VAL DE LA MARE DAM
JERSEY, CHANNEL ISLANDS

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ABSTRACT

Val de la Mare Dam is the principle storage reservoir for the Island of Jersey. The dam has been regularly inspected since completion in 1962.

In January, 1971, small upstream relative movements of the handrail of the crest walkway bridge were noticed. At the same time, darkening and damp patches were observed on the downstream face of the dam and parts of the surface showed random cracking of the concrete.

Ercon were retained to investigate the problem, and a programme of investigations was initiated. Alkali aggregate reactivity was diagnosed as the cause of the defects.

Proposals were developed, and subsequently implemented, for remedial works which included the provision of drainage into the gallery, the grouting of and the installation of anchors in a section of the dam most adversely affected by the reaction, and the installation of appropriate instrumentation to monitor the loads on the anchors, future movements of the particular section, and uplift pressures.

Introduction

The dam has a maximum height of 23 m above the valley floor, a crest length of 170 m, and is constructed of mass concrete in 6.7 m wide blocks with the lift (pour) heights being generally 1.2 m. The foundation is of Precambrian sedimentary shales.

The dam was designed using the middle third rule allowing for an internal uplift pressure of 50% of reservoir head at the upstream face decreasing linearly to zero at the downstream face, based on 100% of plan area.

Construction took place over the period 1957 to 1962, with the coarse aggregate being obtained from a local quarry, and from oversize material from a beach, the fine aggregate from a beach, and the cement being imported from the U.K. A photograph of the dam is shown in Fig. '1'.

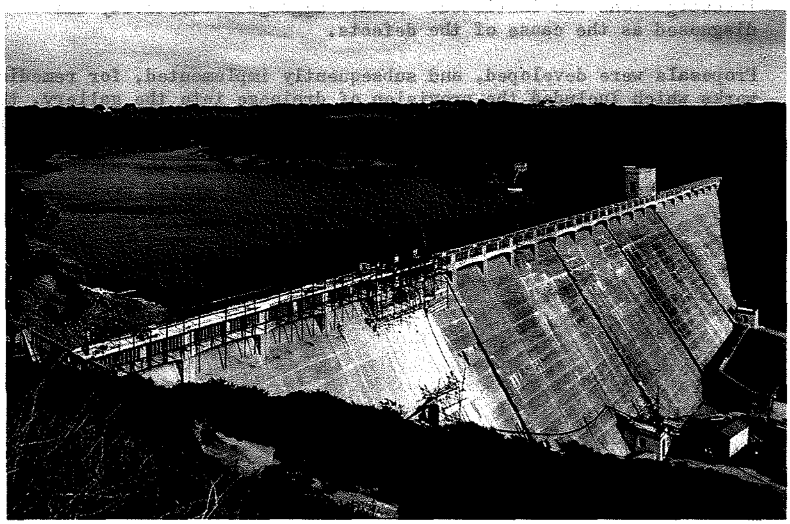

FIG. 1
Val de la Mare Dam

The first signs of trouble appeared in January 1971, following a period of colder than usual weather. Small upstream relative movements in the order of 6 to 12 mm were noticed on certain sections of the concrete handrail of the crest walkway bridge, and darkening and damp patches were observed at the same time on the downstream face of the dam on the same blocks on which the movements had occurred, with the surface showing random hairline cracking.

Following Ercon's initial inspections of the dam and the quarry, it was thought likely that alkali aggregate reaction could be the cause of the problem even though the aggregate quarry had been in operation for several years and there had been no published records of an occurrence on Jersey. A programme of investigations was therefore initiated. Also several theories, including frost action, sulphate attack and earthquake action, were considered and investigated, but eventually discounted.

Investigation Programme

The investigation programme, which was carried out during the period July 1971 to September 1973, included the following :-

(i) Consultations with the Cement and Concrete Association, the American Corps of Engineers, the Concrete Research Laboratory in Karlstrup, and the British Museum (Natural History), Department of Mineralogy. The consultations were linked with a review of the existing literature on the subject which revealed that experience of the phenomenon in the U.K. was limited. Overseas case histories were therefore examined and discussed, and visits were paid to Jersey by specialists in the field of testing and deterioration of concrete.

(ii) Review of the method and sequence of construction of the dam and of the source of supply of the relevant construction materials :- investigations revealed that affected lifts were randomly sited in the dam, but were associated in time.

(iii) Discussions with the U.K. Suppliers on the cement shipped to Jersey during the appropriate period of construction. These discussions revealed that cement shipments to Jersey in 1960 during periods significantly related to the time of construction of affected block lifts contained cement with alkali contents of values up to 0.95 percent measured on a monthly average basis and expressed as a percentage of Na_2O.

