The British Dam Society

Leakage Remediation Works at the Hampton Distributing Reservoir

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SYNOPSIS Hampton Distributing Reservoir is a non-impounding reservoir built in 1900s and located in Hampton, southwest London. The reservoir, formed by a typical puddle clay core embankment, has a total perimeter of 800m and a storage capacity of 32,000m^3.

An increase in embankment settlement was detected, starting from 2011, based on annual crest levelling surveys, which was then followed up with a non-intrusive geophysical survey in 2020. This identified a distinct leakage path at the foundation level of the reservoir embankment. In order to mitigate the risk of seepage-induced instability such as internal erosion, leakage remedial measures were proposed to arrest the leakage.

Limited working space and difficult access were some of the main constraints for the remedial works. Following an optioneering/feasibility study, permeation grouting using Tube-a-Manchette (TaM) was identified as the most practical remedial solution. Grouting works were carried out on both sides of the clay core to target flow paths and create a low permeability zone reducing the leakage/seepage through the dam.

This paper presents the key aspects of the project, from the initial investigative works to construction, covering also the optioneering and design of the grouting works. Challenges and lessons learnt from the project are also highlighted.

INTRODUCTION

Hampton Distributing Reservoir (locally known as 'Red House Reservoir') is located in Hampton, southwest London. It is a small non-impounding reservoir built in 1900s, owned and operated by Thames Water Utilities Limited (TWUL). Water supplied by the Staines Reservoirs Aqueduct is temporarily stored in the Hampton Reservoir, and then gravitates to the Grand Junction Reservoir at the Hampton Water Treatment Works.

The reservoir is formed by a typical puddle clay core embankment with a maximum height of 3m. It is approximately triangular in plan with a length of 250m, base width of 150m, and a total perimeter of 800m. It has a storage capacity of 32,000m^3.

The typical cross section of the embankment is shown in **Figure 1**. The main characteristics of the embankment section are the following:

- Maximum height of 3m with a 1.8m wide crest, 1v:2h downstream slope and 1v:3h upstream slope, the latter protected by concrete slabs from crest to toe.

- Embankment shoulders are formed by clayey sandy Gravel on a stripped surface of original ground level over Kempton Park Gravel Formation.
- A 0.9m wide puddle clay core that passes in a trench through the Kempton Park Gravel Formation and is keyed into the underlying London Clay Formation with a 1.2m deep embedment which results in a total height of 8.5m for the puddle clay core/trench (not fully shown in the cross section below).

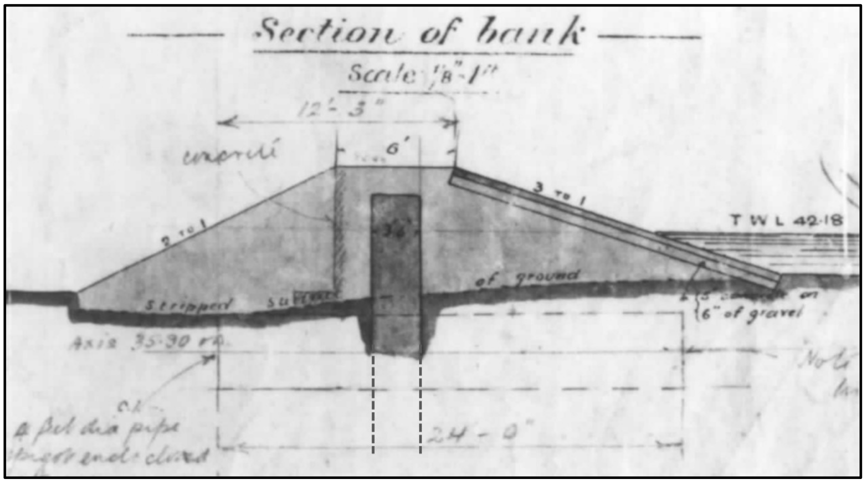

Figure 1. Extract of record drawing showing a typical section of the embankment, with a clay core extending to the London Clay Formation at depth.

THE PROBLEM

Embankment crest levels were monitored annually at nine points on the reservoir rim. In 2011 an inspection was carried out under Section 10 of the Reservoirs Act. As a part of the reservoir inspection, crest surveys data were reviewed with the results showing an average settlement rate of 3mm/year at the southwest side of the dam over the period between 1988 to 2011, which was considered as a normal amount of settlement. There was no significant settlement at other monitoring points. The inspection report recommended that annual monitoring of the embankment to be continued.

