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Wilsons Dam: Design Expectations Compared With Measured Performance

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Summary: Wilsons Dam is a zoned earthfill, water supply dam constructed for the Whangarei District Council between October 2000 and July 2002. The dam foundation comprises a 15m deep sequence of soft alluvial silts and clays, interbedded with permeable sand and gravel lenses. The catchment area is relatively small (3.9 km^2) and a viable water supply in the Northland climate required seepage losses to be minimised.

The foundation conditions raised a number of issues for the design of the dam, including seepage; construction stability; settlement and seismic liquefaction. These were addressed by a range of design provisions, including wick drains; cement-bentonite slurry cut-off wall; wide berms, upstream and downstream; highly plastic core; staged construction and instrumented monitoring of foundation performance.

This paper compares design expectations with dam performance, as measured in key areas, to date. It concludes that the design has successfully addressed the challenges of this site.

INTRODUCTION

Wilsons Dam is an 18m high, zoned earthfill embankment constructed on the Waiwarawara stream in Ruakaka, approximately 25km south along SH1 from Whangarei (ref Figure 1). The reservoir, which forms part of the Whangarei District Council (WDC) water supply network, stores 2.4 Mm^3 for supply in the Bream Bay area. Key dam statistics are provided in Table 1.

Bream Bay is the second largest water supply area within the Whangarei District, the locality is experiencing high growth and has significant zonings for both industrial and coastal residential subdivision. The area is subject to seasonal fluctuations in water demand, varying between 6,000 and 11,000 m^3/day. All consumers are metered and there are 2,328 water connections, including Whangarei's largest water consumer, the Marsden Point oil refinery, which uses between 4,000 and 6,500 m^3/day.

The geological setting, at the dam site, includes abutment ridges of highly-weathered greywacke (Waipapa group) forming a deep valley infilled with sediments of Pleistocene age. The main borrow area comprised tertiary-age siltstones, which have largely weathered to plastic clays.

The presence of deep soft soils, with permeable lenses, in the dam foundation meant that special consideration had to be given to several aspects of the design. The dam profile and construction programme were dictated by the need to avoid overstressing the foundation. A cement-bentonite slurry cut-off wall was used to restrict seepage in the near-surface permeable layers. Settlements in the order of one metre were expected and dam components have to be sufficiently plastic to accommodate the movement without cracking.

Instrumentation was installed to record settlement, lateral displacement and pore pressures. Monitoring of the instruments provided the ability to maintain construction stability by controlling rates of fill placement.

Table 1. Summary Of Dam Features

Dam height & crest length	18 m x 200 m
Reservoir capacity	2.6 Mm^3 (2.4 Mm^3 live storage)
Earthfill volume	210,000 m^3
Service spillway	Uncontrolled 3.9m diameter bellmouth
Diversion	2.55m dia concrete pipe, combined with spillway on left abutment
Auxiliary spillway	60 m wide grassed channel on right abutment
Outlet Pipework	375 mm dia multi level offtake and 3.5 km delivery line to Ruakaka WTP
Owner	Whangarei District Council
Designer	MWH Ltd
Constructor	Roadstone Construction Ltd
Construction period	October 2000 to July 2002
First Fill	15 July 2002 to September 2003 (projected)

