Case History
Submarine Landslides on the South-Eastern Australian Margin

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1. INTRODUCTION

The consequences of submarine landslides include damage to seabed infrastructure (communications cables and buried pipelines), subsidence of coastal areas and the generation of tsunamis (Masson *et al.*, 2009). Our failure to understand the causes of these phenomena means that submarine landslides present a significant risk to coastal and offshore development, and have on occasion resulted in the halting of offshore developments. It has been established that large submarine landslides can produce tsunamis, such as the earthquake triggered submarine slides in 1929 (Grand Banks, USA; Fine *et al.*, 2005) and 1988 (Aitape, Papua New Guinea; Tappin *et al.*, 2001) which both resulted in significant casualties. The large loss of life and damage to infrastructure from the Indian Ocean tsunami of 2004 (Lay *et al.*, 2004) and the recent Japanese event have increased interest in the tsunamigenic potential of large submarine slides. Australia is vulnerable to tsunamis with 85% of the population living within 50 km of the coast and much of the critical infrastructure located close to the coast. It has been suggested (Dominy-Howes, 2008) that the maximum credible tsunami could cover mainly in 6 m of water, and while the possibility of such an event has major implications for risk assessment and siting of critical infrastructure, the likelihood cannot be sensibly determined.

The geological record contains many examples of submarine landslides, which can vary in scale from minor shallow slides to very large slides, such as the Storegga slides off the Norwegian coast which have a total volume of over 3000 km^3 (Haflidason *et al.*, 2005). Statistics on known landslides on the eastern continental slope of North America, which has geological similarities to Australia's eastern margin, have been published by Masson *et al.* (2006). These show that between 30^{o}N and 45^{o}N there are 152 large landslides affecting an area of nearly 40000 km^2. Most failures occur on slopes of between 1^o and 7^o, with the greatest number of failures occurring on slopes of 3.5^o. The area affected by failures decreases as the slope increases, and the depth of water at the slide head ranges from 250 m to 2500 m, with the greatest number of failures occurring at water depths of around 1000 m. Despite extensive literature on the nature and causes of submarine landslides, their dynamics and triggering processes are not well understood (Locat and Lee, 2002; Bardet *et al.*, 2003). One of the principal reasons for this is the limited data on the physical and mechanical properties of the sediments, particularly from the slide plane, as these materials have not traditionally been collected.

In Australia, studies of the southeastern (SE) Australian continental slope (Jenkins and Keene, 1992; Glenn *et al.*, 2008; Boyd *et a*l., 2010) have been very limited until recently. Evidence of submarine landslides on the SE Australian margin was first reported by Jenkins and Keene (1992), but it was not until high resolution, multi-beam bathymetric data became available (Glenn *et al.*, 2008; Boyd *et al.*, 2010) that the true distribution of these slides could be established. The recent collection of high-resolution multibeam echo-sounding and sub-bottom profiling data has provided a detailed view of mass-transport features over a 900 km length of the margin. A wide range of slide features has been detected as well as a series of canyons which cut through the slope sediments. The submarine slides range in scale from hundreds of small slides with volumes of <0.5 km^3 up to the largest documented slide which has a volume of 20 km^3 (Boyd *et al.*, 2010).

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2. SUBMARINE LANDSLIDES

2.1 SOUTHEASTERN AUSTRALIAN MARGIN

The SE Australian continental margin stretches 1500 km north from Bass Strait to the Great Barrier Reef (Boyd et al., 2004). The margin which is by world standards narrow, deep and sediment deficient, was formed by rifting in the Cretaceous period between 90M and 65M years ago (Gaina *et al.*, 1998). Since then, margin subsidence has been relatively minor. The continental shelf ranges between 14 and 78 km wide and is relatively flat with a thin sediment cover. The sediment reaches a peak thickness of about 500 m at the edge of the shelf, which occurs at depths ranging between 55 m and 180 m. The continental slope is the region from the shelf edge to the Tasman Abyssal Plain where the water depth is around 4500 m. The continental slope ranges from 28-90 km wide and has average slopes in the range from 2.8° to 8.5°. The sediment cover generally reduces from the shelf edge to the Abyssal plain, and is absent from the lower slope off southern NSW (Boyd *et al.*, 2004).

Figure 1 shows the regions where detailed bathymetric studies have been conducted in the last 5 years and from which the sediments have been recovered that are discussed later. Two ship surveys have been conducted, one of the continental slope off Brisbane (SS2008-12), and the other off Sydney (SS2006/10). The surveys consisted of both sub-bottom profiles and echo-sounder records to provide a detailed picture of the seafloor and reveal the underlying geology. In addition 26 gravity cores were obtained from these regions from areas within and adjacent to several slide features, and further sediment was dredged from deeper water. An overview showing the bathymetry for both of the studied areas is given in Figure 2. At this scale it is possible to see that a number of canyons cut into the slope sediments and most of these are off the major rivers. Further details of some of the slides are shown in Figures 3 and 4.

Close inspection of the bathymetric data reveals several distinct large sediment slides varying in volume from <0.5 km^3 to 20 km^3 on the upper slope (water depths < 1200 m) of the SE Australian margin (Boyd *et al.*, 2010). The large slides typically comprise a distinct U-shaped trough in cross-section (3-6 km wide and 20-250 m deep) backed by an amphitheatre-shaped crestal zone. This slide morphology is similar to the classical circular failure profile described by Varnes (1978), but they are elongated in longitudinal profile. The sides and head walls of the scarps are relatively steep with slopes of up to 17°. The largest slides are the Bulli (Figure 3c) and Shovel Slides, near Sydney, on slopes of around 4.5° that are up to 13 km long and 5 km wide with volumes of 20 km^3 (Glenn *et al.*, 2008) and the Byron slide (Figure 3b), off Byron Bay, with a volume of 3 km^3 and slope of 6.5° (Clarke *et al.*, 2011). Sub-bottom profiling data from multiple sections across the continental slope and in particular across the slides show the sediment is built up of well stratified beds (Glenn *et al.*, 2008), which have been suggested to be evidence of past slide events. In most locations, sediments derived from the slides cannot be detected on the slope and it appears that the slide material has been transported to the abyssal plain. However, in a small number of locations (Figure 3a) where the slopes are less steep (< 2°), the slide debris flow deposits have remained on the slope and contain blocks up to 350 m wide and 50 m high.

Figure 2 shows a large number of canyons that cut into the continental slope sediments. These have been categorised into large box canyons, and smaller narrow linear canyons. The 46 large box canyons are on average 14 km wide, 20 km long and over 600 m deep. They stretch from the middle slope to the abyssal plain, and have slopes up to 17° on the walls. Narrow linear canyons occur in the upper slope sediments, most located in central NSW off major river systems such as the Shoalhaven, Hunter and Tweed or off Fraser Island. Well developed examples are 800-1900 m wide, 120-320 m deep and extend downslope for 14-22 km. Canyon wall slopes are up to 34°, the steepest slopes found on this margin (Boyd *et al.*, 2010).

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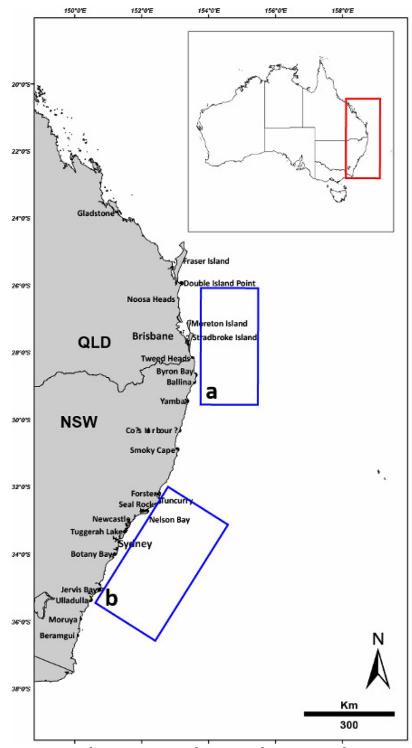

Figure 1: Location map of the two study areas along the southeastern Australian coastline. Blue insets a) and b) mark the location of the bathymetric maps presented in Figure 2.

Further observations from the bathymetry include the widespread slope failures on the mid-slope, shown in Figure 4, which demonstrates an average slope of around 8° and the widespread relatively shallow failures observed on the Nerang plateau shown in Figure 3a, where the average slope is < 2°. There are also circular depressions, referred to as pock marks, off Newcastle which are believed to be associated with gas leakage from the underlying Permian coal measures (Glenn *et al.*, 2008).

The figures reveal evidence of widespread erosional features on the SE Australian continental slope. This is different from other margins of similar age, for example the US Atlantic and Gulf Coasts, where sediment deposition is the more dominant process. However, both margins exhibit extensive slides and other erosional features. The 500 m thickness of the sediments on the SE Australian margin has been taken as evidence of a previous period of deposition (Davies 1979, Boyd *et al.*, 2004), but the sediment thickness is substantially less than other margins. This can in part be explained by the dryness of the Australian continent, the relatively subdued highlands and its small rivers. When the resulting low sedimentation is combined with ongoing subsidence of the abyssal plain caused by initial crustal thinning and later thermal cooling, which has caused the gradients on the margin to slowly increase, the result has been a retrogressive gravity failure over all of the lower slope and much of the upper slope. Thus it is considered that the present state of sediment instability, where the overlying sediment wedge is continually undercut by slope failure over geological time, is the cause for modern episodes of failure (Glenn *et al.*, 2008).

