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LONG-TERM BENEFITS AND PERFORMANCE OF DAMS

Chapter 5. Instrumentation and monitoring

Monitoring of dams in operation - a tool for emergencies and for evaluation of long-term safety

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SYNOPSIS. As the number of new dam projects dropped during the late 1980's, the focus shifted from construction to operation of dams in Norway. In this process, the importance of emergency warning and monitoring of long-term behaviour was realized and resulted in a guideline for monitoring and instrumentation in 1996. After 7 years, the guideline has been revised and this paper summaries the content of the revised guideline [NVE, 2003]. Some Norwegian dams have a dam break warning system and a brief history of this system is also described.

INTRODUCTION

Traditionally, monitoring of Norwegian dams has been limited to the initial filling and the first years after commissioning. Surveillance of long-term performance was generally not systematic and limited to random inspection of dams.

The need for monitoring of long-term performance was visualised when the guideline for inspection and reassessment was introduced in 1994 [NVE, 2002]. According to the guideline, a reassessment is required about every 15^{th} year, which includes a detailed evaluation of the dam and appurtenant structures. An element of the reassessment is an evaluation of the long-term performance of the dam, in order to compare the theoretical and the actual behaviour.

In 1994 a guideline on emergency action planning was also published [Svendsen, Molkersrød and Torblaa, 1997]. The guideline was a result of increasing focus on emergency planning and how to reduce the consequences related to an abnormal situation. It is evident that early warning is important to prevent worsening of the situation and to reduce the consequences.

The realization of the importance of instrumentation to monitor long-term behaviour and for the purpose of emergency warning resulted in a guideline for monitoring and instrumentation in 1996. During 2003, the guideline has been revised and this paper summaries the recommendations of the revised guideline. Some Norwegian dams also have a dam break warning systems, and a brief history related to this system is also included.

LEGAL FRAMEWORK

The practice of public supervision with dams in Norway started in 1909. After almost 100 years, the regulatory authority has been transferred to the Norwegian Water Resources and Energy Directorate (NVE) which reports to the Ministry of Petroleum and Energy.

The Dam safety regulations [NVE, 2000a] represent the legal basis for public supervision and safety control of dams, spillways and hydraulic structures (called watercourse structures as a collective term). More detailed specifications and technical safety recommendations are specified in guidelines. The structure of the regulations is given in figure 1, below.

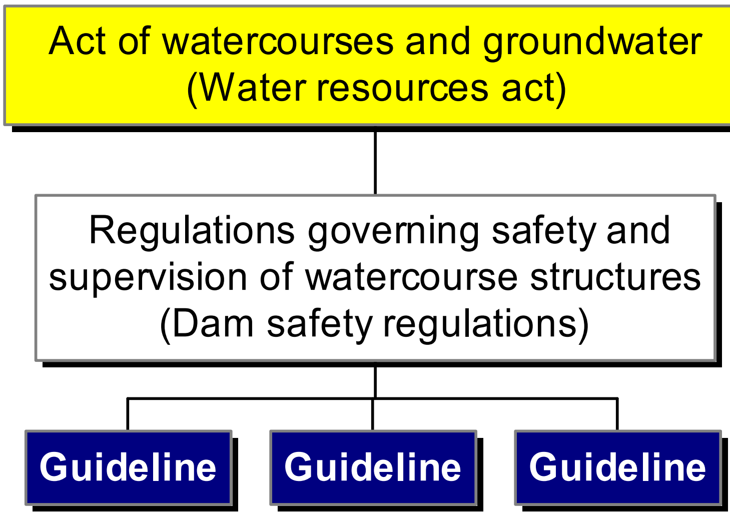

Figure 1: Structure of the legal framework

The guidelines give recommendations on how to fulfil the regulations but do not set any direct requirements. Guidelines are developed and managed by NVE and can easily be revised and amended. The Dam safety regulations and the Water resources act include general requirements and therefore needs a formal approval from the highest level in the administration, a process that is very time-consuming and complicated.

All together, 20 different guidelines are planned [Midttømme, 2003]. Eight of these have been issued, whereof two has been translated to English. The English versions of the regulations and guidelines are available on NVE's Internet site; http://www.nve.no-> English pages-> Safety and supervision-> Legislation

Classification of dams
The regulations define 3 different consequence classes [NVE, 2000b] and requirements for instrumentation are dependent on the classification of the dam. Each class is defined on the basis of the number of houses affected by a dam break, as shown in Table 1. Environmental and economical consequences shall also be assessed as an element of the classification.

Table 1: Classification of dams – definition

Consequence class		**Affected housing units**
Class 1	Low hazard	0
Class 2	Significant hazard	1-20
Class 3	High hazard	More than 20

PLAN FOR MONITORING

A plan for monitoring is important as a basis for the surveillance. A monitoring plan can be part of the inspection program for the dam, since an evaluation of the collected data often is an element of the inspection procedure.

The plan for monitoring will normally contain the following elements:

1. Overview of the different types of instrumentation on each dam.
2. Description of the different measurements that are being carried out and frequency of the readings.
3. Background on the choice of instrumentation or reasons for lack of instrumentation when this does not coincide with the guideline.
4. Description of the location of each individual monitoring device
5. Specifications on the different instruments.
6. Description of the accuracy of the instruments and expected errors in the recorded monitoring data.
7. Plan for calibration of the instruments where necessary.
8. Plan for testing, inspection and maintenance of the instruments.
9. Limit values to initiate actions in case of an emergency.

Limit values are important to give a warning in an emergency situation. The values are worked out as part of the Analytical Phase, which forms the basis for development of an Emergency Action Plan [Svendsen, Molkersrød and

Torblaa, 1997[3]]. Limit values will specify when to intervene and can for example be defined as;

- water level where access to the gatehouse or gate is interrupted
- water level when overtopping of the core will occur
- maximum expected load that the structure can withstand
- normal or largest acceptable leakage

MONITORING AND INSTRUMENTATION

Generally, the need for monitoring and instrumentation will depend on the dam type and consequence class. The number of instruments and their location must be assessed according to the dam type, height and length; the state of the foundation; normal reservoir levels, reservoir size and other factors in the reservoir area.

It is important that the instruments are reliable, accurate and easy to read off. Care must be taken when installing the instrument and the location must be chosen in order to ensure a correct and adequate reading of the monitoring data. For example, the water level sensor should not be located so that gates or spillways can influence the readings. This should be obvious, but experience show that it is not always taken into consideration, fore example in cases where the gates or spillways are seldom in use.

Recommendations for instrumentation of dams in Norway to monitor long-term behaviour and to provide emergency warning are given in the guideline for monitoring and instrumentation [NVE, 2003]. The guideline also specifies the frequency of the readings. Additional instrumentation and other frequencies than recommended by the guideline will need to be evaluated, dependent on the dam in question.

MONITORING OF LONG TERM BEHAVIOUR

Monitoring of long-term behaviour is generally limited to monitoring of the following elements:

- Leakage
- Pore pressure
- Deformation

Leakage

Variations in the recorded leakage will give an indication on the performance of the dam. Decreasing leakage can indicate increasing pore pressures as a result of poor drainage, while increasing leakage may indicate deterioration of elements within the dam or foundations that will need further investigations. Measurements of leakage should be recorded together with water level in the reservoir, precipitation and snow melting, as these factors may influence the readings.

Continuous measurements of leakage are of importance to detect sudden changes in the leakage that may not be determined by a measurement now and then. This is of particular interest for embankment dams, but also for concrete dams founded on soils or rock with weak zones.

Pore pressure

Pore pressure measurements of the foundation will generally be required for dams founded on soils or rock with weakness zones. Additional monitoring of pore pressures can prove necessary, particularly on high concrete dams. However, as a result of the glacial history of Norway, the foundation often consist of hard, resistant and durable rock, and potential monitoring will need to be evaluated in each separate case and this is therefore not a general recommendation. Measurement of pore pressures is recorded together with the water level in the reservoir.

Deformations of concrete dams

Measurements of deformations are particularly recommended for arch dams and dams where alkali aggregate reaction (AAR) are detected or suspected. Deformations should be recorded together with measurements of water level and concrete temperature. Measurements of cracks must also be considered, however, this will be based on an individual assessment of the dam in question.

Deformations of embankment dams

Generally, settlements of the crest will be measured at least once every year. Annual levelling of moraine- and asphalt concrete core is also recommended. In this way any sudden changes in settlement can be detected and a more detailed evaluation can be carried out.

In addition, horizontal and vertical deformations of bolts are measured about every 5th year. Suggested distribution of deformation bolts are given in the guidelines. The need for a more detailed survey may be required to identify local deformations that will not be recorded within the grid of deformation-bolts, e.g. deformations caused by beaching or internal erosion. For this purpose, detailed topographic mapping can be made on basis of aerial photographs or sonar. A picture of sonar mapping is shown in figure 2. Sonar may give a better level of detailing than aerial photographs, but will be limited to the upstream face.

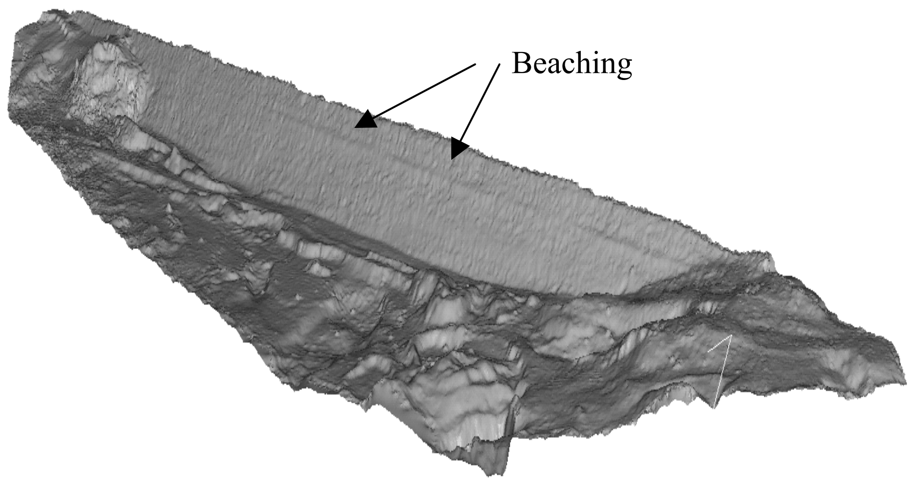

Figure 2: Topographic mapping of the upstream face of an embankment dam (Photo: Artec Subsea AS)

Evaluation of the monitoring data

The monitoring data must be analyzed and evaluated continuously and presented graphically so that both long term and short term tendencies are visualized. Some examples of graphical presentations are illustrated in the guideline [NVE, 2003[1]].

The monitoring data need to be assessed and commented on the basis of the following elements:

- Accuracy of the monitoring data.
- Possible changes in trends.
- Factors that may have influenced the measurements.

MONITORING FOR EMERGENCY WARNING

Instrumentation for the purpose of emergency warning is generally limited to monitoring of water level and leakage. Monitoring of the reservoir water level is recommended on all high hazard dams and dams with gated spillways. Monitoring of leakage for emergency purposes is limited to embankment dams with a central core of moraine and asphaltic concrete or dams founded on soils or rock with weak zones.

Where monitoring for emergency warning is required, a continuous reading and transmission of the data is necessary in order to detect any development of a possible abnormal situation.

In some cases, abnormal situations have not been detected as the instrumentation did not work properly. In such cases a surveillance camera on the dam site would have been useful. A camera on site can also reveal additional information that is not necessarily detectable by instrumentation alone and may be of particular use when the control centre is at another location than the dam.

DAM BREAK WARNING SYSTEMS

Some Norwegian dams also have been required to have a dam break warning system installed. [Martinsen, 1995]. This is not included in the guideline for monitoring and instrumentation, as the dam break warning system is only required on some particular dams.

The first formal warning system of dam failure was established during the Second World War. It covered nine river basins, and was based on the use of telephone or radio at the dam itself, and cars along the main roads with alarm sirens. After the war, this system was abandoned.

In some river basins, dam failures could cause disasters of enormous dimensions, in terms of both loss of lives and material damage and in 1966 a working group for dam safety in emergencies was set up. In 1971, the group concluded that a modern warning system should be developed. The decision was based on the following main factors;

- Even though the likelihood of dam failure is relatively small in times of peace, the possibility of it occurring as result of natural reasons, technical damages or damage caused by terrorism or sabotage should not be ignored. The system should therefore also be operated in times of peace.
- Dams constitute targets for attack in times of war. The primary motive for attack would usually be to exploit the destructive effect of the breach wave in the area below.
- In the event of a dam failure, loss of life can be very extensive. A good warning system allows even people living very close to the dam, to be evacuated.

These arguments, combined with a strong local political pressure, led to development of an electronic warning system for dam failure in two river basins.

The dam owners were requested to install and operate the warning systems, as well as the communications and transfer of the warning signals to a first reporting point. From this reporting point, the Civil Defence services assumed responsibility for installing and operating the system of sirens, and for communication between the first reporting point and the sirens themselves. In this way the warning was transferred to a large number of sirens throughout the river basin.

The warning system is based on two different electronic monitoring systems:

- Four single current electric circuits built into the dam. When one circuit is broken it will be detected.
- Downstream the dam, there are four independent floats that measures the water level, and a warning signal is given at a previous defined water level.

To avoid unnecessary warning, the warning system is only activated if a dam break is indicated by at least three of the independent systems.

CONCLUSIONS

The guideline for monitoring and instrumentation [NVE, 2003] is valuable as a basis to determine a minimum of instrumentation for Norwegian dams. However, an assessment of each individual dam should always be made in order to determine the need for any additional monitoring, or in some cases the need for reduced monitoring compared to the guideline.

The guideline gives recommendations on what to do, when and where. However the human factor should not be forgotten, as instrumentation and monitoring is just a tool on the way to achieve better dam safety. Just as important as the actual instruments are how they are operated and how the monitoring data is analyzed. Valuable information may drown in an enormous amount of data. Further, improper analysis of the data may not reveal information that could have been detected. These factors may prove to be the real challenges when monitoring a dam and should not be overlooked.

