MONITORING OF CONCRETE GRAVITY DAMS

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by

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SUMMARY

Monitoring dams as an essential part of their operation is discussed. The major phenomena that are monitored viz. movements, uplift, leakage, stress, strain, chemical analysis of seepage waters, water table, seismic activity and cracks are described. An account is given of the Sydney Metropolitan Water Sewerage and Drainage Board's system of reporting monitored data. Case histories are described showing how recorded data indicated action was required and what remedial work ensued. Opinions expressed by the author in relation to the case histories are not necessarily supported by other members of the Board's engineering staff.

1. INTRODUCTION

It is doubtful whether any other branch of engineering carries responsibilities to the public equal to those involved in the design and supervision of dams that impound large bodies of water.

Of the phases of engineering effort associated with dams i.e. investigation, design, construction, testing of appurtenant works and operation, increasing attention is being paid in the operational phase to monitoring those phenomena which serve as indicators of the condition and stability of the dam.

Monitoring of dams will become more important in the future as larger dams are built and it becomes necessary to build dams on less favourable sites.

2. WHAT IS TO BE MONITORED AND WHY

This section will deal only with measurements of the behaviour of a dam primarily in its operational phase and will not include monitoring uniquely associated with the construction of a dam or testing of appurtenant works.

Observations can be broadly grouped into six categories according to the function each is intended to perform:-

Group A. Checking of Dam and Foundation Movements

- precise survey of dam and abutments
- clinometers
- collimator shafts
- gap gauges
- measurement of relative movement across inter-monolith joints

Group B. Observation of Dam and Foundation Leakage and Uplift

- flow from foundation drainholes drilled from galleries
- uplift pressure gauges
- flow from wall of dam and past seals
- flow under dam measured by V-notch weirs situated just downstream of dam
- assessment of abutment seepage with water table tell tale holes

Group C. Measurements for Comparison with Design Figures

- stress and strain meters
- uplift pressure gauges

Group D. Analyses to Indicate Long Term Integrity of Dam Concrete and Foundation Materials

- chemical analyses of drainage water

Group E Recording of Earthquakes

- seismographs

Group F. Crack Observations

- measurement of crack widths and lengths

Each of these groups will now be considered separately. Details of instruments will not be discussed as these change continually and information can be obtained from manufacturers.

2.1 Dam and Foundation Movements

All concrete gravity dams move in response to varying heights of water ponded behind them or in response to seasonal temperature variations in exposed concrete or rock. Movements can also occur in response to tectonic movements. A typical plot of precise survey data is shown in Figure 1. in which some of the results of the last seven years surveys done on Nepean Dam are shown.

In general, so long as the movements of both dam and surrounding rock can be seen to be consistent with the loading and recover satisfactorily when the load is removed they do not constitute any cause for concern. Recovery after removal of load is often slow i.e. there is a 'delayed elastic' effect rather than a simple 'elastic' one.

If however, movements are progressive and do not recover the dam could be in an early stage of failure and drastic measures such as immediate lowering of the water level or if this is not possible evacuation of people downstream of the dam may be necessary, while permanent solutions are sought.

Alternatively, movements may not recover but still be in the category of initial adjustment of the foundations to the newly imposed dam and water loads. After initial adjustment has taken place the site may be subsequently stable.

Thus interpretation of this data is often extremely difficult with the real magnitude of the movements clouded with doubts about the accuracy of the survey itself and the stability of reference points. A case in point was the failure of the Malpasset Dam in France in 1959. In hindsight it seems incredible that survey data was available that showed the progressive and almost inevitable process of failure taking place over some considerable time before collapse and no effective action was taken.

Precise survey involves survey of fixed targets on the dam and foundations in relation to targets remote from the dam by precise triangulation and/or direct measurements. At Warragamba Dam this survey system has targets three miles from the dam and has an accuracy of one part in 200,000 (1). Precise survey is expensive and time consuming but still its results are possibly the best indicators of the dams stability. The importance of monitoring movements becomes obvious when it is recognised firstly that 40% of all dam failures are foundation failure (2) and secondly that experience of underground excavations and open pit mining show that nature frequently gives warnings in the form of minor yielding and cracking in advance of a major failure.

Vertical deflections are measured by precise levelling from bench marks situated remote from the dam.

Tilt-meters or clinometers are essentially extremely sensitive spirit bubbles coupled to a micrometer and a reading representing the angle of tilt is recorded.

Collimator shafts may be constructed in a dam to allow precise survey measurements to be taken from top to bottom within the dam. This can be a check on the precise triangulation.

Gap gauges can be installed in joints for reasons other than indicators to assist in intermonolith grouting. For example the apron at Warragamba Dam is not designed to absorb direct thrusts from the main wall. Gap gauges were installed between dam and apron and between apron and rock at the down-stream end of the apron. Gap gauges between apron and dam monitored movements during the early stages of filling before this gap was sealed.

Relative movements across intermonolith joints are generally done by direct measurements (e.g. mechanical strain gauge) on to pins set in either side of the joint.

2.2 Leakage and Uplift

Variations in leakage and uplift can be the first sensitive indicators of any alteration in foundation conditions.

Drainage holes relieve the foundations of some uplift and uplift pressure gauges monitor the residual hydrostatic pressures under the dam and apron and hence the adequacy of the drainage system.

Uplift measuring piezometers generally intercept the rock/concrete junction under a dam. These can be installed either during construction or drilled subsequently from galleries or the downstream face.

Drainage holes are generally drilled from the drainage gallery into the foundations after grouting is complete.

When a gravity dam is designed with a drainage system an assumption is made regarding the magnitude of the uplift. When the recorded uplift is in excess of that assumed in design additional drainage must be provided until actual uplift is within design limits. Since drain holes and uplift holes tend to block up with time their reaming or replacement is a continuing maintenance problem.

Flows from cracks in the dam wall or past seals as well as foundation flows emerging from rock just downstream of the dam can be monitored by reading suitably located V-notch weirs. In the case of weirs situated outside the dam due allowance must be made for the influence of rainfall that has fallen just prior to readings being taken.

Possibly the highest concrete gravity dam to have failed was the 205 ft. high St. Francis Dam in California, U.S.A. The dam was founded partly on schist and partly on conglomerate and was completed in 1926. The failure took place about two years after completion of the dam and was due to disintegration of the conglomerate when saturated. Seepage was noticed from the conglomerate when the dam first filled about a year before the collapse on 12th March, 1928 (3). A proper appraisal of all implications of this leakage may have averted the disaster.

Monitoring of the water table by measuring the height of water in boreholes can indicate the adequacy of wing curtain grouting and drainage. A high water table can induce instability of rock in the abutments due to the introduction of unbalanced water pressures into joints paralleling the gorge downstream of the dam.

2.3 Measurements to compare with Design Figures

All dam analyses are idealised to some extent and it is ususually difficult to make accurate assumptions about such basic quantities as modular ratio of rock and concrete and what residual regional stresses if any exist in the area. Rosettes of stress and/or strain meters placed in the concrete in key locations will provide data for comparison of actual and computed stresses and strains. Such locations are often areas of maximum beam or arch action or areas of main cantilever or torsion stresses.

At Warragamba Dam a total of 164 strain meters in groups of 10 to 12 were cast into the dam wall. These were Carlson meters of the oil sealed electrical resistance type (11). Some stress meters were installed in pairs close to several strain meters to enable checks to be made. Resolution of readings was difficult. After extremely divergent readings were discarded some correlation of results was achieved during the first ten years but generally results were disappointing. After this period progressive failure of many of the gauges made further readings useless.