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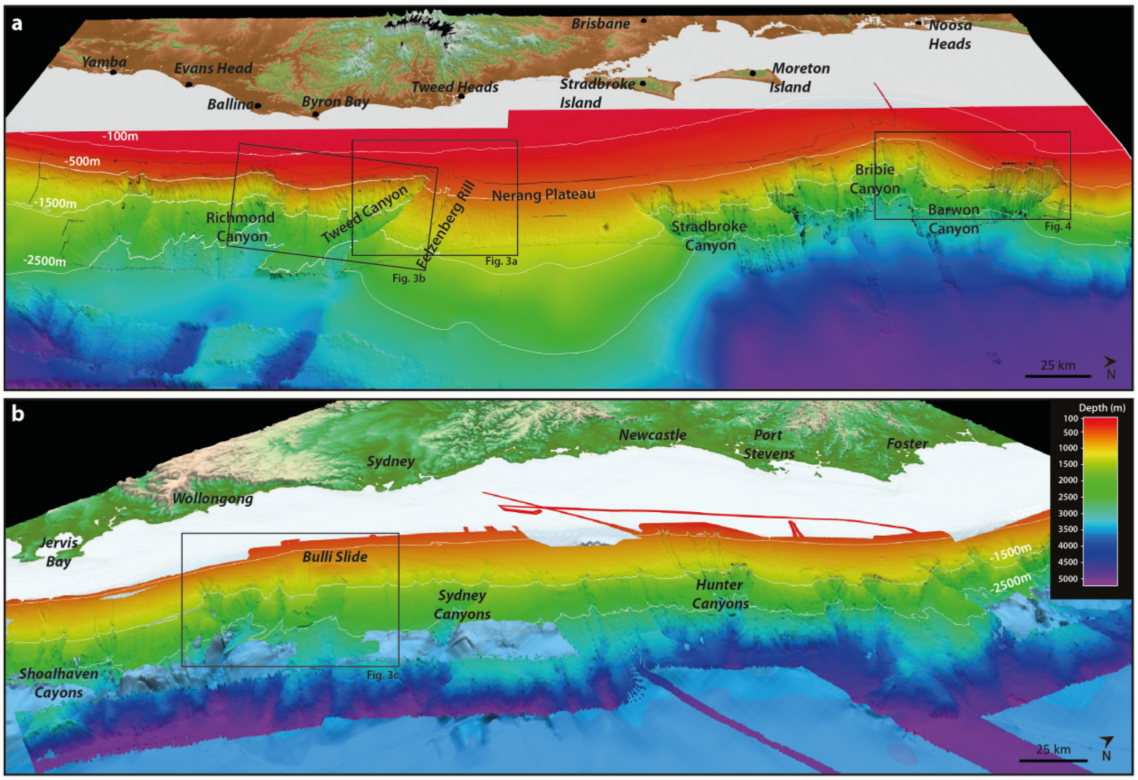

Figure 2: Bathymetric maps of the a) the southern Queensland / northern New South Wales continental slope and b) the mid New South Wales continental slope. Data for these maps were collected on two RV Southern Surveyor voyages: SS2008/12 off the southern Queensland / northern New South Wales coastline (Boyd *et al.*, 2010) and SS2006/10 off the mid New South Wales coastline (Glenn *et al.*, 2008). Insets mark the location of the close-up slopes images presented in Figure 3 and Figure 4.

2.2 TRIGGERS

The literature on submarine landslides summarised by Masson *et al.* (2009) and Locat and Lee (2002) lists a variety of causes for their initiation. These include: earthquakes, storm wave loading, erosion and in particular slope over-steepening, rapid sedimentation leading to under-consolidation, the presence of weak layers, gas hydrate dissociation, sea-level changes, glaciations/isostatic uplift, volcanic activity and diapirs. It is also widely accepted that a combination of these factors is required to initiate a landslide, especially where these occur on very shallow slopes. There is data indicating that several large landslides have coincided with earthquakes (e.g. Tappin *et al.*, 2001; Bardet *et al.*, 2003; Masson *et al.*, 2006; Synolakis *et al.*, 2002). The role of weak layers, oriented parallel to the sedimentary bedding, has long been used to explain the scale of some large slides, but more recently the importance of weak layers in controlling sliding at all scales has been noted (Masson *et al.*, 2009). Despite this Masson *et al.* (2009) also commented that "we know very little about the nature and characteristics of these weak layers, since they have rarely been sampled and very little geotechnical work has been done on sediments recovered from them". An important consideration is the brittleness of the sediments. Weak layers need to lose strength rapidly and pore pressure needs to rise for effective stresses to reduce. Masson *et al.* (2009) suggest that clay rich sediments with high water content and high plasticity are required for this to occur.

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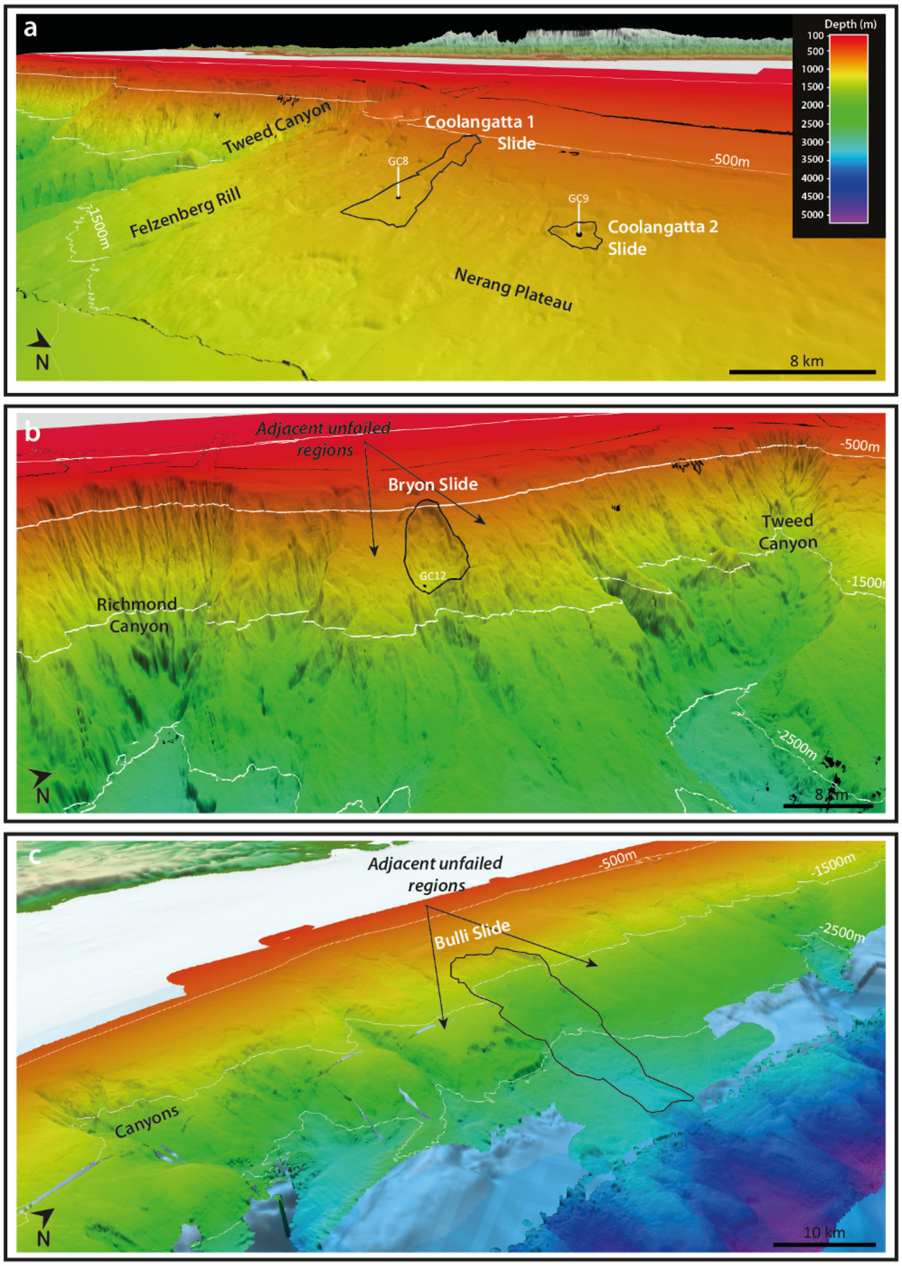

Figure 3: Digital elevation model (DEM) of the slope geometry for four slide sites (outlines denoted by black line): (a) Coolangatta 1 and Coolangatta 2 Slides, (b) Byron Slide, (c) Bulli Slide. Also shown are locations of the three gravity cores (GC8, GC9, GC12) referenced in this study, collected on the RV Southern Surveyor SS2008/12 voyage.

