Monitoring embankment performance during the raising of Abberton Reservoir

G. HIRD, MWH, High Wycombe, UK
R. MORRIS, MWH, High Wycombe, UK
I. CARTER, MWH, High Wycombe, UK

SYNOPSIS. The raising of the main dam at Abberton is one of the key elements of the Abberton Reservoir Enhancement Project, which aims to provide additional water for the Essex Supply Area and Greater London.

The original dam embankment was completed in 1938 and was 15m high and 680m in length. The dam was raised by 3.2m during the enhancement project by adding 300,000m³ of locally won fill material over the crest and downstream shoulder of the original embankment. The reservoir remained fully operational throughout the operation.

A large section of the upstream shoulder of the original dam embankment failed during construction in the late 1930s. An extensive monitoring system was therefore installed in advance of the main earthworks raising in order to monitor the dam embankment, evaluate its performance in comparison to FEA model predictions and thereby enable the raising works to take place in a controlled and safe manner.

The paper describes the monitoring system and the observed performance during construction. The paper also discusses the benefits of a remote monitoring system and provides an insight into some of the lessons learnt from maintaining instrumentation during a major earthworks project.

INTRODUCTION

A perceived shortfall between existing supply and future demand prompted Essex Suffolk & Water, the southern operating arm of Northumbrian Water Ltd, the reservoir owner, to investigate water resource scheme options during the 1990s. This study culminated in a decision to increase the capacity of Abberton Reservoir and upgrade the existing Ely-Ouse to Essex Transfer Scheme, which passes water from Norfolk to Essex.

Abberton Reservoir is located in relatively flat terrain about 4km southwest of Colchester in Essex. The full supply level is to be raised by 3.2m, which will increase its capacity by nearly 60% to 41Mm³. The project is very large

Dams: Engineering in a Social & Environmental Context. Thomas Telford, London, 2012

in scope and the engineering of the various elements has been strongly influenced by the reservoir's existing status as a RAMSAR site, a Site of Special Scientific Interest (SSSI) and Special Protection Area (SPA). Abberton Reservoir is also of strategic importance to the owner and the complexity of the project was further influenced by the need to maintain the reservoir as a live asset throughout the raising works. The reservoir has three distinct elements, which are separated by road causeways, as shown in Figure 1.

Figure 1. General Layout of Abberton Reservoir

The main dam is located at the northern end of the eastern section. The natural inflow is via Layer Brook, which enters the western section. The central and east (main) sections were originally connected via a culvert which passes through the causeway embankment but this was closed as part of the raising works and the causeway transformed into a true embankment dam. In future, the central and main sections of the reservoir will act independently. Only the eastern (main) compartment will be raised and it will operate as a non-impounding reservoir with water pumped into it from the River Stour and other sources.

Ancillary structures including the offtake pumping station (which supplies the nearby treatment works), valve tower and the overflow are all to be raised in line with the new full supply level. Four new col (saddle) dams have also been constructed along the southern side of the reservoir where the shoreline approaches the natural watershed.

The construction contract for the dam raising and appurtenant works commenced in January 2010 and is due for completion in spring 2013.

PROBLEMS DURING ORIGINAL CONSTRUCTION

The original dam embankment was constructed between 1936 and 1938. The crest was 680m long and stood at a nominal level of 19.6mOD, some 15m above the valley floor. The embankment has a central puddle clay core, which is flanked by selected clay fill and outer shoulders of granular fill.

The overflow structure, which in local terminology is referred to as the *swallow hole*, is situated towards the right abutment, close to the southern edge of the original floodplain. The swallow hole connects to a 110m long culvert which passes through the embankment and discharges to the natural water course downstream of the dam. A valve tower directly upstream of the swallow hole houses the reservoir's bottom outlet.

The upstream face is protected by concrete block-work and the downstream shoulder is grassed and slopes down from the crest at 1 on 3 to a sizeable mid-berm, below which it slackens to 1 on 4. A large gravel filled vertical wall drain is located within the downstream shoulder while at each abutment the puddle-clay core and cut-off gives way to a concrete core wall.

Slope failure during original construction

In 1937 the original embankment failed when it was within 2m of its intended final height. The failure occurred to the west of the valve tower over a length of 175m. The upper part of the core was displaced and the crest dropped by up to 3.5m while the upstream toe moved out by some 15m (French *et al*, 2000).

