Enlarged Cotter Dam Saddle Dams – Materials and Construction

Authors

Dr Mark Locke, Principal Engineer, GHD
Scott Kindred, Project Engineer, Bulk Water Alliance

The Bulk Water Alliance (BWA) consisting of ACTEW and ACTEW-AGL, GHD, and John Holland / Abigroup, are delivering the Enlarged Cotter Dam project in Canberra, ACT. The greatly enlarged reservoir will require two central core rockfill saddle dams on a ridge adjacent to the main dam site. Construction of these two dams was completed in early 2011. The challenges of the site and the Alliance delivery model have provided opportunities for innovation in both use of materials and construction.

The dam foundations were variably weathered and fractured with some highly weathered seams extending below the cutoff trench foundation. The foundation was grouted effectively using GIN grouting and the entire cutoff trench was shotcreted to reduce the risk of piping of the dispersive core material.

The steep topography provided very limited sources of material suitable for a dam core. Potential contingency plans considered included bentonite enrichment of the low plasticity materials or a change to a concrete faced rockfill dam. The high cost of these options drove the decision to use the available residual soils from small gullies by selectively winning material with a higher fines content for use below full supply level. The lack of room on the ridge for stockpiling and conditioning of clays lead to trialling of a continuous mixer for mixing and conditioning the core which was found to be highly successful.

Filter materials were crushed sands and gravels produced from nearby commercial quarries. The materials and grading were generally high quality, with some challenges producing coarser filter materials by blending available aggregate products. A range of options were effectively adopted for placement of the filters including loader placement, trench boxes and spreading from a modified ejector dump truck.

Keywords: rockfill dam, clay core, filters, GIN grouting.

Introduction

The Bulk Water Alliance (BWA) was formed in May 2008 and is currently delivering a suite of major water security projects for ActewAGL in Canberra. The BWA partners include GHD, Abigroup, John Holland and ActewAGL. One project within the Alliance includes the design and construction of the Enlarged Cotter Dam (ECD); a water supply reservoir located 18km west of Canberra on the Cotter River.

The project includes the construction of a new 80m high roller compacted concrete dam and two auxiliary saddle dams to the south west of the main concrete dam spanning the neighbouring low lying valleys. The Main Dam and Saddle Dams retain a large reservoir and have an Extreme hazard rating.

Construction of Saddle Dams 1 and 2 was a significant project delivered under the Enlarged Cotter Dam scope of work, with Saddle Dam 1 being 17m high, Saddle Dam 2 21m high and both approximately 300m long.

The work was technically challenging as the saddle dams had an intricate section design, were located in areas of significant subsurface fracturing and used the adaptive Alliance framework to accommodate site won materials. A systematic engineering approach was used through the project lifecycle to reduce the time and cost risk while still producing a quality product for the Client.

A select team of designers, geotechnical specialists and construction engineers then reviewed every aspect of the design and construction process and used a 3D staging model (Figure 1) to identify areas for improvement including:

1. Methods for effective preparation and treatment of a foundation that was highly variable in weathering and jointing;
2. Refining the section design and optimising the filter widths;
3. New techniques for conditioning and curing the 50,000m3 of site clay core material; and
4. Challenging design assumptions on material placement and compaction.

Figure 1 - A 3D staging model was used to analyse the pros and cons of different construction methods.
**Dam Options Study**

The project commenced with a detailed geotechnical investigation of the sub-surface conditions using diamond core drilling, downhole permeability testing, core sampling, seismic refraction surveys, statistical fracture surveys and detailed geological mapping. The outcomes formed the basis for the evaluation of different design options including:

- concrete faced rockfill dam;
- roller compacted concrete dam;
- rockfill dam with central core and bentonite enrichment; and
- rockfill dam with clay core using site won materials.

Initial pre-construction geological investigation for potential clay sources was restricted by approval and heritage clearances. Consequently a number of assumptions were made about the location and quality of clay deposits. A suitable source of clay material could not be identified in the pre-construction phase and hence, option d above was not considered feasible. As options a and b were determined to be prohibitively expensive, the target out-turn cost (TOC) for the project was based on bentonite enrichment of the gravelly residual soils on site. However, it was the aim of the construction team to find a cost effective solution of extracting clay that met the design criteria and hopefully avoided importing bentonite and the expensive blending process.

