SOFT SOIL CASE HISTORY: SHELLHARBOUR SEWAGE TREATMENT PLANT UPGRADE

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ABSTRACT

This paper describes ground engineering challenges and solutions employed at a soft ground site near Shellharbour, New South Wales. Geotechnical issues encountered during the sewage treatment plant upgrade project included:

- Deep peaty soils at the backfilled swamp site
- High groundwater level and potentially high inflows through permeable fill
- Large clarifier tank excavations (approx 80 m x 80 m x 5 m deep)
- Flooding and acid sulphate soils
- Potential settlement impacts on existing infrastructure.

Ground engineering risks were successfully managed through adequate scoping of investigations, numerical modelling of designs and involvement of geotechnical engineers during construction. A comparison of actual versus predicted behaviour for an anchored sheetpile wall is presented, enabling an evaluation of WALLAP and PLAXIS software. The value of geotechnical observations and monitoring during construction is also discussed.

1 INTRODUCTION

This case history provides a summary of design and construction techniques used in soft ground conditions for the upgrade of Shellharbour Sewage Treatment Plant (STP) in New South Wales, Australia. In particular, this paper describes the design and behaviour of a large, sheetpiled excavation at the site and draws conclusions about the performance and possible improvements that may be achieved on similar future projects.

The site is situated in a low-lying area behind the dune environment of Shellharbour Beach, about 1 km north of the town of Shellharbour. Prior to the 2005 upgrade, the treatment plant was constructed in two phases during the 1970s and 1980s. Figure 1 shows the layout of the site.

The site was originally part of Barrack Swamp, which was reclaimed by backfilling with gravelly coal wash reject material. Construction drawings showed that early excavations for in-ground tanks and lagoon structures utilised a perimeter clay cut-off wall to limit groundwater inflow (dashed line around existing structures in Figure 1). Anecdotal evidence indicates that this was not entirely successful and that steel sheet piles were used to provide additional groundwater cut-off and improve excavation stability.

The recent upgrade was commissioned by Sydney Water in late 2004 under a Design and Construction Contract, awarded to United Group (Formerly UKG) as the Contractor and Sinclair Knight Merz as Designer. The $25M augmentation comprised construction of 4 x 30 m diameter buried clarifier tanks, control and distribution chambers, a buried pump station, chlorination and grit facilities, ancillary buildings, tanks, biofilter beds and associated roads, embankments and retaining structures. The focus of this paper includes the investigation, design and construction of the in-ground clarifier tanks.

2 SURFACE AND SUBSURFACE CONDITIONS

The landscape surrounding the extensively filled Shellharbour STP site comprises gently undulating and mounded grassland and bushes, with relief to 3 m. Low lying parts of the site are subject to periodic flooding, associated with the tidal Tongarra Creek running along the western and part of the southern site boundary. The average ground surface level of the site is RL 3 m AHD, which is above the 1 in 100 year flood level (RL 2.8 m AHD). However, a large depression covering much of the area of the proposed clarifier tanks was as low as RL 1 m AHD in places, and frequently flooded.

Adjacent to the creek, the ground surface comprises marshland vegetation of Barrack Swamp and the nearby Shadforth compensatory wetlands. An adjacent cricket oval was located on a former landfill, which was a potential source of contaminated groundwater.

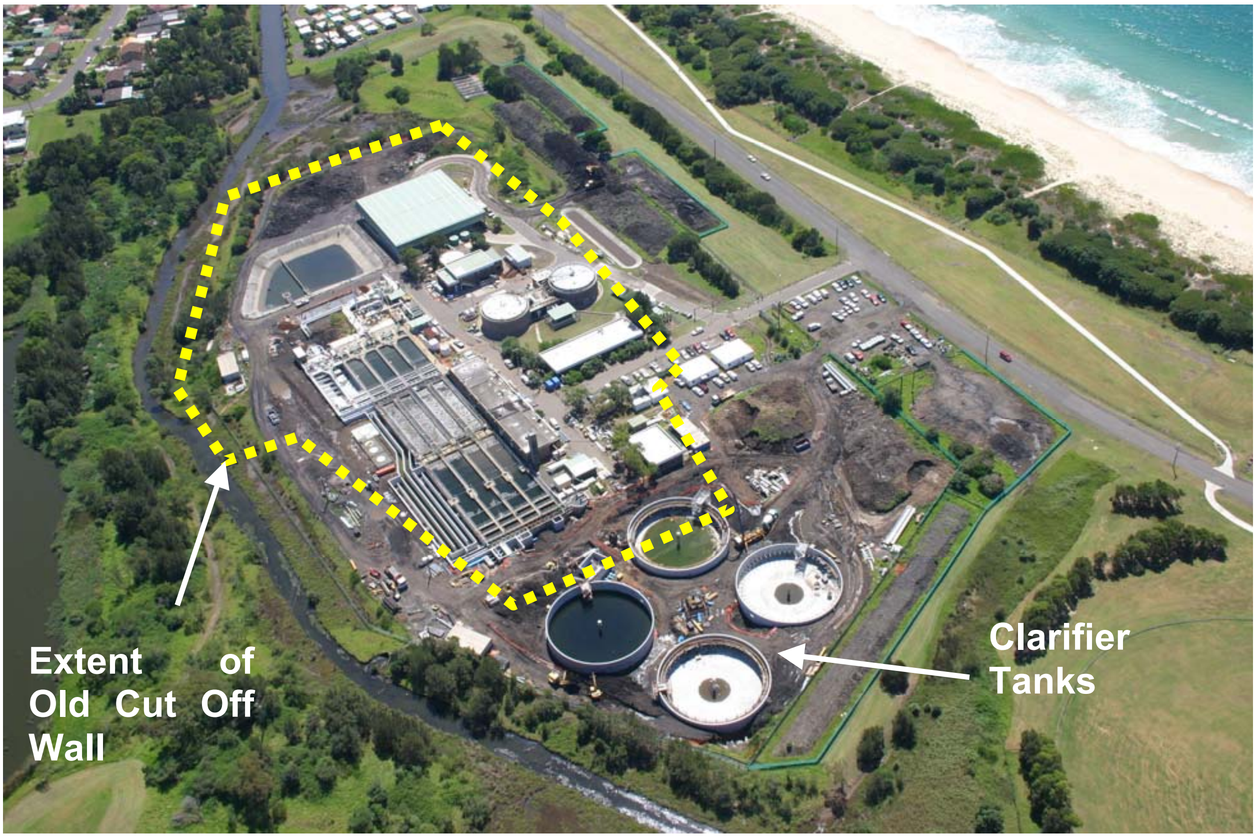

Figure 1: Site layout (during construction).