Experience at Warragamba has shown that the whole process of installation reading and interpretation of results can be extremely complex and must be undertaken only by very skilled personnel.

Uplift pressures are read regularly and compared with design assumptions. Due to the interdependence of uplift pressure and drainage, uplift pressures have been discussed above in 2.2.

2.4 Chemical Analyses of Drainage Waters

Chemical analyses of drainage waters indicate whether materials are being washed or leached out of the foundations or concrete and whether this constitutes significant short or long term deterioration of foundation or dam.

In some older dams constructed of poor concrete and ponding waters with a high capability to dissolve lime, leaching of lime will lead to disintegration of the concrete. It is important to widen the investigations to give some idea of the extent of the affected areas and the amount of lime being removed to ascertain whether remedial action is required or alternatively is not required within the foreseeable life of the dam. Such expansion of the investigation programme could include chemical analyses of cores extracted from the dam.

Washing out of clay seams or faulted zones in the foundation could lead to collapse of foundations and failure of the dam if this phenomenon is not detected.

In cases where significant removal of material is occurring a grouting programme of either cement or chemical grouts or both may be required. Such a solution, however, must be approached with extreme caution to ensure that the stability of the dam is not jeopardised during the period of foundation treatment.

2.5 Seismic Activity

In an area where the crust of the earth is loaded with a large dam and the lake behind it the consequential build up of stress is generally concentrated around structural weaknesses such as faults, joints and fold axes. These stress concentrations can either:-

(a) cause tremors and therefore relieve the stress or,

(b) be relieved by tremors originating elsewhere.

In addition the presence of water lubricates and weakens the joints and movement can be initiated in this way to relieve the stress.

Normally when the lake fills especially for the first time, there is a sharp increase in the frequency of earth tremors in the area. A plot of epicentres in the area is extremely valuable to co-ordinate with the geological knowledge of the area and give an indication as to the faults or weaknesses that are liable to "move" during future earthquakes. An assessment can then be made as to whether this constitutes a danger to the dam or lake.

2.6 Measurement and Recording of Cracks

Most gravity dams contain a large number of cracks usually of minor extent. Cracks can range from minor dry surface shrinkage cracks to larger cracks with water flowing from them. Generally cracks that are random and small are not significant while larger cracks and cracks that leak water and reflect the stress pattern of the structure are significant.

It is desirable to know whether cracks considered significant are extending or widening. At regular intervals crack lengths can be recorded and movement checked by either a mortar pad or a direct measurement across the crack at selected points.

Cracks may be identified from inspections of boreholes and cores in addition to surface inspections. One of the most alarming cases quoted in recent years must be that of Bhandardara Dam constructed between 1910 and 1926 in India (4). The dam is a 270-ft. high mass gravity structure.

Immediately after the 1969 monsoon, excessive seepage and cracks were observed at the toe of the dam. Subsequent drilling showed a crack extensively developed as shown in Figure 2. This crack appears to follow partly the type of failure envisaged in the crack propogation analysis of the U.S.B.R. (6) and discussed by Leliavski (7) and partly the stage construction of the dam (5). It is difficult to imagine a dam closer to failure and the margin between the dam standing or falling was probably only inches of water height at the previous peak flood level. The dam was post-tensioned in 1970.

3. M.W.S. & D.B'S PROCEDURE FOR REPORTING AND ASSESSING DATA RECORDED

At each of the Board's dams is a Resident Officer responsible for a large amount of the day to day operation of the dam and one who provides a continuity of detailed knowledge in carrying out those sections of the monitoring entrusted to him.

3.1 Leakage and Uplift

Leakage and uplift for each dam is read monthly. The readings are processed, recorded and results scanned for deviant values or deviant trends etc. A continuous plot of uplift co-efficients and leakages is kept for each dam. A simplified version of this plot is shown in Figure 3. Results are compiled into a quarterly report for each dam and are submitted to Deputy Engineer-in-Chief level for review.

3.2 Precise Surveys

Precise surveys are carried out every six months on Warragamba Dam and annually on all other major dams and the results comprehensively reported and compared with previous surveys. An effort is made to seek explanations for all types of movement e.g. responses to seasonal effects and changes in water level, and to evaluate any danger and need for remedial action.

These reports are submitted to the Engineer-in-Chief after referral to all Branch Heads involved.

3.3 Annual Inspections

The annual inspection of dams is the responsibility of the Resident Engineer Headworks, the senior site engineer responsible for the operation of all the Board's dams. His report is submitted through Branch Heads involved to the Engineer-in-Chief. At any stage these officers may request further investigation of any aspect of the report.

Reports of annual inspections are presented partly as a completed standard sheet (Figure 4) and partly as a written report.

Annual inspections are intended to cover more than simply a physical inspection of the dam as the reports act as the main source of initiative leading to such things as re-appraisal of the design of the dam or spillway capacity should such things be considered necessary.

4. CASE HISTORIES

The following examples are cases in the Board's experience where recorded data has demanded attention and an outline will be given of how remedial action was taken.

4.1 Nepean Dam

Nepean Dam is a curved mass gravity wall of cyclopean masonry. Its maximum height is 247-ft. and it was completed in 1935.

As part of a general re-appraisal of Nepean Dam in February 1965 a study was made of the uplift under the dam. Although the dam was not considered unsafe it was considered prudent to further increase its factor of safety by reducing the existing high uplift profile.

A typical profile of recent uplift co-efficients under Nepean Dam was plotted and compared with a corresponding profile recorded before 1942. The resultant profiles are shown on Figure 5. At first sight it appeared that considerable deterioration had taken place in the drainage system resulting in large increases in uplift.

When studied in greater detail it was found that the variation in uplift co-efficients had occurred primarily during the period 1936 to 1942.

(Uplift co-efficient = uplift pressure head on foundation divided by head between foundation and reservoir level.)

During this period very great variations in water level ponded behind the dam had occurred but since 1942 the stored water level has for the most part remained close to Full Supply Level and uplift co-efficients have stabilised.

It is probable that the increase in uplift co-efficients was due more to a prolonged initial adjustment of foundations than to any significant deterioration of the drainage system.

Since leakage flows were small and there was no evidence of removal of foundation material it was decided that additional drainage but no grouting would be provided. Reaming of existing drain holes was not possible since these were in the form of vertical pipes with horizontal connectors to the gallery.

To date additional drainage holes drilled have given some reduction of uplift and further drainage holes drilled from the lower gallery will be provided as required at sufficiently close spacing to reduce the uplift profile to that assumed in the design, or lower.

4.2 Avon Dam

Prior to strengthening Avon Dam was a curved mass gravity dam constructed of cyclopean masonry. Its maximum height is 235-ft. and it was completed in 1927.

When Avon Dam filled for the first time in 1929 considerable leaks developed in the eastern abutment. These were plugged by dropping bags of wet sawdust into the water each day until the leakage subsided. In 1930 a quantity of clayey material was extruded from a foundation drain. Extensive amounts of lime were leached from the concrete and deposited in galleries in the form of calcium carbonate. Relative movement between two monoliths was clearly shown by shearing of calcium carbonate deposits and then crystallisation as calcite across the joint. The crystallisation was in the form of oblique parallel crystals spanning the joint. Extensive cracking and some spalling had occurred on the down-stream face. Records showed that both uplift and foundation leakage had increased.

