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Capping design for 2 tailings storage facilities with unique geotechnical and regulatory challenges

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Abstract

This paper describes the design and regulatory challenges encountered in the capping of 2 tailings storage facilities (TSFs) in Western Australia containing slightly radioactive tailings.

The tailings has unique properties, comprising mostly clay-sized particles with a low plasticity index, high liquid and plastic limits, high void ratio and low dry density. The site is regulated under a landfill licence while dam safety guidance also applies.

One of the key challenges of capping soft tailings is to provide safe and stable access for construction equipment. To estimate the tailings strength, existing cone penetrometer test (CPT) data, dynamic cone penetration (DCP) tests and vane shear tests (VST) were used. DCP tests and VSTs carried out at the same time and location yielded conflicting results. DCPs sank under self-weight, indicating very soft soil, while VSTs and test pits confirmed a soft to firm tailings strength. The potential mechanisms for this discrepancy are investigated.

The sites are regulated under a landfill licence and require design of a geomembrane capping system to achieve maximum infiltration requirements. Given the tailings is highly compressible, over 3 m of settlement is expected. The final landform therefore had to be over-built to achieve a self-shedding surface after settlement.

To reduce earthworks, the existing embankments were cut back for one site, requiring consideration of the existing freeboard requirements and geomembrane liner connection details. It is proposed that the required environmental freeboard no longer applies once the tailings is covered and the dam safety freeboards no longer apply once a design storm can be released in a controlled manner.

The slight radioactivity of the tailings was accounted for by using a cap of sufficient thickness to delay the emission of radon until it has mostly decayed. The selected cap comprised general fill, a clay-rich subgrade, geomembrane and a revegetation layer.

Keywords: *TSF closure, dynamic cone penetrometer, vane shear test, geomembrane, NORM, geosynthetics, regulatory requirements, soft tailings*

1 Introduction

Tailings can have a wide range of geotechnical, geochemical and environmental properties that must be accounted for in the cover design for tailings storage facilities (TSFs). ATC Williams Pty Ltd (ATCW) was engaged to design a cover for 2 lined TSFs in Western Australia, referred to as Cell 1 and Cell 2. Both cells store soft clay tailings, containing low levels of naturally occurring radioactive material (NORM).

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Typically, TSFs are regulated under mining guidelines, which in Australia usually stipulate closure to achieve a safe, stable, sustainable and non-polluting (or similar requirements) landform. Unlike most TSFs, these 2 TSFs are regulated under a landfill licence as they form part of a municipal waste management facility.

The TSFs were constructed by partially excavating the ground within the storage area followed by construction of above-ground embankments (turkey's nest–style TSF) on existing ground. The facilities are composite-lined TSFs, with the liner comprising a high-density polyethylene (HDPE) geomembrane overlying a geosynthetic clay liner on a prepared subgrade.

Both TSFs have underdrainage systems above the liner to facilitate drainage and consolidation of the tailings. Underdrainage from Cell 1 gravity drains through the embankment into the leachate pond. Underdrainage from Cell 2 is extracted via pumps from 2 sumps and pumped to the leachate pond.

Tailings in Cell 1 was discharged from 3 central discharge points, creating a mounded landform. Tailings in Cell 2 was discharged from the north, east and south embankments, creating a tailings beach sloping with a gradient 0.6% towards the western embankment.

The site owner's objective is to cap the tailings as soon as feasible to reduce leachate generation from rainfall. Leachate treatment capacity on site is limited and offsite treatment of leachate is costly. To permit capping, the tailings must achieve a minimum strength sufficient to safely support construction equipment. Cone penetrometer test (CPTs) carried out on Cell 1 in 2022 indicated a very low tailings strength at the time of testing. An access causeway constructed at the time to allow access by a CPT rig took weeks to construct and tailings heaved under the weight of the causeway. This paper describes the investigations carried out to derive design parameters.

2 Tailings properties and investigations

2.1 Chemical and waste classification

The waste is referred to as a 'neutralised chloride waste residue' with a typical chemical composition predominantly made up of iron (II) and iron (III) hydroxides, aluminium hydroxide, manganese hydroxide, magnesium hydroxide, calcium chloride, carbon and titanium oxide.

The tailings materials met the requirements for classification as Class 1 waste (the lowest waste classification) based on leaching tests, in accordance with the Western Australian Landfill Waste classification standards (Department of Water and Environmental Regulation 2019). The client elected to line the facility to the standard of a landfill containing Class 3 waste due to low levels of radioactivity of the tailings.

