Advanced Geotechnical Modelling and Monitoring for the Port Coogee Project

Michael Hillman[1], Eric Y. S. Tam[1], Richard Mocke[2] and Bruce Keay[3]
[1] Coffey Geotechnics, Perth, Western Australia
[2]JFA Consultants (seconded to SKM), Fremantle, Western Australia
[3]Sinclair Knight Merz, Perth, Western Australia

Abstract
The Port Coogee is a premier land and marina development being undertaken on Cockburn Sound south of Fremantle, Western Australia, by Australand Holdings Pty, Ltd. Sinclair Knight Merz (SKM) are project manager and civil engineer for the project and Coffey Geotechnics Pty Ltd the project's geotechnical consultants. As part of the marina development, extensive geotechnical modelling and monitoring has taken place, with some parts of the development being located in excess of 8m of water depth, requiring careful consideration of edge wall stability and land reclamation techniques.

Detailed geotechnical stability modelling has produced an edge wall design for the project requiring a combination of stability requirements, depending on the water depth, intended loading conditions and geotechnical properties of reclamation materials. For the reclamation, a detailed programme of settlement and testing methods has been undertaken to monitor the results of high impact rolling of the reclamation areas, giving much greater confidence in the assessment of long term stability of the proposed development areas.

1 Introduction

The Port Coogee development is situated on the shores of Cockburn Sound about 18 kilometres south-west of the Perth CBD and five kilometres south of the port city of Fremantle.

Considered one of the biggest coastal renewal projects ever undertaken in Australia, the $900 million project will see the creation of a world-class marina, marina village and a mix of residential and commercial developments at the site (Figure 1).

The project has involved significant land reclamation in the marine environment, in up to 8m of water depth for the development of waterside commercial and residential lots. Geotechnical design for reclamation areas has involved two major considerations:

- The marina edge walls/ revetment walls were required to be designed with adequate Factor of Safety for the possible modes of failure.
- Limiting post earthworks settlement within areas on which residential structures were to be placed and providing advice on appropriate footing configurations for the conditions created.

In this paper, we discuss the design approach in addressing these considerations.

2 Revetment Stability

2.1 Approach

The primary design approach to revetment stability required analyses for slip circle type failure. Checks run for sliding and overturning of upper wall elements, are not discussed in this paper.

2.2 Wall Geometry

Slope stability analyses for the Port Coogee Marina revetment walls commenced in August 2005. During the subsequent year, alternative designs were considered, influenced by issues of constructability for the water depths required.

A number of wall types were proposed, to address different loading conditions. For example a more robust wall type is required in deeper waters, where residential loading is proposed at relatively short set back from the wall, than for a wall in shallow waters backed by Public Open Space. For the purposes of this paper, only one wall type (Wall Type "C") is discussed. This wall type accommodates most of the load conditions envisaged. The design for other wall types followed similar procedures to those adopted for Wall Type "C". Figure 2 illustrates a typical cross section.

2.3 Design Criteria

The stability of slopes is influenced by the structural loading by water levels and also by earthquake forces. For the initial phases of modelling ("preliminary analyses"), four loading conditions were examined as set out in Table 1. The factors of safety applicable to the rock revetment wall ("Shallow" slip circles) differ from those adopted for a slip circle that intersects a proposed residential structures ("Deep" slip circles).

Water level differences for "normal loading" were based on low tide records. The "rapid drawdown" case refers to the falling water levels after an extreme storm surge. The Tsunami case was based on water levels recorded during the 2004 Boxing Day Tsunami.

For earthquake loading, it was assumed that the water level would be at mid tide. The peak horizontal ground acceleration was derived from the 500 year average recurrence interval peak bedrock acceleration. In accordance with the practice for retaining wall design, the bedrock acceleration was multiplied by a site amplification factor (estimated to be 1.25) and the product halved to allow for inertia effects with a rock & soil slope.

Table 1 - Design Minimum Factors of Safety

Design Condition	Shallow Slip Circle	Deep Slip Circle
Normal Tide	1.35	1.5
Earthquake	1.1	1.15
Rapid Draw Down	1.25	1.35
Tsunami	1.1	1.15

It should be noted that the selection of a potential failure profile for the Shallow Slip Circle cases, is open to interpretation. An "infinite radius" slip circle zone may have the lowest theoretical factor of safety, but ignores the size of the rock "particles", and the higher strength anticipated from the armour face given little confining load.

In view of the above, slip circles selected for the shallow slip circle case penetrate the rock fill bund, and are not confined to the armor rock alone.

