The paper was published in the proceedings of the 11th Australia New Zealand Conference on Geomechanics and was edited by Prof. Guillermo Narsilio, Prof. Arul Arulrajah and Prof. Jayantha Kodikara. The conference was held in Melbourne, Australia, 15-18 July 2012.

Monitoring and Geotechnical Assessment of Stabilised Treated Sediment Bund

Ching Dai

Associate Director, PhD, AECOM, Level 21, 420 George Street, Sydney, NSW 2000; PO Box Q410, QVB PO, Sydney, NSW
T +61 2 8934 0194 F +61 2 8934 0001 email: ching.dai@aecom.com; www.aecom.com

ABSTRACT

This paper presents an assessment of the long term monitoring carried out in conjunction with the bund construction at the Kooragang Island Emplacement Cell (KIEC). As part of an observational approach to the design and construction monitoring was carried out during and following the construction period. A back-analysis check of the slope stability of KIEC has been carried out and future settlements forecast based on the observed movement and interpreted soil strength and deformation parameters.

Factors of Safety (FoS) for the bund embankment stability are compared based on an extrusion failure check, the use of a stability control charts and limit equilibrium slope stability analyses. The assessment confirms that rate of filling was controlled to provide an adequate factor of safety during construction and for the long term.

Keywords: monitoring, back analysis, bund slope stability

1 INTRODUCTION

The KIEC facility is located on Kooragang Island near Newcastle and consists of a series of containment cells designed to provide long term emplacement of treated sediment from a nearby site. AECOM was engaged by Thiess Services Pty Ltd (Thiess) to design and supervise construction of the bund of Cell 3 and the filling of Cell 1 and Cell 2 for KIEC. The subsurface conditions generally consist of an upper layer of fill including coal rejects, slag, ash, sand and gravel (0.3m-11.6m in thickness), soft to firm alluvial upper clay (0.0m – 5.5m in thickness), medium dense to very dense sand (8.4m – 27.3m in thickness), stiff to very stiff estuarine deep clay (0.8m – 34.2m in thickness) and bedrock consisting of siltstone, sandstone and coal. The bund embankment was constructed on the above foundation. The typical bund embankment height is 10m.

Monitoring of the geotechnical performance of the KIEC, during construction and operation was required:

- To ensure safety and integrity of the site and adjoining land during construction and operation
- During construction stages - to assess the response of the natural ground conditions, for comparison to design assumptions and interpretations and to allow refinement of the geotechnical model, control rates of fill placement and provide for the long term performance. At that time the information will assist in developing post development performance of the emplacement cells.

The installed monitoring plan is shown in Figure 1 along with the three individual cells making up the emplacement. While this paper is being prepared, the site construction of sediment fill emplacement has been completed. The results in this paper therefore represent the outcomes relevant to the construction stage.

2 SCOPE OF INSTRUMENTATION

The instruments installed and incorporated in the geotechnical assessment included

- **Inclinometers** (IO) - provide information on lateral movement at the toe of the Cell 3 bund during and post construction. During construction the inclinometer results were reviewed in conjunction with data on the rate and height of fill being placed on site to identify any possible signs of bund instability and if necessary, control the rate of fill placement;

- **Soil settlement gauges** (SSG) - to provide information on vertical movement during and post backfilling in Cells 1, 2 and 3;
- **Vibrating wire piezometers** (VWP) - to provide information on pore water pressure during and post backfilling of the cell. The vibrating wire piezometers were targeted to monitor the dissipation of excess pore water pressure within the upper and lower clay units underlying Cells 1, 2 and 3; and
- **Earth pressure cells** (EP) - to provide information on the magnitude of the vertical stress distributions during and post backfilling of the cell. During backfilling of the cells the earth pressure cell instrument data were correlated with the vertical settlement rate data and pore water pressure data to identify any variations between assumed and actual pressure imposed by filling of the cells and to refine settlement projections within each cell.

