

Hydrogeology of the Ross River Dam

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ABSTRACT

Basic pre-construction foundation investigations for the Ross River Dam were done in the late '60s to early '70s but a more detailed hydrogeological assessment was carried out to investigate and manage waterlogging and salinity, which developed immediately downstream in the late 1970s.

As part of the 2005 Stage 2 to 5 upgrade design, detailed conceptual and numerical hydrogeological modelling was required to predict aquifer response along the embankment and downstream. This required "data mining" and additional drilling and aquifer testing to fill in data gaps, with the filtered and re-interpreted data used to build a 3D conceptual model of the embankment and underlying geology, by a design team comprising specialist hydrogeologists, geologists, geotechnical and dams engineers. This was converted to a 10-layer, 2-million cell numerical model, to enable high-resolution modelling of groundwater behaviour for a range of aquifer properties, flood hydrographs and seepage management options. As well as a design tool, the model is a valuable monitoring tool in confirming the performance of seepage management systems and to provide early warning of seepage management failures.

The study emphasised the need to capture data for a wide range in aquifer stress, to have simple preliminary spreadsheet models to provide a "sanity check" and to collect data away from the embankment to allow a 3D interpretation of the geology, to the assumption of "layer cake" models.

1 Introduction

Ross River Dam, located 20 km south-west of Townsville, was designed by the Queensland State Government, which also oversaw its construction during the 1970s. The dam's two primary purposes are flood mitigation and water storage. The embankment was raised in 1976 to its current crest level.

This paper describes the hydrogeology of the Ross River Dam, and details the limitations of the geological dataset utilised to model the system. The process of evaluation of existing data, or "data mining" is described in the context of developing a basic predictive model for groundwater levels at the toe of the dam.

A detailed description of the geology of the Ross River Dam is provided in the accompanying ANCOLD 2005 paper by Forster & Laxman, "Characterisation of the Ross River Dam Foundations".

2 Background

Following the first filling of the dam to the full supply level in 1974, a large area of waterlogged, and subsequently salt-scalded ground, developed downstream of the embankment between chainage 1000 m and 2 600 m. Investigations carried out from 1980 to 1982 concluded the waterlogging was caused by pressurisation of the more permeable materials within the sedimentary sequence upon which the embankment was constructed.

As a result of the pressurization of the upper and lower aquifers, dominated by reportedly loose or free sands (which have been found to be medium dense to dense sands during the 2004 investigations), a series of pressure relief bores were installed along the toe of the embankment during 1983 to 1984. Groundwater measurements indicate that these relief bores have successfully reduced the artesian head to sub-artesian conditions.

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3 Risk Model

The validation of the design of the Ross River Dam Upgrade has been accomplished using risk based design methods (Barker et al, ANCOLD 2005, *Use of Risk Based Design Validation for Justification of the Ross River Dam Upgrades*). For this validation, the three significant failure mechanisms proposed for the embankment portion of the Ross River Dam were as follows:

- Failure from overtopping.
- Failure from piping through the dam.
- Failure from piping or blowout of foundation sands.

The assessment of groundwater data and development of the hydrogeological model provided critical data for the evaluation of mechanisms for failure involving piping or blowout of the foundation soils.

4 Aquifers Under the Dam

The alluvial sediments of the Townsville coastal plain constitute a regionally significant aquifer, although the area is not classified as a declared sub-artesian area under the Queensland Water Act 2000. The most productive zones are those with deeper basement (i.e. a thicker cover of sediment) allowing greater development of the basal Pleistocene sand and gravel. A detailed discussion of aquifer hydraulic properties is presented herein. The general, idealised hydro-stratigraphic sequence comprises, from the surface down:

- *Surficial clays* with isolated areas of shallow un-cemented sands;
- *Upper aquifer*, comprising structured/fissured clays, cemented sands and basal free or un-cemented sands;
- *Mottled clay*, with minor isolated cemented sand lenses; and
- *Lower aquifer*, comprising clayey to free sands and gravels overlying weathered

and fractured bedrock granite and andesite.

Not all layers are present in all areas of the coastal plain or the Ross and Bohle River Valleys. In some areas, the upper and lower aquifers are in contact or indistinguishable (mottled clay absent). In areas of elevated bedrock, the upper aquifer directly overlies bedrock, indicating an on-lap style of sedimentation. Of significance to embankment stability, some areas of un-cemented sand are present either at or near surface, however a large proportion of the sandy materials have undergone some cementation.

Upper Aquifer Sands

The upper aquifer sands comprise weakly cemented and clayey to free (uncemented) sands and gravels. With the exception of the shallow free sands noted below, the free sands are primarily at the base of the sequence and are generally poorly sorted, but are significantly permeable.

Cemented Sands

Cemented or indurated sands and sandy clays are present at the top, within, or at the base of the structured clays, and are not considered permeable. QWRC (1982) noted that when drilled using augers, disturbed cemented sands appear as sandy clays and careful logging of the materials was required to ensure correct classification for use in the hydraulic modelling.

Structured Clays

The structured clays have moderate secondary permeability through fissures, as indicated by seepage observed in deep test pits constructed by QWRC (1982) and GHD (2004). It was noted that seepage primarily occurred along manganese-stained fissures, and that no water appeared to be seeping through the carbonate veins and pipes identified during the QWRC investigation of the early 1980s.

