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The first 30 years of Lefkara Dam

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The 74 m high Lefkara embankment dam was built in the early 1970s for the Republic of Cyprus Water Development Department in order to supply domestic water to the cities of Famagusta and Larnaca. It is a rockfill dam with a central clay core composed of a combination of residual soils and colluvium supported by diabase rockfill shoulders. The dam was well instrumented using total pressure cells, piezometers and internal settlement gauges as well as surface settlement markers. These instruments are still working and the results are discussed in the paper. The dam is curved in plan since it was anticipated at the time of its design that the arched plan would help resist the thrust from the reservoir, but in reality it made virtually no difference to the behaviour that would be expected from a straight dam. Instead of the conventional vertical valve shaft for housing the water intakes at various levels, a semi-buried inclined gallery (shaft) has been adopted. Leakages of reservoir water through the gallery joints into the gallery have been creating problems at high reservoir levels. The embankment and its foundation proved to be quite watertight and the seepages recorded are remarkably low.

1. INTRODUCTION

Lefkara dam is the first major dam built in the Republic of Cyprus for domestic water supply purposes. It was constructed between 1971 and 1974 as part of the Famagusta Water Supply Project and it was intended to be the main source of domestic water for the towns of Famagusta and Larnaca. It is located about 7 km upstream of Lefkara village on the south-eastern side of the Troodos mountains.

The dam is a 74 m high curved rockfill embankment, with a central clay core, having a total fill volume of 820 000 m^3. The capacity of the reservoir formed by the dam is 13·85 million m^3. The contract price for building the dam was 1 127 000 Cyprus pounds in 1971 (1 Cyprus pound was equal to 1 pound sterling). During tendering the bidders were asked to give quotations for straight and curved embankment shapes. The extra cost quoted by the successful contractor for the setting out and construction difficulties related to the curved shape was 0·2% of the cost for the straight shape and the client accepted it. Construction of the dam was carried out under strict supervision and quality control was performed by an on-site laboratory.

Hydrological studies based on rainfall and runoff records between 1917 and 1970 estimated that the mean yearly runoff that would be impounded by the dam would be of the order of 8·2 million m^3. This figure proved to be an overestimate however, as rainfall in the last 30 years of the twentieth century in Cyprus, was in fact about 14% to 15% lower than the average rainfall of the first 70 years of the century. This had a dramatic influence on runoff, which at Lefkara dam was reduced to about 45% of the original estimate. The dam started impounding water in November 1973 and it overspilled for the first time in February 2004, 30 years after completion. During the winter of 1992/1993 the water level in the reservoir was within 25 cm of the spillway crest and the dam was practically tested at full head. With a series of consecutive dry years (i.e., 1995 to 2000), water levels in the reservoir remained very low, raising questions regarding the state of the clay core at higher elevations. Dramatic water shortage conditions were faced during the summers of 1999 and 2000 when the reservoir was practically empty.

The embankment is well instrumented with total pressure cells, piezometers, internal settlement gauges and surface settlement markers. Most instruments are in good working condition more than 30 years after their installation and produce valuable information on the performance of the dam.

The dam has performed well during its 30 years of operation. Loss of precious water in the form of seepages though the embankment and its foundation was negligible and there were no problems with excessive settlements or deterioration of construction materials. The only significant weaknesses encountered during this period were the leakages of reservoir water through the expansion joints of the inclined gallery which was built instead of the conventional vertical valve shaft for accommodating the various water intakes. Due to the semi-buried nature of this structure it is proving extremely difficult to seal the leakages, which give rise to deterioration of the electromechanical equipment and create problems with their operation.

2. MAIN FEATURES OF THE DAM

The general arrangement of the dam and ancillary works is depicted in Fig. 1. As shown, the embankment is curved upstream with a radius of 240 m. The spillway has been excavated in rock on the right abutment and this is of the

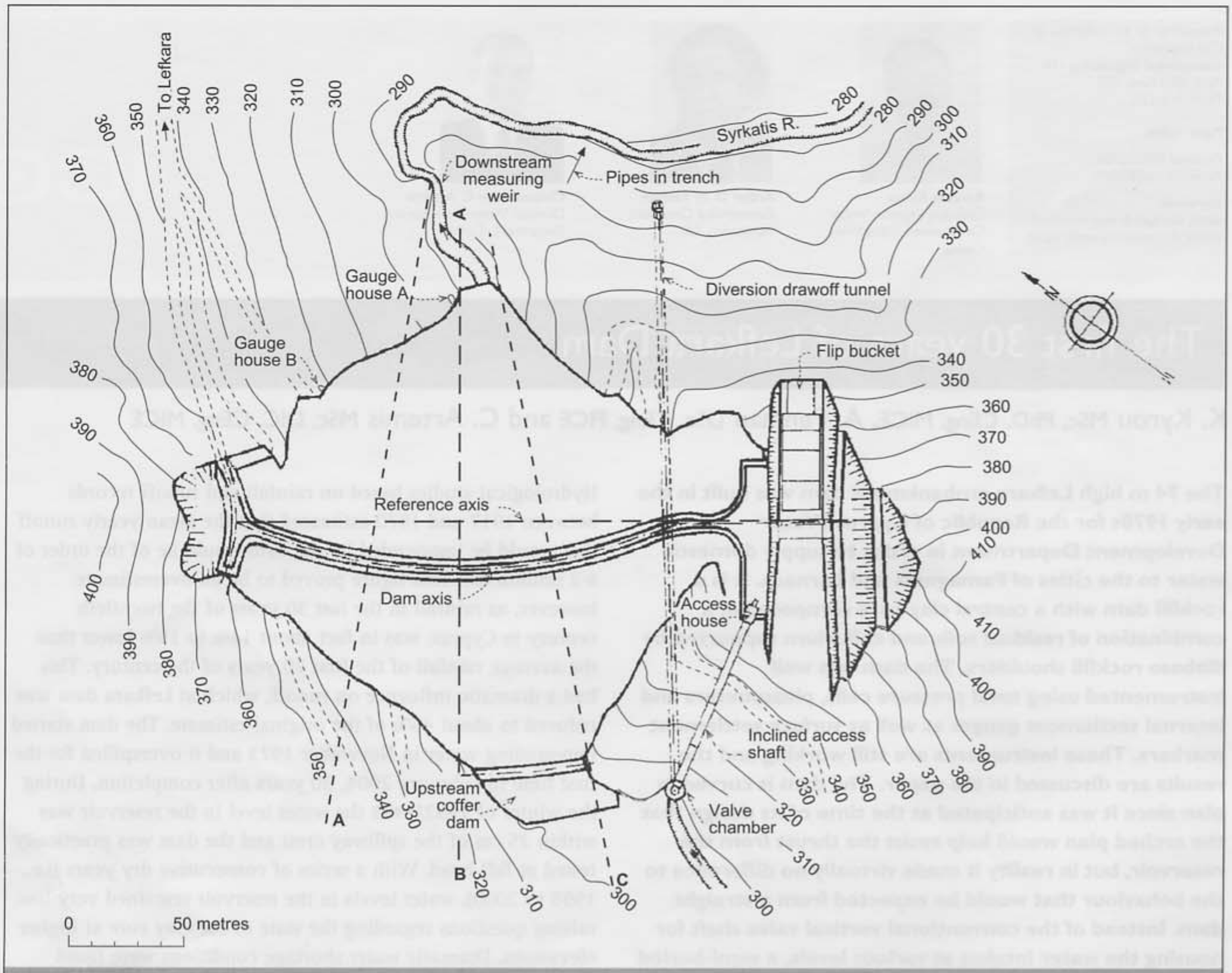

Fig. 1. Lefkara Dam—general arrangement plan (contours in m)

uncontrolled overflow type, with a 20 m wide ogee-crested weir and a flip bucket structure at the lower end of the chute.