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Lee (2009) has shown that landslides were more frequent during and just after the last period of glaciation than they are today. One of the suggested reasons for this is that glacial periods coincide with periods of low relative sea level. Figure 5 shows the relative sea level curve for Australia for the last 0.5 million years, which indicates that the sea level has been over 100 m below its present level on several occasions, and the times of these minima are associated with glacial periods. The lowered sea level can increase the likelihood of sliding because it results in the shoreline migrating closer to the shelf edge, leading to increased erosion and higher rates of sedimentation offshore, which now occurs directly on the slope. The lower water pressures (and possibly changed temperatures) can lead to release of gas from gas hydrates increasing pore pressures and reducing strength, and related changes to stress levels in the crust can increase seismic activity. Increased groundwater flows from underlying rocks can occur also contributing to reduced strength. It has also been suggested that changes to deep ocean currents are associated with glaciations and erosion from these currents can contribute to slope steepening (Hubble *et al.*, 2011).

Another mechanism that has been postulated to explain submarine slides is that of creep, slow down slope movements due to gravity stresses that may lead to failure of the sediment mass or to failure on a weak layer at depth. It has been demonstrated that thick deposits on steep slopes can fail by this process (Silva and Booth, 1984). However, as noted by Hampton *et al.* (1996) proof that creep is significant on continental slopes is elusive.

Observations of the widespread occurrence of submarine slides suggests that weak clay layers cannot be a major cause, and tend to favour earthquakes as the triggering mechanism. Nevertheless, it is widely accepted that neither the submarine sliding process nor the slide triggering mechanisms are very well understood (e.g. Locat and Lee, 2002; Mosher *et al.,* 2009), and this is particularly so in the cases of submarine mass failures that do not appear to be linked to seismic activity.

2.3 Sediment Properties

From the recent ship cruises 26 gravity core samples have been obtained, 14 from the region off Sydney and 12 from the region off Brisbane. For most of the gravity cores the soil has been logged and basic properties, particle size distribution, mineralogy and densities have been obtained. The results from the Sydney region have been reported in detail by Glenn *et al.* (2008) and only typical results are reported here. The basic classification tests have been supplemented by a limited number of triaxial, oedometer, shear box and vane shear tests to investigate the mechanical behaviour of the sediments. The mechanical tests have been performed to investigate the landslide triggering processes and in particular to determine the collapse potential of the sediment and the influence of composition and stress level on this behaviour. A summary of the classification data, which is limited to material from the upper 5.3 m of the sediments as this was the maximum depth of penetration of the gravity corer, is included in Table 1. This shows that the continental slope sediments are predominantly comprised of silt sized material, with about 15% clay, variable amounts of sand sized particles and significant organic content. The sediments contain a significant amount of carbonate grains derived from the remains of living organisms and also significant amounts of terrigeneous material, believed to be transported by the wind from the interior of the continent. Although there is some variability in the composition from core to core there is a broad similarity in the particle distributions all along the margin.

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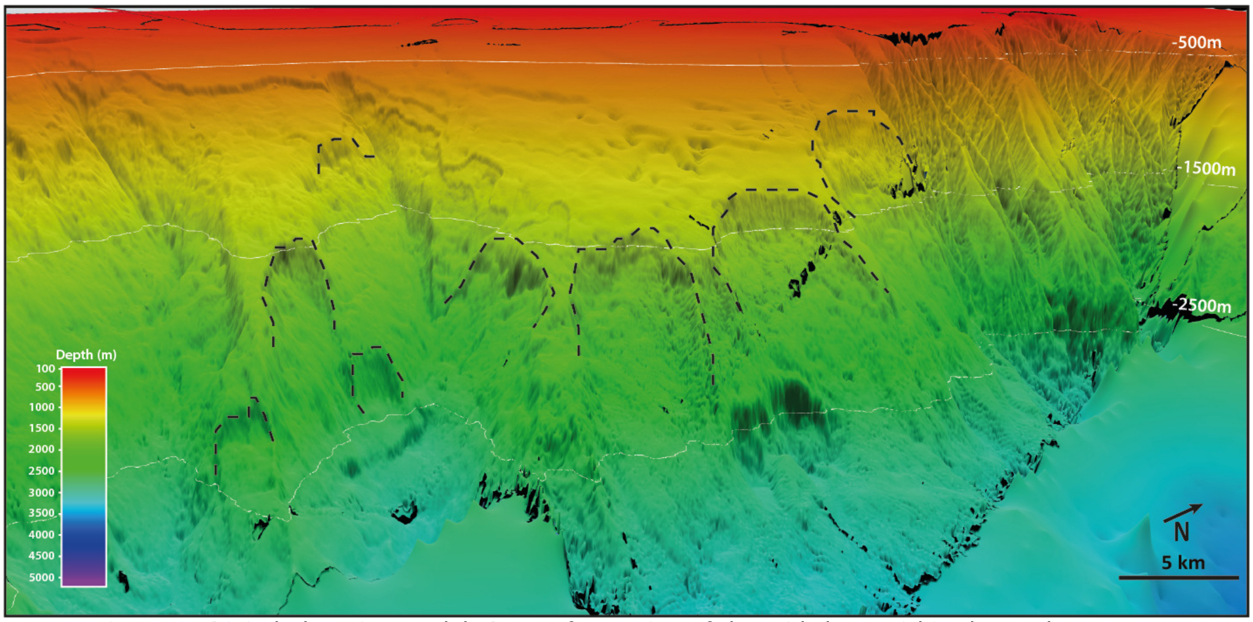

Figure 4: Digital elevation model (DEM) of a section of the mid-slope within the study area. Note the abundance of slump/slide scars presenting arcuate crests (crest outlines denoted by black dashed lines). Modified from Hubble, 2011.

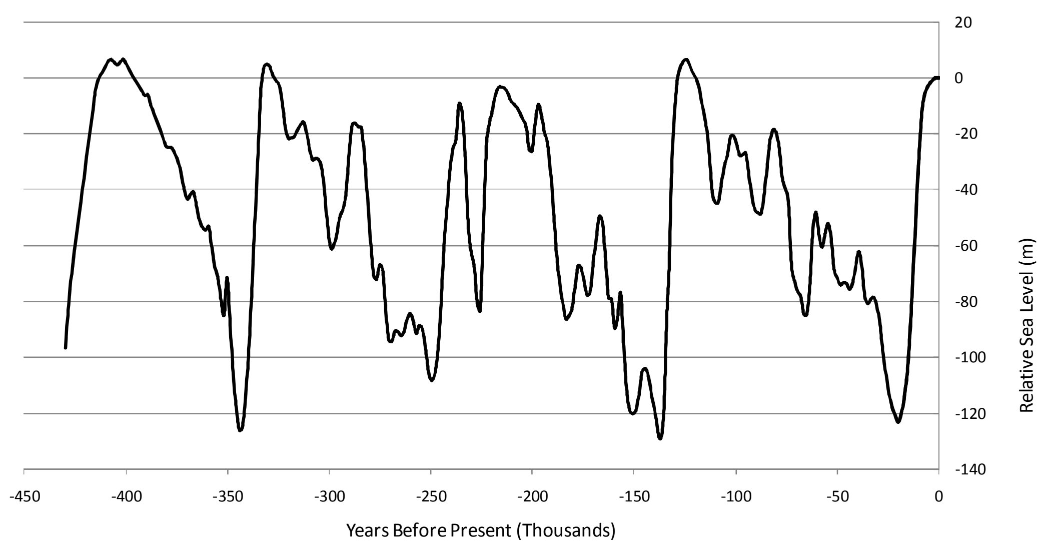

Figure 5: Relative sea level history (after Waelbroeck *et al.*, 2002)

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Table 1: Summary of available classification data

	Clay (%)	Silt (%)	Sand (%)	Carbonate (%)	Organic (%)	LL	Ip	Moisture content (%)
Brisbane	10-20	50-65	15-40	20	8	46.5	9	50-100
Sydney	8-25	30-80	10-60	35	?	44	18	55-85

Several of the gravity cores were obtained from within detected landslide features in an attempt to penetrate through the base of the slides to assist in constraining the slide depths and dates. In most locations this was unsuccessful as the recent sediment drape overlying the slide surface was thicker than the gravity corer could penetrate. However, in three locations off SE Queensland a distinct boundary in the sediments could be detected at depths between 87 cm and 220 cm. The sedimentological and geotechnical properties of the sediments and their variation with depth from one of these cores (GC12; see Figure 3b for core location) are shown in Figure 6. It can be seen from the figure that there is a distinct change in density and moisture content, as well as appearance of the material at a depth of 87 cm. It is also noticeable that this change in density is not associated with any significant change in the grading, carbonate content or organic content of the material.

