Mandena Mine Salinity Control Weir Design and Construction

F.J.G. Gibbons SSI Engineers and Environmental Consultants, Johannesburg, South Africa.
francisg@ssi.co.za
A. Robinson SSI Engineers and Environmental Consultants, Johannesburg, South Africa.
abhijatrir@ssi.co.za

ABSTRACT

As part of the infrastructure for their Mandena mine in southern Madagascar, Rio Tinto recently constructed a rockfill weir on the Anony River. The weir consists of a 150 m long, 9 m high rockfill embankment, with an adjacent 300 m wide rock-cut main spillway, a boat lock and low flow spillway.

Various systems for decreasing the permeability of the rockfill embankment were investigated, including HDPE geomembrane, geosynthetic clay liners and sheet piling. Constructability dictated the final solution which comprised a sand core constructed from local sand deposits enclosed in a heavy duty geotextile filter. The rate of saline water seepage through the weir with this configuration under adverse hydraulic conditions was investigated using finite element modelling and was found to be acceptable. The weir was constructed without coffer dams in 4m water depth and the contractor developed an innovative system for laying geotextile underwater from a purpose made barge.

1. INTRODUCTION AND PROJECT BRIEF

Rio Tinto appointed SSI to carry out the preliminary and detailed design and construction supervision for a salinity control weir that would secure the supply of fresh water for their Mandena mine development in near Tolagnaro (Fort Dauphin) in southern Madagascar. The supply to the mine is drawn from the coastal lake system adjacent to the mining area feeding the Anony River estuary and the weir structure was required to prevent saline water intrusion into the coastal lake system resulting from flow reversals in the estuary as a result of tidal influence and storm surges. The weir also needed to be designed in such a way that it would allow the passage of floods flows from the upstream catchment with a relatively small increase in water levels relative to the natural state, as the village of Andrakaraka is situated on the low lying ground adjacent to Meander River connecting the lakes.

Figure 1. The Anony River Estuary and Partially Completed Weir

2. SITE DESCRIPTION

The site is located approximately 15 km north of Tolagnaro and can be accessed either by boat, along the coastal lakes, or by road. The site chosen for the weir was at a point where the Anony River is 130 m wide, 4km upstream from the river mouth. A major feature in favour of the site was a rock outcrop about 400 m long perpendicular to the river on the left bank. The rock fell off steeply under the river such that the sand cover on the right bank was 15m thick. The right bank comprised a steep primary sand dune approximately 70 m high that is virtually at its natural angle of repose. The maximum water depth in the river channel was about 4.0m with a normal water level 0.3 m above sea level.

Figure 2. The weir site before construction commenced

Table 1. Anony River Flood Flows.

Return Period (years)	Estimated River Flow (m^3/s)
10	581
50	1 213
100	1 551
200	1 926
RMF	2 874
PMF	5 227
10	581

The combined 164 km^2 catchment for the Anony River at the weir site comprises approximately one third mountain forest and two thirds coastal plain underlain by estuarine sands. Design flood flows are summarised in Table 1 above.

3. BACKGROUND WORK

Although not covered in detail in this paper the design team undertook the following background work in order to establish the overall design parameters for the weir.

- Water resources analysis for the mine – this indicated that there was sufficient flow in the two rivers feeding the lake system to meet the town's needs as well as the mine's without providing additional storage. The water level in the lake system could therefore be left at close to the natural state.
- Flood hydrology study for flood level determination – as mentioned above the low lying areas adjacent to the lakes made it essential to limit the increase in flood levels after construction of the weir to a minimum.
- Storm surge levels were investigated to determine the maximum likely downstream estuarine water levels and, from this, the main spillway level. The lower the main spillway level, the greater the risk of overtopping by saline water from the downstream direction under adverse tide and storm surge levels, and conversely the greater the increase in flood levels relative to the natural state.
- A HECRAS unsteady flow model was used to determine water levels for the different spillway and flow options

A main spillway crest level of +1.1 m above mean sea level was eventually adopted which provided an acceptable risk of saline flow reversal and minimal increases in flood levels relative to the natural state. The height of the rockfill embankment was selected at + 5 m above mean sea level, which satisfied a zero freeboard condition under the Probable Maximum Flood.

Very extensive environmental impact assessments were carried out by QMM, the subsidiary of Rio Tinto that developed the mine. Specialist studies undertaken included the investigation into the change in fish species and populations, fish passage through the boat lock, sedimentation upstream of the weir, salinity and water quality changes, the impact on river based transport, including wooden canoes and tourist boats, as well as on tourism and local fishing. Communication of the weir development with local communities, particularly Andrakaraka about 5 km upstream and Evatra 4 km downstream was also extensive. On the basis of this work and SSI's technical study QMM made application to the Madagascar government agencies for the weir construction and gained approval for the works.