The tailings contains a low level of NORM, with an activity of approximately 39 Bq/g, mostly resulting from the decay of uranium and thorium. Once capped, the primary mechanism of exposure to radioactivity from the waste is via inhalation of radon gas, a radioactive decay product with a half-life of 3.8 days. A cap with a minimum soil thickness of 2 m is recommended to mitigate the impact of radon on potential receptors.

2.2 Geotechnical material properties

The tailings classifies as a silty CLAY and has the following average properties:

- >90% of fine particles (<75 µm)
- plasticity index of approximately 15–20
- plastic limit of approx. 80–90
- liquid limit of approx. 95–110
- hydraulic conductivity of <10^{-9} m/s (under expected confining pressure at the base of the TSF).

The material consolidates and drains slowly and has a very low shear strength in an unconsolidated saturated state. Strength increases with consolidation and solids content.

The average dry density is very low, with testing indicating an initial settled dry density of only 0.35 t/m^3. After air drying, the final settled dry density is estimated to increase to 0.6 t/m^3. Slurry consolidation tests carried out by others indicate that even at small vertical effective pressures of 10–100 kPa, significant, albeit slow, consolidation occurs.

2.3 Tailings strength investigation

2.3.1 Overview

At the beginning of the designer's involvement on site, the tailings on Cell 1 had been allowed to dry somewhat, with desiccation cracking on the tailings surface. The designer carried out a field investigation of Cell 1 using dynamic cone penetrometer (DCP) tests and vane shear tests (VSTs) to assess the tailings strength and bearing capacity. Cell 1 tailings was accessed using plywood boards in softer areas, while desiccated areas required no additional support. Cell 2 was still operational at the time of testing and the tailings could not be accessed or tested. The design for Cell 2 was to be carried out based on a minimum assumed tailings strength, with tailings strength to be confirmed prior to construction.

2.3.2 Tailings investigation methodology

2.3.2.1 Overview

The investigation of the tailings comprised DCP tests and VSTs, spread evenly across the tailings surface. Separate investigations were carried out prior to design and prior to construction.

2.3.2.2 Dynamic cone penetrometer

The DCP was developed in the 1950s as a simple method assessing penetration resistance of soil to assist in the design of flexible pavements (Scala 1956). It was developed further over time and in Australia is standardised in AS 1289.6.3.2 (Standards Australia 1997) and in the United States in ASTM D6951 (ASTM 2023). A DCP consists of a hardened steel cone attached to a metal shaft, which is driven into the ground by dropping a standardised weight along the rod onto an anvil attached to the rod as per AS 1289.6.3.2 (Standards Australia 1997). The number of blows required to achieve 100 mm penetration is recorded and used to estimate soil penetration resistance.

Many correlations between the penetration per DCP blow and shear strength, bearing capacity, unconfined compressive strength and California bearing ratio have been developed for cohesive and coarse-grained soils. The DCP is widely used and considered as a reliable, quick and simple method to estimate the strength and bearing capacity of a soil.

Some limitations of the DCP include the following:

- The DCP measures penetration resistance to a dynamic load, which is then correlated to shear strength, bearing capacity and California bearing ratio, relying on empirical relationships derived from a history of DCP testing of soil for pavement and foundation design.
- In the original method description, Scala (1956) states that DCP tests should be supported by regular California bearing ratio tests to confirm the results obtained by the DCP.
- Other field studies indicate that DCP values cannot be universally correlated to published relationships, formulae or charts (Sanglerat 1972; Campbell 2018), in particular in cohesive soil and below the watertable.
- Sowers & Hedges (1966) comment that the dynamic resistance of fine-grained, saturated, cohesionless soil is likely less than the static resistance. Consequently, the dynamic force exerted by the DCP may result in underestimation of the soil's shear strength.

- Lunne et al. (1997) consider that DCP tests have a high applicability in sand and moderate applicability in clay soil.

2.3.2.3 Vane shear test

The VST is a widely used test method to assess the shear strength of cohesive soil. It is particularly suited for soft to firm cohesive soil and often for soil that is considered weak and compressible, such as tailings (Ameratunga et al. 2016). Lunne et al. (1997) consider that VSTs have a high applicability for clay-type soils and that it is unsuitable for coarse-grained soil.