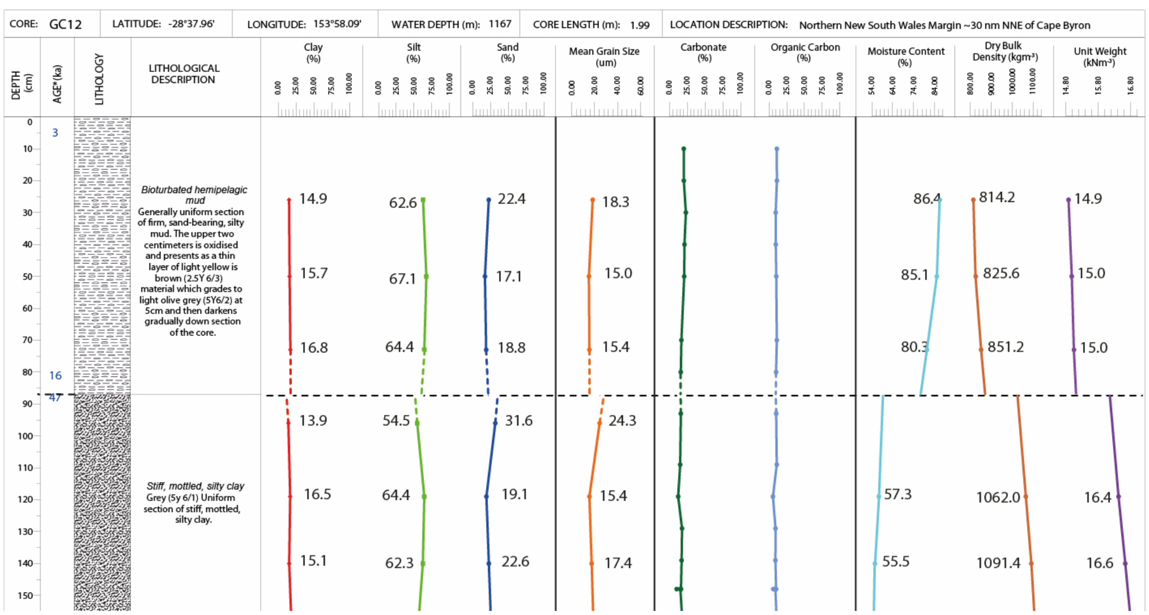

Figure 6: Characterisation of sediment from core GC12, showing physical properties with depth below seabed. Bulk radiocarbon dates are also shown. The presumed slide plane is indicated with a dashed black line at 87 cm depth below seabed (Modified from Clarke *et al*., 2011)

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Additionally, a bulk radiocarbon (14C) age was determined for sediment sampled directly above the slide plane for this core, returning a date of 15.8 ka for the recent material just above the inferred slide surface (Clarke *et al.*, 2011). This date is consistent with sliding occurring around the time of the most recent sea level low shown in Figure 5. Dating for the deeper sediment could not be determined because its age exceeded that for which 14C dating is reliable. The dating has also enabled the rate of sedimentation to be determined, providing values between 0.3 and 1.2 m/10,000 years. As the rate of deposition is expected to have been higher in the past, these rates of sedimentation suggest that sliding must have been a geologically common event since the formation of the margin 60 million years ago as the current sediment deposit is less than 500 m thick.

Using the values of Cc given below, the change in moisture content at the inferred slide plane can be interpreted as representing a slide depth of anywhere between 10 m and 200 m. The depth reconstructed at the GC12 site by replacing the material apparently missing from the U-shaped trough, i.e. by maintaining the continuity and shape of the adjacent slope and projecting it above the GC12 site, is approximately 250 m. Thus while it is possible to date a possible slide at approximately 16,000 years there is insufficient information to determine whether this is the date of the main slide at this location and further mechanical and dating studies are in progress to further constrain the result.

2.4 GEOMECHANICAL BEHAVIOUR

Limited geomechanical data are available from the vicinity of the slides close to Sydney and from the region off Byron Bay. This has consisted primarily of one-dimensionally consolidated undrained triaxial tests, with some additional shear box tests to evaluate residual strength properties for the Sydney sediments.

Figure 7 shows typical compression plots from 1-D compression tests on undisturbed specimens. Results of three specimens from one of the cores (GC9; see Fig 3a for location), from SE Queensland are shown together with a typical specimen from a core off Sydney. Although there is some variability in the responses the similarity of the response of the specimen from Sydney and SE Queensland is remarkable. Based on these very limited data it appears that the grading, mineralogy and compressibility of specimens on the continental slope are similar along most of the 1500 km of the SE Australian margin. The specimens show high compressibility with Cc values ranging from 0.3 to 0.65. This is considerably higher than would be expected from their remoulded index test results, as the correlation Cc = IpGs/2 would suggest Cc values of 0.13 to 0.26 for plasticity indices of 10% to 20%. It can also be noted that the moisture contents in the upper 5 m are significantly higher than the liquid limit and vane shear tests have indicated that the specimens have significant sensitivities (>2). These data indicate that the slope sediments are structured and, while the cause of the sensitivity has not been established, it could be related to the relatively high organic content of up to 8%, which is known to be a factor in sensitivity in other soils.

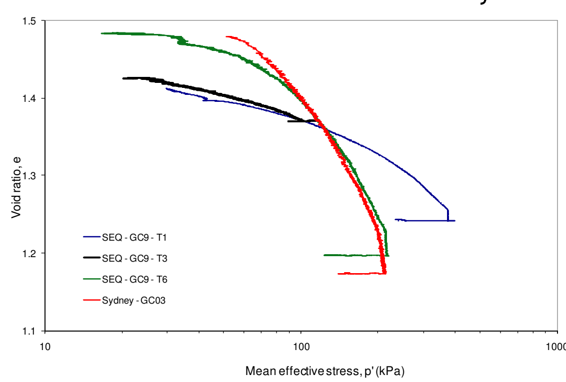

Figure 7: Response of core specimens to 1-D compression

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The responses of 3 specimens to undrained shearing in triaxial compression are shown in Figure 8, and the associated effective stress-strain curves are shown in Figure 9. To enable comparison of the tests the deviator stress and excess pore pressures have been normalised by the vertical effective stress at the start of shearing. Specimens GC9-T1 and GC9-T3 were adjacent specimens and have similar compressibilities, as seen from Figure 7, but they responded differently to shearing. Specimen GC9-T1 at the higher stress level ($\sigma'v$ = 620 kPa) shows a more brittle response with the peak deviator stress attained at a very small strain, after which the resistance rapidly decreases to its ultimate value. In contrast, specimen GC9-T3 at the lower stress ($\sigma'v$ = 167kPa) does not reach a maximum until relatively large strain. From the pore pressure responses and the effective stress paths it can be seen that this difference is a consequence of a transition from dilative to compressive behaviour as the stress level increases. The more compressible (Sydney) specimen shows a response similar to the higher stress GC9-T1 even though the stress level ($\sigma'v$ = 220kPa) is similar to GC9-T3. The shear response of GC9-T6, which has similar compressibility to the Sydney specimen also shows the tendency for increasing brittleness with increasing stress level, however, this result is not considered reliable due to non-uniform deformation during shearing. This pattern of reducing dilation and increasing brittleness with stress level can explain why deeper failure surfaces develop.

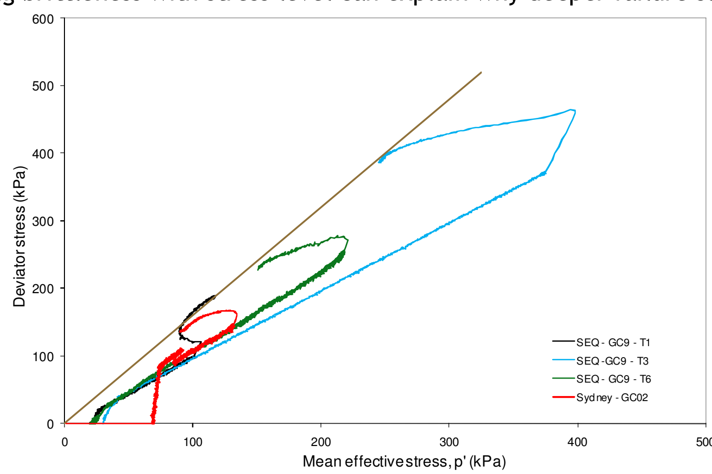

Figure 8: Stress-path curves from triaxial tests.

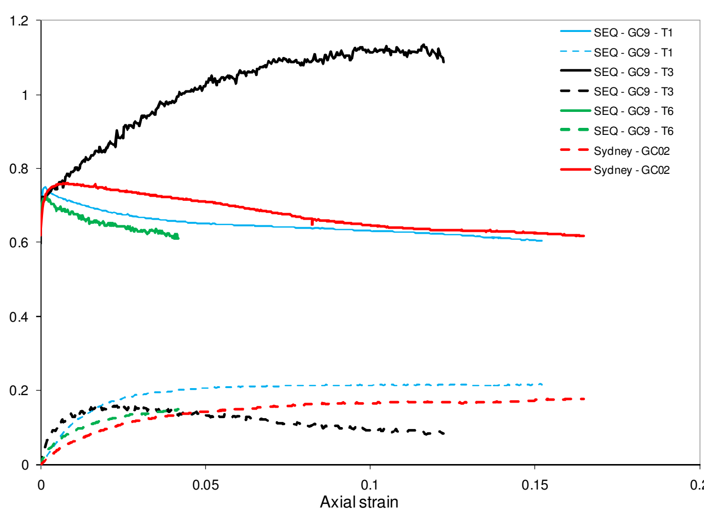

Figure 9: Normalised deviator stress and pore pressure responses from 1-D compressed specimens.

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From Figure 9 it can be seen that all specimens approach a similar ultimate frictional resistance, which for the specimens shown ranged from 37^o to 40^o. Cyclic shear box tests showed no evidence of any lower residual frictional strength (Glenn *et al.*, 2008).