A geotechnical investigation was carried out by SKM in 2004 with the objective of collating existing information and gathering additional information for detailed design. Additional investigation included:

- Desk study, review of existing information including information from four previous investigations, which included 22 boreholes, 2 CPTs and 10 Test Pits;
- Drilling of 9 additional cored boreholes, with *in situ* strength testing and acid sulfate screening;
- Excavation of 9 exploratory test pits using a backhoe and a groundwater pump-out test;
- Installation of 3 standpipe piezometers for groundwater monitoring and falling head tests and
- Geotechnical laboratory testing including classification tests, oedometer tests, triaxial strength tests, aggressivity tests, POCAS tests, organic content tests and Californian Bearing Ratio (CBR) testing.

Table 1: Summary of Ground Profile.

Unit	Depth to top of unit (m)	Thickness (m)	Origin / Type	Description
1	0	1 – 5	Fill	FILL: comprising asphalt paving or topsoil over a highly variable mixture of sandy GRAVEL (coal wash reject), with cobbles of slag and zones of gravelly CLAY.
2	1 – 5	0 – 5	Estuarine Swamp	Clayey PEAT / Peaty CLAY low to medium plasticity, with fibrous material, brown and black. Very soft and highly compressible, odorous.
3	3 – 6	1 – 4	Estuarine	Organic Silty CLAY, medium plasticity, grey. Very soft and compressible. Locally interbedded with Unit 2.
4	6 -9	2 – 5	Alluvial (possibly estuarine)	Sandy Silty Clay, medium to high plasticity, grey with yellow brown. Highly variable mixture of sandy CLAY or clayey SAND. Sand is fine to medium grained.
5	8 – 14	0.5 – 4	Residual	Sandy Silty Clay, medium to high plasticity, grey mottled yellow brown or orange brown stained. Sand is generally fine to coarse grained.
6	10 – 15	-	Bedrock	SANDSTONE, fine to medium grained, volcanic/tuffaceous, distinctly weathered and medium to high strength.

The investigation showed that the uppermost ground profile comprised widespread fill of mostly slag and coal wash reject (also called 'chitter'). Natural soils comprised recent swamp deposits including organic peat and organic clays over soft estuarine clay and stiffer sand/clay mixtures of Quaternary age. The lower alluvial material was judged to be overconsolidated and deposited during an earlier geological epoch. Bedrock was identified in all cored boreholes as Budgong Sandstone, at an average depth of 12 m in the area near the proposed clarifier tanks.

Six geotechnical units were used to build the geotechnical model, which is summarised in Table 1.

A cross section through the site is shown on Figure 2 which illustrates this ground profile in the vicinity of the proposed clarifier tanks. It is worth noting a few additional characteristics of the upper soil units to illustrate the challenging nature of the soft ground conditions and how they affected the design of the proposed clarifier tank excavation.

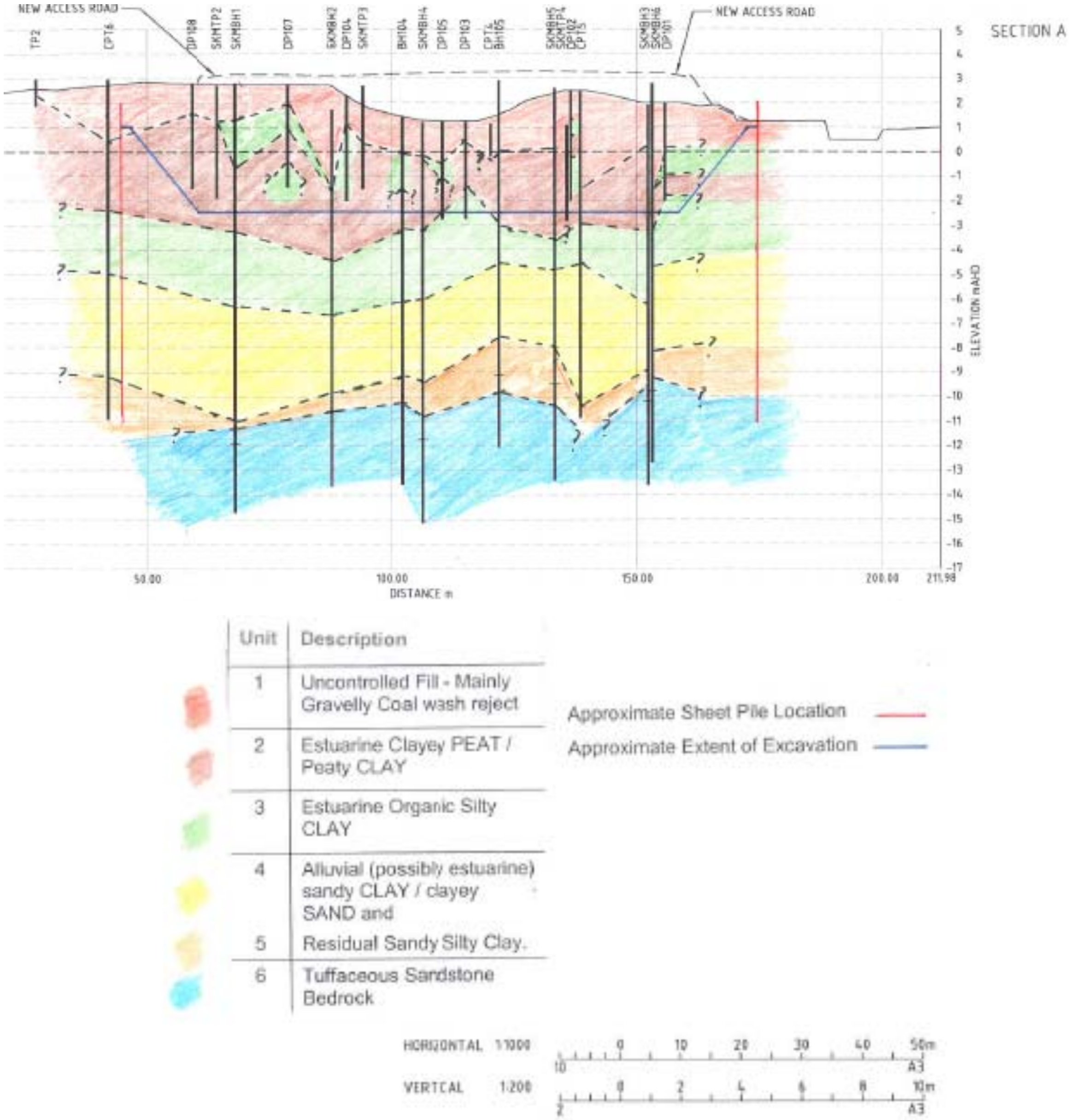

Figure 2: Cross Section through Clarifier Tanks.

