

SYMPOSIUM ON MONITORING OF DAMS

SYDNEY

11th October, 1972

Symposium organised by the Sydney Division of The Institution of Engineers, Australia, and the Sydney Branch of The Australasian Institute of Mining and Metallurgy at Eagle House, Lavender and Alfred Streets, Milsons Point.



DH03123

## ORGANISATION

This symposium has been organised by a committee consisting of members nominated from the Sydney Branch of the Australian Geomechanics Society.

The following members constituted the committee:-

Mr. F. J. Carter

Mr. C. G. C. Coulter

Mr. M. D. Fitzpatrick

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THE AUSTRALIAN GEOMECHANICS SOCIETY - SYDNEY GROUP

WEDNESDAY, OCTOBER 11, 1972

ONE DAY SYMPOSIUM

MONITORING OF DAMS

Auditorium, Eagle House,  
Cnr. Lavender & Alfred Streets,  
Milsons Point

	<u>Programme</u>	<u>Page No.</u>
9.00	Registration	
9.25	Opening (Professor I. K. Lee)	
	MORNING SESSION - CONCRETE DAMS	
9.30	"Concrete Gravity Dams"	
	Chairman: C. R. Longworth	
	Speaker: F. J. Carter .. .. .	1/1
10.30	Morning Tea	
11.00	"Arch Dams"	
	Chairman: J. R. Herington	
	Speaker: M. D. Fitzpatrick .. .. .	2/1
	AFTERNOON SESSION	
1.30	"Earth and Rock Dams"	
	Chairman: J. Wilkins	
	Speakers: C. G. C. Coulter .. .. .	3/1
	A. D. Hosking .. .. .	4/1
3.30	Afternoon Tea	
4.00	"Face Dams"	
	Chairman: M. G. Speedie	
	Speaker: M. D. Fitzpatrick .. .. .	5/1
4.45	Summary	
	Speaker: J. D. Hodgson	

MONITORING OF CONCRETE GRAVITY DAMS

F. J. Carter

Metropolitan Water, Sewerage & Drainage Board

MONITORING OF CONCRETE GRAVITY DAMS

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 downstream 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 ususally 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 propagation 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 downstream 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 propagate 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 1¼" or 1½" 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

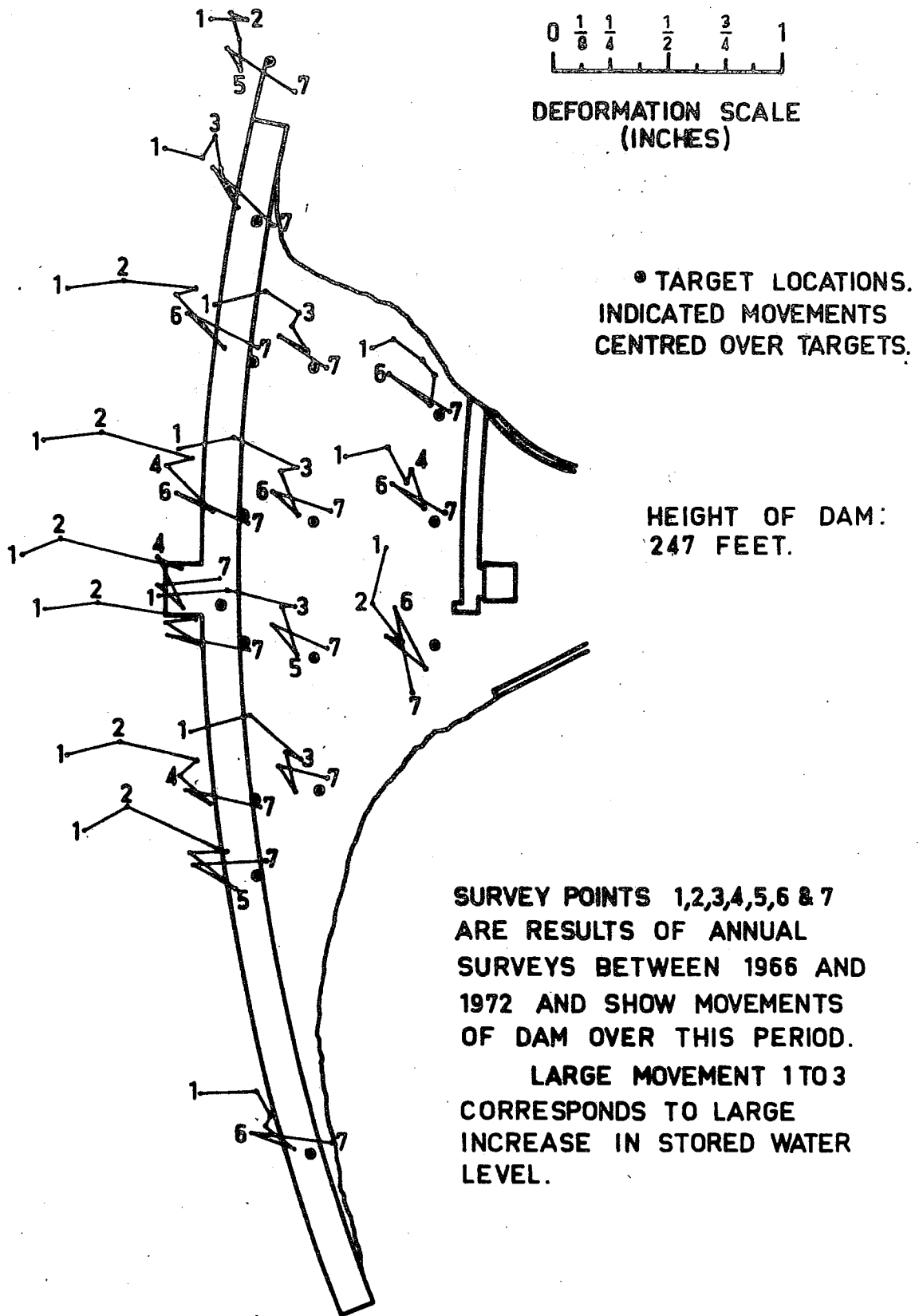
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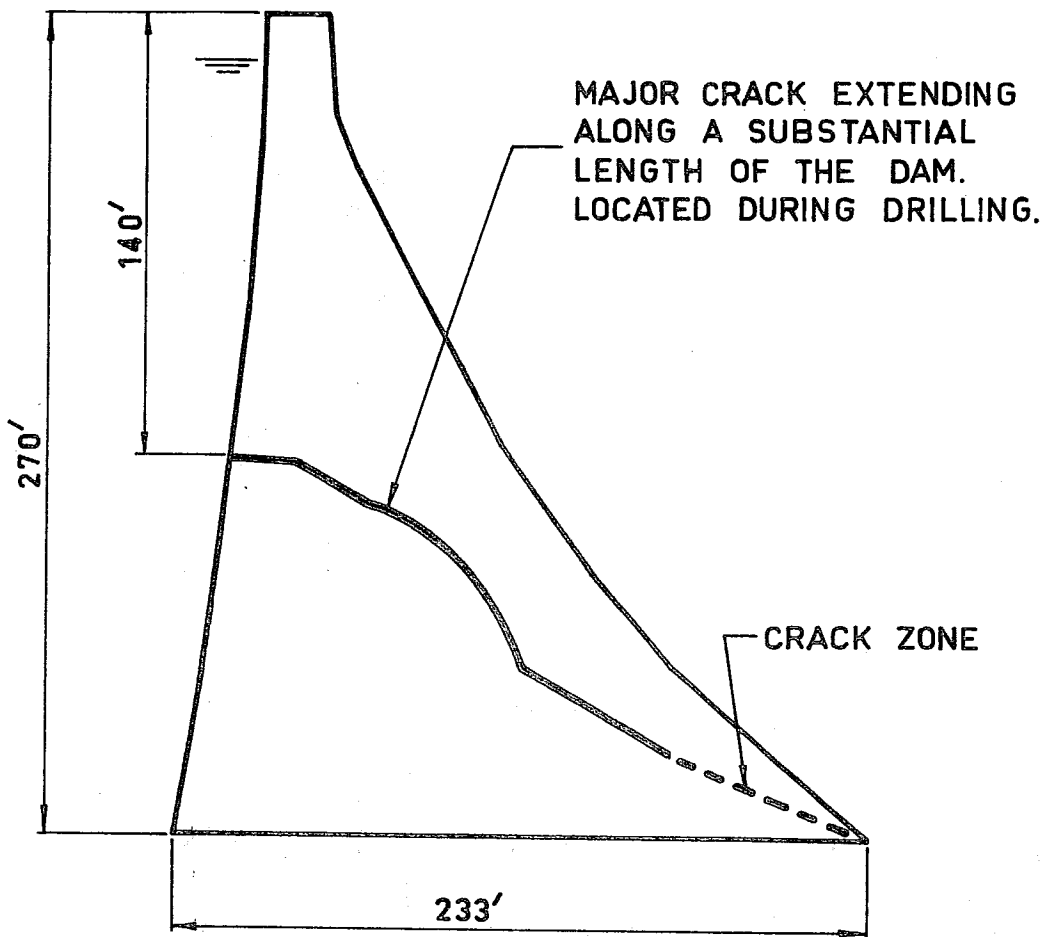
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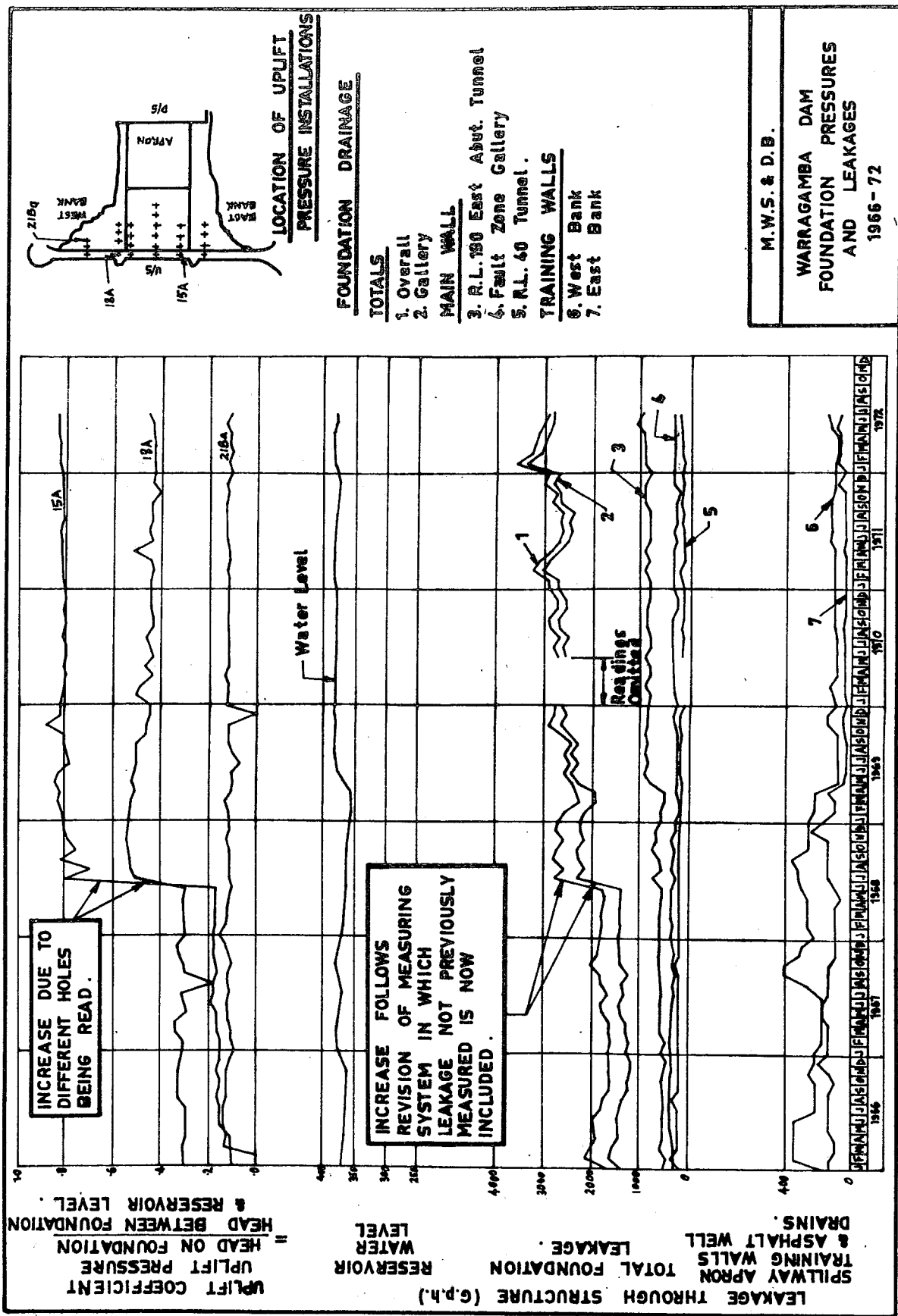
HORIZ. DEFORMATIONS: NEPEAN DAM

FIGURE 1



BHANDARDARA DAM - MAIN CRACK

FIGURE 2



**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.

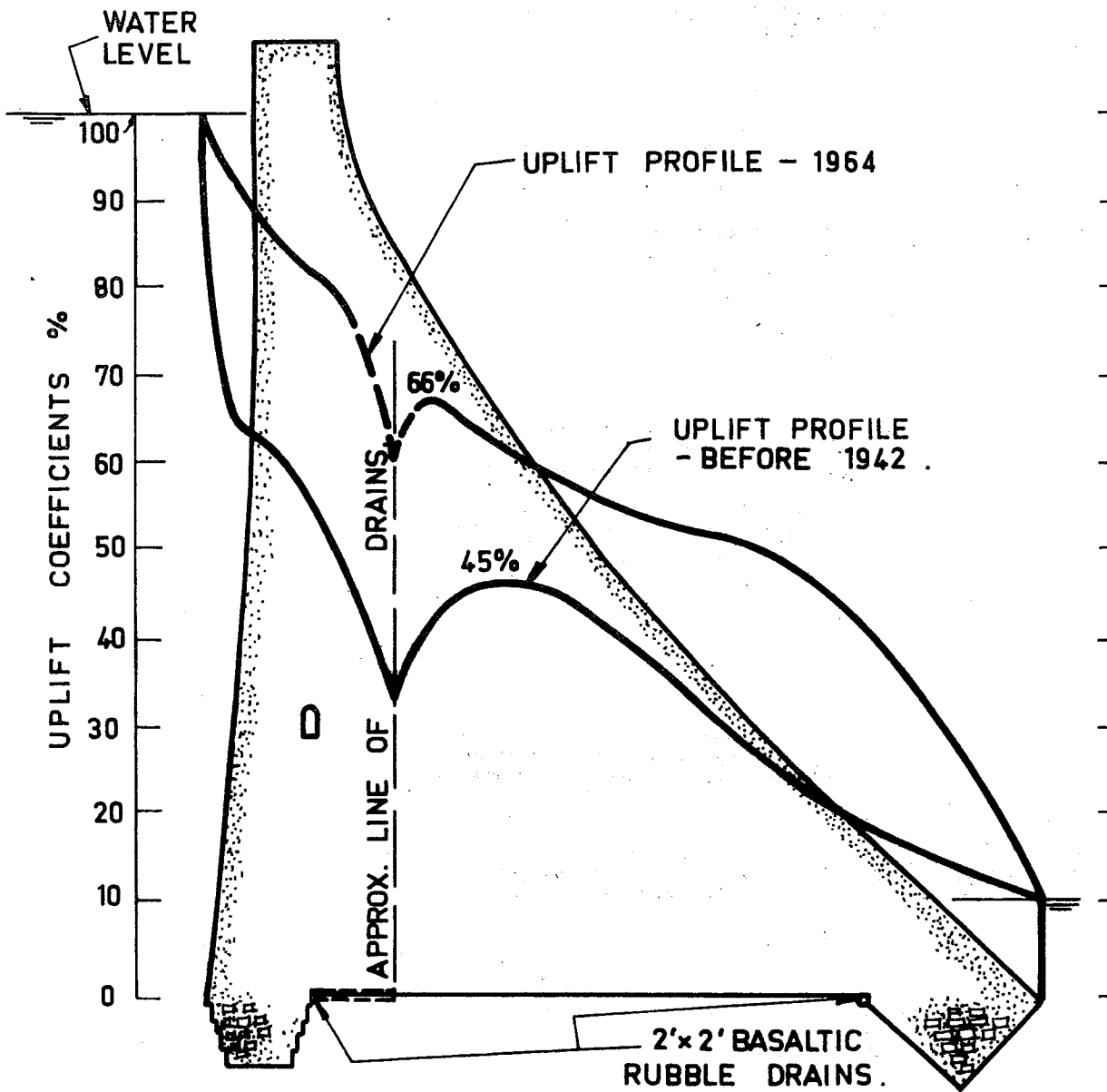
4. OUTLETS

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

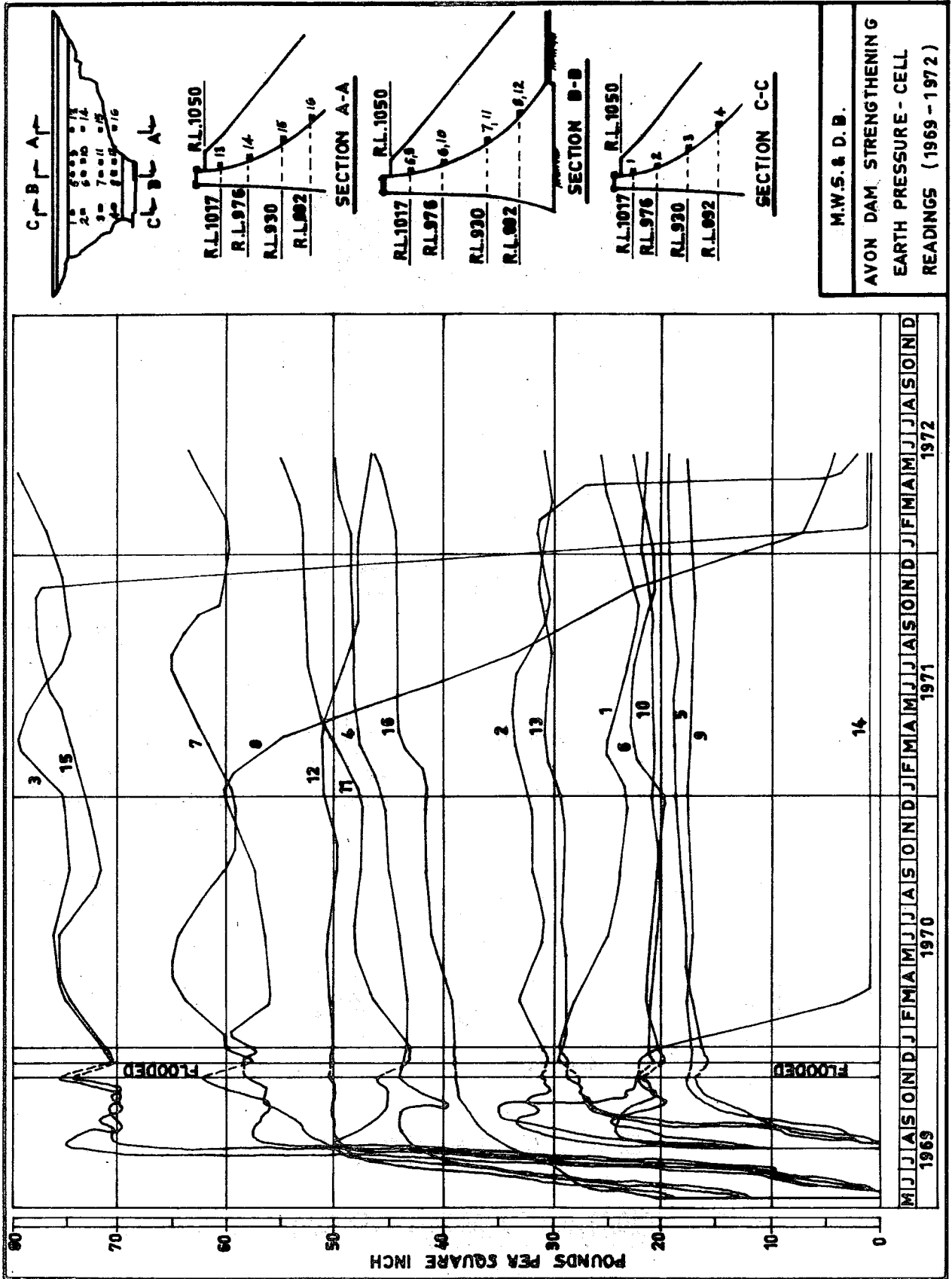
5. SCOUR PIPES

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

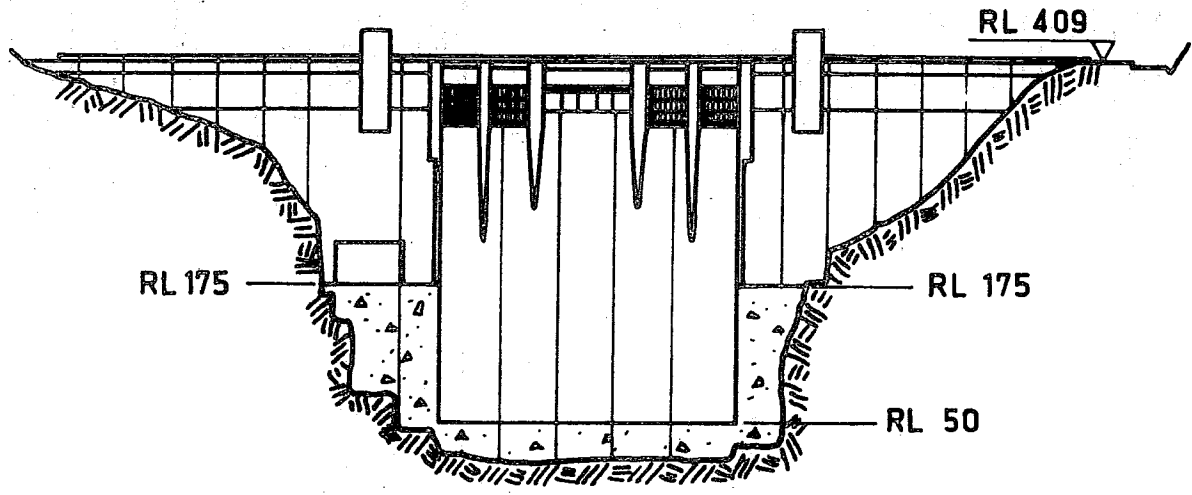
FIGURE 4



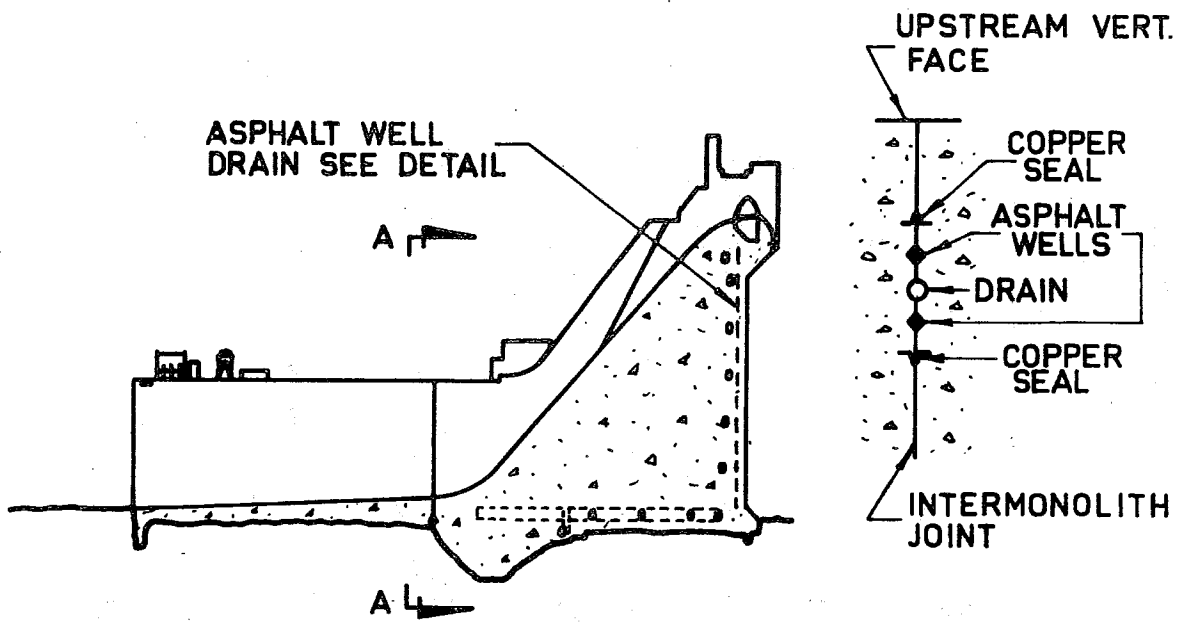
NEPEAN DAM  
FIGURE 5



M.W.S. & D.B.  
 AVON DAM STRENGTHENING  
 EARTH PRESSURE - CELL  
 READINGS (1969 - 1972)