3. ANALYSIS

3.1 Landslide Initiation

Geomechanical modeling of three of the submarine landslides has been undertaken using the slope stability program GEO-SLOPE/W (2007) to examine the influence of cohesion, friction angle and slope geometry on the stability (Clarke *et al*, 2011). As the friction angle is around 40^o and the slopes are from 3^o to 6^o static analyses predict very high factors of safety. Analyses have also been conducted to investigate the effects of earthquake loading by including a factor for seismic accelerations in the standard pseudo-static limit-equilibrium calculations. Selecting an appropriate value for the seismic coefficient acting on the failure mass can be especially difficult (Seed and Martin 1966). A very crude investigation of seismic loading on the slopes indicates that lateral and vertical accelerations of 0.3 g (a_h = 0.3 g, a_v = -0.3 g), the upper limit of those used to investigate the stability of earth dams during earthquakes (Seed and Martin, 1966; Ozkan, 1998), would be sufficient to destabilise the slopes of the seafloor in the present study. While this approach has been widely used to assess the stability of submarine slides to earthquake events, its applicability to such large volumes of soil is questionable and the approach is of limited value in understanding the mechanisms leading to failure.

Puzrin *et al*. (2004) have argued that it is unlikely that a failure can develop over distances of several kilometres instantaneously, and that a progressive failure mechanism must be considered. On land progressive failures are often observed to result from oversteepening of the toe of a slope, where failure at the toe leads to a retrogressive failure that migrates upslope. This mechanism is also considered to be responsible for the large Storegga submarine slide. Puzrin *et al*. (2004) have suggested an alternative progressive failure mechanism that involves a weakened zone propagating down slope. The basis of the analysis can be explained by considering Figure 10. The starting point is that a zone of elevated pore pressures develop (Figure 10a), possibly owing to an earthquake, where the soil reaches a state of failure for which the mobilisable soil resistance is lower than the stresses at equilibrium from the weight of the overlying soil ($\tau_r < \tau_g$ Figure 10b). If the length of this failed zone is sufficient a global and catastrophic failure will occur. However, if the zone of failure is more limited the soil will tend to move downslope into the currently unfailed region. If the failure plane can propagate because the energy released is greater than that needed to progress the failure, then the shear plane can grow, and if conditions are unfavourable, it may continue to advance until it reaches a length where global failure results. For significant energy release to occur the ultimate resistance of the soil needs to be lower than that required to resist the gravitational stresses, and the soil needs to respond in a brittle manner. The triaxial test data shown above display the type of brittle behaviour that can potentially lead to this type of mechanism.

The analysis of Puzrin *et al*. (2004) suggests that failure begins upslope, but depending on the soil type and behaviour progressive failure may be limited or not occur and it is possible that the resulting length of the failure surface may be less than the critical value required for a catastrophic failure. There is some evidence for this from a number of head scarps present on the SE Australian margin where the soil below has not moved significantly. Glenn *et al*. (2008) suggest that these features represent the sites most likely to fail in the future. However, if the analysis of Puzrin *et al*. (2004) is correct, the soil movements make these sites less likely rather than more likely to lead to failure.

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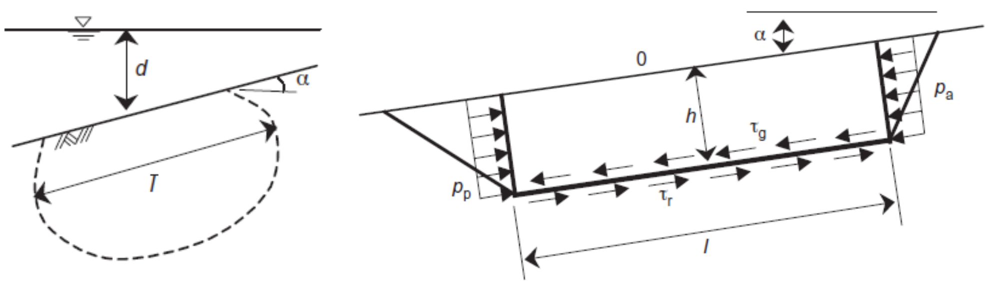

Figure 10: (a) Zone of elevated pore pressures, (b) Slope failure mechanism (after Puzrin *et al.*, 2004)

3.2 Tsunami generation

The viscous drag on the overlying ocean due to the movement of the slide block is responsible for the tsunami generation. Many studies have been conducted into tsunami generation and propagation, but only the work of Ward (2001) will be mentioned here. Ward (2001) presents results of tsunami generation and propagation based on classical tsunami theory and assuming linear wave theory. This theory uses a rigid seafloor overlain by an incompressible, homogeneous and non-viscous ocean subjected to a constant gravitational field. Figure 11 provides an indication of the size of the tsunami from analysis of a rectangular block of length L (km) and width W (km) sliding down an inclined plane for a water depth of 1000 m. The tsunami velocity is given by $\sqrt{(gh)}$ where h is the water depth, and in 1000 m of water this is 99 m/s. There are no reliable data on the speed of submarine slides, although turbidity currents from the 1929 Grand Banks slide travelled at 25 m/s, and based on travel distances of slide debris speeds of up to 80 m/s have been inferred for some large slides (Masson *et al.*, 2006). The fracture mechanics approach proposed by Puzrin *et al.* (2010) enables an estimate of the initial velocity to be determined and values of around 10 m/s were estimated for some reported slide geometries. For the largest slides on the SE Australian margin the water depth is around 1000 m, the maximum slide thickness is 200 m, and assuming a maximum velocity of 20 m/s a peak tsunami wave height of around 100 m can be estimated from Figure 11. As the maximum dimensions of the sliding blocks on the SE Australian margin are 20 km x 5 km some reduction to this height may be appropriate. Further increment in wave height will occur as any waves approach the coast.

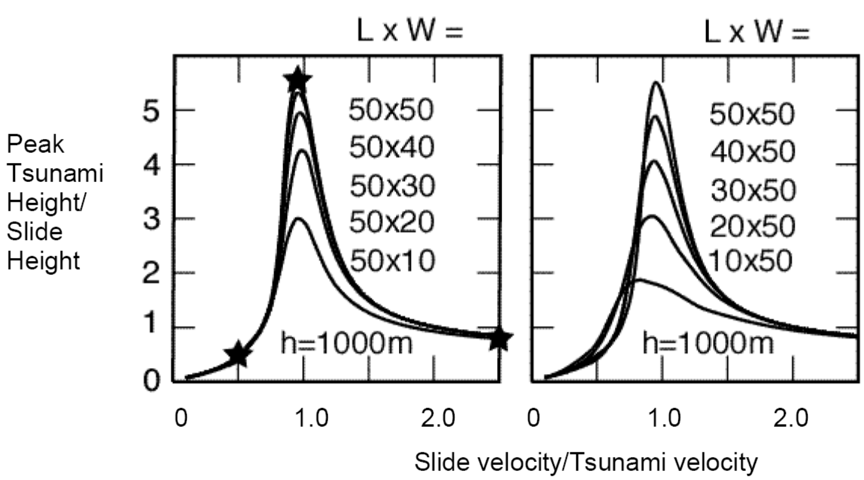

Figure 11: Effects of slide velocity and slide dimensions on peak tsunami height (after Ward, 2001).

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4 SUMMARY

The paper has shown evidence of large scale mass wasting phenomena on the SE Australian continental slope, including the existence of many landslide features. Slides are evident all along the margin from Shoalhaven to the Sunshine Coast, and on slopes ranging from 1^o to 9^o. The soil properties of the upper sediments are similar along the margin and show no evidence of weak clay layers, although they do contain significant amounts of clay. The friction angles of the sediments are in the range of 37^o - 40^o, so that conventional soil mechanics would suggest the slopes have high factors of safety. However this is clearly not the case, as slope failures are widespread. Triaxial tests have indicated a significant increase in the brittleness of the shear response with stress level, and this is thought to be significant in explaining why the slides have thicknessess of 50 m to 200 m. The largest slope failures have a volume of 20 km^3 and have the potential to generate significant tsunami waves.

The dating of the slides suggests that the most recent failures occurred at the time when the sea level was at its minimum during the last glaciation. There are several reasons why the likelihood of slides should increase at these times, however the cause of the slides on the SE Australian margin is not well understood. While the likelihood of slides appears to be lower in inter-glacial periods there are examples of earthquake caused submarine slides that have occurred recently and the possibility that a large submarine slide could occur any day cannot be discounted.

5 ACKNOWLEDGEMENTS

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Case History

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

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1. Introduction

In 2006 to 2007, a reclamation was carried out on a coastal area in south-east Asia for the development of a container terminal which has a design elevation of RL+4.5m. The subsurface profile at the site comprised over 30m of soft to firm clay. The client implemented a program of surcharging with PVD for the container yard to limit the post-construction settlement to 300mm in 20 years, but let the 500m long wharf construction using a Design and Construct delivery mechanism. The wharf construction contract was awarded in 2007, and the successful contractor elected to use CDM in a 24m wide zone behind a piled wharf deck structure to provide the necessary stability for a dredging level which varied from RL-5m at the landward edge of the wharf deck to RL-15.7m at the seaward edge of the wharf.

