IN THE MATTER OF
THE QUEENSLAND FLOODS COMMISSION OF INQUIRY 2011

A COMMISSION OF INQUIRY UNDER THE
COMMISSIONS OF INQUIRY ACT 1950

AND PURSUANT TO THE
COMMISSIONS OF INQUIRY ORDER (No. 1) 2011

# SECOND SUPPLEMENTARY STATEMENT OF BARTON JEFFERAY MAHER

On the 12th day of September, 2011, I, **Barton Jefferay Maher** of [redacted] Junction Road, Karalee, state on oath:

## Introduction

1. I am currently employed by Queensland Bulk Water Supply Authority (***Seqwater***) as Principal Engineer, Dams and Weirs Planning.

2. I make this statement in response to the requirement issued by the Queensland Floods Commission of Inquiry dated 30 August 2011 (the ***Requirement***).

## Brief description of the nature of cracking at Somerset Dam

3. The existence of some minor cracking in the Somerset Dam concrete has been known since at least 1951. It should be noted that cracking of concrete is common in mass concrete. It is the location and extent of the cracking which will dictate the impact of any cracking on the dam's long term performance and stability.

4. A summary of the cracking is contained in the report entitled *Somerset Dam Crack Investigation*, 2008 prepared by SMEC (***SMEC 2008 Report***). It provides that the cracking:

   - is typically evident in the top downstream corner of the upper gallery of the Dam;
   - was not significant upstream of the upper gallery;
   - is also evident across the downstream face of the dam;
   - is likely continuous between the gallery and the downstream face; and
   - is not present on the upstream face of the Dam.

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Filed on behalf of: Queensland Bulk Water Supply Authority trading as Seqwater

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ggkb A0118699321v1 120128021

5. The cracking is likely due to dissipation of heat following the completion of construction. Other factors that may have contributed to the cracking include:

- the long break in the construction of the dam between 1942 and 1948; and
- the use of two grades of concrete in the construction of the Dam wall (with a higher grade being used on the upstream side of the dam).

6. Because the cracking is present on the downstream side of the dam wall (rather than the upstream side), it does not materially impact the structural integrity of the wall. The stability of the dam is dependent upon two key loads. Firstly there is the weight of the concrete which resists overturning and sliding of the dam wall. Secondly there is the water loading on the upstream face of the dam which exerts a force that tries to push the dam wall downstream and, due to the nature of water loading, tries to bend the dam towards the downstream side of the dam wall. This bending, at water levels well above normal operating levels, results in the upstream face experiencing tension (i.e. being pulled apart) and the downstream face experiencing compression (i.e. being pushed together).

7. Therefore, it is the uncracked concrete at the upstream face of the dam which controls the stability of the dam under high loads from extreme flood events. As the water level in the dam increases, the water on the upstream side of the dam wall exerts a pressure which compresses the downstream side of the dam wall, effectively pushing the cracks on the downstream side of the wall together. The presence of the cracks has been taken into account in the analyses performed on the dam previously. The analysis has concluded that the dam is structurally sound. The existence of the cracks does not alter the way in which the dam is operated during flood conditions.

**Brief history of the monitoring of cracks in Somerset Dam to July 2008**

8. Annexed to this statement and marked **BM-1** is a bundle containing all reports in the possession of Seqwater which I have been able to locate which have considered the cracking of Somerset Dam.

9. The reports date back to 1939 with first significant report into cracking in 1951.

10. Significant investigations, monitoring and analysis of the cracking and stability of Somerset Dam have been undertaken since that time.

11. I have been able to locate records of regular monitoring (on a monthly basis) from the early 1980s. Annexed to this statement and marked **BM-2** are copies of those records.

12. The cracking which is described in paragraph 4 above is first documented in 1951 which is included in the bundle of documents in BM-1.

13. There are currently 22 measurement points to monitor the longitudinal crack along the upper gallery. These were installed between 1969 and 1984. File notes indicated that 5 monitoring points were installed in 1969, 4 cores were taken in 1978, 20 pairs of gauge plugs were installed in the centre

gallery cracking to allow a multi position strain gauge to be used to measure crack widths, 2 additional gauge plugs were installed in 1981 in the centre gallery, 6 additional pair of gauge points were installed in 1984 between monoliths G and H, L and M, and Q and R.

14. Additionally, 13 hydraulic piezometers were installed in 1999 at the dam to monitor pore pressure. The piezometers are read monthly.

15. Since Seqwater has owned Somerset Dam it has carried out monthly monitoring of the cracking, as well as yearly monitoring as part of the Annual Dam Safety Inspection.

16. Also, as part of their ordinary duties, the dam operators at Somerset Dam are required to monitor the cracking during and following significant rainfall events. If they notice a significant change, they are to immediately notify the Principal Engineer (Dam Safety), Mr John Tibaldi.

**Program for monitoring the cracks and similar issues at Somerset Dam following receipt of the SMEC 2008 Report**

17. The SMEC 2008 Report recommended that the crack in the upper gallery be monitored in respect to movement or other signs of deterioration every 4 months and, as a minimum, as part of any annual dam safety inspection and immediately following any significant inflows into the dam.

18. Since receipt of the SMEC 2008 Report, Seqwater has continued monitoring the cracking on a monthly basis. To assist the dam operators, the extent of cracking was marked to assist with the visual monitoring of any changes.

19. Seqwater has also continued undertaking routine inspections (on a daily basis) at Somerset Dam (and other dams) in accordance with the DERM and ANCOLD Dam Safety Management Guidelines. The routine inspections at Somerset Dam include inspections of the crack measurement points. Annexed to this statement and marked **BM-3** are copies of these routine inspections.

20. The cracking has also continued to be reviewed as part of the annual dam safety inspections and, as I explained above, the dam operators monitor the cracks during and following significant rainfall events.

**Inspections or additional monitoring of Somerset Dam undertaken as a result of the January 2011 Flood Event**

21. During the January 2011 Flood Event the cracking was monitored daily, with more than one visual inspection carried out on some days during the event due to the rate of rise of the storage. The operators inspected the galleries and sump pumps at Somerset Dam at a maximum interval of two hours throughout the peak of the January 2011 event.

22. Since the January 2011 Flood Event the existing monitoring arrangements described in paragraphs 18 to 20 above have continued.

23. Annexed to this statement and marked **BM-4** is a document detailing the monitoring results relating to the cracking since the event. These results do not indicate any change in the cracking which would cause concern about the stability of Somerset Dam.

**Upgrade of Somerset Dam and final Acceptable Flood Capacity Report**

24. .It is my understanding that Seqwater has not yet given the Department of Environment and Resource Management (***DERM***):

- a cost estimate and the time frame required to upgrade Somerset Dam in accordance to the Acceptable Flood Capacity Guidelines other than the SEQWater, *Provision of Contingency Storage in Wivenhoe and Somerset Dams Feasibility Report*, 2007 (see Exhibit BM-1); or

- a final Acceptable Flood Capacity Report. It is my understanding that a draft of the Somerset Dam Hydrology Study was sent to the DERM Office by the Seqwater Senior Hydrologist. This report is yet to be finalised. This report indicated that the upgrade of Somerset Dam would be required by 2025 under the fall back provision of the DERM Acceptable Flood Capacity Guidelines (2007).

25. The DERM information notice that accompanied the Dam Safety Conditions for Somerset Dam is annexed to this statement and marked **BM-5**. The information notice states that the need to upgrade Somerset Dam should be reviewed when carrying out the detailed design for the Wivenhoe Dam Stage 2 upgrade works.

26. Detailed design of the Wivenhoe Dam Stage 2 upgrade works has not yet commenced.

27. Entura (formally Hydro Tasmania) has been engaged by Seqwater to provide an updated scope of work and cost estimate to complete the matters detailed in paragraph 24 above.

28. Additionally, DERM has been provided with a report entitled *Somerset Dam Concept Review for Dam Raising* (2006) (contained within exhibit BM-1) which details cost estimates for certain upgrade options as part of the SEQWater, *Provision of Contingency Storage in Wivenhoe and Somerset Dams Feasibility Report*, 2007 (see Exhibit BM-1).

29. Further, it is my understanding that Seqwater is currently updating the portfolio risk assessment for all of its referable dams in accordance with the ANCOLD Guidelines on Risk Assessment (2003) (including a risk assessment relating to cascade failure of Somerset and Wivenhoe Dams) to prioritise and cost necessary upgrades to its dams.

30. In my opinion, it is appropriate to address the upgrade of Somerset Dam in the context of Seqwater's broader portfolio of dams so that it can be prioritised appropriately.

**Additional information regarding communications with DERM about the safety of Somerset Dam after July 2008**

31. In October 2008, I discussed progressing the investigations into raising Wivenhoe Dam to provide additional storage following the SEQWater, *Provision of Contingency Storage in Wivenhoe and Somerset Dams Feasibility Report*, 2007 (see Exhibit BM-1) with the DERM Office of Dam Safety. This project had initially looked at the possibility of raising Somerset Dam and had investigated preliminary options to upgrade Somerset Dam to increase the flood security as well as raise the Full Supply Level. This report was supplied to DERM for comment and review.

32. In September 2009, discussions were held with the DERM Office of Dam Safety AFC Project Manager and the Seqwater Senior Hydrologist to progress the hydrology study required as part of the Acceptable Flood Study program for Seqwater. These discussions were around the assumptions that would need to be made to assess the flood capacity of Somerset Dam in isolation from Wivenhoe and provide an estimate of the percentage of the Acceptable Flood Capacity that the dam could safely pass. This was required to determine the upgrade timing using the fallback position in the DERM Acceptable Flood Capacity Guidelines (2007). I cannot now remember the detail of those discussions.

33. I am aware that there was also further correspondence with the DERM Office of Dam Safety around the revision of the Wivenhoe and Somerset Flood Manual, in particular the interaction diagram and the subsequent interaction study in 2009. However, I was not directly involved in this work.

**SWORN** by **BARTON JEFFERAY MAHER** on 12 September 2011 at Brisbane in the presence of:

[redacted]

Deponent

[redacted]

Solicitor

IN THE MATTER OF

THE QUEENSLAND FLOODS COMMISSION OF INQUIRY

A COMMISSION OF INQUIRY UNDER THE
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**Supplementary STATEMENT OF BARTON JEFFERAY MAHER**
**INDEX OF ANNEXURES**

| **Annexure No.** | **Document** | **Date** |
|---|---|---|
| BM-1 | Reports considering the cracking and stability of Somerset Dam<br>1. 10-Aug-39 – Letter - RE Somerset to Chief Engineer Stanley River Works Abroad.<br>2. 16-May-40 – Memo- E.L.Richard to RE Somerset Dam re cracking plus field notes to 1941<br>3. 29-May-40 – Memo- E.L.Richard to RE Somerset Dam re cracking, compares Somerset data to published data on Haweswater Dam<br>4. 01-Jan-41 - Notes left by E.L.Richard on his retirement re Somerset Dam cracking<br>5. 21-Sep-51 – Memo - RE Somerset (Gipps) to Chief Engineer Stanley River Works Board forwarding G M Card's report of 30/7/51<br>6. 29-Sep-69 – Letter - Sheperd to Cowling. Comments on cracking and possible causes<br>7. 07-Nov-77 – Letter - Dept of Works to Cossins & | Various |

| Annexure No. | Document | | Date |
|---|---|---|---|
| | | Clerke. Survey of monoliths along crest | |
| | 8. | 08-Nov-77 – Letter - Copy of 6 (Shepherd's letter) plus notes from G M Card dated 30/7/51 | |
| | 9. | 08-Nov-77 – Letter - From Shepherd to BCC. Prelimary report on inspections of BCC dams. Includes Somerset inspection notes | |
| | 10. | 24-Nov-77 – Notes - Cracking comments from Gil Verrenkamp | |
| | 11. | 30-Nov-77 – Memo - Recommendation of Soiltest gauge for crack measurement | |
| | 12. | 01-Apr-78 – Letter - ANCOLD Bulletin of April 78. Letter from EM Shepherd re dam cracking on page 3 | |
| | 13. | 07-Feb-79 – Memo - Verrenkamp to Clerke – Test Holes – Centre Gallery – Somerset Dam | |
| | 14. | 07-Feb-79 – Memos - Collection of copies of memos and letters which were sent to BAWB for insurance purposes. Copies filed above | |
| | 15. | 20-Feb-79 – Letter - Clerke to Shepherd re drilling of test holes | |
| | 16. | 09-Aug-82 – Letter - O'Connell to Shepherd. Refers to weeps in shaft 13 and proposed trangulation and level surveys | |
| | 17. | 29-Aug-86 - Notes - on Visit to Somerset Dam – 29 August 1986. Ben Russo | |
| | 18. | 31-Aug-87 – Letter - From Board to BCC requesting report on cracking for insurance purposes | |
| | 19. | 05-Oct-87 – Report - Somerset Dam Survelliance – Report on Cracking – Ben | |
| | 20. | 09-Oct-87 – Letter - Letter from BCC to Board re cracking with reference to report from Ben Russo | |
| | 21. | 25-Mar-88 – Fax – From Gil Verrenkamp to George Ilievski – Analysis of Foundation Drain Water Sample Somerset Dam | |
| | 22. | Brisbane City Council – Safety Review, Somerset Dam – July 1988 | |

| Annexure No. | Document | | Date |
|---|---|---|---|
| | 23. | Department of Water Supply and Sewerage – Somerset Dam Inspection Report - 7 April 1995 | |
| | 24. | South East Queensland Water Board – Safety Review, Somerset Dam – September 1995 | |
| | 25. | R. Russo – 1996 GHD Somerset Dam Safety Review Comments – 5 August 1996 | |
| | 26. | South East Queensland Water Board – Scoping Report for Installing of Piezometers to Monitor Uplift in the Foundation – January 1999 | |
| | 27. | South East Queensland Water Board – Foundation Gallery Inspection – 24 December 1999 | |
| | 28. | South East Queensland Water Board – Safety Review – September 2000 | |
| | 29. | South East Queensland Water Board – Preliminary Risk Assessment, Wivenhoe, Somerset and North Pine Dams – Final Report - 2000 | |
| | 30. | SEQWater – Somerset & North Pine Dams, Dam Safety Review – December 2004 | |
| | 31. | WRM Water & Environment – SEQWater – Somerset and Wivenhoe Dam Flood Risk Analysis – 8 October 2004 | |
| | 32. | SMEC – Somerset Dam – Detailed Risk Assessment of Project – 2005 | |
| | 33. | SEQWater - Somerset Dam – Stability of Abutment Monoliths – May 2005 | |
| | 34. | SMEC – Somerset Dam Concept – Review for Dam Raising – September 2006 | |
| | 35. | SEQWater – Condition Assessment of Bulk Water Assets – 28 November 2007 | |
| | 36. | Queensland Government Natural Resources and Water – SEQWater Water Services Dam Safety Audit – June 2007 | |
| | 37. | SEQWater – Provision of Contingency Storage in Wivenhoe and Somerset Dams – February 2007 | |
| | 38. | SMEC – Somerset Dam Investigation – July | |

| Annexure No. | Document | Date |
|---|---|---|
| | 2008<br>39. Seqwater – Annual Dan Safety Inspection 2008 – 17 November 2008<br>40. Seqwater – Annual Dam Safety Inspection – 2 November 2009<br>41. Seqwater – Comprehensive Dam Safety Inspection – September 2010 | |
| BM-2 | Records of Crack Monitoring | Various |
| BM-3 | Records of Routine Inspections | Various |
| BM-4 | Crack monitoring results following January 2011 Flood Event | Various |
| BM-5 | Information Notice accompanying Dam Safety Conditions | 22 May 2009 |

Due to the large number, and size, of the annexures to this statement, it is only possible to publish those annexures specifically referenced in the Commission’s Final Report.

These annexures are:

BM-1: Document 24
BM-1: Document 33
BM-1: Document 38
BM-1: Document 40
BM-1: Document 41

BM-1: Document 24

DAVID GILL

# SOUTH EAST QUEENSLAND WATER BOARD

# SAFETY REVIEW

# SOMERSET DAM

SEPTEMBER 1995

SOUTH EAST QUEENSLAND WATER BOARD
SAFETY REVIEW
SOMERSET DAM

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FIGURES



APPENDICES

# 1 INTRODUCTION

Somerset dam was constructed in the period 1935 to 1953 with a delay in construction as a result of the war in the period 1942 to 1948.

The dam has been constructed across the Stanley River between Little Mount Brisbane and Mount Somerset. It is located at AMTD 4.8 on the Stanley River which is a tributary of the Brisbane River and is located in the upstream limit of the Wivenhoe storage.

Wivenhoe dam was constructed in the period 1976 to 1985.

Somerset dam is a concrete gravity structure with a gate controlled flood storage compartment. The primary features of the dam are indicated in Table 1 and Figure 1.

The dam has a downstream energy dissipater with baffle and side training walls of mass concrete. The distance from the downstream face of the spillway to the baffle is 36.6 m (120 ft) and the overall length of the stilling basin is 58.2 m (191 ft). The concrete in the floor of the stilling basin is about 3 m (10 ft) thick for the first 23.8 m (78 ft) downstream of the spillway and reduces to 1.5 m (5 ft) thickness at the baffle and 0.76 (2 ft 6 inches) at the end sill.

The dam and the stilling basin are provided with a number of drains to limit the build up of pore pressures.

A hydro-electric powerstation on the right side of the spillway currently operates five days a week and currently is the primary method of making releases from the storage.

Four cone valve regulators 2.3 m in diameter are also able to make releases. During the 1974 flood these regulator valves were submerged by the tailwater.

Eight sluice gates have been provided for normal flood releases and eight radial gates were provided to improve the flexibility in operation during flooding. The Full Supply Level of Somerset dam is 1.5 m below the concrete spillway crest level.

A reinforced concrete deck is located 4.88 m above the non-overflow crest level of the dam. This deck carries the winches and gantry crane required to operate the sluices, crest gates and coaster gate.

An inspection of the dam was undertaken in the period 6 to 7 March 1995.

This report has been completed as part of a comprehensive review of the design, construction and performance history of Somerset dam to evaluate its structural and hydraulic integrity. The evaluation has included an examination of existing records, an on site inspection, and a review of the dam against current design and construction standards.

This report has been organised in accordance with the WRC guidelines for preparation of a Dam Safety Review - Procedure DS 005. This report discusses the dam in the following sequence:

- Section 2 provides a summary of the findings and recommendations.
- Section 3 which indicates the sources of the data reviewed for this report.
- Section 4 summarises the operational status of the dam during the inspection.
- Section 5 indicates the summary historical events associated with the dam.
- Section 6 summarises the paths to failure that were considered as part of this review.

- Section 7 summarises the state of preparedness of the dam in an emergency event.
- Sections 8, 9, and 10 summarise the results from the hydraulic, geotechnical and structural evaluation of the dam.

The conclusions and recommendations are based on a thorough examination of the available design files, the available records of construction, the site inspection, examination of instrument records and the stability analyses completed for this report. The conclusions are supported by findings within the report and the Recommendations result from the conclusions. The recommendations are based on safety concerns for the dam and generally relate to the need to collect additional data to confirm the findings of this report.

## TABLE 1
## SOMERSET DAM PRIMARY FEATURES

| | | |
|---|---|---|
| 1. | **DAM** | |
| | Type of dam | Mass concrete gravity |
| | Dam Owner | South East Queensland Water Board |
| | Length of dam at deck level | 308 m (1012 ft) |
| | Maximum height above foundation level | 58 m (189 ft) |
| | Lowest foundation level | RL 54.77 (180 ft) |
| | Total volume of mass concrete | 205 700 $m^3$ (269 057 $yd^3$) |
| | Dam crest level - bridge deck level | RL 112.34 (369 ft) |
| | - non overflow crest level | RL 107.46 (353 ft) |
| | - level of concrete spillway crest | RL 100.45 (330 ft) |
| | Full Supply level | RL 98.93 (325 ft**) |
| | Minimum water level | RL 69.97 (230 ft***) |
| | Peak water level as a result of PMF | One gate out of Service RL 110.7<br>All gates open RL 110.4 |
| | Maximum discharge as a result of PMF | One gate out of Service 7950 $m^3$/s<br>All gates open 8140 $m^3$/s |
| | Tailwater levels* | 1974 flood 1052 cumec Tailwater level 70.9 to RL 72.1<br>PMF 8140 cumec Tailwater level RL 80-82. |
| 2. | **SPILLWAY** | |
| | Number of radial spillway gates | 8 |
| | Size of each crest gate | 7.9 m wide x 7.0 m high - (26 ft wide x 23 ft high) |
| | Top of gates when closed | RL 107.46 (353 ft) |
| | Radius of spillway gates | 7.0 m (23 ft) |
| | Centreline of trunnion bearing | RL 103.73 (340.75 ft) |
| | Clear length of spillway | 63.4 m (208 ft) |
| 3. | **SLUICE GATES** | |
| | Number of Sluice gates | 8 |
| | Size of each sluice gate | 3.66 m high x 2.44 wide - (12 ft high x 8 ft wide) |
| | Invert level of sluice entrance | RL 71.2 m (234 ft) |
| | Top of stilling basin training walls | RL 73.02 (240 ft) |
| | Stilling basin length | 58.2 m (191 ft) |
| | Baffle height | 3.0 m (10 ft) |
| | Stilling basin level | RL 60.83 (200 ft) |
| 4. | **REGULATORS** | |
| | Number of regulators | 4 |
| | Level of centreline of regulators | RL 69.97 (230 ft) |
| | Type and size of regulators | 2.3 m dia fixed cone dispersal valves |
| | Discharge capacity of each regulators at FSL | 79 cumecs (2,800 cusecs) |
| | Discharge capacity of each regulators with water level at RL 250 ft | 28 cumecs (1,000 cusecs) |
| 5. | **STORAGE** | |
| | Reservoir area at F.S.L. | 4,400 ha |
| | Storage capacity at F.S.L. | 369,000 ML |

* Tailwater levels - considerable uncertainty exists in the tailwater level estimates Wivenhoe Dam has a significant influence on the tailwater.

** Full supply level originally 315 ft

*** Centreline of regulator inlet.

( ) Values in brackets are as shown on drawings in imperial units, all metric units are to AHD with RL (AHD) = (RL (ft) - 0.43) x 0.3048

## 2 EXECUTIVE SUMMARY

### 2.1 Summary of Findings

The following items summarise the findings of the Safety Review:

(i) the concrete visible in the upstream face is in a good condition with some minor loss of mortar resulting in exposure of the coarse aggregate. The loss of mortar is a symptom of the presence of aggressive water in the reservoir. Measurements of pH, dissolved gasses and water temperature should be undertaken and Saturation Index or Longellier Index calculated.

(ii) the downstream face of the dam has a dirty appearance in the section of the dam with a slopping face. There are signs of seepage at lift joints where calcite has been deposited "over" the dirty surface.

(iii) the bridge deck is in good condition.

(iv) the crest of the mass concrete section is in good condition with minor cracking only.

(v) The gate pier concrete appears to be generally in a good condition. A pattern of closely spaced cracking was noted on the top of some piers. This may be caused by ageing, temperature effects, or alkali aggregate reactivity.

(vi) The downstream river channel is in good condition. Erosion on the right abutment downstream of the stilling basin has been repaired with large rip-rap.

(vii) The mass concrete training walls are in good condition. However, stability calculations using assumed backfill and water level data indicate that the walls are potentially unstable. The properties and depth of backfill is required to asses the stability of the walls. Water table levels behind the wall are critical to its stability and this requires monitoring. During a flood water pressure resulting from the tailwater can be transmitted through the rockfill behind the walls and increase the pressure on the walls at the upstream end where the depth of water in the dissipator may not be as great as at the downstream limit due to the formation of the hydraulic jump. At these times water is expected to flow out of the weep holes to relieve the water pressure.

(viii) The dissipator floors may be subject to high velocity flow sufficient to cause cavitation during a flood equivalent to the 1974 flood. During major floods the dissipator does not result in complete energy dissipation and high velocity flow is likely beyond the downstream limit of the stilling basin. For floods larger than the 1974 flood the thickness of concrete in the floor may be insufficient to resist the uplift pressures caused by the tailwater. Monitoring of uplift pressures is desirable.

For floods larger than the 1974 flood the dissipator retaining walls are likely to be overtopped. During the PMF there could be about 7 m of water over the top of the walls. This may result in high water loads behind the walls after the flood has receded.

The dissipator should be pumped out and inspected.

(ix) The gallery inspection indicated that:

- there is a continuous crack on the downstream side of the upper gallery within the 0.3 m x 0.3 m fillet at the roof of the gallery.

- cracking on the upstream side of the gallery is much less pronounced.

- cracks in construction joints in the sluice shaft have a build up of calcite. Of particular interest is a construction joint at about RL 91.5 in Monolith I and N. These are reported to be damp at times.

- the contraction joint between Monoliths L and M is open at the deck level less open at the upper gallery level and weeping and closed at the lower gallery level.

- the contraction joint drain in Monolith S has been weeping for some time. A horizontal crack in Monolith S was noted in the upstream face in 1977 at RL 97.2. The seep into this drain may be through this crack.

- the cored drain in Monolith R is weeping and appears to be partly blocked.

- the roof, walls, and floor of the gallery is dry with water draining from the drain holes into the gutter to the sumps which are pumped out weekly.

- on the left hand end of the gallery a crack extends between the upper and lower galleries in the stairway near where the Monolith S and T contraction joint would occur.

(x) The evaluation of the cracking would indicate that the majority of the cracking may result from the loss of heat following construction. Ninety Five percent of the heat would been dissipated by about 1960 (The cracking was not noted in 1951 possibly because this was too soon after the final phase of construction which began in 1948). This heat loss would result in a reduction in volume of the core relative to the outer "shell". This would result in compression loading of the outer "shell" of the dam and tension internally that could manifest as opening of construction joints or cracking of mass concrete.

(xi) The crack is continuing to open at a rate of less than 0.1 mm per year. This may indicate that the core of the dam is continuing to lose the final small amount of heat built up during construction. A 0.1 mm/year movement is approximately equal to a 0.3°C per year reduction in temperature.

(xii) Free water can at present enter the gallery during a flood through:

- the access door to the upstream regulator platform. The invert of this door is at RL108.1 (355 ft) - 2.3 m below PMF level
- when the non-overtoppable section is overtopped the water flowing down the abutments can enter the galleries through the gallery access portals. A bulkhead gate has been provided to prevent this
- when the tailwater level exceeds about RL64.6 m (212.5 ft) water can enter the pipes provided to pump out the sumps. This is prevented by the provision of a reflux valve.

(xiii) The upper portion of joint L/M has opened excessively and resulted in a torn water stop in the upper portions of the joint.

(xiv) The outlets of the drilled foundation drains have been capped with a hole in the cap. These caps should be removed and replaced with a pipe with a right angle bend. Seepage water from each drain should then be monitored at monthly intervals.

(xv) there is no monitoring of uplift undertaken other than the monitoring of water level in the various drains entering the lower gallery. This monitoring effectively measures the effectiveness of the sump pumping arrangement. Installation of effective uplift monitoring systems should be undertaken.

(xvi) A precise survey was undertaken in the early 1980's. This should be repeated now to determine whether there has been significant movement in the period.

(xvii) The outlet sluices were not all inspected. Only sluice gate K was inspected. Cracking repaired in the sluice tunnels in Monolith Q was not inspected.

(xviii) The trash rack structure roof on the left abutment was inspected and the concrete was found to be dirt/silt covered but apparently in good condition.

(xix) The emergency coaster gate is generally in a fair condition. The rollers are in a poor condition. The gate is designed to close safely against flow but at present this capability is in doubt. If a sluice gate were to fail or fail to close there is no other way to repair the gate than by installing the coaster gate into flow. It is essential that this gate be repaired to a standard which would enable it to be close safely against flow or it be replaced.

(xx) On the basis of the assumed "normal" parameters, the dam meets the current structural guidelines with the reservoir at FSL however, if the radial gates are closed during a flood (as is allowed by the current flood operational rules) the upper section of the Monoliths are likely to fail through overturing. Assuming zero tensile strength at the upper gallery level (as a result of the cracking), the imminent failure flood level (IFF) with the radial gates closed is RL 105.7 and the IFF with the radial gates open is RL 109.1. There is insufficient data available to define the loss of strength in the dam caused by the cracking. If lower bound strength parameters are adopted the dam would not meet current structural guideline requirements.

(xxi) The majority of the parameters used to assess the stability of the dam were assumed. It is necessary that the concrete and foundations be investigated to ensure that appropriate parameters are available for the stability assessment. Sub-horizontal foundation joints were identified during construction. The locations, extent and properties of these joints are required to effectively define the stability of the dam.

(xxii) Access to the dam during extreme flood events may be limited.

Overall the dam is assessed to be CONDITIONALLY POOR with the need to reduce the uncertainty associated with the critical analysis parameters through further investigations and monitoring. The dam is likely to be regarded as FAIR when the data is available however the emergency coaster gate and dissipator walls and floor are likely to be classified as POOR and the cracking in the gallery and upstream face of the dam may indicate conditions throughout the dam that would result in the dam being classified as poor.

## 2.2 Conclusion

1 From observations made about the state of the dam during the inspection, it is concluded that the dam shows deterioration in the following areas:

(a) aggressive water in the reservoir has caused loss of mortar from the upstream face of the dam and in 1950 concrete deterioration was noted in Monolith Q at about RL70 in a bell-mouth in Monolith Q.

(b) the gate pier concrete and other areas of concrete have closely spaced cracking that may be associated with alkali-aggregate reactivity.

(c) joint opening has caused tearing of the waterstop in joint L/M.

(d) the outlets of the drilled foundation drains have been capped to prevent dirt entering the holes. These caps have holes in them and some of these holes are becoming blocked.

(e) the emergency coaster gate rollers are in a poor condition and it can no longer be used to close safely against flow.

2 The stability analyses indicate that:

(a) under normal loading conditions using assumed parameters the dam is stable using ANCOLD guidelines. Under PMF conditions with the gates closed the dam exhibits excess tensile stresses which could result in failure. The imminent failure flood level (IFF) with the radial gates closed is RL 105.7 and with the radial gates open it is RL 109.1.

(b) the dissipator training walls would be considered to be unstable using current design criteria.

(c) the dissipator floor and walls are subject to high velocity flow which has the potential to enter the drain/weep holes and cause excess uplift pressures which may cause failure of the stilling basin.

(d) the hydraulic jump may result in excess pore pressure beneath the upstream portion of the stilling basin and behind the upstream end of the stilling basin walls.

(e) The analyses were completed using assumed strength and uplift parameters. These parameters are considered to be reasonable but require validation:

3 A review of the available construction records and the site inspection indicated that:

(a) cracking of concrete was a concern during construction and a crack survey was undertaken in 1951.

(b) a horizontal crack in the upper gallery was reported in 1969. This crack is observed mainly on the downstream side of the gallery and near the roof. It extends for the full length of the gallery and is observed in the access adits at each end.

(c) cracking between the shafts in Monolith Q with pronounced leakage was noted in 1977, and repaired at some later date.

(d) the construction joint drain in Monolith S has been seeping for some time. The horizontal crack noted in the upstream face is likely to be the source of this seepage.

(e) the dam was apparently constructed without any control of placing temperature. Temperature changes are likely to be the main cause of the cracking.

4 Water can enter the gallery through the access door to the upstream regulator/trashrack platform, through the access adits into the gallery and through the sump pump outlet pipe.

5 Monitoring at the dam is not providing adequate data on the structural safety of the dam. Uplift pressure monitoring is not effective; there is no deformation or movement monitoring and there is no seepage monitoring.

6 Access to the dam during extreme flood events may be limited.

### 2.3 Recommendations

1 In order to assess the extent of the deterioration it is recommended that:

(a)
- Measurements of pH, dissolved gasses, water temperature and sulphate be undertaken in the reservoir water at various depths and times of year at the dam. In addition the saturation index or Longellier Index should be calculated.
- Cores be taken from the concrete in the upstream face of the dam to test for deterioration or loss of cement.

(b) Cores be taken from the gate pier concrete for testing to assess whether alkalai aggregate reactions are occurring.

(c) The joint opening between Monoliths L/M be monitored at the deck level, upper gallery level, and lower deck level. If movement is not continuing over a 1 or 2 years period the joint should be repaired and the water stop reinstated.

(d) The caps on the drilled foundation drainage holes should be removed and replaced with 90° elbows and the rate of seepage from each drain monitored.

(e) The emergency coaster gate be repaired or replaced to provide a gate that can be safely placed in flowing water.

2 In order to "prove" the stability model assumptions about drainage, uplift, rock joint strength, cracking, concrete strength and density it is recommended that investigations be undertaken. These investigations should involve drilling and recovery of core with a diameter of not less than 100 mm. This drilling will include:

(a) at least five holes through the dam with some holes angled and extending about 20 m into the foundation.

(b) at least 3 boreholes in the left and right abutment near the toe of the dam to a depth of about 10 m and at least twp sub-horizontal holes drilled into the dam from the dam abutments to inspect cracking and the quality of the concrete/foundation interface.

(c) bore hole camera inspection of the holes in the dam and abutment to define cracking, joint orientation and opening.

(d) at least 6 holes in the left and right side dissipator retaining walls to determine the properties and depth of the backfill behind the walls and to monitor water levels.

(e) inspection of the floor of the dissipator to assess its current condition and if practical undertake drilling in the floor to assess the quality of the concrete/rock interface, and install piezometers to determine uplift pressures beneath the dissipator floor.

(f) Inspect the drain/weep holes in the floor of the dissipator and assess whether they are working and whether high velocity flow could enter the holes. Map the position of the holes.

3 (a) It is recommended that a crack survey be undertaken to locate the position and extent of all visible cracks in the dam. This survey should be in sufficient detail to enable new cracks to be identified in the future.

(b) It is recommended that the frequency of the current crack monitoring in the upper gallery be undertaken at one month intervals rather than weekly.

4 To prevent entry of water into the gallery it is recommended that:

(a) a bulkhead gate be designed, supplied and installed at the access door to the upstream regulator/trashrack platform.

(b) the importance of the existing bulkhead gate to the access adit be fully understood by all staff at the dam and that it should be installed and removed as part of the regular safety inspection.

(c) the reflux valve on the sump pump outlet pipes be regularly inspected and maintained.

5 It is recommended that the following instrumentation be installed to allow effective monitoring of the structural performance of the dam:

(a) piezometers at 5 sections in the dam. These should be located at the following points in the dam:

- upstream of the grout curtain, about 5 m upstream of the grout curtain and about 1 m below the "cut-off" level
- between the grout curtain and drilled drainage holes
- between the upstream foundation drain and upstream foundation tunnel
- at the downstream toe of the dam.

(b) It is recommended that thermometers be installed in the dam to measure the change in response of temperature with depth into the concrete. This should be compared with deformation monitored using plumb bobs and precise survey of the crest.

(c) piezometers in each of the six boreholes drilled behind the dissipator retaining walls.

(d) piezometers in the floor of the dissipator to monitor uplift.

(e) carry out a precise survey at the locations surveyed in the early 1980's, and then at six monthly intervals in the winter and summer.

(f) install plumb bob monitoring in the drill holes installed from the crest of the dam and intersecting the lower gallery.

(g) install surface movement points on the crest of the dam and along the downstream toe of the dam.

(h) install a water meter on the sump pump system to monitor the quantity of water seeping into the under drains and take water samples at six monthly intervals and undertake chemical analyses to assess deterioration of the grout curtain.

About 12 months after installing the monitoring system review the data collected and reassess the stability of the dam.

6 It is recommended that a study be undertaken of the flood limits likely to limit access to the site during extreme flood events, this study would also result in an improved knowledge of the tailwater levels likely to exist at the dam.

7 Until detailed investigations and analyses are completed it is recommended that the imminent failure flood level be adopted as RL 105.7 if the radial gates are closed and RL 109.1 if the radial gates are open.

## 3 DATA REVIEWED

1. Somerset Dam crack measurements for hydrological year 1986/87 to 1990/91.

2. Somerset Dam Drawings - Volumes 1 to 5, South East Queensland Water Board.

3. Somerset Dam - Uplift Surveillance 1st September 1987 to 17 February 1995.

4. Original Design Calculation files:

   SD(M) 1 to 15 and SD(M) 17
   SD(S) 1 to 7 and SD(NS) 1 to SD(NS) 5
   SD(SG) 1 to SD(SG) 25
   SD(PH) 1 to SD(PH) 6 and SD(VH) 1 to SD(VH) 6
   SD(SP.D) 1 to SD(SP.D) 5
   SD(ST.G) 1 to SD(ST.G) 10

5. Original Design Calculation Books

   SD(DC) 1 to SD(DC) 21

6. Original Stanley River Works Board - Somerset Dam - Excavation - Fortnightly Summary.

7. E L Richard "Foundation Grouting at Somerset Dam.

8. R Russo "Somerset Dam Safety Review" July 1988, Brisbane City Council.

9. R Russo "Somerset Dam Safety Inspection" 1994.

10. Report titled "Somerset Dam" introduced by Frank Nicklin Premier of Queensland.

11. Report titled "Somerset Dam" Issued by the Bureau of Industry 1938.

The following correspondence and records relevant to Somerset Dam Safety:

<u>Undated Correspondence and Reports</u>

(a) R Gipps "Permeability Testing of Concrete "A" Class and "C" Class Somerset Dam".

(b) Somerset Dam "Notes on the History of Construction".

(c) Cracking in Galleries - Somerset Dam
Notes of Data which are possibly relevant.

(d) Copy of Letter to the Chief Engineer Stanley River Works Board, "Cracks and Galleries" - written 10 months after forms stripped.

(e) Joint Movement and Cracking.

Dated Correspondence and Reports

(i) Summary of paper by D C Henry Trans Am Soc C.E Vol.99 1934 pp 1041-1123 - "Stability of Straight Gravity Dams"

(ii) Letter from Chief Engineer to H H Dare.

(iii) Notes on cracks 1939 to 1942 - possibly summaries prepared by G M Card.

(iv) Letter from Chief Engineer to the Chairman Stanley River Works Board "Construction of Somerset Dam under War Conditions".

(v) Resident Engineer to the Chief Engineer "Concrete in Mon Q Bell-mouth - 15.2.50.

(vi) G M Card to the Chief Engineer - "Concrete Crack Survey" 21 September 1951.

(vii) G A Cowling to W H R Nimmo "Drainage Measurements - Somerset Dam" - 7.8.62.

(viii) W Nimmo to the Chief Engineer and Manager "Drainage Measurements - Somerset Dam - 22.8.62.

(ix) G A Cowling to W H R Nimmo - water levels when drainage measurements taken 23.8.62.

(x) E M Shepherd to Mr Cowling "Cracking in Galleries at Somerset Dam 29.9.69.

(xi) G Cossins "The Operation of Somerset Dam 1969".

(xii) The operation of Somerset Dam during the flood of January 1974.

(xiii) E M Shepherd "Somerset Dam - Notes on its capacity to cope with extreme floods" 17.11.76.

(xiv) E M Shepherd "A Cracking Pattern in Some Concrete Dams" ANCOLD, 17 Oct 1977.

(xv) N H Lindley "Somerset Dam Investigation on 4 Nov 1977".

(xvi) E M Shepherd "Examination of Brisbane City Council Dams - Preliminary Report" Nov 1977.

(xvii) N H Lindley to G Cossins and J Clerke, Somerset Dam Investigations 22 Nov 1977.

(xviii) G Verrenkamp "Inspection of Somerset Dam" 28 Nov 1977.

(xix) G Verrenkamp "Coaster Gate Operations - Somerset Dam June 1976 to June 1978.

(xx) G Verrenkamp & J Clerke "Test Holes - Centre Gallery Somerset Dam, 7 Feb 1979.

(xxi) B P O'Connell to E M Shepherd "Crack Investigations" 20 Feb 1979.

(xxii) E M Shepherd "Cracking in Concrete Dams" ANCOLD August 1979.

(xxiii) Notes on dam movement.

(xxiv) E M Shepherd to B P O'Connell "Inspection of BCC Dams" 30 May 1980.

(xxv) R Russo "Notes on visit to Somerset Dam" 29 Aug 1986.

(xxvi) R Russo - concrete density etc.

(xxvii) R Russo "Somerset Dam Surveillance", Report on Internal Drainage System 5.11.86.

(xxviii) R Russo "Safety Review of Somerset Dam Interim Report" 1.11.86.

(xxiix) Materials Engineer to Superintendent "Density of Concrete Cores" 12.2.87.

(xxix) R Russo "Somerset Dam Assessment of Uplift Measurement of 7 May 1987" 6.8.87.

(xxx) R Russo "Somerset Dam Surveillance Report on Cracking" 5.10.87.

(xxxi) R Russo "Somerset Dam Assessment of Uplift Measurements of 31.3.88, 19.4.88 and 8.5.88.

(xxxii) R Russo "Somerset Dam Uplift Surveillance Visit of Inspection" 25.8.88.

(xxxiii) R Russo "Somerset Dam - Uplift Surveillance" 29.12.88.

(xxxiv) R Russo "Somerset Dam - Uplift Surveillance" 17.6.89.

(xxxv) R Russo "Somerset Dam - April Floods - Report on Scour Damage Downstream of Dissipator" 19.6.89.

(xxxvi) R Russo "Proposal for inspection procedures in the event of a damaging earthquake" 16.2.90.

(xxxvii) H Holland "Brief for Flood and Dam Break Studies for North Pine, Somerset and Wivenhoe Dams".

(xxxviii) H Holland "Reply to SEQWB letter" 4 Oct 1990.

(xxxix) P Barber "Somerset Dam Reinstatement of Embankments" 15.3.91.

## 4 OPERATIONAL STATUS DURING FIELD INSPECTION

The dam was inspected on 6 and 7 March 1995. The findings from this inspection are summarised in Section 10.4.1 of this report. In the period prior to 11 February 1995 a very dry season had resulted in the storage being at an unusually low level. From 11 February 1995 to 28 February the reservoir rose from RL 90.81 to RL 95.81. On the 6 March the reservoir level was RL 95.81. At this time the stored volume of water was 252,000 ML which is 68% of its design capacity. During the inspection light rain was experienced.

The tailwater level is not monitored at the dam, however, as Wivenhoe dam is low the tailwater level is directly related to the releases from the power station and was at about RL 64.1.

There were no spillway or sluice releases made during the inspection except those made during the exercising of the gates.

The maximum water level ever experienced in the reservoir was RL 106.55 (RL 350) which occurred during the 1974 flood. At that time it was noted that the tailwater level was RL 72.1 (RL 237).

## 5 HISTORICAL EVENTS

(a) Construction

(i) Stanley River Works Board constituted 1934,
(ii) Access road, trial pits and construction of town commenced 1935,
(iii) First concrete placed during October 1937,
(iv) Work was suspended in 1942 due to war. The level of the monoliths at the time of the suspension of work is indicated in an undated report "Notes on the History of Construction" Somerset Dam. This indicates the following:

- The levels of monoliths during the war time close down period was approximately as follows. Where two values are shown the former represents the upstream level and the other downstream level.

- Spillway Monolith C, D, E, F, T, U, V and W were all completed to crest level at RL 107.46 (RL 353). The lower reinforcement of piers was set, but piers were not constructed.

- Spillway monoliths J, L, M and O were completed to crest level at RL 100.45 (RL 330).

- Levels of Monolith:
  - G RL 102.65 to 102.01 (RL 337.2 to RL 335.1)
  - H RL 101.21 to 101.98 (RL 332.5 to RL 335)
  - I RL 87.71 to 87.96 (RL 288.2 to RL 289.0) with a high section at RL 88.14 289.6)
  - K RL 88.11 to 88.57 (RL 289.5 to RL 291)
  - N RL 87.35 to 88.35 (RL 287 to 290.3)
  - P RL 87.35 to 90.39 (RL 287 to RL 297)"

(Note: The invert of the upper gallery is at RL 88.9 (292 ft).

(v) Work recommenced in early 1948 and the last structural concrete was placed in 1953,
(vi) The sluices were operational in 1951,
(vii) The first discharge regulator was installed in 1952,
(viii) The gantry crane was taken over in 1955.
(ix) The eight spillway gates were in place in 1953,

(b) Structural behaviour

(i) Summary of crack monitoring reports in 1939 to 1942,
(ii) Deterioration of concrete in Monolith Q Belmouth 15-2-50,
(iii) Crack survey by GN Card dated 30-7-51,
(iv) Drainage measurements GA Cowling 7-8-62,
(v) EM Shepherd letter to ANCOLD on cracking in some concrete dams October 1977,
(vi) NH Lindley survey of dam movements 7 November 1977,
(vii) EM Shepherd inspection of dams dated 8 November 1977 - discussion of cracking in gate shafts, in Monolith Q and notes on comparison with crack survey completed in 1951,

(viii) G Verrenkamp memo dated 28.11.77, on horizontal cracks in upstream face of dam at RL 95.3 to RL 97.2,
(ix G Verrenkamp - crack test holes - 7 February 1979,
(x) EM Shepherd letter to ANCOLD further discussion on cracking August 1979,
(xi) R Russo notes on site visit 29-8-86,
(xii) R Russo site visit 8 October 1986 - concrete density,
(xiii) Gate in Monolith M scraping against pier first noted in the period 1986 to 1988.

(c) Earthquake activity

The earthquake activity of the area was assessed for this report by Mr G Gibson and R Cuthbertson: Somerset Sam - North Pine Dam; "Review of Seismicity", March 1995. This indicates that there have been 1363 events recorded in South East Queensland and 108 events within 25 km of Somerset dam. The largest of these was in Kilcoy in 1913. This event was of magnitude MP 4.8 and was strongly felt in the central Brisbane valley, with a maximum intensity of 5 assigned to Kilcoy, Esk and Crows Nest. A permanent creek in the Crows Nest area was reported to have gone dry immediately after the earthquake.

This Kilcoy earthquake was only about 15 km from Somerset Dam.

(d) Safety Review's

(i) R Russo "Safety Review - Somerset Dam: Brisbane City Council 1988,
(ii) R Russo "Dam Safety Inspection - Somerset Dam" 1994

## 6 PATHS TO FAILURE

Dams may contain weaknesses that can lead to failure. Where a number of weaknesses lead to a dam failure the incidents are collectively known as paths to failure. The potential paths to failure identified at Somerset dam are indicated in Appendix D.

The paths to failure have been considered as a series of causes and consequences. This is illustrated below by following one of the paths from sheet 2 and sheet 1 of Appendix D.

| Incident | Cause | Consequence |
|---|---|---|
| 1. | Grout curtain failure | Inadequate drains |
| 2. | Inadequate drains | Excess uplift |
| 3. | Excess uplift | Mass concrete dam in stability |
| 4. | Mass concrete dam instability | Structural Inadequacy |
| 5. | Structural Inadequacy | Dam Failure |
| 6. | Dam Failure | Flooding |

The dam Safety Review, Standing Operating Procedures, Operation and Maintenance Manuals, and Tender Documents are developed to ensure that the risk of a path to failure developing is minimised. In the example of a path to failure given above the dam safety review requires that an engineering evaluation of the uplift pressures is undertaken to asses whether the grout curtain is continuing to be effective. The difference in piezometric level upstream and downstream of the grout curtain are reviewed to assess trends. The dam safety review also evaluates uplift pressures and their effect on dam stability as related to the original design concepts and current practice to ensure that the dam is structurally adequate.

The dam safety review must therefore include evaluation of:

- design and construction records
- instrumentation data
- operation and maintenance methods and records
- changes in site conditions (such as tree growth or corrosion)

The paths to failure indicated in Appendix D are a formalised presentation of the identified factors that could lead to dam failure. This presentation does not aim to allocate the risk or probability of the incident causing a dam failure. It is only intended that all the potential paths to failure are identified.

This dam safety review has been completed to evaluate the current condition of the dam with special consideration of the potential paths to failure.

## 7 EMERGENCY PREPAREDNESS

(a) Hazard Classification

The dam is a referable dam with a high hazard classification.

(b) Access to the Site

All weather access is available to the dam but is subject to flooding at Reedy Creek crossing and at other points along the access roads.

During major flood events such as the probable maximum flood access to the dam would be limited due to the high tailwater level from Wivenhoe dam. Access via Kilcoy is also likely to be restricted by flooding in Kilcoy. A detailed evaluation of bridge inundation risk is required and other means of access to the dam during major floods require investigation to ensure that the dam operation is able to be maintained for a major flood event.

(c) Communications

Communications are considered to be adequate for most emergencies. Communication consists of:

- 2 normal telephone lines
- Radio (BCC & SEQWB)
- Mobile phones
- Power station radio link with SEQWB

(d) Warning System

No system has been established.

(e) Auxiliary Power

As indicated in Appendix A:

- the auxiliary power system should be located above the area that may be flooded during a probable maximum flood

- the auxiliary power plant is aging and replacement should be considered

(f) Remote Operation

Remote Operation is discussed in Appendix A. The current operation is entirely manual. It is recommended that the spillway gates be decommissioned and therefore remote operation is not necessary and the sluice gates operation be modified to permit remote operation.

(g) Security of the Site

The South East Queensland Water Board is currently reviewing its security provisions at the site.

(h) Reservoir Evacuation Potential

The eight sluice gates were constructed with an invert level of RL 71.2 (240 ft). This means that the reservoir may be able to be drawn down to about RL 73 with the sluices. This level is expected to be acceptable for most emergency conditions that could develop at the dam.

(i) Operating Instructions

The operating instructions are being upgraded.

## 8 HYDROLOGY

In August 1990, the South East Queensland Water Board commissioned the Department of Primary Industries, Water Resources Business Group to undertake the Brisbane River and Pine Rivers Flood Study. This work involved:

- hydrologic review;
- flood operating procedure;
- hydraulic analysis, flood studies;
- dam break (failure) analysis;
- flood inundation

### (a) Probable Maximum Flood

Under normal spillway gate operation Somerset Dam is capable of passing a Probable Maximum Flood if the gates are operated in accordance with the procedures defined in the Manual of Operational Procedures for flood mitigation for Wivenhoe Dam and Somerset Dam (SEQWB 1992) and ignoring wind set up and wave run-up.

The table below indicates the peak conditions resulting from a PMF at Somerset Dam as derived by DPI WRC (1994).

| | **All gates operating** | **One spillway radial gate or low level sluice gate out service** |
|---|---|---|
| Peak reservoir water level | RL 110.4 | RL 110.7 |
| Peak spillway flow | 8140 $m^3/s$ | 7950 $m^3/s$ |

The non overflow crest level is RL 107.46. This flood would therefore result in up to 3.2 m of water overtopping the non-overflow crest. This may cause flooding of the galleries, power station and erosion near the downstream toe of the dam. Tension stresses in the upstream face of the dam have been calculated during a PMF. These would be made worse as a result of flooding of the gallery. Erosion of some parts of the downstream foundation area is likely and this could lead to failure of the dam if the overtopping lasted for long enough.

The notional probability of exceedance of the PMF as determined by the DPI-WR is 1 in 1 000 000.

### (b) Dam Break

The two dam break failure modes considered by the DPI WRC (1994) were:

- Sunny day failure - sudden and complete removal or opening of two or more of the low level sluice gates.
- Overtopping failure resulting from two or more spillway radial gates remaining closed during a flood and/or failure of two central concrete gravity monoliths down to RL 100.

The DPI-WR study found that the incremental effects of a dam break failure of Somerset dam are relatively minor. The worst impacts would be in the reach of Stanley River immediately below Somerset dam.

The worst conditions occur when an overtopping dam failure of Somerset coincides with flooding in Wivenhoe dam. Under this condition Wivenhoe could discharge flows of sufficient magnitude to cause major flooding in urban communities.

## 9 GEOLOGICAL FEATURES

### 9.1 Regional Geology

The geology of the area is fully discussed in the "Geology of Ipswich and Brisbane 1:250,000 Sheet Areas" by L.C. Cranfield, H. Schwarzbock and R.W. Day, Report No.95, Geological Survey of Queensland, Department of Mines, 1976. In 1979 the Department of Mines published the Caboolture Sheet (9443) geological map at a 1:100,000 scale, which covers the damsite and most of the reservoir area. The very northern part of the reservoir is covered by the Nambour Sheet (9444) 1:100,000 scale, which was published in 1977.

The Geological Survey of Queensland indicates that the Somerset Dam and reservoir area are located on two structural blocks: the Northbrook Block and the Esk Trough (Figure 3).

**Northbrook Block**

This Northbrook block underlies the eastern parts of the reservoir area and comprises Northbrook Beds as shown on Figure 4. This name was used by Bryan and Jones (1944) for the fossiliferous marine Permian rocks identified by Ball (1934a). The Northbrook Beds are composed of volcanic conglomerate, arenite, shale with interbedded siltstone, chert, and volcanics, mainly andesite and andesite tuffs, with minor dacitic flows and tuffs. In the reservoir area the unit consists mainly of volcanics. The dominant volcanic rock is a green and light brown andesite and andesite tuff. Dacitic flows are commonly buff coloured. Conglomerate beds consist of pebbles of volcanics consisting of 90% of the rock in a coarse arenaceous matrix. The siltstone is light greenish grey and massive. The arenite consists of quartz and feldspar set in a dark grey matrix. (Geology of the Ipswich and Brisbane 1:250,000 Sheet Areas.)

In the west, the Northbrook Beds are faulted against the Neara Volcanics of the Esk Trough along the Eastern Border Fault, which is part of the Great Moreton Fault System. The unit trends north-northwest and dips from 25° to 80° to the west. To the west of Somerset Dam this unit appears to be unconformably overlain by the Bryden Formation. The age of the Northbrook beds cannot be determined more precisely than Permian because of poor preservation of the contained faunas.

**Esk Trough**

The Esk Trough comprises a thick sequence of continental sediments and andesite volcanics. The Esk Trough is divided into three units: the Bryden Formation (base), the Neara Volcanics, and the Esk Formation (top) as defined by Cranfield and Schwarzbock in 1972.

The Bryden Formation outcrops in the north-western part of the reservoir area and unconformably overlies, the Northbrook Beds.

The Bryden Formation consists of alternating pebble to cobble conglomerate, sandstone and shale. The conglomerate is green-grey and consists of chert, sandstone and minor plutonic and volcanic pebbles, set in a clay matrix. The sandstone is greenish, fine to medium grained. The shale is dominantly olive-green. In the vicinity of Somerset Dam, the unit dips between 20° and 65° on the flanks of a westerly plunging anticline.

The Bryden Formation is conformably overlain by and in the upper part interfingered with the Neara Volcanics.

The Neara Volcanics outcrop in the southern and south-western parts of the reservoir area, and consists of andesite pebble to boulder conglomerate, andesite flows and agglomerate, rare acid and intermediate tuff, sandstone and shale. Generally volcanic conglomerate at the base interfingers with conglomerate of the underlying Bryden Formation. This is overlain by andesite flows. Shale occurs towards the top of the unit. The unit forms a north trending belt up to 8 km wide and is folded into broad, open anticlines and synclines with north to north-west trends. In the east the unit is faulted against the Northbrook Beds along the Eastern Border Fault. At Somerset Dam the unit is intruded by the Triassic Somerset Dam Igneous Complex. The presence of a thick sequence of pyroclastics in Somerset Dam area (Little Mount Brisbane) suggests that this was a centre of eruption. The Neara Volcanics are of early Middle Triassic age. (Geology of the Ipswich and Brisbane, 1:250,000 Sheet Areas.)

In the reservoir area the Esk Formation was completely eroded and therefore is not described further in this report.

**Somerset Dam Igneous Complex**

The gabro-granophyre association (granophyre = microgranite which displays granophyric texture) called "The Somerset Basic Intrusion" by Mathison (1967), is situated to the west of Somerset Dam township and forms high rugged terrain, 470 to 670 m above sea level, and covers an area of about 10 km$^2$. The unit intrudes the Triassic Neara Volcanics. The northern part of this complex is mainly composed of granophyric intrusions, forming a large irregular mass, as well as small satellite bodies. The rock appears to be either of equigranular or porphyritic texture.

The basic part of the complex is composed of at least 20 saucer-shaped layers of olivine gabbro, leucogabro, troctolite and ferrigabbro (Mathison, 1967). The colour of the gabbro varies from a light coloured massive leucogabbro to a dark grey banded troctolite.

The zone of contact metamorphism is much narrower around the granophyre than that around the gabbro.

Felsite and quartz diorite dykes are numerous in this area, and are probably related to the later stages of crystallisation of the gabbros (McLeod, 1959). The quartz diorite dykes are mineralogically similar to the upper parts of the gabbro, while the felsite dykes are similar to the main mass of the granophyre.

The radiometrical tests carried out on samples of the Somerset Dam Igneous Complex gave ages of 207 to 213 ± 5 my, Middle to Late Triassic age. This age anomaly was explained by periodic magma renewals within the intrusion. (Geology of the Ipswich and Brisbane, 1:250,000 Sheet.)

**Alluvial Deposits**

A significant amount of alluvial deposits adjacent to the mouth of Oakey Creek (oldest terrace), in Oakey Creek and in Splitters Creek, are indicated on Caboolture Sheet 1:100,000 scale. These deposits are mainly composed of gravel and sand in the oldest terraces, and of gravel, sand, silt and clay in recent river deposits.

## 9.2 Geology of the Damsite Area

The only available information regarding engineering geological and geotechnical investigations at the dam site were a geological map of the damsite area (reproduced at 50% of original scale in this report Figure 5), and one page of Geology of the damsite, written by an unknown author, and which refers to the Bell's "Proceedings of the Royal Society of Queensland, Vol.L11, Part 1, P.14 (1940).

The geological map (Figure 5) was produced by C.W. Ball B.Sc., dated 12/10/37. The rocks which underlie the Somerset damsite area belong to two geological units: Neara Volcanics and Somerset Dam Igneous Complex.

### Neara Volcanics

The Neara Volcanics is the intermediate unit of the Esk Trough Formation, conformably overlying and in basal parts interfingering with the Bryden Formation, the base unit of the Esk Trough. In the damsite area this unit is represented by various textural forms of andesites and possibly trachytes. Augite andesite and porphyritic andesite (on Bell's map: Porphyrite, Angie Andesite and Angie Porphyrite), fine grained intermediate volcanic igneous rocks, form most of the damsite area.

Augite Andesite is more basic because of the presence of augite and plagioclase, while the porphyritic andesite is less basic with visible feldspar phenocrysts (light colour minerals). Andesite has an average specific gravity of 2.8, with silica percentage between 55% and 65%.

In his map Bell indicated the presence of andesite and trachyte agglomerate associated with andesite and trachyte flows. A geological reconnaissance around the dam, indicated an insignificant amount of agglomerate, either andesite or trachyte agglomerate, in existing road cuttings and outcrops. In the quarry, some 180 m to the north of the dam, probably less than 5% of the face shows agglomerate pockets of either the andesite or granite type. Therefore, it is believed that Bell probably interpreted porphyritic andesite as either trachyte or andesite agglomerate (in some cases these are very similar to agglomerate).

The andesite flows of the Neara Volcanics, conformably overlying the Bryden Formation were intruded by coarse grained igneous rocks of Middle to Late Triassic age, recognised as the Somerset Dam Igneous Complex. In the damsite area this unit is composed of granite and diorite.

Granite, on Bell's map called Alaskite the petrological name for granites consisting only of quartz and alkali feldspar, is coarse grained acid igneous rock with specific gravity 2.4 to 2.7 and more than 65% of silica.

Diorite, intermediate even-textured coarse grained rock with quartz and plagioclase as dominant feldspar, has an average specific gravity of 2.8 t/m$^3$ and silica percentage normally between 55% and 65%.

Both the diorite intrusions and the andesite flows (at the later stages of crystallisation of the Somerset Dam Igneous Complex, particularly crystallisation of the gabbros), were intruded by steeply sloping pink felsite dykes, which vary in width up to 3 m. Felsite is a petrological name for a fine, evenly grained acid or intermediate rock. At the dam site, the dykes and veins form in both the country rock and in the parent plutonic mass.

**Structures**

In the Somerset Dam area the Neara Volcanics strike north north-east and dip towards the west south-west at 10° to 30° (see Figure 3).

There are three major joint sets developed in andesite flows, described by Bell. One series of joints are dipping at 10° towards west-southwest, or gently in a downstream direction, and two subvertical sets striking north-south and east-west.

Subvertical joints (called "Master Joints") were exposed in several sections of the foundation excavations, and were longer than the width of the dam base. Engineering properties of these obviously continuous joints are not known. In only one sentence of the available report the subvertical joints are described as plane like joints, (currently called planar joints) and the gently downstream dipping joints are described as more continuous but irregular and rolling (curved).

### 9.2.1 Foundation and Abutments

The foundation rock was described as a dark grey andesitic prophyrite (ie. porphyritic andesite). This is the only description of the foundation rock, and it is not known whether the foundation excavations were mapped in detail.

A number of continuous discontinuities were exposed in the foundation segments, with a very short and poor description of their characteristics.

The final foundation was obtained by picks, bars and sledge hammers working on the surfaces of the discountinuities. It is reported that: "In general the joint surfaces were discontinuous and the final foundation had a chunky appearance from the almost perpendicular steps, usually about 12" high." (Note "in general".)

The most extensive discontinuity was exposed in the left abutment from Monolith U to Monolith Q, along the downstream toe of the dam. A number of joints perpendicular to the dam axis were confined by this joint.

A continuous discontinuity striking N-S across Monolith S, extending from the upstream excavation to the continuous discontinuity mentioned above was also exposed during the foundation excavation.

The final surface of foundation excavation for the downstream toe of Monoliths N and O, was developed along a smooth continuous discontinuity dipping in a downstream direction (the angle is not measured). To prevent sliding of the dam the foundation surface was drilled at intervals (no detailed description) and short lengths of old drill steel were grouted into the holes.

During excavation for Monolith G, a flat continuous water bearing seam at a level of about RL 61 was exposed. The seam was about 75 mm thick and was composed of a small andesite particles embedded in a clay matrix. The seam was sealed by driving a tunnel along it for 36.5 m under the abutment and filling the tunnel with concrete and grouting. The detailed description of this work is not available.

A series of subvertical discontinuities striking E-W were revealed during the excavation of the upper portion of the foundation for the left abutment. To prevent the leakage from the reservoir, the discontinuities were sealed by constructing a concrete filled tunnel excavated at right angle to the strike of the discontinuities. There is no detailed description of this work.

### 9.2.2 Grouting

A summary of the impact of grouting is included in Section 10.3 of this report.

### 9.2.3 Evaluation

The available information regarding geological features produced by the geologists and related engineers prior to and during the construction of the Somerset Dam, is very limited. Only the geological map of the dam site area produced by C.W. Ball and the two page report written by an unknown author are available.

In light of all of this the following can be concluded:

(i) The significance of the contact between the andesite flow and the underlying diorite probably was not understood. If the dam was completely founded on the andesite, then how far from the foundation excavation is the contact, and what are the characteristics of this usually soft (extremely weathered) zone.

(ii) A number of felsite dykes shown on the map in the area of the foundation excavations are not mentioned in the report.

(iii) Significant continuous discontinuities were exposed during foundation excavations. Of particular importance to the stability of the dam is a set of discontinuities dipping at 10° towards the west, south-west, or gently in a downstream direction. The geotechnical parameters of these and other important discontinuities are not known. For this report it has been assumed that the ultimate strength of this set of discontinuties is greater than the equivalent of C=1400kPa & $\phi$=45°. This primary assumption of shear strength is based on the ANCOLD guidelines which states that "in the absence of more reliable data preliminary analysis of foundations on sound jointed hard rock where sub-horizontal joints are not continuos" the above strengths can be adopted. The impact of the sub-horizontal joints is not known until investigations have been carried out. To further assess the behaviour on the joint it has been assumed that c = o and $\phi$ = 45°. It is essential that investigations and the lower bound strength is given by testing be undertaken to define the strength of the foundation

(iv) The treatment of a flat water bearing seam encountered in Monolith G, and a zone of E-W striking subvertical discontinuities intersected in the upper portion of the left abutment, as well as other important defects in the rock mass are not detailed.

### 9.2.4 Reservoir

A reconnaissance of the lake found no indication of any significant landslides, soil creep or any other natural movements of soil or rock slides were found. Also, there are no records of any landslides occurring after the construction of the dam. A minor scree slope has developed upstream of the dam on the Left Abutment.

## 10 STRUCTURAL FEATURES

### 10.1 Seepage

(a) General

The dam was constructed by reference to the "N line" shown on the drawings at the upstream edge of the vertical portion of the dam above RL 100.45 (330 ft) (Figures 1 and 6).

The drainage system as shown on Figure 6 consists of:

- foundation drain holes reported by Richard (1949) to be at 3 m centres and drilled 9 m into the rock and located 4.5 m downstream of the upstream heal. The drawings indicate that the holes were drilled to RL 45.6 and 3 m downstream of the 'N line'.

- a brick foundation drain 0.38 m x 0.38 m (15" x 15") constructed on the foundation and located 4.57 m (15 ft) downstream of the "N line".

- an upstream foundation tunnel 0.76 m wide by 1.83 m high located 12.19 m (40 ft) downstream of the "N line".

- a brick foundation drain 0.38 m x 0.38 m constructed on the foundation and located 24.38 m (80 ft) downstream of the "N line".

- a downstream foundation tunnel 0.76 m wide by 0.91 m high located 39.01 m (128 ft) downstream of the "N line" in Monoliths H to Q inclusive.

All of the seepage water from the under drains, drains to sumps in Monoliths H and Q on the right and left abutments. The sumps are connected by a drainage tunnel and the sump on the left abutment is lower than the sump on the right abutment. Pumping from the left abutment is generally the only sump water is pumped from.

A 150 mm (6") diameter drain has been provided in Monolith H and Q for drainage of water pumped from the sumps and released into the stilling basin. This drain has flaps on the downstream end to prevent water entering the gallery when the tailwater level is high. The drain outlet has been constructed at RL 64.5 (212.2 ft). This is 2.5 m below the FSL of Wivenhoe and therefore it is now generally below the tailwater level. Water is not reported to back up this pipe into the gallery which indicates that the flaps are currently effective in keeping water out of the gallery. These must be regularly maintained to ensure that they are effective.

The sumps pumps are operated weekly to ensure that they are operating correctly. The procedure adopted to operate the pumps requires that water is put into the sumps until the automatic trip is activated and the pumps operated till the lower limit of the automatic trip again operates. No monitoring of the water quantities is undertaken. Observations by operating staff indicate that some time ago the seepage level in the upstream foundation tunnel was generally about half the height of the tunnel. Now it is kept at a much lower level. The time required to pump the water out of the sump is monitored. A plot of this is indicated in Figure 5. No conclusions can be drawn from this plot possibly due to the quantity of water added to activate the pumps.

(b) Within the galleries

Within the galleries there is generally very little water flowing in the side drains. In August 1962 GA Cowling reports that "all the expansion joint drains with the exception of the K/L drain, were weeping into the lower gallery. All the vertical foundation drains with the exception of the one measured in monolith J were weeping into the lower gallery". In February 1995 only monolith U is reported to have drains full. Discussions with operating staff suggest that one of the drains may flow under pressure when the reservoir level is high. Some of the drains have become partially blocked where they enter the gallery.

(c) Stilling Basin Retaining Walls

The weep holes on the right hand side wall of the stilling basin indicated that water is collecting behind the wall. It is not known whether this is from seepage or rainfall.

### 10.2 Review of Design

The basis for the design of the dam is detailed in the calculation files and summarised in a letter from the Chief Engineer to HH Dare on 5 March 1940. This states that:

> **"Stability of Monoliths as Designed**
>
> The stability of monoliths is dealt with in my report to the Board "Design of Dam and Progress of Works" dated 18 January 1938, of which you should have a copy. (No copy has been found.)
>
> Briefly stated, the basis of design was that with an uplift coefficient of 0.70 (uplift varying linearly in intensity from headwater and tailwater but acting on 0.70 of base area), there should be no tension and the sliding factor (ratio horizontal to vertical force) should not exceed 0.67, this being adopted as the coefficient of friction of concrete on rock.
>
> It was found to be impossible to keep the sliding factor down to 0.67 without assuming portion of the thrust to be resisted by shear in the concrete of the cut-off wall and the triangular extension of the cut-off wall.
>
> The designed values of these quantities are:-
>
> **37' 4" Monoliths J and O**

| | |
|---|---|
| Coefficient of uplift just producing no tension with dam full and tailwater. | 0.81 |
| For uplift 0.81 and sliding factor of 0.67, shear in concrete is. | 27.5 lbs/sq inch |
| For uplift 0.70 and sliding factor 0.67 | 22.5 lbs/sq inch |

> **Central Monoliths L and M**

| | |
|---|---|
| Coefficient of uplift, for no tension | 0.85 |
| Shear in concrete, uplift 0.85 | 28.5 lbs/sq inch |

| | | |
|---|---|---|
| Shear in concrete, uplift 0.70 | less than | 22.5 lbs/sq inch |

**26' 0" Monoliths I, K, N and P**

| | | |
|---|---|---|
| Coefficient of uplift for no tension | | 0.71 |
| Shear in concrete, uplift 0.71 | | 30.1 lbs/sq inch |
| Shear in concrete, uplift 0.70 | less than | 30 lbs/sq inch" |

Detailed results from calculations completed during the design are included in Appendix C.

### 10.3 Review of Construction

Details on construction quality control is limited. The sources of information on construction consist of:

- Somerset Dam - booklet
- Foundation Grouting at Somerset Dam by EL Richards IE Aust May 1949
- Results from tests on cement used in the dam

(a) Grouting

Foundation grouting was completed from the lower gallery. Richards (1949) reports that grout holes at 1.5 m centres and 9 m depth into the rock were drilled. Some holes were drilled to 18 m depth. The drawings indicate that the drilling was to RL 45.6.

Grouting was undertaken at a pressure of 1,380 kPa at the gallery level which was 10 m above the foundation in the spillway portion of the dam.

The grout mix used varied from 1 to 5 to 1 to 10 by volume with 2% bentonite.

The amount of cement injected into any one hole varied from nothing to 12 tonnes at the first grouting in any one stage. The average rate of take of cement was 23 kg/m. For those stages that actually took grout the average take was 53 kg/m.

The grouting was undertaken with the aim of reducing the uplift pressures beneath the dam. It is now generally accepted that a single line grout curtain can reduce the rate of loss of water through the foundation but has a limited influence on the uplift pressures. The drainage system has the greatest influence.

The general impression gained by Richards (1949) from the grouting was that generally the "grout was injected along a series of semi-parallel surfaces, which were generally parallel to the excavated surface". There was also a series of steeply sloping fissures roughly perpendicular to the other fissures.

The great majority of holes at Somerset gave water test results of less than 1 Lugeon when tested before grouting. It was expected that very little cement would be needed to seal the foundation and this was found to be the case.

During most of the grouting the lake level was at RL 88.3 (290 ft) - (2/3 full).

Two holes were drilled during construction to determine the effectiveness of the grout curtain. The reservoir level during this testing was RL 88.0 (289 ft). Hole 137 was drilled upstream and hole 138 was drilled downstream. At the upstream heel of the dam at the concrete-rock joint the water pressure measured was equivalent to RL 86.9 m (285.4 ft) at a depth of 9 m depth (bottom of grout curtain) the measured water pressure was equivalent to RL 84.3 m (276.9 ft). The method used to isolate the lower part of the hole is not indicated in the report, and it is likely that the pressure measured was the average for the full depth of the hole. The downstream hole had a water pressure less than the invert level of the gallery level.

This test was assessed by Richards (1949) to indicate that:

- the area under the cut-off was jointed before grouting and that the grouting had sealed the joints.
- the effect of the grouting was confined to a comparatively narrow band surrounding the line of the grout holes because both the test holes passed through jointed and unsealed rock.

Richards also reported that:

"The practice of drilling and grouting every second hole before the intermediate holes gave evidence of the sealing effect of the grouting, although frequent anomalies showed that the 1.5 m (5 ft) spacing adopted for the grout holes was not too close."

High uplift pressures were measured in the grout holes after grouting.

(b) Uplift

Pressure measurements (method unknown) made in about September 1946 (Richards) indicated that about $^1/_3$ of the holes at the grout curtain had water pressures greater than would be expected if "a straight line distribution of pressure from reservoir level to tailwater level at the line of the brick drains" is assumed.

(c) Concrete

Coarse aggregate for concrete was obtained from a quarry about 500 m from the dam. Sand was obtained from natural deposits in the Stanley River and nearby streams.

Queensland cement was used throughout.

All concrete was batched by weight. Two sizes of coarse aggregate with a 75 mm maximum size and one of sand was used. The body of the dam was constructed with concrete containing 200 kg/m$^3$ cement (4 cubic feet bags per cubic yard and adopting cement 94 lb/ft$^3$ as indicated by Richard 1949). Near the face of the dam the concrete contains 299 kg/m$^3$ of cement (6 cubic feet bags per cubic yard).

Hand written notes included in the files in about 1940 indicated test results on concrete as follows;

- No 10 from Q/57 from top of unit 2.46 t/m$^3$ (153.8 lb/ft$^3$)
- No 7 from R/61260 from near bottom of unit 2.52 t/m$^3$ (157.6 lb/ft$^3$)
- No 9 from P? from near bottom of unit 2.52 t/m$^3$ (157.1 lb/ft$^3$)

In February 1987 the dry density and saturated density of concrete cores was measured on five samples of core - presumably from the test holes drilled into the upper gallery cracks. The data from these tests is indicated below:

| Core No | Dry Density (kg/m$^3$) | Saturated Density (kg/m$^3$) |
|---|---|---|
| 1 | 2 350 | 2450 |
| 2 | 2 480 | 2550/2580 |
| 3 | 2 440 | 2520 |
| 4a | 2 440 | 2520 |
| 4b | 2 440 | 2520 |

A concrete testing laboratory is reported to have regularly examined concrete samples for compressive strength and permeability. Only the results of strength tests on the cement have been found at this stage.

Very dry mixes were normally used, and all concrete was placed with the help of vibrators. Anecdotal evidence suggests that when there was no direct supervision the concrete may have been placed with a high water cement ratio.

Provision for grouting of the Monolith joints is indicated on the Drawings, however Shepherd (1976) indicates grouting of the joints was not undertaken.

(d) Foundation Excavation

The following description of the foundation is provided by Richards (1949):

> "The foundation rock is a dark grey andesitic prophyrite, intruded by steeply sloping quartz-felsite dykes which are up to 10 feet wide. The rock is very hard and weather resistant and forms an excellent foundation. Three series of joints occur in the prophrite. One series dips gently (10°) in a downstream direction and the other two series are semi vertical and strike roughly N-S and E-W. The axis of the dam lies approximately in a N.W.-S.E. direction. The joints are usually very tight, although water staining was noticed in diamond drill cores as deep as 50 feet below the original stream bed. The percentage of core recovery was very high, over 95%, and shows that no great weathering had taken place at the joint surfaces. Occasionally the joints showed a hard thin calcite lining and occasionally the driller reported "Open joint" but no estimate could be made of the width of such openings.
>
> The joint surfaces acted as planes of cleavage during excavation and the final foundation surface was prepared for concrete by working up to them with picks and bars. The joints also form the only possible path for percolation of water beneath the dam and therefore the aim of the grouting programme was to seal up them as close to the upstream face of the dam as possible."

(e) Evaluation of Construction Standards

On the basis of the documents listed we are of the opinion that:

- the dam was constructed on a jointed rock foundation and significant effort was made to ensure that weak zones were removed.
- the grouting was completed in a professional manner and was successful in reducing the rate of seepage.
- the designers understood the importance of good drainage.
- the concrete was placed with no control on placing temperature and cracking of concrete resulted both during construction and after construction.
- the coarse concrete aggregate consisted of a basic rock which is unlikely to be subject to alkalai-aggregate reactivity. The nature of the sand however, is unknown.
- the concrete in the core of the dam contained about 200 kg/m$^3$ of cement.

This information suggests that standard rock strength and concrete parameters can be adopted for analysis. However, the existence of shallow dipping planar joints in the dam foundation and the lack of "physical" data requires that investigations be undertaken to determine the actual parameters of the concrete and rock at the dam.

## 10.4 Evaluation of Existing Condition

### 10.4.1 Inspection

On the 6 March the dam was inspected. Appendix B is a photographic record of various features at the dam. The primary features of the dam and their current condition are:

(a) Upstream Face

A horizontal crack was observed on the left bank in Monolith V and penetrated up to the bulkhead gate storage building. This crack is located at about RL 100.4. On the downstream face the 0.75 H:1V slope begins at about this level. At the time of the inspection the crack was about 1 mm wide.

Cracking was also noted in the spillway gate piers at about RL 100.5 on the right abutment above the intake to the regulators.

(b) Downstream Face

A horizontal crack was observed at the gallery portal on the left bank. This crack is a continuation of the continuous crack found in the upper gallery.

Surface cracking is presenting as a "crazing" pattern on the top of gate piers. The cracks in this pattern extends about 100 to 150 mm in depth when they exit at corners in the concrete. The pattern is coarse with cracks about 70 to 100 mm apart. This may be a sign of aging of the concrete, temperature effects or due to alkali aggregate reactivity.

(c) Galleries

The upper gallery (floor EL 89) has a continuous crack on the downstream wall at the roof fillet/wall corner. The crack runs for the full length of the gallery and along the entry adits. The crack width varies up to about 3 mm in width. The upper part of the crack appears to have displaced approximately 1 mm upstream relative to that part below the crack. Cored holes were drilled in 1979. The 100 mm diameter holes were drilled to a maximum depth of 1.3 m and it was reported that the crack width was uniform in width over this distance. The cracks were felt up to "arm" length (approximately 0.6 m) and some lateral displacement (left to right) may have occurred. This is difficult to tell by "touch" but needs checking. If this is so the movement has occurred since 1979.

The contraction joint between monoliths L and M (near the centre of the river and where the pier is twice the pier thickness of all others) has opened about 30 mm near the dam crest. The copper waterstop in the upper part of the dam (above the top of the gates) is torn and "daylight" can be seen looking through the joint. In the upper gallery the joint is much less open as indicated in the photographs and in the lower gallery the joint is closed and possible compressed.

(d) Abutments

(i) Left Abutment

On the upper upstream left abutment some 50 m upstream and 30 m above the dam crest a minor rockfall/surface scree is evident. This does not appear to pose a threat to the dam, as massive rock outcrops between it and the dam.

Downstream of the dam on the left abutment there is some evidence of seepage. This may be caused by surface run-off rather than seepage.

(ii) Right Abutment

Good massive rock in outcrops occurs above the access road and along the downstream toe of the dam.

(e) Spillway

Concrete in the spillway is in moderate condition with no evidence of cavitation damage.

The gate pier on the right hand side of monolith M has been cut back at the top to maintain clearances for the gate. Gil Verrenkamp reports that the problem with this gate was first noted in the period 1986 to 1988. Gouging of the concrete below the cut back suggests that further movement may have occurred. It should be noted that in early 1986 a magnitude 3 earthquake was recorded as occurring close to Somerset dam.

The dissipator basin was not drained for the inspection. There are no records of the dissipator being inspected. A detailed inspection is required.

(f) General Comments

The probable maximum flood will result in water flowing over the abutment monoliths "non over toppable" portion. If this were to occur water would enter the galleries through the access door, gate shafts and gallery access adit. The drainage system would be flooded and pressurised. A bulkhead gate has been provided to prevent this occurring. It is essential that site staff are made fully aware of the purpose of this bulkhead gate.

### 10.4.2 Dam and Spillway Monitoring

No instruments were included in the dam for monitoring its behaviour.

Crack opening measurements across the cracks in the gallery have been monitored since about 1979. This monitoring has indicated that the crack opening is affected primarily by the seasonal variation in temperature. An analysis of the graphic presentation of crack with variation by R Russo in 1994 suggests that the crack is opening at a rate of 0.1 mm/year.

Water levels in the drainage holes were measured in August 1962. At that time all expansion joint drains except K/L were weeping into the lower gallery and all the vertical foundation drains with the exception of monolith J were weeping into the lower gallery. In 1989 the water in the holes was lowered and the time for the water level to recover monitored. At that time some drains in Monoliths H, M, N and O were not flowing, a drain in Monolith J was not flowing and the remaining drains indicated various rates of recovery. From this it appears that some of these drains have become blocked since 1966.

Uplift monitoring by measuring the water level in the drainage holes leading to the brick drains commenced on a regular basis in 1987. In 1994/5 only monolith U had upstream drains that were full.

From 15 December 1983 to the present, the time taken to pump out water from the sump in monolith Q is measured. Results of this monitoring are plotted in Figure 5 and indicate no clear trends. This could be because the system used to test the pumps each week involves filling the sump to trip the automatic switch on the pumps.

A precise survey of monuments on the dam was undertaken in 1982 but no follow up survey has been undertaken.

### 10.4.3 Stability Checks

(a) Definitions and Analysis Method

The safety of the Somerset Dam is assessed by re-evaluating the stability of the structure using current design criteria. The current Australian method of design of concrete gravity dams uses the ANCOLD publication "Guidelines on Design Criteria for Concrete Gravity Dams" (1991) along with recommendations from the international organisation ICOLD.

The loads used in the analysis of a concrete gravity dam comprise three categories:

- Normal Loads
- Extreme Loads

Normal loads are those loads which the structure is likely to experience during the expected life of the structure. Such loads include the normal reservoir loads, significant but not extremely large floods and small earthquakes such as the operating basis earthquake. The dam must be designed to withstand these loads with no damage to any part of the structure or appurtenant works.

The Operating Basis Earthquake (OBE) represents the maximum earthquake that is likely to occur during the economic life of the structure. Under this event the dam and its appurtenant structures must remain undamaged and operable.

Extreme loads represent those loads which may conceivably be imposed on the structure but have an extremely low probability of occurrence and thus warrant the use of a low factor of safety. The structure may sustain damage but must be designed to prevent an uncontrolled loss of the reservoir under these loads. Loads which fall into this category are the Probable Maximum Flood (representing the maximum possible flood load) and the Maximum Design Earthquake (representing the maximum earthquake likely to be experienced in the area).

Under current design criteria the dam must have sufficient resistance to prevent failure by:

- overturning
- rupture
- or sliding.

Under the 1991 ANCOLD "Guidelines on Design Criteria for Concrete Gravity Dams" the criteria above are checked by using a limit state analysis method. Structure loads are factored toward the conservative by a ratio reflecting the certainty of the magnitude of the applied load and the probability that the load will be applied. For example the dam weight is factored by 0.95 under normal load cases reflecting the permanent nature of the load and the stabilising effect that the load has. Strengths of the dam and foundation are reduced by a factor reflecting the probability of occurrence of the load case being analysed and the degree of certainty of the strength parameter used. For example the frictional strength of the joint or foundation under analysis is reduced by a factor of 0.3 under normal load cases unless there are tests on the strength parameters which allow you to argue for a smaller reduction.

The ANCOLD factors used for this analysis are tabulated below.

| Load | Normal Load | Extreme Load |
|---|---|---|
| Dam Weight | 0.95 | 1.0 |
| Reservoir vertical force | 0.95 | 1.0 |
| Reservoir horizontal force & uplift | 1.05 | 1.0 |
| Tailwater forces | 0.95 | 1.0 |
| Silt force | 1.5 | 1.0 |
| Concrete compressive strength | 0.4 | 1.0 |
| Joint or foundation friction | 0.3 | 0.8 |
| Joint or foundation cohesion | 0.3 | 0.8 |

To check the resistance of the dam against rupturing and sliding the ratio between the factored shear strength and factored resultant horizontal force is calculated. This assesses the available frictional resistance and cohesion along the foundation as a ratio of the applied sliding forces. For adequate stability this factor must be greater than 1.0.

To check the stability of the dam against overturning the sum of moments about the dam toe are calculated. To remain stable the dam must exhibit a net stabilising moment.

The earthquake stresses can be calculated with a very rigorous finite element method or using well established simplified methods. The finite element method is expensive and time consuming as it requires a detailed model to be constructed of the dam and its foundation. This model is then subjected to a numerical representation of an earthquake record (for a time history analysis) or more often is analysed for significant resonant behaviour (modal analysis). A modal analysis requires combining each resonant frequency to give an approximation of the stresses expected under seismic loads. There are various simplified methods used which approximate the modal analysis method but do not require the construction of a rigorous mathematical model. The method used in this study is from a well recognised Earthquake Engineering Research Centre (University of California, Berkeley) publication by Fenves and Chopra (1986) "Simplified Analysis for Earthquake Resistant Design of Concrete Gravity Dams". This method has had considerable correlation checking with sophisticated finite element work and is proven to be a reasonable approximation of expected seismic behaviour.

(b) Model Adopted and Loads Applied

To check the stability of the Somerset Dam under current design criteria two representative models were selected. One model was based on the spillway blocks of monolith K and monolith L. To check a representative non overflow block, monoliths Q and R were chosen. The highest block sections were used in the analysis as these generally are the least stable sections.

The load cases analysed are as follows:

(i) Normal Loads

- normal full supply level
- maximum historical flood level (1974 flood)
- operating basis earthquake. The operating basis earthquake is defined by ICOLD as the earthquake likely to be experienced during the life of the dam.

(ii) Extreme Loads

- probable maximum flood
- maximum design earthquake.

The first three cases above represent the normal loads of the structure. The maximum historical flood level was dependent on operating policy. Here the gates were closed during a significant flood event in 1974 causing the reservoir level to rise to RL 106.55 m (approximately 1 m below the non overtoppable crest level).

The extreme loads of the Probable Maximum Flood (PMF) and the Maximum Design Earthquake (MDE) represent the absolute maximum flood and earthquake that the structure be designed for to prevent an uncontrolled loss of reservoir storage. Other damage to the structure is permitted.

(c) Model Properties

The material properties of the dam have been chosen from a combination of guidelines from ANCOLD, internationally accepted industry standards and factual reports from Somerset Dam files.

| Property | Value adopted | Comment |
|---|---|---|
| Concrete density | 2.4 t/ $m^3$ | Mean dry density of concrete tests in 1987 |
| Concrete compressive strength | 20 MPa | Assumed |
| Allowable compressive stress | 8000 kPa normal | Assumed |
| | 20,000 kPa extreme | Assumed |
| Concrete friction strength | 45 deg | ANCOLD |
| Concrete cohesion | 1400 kPa | ANCOLD |
| Concrete tension | 0 kPa normal | ANCOLD |
| | 90 kPa servicibility | 0.2 * (fc')^0.5 * 1/10 - ANCOLD |
| | limit for unusal & extreme loads | |
| Dynamic tension capacity | 134 kPa OBE | ANCOLD 0.3 (fc')^0.5 = 1340 |
| | 1340 kPa MDE | |
| Foundation compressive strength | 20 Mpa | assume same as concrete |
| Foundation friction | 45 deg | ANCOLD recommendation for |
| Foundation cohesion | 1400 kPa | foundation of sound joined hard rock where sub horizontal joints are not continuous* |
| Foundation tension | 0 kPa | Assumed |

* The properties of the foundation are assumed. Analysis of sub-horizontal joints in the foundation has not been undertaken. Investigation are required to identify the location and extent of the sub-horizontal joints.

On the basis of experience ANCOLD have adopted an allowable tensile strength of zero kPa for normal loads the serviceability requirement of 90 kPa applies for unusual loads and for extreme loads. Somerset dam has a number of cracks through the dam. The allowable tensile strength must be conservatively evaluated. The cracking in Somerset dam may mean that the allowable tension be zero even for unusual and extreme loads. Investigations and testing is required to allow tension stresses greater than zero.

The lack of information available regarding as constructed properties of the structure has meant that all key properties were estimated. These estimations were made after considering industry accepted recommendations for design properties taking into consideration the typical construction procedures and specifications that would have been likely to have been used at the time the structure was designed and built. An assessment of local foundation strength is based on design of other projects on similar foundations.

The concrete density listed in the above table is taken from a memo on concrete densities noted in the design files and measured well after construction as indicated in Section 10.3(c) above. The range in concrete density measured in a small numbers of tests is relatively large so sensitivity studies were carried out to assess the effect of different concrete densities.

The range in concrete density measured in a small numbers of tests is relatively large so sensitivity studies were carried out to assess the effect of different concrete densities.

The tensile strength of the dam concrete under seismic loading is an area of ongoing research and an assessment of the "state of the art" allowable strengths has been made. The uncertainty arises from the lack of performance history of concrete dams under strong earthquake shaking and the uncertain behaviour of concrete gravity dams under large transient and cyclic loading. In a discussion of earthquake response of concrete gravity dams the effect of loading rate on concrete tensile strengths was investigated (Chopra, "Earthquake Response Analysis of Concrete Dams" in Advanced Dam Engineering for Design, Construction and Rehabilitation edited by R Jansen, 1988). A relationship between the concrete cylinder compressive strengths (as determined under normal quality control testing) and the allowable tensile strength was presented. A tensile strength under seismic loading was proposed of $2.6^{*}fc^{2/3}$ where fc is the standard cylinder compressive strength (in psi). A higher allowable tensile strength may be used to allow for non linear behaviour of the concrete at failure. In this case the strength can be increased to $3.4^{*}fc^{2/3}$.

A brief summary of key dimensions and reservoir levels of the sections modelled are included in the table below:

| Monolith | K/L | Q/R |
|---|---|---|
| crest RL | 100.5 m | 107.5 m |
| block height | 45.8 m | 52.8 m |
| crest width | ogee | 4.27 m |
| base width | 43.7 m | 41.9 m |
| D/s slope | 0.7 | 0.75 |
| U/s slope | 0.05 | 0.05 |
| Headwater Levels | | |
| Minimum Operating Level | 70 m RL | 70 m RL |
| Full Supply Level | 98.93 m RL | 98.93 m RL |
| 1974 flood | 106.55 m RL | 106.55 m RL |
| PMF | 110.7 m RL | 110.7 m RL |
| Upstream silt level | 70 m RL | 70 m RL |

The tailwater level to be adopted for analysis of stability is complicated by the presence of Wivenhoe. The FSL in Wivenhoe is RL67 and the crest of the saddle dams is RL79.0. The original tailwater curve is presented by R Russ in his 1988 Safety Review, this has been adopted as one limit for the tailwater levels. During the 1974 flood recorded water levels were about 5 m above the calculated tailwater level and this has been adopted as the upper limit for the tailwater level. The adopted tailwater levels are indicated below.

| Design Condition | Minimum Tailwater Level | Maximum Tailwater Level |
|---|---|---|
| Normal | 64.0[1] | 67.0[2] |
| 1974 Flood | 67.2[3] | 72.1[4] |
| PMF | 80.0[5] | 82.4[6] |

**Notes:**

1. Bed level downstream of stilling basin.

2. FSL in Wivenhoe.

3. Lowest tail water level reported by R Russo in 1988.

4. Maximum tailwater level reported by site staff during the 1974 flood.

5. Adopted tailwater level during restricted releases from Somerset Dam when water level is threatening to breach Wivenhoe saddle dams.

6. Water level extrapolated by R Russo with a flow of 7950 $m^3/s$.

The loads for the structure have been compiled from historical records for the dam, DPIWR PMF studies and the RMIT Seismicity Study for the dam site.

Normal operating levels and historical flood levels have been taken directly from historical records.

The PMF level has been taken from the DPIWR report "Brisbane River and Pine River Flood Study - Initial Draft Report" (December 1994). The PMF scenario where one of the spillway or sluice gates remains closed has been adopted. This is an increase of 0.3 m above that with all gates operating. This is in keeping with the high hazard nature of the dam where suitably conservative extreme load combinations are adopted.

The following uplift assumptions were made for the analysis:

| Load Case | Silt Level | Reservoir Water Level | Distance To Drains from heel (m) | Uplift At Drains | Tailwater Level |
|---|---|---|---|---|---|
| FSL | 70 | 98.93 | 5.33 | 75.64 | 64 |
| Max past flood | 70 | 106.55 | 5.33 | 83.58 | 72.1 |
| PMF | 70 | 110.7 | 5.33 | 90.23 | 80 |

There is no data available on the actual uplift being experienced by the dam. Uplift is a major loading on concrete gravity dams and should be measured to establish whether "normal" design assumptions are achieved.

The seismic loads have been obtained from Gibson and Cuthbertson "Review of Seismicity, Somerset Dam" (March 1995, RMIT and University of Queensland) for the area surrounding Somerset, Wivenhoe and North Pine dam. A rock site spectra has been used to assess the spectral acceleration at the resonant frequency of the dam. This represents the combination of accelerations present in a earthquake which will change with the frequency of vibration of shaking.

The Gibson and Cuthbertson (1995) report provides estimates of:

- peak ground acceleration recurrence
- smoothed ground motion recurrence spectra which represents the acceleration present in an earthquake with the frequency of vibration
- and a response spectra for structures following damped harmonic motion with a range of damping values for the 10 000 year event.

The peak ground acceleration is an unreliable indicator of earthquake hazard as it does not include consideration of the harmonic motion of the structure. The smoothed ground motion spectra give a better indication of earthquake behaviour but they are largely based on measurements taken in California.

The calculated fundamental vibration period for the dam including reservoir interaction is:

- non-overflow section - Tr = 0.17 seconds
- overflow section - Tr = 0.17 seconds

The relevant acceleration derived from the review are indicated below:

| Return Period | Peak Ground Acceleration | Peak Acceleration at Fundamental Period |
|---|---|---|
| | | Tr = 0.17 sec |
| 100 | 0.052 | 0.03 |
| 300 | 0.088 | 0.09 |
| 1 000 | 0.152 | 0.20 |
| 3 000 | 0.246 | 0.39 |
| 10 000 | 0.392 | 0.71 |
| 20 000 | 0.305 | 1.11 |
| 30 000 | | 1.68 |
| 100 000 | | |

The Gibson & Cuthbertson (1995) review concludes that "the majority of reismicity data used covers only 15 years, and the ground motion recurrence estimates must be considered as preliminary".

The adopted spectral accelerations used in the earthquake analyses were:

- operating basis earthquake 0.16 g
- maximum design earthquake 1.35 g

These accelerations are approximately equivalent to Modified Mercelli intensities of 6.9 and about 10.

The ordinary basis earthquake acceleration adopted for the analysis could be experienced during a 300 to 1 000 year return period earthquake. The maximum design earthquake acceleration could be experienced during a major earthquake with a return period in excess of 10 000 years and possibly in excess of 50 000 years.

The ICOLD recommendation for selection of earthquake magnitude requires that:

- the ordinary basis earthquake be selected on the basis that it is the event that occurs once during the life of the dam and minimal damage results from the event.
- the maximum credible earthquake be used to assess the dam and even though significant damage could occur as a result of this earthquake the dam should not fail. Because of the difficulty in defining this event it is sometimes defined as the 10 000 or 100 000 year event.

The 300 to 1 000 year return period earthquake selected for the ordinary basis earthquake in this case is assessed to provide a reasonable provision for potential earthquake during the remaining life of the dam. The maximum design earthquake acceleration is assessed to provide a reasonable estimate of the extreme loading applicable for this dam at this site.

(d) Stability Results

A summary of the results from the stability calculation and data used have been included in Appendix E.

(i) Stability under Normal Full Supply Level

Both the non overflow and spillway monoliths are stable under normal reservoir conditions. Stresses are below 1 Mpa and are all compressive. There is no tendency for cracks to open. A net stabilising moment is present. The ratios of factored shear strength to factored horizontal force are above 2 and are acceptable.

(ii) Stability under the Maximum Historical Flood (1974 Flood)

Under this load condition minor tensile stresses occur at the upstream face of both monolith types.

For the overflow section 23 kPa tension is present at RL 96 when the radial gates are closed and 29 kPa compression is present at the base. Compressive stresses are present in the downstream face and over most of the upstream face.

For the non overflow monolith a tension of 30 kPa is present at the base and up to about RL 58. No other areas of tensile stresses are present in this monolith. This exceeds the limit of zero tension in the rock indicated in Section 10.4.3(c) above. Investigations will be required to prove that this magnitude of tension is acceptable at the foundation level.

ANCOLD guidelines suggest an allowable tensile stress of zero kPa under long term, normal loads. It should be noted that G Verrenkamp has reported horizontal cracking in the upstream face of the dam at elevation 95.3 and 97.2 in Monoliths F and S which are both non-overflow Monoliths.

The effect of the actual concrete density being different from the adopted density of 2.4 t/ cu m has been investigated. A more detailed discussion is presented in a subsequent section but as expected decreasing the concrete density has the effect of raising the tensile stresses at the upstream face of the dam.

(iii) Stability under Operating Basis Earthquake

The peak deflection of the structure under a 150 year return period earthquake is 2.8 mm at the crest for block Q/R and 1.9 mm at the ogee crest for block K/L. The peak accelerations at the crest are 0.38 g for block Q/R and 0.27 g for block K/L using the Earthquake Engineering Research Centre (EERC) analysis method described earlier.

The differences between the blocks largely reflects the different block heights. This analysis has not considered the crest bridge as part of the structure. The accelerations and deformations are likely to be similar for both monolith types at the crest bridge level.

Stresses under OBE loading are all compressive and within acceptable limits with one notable exception. The upper level of the non overflow monolith is showing tensile stress of 112 kPa at 102 m elevation (approximately at the level of the abrupt change in downstream slope). This is because of the concentrated mass above this section (arising from the vertical downstream face above the section change). This stress, is above the ANCOLD allowable tensile stress of 0 kPa, set by ANCOLD for normal loads. The dynamic tensile stresses are much higher than the allowable static tensile stresses reflecting the very short term and cyclic nature of the stress condition, but are all dependent on the concrete being sensibly intact. Cracking in the dam has meant that at a number of places the strength has been reduced to zero. Investigations are required to assess the actual strength of the dam.

The structure has a net stabilising moment so will not fail from overturning under an OBE event.

There is the possibility of some damage to crest equipment from peak accelerations in excess of 0.38 g. Holding down bolts for winch motors and similar equipment, if not adequately designed for seismic loads, are likely to fail under the high crest accelerations. It can be seen that the amplification of earthquake loads from the ground level to the crest is significant. This can cause problems in ensuring that appurtenant equipment remains operable during and after the operating basis earthquake.

(iv) Stability under Probable Maximum Flood -

The upper sections of both monolith types are likely to fail through overturning under the PMF load if the concrete tensile capability is exceeded.

The stability analysis for Somerset dam have been completed assuming that some gates are closed. The requirement to analyse the dam for this condition has been necessitated by the "Manual of Operational Procedures For Flood Mitigation For Wivenhoe Dam And Somerset Dam" which requires that:

> "Somerset Dam should, if possible, not be overtopped by flood water but, if Wivenhoe Dam is threatened by overtopping, the release of water from Somerset Dam is to be reduced, for example by the use of its spillway gates, even at the risk of overtopping Somerset Dam in order to prevent if possible, the overtopping of Wivenhoe Dam."

The notional probability of a flood overtopping Wivenhoe dam is 1:14300. This is significantly short of the PMF and therefore increases the probability that Somerset Dam will require operation that involves closing the spillway gates during a major flood.

With the gates closed the spillway monolith shows a peak tensile stress in the lower part of the monolith at the base of 46 kPa and 151 kPa at RL 96 and this is greater than the allowable at the foundation level indicated in Section 10.4.3(c) above. For the spillway monolith, opening the gates has an immediate stabilising effect. The stresses at the base of the structure on the upstream face reduces from 111 kPa tension to 33 kPa compression. Likewise the stress at 96 m reduces from 151 kPa tension to 30 kPa compression. The non-overflow monolith shows peak tensile stresses of 111 kPa at the heel of the dam and a tensile stress of 171 kPa at RL 102. There is a net stabilising moment at the base of the structure and the shear friction factors show adequate resistance to shearing at the structure's base.

The tensile resistance of the concrete is uncertain. Testing is required to define the allowable tensile strength of the concrete. Also crack mapping and borehole investigation is required to determine areas of zero tensile strength.

(v) Reservoir Water Level Limitations (Imminent Failure Flood Level)

A study of the tensile stresses in the dam for different water levels was undertaken to evaluate the imminent failure flood level. Also included in this evaluation was a consideration of the resistance of the upper portion of the dam to sliding if the cracking in the gallery is continuous and results in the cohesion reducing to zero with $\phi = 45°$. The results of this analysis are indicated below:

| Block | Res Level | Peak "Stress" | at elev. | with SF = |
|---|---|---|---|---|
| Spillway FSL | 98.9 | + 467 kPa | 54.65 | 2.18 |
| Spillway gate closed | 105.7 | - 1 kPa | 97.4 | 0.99 |
| Spillway gate open | 109.1 | + 31 kPa | 97.4 | 1.00 |
| Non overtop FSL | 98.9 | + 451 kPa | 54.65 | 2.00 |
| Non overtop | 107.75 | - 1 kPa | 100.5 | 1.80 |
| Non overtop | 109.8 | - 128 kPa | 100.5 | 0.99 |

+ ve = compressive

This indicates that the imminent failure flood level occurs when the reservoir water level is at RL 105.7 if the gates are closed and RL 109.1 if the gates remain open and sliding resistance is marginal at these reservior levels if cracking results in the shear strength resucing to c = 0 and $\phi = 45°$.

(vi) Stability under Maximum Design Earthquake

The maximum design earthquake for this structure has been adopted as a 20,000 year return period event.

The peak deformations and accelerations of the structure have been assessed for the resonant frequency of the dam which has a peak spectral ground acceleration of 1.35 g. The accelerations at the crest of the structure are calculated to be 3.22 g for blocks Q/R and 2.24 g for blocks K/L. The deformations for this earthquake are 24 mm and 16 mm respectively. The peak accelerations and deformations have been assessed at the crest of the structural part of the dam. Peak accelerations and deformations for the crest bridge will be significantly higher again as these accelerations increase with structure height.

The spillway monolith shows tensile stresses up the upstream face of the structure and a net overturning moment under MDE load. The tensile stress at the base is 1035 kPa and reduces to 158 kPa tension at a section analysed at 96 m elevation. These stresses are less than the 1340 kPa recommended by ANCOLD which allows for the rapid and transient nature of the earthquake load but due to the cracking in the dam this limit may not be appropriate. Likewise the net overturning moment is cyclic and while it may permit cracking to form within the structure is unlikely to lead to failure of the dam.

The non overtoppable monolith also shows tensile stresses up the upstream face of the structure and an overturning moment. The tensile stress at the base is 1420 kPa and peaking at 1817 kPa at a section at elevation of 102m. This is greater than the ANCOLD recommendation of 1340 kPa.

Due to the cracking of the concrete in the dam the tensile resistance of the concrete are uncertain until investigations and testing is completed to define the overall properties of the dam.

(vi) Sensitivity of the Analyses to Concrete Density

A range of concrete densities was used for key load cases above to assess the effect of the density assumed on the stability of the structure. Densities ranging from 2.3 to 2.6 t/ cu m (in 0.1 t/cu m increments) were used.

As could be expected, reducing the concrete density reduces the stabilising effect of the dam weight thereby reducing the overall stability of the dam. Lower densities allow greater tensile stresses to form at the upstream face of the dam under flood loads. The increase in tensile stress is greater at the base of the dam as the concrete weight has a proportionally greater effect on dam stability at lower elevations.

As small tensile stresses are present under large flood events for the preferred density model described above, reducing the density modelled allows these stresses to exceed recommended limits. Thus, if the concrete had a lower density or had a lower tensile strength than modelled it would be reasonable for cracking to be observed in the lower portion of the structure following the 1974 flood event.

(c) Cracked Section Studies

Once cracking has been initiated, the behaviour of a dam monolith can be assessed using an approximation of the changed uplift and stress condition brought about by the onset of cracking. There are several methods used to assess cracked section stability these include: the ANCOLD method given in their 1991 guidelines and the internationally accepted Levy and Hoffman criteria. The levy and Hoffman criteria have been adopted in this analysis.

The Levy and Hoffman method uses a combination of beam theory (using external loads) and uses a pseudo internal pore pressure within the concrete. The uplift distribution used in this method is full reservoir head over the cracked length (which is equivalent to the most conservative of the ANCOLD uplift distributions) and the pore pressure through the uncracked portion is equivalent to a linear drop in uplift to tailwater (again equivalent to the ANCOLD assumed distribution). The Levy and Hoffman Criteria are two separate criteria used to describe the cracked stress distribution within the structure. Each criteria is discussed below.

The Levy Criteria assesses the stress condition at the head of the crack. The external loads are the normal ANCOLD reservoir and concrete loads along with the uplift load applied over the cracked length. The stress is calculated at the heel of the intact portion. If this stress, less the uplift pressure, is more tensile than the allowable tensile strength, the crack is considered to propagate.

The Hoffman Criteria simply assesses the change in stress in the crack with change in crack length. If the stress becomes more tensile with increasing crack length then a crack, if it were to form, would propagate until failure of the monolith occurred.

If the Levy condition is not met then the structure must meet the Hoffman Criteria to remain stable. In this case the crack will propagate to a stable crack length. If both the Levy and Hoffman Criteria are not met then a crack will form and this crack will propagate until failure of the structure occurred.

For this structure the results of the analysis are presented in tabular form below.

| Load Conditions (Water Level) | | Non Overflow Block | | | Spillway Block | | |
|---|---|---|---|---|---|---|---|
| | | Initial Crack Length | Levy | Hoffman | Initial Crack Length | Levy | Hoffman |
| FSL | | 0 | OK Compressive | OK | 0 | OK Compressive | OK |
| 1974 | | 1.6 m | 54 kPa | Unstable (Failed) | 0 | Failed 12 kPa tension | Unstable (failed) |
| PMF | (gate closed) | 4.7 m | 282 kPa | Unstable (Failed) | 2.2 m | 75 kPa Failed | Unstable |
| | (gates open) | | | | 0 | 110 kPa Failed | Unstable |

Note tensile stress +ve

In all cases the allowable tensile stress assumed is zero. The calculated cracked section stress at the initial crack length is included in the table above to allow a comparison with higher allowable tensile stresses. In all cases above the stresses have been calculated for a section at the structure's base.

It can be seen from the table above that the monoliths, are unstable once cracking is initiated for a section at the base. This occurs because of the high reservoir pressures and changes from stable cracked sections at full supply level to unstable during the 1974 flood. This is only a problem once cracking is initiated through overstressing.

(d) Discussion of Stability Results

The dam is stable with the reservoir at full supply level. For reservoir levels exceeding RL 105.7 with the gates closed and RL 109.1 with the gates open, the dam would be considered to be unstable using the current ANCOLD guidelines.

Under earthquake loadings the stresses exceed the ANCOLD guidelines.

Within the guidelines set by ANCOLD the structural evaluation of the dam would be CONDITIONALLY POOR as defined by the USBR (refer Appendix G for definition of terms)

## 10.5 Review of Concrete Cracking

### 10.5.1 Cracking During Construction

Cracking of the concrete placed in Somerset Dam was observed from the earliest stages of construction. Hand written notes and sketches indicate that cracking was being monitored in the galleries and in the baffle block in August 1939 and continued to 1942 and summary of these notes is indicated below:

(a) Foundation Tunnel (November 1939)

Sketches of the foundation tunnel through Monolith P, R and S indicates a number of subvertical cracks with some weeping.

(b) Stilling Basin Baffle Wall and End Sill Wall (December 1940 to February 1941).

On 6 December 1940 block 98 of the baffle wall was poured and on 9 December cracking was reported on all five exposed faces in a vertical and horizontal square pattern. On 14 February 1941 cracking is reported in most of the baffle wall blocks completed to that date at about the same time cracks are reported in the downstream sill wall.

(c) Lower Gallery (June 1939 to February 1941)

In the lower gallery the following cracking was reported in 1939.

- Monolith L - no cracks
- Monolith M - upstream face and roof crack perpendicular to N line
  - 3 transverse cracks, which connect with upstream face
  - crack approximately parallel to the N line on the roof
- Monolith P - no cracks
- Monolith Q - stairway faint cross crack
- Monolith R - cross crack on roof
  - 3 vertical cracks on the upstream face which extend to water face
  - 3 transverse cracks on the roof which apparently extend to the upstream face
- Monolith S - a number of cracks near the corners of the roof and following the line of the steps
- Monolith T - 3 vertical cracks on the upstream face
  - cracks at corners of the roof following the line of the galleries.

(d) Upper Gallery (November 1940 to March 1941)

- Monolith Q - upstream wall cracks between both gate shafts and gallery wall
  - transverse cracks in the roof spaced evenly across the Monolith

- Monolith R - three transverse cracks roughly equally spaced
- Monolith S - two transverse cracks each approximately 6 m (20 ft) from the construction joint
  - longitudinal crack near upstream edge.
- Monolith U - 4 cracks across the roof the second crack from the right hand side was weeping.
  - 2 vertical cracks in the upstream face of the gallery.

(e) Sluice Tunnels (March 1942)

- Sluice J - percolation through construction joints
- Sluice K - extensive percolation through construction joints and cracks on sides
  - extensive percolation through construction joint in roof
- Sluice M - very little leakage from contraction joints.

(f) Monolith G

Cracks in hydro-electric outlet roof and walls are reported between September 1940 and August 1941.

### 10.5.2 Post Construction Cracking

(a) Monolith Q Bellmouth

In February 1950 Mr R de V Gipps reported serious deterioration of the concrete in box Q/55, Q45, 49 and 59 extending from approximately RL 70.1 m (230.5 ft) to 71.3 m (234.5 ft) and up to about 3 mm ($^1/_8$ inch) depth.

(b) Concrete Crack Survey

In July 1951 Mr G N Card completed a crack survey of the dam.

(c) Post 1950 Cracking

(i) In September 1969 Mr E M Shepherd inspected the cracking in the galleries and noted that the horizontal crack on the downstream side of the upper gallery was not reported in the 1951 survey. At this time Mr Shepherd suggested that the cracking was caused by drying shrinkage.

(ii) In October 1977 Mr E M Shepherd wrote to ANCOLD to gain discussion on the cracking.

(iii) In November 1977 Mr E M Shepherd reported on his inspection of the dam. This report noted that:

- the cracking between shafts 12 and 13 in regulator Monolith Q resulted in pronounced leakage of water from one shaft to the other. The crack could not have existed in the early 1950's when an extensive maintenance programme was carried out on the regulator outlets. This suggests a long slow build up of stress.

- the sluice in Monolith Q when inspected in the early 1950 showed some untidy cracking and at the 1977 inspection the cracks were quite visible but probably no more prominent.
- a seeping crack existed in the upstream face of the gate chambers in Monolith I and was not reported in 1951.