A vane shear comprises 2 thin metal veins perpendicular to each other attached to a metal rod. The veins and rod are inserted into the soil and rotated by hand at a standardised rate. Readings for peak and residual strength of the soil are read from a dial and converted, depending on the vane size selected.

The plasticity index of the soil has a major impact on the measurements collected with the VST. To allow for this effect, the correction factors as shown in Figure 1 have been proposed (Ameratunga et al. 2016).

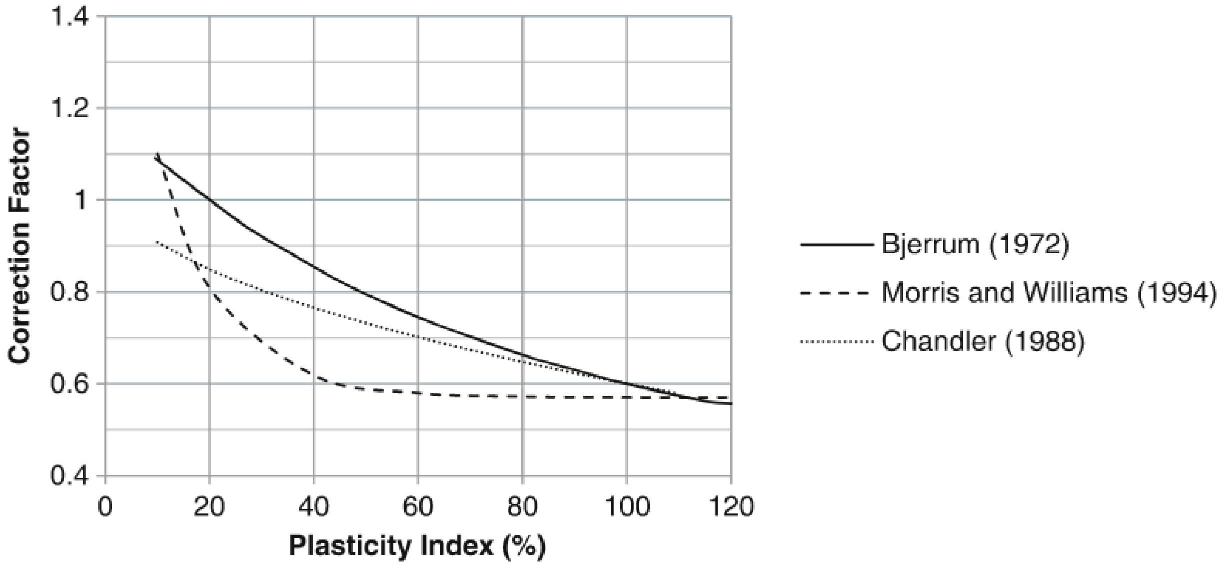

Figure 1 Field vane correction factor versus plasticity index (Ameratunga et al. 2016)

Morris & Williams (1994) proposed correction factors depending on the liquid limit of a soil, using the equation:

$$\mu = 7.01 \, e^{-0.08(LL)} + 0.57 \, (\text{for} LL > 20) \quad (1)$$

Using the relations proposed by Morris & Williams (1994), correction factors of 0.57 (for liquid limit) or 0.80 (for plasticity index) were calculated based on the geotechnical properties of the tailings discussed in Section 2.2.

2.3.3 Results and interpretation

2.3.3.1 DCP and SVT results

The DCP tests were undertaken in 15 locations in May 2024. At all tested locations, the DCP rod sank under self-weight (one blow per metre or less) to the depth of the test (Figure 2). The results indicated a very soft material with a very low shear strength. Zero to one DCP blow per 100 mm corresponds to an undrained shear strength of 0–12 kPa. In the case of the DCP dropping under self-weight by 1 m, this would imply an undrained shear strength approaching 0.

(a) (b)

Figure 2 DCP sinking under self-weight

ATCW and the site owner carried out SVTs as follows:

- May 2024: 9 SVTs (ATCW)
- February 2025: 18 SVTs (ACTW)
- April 2025: 19 SVTs, part of construction testing
- May 2025: 6 SVTs, part of construction testing.

The investigations indicated strength increase since the 2022 CPT investigation, which is consistent with tailings consolidation and drying. The investigation indicated a well-formed crust with an average waste strength ranging from 25 to >60 kPa to a depth of 2 m and more. Using the correction factor for liquid limit estimated in Section 2.4.2, the shear strength of the tailings may reduce to 14 and >34 kPa.

