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Innovative earthworks to achieve settlement performance of a very deep fill

Une approaches innovante d'evaluation des tassements de remblai épais

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ABSTRACT: A quarry in the Melbourne suburb of Lilydale reached the end of its productive life and was sold together with surrounding land for residential and commercial development. The quarry is up to 140m deep and as part of the planned development the client wished to backfill the quarry with approximately 9 million cubic metres of stockpiled overburden accumulated from the quarrying operation. An assessment of the potential settlement of the fill was made using oedometer tests to simulate the initial compression, with samples then saturated to simulate the potential for groundwater recovery to cause additional settlement. The quarry filling has reached approximately 50% of its planned height and settlement monitoring has been undertaken. This paper details the innovative earth filling strategy, the site work undertaken and assesses the measured behaviour of the unsaturated compression that has taken place to date from a theoretical and practical perspective. The findings of this paper will provide meaningful technical guidance and reference for future large-scale earthworks projects.

RÉSUMÉ : Une carrière de 140m de profondeur dans la banlieue de Melbourne de Lilydale a atteint la fin de sa vie productive et a été vendue pour permettre le développement urbain. Le client souhaitait remblayer la carrière avec environ 9 millions de mètres cubes accumulés au cours de l'exploitation. Une évaluation des tassements potentiel du remblai a été faite à l'aide d'essais oedomètre pour simuler la compression initiale. Les échantillons on été ensuite saturés pour simuler le potentielle reéquilibrage des nappes phréatiques sur le long terme. Le remblayage de la carrière a atteint environ 50% de sa hauteur prévue et une surveillance des tassements a été entreprise. Cet article détaille la stratégie innovante des travaux de remblai et presente le comportement mesuré de la compression non saturée qui a eu lieu à ce jour d'un point de vue théorique et pratique. Les conclusions de ce document fournissent des conseils techniques et des références pour les futurs projets de terrassement à grande échelle.

KEYWORDS: Quarry backfill, lab testing, settlement prediction, monitoring data and final filling strategy.

1 INTRODUCTION

The former Lilydale Quarry Redevelopment comprises 163 Ha of land proposed for residential, retail, commercial and community uses. The site previously incorporated a limestone quarry and lime production facility which operated since the late-19th century. The quarry is up to 140 m deep (RL 155m to RL15m) and covers an area of approximately 25 Ha at the surface, where RL is reduced level in AHD. Subject to settlement performance of the filling it is proposed that the central portion of this area will be developed as a 'neighbourhood centre', with medium density townhouses, commercial and retail facilities, and potentially higher density residential apartments of 4 to 8 storeys. A new railway station is proposed beyond the western edge of the existing quarry. The northern part of the quarry is planned as public open space which will be handed back to local Council.

A volume of 8.6 million m^3 of stockpiled overburden material had been placed immediately east of the quarry and an additional 1 million m^3 of material is expected to be won through general site earthworks of the surrounding subdivision works. The volume of space to be filled inside the quarry has been estimated to be approximately 9.6 million m^3. After regrading of the surrounding ground and filling to the required design levels the maximum depths of fill will range from 110m to 120m.

2 GEOTECHNICAL INVESTIGATIONS

The site has been the subject of considerable geotechnical investigation covering numerous issues related to pit batter stability and backfilling. This paper will only deal with the consideration of the settlement of the fill material and how the filling has been specified and controlled to limit future settlement.

Samples of the borrow material were placed in oedometers at a range of dry density ratios from 95% to 110% Standard (AS 1289 5.1.1). The samples were initially loaded in a dry state up to 1600kPa to simulate initial loading that was aiming to represent in-situ compaction. They were then saturated to simulate rising groundwater to assess the impact of saturation. Samples selected varied from clay, silty clay and gravelly clay to represent variations of potential backfill materials with key test results as summarised in Table 1.