(iv) In November 1977 Mr G Verrenkamp reported two horizontal cracks in the upstream face of the dam. These were:

- Monolith F on the right side and extending half way across the Monolith at RL 95.3 (313.25 ft)
- Monoliths S and R extending across the full width of Monolith S and about 1/3 the widths of Monolith R at RL 97.2 (319.2 ft)

(v) In February 1979 core was taken along the cracks in the gallery.

(vi) In August 1978 Mr E M Shepherd suggested that the use of two grades of concrete with a system of vertical cored drains leading into the galleries and located near the downstream limit of the richer concrete could have had some influence on the cracking.

The testing of concrete cores in February 1987 indicated that there was a large increase in density, from the dry density to the saturated density. The reason for this is not known but requires checking.

### 10.5.3 Review of Possible Causes of Cracking

Most cracks in concrete are primarily caused by tensile stresses due to internal or external restraint produced by volume change. (Leonhardt (1977), Carlson (1970), ACI Committee 207 (1970); CIRIA Report 91). Volume changes are caused by:

- changes in moisture content of the concrete
- chemical reactions
- changes in temperature
- construction procedures
- stresses from applied loads.

(a) Drying Shrinkage

Drying shrinkage can range from 200 microstrain for low slump lean mixes to over 1 000 microstrain for rich mixes containing excess water and poor quality aggregate.

(b) Chemical Reactions/Autogenous Volume Change

Autogenous volume change results from the chemical reaction within the concrete. The net autogenous volume change of most concrete is a shrinkage of from 0 to 150 microstrain. At Dvorshak Dam increasing cement content caused higher autogenous shrinkage and the volume change was still continuing after 36 months of testing.

(c) Changes in Temperature

The most significant factor affecting volume change in mass concrete dams is temperature. Temperatures of 80°C have been measured in mass concrete. The thermal strain resulting from temperature change is about 10 microstrain for each °C. There are two causes of temperature changes in mass concrete dams. These are:

- heat of hydration
- environmental temperatures

The heat of hydration in concrete can be very high and can take long times to dissipate. At Somerset dam the hearting concrete contained 200 kg/m$^3$ of cement. The peak hydration temperature that developed in the core of Somerset dam is difficult to determine due to the long construction period. However, it is unlikely that the peak temperature was less than 50°C.

The following guidelines can be used to evaluate the time required to dissipate temperature generated due to hydration:

- 95% of heat is lost in a 15 cm thick wall in 1 1/2 hours.
- 95% of heat is lost from a 1.5 m thick wall in one week.
- 95% of heat is lost from a 15 m thick wall in 2 years.
- 95% of heat is lost from a 150 m thick dam in 200 years.

The maximum thickness of Somerset dam is about 38 m and therefore 95% of the heat would be lost in about 20 years. The largest portion of heat loss would have occurred by about 1960. It should be noted that the main horizontal crack in the upper gallery was not noted in 1951 but was noted in 1969. By 1969 the majority of the heat would have been dissipated.

The behaviour of the surface of mass concrete structures is affected by daily and annual cycles of temperature. The following environmental temperature responses are reported by the ACI committee on the Mass Concrete for Dams and other Massive Structures:

- at the surface the temperature of the concrete responds almost directly to <u>daily</u> variation in temperature.
- 0.6 m from the surface only 10% of the <u>daily</u> surface temperature variation is felt in the concrete.
- 7.5 m from the surface 10% of the <u>annual</u> variation in temperature is felt in the concrete.
- 15 m from the surface 1% of the <u>annual</u> variation in temperature is felt in the concrete.

The downstream face of the upper gallery in Somerset dam is about 6 m and about 8 m from the downstream face of the dam at the non-overflow and overflow sections of the dam respectively. The upstream face of the upper gallery is about 4.8 m from the upstream face of the dam. This means that the annual variation in temperature will be "felt" at the upper gallery location on the downstream side of the gallery. On the upstream side of the gallery temperature effects will be dampened by the presence of more uniform temperature associated with the temperature of the water.

(d) Construction Procedures

Construction procedures have a significant impact on the development of cracks. The procedures that have an impact include:

- temperature of the concrete at the time of placing which can effect early-age thermal cracks and the peak hydration temperature.
- the duration between successive lifts.
- curing time and rate of drying of the concrete surface.
- air temperature during curing
- time for striking of forms
- high early age strength

Of these a significant feature of the construction of Somerset dam was the time between lifts and the long break in construction between 1942 and 1948. The dam construction in 1942 was generally completed to its finished crest level in Monoliths J, L, M and O. Monoliths I, K, N and P were completed to approximately RL 87 to RL 90 ie at upper gallery floor level to 2 m below the upper gallery floor level.

It is recognised that whenever irregular placement occurs the construction joints in the old lifts are rigid and restrain dimensional change of the newly placed concrete. Restraint to construction or shrinkage of the concrete induces tensile stresses. The restraint effect reduces with height above the "rigid" surface.

The significant horizontal crack in the downstream roof is located where this effect could be significant.

(e) Evaluation

The most likely cause of the significant horizontal crack in the downstream roof of the gallery is volume change due to dissipation of heat following completion of construction in about 1951. The continuing opening and closing of the crack is caused by the seasonal variation in temperature. There is an apparent trend of increase in joint opening of up to about 0.1 mm per year. This magnitude of movement is too small to be able to define the cause however, a temperature reduction in the core or increase in temperature of the outer "shell" of only 0.3°C per year could cause this amount of joint opening.

(f) Discussion on Cracking

**1. General**

In concrete dams cracks may be subdivided as follows:

Type 1 - Hairline cracks, minor, discontinuous shrinkage cracks, weaknesses associated with construction joints.

These are normal and expected in any such structure. The selected parameters are based on this situation - subject to adequate concrete quality.

Type 2 - Open cracks on Possible Failure Surfaces - essentially sub-horizontal.

Such cracks will sustain zero tension and the available shear strength depends on the roughness of the surface and the overlaying stress. One way of assessing this is to determine a roughness angle and add that to the friction angle (45°).

Type 3 - Open cracks that have healed somewhat by Autogenous Healing.

These must by positively identified or treated as Type 2. The tensile strength will depend on the degree of healing and strength of the infill - generally between 0 and same value less than for Type 1.

The shear resistance will also depend in the degree of healing, roughness of the crack surface extent of the crack and strength of the infill. Generally the Type 2 strength plus cohesion due to the infill.

Hence the strength (tensile or shear) will depend in the type of crack together with the percentage of the failure surface that is made up by the crack.

## 2. Cracks Mapped in 1951

Most of these cracks were not identified in the latter mapping. Where this is the case they are either Type 1 or Type 3 cracks. As they will only make up a small part of any failure surface it is considered that use of Type 1 parameters is realistic.

Cracks mapped in both 1951 and 1977 are discussed later.

## 3. Crack at Top of Upper Gallery

This is a very noticeable crack that is continuous along the top of the downstream side of the gallery and extends along the sides of the access galleries. There is also an indication that it is exposed on the downstream face of the dam. In the gallery it is generally open. There are also indications that the crack extends upstream of the gallery in some locations, in such cases it is closed.

Holes drilled in the gallery in 1979 show it extends downstream at least 1 m and stays open in this length. It is not clear if the crack forms a continuous plane downstream of the gallery but it must be considered a possibility until demonstrated otherwise.

The surface where visible is relatively rough in a downstream direction. Hence the tensile strength should be taken as zero and the shear strength based on a friction angle somewhat greater that 45°. The use of a strength reduction factor of 0.3 under thus condition is considered to be low.

Under these circumstances there is inadequate resistance to sliding with the gates closed and reservoir above RL 105.7. As there is also some shear strength reduction on the upstream side of the gallery the situation is currently unacceptable.

Therefore it is essential to either demonstrate the crack is sufficiently discontinuous and/or sufficiently rough that there is adequate shear strength resistance, or the crack should be strengthened or the reservoir should not be allowed to rise above the critical level (ie. the operating rules would have to be changed).

In order to resolve this matter the following is recommended:

1. Retailed mapping of the crack to identify both the location and the roughness. This will probably involve cleaning of the downstream face (and possibly a detailed inspection of the upstream face).
2. A detailed concrete cooling study taking into account the construction break for the work in order to better understand the cracking mechanisms.
3. Drilling of strategically placed holes from the crest of the dam to intersect the crack to enable determination of the internal roughness and laboratory shear testing of the joint. Such testing must be in the correct direction.
4. TV inspection of the drill holes.
5. Consideration should also be given to injecting dye at locations where the tensile strength of the concrete may be exceeded to check for leakage from the face.
6. Continuing crack monitoring and determine the need for additional movement measurement locations.
7. Monitoring temperature of concrete in selected locations.
8. Determine generally methods and costs of strengthening the joint.

## 4 Other Sub Horizontal Cracks - eg. at Construction Joints

The development of the crack at the top of the upper gallery is believed to be a result of concrete volume change due to cooling and the stress concentration effects of the corner of the gallery. However, it is possible that this crack has not relieved all the built up stresses, particularly as it could be induced by vertical shrinkage. Hence there could be other similar cracks, possibly associated with construction joint weaknesses. There is some evidence of seepage on the downstream face that provides a further indication.

Although there would almost certainly by Type 3 cracks and this provides reasonable resistance to sliding it is considered necessary to investigate this matter. Work would essentially be the same as required for the gallery cracks as follows.

1. Mapping of downstream face.
2. Drilling of cored holes from the crest, TV logging and possibly dye testing.

## 5 Discontinuous Horizontal Cracks in the Upper Upstream Face

Analyses show that there is tensile stress in this area under the 1974 flood (with gates closed) and under higher reservoir levels. Once again the extent and nature of the cracks are not known. It is believed that they are of no risk to the structure but more data is needed. The following is recommended:

1. Detailed crack mapping
2. Determination of possible cause (Cooling, 1974 flood)
3. Drilling as per other cracks

## 6 Opening of Joint in the Monolith L/M Joint

As detailed this joint is open at the top to the extent that the waterstop is torn, is open at the upper gallery and is closed at the lower gallery. Hence, with high water levels water is introduced to the joint. It would be desirable to develop an understanding of the mechanism causing the opening. The following is recommended:

1. More detailed mapping
2. Monitoring
3. Assess possible mechanism,
4. Repair waterstop, if joint movement is not continuing.

## 7 Cracking on Top of Spillway Gate Piers

These would appear to be due to concrete deterioration. This cracking is currently not significant structurally, however, if the gates are to be retained the cause of the cracking needs to be identified and fixed to ensure that the dam has a long life.

The following is required:

1. Determine cause of cracking
2. Determine depth and extent of cracking
3. Fix

## 10.6 Dissipator

1. Retaining Walls

The dissipator retaining walls are designed as mass concrete walls. The wall dimensions as shown on the available drawings are not certain. The following dimensions are interpreted from the drawings:

- top of wall RL 73.02 (240 ft)
- stilling basin floor level RL 60.83 (200 ft)
- lowest level of concrete RL 57.78 (190 ft)
- width of top of wall 0.6 m (2 ft)
- inside slope of wall 0.25H:1V
- hand placed rock is placed behind the wall with a width of 0.46 m (18 inch) at the top, and has a slope of 0.125H:1V. Where a narrow gap was formed between the rock surface and the formed concrete wall the gap was filled with hand placed rock.

- The following minimum thicknesses of concrete were set:

| **Height Of Wall Above Rock** | **Minimum Concrete Thickness** |
|---|---|
| 4.88 m (16 ft) | 2.44 m (8 ft) |
| 6.10 m (20 ft) | 2.74 m (9 ft) |
| 7.32 m (24 ft) | 3.05 m (10 ft) |

The maximum thickness of concrete was 4.27 m (14 ft) at RL 60.84 (200 ft).

- Weep holes were placed at RL63.88 m (210 ft), RL68.45 m (225 ft), RL70.89 m (233 ft) and RL72.11 m (237 ft). The left abutment may not have had weep holes installed.

A check on the stability of the walls was undertaken assuming that:

- the maximum water level behind the walls is RL 65.71 m (216 ft)
- concrete is poured directly against the rock up to RL 64.49 m (212 ft)
- the wall is rigid and earthfill pressure can be calculated from Ko=1-SIN$\phi$ and assuming that $\phi$=35° and the bulk density of the fill is 20 kN/m$^3$
- the density of concrete is 2.4 t/m$^3$

This analysis is included in Appendix F and indicates that:

- at RL 65.71 m (216 ft)
  - if the concrete is intact the maximum tensile stress is 118 kPa and the maximum compressive stress is 324 kPa
  - the resultant is located 0.437 m from the water face ie just within the middle 50%
- at RL 60.83 m (200 ft)
  - if the concrete is intact the maximum tensile stress is 434 kPa and the maximum compressive stress is 778 kPa
  - the resultant is located outside the base under these conditions and the wall would be considered to be unstable.

On the basis of the assumed loadings the dissipator walls do not meet the current design requirements for normal static loadings and would be less stable under extreme or dynamic loadings.

2. Floor

The floor of the dissipator consists of concrete:

- 3.05 m (10 ft) thick for a distance of 21.03 m (69 ft) downstream of the dam
- thickness reducing from 3.05 m (10 ft) to 1.52 m (5 ft) for the next 12.80 m (42 ft)
- 1.52 m (5 ft) thick beneath the baffle and for a distance of 8.23 m (27 ft)
- thickness reducing from 1.52 m (5 ft) to 0.76 m (2.5) for the next 6.25 m (20.5 ft)
- thickness of 0.76 m (2.5 ft) over the final 8.08 m (26.5 ft)

The total length of the dissipator is 58.22 m (191 ft) with the baffle 33.83 m (111 ft) downstream of the dam.

Drains have been cast into the concrete. These consist of 75 mm diameter GSI pipes and are shown on the drawings at about 6 m (20 ft) centres.

No anchors are indicated on the available drawings.

The thickness of concrete would be adequate to resist uplift pressures equivalent to a ground water level of about RL 68.03 m (224 ft). The system of drains installed in the dam and dissipator walls will ensure that this occurs (if they are not blocked), when the spillway is not operating. Operation of the spillway may change these uplift pressures.

3. Hydraulics

For analysis of the hydraulics of the dissipator the 1974 flood is used to evaluate the stilling basin hydraulics. The adopted hydraulic characteristics of the spillway during this flood are:

- Peak reservoir level RL 106.55
- Peak tailwater level RL 72.1 (as reported by G Verrencump)
- Peak outflow 1052 cumecs

For these conditions the following characteristics are calculated:

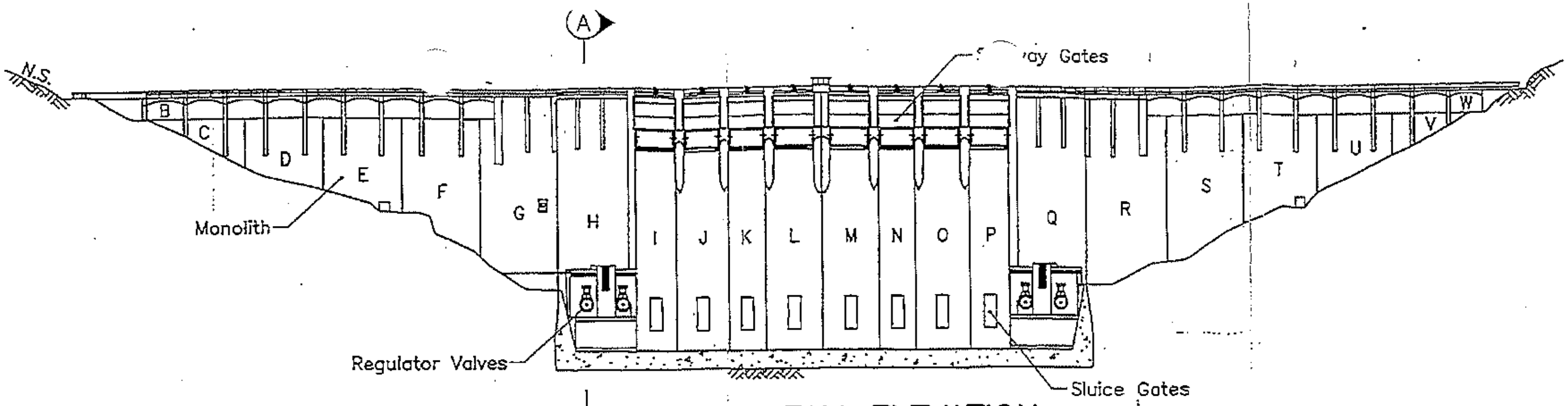
- maximum velocity at entry to dissipator 29 m/s
- inflow depth of water 3.5 m
- Froude number 4.9
- minimum tailwater depth for full development of the hydraulic jump 23 m
- actual tailwater depth 11.3 m
- the length of the hydraulic jump is about 87 m and the stilling basin is 58 m long.

This indicates that the energy loss in the dissipator is not as complete as would occur in a fully developed hydraulic jump dissipator. This means that high velocity flow is likely beyond the end of the dissipator. The rock surfaces downstream of the dissipator have resisted these flows but some erosion of the right abutment downstream of the dissipators has occurred.

The high velocity flow in the dissipator is sufficient to cause cavitation at discontinuous or rough surfaces in the concrete.

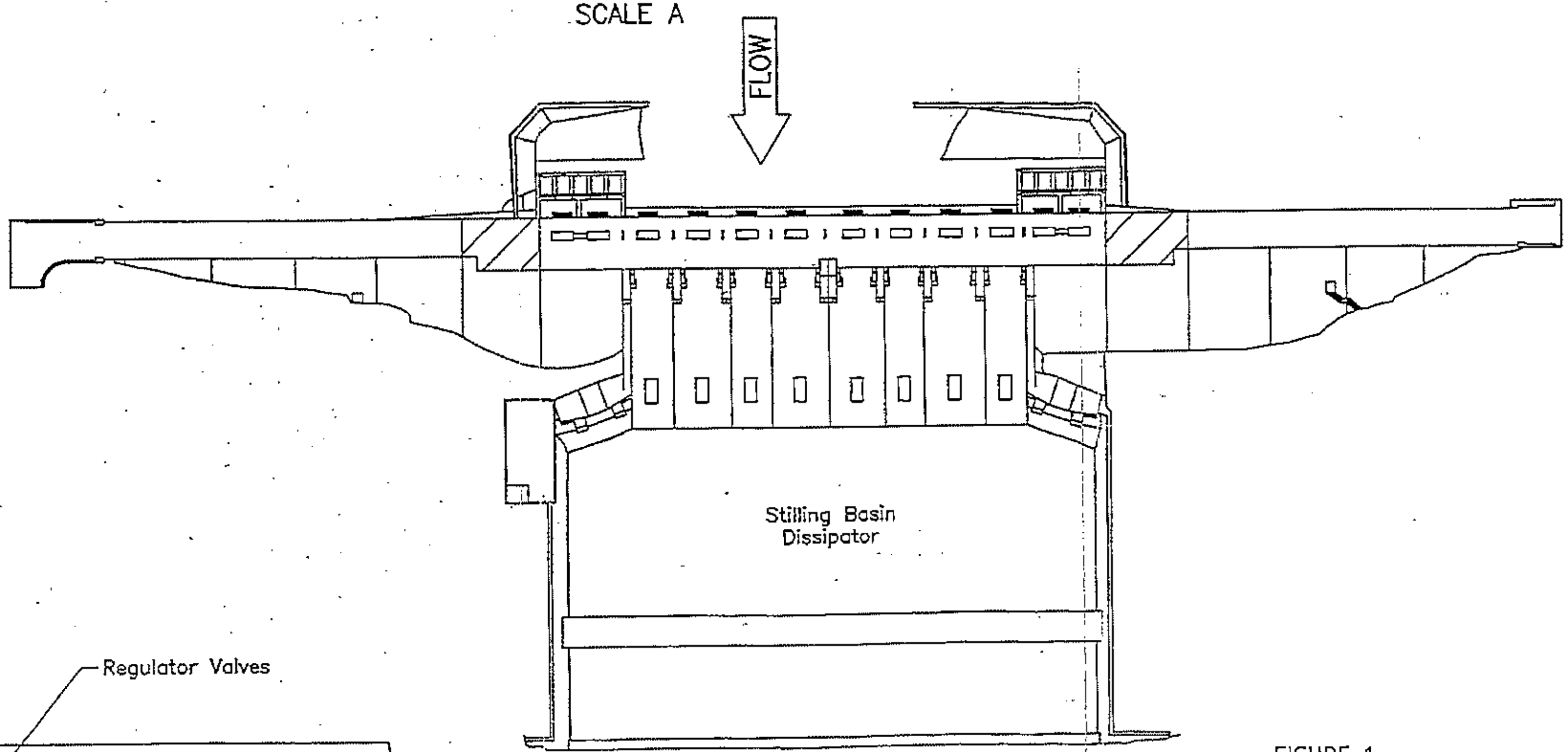
The drain holes if located incorrectly can result in high velocity water entering the pipes and resulting in high uplift pressures beneath the concrete floor. These pressures can be sufficient to "jack" the concrete off the rock and result in failure.

During the 1974 flood the tailwater level was 11.3 m above the floor level and the incoming water level was 3.5 m above the floor level. If the downstream tailwater level connects directly to the ground water pressure in the entrance, the uplift pressure would be just balanced by the weight of concrete and water at the upstream section of the dissipator. It should be noted that the tailwater pressure will be transmitted through the rockfill behind the retaining walls and through open joints in the foundation rock. The "downstream" drainage (foundation) tunnel will help to dissipate the uplift pressures



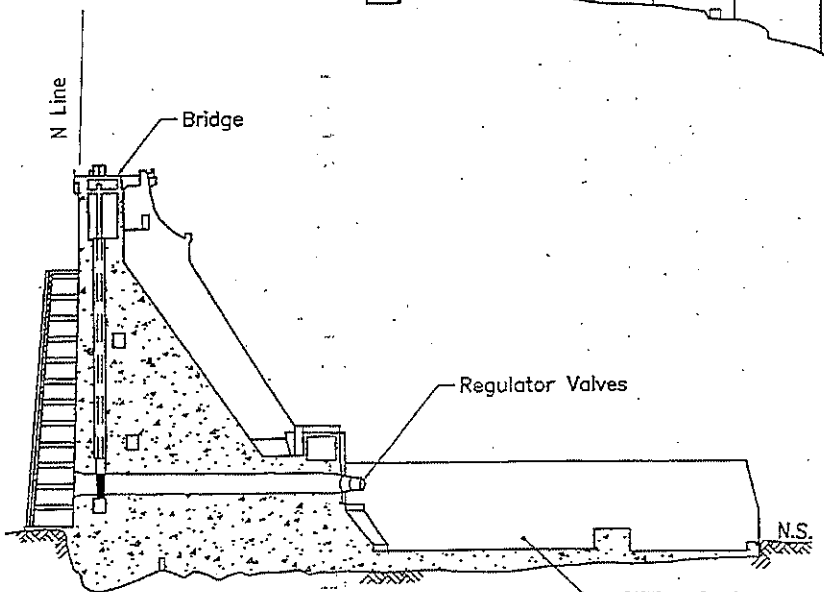

DOWNSTREAM ELEVATION

SCALE A




PLAN

SCALE A




SECTION A – A

SCALE B

NOTE
All distances in metres
unless otherwise specified.

SCALE A 0 20 40 60 METRES 1:1000

SCALE B 0 15 30 45 METRES 1:750

FIGURE 1

DEPARTMENT OF PRIMARY INDUSTRIES

QUEENSLAND WATER RESOURCES

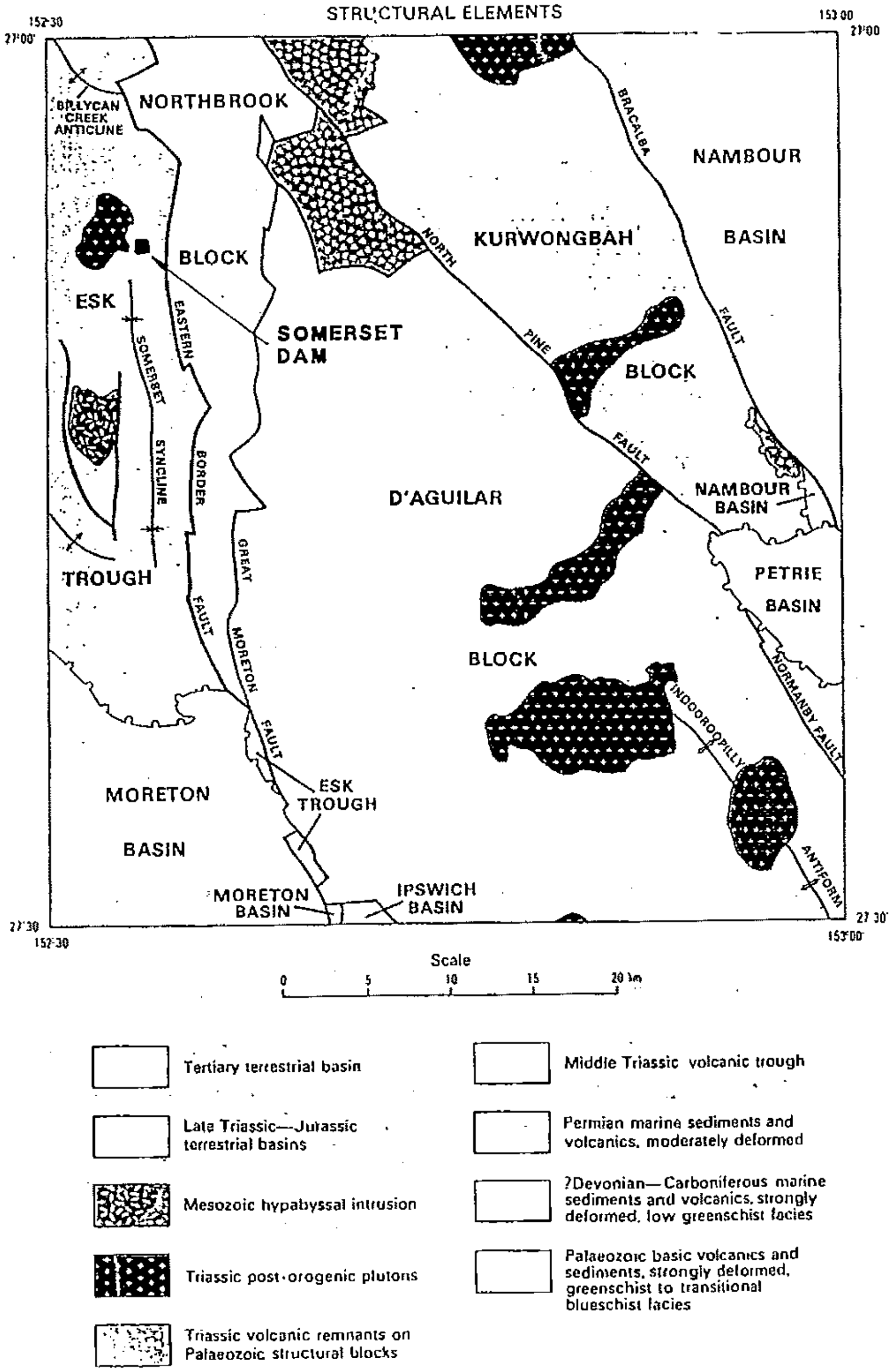
BRISBANE RIVER FLOOD STUDY

SOMERSET DAM

MAIN DAM ARRANGEMENT

15.12.94

A3–104021

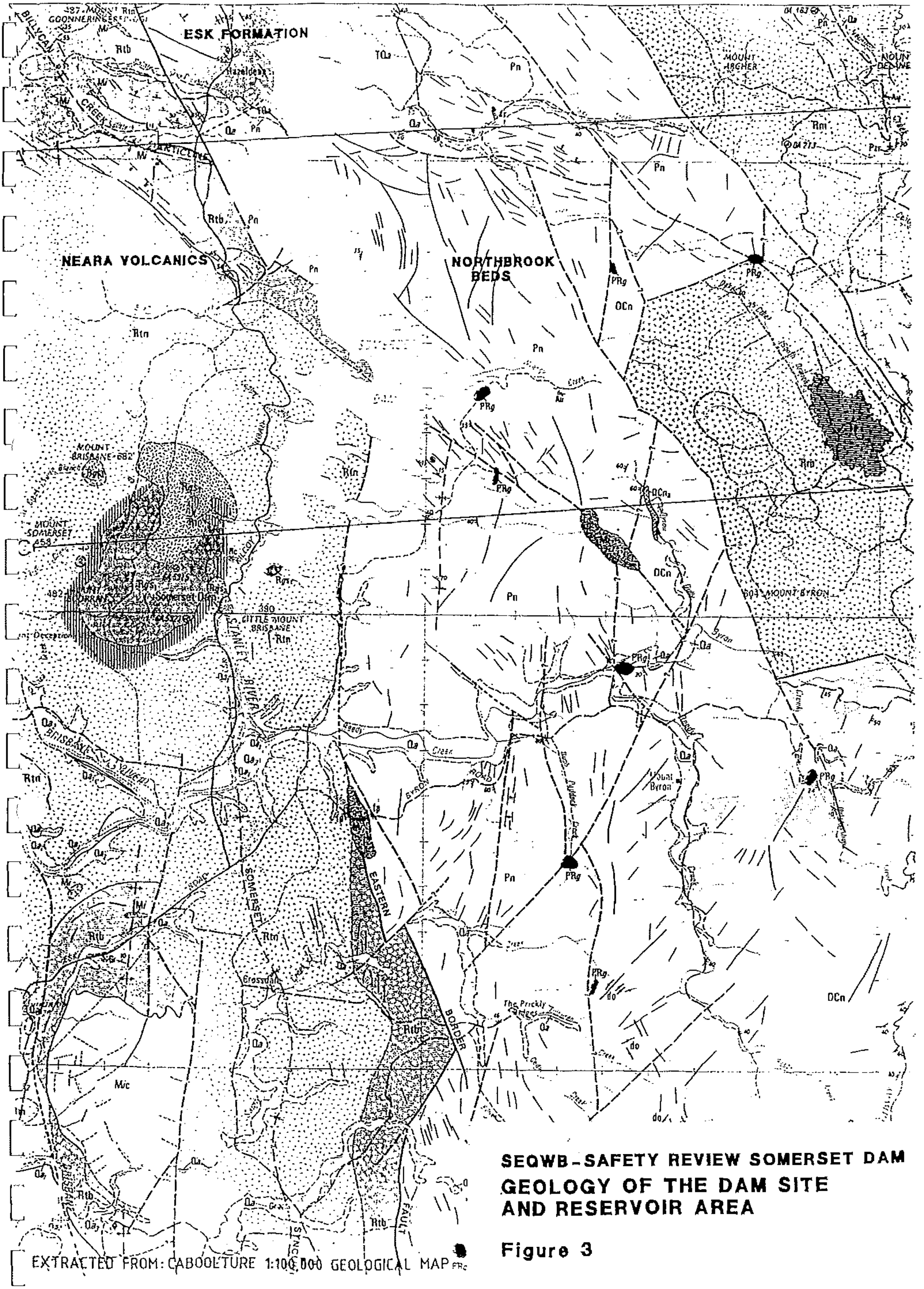



EXTRACTED FROM : CABOOLTURE 1:100,000 GEOLOGICAL MAP

**SEQWB - SAFETY REVIEW SOMERSET DAM**

**STRUCTURAL ELEMENTS**

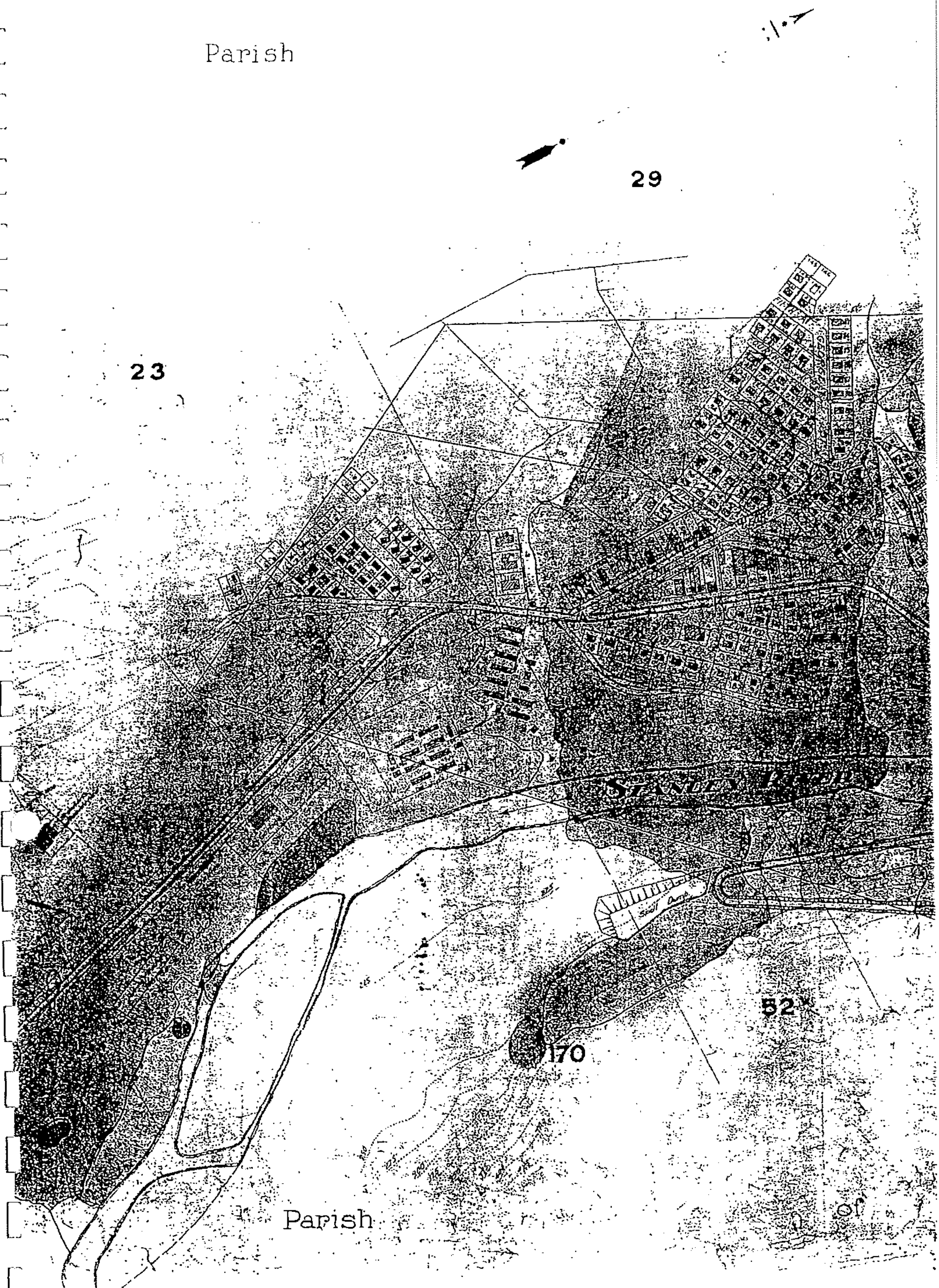
**Figure 2**




**SEQWB-SAFETY REVIEW SOMERSET DAM**

**GEOLOGY OF THE DAM SITE AND RESERVOIR AREA**

**Figure 3**



Parish
29
23
Stanley Road
52
170
Parish
of

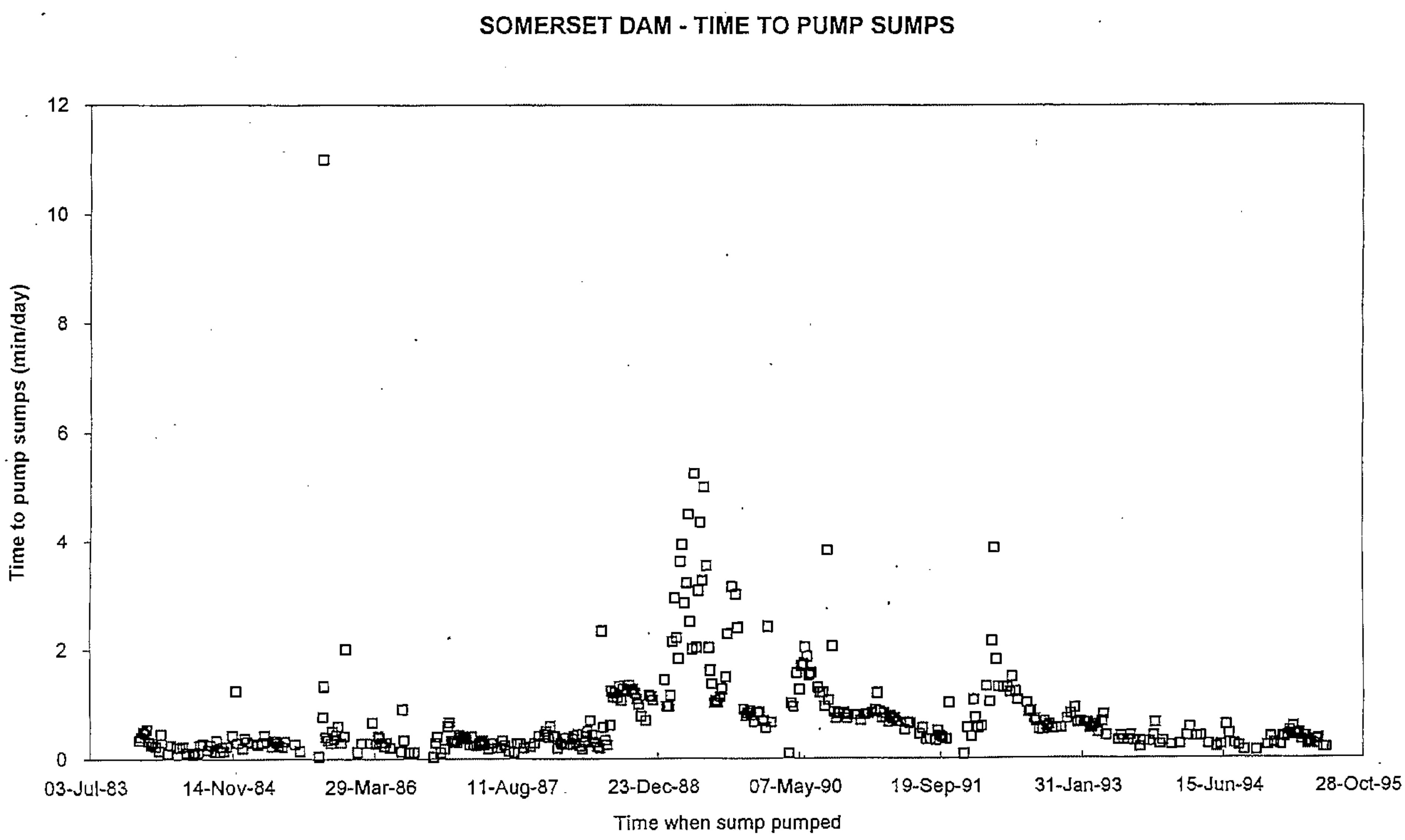
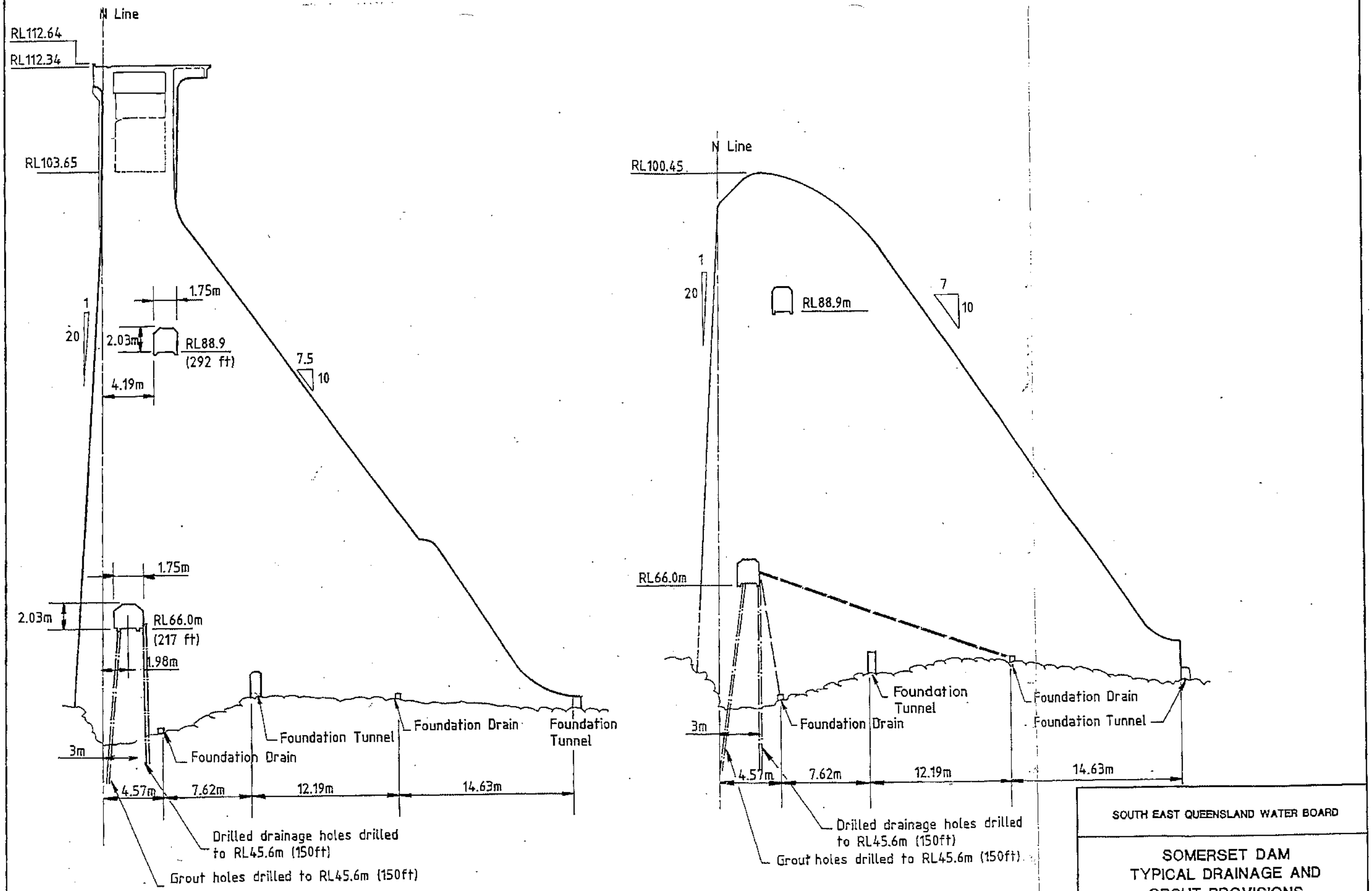

SOMERSET DAM - TIME TO PUMP SUMPS
Time to pump sumps (min/day)
0
2
4
6
8
10
12
03-Jul-83
14-Nov-84
29-Mar-86
11-Aug-87
23-Dec-88
07-May-90
19-Sep-91
31-Jan-93
15-Jun-94
28-Oct-95
Time when sump pumped

FIGURE 5




SOUTH EAST QUEENSLAND WATER BOARD

SOMERSET DAM
TYPICAL DRAINAGE AND
GROUT PROVISIONS

FIGURE 6

# APPENDIX A

## Safety Evaluation of Mechanical Equipment

SOUTH EAST QUEENSLAND WATER BOARD

# SOMERSET DAM

# SAFETY EVALUATION
# OF MECHANICAL EQUIPMENT

Inspection Report
on
Mechanical Equipment
by

G.M.Thyer

March 1995

HEC Enterprises Corporation

# SOMERSET DAM

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## 1. INTRODUCTION

This report has been prepared as part of a Dam Safety Review of Somerset Dam. The mechanical equipment at Somerset Dam was inspected by G.M.Thyer of HEC Enterprises Corporation on 6th and 7th of March 1995. Operator's representative present: T.Cannell.

The report reviews and evaluates the operational status of the equipment in terms of public safety and operator safety (and asset safety only in so far as it affects the first two).

## 2. SUMMARY

In the course of this part of the review the following equipment was inspected at Somerset Dam:

- Spillway Gates
- Sluice Gates
- Emergency Coaster Gate
- Regulator Valves
- Baulks
- Sump Pumps
- Gantry Crane
- Diesel Generator Set

Tests were carried out on the Spillway Gates, Sluice Gates and the Regulators. The Gantry Crane was tested only briefly.

The report includes a review of the equipment and comments regarding comparisions with current design practice.

Discussions were held with the operators as to operating and maintenance procedures and records.

## 3. BACKROUND

Somerset Dam has eight spillway gates and eight bottom sluice outlets in the central blocks I - P of the concrete gravity dam. Two discharge regulators (No.1 & No.2 Regulators) are located adjacent to the sluices in right abutment block H, and two (No.12 & No.13 Regulators) are located in block Q on the left abutment.

An additional sluice gate (No.1 Gate) is located in block G of the right abutment and the conduit feeds a small power station of about 4 MW.

An emergency coaster gate is provided for shutting off the sluice outlets at the upstream entrance.

Baulks are provided for servicing the hydro outlet and regulators.

A gantry crane travelling the entire crest serves for maintenance of all gates and operation of the coaster gate.

Backround data:-

| | |
|---|---|
| Full supply level: | R.L. 99.0m |
| Spillway Gates: | Radial type gates, welded and bolted construction.<br>width = 7.9m, height = 7.0m. Free surface type.<br>Wire rope hoists, gates counterweighted. |
| Sluice Gates: | Caterpillar roller type, rivetted and bolted construction.<br>width = 2.44m opening, height = 3.66m opening.<br>Wire rope hoists. |
| Regulator gates: | Caterpillar roller type, rivetted and bolted construction.<br>width = 2.74m opening, height = 2.74m opening.<br>Wire rope hoists. |
| Emergency Coaster Gate: | Caterpillar roller type, rivetted construction.<br>width = 3.7m, height = 4.9m at entrance, roller track span = 4.6m. Handled by gantry crane. |
| Baulks: | One regulator baulk, one hydro outlet baulk.<br>To suit 2.74m dia. conduit entrance. |
| Trashracks: | 5 bays per intake structure each side. 1 spare set of trashracks. |

| | |
|---|---|
| Regulators: | 2286mm dia. Fixed Cone valves, operated by electric actuator via linkage. |
| Gantry Crane: | 100 ton capacity, plus 14 ton auxiliary hoist for general maintenance & handling of trashracks. |

## 4. INSPECTION REPORT

### 4.1 Spillway Gates

The spillway radial gates were inspected on 6-3-95 and found to be in fairly good condition. Corrosion protection appeared adequate. (The gates spend a lot of the time out of the water which helps in this regard).

Structurally the gates appeared fairly sound. Although there are no side seals, occasional binding has been observed on the concrete side walls.
This may be due to a lack of torsional stiffness in the gates. At the back of the gates the central panel between the main beams does not have diagonal bracing. Bracing in this panel would add to the torsional stiffness of the gates and reduce the binding effect.

The trunnion bearings at the ends of the arms are articulated with a vertical pin allowing movement in the lateral direction as well as vertical. This may be contributing to the freedom of movement of the gates, causing the binding action at the side walls. The lack of side seals also contributes to the free movement of the gates and the tendency to run roughly against the concrete side walls.

Bottom seals consist of timber blocks bolted to the bottom of the gate - this is all that is required for this installation as sealing is not critical.

The hoisting machinery for the gates was inspected and found to be in fair condition. The curved slide type counterweight system has caused a few problems with wheels seizing up occasionally. There is a slight risk here of malfunction of the gates because of this although regular maintenance should keep this risk to a minimum.

The hoisting machinery is reasonably simple and rugged which fulfills the most basic requirements for gates which are to be used in an emergency. However the following points are made in relation to good operating practice for spillway gates:-

1) Machinery and operators at Somerset Dam are exposed to the elements and there is some doubt as to whether the operating environment is safe enough in high winds and rain, especially in a severe flood situation.

| | |
|---|---|
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1) Machinery and operators at Somerset Dam are exposed to the elements and there is some doubt as to whether the operating environment is safe enough in high winds and rain, especially in a severe flood situation.

Shelters for each set of hoisting machinery should be considered if the gates are going to continue to be operated locally. Even if remote control is installed provision for local control must still be sound.

2) Remote control and/or monitoring of the gates while spilling in major floods, from an observation station on or near the dam, is strongly recommended as an operator safety measure.

Power supply to the gates is via the old switchboards in the right abutment chamber. These boards could be flooded in a major flood. This arrangement is now being changed to provide greater security for the power supply. The supply also has north and south feeders which can be interchanged if one fails.

Auxiliary power is available from the 75 kVA standby diesel generator set located in the right abutment chamber. Again this power supply is vulnerable in a major flood and should be relocated to a higher level. The set is fairly old and a new diesel generator set should be installed to provide maximum reliability.

An alternative means is provided for operating the spillway gates. This consists of a portable electric gearmotor unit which is coupled to the main hoist to operate a gate. This is good, but still does not meet current standards of acceptable practice for standby operation in that it is not a completely independent means of operating the gates - it still depends on the same prime source of drive - electrical power via the spillway wiring.

A portable diesel engine auxiliary, or a portable diesel driven hydraulic unit would fulfil the requirements for standby operation in an emergency.

There is no position indication on the spillway gates. Settings are therefore approximate only and are at the discretion of the operators. Possibly only rough settings are all that is required, but this should be clarified for the longer term position on the spillway gates. The operators report that the gates have only been operated once in the last three years, and are generally fully opened when required to operate. The gates were originally intended for regulation of releases before Wivenhoe was built, but no longer serve that function. Consideration could be given to de-commissioning the gates and concentrating resources on the sluice outlets and regulator outlets which are vital for storage control and releases.

Operating instructions for the gates are fairly basic. Documentation is generally out of date and will be upgraded as part of this consultancy. Training instructions and manuals likewise will be upgraded.

The spillway gates are tested monthly at present. Sometimes they are tested more frequently if convenient. Regular testing of spillway gates is a good safety feature as it keeps operating staff familiar with the equipment and procedures. Monthly testing is recommended. Standby equipment should be tested more frequently.

In summary, the main advantage of the spillway gates at Somerset is that the equipment is fairly rugged and simple to operate. This is a good feature in emergency situations as the operators are confident that they can operate the gates without any major problems. However, they do not fully meet modern design criteria in respect of:-

1. Emergency standby operation provision.

2. Position indication provisions.

### 4.2 Spillway Gate Tests

Tests were carried out on three spillway gates on 7-3-95. Lake level was 95.79m.

1) No.2 Gate was opened on normal power using the local pushbutton station on the hoist. The gate was fully opened. Opening time 3mins 40secs.

   The gate was then closed.

   Gate operation was very smooth throughout opening and closing cycle with no evidence of vibration or binding.

   Note: when fully closed, the pushbutton 'open' is operated briefly to release tension on the counterweight spring.

2) No.6 Gate was then opened about $1^1/_2$ metres using the auxiliary gearmotor unit. (1 h.p. electric drive). This unit connects to the hand drive shaft on the main hoist.

   The gearmotor unit was set up, trimmed and connected. Power was from the sluice gate control panel nearby. The main solenoid brake was released manually.

   Operation is slow in this mode (about 17mins full stroke).
   Operation was very smooth as for Gate 2.

3) No.4 Gate was opened using the auxiliary diesel set in the right abutment chamber. Procedure as for normal power operation.

   Note: the diesel set switches in automatically in the event of power failure - starts up within 5 seconds and changes over to emergency power for whole dam.

Gate opened to about 50%. Operation smooth as before.

### 4.3 Sluice Gates

Two of the sluice gates were inspected on 7-3-95, Sluice Gate 'K' and No.3 Regulator sluice gate.

1) No.3 Regulator Sluice Gate

This gate was accessed through the No.3 Regulator valve at the downstream end of the conduit.

The sluice gate was in fairly good condition. New stainless steel roller tracks were fitted in 1984. Gate rollers and pins have been replaced with stainless steel components over the last few years.

Gate leakage was moderate only, mainly at the top corners and some at the bottom seal.

The steel lining downstream of the gate was repainted in 1984 and has been patch-painted since then. The liner surface appeared to be in good condition.

2) Sluice Gate 'K'

This gate was accesssed via the gate shaft from the dam upper gallery.

The gate was stripped down in 1982 and reconditioned. Only the roller tracks remain to be refurbished and this will complete the replacement of all roller tracks on all sluice gates at Somerset. Gate rollers and pins have been replaced with stainless steel.

The steel liner which extends for about 6 metres downstream was in fairly good condition. Concrete surfaces beyond the steel liner were moderately rough but still quite servicable.

Gate leakage was moderate only.

Sluice gate handling gear was also inspected. This equipment was in good condition and has been improved over the years. It is used fairly regularly for maintenance operations.

Wire rope hoists for the sluice gates were also inspected and appeared to be in good condition. Gearing, sheaves and wire ropes were well greased.

The hoists have been re-roped in the last 12 months. Only basic maintenance is required on these hoists - bearings, ropes etc.

The sluice gates are tested every 6 months under flow conditions. They are opened to 1.8 metres, closed again, then the coaster gate put down and the conduit between the entrance and the sluice gate inspected. This is done for all 8 gates. The hydro outlet sluice gate is tested monthly, and is shut down annually for maintenance.

The sluice gates provide a means of dewatering the downstream conduit fairly rapidly for inspection and maintenance.

Power supply arrangements are generally as for the spillway gates and should be adequate with improvement of the standby generator facility.

In summary, the sluice gates and operating machinery are considered serviceable and should continue to meet their required function provided routine maintenance and regular testing is continued.

### 4.4 Sluice Gate Tests

Two sluice gates were tested under flow conditions on 7-3-95, Sluice Gate 'P' and Sluice Gate 'M'.

1) Sluice Gate 'P'

   This gate was opened on local control from the pushbutton station on the dam crest.

   The gate was opened approximately 1.8 metres (50%) and closed again. Opening time = 2 minutes. Operation smooth, no vibration. Re-sealing of the gate was relatively good (resealing can be a problem with these gates).

2) Sluice Gate 'M'

   Gate opened to 1.8m on local control. Operation smooth. Resealing satisfactory.

### 4.5 Emergency Coaster Gate

The emergency coaster gate was inspected on 7-3-95 in its storage chamber on the left abutment of the dam.

The gate was only in fair condition. Rollers were in poor condition.

This gate is the main 'problem' gate at Somerset. It is a large caterpillar roller type coaster gate designed to close against flow at the entrance to the sluice outlets. It has never been tested in flow conditions. The gate seals are very small and are in bad condition. It is intended to replace these seals to make the gate more useful as as bulkhead for maintenance of the upstream portion of the sluice conduits. At present a reasonable degree of sealing can be achieved using sawdust dropped on the upstream side of the closed gate.

The gate structural design is probably adequate for closure against flow. Many large holes are provided in the transverse beam webs and in the vertical web members. This would help in equalising the severe pressure differentials which can occur in the lower part of a coaster gate when closing against high head. However, three factors are present which make successful closure in flow conditions unlikely:-

1) downstream sealing coaster gates are prone to suffer strong downpull effects during closure against flow.

2) the long wire rope suspension can lead to severe 'bouncing' effects when closing against flow.

3) there is uncertainty about the condition of the gate frames. The sealing frames and roller tracks were inspected by a diver about 10 years ago and reported as O.K. but the true condition may be questionable now. Further investigation of this aspect is recommended.

The spring loaded guides were inspected. These appeared to have never moved properly and could jam if the gate was closed in flow conditions.

The gate is placed in position using the 100 tonne gantry crane.

Based on the above observations the following conclusions are drawn:-

1. The ability of the coaster gate to close safely against flow is doubted.

2. The gate probably should be downgraded from emergency gate status to stillwater bulkhead status.

This would be acceptable for the regulator outlets which have steel liners upstream of the sluice gate, and regulator valves protected by the sluice gates.

In the case of the sluice outlets, emphasis should be placed on continuing to upgrade the sluice gates and maintain their operational status.

### 4.6 Regulator Valves

The four regulators were inspected on 6-3-95 and 7-3-95.

All valves appeared to be in good condition. Controls were in fair condition, although electrical wiring required upgrading. This is planned.

The regulators are tested every month to 15% opening (dry test) and once per year to 100% opening with water on.

Provided the valves are properly maintained and tested regularly there should be no major safety concerns here.

### 4.7 Regulator Valve Tests

An operating test was carried out on No.13 Regulator on 7-3-95.

The regulator was opened to 40% on normal local control from the control room on the dam crest and closed again. Operation was satisfactory.

The valve controls have a key interlock for security between the two control positions at the dam crest and in the valve control room.

The operators reported that new pushbutton controls will be installed.

### 4.8 Baulks

These bulkheads are used for maintenance only and are not suitable for use in flow.

Their operation was reported as generally satisfactory. A diver is required while placing/removing the baulks. Sealing is by timber (pine) strips attached to the baulks.

### 4.9 Sump Pumps

Two sump pumps are located in a sump at each end of the dam gallery.
One is submerged and one about 3 metres higher for emergency use only. There is a linking chamber from the power station for dewatering purposes.

Condition of the pipework appeared satisfactory and operation of the pumps reported as O.K.

The pumps are tested weekly and drains checked.

Provided the pumps are properly maintained, and tested regularly, there should be no major safety concerns here.

### 4.10 Gantry Crane

The gantry crane was inspected on 7-3-95.

The main and auxiliary hoisting equipment was in fair condition. The operators reported no major problems, the crane generally performing its functions fairly well.

A catwalk has been added to provide better and safer access to the auxiliary hoist.

The new cable reeler makes operation of the gantry much more efficient - 18 minutes for full travel of the crest compared to $1^1/_2$ hours previously.

The gantry was tested on the travel motion and the hoist raise and lower motions.

### 4.11 Diesel Generator Set

The 75kVA standby generator located in the right abutment chamber was inspected. This unit supplies power for the entire dam in the event of failure of the main supply. The unit was installed in the mid '50s and is probably nearing the end of its useful life.

Automatic changeover to start the diesel on failure of the main supply was added last year.

The main risk with the diesel generator set is that it is located in a room vulnerable to flooding. The floor was only 3 feet clear of the 1974 flood. An emergency power provision should be located in a secure area even in the maximum flood. Relocation at a higher level should therefore be a high prioity.

The set is test run every two weeks.

### 4.12 Documentation

Documentation for operation and maintenance of the mechanical equipment was discussed with the operators. The primary documents existing at present are:-

1) "Manual of Maintenance and Operating Procedures of Mechanical and Electrical Services" Brisbane City Council Department of Water Supply and Sewerage.

This document was prepared about 15 years ago, and includes procedures for flood releases, procedures for operation of equipment, maintenance procedures, staff training/briefing, emergency procedures (1 page only, needs corrections), and drawings.

Operation and maintenance documentation will be extensively updated as part of this consultancy.

2) "Manual of Operational Procedures for Flood Mitigation for Wivenhoe Dam and Somerset Dam" ( 65 pages) 10 Sept.1992

This manual covers staffing, communications, objectives, flood classification, emergency procedures, operation considerations, hydrological data, etc. for Wivenhoe and Somerset Dams.

Documentation on these aspects will also be updated as part of this consultancy.

Maintenance procedures were discussed. Contractors are used for some of the mechanical and electrical equipment maintenance e.g. Demag for the checks on hoists, Westinghouse for checks on some of the electrical equipment, safety specialists for ladders, etc.

Routine maintenance records are kept on a card system. A computer-based record system will be introduced to replace this.

## 4.13 Communications

Communications at Somerset Dam consist of:-

Phones (2 lines)
Radios (2) in BCC radio link system - 1 with repeater
SEQWB - own radio system
Mobile phones (all rangers)
Power Station - radio link with SEQWB
Vehicle radios (all vehicles)

Adequate communication systems appear to be available for most emergencies. This will be addressed in the documentation prepared as part of this consultancy.

## 5. RECOMMENDATIONS

The following recommendations are made in relation to the mechanical equipment at Somerset Dam:-

1. Remote control and/or monitoring of the gates while spilling in major floods, from an observation station on or near the dam, is strongly recommended as an operator safety measure.

2. An alternative means of operating the spillway gates, independent of the main electrical supply wiring, should be provided. A portable diesel engine auxiliary, or a portable diesel driven hydraulic unit would fulfil the requirements for standby operation in an emergency.

3. The existing diesel generator set should be upgraded and relocated to a higher level above potential flooding zones.

4. Shelters should be provided for each set of hoisting machinery for the spillway gates to allow safe local operation of the gates in extreme weather conditions.

5. Local and remote position indication should be provided for the spillway gates.

6. Means of increasing the torsional stiffness of the spillway gates should be investigated in order to reduce the potential of the gates to bind on the side walls because of racking of the gates.

7. Consideration should be given to de-commissioning the spillway gates and concentrating resources on the sluice outlets and regulator outlets which are vital for storage control and releases.

8. Efforts to improve the performance of the sluice gates should be continued as more reliance will be placed on these gates in the future.

9. The emergency coaster gate should be downgraded to stillwater bulkhead status and increased reliance placed on the sluice gates. The condition of the gate sealing frames should also be investigated in more detail.

# APPENDIX B

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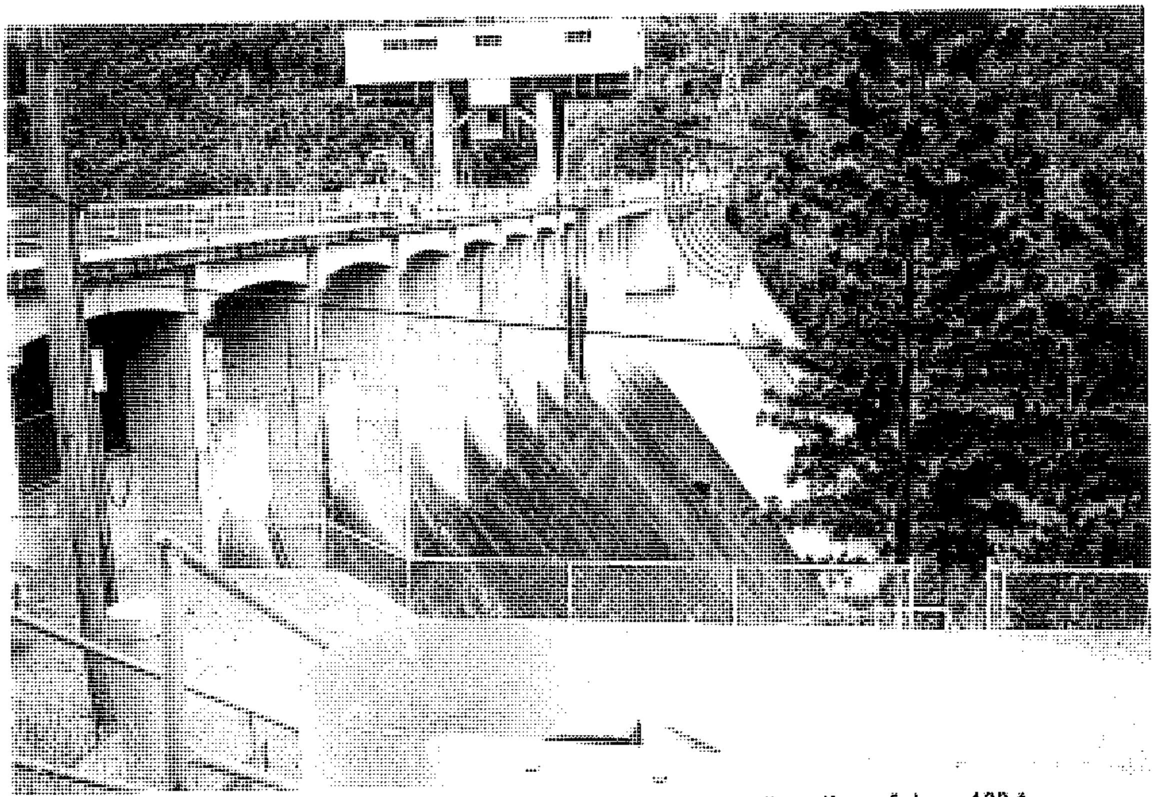
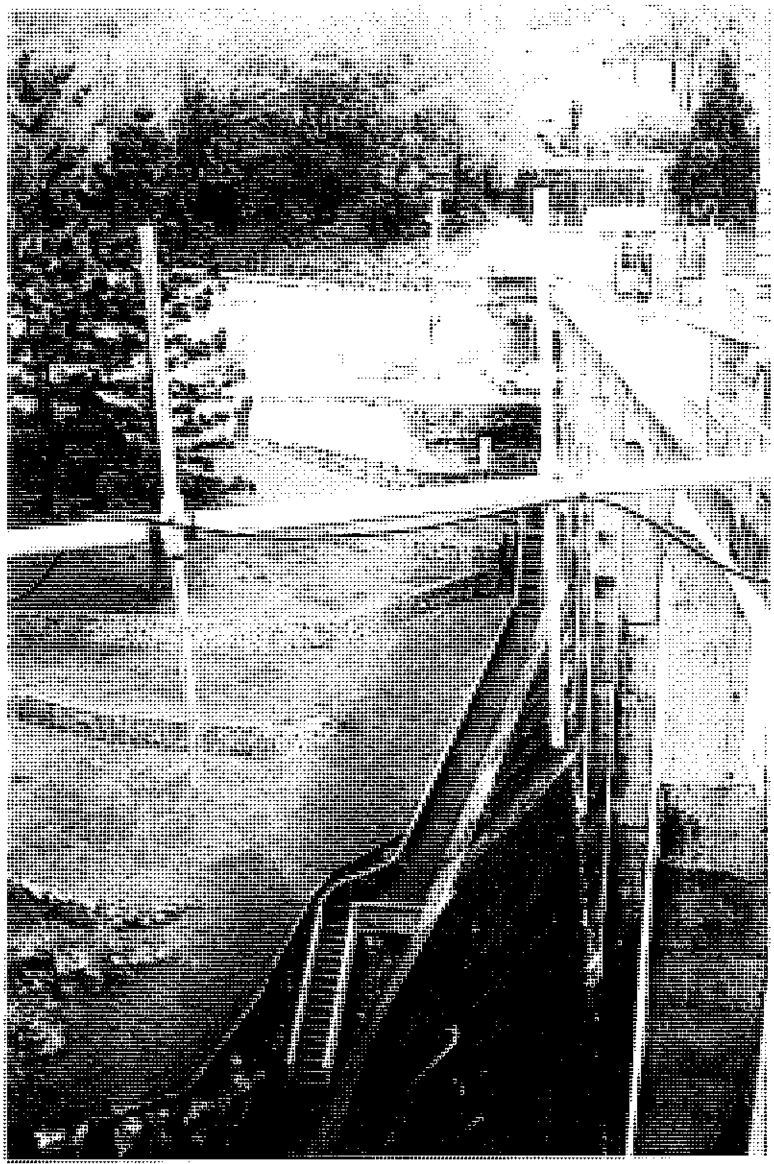
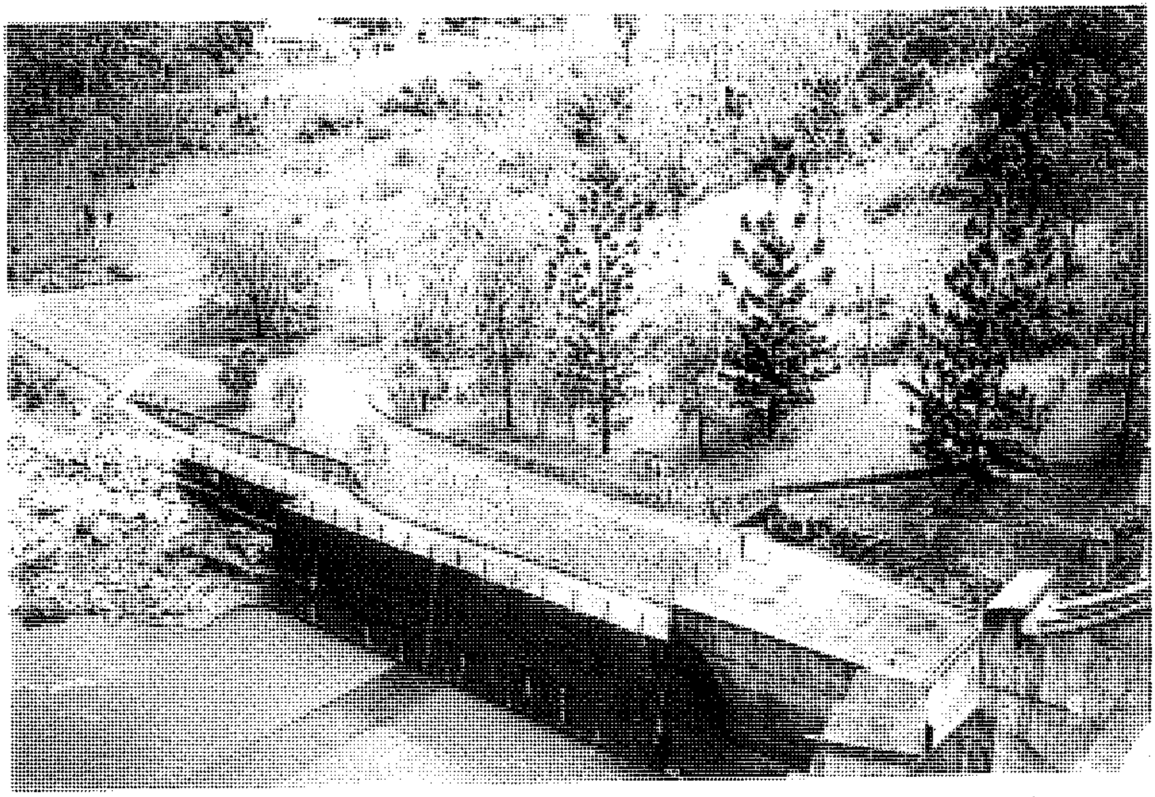
## Somerset Dam Photographs
## Taken on 6.3.95



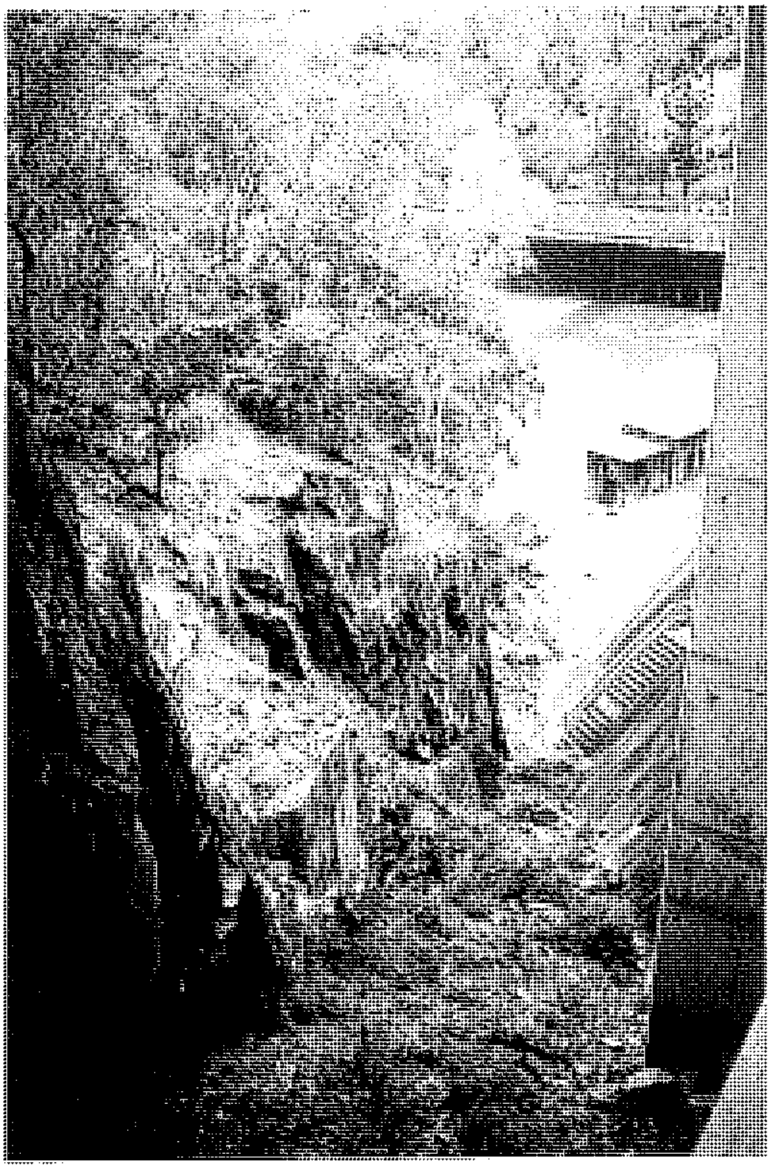
Somerset Dam from Right Abutment - Note: "non overflow" portion of dam, 100 t gantry crane, counterweight slides for spillway gates, gantry tower used for cable way during construction of the dam.



Right Abutment of Somerset Dam from Spillway - Note power pole placed downstream of "non overflow" section of dam. BCC vehicle at entrance to diesel emergency backup workshop entrance and landscaping

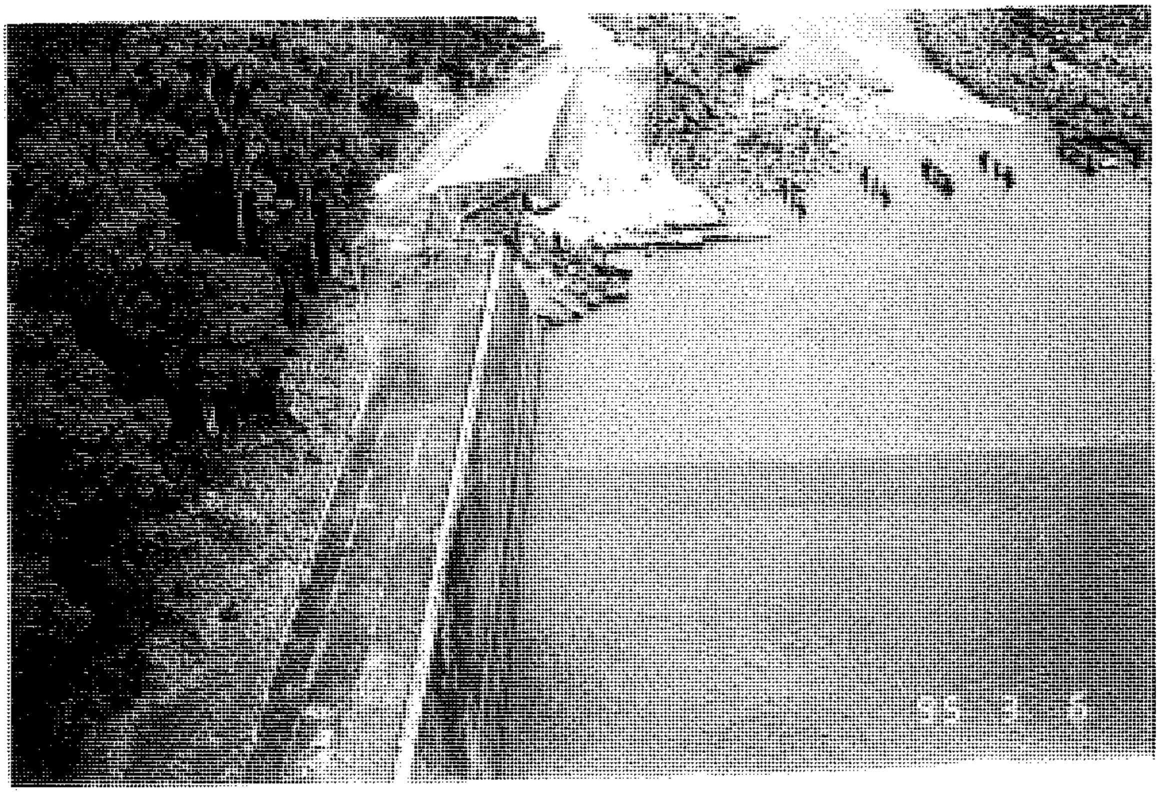


Right Abutment Stilling Basin Walls - Note weep holes, rock outcrops counterweight slides for spillway gates, signs of weathering "alkali; aggregate reactivity" in gate pier, backfilling behind retaining walls, stilling basin baffle, rip-rap downstream of stilling basin.

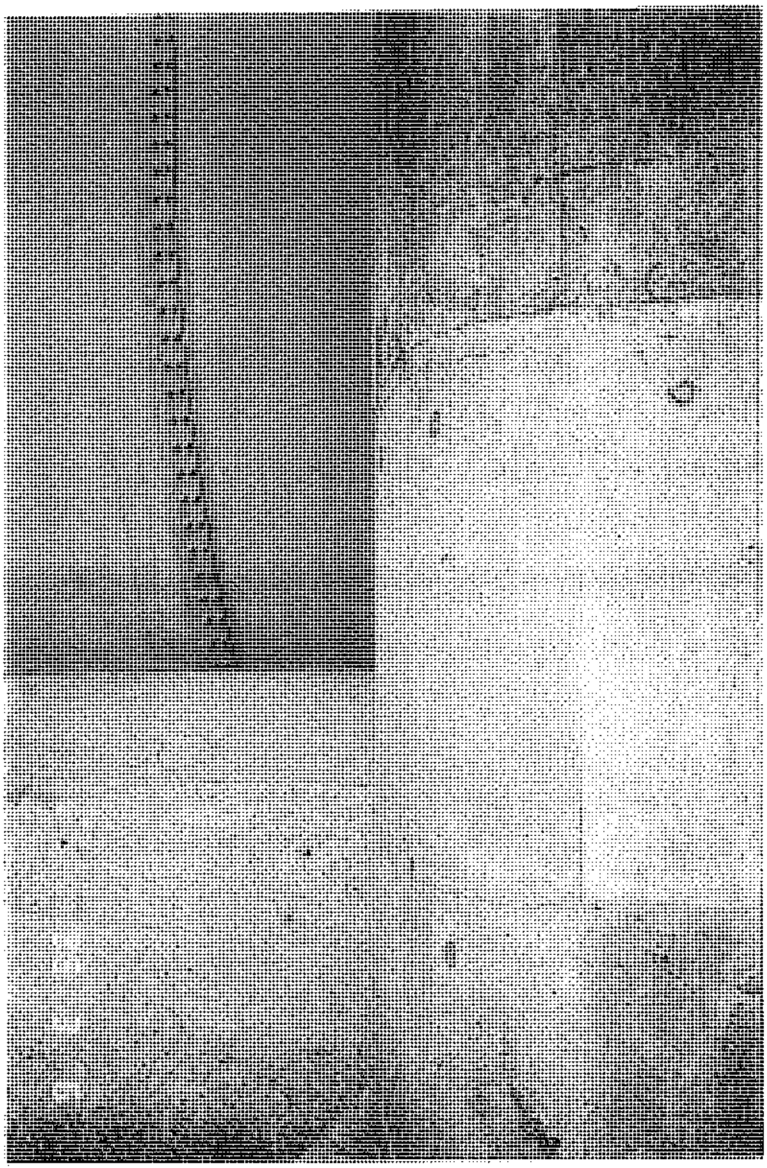


Left Abutment - Note dyke which is identified on the geological plan on local grid between P950 and P1200 and concrete block apparently between andesite and dyke.

Upstream Face of Dam on Left Abutment - crack about 0.5 m above FSL - approximately 0.5 mm to 1 mm wide.



Left Abutment Stilling Basin Wall - Note backfill to full height, no weep holes, grass growing in joints on top sealing slab.



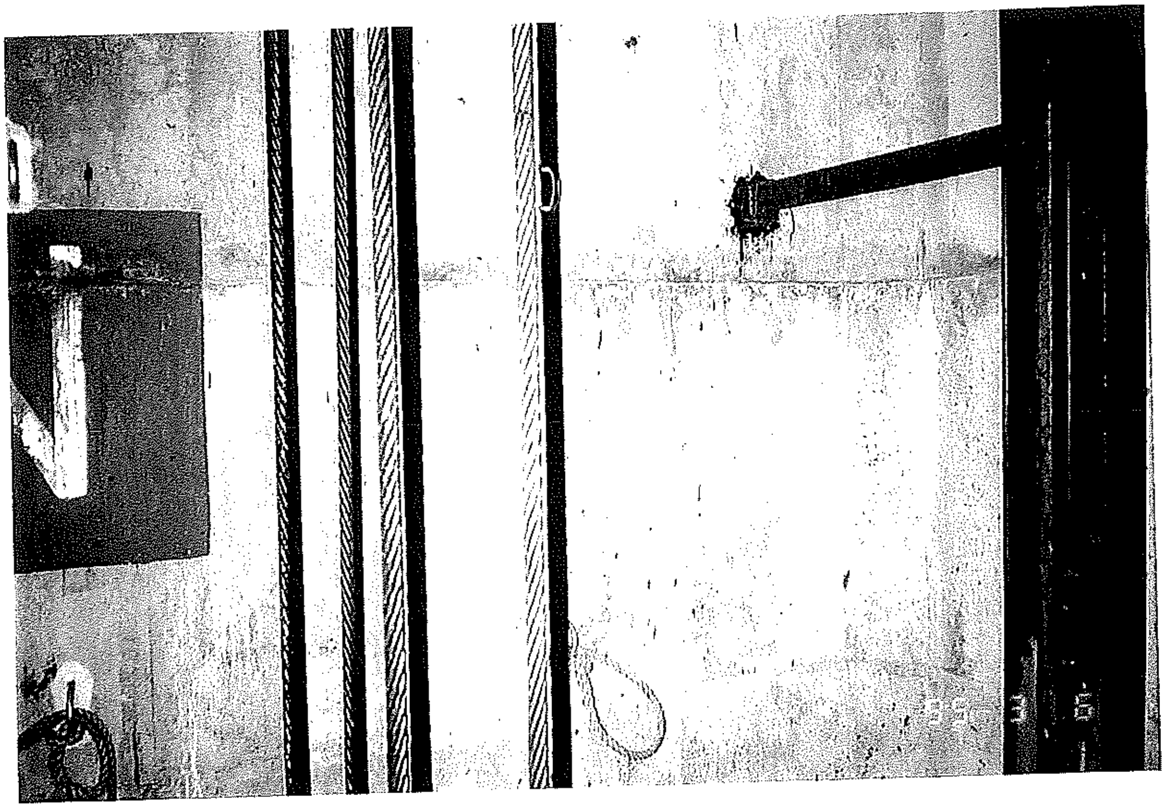
Right Hand Side of Spillway, Note cracking in joint between pours.

Upstream Face of Dam - Note gantry, intakes to regulators diversion baulk, "non flow" section of dam, access door to intake structure at same level as "non overflow" section.

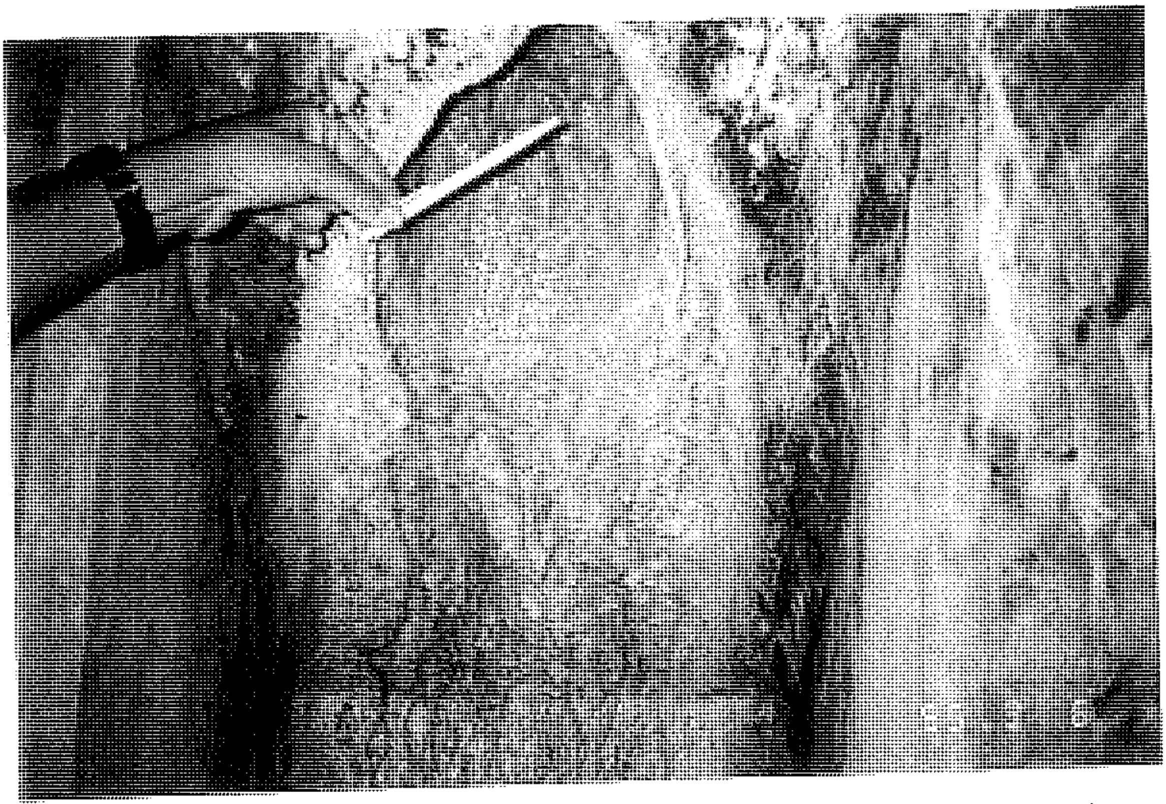
Downstream Face of Dam - Note calcite staining along construction joints.

Sluice Shaft Construction Joint in Monolith I about 3 m above upper gallery level (approximately RL 91.5). A crack in the upstream face of the dam was noted at RL 95.3 in Monoliths F, S and R.


Seepage through Joint L/M - Water entering about 2 m above gallery floor.

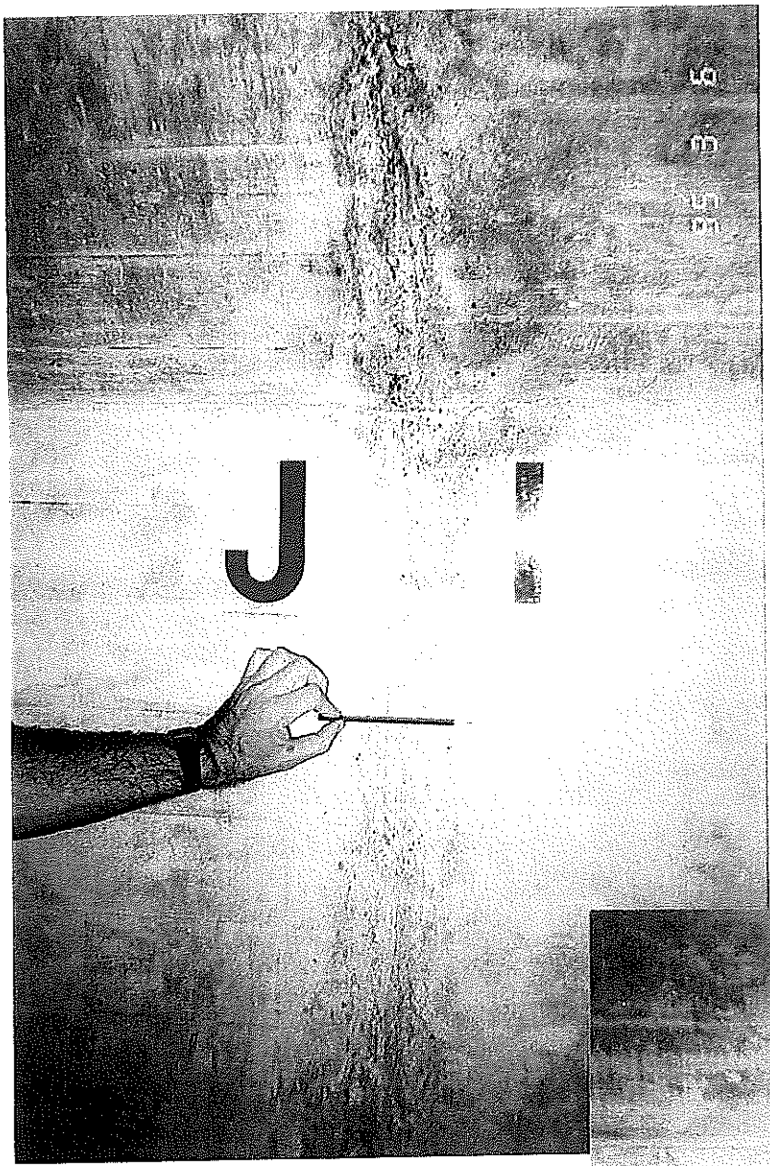


Calcite from old weep in sluice shaft in Monolith N.



Contraction Joint Drain in Monolith S lower gallery - Note that in 1977 a horizontal crack was found in the upstream face of the dam at about RL 97.2.

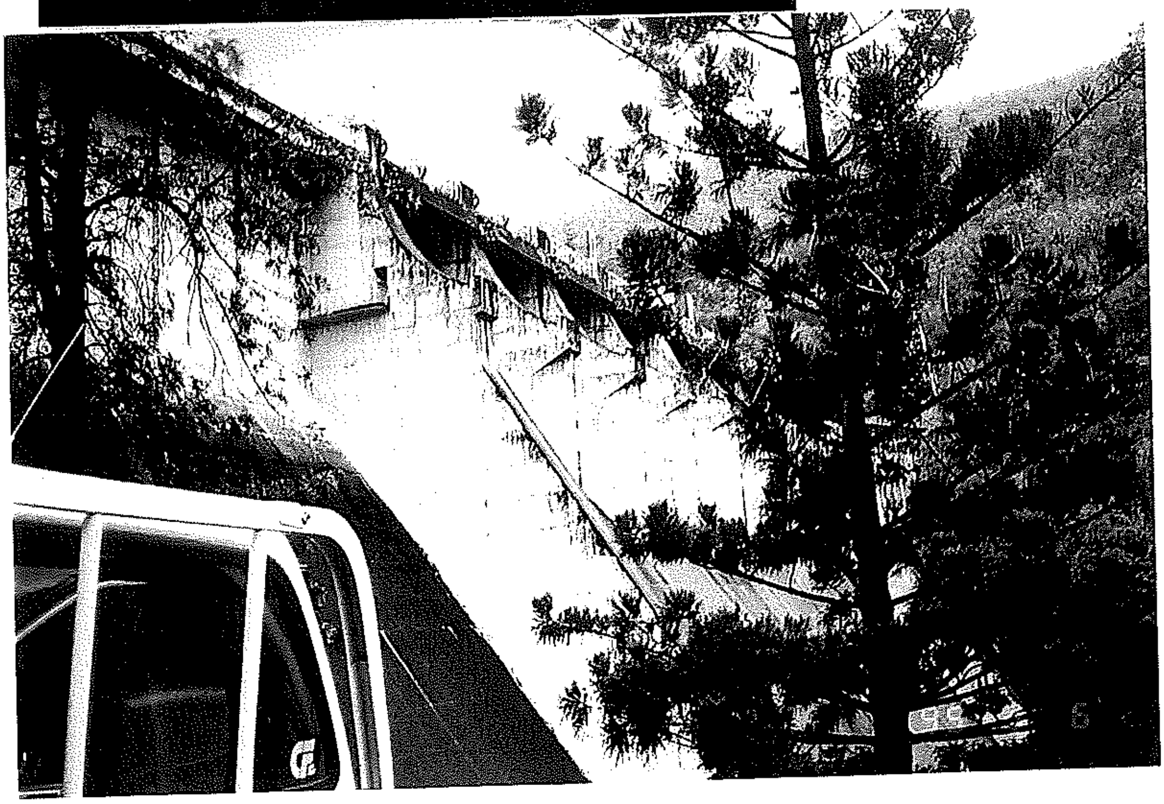
Cored Drain in Monolith R on downstream side of gallery.




Downstream face of joint between Monoliths I and J in lower gallery. Seepage from downstream face indicates drains not acting to completely stop seepage.


Downstream face of contraction joint between Monoliths L and M in lower gallery. Note that at the crest this joint is open, in the upper gallery the joint indicates a crack has developed and in the lower gallery the joint may show signs of being compressed.

Contraction Joint Drain between Monoliths I and J.



Downstream Face at Dam - Note staining on gate piers.

# APPENDIX C

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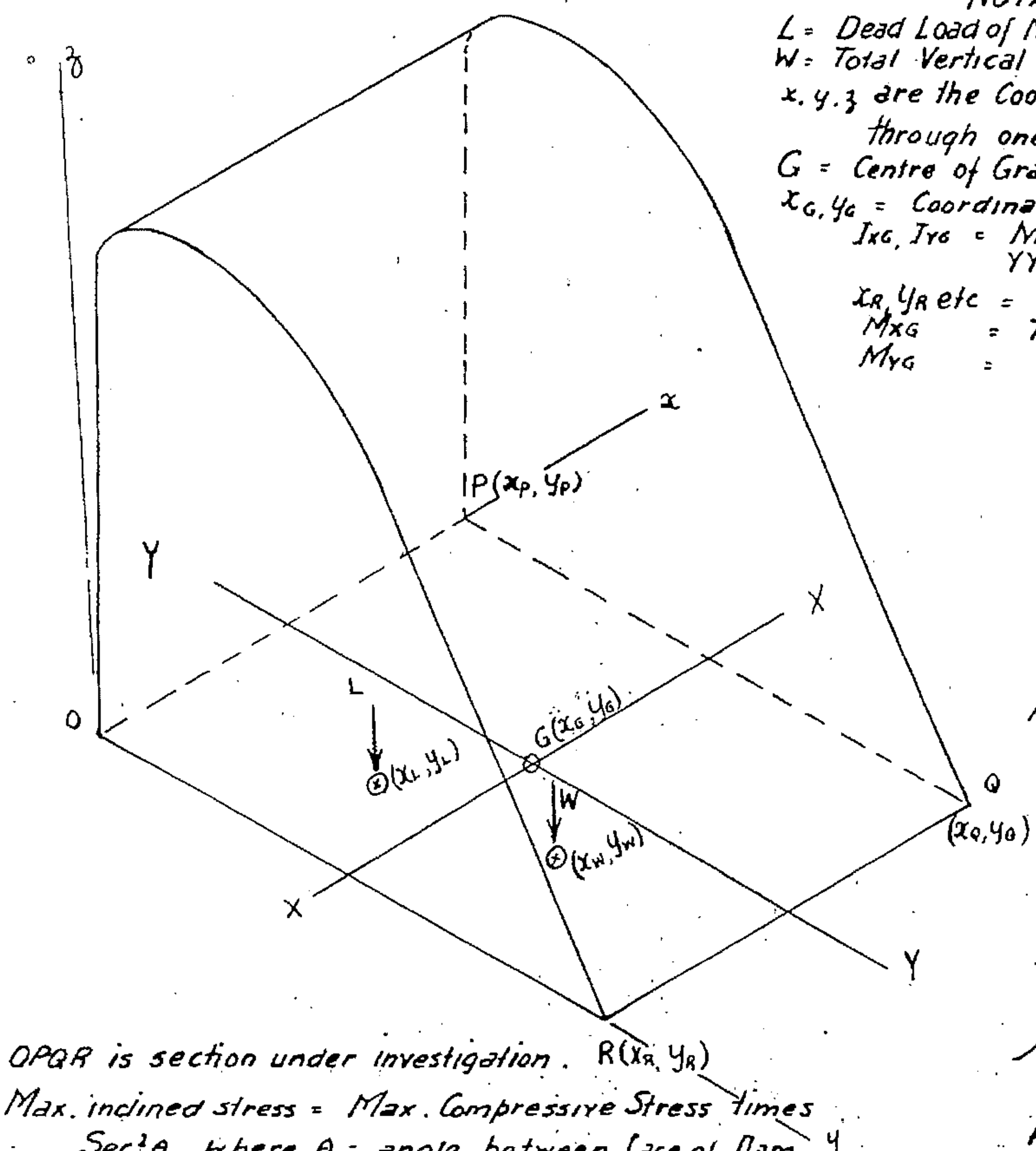
# Summary of Results from Original Design Calculations




OPQR is section under investigation.

Max. inclined stress = Max. Compressive Stress times $Sec^2\theta$, where $\theta$ = angle between face of Dam and Vertical.

## NOTATION

L = Dead Load of Masonry, Gates, etc. above section.
W = Total Vertical Load.
x, y, z are the Coordinates of points of action referred to axes $O_x$, $O_y$, $O_z$, through one corner of the section.
G = Centre of Gravity of the section.
$x_G$, $y_G$ = Coordinates of G
$I_{XG}$, $I_{YG}$ = Moments of Inertia of Section OPQR, referred to axes XX, YY, passing through G, and parallel to the sides of the section
$x_R$, $y_R$ etc = coordinates of the corners of the section.
$M_{XG}$ = Total moment about XX (+ve clockwise) looking along Ox
$M_{YG}$ = " " " YY ( " " ) " " Oy

## STRESSES

Compressive taken as positive

$$f_R = \frac{W}{A} + \frac{M_{XG}(y_R - y_G)}{I_{XG}} - \frac{M_{YG}(x_R - x_G)}{I_{YG}}$$

$$f_Q = \frac{W}{A} + \frac{M_{XG}(y_Q - y_G)}{I_{XG}} - \frac{M_{YG}(x_Q - x_G)}{I_{YG}}.$$

$$f_P = \frac{W}{A} + \frac{M_{XG}(y_P - y_G)}{I_{XG}} - \frac{M_{YG}(x_P - x_G)}{I_{YG}}$$

$$f_O = \frac{W}{A} + \frac{M_{XG}(y_O - y_G)}{I_{XG}} - \frac{M_{YG}(x_O - x_G)}{I_{YG}}$$

NOTE: $y_R = y_Q$, $x_R = 0$, $y_P = 0$, $x_Q = x_P$, $x_O = 0$, $y_O = 0$

$e_1$ = distance from centroid to line of action of Resultant, which would give no stress on upstream face, and = $\frac{I_{XG}}{A(y_R - y_G)}$

$e_2$ = distance from centroid to line of action of Resultant, which would give no stress on downstream face, and = $\frac{I_{XG}}{A(y_G - y_P)}$

$\mu_1$ = distance from centroid to line of action of Resultant, which would give no stress on left hand face, looking down stream, and = $\frac{I_{YG}}{A(x_G - x_O)}$.

$\mu_2$ = distance from centroid to line of action of Resultant, which would give no stress on right hand face looking downstream, and = $\frac{I_{YG}}{A(x_P - x_G)}$.

A = nett area of section under investigation.

# PROPERTIES 26' MONOLITHS I, K, N, & P

| Section | R.L. | Base Width | Area Sq. Ft. | Volume above Section Cu. Ft. | Volume above Section Cu. Yds. | $X_G$ | $Y_G$ | $I_{xG}$ | $I_{yG}$ | $e_1$ | $e_2$ | $\mu_1$ | $\mu_2$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 301·43 | 51·43 | 1172·0 | 23119 | 856 | 12·50 | 27·10 | 257383 | 64783 | 9·03 | 8·10 | 4·42 | 4·42 |
| 2 | 298·67 | 53·50 | 1123·7 | 26423 | 979 | " | 28·16 | 289790 | 67477 | 9·35 | 8·41 | 4·41 | 4·41 |
| 2.a. | 298·67 | 53·50 | 1079·9 | 26423 | 979 | " | 29·49 | 273208 | 59990 | 10·54 | 8·58 | 4·44 | 4·44 |
| 3. | 292 | 58·50 | 1205·0 | 34036 | 1261 | " | 32·17 | 355116 | 66503 | 11·19 | 9·16 | 4·42 | 4·42 |
| 3.a. | 292 | 58·50 | 1348·7 | 34036 | 1261 | " | 30·71 | 379321 | 73990 | 10·12 | 9·16 | 4·39 | 4·39 |
| 4 | 270 | 75·00 | 1814·6 | 69419 | 2571 | " | 38·32 | 841319 | 96922 | 12·64 | 12·10 | 4·27 | 4·27 |
| 5 | 252 | 88·5 | 2154·8 | 105143 | 3894 | " | 45·07 | 1388939 | 114532 | 14·84 | 14·30 | 4·25 | 4·25 |
| 6 | 246 | 93·0 | 2205·4 | 118470 | 4388 | " | 48·611 | 1480692 | 119650 | 15·13 | 13·81 | 4·34 | 4·34 |
| 6.a. | 246 | 93·0 | 2087·1 | 118470 | 4388 | " | 50·48 | 1340074 | 119019 | 15·10 | 12·72 | 4·56 | 4·56 |
| 7 | 243 | 95·25 | 1960·9 | 124409 | 4608 | " | 53·07 | 1388833 | 120977 | 16·79 | 13·35 | 4·94 | 4·94 |
| 8 | 240 | 97·5 | 1919·5 | 130322 | 4827 | " | 54·31 | 1490335 | 123385 | 17·98 | 14·30 | 5·14 | 5·14 |
| 9 | 234 | 102·0 | 1897·6 | 141689 | 5248 | " | 55·23 | 1652007 | 128208 | 19·22 | 16·28 | 5·58 | 5·58 |
| 9.a. | 234 | 102·0 | 1992·3 | 141689 | 5248 | " | 52·16 | 1880069 | 129028 | 18·93 | 18·09 | 5·18 | 5·18 |
| 10 | 213·56 | 117·33 | 2933·2 | 191540. | 7094 | " | 58·66 | 3364742 | 152769 | 19·55 | 19·55 | 4·17 | 4·17 |
| 11 | 200 | 127·5 | 3187·5 | 233053 | 8632 | " | 63·75 | 4318059 | 166015 | 21·25 | 21·25 | 4·17 | 4·17 |
| 12. | 190 | 135·0 | 3375·0 | 265863. | 9847 | " | 67·50 | 5125780 | 175781 | 22·50 | 22·50 | 4·17 | 4·17 |
| Inclined | | 116·6 | 1940·7 | 145467 | 5388 | " | 59·37 | 2136080 | 145784 | 19·24 | 18·54 | 6·01 | 6·01 |

# DAM EMPTY 26' MONOLITHS I, K, N & P

| Section | R.L | Volume above Section | | Weight above Section, including gates. TONS | $X_L$ | $Y_L$ | Stress at Corners Tons per Sq. ft. | | | | Max. Inclined Stress | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. Ft. | Cu. Yds | | | | O | P | Q | R | O | P |
| 1 | 301·43 | 23119 | 856 | 1537·2 | 12·43 | 22·23 | 2·12 | 2·08 | ·58 | ·63 | 2·13 | 2·09 |
| 2 | 298·67 | 26423 | 979 | 1754·0 | 12·44 | 23·03 | 2·33 | 2·29 | ·63 | ·67 | 2·34 | 2·30 |
| 2.a. | 298·67 | 26423 | 979 | 1754·0 | 12·44 | 23·03 | 2·87 | 2·82 | ·61 | ·65 | 2·88 | 2·83 |
| 3. | 292 | 34036 | 1261 | 2253·6 | 12·45 | 25·06 | 3·34 | 3·30 | ·66 | ·70 | 3·35 | 3·31 |
| 3.a. | 292 | 34036 | 1261 | 2253·6 | 12·45 | 25·06 | 2·72 | 2·68 | ·72 | ·76 | 2·73 | 2·69 |
| 4 | 270 | 69419 | 2571 | 4575·7 | 12·48 | 30·58 | 4·15 | 4·12 | ·96 | ·99 | 4·16 | 4·13 |
| 5 | 252 | 105143 | 3894 | 6920·0 | 12·48 | 35·12 | 5·46 | 5·43 | 1·05 | 1·07 | 5·47 | 5·44 |
| 6 | 246 | 118470 | 4388 | 7794·6 | 12·49 | 36·63 | 6·61 | 6·59 | ·72 | ·75 | 6·62 | 6·61 |
| 6.a. | 246 | 118470 | 4388 | 7794·6 | 12·49 | 36·63 | 7·81 | 7·79 | ·30 | ·32 | 7·83 | 7·81 |
| 7 | 243 | 124409 | 4608 | 8184·4 | 12·49 | 37·53 | 9·04 | 9·02 | ·30 | ·32 | 9·06 | 9·04 |
| 8 | 240 | 130322 | 4827 | 8572·4 | 12·49 | 38·38 | 9·45 | 9·43 | ·50 | ·52 | 9·47 | 9·45 |
| 9 | 234 | 141689 | 5248 | 9360·6 | 12·49 | 39·84 | 9·92 | 9·90 | 1·00 | 1·03 | 9·94 | 9·92 |
| 9.a. | 234 | 141689 | 5248 | 9360·6 | 12·49 | 39·84 | 7·91 | 7·89 | 1·63 | 1·65 | 7·93 | 7·91 |
| 10 | 213·56 | 191540 | 7094 | 12632·0 | 12·49 | 44·08 | 7·53 | 7·51 | 1·09 | 1·10 | 7·55 | 7·53 |
| 11 | 200 | 233053 | 8632 | 15356·3 | 12·49 | 47·74 | 8·46 | 8·44 | 1·18 | 1·20 | 8·48 | 8·46 |
| 12 | 190 | 265863 | 9847 | 17509·5 | 12·49 | 50·41 | 9·13 | 9·12 | 1·24 | 1·26 | 9·15 | 9·14 |
| Inclined | | 145467 | 5388 | 9566·4 | 12·49 | 44·10 | 8·73 | 8·71 | ·98 | 1·00 | 9·07 | 9·05 |

# DAM FULL 26' MONOLITHS I.K.N & P

| Section | R.L. | Volume above Section | | Weight above Section including Gates TONS | $X_w$ | $Y_w$ | Stress at Corners Tons per sq. ft. | | | | Max. Inclined Stress | | Stress at up-stream Toe | Pressure of full head of water | Uplift Coeff. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. Ft. | Cu. Yds | | | | O | P | Q | R | Q | R | | | |
| 1 | 301·43 | 23119 | 856 | 1911·9 | 12·44 | 24·90 | 2·10 | 2·05 | 1·21 | 1·25 | 1·81 | 1·86 | 2·05 | 1·50 | 1·37 |
| 2 | 298·67 | 26423 | 979 | 2134·2 | 12·45 | 26·37 | 2·14 | 2·10 | 1·39 | 1·43 | 2·07 | 2·13 | 2·10 | 1·57 | 1·33 |
| 2.a. | 298·67 | 26423 | 979 | 2134·2 | 12·45 | 26·37 | 2·72 | 2·67 | 1·37 | 1·41 | 2·04 | 2·10 | 2·67 | 1·57 | 1·70 |
| 3 | 292 | 34036 | 1261 | 2648·2 | 12·45 | 30·06 | 2·73 | 2·68 | 1·76 | 1·81 | 2·62 | 2·70 | 2·68 | 1·76 | 1·53 |
| 3.a. | 292 | 34036 | 1261 | 2648·2 | 12·45 | 30·06 | 2·12 | 2·08 | 1·82 | 1·86 | 2·71 | 2·77 | 2·08 | 1·76 | 1·19 |
| 4 | 270 | 69419 | 2571 | 5029·3 | 12·48 | 40·27 | 2·34 | 2·31 | 3·19 | 3·21 | 4·75 | 4·78 | 2·31 | 2·37 | ·97 |
| 5 | 252 | 105143 | 3894 | 7435·1 | 12·49 | 48·40 | 2·66 | 2·64 | 4·21 | 4·24 | 6·27 | 6·32 | | | |
| 6 | 246 | 118470 | 4388 | 8021·0 | 12·49 | 52·81 | 2·54 | 2·52 | 4·64 | 4·66 | 6·91 | 6·94 | | | |
| 6.a. | 246 | 118470 | 4388 | 8021·0 | 12·49 | 52·81 | 3·15 | 3·13 | 4·43 | 4·45 | 6·60 | 6·63 | | | |
| 7 | 243 | 124409 | 4608 | 8422·9 | 12·49 | 54·37 | 3·89 | 3·87 | 4·61 | 4·63 | 6·87 | 6·90 | | | |
| 8 | 240 | 130322 | 4827 | 8823·2 | 12·49 | 55·89 | 4·10 | 4·08 | 4·99 | 5·01 | 7·44 | 7·46 | | | |
| 9 | 234 | 141689 | 5248 | 9637·1 | 12·49 | 58·74 | 4·12 | 4·11 | 6·19 | 6·21 | 9·22 | 9·25 | | | |
| 9.a. | 234 | 141689 | 5248 | 9637·1 | 12·49 | 58·74 | 3·09 | 3·07 | 6·51 | 6·53 | 9·70 | 9·73 | | | |
| 10 | 213·56 | 191540 | 7094 | 13351·8 | 12·49 | 65·63 | 2·94 | 2·92 | 6·17 | 6·18 | 9·19 | 9·21 | | | |
| 11 | 200 | 233053 | 8632 | 16149·0 | 12·49 | 71·72 | 3·17 | 3·16 | 6·96 | 6·97 | 10·37 | 10·39 | | | |
| 12 | 190 | 265863 | 9847 | 18360·1 | 12·49 | 76·18 | 3·35 | 3·33 | 7·53 | 7·55 | 11·22 | 11·25 | | | |
| Inclined | | 145467 | 5388 | 9794·4 | 12·49 | 64·70 | 3·12 | 3·10 | 5·57 | 5·59 | 12·96 | 13·00 | | | |

NOTE: Uplift coefficient for this and for other monoliths is the ratio between minimum pressure on upstream edge of section, and full water pressure.