The CDM zone immediately behind the wharf was constructed using overlapping 1.6m diameter columns to form interlocking 24m wide panels that run normal to the wharf alignment at a spacing of 3.6m centres, giving an area replacement ratio of 40%. The CDM columns were fully penetrating to RL-35m into very stiff to hard clay, and the CDM mix provided an average unconfined compressive strength of 1.3MPa (design strength of 1MPa). This zone is thus relatively stiff, and the post-construction settlement under the design loading of 40kPa was assessed to be 35mm. Based on monitoring results of the surcharged container yard, post-construction settlement in 20 years was estimated to be 315mm.

Therefore, the challenge was to design the transition zone to limit differential settlement to acceptable limits for drainage and pavement performance.

2. Differential settlement design criteria

The following design criteria were specified by the client with respect to limitations on differential settlement for an applied loading of 40kPa in the transition area:

- In 20 years, the minimum ground slope shall not be less than 0.7% in order to maintain adequate drainage within the container handling area.
- Within the transition zone, the differential settlement shall not be more than 0.3% change in grade from the general ground slope for satisfactory performance of the pavement.

Another challenge was that there were considerable uncertainties in post-construction settlement predictions particularly for the surcharged PVD area which is beyond the responsibility of the wharf construction contractor, but has direct impact on the differential settlement of the transition area.

3. Subsurface profile and soil properties

The soil profile at this site is relatively uniform, and comprised a thick soft to firm clay layer that exhibits linearly increasing stiffness and strength with depth. The soft to firm clay is underlain by a medium dense to dense sand followed by a deep sequence of very stiff to hard clays. The adopted parameters for the soft clay layer are summarised in Table 1.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

Table 1 – Geotechnical Model Adopted

Layer	Elevation RL (m)	Thickness (m)	CR = $C_c/(1+e_o)$	CRR = $C_r/(1+e_o)$	$C_{\alpha\varepsilon}$	σ_{vo}' (kPa)	OCR	S_u (kPa)
1	+2 to +0	2	0.280	0.045	0.0112	13.22	4.9	11
2	+0 to -3	3	0.469	0.047	0.0188	28.28	3.5	15
3	-3 to -8	5	0.573	0.101	0.0229	43.18	1.9	19
4	-8 to -12	4	0.512	0.091	0.0205	66.69	1.6	25
5	-12 to -18	6	0.807	0.103	0.0323	98.17	1.7	33
6	-18 to -22	4	0.675	0.107	0.0270	122.68	1.8	49
7	-22 to -25	3	0.541	0.116	0.0217	139.16	1.6	56
8	-25 to -28	3	0.490	0.102	0.0196	156.52	1.6	63
9	-28 to -32	4	0.379	0.105	0.0152	174.47	1.5	72

where:
C_c = Compression Index
C_r = Recompression Index
e_o = Initial void ratio
$C_{\alpha\varepsilon}$ = Creep strain rate
σ_{vo}' = Initial effective vertical stress
S_u = Undrained shear strength
OCR = over-consolidation ratio

4. Adopted solution

The strategy adopted for the transition zone to meet the differential settlement design criteria was as follows:

- Surcharge the transition zone (5m surcharge height), which was carried out over a period of only 3 months due to time limitations. Only 0.65m of settlement was achieved in the surcharged transition area compared to about 3m in the surcharged PVD area.
- Provide an initial ground slope of 2.1% at the transition zone, sloping down from the edge of the surcharged PVD area to the edge of the wharf CDM area. This slope was chosen on the basis of the maximum slope at which the container handling over-head gantry will be able to operate. It is expected that this slope will reduce with time as the surcharged PVD area will settle more than the transition zone.
- Provide stepped CDM ground improvement in the 30m wide transition zone to provide a gradual increase of settlement towards the surcharged PVD area.

The CDM in the transition zone comprised twin 1.6m diameter columns at 4.8m lateral spacing and 3.5m longitudinal spacing, giving an area replacement ratio of 22.8%. An average of 8 rows of twin CDM columns was used across the transition area. The adopted strategy in the transition zone is illustrated in Figure 1 below.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

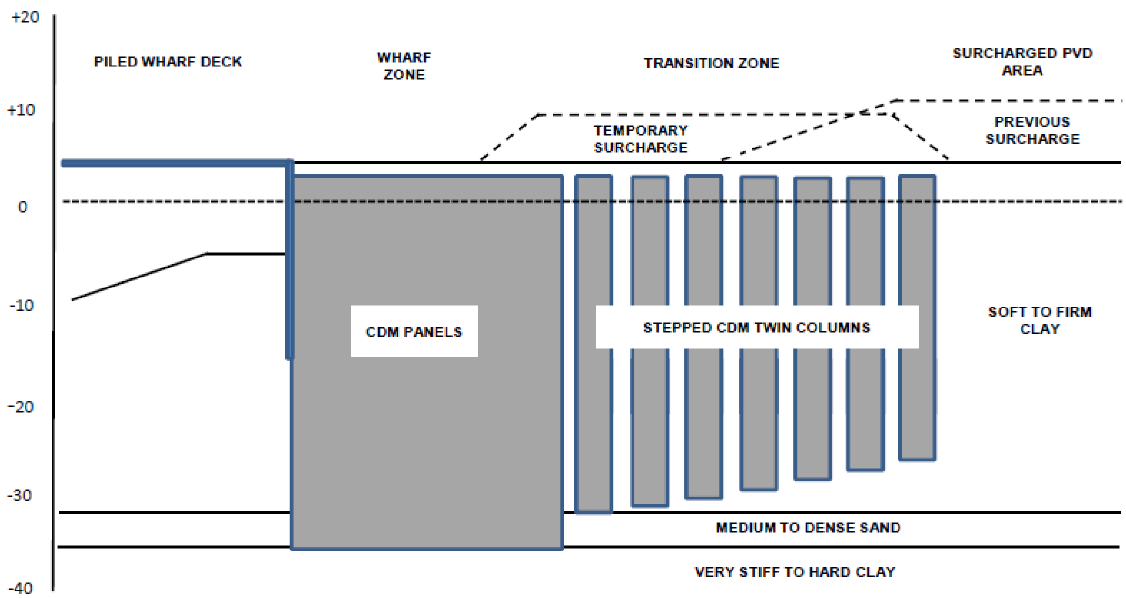

Figure 1 – Illustration of Ground Improvement Adopted in Transition Zone

5. Analysis methodology and results

The analysis method adopted for the design comprised primarily of simple one-dimensional consolidation analysis based on the method described in SGF (1997). The elastic modulus of the CDM column material was assessed to be 153MPa with a standard deviation of 28MPa based on 14 batches of tests (over 80 samples). The soil compressibility values given in Table 1 were converted to constrained modulus values based on initial and final stress levels, and OCR of the soil layers. An equivalent constrained modulus of the soil was then calculated in the CDM treated zone based on the area replacement ratio in accordance with SGF (1997). In the untreated zone below the toe of the stepped CDM columns, the soil compressibility is unchanged. Post-construction creep following preloading was assessed using the method described in Wong (2007).

After the design was approved by the client, numerical analyses were carried out using the commercially available finite element analysis software package PLAXIS. However, this paper will focus on the reliability assessment of the results rather than the analytical and numerical settlement analysis procedures.

By progressively lifting the toes of the twin CDM columns by 0.97m increments for each row moving landwards, the post-construction settlement in 20 years was estimated to range from about 60mm to 250mm, with increasing settlement towards the surcharged PVD area. With the initial ground slope set at 2.1%, the estimated settlement will reduce the initial slope to 1.4% in 20 years and this allows for uncertainty in predictions in meeting the minimum gradient of 0.7% for drainage requirements.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

The numerical analysis showed that because there is at least 2.5m of compacted fill over the soft clay, the differential settlement between CDM columns to be well within the design limit of 0.3% change from the general ground slope. Geogrid reinforcement in the fill was used to provide additional safety to even out settlement.

6. Reliability assessment

To assess the confidence level of the post-construction settlement estimate, a reliability assessment based on the procedure described by Duncan (2000) was carried out. The steps involved in the reliability assessment procedure is summarised briefly as follows:

(1) Assess the most likely values (MLV) for each parameter.

(2) Using the MLV for all parameters, calculate the most likely settlement estimate S_{MLV}.

(3) Assess the standard deviation (SD) of each parameter that involves uncertainty. In the absence of adequate statistical data, the standard deviation may be estimated in two ways (a) by using the three-sigma rule - estimating the highest conceivable value (HCV) and the lowest conceivable value (LCV) and estimating the standard deviation as (HCV – LCV)/6, and (b) by using published literature on the coefficient of variation (CV) and estimating the standard deviation as CV x MLV.

(4) Compute the settlement with each parameter increased by one SD and then decreased by one SD, while maintaining all other parameters at the MLV.