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Long-term stress measurements in the clay cores of storage reservoir embankments

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SYNOPSIS. In 1987 push-in spade-shaped earth pressure cells and BRE miniature push-in earth pressure cells were installed to study stresses within the puddle clay cores of Staines South and King George VI storage reservoirs in west London. The spade cells were installed to measure horizontal stress and the miniature cells were installed to measure both horizontal and vertical stresses. In 1998 spade cells were also installed at various sections in the rolled clay cores of Queen Mother and Wraysbury reservoirs. This paper outlines the monitoring programme and briefly describes the instrumentation and installation techniques. Selected data sets demonstrate the reliability and longevity of the instrumentation. The results show that these instruments can provide valuable long-term information on stress levels within clay cores and, in particular, the effects of reservoir drawdown and refilling on the magnitude of these stress levels in relation to reservoir water pressure.

INTRODUCTION

A survey by Charles and Boden [1] of nearly 100 cases of unsatisfactory performance of embankment dams in the UK suggested that the most serious hazard for old earth dams as they age in service is associated with internal erosion. Hydraulic fracture of a clay core is one possible mechanism which can initiate internal erosion and it has been postulated that hydraulic fracture can occur if the water pressure from the reservoir exceeds the minimum total earth pressure acting on a transverse plane within the body of the core. The state of stress within clay cores is therefore of considerable interest. Charles [2] has reviewed case histories of the deterioration of puddle clay cores and Charles and Watts [3] have described a programme of field measurements to examine the horizontal pressures within puddle clay cores and puddle-filled cut-off trenches of old earth dams.

With uniform ground conditions and a level ground surface it is usually assumed that the vertical total stress can be calculated with sufficient accuracy by multiplying the depth below ground level (z) by the mean bulk unit weight of the soil above that depth (γ). However, there are situations where the vertical stress is significantly different from the calculated overburden pressure. An important example is where there is "arching" involving stress transfer between soils with different stiffness such as between the clay core and shoulders of an embankment dam. In such cases the vertical total stress may be significantly smaller than γz and the horizontal stress will be a complex function not only of depth and unit weight of the soil, but also of the stress-strain relation and stress history of the soil. Reliable determination of vertical and horizontal stress usually requires in-situ measurement.

INSTRUMENTATION

Two types of pressure measuring device have been installed to monitor the stresses within the cores of four embankment dams.

Spade type pressure cells

The use of push-in spade-shaped earth pressure cells (“spade cells”) in various types of clay has been described by Penman and Charles [4] and Tedd and Charles [5]. Spade cells have proved to be very simple and reliable for stress measurement although there is a tendency for them to over-read even when the excess pore pressure set up during installation has dissipated. The amount by which spade cells over-read has been investigated and a simple empirical correction of half the undrained shear strength ($0.5c_u$) has been proposed by Tedd and Charles [5]. Ryley and Carder [6] have found that a larger correction is needed where c_u >150 kN/m^2 but this is of no significance for the work reported in this paper.

The spade cells used in the investigations were manufactured by Soil Instruments Ltd. and consist of an oil filled steel envelope approximately 200mm long x 100mm wide and 6mm thick. Each spade cell incorporates a piezometer above the pressure cell and the pressures are measured by pneumatic transducers.

Installation of the cells was accomplished by pushing the spade cell about 1m beyond the bottom of a vertical borehole and all the cells were aligned to measure horizontal stress along the axis of the dam (σ_{ha}). Several weeks had to elapse after installation before the decay of the excess pressures, which were generated by pushing the cells into the clay, was complete.

BRE miniature pressure cells

Cells pushed into the soil from the bottom of vertical boreholes can only be aligned to measure horizontal stress. In the situations where the measurement of vertical stress is required, a vertical borehole may provide the only access for in-situ measurement. The BRE push-in miniature earth pressure cell ("miniature cell") is designed to be jacked horizontally into soft clay from a vertical 150mm diameter borehole.

The miniature cell consists of a 2.4mm thick oil filled envelope attached to a wedge shaped slim body. It has a measuring area 44mm in diameter, an overall length of 115mm and a maximum body thickness of 20mm in the direction of stress measurement. The miniature pressure cell operates on similar principles to the larger spade cells.

Miniature cells are installed using a special placing device which is lowered down a vertical borehole. Cells are pushed horizontally about 450mm beyond the borehole wall and multiple installations can be carried out at different elevations within a single borehole. The cell can be pushed into the undisturbed soil with an attitude to measure either vertical stress or horizontal stress. The system has been described by Watts and Charles [7] and Watts [8].

It has been found that, generally, shorter times can be allowed after the installation of miniature cells for the dissipation of excess pressures than for the larger spade cells. Generally the correction for over-read is smaller than that required for a spade cell. No corrections have been applied to the data presented in this paper for spade or miniature type cells.

PROGRAMME OF FIELD MEASUREMENTS

The study of stresses within the puddle clay cores of King George VI and Staines South storage reservoirs in west London commenced in 1987. Further installations were carried out between 1991 and 1997 to investigate potential for hydraulic fracture on several sections at both dams. In 1998 instrumentation was installed in the rolled clay cores of Queen Mother and Wraysbury reservoirs, also situated in west London. All the dams comprise continuous embankments encircling non-impounding reservoirs which store water above the surrounding natural ground level.

Cross-sections of the central parts of all four embankment dams with the elevations of the pressure cells within the clay cores are shown in Figure 1. All cells are located on the centre-line of the cores. The pressures measured at, or close to, reservoir full condition at each of the dams are plotted in relation to the elevation of the cells in metres above Ordnance Datum. The

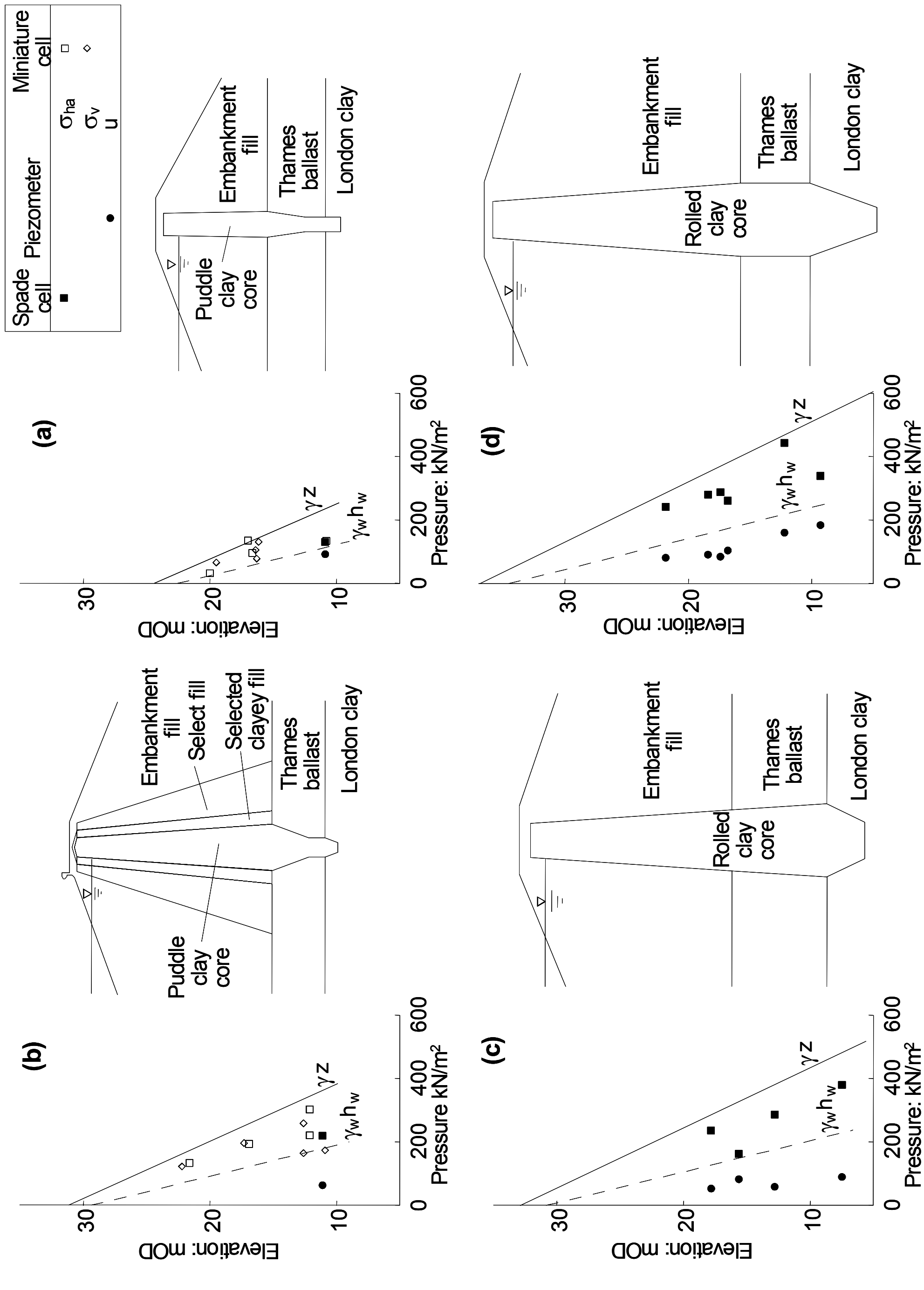

Figure 1: Pressures measured on the centre-line of the clay core at:
(a) Staines South; (b) King George VI; (c) Wraysbury; (d) Queen Mother

readings represent the equilibrium pressures measured after excess pore water pressures generated during installation had fully dissipated and therefore the individual cell readings at a particular dam were not all taken at the same time. The pressure generated by the reservoir water ($\gamma_w h_w$) on the upstream face of the core, the theoretical overburden pressure (γz) within the core and the measured pore water pressures are also shown.

STAINES SOUTH

Staines South reservoir was completed in 1903. It is part of a twin reservoir and shares a common embankment with Staines North reservoir and has a top water level approximately 3m lower than Staines North. The dam has a maximum height of 9m. The central puddle clay core extends 6m to 8m below original ground level to form a cut-off through the Thames ballast and is keyed a short way into the underlying London clay. The plasticity results of the puddle clay plot above the A-line of the plasticity chart and are classified according to BS 5930 [9] as very high plasticity (CV). Undrained shear strengths (c_u) measured from samples from the core were in the range 20-30 kN/m^2.

Miniature cells were installed in the puddle clay core at 8m below crest level to measure vertical stress and horizontal stress in the axial direction at a section where the embankment height was 9m above original ground level. Another miniature cell was installed at 5m below crest level to measure vertical stress. A spade cell was installed to measure horizontal axial stress 13.5m below the crest in the clay filled cut-off trench at the level of the boundary between Thames ballast and London clay. In 1993, a similar installation was carried out at another location comprising a spade cell at 13.6m below the crest and miniature cells to measure vertical and horizontal stresses at 8m and 5m below crest level.

The spade cell and miniature cell orientated to measure horizontal stress along the dam axis (σ_{ha}) in the deepest part of the clay cut-off are in close agreement and show pressures slightly above reservoir pressure. Data from miniature cells installed in the puddle clay at a depth of 8m below the top of the embankment are shown in Figure 2. These typical measurements demonstrate the longevity and stability of the pressure cells over a long period. The vertical stress at this position is less than the horizontal stress.

The maximum level at which the reservoir has been held has varied, but a broadly consistent pattern of earth pressure changes has been observed. The variations in vertical and horizontal pressure due to a number of major drawdown events have been monitored and the drawdowns in 1994, 1997 and 1998/99 when the reservoir was emptied are of particular interest.

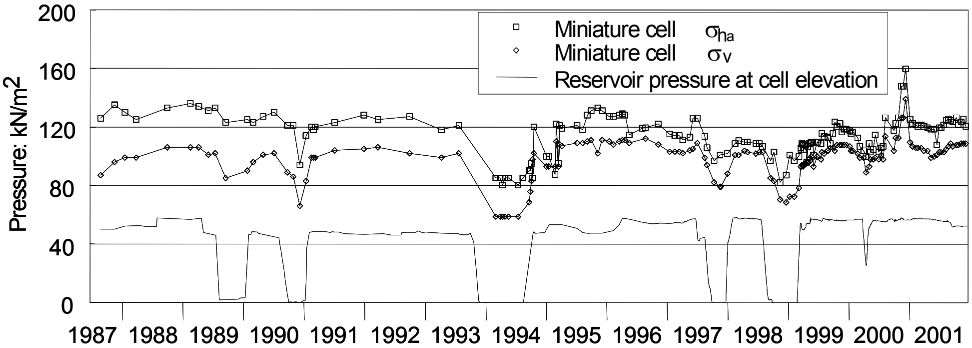

Figure 2: Measurements of vertical and horizontal pressures within the puddle clay core of Staines South at 17mOD.

On reservoir drawdown, the earth pressures have shown substantial reductions and the measured pressures have then risen rapidly in response to reservoir refilling. However, there has been some delay in returning to the pre-drawdown stress levels with final recovery in stress occurring over a period following refilling.

The anomalous peak in the data in late 2000 is more likely to be related to a common readout fault or operator error than to actual ground behaviour. There has been a general convergence of the vertical and the horizontal pressure readings since 1987, principally as a result of a steady increase in vertical stress. This pattern may be associated with the major drawdown events.

KING GEORGE VI

King George VI reservoir was completed in 1947. It has a maximum height of 17m above original ground level. The embankment is constructed of ballast excavated from the centre of the reservoir and contains zones of selected fill either side of a puddle clay core. Although slightly wider, the geometry of the cut-off through the Thames ballast is similar to that at Staines South. The plasticity results of the puddle clay plot above the A-line and vary from high (CH) to very high plasticity (CV). Undrained shear strengths of recovered samples were also in the range 20-30 kN/m^2 and were generally found to increase with depth.