Extensive remedial works were undertaken to remove the slip and rebuild the embankment on an upstream slope which progressively slackens from 1 on 7 to 1 on 11. The upstream shoulder was also supported by a 2.5m high mass concrete capping beam to a sheet pile wall installed along the upstream toe in the valley bottom.

Subsequent studies have been undertaken to analyse the cause of failure. French *et al* (2000) considered two possible failure mechanisms; a non-circular mechanism running through the Alluvium and a deeper failure mechanism running through the top of the London Clay. The study concluded that the most likely location for the slip surface was through the top of the London Clay and analysis suggested that the failure occurred because excess pore pressures developed within the clay fill and foundation.

DESIGN OF NEW RAISED DAM

Draining the reservoir for the duration of the main dam raising works was not an option because of the need to maintain the reservoir in full operation at all times. The only available option was therefore to raise the crest and downstream shoulder and move the axis of the dam downstream. The new crest is to be raised by 3.2m and the downstream toe will extend northwards by up to 50m. The existing toe drain system has been extended out to the new toe and the underlying alluvium in the valley bottom has been improved by a network of over 1300 wick drains.

The new downstream shoulder has a bermed profile with the lower, outer section formed in dry, cohesive fill with a 1 on 5 slope and an inner section formed by granular fill at 1 on 3 leading up the finished crest level of 22.70mOD. The existing crest was reduced to 16mOD (i.e. by about 3m) prior to raising the dam in order to secure a robust watertight connection between the existing puddle clay core and 'selected clay' fill and the new rolled clay core material.

The downstream side of the rolled clay core is protected by a composite wall drain comprising fine and coarse filters. A coarse filter was also laid on the face of the original downstream shoulder and both systems connect to a composite drainage blanket laid beneath the new downstream shoulder which discharges into a new toe drain pumping station. The upstream face of the raised embankment has been protected by open stone asphalt (OSA) capped by *Ronacoast*, a resin bound shingle material.

New Instrument Monitoring System

A new comprehensive monitoring system was designed to replace the existing ad-hoc system which has been installed during multiple phases of investigations carried out over the preceding twelve year period. However, eight 19mm diameter standpipe piezometers installed along the centre line of the 1937 failure were converted for remote monitoring and included in the new monitoring system in order to provide continuity between the old and new systems. All remaining instruments were decommissioned.

The new system comprises a fully automated network of remotely monitored vibrating wire piezometers, in-place inclinometers and toe drain flow monitoring system, which are supplemented by a series of manually monitored conventional instruments, including inclinometers, magnetic settlement gauges and survey markers.

Layout

The new monitoring system is arranged on three main instrumented sections, which are located in the valley bottom alongside the outlet culvert (V-V), near the maximum section (U-U) and on the left flank in the vicinity of solifluction lobes (S-S), as shown on Figure 2.

Each instrumented section is orientated north-south through the axis of the dam and has five distinct instrument clusters distributed between the upstream shoulder of the original dam and the downstream toe of the raised dam. A summary of the instrument cluster locations relative to the embankment geometry is detailed in Table 1 and is shown on a typical section through the embankment on Figure 3.

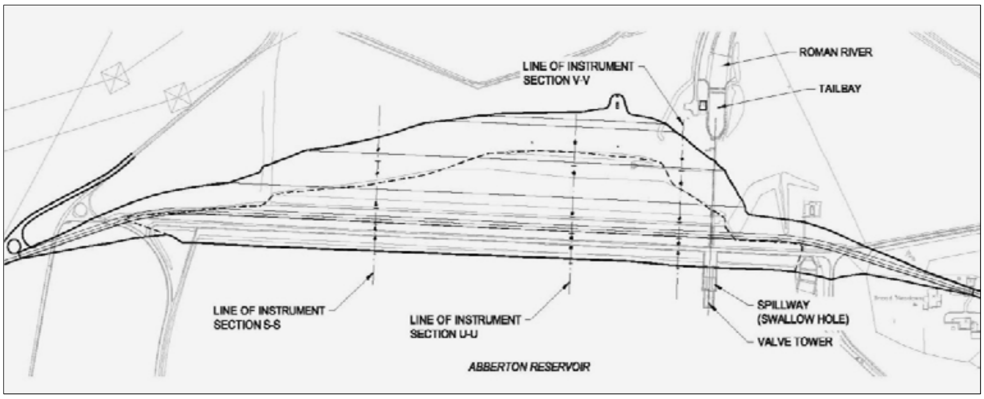

Figure 2 – Layout of instrumented sections at the main dam

Pore-water pressures within the embankment and its foundation are monitored by sixty-five vibrating wire piezometers (VWP) strategically distributed throughout the instrument clusters. Each VWP is connected via armoured cabling to a datalogger located at the downstream toe which enables all data to be transmitted via a GPRS datacard to a data management, calculation and presentation website known as *ARGUS*.