Between 2011 and 2019, average settlement continued at just under 3mm/year, except at one monitoring point at the south side of the dam, which recorded an increase in average settlement of 6.4mm/year, with two years where settlement exceeded 10mm. The QCE (Qualified Civil Engineer under the Reservoirs Act) was consulted, and the crest surveying frequency increased.

Due to a continuing trend of settlement, in 2020 the reservoir Supervising Engineer (under the Reservoirs Act) requested a geophysical seepage survey in order to investigate potential leakage problem in that section of the embankment. The survey identified a zone of leakage extending some 20m on the south side of the dam (Figure 2) at the same location where the larger settlement was recorded.

Seepage survey results showed the leakage at a depth of approximately 9m below the crest level which corresponds to the bottom of the puddle clay trench. It was suspected that the leakage passed through the clay core at the interface with the London Clay formation. The concentrated leakage paths could lead to internal erosion of embankment materials. If the internal erosion was allowed to develop further, the integrity of the dam could be compromised, which could eventually lead to its failure.

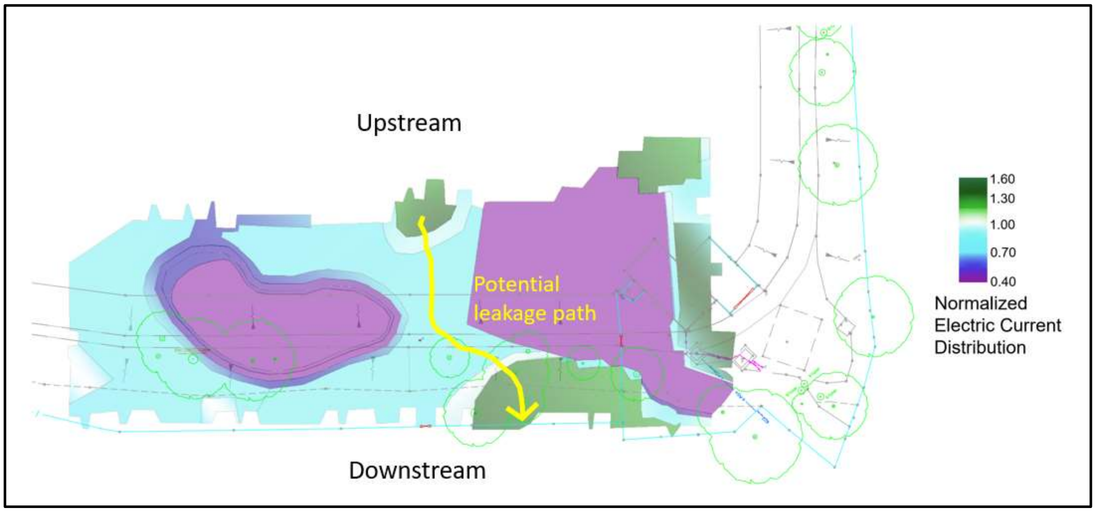

Figure 2. Geophysical seepage survey showing the leakage zone

TWUL (the Client) commissioned MWH Treatment (MWHT, the Main Contractor) to undertake leakage remediation works and subsequently MWHT commissioned AtkinsRéalis as designer to support the implementation of the project during design and construction. The assignment started with a ground investigation to better understand the embankment characteristics, an options appraisal to identify an appropriate solution for the remedial works, and was followed by the design and construction support. MWHT commissioned Keller as the geotechnical contractor who provided technical advice for the grouting works and carried out the construction.

OPTIONEERING

The optioneering study was carried out to identify the most appropriate leakage remedial solution in terms of the effectiveness, buildability, sustainability and cost. Remedial solutions using either a piled cut-off wall or grouting were considered.

Difficult site access and limited working space were the main challenges in the project. The width of the embankment is only 1.8m. The embankment slope and downstream toe are populated by some large trees and vegetation which limits the headroom on crest, as shown in Figure 3 below. The reservoir area is a Ramsar site and a ‘Site of Special Scientific Interest (SSSI)’.