Table 1 Oedometer test results on potential fill material (23 tests)

Soil Type	Initial Moisture Content (w_o, %)	CCR (%)	Initial Dry Density ($\gamma_{d,0}$, t/m^3)	CR	p_c' (kPa)
Clay	16.0 - 19.9	0.00 - 0.73	1.56 - 1.91	0.06 - 0.12	65 - > 1600
Gravelly Clay	20.0 - 21.7	0.00 - 8.29	1.47 - 1.77	0.05 - 0.16	480 - 820
Gravelly Clay	18.9 - 19.2	0.00 - 0.69	1.75 - 1.80	0.05 - 0.16	400 - > 1600
Clay	21.0 - 21.3	0.44 - 0.54	1.66 - 1.73	0.05 - 0.16	> 1600
Silty Clay	23.0 - 25.1	0.69 - 11.28	1.42 - 1.65	0.05 - 0.16	250 - > 1600

The purpose of these tests is to understand how each sample would behave under various loading criteria and to develop empirical relationships between key settlement related parameters (compression ratio CR = $C_c/(1+e_0)$, collapse compression ratio CCR = $\Delta e_w/(1+e_0)$, pre-consolidation pressure p_c') versus initial dry density ($\gamma_{t,0}$), where C_c is compression index; Δe_w is change in void ratio due to collapses at macro and micro structural levels within the sample attributed to sample saturation). The test data was found to be very scattered when plotted against initial dry density. With increasing initial dry density, both CR and CCR were found to decrease and p_c' to increase. As expected, the samples had the potential to collapse due to saturation under a constant stress. The CCR values dropped with increased initial dry density to a point (approximate 1.62 - 1.65 t/m^3) where a trend was observed without much change in CCR values despite increasing initial dry density. This phenomenon is believed to reflect that collapse compression took place at a macro and micro level within the samples. It was therefore concluded that saturation collapse compression could be limited by maximising the compacted dry density in the fill. It was also observed that compression took place rapidly for a sample prepared at a state with initial moisture content drier than the Standard Optimum Moisture Content (SOMC). This behaviour is believed to be attributed to the presence of a large volume of air retained within the sample. i.e. high air voids, which could be reduced at a particular moisture content by increasing the compacted density. Based on this finding, it is postulated that the rate of compression can be reduced by compacting the fill material to have low air voids even when placed dry (up to 5%) of SOMC. More discussion can be referred in Section 8 of "Settlement Model" below.

3 GROUNDWATER

The natural groundwater level at the quarry had been lowered over time as the depth of the quarry floor increased. The initial groundwater level at the time of the commencement of rehabilitation was at RL 0 m.

As part of planning for the quarry to be backfilled to facilitate residential development simple 2D modelling was undertaken to assess the recovery rate of groundwater levels within the quarry once filled. The groundwater level in the former quarry area is expected to rebound up to RL 88m but this could take many years.

A permanent sump was constructed from the base of the quarry and raised progressively as the filling occurred. This allows groundwater levels to be controlled by pumping from the sump. To date the groundwater level has been maintained at about RL 15m but recently the pumps were turned off to allow groundwater to rise and study how quickly groundwater recovery occurred and any impact on the monitored rates of settlement.

Consideration is being given to permanent dewatering if the rate of groundwater recovery and impact of settlement cannot be reliably predicted. The groundwater currently sourced from dewatering is already providing benefits to local water courses and could provide further benefits in irrigation of the developed land.

4 QUARRY PREPARATION

At handover, the quarry was left in an operational state with haul roads into the base of the pit and some fill on the quarry floor. Due to the uncontrolled nature of this fill, it was removed from the quarry with the oversize rock separated, and the remaining material replaced in a controlled manner. All surfaces were cleared of uncontrolled fill and these will continue to be inspected by geotechnical staff prior to placing engineered fill. This includes removal and disposal of any vegetation.

Loose rocks were removed from the quarry walls so that the exposed faces were intact and solid, allowing engineered fill to be compacted against the walls. Where cavities have been observed in the face of the quarry walls, flowable cementitious fill and/or concrete has been used to seal these off from the fill material so that the engineered fill cannot be eroded into the cavities when the water table is allowed to rise.

A drainage blanket was placed over the entire floor of the quarry to direct water to the sump located in the northern end of the pit. The drainage blanket was constructed from boulders in the stockpile that were crushed on site to produce a well graded material with a maximum particle size of 300 mm and no particles finer than 50 mm. The blanket has a minimum thickness of 1 m but in many places where uncontrolled fill was removed from the floor of the pit, the thickness is much greater.

The material was placed in 500mm loose layers and compacted with a 16t vibrating smooth drum roller. Each layer was proof rolled with 100t fully laden dump trucks. The surface of the blanket was topographically surveyed.

Above the drainage blanket a 300 mm thick coarse filter was placed. This material was also sourced from on-site crushing and comprises a material of 50 mm maximum particle size and no material finer than 20 mm. A Bidim A34 geotextile was then placed above the coarse filter to restrict the migration of fines from the engineered fill in the event that surface water penetrates the fill.