**Use of Site Won Residual Soils**

**Site Investigations**

Importing offsite materials was to be minimised for cost reasons and to mitigate community impacts. An extensive investigation program including 52 test pits was carried out along the ridge to the west of the saddle dams to search for potentially suitable materials close to the site. The deposits of residual soils were quite variable, ranging from high plasticity clays when completely weathered to gravelly sands with a small clay fraction when less weathered. The depth of the weathering profile was also highly variable from about 0.3m to 3m. A thorough program of test pitting was undertaken to determine the available quantities of potential borrow materials and the team persevered with exhaustive laboratory testing of the materials. Ultimately permeability testing was used to confirm its suitability as an impermeable clay core and particle size distributions and Atterberg limits were used to classify the materials.

**Zoned Core Optimisation**

Due to the lack of high quality clay materials, the design focussed on utilising the available materials most effectively and the core arrangement was optimised to the section shown in Figure 2, to meet the anticipated quantities of material. The borrow materials investigation identified a sufficient total volume of material for construction of a narrow clay core but approximately 60% of this was the higher permeability, less weathered material which did not meet conventional requirements for a clay core.

The borrow material was divided into two main material types. Zone 1A was a low permeability material consisting of clayey sand and sandy clay material. This was a mixture of the residual soil and selective inclusion of completely weathered rock from weathering of the underlying rhyolites. Samples had an average permeability of approximately $1 \times 10^{-9}$ m/s, which is appropriate for a low permeability core material.

Zone 1C was a silty / clayey sand with gravel, typically sourced from moderately to highly weathered rock, providing a semi-pervious to impervious material. Permeability testing on samples of zone 1C material gave a wide range of results from $10^{-10}$ to $10^{-7}$ m/s depending on the grading and PI of the sample. Seepage analysis was carried out considering steady conditions at full supply level and transient analysis for the design flood event to examine a range of options for the core arrangement to optimise the use of available materials. Based on the materials balance available, the feasible core arrangements considered included:

1. Bentonite enrichment of the available material to produce a conforming zone 1A material.
2. Core zone up to 1m above FSL as zone 1A, above this as zone 1C.
3. Upstream half of core up to 1m above FSL as zone

**Figure 2 - The saddle dam section design was complex and involved three types of clay, three filter materials and four types of rockfill.**
1A, zone 1C elsewhere.

4. Blend all available material to produce a single product with an intermediate permeability.

Modelling of the effect of the design flood event was carried out using a transient analysis which adjusted the upstream boundary conditions based on the flood hydrograph. A potential concern was that Seep/W sets the initial conditions based on the full supply level case assuming all water has drained from the unsaturated zone. In reality, construction pore pressures and rainfall will maintain some moisture in the core above the phreatic surface. This difference would result in underestimation of the flow through the core both because water would be stored in the voids in the core and the permeability drops rapidly with negative pore pressure. A range of models were considered to address this potential problem. These included the basic model case, changing the permeability function so that there is no reduction in permeability with negative pore pressure, i.e. a flat line function at the saturated permeability, or applying an infiltration to the core surface to simulate approximately 90% saturation (equivalent to the estimated construction moisture content) prior to the flood rise. It was found that the infiltration model gave the greatest degree of saturation and flow during the flood event, although the difference was not great.

A typical output from the seepage analysis is shown in Figure 3. The predicted seepage loss for both dams is listed in Table 1. The results of the seepage analysis were:

- for the steady state (FSL) case, the total estimated seepage is in the order of one l/min for a zone 1A core (options 1 and 2), which is relatively low. The proposed core arrangement has an estimated seepage of about 1 l/min, while the option of combining the two core materials into a single product has an estimated seepage about 10 times higher, this demonstrates the advantage of using the better quality material below full supply level.

- For the design flood case, the seepage through the higher permeability zone 1C material was high, but considered acceptable as the filter and downstream rockfill could easily handle this quantity of seepage.