A concrete-lined tunnel, 2·7 m in diameter was driven through the right abutment and used for diversion purposes during the construction stage. After completion of the embankment, the tunnel was sealed off at its upstream end with a valve chamber and used for housing the draw-off pipework as well as for conveying along its invert the scour water from a 914 mm (36 inch) scour outlet.

Instead of the conventional valve shaft for housing the raw water intakes, a semi-buried inclined gallery 2·8 m in diameter was built on the right flank. At crest level the gallery ends in an access house and control room and at riverbed level it is connected to the tunnel entrance via a massive valve chamber.

A typical section of the embankment is shown in Fig. 2. It consists of a central clay core with a width of roughly half the water head, flanked both upstream and downstream with 5 m wide sandy gravel filters. The embankment is supported by rockfill shoulders with an upstream slope of 1:1·5 and a downstream slope of 1:1·6. A 14 m high cofferdam was initially built at the upstream toe of the dam for diversion purposes and this was embodied into the main embankment during construction.

The embankment is founded on igneous diabase rocks throughout. Alluvial deposits at the dam site were virtually non-existent and there were extensive bedrock exposures along the riverbed. Valley side slopes were quite steep and this is demonstrated by the short crest length of 233 m.

At the beginning of construction, a grout curtain was formed along the curved axis. It consists, as described below, of a central single row of grout holes, flanked by two rows of contact grout holes on each side.

3. EMBANKMENT CONSTRUCTION

Construction work for the dam began in June 1971 and was completed by the end of 1973. Some finishing work was carried out, mostly at the crest area, during the early part of 1974.

The shoulders of the embankment were built with sound diabase rockfill won from a single quarry, which was in an extensive diabase outcrop situated on the river bend, just downstream of the spillway discharge area. Good

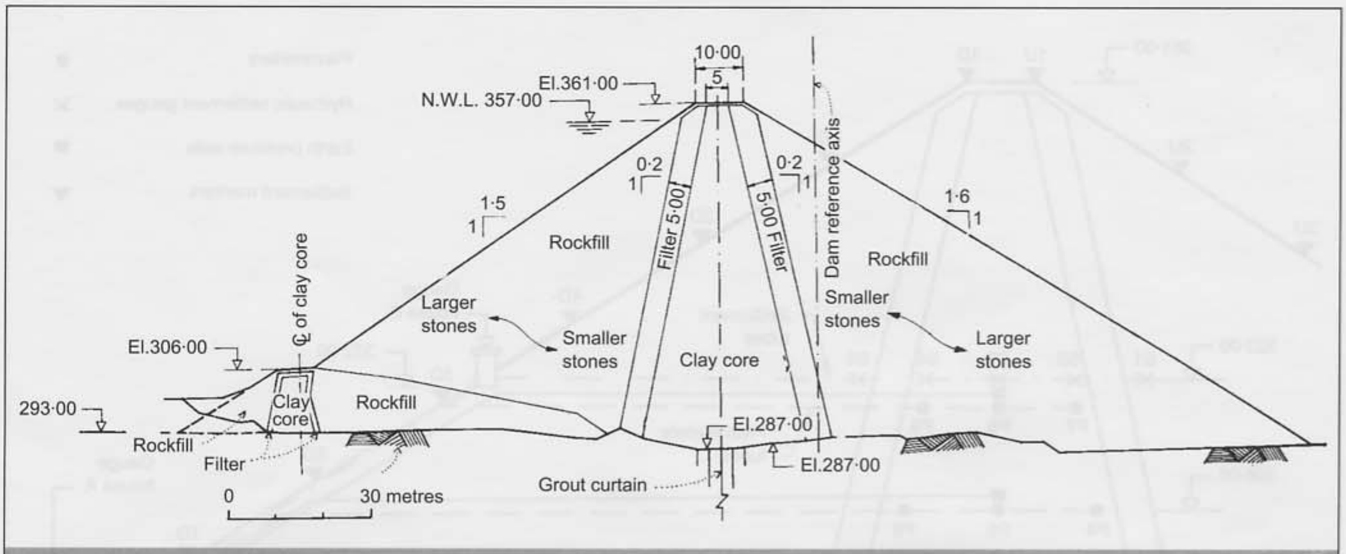

Fig. 2. Typical embankment section (elevations in m)

fragmentation of the rock was achieved by blasting, but on occasions the presence of shear zones and softer seams produced a large proportion of fines. A certain amount of selection was thus exercised at the quarry during loading to avoid excessively large amounts of fines, which would have reduced the permeability of the shoulders. The rockfill was spread in a maximum layer thickness of 1 m and compacted with vibrating rollers having a static weight of 13·5 t. Sluicing of the rockfill with 2 m^3 of water for each m^3 of rockfill was specified, but in practice the amount of sluicing water was less. A continuous effort was made during placement of the rockfill to select the larger fragments of rock for placement near the slopes, leaving the finer rockfill adjoining the filters.

The clay core was constructed with red clays won from the riverbed area within the reservoir and downstream from the dam. These soils were the weathered product of diabase and could be described as a combination of residual soils and colluvium and they are classified as clays of medium plasticity. Placement in the core of the dam was carried out at Proctor optimum moisture (±2%). Wetter clay was placed in contact with the rock foundation in order to achieve a more plastic fill against the steep abutments (opt +4%). The clay was compacted to an average dry density 99% of the Proctor maximum. Field permeabilities varied between 10^{-9} and 10^{-10} m/s.

The filter material was won from borrow areas in the riverbed downstream from the dam and included gravels from the riverbed and adjacent terrace deposits. Most of the sandy gravel material was used 'as dug' but stones greater than 127 mm (5 inches) were removed during placement. Minimal compaction was applied to reduce transmittance of arching forces from the clay core to the shoulders. Filter permeabilities measured on-site varied between 10^{-5} and 10^{-7} m/s.

Foundation treatment included application of dental concrete and slush grout in the clay contact area and the formation of a grout curtain underneath the centreline of the embankment. Grouting was carried out from five rows of holes, the central row regarded as the main grout curtain, reaching a depth of one-third of the head of water plus 17 m. The adjacent rows were much shallower, and were placed to form the contact grout area. Grout takes were generally low and only 132 t of cement were used. Average takes were below 20 kg/m depth of hole.