A full scale re-appraisal of the dam was undertaken (8) which included:-

- examination of records including uplift and leakage
- structural analysis of the dam
- re-assessment of the design flood
- detailed study of deterioration of concrete strength due to lime leaching
- chemical analysis of drainage water
- geological survey
- fluorescein dye tests to establish leakage paths
- exploratory diamond drilling and core testing.

These studies led to the expenditure of nearly $4M on the following remedial work (9):-

- provision of new foundation grout curtain

- provision of new foundation drainage curtain
- strengthening of the dam by placing rock fill against the downstream face
- reconstruction of original spillway to give greater capacity

4.2.1 Avon Dam Earth Pressure Cells

An aspect of the strengthening of Avon Dam that was of particular interest was the monitoring of the earth pressures exerted by the heavily compacted sloping earth fill on the downstream face of the existing dam. Details of design loadings, earth pressure cells and initial readings are described in (9).

In the initial designs values of earth pressure coefficients ranging from 0.2 (active) to 0.82 (at rest) were considered. Although not designed for, it was assumed that passive values of earth pressure coefficient would be higher than these values. It was considered that passive earth pressures would not develop over the range of deflections anticipated from the concrete wall. The earth pressure cell readings showed clearly when arching in the fill developed and that a K factor of the order of 0.85 appeared to best fit the readings obtained.

In discussion on this point (10) Professor Lee has pointed out that in a lower bound solution for an associated flow rule material a K factor of the order of 0.8 would correspond to the passive earth pressure coefficient for an embankment fill of this shape. Thus the monitored results together with the discussion that followed the publishing of the paper on Avon Dam gave further confidence that the embankment was adequately loading the concrete dam for stability purposes.

The monitored results of the earth pressure cells are shown plotted in Figure 6. Since completion of the embankment there appears to have been little movement of the fill. A number of gauges appear to have failed and now give zero readings although there is always the possibility that these gauges have become unweighted due to local movements of the fill.

4.3 Warragamba Dam - Leakage into Asphalt Well Drains

Warragamba Dam is a straight gravity dam with a central overfall spillway and an overall height of about 450-ft. completed in 1960 (11).

In September 1963 a sudden large increase of leakage into some of the asphalt well drains was reported. The location of these drains is shown in Figure 7. Flows from individual drains increased from zero to a maximum of 1029 gallons per hour. Investigation with a closed circuit T.V. camera lowered down the drainhole showed the leakage to be coming from a horizontal crack at approximately R.L. 175.

A review of stress analyses showed theoretical compressive stresses at that elevation on the upstream face and it was evident that stress calculations could not show the cause of the cracking.

However, studies of foundation deflection showed that settlement had occurred over the spillway section while the abutment sections showed smaller settlement. Figure 7. also shows a pronounced change in the foundation slope at R.L. 175 on both sides of the gorge which could cause arching of the upper part of the dam tending to cause separation of the upper and lower parts of the dam when considered in relation to the rock movement mentioned above.

Another cause could have been the fact that a three months' delay in pouring concrete occurred in a number of monoliths at this level. It is possible that the lapse of time may have prevented an effective bond forming between successive concrete pours.

Although consideration was given to grouting the cracks it was concluded that this should not be attempted until the crack had reached maximum opening and it could be ensured that grouting would not propogate the crack.

It was finally decided that action would be deferred and the leakages read and forwarded fortnightly to Design Branch. Possible remedial action was to grout the drains and then re-drill them.

Over the next 18 months probably due to further foundation adjustments the crack gradually closed, leakage became very small, and the need for remedial action lapsed. Monthly reading and reporting of flows from each of the asphalt well drains is now a permanent part of the monitoring of the dam.

4.4 Warragamba Dam - Crack in Downstream Wall of Hydro-Electric Power Station (H.E.P.S.)

In April 1966 a diagonal crack (0.01" average width) was reported extending for some 50-ft. in the downstream wall of the H.E.P.S. as shown in Figure 8. The cracked section of wall is 10-ft. thick and heavily reinforced with $1\frac{1}{4}$" or $1\frac{1}{2}$" diameter bars at 9" or 12" centres both ways on both faces. Checking found this reinforcement adequate for the normal loads that might be expected on the wall.

It was considered that the causes of the cracking are probably related to the dam and foundation movements that had taken place prior to April 1966. Investigation of these movements suggested three effects which may have given rise to the cracking.

4.4.1 Effect 1. Gorge Contraction

Recorded measurements of dam movements had shown that the Warragamba gorge was contracting in width. This could have been due to swelling of rocks due to saturation in the upper parts of the gorge and/or a regional compressive state of stress in the rocks.

A regional compression gives rise to a stress picture as shown in Figure 8 with the crack following the line of maximum shear.

4.4.2 Effect 2. Foundation Movements

Consideration of measured upward and downward foundation movements in the vicinity of the dam also indicated that the crack may have been due to shear effects. Contours of foundation movements and the local effect at the downstream wall of the H.E.P.S. are shown in Figure 9.

4.4.3 Effect 3. The Three Dimensional Effect - Cracking Occurs Inside the Wall and Not Outside

The strutting effect of the dam wall across the gorge would tend to modify the regional stress picture in the vicinity of the dam itself. Instead of acting directly across the gorge in a S.E.-N.W. direction the principal compressive stress in the region of the H.E.P.S. would act in a more E - W direction as shown in Figure 8. This would tend to open up a crack on the inside of the wall and not on the outside.

4.4.4 Action Taken

Since the cracks have occurred only on the inside wall of the H.E.P.S. deterioration due to corrosion of steel is not expected to occur. In its present state the crack is not considered to be serious but should it continue to open then remedial measures such as prestressing the wall etc. may have to be considered.

Eight measuring points were set out across the crack and these were monitored monthly for a period of two years. During this period the crack remained stable and no further action was taken other than to include its inspection in the annual report.

5. Conclusions

5.1 Although a concrete gravity dam is generally considered to be a static structure composed of and built on stable materials it nevertheless can deteriorate and its foundations can deteriorate. It moves and there are a number of physical and chemical properties that should be regularly measured and assessed to ensure its continued stability.

5.2 The proper reporting of monitored data of existing dams and its reference to senior and experienced engineers is probably the most important safeguard that can be implemented to ensure that dam failures will not occur. The Board's system of reporting leakage and uplift, precise survey and annual inspections is comprehensive and to date has been successful in pointing to the need for remedial works as they occur.

5.3 History has shown that despite the ever increasing advances in technology, dam failures continue to occur and loss of life and damage to property appear to be increasing. In addition since bigger dams are now being built and it sometimes becomes necessary to develop poorer sites than have been chosen in the past, proper monitoring of dams becomes increasingly important.

5.4 As the case histories described above have shown some problems do not yield obvious solutions. Courses of remedial action based on insufficient data could be detrimental and it is often necessary to defer action while more data is assembled. Considerable judgement and experience is required in these circumstances.

5.5 Perhaps a lesson to be learned from the disasters of the Malpasset and Vaiont dams, which were being monitored, is that with their great size dams tend to mesmerise us into believing them invulnerable. Properly monitored and appraised data are the sensitive indicators of a dam's true condition rather than its external appearance.