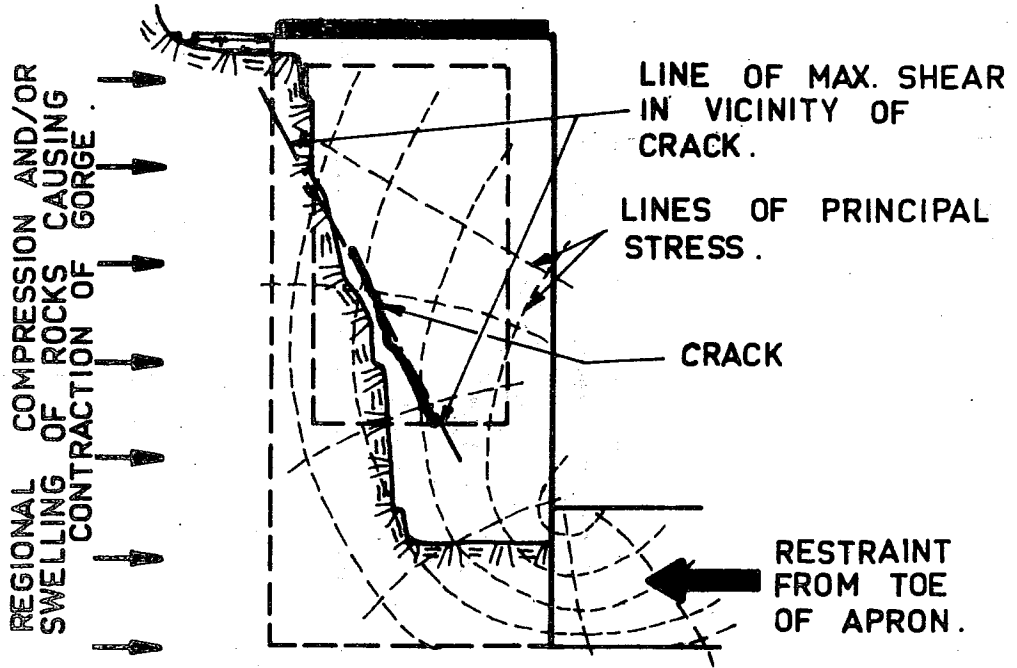
SECTIONAL ELEVATION A-A



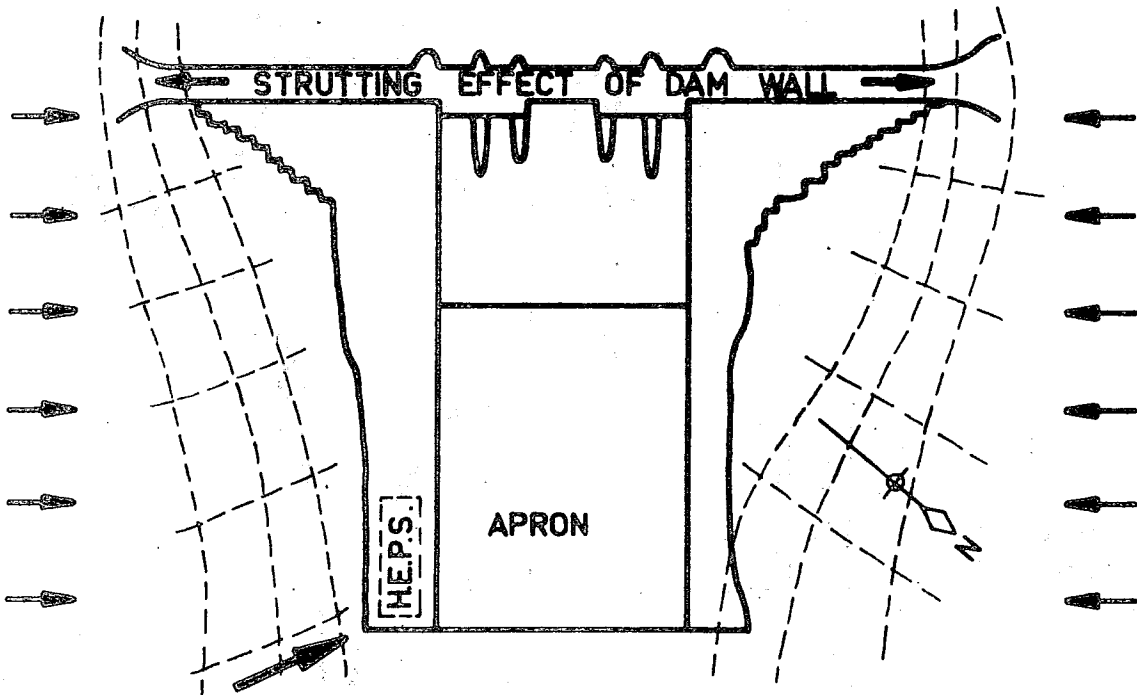
SPILLWAY SECTION

WARRAGAMBA DAM

FIGURE 7

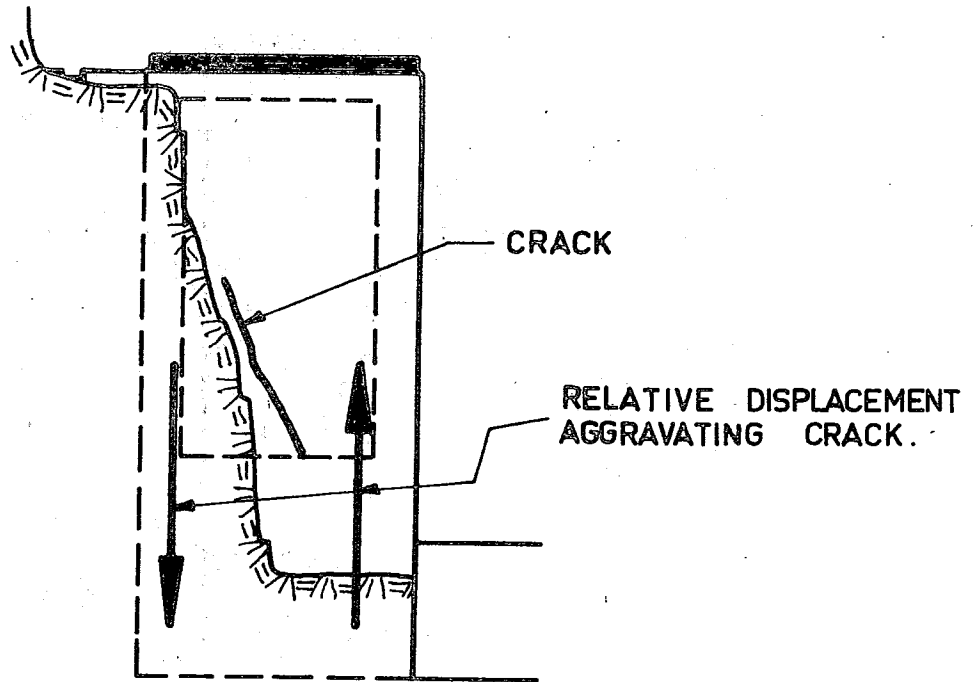


END ELEVATION OF EAST BANK TRAINING WALL - CRACK FOLLOWS LINE OF MAXIMUM SHEAR

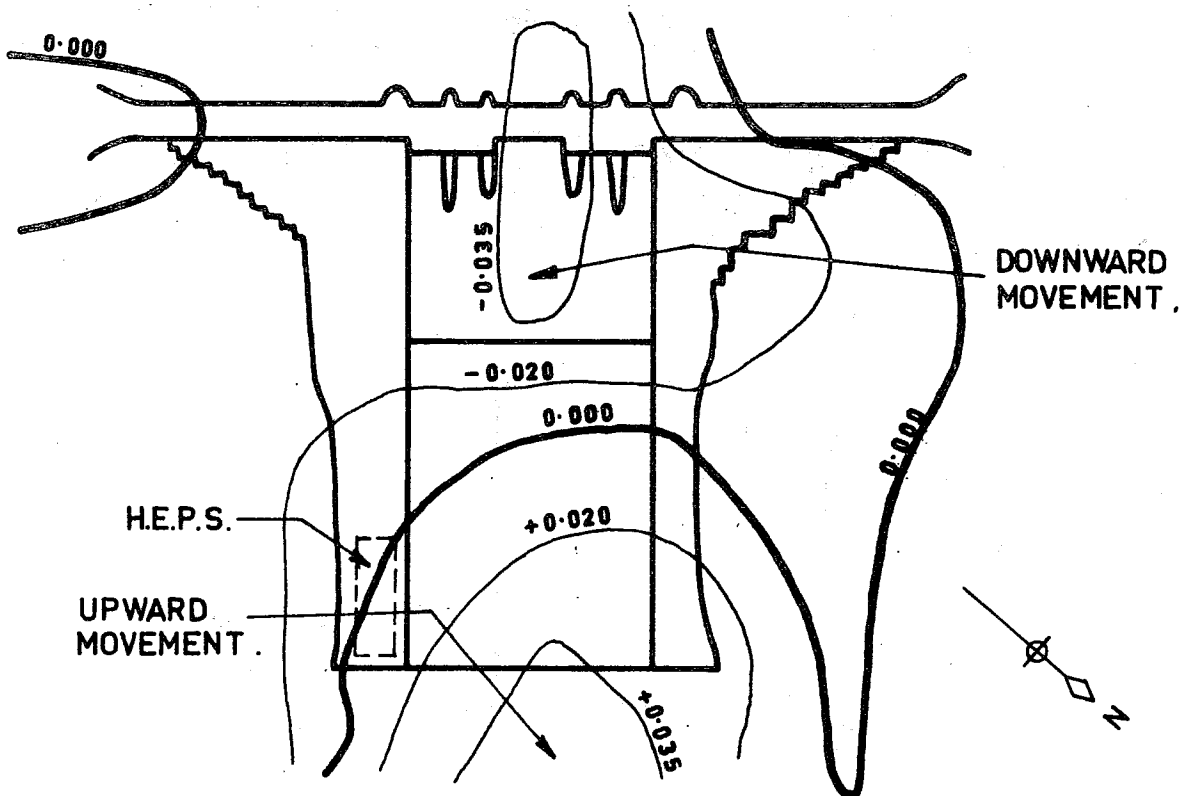


MODIFICATION OF DIRECTION OF REGIONAL COMPRESSION IN VICINITY OF DAM

FIGURE 8



FOUNDATION MOVEMENTS UNDER H.E.P.S.



CONTOURS OF VERT. FOUNDATION MOVEMENT

FIGURE 9

MONITORING OF ARCH DAMS

M.D. Fitzpatrick

Section Engineer, Civil Design Division

The Hydro-Electric Commission

INTRODUCTION

The design procedure for arch dams is now well established and involves extensive analytical and model studies in which many simplifying assumptions are made to enable the real and complex structure to be treated as a mathematical or structural model. Despite these simplifications there is substantial evidence indicating that the design methods used result in structures whose stresses and deflections agree well with calculated values. The evidence has been obtained by extensive instrumentation and observation on a number of dams with varying valley shape and foundation conditions, the results showing clearly that reasonable agreement exists between prototype behaviour and that predicted in the design. Most of the important contributions in this field have come from the United States Bureau of Reclamation (U.S.B.R.), the Laboratorio Nacional de Engenharia Civil (L.N.E.C.), Portugal, and the Istituto Sperimentale Modelli e Strutture (I.S.M.E.S.), Italy.

In view of this advanced state of the art it is considered that instrumentation to check in detail the stressed state of the dam is not warranted in the case of a structure whose size is well within the limit of international experience. Not only are such studies costly involving as they must complementary studies of the elastic, creep and shrinkage properties of the concrete but considerable resources are required for their proper and thorough execution.

Where the safety control of the dam is the prime objective it is considered sufficient to monitor only the overall pattern of deformation of the dam and foundation, which, to some extent at least, forms a gross check on its stressed state.

The Hydro-Electric Commission, Tasmania, has 28 dams in service or under construction all of which have been built since 1945.

The number of each type is as follows:-

Earth	4 (including one under construction)
Rockfill	8 (either central or sloping core)
Gravity	3
Prestressed	2
Buttress	1
Faced Rockfill	6 (including one under construction)
Arch	4 (including one under construction)

Monitoring of all these dams is carried out, the extent and frequency being dependent on the size and type of dam and the length and history of service behaviour.

Because of the fairly rapid increase in the number of dams being monitored (18 since 1960) the Commission considered it necessary to centralise all records on its dams and for this purpose formed the Safety of Dams Unit within the Civil Design Division in 1971. This Unit is now responsible for collating and documenting in detail the complete history of each dam and for co-ordinating, assessing and presenting the results of all monitoring observations on dams in service.

This paper describes the types and methods of measurement made on three arch dams and presents the results of some observations. The instrumentation which will be installed in Gordon Dam, now under construction, is also given.

#### TYPES OF MEASUREMENT

A very good reference on what measurements should be made on an arch dam is the I.C.O.L.D. publication entitled

"General Considerations Applicable to Instrumentation  
for Concrete Dams"

written by the Committee on Observation of Dams and Models under chairman, Professor G. Oberti. This paper not only describes the types and methods of measurement and the purpose for which they are made but also gives an indicative list of the instruments available commercially and the companies which manufacture them.

The type of measurements made on the Commission's arch dams are

- (i) absolute horizontal and vertical displacement of points on the dam and foundation.
- (ii) differential displacement of the dam and foundation with respect to a vertical line consisting of a pendulum (or plumb bob) wire.
- (iii) differential displacement of points in the foundation with reference to a simple alignment provided by a tensioned tape or a steel rod anchored at one end.
- (iv) rotation with reference to a horizontal plane
- (v) strain deformation
- (vi) contraction joint gap
- (vii) temperature
- (viii) uplift or pore pressure
- (ix) leakage

#### METHODS OF MEASUREMENT AND INSTRUMENT DETAILS

##### 1) Surveying

Detailed description of the survey procedures used in the Commission in obtaining both absolute and relative displacements have been described by Jenkins and Funnell - ref (1).

They include:-

Base Line measurement using 200ft. long invar tapes standardised at the National Standards Laboratory, Sydney, every 2 or 3 years.

Triangulation observations for which a Wild T3 centesimal theodolite is used having an absolute accuracy of approximately  $\pm 3$  centesimal seconds (or  $\pm 1$  sexagesimal second). The overall accuracy of deflection measurements is considered to be  $\pm 0.002$  ft.

Collimation survey using Galileo adjustable targets and the Wild T3 theodolite as the collimating instrument. The absolute error in the measured deflections is approximately  $\pm 0.005$  ft. per 1000 ft. length of observation distance.

Precise levelling for which a Zeiss Ni 2 automatic level is normally used. A Zeiss Ni 1 is used for work of very high precision.

For lengths of run from 500 to 10,000 ft. and the number of set-ups from 8 to 80 per run the average misclose from an analysis of 94 runs was 0.004 ft. with a maximum of 0.015 ft.

Co-ordinometer measurements using a Galileo optical co-ordinometer to measure the movement of a suspended or inverted pendulum wire with reference to the co-ordinometer base. It provides direct measurement of the x and y components of movement to an accuracy of  $\pm 0.02$  mm, although the actual error is influenced by the stability of the pendulum wire at the time of measurement.

Rotation with respect to a horizontal plane is measured with an instrument called a clinometer. With a base length of 1 m the accuracy is  $\pm 2$  seconds of arc. Both Huggenberger and Galileo clinometers are used.

An electrical clinometer produced by Kyowa (CKE-33A) has a least reading of  $\pm 8$  seconds. This instrument may be used if telemetering of the measurement is required.

## 2) Measurement of Differential Displacement of Points in the Foundation

Two methods are used. The first involves measuring the change in length between the dead end of a drill hole and its collar by means of a rock compression gauge which consists of a steel rod anchored at the dead end and free to move relative to the rock at the collar. The movement is measured either by vernier callipers relative to a reference point anchored to the rock or by a Kyowa (CJ-12G) joint meter fixed to the rock at one end and attached to the steel rod at the other end. Accuracy is approximately  $\pm 0.001$  inch.

In the second method the change in length along an adit is measured by a tensioned tape anchored at one end and supported at intermediate measuring stations spaced at 10 to 20 ft. intervals. Accuracy is approximately  $\pm 0.02$  inch.

## 3) Measurement of Strain, Joint Gap and Temperature

Strain is measured by a Carlson type strain meter (Kyowa CS-25E) which has a gauge length of 250mm and a least reading of  $4 \times 10^{-6}$  units of strain.

Contraction joint gap movements are measured by the Kyowa CJ-12G joint meter. Gauge length is 260mm; range is 12mm for tension and 0.7mm for compression; least reading is 0.04mm.

Both the above meters also provide measurement of temperature to an accuracy of  $\pm 1^{\circ}\text{C}$ . Greater accuracy ( $\pm 0.3^{\circ}\text{C}$ ) is obtained from the Kyowa CTS-100N resistance thermometer.

The electrical conductors for these instruments are embedded directly in the concrete and are routed to multiple switch boxes (Kyowa NS-F series) located in the dam galleries. The maximum number of channels is 66 per box.

#### 4) Measurement of Pore Pressure

Pore pressures (or uplift pressures) have not been measured at any of the 3 existing arch dams. However, at Gordon Dam the effectiveness of the grout and drainage curtains in the steep, high abutments will be measured by monitoring pore pressures within the foundations. The Kyowa pore pressure meter (CP-N series) will be used by sealing them within specially located drill holes. This is an electrical instrument which measures the deformation of a diaphragm due to change in pore pressure by means of a strain meter.

#### 5) Measurement of Leakage

This is done either by channelling the water from a number of drain holes to a V-notch weir or by timing the discharge from a single hole into a calibrated container.

### RESULTS OF MEASUREMENTS

#### 1) Clark Dam

##### a) Description

Clark Dam is a constant radius structure located in the central part of the state on the Derwent River. Originally, when completed in 1949, the dam had a height of 200 ft. and a crest length of 1100 ft. Base thickness is 75 ft. In 1966 its height was increased to 221 ft. by the construction of a prestressed concrete cantilever wall on the original crest. Storage volume was thereby increased by some 70%.

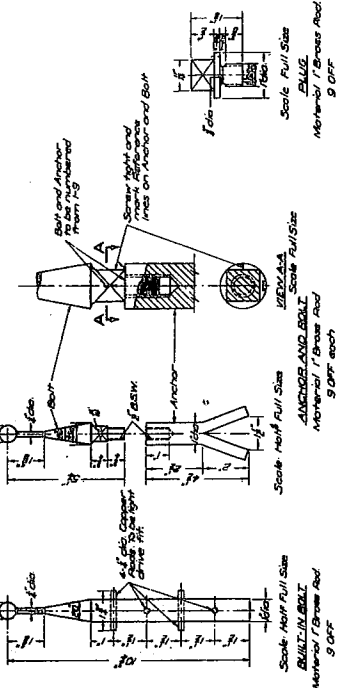
##### b) Instrumentation

The location of survey targets is shown in fig 1.

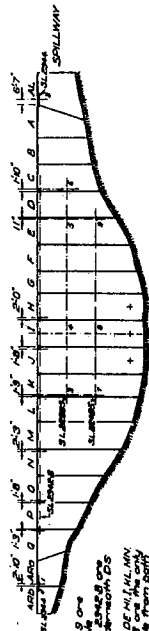
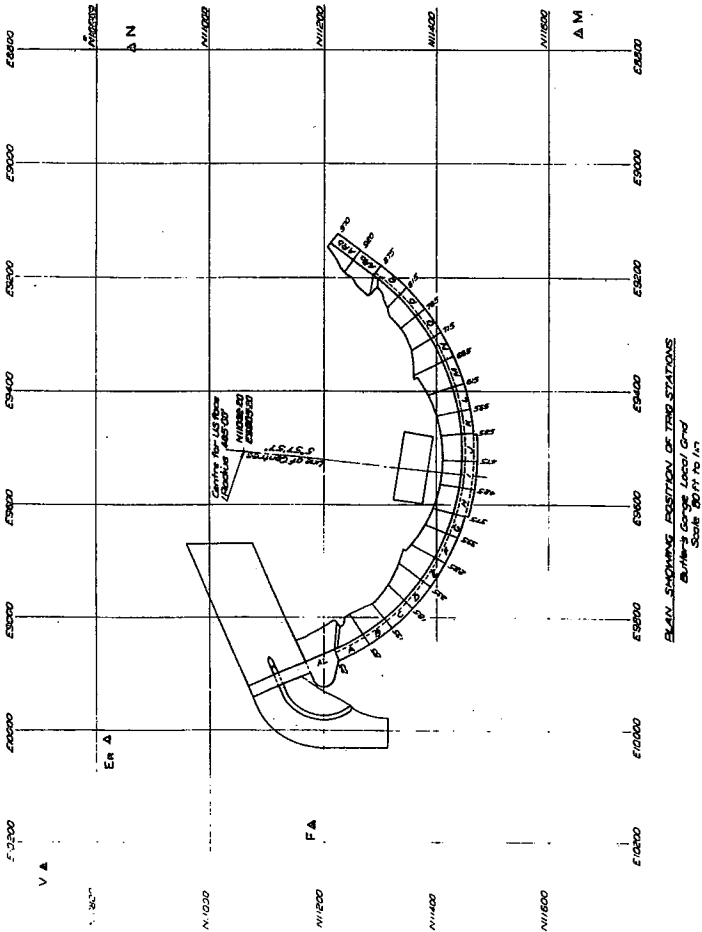
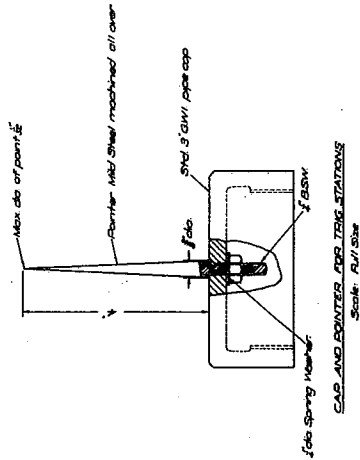
**LOCATION OF TIE STATIONS AND SECTION AND ELEVATION OF LINES**

Station	Section	Elevation
V	VER	5000
E	ER	5000
F	ER	5000
M	ER	5000
N	ER	5000

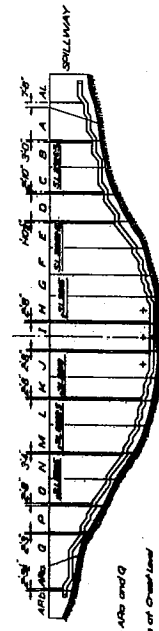
1. Concrete pillars placed with reference to center of section for "N" tie station.
2. Stations A and E are for use as observation points and are placed in permanent position.
3. Stations V, F, M and N are for use as reference points.



**SIGHTING MARKS ON DAM**



**POSITION OF SIGHTING POINTS ON ADMINISTRATIVE FACE OF DAM**  
Scale: 80 FT TO 1 IN



**POSITION OF HOLES FOR BLUES BOSS**  
Scale: 80 FT TO 1 IN

**SIGHTING POINTS** at A, B, C, D, E, F, G, H, I, J, K, L, M, N, O, P, Q, R, S, T, U, V, W, X, Y, Z, AA, AB, AC, AD, AE, AF, AG, AH, AI, AJ, AK, AL, AM, AN, AO, AP, AQ, AR, AS, AT, AU, AV, AW, AX, AY, AZ, BA, BB, BC, BD, BE, BF, BG, BH, BI, BJ, BK, BL, BM, BN, BO, BP, BQ, BR, BS, BT, BU, BV, BW, BX, BY, BZ, CA, CB, CC, CD, CE, CF, CG, CH, CI, CJ, CK, CL, CM, CN, CO, CP, CQ, CR, CS, CT, CU, CV, CW, CX, CY, CZ, DA, DB, DC, DD, DE, DF, DG, DH, DI, DJ, DK, DL, DM, DN, DO, DP, DQ, DR, DS, DT, DU, DV, DW, DX, DY, DZ, EA, EB, EC, ED, EE, EF, EG, EH, EI, EJ, EK, EL, EM, EN, EO, EP, EQ, ER, ES, ET, EU, EV, EW, EX, EY, EZ, FA, FB, FC, FD, FE, FF, FG, FH, FI, FJ, FK, FL, FM, FN, FO, FP, FQ, FR, FS, FT, FU, FV, FW, FX, FY, FZ, GA, GB, GC, GD, GE, GF, GG, GH, GI, GJ, GK, GL, GM, GN, GO, GP, GQ, GR, GS, GT, GU, GV, GW, GX, GY, GZ, HA, HB, HC, HD, HE, HF, HG, HH, HI, HJ, HK, HL, HM, HN, HO, HP, HQ, HR, HS, HT, HU, HV, HW, HX, HY, HZ, IA, IB, IC, ID, IE, IF, IG, IH, II, IJ, IK, IL, IM, IN, IO, IP, IQ, IR, IS, IT, IU, IV, IW, IX, IY, IZ, JA, JB, JC, JD, JE, JF, JG, JH, JI, JJ, JK, JL, JM, JN, JO, JP, JQ, JR, JS, JT, JU, JV, JW, JX, JY, JZ, KA, KB, KC, KD, KE, KF, KG, KH, KI, KJ, KL, KM, KN, KO, KP, KQ, KR, KS, KT, KU, KV, KW, KX, KY, KZ, LA, LB, LC, LD, LE, LF, LG, LH, LI, LJ, LK, LL, LM, LN, LO, LP, LQ, LR, LS, LT, LU, LV, LW, LX, LY, LZ, MA, MB, MC, MD, ME, MF, MG, MH, MI, MJ, MK, ML, MM, MN, MO, MP, MQ, MR, MS, MT, MU, MV, MW, MX, MY, MZ, NA, NB, NC, ND, NE, NF, NG, NH, NI, NJ, NK, NL, NM, NN, NO, NP, NQ, NR, NS, NT, NU, NV, NW, NX, NY, NZ, OA, OB, OC, OD, OE, OF, OG, OH, OI, OJ, OK, OL, OM, ON, OO, OP, OQ, OR, OS, OT, OU, OV, OW, OX, OY, OZ, PA, PB, PC, PD, PE, PF, PG, PH, PI, PJ, PK, PL, PM, PN, PO, PP, PQ, PR, PS, PT, PU, PV, PW, PX, PY, PZ, QA, QB, QC, QD, QE, QF, QG, QH, QI, QJ, QK, QL, QM, QN, QO, QP, QQ, QR, QS, QT, QU, QV, QW, QX, QY, QZ, RA, RB, RC, RD, RE, RF, RG, RH, RI, RJ, RK, RL, RM, RN, RO, RP, RQ, RR, RS, RT, RU, RV, RW, RX, RY, RZ, SA, SB, SC, SD, SE, SF, SG, SH, SI, SJ, SK, SL, SM, SN, SO, SP, SQ, SR, SS, ST, SU, SV, SW, SX, SY, SZ, TA, TB, TC, TD, TE, TF, TG, TH, TI, TJ, TK, TL, TM, TN, TO, TP, TQ, TR, TS, TT, TU, TV, TW, TX, TY, TZ, UA, UB, UC, UD, UE, UF, UG, UH, UI, UJ, UK, UL, UM, UN, UO, UP, UQ, UR, US, UT, UY, UZ, VA, VB, VC, VD, VE, VF, VG, VH, VI, VJ, VK, VL, VM, VN, VO, VP, VQ, VR, VS, VT, VU, VV, VW, VX, VY, VZ, WA, WB, WC, WD, WE, WF, WG, WH, WI, WJ, WK, WL, WM, WN, WO, WP, WQ, WR, WS, WT, WU, WV, WW, WX, WY, WZ, XA, XB, XC, XD, XE, XF, XG, XH, XI, XJ, XK, XL, XM, XN, XO, XP, XQ, XR, XS, XT, XU, XV, XW, XX, XY, XZ, YA, YB, YC, YD, YE, YF, YG, YH, YI, YJ, YK, YL, YM, YN, YO, YP, YQ, YR, YS, YT, YU, YV, YW, YX, YY, YZ, ZA, ZB, ZC, ZD, ZE, ZF, ZG, ZH, ZI, ZJ, ZK, ZL, ZM, ZN, ZO, ZP, ZQ, ZR, ZS, ZT, ZU, ZV, ZW, ZX, ZY, ZZ.

THE HYDRO-ELECTRIC COMMISSION, TAIWAN

**CLARK DAM**

DEFLECTION MEASUREMENTS  
INSTALLATION OF EQUIPMENT

NO.	DATE	BY	SCALE
1	1957	...	...
2	1957	...	...
3	1957	...	...
4	1957	...	...
5	1957	...	...
6	1957	...	...
7	1957	...	...
8	1957	...	...
9	1957	...	...
10	1957	...	...

PROJECT NO. 100-100-100-100

DATE: 1957

BY: ...

SCALE: ...

Handwritten signature and date.

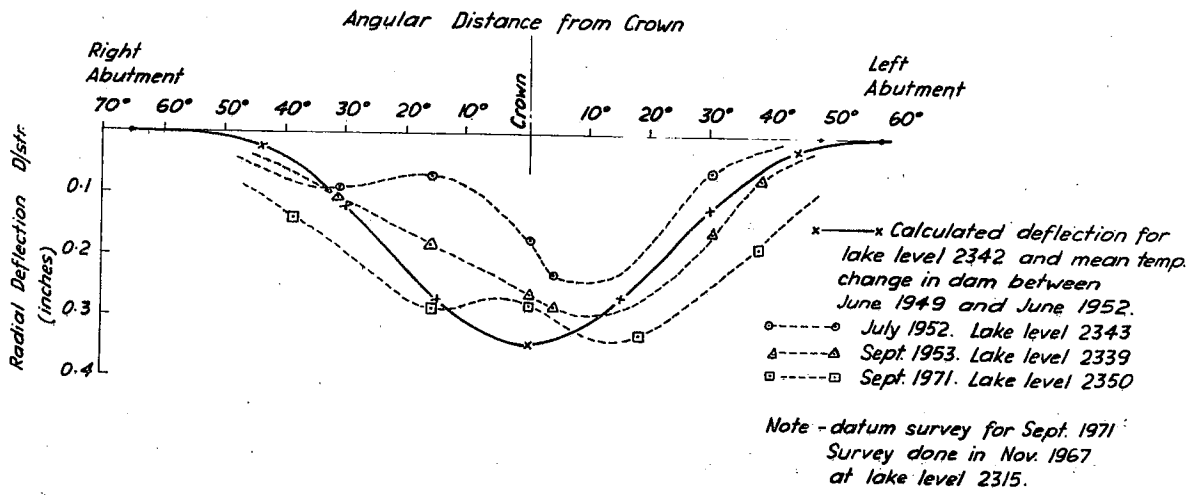


Fig. 2 - CLARK DAM - Radial Deflection of Crest Arch

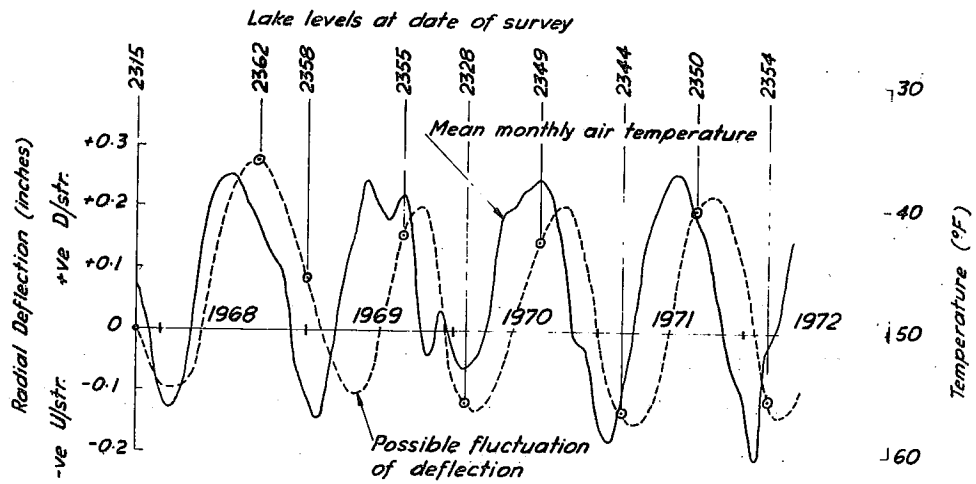


Fig. 3 - CLARK DAM - Radial Deflection at Centre of Crest.