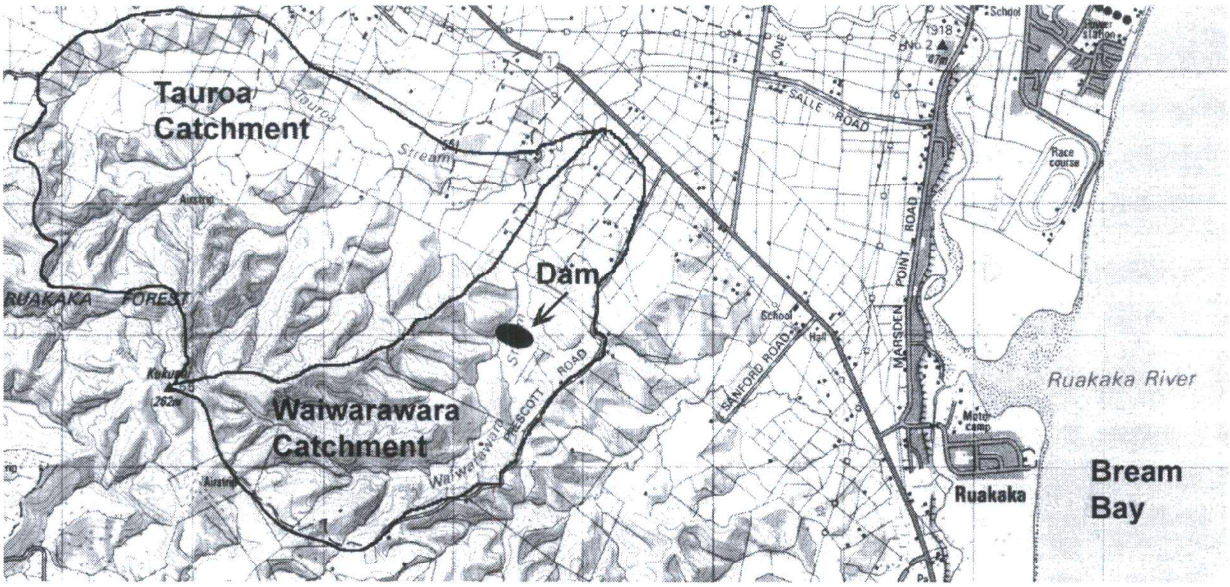

Figure 1. Location Diagram

SITE SELECTION

The site was first investigated in 1978 by the former County Council in a report covering five potential water supply schemes for the Bream Bay Water Supply Area. A 1987 Engineering and Environmental Impact report by Tonkin & Taylor, identified Wilsons Dam as the preferred option, based on economic and environmental issues, together with its close proximity to the Ruakaka Area.

Following the 1987 report, Council commenced land purchase for the dam and reservoir and obtained resource consent to dam and abstract water. Meritec undertook a final stage feasibility study in 1999.

In late 1999 MWH were appointed to provide final engineering design and contract procurement services for the project. Peer review contracts were awarded to Tonkin & Taylor for the design and Meritec during construction. The first phase of final design confirmed that Wilsons Dam could meet WDC's objective of providing a 1 in 50 year Return Period Drought supply capability at the end of a 50 year planning period.

FOUNDATION CHARACTERISATION

The foundation soils were investigated between 1987 and 2000 during feasibility studies and the final design. In all 15 trial pits, 10 boreholes and 19 CPT's were undertaken, with laboratory and field testing for plasticity, consolidation properties, pore pressure dissipation, triaxial shear strength, vane shear strength and permeability. Significant uncertainties associated with foundation permeability, consolidation characteristics and liquefaction risk were targeted in the design phase investigations.

The foundation model included a 3m thick crust of 'recent alluvium' (silts), overlying 'upper gravels' 2m thick, over 'estuarine silts' 10m thick and 'lower gravels' 5m thick before reaching the basement greywacke at approximately 20m depth. The investigation recognised that the 'upper gravels' appeared to be laid down in channels and did not form a continuous sequence. Several layers of varying thickness and depth, within the 'estuarine silts', could be found at any particular location. Groundwater was identified between 1 and 3m below the ground surface with some tendency to artesian pressures noted from deeper aquifers.

During construction, additional geological information was obtained from drilling for instrumentation and from CPT's used to confirm the cut-off wall depth. In general terms, the overall foundation model was confirmed and in particular, further evidence for the complexity of the 'upper gravels' sequence was found.

DESIGN CONSIDERATIONS

The dam foundation of deep soft soils, with permeable lenses, has driven many aspects of the dam design. Special consideration needed to be given to construction stability, settlement, foundation seepage and liquefaction. Design features specifically incorporated to deal with these issues include:

- Wick drains, to accelerate drainage and consolidation of the soft foundation soils
- A cut-off wall formed from cement-bentonite slurry, minimising seepage through the 'upper gravels'
- 30m wide berms, upstream and downstream, to consolidate and strengthen a wide area of the foundation
- A plastic core zone, to accommodate substantial settlements
- Staged construction, allowing dissipation of excess pore pressure between earthworks seasons
- Instrumentation along the crest and on two instrumented cross-sections, comprising 54 vibrating wire (VW) piezometers in the foundation, 16 VW piezometers in the fill and 15 combined inclinometer / extensometer tubes extending from bedrock up to the dam surface.

An indicative cross-section through the dam and its foundation is shown in Figure 2.