The sump riser is a reinforced concrete structure founded on the base of the quarry, with holes in its base to allow water to enter from the drainage layer. It is founded on solid rock at the quarry base and will rise to at least the expected groundwater level of RL 88 m, but potentially to the final filled surface.

A 1 m wide layer of coarse rock is placed around the sump to allow surface water to drain to the floor and be pumped out. A layer of Bidim A34 is placed between the engineered fill and the drainage material to stop fines being eroded from the engineered fill.

5 FILLING PROCESS

The backfilling plan for the quarry includes placing fill up to approximately 120 m deep. A review of international case studies yielded no relevant examples of projects where development has taken place on such a depth of filling. There are local examples in Victoria where quarries have been backfilled to depths in the order of 30 m to 40 m and developed.

The client for the project has been made aware that the estimated settlement cannot be predicted with confidence at this time, due to the uncertainty about the compressibility of the fill. Hence it is proposed to use an observational approach to the settlement of the filling and to modify the placement methods if needed as the work progresses. To this end, a collaborative contracting approach has been used to engage the contractor to undertake the filling of the quarry so that the developer can maintain control over the quality of the work.

The relevant Australian Standard for Earthworks for Residential Development is AS3798. This Standard was first developed in the 1990s to provide a consistent approach to such work and to overcome issues with uncontrolled fill on residential building lots.

AS3798 was not intended to be suitable for placing fill up to 120 m deep. Such filling is usually only undertaken in the construction of large dams. To assess the method for undertaking filling, the investigation work detailed above was undertaken, which led to the development of a bespoke specification .

6 ENGINEERING FILLING SPECIFICATION SUMMARY

The material for the engineered fill is sourced from overburden stockpiles and excess cut material on site. During quarrying works the material had been mostly placed on the eastern side of the quarry in terraced platforms in the order of 40 m deep.

This material has been sampled by drilling boreholes and excavating test pits and comprises a mixture of materials removed as part of the overburden stripping and includes very large boulders through to clay fines. Any observed contaminated material, topsoil or organic material will be precluded from the engineered fill..

The primary objective of the specification is to achieve economical backfilling of the quarry to a standard which will allow development at some time in the future. Filling is planned to take about five years and has been taking place for over 2 years with all uncontrolled fill removed and preparatory works completed (2021). Approximately 3 million m^3 of fill has been placed up to about RL70m – RL90m at time of preparing this paper. During this time settlement monitoring has been undertaken and compared to the original settlement estimates.

Based on the information derived from the investigations, a minimum average dry density ratio of 101% Standard for a day's placement has been adopted, with no test result below 98% Standard. This compares to the requirements of AS3798 (the Australian Standard for Earthworks for Residential Development) which require a dry density ratio of 95% Standard for residential development. To achieve this density, initial roller trials adopted 8 passes of a Cat 825 compactor with a loose layer thickness of 400 mm. This has now been demonstrated to achieve and often exceed the minimum density requirements. The average Density Ratio to date is 102.6%. A maximum particle size of 300 mm has been adopted and the moisture content has been maintained at that from the stockpiles, which is generally dry of Standard Optimum Moisture Content, which facilitates achieving higher density.

To compact material close to the walls of the quarry a smaller compactor has been used. This is a 16t vibrating sheepsfoot roller, and again it has been demonstrated that 8 passes of this roller achieves the specified minimum density. Both rollers are equipped with location sensing equipment which allows their tracking to be recorded over any point in space. This technology can be used to verify that each layer does not exceed the maximum thickness and has at least 8 passes. All earthworks are being conducted under Level 1 supervision as defined in AS 3798.

7 MONITORING

To compare the actual performance of the fill material and groundwater to the modelled estimates, monitoring is being undertaken during construction of the quarry backfill and will continue after filling is completed. This will allow the performance to be compared to the geotechnical and hydrogeological models that have been developed and used to make the estimates of settlement and groundwater rebound.

The proposed instrumentation to allow this monitoring to take place includes 5 settlement arrays across the quarry width. The first two arrays were installed at RL 35m (southern array) and RL 25m (northern array) with shape array locations presented in Figure 1. The plots of settlement and fill level versus time are shown in Figure 2 (southern array) and Figure 3 (northern array) below. A third array was recently installed at RL66m with two settlement plates. Two more arrays will be installed at higher levels in the fill, together with some settlement plates for the purpose of counter check and calibration. Piezometers are also installed in each settlement array but as, yet no positive pore pressures have been measured.