4. PERFORMANCE OF THE DAM

The performance of the dam during its 30 years of operation has been continuously monitored using the instrumentation installed during construction and through visual observation. Fig. 3 shows the installation of some of the instruments. Two sections (i.e., the maximum (section B) and the one on the left abutment (section A)) were equipped with instruments within the body of the dam as well as settlement markers on the surface. The third section (on the right abutment (section C)) was provided with surface settlement markers only. Instrumented section B (maximum section) is shown in Fig. 4. The following equipment was installed.

(*a*) Fifteen Bishop-type, high-air-entry hydraulic piezometers.
(*b*) Ten BRS-type vibrating-wire total pressure cells arranged in two clusters of five cells each at the maximum section.
(*c*) Five hydraulic overflow-type internal settlement gauges at approximately mid-height of the maximum section.
(*d*) Thirty-six surface settlement markers at the three sections.

There are two instrument houses, one on the downstream slope


Fig. 3. Installation of some of the instruments

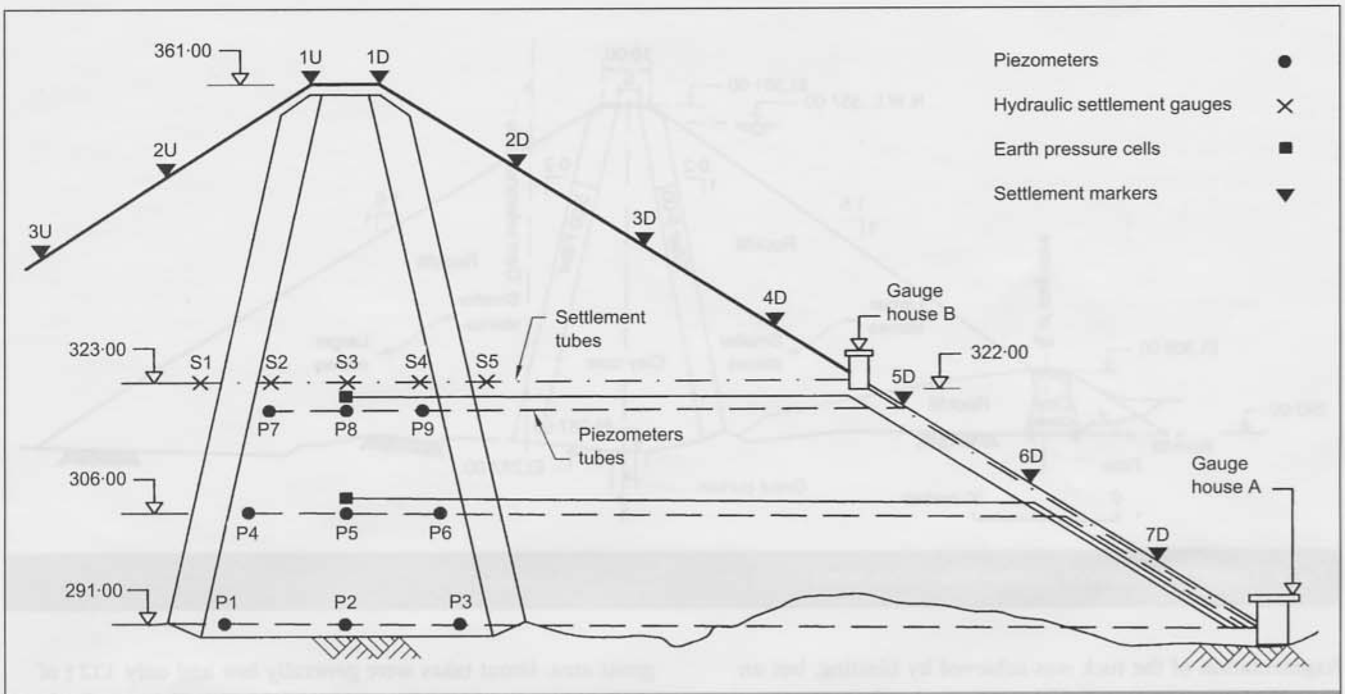

Fig. 4. Instrumented section B (elevations in m)

and one at the toe to record the instruments embedded in the embankment.

4.1. Embankment settlements

Internal embankment settlements have been recorded by the five hydraulic overflow settlement cells installed just below mid-height at the max section (B) as shown in Fig. 4. These instruments performed well during the first ten years of their operation but subsequently great effort was needed to ensure that the three tubes connected to the cells were sufficiently clear to ensure that air pressure in the cells was atmospheric. The most probable cause of these problems was blockage of the drain tubes by growth of algae. Readings were obtained up until 1994 (i.e., for more than 21 years) when these instruments were abandoned. The instruments have nevertheless recorded the development of settlements during this period accurately, as shown in Fig. 5.

As would be expected, maximum settlement occurred at mid-

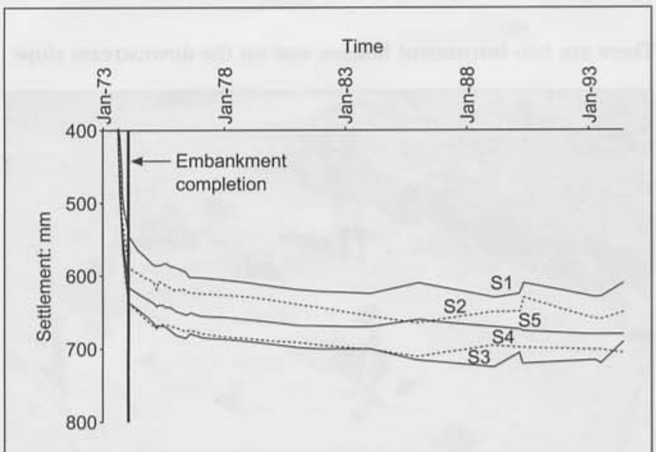

Fig. 5. Internal settlements recorded by the hydraulic overflow settlement cells

width of the core (S_3). This maximum settlement was of the order of 720 mm and out of this about 600 mm occurred during construction and about 120 mm after completion of the construction works. The average post-construction settlements recorded by the three cells installed in the clay core (i.e. S_2, S_3 and S_4) was of the order of 80 mm. Although two of the instruments (S_1 and S_5) were installed in the rockfill and the remaining three in the clay core, settlements were of comparable magnitudes and demonstrate a smooth transition between the rockfill and the clay. It is usual for the maximum construction settlements in dams to occur at between one-third and one-half of the embankment height and it seems reasonable to assume that this pattern applied at Lefkara. The settlements recorded by the internal settlement cells at mid-height of the embankment are therefore expected to be very close to the true maximum settlements of the embankment. The magnitude of these settlements is slightly lower than the corresponding settlements from other rockfill dams in Cyprus and this may be attributed to the good quality of rockfill, the effect of sluicing and the high degree of compaction.