2.3.3.2 Further test pit investigation

The DCP test results were inconsistent with the VST results and relying on the DCPs would result in an excessively conservative design or even a design that could not be implemented.

To further assess the discrepancy between the test results, the site owner excavated a test pit from a previously placed earthfill causeway on Cell 1. The test pit was located adjacent to a previous VST location and was excavated to a depth of 2.0 m below ground level. The test pit remained open without wall collapse as shown in Figure 3. This indicated a soil shear strength in line with the VST results, not the DCP testing.

Figure 3 Test pit excavated in Cell 1 tailings

2.3.3.3 Interpretation and adopted parameters

The DCP and VST results varied greatly. The DCP results indicated very soft ground conditions, requiring reinforcement with geosynthetics and the use of low ground pressure equipment, significantly increasing construction costs.

The DCP test results were contradicted by VST results and a test pit excavated in the tailings, both of which indicated soft to firm ground conditions. Conversely, soft to firm ground condition would allow placement of soil without reinforcement and would allow use of larger equipment with higher ground pressure.

While DCPs are widely used to assess tailings strength, in this case the nature of the tailings resulted in non-representative results, despite existing correlations. Due to the limitations of the DCP (refer Section 2.3.2.2), especially in soft clays with unique properties, the DCP results were not considered to be representative of ground conditions. Potential localised liquefaction of tailings under the dynamic load of the DCP is a possibility. A CPT investigation of the Cell 1 tailings after completion of the first stage of capping indicated that the tailings may be prone to cyclic liquefaction. Localised compaction of the compressible tailings under the DCP point load is also a possible explanation of the unusual DCP results.

The VST results were used for estimation of the tailings strength and the DCP test results were disregarded. Using the correction factor for liquid limit estimated in Section 2.4.2, the adopted shear strength of the tailings ranged from 25 kPa for the crust and 10 kPa for the underlying tailings for the purpose of a bearing capacity assessment.

Where soil or tailings with different properties are tested, especially soft clay, the DCP results should be critically evaluated and may not be representative as demonstrated in this case study. As with any engineering, the correlations for test equipment and testing methods need to be understood and questioned where required.

The results from the tailings strength testing informed the capping system design as discussed in the following sections.

3 Capping system design

3.1 Overview

The tailings strength investigation and assessments directly inform the capping design and whether a cap construction is feasible, given the tailings strength. The regulatory requirements, the selected capping system and the design assessments are discussed in the following sections.

3.2 Regulatory requirements

3.2.1 Waste and landfill guidance

Cells 1 and 2 are part of a municipal waste management facility and while the cells are technically TSFs, from a regulatory standpoint, landfill guidance applies. In Western Australia, the landfill guidance adopted is EPA Victoria's 'Siting, design, operation and rehabilitation of landfills' (EPA Victoria 2015).

The guidelines' design objectives for the final landfill surface or capping are:

- Minimise infiltration of water into the waste, ensuring that the infiltration rate does not exceed the seepage rate through base of the landfill.
- Ensure the capping system has no more than 75% of the seepage through a base liner that meets best practice requirements.
- Provide a long-term stable barrier between waste and the environment to protect human health and the environment.
- Prevent the uncontrolled escape of landfill gas.
- Provide land suitable for its intended post-mining use.

To achieve the maximum allowable seepage rate, a geomembrane liner is required.

The site is subject to an environmental licence, which makes several requirements with regards to monitoring and minimum operational and flood storage freeboard.

3.2.2 Mining

The site is subject to a regulator-approved closure and rehabilitation plan. Therefore, mine closure guidelines issued by the Western Australian Department of Mines, Petroleum and Exploration are not considered mandatory but were consulted for standard industry practice.

3.2.3 Naturally occurring radioactive material

As part of the licence, the facility must comply with 'Managing naturally occurring radioactive material – 4.2 (NORM – 4.2) in mining and mineral processing – Guideline', issued by the Government of Western Australian Department of Mines and Petroleum Resources Safety, 2010, which for mine waste or tailings suggests typically, a 0.5 m layer of roller-compacted clay soil mixture will be effective in attenuating radon/thoron release to the atmosphere.

A radiation management plan for Cell 1 and Cell 2 requires the tailings to be covered by a minimum of 2 m of soil to control emission of radioactivity. This plan was not revised or challenged by the designer as a minimum cap thickness of 2 m was achieved due to other design requirements.