Once the quarry filling is completed, a comprehensive surface settlement monitoring network will also be set up to allow ongoing survey of the surface settlement. This data will be compared to the estimates from the modelling.

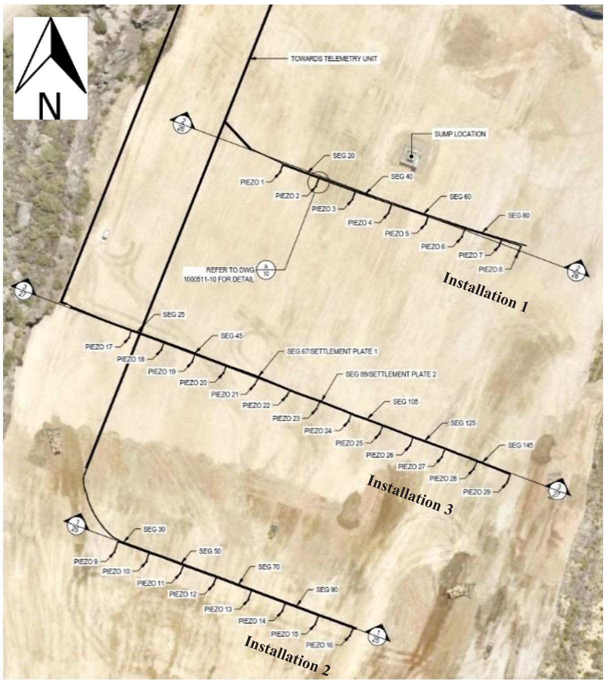

Figure 1. Shape Array Location Plan

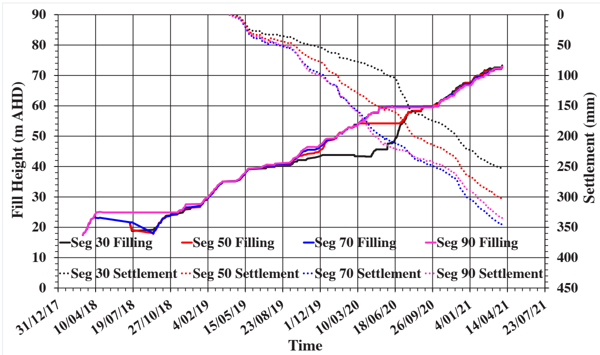

Figure 2. Plots of measured settlement (southern array) and fill level versus time

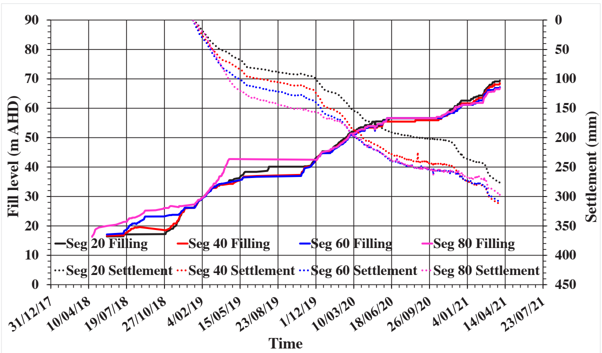

Figure 3. Plots of measured settlement (northern array) and fill level versus time

As shown in Figure 1, the shape arrays have been installed across the width of the quarry pit with differing depths of fill beneath them. Approximately 9.6m (min. 9.45m, max. 9.7m) has been placed in the north and 17.5m (min. 17.49m, max. 17.67m) in the south before installation of the shape arrays. The shape array settlement data compared with the filling profile shows self-weight compaction as the fill is being placed, and consolidation/creep settlement once the filling ceases for long periods of time. It has been noted that approximately 75% to 85% of the observed settlement was attributed to immediate compression and approximately 15% to 25% attributed to primary consolidation due to likely dissipation of induced air pore pressure as a result of earth filling. The rate of primary consolidation due to likely dissipation of induced air pore pressure would be expected to slow down due to the increase in the length of drainage path as a result of earth filling. At this time there appears to be no saturation induced settlement. As noted in Section 9 there are recent indications of pore water in piezometers but this may be due to saturation and not groundwater infiltration.

