Puketutu Island Rehabilitation Project – Engineering of a Biosolids Monofill Facility

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ABSTRACT

Watercare Services Limited (WSL) operate a very large municipal wastewater treatment plant at Mangere, Auckland. A by-product of the treatment process is the creation of a stabilised biosolid. Through a review of operations an opportunity was identified to rehabilitate a retired quarry on the adjacent Puketutu Island site, both improving the Islands environment and reducing the cost and transport burden to take this material to commercial landfill. The new facility, referred to as the Puketutu Island Rehabilitation Project, is a modern fully lined monofill facility located in a central volcanic cone that has been historically quarried out and partially backfilled.

CH2M Beca Ltd. were engaged to prepare the detailed design and construction management of the first two phases of the facility and this paper presents some of the challenges faced in preparing those designs and the subsequent construction.

The nature of this past site use created some design challenges for construction within the consented footprint. This included backfilling of large ponds, rock blasting, a large landslip, and known and unknown wet bin locations. The facility was designed to a similar standard as a landfill, with full liner, leachate collection and biosiolds affected stormwater capture.

A 3D geological model was created to better understand the complex underlying geology. Some interesting aspects include the design of the embankments using the variable materials on site and underlain by existing uncontrolled fill and wet bins to achieve stability under a 1/2500 AEP earthquake, confirming liner integrity, staging of the project while maintaining a fully operational facility, and construction of internal access roads to allow the biosolids to be placed in cells by the tractor/trailer units. Additionally, it was necessary for the construction to be able to take place year round to meet the project timeframes.

1 INTRODUCTION

Puketutu Island is located in the Manukau Harbour in Auckland, New Zealand. It is approximately 195 hectares in total and has a central volcanic cone with associated ash deposits (tuff) and lava flows (basalt) around the island perimeter. Within the footprint of the Puketutu Island Rehabilitation Project Monofill Facility, extensive quarrying and clean fill operations have occurred over the past five-decades.

The project involves rehabilitating part of the quarried areas of the island with biosolids to create a landform that is consistent with the remaining geology of the island. The landform will comprise an outer embankment within which the biosolids will be placed in order to create an elevated central landform. The Island will eventually become a regional park, with parts of the island being opened to the public as soon as practicable.

The main objectives of this rehabilitation project, which form the basis from which the design philosophy and rehabilitation project have been developed, included providing a sustainable and economic use of biosolids from the population served by WSL, incorporating design features that manage any potential impacts on the environment and be considerate of any cultural or social implications.

The first two phases (referred to as Phase 1, but split into two sub phases of 1A, 1B and 1C – see Figure 1) of design, discussed in this paper, have been peer reviewed and approved by Auckland Council to ensure that the key elements of the consented concept design have been maintained and equivalence proven when changes to the concept have been implemented.

The resource consent process was completed prior to Beca's involvement; therefore any changes to the consented design needed to go through a thorough peer review process to prove equivalence and gain acceptance.