3.2.4 Summary

Regulatory requirements for capping and closure of the 2 TSF cells are different to a standard TSF closure. The design is to meet maximum infiltration requirements, and a minimum cap thickness of 2 m is required to control the emission of radioactive decay products. Standard industry practice for capping of TSFs was considered as the sites are TSFs.

3.3 Selected capping system

To meet the regulatory and design requirements, the capping system described in Table 1 and illustrated in Figure 4 was selected. Design assessments are described in Section 4.3.

Table 1 Selected capping system and rationale

Cap component	Rationale
1.3 m revegetation layer	Meets approved closure plan requirements and allow sufficient soil depth for vegetation growth, given the local climate.
Geocomposite drainage net	Allows subsurface drainage of rainwater.
1.5 mm LLDPE geomembrane	LLDPE geomembrane can tolerate strain from the expected settlement.
0.3 m select fill (clay rich) subgrade	Provides firm, relatively low-permeability subgrade to achieve a good, low-permeability interface between geomembrane and subgrade.
Underliner drainage system	A series of gravel-filled trenches with pipework is installed underneath the liner to allow removal of interstitial water rising upwards from tailings consolidation. Water will be extracted via gravity sumps along the western and southern embankments.
Bulk fill, including 1 m thick first fill layer	Bulk fill thickness varies to achieve desired post-settlement landform and achieve minimum separation of 2 m between tailings and the surface. First layer of fill to be 1 m thick to achieve safe working platform for equipment.
High-strength geotextile	Two layers of uniaxial high-strength geotextile placed perpendicular to each other and anchored on the edges. High-strength geotextile is only required in select areas of Cell 1 and all of Cell 2, subject to tailings strength.

LLDPE - linear low-density polyethylene

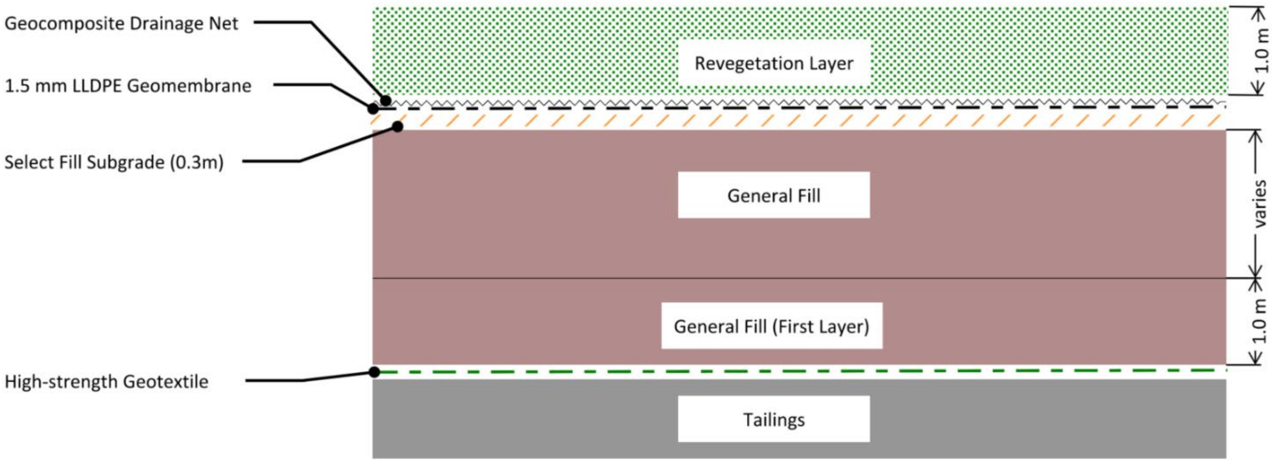

Figure 4 Proposed capping system

The following alternative capping system options were considered:

- Phytocap/evapotranspiration cap:
  - A phytocap trial had previously been carried out at the site and was considered unsuccessful. Therefore, this capping option was not considered further.
- Compacted clay liner with cover soil:
  - There is a lack of suitable clay on site. In addition, a clay cap may not reliably achieve the infiltration target due to cracking as a result of the significant settlement and potential desiccation due to the site's Mediterranean climate.
- Soil-only cover:
  - A soil-only cover or a cover including low-permeability and capillary break layers is unlikely to demonstrate the infiltration requirement unless it can be demonstrated with modelling.
  - The option is seen as higher risk as the landfill regulator may be unfamiliar with this type of capping system.