4. EMBANKMENT OPTIONS CONSIDERED

4.1 Introduction

A number of different rockfill embankment arrangement options were formulated and evaluated under all anticipated flow conditions. It was important to ensure that any option selected complied with the following criteria:

- Seepage through the embankment should be limited to acceptable levels, especially under reverse hydraulic gradients when there could be saline water intrusion.
- Erosion of the highly sensitive dune on the right bank during initial closure of the river or as a result of overtopping of the embankment had to be prevented.
- Piping occurring either in the right bank or under the embankment through the sand foundation under the various hydraulic gradients had to be prevented.
- The embankment should be stable under all expected flow conditions including flood flows and expected reverse flows resulting from high downstream levels caused by storm surge.
- The embankment needed to be constructible in the wet – i.e. without coffer dams. The reasoning behind this criterion was that the cost to construct the upstream and downstream coffer dams required to enable the embankment to be constructed in the dry would not be very much less than to construct the embankment itself in the wet.

The table below summarises the key options considered and their respective advantages and disadvantages.

Table 2 - Summary of Embankment Options Considered

Option	Advantages/Disadvantages
Overtopping rockfill structure	To ensure stability of the downstream flank of the rockfill structure under the higher flood flows the downstream slope of the embankment has to be very flat, which renders this option uneconomical. This option would also require careful protection of the right bank dune.
Rockfill embankment with HDPE geomembrane liner. Geomembrane would be protected from damage by sand layers on either side	The geomembrane liner would be difficult to join and place underwater. Damage to the HDPE liner during construction was a big risk. The smooth liner was likely to affect the stability of any rockfill placed over the liner to secure it in place. Possible reverse heads on the embankment required a significant amount of material to "hold down" an HDPE liner.
Sand core rockfill embankment with steel or concrete sheet pile providing the impermeable barrier	Expensive to install because materials largely imported and construction plant not available in Madagascar. Long term corrosion resistance of steel sheetpiles is an issue. An advantage is that the sheet piles can be driven down to rock level to seal the underlying sand layer

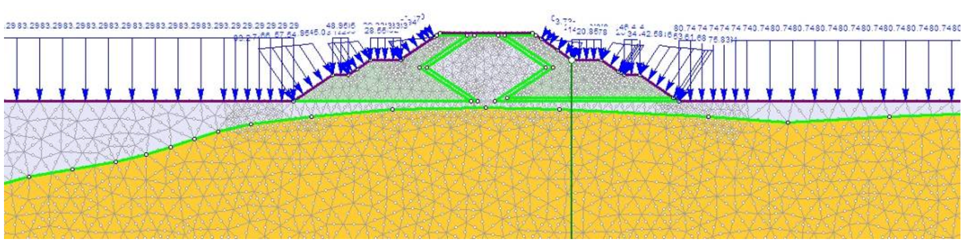
Option	Advantages/Disadvantages
Diaphragm wall sealed sand core rockfill embankment	Expensive to install because construction plant not available in Madagascar and requiring special construction. An advantage is that the diaphragm walling can be constructed down to rock level to seal the underlying sand layer
Sand embankment with 1:10 side slopes	Rock would still need to be excavated for spillway construction and would need to be spoiled elsewhere. The 1:10 slopes would result in a much larger footprint and the spillway end wall would need to be much longer to contain the sand.
Sand core rockfill embankment (chosen option)	Higher permeability through the core but still within acceptable limits. Readily available materials. This option was ultimately chosen because of simplicity and lowest cost.

An option comprising a geosynthetic clay liner was dropped at an early stage because of the adverse performance of bentonite in potentially saline conditions.

4.2 Review of Options

The above options were reviewed initially in general terms to see if they would work under the given flow conditions. A number were then modelled in detail using the Phase 2 finite element modelling package. Key aspects included in the modelling were stability of the structure under hydraulic and earthquake loads and seepage through the structure. The conditions modelled were the highest head differential based on the HECRAS model outputs under various flood flow conditions as well as under reverse flow conditions.

