GEOTECHNICAL OFFSHORE SITE INVESTIGATION AND RECLAMATION DESIGN AT PORT KEMBLA

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ABSTRACT

Port Kembla is an active seaport situated approximately 90 kilometres south of Sydney. The majority of the current port activities are focussed in the Inner Harbour area of the Port. However, as this is reaching capacity, the port authority is turning its focus onto the planning of the development of the mostly undeveloped Outer Harbour. Stage 1 and 1A (Phase 1) development of the Port Kembla Port Corporation (PKPC) Outer Harbour master plan would create one additional bulk cargo berth and approximately 10 hectares of reclaimed land. In February of 2010, PKPC awarded Snowy Mountains Engineering Corporation (SMEC) the contract to undertake Phase 1 detailed geotechnical site investigation works, and the associated reclamation and berth designs.

The Outer Harbour has been subjected to deposition of materials from five previous disposal campaigns, whereby dredged sediment from the Inner Harbour was disposed of within the Outer Harbour. Underwater containment bunds of uncrushed blast furnace slag were constructed for one of the disposal campaigns, and the contained areas were filled with spoil that typically consists of unconsolidated, very soft, compressible clay. This is consistent with geotechnical interpretation based on site investigation data which found that unconsolidated dredged fill, up to eight metres thick, underlies the majority of the Stage 1A and 1B development, generally thickening towards the east and southeast.

Phase 1 geotechnical design for the Outer Harbour development includes the design of containment bunds and land reclamation design associated with subsequent infilling with appropriate select fill material. Various design options were considered for both the bund and reclamation construction. Instrumentation and monitoring were proposed as part of the detailed design in order to confirm design assumptions and monitor the performance of the reclamation.

As the detailed design progressed, PKPC made the decision that the conforming design, which satisfied the original scope of works and settlement criteria, would not be constructed. Their preference instead was for a reclamation design that eliminated the need for removal of any of the underlying dredged spoil and did not utilise ground treatments other than passive preloading and surcharging techniques. The developed design has since been issued for tender and a constructor selected with construction about to commence.

1 INTRODUCTION

Port Kembla is an active seaport situated on the north side of Red Point, approximately 90 km south of Sydney. As a result of the State Government's New South Wales Ports Growth Plan, a proportion of the shipping and cargo previously handled by Port Jackson has been relocated to Port Kembla. This development, combined with an on-going shortage of land within the Inner Harbour is understood to be the key incentive for the redevelopment of the Outer Harbour. In 2008 a major review of the development options for the Outer Harbour was performed, which considered contemporary commercial and trade related realities, and led to the proposed development being altered significantly from that of the previous development strategy. Prior to this, dredged spoil was deposited in the Outer Harbour within what was the footprint of the future reclamation. These activities resulted in a minimum of 460,000 m^3 of both imported slag and dredged spoil from the Inner Harbour being deposited in the Outer Harbour, over five disposal campaigns.

The PKPC Outer Harbour master plan proposes the reclamation of least 42 hectares of additional port area over two stages of reclamation works, and the addition of 1770 m of new berth length. Stage 1 and 1A of the Outer Harbour development would create one additional bulk cargo berth and approximately 10 hectares of reclaimed land.

The overall Outer Harbour development has been divided into the following stages:

- Stage 1 and 1A - to create one additional bulk cargo berth and approximately 10 hectares of reclaimed land together with road connections (Phase 1).
- Stage 1B - the extension of the reclamation to the south, and eventually to the north, to incorporate the existing Port Kembla Gateway facility. This would then allow the extension of the bulk berth north and south to form a three berth facility (Phase 1).

- Stage 2A and 2B - to add a two berth container terminal and associated rail infrastructure (Phase 2).
- Stage 3 - to add two more container berths and associated reclamation, together with further development of associated rail and road infrastructure (Phase 2).

The PKPC Outer Harbour master plan showing these various stages are shown in Figure 1.