(iv) Geological survey of, and testing programme on aggregate samples from, the local quarry, and examination of the sources and testing of samples of the local beach sand. Reactive materials were identified in some of the materials obtained both from the local quarry and from the beach, and a materials testing programme was initiated.

(v) Core drilling in specific sections of the dam, followed by laboratory description and testing.

(vi) Petrographic examinations of thin sections of the concrete cores and of the aggregates from sources considered to be the same as used in the concrete and from various other localitities on the island - carried out by Ercon and by the Building Research Establishment. The examinations showed that the samples contained reactive silica which could give rise to alkali aggregate expansive reactions. The silica was present as either opal or chalcedony.

(vii) Expansion testing on cores taken from the dam and on mortar bars made from local quarry rock and beach sand - carried out by the Building Research Establishment. The mortar bar tests on the aggregate samples showed that the aggregates were expansive.

(viii) In-situ sonic velocity measurements in each accessible lift of each block of the dam. A first series of measurements was carried out in September 1972, and a second series in July 1973. The technique was used largely to give a qualitative assessment of the concrete quality, and four classifications of concrete were made on the basis of the measured velocities. Comparisons were made between the two series of measurements, and areas of significant deterioration were identified by significantly lower velocities.

(ix) Installation of electrically operated piezometers in certain sections of the dam, the locations chosen on the basis of the sonic test results. The limited number of piezometers installed indicated that the original design uplift pressures were not being exceeded, except in the case of Block 6 where the higher pressures were resulting in a reduction in the design factors of safety against overturning and sliding.

Assessment of Remedial Measures

Following the investigation programme and instrument readings taken over a period of more than two years, the decisions then had to be made on:-

(a) Whether or not the dam would eventually deteriorate to the point of being unserviceable.

(b) If so, how long could it remain in service before an alternative water supply source had to be found.

(c) If not, what remedial works were required to keep the dam in a safe stability condition.

It was not possible to make a completely quantitative decision on the matter. Engineering judgement, based on the whole range of accumulated information, and taking into account social and economic factors, had to enter into the decision.

Several general factors were of significance and influenced the ultimate decision :-

(i) Although the reactive material had to be assumed to be present throughout the dam, the degree of attack and concrete deterioration varied. This was borne out by the sonic test results, the crest movements and visual inspection of the downstream face of the dam. The known variation of alkali content of the cement delivered and the fact that most of the reactive material was thought to be derived from a number of veins in the quarry and from beach pebbles, containing either chalcedony or opal, also supported this assessment.

(ii) The expansion test results carried out by the Building Research Establishment on concrete cores from the dam containing suspected reactive material and on mortar bars manufactured from coarse aggregate from the Ronez Quarry and beach pebbles, all containing reactive silica, indicated that the degree of expansion was not as high as the values considered unacceptable in the U.S.A. and Canada, based on A.S.T.M. C. 33-67 and C.227-69.

(iii) Case studies of other dams indicated that the initial large rate of expansion caused by the reaction was not maintained with time.

(iv) There was no proven method of chemical treatment that would either stop or reduce the reaction and expansion.

(v) The readings of internal uplift pressures from the installed piezometers were acceptable, except in the case of Block 6.

Based on the evidence accumulated on this particular dam and on the results of other cases, a conclusion was reached that the concrete would not deteriorate to the extent that it would be incapable of taking the required compressive loads. The danger was that expansive cracking could lead to higher internal uplift pressures than had been allowed for in the design, resulting in instability.

Hence, the policy was adopted of proceeding with remedial works to ensure that the stability of the dam was maintained against increased internal uplift, assuming that the concrete would be capable of taking all the stresses applied to it in the future. Also, as the alkali aggregate reaction varied over the dam, only isolated bad sections would be dealt with, and the remedial method adopted should be capable of being extended in stages to the complete dam at a later date should further deterioration occur. Any remedial works adopted had to be such that the reservoir could be kept in operation at all times, and a minimum of restriction placed on the water level during the work.

Decision on Remedial Measures

After consideration of many alternatives, the decision on the type and extent of remedial measures was made related to Block 6 and Blocks 10-20. Block 6 is situated near the right abutment, and is clearly indicated on Fig. 1 by the presence of scaffolding and equipment. Blocks 10-20 are in the centre section of the dam between the two gallery openings sited against the downstream toe.