A typical output from the seepage analysis is shown in Figure 3. The predicted seepage loss for both dams is listed in Table 1. The results of the seepage analysis were:

<table>
<thead>
<tr>
<th>Seepage Analysis Case</th>
<th>Steady Seepage (L/min)</th>
<th>Seepage Flood Case (L/min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>All 1A (blend and bentonite enrich)</td>
<td>0.9</td>
<td>2</td>
</tr>
<tr>
<td>Upper 1C, Lower 1A (proposed design)</td>
<td>0.9</td>
<td>77</td>
</tr>
<tr>
<td>1A upstream half of core, 1C elsewhere</td>
<td>4</td>
<td>36</td>
</tr>
<tr>
<td>Blend 1A and 1C to single material</td>
<td>11</td>
<td>27</td>
</tr>
</tbody>
</table>

Table 1 – Results of Seepage Analysis

On the basis of the seepage analysis it was decided to adopt the higher cost approach of selectively winning two core material products but to avoid the complex and costly process of bentonite enrichment.

Material Specifications

Specification requirements for zone 1A were a 75mm minus material with more than 75% of particles passing 4.75mm and more than 30% fines (passing 0.075mm). The plasticity index was greater than or equal to 8.

The placement specification required that Zone 1A be moisture conditioned to 0% to 3% above optimum moisture content. This material was then compacted in 150mm lifts after compaction using tamping rollers to at least 98% of standard maximum dry density.

Specification requirements for the 1C material were a 75mm minus material with more than 75% passing the 4.75mm sieve, greater than 20% fines, and a plasticity index greater than 6% to exclude topsoil and silt materials. Hence, this was more like a road pavement material than a traditional dam core. Moisture conditioning and compaction requirements for Zone 1C were the same as Zone 1A. It was found that this low fines material could not achieve the target compaction at greater than 2% above optimum moisture content.

Clay Winning

Clay material was won from multiple onsite borrow pits with material properties varying at each location. A complicated earthworks process was followed that required careful management to produce to a quality result. Test pitting in the borrow areas and stripping of the quarry area showed that the weathering profile was quite
variable, with high points of less weathered rock within the residual soil. More accessible deposits were excavated with scrapers working under the site supervision of a Geotechnical Engineer to separate zone 1A and 1C material at the borrow pit and direct these materials to different stockpiles. Smaller deposits were worked with a bulldozer to push up stockpiles then excavator and truck to haul materials to stockpiles. Despite the efforts at selective extraction of the material, some less weathered rock was collected with the zone 1A material.

The material passed through a rigorous testing and classification process before being screened over a mobile screen to remove material greater than 50mm (Figure 4). This screening was shown to both remove large rock and to break up large clods of clay to facilitate blending and moisture conditioning.

The zone 1C material was largely won by screening fines from the overburden stripped from construction areas, particularly the quarry area, after removal of topsoil. The overburden consists of the completely to moderately weathered rock. The screening process produced a range of useful site construction materials including large rock suitable for erosion protection, a 75mm to 6mm product suitable for road surfacing and a 6mm minus screened product which was used as zone 1C. The zone 1C material was also won from the lower levels of the zone 1A borrow pits as the rock become less weathered. Using these methods the zone 1C material was quite economical to win.

Moisture Conditioning by Continuous Mixer

Typically moisture conditioning includes watering layers in the borrow area and multiple passes with an elevator scraper (or similar) to ensure the clay material is completely blended and has a uniform moisture content. The material is then stockpiled for several weeks to ensure uniform condition before placement in the permanent works. With space limitations and an ambitious construction program, the team conducted a robust trial process using a pugmill; a mobile continuous mixer as an alternative conditioning process.

Several loads of clay material were hauled over 400km to the nearest available pugmill to confirm the process would achieve the desired results and identify the parameters necessary for success. The material was then hauled back to site and two trial embankments were constructed with both the pugmill mixed clays (placed within 4 hours of mixing) and material conditioned using traditional methods (stockpiled for 2 weeks). The pugmill was able to evenly condition these lower fines content clays and demonstrated no reduction in the quality of the material from test results and observations of the trial embankments. This ultimately meant the clay could be conditioned and placed directly into the permanent works with no requirement to cure the clay in stockpile to achieve uniformity. The fact that this innovative process was accepted, in spite of it being extremely uncommon, demonstrates the rigour with which the trial was conducted and the collaborative efforts of the Alliance project team.

Figure 4 - Clay stockpiles being screened, classified and then moisture conditioned on demand using a continuous mixer
As the borrow operations moved to higher plasticity deposits, the load on the mixing paddles within the mixing chamber of the pugmill increased. It was ultimately found that the pugmill could not handle the site’s highest plasticity clays, and traditional conditioning and stockpiling methods were required for these materials.