Also shown are the locations of the pendulum shafts - however, due to poor installation of the pipes forming the shafts resulting in deviations from the vertical of almost half the pipe diameter the pendulums were not installed. In addition a number of resistance thermometers were embedded in 4 blocks of the dam and gap gauges were installed across contraction joints to control joint grouting operations.

c) Results

The reservoir (Lake King William) was filled between 1949 and 1952. Deflection surveys carried out in July 1952 and September 1953, have been described by Thomas (2). The measured radial deflections obtained from those two surveys, the survey in September, 1971 and the calculated deflection curves are shown in fig 2. The asymmetry of the measured deflection curve was ascribed to differential temperature between the two abutments.

No further measurements were made until the dam height was increased. At that time it was found necessary to modify the triangulation scheme used in the original surveys.

The new datum survey was carried out in November, 1967 when the lake was at SL 2315. Since then two surveys have been done each year. Fig 3 shows the radial deflection at the centre of the crest for the various surveys, the corresponding water level and the plot of mean monthly air temperature. The correlation with temperature is obvious and masks any influence of varying water load.

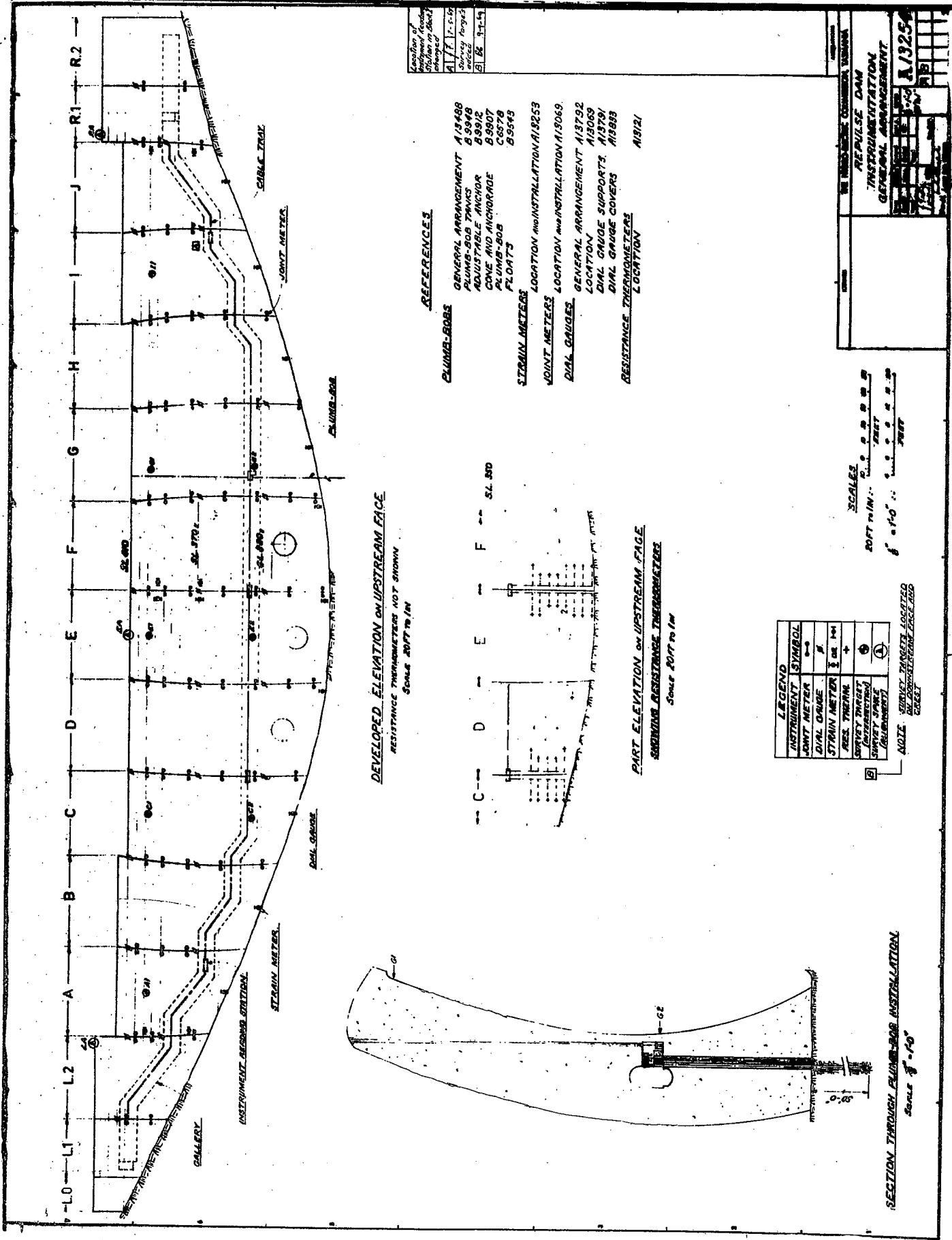
2) Repulse Dam

a) Description

Repulse is a cupola shaped dam, 139 ft. high, 679 ft. crest length and with thickness varying from 30 ft. at the base to 20 ft. at the crest. The dam is located on the Derwent River about 50 miles north west of Hobart. It was completed in 1968.

b) Instrumentation

The general arrangement of the instrumentation is shown in fig 4.



**DEVELOPED ELEVATION ON UPSTREAM FACE**

RESISTANCE THERMOMETERS NOT SHOWN  
SCALE 20 FT TO 1 IN

**REFERENCES**

- PLUMB-BOBS**
  - GENERAL ARRANGEMENT A13488
  - PLUMB-BOB TANKS B9948
  - ADJUSTABLE ANCHOR B9912
  - CONE AND ANCHORAGE B9907
  - PLUMB-BOB C6578
  - FLOATS B9543
- STRAIN METERS**
  - LOCATION AND INSTALLATION A13253
- JOINT METERS**
  - LOCATION AND INSTALLATION A13069
- DIAL GAUGES**
  - GENERAL ARRANGEMENT A13732
  - LOCATION A13069
  - DIAL GAUGE SUPPORTS A13731
  - DIAL GAUGE COVERS A13883
- RESISTANCE THERMOMETERS**
  - LOCATION A13121

INSTRUMENT	SYMBOL
JOINT METER	⊕
DIAL GAUGE	⊖
STRAIN METER	⊕
RES. THERM.	⊕
SAFETY TARGET (INTERSECTION SUPPORT SPARS (REINFORCEMENT))	⊕

NOTE: SAFETY TARGET LOCATED ON ADJACENT DAM WALL AND CHECK

SCALE  
20 FT TO 1 IN  
1" = 20 FT

**SECTION THROUGH PLUMB-BOB INSTALLATION**

SCALE 1" = 1'-0"

Location of Instrument Facing Direction of Shot

A13488	1-1-50
B9948	1-1-50
B9912	1-1-50
B9907	1-1-50
C6578	1-1-50
B9543	1-1-50

REVISIONS

NO.	DATE	DESCRIPTION
1	11/25/54	REVISED DAM INSTRUMENTATION GENERAL ARRANGEMENT

PROJECT NO. A13253

Fig. 4

The system consists of:-

- 7 survey targets for triangulation observations
- 3 collimation targets - one at the crest centre and one on each thrust block
- 1 suspended and 2 inverted pendulums located in G block
- 122 joint meters + 58 dial guage measuring stations also used to measure joint gaps
- 36 strain meters
- 27 thermometers

### c) Results

The strain meters were installed as single instruments and not in rosettes. Their purpose was to obtain some indication of vertical stresses around the base and of vertical and horizontal stresses near crest level. No results of any real value were obtained from them.

The datum survey was carried out in February, 1968 just prior to filling the dam in early March, 1968. Radial deflection at the centre of the dam 10 ft. below crest level, is shown in fig 5 together with mean monthly air temperature and the mean temperature of the crown cantilever. It may be seen that very little change in mean temperature occurred during filling so that the corresponding deflection is due almost entirely to the water load.

In fig 6 the deflection due to the first filling is compared with the calculated deflection due to water load only for the crest arch, the crown cantilever and the G block cantilever. The calculated deflection was obtained by trial load analysis using a value of 5 for the ratio of elastic modulus of concrete to foundation and an elastic modulus for concrete of  $3 \times 10^6$  lb/in<sup>2</sup>. The measured deflection was only 40% of that calculated indicating a considerably greater foundation modulus than that adopted.

Also shown in fig 6 are the calculated deflection curves for water load plus a 9°F temperature rise and water load plus an 8°F temperature fall, the range in deflection for the 17°F change in temperature being 0.59 inches. The measured ranged in deflection for a 12°F change in mean temperature of the crown cantilever has been 0.37 inches.

Comparison between the co-ordinometer measurements on the pendulums and the triangulation surveys shows a maximum difference of inches in the radial direction - fig 7.

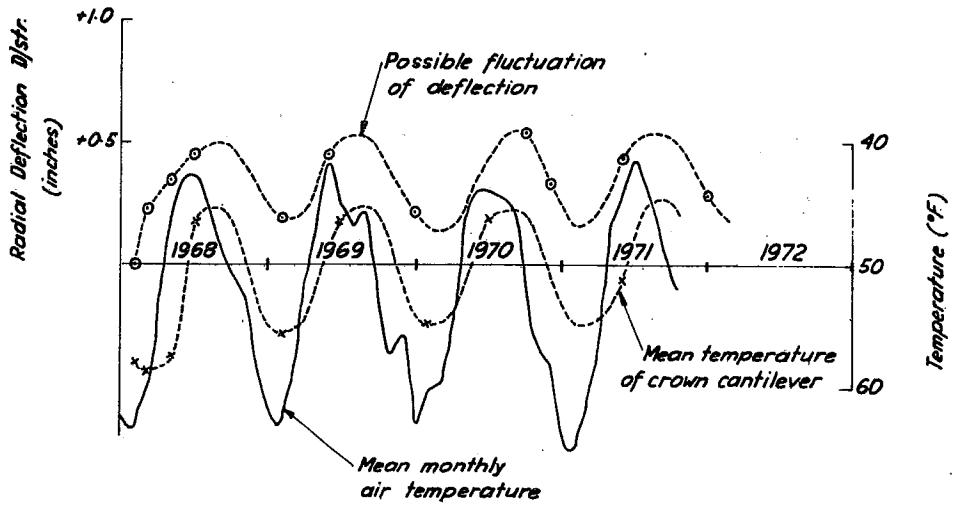


Fig. 5 - REPULSE DAM - Radial Deflection at Centre of Crest

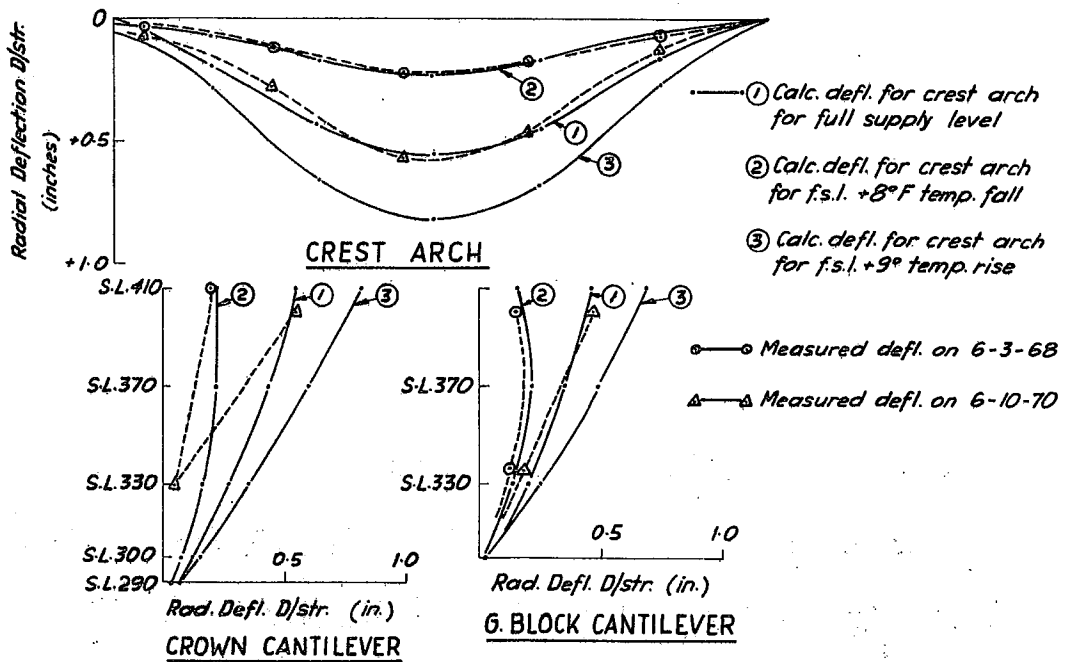
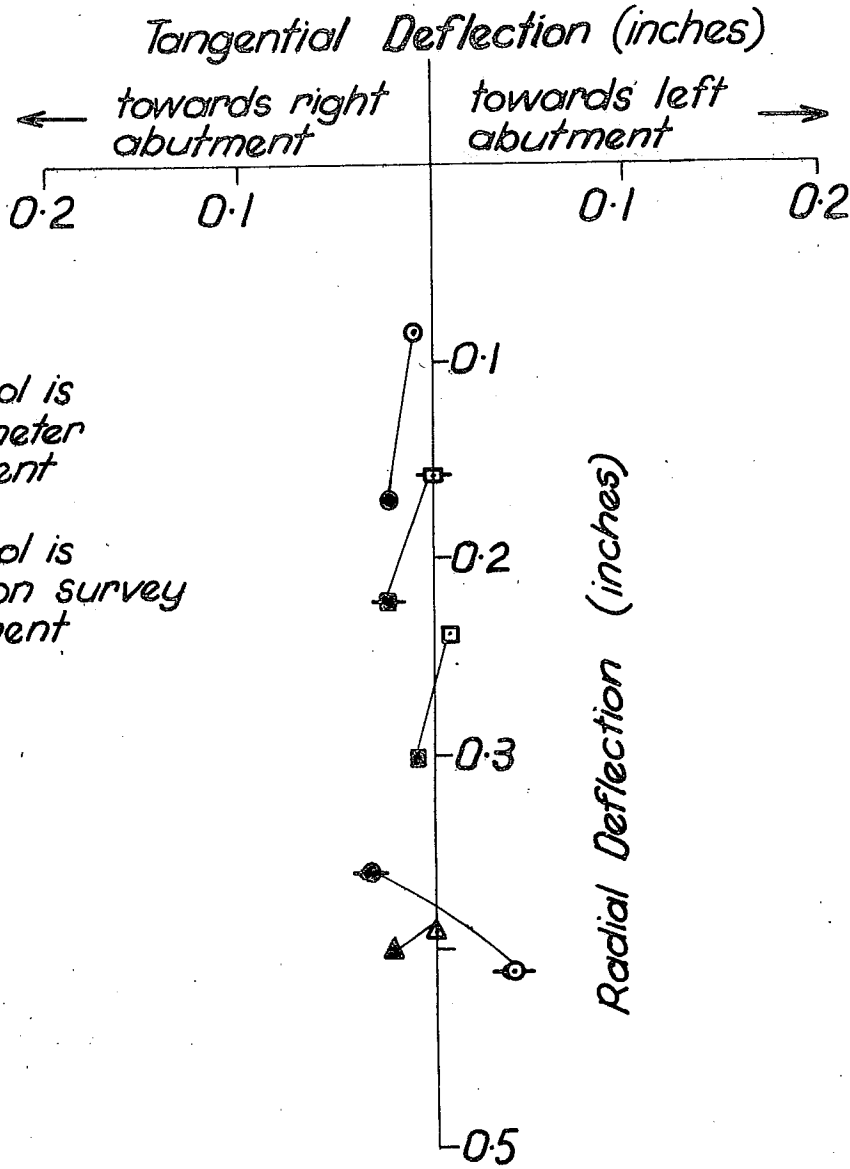


FIG. 6 - REPULSE DAM - Radial Deflection of Crest Arch and Two Cantilevers



Open symbol is  
co-ordinometer  
measurement

Solid symbol is  
triangulation survey  
measurement

- ) 6-3-68
- ) 16-5-68
- △) 4-7-68
- ) 9-6-71
- ) 13-1-72

Fig 7 - REPULSE DAM  
Comparison of Co-ordinometer and  
Triangulation Survey Measurements  
for Target G1.

3) Devils Gate Dama) Description

Devils Gate is a thin cupola shaped dam 275 ft. high and 440 ft. crest length, with thickness varying from 17 ft. at the base to 9 ft. at the crest. The dam is located on the Forth River in the north of the state 20 miles south of Devonport. It was completed in 1969.

b) Instrumentation

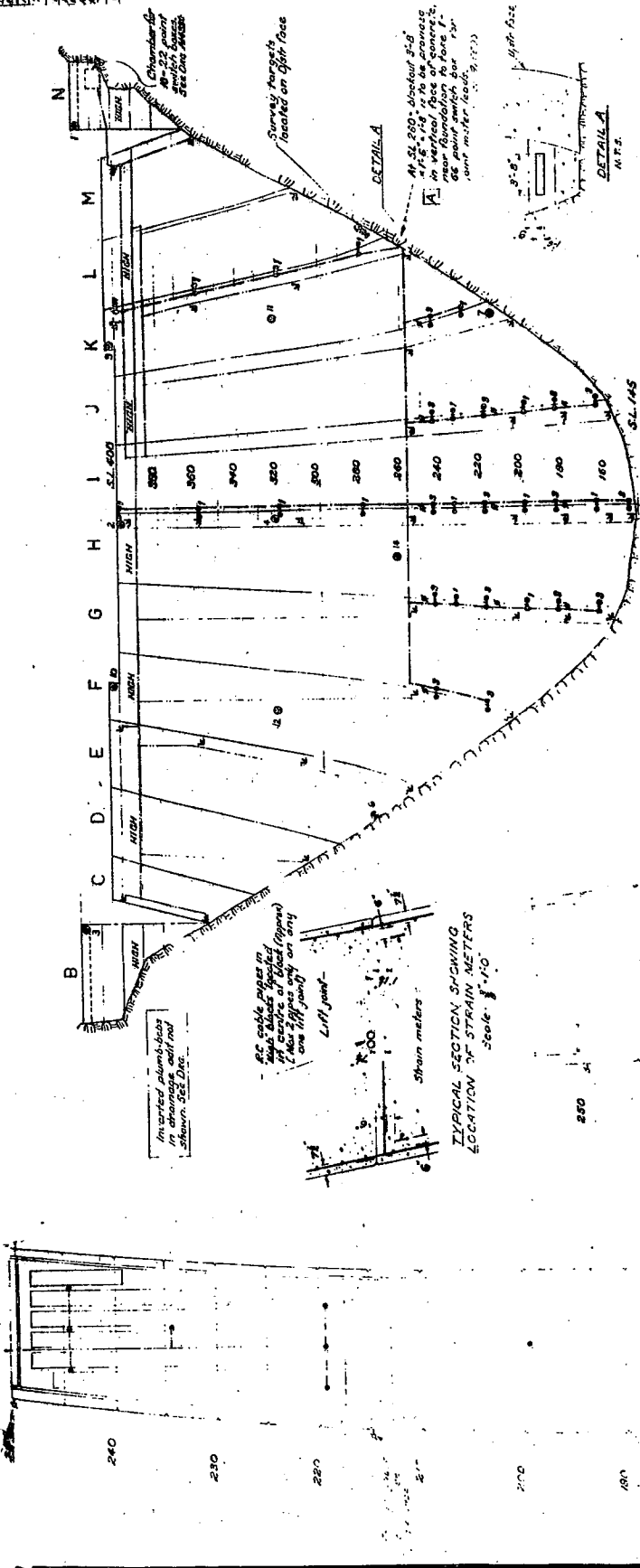
Because of the pronounced vertical curvature of the dam it was only possible to grout the lower one third of the contraction joints and this limitation introduced a high temperature load on the dam. Under the combined loading of own weight plus water load plus temperature load the analysis indicated that significant tensile stresses would occur in several locations. These were the extrados foundation region where both arch and cantilever stresses were in the range 150 to 300 lb/in<sup>2</sup> and the cantilever extrados at the top of the grouted zone where calculated cantilever tensions were in the range 150 to 200 lb/in<sup>2</sup>. It was decided to accept these tensions and to instrument the dam with strain meters to obtain an indication of whether the distribution of stresses was generally in accord with the design.

Fig 8 shows the instrumentation system installed and consisting of:-

- 12 survey targets for triangulation observations
- 70 strain meter rosettes of 3 meters each
- 61 joint meters (plus 22 dial gauge measuring stations for joint gap measurements)

No pendulums were installed because of the pronounced vertical curvature. In addition to the above and not shown in fig 8 two adits in a low modulus zone in the left abutment were instrumented to measure foundation deformation. In the adit at SL 350 there are 3 rock compression gauges of 50 ft. length, one in the radial direction and two in the tangential direction, and tape gauges along the 115 ft. length of adit which is approximately in the radial direction. In the adit at SL 284 there are 5 rock compression gauges each 50 ft. long. Two of these are in the vertical direction, two are in the horizontal tangential direction and one in the horizontal radial direction.

DATE	1/14/52
BY	J. H. ...
CHECKED BY	...
SCALE	1" = 20 FT
PROJECT	...
NO.	...



**DEVELOPED ELEVATION ON UPSTREAM FACE**  
Scale: 20 FT TO 1 IN

**POSITION OF JOINT METERS (Carbon Type Gages)**

1. The location of the joint meters is as shown in the drawing. The meters are to be installed in the concrete at the following locations:
  - (a) about 2 ft from the face of the concrete at the joint.
  - (b) about 2 ft from the face of the concrete at the base of the dam.
  - (c) about 2 ft from the face of the concrete at the top of the dam.
  - (d) about 2 ft from the face of the concrete at the bottom of the dam.
2. The location of the strain meters is as shown in the drawing. The meters are to be installed in the concrete at the following locations:
  - (a) about 2 ft from the face of the concrete at the joint.
  - (b) about 2 ft from the face of the concrete at the base of the dam.
  - (c) about 2 ft from the face of the concrete at the top of the dam.
  - (d) about 2 ft from the face of the concrete at the bottom of the dam.

**NOTES**

1. The location of the joint meters is as shown in the drawing. The meters are to be installed in the concrete at the following locations:
  - (a) about 2 ft from the face of the concrete at the joint.
  - (b) about 2 ft from the face of the concrete at the base of the dam.
  - (c) about 2 ft from the face of the concrete at the top of the dam.
  - (d) about 2 ft from the face of the concrete at the bottom of the dam.
2. The location of the strain meters is as shown in the drawing. The meters are to be installed in the concrete at the following locations:
  - (a) about 2 ft from the face of the concrete at the joint.
  - (b) about 2 ft from the face of the concrete at the base of the dam.
  - (c) about 2 ft from the face of the concrete at the top of the dam.
  - (d) about 2 ft from the face of the concrete at the bottom of the dam.

**DETAIL A**  
M.P.S.

**DETAIL B**

**DETAIL C**

**LEGEND**

SYMBOL	DESCRIPTION
○	JOINT METER
●	STRAIN METER
□	DIAMETER
△	SURVEY TARGET

**SCALES**

200 FT TO 1 IN  
1" = 20 FT  
1" = 10 FT  
1" = 5 FT  
1" = 2 FT

**ELEVATION ON JOINT F-G**  
Position of strain meters not shown.  
Scale: 1" = 10'

**ELEVATION ON JOINT H-I**  
Position of strain meters not shown.  
Scale: 1" = 10'

THE WASHINGTON CONSULTING ENGINEERS  
114793  
ARCH INSTRUMENTATION  
GENERAL ARRANGEMENT

DEVELOPED ELEVATION ON UPSTREAM FACE  
Scale: 20 FT TO 1 IN

POSITION OF JOINT METERS (Carbon Type Gages)

NOTES

DETAIL A

DETAIL B

DETAIL C

LEGEND

SCALES

ELEVATION ON JOINT F-G

ELEVATION ON JOINT H-I

Fig. 8

c) Results

The reservoir (Lake Barrington) was filled between 26th July and 4th September, 1969.

Strain measurements were converted to stresses using a single value of Young's modulus and without any allowance for creep and shrinkage. The presence of the tension zones in the extrados foundation region and across the extrados at the top of the grouted zone was confirmed. The results also indicated that the whole of the downstream face is in compression.

The deflection at the centre of the crest is shown in fig 9 together with a correlated mean monthly air temperature. It will be noted that there is virtually no lag between the mean monthly temperature and the temperature movement of the dam and this is to be expected in such a thin structure.

The calculated deflections, due to water load, of the arch at SL 400 (crest) and SL 310 are shown in fig 10 together with the maximum and minimum measured deflections obtained to date. The same information for the crown cantilever is shown in fig 11. The mean temperature and temperature gradients in the crown cantilever are shown in fig 12 for the date of the datum survey and 3 subsequent dates. Because of the temperature changes that took place during filling it is not possible at this stage to determine the deflection due to the water load alone. When a few more deflection surveys have been carried out with simultaneous measurement of temperature distribution it will be possible to determine the separate contribution of water and temperature loading statistically.

The tangential deflection of both point 5 on the right abutment and point 6 on the left abutment has gradually increased to about  $\frac{1}{8}$ th inch in the 3 years since filling. Smaller foundation deformation was obtained from the rock compression gauges in the left abutment at about the same level as point 6, results being in the range zero to  $\frac{1}{16}$  inch.

GORDON DAM

1) Description

Gordon Dam will be a double curvature structure 460 ft. high with a thickness of 50 ft. at the base reducing to 9 ft. at the crest. The crest length will be 625 ft.

