Design Considerations for Water Storage Structures on Porous Foundation in South West Western Australia

Michael Ashley, Robert Wark
GHD

The construction of service reservoirs has been an integral part of the development of water supply systems throughout Western Australia, and many such developments have occurred in coastal regions. The porous and highly soluble limestone foundations that are found in coastal regions pose specific challenges and risks for the long term management of these structures. Minimising leakage rates has been traditionally driven by economic losses. However, it has become apparent that the leakage has caused long term structural damage to the foundations of the structures.

Based on four case studies from south west Western Australia, this paper describes the extent of the problem, investigation and testing methods, design challenges and construction issues to be considered when constructing water storages on porous foundations.

Keywords: porous foundations, leakage

Introduction

The occurrence of leakage problems in service reservoirs throughout Western Australia has caused issues with the structural stability of foundations and embankments. Minimising leakage has historically been driven by economic factors, however issues with the structural performance of foundations due to significant leakage has been observed in coastal limestones and similar materials. There have been cases where foundations have dissolved, eroded or settled due to leakage, which caused further damage to liner systems and further leakage problems. There is a risk in some cases that the leakage could progress and increase the risk of a catastrophic failure of the storage.

The causes of the problems experienced with service reservoirs throughout Western Australia include the following:

- Design of single layer polyethylene liner systems with no leakage detection or protection systems. If damage to the liner system occurs and a leak develops, there is no method of quantifying the leakage rate and potential foundation damage.
- Inadequate joint detailing in concrete lined storages. Typically, there is a higher risk of leakage at the joints between slabs of concrete lined storages compared to within the slabs, and this is often due to insufficient water stop detailing at the joints.
- Site geology. The site geology can have a significant impact on the porosity of foundations. Foundations at sites located in predominantly coastal areas which include limestone have been observed to dissolve and cause structural collapse.
- Inadequate construction supervision. If the construction supervision of lined storages is not adequate and quality control procedures are not adhered to effectively, there can be future issues with leakage and structural performance of foundations.
- Measurement accuracy. Traditional leakage measurement tests such as “drop tests” are not sensitive enough to detect significant leakage before it can affect the structural performance of foundations.

The extent of leakage and foundation problems, site investigations, testing methods, design considerations and construction aspects were investigated for four case studies in coastal regions of WA.

Case study 1 - Kwinana Water Treatment Ponds

General

Kwinana is an industrial suburb located approximately 36 km south of Perth in Western Australia.

The Kwinana Water Reclamation Plant (KWRP) supplies highly treated waste water to industries within the Kwinana industrial precinct for use as process water and in cooling systems. KWRP is one of the largest water recycling facilities in Australia and is owned, operated and maintained by the Water Corporation of WA and its Alliance partners. Photograph 1 shows the Northern Pond before remedial works.

Photograph 1 Northern Pond at KWRP prior to remedial works

KWRP has been in operation since 2004, and at full capacity since 2008. The two storage ponds at the site have a total capacity of 20 ML. Each pond consists of a lined earth embankment with a flexible floating cover.

Extent of the problem

Leakage monitoring showed water losses in the ponds increased from negligible levels in around 2004 to approximately 1,000 to 1,500 ML/d in around 2008. "Drop tests" indicated that the storage level in the ponds was dropping substantially, although leakage from valves may have contributed to the losses.

A diver inspection within the Northern Pond in 2010 revealed several cracks and tears in the single layer HDPE liner. A typical tear is shown in Photograph 2.

Photograph 2 Tear near weld seam (dye check)

As a result, the liner was removed and geotechnical investigations revealed several soft depressions in the embankment and foundation material. Laboratory test results on representative sample classified the embankment and foundation material as calcareous marl with up to 60% carbonate content, although in the areas of leakage the carbonate was almost completely dissolved.