Table 1. Instrument Cluster Locations

Instrument Cluster	Typical Location
1	Immediately upstream of the original puddle clay core
2	Immediately downstream of the original puddle clay core
3	Mid-berm of original embankment
4	Downstream toe of original embankment
5	Within enlarged footprint of raised embankment

The *ARGUS* system, hosted by *Soil Instruments Ltd*, translates raw signal into engineering units and presents the data on a site plan backdrop on a password protected website. Each sensor, displayed as a ‘sensor box’ on the plan, displays the latest measurement result and the current alarm level status which is calculated as a percentage of the threshold level for each construction increment modelled in the finite element analysis. In the event that an alarm level threshold is exceeded then an automated alarm is raised to the designated users via email and SMS messaging.

Each sensor, which is updated at six hourly intervals, can be interrogated online to produce 10-day trend plots, or the data can be downloaded by the end-user for analysis in Microsoft Excel. Horizontal movements are monitored via a series on three In-Place Inclinometers (IPI) which are located below future top water level on the upstream face and twelve standard inclinometers located across the downstream shoulder.

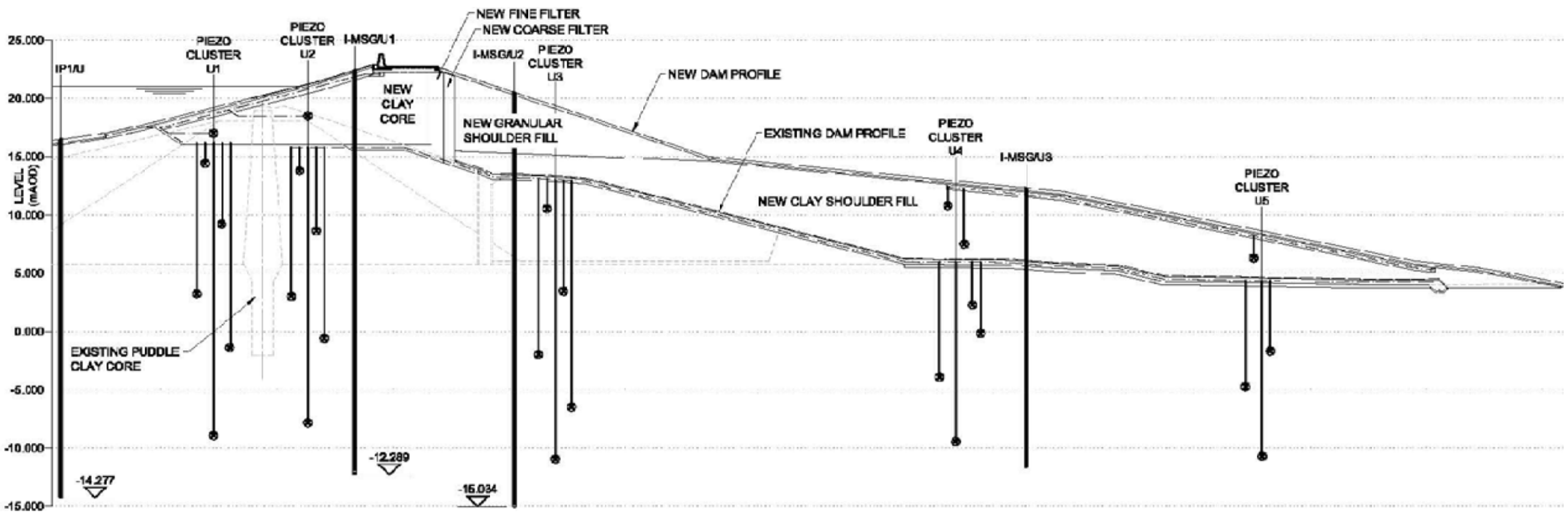

Figure 3 . Cross section through instrumented section U-U

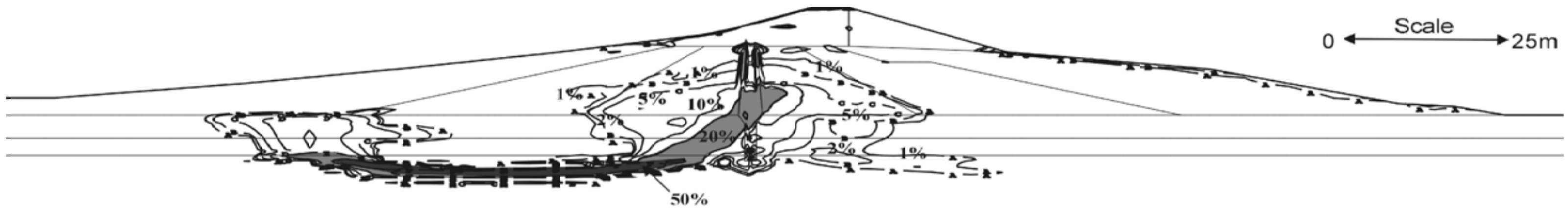

Figure 4 . FEA showing plastic shear strain developed during back-analysis of original failure (GCG, 2009)

Each IPI installation, comprising a string of multiple IPI gauges installed at 2m centres, is connected to the *ARGUS* system via the datalogger to enable continuous remote monitoring of the upstream face of the dam. The standard inclinometers are monitored manually.

Settlement is measured by a series of eleven manually monitored Magnetic Settlement Gauges (MSG) installed across the downstream shoulder of the dam and a network of topographical survey monuments.

FINITE ELEMENT ANALYSIS

A comprehensive finite element analysis (FEA) of the raised embankment design was carried out during the detailed design stage by *Geotechnical Consulting Group* (GCG) (GCG, 2009 and Kovacevic *et al*, 2010). It analysed the original 1937 failure, the performance of the re-constructed embankment and the suitability of the raised embankment design.

The analysis looked at two main sections through the embankment which coincide with instrumented sections. So that the behaviour of the failed part of the original embankment could be further investigated and assessed, instrument section U-U was modelled. Section S-S, was chosen in order to assess the impact of excavations at the toe required to remove soliflucted deposits from the enlarged footprint of the raised dam.