The Unit 2 peaty soils were typically dark brown and where encountered, varied from pure peat of almost completely un-humified, fibrous plant remains to highly amorphous peat and clay mixtures. The Unit 2 soils were typically 3 m thick with a very high acid sulfate soil potential upon oxidation. Unit 2 peaty soils were found to vary from very soft to soft and spongy. Tree trunks and obstructions were also identified in the upper swamp deposits and fill. Corresponding organic content test results in Unit 2 soils varied from 60% to less than 10%. Samples having a high percentage of organic matter had high moisture contents of up to 339%, and material density as low as 11.6kN/m^3.

Below peaty soils, a layer of very soft to firm grey silty clay of high plasticity (Unit 3) was observed. To illustrate the low strength of these soils, the authors noted during investigation that the consistency of the estuarine clay in the softest, low lying areas was similar to toothpaste. In some areas, shells and shell fragments were observed, indicating an estuarine alluvial origin.

The thickness of this silty clay unit was about 3 m on average, although in many cases there was no clear boundary with the underlying alluvial unit. Permeability and therefore excavation inflows associated with this unit were expected to be low.

The clarifier excavation works had significant potential to generate acid sulphate soil and groundwater due to:

- Oxidation of potential Acid Sulfate Soils (ASS) exposed in excavation spoil and side walls of excavations and
- The oxidation of potential ASS soils due to changes in groundwater level.

An acid sulfate management plan was developed to prevent migration of impacted groundwater to off site receptors and control the handling and disposal of acid sulphate spoil, leachate and excavation water. The plan incorporated avoidance and mitigation strategies and detailed the management of stockpiles, neutralisation of impacted water and included a monitoring program and contingency measures.

3 GROUNDWATER

Groundwater exists at shallow depth throughout the low-lying site and its behaviour is influenced by the surrounding tidal creek. Groundwater levels measured during fieldwork were commonly in the range 1.0 m to 1.5 m AHD and high groundwater inflows were expected for excavations intersecting the highly permeable gravelly fill. In particular, control of groundwater at the large clarifier tank excavation (up to 6.5 m deep) was identified as a major issue at tender stage.

The reference design at tender stage included groundwater cut-off works comprising an 80 m length of sheet piling along the creek boundary adjacent to the clarifier excavation, with the objective of reducing groundwater inflow to a limit of 1 ML per day. An additional rate item was included with the tender to cover additional sheet piling if required. Subsequent analysis carried out as part of design estimated inflows of up to 2.2 ML/day for an open cut clarifier excavation, which led to the adoption of a circumferential cut-off wall to reduce groundwater inflow.

Following award of the tender, it was necessary to check the hydraulic connection between the tidal creek and the permeable fill and refine likely groundwater inflows and control options. Groundwater response in standpipe piezometers within the fill was periodically measured during the site investigation and compared against tidal variations. However, no consistent relationship was established between the groundwater and tidal fluctuations, probably due to the irregularity of manually read piezometer data.

Additional groundwater observations, including falling and rising head tests and pump out tests from pits during the investigation, indicated that the transmissivity of the fill was very high. It was inferred that significant recharge would occur in this layer, given the proximity to the tidal creeks and likely degree of hydraulic connection.

Despite the high transmissivity, it later became apparent that the fill was not so well connected to the creek and that once drained, the long term, steady state recharge of the gravel aquifer was less than expected. In this regard, a longer term pumping test utilising a production well and monitoring bores along with continuously datalogged piezometers would have been beneficial.

4 DESIGN PARAMETERS

The soil and rock design parameters adopted for analysis are shown on Table 2. These were based on a moderately conservative design line selected through combined test results and correlations for all available investigation data (current and historical). The corresponding anchor loads and bending moments from WALLAP analyses were treated as working load values to which appropriate safety factors were then applied.

Preliminary anchor design was carried out using limit state design methods according to the Australian Standard for Earth Retaining Structures (AS4678, 2002).

Table 2: Soil and Rock Design Parameters.

Unit	SPT 'N'	Bulk Density, γb (kN/m3)	In-situ Stress, K'o	Undrained Modulus Eu (MPa)[1]	Friction Angle, φ' (Deg)	Drained Cohesion c' (kPa)	Undrained Shear Strength Su (kPa)[1]	Permeability, K (m/s)
1. Gravelly FILL	3 - 29 Ave 14	18	0.44	2Z	34	0	0	1×10^{-3}
2. Estuarine Clayey PEAT / Peaty CLAY	0 - 4 Ave 1	14	0.69	1+0.35Z	18	2	3+1.2Z (Ave 8)	1×10^{-5}
3. Estuarine Organic Silty CLAY	0 - 5 Ave 4	16	0.62	2+0.34Z	22	2	6+1.1Z (Ave 14)	1×10^{-8}
4 & 5 Alluvial / Residual SAND & CLAY.	5 - 22 Ave 12	18	0.56	4.5+0.75Z	26	2	15+2.5Z (Ave 40)	1×10^{-8}
6. Tuffaceous SANDSTONE Bedrock	Refusal	24	-	100	42	200	-	1×10^{-7}

Note:

1. Where Z is depth measured below ground surface

5 DESIGN APPROACH

This section deals with the soft ground engineering issues relating to the large temporary excavation containing the four circular concrete clarifier tanks. Final excavation levels varied from RL -2.5 m at the outer edge of the tanks to -3.5 m AHD at the central sump of each tank. An additional 1 m of over-excavation was required for an under-tank drainage layer. Resulting excavation depths in the area of the buried clarifiers varied from 4.5 m to 6.5m.

During tender and preliminary design stages of the project, the following main excavation options were identified:

- Open cut (approx 100 m x 100 m x 5 m average depth) with shallow treatment / cut-off to reduce inflow;
- Secant piled tank structures installed from ground surface;
- Cantilevered sheet pile retaining wall around perimeter with supporting berm and
- Anchored sheet pile wall around perimeter without supporting berm.

