*Third A-NZ Young Geotechnical Professionals Conference, Cameron, Collingwood & Slatter (eds), Feb. 1998, Melbourne, Australia.*

# Design and Construction of Sand Dam to Float Dredge over Buried Pipelines

Peter W Wright Geotechnical Engineer/Hydrogeologist Douglas Partners Pty Ltd


## SUMMARY

Mineral sands mining by dredging is carried out within the Tomago Sand-beds aquifer, which is also used to supply potable water to the Newcastle region. In order to transfer a floating dredge and separator plant over twin 1 m diameter water supply pipelines, a large temporary lock system was constructed of sand, comprising 4 m high dam walls. The water level was raised to 3 m above original ground level to enable the mining plant to float across the pipelines.

Engineering design and construction of the lock system was required to prevent damage to the strategically important pipelines. The design included slope stability and seepage analyses as well as analysis of stresses imposed on the pipeline by the embankment. The necessity to control stress on the pipeline required spanning over the pipeline with a steel plate structure underlain by a void. The presence of the void within the embankment placed particular emphasis on the control of piping, which had potential to cause both filling of the void and embankment failure. After consideration of both cut-off walls and lining systems, piping was controlled using a combination of sand and gravel toe berms underlain by filter fabric, as well as baffles along the steel plate. A void was maintained below the steel plate utilising reinforced filter fabric.


## 1 INTRODUCTION

The aim of the project was to design and construct a lock system to enable safely floating a sand mining dredge over strategically important water supply pipelines. The primary constraint on the project was the necessity to use the sands on site for construction of the dam walls. This constraint required the design to overcome some interesting problems as discussed in the following sections.

Involvement in the project comprised three main stages as follows:

- preliminary design;
- detailed design;
- construction testing.

The Tomago Sand Beds are a major source of potable water for the Newcastle Region. Douglas Partners has had extensive involvement in water quality management of the aquifer over the last 20 years and thus have significant data on the site as discussed in the following section.

## 2 SITE CHARACTERISTICS

The Tomago Sand Beds comprise an extensive unconfined sand aquifer. The aquifer has an average thickness of around 20 m and is underlain by clay and sedimentary bedrock. The sands are generally fine to medium grained, in-situ and laboratory testing suggests a typical hydraulic conductivity of $1\times10^{-4}$ m/s to $2\times10^{-4}$ m/s.

Antecedent groundwater levels at the crossing location are generally between one and three metres below ground level. Hydraulic gradients in the area were generally in the range 0.003 to 0.004.

The site contains both previously mined and natural unmined sand. Cone penetration testing undertaken in the sandbeds indicates that areas of hydraulically placed mine tailings are generally in a very loose to loose condition. Unmined ground is typically in a medium dense to dense condition.

Existing twin 1.0 m diameter water supply pipelines cross the site. The pipelines are of concrete lined steel construction with their obvert level 1 m below ground surface level.

## 2 PRELIMINARY DESIGN

### 2.1 General

The preliminary stages of design work comprised assessment of the practicality to construct the dam walls from clean sands. Assessment of seepage flows through the relatively permeable sands was a primary consideration together with slope stability analysis.