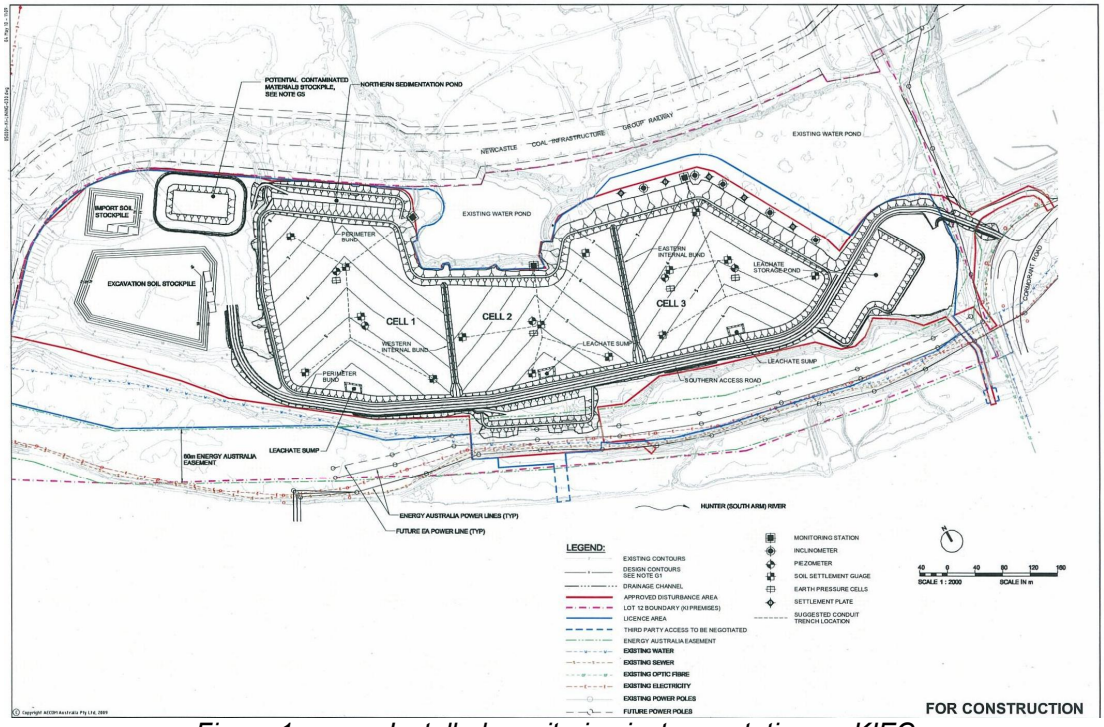

Figure 1 Installed monitoring instrumentation on KIEC

3 ASSESSMENT OF DEFORMATION

Assessment of the monitoring data was carried out at key sections. In the following the data is considered for the north east edge of cell 3. This was a critical area of the site because of the presence of an adjacent pond and associated thicker soft clays.

3.1 Lateral Deformation and Back analysis from Inclinometer Observations (I01)

Figure 2 (Left) shows the cumulative horizontal movement recorded in inclinometer I01. A back analysis check using a 1-D consolidation program for curve fitting was carried out to relate the observed movement of I01 to the likely soft soil strength and deformation parameters. The back analysis obtained reasonable agreement with the monitoring data. Figure 2 (right) shows the theoretical expected lateral movement of IO1 by back analysis compared to the measured data..

It can be seen that the fastest lateral movement predictably occurred during bund construction. It was also observed that the monitored lateral deformation continued post completion of the bund construction, however at a lesser rate.