Secant piles were eliminated at the pre-tender stage, due mainly to cost. The open cut option without cut-off walls was impractical due to space restrictions, dewatering impacts and slope instability associated with high slope pore pressure. Much of the design effort therefore went into developing a cost effective perimeter wall solution.

The cantilever wall with internal buttress berm was initially favoured to save on the cost of anchors. A sub-option of vinyl sheet piles to provide groundwater cut-off was considered, but wall displacements and space restrictions were problematic. To reduce movements of the cantilever wall to an acceptable level it was necessary to install the wall close to rock surface in combination with an internal berm (1V:4H). A series of limit equilibrium analyses were carried out using the software package SLOPE/W to optimise the slope geometry and check the internal berm stability (See Figure 3).

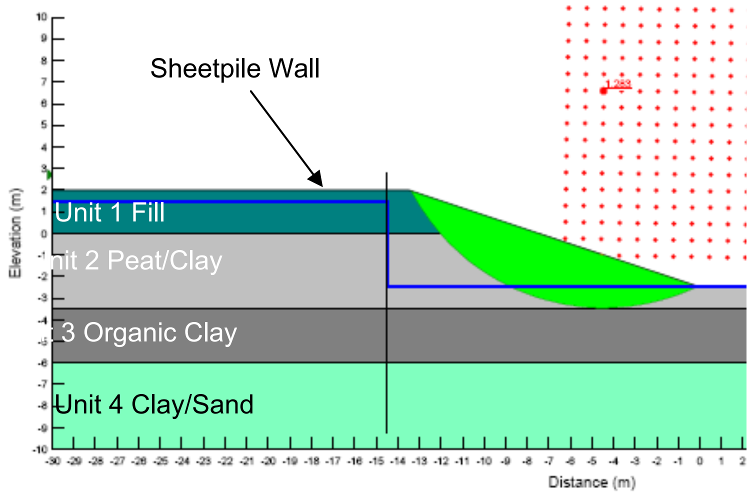

Figure 3: Stability Analysis for Internal Berm (fully drained).

As the design progressed, space constraints became significant so the internal berm was replaced by anchors, except in the area of the eastern entry ramp. Two types of sheet pile section were adopted (Larssen LX 16 and LX6W), depending on the retained height and proximity to sensitive structures. A schematic design in the vicinity of existing structures is shown in Figure 4.

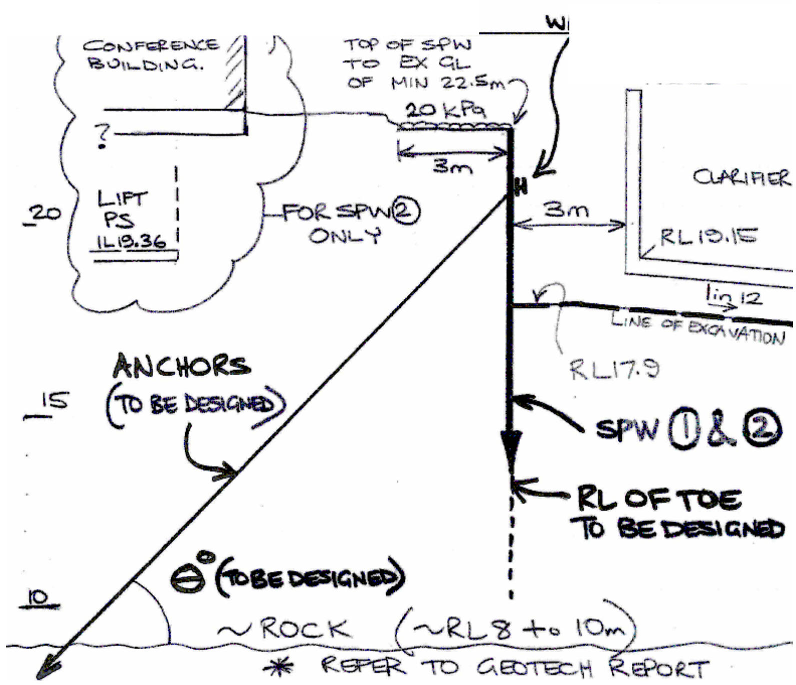

Figure 4: Schematic design of anchored wall (Note: Sydney Water datum used +20 m AHD).

The main design tool used to assess anchor and wall behaviour was WALLAP software. Groundwater behaviour near excavation works was analysed using the finite element package SEEP/W. Stability assessments for the berm and cantilevering wall options were checked using SLOPE/W.

Two main analyses were carried out:

- Case 1 – LX16 anchored sheetpile wall, as used near existing structures and
- Case 2 – LX6W cantilevering sheetpile wall with buttressing berm near location of haulage ramps.

Drained conditions in cohesive materials were not expected to occur within the construction timeframe for excavation Stage 1. Stage 2 was checked using drained and undrained soil parameters, because effective stress conditions had the potential to develop over the expected construction period of 6 months.

Checks were then carried out on wall displacement, safety factors, bending moment, shear force and wall buckling capacity. The whole design optimisation process and sensitivity assessment involved running over 100 WALLAP models to examine the effect of variables such as soil strength, drainage condition, wall toe level, anchor pre-stress and live load at each key cross section.

6 DESIGN OUTCOMES

For brevity only Case 1, incorporating the anchored sheet pile wall driven to rock, is discussed further in this paper. The results of critical WALLAP analyses are shown in Figure 5 and are discussed further below.

The factor of safety using a gross pressure method was calculated to be consistently above 1.8, and the controlling factor in the design was the management of wall movements to avoid impacting nearby structures – particularly along the northern wall.

Due to the low soil stiffness values used, wall deflections were found to be highly sensitive to surface loading and anchor pre-stress. Application of a full 20 kPa surcharge (i.e. large tracked excavator) during the cantilever Stage 1 excavation was predicted to cause unacceptable movements at nearby structures, and hence a construction plant exclusion zone within 3m of the wall crest was established along the critical northern anchored wall.

Key design findings are summarised below:

- Horizontal design waler loading at RL 0.5m AHD was approximately 130 kN/m run, based on a 30º inclination from horizontal, assumed anchor stiffness and pre-stress to 50% of anchor load.
- Sheet piles were not required to be driven to rock for stability or groundwater cut-off reasons, but were extended to sandstone rock surface to reduce the risk of wall movement under the vertical component of anchor loading.
- Anchor pre-stress of 50% working load (rather than 100%) was specified in order to accommodate additional anchor loads that WALLAP predicted would occur as excavation proceeded and reduce predicted movements back into retained soil. All temporary anchors were proof load tested to a minimum 125% working load.
- Estimates of vertical and horizontal ground movements away from the sheet pile wall were carried out using empirical design envelopes (Clough *et al.*, 1990). Limiting the maximum wall movement to 100 mm was found to result in acceptable movements at sensitive nearby structures which had to remain operational during the works.
- During construction, survey monitoring was used to check ground movements outside the excavation and wall crest movements. Anchor load behaviour was also monitored by a hydraulic load cell (See Section 8).