**Granular Filters**

**Optimisation of Filter Widths**

The filters for the embankment consisted of a double 2A / 2B filter downstream before transitioning to a 3A fine rockfill. Upstream a single 2C filter zone was used before the 3A rockfill.

The imported sand and gravel filters were a large portion of the overall cost of the two saddle dams. Through a series of workshops the individual filter widths were reduced from 2.0m to 1.0m, which is considerably narrower than convention. The design team was required to submit the proposed reduction to the Dam Safety Regulator and the Technical Review Panel and prove the filters would still perform their critical function within the overall dam performance. Filter placement methodologies were developed to ensure that the 1.0m minimum thickness could be ensured.

**Filter Placement Methodology**

With the filter material reduced to an unusually narrow width, it was now up to the construction team to develop an efficient methodology of placing the materials. Three different placement methodologies were used successfully in different locations.

Near foundations and at the end of each lift in the abutments, where the other methods were not feasible, the 2A filter was overplaced with a small wheeled loader to fill the entire zone between the core zone and foundation or downstream blanket filter.

For confined areas, the filters were placed using 1m wide trench boxes which were lifted into place and checked by survey. A double box for placement of 2A and 2B material was developed for the downstream filters (Figure 5) and a single box upstream. The boxes were useful both as confinement for the filter to minimise material losses through the ‘christmas tree’ effect of normal placement methods, and as formwork for the core to ensure good compaction. Filter material was placed in a single lift in the 600mm high boxes and flooded with water then the surface compacted with a 60kg vibrating plate compactor. This was demonstrated to provide adequate compaction with a density index of approximately 0.7. After a 600mm lift of core (4 x 150mm layers), filter and rockfill, the boxes were lifted with an excavator and surveyed into location to start the next lift. The minor disturbance caused by lifting the boxes was repaired by running over the junction at the edges of the filter zones with a plate compactor.

Once the working level had become less confined, the regular relocation of the trench boxes was found to limit construction productivity. The team developed a continuous placement methodology using a customised modification to an ejector moxy truck involving a twin spreading chute on the tailgate (Figure 6). The alignment of the filter was set out by a surveyor, then the trained operator would drive carefully along this line spreading the 2A and 2B filter material at a controlled rate. A D4 bulldozer was fitted with a spreading blade to push the filter piles into a 300mm high lift. Because of the reduced certainty in filter location, the target width for filter zones
using this alternative method was 1.5m to ensure the minimum 1.0m was maintained. The increased filter quantities were significantly outweighed by the production efficiencies as it took the filter placement off the critical path and returned the core placement and compaction process to the governing task in the layer cycle.

Filter Materials

No natural sand or gravel deposits were identified on site for suitable filter materials, and the site crusher was fully utilized producing aggregate for the main dam, hence, filter materials had to be imported. Commercial aggregate suppliers were invited to provide tenders for the supply of the three filter products. The specification provided to the suppliers was a tight grading limit (ratio between maximum and minimum sizes was about 2.5) and a maximum fines content as tested in the suppliers stockpile of 2%. This low fines content was specified to ensure that after transport, handling and placement, the fines content on the bank would be less than 5%

The commercial suppliers were not willing to modify their crushing equipment to produce these relatively small volumes of specialised gradings which would disrupt their regular supply of aggregates for concrete and road purposes. Instead, they proposed some modifications to the design gradations to suit what could be economically produced by blending or screening available products. Several trial blends and revisions to the filter design occurred to optimize the filter materials. The main outcomes of this were:

- All products were crushed from local igneous rocks. A natural sand deposit was considered for the 2A filter, however, local sands had a relatively high clay content which is difficult to wash out.
- Only one supplier was able to produce 2A material. They had no problem meeting the 2% fines limit or tight filter band. The fine limit of the 2A filter was made coarser based on their submitted grading to allow a coarser 2B filter.

