Sweet Smell of Successful Risk Management

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Tropical Forestry Services (TFS) currently (2015) leases Arthur Creek Dam from the West Australian state government and utilises the water source to drip irrigate its Indian sandalwood (Santalum album) plantation. Arthur Creek Dam is located approximately 70 km south west of Kununurra in the East Kimberley region of Western Australia. TFS grows and processes the sandalwood to produce oil that is used extensively in the global fragrance perfume market. TFS took over the lease of the 26 m high zoned earth core and rock fill dam in 2007 and has systematically carried out remedial works to the structure to lower the f-N curve below the ANCOLD “Limit of Tolerability” and to well within the ALARP zone. This paper describes the proactive risk management approach TFS has undertaken to address dam safety issues. It also specifically describes the most recent management issue, being the outlet pipe refurbishment.

A number of dam safety issues were identified during the initial surveillance and subsequent annual surveillance inspections. Issues include insufficient spillway capacity, seepage from the right abutment and deterioration of the steel outlet pipe. The remedial works to the outlet pipe were completed in late 2014 and involved close collaboration between TFS, the contractor and the designer. The outlet pipe re-sleeving operation was complex as the dam had to remain in operation and the water level could not be artificially lowered. In addition, the original outlet pipe was asymmetrical along both the vertical and horizontal axes, close to the bulkhead gate structure. Contingency measures were employed to enable the dam to remain in operation with 3 DN 400 HDPE siphon pipes installed.

The completion of the refurbishment of the outlet pipe by sleeving the pipe reduced the risk posed by this structure by an order of magnitude. Planned future risk reduction measures include the treatment of seepage within the upper right abutment and rebuilding the crest. These actions will further reduce the risk of dam failure through piping and overtopping of the dam crest.

**Keywords:** risk, ALARP, outlet pipe, re-sleeving.

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**Project Information**

Tropical Forestry Services (TFS) grow and process Indian Sandalwood (Santalum album) to produce Pharmaceutical Grade Sandalwood oil. This is used primarily in an expanding range of medicinal products and also in the elite global fragrance industry. Wood from the plantations will, for the first time in history, supply the Asian fragrant wood market with legally traded wood. TFS leases the Arthur Creek Dam from the West Australian state government and uses the water source to irrigate their oil producing sandalwood plantation.

Arthur Creek Dam is located approximately 70 km south west of the Kununurra townsite in the East Kimberley region of Western Australia and reached via the Victoria and Great Northern Highways (see Figure 1). The dam was constructed in 1971 for Goddard Australia, which at the time was one of the largest private irrigation scheme owners in the state. Arthur Creek is classified as a “High B” Consequence Category dam, based partly on the risk to life and also because of its potential impact on the business downstream.

**Dam Features**

Arthur Creek Dam consists of the main embankment, outlet works, spillway and reservoir. The dam is approximately 26 m high, stores 74 GL of water and has a catchment area of 92 km². There are two natural spillway saddles, which are located at about opposite sides of the dam and reservoir (Refer to Figure 2). The original drawings of the project included an embankment through one spillway saddle, presumably to reduce the likelihood of flow over this, the lower of the two saddles. The other saddle was to be excavated over a width of 115 m.
The embankment is of clay core, rockfill type. The supporting shells were composed of a fine sand inner zone and a coarse gravel and cobble outer zone with a surfacing layer of shale/weathered dolerite over the outside face of the dam. These zones of sand and gravel should provide a very good filtering function, but tend to lack the overall stability that would be provided by a genuine rockfill zone.

The upstream and downstream slopes are 1V:1.7H for the top 7 m and 1V:1.9H for the lower portion of the slope.

**Operating History**

During the first wet season, hillside runoff on the downstream right abutment was directed back onto the right hand groin of the dam by construction haul roads. This resulted in a significant erosion gully forming on the right abutment that was filled in and corrected.

The original construction access to the site was from the upstream side along the Bedford Stock Route. Once the reservoir filled, access to the embankment by road was almost impossible. The only options were to drive to the edge of the storage along the Bedford Stock Route and then take a boat across the storage to the dam site, or to walk up the valley from the diversion dam site. This latter option was very difficult due to the thick masses of vegetation and walking across the hills was often an easier option.

The first dam inspection of the site by the second author was in March 1974, three years after construction was completed. This visit identified excessive seepage from the toe and from the abutments of the dam. The inspection report recommended that the storage be operated at reduced levels to minimise the risk of a piping failure. The PWDWA and later the Water Authority of Western Australia (WAWA) monitored seepage from the dam up until 1988, initially on a monthly basis and later on a three monthly basis until they ceased to have any interest in the dam.