2.4 Results of Preliminary Analyses

Extensive analyses were undertaken for all the above loading cases during early stages of wall design.

The analyses consistently demonstrated that the critical design criterion was to achieve the FOS required for a "normal" tide loading condition (occurring at low tide, when submergence is at a minimum). Subsequent analyses have concentrated on this load condition.

The original design under which these earlier analyses were based, differs in a number of aspects from the design currently adopted and illustrated in Figure 2. The primary design difference arises from constructability issues, and involves the placement of a rock fill bund to provide a stable platform against which General Fill (GF) is placed subaqueously (see Figure 3), rather than attempting to place a filter rock against a GF embankment. Further consideration was given to strength parameters assigned, to the design of the geogrid, and in ignoring any contribution to stability from the lateral resistance provided by piling.

2.5 Adopted Geotechnical Model

It was not considered practical to undertake laboratory scale test work to establish strength parameters for rock armour and rock fill material. Consideration was given to attempting to measure the angle of repose as a design friction angle. This proved unsuccessful because of tidal and wave surge in the trial area, and constraints on testing within a site where construction activities were on-going. Strength parameters were therefore adopted based on a research of published data, with allowances for differences in strength for material placed subaqueously (and therefore comparatively loose), in comparison with the strength parameters for material placed above the water table.

High Energy Impact Compaction (HEIC) was undertaken to improve subsurface conditions (discussed later in this paper, also see Figure 4) and Cone Penetrometer (CPT) probing undertaken to assess the improved condition.

Strength parameters adopted for the numerical modelling are summarised in Table 2.

Table 2 Adopted Geotechnical Parameters

SOIL/ ROCK UNIT	Unit Weight (kN/m^3)	Friction Angle (degrees)
Natural Sand	17	30
General Fill: < -2m AHD	17	32
General Fill: -2m AHD to +0.25m AHD	18	35
General Fill: >+0.25m AHD	18	38
Rock Fill: <-2m AHD	19	37
Rock Fill: >-2m AHD	19	40
Armour	20	45

2.6 Analyses

Analyses were undertaken using the proprietary program SLOPE/W distributed by Geo-Slope International.

For the final design, a wall geometry was selected that it was agreed with the contractor represented the flattest overall slope that could be constructed by the equipment he was able to mobilise for this project. Given this external geometry, then three parameters remained that could be varied. These were:

- Rock armour thickness, which influenced the FOS for the "shallow" slip circle, but had little influence on the "deep" slip circle.

- Provision of a toe berm placed as a separate construction activity to the wall building. This toe berm was required to both reduce the length of the slope within which a shallow slip could develop, but more importantly, to contribute to the overall FOS for a deep slip circle failure mode.
- Provision of a geogrid providing lateral restraint for a deep slip circle failure mode, to improve the overall FOS where structures may be at risk.

Figure 5 illustrates an example of the slope stability output with geogrid and toe berm in place.

2.7 Design outcomes

2.7.1 Rock Armour Thickness

The extent to which armour rock emerges above normal low tide level as well as the length and gradient of the slope has a significant influence on the shallow slip circle failure mode, because of buoyancy effects (higher effective weight above water). This observation was significant in comparing other possible wall geometries (and other adopted Wall Types) to the final Wall Type C configuration.

2.7.2 Toe berm

The height of toe berm limits the length of revetment slope (shallow slip circle failure mode). The weight, the strength and the lateral extent (length) of the toe berm contributes significantly to the FOS for the deep slip circle failure mode, by extending the failure circle and weighting the toe. The sizing of Toe Berm for the deep water (deeper than -4.5m AHD) takes into account the contribution of geogrid to stability.

Toe berms were not considered for shallow water depths. A design with geogrid does not require them, and they would interfere with navigation. It was considered that toe berms above -3m AHD would provide an unacceptable risk to boating in the marina.

2.7.3 Geogrid

The design FOS is increased from 1.35 to 1.5 where slip circle failure would put residential structures at risk (see Table 1). Geogrids were considered a cost effective means of improving the FOS. The selected geogrid was placed close to the base of the vertical wall element and near the crest of the revetment, and was extended from that wall element to anchor in soil beneath the proposed residences (nominally 9m in total length).

The type geogrid proposed for water depths in excess of 4m, was a single layer developing more than 50kN/m resistance over the design life of a residence (nominally 50 years). It was noted that such failure would represent an "ultimate" design event (strain leading to failure can be large).