The cause of the liner failure was not conclusively determined; however the following potential reasons were identified:

- Rapid loading of the liner stressed the weld/joints and may have caused tearing. Tearing may have occurred as the pond was emptied and refilled at a rate which exceeded operating guidelines.
- Anecdotal evidence suggested that the original design included a diffuser inlet located centrally in the pond. This system was changed to a simple inlet pipe located at the southeast corner of the pond. Movement of the inlet pipe may have resulted in the tearing of the liner joints and welds in the area.
- The embankments and foundations were constructed with the limestone marl material available on site. The materials were highly calcareous and may have adversely reacted with the slightly acidic treated water, resulting in partial dissolution of sections of the embankment and foundation and the creation of voids beneath the liner. The voids were likely to have caused the liner to stretch, causing new tears and widening of existing tears.

Geology

The site was primarily located on the coastal limestone system, and the Tamala Limestone was the principal geological unit in the area.

Site investigations

Visual inspections and geotechnical investigations were undertaken to investigate the cause of the leakage in addition to the diver inspections.

A visual inspection was undertaken in March 2010 which identified that the liner weld seams had deteriorated. Depressions were observed in areas of the liner, suggesting that the foundation material had eroded or scoured.

Geotechnical investigations were undertaken to determine the foundation conditions. Geotechnical investigations consisted of Perth Sand Penetrometer (PSP) tests and test pits. The PSP results showed areas of very loose to loose sand up to 900 mm deep in areas where depressions were observed. The foundation material was classified as highly calcareous sand.

The process water stored in the ponds is slightly acidic and leakage is likely to react with the foundation material and result in voids.

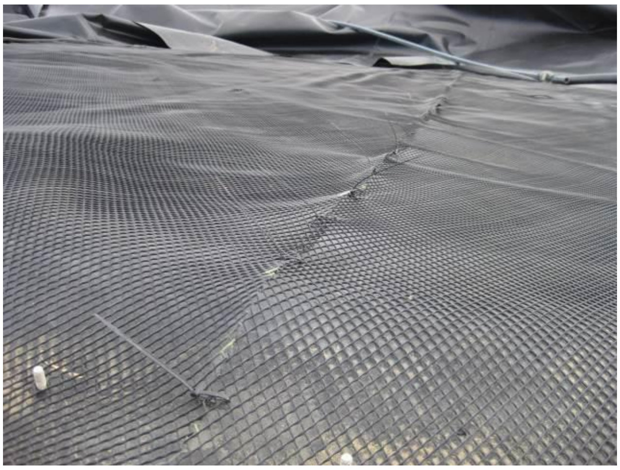
The site investigations concluded that the liner foundations did not provide adequate structural strength to support the liner.

Design approach

Remedial works consisted of foundation reconstruction and replacement of the liner. The following liner replacement options were considered:

- Geosynthetic clay liner (GCL) with a nominal 150 mm thick protective non-calcareous sand layer, covered by a single layer of HDPE.
- 300 mm thick compacted clay liner.
- Double HDPE liner with an intermediate permeable grid. This arrangement allows seepage through the top liner to be collected.

A clay liner was not considered further as it would be difficult to construct due to the steep embankments, and suitable clay material was not readily available. A double HDPE liner with intermediate permeable grid was preferred as the long-term solution as it provided two layers of leakage protection and reduced the risk of the chemically aggressive water dissolving the carbonate content of the foundation soils. If a leak occurred in the top liner layer, the design was that the leakage would be collected by the permeable grid layer. A similar liner system was successfully used for several ponds at the nearby Woodman Point Wastewater Treatment Plant. Photograph 3 shows the HDPE liner and permeable grid installation.



Photograph 3 HDPE liner and permeable grid installation

The existing inlet and outlet pipework was also upgraded from PE pipes to stainless steel pipes including a diffuser with concrete encasement and anchor blocks. The pipe upgrade was designed to limit pipe movement and reduce the risk of vortex formation and foundation movement.

The Northern and Southern Ponds have now been successfully remediated. Photograph 4 shows the Northern Pond after remedial works.

Photograph 4 Northern Pond after remedial works

Case study 2 - Typical concrete lined reservoir

General

A case study was undertaken on a typical concrete lined reservoir in south west Western Australia.