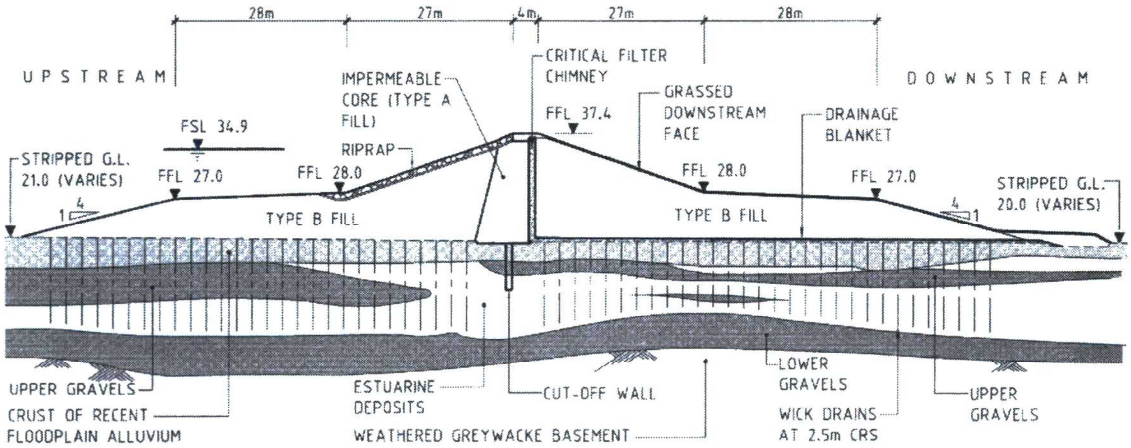

Figure 2. Dam Cross Section

DESIGN SOLUTIONS AND RECORDED PERFORMANCE

The following sections discuss how three key issues were addressed and compare design predictions with monitored performance.

Construction Stability

The 1995 failure of a Northland irrigation dam, during construction on similar foundations (Freer, 1997) illustrates the need for extreme caution in the design and construction of such dams. The key to embankment construction on soft ground is to avoid overstressing and yield of the foundation soils. At Wilsons Dam this was achieved by the adoption of staged construction; the provision of wick drains to speed consolidation; close monitoring of foundation piezometric pressures to control fill rates; and stability reviews at critical points based on measured pore pressures and foundation shear strengths.

Wick drains were used to speed consolidation and in particular to allow rapid drainage from the middle of thick layers of silts, thereby reducing the risk of any laterally continuous weak zones. Wick design was based on Bru's Chart 1981 (from Leroueil et al, 1990) and a square grid spacing of 2.5 m was adopted. One rig took four months to install 25 km of wick to an average 11m depth (14m max). The longitudinal extent of installation was limited to where fill depth exceeded 2.5 m, wicks were also curtailed where the valley sides reduced the thickness of soft foundation soils. A section of the foundation 12.5m wide, beneath the core, was left without wicks. The upstream wicks were collected into a piped drainage system, discharging to the diversion tunnel; flows of up to 4.5 litres per minute were observed during construction. The downstream wicks are terminated in the drainage blanket, which discharged freely to a temporary open channel throughout construction; flows were too small to be measured.

An undrained shear strength profile, based on the work of Leroueil et al, was developed for the design. Gains in shear strength, resulting from consolidation, were predicted using the increase in effective stress from embankment fill. During construction an indication of actual strength gains in the foundation were measured using 'Geonor' shear vane tests. Figure 3 compares the predicted 'design' strength profiles with measured shear values and 'reasonable lower bound' actual strength profiles. The comparison demonstrates that, while there was significant scatter in the test results, trends in actual strength gain were reasonably approximated by the design profiles.

