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Cockatoo Island stage 3: seawall failure and remediation

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Abstract

The hematite mining operation at Cockatoo Island, Western Australia, required the construction of a 13 m high, composite earth and rockfill seawall to exclude 10 m tides from the Indian Ocean. The Stage 3 seawall is underlain by up to 30 m of soft, low permeability coralline sediments which are in turn underlain by stronger and higher permeability marine sediments and hematite scree layers. During initial construction, a 140 m section of the seawall failed when a height of 8.7 m was reached.

This paper describes the post failure investigation and analysis results, and the remedial works which were successfully completed. The investigation results indicated the basal drainage characteristics of the coralline sediments to be variable along the length of the seawall. At the location of the failure, basal drainage was very low and this was considered to be a major contributing factor to the failure coupled with rapid construction of the embankment. Instrumentation and monitoring prior to the failure was limited, partly due to difficulties associated with high tidal fluctuations. In contrast, the remedial work was carried out successfully with a significantly improved instrumentation and monitoring system, including inclinometers, extensometers, piezometers, settlement plates, survey prisms and total pressure cells. A rigorous review and approval process was developed in conjunction with the client, Cockatoo Mining, using an observational method to assess the risk of instability prior to placing each additional embankment layer.

The degree of client involvement and the collaborative approach adopted in relation to the observational method, approvals, data transfer, and joint management for the remedial work is extremely rare on most mines. The successful completion of the remedial work demonstrated that such a collaborative approach is a key ingredient to the successful completion of high risk projects in complex ground conditions.

1 Introduction

Cockatoo Island is situated in the Kimberley Region of Western Australia, approximately 1,900 km from the city of Perth as shown in Figure 1. The hematite mine is located on the southern edge of the island.

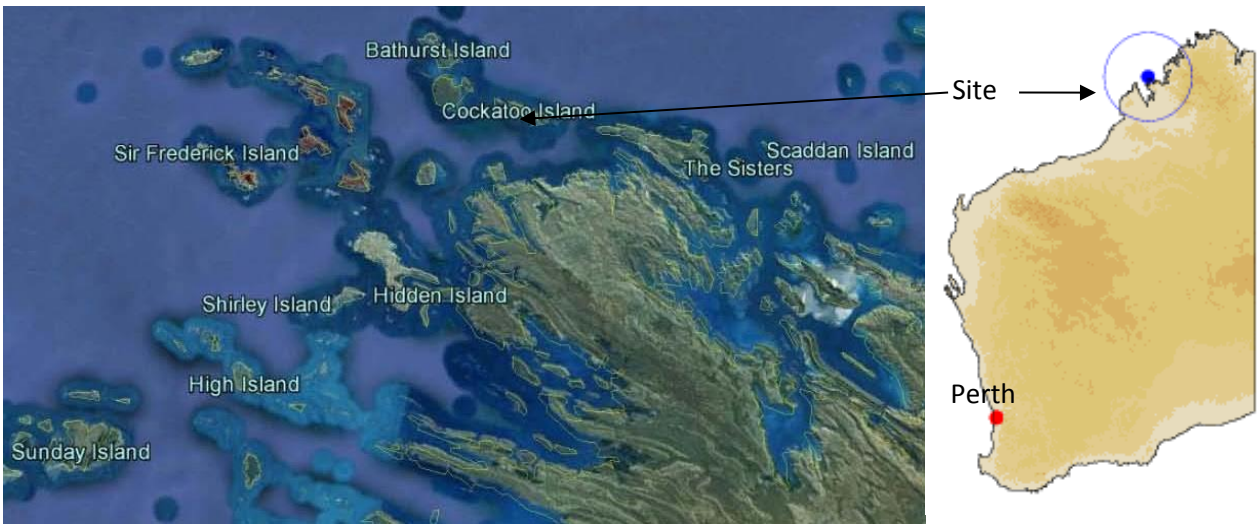


Figure 1 Site location

The Cockatoo Island Operations were a joint venture between HWE Mining Pty Ltd (HWE) and Cliffs Natural Resources Pty. Ltd. (Cliffs), and were carried out in a number of stages extending from west to east along the southern coastline of the Island. Until late 2008 iron ore was extracted from the Stage 1 and 2 open pits, predominantly from below sea level behind a zoned earth fill embankment seawall. Construction of a Stage 3 extension to the existing Stage 1/Stage 2 seawall was necessary to allow continuation of mining into the Stage 3 orebody.

2 Site conditions and background of failures

The site of the seawall was located about 50 m offshore, which had a seabed level at approximately RL 0 m Cockatoo Island Data (CID). The site was underlain mainly by a thick (20 to 30 m) layer of soft coralline sediment (silty clay with coral nodules) beneath a thin (1 to 2 m) layer of beach sand and coral debris as shown in Figure 2. The coralline sediment has the behaviour of soft clay, having relatively low permeability, with its thickness increasing seawards. The coralline sediment was underlain by marine sediments or hematite scree layer (landward side); these layers have higher permeability than the overlying coralline sediment, and were originally expected to provide basal drainage during the consolidation process (see Section 3 for further discussions). An important aspect of the site was that it has a maximum tidal range of about 11 m ranging from RL 0 to 11 m.

The original seawall design comprised a rockfill embankment with armour rock on the seaward batter and a central clay core. However, due to the lack of suitable material on site that would meet the core specification, a steel sheetpile was designed to be installed along the central core centreline when the embankment construction reached RL 10 m. The embankment had design batters of 2.5H:1V on the seawards side and 2H:1V on the landward side, with staged construction to allow consolidation of the underlying soft coralline sediment, and sufficient strength gain in the foundation to maintain stability during construction. The position of the seawall was driven by a limited offshore lease boundary which governed the seaward toe extent, and the need to minimise the footprint on the landward side to enable maximum extraction of the hematite ore. Whilst embankment stability during construction was important, long-term stability of the seawall was just as critical because the seawall eventually formed the barrier against the ocean to enable open pit mining to be conducted safely to RL -50 m. Factors of Safety of 1.3 and 1.5 were chosen as design values for the construction case and long-term case respectively. A typical geological profile of the site, proposed geometry of the seawall and the proposed mining operation is presented in Figure 2.