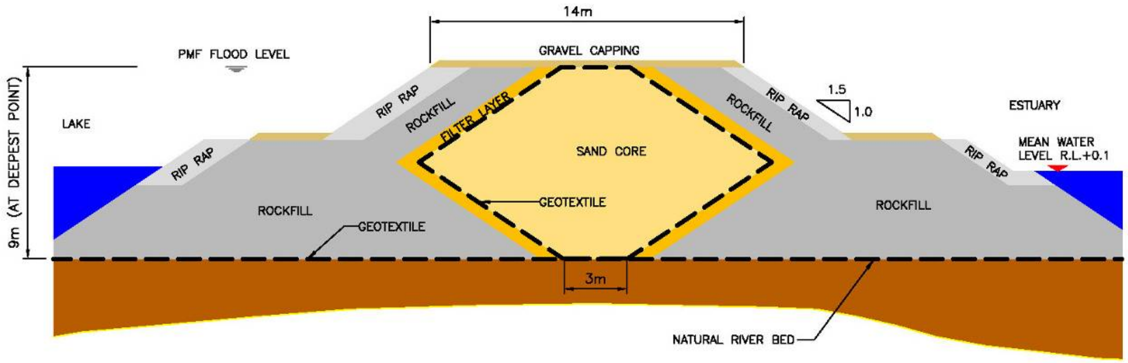
Figure 3. Layout of finite element model showing meshing and water levels during the RMF flood at peak water levels, downstream to the right



The geometry of the cross-section was refined during the stability analysis. The final selected design was based on the dimensions required to build the embankment rather than the smallest dimensions required for impermeability or stability reasons. Although at first sight the adopted cross-section would appear to be a very simple solution, the final design of the rockfill embankment was developed over numerous iterations, each subject to rigorous scrutiny to ensure stability, sufficient impermeability and constructability.

The concept provided for two pioneer embankments to be constructed out across the river initially to 1,5 m above mean seal level as described further below; these pioneer embankments formed the upstream and downstream containment for the sand core below water level, and they were topped with gravel to provide access to the sides of the embankment. On the right bank the two berms thus formed ramp up to provide access to the top of the embankment, as well as thickening the embankment to provide further protection against damage to the sensitive dune on the right bank.

Figure 4. Typical Cross Section of Rockfill Embankment – Selected Option



The Phase 2 software was also used to check the rate of seepage through the embankment. Seepage flows per metre length are given in Table 3 for low and high permeability scenarios.

Table 3. Seepage flows per metre length through the weir embankment for the high and low permeability scenarios

Upstream water level (m)	Downstream water level (m)	Seepage flow for low permeability scenario ($m^3/s/m$)	Seepage flow for high permeability scenario ($m^3/s/m$)
1.46	-0.21	1.70×10^{-6}	0.00272
4.49	4.23	0.07×10^{-6}	0.00023
0.30	1.50	-1.22×10^{-6}	-0.00199

The results show that the highest seepage flows occur during the maximum head difference .Thus for an embankment length of 150m, which is approximately the length of the weir embankment from the masonry wall to the dune, a seepage of 0.40 m^3/s can be expected. This of course is insignificant in comparison to the flow over the spillway for this condition.

4.3 Geotextile Layers

Although the cost of the geotextile foundation and containment layers were relatively minor in relation to the cost of the whole embankment, they are a critical component of the structure and the laying thereof in the wet required particular attention during construction. These geotextile layers would be susceptible to damage during construction and, in addition, the competency of the completed layers after placing of the rockfill could not be checked. Any significant damage could be the source of piping over time.