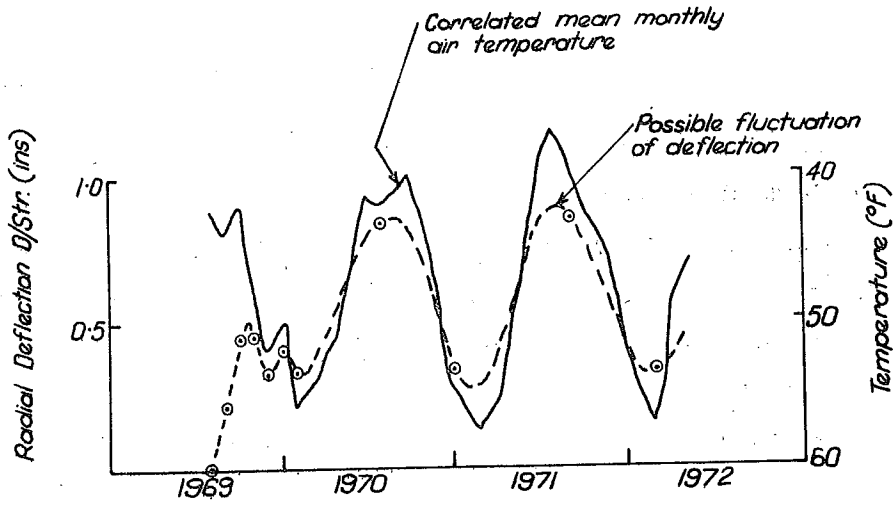


Fig 9 - DEVILS GATE DAM - Radial Deflection at Centre of Crest

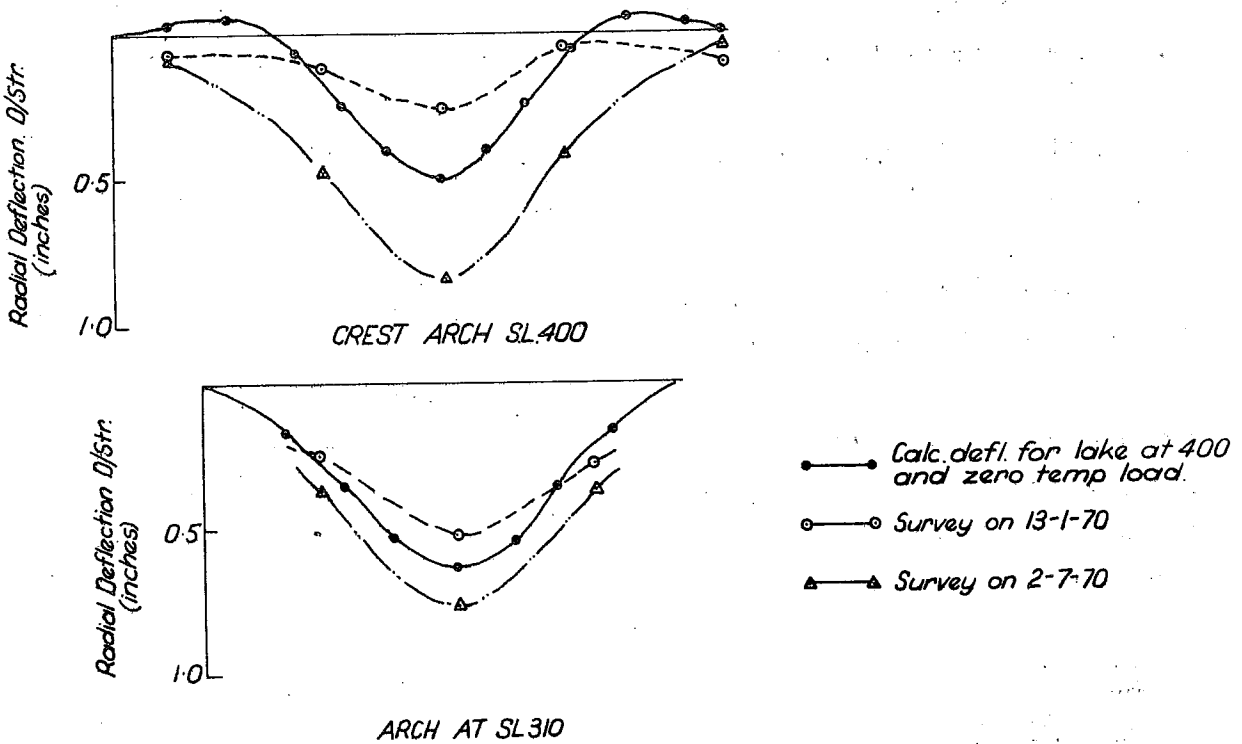


Fig 10 - DEVILS GATE DAM - Radial Deflection of Two Arches

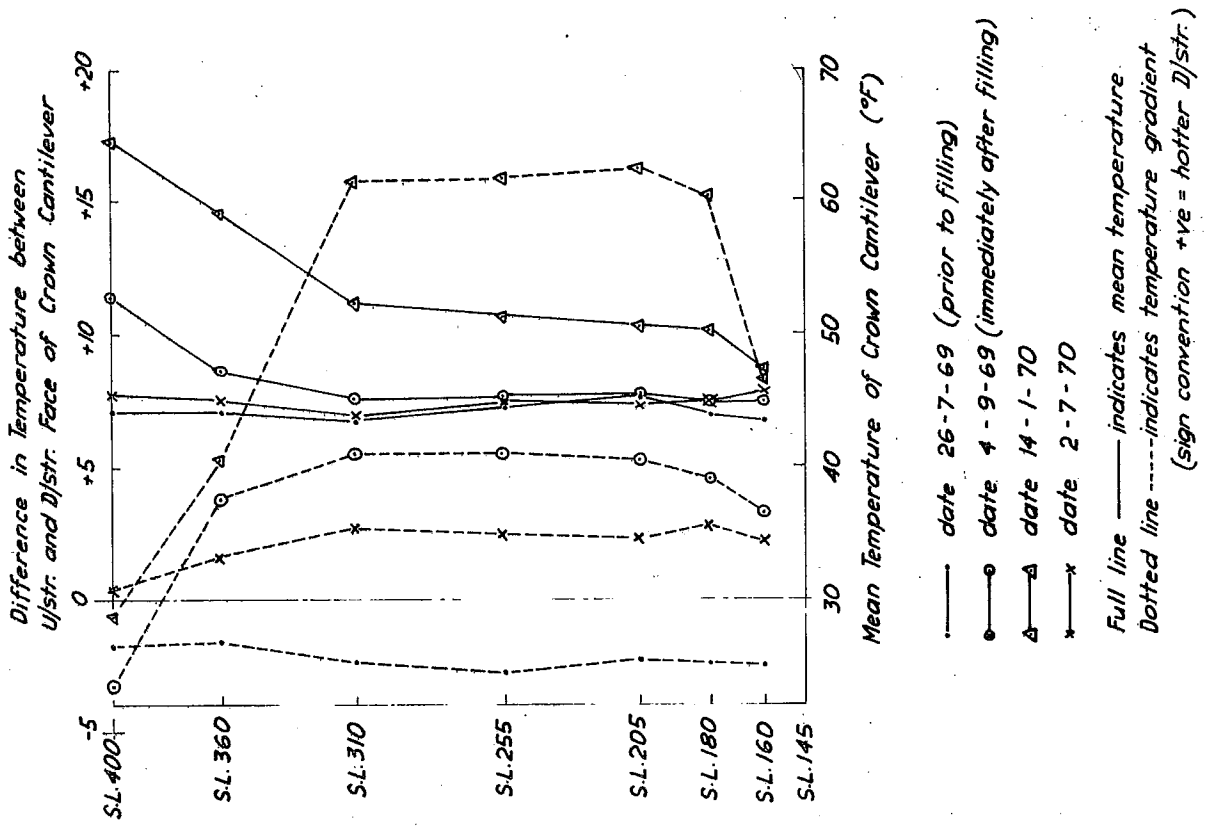


FIG. 12 - DEVILS GATE DAM  
Temperature of Crown Cantilever

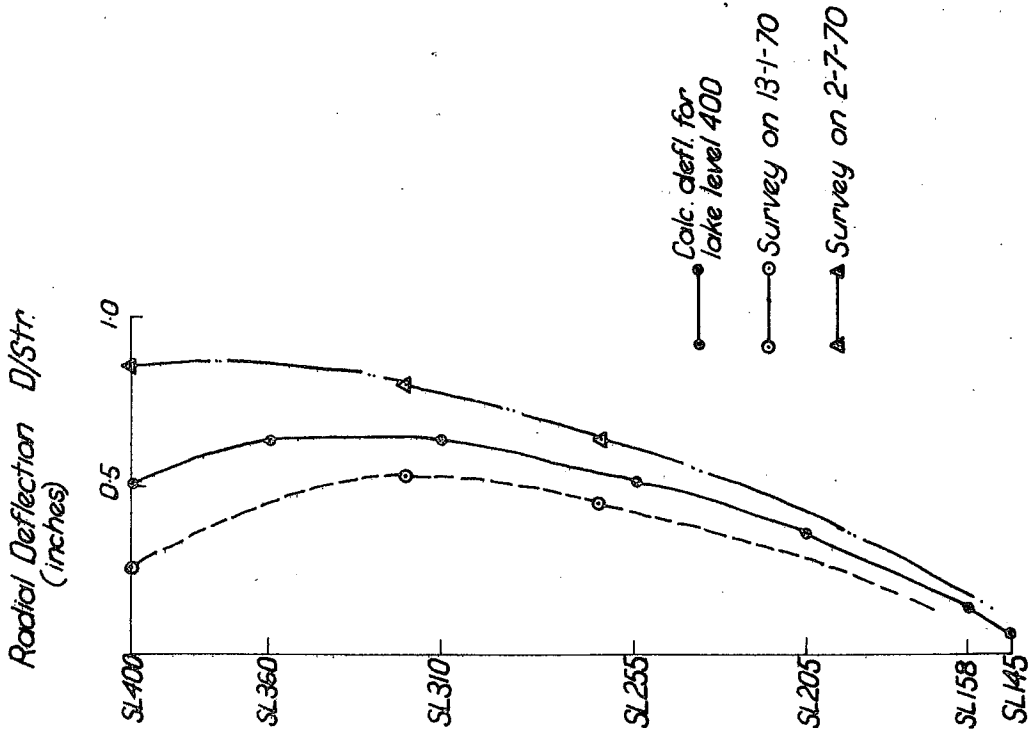


Fig 11- DEVILS GATE DAM  
Radial Deflection of  
Crown Cantilever

The dam is under construction at the present time in the south west region of the state. Concrete construction is in progress and approximately 10% of the 200,000 yd<sup>3</sup> of concrete has been placed.

2) General

Design studies for the dam were extensive and included:-

trial load analysis with radial and tangential adjustment

structural model tests at the Commission's Civil Engineering Laboratory and at L.N.E.C., Portugal

3 dimensional finite element analysis by Professor Zienkiewicz at University of Swansea

The agreement between all studies was in general excellent and showed that no tensions were present for the combined loading of own weight plus water load and that with temperature load added tensile stresses greater than 150 lb/in<sup>2</sup> were present in only three very localised areas.

The reservoir, Lake Gordon, and the adjacent Lake Pedder with a combined storage of 12 million acre feet will constitute a very large crustal load of  $1.4 \times 10^{10}$  tons. The possibility of increased seismic activity in the region due to this load was considered real and various seismic and related projects have been undertaken to monitor seismic events. These include:-

Extension of the 12 years old state seismic net into the south-west area

Installation of several micro-earthquake stations in the region

Precise repeat levelling and trilateration

In addition several strong motion instruments will be located at the dam.

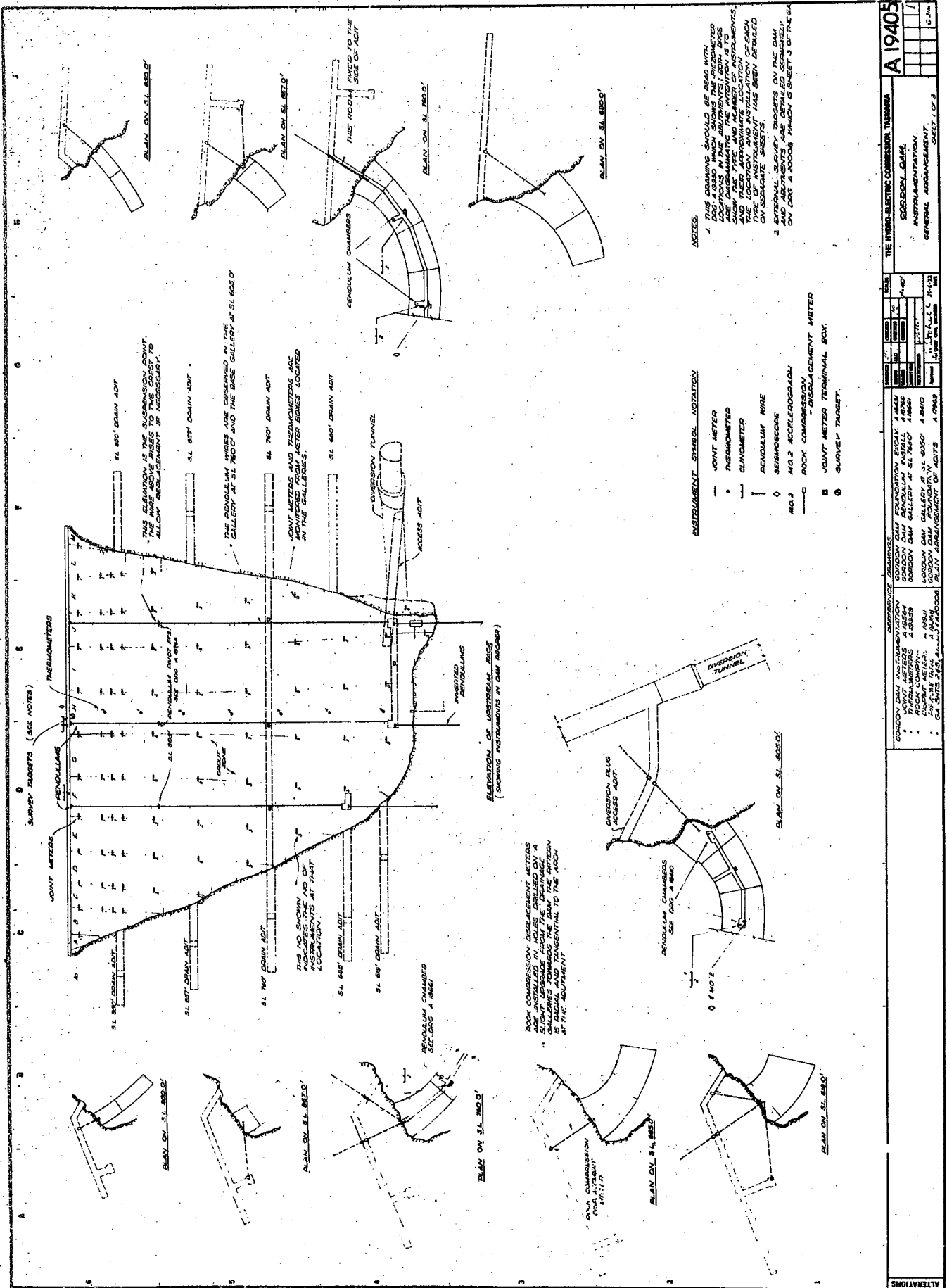
3) Instrumentation

As mentioned above considerable work is involved in first instrumenting to measure strains and then in calculating stress from measured strain in concrete. In addition to the cost of instrumentation there is the cost of the necessary creep studies and of applying the laboratory data to measured strains in order to compute stress. Work on Repulse and Devils Gate showed that credible stress values would not be obtained without making the correct allowance for creep. In view of the cost and resources required for this work and because of the satisfactory state of calculated stress under all loading conditions it was decided not to install any strain meters but simply to monitor the deformations of the dam and foundation as a whole.

The instrumentation which will be installed in the dam is shown in figs 13 to 18 and consists of

- 23 survey targets (fig 15)
- 3 suspended and 3 inverted pendulums (fig 15). The suspended pendulums will hang from a suspension point at SL 905, pass through intermediate measuring chambers at SL 760 and terminate near the base of the dam. The inverted pendulums will be anchored 50 to 80 ft. below the foundation surface in cased 7 in. dia. drill holes and extend to the same chambers in which the suspended pendulums are terminated.
- 18 clinometer positions fig 13 of 1 in. base length, generally in orthogonal pairs to measure rotation in the radial and tangential directions.
- 150 joint meters, fig 13 and 16
- 30 resistance thermometers fig 17
- 32 piezometers fig 14
- 18 rock compression meters fig 14 and 18
- 1 MO2 accelerograph fig 13
- 4 Wilmot seismoscopes fig 13

In order to increase the accuracy of triangulation deflection surveys and reduce the time required to carry out each survey the Commission has placed on order a Kern Mekometer.



**NOTES**

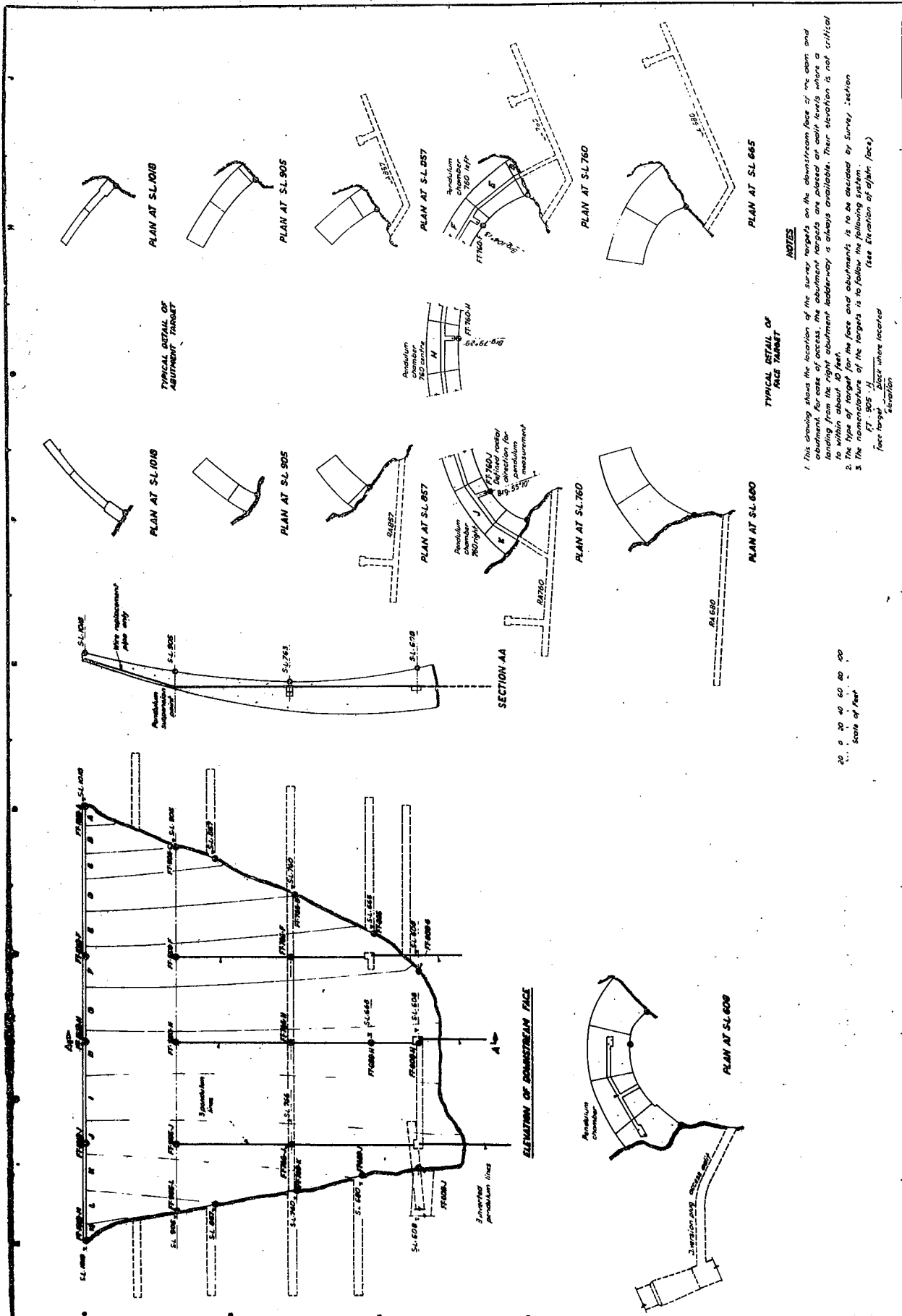
1. THIS DRAWING SHOULD BE READ WITH Dwg. A 1940 WHICH SHOWS THE INSTRUMENT LOCATIONS IN THE ADJUSTMENT ROOMS TO WHICH THE TYPE AND NUMBER OF INSTRUMENTS, THE LOCATION AND INSTALLATION OF EACH TYPE OF INSTRUMENT HAS BEEN DETAILLED ON SEPARATE SHEETS.
2. SURVEY TARGETS ARE ON THE DAM AND METER BONES ARE DETAILLED SEPARATELY ON Dwg. A 2008 WHICH IS SHEET 3 OF THE 6.

- INSTRUMENT SYMBOL NOTATION**
- JOINT METER
  - PRESSUREMETER
  - CLAMMETER
  - PENDULUM WIRE
  - ◇ SEISMOSCOPE
  - NO. 2 ACCELEROGRAM
  - ROCK COMPRESSION - DISJUNCTION METER
  - JOINT METER TERMINAL BOX
  - SURVEY TARGET

THE HYDRO-ELECTRIC COMMISSION TUZUMA GORDON DAM		A 1940S	
GENERAL ARRANGEMENT		SHEET 1 OF 3	
DATE	1954	BY	J. L. L. 11.11.54
SCALE	AS SHOWN	CHECKED	J. L. L. 11.11.54
PROJECT	GORDON DAM	DESIGNED	J. L. L. 11.11.54
NO.	1940S	APPROVED	J. L. L. 11.11.54
<p><b>DESCRIPTION</b></p> <p>ROCK COMPRESSION DISJUNCTION METERS                  JOINT METERS                  PRESSUREMETERS                  CLAMMETERS                  PENDULUM WIRE                  SEISMOSCOPE                  NO. 2 ACCELEROGRAM                  ROCK COMPRESSION - DISJUNCTION METER                  JOINT METER TERMINAL BOX                  SURVEY TARGET</p>			

FIG. 13





**NOTES**

1. This drawing shows the location of the survey targets on the downstream face of the dam and abutment. For ease of access, the abutment targets are placed at odd levels where a landing from the right abutment ladder-way is always available. Their elevation is not critical to within about 10 feet.
2. The type of target for the face and abutments is to be decided by Survey Section.
3. The nomenclature of the targets is to follow the following system:  
 (a) 'SL' - Station where located  
 (b) 'Face' - Face where located  
 (c) 'Target' - Target where located

Scale of Feet  
 0 10 20 30 40 50 60 70 80 90 100

<p>THE HYDRO-ELECTRIC COMMISSION, TASMANIA  <b>GORDON DAM</b>                  INSTRUMENTATION                  GENERAL ARRANGEMENT. SHEET 3 OF 3</p>	
<p>PROJECT NO. A20008                  DRAWING NO. 4076                  SCALE 1/4" = 1'-0"                  DATE 1/1/50</p>	<p>DESIGNED BY                  CHECKED BY                  APPROVED BY</p>







This is a direct distance measuring instrument of a high order of accuracy. Experience with the 3 or 4 instruments which exist in England and Switzerland shows the accuracy to be within  $\pm 0.1 \text{ mm} = 3$  parts per million of the site distance. Thus over a site distance of 300m the accuracy would be  $\pm 1 \text{ mm}$ .

Also under investigation at the present time is the possibility of obtaining measurements from the pendulums by means of electrical transducers. This is part of an overall proposal to telemeter measurements from a selected group of instruments either to Hobart or to the Gordon underground powerstation. The object of the proposal is to provide a more effective safety control procedure at considerably reduced cost during the life of the dam.

#### 4.) Assessment of Results

As stated above monitoring of Clark, Repulse and Devils Gate Dams is carried out two times per year. Assessment of the results consists of comparison with calculated values to see that measured deflections are of the right order and comparison of the seasonal observations to see that the expected temperature dependent fluctuations occur. The assessment is qualitative and is not capable of detecting whether an apparent anomaly is due to the particular loading conditions at the time of measurement or whether it is due to some unexpected phenomenon.

For Gordon Dam it is intended to use the measurements to establish empirically, by statistical methods, the response of the dam to the fluctuations of load, both water and thermal. When a correlation has been established detection of an abnormality will be an easier task, and the assessment of the safety of the structure will be soundly based on a variation from an observed repeatable pattern. This method of safety control is used extensively in Portugal - (3) and (4) and Italy (5).

#### CONCLUDING REMARKS

The monitoring of an arch dam for safety control requires systematic observation of the deflections of the structure and its foundation during construction and reservoir filling and throughout its life. Where possible two or three different methods of measurement should be employed to give greater overall confidence in the results.

Only by comparison between parallel sets of measurements can a systematic error in any one method be detected.

The deflection of the dam during filling is dependent on the increase in water load, the change in temperature and also creep, if the filling rate is slow. Subsequently, if the lake level remains essentially unchanged, deflection will fluctuate with temperature and increase asymptotically due to creep. Determination of the separate components of deflection due to water load, temperature and creep is of importance in assessing the results in order to find out whether an apparent anomaly is due to the specific loading conditions at the time or whether it is due to some unexpected phenomenon. The separate contribution of the various load parameters may be found simply by statistical methods. It is proposed to carry out this form of quantitative assessment on observations of Gordon Dam.

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MONITORING OF EARTH AND ROCK DAMS

C. G. C. Coulter

Electricity Commission of N.S.W.

MONITORING ELECTRICITY COMMISSION OF NEW  
SOUTH WALES' DAMS

1. INTRODUCTION

The Electricity Commission of New South Wales owns and operates eleven dams. The purpose and some salient details of these structures and the instruments installed to monitor performance are given in Table 1A and 1B.

It is standard Commission practice for the dam designers to be responsible for obtaining and interpreting performance data during the construction and contract maintenance periods. At the end of the maintenance periods, responsibility for this work is transferred to the Power Development Division and it is this phase of the work which is covered in this paper.

Each dam is inspected at least once a year and a formal inspection report prepared. Depending upon conditions of the structures and their past performances, additional inspections are made at more frequent intervals of those features of a dam which may be causing concern.

With the exception of Tallawarra No.1 Ash Dam, Wangi Ash Dam and Ulan Water Supply Dam, each dam has been instrumented to some extent with ground water level observation pipes, piezometers, settlement cross arm installations, as well as survey monuments to observe horizontal and vertical movements and weirs to measure seepage rates. The numbers and types of instruments installed have been determined having regard for the size, importance and condition of the structure, and to some extent the consequences of failure.

Immediately after construction, surveys to determine settlements and deformations are usually made at three monthly intervals, but this is reduced progressively to six monthly and yearly intervals once the reservoir has been filled and the pattern of movements has been established.

In the case of the critical section of Tallawarra No.2 Ash Dam (See Section 3 below), surveys were taken as frequently as once weekly during the period near the end of construction and immediately thereafter and, for this structure, the time intervals between surveys are now still assessed having regard for the previous results.

Ground water levels, piezometers and seepage flows are usually observed at monthly intervals during filling periods and this is reduced to three or six monthly intervals after seepage conditions have stabilised or when the observations show that changes are occurring slowly.

Piezometer, ground water level and seepage observations are made by a Technical Officer working under the supervision of a senior engineer. The Technical Officer is also responsible for plotting and recording results. He has developed some expertise in the field and now makes the annual routine inspections of the minor structures.

Surveys to determine horizontal and vertical movements are made by officers from the Commission's Survey Branch.

Interpretation and assessment of the observations are made by senior engineers. This usually amounts to an examination of the trends in the rates of change of seepage, settlement, horizontal movement and pore pressures. The piezometer and ground water level readings are checked for high pore pressures in areas protected by filters and near the downstream toes. For recently constructed dams the piezometer readings are further examined to check whether steady state seepage conditions are being reached and to ensure that filters are effective.

To illustrate this work, further details are given hereunder of the Liddell Cooling Pond Dam and Tallawarra No.2 Ash Dam. These two examples represent extreme cases of our experience. The cooling pond dam is the largest structure monitored. Its performance has been satisfactory and

the major monitoring difficulty has been the establishment of survey procedures to measure settlement and deformation. By comparison, the Tallawarra ash dam is a relatively low structure which has been deliberately built to a low factor of safety. In this case, the performance observations have been used to assess the limits of safe construction and operating levels, and thus the need for augmentation.

## 2. LIDDELL COOLING POND DAM

Liddell Cooling Pond Dam is shown in plan on Figure 1 and cross section on Figure 3. Details of the structure have been published previously in Reference 1 and some behaviour data in Reference 2.

