The Performance of Thika Dam, Kenya

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SYNOPSIS. Thika Dam is situated on the Thika River about 60km north of Nairobi in the foothills of the Aberdare Mountains where the river bed is 1985m above mean sea level (AMSL). The dam forms a major element of the Third Nairobi Water Supply Project which was constructed between 1990 and 1995.

The dam incorporated a range of instrumentation and monitoring systems. The data from the instrumentation was collected regularly for 5 years after construction with periodic reviews of the dam performance.

The purpose of the paper is to show that the dam constructed from halloysitic clay has performed satisfactorily and to compare the behaviour with dams constructed from conventional materials.

INTRODUCTION

Thika Dam is a 70m high earthfill embankment constructed entirely from a residual soil rich in halloysite. The unusual behaviour of residual soil rich in halloysite has been discussed by Terzaghi (1958) in relation to the construction and performance of Sasumua Dam in Kenya. Thika Dam incorporated vibrating wire piezometers, total pressure cells, inclinometers and settlement gauges. Surface monuments were installed on the embankment to monitor surface movements and seepage was collected in a measurement chamber at the downstream toe. The dam is shown in Figure 1

This paper examines the performance of the dam over the first 5 years of operation. The post construction settlement behaviour is examined and compared with settlement data from other dams. Results of seepage measurements from the drainage blanket are discussed and the pore pressure response of the upstream shoulder is also discussed. Comparisons are made with design stage predictions and data from other dams.

Figure 1 – Thika Dam, Kenya

GEOLOGY

The rocks underlying the area are of Pleistocene age and are of volcanic origin being predominantly Pyroclastic tuffs. Two origins of these tuffs can be recognised:

- Pyroclastic flows consisting of fragments of rock dispersed in a medium of fluidised fine material.
- Pyroclastic falls from material thrown into the air by the volcanic explosion.

The remainder of the volcanic sequence comprises flows of phonolite lava. Lavas represent the height of volcanic activity with eruptions occurring from localised vents. Their deposition was sometimes accompanied by air fall activity and thus the phonolite may be found either as massive units or interbedded with the tuffs.

Six periods of volcanic activity can be recognized. The end of each deposition period is marked by a weathered horizon at the top of the sequence. The presence of these residual soil horizons indicates ancient erosion surfaces which were subsequently covered by later volcanic deposits.

The modern drainage pattern has deeply dissected this volcanic sequence and a highly to completely weathered material covers the slopes. Outcrops of rock are restricted to small areas of very steep slope in the valley sides and to water falls formed in the valley floor where streams flow over the more resistant lavas.

DAM DESIGN

Embankment

The embankment is 450m long (curved in plan), 70m high and is constructed of residual volcanic soil. The dam is homogeneous with the exception that the upstream sloping core was placed at a higher moisture content than the shoulders, from which it is separated by a chimney drain. The higher moisture content in the core was intended to make the core sufficiently plastic to maintain high post construction total stresses. Lower moisture content in the shoulders was necessary to minimise construction pore pressures and hence maximise strength. A section through the embankment at maximum height is shown in Figure 2.

During construction higher than expected construction pore pressures were experienced in the downstream shoulder. To ensure construction stage stability 125 vertical sand drains, average depth 15m and 4 drainage blankets, at 5m vertical intervals, were installed in the downstream shoulder. A 1:10 toe weight was constructed from reject material at the upstream toe.

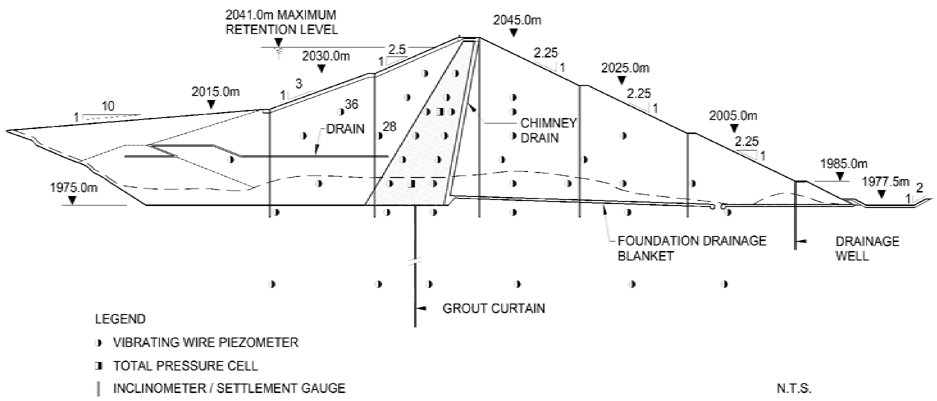

Figure 2 – Cross Section at Maximum Height

Drainage

Seepage through the core and foundation is collected by the chimney drain and a foundation drainage blanket. On the steep abutments the drainage blanket was replaced with a series of finger drains. Further drainage measures were provided by a line of drainage wells along the downstream toe of the valley section of the dam, and by two drainage adits, one in each abutment.