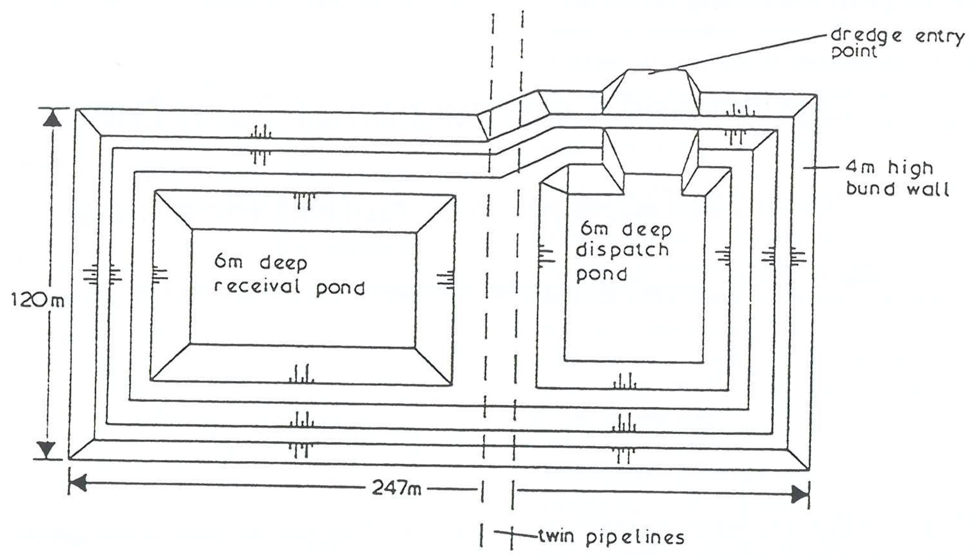


Figure 1 – Sketch of overall layout

The overall proposed lock system comprised a rectangular dam 240 m long and 115 m across, with ponds excavated within the bunded area either side of the pipeline. The overall scheme layout is shown on Figure 1. The mining plant required at least 3 m depth of water, thus dam walls were 4 m high to allow 1 m freeboard, the ponds required excavating to approximately 6 m below existing ground level.

## 2.2 Seepage

The seepage analyses was carried out using the computer modelling package SEEP/W, which allows transient analyses and unsaturated flow. In order to model unsaturated flow the program makes use of functions to relate both hydraulic conductivity and volumetric water content to pore pressure. Standard library functions appropriate for a fine to medium grained sand were used, and are presented in Figure 2 below. The functions take into account the reduction in hydraulic conductivity and water content for negative pore pressures (unsaturated conditions).

The seepage analyses and stability analyses are somewhat interdependent and preliminary seepage analyses were based on batter slopes of 2H : 1V. The cross section was assumed homogeneous and isotropic. A sensitivity analysis was performed on the saturated hydraulic conductivity, ranging it from $1x10^{-4}$ m/s to $2x10^{-4}$ m/s whilst retaining the general shape of the function appropriate for unsaturated conditions.

Steady state modelling indicated that with the dam full, hydraulic gradients were less than the critical gradient appropriate for piping of the sands. The primary concern was surface erosion of the sands at the toe, due to seepage exiting the lower slopes of the batter. As such, the seepage analyses concentrated on the time taken for seepage to surface at the downstream toe.

It was estimated a period of at least twelve hours was required to fill the dam and the transient analyses indicated that the wetting front was likely to reach the downstream toe within six to eight hours of the dam being full. Both partial lining of the upstream batters and a cut-off wall 10 m to 15 m deep were modelled. Each case still allowed propagation of the wetting front to the toe within the period required to transfer the dredge, however hydraulic gradients at the toe were significantly reduced thus reducing the likelihood of piping.

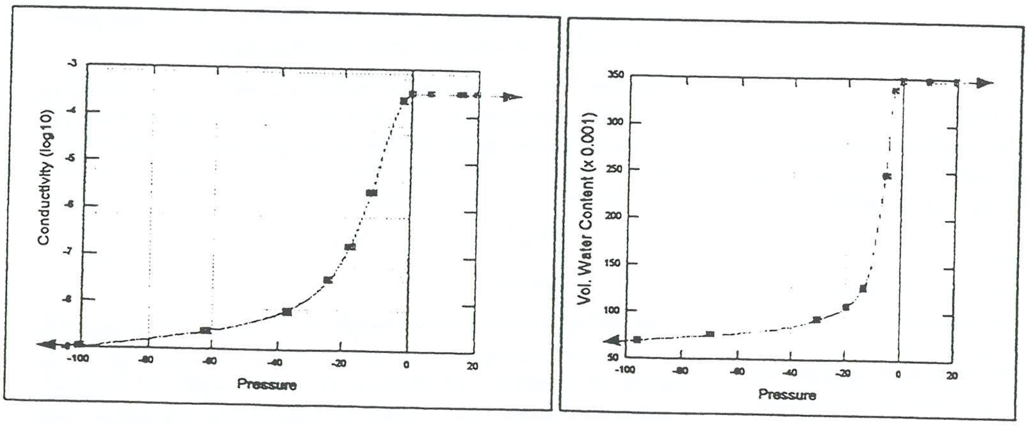


Figure 2 – Hydraulic conductivity and volumetric water content functions

Due to the uncertainty of the effect of seepage at the toe, as well as cost and construction considerations, a third option was adopted which involved placing filter fabric at the toe beneath a 1 m high berm constructed of sand. The