A sheet piled cut-off wall is a proven method to provide a low permeability continuous barrier along an embankment, which was recently used in other reservoirs in the London region such as Island Barn, William Girling and King George V reservoirs. This solution for the Hampton Distributing reservoir would require installation of 10m long sheet piles from the embankment crest through the puddle clay core into the underlying London Clay formation.

However, due to the very narrow crest, piling works would have to be assisted by a mobile crane set up either at the toe of the embankment or on a floating pontoon on the reservoir. Either option would have required significant enabling works. Considering the site constraints and ecological sensitivity of the site, the pile cut-off wall solution was not considered feasible.

An alternative remedial solution using permeation grouting was proposed. The technique involves injecting low pressure cement grout into the ground using the Tube-a-Manchette (TaM) method. The grouting works would not require heavy plant hence avoiding the need for significant enabling works. Drilling works could be conducted on the narrow crest by a small drilling rig to create boreholes for TaM pipe installation.

GROUND INVESTIGATION

In September 2022, a new ground investigation (GI) was carried out by Structural Soils Ltd in order to better understand the ground conditions and provide geotechnical parameters for the design of remediation works. In addition, the new GI also provided confirmation of width, depth and position of the puddle clay core in the works area.

The GI works started with hand-dug slit trenches on the embankment crest to expose the clay core and to confirm its alignment. Dynamic probing was conducted at three locations, followed by low-vibration percussive boreholes through the centre of the clay core down to the London Clay formation. A small Windowless Sampling rig compactible for drilling works on the narrow crest was used (Figure 3). These exploratory holes were spread through the 30m chainage, to confirm the depth and condition of the clay core and the London Clay where the core keyed in. The boreholes were fully cased which protected the thin clay core from hydraulic fracturing and hole collapse. Verticality was checked throughout the drilling works in order to reduce the risk of penetrating the sides of the clay core.

Figure 3. Small portable drilling rig on narrow crest

During the GI, two boreholes were terminated at a shallower depth after water strikes were observed at 6m to 8m below crest level within the suspected leakage zone. The soil samples at these levels showed that the puddle clay core was very soft with high moisture content. The levels where water strikes were encountered were slightly higher than the leakage zone determined in geophysical seepage survey (9m below crest level), which suggested that the problems in the clay core could be more widespread than originally anticipated.

A percussive borehole was carried out at the toe of the embankment to provide samples and data for the natural strata. The level of the interface between the Kempton Park Gravel and London Clay formation was also determined. In situ permeability testing was conducted to determine the permeability of the soil (Kempton Park Gravel) underlying the embankment.

The particle size distribution and permeability of the foundation materials were used to inform the grouting design.

THE GROUTING SOLUTION

Kempton Park Gravel (KPG) formation beneath the embankment consisted of a clean sand-gravel mixture with a permeability generally ranging between 10^{-4} and 10^{-5}m/s. The geophysical survey and dam settlement monitoring indicated that pronounced water flow paths had developed in discrete locations. It was, therefore, predicted that zones of higher permeability would be present where the finer grained elements of the soil had been eroded.

To target the erosion paths, a grid of grout injection points was established using the Tube-a-Manchette (TaM) system. Each TaM pipe consisted of a tube with injection ports at regular centres over the intended grout injection zone. The injection sleeves consisted of perforations, covered with a rubber sleeve to form simple non-return valve. The TaM pipes were sealed into the ground with a low strength sleeve grout. Each injection sleeve could be isolated with the use of a double inflatable packer to allow the precisely controlled grout injection in the target soil at the required pressure. Each injection sleeve could be used multiple times to allow a phased approach to the grout injection.

A cross section of the proposed target zone for permeation grouting is presented in Figure 4. Two rows of TaM pipes were installed upstream of the dam core and two more rows were installed downstream of the core. The inner row grout holes were vertical. However, due to the limited crest width, the grout holes on the outermost row were inclined ('raked') with an angle of 10° which provided a broader grouted zone at the base where leakage was predicted to be most pronounced. This approach provided sufficient space for personnel to safely work on the crest.