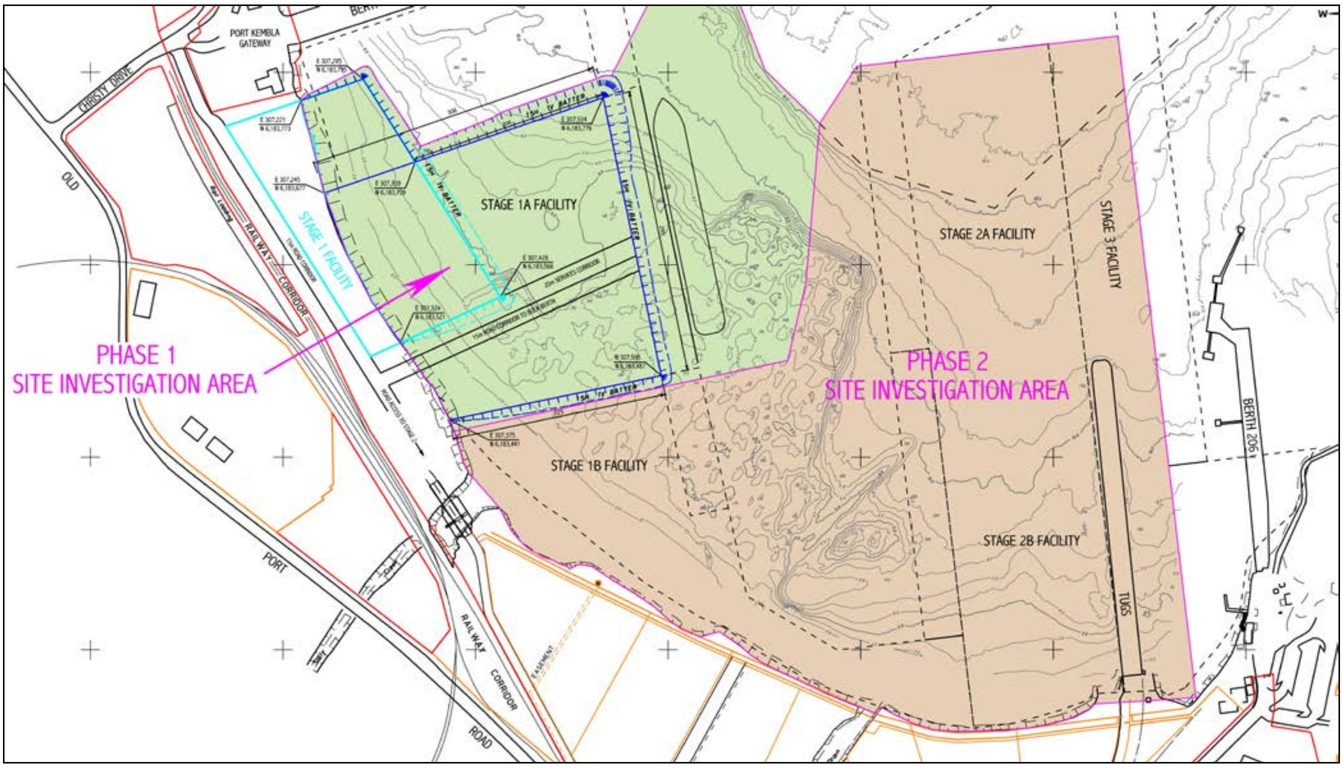

Figure 1: Port Kembla Port Corporation Outer Harbour master plan

In February 2010, PKPC awarded SMEC the contract to undertake both Phase 1 and Phase 2 detailed geotechnical site investigation works, the associated detailed design of reclamation Stages 1 and 1A, and concept design options for Stage 1A berth.

This paper presents the findings of the offshore geotechnical site investigation and the design methodology and analysis results for the associated reclamation design of Stage 1 and 1A.

2 REGIONAL GEOLOGY

The site is situated near the southern margin of the Sydney Basin. The 1:100,000 scale geological map of Wollongong-Port Hacking indicates that the site is directly underlain by Quaternary quartz and lithic fluvial sand, silt and clay. Immediately to the west of the site the Dapto Latite Member is indicated, comprising a melanocratic coarse grained and porphyritic latite. The Budgong Sandstone Formation is indicated approximately 3 km north-west of the site.

Bedrock at the site comprises the Budgong Sandstone Formation, derived from the lithification of a Permian marine deltaic sand. The Budgong Sandstone is the uppermost unit of the Shoalhaven Group, and outcrops along the coastal plain of Wollongong. The contact with the overlying Illawarra Coal Measures is conformable. It contains minor, interbedded, thin laminated siltstone, thin lenticular conglomerates and five tabular latite bodies. The sandstone is lithic to felspathic litharenite, and comprises mainly volcanic rock fragments and feldspar clasts (Bowman, 1971). Most of the Budgong Sandstone is planar bedded in laterally discontinuous units varying in thickness from several centimetres to 3 m. Bioturbation completely obliterates most bedding structures (Sherwin & Holmes, 1982).

The Dapto Latite Member is the most mafic of all the flows in the Gerringong Volcanic Facies. Petrologically, the Dapto Latite Member is basalt which varies in texture from aphanitic to porphyritic with a crystalline groundmass (Bowman, 1971). The Dapto Latite Member exhibits columnar jointing in some areas; it also contains partially or completely filled vesicles apparently elongated in stringers parallel to the flow direction. The Dapto Latite probably flowed into shallow water offshore, intruding soft sediments in part. Where the flows are in contact with the Budgong Sandstone, significant changes to weathering effects have not been reported.

Overlying the old eroded land surface are unconsolidated sediments. The clay, sand and gravel basal units are believed to be Quaternary alluvium, overlain by unconsolidated silts and clays, interpreted as modern marine and estuarine sediments.

3 GEOTECHNICAL OFFSHORE SITE INVESTIGATION

As part of the development planning of the Outer Harbour, numerous geotechnical investigations had been undertaken. These include the drilling of 40 boreholes and vibrocores between 1977 and 2008.