In 1987 a spade cell was installed to measure horizontal stress in the axial direction at 20m below crest level in the clay filled cut-off trench. Miniature earth pressure cells were installed in adjacent boreholes to measure both vertical and axial stresses at approximately 19m and 10m below crest level. In 1991 five additional miniature cells were installed in a single borehole

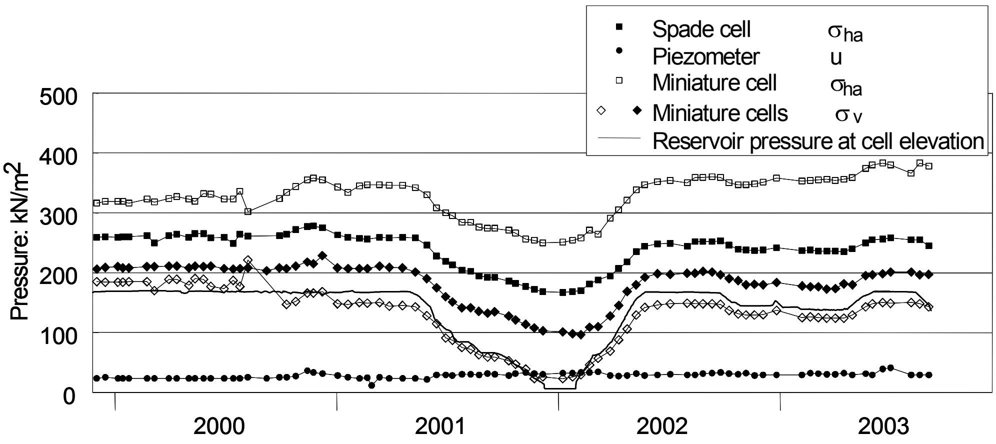

Figure 3: Measurements of vertical and horizontal pressure within the puddle clay core of King George VI at 12mOD.

close to the 1987 installations to measure vertical pressure at 20m and vertical and horizontal pressure at 19m and 14m below crest level. A further four cells were installed to measure vertical and horizontal pressure between 20m and 18m below crest level at another location in 1997.

The two miniature cells which are orientated to measure vertical stress (σ_v) in the clay filled cut-off trench have consistently measured reservoir pressure since, or shortly after, the time of installation. One cell encountered some granular material during installation and did not register excess pressures during or after installation.

In Figure 3 the pressures measured by a spade cell with piezometer and three of the miniature cells installed in adjacent boreholes at an average depth of 19m below top of embankment are plotted along with the reservoir pressure at that elevation. The figure covers the period from 2000 to 2003, during which the reservoir underwent a major drawdown and was empty for about 1 month during the winter 2001/02. The response to such an operational event indicated by the short-term detailed observations illustrates the benefits of instrument stability and regular monitoring.

The profile of pressure reduction measured by the cells closely follows the reservoir drawdown. The measured earth pressures generally recovered to values similar to those before drawdown, but the horizontal pressure (σ_{ha}) measured by a miniature cell located about 1m above the spade cell has continued a rising trend which was evident before the drawdown. All the cells showed some small time delay in reaching a maximum pressure reading some time after the maximum reservoir level had been fully reinstated.

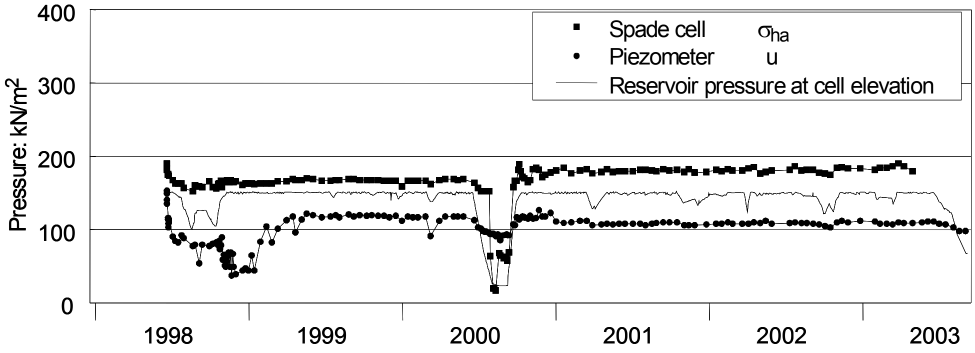

Figure 4: Measurement of horizontal pressure within the rolled clay core of Wraysbury at 15.6mOD.

WRAYSBURY

Wraysbury reservoir was constructed between 1965 and 1970 and comprises a zoned embankment with rolled clay core and gravel shoulders with clay layers within the upper part of the downstream shoulder. The embankment has a maximum height of 17m above foundation level.

The rolled clay core extends 11m below surrounding ground level to form a cut-off through the Thames ballast and keys a minimum of 3m into the underlying London clay formation. The plasticity results of the rolled clay core generally plot above the A-Line and are classed as high plasticity (CH). Water was added to the clay, which was placed to a specified undrained shear strength of about $80kN/m^2$ with air voids not greater than 3% [10].

In 1998 single spade cells were installed at four locations between 15m and 25m below crest level. These installations were carried out to investigate stress levels within the core and changes in horizontal pressure in response to rapid operational changes in reservoir level.

The deepest and shallowest cells illustrated in Figure 1(c) have given remarkably stable readings since installation in 1998. The pressures measured by these cells during reservoir full conditions remained unchanged after a 7m drawdown during August and September 2000. The cell at elevation 12.8mOD has indicated a steady fall in horizontal stress σ_{ha} of about 50kPa or 5.0m water head since 1998. The cell at elevation 15.6mOD, which measured pressures close to reservoir full level after installation has shown a quite different trend as shown in Figure 4. There has been a marked and sustained rise in earth pressure at this cell after the reservoir drawdown while the piezometer at this location has measured a small but sustained fall in pore water pressure.

QUEEN MOTHER

Queen Mother (formerly known as Datchet) reservoir was constructed between 1969 and 1974 and comprises a zoned embankment with rolled clay core and gravel shoulders with clay layers within the downstream shoulder. It has a maximum height of 20m above foundation level.

Core clay properties are similar to Wraysbury. The specified placed undrained shear strength for the clay was also about 80kN/m^2 with air voids not greater than 3%[11].

In 1998 single spade cells were installed at six locations at depths between 15m and 28m below the crest. The installations were also carried out to investigate stress levels within the core and changes in horizontal pressure in response to rapid operational changes in reservoir level.

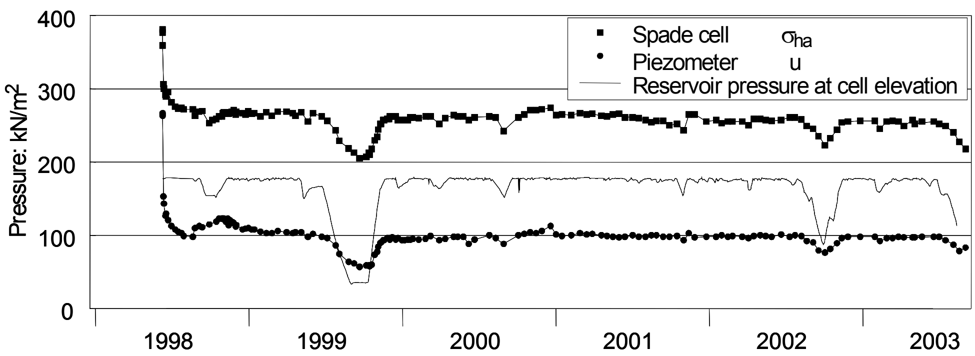

Figure 5: Measurement of horizontal pressure within the rolled clay core of Queen Mother at 17mOD.

The long-term measurements obtained from the pressure cells installed at Queen Mother indicate a rather different pattern of behaviour to that observed at Wraysbury. Figure 5 shows pressures measured by a spade cell and piezometer installed 20m below the crest of the dam. It is one of a cluster of three cells at similar depths shown in Figure 1(d). Initial readings after installation in June 1998 show the dissipation of excess pressure due to installation. The cell was installed during a period when the reservoir was at, or close to normal top water level.

Since cell installation there have been two periods when the reservoir level was reduced by a significant amount and a number of minor reductions in level have occurred over shorter time periods. The plot shows the sensitivity of the pressure cell to changes in reservoir level and hence variations in horizontal pressure within the clay core. During 1999 the reservoir level was reduced by approximately 14.5m and held at the reduced level for just over one month. This resulted in a reduction in σ_{ha} of 65 kN/m^2, equivalent to about half the reduction in reservoir pressure. This reduction was time dependent and the pressure was still falling when the reservoir level was rapidly raised. The rates of change of the earth pressure and the pore water pressure measured by the associated piezometer appear to be closely related.

There has also been a small but steady decline of about 15kN/m^2 or 1.5m head in the earth pressure measured by this cell for reservoir full conditions over the five years since installation. The measurements obtained from this cell are typical for the dam and a very similar pattern of reaction to reservoir drawdown and a steady decline in pressure is repeated for all the cells at different locations along the dam.

DISCUSSION

A considerable volume of data now exists for all the four monitored dams. The oldest instrumentation, which is in Staines South and King George VI, has provided a continuous record of pressures over a period of 16 years. The vast majority of the spade and the miniature cells have given realistic and consistent measurements throughout the monitoring period. The instruments have made possible the measurement of in-situ stress under static reservoir full conditions and with fluctuating reservoir levels.

Under static conditions with reservoir full, the earth pressures within the clay cores generally are significantly above the reservoir pressure at that particular depth. Vertical and horizontal pressures measured at similar elevations in the puddle clay cores of Staines South and King George VI are generally similar in magnitude and no consistent pattern as to a dominant direction has emerged.

The situation in the narrow clay cut-offs of puddle core dams is somewhat different. Stresses at or close to reservoir pressure have been monitored at both King George VI and Staines South.

Pressures at all elevations in the rolled clay cores at Wraysbury and Queen Mother are generally well above the reservoir pressure. One exception is a cell at Wraysbury which was installed at a predetermined elevation within a softened zone. Its elevation is coincident with the boundary between embankment fill and foundation ballast and this may be of significance.

CONCLUSIONS

1. The field measurements have demonstrated that spade cells and miniature cells can provide a reliable means of monitoring stresses in clay cores over long periods.

2. Reservoir refilling following a major drawdown is a critical time for hydraulic fracture and the instrumentation can be used to monitor stresses during this period.

3. The stress conditions in narrow clay cut-offs tend to be more adverse than in the clay cores within the embankments.

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Glacial risk and reservoir management: the Lago della Rossa reservoir example (Valli di Lanzo, Western Alps, Italy)

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SYNOPSIS. The climatic evolution of recent years, characterised by slight winter snowfalls and very high summer temperatures, is causing a progressive loss in ice mass combined with the increase of glacial risk. Besides the widespread retreat of glaciers, one of the most evident consequences of temperature increase is the formation of epiglacial lakes, like the one formed on the left side of the Croce Rossa glacier in 1998. The glacier overhangs the reservoir of Lago della Rossa, the highest in Italy. A complete survey (GPS surveyed strain net, ablation stakes, radar echo sounding, automated air and ice temperature measurement) has been carried out since 1998 in order to establish the main features of the glacier and monitor its evolution. Mathematical and physical models have been applied in order to evaluate glacier stability and future scenarios in case of epiglacial lake outburst.

Introduction

The formation of ponds and supraglacial lakes even at an elevation higher than 3000 m a.s.l. represents one of the main consequences of the atmospheric warming which presently affects high mountain regions where glacier- and permafrost-related hazards are rapidly increasing (Mercalli et al., 2002a; Mercalli et al., 2002b; Tamburini et al., 2003; Kääb et al., in press).

Once formed, supraglacial lakes tend to expand due to thermokarst process. The appearance of a glacial lake represents a cause for concern about potential glacial lake outburst flooding (GLOF) (Mercalli et al, 2002a).

In order to either prevent or reduce the accumulation of large volumes of water and to prevent dangerous situations, practical measures have been successfully taken in many cases (pumping, syphoning, oblique drilling or tunneling through the ice body, excavation of drainage channels).

In 1818 at the ice-dammed Giétroz lake (Swiss Alps), despite the extreme technical and environmental conditions, a drainage tunnel about 200 m long was excavated by hand through ice (UNST, 1981; Hambrey et al., 1992).

In 1996 at the Gruben Glacier (Swiss Alps) a drainage channel was excavated in order to prevent the occurrence of outburst floods from a periglacial lake (Haeberli et al, 2001).

Drainage channels should be oriented according to the sliding direction of the glacier, in order to increase their effective life and efficiency even in cases where the speed of the glacier itself is high. Once the channel is excavated, water flowing through it rapidly melts ice and increases the width of the channel, so enhancing the excavation process.

As part of glacial lake hazards mitigation at Hualcán (Cordillera Blanca, Peru) in 1993, following installation of siphons, the construction of a 2-m diameter tunnel started. The tunnel was 155 m long beneath a rock bar below the moraine dam (Reynolds et al., 1998).

In the Himalaya remediation strategy to reduce the GLOF hazard at the Tsho Rolpa (Nepal) consisted in syphon pipes to augment the original trial pipe and an artificial spillway ((Reynolds, 1999).

Description of the investigated area

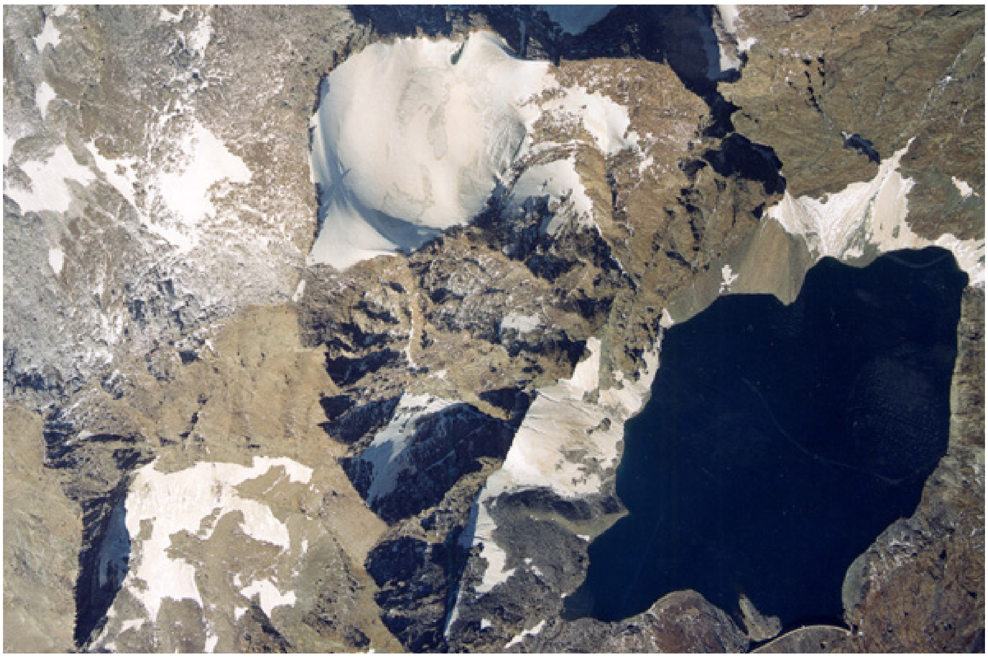


Figure 1: Aerial view of the Lago della Rossa reservoir and Croce Rossa Glacier (CNR-IRPI, 16/09/2001)

The Croce Rossa Glacier (Fig. 1 and 2) is located on the NE slope of Croce Rossa peak (3556 m a.s.l.) in Valle di Lanzo (Western Italian Alps). The steep terminus of the glacier (3380 m a.s.l.) overhangs the Lago della Rossa reservoir, the highest in Europe (2715 m a.s.l.), which is located about 700 m below.