More recently wave action during a major storm event (2006) caused serious erosion of the upstream face of the dam and the crest. Verbal advice is that the damage caused a scarp several metres high to form on the upstream face and resulted in a substantial narrowing of the crest width in places. A scarp at the current water levels was still visible on the upstream face, although the major damage has been repaired. GHD understood that the owner stripped about 200 mm from the top
of the dam and then bulldozed rock scree material across the crest to reinstate the upstream face and crest. Apparently no attempt was made to restore the zoning in the embankment.

**Risk Management**
The initial review in 2008 found that the main risks were
- Inadequate spillway capacity
- Lack of rip rap on the upstream face
- The risk of piping through the crest at high water levels due to lack of filters
- The high rates of seepage through the right abutment
- The potential loss of control of the outlet works due to valve failure
- The potential for corrosion to reduce the effective thickness of the pipe wall with an associate risk of a pipe collapse or pipe burst

**Reduced PAR**
TFS has undertaken a very active approach to managing these risks. One of the very first decisions was to site all new housing and workshop sites out of the dam break path thus reducing the population at risk (PAR) from 10 to 2.

**Improved Inspection Regime**
TFS has also been undertaking routine monitoring and inspection of the dam on a daily basis. This work includes measuring the water level in the reservoir and measuring seepage flows as well as recording valve settings and making routine visual observations of the key parts of the embankment and the surrounds.

**Spillway Upgrade**
The original design drawings show a spillway design involving an embankment across the most westerly of the saddles and an excavation in the more easterly saddle (*Figure 2* shows the location of Spillway Saddle 1 and Saddle 2). After TFS assumed the lease of the dam, a review in 2008 discovered that the spillway had not been constructed as designed and would have a capacity to only be able to pass a flood with an Annual Exceedance Probability (AEP) of the order 1:100.

In 2008 TFS undertook an immediate enlargement of the spillway by lowering spillway saddle No 2 by just over 2 m. This gave an interim spillway capacity just in excess of 1:1000 AEP flood.

*Photograph 1 Spillway No 2 excavation and first flow February 2009*
Outlet Works

The original outlet works consisted of a 900 mm diameter steel pipe with a coarse screen on the upstream end. The original arrangement included a gate valve on the downstream end to provide a service valve function and a clam shell valve to provide the flow regulation function. Under this arrangement the gate valve would not be used to control the flow; this would be done with the clam shell valve. When the clam shell was closed, the gate valve could be closed. Provision was made for a bulkhead gate to be installed by divers at the upstream end of the outlet conduit. From the drawings it is understood that the steel pipe was concrete encased or at least seated on a concrete support block where it passed through the core. Subsequent inspections showed that the pipe was given an epoxy paint lining. The external coating was believed to be a bitumen enamel wrap.

Works undertaken in 2009 replaced the existing gate valve with a high performance butterfly valve and the clam shell was refurbished. A new bulkhead gate was built and installed by divers. While the outlet works were dewatered an eddy current inspection of the pipework was undertaken to determine the remaining thickness of material in the pipe wall. The pipe inspection was carried out by Applus (Previously known as Pilbara NDT) and the test used was termed an Inco Test. The measurement of the pipe wall thickness indicated that while the pipe had adequate margins of safety at the time, work to upgrade it would be required in the future.

With the passage of time and continued deterioration of the pipework, coupled with a reassessment of the techniques used to assess the remaining life of the pipe based on work at Samson Brook Dam, the decision was made to refurbish the pipe where it ran through the embankment. The 2014 remedial work involved sleeving a DN 750 by 12.4 mm wall thickness Grade 350 steel pipe inside the original DN 900 pipe and grouting the annulus.
The outlet works are described below.

**Embankment Reinstatement**
During 2012, a site inspection identified that the upstream face had lost its rock protection and works were undertaken to restore the riprap and restore the crest level to RL 105.2 m AHD. This slight raise of the embankment crest further reduced the flood risk by an order of magnitude.

**Achieving ALARP**
The $f$-$N$ curve for the project is shown in Figure 3. The initial status of the risk assessment put the project well above the limit of tolerability. The work to date, reducing the PAR, improved surveillance, upgrading the spillway and the outlet works, has reduced the risks into the ALARP zone.

However there are some risks remaining and further works are planned to ensure that an acceptable ALARP status is maintained. The work over the next few years will include raising of the embankment crest level to include filters and rip rap as well as providing improved flood security.