3 Ground Improvement

3.1 Placement of Fill

The reclamation method involves pushing fill out by dozer from the shoreline over previously placed fill. Initially a rock fill bund is extended, and this is followed by the placement of general fill behind the bund to +0.5mAHD. Ground improvement using the High Energy Impact Compaction (HEIC) was undertaken at about +0.5m AHD. Subsequent filling is taking place as an engineered fill under normal engineering constraints to Final Earthworks Level at +3.3m AHD.

3.2 High Energy Impact Compaction (HEIC)

3.2.1 HEIC Trials

A trial of HEIC was carried out in the initial northern reclamation area of the marina. The reclaimed area was approximately rectangular in shape with approximate plan dimensions of 300m length by 100m width.

The HEIC was carried out by specialist contractor, Broons Hire Pty Ltd, using a 'Square' impact roller of model BH-1300HD with 12 tonnes of module weight, towed by a CASE tractor at an average speed of 10-12 km per hour (see Figure 4). As the fill platform was about 0.8m above water level at the time of the work, the HEIC was carried out in a dry condition.

3.2.2 CPT Testing and Ground Settlement Monitoring

CPT was carried out using a combination of mechanical cone and electric friction cone penetrometers. Fifteen (15) monitoring points were established for CPT testing over the first area of reclamation (100m x 300m). The testing was carried out between 7 November 2006 and 10 January 2007.

Pre-HEIC readings were undertaken to provide baseline data, and subsequent CPT testing was undertaken at the completion of 5 passes, 10 passes, 20 passes and 30 passes, with 40 passes being applied over areas of deeper waters.

The results indicated that the fill prior to HEIC contains 'weak' (loose) pockets. Application of HEIC does not noticeably improve already adequately compacted fill with q_c > 10MPa, but does appear to improving the weaker zones probably to depths exceeding 4m below the test level. A minimum CPT cone resistance q_c of about 5 MPa has been achieved to this depth. The magnitude of improvement in q_c is noticeable between 10 passes and 30 passes, but there appeared to be little improvement beyond 30 passes. A typical CPT combined plot of cone resistance versus depths of monitoring point R15 at the deeper water section is shown in Figure 6.

During the same period, ground surface settlement monitoring was carried out and the survey levels interpreted using SURFER contouring and data analysis software. A ground settlement of up to 100mm was observed in the general reclamation area between the pre impact rolling and post 30 passes impact rolling.

Earthworks are continuing based on a requirement of 30 passes using the HEIC.

4 Estimates of Settlement

4.1 Preliminary Assessment

The issue of settlement in subaqueously placed fill was investigated by the project manager (Sinclair Knight Merz) on a similar in-shore development in Jervoise Bay, Western Australia, involving similar reclamation techniques (though slightly deeper waters). HEIC was undertaken on that project.

The testing at the Jervoise site was undertaken over a trial area about 70m in length and 40 m in width, instrumented with 17 settlement monitoring plates. The settlement plates were installed at the final earthworks level of about 4m AHD, in an area with subaqueous deposition some 10m thick (a total fill depth of about 14 m). The natural soil profile below sea floor comprised up to 3m of loose sand over limestone. A surcharge load comprising 4m of rubble fill was placed over and above final earthworks level for a 3 month period.

Within the centre of the group of plates (i.e. removed from edge effects), settlements ranged between 54mm and 93mm, indicating a differential settlement due to variability in the subsurface profile alone (fill and loose sand) alone, of almost half the maximum settlement. Most of the settlement had occurred within days of load application with negligible noticeable settlement after one month under constant load.

The authors were unable to locate any CPT records for the trial site. Given its close proximity to the Port Coogee site, and the same geological environment, use has been made of the Port Coogee CPT data in deep water to assess the likely distribution of modulus values through the subaqueous profile into the loose sand overlying limestone.

The typical profile that is likely to have existed is set out in Table 3.

Elastic modulus E is frequently derived from CPT cone resistance q_c based on a linear relationship with a constant of proportionality "a" ($a=E/q_c$) ranging from 1.5 to 8 (e.g. Sutcliffe and Waterton 1983).

Table 3 Typical Offshore Subsurface Profile

Depth Interval	CPT Cone Resistance "q_c"	COMMENT
Sea level to -2m AHD	10MPa	
-2m AHD to Seabed	5MPa	
Subseabed loose sand	1MPa	Nominally 0m to 3m thickness
Limestone subcrop	10MPa	Allow for some karstic activity
Limestone Deep	>30Mpa	

Work undertaken on the investigation of the settlement of tanks in Kwinana immediately to the south of Port Coogee and on similar geotechnical conditions (Hillman, Cocks and Szymakowski (1999)), provided considerable support to this simple model, noting however:

- The ratio of stiffness E to cone resistance q_c was dependent on prior stress history. It was some 5 times less below a threshold stress, than the rate of increase in stiffness above that stress (this threshold is likely to be related to preconsolidation / aging/ cementing effects – refer Fahey et al 2003); and
- The ratio decreases with low values of q_c.