A total of sixty-eight hydraulic piezometers were installed in the dam at the two sections shown on Figure 1. Twenty-nine ground water observation holes were installed in abutments and downstream of the dam. Four cross arm settlement installations were installed near the piezometer installations. A total of twenty three survey monuments were installed on the dam crest and on the downstream face of the dam to check settlement and horizontal movements.

The drainage system from the inclined drain and filter under the downstream part of the dam is collected at a point adjacent to the original creek and is measured with a weir. A number of weep holes in the diversion tunnel downstream of the plug have discharged a considerable quantity of water and provision has been made for fitting a temporary weir in the drainage channel in the floor of the tunnel to measure this flow. The coal seam which outcrops within the storage near the left abutment has been worked and the tunnels extend to within about 10 chains of the reservoir. Seepage into these tunnels flows to the current working sections of Liddell Colliery. The tunnels are inspected annually and seepage rates estimated.

Details of the survey net and procedures warrant special mention because of the difficulties which have been experienced in obtaining satisfactory results. The control stations and observation points are shown on Figure 1.

It was originally planned to observe the settlement and deformation points on the crest and downstream slope of the dam from survey stations located on the dam abutments. The stations were established but it was found that relationships between them were not being maintained. Four additional key control stations were therefore established downstream of the dam at the locations shown on Figure 1. Selection of the sites for these stations was made having regard for underground coal mining operations downstream of the dam and possible earth movements caused by subsidence. The sites were finally located on mining barriers where the effects of subsidence could be expected to be minimal.

There has been a history of instability with survey reference marks in the Liddell area. Measurements on 1 cubic foot concrete survey monuments have shown vertical movements of up to 0.1 feet and horizontal displacements of 0.03 feet. In order to limit such movements the key control stations were constructed of 8 cubic yard blocks of concrete cast in place below ground level.

Special precautions have been taken to ensure that the survey net provided by the key control stations is secure and that the net can be re-established even after the loss of two stations. The net has been defined by high accuracy distance measurements using an electro-optical machine combined with high precision triangulation. Instruments and targets are fixed with position locating pins cast into the blocks.

Refraction problems generally require that surveys be carried out at night.

At the start of each settlement and deflection survey, the levels of the corners of the key control stations are levelled to ensure that tilting has not occurred. It has been found that the large reference marks have tilted up to 0.01 feet. The control net is then surveyed to ensure that the interrelationship between marks has been maintained. The positions of the observing survey stations on the dam abutments are observed and these stations are then used to fix settlement point positions. Adjustments of the triangulation nets and determination of the correct settlement point positions are calculated using a computer. The program also determines the error probability of the position fix.

In the absence of changes in the position of the key control stations, it is estimated that the positions of the settlement points are defined to an accuracy of about 0.01 feet in the transverse direction and to about 0.02 feet in the longitudinal direction.

Difficulties have been experienced in providing a secure base for level measurements and a station remote from the dam is used for this purpose. The relative levels between the reference station and settlement points is determined to an accuracy of  $\pm 0.005$  feet but the absolute level accuracy is probably of the order of  $\pm 0.02$  feet.

Surveys are taken annually and the results of the recent survey are shown on Figure 2. Post-construction settlements and movements have been small and give no rise for concern for the stability of the structure.

The pore pressures measured at cross section chainage 11 + 72 are shown on Figure 3. Examination of these show that as of March, 1972, construction pore pressures have not completely dissipated and that steady

state seepage conditions have not been reached. The piezometer readings upstream and downstream of the chimney drain indicate that it is working effectively and piezometer readings in the foundation under the downstream slope indicate that the blanket drainage facilities are performing satisfactorily.

Since the dam was filled in 1969, seepage from the dam drainage system and from the diversion tunnel has remained sensibly constant at about 0.1 and 0.2 cusecs respectively.

High ground water levels have developed in the left abutment near the zone of high seepage into the diversion tunnel. The ground water levels in this area increased at a rate corresponding to the pond water level. There has been no noticeable increase in seepage into the coal mine since the pond was filled.

### 3. TALLAWARRA NO.2 ASH DAM

The Tallawarra ash dams are ring dams as shown on Figure 4. The dams have a maximum height of about 30 feet, and average about 20 feet. No.1 pond, of approximately 400 acre feet storage, was filled in 1961. No.2 pond has a capacity of about 2,400 acre feet of which about 1,800 acre feet has been filled by mid 1972.

Between chainages 1200 and 4400, the No.2 dam is founded on soft marine clays up to 60 feet thick, which have given difficult foundation conditions. Elsewhere, the dam is founded on stiff residual soils which have given satisfactory foundations.

The deepest section of soft foundation material is located between chainages 2800 to 4400 and the remainder of this discussion will be concerned with this part of the dam.

The section of the dam of interest was built in stages as shown on Figure 5. The stages of construction were:

- 1st Stage: November, 1961 To R.L. 12 with 90 feet crest width
- 2nd Stage: Early 1963 To R.L. 20 with 15 feet crest width
- 3rd Stage: Feb.-May, 1965 To R.L. 30 with 20 feet crest width

The factor of safety for the third stage of construction was assessed to be unity when constructed to R.L. 30. Consideration was given (in 1965) to constructing the dam to a safer lower level but there were a number of impelling reasons for building the dam as high as possible at that time. To achieve this, it was decided to use lightweight fly ash fill (85 lb./cu.ft. bulk density compared with 130 lb./cu.ft. for the alternative clay fill available), to measure settlements and horizontal movements of reference marks installed on the downstream side of the dam and to assess from the deformation observations when an unsafe condition was being reached so that construction could be stopped. The layout of the reference marks is shown on Figure 6. Observation survey stations were located on the stable sections of the dam at chainages 2850 and 4000 and these are checked against key reference survey stations located away from the dam.

Figure 5 also shows the settlement and deformation observations obtained during the construction period (ending May, 1965) and through to the end of 1966.

The fill was placed in about 2 foot lifts during discrete time intervals and each period of fill placement was followed by a rest period of about a week. It was found that the addition of fill caused the settlement points on the lower berm and at the toe of the dam to rise (see Figure 5 - settlement point 12 and pipe). During the rest period, the level of these marks fell. Figure 5 clearly shows three periods of fill placement followed by rest periods. It was planned during construction, that if the reference marks did not fall during any rest period to cut down the previously

placed lift and to finish off the dam at the lower level. This did not occur and the dam was finished off at the target level of R.L. 30.

Figure 6 shows the total settlements and lateral movements recorded between 1965 and 1972. Settlements have been reasonably uniform. There is a rapid increase in settlement (0 to 2 feet) over the 200 feet length of dam between settlement points 25 and 24 and this severe deformation has caused the dam to crack transversely at the location shown on Figure 6. The differential settlements in this area are caused by steeply dipping base-rock surface and a corresponding rapid change in the thickness of the compressible clay layer.

Horizontal movements have been much less uniform along the length of the dam shown on Figure 6. These range from 0.05 feet on survey mark No. 27 to 0.7 feet on mark No. 16. The small displacements at the bend in the dam are probably related to the spread (three dimensional) support provided at corner and also by the reduced thickness of soft material in this area. The effect of differences in thickness of the soft stratum has not been reflected in settlements at this stage of consolidation.

To the east of the corner (settlement point No. 27) the dam has moved generally about 0.6 feet laterally and the movements have been reasonably uniform throughout the cross section. To the west of the corner the transverse horizontal movements have been greater at the toe of the dam than at the centreline.

Approximately one half of the total recorded horizontal movements occurred within two years of completing Stage 3. The movements are still continuing at a slow rate.

A total of 31 piezometers (stand pipe type) have been installed in the soft clay foundations. Figure 7 shows typical piezometer records plotted against height of dam, ponding level and tailwater level. Piezometer tip No. 18 was located at chainage 3450 near the centre of the clay stratum and approximately under the centre line of the 2nd Stage west. Piezometer tip No. 33B2 is located in the same relative position at chainage 3350. The example shown is typical of our experience with these instruments. It has been found that the standpipes are readily damaged despite all reasonable precautions taken to protect them, and if replaced by new instruments located at virtually identical positions, the two records are often inconsistent.

As shown on Figure 7, pore pressures are still high in the centre of the clay layer but they have fallen in the upper and lower parts of the clay stratum. The increase in pore pressure reading observed by tip No. 33B2 from 1968 to 1972 is undoubtedly related to the increase in pond level. However, the relatively rapid increase during 1970 cannot be explained with any certainty.

The factor of safety of the structure against a slip circle failure has been estimated recently using soil strength data and a pore pressure field based on the piezometer observations. The computed values ranged from 0.7 to 1.3 depending upon the method of analysis used. The wide range in values appears to be typical of this situation where the angle subtended by the critical slip circle is large and foundation pore pressures are high.

The future stability of the structure depends on the rate of pore pressure dissipation and the rate of increase in storage level with further ponding of ash. It is considered that calculated factors of safety are not precise and cannot be used for deciding whether the overall stability of the structure is improving or deteriorating. In this situation, recourse

must continue to be made to the procedure which has been evolved for observing trends in settlements and movements in order to decide when stability is deteriorating and if left unchecked would lead to failure. When it is considered that such a situation has been reached, construction will be started on the fourth stage embankment located on the ash deposits upstream of the existing dam.

#### 4. DISCUSSIONS

The case histories which have been briefly described above highlight the different approaches which have been taken to monitor and assess the performance of two widely different types of structures. The Liddell cooling pond dam is a major structure which has been designed and built to high standards. Failure of this structure would have far-reaching consequences and all precautions must be taken to ensure that its future operation is satisfactory. The Tallawarra ash dam on the other hand is a relatively minor structure which has deliberately been built to a low factor of safety. This has been done having regard for the purpose of the structure (which ponds ash and not water against the weak section of the embankment), the minimal consequences of a slip and the real incentives for making best use of the storage provided at the site.

It has been our experience that it is not possible to have standard instrumentation systems and procedures for monitoring the performance of dams. Each structure must be treated differently according to its particular features. Having obtained the field data, the engineer responsible for the safety of the structure has to decide on the criteria he will adopt to assess whether a structure is safe, unsafe, or whether he should defer a decision to implement maintenance works until further performance data is available. The decision is often difficult and must be made having regard for all the performance data available, the large sums of money which may be involved in strengthening a structure and the consequences which would occur if failure took place. The literature is of little value in this work as

there are few fully documented cases of dam or embankment performance immediately prior to failure, and what is available is usually only relevant to the situations described.

In our work, considerable emphasis has been placed on the measurement of seepage flows and the settlements and movements of reference marks because it is considered that the best indication of deteriorating conditions will show up by increased seepage or as accelerating movements. As the criteria are based on changes or trends, it is most important that the measurements be taken accurately so as to ensure that slow changes are not marked by errors in the observations.

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#### Acknowledgements

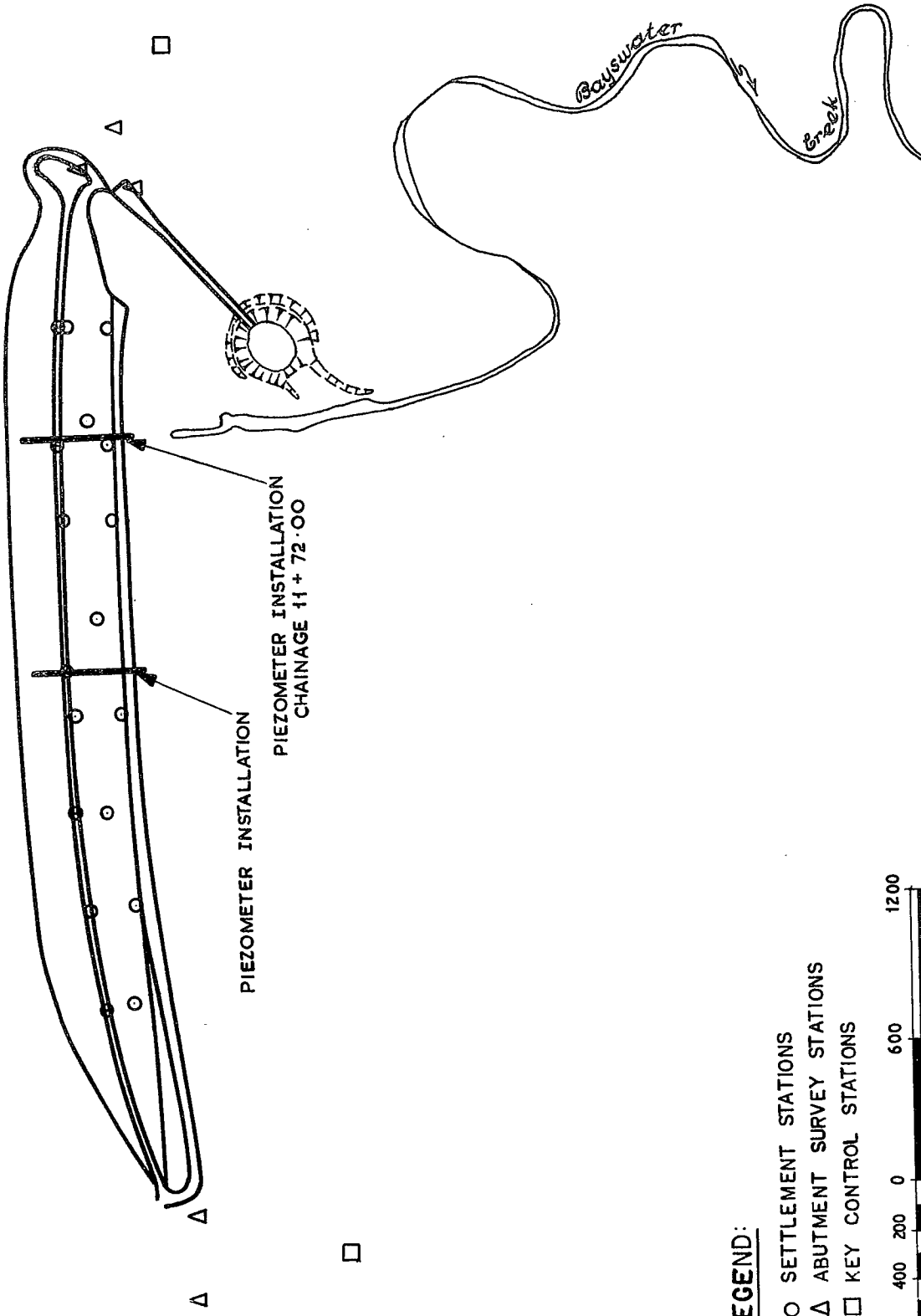
I acknowledge with thanks, the permission of the Electricity Commission of N.S.W. to publish this paper and my colleagues who participate in this work.

TABLE 1A  
ELECTRICITY COMMISSION OF N.S.W. DAMS - SALIENT FEATURES

Dam	Type	When Constructed	Max. Height (Feet)	Crest Length (Feet)	Capacity (Acre Feet)	Catchment (Square Miles)
Bega Hydro Dam	Earthfill	1958	94	1,200	2,500	13.6
Liddell P.S. Cooling Pond Dam	Earthfill	1967/8	145	4,250	120,000	27.0
Liddell P.S. Water Supply Dam	Earthfill	1969	105	1,350	3,700	0.78
Liddell P.S. Ash Dam	Earthfill	1971	108	2,830	11,200	4.0
Munmorah P.S. Ash Dam	Earthfill	1966	20	6,200	4,000	3.0
Tallawarra No. 1 Ash Dam	Earthfill	1953	20	5,000	400	0
Tallawarra No. 2 Ash Dam	Earthfill Staged Construction	1961 1963 1965	12 20 30	10,500	2,400	0
Ulan P.S. Water Supply Dam	Rockfill and Concrete Gravity	1957	34	600	135	45.0
Wallerawang P.S. Ash Dam	Earthfill	1967	30	3,600	270	0.5
Wangi P.S. Ash Dam	Earth and Ashfill Staged Construction	1955 1960 1967	10 20 30	2,500	2,500	1.5
Vales Pt. P.S. Ash Dam	Earthfill	1963	40	3,000	7,000	9.0

TABLE 1B  
ELECTRICITY COMMISSION OF N.S.W. DAMS - INSTRUMENTATION DETAILS

Dam	Piezometers		Ground Water Holes		Settlement Surveys		Horizontal Movement Surveys		Seepage Weirs	
	No.	Read	No.	Read	No. of Stations	Read	No. of Stations	Read	No.	Read
Bega Hydro Dam	11	Monthly	-	-	13	Yearly	13	Yearly	2	Weekly
Liddell P.S. Cooling Pond Dam	68	6-Monthly	29	6-Monthly	4 sets of cross arms & 27	6-Monthly	23	Yearly	2	6-Monthly
Liddell P.S. Water Supply Dam	20	6-Monthly	9	6-Monthly	19	6-Monthly	9	Yearly	1	6-Monthly
Liddell P.S. Ash Dam	27	3-Monthly	13	3-Monthly	32	6-Monthly	32	Yearly	1	3-Monthly
Munmorah P.S. Ash Dam	-	-	19	6-Monthly	24	Yearly	-	-	-	-
Tallawarra No. 1 Ash Dam	-	-	-	-	-	-	-	-	-	-
Tallawarra No. 2 Ash Dam	44	3-Monthly	6	3-Monthly	15	Variable (6-Monthly)	15	Variable (6-Monthly)	-	-
Ulan P.S. Water Supply Dam	-	-	-	-	-	-	-	-	-	-
Wallerawang P.S. Ash Dam	-	-	11	6-Monthly	-	-	-	-	1	6-Monthly
Wangi P.S. Ash Dam	-	-	-	-	-	-	-	-	-	-
Vales Pt. P.S. Ash Dam	16	6-Monthly	-	-	9	Yearly	9	Yearly	-	-



**LEGEND:**

- SETTLEMENT STATIONS
- △ ABUTMENT SURVEY STATIONS
- KEY CONTROL STATIONS

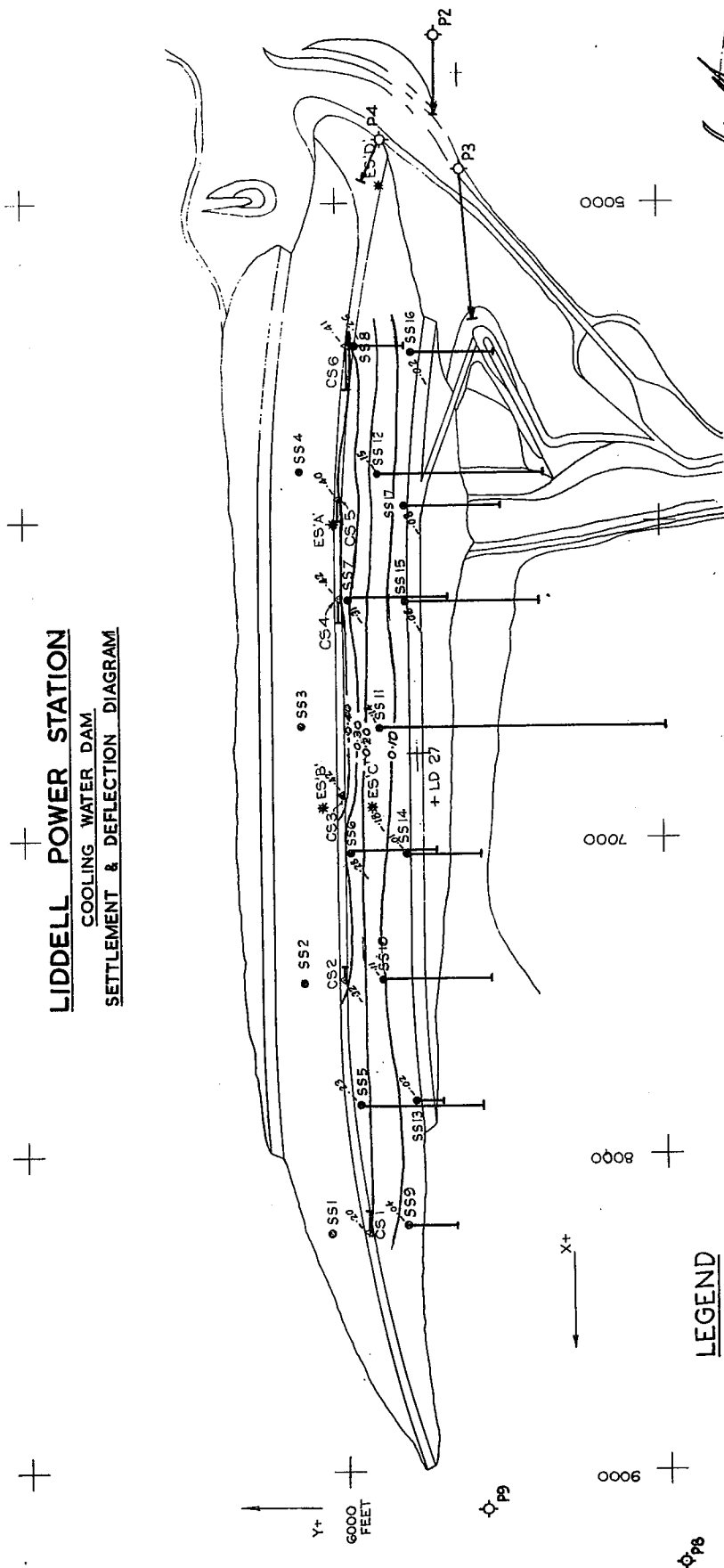


**LIDDELL POWER STATION**  
**COOLING WATER DAM**  
**INSTRUMENTATION**

FIG. 1

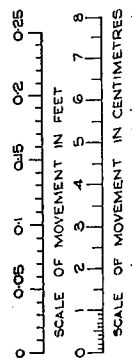
Figure 1.

**LIDDELL POWER STATION  
COOLING WATER DAM  
SETTLEMENT & DEFLECTION DIAGRAM**

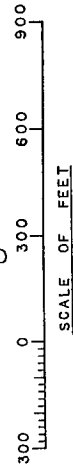


**LEGEND**

- \*ES'B Cap for Electric Settlement Installation
- △CS'6 Crest Settlement Point
- SS'12 Surface Settlement Point



Date of this reading ..... 1-5-72  
 Water level in reservoir ..... ft  
 Initial reading on 19-9-68 .....



**NOTES**

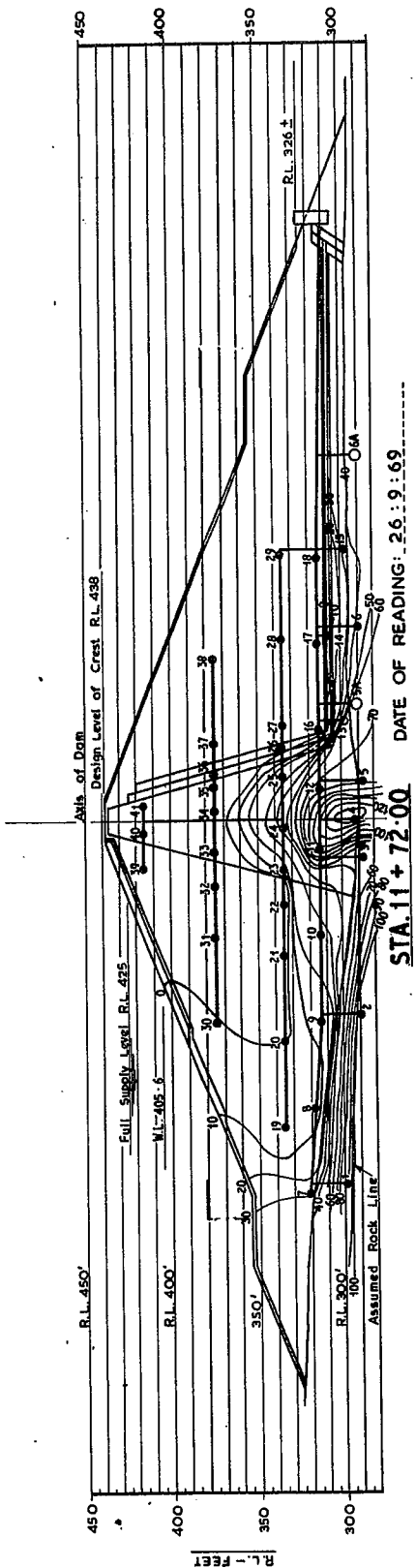
1. Deflections of Concrete Observing Pillars (Shown thus ○) are horizontal vectors.
2. Deflections of Crest Settlement Points (Shown thus △) are components parallel to dam axis.
3. Deflections of Surface Settlement Points (Shown thus ○) are components perpendicular to dam axis.