Post-construction surface settlements have been monitored by recording the settlements of surface settlement markers installed along lines A, B and C using high-precision survey equipment to relate their movements to stable reference datum positions on rock outcrops away from the dam. The markers were installed in February and March 1974, two to three months after completion of the embankment. They have now (2004) recorded maximum post-construction settlements of the order of 245 mm on the maximum section. The development of post-construction settlements from six surface settlement markers (three upstream and three downstream of the dam centreline) on section B is shown in Fig. 6. These settlements were found to have an obvious degree of dependency on the depth of fill below, although it needs be pointed out that due to the effect of saturation of the rockfill, settlements on the upstream shoulder were greater than on the downstream

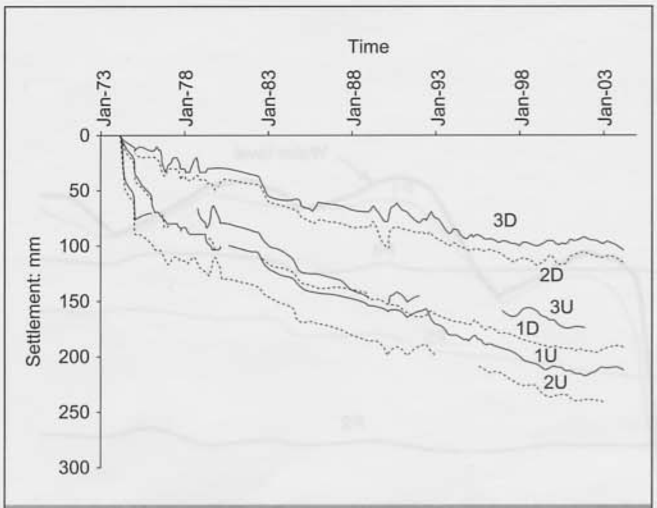

Fig. 6. Development of post construction settlements of six surface settlement markers (line B)

shoulder, as demonstrated by the plot of depth of fill against settlement in Fig. 7. A comparison of the internal settlements recorded by the hydraulic overflow cells and the surface settlements on line B clearly demonstrates that post-construction compression of the upper half of the embankment represents more than 60% of the post-construction compression of the entire embankment.

4.2. Pore water pressures in the clay core

The build up of pore water pressures within the dam core during and after construction has been recorded by the 15 high-air-entry hydraulic piezometers installed at instrumented sections A and B. Pressures were initially being measured with very accurate mercury manometers, but 15 years after installation readings became extremely difficult due to excessive staining inside the manometer tubes and the panels of mercury manometers were subsequently replaced by a new panel incorporating a digital transducer. All piezometers are operational 30 years after their installation.

The piezometers have given a clear picture of the build-up of pore water pressures in the clay core during construction and their subsequent dissipation, as well as the variation of seepage pressures with the water levels in the reservoir. The variation of

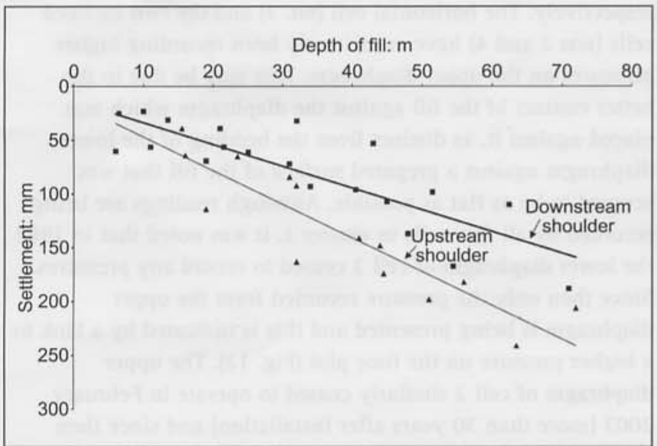

Fig. 7. Relation between post construction settlements and depth of fill

pore water pressures with time during construction and the first five years of operation of the dam is typically represented by instruments P_1, P_2, P_3, P_5 and P_8 in Fig 8. Sharper responses to the weight of fill material placed above the piezometers are shown by the instruments located in the centre of the core (i.e., P_2, P_5 and P_8), due to the associated longer drainage paths. The variation of typical values of the pore pressure ratio, r_u (defined as the ratio of the pore water pressure to the total fill pressure), with depth of fill is shown in Fig. 9 which clearly demonstrates that piezometer P_2, situated in the middle of the clay core at the lowest elevation, recorded the highest r_u value (i.e., 0·40). The slow dissipation is an indication of the impermeable nature of the foundation. Instruments at higher elevations recorded lower r_u values, possibly due in part to the smaller confinement due to the reduced amount of rockfill on either side. It is interesting to note the faster reduction in the r_u values towards the final stages of construction when fill placement was slower.

The different r_u values recorded by piezometers P_1 and P_3 situated on either side of piezometer P_2 at the lower level, may be attributed to: (1) the ponding of sluicing water upstream from the clay core; (2) possible higher placement moisture content around instrument P_1; and (3) a higher degree of lateral confinement on the upstream side due to the slope of the rock foundation. After placement of the rockfill and filters, the clay may have been prevented from moving laterally in an upstream direction due to the rock foundation, which is sloping towards the clay core. On the downstream side, the slope is away from the clay core giving the core a better chance to expand downstream when compacted. The piezometric elevations recorded by the same instruments during the last 15 years are shown in Fig. 10 and they clearly represent the pore water pressures due to the transient conditions of seepage.

4.3. Total pressures in the clay core

Total pressures are measured at the centre of the clay core, at two different elevations (306 and 323 m amsl) as shown by Fig. 4. Measurements are being obtained from ten BRS-type, vibrating-wire earth-pressure cells, arranged in two clusters of five cells each. The design and construction of the BRS earth-pressure cells has been described by Thomas and Ward.[1] This type of cell is basically a steel capsule 280 mm in diameter and 38 mm in height which encloses a space 152 mm in diameter and 25 mm high. It incorporates two diaphragms, one on each side, the deflections of which are measured with vibrating-wire strain gauges. Their frequencies of vibration are measured in an instrument house with a stable oscillator. The two diaphragms of each buried cell measure the component of earth pressure in a direction normal to their surface. Under ideal conditions the readings from the two diaphragms should be similar but as these are rarely the same they are averaged to give the magnitude of total stress in a given direction. Each pressure cell is provided with two air lines for the circulation of dry nitrogen gas to keep the inside of the cell at atmospheric pressure and to prevent corrosion of the vibrating wires.