Figure 2: The Croce Rossa Glacier (Tamburini, 05/08/2003)

In 1998 a small supraglacial lake (Fig. 3) formed near the left margin of the glacier at an elevation of about 3450 m a.s.l..

Figure 3: The supraglacial lake appeared in 1998 (Tamburini, 2000)

The presence of the supraglacial lake represents a cause for concern due to the increased risk of triggering ice avalanches into the Lago della Rossa reservoir. If waves should be generated by such events, they could seriously damage the dam and overtop the dam crest, with resulting floods in the downstream valley.

For this reason surveys were immediately carried out on the glacier in order to assess and reduce hazards and a monitoring system was installed.

Studies on the Croce Rossa Glacier

The following studies were carried out in the Croce Rossa Glacier area:

- Aerial photogrammetric survey, georeferenced with GPS, provided a map at 1:2000 scale of an area including the glacier and the reservoir below
- GPS-assisted georadar surveys from glacier surface provided glacier bed morphology and glacier volume
- A physical model was performed in order to assess the water waves generated by a glacier avalanche in the Rossa reservoir

Moreover a monitoring system for the measurement of ice temperatures and glacier displacement rate has been established.

GPS-assisted GPR surveys: methodology, results and evolutive scenarios

GPS-assisted GPR (Ground Penetrating Radar) surveys from the glacier surface were carried out in December 1998 and completed in April 1999, in order to determine the ice thickness and the depth of the glacier bed. Four transverse and two longitudinal profiles were surveyed in locations determined by the glacier morphology and where access was possible.

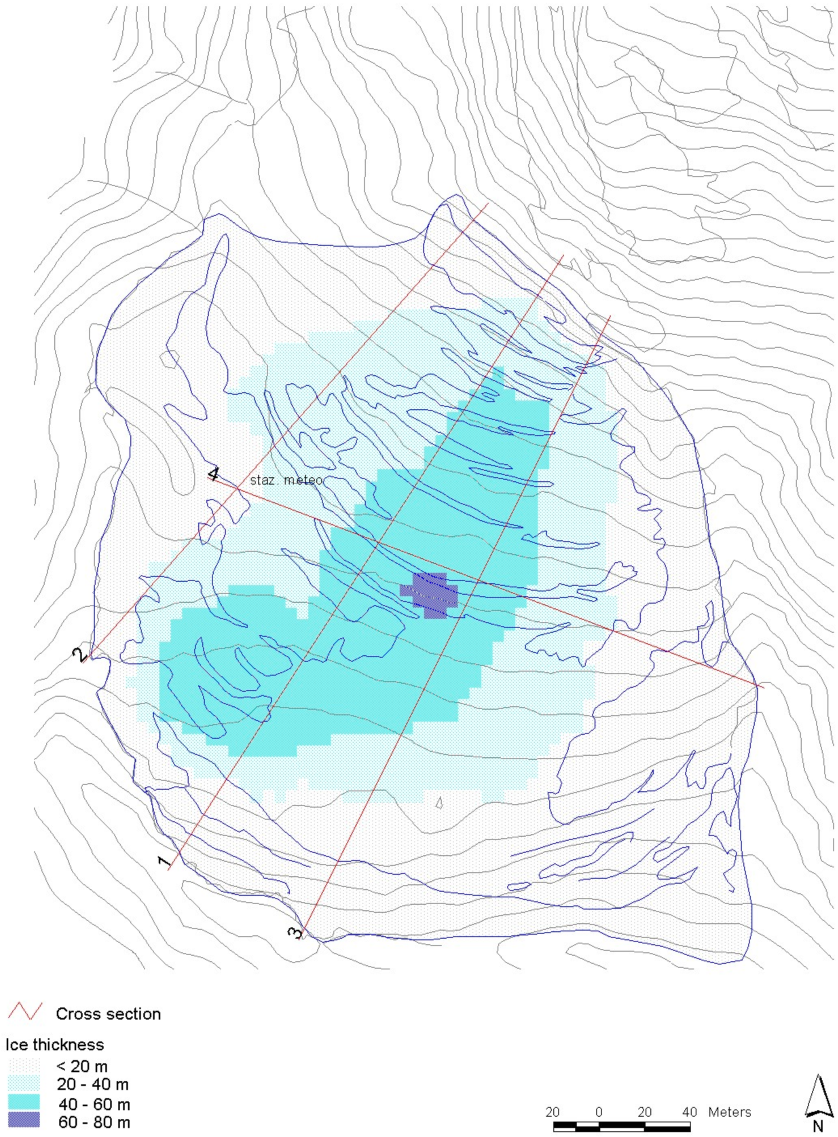
The following equipment was used:

- GSSI SIR-2 radar system
- RADARTEAM Subecho 40 antenna, with 35 MHz base frequency
- differential GPS positioning system

Raw GPR data were processed with GSSI WINRAD Software, in order to convert the original time vs. time profiles into distance vs. depth cross sections of the glacier. A map of the glacier bed was obtained in ArcView environment, by creating and contouring a DEM (Digital Elevation Model) obtained from interpolating glacier bed elevation data. By subtracting the glacier surface and glacier bed DEMs in ArcView environment, the overall volume of the glacier and a map of the ice thickness were obtained (Fig. 4). This map was later used for physical model performance.

The most relevant results obtained by GPR investigations are listed below:

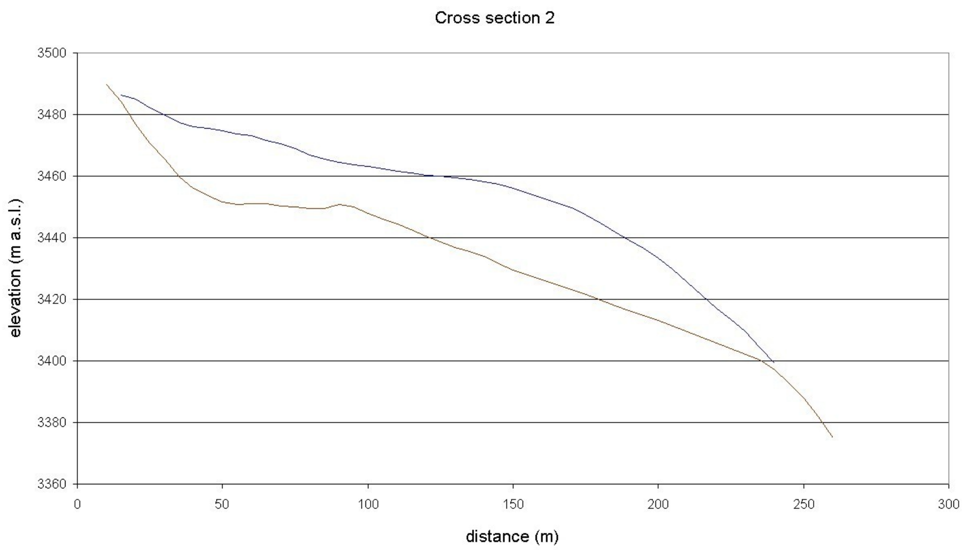
- the overall volume of the glacier was calculated as about 1.5 million m^3
- a maximum ice thickness of about 60 m was detected in the central part of the glacier
-



a change in glacier bed dip, corresponding to a rather flat area in the central part of the glacier, resulted from interpolating glacier bed elevation data; such a morphological step has been considered as favouring stability of at least the upper part of the glacier (Fig. 5)

Figure 4: Ice thickness map obtained with GPR survey

Due to the morphology of the glacier bed resulting from GPR surveys, a sudden collapse of the entire glacier was considered less probable than a partial collapse in case of outburst of the epiglacial lake and subsequent sudden increase of water pressure at the base of the glacier. Considering both the major crevasse pattern and the epiglacial lake position, some hypothesis about the unstable portion of the glacier have been carried out.



The worst scenario refers to the collapse of a volume of about 500,000 m^3 of ice.

Figure 5: Longitudinal cross section of the Croce Rossa Glacier.

Physical model

A physical model of the glacier and the reservoir below was carried out in the Enel.Hydro labs in Milan, with the aim of assessing the water waves generated by a glacier avalanche in the Lago della Rossa reservoir (Brambilla et al., 2000). Several scenarios have been simulated, with different ice volumes and reservoir levels, in order to define threshold values for the reservoir level. The temporal development of an ice slide generated by manual and "instantaneous" removal of a gate is shown in Fig. 6.

The main results obtained by physical modelling are listed below:

- the time span occurring between the collapse and the dam overflow is lower than 1 minute
- the model provided a table of water level vs. ice avalanche volume, showing that no overflow of the Lago della Rossa dam occurs in case of collapse of an ice volume lower than 200,000 m^3
- in the worst case (500,000 m^3) only the right part of the dam top is overflown

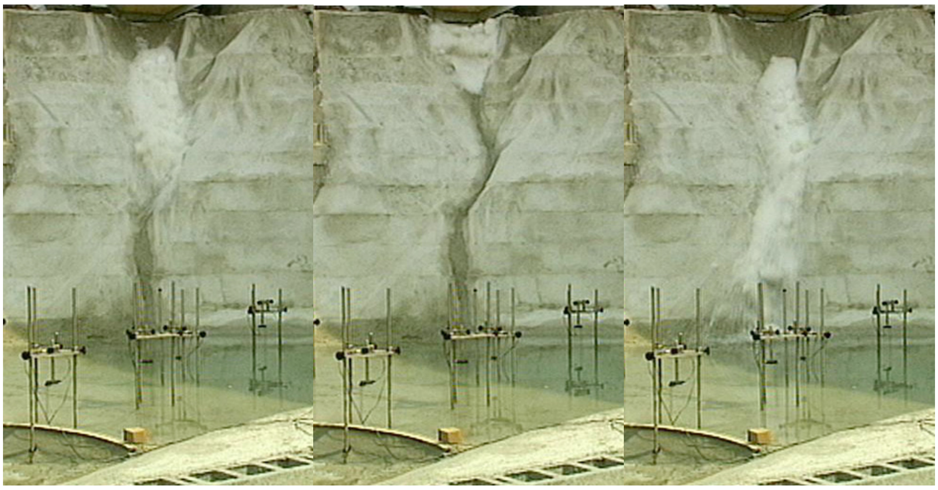


Figure 6: Temporal development of ice-slide 5, 10 and 16 seconds after the release

According to the results of the model, the maximum water level in the reservoir has been fixed 9 m lower than the maximum water level during the critical season (approximately July to September). Unfortunately, as the maximum water level is generally reached at the end of September, the above restriction represents a significant limitation on reservoir management. For this reason, appropriate measures have been proposed in order to enable a sustainable management of the reservoir itself, based on the improved knowledge of the glacier evolution acquired during about five years of glacier monitoring

Monitoring system installed on the Croce Rossa Glacier

The following devices were installed on the Croce Rossa Glacier:

- ablation stakes, set in holes carried out by steam drill device, provide data for mass balance and glacier sliding evaluation
- thermometers, inserted at different depths in holes, provide ice temperature measurement; both ice and air temperature data are automatically acquired and stored locally in a datalogger

Surface displacements are measured with GPS (5 stakes) and EDM (3 stakes) devices. EDM measurements are taken from the Rossa dam.

The five ablation stakes, each one formed by 5 wooden sections for a total length of 10 m, were installed in holes carried out by steam drill device. As the stakes are not visible from the below dam, static GPS measurements are regularly carried out twice a year. A GPS reference station was established on outcropping rock along the left margin of the glacier in order to simplify site operations.

Three stakes equipped with reflecting prism were installed near the glacier terminus, in order to enable glacier sliding measurement from a remote measuring topographic station from directly the below dam. Due to distance (about 1500 m) and difference in elevation (about 1000 m) traditional topographic measurements are affected by a high error, so only distance measurements can be considered reliable for the control of the glacier terminus. Distance measurements can be easily automated by using a motor driven theodolite.

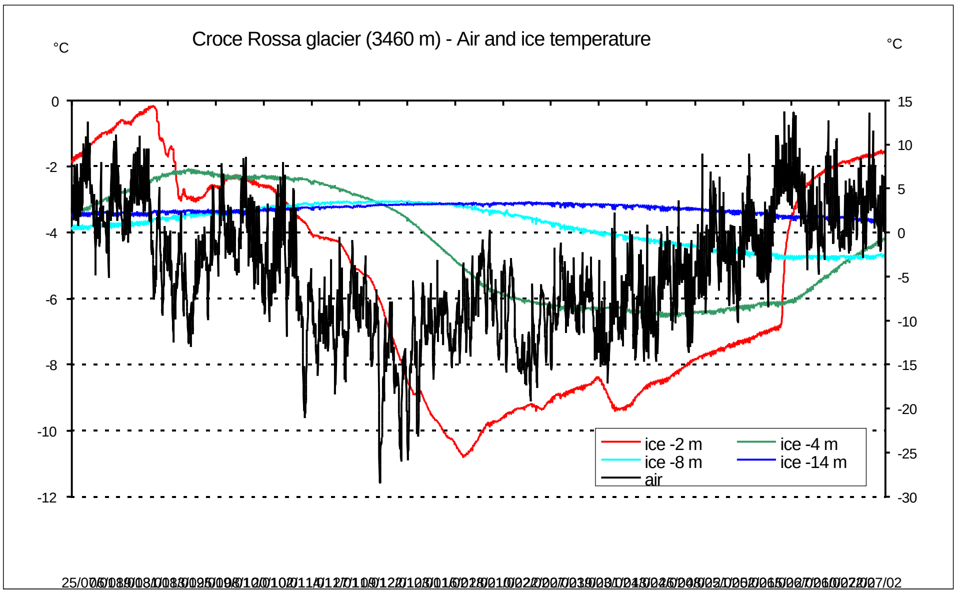
Four ice thermometers were installed in holes carried out by steam drill device near the internal side of the supraglacial lake, at depths of –2, -4, -8 and –14 m respectively from glacier surface. The deepest hole reached the base of the glacier, so providing temperature values at the ice-rock contact.