3.4 Design assessments

3.4.1 Settlement

The tailings is expected to settle significantly over time, with a maximum settlement of up to 3 m expected because of consolidation under self-weight and the weight of the cap. The post-settlement surface of the cells is required to have a continuous gradient, allowing shedding of surface water and subsurface drainage. The author estimated tailings settlement using site specific tailings consolidation testing data collected by others and ATCW's database of tailings consolidation data.

A 2-dimensional settlement model was developed to estimate the settlement and develop a landform with gradients sufficient to counter settlement. A liner strain assessment was carried out using the PLAXIS software, with a maximum liner strain of 6% estimated. This is less than the maximum allowable liner strain of 10% for a linear low-density polyethylene (LLDPE) geomembrane and an LLDPE geomembrane was therefore selected.

3.4.2 Bearing capacity

The bearing capacity assessment for Cell 1 indicated that most of the tailings has a bearing capacity sufficient to support a CAT D6T LGP dozer placing a first layer of fill. High-strength geotextile was used as reinforcement in a portion of Cell 1 with potentially soft tailings.

The bearing capacity for Cell 2 indicates that a crust with a bearing capacity of 25 kPa and an underlying soft tailings strength of 6 kPa is required before capping can commence with the placement of a 1 m-thick layer of fill. A high-strength geotextile reinforcement used 2 layers of uniaxial geotextile placed perpendicular to each other. Tailings strength is to be confirmed before any construction works occurs.

3.4.3 Infiltration modelling

Infiltration modelling comprised using the approach by Rowe (2012), which estimates leakage through a geomembrane liner based on the number of defects and wrinkles. The modelling indicated that the proposed lining system meets the regulatory requirements.

3.4.4 Slope stability

Slope stability models indicated that the TSF embankments – with the additional soil placed for capping – meet an acceptable Factor of Safety for slope stability.

4 Embankment cutback

4.1 Rationale

Cell 2 will not reach its final designed capacity. The lowest final tailings level approximately 2.5 m below the embankment crest level. To achieve a positively grading surface after settlement, this 2.5 m space would need to be filled with soil, requiring over 100,000 m^3 of earthworks. To reduce earthworks, reducing the embankment height of the western embankment by cutting the liner and then the embankment itself, is proposed. Figures 5 and 6 schematically depict the reduction of earthworks achieved by cutting back the embankment.