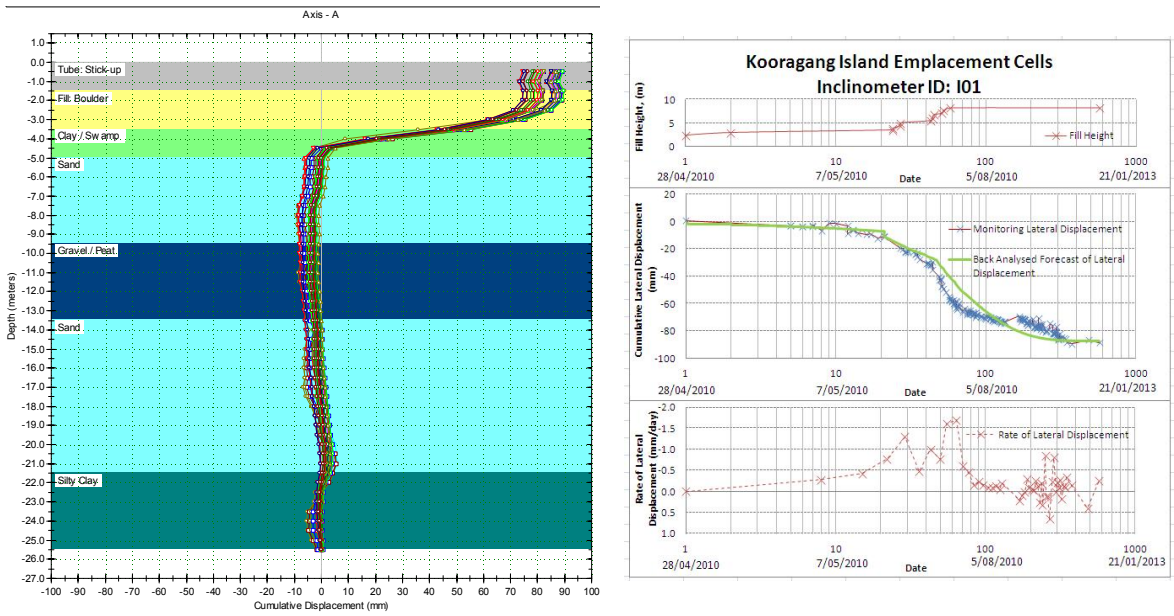

Figure 2 (Left) Cumulative horizontal movement
(Right) Back analysis - Horizontal deformation vs. date

3.2 Vertical Settlement Analysis

A typical vertical settlement forecast is shown in Figure 3. Based on the SSG installation log at the relevant location, the soil profile consists of an upper layer of fill including gravel and clayey gravel (approximately 4m in thickness), underlain by alluvial upper clay (approximately 4m in thickness) and dense sand of approximately 7.5m in thickness.

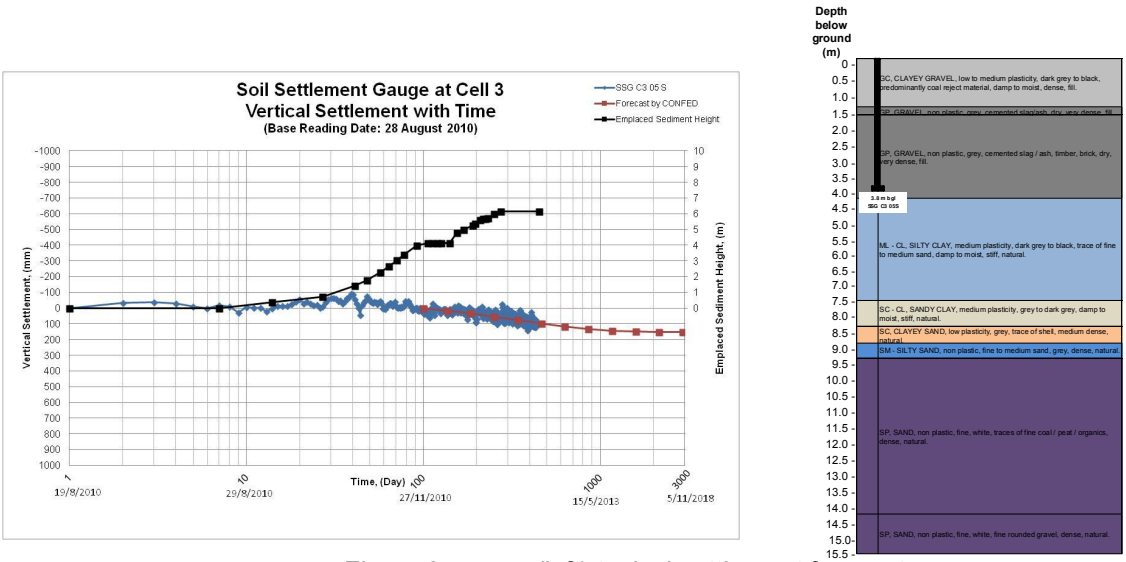

Figure 3 (left) typical settlement forecast;
(right) at a typical geotechnical model

The back analysis for SSG readings in Cell 3 shows primary settlement completion values consistent the typical forecasted value of 185mm.