Construction activities focused on avoiding contamination of filters both in stockpiles and in the embankment. The foremen and quality staff would regularly inspect the filters for contamination and two labourers with shovels removed contaminated material from every lift. This manual removal process was a basic and effective means of managing contamination rather than trying to prevent it entirely during bulk filter placement. The construction quality control included stringent monitoring and testing of filter properties. This confirmed a very regular product from the supplier and placed on the bank with only 4% of filter properties. This confirmed a very regular product both in stockpiles and in the embankment. The foremen and quality staff would regularly inspect the filters both in stockpiles and in the embankment. The foremen and quality staff would regularly inspect the filters both in stockpiles and in the embankment.

Foundation Treatment

Foundation Excavation

The foundations of the saddle dams were highly variable in weathering and jointing. It was not considered feasible to excavate to a sound, tight foundation. The target foundations were moderately weathered rock or better in the core trench and highly weathered rock or better in the shoulder foundations, but noting that seams of completely weathered rock would still be present within the foundation. These were to be chased out where possible.

The general process for foundation excavation was:

- After removing topsoil, scrapers were used to remove residual soils for potential core material, this was typically only 300mm to 700mm deep.
- Bulk excavation was done with a large D11 dozer. The shoulder foundations were defined as blade refusal while a single tyne was used to rip as deep as possible in the core trench.
- After bulk cleanout, the foundation was inspected by the site geological team to direct where further bulk excavation was required.
- Additional bulk excavation was carried out with a toothed bucket, rock pick or hydraulic rock hammer on an excavator. Better quality rock that
surrounded isolated areas of weak material was typically also removed to achieve a core trench that freely drained to a single low point;

- The foundation was blown down with compressed air and water as a first stage clean up. The traditional hand blow-pipes were considered an unacceptable safety risk for operators, instead an excavator was modified with an air hose on a rock pick and connected to a 900CFM high pressure compressor to perform this clean-down.
- Following this clean-down the foundation was again inspected to identify further areas for bulk excavation and weak seam treatment.
- Hand excavation using hand picks, shovels, crowbars and wash down with high pressure washers was then carried out to reach the final foundation. This was a slow task for a team of labourers. To improve efficiency, a vacuum truck was used at key times during hand clean-down to remove loose rock and water from the low lying areas of the core trench.

A Foundation Inspection Committee (FIC) was formed to provide overall approval of the foundation. This consisted of a Principal Geologist, Principal Engineer and Construction Manager along with the site geological team. The intention of the FIC was to provide guidance to the site geological team on what was an acceptable foundation and methods for dealing with difficult foundation features. The FIC met approximately monthly during the foundation excavation process.

A difficult feature of the foundations was the presence of a predominant near horizontal joint set with highly to completely weathered material on the joints (Figure 8). This was difficult to identify during bulk excavation as machinery would refuse on the overlying hard rock and it was not until the foundation was cleaned out that the weak zone beneath could be seen. This usually resulted in much frustration as excavators and other heavy machinery were brought back to the location to break out the sound rock and remove the underlying weak material. Often this required removal of a large quantity of sound rock as the joint set was chased into the abutment with little improvement in rock quality. Typical joint treatment rules such as cleaning them out to twice their width did not work as it would not be possible to backfill the horizontal joint with grout and ensure it was fully sealed against the roof of the joint.

The foundation excavation process took considerable time and numerous iterations of clean down, inspection and further excavation. The Construction Team held a workshop after completion of the first saddle dam foundation to improve the process for the second foundation. This improved process consisted of:

- An ongoing front from one end of the abutment to the low point in the foundation.
- After bulk excavation, providing sufficient clean down with the excavator mounted airhose to properly inspect the foundation. No hand clean-down during this stage.
- General mindset that it is more efficient to remove rock early rather than late – ‘if in doubt, take it out’ during bulk excavation.
- When the site geology team accepted the bulk excavation a red line was painted at the extent of approved foundation. This red line meant that hand cleanup could commence.
- After hand cleanup was completed and approved, a green line was painted on the foundation identifying the extent of completed foundation. Hence, the hand-cleanup crew knew to only work between the red and green lines.

This process was generally effective in eliminating or at least minimising the repeated iterations of hand-cleanup and machine excavation and gave site foremen and labourers a more clear understanding of the requirements. It did result in more material being removed from the foundation and consequent additional materials for construction.

Another observation of the first saddle dam construction was that localised hollows regularly filled with water and needed to be pumped out and cleaned. To avoid this time-consuming task, a considerably greater volume of dental concrete was used in the second dam foundation to ensure the abutments would drain to the central low point.