At Lefkara, four of the cells of each cluster were arranged to form a 45° rosette with a check gauge, which is the normal method used for carrying out two-dimensional stress measurements. These cells measure stresses across the dam axis

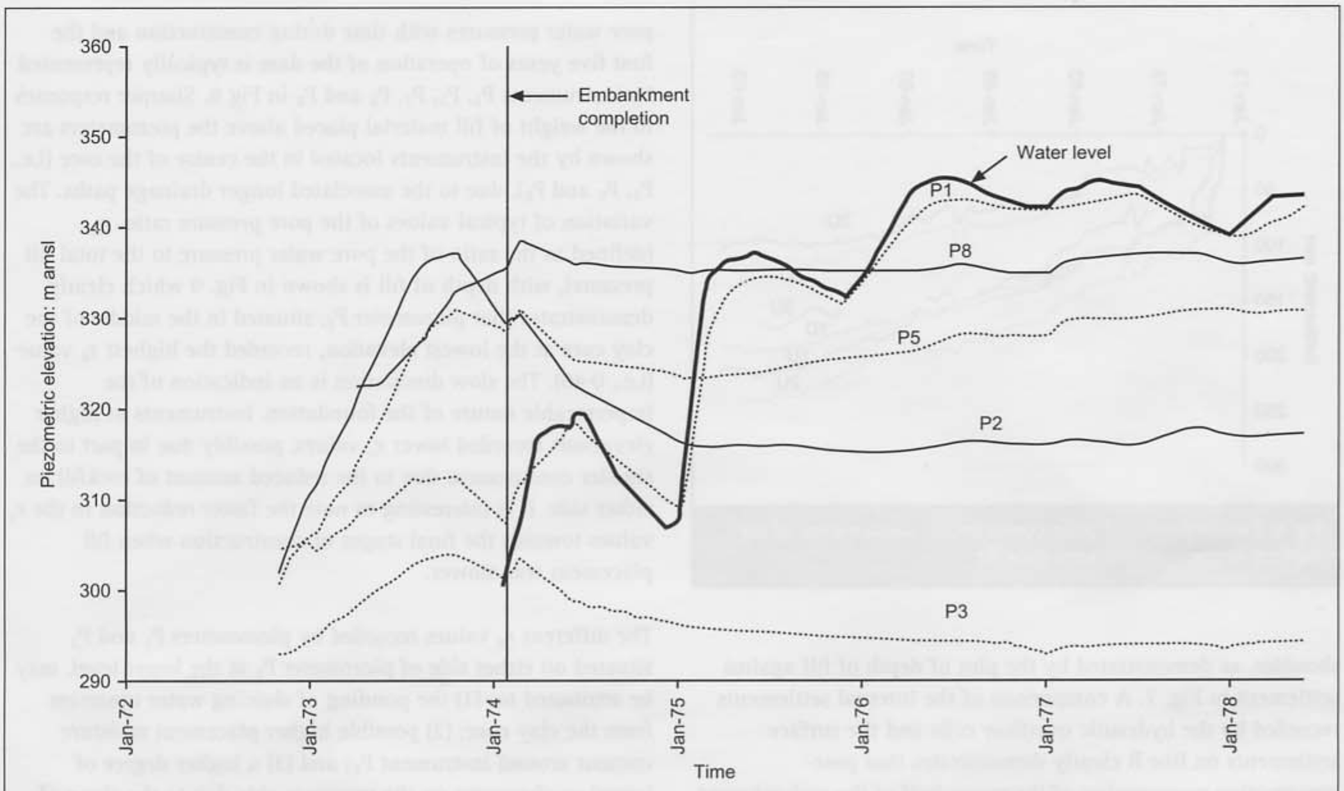

Fig. 8. Variation of pore water pressure during construction and during the first five years of operation (amsl, above mean sea level)

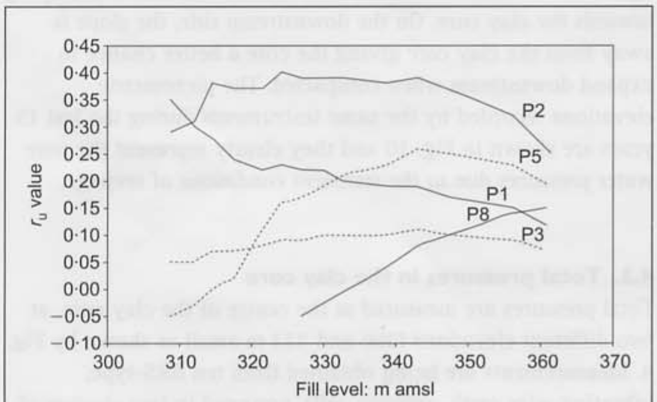

Fig. 9. Variation of r_u value with placement of fill during construction (amsl, above mean sea level).

but a fifth cell has also been installed to measure stresses along the dam axis, as indicated by Fig. 11.

The variation with time of average total pressures recorded from the two clusters of cells is shown by Figs 12 and 13. For comparison purposes these figures also show the variation of reservoir head above the level of the pressure cells (expressed in kPa) and the pore water pressure recorded by the two hydraulic piezometers installed at the location of each cluster of total pressure cells (P_5 and P_8). All instruments have shown a rapid response to the overburden pressures when building the embankment and a subsequent drop in pressure just after completion of the construction work, most probably due to consolidation effects. The magnitudes of the average recorded pressures remained relatively unaffected with time, although some fluctuations have been observed to take place with the variation of the water level in the reservoir. This variation can be expected to vary the total thrust on the upstream face of the clay core. As expected, these differences were more pronounced in the case of cells 4 and 9 which were installed with a 45° upstream inclination, the surfaces of which seem to coincide with the direction of total thrust.

In cluster 1 installed at an elevation of 306 m amsl, the highest average pressure is recorded by the horizontal cell (no. 3), followed by the two cells installed at 45° (nos 2 and 4) and then by the vertical cell measuring horizontal pressures in an upstream–downstream direction. The lowest pressures were recorded in a direction along the dam axis (no. 1). Considerable differences have been recorded by the two diaphragms of each cell. These different pressures can be expressed as percentages of the higher measured pressure and these were typically of the order of 4%, 8%, 28%, 38% and 15% for cells 1 to 5, respectively. The horizontal cell (no. 3) and the two inclined cells (nos 2 and 4) have consistently been recording higher pressures on the upper diaphragm. This may be due to the better contact of the fill against the diaphragm which was placed against it, as distinct from the bedding of the lower diaphragm against a prepared surface of the fill that was scraped to be as flat as possible. Although readings are being recorded by all five cells in cluster 1, it was noted that in 1988 the lower diaphragm of cell 3 ceased to record any pressures. Since then only the pressure recorded from the upper diaphragm is being presented and this is indicated by a kink to a higher pressure on the time plot (Fig. 12). The upper diaphragm of cell 2 similarly ceased to operate in February 2003 (more than 30 years after installation) and since then only pressures from the lower diaphragm are presented.