Block 6

The remedial measures chosen for Block 6 had two main aims. Firstly, grouting was proposed as a trial measure to ascertain whether a curtain could be introduced near the upstream face of the block to reduce seepage uplift pressures and to reduce future alkali aggregate reaction by limiting available water. Piezometers could also be installed to check the effects of the curtain.

Secondly, to counteract the higher internal uplift pressures, it was proposed to anchor the block to the foundation rock beneath the dam by installing prestressing tendons. The required anchor force would be determined from static considerations of stability to provide for a minimum factor of safety against overturning of 1.7 under full flood condition loading and with hydrostatic uplift assumed to act over 100% of the plan area of the block, with full hydrostatic head at the upstream face decreasing linearly to zero at the downstream face.

Blocks 10-20

On the remainder of the dam, it was not considered essential to undertake any remedial works. However, since drilling equipment would be mobilised, for Block 6 work, it was decided to take advantage of this by drilling drainage holes approximately 1.2 m from the upstream face of the dam in Blocks 10-20 where access was available from the inspection gallery. It was hoped that if cracking became serious, the holes would intercept seepage flow and reduce the internal uplift pressure downstream of the holes.

Remedial Works

The remedial works were carried out during the period June to December, 1974.

Site Investigation

As a necessary preliminary to the design of the anchoring of Block 6, an investigation of the foundation rock beneath the block was carried out. Two vertical boreholes and one inclined borehole were drilled from the downstream toe of the block. The inclined borehole was angled to penetrate through the middle of the proposed anchorage zone.

In addition, one vertical borehole was drilled from the crest of Block 6, down through the concrete of the dam into the proposed anchorage zone. The position in plan of this hole coincided with one of the proposed grout holes in the centre of the block (refer to Fig. 2). Laboratory tests were carried out on cores recovered from the boreholes - the tests produced data on the elastic modulus and poissons ratio of the rock, and the shear strength of the joints. All boreholes encountered very jointed and fissured rock.

In addition to the boring, rock outcrops in the vicinity of the dam were mapped geologically to supplement the borehole information.

Grouting of Block 6

Six holes were drilled vertically down through the concrete of the dam at a distance of 1.2 m from the upstream face and between 0.9 m and 1.32 m apart. Five of the holes were drilled in 80 mm diameter - the depth of the holes varied from 13.5 m to 17.0 m such that the bottoms of the holes were approximately 0.6 m into the concrete of the cut-off trench. The sixth hole, chosen as the position of the site investigation borehole, was cored in 100 mm through the concrete and foundation rock for a total depth of 34.5 m The positions of the grout holes are shown on Figs. 2 and 3.

A water test was carried out in each hole to test the permeability of the surrounding concrete - the tests showed the permeability to be very low.

The material chosen for grouting was Polythixon 60/40 DR grout. This is an oil-based chemical grout supplied in the form of two liquid phases which, when mixed, form a cross-linked polymer that sets to form a rubber-like substance. At the time of mixing, the grout has a very low viscosity and is therefore suitable for penetrating fine cracks. The quantity of Polythixon used in the grout holes was only slightly greater than the quantity of water used in the water tests, so it is probable that very little penetration was achieved.

Anchoring of Block 6

In order effectively to distribute the loads from the anchors, a high-strength reinforced concrete spreader beam was introduced as part of the crest lift of the downstream face extending for the full block width between the walk-way supports. The spreader beam was installed before any drilling work was carried out for the grouting, anchor tendons and instrumentation, and to avoid drilling through the freshly placed concrete of the spreader beam for the grout holes and the tendon holes, mild steel tubes were fixed in appropriate positions through the beam.

The anchoring of Block 6 consists of 3 No. 40 mm diameter 'Macalloy' high tensile anchors fixed in 115 mm diameter holes positioned symmetrically across the width of the Block at 2.23 m centres and drilled through the concrete of the dam into the foundation rock - refer to Figs. 2 and 3.

Calculations for the proposed depths and lengths of the anchorage zones were carried out using data from the laboratory testing of rock cores. Using the depths and anchorage lengths resulting from the analysis, a computer analysis of stresses and displacements in the dam and the foundation was carried out using a finite element technique based on the linear elastic plane strain model. The results of the computer analysis showed that there would be no areas of high stress concentration after the installation of the anchors and no excessive movements of the structure.

The final depths at which the anchorage zones were established were in fact much deeper due to the rock conditions encountered during drilling for the anchors. A major shear zone was detected below Block 6, lying parallel to the bedding and therefore dipping steeply upstream and towards the centre of the valley. Numerous fissures and areas of highly fractured brecciated rock associated with this shear zone were observed.