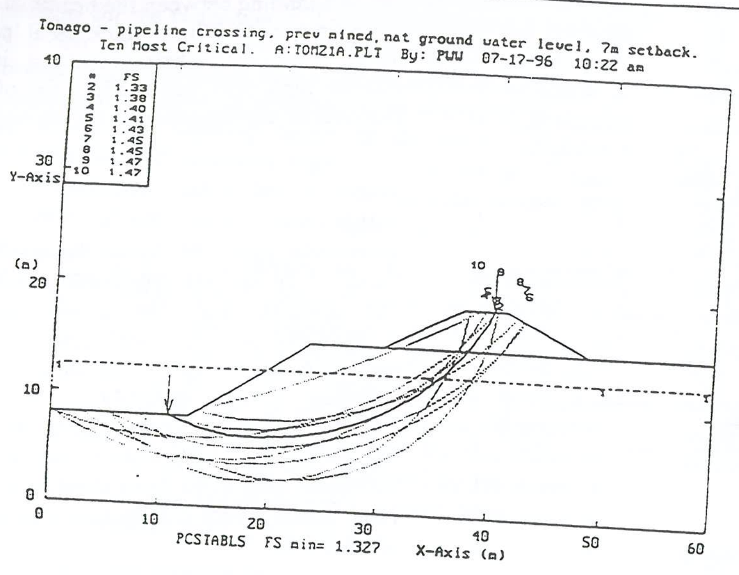


Figure 3 – Upstream stability output

ballasted filter fabric was to provide control of piping and the berm provided enhanced down stream stability of the embankment.

Steady state modelling of the toe berm option indicated seepage rates in the range 10 to 20 litres/min/m run of the dam. The estimated seepage rates were then used for preliminary assessment of pumping requirements to fill the lock system.

### 2.3 Stability

Stability analyses was carried out using the slope stability model PCSTABL 6.0, using the following strength parameters;

- Previously mined ground : $\phi' = 28°$
- Natural unmined ground : $\phi' = 30\text{-}32°$
- Constructed bund wall : $\phi' = 32\text{-}34°$

The strength parameters were considered relatively conservative, however they were based on limited site specific data, and allowed for refinement of the analyses during final design work if required. A minimum density index of 75 % was assumed for the constructed bund wall. A minimum acceptable factor of safety of 1.2 was adopted based on the temporary nature of the works. Slip circle type analyses was carried out based on various seepage conditions.

The modelling indicated that the antecedent water levels were critical for upstream stability and the elevated water levels, calculated from the seepage analyses, were critical for downstream stability. For 2H:1V batters of the constructed dam, downstream stability was acceptable particularly if the 1 m high berm discussed in the previous section was constructed at the toe.

The upstream profile was assessed based on a batter slope of 1.5H:1V for the excavated pond walls and a 2H:1V for the constructed dam wall. The modelling indicated that a setback distance between the toe of the bund and the edge of the pond was required, varying from 2.5 m for unmined ground to 7 m for previously mined ground. A typical stability output is presented on Figure 3. Minor slumping of the saturated pond walls was not considered to be critical to overall stability.

## 3 DETAILED DESIGN

### 3.1 General

Once the viability of the project was confirmed detailed design was undertaken. Particular attention was placed on protection of the buried pipelines. Fretting and overall stability of the excavated pond walls adjacent to the pipelines; the effect of the dredge landing above the pipeline in the event of a breach of the dam; and the effect of the surcharge load on the pipeline from the embankment were all considered.

### 3.2 Stability

A ten metre setback between the edge of the pond and the edge of the pipelines was adopted to provide protection from fretting but more particularly to increase stability in the case that the dredge landed above the pipelines.

### 2.3 Stress on Pipelines

Stress analyses on the pipeline, undertaken by the civil consultants involved in the project Advitech Pty Ltd, estimated that the pipelines could sustain a safe applied load equivalent to approximately 1 m thickness of sand without cracking of the concrete liner occurring. The ratio of vertical to horizontal stress on the pipelines was found to be particularly important. As 4 m of compacted sand was to be