ACKNOWLEDGEMENTS

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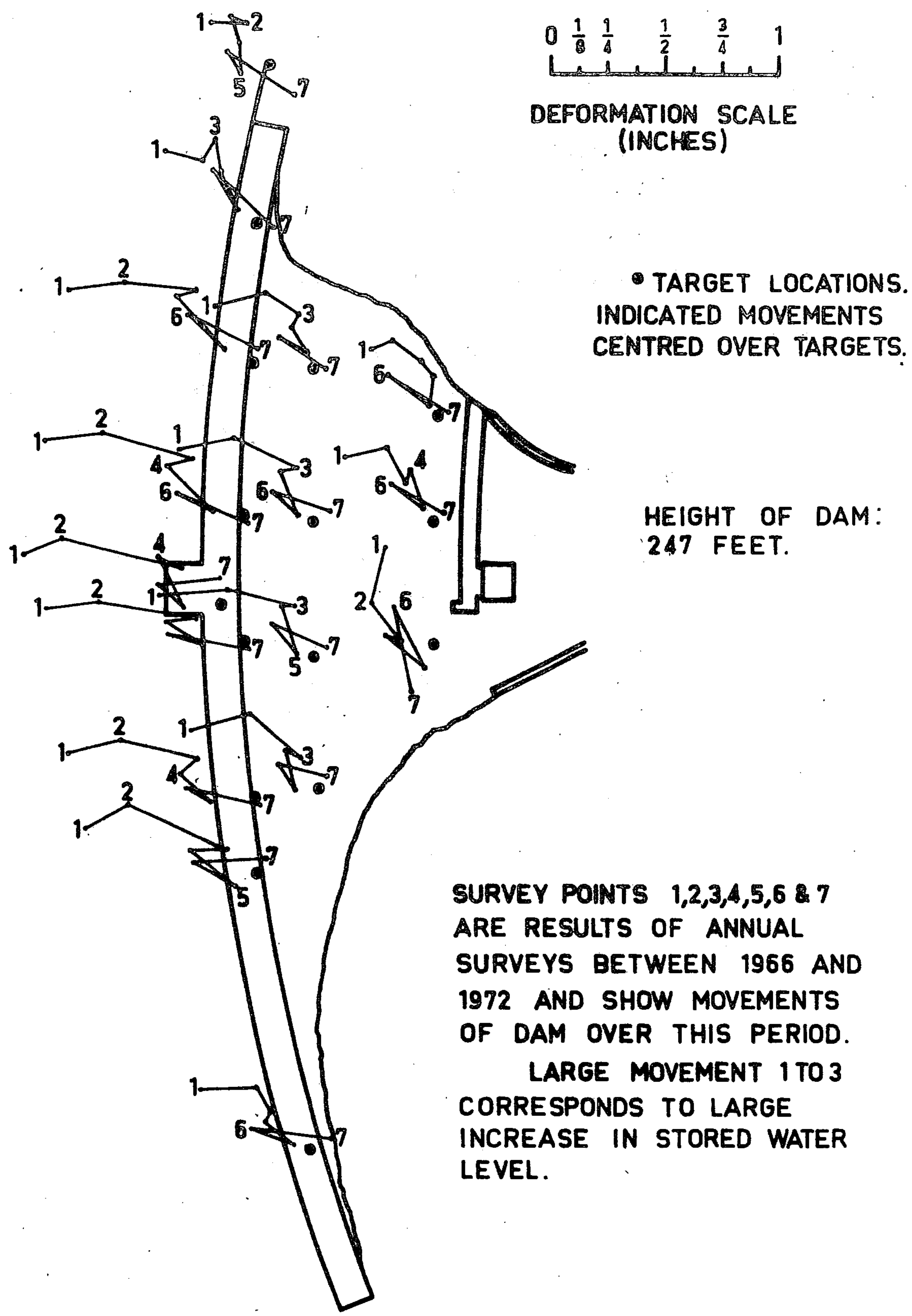
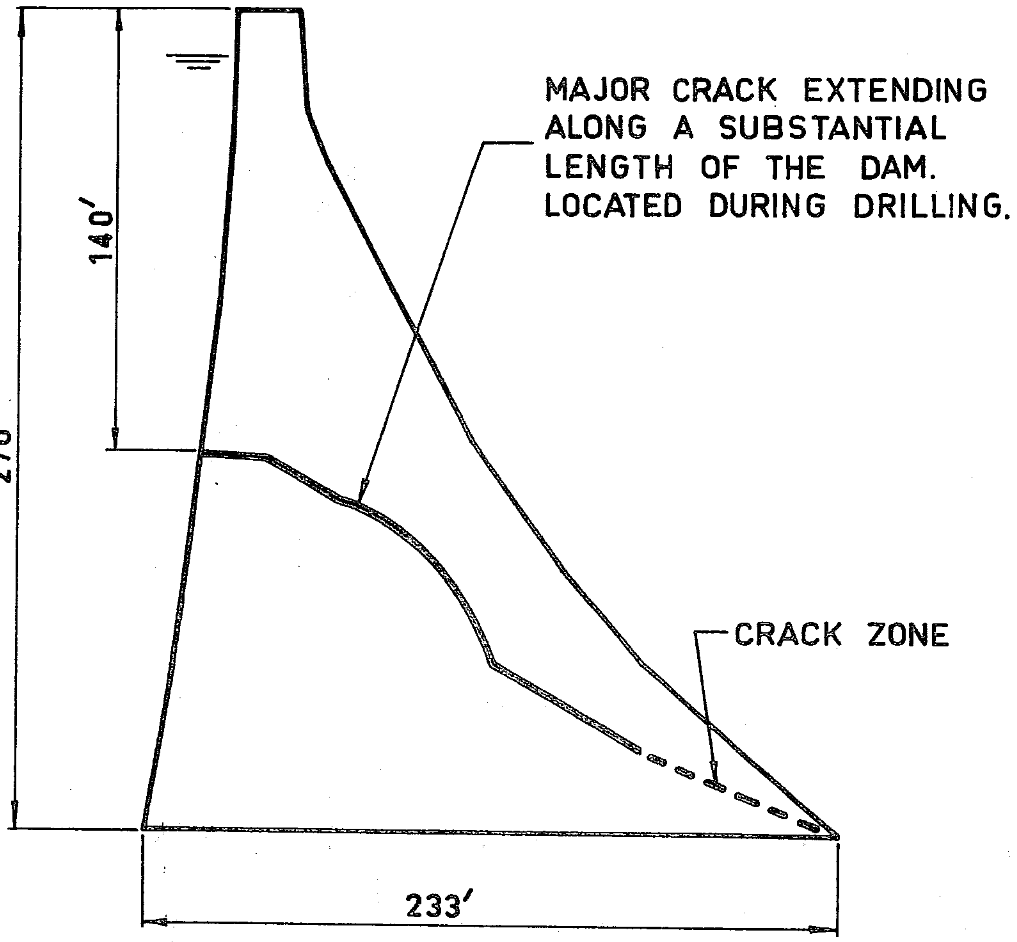