A total of eighteen (18) boreholes (BHS101 to BHS118) were completed as part of the Phase 1 investigations, using a combination of washbore and bedrock core drilling methods for soil and rock respectively. Investigation locations were targeted to provide geotechnical data on the proposed areas of reclamation, dredging and construction. Borehole locations, the footprint of existing together with nomenclature of bunds and reclamation areas are presented in Figure 2. For clarity and ease of reference, the Stage 1 and 1A containment bunds and reclamation have been divided into discrete sections and areas, and are also shown in Figure 2, as summarised below:

- Stage 1 containment bunds: B1 to B4
- Stage 1 reclamation areas: “General Area” and “Service and Road Corridor”
- Stage 1A containment bunds: B5 to B10
- Stage 1A reclamation areas: Areas R1 to R4

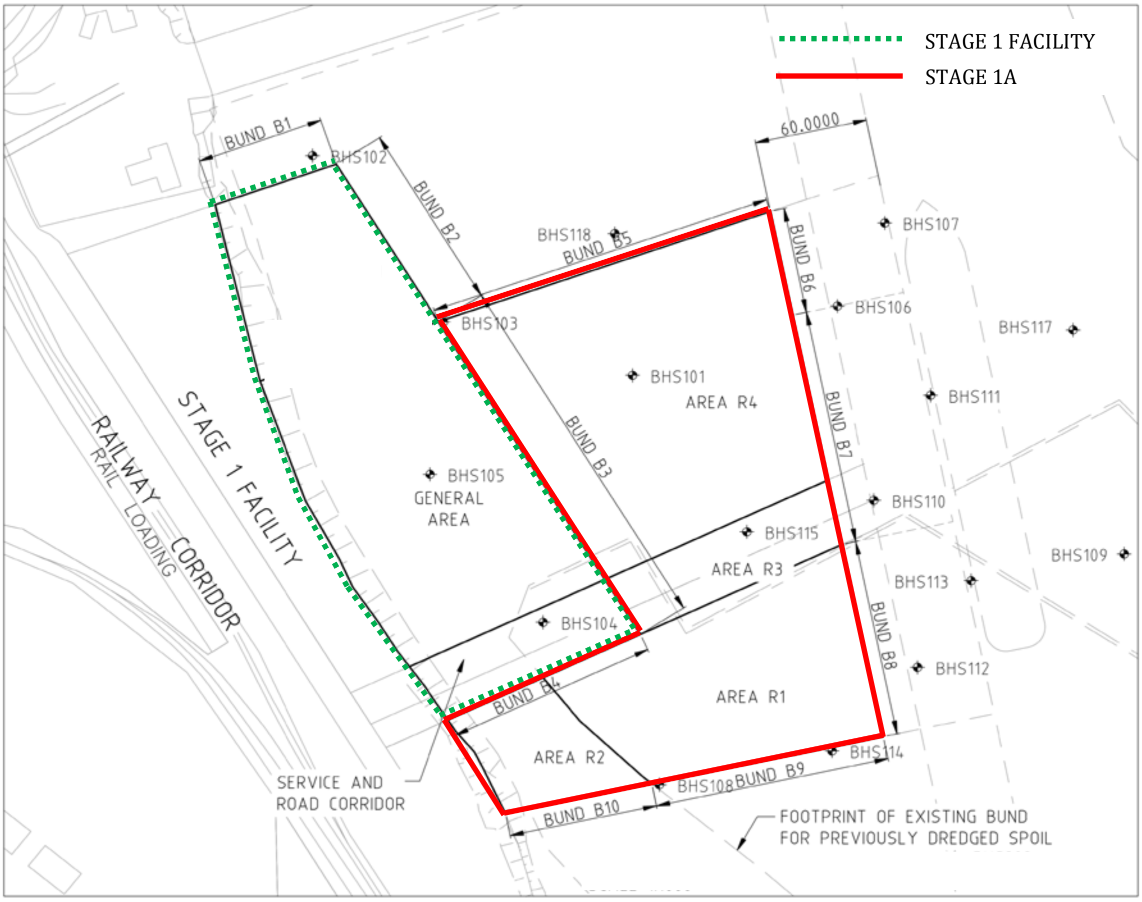

Figure 2: Geotechnical investigation location plan for Phase 1 investigations.

The recovery of undisturbed samples of the fill that were of sufficient size to permit triaxial and/or oedometer tests proved unsuccessful. This was due to a combination of particularly low shear strengths and relative densities allowing samples to ‘flow’ out of the U50/U63 tubes, and the presence of obstructions (such as slag) within the fill, which inhibited sample collection. The latter of these occurred twice in fill soils, resulting in damage to the sampling tube. Of the 58 SPT samples recovered from the investigation, five samples of fill were lost due to poor consolidation and three samples lost due to obstructions (metal wire/slag).

4 SITE GEOTECHNICAL CONDITIONS AND GEOTECHNICAL INTERPRETATION

4.1 GEOTECHNICAL CONDITIONS IN STAGE 1 FACILITY

The general area of the Stage 1 facility lies outside of the existing footprint of the perimeter bund for spoil disposal, and was subsequently not subject to filling. Marine deposits (approximately 1.0 m thick, increasing to up to 3 m thick towards the north) were encountered and described as firm to stiff silty clay. Very loose to dense alluvial sands were also encountered throughout the area directly overlying the residual soils, with a typical thickness of between 1 m and 2 m.

Residual soils directly overlie the bedrock across the Stage 1 facility, and were typically described as a stiff to hard clay. The residual soils thicken in the southern area of Stage 1, where up to 3 m was encountered.

The 20 m wide service corridor and 15 m wide road corridor at the southern end of the facility encroach into the existing spoil disposal cell. Up to 1.5 m of dredged spoil is anticipated to be present within this area.