Foundation Treatment

The embankment was founded primarily on residual soil, with typically 3 m of stripping to remove all organic material and provide a suitable profile for filling. On the line of the original river bed, about Chainage (Ch) 290, the embankment was founded on Grade III Lapilli tuff which occurred at a convenient level.

A grout curtain was provided to limit the seepage through the moderately permeable foundations (Lugeon values in the range 5 to 50). The grout curtain was constructed by means of jet grouting in the upper part and by conventional grouting in the lower part. The decision to incorporate a grout curtain through the residual soil was influenced by the lack of precedent for founding a 70m high embankment on up to 35m of residual soil without positive foundation treatment.

This decision was vindicated during construction when an underground cavern, with a volume of at least $8m^3$ was encountered on the right abutment of the upstream shoulder. Similar caverns were exposed in the borrow area upstream of the dam and appeared to occur at depths of up to 10m. The jet grouting of the upper part of the foundation provided security against the possible inter-linking of such caverns.

Further information on the jet grouted cut-off is provided in Attewill et al (1992) and Attewill and Morey (1994)

Instrumentation

The dam was instrumented primarily at three sections; Ch 120, Ch 200 and Ch 290. The section at Ch 290 is at the maximum embankment section, is the most comprehensively instrumented and is shown in Figure 2.

Instrumentation comprised the following:
- 92 vibrating wire piezometers (embankment and foundation)
- 5 inclinometer / settlement gauges

- 26 survey monuments
- 3 total pressure cell arrays (5 in each array)
- 33 observation wells in the abutments
- 18 double installation observation wells along the rim of the reservoir

A number of instruments have failed over the years; 50 piezometers and one total pressure cell array remain functional. Possible causes of these failures were ineffective cable joints and horizontal strain in the embankment. Joins in cables were carried out using proprietary epoxy jointing kits but these may not have been effective in all cases. The cables also passed through materials with different stiffness and a horizontal displacement of up to 330mm was observed in the deepest inclinometer. Despite snaking the cables during installation and the use of a special cable with large strain properties, the strain in some of the cables may have exceeded the failure strain.

Seepage was measured manually by means of V-notch weirs in a seepage measurement chamber which collected flows from the blanket drain.

Material Properties

The material used to construct the dam was predominantly the red to reddish-brown residual soil which formed a mantle up to 6m deep over the borrow area. The results of X-ray diffraction analysis indicated a halloysite content of 60 – 65%. Terzaghi (1958) and Wesley (1973) have noted that halloysitic rich clays exhibit abnormal properties in comparison with sedimentary clays from more temperate regions.

The plasticity index is much lower than that of a sedimentary clay with equal liquid limit. The angle of internal friction and permeability are higher and the compressibility lower than the corresponding properties for a clay with equal liquid limit. Irreversible changes also take place on drying and affect the Atterberg limits, particle size tests and compaction test results. Terzaghi attributed this abnormal behaviour to the clay fraction occurring in clusters or aggregates rather than as individual particles. Moreover, water is located in the voids between the clusters and in their solid structure. The water in the solid structure is inert and has no influence on the mechanical behaviour (Geological Society (1997)).

The Atterberg limits plotted well below the A – Line on Unified Soil Classification System plasticity chart with liquid limit in the range 80% to 100% and Plasticity Index in the range 30% - 40%.

The peak effective stress parameters, as measured in isotropic consolidated triaxial tests with pore water pressure measurement, were
c' (apparent) = 10kPa and $\varphi' = 33^{o}$. These parameters were used in the design and were confirmed by laboratory tests carried out during construction.

The field dry density of the fill, depending on whether placed in the shoulder or the core, was in the range $1.1t/m^3$ to $1.2t/m^3$. The field water contents for the shoulder and core were 46% to 53% respectively with corresponding laboratory optimum water contents of 45.5% and 48%.

Further information on the material properties is given in Attewill and Bruggemann (1997).