HORIZ. DEFORMATIONS: NEPEAN DAM

FIGURE 1



BHANDARDARA DAM-MAIN CRACK

FIGURE 2

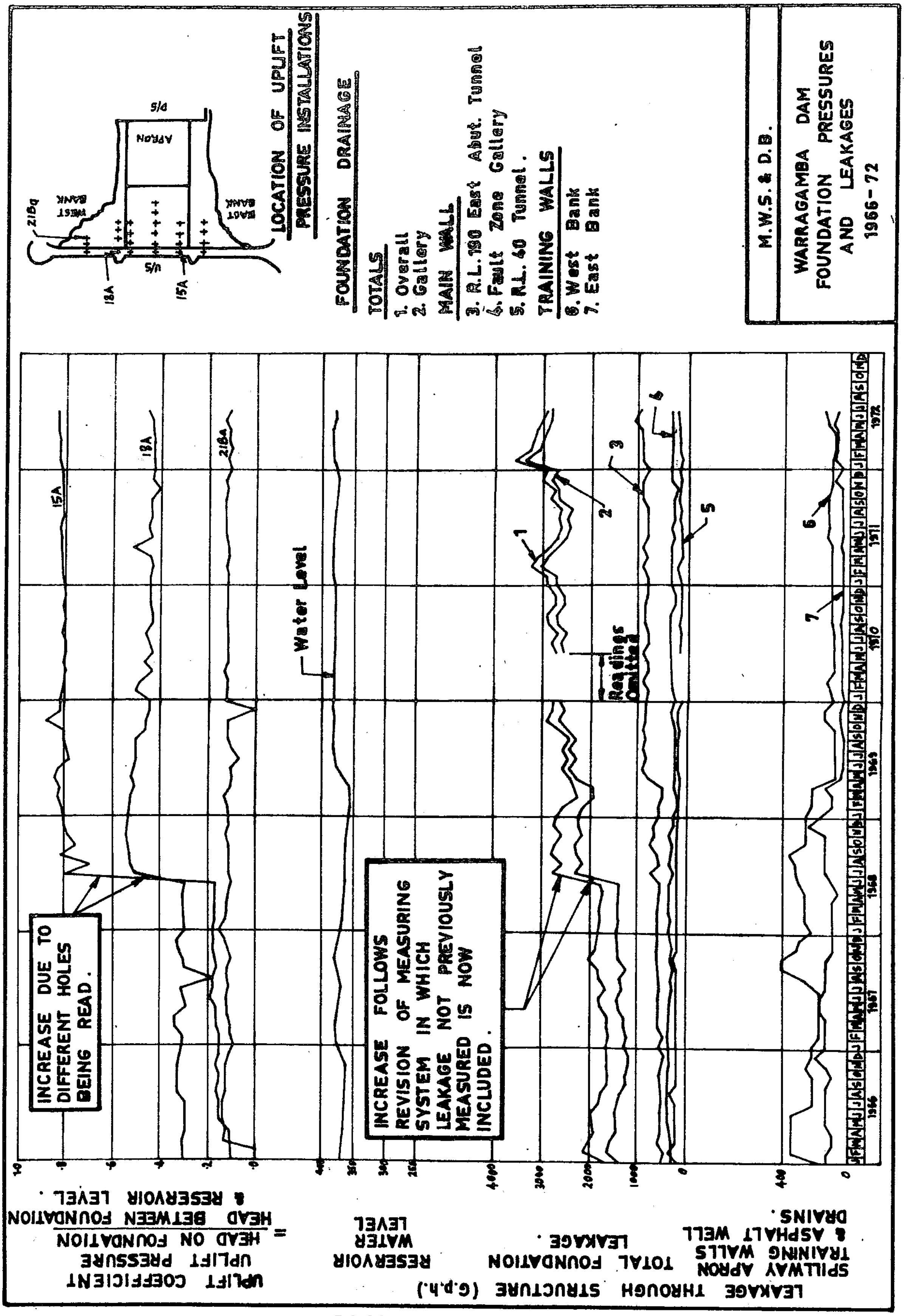

M.W.S. & D.B.
WARRAGAMBA DAM
FOUNDATION PRESSURES
AND LEAKAGES
1966 - 72
LOCATION OF UPLIFT
PRESSURE INSTALLATIONS
FOUNDATION DRAINAGE
TOTALS
1. Overall
2. Gallery
MAIN WALL
3. R.L. 190 East Abut. Tunnel
4. Fault Zone Gallery
5. R.L. 40 Tunnel.
TRAINING WALLS
6. West Bank
7. East Bank
INCREASE DUE TO DIFFERENT HOLES BEING READ.
INCREASE FOLLOWS REVISION OF MEASURING SYSTEM IN WHICH LEAKAGE NOT PREVIOUSLY MEASURED IS NOW INCLUDED.
Water Level
Readings Omitted
15A
18A
21BA
UPLIFT COEFFICIENT
UPLIFT PRESSURE = HEAD ON FOUNDATION / HEAD BETWEEN FOUNDATION & RESERVOIR LEVEL.
RESERVOIR WATER LEVEL
TOTAL FOUNDATION LEAKAGE.
SPILLWAY APRON TRAINING WALLS & ASPHALT WELL DRAINS.
LEAKAGE THROUGH STRUCTURE (G.p.h.)
1966
1967
1968
1969
1970
1971
1972

FIGURE 3

INSPECTION OF GRAVITY MASONRY DAMS

NAME OF DAM

1. STABILITY

(1) Quality of Foundations.
(2) Effect of water on nature of foundations. ... ...
(3) Condition of Upstream face.
(4) Condition of concrete in downstream face. ... ...
(5) Deposits on Downstream face -

(i) Nature
(ii) Extent ... ...

(6) Cracks on Downstream face -

(i) Nature
(ii) Extent ... ...

(7) Deposits in Inspection Gallery -

(i) Nature
(ii) Extent ... ...

(8) Cracks in Inspection Gallery -

(i) Nature
(ii) Extent ... ...

(9) Effect of any cracks on the stability of the Dam.
(10) Leakage through downstream face... ... ...
(11) Leakage through contraction joints in Inspection Gallery.
(12) Leakage through cracks in Inspection Gallery ...
(13) Flow through drains in Inspection Gallery -

(i) Foundation drains.
(ii) Drains from body of dam ... ...

(14) Erosion at toe of dam.

2. SEEPAGE

(1) Leakage round sides of dam (nature and quantity) -

(a) Through pores in foundation.
(b) Through seams in foundation ... ...
(c) Through fissures in foundation.

(2) Leakage under dam (nature and quantity) -

(a) Through pores in foundation.
(b) Through seams in foundation... ... ...
(c) Through fissures in foundation.

(3) Effect of leakage on foundations. ... ...

3. SPILLWAY

(1) General location relative to dam
(2) History of flooding. ... ...
(3) Condition of spillway.

(a) Invert.
(b) Walls. ... ...

(4) Effect of any erosion on dam.