![Figure 3 $f$-$N$ curve for Arthur Creek Dam](image-url)

**Figure 3 $f$-$N$ curve for Arthur Creek Dam**
Outlet Works Refurbishment

During 2014, TFS took the decision to refurbish the outlet works.

The project timeline was approximately as follows:

- Specification and design: Early May 2014
- Planning work: May 2014
- Mobilise to site: End of August 2014
- Install bypass pipework: First two weeks of September 2014
- Mob divers, pipe jacking crew: Last week in September 2014
- Pipe jacked into position: 1-5 October 2014
- Grout Pipe: 14 November 2014
- Complete Installation: 21 November 2014

As part of the refurbishment, the original outlet works had been decommissioned by placing the bulkhead gate in position on the inlet, dewatering the outlet works and removing all the pipe work and valves downstream of the anchor block. The leakage from the pipe when measured on 5 October 2014 was 0.3 L/sec. The final section of DN 750 steel pipe was pushed into position mid-morning on 5 October 2014 ready for final grouting of the annulus.

The assessment made by the site staff and the divers was that the original pipe had settled by about 120 mm in the middle of the pipe, that it deviated by about 100 mm in alignment and that the pipe had been flattened by about 50 mm under the loads.

Video Camera Inspection of Internal Section of Original Pipe

The overall length of the pipe was found to be 92.4 m with an end clearance measurement of 0.46 m. This was significantly longer than had been identified in either of the previous inspections (85 m), either by camera or by the divers with an ROV. The video inspection of the cleaned surfaces of the pipe showed at least six areas of the pipe where the existing coating was no longer effective, all of which were in the most upstream 56 m of the pipe.
A temporary siphon was installed to maintain flows while the outlet works were being reinstated. The system consisted of three DN 400 HDPE pipelines with an intake in the reservoir and steel pipe sections across the crest with vacuum generating equipment to keep the line de-aired. Each pipe was fitted with a magnetic flow meter. The system had a nominal capacity of 2000 L/sec and was operating at between 200 and 500 L/sec during the site visit on 5 October 2014. The inlet had been fitted with a screen to prevent wildlife and debris getting sucked in and blocking the pipework.

### Pipe Jacking

The pipe installation process consisted of:

- Setting up the first pipe on the jack bed
- Pushing it into position with the 100 tonne jack
- Placing the next pipe on the jacking bed and welding to the already installed pipe
- Conducting air pressure test of joint and magnetic particle test of fillet welds
- Pushing the pipe string along the carrier pipe

This procedure was repeated until the full pipe string had been inserted in the carrier pipe. Photograph 6 to Photograph 11 illustrate the steps involved.
The sleeve pipe was a DN 765 by 12.7 Grade 350 steel pipe with cement mortar lining manufactured by PLC of Rutherford in NSW. The pipe was delivered to DM Civil’s yard in Maddington. OJG completed weld preparation for the joints before the pipe was transported to site.
The jack used for the project had a capacity of 100 tonnes force (1000 kN). The final push home was at the limit of the capacity of the jack. The additional force required to push the pipes home is thought to be due to the distortion of the original carrier pipe.

**Packer**

The final grouting of the pipe sleeve was not completed until 14 November 2014. The original planning had been to place a packer on the end of the sleeve pipe before the first section of pipe was installed in the outlet works. However after the pipe string had been jacked into position and the packer inflated, the packer suddenly deflated some 30 minutes later. Following removal it was found to have punctured (Refer to Photograph 12 and Photograph 13). The puncture was believed to be largely caused by the eccentricity of the sleeve pipe within the outer carrier pipe.

In order to maintain the security of the outlet works while a revised packer system was designed and manufactured, the downstream flange was reinforced to carry the full water load (Refer to Photograph 14).
The divers were able to manufacture a template of the opening to be plugged (Refer to Photograph 15). This photograph shows the template that was taken to allow for the design of the new packer and shows the significant offset of the pipes at the upstream end.

The redesigned packer consisted of 5 layers of 40 durometer hardness natural rubber, 20 mm thick, cut to the template shape and bolted together. The packer was designed to be inserted with the nuts loose; these would be tightened when the packer was positioned.
The initial design had two sections of the packer, but this was trimmed to one section to ease the difficulties for the divers in the installation process. The narrowest section of the packer was only 30 mm wide and tended to distort when the clamp bolts were tightened. This was rectified by adding a stiffener to this section of the metal facings. A test of the packer found that it adequately held the pressures expected during the grouting operation.