Shallow Free Sands

The free sands, which comprise part of the “shallow sands and gravels” and “shallow clayey sands and gravels” are the most

permeable of the materials in the upper aquifer, and occur as a series of separate bodies. The basal sands and gravels have similarly high hydraulic conductivity.

5 Data Mining

Historical Data Sources

Testing was carried out as part of the pre-construction and seepage investigations. A total of 29 in-situ falling/rising head tests, along with some grainsize analyses, were carried out by Ground Test Australia (1968) to assess the permeability of individual soil types.

QWRC (1982) completed a series of in-situ falling head and pumping tests to obtain transmissivity and storativity data for the overall aquifer as well as that of the structured clays. In excess of 220 boreholes were constructed for this testing, which was state-of-the-art in 1982. Review of this data as part of the current study confirmed the high quality of the work undertaken by QWRC.

Additional testing was undertaken by Australasian Groundwater & Environmental Consultants Pty Ltd (2003) but the results were of limited reliability due to pumping rate restrictions.

In addition to the test data above, groundwater records have been collected intermittently over the past 25 years as part of NQ Water's ongoing monitoring. A typical plot of groundwater level and corresponding reservoir level with time is shown in Figure 1.

This figure shows some key characteristics of the groundwater measurements at the Ross River Dam. The ground level at the bore is shown on the graph at 32.4 mAHD. The period of artesian pressures can be seen in the graph for readings recorded prior to early 1980. This bore is located near the trial relief bore system installed by QWRC at the end of 1980. The permanent system was commissioned in 1984 and the effect of this system can be seen in the depressed groundwater readings since 1984, even with the increase in reservoir storage level from 1989.

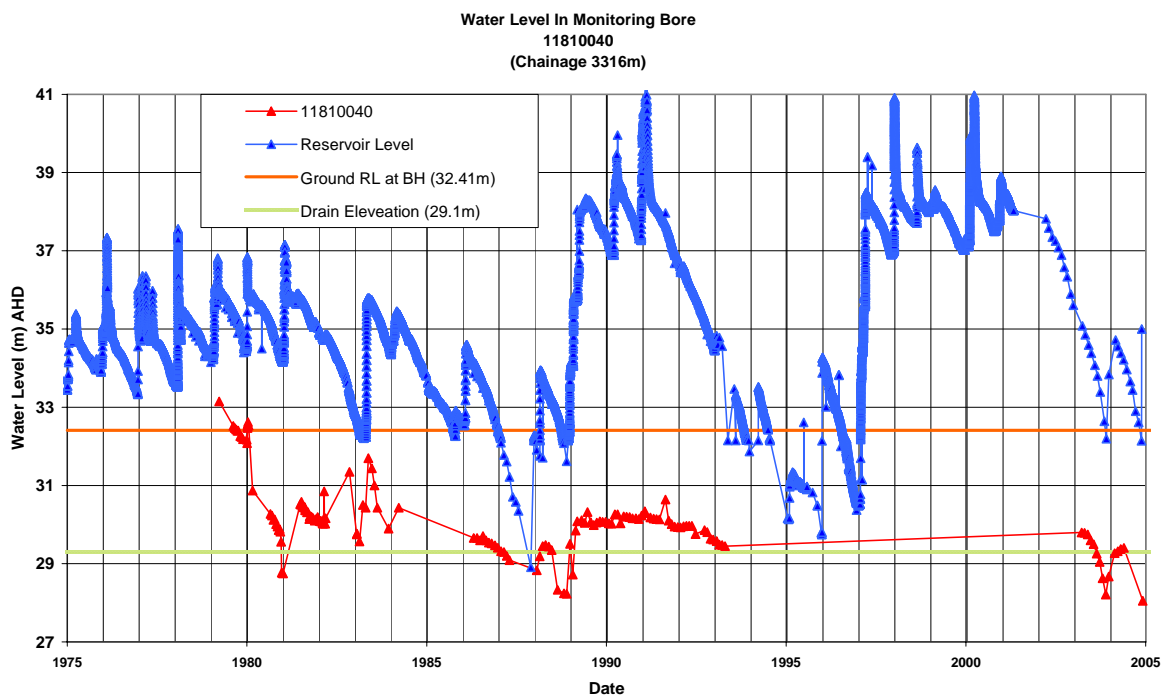


Figure 1 Typical Groundwater Level Record with Time

The layout of the relief bore relative to the downstream toe of the embankment is shown in Figure 2. This shows the drainage of the bore into the seepage drain pipe via a Tee

flowing into the top of the pipe. The seepage drain pipe runs from Ch 6200 discharging to the Ross River at approximately Ch 300 and has an invert approximately 4 m below ground

surface. The relief bores only remove water above the elevation of the Tee as flow into the seepage drain pipe occurs under gravity, thereby limiting the amount of depressurisation

that the bores can provide. This is shown as the green line at RL 29.1 on the plot of level with time in Figure 1.

