing seabed sediments at PBE.

During early construction, additional investigation works identified the presence of a buried organic clay channel, which was only found after post filling CPTs were commenced. The diver probing and sampling works had targeted the area, but as the clay layer was located approximately 3 m below the existing seabed level the diver investigation did not encounter the layer. Only one pre-tender borehole had been located on the seabed

feature and the single CPT that had been scheduled in the area was not completed after the piezocone was damaged. The layer had potential to affect haul road and sensitive wall transporters and so both short and long term remediation was required.

The area was subsequently treated with wick drains and preloaded to achieve acceptable post-handover settlement performance. Predictive PLAXIS models were regularly calibrated and re-calibrated against monitoring equipment, comprising; settlement plates, vibrating wire piezometers, inclinometers. CPTs were also completed at different times, allowing repeated strength-gain observations to be made. The unbound haul road was resurfaced during these works.

6.3 FINES / CLAY BALLS IN RECLAMATION FILL

The dredging of the source material was controlled by the 3D Vulcan model, which included the presence of unsuitable materials based on pre-tender boreholes. However, as the source materials included unsuitable pockets or layers that sometimes occurred between discrete boreholes, it was probable that the dredging operation would excavate some unsuitable materials. On the PBE site this was evidenced by the presence of “clay balls” which were visible on the completed reclamation surface.

In assessing dredging and reclamation quality control processes there should be a distinction between localised / minor amounts of unsuitable material against large quantities of unsuitable material, i.e. a major clay layer which should be disposed of. The dredging process itself involves transporting the dredged material with a large quantity of water. By this process fines in the source material can separate from the sand fill and will either remain suspended in the water after placement, or may accumulate at the base of reclamation fill slopes as “slimes”. Dredging QA / QC processes need to be developed for this, although we note that this effect can help to produce a cleaner end product, than the original source material. The loss of fines by suspension also makes effective exclusion of dredging areas, via silt curtains, an important environmental factor to consider.

As fines may segregate or accumulate during the dredging and reclamation processes, steps should be taken to mitigate the risk of this occurring. Dredging QA processes form one part of this, for example, the dredging and reclamation plan for PBE included measures to identify changes in dredge material by occurrence of clay balls or large amounts of clay at the dredge outlet, colour changes or odour, together with comprehensive testing of materials both at the reclamation point and from within the dredge pumping system. During dredging and placement operations, a number of controls were applied as outlined below:

- A sample was taken of the reclamation material near discharge to visually assess the fines content. Samples were sieve tested every 2,250 m³ to assess the grading and photographs taken and
- To check if sediments were building up ahead of the advancing reclamation slope, regular underwater sampling and laboratory testing was undertaken. Sampling was carried out using either a multi-sampler, beaker sampler or Van Veen grab from a support vessel every 4,000 m³ of placed material.

Other considerations include producing working methodologies that minimise the risk of accumulation of fines in critical areas, for example close to berth structures, where generally performance criteria may be stricter in terms of differential settlement limits.

6.4 EARLY SETTLEMENT MONITORING DURING RECLAMATION FILLING

To validate design compressibility parameters it is beneficial to try to obtain early settlement data of the response to the reclamation filling. This was undertaken in shallow areas at PBE by the use of settlement plates, with steel rods fitted. During the reclamation process some of these monitoring points were lost or damaged before fill reached the final surface level. The construction team did attempt to keep these monitoring points operational, but with a large dredging / reclamation operation there are many things that can destroy such points and some redundancy or provision is needed to account for damage. An example of settlement plate rod monitoring from PBE is included in Figure 14.

Consideration was given to alternative other progressive filling settlement measurement systems in deep areas including installation of large metal plates on the existing seabed prior to filling commencing with the aim of drilling and casing down to these plates after filling. Another method which was trialled unsuccessfully involved attaching vibrating wire piezometers to seabed plates placed prior to filling and monitoring the change in pore pressure response due to settlement. It was found that the tidal response was inconsistent and muted and any settlement related trend in the data could not be reliably interpreted.