4.2 GEOTECHNICAL CONDITIONS IN STAGE 1A FACILITY

The underlying geotechnical conditions in the Stage 1A facility are significantly more complex than the Stage 1 area.

Unconsolidated dredged fill underlie the majority of the Stage 1A area and generally thicken towards the east and south-east of the area. The thickest sequences of dredged fill were encountered below the proposed eastern batter (Bunds B6 to B8), with no fill encountered at the northernmost edge of the proposed batter (Bund B5), increasing to 7.5 m of fill at the southernmost edge (Bunds B8 and B9). Fill was also absent from the far south-west corner (Bund B10). The nature of the fill is noted to alter towards the southern part of the area, as it changes from a granular material to a cohesive material.

Unconsolidated marine soils were encountered sporadically throughout the area, with thin (0.3 m to 0.5 m) laterally discontinuous occurrences. Alluvial soils were encountered throughout the area, with typical thicknesses of 1 m to 2 m. Thicker sequences of alluvium (up to 4.7 m) were noted to occur beneath the proposed southern batter in the vicinity of Salty Creek.

Residual soil comprises the base of the soil units and is encountered directly overlying bedrock. The thickness of the residual soils varies between 0.4 m and 3.0 m and is typically between 1 m to 2 m. The residual soils were effectively the only consolidated soil horizon in the Stage 1A area and were typically stiff to hard clays.

The depth below existing seabed level of the residual soils is noted to increase in a southerly direction. Towards the central and southern part of the Stage 1A area the residual soils were encountered at increasingly greater depths as the thickness of the overlying alluvial and dredged fill increases.

4.3 GEOTECHNICAL UNITISATION

Soil and rock units encountered concur well with published geological data, with six separate units being encountered during the investigation. The units encountered are presented in Table 1.

Table 1: Geotechnical unit descriptions

Unit	Description	Thickness (m)	Typical composition
1a 1b	Fill (cohesive) Fill (granular)	0.0 to 8.2	Poorly consolidated clay and sand fill mixed with variable minor fractions. Clay fill is very soft to firm, plasticity is variable. Sand fill is very loose to medium dense. Man made artefacts include charcoal, ash, slag gravels, possible coal-wash and metal wire.
2	Marine and estuarine sediment	0.0 to 1.0	Very soft to soft silty clay of variable plasticity. Only encountered as thin layers in BHS102, BHS106 and BHS111.
3a	Quaternary alluvium (cohesive)	0.0 to 3.4	Soft to firm clays were encountered within this unit. Shell fragments noted throughout.
3b	Quaternary alluvium (granular)	0.0 to 2.6	Typically very loose to medium dense sand with variable minor fractions. Shell fragments noted throughout.

Unit	Description	Thickness (m)	Typical composition
4	Residual Soil	0.4 to 3.7	Typically very stiff to hard clays of low plasticity, with gravels of latite, sandstone and siltstone noted throughout. Sand and gravel units also encountered.
5	Dapto Latite Member	0.0 to > 1.0	Extremely weathered to highly weathered fine to coarse grained latite with medium to coarse gravels.
6	Budgong Sandstone Formation	>12.6	Extremely weathered becoming fresh sandstone and siltstone. Defect spacing and rock strength noted to increase markedly with depth.

4.4 GEOTECHNICAL DESIGN PARAMETERS

A suite of geotechnical design parameters was developed for the design of the reclamation. These parameters were derived from project specific *in situ* and laboratory tests where available, and are considered to be representative of the properties of the material in its current condition. The geotechnical design parameters developed include:

- Bulk unit weight γ (kN/m^3)
- Undrained shear strength c_u (kPa)
- Effective cohesion c' (kPa) and effective friction angle ϕ' (degrees)
- Modified compression index $C_{c\varepsilon}$
- Modified recompression index $C_{r\varepsilon}$
- Modified secondary compression index $C_{\alpha\varepsilon}$
- Coefficient of consolidation C_v (m^2/year)
- Drained elastic modulus E' (MPa)

A summary of the adopted geotechnical design parameters is given in Table 2.

Table 2: Geotechnical design parameters

Unit	Description	γ (kN/m^3)	c_u (kPa)	c' (kPa)	ϕ' (deg)	$C_{c\varepsilon}$	$C_{r\varepsilon}$	$C_{\alpha\varepsilon}$	C_v (m^2/yr)	E' (MPa)
1a	Fill (cohesive)	16	7.5	0	25	0.250	0.025	0.0125	10	-
1b	Fill (granular)	16	-	0	30	-	-	-	-	7
2 / 3a	Marine and estuarine Sediment / Quaternary alluvium (cohesive)	17	10	0	22	0.250	0.025	0.0125	5	-
3b	Quaternary alluvium (granular)	19	-	0	34	-	-	-	-	40
4	Residual Soil	19	150	5	28	0.100	0.010	0	50	-

5 DESIGN OPTIONS

As part of the design development, a number of different schemes were considered for the design of both the containment bund and the reclamation. The selection of the adopted solution and extent of ground improvement (if required) is highly dependent on the following factors:

- Capital cost
- Whole-of-life budgetary constraints
- Total and differential settlement criteria for the proposed use of the reclaimed land
- Construction program

5.1 DESIGN OPTIONS FOR CONTAINMENT BUNDS

The containment bunds could be placed directly on the seabed at locations where the geotechnical conditions are favourable, i.e. with little or no dredged fill and/or soft marine or alluvial soils. This applies to Bunds B1 to B3 of Stage 1, and Bunds B5 and B10 of Stage 1A.