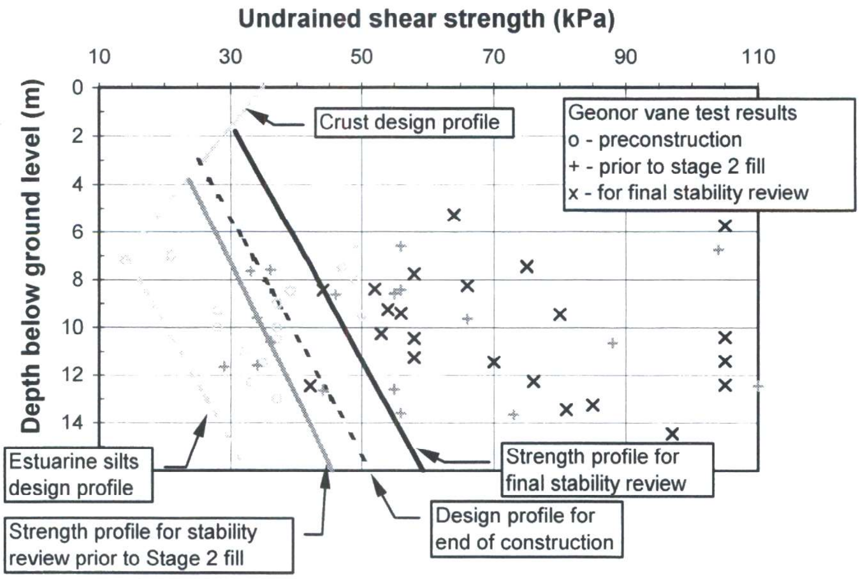

Figure 3. Shear Strength Profiles

Stability analyses were conducted using the predicted undrained strength parameters. Slope/W software was used with the Bishop Method for circular failure surfaces, or the Janbu method for non-circular failure surfaces.

Stability reviews during construction were based on revised foundation strengths from the Geonor testing and piezometer data. The fill density was also adjusted in accordance with data from fill monitoring. Factors of safety in excess of 1.3 were calculated for all temporary load cases at the design stage and these were confirmed during construction.

An additional goal of the stability reviews was to consider the possibility of raising the dam crest height and increasing reservoir storage. A poor foundation response would mean the crest height was limited to the design target of RL36.3 whereas a good response would allow construction to proceed to a maximum of RL37.5 giving a 30% increase in live storage volume.

The very wet conditions during all of 2001 severely restricted the contractor's ability to place fill. Towards the end of the first construction season, it was recognised that there was insufficient fill in the downstream platform to achieve the required strength gain in the foundation soils. Accordingly, eight days in late May 2001 were spent pre-loading this area with uncontrolled fill, which was removed at the commencement of the next construction season. Berm level was not reached until early February 2002 and a stability review at that time suggested a maximum final crest level of RL36.5. A final stability review was conducted as the fill neared that level and it was found that the interim strength gain allowed the final crest height to be raised to RL37.4. This 'extra' strength is attributed to the increased time taken to place the second stage fill and hence the greater consolidation that occurred in the foundation. The strength profiles used in these stability reviews are shown in Figure 3.

When normally consolidated soils are loaded in undrained conditions, pore pressures increase proportionally with load. However, as loading continues the soil will eventually begin to yield, at which point the excess pore pressure rises at a rate greater than the applied load. Routine monitoring of fill rates was achieved by plotting pore pressure rise in the foundation piezometers vs estimated fill load, as shown by sample traces of two instruments in Figure 4. The contract conditions included provisions to suspend filling if pore pressures were judged to be excessive. However, in the reality of the 2001 earthworks seasons, this option was not exercised and fill rates were effectively constrained by weather.

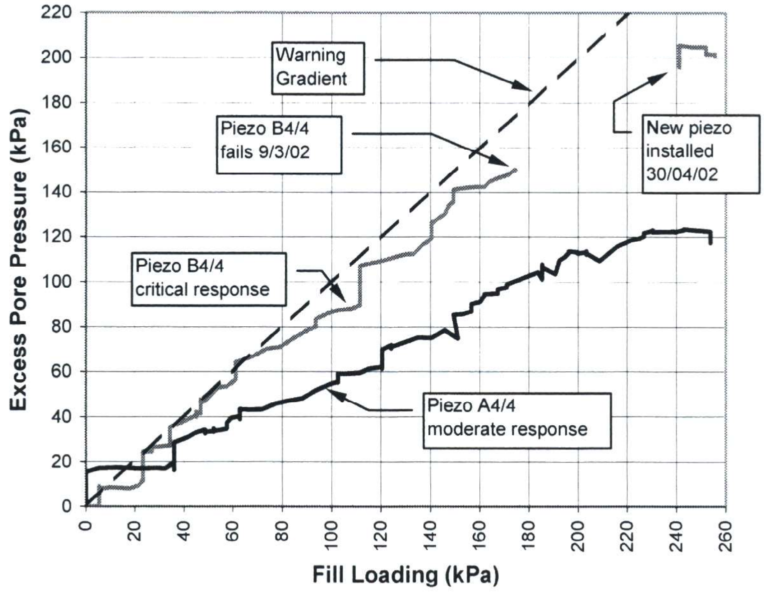

Figure 4. Pore Pressure Response Monitoring

Settlement

Foundation settlements under the dam considered components of elastic settlement, primary consolidation and secondary compression, predicted from conventional consolidation theory. Predictions are summarised in Table 2.