8 SETTLEMENT MODEL AND PREDICTION

As discussed in Section 2, significant scatter was observed within the laboratory test data, likely due to the natural variability of the soil within the stockpiles. From observations, the characteristic of the CCR due to saturation was very much dependent on the initial dry density. Despite the test data scatter, general trends and relationships have been still evident for CR, CCR, and p_c'.

$CR = 0.485 - 0.227\gamma_{d,0}$ (mean + one standard deviation for upper bound settlement estimation) (1a)

$CR = 0.349 - 0.167\gamma_{d,0}$ (mean - one standard deviation) (1b)

$CRR = 0.15\ CR$ (2)

$CCR = 0.616 - 0.357\gamma_{d,0}$ (where $\gamma_{d,0} < 1.62\ t/m^3$) (3a)

$CCR = 0.0060$ (where $\gamma_{d,0} > 1.62\ t/m^3$) (3b)

$CCR = 0.581 - 0.341\gamma_{d,0}$ (where $\gamma_{d,0} < 1.62\ t/m^3$) (4a)

$CCR = 0.0030$ (where $\gamma_{d,0} > 1.62\ t/m^3$) (4b)

$LOG\ (p_c') = LOG\ (7.297) + 0.907\gamma_{d,0}$ (mean - one standard deviation for upper bound settlement estimation) (5a)

$LOG\ (p_c') = LOG\ (19.808) + 0.907\gamma_{d,0}$ (mean + one standard deviation) (5b)

Where, Equations 3a and 3b (mean + one standard deviation) are for upper bound collapse settlement estimation; Equations 4a and 4b (mean - one standard deviation) are for lower bound collapse settlement estimation; $\gamma_{d,0}$ is the initial dry density in t/m^3; CRR is re-compression ratio, i.e., $CRR = C_r/(1+e_0)$.

These relationships have been used in the settlement predictions described in the section below. Equations 1a & 5a have been used for upper bound settlement estimation. Equations 1b & 5b have been used together with Equations 1a & 5a for lower bound settlement estimation, i.e., average of settlement estimated using Equations 1a & 5a and Equations 1b & 5b is defined as lower bound settlement. However, it needs to be recognised that the samples tested only represent the finer portion of the material taken from stockpiles. The coarse component was removed as the samples are only 19mm in height. The coarser fraction can be expected to reduce the settlement estimates made from the oedometer tests. Nevertheless, it is critical to know what materials are used and the level of compaction achieved in the field in terms of assigning representative relevant design parameters.

The settlement of the earthfill in the pit will most likely consist of three components, i.e., settlement due to self-weight compaction, settlement due to wetting as groundwater rises, and long-term creep settlement. The laboratory tests show the compacted samples to exhibit typical consolidation behaviour, with an inherent pre-consolidation pressure that controls when the sample yields under external applied pressure. This inherent pre-consolidation pressure is dependent on the material type, the compaction energy applied, and the initial moisture content. As the load increments were increased while the samples were "dry", the samples would still be partially saturated, and voids would contain both air and water before testing. The samples would start to yield once the applied pressure exceeds the pre-consolidation pressure, and significant excessive pore water pressure would also be developed as the applied pressure increases. It was also observed from the permeability tests that the partially saturated samples were generally of low permeability. In theory, therefore, the dissipation of the excess pore water pressure should be very slow. However, our observations from the oedometer tests were that the rate of consolidation was much quicker than expected. This behaviour may be attributed to the following two factors: 1) the presence of air allows the sample to be compressed much quicker; 2) the water within the sample was not free water, but was bonded to the clay mineral structure, and therefore not mobile. For self-weight compression, only the total settlement has been assessed. Prediction of associated excessive pore water pressure and its dissipation has not been carried out as prediction of excess pore pressure generation in unsaturated soil is very dependent on many factors including recovery of groundwater.

Based on the available in situ density testing, the fill materials have been compacted above the Standard maximum dry density, with an average dry density of 2.027 t/m^3 and a standard deviation of 0.158 t/m^3. The average dry density minus one standard derivation (1.869 t/m^3) was adopted for the settlement estimates. Using Equations 1 to 5, the compression ratio (CR) and pre-consolidation pressure (p_c') were estimated to be 0.0597 and 361kPa, respectively for upper bound estimation. The recompression ratio (CRR) was obtained by adopting a ratio of 0.15 for CRR / CR, giving an estimated value of 0.0089.