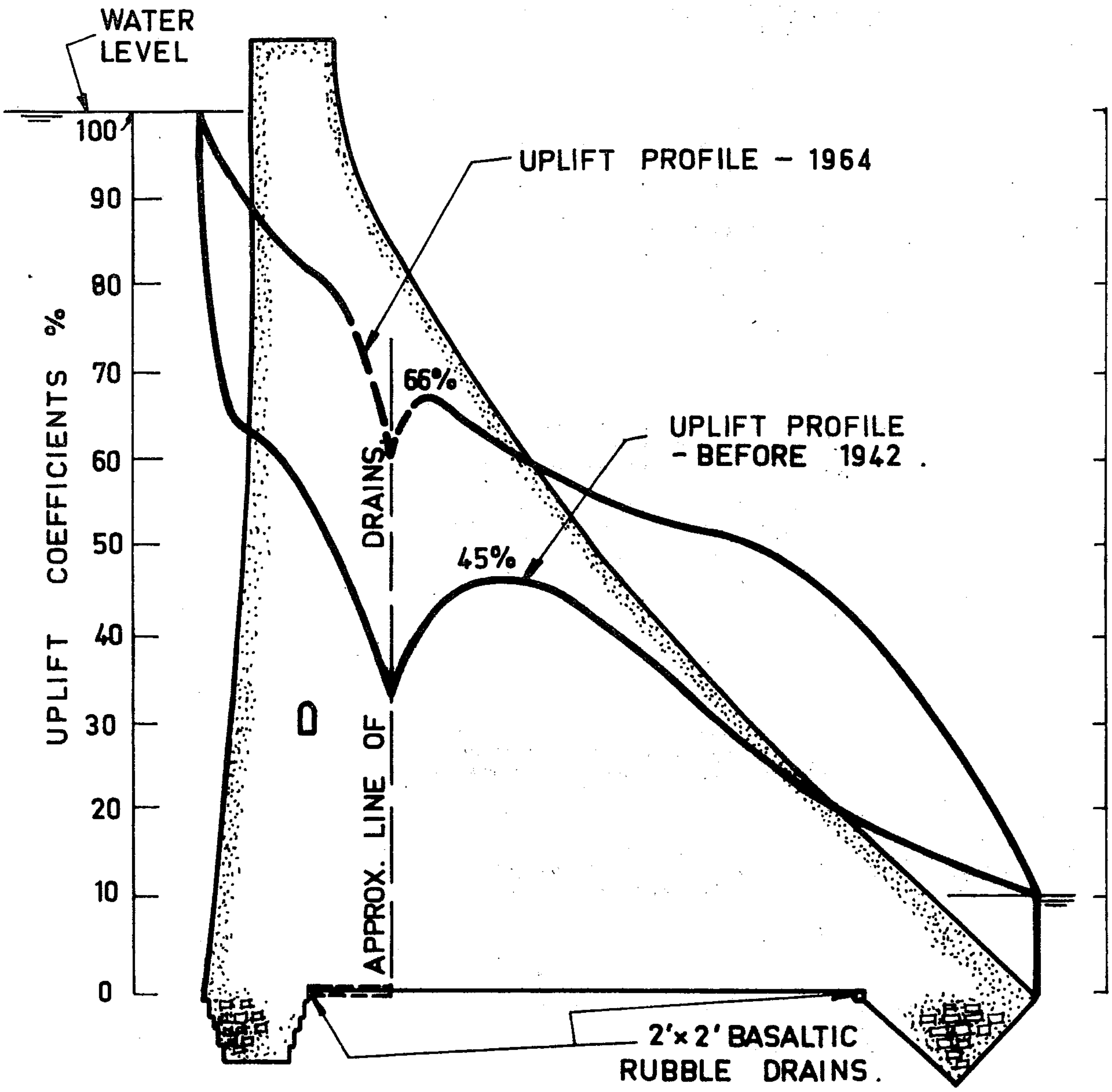
4. OUTLETS

(1) History ... ...
(2) Condition
(3) Adequacy ... ...

5. SCOUR PIPES

(1) History.
(2) Condition. ... ...