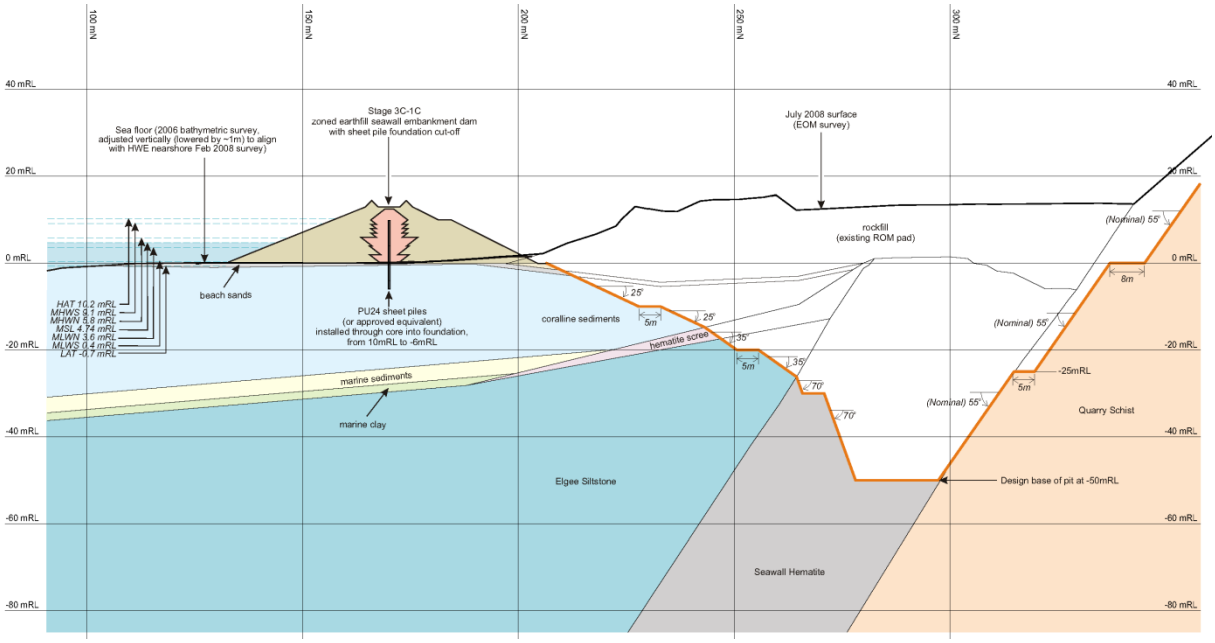


Figure 2 Geological section and proposed seawall and mining profile

The design profile of the Stage 3 seawall was essentially the same as that for Stages 1 and 2. The construction methodology was also similar, comprising initial end dumping of rockfill to RL 4 m, excavation for the core trench and placement of core material during low tides. Time windows for construction during the low tides were typically restricted to two to four hours, and placement of core material in wet conditions often proved difficult. A description of the Stage 2 seawall construction was given by Phillips et al. (2004) who also reported that a deep seated rotational failure of about 100 m length occurred seawards during construction near the eastern end of the Stage 2 seawall (i.e. close to the western end of the Stage 3 seawall). In fact, there were two such failures during construction of the Stage 2 seawall, with the larger one being about 130 m long further to the west as shown in Figure 3.

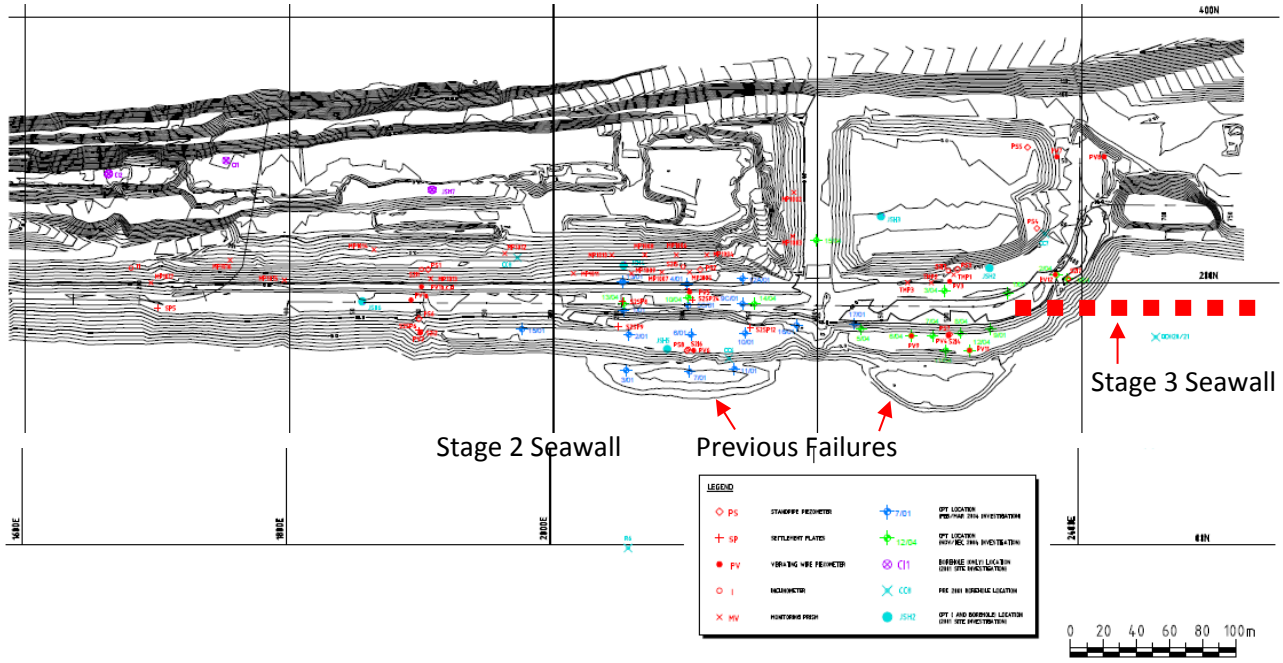


Figure 3 Plan showing previous stage 2 seawall failures

During construction of the Stage 3 seawall, a portion of the embankment of approximately 140 m in length between 2,500 and 2,640 mE failed on 24 April 2009 when the embankment crest reached an elevation of RL 8.7 m. The failure occurred soon after raising the height of the embankment crest and during low water spring tides. At low water spring tide (RL 1.3 m) on the date of the Stage 3 seawall failure nearly the full embankment height was above sea level. The fact that the embankments failed at low spring tide is not surprising, as the maximum stresses on the wall occur immediately after fill placement when the sea level is at its lowest.

Figure 4 shows two photographs taken in the area of the failed embankment, both clearly showing a rear scarp almost 6 m deep, and rotation of the failed embankment surface.

(a) (b)

Figure 4 (a) failed area looking west; (b) failed area looking east

A plan of the failed Stage 3 seawall is shown in Figure 5, showing a seaward movement of about 30 m. Tension cracking was observed to the immediate north and west of the failure, extending in a pattern generally sympathetic to the orientation of the back scarp and western edge of the failure.

Examination of the failure geometry suggested that the failure surface was approximately circular and deep seated, extending to below RL -15 m (based on post-failure investigations). Subsequent examination of the monitoring data indicated that the embankment failure occurred between 04:00 and 05:00 on 24 April 2009. It is possible that some settlement and tension cracking could have developed over a period of minutes and perhaps hours, prior to the main failure event, but there is little monitoring data to substantiate this as discussed in Section 3.2.