Seepage analyses indicated that groundwater cut-off was required through permeable fill around most of the excavation perimeter to reduce groundwater inflow to less than the required 1 ML/day. This flow rate was inferred to be the maximum amount that could be pumped into the sewage treatment plant. The tank design incorporated under-tank drainage to prevent buoyancy forces when tanks are emptied for periodic maintenance. To reduce the amount of under-tank drainage required, each tank incorporated a circumferential clay cut-off wall extending below the tank and under-tank drainage fill layer.

The SEEP/W seepage model extended about 50 m laterally westwards from the excavation crest to include the tidal creek, which flowed along the western site boundary and acted as a source of groundwater. The recharging effect of the dune aquifer and sea some 200 m to the east, was modelled using free and fixed head conditions at an infinite boundary. The actual response of the eastern boundary may lie somewhere between these conditions. A SEEP/W output showing the cut-off effect of the sheet pile wall with berm is shown in Figure 5.

Various sheetpile and berm configurations were modelled to establish cut-off depths and inflows during excavation and periodic dewatering of the under-tank drainage layer during maintenance. The key findings of the seepage analyses are presented below:

- It was found unnecessary to extend sheet piles to rock surface for adequate seepage cut-off, however there were overriding requirements to provide sufficient embedment for wall stability and to control displacements.
- During excavation, perimeter sheet pile walls installed near to rock reduced predicted steady state inflows into the clarifier from 2.2 ML/day for no cut off, 1.75 ML/day for 80 m of sheetpile wall, 1 ML/day for 220 m of sheet pile wall and 0.02 ML/day for full perimeter cut-off. Approximately 300 m of sheet pile wall was eventually installed, which resulted in estimated inflows of 0.01 to 0.1 ML/Day during excavation, based on intermittent use of a 50 mm diameter petrol powered submersible pump.

- Post construction steady state flows into tank drainage during operational maintenance were expected to be low at approximately 180-220 Litres/Day/Tank depending on whether full perimeter cut-off works were used. On this basis, permanent sheet piling was not necessary for the periodic, post-construction dewatering of the clarifiers. The design of the under-tank layer and drainage system included further work to optimise the size and layout of drainage pipes, control the grading of drainage fill and reduce clogging on the underside of the drainage layer.
- Groundwater drawdown and associated settlements outside the excavation were not significant for sheet piled options, which effectively prevented ingress into the excavation and limited impacts on external groundwater conditions. The volume of water pumped from the excavation was sufficiently low so that it could be treated on-site as part of normal effluent acid sulfate run-off.

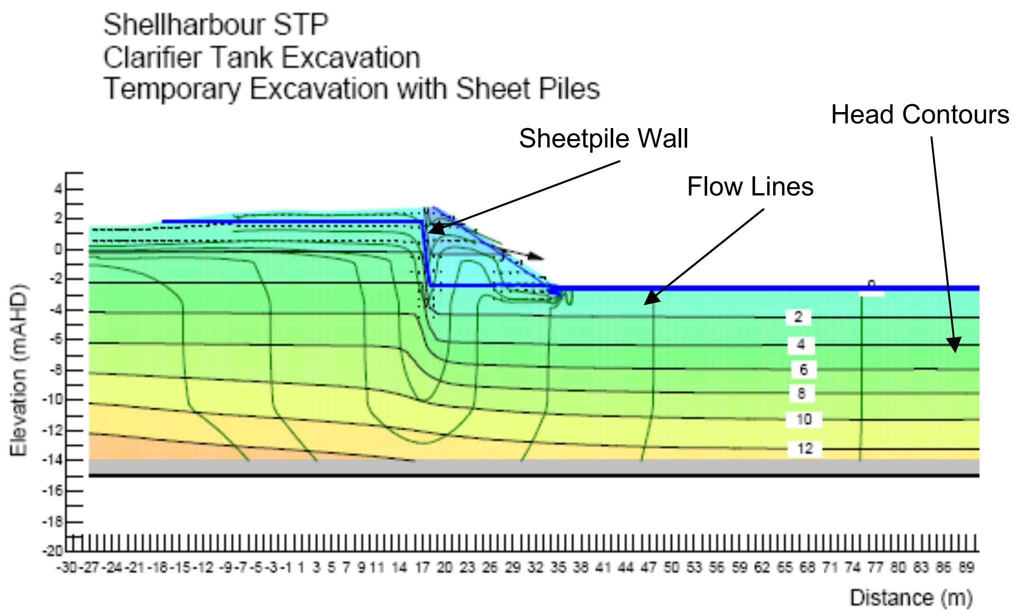

Figure 5: SEEP/W output.

Other design issues included providing suitable details for 'floating' (or un-piled) inter-tank pipework and thrust blocks taking lateral load. In particular, there were concerns that backfilling to higher site levels would cause significant loading of pipes between piled supports due to downdrag as underlying soils consolidated.

Load transfer from overlying soils onto to the 'bridging' pipe sections was estimated by considering both 'pull out' wedge mechanisms of material above the pipe and local pipe bearing capacity failure in uplift. The latter was analogous to the resistance available to laterally loaded piles. In all such cases, it was assumed that pipe bedding and trench details were in accordance with the Sydney Water specifications for Standard Trench Details.

7 CONSTRUCTION ISSUES

As excavation for the clarifier tanks and other subsurface structures was underway, it was observed that groundwater inflows through the gravelly fill layer were significantly less than anticipated. Excavations in fill for the chlorination tanks did not incorporate groundwater cut-off yet steady state recharge was low and manageable by intermittent pumping from a single sump point once the fill aquifer was drained. It was inferred from these observations that the hydraulic connection between the permeable fill layer and sources such as the tidal creek were not as well established as initially thought. It is thought that the original slurry wall may also have helped to reduce inflow and hydraulic connection. On this basis, a decision was made to replace a 60 m length of the sheet piled wall with a 1V:4H berm where space constraints and sensitive structures were not critical.

Figure 6 shows the clarifier excavation close to formation level.