A cable connection between the ice thermometer and a permanent meteorological station installed near the GPS reference station enables continuous automatic ice and air temperature measurement. The data acquisition rate is 12 per day (every 2 hours).

Mass balance is evaluated on five ablation stakes, the same used for surface movement detection with GPS measurements. Twice per year, when glacier surface displacement

measurements are carried out, snow thickness, stratigraphy, density, etc. measurement are taken in order to calculate the mass balance of the glacier.
Finally, photographs of the glacier and epiglacial lake are regularly taken at least twice per year: a comparison with previous photographs allows the integration of instrumental data for an overall monitoring of the glacier evolution.
Monitoring system installation
Two programmes have been carried out in order to investigate the glacier and install the monitoring system operating at present:

- at the end of December 1998:
  main GPR survey (completed on April 1999)
  installation of ice thermometers at different depths, as described, ith a battery powered datalogger
  installation of ablation stakes; the stakes close to the terminus were equipped with reflecting prisms at the top, in order to enable EDM measurements from below the Rossa dam
  installation of a reference point both for GPS and EDM measurements on outcropping rock near the left margin of the glacier
- on July 2001:
  installation of a meteorological station at an elevation of about 3450 m a.s.l., anchored on an outcrop near the left margin of the glacier, powered by solar cells for automatic acquisition and storage of air and ice temperature data
  completion of the ablation stakes network; the stakes close to the terminus had to be replaced as they got lost



Results of glacier monitoring campaigns
Figure 7: Air and ice temperatures recorded from July 2001 to July 2002
The glacier has been regularly surveyed for five years. The main results provided by the monitoring activity are listed below:

- the average surface planimetric displacements are about 2 m per year in the centre of the glacier, lower than 40 cm/yr near the left margin, where the thermometers are located; such values seem to be constant within the considered period (Fig. 8)
- up to 8 m of depth temperatures in ice are subject to seasonal variations; a delay in the yearly maximum value can be observed: such delay increases with depth, up to

6 months at –8 m from the surface (Fig. 7). This is in agreement with what was observed by Paterson, 1981.

- at a depth of –14 m from the surface, temperature is about constant all the year round, varying from –3.1 to –3.7 °C; this confirms that at a depth of –14 m ice is not subject to seasonal variation; moreover, the presence of a cold ice layer at the base plays an important role in increasing the glacier stability, as low temperature is responsible for the adhesion of ice to the rock below (Luthi, 1994)
- during 1998-1999 and 1999-2000 glaciological year, an intense ablation was observed; the effects of the snowy 2000-2001 winter have been observed during the following two years, characterised by a slightly positive mass balance. Finally, the exceptionally warm 2003 summer is responsible for a strongly negative mass balance (–2.5 to –3 m w.e.); the above data suggest the opportunity of performing a new photogrammetric survey, in order to calculate the overall mass reduction and upgrade the instability scenarios outlined after the first investigation programme in 1998
- water release from the terminus was never observed; this could confirm the adhesion between ice and rock at the base of the glacier, as previously observed

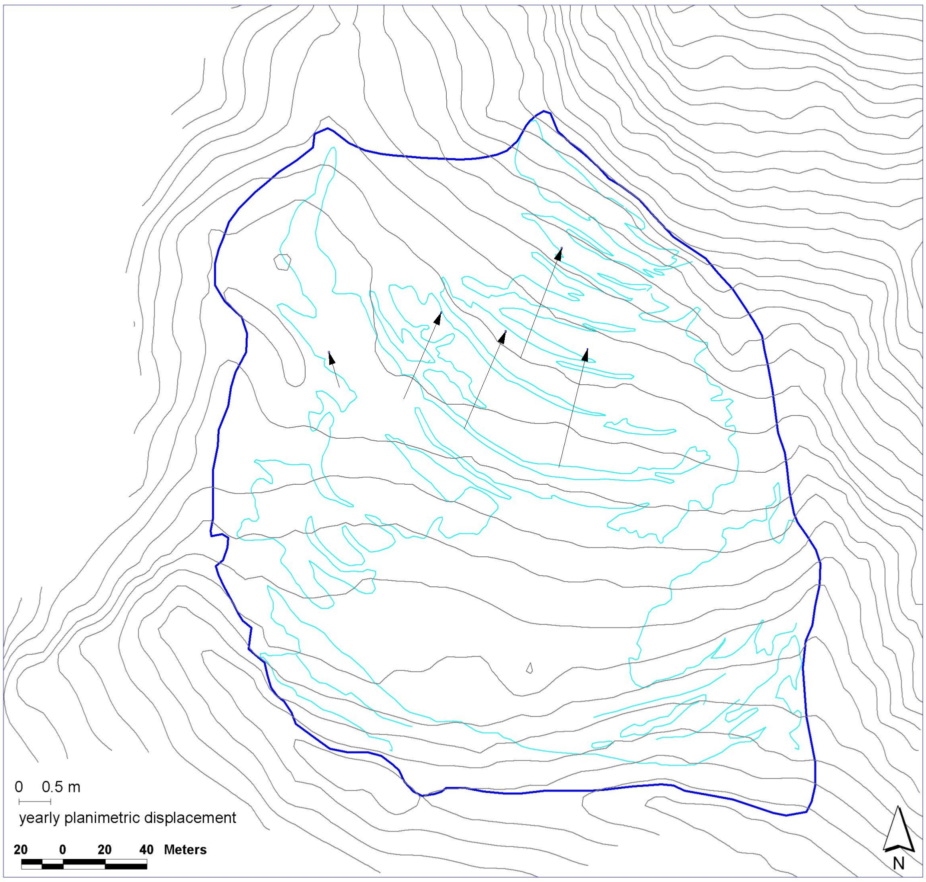


Figure 8: Yearly planimetric displacement vectors (from September 2002 to August 2003)

Scenarios for the future

According to the results provided by monitoring, the behaviour of the glacier seems to be constant for the considered period. The dynamics of the glacier seem not to have been influenced by the high temperatures recorded during summer 2002 and 2003. Anyway, in order to verify the absence of a long term effect of the exceptionally warm 2003 summer, a

further surface displacement measurement programme is progressing.
Two main collapse scenarios can be outlined for the Croce Rossa Glacier:

- sudden collapse of the lower portion of the glacier (the original volume of about 500,000 m^3 should be verified) due to outburst of the epiglacial lake and subsequent sudden increase of water pressure at the base of the glacier
- gradual increase of displacement rate at the glacier terminus before the collapse of the lower part of the glacier.

The former can be considered instantaneous, the latter can be forecast by measuring the displacement rate at the glacier terminus. In any case the main hazard is represented by water stored in the epiglacial lake, which should be removed.

Future developments

In order to substantially reduce hazard and enable efficient management of the Lago della Rossa reservoir, further investigations and interventions have been planned. The main activities are listed below:

- drainage of the epiglacial lake, which could trigger an unpredictable collapse of the glacier
- performance of a new aerial photogrammetric survey, in order to evaluate the glacier volume decrease and upgrade the collapse scenarios
- set of new ice thermometers closer to the glacier terminus at different depths, including the glacier base, in order to assess the glacier stability; a proper hole must be drilled to a depth of about 50 m
- mathematical model application in order to evaluate the long term effects of increasing temperature on the glacier dynamics
- reservoir level threshold value upgrade, taking into account the reduced volume of the unstable portion of the glacier and the possibility of forecasting the collapse and the discharge of water from the reservoir in case of emergency; water level vs. time curves should be computed for different discharge rates.

To assist glacier monitoring after draining the epiglacial lake, surface displacements and temperature (air and ice) measurements should be automated, in order to enable a real time control of the glacier evolution..

Automatic EDM measurements, by means of a motor driven theodolite, should be carried out in summer and early autumn (June to October), when the reservoir level reaches its maximum and the possibility of glacier collapse is higher. In case of displacement rate increase, the application of on line Voight model approach in order to forecast the collapse time could be helpful for hazard management (Voight, 1988; Voight, 1989).

Ice and air temperature, at present automatically stored in the datalogger of the meteorological station installed of the glacier, should be transmitted by either radio or satellite to the dam, enabling their automatic processing and comparison with threshold values.

Moreover periodic inspections on the glacier, will integrate instrumental measurements and provide data for mass balance evaluation.

Conclusions

The results of the studies carried out on the Croce Rossa Glacier and the data collected by the monitoring system during the last five years showed that:

- the main cause for concern is represented by the supraglacial lake, which must be drained in order to eliminate the main cause of sudden collapse of the lower portion of the glacier
- mass balance results showed that the overall volume of the glacier has been significantly reduced
- ice temperature values indicate the presence of a cold ice layer at the base of the glacier, which plays an important role in increasing the glacier stability
- the sliding velocity of the glacier is known and seems to be constant during the considered period, with a maximum of 2 m per year in the central part; an increase

in sliding velocity can be identified if an automatic monitoring system is operating during the critical season (June to September).

Once the supraglacial lake has been drained, an automatic monitoring system, based on displacement and ice temperature measurement, will enable the possibility to manage the reservoir, according to what happens in other similar situations in the Alps, like the Jungfrau railway below the Eiger Glacier or the Mauvoisin dam below the Giétroz Glacier (UNST, 1981; Luthi, 1994)

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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.

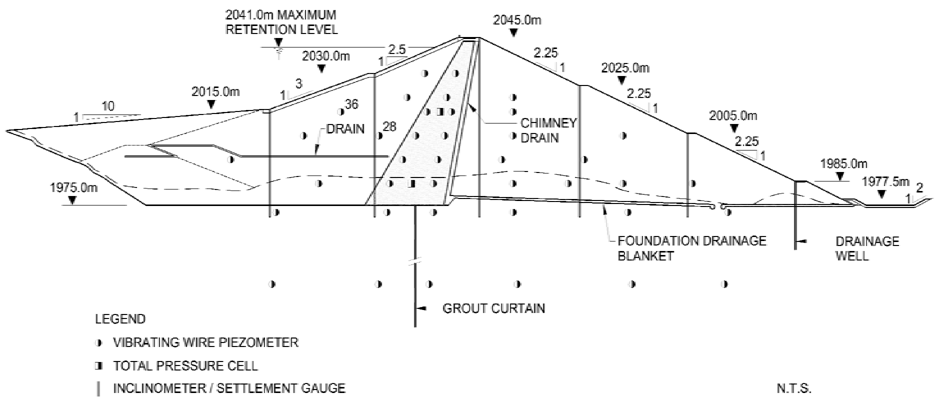

Figure 2 – Cross Section at Maximum Height

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 φ' = 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.

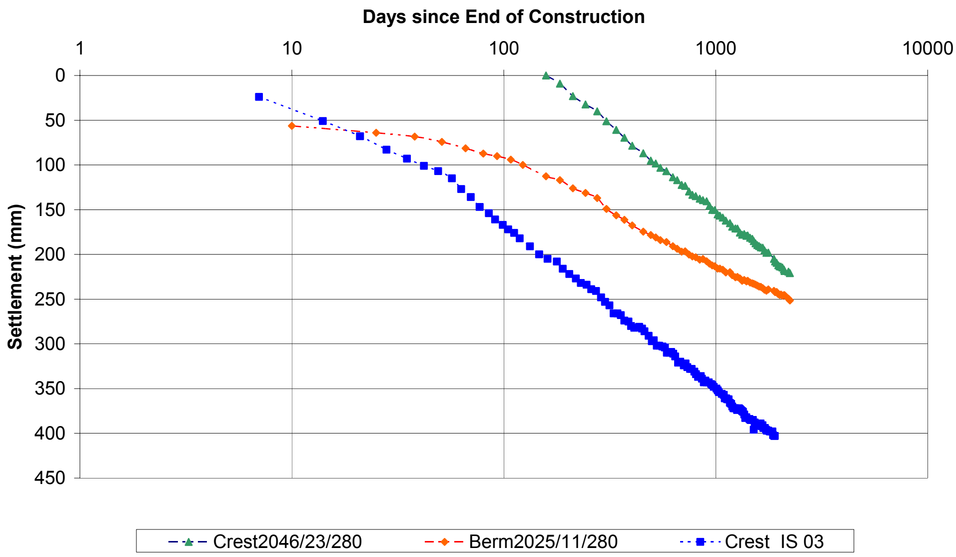

Figure 3 – Post Construction Settlement

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.

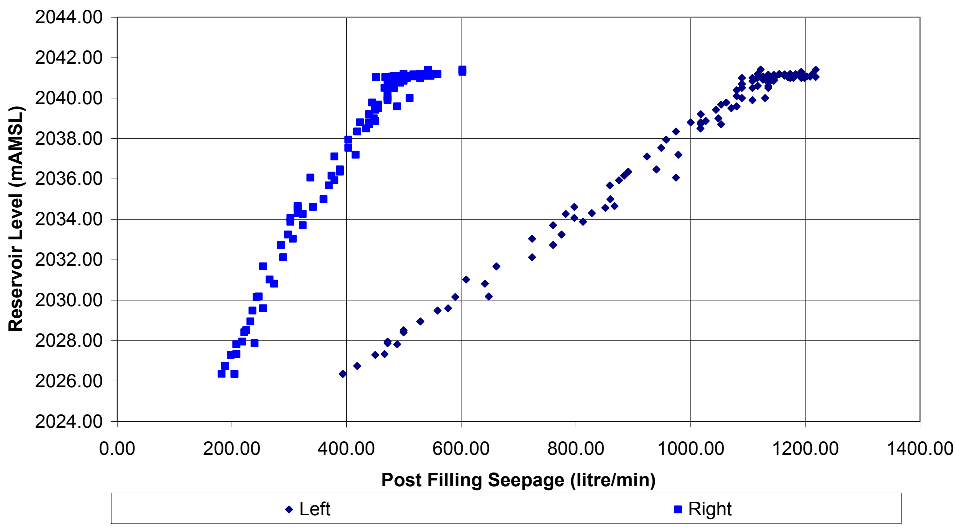

Figure 4 – Blanket Drain Seepage

<u>Piezometer Response</u>

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.