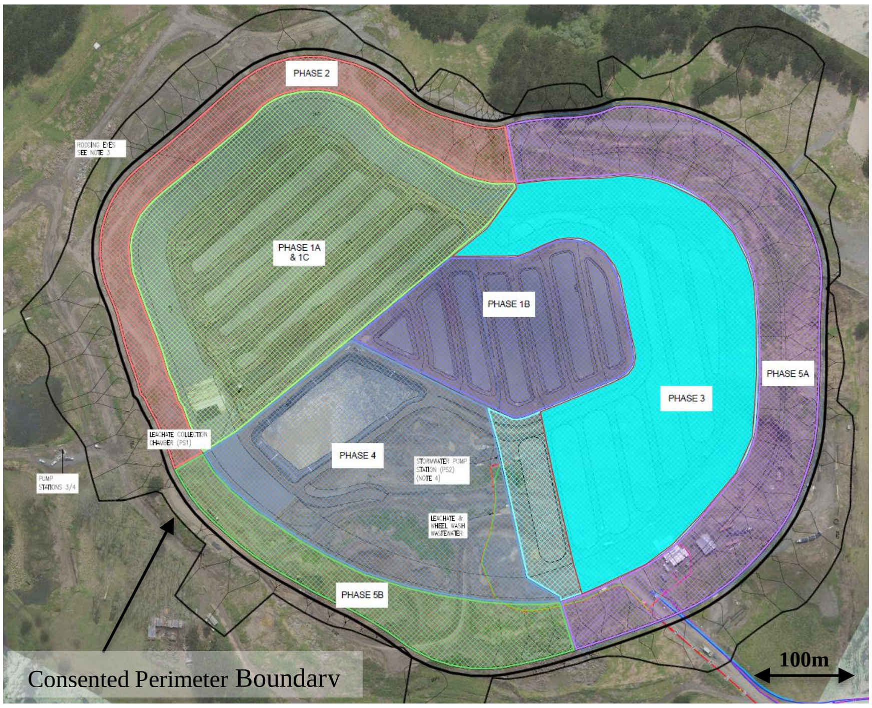

Figure 1: Phase 1 Layout

2 DESIGN DETAILS

The core geotechnical analyses completed for the design of the facility included slope stability analyses (for short term, long term, seismic and during construction), settlement analyses, liner design and groundwater modelling.

Figure 2 shows a typical leapfrog geological cross section; running approximately E-W across the site.

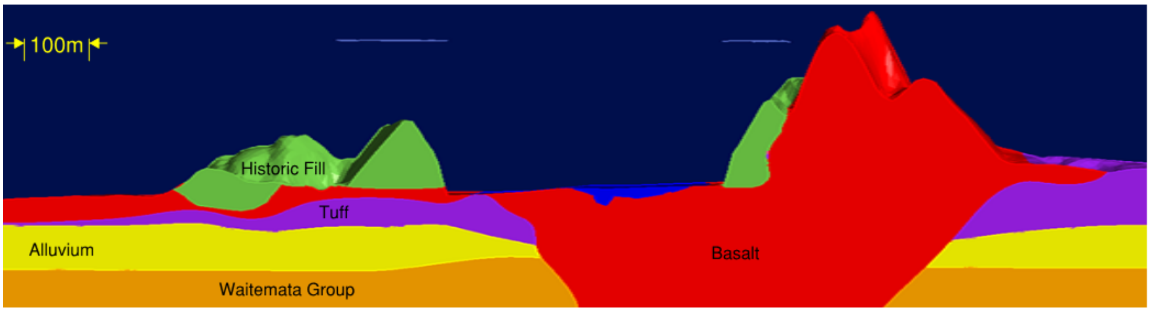

Figure 2: Geological Section E-W across the site (5x vertical exaggeration)

2.1 Site Layout

The overall facility has been designed with external perimeter embankment slopes no steeper than 4H:1V and internal perimeter embankment slopes of 3H:1V with a mid-height bench.

Internal haul roads have been designed with a top width of 6.5m, 1H:1V side slopes and at a spacing of typically 30m to suit the maximum reach of the biosolids placement operating equipment. The maximum lift height (between cells) is 2m; which are constructed in a “Christmas tree” pattern with geogrids placed at the base of each lift.

Perimeter haul roads have been designed so that a 12.5m long truck can manoeuvre safely around curves in either direction. These roads have internal 1:1 side slopes, 1.5:1 external side slopes, and both reinforced with geogrids. It is important to note that these are temporary embankments located only on the floor of the facility which are designed for short term conditions and should not be confused with the permanent flatter perimeter embankment slopes of the overall consented facility.

A typical layout of the biosolids placement on the facility floor is shown in Figure 3.

The staging of these roads (and other site infrastructure) enabled the full facility to be constructed in stages but still be fully functional without disruption to operations in the future.