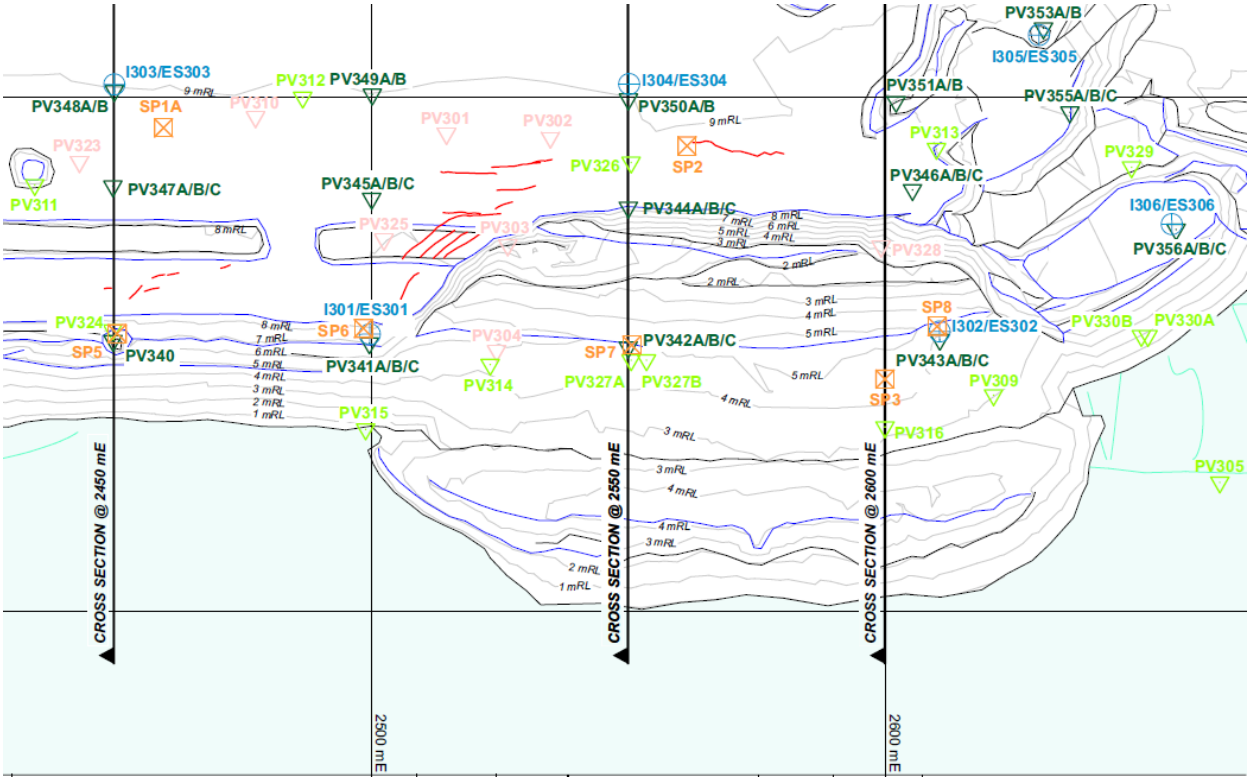


Figure 5 Plan of failed section of the Stage 3 seawall together with instrumentation locations; SP = settlement plates; PV = piezometers; I = inclinometers; ES = downhole extensometers

3 Pre-failure investigation and instrumentation

3.1 Pre-failure investigation

A reasonably detailed program of geotechnical investigations had been carried out for the design of the seawall. At the investigation locations, the combined thickness of the beach sand and coral debris layers was typically more than 3.5 m, with the underlying soft, coralline sediment layer extending to RL -25 m, i.e. well below the estimated depth of the failure.

Results of soil indices and measured ratios of undrained shear strength to in-situ vertical effective stress (S_u/σ_{vo}'), from laboratory staged triaxial consolidated undrained tests with pore pressure measurements, are presented in Figures 6(a) and (b) respectively. The test results above RL 0 m represent the placed core material while those below RL -5 m represent the coralline sediment. Figure 6(a) shows the in-situ moisture content of the coralline sediment to be generally close to the Liquid Limit (typically 45 to 60%). Figure 6(b) shows a reduction of S_u/σ_{vo}' from about 0.4 near the top of the coralline sediment layer, to about 0.25 at the bottom of the layer. The reduction of S_u/σ_{vo}' ratio with depth was inferred to be indicative of

over–consolidation near the top of the layer, and reaching a normally consolidated state at the base of the layer.

The measured friction angles from the staged triaxial tests are shown in Figure 6(c), showing friction angles typically more than 35° with the occasional test results between 27° and 30° for the coralline sediment.

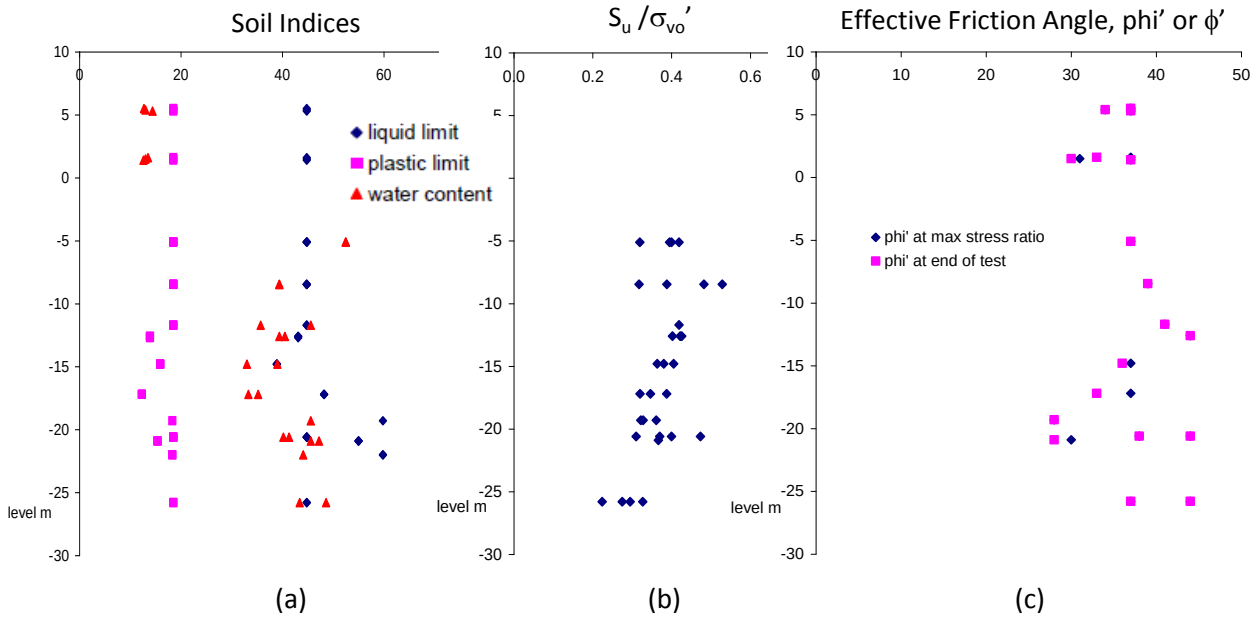


Figure 6 (a) soil indices; (b) S_u/σ_{vo}' ratios; (c) effective friction angle

Pre-failure instrumentation comprised the installation of settlement plates, piezometers, and six dual extensometers/inclinometers, installed when the seawall reached RL 5.2 to 6.8 m during January and March 2009.

Re-examination of the boreholes drilled as part of the original instrumentation program revealed the following features:

- Comparison of the original surface levels with the level of the base of fill intersected in the piezometer bores suggested that approximately 0.8 to 1.2 m of settlement had occurred beneath the embankment at the time of drilling (January to March, 2009) when the embankment construction reached RL 5.2 to 6.8 m.
- The hematite beach sands were typically about 1 m thick under the area of the failure but in some locations were thinner or not present.
- Beneath the hematite beach sands was a layer of coral reef debris of varied thickness. The coral reef debris was a distinct layer of both intact and broken coral reef within a matrix of coralline sediments. It was significant to note that within the failed area significantly less coralline reef debris was detected in some boreholes, in comparison to areas outside the failure where the reef debris encountered was consistently 1 to 2.5 m in thickness.