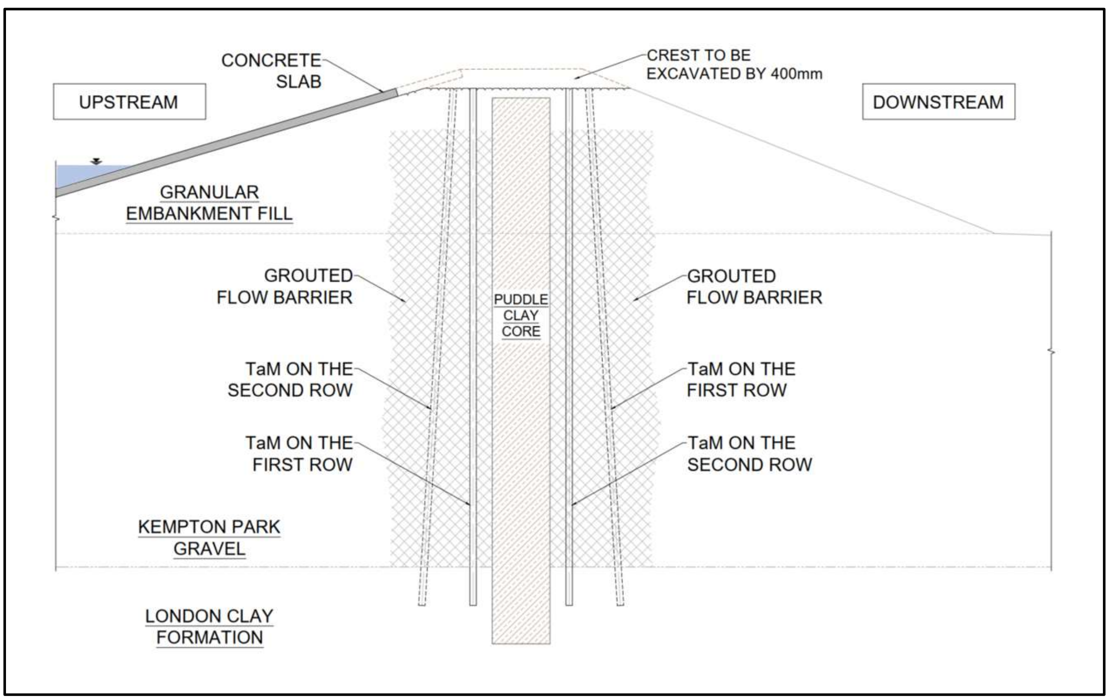

Figure 4. Grout injection zone within the dam

The grout holes were provided with a minimum 1m toe-in to the London Clay in order to achieve a good contact. The findings from the GI works indicated that the leakage zone may extend higher than the most pronounced paths determined by the geophysical survey. Therefore, the targeted zone of grouting was 8m deep extending from the dam shoulder into the London Clay formation, below the base of the Kempton Park Gravel.

A plan view of the grout hole arrangement on the embankment is presented in Figure 5. Two rows of TaM pipes were installed on an equilateral triangular grid on each side of the puddle clay core. The holes were spaced at 1m centres, in line with the CIRIA C774 (CIRIA, 2018) recommendation for medium to fine sand permeability ranges between 10^{-4} and 10^{-5}m/s.

The grout injection sequence was agreed with the QCE. Alternate primary and secondary grouting sequence was adopted. Injection data including grout injection volumes and flow rates were reviewed after each grouting cycle. The data were then used to identify zones of high-volume grout take and to determine the need of grout injections in the next phase.

The grouting works were carried out in five phases in the following sequence:

Phase 1: Trial grouting

Phase 2: Injection of Primary TaMs of the first row at downstream and upstream

Phase 3: Injection of Secondary TaMs of the first row at downstream and upstream

Phase 4: Injection of Primary TaMs of the second row at downstream and upstream

Phase 5: Injection of Secondary TaMs of the second row at downstream and upstream

The primary/secondary TaMs and the first/second rows are defined in Figure 5.