(3) Adequacy

FIGURE 4



NEPEAN DAM

FIGURE 5

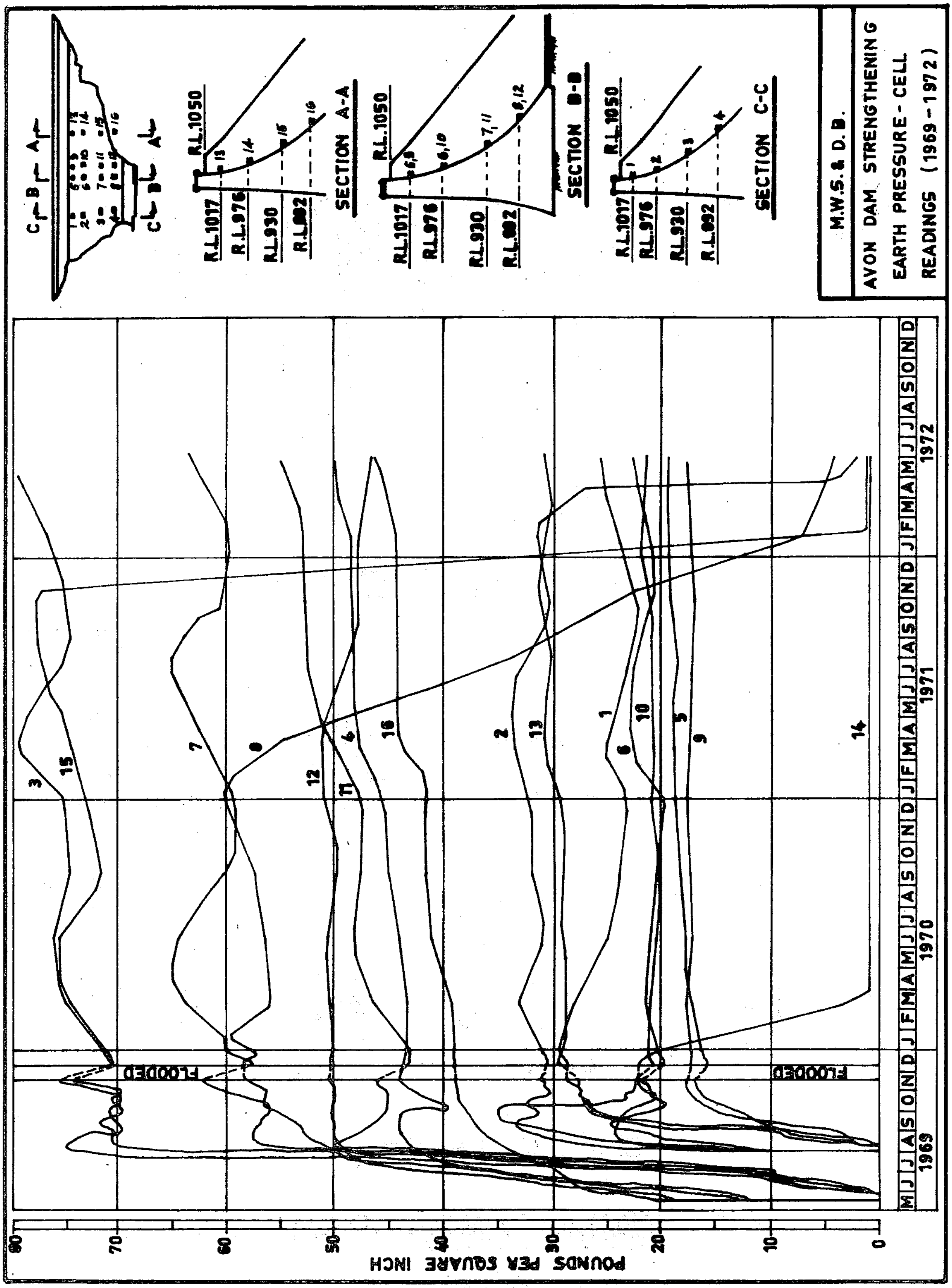
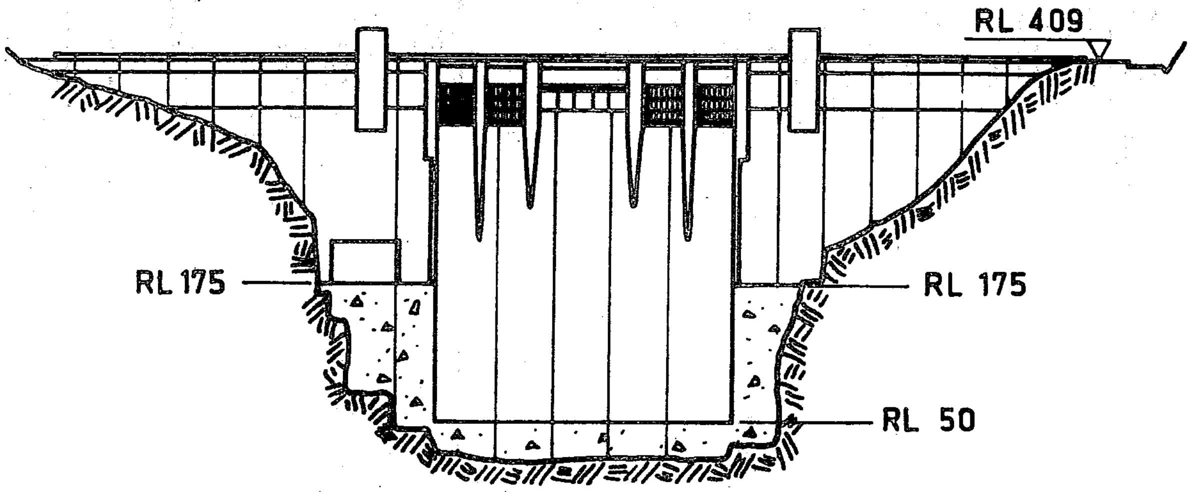
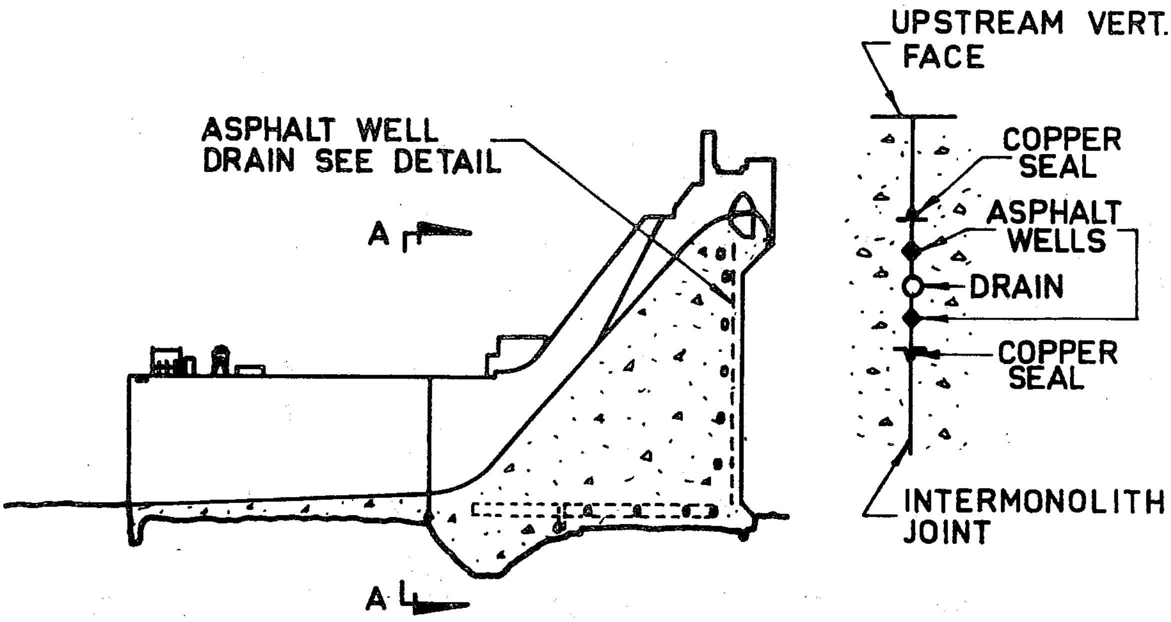