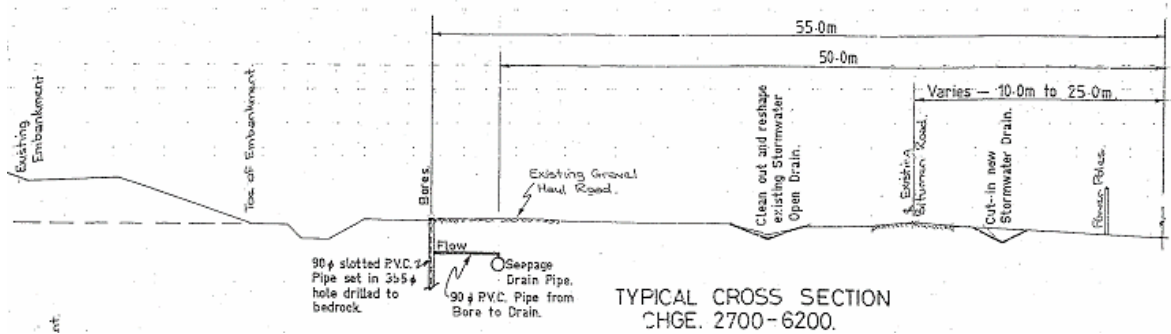


Figure 2 Typical Relief Bore Detail

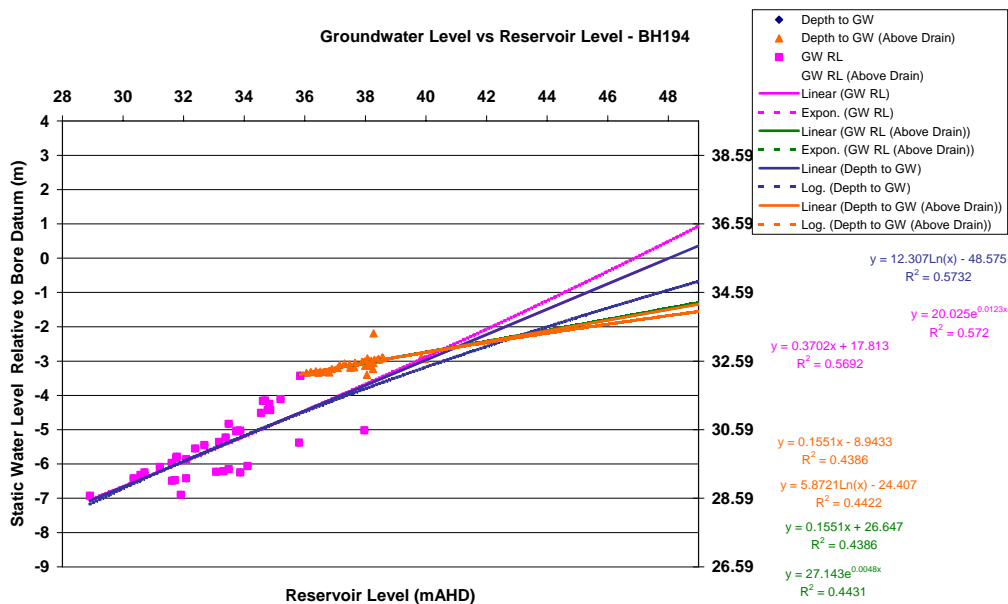


Figure 3 Plot of Groundwater Level versus Reservoir Level

Figure 3 shows the typical variation in slope of the fitted curve for groundwater levels above and below the Tee invert. For the borehole data shown above, failure to differentiate the different response in the groundwater levels above and below the drainage level of the relief bore, results in an overestimate of extrapolated groundwater levels of up to 2.5 m at the PMF reservoir level of RL 48.5 m. For monitoring bores a significant distance away from relief bores, this effect is less pronounced.

Accordingly, these plots also indicate the efficiency of the relief bores on groundwater response.

An assessment of the expected (predictive) groundwater levels at the toe of the dam can be determined using the data fit curves (shown above) when assessed for every groundwater bore for which data is available. Combining all of the available best-fit curves provided the predicted groundwater levels for various reservoir water levels, as shown in Figure 4.