DAM PERFORMANCE

Embankment construction commenced in October 1991 and was substantially completed in February 1994. The post construction performance of the embankment is discussed in the sections that follow.

Settlement

Settlement was monitored by plate magnets incorporated in the inclinometer installations and by surface monuments. Figure 3 shows the post construction settlement of the crest and the downstream berm at elevation 2025mAMSL. Crest settlements are shown for the surface monuments and the top plate magnet on the crest inclinometer / settlement gauge. The record for the surface monument is shorter than that for the plate magnet because the surface monuments were installed only after the crest road and wave wall construction was completed.

The cumulative settlement of the crest, as measured by the top magnet, since the end of construction was about 400mm or 0.6% of the maximum embankment height. The crest was provided with a 1m camber and thus the settlement was still within design provisions.

The top magnet at the crest shows that there was a slight increase in the rate of settlement about 60 days after the end of construction and since then, at an average rate of about 55mm/year. The value of the crest settlement index, proposed by Charles (1986), was estimated to be 0.003. The index was developed for puddle clay core dams and a range of 0.002 to 0.074 is given in Johnston et al (1999), nevertheless the index for Thika Dam is at the lower end of the range and suggests the dam's performance appears to be in keeping with other types of dam.

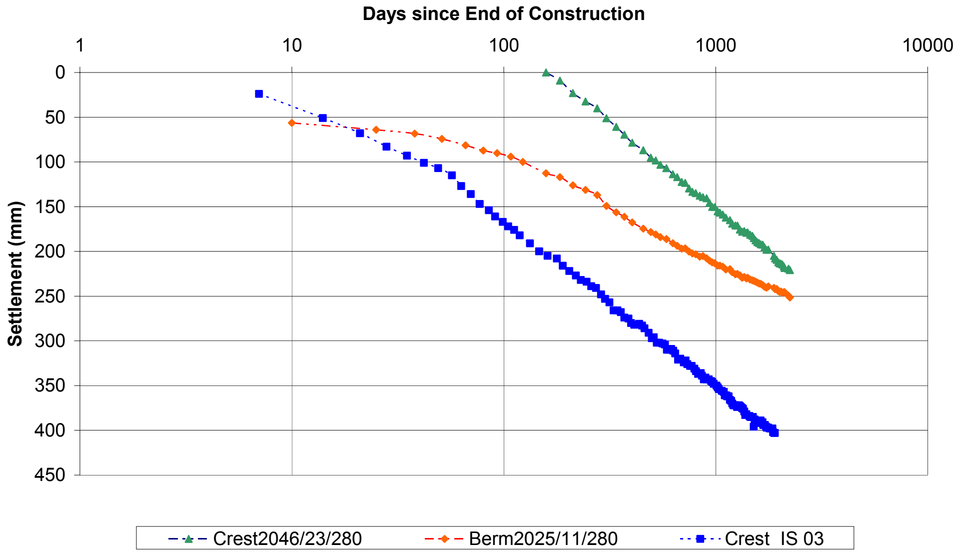

Figure 3 – Post Construction Settlement

The value of the drawdown settlement index, proposed by Johnston et al (1999), was estimated for drawdown events during the first 5 years. Five drawdown events provided a cumulative drawdown of 24m. The settlement associated with these drawdown events amounted to 38mm, yielding an index of 0.023mm/m^2. The index for the individual events varied from 0.013m/m^2 to 0.058mm/m^2. These values are generally towards the lower end of the range of values given by Johnston et al (1999) and suggest satisfactory performance.

Seepage

Seepage through the dam is collected by the chimney drain which is connected to the foundation drainage blanket. The foundation drainage blanket was divided at the valley bottom so that flows from each side of the valley were monitored by V notch weirs. The seepage flow is plotted against reservoir water level in Figure 4. The maximum flow from the left hand side was about 1,200litres/minute (20litres/second) and from the right about 600litres/minute (10litres/second) to give a total seepage from the drainage blanket of about 1,800litres/minute (30litres/second). The minimum compensation release required downstream was 15,000litres/minute (250litres/second) and seepage flow made a contribution to the compensation flow.

The seepage estimates made at design stage were based on conventional hand sketched flow nets with the foundation permeability ten times the embankment permeability and an impervious cut-off. The seepage was estimated at 17litres/second.

This seepage is about 60% of the maximum experienced during operation of the dam and reinforces the recommendation of Cedergren (1967) that drain designs should be based on liberal factors of safety. The magnitude of the seepage experienced in the field suggests that care needs to be exercised in the estimation of the foundation bulk permeability from permeability values determined during the ground investigation.