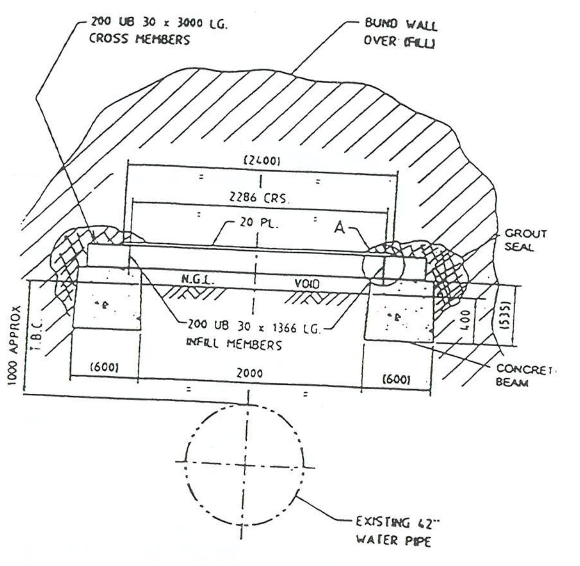


Figure 4 – Section of structure over pipelines

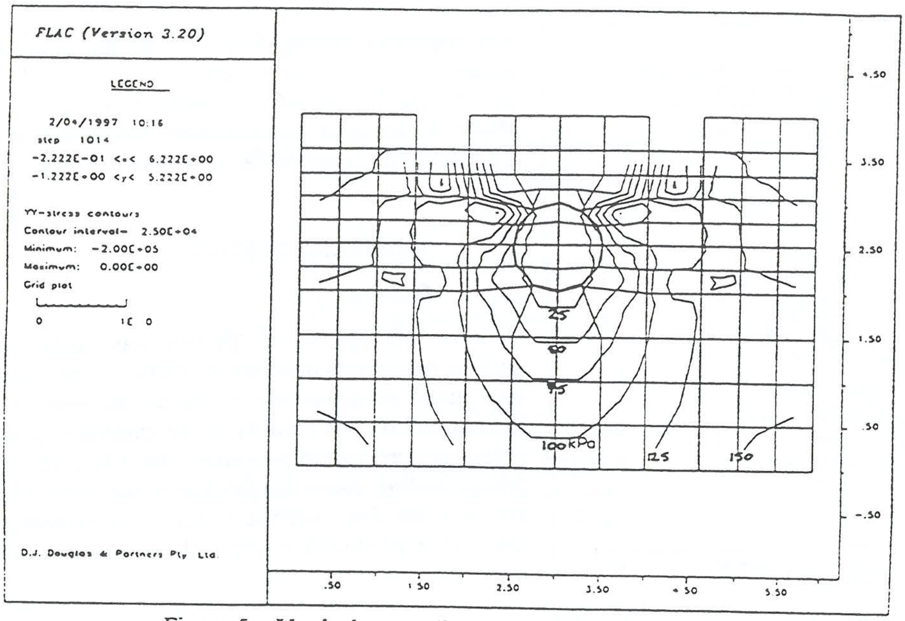


Figure 5 – Vertical stress distribution around pipeline

placed over the pipelines, measures to reduce stress on the pipelines were required.

Several options were considered to control stress on the pipelines including concrete encasement of the pipeline, however the Hunter Water Corporation, owners of the pipeline, required that excavation around the pipelines was kept to a minimum.

The preferred option was the construction of a structure which spanned across the pipelines, distributing the load of the overlying embankment either side of the pipeline. The structure comprised a concrete beam placed each side of each pipeline, in parallel, 2.5 m apart. Steel beams were then placed spanning between the beams at 1.5 m centres, with 20mm thick steel plate placed over the beams, resulting in a 300mm high void below the steel plate. The overall configuration is shown on Fig 4.

Analyses of the resulting stress distribution around the pipeline was undertaken using the modelling package FLAC. A Mohr-Coulomb/Elastic analysis was adopted and the pipeline was incorporated into the model as a solid with an equivalent modulus. The modelled stress distribution indicated maximum vertical total stresses at the pipeline of 50 kPa and a vertical to horizontal total stress ratio close to unity. The vertical stress distribution is shown on Figure 5.

### 3.4 Seepage

The restriction that no more than one metre of filling could be unsupported above the pipeline meant the void ran almost the full width of the embankment. The continuation of the void close to the downstream toe significantly increased the risk of piping. The void would provide a preferred path for flow beneath the embankment resulting in high hydraulic gradients at the toe. The impermeable steel plate above the void would force seepage flowing through the embankment to be concentrated at toe where the height of the embankment above the steel plate was less than a metre. This would increase the hydraulic gradient as well as force the seepage exit point further up the embankment.