3.2 Pre-failure monitoring results

Due to the high tidal range at the site, a decision was made to provide casing protection to the extensometer/inclinometer with the collar installed at RL 10 m. As such, these instruments could not be read until the seawall reached a level high enough for access to be made, and as a result, no extensometer and inclinometer readings were made prior to the failure. Settlement plates were fitted with extension rods and prisms above the high tide level for measurement of lateral movements as well as settlement. However, the extension rods were often bumped during the filling process and also subjected to sway due to wind and wave actions, thus limiting their reliability.

Pore pressure measurements showed relatively fast rates of pore pressure dissipation in the piezometers installed in the upper part of the coralline sediment. However, within the failed area, relatively slow rates of pore pressure dissipation were observed in the deep piezometers. The pore pressure readings pre- and post-failure are further discussed below.

4 Post-failure investigations

4.1 Post-failure investigations

On 1 May 2009, seven days after the seawall failure, interim remediation was undertaken which comprised flattening of the failure back scarp and placement of rockfill protection over the core exposed at either end of the failure. A working platform was constructed at RL 4 m to facilitate post-failure investigations and allow additional instrumentation to be installed to replace damaged instrumentation. Post-failure site investigations included additional boreholes, cone penetration tests (CPT), dilatometer (DMT) and shear vane testing. The investigation results indicated that in the failed area, the total thickness of the beach sand and coral reef debris layer was relatively thin (1 to 2 m), and confirmed that the rotational slip surface extended down to about RL -15 m as shown in the inferred geological section presented in Figure 7.

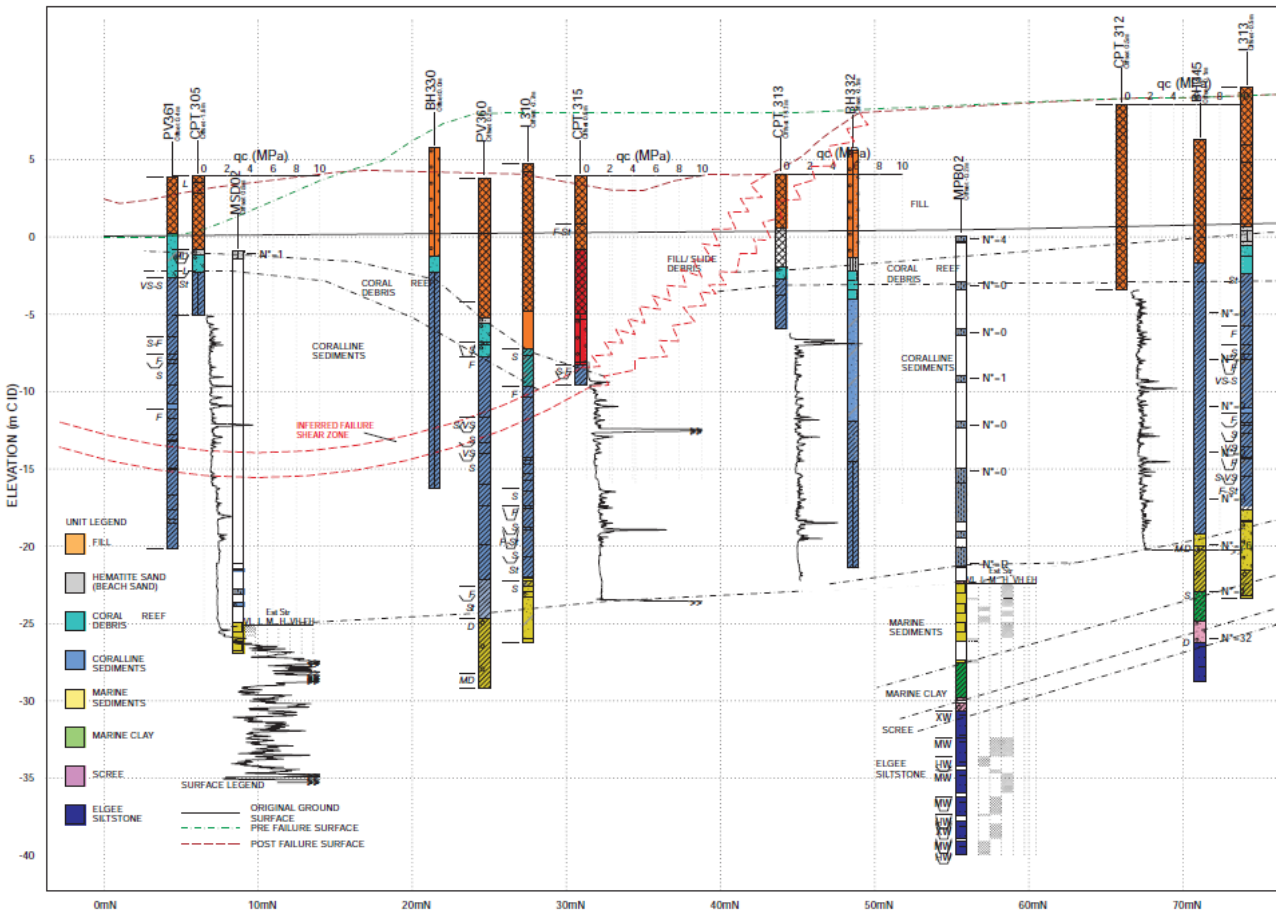
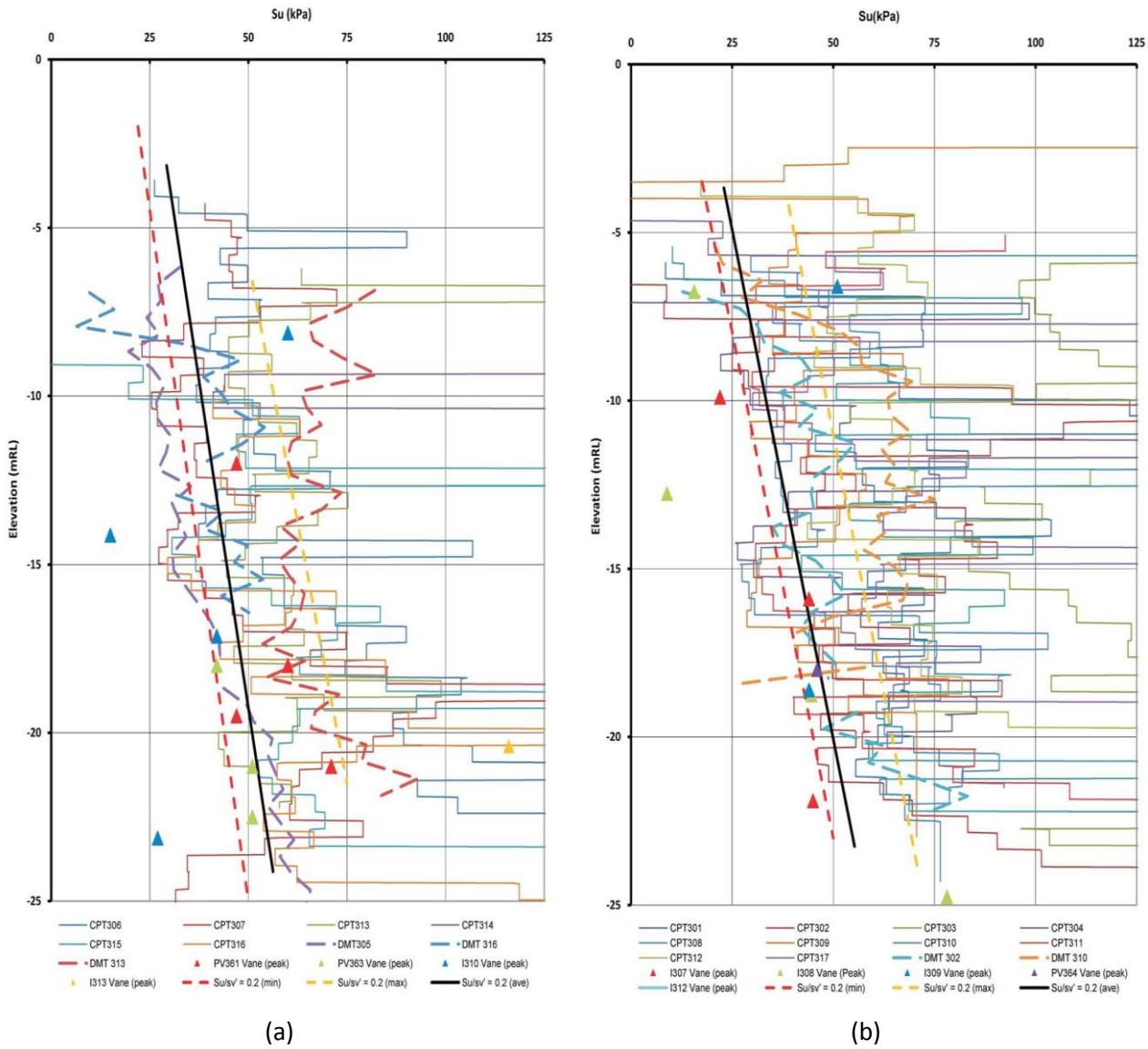