As discussed above, the relevant parameters defined in Equations 1 to 5 have been used to calculate the one-dimensional consolidation of the fill for a typical section through the quarry pit, considering various fill thicknesses. Both the total settlement due to self-weight compression and the estimated measurable settlement on the site were assessed, and the results are presented in Figures 4 and 5, respectively. Figure 4 presents plots of ground profile and upper bound total construction settlement, collapse settlement and creep settlement versus horizontal distance. Figure 5 shows the expected measurable profile that could be measured using typical settlement plates, extensometers, or horizontal settlement profilers.

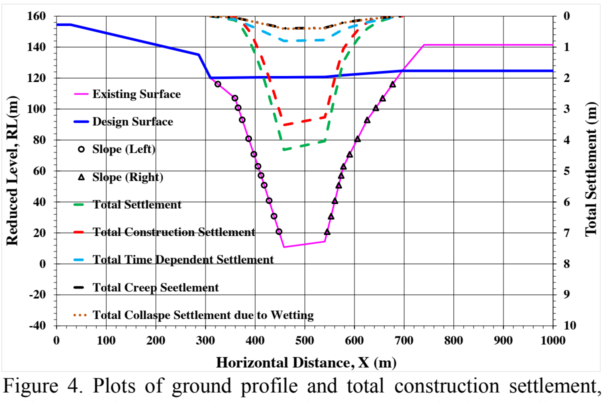

Figure 4. Plots of ground profile and total construction settlement, collapse settlement and creep settlement versus horizontal distance (upper bound)

From Figures 4 and 5, the total settlement due to self-weight compaction is estimated to be in the order of 2.2 m - 3.4 m, although most of this settlement is expected to occur during construction. It is also expected that some differential settlement will occur across the section, particularly where the fill thickness varies significantly over a short distance at the quarry faces.

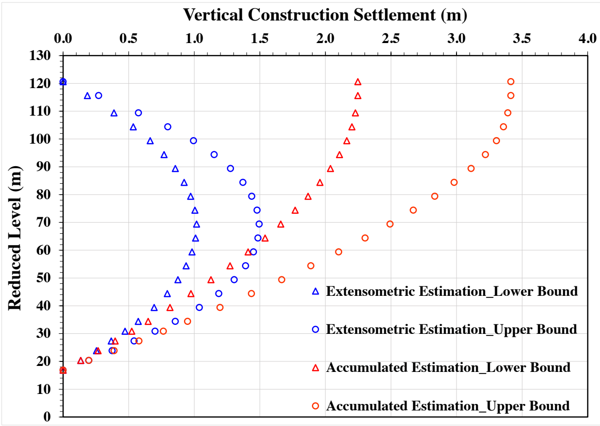

Figure 5. Plot of total measurable vertical settlement versus reduced level at end of construction

If pumping ceases, groundwater is expected to rise slowly to approximately RL88 m AHD. As a result, the fill within the pit will eventually become fully saturated below this level. The time for the fill to achieve full saturation is dependent predominantly on the permeability of the compacted material, the presence of drainage paths, and the time taken for groundwater to recharge outside the pit walls. The total collapse settlement due to wetting is a function of the collapse compression ratio, thickness of fill, and heave, and can be estimated from: Collapse Settlement = CCR × Fill Thickness-Heave. The estimated maximum total collapse settlement due to wetting is estimated to be approximately 0.2 m to 0.4 m. The settlement will occur as the groundwater level rises, which has been estimated to take in the order of twenty years.

Creep settlement of fill is likely to occur even in granular soils. Creep settlement would occur after completion of filling and can be estimated from: Creep Settlement = $c_{\alpha e}$ × Fill Thickness × LOG((t_2+t_1))/t_1). Where, t_2 is the design life, t_1 is the time between completion of filling and commencement of construction, and $c_{\alpha e}$ =3% of CR. Assuming creep commences at the end of construction, it is estimated that approximately 0.3 m to 0.4 m of settlement will occur over the next 50 years.

Based on the assumption that creep and collapse due to groundwater commence at the end of construction, it is estimated that settlement in the order of 0.5 m to 0.8 m will occur over the next 50 years. Depending on the design settlement for the facilities to be built and the method of construction, a delay is likely to be required after filling to allow for some of this settlement to take place.

If it is assumed that construction is delayed for 5 years after filling, the modelling estimates that design settlement of approximately 300 mm will occur over a 50-year design life if pumping is decided to be in continuous operation. This will be most likely creep settlement. The actual delay time will be determined based on actual performance of the settlement as measured and the design settlement criteria which are required to be met for the asset types to be constructed in different parts of the filled area.