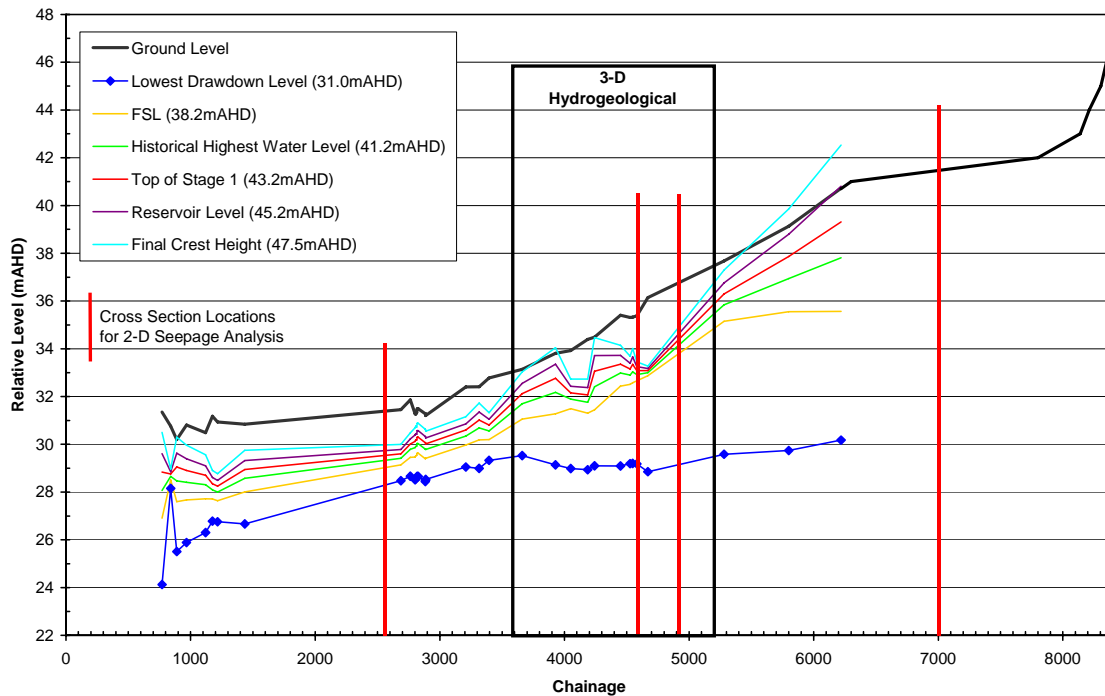


Figure 4 Predicted Groundwater Levels as a Function of Reservoir Level

Figure 4 also shows areas where 2D SEEP/W models and 3D GMS MODFLOW hydrogeological models were developed to assess groundwater response to changing reservoir levels and upgrade strategies. This predicted groundwater plot allowed a cross check against the theoretical models developed.

Hydraulic Conductivities

The existing data from the QWRC investigations was used as a basis for the 2004 modelling, supplemented by additional field testing to verify the original data and provide new data in areas where data gaps existed.

In-situ rising and falling head testing was completed on selected monitoring bores constructed as part of the 2004 investigation. Bores with sufficient water above the top of the screen were tested using pneumatically displaced rising head tests. Selected additional bores were tested with falling head tests.

Methodology of Hydraulic Testing

Slug testing was utilised to enable determination of aquifer properties, specifically hydraulic conductivity, for the Ross River Dam hydrogeological study. Slug tests are often identified as being the most accurate method to determine the permeability of an aquifer when conducted carefully and interpreted with consideration of site hydrogeological conditions (Hamilton and Li 2003).

Prior to undertaking the slug testing, hydrographs for those monitoring bores selected for testing were assessed to determine whether they were providing water levels representative of expected aquifer conditions. Groundwater monitoring bores that had not been used (sampled/developed) in the previous 12 months, or displayed suspicious water level trends, were developed by air lift prior to testing.

Development removed a number of other materials from the bores including algal/bacterial buildup and organic matter (including mice, frogs and cane toads).

Hydraulic Testing

Two different slug testing methods were utilised at Ross River Dam: conventional slug testing and pneumatic slug testing. Pneumatic slug testing was performed on a total of 43 bores, covering 4 different water bearing units (dominated by sandy horizons). Bores with water levels within the screen can not be tested pneumatically as air escapes through the screen preventing pressurisation. Fourteen bores were tested conventionally with water slug.

Where a number of sand lenses/aquifers were screened at the one location, at least one day was given between testing of the aquifers to ensure that there was no inter-aquifer interference.

The 'conventional' slug test involves the rapid addition of a slug of water to the bore, with the rate of fall in water level measured. A 10 litre slug of water was utilised as this was considered large enough to provide a measurable water level change (10 L relates to approx 5 m rise in 50 mm bore), while being small enough to be handled.

A Greenspan PS2100 pressure transducer and temperature data logger was used to record the change in water level in the bore. The recorded data was downloaded to a laptop computer in the field and plotted to ensure the pressure transducer had functioned correctly and the data appeared to resemble expected recovery. If the results were not as expected, the slug test was re-run at a later date, allowing the aquifer to re-equilibrate prior to the second test.

The water level in the bore was recorded manually with an electronic probe prior to and immediately following the test to provide data for comparison and calibration purposes.

Review of the data from the conventional slug testing identified a number of issues. Making an 'instantaneous' water level change in an aquifer with a conventional 'water' slug is not easy. In addition, measurement of accurate water level change in high permeability aquifers, particularly early-time response, is difficult with conventional slug tests as aquifer response is typically very rapid (Hamilton and Li 2003). While the use of the pressure

transducer and data logger can overcome this problem in theory, disruption to pressure records during the insertion or removal of the slug, particularly in the narrow diameter bore, can cause inaccuracies in readings.

As a result, pneumatic slug testing was favoured over conventional slug testing. This was also supported by previous investigations that had identified sandy materials at the site, which inherently have high permeabilities.

Pneumatic slug tests require the water level in the bore to be depressed through the application of compressed air to the bore above the water level, effectively pushing the water level downwards. This was achieved using an air compressor and a custom built 'Christmas Tree' (constructed of UPVC pressure pipes and fittings). A petrol powered 30 cubic feet per minute (cfm) air compressor was used to depress the water level in the bore being tested. The Christmas tree used is pictured in Figure 5. The Christmas Tree has a series of valves and pressure gauges allowing the pressure in the hole to be regulated and released instantaneously.