At the location of cluster 1 the overburden has been estimated

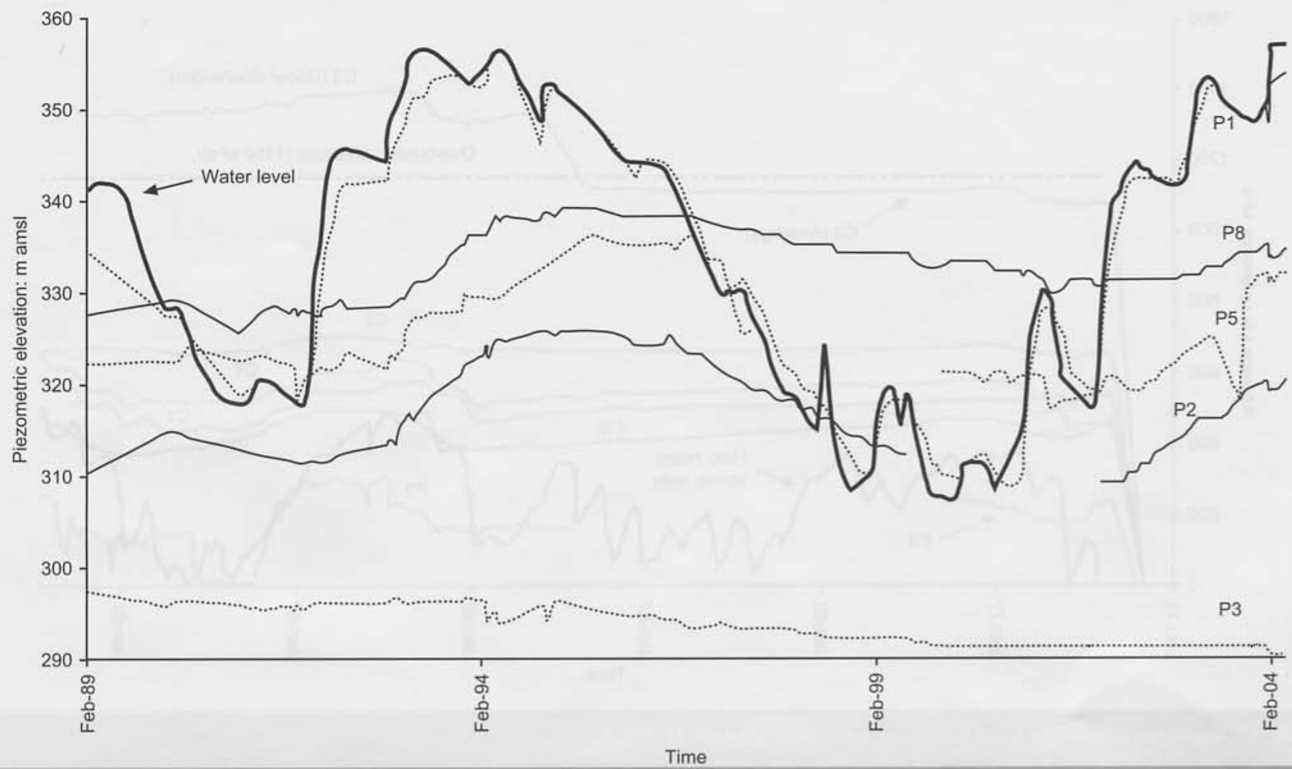

Fig. 10. Variation of pore water pressure recorded by piezometers during the last 15 years of operation (amsl, above mean sea level)

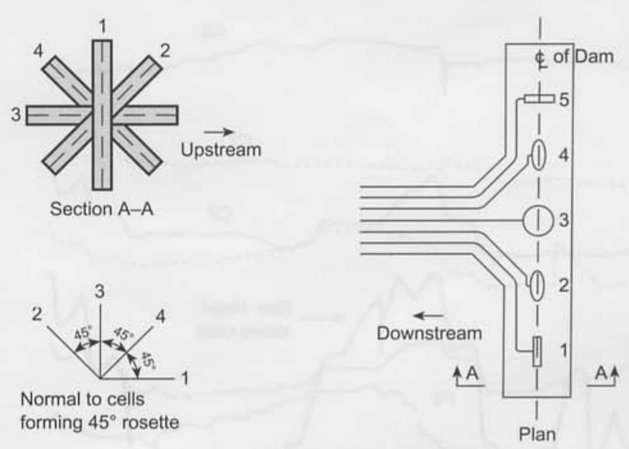

Fig. 11. Arrangement of total pressure cells of cluster 1 and 2

to be 1150 kN/m^2. The average pressure recorded by cell 3 up until 1988 was slightly below this value but the pressures from the upper diaphragm presented thereafter are between 16% and 21% higher than the overburden. Assuming this value is an overestimate and by applying a cell action factor of 1·09 as suggested by Buck,[2] the corrected pressure given by Fig. 12 is between 8% and 13% higher than the overburden. Because of the expected and measured greater settlement of the core, some transfer of loading from the core to the rockfill shoulders was expected, so that it was concluded that the pressure recorded is in fact an overestimate. On the other hand, there is no need to expect the total pressure to exactly reflect the height of overburden above a particular point in the complex three-dimensional shape of the dam, made up as it is from several materials of varying properties. It is also noted that although total pressure cells are accurate, their interaction with the environment is very complicated and in some cases measurements may differ from the true *in situ* pressures.[3,4]

Regarding the possibility of hydraulic fracture in the clay core,[5,6] it is noted that the pore water pressures recorded by piezometer P_5, installed adjacent to the cluster of total pressure cells, has always been below any of the corresponding total pressures recorded, hence effective stresses in all directions are positive.

The behaviour of cluster 2 was to a certain extent similar to cluster 1. Pressures in a vertical direction at the horizontal cell (no. 8) were the highest, with next lower pressures measured by the two inclined cells (nos 7 and 9). The order of stresses recorded by the vertical cells 6 and 10 was different however, with the horizontal stresses in the direction of the dam axis (no. 10) being higher than those normal to it (no. 6). Evidence of this higher horizontal pressure along the dam axis with its associated compression strains has been observed by arching of kerbing stones at crest level as shown by Fig. 14. As in the case of cluster 1 there were differences between the two diaphragms of the cells, which were typically of the order of 8%, 9%, 18%, 45% and 6% of the higher measured pressures for cells 6 to 10, respectively. Similar to cluster 1, the upper diaphragms also record higher pressures than the lower ones. It is noted that one of the diaphragms of cell 6 ceased to operate in 1995 so the pressure presented thereafter is that recorded by the remaining diaphragm.