Overall, the Construction Team believed this approach was an improvement and the efficiencies outweighed the additional material costs.

### Shotcreting of Core Trench Foundation

As the clay core material was dispersive, there was a risk of the core material eroding into open joints in the foundation which could lead to piping failure. To minimise this risk, the entire foundation for the core zone of the saddle dams was coated with steel fibre reinforced

---

**Figure 9** - *The entire core trench was shotcreted, and smooth finished with a steel hand trowel*
shotcrete after approval of the cleaned foundation (Figure 9). The shotcrete was hand finished with a steel trowel to remove any potential seepage path through the open ‘off-gun’ finish. Shotcreting the entire core foundation is unusual for embankment dams but was considered necessary here due to the combination of an extreme hazard dam, jointed foundation and dispersive core materials.

GIN Grouting

The rock foundations were grouted with a single line grout curtain, typically 30m deep, to reduce subsurface seepage through the foundation. The curtain comprised of vertical holes at maximum six metres centres. The target permeability value was 6 lugeon or below and where necessary, additional grout holes were installed at three metres or even one and a half metre centres.

It was decided to use the state-of-the-art GIN (Grout Injection Number) grouting method for the saddle dam foundations. GIN grouting is a computer controlled and recorded method which aims to maximise the efficiency of the grouting process and minimise damage to the foundation rock. The method uses a single stable grout mix and controls the volume of grout injected and pressure at which it is pumped. The process was developed in Europe and is a relatively new technology in Australia.

The concept of GIN was first proposed by Lombardi and Deere (1993). The grout pressure that can be safely used without jacking or damaging the formation is directly related to the area of defect surface in contact with fresh grout. While estimating the size of the area affected by the grout at any particular time is not possible, Lombardi and Deere proposed that it would roughly correlate to the quantity of grout injected. Thus, the GIN was set as the product of the injected volume and the grout pressure at a given time.

A GIN curve defining the maximum combination of pressure and volume of grout injected for a range of depths was developed as shown in Figure 10. A computerised system controlled the grouting process and ensured the maximum pressure or volume product were not exceeded based on the stop criteria in Table 2.

<table>
<thead>
<tr>
<th>Table 2 – GIN Grouting Stop Criteria</th>
</tr>
</thead>
<tbody>
<tr>
<td>At limit on GIN curve</td>
</tr>
<tr>
<td>On reaching ( P_{\text{max}} )</td>
</tr>
<tr>
<td>On intersecting the GIN curve</td>
</tr>
<tr>
<td>On reaching ( V_{\text{max}} )</td>
</tr>
</tbody>
</table>

The core trench foundations were expected to contain closely fractured zones and there was concern that the grout packers may not be able to adequately seal to apply pressure for grouting. For this reason, a grout cap was included in the design; this was seen as a complex and expensive construction operation. A grouting trial was carried out during the pre-construction phase to test the ability to grout the moderately weathered rock foundations. The trial holes were water pressure tested up to a pressure of 1 bar, and then grouted at pressures up to 4 bar. The packers were able to sustain these high pressures indicating that a grout cap was not necessary to seal the fractured rock.

Conclusions

The integrated project team worked effectively from start to finish delivering value for money without compromising the design intent. Extensive testing and analysis was required of the onsite clay borrow pits to prove the material was suitable for the saddle dam core material. Ultimately the design was adjusted into a zoned core with the best material being used lowest in the core where the hydraulic pressures are highest. Assumptions continued to be challenged as the team established a means of conditioning the clay core material with a mobile continuous mixer. An innovative process that meant the clay could be conditioned and placed on demand without the need to cure it in stockpile for two weeks. A modified filter placement moxy truck, a mechanised foundation preparation process, GIN foundation grouting and fibre reinforced shotcreted foundation are amongst the other engineering solutions that resulted in the saddle dams reaching completion in February 2011 some 5 months ahead of the required program and 10% under budget.

The process of adapting the design to suit the available onsite materials pushed the boundaries of the embankment dam design and maximized the flexible nature of an alliance framework.

Acknowledgements

The authors would like to acknowledge the many people who provided a positive contribution to the project through the Alliance framework and ACTEW/AGL for encouraging presentation of this paper.