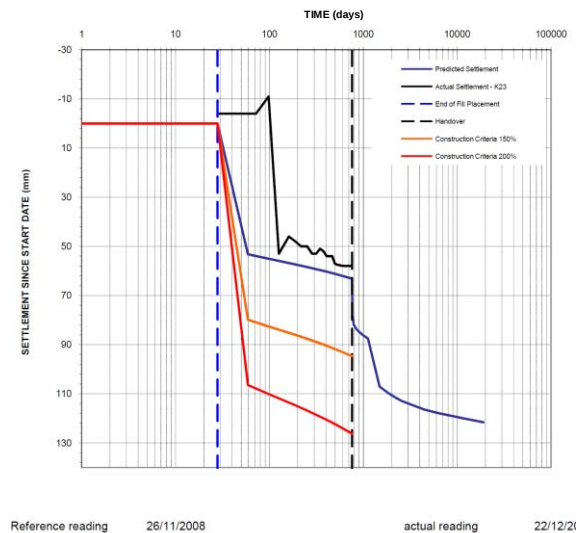


Figure 14: Settlement Plate Monitoring Results.

6.5 TEMPORARY RECLAMATION STABILITY

Due to the large pumping rates achievable with modern cutter suction dredges (up to 100,000 m³ per day on PBE) it is important to consider the method and working sequence of the reclamation process. Simplistically the dredge is a large capacity pump, which will pump water (and some sand) into the new reclamation area. It is important to consider the segregation and sedimentation processes during reclamation, which can result in accretion of “slimes” layers that affect stability of rapidly placed sand fill.

The importance of balancing dredging and reclamation production rate, while minimising risk of instability is known and has been discussed previously, in relation to other large scale ports projects at Malaga and Barcelona (Del Campo *et al.*, 2011). The lesson here is that even in granular fill materials, very rapid fill placement rates can result in slope failures as high pore pressures do not have time to dissipate. As a result of site observations and back-analyses undertaken during PBE, project controls were placed on the reclamation process, including a maximum height increase per hour and a maximum lateral beach reclamation rate.

6.6 STRENGTH AND STIFFNESS OF COMPACTED FILL

As filling and early works got underway, progressive CPT testing was carried out at targeted locations to confirm that the thickness of unsuitable materials in seabed and fill was within allowable limits and to confirm the predictive model for compacted fill and consolidating seabed sediments.

Based on the CPT strength correlations (Robertson *et al.*, 1983) it was shown that achieving friction angle $\geq 37^\circ$ can reasonably be expected throughout the full depth of compacted reclamation fill. The PSTR limited design values to 35° in non-trench fill. Site specific CPT correlations were substantiated against results from dilatometer and shear box test during field trials and production testing.

Design studies for marine trenches showed that the compaction required to achieve the target trench stiffness ($E_s' = 100$ MPa) was more onerous than that required to achieve the target trench friction angle (37°). In practice the design was more sensitive to stiffness variations of the trench foundation materials than to strength variations. There were rarely any continuous weak layers and there was often significant capacity in strength design. It was also shown that using peak strength values was compatible with the strain field and that only a proportion of the available shear strength was mobilised in backfill under operational stresses.

For the foundation trench, an additional set of screening rules, re-testing protocols, check-analysis procedures and acceptance criteria were developed and substantiated through extensive modelling.

In shallow fill, trials were undertaken to explore whether site specific CPT density correlations could be used in lieu of field density tests. This method would have reduced disturbance and the need to re-compact pitted areas subject to conventional earthworks testing. However, this approach was subsequently not adopted for compliance testing due to statistical variability. Sensitivity tests were also undertaken to assess the influence of standard vs. modified compaction, surcharge weights and compaction methodology on CBR test results.

The use of the vibrated-in, perforated sheetpile in lieu of VC in confined situations was adopted after a trial in completely uncompacted fill (average $D_r = 10$ to 20%) indicated that improvements within a metre or so of the

sheetpile could be achieved (up to $D_r = 50\%$). Improvements due to the perforated sheetpile were reduced, or even neutral, in zones of fill which had been previously improved by adjacent ground treatment. The technique was adopted only in confined situations near wall structures in very loose fill, where a low level of improvement was required to satisfy liquefaction and settlement targets.

7 CONCLUSIONS

This paper provides an insight into the challenging geotechnical aspects of the dredging and reclamation design and construction for the Port Botany Expansion (PBE) project. Key outcomes are as follows:

1. Substantial dredging and backfilling works were required for reclamations and for deep foundation trenches to RL -30 m CD to remove unsuitable clay material below the wharf structures. Filling above the natural seabed included backfilling of former borrow pits up to about 30 m depth of new sand fill over a total area of 63 Ha.
2. 3D mapping was undertaken to create a spatial model of unsuitable reclamation source materials. This model was used in real-time by dredge operators to manage dredging and reclamation processes so as to reduce the risk of placing unsuitable materials in the reclamation.
3. Extensive trials were carried out to quantify effects of vibrocompaction (VC) and dynamic compaction (DC) and to refine compaction targets. Compaction trials near structures were focussed on achieving compaction without adversely affect wall structures. This led to tailoring of VC techniques (reduced power, higher lift rates) and introduction of innovative vibrating sheetpile compaction methods. Iterative soil-structure design was also undertaken to achieve compatibility between geotechnical and structural models and to maximise the efficiency of wall design (Davies, 2010).
4. Construction stage observations included recognising the importance of understanding the thickness and distribution of seabed sediments, controls on reclamation filling/stockpile placement rates, putting in earthworks testing screening rules around the occurrence and significance of fines and clay balls and applying innovative techniques, to early settlement monitoring during filling.
5. Development of a robust earthworks verification process included confirmation of design assumptions through progressive testing and over 2,000 'as-built' settlement checks, which resulted in generally good agreement between monitored and predicted settlements. These provided confidence that design criteria and long term performance would be achieved.

Reclamation and construction commenced onsite in August 2008; construction was formally completed in June 2011. The first berths are anticipated to commence stevedoring operations early in 2012.

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9 REFERENCES

- Albani, A.D. and Rickwood, P.C. (1998). "The Botany Basin: its bedrock morphology and recent geological history". In McNally, G.H and Jancowski, J, (Ed), *Collected Case Studies in Engineering Geology, Hydrogeology and Environmental Geology*, Geological Society of Australia, 190-196.
- Baldi, G. Belotti, R. Ghionna, V. Jamiolkowski, M. and Pasqualini, E. (1986). "Interpretation of CPT's and CPTU's, 2nd part: Drained Penetration of Sands". Fourth International Geotechnical Seminar, Field Instrumentation and In-Situ Measurements, Nanyang Technological Institute, Singapore, 143-156.
- Davies, P.R. and McIlquham, J.D. (2011). "Geotechnical Design for the Port Botany Expansion Project, Sydney". *Geotechnical Engineering*, 164, GE3, 149-167, doi: 10.1680/geng.10.00052.
- Davies, P and Masters, P (2010). "Soil-Structure interaction considerations for Retaining Walls on the Port Botany Expansion", *Soil-Structure interaction considerations for Retaining Walls on the Port Botany Expansion; The 5th Civil Engineering Conference in the Asian Region and Australasian Structural Engineering Conference 2010*.
- Del Campo, J.M. and Negro, V. (2011). "Failures of Harbour Walls at Malaga and Barcelona", *Bulletin of Engineering Geology and the Environment*, 70, 1-6.
- Fell, Robin. (2006). Private Communication "Review of fissured clay strength, Port Botany Expansion", New South Global Consulting.

- Greenwood, D.A. and Kirsch, K. (1984). "Specialist ground treatment by vibratory and dynamic methods". Piling and Ground Treatment, Thomas Telford Ltd. London, 17-45.
- Look, B. (2007). "Handbook of Geotechnical Investigation and Design Tables", Balkema, Leiden
- Lunne, T. Robertson, P.K. Powell, J.J.M. (1997) "Cone Penetration Testing in Geotechnical Practice", Blackie Academic & Professional, London.
- Massarch, K.R. and Fellenius, B.H.(2002). "Vibratory compaction of coarse-grained soils". Canadian Geotechnical Journal, 39, 695-709.
- Moss-Morris, A. and Hodge, N.M. (1981). "Design and Construction of Quay Walls for a Container Terminal Founded on Fissured Clays". 25th International Conference of Navigation, Edinburgh, 4, 557-568.
- Robertson, P.K, and Campanella, R.G. (1983). "Interpretation of Cone Penetration Tests (Part I)". Canadian Geotechnical Journal, 20, No. 4.
- Thorne, C.P. (1984). "Strength assessment and stability analyses for fissured clays". Geotechnique, 34, No. 3, 305-322.
- Thorne, C.P. (1985). "Deep marine fissured clays", Engineering Geology of the Sydney Region, Edited by PJN Pells, Balkema, 63-79.
- Terzaghi, K. Peck, R. and Mesri, G. (1996). "Soil Mechanics in Engineering Practice", Third Edition
- Thorne C.P. (1997). "Expressions Linking Drained and Undrained Parameters". Australian Geomechanics, December 1997, 38-47.
- Youd *et al.* (2001). "Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils" J. Geotechnical and Geoenvironmental Engineering., ASCE, 127(10), 817-833.