# DAM FULL WITH BACKWATER 26' MONOLITHS I, K, N & P.

| Section | R.L. | Volume above Section | | Weight above section, including Gates TONS | Xw | Yw | Stress at Corners Tons per sq. ft. | | | | Max. Inclined Stress | | Stress at Upstream Toe | Pressure of full head of water | Uplift Coeff. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. Ft. | Cu. Yds. | | | | O | P | Q | R | Q | R | | | |
| 5 | 252 | 105143 | 3894 | 7516·6 | 12·49 | 48·69 | 2·62 | 2·59 | 4·33 | 4·35 | 6·45 | 6·48 | 2·59 | 2·87 | ·90 |
| 6 | 246 | 118470 | 4388 | 8123·1 | - | 53·41 | 2·41 | 2·40 | 4·84 | 4·86 | 7·21 | 7·24 | 2·40 | 3·04 | ·79 |
| 6.a. | 246 | 118470 | 4388 | 8123·1 | - | 53·41 | 3·01 | 2·99 | 4·64 | 4·65 | 6·91 | 6·93 | 2·99 | 3·04 | ·98 |
| 7 | 243 | 124409 | 4608 | 8608·0 | - | 55·30 | 3·60 | 3·57 | 4·86 | 4·89 | 7·24 | 7·29 | 3·57 | 3·13 | 1·14 |
| 8 | 240 | 130322 | 4827 | 8839·5 | - | 56·72 | 3·84 | 3·82 | 5·22 | 5·23 | 7·78 | 7·79 | 3·82 | 3·21 | 1·19 |
| 9 | 234 | 141689 | 5248 | 9576·8 | - | 59·48 | 3·86 | 3·84 | 6·36 | 6·37 | 9·48 | 9·49 | 3·84 | 3·38 | 1·14 |
| 9.a. | 234 | 141689 | 5248 | 9576·8 | - | 59·48 | 2·87 | 2·85 | 6·66 | 6·67 | 9·92 | 9·94 | 2·85 | 3·38 | ·84 |
| 10 | 213·56 | 191540 | 7094 | 14148·3 | - | 66·26 | 2·96 | 2·94 | 6·69 | 6·71 | 9·97 | 10·00 | 2·94 | 3·95 | ·75 |
| 11 | 200 | 233053 | 8632 | 17054·1 | - | 72·35 | 3·19 | 3·18 | 7·51 | 7·52 | 11·19 | 11·20 | 3·18 | 4·33 | ·74 |
| 12 | 190 | 265863 | 9847 | 18997·5 | - | 76·96 | 3·27 | 3·26 | 7·99 | 8·00 | 11·90 | 11·92 | 3·26 | 4·60 | ·71 |
| Inclined | | 145467 | 5388 | 9915·5 | - | 66·82 | 2·73 | 2·71 | 6·30 | 6·32 | 14·66 | 14·70 | 2·71 | 3·05 | ·89 |

Vide Note, Page 4.

# PROPERTIES 37' 4" MONOLITHS J & O

| Section | R.L | Base Width | Area Sq. ft. | Volume above Section Cu. Ft. | Volume above Section Cu. Yds | $X_G$ | $Y_G$ | $I_{xG}$ | $I_{YG}$ | $e_1$ | $e_2$ | $\mu_1$ | $\mu_2$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 301·43 | 51·43 | 1806·2 | 62425 | 2312 | 18·67 | 26·62 | 397979 | 220818 | 8·88 | 8·28 | 6·55 | 6·55 |
| 2 | 298·67 | 53·50 | 1883·5 | 67515 | 2501 | .. | 27·67 | 448042 | 229790 | 9·21 | 8·60 | 6·54 | 6·54 |
| 2.a. | 298·67 | 53·50 | 1668·8 | 67515 | 2501 | - | 28·89 | 425677 | 204858 | 10·36 | 8·83 | 6·58 | 6·58 |
| 3. | 292 | 58·50 | 1855·5 | 79257 | 2935 | .. | 31·56 | 552933 | 226547 | 11·06 | 9·44 | 6·54 | 6·54 |
| 3.a. | 292 | 58·50 | 2070·2 | 79257 | 2935 | .. | 30·20 | 586176 | 251479 | 10·01 | 9·38 | 6·51 | 6·51 |
| 4 | 270 | 75·00 | 2739·6 | 132752 | 4917 | .. | 38·04 | 1275320 | 324472 | 12·60 | 12·24 | 6·35 | 6·35 |
| 5 | 252 | 88·50 | 3246·2 | 186624 | 6912 | .. | 44·79 | 2101844 | 383041 | 14·81 | 14·46 | 6·32 | 6·32 |
| 6 | 246 | 93·00 | 3352·4 | 206666 | 7654 | .. | 47·89 | 2310747 | 401813 | 15·28 | 14·39 | 6·42 | 6·42 |
| 6.a. | 246 | 93·00 | 3234·0 | 206666 | 7654 | .. | 49·07 | 2178516 | 401182 | 15·33 | 13·73 | 6·65 | 6·65 |
| 7 | 243 | 95·25 | 3135·7 | 216089 | 8003 | .. | 51·03 | 2298763 | 409975 | 16·58 | 14·37 | 7·00 | 7·00 |
| 8 | 240 | 97·50 | 3122·0 | 225566 | 8354 | .. | 52·17 | 2465774 | 419200 | 17·42 | 15·14 | 7·19 | 7·19 |
| 9 | 234 | 102·00 | 3095·6 | 244315 | 9049 | - | 53·51 | 2756032 | 437676 | 18·36 | 16·64 | 7·57 | 7·57 |
| 9.a. | 234 | 102·00 | 3250·2 | 244315 | 9049 | - | 51·71 | 2971784 | 438496 | 18·18 | 17·68 | 7·23 | 7·23 |
| 10 | 213·56 | 117·33 | 4380·2 | 321338 | 11901 | - | 58·67 | 5024691 | 508739 | 19·56 | 19·56 | 6·22 | 6·22 |
| 11 | 200 | 127·50 | 4760·0 | 383326 | 14197 | .. | 63·75 | 6448312 | 552850 | 21·25 | 21·25 | 6·22 | 6·22 |
| 12 | 190 | 135·00 | 5040·0 | 432326 | 16012 | - | 67·50 | 7654500 | 585385 | 22·50 | 22·50 | 6·22 | 6·22 |
| Inclined | | 116·59 | 3378·6 | 250416 | 9275 | .. | 58·91 | 3765806 | 499514 | 19·33 | 18·92 | 7·92 | 7·92 |

# DAM EMPTY 37'4" MONOLITHS J & O

| Section | R.L. | Volume above Section Cu. Ft. | Volume above Section Cu. Yds. | Weight above Section including Gates TONS | $X_L$ | $Y_L$ | Stress at Corners Tons per Sq. ft. O | P | Q | R | Max. Inclined Stress O | P |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 301·43 | 62425 | 2312 | 4116·6 | 18·64 | 20·94 | 3·85 | 3·83 | ·81 | ·83 | 3·86 | 3·84 |
| 2 | 298·67 | 67515 | 2501 | 4450·7 | 18·64 | 21·54 | 4·06 | 4·04 | ·78 | ·80 | 4·07 | 4·05 |
| 2.a. | 298·67 | 67515 | 2501 | 4450·7 | 18·64 | 21·54 | 4·89 | 4·88 | ·77 | ·79 | 4·90 | 4·89 |
| 3 | 292 | 79257 | 2935 | 5221·3 | 18·65 | 23·13 | 5·33 | 5·31 | ·66 | ·68 | 5·34 | 5·32 |
| 3.a. | 292 | 79257 | 2935 | 5221·3 | 18·65 | 23·13 | 4·43 | 4·42 | ·73 | ·75 | 4·44 | 4·43 |
| 4 | 270 | 132752 | 4917 | 8731·9 | 18·65 | 28·40 | 5·70 | 5·69 | ·74 | ·75 | 5·71 | 5·70 |
| 5 | 252 | 186624 | 6912 | 12267·2 | 18·66 | 32·94 | 6·88 | 6·87 | ·75 | ·76 | 6·90 | 6·89 |
| 6 | 246 | 206666 | 7654 | 13582·5 | 18·66 | 34·48 | 7·83 | 7·82 | ·49 | ·50 | 7·85 | 7·84 |
| 6.a. | 246 | 206666 | 7654 | 13582·5 | 18·66 | 34·48 | 8·67 | 8·66 | ·20 | ·21 | 8·69 | 8·68 |
| 7 | 243 | 216089 | 8003 | 14200·8 | 18·66 | 35·32 | 9·48 | 9·48 | ·23 | ·24 | 9·50 | 9·50 |
| 8 | 240 | 225566 | 8354 | 14822·8 | 18·66 | 36·13 | 9·78 | 9·77 | ·37 | ·38 | 9·80 | 9·79 |
| 9 | 234 | 244315 | 9049 | 16095·4 | 18·66 | 37·63 | 10·17 | 10·16 | ·70 | ·71 | 10·20 | 10·19 |
| 9.a. | 234 | 244315 | 9049 | 16095·4 | 18·66 | 37·63 | 8·90 | 8·89 | 1·11 | 1·12 | 8·92 | 8·91 |
| 10 | 213·56 | 321338 | 11901 | 21150·0 | 18·66 | 42·39 | 8·85 | 8·84 | ·81 | ·82 | 8·87 | 8·86 |
| 11 | 200 | 383326 | 14197 | 25218·0 | 18·66 | 46·06 | 9·71 | 9·71 | ·88 | ·89 | 9·73 | 9·73 |
| 12 | 190 | 432326 | 16012 | 28433·6 | 18·66 | 48·74 | 10·35 | 10·34 | ·94 | ·94 | 10·38 | 10·37 |
| Inclined | | 250416 | 9275 | 16452·4 | 18·66 | 41·80 | 9·00 | 8·99 | ·54 | ·55 | 9·34 | 9·33 |

# DAM FULL 37' 4" MONOLITHS J & O.

| Section | R.L. | Volume above Section Cu. Ft. | Volume above Section Cu. Yds | Weight above Section including Gates TONS | Xw | Yw | Stress at Corners Tons per sq. ft. O | P | Q | R | Max. Inclined Stress Q | R | Stress at up-stream Toe. | Pressure of full head of water | uplift. Coeff. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 301·43 | 62425 | 2312 | 4379·8 | 18·64 | 28·15 | 1·98 | 1·97 | 2·84 | 2·85 | 4·23 | 4·25 | 1·97 | 1·50 | 1·32 |
| 2 | 298·67 | 67515 | 2501 | 4721·7 | 18·64 | 29·24 | 2·06 | 2·04 | 2·93 | 2·94 | 4·37 | 4·38 | 2·04 | 1·57 | 1·30 |
| 2.a. | 298·67 | 67515 | 2501 | 4721·7 | 18·64 | 29·24 | 2·73 | 2·71 | 2·92 | 2·93 | 4·35 | 4·37 | 2·71 | 1·57 | 1·72 |
| 3 | 292 | 79257 | 2935 | 5513·0 | 18·65 | 32·14 | 2·80 | 2·78 | 3·12 | 3·14 | 4·65 | 4·68 | 2·78 | 1·76 | 1·58 |
| 3.a. | 292 | 79257 | 2935 | 5513·0 | 18·65 | 32·14 | 2·12 | 2·11 | 3·17 | 3·19 | 4·72 | 4·75 | 2·11 | 1·76 | 1·20 |
| 4 | 270 | 132752 | 4917 | 9108·5 | 18·65 | 41·02 | 2·52 | 2·51 | 4·11 | 4·12 | 6·12 | 6·14 | 2·51 | 2·37 | 1·06 |
| 5 | 252 | 186624 | 6912 | 12731·9 | 18·66 | 48·53 | 2·91 | 2·90 | 4·91 | 4·92 | 7·32 | 7·33 | | | |
| 6 | 246 | 206666 | 7654 | 13768·6 | 18·66 | 52·07 | 2·92 | 2·91 | 5·23 | 5·24 | 7·79 | 7·81 | | | |
| 6.a. | 246 | 206666 | 7654 | 13768·6 | 18·66 | 52·07 | 3·33 | 3·32 | 5·08 | 5·09 | 7·57 | 7·58 | | | |
| 7 | 243 | 216089 | 8003 | 14404·2 | 18·66 | 53·46 | 3·82 | 3·81 | 5·26 | 5·27 | 7·84 | 7·85 | | | |
| 8 | 240 | 225566 | 8354 | 15044·0 | 18·66 | 54·84 | 3·97 | 3·96 | 5·56 | 5·57 | 8·28 | 8·30 | | | |
| 9 | 234 | 244315 | 9049 | 16353·4 | 18·66 | 57·49 | 4·02 | 4·01 | 6·42 | 6·43 | 9·57 | 9·58 | | | |
| 9.a. | 234 | 244315 | 9049 | 16353·4 | 18·66 | 57·49 | 3·39 | 3·38 | 6·63 | 6·64 | 9·88 | 9·89 | | | |
| 10 | 213·56 | 321338 | 11901 | 21893·6 | 18·66 | 64·99 | 3·38 | 3·38 | 6·61 | 6·62 | 9·85 | 9·86 | | | |
| 11 | 200 | 383326 | 14197 | 26066·3 | 18·66 | 70·86 | 3·65 | 3·64 | 7·31 | 7·31 | 10·89 | 10·89 | | | |
| 12 | 190 | 432326 | 16012 | 29365·3 | 18·66 | 75·19 | 3·84 | 3·83 | 7·82 | 7·83 | 11·65 | 11·67 | | | |
| Inclined | | 250416 | 9275 | 16642·1 | 18·66 | 63·11 | 3·36 | 3·35 | 5·24 | 5·25 | 12·19 | 12·21 | | | |

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# DAM FULL WITH BACKWATER 37' 4" MONOLITHS J & O

| Section | R.L. | Volume above Section | | Weight above Section including Gates. TONS | Xw | Yw | Stress at Corners Tons per sq. ft. | | | | Max. Inclined Stress | | Stress at up-stream Toe | Pressure of full head of water | Uplift Coeff. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. Ft. | Cu. Yds | | | | O | P | Q | R | Q | R | | | |
| 5 | 252 | 186624 | 6912 | 12879·6 | 18·66 | 48·70 | 2·90 | 2·89 | 5·01 | 5·02 | 7·46 | 7·48 | 2·89 | 2·87 | 1·01 |
| 6 | 246 | 206666 | 7654 | 13972·1 | " | 52·42 | 2·86 | 2·85 | 5·40 | 5·41 | 8·05 | 8·06 | 2·85 | 3·04 | ·94 |
| 6. a. | 246 | 206666 | 7654 | 13972·1 | " | 52·42 | 3·27 | 3·26 | 5·26 | 5·27 | 7·84 | 7·85 | 3·26 | 3·04 | 1·07 |
| 7 | 243 | 216089 | 8003 | 14534·8 | " | 54·02 | 3·67 | 3·67 | 5·47 | 5·48 | 8·15 | 8·17 | 3·67 | 3·13 | 1·17 |
| 8 | 240 | 225566 | 8354 | 15160·4 | " | 55·47 | 3·80 | 3·79 | 5·77 | 5·78 | 8·60 | 8·61 | 3·79 | 3·21 | 1·18 |
| 9 | 234 | 244315 | 9049 | 16437·7 | " | 58·08 | 3·86 | 3·85 | 6·63 | 6·64 | 9·88 | 9·89 | 3·84 | 3·38 | 1·14 |
| 9. a. | 234 | 244315 | 9049 | 16437·7 | " | 58·08 | 3·24 | 3·23 | 6·82 | 6·83 | 10·16 | 10·18 | 3·23 | 3·38 | ·96 |
| 10 | 219·56 | 321338 | 11901 | 22914·2 | " | 65·60 | 3·38 | 3·37 | 7·08 | 7·09 | 10·55 | 10·56 | 3·37 | 3·95 | ·85 |
| 11 | 200 | 383326 | 14197 | 27248·2 | " | 71·50 | 3·64 | 3·63 | 7·81 | 7·82 | 11·64 | 11·65 | 3·63 | 4·33 | ·84 |
| 12 | 190 | 432326 | 16012 | 30214·1 | " | 75·91 | 3·76 | 3·75 | 8·23 | 8·24 | 12·28 | 12·28 | 3·75 | 4·60 | ·81 |
| Inclined | | 250416 | 9275 | 17069·9 | " | 65·03 | 3·07 | 3·06 | 5·96 | 5·97 | 13·87 | 13·89 | 3·06 | 3·05 | 1·00 |

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# PROPERTIES 37'-4" CENTRAL MONOLITHS L & M

| SECTION | RL | BASE WIDTH | AREA Sq. ft. | VOLUME ABOVE SECTION Cu. ft. | VOLUME ABOVE SECTION Cu. Yds. | $X_G$ | $Y_G$ | $I_{XG}$ | $I_{YG}$ | $e_1$ | $e_2$ | $\mu_1$ | $\mu_2$ |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | 301·43 | 51·43 | 1806·2 | 62064 | 2299 | 18·67 | 26·62 | 397979 | 220818 | 8·88 | 8·28 | 6·55 | 6·55 |
| 2 | 298·67 | 53·50 | 1883·5 | 67154 | 2487 | " | 27·67 | 448042 | 229790 | 9·21 | 8·60 | 6·54 | 6·54 |
| 2a | 298·67 | 53·50 | 1668·8 | 67154 | 2487 | " | 28·89 | 425677 | 204858 | 10·36 | 8·83 | 6·58 | 6·58 |
| 3 | 292 | 58·50 | 1855·5 | 78896 | 2922 | " | 31·56 | 552933 | 226547 | 11·06 | 9·44 | 6·54 | 6·54 |
| 3a | 292 | 58·50 | 2070·2 | 78896 | 2922 | " | 30·20 | 586176 | 251479 | 10·01 | 9·38 | 6·51 | 6·51 |
| 4 | 270 | 75·00 | 2739·6 | 132391 | 4903 | " | 38·04 | 1275320 | 324472 | 12·60 | 12·24 | 6·35 | 6·35 |
| 5 | 252 | 88·50 | 3246·2 | 186263 | 6899 | " | 44·79 | 2101844 | 383041 | 14·81 | 14·46 | 6·32 | 6·32 |
| 6 | 246 | 93·00 | 3352·4 | 206305 | 7641 | " | 47·89 | 2310747 | 401813 | 15·28 | 14·39 | 6·42 | 6·42 |
| 6a | 246 | 93·00 | 3234·0 | 206305 | 7641 | " | 49·07 | 2178516 | 401182 | 15·33 | 13·73 | 6·65 | 6·65 |
| 7 | 243 | 95·25 | 3135·7 | 215728 | 7990 | " | 51·03 | 2298763 | 409975 | 16·58 | 14·37 | 7·00 | 7·00 |
| 8 | 240 | 97·50 | 3122·0 | 225205 | 8341 | " | 52·17 | 2465774 | 419200 | 17·42 | 15·14 | 7·19 | 7·19 |
| 9 | 234 | 102·00 | 3095·6 | 243954 | 9035 | " | 53·51 | 2756032 | 437676 | 18·36 | 16·64 | 7·57 | 7·57 |
| 9a | 234 | 102·00 | 3250·2 | 243954 | 9035 | " | 51·71 | 2971784 | 438496 | 18·18 | 17·68 | 7·23 | 7·23 |
| 10 | 213·56 | 117·33 | 4380·2 | 320977 | 11888 | " | 58·66 | 5024691 | 508739 | 19·56 | 19·56 | 6·22 | 6·22 |
| 11 | 200 | 127·50 | 4760·0 | 382965 | 14184 | " | 63·75 | 6448312 | 552850 | 21·25 | 21·25 | 6·22 | 6·22 |
| 12 | 190 | 135·00 | 5040·0 | 431965 | 15999 | " | 67·50 | 7654500 | 585385 | 22·50 | 22·50 | 6·22 | 6·22 |
| Inclined | | 116·59 | 3378·6 | 250055 | 9261 | " | 58·91 | 3765806 | 499514 | 19·33 | 18·92 | 7·92 | 7·92 |

# DAM EMPTY 37'-4" CENTRAL MONOLITHS L&M

| SECTION | RL | VOLUME ABOVE SECTION | | WEIGHT ABOVE SECTION Including Gates Tons | $X_G$ | $Y_G$ | STRESS AT CORNERS Tons per Sq ft | | | | MAX INCLINED STRESS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. ft. | Cu. Yds | | | | O | P | Q | R | O | P |
| 1 | 301·43 | 62064 | 2299 | 4092·0 | 18·73 | 20·96 | 3·79 | 3·84 | ·84 | ·80 | 3·80 | 3·85 |
| 2 | 298·67 | 67154 | 2487 | 4426·1 | 18·73 | 21·56 | 4·00 | 4·04 | ·81 | ·77 | 4·01 | 4·05 |
| 2a. | 298·67 | 67154 | 2487 | 4426·1 | 18·73 | 21·56 | 4·83 | 4·88 | ·80 | ·75 | 4·84 | 4·89 |
| 3 | 292 | 78896 | 2922 | 5196·7 | 18·72 | 23·16 | 5·27 | 5·31 | ·70 | ·65 | 5·28 | 5·32 |
| 3a. | 292 | 78896 | 2922 | 5196·7 | 18·72 | 23·16 | 4·38 | 4·42 | ·76 | ·72 | 4·39 | 4·43 |
| 4 | 270 | 132391 | 4903 | 8707·2 | 18·70 | 28·43 | 5·66 | 5·69 | ·77 | ·74 | 5·67 | 5·70 |
| 5 | 252 | 186263 | 6899 | 12242·6 | 18·69 | 32·97 | 6·84 | 6·87 | ·78 | ·75 | 6·86 | 6·89 |
| 6 | 246 | 206305 | 7641 | 13557·8 | 18·69 | 34·50 | 7·79 | 7·82 | ·51 | ·49 | 7·81 | 7·84 |
| 6a. | 246 | 206305 | 7641 | 13557·8 | 18·69 | 34·50 | 8·63 | 8·65 | ·22 | ·20 | 8·65 | 8·67 |
| 7 | 243 | 215728 | 7990 | 14176·2 | 18·69 | 35·35 | 9·44 | 9·47 | ·26 | ·23 | 9·46 | 9·49 |
| 8 | 240 | 225205 | 8341 | 14798·2 | 18·69 | 36·16 | 9·74 | 9·76 | ·40 | ·37 | 9·76 | 9·78 |
| 9 | 234 | 243954 | 9035 | 16070·7 | 18·68 | 37·65 | 10·13 | 10·15 | ·72 | ·70 | 10·16 | 10·18 |
| 9a. | 234 | 243954 | 9035 | 16070·7 | 18·68 | 37·65 | 8·87 | 8·89 | 1·13 | 1·11 | 8·89 | 8·91 |
| 10 | 213·56 | 320977 | 11888 | 21125·2 | 18·68 | 42·41 | 8·82 | 8·84 | ·82 | ·80 | 8·84 | 8·86 |
| 11 | 200 | 382965 | 14184 | 25193·4 | 18·68 | 46·07 | 9·69 | 9·71 | ·90 | ·88 | 9·71 | 9·73 |
| 12 | 190 | 431965 | 15999 | 28409·6 | 18·68 | 48·77 | 10·32 | 10·34 | ·95 | ·94 | 10·35 | 10·37 |
| Inclined | | 250055 | 9261 | 16429·0 | 18·68 | 41·83 | 8·96 | 8·98 | ·56 | ·54 | 9·31 | 9·33 |

# DAM FULL 37'-4" CENTRAL MONOLITH

| SECTION | RL | VOLUME ABOVE SECTION | | WEIGHT ABOVE SECTION Including Gates Tons | $X_w$ | $Y_w$ | STRESS AT CORNERS Tons. per Sq. ft. | | | | MAX. INCLINED STRESSES | | STRESS AT UPSTREAM TOE | PRESSURE OF FULL HEAD OF WATER | UPLIFT COEFF. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. ft. | Cu. Yds. | | | | O | P | Q | R | Q | R | | | |
| 1 | 301·43 | 62064 | 2299 | 4384·7 | 18·60 | 27·13 | 2·30 | 2·26 | 2·55 | 2·59 | 3·80 | 3·86 | 2·26 | 1·50 | 1·51 |
| 2 | 298·67 | 67154 | 2487 | 4726·6 | 18·61 | 28·22 | 2·37 | 2·33 | 2·64 | 2·68 | 3·93 | 4·00 | 2·33 | 1·57 | 1·48 |
| 2a | 298·67 | 67154 | 2487 | 4726·6 | 18·61 | 28·22 | 3·07 | 3·02 | 2·63 | 2·68 | 3·91 | 3·99 | 3·02 | 1·57 | 1·92 |
| 3 | 292 | 78896 | 2922 | 5517·9 | 18·62 | 31·12 | 3·14 | 3·09 | 2·83 | 2·88 | 4·22 | 4·29 | 3·09 | 1·76 | 1·96 |
| 3a | 292 | 78896 | 2922 | 5517·9 | 18·62 | 31·12 | 2·43 | 2·39 | 2·89 | 2·93 | 4·30 | 4·36 | 2·39 | 1·76 | 1·36 |
| 4 | 270 | 132391 | 4903 | 9113·3 | 18·64 | 40·10 | 2·78 | 2·75 | 3·86 | 3·89 | 5·75 | 5·79 | 2·75 | 2·37 | 1·16 |
| 5 | 252 | 186263 | 6899 | 12736·8 | 18·65 | 47·70 | 3·15 | 3·12 | 4·68 | 4·71 | 6·98 | 7·01 | | | |
| 6 | 246 | 206305 | 7641 | 13773·5 | 18·65 | 51·25 | 3·16 | 3·14 | 5·00 | 5·03 | 7·45 | 7·49 | | | |
| 6a | 246 | 206305 | 7641 | 13773·5 | 18·65 | 51·25 | 3·60 | 3·57 | 4·85 | 4·88 | 7·23 | 7·27 | | | |
| 7 | 243 | 215728 | 7990 | 14409·1 | 18·65 | 52·65 | 4·09 | 4·06 | 5·03 | 5·06 | 7·50 | 7·53 | | | |
| 8 | 240 | 225205 | 8341 | 15048·8 | 18·65 | 54·04 | 4·24 | 4·21 | 5·33 | 5·35 | 7·94 | 7·97 | | | |
| 9 | 234 | 243954 | 9035 | 16358·3 | 18·65 | 56·71 | 4·28 | 4·26 | 6·19 | 6·22 | 9·23 | 9·26 | | | |
| 9a | 234 | 243954 | 9035 | 16358·3 | 18·65 | 56·71 | 3·62 | 3·60 | 6·41 | 6·43 | 9·55 | 9·58 | | | |
| 10 | 213·56 | 320977 | 11888 | 21898·4 | 18·65 | 64·30 | 3·57 | 3·55 | 6·43 | 6·45 | 9·58 | 9·61 | | | |
| 11 | 200 | 382965 | 14184 | 26071·3 | 18·66 | 70·21 | 3·82 | 3·80 | 7·13 | 7·15 | 10·63 | 10·65 | | | |
| 12 | 190 | 431965 | 15999 | 29370·9 | 18·66 | 74·57 | 4·00 | 3·99 | 7·65 | 7·67 | 11·40 | 11·42 | | | |
| Inclined | | 250055 | 9261 | 16647·0 | 18·65 | 62·20 | 3·57 | 3·56 | 5·04 | 5·06 | 11·73 | 11·77 | | | |

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# DAM FULL WITH BACKWATER 37'-4" CENTRAL MONOLITH

| SECTION | RL | VOLUME ABOVE SECTION | | WEIGHT ABOVE SECTION Including Gates Tons | $X_w$ | $Y_w$ | STRESS AT CORNERS Tons per Sq. ft. | | | | MAX. INCLINED STRESSES | | STRESS AT UPSTREAM TOE | PRESSURE OF FULL HEAD OF WATER | UPLIFT COEFF. |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Cu. ft. | Cu. Yds. | | | | O | P | Q | R | Q | R | | | |
| 5 | 252 | 186263 | 6899 | 12884·5 | 18·65 | 47·88 | 3·14 | 3·11 | 4·78 | 4·81 | 7·13 | 7·17 | 3·11 | 2·87 | 1·08 |
| 6 | 246 | 206305 | 7641 | 13977·0 | 18·65 | 51·62 | 3·10 | 3·08 | 5·17 | 5·20 | 7·71 | 7·75 | 3·08 | 3·04 | 1·01 |
| 6a | 246 | 206305 | 7641 | 13977·0 | 18·65 | 51·62 | 3·53 | 3·51 | 5·03 | 5·05 | 7·49 | 7·53 | 3·51 | 3·04 | 1·15 |
| 7 | 243 | 215728 | 7990 | 14539·7 | 18·65 | 53·22 | 3·94 | 3·92 | 5·24 | 5·26 | 7·80 | 7·84 | 3·92 | 3·13 | 1·25 |
| 8 | 240 | 225205 | 8341 | 15165·3 | 18·65 | 54·68 | 4·07 | 4·04 | 5·55 | 5·57 | 8·26 | 8·30 | 4·04 | 3·21 | 1·26 |
| 9 | 234 | 243954 | 9035 | 16442·6 | 18·65 | 57·31 | 4·11 | 4·09 | 6·40 | 6·42 | 9·53 | 9·57 | 4·09 | 3·38 | 1·21 |
| 9a | 234 | 243954 | 9035 | 16442·6 | 18·65 | 57·31 | 3·47 | 3·45 | 6·61 | 6·63 | 9·84 | 9·87 | 3·45 | 3·38 | 1·02 |
| 10 | 213·56 | 320977 | 11888 | 22919·0 | 18·66 | 64·94 | 3·56 | 3·54 | 6·90 | 6·92 | 10·28 | 10·31 | 3·54 | 3·95 | ·90 |
| 11 | 200 | 382965 | 14184 | 27253·1 | 18·66 | 70·88 | 3·81 | 3·80 | 7·64 | 7·65 | 11·38 | 11·40 | 3·80 | 4·33 | ·88 |
| 12 | 190 | 431965 | 15999 | 30361·2 | 18·66 | 75·31 | 3·94 | 3·93 | 8·11 | 8·12 | 12·08 | 12·10 | 3·93 | 4·60 | ·85 |
| Inclined | · | 250055 | 9261 | 17082·6 | 18·65 | 64·18 | 3·29 | 3·27 | 5·77 | 5·78 | 13·42 | 13·45 | 3·27 | 3·05 | 1·07 |

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# SHEAR STRESSES

| Section | R.L | 26' Monolith | 37'4" Monolith | Central Monolith |
|---|---|---|---|---|
| | | Shear | Shear | Shear |
| 1 | 301·43 | 10·81 lb/□" | 15·09 lb/□" | 14·05 lb/□" |
| 2 | 298·67 | 11·75 " | 15·78 " | 14·78 " |
| 2.a. | 298·67 | 13·31 " | 17·81 " | 16·68 " |
| 3 | 292 | 15·66 " | 19·49 " | 18·47 " |
| 3.a. | 292 | 13·99 " | 17·47 " | 16·56 " |
| 4 | 270 | 20·52 " | 22·83 " | 22·14 " |
| 5 | 252 | 26·14 " | 27·71 " | 27·13 " |
| 6 | 246 | 28·80 " | 29·91 " | 29·35 " |
| 6.a. | 246 | 30·43 " | 31·00 " | 30·42 " |
| 7 | 243 | 34·30 " | 33·69 " | 33·09 " |
| 8 | 240 | 37·04 " | 35·60 " | 35·00 " |
| 9 | 234 | 43·04 " | 39·61 " | 39·01 " |
| 9.a. | 234 | 39·69 " | 37·73 " | 37·15 " |
| 10 | 213·56 | 37·28 " | 37·91 " | 37·49 " |
| 11 | 200 | 41·42 " | 41·73 " | 41·34 " |
| 12 | 190 | 44·47 " | 44·56 " | 44·19 " |
| Inclined | | 51·65 " | 47·93 " | 47·44 " |

# SLIDING FACTORS

| Section | R.L | 26' Monolith | | | | 37'4" Monolith | | | | Central Monolith | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | Uplift Coeff. | Sliding Factor | Uplift Coeff | Sliding Factor | Uplift Coeff | Sliding Factor | Uplift Coeff | Sliding Factor | Uplift Coeff | Sliding Factor | Uplift Coeff | Sliding Factor |
| 10 | 213·56 | ·75 | ·83 | ·46 | ·67 | ·85 | ·77 | ·65 | ·67 | ·90 | ·79 | ·67 | ·67 |
| 11 | 200 | ·73 | ·81 | ·46 | ·67 | ·84 | ·77 | ·65 | ·67 | ·88 | ·78 | ·67 | ·67 |
| 12 | 190 | ·71 | ·79 | ·48 | ·67 | ·81 | ·75 | ·64 | ·67 | ·85 | ·77 | ·66 | ·67 |

# INCLINED SECTIONS

## DAM FULL WITH BACKWATER-SLUICE OPEN

| MONOLITH | SECTION | R.L. | VOLUME ABOVE SECTION | | WEIGHT ABOVE SECTION Including Gates Tons. | $X_w$ | $Y_w$ | STRESS AT CORNERS Tons per Sq. ft. | | | | MAX. INCLINED STRESS | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | Cu. ft. | Cu. Yds. | | | | O | P | Q | R | Q | R |
| 26' | INCLINED | | 145647 | 5388 | 9766·6 | 12·49 | 66·72 | 2·70 | 2·68 | 6·16 | 6·18 | 14·33 | 14·38 |
| 37'-4" | " | | 250416 | 9275 | 16921·5 | 18·66 | 64·95 | 3·05 | 3·04 | 5·88 | 5·89 | 13·68 | 13·70 |
| CENTRAL | " | | 250055 | 9261 | 16926·4 | 18·65 | 64·08 | 3·27 | 3·25 | 5·67 | 5·70 | 13·20 | 13·26 |

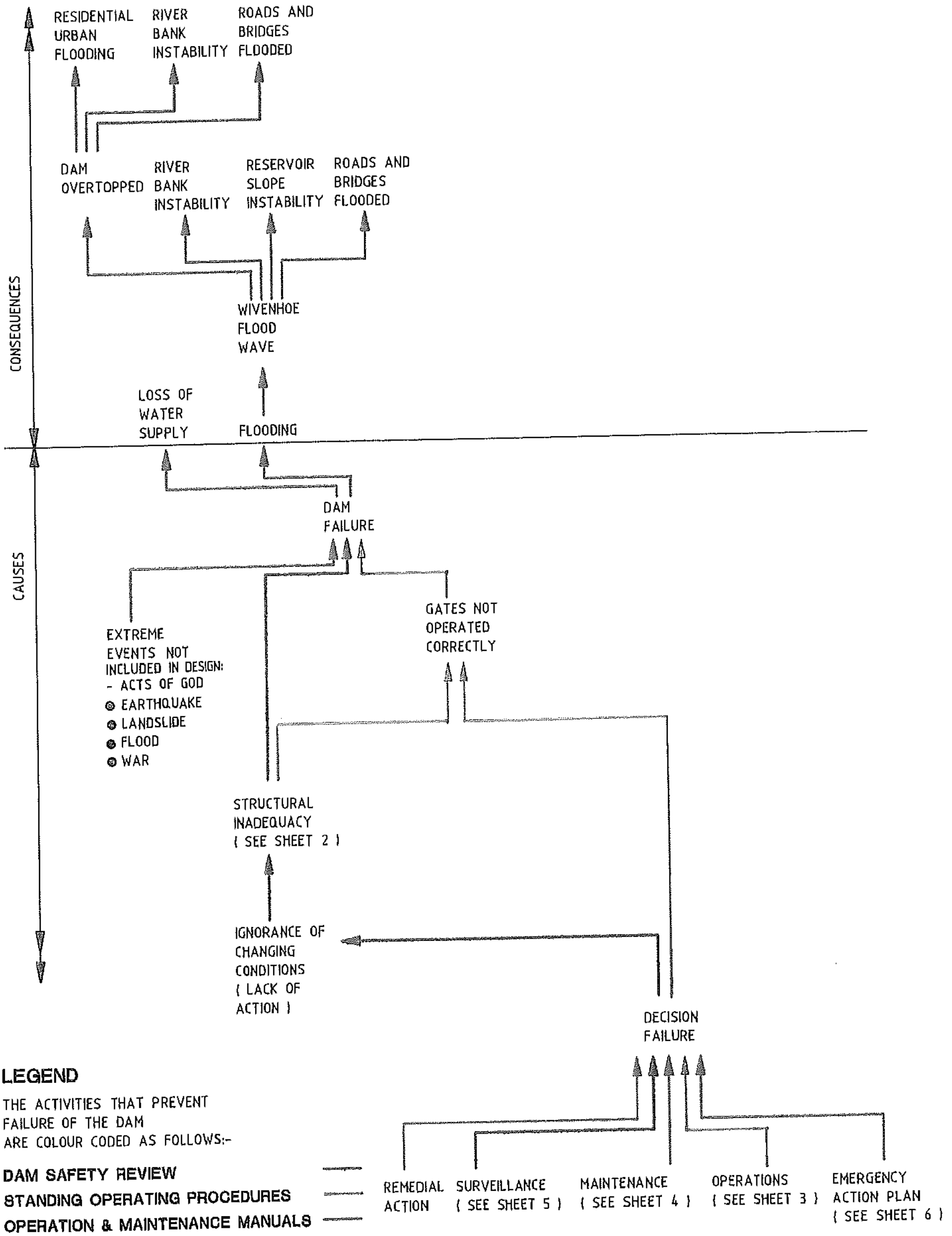
# SLIDING FACTORS

| SECTION | R.L. | SLIDING FACTORS | | |
|---|---|---|---|---|
| | | 26' MONO. | 37'-4" MONO | CENT. MON. |
| 1 | 301·43 | ·43 | ·40 | ·37 |
| 2 | 298·67 | ·43 | ·41 | ·38 |
| 3 | 292 | ·46 | ·42 | ·40 |
| 4 | 270 | ·48 | ·44 | ·43 |
| 5 | 252 | ·49 | ·45 | ·44 |
| 6 | 246 | ·51 | ·47 | ·46 |
| 7 | 243 | ·51 | ·47 | ·46 |
| 8 | 240 | ·52 | ·48 | ·47 |
| 9 | 234 | ·53 | ·48 | ·47 |
| 10 | 213·56 | ·53 | ·49 | ·48 |
| 11 | 200 | ·53 | ·49 | ·49 |
| 12. | 190 | ·53 | ·49 | ·49 |

NOTE :— In the above calculations backwater and uplift were neglected.

# APPENDIX D

# Potential Paths to Failure




## LEGEND

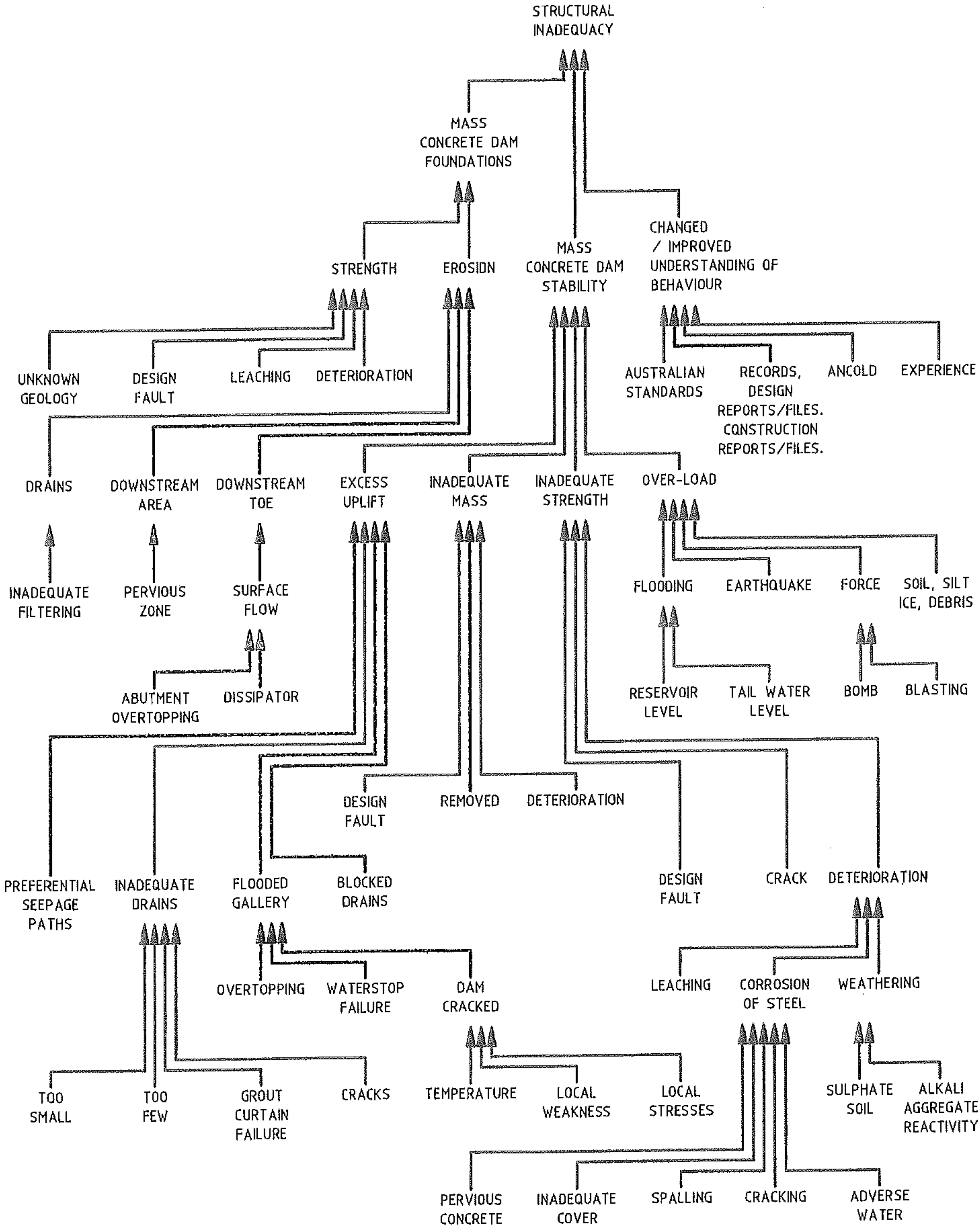
THE ACTIVITIES THAT PREVENT FAILURE OF THE DAM ARE COLOUR CODED AS FOLLOWS:-

**DAM SAFETY REVIEW**
**STANDING OPERATING PROCEDURES**
**OPERATION & MAINTENANCE MANUALS**
**DAM INSPECTIONS BY DAM ENGINEER**
**MANAGEMENT DECISIONS**
EMERGENCY ACTION PLAN

# SEQWB
# SOMERSET DAM - PATHS TO FAILURE
# GENERAL OVERVIEW

**SHEET 1**

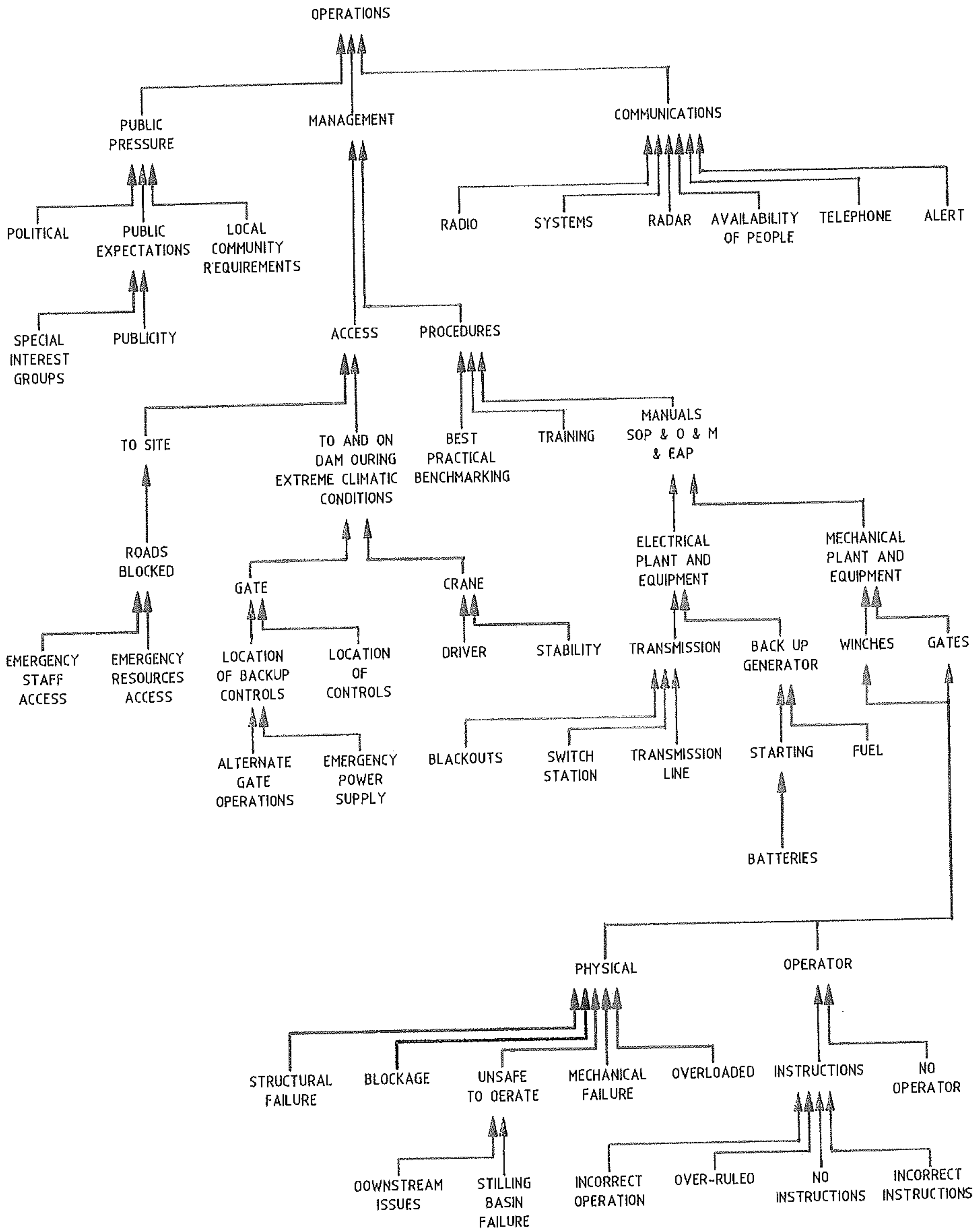
F:\HOME\4II\14019\SOMERSET\SHEETS\14019SH1.dwg




**SEQWB**
**SOMERSET DAM - PATHS TO FAILURE**
**STRUCTURAL INADEQUACY**

**SHEET 2**

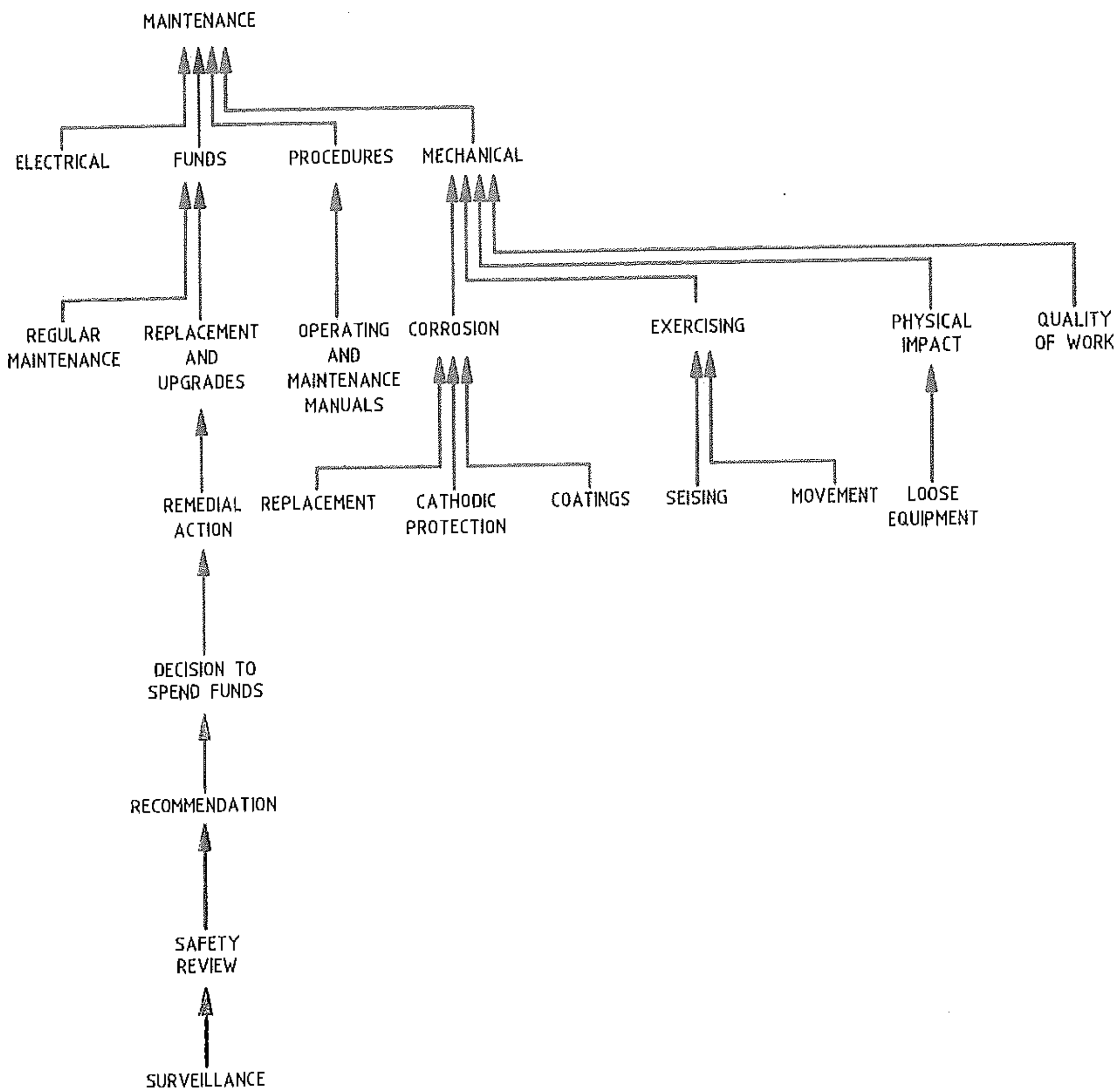
F:\HOME\411\14019\SOMERSET\SHEETS\14019SH2.dwg




**SEQWB**
**SOMERSET DAM - PATHS TO FAILURE**
**OPERATIONS FAILURE**

**SHEET 3**

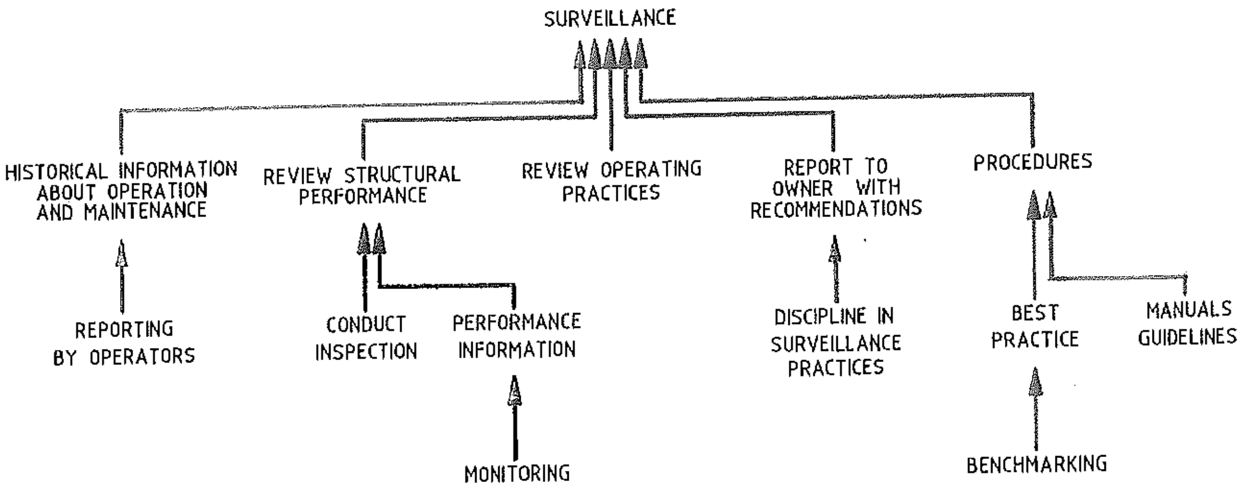
F:\HOME\411\14019\SOMERSET\SHEETS\14019SH3.dwg




**SEQWB**
**SOMERSET DAM - PATHS TO FAILURE**
**MAINTENANCE FAILURE**

**SHEET 4**

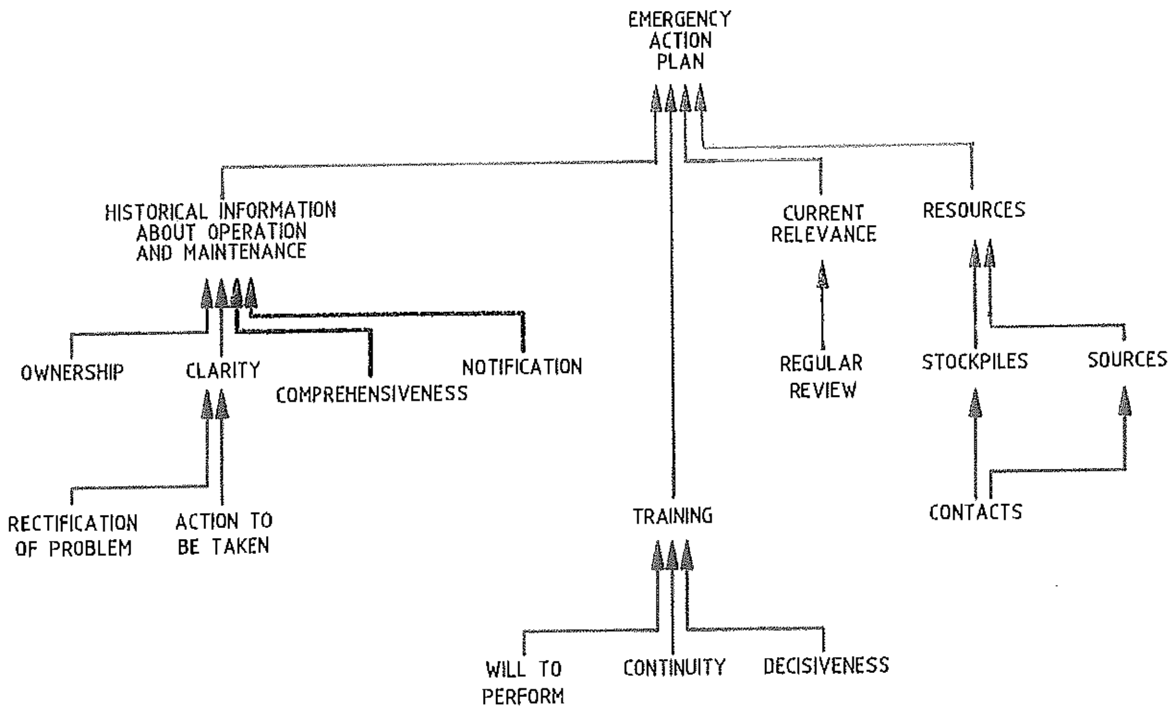
F:\HOME\411\14019\SOMERSET\SHEETS\14019SH4.dwg




**SEQWB**
**SOMERSET DAM - PATHS TO FAILURE**
**SURVEILLANCE FAILURE**

**SHEET 5**

F:\HOME\411\14019\SOMERSET\SHEETS\14019SH5.dwg




**SEQWB**
**SOMERSET DAM - PATHS TO FAILURE**
**EMERGENCY ACTION PLAN**

**SHEET 6**

F:\HOME\411\14019\SOMERSET\SHEETS\14019SH6.dwg

# APPENDIX E

# Summary of Stability Analyses

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:38
The input file used was g:\somerset\so_qrfsl.dat
and this output file is g:\somerset\so_qrfsl.out

Somerset Dam - Non overtoppable block at full supply level

Input Data

| | | |
|---|---|---|
| Water Unit Weight | (kN/m3) : | 9.81 |
| Base Friction Strength (Phi) | (ø) : | 45 |
| Base Cohesion | (kPa) : | 1438 |
| Concrete Strength (exterior) | (kPa) : | 20000 |
| Concrete Strength (interior) | (kPa) : | 10000 |
| Foundation Bearing Strength | (kPa) : | 10000 |
| Concrete Unit Weight | (kN/m3) : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 107.5 | 4.267 | .0001 | .0001 |
| 101.8 | 4.26814 | .0001 | .75 |
| 100.5 | 5.243273 | .05 | .75 |
| 54.7 | 41.88327 | .05 | .75 |

External Dam Loads

| | | |
|---|---|---|
| Spilway gates open | = | n |
| Reservoir Level | (m) : | 98.93 |
| Spillway crest level | (m) : | 107.5 |
| Tailwater Level | (m) : | 64 |
| Silt Level | (m) : | 70 |
| Silt Unit Weight | (kN/m3) : | 15.9903 |
| Silt earth pressure coefficient | : | 1 |
| Distance to Drains from heel | (m) : | 5.33 |
| ratio of HWl at Drains | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1.05 |
| Concrete and Tailwater | : | .95 |
| Silt | : | 1.5 |
| cohesion and friction | : | .3 |
| concrete strength | : | .4 |
| foundation bearing | : | .3 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN); E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 26,176 | 27.195 | 9,617 | 14.760 | 481 | 41.185 | 728 | 5.117 | 36 | 41.667 |
| 59.94 | 21,223 | 24.474 | 7,459 | 12.998 | 373 | 37.045 | 313 | 3.355 | 16 | 37.528 |
| 65.22 | 16,795 | 21.754 | 5,574 | 11.237 | 279 | 32.905 | 71 | 1.593 | 4 | 33.388 |
| 70.51 | 12,894 | 19.036 | 3,963 | 9.475 | 198 | 28.766 | 0 | 0.000 | 0 | 0.000 |
| 75.79 | 9,519 | 16.318 | 2,626 | 7.713 | 131 | 24.626 | 0 | 0.000 | 0 | 0.000 |
| 81.08 | 6,670 | 13.600 | 1,564 | 5.952 | 78 | 20.486 | 0 | 0.000 | 0 | 0.000 |
| 86.36 | 4,347 | 10.876 | 775 | 4.190 | 39 | 16.346 | 0 | 0.000 | 0 | 0.000 |
| 91.65 | 2,550 | 8.123 | 260 | 2.428 | 13 | 12.206 | 0 | 0.000 | 0 | 0.000 |
| 96.93 | 1,279 | 5.251 | 20 | 0.667 | 1 | 8.066 | 0 | 0.000 | 0 | 0.000 |
| 102.22 | 531 | 2.134 | 0 | 0.000 | 0 | 0.000 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN); C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN); E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm);K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 7,153 | 25.155 | 429 | 3.117 | 322 | 2.337 | 10,783 | 18,154 | 354,943 | 1.410 |
| 59.94 | 4,569 | 24.382 | 81 | 1.355 | 61 | 1.016 | 8,224 | 15,791 | 286,929 | 0.677 |
| 65.22 | 2,726 | 24.142 | 0 | 0.000 | 0 | 0.000 | 5,958 | 13,362 | 220,897 | 0.202 |
| 70.51 | 2,102 | 21.162 | 0 | 0.000 | 0 | 0.000 | 4,161 | 10,231 | 152,458 | -0.282 |
| 75.79 | 1,551 | 18.149 | 0 | 0.000 | 0 | 0.000 | 2,758 | 7,539 | 99,807 | -0.733 |
| 81.08 | 1,073 | 15.091 | 0 | 0.000 | 0 | 0.000 | 1,642 | 5,283 | 60,916 | -1.138 |
| 86.36 | 669 | 11.972 | 0 | 0.000 | 0 | 0.000 | 814 | 3,464 | 33,696 | -1.450 |
| 91.65 | 337 | 8.762 | 0 | 0.000 | 0 | 0.000 | 273 | 2,081 | 16,061 | -1.556 |
| 96.93 | 79 | 5.415 | 0 | 0.000 | 0 | 0.000 | 21 | 1,133 | 5,926 | -1.179 |
| 102.22 | 0 | 0.000 | 0 | 0.000 | 0 | 0.000 | 0 | 504 | 1,077 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa); C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa) E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | 346 | 520 | 0 | 813 | 23,532 | 2.18 |
| 59.935 | 374 | 464 | 0 | 725 | 20,999 | 2.55 |
| 65.220 | 385 | 414 | 0 | 646 | 18,446 | 3.10 |
| 70.505 | 370 | 330 | 0 | 515 | 15,683 | 3.77 |
| 75.790 | 354 | 248 | 0 | 388 | 13,052 | 4.73 |
| 81.075 | 338 | 171 | 0 | 267 | 10,551 | 6.43 |
| 86.360 | 319 | 99 | 0 | 155 | 8,181 | 10.05 |
| 91.645 | 297 | 41 | 0 | 64 | 5,942 | 21.74 |
| 96.930 | 262 | 18 | 0 | 28 | 3,834 | %186.11 |
| 102.215 | 118 | 118 | 0 | 118 | 1,993 | |

Levy and Hoffman Criteria
Concrete Gravity Dam Crack Stability Assessment

Levy and Hoffman PWP distribution
full uplift over crack and no PWP thereafter

| | | | |
|---|---|---|---|
| Dam - | Example Dam | Load Case: | Full Supply Level |
| Client - | SECWB | Comments: | Non overflow block OPR |
| Date - | 23-Aug-95 | | at block base |
| By - | J Williams | | |
| Checked by - | | | |

Crack stability at a Dam Section

[illegible]

SUMMARY

| Length | Net Upstream Stress | Levy | Hoffman |
|---|---|---|---|
| 0 | -221.57608 | OK | |
| 4.8 | -308.43627 | OK | OK |

Tens +ve

| | |
|---|---|
| If Levy is not met the crack length increases. | |
| If Levy is met the crack length is stable | X |
| If Hoffman is met the crack increases to a stable point | X |
| If Hoffman is not met the crack propagates until failure | |

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:50
The input file used was g:\somerset\so_qr_74.dat
and this output file is g:\somerset\so_qr_74.out

Somerset Dam - Non overtoppable block passing max past flood in 1974

Input Data

| | | |
|---|---|---|
| Water Unit Weight | (kN/m3) : | 9.81 |
| Base Friction Strength (Phi) | (ø) : | 45 |
| Base Cohesion | (kPa) : | 1438 |
| Concrete Strength (exterior)(kPa) | : | 20000 |
| Concrete Strength (interior)(kPa) | : | 10000 |
| Foundation Bearing Strength (kPa) | : | 10000 |
| Concrete Unit Weight | (kN/m3) : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 107.5 | 4.267 | .0001 | .0001 |
| 101.8 | 4.26814 | .0001 | .75 |
| 100.5 | 5.243273 | .05 | .75 |
| 54.7 | 41.88327 | .05 | .75 |

External Dam Loads

| | | |
|---|---|---|
| Spilway gates open | = | n |
| Reservoir Level | (m) : | 106.55 |
| Spillway crest level | (m) : | 107.5 |
| Tailwater Level | (m) : | 72.1 |
| Silt Level | (m) : | 70 |
| Silt Unit Weight | (kN/m3) : | 15.9903 |
| Silt earth pressure coefficient | : | 1 |
| Distance to Drains from heel | (m) : | 5.33 |
| ratio of HWL at Drains | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1.05 |
| Concrete and Tailwater | : | .95 |
| Silt | : | 1.5 |
| cohesion and friction | : | .3 |
| concrete strength | : | .4 |
| foundation bearing | : | .3 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN);E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 26,176 | 27.195 | 13,212 | 17.300 | 652 | 41.079 | 728 | 5.117 | 36 | 41.667 |
| 59.94 | 21,223 | 24.474 | 10,658 | 15.538 | 650 | 36.852 | 313 | 3.355 | 16 | 37.528 |
| 65.22 | 16,795 | 21.754 | 8,379 | 13.777 | 650 | 32.624 | 71 | 1.593 | 4 | 33.388 |
| 70.51 | 12,894 | 19.036 | 6,373 | 12.015 | 650 | 28.396 | 0 | 0.000 | 0 | 0.000 |
| 75.79 | 9,519 | 16.318 | 4,641 | 10.253 | 650 | 24.168 | 0 | 0.000 | 0 | 0.000 |
| 81.08 | 6,670 | 13.600 | 3,183 | 8.492 | 650 | 19.940 | 0 | 0.000 | 0 | 0.000 |
| 86.36 | 4,347 | 10.876 | 1,999 | 6.730 | 650 | 15.712 | 0 | 0.000 | 0 | 0.000 |
| 91.65 | 2,550 | 8.123 | 1,090 | 4.968 | 650 | 11.484 | 0 | 0.000 | 0 | 0.000 |
| 96.93 | 1,279 | 5.251 | 454 | 3.207 | 650 | 7.256 | 0 | 0.000 | 0 | 0.000 |
| 102.22 | 531 | 2.134 | 92 | 1.445 | 0 | 4.268 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN); C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN); E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm); K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 10,438 | 23.796 | 1,494 | 5.817 | 1,120 | 4.362 | 13,546 | 15,625 | 209,635 | 7.545 |
| 59.94 | 7,522 | 22.163 | 726 | 4.055 | 544 | 3.041 | 10,971 | 13,413 | 170,605 | 6.129 |
| 65.22 | 5,044 | 20.825 | 232 | 2.293 | 174 | 1.720 | 8,683 | 11,446 | 136,491 | 4.809 |
| 70.51 | 3,005 | 20.166 | 12 | 0.532 | 9 | 0.399 | 6,680 | 9,721 | 106,706 | 3.643 |
| 75.79 | 2,062 | 18.149 | 0 | 0.000 | 0 | 0.000 | 4,873 | 7,496 | 73,240 | 2.735 |
| 81.08 | 1,532 | 15.091 | 0 | 0.000 | 0 | 0.000 | 3,342 | 5,346 | 45,844 | 1.816 |
| 86.36 | 1,074 | 11.972 | 0 | 0.000 | 0 | 0.000 | 2,099 | 3,619 | 26,986 | 0.822 |
| 91.65 | 690 | 8.762 | 0 | 0.000 | 0 | 0.000 | 1,144 | 2,316 | 14,739 | -0.202 |
| 96.93 | 379 | 5.415 | 0 | 0.000 | 0 | 0.000 | 477 | 1,435 | 7,181 | -0.954 |
| 102.22 | 144 | 2.192 | 0 | 0.000 | 0 | 0.000 | 97 | 354 | 606 | 0.420 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa); C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa) E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | -30 | 775 | 0 | 1,213 | 22,051 | 1.63 |
| 59.935 | 9 | 703 | 0 | 1,098 | 20,286 | 1.85 |
| 65.220 | 47 | 637 | 0 | 995 | 17,871 | 2.06 |
| 70.505 | 84 | 581 | 0 | 908 | 15,530 | 2.33 |
| 75.790 | 103 | 496 | 0 | 775 | 13,039 | 2.68 |
| 81.075 | 122 | 392 | 0 | 613 | 10,570 | 3.16 |
| 86.360 | 154 | 284 | 0 | 443 | 8,228 | 3.92 |
| 91.645 | 206 | 169 | 0 | 265 | 6,013 | 5.26 |
| 96.930 | 302 | 52 | 0 | 81 | 3,925 | 8.23 |
| 102.215 | 34 | 132 | 0 | 132 | 1,947 | 20.12 |

Levy and Hoffman Criteria
Concrete Gravity Dam Crack Stability Assessment

Levy and Hoffman PWP distribution
Full uplift over crack and no PWP elsewhere

| | | | |
|---|---|---|---|
| Dam - | Somerset Dam | Load Case | 1974 flood |
| Client - | SEQWB | Comments | Non overflow block Q/R at block base |
| Date - | 23-Aug-95 | | |
| By - | J Williams | | |
| Checked by - | | | |

Crack Stability at a Dam Section

Allowable tens. (kPa) 0

Section Width 41.9 Cracked Width 40.34 Head at section (m) 51.85 Tailwater (m) 17.4

| Elevation | Conc Wt | W cog | Res H | H cog | Res V | V cog | Silt Hs | Hs cog | Tail Ht | Ht cog | Others V | Others cog |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 54.7 | 26176 | 27.195 | 13212 | 17.3 | 652 | 41.079 | 728 | 5.117 | 1494 | 5.817 | 1156 | 5.5237474 |

| | | | | | | |
|---|---|---|---|---|---|---|
| Uncracked | Sum Horiz | 12446 | Sum Vert | 17546 | Sum Mom | 273040.45 |
| Cracked | Sum Horiz | 12446 | Sum Vert | 27190.508 | Sum Mom | 488794.72 |
| Cracked no U | Sum Horiz | 12446 | Sum Vert | 27984 | Sum Mom | 491040.16 |

z and p not used
z = 0
p = 1

Normal Uplift / Cracked Uplift

| Uplift U | U cog | U1 | U1 cog | U2 | U2 cog | U3 | U3 cog | U4 | U4 cog |
|---|---|---|---|---|---|---|---|---|---|
| 10438 | 23.796 | 793.49166 | 41.12 | 0 | 0 | 0 | 0 | 0 | 0 |

Uplift according to Levy and Hoffman
uplift over crack only, pore pressure over crack

Stress at Uncracked Upstream Face (for checking purposes)
Stress u = Sum Vert/width*(1-6*(0.5 width - Sum M/Sum Vert)/width)
Stress u = 95.6293961 Stress d = 741.8885

Stress at Cracked Section including Uplift
Stress u = 454.145549 Stress d = 893.9213
less uplift 508.6485
net stress 54.5029509 less than? 0 allowable tension NB Tension +ve
Levy = Failed
Hoffman rate of change of Levy with increasing crack must be less than zero (increasing stability)
Uncracked portion 40.24 ie more compressive stress with increasing crack

Cracked with U Sum Horiz 12484 Sum Vert 27477.643 Sum Mom 487810.28
z = 0
p = 1

Normal Uplift / Cracked Uplift

| Uplift U | U cog | U1 | U1 cog | U2 | U2 cog | U3 | U3 cog | U4 | U4 cog |
|---|---|---|---|---|---|---|---|---|---|
| 10438 | 23.796 | 844.35651 | 41.07 | 0 | 0 | 0 | 0 | 0 | 0 |

Uplift according to Levy and Hoffman
uplift over crack only, pore pressure over crack

Stress at Cracked Section including Uplift
Stress u = 441.845036 Stress d = 923.84301
less uplift 508.6485
net stress 66.8034639 less than? 0 allowable tension NB Tension +ve
Levy = Failed
Hoffman = Unstable because crack length increases to failure

SUMMARY

| Crack Length | Net Upstream Stress | Levy | Hoffman |
|---|---|---|---|
| 1.56 | 54.5029509 | Failed | |
| 1.66 | 66.8034639 | Failed | Unstable |
| | Tens +ve | | |

| | |
|---|---|
| If Levy is not met the crack length increases. | X |
| If Levy is met the crack length is stable | |
| If Hoffman is met the crack increases to a stable point | |
| If Hoffman is not met the crack propogates until failure | X |

| | A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|---|
| 1 | EERC Simplified Seismic Analaysis for Concrete Gravity Dams | | | | | | |
| 2 | for Somerset Dam - Non Overflow Block - OBE | | | | | | |
| 3 | Note for spreadsheet users | Pink indicates data from model | | Red is data from EERC figures and tables | | | |
| 4 | geometry | | | SI | million psi | Ratios | |
| 5 | H = Height water | | | 44.23 | | | |
| 6 | Hs = Ht structure | | | 52.80 | | H/Hs | 0.84 |
| 7 | Es = E Strcuture MPa | | | 22600.00 | 3280.12 | Es/Ef | 0.75 |
| 8 | Ef = E fdn MPa | | | 30000.00 | 4354.14 | | |
| 9 | Concrete Density (kN/m3) | | | 23.50 | | | |
| 10 | | | | | | | |
| 11 | Refer EERC Calculation Steps | | | | | | |
| 12 | T1 = Fundamental Period (s) | | | 0.13 | =0.36*Hs/ sqrt(Es) | | |
| 13 | Rr = Reservoir Interaction factor | | | 1.13 | from Fig 4 | H/Hs | 0.84 |
| 14 | Tr = Period Modified for Reservoir | | | 0.15 | Rr * T1 | | |
| 15 | Rw = Period Ratio | | | 0.81 | =(4H / 1440) / Tr | | |
| 16 | Rf = Foundation Interaction factor | | | 1.14 | from Fig 6 | Es/Ef | 0.75 |
| 17 | T'1 = Modified Fundamental Period | | | 0.17 | = Rr * Rf * T1 | | |
| 18 | e'1 = Damping Ratio | | | 0.12 | = 1/Rr 1/Rf^3 e1 + er + ef | | |
| 19 | where e1 is rock damping | | | 0.10 | | | |
| 20 | and er = reservoir damping factor | | | 0.01 | from Fig 5 | H/Hs | 0.84 |
| 21 | and ef = foundation damping factor | | | 0.06 | from Fig 7 | | |
| 22 | IF e'1 < e1 use e1 | | | 0.12 | | | |
| 23 | GO TO TABLE | | | | | | |

Taken from
"Simplified Analysis for Earthquake Resistant Design
of Concrete Gravity Dams"
by G Fenves and A Chopra
Earthquake Engineering Research Center
UC - Berkeley, June 1986

| Modal Participation Mass | | | | | |
|---|---|---|---|---|---|
| M1 = | 1323.60 | L1 = | 3807.16 | | |
| Period (s) = | 0.17 | Freq (Hz) = | 5.82 | | |
| Sa @ Freq | 0.16 | | | | |
| M'1 = Rr^2 * M1 = | | 1690.10 | | | |
| L'1 = L1 + 1/g Fst (H/Hs)^2 Ap | | | | | |
| Where Fst = 0.5wH^2 = | | 9595.62 | | | |
| Ap f(Rw, a) from Table 5 | | 0.322 | Rw = | | 0.81 |
| L'1 = | | 4026.18 | a = | | 0.90 |
| L'1/M'1 * Sa = | | 0.38 | L'1/M'1 = | | 2.38 |

| | A | B | C | D | E | F | G | H | I | J | K | L | M | N | O | P | Q | R | S | T | U |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 23 | | | | | | | | | | Rw = | | | | | | | | | | | |
| 24 | | | | | | | | | | 0.81 | | | Westergaard | | | | | | | | |
| 25 | Crest RL then RL | | | | | | | | | hydrodynamic masses | | | hydrody | | | | | | F1(y) = | | using /ht |
| 26 | at centroid | Ht (m) | blk ht | av width | mass (t) | wt | Wt / Ht | y/H | y/Hs | gp(y)/wH | gp(y) | gp | mass | mode 1 phi | w phi | w phi^2 | w/ht *phi | L/M Sa | l/m Sa * (w*phi+gp) | | at joint |
| 27 | 107.50 | at centroid | | (at centroid) | | (kN/m3)/m | kN/m3/m2 | | | | N/m/m | N/m | * blk ht | | | | | for Rr | /ht | /blk | * ht |
| 28 | 104.85 | 50.15 | 3.98 | 4.27 | 40.63 | 398.59 | 100.27 | 1.13 | 0.95 | 0.000 | 0.00 | 0.00 | 0.00 | 0.866 | 345.16 | 298.93 | 86.84 | 0.38 | 33.11 | 131.63 | 131.63 |
| 29 | 99.55 | 44.85 | 5.28 | 6.20 | 78.35 | 768.57 | 145.70 | 1.01 | 0.85 | 0.000 | 0.00 | 0.00 | 0.00 | 0.619 | 475.74 | 294.49 | 90.19 | 0.38 | 34.39 | 181.42 | 181.42 |
| 30 | 94.30 | 39.60 | 5.25 | 10.20 | 128.32 | 1258.80 | 239.77 | 0.90 | 0.75 | 0.124 | 37.76 | 198.23 | 644.89 | 0.455 | 572.75 | 260.60 | 109.10 | 0.38 | 56.00 | 294.01 | 294.01 |
| 31 | 89.05 | 34.35 | 5.28 | 14.40 | 182.00 | 1785.43 | 338.47 | 0.78 | 0.65 | 0.160 | 48.72 | 257.00 | 946.53 | 0.334 | 596.33 | 199.18 | 113.05 | 0.38 | 61.69 | 325.41 | 325.41 |
| 32 | 83.75 | 29.05 | 5.30 | 18.51 | 234.96 | 2305.17 | 434.94 | 0.66 | 0.55 | 0.179 | 54.51 | 288.89 | 1178.82 | 0.240 | 553.24 | 132.78 | 104.39 | 0.38 | 60.59 | 321.14 | 321.14 |
| 33 | 78.45 | 23.75 | 5.30 | 22.88 | 290.53 | 2850.08 | 537.75 | 0.54 | 0.45 | 0.182 | 55.42 | 293.73 | 1369.23 | 0.165 | 470.26 | 77.59 | 88.73 | 0.38 | 54.97 | 291.34 | 291.34 |
| 34 | 73.15 | 18.45 | 5.28 | 27.14 | 342.93 | 3384.09 | 637.74 | 0.42 | 0.35 | 0.180 | 54.81 | 289.13 | 1528.97 | 0.108 | 363.32 | 39.24 | 68.88 | 0.38 | 47.17 | 248.81 | 248.81 |
| 35 | 67.90 | 13.20 | 5.25 | 31.32 | 393.93 | 3864.48 | 736.09 | 0.30 | 0.25 | 0.177 | 53.90 | 282.96 | 1669.50 | 0.063 | 243.46 | 15.34 | 46.37 | 0.38 | 38.24 | 200.75 | 200.75 |
| 36 | 62.65 | 7.95 | 5.28 | 35.52 | 448.88 | 4403.52 | 834.79 | 0.18 | 0.15 | 0.171 | 52.07 | 274.67 | 1813.81 | 0.034 | 149.72 | 5.09 | 28.38 | 0.38 | 30.68 | 161.84 | 161.84 |
| 37 | 57.35 | 2.65 | 3.98 | 39.76 | 378.63 | 3714.36 | 934.43 | 0.06 | 0.05 | 0.167 | 50.85 | 202.14 | 1463.24 | 0.010 | 37.14 | 0.37 | 9.34 | 0.38 | 22.96 | 91.25 | 91.25 |
| 38 | 54.70 | | sum | 41.88 | total | total | | | | from FIG 6 | /blk ht | total | total | from FIG 3 | total L1 | total M1 | | | | total H | |
| 39 | to base RL | | 52.80 | | 2519.17 | 24713.09 | | | | or Table 4 | | 2086.75 | 10614.99 | or Table 1 | 3807.16 | 1323.60 | | | | 2247.60 | |
| 40 | | | check hand calcs | | ? | ? | | | | | | | | | check L/M = 3-4 | | | | | | |
| 41 | FORMULAE | | | | | | | | | | | | | | 2.88 | | | | | | |
| 42 | User | RL - base RL | | User | Width*Ht*Conc Wt/9.81 | | Unit Wt/Blk Ht | | Ht / H | | Col J * 9.81* Water Ht*(H/Hs)^2 | | | from FIG 3 | | | | | | | |
| 43 | | | 0.5 (RLup-RL)+ 0.5(RL-RLlo) | | | Width*Ht*Conc Wt | | Ht / Hs | | from FIG 8 | | Col K * Blk Ht | | or Table 1 | | Blk wt * Col N^2 | | See Above (S20) | | Col R * (Col O + Col L) | |
| 44 | | | | | | | | | | or Table 4 | | to lumped mass | | | blk wt * Col N | | Col G * Col N | | Col R * (Col Q + Col J) | | Col S * Col |
| 45 | What it means | | (align left of explanation with column) | | | | | | | | | | | | | | | | | | |
| 46 | node RL | node height | | lumped mass at node | | weight per unit height in block | | | | percentage of hydrodynamic force acting along block | | | | | | | | Modal Participation Factor | | | |
| 47 | | | | | lumped weight at node | | | | | | hydrodynamic force acting along block | | | | | | | | lateral EQ force over block height | | |
| 48 | | | | | | | | | | | | hydronamic weight acting at node | | | | | | | | lateral EQ force lumped | |
| 49 | | | | | | | | | | | | | Westergaard check | | | | | | | | check |
| 50 | | | | | | | | | | | | | | mode shape | | | | | | | |

51 Dynamic Identities
52 from Chopra "Dynamics of Structures - A Primer"
53
54 maximum displacement at a node
55 u(j) = L/M Sd phi(j)
56 maximum lateral force at a node and acceleration is
57 f(j) = L/M Sa m(j) phi(j) a(j) = f(j) / m(j)
58 relationship between spectral acceleration and spectral displacement
59 Se = w Sv = w^2 Sd
60 where w is the eigenvalue = 2*pi / T where T is the period.
61 All identities above are for a single mode of vibration.
62 All accelerations are quoted as a ratio of gravity.

Dynamic Identities using EERC Method

force at node, all below for 1st mode, Sa expressed in g
f(y) = L'1/M'1 * Sa * sum over ht( wt * phi(y) + gp(y) )

acceleration at node, in g
a(y) = f(y) / wt/ht(y) = 1/g * f(y) / m(y)

deformation at a node in mm is
u(y) = a(y) / (2 pi / T)^2 * 9.81 * 1000
to convert previous accelerations in g to m then convert to mm

| | V | W | X | Y | Z | AA | AB | AC | AD | AE | AF | AG | AH | AI | AJ | AK |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|

Reservoir wave absorbtion

GRAVDAM.BAS MODEL

| Model elev | Sum H kN | Sum V kN | Sum M kNm | eccentricity e (m) | width T (m) | Stress U/S kPa | Stress D/S kPa |
|---|---|---|---|---|---|---|---|
| 107.50 | | | | | | | |
| 102.20 | 0.00 | 504.00 | 1077.00 | 0.000 | 4.274 | 117.93 | 117.93 |
| 96.93 | 21.00 | 1133.00 | 5926.00 | -1.179 | 8.103 | 261.91 | 17.75 |
| 91.68 | 273.00 | 2081.00 | 16061.00 | -1.556 | 12.324 | 296.78 | 40.94 |
| 86.40 | 814.00 | 3464.00 | 33696.00 | -1.450 | 16.555 | 319.20 | 99.28 |
| 81.10 | 1642.00 | 5283.00 | 60916.00 | -1.138 | 20.785 | 337.67 | 170.68 |
| 75.80 | 2758.00 | 7539.00 | 99807.00 | -0.733 | 25.012 | 354.42 | 248.42 |
| 70.53 | 4161.00 | 10231.00 | 152458.00 | -0.282 | 29.239 | 370.16 | 329.66 |
| 65.28 | 5958.00 | 13362.00 | 220897.00 | 0.202 | 33.467 | 384.79 | 413.71 |
| 60.00 | 8224.00 | 15791.00 | 286929.00 | 0.677 | 37.695 | 373.77 | 464.06 |
| 54.70 | 10783.00 | 18154.00 | 354943.00 | 1.410 | 41.924 | 345.64 | 520.41 |

| | moment KN m /m | stress kPa | accel (g) @ crest | Deformation (mm) @ cres | Revised Resultant Forces due to Additional Earthquake forces elev | Sum H | Elev | Sum H | Sum V | sum M | width | eccentricity | New Resultant Stresses Stress U/S kPa | Stress D/S kPa | Elev |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | M*6/t^2 | 3.22 | 23.63 | 107.50 | | | | | | | | | | |
| | 0.00 | 0.00 | 2.79 | 20.47 | 104.85 | 1110.64 | 102.20 | 1110.64 | 504.00 | -4809.41 | 4.27 | 11.68 | -1815.69 | 2051.55 | 102.20 |
| | 5886.41 | 918.79 | 1.99 | 14.63 | 99.55 | 2641.39 | 96.93 | 2662.39 | 1133.00 | -13827.68 | 8.10 | 16.26 | -1543.34 | 1623.00 | 96.93 |
| | 19753.68 | 1138.53 | 1.70 | 12.51 | 94.30 | 5122.09 | 91.68 | 5395.09 | 2081.00 | -30583.66 | 12.32 | 20.86 | -1545.94 | 1883.66 | 91.68 |
| | 46644.66 | 1349.11 | 1.34 | 9.87 | 89.05 | 7867.77 | 86.40 | 8681.77 | 3464.00 | -54647.85 | 16.55 | 24.05 | -1614.86 | 2033.35 | 86.40 |
| | 88343.85 | 1547.42 | 1.04 | 7.67 | 83.75 | 10577.38 | 81.10 | 12219.38 | 5283.00 | -83487.96 | 20.79 | 26.20 | -1667.84 | 2176.18 | 81.10 |
| | 144403.96 | 1654.64 | 0.78 | 5.74 | 78.45 | 13035.58 | 75.80 | 15793.58 | 7539.00 | -113685.52 | 25.01 | 27.59 | -1693.22 | 2296.06 | 75.80 |
| | 213492.52 | 1739.31 | 0.57 | 4.22 | 73.15 | 15134.89 | 70.53 | 19295.89 | 10231.00 | -140492.68 | 29.24 | 28.35 | -1685.81 | 2385.63 | 70.53 |
| | 292950.68 | 1791.51 | 0.41 | 3.00 | 67.90 | 16828.70 | 65.28 | 22786.70 | 13362.00 | -154573.47 | 33.47 | 28.30 | -1626.53 | 2425.03 | 65.28 |
| | 375470.47 | 1785.29 | 0.29 | 2.14 | 62.65 | 18194.21 | 60.00 | 26416.21 | 15791.00 | -176802.35 | 37.69 | 30.04 | -1584.41 | 2422.25 | 60.00 |
| | 463731.35 | 1759.78 | 0.20 | 1.44 | 57.35 | 18964.12 | 54.70 | 29747.12 | 18154.00 | -162412.29 | 41.92 | 29.91 | -1420.49 | 2286.54 | 54.70 |
| at base | 517355.29 | 1769.56 | | | | | | | | | | | | | |

Two Columns above sum | Col W*6 / Col C^2 | Col Y /(2pi/T)^2 * 9.81*1000

F1(blk) * centroid dist | Col S / Col G

EQ stress add to other stresses calculated

t node | EQ acceleration

Deformation using dynamic Identities

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:52
The input file used was g:\somerset\so_qrpmf.dat
and this output file is g:\somerset\so_qrpmf.out

Somerset Dam - Non overtoppable block passing PMF with one gate closed weight of water flowing over the top of block ignored

Input Data

| | | |
|---|---|---|
| Water Unit Weight | (kN/m3) : | 9.81 |
| Base Friction Strength (Phi) | (ø) : | 45 |
| Base Cohesion | (kPa) : | 1438 |
| Concrete Strength (exterior) | (kPa) : | 20000 |
| Concrete Strength (interior) | (kPa) : | 10000 |
| Foundation Bearing Strength | (kPa) : | 10000 |
| Concrete Unit Weight | (kN/m3) : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 107.5 | 4.267 | .0001 | .0001 |
| 101.8 | 4.26814 | .0001 | .75 |
| 100.5 | 5.243273 | .05 | .75 |
| 54.7 | 41.88327 | .05 | .75 |

External Dam Loads

| | | |
|---|---|---|
| Spilway gates open | = | y |
| Reservoir Level | (m) : | 110.7 |
| Spillway crest level | (m) : | 107.5 |
| Tailwater Level | (m) : | 80 |
| Silt Level | (m) : | 70 |
| Silt Unit Weight | (kN/m3) : | 15.9903 |
| Silt earth pressure coefficient | : | 1 |
| Distance to Drains from heel | (m) : | 5.33 |
| ratio of HWL at Drains | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1 |
| Concrete and Tailwater | : | 1 |
| Silt | : | 1 |
| cohesion and friction | : | .8 |
| concrete strength | : | 1 |
| foundation bearing | : | .8 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN);E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 26,176 | 27.195 | 15,359 | 18.568 | 745 | 41.041 | 728 | 5.117 | 36 | 41.667 |
| 59.94 | 21,223 | 24.474 | 12,590 | 16.795 | 744 | 36.814 | 313 | 3.355 | 16 | 37.528 |
| 65.22 | 16,795 | 21.754 | 10,095 | 15.020 | 744 | 32.586 | 71 | 1.593 | 4 | 33.388 |
| 70.51 | 12,894 | 19.036 | 7,874 | 13.241 | 744 | 28.358 | 0 | 0.000 | 0 | 0.000 |
| 75.79 | 9,519 | 16.318 | 5,928 | 11.458 | 744 | 24.130 | 0 | 0.000 | 0 | 0.000 |
| 81.08 | 6,670 | 13.600 | 4,255 | 9.667 | 744 | 19.902 | 0 | 0.000 | 0 | 0.000 |
| 86.36 | 4,347 | 10.876 | 2,856 | 7.865 | 744 | 15.674 | 0 | 0.000 | 0 | 0.000 |
| 91.65 | 2,550 | 8.123 | 1,731 | 6.045 | 744 | 11.446 | 0 | 0.000 | 0 | 0.000 |
| 96.93 | 1,279 | 5.251 | 880 | 4.188 | 744 | 7.218 | 0 | 0.000 | 0 | 0.000 |
| 102.22 | 531 | 2.134 | 303 | 2.244 | 0 | 4.268 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN);                    C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN);        E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm);K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 13,332 | 22.939 | 3,152 | 8.450 | 2,364 | 6.337 | 12,935 | 15,989 | 190,815 | 9.028 |
| 59.94 | 10,114 | 21.045 | 1,975 | 6.688 | 1,481 | 5.016 | 10,929 | 13,349 | 142,635 | 8.162 |
| 65.22 | 7,335 | 19.241 | 1,071 | 4.927 | 804 | 3.695 | 9,095 | 11,011 | 105,095 | 7.189 |
| 70.51 | 4,994 | 17.594 | 442 | 3.165 | 332 | 2.374 | 7,432 | 8,976 | 76,600 | 6.086 |
| 75.79 | 3,091 | 16.263 | 87 | 1.403 | 65 | 1.052 | 5,841 | 7,237 | 55,285 | 4.866 |
| 81.08 | 1,781 | 15.091 | 0 | 0.000 | 0 | 0.000 | 4,255 | 5,632 | 37,503 | 3.733 |
| 86.36 | 1,295 | 11.972 | 0 | 0.000 | 0 | 0.000 | 2,856 | 3,795 | 20,965 | 2.754 |
| 91.65 | 882 | 8.762 | 0 | 0.000 | 0 | 0.000 | 1,731 | 2,411 | 11,033 | 1.588 |
| 96.93 | 542 | 5.415 | 0 | 0.000 | 0 | 0.000 | 880 | 1,480 | 5,462 | 0.360 |
| 102.22 | 281 | 2.192 | 0 | 0.000 | 0 | 0.000 | 303 | 250 | -162 | 2.783 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa);              C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa)    E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | -111 | 874 | D | 1,396 | 53,978 | 4.17 |
| 59.935 | -106 | 814 | 0 | 1,301 | 47,556 | 4.35 |
| 65.220 | -95 | 753 | 0 | 1,202 | 41,748 | 4.59 |
| 70.505 | -76 | 690 | 0 | 1,096 | 36,633 | 4.93 |
| 75.790 | -48 | 627 | 0 | 987 | 32,155 | 5.51 |
| 81.075 | -21 | 563 | 0 | 881 | 27,486 | 6.46 |
| 86.360 | 0 | 458 | 0 | 716 | 22,081 | 7.73 |
| 91.645 | 44 | 347 | 0 | 542 | 16,110 | 9.31 |
| 96.930 | 134 | 231 | 0 | 362 | 10,502 | 11.94 |
| 102.215 | -171 | 288 | 0 | -257 | -2,041 | -6.74 |

**Levy and Hoffman Criteria**
**Concrete Gravity Dam Crack Stability Assessment**

Levy and Hoffman PWP distribution
Full uplift over crack and no PWP elsewhere

| | | | |
|---|---|---|---|
| Dam - | Somerset Dam | Load Case | PMF Flood - one gate inoperative |
| Client - | SEQWB | Comments | Non overflow block Q/R at block base |
| Date - | 23-Aug-95 | | |
| By - | J Williams | | |
| Checked by - | | | |

**Crack Stability at a Dam Section**

Allowable tens. (kPa) 0
Head at section (m) 56
Tailwater (m) 25.3

| Section Width | 41.9 | Cracked Width | | 37.18 | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Elevation | Conc Wt | W cog | Res H | H cog | Res V | V cog | Silt Hs | Hs cog | Tail Ht | Ht cog | Others V | Others cog |
| 54.7 | 26176 | 27.195 | 15359 | 18.658 | 745 | 41.041 | 728 | 5.117 | 3152 | 8.45 | 2400 | 6.86695 |

| | | | | | | |
|---|---|---|---|---|---|---|
| Uncracked | Sum Horiz | 12935 | Sum Vert | 15989 | Sum Mom | 189430.8 |
| Cracked | Sum Horiz | 12935 | Sum Vert | 26728.021 | Sum Mom | 392727.15 |
| Cracked no U | Sum Horiz | 12935 | Sum Vert | 29321 | Sum Mom | 444883.17 |

z and p not used
z = 0
p = 1

Normal Uplift / Cracked Uplift

| Uplift U | U cog | U1 | U1 cog | U2 | U2 cog | U3 | U3 cog | U4 | U4 cog |
|---|---|---|---|---|---|---|---|---|---|
| 13332 | 22.939 | 2592.9792 | 39.54 | 0 | 0 | 0 | 0 | 0 | 0 |

Uplift according to Levy and Hoffman
uplift over crack only, pore pressure over crack

Stress at Uncracked Upstream Face (for checking purposes)
Stress u = Sum Vert/width*(1-6*(0.5 width - Sum M/Sum Vert)/width))
Stress u = -115.79645 Stress d = 878.99454

Stress at Cracked Section including Uplift
Stress u = 266.840257 Stress d = 1170.9231
less uplift 549.36
net stress 282.519743 less than? 0 allowable tension NB Tension +ve
Levy = Failed
Hoffman rate of change of Levy with increasing crack must be less than zero (increasing stability)
Uncracked portion 37.08 ie more compressive stress with increasing crack

| | | | | | | |
|---|---|---|---|---|---|---|
| Cracked with U | Sum Horiz | 14631 | Sum Vert | 27425.085 | Sum Mom | 381657.05 |

z = 0
p = 1

Normal Uplift / Cracked Uplift

| Uplift U | U cog | U1 | U1 cog | U2 | U2 cog | U3 | U3 cog | U4 | U4 cog |
|---|---|---|---|---|---|---|---|---|---|
| 13332 | 22.939 | 2647.9152 | 39.49 | 0 | 0 | 0 | 0 | 0 | 0 |

Uplift according to Levy and Hoffman
uplift over crack only, pore pressure over crack

Stress at Cracked Section including Uplift
Stress u = 186.263059 Stress d = 1292.9756
less uplift 549.36
net stress 363.096941 less than? 0 allowable tension NB Tension +ve
Levy = Failed
Hoffman = Unstable because crack length increases to failure

**SUMMARY**

| Crack Length | Net Upstream Stress | Levy | Hoffman |
|---|---|---|---|
| 4.72 | 282.519743 | Failed | |
| 4.82 | 363.096941 | Failed | Unstable |
| | Tens +ve | | |

| | |
|---|---|
| If Levy is not met the crack length increases. | X |
| If Levy is met the crack length is stable | |
| If Hoffman is met the crack increases to a stable point | |
| If Hoffman is not met the crack propogates until failure | X |

## EERC Simplified Seismic Anaalysis for Concrete Gravity Dams
## for Somerset Dam - Non Overflow Block - MDE

Note for spreadsheet users   Pink indicates data from model   Red is data from EERC figures and tables

| geometry | SI | million psi | Ratios | |
|---|---|---|---|---|
| H = Height water | 44.23 | | | |
| Hs = Ht stucture | 52.80 | | H/Hs | 0.84 |
| Es = E Strcuture MPa | 22600.00 | 3280.12 | Es/Ef | 0.75 |
| Ef = E fdn MPa | 30000.00 | 4354.14 | | |
| Concrete Density (kN/m3) | 23.50 | | | |

Taken from
"Simplified Analysis for Earthquake Resistant Design
of Concrete Gravity Dams"
by G Fenves and A Chopra
Earthquake Engineering Research Center
UC - Berkeley, June 1986

| Refer EERC Calculation Steps | | | | |
|---|---|---|---|---|
| T1 = Fundamental Period (s) | 0.13 | =0.38*Hs/ sqrt(Es) | | |
| Rr = Reservoir Interaction factor | 1.13 | from Fig 4 | H/Hs | 0.84 |
| Tr = Period Modified for Reservoir | 0.15 | Rr * T1 | | |
| Rw = Period Ratio | 0.81 | =(4H / 1440) / Tr | | |
| Rf = Foundation interaction factor | 1.14 | from Fig 6 | Es/Ef | 0.75 |
| T'1 = Modified Fundamental Period | 0.17 | = Rr * Rf * T1 | | |
| e'1 = Damping Ratio | 0.12 | = 1/Rr 1/Rf^3 e1 + er + ef | | |
| where e1 is rock damping | 0.10 | | | |
| and er = reservoir damping factor | 0.01 | from Fig 5 | H/Hs | 0.84 |
| and ef = foundation damping factor | 0.08 | from Fig 7 | | |
| IF e'1 < e1 use e1 | 0.12 | | | |
| GO TO TABLE | | | | |

| Modal Participation Mass | | | |
|---|---|---|---|
| M1 = | 1323.60 | L1 = | 3807.16 |
| Period (s) = | 0.17 | Freq (Hz) = | 5.82 |
| Sa @ Freq | 1.35 | | |
| M'1 = Rr^2 * M1 = | 1690.10 | | |
| L'1 = L1 + 1/g Fst (H/Hs)^2 Ap | | | |
| Where Fst = 0.5wH^2 = | 9595.62 | | |
| Ap f(Rw, a) from Table 5 | 0.322 | Rw = | 0.81 |
| L'1 = | 4028.18 | a = | 0.90 |
| L'1/M'1 * Sa = | 3.22 | L'1/M'1 = | 2.38 |

| A | B | C | D | E | F | G | H | I | J | K | L | M | N | O | P | Q | R | S | T | U |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | | | | | Rw = | | | | | | | | | | | |
| | | | | | | | | | 0.81 | | | Westergaard | | | | | | | | |
| Crest RL then RL | | | | | | | | | hydrodynamic masses | | | hydrody | | | | | | F1(y) = | | using /ht |
| at centroid | Ht (m) | blk ht | av width | mass (t) | wt | Wt / Ht | y/H | y/Hs | gp(y)/wH | gp(y) | gp | mass | mode 1 phi | w phi | w phi^2 | w/ht *phi | L/M Sa | l/m Sa * (w*phi+gp) | | at joint |
| 107.50 | at centroid | | (at centroid) | | (kN/m3)/m | kN/m3/m2 | | | | N/m/m | N/m | * blk ht | | | | | for Rr | /ht | /blk | * ht |
| 104.65 | 50.15 | 3.98 | 4.27 | 40.63 | 398.59 | 100.27 | 1.13 | 0.95 | 0.000 | 0.00 | 0.00 | 0.00 | 0.866 | 345.18 | 298.93 | 86.84 | 3.22 | 279.41 | 1110.64 | 1110.64 |
| 99.55 | 44.85 | 5.28 | 6.20 | 78.35 | 768.57 | 145.70 | 1.01 | 0.85 | 0.000 | 0.00 | 0.00 | 0.00 | 0.619 | 475.74 | 294.49 | 90.19 | 3.22 | 290.19 | 1530.74 | 1530.74 |
| 94.30 | 39.60 | 5.25 | 10.20 | 128.32 | 1258.80 | 239.77 | 0.90 | 0.75 | 0.124 | 37.76 | 196.23 | 644.89 | 0.455 | 572.75 | 260.60 | 109.10 | 3.22 | 472.52 | 2480.71 | 2480.71 |
| 89.05 | 34.35 | 5.28 | 14.40 | 182.00 | 1785.43 | 338.47 | 0.78 | 0.65 | 0.160 | 48.72 | 257.00 | 946.53 | 0.334 | 596.33 | 199.18 | 113.05 | 3.22 | 520.51 | 2745.68 | 2745.68 |
| 83.75 | 29.05 | 5.30 | 18.51 | 234.98 | 2305.17 | 434.94 | 0.65 | 0.55 | 0.179 | 54.51 | 288.89 | 1178.82 | 0.240 | 553.24 | 132.78 | 104.39 | 3.22 | 511.25 | 2709.61 | 2709.61 |
| 78.45 | 23.75 | 5.30 | 22.88 | 290.53 | 2850.08 | 537.75 | 0.54 | 0.45 | 0.182 | 55.42 | 293.73 | 1369.23 | 0.165 | 470.26 | 77.59 | 88.73 | 3.22 | 463.81 | 2458.20 | 2458.20 |
| 73.15 | 18.45 | 5.28 | 27.14 | 342.93 | 3364.09 | 637.74 | 0.42 | 0.35 | 0.180 | 54.81 | 289.13 | 1528.97 | 0.108 | 363.32 | 39.24 | 68.88 | 3.22 | 397.97 | 2099.31 | 2099.31 |
| 67.90 | 13.20 | 5.25 | 31.32 | 393.93 | 3864.48 | 736.09 | 0.30 | 0.25 | 0.177 | 53.90 | 282.95 | 1669.50 | 0.063 | 243.46 | 15.34 | 46.37 | 3.22 | 322.63 | 1693.81 | 1693.81 |
| 62.65 | 7.95 | 5.28 | 35.52 | 446.88 | 4403.52 | 834.79 | 0.18 | 0.15 | 0.171 | 52.07 | 274.67 | 1813.81 | 0.034 | 149.72 | 5.09 | 28.38 | 3.22 | 258.87 | 1365.51 | 1365.51 |
| 57.35 | 2.65 | 3.98 | 39.76 | 378.63 | 3714.36 | 834.43 | 0.06 | 0.05 | 0.167 | 50.85 | 202.14 | 1463.24 | 0.010 | 37.14 | 0.37 | 9.34 | 3.22 | 193.69 | 769.91 | 769.91 |
| 54.70 | | sum | 41.88 | total | total | | | | from FIG 8 | /blk ht | total | total | from FIG 3 | total L1 | total M1 | | | | total H | |
| to base RL | | 52.80 | | 2519.17 | 24713.09 | | | | or Table 4 | | 2086.75 | 10614.99 | or Table 1 | 3807.16 | 1323.60 | | | | 18964.12 | |
| | | check hand calcs | | ? | ? | | | | | | | | | check L/M = 3-4 | | | | | | |
| FORMULAE | | | | | | | | | | | | | | 2.88 | | | | | | |
| User | RL - base RL | | User | Width*Ht*Conc Wt/9.81 | | Unit Wt/Blk Ht | | Ht / H | | Col J * 9.81* Water Ht*(H/Hs)^2 | | | from FIG 3 | | | | | | | |
| | | 0.5 (RLup-RL)+ 0.5(RL-RLlo) | | | Width*Ht*Conc Wt | | Ht / Hs | | from FIG 8 | | Col K * Blk Ht | | or Table 1 | | Blk wt * Col N^2 | | See Above (S20) | | Col R * (Col O + Col L) | |
| | | | | | | | | | or Table 4 | | to lumped mass | | | blk wt * Col N | | Col G * Col N | | Col R * (Col Q + Col J) | | Col S * Col |
| What it means | | (align left of explanation with column) | | | | | | | | | | | | | | | | | | |
| node RL | node height | | lumped mass at node | | weight per unit height in block | | | | percentage of hydrodynamic force acting along block | | | | | | | | Modal Participation Factor | | | |
| | | | | lumped weight at node | | | | | | hydrodynamic force acting along block | | | | | | | | lateral EQ force over block height | | |
| | | | | | | | | | | | hydronamic weight acting at node | | | | | | | | lateral EQ force lumped | |
| | | | | | | | | | | | | Westergaard check | | | | | | | | check |
| | | | | | | | | | | | | | mode shape | | | | | | | |

Dynamic Identities
from Chopra "Dynamics of Structures - A Primer"

maximum displacement at a node
*u(i) = L/M Sd phi(i)*
maximum lateral force at a node   and acceleration is
*f(i) = L/M Sa m(i) phi(i)*   *a(i) = f(i) / m(i)*
relationship between spectral acceleration and spectral displacement
*Sa = w Sv = w^2 Sd*
where w is the eigenvalue = 2*pi / T where T is the period.
All identities above are for a single mode of vibration.
All accelerations are quoted as a ratio of gravity.