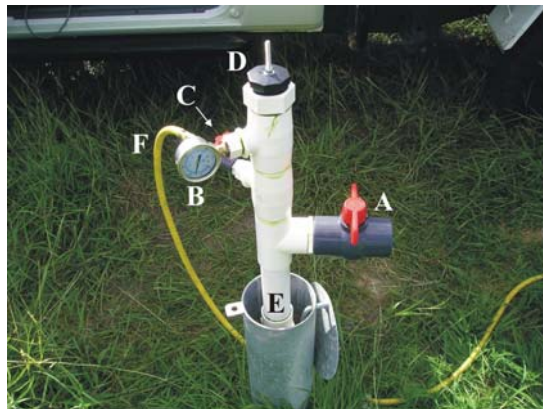


Figure 5 Christmas Tree. A) Instant pressure relief valve, B) Pressure gauge (See Figure 6), C) Pressure control valve, D) Data logger access port, E) Threaded connection to bore, & F) Compressed air supply hose.



Figure 6 Pressure gauge allowing depth of depression to be set using ball valves (red handle in lower right).

Once an airtight seal had been achieved, the bore was pressurised (1 kPa is equivalent to approximately 102 mm water level change (AS 2368-1990)), effectively forcing the water table downwards. The desired extent of water table decrease was calculated for each bore prior to the test being conducted to ensure that the water level was not allowed to fall below the top of the screen. This ensured that the aquifer was not aerated with compressed air, which could alter the aquifer's hydraulic properties. A pressure of between 10 and 60 kPa was applied to the bores, with the majority being pressurised to 50 kPa. The system was pressurised and held steady for approximately 2 minutes prior to the commencement of recording for each test. This provided a marker to check that the water level had stabilized at the desired level prior to the release of the pressure.

Pressure was released from the system by opening the 50 mm diameter ball valve, effectively providing an instantaneous release of stored pressure. This allowed the water table to recover (i.e. the slug test). This recovery was measured with a Greenspan PS2100 pressure

transducer and temperature data logger, which was suspended from the Christmas Tree, inside the bore casing. Water level recovery was logged for approximately 30 minutes to ensure full recovery was achieved. As for the conventional slug testing, the recorded data was downloaded to a laptop onsite to ensure data quality.

In general, the pneumatic slug testing was successful, although the overall population of slug-tested hydraulic conductivities for the aquifers were up to an order of magnitude lower than results derived from long-term constant rate pumping tests. This is probably partly due to skin effects due to smearing of clays during drilling of the bores, and the fact that by their nature, constant rate pumping tests will only be carried out in highly permeable materials. In order to minimise the effects of smearing, bores must be thoroughly developed using the relatively aggressive means noted above, often for times in the order of an hour or more, which is significantly greater than the standard purge requirement of between 3 & 5 casing volumes, over a few minutes, sometimes erroneously considered to equate to successful development.

Foundation Hydraulic Conductivity Distribution by Lithological Type

Based on the results of the Ground-Test (1968), QWRC (1982) and GHD's 2004 data, the relationship between hydraulic conductivity and lithology was assessed.

The data indicate the highest hydraulic conductivities were recorded in the lower aquifer sands although the upper sands were of a similar order.

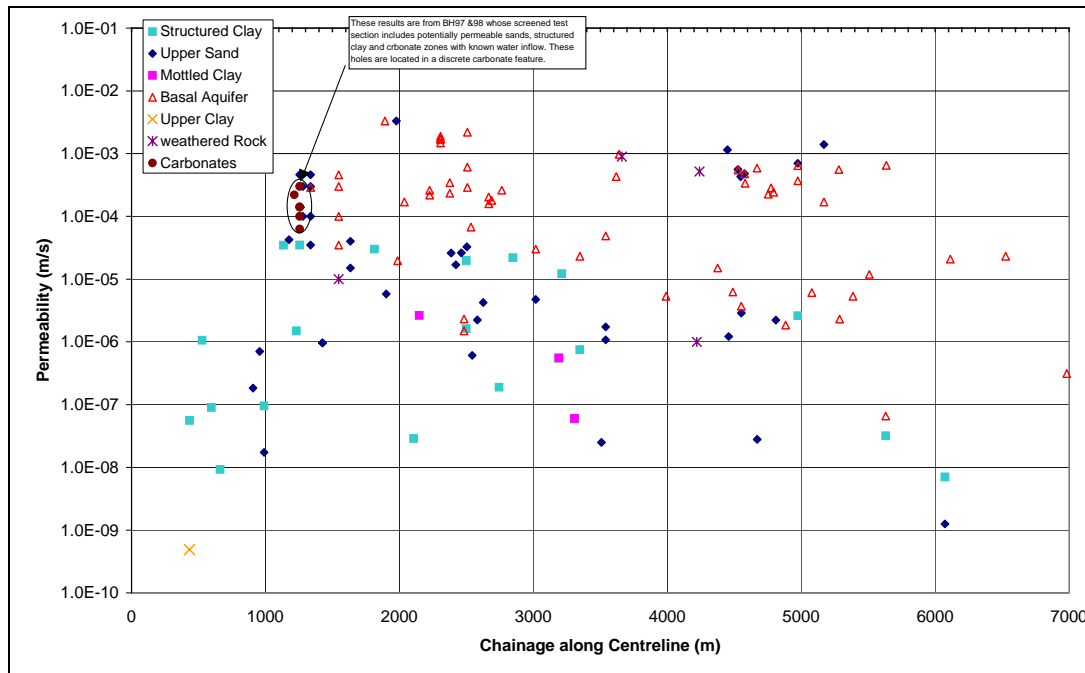


Figure 7 Measured Permeabilities by Chainage and Soil Type

Figure 7, which is the variation in permeabilities for each soil type plotted by chainage shows:

- the structured clay having a permeability of between 3×10^{-5} to 1×10^{-8} m/s,
- the (upper) sands having a permeability of 2×10^{-3} to 1×10^{-9} m/s (reflecting the varying amounts of clays within the sands),
- the mottled clay having a permeability of 3×10^{-6} to 5×10^{-8} m/s, and
- the basal sands and gravels having a large range from 3×10^{-3} to 5×10^{-8} m/s.