M.W.S. & D.B.
AVON DAM STRENGTHENING
EARTH PRESSURE - CELL
READINGS (1969 - 1972)
R.L.1050
R.L.1017
R.L.976
R.L.930
R.L.892
SECTION A-A
SECTION B-B
SECTION C-C
FLOODED
FLOODED
POUNDS PER SQUARE INCH
1969
1970
1971
1972

FIGURE 6



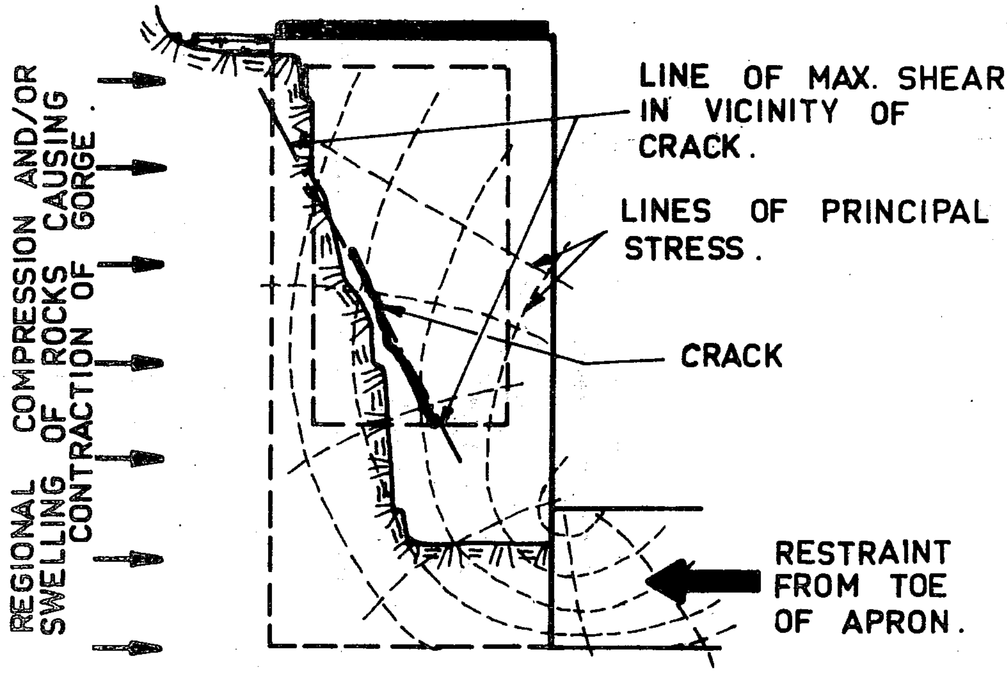
SECTIONAL ELEVATION A-A



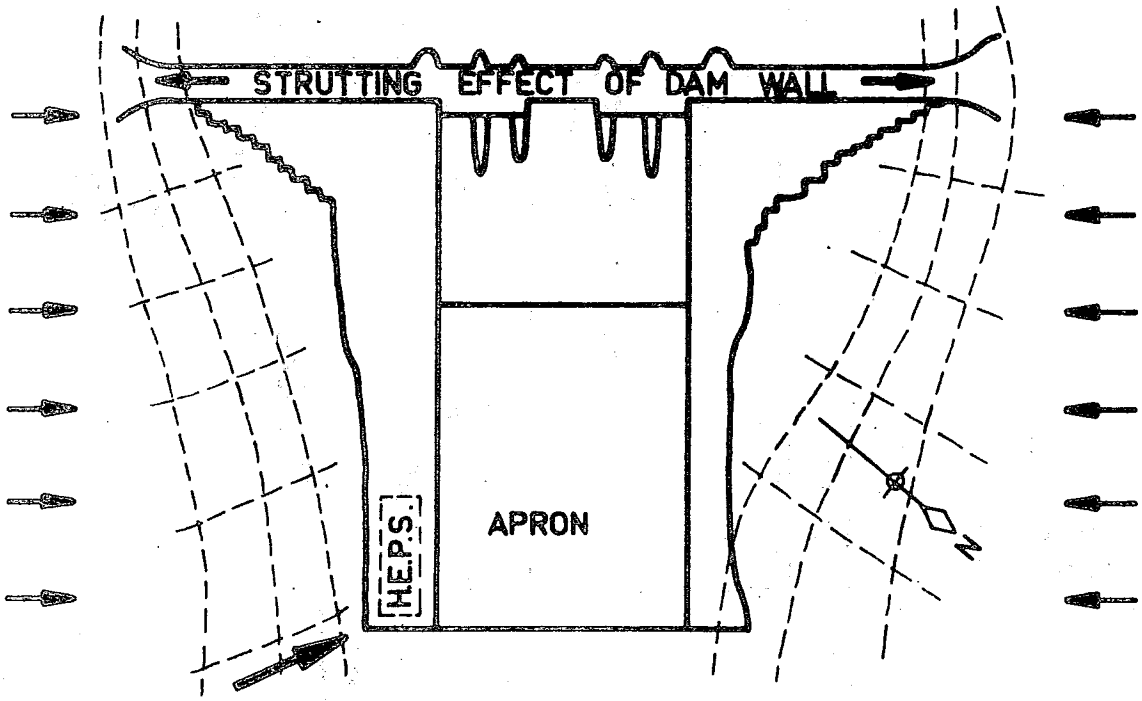
SPILLWAY SECTION

WARRAGAMBA DAM

FIGURE 7

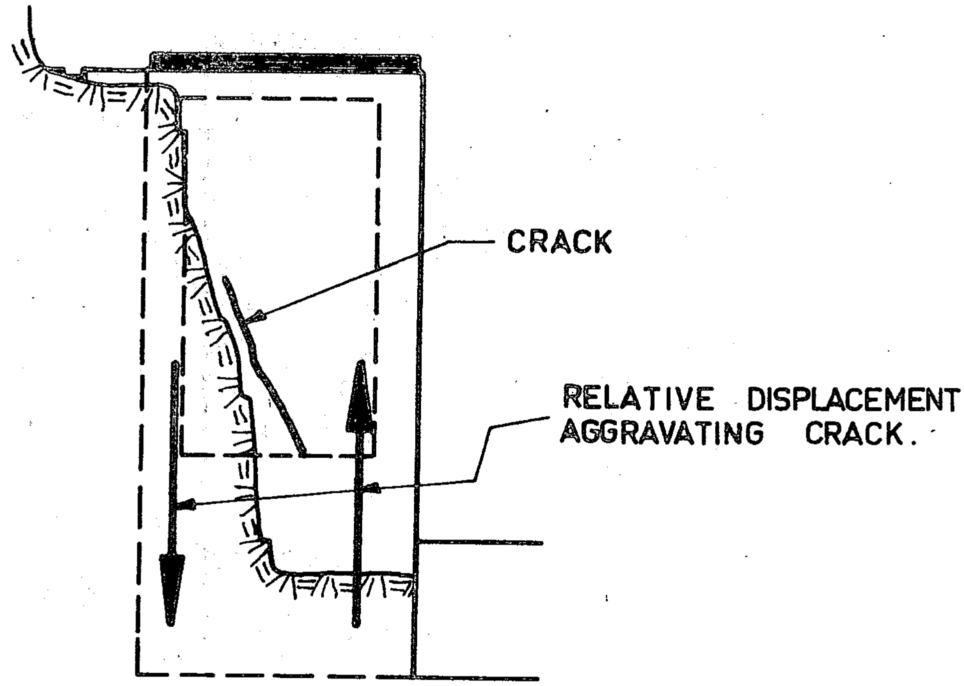


END ELEVATION OF EAST BANK TRAINING WALL – CRACK FOLLOWS LINE OF MAXIMUM SHEAR

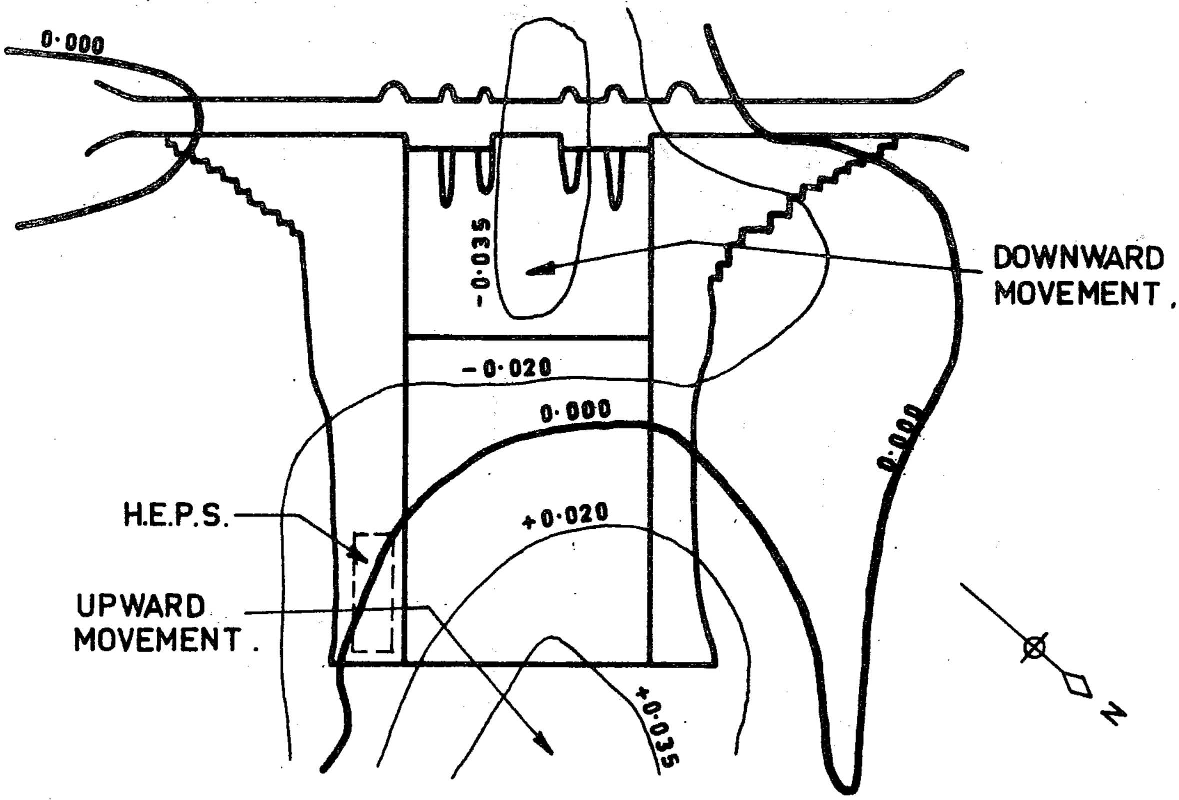


MODIFICATION OF DIRECTION OF REGIONAL COMPRESSION IN VICINITY OF DAM

FIGURE 8



FOUNDATION MOVEMENTS UNDER H.E.P.S.



CONTOURS OF VERT. FOUNDATION MOVEMENT

FIGURE 9