Figure 7 Interpretation of the post-failure subsurface profile within the failed area

Undrained shear strengths of the coralline sediments were measured from CPT and DMT tests with calibration using the vane shear test results as shown in Figure 8. There is a considerable scatter of results, with the areas outside the failed area having several (CPT interpreted) Su profiles that are higher than those within the failed area. This may be partly attributed to variability in material properties, but may also be attributed to some of the areas outside the failed area having different drainage properties and thus having experienced a greater degree of consolidation than within the failed area (see Section 4.2). However, there is little difference in the measured vane shear strength results at the lower range of

measured results. Therefore, when the embankment was of sufficient height to cause the coralline sediments to become normally consolidated, a common $S_u/\sigma_{vo'}$ ratio of 0.2 was adopted for the remedial works design. This ratio is consistent with the value of 0.21 quoted by Mesri (1975) and 0.21 ± 0.015 (for Plasticity Index >20%) quoted by Ladd and DeGroot (2003).



(a) (b)

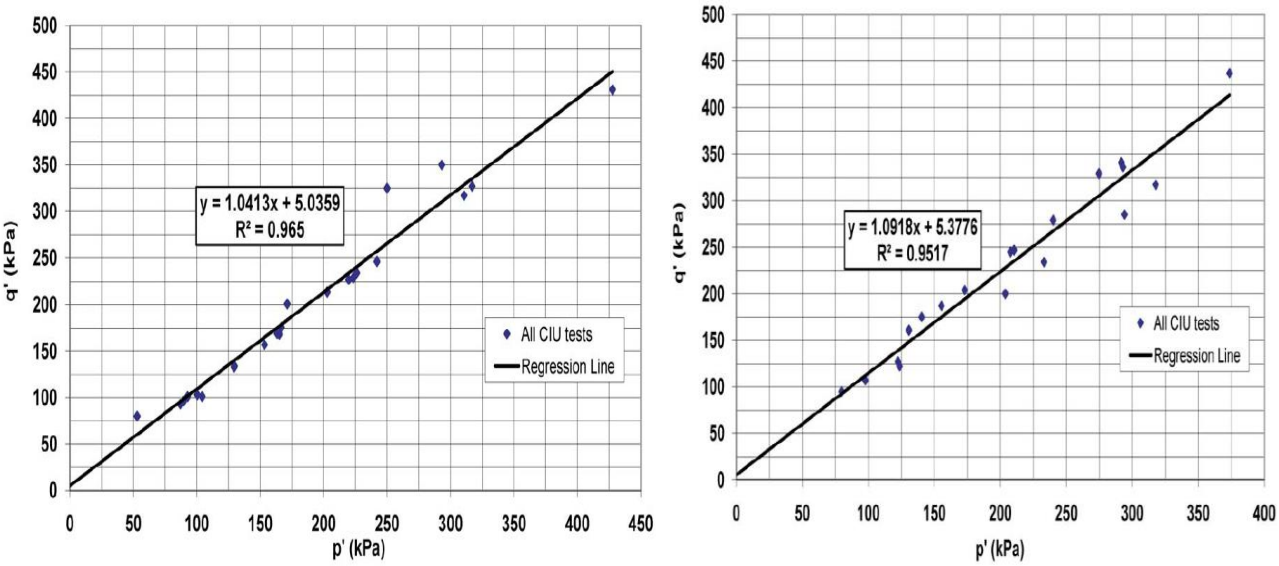
Figure 8 (a) measured S_u profile (failed area); (b) measured S_u profile (outside failed area)

A program of staged consolidated undrained triaxial tests with pore pressure measurements (CIU) was also conducted on recovered tube samples of coralline sediment. Fifteen of the samples were isotropically consolidated, with three K_o consolidated CIU tests. The individual CIU test results gave an effective cohesion ranging from 1 to 11 kPa and effective friction angle ranging from 24.5 to 29.5°.

Cambridge p'-q plots of CIU test results from samples recovered from within and outside of the failed area are presented in Figure 9. The lines of best fit to the data in Figure 9 result in a low cohesion. If it is assumed that the cohesion is zero, and the lines are drawn through zero, the effective friction angle, ϕ' becomes 23 to 32° for individual tests and an average of 29° when the two sets of tests were combined. The three K_o CIU tests gave an average effective friction angle of 30.9°.

These results are somewhat lower than the pre-failure investigation results shown in Figure 6(c). It is not clear whether the difference is caused by strain softening effects, or natural variability in the coralline sediment. To be prudent, it was decided to adopt a friction angle of 29° for the coralline sediment prior to

failure and outside the failed area. Within the failed area, a reduced residual friction angle of 25° was assessed using the methodology described in Khalili et al. (1996).



(a) (b)

Figure 9 (a) p'–q' plots (failed area); (b) p'–q' plots (outside failed area)

4.2 Post-failure back-analyses

After the failure, back-analyses were carried out using both a total stress method (slip circle limit equilibrium analysis using the computer program SLIDE (Rocscience Inc., 2013)) and an effective stress method (coupled consolidation/stability analysis using the finite element analysis computer program PLAXIS (Plaxis bv (2013)).