At other locations, bund construction directly over dredged fill or unconsolidated soils would increase the risks of slope instability, thereby introducing an unacceptable element of risk to site operations in the short term, and in the long term over and beyond the project duration. This is particularly applicable for Bunds B6 to B8 of Stage 1A, where dredging would be undertaken in front of the bund for the Stage 1A berthing box, to RL-16.5m (PKD) well below the soil deposits and into the underlying rock mass.

Options that were considered to minimise the risk of slope instability include:

- Stabilising berms with/without high strength geotextile. This is applicable for Bunds B4 and B9/B10 that would be buried by future reclamations, and do not warrant complex or expensive treatment options.
- Dredging of soft sediment under the foundation of the bund. Subsequent disposal would be required prior to placement of bund materials. This is applicable for Bunds B6 to B8.
- Ground improvement of the soft materials *in situ*, with various ground improvement techniques prior to bund construction. This is applicable for Bunds B6 to B8.

Preliminary slope stability analyses were undertaken for Bunds B6 to B8 for three options - dredged bund foundation, ground treatment with stone columns and ground treatment with concrete injected columns. Comparative budget cost estimates were developed and it was found that ground treatment options (either stone columns or concrete injected columns) would cost approximately $10 million more than a dredged bund foundation option.

Moreover, the use of ground treatment to improve stability would require strict quality control during construction and adequate inspection and field testing to ensure that the design assumptions are met on site. This presented as an additional risk element to the design, is labour and time intensive, and any non-conformance would require additional design and remedial measures to be implemented during the construction phase.

Consequently, the detailed design for the containment bund involved:

- No treatment for Bunds B1 to B3 and B5.
- Use of stabilising berms and high strength geotextile (where required) for Bunds B4, B9 and B10.
- Dredging directly adjacent to the bund toe foundation along the eastern arm of Stage 1A (Bunds B6 to B8).

Removing dredged fill and soft sediments is a relatively lower risk option, as it does not rely on strict quality control during construction to ensure the installed ground improvement conform to design assumptions. To contain the dredged spoil, an additional containment bund would have to be constructed within the footprint of the future Stage 2A and 2B facility to contain the disposed material.

5.2 DESIGN OPTIONS FOR RECLAMATION

Very soft to firm cohesive dredged fill and normally consolidated soft soils underlie the Stage 1A area south of the service corridor. Excessive consolidation settlement would occur if the reclamation fill and long-term design load are applied directly on these soft materials.

Consolidation settlement is the vertical displacement of the surface corresponding to the volume change due to the discharge of excess pore pressure set up by the increase in overburden load. In this instance, the overburden load equals the loading imposed by reclamation fill and long-term design load. The consolidation process continues until all the excess pore water pressure has completely dissipated.

Constructing buildings and infrastructure on under-consolidated ground may adversely impact their operation and performance, as excessive differential settlement may result in damage. Various ground improvement options have been considered to limit the post construction settlement. The possible options that could be adopted for the soft foundation materials include:

- Removal and replacement with reclamation fill.
- Preloading or surcharging to improve the *in situ* ground after the reclamation. In this option, prefabricated vertical drains (PVD) can be installed into the soft materials to accelerate the discharge of excess pore pressure, if required.
- Installation of stone columns prior to the reclamation, followed by preloading and surcharging.
- Installation of rigid inclusions e.g. concrete injected columns after the reclamation has been completed to above the tidal zone (i.e. RL +2.2 m)

The southern area of Stage 1A is underlain by soft marine sediments and cohesive dredged spoil deposited during past dredging campaigns. Hence, the reclamation may be prone to bearing capacity failure and excessive settlement. The prediction of post construction settlement for sites underlain by deep soft soil is associated with considerable uncertainties. Uncertainties in soil properties, including creep behaviour, and use of different design methods would alter the results.

The risk of the actual settlement exceeding the design value would be increased for non-rigid ground treatment options such as preloading, and reduced for structural support treatment options such as rigid inclusion techniques (ie. CICs).

A qualitative risk appraisal of the ground treatment methods is presented in Table 3.

Table 3 – Risk appraisal of the proposed ground treatment for the reclamation

	Remove and Replace	**Surcharge and Preload**	**Stone Column with Surcharge and Preload**	**Concrete Injected Columns (CICs)**
Containment Bund failure during/post construction	Very Low	Medium / High	Low / Medium	Low
Required settlement period significantly longer than predicted	Very Low	Medium	Low	Very Low
Post construction settlement magnitude significantly larger than predicted	Very Low	Medium	Low	Very Low

Preliminary settlement analyses were undertaken for the last three options and comparative budget cost estimates were developed. The substantial volume of spoil generated by removal and replacement, the tight construction program and high costs for the stone column solution preclude these options from being adopted.

The detailed design hence included surcharge and preload for Areas R1 to R3, and the use of concrete injected columns for Area R4 where the thickest sequences of existing dredged spoil (Unit 1b) and marine/estuarine sediments and soft alluvial soils (Unit 2/3a) are present.