Extent of the problem

The original reservoir was extended and the extension had a history of leakage.

Leakage was reported almost immediately upon filling after completion of the extension works area. Leakage was not reported in the original reservoir area. Reportedly, there were issues related to poor workmanship and quality control during construction of the extension. The following possible reasons for leakage from the reservoir were identified:

- Poor joint detailing leading to a failure of the joint sealant between slabs. A typical joint is shown in Photograph 5.
- Use of coastal limestone as bedding layer material. The bedding materials may have adversely reacted with the stored water, resulting in partial dissolution and voids beneath the liner.
- Structural failures of the slab foundations caused by insufficient compaction of the upper zone of sandy material below the reservoir, solution of lime or lime cementing.

Photograph 5 Typical joint between concrete slabs

Geology

The reservoir site was located on the Spearwood Dune System which consists of calcareous sands, limestone pinnacles and calcareous sandstones.

Borehole drilling intersected limestone approximately 2 m below the planned floor level of the reservoir, however it was reported that limestone boulders and outcrops were intersected during construction.

Site investigations

Micro seismic testing and coring investigations were undertaken which confirmed the deficiencies of the subgrade below the parts of the reservoir. The voids identified below the base slabs were subsequently grouted and the entire reservoir was lined with an unbonded HDPE liner. Photograph 6 shows a typical core drilled in the concrete at a joint.

Photograph 6 Typical concrete core at joint

At a later date, a diving inspection was undertaken which identified tears through the HDPE liner. This was followed by non-destructive survey of the foundations beneath the floor and batter slabs using ground penetrating radar and laser levelling. Some anomalies were detected, but the cause was not confirmed.

After the diving inspection, it was reported that the reservoir leakage increased significantly over an extended period of time. An additional inspection by divers confirmed a number of ruptures in the liner. The ruptures were repaired and a "drop test" was undertaken, however the leakage continued. Further inspection by divers located new leaks, often associated with previous repairs. The common causes for the leaks were believed to be movements between the slabs caused by the foundation conditions. The foundation was constructed with limestone materials and not crushed aggregates, and movements caused localised increases of stresses induced in the repair extrusion welds and subsequent failure.

The foundation voids increased the risk to the short-term stability issues. The possible failure mechanism was that unsupported sections of the concrete slabs could subside and crack, which in turn could lead to a failure of the HDPE liner, leading to an uncontrolled release and possible catastrophic failure.

Design approach

The following short term remedial works were recommended as a trial treatment of about 60% of the floor area of the extended reservoir:

- Removal existing HDPE liner progressively.
- Grout voids under slabs and core holes by jet grouting.
- Reinforce and reconstruct slab contraction joints with reinforced concrete.
- Apply an external polyolefin seal to seal the reconstructed joints.
- Repair and seal the edges of the remaining liner.
- Undertake ongoing monitoring and assessment of piezometers and water balance data.

Early assessment of the remedial works indicated that the works were successful. Photograph 7 shows a typical joint seal repair.

Photograph 7 Typical joint seal repair

The trial repair has been successful in that all the sealed joints are 100% water tight when tested by divers. However leakage through the remaining HDPE liner and concrete joints and further stages of the remedial works are to be undertaken to fully remedial the failed portions of the concrete liner.

Case study 3 - Albany Wastewater Scheme dams

General

The Albany wastewater scheme is located 14 km north-west of the town of Albany, approximately 400 km south-east of Perth.

The first storage dam for the Albany Wastewater disposal scheme was constructed in 1994. The dam was designed as an earthfill embankment dam, with a storage volume of 600 ML and maximum height of 9 m. The dam was designed to store treated wastewater for later use irrigating blue gum plantations.

The second storage dam was designed in 2006 as an earthfill embankment dam with an upstream clay blanket and internal chimney filters. The maximum height of the embankment was 10 m and the storage volume was 800 ML.