(5) Calculate the difference in settlement (ΔS_i) between (MLV + SD) and (MLV – SD) for each of the parameters, and use Taylor's series to calculate the combined standard deviation of S_{MLV} as follows:

$$SD_{MLV} = \sqrt{\left(\frac{\Delta S_1}{2}\right)^2 + \left(\frac{\Delta S_2}{2}\right)^2 + \left(\frac{\Delta S_3}{2}\right)^2 + \dots\dots\dots\dots} \qquad \text{Eq. [1]}$$

(6) Calculate the coefficient of variation of S_{MLV} as $CV_{MLV} = SD_{MLV} / S_{MLV}$

(7) Assess the probability of the actual settlement being greater than SR x S_{MLV} where SR (settlement ratio) = actual settlement/S_{MLV} using a lognormal reliability index, β_{LN} as follows:

$$\beta_{LN} = \frac{\ln\{(SR)\sqrt{1 + CV_{MLV}{}^2}\}}{\sqrt{\ln(1 + CV_{MLV}{}^2)}} \qquad \text{Eq. [2]}$$

(8) Using the built-in function NORMDIST in Excel, the probability that the settlement ratio SR may be exceeded is {1 – NORMDIST(β_{LN})}

For the row of CDM columns on the side of the PVD area which are to be installed with their toes at about mid-depth of Layer 8, there are 16 parameters that involve uncertainties which will affect the estimated settlement. The adopted MLV, CV, SD, (MLV + SD) and (MLV – SD) for these parameters are presented in Table 2.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

Table 2 – Parameters Adopted for Reliability Assessment

Description	Symbol	CV	SD	MLV	MLV+SD	MLD – SD
Applied stress	Δp	0.14	10	70.3	80.3	60.3
No of log time cycles for creep calculation	Ncreep	0.20	0.2	1	1.2	0.8
CDM treated zone	D'eq	0.20	7.3	36.5	43.8	29.2
	H(CDM)	0.02	0.5	28.5	29	28
Untreated bottom 1.5m thickness of Layer 8	CR	0.20	0.0980	0.4900	0.5880	0.3920
	CRR	0.20	0.0204	0.1020	0.1224	0.0816
	Cαε	0.20	0.0039	0.0196	0.0235	0.0157
	σvo'	0.05	8	160.1	168.1	152.1
	σp'	0.20	50.5	252.4	302.9	201.9
	H(clay)	0.00	0	1.5	1.5	1.5
Untreated clay layer 9	CR	0.20	0.0760	0.3790	0.4550	0.3030
	CRR	0.20	0.0210	0.1050	0.1260	0.0840
	Cαε	0.20	0.0030	0.0152	0.0182	0.0122
	σvo'	0.05	8.75	174.5	183.25	165.75
	σp'	0.20	52.8	264	316.8	211.2
	H(clay)	0.10	0.4	4	4.4	3.6

The adopted MLV and SD values were evaluated from an extensive set of testing results, together with back-analysis results of settlement from both the surcharge PVD area and the wharf CDM area.

The results from Step (4) of the reliability assessment procedure are presented in Table 3.

Table 3 – Computed Settlement from MLV + SD and MLV – SD for various Parameters

Description	Symbol	Settlement (m)			
		S(MLV + SD)	S(MLV - SD)	ΔS/2	$(\Delta S/2)^2$
Applied stress	Δp	0.260	0.235	0.012	0.000154
No of log time cycles for creep calculation	Ncreep	0.266	0.230	0.018	0.000325
CDM treated zone Untreated bottom 1.5m thickness of Layer 8	D'eq	0.238	0.261	0.011	0.000131
	H(CDM)	0.249	0.247	0.001	0.000001
	CR	0.250	0.246	0.002	0.000004
	CRR	0.251	0.244	0.004	0.000014
	Cαε	0.253	0.242	0.006	0.000035
	σvo'	0.254	0.240	0.007	0.000049
	σp'	0.243	0.304	0.031	0.000939
	H_c	0.248	0.248	0.000	0.000000
Untreated clay layer 9	CR	0.253	0.242	0.005	0.000030
	CRR	0.257	0.238	0.009	0.000086
	Cαε	0.260	0.235	0.012	0.000148
	σvo'	0.261	0.234	0.013	0.000173
	σp'	0.236	0.354	0.059	0.003475
	H_c	0.261	0.234	0.013	0.000180
			Standard Deviation		**0.076**

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

The combined SD on S_{MLV} calculated using Equation 2 is 0.076m (i.e. 76mm), giving a coefficient of variation CV_{MLV} of 76/250 = 0.3 (or 30%).

From Steps 7 and 8 of the reliability assessment procedure, the probability of exceeding a particular multiple of S_{MLV} (i.e. Settlement Ratio, SR) has been computed and shown in Figure 2 below:

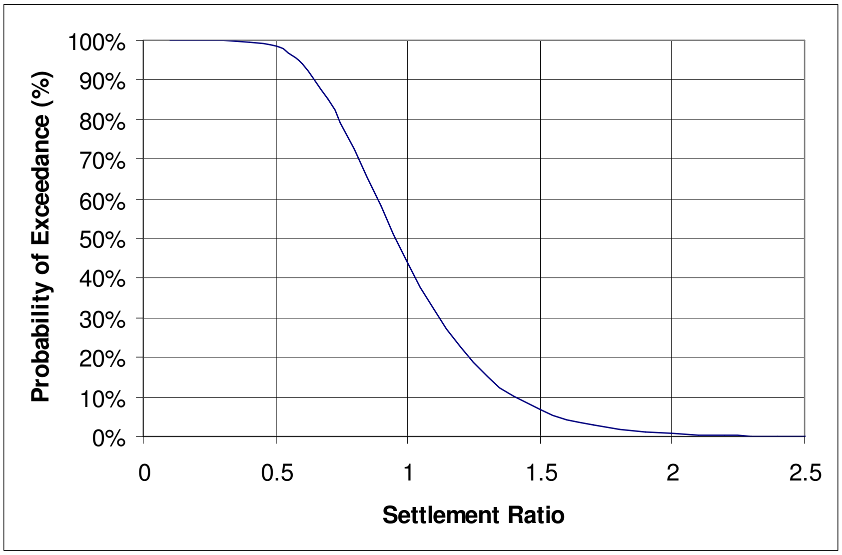

Figure 2 – Computed Probability Distribution of Settlement Estimate

For ground improvement design to limit settlement and differential settlement of civil infrastructures, a confidence level of 95% is generally considered to be a stringent design requirement. From Figure 2, it can be seen that there is only a 5% probability that the actual settlement would exceed the most likely estimate of 250mm by more than 1.6 times.

Assuming that the wharf CDM treated area would settle 60mm as estimated, and the settlement could be as much as 400mm (i.e. 1.6 x 250) in 20 years, the ground slope would reduce from the initial slope of 2.1% to about 1% which meets the specified minimum slope of 0.7% for drainage. Even if the wharf CDM treated area does not settle, a minimum slope of 0.7% in 20 years would be met.

7. Post-construction performance

Unfortunately, the author was unable to obtain any monitoring data from our client on this project post-construction. However, verbal information from our client is that the container terminal is performing satisfactorily to date. Plate 1 shows the pavement condition during operation of the container terminal.

Case History (continued)

A Case Study on the Design of Transition Zone for Cement Deep Mixing for a Port Reclamation Project

Plate 1
Finished pavement surface during container handling

8. Conclusion

Different ground stiffness between the main container storage yard and the wharf area of this container terminal project was caused by different ground treatment adopted. The main container storage yard was treated using surcharge with PVD while the wharf area was treated using CDM with fully penetrating columns to RL-35m. This situation presented a significant challenge in the design of the transition zone between these two areas to meet the differential settlement criteria for serviceability of the container handling equipment, and surface drainage.

A 30m wide stepped CDM zone together with setting the initial ground slope upwards to the landside provided a satisfactory solution to the challenge. The use of the simple reliability assessment procedure described by Duncan (2000) provided a useful quantification of possible uncertainties, and provided confidence to the client that the adopted solution is sound, and has enough built-in safety to cater for potential uncertainties in material properties and design assumptions.

9. REFERENCES

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Case History
Geotechnical offshore site investigation and reclamation design at Port Kembla

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Tonilee Andrews, Port Kembla Port Corporation, Port Kembla

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 at 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.

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.

Case History
Geotechnical offshore site investigation and reclamation design at Port Kembla

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).