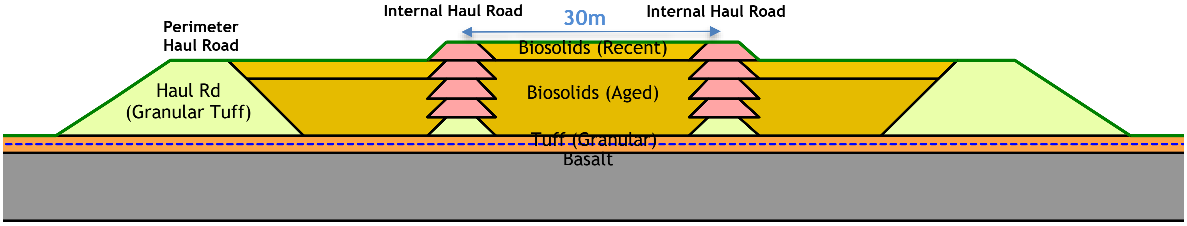

Figure 3 –Biosolids Placement on Facility Floor

2.2 Materials

Numerous investigations (approx. 170 boreholes) have been undertaken at the site since 1997 with targeted investigations for this design completed since 2007. A 3D geological model (using Leapfrog) was created to better understand the complex underlying geology (including historic fill) and this enabled us to investigate specific areas of concern during detailed design (for example, to compare and cut critical cross sections for the settlement/stability analyses).

The key materials used in the detailed design analyses, as identified from the investigations, are biosolids, granular tuff/basalt (used for haul road construction and floor foundation materials), historic fill, imported fill (clay liner) and other soils that are present occasionally throughout the site with varying thicknesses (Scoria, Alluvium and Waitemata group soils).

A review of the historical geotechnical investigations and laboratory testing (Kayser 2012) were used to assess material parameters for these soils and the biosolids.

2.3 Seismic Design

The facility has been designed for both SLS and ULS events, with differing performance criteria for the internal haul roads and perimeter embankments.

The peak ground accelerations (PGAs) for the project have been derived from NZS1170.5:2004. The project site was classed as soil class C.

The consequences of failure of the structure are considered to be high, with 'very great' environmental consequences (Table 3.1 AS/NZ1170.0:2002). The biosolids facility is not considered to be a post-disaster facility (Table 3.2) and as such, has been assigned an Importance Level of 3. The required design working life of the haul road bunds was 25 years; which based on Table 3.3 in AS/NZ1170.0:2002, corresponds to a ULS event annual probability of exceedance (APE) of 1 in 500 years and an SLS APE of 1 in 25 years. For the final facility perimeter embankments, a required design working life was 100 years, which corresponds to a ULS APE of 1 in 2500 years and an SLS APE of 1 in 25 years. A 20mm maximum displacement criteria was adopted for the seismic design of the perimeter slopes, with no failure anticipated to breach the liner.

We have also considered the appropriate earthquake loads for landfills as this monofill has some design similarities to those facilities. Guidance from the EPA (1995) requires the seismic design of landfills be undertaken for an earthquake having a 90% probability of non-exceedance in 250 years (approximate return interval of 2500 years). This indicates that the ULS design earthquake derived from AS/NZ1170.0:2002 for this facility is appropriate.

3 DESIGN CHALLENGES

A number of key elements provided design challenges; these included backfilling deep ponds, rock blasting, uncontrolled fill, historic wet bins, perimeter embankments, a landslip, and an alternative lining system.

Some of the challenges were due to a differing detailed design elements versus the consented concept design (i.e. requirement to prove as equivalent or better), while others were due to the uncertainties around past site use (i.e. uncontrolled fill and wet bins).

3.1 Backfilling Deep Ponds

Two large ponds (up to 7m deep with one below sea level) had been formed on the floor footprint during quarrying activities and needed to be backfilled during Phase 1.

The construction challenges included dewatering the ponds which sit within highly permeable basalt and tuff zones at pumping rates below the consented limits and then backfilling, both safely and practically through water. Not all water could be pumped in isolated areas (typically a few meters of water remained) during backfilling due to the very high recharge rates. Only one of the two ponds was able to be fully pumped and backfilled in dry conditions.

The design challenge with this backfilling method was to ensure that base settlement was controlled so that no damage to the liner system (in particular liner strain) would occur. In addition, placement of rock, soil and geotextiles needed to be controlled to ensure that filter compatibility (i.e. a graded fill approach adopted based on Fell et al 2005) was achieved from soil/rock layering and that groundwater could flow between the layers to avoid pore pressure build up on the base of the liner.

To minimize the risk of backfilling in the deeper partially saturated pond, Beca developed a methodology with the contractor to ensure the backfilling could be completed safely and to ensure it would meet the required design outcome. A graded fill approach was adopted, which also included the use of sacrificial risers to continue pumping during backfill and incorporation of geotextiles to bridge over the pond backfill rock blinding layer before placement of the hard fill to subgrade level. Additionally, the ponds were backfilled well in advance of the liner being placed, which allowed monitoring of the base above the pond infills for defects prior to the placement of the liner system.