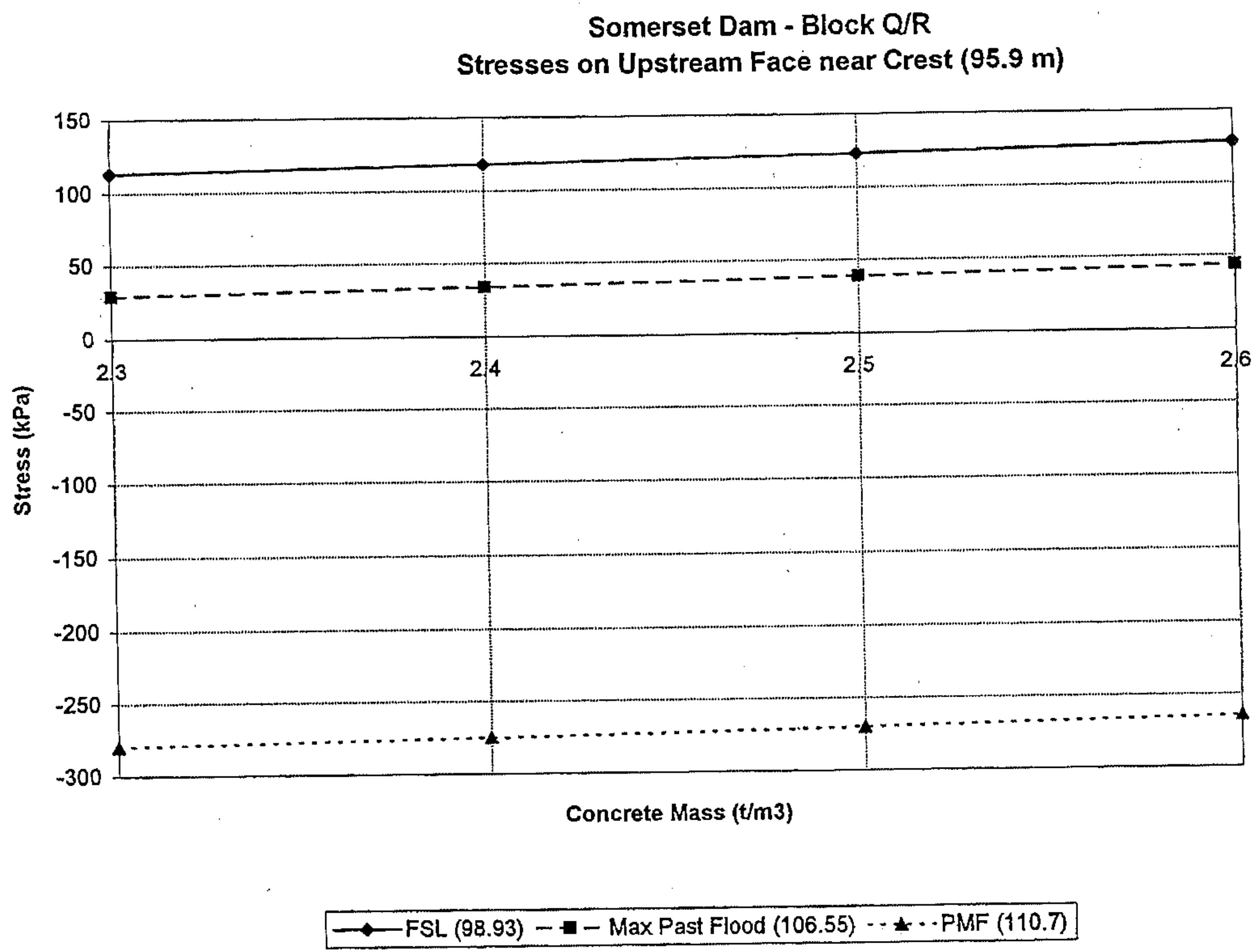
Dynamic Identities using EERC Method

force at node, all below for 1st mode, Sa expressed in g
*f(y) = L'1/M'1 * Sa * sum over hi( wt * phi(y) + gp(y) )*

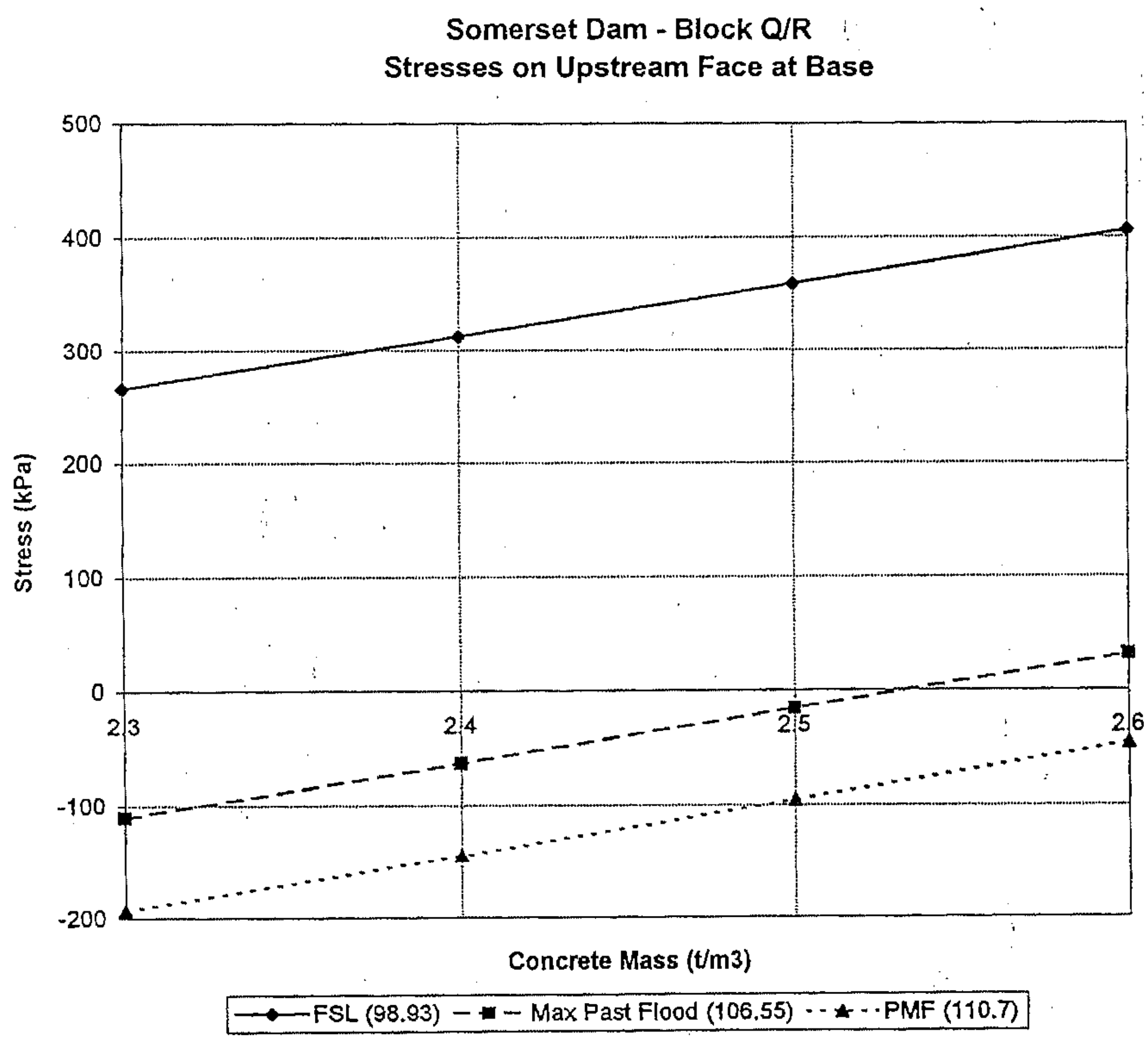
acceleration at node, in g
*a(y) = f(y) / wt/m(y) = 1/g * f(y) / m(y)*

deformation at a node in mm is
*u(y) = a(y) / (2 pi / T)^2 * 9.81 * 1000*
to convert previous accelerations in g to m then convert to mm

| | V | W | X | Y | Z | AA | AB | AC | AD | AE | AF | AG | AH | AI | AJ | AK |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1 | | | | | | | | | | | | | | | | |
| 2 | | | | | | | | | | | | | | | | |
| 3 | | | | | | | | | | | | | | | | |
| 4 | | | | | | | | | | | | | | | | |
| 5 | | | | | | | | | | | | | | | | |
| 6 | | | | | | | | | | | | | | | | |
| 7 | | | | | | | | | | | | | | | | |
| 8 | | | | | | | | | | | | | | | | |
| 9 | | | | | | | | | | | | | | | | |
| 10 | | | | | | | | | | | | | | | | |
| 11 | | | | | | GRAVDAM.BAS MODEL | | | | | | | | | | |
| 12 | | | | | | Model elev | Sum H | Sum V | Sum M | eccentricity | width | Stress U/S | Stress D/S | | | |
| 13 | | | | | | 107.50 | kN | kN | kNm | e (m) | T (m) | kPa | kPa | | | |
| 14 | | | | | | 102.20 | 0.00 | 504.00 | 1077.00 | 0.000 | 4.274 | 117.93 | 117.93 | | | |
| 15 | | | | | | 96.93 | 21.00 | 1133.00 | 5926.00 | -1.179 | 8.103 | 261.91 | 17.75 | | | |
| 16 | | | | | | 91.68 | 273.00 | 2061.00 | 16061.00 | -1.556 | 12.324 | 296.78 | 40.94 | | | |
| 17 | | | | | | 86.40 | 814.00 | 3464.00 | 33696.00 | -1.450 | 16.555 | 319.20 | 99.26 | | | |
| 18 | | | | | | 81.10 | 1642.00 | 5283.00 | 60916.00 | -1.138 | 20.785 | 337.67 | 170.68 | | | |
| 19 | | | | | | 75.80 | 2756.00 | 7539.00 | 99807.00 | -0.733 | 25.012 | 354.42 | 246.42 | | | |
| 20 | Reservoir wave absorbtion | | | | | 70.53 | 4161.00 | 10231.00 | 152458.00 | -0.282 | 29.239 | 370.16 | 329.66 | | | |
| 21 | | | | | | 65.28 | 5958.00 | 13362.00 | 220697.00 | 0.202 | 33.467 | 384.79 | 413.71 | | | |
| 22 | | | | | | 60.00 | 8224.00 | 15791.00 | 286929.00 | 0.677 | 37.695 | 373.77 | 464.06 | | | |
| 23 | | | | | | 54.70 | 10783.00 | 18154.00 | 354943.00 | 1.410 | 41.924 | 345.64 | 520.41 | | | |
| 24 | | | | | | | | | | | | | | New Resultant Stresses | | |
| 25 | | moment | stress | accel (g) | Deformation | Revised Resultant Forces due to Additional Earthquake forces | | | | | | | | | | |
| 26 | | KN m /m | kPa | @ crest | (mm) @ cres | elev | Sum H | Elev | Sum H | Sum V | sum M | width | eccentricity | Stress U/S | Stress D/S | Elev |
| 27 | | | M*6/t^2 | 0.38 | 2.80 | 107.50 | | | | | | | | kPa | kPa | |
| 28 | | 0.00 | 0.00 | 0.33 | 2.43 | 104.85 | 131.63 | 102.20 | 131.63 | 504.00 | 379.35 | 4.27 | 1.38 | -111.24 | 347.10 | 102.20 |
| 29 | | 697.65 | 108.89 | 0.24 | 1.73 | 99.55 | 313.05 | 96.93 | 334.05 | 1133.00 | 3584.82 | 6.10 | 0.89 | 47.95 | 231.71 | 96.93 |
| 30 | | 2341.18 | 134.94 | 0.20 | 1.48 | 94.30 | 607.06 | 91.68 | 880.06 | 2061.00 | 10532.74 | 12.32 | 1.10 | 78.38 | 259.34 | 91.68 |
| 31 | | 5528.26 | 159.89 | 0.16 | 1.17 | 89.05 | 932.48 | 86.40 | 1746.48 | 3464.00 | 23225.62 | 16.55 | 1.57 | 89.98 | 328.50 | 86.40 |
| 32 | | 10470.38 | 183.40 | 0.12 | 0.91 | 83.75 | 1253.62 | 81.10 | 2895.62 | 5283.00 | 43801.46 | 20.79 | 2.10 | 99.98 | 408.37 | 81.10 |
| 33 | | 17114.54 | 196.11 | 0.09 | 0.68 | 78.45 | 1544.96 | 75.80 | 4302.96 | 7539.00 | 74504.18 | 25.01 | 2.62 | 111.74 | 491.10 | 75.80 |
| 34 | | 25302.82 | 206.14 | 0.07 | 0.50 | 73.15 | 1793.76 | 70.53 | 5954.76 | 10231.00 | 117737.92 | 29.24 | 3.11 | 126.49 | 573.33 | 70.53 |
| 35 | | 34720.08 | 212.33 | 0.05 | 0.36 | 67.90 | 1994.51 | 65.28 | 7952.51 | 13362.00 | 176396.80 | 33.47 | 3.53 | 146.42 | 652.09 | 65.28 |
| 36 | | 44500.20 | 211.59 | 0.03 | 0.25 | 62.65 | 2156.35 | 60.00 | 10380.35 | 15791.00 | 231968.25 | 37.69 | 4.16 | 141.69 | 696.14 | 60.00 |
| 37 | | 54960.75 | 208.57 | 0.02 | 0.17 | 57.35 | 2247.60 | 54.70 | 13030.60 | 18154.00 | 293626.82 | 41.92 | 4.79 | 136.32 | 729.73 | 54.70 |
| 38 | at base | 61316.18 | 209.73 | | | | | | | | | | | | | |
| 39 | | | | | | | | | | | | | | | | |
| 40 | | | | | | | | | | | | | | | | |
| 41 | | | | | | | | | | | | | | | | |
| 42 | | | | | | | | | | | | | | | | |
| 43 | Two Columns above sum | | Col W*6 / Col C^2 | | Col Y /(2pi/T)^2 * 9.81*1000 | | | | | | | | | | | |
| 44 | | F1(blk) * centroid dist | | Col S / Col G | | | | | | | | | | | | |
| 45 | | | | | | | | | | | | | | | | |
| 46 | | | | | | | | | | | | | | | | |
| 47 | | | EQ stress add to other stresses calculated | | | | | | | | | | | | | |
| 48 | t node | | | EQ acceleration | | | | | | | | | | | | |
| 49 | | | | | Deformation using | | | | | | | | | | | |
| 50 | | | | | dynamic Identities | | | | | | | | | | | |
| 51 | | | | | | | | | | | | | | | | |
| 52 | | | | | | | | | | | | | | | | |
| 53 | | | | | | | | | | | | | | | | |
| 54 | | | | | | | | | | | | | | | | |
| 55 | | | | | | | | | | | | | | | | |
| 56 | | | | | | | | | | | | | | | | |
| 57 | | | | | | | | | | | | | | | | |
| 58 | | | | | | | | | | | | | | | | |
| 59 | | | | | | | | | | | | | | | | |
| 60 | | | | | | | | | | | | | | | | |
| 61 | | | | | | | | | | | | | | | | |
| 62 | | | | | | | | | | | | | | | | |



Somerset Dam - Block Q/R
Stresses on Upstream Face near Crest (95.9 m)
150
100
50
0
-50
-100
-150
-200
-250
-300
Stress (kPa)
2.3
2.4
2.5
2.6
Concrete Mass (t/m3)
FSL (98.93)
Max Past Flood (106.55)
PMF (110.7)


Somerset Dam - Block Q/R
Stresses on Upstream Face at Base
500
400
300
200
100
0
-100
-200
Stress (kPa)
2.3
2.4
2.5
2.6
Concrete Mass (t/m3)
FSL (98.93)
Max Past Flood (106.55)
PMF (110.7)

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:40
The input file used was g:\somerset\so_klfsl.dat
and this output file is g:\somerset\so_klfsl.out

Somerset Dam - Spillway block at full supply level

Input Data

| | | |
|---|---|---|
| Water Unit Weight | (kN/m3) : | 9.81 |
| Base Friction Strength (Phi) | (ø) : | 45 |
| Base Cohesion | (kPa) : | 1438 |
| Concrete Strength (exterior)(kPa) | : | 20000 |
| Concrete Strength (interior)(kPa) | : | 10000 |
| Foundation Bearing Strength (kPa) | : | 10000 |
| Concrete Unit Weight | (kN/m3) : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 100.5 | 0 | 1.67 | 3 |
| 98.6 | 8.873007 | .05 | .97 |
| 91.7 | 15.91101 | .05 | .7 |
| 54.7 | 43.661 | .05 | .7 |

External Dam Loads

| | | |
|---|---|---|
| Spilway gates open | = | n |
| Reservoir Level | (m) : | 98.93 |
| Spillway crest level | (m) : | 100.5 |
| Tailwater Level | (m) : | 64 |
| Silt Level | (m) : | 70 |
| Silt Unit Weight | (kN/m3) : | 15.9903 |
| Silt earth pressure coefficient | : | 1 |
| Distance to Drains from heel | (m) : | 5.33 |
| ratio of Hwl at Drains | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1.05 |
| Concrete and Tailwater | : | .95 |
| Silt | : | 1.5 |
| cohesion and friction | : | .3 |
| concrete strength | : | .4 |
| foundation bearing | : | .3 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN); E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 28,210 | 27.608 | 9,617 | 14.760 | 482 | 42.958 | 728 | 5.117 | 36 | 43.443 |
| 59.24 | 23,679 | 25.359 | 7,729 | 13.232 | 481 | 39.520 | 358 | 3.588 | 18 | 40.080 |
| 63.82 | 19,518 | 23.108 | 6,046 | 11.703 | 481 | 36.081 | 118 | 2.060 | 6 | 36.718 |
| 68.40 | 15,729 | 20.854 | 4,570 | 10.175 | 481 | 32.642 | 8 | 0.532 | 0 | 33.356 |
| 72.99 | 12,311 | 18.597 | 3,300 | 8.647 | 481 | 29.203 | 0 | 0.000 | 0 | 0.000 |
| 77.57 | 9,264 | 16.339 | 2,237 | 7.118 | 481 | 25.765 | 0 | 0.000 | 0 | 0.000 |
| 82.16 | 6,589 | 14.088 | 1,379 | 5.590 | 481 | 22.326 | 0 | 0.000 | 0 | 0.000 |
| 86.74 | 4,285 | 11.862 | 728 | 4.062 | 481 | 18.887 | 0 | 0.000 | 0 | 0.000 |
| 91.33 | 2,351 | 9.744 | 283 | 2.533 | 481 | 15.448 | 0 | 0.000 | 0 | 0.000 |
| 95.91 | 846 | 6.646 | 45 | 1.005 | 14 | 11.465 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN); C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN); E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm);K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 7,417 | 26.173 | 429 | 3.117 | 300 | 2.182 | 10,783 | 19,790 | 404,467 | 1.411 |
| 59.24 | 5,094 | 25.646 | 111 | 1.588 | 78 | 1.112 | 8,547 | 17,694 | 342,949 | 0.747 |
| 63.82 | 3,081 | 26.317 | 0 | 0.060 | 0 | 0.042 | 6,526 | 15,770 | 285,364 | 0.315 |
| 68.40 | 2,464 | 24.082 | 0 | 0.000 | 0 | 0.000 | 4,811 | 12,813 | 215,391 | -0.120 |
| 72.99 | 1,948 | 21.660 | 0 | 0.000 | 0 | 0.000 | 3,466 | 10,107 | 156,567 | -0.519 |
| 77.57 | 1,484 | 19.218 | 0 | 0.000 | 0 | 0.000 | 2,349 | 7,700 | 108,915 | -0.892 |
| 82.16 | 1,071 | 16.749 | 0 | 0.000 | 0 | 0.000 | 1,448 | 5,592 | 71,448 | -1.245 |
| 86.74 | 710 | 14.245 | 0 | 0.000 | 0 | 0.000 | 765 | 3,782 | 43,189 | -1.606 |
| 91.33 | 400 | 11.697 | 0 | 0.000 | 0 | 0.000 | 297 | 2,271 | 23,157 | -2.104 |
| 95.91 | 136 | 8.207 | 0 | 0.000 | 0 | 0.000 | 47 | 674 | 4,271 | -0.532 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa); C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa) E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | 365 | 541 | 0 | 806 | 24,788 | 2.30 |
| 59.235 | 391 | 488 | 0 | 728 | 22,676 | 2.65 |
| 63.820 | 406 | 450 | 0 | 671 | 20,616 | 3.16 |
| 68.405 | 392 | 376 | 0 | 560 | 18,245 | 3.79 |
| 72.990 | 373 | 302 | 0 | 451 | 15,950 | 4.60 |
| 77.575 | 349 | 232 | 0 | 345 | 13,744 | 5.85 |
| 82.160 | 321 | 164 | 0 | 244 | 11,628 | 8.03 |
| 86.745 | 287 | 98 | 0 | 146 | 9,602 | 12.56 |
| 91.330 | 250 | 31 | 0 | 46 | 7,665 | 25.77 |
| 95.915 | 74 | 42 | 0 | 82 | 5,211 | %111.31 |

Levy and Hoffman Criteria
Concrete Gravity Dam Crack Stability Assessment

| | | | |
|---|---|---|---|
| Dam - | [illegible] Dam | Load Case | Full Supply Level |
| Client - | [illegible] | [illegible] | Spillway block RL |
| Date - | [illegible] | | at block base |
| By - | J Williams | | |
| Checked by - | | | |

[illegible]

SUMMARY

| Crack | Length | Max Upstream Stress | Levy | Hoffman |
|---|---|---|---|---|
| | 6 | -309.59205 | OK | |
| | 4.1 | -317.79019 | OK | OK |

Tens +ve

| | |
|---|---|
| Levy is not met the crack length increases | |
| Levy is met the crack length is stable | N |
| If Hoffman is met the crack increases to a stable point | Y |
| If Hoffman is not met the crack propagates until failure | |

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:53
The input file used was g:\somerset\so_kl_74.dat
and this output file is g:\somerset\so_kl_74.out

Somerset Dam - Spillway block passing max past flood in 1974

Input Data

| | | | |
|---|---|---|---|
| Water Unit Weight | (kN/m3) | : | 9.81 |
| Base Friction Strength (Phi) | (ø) | : | 45 |
| Base Cohesion | (kPa) | : | 1438 |
| Concrete Strength (exterior) | (kPa) | : | 20000 |
| Concrete Strength (interior) | (kPa) | : | 10000 |
| Foundation Bearing Strength | (kPa) | : | 10000 |
| Concrete Unit Weight | (kN/m3) | : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 100.5 | 0 | 1.67 | 3 |
| 98.6 | 8.873007 | .05 | .97 |
| 91.7 | 15.91101 | .05 | .7 |
| 54.7 | 43.661 | .05 | .7 |

External Dam Loads

| | | | |
|---|---|---|---|
| Spilway gates open | | = | n |
| Reservoir Level | (m) | : | 106.55 |
| Spillway crest level | (m) | : | 100.5 |
| Tailwater Level | (m) | : | 72.1 |
| Silt Level | (m) | : | 70 |
| Silt Unit Weight | (kN/m3) | : | 15.9903 |
| Silt earth pressure coefficient | | : | 1 |
| Distance to Drains from heel | (m) | : | 5.33 |
| ratio of Hwl at Drains | | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1.05 |
| Concrete and Tailwater | : | .95 |
| Silt | : | 1.5 |
| cohesion and friction | : | .3 |
| concrete strength | : | .4 |
| foundation bearing | : | .3 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN); E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 28,210 | 27.608 | 13,212 | 17.300 | 863 | 42.141 | 728 | 5.117 | 36 | 43.443 |
| 59.24 | 23,679 | 25.359 | 10,981 | 15.772 | 862 | 38.702 | 358 | 3.588 | 18 | 40.080 |
| 63.82 | 19,518 | 23.108 | 8,956 | 14.243 | 862 | 35.264 | 118 | 2.060 | 6 | 36.718 |
| 68.40 | 15,729 | 20.854 | 7,137 | 12.715 | 862 | 31.825 | 8 | 0.532 | 0 | 33.356 |
| 72.99 | 12,311 | 18.597 | 5,524 | 11.187 | 862 | 28.386 | 0 | 0.000 | 0 | 0.000 |
| 77.57 | 9,264 | 16.339 | 4,118 | 9.658 | 862 | 24.947 | 0 | 0.000 | 0 | 0.000 |
| 82.16 | 6,589 | 14.088 | 2,918 | 8.130 | 862 | 21.509 | 0 | 0.000 | 0 | 0.000 |
| 86.74 | 4,285 | 11.862 | 1,924 | 6.602 | 862 | 18.070 | 0 | 0.000 | 0 | 0.000 |
| 91.33 | 2,351 | 9.744 | 1,136 | 5.073 | 862 | 14.631 | 0 | 0.000 | 0 | 0.000 |
| 95.91 | 846 | 6.646 | 555 | 3.545 | 256 | 10.008 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN); C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN); E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm);K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 10,842 | 24.767 | 1,494 | 5.817 | 1,046 | 4.072 | 13,546 | 17,263 | 260,689 | 6.748 |
| 59.24 | 8,249 | 23.490 | 812 | 4.288 | 568 | 3.002 | 11,296 | 15,209 | 220,518 | 5.631 |
| 63.82 | 5,965 | 22.438 | 336 | 2.760 | 235 | 1.932 | 9,261 | 13,327 | 184,027 | 4.602 |
| 68.40 | 3,991 | 21.841 | 67 | 1.232 | 47 | 0.862 | 7,442 | 11,616 | 150,983 | 3.693 |
| 72.99 | 2,520 | 21.660 | 0 | 0.000 | 0 | 0.000 | 5,801 | 9,868 | 118,532 | 2.960 |
| 77.57 | 2,013 | 19.218 | 0 | 0.000 | 0 | 0.000 | 4,324 | 7,506 | 81,848 | 2.348 |
| 82.16 | 1,557 | 16.749 | 0 | 0.000 | 0 | 0.000 | 3,064 | 5,443 | 53,494 | 1.705 |
| 86.74 | 1,153 | 14.245 | 0 | 0.000 | 0 | 0.000 | 2,020 | 3,678 | 32,490 | 0.980 |
| 91.33 | 801 | 11.697 | 0 | 0.000 | 0 | 0.000 | 1,193 | 2,212 | 17,858 | 0.020 |
| 95.91 | 480 | 8.207 | 0 | 0.000 | 0 | 0.000 | 583 | 543 | 1,579 | 2.900 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa); C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa) E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | 29 | 761 | 0 | 1,134 | 24,030 | 1.77 |
| 59.235 | 61 | 695 | 0 | 1,035 | 21,931 | 1.94 |
| 63.820 | 91 | 633 | 0 | 944 | 19,883 | 2.15 |
| 68.405 | 117 | 579 | 0 | 863 | 17,886 | 2.40 |
| 72.990 | 134 | 525 | 0 | 782 | 15,878 | 2.74 |
| 77.575 | 133 | 434 | 0 | 646 | 13,686 | 3.17 |
| 82.160 | 131 | 341 | 0 | 508 | 11,584 | 3.78 |
| 86.745 | 131 | 244 | 0 | 363 | 9,571 | 4.74 |
| 91.330 | 136 | 138 | 0 | 205 | 7,647 | 6.41 |
| 95.915 | -23 | 117 | 0 | 242 | 3,924 | 6.74 |

Levy and Hoffman Criteria
Concrete Gravity Dam Crack Stability Assessment

Levy and Hoffman PWP distribution
Full uplift over crack and no PWP distribution.

| | | | |
|---|---|---|---|
| Dam : | Somerset Dam | Load Case | 1974 flood |
| Client : | SEQWB | Comments | Spillway block [illegible] |
| Date : | 21-Aug-95 | | at block base |
| By : | J Williams | | |
| Checked by : | | | |

Crack Stability at a Dam Section

Section Width 43.7 Cracked Width 43.7

[illegible]

Levy = Failed

Hoffman = Unstable - increases crack length increases to failure

| SUMMARY | | | |
|---|---|---|---|
| Crack Length | Net Upstream Stress | Levy | Hoffman |
| 0 | 11.7553473 | Failed | |
| 8.1 | 24.7262278 | Failed | Unstable |

Tension

| | |
|---|---|
| If Levy is not met the crack length increases. | X |
| If Levy is met the crack length is stable | |
| If Hoffman is met the crack increases to a stable point. | |
| If Hoffman is not met the crack propagates until failure | X |

## EERC Simplified Seismic Anaalysis for Concrete Gravity Dams

### for Somerset Dam - Spillway Block - OBE

Note for spreadsheet users  Pink indicates data from model  Red is data from EERC figures and tables

| geometry | SI | million psi | Ratios | |
|---|---|---|---|---|
| H = Height water | 44.23 | | | |
| Hs = Ht structure | 45.80 | | H/Hs | 0.97 |
| Es = E Strcuture MPa | 22600.00 | 3280.12 | Es/Ef | 0.75 |
| Ef = E fdn MPa | 30000.00 | 4354.14 | | |
| Concrete Density (kN/m3) | 23.50 | | | |

Taken from
"Simplified Analysis for Earthquake Resistant Design
of Concrete Gravity Dams"
by G Fenves and A Chopra
Earthquake Engineering Research Center
UC - Berkeley, June 1986

| Refer EERC Calculation Steps | | | |
|---|---|---|---|
| T1 = Fundamental Period (s) | 0.12 | =0.38*Hs/ sqrt(Es) | |
| Rr = Reservoir Interaction factor | 1.29 | from Fig 4 H/Hs | 0.97 |
| Tr = Period Modified for Reservoir | 0.15 | Rr * T1 | |
| Rw = Period Ratio | 0.82 | =(4H / 1440) / Tr | |
| Rf = Foundation Interaction factor | 1.14 | from Fig 6 Es/Ef | 0.75 |
| T'1 = Modified Fundamental Period | 0.17 | = Rr * Rf * T1 | |
| e'1 = Damping Ratio | 0.13 | = 1/Rr 1/Rf^3 e1 + er + ef | |
| where e1 is rock damping | 0.10 | | |
| and er = reservoir damping factor | 0.02 | from Fig 5 H/Hs | 0.97 |
| and ef = foundation damping factor | 0.06 | from Fig 7 | |
| IF e'1 < e1 use e1 | 0.13 | | |
| GO TO TABLE | | | |

| Modal Participation Mass | | | |
|---|---|---|---|
| M1 = | 1875.62 | L1 = | 4894.74 |
| Period (s) = | 0.17 | Freq (Hz) = | 5.87 |
| Sa @ Freq | 0.16 | | |
| M'1 = Rr^2 * M1 = | 3121.22 | | |
| L'1 = L1 + 1/g Fst (H/Hs)^2 Ap | | | |
| Where Fst = 0.5wH^2 = | 9595.62 | | |
| Ap f(Rw, a) from Table 5 | 0.322 | Rw = | 0.82 |
| L'1 = | 5188.48 | a = | 0.90 |
| L'1/M'1 * Sa = | 0.27 | L'1/M'1 = | 1.66 |

Rw = 0.82

| Crest RL then RL at centroid | Ht (m) at centroid | blk ht | av width (at centroid) | mass (t) | wt (kN/m3)/m | Wt / Ht kN/m3/m2 | y/H | y/Hs | hydrodynamic masses gp(y)/wH | gp(y) N/m/m | gp N/m | Westergaard hydrody mass * blk ht | mode 1 phi | w phi | w phi^2 | w/ht *phi | L/M Sa for Rr | F1(y) = l/m Sa * (w*phi+gp) /ht | /blk | using /ht at joint * ht |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 100.50 | | | | | | | | | | | | | | | | | | | | |
| 98.20 | 43.50 | 3.45 | 5.81 | 48.05 | 471.37 | 136.63 | 0.96 | 0.95 | 0.070 | 28.33 | 97.74 | 168.27 | 0.866 | 408.21 | 353.51 | 118.32 | 0.27 | 39.00 | 134.57 | 134.57 |
| 93.60 | 38.90 | 4.58 | 13.92 | 152.55 | 1496.47 | 327.10 | 0.88 | 0.85 | 0.130 | 52.61 | 240.70 | 602.96 | 0.619 | 926.31 | 573.39 | 202.47 | 0.27 | 67.85 | 310.39 | 310.39 |
| 89.05 | 34.35 | 4.55 | 17.90 | 195.09 | 1913.85 | 420.63 | 0.78 | 0.75 | 0.170 | 68.80 | 313.04 | 816.44 | 0.455 | 870.80 | 396.21 | 191.39 | 0.27 | 69.20 | 314.87 | 314.87 |
| 84.50 | 29.80 | 4.58 | 21.31 | 233.56 | 2291.20 | 500.81 | 0.67 | 0.65 | 0.179 | 72.44 | 331.42 | 992.11 | 0.334 | 765.26 | 255.60 | 167.27 | 0.27 | 63.76 | 291.69 | 291.69 |
| 79.90 | 25.20 | 4.60 | 24.76 | 272.85 | 2676.66 | 581.88 | 0.57 | 0.55 | 0.181 | 73.25 | 336.95 | 1145.55 | 0.240 | 642.40 | 154.18 | 139.65 | 0.27 | 56.63 | 260.48 | 260.48 |
| 75.30 | 20.60 | 4.60 | 28.21 | 310.87 | 3049.61 | 662.96 | 0.47 | 0.45 | 0.182 | 73.66 | 338.81 | 1276.51 | 0.165 | 503.19 | 83.03 | 109.39 | 0.27 | 48.68 | 223.95 | 223.95 |
| 70.70 | 16.00 | 4.58 | 31.66 | 346.99 | 3403.95 | 744.03 | 0.36 | 0.35 | 0.177 | 71.63 | 327.72 | 1387.66 | 0.108 | 367.63 | 39.70 | 80.36 | 0.27 | 40.42 | 184.94 | 184.94 |
| 66.15 | 11.45 | 4.55 | 35.07 | 382.29 | 3750.29 | 824.24 | 0.26 | 0.25 | 0.174 | 70.42 | 320.40 | 1487.14 | 0.063 | 236.27 | 14.88 | 51.93 | 0.27 | 32.54 | 148.06 | 148.06 |
| 61.60 | 6.90 | 4.58 | 38.49 | 421.79 | 4137.73 | 904.42 | 0.16 | 0.15 | 0.170 | 68.80 | 314.76 | 1595.72 | 0.034 | 140.68 | 4.78 | 30.75 | 0.27 | 26.48 | 121.13 | 121.13 |
| 57.00 | 2.30 | 3.45 | 41.94 | 346.58 | 3399.98 | 985.50 | 0.05 | 0.05 | 0.167 | 67.58 | 233.17 | 1275.31 | 0.010 | 34.00 | 0.34 | 9.85 | 0.27 | 20.60 | 71.06 | 71.06 |
| 54.70 | | sum | 43.66 | total | total | | | | from FIG 8 | /blk ht | total | total | from FIG 3 | total L1 | total M1 | | | | total H | |
| to base RL | | 45.80 | | 2710.61 | 26591.09 | | | | or Table 4 | | 2854.69 | 10747.67 | or Table 1 | 4894.74 | 1875.62 | | | | 2061.13 | |
| | | check hand calcs | | ? | ? | | | | | | | | | check L/M = 3-4 | | | | | | |
| | | | | | | | | | | | | | | 2.61 | | | | | | |

FORMULAE

| A | B | C | D | E | F | G | H | I | J | K | L | M | N | O | P | Q | R | S | T | U |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| User | RL - base RL | 0.5 (RLup-RL)+ 0.5(RL-RLlo) | User | Width*Ht*Conc Wt/9.81 | Width*Ht*Conc Wt | Unit Wt/Blk Ht | Ht / Hs | Ht / H | from FIG 8 or Table 4 | Col J * 9.81* Water Ht*(H/Hs)^2 | Col K * Blk Ht to lumped mass | | from FIG 3 or Table 1 | blk wt * Col N | Blk wt * Col N^2 | Col G * Col N | See Above (S20) | Col R * (Col Q + Col J) | Col R * (Col O + Col L) | Col S * Col |

What it means (align left of explanation with column)

| A | B | D | E | F | J | K | L | M | N | S | T | U |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| node RL | node height | lumped mass at node | lumped weight at node | weight per unit height in block | percentage of hydrodynamic force acting along block | hydrodynamic force acting along block | hydronamic weight acting at node | Westergaard check | mode shape | Modal Participation Factor | lateral EQ force over block height | lateral EQ force lumped / check |

**Dynamic Identities**
from Chopra "Dynamics of Structures - A Primer"

maximum displacement at a node
*u(j) = L/M Sd phi(j)*
maximum lateral force at a node — and acceleration is
*f(j) = L/M Sa m(j) phi(j)* — *a(j) = f(j) / m(j)*
relationship between spectral acceleration and spectral displacement
*Sa = w Sv = w^2 Sd*
where w is the eigenvalue = 2*pi / T where T is the period.
All identities above are for a single mode of vibration.
All accelerations are quoted as a ratio of gravity.

**Dynamic Identities using EERC Method**

force at node, all below for 1st mode, Sa expressed in g
*f(y) = L'1/M'1 * Sa * sum over ht( wt * phi(y) + gp(y) )*

acceleration at node, in g
*a(y) = f(y) / wt/ht(y) = 1/g * f(y) / m(y)*

deformation at a node in mm is
*u(y) = a(y) / (2 pi / T)^2 * 9.81 * 1000*
to convert previous accelerations in g to m then convert to mm

| GRAVDAM.BAS MODEL | | | | | | | |
|---|---|---|---|---|---|---|---|
| Model elev | Sum H | Sum V | Sum M | eccentricity | width | Stress U/S | Stress D/S |
| 100.50 | kN | kN | kNm | e (m) | T (m) | kPa | kPa |
| 95.90 | 47.00 | 674.00 | 4271.00 | -0.532 | 11.610 | 74.02 | 42.09 |
| 91.33 | 297.00 | 2271.00 | 23157.00 | -2.104 | 16.186 | 249.74 | 30.88 |
| 86.78 | 765.00 | 3782.00 | 43169.00 | -1.606 | 19.627 | 287.29 | 98.09 |
| 82.20 | 1448.00 | 5592.00 | 71448.00 | -1.245 | 23.064 | 320.99 | 163.93 |
| 77.60 | 2349.00 | 7700.00 | 108915.00 | -0.892 | 26.506 | 349.16 | 231.85 |
| 73.00 | 3466.00 | 10107.00 | 156567.00 | -0.519 | 29.944 | 372.63 | 302.43 |
| 68.43 | 4811.00 | 12813.00 | 215391.00 | -0.120 | 33.381 | 392.12 | 375.57 |
| 63.88 | 6526.00 | 15770.00 | 285364.00 | 0.315 | 36.821 | 408.31 | 450.28 |
| 59.30 | 8547.00 | 17694.00 | 342949.00 | 0.747 | 40.258 | 390.58 | 488.44 |
| 54.70 | 10783.00 | 19790.00 | 404467.00 | 1.411 | 43.698 | 365.14 | 540.62 |

Reservoir wave absorbtion

| | moment | stress | accel (g) | Deformation | Revised Resultant Forces due to Additional Earthquake forces | | | | | | | | New Resultant Stresses | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | KN m /m | kPa | @ crest | (mm) @ cres | elev | Sum H | Elev | Sum H | Sum V | sum M | width | eccentricity | Stress U/S | Stress D/S | Elev |
| | | M*6/t^2 | 0.27 | 1.92 | 100.50 | | | | | | | | kPa | kPa | |
| | 0.00 | 0.00 | 0.24 | 1.70 | 98.20 | 134.57 | 95.90 | 181.57 | 674.00 | 3651.99 | 11.61 | 0.39 | 46.46 | 69.65 | 95.90 |
| | 619.01 | 19.17 | 0.18 | 1.29 | 93.60 | 444.96 | 91.33 | 741.96 | 2271.00 | 20513.44 | 16.19 | -0.94 | 189.20 | 91.42 | 91.33 |
| | 2643.56 | 49.51 | 0.14 | 1.02 | 89.05 | 759.83 | 86.78 | 1524.83 | 3782.00 | 37088.23 | 19.63 | 0.01 | 192.27 | 193.11 | 86.78 |
| | 6100.77 | 80.60 | 0.11 | 0.80 | 84.50 | 1051.51 | 82.20 | 2499.51 | 5592.00 | 60510.27 | 23.06 | 0.71 | 197.62 | 287.30 | 82.20 |
| | 10937.73 | 107.04 | 0.09 | 0.62 | 79.90 | 1311.99 | 77.60 | 3660.99 | 7700.00 | 91942.10 | 26.51 | 1.31 | 204.21 | 376.80 | 77.60 |
| | 16972.90 | 127.96 | 0.07 | 0.48 | 75.30 | 1535.94 | 73.00 | 5001.94 | 10107.00 | 132528.77 | 29.94 | 1.86 | 211.78 | 463.29 | 73.00 |
| | 24038.23 | 143.88 | 0.05 | 0.36 | 70.70 | 1720.88 | 68.43 | 6531.88 | 12813.00 | 183522.76 | 33.38 | 2.37 | 220.52 | 547.17 | 68.43 |
| | 31858.24 | 155.43 | 0.04 | 0.26 | 66.15 | 1868.94 | 63.88 | 8394.94 | 15770.00 | 245604.35 | 36.82 | 2.84 | 230.35 | 626.23 | 63.88 |
| | 39759.65 | 161.06 | 0.03 | 0.20 | 61.60 | 1990.08 | 59.30 | 10537.08 | 17694.00 | 295469.53 | 40.26 | 3.43 | 214.81 | 664.21 | 59.30 |
| | 47479.47 | 161.99 | 0.02 | 0.14 | 57.00 | 2081.13 | 54.70 | 12844.13 | 19790.00 | 351638.21 | 43.70 | 4.08 | 199.14 | 706.62 | 54.70 |
| at base | 52828.79 | 166.28 | | | | | | | | | | | | | |

Two Columns above sum | Col W*6 / Col C^2 | Col Y /(2pi/T)^2 * 9.81*1000

F1(blk) * centroid dist | Col S / Col G

EQ stress add to other stresses calculated

t node | EQ acceleration

Deformation using dynamic identities

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:55
The input file used was g:\somerset\so_klpmf.dat
and this output file is g:\somerset\so_klpmf.out

Somerset Dam - Spillway block passing PMF with one gate closed Block with gate closed

Input Data

| | | |
|---|---|---|
| Water Unit Weight | (kN/m3) : | 9.81 |
| Base Friction Strength (Phi) | (ø) : | 45 |
| Base Cohesion | (kPa) : | 1438 |
| Concrete Strength (exterior)(kPa) | : | 20000 |
| Concrete Strength (interior)(kPa) | : | 10000 |
| Foundation Bearing Strength (kPa) | : | 10000 |
| Concrete Unit Weight | (kN/m3) : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 100.5 | 0 | 1.67 | 3 |
| 98.6 | 8.873007 | .05 | .97 |
| 91.7 | 15.91101 | .05 | .7 |
| 54.7 | 43.661 | .05 | .7 |

External Dam Loads

| | | |
|---|---|---|
| Spilway gates open | = | n |
| Reservoir Level | (m) : | 110.7 |
| Spillway crest level | (m) : | 100.5 |
| Tailwater Level | (m) : | 80 |
| Silt Level | (m) : | 70 |
| Silt Unit Weight | (kN/m3) : | 15.9903 |
| Silt earth pressure coefficient | : | 1 |
| Distance to Drains from heel | (m) : | 5.33 |
| ratio of Hwl at Drains | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1 |
| Concrete and Tailwater | : | 1 |
| Silt | : | 1 |
| cohesion and friction | : | .8 |
| concrete strength | : | 1 |
| foundation bearing | : | .8 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN);E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 28,210 | 27.608 | 15,410 | 18.683 | 1,082 | 41.913 | 728 | 5.117 | 36 | 43.443 |
| 59.24 | 23,679 | 25.359 | 12,992 | 17.155 | 1,080 | 38.474 | 358 | 3.588 | 18 | 40.080 |
| 63.82 | 19,518 | 23.108 | 10,780 | 15.627 | 1,080 | 35.036 | 118 | 2.060 | 6 | 36.718 |
| 68.40 | 15,729 | 20.854 | 8,774 | 14.098 | 1,080 | 31.597 | 8 | 0.532 | 0 | 33.356 |
| 72.99 | 12,311 | 18.597 | 6,975 | 12.570 | 1,080 | 28.158 | 0 | 0.000 | 0 | 0.000 |
| 77.57 | 9,264 | 16.339 | 5,382 | 11.042 | 1,080 | 24.719 | 0 | 0.000 | 0 | 0.000 |
| 82.16 | 6,589 | 14.088 | 3,995 | 9.513 | 1,080 | 21.281 | 0 | 0.000 | 0 | 0.000 |
| 86.74 | 4,285 | 11.862 | 2,815 | 7.985 | 1,080 | 17.842 | 0 | 0.000 | 0 | 0.000 |
| 91.33 | 2,351 | 9.744 | 1,840 | 6.457 | 1,080 | 14.403 | 0 | 0.000 | 0 | 0.000 |
| 95.91 | 846 | 6.646 | 1,072 | 4.928 | 400 | 9.953 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN); C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN); E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm);K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 13,863 | 23.883 | 3,152 | 8.450 | 2,206 | 5.915 | 12,986 | 17,672 | 242,713 | 8.115 |
| 59.24 | 11,024 | 22.370 | 2,115 | 6.922 | 1,480 | 4.845 | 11,235 | 15,233 | 193,799 | 7.408 |
| 63.82 | 8,495 | 20.931 | 1,284 | 5.393 | 899 | 3.775 | 9,614 | 13,008 | 152,913 | 6.655 |
| 68.40 | 6,275 | 19.610 | 659 | 3.865 | 462 | 2.706 | 8,123 | 10,996 | 119,193 | 5.852 |
| 72.99 | 4,365 | 18.505 | 241 | 2.337 | 169 | 1.636 | 6,734 | 9,196 | 91,763 | 4.993 |
| 77.57 | 2,763 | 17.857 | 29 | 0.808 | 20 | 0.566 | 5,353 | 7,602 | 69,341 | 4.130 |
| 82.16 | 1,822 | 16.749 | 0 | 0.000 | 0 | 0.000 | 3,995 | 5,847 | 47,282 | 3.446 |
| 86.74 | 1,395 | 14.245 | 0 | 0.000 | 0 | 0.000 | 2,815 | 3,970 | 27,753 | 2.823 |
| 91.33 | 1,019 | 11.697 | 0 | 0.000 | 0 | 0.000 | 1,840 | 2,413 | 14,670 | 2.014 |
| 95.91 | 667 | 8.207 | 0 | 0.000 | 0 | 0.000 | 1,072 | 578 | -1,160 | 7.811 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa); C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa) E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | -46 | 855 | 0 | 1,278 | 61,538 | 4.74 |
| 59.235 | -39 | 796 | 0 | 1,189 | 56,093 | 4.99 |
| 63.820 | -30 | 736 | 0 | 1,099 | 50,976 | 5.30 |
| 68.405 | -17 | 676 | 0 | 1,008 | 46,206 | 5.69 |
| 72.990 | -0 | 614 | 0 | 915 | 41,796 | 6.21 |
| 77.575 | 19 | 555 | 0 | 827 | 36,572 | 6.83 |
| 82.160 | 26 | 481 | 0 | 716 | 31,213 | 7.81 |
| 86.745 | 28 | 377 | 0 | 561 | 25,755 | 9.15 |
| 91.330 | 38 | 260 | 0 | 388 | 20,553 | 11.17 |
| 95.915 | -151 | 251 | 0 | -373 | -6,456 | -6.02 |

Levy and Hoffman Criteria
Concrete Gravity Dam Crack Stability Assessment

Levy and Hoffman PWP distribution
Full uplift over crack and no PWP elsewhere

| | | | |
|---|---|---|---|
| Dam - | Hammond Dam | Load Case | PMF Flood - not gate [illegible] |
| Client - | [illegible] | Comments | Spillway block N1 |
| Date - | 23-Aug-95 | | at block base |
| By - | P Williams | | block with gates closed |
| Checked by - | | | |

Crack Stability of a Dam Section

Section Width 43.7 | Cracked Width 41.47 | [illegible] 0 | Head at bottom (m) 56 | Tailwater (m) 15.1

[illegible]

## SUMMARY

| Crack Length | Net Uncracked Stress | Levy | Hoffman |
|---|---|---|---|
| 2.23 | 73.7157436 | Failed | |
| 2.33 | 143.881892 | Failed | Unstable |

Time step

| | |
|---|---|
| If Levy is not met the crack length increases. | X |
| If Levy is met the crack length is stable | |
| If Hoffman is met the crack increases to a stable point | |
| If Hoffman is not met the crack propagates until failure | X |

ANCOLD LIMIT STATE ANALYSIS
GRAVDAM.BAS Gravity Dam Analysis
This analysis was performed on 08-23-1995 at 09:56
The input file used was g:\somerset\so_klpm2.dat
and this output file is g:\somerset\so_klpm2.out

Somerset Dam - Spillway block passing PMF with one gate closed Blocks with gates open

Input Data

| | | |
|---|---|---|
| Water Unit Weight | (kN/m3) : | 9.81 |
| Base Friction Strength (Phi) | (ø) : | 45 |
| Base Cohesion | (kPa) : | 1438 |
| Concrete Strength (exterior)(kPa) | : | 20000 |
| Concrete Strength (interior)(kPa) | : | 10000 |
| Foundation Bearing Strength (kPa) | : | 10000 |
| Concrete Unit Weight | (kN/m3) : | 23.544 |

DAM CROSS SECTION GEOMETRY

| Elevation of top of section | Crest width | Upstream slope | Downstream slope |
|---|---|---|---|
| 100.5 | 0 | 1.67 | 3 |
| 98.6 | 8.873007 | .05 | .97 |
| 91.7 | 15.91101 | .05 | .7 |
| 54.7 | 43.661 | .05 | .7 |

External Dam Loads

| | | |
|---|---|---|
| Spilway gates open | = | y |
| Reservoir Level | (m) : | 110.7 |
| Spillway crest level | (m) : | 100.5 |
| Tailwater Level | (m) : | 80 |
| Silt Level | (m) : | 70 |
| Silt Unit Weight | (kN/m3) : | 15.9903 |
| Silt earth pressure coefficient | : | 1 |
| Distance to Drains from heel | (m) : | 5.33 |
| ratio of HWL at Drains | : | .3333 |

Load factors used

| | | |
|---|---|---|
| U/s Water and Uplift | : | 1 |
| Concrete and Tailwater | : | 1 |
| Silt | : | 1 |
| cohesion and friction | : | .8 |
| concrete strength | : | 1 |
| foundation bearing | : | .8 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Weight of conc (kN); C) Hor dist c of g from D/S toe (m)
D) Hor reservoir water force (kN); E) Ht to c of g of water force (m)
F) Vert res water force (kN); G) Hor dist c of g from D/S toe (m)
H) Horizontal silt force (kN); I) Ht to c of g of silt force (m)
J) Vertical silt force (kN); K) Hor dist c of g silt force (m)

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 28,210 | 27.608 | 14,899 | 17.636 | 1,082 | 41.913 | 728 | 5.117 | 36 | 43.443 |
| 59.24 | 23,679 | 25.359 | 12,481 | 16.030 | 1,080 | 38.474 | 358 | 3.588 | 18 | 40.080 |
| 63.82 | 19,518 | 23.108 | 10,270 | 14.412 | 1,080 | 35.036 | 118 | 2.060 | 6 | 36.718 |
| 68.40 | 15,729 | 20.854 | 8,264 | 12.777 | 1,080 | 31.597 | 8 | 0.532 | 0 | 33.356 |
| 72.99 | 12,311 | 18.597 | 6,465 | 11.122 | 1,080 | 28.158 | 0 | 0.000 | 0 | 0.000 |
| 77.57 | 9,264 | 16.339 | 4,872 | 9.441 | 1,080 | 24.719 | 0 | 0.000 | 0 | 0.000 |
| 82.16 | 6,589 | 14.088 | 3,485 | 7.723 | 1,080 | 21.281 | 0 | 0.000 | 0 | 0.000 |
| 86.74 | 4,285 | 11.862 | 2,304 | 5.954 | 1,080 | 17.842 | 0 | 0.000 | 0 | 0.000 |
| 91.33 | 2,351 | 9.744 | 1,330 | 4.111 | 1,080 | 14.403 | 0 | 0.000 | 0 | 0.000 |
| 95.91 | 846 | 6.646 | 562 | 2.152 | 400 | 9.953 | 0 | 0.000 | 0 | 0.000 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Uplift force (kN); C) Hor dist c of g from D/S toe (m)
D) Hor tailwater force (kN); E) Ht to c of g of tailwater force (m)
F) Vertical tailwater force (kN); G) Hor dist c of g from D/S toe (m)
H) Sum of horizontal forces (kN); I) Sum of vertical forces (kN)
J) Sum moments abou D/S toe (kNm);K) Eccentricity e

| A | B | C | D | E | F | G | H | I | J | K |
|---|---|---|---|---|---|---|---|---|---|---|
| 54.65 | 13,863 | 23.883 | 3,152 | 8.450 | 2,206 | 5.915 | 12,475 | 17,672 | 267,846 | 6.693 |
| 59.24 | 11,024 | 22.370 | 2,115 | 6.922 | 1,480 | 4.845 | 10,724 | 15,233 | 216,593 | 5.911 |
| 63.82 | 8,495 | 20.931 | 1,284 | 5.393 | 899 | 3.775 | 9,103 | 13,008 | 173,367 | 5.083 |
| 68.40 | 6,275 | 19.610 | 659 | 3.865 | 462 | 2.706 | 7,612 | 10,996 | 137,306 | 4.205 |
| 72.99 | 4,365 | 18.505 | 241 | 2.337 | 169 | 1.636 | 6,224 | 9,196 | 107,537 | 3.277 |
| 77.57 | 2,763 | 17.857 | 29 | 0.808 | 20 | 0.566 | 4,843 | 7,602 | 82,776 | 2.363 |
| 82.16 | 1,822 | 16.749 | 0 | 0.000 | 0 | 0.000 | 3,485 | 5,847 | 58,376 | 1.549 |
| 86.74 | 1,395 | 14.245 | 0 | 0.000 | 0 | 0.000 | 2,304 | 3,970 | 36,507 | 0.618 |
| 91.33 | 1,019 | 11.697 | 0 | 0.000 | 0 | 0.000 | 1,330 | 2,413 | 21,085 | -0.645 |
| 95.91 | 667 | 8.207 | 0 | 0.000 | 0 | 0.000 | 562 | 578 | 2,915 | 0.766 |

For the Table Below.....
A) Elevation stresses calculated (m)
B) Stress U/S face (kPa); C) Stress D/S face (kPa)
D) Stress D/S if cracked (kPa) E) Maximum D/S stress (kPa)
F) Resistance to sliding which must be greater than Sigma H
G) Resistance to sliding / Sigma H

| A | B | C | D | E | F | G |
|---|---|---|---|---|---|---|
| 54.650 | 33 | 776 | 0 | 1,156 | 64,408 | 5.16 |
| 59.235 | 45 | 712 | 0 | 1,060 | 58,501 | 5.45 |
| 63.820 | 61 | 646 | 0 | 962 | 52,766 | 5.80 |
| 68.405 | 80 | 578 | 0 | 862 | 47,200 | 6.20 |
| 72.990 | 105 | 509 | 0 | 758 | 41,803 | 6.72 |
| 77.575 | 133 | 440 | 0 | 656 | 36,572 | 7.55 |
| 82.160 | 151 | 356 | 0 | 530 | 31,213 | 8.96 |
| 86.745 | 164 | 240 | 0 | 358 | 25,755 | 11.18 |
| 91.330 | 185 | 113 | 0 | 169 | 20,553 | 15.45 |
| 95.915 | 30 | 70 | 0 | 135 | 13,821 | 24.60 |

**Levy and Hoffman Criteria**
**Concrete Gravity Dam Crack Stability Assessment**

Levy and Hoffman PWP distribution
Full uplift over crack and no PWP elsewhere

| | | | |
|---|---|---|---|
| Dam - | Somerset Dam | Load Case | PMF Flood - one gate inoperative |
| Client - | SEQWB | Comments | Spillway block KL<br>at block base<br>block with gates open |
| Date - | 23-Aug-95 | | |
| By - | J Williams | | |
| Checked by - | | | |

Crack Stability at a Dam Section

Allowable tens. (kPa) 0

Section Width 43.7 Cracked Width 43.7 Head at section (m) 56 Tailwater (m) 25.3

| Elevation | Conc Wt | W cog | Res H | H cog | Res V | V cog | Silt Hs | Hs cog | Tail Ht | Ht cog | Others V | Others cog |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 54.7 | 28210 | 27.608 | 14899 | 17.636 | 1082 | 41.943 | 728 | 5.117 | 3152 | 8.45 | 2242 | 6.5175905 |

| | | | | | | |
|---|---|---|---|---|---|---|
| Uncracked | Sum Horiz | 12475 | Sum Vert | 17671 | Sum Mom | 267876.88 |
| Cracked | Sum Horiz | 12475 | Sum Vert | 31534 | Sum Mom | 598966.9 |
| Cracked no U | Sum Horiz | 12475 | Sum Vert | 31534 | Sum Mom | 534125.84 |

z and p not used
z = 0
p = 1

Normal Uplift / Cracked Uplift

| Uplift U | U cog | U1 | U1 cog | U2 | U2 cog | U3 | U3 cog | U4 | U4 cog |
|---|---|---|---|---|---|---|---|---|---|
| 13863 | 23.883 | 0 | 43.7 | 0 | 0 | 0 | 0 | 0 | 0 |

Uplift according to Levy and Hoffman
uplift over crack only, pore pressure over crack

Stress at Uncracked Upstream Face (for checking purposes)
Stress u = Sum Vert/width*(1-6*(0.5 width - Sum M/Sum Vert)/width)
Stress u = 32.8932183 Stress d = 775.8482

ess at Cracked Section including Uplift
Stress u = 438.6732 Stress d = 1004.5305
less uplift 549.36
net stress 110.6868 less than? 0 allowable tension NB Tension +ve
Levy = Failed
Hoffman rate of change of Levy with increasing crack must be less than zero (increasing stability)
Uncracked portion 43.6 ie more compressive stress with increasing crack

Cracked with U Sum Horiz 14171 Sum Vert 32389.064 Sum Mom 589406.86
z = 0
p = 1

Normal Uplift / Cracked Uplift

| Uplift U | U cog | U1 | U1 cog | U2 | U2 cog | U3 | U3 cog | U4 | U4 cog |
|---|---|---|---|---|---|---|---|---|---|
| 13863 | 23.883 | 54.936 | 43.65 | 0 | 0 | 0 | 0 | 0 | 0 |

Uplift according to Levy and Hoffman
uplift over crack only, pore pressure over crack

Stress at Cracked Section including Uplift
Stress u = 374.607982 Stress d = 1111.1289
less uplift 549.36
net stress 174.752018 less than? 0 allowable tension NB Tension +ve
Levy = Failed
Hoffman = Unstable because crack length increases to failure

**SUMMARY**

| ck Length | Net Upstream Stress | Levy | Hoffman |
|---|---|---|---|
| 0 | 110.6868 | Failed | |
| 0.1 | 174.752018 | Failed | Unstable |
| | Tens +ve | | |

| | |
|---|---|
| If Levy is not met the crack length increases. | X |
| If Levy is met the crack length is stable | |
| If Hoffman is met the crack increases to a stable point | |
| If Hoffman is not met the crack propogates until failure | X |

## EERC Simplified Seismic Anaalysis for Concrete Gravity Dams
## for Somerset Dam - Spillway Block - MDE

Note for spreadsheet users   Pink indicates data from model   Red is data from EERC figures and tables

| geometry | SI | million psi | Ratios | |
|---|---|---|---|---|
| H = Height water | 44.23 | | | |
| Hs = Ht structure | 45.80 | | H/Hs | 0.97 |
| Es = E Strcuture MPa | 22600.00 | 3280.12 | Es/Ef | 0.75 |
| Ef = E fdn MPa | 30000.00 | 4354.14 | | |
| Concrete Density (kN/m3) | 23.50 | | | |

Taken from
"Simplified Analysis for Earthquake Resistant Design
of Concrete Gravity Dams"
by G Fenves and A Chopra
Earthquake Engineering Research Center
UC - Berkeley, June 1986

| Refer EERC Calculation Steps | | | |
|---|---|---|---|
| T1 = Fundamental Period (s) | 0.12 | =0.38*Hs/ sqrt(Es) | |
| Rr = Reservoir Interaction factor | 1.29 | from Fig 4 H/Hs | 0.97 |
| T'r = Period Modified for Reservoir | 0.15 | Rr * T1 | |
| Rw = Period Ratio | 0.82 | =(4H / 1440) / T'r | |
| Rf = Foundation Interaction factor | 1.14 | from Fig 6 Es/Ef | 0.75 |
| T'1 = Modified Fundamental Period | 0.17 | = Rr * Rf * T1 | |
| e'1 = Damping Ratio | 0.13 | = 1/Rr 1/Rf^3 e1 + er + ef | |
| where e1 is rock damping | 0.10 | | |
| and er = reservoir damping factor | 0.02 | from Fig 5 H/Hs | 0.97 |
| and ef = foundation damping factor | 0.06 | from Fig 7 | |
| IF e'1 < e1 use e1 | 0.13 | | |
| GO TO TABLE | | | |

| Modal Participation Mass | | | |
|---|---|---|---|
| M1 = | 1875.62 | L1 = | 4894.74 |
| Period (s) = | 0.17 | Freq (Hz) = | 5.87 |
| Sa @ Freq | 1.35 | | |
| M'1 = Rr^2 * M1 = | 3121.22 | | |
| L'1 = L1 + 1/g Fst (H/Hs)^2 Ap | | | |
| Where Fst = 0.5wH^2 = | 9595.62 | | |
| Ap f(Rw, a) from Table 5 | 0.322 | Rw = | 0.82 |
| L'1 = | 5188.48 | a = | 0.90 |
| L'1/M'1 * Sa = | 2.24 | L'1/M'1 = | 1.66 |

Rw = 0.82

| Crest RL then RL at centroid | Ht (m) | blk ht | av width (at centroid) | mass (t) | wt (kN/m3)/m | Wt / Ht kN/m3/m2 | y/H | y/Hs | hydrodynamic masses gp(y)/wH | gp(y) N/m/m | gp N/m | Westergaard hydrody mass * blk ht | mode 1 phi | w phi | w phi^2 | w/ht *phi | L/M Sa for Rr | F1(y) = l/m Sa * (w*phi+gp) /ht | /blk | using /ht at joint * ht |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 100.50 | at centroid | | | | | | | | | | | | | | | | | | | |
| 98.20 | 43.50 | 3.45 | 5.81 | 48.05 | 471.37 | 136.63 | 0.98 | 0.95 | 0.070 | 28.33 | 97.74 | 168.27 | 0.866 | 408.21 | 353.51 | 118.32 | 2.24 | 329.10 | 1135.40 | 1135.40 |
| 93.60 | 38.90 | 4.58 | 13.92 | 152.55 | 1496.47 | 327.10 | 0.88 | 0.85 | 0.130 | 52.61 | 240.70 | 602.96 | 0.619 | 926.31 | 573.39 | 202.47 | 2.24 | 572.44 | 2618.93 | 2618.93 |
| 89.05 | 34.35 | 4.55 | 17.90 | 195.09 | 1913.85 | 420.63 | 0.78 | 0.75 | 0.170 | 68.80 | 313.04 | 816.44 | 0.455 | 870.80 | 396.21 | 191.39 | 2.24 | 583.89 | 2656.70 | 2656.70 |
| 84.50 | 29.80 | 4.58 | 21.31 | 233.56 | 2291.20 | 500.81 | 0.67 | 0.65 | 0.179 | 72.44 | 331.42 | 992.11 | 0.334 | 765.26 | 255.60 | 167.27 | 2.24 | 537.95 | 2461.10 | 2461.10 |
| 79.90 | 25.20 | 4.60 | 24.76 | 272.85 | 2676.66 | 581.88 | 0.57 | 0.55 | 0.181 | 73.25 | 336.95 | 1145.55 | 0.240 | 642.40 | 154.18 | 139.65 | 2.24 | 477.78 | 2197.80 | 2197.80 |
| 75.30 | 20.60 | 4.60 | 28.21 | 310.87 | 3049.61 | 662.96 | 0.47 | 0.45 | 0.182 | 73.66 | 338.81 | 1276.51 | 0.165 | 503.19 | 83.03 | 109.39 | 2.24 | 410.78 | 1889.57 | 1889.57 |
| 70.70 | 16.00 | 4.58 | 31.66 | 346.99 | 3403.95 | 744.03 | 0.36 | 0.35 | 0.177 | 71.63 | 327.72 | 1387.66 | 0.108 | 367.63 | 39.70 | 80.36 | 2.24 | 341.08 | 1560.45 | 1560.45 |
| 66.15 | 11.45 | 4.55 | 35.07 | 382.29 | 3750.29 | 824.24 | 0.26 | 0.25 | 0.174 | 70.42 | 320.40 | 1487.14 | 0.063 | 236.27 | 14.88 | 51.83 | 2.24 | 274.56 | 1249.24 | 1249.24 |
| 61.60 | 6.90 | 4.58 | 38.49 | 421.79 | 4137.73 | 904.42 | 0.16 | 0.15 | 0.170 | 68.80 | 314.76 | 1595.72 | 0.034 | 140.68 | 4.78 | 30.75 | 2.24 | 223.40 | 1022.07 | 1022.07 |
| 57.00 | 2.30 | 3.45 | 41.94 | 346.58 | 3399.96 | 985.50 | 0.05 | 0.05 | 0.167 | 67.58 | 233.17 | 1275.31 | 0.010 | 34.00 | 0.34 | 9.85 | 2.24 | 173.79 | 599.56 | 599.56 |
| 54.70 | | sum | 43.66 | total | total | | | | from FIG 8 | /blk ht | total | total | from FIG 3 | total L1 | total M1 | | | | total H | |
| to base RL | | 45.60 | | 2710.61 | 26591.09 | | | | or Table 4 | | 2854.69 | 10747.67 | or Table 1 | 4894.74 | 1875.62 | | | | 17390.82 | |
| | | check hand calcs | | ? | ? | | | | | | | | | check L/M = 3-4 | | | | | | |
| | | | | | | | | | | | | | | 2.61 | | | | | | |

FORMULAE

| User | RL - base RL | User | Width*Ht*Conc Wt/9.81 | Unit Wt/Blk Ht | Ht / H | from FIG 8 or Table 4 | Col J * 9.81 * Water Ht*(H/Hs)^2 | Col K * Blk Ht to lumped mass | from FIG 3 or Table 1 | blk wt * Col N | Blk wt * Col N^2 | Col G * Col N | See Above (S20) | Col R * (Col Q + Col J) | Col R * (Col O + Col L) | Col S * Col |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | 0.5 (RLup-RL)+ 0.5(RL-RLlo) | Width*Ht*Conc Wt | | Ht / Hs | | | | | | | | | | | |

What it means (align left of explanation with column)

| node RL | node height | | lumped mass at node | weight per unit height in block | | percentage of hydrodynamic force acting along block | hydrodynamic force acting along block | hydronamic weight acting at node | Westergaard check | mode shape | Modal Participation Factor | lateral EQ force over block height | lateral EQ force lumped | check |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | lumped weight at node | | | | | | | | | | |

Dynamic Identities
from Chopra "Dynamics of Structures - A Primer"

maximum displacement at a node
u(j) = L/M Sd phi(j)
maximum lateral force at a node    and acceleration is
f(j) = L/M Sa m(j) phi(j)    a(j) = f(j) / m(j)
relationship between spectral acceleration and spectral displacement
Sa = w Sv = w^2 Sd
where w is the eigenvalue = 2*pi / T where T is the period.
All identities above are for a single mode of vibration.
All accelerations are quoted as a ratio of gravity.

Dynamic Identities using EERC Method

force at node, all below for 1st mode, Sa expressed in g
f(y) = L'1/M'1 * Sa * sum over ht( wt * phi(y) + gp(y) )

acceleration at node, in g
a(y) = f(y) / wt/ht(y) = 1/g * f(y) / m(y)

deformation at a node in mm is
u(y) = a(y) / (2 pi / T)^2 * 9.81 * 1000
to convert previous accelerations in g to m then convert to mm

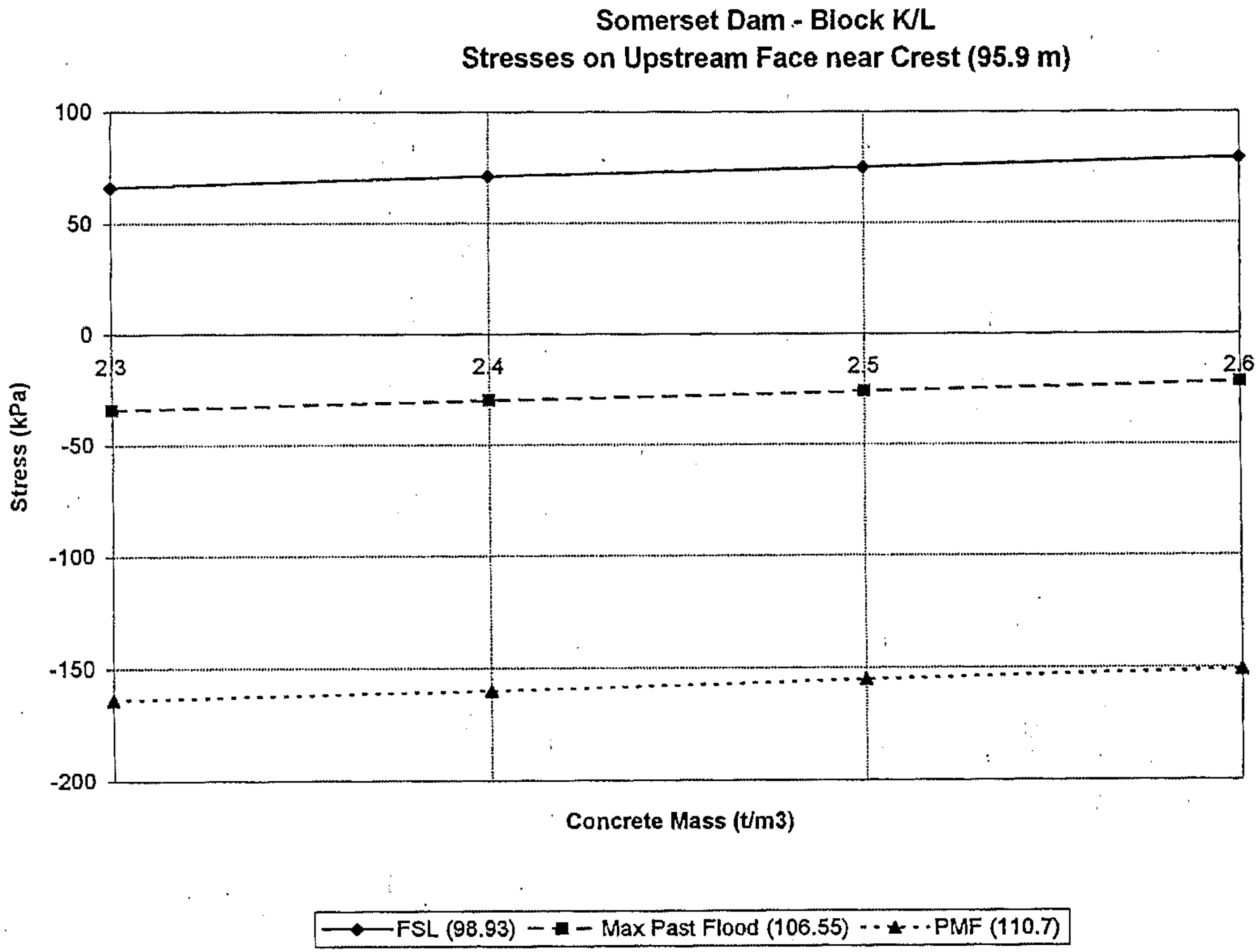
Reservoir wave absorbtion

| GRAVDAM.BAS MODEL | | | | | | | |
|---|---|---|---|---|---|---|---|
| Model elev | Sum H | Sum V | Sum M | eccentricity | width | Stress U/S | Stress D/S |
| 100.50 | kN | kN | kNm | e (m) | T (m) | kPa | kPa |
| 95.90 | 47.00 | 674.00 | 4271.00 | -0.532 | 11.610 | 74.02 | 42.09 |
| 91.33 | 287.00 | 2271.00 | 23157.00 | -2.104 | 16.186 | 249.74 | 30.88 |
| 86.78 | 765.00 | 3782.00 | 43189.00 | -1.606 | 19.627 | 287.29 | 98.09 |
| 82.20 | 1448.00 | 5592.00 | 71448.00 | -1.245 | 23.064 | 320.99 | 163.93 |
| 77.60 | 2349.00 | 7700.00 | 108915.00 | -0.892 | 26.506 | 349.16 | 231.85 |
| 73.00 | 3466.00 | 10107.00 | 156567.00 | -0.519 | 29.944 | 372.63 | 302.43 |
| 68.43 | 4811.00 | 12813.00 | 215391.00 | -0.120 | 33.381 | 392.12 | 375.57 |
| 63.88 | 6526.00 | 15770.00 | 285364.00 | 0.315 | 36.821 | 406.31 | 450.28 |
| 59.30 | 8547.00 | 17694.00 | 342949.00 | 0.747 | 40.258 | 390.58 | 488.44 |
| 54.70 | 10783.00 | 19790.00 | 404467.00 | 1.411 | 43.698 | 365.14 | 540.62 |

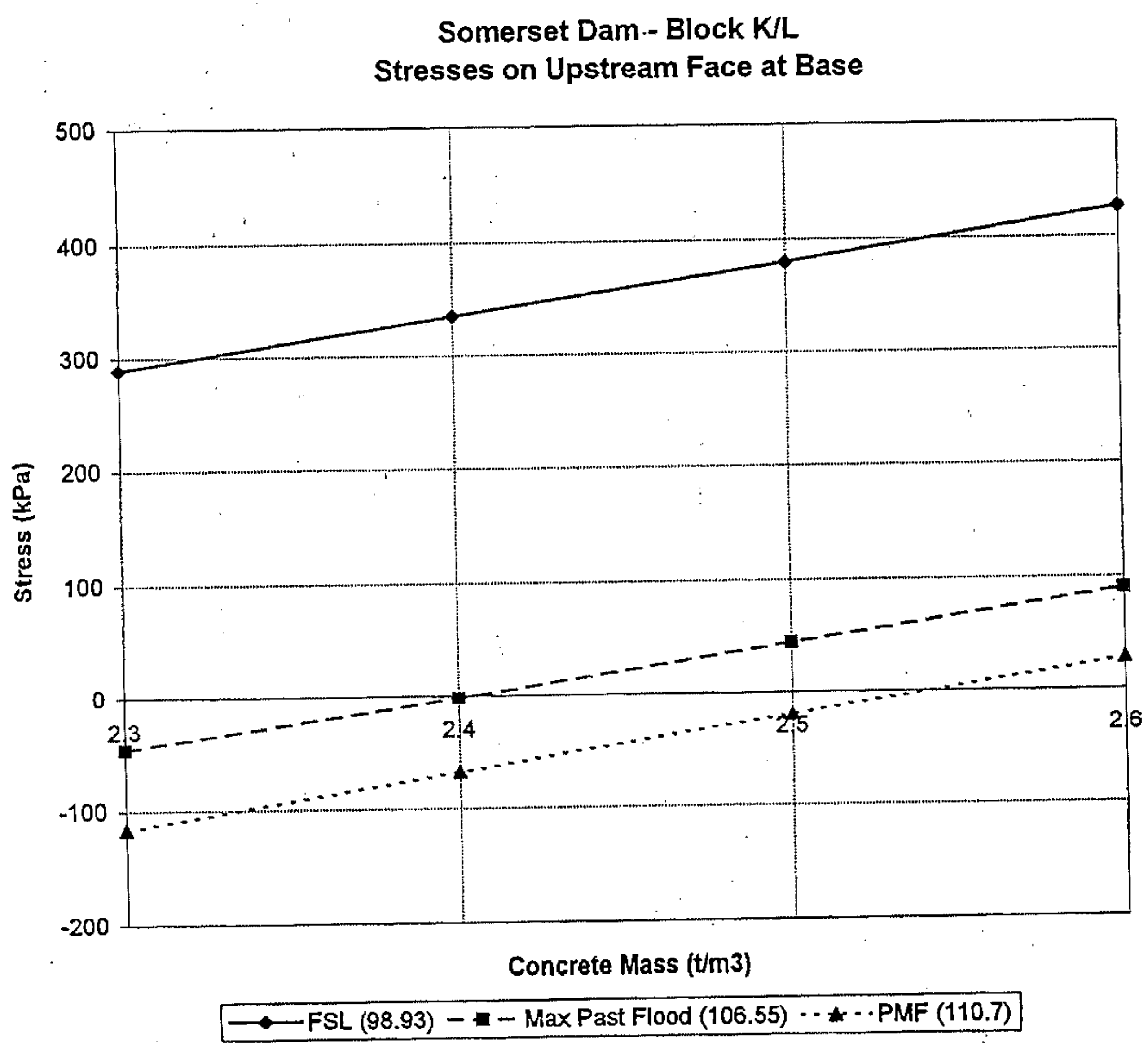
Revised Resultant Forces due to Additional Earthquake forces; New Resultant Stresses

| | moment | stress | accel (g) | Deformation | elev | Sum H | Elev | Sum H | Sum V | sum M | width | eccentricity | Stress U/S | Stress D/S | Elev |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | KN m /m | kPa | @ crest | (mm) @ cres | | | | | | | | | kPa | kPa | |
| | | M*6/t^2 | 2.24 | 16.16 | 100.50 | | | | | | | | | | |
| | 0.00 | 0.00 | 2.00 | 14.37 | 96.20 | 1135.40 | 95.90 | 1182.40 | 674.00 | -951.86 | 11.61 | 7.22 | -158.48 | 274.59 | 95.90 |
| | 5222.86 | 161.75 | 1.51 | 10.86 | 93.60 | 3754.33 | 91.33 | 4051.33 | 2271.00 | 851.92 | 16.19 | 7.72 | -261.11 | 541.73 | 91.33 |
| | 22305.08 | 417.73 | 1.19 | 8.59 | 89.05 | 6411.03 | 86.78 | 7176.03 | 3782.00 | -8266.27 | 19.63 | 12.00 | -514.44 | 899.83 | 86.78 |
| | 51475.27 | 680.05 | 0.94 | 6.76 | 84.50 | 8872.13 | 82.20 | 10320.13 | 5592.00 | -20839.08 | 23.06 | 15.26 | -719.98 | 1204.90 | 82.20 |
| | 92287.08 | 903.14 | 0.73 | 5.25 | 79.90 | 11069.94 | 77.60 | 13418.94 | 7700.00 | -34293.80 | 26.51 | 17.71 | -873.89 | 1454.90 | 77.60 |
| | 143208.80 | 1079.65 | 0.56 | 4.02 | 75.30 | 12959.51 | 73.00 | 16425.51 | 10107.00 | -46255.53 | 29.94 | 19.55 | -984.59 | 1659.65 | 73.00 |
| | 202822.53 | 1214.00 | 0.42 | 3.01 | 70.70 | 14519.95 | 68.43 | 19330.95 | 12813.00 | -53497.31 | 33.38 | 20.87 | -1055.76 | 1823.44 | 68.43 |
| | 268888.31 | 1311.45 | 0.31 | 2.21 | 66.15 | 15769.20 | 83.88 | 22295.20 | 15770.00 | -50108.07 | 36.82 | 21.59 | -1078.34 | 1934.92 | 63.88 |
| | 335472.07 | 1358.95 | 0.23 | 1.65 | 61.60 | 16791.26 | 59.30 | 25338.26 | 17694.00 | -57659.03 | 40.26 | 23.39 | -1092.47 | 1971.50 | 59.30 |
| | 400608.03 | 1368.77 | 0.17 | 1.19 | 57.00 | 17390.82 | 54.70 | 28173.82 | 19790.00 | -41275.92 | 43.70 | 23.93 | -1035.46 | 1941.22 | 54.70 |
| at base | 445742.92 | 1402.97 | | | | | | | | | | | | | |

| | | | | |
|---|---|---|---|---|
| Two Columns above sum | | Col W*6 / Col C^2 | | Col Y /(2pi/T)^2 * 9.81*1000 |
| | F1(blk) * centroid dist | | Col S / Col G | |
| | | EQ stress add to other stresses calculated | | |
| t node | | | EQ acceleration | |
| | | | | Deformation using dynamic identities |



Somerset Dam - Block K/L
Stresses on Upstream Face near Crest (95.9 m)
100
50
0
-50
-100
-150
-200
Stress (kPa)
2.3
2.4
2.5
2.6
Concrete Mass (t/m3)
FSL (98.93)
Max Past Flood (106.55)
PMF (110.7)

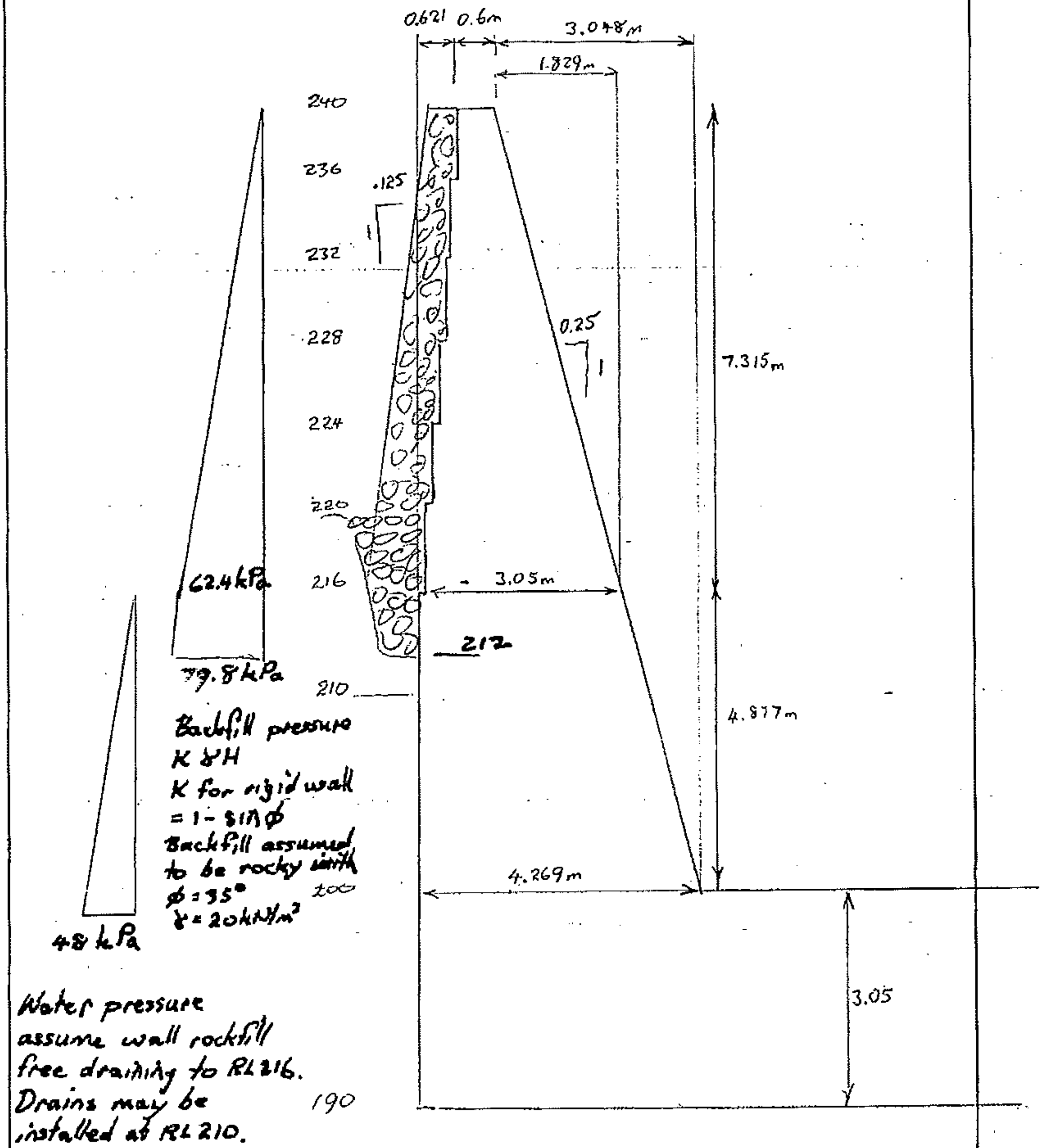

Somerset Dam - Block K/L
Stresses on Upstream Face at Base
Stress (kPa)
500
400
300
200
100
0
-100
-200
2.3
2.4
2.5
2.6
Concrete Mass (t/m3)
FSL (98.93)
Max Past Flood (106.55)
PMF (110.7)

# APPENDIX F

## Somerset Dam Dissipator Wall Stability

# CALCULATION SHEET

| | | | | | |
|---|---|---|---|---|---|
| CLIENT: | SEQWB | File No.: | 311/14019/00 | Sheet 1 of 3 | |
| PROJECT: | Somerset Dam | Calc. by: | cb | Date: 1-8-95 | |
| JOB No.: | 311/14019/00 | Checked by: | | Date: | |




Dissipator Retaining Wall

Check Section

| | | |
|---|---|---|
| CLIENT: | File No.: | Sheet 2 of 3 |
| PROJECT: | Calc. by: | Date: |
| JOB No.: | Checked by: | Date: |

Check design at RL 216

$$V = \frac{62.4 \times 7.315}{2} = 228 \text{ kN}$$

$$M = 228 \times \frac{7.315}{3} = 556.5 \text{ kNm}$$

$$\text{Mass of Concrete} = \frac{3.05 + 0.6}{2} \times 7.315 \times 2.4$$

$$= 32.04 \text{ tonne}$$

$$\text{Weight of Concrete} = 314.3 \text{ kN}$$

$$\text{C of G of concrete} = \frac{\frac{1.829 \times 7.315}{2} \times \frac{2}{3} \times 1.829 + 0.6 \times 7.315 \times 2.129 + \frac{0.621 \times 7.315}{2} \ldots}{13.35}$$

$$= \frac{16.314 + 9.344 + 11.974}{13.35}$$

$$= \frac{29.475}{13.35}$$

$$= 2.208$$

Stresses

$$\sigma = \frac{314.3}{3.05} \pm \frac{314.3 \times 0.683 \times 1.525}{2.364} \pm \frac{556.5 \times 1.525}{2.364}$$

$$= 103.05 \pm 138.48 \pm 358.99$$

$$\sigma_{\text{fill side}} = 103.05 + 138.48 - 358.99 = -117.5 \text{ kPa}$$

$$\sigma_{\text{water side}} = 103.05 - 138.48 + 358.99 = 323.6 \text{ kPa}$$

If tensile strength of concrete is zero

Location of resultant $= \frac{314.3 \times 2.208 - 556.5}{314.3} = 0.437 \text{ m}$

$e = 1.088 \text{ m}$

Crack 1.738m long

Maximum comp stress $= \frac{2 \times 314.3}{1.312} = 479.1 \text{ kPa}$

Check design at RL 200

$V = 62.4 \times 7.315/2 + \frac{62.4+79.8}{2} \times 1.2 + \frac{48 \times 4.877}{2} =$ 430.6 kN

$M = \frac{62.4 \times 7.315}{2}\left(4.877 + \frac{7.315}{3}\right) + 62.4 \times 1.2 \times (4.877 - 0.6)$

$+ \frac{(79.8-62.4) \times 1.2}{2} \times (4.877 - \tfrac{2}{3} \times 0.6) + \left(\frac{48 \times 4.877}{2}\right) \times 4.877/3$

$= 1669.6 + 320.3 + 46.7 + 190.3$

$= 2226.9$ kNm

Weight of concrete $= \left[\left(\frac{0.6+3.05}{2}\right) \times 7.315 + \left(\frac{3.05+4.269}{2}\right) 4.877\right] \times 2.4 \times 9.81$

$= (13.35 + 17.85) \times 23.54$

$= 734.57$ kN

C of G of Concrete $= \left[\frac{12.192 \times 3.048}{2} \times 3.048 \times 2/3 + 0.6 \times 12.192 \times (3.048 + 0.3)\right.$

$+ \frac{7.315 \times 0.621}{2} \times \left(3.648 + \frac{0.621}{3}\right)$

$\left. + 4.877 \times 0.621 \times (3.648 + 0.621/2)\right] / 31.205$

$= (37.76 + 24.49 + 8.76 + 11.99)/31.205$

$= 2.66$ m $\quad e = 0.5255$

$\sigma = \frac{734.57}{4.269} \pm \frac{734.57 \times 0.5255 \times 2.1345}{6.483} \pm \frac{2226.9 \times 2.1345}{6.483}$

$= 172.07 \pm 127.09 \pm 733.20$

$\sigma_{fill\ side} = -434.04$ kPa

$\sigma_{water\ side} = 778.18$ kPa

If tensile strength of concrete is zero

Location of resultant $= \frac{734.57 \times 2.66 - 2226.9}{734.57} = -0.37$

Resultant is outside base – wall is unstable under these loadings.

# APPENDIX G

# Definition of Overall Dam Safety Evaluation Terms

# EXTRACT FROM

USBR - Safety Evaluation of Existing Dams

A manual for the Safety Evaluation of Embankment and Concrete Dams

A Water Resources Technical Publication
Denver Colorado 1983

The terms satisfactory, fair, poor, and unsatisfactory are used in a general sense throughout the Examination Report describing the structural or the operational condition of the equipment; but, when they appear capitalised in the SEED Report they denote the overall classification of the dam as follows:

- SATISFACTORY

  No existing or potential dam safety deficiencies are recognised. Safe performance is expected under all anticipated loading conditions, including such events as the MCE (maximum credible earthquake) and the PMF (probable maximum flood).

- FAIR

  No existing dam safety deficiency is recognised for normal loading conditions. Infrequent hydrologic and/or seismic events would probably result in a dam safety deficiency.

- CONDITIONALLY POOR

  A potential dam safety deficiency is recognised for unusual loading conditions which may realistically occur during the expected life of the structure. CONDITIONALLY POOR may also be used when uncertainties exist as to critical analysis parameters which identify a potential dam safety deficiency; further investigations and studies are necessary.

- POOR

  A potential dam safety deficiency is clearly recognised for normal loading conditions. Immediate actions to resolve the deficiency are recommended; reservoir restrictions may be necessary until problem resolution.

- UNSATISFACTORY

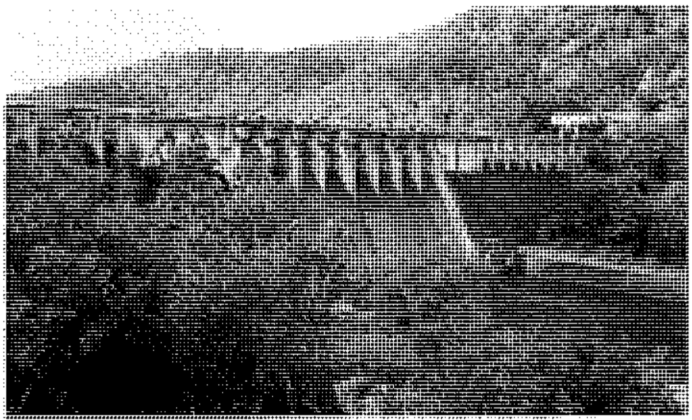
  A dam safety deficiency exists for normal conditions. Immediate remedial action is required for problem resolution.

BM-1: Document 33

# SEQWater

## SOMERSET DAM

## STABILITY OF ABUTMENT MONOLITHS



**Report No: DC 05099.**
**May, 2005**

NSW Department of Commerce

Level 13, 2 – 24 Rawson Place, Sydney NSW 2000

**Dams & Civil** **Water Technologies**

# EXECUTIVE SUMMARY

### *Background*

Somerset and Wivenhoe Dams are dual purpose storages for water supply and flood mitigation providing a safe water supply to Brisbane and adjacent Local Authorities and flood mitigation benefits for the Brisbane and Ipswich areas.

Modern design criteria for dams have changed considerably since the construction of these dams. The current requirements for flood handling capability and earthquake are more onerous and Wivenhoe in particular has inadequate spillway capacity.

A 2 stage upgrade program has been adopted to provide a major increase in Wivenhoe's flood handling capability. Stage 1, currently under construction by the Wivenhoe Alliance, will provide a spillway capacity to handle the 1 in 100,000 AEP (Annual Exceedance Probability) flood event. Stage 2 will provide a further increase in flood handling capacity to accommodate the Probable Maximum Flood.

Previous studies, GHD (2000) and SKM (2000), have judged that Somerset Dam is likely to fail under extreme flood or earthquake events. The most vulnerable areas are the upper levels of the abutment monoliths.

As a stand-alone structure, Somerset Dam poses a societal risk that is well below the ANCOLD Limit of Tolerability. The main concern from a dam safety perspective is whether failure of Somerset Dam could trigger a premature cascade failure of the downstream Wivenhoe Dam following Stage 1 works.

If Somerset Dam operations do not compromise the Wivenhoe Stage 1 works then a Somerset upgrade, if required at all, would reasonably attract the same degree of urgency as Stage 2 Wivenhoe works. If the 1 in 100,000 standard adopted for Wivenhoe was seriously compromised by Somerset Dam operations, then the upgrade could warrant more urgent action.

### *Previous Studies*

Previous studies, SKM (2000) and GHD (2000), provided stability analyses for Somerset while SMEC (2004) incorporated this work in a risk assessment study. The failure flood levels adopted by SMEC (2004) were:

- EL 109.7 for failure at the *"Change of Slope"* in the upper abutment monoliths;
- EL 110.0 for failure at the *"Upper Gallery"* level of the abutment monoliths.

SMEC (2000) noted that "the results from previous stability analyses are at odds" and that "the reasons for the differences are not apparent". In addition, the DPI (1994) Report quotes a Ben Russo conclusion that differs from both of these studies.

Studies by Commerce (2004) demonstrated that a 1 in 100,000 AEP flood event would produce storage levels of up to RL 109.75 in Somerset Dam. Failure of Somerset would produce Wivenhoe storage levels very close to embankment crest level but would not overtop.

Given the small margin for safety, the apparent conflict in previous studies and the concern with extensive cracking at the upper gallery, a decision was taken to review the stability of Somerset Dam at the two critical locations previously identified.

This Report summarises the results of the new stability analyses and comments on the key assumptions that produced the conflicting results in previous reports. It is intended that it be read in conjunction with Commerce (2004).

***Review of Stability Analyses for "Change of Slope" Section***

The Somerset Dam abutment monoliths comprise a wide range of layouts with different structural arrangements in the upper levels, different internal drainage arrangements, and different gallery layouts. A simple analysis using a standard section is suitable only for preliminary assessments.

Analyses showed that all monoliths satisfy stability criteria up to a storage level of RL 111.0, well above the critical level of RL 109.75. Some monoliths (G, H and Q) are more stable. Monolith R is somewhat less stable and approaches instability with a storage level of RL 110.6. However, this monolith would receive support from adjacent more stable monoliths.

The GHD (2000) analysis indicates failure of a typical section at RL 109.7. The major differences between it and the Commerce analyses are:

- GHD (2000) uses the monolith weights up to overflow level of RL 104.47. No allowance is included for the dead weight of the piers and hoist bridge;
- GHD (2000) slightly underestimates the downstream profile of the dam, producing a slightly smaller dead load and a slightly shorter base width;
- GHD (2000) makes no allowance for the weight of water on the crest during overtopping flows.

These additional dead loads make a considerable difference to the stability at high levels such as the *"Change of Slope"* section, where the mass concrete weights are relatively low. The impact of pier and bridge loads becomes less significant at lower levels in the dam. Insufficient detail was available from the SKM (2000) analysis for comparison.

***Review of Stability Analyses for Upper Gallery" Section***

Stability criteria for the "Upper-Gallery section" were satisfied with a storage level of RL109.75. While the concrete is cracked, known cracks emerge in and drain to the gallery, making this section as stable (in some respects more stable) at high storage levels than adjacent sound concrete. The abutment monoliths satisfy stability criteria for storage levels up to RL 110.9. Theoretical failure occurs in the sound concrete below the gallery at a storage level of RL 111.4.

The results provided by GHD (2000) and SKM (2000) indicate failure at lower levels of RL 110.7 and RL 110.5 respectively. The GHD (2000) results are partly due to the same factors noted above. In addition, the current analysis recognises uplift reduction from internal drainage upstream of the gallery that was not included in the GHD analysis. SKM (2000) is likely to have used a similar approach to GHD.

It is possible that cracking also exists above or below the gallery. This is a far more serious situation in that such cracks cannot drain to the gallery. If these conditions exist, the dam just satisfies stability criteria for a storage level of RL 109.7. Conditions rapidly deteriorate with higher storage levels and failure could occur with a storage level as low as RL 110.1.

***Stability Assessment***

Somerset Dam, on the basis of its known condition, satisfies stability criteria for a storage level of RL 109.75 and will safely handle the 1 in 100,000 AEP flood event. This in turn ensures that the Stage 1 upgrade works for Wivenhoe Dam are not compromised by any Somerset Dam deficiencies.

On this basis upgrade work at Somerset Dam, if required at all, would reasonably attract the same degree of urgency as Stage 2 Wivenhoe works. It is recommended that any upgrading of Somerset Dam be considered at the time that Stage 2 Wivenhoe works are assessed.

?

There is concern that cracking observed in the Upper Gallery walls may also exist above or below the Gallery. While such cracked concrete would just satisfy stability criteria for a storage level of RL 109.75, stability reduces rapidly for higher storage levels and failure could occur at RL 110.1. It is recommended that some exploratory drilling be carried out to determine whether such cracks do exist. A similar recommendation was made in GHD (2000).

***Risk Assessment***

The stability analyses summarised above demonstrate that the risk profile developed by SMEC (2004) makes conservative assumptions for structural failure in the upper levels of the abutment monoliths. On this basis, there is no need for further development of the Somerset Dam risk profile as a stand alone document.

However, the current risk analysis for Wivenhoe Dam is a modification of the Preliminary Risk Assessment produced by SKM and uses the SKM loss of life data. The Wivenhoe/Somerset combined risk profile is a borderline case in terms of the ANCOLD criteria. Given the importance of these dams and the flood mitigation benefits provided to Brisbane and adjacent areas, consideration should be given to a detailed assessment of the combined risk profile. This would require among other assessment work, new determinations for consequences, particularly the loss of life figures.

***Other Considerations***

If the WIVOPS flood operation program still requires that the Somerset spillway gates be lowered if Wivenhoe Dam is in danger of being overtopped, then this Report should be reviewed and the spillway examined in detail to ensure these operations can be undertaken successfully. This type of gate operation is not recommended.

This Report assumes that the gallery systems are not flooded by water overtopping the abutment monoliths. The dam layout should be reviewed to ensure this is the case and waterproof doors installed where necessary.

The internal drainage system is complicated, particularly at the lower levels of the dam. It is recommended that the drainage system be documented, critical drains be monitored and maintenance carried out where necessary.

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## TABLES



## FIGURES

# 1 INTRODUCTION

## 1.1 Somerset Dam

Somerset Dam is a 47m high concrete gravity dam on the Stanley River upstream of Wivenhoe Dam. Both are dual purpose dams for water supply and flood mitigation providing a safe water supply to Brisbane and adjacent Local Authorities and flood mitigation benefits for the Brisbane and Ipswich areas.

Modern design criteria for dams have changed considerably since the construction of these storages. The current requirements for flood handling capability and earthquake are more onerous and Wivenhoe in particular has inadequate spillway capacity.

Previous studies, GHD (2000) and SKM (2000), have judged that Somerset Dam is likely to fail under extreme flood or earthquake events. The most vulnerable areas are the upper levels of the abutment monoliths.

However, as a stand-alone structure, Somerset Dam poses a societal risk that is well below the ANCOLD Limit of Tolerability. The main concern from a dam safety perspective is whether failure of Somerset Dam could trigger a premature cascade failure of the downstream Wivenhoe Dam. Extreme earthquake loadings have not been considered here, as the risk analysis results plot well within the ANCOLD Limit of Tolerability and Wivenhoe Dam has sufficient capacity to absorb a Somerset sunny day failure without initiating the fuse plug embankments.

## 1.2 Previous Studies

Three reviews of Somerset Dam have been carried out by various consultants for SEQWater in recent years:

- SKM (2000) provided a Preliminary Risk Assessment for Wivenhoe, Somerset and North Pine Dams that assessed the failure modes and consequence of failure for all three dams, produced risk profiles and outlined risk reduction options;
- GHD (2000) provided a detailed dam safety review of Somerset dam, covering the hydraulic, geotechnical and structural evaluations of the dam;
- SMEC (2004) provided a detailed risk assessment for Somerset Dam using the data provided in the two earlier reports together with new geological and other investigation studies. It also included the results of a hydrology analysis undertaken by the Wivenhoe Alliance.

While the SMEC (2004) Report provided an assessment of Somerset Dam in isolation, it did not address the cascade failure issue in detail. Upgrade studies for Wivenhoe Dam were under development by the Wivenhoe Alliance at the time this Report was produced and final details were not available to SMEC.

A subsequent review of Somerset Dam, Commerce (2004), identified the impact of a Somerset Dam failure on Wivenhoe Dam after completion of the Stage 1 Upgrading and made recommendations on future actions. It included additional hydrological and hydraulic studies to supplement the data provided above and expanded the risk assessment for Wivenhoe to include the risks associated with a Somerset failure.

### 1.3 Impact of Somerset Dam on Wivenhoe Dam

Wivenhoe Dam is a 56 m high, zoned earth embankment with a concrete gravity spillway, and a 2 stage upgrade program has been adopted to provide a major increase in flood handling capability. Stage 1, currently under construction by the Wivenhoe Alliance, will provide a spillway capacity to handle the 1 in 100,000 AEP flood event. Stage 2 will provide a further increase in flood handling capacity to accommodate the Probable Maximum Flood.

If the risks associated with a Somerset failure do not compromise the standards adopted for Stage 1 works at Wivenhoe then a Somerset upgrade, if required at all, would reasonably attract the same degree of urgency as Stage 2 Wivenhoe works. If the 1 in 100,000 standard adopted for Wivenhoe was seriously compromised by Somerset Dam risks, then the upgrade could warrant more urgent action.

Studies by Commerce (2004) demonstrated that a 1 in 100,000 AEP flood event would produce storage levels of up to RL 109.75 in Somerset Dam. Failure of Somerset Dam at these storage levels would produce storage levels at Wivenhoe Dam very close to the embankment crest level but would not trigger a cascade failure of Wivenhoe Dam.

The critical flood levels adopted for previous risk analysis studies, SMEC (2004), were:

- EL 109.7 for the Change of Slope failure;
- EL 110.0 for the Upper Gallery failure.

These levels adopted by SMEC (2004) were based on separate stability analyses by GHD (2000) and SKM (2000). SMEC (2004) noted that "the results from the two analyses are at odds" and that "the reasons for the differences are not apparent". In addition, the DPI (1994) Report quotes a Ben Russo conclusion that differs from both of these studies.

Given the small margin for safety, the apparent conflict in previous studies and the concern with extensive cracking at the upper gallery, a decision was taken to review the stability of Somerset Dam at the two critical locations.

This Report summarises the results of this work and comments on the key assumptions that produced the conflicting results in previous reports. It is intended that it be read in conjunction with Commerce (2004).

### 1.4 Stability Analyses

The ANCOLD Guidelines are based on a limit state design method that uses load factors. In practice this has proven to cause some difficulties, particularly when considering existing dams that may be just satisfactory or have marginal stability. The ANCOLD guideline is under review, and is generally not used to analyse existing dams in Australia.

The stability analyses carried out for this Report use the traditional working stress approach in accordance with international practice. This is consistent with the approach taken by both GHD (2000) and SKM (2000). The Canadian Dam Safety Association (CDSA) "Dam Safety Guidelines", 1999 have been adopted unless otherwise noted.

This Report deals only with flood loadings with a very low probability of occurrence, typically an AEP of around 1 in 100,000 or more. It is accepted that the structure may be damaged under these loadings but that there will be no uncontrolled loss of reservoir storage. They are treated as "Unusual" or "Extreme Loads" with a low factor of safety under CDSA (1999) and other standards.

## 1.5 Limitations

This Report makes the following assumptions:

- Somerset Dam will not be surcharged during extreme flood events if Wivenhoe Dam is in danger of being overtopped, as was recommended in the flood operations program;
- The gallery system is not flooded by overtopping flows;
- The internal drainage system is maintained in good condition.

**Figure 1-1 - Somerset Dam General Arrangement**

# 2 DATA & ASSUMPTIONS

## 2.1 Somerset Dam

Somerset Dam is a 47m high concrete gravity dam with a central gated overflow spillway, controlled by 8 radial gates and 8 low level sluice gates. Full Supply Level (FSL) is located at EL 98.93, some 1.52 m below the spillway fixed crest and the gates are used only for flood control purposes. There are 4 low-level outlets through the abutment units and a pipeline leading to the power station. Water is released as required from Somerset Dam to supplement the downstream Wivenhoe Dam.

There are 8 mass concrete abutment units on each side of the central spillway structure supporting a road bridge at EL 112.34. Five abutment units on each side are constructed with an open overflow section below the bridge at EL 107.46. Flood water discharging through these openings flows down the back face of the dam and impacts on an unprotected rock foundation, before flowing laterally towards the central spillway channel.

The concrete dam is a conventional mass concrete construction with upstream slopes of 0.05H:1V and downstream slopes of 0.7H:1V in the central overflow section and 0.75H:1V in the abutment units. There is an abrupt "change of slope" above FSL in the abutment units that provides a constant width of nominally 4.3 m in the top section. This "change of slope" discontinuity provides a critical section for dam stability.

Two drainage galleries are provided in the dam at EL 88.6 and EL 66.0. Extensive concrete cracking has occurred at the Upper Gallery providing the second critical section for dam stability.

Recent geological investigation studies (SMEC 2004) recorded the foundations to be generally slightly weathered and assessed visually to be of very high strength and high durability, showing no signs of significant degradation or weathering upon exposure. The dam was excavated into high strength, tight rock and while erosion of near surface materials below the dam could be expected under low to medium flows, the rock mass was tight at depth and was judged to have a high resistance to erosion.

## 2.2 The Abutment Monoliths

### *2.2.1 General*

There are 5 major abutment monoliths and 3 smaller units on each side of the gated spillway structure:

- Smaller monoliths are A, B and C on the far right abutment and V, W and X on the far left abutment;
- Major monoliths are D, E, F G and H on the right abutment and U, T, S, R and Q on the left abutment;

- The two monoliths either side of the central spillway structure, H and Q, are full height units founded at river bed level and extending to road level with no overflow facility. They each contain two low level outlets and associated gate shafts;
- The other monoliths are founded on the abutment slopes with higher foundation levels. Monoliths G and R are predominantly full height monoliths with short overflow sections. Monolith G contains the outlet to the power house and a singe gate shaft;
- Monoliths B to F and S to W have overflow sections that permit overtopping when the storage rises above RL 107.46;
- The widths of the major monoliths are generally 15,85m with Monoliths H and Q slightly longer.

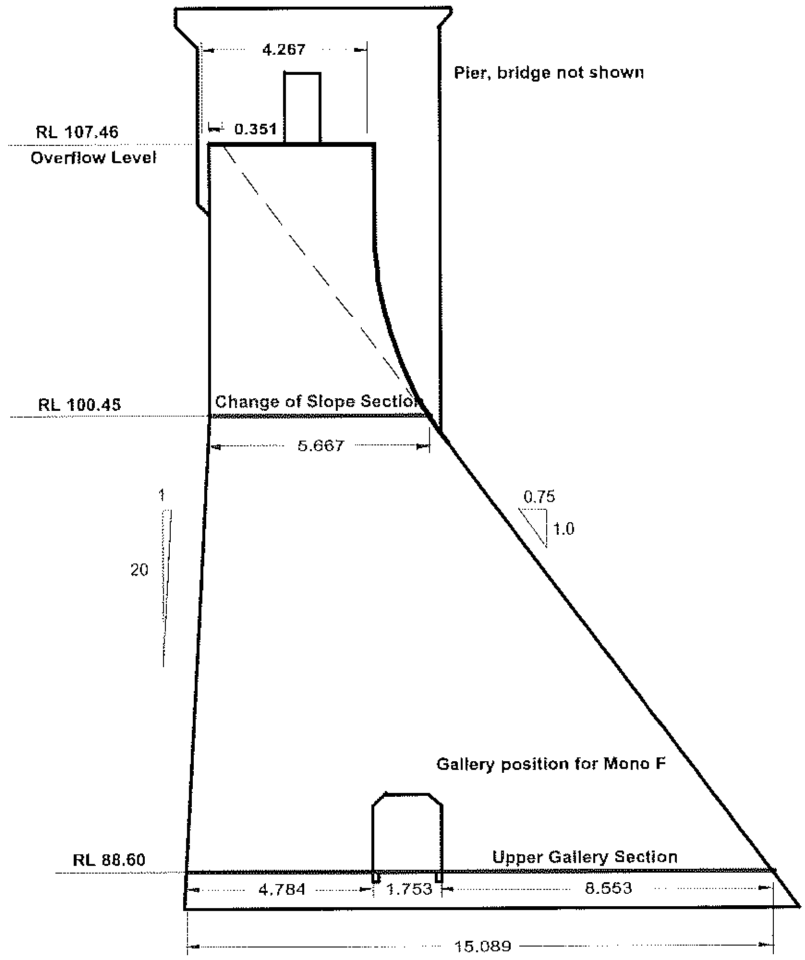



**Figure 2-1 - Somerset Abutment Unit**

Key dimensions for the abutment units are provided at **Table 2-1** and Figure 2-1, while monolith layouts affecting the two critical failure levels are summarised at Table 2-2.

**Table 2-1 - Somerset Abutment Monoliths**

| Item | Change of Slope | Upper Gallery |
|---|---|---|
| Full Supply Level | RL 98.93 | RL 98.93 |
| 1974 Flood of Record | RL 106.55 | RL 106.55 |
| Base Level for Analysis | RL 100.45 | RL 88.60 |
| Monolith Crest overflow level | RL 107.46 | RL 107.46 |
| Height of Block to overflow level | 7.01 m | 18.86 m |
| Base Width | 5.666 m | 15.09 m |
| D/S Slope | Located on curved transition | 0.75H:1V |
| U/S Slope | Vertical | 1 H:20V |
| Distance to drains from u/s face | Vary see Table 2-2 | Vary see Table 2-2 |
| Tailwater Level | N.A. – below base | N.A. – below base |

There is considerable variation in the layout of individual monoliths with both gallery layouts and internal drainage facilities varying across the dam. The stability of the abutment monoliths cannot be assessed from one standard section.

Other features that affect Monoliths G, H and Q are gate shafts located 0.91 m (3 feet) downstream of the vertical water face. The concrete upstream of the shafts is reinforced, providing additional tensile strength. The shafts provide extensive drainage capacity and it is difficult to see any significant uplift developing downstream of the shafts. The low uplift and the upstream reinforcement make these units more stable than the others and stability analyses have concentrated on the smaller monoliths.

### *2.2.2 The "Change in Slope"*

The change of slope in the upper abutments occurs at RL 100.45 on the upstream face. The downstream face in this area has a curved profile that is tangent to the 0.75H:1V sloping backface at RL 99.81 and tangent to the vertical backface at RL 105.29. For the purpose of this Report, the level for the "change in slope" analysis is taken as RL 100.45.

GHD (2000) appear to have also used RL 100.45 while. SKM (2000) nominate RL 99.19 as the "neck level".

The concrete appears to be in good condition and there are no indications of deterioration or other problems in reports by the previous consultants.

Monoliths D, E and F on the right abutment and S, T and U on the left abutment have similar layouts referred to in this Report as the ***typical section***. Drainage in the form of cored holes is provided at distances of 2.1 m to 2.5 m from the upstream face.

Monoliths G, H, Q and R are full height units with a different profile, different dead loads and different internal drainage arrangements. Some of the important variations are summarised at Table 2-2.

### *2.2.3 The "Upper Gallery"*

The Upper Gallery is 2.03 m high and 1.75 m wide with the floor level typically at RL 88.6. In the higher monoliths on the right hand side (G and H) the upstream face of the gallery is located 4.8m from the water face. The gallery layout becomes more complicated in the next two smaller monoliths, with the Upper Gallery moving further downstream to permit a stairway to the lower gallery between it and the upstream face and an access from the downstream face.

The internal drainage in the larger blocks such as G and H is located in blockouts in the downstream wall of the gallery. These cored holes emerge at roof level and are cored from floor level to the upstream face of the Lower Gallery. This places the drains at the Upper Gallery level some 6.77 m from the upstream water face. However, gate shafts upstream of these drainage holes would have a far larger impact on uplift.

Monoliths F and the right hand end of G have the internal drainage located on the upstream wall of the stairway leading to the Lower Gallery. This drainage location varies from 2.65m to 3.14m from the water face of the dam. Drainage in Monolith E is located at the upstream side of a short extension gallery with holes at 3.14 m from the upstream face.

All drainage consists of 150 mm diameter cored holes, generally at 3 m centres, although the spacing varies in some areas. The left hand abutment has a roughly similar layout but with variations in some monoliths.

### *2.2.4 Cracking of Concrete in the Upper Gallery*

There is considerable horizontal cracking exposed in the gallery walls, presumably from temperature and shrinkage effects. The main cracks are located on the downstream side of the gallery wall, one about 0.4 m above floor level and the other 1.6 m to 1.8 m above floor level. The latter crack extends for most of the length of the gallery and appears to be at the same level as a construction joint in the downstream face of the dam. Cracks can also be seen extending to the downstream face in the two access adits at each end of the dam.

Horizontal hairline cracking can also be seen in the upstream gallery wall and in the stairways to the lower gallery. In one spillway monolith the crack emerges in the upstream face of the gate shaft and there has been long term leakage. There is no indication of leakage elsewhere in the Upper Gallery.

Investigation work by SMEC included horizontally drilled holes into the downstream gallery wall. There was some difficulty in following the cracks with horizontal holes as the cracks deviated around 50 mm along the drilled length. The surface of the cracks was irregular and rough. Drilling water returned along the crack for 0.5m either side of the borehole collar.

**Table 2-2 - Abutment Monolith Layouts**

| Monolith | General Description | Change of Slope at RL 100.45 | Upper Gallery at RL 88.6 |
|---|---|---|---|
| **A & X** | Small monument at end of each abutment | Unit above RL 100.45 | Unit above Upper Gallery |
| **B & W** | Small monument with overflow capacity | Unit above RL 100.45 | Unit above Upper Gallery |
| **C $ V** | Small monument with overflow capacity | Half Unit above RL 100.45 | Unit above Upper Gallery |
| **D & U** | Main abutment units with overflow capacity | Typical cross section for overflow unit Internal drains at 2.5 m from water face. | Unit above Upper Gallery |
| **E & T** | Main abutment units with overflow capacity; short Upper Gallery at 2.3 m from u/s face, d/s access to upper gallery | Typical cross section for overflow unit Internal drains at 2.5 m from water face | Drainage from short gallery 3.1 m d/s of water face. |
| **F & S** | Main abutment units with overflow capacity; Main Upper Gallery angled diagonally across unit, u/s stair access to Lower Gallery near water face. | Typical cross section for overflow unit Internal drains at 2,1 to 2.5 m from water face | Drainage from stair access at average of 2.7 to 3.1m from water face. The Main Upper Gallery will intercept known cracks at 4.8 to 7.5m from the water face. |
| **G** | Main abutment unit on right side of spillway, full height unit for 70% width, overflow capacity over right hand side for 3.58 m.<br>Contains conduit to power station and a single gate shaft | Gate shaft prevents extensive uplift development and reinforcement u/s of gate shaft provides tensile capability preventing crack development | Gate shaft prevents extensive uplift development and reinforcement u/s of gate shaft provides tensile capability preventing crack development |
| **R** | Main abutment unit on left side of spillway. Similar to G without the conduit, gate shaft or reinforcement in water face | Different dead loads and horizontal water load to typical section. Internal drains at 4.6 m from water face | Main Upper Gallery will intercept known cracks at 4.8 m from water face. Drains above and below gallery at 5.2 m from water face... |
| **H & Q** | Full height units, founded at river bed, contain low level outlets and 2 * 1.52 m wide gate shafts | Gate shaft prevents extensive uplift development and reinforcement u/s of gate shaft provides tensile capability preventing crack development | Gate shaft prevents extensive uplift development and reinforcement u/s of gate shaft provides tensile capability preventing crack development |

The drilling showed the cracks were open for at least 1 to 2 m from the downstream face of the gallery. At some stage it reduces to a hairline crack that appears to extend to the downstream face, as seen in the access adits

It appears that the concrete is arching across the gallery and downstream cracked area and supported by concrete near the upstream face and downstream face. Bearing pressures calculated in this area are therefore nominal values only. Downstream bearing pressures under high water loads may be substantially higher than those calculated. However the height of the dam at the Upper Gallery level is not sufficient to produce bearing pressures approaching anywhere near the strength of the concrete, even if increased by a factor of 2 or 3.