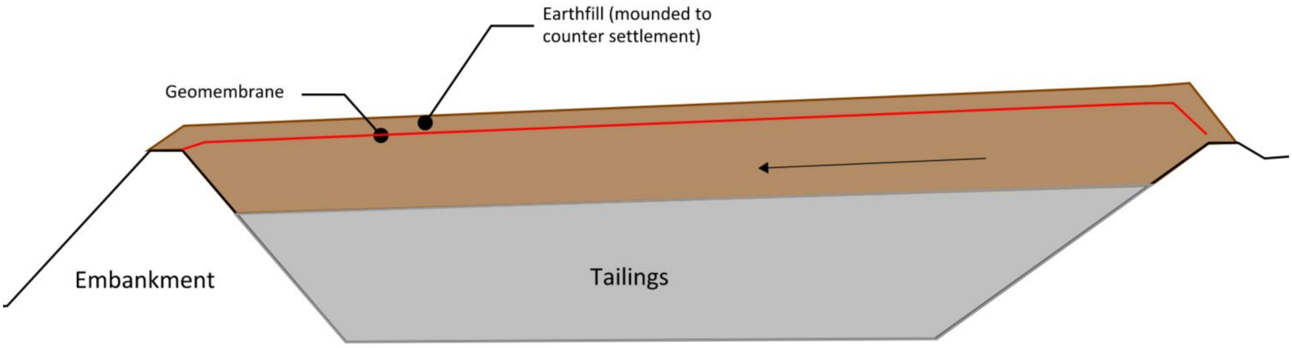

Figure 5 Schematic TSF section, no cutback

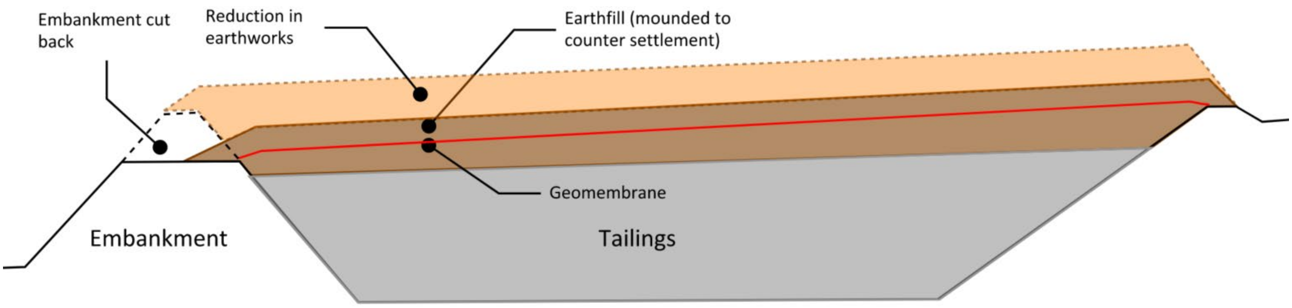

Figure 6 Schematic TSF section, with cutback

4.2 Freeboard

Cell 2 is required to maintain freeboard of a minimum 1.7 m along the western embankment while tailings is exposed. The freeboard comprises both environmental freeboard (run-off from a 90th percentile wet season less average wet season) and dam safety freeboard as per ANCOLD guidance for a Low Consequence Category TSF (24 hours, 5% annual exceedance probability, AEP) plus additional freeboard. To allow embankment cutback and capping of the facility as designed, the freeboard can no longer apply.

To negate the need to maintain environmental freeboard, the design includes placement of the high-strength geotextile and the first 1 m layer of fill before cutting back the embankment. This means the tailings is no longer exposed and rainfall falling on the facility is no longer considered to be leachate. Dam safety freeboard is no longer required once the embankments have been cut back to the extent that a design rainfall event could be safely released without the risk of an embankment failure. Once the 'dam' ceases to be a 'dam', freeboard no longer applies. Embankment cutback is to occur during the generally dry summer months, reducing water management requirements.

4.3 Cutback design

Figures 7 and 8 show the proposed embankment cutback after placement of the initial 1 m of earthfill on the tailings. The existing double liner will need to be cut, rolled back and re-anchored once the embankment has been cut back. This is a labour-intensive process, but cost estimates indicated that significant cost savings can be achieved by the reduction in earthworks.

An additional benefit of the embankment cutback is that the Factor of Safety for slope stability is increased.

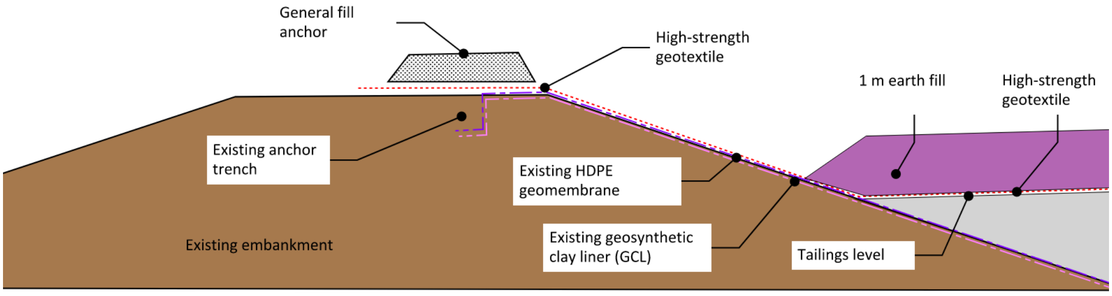

Figure 7 Embankment before cutback

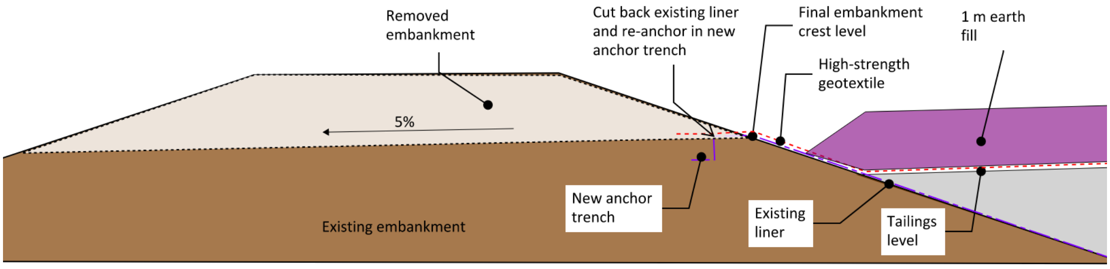

Figure 8 Embankment after cutback

5 Construction of Cell 1

Following design and regulatory approval, construction of Cell 1 commenced in early 2025. Construction was carried out in 2 stages, with bulk earthworks to the underside of cap occurring in summer 2025 and lining works commencing in late 2025.