Storativity

Storativity values were calculated by QWRC (1982) from the results of several pumping tests where water levels were recorded in observation bores as well as the pumped bore. The results indicate the aquifer is partially

confined as the higher values are consistent with values for unconfined aquifers but the lower values are consistent with confined, partially consolidated aquifers.

Statistic	Storativity
Arithmetic Mean	0.01
Geometric Mean	0.0001
Maximum	0.06
Minimum	2×10^{-4}

Table 1 Foundation Storativity Values

6 Hydrostratigraphy

The hydrostratigraphy near Chainage 3600 m to 5200 m is summarised in Table 2.

Model Layer	Unit	Description	Regional Average Thickness (m)
1	Surface Soils	High plasticity clays. Isolated pockets and channels of surficial channel sands. Borrow areas upstream from dam wall, exposing underlying horizons. Dam embankment.	~5 m except embankment areas
2, 3 & 4	Structured Clays	Structured clays (2 & 4) and local indurated sands (3). Three layers were required to allow indurated sands to be at the top, middle or base of structured clays.	5-10 m
5 & 6	Shallow Sands	Variable well-sorted gravels, sands, clayey sands to sandy clays. Tends to be fining upwards with clayey sands at top.	5-15 m
7	Mottled Clay	Mottled clays separating upper and lower aquifers	5-10m
8 & 9	Basal Sands	Variable well-sorted gravels, sands, clayey sands to sandy clays. Tends to be fining upwards with clayey sands at top.	5-10 m
10	Weathered bedrock	Decomposed/extremely weathered and fractured bedrock granite and andesite/rhyolite volcanics	5-10

Table 2 Hydrostratigraphy of Conceptual Model

7 Hydrogeological Modelling

Modelling Challenges

The geology of the Ross River Dam was defined through the sedimentological interpretation and basin analysis of all the boreholes located along, and close to, the embankment. Although a large number of boreholes (several hundred) and test pits have been completed along the dam embankment, there were only three areas where sections normal to the axis of the embankment could be constructed. Consequently, a traditional three-dimensional numerical model could not be developed for the entire embankment and reservoir areas. Although the problem should, ideally, have been modelled in 3D, the nature of the data resulted in the application of several 2D models, constructed at locations along the

embankment where sufficient data existed to allow accurate geological interpretation of the sub-strata.

Seepage calculations were completed at these locations using SEEP/W, with the models being calibrated against groundwater monitoring data (where available), reservoir levels (hydrographs) and reported scalding/waterlogging downstream.

Following the completion of additional drilling in the vicinity of chainage 4 500 m, a small area (about 1 km²) could be modelled in 3D, allowing the 2D models to be checked and adjusted as required.

As the embankment was constructed along the natural/existing levee of the Ross River, buried channels cross the embankment at an acute

angle, neither normal to or parallel to the alignment of the embankment (Figure 8).

As a result, 2D and 3D models had to be constructed to allow cross drainage/seepage to be considered.

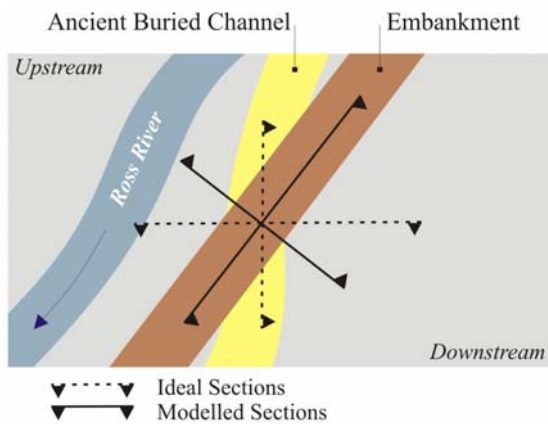


Figure 8 Orientation of Ideal model alignments versus those able to be used.

Model Data

Data collated for the model included:

- ▶ Spot elevations derived from contours of the area upstream from the embankment, and borehole collar heights, a series of traverses downstream from the embankment and surveys of ground surface at bore locations. The topography of the predictive models was modified to reflect the proposed final embankment height;
- ▶ Stratigraphic horizon elevations interpreted from bore logs.
- ▶ Groundwater level data corresponding to a reservoir level of 38.2 mAHD;
- ▶ Hydraulic conductivity, storativity and porosity data derived from previous and current studies;
- ▶ Depressurisation bore locations; and
- ▶ Black & White and Colour aerial photography, flown in 1960s and 1999.