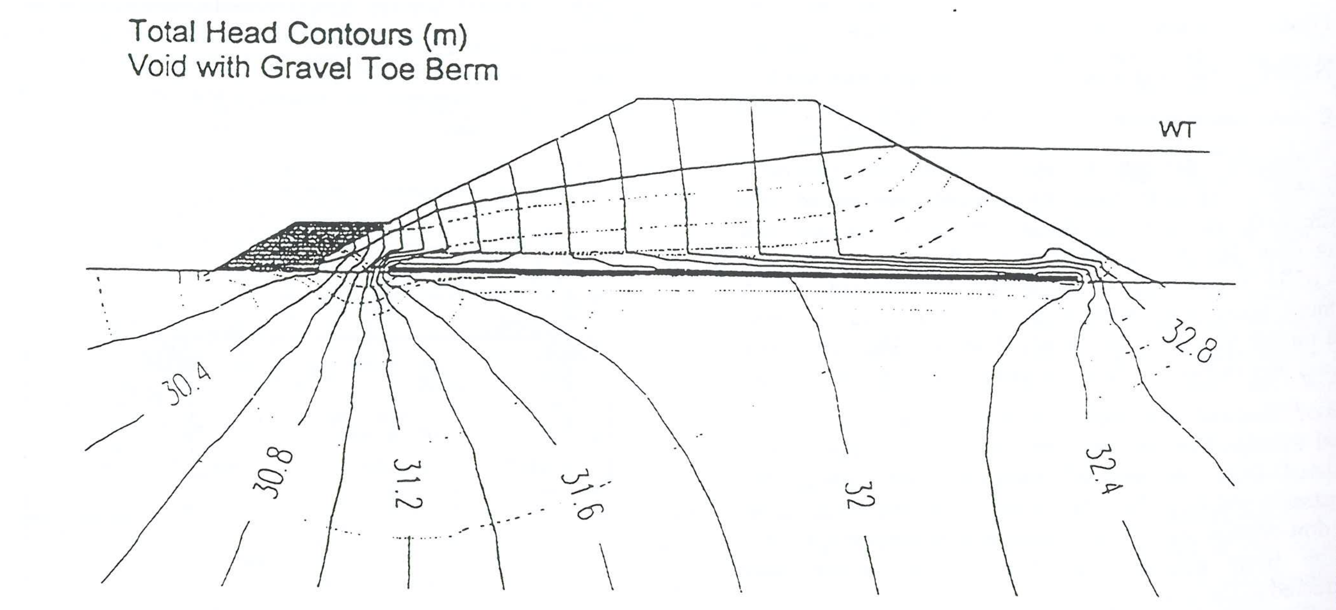


Figure 6 – Total head distribution for seepage with void and gravel toe berm.

In order to control the seepage exit point, the sand toe berm as specified for the remainder of the embankment was replaced by a free draining gravel, underlain by filter fabric. The seepage regime modelled with SEEP/W is shown on Figure 6. A distinct draw-down of the water table is evident at the toe due to the free draining nature of the gravel. Hydraulic gradients are still relatively high, however the gravel berm allowed the use of a higher critical hydraulic gradient and thus an acceptable factor of safety against piping.

Additional measures to reduce the risk of piping included welding 300 mm high baffle plates to the top of the steel plate where the smooth interface otherwise provided potential for piping to develop. Due to the constraint that the gravel berm could only be one metre high, additional filter fabric was placed along the surface of the batter slope immediately above the berm in case the seepage did exit the face above the gravel berm. The filter fabric was anchored by the gravel at the bottom end and anchored between sand layers, during embankment construction, at the top end.

A secondary consideration was mobilisation of the sand at the upstream end of the void such that it filled the void, thereby transferring load from the embankment onto the pipelines. This concern was addressed by spanning reinforced filter fabric between the concrete girders such that it held the underlying sand in place. Figure 7 shows the details of preventative measures used to stabilise the sands.

## CONSTRUCTION

Performance of the lock system was very much dependant on careful construction of the bund walls and attention to detail at the pipelines. Full time construction supervision was provided by Advitech Pty Ltd, with Douglas Partners providing earthworks inspections and testing.

Compaction was primarily checked using dynamic penetrometer testing. Calibration of the penetrometer was undertaken under laboratory conditions using sand from site placed at a controlled density index within a 200 litre mould. Ongoing correlation of penetrometer testing against field density testing and laboratory compaction was undertaken throughout construction.

The construction of the steel structure resulted in the ends of the void being open, as well a number of openings along the side of the void. The need for a low shrinkage, low cost and reasonably high strength sealing material, which could be placed with practical ease, resulted in the use of a dry mix of sand, cement and fly-ash. The material was placed dry to reduce shrinkage and was compacted using a vibrating plate, then overlain with wet sand to allow hydration. A cement/fly-ash content of 20% was used to provide sufficient strength to inhibit cracking due to deflection of the steel plate, the fly-ash content reducing shrinkage and cost.