The data shows that seepage from the left hand side was twice that from the right hand side of the valley. This behaviour could be a result of the foundation conditions on the left abutment, where a layer of boulders was encountered in the area of the jet grouted cut-off. The interlocking of the jet grouted columns was unlikely to be as efficient as when installed in residual soil, as on the right abutment. The greater seepage from the left hand side supports Casagrande (1961) that small imperfections in a cut-off can have a major influence on the overall performance of a cut-off.

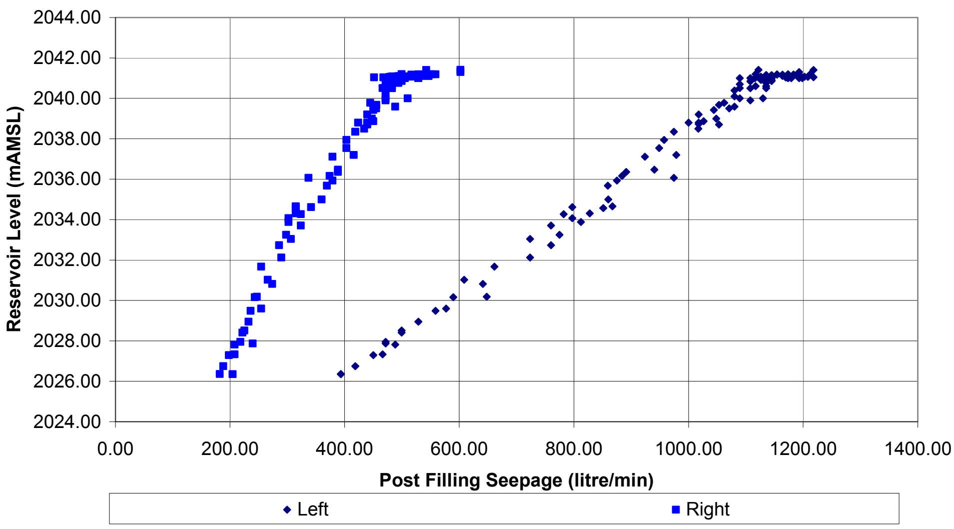

Figure 4 – Blanket Drain Seepage

Piezometer Response

The response of four vibrating wire piezometers situated in the upstream shoulder to an operational drawdown is illustrated in Figures 5 and 6. At the start of the drawdown all four piezometers measured pore pressure closely reflecting the water level in the reservoir. Although the fill material in both the shoulder and the core has relatively low permeability there is very little head drop in the upstream shoulder.

The drawdown took place between July 1998 and April 1999 from near top water level at 2040.8mAMSL to 2028.7mAMSL. The rate of drawdown initially averaged 0.04m/day but after reaching elevation 2034mAMSL it increased to around 0.1m/day.