A flood event also disrupted the initial excavation works, as a strategic decision had been made to set the wall crest at an elevation close to the 1 in 2 year flood event. However, the topography of the site was such that construction of a low embankment behind the wall crest along the western boundary provided significantly improved flood mitigation.

Additional analysis was then carried out to assess what the potential impact of embankment construction would be on the anchored wall. The geometry and location of the flood control embankment was adjusted to reduce the impact on the wall to an acceptable level and construction proceeded without further flooding.

Figure 6: Approaching final excavation level.

In regard to the design and installation of ground anchors, a preliminary anchor design featuring a fixed length in rock was carried out in order to finalise the anchored wall design and for cost estimation purposes. The successful anchor contractor put forward a non-conforming design which included multi-stage grouting of soil anchors in Unit 3 and 4 soils. Suitability tests on these anchors resulted in poor capacity, and they were rejected at an early stage. Further discussion on the effects of anchor pre-stress is contained in Section 8.

Special attention was also given to the design of the construction platform which was formed at the base of the clarifier excavation to provide support for piling rigs and to act as an under-tank drainage layer during periodic, post-construction tank dewatering. The platform was constructed from the gravelly coal wash reject material ("chitter") which was subjected to a range of laboratory tests to assess strength, durability, grading and resistance to weathering/breakdown. Tests included Particle Size Distribution Test (AS1289 3.6.1), a Slake Durability Test (ISRM) Point Load Strength Tests on Aggregate Samples (ISRM, 1985) and an Accelerated Weathering Test (RTA T103).

Interpretation of the test results indicated that nearly half of the material would breakdown under repeated wetting, drying and abrasion. It was inferred that the upper part of the as-placed construction platform would behave in a similar way under construction traffic, piling rig disturbance, etc. Based on the expected deterioration of the uppermost chitter material under trafficking, a 900 mm thick chitter layer was specified with an additional 300 mm sacrificial layer in areas trafficked by piling rigs. Following analysis, two layers of SS30 geogrid and geofabric were included in the platform.

8 OBSERVATION AND MONITORING

A monitoring plan was developed to include survey monitoring and anchor load monitoring. The plan included trigger levels which were divided into three levels of severity, based on a traffic light analogy (Green, Yellow and Red), in order of increasing severity. Corresponding mitigation actions were then tailored to reflect the sensitivity of particular structures. As with any monitoring plan, the purpose was to:

- Enable checking of actual versus predicted performance;
- Provide early warning of unexpected behaviour;
- Facilitate a managed response strategy (i.e. escalating trigger levels) and
- Protect stakeholders in the event of a dispute.

Following baseline and dilapidation surveys, monitoring was carried out during excavation and construction, at a frequency which was reviewed regularly and which was linked to the performance of the works. The initial frequency of displacement and anchor load monitoring took place on a two-weekly basis over the duration of excavation until movements and loads stabilised.

Trigger criteria were established to provide a framework of limiting thresholds, against which measured displacements/loads could be compared to determine what action would be needed to facilitate safe excavation. The assessment of tolerable ground and structural movements was based on numerical analyses and commonly adopted serviceability criteria for structures. Traffic light criteria were defined as follows:

- Monitoring results in the "green" range are acceptable and require no action, beyond continuation of monitoring, if appropriate.
- Results in the "yellow" range require immediate investigation to determine the cause of the deformations and possibly more frequent or additional monitoring.
- Movements or loads in the "red" zone require stoppage of work until the nature of the deformations and their potential effects have been fully assessed. In this case remedial works may be required, which could potentially include installation of additional anchors.

The specific trigger level criteria adopted for this project are shown in Table 3:

Table 3: Monitoring Criteria.

Item	**Green**	**Yellow**	**Red**	
			Max. Displ. (mm)	**Max. Angular Distortion (Δl/l)**
Lateral movement at crest of northern anchored wall (near structures)	<100 mm	101-150 mm	>150 mm	N/A
Lateral movement at crest of other walls	<100 mm	101-200 mm	>201 mm	N/A
Structures - high sensitivity (e.g. PST, Chlorination Channels, Tank Structures)	<5 mm	5 – 10 mm	>10	1/500
Structures: low sensitivity (e.g. Conf. Centre)	<15 mm	15 – 40 mm	>40	1/350
Road / footpath Areas	<30 mm	30 – 50 mm	>50	1/100
Anchor load	< 100% working load	100-120% working load	>120% working load	

Selected wall crest displacements are shown on Figures 7 and 8.

Figure 7: Measured displacements around clarifier excavation.

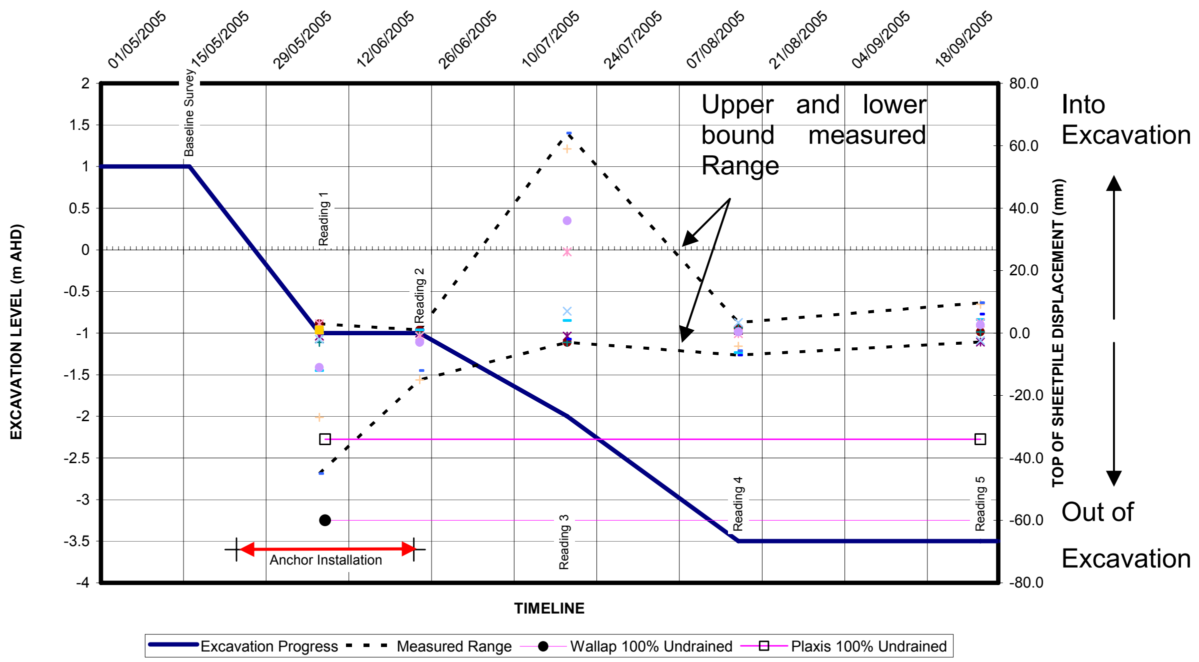

Figure 8: Measured and Predicted Displacements along Northern Wall.