However, on average approximately 40 to 75% of consolidation has been completed for each of the reliable soil settlement gauges. Through back analysis it is predicted that most primary settlement will be completed in approximately two to three years.

3.3 Correlation of Monitoring Data

The earth pressure cell data shows pressure increases as the emplaced sediment material weight increases, which indicated the total stress increment is relevant consistent with emplaced fill weight.

However there is not obviously correlation between excess pore pressure and primary consolidation settlement completion. The maximum fill height achieved in Cells 1, 2 and 3 is approximately 10m. This means the additional load on the ground due to the treated fill is approximately 170kPa (10m x 17kN/m^3). Currently all vibrating wire piezometers are showing maximum pore water pressure increases of less than 10kPa, or less than 10% of the placed load. This implies excess pore pressure dissipation of at least 90%. However, as comparison, the actual consolidation settlement is approximately 40% to 75% complete. It is not sure why they are inconsistent. One reason for this discrepancy might be that there might have been lateral water flow from the cells into a nearby pond, so the excess water pressure has quickly dissipated. The discrepancy might also be caused by issues with the reliability of either the VWP or SSG instruments.

Nevertheless, for the assessment of strength gain to be used in stability analyses lesser figure of consolidation ratio of 40% was assumed

4 STABILITY OF CELL 3 SEDIMENT EMBANKMENT

Figure 4 shows the as-built embankment profile with the typical geotechnical model. The design bund batter slope was 1 (V) to 3 (H). In the following the process used to interpret strength gain is described along with stability assessments based on soft clay extrusion, use of a stability control chart and limit equilibrium analyses of global stability.

4.1 Design Criteria

The key design criteria were as follows:

Minimum Factor of Safety (FoS) = 1.5 was required for global stability in the long term; FoS = 1.25 for short-term, and FoS of 1.1 for seismic case. The short term stability with a rapid rate of fill placement was the critical case. Therefore the discussion below is only targeted on short term performance and the improvement in the bund stability.

FoS of 1.3 is adopted for potential extrusion failure and embankment stability by the control chart, because both of them are basically for a short term performance.

4.2 Strength gain in soft to firm alluvial upper clays

At the design stage, based on existing information of CPTs and plasticity index, the Cu values were interpreted as a normalised strength ratio of (AECOM 2010).

$$\Delta Cu=0.23\ \Delta\sigma'_v \qquad (1)$$

After construction of the fill emplacement (about 10m high), due to the presence of the HDPE liner, further deep site investigation was not possible. Following up the equation 2 and assuming 40% consolidation, the strength gain would be 13.8kPa. 10kPa was adopted in the following sections where it is necessary. This would be expected to be a lower bound strength gain for the alluvial upper clays because the actual primary consolidation completion was about 40% to 75%.

4.3 Potential extrusion Failure Check

An analysis for potential extrusion failure of foundation soil from beneath the embankment was carried out based on British Standard (BS8006). . The predicted FoS against extrusion failure under the embankment bund was assessed to be 1.3 prior to filling of Cell 3. This FoS marginally satisfied the design criterion.

As indicated in section 4.2, the adopted rate of filling enabled some strength gain during construction. This lead to a factor of safety increasing. Based on assessed conservative strength gain, the current FoS to against potential extrusion failure is greater than 1.6 for the current embankment situation. The

gained shear strength under the embankment can cater for the outward shear stresses within the soft foundation soil during and post construction.