*[Signature]*  
Principal Surveyor

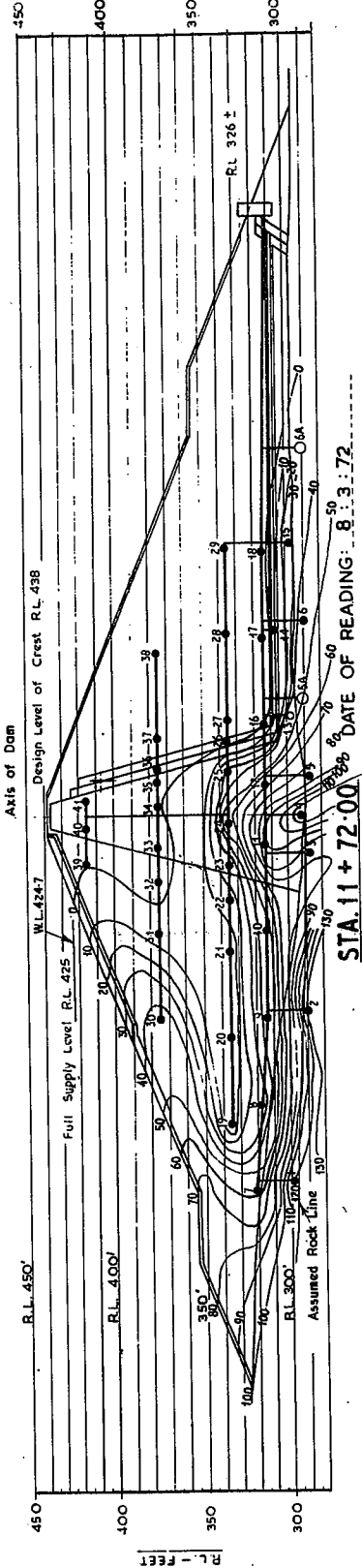
**FIG. 2**

C.I. 5267

Figure 2



STA. 11 + 72.00 DATE OF READING: 2.6.9.69



STA. 11 + 72.00 DATE OF READING: 8.3.72

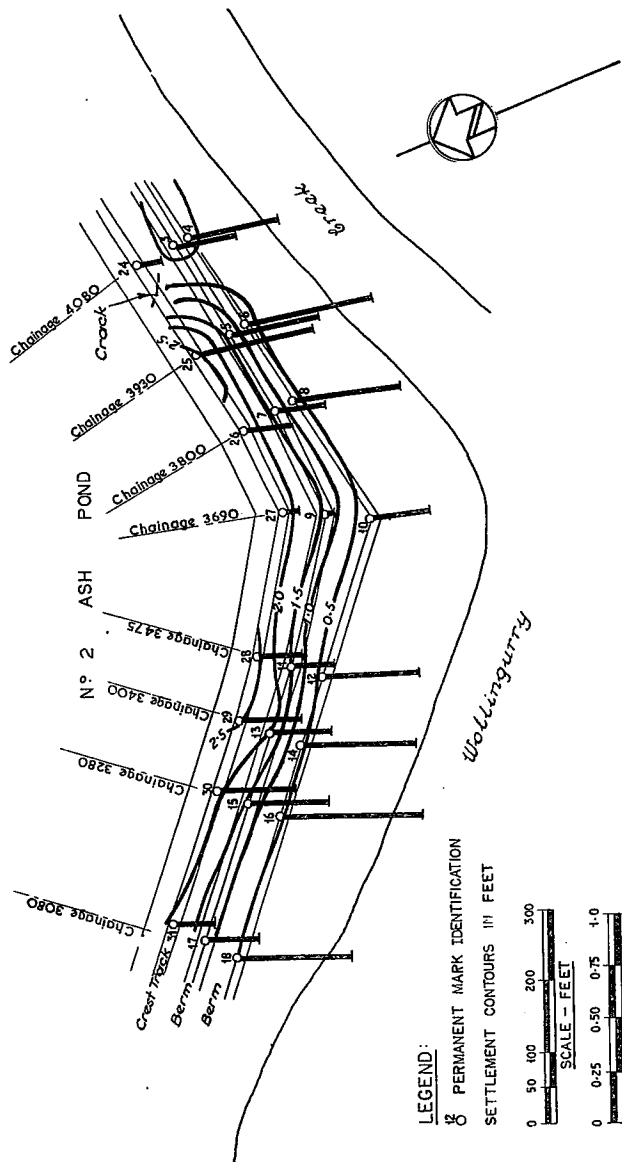
LIDDELL POWER STATION  
COOLING WATER DAM  
PORE PRESSURE DIAGRAMS

FIG. 3

THE ELECTRICITY COMMISSION OF N.S.W. POWER & TRANSMISSION DEVELOPMENT DIVISION			
DRN	H.C.M.	LIDDELL POWER STATION - COOLING WATER DAM	APPROVED
TCD	F.N.G.	PORE PRESSURE DIAGRAM	DATE
CKD		26:9:69 & 8:3:72	
			C.I. 5264
			C.I. 3724





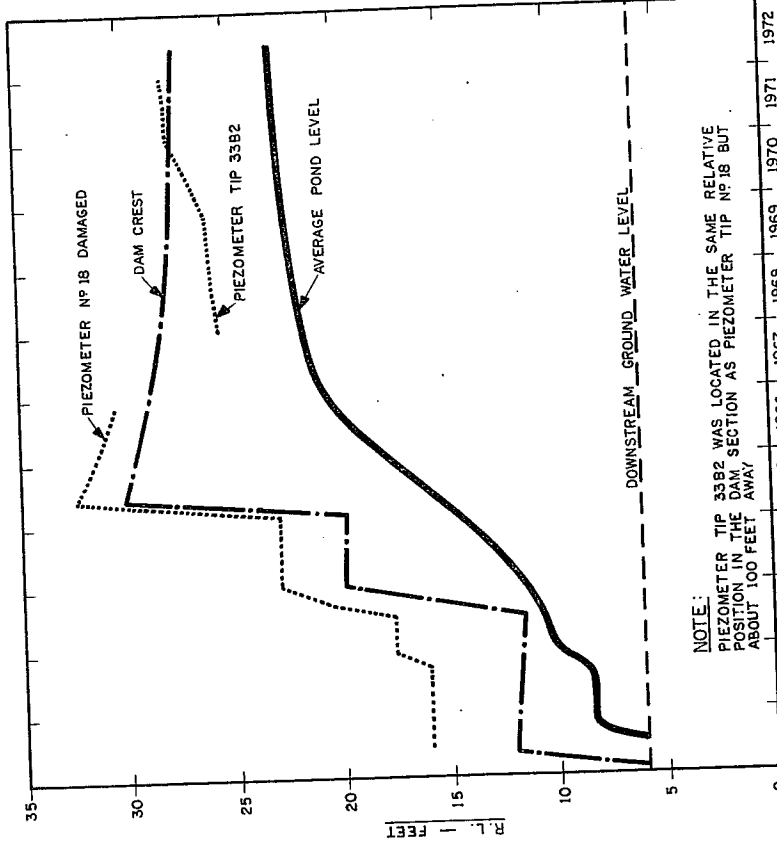


**TALLAWARRA N<sup>o</sup> 2 ASH DAM**  
**TOTAL OBSERVED SETTLEMENTS & HORIZONTAL MOVEMENTS**  
 MAY 1965 TO NOVEMBER 1966

FIG. 6

FIG. 6

THE ELECTRICITY COMMISSION OF N.S.W.		POWER DEVELOPMENT DIVISION		DATE
DRN	C.G.C.	TALLAWARRA P.S. - ASH DISPOSAL DAM N <sup>o</sup> 2	APPROVED	
TCD	F.N.G.	TOTAL OBSERVED SETTLEMENTS & HORIZONTAL MOVEMENTS		C.I. 5269
CKD	N.M.	MAY 1965 TO NOVEMBER 1966		



NOTE:  
PIEZOMETER TIP 3382 WAS LOCATED IN THE SAME RELATIVE POSITION IN THE DAM SECTION AS PIEZOMETER TIP NO 18 BUT ABOUT 100 FEET AWAY

TALLAWARRA NO 2 ASH DAM  
DAM HEIGHT, POND LEVEL & PIEZOMETER READING AT CHAINAGE 3300

FIG. 7

THE ELECTRICITY COMMISSION OF N.S.W. POWER DEVELOPMENT DIVISION			
DRN	C.G.C.	TALLAWARRA P.S. - ASH DISPOSAL DAM NO 2	APPROVED
TCD	F.N.G.	DAM HEIGHT, POND LEVEL & PIEZOMETER READINGS AT CHAINAGE 3300	
CKD		CI. 5266	
			DATE

OMISSION

Mr. A. D. Hosking's paper on "Earth and Rock Dams" was not received in time for printing. An addendum will be issued at the Symposium.

c.f. *AUST. GEOMECH. JOURNAL*  
64 N1, 1974, pp1-12

MONITORING OF FACE DAMS

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Section Engineer, Civil Design Division

The Hydro-Electric Commission

## INTRODUCTION

The term Face Dam is used to mean a fill dam having an impervious membrane on the upstream face. The membrane may be of reinforced concrete, bituminous concrete, timber, steel or any other suitable material.

The overall performance of this type of dam depends almost entirely on the compressibility of the embankment material and the ability of the membrane and its joints to withstand the deformation to which they are subjected. The fill material will normally be non-cohesive and relatively pervious and may be a sandy gravel or, more commonly, a quarried rockfill. If the fill is highly compressible large deflections of the membrane will result leading to large strains in the membrane and the likelihood, in the case of a high dam, of rupture and consequent leakage. This is what occurred at such dams as Dix River (275 ft. - 1925), Salt Springs (328 ft. - 1931), Paradelá (355 ft. - 1958), Wishon (290 ft. - 1958) and Courtright (310 ft. - 1959). Leakage flow at these dams has been in the range 20 to 130 cusecs. All were constructed of quarried rockfill placed by dumping it in high lifts.

In recent years the trend has been towards improving the deformation properties of the fill and this has been achieved by placing rockfill in thin layers and compacting with heavy vibratory rollers.

This paper presents details of various types of instrumentation used to monitor the behaviour of several rolled rockfill face dams recently completed by the Hydro-Electric Commission, Tasmania. Measurements from different types of instrument are compared and the results of some observations are given.

## DATA ON FACE DAMS COMPLETED

The Commission has completed the construction of 5 face dams in the last 3 years. A sixth dam, Scotts Peak, is also very nearly complete, the final phase consisting of the construction of a 12 ft. high wave wall along the 3500 ft. length of crest. The dams are listed in Table 1 below together with other relevant details.

In all cases the membrane was constructed after the rockfill had reached virtual crest level this being done to preclude deformation of the membrane due to construction settlement. The reinforced concrete membranes were constructed by slipforming 40 ft. wide slabs from toe to crest. Horizontal contraction joints were almost entirely eliminated.

Table 1 DETAILS OF FACE DAMS

Name of Dam	Ht (ft)	Lgth (ft)	Vol. of Fill 10 <sup>6</sup> yds <sup>3</sup>	Type of Membrane	Rockfill Details			
					Type of Rock	Zone 2	Zone 3A	Zone 3B
Wilmot	110	450	0.160	r.c.	Greywacke	1'-6"* -9"***	4'-6" -24"	4'-6" -24"
Cethana	360	700	1.800	r.c.	Quartzite	1'-6" -9"	3'-0" -36"	4'-6" -54"
Paloona	130	560	0.180	r.c.	Chert and Argillaceous Chert	1'-6" -9"	2'-0" and -24"	4'-6" -24"
Serpentine	130	430	0.170	r.c.	Quartzite and schist	thin layer on face -6"	3'-0" -36"	3'-0" -36"
Mackenzie	50	3200	0.230	b.c.	Dolerite	thin layer on face -4"	4'-6" -54"	4'-6" -54"
Scotts Peak	150	3500	0.765	b.c.	Argillite	gravel 1'-6" -3"	3'-0" -36"	3'-0" -36"

r.c. = reinforced concrete

b.c. = bituminous concrete

\* layer thickness

\*\* maximum particle size

Information of the design, instrumentation and performance of some of these dams has already been published - ref (1), (2) and (3). Two companion papers on Cethana Dam have been submitted to the 1973 ICOLD Congress in Madrid - ref (4) and (5).

#### INSTRUMENTATION

Measurement is made of:-

- a) vertical settlement within the embankment
- b) deflection in 3 co-ordinate directions of crest and downstream face targets
- c) deflection of the membrane in the vertical, slope and normal directions
- d) movement between the membrane and the plinth in the plane of the membrane and normal to it

- e) opening and closing movement of joints in the slope direction above full supply level
- f) strain in the membrane
- g) leakage

Not all of these measurements have been carried out on each of the dams. The most comprehensive instrumentation system was provided at Cethana Dam as it is a high dam and one of the first of its type and size to be constructed of rolled rockfill. The general arrangement of Cethana instrumentation is shown in fig 1. Instrumentation of the other dams is shown in fig 2 - Wilmot; fig 3 - Paloona; fig 4 - Serpentine; and fig 5 - Scotts Peak. There are no embedded instruments in Mackenzie Dam on which monitoring is minimal and limited to settlement observations on a few crest targets and leakage measurement.

#### METHODS OF MEASUREMENT AND INSTRUMENT DETAILS

##### a) General

The operation of the reservoir behind each of the dams is such that the level is controlled close to full supply level at all times. Measurements on the upstream face therefore are carried out under water.

Where possible 2 different methods of measurement are made to provide a check on the results of observations.

##### b) Measurement of Vertical Settlement

The hydrostatic settlement cell is the instrument used to measure settlement at points within the rockfill.

The conventional vertical cross-arm settlement installation is not particularly suitable in rockfill because of the difficulty of digging back each time a vertical extension is required. Cross-arms were installed in Rowallan Dam (6) without digging back but were unsuccessful due to disturbance from vibratory roller compaction.

The settlement cell used is similar to that developed by Russel (7) and contains 2 independent weirs connected by  $\frac{1}{2}$  in. dia. rigid polyvinyl chloride tubing to a gauge board at the downstream toe. In addition to the 2 levelling tubes there is an air tube to maintain atmospheric pressure in the cell and a fourth tube to drain off excess water. The accuracy of measurement is  $\pm 0.01$  ft.

Conventional survey methods are used to measure settlement and deflection of crest and downstream face targets.

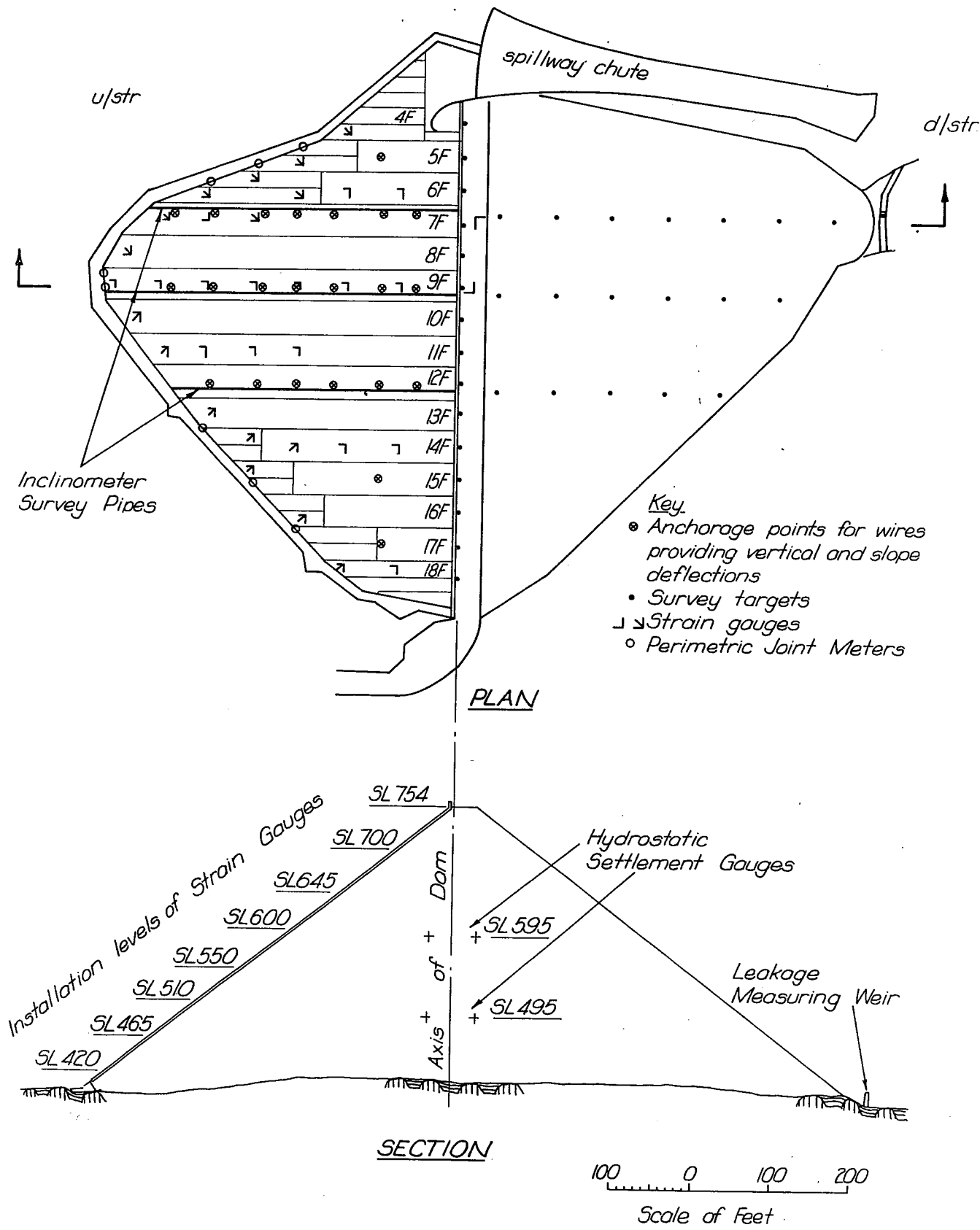
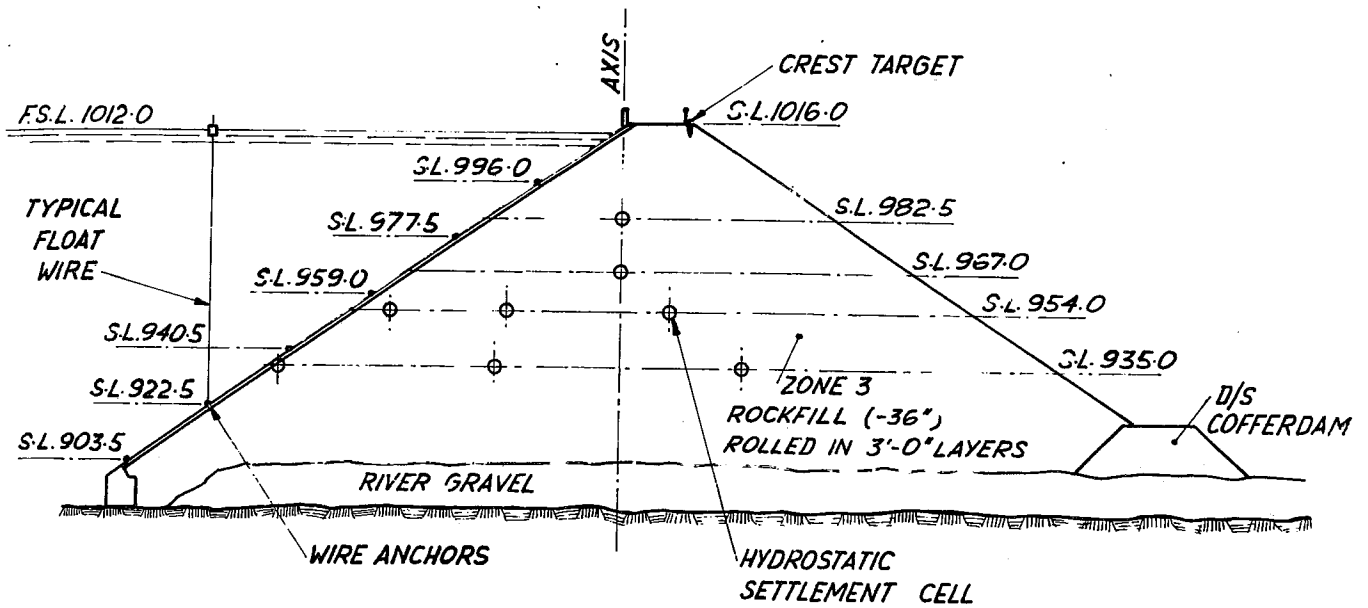
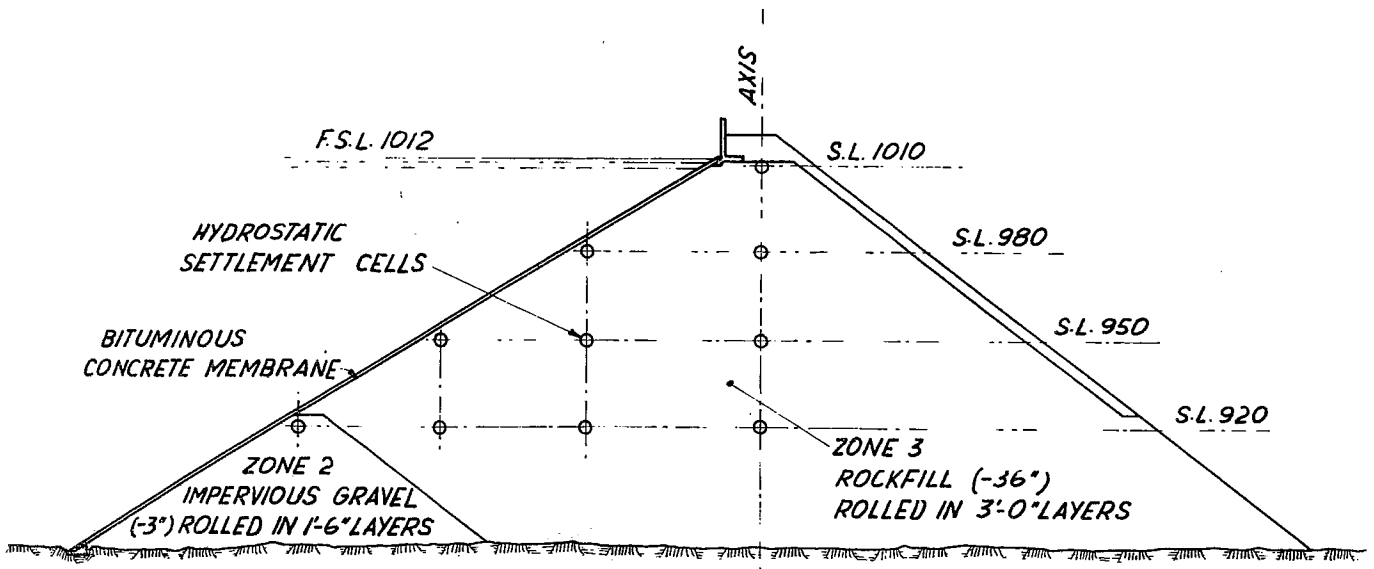
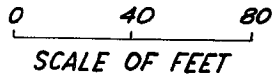


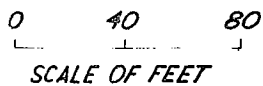
Fig. 1. CETHANA DAM - GENERAL ARRANGEMENT OF INSTRUMENTATION



**FIG. 4 SERPENTINE DAM - INSTRUMENTATION**



**FIG. 5 SCOTTS PEAK DAM - INSTRUMENTATION**



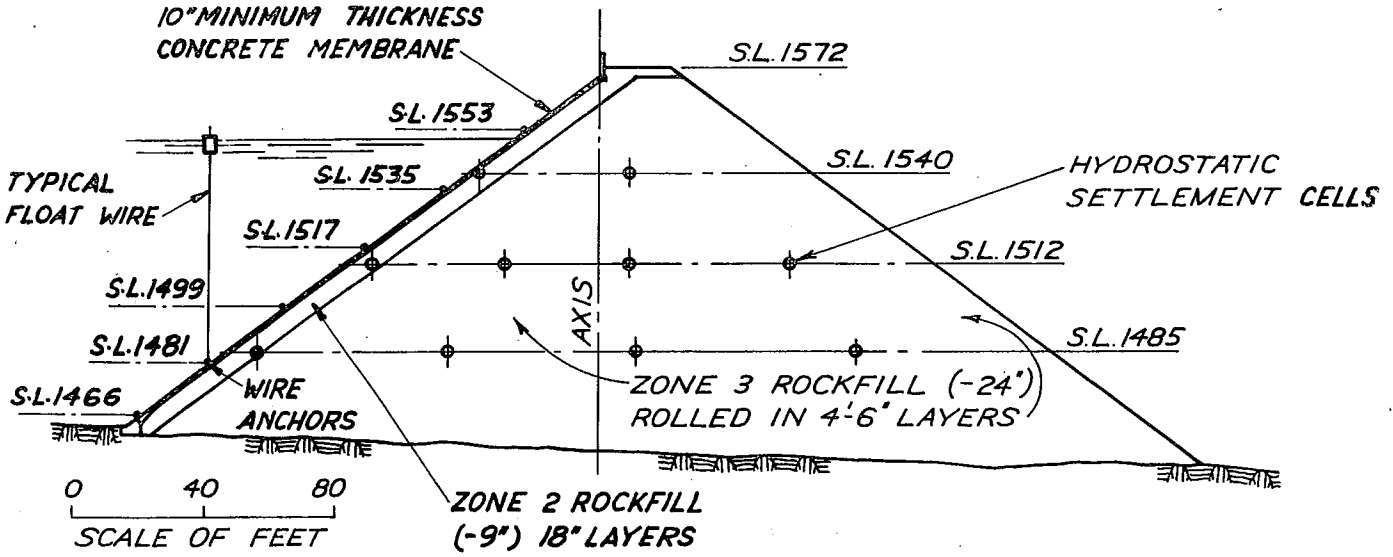


FIG. 2 WILMOT DAM - INSTRUMENTATION

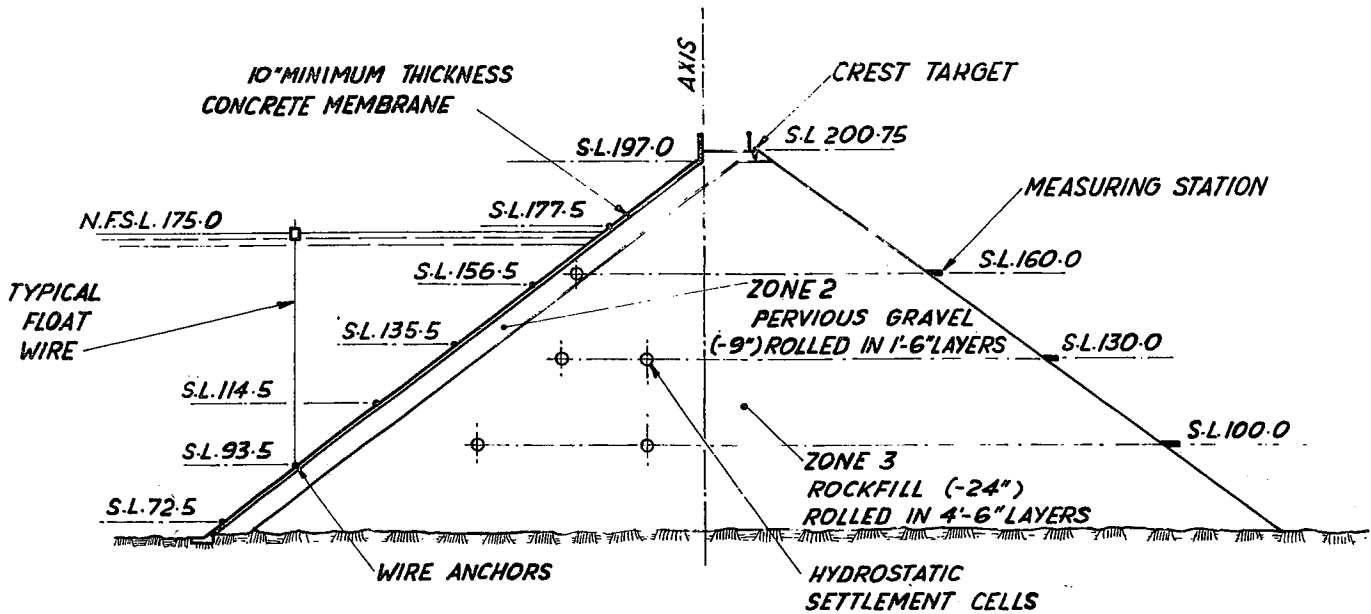
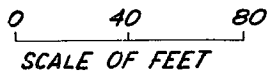


FIG. 3 PALOONA DAM - INSTRUMENTATION



c) Measurement of Membrane Deflection

Above lake level vertical and horizontal deflection is measured by conventional survey methods.

Vertical deflection is measured below water level by what is termed a float wire system and by hydrostatic settlement cells installed in the rockfill just beneath the membrane.

Slope deflection (that is, deflection up or down the slope of the face) is measured by a system of fixed wires from points on the face to the crest. When strain meters are installed the slope deflections are calculated by integration of strain. At the upstream toe the slope deflection of the membrane relative to the plinth is measured by a Carlson type joint meter.

Normal deflection is measured by surveying along the inside of a 3 in. dia. pipe attached to the membrane using a portable inclinometer which is a bore hole survey type of instrument. At the upstream toe the normal deflection of the membrane relative to the plinth is also measured by a joint meter. Instrument details are as follows:-

(i) Float Wire System

At each point where measurement of vertical deflection is required, a stainless steel non-fouling swivel anchor was fixed to the membrane and its elevation determined. A single strand stainless steel wire, 1 mm diameter and graduated with tags at 5 ft. intervals, is attached to the anchor and extended up the slope to the crest. When a measurement is to be made, the wire is detached from the crest and taken by boat to a position above the anchor. A special float carrying a survey staff is then attached to the wire and the immersion of the float is adjusted to obtain a specified tension in the wire. The float brings the wire to a vertical position in the same manner as an inverted plumb bob. The new elevation of the anchor on the membrane is determined by reading the survey staff with a level from the reservoir bank and measuring from the staff reading to the closest wire graduation. This work has to be carried out in very calm conditions. Corrections are made for the effect of tension and temperature on the wire.

(ii) Fixed Wire System

At each point where measurement of slope deflection is required, a single strand stainless steel wire 1 mm diameter is attached to an anchor and extended up the membrane, through nylon guides at 40 ft. spacing, to a fixing on the parapet wall.

The wire passes over a datum consisting of a scribed line on a metal plate fixed to the parapet wall. Measurement of slope deflection is made by tensioning the wire and measuring the distance between a tag graduation attached to the wire and the scribed line. The measurement is corrected for movement of the datum and for the effect of tension and temperature on the wire.

(iii) Inclinometer

The instrument was designed and constructed by Mr. P.A. Watt of the University of Tasmania and is called the Watt Inclinometer. It comprises a multi-wheeled  $2\frac{1}{2}$  in diameter by 5 ft long probe, a power supply and digital display unit - fig 8. The probe contains two force balanced servo - accelerometers and associated electronic equipment. The mounting containing the accelerometers is automatically rotated so that the axis of one of the accelerometers is maintained normal to a vertical plane through the probe. This accelerometer measures the inclination of the probe axis and its output is digitised and telemetered to the display unit at the crest of the dam. The probe is lowered down the pipe by 5 ft long steel rods and readings are taken at rod length intervals.

The instrument accuracy of the inclinometer over the measuring range of  $37^{\circ} \pm 11\frac{1}{2}^{\circ}$  is  $\pm 5$  seconds of arc. The accuracy of a closed survey on a 500 ft long pipe is  $\pm \frac{1}{2}$  in. at the mid point.