Notable outcomes from the monitoring exercise are summarised below:

- Measured displacements and loads were all within the green or yellow range and no additional monitoring was necessary after the excavation phase. A plot showing the range of recorded movements is shown in Figure 8, which shows that actual crest movements were less than predicted.
- The anchor load cell was initially fitted to an anchor on the northern side of the excavation which was pre-stressed to 100% of working load. Load dropped 15% then recovered slightly, during a period of no excavation. This load change is inferred to be due to compression of retained soil and fill near anchor level.
- Due to a change in excavation programming, the load cell was relocated and the anchor only stressed to 50% of working load, in accordance with design recommendations. About 8 weeks after installation, following excavation to full depth, the anchor load had increased by 20%.
- The highest wall crest displacements of up to 124 mm were recorded in the vicinity of the cantilevering sheet pile walls at the eastern haulage ramp. The trend of data correlated well with proximity to the large spoil stockpiles, which were in the order of 5 m high (See Figure 7). Recommendations were provided at investigation stage to limit the height, slope and position of the spoil stockpiles and were reiterated during construction.

Figure 9 shows how the corresponding anchor loads varied over time, compared to loads predicted by PLAXIS and WALLAP software.

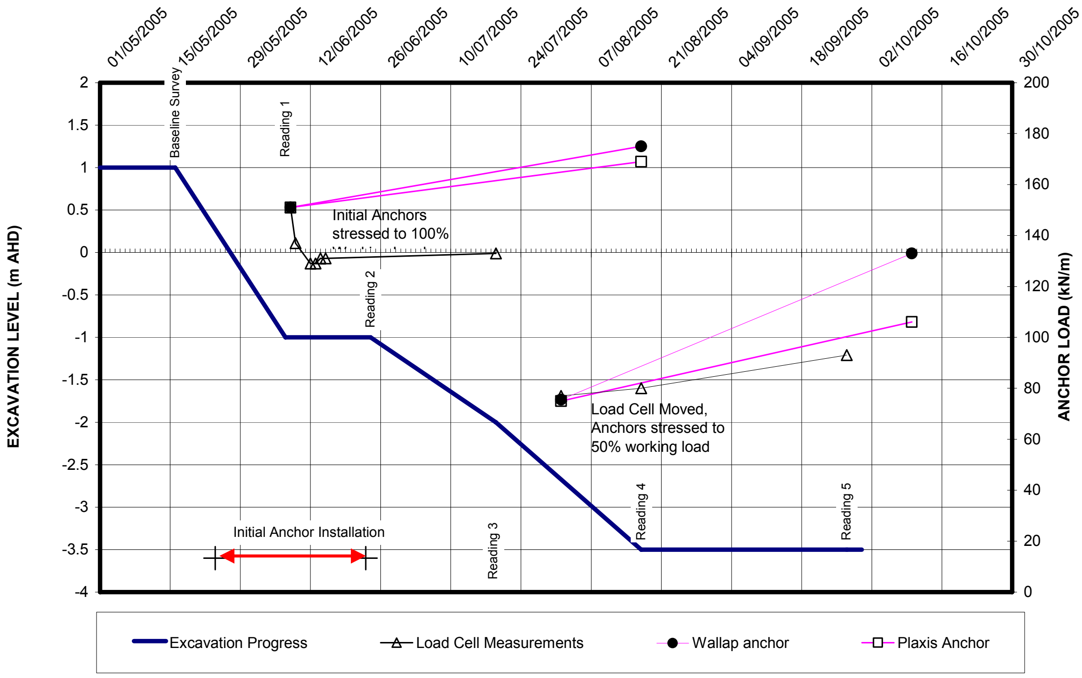

Figure 9: Measured and Predicted Anchor Loads.

9 PLAXIS COMPARISON

The authors felt it worthwhile to explore the issue of anchor behaviour and carry out a comparison against WALLAP, using PLAXIS finite element software. Where possible, the same input parameters were used for both programs (based on Table 2), although some judgment was necessary when considering input conditions such as wall/soil friction and constitutive model (elasto plastic, mohr coulomb used in PLAXIS).

Results for the two main construction stages comprising initial excavation to anchor level (Stage 1) and subsequent excavation to full depth after anchor installation are shown on Figure 10.

It can be seen from Figure 10 that the predicted movements at the initial cantilevering wall excavation stage are very similar and close to the upper range of measured wall displacements. Only results from undrained analyses are shown at Stage 1, due to the relatively short timeframe of this stage. Following anchor installation and excavation to full depth (Stage 2), the familiar anchored wall deflection profile is generated, although with a wide range of possible displacements depending on the amount of anchor pre-stress used and drainage condition.

Intuitively, the lower pre-stress value of 50% working load results in greater movement into the excavation. In these soft soil conditions, using 100% pre-stress resulted in large movements back into the retained soil. In order to limit the potential for such movement and to allow some spare capacity for the predicted rise in anchor loads as excavation proceeded, a 50% anchor pre-stress was specified on construction drawings. Interestingly, WALLAP predicted significantly larger movements into the retained soil than PLAXIS.

Initially, the anchor contractor stressed all anchors to 100% working load, and hence the applicable curves on Figure 10 (Stage 2) are those with negative value crest movements. The range of predicted wall movements (-30 mm to -80 mm) exceed the range of measured net crest movements along the northern wall (+/- 10 mm), although almost all of the 40 mm movement into the excavation was recovered.

The lack of movement back into retained materials may be due to a number of factors including higher-than-modelled soil stiffness, inappropriately modelled soil/wall friction in the anchor induced passive zone and the fact that large forces are required to initiate significant yielding in the passive anchor zone.