Placement of cover soil confirmed the tailings strength estimated using the VSTs. Figure 9 shows placement of the first layer of soil on the tailings. There was minimal to no deflection of the tailings and fill during placement. Construction controls included VSTs before construction and regular survey of the first fill layer placed to assess if an unusual amount of settlement occurs.

(a) (b)

Figure 9 Photos showing placement of cover soil on Cell 1

At the time of preparation of this paper, the Cell 2 design was undergoing review by the client while the tailings was undergoing drying and consolidation. Cell 2 tailings settlement and drying is in the process of being monitored using aerial surveys and site photographs.

6 Conclusion

6.1 General

This paper presents a case study for the development a closure designs for 2 TSFs containing tailings with unusual geotechnical properties. One cell was successfully closed while the client is currently considering the second cell design. The paper demonstrates how successful closure of an unusual site can be achieved using engineering judgement and careful interpretation of investigation data.

6.2 Tailings strength testing

Strength test results with DCPs and VSTs resulted in conflicting results, with VST results selected as the suitable method for this case. Even though long-established correlations for DCP blow count with soil shear strength exist, it is an important reminder that DCP blows are not direct measurements, but correlations for commonly encountered soil and tailings. Where soil or tailings with non-standard properties are tested, especially soft clay, the DCP results should be critically evaluated and may not be representative at all – as demonstrated in this case study. As a general lesson, all geotechnical engineering test results need to be interpreted based on site conditions and the materials tested.

The designer focused on constructability in the selection of the design and capping system and suited to the tailings encountered. Delaying construction has the potential to significantly increase leachate management cost. Tailings capping must therefore occur as soon as feasible. The design developed achieves this.

The tailings strength testing and the discarding of DCP data in favour of SVT and test pit data reduced the need for a high-strength geotextile for Cell 1, significantly reducing cost. It also provided an important learning for interpreting future investigations of Cell 2 tailings.

Construction of Cell 1 capping confirmed that tailings strength was as per the SVT results.

6.3 Unusual regulatory requirements and cap design

The 2 TSFs operate under a landfill regulatory environment, which is unusual for TSFs. The designer developed a design suited to both the landfill regulator expectations and standard industry practice for TSF closure design. In this case, the cap design includes an LLDPE geomembrane and a geocomposite drainage net, which is an unusual arrangement for TSFs.

In general, regulatory requirements for any site need to be clearly understood and can result in different rehabilitation outcomes. In the case of the 2 TSF cells discussed here, a different rehabilitation method may have been selected for a site regulated solely by mining guidelines.

6.4 Tailings consolidation

Both cells contain tailings that is subject to significant settlement of up to 3 m. A 2-dimensional settlement model was developed using available consolidation test data and a final landform intended to counter the settlement was developed. Liner strain due to settlement was acceptable for the LLDPE geomembrane selected.

6.5 Freeboard and embankment cutback

An embankment cutback is proposed for Cell 2 to reduce the earthworks volume required for capping. This required encroaching the TSF’s freeboard of 1.7 m. The design adopted included an approach by which first a 1 m layer of fill was placed on the tailings to remove the need for environmental freeboard.

The embankment is then cut to allow safe discharge of a dam safety–related design storm. Once a 'dam ceases to be a dam', freeboard requirements can be reinterpreted and in this case, maintaining a dam safety freeboard is no longer required.

The embankment cutback requires cutting and re-anchoring of the liner, which is labour intensive, but in this case will likely result in potential cost savings, while also improving long-term slope stability and reducing erosion risk of the final landform. Reducing embankment height for a turkey's nest–style TSF with remaining capacity can achieve significant savings in earthworks and cost.

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Use of artificial intelligence disclosure statement

In the preparation of this work, the author used Microsoft CoPilot (Basic) to assess the document for general flow and completeness of sections with respect to the abstract. No content was generated by artificial intelligence (AI). The author has reviewed the content and edited it and take full responsibility for the paper.

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