The maximum combined settlement for the 50 years post commencement of filling is therefore estimated to be in the order of 3.0 m (2.2 m + 0.8m) - 4.2 m (3.4 m + 0.8 m). The settlement estimates presented here are preliminary and subject to verification by field measurements. Back analysis should be carried out using in situ field measurements to revise key design parameters. A further review of the settlement predictions will be undertaken as settlement is measured during construction, to allow calibration of the model and improved confidence in settlement estimates. As the current estimates are based on limited laboratory data, the settlement estimates within this paper should only be used as a guide for the magnitudes expected both during and following the filling works.

9 COMPARISON OF ACTUAL TO PREDICTED SETTLEMENT

As discussed above, the shape arrays have been installed across the width of the quarry pit with differing depths of fill beneath them. The shape array settlement data compared with the filling profile shows self-weight compression as the fill is being placed, and consolidation settlement once the filling ceases for extended periods of time.

The vibrating wire piezometers (VWPs) have been installed across the width of the quarry pit to observe development of positive pore water pressure as groundwater recovers or positive pore pressures are developed in the fill (refer to Figure 6 & Figure 7). As groundwater pumping has now ceased as part of the groundwater recharge trial, the groundwater table has begun to recover in the ground surrounding the quarry, and in the sump, which is hydraulically connected to the surrounding ground through the drainage blanket. At this stage, the VWPs of the southern array have not shown any signs of groundwater ingress. There is some fluctuation in the data from installation 1 (north), however, the cause of this has not been confirmed. Hence the rate of groundwater infiltration into the quarry backfill is not known at this stage.

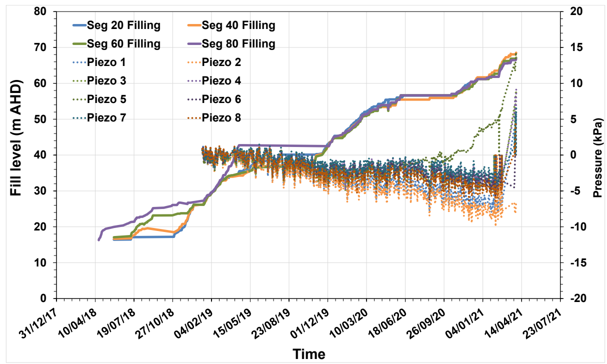

Figure 6. Plot of Vibrating Wire Piezometers installation 1 versus fill height above shape array (north)

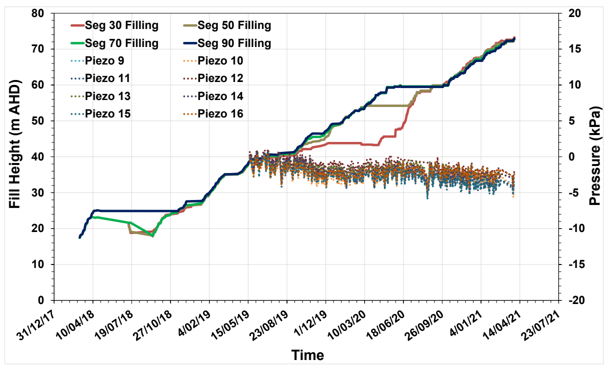

Figure 7. Plot of Vibrating Wire Piezometers installation 2 versus fill height above shape array (south)

Figures 8 and 9 below, present the measured settlement data from each of the shape arrays compared with the settlement estimates, respectively. The estimated settlement bounds have been presented in different ways according to different locations to reflect actual performance of the earthfill. For example, the measured settlements fit better with predicted values on upper bound under shape array 1 (northern) (Figure 8) but match better with predicted values on lower bound under shape array 2 (southern) (Figure 9).

From these comparisons of the measured values versus the estimated settlement values, the performance of the fill under the southern shape array is better than the fill under the northern shape array. The reason for this variation in performance is not known at this stage but is most likely a reflection of the variance in the fill behaviour that is to be expected throughout the quarry. The material used for filling under both shape arrays was similar, having been sourced predominantly from the eastern stockpile, and compaction results were consistent in both areas. In both cases the actual settlement is within the estimated settlement range. This will continue to be reviewed as further settlement data is compiled and analysed together with additional shape arrays as they are installed.