6 CONTAINMENT BUND AND RECLAMATION DESIGN

6.1 KEY DESIGN CRITERIA

6.1.2 Stability criteria

The following stability design criteria were adopted in the detailed design of the containment bunds. Two separate criteria were developed for permanent and temporary bunds.

Bunds B1, B2, B5 to B10 are considered permanent. They are exposed for an extended period of time before the reclamation is extended to the north (Bunds B1, B2 and B5) and to the south (Bunds B9 and B10) for the Stage 1B Facility, or before the wharf structure is constructed in front of Bunds B6 to B8. The minimum factors of safety adopted for design are summarised in Table 4 below.

Table 4: Summary of stability design criteria.

Analysis case	**Permanent bunds (B1, B2, B5 to B10)**	**Temporary bunds (B3, B4)**
Short term	1.30	1.20
Long term	1.50	1.30
Seismic	1.10	1.10

6.1.2 Settlement criteria

Taking into consideration the intended future land usage, the design settlement criteria have been established for different areas, as shown in Table 5.

Table 5: Summary of settlement design criteria.

Stage	Area	Loading (kPa)	Total Post Construction Settlement Criteria
Stage 1	General area	20	50 mm PCS in 10 years
Stage 1	Service and road corridor	20	50 mm PCS in 10 years
Stage 1A	R1	50	50 mm PCS in 10 years
Stage 1A	R2	50	200 mm PCS in 10 years
Stage 1A	R3	20	50 mm PCS in 10 years
Stage 1A	R4	50	50 mm PCS in 10 years

6.2 DESIGN METHODOLOGY

The design of the reclamation was undertaken by considering the following:

- Global stability of containment bund. The analysis determined the slope stability of the reclamation and containment bund under short and long term loading, as well as during seismic events. The analysis was undertaken using the limit equilibrium software SLOPE/W, for both circular and non circular slip surfaces.
- Assessment of primary and post construction settlement of reclamation. The analysis has taken into account the preload and surcharge requirements, or the arrangement of ground treatment to satisfy the design criteria, in each reclamation area.
  The primary settlement and degree of consolidation was determined using the finite element program PLAXIS for the construction duration specified. When soft soil has been surcharged, the creep strain rate would reduce depending on the over-consolidation ratio achieved by the surcharge process. From the PLAXIS model, the degree of consolidation at surcharge removal was used to estimate the creep strain rate reduction (C_α'/C_α), which was then used to estimate the creep settlement, based on the method suggested by Stewart *et al*. (1994).
- Assessment of volumes of dredging, slag (a co-product of the iron making process) and interburden rock (latite breccia available from local quarries) required for bund construction and volumes of slag required for reclamation.

6.3 DESIGN SUMMARY

6.3.1 Stage 1

The geotechnical conditions within the footprint of the Stage 1 facility are relatively favourable and no foundation treatment is required for the construction of the bund. The northern and northeastern arms (Bunds B1 and B2) are “permanent” and will be constructed of interburden rock as Stage 1B would only be extended to the north after a minimum of 12 years. The southeastern (Bund B3) and southern (Bund B4) arms of the bund are only temporary and will be constructed of slag as they would be buried by the reclamation for Stage 1A, planned to be undertaken within the next two years.

No foundation treatment is required for the general area of the reclamation. However, the service and road corridor straddles the original footprint of the perimeter bund that contains the previously dredged spoil. To minimise any total and differential settlement, the foundation treatment consists of 2.3 m of surcharge (equivalent to 36 kPa) above RL +4.0 m for 3 months.

6.3.2 Stage 1A

The geotechnical conditions under the northern arm (Bund B5) are relatively favourable and no foundation treatment is required for the construction of these bunds. The eastern arm (Bunds B6, B7 and B8) is the most critical section, as dredging would be undertaken in front of it for the Stage 1A berthing box down to RL -16.5 m. A dredged bund foundation is adopted for the entire length. The temporary trench would be filled with slag which would form the foundation of the bund up to RL -4.0 m. The bund would then be constructed of interburden rock directly on top of this slag foundation up to RL +2.2 m. For the southern arm (Bunds B9 and B10) the use of high strength geotextiles and stabilising berms are required to ensure slope stability.

For settlement control, the following are adopted for each reclamation area:

- Area R1
  This area is underlain by up to 7.5 m of dredged deposits and, in order to meet the settlement criteria, the foundation treatment for Area R1 consists of 0.45 m diameter CICs at 1.2 m centre to centre spacing in a triangular pattern.
- Area R2
  The foundation treatment for Area R2 consists of 5 m of surcharge (equivalent to 80 kPa) above RL +4.0 m for 3 months.
- Area R3
  The foundation material for Area R3 consists of with 2.3 m of surcharge (equivalent to 36 kPa) above RL +4.0 m for 3 months.
- Area R4
  The foundation treatment for Area R4 consists of 5 m of surcharge (equivalent to 80 kPa) above RL +4.0 m for 3 months.

Perspective view showing idealised profiles of the design, including the containment bunds and dredging for the bund foundation is shown in Figure 3 below.

This image was extracted from the three dimensional model developed for the reclamation which was utilised to develop the construction staging approach and materials volumes estimation required for the accurate pricing of the proposed works.