On the evidence available, it is assumed that a crack exists across the full width and length of the monolith blocks with a far more prominent crack for at least 2 metres downstream from the gallery. If the dam is subjected to unprecedented water levels, it is reasonable to assume that the upstream cracks could develop significant uplift pressures.

A similar assumption was made by GHD (2000). SKM (2000) took the view that continuous cracking was a conservative assumption but accepted it for stability analyses.

A critical unknown is whether cracking exists above or below the gallery. Cracks that emerge in the gallery walls will be drained by the gallery and are not necessarily a significant stability problem. If similar cracks exist above or below the gallery, these become a plane of weakness with uplift relieved only by the internal drains. Russo (1996) mentions cracking has been observed at RL 95.3 and RL 97.2.

## 2.3 Concrete Properties

### *2.3.1 General*

The material properties adopted for this Report are summarised at Table 2-3. They are consistent with properties used by the previous three consultants wherever possible and are also consistent with internationally accepted standards

The critical parameters for this study are the allowable tensile stress and the shear strength parameters for the cracked concrete.

### *2.3.2 Tensile Strength Parameters*

Research by UNSW (Khabbaz & Fell (1999)) report typical tensile strengths for 20 MPa concrete of 1,100 kPa and the 95% lower confidence limit of 200 kPa, the latter being the source of the SKM assumption. ANCOLD recommend a tensile strength of $\underline{0.2^* \sqrt{f'_c}}$ (900 kPa for 20 MPa concrete) and an allowable tensile stress of 10% of this value to allow for stress concentrations at the crack. This is the source of the 90 kPa adopted by GHD.

**Table 2-3 - Material Properties**

| Property | GHD (2000) | SKM (2000) | Value Adopted | Notes |
|---|---|---|---|---|
| Concrete Weight | 23.5 kN/m$^3$ | 23.5 kN/m$^3$ | 23.5 kN/m$^3$ | GHD refer to test results in 1987 & 1999 |
| Concrete Compressive Strength | 40 MPa | 20 MPa | 20 MPa | GHD refer to 1999 core testing, but used 20 MPa for tensile strength calc |
| Concrete Friction Strength, cracked & uncracked | 45 deg | 45 deg | 45 deg | ANCOLD |
| Concrete Cohesion, uncracked | 1,400 kPa | 1,400 kPa | 1,400 kPa | ANCOLD |
| Concrete Cohesion, cracked concrete at gallery | 100 kPa | 100 kPa | 100 kPa | Assumed |
| Concrete Tensile Strength uncracked concrete | 90 kPa | 200 kPa | 90 kPa | ANCOLD |
| Concrete Tensile Strength cracked concrete | 0 kPa | 0 kPa | 0 kPa | Normal practice |

SMEC (2004) in its risk assessment workshop adopted a tensile strength of 200 kPa, while recognising that it "could vary, from effectively zero for a cracked section, to possibly greater than 1 MPa for an intact section.

The conservative 90 kPa value for sound concrete adopted by GHD has been used here for consistency, with discussion of higher values where appropriate. For cracked concrete the tensile strength is taken as zero.

### *2.3.3 Shear Strength*

The shear strength parameters adopted for mass concrete are cohesion of 1,400 kPa and friction angle of 1.0. These are ANCOLD recommendations for concrete of uncertain quality in existing dams. For cracked concrete, the cohesion is reduced to 100 kPa on the basis that the crack is rough with some aggregate interlock.

These assumptions are consistent with previous work by GHD and SKM.

## 2.4 Uplift Assumptions

### 2.4.1 *"Change in Slope"*

At the "change in slope", the drainage holes are located between 2.1 to 4.6 m from the upstream face. A typical drainage location of 3.2 m from the upstream face was adopted in line with GHD (2000) although this is a little conservative and 2.5m would have been more appropriate. A separate analysis for Monolith R used a distance of 4.6 m.

Cracked analyses adopted uplift pressures as recommended in ANCOLD (1991) and shown at Figure 2-2 (with 0.33H replaced by 0.25H). For normal drained sections, the pressure at the line of drains was assumed to be 25% of full storage head in view of the good quality concrete and the location well above full supply level. For cracked analyses where the non-compression area passes the line of drains, the uplift was assumed to be 85% of full storage head. Both figures are considered conservative for sound concrete but reasonable for cracked concrete.

### 2.4.2 *"Upper Gallery*

At the "Upper Gallery" at RL 88.6, the known cracks are intersected by internal drainage holes and then emerge in the Upper Gallery. Uplift cannot bypass the gallery and zero uplift was assumed for cracks downstream of the gallery.

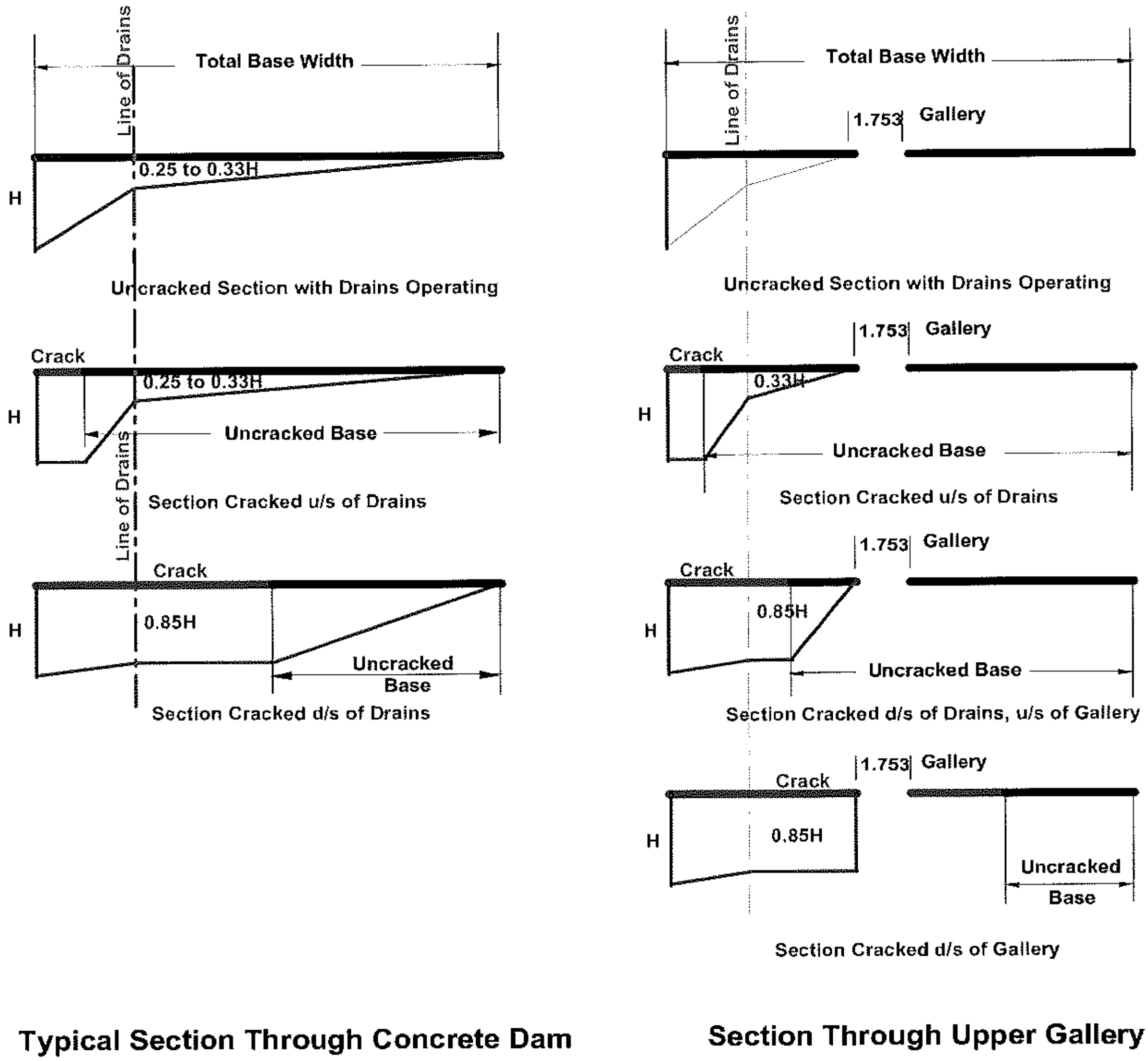
Upstream of the gallery, uplift in cracks was assumed to vary from full storage pressure at the water face, to 33% of storage head at the line of drains and then reduce to zero at the upstream gallery wall. If non-compression areas developed upstream of the drains, they were assumed to attract full storage head. Where the non-compression area passes the line of drains, the uplift was adjusted as shown at Figure 2-2.

The drainage effect provided by the gallery can make horizontal planes more stable than planes above or below the gallery. Additional analyses were carried out for:

- Planes below the gallery in good quality concrete;
- Planes below the gallery in cracked concrete.

For these planes, conventional drainage assumptions were made as recommended in ANCOLD (1991) and shown at Figure 2-2. For normal drained sections, the pressure at the line of drains was assumed to be 33% of full storage head. If non-compression areas developed upstream of the drains, they were assumed to attract full storage head. Where the non-compression area passes the line of drains, the uplift was adjusted as shown at Figure 2-2. Both figures are considered conservative for sound concrete but reasonable for cracked concrete.

## Figure 2-2 - Typical Uplift Assumptions



## 3 STABILITY AT CHANGE OF SLOPE; RL 100.45

### 3.1 Stability with Storage Level at RL 109.7

#### *3.1.1 Current Analysis*

The storage level of RL 109.75 is the critical level required to handle the 1 in 100,000 AEP flood event and ensure that the Stage 1 works at Wivenhoe are not comprised. The results of the stability analysis are provided at for the typical section representing Monoliths D, E and F on the right abutment and S, T and U on the left abutment. As noted above, Monoliths G, H and Q are relatively more stable than the above.

Separate results are given for Monolith R, which as a full height monolith over most of its width attracts additional horizontal water load. It has a higher dead load but the drains being located further downstream offset this.

**Table 3-1 - "Change of Slope" Stability for Storage Level of RL 109.7**

| Analysis | Parameter | Criteria | Typical Section | Monolith R |
|---|---|---|---|---|
| **Uncracked Section Analysis Sound Concrete** | U/S Bearing Stress | Tension < 90 kPa | 47 kPa tension | 102 kPa tension |
| | D/S Bearing Stress | Compression < 9 MPa | 303 kPa compression | 420 kPa compression |
| | Resultant Location*<br>% of Base | ***<br>*** | 1.54 m<br>27% | 1.28 m<br>22.6% |
| | Sliding Factor | > 2 | 21.8 | 21.3 |
| | | | | |
| **Cracked Analysis for zero tensile strength** | Crack Length<br>Crack Length as %age of Base ** | <75% | 1.57<br>28 % | 2.45<br>43 % |
| | D/S Bearing Stress | Compression < 9 MPa | 329 kPa compression | 508 kPa compression |
| | Resultant Location*<br>% of Base | *** | 1. 36<br>24% | 1.07<br>19 % |
| | Sliding Factor | > 2 | 16.1 | 12.9 |

* Resultant location is the distance the resultant force acts from the d/s toe

** There is no consistent agreement on maximum crack lengths. A maximum crack length of 75% of the base length is adopted.

The results are given for a storage level of RL 109.7 to correspond with previous work. Comments and conclusions also apply to a storage level of RL 109.75.

Stability analyses for the ***typical section*** give a tensile stress of 47 kPa for this water level. This is not a particularly high tensile stress and should be well within the tensile strength of the concrete. The sliding factor is well above requirements and the compressive stress is low, as would be expected for the height of structure available.

If the section is allowed to crack (zero tensile strength), the crack would extend for some 1.6 m equivalent to 28% of the base width.

These results would satisfy international criteria for an extreme flood event.

Stability analyses for the ***Monolith R*** give a tensile stress of 102 kPa for this water level, substantially more than for the critical section. The sliding factor is still well above requirements and the compressive stress is low. The cracked analysis produces a crack length of 2.45m or 43% of the base width.

This is still a satisfactory result at this water level. Monolith R would also benefit from restraint from the adjacent more stable monoliths.

## 3.2 Stability at Higher Storage Levels

A storage level of RL 110.7 produces tensile stresses of 98 kPa in the ***typical section***. The cracked analysis produces a crack length of 3.5m (62% of base width) but still has a high sliding factor of 7.5. Stability reduces rapidly as the water level increases and the crack extends to 75% of the base width at a storage level of RL 111.0 with theoretical failure occurring at a storage level of RL 111.6.

Tensile stresses are still low with the storage at RL 111.6 however (148 kPa tension), the tensile strength of 90 kPa is conservative, and it is possible that cracking would not occur. A storage level of RL 112 produces tensile stresses of 168 kPa, still below the 200 kPa tensile strength used by SKM (2000) and SMEC (2004). At this storage level, the water is likely to impact on the bridges and increase the horizontal water loading. No attempt has been made to analyse this scenario.

Monolith R is less stable but satisfies stability criteria for storage levels up to RL 110.6 with theoretical failure occurring at a storage level of RL 111.1. Tensile stresses of 200 kPa develop at a storage level of RL 110.8.

Overall, the abutment monoliths satisfy stability criteria for storage levels up to RL 111.0. Previous flood data for Somerset Dam indicates a 1 in 200,000 AEP for a storage level of RL 111.0 (all gates operating). This data is based on flood operational rules prior to the Wivenhoe Upgrade (as given in the WIVOPS program). Revised estimates with new flood operating rules would reduce the probability.

## 3.3 Comparison with Previous Reports

GHD (2000) results indicate a far less stable situation with failure of the ***typical section*** at a storage level of RL 109.7. For comparison, a storage level of 110.7 produces tensile stresses of 179 kPa against the 98 kPa tension for the current analysis.

The GHD (2000) analysis applies only to the ***typical section*** and no analysis was carried out for Monolith R. The major differences between the two analyses are that GHD (2000):

- Uses the monolith weights up to overflow level of RL 104.47. No allowance is included for the dead weight of the piers and hoist bridge;
- Slightly underestimates the downstream profile of the dam, producing a slightly smaller dead load and a slightly shorter base width;
- Makes no allowance for the weight of water on the crest during overtopping flows.

These additional dead loads make a considerable difference to the stability at high levels such as the change of slope, where the mass concrete weights are relatively low. The impact of pier and bridge loads becomes less significant at lower levels in the dam.

The GHD stresses have been reproduced by using GHD parameters with the current model. The GHD (2000) analysis is a preliminary analysis that under-estimates the stability of the upper abutment units.

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# 4 STABILTY AT UPPER GALLERY; RL 88.6

## 4.1 Stability Analyses

The basic analysis for this area of the dam, ***Case A***, assumes continuous cracks exist across the full width of the monoliths with uplift pressures as described in Section 2.4. Two additional analyses have been carried out:

- ***Case B*** assumes sound concrete above or below the gallery with internal drainage from the drainage holes, not from the gallery
- ***Case C*** assumes cracked concrete above or below the gallery with internal drainage from the drainage holes, not from the gallery.

Analyses were carried out for Monolith F using the mid-unit location of the upper gallery. Other monoliths such as E, R and S have the gallery located a little closer to the upstream face and would be slightly more stable. The small monoliths are founded above the Upper Gallery level.

As noted previously, Monoliths G, H and Q are more stable due to drainage provided by the gate shafts and strength provided by water face reinforcement.

## 4.2 Stability at Storage Level of RL 109.7

The results of analyses are summarised at Table 4-1.

Cracked concrete that crosses the Upper Gallery (Case A) is stable with compression stresses across the full width of the dam. Compression stresses at the downstream toe are low at 454 kPa and the sliding factor of 2.1 is just adequate. The stability comes from the gallery draining cracked concrete that emerges in the upstream wall and from the absence of uplift pressures in the major cracks downstream of the gallery. The results satisfy stability criteria for an extreme flood loading.

For comparison purposes, sound concrete just below the gallery (Case B) would experience small tensile stresses of 10 kPa at the upstream face for a storage level of RL 109.7. The sliding factor for the intact concrete however is much larger at 11.2.

If additional cracked concrete exists just above or below the gallery (Case C), the small tensile stress of 10 kPa at the upstream face would permit full storage head to penetrate the cracks and the non-compression area would extend for 1.1 m past the upstream concrete face. The sliding factor of 1.99 is adequate and Case C would also satisfy stability criteria for an extreme flood loading.

**Table 4-1 - Upper Gallery Stability for Storage at RL 109.7**

| Analysis | Parameter | Criteria | Monolith F | | |
|---|---|---|---|---|---|
| | | | Case A | Case B | Case C |
| **Uncracked Analysis** | U/S Bearing Stress | Tension < 90 kPa | 49 kPa compression | 10 kPa tension | 10 kPa tension |
| | D/S Bearing Stress | Compression < 9 MPa | 448 kPa compression | 405 kPa compression | 405 kPa compression |
| | Resultant Location<br>% of Base | <br>*** | 5.22<br>35% | 4.91 m<br>33% | 4.91 m<br>33% |
| | Sliding Factor | > 2 | 2.16 | 11.2 | 2.08 |
| | | | | | |
| **Cracked Analysis for zero tensile strength** | Crack Length<br>Crack Length as %age of Base ** | <br>75% | No crack | 1.1 m<br>7% | 1.1 m<br>7% |
| | D/S Bearing Stress | Compression < 9 MPa | | 414 kPa compression | 414 kPa compression |
| | Resultant Location<br>% of Base | <br>*** | | 4.67<br>31% | 4.67<br>31% |
| | Sliding Factor | > 2 | | 10.4 | 1.99 |

* Resultant location is the distance the resultant force acts from the d/s toe

** There is no consistent agreement on maximum crack lengths. A maximum crack length of 75% of the base length is adopted.

## 4.3 Stability at Higher Storage Levels

### *4.3.1 Case A: Cracked Concrete Surface through the Gallery*

At a storage level of RL 110.8, tensile stresses begin to develop at the upstream face. The non-compression area permits full storage head to enter the cracked concrete and the sliding factor drops below the criteria value of 2.0 with a storage head at RL 110.9

With the storage level at RL 111.5 the non-compression extends to the gallery and the sliding factor reduces to 1.36. A storage level of RL 112 cracks the section for 58% of the base width and the shear factor reduces to 1.27. At this stage the monolith is depending on the 100 kPa cohesion for stability. The theoretical failure occurs at a storage level above RL 112.

### *4.3.2 Case B: Sound Concrete below the Gallery*

A storage level of RL 111.4 would produce tensile stresses of 90 kPa. If the sound concrete cracked under this stress, the crack would proceed to failure. The stability is highly dependent on the tensile strength of the concrete and this is a conservatively low estimate of tensile stress.

### *4.3.3 Case C: Cracked Concrete Above or Below the Gallery*

Increasing the storage level produces a rapid reduction in the sliding factor with a sliding failure occurring at a storage level of RL 110.1.

## 4.4 Conclusion and Comparison with Previous Reports

Stability criteria for the "Upper-Gallery section" were satisfied with a storage level of RL109.75. While the concrete is cracked, known cracks emerge in and drain to the gallery, making this section as stable (in some respects more stable) at high storage levels than adjacent sound concrete. The abutment monoliths satisfy stability criteria for storage levels up to RL 110.9. Theoretical failure occurs in the sound concrete below the gallery at a storage level of RL 111.4.

If cracked concrete is present above or below the gallery, the stability reduces rapidly as the storage level rises above RL 109.7 with theoretical failure at a storage level as low as RL 110.1.

GHD (2000) does not give a storage level for failure (as this would have been above the PMF level current at that time) but provides stability results for a storage level of 110.7. At this level, the tensile stress for an uncracked analysis is given as 121 kPa tension, compared with 49 kPa tension from the current analysis. GHD (2000) indicates the non-compression area would extend to within 2.7m of the downstream toe (equivalent to roughly 12.3m from the upstream face and the section would be stable for sliding.

The difference between the two analyses is that GHD (2000) analysis:

- uses the monolith weights up to overflow level of RL 104.47. No allowance is included for the dead weight of the piers and hoist bridge;
- slightly underestimates the downstream profile of the dam, producing a slightly smaller dead load and a slightly shorter base width;
- makes no allowance for the weight of water on the crest during overtopping flows.

In addition, the current analysis recognises uplift reduction from internal drainage upstream of the gallery that was not included in the GHD analysis. SKM (2000) is likely to have used a similar approach to GHD.

Russo (1996) reports that a check analysis of a cracked section at the Upper Gallery showed it could handle PMF loadings (understood to be RL 110.7 at that time) with zero cohesion in the cracked concrete. This is in agreement with the current analysis.

# 5 CONCLUSIONS

## 5.1 Stability Considerations

The Somerset Dam abutment monoliths comprise a wide range of layouts with different structural arrangements in the upper levels, different internal drainage arrangements, and different gallery layouts. A simple analysis using a standard section is suitable only for preliminary assessments.

The storage level of RL 109.75 is the critical level required to handle the 1 in 100,00 AEP flood event and ensure that the Stage 1 works at Wivenhoe are not compromised. The stability analyses show that all monoliths satisfy stability criteria at the two critical sections:

- The "change of slope" at RL 100.45 and
- The "Upper Gallery" level of RL 88.6 where cracked concrete is observed in the gallery walls.

The stability results for higher water levels are summarised at Table 5-1, with critical water levels where stability criteria are no longer satisfied and also levels for theoretical failure of the units.

**Table 5-1 - Stability Results for Abutment Monoliths**

| Location & Condition | Critical Water Level | | Notes |
|---|---|---|---|
| | Stability Criteria | Monolith Failure | |
| Change of Slope, good quality concrete | RL 111.0* | RL 111.6 | *75% of base is cracked |
| Upper Gallery, cracked concrete; Case A | RL 110.9* | > RL 112 | *Shear factor falls below 2.0 |
| Below Upper Gallery, sound concrete; Case B | RL 111.4 | RL 111.4* | *U/S tensile stress of 90 kPa initiates crack. Cracked analysis fails |
| Below Upper Gallery, cracked concrete; Case C | RL 109.7* | RL 110.1** | *Shear factor falls below 2.0<br>**Fails in shear |

For the "change in slope" location, some monoliths (G, H and Q) are more stable. Monolith R is somewhat less stable and satisfies stability criteria for storage levels only up to RL 110.6. However, this monolith would receive support from adjacent more stable monoliths.

Previous flood data for Somerset Dam indicates a 1 in 200,000 AEP for a storage level of RL 111.0 (all gates operating). This data is based on flood operational rules prior to the Wivenhoe Upgrade (as given in the WIVOPS program). Revised estimates with new flood operating rules would reduce the probability.

It is not known whether cracking observed in the Upper Gallery also occurs above or below the gallery. If cracking does exist (Case C), the dam would just satisfy stability criteria for a storage level of RL 109.7. However, stability would reduce rapidly with higher water levels and the section theoretically fails in shear with a storage level of RL 110.1.

## 5.2 Limitations & Recommendations

A number of considerations that affect the conclusions drawn in this Report are discussed below:

### *5.2.1 Surcharging of Somerset Dam during Extreme Flood Events*

It is understood that the WIVOPS flood operation program required that the Somerset spillway gates be lowered if Wivenhoe Dam is in danger of being overtopped. With the Stage 1 spillway upgrade this situation would only occur for extreme floods. It is suggested at Commerce (2004) that lowering the gates back into a major spillway discharge is unlikely to happen in practice and this procedure was not included in the hydraulic analyses. It is noted that a short review of the initial GHD Report, Russo (1996) offers the same comment and recommends that gates should be locked in the raised position during extreme flood events to ensure it does not happen. In either case, the gate operation manual should clearly document the procedure.

If it is intended that Somerset Dam be surcharged in this manner to minimise flood levels at Wivenhoe Dam then this Report should be reviewed and the spillway examined in detail to ensure these operations can be undertaken successfully.

### *5.2.2 Flooding of the Gallery System*

Storage levels above RL 107. 46 overtop most of the abutment monoliths and discharge water onto the backface of the dam. The dam has a variety of entrance doors, gate shafts, galleries etc and a common problem with dams such as this is that gallery and drainage systems will be flooded when they are most needed.

This Report assumes that the gallery systems are not flooded. Flooding would introduce full storage head into the interior of the dam and negate the drainage provisions. The stability would be markedly reduced.

The dam layout should be reviewed to ensure this is the case and waterproof doors installed where necessary. There are references to gallery flooding in several of the Reports including Russo (1996).

### *5.2.3 Internal Drainage*

The internal drainage system is complicated, particularly at the lower levels of the dam. It is understood the internal drainage and the role it plays is not documented in the Operation and Maintenance Manuals. Internal drainage at the higher levels relevant to this Report has been briefly inspected and appears to be in good condition. Drains at lower levels in some cases are blocked entirely by calcite. Many of the vertical drains are capped to prevent dirt entering the system, but caps are jammed and likely to prevent proper operation of the system.

The stability assessment assumes the internal drainage system is working effectively to reduce uplift. Failure of the internal drainage system can produce higher uplift pressures, markedly reducing stability.

It is strongly recommended that the drainage system be documented, critical drains be monitored and maintenance carried out where necessary.

### *5.2.4 Cracked Concrete at the Upper Gallery*

Stability analyses indicate that cracked concrete surfaces above or below the Upper Gallery are stable for a storage level of RL 109.7 but that this stability reduces rapidly with higher storage levels with failure indicated for a storage level of RL 110.1.

The cracks are most probably due to volume change due to dissipation of heat following completion of construction (as suggested in GHD (2000)). This would be consistent with cracks first being noticed until the 1960's. Other factors that would contribute would be the long break in construction between 1942 and 1948, the use of two grades of concrete and the high cement content of the upstream concrete.

Given the size of the cracks and the rapid reduction in stability with a small increase in water level, it would be good practice to investigate this area more closely. The purpose is to gain a better understanding of the dam and its limitations.

It is recommended that some exploratory drilling be carried out to determine whether such cracks do exist. A similar recommendation was made in GHD (2000).

### *5.2.5 Risk Assessment*

The stability analyses summarised above demonstrate that the risk profile developed by SMEC (2004) makes conservative assumptions for structural failure in the upper levels of the abutment monoliths. On this basis, there is no need for further development of the Somerset Dam risk profile as a stand alone document.

However, the current risk analysis for Wivenhoe Dam is a modification of the Preliminary Risk Assessment produced by SKM and uses the SKM loss of life data. The Wivenhoe/Somerset combined risk profile is a borderline case in terms of the ANCOLD criteria. Given the importance of these dams and the flood mitigation benefits provided to Brisbane and adjacent areas, consideration should be given to a detailed assessment of the combined risk profile. This would require among other assessment work, more detailed engineering assessments and new determinations for consequences, particularly the loss of life figures. It would be a significant project and would need some careful definition of the scope of work to identify the areas where additional studies would provide value for money.

## 6 REFERENCES


ANCOLD (1999): Guidelines on Design Criteria for Gravity Dams, produced by Australian National Committee on Large Dams, 1991.

CDSA (1999): Dam Safety Guidelines produced by the Canadian Dam Safety Association

Commerce (2004): Somerset & North Pine Dams, Safety Review, produced by the Department of Commerce in December 2004.

DPI (1994): A 20 volume study undertaken by Department of Natural Resources, extracts of which are included at Appendix 3.6 of SMEC (2004).

GHD (2000): Somerset Dam Safety Review, Report prepared for SEQWater by GHD Australia Limited 2000.

Russo (1996): 1996 GHD Somerset Dam Safety Review, Comments by R. Russo, a short review of the initial GHD Report

SKM (2000): Preliminary Risk Assessment Wivenhoe, Somerset and North Pine Dams, Report prepared for SEQWater by SKM Australia Limited, 2000

SMEC (2004): Somerset Dam Detailed Risk Assessment Stage 2, Report prepared for SEQWater by SMEC Australia Limited 2004

# Appendix A

Stability Analyses

For

Change of Slope

| Section at Change of Slope | | | Sound Concrete | | | |
|---|---|---|---|---|---|---|
| Storage Level 109.70 | | | at RL 100.45 | | | |
| **Data** | **Uncracked Section Analysis** | | | **Cracked Section Analysis** | | |
| Base Length T = | 5.666 | | | 5.666 | | |
| Crack Length | 0.000 | | | 1.570 | | 28% |
| Drains from U/S Face Du = | 3.200 | | | 3.200 | | |
| Drain Factor p = | 0.250 | | | 0.250 | | |
| Area Uncracked Base A = | 5.666 | | | 4.096 | | |
| U/S Section Modulus Zu = | 5.351 | | | 2.796 | | |
| D/S Section Modulus Zd = | 5.351 | | | 2.796 | | |
| **Loading** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** |
| **Vertical Loads** | | | | | | |
| Vert Dead Load | 879 | -3.298 | -2,899 | 879 | -3.298 | -2,899 |
| Vert Water u/s face | 0 | | 0 | 0 | | |
| Vert Water on Overflow | 57 | -3.291 | -187 | 57 | -3.533 | -200 |
| Uplift | -209 | -4.020 | 842 | -263 | -4.031 | 1,060 |
| **Total Vertical Loads** | **726** | **-3.089** | **-2,244** | **673** | **-3.031** | **-2,040** |
| **Total Horizontal Loads** | **397** | **2.822** | **1,122** | **397** | **2.822** | **1,122** |
| **Summary Data** | | | | | | |
| Resultant -D/S Toe | 726 | -1.545 | -1,122 | 673 | -1.364 | -918 |
| **Stability Parameters** | | | | | | |
| U/S Bearing Pressure kPa | -47 | | | 0 | | |
| D/S Bearing pressure kPa | 303 | | | 329 | | |
| Friction Factor; C=100, Tan Ø= 1.0 | 3.25 | | | 2.72 | | |
| Friction Factor; C=1,400, Tan Ø= 1.0 | 21.78 | | | 16.12 | | |

**Note: Cracked section unlikely as u/s tensile stress of 47 kPa well below 90 kPa criteria**

## Section at Change of Slope

**Sound Concrete**

**Storage Level 111.04** **at RL 100.45**

| Data | | Uncracked Section Analysis | | | Cracked Section Analysis | | |
|---|---|---|---|---|---|---|---|
| Base Length | T = | 0.000 | | | 5.666 | | |
| Crack Length | | 0.000 | | | 4.250 | | 75% |
| Drains from U/S Face | Du = | 3.200 | | | 3.200 | | |
| Drain Factor | p = | 0.250 | | | 0.250 | | |
| Area Uncracked Base | A = | 5.666 | | | 1.416 | | |
| U/S Section Modulus | Zu = | 5.351 | | | 0.334 | | |
| D/S Section Modulus | Zd = | 5.351 | | | 0.334 | | |
| **Loading** | | **Load** | **L arm D/S Toe** | **Mom D/S Toe** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** |
| **Vertical Loads** | | | | | | | |
| Vert Dead Load | | 879 | -3.298 | -2,899 | 879 | -3.298 | -2,899 |
| Vert Water u/s face | | 0 | | 0 | 0 | | 0 |
| Vert Water on Overflow | | 91 | -3.291 | -298 | 91 | -3.533 | -320 |
| Uplift | | -240 | -4.020 | 964 | -463 | -3.247 | 1,503 |
| **Total Vertical Loads** | | **730** | **-3.060** | **-2,233** | **507** | **-3.386** | **-1,716** |
| **Total Horizontal Loads** | | **494** | **2.993** | **1,477** | **494** | **2.993** | **1,477** |
| **Summary Data** | | | | | | | |
| Resultant -D/S Toe | | 669 | -0.791 | -529 | 507 | -0.472 | -239 |
| **Stability Parameters** | | | | | | | |
| U/S Bearing Pressure | kPa | -116 | | | 0 | | |
| D/S Bearing pressure | kPa | 374 | | | 716 | | |
| Friction Factor; C=100, Tan Ø= 1.0 | | 2.63 | | | 1.31 | | |
| Friction Factor; C=1,400, Tan Ø= 1.0 | | 17.55 | | | 5.04 | | |

Note: Cracked extends through 75% of base

# Appendix B

Stability Analyses

For

Upper Gallery

Case A

| Section Through Upper Gallery | | | | Cracked Concrete | | |
|---|---|---|---|---|---|---|
| Storage Level 109.70 | | | | at RL 88.6 | | |
| | | Case A | | | | |
| **Data** | **Uncracked Section Analysis** | | | **Cracked Section Analysis** | | |
| Base Length T = | 15.090 | | | | | |
| Crack Length Lc = | 0.000 | | | | | |
| Drains from U/S Face Du = | 2.600 | | | | | |
| Gallery from U/S Face | 6.100 | | | | | |
| Drain Factor p = | 0.330 | | | | | |
| Area Uncracked Base A = | 13.337 | | | | | |
| U/S Section Modulus Zu = | 37.440 | | | | | |
| D/S Section Modulus Zd = | 38.190 | | | | | |
| **Loading** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** |
| **Vertical Loads** | | | | | | |
| Vert Dead Load | 3,675 | -9.995 | -36,733 | | | |
| Vert Water u/s face | 88 | -14.832 | -1,308 | | | |
| Vert Water on Overflow | 57 | -12.364 | -701 | | Not | |
| Uplift | -480 | -13.331 | 6,392 | | | |
| | | | | | Applicable | |
| **Total Vertical Loads** | **3,341** | **-9.684** | **-32,350** | | | |
| **Total Horizontal Loads** | **2,162** | **6.904** | **14,923** | | | |
| **Summary Data** | | | | | | |
| Resultant -D/S Toe | 3,341 | -5.217 | -17,426 | | | |
| **Stability Parameters** | | | | | | |
| U/S Bearing Pressure kPa | 49 | | | | | |
| D/S Bearing pressure kPa | 448 | | | | | |
| Friction Factor; C=0, Tan Ø= 1.0 | 1.545 | | | | | |
| Friction Factor; C=100, Tan Ø= 1.0 | 2.162 | | | | | |
| Friction Factor; C=1,400, Tan Ø= 1.0 | 10.184 | | | | | |

## Section Through Upper Gallery — Cracked Concrete

**Storage Level** 110.87 — at RL 88.6

Case A

| Data | | Uncracked Section Analysis | Cracked Section Analysis |
|---|---|---|---|
| Base Length | T = | 15.090 | 15.090 |
| Crack Length | Lc = | 0.000 | 0.350 |
| Drains from U/S Face | Du = | 2.600 | 2.600 |
| Gallery from U/S Face | | 6.100 | 6.100 |
| Drain Factor | p = | 0.330 | 0.333 |
| Area Uncracked Base | A = | 13.337 | 13.0 |
| U/S Section Modulus | Zu = | 37.440 | 35.5 |
| D/S Section Modulus | Zd = | 38.190 | 36.5 |

| | Load | L arm D/S Toe | Mom D/S Toe | Load | L arm D/S Toe | Mom D/S Toe |
|---|---|---|---|---|---|---|
| **Vertical Loads** | | | | | | |
| Vert Dead Load | 3,675 | -9.995 | -36,733 | 3,675 | -9.995 | -36,733 |
| Vert Water u/s face | 95 | -14.830 | -1,409 | 95 | -14.830 | -1,409 |
| Vert Water on Overflow | 86 | -12.364 | -1,067 | 86 | -12.364 | -1,067 |
| Uplift | -506 | -13.331 | 6,747 | -531 | -13.369 | 7,104 |
| **Total Vertical Loads** | **3,350** | **-9.689** | **-32,462** | **3,325** | -9.655 | **-32,105** |
| **Total Horizontal Loads** | **2,381** | **7.151** | **17,029** | **2,381** | **7.151** | **17,029** |
| **Summary Data** | | | | | | |
| Resultant -D/S Toe | 3,350 | -4.606 | -15,433 | 3,325 | -4.534 | -15,076 |

| Stability Parameters | | Uncracked | Cracked |
|---|---|---|---|
| U/S Bearing Pressure | kPa | -5 | 0 |
| D/S Bearing pressure | kPa | 502 | 505 |
| Friction Factor; C=0, Tan Ø= 1.0 | | 1.41 | 1.40 |
| Friction Factor; C=100, Tan Ø= 1.0 | | 1.97 | 1.94 |
| Friction Factor; C=1,400, Tan Ø= 1.0 | | 9.25 | 9.03 |

# Appendix B

Stability Analyses

Below

Upper Gallery

Case B & Case C

**Section Below Upper Gallery** | **Sound Concrete or Concrete with Cracks**
**Storage Level 109.70** | **at RL 88.6**
**Applies Case B & C**

| Data | | Uncracked Section Analysis | | | Cracked Section Analysis | | |
|---|---|---|---|---|---|---|---|
| Base Length | T = | 15.090 | | | 15.1 | | |
| Crack Length | | 0.000 | | | 1.1 | | |
| Drains from U/S Face | Du = | 3.200 | | | 3.2 | | |
| Drain Factor | p = | 0.330 | | | 0.3 | | |
| Area Uncracked Base | A = | 15.090 | | | 14.0 | | |
| U/S Section Modulus | Zu = | 37.951 | | | 32.6 | | |
| D/S Section Modulus | Zd = | 37.951 | | | 32.6 | | |
| **Loading** | | **Load** | **L arm D/S Toe** | **Mom D/S Toe** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** |
| **Vertical Loads** | | | | | | | |
| Vert Dead Load | | 3,675 | -9.995 | -36,733 | 3,675 | -9.995 | -36,733 |
| Vert Water u/s face | | 88 | -14.832 | -1,308 | 88 | -14.832 | -1,308 |
| Vert Water on Overflow | | 61 | -12.364 | -759 | 61 | -12.364 | -759 |
| Uplift | | -847 | -10.961 | 9,279 | -928 | -11.171 | 10,362 |
| **Total Vertical Loads** | | **2,978** | **-9.912** | **-29,521** | **2,897** | **-9.816** | **-28,438** |
| **Total Horizontal Loads** | | **2,162** | **6.904** | **14,923** | **2,162** | **6.904** | **14,923** |
| **Summary Data** | | | | | | | |
| Resultant -D/S Toe | | 2,978 | -4.901 | -14,597 | 2,897 | -4.665 | -13,514 |
| **Stability Parameters** | | | | | | | |
| U/S Bearing Pressure | kPa | -10 | | | 0 | | |
| D/S Bearing pressure | kPa | 405 | | | 414 | | |
| Friction Factor; C=0, Tan Ø= 1.0 | | 1.38 | | | 1.34 | | |
| Friction Factor; C=100, Tan Ø= 1.0 | | 2.08 | | | 1.99 | | |
| Friction Factor; C=1,400, Tan Ø= 1.0 | | 11.15 | | | 10.40 | | |

| Section Below Upper Gallery | | Cracked Concrete |
|---|---|---|
| Storage Level | 110.10 | at RL 88.6 |
| | | Case C |

| Data | | Uncracked Section Analysis | Cracked Section Analysis |
|---|---|---|---|
| Base Length | T = | 15.090 | 15.090 |
| Crack Length | | 0.000 | 9.0 m |
| Drains from U/S Face | Du = | 3.200 | 3.200 |
| Drain Factor | p = | 0.330 | 0.330 |
| Area Uncracked Base | A = | 15.090 | 6.090 |
| U/S Section Modulus | Zu = | 37.951 | 6.181 |
| D/S Section Modulus | Zd = | 37.951 | 6.181 |

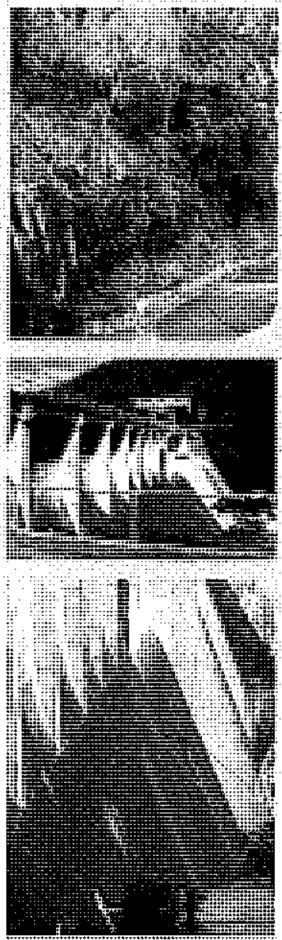
| Loading | Load | L arm D/S Toe | Mom D/S Toe | Load | L arm D/S Toe | Mom D/S Toe |
|---|---|---|---|---|---|---|
| **Vertical Loads** | | | | | | |
| Vert Dead Load | 3,675 | -9.995 | -36,733 | 3,675 | -9.995 | -36,733 |
| Vert Water u/s face | 91 | -14.831 | -1,343 | 91 | -14.831 | -1,343 |
| Vert Water on Overflow | 72 | -12.364 | -894 | 72 | -12.364 | -894 |
| Uplift | -863 | -10.961 | 9,455 | -2,210 | -9.056 | 20,013 |
| **Total Vertical Loads** | **2,975** | **-9.920** | **-29,515** | **1,628** | **-11.644** | **-18,957** |
| **Total Horizontal Loads** | **2,483** | **7.253** | **18,010** | **2,236** | **6.993** | **15,640** |
| **Summary Data** | | | | | | |
| Resultant -D/S Toe | 2,975 | -4.663 | -13,874 | 1,628 | -2.037 | -3,316 |

| Stability Parameters | | Uncracked | Cracked |
|---|---|---|---|
| U/S Bearing Pressure | kPa | -29 | 2 |
| D/S Bearing pressure | kPa | 423 | 533 |
| Friction Factor; C=0, Tan Ø= 1.0 | | 1.33 | 0.73 |
| Friction Factor; C=100, Tan Ø= 1.0 | | 2.01 | 1.00 |
| Friction Factor; C=1,400, Tan Ø= 1.0 | | 10.78 | 4.54 |

| Section Below Upper Gallery | | Sound Concrete Only | | | | | |
|---|---|---|---|---|---|---|---|
| Storage Level | 111.41 | at RL 88.6 | | | | | |
| | | Case B | | | | | |

| Data | | Uncracked Section Analysis | | | Cracked Section Analysis | | |
|---|---|---|---|---|---|---|---|
| Base Length | T = | 15.090 | | | | | |
| Crack Length | | 0.000 | | | | | |
| Drains from U/S Face | Du = | 3.200 | | | | | |
| Drain Factor | p = | 0.330 | | | Section | | |
| Area Uncracked Base | A = | 15.090 | | | Fails | | |
| U/S Section Modulus | Zu = | 37.951 | | | when | | |
| D/S Section Modulus | Zd = | 37.951 | | | Cracked | | |
| **Loading** | | **Load** | **L arm D/S Toe** | **Mom D/S Toe** | **Load** | **L arm D/S Toe** | **Mom D/S Toe** |
| **Vertical Loads** | | | | | | | |
| Vert Dead Load | | 3,675 | -9.995 | -36,733 | | | |
| Vert Water u/s face | | 98 | -14.828 | -1,455 | | | |
| Vert Water on Overflow | | 108 | -12.364 | -1,338 | | | |
| Uplift | | -915 | -10.961 | 10,031 | | | |
| **Total Vertical Loads** | | **2,966** | -9.943 | **-29,495** | | | |
| **Total Horizontal Loads** | | **2,483** | **7.253** | **18,010** | | | |
| **Summary Data** | | | | | | | |
| Resultant -D/S Toe | | 2,966 | -3.872 | -11,485 | | | |
| **Stability Parameters** | | | | | | | |
| U/S Bearing Pressure | kPa | -91 | | | | | |
| D/S Bearing pressure | kPa | 484 | | | | | |
| Friction Factor; C=0, Tan Ø= 1.0 | | 1.19 | | | | | |
| Friction Factor; C=100, Tan Ø= 1.0 | | 1.80 | | | | | |
| Friction Factor; C=1,400, Tan Ø= 1.0 | | 9.70 | | | | | |

BM-1: Document 38



SEQWater

# Somerset Dam Crack Investigation

Project No. 3003301

SMEC

## Table of Contents



## Figures

# 1 Introduction

South East Queensland Water (SEQWater) owns and operates Somerset Dam. The concrete gravity dam has a well developed crack on the downstream side of the dam that is visible on the downstream face and within the upper gallery of the dam.

This crack has been in existence for a number of years and several attempts have been made to investigate the crack and its impact on the dam.

In order to further quantify the extent of cracking and whether the existing crack was the location of movement within the dam, SEQWater submitted a project brief titled "Somerset Dam Crack Investigation".

This report details the outcome of a drilling and core logging program undertaken in accordance with the above brief.

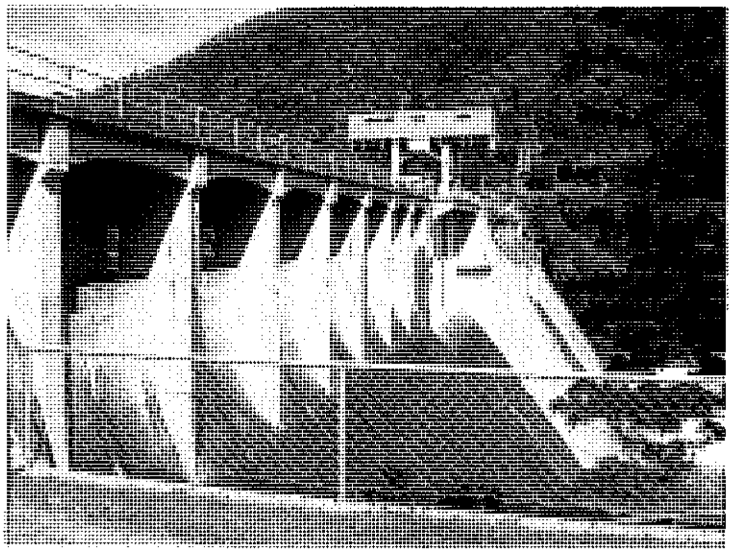


*Figure 1. Somerset Dam*

# 2 Project Outline

Somerset Dam comprises a 47 m high concrete gravity dam with a crest length of 308 m. The dam was constructed in two stages commencing in 1935 but construction was suspended in 1942 due to the impact of the Second World War. Work recommenced on the dam in 1948 and was completed in 1953.

There are two galleries within the concrete structure. The upper inspection gallery at RL 88.9 m provides access to other internal galleries associated with the baulks and stop gates.

The lower gallery at RL 66.0 m is a drainage gallery. The two galleries are joined by an internal stair well at each end of the galleries.

Both galleries are approximately 1.75 m wide and 1.9 m high.

The cracking at Somerset Dam is typically evident in the top downstream corner of the upper gallery. The crack runs nearly for the full width of the dam at the level of the gallery. The crack is also evident in a number of areas across the downstream face of the dam. Due to the presence of the crack both in the gallery and on the external downstream face of the dam it is likely that the crack is continuous between these two locations.

Previous investigation saw a limited number of cores being taken from within the gallery along the line of the cracks. These cores were drilled horizontally from within the gallery and limited to a depth of approximately 1 to 2 metres and provided a limited degree of information in respect to the cracking.

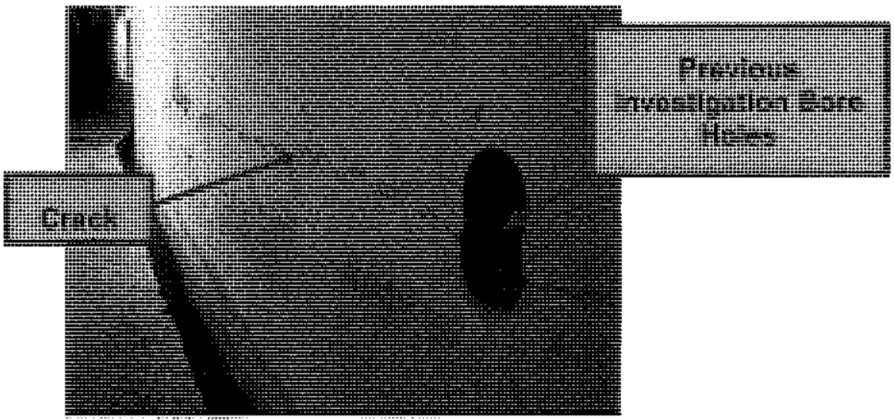



*Figure 2. Crack in Upper Gallery – Somerset Dam*

In order to more fully understand the nature of the cracking, a more comprehensive drilling program was proposed. This program was intended to:-

- Confirm that the cracks extended from the upper gallery towards the downstream face of the dam;
- Provide core samples from the region of the crack for further examination;
- Undertake drilling and coring of the concrete upstream of the upper gallery to determine if the cracking was extending towards the upstream face.

The project brief allowed for drilling from both the upper gallery and from the top of the dam, with drill holes angled so as to intercept any cracking either upstream or downstream from the gallery.

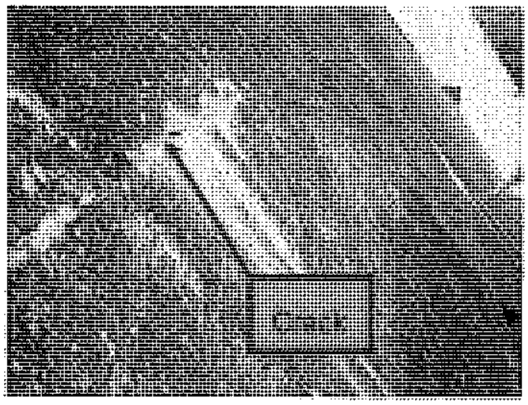


*Figure 3. Crack across Downstream Face – Somerset Dam*

The drilling and coring program covered examination of Somerset Dam in a limited number of areas (Monolith Blocks) only and from the crest of the dam to a depth of only approximately 20 metres. The investigation did not extend into the lower sections of the dam below the level of the upper gallery.

# 3 Drilling Program

Undertaking a drilling program at Somerset Dam to investigate the existing cracking has a number of issues including:-

- The existing crack is located at a level well below the road deck and also the dam crest level (located at a level below the road deck);
- The gallery has a small cross section only and limits the size of the equipment that can be used within the gallery and the length of cores that can be extracted;
- Access to the dam crest level is restricted by narrow and relatively low access openings through the piers supporting the road deck.

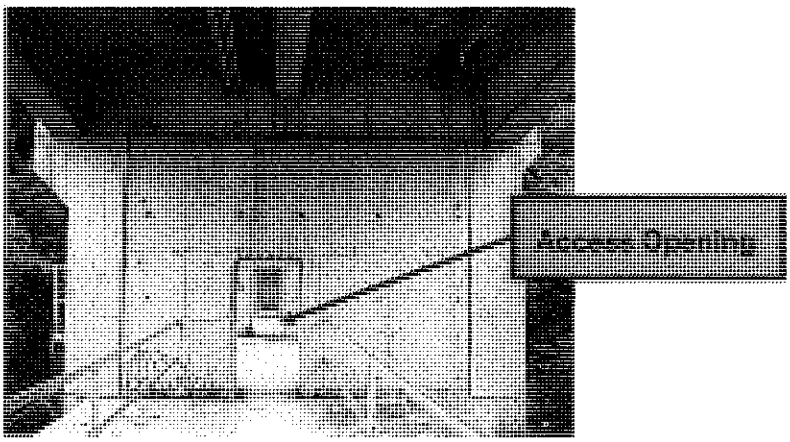



*Figure 4. Somerset Dam Crest Level Access*

Drilling from the upper gallery was considered to be unviable due to:-

- Restricted the size of equipment that could be used, effectively restricting the depth of any drill holes;
- The gallery contains a number of services, which were not to be disrupted. The electrical services in particular were located in the upper corners of the gallery and most likely would have required relocation;
- The limited cross section of the gallery would have made it difficult to carry out the work and for SEQWater operators to access the gallery.

The preferred method was using a portable drilling rig that could be largely dismantled and manhandled through the pier access openings, and drilling from the dam crest level. Whilst this meant a longer length of drilling overall, it was more easily and quickly carried out.

In addition, the preferred option provides cores of the upper section of Somerset Dam above the level of the gallery which can be used as future references.

The drilling program was undertaken by "Mulligans Drilling" from Khancoban (NSW) using a portable drilling rig that could be manhandled through the access openings but also provided the ability to drill and extract longer cores.

Supervision of the drilling and logging of the extracted cores was undertaken by Geotechnical Engineering Staff from SMEC's Brisbane Office. The bore logs including photographs are detailed in the attached Geotechnical Memorandum.

# 4 Issues Encountered

In addition to the issues around gaining access to the dam crest level for the drilling rig, other issues associated with the project included:-

- Locating the drill rig and its inclination for the angled holes. These holes were to pass at an angle on the downstream side of the upper gallery. In several instances, the bore holes intercepted the gallery.
- Core samples could not be retrieved in a single piece and a number of drilling induced breaks in the cores occurred. All breaks were examined at the time of extraction and the type of break was noted (ie existing crack versus drilling induced).
- Breaks were recorded at various depths including well above the upper gallery in both vertical and angled holes (both upstream and downstream of the gallery). Whilst these were recorded as breaks, it is possible that some of these are the joints between successive pours in the monoliths.

Confirmation of the cracks was undertaken by examination of the core samples. The cracks were identified by a "weathered" appearance of the crack or the build up of calcite at the crack (an indication of possible water flow). Observations of the drillers also noted a change in the drilling fluid at cracks (loss of drilling fluid) at some locations.

## 5 Core Samples

The drilling program resulted in a large number of core samples being taken from the dam. All core samples were logged, numbered and placed in core trays and have been placed in storage at Somerset Dam.

The detailed core logs and photographs of the cores are contained in the Geotechnical Memorandum included in this report. The memorandum also includes detailed drawings on the location and angle of each cored hole.

Where identified the existing crack has been photographed in detail.

The attached memorandum includes a tabulation for each core indicating the location of the any observed breaks.

# 6 Examination of Cores

Following the extraction of the cores, the cores were examined by Peter Darling and Howard Baldwin of SMEC at Somerset Dam. The following observations were noted:-

- The cores recovered from the drilling indicate the concrete in the dam in the section above the upper gallery to be of good quality. There were no signs of defects or other deficiencies (other than as noted below) and very little evidence of air bubbles entrained in the concrete.

- The aggregate use for the concrete is relatively large, possibly a maximum size of 4" to 6" (100 mm to 150 mm). The cores indicate that the aggregate was produced from a crushing process with little evidence of rounded or washed material.

- There were no signs of AAR (Alkali Aggregate Reaction) in the cores examined.

- The cracks (other than drill breaks) encountered in the cores (2 sighted) had a 'weathered' appearance. There were no signs of any significant deterioration or deposition on the surface of the cracks.

- The drill breaks in the core have a different appearance to the surface of the cracks encountered in the cores – they appeared 'fresh' with no surface deterioration or deposition.

- A number of drill breaks were noted along the aggregate/paste interface. The interfaces have a relatively 'smooth' appearance, unlike the breaks through the adjacent cement paste which is 'rougher'. The surface of the exposed interface when rubbed left a 'cement' deposit, and the interface represents a 'weakness', although how weak is not known. The 'powdery' interface is a localized effect, and limited to the piece of aggregate in question. This weakness is not considered to be extensive but more localised to individual aggregate pieces – cores were typically 200mm to 600mm long between breaks. This feature would not appear to impact on the overall integrity (strength) of the concrete to any significant degree.

- An occasional 'boundary' between the aggregate and the paste was observed in the core; this feature may represent the 'weaker' aggregate/paste interface indicating it is not an extensive feature.

- Little if any breaks in the aggregate itself were noted.

# 7 Conclusions

The coring program identified a number of "breaks" including multiple breaks in individual holes, including at levels well above the upper gallery and also upstream of the upper gallery. The majority of these breaks are not evident in the wider structure and are considered to be associated with construction joints (joints between successive pours).

Examination of the core breaks at a depth that coincides with the upper gallery indicated that there does not appear to be any movement taking place at the joint (no evidence of abrasion or other signs of movement at the break other than vertical displacement).

At this stage the existing crack downstream of the upper gallery does not appear to be impacting on the operation of Somerset Dam or its overall structural stability. No significant cracking was identified upstream of the upper gallery.

The concrete in the upper section of the dam appears to be relatively uniform in nature and no defects such as large amounts of entrained air, poorly compacted concrete or Alkali Aggregate Reaction were observed in the core samples.

The observations and conclusions are applicable only to the monoliths that were cored and for the area of the Somerset Dam above the upper gallery. This investigation did not include any section of the dam below the level of the upper gallery.

# 8 Recommendations

The coring program confirmed the presence of the crack in the area downstream of the upper gallery at a number of locations. Examination of the cores have indicated that little if any movement is taking place and that the concrete in the area of the cracks does not appear to be deteriorating or indicating signs of any further issues.

Whilst cracks were identified at a number of levels in the area upstream of the upper gallery, they appear to be isolated and most likely associated with the joint between successive concrete pours.

Whilst the cracks do not appear to be deteriorating or extending at this time or presenting a structural issue that is impacting on the operation of Somerset Dam, it is recommended that:-

- The cores taken from Somerset Dam by this program be retained in storage and used as a future reference;
- The existing crack in the upper gallery continues to be monitored in respect to movement or other signs of deterioration. It is recommended that monitoring be undertaken at least 4 monthly and as a minimum as part of any annual dam safety inspection and immediately following any significant inflows to the dam;
- That the monitoring include measurement of both horizontal and vertical movement of the crack at a minimum of 6 No. locations in the upper gallery parallel to the main axis of the dam and at each of the gallery entrances (perpendicular to the main axis of the dam);
- The results of the crack movement monitoring be examined further at the next comprehensive dam safety review;
- Any noticeable change or movement of the crack is immediately investigated and appropriate actions implemented to ensure the safety and integrity of Somerset Dam.

# 9 Geotechnical Memoranda

SMEC
SMEC Australia Pty Ltd

**MEMORANDUM**

**TO:** [redacted]
**FROM:** [redacted]
**SUBJECT:** SOMERSET DAM CRACK INVESTIGATION
**PROJECT NO:** 3003301
**DATE:** 28/02/2008
**CC:** [redacted]

## INTRODUCTION

This memorandum details the findings of the completed drilling works at Somerset Dam. The drilling works were undertaken as part of a wider risk assessment of the dam's long term stability and involved investigating known and unknown cracks within the dam's mass concrete structure. Included in the following document are an outline/brief on the project itself, details of the works completed and the resulting data and information gathered.

No recommendations or conclusions have been drawn from the works to date. This memorandum is a factual report of observations and findings during the drilling works.

More detailed background information on the dam and the original proposal as initially outlined by SMEC are contained within document titled "Proposal for Somerset Dam Crack Investigation" dated January 2007.

## PROJECT BRIEF

The requirements of the original proposal from which the current works were based on was for a series of 16 angled and vertical core holes (8 of each) to be drilled into the dams structure within monoliths F, G, R and S. These core holes were required to investigate the condition and the extent of internal cracking that may potentially exist above and below the dam's upper inspection gallery.

A previous investigation into the known existing crack in the upper inspection gallery involved the drilling of a number of short horizontal 100mm cores into the exposed crack within the upper gallery. This existing crack can be seen to propagate horizontally along the length on the upper gallery and can also be seen to a lesser degree on the down stream face of the dam wall at a similar RL as the crack within the gallery.

The intent of the current crack investigation was to determine the extent of this propagation and if possible find if any other cracks existed above or below this existing crack. The vertical holes on the upstream side of the upper inspection gallery were to investigate the extent of the existing crack on the downstream and upstream sides of the dam (or to confirm if it was isolated to the down stream side). The angled holes were drilled to intersect any cracks on the downstream side of the gallery (above and below the existing crack).

## DRILLING WORKS

The drilling works were completed over a 17 day period from the 6th to the 22nd of February and included 15 working days where drilling was performed. Drilling contractor "Mulligan's Drilling" from Khancoban, NSW were engaged to undertake the drilling works. Their selection as the contractor of choice was based on their previous drilling experience at Somerset Dam, their expertise in difficult access drilling work and their availability. A summary of completed holes with their finished depth, angle and monolith they were drilled in is tabled below.

| HOLE ID | DEPTH | MONOLITH | INCLINATION |
|---|---|---|---|
| CHSV1 | 21.48 | S | -90 |
| CHSA1 | 22.32 | S | -70.5 |
| CHTV1 | 21.30 | T | -90 |
| CHTA1 | 18.00 | T | -71.5 |
| CHTA2 | 21.92 | T | -66 |
| CHSV2 | 21.25 | S | -90 |
| CHSA2 | 22.45 | S | -67 |
| CHEV1 | 21.79 | E | -90 |
| CHEA1 | 5.00 | E | -67.8 |
| CHEA2 | 18.60 | E | -67 |
| CHFV1 | 21.25 | F | -90 |
| CHFA1 | 22.90 | F | -66 |
| CHFV2 | 21.33 | F | 90 |
| CHFA2 | 22.93 | F | -65 |
| CHGA1 | 22.62 | G | -64.5 |

**Table 1.** Hole summary

### *Access*

Due to access constraints (0.8m opening width between monolith bays) and the required location of the coreholes, the choice of drill rigs was limited to those that could be maneuvered into place through the existing opening and/or assembled and disassembled over each of the designated holes. Mulligan's chose to use a "Scout" rig which could be assembled and disassembled as required.

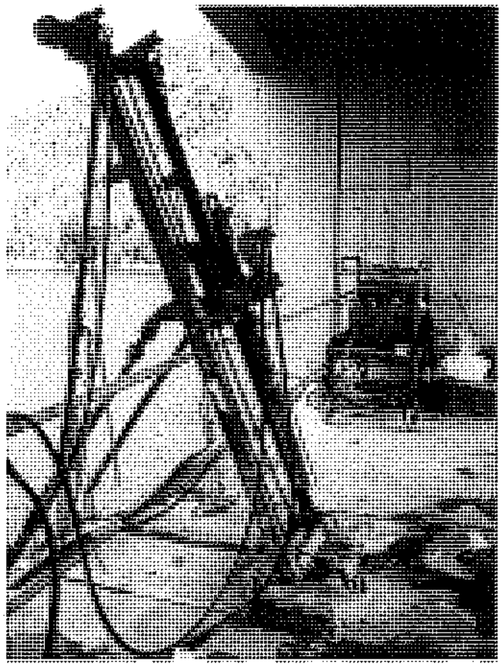


**Photo 1.** Scout rig positioned over hole.

### *Supervision Requirements*

SMEC's on site supervision requirements involved coordination of the drilling works, determining the location and angles of each of the holes to be drilled, recording observations of core as it was extracted (logging, photographing etc) and the labeling and storage (on-site) of core boxes on completion of works.

### *Drilling and Survey*

Drilling of holes involved first setting up of the rig over the designated position, using "Diatube" to core out a short 100mm diameter hole in which to set the T piece/collar, grouting in the T piece/collar and then drilling using NMLC coring methods. Each retrieved core was approximately 1.5m in length and required breaking in order to fit into the 1.05m core boxes. Core was logged as it came up noting all apparent drill breaks "DB", hand breaks "HB" and any cracks. On completion of the holes, each was surveyed at a point 7m from the top and at the bottom of each the hole using "Reflex EZ-TRAC" down hole survey equipment which gave a read out of the holes azimuth and inclination at selected points down the hole. The information gathered from these down hole surveys allows for the accurate plotting of the holes position (and any cracks found) in relation to the upper gallery.

### *Hole Identification*

To aid in recognition of holes position at a later date, coreholes were named according to the monolith they were drilled in, orientation of the hole and the order/sequence that they were drilled in i.e. CHSA2 designated? the 2nd angled hole drilled in monolith S, where "CH" is an abbreviation of corehole, "S" represents the monolith hole was drilled in, "A" represents that hole was angled as opposed to "V" for vertical and "2" represents that the hole was the second angled hole drilled in monolith S.

## CONSTRAINTS

As previously mentioned, the access constraints required that a portable "scout" rig be used. The position of the upper gallery and the downstream face meant that to avoid drilling coming out of the DS face of the dam or going through the upper gallery, correct angles would need to be determined prior to drilling. As the upper galleries alignment changed (was not in a straight line) in relation to the position of drilling on the upper deck, it was necessary to adjust the angles for each of the holes dependant on which monolith drilling was being done in.

## IDENTIFICATION OF CRACKS

NMLC drilling methods with a series 10 bit were used on all 15 holes giving an outside hole diameter of approximately 75mm and a recovered core diameter of 50mm. Due to the size of the core samples, the retrieved core displayed a tendency towards drilling induced cracking. Because of these drilling induced cracks the identification of existing cracks was not always immediately obvious. The use of a larger diameter core may have reduced or avoided drill breaks within the core but would have made storage and moving difficult.

Cracks were often tight and almost hairline in appearance. Confirmation of existing cracks, if not obvious involved examining the surface deposits of each of the cracks surfaces for evidence of calcite formation. The existence of calcite on the joint/cracks surface was taken as an indicator of water flow through the crack at one point in time. The alternate method used to identify possible cracks while drilling was feedback from the drillers on any loss of drilling fluid/water. This was taken as a sign of a possible crack or void within the hole.

Identified cracks were logged noting the level/amount of calcite on surfaces, degree of weathering (if any) and the apparent tightness of joint. Each crack was photographed and its position (depth, hole number) also noted.

## CONCRETE CONDITION

Concrete taken from all 15 coreholes was generally uniform in composition (sand particle size, aggregate size, and distribution and material/rock type used) throughout the cores retrieved. There were slight variations in the composition (size and distribution of aggregate) at some points of the retrieved samples (possibly an indication of a separate lift), but this varied only slightly and was not significant enough to make a definitive difference to the overall appearance or description of the cores.

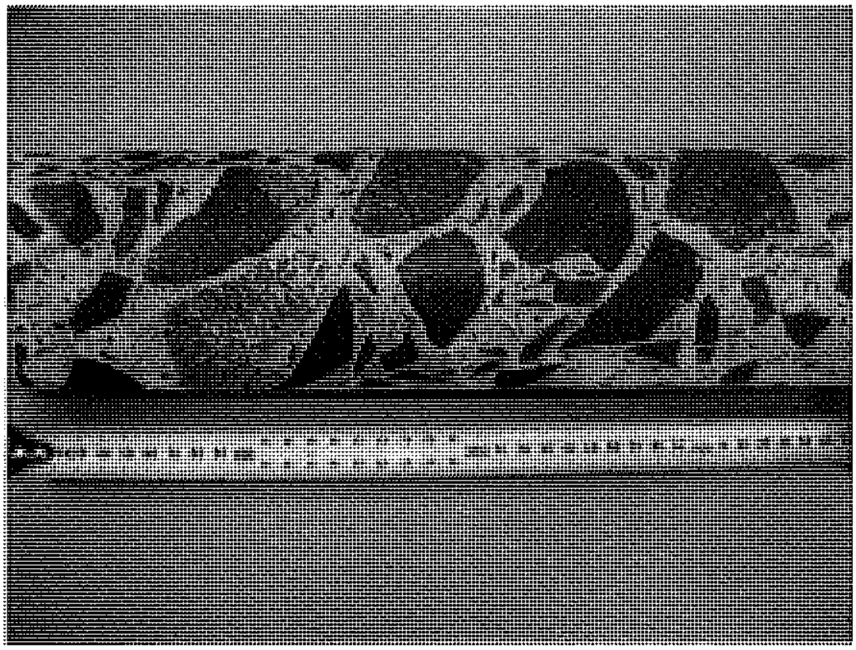


**Photo 2.** Photograph of 100mm diameter core.

The following note gives a general overall description of the concrete core retrieved.

- CONCRETE: medium to coarse sub angular to sub rounded sand, fine to coarse angular gravel with cobble sized inclusions (up to 100mm in some cases). Evidence of small pores/voids (1-5mm in size), aggregate a mixture of locally founded materials/rock with the majority of it being grey wacke/quartzite with the occasional inclusion of chert and other minor components.

The overall concrete condition appears to be excellent, with the surfaces of lifts difficult to detect from extracted cores and only minor voids visible.

## COMPLETION OF HOLES

On completion of holes the initial proposal for the completed holes was to install appropriately sized gattic covers that would be flush mounted on the surface to cover the hole left behind after coring. The decision was made onsite by SEQ water's project manager, Barton Maher to instead backfill the holes on completion with grout. This back filling was done on the final two days of the job and due to the existence of cracks within several of the hole there were some difficulties in filling these problem holes. To aid in back filling it was decided on two of the problem holes to use the larger diameter cores taken from the top to plug the holes and avoid any further loss of grout mix into any cracks or voids.

Prior to backfilling all holes were surveyed as previously stated using the "EZ-TRAC" system and their azimuth and inclination taken at two points in each hole as per requirements.

## VARIATIONS FROM ORIGINAL PROPOSAL

The initial proposal was for the drilling of 16 holes within monoliths F,G,R and S with the holes to be taken down to a depth of between 17 and 22m. This plan had to be revised on site due to constraints with respect to the dam's geometry, location of internal features (i.e. gallery, internal stairwells etc) and drilling practicalities.

The plan to drill 4 holes in monoliths "G" and "R" required drilling within the central core area (spillway area) with difficult access and working space constraints for the drill rigs and operators (hence safety issues).

Because of these issues it was decided by SEQ water's and SMEC's representative on site to reduce from 16 holes to 12 holes with allowance for adding holes if required. This meant that the revised plan would have two holes in each of the three bays radiating out from the central core area on each side, giving 4 in both "F" and "S" monoliths and two in each of "E" and "T" monoliths.

The existence of the stairwell from the upper inspection gallery down to the lower inspection galley in monoliths E and T meant that it was not possible to drill two vertical holes (in these two monoliths) to the required depth. It was therefore decided to omit the outermost vertical and angled holes for both from the plan (leaving just two holes within each of monolith T and E).

### *Interception of observation Gallery*

Hitting of the gallery while drilling holes CHTA1 and CHEA2 in monoliths "T" and "E" (due to selection of too small an angle) and the abandonment of core hole CHEA1 at 5m in depth (due to interception of large crack/void) added three holes on top of the revised number of 12 holes giving a total of 15 holes.

Holes CHTA1 and CHEA2 both intercepted the upper observation gallery at identical positions (top corner of the downstream gallery wall). Drilling was ceased immediately and gallery inspected for any damage and drill cuttings.

**Photo 3.** Photo showing CHEA1 intercepting upper observation gallery

## TESTING

No allowance was made for testing within the original proposal. 100mm cores taken from the top of each core hole using the diatube drill bit were removed from site by SMEC's onsite representative for possible strength testing at a later date if required.

## RESULTS

Results of observation and findings (photographs, location of identified cracks) are detailing in the attached Appendices.

Regards,

Roa Ginn

Geotechnical Engineer

Andrew Houghton

Geotechnical Manager

Appendix:

A: Hole details (including photographs of core and cracks)

B: Drawings showing Dam and hole locations in relation to upper observation gallery

# APPENDIX A

**RESULTS**

None = No visible signs of calcite on surfaces/ perceivable weathering

Slight = visible signs/patches of calcite build up on surface. Observable signs of weathering on surfaces

Moderate = Clearly visible signs, both surfaces have build up on them. Weathering of cement detectable visually

## CHEA1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | AZIMUTH | COMMENT |
|---|---|---|---|---|---|---|
| CHEA1 | | 5.0 | E | -67.8 | 230.5 | Terminated hole @ 5.0m due to continued loss of drilling fluids |

| HOLE ID | DEFECT TYPE | DEPTH (m) | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHEA1 | Crack/Void | 1.90 | None | None | Honeycombing | Large loss of water and drill cuttings. Possible crack linking adjacent drain holes between adjacent monoliths. Large amount of cuttings on gallery floor on inspection |

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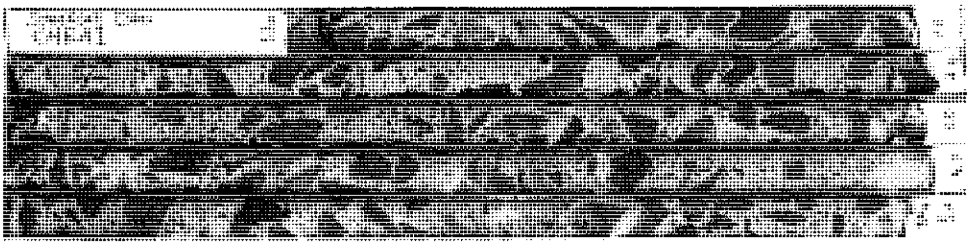
# PHOTOGRAPH OF CORE

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHEA1 |
| Project: | Somerset Dam | Coordinates E: | | Sheet No: | 1/1 |
| Feature: | | N: | | Project No: | 3003301 |
| Location: | Monolith E | Surface RL: | 107.6 | Datum: | AHD |
| | | | | Depth: | |
| | | | | Machine: | Scout |



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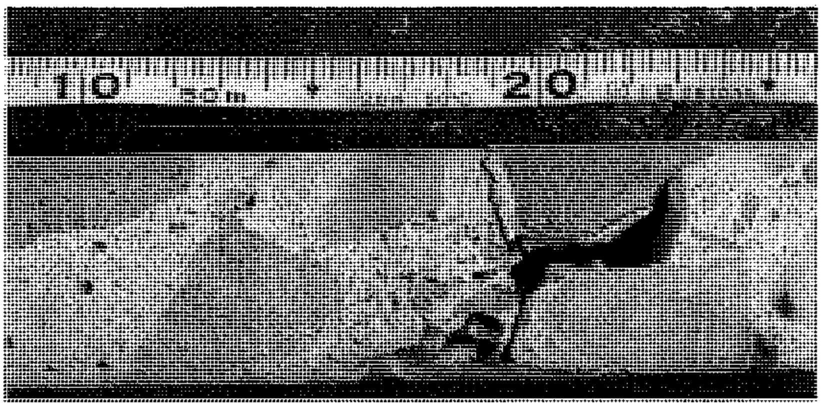
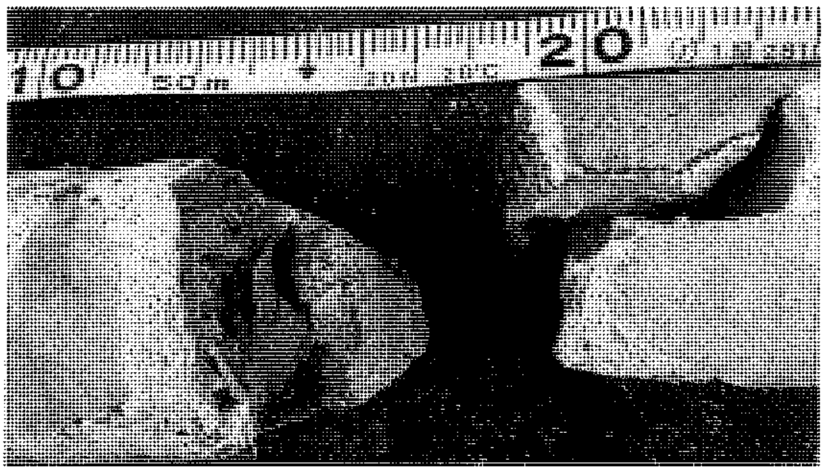
## PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHEA1 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/1 |
| **Feature:** | | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH E | **Surface RL:** | 107.6 | **Datum:** | AHD |
| | | | | **Depth:** | 1.90m |
| | | | | **Machine:** | Scout |



## CHEA2

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @18.0 | @7m | @18.0 | |
| CHEA2 | | 18.60 | E | -67.3 | -66.9 | 231.2 | 220.2 | No cracks/defects found. Ended hole at 18.60m due to intercepting top corner of upper observation gallery. Approximately 20cm short of gallery crack. |

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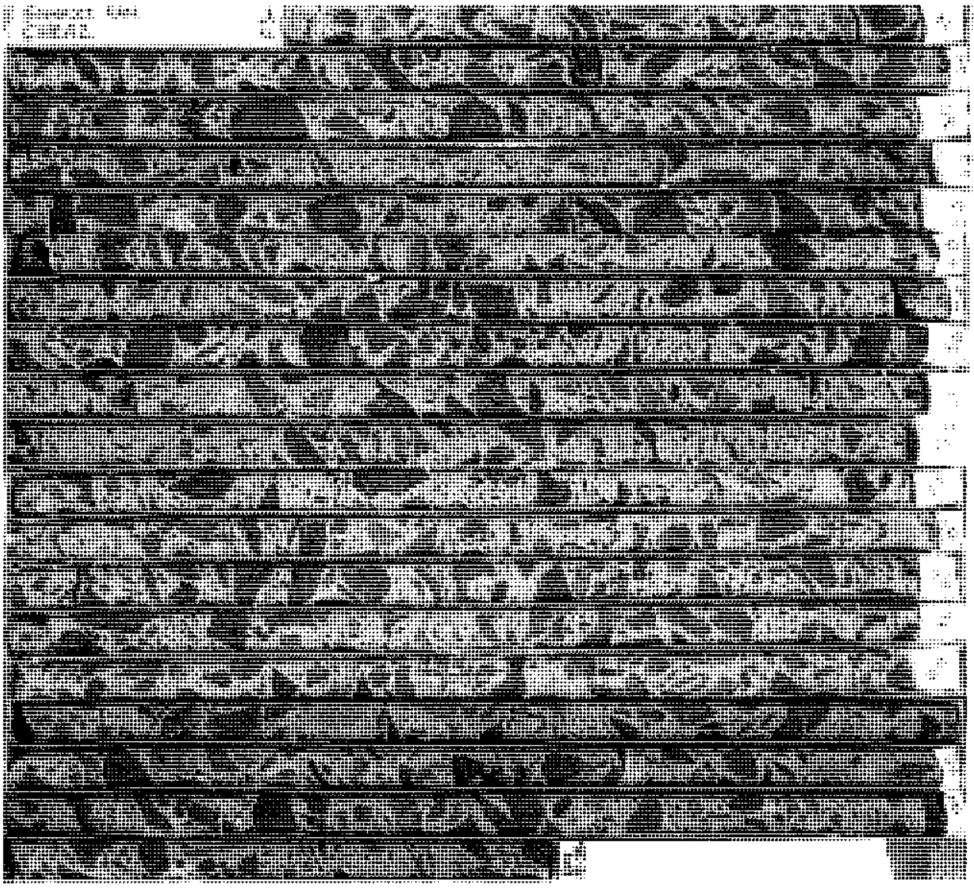
**PHOTOGRAPH OF CORE**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHEA2 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/1 |
| **Feature:** | | **N:** | | **Project No:** | 3003301 |
| **Location:** | Monolith E | **Surface RL:** 107.6 | **Datum:** AHD | **Depth:** | |
| | | | | **Machine:** | Scout |



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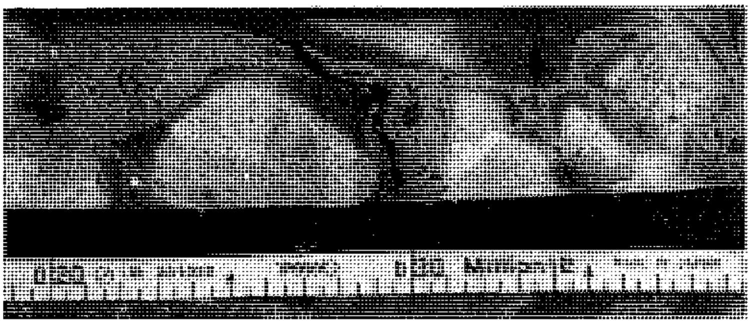
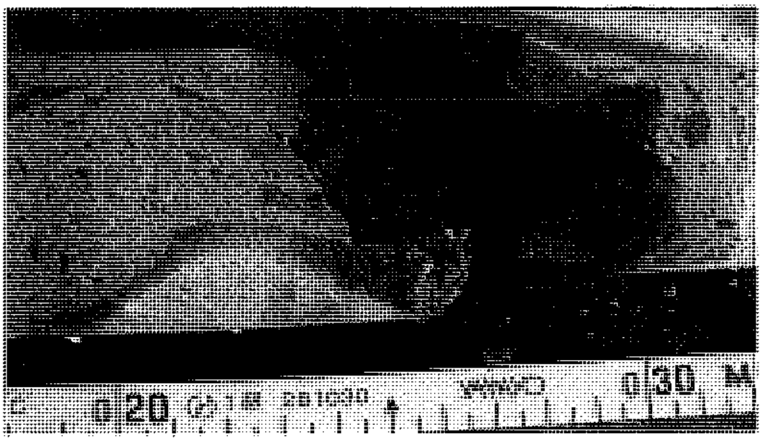
**PHOTOGRAPH OF CORE CRACKS**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHEA2 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/1 |
| **Feature:** | Crack | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH E | **Surface RL:** 107.6 | **Datum:** AHD | **Depth:** | 13.20m |
| | | | | **Machine:** | Scout |





## CHEV1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @21.79 | @7m | @21.79 | |
| CHEV1 | | 21.79 | E | -90 | -90 | NA | NA | No cracks/defects found |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHEV1 | Crack | 11.70 | Moderate | Slight | Tight | Possible lift surface, calcite visible on both surfaces |

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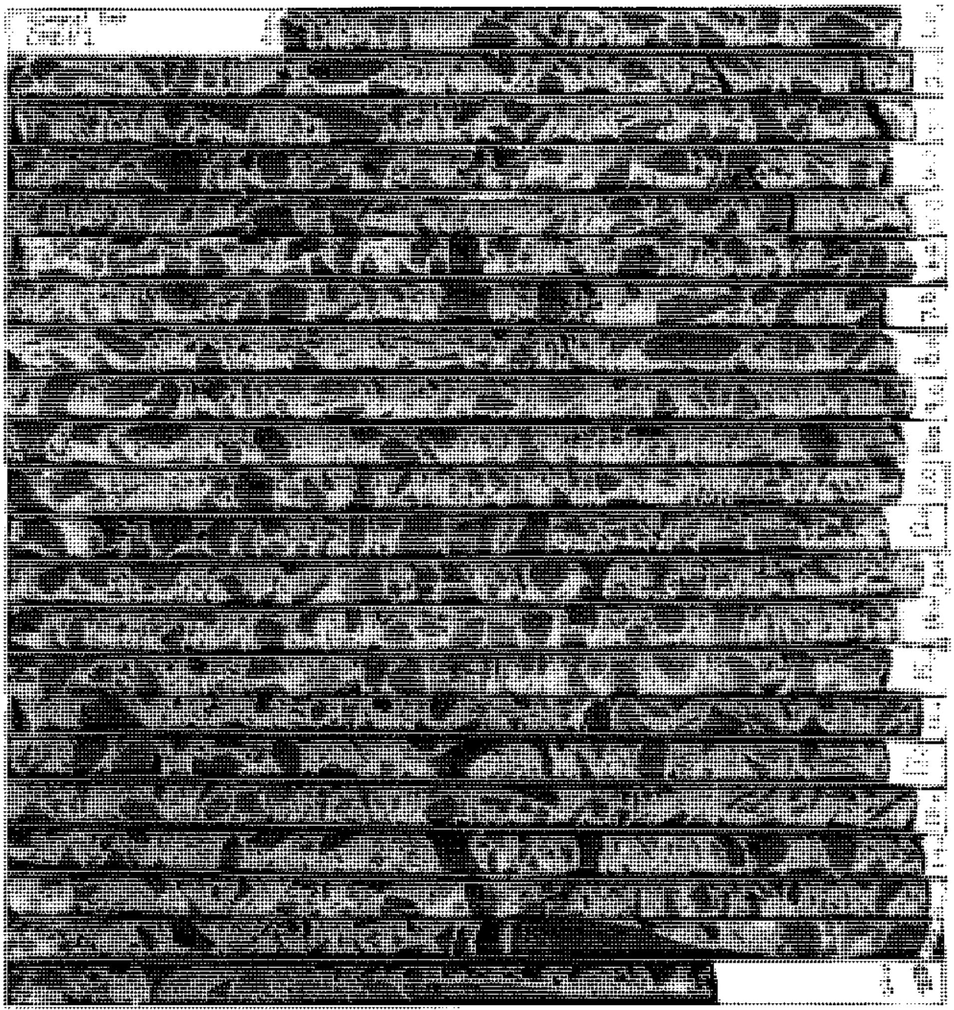
**PHOTOGRAPH OF CORE**

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | | Core Hole: CHEV1 |
| | | | | | Sheet No: 1/1 |
| | | | | | Project No: 3003301 |
| Project: | Somerset Dam | Coordinates | E: | | |
| Feature: | | | N: | | Depth: |
| Location: | Monolith E | Surface RL: | 107.6 | Datum: AHD | Machine: Scout |



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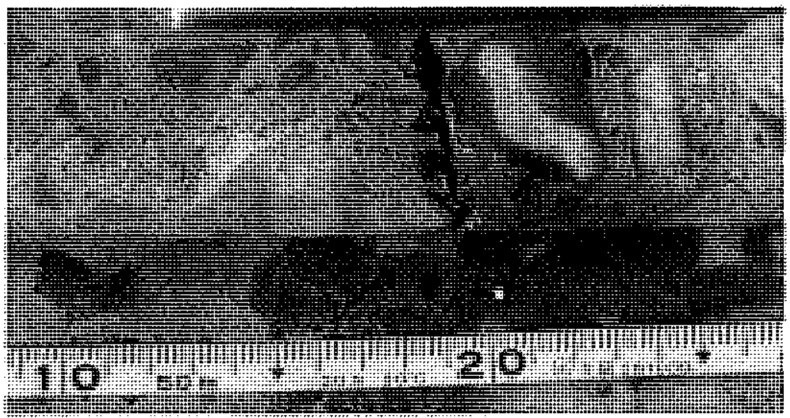
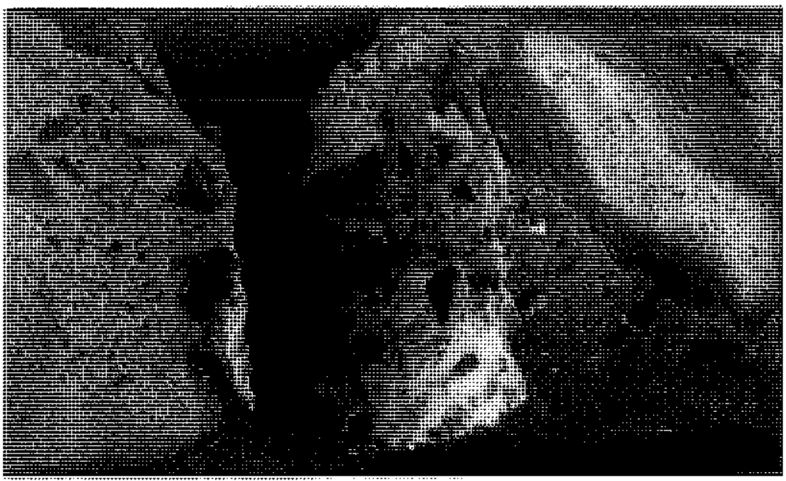
**PHOTOGRAPH OF CORE CRACKS**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHEV1 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/1 |
| **Feature:** | Crack | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH E | **Surface RL:** | 107.6 | **Depth:** | 11.70m |
| | | **Datum:** | AHD | **Machine:** | Scout |



## CHFA1

| HOLE ID | CHAINAGE | HOLE DEPTH (m) | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @22.90m | @7m | @22.90m | |
| CHFA1 | | 22.90 | F | -66.6 | -66.0 | 235.3 | 219.1 | |

| HOLE ID | DEFECT TYPE | DEPTH (m) | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHFA1 | Crack | 11.10 | Slight | Moderate | Loose | 10mm at its widest, infilled with weathered cement, slight sandy feel when rubbed, no signs of calcite on surface. loss of water after rods removed. |
| CHFA1 | Crack | 12.42 | None | moderate | Loose/Fragmented | Possible lift surface, fragmented pieces in joint/crack |
| CHFA1 | Crack/void | 14.4 | None | Slight | Loose/Fragmented | Possible lift surface, voids/hole showing calcite deposits on surfaces of void. Fragmentation possibly due to drilling. |
| CHFA1 | Crack | 18.8 | Slight | Slight | Loose-tight | Loss of water on run 18.3 to 19.85, surfaces of joint feel sandy, possibly signs of slight weathering. |
| CHFA1 | Crack | 20.10 | Slight | None | Fragmented | Possible lift surface, fragmented pieces in crack. Crack approximately 30mm at its widest. |

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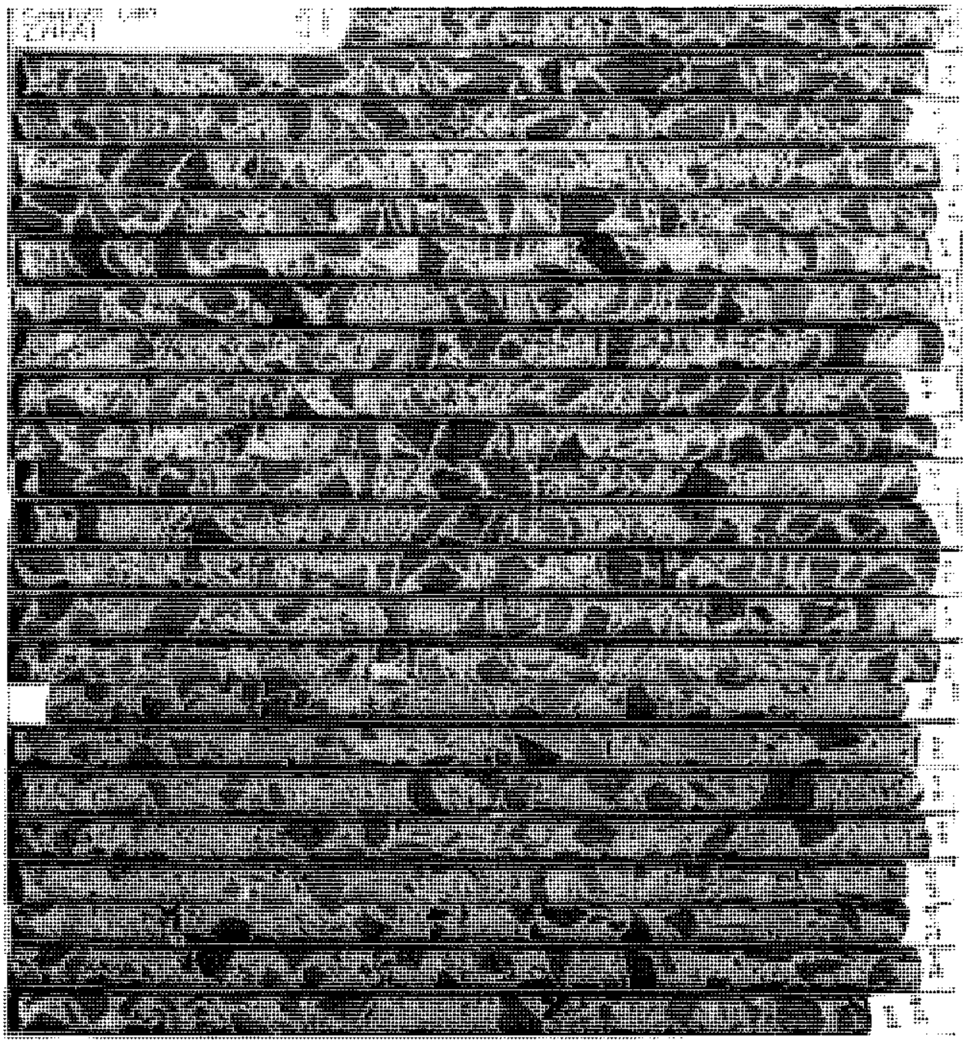
# PHOTOGRAPH OF CORE

| | | | | | |
|---|---|---|---|---|---|
| Core Hole: | CHFA1 | | | | |
| Sheet No: | 1/1 | | | | |
| Project No: | 3003301 | | | | |
| Client: | SEQ water | Coordinate System: | | | |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | | N: | | Depth: | |
| Location: | Monolith F | Surface RL: | 107.6 | Datum: AHD | Machine: Scout |



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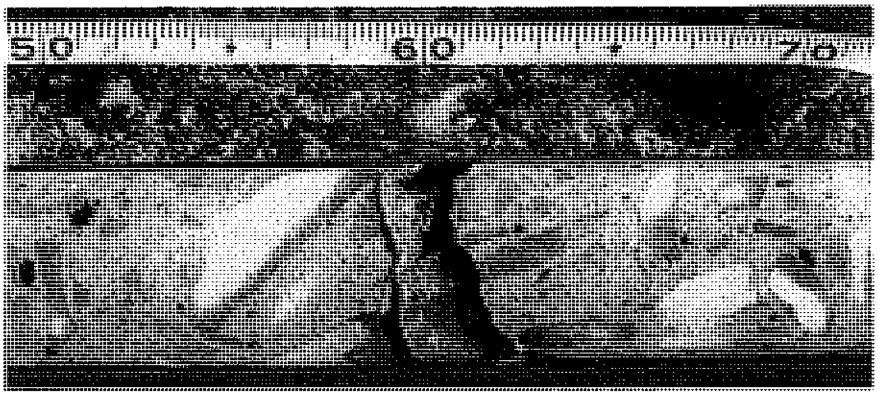
## PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHFA1 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/4 |
| **Feature:** | Crack | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH F | **Surface RL:** 107.6 | **Datum:** AHD | **Depth:** | 11.10m |
| | | | | **Machine:** | Scout |



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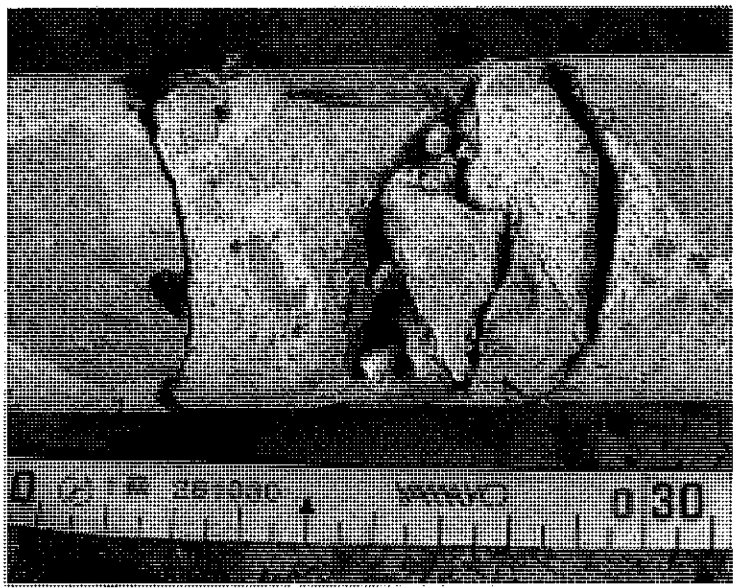
## PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHFA1 |
| | | | | **Sheet No:** | 2/4 |
| | | | | **Project No:** | 3003301 |
| **Project:** | Somerset Dam | **Coordinates E:** | | | |
| **Feature:** | Crack | **N:** | | **Depth:** | 12.42m |
| **Location:** | MONOLITH F | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** | Scout |



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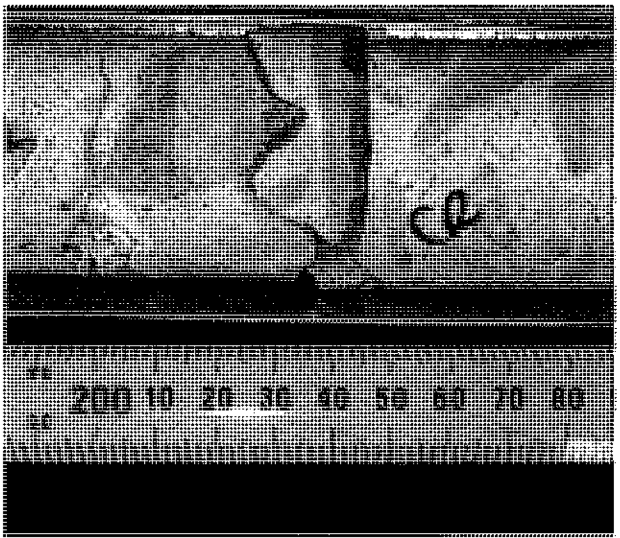
## PHOTOGRAPH OF CORE CRACKS

| | | | |
|---|---|---|---|
| **Core Hole:** CHFA1 | | | |
| **Sheet No:** 3/4 | | | |
| **Project No:** 3003301 | | | |
| **Client:** SEQ water | **Coordinate System:** | | |
| **Project:** Somerset Dam | **Coordinates E:** | | |
| **Feature:** Crack | **N:** | | **Depth:** 14.40m |
| **Location:** MONOLITH F | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** Scout |

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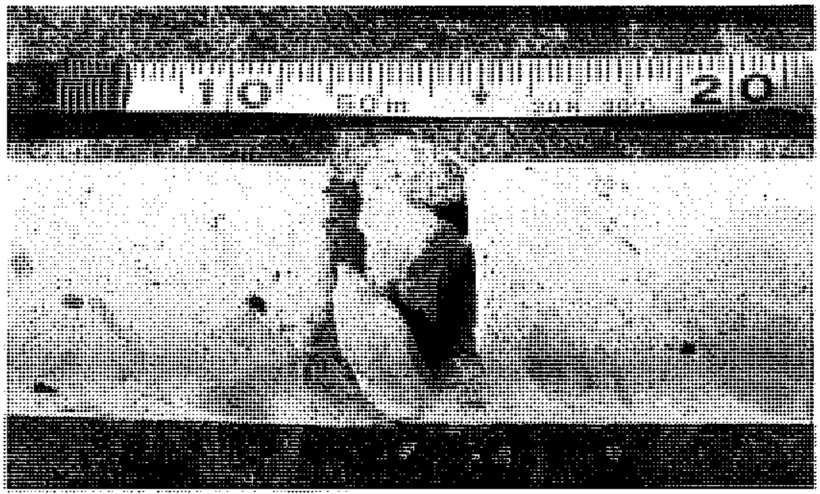
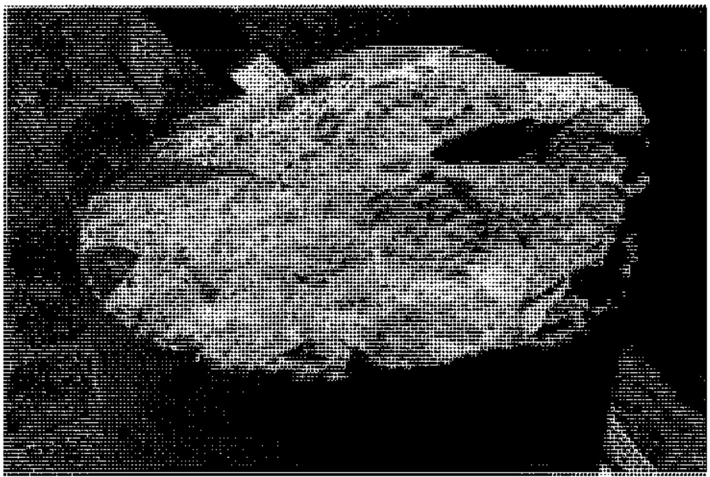
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHFA1 |
| | | | | **Sheet No:** | 4/4 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Project No:** | 3003301 |
| **Feature:** | Crack | **N:** | | **Depth:** | 20.10m |
| **Location:** | MONOLITH F | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** | Scout |



## CHFV1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @21.25 | @7m | @21.25 | |
| CHFV1 | | 21.25 | F | -90 | -90 | NA | NA | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHFV1 | Crack/Void | 17.30 | None | None | Void | Honeycombing/Void from 17.23 to 17.32m, Large loss of water seeping out of 2 vertical drains coming out of the ceiling of the stairwell leading down to the lower observation gallery at the intersection between F and G monoliths. |

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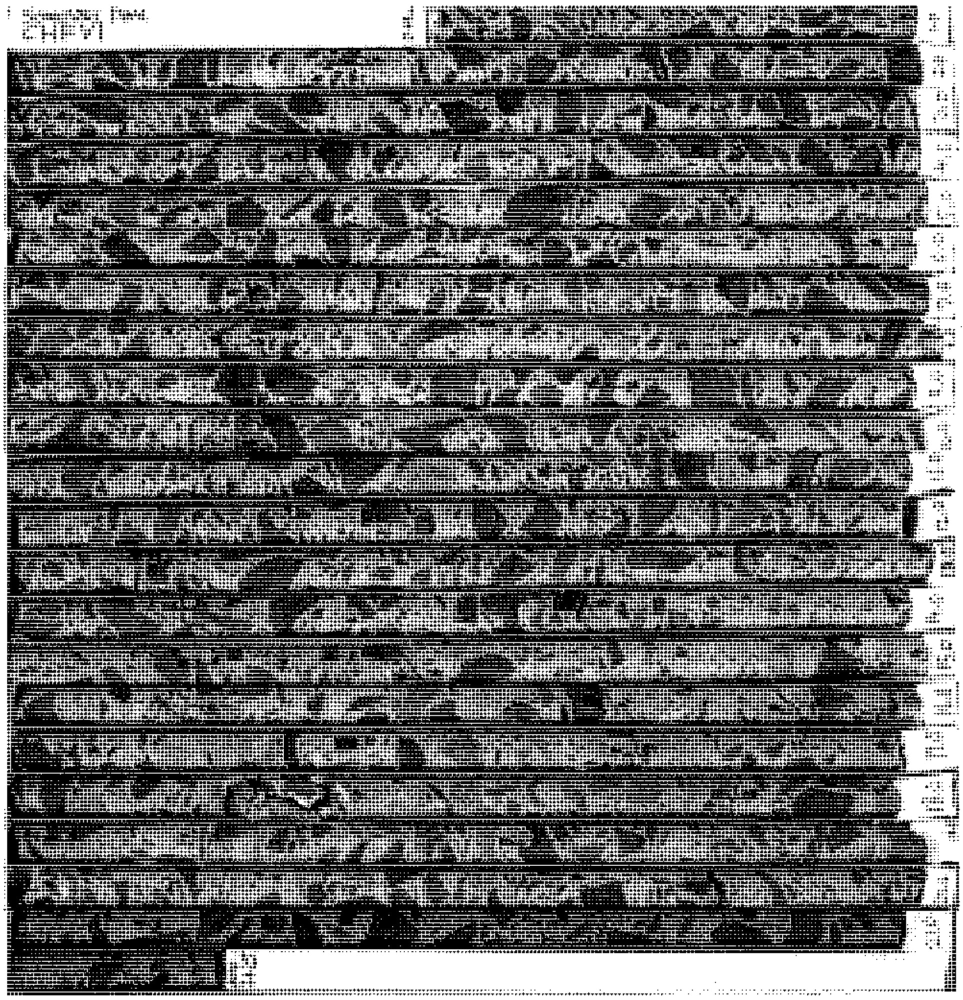
# PHOTOGRAPH OF CORE

| | | | | | |
|---|---|---|---|---|---|
| | | | | Core Hole: | CHFV1 |
| | | | | Sheet No: | 1/1 |
| Client: | SEQ water | Coordinate System: | | Project No: | 3003301 |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | | N: | | Depth: | |
| Location: | Monolith F | Surface RL: | 107.6 | Datum: | AHD |
| | | | | Machine: | Scout |



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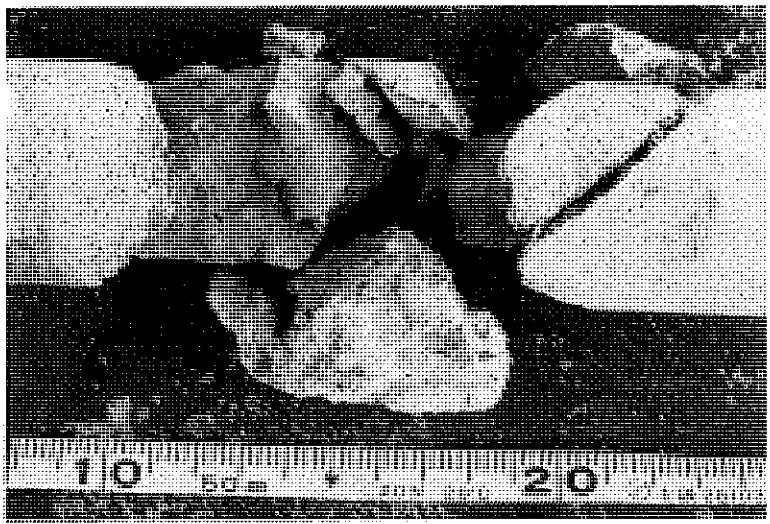
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHFV1 |
| Project: | Somerset Dam | Coordinates E: | | Sheet No: | 1/1 |
| Feature: | Crack | N: | | Project No: | 3003301 |
| Location: | MONOLITH F | Surface RL: 107.6 | Datum: AHD | Depth: | 17.30m |
| | | | | Machine: | Scout |



## CHFA2

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @22.93 | @7m | @22.93 | |
| CHFA2 | | 22.93 | F | -65.1 | -64.4 | 218.9 | 218.1 | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHFA2 | Inclusion | 11.05 | NA | NA | NA | Wood inclusion found embedded in core. |
| CHFA2 | Crack | 11.15 | None | Moderate | Slightly open | 10mm at its widest, infilled with weathered cement, slight sandy feel when rubbed, no signs of calcite on surface. Large loss of water after rods removed. Water seen seeping out of down stream face of dam approximately 5m in width and at same RL as drilling depth. |
| CHFA2 | Crack | 14.53 | moderate | Slight | Tight | Possible lift surface, signs of calcite of crack surfaces, loose appearance of crack. Loose pieces of cement in split, possibly from drilling. |
| CHFA2 | Crack | 20.40 | None | Moderate | Slightly open | 10mm at it's widest, infilled with weathered cement, slight sandy feel when rubbed, no signs of calcite on surface. |

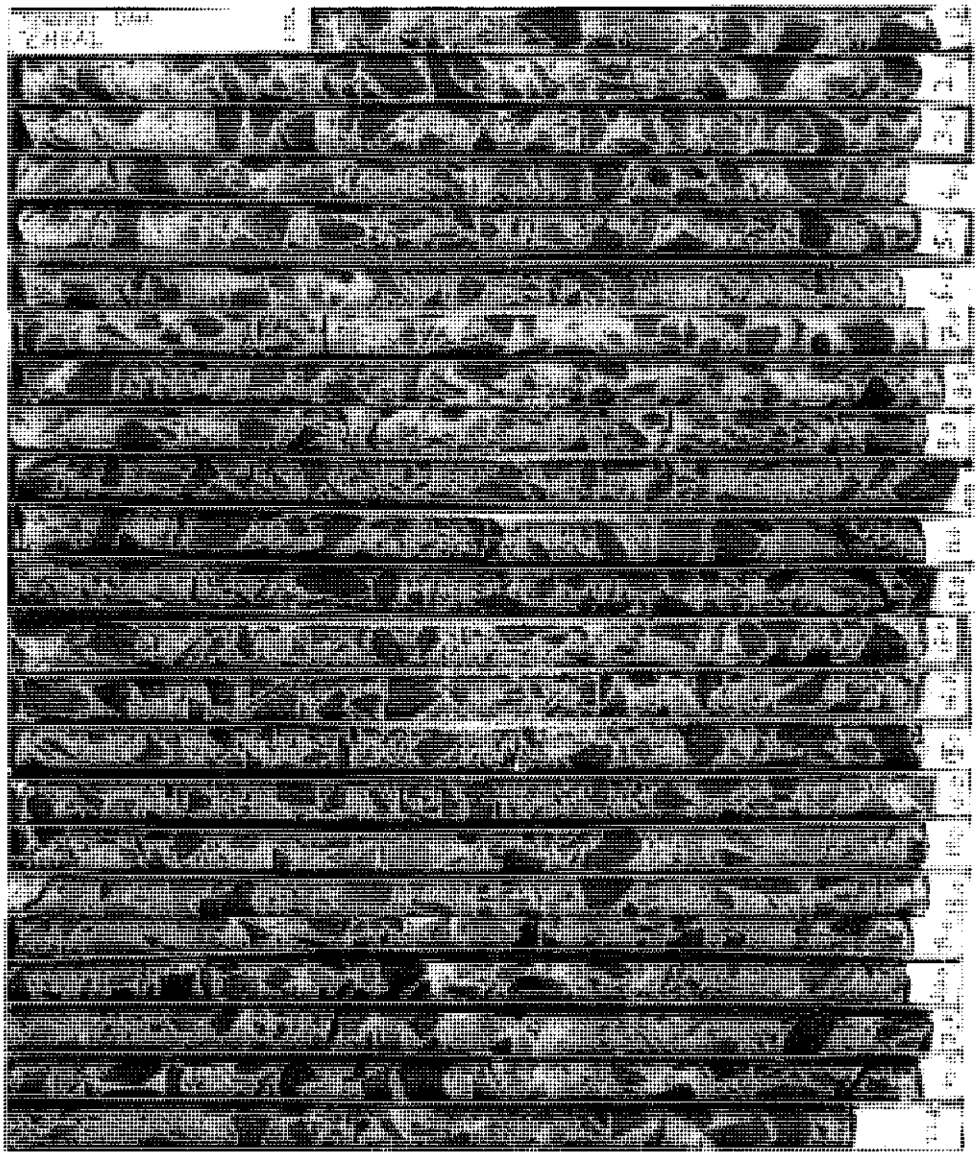
| SMEC<br>SMEC Australia Pty Ltd | PHOTOGRAPH OF CORE | | Core Hole: CHFA2<br>Sheet No: 1/1<br>Project No: 3003301 |
|---|---|---|---|