No problems were encountered drilling through the concrete of the dam - drilling was carried out using a 115 mm diameter down-the-hole hammer.

Drilling through rock was good initially until the highly fractured zones were encountered - in these zones the drill repeatedly jammed. When the hammer was freed and operating again, gravel size fragments of mudstone and some clay were flushed up the borehole. Collapsing of the holes frequently occurred, giving concern over the possibility of cavities being created in the fault zone.

In order to prevent further collapse in the holes and the formation of cavities, the method of drilling was changed from rotary percussive to coring using water flush, and a drilling/pressure grouting technique was employed. Repeated pressure grouting and re-drilling resulted in a build-up of grout in the fault zone, and the area was eventually stabilised. One of the advantages of the change in drilling method was that the rock cores could be examined and a better appreciation of the geology of the zone could be obtained.

Although the cores showed the rock in the chosen anchorage zones to be suitable, fissures and joints were noted in these cores, and water tests showed a sufficiently large take from the anchor holes for fissure grouting to be considered necessary.

Grouting was carried out in stages up each hole using hydraulic packers - the quantities of grout pumped were not high indicating that no large voids were present. When set, the grout was redrilled and the grouting repeated until water tests showed the anchorage zones to be tight.

As the anchors were being installed, denso tape was wrapped around the bars for approximately 3.0 m above the bonded zone and above this silicon grease was sprayed onto the bars. The presence of the denso tape immediately above the bonded zone ensured that there would be no transfer of bond stress between the anchor tendon and the rock at a higher level than was desirable.

The anchors were bonded to the rock over a 9.75 m anchorage length using a 0.45 w/c low alkali cement grout with a 14-day cube strength of $35 N/mm^2$. The grout was pumped through a 12 mm diameter pipe that was temporarily fixed to the lower end of the anchor sleeve and raised as grouting proceeded.

As a test of the ability of the foundation rock to carry successfully the anchor loads at the chosen depths, the centre anchor was stressed before drilling of the two outer anchors had been completed.

Stressing on all three anchors took place in 10-tonne increments up to 90 tonnes with readings being taken of the anchor bar extension, the load cells, the electrolevel and the inverted pendulum after each increment. Readings were plotted so that anything unusual would have been detected. After a waiting period of approximately 15 minutes, the load was increased by a further increment. At 90 tonnes, the load was held for 30 minutes before being relaxed to the design figure of 85 tonnes, and locked off. Readings of the demec points and the two extensometers were taken before and after each stressing operation.

When the two anchors had been stressed, the annular space above the bonded length of each anchor tendon was filled with Polythixon FR special grout to prevent corrosion of the tendons.

Instrumentation Installation

Instruments were installed in and adjacent to Block 6 to monitor behaviour during the stressing of the anchors and at regular intervals after the stressing. The individual sitings of the instrumentation are indicated diagrammatically on Figs. 2 and 3.

In Block 6 itself, the following instruments were installed :-

(i) Vibrating wire load cells at the head of each anchor.

(ii) A vertical extensometer down from the walkway bridge to a level five feet below the deepest anchor.

(iii) An inclined extensometer from a point two-thirds down the exposed downstream face of the dam through the cut-off trench into the foundation zone immediately upstream of the dam.

(iv) An inverted pendulum vertically down from the walkway bridge to the middle of Lift No. 7.

(v) Two electrically operated piezometers inclined down from the downstream face of the dam into the middle of Lift No. 5, the individual piezometers being positioned 1.5 m and 3.35 m from the upstream face of the dam.

(vi) Twenty four mechanical strain gauge positions (three points per position) on the downstream face of the dam distributed across the lift joints and the vertical joints at each side of the block.

In addition to the above, four electrolevels were positioned on the walkway piers, each electrolevel spanning the joint between adjacent blocks for Block Nos. 4 to 8.

Relief Drainage Holes in Blocks 10 - 20

The relief drainage works consist of 75 mm diameter holes drilled from the roof of the drainage gallery up to near the crest of the dam, and from the bottom of the gallery down to elevation 3.35 m as shown on Fig. 4.

The upward holes are vertical, with respect to the dam elevation, in Blocks 10 to 18, and were drilled generally at 3.35 m centres, except in three blocks where the regular spacing was adjusted to avoid clashing with existing piezometer installations. Inclined holes, with respect to the dam elevation, were provided for Blocks 19 and 20. The downward holes are vertical with respect to the dam elevation, and are again generally at 3.35 m centres, and were extended into the foundation rock to supplement the existing foundation relief drainage system.