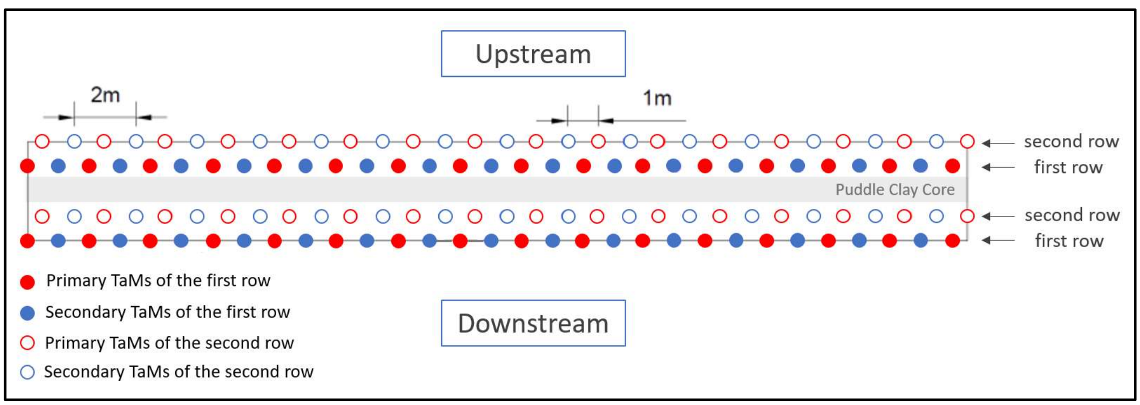

Figure 5. Grout borehole arrangement

After the five phases of grouting, additional reinjections were commenced on the selected sleeves where both high injection volumes and high flow rates were observed. The data was again reviewed and if necessary, the grouting was extended or repeated until satisfactorily low grout volumes and low flow rates were observed.

Cement based grouts were used to provide the longevity required. A cement bentonite grout mix was used as sleeve grout to seal the TaM pipes in place. It was also used in the initial grout injections to provide a low-cost solution to grout the most pronounced leakage paths.

The geotechnical contractor provided quotes for the grout mixes in Table 1. Microfine or Ultrafine cement grout were also considered due to their enhanced penetrability compared to cement bentonite grout. Several grout mixes were tested during trial injections and microfine cement was selected for the grout injection, to permeate as much of the soil as practical.

Table 1. Proposed grout mix

Type	Cement	Additive	Mix design	Particle size
Cement bentonite grout	Ordinary Portland Cement (CEM II)	Bentonite	1:10:20 Bentonite-Cement-Water sleeve grout mix	D95 < 50 – 75 µm
Microfine cement grout	Microfine cement (MasterRoc MP650 SR)	Superplasticiser: MasterRheobuild 1000	1:1 water cement ratio with 1.5% additive	D95 < 16 µm
Ultrafine cement grout	Ultrafine cement (MasterRoc MP800 SR)	Superplasticiser: MasterRheobuild 1000	1:1 water cement ratio with 1.5% additive	D95 < 12 µm

CONSTRUCTION

The construction phase commenced in late August 2023. Firstly, a temporary wider working platform was constructed by lowering the crest to allow for sufficient working space and to ease the drilling of the holes further away from the centre of the crest. In addition, a flat compound area of approximately 10m by 10m was used for material storage and equipment such as grout pump module and grout mixer, as shown in Figure 6.

Figure 6. Site compound area for grouting equipment

Because of the requirements of maintaining freeboard and allowing for sufficient cover to the puddle clay core, the maximum depth of excavation to create a wider working platform was limited to 400mm. Due to limited working space and difficult access, the geotechnical contractor used a small drilling rig (Klemm 702) with a width of only 750mm when it is tracked into position, which helped to overcome the accessibility constraints (Figure 7).

Figure 7. Drilling works on the crest

Before the construction, precautionary measures were put in place to minimise noise and vibration due to the ecological sensitivity of the site. Sound barrier blankets were installed around the works area and routine noise monitoring was carried out to ensure noise levels were within acceptable limits. The small earthwork equipment and drilling rigs also helped to minimise vibrations.

Since the works area was in close proximity to the reservoir, a containment system was installed on the crest to contain drill and grout arisings during the construction. Contamination risk to groundwater and reservoir water was managed through a careful control of the maximum grout volume per sleeve and injection pressure in order to limit the grout spread. In addition, routine sampling and testing for pH value and turbidity were carried out throughout the construction period.

The reservoir was in operation during construction. Access for the operational staff was maintained during the works. Given the limited working area, careful planning was carried out to ensure that site activities did not obstruct access to the outlet screen, the remaining part of the crest, the overflow weir or any operational valves.