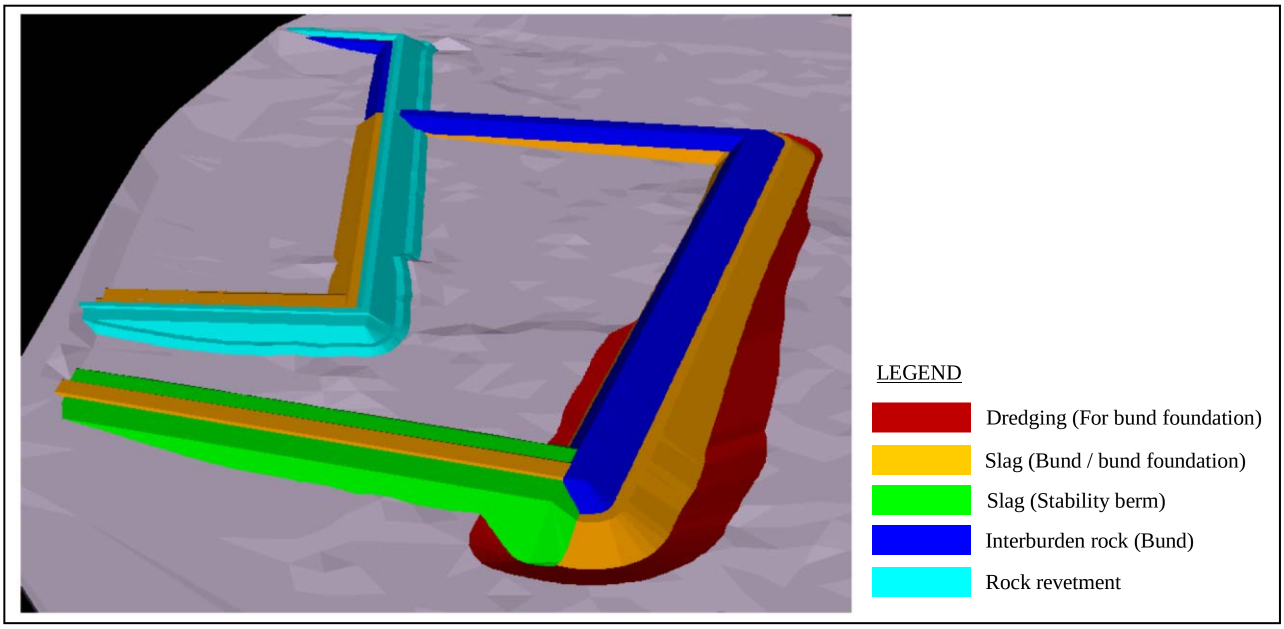

Figure 3: Perspective view showing completed Stage 1 and 1A bunds

7 INSTRUMENTATION AND MONITORING

Geotechnical instrumentation on the bund and reclamation area are required during and after construction in order to provide data that would enable:

- Confirmation of design assumptions e.g. the *in situ* shear strength, the compressibility and the rate of consolidation. Due to the formation process of the cohesive dredged fill, it is expected that the properties would vary significantly across the site.
- Decision for preload/surcharge removal to be made by the Principal based on the performance during preloading. There are opportunities for early preload/surcharge removal if the rate of consolidation is faster than the predicted value. If necessary, contingency measures to be implemented in a timely manner.
- Recording of reclamation performance during construction to be kept for future reference.

- The geotechnical models representing the site conditions can be calibrated against the field measurements and performance. The calibrated geotechnical models can then be used to refine the post construction settlement predictions.

It was proposed that field monitoring be carried out regularly during bund construction and land reclamation in order to provide an early indication on any impending instability problems, and to monitor the performance of the preload and embankment founded in the soft soil areas. This included the installation of both settlement plates and settlement pins across the reclamation. The data from these instruments would be used to both confirm the design assumptions and also to establish the stability status of the bund and reclamation as they are being built.

The field monitoring would allow the risk of failure along the bund to be minimised and allow refinement of geotechnical models to update post construction settlement predictions. The following mitigating measures can be implemented in the event failure becomes imminent, without undue construction safety risk:

- Reducing the height of the reclamation
- Extending the period between lifts and wait for strength gains of the underlying soft cohesive soil

In the event the rate of settlement of preloaded embankment is slower than expected, the following measures could be adopted to rectify the situation:

- Leaving the preload/surcharge in place for an extended period of time
- Increasing the preload/surcharge height
- In extreme cases, contingency measures could include using ground inclusions to improve the strength of the ground.

8 CONFORMING AND VARIATION DESIGNS

8.1 CONFORMING DESIGN

The design detailed in Sections 6 and 7 above was the “conforming design”, which conformed to the original scope agreed with PKPC. It assumed that Stages 1 and 1A would be constructed in two stages, and a time lapse exists between the completion of Stage 1 and the commencement of construction of Stage 1A. Both Stages 1 and 1A (including all bunds and reclamation areas) would be constructed to their final configuration.

8.2 VARIATION DESIGN

In October 2010, following detailed pricing of the proposed scheme and based on the direction from PKPC, the need for a lower cost solution was defined, leading to a revised concept of the containment bunds and reclamation for Stages 1 and 1A being developed. This revised concept, termed the “variation design”, adopted a high risk profile to the bund and reclamation design with lower performance requirements needing to be achieved.