**Client:** SEQ water  **Coordinate System:**

**Project:** Somerset Dam  **Coordinates** E:

**Feature:**  N:  **Depth:**

**Location:** Monolith F  **Surface RL:** 107.6  **Datum:** AHD  **Machine:** Scout



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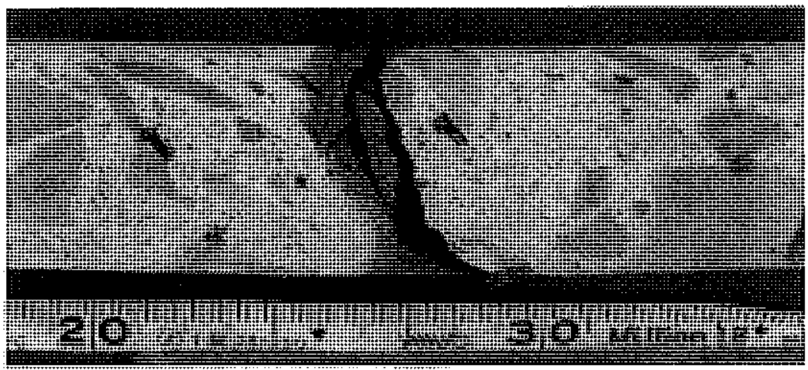
# PHOTOGRAPH OF CORE CRACKS

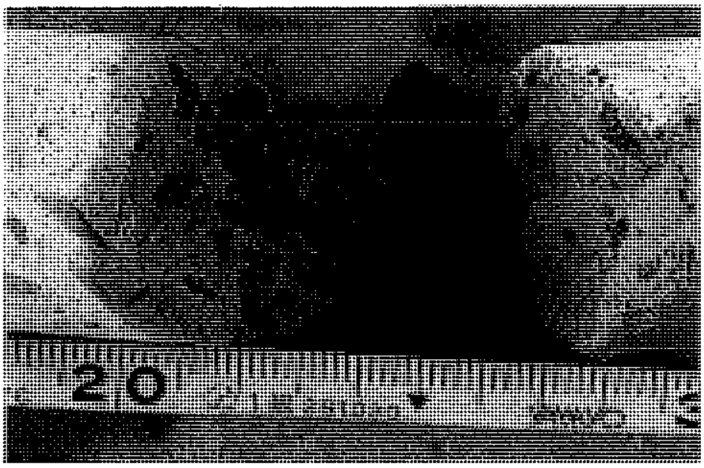
**Core Hole:** CHFA2
**Sheet No:** 1/3
**Project No:** 3003301

**Client:** SEQ water
**Project:** Somerset Dam
**Feature:** Crack
**Location:** MONOLITH F

**Coordinate System:**
**Coordinates E:**
**N:**
**Surface RL:** 107.6 **Datum:** AHD

**Depth:** 11.15m
**Machine:** Scout



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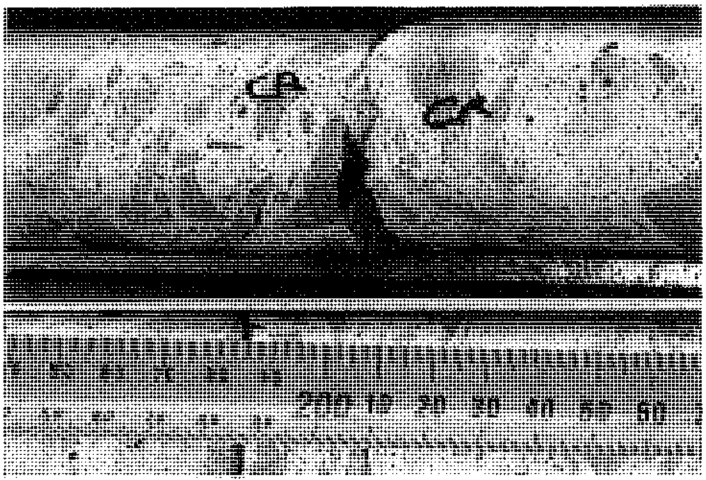
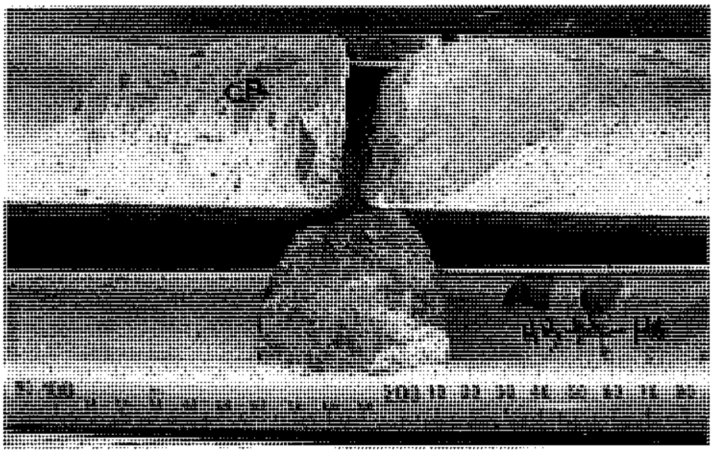
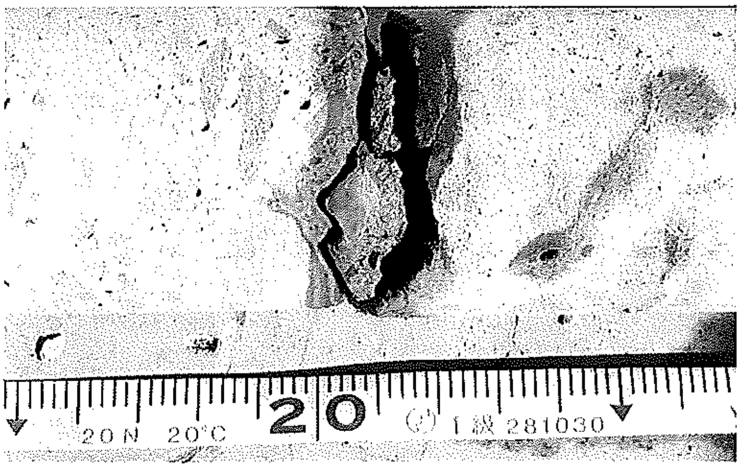
## PHOTOGRAPH OF CORE CRACKS

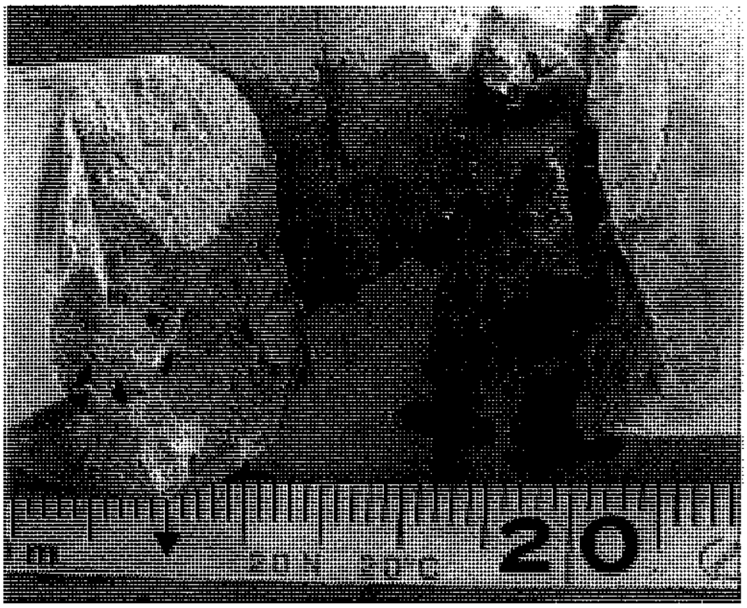
| | | | | | |
|---|---|---|---|---|---|
| **Core Hole:** CHFA2 | | | | | |
| **Sheet No:** 2/3 | | | | | |
| **Project No:** 3003301 | | | | | |
| **Client:** | SEQ water | **Coordinate System:** | | | |
| **Project:** | Somerset Dam | **Coordinates** | **E:** | | |
| **Feature:** | Crack | | **N:** | | **Depth:** 14.53m |
| **Location:** | MONOLITH F | **Surface RL:** | 107.6 | **Datum:** AHD | **Machine:** Scout |





| SMEC SMEC Australia Pty Ltd | PHOTOGRAPH OF CORE CRACKS | | Core Hole: CHFA2 |
|---|---|---|---|
| | | | Sheet No: 3/3 |
| | | | Project No: 3003301 |
| **Client:** SEQ water | **Coordinate System:** | | |
| **Project:** Somerset Dam | **Coordinates** **E:** | | |
| **Feature:** Crack | **N:** | | **Depth:** 20.40m |
| **Location:** MONOLITH F | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** Scout |



## CHFV2

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @21.33 | @7m | @21.33 | |
| CHFV2 | | 21.33 | F | -90 | -90 | NA | NA | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHFV2 | Crack | 11.73 | Moderate | Moderate | Tight | Calcite deposits and powdery feel on both surfaces. Visually tigh, undulating, rough crack surface. |
| CHFV2 | Crack | 14.45 | None | Slight | Tight | No visible calcite on surfaces of crack, slightly weathered. |
| CHFV2 | Crack/Void | 15.46 | Slight | Slight | Tight | Possible lift surface. Fractured material around crack visible when split placed on ground. Evidence of honeycombing and voids. Hairline crack surrounding void. |

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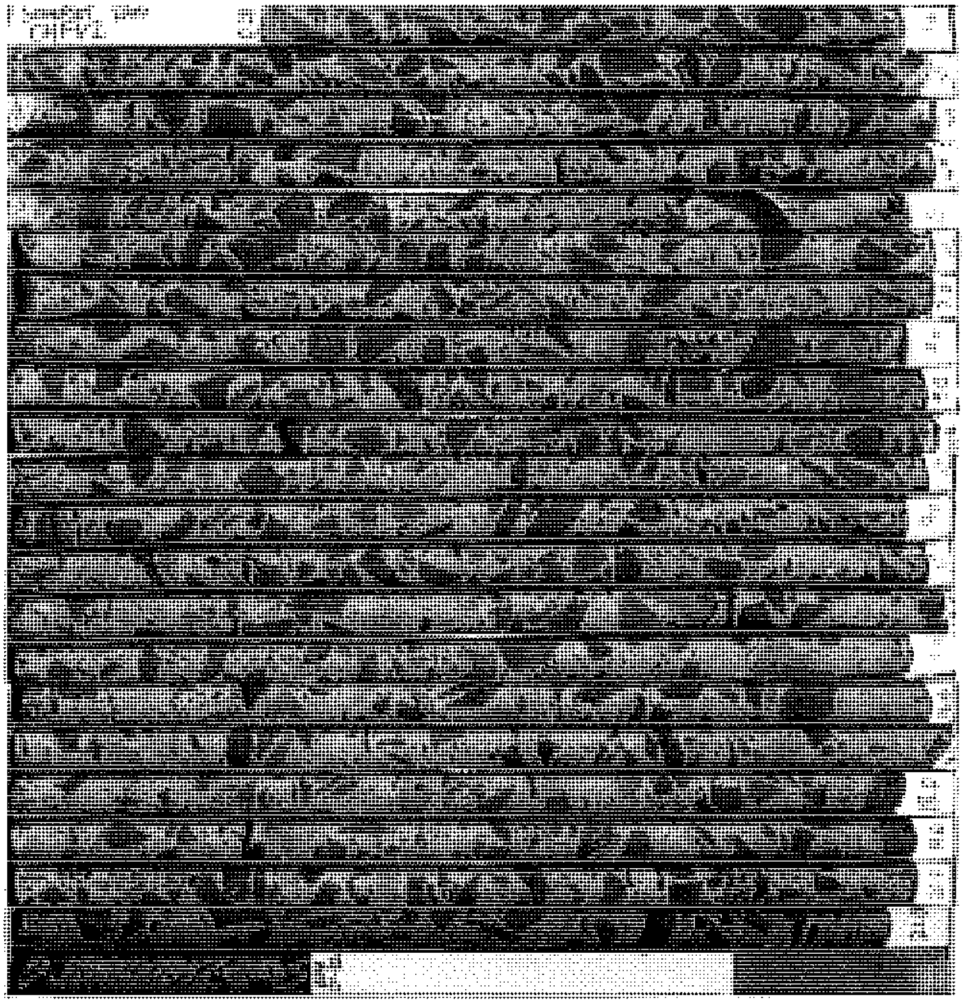
**PHOTOGRAPH OF CORE**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHFV2 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/1 |
| **Feature:** | | **N:** | | **Project No:** | 3003301 |
| **Location:** | Monolith F | **Surface RL:** | 107.6 | **Datum:** | AHD |
| | | | | **Depth:** | |
| | | | | **Machine:** | Scout |



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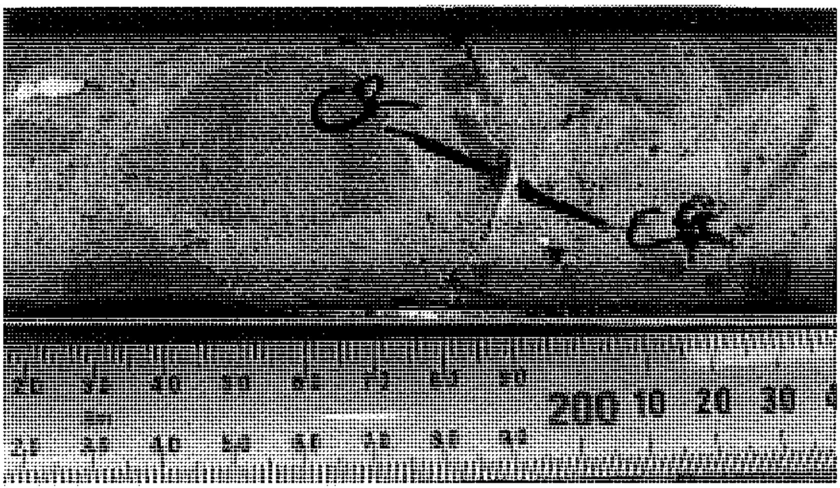
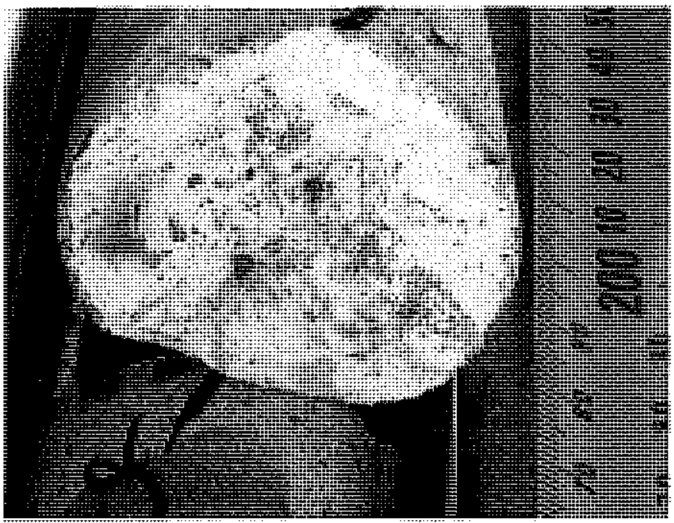
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHFV2 |
| | | | | Sheet No: | 1/3 |
| | | | | Project No: | 3003301 |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | Crack | N: | | Depth: | 11.73m |
| Location: | MONOLITH F | Surface RL: 107.6 | Datum: AHD | Machine: | Scout |



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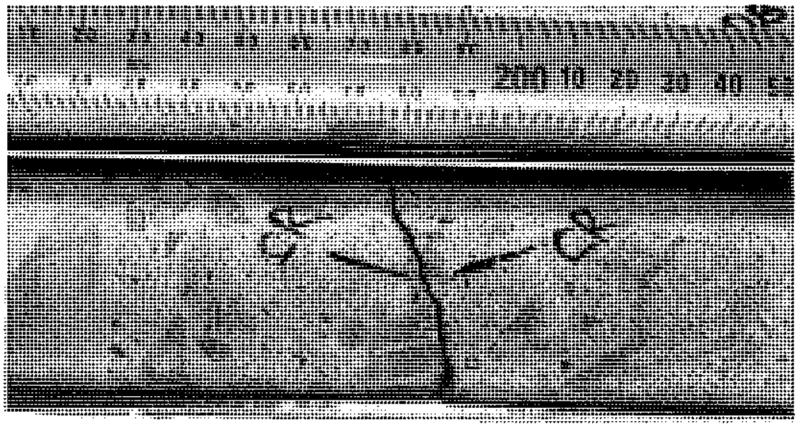
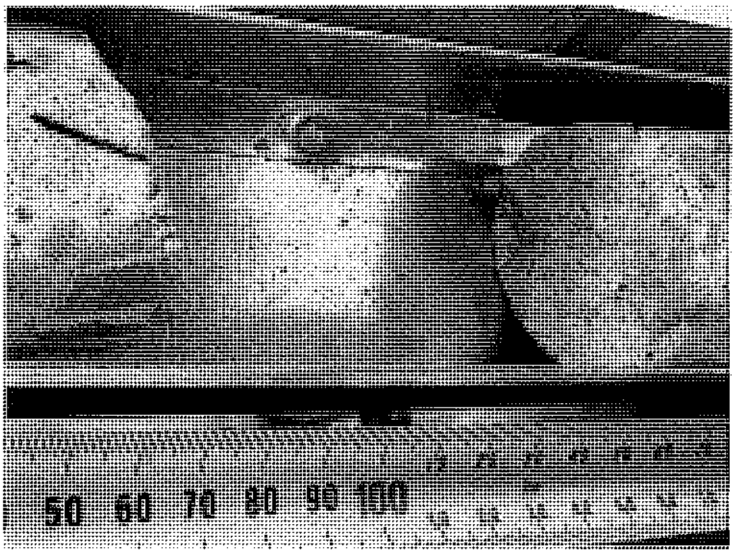
# PHOTOGRAPH OF CORE CRACKS

Core Hole: CHFV2
Sheet No: 2/3
Project No: 3003301

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | | |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | Crack | N: | | Depth: | 14.45m |
| Location: | MONOLITH F | Surface RL: 107.6 | Datum: AHD | Machine: | Scout |



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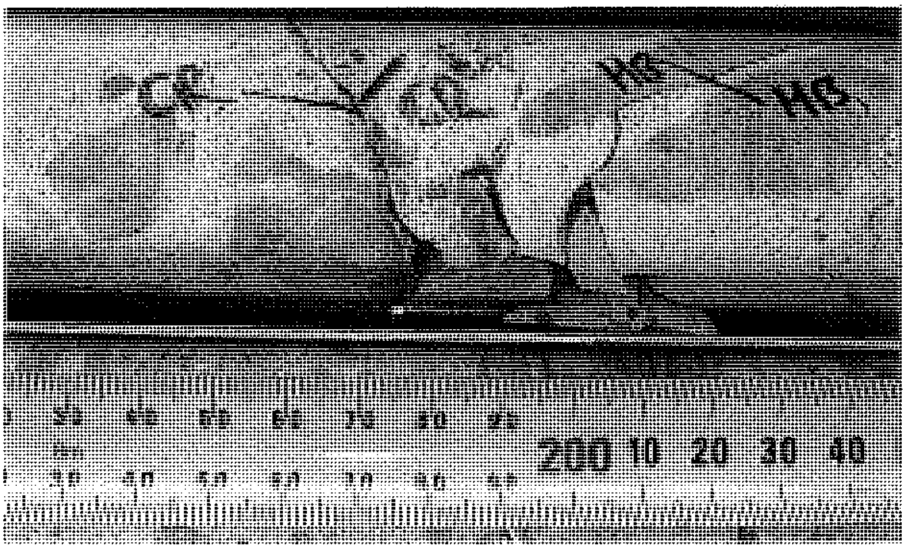
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHFV2 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 3/3 |
| **Feature:** | Crack | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH F | **Surface RL:** 107.6 | **Datum:** AHD | **Depth:** | 15.46m |
| | | | | **Machine:** | Scout |



## CHGA1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @22.62 | @7m | @22.62 | |
| CHGA1 | | 22.62 | G | -64.5 | -64.3 | 204.9 | 216.5 | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHGA1 | Crack/Void | 16.0 | Slight | Slight | Void | Possible lift surface. Signs of honeycombing/void on crack and fracturing. Loss of smaller pieces. Large loss of water when rods pulled at end of run. |
| CHGA1 | Crack/Void | 21.72 | Slight | Moderate | Void | Possible lift surface, powdering feel to crack surface. Slight evidence of calcite on lower surface of crack. |

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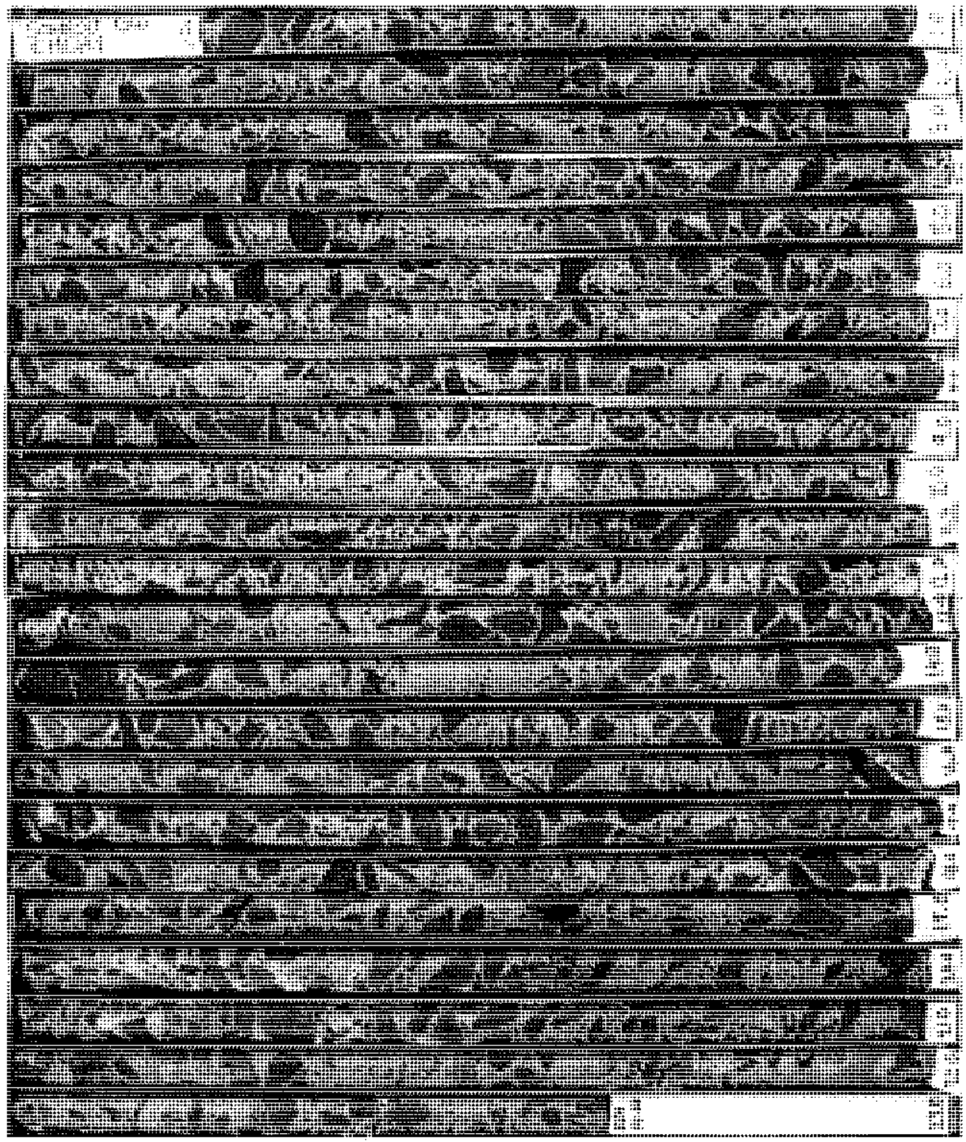
## PHOTOGRAPH OF CORE

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHGA1 |
| Project: | Somerset Dam | Coordinates E: | | Sheet No: | 1/1 |
| Feature: | | N: | | Project No: | 3003301 |
| Location: | Monolith G | Surface RL: 107.6 | Datum: AHD | Depth: | |
| | | | | Machine: | Scout |



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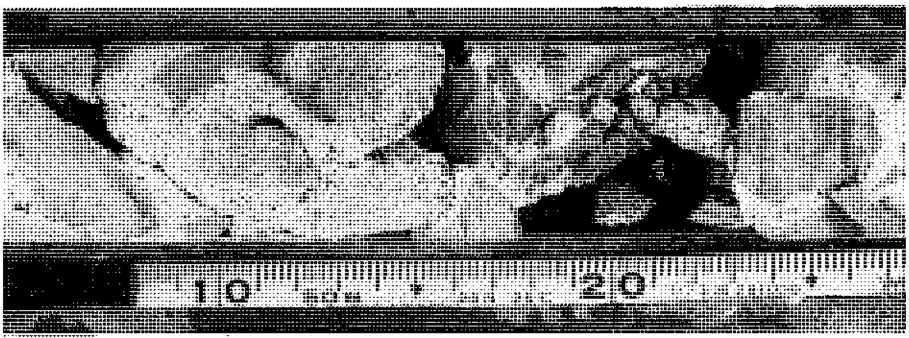
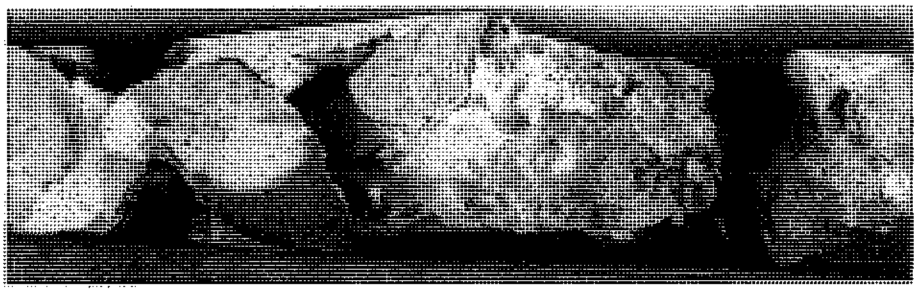
## PHOTOGRAPH OF CORE CRACKS

| | |
|---|---|
| Core Hole: | CHGA1 |
| Sheet No: | 1/2 |
| Project No: | 3003301 |

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | | |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | Crack | N: | | Depth: | 16.0m |
| Location: | MONOLITH G | Surface RL: 107.6 | Datum: AHD | Machine: | Scout |



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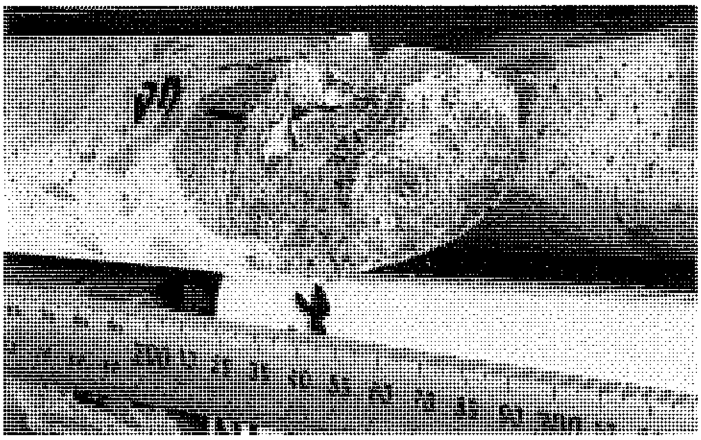
## PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHGA1 |
| Project: | Somerset Dam | Coordinates E: | | Sheet No: | 2/2 |
| Feature: | Crack | N: | | Project No: | 3003301 |
| Location: | MONOLITH G | Surface RL: 107.6 | Datum: AHD | Depth: | 21.72m |
| | | | | Machine: | Scout |



## CHSA1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @22.32 | @7m | @22.32 | |
| CHSA1 | | 22.32 | F | -70.5 | -70.3 | -219.2 | -213.9 | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHSA1 | Crack | 14.85 | Moderate | Moderate | open | 15cm long in core. Vertical open crack (approximately 20 degrees in core), Calcite on surfaces showing signs of weathering. Identical location and appearance as crack at 14.80 in CHTA1. |
| CHSA1 | Crack/Void | 18.3 | No | Mod to High | Infilled | Weathered seam of cement. Seepage evident from crack in upper observation gallery. 10-20mm at its widest point in core. |
| CHSA1 | Crack | 21.45 | Slight | Slight | Tight | Slight evidence of calcite on one side of core sample at crack location. |

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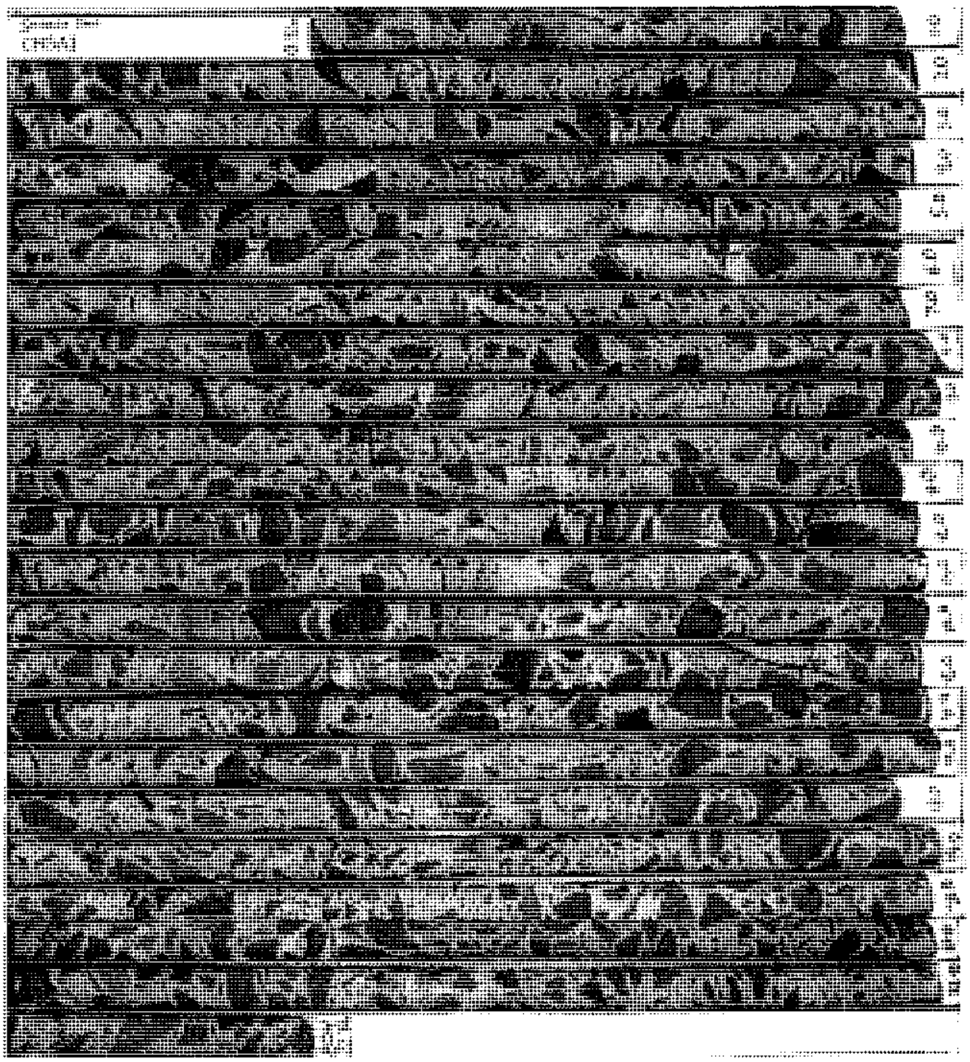
**PHOTOGRAPH OF CORE**

| | |
|---|---|
| Core Hole: | CHSA1 |
| Sheet No: | 1/1 |
| Project No: | 3003301 |

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | | |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | | N: | | Depth: | |
| Location: | Monolith S | Surface RL: | 107.6 | Datum: AHD | Machine: Scout |



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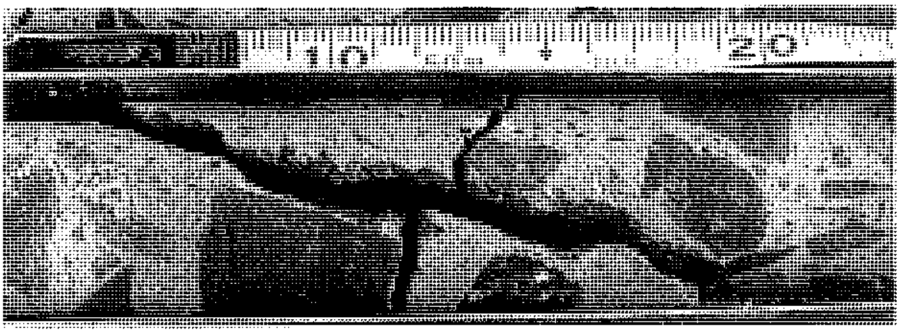
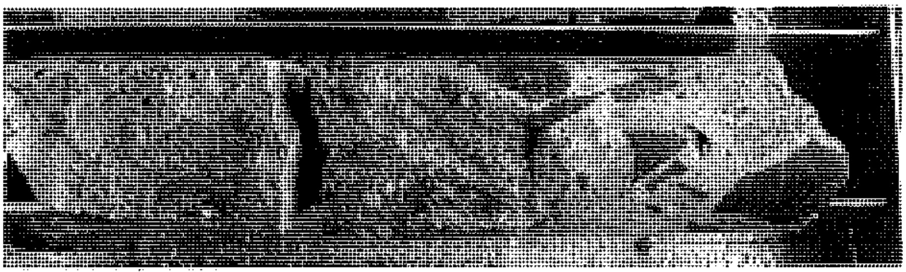
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Core Hole:** CHSA1 | | | | | |
| **Sheet No:** 1/3 | | | | | |
| **Client:** | SEQ water | **Coordinate System:** | | **Project No:** | 3003301 |
| **Project:** | Somerset Dam | **Coordinates** | **E:** | | |
| **Feature:** | Crack | | **N:** | **Depth:** | 14.85m |
| **Location:** | MONOLITH S | **Surface RL:** | 107.6 | **Datum:** AHD | **Machine:** Scout |





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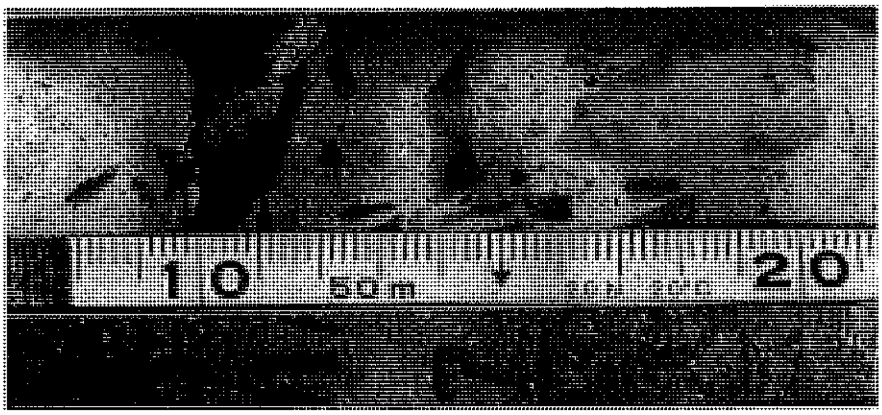
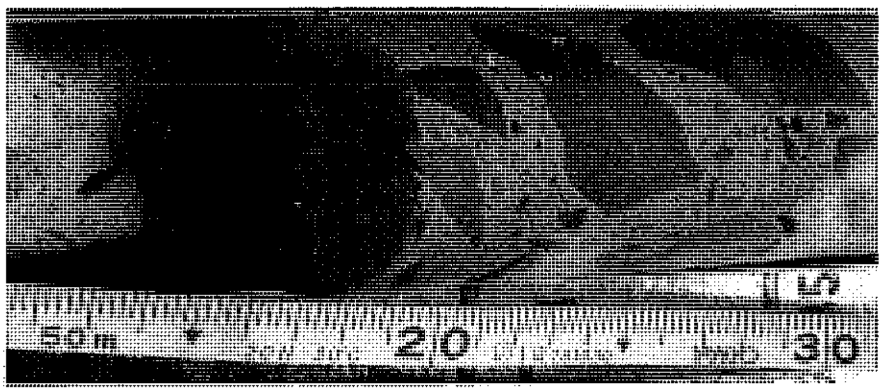
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSA1 |
| | | | | **Sheet No:** | 2/3 |
| | | | | **Project No:** | 3003301 |
| **Project:** | Somerset Dam | **Coordinates E:** | | | |
| **Feature:** | Crack | **N:** | | **Depth:** | 18.30m |
| **Location:** | MONOLITH S | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** | Scout |



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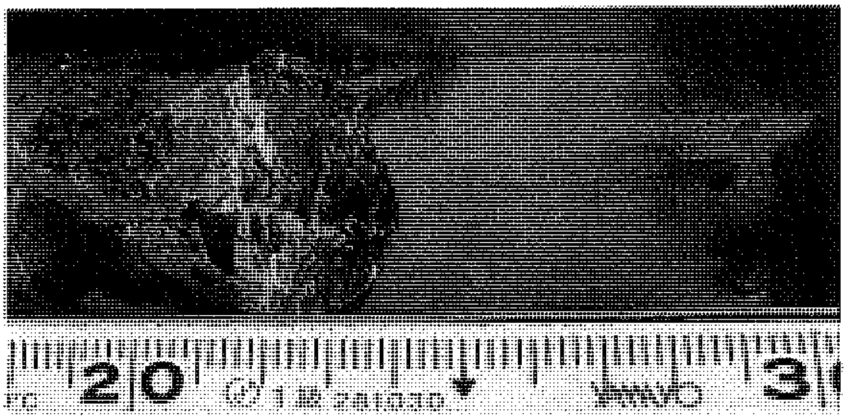
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSA1 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 3/3 |
| **Feature:** | Crack | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH S | **Surface RL:** | 107.6 | **Depth:** | 21.45m |
| | | **Datum:** | AHD | **Machine:** | Scout |



## CHSV1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @21.48 | @7m | @21.48 | |
| CHSV1 | | 21.48 | S | -90 | -90 | NA | NA | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHSV1 | Crack | 19.90 | Slight | Slight | Loose to Tight | Possible lift surface, calcite visible around aggregate. |

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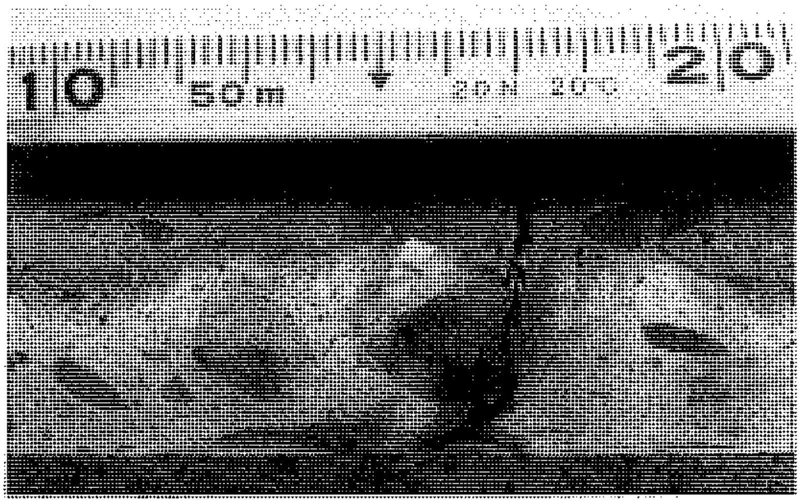
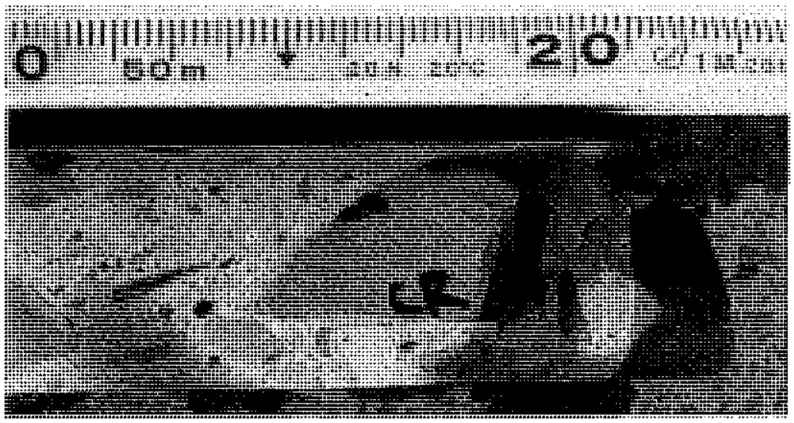
**PHOTOGRAPH OF CORE CRACKS**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSA2 |
| | | | | **Sheet No:** | 1/4 |
| | | | | **Project No:** | 3003301 |
| **Project:** | Somerset Dam | **Coordinates E:** | | | |
| **Feature:** | Crack | **N:** | | **Depth:** | 6.0m |
| **Location:** | MONOLITH S | **Surface RL:** | 107.6 **Datum:** AHD | **Machine:** | Scout |





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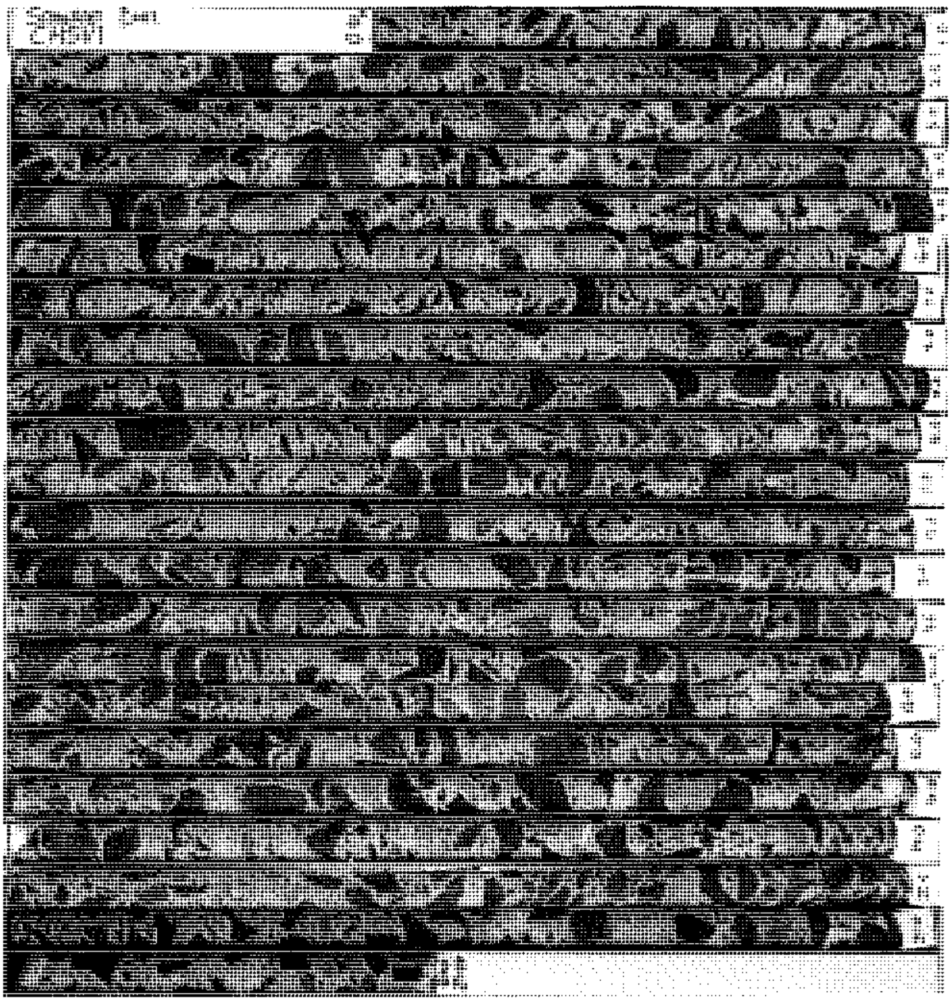
**PHOTOGRAPH OF CORE**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSV1 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/1 |
| **Feature:** | | **N:** | | **Project No:** | 3003301 |
| **Location:** | Monolith S | **Surface RL:** | 107.6 | **Depth:** | |
| | | **Datum:** | AHD | **Machine:** | Scout |



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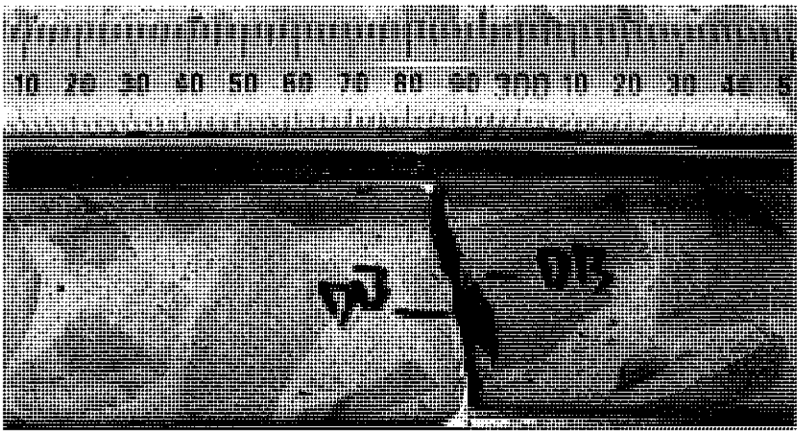
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHSV1 |
| | | | | Sheet No: | 1/1 |
| Project: | Somerset Dam | Coordinates E: | | Project No: | 3003301 |
| Feature: | Crack | N: | | Depth: | 19.90m |
| Location: | MONOLITH S | Surface RL: 107.6 | Datum: AHD | Machine: | Scout |




## CHSA2

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @22.32 | @7m | @22.32 | |
| CHSA2 | | 22.45 | S | -67.1 | -66.7 | 221.4 | 220.3 | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHSA2 | Crack | 6.0 | None | Slight-Moderate | Infilled | Weathered seam of cement 1-5mm at its widest, possible lift surface. |
| CHSA2 | Crack | 12.73 | Moderate | Slight | Tight | No obvious weathering, signs of calcite on both surfaces of joint. |
| CHSA2 | Crack | 18.05 | Slight | Slight | Tight | Slight evidence of calcite on surface. |
| CHSA2 | Crack | 18.68 | No | Mod to High | Infilled | Weathered seam of cement. 5-10mm at its widest point in core. Loose fragments from joint. |

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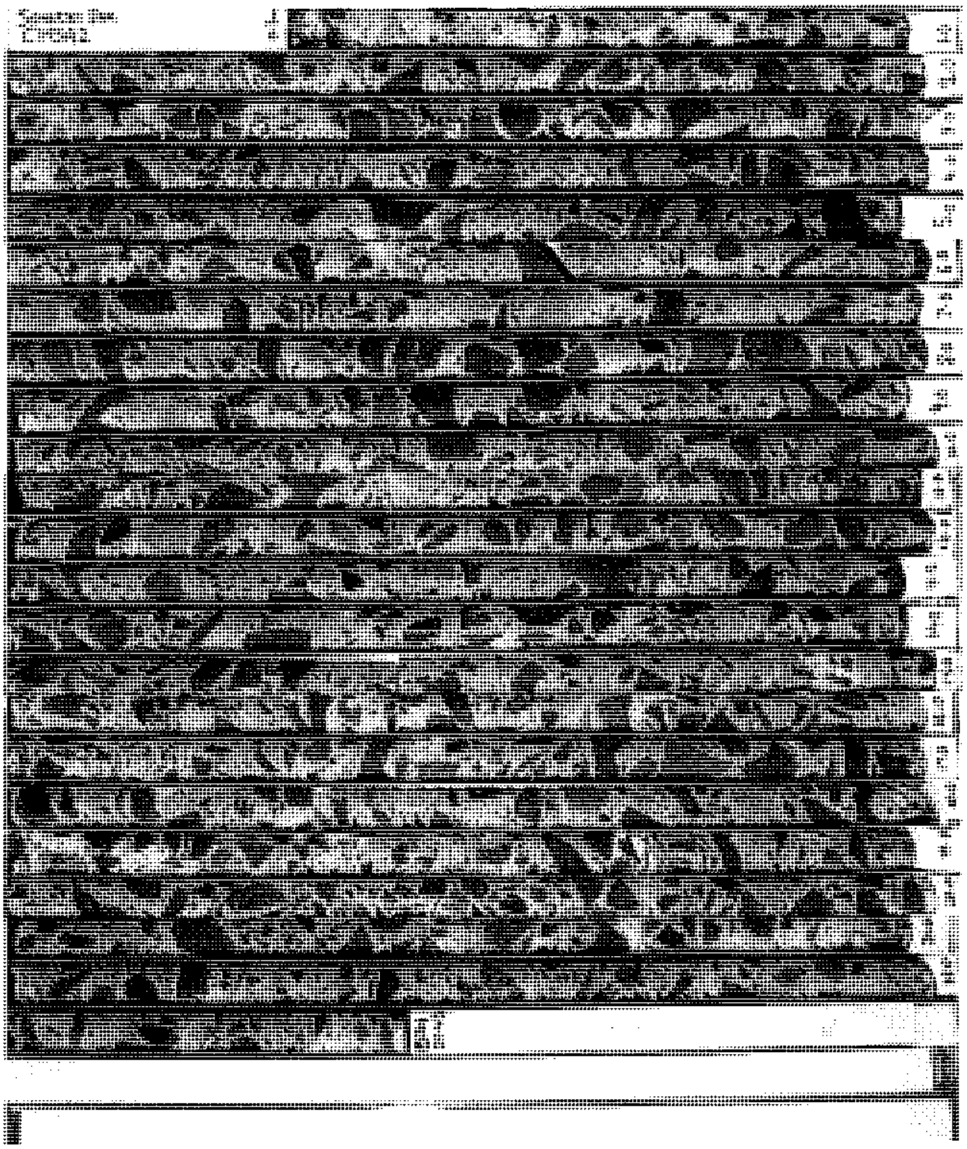
# PHOTOGRAPH OF CORE

**Core Hole:** CHSA2
**Sheet No:** 1/1
**Project No:** 3003301

**Client:** SEQ water
**Project:** Somerset Dam
**Feature:**
**Location:** Monolith S

**Coordinate System:**
**Coordinates** **E:**
**N:**
**Surface RL:** 107.6 **Datum:** AHD

**Depth:**
**Machine:** Scout



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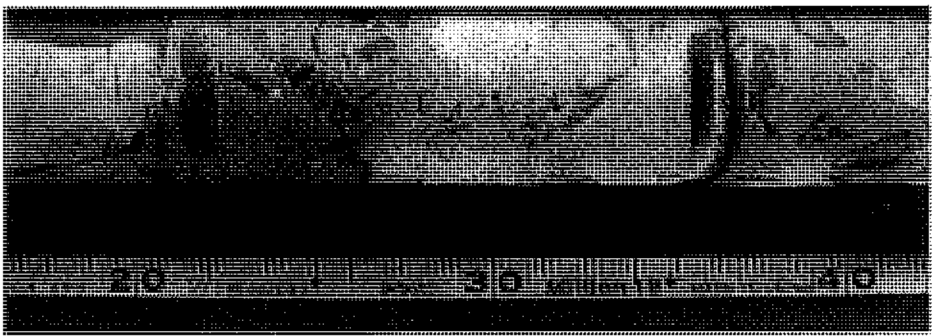
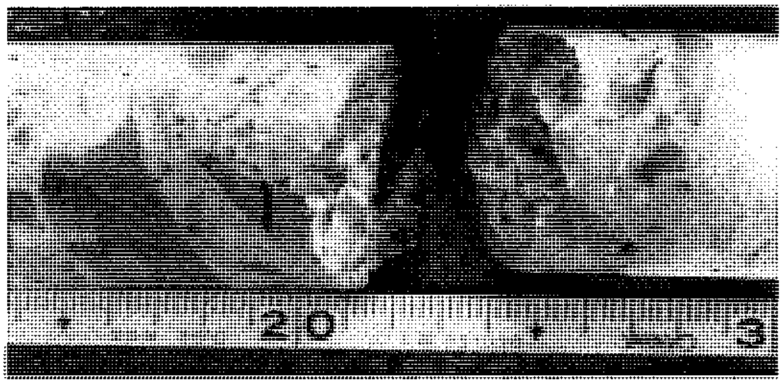
**PHOTOGRAPH OF CORE CRACKS**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSA2 |
| | | | | **Sheet No:** | 2/4 |
| **Project:** | Somerset Dam | **Coordinates** | **E:** | **Project No:** | 3003301 |
| **Feature:** | Crack | | **N:** | **Depth:** | 12.73m |
| **Location:** | MONOLITH S | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** | Scout |



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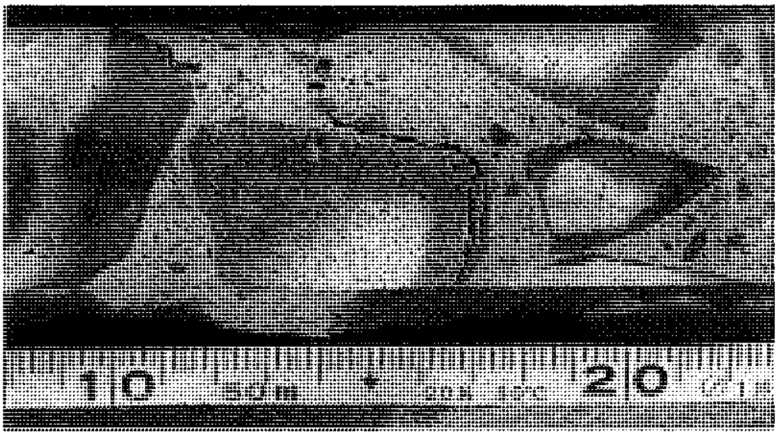
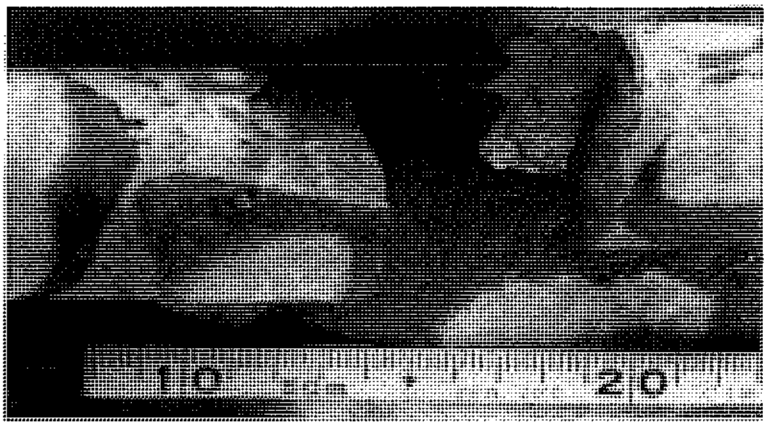
**PHOTOGRAPH OF CORE CRACKS**

| | | | |
|---|---|---|---|
| Core Hole: | CHSA2 | | |
| Sheet No: | 3/4 | | |
| Project No: | 3003301 | | |
| Client: | SEQ water | Coordinate System: | |
| Project: | Somerset Dam | Coordinates E: | |
| Feature: | Crack | N: | |
| Location: | MONOLITH S | Surface RL: | 107.6 |
| Datum: | AHD | Depth: | 18.05m |
| Machine: | Scout | | |



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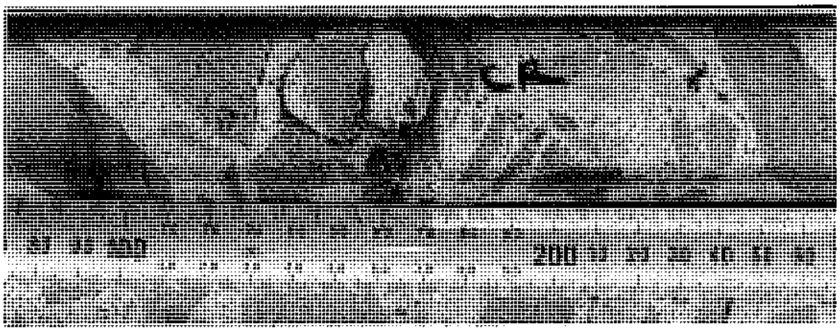
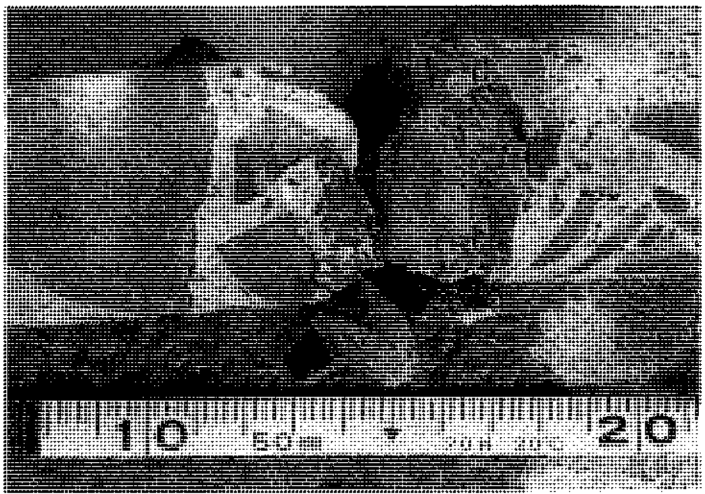
## PHOTOGRAPH OF CORE CRACKS

| | | | |
|---|---|---|---|
| Core Hole: | CHSA2 | | |
| Sheet No: | 4/4 | | |
| Project No: | 3003301 | | |
| Client: | SEQ water | Coordinate System: | |
| Project: | Somerset Dam | Coordinates E: | |
| Feature: | Crack | N: | Depth: 18.68m |
| Location: | MONOLITH S | Surface RL: 107.6 | Datum: AHD |
| | | | Machine: Scout |





## CHSV2

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @21.33 | @7m | @21.33 | |
| CHSV2 | | 21.25 | S | -90 | -90 | NA | NA | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHSV2 | Crack | 20.40 | Slight | Slight | Tight | Possible lift surface, weak cement possibly due to weathering. Slight signs of honey combing. |

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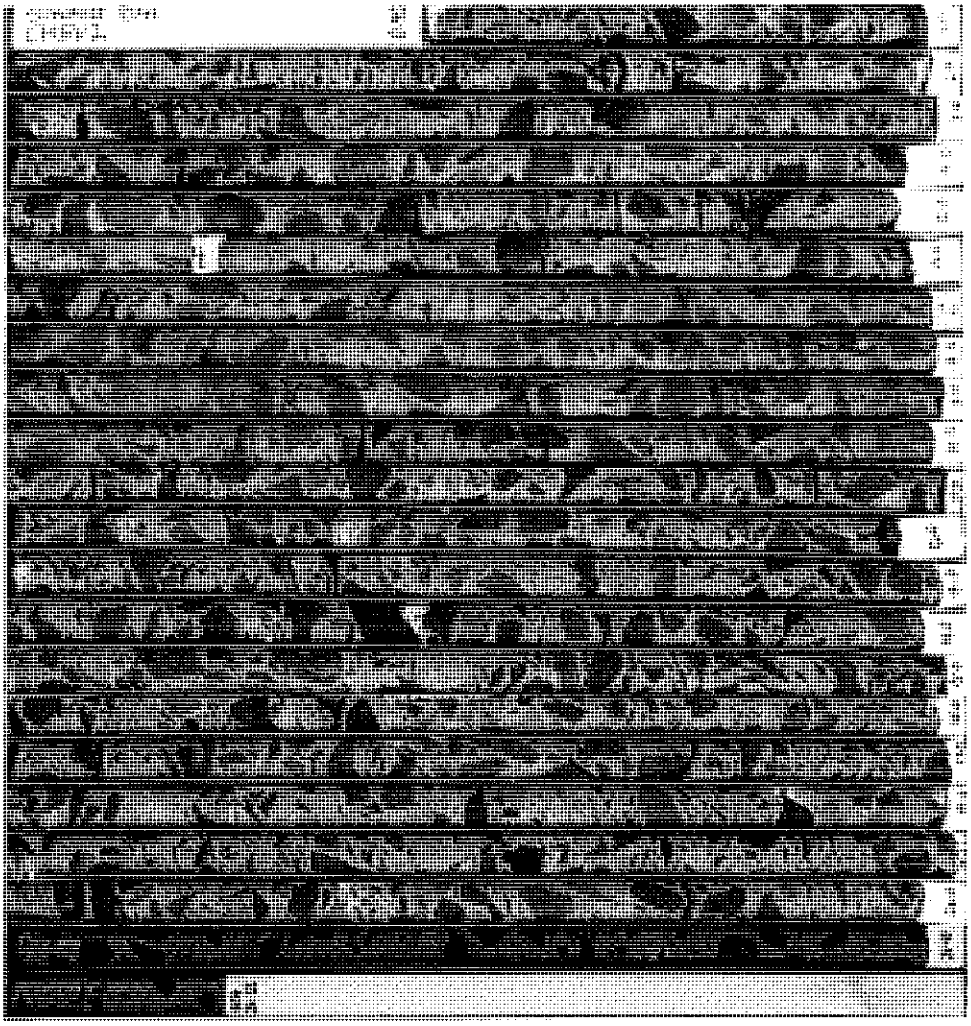
# PHOTOGRAPH OF CORE

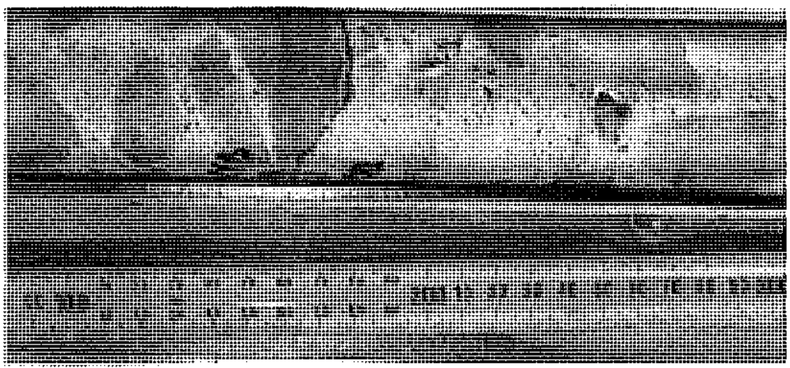
| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSV2 |
| **Project:** | Somerset Dam | **Coordinates** | **E:** | **Sheet No:** | 1/1 |
| **Feature:** | | | **N:** | **Project No:** | 3003301 |
| **Location:** | Monolith S | **Surface RL:** | 107.6 | **Datum:** | AHD |
| | | | | **Depth:** | |
| | | | | **Machine:** | Scout |

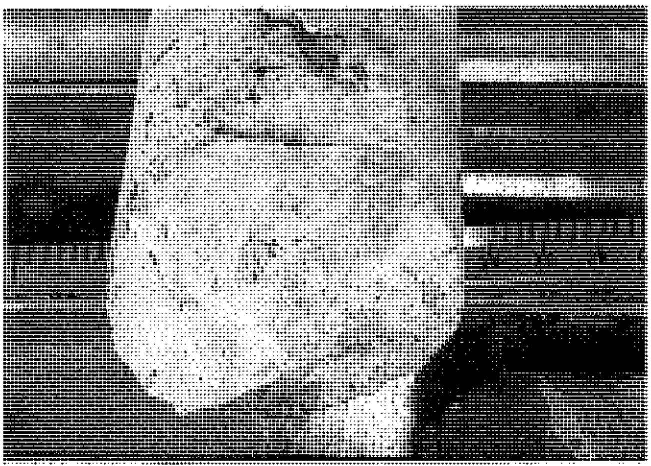


SMEC
SMEC Australia Pty Ltd

# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHSV2 |
| | | | | **Sheet No:** | 1/1 |
| | | | | **Project No:** | 3003301 |
| **Project:** | Somerset Dam | **Coordinates E:** | | | |
| **Feature:** | Crack | **N:** | | **Depth:** | 20.40m |
| **Location:** | MONOLITH S | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** | Scout |





## CHTA1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @18.0 | @7m | @18.0 | |
| CHTA1 | | 18.0 | T | -71.7 | -71.3 | 216.8 | 215.5 | Ended hole at 18.0m due to intercepting top corner of upper observation gallery. Approximately 20cm short of gallery crack. |

| HOLE ID | DEFECT TYPE | DEPTH (m) | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHTA1 | Crack | 12.66 | Moderate | Slight | Tight | Crack appears tight, distinct deposits of calcite on both surfaces. |
| CHTA1 | Crack | 14.80 | Moderate | Moderate | open | Vertical open crack (approximately 20 degrees in core), Calcified surface showing signs of weathering. |

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**PHOTOGRAPH OF CORE**

| | | | | | |
|---|---|---|---|---|---|
| **Core Hole:** | CHTA1 | | | | |
| **Sheet No:** | 1/1 | | | | |
| **Project No:** | 3003301 | | | | |
| **Client:** | SEQ water | **Coordinate System:** | | | |
| **Project:** | Somerset Dam | **Coordinates** | **E:** | | |
| **Feature:** | | | **N:** | **Depth:** | |
| **Location:** | Monolith T | **Surface RL:** | 107.6 | **Datum:** AHD | **Machine:** Scout |

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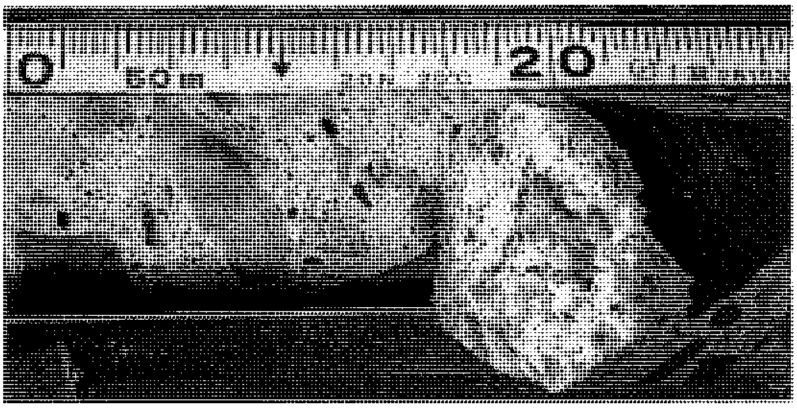
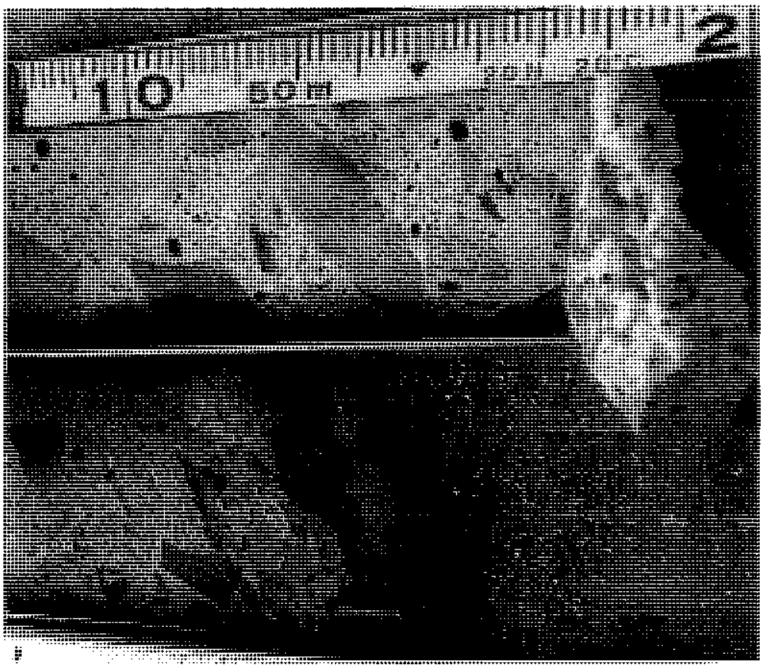
**PHOTOGRAPH OF CORE CRACKS**

| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHTA1 |
| | | | | **Sheet No:** | 1/2 |
| | | | | **Project No:** | 3003301 |
| **Project:** | Somerset Dam | **Coordinates** | **E:** | | |
| **Feature:** | Crack | | **N:** | **Depth:** | 12.66m |
| **Location:** | MONOLITH T | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** | Scout |





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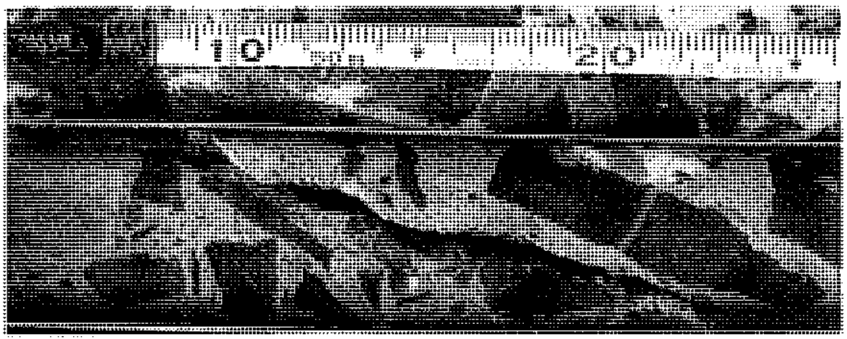
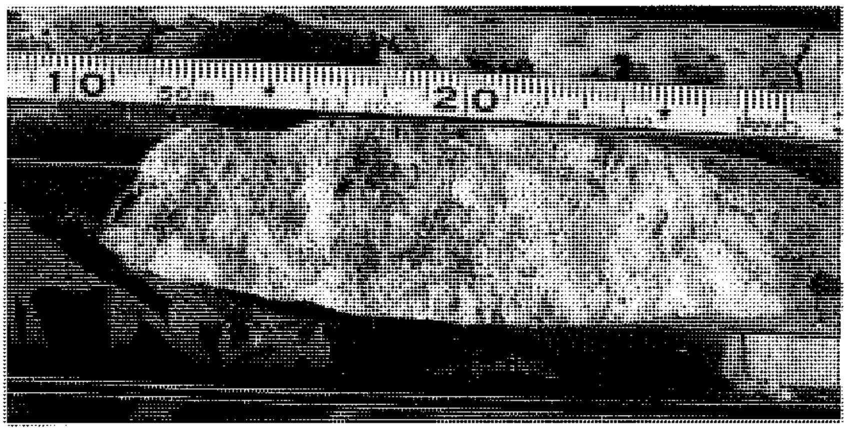
# PHOTOGRAPH OF CORE CRACKS

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHTA1 |
| | | | | Sheet No: | 2/2 |
| | | | | Project No: | 3003301 |
| Project: | Somerset Dam | Coordinates E: | | | |
| Feature: | Crack | N: | | Depth: | 14.80m |
| Location: | MONOLITH T | Surface RL: 107.6 | Datum: AHD | Machine: | Scout |



## CHTV1

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @21.30 | @7m | @21.30 | |
| CHSV1 | | 21.30 | T | -90 | -90 | NA | NA | No cracks/defects found |

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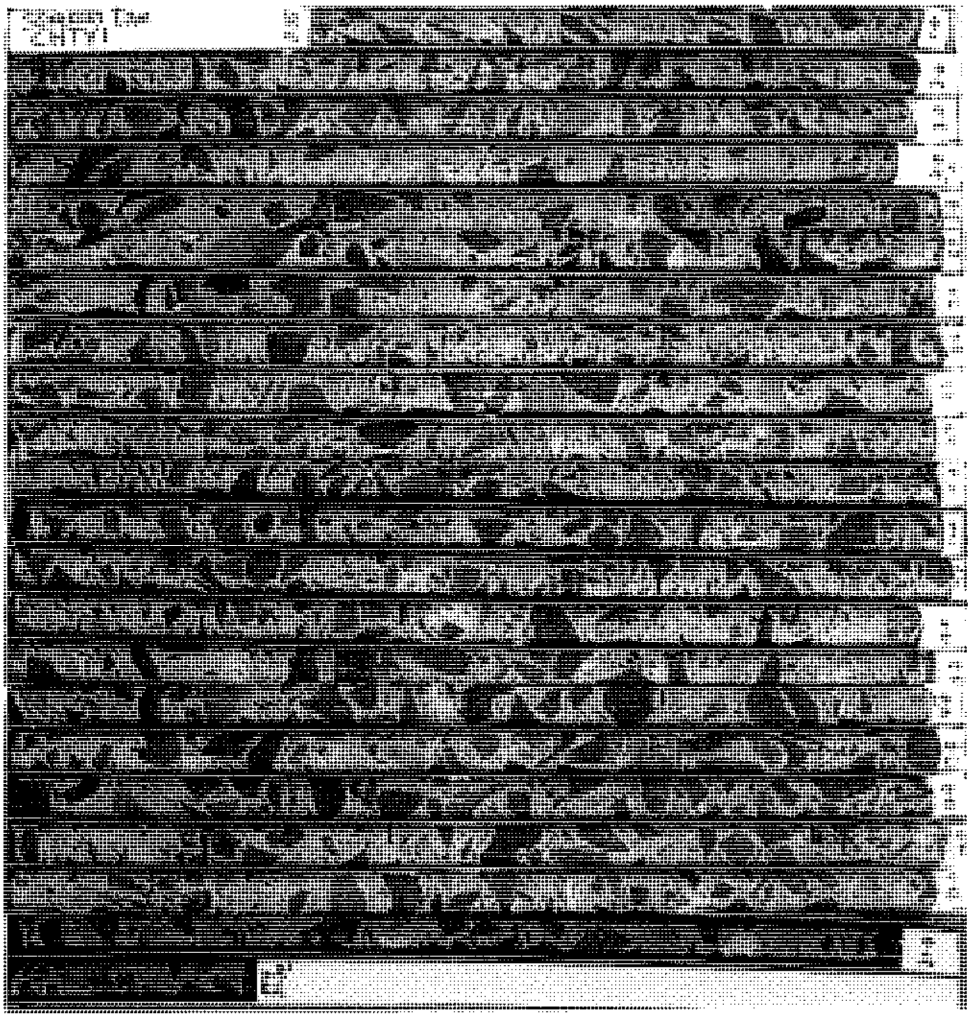
## PHOTOGRAPH OF CORE

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | Core Hole: | CHTV1 |
| Project: | Somerset Dam | Coordinates E: | | Sheet No: | 1/1 |
| Feature: | | N: | | Project No: | 3003301 |
| Location: | Monolith T | Surface RL: 107.6 | Datum: AHD | Depth: | |
| | | | | Machine: | Scout |



## CHTA2

| HOLE ID | CHAINAGE | HOLE DEPTH | MONOLITH | INCLINATION | | AZIMUTH | | COMMENT |
|---|---|---|---|---|---|---|---|---|
| | | | | @7m | @18.0 | @7m | @18.0 | |
| CHTA2 | | 21.92 | T | -66.7 | -65.9 | 208.7 | 218.0 | |

| HOLE ID | DEFECT TYPE | DEPTH | CALCITE | WEATHERING | JOINT TIGHTNESS | COMMENTS |
|---|---|---|---|---|---|---|
| CHTA2 | Crack | 12.66 | Slight | None | tight | Slight evidence of calcite on surfaces |
| CHTA2 | Crack | 19.1 | Slight | Slight | Tight | Slight evidence of calcite and weathering on surfaces |

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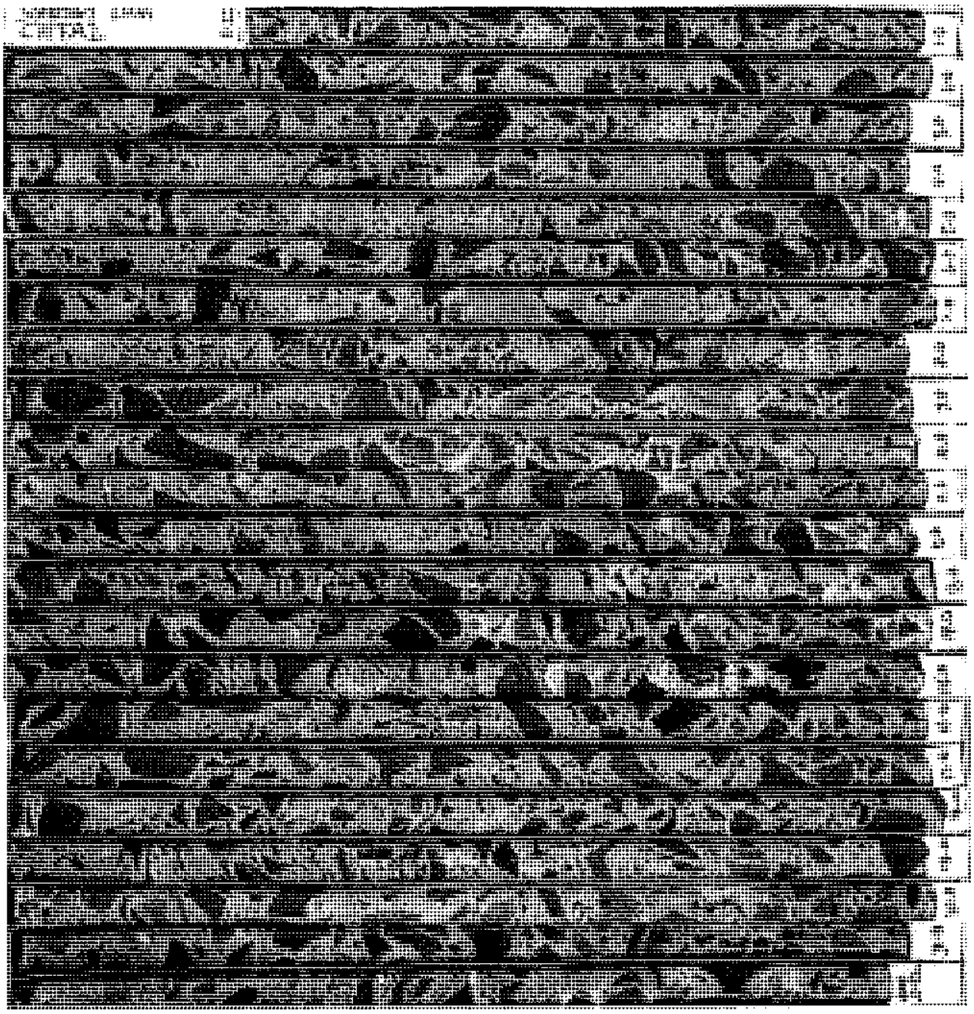
# PHOTOGRAPH OF CORE

Core Hole: CHTA2
Sheet No: 1/1
Project No: 3003301

| | | | | | |
|---|---|---|---|---|---|
| Client: | SEQ water | Coordinate System: | | | |
| Project: | Somerset Dam | Coordinates | E: | | |
| Feature: | | | N: | Depth: | |
| Location: | Monolith T | Surface RL: | 107.6 | Datum: AHD | Machine: Scout |



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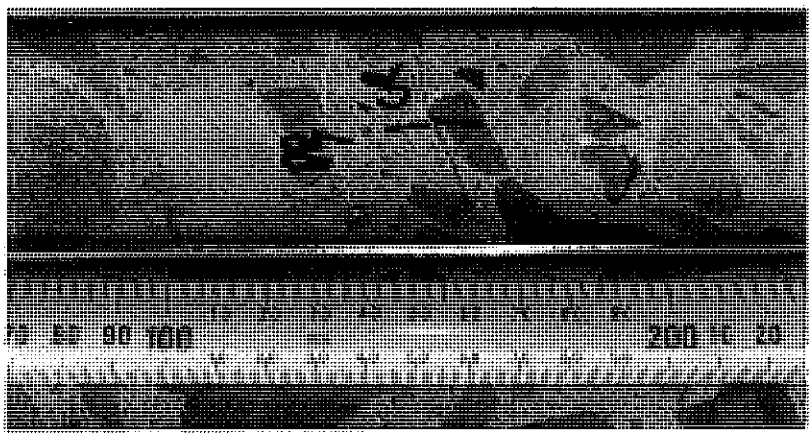
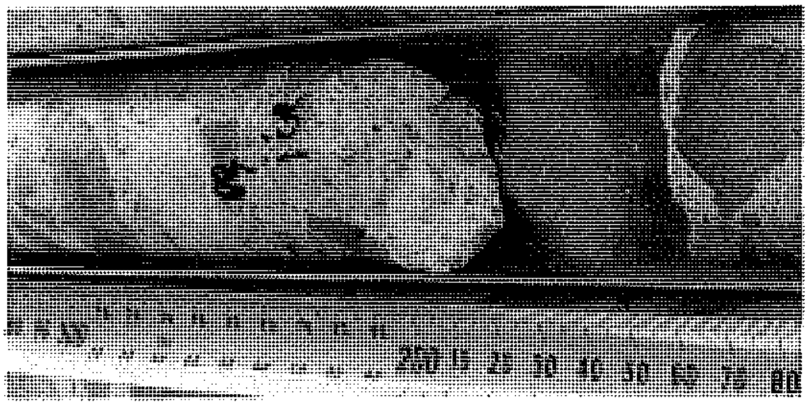
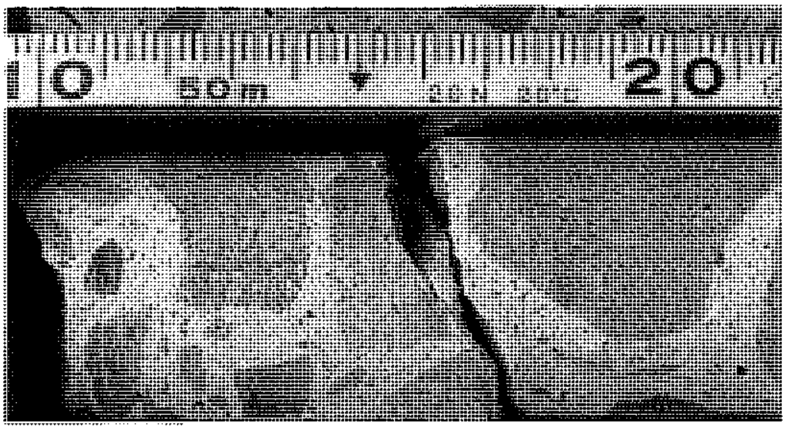
## PHOTOGRAPH OF CORE CRACKS

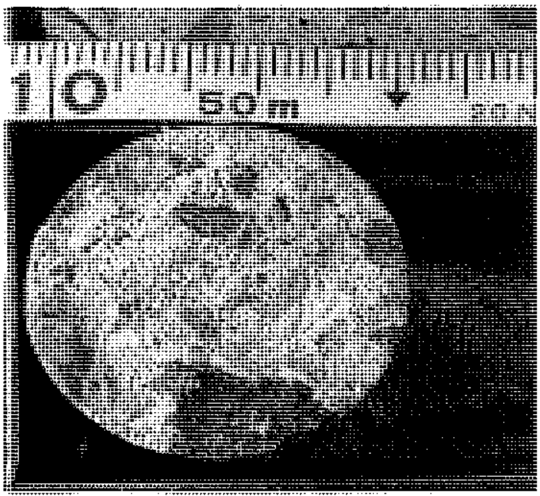
| | | | | | |
|---|---|---|---|---|---|
| **Client:** | SEQ water | **Coordinate System:** | | **Core Hole:** | CHTA2 |
| **Project:** | Somerset Dam | **Coordinates E:** | | **Sheet No:** | 1/2 |
| **Feature:** | Crack | **N:** | | **Project No:** | 3003301 |
| **Location:** | MONOLITH T | **Surface RL:** 107.6 | **Datum:** AHD | **Depth:** | 12.66m |
| | | | | **Machine:** | Scout |



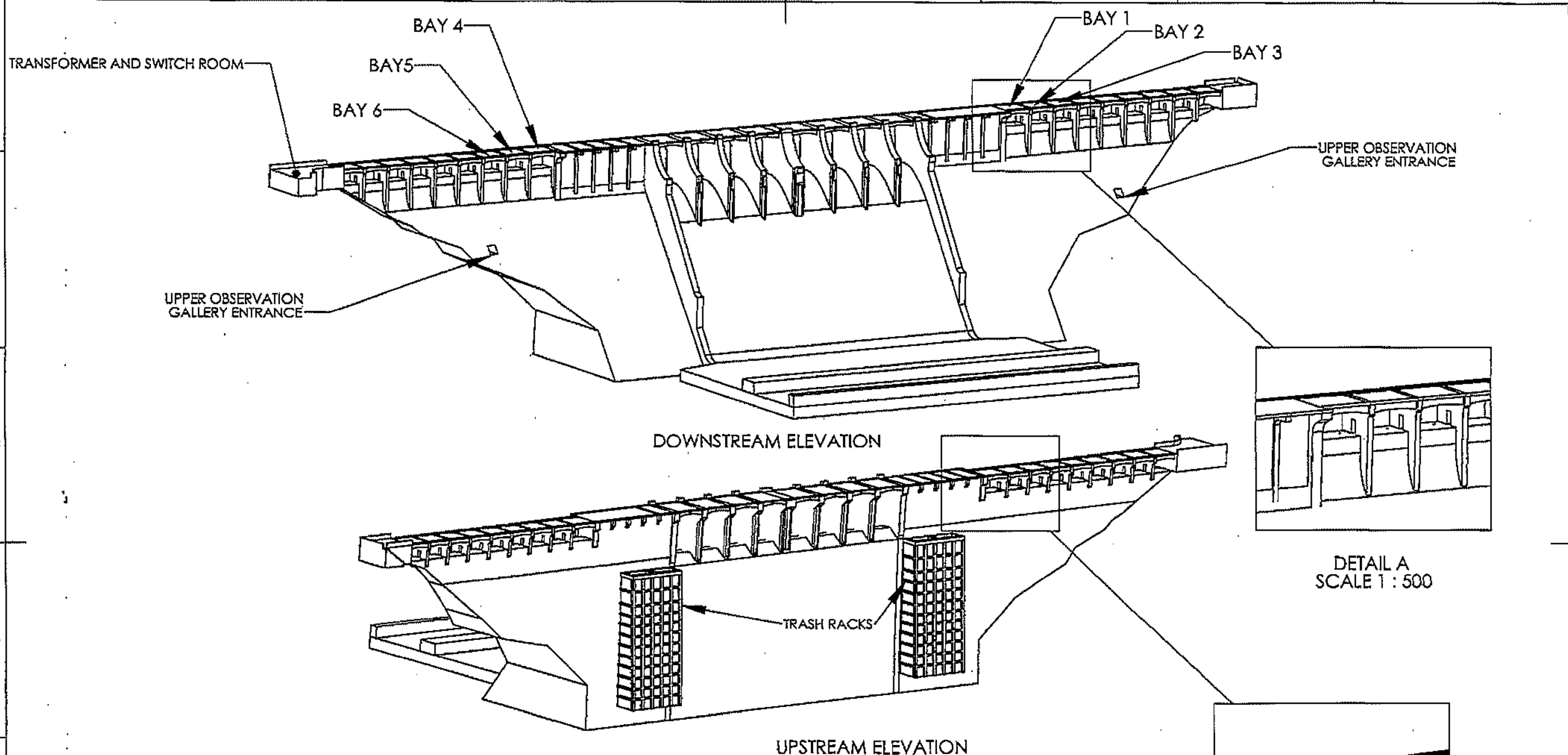


| | | | |
|---|---|---|---|
| SMEC<br>SMEC Australia Pty Ltd | **PHOTOGRAPH OF CORE CRACKS** | | **Core Hole:** CHTA2<br>**Sheet No:** 2/2<br>**Project No:** 3003301 |
| **Client:** SEQ water | **Coordinate System:** | | |
| **Project:** Somerset Dam | **Coordinates E:** | | |
| **Feature:** Crack | **N:** | | **Depth:** 19.10m |
| **Location:** MONOLITH T | **Surface RL:** 107.6 | **Datum:** AHD | **Machine:** Scout |





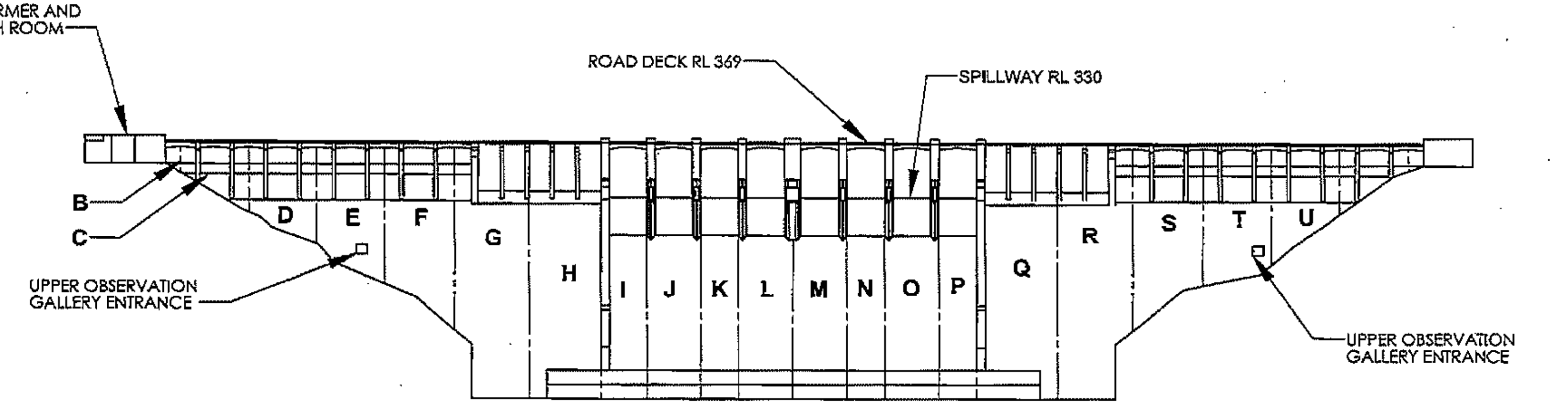
# APPENDIX B



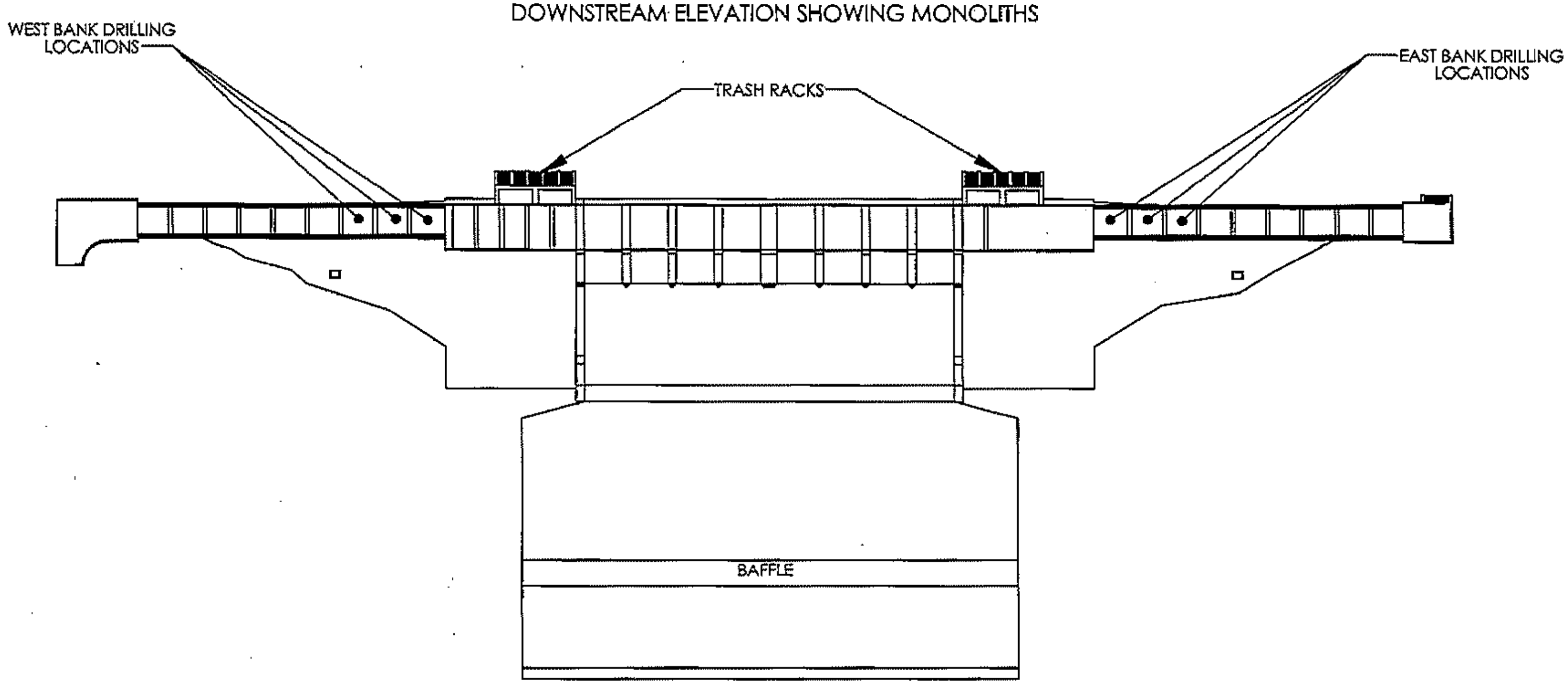

**HOLE DETAILS**

| HOLE ID | MONOLITH | BAY | INCLINATION | DEPTH | DRAWING No |
|---|---|---|---|---|---|
| CHSV1 | S | 1 | VERTICAL | 21.48 | 5 |
| CHSA1 | S | 1 | 19.5 | 22.32 | 5 |
| CHSV2 | S | 2 | VERTICAL | 21.25 | 6 |
| CHSA2 | S | 2 | 23 | 22.45 | 6 |
| CHTV1 | T | 3 | VERTICAL | 21.30 | 7 |
| CHTA1 | T | 3 | 18.5 | 18.00 | 7 |
| CHTA2 | T | 3 | 24 | 21.92 | 7 |
| CHFV1 | T | 4 | VERTICAL | 21.25 | 8 |
| CHFA1 | E | 4 | 24 | 22.90 | 8 |
| CHGA1 | E | 4 | 25.5 | 22.62 | 8 |
| CHFV2 | E | 5 | VERTICAL | 21.33 | 9 |
| CHFA2 | F | 5 | 25 | 22.93 | 9 |
| CHEV1 | F | 6 | VERTICAL | 21.79 | 10 |
| CHEA1 | F | 6 | 22.2 | 5.00 | 10 |
| CHEA2 | F | 6 | 23 | 18.60 | 10 |

| | | | |
|---|---|---|---|
| DRAWN | RG | CLIENT: SEQ WATER | JOB No: 3003301 |
| CHECKED | RG | SMEC | DWG NO. ISOMETRIC VIEW OF DAM |
| DATE | 6/05/08 | SMEC Australia Pty Ltd | A3 |
| PROJECT | | SOMERSET DAM | SCALE:1:1000 SHEET 1 OF 10 |

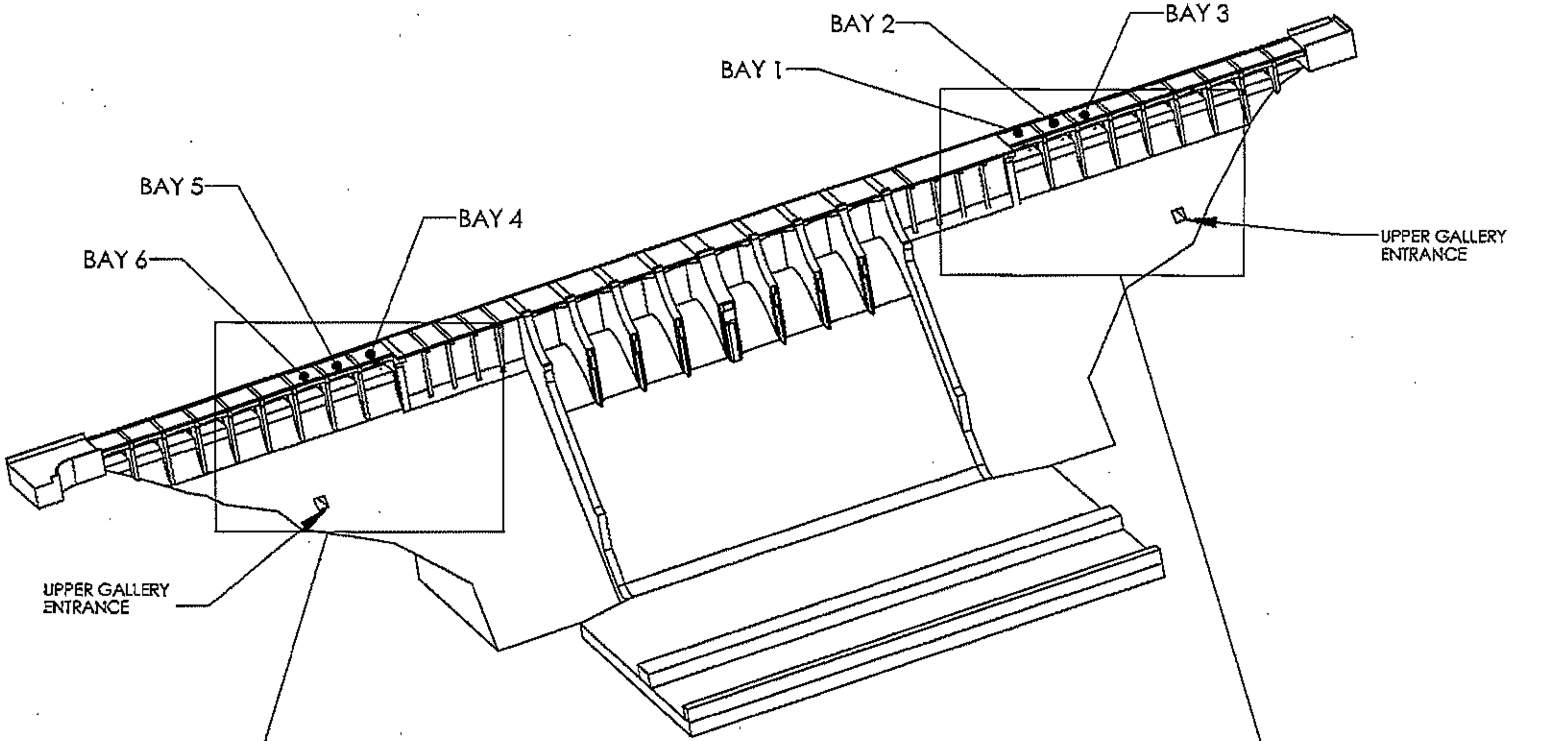


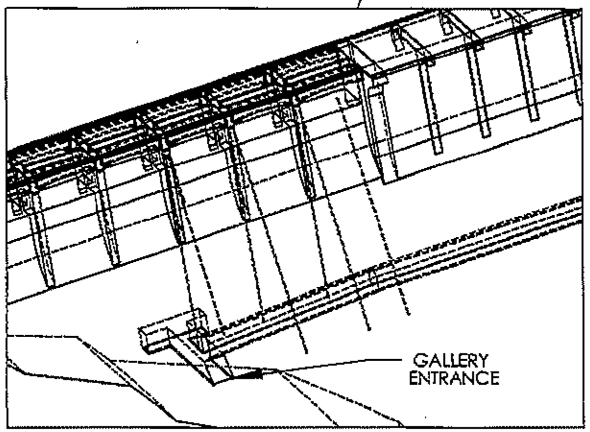

DOWNSTREAM ELEVATION SHOWING MONOLITHS



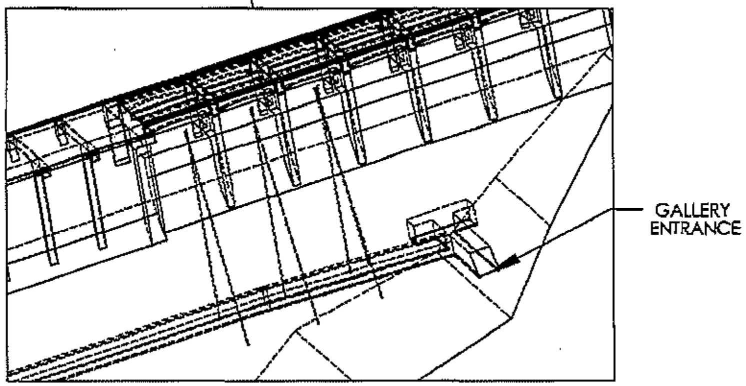

PLAN VIEW

| | | | | | |
|---|---|---|---|---|---|
| | | CLIENT | SEQ WATER | JOB No: 3003301 | |
| DRAWN | RG | SMEC | | DWG NO. MONOLITH LOCATIONS | A3 |
| CHECKED | | SMEC Australia Pty Ltd | | | |
| DATE | 6/05/08 | | | | |
| PROJECT | | SOMERSET DAM | | SCALE:1:1000 | SHEET 2 OF 10 |



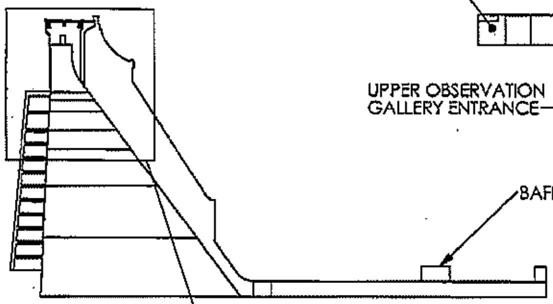




LOCATION OF GALLERY IN RELATION TO CORE HOLES ON WEST BANK
SCALE 1 : 500

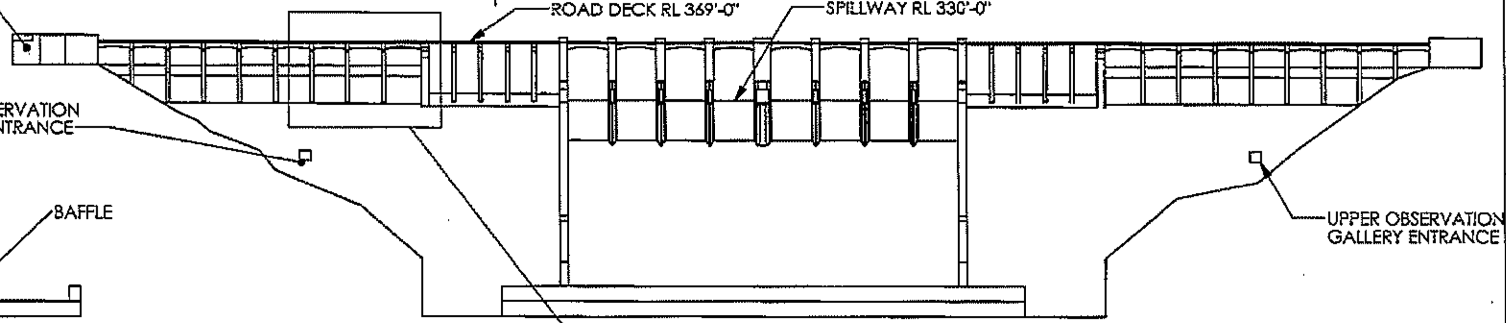



LOCATION OF GALLERY IN RELATION TO CORE HOLES ON EAST BANK
SCALE 1 : 500

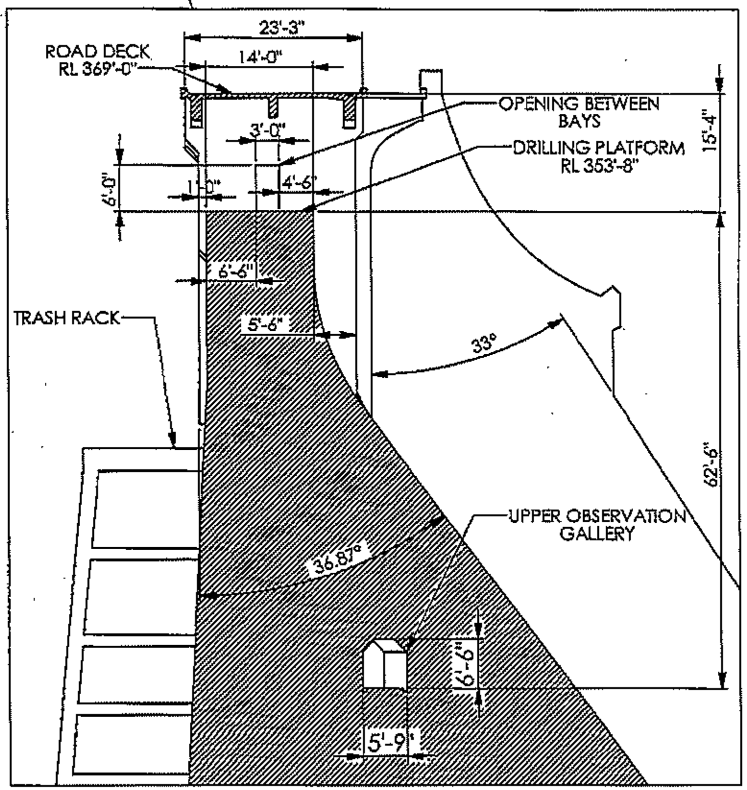
| | | | | | |
|---|---|---|---|---|---|
| | | CLIENT | SEQ WATER | PROJECT No: 3003301 | |
| DRAWN | RG | SMEC | | DWG NO. GALLERY POSITION | A3 |
| CHECKED | | | | | |
| DATE | 6/05/08 | SMEC Australia Pty Ltd | | | |
| PROJECT | SOMERSET DAM | | | SCALE:1:1000 | SHEET 3 OF 10 |



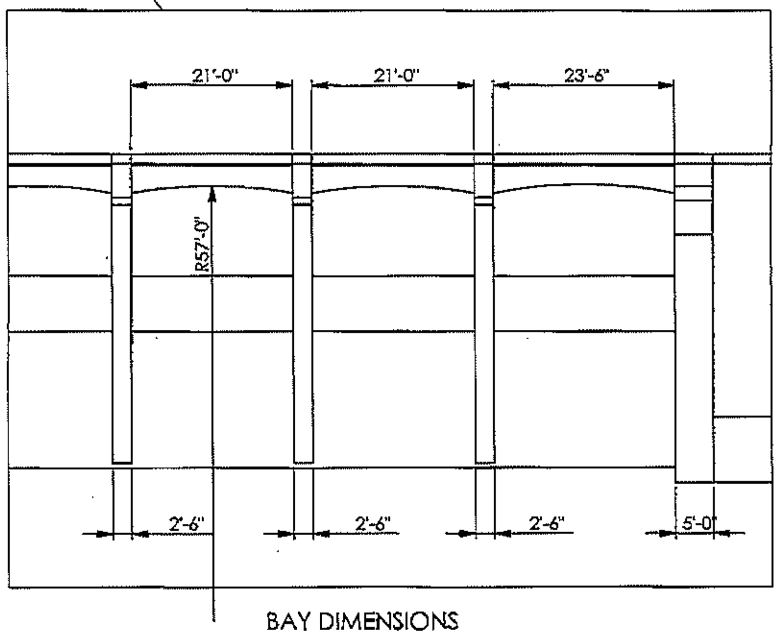

SIDE ELEVATION




DOWN STREAM ELEVATION

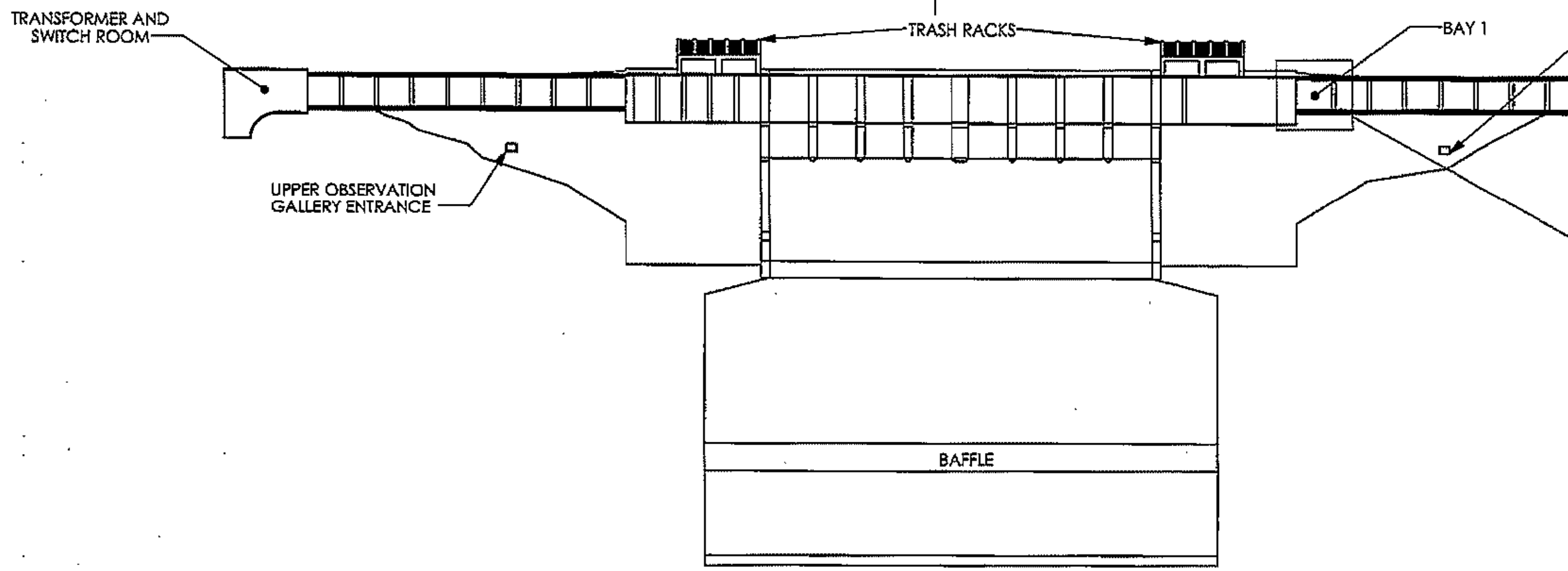


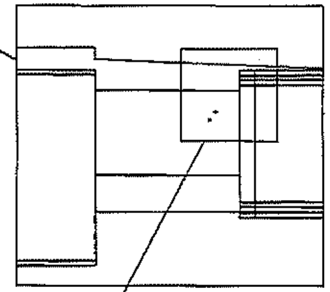

CROSS SECTION DETAILS SHOWING DIMENSIONS
SCALE 1 : 200



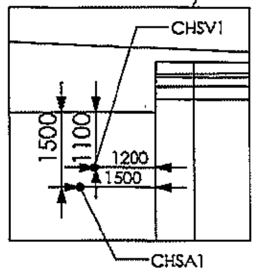

BAY DIMENSIONS
SCALE 1 : 200

| | | | |
|---|---|---|---|
| | CLIENT: SEQ WATER | PROJECT No: 3003301 | |
| DRAWN: RG | SMEC<br>SMEC Australia Pty Ltd | DWG NO. BAY AND DAM DIMENSIONS | A3 |
| CHECKED | | | |
| DATE: 6/05/08 | | | |
| PROJECT | SOMERSET DAM | SCALE 1:1000 | SHEET 4 OF 10 |