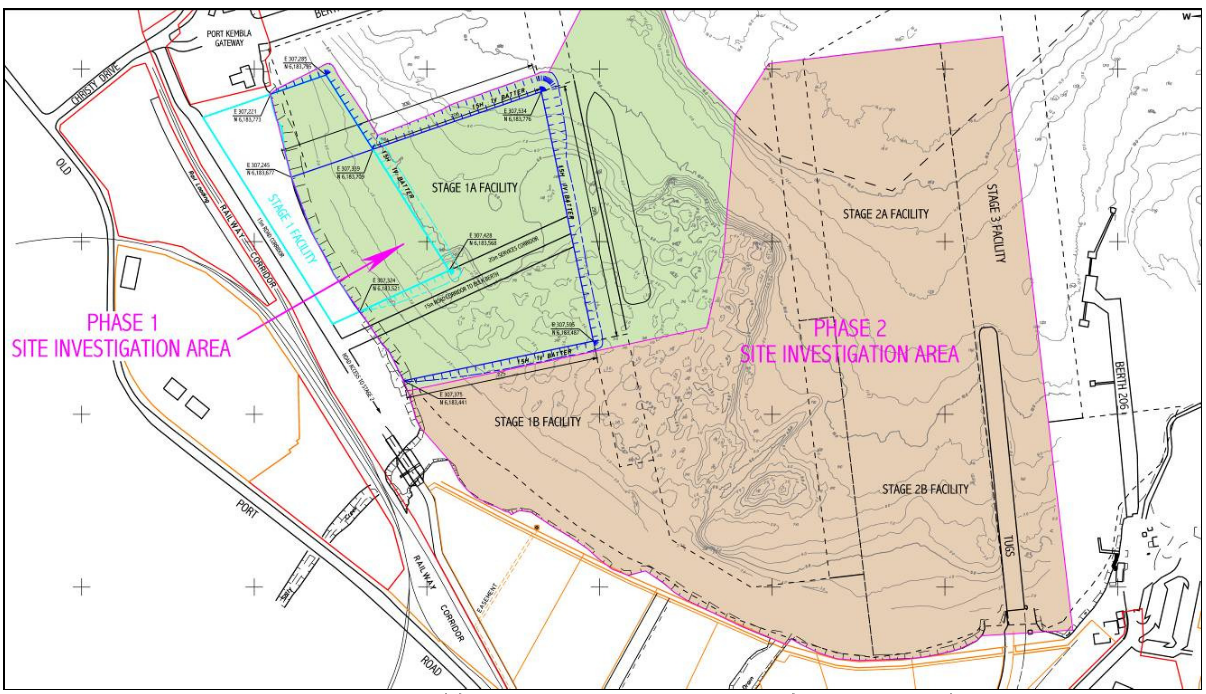

Figure 1: Port Kembla Port Corporation Outer Harbour master plan

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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 bunds 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

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 1.0m 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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

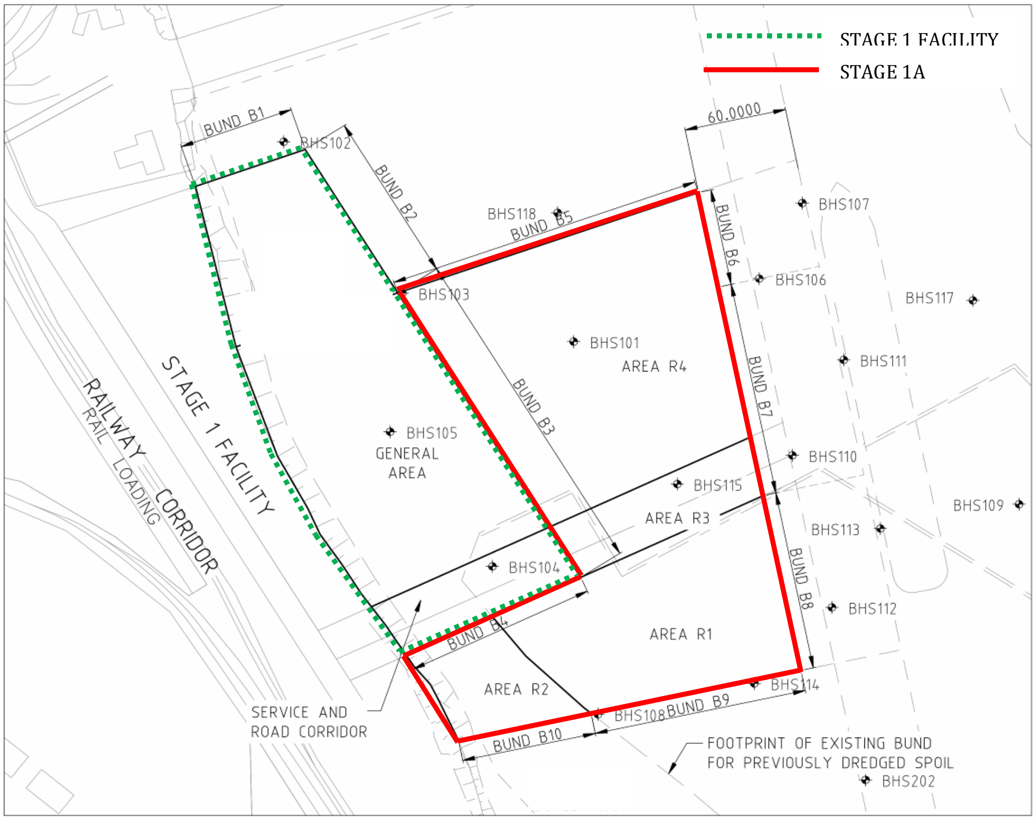

Figure 2: Geotechnical investigation location plan for Phase 1 investigations

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 underlies the majority of the Stage 1A area and generally thickens 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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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 artifacts 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.
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 Ggeotechnical 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)

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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	0.013	10	-
1b	Fill (granular)	16	-	0	30	-	-	-	-	7
2 / 3a	Marine estuarine Sediment Quaternary alluvium (cohesive)	17	10	0	22	0.250	0.025	0.013	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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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 (ie. 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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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

6.1.1 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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

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.

Case History (Continued)
Geotechnical offshore site investigation and reclamation design at Port Kembla

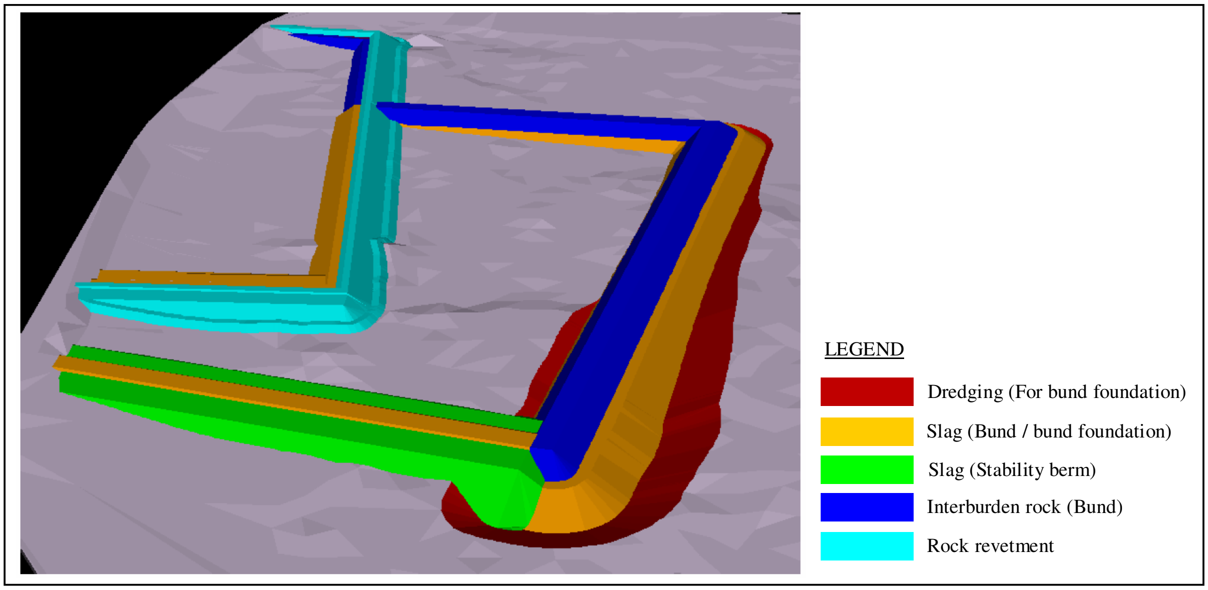

Figure 3: Perspective view showing completed Stage 1 and 1A bunds

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:

Case History (Continued)
Geotechnical offshore site investigation and reclamation design at Port Kembla

- 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.

Case History (Continued)

Geotechnical offshore site investigation and reclamation design at Port Kembla

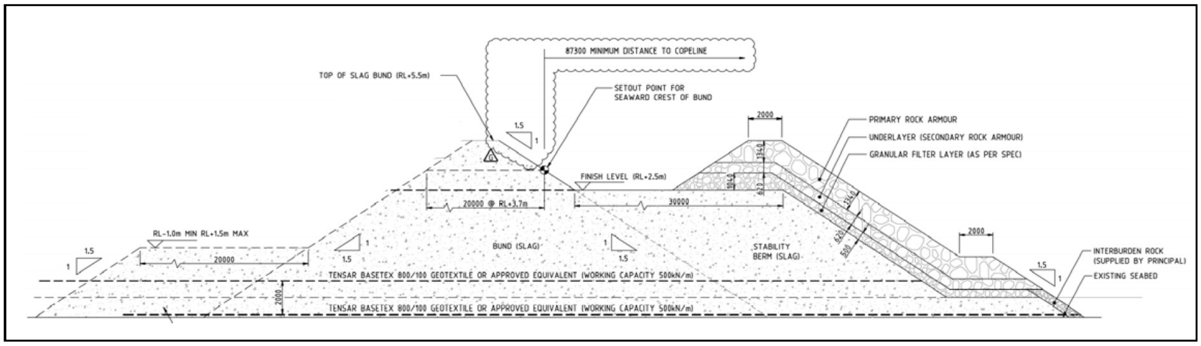

Figure 4: Typical section of bund for proposed variation design

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.

8. 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.

Case History (Continued)
Geotechnical offshore site investigation and reclamation design at Port Kembla

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.

9. REFERENCES

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