Table 2. Calculated Foundation Settlements

Component	Lower bound	Upper bound	Best estimate
Elastic settlement	20	75	35
Primary consolidation	575	1400	1075
Secondary compression	25	275	60
Total	**620 mm**	**1750 mm**	**1170 mm**

Primary consolidation settlement was estimated from laboratory oedometer testing. The observed range of results for the coefficient of compressibility (m_v) was 0.15 to 0.37 m^2/MN. The best estimate is based on m_v = 0.27 m^2/MN.

Due to a lack of site specific data secondary compression was estimated from the natural moisture content of the foundation soils. Mesri's Chart (Mesri 1975) indicated that the secondary compression ratio would probably lie between 0.07% and 0.9% with the most likely value being 0.21%.

Construction settlements depend on the efficiency of the wick drainage and the response of the foundation soils to loading. Expectations were that the degree of consolidation would be at least 50% and possibly 75% by the end of construction. Settlement predictions during construction were thus in a range of 570 to 845 mm. By the end of construction settlement under the crest had reached 900mm. Assessments of actual consolidation by the end of construction were higher than expected at 80 - 90%, this is also a result of the extended duration for fill placement.

Results of settlement measurement at the crest and other key locations are shown in Figure 5 and indicate a reasonable degree of conformity with the original predictions.

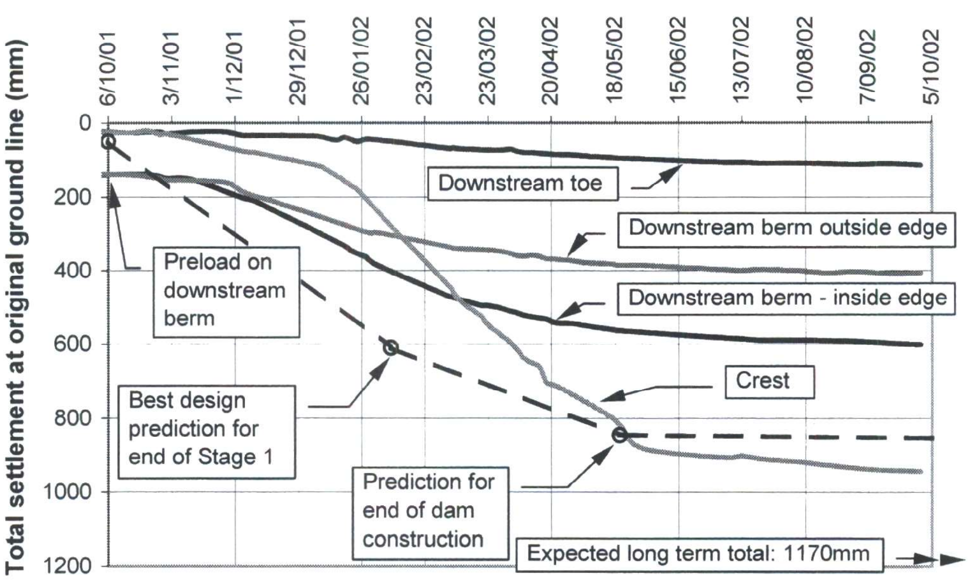

Figure 5. Measured Foundation Settlements – Mid Valley

Seepage

Reservoir losses through the dam, its foundation and the abutments were modeled using Seep/W software. The seepage model differs from the stability model to include appropriately conservative assessments of permeable layer thickness, allowing for the interlayering of permeable lenses within the estuarine silts. The model also extends hundreds of metres upstream and downstream of the dam to account for losses through the 'lower gravels'.

Seepage modelling was based on foundation permeability's ranging from 10^{-4}–10^{-6} m/s for the gravels to 10^{-8}–10^{-10} m/s in the 'estuarine silts' and 'recent alluvium'. A permeability of 10^{-10} m/s was deemed 'most likely' for the dam fill. Specific steps taken to minimise foundation seepage included construction of a cement-bentonite slurry cut-off wall through the 'upper gravels' and minimising the interconnection of wick drains with the 'lower gravels'.