## CONCLUSIONS

The mining equipment was successfully floated over the pipeline in March 1997. Subsequent camera inspection inside the pipeline confirmed no damage to the concrete lining of the pipelines. Observation made while the water level was at full height confirmed that the seepage and piping control measures had worked. Seepage was observed emanating from the gravel toe berm and elsewhere only minor slumping of the sand occurred at one corner of the dam. The dam has since been removed.

The project provided a number of challenging problems, which required practical engineered solutions to allow simplicity of construction and control of costs. Dam design and construction always provides interesting design issues, however the necessity to use relatively permeable sands for construction of the lock system and the added complexity of controlling stresses on the pipeline made this project particularly unusual.

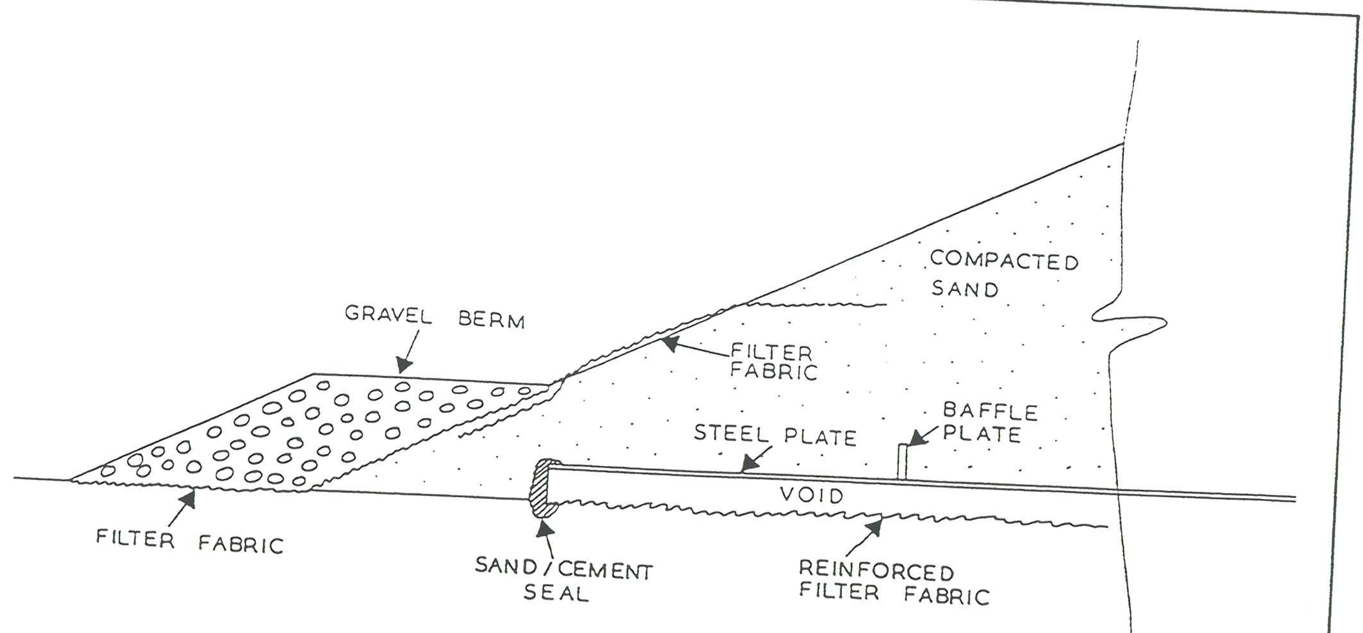


Figure 7 – Sketch of measures used to stabilise sand around the void structure.

## BIBLIOGRAPHY


- Carter M, "Geotechnical Engineering Handbook", Pentech Press.
- Douglas Partners Pty Ltd, "Report on Geotechnical Assessment, RZM Pipeline Crossing, Tomago", Project no 18471, March 1996.
- Douglas Partners Pty Ltd, " RZM Pipeline Crossing, Tomago", Project no 18471A, August 1996.
- Douglas Partners Pty Ltd, " RZM Pipeline Crossing, Tomago", Project no 18471A/1, February 1997.
- Geoslope International, "SEEP/W user manual", Ver 4.0, 1996.
- Lee, White & Ingles, "Geotechnical Engineering",1983, Pitman.