The total stress analysis was used to obtain the average undrained cohesion (S_u) which may have been operating at the time of failure when the embankment was raised from RL 8 to 8.7 m. This was done by setting $S_u/\sigma_{vo}' = K$ and iterating the value K until the Factor of Safety (FS) reached unity in the failed section. Using measured excess pore pressures to assess the effective stress achieved just prior to the final lift, and $S_u/\sigma_{vo}' = 0.2$, the computed FS was found to be 1 at Ch2,550 mE and Ch2,600 mE (central section of the failed area) and 1.1 at Ch2,450 mE (west of the failed area). The depth of the critical slip surface in the back-analysis matched closely with the inferred depth of the slip surface in the failed area. Sensitivity analyses were also carried out on a number of parameters including subsurface profile, water level lag behind the core relative to the spring low tide, effective stress achieved in the coralline sediment, density of the rockfill, etc. It was concluded from the sensitivity analyses that the back-analysis results were most sensitive to the assumed effective stress achieved just prior to the failure and the thickness of the beach sand and coral reef debris which provided additional restoring moment with increasing thickness. Within the failed area, the average total thickness of beach sand and coral reef debris was 2.2 m compared to 3 m outside the failed area.

The total stress analysis had a number of drawbacks including uncertainties in estimating the degree of consolidation achieved, and the fact that the ratio S_u/σ_{vo}' was unlikely to be a constant beneath the batter and beyond the toe of the embankment where the loading was smaller than beneath the crest of the embankment, and some over-consolidation of the in-situ material existed as discussed in Section 3.1. To overcome these drawbacks, effective stress back-analyses were also carried out to enable excess pore pressures to be predicted during staged construction, using a coupled consolidation/stability assessment method. The permeability of the material was modified to match the observed excess pore pressure profile just prior to the failure coupled with a c'-phi' strength reduction when the embankment reached RL 8.7 m and the sea level dropped to RL 1.3 m (spring low tide at the time of failure). The objective of the coupled

numerical back-analyses was to calibrate a credible combination of material permeability and strength parameters that would model the failure, and then be used for design of the remedial works. In the numerical analysis, the coralline sediment was modelled using the PLAXIS Soft Soil model which is based on the modified Cam–Clay constitutive model (Muir Wood, 1990) with the Mohr–Coulomb model used for all other materials. Based on a number of trials, the set of parameters shown in Tables 1 to 3 were adopted.

Table 1 General parameters adopted for numerical coupled consolidation/stability analysis

Layer	Cohesion c' (kPa)	Friction Angle ϕ' (°)	Saturated Unit Weight (kN/m³)	Young's Modulus (MPa)	Poisson's Ratio	Vertical Permeability, k_v (m/d)	k_h[6]/ k_v
Rockfill	1	40	24	30	0.25	10	1
Earthfill core	1 to 3	31	24	40	0.3	0.00432	1
Beach sands	1	35	29	*10*	0.3	10	1
Coral reef debris	0.1	32	20	5	0.3	1	1
Coralline sediments	(0 to 3)[1]	29[2]	18	–[3]	–[3]	0.001	1
Marine sediments	5	32	21.3	35	0.3	0.002[4]	1
Toe berm in failed area	0.1[6]	32	20[5]	5	0.3	1	1

Notes: [1] Refer to Table 2 for c' profile adopted.
[2] Reduced to 25° after failure in failed area.
[3] Refer to Table 3 for PLAXIS Soft Soil parameters adopted.
[4] Relatively low permeability value adopted to match relatively low basal layer drainage characteristics observed from the deep piezometers in the failed area.
[5] The existing toe berm in the failed area comprise a combination of heaved up natural soils and rockfill, and has been disturbed, hence a lower unit weight is adopted.
[6] kh = Horizontal permeability (ratio of kh/kv set to 1 for simplicity to model average permeability, and was also found to give reasonable results in the subsequent groundwater modelling).

Table 2 Adopted over-consolidation ratio (OCR) and effective cohesion for coralline sediments

Outside Failed Area			Failed Area		
RL (m)	OCR	c' (kPa)	RL (m)	OCR	c' (kPa)
-3 to -4	3	3	-2.2 to -4	3	3
-4 to -6.5	1.5	2	-4 to -6.5	1.5	2
-6.5 to -8.5	1.25	1	-6.5 to -8.5	1.25	1
below -8.5	1	0	below -8.5	1	0

Table 3 Adopted PLAXIS Soft Soil parameters for coralline sediments

Layer	Initial Void Ratio (e_0)	Compression Index (C_c)	Swelling Index (C_s)	λ[1]	κ[1]
Coralline sediments	0.98	0.39	0.029	0.086	0.013

Notes: [1] PLAXIS 2D (Plaxis bv, 2011) Soft Soil model parameters (calculated from C_c, C_s and e_o)

PLAXIS analyses were carried out for both the failed and non-failed areas, although discussions will focus on the failed area. Figure 10 shows the computed pore pressure distribution beneath the seawall in the failed area just prior to the failure on 24 April 2009. The embankment was raised to RL 8.7 m using the as-built construction sequence and the sea level was dropped to RL 1.3 m to simulate the spring low tide at the time of failure. A FS of one was back-analysed using the c'-phi' reduction method.

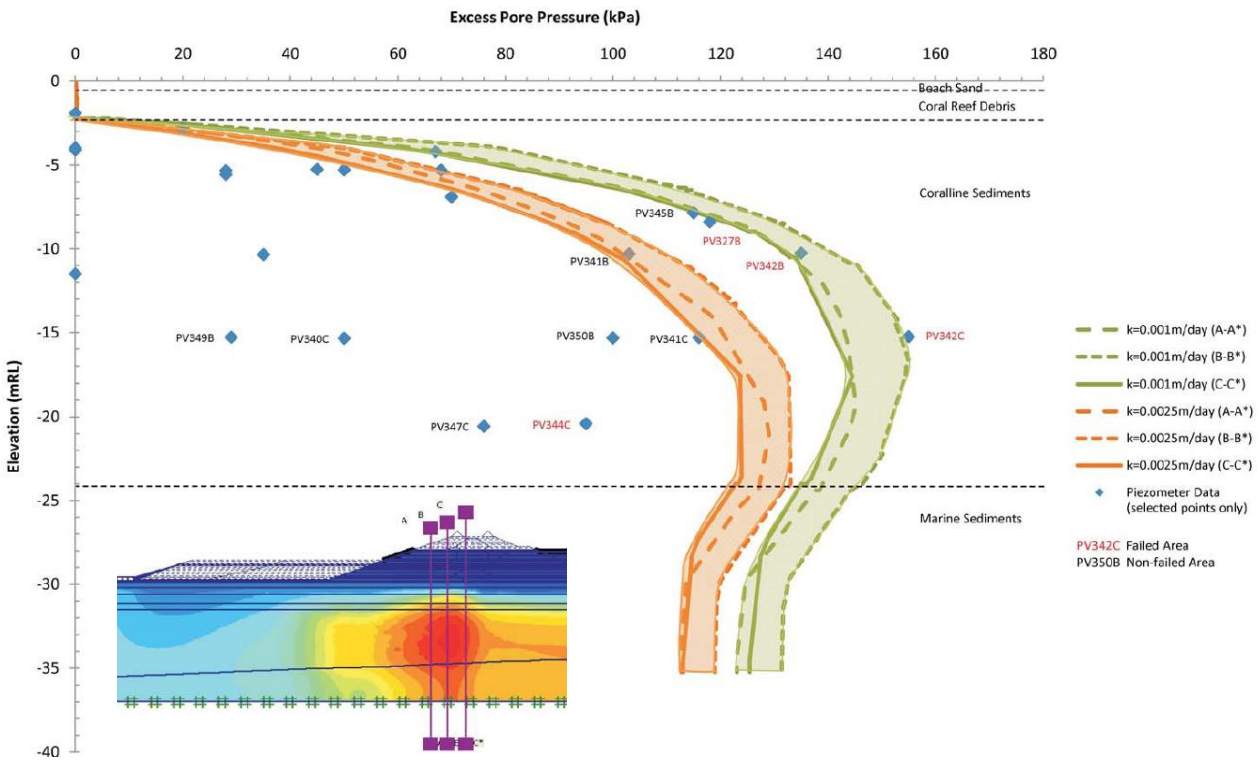


Figure 10 Computed excess pore pressure distribution within failed area

The following observations were made from the back-analysis results:

- Based on the relatively high excessive pore pressures measured at depth within and on the western edge of the failed area, there appeared to be a lack of basal drainage in the failed area compared to outside the failed area. To inhibit basal drainage in the modelling, a permeability $k_V = k_H = 0.002$ m/day was adopted for the marine sediments underlying the coralline sediment.
- An adopted permeability $k_V = k_H = 0.001$ m/day for the coralline sediment provided a reasonable match of excess pore pressures measured at depth in piezometer PV342 (located at the western third point of the failed area Ch2,550 mE). This section appeared to exhibit the highest excess pore pressures. PV344C, also located on the same section but on the landward side of the failure scarp, showed lower excess pore pressures.
- PV341, located at the western edge of the failed area, showed lower excess pore pressures which matched the back-analysis results for a higher coralline sediments permeability of $k_V = k_H = 0.0025$ m/day.

5 Design of remedial works

The remedial works comprised construction of a 20 m wide berm to RL 2 m in the area west of the failed area, and utilising the failed debris to form a 35 m wide counterweight toe berm at RL 4 m within the failed area to increase stability. More importantly, it was recognised from the failure and back-analyses that the failure was caused by constructing the embankment too quickly which did not allow for sufficient consolidation and drainage, and the lack of appropriate instrumentation (particularly the ability to read inclinometers for signs of lateral movements and potential development of shear planes).

For the reconstruction, the analysis showed that a rate of construction of 0.5 m per fortnight was required to allow sufficient consolidation to occur so as to limit the increase of excess pore pressures. The analyses indicate that it was theoretically possible to maintain a minimum FS of 1.3 for construction to RL 13 m in the area outside the failed area, but was only able to maintain this minimum FS in the failed area for reconstruction to RL 10 m.

As the adopted reduced friction angle of 25° for the coralline sediments within the failed area was considered to be on the conservative side, and as the design was based on the worst case measured pore pressures at Ch2, 550 mE, the proposed construction rate was adopted as an acceptable solution, to be reviewed during construction.

In order to avoid failure again during reconstruction, an observational method was adopted with a detailed instrumentation and monitoring plan which also incorporated a response action plan. The Observational Method has been described in detail by Peck (1969); in brief, it involved:

- Conducting sensitivity analyses of potential variations in design parameters to assess a range of 'what if' scenarios.
- Developing potential contingency measures to react to potential 'what if' scenarios.
- Devising an instrumentation and monitoring plan to enable the critical parameters (based on the sensitivity analyses) to be reviewed during construction.
- Rigorous implementation of the instrumentation and monitoring plan including regular review of the results and design during construction.
- Implementing contingency plans or modifying the design and/or construction staging if required. The mine operator also needs to have processes in place and lines of responsibility defined so that contingency actions are taken when advised by the design consultant.

A contingency plan was later developed involving the construction of an interim bund between RL 10 and 13 m on the landward side of the seawall to provide more time for excess pore pressures to take place when the main embankment reached RL 10 m. This plan was implemented during construction, with the interim bund also used as a temporary haul road and to allow interim pit mining of the overburden to take place to RL 0 m.

6 Instrumentation and monitoring

The instrumentation and monitoring system was of significant importance as it was the main method of assessing the performance of the seawall during construction and subsequent excavation of the adjoining pit. It enabled measurement of ground movements and groundwater conditions, and assessment of their changes with time, to confirm that these parameters did not exceed allowable design levels.

6.1 Instrumentation design

As part of the remediation design, and prior to recommencement of seawall reconstruction, an improved instrumentation and monitoring system was instated comprising the following:

- 20 inclinometers and extensometers, installed predominantly on the counterweight berm, close to the seaward toe of the seawall, to monitor lateral and vertical movements of the ground respectively. Each extensometer was installed in the same borehole as each inclinometer, with sensor rings fitted to the inclinometer casing at 1 m intervals, and was manually read using a probe and readout unit.
- 23 settlement plates, installed at the interface of the embankment fill and the preconstruction ground surface, to monitor vertical movements within the fill. Settlement plates were fitted with extension rods, on which survey prisms were mounted to enable settlement data to be recorded using the automated site survey system.
- Numerous survey markers were also installed, in addition to settlement plates, to monitor lateral surface displacement via the automated site survey system.
- 148 vibrating wire piezometers (VWP) to measure pore pressures within the coralline sediment. Typically three piezometer sensors were grouted at each installation at varying depths to allow a

piezometric profile (with depth) to be assessed. Piezometers were connected to data loggers to provide continuous readings at regular intervals. In addition, data was telemetered to the mine office where it was validated and then assessed in further detail.

6.2 Tidal restraints

Due to the significant tidal range, careful consideration was required to ensure that the instrumentation and monitoring system could operate effectively during all stages of the tidal cycle. The piezometer system was of particular concern as the data loggers and associated telemetry equipment needed to be kept dry.

To overcome the tidal restraints, three main systems were developed:

- *Floating buoy system.* In this system floating buoys were anchored at designated locations, which housed the data logger, solar panel, battery, and telemetry systems. Nearby piezometer cables were routed to these floating buoys and each buoy transmitted data back to the mine office. Maintenance/repairs could be carried out during low tide or via a dingy. This system was particularly effective for clustered piezometer groups.
- *Piezometer riser pipe systems.* In this system, piezometers that had surface elevations just below the highest astronomical tide (HAT) level, or those not in clusters, were extended vertically so that their riser pipe was above HAT level. Custom mounts were made on top of the riser pipes to house the data logger and telemetry equipment.
- *Multiple instrumentation enclosures.* This system was implemented near seawall completion, and was used for subsequent monitoring of the pit excavation. In this system, a total of seven custom built enclosures were installed along the completed crest of the seawall, each housing multiple data loggers capable of connecting many piezometric sensors. A more robust telemetry system was also installed for each enclosure, to cater for the increased number of connections. This system significantly reduced labour and maintenance requirements, and eliminated many tidal restraints, as the majority of issues could be addressed from these dedicated boxes well above the tidal level.

6.3 Instrumentation documentation

The instrumentation and monitoring system was operated according to the following documents and manuals:

- *Instrumentation and monitoring plan.* This document provided an overview of the design philosophy and prescribed in detail the specific requirements of each aspect of both the physical instrumentation system, and the associated management and review system. It included details on the frequency of data readings required for each instrument, depending on the type of instrument and its location with respect to where seawall construction was being undertaken.
- *Process control plan.* This document, in the form of a flow chart, provided a visual representation of the methods and processes involved. It included instrumentation specific details and processes including instrumentation installation, data acquisition and validation, protection of instrumentation, emergency responses to trigger events, requirements and response to cyclones and also defined the actions and responsibilities of all relevant staff.
- *Operating manuals.* Numerous manuals were prepared to provide operating staff with detailed information pertaining to instrumentation installation, data acquisition, maintenance, repairs and related day-to-day duties. In addition, these manuals provided specific information and processes to enable them to manage the site instrumentation system as effectively as possible.