4.4 Global Stability of the Sediment Embankment

The limit equilibrium method (Slope/W) was used to assess the slope global stability of the proposed backfill slope. For the construction stage undrained conditions were assumed. The sensitivity analysis and comparison of FoS vs strength gain for alluvial upper clay are listed below:

- FoS was about 1.25 for the design strength profile of 20kPa without strength gain at the design stage;
- FoS > 1.5 for the design strength of 20kPa with 5kPa strength gain;
- FoS > 1.7 for the design strength of 20kPa with 10kPa strength gain

These indicate that FoS of global stability of the sediment embankment is increased during fill emplacement construction.

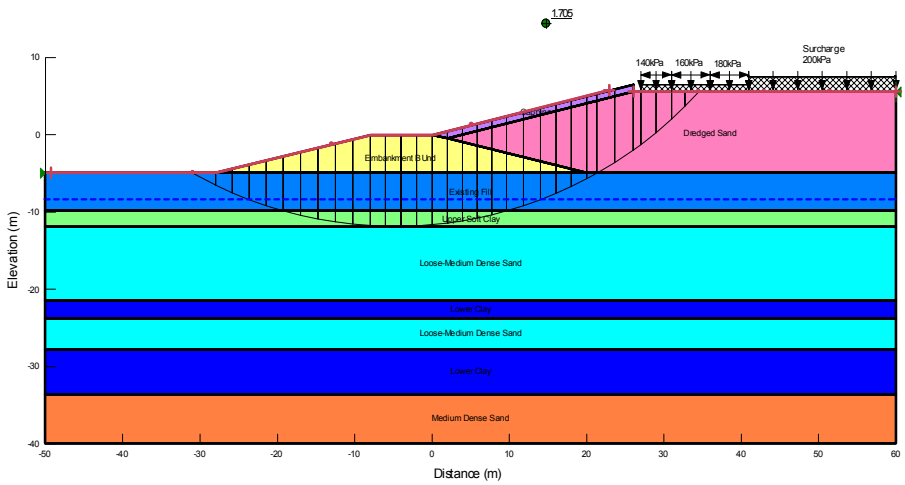

Figure 4 Typical global stability analysis indicated FoS for the current embankment stability is greater than 1.7

5 EMBANKMENT STABILITY CONTROL CHART

A further assessment of the field performance against predictions has been made using a Construction Control Chart (Wakita and Matsuo, 1994 and CIRIA Report 185), which is an observational approach. In this case, consideration of the shear strength is not required. The chart was developed for embankments founded on soft clays and uses the ratio between the maximum lateral deflection (h) beneath the embankment toe and embankment settlement (d). The resulting FoS is defined as a ratio of embankment load failure / embankment load and was developed by Wakita and Matsuo based on observational statistics and finite element method correlation. This has been plotted in Figure 5 and provides a further means of assessing the likely FoS against global failure.

The FoS is determined by processing the data for in-situ monitored lateral movement of I01 (inclinometer showing critical lateral movement) and in-situ monitored vertical settlement at centre of base of embankment.

The plot was selected at three reading dates: in Dec 2010, h=75mm, d=47mm; in July 2011, h=80mm, d=61mm; in Nov 2011, h90mm, d=110mm. These three points are plotted as t1, t2 and t3 respectively in Figure 5. When monitoring first began in Dec 2010, the FoS was approximately 1.4. By November 2011 the value had increased to greater than 1.5, which means that FoS is increased during and after construction of fill emplacement. It approved that the actual fill rate and program was proper to obtain some strength gain. It also indicates the long-term factor of safety criteria for embankment stability has been achieved. The ongoing result by the stability control chart here gives additional confidence to the stability assessment.