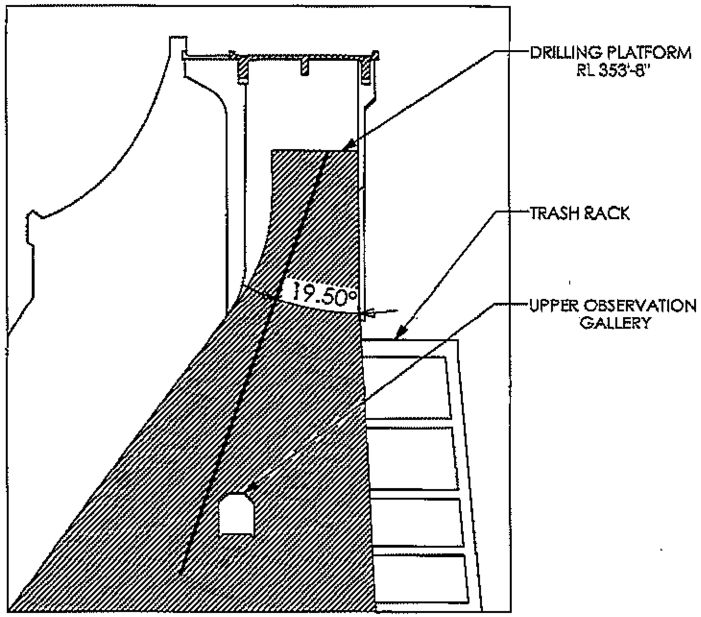


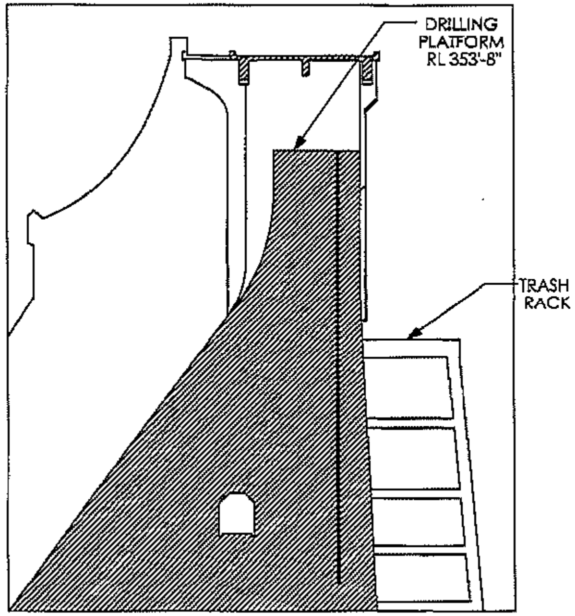
BAY 1
SCALE 1 : 250




HOLE LOCATIONS BAY 1
SCALE 1 : 100

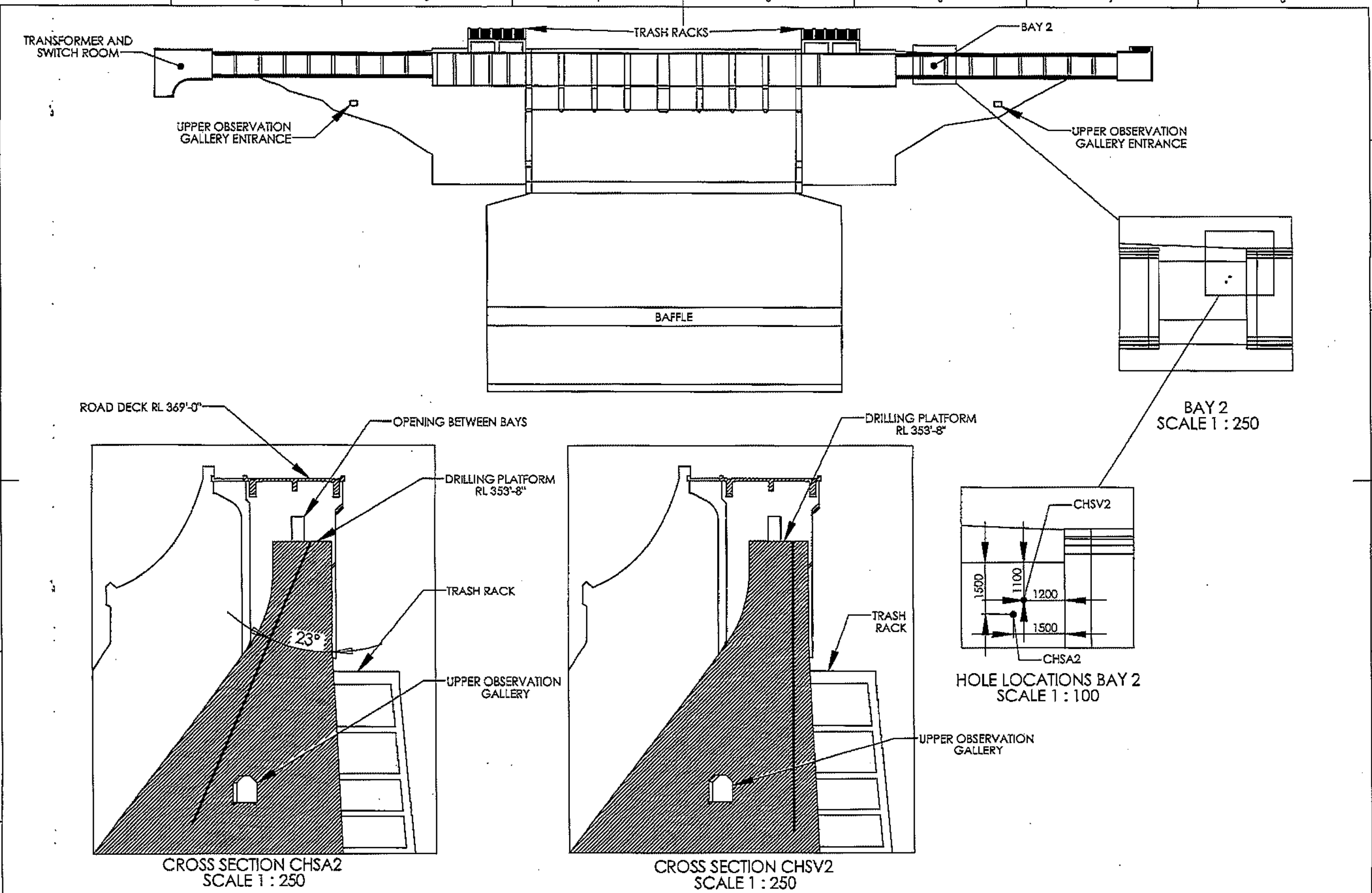



CROSS SECTION CHSA1
SCALE 1 : 250

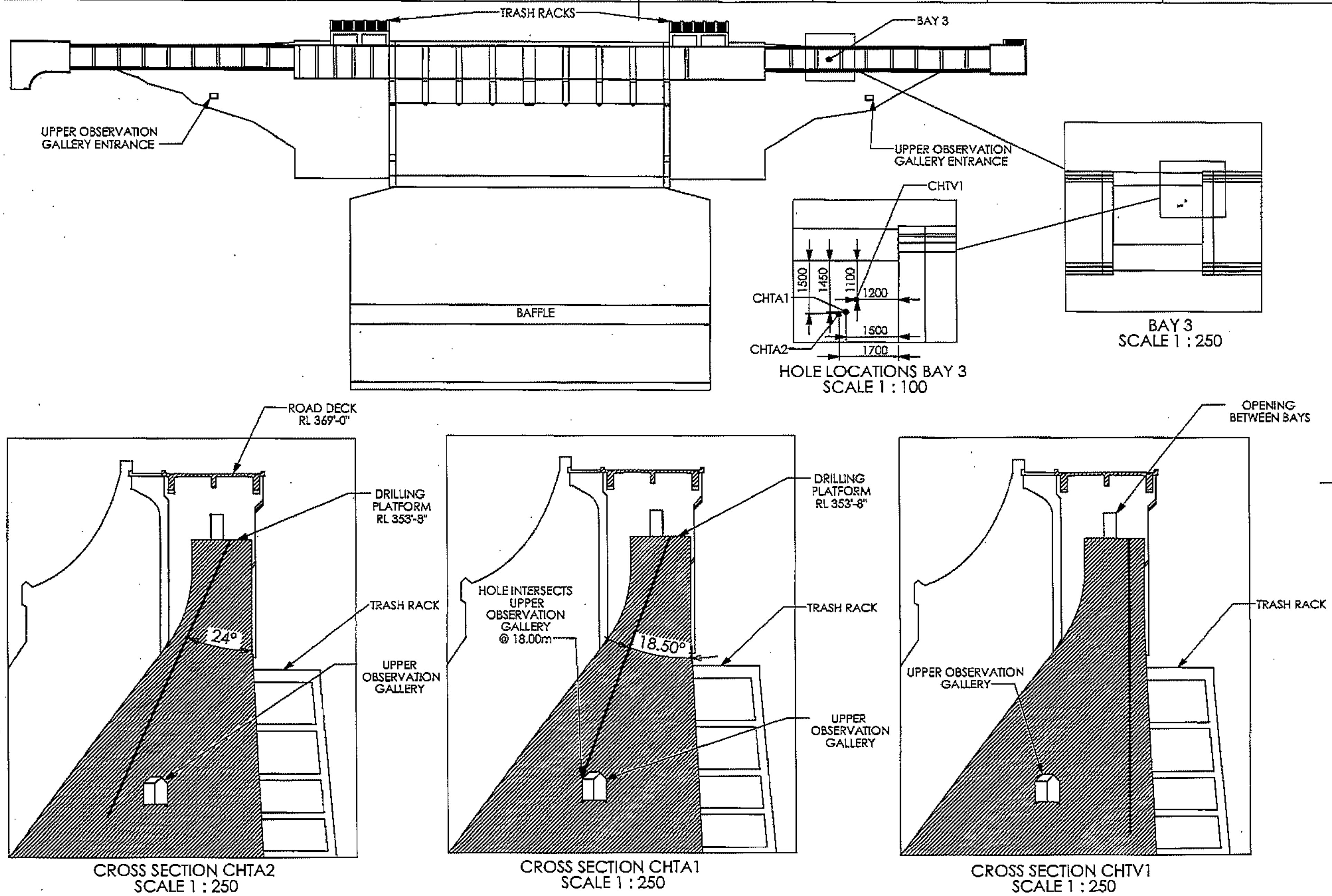



CROSS SECTION CHSV1
SCALE 1 : 250

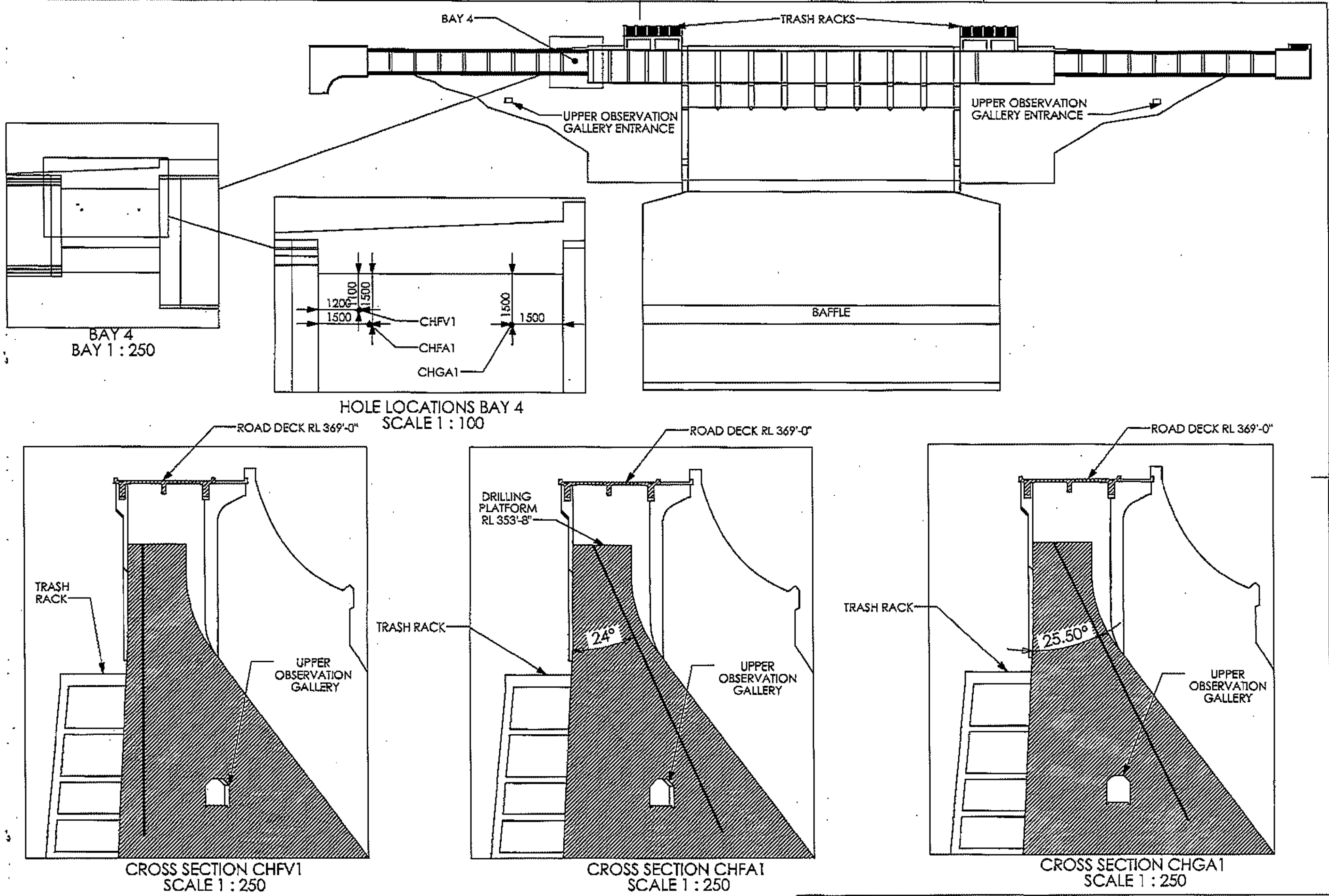
| | | | |
|---|---|---|---|
| | CLIENT SEQ WATER | PROJECT No: 3003301 | |
| DRAWN RG | SMEC<br>SMEC Australia Pty Ltd | DWG NO.<br>S MONOLITH - BAY 1 | A3 |
| CHECKED | | | |
| DATE 6/05/08 | | | |
| PROJECT | SOMERSET DAM | SCALE 1:1000 | SHEET 5 OF 10 |



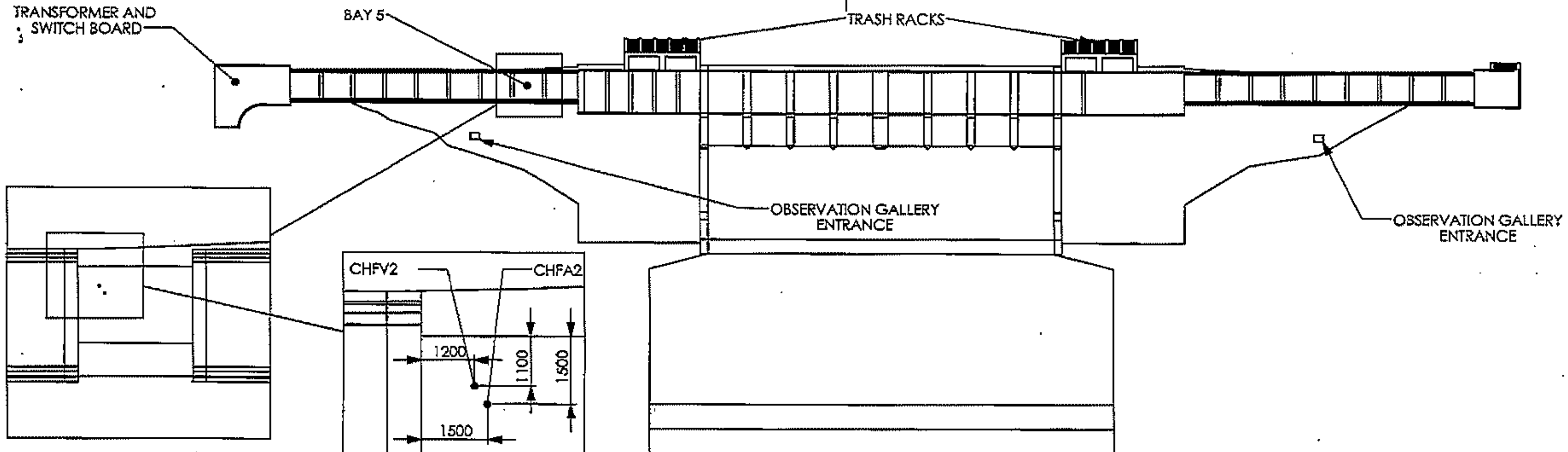

| | | | |
|---|---|---|---|
| | | CLIENT SEQ WATER | PROJECT No: 3003301 |
| DRAWN | RG | SMEC | DWG NO. S MONOLITH - BAY 2 | 
| CHECKED | | SMEC Australia Pty Ltd | A3 |
| DATE | 6/05/08 | | |
| PROJECT | | SOMERSET DAM | SCALE 1:1000 · SHEET 6 OF 10 |




| | | | | | |
|---|---|---|---|---|---|
| | | CLIENT | SEQ WATER | PROJECT No: 3003301 | |
| DRAWN | RG | SMEC | | DWG NO. T MONOLITH - BAY 3 | A3 |
| CHECKED | | | | | |
| DATE | 6/05/08 | SMEC Australia Pty Ltd | | | |
| PROJECT | · | SOMERSET DAM | | SCALE 1:1000 | SHEET 7 OF 10 |

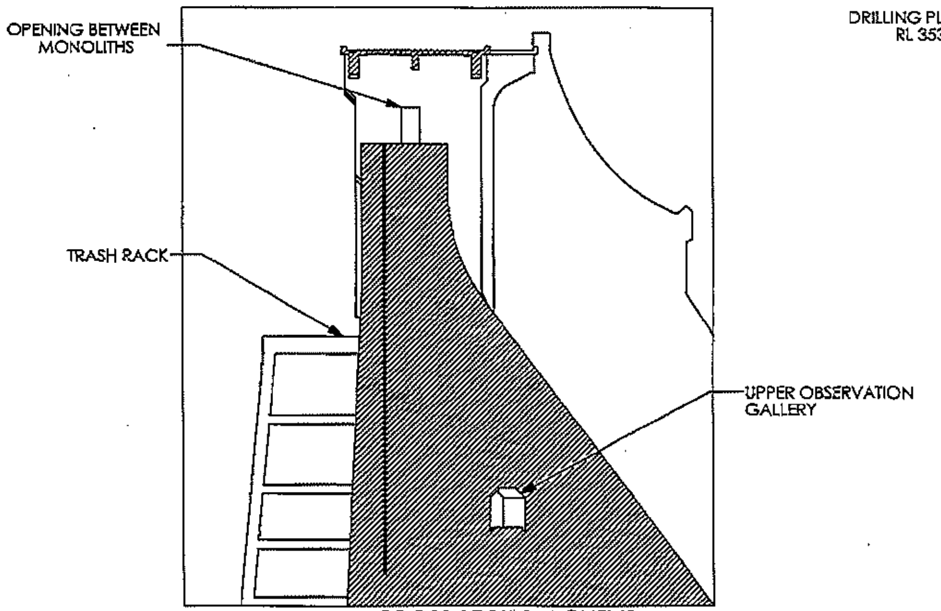



| | | | | |
|---|---|---|---|---|
| | | CLIENT SEQ WATER | PROJECT No: 3003301 | |
| DRAWN | RG | SMEC<br>SMEC Australia Pty Ltd | DWG NO.<br>F MONOLITH - BAY 4 | A3 |
| CHECKED | | | | |
| DATE | 6/05/08 | | | |
| PROJECT | SOMERSET DAM | | SCALE 1:1000 | SHEET 8 OF 10 |

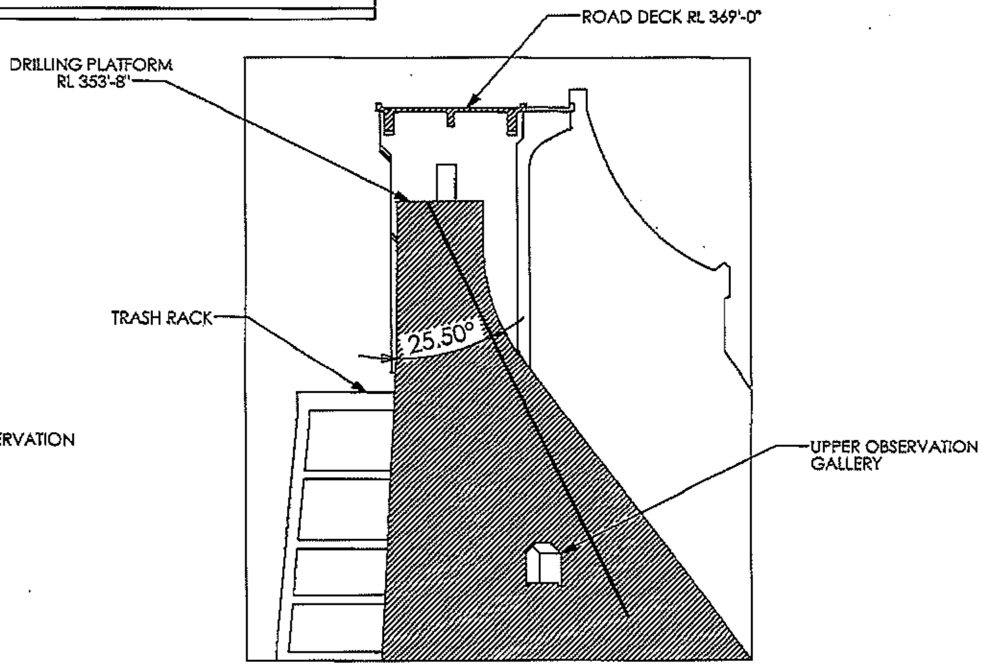



BAY 5
SCALE 1 : 250

HOLE LOCATIONS BAY 5
SCALE 1 : 100

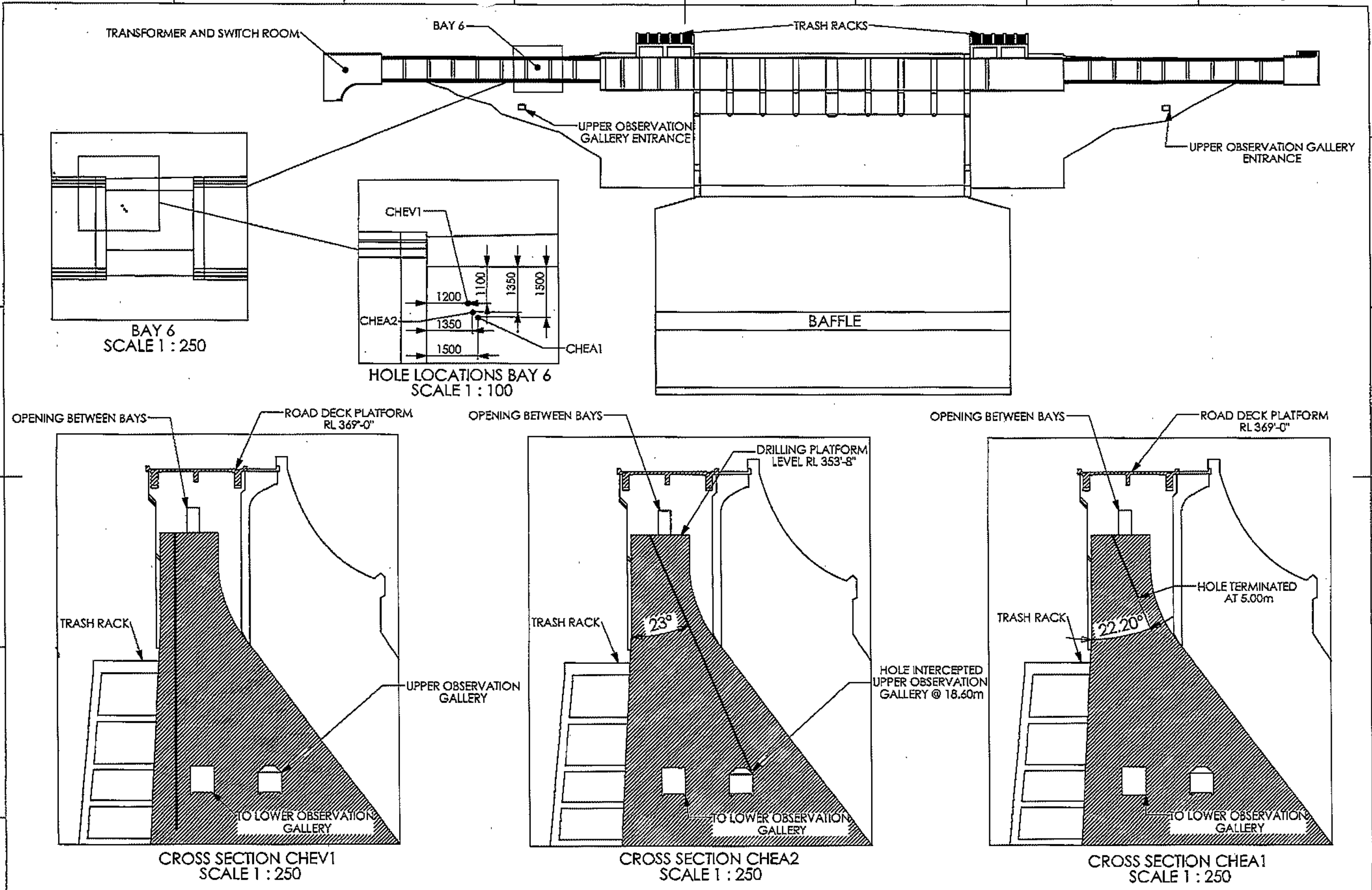



CROSS SECTION CHFV2
SCALE 1 : 250




CROSS SECTION CHFA2
SCALE 1 : 250

| | | | |
|---|---|---|---|
| | | CLIENT SEQ WATER | PROJECT No: 3003301 |
| DRAWN | RG | SMEC | DWG NO. F MONOLITH - BAY 5 — A3 |
| CHECKED | | SMEC Australia Pty Ltd | |
| DATE | 6/05/08 | | |
| PROJECT | | SOMERSET DAM | SCALE 1:1000 — SHEET 9 OF 10 |



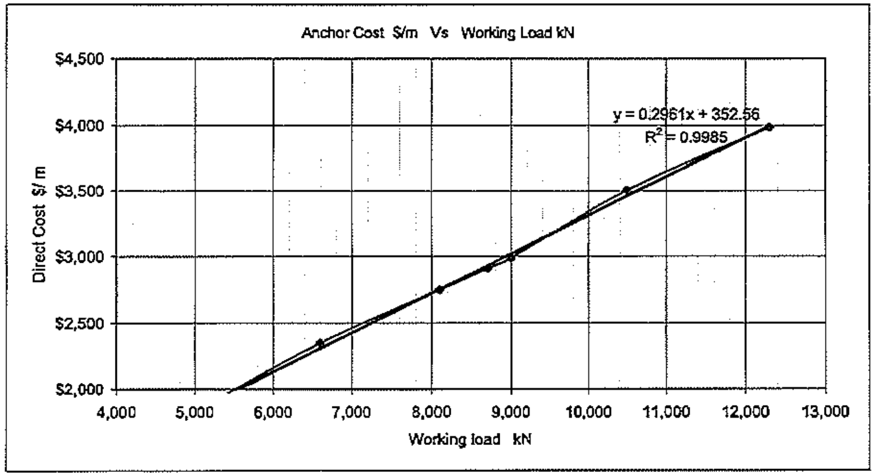

| | | | |
|---|---|---|---|
| DRAWN | RG | CLIENT: SEQ WATER | PROJECT No: 3003301 |
| CHECKED | | SMEC | DWG NO. E MONOLITH - BAY 6 |
| DATE | 6/05/08 | SMEC Australia Pty Ltd | A3 |
| PROJECT | | SOMERSET DAM | SCALE 1:1000 — SHEET 10 OF 10 |

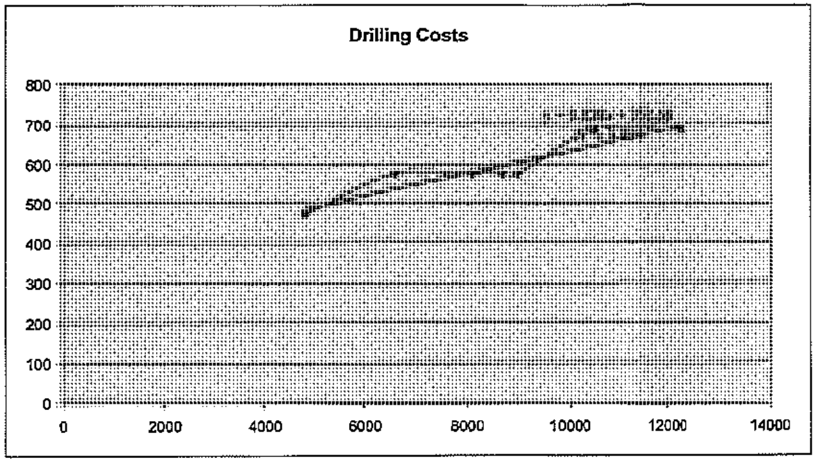
| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| $72,600 | $97,291 | $1,769 | 300 | 460 | 115 | 575 | $31,625 | $0 | $128,916 | **$2,344** |
| $89,100 | $119,403 | $2,171 | 300 | 460 | 115 | 575 | $31,625 | $0 | $151,028 | **$2,746** |
| $139,200 | $186,541 | $2,332 | 300 | 460 | 115 | 575 | $46,000 | $0 | $232,541 | **$2,907** |
| $126,000 | $168,852 | $2,412 | 300 | 460 | 115 | 575 | $40,250 | $0 | $209,102 | **$2,987** |
| $178,500 | $239,207 | $2,814 | 350 | 550 | 137.5 | 687.5 | $58,438 | $0 | $297,645 | **$3,502** |
| $209,100 | $280,214 | $3,297 | 350 | 550 | 137.5 | 687.5 | $58,438 | $0 | $338,651 | **$3,984** |

sembly and installation

**TS**

$5,000




REST FSL 105,

| No. Anchors | Working load | Length | Cost per metre | Direct Cost of Anchors |
|---|---|---|---|---|
| No. | kN | m | $/m | |
| 15 | 8100 | 55 | 2780 | $2,293,500 |
| 15 | 12300 | 85 | 4040 | $5,151,000 |
| 16 | 12600 | 75 | 4130 | $4,956,000 |
| 18 | 12300 | 85 | 4040 | $6,181,200 |
| 13 | 8100 | 55 | 2780 | $1,987,700 |
| | | | | $20,569,400 |

alternative costing

**Cost = $30,000 $/tonne strand**

Approx cost per tonne supplied by VSL 30/8/06

Weight strand per anchor-
1.125 kg/m/strand

| Mass/m | Mass/anchor | | | | Drilling costs/m | Drilling |
|---|---|---|---|---|---|---|
| kg/m | kg/anchor | tonne/anchor | $/ANCHOR | Total $ | | |
| 60.75 | 3341.25 | 3.34 | $100,238 | $1,503,563 | $579 | $1,719,036 |
| 92.25 | 7841.25 | 7.84 | $235,238 | $3,528,563 | $696 | $4,853,908 |
| 94.5 | 7087.5 | 7.09 | $212,625 | $3,402,000 | $705 | $4,440,240 |
| 92.25 | 7841.25 | 7.84 | $235,238 | $4,234,275 | $696 | $4,853,908 |
| 60.75 | 3341.25 | 3.34 | $100,238 | $1,303,088 | $579 | $1,719,036 |
| | | | | $13,971,488 | | $17,586,128 |
| | | | | | | $31,557,616 |

RADIAL GATES

| No. Strand | No. Anchors | Working load | Length | Cost per metre | Direct Cost of Anchors |
|---|---|---|---|---|---|
| No. | No. | kN | m | $/m | |
| 32 | 15 | 4800 | 50 | 1790 | $1,342,500 |
| 58 | 15 | 8700 | 80 | 2960 | $3,552,000 |
| 60 | 16 | 9000 | 70 | 3050 | $3,416,000 |
| 58 | 18 | 8700 | 80 | 2960 | $4,262,400 |
| 32 | 13 | 4800 | 50 | 1790 | $1,163,500 |
| | | | | | $13,736,400 |

REST 103

| No. Anchors | Working load | Length | Cost per metre | Direct Cost of Anchors |
|---|---|---|---|---|
| No. | kN | m | $/m | |
| 15 | 6600 | 55 | 2330 | $1,922,250 |
| 15 | 10500 | 85 | 3500 | $4,462,500 |
| 16 | 11550 | 75 | 3815 | $4,578,000 |
| 18 | 10500 | 85 | 3500 | $5,355,000 |
| 13 | 6600 | 55 | 2330 | $1,665,950 |
| | | | | $17,983,700 |

FIXED CREST 101

| No. Strand | No. vnchor | Working load | Length | Cost per metre | Direct Cost of Anchors |
|---|---|---|---|---|---|
| No. | No. | kN | m | $/m | |
| 32 | 15 | 4800 | 50 | 1790 | ##### |
| 58 | 15 | 8700 | 80 | 2960 | ##### |
| 60 | 16 | 9000 | 70 | 3050 | ##### |
| 58 | 18 | 8700 | 80 | 2960 | ##### |
| 32 | 13 | 4800 | 50 | 1790 | ##### |

BM-1: Document 40

**SOMERSET DAM**

**ANNUAL DAM SAFETY INSPECTION 2009**

| | | |
|---|---|---|
| **Date of Inspection:** | 2 November 2009 | |
| **Inspected by:** | John Tibaldi – Seqwater | |
| **Others present:** | Agg Dagan - Seqwater | |
| **Report Prepared by:** | John Tibaldi (RPEQ 2525) | |
| **Field Conditions:** | Clear | ☒ |
| | Cloudy | ☐ |
| | Overcast | ☐ |
| | Rain | ☐ |
| | Rainfall | Nil |

# EXECUTIVE SUMMARY

Somerset Dam has been owned by Seqwater since 1 July 2008. In accordance with the Dam safety Conditions for Somerset Dam issued in accordance with the Water Supply Act 2008, Seqwater must undertake an Annual Inspection of the dam each year in accordance with the *Queensland Dam Safety Management Guidelines.* This report contains the results of the inspection undertaken in September 2009. The previous inspection was undertaken in November 2008.

The dam is generally in good condition and no significant dam safety issues were identified during the inspection. The main concerns from the inspection relate to the discontinuation of the critical infrastructure condition monitoring programs at the dam and the outstanding corrective maintenance works.

It is also recommended that the dam safety issue associated with the concrete cracking in the upper gallery be revisited, as the instrumentation records show that the crack is continuing to widen over time.

The next inspection is scheduled for October 2010.

# TABLE OF CONTENTS

# 1.0 DESCRIPTION

| | |
|---|---|
| POPULATION AT RISK | - Sunny Day Failure...... > 1000<br>- Flood.............................. > 1000 (Not fully assessed) |
| Failure Impact Rating | 2 |
| Hazard Category | Extreme |
| Name of Reservoir | Lake Somerset |
| Year Complete | 1953 |
| Location | Stanley River near Kilcoy |
| Dam Owner | Seqwater |
| Water Course | Stanley River |
| Purpose | Town water |
| Type of Construction | Mass concrete gravity dam |
| Outlet Works | 8 radial gates, 8 sluice gates and 4 cone dispersion valves |
| Catchment Area | 1,340 $km^2$ |
| FSL | 99.00 m AHD |
| Nominal Full Supply Capacity | 380,000 ML |
| Surface Area at FSL | 4,210 ha |
| Main Dam Crest | 107.46 m AHD |
| Bridge Deck Level | EL 112.34 m |
| Main Dam Embankment Length | 305 m |
| Maximum Height of Main Dam Embankment | 58.0 m |
| Width at Top of Main Dam Embankment | 6.5 m |
| Spillway Crest | 100.45 m AHD |

| | |
|---|---|
| Spillway Length | 63.4 m |
| Gates | 8 gates – wide 7.9 m x 7.0 m high |
| Saddle Dam Crest | Not Applicable |
| Saddle Dam Length | Not Applicable |
| Maximum Height of Saddle Dam Embankment | Not Applicable |
| Peak water level as a result of PMF | EL 112.0 m |
| Spillway Capacity (Including sluice gates) | 4700 $m^3/s$ (EL 107.5 m) |
| Maximum discharge as a result of PMF | 9600 $m^3/s$ |
| AEP of Spillway Capacity (Including sluice gates) | 1 in 10 000 (EL 107.5 m) |
| Regulator valves | 4 x 3 m cone dispersion valves |
| Mean annual pan evaporation | 1600 mm estimated from BoM maps |
| Mean annual rainfall | 986 mm at 040189 Somerset Dam |
| Hydroelectric Facilities | 4 mw generator |
| Notable events | 1974 |
| Maximum Historic Storage Level | 106.26 m |
| COMMENT | |

## 2.0 DAM STATUS

| | |
|---|---|
| **Date** | 2 November 2009 |
| **Reservoir Water Surface Elevation** | 97.38 m |
| **Percentage Full** | 83.3 % |
| **Reservoir Water Level Relative to FSL** | -1.62 m |
| **Spillway Releases:** | Nil |

## 3.0 DAM OPERATION AND DOCUMENTATION

The following Dam Safety Documentation is held by Seqwater for Somerset Dam:

- Emergency Action Plan.
- Standing Operating Procedures.
- Operation and Maintenance Manual.
- Data Book.
- Dam safety Review.

As part of the Annual Inspection process, the above documents were reviewed and found to be in accordance with the Queensland Dam Safety Management Guidelines. The Emergency Action Plan and Standing Operating Procedures are in a standard Seqwater format. The contact details contained in the Emergency Action Plan were updated in July 2009. The Operation and Maintenance Manual is currently being reviewed and updated and the Data Book remains in the format of the previous owner of Somerset Dam and are scheduled for conversion to a standard Seqwater format in 2009/10.

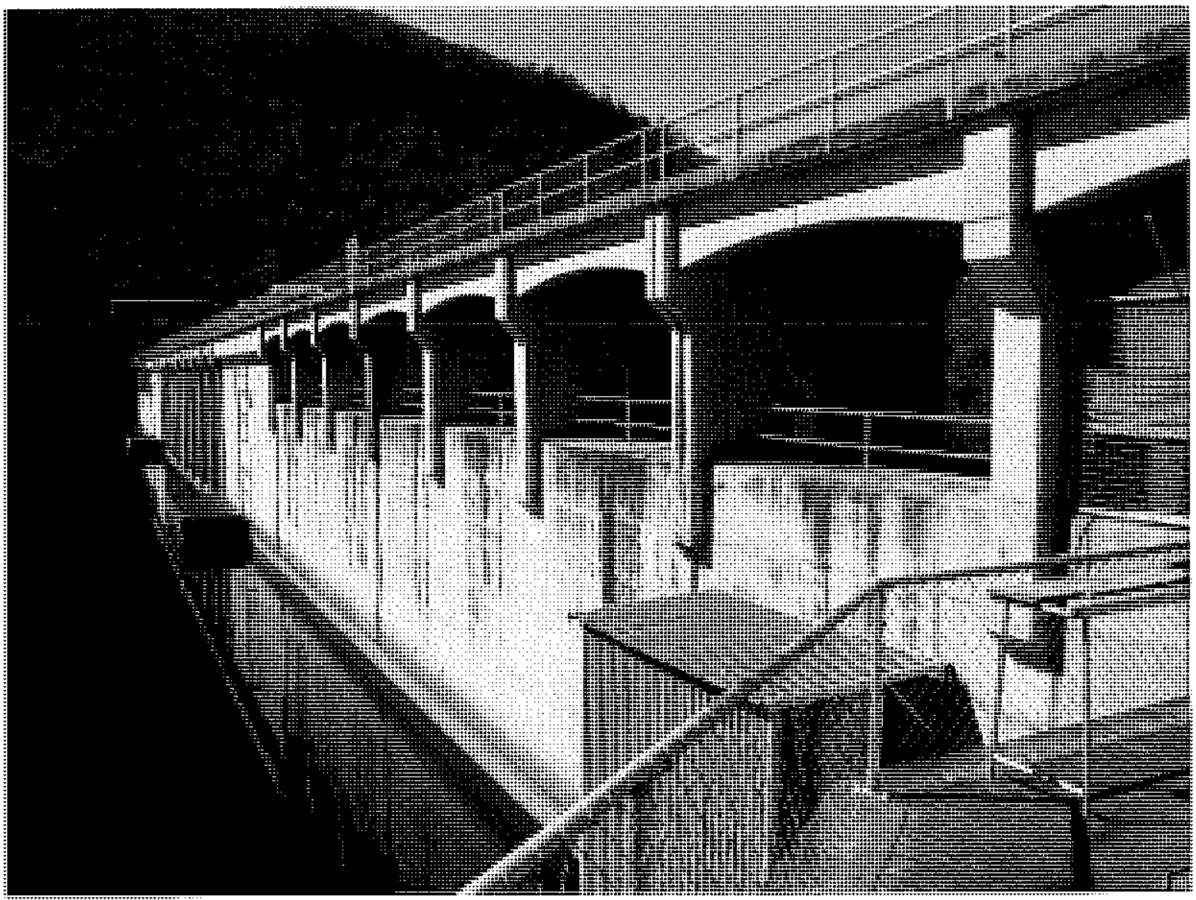
## 4.0 ROUTINE INSPECTION AND LOGBOOK

Routine Inspections at Somerset Dam are undertaken on a daily basis in accordance with the Queensland Dam Safety Management Guidelines. Complete records of all inspections are kept on site and at Seqwater's Karalee Office. No significant dam safety issues were identified from a review of the inspection records over the last 12 months.

A traditional logbook is currently in place at Somerset Dam. A logbook is maintained generally to record the date of inspections and instrument monitoring, to note maintenance undertaken, and to note unusual events (e.g. seismic activity, floods, change in seepage patterns, etc.) to assist in maintaining the safety management of the dam. Notes in the logbook may assist in identifying the time and cause of incidents which may provide early warning for potential failure mechanisms.

## 5.0 DAM EMBANKMENT

Somerset Dam is a 47 metre high concrete gravity dam on the Stanley River upstream of Wivenhoe Dam. The dam is of conventional mass concrete construction. There are seven mass concrete abutment units on each side of the central spillway structure that supports a road bridge at EL 112.3. The abutment units are constructed with an open overflow section below the bridge at EL 107.5. Flood flows passing through these openings flow down the back face of the dam and impact on an unprotected rock foundation rock, before flowing laterally towards the central spillway channel.



**Somerset Dam**

The concrete embankment was inspected and was found generally to be in good condition. Some vegetation was observed growing on the embankment and within five metres of the embankment abutments that should be removed. This aspect of the embankment is certainly improved since the 2008 inspection, with several large trees removed from the right bank.

There are a number of galleries within the dam. Concrete cracking has occurred at the Upper Gallery and there is considerable horizontal cracking exposed in the gallery walls. There is no indication that the cracking is worsening over time.

***Inspection Recommendations:***

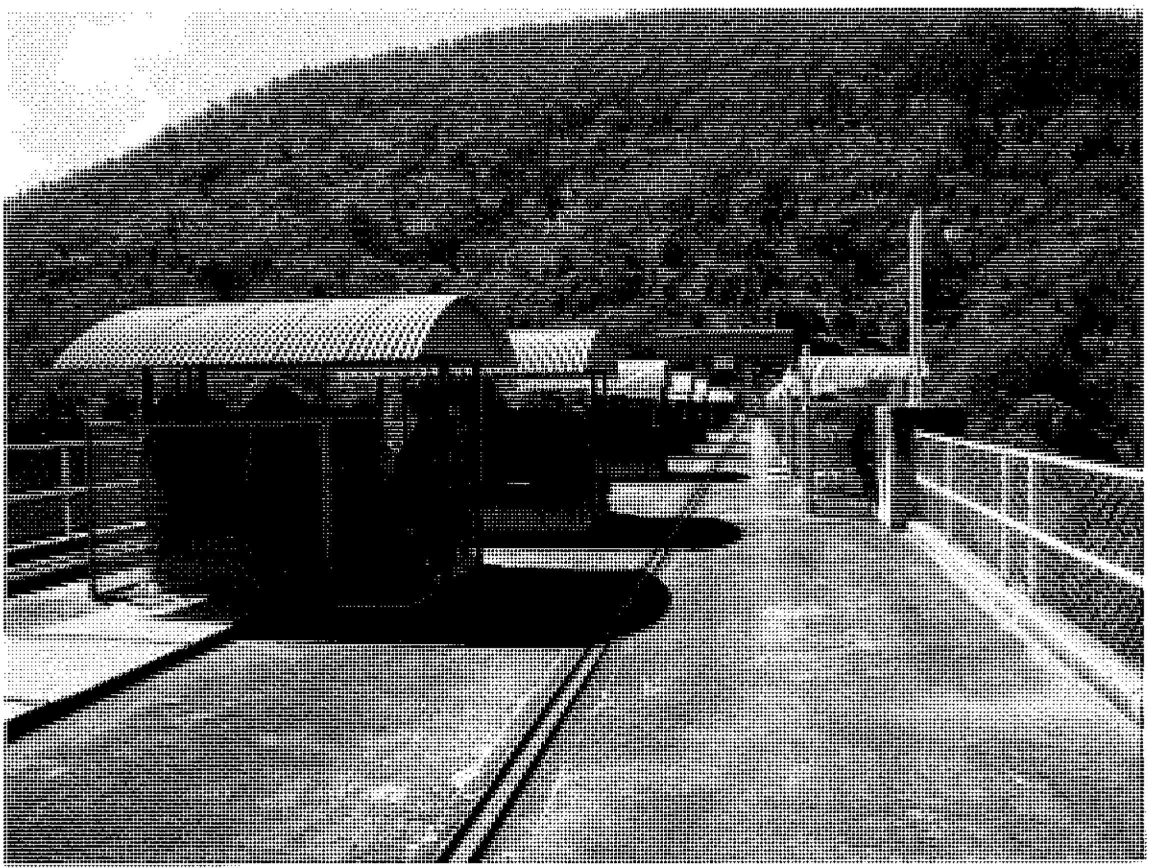
- ***Vegetation growing on the embankment is to be removed and/or sprayed with a suitable herbicide. This includes the trees growing within five metres of the abutments.***

**Trees and Vegetation to be removed beside embankment abutments**

## 6.0 SPILLWAY

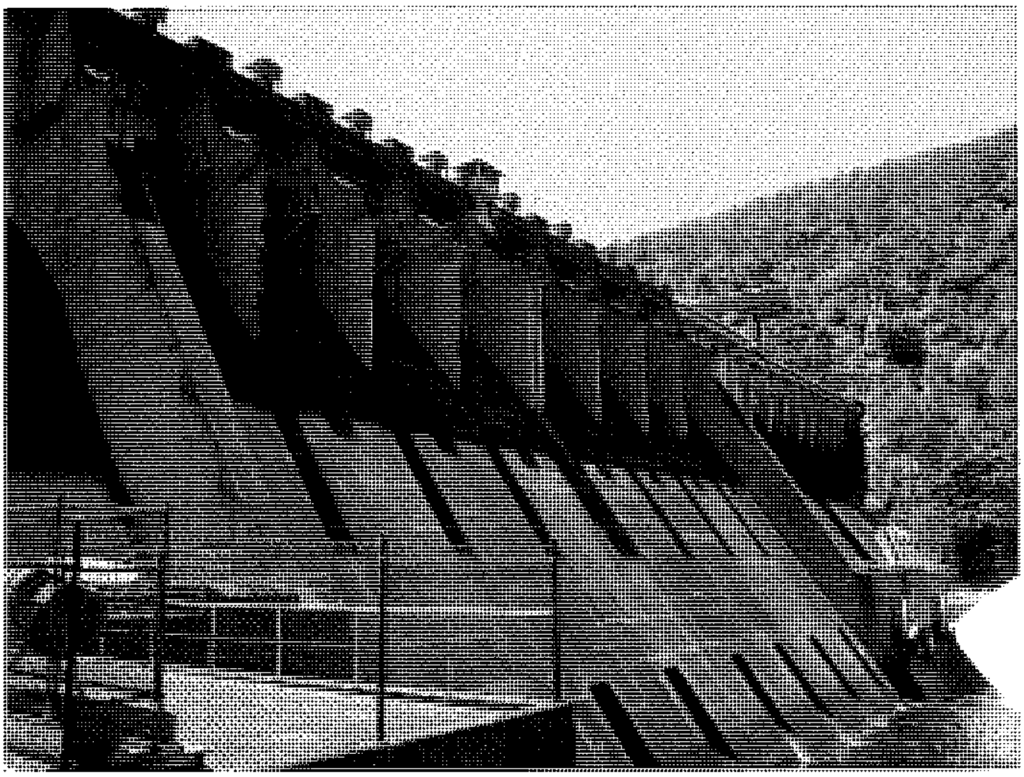
The dam has eight radial gates (sector gates) installed on the top of the spillway. The eight radial gates are each 7 metres high by 8 metres long and are installed above full supply level. The gates are counterbalanced so that the hoist does not have to lift the full weight of the gate.

The radial gate winch units comprise a six-pole electric motor close-coupled to a worm reduction gear set. The output of the worm reduction passes through three sets of spur gears, the last spur gear being bolted to the rope drum. The rope is attached directly to the centre of the gate without any intermediate pulleys, while the counterweight is attached to both ends of the gate. An electric brake operates on the motor-coupling drum. A parking brake is operated by a hand-wheel applying a band brake to a drum mounted on the last spur gear drive shaft.



**Sluice Gates hoisting gear**

The spillway and associated gates, hoisting gear and emergency coaster gate looked to be in good condition, with only some minor issues identified. Undertaking regular routine maintenance in accordance with the dam Operation and Maintenance Manuals appears to be producing good results and it is important that this program is continued. However, it was noted that the electrical condition monitoring program seems to have been discontinued and this is of concern. Replacement of the Radial gate counter weight buffers remains outstanding from the 2008 inspection.



**Spillway**

***Inspection Recommendation:***

- ***Vegetation growing on the spillway is to be removed and/or sprayed with a suitable herbicide.***
- ***The rotten wooden buffers associated with the radial gate counter weights are to be replaced.***
- ***Recommence the electrical condition monitoring program associated with the radial gates, gantry crane and standby diesel generator.***

## 7.0 RESERVOIR RIM AND DOWNSTREAM WATERWAY

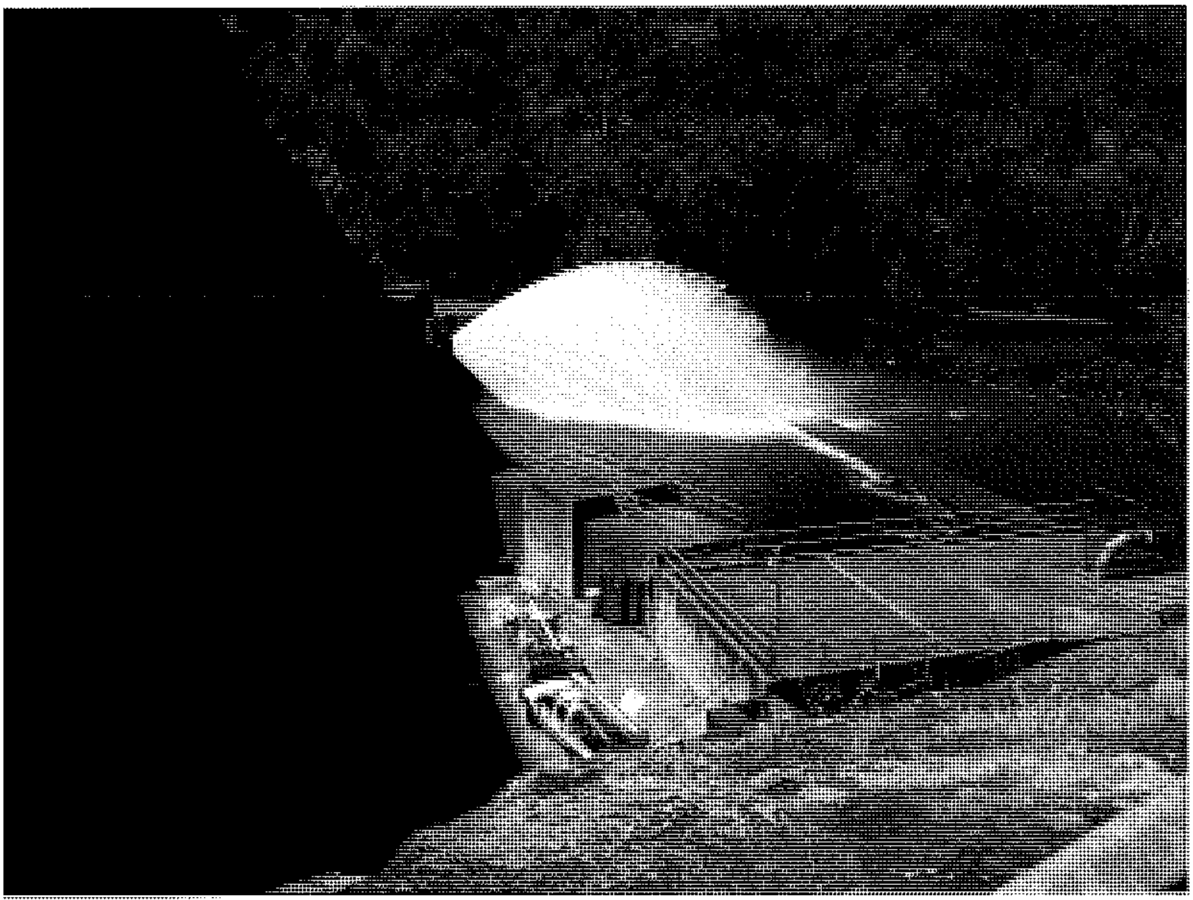
The reservoir rim slopes appear generally stable and above the Full Supply Level are relatively well vegetated with no signs of slips or movement that would be of concern from a dam safety perspective.

There were also no slips or restrictions that would prevent spillway outflow or raise tail water levels to an unacceptable level during a dam outflow event.

# 8.0 OUTLET WORKS

The outlet works consist of thirteen conduits or sluice-ways through the bottom of the dam wall. One of the conduits supplies a mini-hydro power station, four are connect to fixed cone dispersion valves and the eight sluice-ways constitute the main outlet regulating capacity.

The eight main sluice gates are each 3.7 metres high by 2.4 metres wide. The gates are not counterbalanced, and are hoisted by two ropes, each rope being reeved into a four-part system. The conduits connected to the mini-hydro and the fixed cone dispersion valves are protected by similar roller gates with hoists essentially identical to the main sluice gate hoists, the differences relating to the rope drums.



**Fixed Cone Dispersion Valve**

Each winch unit comprises a six-pole electric motor close-coupled to a worm reduction gear set. The output of the worm reduction passes through two sets of spur gears, the last spur gear being bolted to the rope drum. The rope drum is a double drum with two ropes attached. Each rope is reeved through pulleys to create a four-fall rope system connected to an equalising beam on the top of the gate. An electric brake operates on the motor to worm pinion coupling. A band brake is hand-wheel applied to a drum bolted to the rope drum for added security.

A 100 tonne travelling gantry crane on the dam deck serves to handle the emergency coaster gate used for maintenance of the sluice gates. This crane appeared to be in good condition, however it was noted that some recommendations from the 2008 mechanical inspection of the crane remain outstanding.

**Gantry Crane**

The other mechanical equipment associated with the outlet works was inspected and found to be in generally good condition. The planned maintenance program for refurbishing the sluices, tunnels, regulators and regulator conduits is producing good results and it is important that this program is continued with Conduit 13, with associated sluice and regulator, scheduled for refurbishment in the current financial year.

There is a concern that the electrical condition monitoring program seems to have been discontinued and recommendations from the last condition monitoring round associated with repairing the electric brakes on the sluice hoisting gear has not been attended to. Internal inspection of the conduits and valves had occurred within the last five years and will occur again at or before the next five year comprehensive inspection.

The main concern associated with the outlet works is the condition of the guides associated with the placement of the coaster gate. These guides were last inspected over 5 years ago and a diving inspection should be programmed within the next 12 months to assess the condition of this infrastructure.

*Inspection Recommendations:*

- ***Recommence the electrical condition monitoring program associated with the sluice gates, radial gates, regulators, gantry crane and standby diesel generator.***
- ***The electric brakes on the sluice gates are to be repaired in accordance with the recommendations made during the last round of condition monitoring.***
- ***An underwater inspection of the coaster gate guides is to be undertaken as soon as possible.***
- ***The works recommendations arising from the August 2008 crane inspection are to be completed as soon as possible.***

# 9.0 INSTRUMENTATION

Surveillance instrumentation at the dam monitors movement of the dam embankment, seepage and pressure within the embankment. The instrumentation consists of:

- 34 crack measurement points.
- 13 vibrating wire piezometers.
- 56 pressure relief wells.
- 1 automatic water level recorder

Graphs of the piezometer and uplift pressure data are shown in Appendix A. This data was examined and no trends of concern were identified. Some spikes are apparent in the pressure relief wells during the May 2009 flood event. Historically, these spikes seem to occur when the sluice gates are opened, but return to normal soon after the sluice gates are closed. No associated spikes in the piezometers are apparent.

The crack is the upper gallery continues to slowly widen, with an average increase in crack width of around 0.5 millimetres in the last ten years. This issue was last examined several years ago and it is recommended that the issue be revisited to fully understand any dam safety issues associated with this cracking.

The instrumentation was then inspected, and the following works recommendations were made:

***Inspection Recommendations:***

- ***All measuring points are to be suitably labelled and numbered on site and a suitable engineering plan prepared to show instrumentation point locations and corresponding numbering.***
- ***Re-examine the dam safety issues associated with the concrete cracking in the upper gallery.***

## 10.0 RECOMMENDATIONS

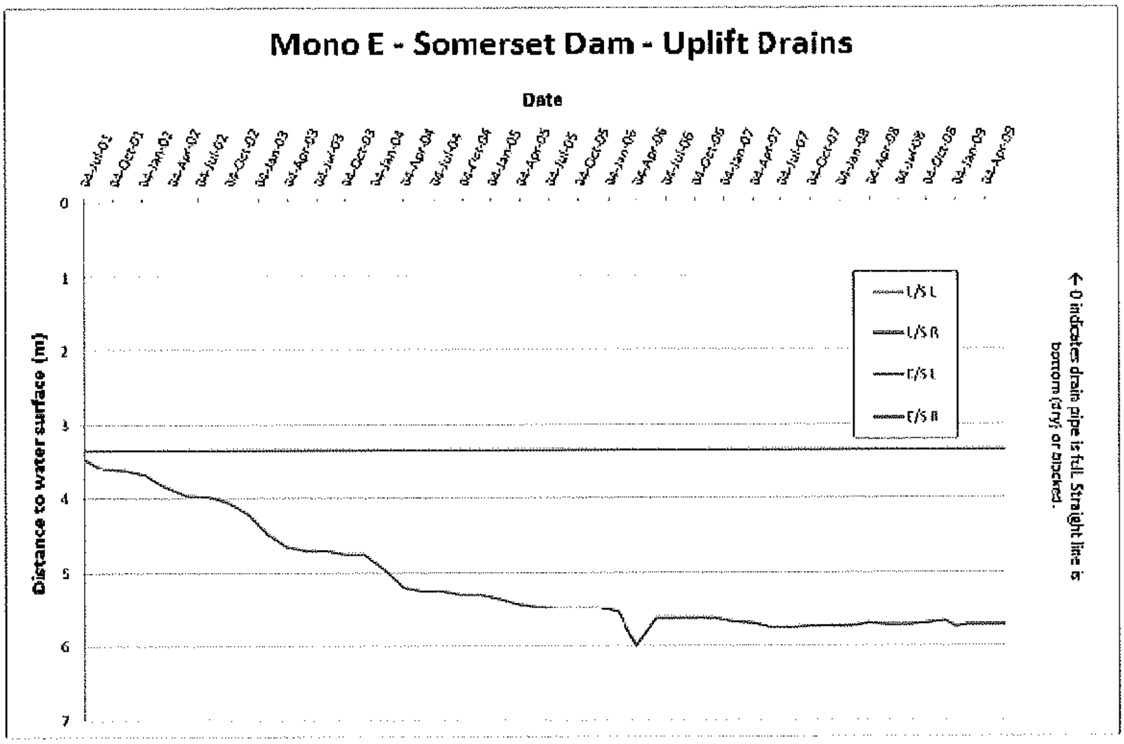
**ANNUAL DAM SAFETY INSPECTION SEPTEMBER 2008**

| Section Reference | Recommendation | Rating (See Below) |
|---|---|---|
| 5.0 | • *Vegetation growing on the embankment is to be removed and/or sprayed with a suitable herbicide. This includes the trees growing within five metres of the abutments.* | 3 |
| 6.0 | • *Vegetation growing on the spillway is to be removed and/or sprayed with a suitable herbicide.* | 3 |
| | • *The rotten wooden buffers associated with the radial gate counter weights are to be replaced.* | 2 |
| | • *Recommence the electrical condition monitoring program associated with the radial gates, gantry crane and standby diesel generator.* | 1 |
| 8.0 | • *Recommence the electrical condition monitoring program associated with the sluice gates, radial gates, regulators, gantry crane and standby diesel generator.* | 1 |
| | • *The electric brakes on the sluice gates are to be repaired in accordance with the recommendations made during the last round of condition monitoring.* | 1 |
| | • *An underwater inspection of the coaster gate guides is to be undertaken as soon as possible.* | 2 |
| | • *The works recommendations arising from the August 2008 crane inspection are to be completed as soon as possible.* | 2 |
| 9.0 | • *All measuring points are to be suitably labelled and numbered on site and a suitable engineering plan prepared to show instrumentation point locations and corresponding numbering.* | 3 |
| | • *Re-examine the dam safety issues associated with the concrete cracking in the upper gallery.* | 3 |

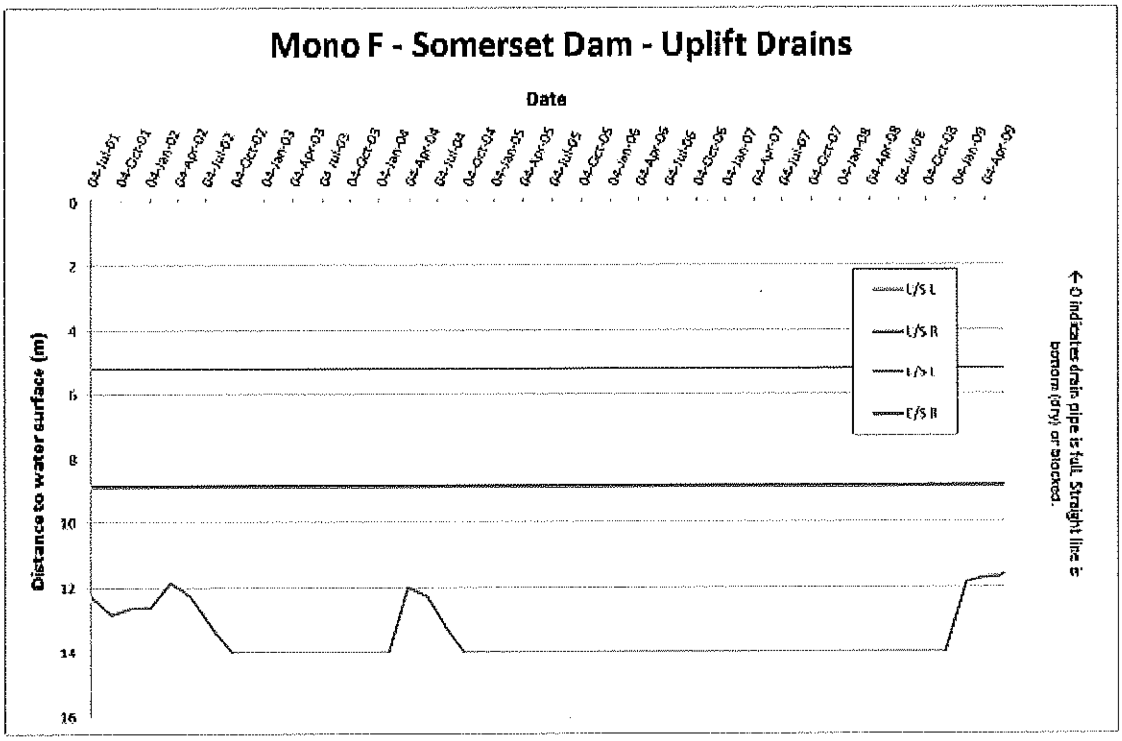
**Legend of Criticality Rating**

| | |
|---|---|
| Rating 1 | Rectification required immediately, i.e. within 1 month |
| Rating 2 | Rectification required within 3 months |
| Rating 3 | Rectification required within 12 months |
| Rating 4 | Ongoing |

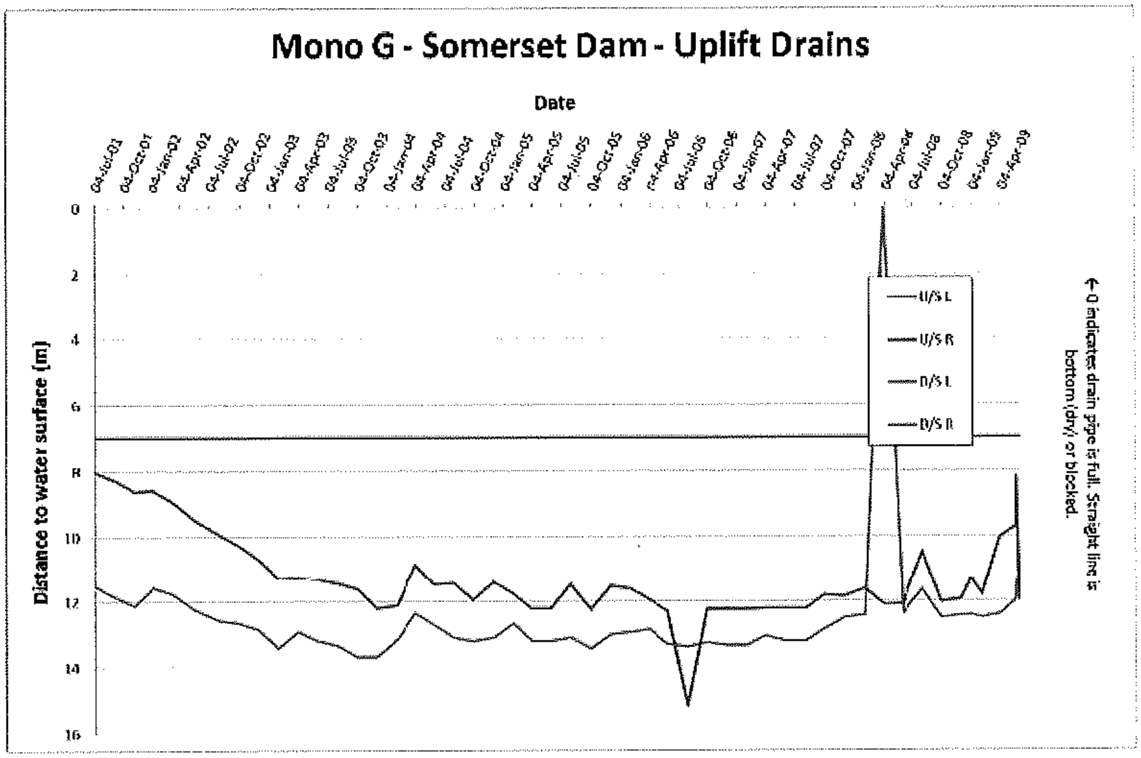
# APPENDIX A
# INSTRUMENTATION DATA

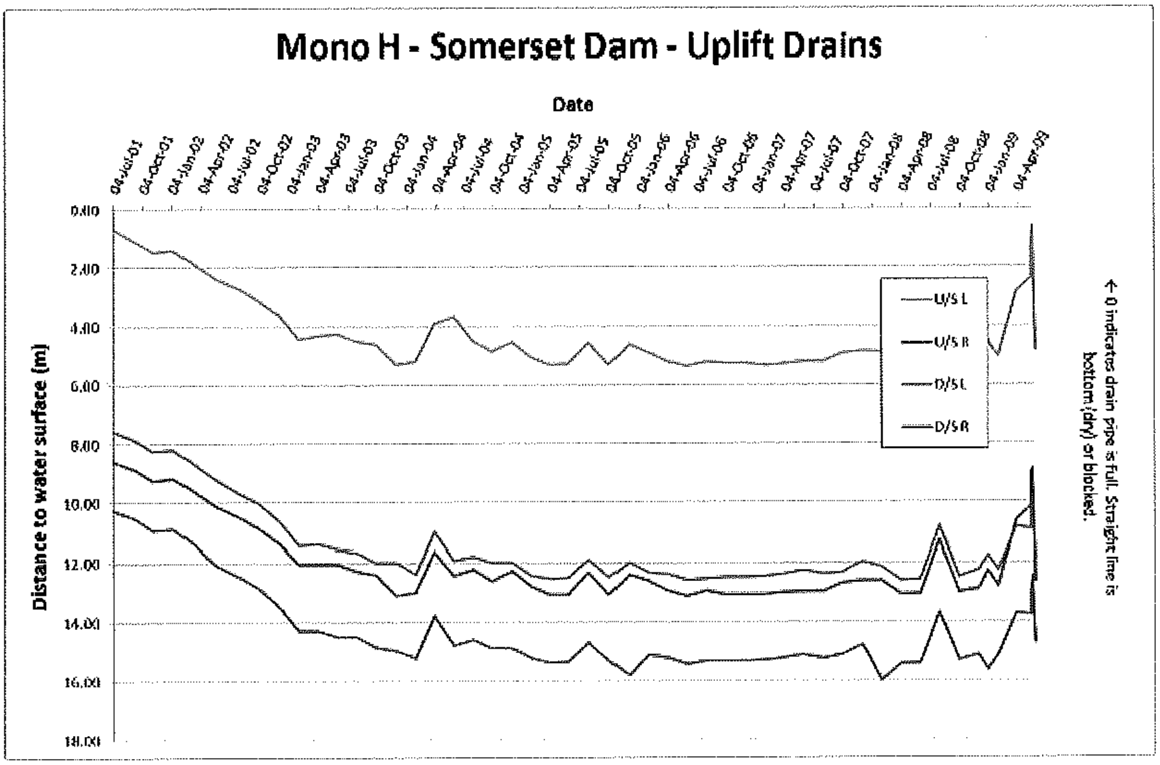


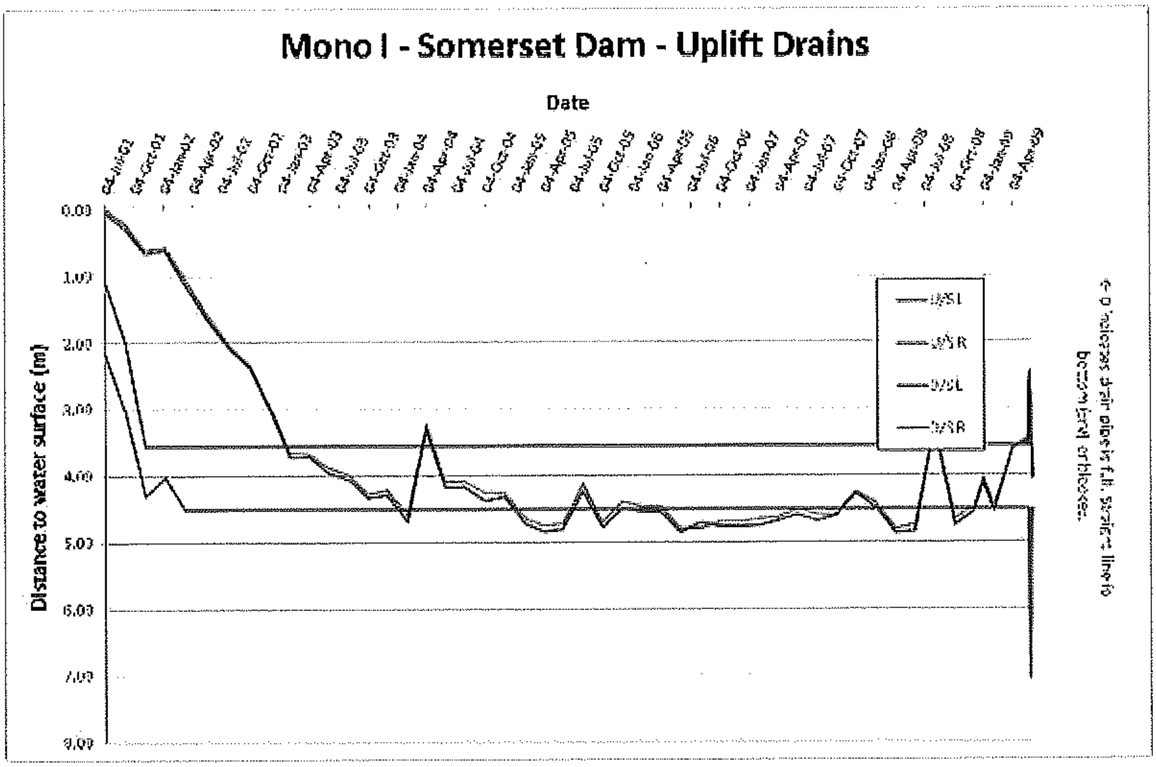
Mono E - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

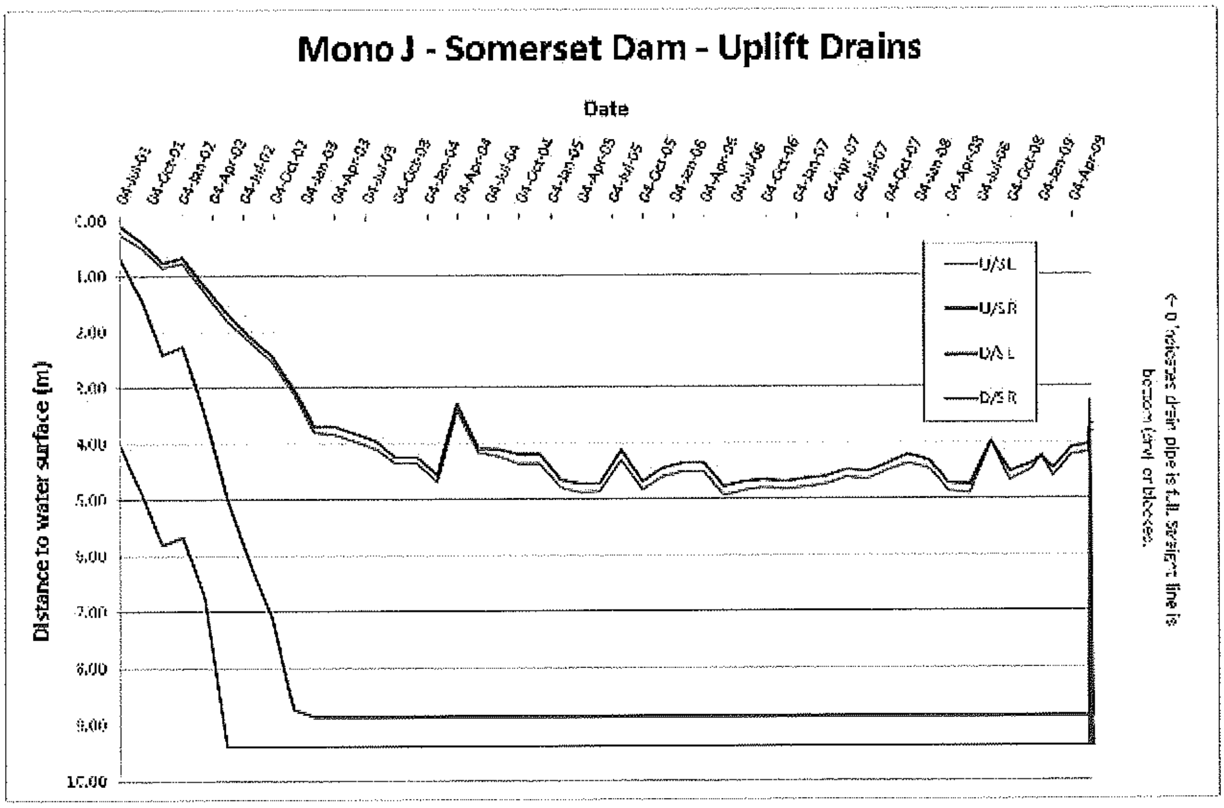



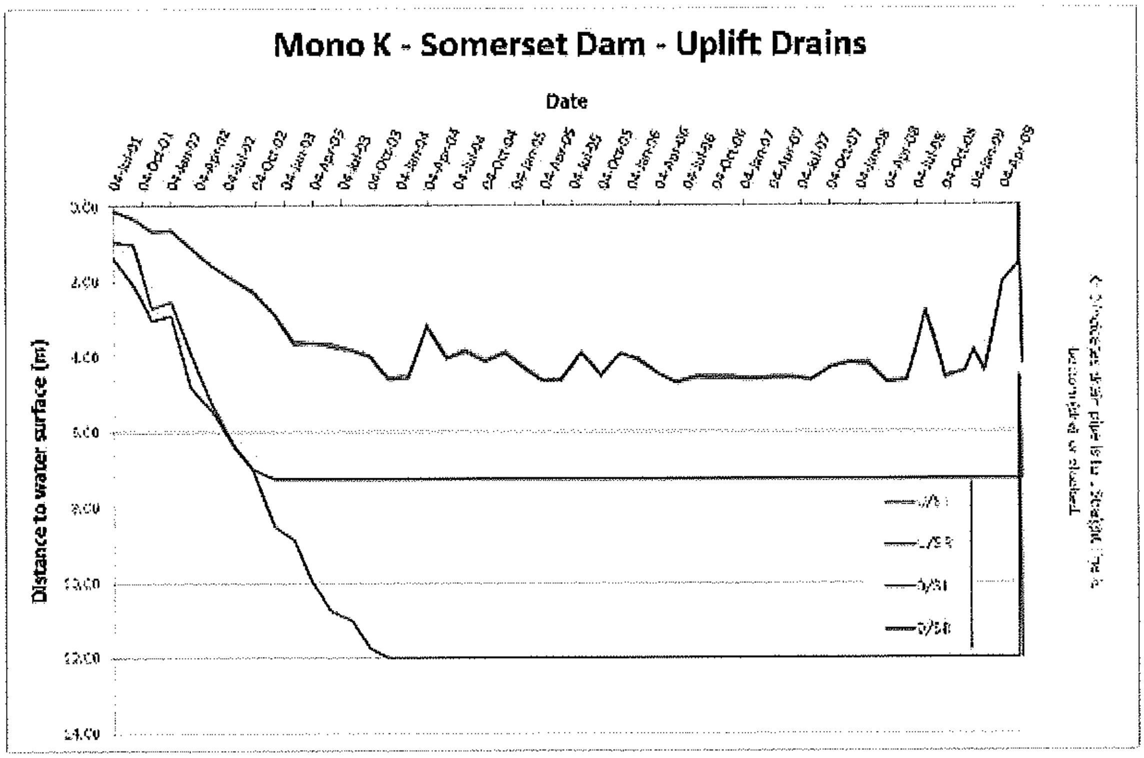
Mono F - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.


Mono G - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

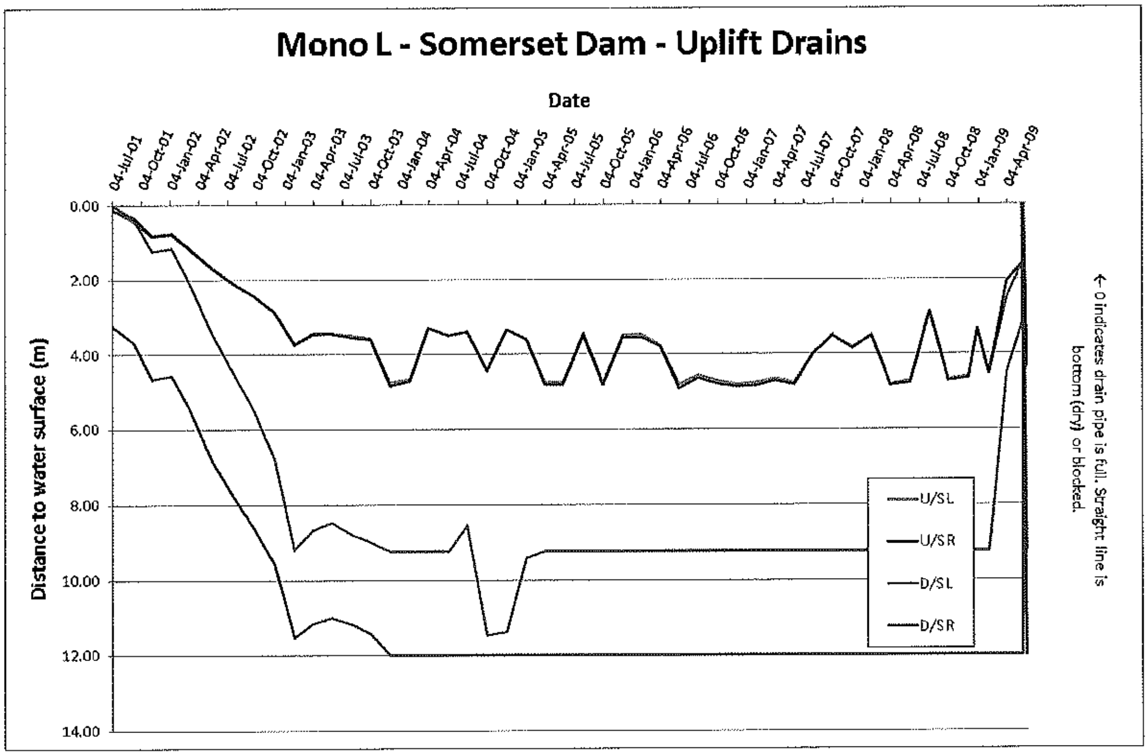

Mono H - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.


Mono I - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom level or blocked.


Mono J - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom level or blocked.



Mono K - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)

Mono L - Somerset Dam - Uplift Drains
Date
04-Jul-01
04-Oct-01
04-Jan-02
04-Apr-02
04-Jul-02
04-Oct-02
04-Jan-03
04-Apr-03
04-Jul-03
04-Oct-03
04-Jan-04
04-Apr-04
04-Jul-04
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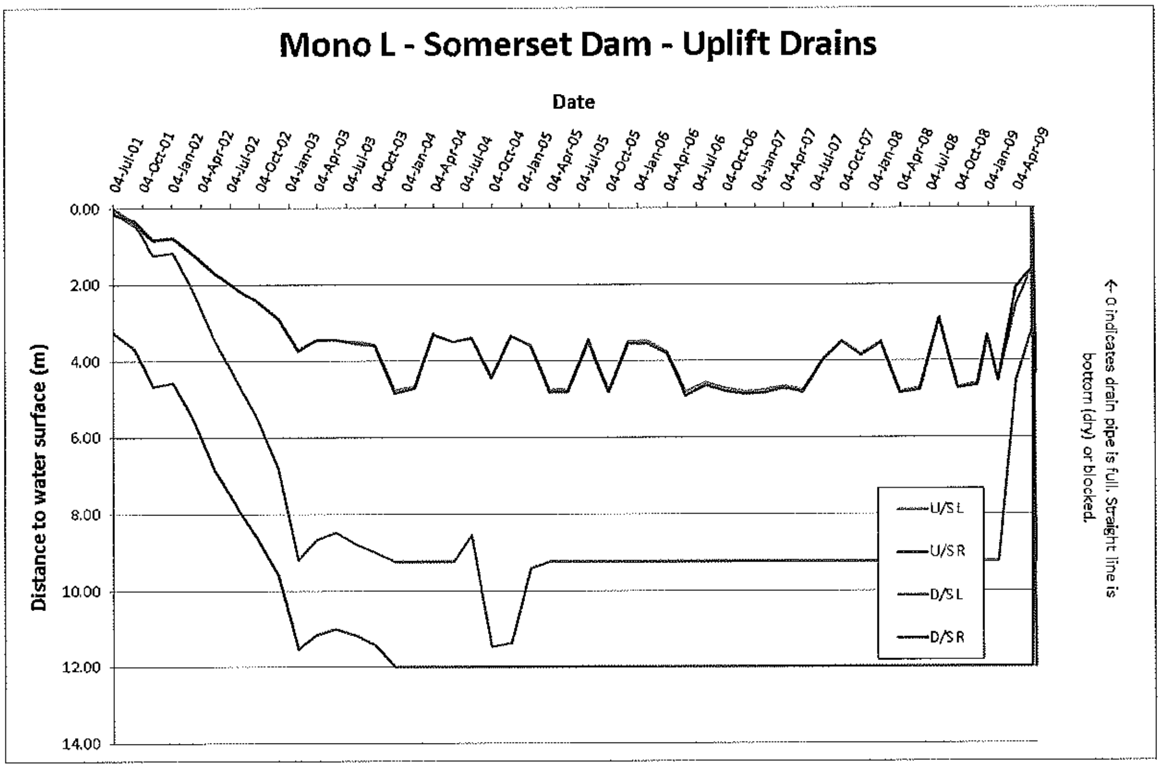
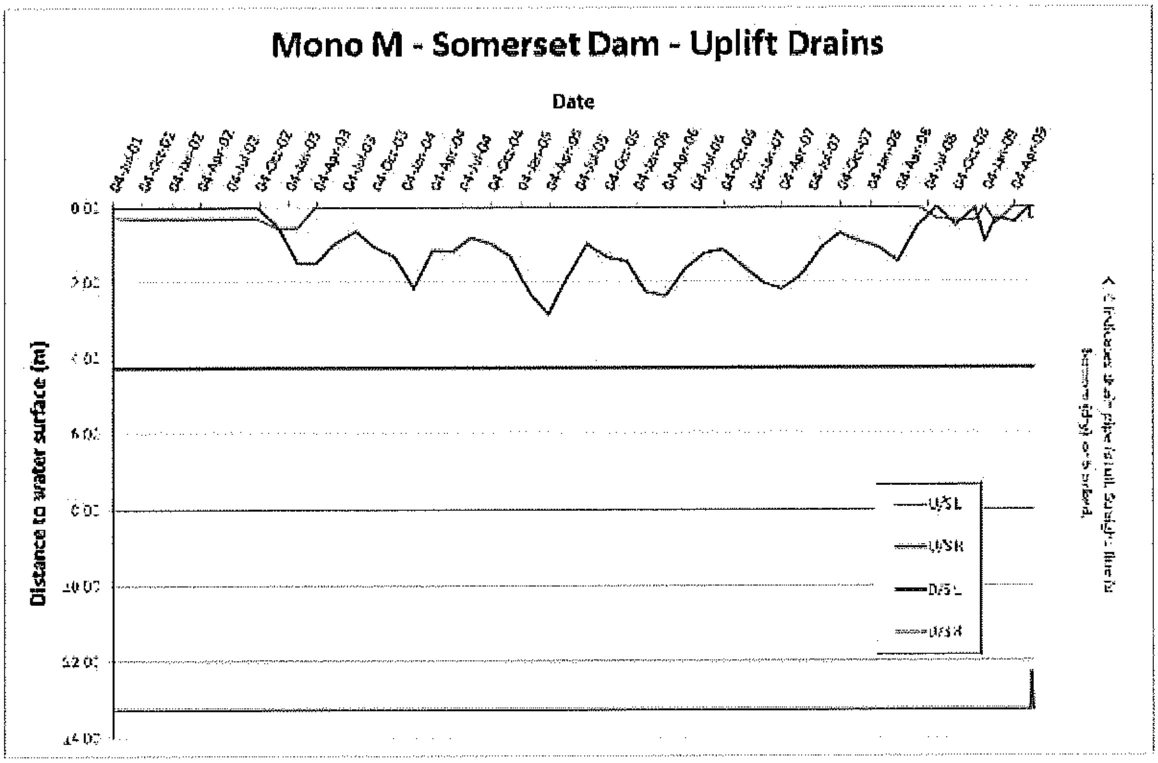
04-Apr-07
04-Jul-07
04-Oct-07
04-Jan-08
04-Apr-08
04-Jul-08
04-Oct-08
04-Jan-09
04-Apr-09
Distance to water surface (m)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
U/SL
U/SR
D/SL
D/SR
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.


Mono L - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
<- 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.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Date
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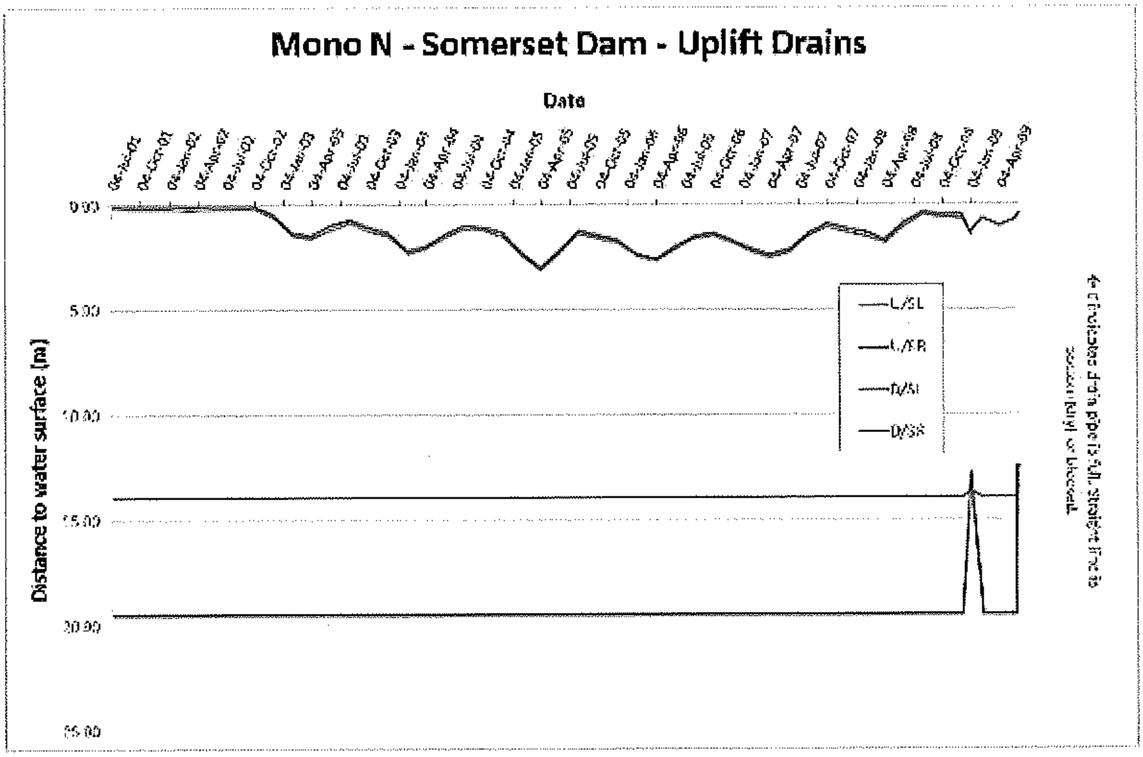
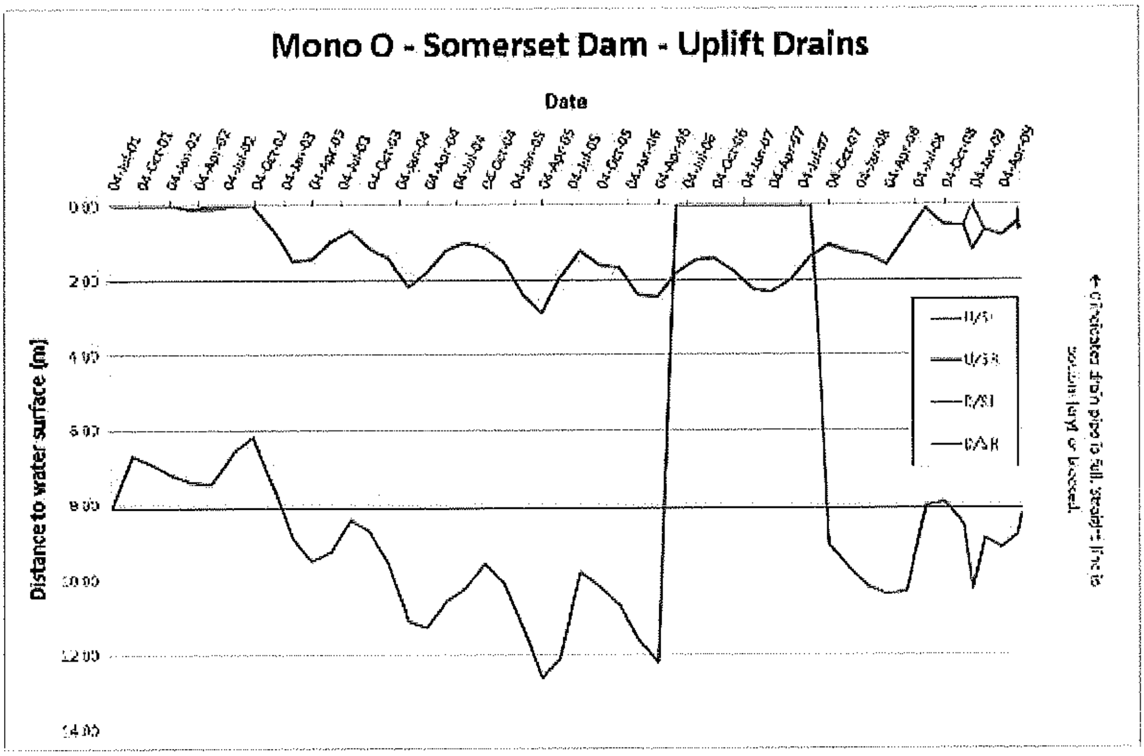
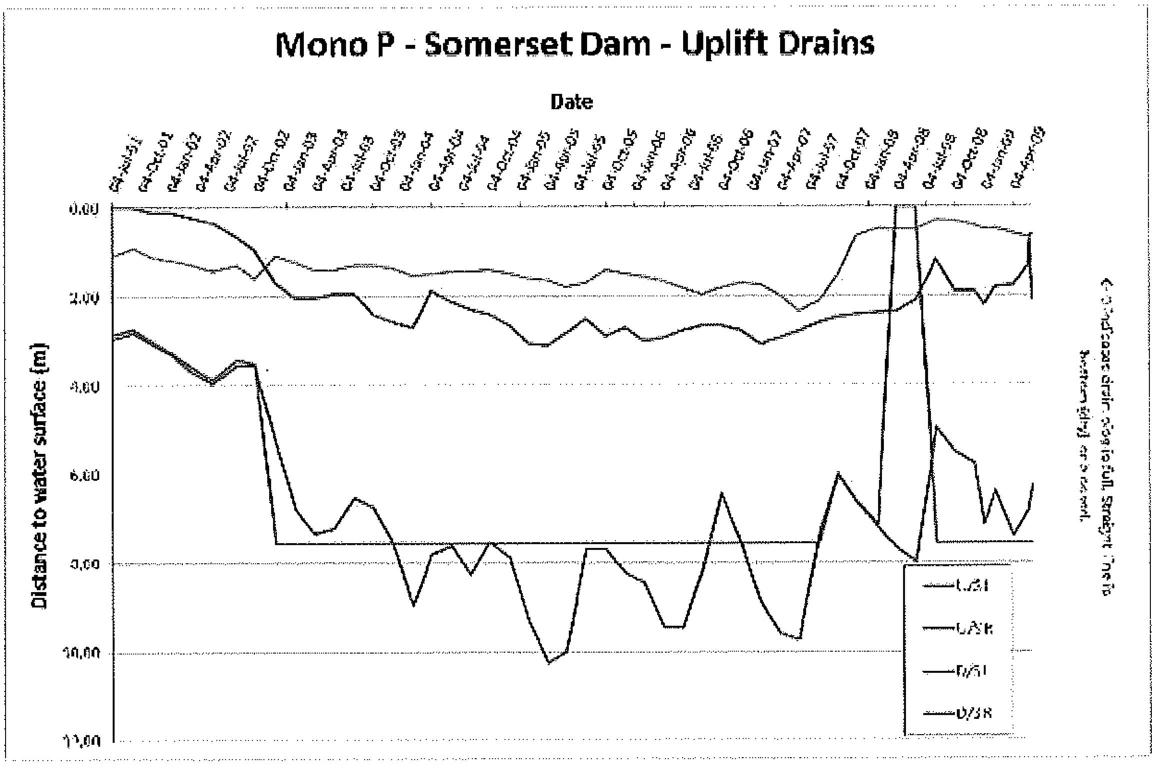
U/S R
D/S L
D/S R

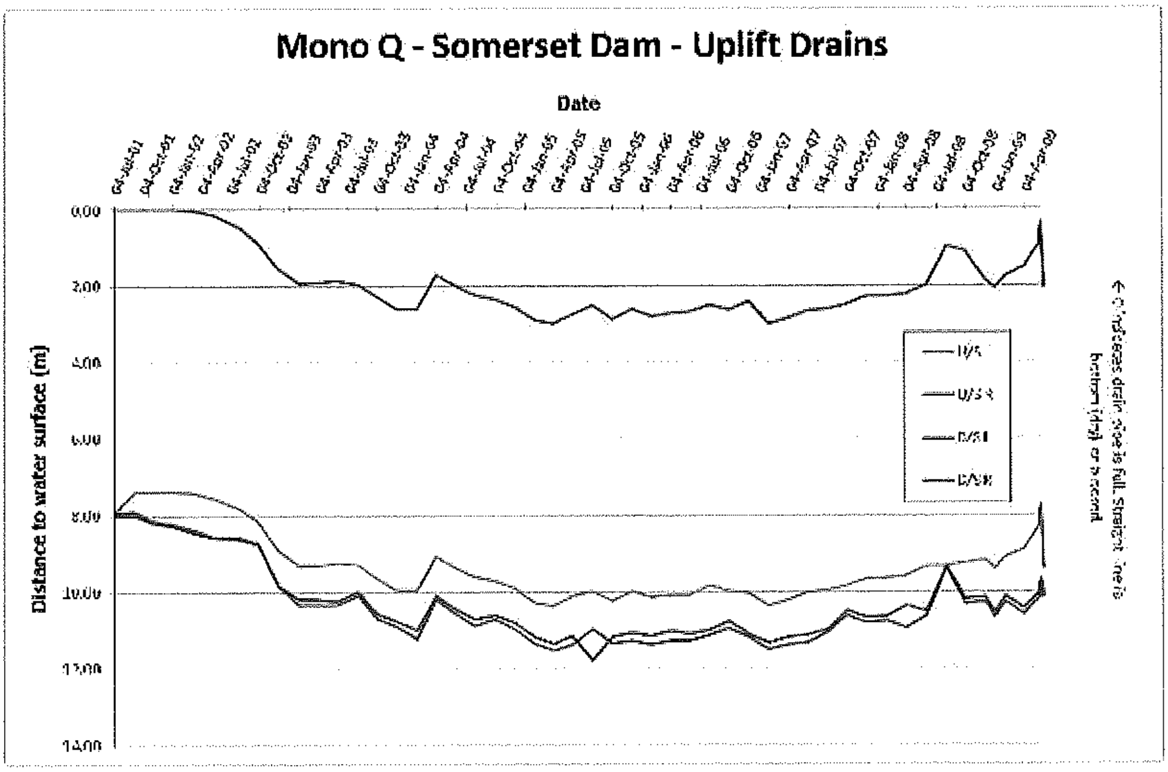


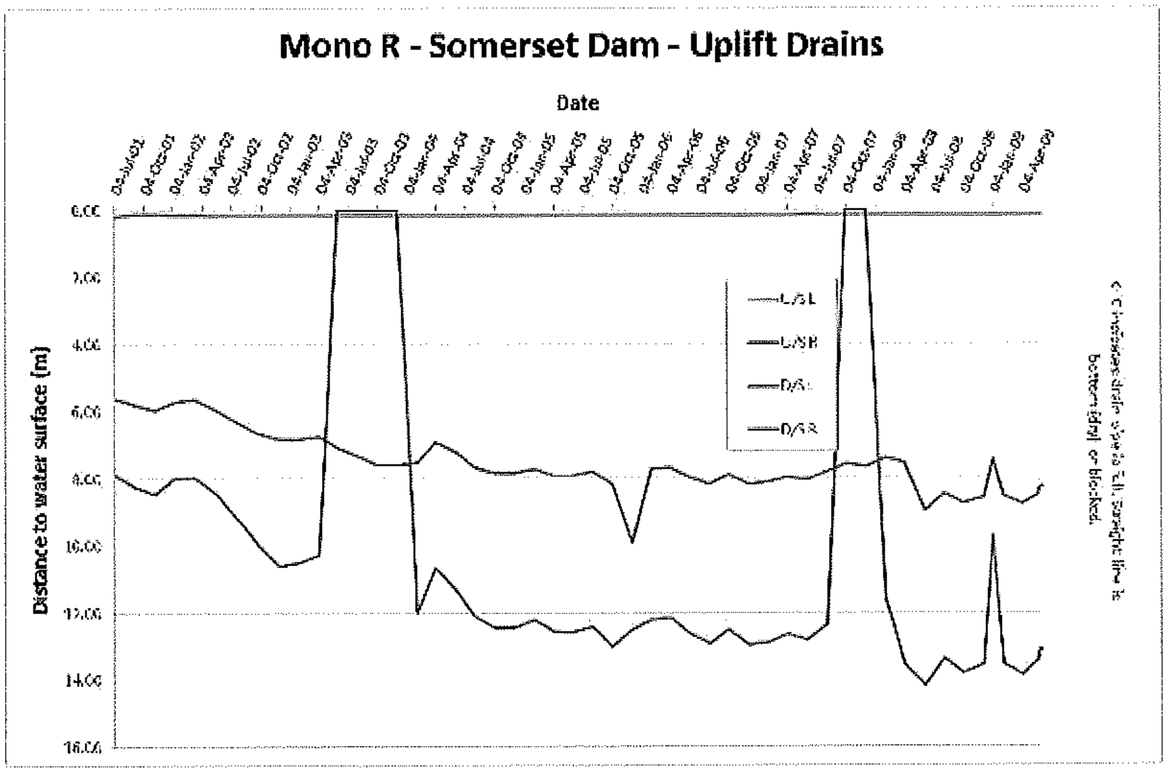
Mono N - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)

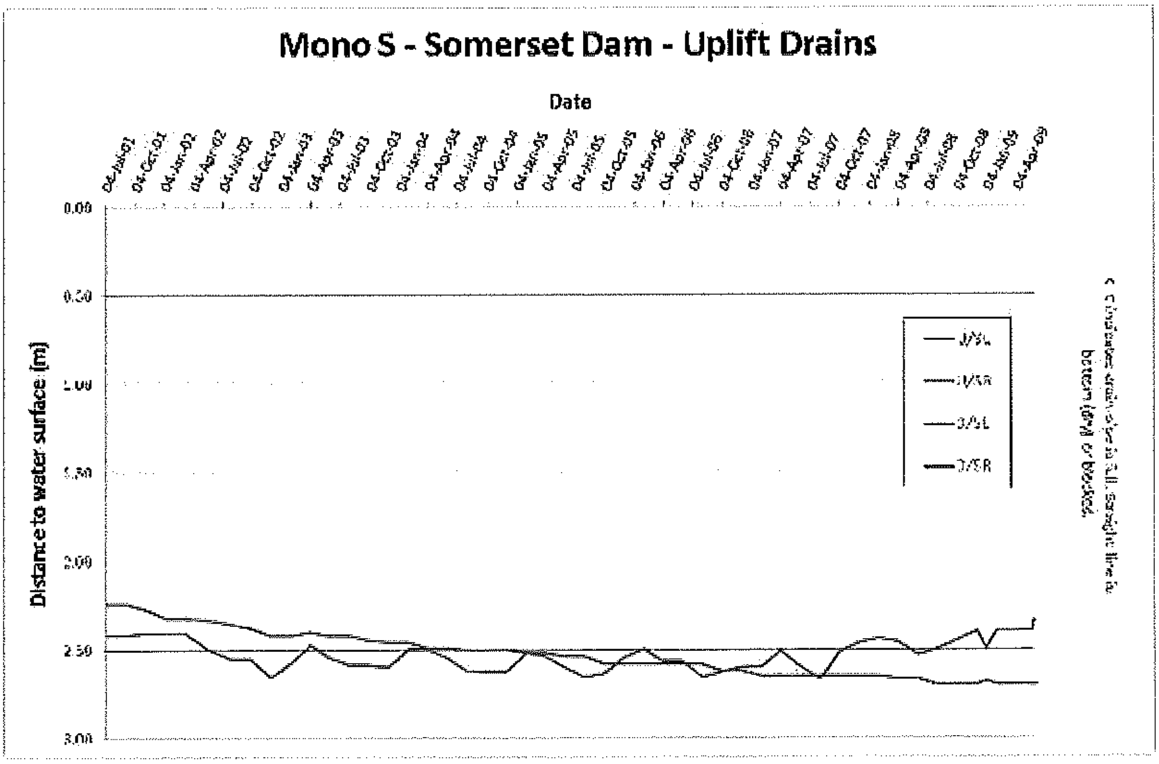



Mono O - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
D/SL
D/SR


Mono P - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R


Mono Q - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)