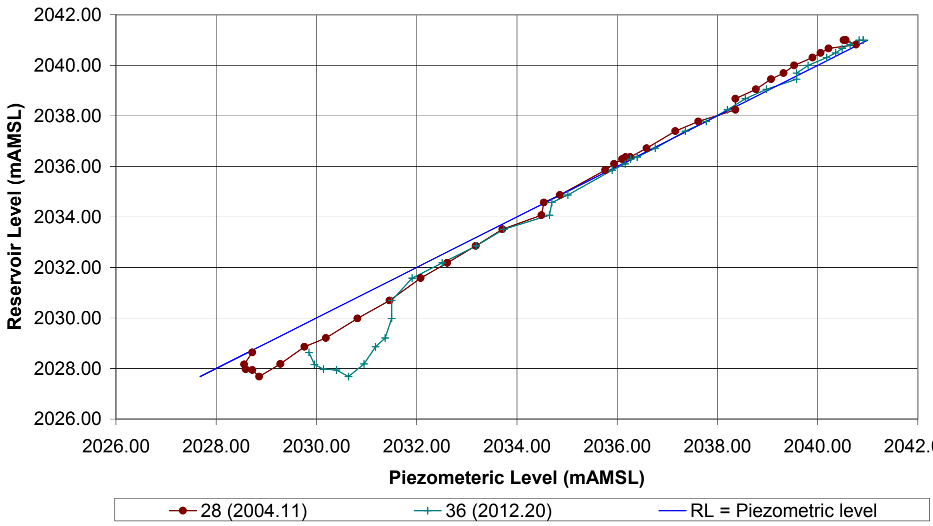

Figure 5 – Upstream Piezometer Response to Drawdown CH 290

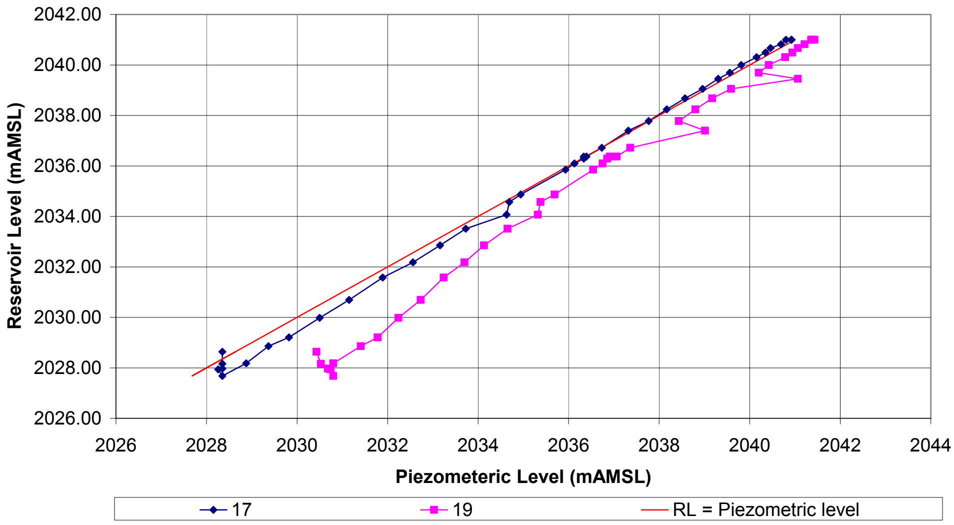

Figure 6 – Upstream Piezometer Response to Drawdown Ch 200

At the slower rate of 0.04m/day the pore pressure response of the four piezometers was similar with Bbar (Δu / $\Delta\sigma_v$) values around 0.95. This value of Bbar reflects efficient drainage with the pore pressure dropping at the same rate as the reservoir. When the drawdown rate accelerated, the response of the piezometers varied with Bbar values dropping to between 0.6 and 0.9. At these values of Bbar pore pressures will lag behind the reservoir level reduction and the factor of safety of the embankment against slope stability failure will reduce. There is no obvious reason for the varying values of Bbar solely as a result of the location of the individual piezometers. This variation is more likely to be a reflection of local variations in fill material and preferential drainage paths.

Two cases for drawdown were used in the original design analysis; emergency drawdown to elevation 2015mAMSL and operational drawdown to 2000mAMSL at drawdown rates of 0.42 and 0.11m/day respectively. The operational drawdown rate is comparable to that observed during drawdown in 1998/99. The r_u values at the end of the operational drawdown are given in the design report and varied at the piezometer locations from 0.2 to 0.4. To make a comparison between the 1998/99 drawdown and the design analysis the r_u values for the 1998/99 drawdown have been estimated assuming:

- Drawdown continues to elevation 2000mAMSL at a rate of 0.1m/day
- Bbar values remain unchanged below elevation 2028mAMSL
- No further dissipation of pore pressures occurs once the drawdown continues below the elevation of the piezometer tip

Using these assumptions the estimated r_u values range from 0.1 to 0.45. These correlate well with the design analysis and demonstrate that the embankment is behaving as predicted and will have an adequate factor of safety.

CONCLUSIONS

The paper has examined the post construction performance of Thika Dam with respect to settlement, seepage and pore water pressures.

The settlement data suggests that adequate allowance for post construction settlement was incorporated at design stage. The settlement indices determined from the settlement data were at the low end of the range published for UK dams.

The seepage flow at maximum retention level was about twice that estimated at design stage. The low seepage at design stage was probably a result of an under-estimate of the foundation bulk permeability from "point" values determined during the ground investigation. The efficiency of the cut-off may also have been reduced by the inclusion of minor imperfections. The need to design drains with liberal factors of safety was confirmed. The observed seepage was about 12% of the minimum required compensation flow and thus contributed to this requirement.

Piezometers in the upstream shoulder indicated that design assumptions of the pore water pressure response during drawdown were consistent with the observed response. This behaviour suggested that there was a satisfactory factor of safety during reservoir drawdown.

The Thika Dam embankment appears to be behaving satisfactorily.

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