In order to confirm the assumptions such as grout mix and grout pressures, trial grouting was carried out. Injection data such as grout volume and grout flow rates were extracted from the pump module, which allowed monitoring and confirmation of the effectiveness of grouting.

Cement bentonite grout was tested in the trial grouting initially as it is a more economic option. However, the volume of grout take at each sleeve was much lower than the targeted volume. Therefore, a microfine cement grout mix was also tested, which generally allowed a higher grout injection volume, indicating more effective permeation of the soil in the leakage zone. It was concluded that microfine cement grout would ensure better results hence it was used in the grouting works.

Grouting was carried out on the embankment, starting firstly with the Phase 2 (i.e. Injection of Primary TaMs of the first row as shown on Figure 5). In each phase, the downstream row of grout holes was grouted first, followed by the upstream row. The aim was to allow grout injected on the upstream row to flow into any gaps between the zones of grout injection on the downstream row (CIRIA, 2018).

To maximise the efficiency of the works, all the TaM pipes were installed prior to injection of the microfine cement grout. This allowed the drilling rig to operate in a systematic sequence in the constrained workspace.

The grout was injected from the bottom of the TaM pipe, progressing upwards with each sleeve in turn. The grout volume, average flow rate and flow rate at termination were recorded for subsequent review. The grout injection parameters were also recorded and graphed against time using the computer-controlled grout injection pumps. This allowed careful monitoring of grout takes, pressures and flow rates against the depth/zone being injected.

The target injection pressure was limited to soil overburden pressure during injection. Grouting was carried out at this target pressure at each port, until the termination criteria, either flow rate of less than 2 litres per minute or total grout take of 100 litres was reached.

Following the completion of daily grouting work cycle, grout data saved in the pump module was extracted and subsequently fed into a 3D model. Graphical output from the 3D model was generated to present the injection parameters at the as-built locations of each grout port. This allowed daily recording and monitoring of grouting parameters as the work progressed. It also facilitated the effective use of the observational approach, in which regular reviews of the grouting data was used to determine the extent of the subsequent grout injections. Figure 8 shows the graphical output of the 3D model which presents grout volume at each TaM sleeve.

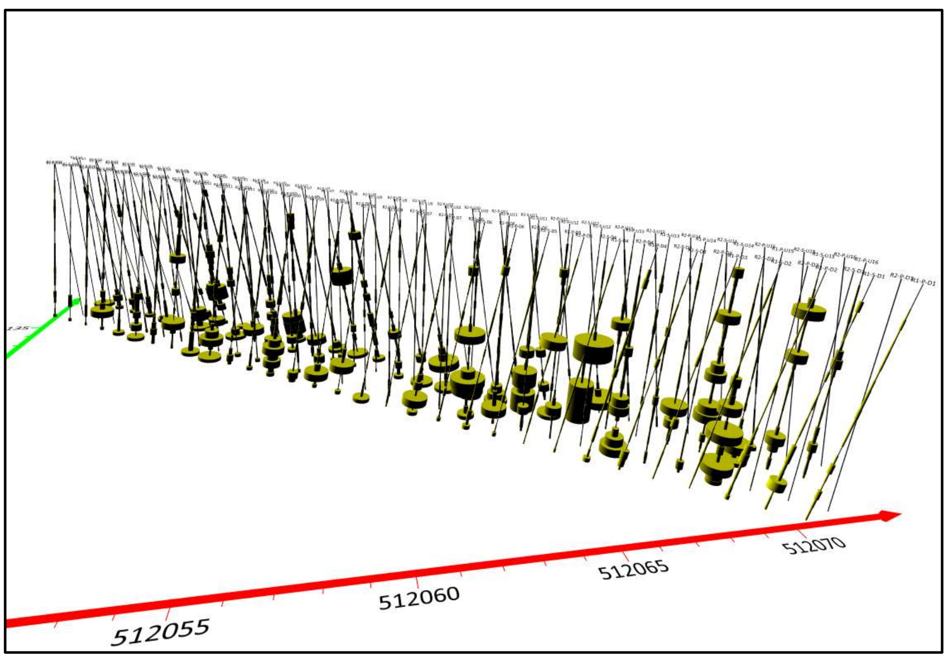

Figure 8. Graphical output from 3D model showing volume of grout take at each port

A swift decision was required to meet the construction programme as the next phase of grouting was determined based on the available data from previous injections. An efficient communication chain was established between the QCE, contractors and the designer's site representative. In order to facilitate communication, grout data and findings were shared to the wider project team after each grouting cycle, usually on a daily basis. The findings were also discussed during frequent meetings (twice a week) and emails which allowed collaborative decision making between the QCE and the geotechnical contractor.