Pipe Grouting

The final scope of works to complete the reconstruction of the outlet works included the following steps:

- Late October 2014 Redesign of a plug to suit the opening, and testing
- 10 November 2014 Remobilise to site
- 11-12 November 2014 Divers install plug
- 13 November 2014 Divers relocate bulkhead gate, prepare for grout
- 14 November 2014 Grout injection to annulus
- 15 November 2014 Check for re-grout

The grout procedure involved filling the annulus and the pipework with water, injecting grout from the base of the downstream bulkhead and gradually displacing the water from the annulus using the various injection lines and air bleed locations. The overall operation took place from 0300 to 0800 hours on the morning of 14 November 2014. Ambient temperatures on site ranged from 28 to 31°C.

The grout used was initially planned to have a water cement ratio of 0.45. However even with the optimum doses of set retarder and superplasticiser, the grout would have been difficult to pump and would have taken up its initial set too quickly in the hot conditions. The decision was made to trial a 0.5:1 water cement ratio mix, with 300 ml/100 kg of cement of Sika Retarder N (set retarder admixture Type RE) and 300 ml/100 kg of cement of Visco crete 10 a high range water reducer set retarding admixture (Type HWRRe).
The overall volume of grout required to fill the annulus was 23 m³, requiring just over 28 tonnes of cement in 42 tonnes of grout. The theoretical specific gravity (SG) of the grout is 1.83. Check values of the SG were used to verify the batching and mixing of the grout and these closely followed the theoretical values (Table 1). Each batch weighed had a mass of 450 kg (nominally) and consisted of 300 kg of cement and 150 kg of water giving a batch volume of 245 L. A total of 97 batches were required to complete the grouting.

Photograph 20 Night grouting operation in progress 14 November

One of the key production parameters was the fluidity of the mix. The flow cone used for the test was the AS 1478.2 cone with a 12.7 mm outlet, which is primarily used for relatively viscous grouts such as are being used for this installation. The cone test measures the time taken for 1.725 L of grout to flow through the cone. The more fluid the grout, the shorter the time for the flow to complete. The limit of the fluidity that would enable satisfactory pumping was 12 seconds, the mixes averaged between 10 and 12 seconds for the duration of the grouting procedure.

Table 1 Grout Quality Control Parameters

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<th>Flow cone time (secs)</th>
<th>Specific Gravity</th>
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</thead>
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<td>1</td>
<td>12.5</td>
<td>1.84</td>
</tr>
<tr>
<td>2</td>
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<tr>
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Samples of grout were tested for strength at 28 and 56 days of age. The mean strength values are shown in Figure 4. The grout achieved the target strengths expected for this grout.

Figure 4 Grout strength data

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Contemporary Challenges for Dams
In the planning for the grouting operation the trials showed that it was likely that the mix would bleed slightly, between 1 and 2%. Based on previous experience it was expected that this would leave a void at the upstream end that would need regrouting. Provision was made in the plug for the divers to attach fittings to enable the potential void to be refilled (see Photograph 16). When the divers attempted to unblock the grout fittings on the plug it was found that the grout was solid and the void was completely filled.

Photograph 21 shows the final stages of the grouting process with water and then grout being discharged from the bleed tubes.

Photograph 21 Final stages of grouting, 0700 November 14, 2014

Photograph 22 Completed works

Final Comments

The upgrade of the outlet works was completed successfully and they are now in first class condition. Photograph 22 also shows the valves and platforms reinstalled on the outlet works, with modifications to the clam shell operating worm drive support frame that eliminated the potential for fingers to be caught on the frame.

The proposed scope of remedial work post 2014 includes the following elements:

- Raising the main embankment by 2.3 m to RL 107.5 m AHD to provide adequate freeboard for the Probable Maximum Precipitation Design Flood (PMPDF) reservoir level
- The raised embankment will include a raised impervious core zone, protected by filters and supported by a rockfill shell
- Reconstruction of the downstream face of the embankment to support the raised crest level
• Reconstruction of the upstream face of the embankment to include a layer of larger riprap rock to provide protection against wave action for the crest when reservoir levels are high

• Improving the foundation cutoff on the upper right abutment of the embankment above RL 98 m AHD

Acknowledgements
GHD would like to firstly acknowledge Ben Wilson from Tropical Forestry Services for all his desire and willingness to support upgrade measures to the Arthur Creek Dam since GHD ongoing involvement from 2008. Secondly GHD would like to acknowledge Russell Blakely - General Manager of Contracts at DM Civil for his cooperate and collaborative approach with designer and client on the Arthur Creek Dam outlet remedial works.