Extent of the problem

Soon after the first dam was constructed, significant water losses were identified. An investigation found that the borrow pit development for the construction of the dam intersected a sequence of sandy lenses in the reservoir basin. Although the borrow pits were capped, water balance analyses indicated that seepage losses from the storage were approximately 400 ML/annum.

The downstream toe area at the first dam was observed as very wet throughout the year, which is an indication of significant seepage through the foundation. Photograph 8 shows seepage observed at the toe of the dam.

Photograph 8 Seepage at downstream toe

Geology

The dam site at Albany is underlain by siltstones of the Plantagenet Group and alluvial sands exist along the watercourses. Zones of laterite cap rock, boulders and gravel occur on the more elevated ridge tops in the area.

The Plantagenet Group is divided into the lower Werillup Formation and the overlying Pallinup Siltstone. The Werillup Formation occurs at depths of more than 40 m below surface at the dam site.

The Werillup Formation consists of basal conglomerate, coarse sandstones, peat and siltstone. The overlying Pallinup Formation consists of very fine sandstone, siltstones and spongolite. The Werillup Formation has major aquifer potential and the Pallinup Formation has minor aquifer potential.

Site investigations

Investigations were undertaken to determine the cause of the water losses. The initial geotechnical investigation identified a sequence of laterised silt and clay soils at the dam foundation. A subsequent investigation found that the borrow pit intersected a sequence of sandy lenses in the reservoir basin.

During the expansion planning stage, an investigation of the source of the observed seepage at the toe of the original dam was undertaken. The downstream toe area was excavated to uncover the blanket filter and a rock toe drainage zone as included in the design. However it was found that the blanket drain was not constructed as designed. The seepage collection trench and rock toe was subsequently reconstructed and the seepage and water logging was reduced.

Photograph 9 shows a sand filled root hole located in the dam foundation during the geotechnical investigation.

Photograph 9 Sand filled root hole in foundation

Design approach

As a result of the findings of the investigations and the leakage at the first storage, the storage basin of the second dam was fully lined with a clay liner. The clay liner included a maximum hydraulic conductivity of $1x10^{-9}$m/s in accordance with the permeability guidelines contained in the Water & Rivers Commission Water Quality Protection Note “Low Hazard Wastewater Containment with Non-Synthetic (Clay) Liners”. Photograph 10 shows the reservoir basin lining works.

Both dams are now operating as intended.

Photograph 10 Reservoir basin lining works

Hopetoun Dam

General

Hopetoun is a town located approximately 800 km south-east of Perth in Western Australia. Hopetoun Treated Wastewater Storage Dam (TWSD) was designed in 2007 as a homogeneous earthfill embankment dam with a clay liner. The maximum height of the embankment was 5.6 m and the storage volume was 89 ML.

Extent of the problem

The design of the storage required consideration of the limestone foundations and their susceptibility to dissolution.

Geology

The site geology includes significant areas of coastal limestone. The Hopetoun area is made up of predominantly Quaternary to Cainozoic deposits ranging from mobile beach sand at the coast to predominantly stabilised coastal dune sands and consolidated calcarenite deposits slightly inland.

The coastal dune sands are comprised of a fine to medium grained calcareous sand derived from the weathering of the underlying limestone rock. The underlying limestone consists of a siliceous calcarenite with variable quartz, shell debris and calcareous cement. The limestone is coastal limestone, commonly found along the coast of Western Australia.

Site investigations

The geotechnical investigations indicated the geology at the dam site consisted of a layer of topsoil underlain by sand. Limestone was encountered at depths of 0.8 m and 5.3 m below surface level in a number of test pits at the dam site. Photograph 11 shows a typical limestone pinnacle encountered in the dam foundation.

Photograph 11 Limestone pinnacle in foundation

Design approach

The design included a 300 mm thick clay liner on the base and side slopes with a maximum hydraulic conductivity of $1x10^{-9}$m/s. The thickness of liner was selected based on the permeability guidelines contained in the Water & Rivers Commission Water Quality Protection Note “Low Hazard Wastewater Containment with Non-Synthetic (Clay) Liners”. The upstream faces were set at 1V:3H to enable the clay liner to be placed on the embankment earthfill with reasonable efficiency. The minimum drawdown level was set at 0.5 m above the reservoir floor level to reduce the risk of the clay liner drying out and cracking.