The cut-off wall was specified as a very weak cement-bentonite slurry with requirements on plasticity, permeability and a 28 day unconfined compressive strength of 150 kPa. Australian consultants M.P.A.Williams and Associates developed the mix design and installation techniques. Roadstone planned and managed the construction to use locally available resources, with assistance from Richardson Drilling for slurry mixing. MWH set the target depth based on a series of CPT's along the dam centreline and construction was monitored closely to ensure that the wall extended into the 'estuarine silts' layer.

Some problems of trench collapse occurred, where gravel deposits were thick and close to the surface. However, initial concerns that these collapses would bridge the trench have been allayed by comparison of piezometer records (ref Figure 6) between Line B where trench collapse occurred and Line A, where the trench appeared to remain stable.

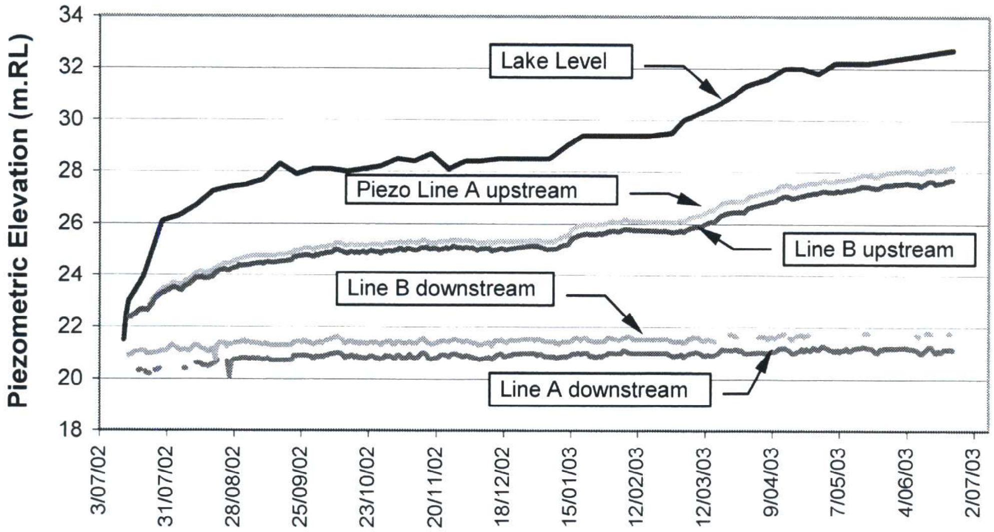

Figure 6. Head Loss Across Slurry Cut-Off Wall

The final installed depth for wick drains was determined by observation of penetration resistance for a pilot series of wicks on a 10m grid. A 'lower gravels' surface was then interpolated from this grid and intermediate wicks were installed with a nominal metre clearance from this surface.

Best estimates of seepage, with the cut-off wall installed, range from 0.4 to 1.2 l/s depending on the degree of interconnection between wicks and 'lower gravels'. Upper bound seepage is an order of magnitude greater at 4 to 12 l/s. Conjunctive use water modelling demonstrates that seepage losses up to 12 l/s will not affect reservoir viability. Measured seepage during commissioning has not exceeded 0.45 l/s with the reservoir at 1.5m below full service level (June 2003).

Seepage is collected and measured in a drainage system leading to a series of manholes across the downstream toe of the dam. The drainage blanket is separated, by bunds, into six zones in order to differentiate between flow sources. Another drain collects seepage along the toe of the right abutment. It is assumed that the wicks will provide a preferential path for deep seepage to enter the collection system via the downstream drainage blanket. However, it is possible that some seepage will exit the site in either the 'upper' or 'lower gravels' without being measured.

SUMMARY AND CONCLUSIONS

1. Foundation conditions at this site presented significant difficulties for dam design and construction. Monitoring to date has confirmed the success of the design.
2. An undrained shear strength model, based on the work of Leroueil et al, adequately predicted the strength gain in the foundation soils.
3. Traditional consolidation theory adequately predicted the range of settlement experienced to date.
4. Continual monitoring and updating of stability predictions recalibrated with actual field data, allowed the dam height and reservoir volume to be maximised during construction.
5. Seepage losses have been minimised through careful installation of wick drains and by provision of a cement-bentonite slurry cut-off wall through the permeable 'upper gravels'.

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Slope/W and Seep/W, software products of Geo-Slope International.