The pipe extends from a hinged mounting on the plinth to the crest and is attached to the face by galvanized steel brackets providing full restraint normal to the pipe axis and no restraint in the axial direction. To minimise corrosion, galvanized pipes coated externally with high density polyethylene, and filled with a 0.2% solution of calcium hydroxide giving a pH of 12, were used.

In use the procedure adopted is as follows:-

The inclinometer is traversed down the pipe in a constant orientation and readings taken at 5 ft intervals. At the bottom it is turned through  $180^{\circ}$  and traversed up the pipe again with a constant orientation and readings at 5 ft intervals.

The two readings for each 5 ft interval are averaged and the change in inclination from the datum position determined.

The normal deflection at the crest is found from conventional survey measurements and the pipe hinge point on the plinth is assumed to be fixed.

Uncorrected normal deflections and the misclose at the hinge point are calculated by summation of the angular change times the distance traversed (5 ft). The adopted deflections are obtained by a linear correction for the misclose.

(iv) Joint Meters

The instrument used to measure deflection of the membrane relative to the plinth is a 360 mm long Carlson type joint meter (Kyowa CJ-40G). The meter is attached to an anchor on the membrane and is connected to an anchor on the plinth by a 2 ft long by  $\frac{1}{4}$  in. dia. stainless steel rod. This arrangement, shown in fig 6, was designed to avoid shear on the meter due to movement normal to its axis.

d) Measurement of Strain and Temperature in the Membrane

The strain meters were designed and manufactured by the Hydro-Electric Commission and consist of a 5 mm range Carlson type joint meter (Kyowa Model CJ-5G), an extension rod, a thin walled steel tube 2 in. dia., and two end flanges  $2\frac{3}{4}$  in. dia. The 260 mm long joint meter is fixed to one end flange and is connected to the other end flange by the extension rod which is held centrally in the tube by blocks of polyurethane foam. The tube is filled with grease. Each strain meter was adjusted to half the total extension of the joint meter so that equal tensile and compressive strains could be measured. The long gauge length was chosen to minimise any local effects caused by reinforcement or tensile cracking.

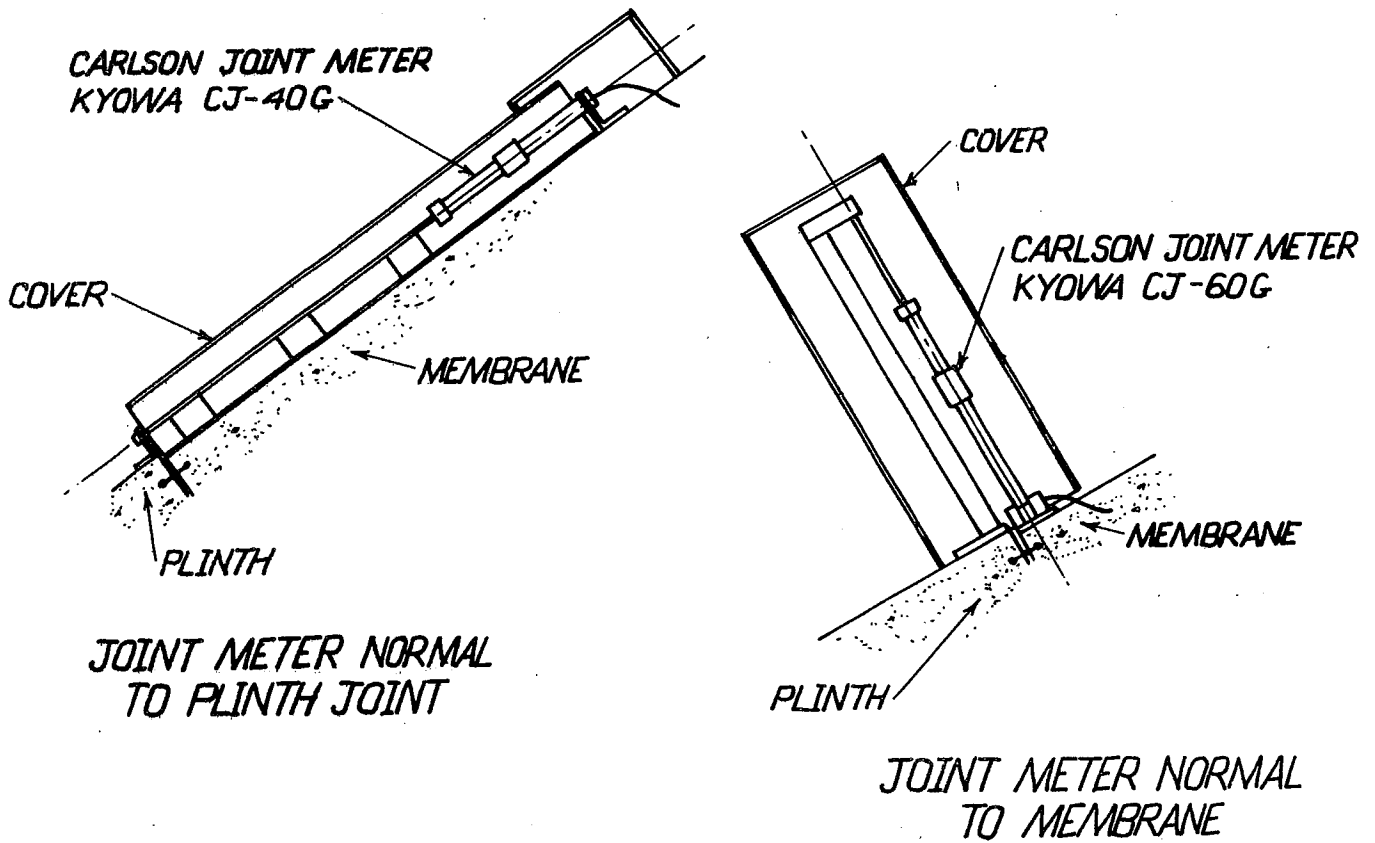
The strain meters are installed in  $45^\circ$  and  $90^\circ$  rosettes in the centre of the membrane and mid-way between contraction joints in the slope direction. All electrical conductors are embedded in the membrane and routed to the terminal installation in the crest parapet.

In addition to providing strain readings the meters also provide temperature readings.

RESULTS OF MEASUREMENTS

a) Crest Settlement with Time

Crest settlement since the end of construction of 7 dams is shown in fig 7. Settlement is expressed as a percentage of the height of the dam. Two central core dams, Rowallan and Parangana, have been included because they are representative of rolled fill construction.



**FIG 6 - JOINT METER INSTALLATION**

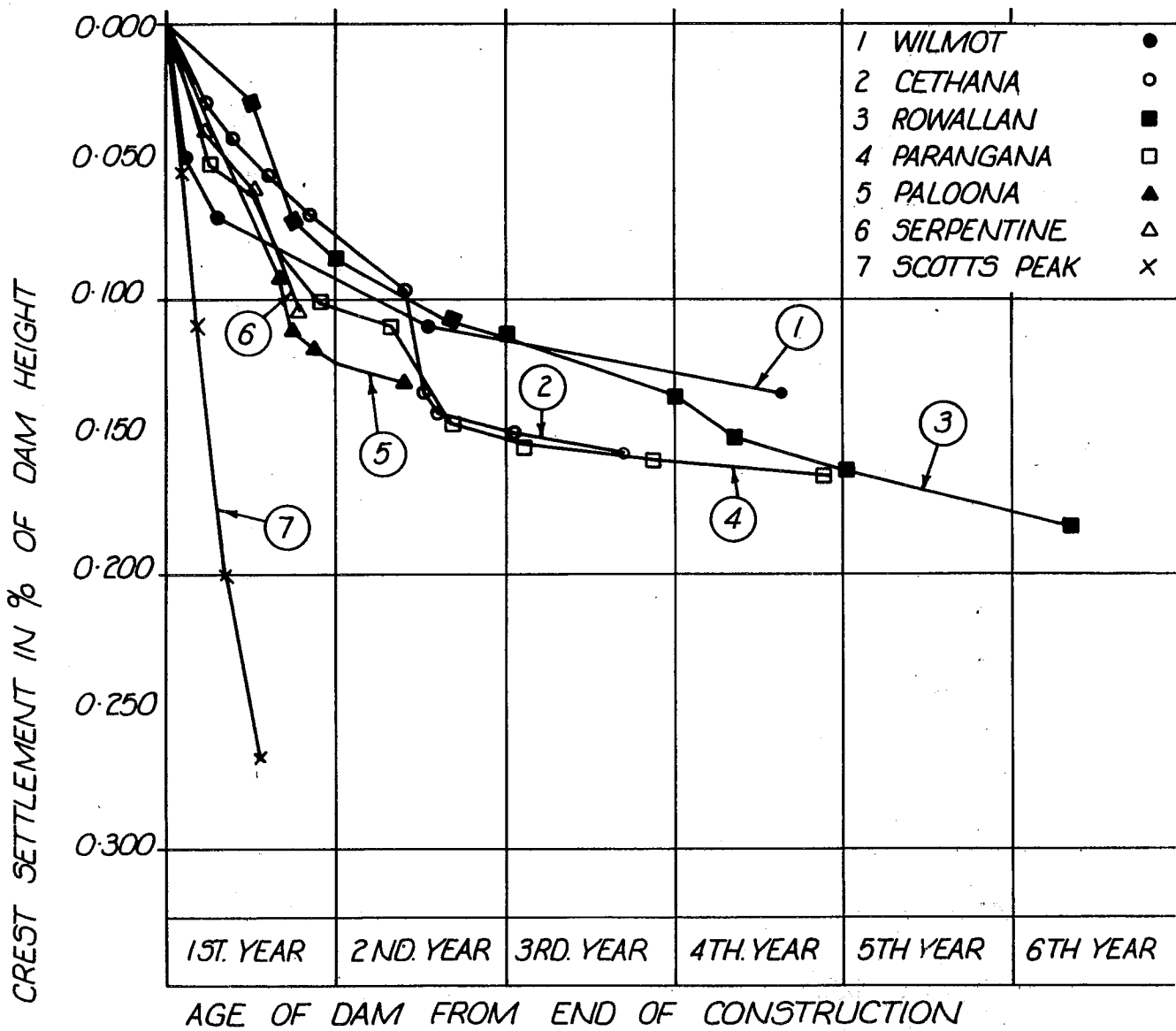
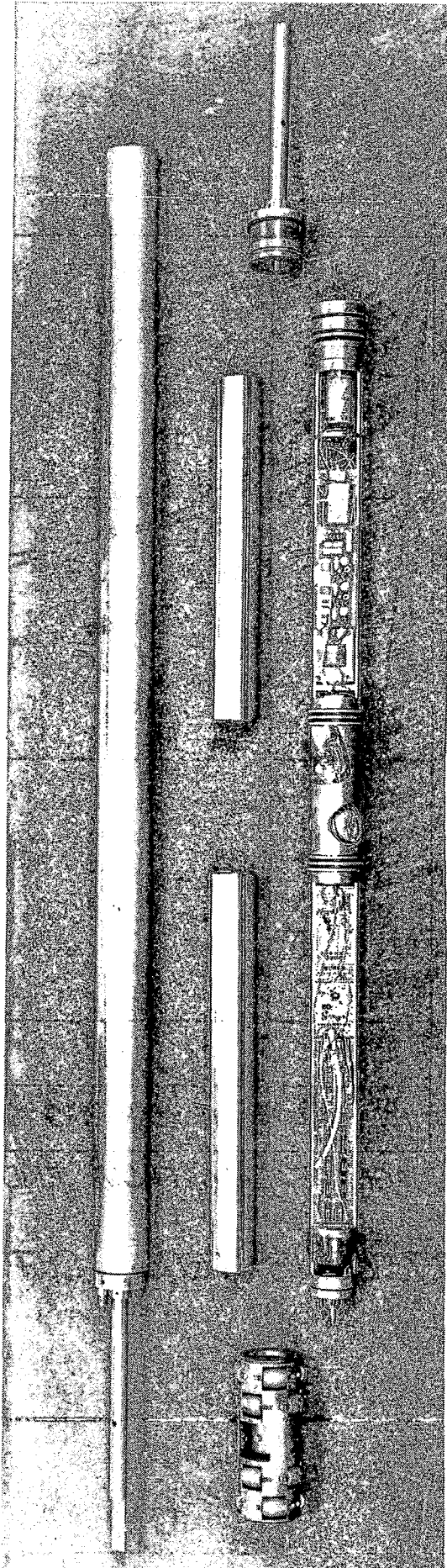
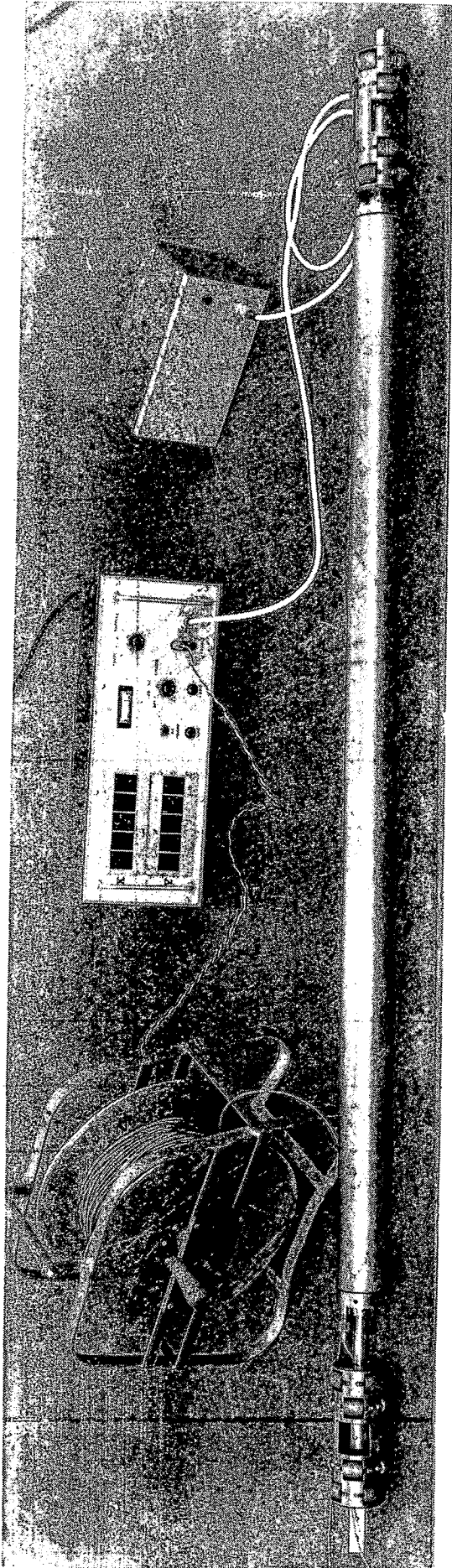


FIG.7- LONG TERM SETTLEMENT



above: - probe, cable drum, digital display unit and power supply  
below: - probe disassembled

FIG 8 - WATT INCLINOMETER

The cores of both dams are composed of non-plastic material - a well graded glacial till in the core of Rowallan and a decomposed granodiorite at Parangana - and the rockfill zones were placed and compacted in layers.

As the results are from the end of construction the reservoir filling period is included. In the case of Cethana this took place midway through the 2nd year and is the reason for the increased rate measured at that time.

It is too early yet to obtain any indication of the rate of long term settlement at Serpentine and Scotts Peak. The indicated rate of long term settlement for the other 5 dams varies from 0.006% per year for Parangana to 0.015% per year for Rowallan. This is to be compared with the long term settlement of dumped rockfill which has been measured at Dix River and Salt Spring Dams for over 30 years and has averaged 0.033% per year - ref (7). In addition the primary settlement occurring in the first year is only about one quarter of that for dumped rockfill. The rather larger settlement at Scotts Peak is due to the relatively weak argillite used as rockfill.

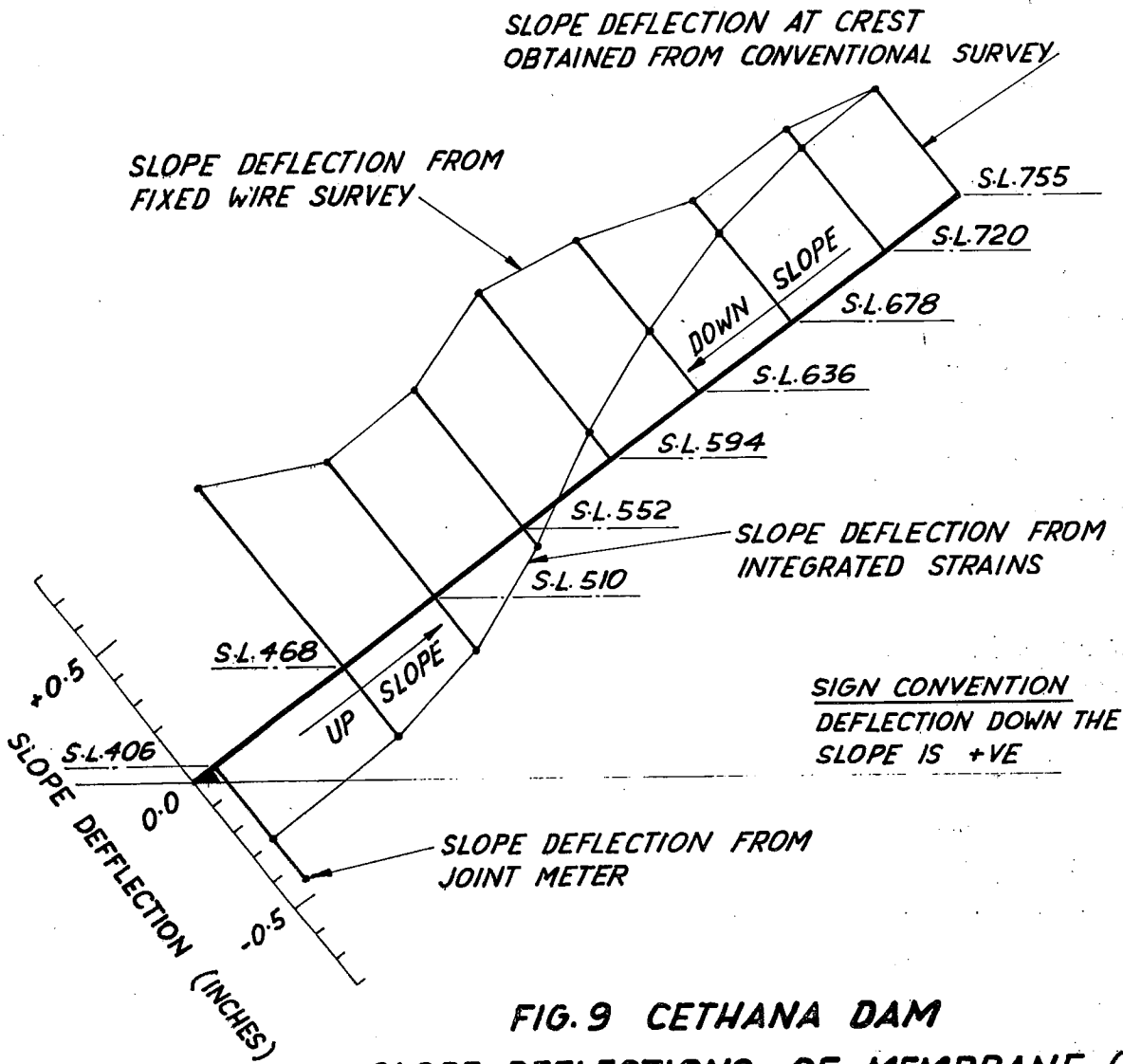
## b) Membrane Deflections

### (i) Cethana Dam

The slope deflections at Cethana are measured by conventional survey at the crest, by fixed wire survey and integrated strains for points on the membrane and by joint meters at the toe. The results for 3 slabs are shown in fig 9 where it may be seen that the wire survey and integrated strain methods are not even in approximate agreement.

There is reasonable agreement, however, between the total shortening of the slab from strain integration and the total change in length as measured at the crest and toe of the slab. For this reason it is considered that the fixed wire survey is the more likely to be in error.

The vertical deflections at Cethana are measured by conventional survey at the crest and by float wire survey for points on the membrane. Where the slope deflection is small compared with the vertical deflection, as is the case at Cethana, the normal deflection is obtained with quite acceptable accuracy from the vertical deflection by neglecting the slope deflection. The normal deflections calculated from the float wire survey are shown in fig 10.



**FIG. 9 CETHANA DAM  
SLOPE DEFLECTIONS OF MEMBRANE (SLAB 9F)  
(8-12-71)**

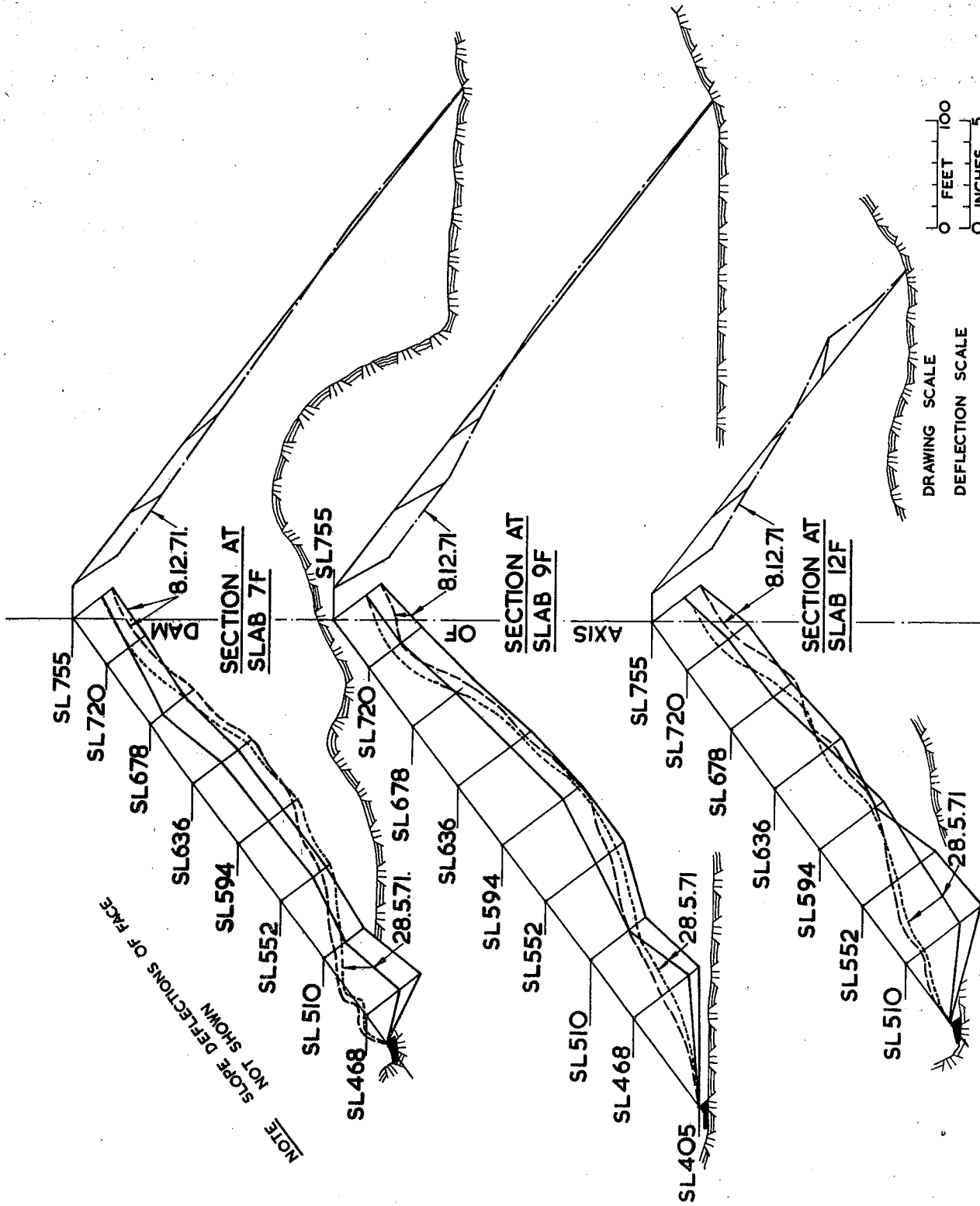


FIG. 10 NORMAL DEFLECTIONS OF UPSTREAM FACE AND DEFLECTIONS OF DOWNSTREAM FACE

Also shown in fig 10 are the normal deflections obtained from the inclinometer survey. While in reasonable agreement over the upper two thirds of the membrane there is considerable disagreement in the lower one-third. The float wire survey shows the greater deflections in the lower region.

The maximum normal deflection at the centre of Slab 9F is about 4.5 in. by both methods.

(ii) Wilmot

Slope deflections from the fixed wire survey are all less than  $\frac{1}{8}$  in. and have been neglected in calculating the normal deflections.

Vertical deflections are measured by float wire survey and by hydrostatic settlement cells. The normal deflections calculated from the two sets of vertical deflection measurements are shown in fig 11.

There is quite a significant difference in the results from the two methods, the wire survey indicating the larger deflections, with a maximum of less than 1 inch. The hydrostatic cells are not located directly on the underside of the membrane but within the rockfill at a depth of 3 or 4 ft from the face. Some of the difference is therefore due to the compression of this small depth of fill but it is considered likely that this could account for only a small fraction of the difference.

(iii) Paloona

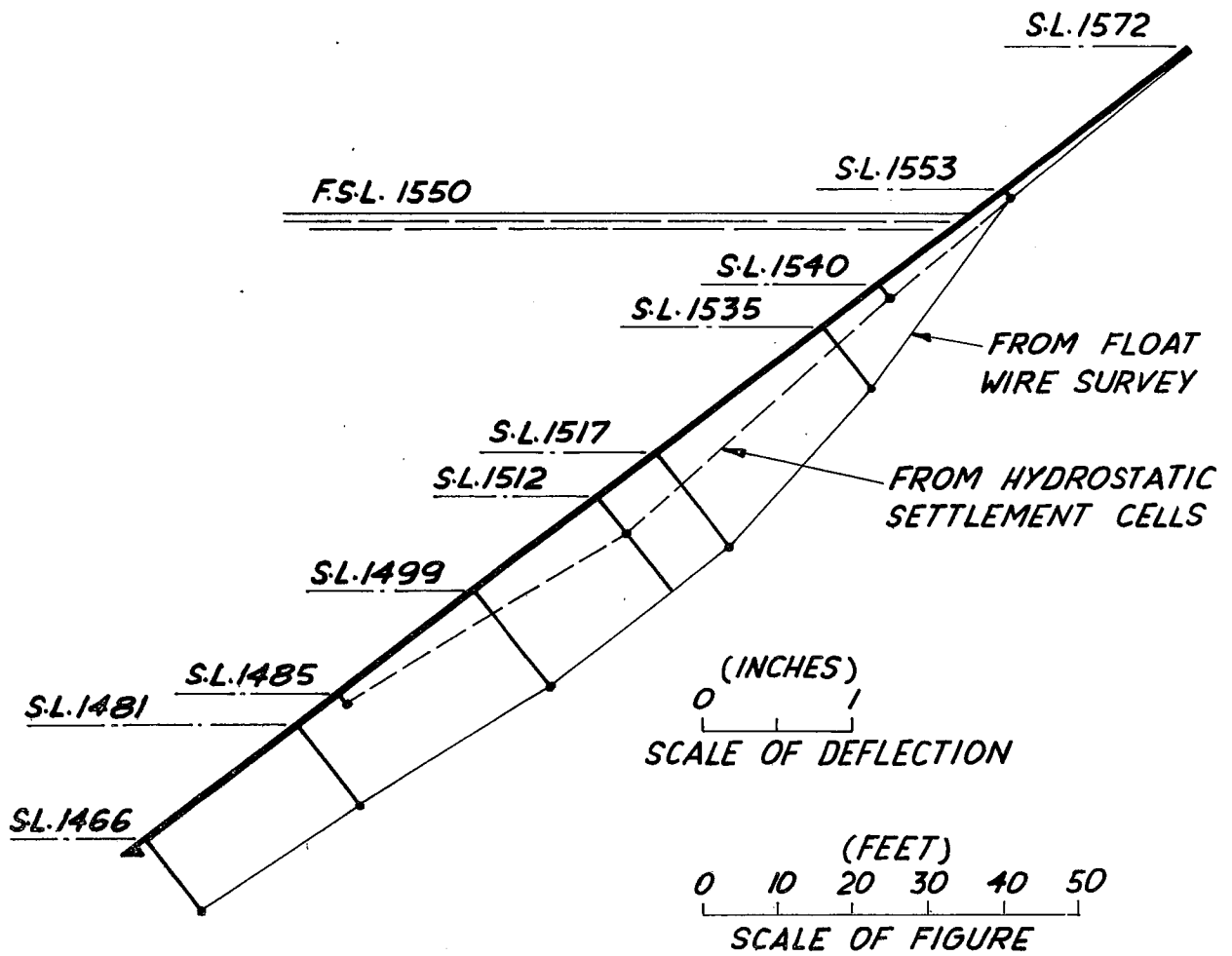
Fig 12a shows the normal deflections of the central section of the membrane as calculated from the float wire survey. The one hydrostatic cell very close to the underside of the membrane at SL 160 is in good agreement. However, at the toe the wire surveys give quite a different result from that measured by the joint meters - see fig 12b.