A detailed review of the ground investigation data was carried out by GCG in 2009 to enable an assessment of material properties and the derivation of model parameters to be made to ensure that the correct soil models were adopted in the analysis. A post-peak strain-softening constitutive model of the Mohr-Coulomb type was adopted to examine the potential for progressive failure in the London Clay, while the Mohr-Coulomb model, without strain-softening, was adopted for the granular fill. An elasto-plastic model of the critical state type (modified Cam Clay) was used to characterise the pre-peak plastic behaviour of the clay fill used to construct the selected clay and puddle clay core fill in the original dam and the rolled clay fill used to raise the embankment. The modified Cam Clay model was adopted due to the destruction of the fabric of the *in situ* clay caused by the process of excavating, remoulding, placing and compacting the material.

The FEA succeeded in correctly predicting the failure of the original construction, both in terms of the construction level and timing. It also identified that the failure was deep seated and indicated that the rupture surface most probably developed about 3m into the London Clay. It probably occurred at this depth because of consolidation and strength gain in the uppermost layers due to vertical drainage from the London Clay into the overlying silty Alluvium, which is significantly more permeable.

An extract from GCGs report (2009), reproduced in Figure 4 shows the distribution of the plastic shear strain generated immediately prior to failure of the original embankment.

The successful back-analysis of the original failure helped to calibrate the model and confirm that the FEA had adopted appropriate parameters. The construction sequence for the embankment raising and beyond was modelled using increments based on a series of key milestone dates that were enshrined within the construction contract to ensure that an appropriate rate of fill-raising was maintained throughout the works. The operational cycle of the reservoir and the post-construction stage of the raised embankment design life were also modelled.

An extended period of inclement weather towards the end of 2010 and early 2011 meant that construction at the main dam was some three months behind programme. To mitigate this delay, the contractor proposed an expedited construction programme resulting in the key milestone dates being revised to enable an increase rate of construction and enable completion of the bulk filling by August 2011, prior to the removal of the embankment crest which had been programmed to coincide with a period of contractually enforced reduced reservoir levels.