3.2 Rock Blasting

Historic fill was removed from the floor below design subgrade level and replaced with compacted hard fill (tuff) prior to constructing the liner system. However, at some locations basalt was encountered above the design subgrade level. The basalt was too strong to be excavated by ripping, therefore rock blasting was required in isolated areas.

In order to avoid sharp contacts below the floor lining system, which could cause large differential settlements (and therefore high liner strain, see section 3.5), 4H:1V slopes were formed if within 2m of the subgrade design level (reduced to 3H:1V if below 2m). “Dental concrete” was also used as required to provide a smoother finish. Plaxis analyses was completed to assess the settlement and liner strains on the site floor and establish the construction criteria.

3.3 Uncontrolled Fill

Historically the site operated as a basalt rock quarry and then as a managed fill site. However, historic fill was placed in an uncontrolled manner and investigations indicate that the quality and strength is variable throughout the site.

The uncontrolled fill was historically placed to form the majority of the embankments around the site, therefore the design allowed for this fill to remain in place, which is discussed further in the embankment design section below.

This uncontrolled fill, which needed to be cut to form the designed facility layout, was able to be reused where drying was possible to achieve a reasonable quality of material as discussed in section 3.6.

3.4 Wet Bins

There are wet bins (i.e. soft compressible soil with Su<25kPa) located within the historic fill, formed by end tipping of saturated/uncompacted fill by the quarry operator during winter placement. These zones have been identified based on past site experience and from review of historical air photos. However, it is acknowledged that there is a risk that wet bins exist at additional locations across the site, not identified.

In order to minimize this risk of wet bins for the embankment design, we modelled large “soft” zones in our settlement analyses (with $C_c/(1+e_0)$=12% using Plaxis). These simulated a weak zone located within the final perimeter embankment and quantify the impact on the lining system, in terms of total settlement, differential settlement and more importantly liner strain. This option also provided a safer design for construction because the contractor was not required to complete temporary works for the removal of the soft soil while working on the existing quarry slopes, which were steeper than the designed slopes.

Ongoing review of boreholes and test pits indicate that large soft wet zones may not exist today or may not be as bad as modelled, even at locations where previous site activities indicated that they may exist. There were isolated soft, wet layers identified within the borehole logs, but not as thick as modelled in our analysis.

3.5 Embankment Design

The perimeter embankment has been designed to tie into the overall consented surface, plus leave the historic uncontrolled fill and wet bins in place, while providing a practical and stable platform for the construction of the liner and the placement of the biosolids.

It has been designed to be constructed in two stages with an internal mid-height bench between stages to allow the liner to be joined on a horizontal surface. However, the first phase also analysed the ultimate profiles, which allowed for the completion of the embankments in future construction phases, and for some flexibility in the construction process.

It was necessary for the construction of the facility to be able to take place year round to meet the project timeframes. Therefore, the design allowed for the uncontrolled fill and wet bins to remain in place, resulting in less earthworks and less area exposed. Slightly conservative parameters (i.e. Su=25kPa, c'=3kPa and Φ'=25° with sensitivity checks completed as part of the probabilistic analyses discussed below) were adopted to enable this; however, it avoided the need for double handling of the fill and the cost, plus adverse environmental effects of importing/exporting large quantities of fill to/from the Island.

A key feature in the design, which enabled the historic fill to remain in place, was the construction of a 10m wide material replacement zone (MRZ) on the internal face of the perimeter embankment. The MRZ provided confidence in the strength and behaviour of this upper/outer layer by re-using the historic fill, but controlling the quality and compaction effort required during constriction. The MRZ has been designed to perform several functions, including:

- Improve stability of the slope, particularly against shallow failures,
- Provide a bridging or rafting layer over areas of softer/weaker soils (i.e. wet bins), particularly in relation to differential settlements and liner strain, and
- Provide a layer of known cohesive soil to act as a diffusive layer for the liner (liner system discussed later).

Tensile strain on the liner was estimated as < 0.7% for all cases on side slopes with "soft zones" and on the floor where sharp contacts with basalt was encountered. The estimated strains are much lower than the typical strains of 5% for a GCL and 2% for a CCL (LaGatta et el, 1997).