Rip rap was included on the upstream face protection to provide protection against wave action and run up.

The technical specification required the Contractor to construct the embankment by lifting in horizontal layers. Constructing the liner up the slope was not permitted.

Photograph 12 shows the clay liner construction.

The construction was completed successfully and the dam is now functioning as intended.

Photograph 12 Clay liner construction

Discussion

The cases of leakage causing structural issues with foundations in service reservoirs in south west Western Australia suggests that further consideration should be given to traditional design criteria adopted for foundation and liner design.

The limited solubility of calcium carbonate in water limits the maximum rate of dissolution to approximately 45 gm/kL of water. However, with relatively large volumes of water leaking into some reservoir foundations, the volumes of lime dissolved over time can become significant. In coastal limestone areas, the carbonate content in the calcareous sands can lightly cement the sand particles forming a bridging structure. The application of substantial quantities of water can result in a collapse mechanism, which can produce significant settlement in soils with relatively low concentrations of carbonate.

The presence of open solution tubes, often believed to have formed around tree roots similar to the Albany Wastewater Scheme dam site, has also been observed at several other sites, most notably at Tamworth Reservoir in south west Western Australia. The washing of surface soils into these features can result in the formation of voids beneath liners.

The traditional method of measuring the performance of storage reservoirs founded on coastal limestone is by conducting “drop tests”. Drop tests typically involve shutting off all the inflows and outflows to the storage and measuring the drop in water levels over a period of several days. However, the accuracy of drop tests is not adequate after factors such as evaporation, rainfall if appropriate and temperature changes are considered, and foundation may be at risk.

Conclusions

The following design aspects should be considered when designing lined storage reservoirs in these areas:

- All tanks and reservoirs located on coastal limestones or similar foundation types should include leak collection and measurement systems. This allows for objective assessment of leakage and collection of leakage before potentially reacting with foundations.

- Geotechnical properties of foundations and bedding materials should be investigated for susceptibility to dissolution from leakage to minimise the risk of movement or voids being created under liner systems. Consideration should be given to foundation treatments and importing bedding materials if the materials available on site are susceptible to dissolution.

- Double liners should be considered for storages with a polyethylene liner system to provide an additional layer of protection and allow drainage between layers.

- Leak collection and detection systems should be considered for double polyethylene liner systems. Typical leak detection systems include drainage media installed between liner layers designed to convey leakage flows, and a collection sump or pump where leakage flows can be measured.

- Robust joint detailing should be considered for concrete lined storages. Consideration should be given to detailing the joints with double water stops. Joint sealant properties, such as durability and flexibility are particularly important. In chlorinated water systems the sealants should be assessed for resistance to attack by chlorine.

- Adequate construction supervision should be provided to allow the works to be constructed in accordance with the design intent and specifications. Consideration should be given to support during construction from design engineers on a full time basis, or alternatively inspections can be undertaken at designated hold points.

Acknowledgements

The authors wish to thank the Water Corporation for permission to use some of the information from their sites as case studies for this paper.

References

GHD, 2012. Design and Construction Report for KWRP, Northern Pond Reconstruction.

GHD, 2007. Design Report, Hopetoun Treated Wastewater Storage Dam – Part 2.

GHD, 2008. Detail Design Report, Albany Treated Wastewater Storage Dam Number Two.

GHD, 2007. Geotechnical Investigation Report, Albany Treated Wastewater Dam Number Two.

GHD, 2007. Hopetoun Wastewater Treatment Plant, Additional Geotechnical Investigations.

GeoEng, 2001. Tamworth Reservoir Rehabilitation, Geotechnical Investigation Report, Volume I.

Water Corporation, 2010. Kwinana Water Recycling Plan (KWRP) Northern Pond Liner and Soil Condition Assessment – Technical Report.