To ensure that the increased rate of construction had no detrimental impact, the FEA was re-run and the revised output used to provide a benchmark against which the embankment performance during construction could be monitored. The output was used to generate a series of plots for each instrument against which the observed data was compared.

MONITORING DURING CONSTRUCTION

Due to the historical problems associated with the main dam coupled with the strategic importance of the asset to the owner, a full-time supervisor was employed to monitor the performance of the embankment throughout the duration of the contract. The monitoring regime adopted throughout construction is summarised in Table 2.

Table 2. Instrument Monitoring Frequency

Instrument Type	Monitoring Frequency
Vibrating Wire Piezometer	Continuously via *ARGUS*
In-Place Inclinometer	Continuously via *ARGUS*
Standard Inclinometers	Fortnightly
Magnetic Settlement Gauges	Fortnightly
Survey Markers	Weekly
Seepage (via toe drain pumping station)	Continuously

The main earthworks commenced in February 2011 and followed the expedited 'recovery' programme. 'Continuous' pore-water pressure measurements were used as the primary tool to control the rate of rise while the inclinometers and magnetic settlement gauges were used to confirm that no unwanted movements were occurring within the embankment.

The automated remote monitoring system monitored trends as the pore-water pressures developed in response to loading and the data was then downloaded from *ARGUS* and imported into Microsoft Excel to enable comparison with the FEA data to be undertaken to ensure that the embankment was behaving as expected.

This method worked well throughout and proved invaluable during the early stages of construction as the earthworks contractor rapidly increased his rate of fill placement in an attempt to recover lost time and to build in some additional programme float for later use. This resulted in the rapid generation of pore-water pressures in the alluvium beneath the new enlarged toe which rapidly approached the limits set out by the FEA. As a result the contractor was directed to reduce the rate of placement to an acceptable level with the corresponding dissipation in pore-water pressures recorded by the instruments.

The results of the analyses clearly demonstrated that the pore-water pressures monitored during construction closely followed those profiles generated by the FEA, although it was found that the actual data was typically around 30kPa lower than predicted. The cause for this over-estimation of predicted pore-water pressures was thought to be variations in the bulk permeability of the embankment fill and its foundations not picked up in the model. An extract from the pore-water pressure monitoring relative to rate of fill placement and FEA data, which demonstrates the accuracy of the trend prediction, is presented in Figure 5. It should be noted that the deviation from predicted shown after the end of phase 1 core raising is the result of no further fill placement having taken place by the time this paper was produced (March 2012) due to problems caused by inclement weather. Further increases in pore-water pressure are expected once fill placement at the dam restarts in the coming weeks.

All VWPs installed at the main dam have survived the main earthworks programme so far and continue to provide continuous 'real-time' data on the performance of the embankment. Protection of the equipment, during and immediately after installation was of paramount importance and careful programming of the works by the main Contractor and his earthworks subcontractor ensured that damage was kept to an absolute minimum.

Monitoring of the VWPs in the interim stage between installation and the commissioning of the *ARGUS* system was carried out on a manual basis using a hand-held readout unit which was a labour intensive and time consuming activity. Monitoring of the same network, complete with data interpretation, using the automated system now takes a fraction of the time. The continuous data will enable a better understanding of trends associated with reservoir level.