During the Phase 2 grouting in the primary grout holes, high volumes of grout take and flow rates were observed at the interface between the puddle clay core and London Clay where leakage was found. Fissures at the top of Weathered London Clay could have contributed to the high injection volumes at those levels.

After the Phase 2 grouting and in discussion with the QCE, it was decided that more grouting was required due to high volumes of grout take. Therefore, the remaining three phases of grouting were carried out sequentially. Grout data at each sleeve was monitored throughout each grouting phase.

Generally, high injection volume was observed in the leakage zone in all four phases of grouting. However, there was an obvious trend of decreasing injection volume in each sleeve as the grouting works advanced. After the completion of all four phases, there was a small number of sleeves where high injection volume was recorded. Additional reinjection was conducted in the selected sleeves where both high grout volume and high flow rates were observed. The volume of grout take in the regrouted sleeves was small (<10L per sleeve). It was then considered that no further grouting would be practical or required.

The construction lasted for approximately three months starting from mid-August 2023. In total 112 no. grout holes were constructed along the 30m long leakage zone. The total grout injection volume using the TaM system was approximately 27m^3. The average volume of grout take per metre (length along the chainage) was 0.9m^3.

A post construction geophysical seepage survey was carried out in November 2023 as a 'compare' investigation to identify effectiveness of the remediation works. The results showed that leakage path through the dam has been successfully stemmed by the grouting works.

CONTRACTUAL ARRANGEMENT

The grouting works consisted of five phases in which the first two phases were fixed scope of works. This was the minimum grouting works that the contractor was requested to carry out. The remaining three phases of grouting would depend on grout injection data from the prior phase. On this basis, a lump sum cost was defined for the first two phases of works in the contract. Grouting works for the remaining phases were re-measurable based on actual injection volumes and number of grouted holes.

Early input from the geotechnical contractor was essential in the tender design stage as it helped minimise risks and aid constructability. Although the grouting design was carried out by the designer, it happened in a collaborative manner with the technical advice from the geotechnical contractor being incorporated in the construction package.

CONCLUSION

A potential leakage problem at the Hampton Distributing Reservoir was identified by a review of settlement monitoring data. The investigation was followed up with a geophysical seepage survey which identified a distinct leakage path through the embankment dam. During the investigation phase, a combination of geophysical seepage survey and ground investigation was helpful to confirm the extent/location of leakage. The leakage path could have developed further overtime and led to dam failure due to internal erosion. In 2023, permeation grouting was carried out within the 30m long leakage zone which successfully arrested leakage through the dam.

When challenging constraints such as difficult access, limited working space and ecological sensitivity are encountered on a site, cut-off wall solutions may not be practicable due to their disruptive nature and significant enabling works required. In these situations, grouting is a proven method which works well at small reservoir sites, especially where heavy machinery and large lay-down area are not allowed. Grouting could also provide a cost-effective solution and reduce the carbon footprint of the project, as it does not require significant enabling works.

Identification of the key seepage paths allowed an effective grouting solution to be planned. Analysis of the grout injection data through daily 3-dimensional modelling, allowed the observational method to be used to identify and target the key seepage paths. The rapid assimilation and visualisation of the grouting data allowed all parties to work as one team, with quick decision making that focused the grouting in the zones where it was most required. This focused approach contributed to an effective use of grouting, minimising the costs and allowing the works to be completed within programme.

ACKNOWLEDGEMENT

The project was overseen by Dr Andy Hughes from Dams and Reservoirs Ltd. who was appointed by Thames Water as the Qualified Civil Engineer for the works at the Hampton Distributing Reservoir. The support and direction by Dr Hughes to the successful delivery of the project is greatly appreciated.

REFERENCE

CIRIA (2018). *Grouting for reservoir dams - a good guide for practice (C774)*. CIRIA, London