A bimodal relationship was presented in Hillman et al 1999 as follows:

$$E = a \times q_c \times \{(q_c^3+1)/(q_c^3+6)\} \quad \text{-------} \; (1)$$

Where E = Elastic Modulus (MPa)

q_c = CPT cone resistance (MPa)

"a" is a dimensionless constant of proportionality dependent on stress history, with a = 3 where there is no surcharge, aging or cementing effects.

Back analysis of the observed settlements for Jervois has been undertaken using the Sydney University programme FLEA (finite layer elastic analysis) to derive a representative value "a" for the maximum observed settlement of 93mm.

The following assumptions were made in the analysis:

- The surcharge load applied by 4m of loose rubbly surcharge is approximately 76kPa.
- Stiffness of engineered fill above the water table was E = 50MPa.

- Stiffness of deeper limestone was E > 120MPa.
- Poisson's ratio for all layers was 0.3
- The loose sand zone and the fill shows minimal surcharge preloading effects (that might arise for example as a result of compactive effort above sea level).

The results of the back analysis demonstrated a fit for a constant of proportionality a = 4. This figure is higher than the value presented in Hillman et al (1999) for a soil that has not been surcharged, and may reflect some surcharge effect from the HEIC and from compaction of fill to final earthworks level.

4.2 Port Coogee Settlement Estimates

Equation (1) has been used to derive settlement estimates for residential loading over the Port Coogee profile. A continuous strip of housing of about 20m depth has been assumed for the modelling, and a uniformly distributed load of about 25kPa has been adopted, which is less than the load adopted in edge wall stability calculations, but allows for the separation likely to exist between individual residences. The constant of proportionality used is a=4

The estimated settlement based on these modulus values and using FLEA is less than 25mm in the deepest water, reducing in shallower waters. This settlement is anticipated to occur within weeks of application of load. The estimate assumes a deep loose sand profile over which the fill has been placed (up to 3m thick, based on probing to-date). However for much of the site the loose sand is very thin or absent, and therefore settlements in such areas would be expected to be significantly less than the above. Re-analysis of the settlement model for a weak limestone layer in place of the sand suggests a total settlement of the order of half the value that applies with loose sand. Therefore differential settlements associated with variability in geotechnical conditions have been estimated at half maximum settlements – and this is similar to the range in settlements observed in the Jervois trial.

Final settlement estimates for individual Lots will be based on the CPT probe results applicable to the Lots on which they are taken.

4.3 Long Term Settlement

The above settlements are in effect "immediate" settlements. Long term creep can be anticipated in calcareous sands. For example long term creep settlement for large tanks in the Kwinana Industrial area, over a 50 year life, were of the order of 60% of initial settlement (Hillman et al (1999)).

The soils at this site are not stressed to the magnitude of stress that applies under the Kwinana tanks, and the anticipated strains are comparatively small (almost an order of magnitude lower). As noted by Fahey (2003), the stiffness response of a soil is strain dependent. Nevertheless, the Port Coogee soils are subject to tidal buoyancy effects (small load / unload cycles), and can be expected to show some creep settlement. Long term settlement estimates have made allowance for a 50 year life of a residence, of 35% of initial settlement.

5 Monitoring of Ground Movements

In order to provide factual data against which to compare settlement estimates, the developer has installed settlement monitoring plates at the level at which HEIC was undertaken (about +0.5m AHD) and will be recording the settlements that arise as filling proceeds to final earthworks level of +3.3m AHD. The success of this monitoring programme is still to be tested, as filling has not (at the time of preparation of this paper) been completed.

Other monitoring being undertaken by the developer includes the establishment of a network of survey "pins" along the rock core edges to monitor for settlement of the rock fill bund as a result of both short term (self weight and compaction of the adjacent reclamation areas) and long term creep (again, with the action of self weight and tidal surging). The monitoring will be undertaken in x, y, z directions.

6 Conclusions

Extensive geotechnical testing, design, and numerical modelling has already been undertaken for the edge walls and reclamation areas of the Port Coogee development.

Construction is now underway and test work and monitoring will continue to monitor edge stability and to provide factual data against which settlements for individual residential Lots can be based.