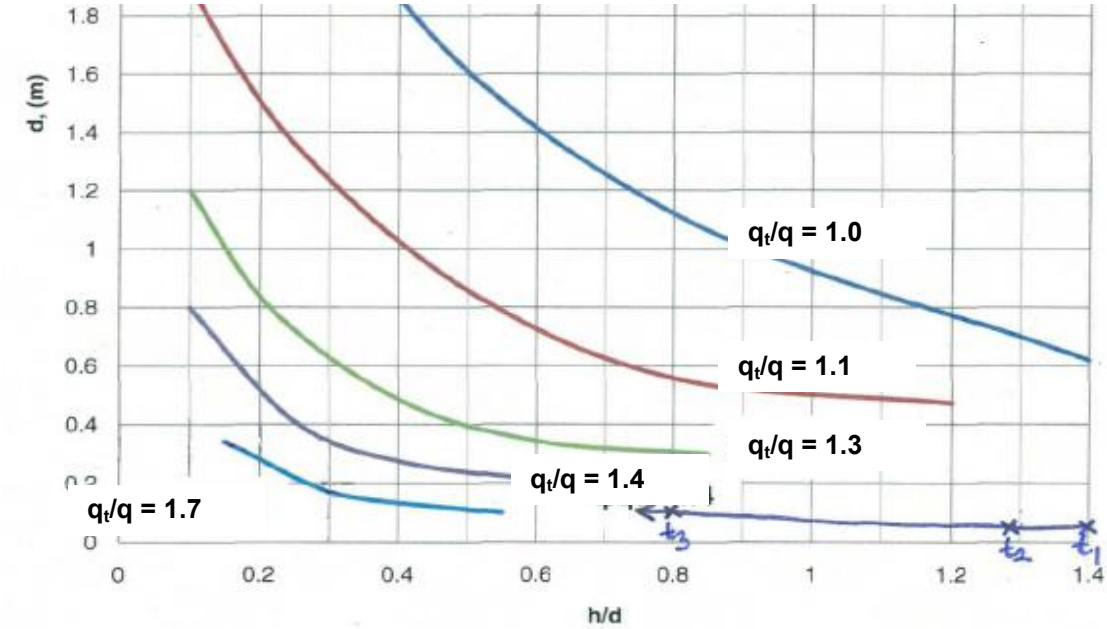

Figure 5 FoS plot on Construction Control Diagram (after Wakita and Matsuo, 1994)

Where h = denote lateral movements at the embankment toe
d = denote vertical settlement at centre of the base of an embankment
q = denote embankment load
q_t = embankment load failure

6 CONCLUSION

From the analysis undertaken and presented in this paper the following conclusions can be drawn.

The monitor data indicates that the complete consolidation time is approximately 2 to 3 years. Both observational approach by Stability Control Chart and numerical analyses indicate that the adopted fill rate was suitable to the foundation soil mass to obtain proper strength gain for the potential stability issue. The stability of the bund embankment has been increasing during and after fill construction. The FoS of the bund has been increased from 1.3 to be greater than 1.5 after filling of the cells has been completed. As a result, the design criterion of a FOS of at least 1.5 for the long term has been satisfied. Based on the analysis and results achieved for settlement predictions and bund stability capacity, it is concluded that the potential for future development is preserved in context with the bund embankment stability requirement by Thiess.

The comparison of the suite of monitoring data indicated that the outcomes from the VWP were not consistent with SSG data. Due to the lack of the correlation between excess pore pressure and consolidation settlement completion, the ongoing monitoring action (including surface settlement plates and surface survey reading) were required to be conducted. The uncertainties should be clarified and understood by the ongoing monitoring data.

7 ACKNOWLEDGEMENTS

The author is thankful to Thiess Services Pty Ltd for giving permission to use the data and publish this paper.

REFERENCES

AECOM (2010), “Report on Kooragang Island Emplacement Cell slope stability Assessment”, 60100219, 7 May, 2010
AECOM (2011), “Report on Kooragang Island Emplacement Cell Geotechnical Assessment – monthly monitoring data report”, 60100219, Dec. 2011
AECOM (2011), “Report on Long term development potential of Kooragang Island Emplacement Cells”, 60100219, Dec, 2011
BS8006 (1995), “Code of Practice for strengthened / Reinforced soil and other fills”, 143pp
CIRIA 1999, Report 185, the observational Method in ground Engineering: Principles and Applications, London, 159pp.
Wakita, E and Matsuo, M (1994), Observation design method for earth structures constructed on soft ground, Geotechnique, 44, (4), 747-755pp