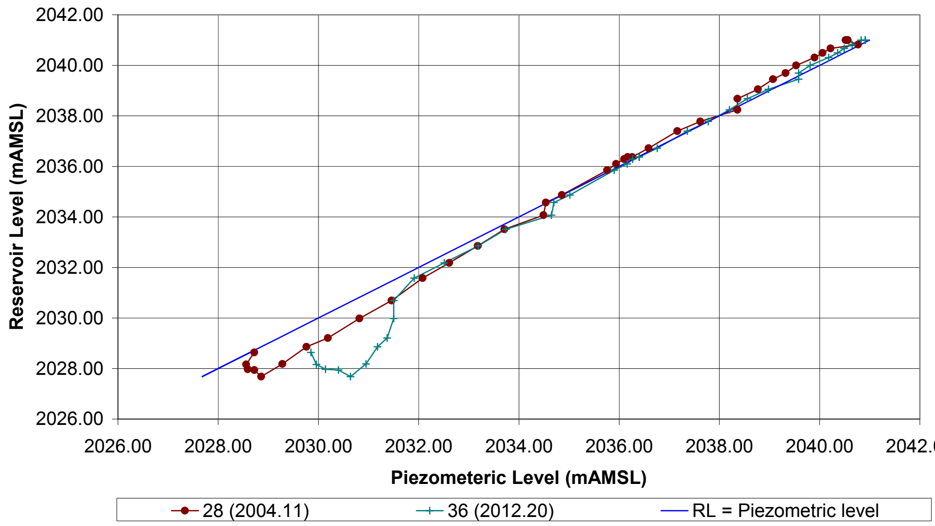

Figure 5 – Upstream Piezometer Response to Drawdown CH 290

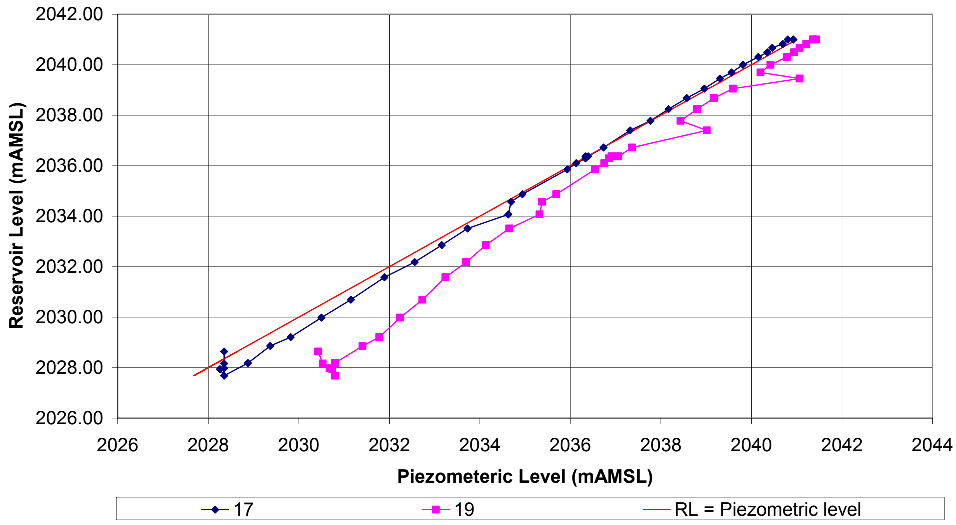

Figure 6 – Upstream Piezometer Response to Drawdown Ch 200

At the slower rate of 0.04m/day the pore pressure response of the four piezometers was similar with Bbar ($\Delta u / \Delta \sigma_v$) 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.

ACKNOWLEDGEMENTS

The authors wish to thank Nairobi City Council for permission to publish this paper. Acknowledgement is also due to the staff of Howard Humphreys (East Africa) Ltd. who diligently and reliably collected, recorded and summarised instrument data over the first 5 years of operation of the dam. The late Dr. Geoff Sims also requires special mention as he carried out the 5 year Inspection of the dam and was always keen to have information published on the performance of Thika Dam.

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Masjed-e-Soleiman Dam instrumentation

PAUL J WILLIAMS, Halcrow Group Limited

SYNOPSIS. The 187m high Masjed-e-Soleiman clay core rockfill dam forms part of a 1,000MW hydropower scheme. The dam was constructed between 1996 and 2001 and impounding commenced in late 2000. The instrumentation installed at the dam was designed to meet international guidelines for the primary purposes of monitoring construction, impounding and long term performance. The instrumentation comprises earth pressure cells, extensiometers, piezometers, groundwater observation holes, survey monuments and seismic monitors. In the event a significant proportion of the buried instrumentation failed during construction. A study was undertaken to review the performance of the instrumentation, evaluate available monitoring data and develop the criteria for control of the impounding.

INTRODUCTION

Masjed-e-Soleiman is a clay core rock fill dam situated in a narrow gorge on the lower reaches of the Karun River in southwestern Iran. The Karun River rises in the Zagros Mountains in western Iran and flows southward to the Persian Gulf. A number of dams are constructed along the Karun River and more are planned as shown in Figure 1.

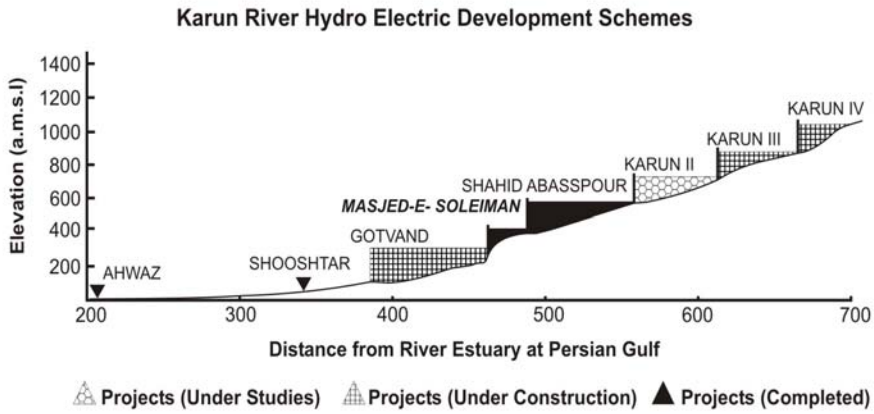

Figure 1: Karun River Cascade Development

Long-term benefits and performance of dams. Thomas Telford, London, 2004

The catchment area of Masjed-e-Soleiman reservoir is 27,550km^2. The reservoir has a total storage of 285 million m^3 and a live storage of 90 million m^3 between elevations 363m and 380m. Peak flood inflows for the 1,000 year and 100 year floods are 9,300 m^3/s and 6,800 m^3/s respectively.

The purpose of the dam is to provide river regulation and storage for hydropower generation. Clay core rockfill construction was favored because of the relative seismicity of the area and the availability of suitable clay and rockfill materials nearby.

The dam comprises a rockfill embankment with clay core and upstream and downstream filters. The upstream slope of the dam is at 1^V to 2^H and incorporates a rockfill cofferdam with upstream clay membrane at its toe. The overall downstream slope is at 1^V to 1.8^H and incorporates an access roadway for construction. The clay core of the dam has a minimum width at the crest of 10m increasing in width by 0.4m for every 1m below crest elevation. Each of the filters has a nominal width of 5m. A typical section through the dam is shown in Figure 2.

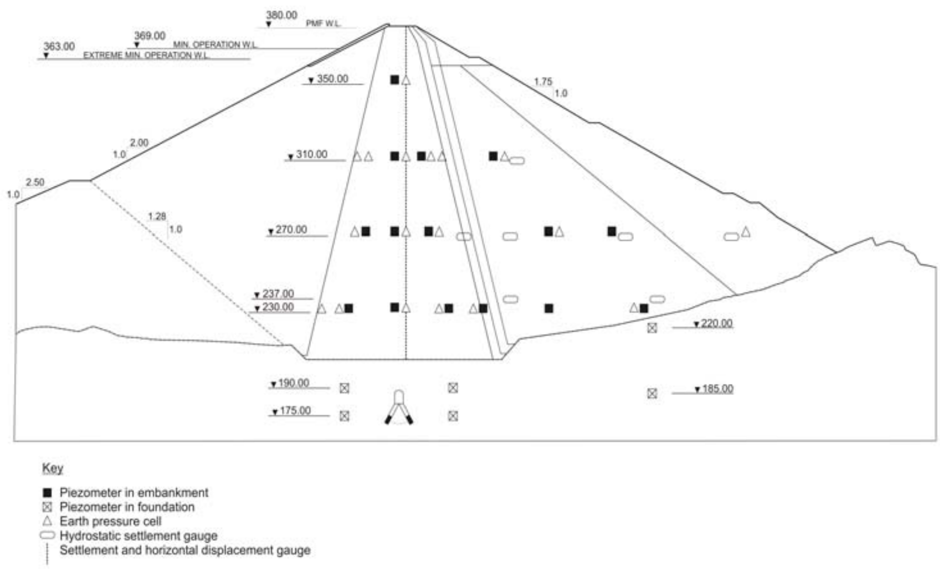

Figure 2: Typical Instrumentation Arrangement at Masjed-e-Soleiman

The clay core was placed at approximately 2% wet of optimum moisture content of 14.2% and compacted densities of 98.5% of maximum were maintained. The core material has a plasticity index of 19.9% and a permeability of between 10^{-8} and 10^{-9} m/s.

The dam foundation comprises alternating layers of permeable sandstone and impermeable claystone with a dip toward the upstream. In general the grout take for the cutoff curtain was low, with the only area of high grout take being on the left abutment between the upper and lower galleries. For this reason the arrangements of the groundwater observation holes in the galleries were amended to give clear indications of seepages in this area.

The site experiences temperature extremes ranging between +55^{o}C and 0^{o}C. The high temperatures through July and August were particularly disruptive to construction and made it difficult to control the moisture content of the clay fill.

INSTRUMENTATION

The primary objectives of the instrumentation and monitoring systems can be summarized as follows:

- To confirm the design assumptions and predictions of performance at the construction phase
- To monitor performance of the embankment during the impounding of the reservoir
- To confirm safe operation through the life of the dam including the provision of early warning of the development of unsafe trends in behaviour
- To verify the safe aging of the structure

The instrumentation installed, at Masjed-e-Soleiman was evaluated against international guidelines for dam instrumentation including the ICOLD Bulletin No 60 and the US Army Corps of Engineers manual EM 1110-2-1908. In general it would be considered a well instrumented dam if compared to other rockfill dams of a similar size worldwide. Both guidelines promote the monitoring of ground water, pore pressure, movements, deformation and fill pressure whilst recognizing that every instrument system is unique and that a significant amount of engineering judgment must be applied.

The instrumentation installed in the Masjed-e-Soleiman embankment comprises the following equipment:

Foundation Peizometers
Embankment Piezometers
Earth Pressure Cells
Hydrostatic Settlement Gauges
Settlement Inclinometers
Surface survey monuments
Standpipe Piezometers
Casagrande Piezometers
Groundwater Observations Holes
Seepage measuring weir
Earthquake Accelerometers

The instrumentation for the dam was intended to assist in the evaluation of the performance relating to the following areas:

- Seepage and leakage
- Deformation due to
    - Slope instability
    - Settlement due to internal erosion
    - Consolidation of fill
    - Consolidation of foundation strata
    - Secondary consolidation of fill and foundation
    - Volume change in clay
    - Changes in reservoir levels
- Seismic disturbance

INSTALLATION & INSTRUMENT FAILURES
During the construction of the embankment a significant number of the instruments were damaged or became inoperable. The reasons for the malfunctions included incorrect installation, damage by construction plant, use of incorrect equipment and faulty instruments. Table 1 sets out details of the instrumentation installed and their operational condition at impounding.

The instrumentation for the embankment was largely installed as the filling progressed. The foundation piezometers were installed in boreholes prior to starting the dam filling.

Hydraulic type peizometers were installed in the downstream shoulder together with hydrostatic type settlement gauges. These instruments could not be used to monitor the construction of the dam because the instrument houses could not be constructed until construction of the embankment was complete. This meant that potentially beneficial monitoring data could not be used to analyse the performance of the dam until after impounding.

There were three types of instrument, which showed significant rates of failure. These were the vibrating wire foundation piezometers, the earth pressure gauges and the settlement inclinometers, each with failure rates well outside of the range that would be expected from equipment malfunction, given careful installation.

Table 1 – Operational Instrumentation at the end of construction

Type	**Location**	**Installed No.**	**Operational No.**	**Defective**
Peizometers – foundation - vibrating wire	Core D/S fill	14 5	6 4	57% 20%
Piezometers -dam – vibrating wire	Core	25	18	28%
Pore pressure –dam – hydraulic type	D/S fill	10	Unable to monitor	
Standpipe piezometers	Abutments / galleries	22	22	
Casagrande piezometers	D/S toe	3	3	
Groundwater observation holes	Abutments	15	15	
Earth pressure gauges	Core D/S fill	39 9	30 8	23% 11%
Hydrostatic settlement gauge	D/S fill	13	Unable to monitor	
Settlement inclinometers	Core D/S fill	4 (519m) 4 (288m)	4 (340m) 4 (263m)	35% 9%
Earthquake accelerometer	Crest/Core/ Gallery/face	1/2 1/1	Installed later	

Foundation piezometers

The vibrating wire foundation piezometers monitoring data is of importance at construction stage to evaluate the stability of the foundation under the loading imposed by the dam. This is particularly important when construction is rapid and pore pressures do not have time to dissipate. With the majority of the piezomters upstream of the grout curtain inoperable, the ability to evaluate pore pressure distribution across the grout curtain at impounding was compromised. It was not considered advisable to proceed to impounding without establishing a method of monitoring foundation pore pressures upstream and downstream of the grout curtain.