7 References

Fahey M, Lehane B and Stewart D (2003) "Stiffness for Shallow Foundation Design in Perth CBD" *Australian Geomechanics,* Volume 38 No 3

Hillman M.O., Cocks G.C., and Szymakowski J.B. (1999) "Tank Settlements on Calcareous Soils, Kwinana, Western Australia" *Australian Geomechanics*, Volume 34, No 3

Sutcliffe G and Waterton C (1983) "Quasi-static Penetration Testing" in *In-situ Testing for Geotechnical Investigations,* M.C.Ervin (ed)

8 Acknowledgements

The authors would like to sincerely thank Australand Pty Ltd and, in particular, Mr Richard Fulcher, Australand Project Manager for the Port Coogee Project for their generous support and permission to publish this paper.

Figure 1: Port Coogee – Development Layout

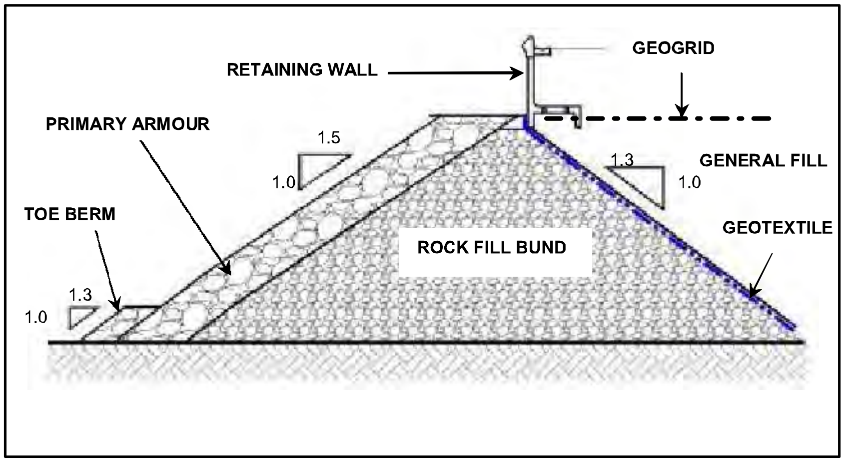

Figure 2: Schematic of Wall Type C cross section

Figure 3: Placement of rock core bund

Figure 4: High Impact Rolling

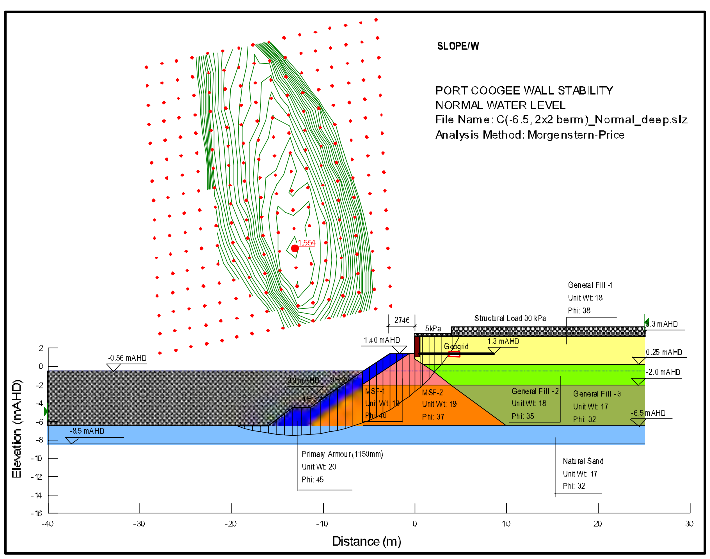

Figure 5: Typical Output of Slope Stability Analysis

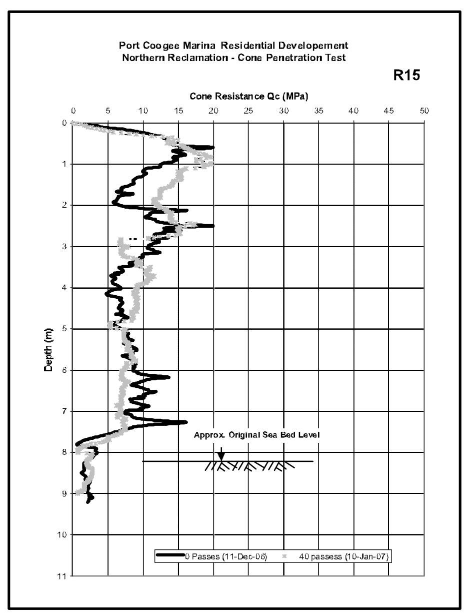

Figure 6: Typical CPT Combined Plot.