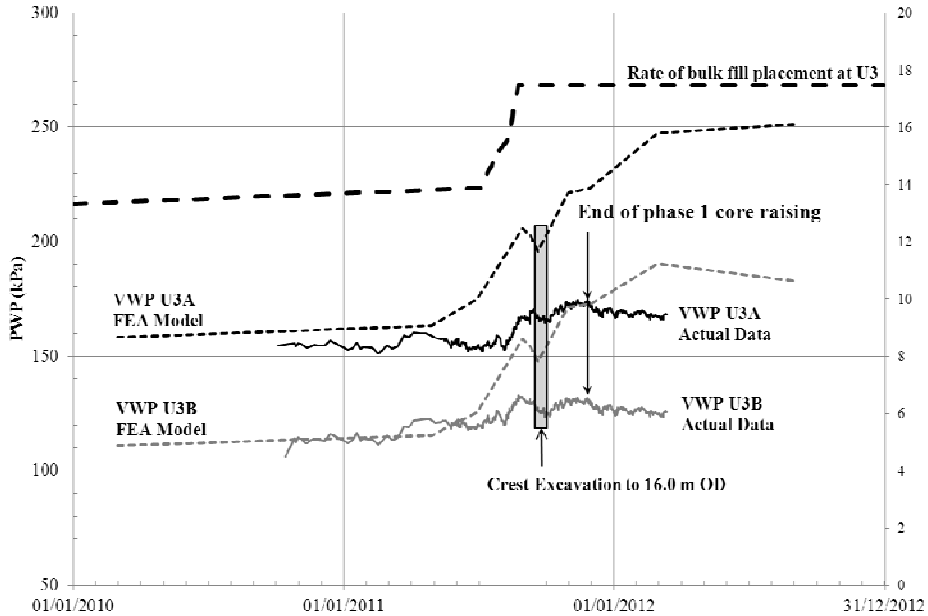

Figure 5. Actual pore-water pressure v FEA predictions

The use of in-place inclinometers at Abberton has proven to be a major success with the instruments providing continuous monitoring of the upstream face. Installation of the instruments did not take place until summer 2011, once the reservoir level had reduced to an acceptable level, meaning that early comparison with the FE data was not possible. Complications during commissioning, coupled with some damage to the signal cables, meant that this part of the automated system was delayed until September 2011. Installation was completed in time to enable monitoring of the crest removal, core reconstruction and winter reservoir recharge to be undertaken, all of which performed in line with the level of movement predicted by the FEA.

In contrast to the success of the in-place inclinometers, the monitoring and maintenance of standard inclinometers during a major earthmoving contract has proved to be a challenge.

The use of standard inclinometers to monitor horizontal movements within the embankment dams is normal practice, however obtaining good quality and reliable data with a high level of repeatability in the middle of a major earthworks contract is exceedingly difficult. All installations have remained intact (just), although monitoring has been beset with difficulties associated with issues such as the raising of access tubing, contamination of the keyways and human error making interpretation of the data a complicated procedure with actual movement sometimes masked by noise. The authors believe that many of these problematic issues would be nullified by the more widespread use of in-place inclinometers.

The new automated toe drain system, which runs along the length of the new downstream toe, collects seepage waters that may pass through or under the dam. A network of pipes conveys flow to a pumping station located in the valley bottom. A series of thirteen inspection chambers evenly spaced along the length of the main carrier drain enable maintenance, inspection and manual measurement of individual flows to be made.

The toe drain pumping station comprises two chambers separated by a V-notch weir which enables inflows to be measured via an ultrasonic sensor. Flows pass over the V-notch before being discharged via one of two (duty and stand-by) automatic bowl pumps into the tailbay channel.

All data is transmitted to an HMI control panel housed in a nearby building which enables real-time flow data to be observed. All data from the HMI is automatically transmitted via a fibre optic link to the main control room at the nearby works where controllers can react to any alarm events. All data is uploaded to the owners' internal database from which the data is retrieved and compared against rainfall and reservoir data.

CONCLUSION

The monitoring system installed in the raised Abberton dam comprises a fully automated network of remotely monitored vibrating wire piezometers, in-place inclinometers and toe drain monitoring system. The automated remote monitoring system, known as *ARGUS* has enabled continuous remote monitoring, which when coupled with more traditional instruments and a comprehensive FE analysis has proven invaluable in controlling the rate of fill placement while ensuring that the embankment continues to perform as expected.

The acquisition of reliable, high quality data on soil displacement in a live earthmoving environment using traditional inclinometers has proved to be very difficult. Despite the obvious short-term cost implications associated with procurement, installation and set-up of a remotely monitored network, it is felt that the resulting high quality data, coupled with the long term reduction in time consuming monitoring costs would be well worth the extra investment.

REFERENCES

French, Woolgar & Saynor (2000). Geotechnical Investigations at Abberton Dam, Essex. *Dams 2000 – Proceedings 11th Conference British Dam Society, Bath*, pp. 345 – 359. Thomas Telford, London.

Kovacevic N, Hight D, Potts D & Carter IC (2012). *Finite element analysis of the main dam embankment at Abberton Reservoir, Essex.* Geotechnique (in review).

Geotechnical Consulting Group (2009). *Abberton Reservoir, Finite Element Analysis of Main Dam, Final Report.*

Geotechnical Consulting Group (2009). *Abberton Reservoir, Soil Parameters, Final Report.*