A series of vibrating wire piezometers were retrofitted from the foundation grouting gallery to replace those piezometers, which were inoperable. These piezometers were installed by drilling inclined upward holes from the gallery to position a piezometer tip close to the position of the inoperable piezometers. The installation of these piezometers went well and presented no problems. The proposed piezometers were installed before impounding and gave plausible readings.

Earth pressure gauges

The earth pressure gauges provided important information relating to the build-up of earth stresses as the fill progressed. This information was used to monitor embankment stability during the construction period when pore pressures were particularly high and effective stresses low. Fortunately, a sufficient number of the instruments remained operational to allow the stability of the dam to be assessed. The importance of the monitoring data from the earth pressure gauges reduces as the fill continues to settle and construction pore pressures dissipate thereby increasing the effective stress.

Although there are earth pressure gauges available that could have been installed in a borehole from the surface, it was considered that, these were difficult to install and because they were relatively small their effectiveness would be limited. It was decided that it was not cost effective nor technically beneficial to install additional earth pressure gauges.

Settlement inclinometers

The settlement inclinometers would normally provide useful monitoring data on the distribution of settlement within the body of the embankment. The settlement inclinometers consist of inclinometer tubes installed with magnetic ring plates fixed to the outside of the tube at regular intervals. The ring plates are arranged to ensure that they settle along with the fill thus compressing the tube system. A probe is then lowered down the tube to record the relative positions of the magnetic rings. The tubes can also be used as traditional inclinometers to monitor horizontal displacements in any direction. In the event all inclinometers below EL 290m became inoperable and therefore it was not possible to assess the consolidation of the fill. Although there were hydrostatic settlement gauges below this level in the downstream shoulder these instruments could not be read until after impounding when the permanent instrument houses were completed.

It would have been possible to drill boreholes and to install settlement inclinometers in a borehole but this is generally only done for shallow holes. To reinstate the inclinometers in the dam core below EL 290m would have necessitated drilling holes to a depth of 170m through the clay. Such drilling would be very expensive and particularly disruptive to the core itself.

Failures

The reasons for the failure of the foundation and core piezometers and earth pressure cells could not be established from an analysis of the records. Given that in general it was the instruments in the upstream shoulder and in the core which had failed it was considered likely that the reason for loss of readings was due to damage of the connections from the instruments to the monitoring points on the downstream face. The cables and tubing had been laid in sand filled trench across the filter and shoulders and at the transitions between core, filters and shoulder the cable had been ‘snaked’ in the trench to allow for drawing out due to differential settlement. It was concluded that the friction on the cabling or tubing as the fill continued was too great to allow any movement. This in turn led to high stresses in the cable or tubing and failure at the interfaces where differential settlement occurred.

The reason for the failure of the settlement inclinometers was easier to determine when photographic records were studied and a failed section uncovered. The inclinometer tubes had been joined by use of an outer sleeve as per manufacturers instructions but the installer had failed to leave a gap between the tube sections to allow for telescoping of the tubes under consolidation of the fill. Subsequent settlement caused buckling of the tubes at the joints, which meant that it was not possible to pass the probe down the tube.

EVALUATION OF EMBANKMENT

The instrumentation suite was designed with particular interest in monitoring pore pressures, stress and deformation in the clay core. This was to enable an assessment to be made of the transfer of total stress to effective stress as pore pressures dissipated allowing consolidation to occur in the form of vertical displacement.

Before impounding could proceed it was necessary to demonstrate that the embankment section was stable and that there would be no safety problems associated with the excess pore water pressures that were being experienced. On completion of the embankment construction it was found that pore water pressures in the core were particularly high and that even at the lower elevations there had been very little dissipation of pressure. This meant that with low effective stresses the shear strength of the clay core was also reduced. Figure 3 shows the recorded pore pressure ratio r_u values on the highest section of the dam at the end of construction in 2000, despite the fact that construction at the foundation level had commenced in 1997. It was estimated that it would take a further 1-2 years for the pore pressures to dissipate significantly.

For this reason it was necessary to evaluate whether the reservoir could be impounded immediately after completion of construction and if so at what rate the water level could be raised.

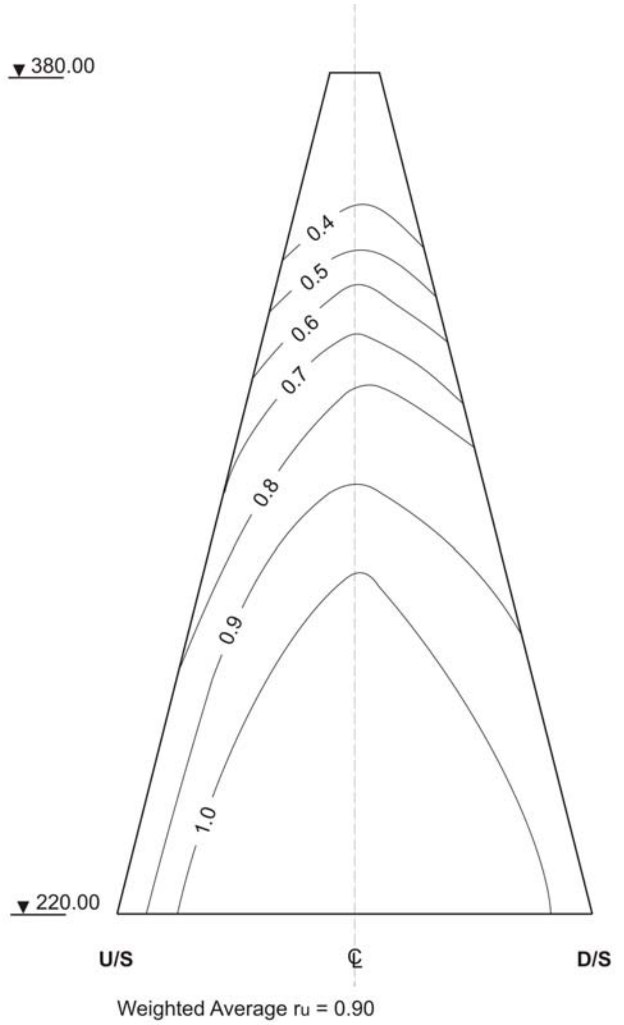

Figure 3: r_u values for the dam core at completion of construction

Of particular concern was the stability of the upstream face of the embankment for the condition of embankment at full height and also for initial impoundment. The slope stability issue was exacerbated by a layer of alluvium under the cofferdam forming the upstream toe, which had a potentially lower angle of shearing resistance. This layer had a thickness of up to 8m and had not been stripped from the foundation when the cofferdam was constructed. The uncompacted toe layer when coupled with the high r_u values in the core meant that it was possible to generate a non-circular slip failure surface through the core and upstream toe foundation, which was barely above unity for the condition of unregulated impounding. A separate study showed that provided the impounding rate was controlled to allow pore pressure dissipation in the core then the short-term factor of safety for impounding could be maintained above 1.3.

The need to make a controlled impounding became more pronounced shortly after embankment construction was completed as the single operable diversion tunnel suffered major damage to the concrete lining when the other diversion tunnel was taken out of service for conversion to a bottom outlet. The situation that developed meant that unless the remaining diversion tunnel could be closed to allow repairs the tunnel would erode further and lead to collapse. It was against this scenario that a balanced impounding procedure was developed. The impounding procedure allowed for initial impounding to EL 303m whilst the bottom outlet was brought into service and then for the water level to be held at this level using the bottom outlet, with its inlet sill at EL 300m, for a period of 3 months to allow the embankment pore pressures to continue to dissipate before filling continued to spillway sill level at EL 350m at a target rate of 0.5m/day.

The early impoundment was complicated by the fact that the upstream shoulder had a clay upstream face to elevation EL 300m. Therefore water ingress into the upstream shoulder was limited to the seepage through the clay membrane until such time as it was overtopped at EL 300m. For this reason the rate of rise immediately above EL 300m was very gradual to allow slow filling of the shoulder and stabilization.

The operational earth pressure cells show a pronounced variation in earth pressure across the core, filters and downstream fill at a number of sections. A typical effective stress distribution has been presented in Figure 4 and represents a series of six working cells. Figure 4 compares the actual minimum effective stresses against the theoretical total stress, calculated as the overburden pressure, and a lower limit of 70% theoretical, which was set as the stable limit for impounding by the designers. This represents a section towards the highest part of the dam at EL 310m some 70m below crest level and is at the time of completion of the embankment filling. Figure 4 clearly shows that the minimum effective stress in the core was considerably less than that in the filters and the shoulders. It was apparent that with effective stresses in the core of only 60% of theoretical overburden stress the core was effectively hanging up on the shoulders which themselves were being crushed with stresses up to 130% of theoretical.

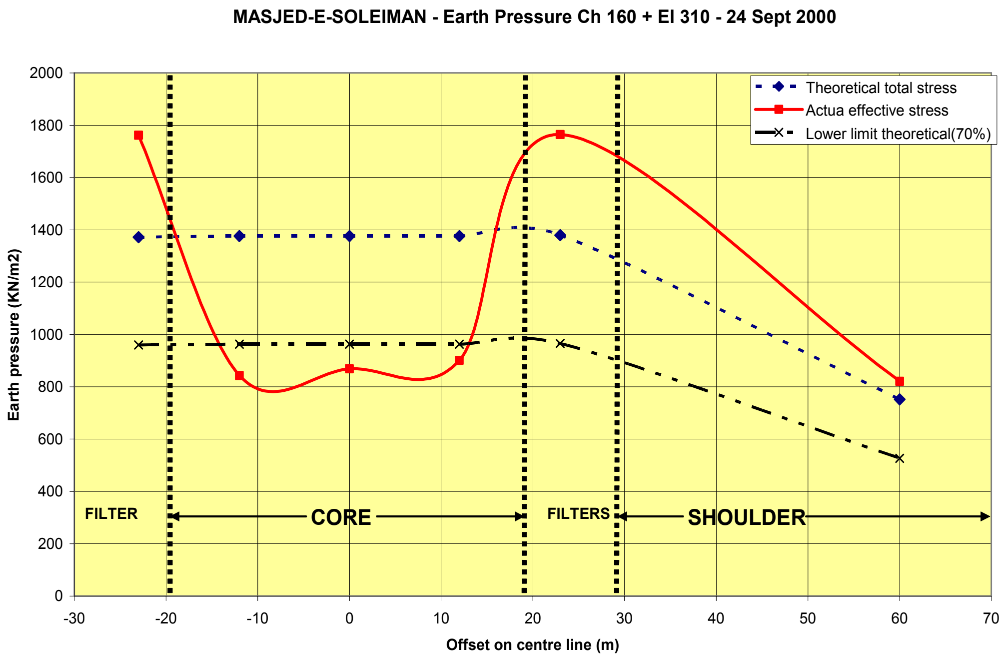

Figure 4: Distribution of effective stress across core and filters

The principal concern with the clay core was that on impounding the hydrostatic pressure of the water in the upstream shoulder would exceed the effective stress in the core material leading to potential fracture. For this reason it was decided to regulate the impounding to ensure that the hydrostatic pressures from the rising water level were not allowed to exceed 80% of the actual recorded minimum effective stress at that level in the core. To achieve this it was necessary to monitor increases in water level and the actual earth pressures as consolidation and pore water pressure dissipation in the core continued. Although it was recognized that the filters may have crushed in the zone against the core it was considered that they would continue to function as an effective filter in the event of leakage through the core.

There was a risk that if there was a significant flood event then it would not be possible to control the rate of impounding with the 330 m^3/s capacity bottom outlet and that this would result in uncontrolled reservoir rising to gated spillway sill at EL 350m. With Shahid Abasspour dam upstream the risk of uncontrolled flooding was mitigated somewhat as it was possible to create nearly 400 million m^3 of storage in the reservoir, under the operational rule curve to attenuate a flood. This meant that the risk of uncontrolled flooding through the winter/spring impounding was reduced to a 1 in 10 year event.

CONCLUSIONS

It was concluded that whilst the instrumentation at Masjed-e-Soleiman was consistent with current international practice the number of instrument failures were significantly higher than would be normally expected. In particular the failure of the vibrating wire piezometers, earth pressure gauges and settlement inclinometers make thorough analysis of the embankment difficult. This demonstrated the importance of adopting good installation procedures.

The analysis showed that the pore pressures within the core were just within acceptable limits but were dissipating far slower than had originally been envisaged. Because of the possible risk of arching and hydro-fracture of the clay core it was concluded that the impounding of the reservoir should only be made under strictly controlled conditions to prevent excess hydrostatic pressures in the upstream shoulder. The impounding was staged to allow dissipation of pore pressures in the core to ensure that at no stage would the hydrostatic pressure in the upstream shoulder exceed the minimum effective stress in the core.

There remained a slight risk that uncontrolled impounding would occur under a flood event but this was mitigated against by using the upstream reservoir to provide storage.

The instrumentation provided sufficient data to determine the behavior in terms of pore pressure and soil pressure but there was no effective measurement of settlement / consolidation of the fill due to the failure of the settlement inclinometers and the inability to use the hydrostatic settlement cells until after construction was complete. This was because the hydrostatic instruments require that the instrument houses are constructed at close the elevation of the instrument.

In the event the impounding went well and in accordance to the criteria set out. By early 2003 pore pressures were continuing to dissipate slowly and seepages remained negligible.

ACKNOWLEDGEMENTS

The author would like to acknowledge the valuable advice and assistance given by Iran Water and Power Resources Development Corporation (IWPC) site staff, Nippon Koei Ltd design staff and instrumentation monitoring staff with Daelim Industrial Co Ltd.