6.4 Data acquisition, validation and review

Throughout seawall remediation, the instrumentation system was continuously improved and manual systems were transitioned to automated systems where possible. Prior to construction, all data acquisition involved manual processes and, whilst significant improvements were made, there remained some manual processes at the completion of the seawall project, namely inclinometer and extensometer readings. Accordingly, and due to the sheer number of instruments in operation, a site-based data monitoring software system was installed, which could retrieve automated data, allowed user input of manually acquired data and provided site based engineering and management staff with access to real-time monitoring data.

As part of the ongoing geotechnical analyses for the seawall, 'trigger' values were periodically assessed which allowed thresholds for pore pressure, settlement and lateral movements to be set. If monitoring data exceeded these thresholds, the software system notified all users and a series of actions were undertaken depending on the severity of the exceedance. Trigger values were regularly updated throughout construction to account for changing conditions and the acquired monitoring data. This process allowed for a more effective data monitoring and review process, as analysts could very quickly identify unusual or concerning trends and focus their attention on those instruments.

Data validation and review was carried out by an off-site (office based) engineering team, and a strict process was implemented to ensure that all relevant data had been properly checked and reviewed. Close communication was maintained between all site-based and office-based personnel to ensure that (1) the required monitoring data was being obtained at the required frequency, (2) any maintenance requirements were promptly addressed and (3) any damaged or non-operating instruments did not falsely trigger any exceedance alarms.

7 Client involvement

As the client was ultimately responsible for scheduling and constructing the seawall, which itself was highly dependent on a numerous of geotechnical conditions, all parties were required to work closely to ensure that works were carried out as effectively as possible and in a timely manner. Systems were established to ensure that no works were undertaken without the appropriate level of geotechnical review, and formal geotechnical approval was required prior to proceeding with any construction works. As part of this process, a large amount of information was routinely provided by the client, such as detailed daily survey records, monthly look-ahead construction schedules, requests for approvals, etc.

The client also placed management of the instrumentation and monitoring system in the hands of the geotechnical team, and thus instrumentation requirements that were considered to be critical to the seawall project did not require rigorous justification and could be approved and implemented at very short notice. This served to reduce construction delays and resulted in a more robust instrumentation network, increased reliability and quality of monitoring data and improved confidence in the geotechnical design.

It is the opinion of the authors that a major contributing factor to the successful completion of the project was the high level of commitment by both the client's operating and management staff. They maintained compliance within the established control plans and showed unprecedented understanding and trust, despite the occurrence of the failure and even in the occasional event where works needed to be halted until appropriate conditions had been met. Furthermore, due to the inherent team relationship that was developed, by the completion of the project it was also evident that the client operational and management staff had developed a much better understanding of the technical specifics, demonstrating on many occasions a greater appreciation for the geotechnical issues and risks associated with the project.

The degree of client involvement and collaboration is sometimes rare, however on this project, provided a good example of how such an approach can provide significant benefits, and can result in successful completion of high risk projects in complex ground conditions.

8 Reconstruction

Reconstruction of the seawall was undertaken in the following broad stages:

1. Construction of counterweight berms. Competent rockfill materials were progressively constructed over several weeks during available tidal windows. Safety protocols were maintained to ensure that equipment remained well behind the advancing berm crest. Rockfill was hauled and tipped in designated areas, and bulldozers then pushed this material to progress the berm outwards.
2. Replacement and reconstruction of lower core to RL 4 m. As a result of the failure, the existing core material in the failed area had been significantly displaced or cracked. Accordingly, the core material in the failed area was excavated and properly reinstated. Working at these low elevations required astute planning and implementation due to the very short tidal windows available.
3. Progressive raising of the seawall from RL 4 to 10 m. This process was carried out over a period of approximately six months, under close monitoring and review.
4. Installation of sheet piles along core alignment. Sheet piles were driven along the seawall centreline from RL 10 m to a toe level of approximately RL -5 m, to provide seepage cut-off of the relatively high permeability beach sands unit. Upon completion of sheetpiling, pore pressures behind (north of) the sheetpile wall were noted to rapidly fall in response, which matched the modelling predicted in the design and confirmed that the sheetpiling had met its design purpose.
5. Construction of setback bund and interim pit mining. Once the seawall had been constructed above the higher tide levels, studies were undertaken to assess the feasibility of constructing a smaller 'interim pit' so that mining could recommence. A design was prepared involving construction of a rockfill bund located inside, and parallel to, the seawall and establishment of an interim pit set back approximately 50 m from the seawall.
6. The interim pit was designed to an elevation of approximately RL -25 m, which required excavation through the soft underlying sediments. This provided an invaluable opportunity to assess the behaviour of the sediments during unloading. In one trial, a small area of soft sediments was excavated at an angle above the design slope angle. For several days this slope remained stable, and may have provided a false sense of confidence, before eventually failing. This demonstrated that the undrained shear strength provided temporary strength, and a higher short-term FS, but this strength and FS continued to fall until driving forces overcame the resisting soil strength. Knowledge gained during this period proved invaluable for mining back to the final pit slope, and contributed to the overall success of the project.
7. Progressive raising of the seawall from RL 10 to 12.5 m. By this stage interim pit mining had commenced, instrumentation systems had been vastly improved and, as the seawall was above the tidal zone, construction limitations were minimal. However, this part of the seawall construction was in fact the most critical, and the overall Factors of Safety typically reduced with each lift placed. Upon completion to RL 12.5 m, permanent windrows were instated and preparation began for mining of the final pit.

9 Conclusions

The Cockatoo Island Stage 3 seawall project is a good example of how the Observational Method, which is more commonly associated with civil and geotechnical projects, can be successfully applied in a mining setting. It also highlights the importance and benefits of maintaining strong client involvement and support throughout the construction process, both from a management and operational perspective. This requires a financial and operational commitment from the client, but provides them with increased confidence in the project and a systematic and controlled process system.

This project also highlights the importance of a robust and well managed instrumentation and monitoring system. This holds particularly true in a mining environment where accepted Factors of Safety are often much lower than those of civil and geotechnical projects, as they typically have a direct impact on the ore recovery and project revenue. For such projects, it is important that the reduced Factors of Safety are partly offset by an increased instrumentation and monitoring system and that this system properly accounts for the mining environment, which often has unique factors that may require additional design considerations. The high degree of client involvement and collaboration in the instrumentation and monitoring program was also a key to success.

When multiple large scale failures occur on a project it is usually an indication that (1) the ground conditions are highly variable and this variability has not been properly accounted for in the design, (2) there is lack of adequate understanding or investigations carried out following the initial failure to properly assess the cause of the failure, or (3) there is an inadequate monitoring system in place throughout construction to provide the level of review necessary to make design adjustments as required. Whilst there is typically a very strong drive in remediating the failed area as quickly as possible, it must be noted that when failures do occur, they provide a good opportunity to calibrate the geotechnical and hydrogeological models through back-analysis. Thus a thorough investigation of the failure and failed areas is considered invaluable, particularly for those projects that are expected to continue for many years.

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