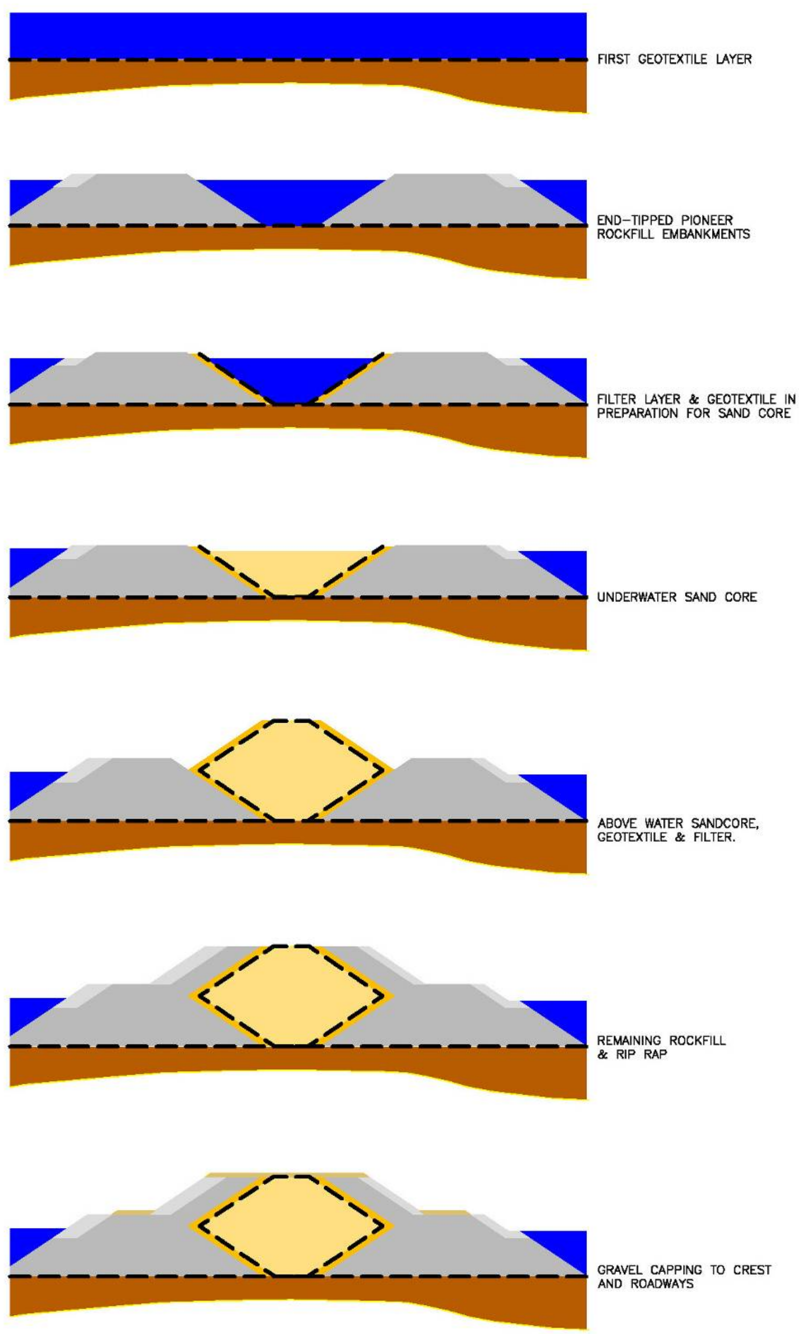
The foundation geotextile layer was laid under the entire rockfill embankment. On the downstream side this formed a barrier to prevent sand getting carried into the rockfill by the seepage flow passing under the embankment. It also generally acted as a reinforcing barrier between the rockfill and the soft sand layer in the river bed that varies in thickness from 1 to 15 m. A key decision with this geotextile layer was whether to protect this layer with a sand layer before placing rockfill on top of it. It was decided however after some consultation with the suppliers to specify a heavy grade geotextile (550g/m^2) and to omit the protective layer. This would also simplify and speed up the construction in the river consequently reducing the risk of flood damage to a partially complete embankment.

The containment geotextile layer contains the sand core at the centre of the embankment which forms the main barrier to seepage through the structure. This layer was easier to construct because it is within the structure however it is still a critical part of the embankment. Again a 550 g/m^2 grade geotextile was specified to reduce the likelihood of damage as much as possible.

5. CONSTRUCTION CHALLENGES

The construction contract was awarded to Colas Madagascar. Prior to construction starting in earnest a workshop of the weir design team and construction team was convened in Fort Dauphin to go over the detailed construction methodology. Each aspect of the construction was reviewed in detail with all the likely advantages and shortcomings. This meeting went a long way to pre-empting possible construction problems and resolving them early. The embankment construction stages are as shown on Figure 5.

Figure 5. Embankment Construction Stages



Some of the challenges encountered during construction and the solutions developed to overcome them are presented below.

5.1 Floods

Whilst the normal flow in the 120 m wide, 4 m deep river varies from 6 to10 m^3/s, flood flows can be significant, as shown in Table 1 above. The rockfill embankment construction was scheduled for the driest part of Madagascar's year from August to October when approximately 15% of the annual rainfall occurs. The critical stages were laying geotextile on the river bed and closing the rockfill embankment against the right bank sand dune.

Figure 6. The 1:10 year flood in February 2008

Figure 7. Excavation on the main spillway

As it turned out a 1:10 year flood occurred during construction. It was however at a late enough stage that damage to the works was minimal and work resumed quite soon after the passage of the flood.