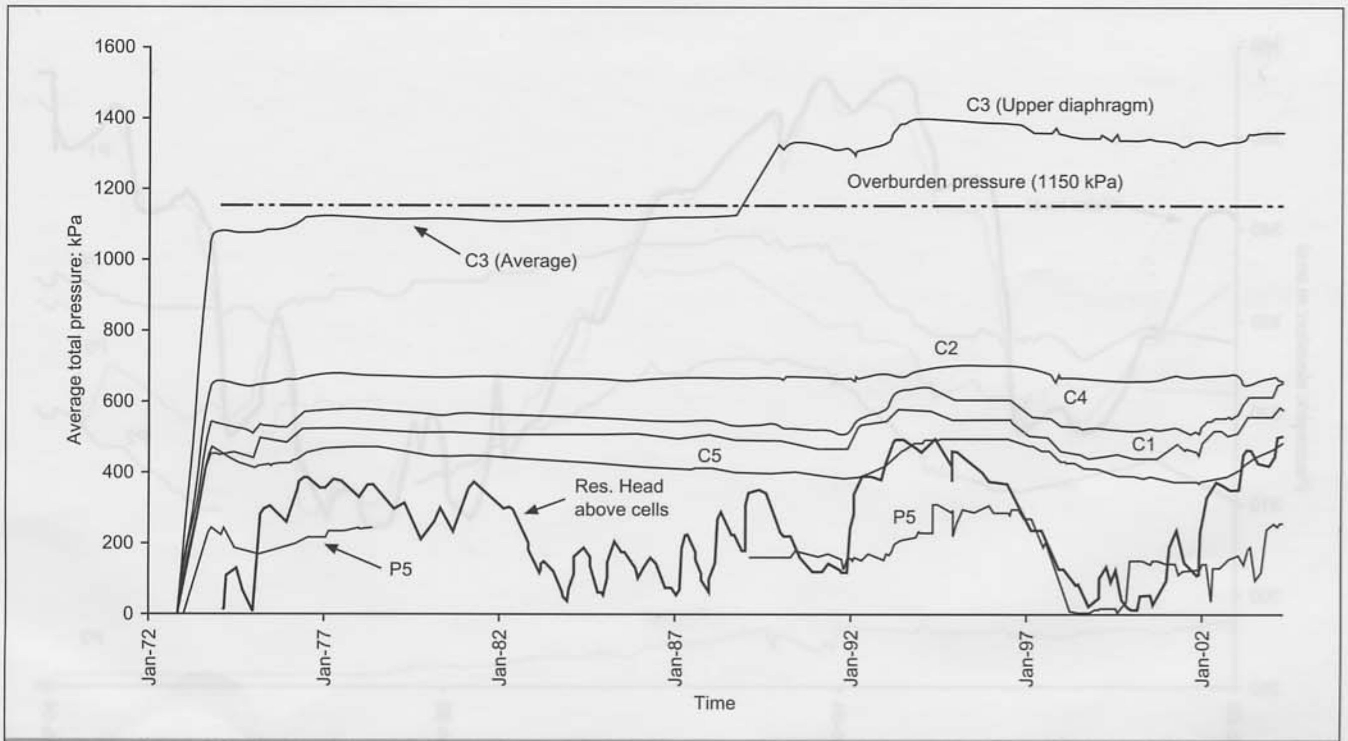

Fig. 12. Variation of average total pressures with time (cluster 1)

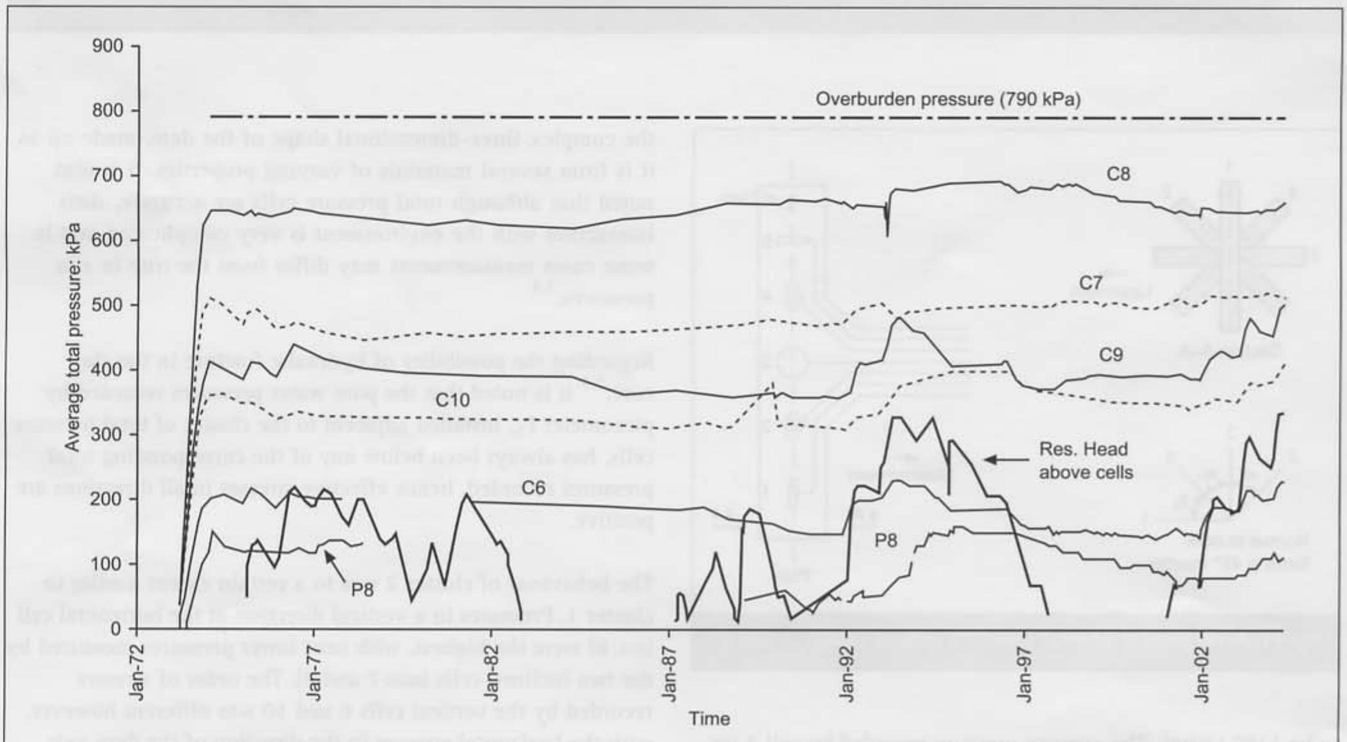

Fig. 13. Variation of average total pressures with time (cluster 2)

At the location of cluster 2 the overburden has been estimated to be 790 kN/m^2 and the vertical pressure recorded by cell 8 is between 80% and 87% of this value. Despite the high horizontal pressure, a more acute arching action at this position may have carried some of the overburden weight. Comparing the pore water pressures recorded by piezometer P_8 and the lowest total horizontal pressures recorded by the cells of cluster 2 it is evident that effective stresses in all directions are positive at all times. The total pressures, shown by Figs 12 and 13, have subsequently been corrected by applying the cell action factors proposed by Buck[2] and adjusted to fit a Mohr's stress circle.[1]

The ratio of the major to the minor principal stresses was found to vary between 0·32 and 0·52 in cluster 1 and 0·28 and 0·44 in cluster 2. The major principal stresses were found to dip upstream between 2 and 10° at cluster 1 and between 0 and 9° in cluster 2.

Fig. 14. Photograph of kerb stones arched by horizontal strains along crest

During the periods of high reservoir levels there is a rotation of principal stresses and the dip angle becomes nearer to the vertical. The tendency for principal stresses to dip upstream even when the reservoir contains substantial amounts of water may be attributed to a certain degree to the topography of the dam site and its steep downstream inclination but also to the steeper upstream embankment slope.