Probabilistic analysis was undertaken (in Slope/W) to assess the sensitivity of embankment stability (for static, seismic and r_u scenarios) to changes in soil strength parameters, which was very important given the variability of the uncontrolled historic fill placed around the site. They show that the design is robust for static and seismic scenarios, but the stability is sensitive to r_u; therefore, vibrating wire piezometers were installed and monitored (with trigger levels identified) during construction.

3.6 Landslip

A landslip occurred within the historic fill placed to the east of the Phase 1 area, but within the future extent of the facility (currently being designed). The cause has been contributed to a previous site activity of end tipping clean fill behind a narrow bund (essentially creating a dam) at the site, similar to the wet bins discussed previously. It is understood that this material was likely placed wet, was exposed and ponded water added to the trigger of the bund breach and slope flow failure.

Investigations have been completed to determine the vertical extent of the failure. A methodology has been developed for the safe removal of the slip material for future expansion of the site. It has been confirmed that all slip material will need to be removed for future expansion in that area.

3.7 Lining System

The lining system for both the floor and side slopes were revised from the consented design. The alternative liners were peer reviewed and accepted as having 'equivalent chemical and hydraulic containment'.

The alternative lining systems reduced the amount of controlled clay fill required to be imported to site and allowed for year round construction at the site. Note that the Phase 1 floor and first 30m (i.e. 10m vertical) up the side slopes was successfully constructed over a two week window in August over winter.

The consent application liner comprised a 900mm thick clay liner on the side slopes and a 600mm thick clay liner overlain by a 1.5mm flexible membrane liner (FML) on the floor. The clay liner specified a permeability of not more than $1x10^{-9}$ m/s)

There was a change to the consent conditions to permit alternatives with equivalent containment. The liner for the floor was changed to 300 mm thick cohesive subsoil layer overlain by a geosynthetic clay liner (GCL), and then an FML. The liner for the perimeter embankment internal side slopes consists of a 450 mm thick cohesive subsoil layer, overlain by a GCL and then an FML. The cohesive subsoil layer specified a permeability $<1 \times 10^{-8}$ m/s.

These alternative lining systems can be constructed in short periods over winter to meet the project requirements, while maintaining equivalence to the consented design.

4 FUTURE SITE DEVELOPMENT/EXPANSION

The facility has been designed and constructed for up to 5 years of biosolids placement. However, there is a requirement for up to 30 more years of placement at the site; which requires the facility to expand both laterally and vertically. Although the overall facility has been designed and generally accepted, future phases will require the same peer review and council acceptance on a phase by phase basis.

5 CONCLUSION

There were design and construction challenges encountered during the first phase to allow up to 5 years of biosolids placement. And although additional design packages are required for the subsequent design phases to expand the facility for the full 35 year storage space required, the bulk of the geotechnical driven design elements has been completed. This is because many of the key challenge solutions, which are common across the entire site, have been peer reviewed and agreed with the consenting authorities. The most significant challenges and the solutions included:

- *Perimeter Embankment Design*: The design considered the variable conditions around the full consented facility, not just the Phase 1 footprint. It allowed for historically placed uncontrolled fill and wet bins to remain in place, but still meet the consented design criteria. Leaving them in place also meant there was less earthworks involved and less area exposed, which was favourable for year round construction.
- *Internal Haul Roads:* The biggest challenge for design/construction of the internal haul roads was the backfilling of the deep ponds and rock blasting on the facility floor. It required design to ensure that the liner constructed above would not be damaged due to the construction of the floor subgrade and planning to ensure it could be completed safely by the contractor. Also, by designing the haul roads using a granular hard fill, construction could continue over the winter and the facility could remain operational year round.
- *Alternative Lining Systems:* It was necessary for the construction to be able to take place year round to meet the project timeframes. So the alternative lining system (equivalent to the consented) provided an opportunity that could be constructed in short periods over winter.

6 ACKNOWLEDGEMENTS

Too many people have been involved in Phase 1 of the project to name individually; however there were a few people that we would like to acknowledge for their contributions, including:

- David Anstiss for his involvement with the project since 2012, with his main inputs on the design of the lining system and assessment of the uncontrolled fill quality, as well as ongoing construction inspections/advice.
- WSL project and operations teams for on-going input and collaboration during the initial design and construction phases.
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