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SESSION 6
INSTRUMENTATION AND MONITORING OF RESERVOIR PERFORMANCE

Chairman Keith Gardiner
Technical Reporter Iain Hampson

Papers presented

1. Long-term stress measurements in the clay cores of storage reservoir embankments
   K.S Watts, A Kilby & J.A Charles

2. Monitoring of dams in operation – a tool for emergencies and for evaluation of long-term safety
   T Konow

3. An update on perfect filters
   P Vaughan & R Bridle

4. Glacial risk and reservoir management: the Lago della Rossa reservoir example(Valli di Lanzo, Western Alps, Italy)
   A Tamburini, G Mortara, L Percalli & M Lucignani

Papers not presented

5. The performance of Thika Dam, Kenya
   T Bruggemann & J Gosden

6. Masjed-e-Soleiman dam instrumentation
   P Williams

Keith Gardiner (Chairman)
As chairman's prerogative I'd like to ask the first question of Thomas of the siren systems you have to evacuate people downstream. I just wondered what the population thought of these and has it affected house prices or insurance rates?

Thomas Konow (Norwegian Water Resources & Energy Directorate)
These were installed in the 70's and actually they were installed because of local political pressure. They wanted the sirens as a safeguard if something should happen and I don't think it has affected house prices or anything like that.

Peter Vaughan (Imperial College)
I need to add a little bit to what Rodney Bridle has said which I don't disagree with at all. The first point is where this originates from and you must remember that at the time the holes appeared at the core at Balderhead we were in the process of designing a second dam in the same valley out of the same clay core. It became most important to be able to say to ones client 'this cannot happen again' and it did. The basis of the design approach that you cannot get erosion, is that you have a perfect filter, whether the problem is hydraulic factor or giant

worms. There is nothing wrong with the design principle and indeed I think that we should adopt it where we are faced with something critical as dam safety.

John Sammonds (Independent Consultant)
A question for Ken Watts. Measurements of in-situ stress presented look consistent and are comfortably in the range one would expect or hope to see. How sure are you, however, that readings are not significantly affected either by the process of installation or by the actual presence of the earth pressure cells within the fill, particularly in the case of the rolled clay core at Wraysbury?

I see on Page 397 that a simple correlation of about 0.5 c_u over-read from push in spade cells has been proposed by Tedd and Charles but the correction has not been applied here.

Ken Watts (BRE)
The answer to your question is we are quite confident that the cells are giving answers which are believable. There have been a number of investigations using both spade cells and miniature earth pressure cells carried out in order to calibration against known circumstances or those which can be assumed with some confidence. Paul Tedd has installed spade type cells horizontally under known overburden pressure to measure vertical stress and BRE has also installed a number of both types of cells at the Bothkennar soft soil test-bed site and calibrated them against a number of other measuring devices plus what is known and generally accepted about the soil stress condition at that site. In all cases that I'm familiar with they produce extremely believable and indeed very accurate readings, particularly in response to changes in stress. So there is no real underlying concern about the validity of the data. The 0.5 c_u correction was proposed for spade type cells installed in stiff clay. Miniature earth pressure cells calibrated in the field and against known applied stresses in laboratory investigations indicate a much smaller correction for soft to firm clays for which that system is designed. However, it is true that no corrections have been applied to the data presented.

Peter Vaughan (Imperial College)
Stress measurements. I think these are rather important and I say this in full awareness of difficulties such as interaction factors. The changes may be more important than absolute values because recent work with the computer is rather model dependent. The magnitude of the result is model dependent, mathematical model dependent, although there is no evidence that the trend is different but they suggest that cycling the reservoir increases the stresses and improves the safety. Now if that is collaborated and known it makes us all able to go round to the pub and relax a little.

Rodney Bridle (Independent Consultant)
Are people aware of what is in the downstream shoulder of the dams they look after? Do they know if the shoulder fills are granular or cohesive? This matters because if you want to use the perfect filter method to check how a dam might perform during an internal erosion event, you need to know what fill material is in the downstream shoulder. Granular fills may provide some filtering capability and failure by internal erosion might be slow as sinkholes form, giving plenty of time to warn people at risk and take evasive action, such as emptying the reservoir.

Peter Vaughan (Imperial College)
What could be added to the discussion over this. Surprisingly many old dams of the old puddle clay probably had filters, if not perfectly employed because they put the puddle in 6ft

layers and they tended to put it in with timbers to form the edges because it was rather expensive but you don't place big rock fill beside that, you put in fine material. In fact when we evaluated Ladybower we found that it had a very good filter and it was one of the last puddle clay dams but it is a puddle clay dam, built pre soil mechanics and they don't need filters at all, but the bit over-used weathered rock is in fact well formed fortunately to meet the perfect filter requirement. There is no risk of internal erosion in my view of that core except if there is some fault or other that they didn't realise was significant.

Andy Hughes (KBR)
A question for Thomas. Your measurement of movement is by sonar. Could you tell us if you've had any problems with that process and the level of accuracy you're able to achieve of your moving parts?

Thomas Konow (Norwegian Water Resources & Energy Directorate)
I have never really watched how they do this sonar mapping, I have just seen reports of it so I can't really answer you there, but the level of detailing is within 20 cm accuracy of the measurements.

Alan Brown (KBR)
Rod two questions for you. Firstly how variable would you expect the fill in the downstream shoulder of any one dam to be? Secondly if a permeability test was used to assess its effectiveness as a perfect filter, what is the risk that over time the permeability of the downstream shoulder may change for example if the fill was internally unstable and experiencing suffusion. Both of these would be uncertainties in any risk assessment. Do you have any comments on how major those uncertainties are going to be?

Rod Bridle (Independent Consultant)
Clearly there is much uncertainty and engineering judgement will play a large part. In a situation where the downstream fill was internally unstable, it would probably have some filtering capability, and there may be signs of internal erosion occurring prior to piping or failure by suffusion, which may provide time for evasive action. Boreholes would give an indication of the variability of the downstream fill. Even though our predecessors placed the fill using different methods to us, it was presumably a continuous process, excavating fill materials from local quarries and borrow pits. With boreholes, an investigation of local geology and any records, and judgement, we should be able to build up a picture of the fills used, but, of course, you cannot know every detail, and there will be uncertainty; the residual risk.

Keith Gardiner
Rod, one of the places that puddle clay core tends to go is down in the underlying rock and the core changes then would be quite narrow and there is always a risk then of arching across and even less stress down in the puddle clay at that level. It's quite often gone down through rock which is quite shattered and is there any work being to look at the risk of erosion at that level?

Rod Bridle
I should say that I don't know how to link the process of erosion to determining the probability of it occurring, but I am recommending that we use the permeability - filtering capability relationship of the perfect filter method to assess the filtering capability offered by the downstream fills in our dams. I am unable to enumerate the risk of internal erosion

failure through a clay cut-off trench. Alan Brown knows how to enumerate risks and probability. However, the risk of failure by internal erosion at depth in a cut-off is likely to be low because the velocity of water flowing through the cut-off is often limited by the restricted flow areas available in the joints and cracks in the rock upstream and downstream of the cut-off. If erosion did occur, you might hope that the water containing eroded materials would flow upwards and be filtered by the fill in the toe of the dam. Clearly there's much judgement in this but if you have an understanding of the grading and permeability of the fills and the floc sizes of the clay in the core and cut-off, then you have some criteria on which to judge how the internal erosion process might advance, particularly if it would advance rapidly, and give no warning before failure, or if it would advance slowly, reveal itself with sinkholes, perhaps, and allow time to take evasive action.

Keith Gardiner
Yes, but maybe it doesn't matter if water's going through it.

Rod Bridle
Leaks themselves do not often matter; they are dangerous if the leaking water causes erosion. This is why it is usually a good option to construct a filtered drain on the outlet of a leak; this will trap any eroded particles and limit erosion, while allowing the water to escape freely. The alternative of trying to block leaks by grouting or similar offers no such assurances.

Written contribution to discussions by R C Bridle
An update on perfect filters, Vaughan & Bridle (p 516)

CORRECTION

With sincere apologies, I have to report (December 2004) that the 'perfect filter' equation presented in the paper is incorrect. It was wrongly transposed from equation 4 in Vaughan & Soares (1982). It also appears in the incorrect form in Vaughan (2000a), (2000b).

The Vaughan & Soares equation is as follows:

$k=6.7*10^{-6}*\ \delta_R{}^{1.52}$

where: δ_R = size of smallest particle retained in microns (10^{-6} mm)
k = permeability of filter (m/s)

The same equation is stated on pages 522 and 526 in this paper as follows:

$\delta_R = 1.49 * 10^3 *(k)^{0.658}$

This is incorrect, the CORRECT VERSION IS AS FOLLOWS:

$\delta_R = 2.54 * 10^3 *(k)^{0.658}$

where: δ_R = size of smallest particle retained in microns (10^{-6} mm)
k = permeability of filter (m/s)

This error has repercussions in Table A on page 523, which is additionally complicated, following discussions on the paper with the ICOLD Technical Committee on Materials for

Fill Dams, where it was pointed out that there appeared to be errors in the table because the well known upper limit of 0.7 mm (700 microns) for D_{15} filter size of Group 2 core soils did not appear. Prof Vaughan prepared the table from grading information available to him sometime ago and, knowing that he will have considered all details with his characteristic thoroughness, we all agreed that it was unlikely to be wrong! However, I have used the USDA SCS 1986 paper (full reference in the paper), which includes examples of base soils and filters, to revise the table, as below:

The floc sizes retained by critical filters can be estimated by using the two relationships below:

$$\delta_R = 2.54 * 10^3 *(k)^{0.658}$$

where: δ_R = size of smallest particle retained in microns (10^{-6} mm)
k = permeability of filter (m/s)

$$k = 3 * 10^{-8} (D_{15})^{1.767}$$

where: D_{15} = D_{15} size of uniform filter (in µm, microns)
k = permeability of filter (in m/s)

Table A (Revised): Comparison of perfect and critical filters

Dam	Perfect Filter Design		Critical filter details deduced from Sherrard & Dunnigan (1989) Filter Provided			
	Floc Size δ_R (μm)	Permea-bility k (10^{-5} m/s)	Core Soil Group	D_{15} of filter (μm)	Permeability (10^{-5} m/s)	Size re-tained δ_R (μm)
Ardingly, UK	10	22	2	200-700	35-319	13-57
Carsington, UK	8	16	1	180-200	29-35	12-13
Cow Green, UK	6	10	2	200-700	35-319	13-57
Dhypotamus, Cyprus	6	10	2	200-700	35-319	13-57
Empingham, UK	10	22	1	90-200	9-35	5-13
Evinos, Greece	11	26	2	200-700	35-319	13-57
Kalavasos, Cyprus	5	8	2	200-700	35-319	13-57
Monasavu, Fiji	20	64*	1	70-200	5-35	2-8
Balderhead, UK	7	13	2	200-700	35-319	13-57

* entered as 13 in error previously

The comparison of actual floc size (extreme left column) with critical filter floc size retained (far right column) shows considerable differences from the original Table A, but the conclusions are unaltered:

- Critical filters for Group 1 core soils may be too coarse (e.g. Carsington) or unnecessarily fine (e.g. Monasavu)
- Critical filters may be too coarse to retain Group 2 core soils (e.g. Cow Green)

The permeabilities, D_{15} sizes and textures of perfect filters given in the table near the bottom of page 524, are also incorrect. The corrected version is as follows:

Mins to drop 300 mm	Terminal velocity mm/s	Floc size microns	Floc texture	Perme-ability perfect filter m/s	D_{15} uniform perfect filter mm	Texture D_{15} perfect filter
5	1.00	32.9	Coarse silt	1.35E-03	0.431	Medium sand
15	0.33	19.0	Medium silt	5.88E-04	0.269	Medium sand
45	0.11	11.0	Medium silt	2.55E-04	0.167	Fine sand
90	0.0556	7.8	Medium silt	1.51E-04	0.124	Fine sand
180	0.0278	5.5	Fine silt	8.90E-05	0.092	Fine sand
360	0.0139	3.9	Fine silt	5.25E-05	0.068	Fine sand
1080	0.0046	2.2	Clay (defloc-culated)	2.28E-05	0.043	Coarse silt

The corrected results highlight the fineness of perfect filters. Filter gradings to retain clay flocs (5-20 microns) require some fine particles of coarse silt size but such gradings have been provided at several dams and the filters have been demonstrated to be non-cohesive.

The error also has affects Table B on page 527. The corrected table, in which the correct expression has been used to calculate the size retained by the filter provided (one column in from left), is as follows:

Table B (Revised): Dams with perfect filters

Dam	Perfect Filter Design		Filter Provided			
	Floc Size (μm)	Perm-eability (10^{-5} m/s)	Filter Soil Type*	Perm-eability (10^{-5} m/s)	Size re-tained δ_R (μm)	D_{15} of filter (μm)
Ardingly, UK	10	22	ns	9	6	230
Carsington, UK	8	16	psg	1 to 10	1 to 6	80-170
Cow Green, UK	6	10	ns	2	2	110
Dhypotamus, Cyprus	6	10	sng	1	1	1000
Empingham, UK	10	22	ng	8	5	100
Evinos, Greece	11	26	sng	10	6	220
Kalavasos, Cyprus	5	8	sng	4	3	600
Monasavu, Fiji	20	64**	cr	4	3	210
Balderhead, UK	7	13				
Melton Mowbray, UK	4***	12	ns	10	6	150
Audenshaw, UK	6	23	ns	10	6	

* ns = natural sand psg = processed sand and gravel ng = natural gravel
cr = crushed rock sng = natural sand and gravel screened to remove coarse sizes

** entered as 13 in error previously

*** subsequently found that test results were wrongly analysed, floc size actually measured was 10 microns.

This completes the corrections, my apologies once again. I will circulate this information, and a corrected version of the paper, to all those to whom I remember sending a copy previously. If others are interested, please contact me on rodney.bridle@damsafety.co.uk. Please do not let the mistakes deter you from using the perfect filter method, which offers much, particularly in helping us to understand how existing dams will behave during internal erosion incidents, as my final comment below shows.

Filtering potential of downstream fills in old dams

The downstream fills at many of our old dams will probably be stony because stone was much easier to win at quarries, easier to load on to carts and easier to place on the dam than clay, which required much hard hand digging by the navvies. This means that the downstream fill at many of our dams will be granular and may have some filtering capacity, thereby reducing risks of fast catastrophic failure from internal erosion, and prolonging time to failure, perhaps sufficiently to let water out of the reservoir to an extent that a breach would no longer threaten lives.