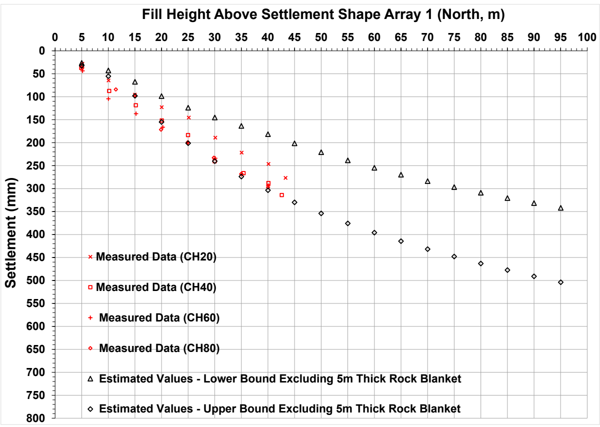

Figure 8. Plot of settlement versus fill height above settlement shape array 1 (north)

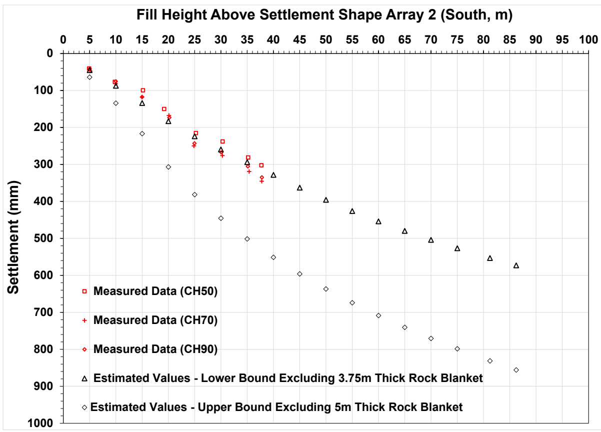

Figure 9. Plot of settlement versus fill height above settlement shape array 2 (south)

To date the measured settlement data indicates the fill performance at both shape arrays is lying within the estimated settlement range. The difference in stiffness of the soil underlying the southern and northern shape arrays is possibly natural variation in materials.

10 FINAL SETTLEMENT ESTIMATES AND IMPLICATIONS FOR DEVELOPMENT

If the masterplan development is to proceed then structures and the supporting services will need to be designed to accommodate the predicted total and differential settlement. For the reasons outlined in this paper, it is premature to confidently predict what this design will require and how long it will take for settlement to reach acceptable levels prior to development. However, if settlement monitoring provides confidence that the development can proceed, a variety of concepts will be considered in designing structures to accommodate the predicted settlement. These concepts could include one or more of the followings:

- Surcharging of the fill in areas of proposed development.
- Use of basements to unload the fill under structures.
- Stiffened raft foundations to reduce differential settlement in the structures.
- Piling to quarry benches at the quarry perimeter.
- Flexible service connections to accommodate differential movements.

Such designs will be undertaken to meet all appropriate design and construction standards for the structures proposed.

The settlement monitoring of the fill will be crucial in gaining confidence in future settlement predictions. Settlement will be measured at different levels in the filling using a variety of methods and at the finished surface. Results will be gathered at a regular frequency so that reliable data is available to analyse and from which to make predictions.

The settlement behaviour of deep fill is complex and dependent on several variables. Broadly speaking, the settlement can be separated into three main components, i.e., self-weight compaction, wetting induced settlement, and creep settlement as discussed above.

Much of the settlement resulting from self-weight compaction is likely to occur during the filling works, while the wetting induced collapse settlement and creep settlement are time dependent. Preliminary estimates indicate the settlement from self-weight compaction could be in the order of 2.2 m to 3.4 m, and the time dependent settlement could be up to 0.5 m to 0.8 m over 50 years, giving a total settlement in the order of 2.7 m to 4.2 m.

The current modelling suggests indicative settlement of 300 mm over 50-years if construction of assets is delayed for 5 years post filling and dewatering is continued. Variations on this prediction are likely based on the time it takes to physically complete the filling works, actual settlement performance of the fill, the assets being developed on the fill and any other actions taken to mitigate the effects of settlement on the assets.

The estimates are preliminary and based on a limited number of laboratory tests. Back analysis will be carried out using data from in situ field measurements, and the prediction models updated and refined based on actual settlement values. By the time construction of the earthfill is complete the model will be updated using actual settlement and pore pressure measurement data. This will provide more confidence in settlement predictions for design of assets.

11 ACKNOWLEDGEMENTS

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