The upward holes were capped at their exit points near the crest to prevent the ingress of water and dirt - the caps are removable for inspection and maintenance purposes. Both the upward and downward holes were provided with suitable capped ends in the gallery to facilitate measurement of flow.

Future Monitoring

Surveillance of the dam is continuing, principally by continual visual observation of the downstream face and of any movement at the crest of the dam, supported by readings on the piezometers and other instruments in Block 6 and by records of the flow from the drainage holes in Blocks 10 - 20. Sonic testing is also planned at three-year intervals.

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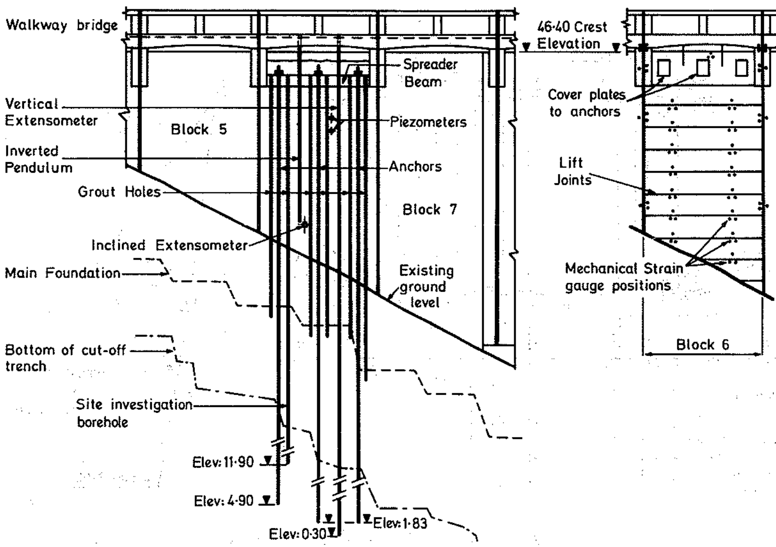

Fig.2 Locations of anchors, grout holes and instrumentation in Block 6.

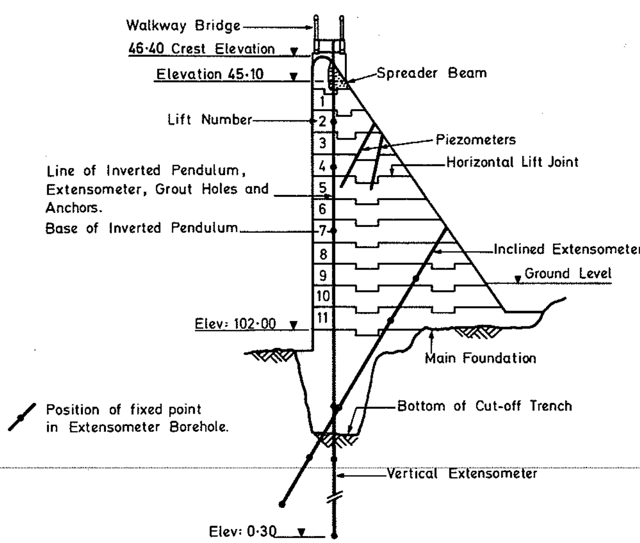

Fig.3 Section through Block 6 showing arrangement of anchors, grout holes and instrumentation.

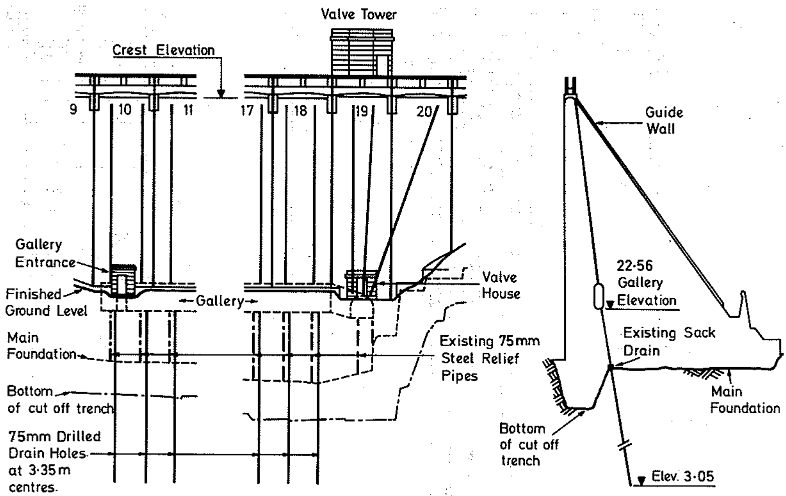

Fig.4 Relief Drainage Holes.