(iv) Comments on Methods of Deflection Measurement

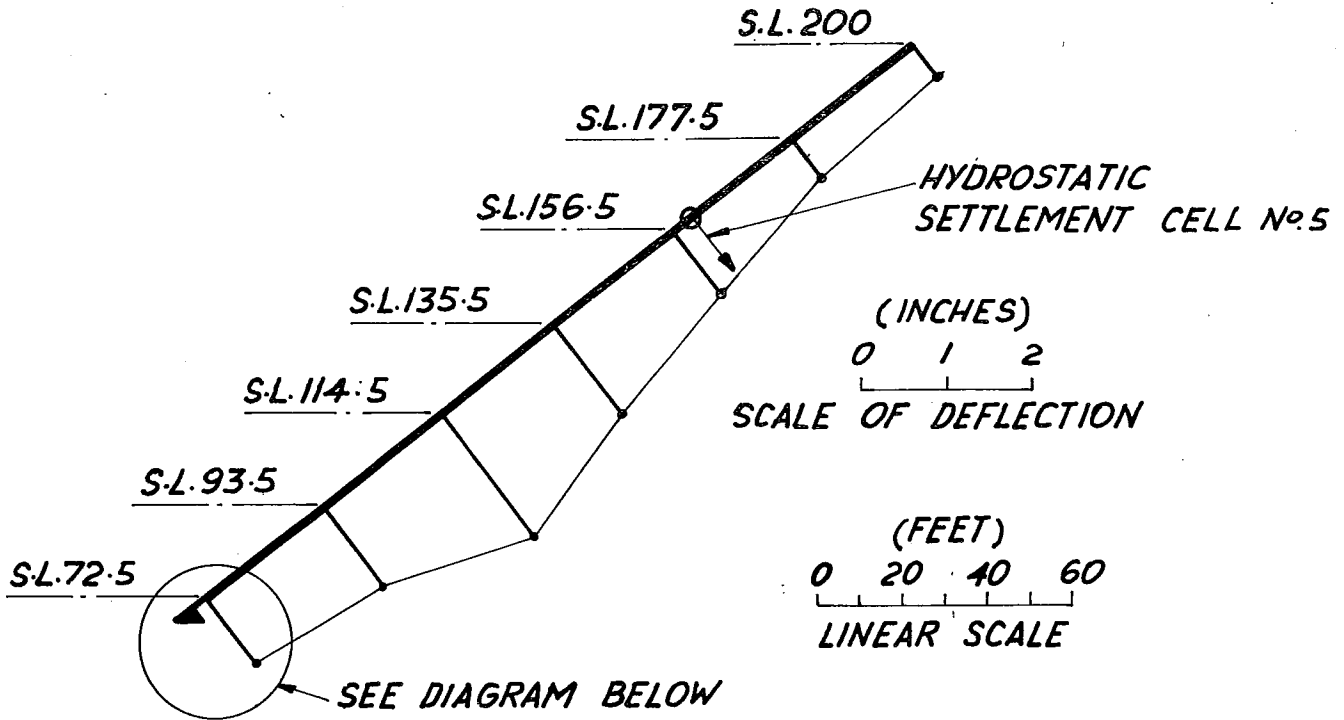
The results presented above show that the deflections obtained from the float and fixed wire systems are generally greater than those obtained by the other methods.

There is not a great deal of evidence on which to base a judgement but it is considered by the author that the wire measurements are likely to be the less reliable. This view is based mainly on the proven reliability, both in the laboratory and in the field, of hydrostatic settlement cells and electrical resistance joint meters.

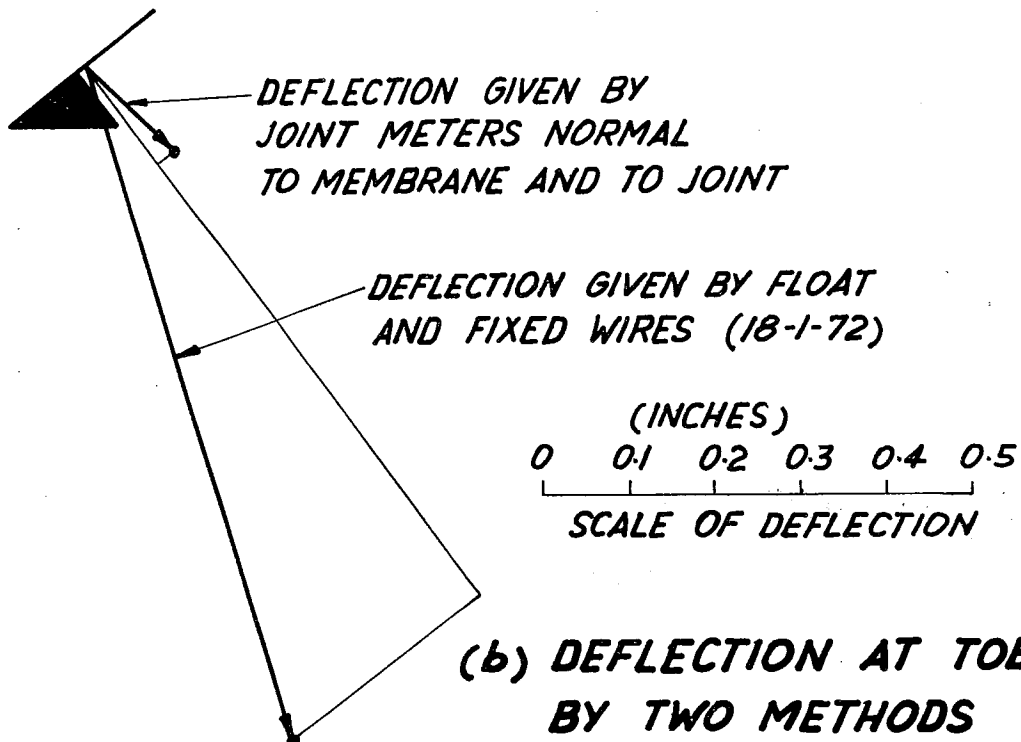
If the wire measurements are in error the reasons for this are not easy to ascertain or determine and as yet no answer has been found. The system is a simple one but does rely on corrections for temperature and tension for accuracy.



**FIG.II WILMOT DAM - NORMAL MEMBRANE DEFLECTION  
(24-4-70)**



**(a) NORMAL DEFLECTION OF MEMBRANE FROM FLOAT WIRES (18-1-72)**



**(b) DEFLECTION AT TOE BY TWO METHODS**

**FIG.12 PALOONA DAM - MEMBRANE DEFLECTION (18-1-72)**

In order for the corrections to be accurate the coefficient of linear expansion and Young's modulus must be uniform and the changes in temperature and tension must be measured with little error.

With regard to the fixed wires it is thought that the assessment of temperature distribution along the wire, firstly in air at the time of datum measurements and subsequently in water and air at the time of each survey, has not been sufficiently accurate to calculate the true temperature correction which is equal to or larger than the measurement itself. In retrospect this could have been overcome by having two wires of different coefficients of linear expansion attached to each point and using the two readings to calculate the temperature change.

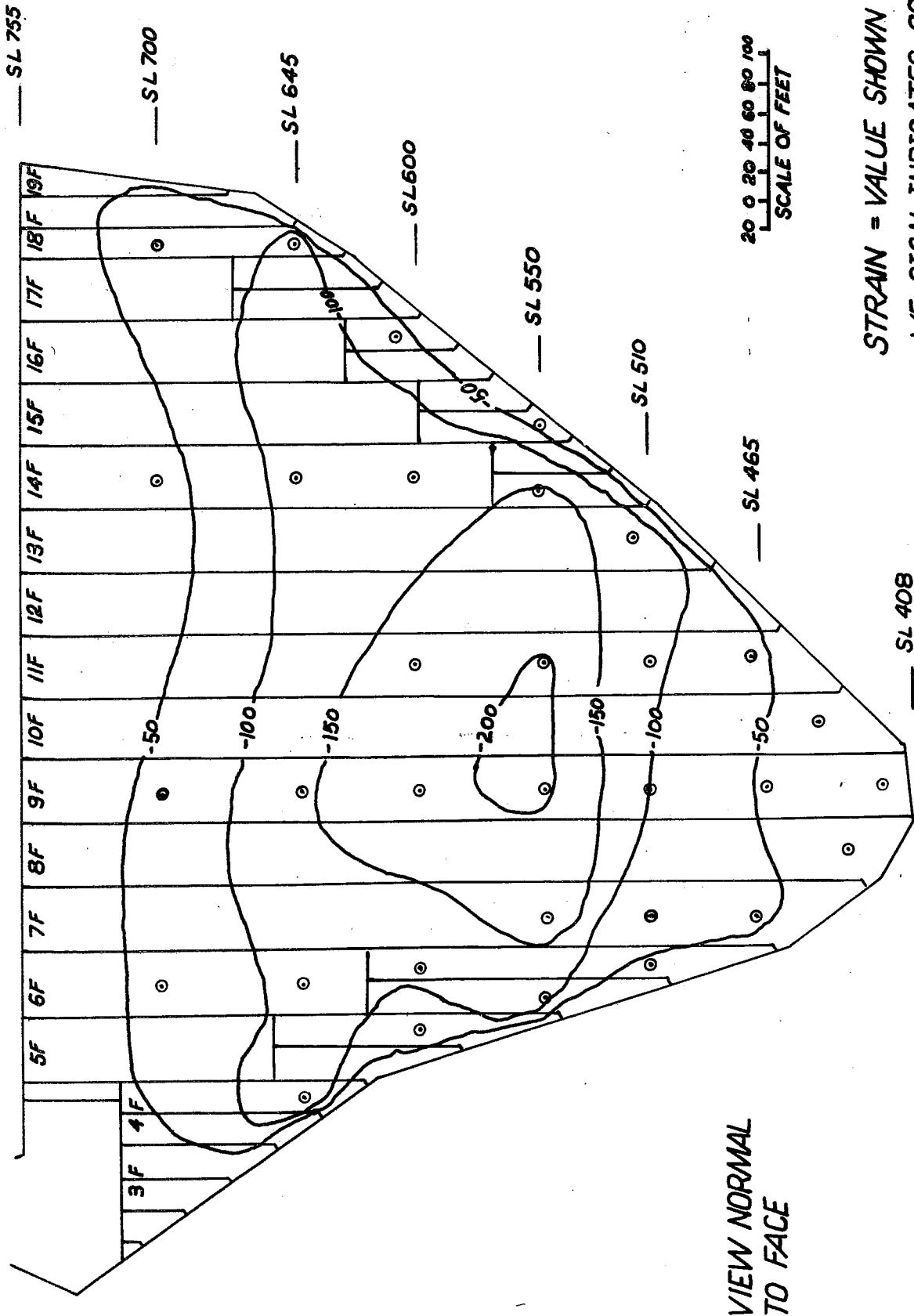
For the float wire measurements the temperature corrections are small compared with the deflections measured so that a small error in the correction could not account for the apparent discrepancies. That the wire is vertical when the measurement is made has been checked by triangulation on the float which was found to be well within any significant radius of error. A remote possibility is that the non-fouling swivel correction to the membrane may not be fool proof. What has not been done, and in hindsight should have been done, is a proper check on the validity of the method by anchoring a wire onto the plinth which could be considered to have negligible movement.

#### STRAIN AND STRESS IN CETHANA MEMBRANE

Cethana Dam is the only one in which membrane strains have been measured. The patterns of strain and stress developed in the membrane between the start of reservoir filling, 4th February 1971, and the 8th December 1971 are shown in figs 13, 14, 15 and 16.

The pattern of strain in the slope direction, fig 13, is reasonably symmetrical about slabs 9F and 10F in the centre of the membrane. Over a fairly large area the strain is greater than  $150 \times 10^{-6}$  units and in a much smaller region within this central area it is a little greater than  $200 \times 10^{-6}$  units.

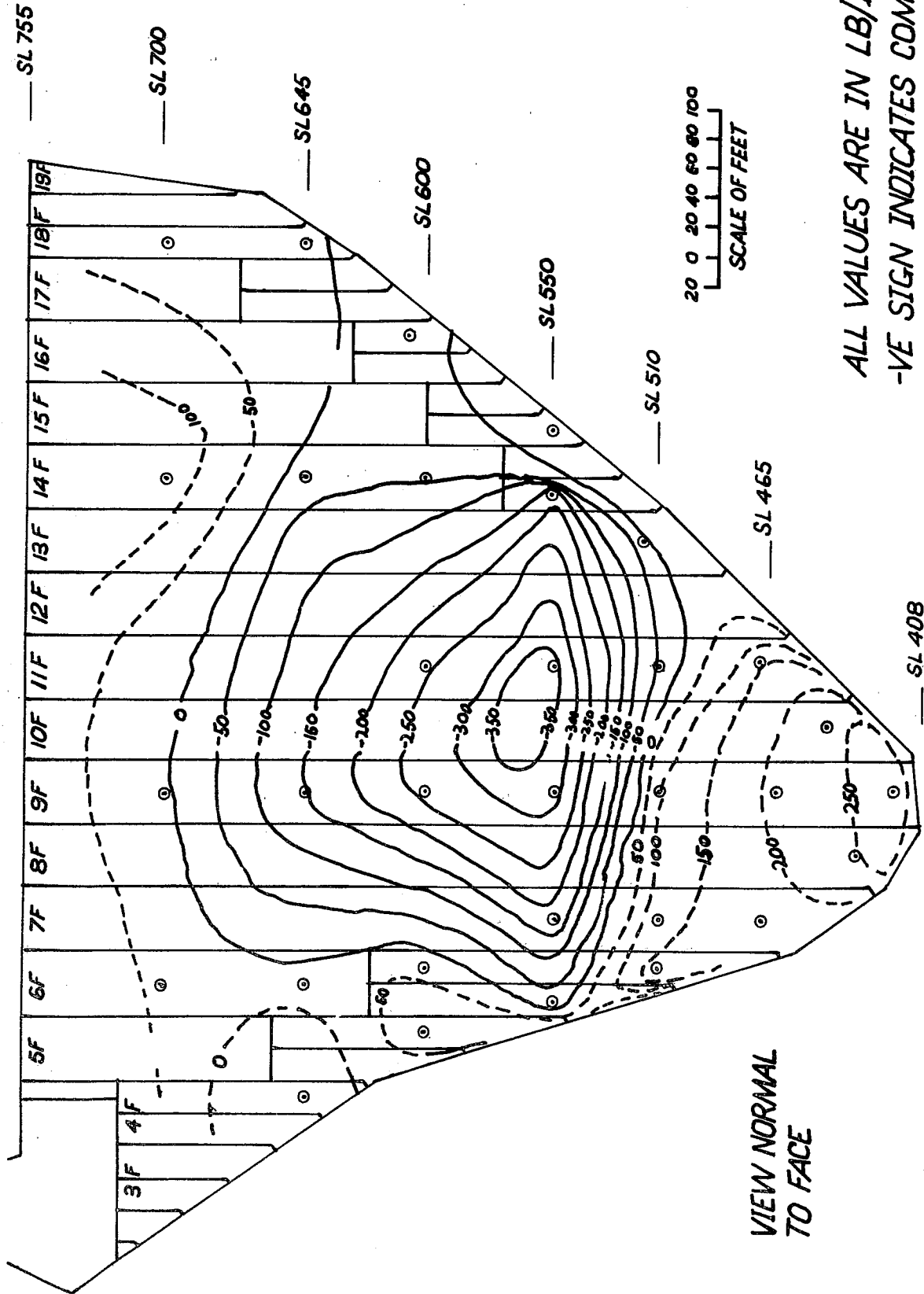
Stress in the slope direction, fig 14, reflects the strain pattern with maximum stress in the central region being about  $350 \text{ lb/in}^2$ . Tensile stresses exist in the lower one third of the membrane, in a narrow region up each abutment and across the top portion of the membrane. The maximum tension is  $250 \text{ lb/in}^2$  near the toe in slab 9F.



STRAIN = VALUE SHOWN  $\times 10^{-6}$

-VE SIGN INDICATES COMPRESSION

FIG 13. CETHANA DAM  
CONTOURS OF STRAIN IN MEMBRANE  
IN SLOPE DIRECTION. (4-2-71 to 8-12-71)



ALL VALUES ARE IN LB/IN<sup>2</sup>  
-VE SIGN INDICATES COMPRESSION

FIG 14. CETHANA DAM  
CONTOURS OF STRESS IN MEMBRANE  
IN SLOPE DIRECTION. (4-2-71 to 8-12-71)

VIEW NORMAL  
TO FACE

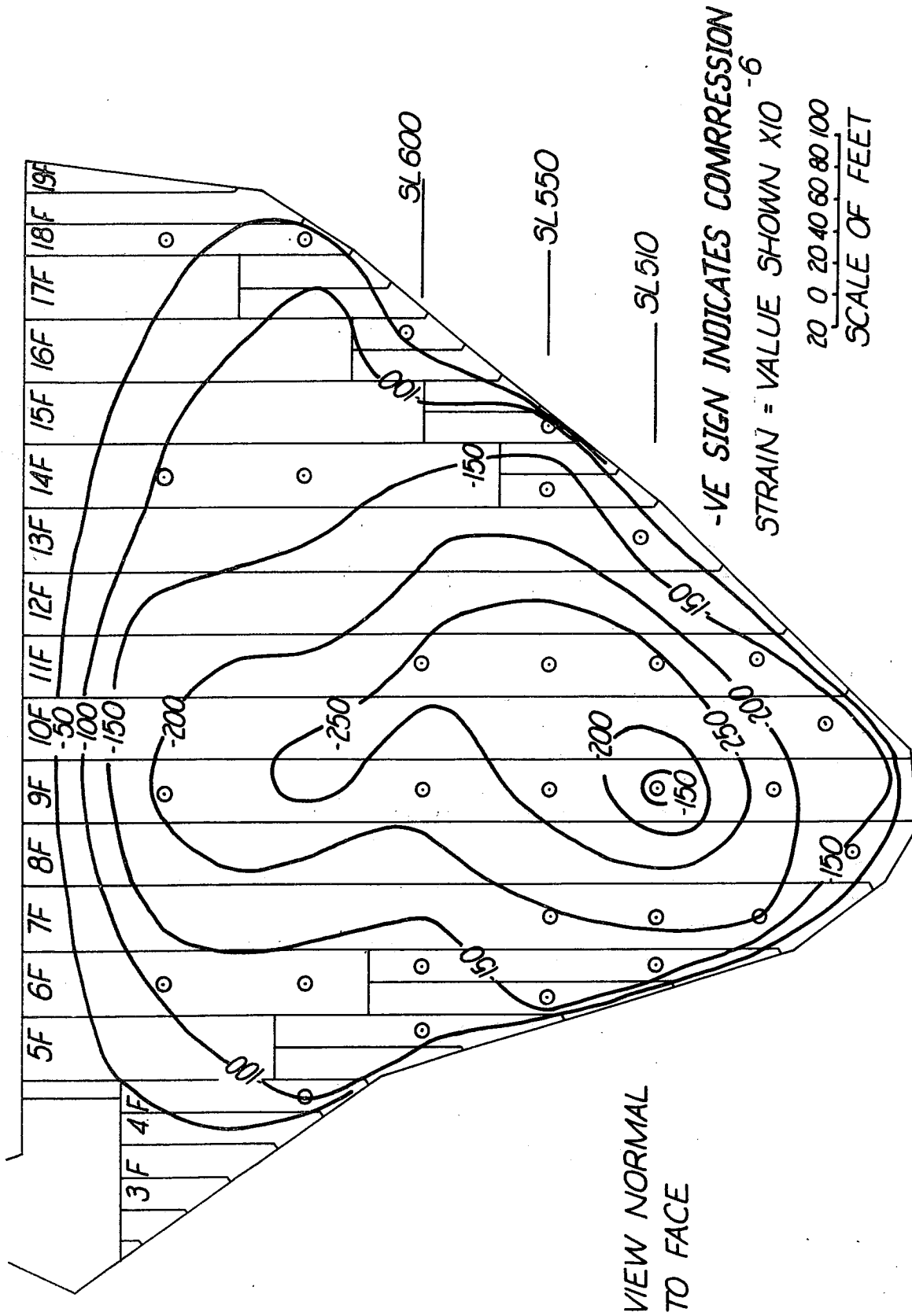


FIG.15 CETHANA DAM - CONTOURS OF STRAIN IN MEMBRANE IN HORIZONTAL DIRECTION

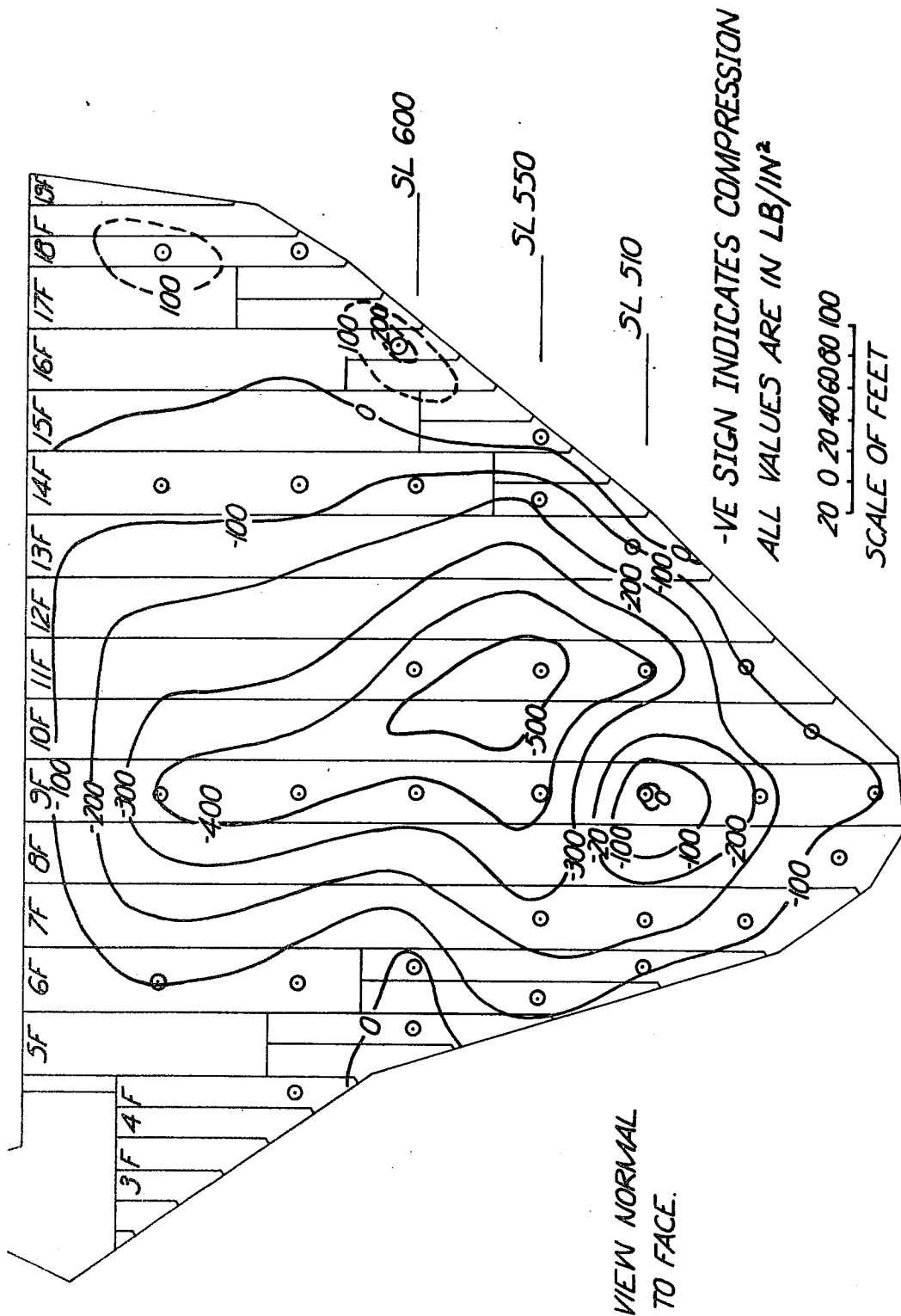


FIG 16. CETHANA DAM - CONTOURS OF STRESS IN MEMBRANE  
IN HORIZONTAL DIRECTION  
(4.2.71-8.12.71)

Stresses were calculated using values of the elastic constants of  $3 \times 10^6$  lb/in<sup>2</sup> for Young's Modulus and 0.2 for Poisson Ratio.

Strain in the horizontal direction, fig 15, appears to reflect the influence of the boundary shape of the left abutment. About one third of the way up slab 9F there is a local depression in the strain pattern where the magnitude drops to  $150 \times 10^{-6}$  units in an area which is generally above  $250 \times 10^{-6}$  units. This appears a little anomalous.

The corresponding stress pattern in the horizontal direction, fig 16, shows this anomalous looking result more clearly. Maximum compressive stress is 500 lb/in<sup>2</sup> in the central region.

The overall pattern of stress and strain is indeed very satisfactory and reflects the uniformity of loading and support of the membrane. The maximum compressions of 350 lb/in<sup>2</sup> (slope direction) and 500 lb/in<sup>2</sup> (horizontal) are very moderate.

It may be shown that under the action of the normal water load the friction on the underside of the membrane is quite sufficient to provide full restraint against membrane movement relative to the rockfill except close to a free boundary. Conversely, the strain in the membrane will be that imposed on it by the movement of the rockfill. As the strains developed in the Cethana membrane due to water load are all compressive there is little doubt that the tensile stresses are due to the temperature drop of 20 to 25°F that occurred during and after filling.

#### LEAKAGE

The discharge measured at the downstream toe of a face dam includes seepage through the membrane and joints in the membrane, seepage through the foundation rock beneath the plinth, and local run-off from the dam and the abutments between the dam crest and the measuring weir.

At Cethana the discharge increased as the reservoir level rose during the initial filling. After the reservoir level reached full supply level in April, 1971 the weir discharge remained reasonably constant at 1.7 cusecs throughout the winter. Fluctuations above this occurred during periods of high rainfall. After March, 1972 the discharge reduced slowly to just under one cusec at which it has remained constant for several months.

The leakage at Paloona Dam is 0.1 cusecs. No figure is available for Wilmot as the measuring weir was covered with eroded bed material during spillway discharge which occurred straight after filling.

19F  
18F  
17F  
16F  
15F  
14F  
13F  
12F  
11F  
10F  
9F  
8F  
7F  
6F  
5F

CONCLUSIONS

Monitoring of several face dams constructed of rolled rockfill has been carried out to check their performance and to obtain data on which studies of behaviour could be based.

That this type of dam is highly successful has been shown by the small magnitude of the deflections and the leakage.

The importance of duplicating the measurements where possible with different types of instrumentation has also been demonstrated.

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