Data Storage

Spatial data was loaded in to the US Army Corps of Engineers (USACE) Groundwater Modelling System (GMS). Data included:

- ▶ Bore files with simplified bore logs with lithological horizons assigned;
- ▶ Material files for each of the stratigraphic horizons comprising;
 - Horizontal and vertical hydraulic conductivity (K_h , K_v)
 - Specific storage;
 - Specific yield;
- ▶ Background image: registered aerial photomosaic;
- ▶ Groundwater level observation points with static water levels corresponding to reservoir level 38.2 mAHD;
- ▶ MODFLOW Drain cells representing existing depressurisation bores with head levels set at the obvert of the collection pipe, and conductance set at above highest foundation permeability so that inflow is controlled by the surrounding aquifer rather than the drain cell;
- ▶ Aerial distribution (polygons) of variable hydraulic conductivity, primarily in the area of the river channel;
- ▶ Polygons of MODFLOW boundary conditions including;
 - No Flow boundaries along regional flow lines;
 - General Head boundaries within the reservoir, with heads varied according to reservoir levels and conductance set according to surface sediment types; and
 - Constant Head boundaries set at regional groundwater level (29 mAHD) representing the northwestern limit of the model. Although water levels in this location vary by up to 0.2 m, the

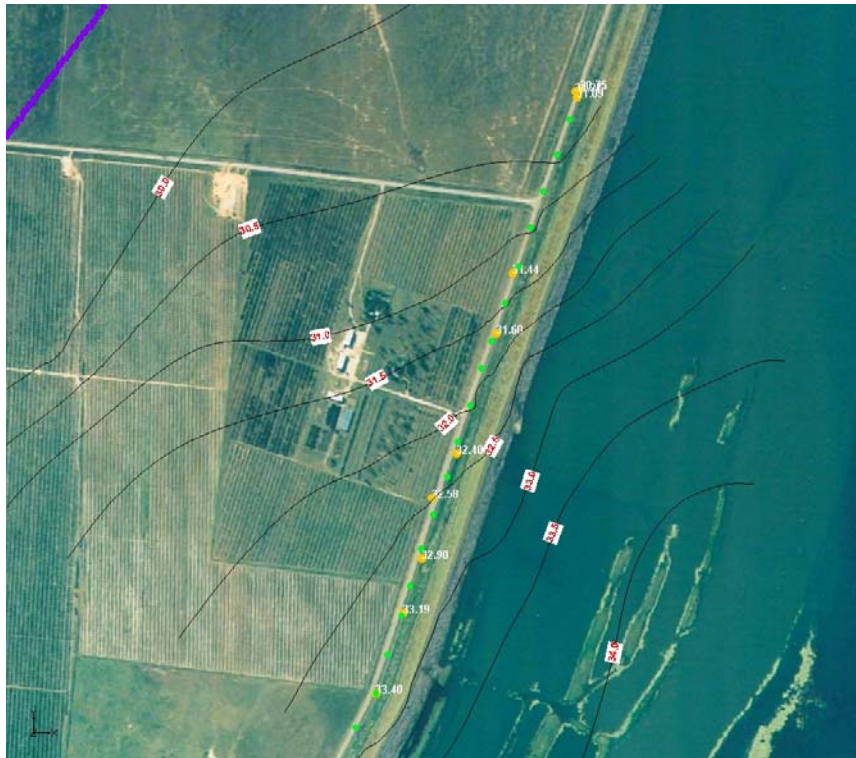


Figure 11 Calibrated Baseline Model RRD11_382– Contoured Predicted SWL with Observed SWL (mAHD)

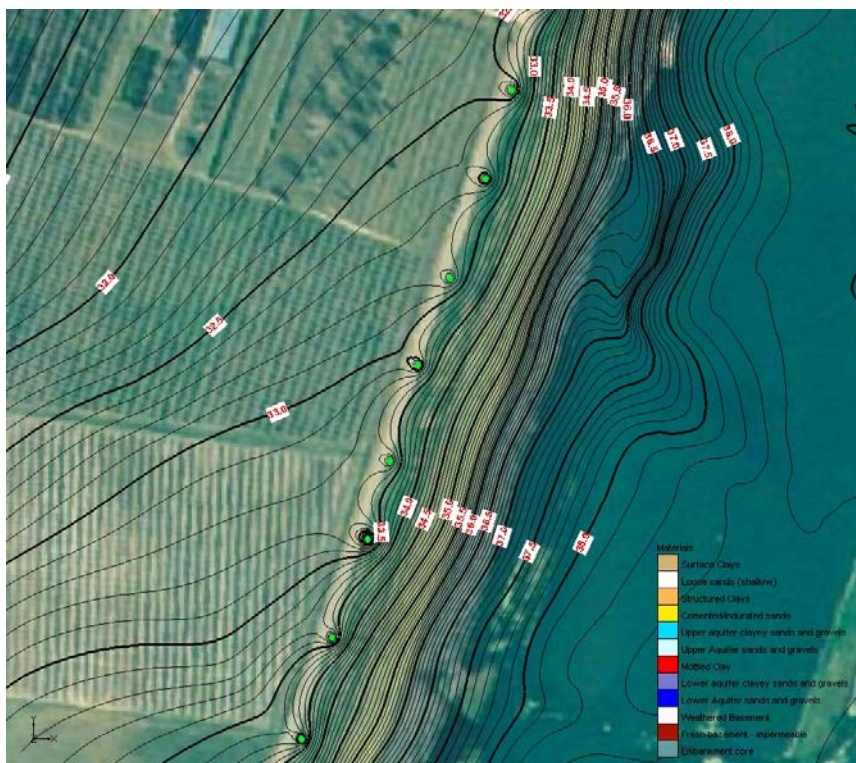


Figure 12 Groundwater Levels in Upper Structured Clays (Layer 2) RRD7_3855. Note the effect of the pressure relief system on the piezometric surface.