In the “variation design”, the original Stage 1 and 1A would be constructed in one single stage, although the seaward bund of Stage 1A would be required to be located closer to the shore than in the original scope. The original Stage 1 area would be fully constructed, while the construction of the proposed bunds (B5 to B10) of the original Stage 1A would also be fully constructed. The remaining portions of the original Stage 1A may be constructed in separable portions.

No ground improvement or replacement was to be adopted for the variation design except in the areas which form the spine road and service corridor for the reclamation area. Early construction of pavements and services and hence controlled consolidation of this area are required. The ground improvement adopted in this area includes preloading the area with the proposed fill materials for a specified period of time to over-consolidate the soil, and then surcharging the ground with additional fill materials to achieve a reduction in post construction settlement. The variation bund design included the use of stability berms (and high strength geotextile where required) directly founded on the seabed, with no dredging of the existing spoil material. A typical section of the proposed design is given in Figure 4 below, which shows the slag bund, slag stabilising berms and high strength geotextiles and rock revetment which consists of primary and secondary rock armours.

For all other areas, the reclamation is allowed to settle, with no total or differential settlement criteria imposed on their performance. Notwithstanding this, at the southeastern corner of the reclamation where unconsolidated dredge material is the thickest, preloading of this area was recommended for a period of 6 months in order to allow the early stages of settlement to get underway and allow confirmation and future revision of the settlement performance for the area. The predictions for this area indicate that up to 1.2m of settlement may occur during this period, which would account for the majority of the predicted settlement, and would identify soft spots as a result of differential settlement. This design has been put out to tender, a constructor selected, and construction is due to commence imminently.

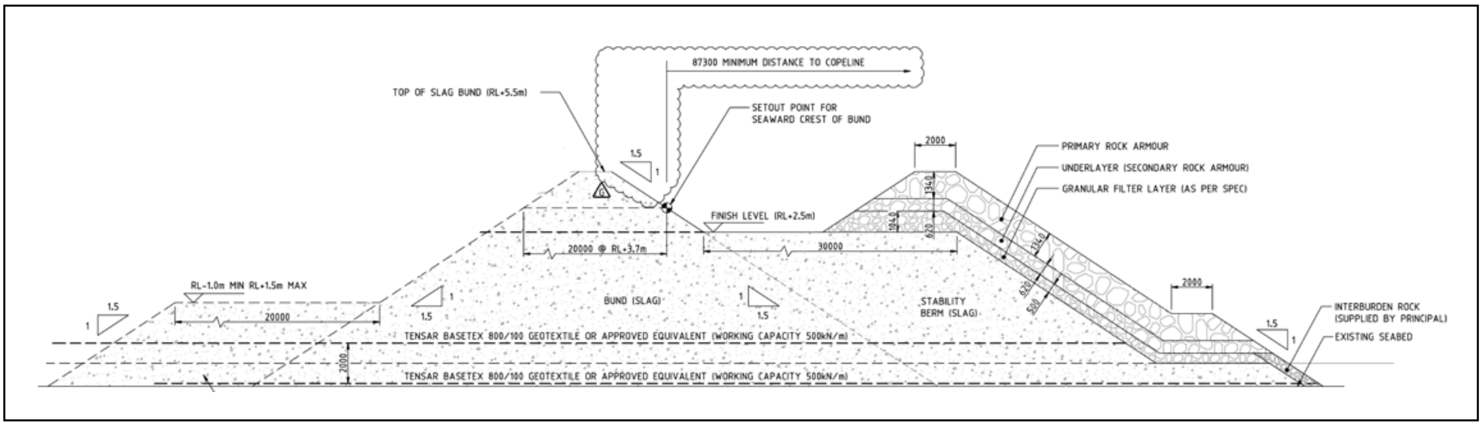

Figure 4: Typical section of bund for proposed variation design

9 CONCLUSIONS

The Outer Harbour of Port Kembla has been subjected to deposition of materials in the central and southeast sections of the works from five previous disposal campaigns, whereby dredged sediment from the Inner Harbour was relocated to the Outer Harbour. This paper has presented the methodology and results of geotechnical offshore site investigation at the Outer Harbour and the associated detailed design of the reclamation.

Unconsolidated dredged fill underlie the majority of the works and generally thicken towards the east and southeast, where a maximum thickness of eight metres of dredged spoil was encountered. This presented a significant challenge to the design as the reclamation fill material would need to be founded on these soft deposits.

Phase 1 geotechnical design for the Outer Harbour development includes the design of containment bunds and land reclamation design associated with subsequent infilling with appropriate select fill material. Various design options were considered for both the bund and reclamation construction. Instrumentation and monitoring were proposed as part of the detailed design to confirm design assumptions and monitor the performance of the reclamation.

As the detailed design progressed, it was decided by PKPC that the conforming design which satisfies the original scope of works would not be constructed. Instead, a variation design consisting of the construction of all bunds, and reclamation areas without any intrusive ground improvement and an observational approach to the settlement performance was developed. No ground treatment was adopted, except for the areas which form the spine road and service corridor. This design has been adopted and will be implemented for construction commencing soon.

Construction is commencing imminently and SMEC has been engaged by PKPC to act as the Principal's Representative to review the monitoring data obtained and provide design advice during construction.

10 REFERENCES

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