5. SEEPAGE MEASUREMENTS

Seepages are being measured at the dam in order to assess the loss of water through the embankment/foundation and the outlet tunnel. Embankment seepages are being monitored through a sophisticated system of underdrains, which was designed specifically to identify broader zones through which seepages are occurring. Underdrains were thus constructed to measure seepages in two zones between 295 and 309 m amsl (one on each flank) and in two zones above an elevation of 309 m amsl (one on each flank). Total flows which include flows between the riverbed and elevation 295 m amsl are being monitored at a measuring weir about 50 m downstream of the embankment toe.

Measurements during the 30 years of operation of the dam have shown that seepages through the embankment and its foundation are very low. Out of the four underdrains above 295 m amsl only one has ever shown any measurable seepage (10 l/min) whilst the total seepage recorded at the weir was less than 60 l/min when the dam was full.

The corresponding seepages through the diversion tunnel have reached 50 l/min when the reservoir was nearly full but these came mainly from the inclined gallery where reservoir water leaks into the gallery through the expansion joints.

6. PROBLEMS ARISING FROM THE LEAKAGES INSIDE THE INCLINED GALLERY

The inclined gallery is a reinforced-concrete structure resting on sound rock foundation at a slope of 37° to the horizontal. It has an internal diameter of 2·8 m and a square external section. Considerable excavation was required to reach sound rock and in effect the structure is half buried. The gallery accommodates four raw-water intakes at various levels with the water main running along its invert.

The gallery, shown by Fig. 15, was cast in 6 m long monolithic sections with expansion joints incorporating rubber centre bulb water stops. With the rise of reservoir water it was noticed that some of these joints leaked. Attempts to seal the leakages from

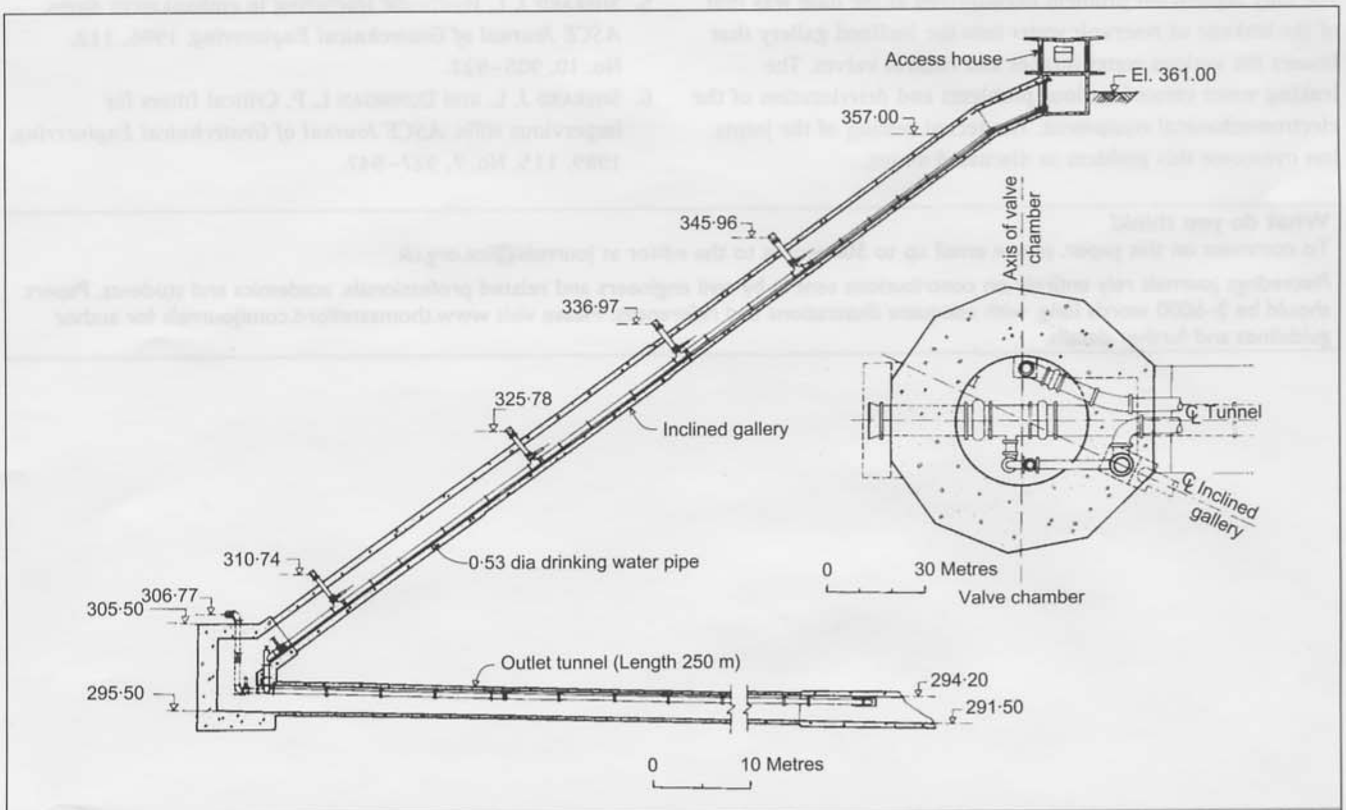

Fig. 15. Section through the inclined gallery (elevations in m)

inside the gallery have been unsuccessful for many years especially at the lower elevations where pressure is in excess of 60 m head of water. Sealing the joints from outside seems an impossible task in view of the half-buried nature of the gallery, but recently a new approach has been used whereby small holes are drilled into the side of the joint and a cement grout injected. This method proved successful and the leakage into the galley is now negligible. This work was carried out at the end of 2003 and movements of the joints may produce some leakage again.

Before the effective work of sealing the joints was carried out, the flow of water inside the gallery created numerous problems which included access difficulties, danger from electric shock, deterioration of electromechanical equipment, accelerated corrosion etc.

7. CONCLUSIONS

During its first 30 years of operation Lefkara dam has served well its purpose of impounding good-quality water for the domestic water supply needs of Famagusta and Larnaca. Due to the reduction in rainfall during the last 30 years of the twentieth century, the yield of the dam has been well below that anticipated.

The embankment performed very well during this period with little deterioration of the construction materials, very small seepage losses and small deformations. The instrumentation installed in the dam during its construction is still functional, more than 30 years after its installation, providing valuable information on the behaviour of the dam. Particularly impressive is the performance and durability of the hydraulic piezometers and the BRS-type total pressure cells.

The only significant problem encountered at the dam was that of the leakage of reservoir water into the inclined gallery that houses the various water intakes and control valves. The leaking water created various problems and deterioration of the electromechanical equipment. The recent sealing of the joints has overcome this problem as discussed above.

Overall the dam is performing well and with proper maintenance it is expected to serve its purpose for many more decades.

8. ACKNOWLEDGEMENTS

The authors are grateful for the help they have received from their colleagues, particularly those who have taken the numerous readings from the instruments from the time they were first installed. The regular and reliable supply of dry nitrogen for the pressure gauges, involving the manhandling of very heavy steel storage cylinders, and the skilful replacement of the mercury manometer panels by new electronic measuring equipment is work of particular note. The authors are indebted to the Ministry of Agriculture, Natural Resources and Environment of the Republic of Cyprus for permission to publish this paper.

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