5.2 Clearing the River Bed

As the geotextile was to be laid on the river bed it was critical that the whole area was inspected first and any obstructions removed. The contractor deployed divers with video cameras who filmed along predefined routes marked with line underwater. The video for each section was reviewed by the resident engineer and the design team. We were expecting to find debris or possibly large rocks on the river bed but eventually the main issue was a colony of oysters about 20 m^2 established both on sand and rock on the river bed. These were removed by hand because their sharp edges could have damaged the geotextile particularly when rockfill was being placed on top of it.

5.3 Placing Foundation Geotextile Layer on the River Bed

A key aspect of the construction was to ensure that the foundation geotextile layer was properly placed on the riverbed with the correct laps (2 m specified). A number of ways were discussed for this including sewing large sheets together and pulling them out from the bank over the river. The final solution however included the construction of a purpose made barge by the contractor. Large panels of geotextile were pre-sewn together and fitted with 250 mm long pieces of 30 mm reinforcing bar placed at 2 m centres. The panels were 12 m wide so the net coverage after laying was 10 m. The contractor rolled each pre-sewn section of geotextile onto a large roller (a 12 m long 150 dia. steel tube). This was fitted on the rear of the purpose made barge.

Figure 8. Preparing geotextile sheets prior to laying

Figure 9. The cable used to guide the barge during laying operations

The position of the barge was accurately controlled by two steel cables that were strung across the river and securely anchored on each side. Slowly the geotextile was rolled off the roller as the barge was pulled across the river. The steel rebar weights made the geotextile sink and this was further aided by placing sandbags on the completed sections from a motor boat. Later a diver went down to both inspect the completed geotextile and remove the rebar weights so that they could be reused. The construction started at the downstream end of the embankment and proceeded upstream so that each new section of geotextile would lap over the previous section and prevent lifting due to flow. The final section of geotextile was securely anchored on the upstream side using sandbags. This was to reduce the possibility of a whole panel of geotextile being washed down the river in the event of a flood.

5.4 Pioneer Rockfill Embankments

After placing the foundation geotextile layer on the river bed, the contractor proceeded with the construction of the two pioneer rockfill embankments. The contractor chose to place these in two stages, the first stage entailing the construction of both embankments to just below the river water level, working from left bank to right (see Figure 11) and in the second stage raising the pioneer embankments to the design level of 1.5 m above sea level, working from right bank back to left. This two stage process allowed the contractor to place loads of rockfill on the geotextile more accurately with a back actor and gave more working space on top of the embankment. Safety on the project was a high priority and the construction vehicles were carefully directed when crossing the new embankments.

Figure 10. Laying the foundation geotextile layer from a specialised barge

Figure 11. The two pioneer rockfill embankments as they progress across the river just below water level

5.5 Closing against the Right Bank Sand Dune

One of the major challenges was the removal of 600 mm dia mangroves from under the embankment on the right bank. It was decided to leave the root systems in place to prevent damage to the sand dune and only remove the top part of the trees to below ground level. The contractor first tried to do this by hand with axes and saws but found it almost impossible. Finally they completed the clearing with an excavator immediately prior to closing against the right bank when the tidal flow in the river had been stopped by artificially closing off the estuary mouth. The holes left by removal of these trees were filled with sand prior to placing of the geotextile.

Figure 12. One of the mangroves on the right bank

Figure 13. Closing the embankment against the right bank

5.6 Sand Core

As mentioned previously the sand core is a critical part of the structure. The 3 m wide base of the core is the “path of least resistance” for seepage and therefore it was carefully inspected by divers with video camera prior to laying the containing geotextile on the base to ensure that no large rocks had rolled into this area. The 3 m wide base of the sand core therefore had a double layer of geotextile installed to further act as a safeguard for this critical area. This also served to simplify the construction of the sand core.

Figure 14. Placing the containing geotextile for the sand core

Figure 15. Final stages of the embankment construction showing the above water level sand core, geotextile, filter, rockfill and riprap layers

5.7 Final Stages of the Embankment Construction and the Masonry End Wall

After the flood in February 2008 it was a relatively straightforward task to complete the remainder of the embankment by adding the required layers. This included the construction of a masonry end wall against which the embankment was finished. The local Malagasy masons made an excellent job of this wall which serves to protect the end of the rockfill embankment from the higher velocity flows during floods.

6. CONCLUSION

The design of the embankment for the salinity control structure comprising a geotextile-contained sand core provided an economical solution in this particular situation where the head difference across the embankment is relatively low and seepage associated therewith is acceptable. The challenges of constructing the embankment without coffer dams in 4 m of water were overcome through close co-operation between the design team and the contractor from an early stage and the contractor's adoption of sound construction techniques, especially for laying of the geotextile underwater.

Figure 16. The completed embankment, boat lock and low flow spillway