It is interesting to note that anchors installed with higher pre-stress lost load, whereas those with lower pre-stress picked up load (Figure 9). Loss of load may be partly due to consolidation of retained materials under highly pre-stressed anchor loading. Unfortunately no excavation occurred before the anchor was relocated, so we do not know whether loads would have increased as excavation took place. Conversely, the uptake of load in more lightly loaded anchors appears to support the modelling work, which may be a significant factor when considering appropriate pre-stress values and anchor capacity in deep soft ground excavations.

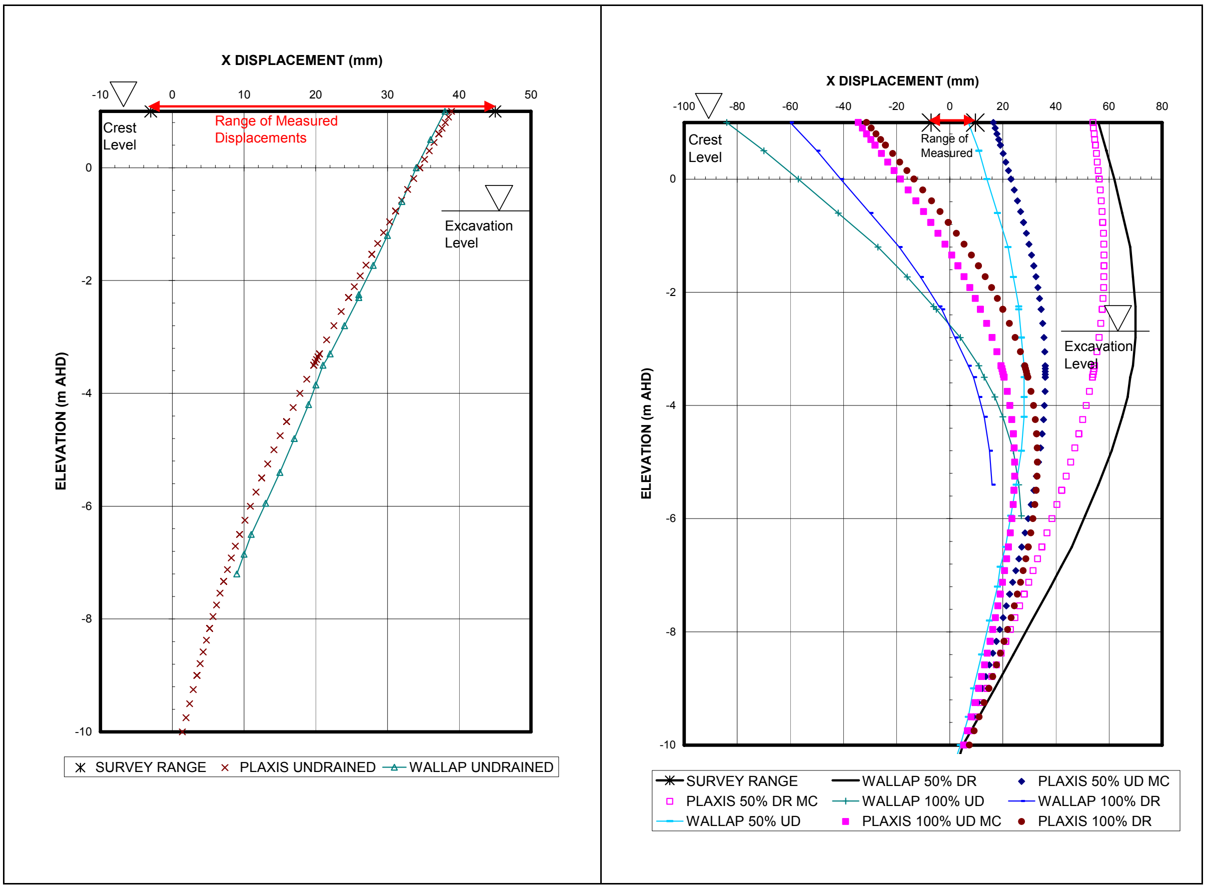

Figure 10: Measured and Predicted Displacements at Cantilever (Stage 1) and Full Depth (Stage 2).
Note: percentage relates to amount of anchor working load mobilised at installation.

10 CONCLUSIONS

This paper presents the findings of a case study on the investigation, design and construction of a large excavation as part of the upgrade of Shellharbour Sewage Treatment Plant (STP). The key outcomes of the case study are summarised below:

- The site is located in a former low-lying swamp, which has been filled over a wide area. A thorough site investigation provided a sound basis for design and revealed that the ground profile comprised gravelly fill (slag and coal wash reject) above organic peat and clays, overlying soft estuarine clay and progressively stiffer alluvial sandy clays. Bedrock surface (Tuffaceous Sandstone) occurred between RL-8 m and -14 m AHD.
- Groundwater was found at shallow depth throughout the low-lying site, and the surrounding tidal creeks influenced its behaviour. Actual inflows into excavations were significantly less than anticipated, which may be associated with the presence of an old slurry wall. An extended pump test using data loggers may have identified this at an early stage. Inflows were manageable during construction and under-tank drainage has performed well during post-construction dewatering.
- Selection of excavation support systems utilised WALLAP, SLOPE/W and SEEP/W numerical models. Both packages matched cantilever movements well, although over-predict movements into retained soil due to anchor loading. Post installation anchor loads varied by up to 20%.
- The authors consider that the use of the latter "one stop" design software incorporating stability, seepage and soil-structure interaction analyses would be used on similar future design tasks for convenience. In this case, further consideration of the most appropriate constitutive model would have been needed.
- Risks to existing infrastructure were successfully managed through the implementation of a monitoring plan incorporating red/yellow/green "traffic light" criteria. Measured displacements and loads were all within green or yellow range and no additional monitoring was necessary after the excavation phase.

- Construction issues were successfully resolved through the continuing, close involvement of geotechnical engineers through the construction stage. Examples include:
  - Flooding, which was successfully managed through construction of an additional embankment;
  - Construction platform, which involved thorough materials testing to verify the suitability of on-site materials for re-use as structural fill with good drainage properties; and
  - Reduced groundwater flow and monitoring, meaning savings could be achieved by replacing a significant section of sheet pile wall with an open cut slope.
- The response of displacements and anchor loads to the level of anchor pre-stress is of interest. The limited data in these soft soil conditions supports the adoption of reduced pre-stress so as to provide spare anchor capacity, although mobilising high pre-stress led to less uptake of load and did not appear to cause significant movement into the retained soil. Further research into the response of anchor loads in soft soil conditions would help to clarify these mechanisms and guide future design.

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