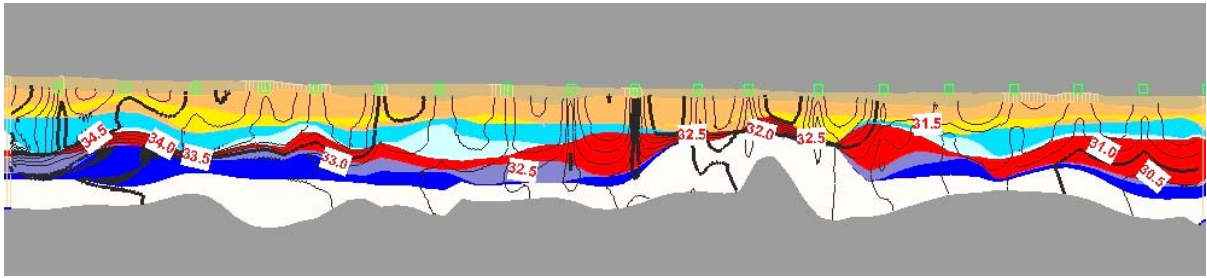


Figure 13 Embankment Long section RRD11_412 showing effect of pressure relief system.

8 Transient Modelling

A transient model was developed to assess the rate of groundwater rise (pressurisation of the foundation) during and following a flood event. The transient model was run with an instantaneous rise in reservoir level from RL 38.2m to RL 47.5m to evaluate the rate of rise, as shown on Figure 14. The analysis showed a relatively slow rise in groundwater level of approximately 0.5 m over 17 days followed by a decreasing rate of rise thereafter.

Selected bores, typically those with a reasonable historical record of groundwater levels, were modelled against the water level in the reservoir. This allowed predictive flood stage hydrographs to be developed for the bores. As illustrated in Figure 15, the peak piezometric level is achieved approximately one day after the peak flood level in the reservoir, after which the piezometric surface

decays relatively rapidly. The surface soils retain water for a longer period of time as a result of the topography of the site and the presence of a relatively low permeability layer directly beneath the soils.

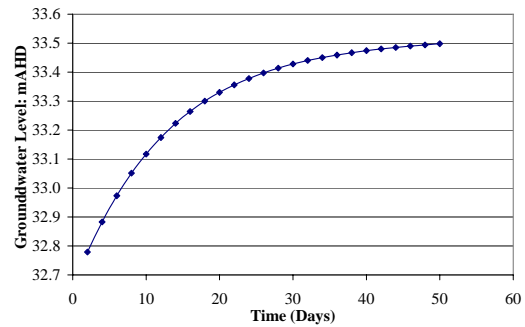


Figure 14 Plot of groundwater levels over time in the surface soils (Layer 1) at Ch 4 500, assuming reservoir head constant at RL 47.5m through time.

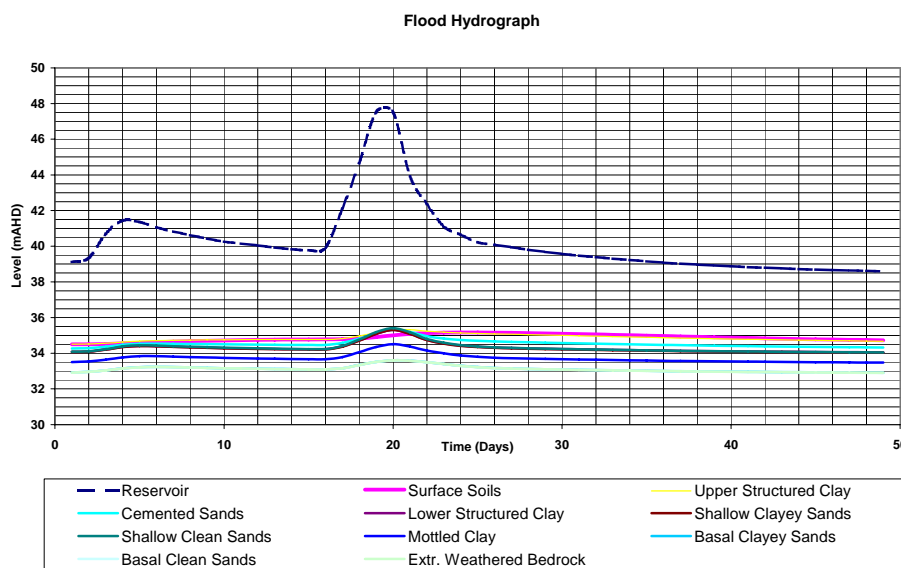


Figure 15 Flood Reservoir and groundwater hydrographs.

9 Conclusions

Based on conservative steady-state modelling, groundwater levels in the surface clays rise above ground surface in areas immediately adjacent to the embankment toe, for reservoir levels of 43.2 mAHD and above.

Transient modelling, however, indicates that, due to the relatively short periods where reservoir levels are above full supply level, and the time lag between reservoir and groundwater level rise, the current drainage system appears to be adequate in the area modelled.

Modelling of the proposed sand drain system under steady-state embankment level conditions indicates the embankment foundations are adequately drained with no areas with a piezometric surface above the ground surface.

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