Mono R - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/SL
U/SR
D/SL
D/SR


Mono S - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
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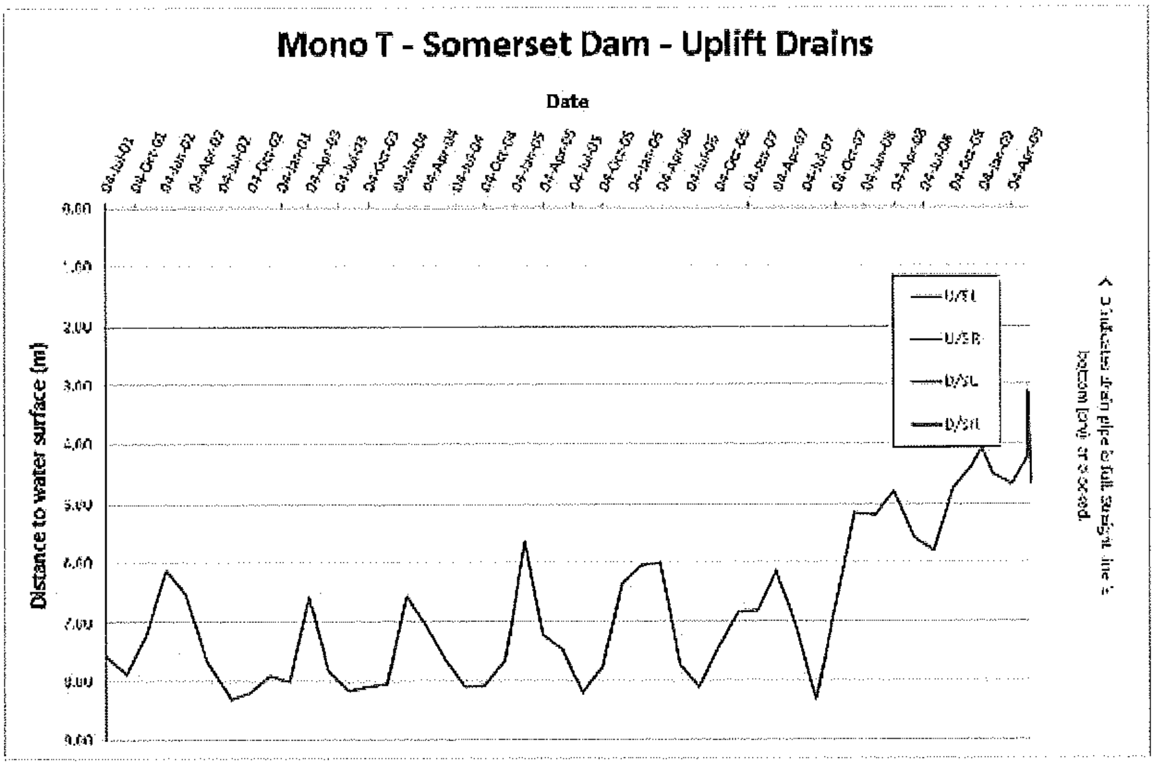
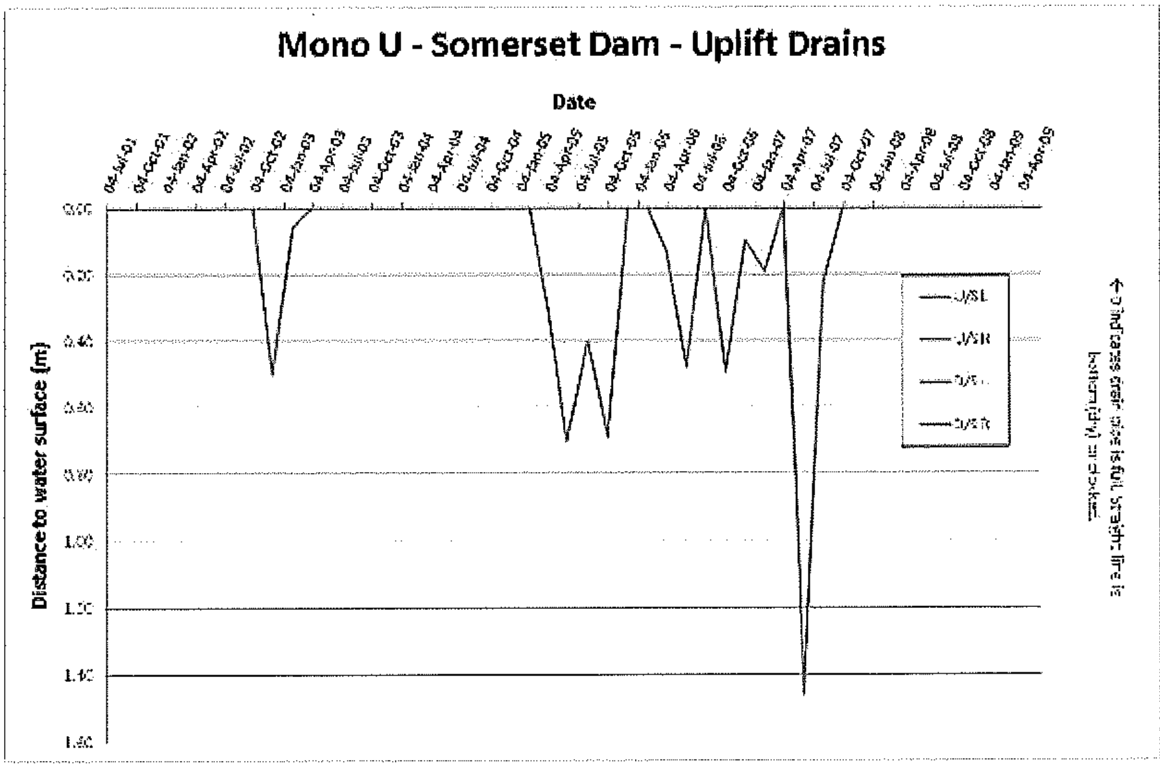
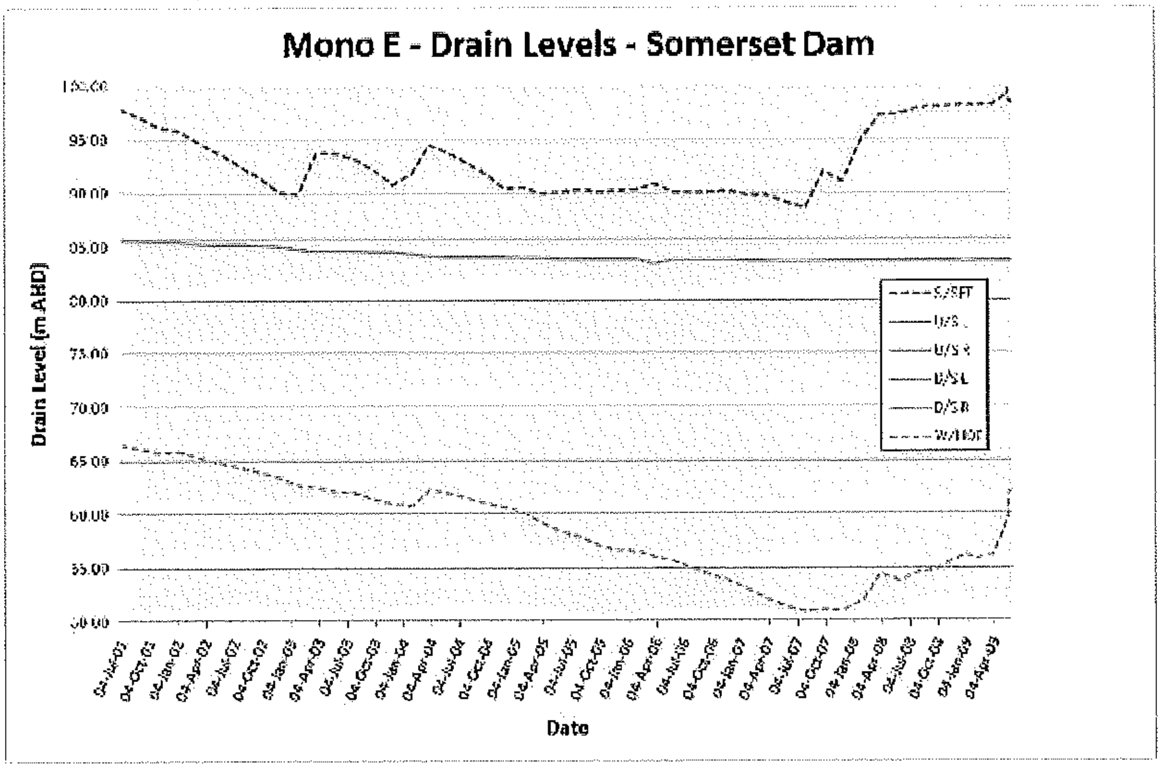
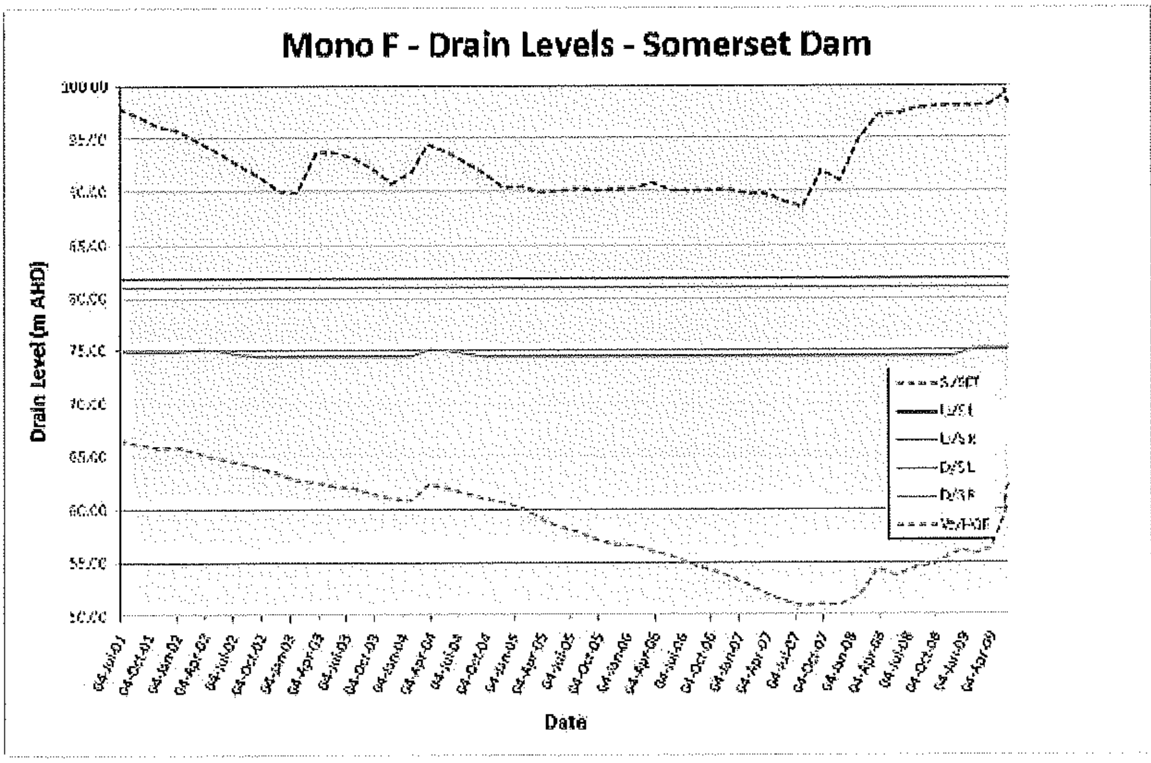
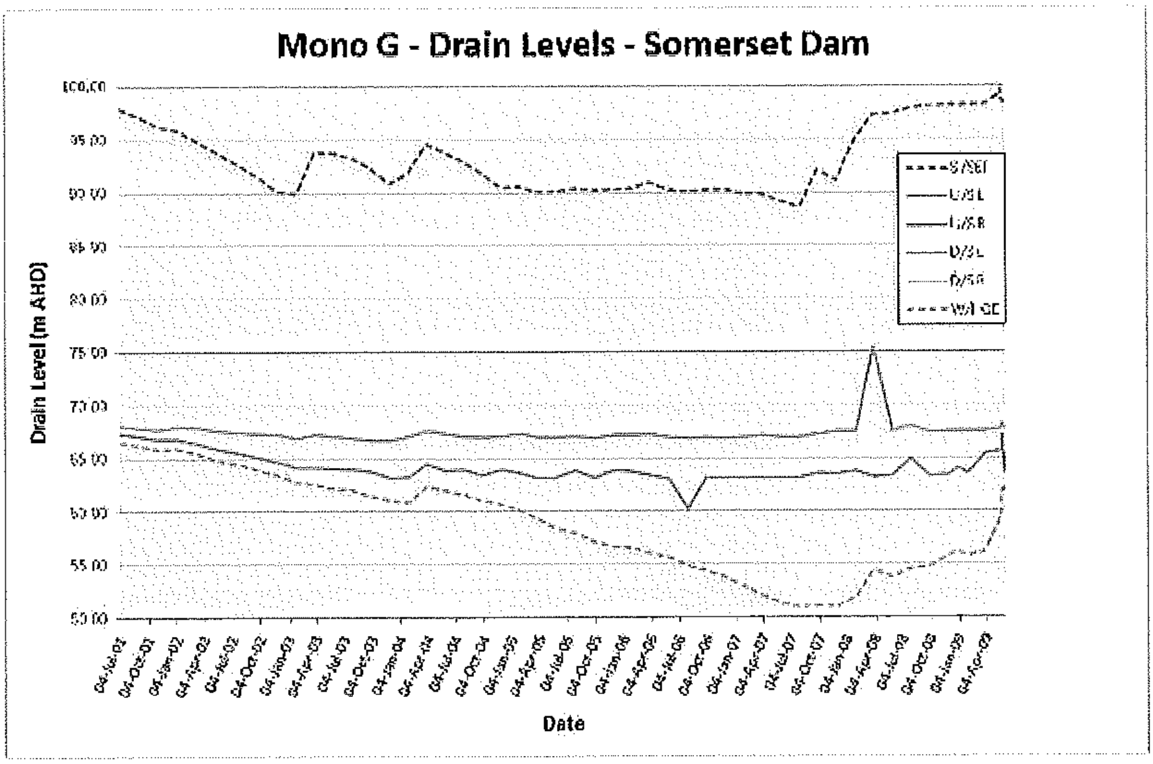
0.50
1.00
1.50
2.00
2.50
3.00

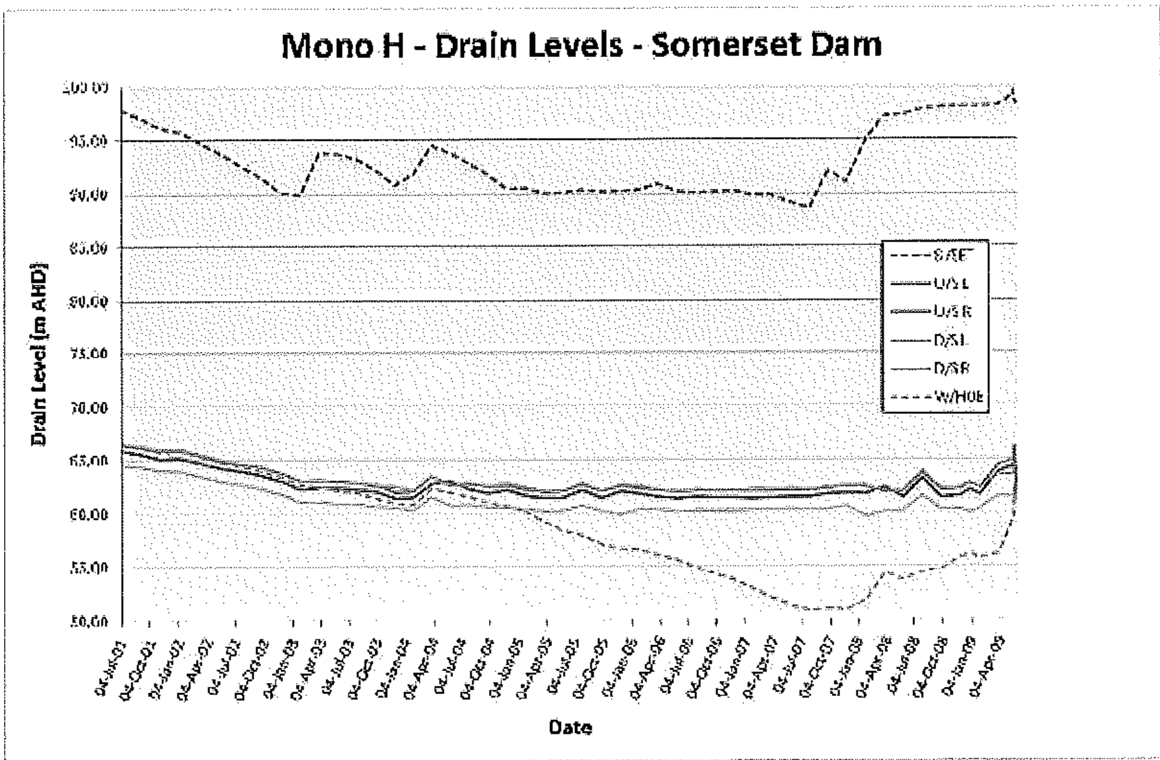

Mono T - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/SL
U/SR
D/SL
D/SR

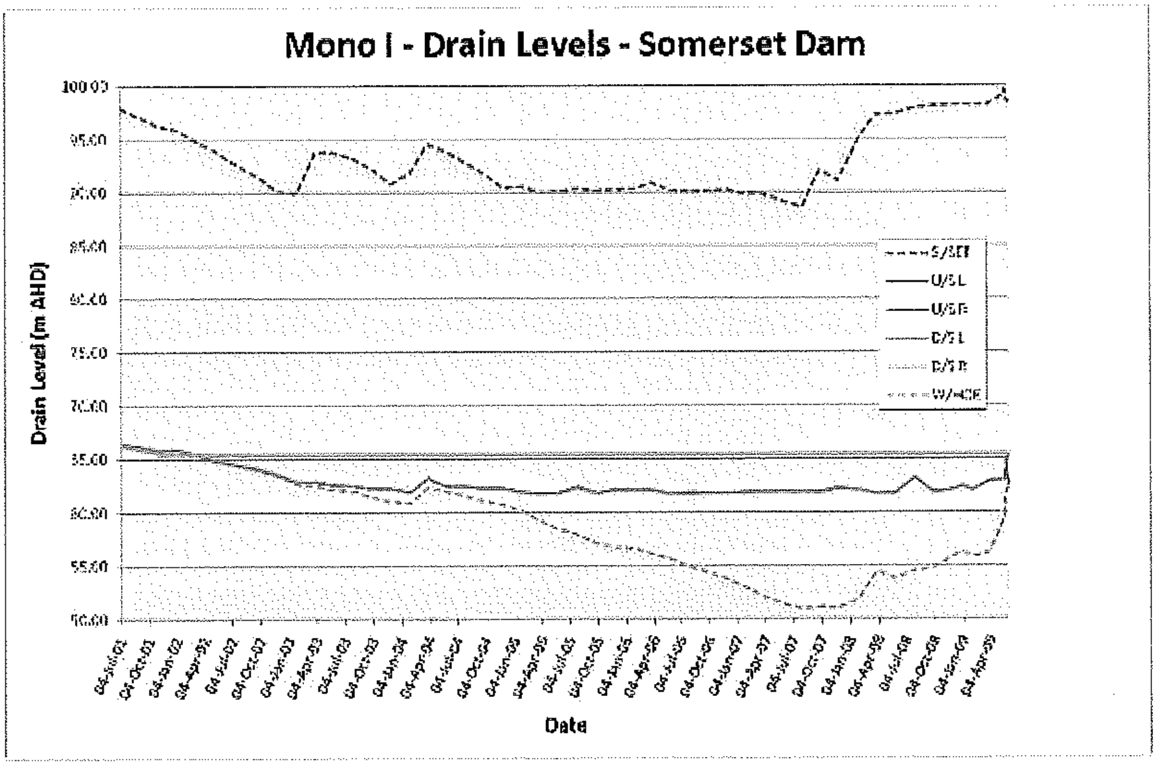

Mono U - Somerset Dam - Uplift Drains
Date
Distance to water surface (m)
U/SL
U/SR
D/SL
D/SR

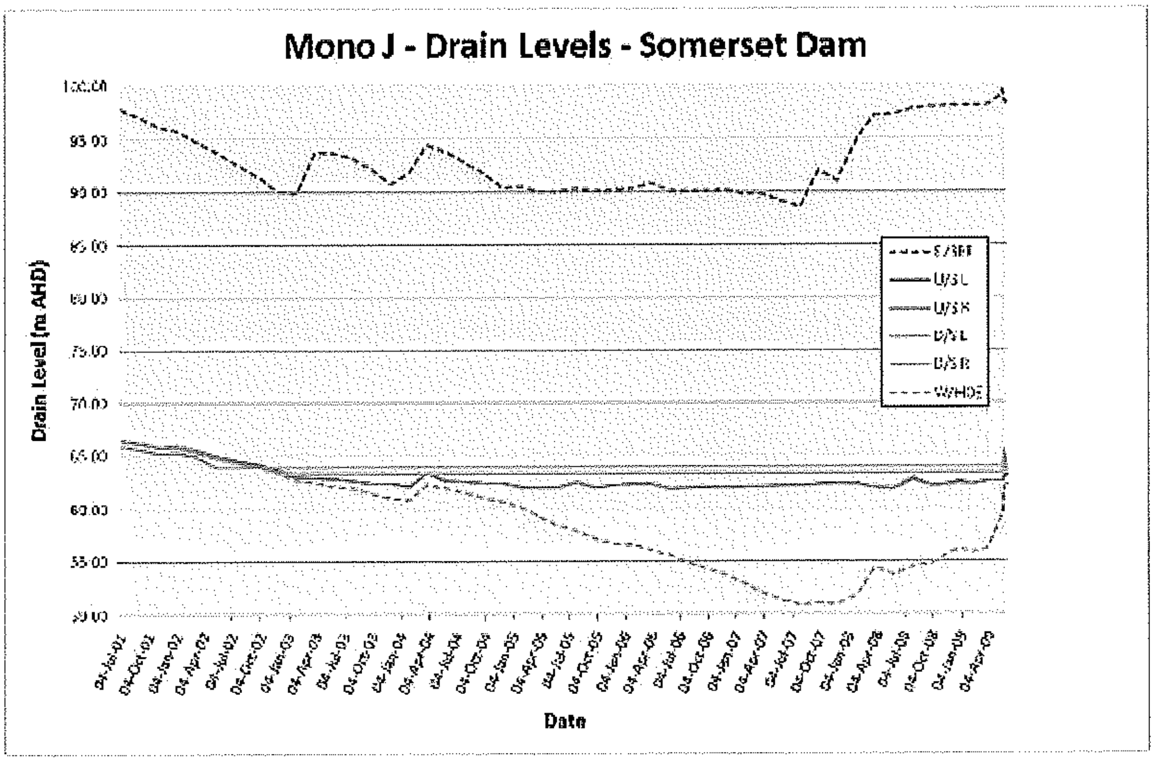

Mono E - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date
S/SET
U/S L
U/S R
D/S L
D/S R
W/HDF

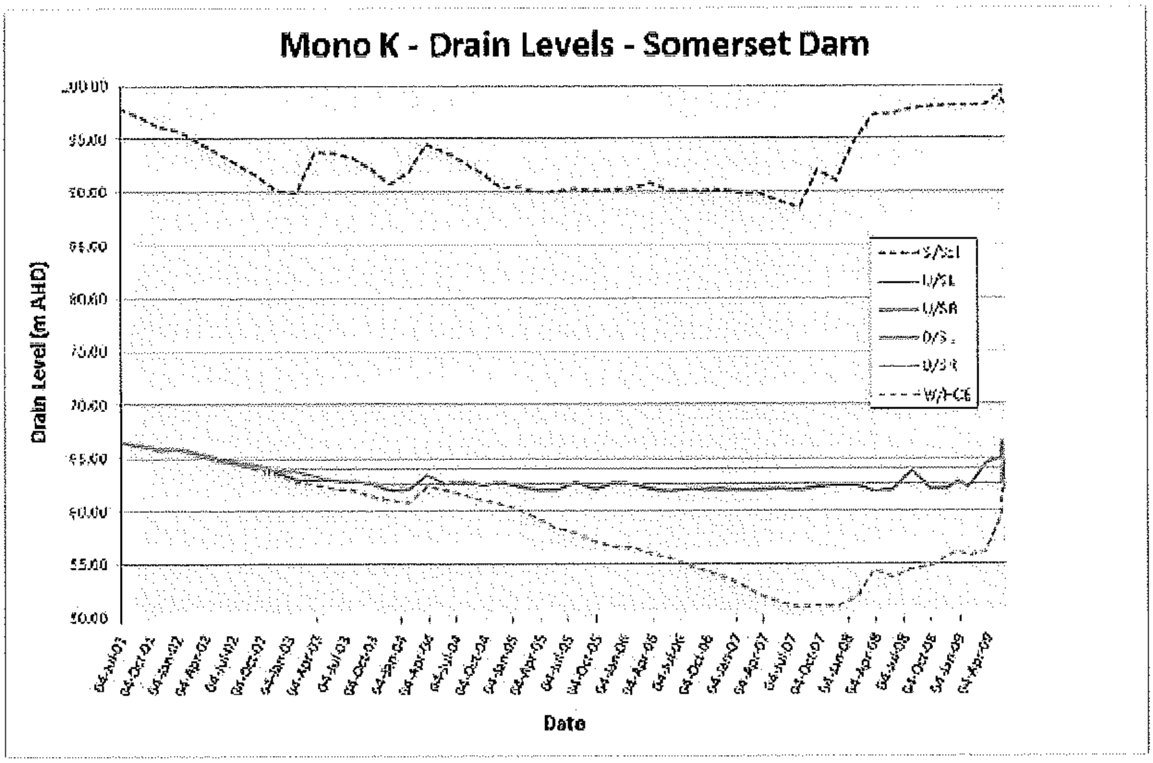

Mono F - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

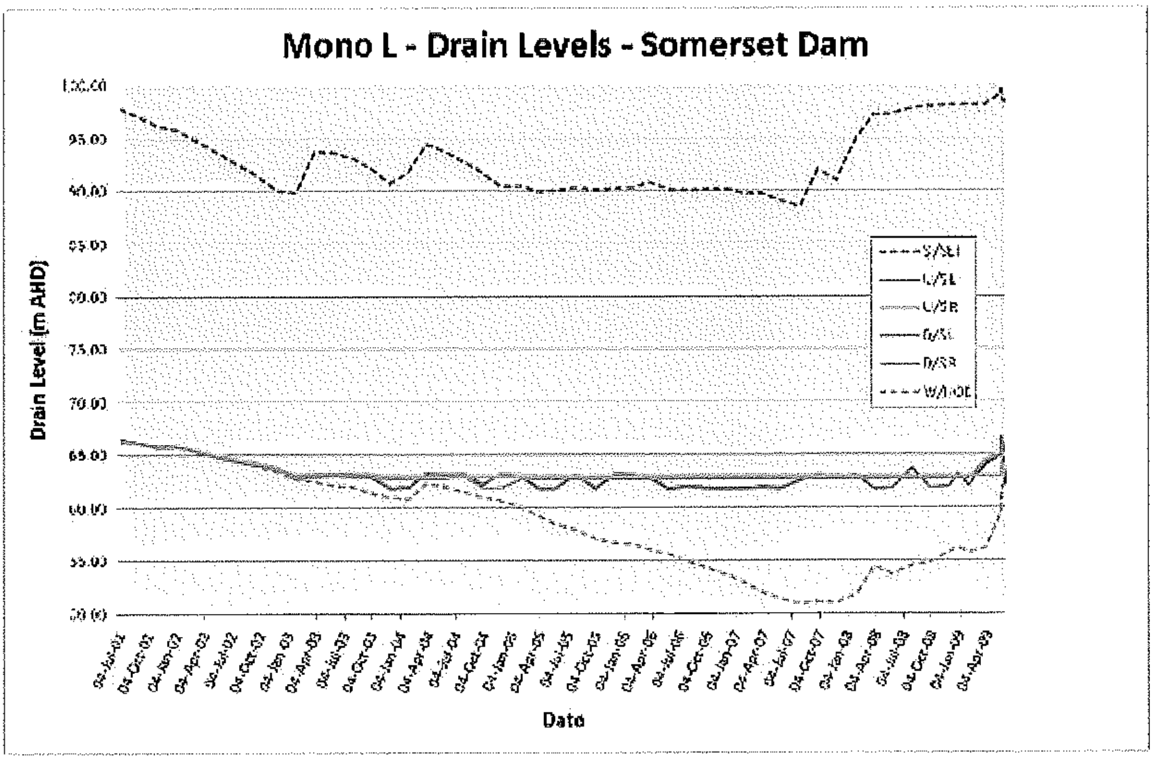

Mono G - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

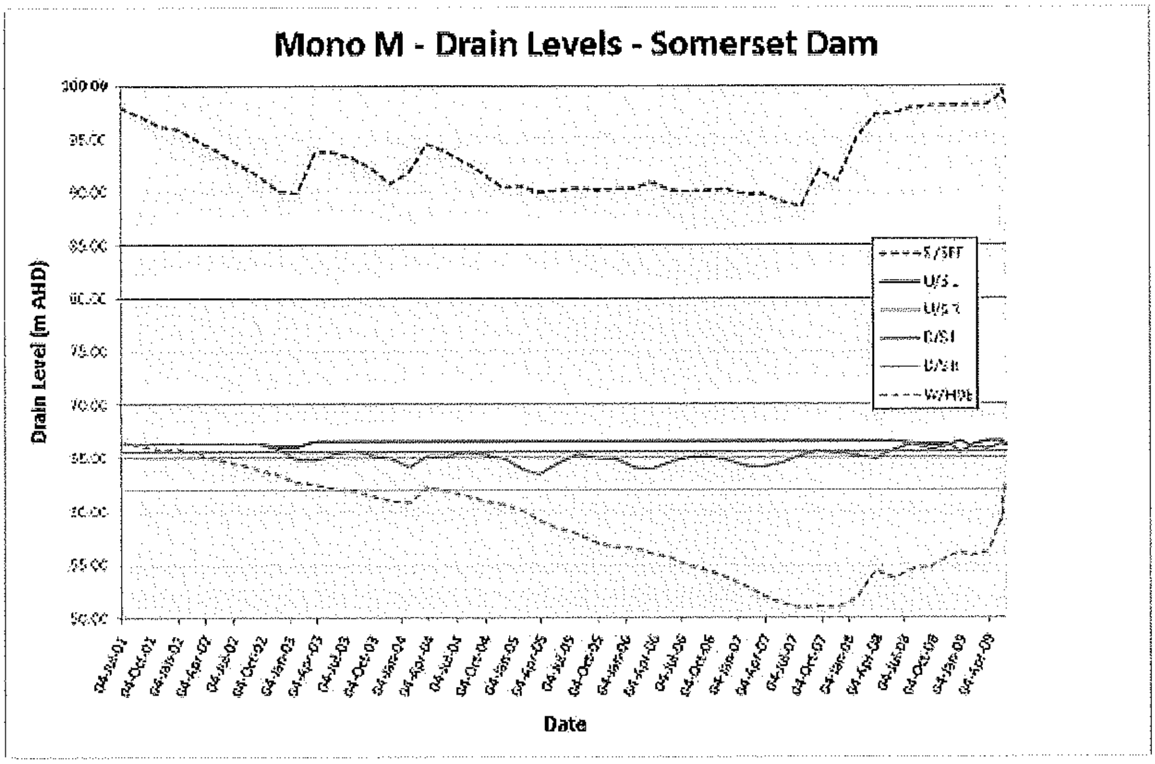

Mono H - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

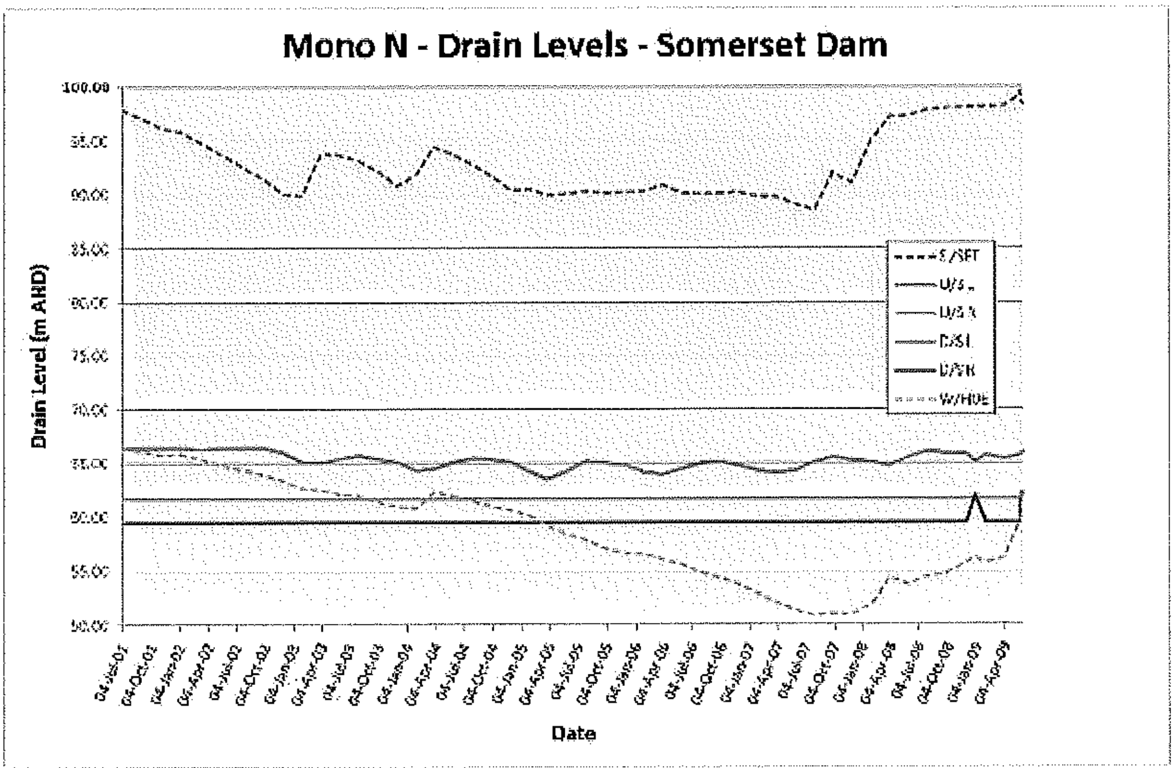

Mono I - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date
S/SET
U/S L
U/S R
D/S L
D/S R
W/HDE

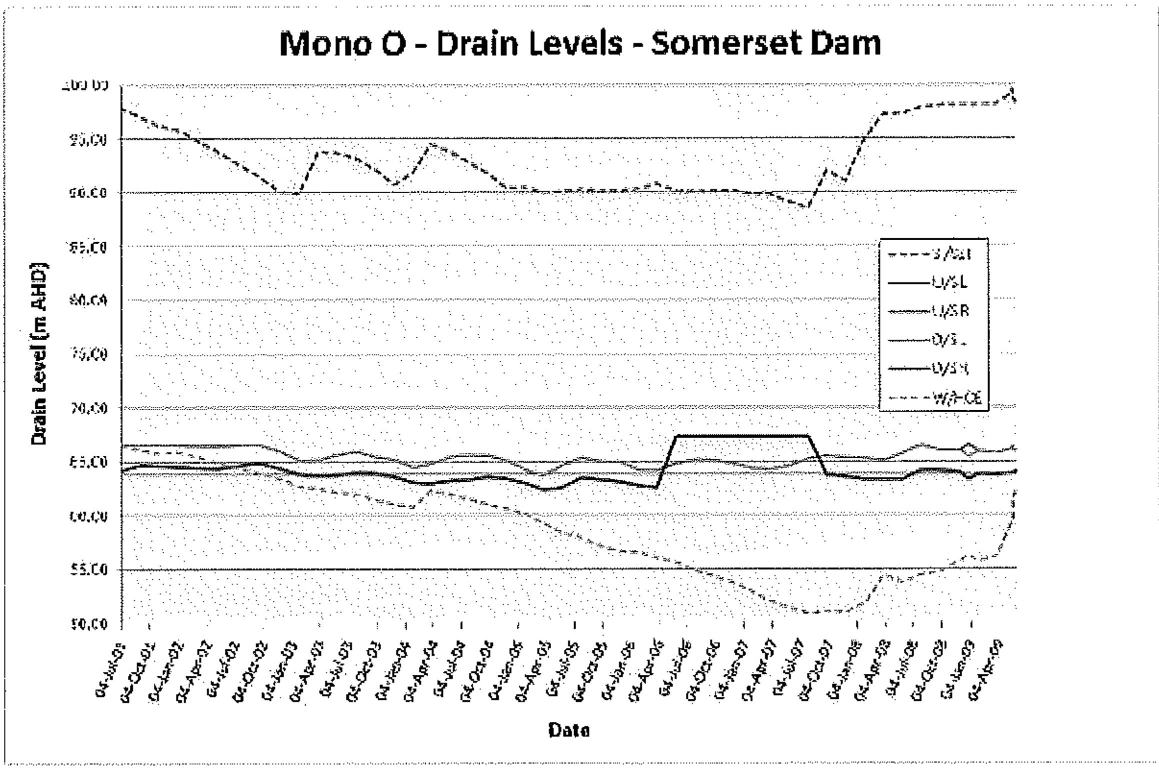

Mono J - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

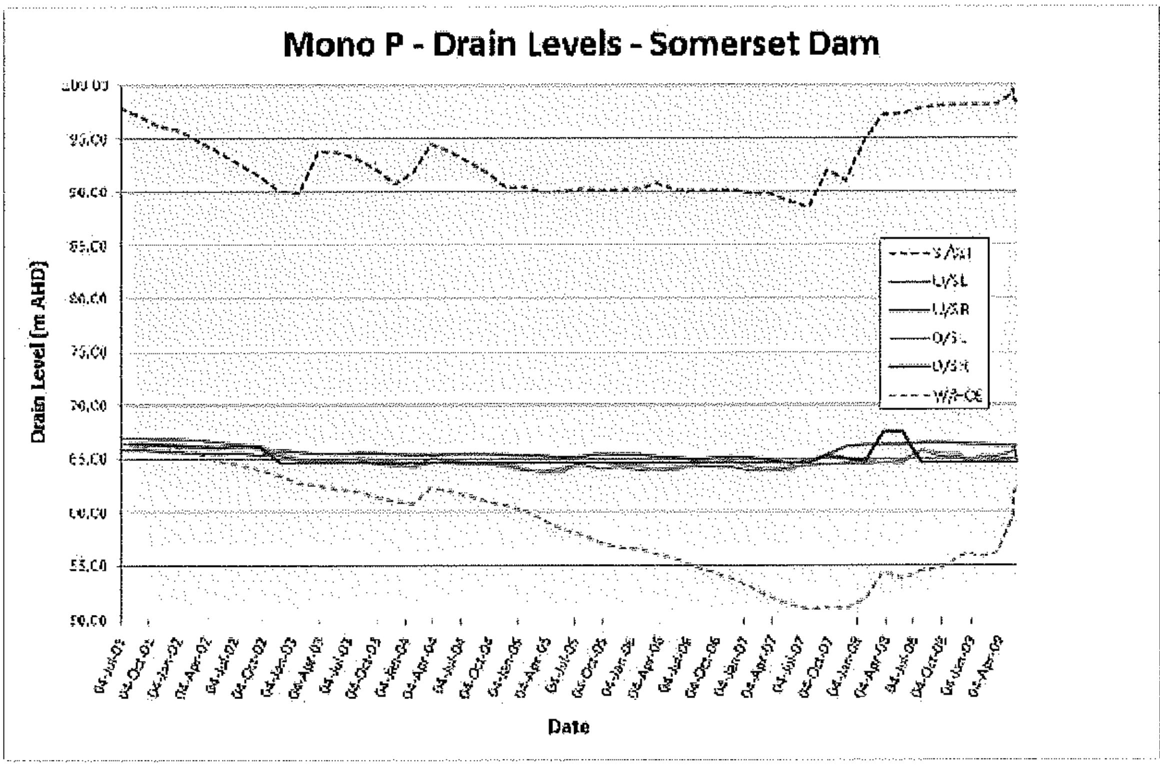

Mono K - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

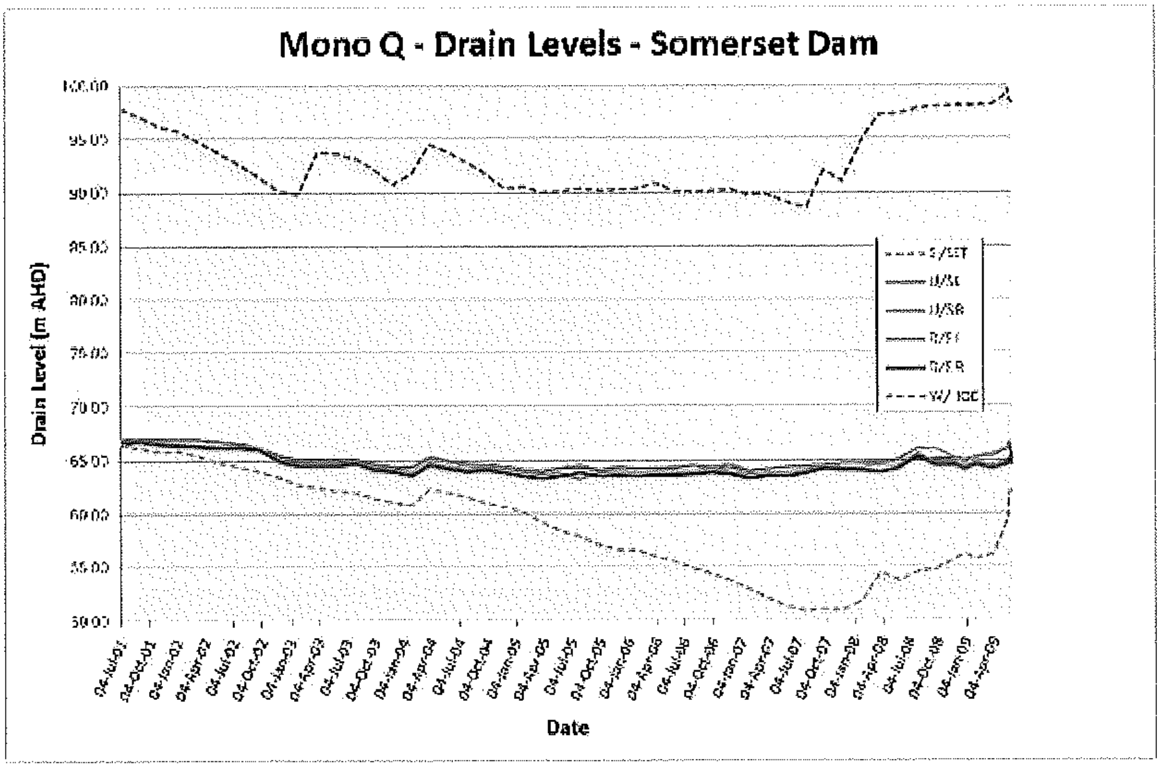

Mono L - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

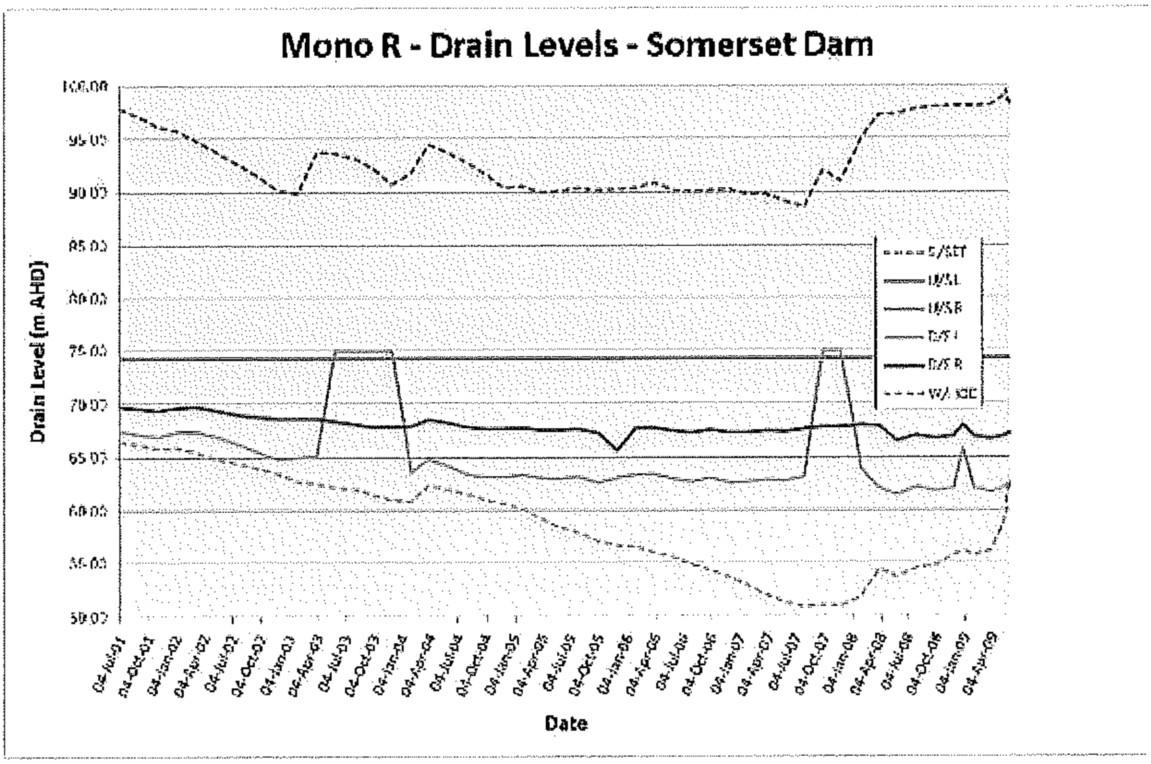

Mono M - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date
S/SET
U/S L
U/S R
D/S L
D/S R
W/HOE

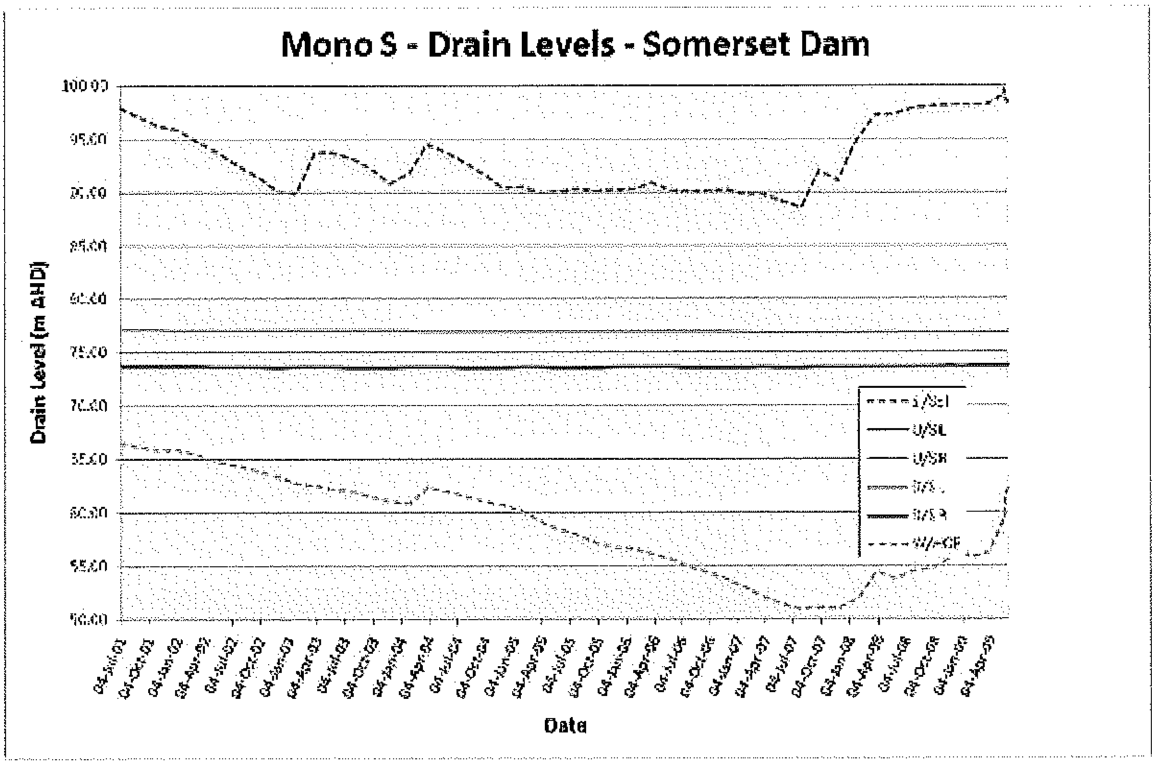

Mono N - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date
S/SET
U/S L
U/S R
D/S L
D/S R
W/HOE

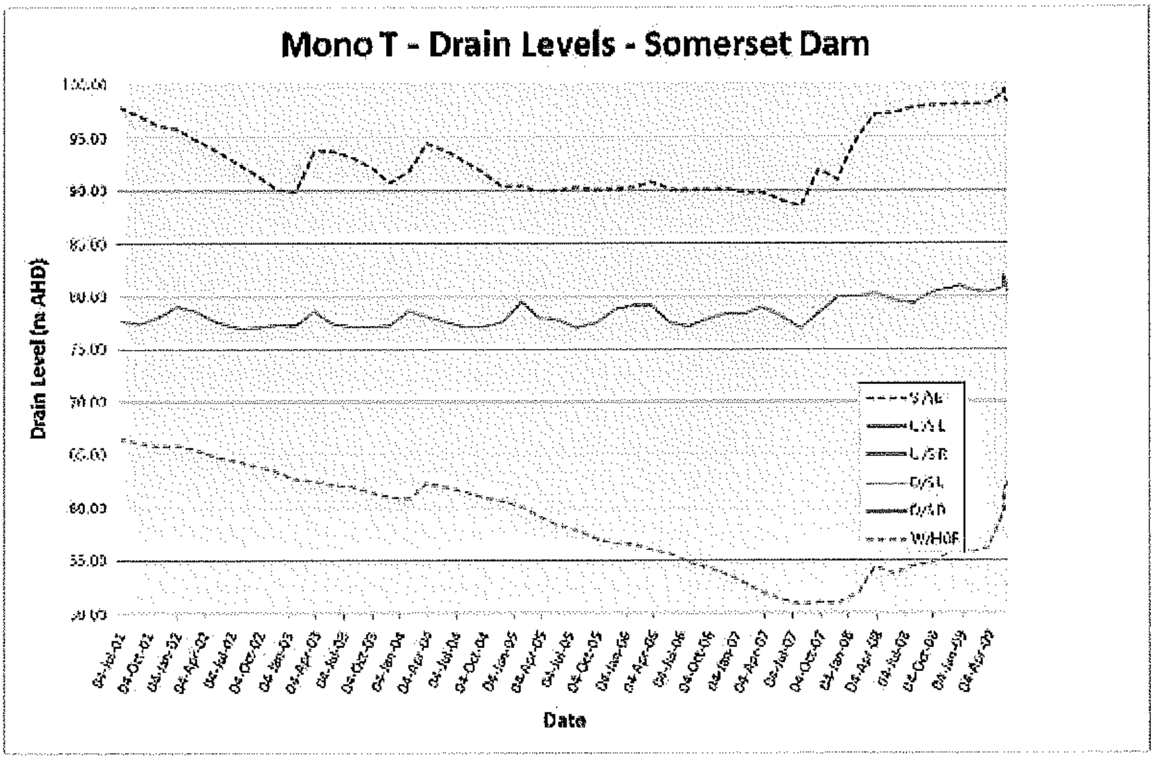

Mono O - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date


Mono P - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

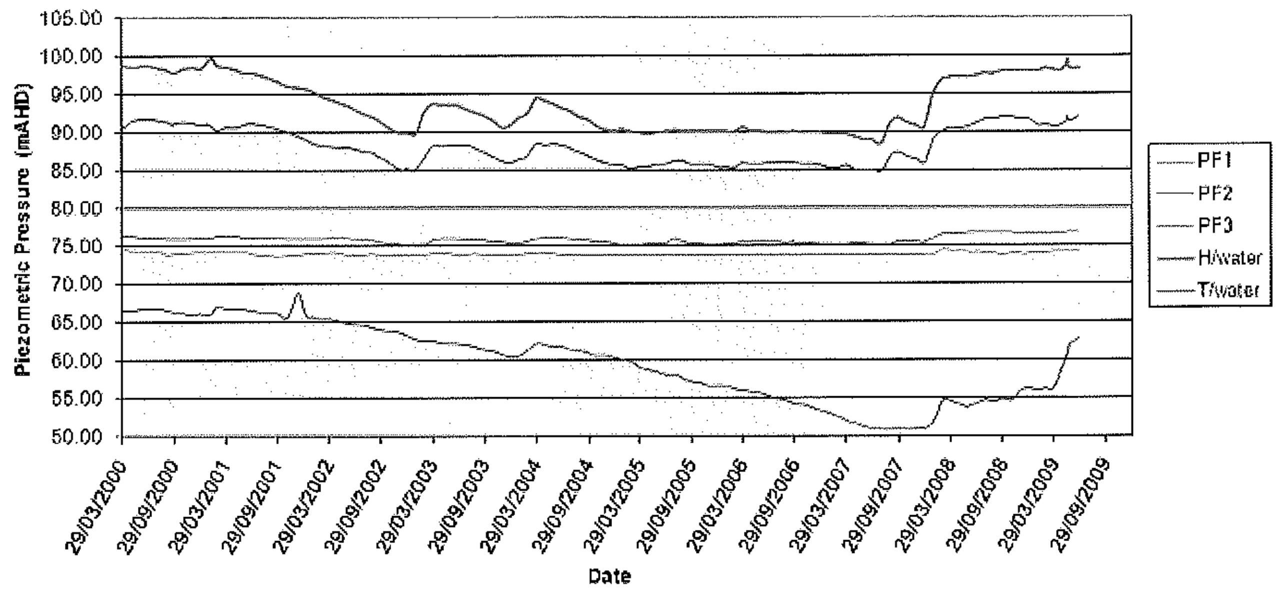

Mono Q - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date


Mono R - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date

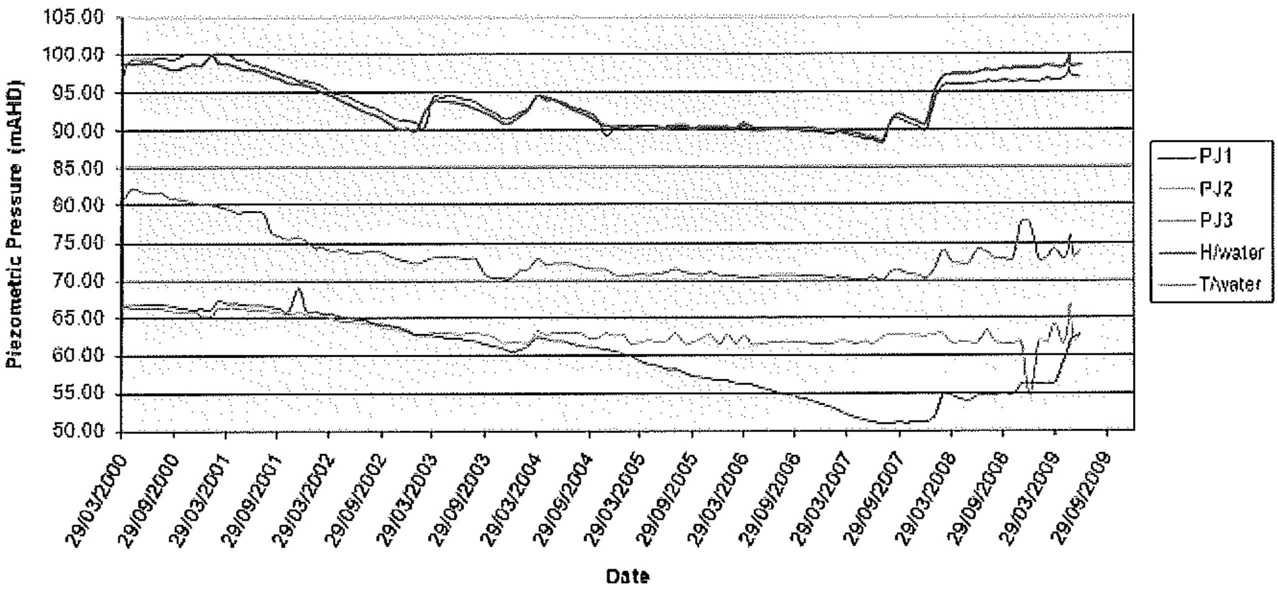

Mono S - Drain Levels - Somerset Dam
Drain Level (m AHD)
Date


Mono T - Drain Levels - Somerset Dam
95.00
90.00
75.00
55.00
Drain Level (m AHD)
W/HGF
Date

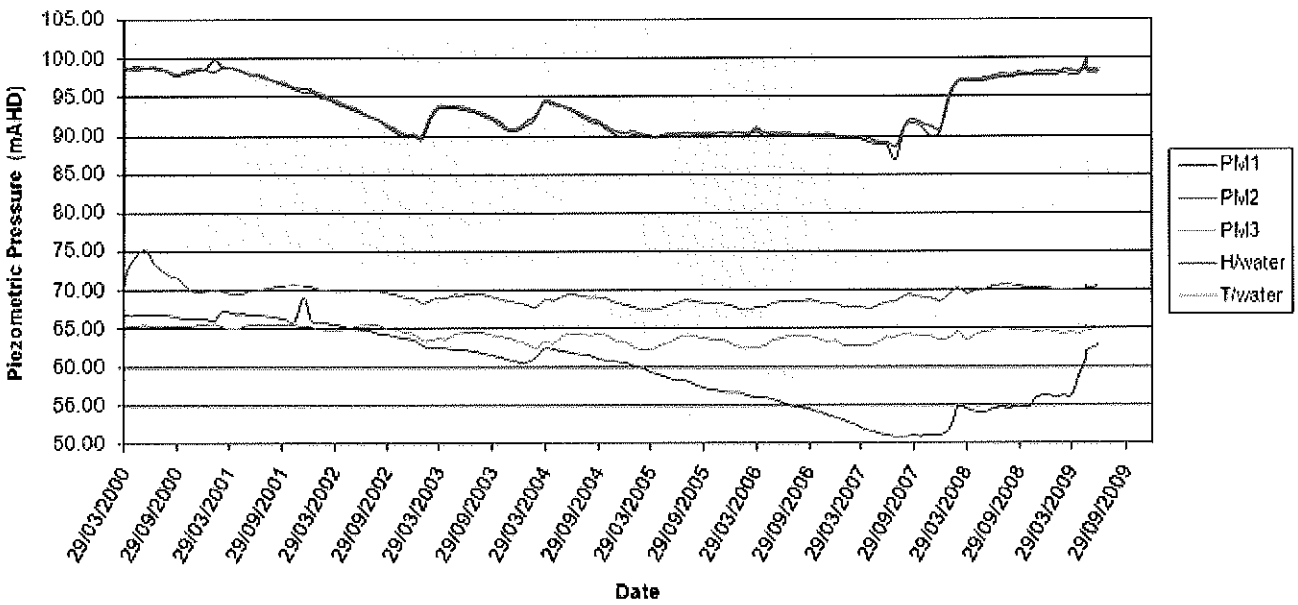
**Somerset Dam - Piezometers Monolith F**



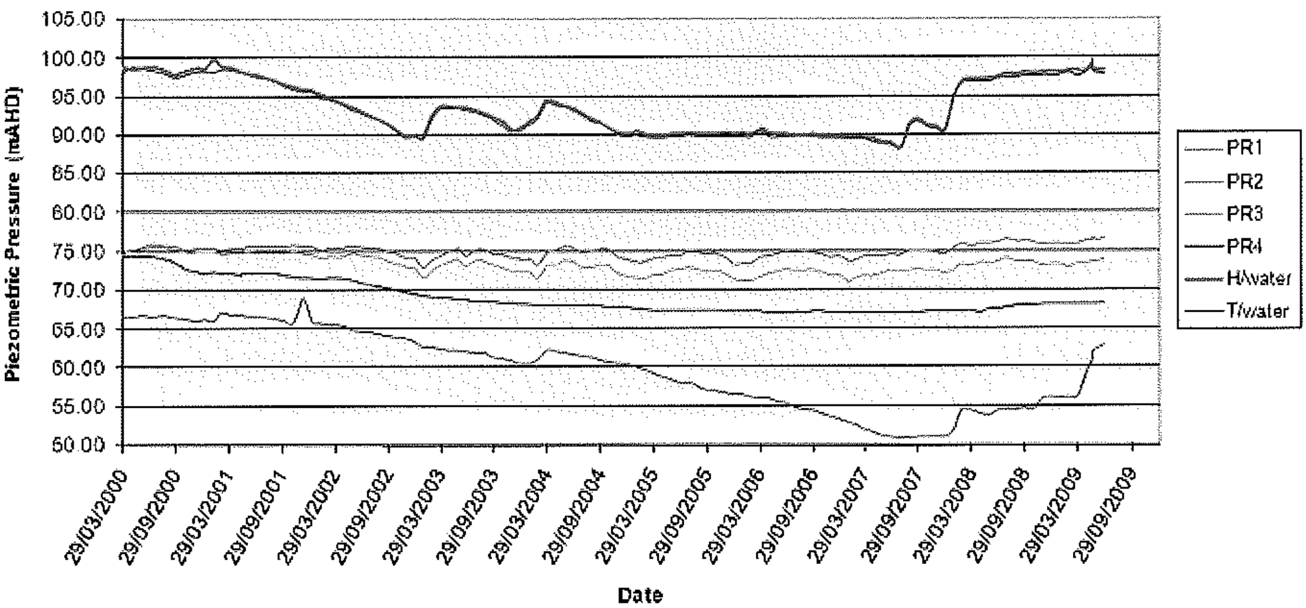

**Somerset Dam - Piezometers Monolith J**



**Somerset Dam - Piezometers Monolith M**




**Somerset Dam - Piezometers Monolith R**



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seqwater
WATER FOR LIFE

# SOMERSET DAM

## FIVE YEAR COMPREHENSIVE DAM SAFETY INSPECTION REPORT

September 2010

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# 1 INTRODUCTION

The Queensland Bulk Water Supply Authority trading as Seqwater took over ownership of Somerset Dam from the previous SeqWater on 1 July 2008. The Dam Safety Conditions issued by the Dam Safety Regulator for Somerset Dam in accordance with the Water Supply Act 2008 requires the following:

- The dam owner must carry out a comprehensive inspection of the dam in accordance with the Queensland Dam Safety Management Guidelines, on or before 1 October 2010.
- The comprehensive inspection must incorporate a review of the dam safety standards of the existing dam against current standards, a review of the adequacy of the dam safety documentation for the dam and reviews of the status on recommended actions from previous inspections.
- A Comprehensive Inspection Report detailing the findings of the comprehensive inspection in accordance with the Queensland Dam Safety Management Guidelines must be submitted to the Dam Safety Regulator, within three months after completion of the comprehensive inspection.

The Queensland Dam Safety Management Guidelines define the purpose of a Comprehensive inspection to be a periodic inspection of the dam and a review of the owner's whole dam safety management program. The Guidelines require the Inspection Report to assess all aspects of the dam safety management program and fully document:

- Deficiencies identified in the dam safety management program and its documentation
- A strategy for overcoming the deficiencies (including prioritisation of actions if several deficiencies are identified).

The Guidelines also require the inspection to be undertaken by an experienced dam's engineer who is a RPEQ. This inspection is to incorporate:

- A periodic inspection.
- An assessment of the appropriateness and adequacy, the effectiveness and application (including the owner's response to inspection report and Safety Review recommendations) of the dam safety management program and documentation for the dam.

This is considered to be the third Comprehensive Five Yearly Inspection of Somerset Dam undertaken in accordance with the Queensland Dam Safety Management Guidelines. The first formal dam safety review was undertaken by Gutteridge, Haskins & Davey Pty Ltd in 2000. Seqwater also has records of a 2006 inspection undertaken by NSW Department of Commerce. It is understood that no final report was issued in relation to this inspection; however Seqwater has a copy of an advanced draft of the final report.

A number of other recent investigations and reports have examined the design and structural performance of Somerset Dam. These include:

- Somerset Dam – Detailed Risk Assessment (SMEC) – 2005.
- Somerset Dam – Stability of Abutment Monoliths (NSW – Dept of Commerce) – 2005.
- Somerset Dam – Concept Review for Dam Raising (SMEC) – 2006.
- Somerset Dam – Crack Investigation (SMEC) – 2008.

These reports generally conclude that the structure of the dam is sound, however the reports raise several relatively minor issues requiring further investigation and these issues have been listed in the recommendations of this report for further action by Seqwater.

## 2 EXECUTIVE SUMMARY

Somerset Dam is generally in very good condition. The recent comprehensive design reviews undertaken by the NSW Department of Commerce and SMEC in 2005 and 2006 respectively conclude that the design of the dam is in accordance with modern day standards and that there are no significant outstanding design issues that require investigation at the present time.

Results from the dam safety instrumentation at the dam show that the structural performance of the dam in satisfactory and in accordance with design expectations. No issues were identified as a result of the comprehensive review of the instrumentation data.

The physical infrastructure at the dam is generally maintained in good condition, with all major dam components generally performing in a satisfactory manner and maintained in a satisfactory condition.

The main outstanding issues requiring resolution relate to optimising the conditioning monitoring program relating to the mechanical and electrical equipment at the dam and fixing an outstanding issue relating to the operation of the electric brakes on the sluice gates. Neither of these issues is considered high risk, but both have been outstanding since 2008.

# 3 DESCRIPTION

| seqwater WATER FOR LIFE | **SOMERSET DAM** |
|---|---|
| **Population at Risk** | Sunny Day Failure: 1,000<br>Flood: >1,000 (not fully assessed) |
| **Failure Impact Rating** | 2 |
| **Hazard Category** | Extreme |
| **Dam Owner** | Seqwater |
| **Name of Reservoir** | Lake Somerset |
| **Year Complete** | 1953 |
| **Location** | Stanley River near Kilcoy |
| **Water Course** | Stanley River |
| **Purpose** | Town water |
| **Type of Construction** | Mass concrete gravity dam |
| **Outlet Works** | 8 radial gates, 8 sluice gates and 4 cone dispersion valves. |
| **Catchment Area** | 1,340km$^2$ |
| **FSL** | 99m AHD |
| **Full Supply Capacity** | 379,849 ML |
| **Surface Area at FSL** | 4,210ha |
| **Main Dam Crest** | 107.46m AHD |
| **Bridge Deck Level** | EL 112.34 m |
| **Main Dam Embankment Length** | 305m |
| **Maximum Height of Main Dam Embankment** | 58m |
| **Width at Top of Main Dam Embankment** | 6.5m |
| **Spillway Crest** | 100.45m AHD |
| **Spillway Length** | 63.4m |
| **Gates** | 8 gates wide 7.9 m x 7.0 m high |
| **Saddle Dam Crest** | Not Applicable |
| **Saddle Dam Length** | Not Applicable |

| | |
|---|---|
| **Maximum Height of Saddle Dam Embankment** | Not Applicable |
| **Peak Water Level as a Result of PMF** | EL 112.0m |
| **Spillway Capacity (including sluice gates)** | 4,700$m^3$/s (EL 107.5m) |
| **Maximum Discharge as a Result of PMF** | 9,600 $m^3$/s |
| **AEP of Spillway Capacity (including sluice gates)** | 1 in 10,000 (EL 107.5) |
| **Regulator valves** | 4 x 3m cone dispersion valves |
| **Mean annual pan evaporation** | 1,600mm (estimated from BOM maps) |
| **Mean annual rainfall** | 986mm at 040189 Somerset Dam |
| **Hydroelectric Facilities** | 4 mw generator |
| **Notable events** | 1974 |
| **Maximum Historic Storage Level** | 106.26 m AHD Jan 1974 |

# 4 DAM HISTORY

Somerset dam was constructed in the period 1935 to 1953 with a delay in construction as a result of the war in the period 1942 to 1948. The dam has been constructed across the Stanley River between Little Mount Brisbane and Mount Somerset. It is located at AMTD 4.8 on the Stanley River which is a tributary of the Brisbane River and is located in the upstream limit of the Wivenhoe storage which was constructed in the period 1976 to 1985.

Somerset dam is a concrete gravity structure with a gate controlled flood storage compartment. The dam has a downstream energy dissipater with baffle and side training walls of mass concrete. The distance from the downstream face of the spillway to the baffle is 36.6 metres and the overall length of the stilling basin is 58.2 metres. The concrete in the floor of the stilling basin is about 3 metres thick for the first 23.8 metres downstream of the spillway and reduces to a 1.5 metres thickness at the baffle and 0.76 metres at the end sill. The dam and the stilling basin are provided with a number of drains to limit the build up of pore pressures.

A hydro-electric power station is provided on the right side of the spillway which at the time of the inspection was being refurbished. Four regulator cone valve 2.3 metres in diameter are the primary method of making releases from the storage. During the 1974 flood these regulator valves were submerged by tail water and it is understood that prior to 1996, the valves may have been operated for extended periods in a partially of fully submerged state. The resulting cavitation may have damaged the valves and extensive repairs are evident within the internal structures of these valves.

Eight sluice gates and eight radial gates have been provided for normal flood releases. The Full Supply Level of Somerset dam is 1.5 metres below the concrete spillway crest level controlled by the radial gates. A reinforced concrete deck is located 4.88 metres above the non-overflow crest level of the dam. This deck carries the winches and gantry crane required to operate the sluices, crest gates and coaster gate.

It is understood that a roller chain from a sluice gate came apart during an operation of a gate in either the 1980s or the early 1990s. No official records relating to the incident can be found, but since that time the circlips holding individual rollers in place on the roller chains on each sluice gate have been welded closed to prevent a another incident of this nature.

An inspection of the dam was undertaken in the period 6 to 7 March 1995. The subsequent report recommended the installation of piezometers, seepage monitoring and crack width monitoring facilities. This installation work was undertaken in the late 1990s (see Section 11).

There is considerable horizontal cracking exposed in the upper gallery walls. The main cracks are located on the downstream side of the gallery wall, one about 0.4 metres above floor level and the other 1.6 metres to 1.8 metres above floor level. The latter crack extends for most of the length of the gallery and appears to be at the same level as a construction joint in the downstream face of the dam. This crack has been investigated in a series of reports, the most recent being in 2008 by SMEC in a report entitled "Somerset Dam – Crack Investigation". Generally, the reports conclude that the cracking in the Upper gallery is not of structural concern, but that the current monitoring program should continue so that a further investigation trigger can be initiated should the crack begin to significantly change in nature over time (See Section 11).

# 5 DATA BOOK

In accordance with the Dam Safety Conditions for Somerset Dam, the dam owner must update and maintain a Data Book in accordance with the Queensland Dam Safety Management Guidelines. The Data Book must include all pertinent records and history relating to the dam and encompass the documentation of investigation, design, construction, operation, maintenance, surveillance, monitoring measurements and any remedial action taken relating to the dam.

The Dam Data Book is held in Seqwater's Karalee Office and is generally comprehensive and contains information in accordance with the Data Book Checklist in the Queensland Dam Safety Management Guidelines. Seqwater has also taken electronic copies of the Data Book information and saved this information within a system that includes a similar provision for saving and back-up.

No recommendations for updating the Somerset Dam Data Book are considered necessary at this time.

# 6 STANDING OPERATING PROCEDURES

In accordance with the Dam Safety Conditions for Somerset Dam, the dam owner must develop Standing Operating Procedures in accordance with the Queensland Dam Safety Management Guidelines. The purpose of the Procedures is to:

- Define responsibilities for actions critical to the safety of the dam.
- Identify procedures for particular daily activities, which ensure that these activities are done safely, in the same way each time and in accordance with development permit conditions.
- Ensure appropriate people are notified when unforseen or unusual events occur.

Seqwater submitted copies of the Standard Operating Procedures for Somerset Dam to the Dam Safety Regulator in March 2009 following an extensive update and review of the Procedures. Controlled copies of the document have also been issued to the following Seqwater staff:

- Principal Engineer Dam Safety.
- Operations Coordinator responsible for Somerset Dam
- Somerset Dam Storage Supervisor.

The dam is generally operated in accordance with the Standing Operating Procedures and the Procedures have been prepared in accordance with the Queensland Dam Safety Management Guidelines. The Procedures address the following areas:

**DAM EMERGENCIES**

Section 1 Dam Safety Organisational Structure and Responsibilities

Section 2 Emergency Action Planning

Section 3 Loss of Communication during an Emergency Event

Section 4 Dam Security and Restricted Areas

**DAM SAFETY SURVEILLANCE**

Section 5 Dam Attendance

Section 6 Dam Operating Log

Section 7 Dam Surveillance and Routine Inspection

| | |
|---|---|
| Section 8 | Dam Instrumentation Data Collection and Management |
| Section 9 | Annual Dam Inspections |
| Section 10 | Comprehensive Five Yearly Dam Safety Inspections |
| Section 11 | Unscheduled Dam Inspections |

**DAM OPERATIONS AND MAINTENANCE**

| | |
|---|---|
| Section 12 | Routine Dam Maintenance |
| Section 13 | Routine Dam Operations |
| Section 14 | Storage Inflow Control |
| Section 15 | Renewal and Refurbishment of Dam Infrastructure |
| Section 16 | Regulated Water Releases |
| Section 17 | Uncontrolled Water Releases |

**DAM SAFETY ADMINISTRATION AND REGULATORY REQUIREMENTS**

| | |
|---|---|
| Section 18 | Reporting |
| Section 19 | Dam Safety Documentation |
| Section 20 | Regulatory Requirements and Dam Safety Conditions |
| Section 21 | Training |

Some internal restructuring of Seqwater occurred in 2010 and this has resulted in a change to some Position Titles referred to in the Standing Operating Procedures. Accordingly it is recommended that the Standing Operating Procedures be amended to account for these changes. No other recommendations for updating the Somerset Dam Standing Operating Procedures are considered necessary at this time.

**RECOMMENDATION**

**Review and update the Standing Operating Procedures to account for the change in position titles due to the recent restructure of Seqwater.**

# 7 OPERATION AND MAINTENANCE MANUAL

In accordance with the Dam Safety Conditions for Somerset Dam, the dam owner must develop Operation and Maintenance Manuals in accordance with the Queensland Dam Safety Management Guidelines. The purpose of the Manuals is to provide instruction on how to operate, maintain and overhaul individual pieces of equipment for a dam and its associated structures. The manuals should contain the following:

- Work Instructions, which detail the way in which equipment should be operated and outline the steps involved in performing a task.
- Maintenance Schedules, which detail the asset, description of task, frequency of maintenance and special requirements for servicing and maintaining the equipment.
- Equipment data sheets or Manufacturer's Manuals, which comprise technical information needed for maintenance, repair and overhaul of equipment.

Seqwater has updated and reviewed the Operation and Maintenance Manual for Somerset Dam. Controlled copies of the document have also been issued to the following Seqwater staff:

- Principal Engineer Dam Safety.
- Operations Coordinator responsible for Somerset Dam
- Somerset Dam Storage Supervisor.

The dam is generally maintained and operated in accordance with the Operation and Maintenance Manual and the Manual has been prepared in accordance with the Queensland Dam Safety Management Guidelines. No recommendations for updating the Somerset Dam Operation and Maintenance Manuals are considered necessary at this time.

# 8 EMERGENCY ACTION PLAN

In accordance with the Dam Safety Conditions for Somerset Dam, the dam owner must develop an Emergency Action Plan in accordance with the Queensland Dam Safety Management Guidelines. The purpose of the Plan is to:

- Identify emergency conditions that could endanger the integrity of the dam and that require immediate action;
- Prescribe procedures that should be followed by the dam owner and operating personnel in the event of an emergency;
- Provide procedures to allow timely warning to Emergency Response Agencies for their implementation of protection measures to downstream communities.

Seqwater submitted copies of the Emergency Action Plan for Somerset Dam to the Dam Safety Regulator in November 2010 following an extensive update and review of the previous Plan. Controlled copies of the document have also been issued to the following Seqwater staff and external agencies:

- Principal Engineer Dam Safety.
- Dam Operations Manager
- Somerset Dam Storage Supervisor.
- Operations Coordinator responsible for Somerset Dam
- Seqwater/SunWater Flood Operations Centre
- Department of Emergency Services
- Brisbane City Council
- Emergency Management Queensland

Relevant dam personnel are generally familiar with the Emergency Action Plan and the Plan has been prepared in accordance with the Queensland Dam Safety Management Guidelines. The Plan contains the following information:

- Register of notification.
- Emergency contacts.
- Emergency action triggers.
- Routes to the dam site.
- Flood maps.

Emergency communication from the dam itself is reasonable. No recommendations for updating the Somerset Dam Emergency Action Plan are considered necessary at this time.

# 9 ROUTINE INSPECTIONS

The Hazard Category of Somerset Dam is Extreme. Routine Inspections are undertaken daily in accordance with ANCOLD Guidelines. Rainfall, storage level and seepage are also measured during these inspections. A typical inspection sheet is shown below.

| | |
|---|---|
| **DAM SURVEILLANCE – SOMERSET DAM** | **Week Commencing 30 October 2010** |
| **Daily** | **FSL EL 99.00** |

| **READINGS** | **Sat** | **Sun** | **Mon** | **Tue** | **Wed** | **Thu** | **Fri** |
|---|---|---|---|---|---|---|---|
| Date | 30/10/10 | 31/10/10 | 01/11/10 | 02/11/10 | 03/11/10 | 0411/10 | 05/11/10 |
| Time | 0700 | 0700 | 0700 | 0700 | 0700 | 0700 | 0700 |
| Rainfall (mm) | | | | | | 2 | 1.6 |
| Storage Level (m AHD) | 99.25 | 99.29 | 99.27 | 99.27 | 99.29 | 99.23 | 99.20 |
| Temperature (°C) | 29 | 30 | 27 | 27 | 27 | 23 | 25 |
| | | | | | | | |
| **CONDITION REPORT** | | | | | | | |
| **Main Embankment** | | | | | | | |
| Concrete Condition | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| Erosion | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | | | | | | | |
| **Instrumentation** | | | | | | | |
| Vibrating Wire Piezometers | | | | | | | |
| Uplift Pressure Gauges | | | | | | | |
| Crack Measurement Points | | | | | | | |
| | | | | | | | |
| **Spillway (Crest EL 100.45)** | | | | | | | |
| Concrete Condition | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| 8 x Sluice Gates Condition | | | | | | | |
| Erosion | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ | ✓ |
| | | | | | | | |
| **River and Ponded Area Banks (Slumping and or Erosion)** | | | | | | | |
| Downstream River Condition | | | | | ✓ | | |
| Ponded Area & Confluences | | | | | ✓ | | |
| Photo's taken | | | | | No | | |
| | | | | | | | |
| Reporting Officer's Initials | GF | GF | GF | GF | GF | GF | GF |

(✓) Indicates Satisfactory Condition
(X) Indicates Action Required (Provide details in Comments Section below)

COMMENTS
...3-11-2010Open No 2 regulator 100% during working hours Release 1680ML

...4-11-2010Open No 2 regulator 100% during working hours Release 1785 ML

Completed form to be emailed or fixed to Dam Operations Coordinator Central Region (Jayam Tennakoon)

Jayam Tennakoon – Fax 5426 1097 Email; jtennakoon@seqwater.com.au

Report Submitted by Graham Francis..............Signature ........................Date 05/11/2010.......................

Seqwater commenced the current system of undertaking routine inspections in June 2009. Prior to that date, although routine inspections were being undertaken, the records were maintained on a weekly rather than on a daily basis. Seqwater now has a robust system in place for ensuring that Routine Inspections are undertaken daily (see SOP 4.3 – Dam Surveillance and Routine Inspection) in accordance with ANCOLD guidelines.

# 10 ANNUAL INSPECTIONS

The most recent Annual Inspection at the dam was undertaken in November 2009 by John Tibaldi (RPEQ 02525). The significant recommendations arising from the inspection and their current status are shown in the following table:

| RECOMMENDATION | STATUS |
|---|---|
| • ***Vegetation growing on the embankment is to be removed and/or sprayed with a suitable herbicide. This includes the trees growing within five metres of the abutments.*** | Complete |
| • ***Vegetation growing on the spillway is to be removed and/or sprayed with a suitable herbicide.*** | Complete |
| • ***The rotten wooden buffers associated with the radial gate counter weights are to be replaced.*** | Complete |
| • ***Recommence the electrical condition monitoring program associated with the radial gates, gantry crane and standby diesel generator.*** | Not Complete<br>(see Section 12) |
| • ***The electric brakes on the sluice gates are to be repaired in accordance with the recommendations made during the last round of condition monitoring.*** | Not Complete<br>(see Section 12) |
| • ***An underwater inspection of the coaster gate guides is to be undertaken as soon as possible.*** | This work has commenced but has been delayed by the rising water levels in the dam. Initial investigations uncovered no major issues and the work has been rescheduled for 2011.<br>(see Section 12) |
| • ***The works recommendations arising from the August 2008 crane inspection are to be completed as soon as possible.*** | Complete |
| • ***All measuring points are to be suitably labelled and numbered on site and a suitable engineering plan prepared to show instrumentation point locations and corresponding numbering.*** | Complete |
| • ***Re-examine the dam safety issues associated with the concrete cracking in the upper gallery.*** | Complete |

The most recent Comprehensive Inspection at the dam was undertaken in July 2006 by the NSW Department of Commerce. There are no recommendations from this report that are not accounted for in the above table or elsewhere in this report.

In summary, there are no outstanding items critical to the safety of the dam and all outstanding work has been addressed in the recommendations of this report.

# 11 INSTRUMENTATION

A summary of the dam safety monitoring instrumentation at Somerset Dam is contained in the following table.

| SOMERSET DAM – DAM SAFETY INSTRUMENTATION | |
|---|---|
| **TYPE** | **DESCRIPTION** |
| Pore Pressure<br><br>(Vibrating Wire Piezometers) | There are 13 hydraulic piezometers installed at the dam to monitor pore pressure. The piezometers are located in the dam foundations under monoliths F, J, M and R. Readings are taken from the lower gallery. Three piezometers are installed in each of monoliths F, J and M, with four in Monolith R. The upstream piezometer in each monolith is connected to the storage. These instruments are read monthly. |
| Pore Pressure<br><br>(Pressure relief drains) | As shown in the plan in Appendix A labelled Figure 6, the system for to relieve pressure in the dam foundations consists of:<br><br>• Foundation drain holes within monoliths E to U originating in the lower gallery. There are generally 4 drains per monolith, two upstream drains and two downstream drains, that are drilled approximately 9 metres into the rock foundations. Water levels in these drain holes are read monthly. Water in the lower gallery drains to the dam sump pumps.<br><br>• Two square brick foundation drains of dimension 0.38 metres x 0.38 metres located on the foundation and one approximately 7.5 metres downstream of upstream heal and the other the approximately 27 metres downstream of upstream heal. These drains are connected by drain pipes to the lower gallery. Water levels in these drains are not currently measured.<br><br>• A foundation tunnel of dimension 0.76 metres wide by |

| | |
|---|---|
| | 1.83 metres high located approximately 15 metres downstream of upstream heal. This tunnel drains directly to the dam sump pumps.<br>• A foundation tunnel of dimension 0.76 metres wide by 0.91 metres high located along the downstream toe. This tunnel drains directly to the dam sump pumps.<br>There are flow metres on the dam sump pumps to record water pumped. This information is not currently monitored over time. |
| Deformation Monitoring | There are 22 movement monitoring points installed at the dam to measure embankment movement. A deformation survey is undertaken annually. |
| Structural Movement | There are 22 measurement points to monitor the longitudinal crack along the upper gallery. These instruments are read monthly. |
| Seepage | All gallery seepage is directed to the dam sump pumps. This includes water flowing from the pressure relief drains. There are flow metres on the dam sump pumps to record water pumped. This information is not currently monitored over time, but overall it appears that seepage is minimal. |
| Rainfall and Storage Level | Alert canisters transmit this data continuously in real time. All data is stored by Seqwater within a database. |

**Pore Pressure (Vibrating Wire Piezometers)**

A summary of the piezometer results is shown in the table below. Detailed graphs showing the behaviour of individual piezometers over time are contained in Appendix B. No plans showing the locations of the piezometers are currently available. Based on the stability assessment in Appendix 3.8 of the Somerset dam Risk Assessment undertaken by SMEC in 2005, under normal load conditions, crack length should not exceed 10 metres and uplift pressure downstream of the crack should not exceed 78 metres. Therefore a trigger level of 77.5 metres has been adopted for the downstream piezometers. If such a level is reached, further structural analysis should be undertaken, although it should be noted that this trigger level certainly exceeds the uplift assumed in the 2005 analysis undertaken by the NSW Department of Commerce. Based of observed piezometer readings that were available when this report was being compiled, the 2005 analysis assumptions are considered questionable.

| PIEZOMETERS | DESCRIPTION | COMMENTS | TRIGGER |
|---|---|---|---|
| PF01<br>PF02<br>PF03 | These three piezometers are located within the foundations under Monolith F. PF01 is the most upstream piezometer and PF03 is the most downstream piezometer. | PF01 is connected to the storage. PF02 and PF03 have shown very little movement since installation. | Maximum uplift recorded at PF03 is 74.66; latest reading on a full storage is 74.32. A drop in pressure is evident between PF01 and PF03. There are no current issues of concern. |
| PJ01<br>PJ02<br>PJ03 | These three piezometers are located within the foundations under Monolith J. PJ01 is the most upstream piezometer and PJ03 is the most downstream piezometer. | PJ01 is connected to the storage. PJ02 and PJ03 also react to movements in storage level, but a generally at relatively low levels. | Maximum uplift recorded at PJ03 is 66.56; latest reading on a full storage is 66.34. A drop in pressure is evident between PJ01 and PJ03. There are no current issues of concern. |
| PM01<br>PM02<br>PM03 | These three piezometers are located within the foundations under Monolith M. PM01 is the most upstream piezometer and PM03 is the most downstream piezometer. | PM01 is connected to the storage. PM02 and PM03 have shown very little movement since installation. | Maximum uplift recorded at PM03 is 65.50; latest reading on a full storage is 65.32. A drop in pressure is evident between PM01 and PM03. There are no current issues of concern. |
| PR01<br>PR02<br>PR03<br>PR04 | These four piezometers are located within the foundations under Monolith R. PR01 is the most upstream piezometer and PR04 is the most downstream piezometer. | PR01 is connected to the storage. PR02 and PR03 have shown very little movement since installation. PR04 seems to be connected to tail water levels. | Maximum uplift recorded at PR03 is 75.64; latest reading on a full storage is 73.96. A drop in pressure is evident between PR01 and PR03. There are no current issues of concern. |

The instruments are read and serviced in accordance with ANCOLD Guidelines.

**Pore Pressure (Pressure Relief Drains)**

Detailed graphs showing the behaviour of individual drains over time are contained in Appendix B. Following a detailed site inspection, it appears that some of the drains that are currently measured relate to the drain connections to the brick foundation drains. Accordingly a detailed study is required to define the function of each measured relief drainage point and the appropriate monitoring arrangements for each point. Generally though, the graphs contained in Appendix B show that the drains are not blocked and are generally functioning as intended. Many off the drains also appear to be flowing slowly into the lower gallery, which also provides a good indication of their functionality.

**Deformation Monitoring**

Deformation surveys are undertaken annually at Somerset Dam. The last survey was undertaken in June 2010 and the detailed results are contained in Appendix B. Following a review of the movements, no values could be found that are cause for any concern.

Maximum settlement since the base survey has been 1 millimetre with a maximum rise of 5 millimetres at points 14 and16. It is worth noting however that for points 14 and 16, the amount of movement has decreased over the last period. Maximum horizontal movement has been 9 to 10 mm since the base survey with Point 14 showing the maximum variability in all three directions.

The only drawing showing the location of the deformation survey marks is A3-213650. It is recommended that a new drawing be prepared to clearly show the survey marks on a general arrangement to allow movements to be related to the structural monoliths.

**Structural Movement (Crack Measurement)**

A summary of the crack measurement results is shown in the table below. The crack has been the subject of many studies and investigations in recent years with the general conclusion being that the crack is not presently an issue of structural concern, but that monitoring of the crack should continue.

Generally, the crack continues to open at a rate of around 0.064 mm/year. Detailed graphs showing the behaviour of individual instruments over time are contained in Appendix B. The most recent detailed investigation of the crack was undertaken by SMEC in 2008 and the issue will be examined again in detail as part of the next Comprehensive Inspection in 2015.

| CRACK MEASUREMENT POINT | ESTIMATED TOTAL ADDITIONAL OPENING OVER THE LAST ELEVEN YEARS |
|---|---|
| 1 | 0.6 millimetres |
| 2 | 0.7 millimetres |
| 3 | 0.7 millimetres |
| 4 | 0.8 millimetres |
| 5 | 0.7 millimetres |
| 6 | 1.1 millimetres |
| 7 | 0.8 millimetres |
| 8 | 0.8 millimetres |
| 9 | 0.7 millimetres |
| 10 | 0.6 millimetres |
| 11 | 0.6 millimetres |
| 12 | 0.7 millimetres |
| 13 | 0.8 millimetres |
| 14 | 0.7 millimetres |
| 15 | 0.8 millimetres |
| 16 | 0.6 millimetres |
| 17 | 0.8 millimetres |
| 18 | 0.7 millimetres |
| 19 | 0.5 millimetres |
| 20 | 0.5 millimetres |
| 21 | 0.7 millimetres |
| 22 | 0.6 millimetres |

### Seepage

All gallery seepage is directed to the dam sump pumps. This includes water flowing from the pressure relief drains. There are flow metres on the dam sump pumps to record water pumped. This information is not currently monitored over time, but overall it appears that seepage is minimal. Regardless, it is recommended that graphing of sump pump flows over time commence.

### Monitoring Frequencies

Monitoring frequencies are undertaken in accordance with the following table that meets ANCOLD Guidelines.

| SOMERSET DAM – DAM SAFETY INSTRUMENTATION | |
|---|---|
| **TYPE** | **MONITORING FREQUENCY** |
| Rainfall and Storage Level | Monitored daily through ALERT |
| Seepage | Continuous |
| Pore Pressure | Monthly |
| Structural Movement | Monthly |
| Deformation Monitoring | Yearly |

The Hazard Category of Somerset Dam is Extreme. Instrumentation monitoring is undertaken in accordance with ANCOLD Guidelines.

### Recommendations

- **A new drawing is to be prepared to clearly show the survey marks on a general arrangement to allow movements to be related to the structural monoliths.**
- **The depth and water level in each foundation drain measuring point in the lower gallery is to be accurately determined. Following this an engineering study is to**

**determine the appropriate monitoring frequency and analysis method for gathered data associated with these drains.**

- **Graphing of sump pump flows over time is to commence.**

# 12 INSPECTION

## 12.1 Inspection Team

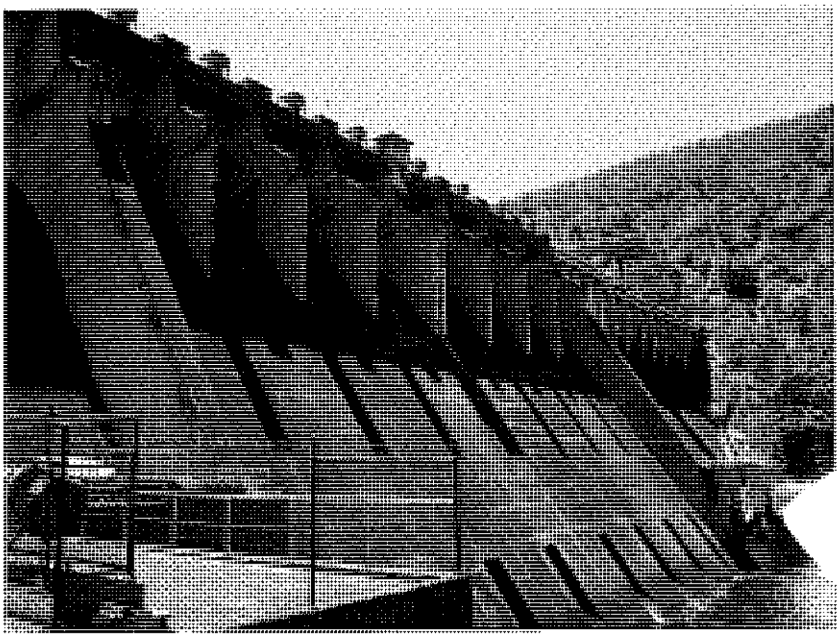
John Tibaldi, Principal Engineer Dam Safety (Seqwater)
Louw Van Blerk, Engineer Dam Safety (Seqwater)

## 12.2 Operational status at time of inspection

**Date of Inspection:** 16 September and 25 November 2010.
**Reservoir Water Surface Elevation:** near FSL

## 12.3 Dam Embankment

Somerset Dam is a 47 metre high concrete gravity dam on the Stanley River upstream of Wivenhoe Dam. The dam is of conventional mass concrete construction. There are seven mass concrete abutment units on each side of the central spillway structure that supports a road bridge at EL 112.3. The abutment units are constructed with an open overflow section below the bridge at EL 107.5. Flood flows passing through these openings flow down the back face of the dam and impact on an unprotected rock foundation rock, before flowing laterally towards the central spillway channel.



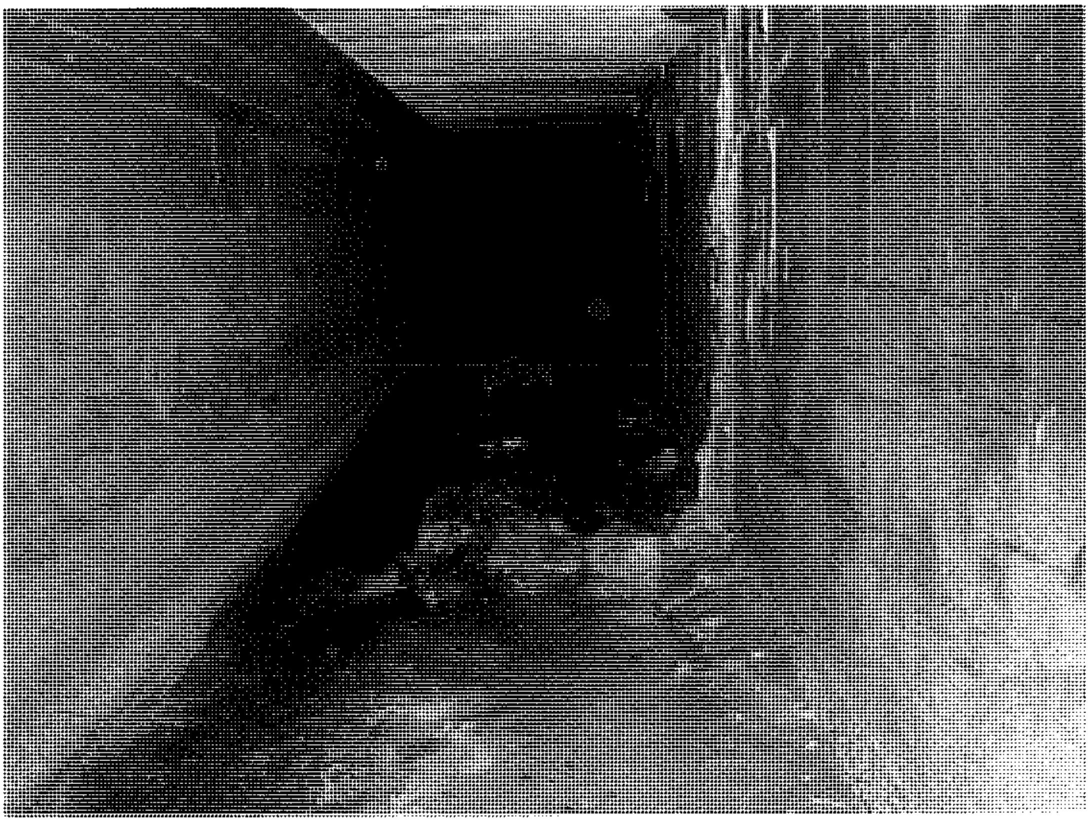
**Somerset Dam**

The concrete embankment was inspected and was found generally to be in good condition. Some vegetation was observed growing on the embankment that should be removed.

There are a number of galleries within the dam. Concrete cracking has occurred at the Upper Gallery and there is considerable horizontal cracking exposed in the gallery walls. There is no indication that the cracking is significantly worsening over time (see also Section 11).

**Inspection Recommendations:**

- **Vegetation growing on the embankment is to be removed and/or sprayed with a suitable herbicide.**

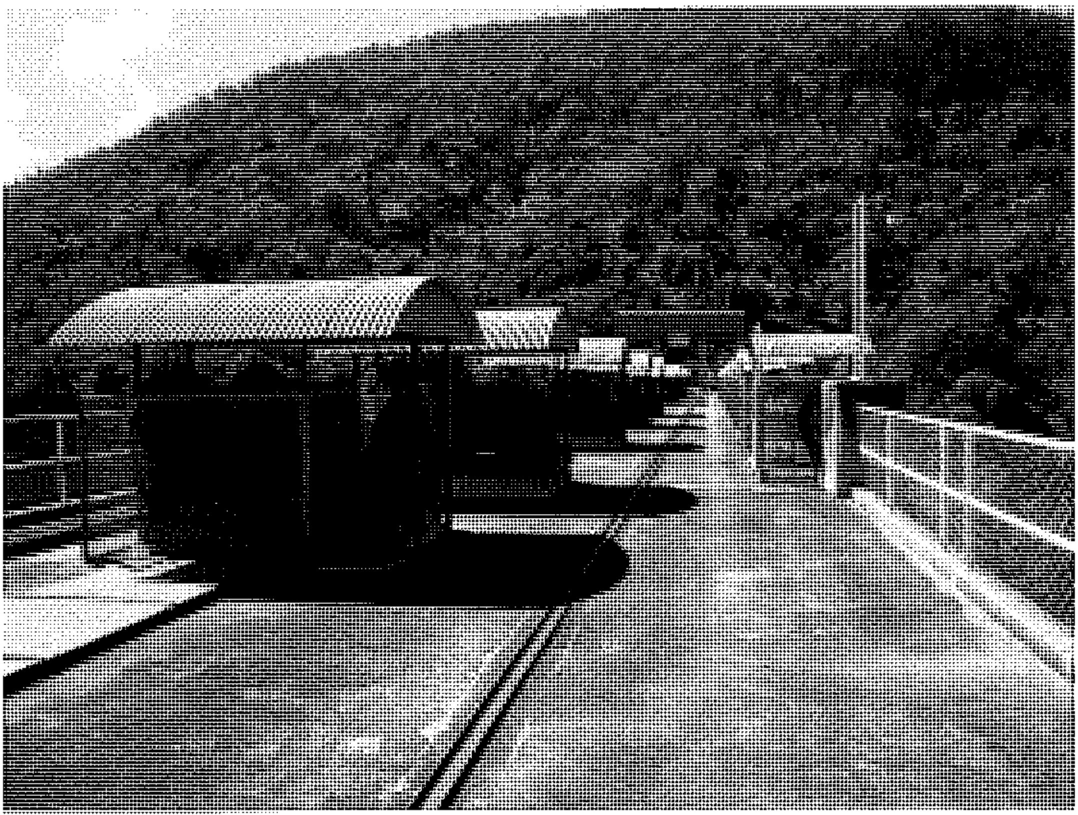


**Concrete in Drainage Gallery**

### 12.4 Spillway

The dam has eight radial gates (sector gates) installed on the top of the spillway. The eight radial gates are each 7 metres high by 8 metres long and are installed above full supply level. The gates are counterbalanced so that the hoist does not have to lift the full weight of the gate.

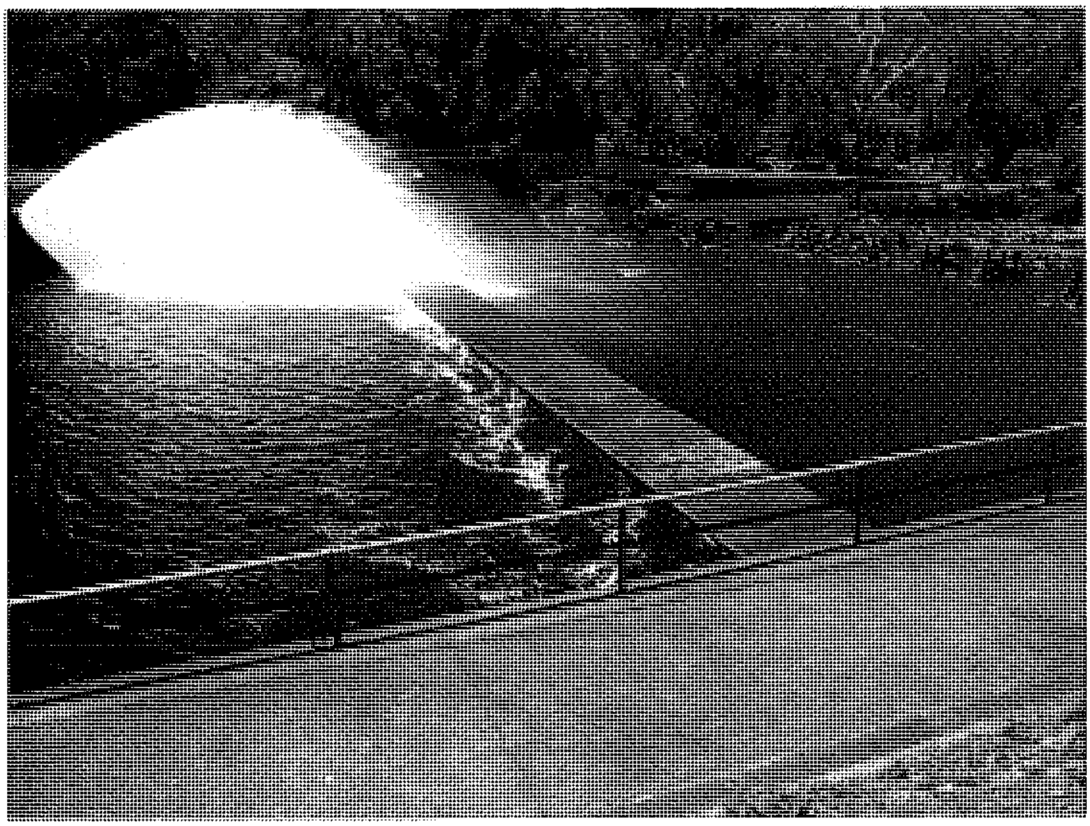
The radial gate winch units comprise a six-pole electric motor close-coupled to a worm reduction gear set. The output of the worm reduction passes through three sets of spur gears, the last spur gear being bolted to the rope drum. The rope is attached directly to the centre of the gate without any intermediate pulleys, while the counterweight is attached to both ends of the gate. An electric brake operates on the motor-coupling drum. A parking brake is operated by a hand-wheel applying a band brake to a drum mounted on the last spur gear drive shaft.



**Sluice Gates hoisting gear**

The spillway and associated gates and associated hoisting gear looked to be in good condition. All equipment was operated under full flow conditions in the recent flood event in

October and no operational issues with the equipment were identified. Undertaking regular routine maintenance in accordance with the dam Operation and Maintenance Manuals appears to be producing good results and it is important that this program is continued. It was noted that recent structural reports calculate that the dam is over stressed if the radial gates are closed when the dam lake level exceeds 105.7 metres AHD and this issue should be noted in the flood operations procedures.



**Dissipater**

Recent comprehensive inspections have raised concerns with the ability of the dissipater to withstand high outflow conditions and although no damage is apparent, an underwater inspection should be undertaken to confirm that the dissipater is structurally sound.

**Inspection Recommendation:**

- **Vegetation growing on the spillway is to be removed and/or sprayed with a suitable herbicide.**
- **Undertake underwater inspection of the dissipater area.**

- **Insert appropriate arrangements into the dam Flood Operations Procedures to account for the dam becoming over stressed if the radial gates are closed when dam water levels exceed 105.7 metres AHD.**

## 12.5 Reservoir Rim and Downstream Waterway

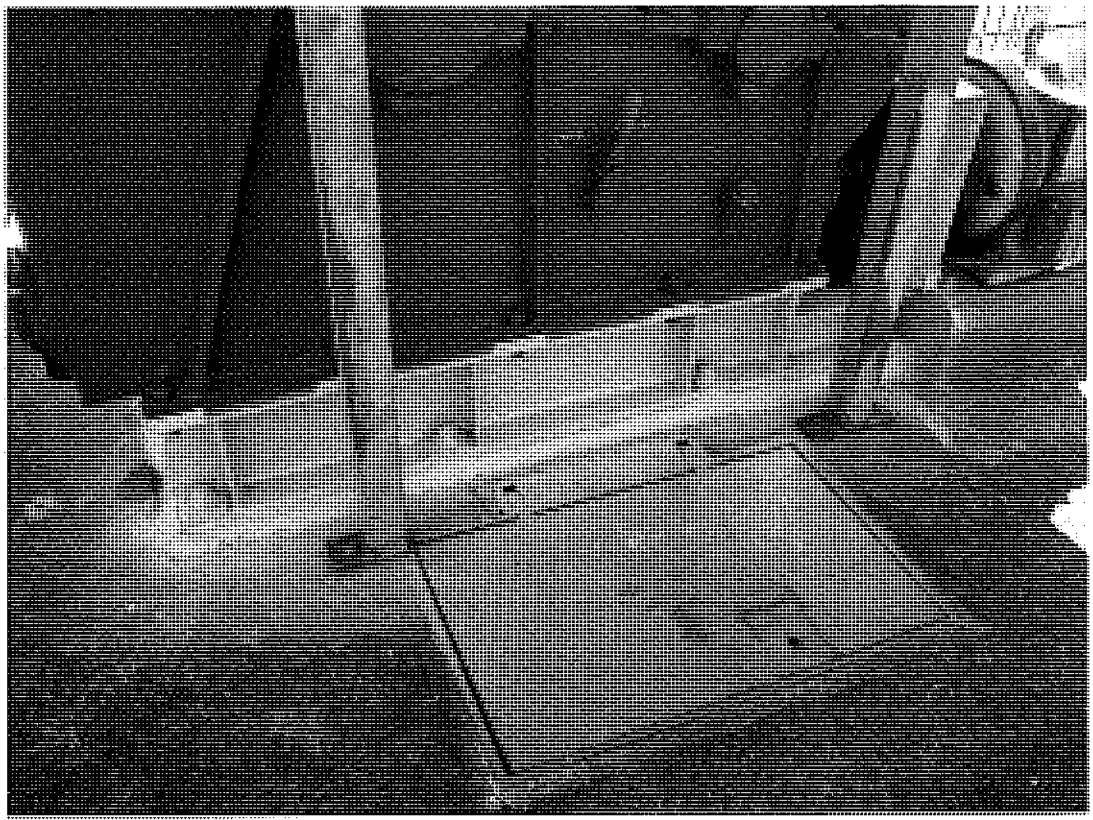
The reservoir rim slopes appear generally stable and above the Full Supply Level are relatively well vegetated with no signs of slips or movement that would be of concern from a dam safety perspective.

There were also no slips or restrictions that would prevent spillway outflow or raise tail water levels to an unacceptable level during a dam outflow event.

## 12.6 Outlet Works

The outlet works consist of thirteen conduits or sluice-ways through the bottom of the dam wall. One of the conduits supplies a mini-hydro power station, four are connect to fixed cone dispersion valves and the eight sluice-ways constitute the main outlet regulating capacity.

The eight main sluice gates are each 3.7 metres high by 2.4 metres wide. The gates are not counterbalanced, and are hoisted by two ropes, each rope being reeved into a four-part system. The conduits connected to the mini-hydro and the fixed cone dispersion valves are protected by similar roller gates with hoists essentially identical to the main sluice gate hoists, the differences relating to the rope drums.



**Sluice Hoisting Gear**

Each winch unit comprises a six-pole electric motor close-coupled to a worm reduction gear set. The output of the worm reduction passes through two sets of spur gears, the last spur gear being bolted to the rope drum. The rope drum is a double drum with two ropes attached. Each rope is reeved through pulleys to create a four-fall rope system connected to an equalising beam on the top of the gate. An electric brake operates on the motor to worm pinion coupling. A band brake is hand-wheel applied to a drum bolted to the rope drum for added security.

A 100 tonne travelling gantry crane on the dam deck serves to handle the emergency coaster gate used for maintenance of the sluice gates. This crane appeared to be in good condition, however it was noted that one recommendation from the 2008 mechanical inspection of the crane remain outstanding. This recommendation relates to corrosion repair and although relatively minor in nature, it should be attended to.

The other mechanical equipment associated with the outlet works was inspected and found to be in generally good condition. All sluice gates were operated under full flow conditions in the recent October flood event and no issues of concern were detected. The Coaster Gate has recently been inserted against all sluice tunnels successfully. The planned maintenance program for refurbishing the sluices, tunnels, regulators and regulator conduits is producing good results and it is important that this program is continued with a further tunnel and associated sluice scheduled for refurbishment in the current financial year.

Some concerns remain from the 2008 inspection that the electrical condition monitoring program seems to have been discontinued and recommendations from the 2008 condition monitoring round associated with repairing the electric brakes on the sluice hoisting gear has not been attended to. Internal inspection of the conduits and valves has occurred in 2010, with no significant issues of concern detected.

The only concern associated with the outlet works is the condition of the guides associated with the placement of the coaster gate. These guides were last inspected over 5 years ago and a diving inspection should be programmed as soon as a drop in dam water level occurs to assess the condition of this infrastructure.

**Inspection Recommendations:**

- **Recommence the electrical condition monitoring program associated with the sluice gates, radial gates, regulators, gantry crane and standby diesel generator.**
- **The electric brakes on the sluice gates are to be repaired in accordance with the recommendations made during the last round of condition monitoring.**
- **An underwater inspection of the coaster gate guides is to be undertaken as soon as possible.**

- **The works recommendations arising from the August 2008 crane inspection are to be completed as soon as possible.**
- **All wire ropes are to be tested against design specifications during the 2011/12 financial year. This work is in accordance with the inspection and testing schedule that was established in 1996 and has been in place since that time.**
- **Continue the rolling program of sluice gate, sluice tunnel and trash rack painting.**

## 12.7 Instrumentation

Surveillance instrumentation at the dam monitors movement of the dam embankment, seepage and pressure within the embankment. The instrumentation consists of:

- 22 crack measurement points.
- 13 vibrating wire piezometers.
- 56 pressure relief wells.
- 1 automatic water level recorder

Graphs of the piezometer and uplift pressure data are shown in Appendix A. This data was examined and no trends of concern were identified (see Section11).

The instrumentation was inspected, and the following works recommendations were made:

**Inspection Recommendations:**

- **The sump pump operating level is to be lowered to allow the foundation tunnel to be drained at all times. A manual switch is to be added to facilitate sump pump testing and drainage.**
- **All measuring points are to be suitably labelled and numbered on site and a suitable engineering plan prepared to show instrumentation point locations and corresponding numbering.**
- **Plastic covers or similar are to replace the existing screwed metal claps on the foundation drainage points to facilitate access, monitoring and maintenance.**
- **The wooden covers on the pressure relief points in the floor of the lower foundation gallery that are jammed in place are to be removed and replaced with covers that are easily removable and will be displaced by any drain uplift flow..**
- **The foundation drainage points in the floor of the lower gallery that are causing tripping hazards are to be suitably modified.**

# 13 RECOMMENDATIONS

| Recommendation | Rating (See Below) |
|---|---|
| • Review and update the Standing Operating Procedures to account for the change in position titles due to the recent restructure of Seqwater. | 3 |
| • A new drawing is to be prepared to clearly show the survey marks on a general arrangement to allow movements to be related to the structural monoliths. | 3 |
| • The depth and water level in each foundation drain measuring point in the lower gallery is to be accurately determined. Following this an engineering study is to determine the appropriate monitoring frequency and analysis method for gathered data associated with these drains. | 3 |
| • Graphing of sump pump flows over time is to commence. | 2 |
| • Vegetation growing on the embankment is to be removed and/or sprayed with a suitable herbicide. | 3 |
| • Vegetation growing on the spillway is to be removed and/or sprayed with a suitable herbicide. | 3 |
| • Undertake underwater inspection of the dissipater area. | 3 |
| • Insert appropriate arrangements into the dam Flood Operations Procedures to account for the dam becoming over stressed if the radial gates are closed when dam water levels exceed 105.7 metres AHD. | 2 |
| • Recommence the electrical condition monitoring program associated with the sluice gates, radial gates, regulators, gantry crane and standby diesel generator. | 2 |
| • The electric brakes on the sluice gates are to be repaired in accordance with the recommendations made during the last round of condition monitoring. | 2 |
| • An underwater inspection of the coaster gate guides is to be undertaken as soon as possible. | 3 |
| • The works recommendations arising from the August 2008 crane inspection are to be completed as soon as possible. | 3 |

| | |
|---|---|
| • **All wire ropes are to be tested against design specifications during the 2011/12 financial year. This work is in accordance with the inspection and testing schedule that was established in 1996 and has been in place since that time.** | **3** |
| • **Continue the rolling program of sluice gate, sluice tunnel and trash rack painting.** | **4** |
| • **The sump pump operating level is to be lowered to allow the foundation tunnel to be drained at all times. A manual switch is to be added to facilitate sump pump testing and drainage.** | **2** |
| • **All measuring points are to be suitably labelled and numbered on site and a suitable engineering plan prepared to show instrumentation point locations and corresponding numbering.** | **3** |
| • **Plastic covers or similar are to replace the existing screwed metal claps on the foundation drainage points to facilitate access, monitoring and maintenance.** | **3** |
| • **The wooden covers on the pressure relief points in the floor of the lower foundation gallery that are jammed in place are to be removed and replaced with covers that are easily removable and will be displaced by any drain uplift flow.** | **3** |
| • **The foundation drainage points in the floor of the lower gallery that are causing tripping hazards are to be suitably modified.** | **3** |

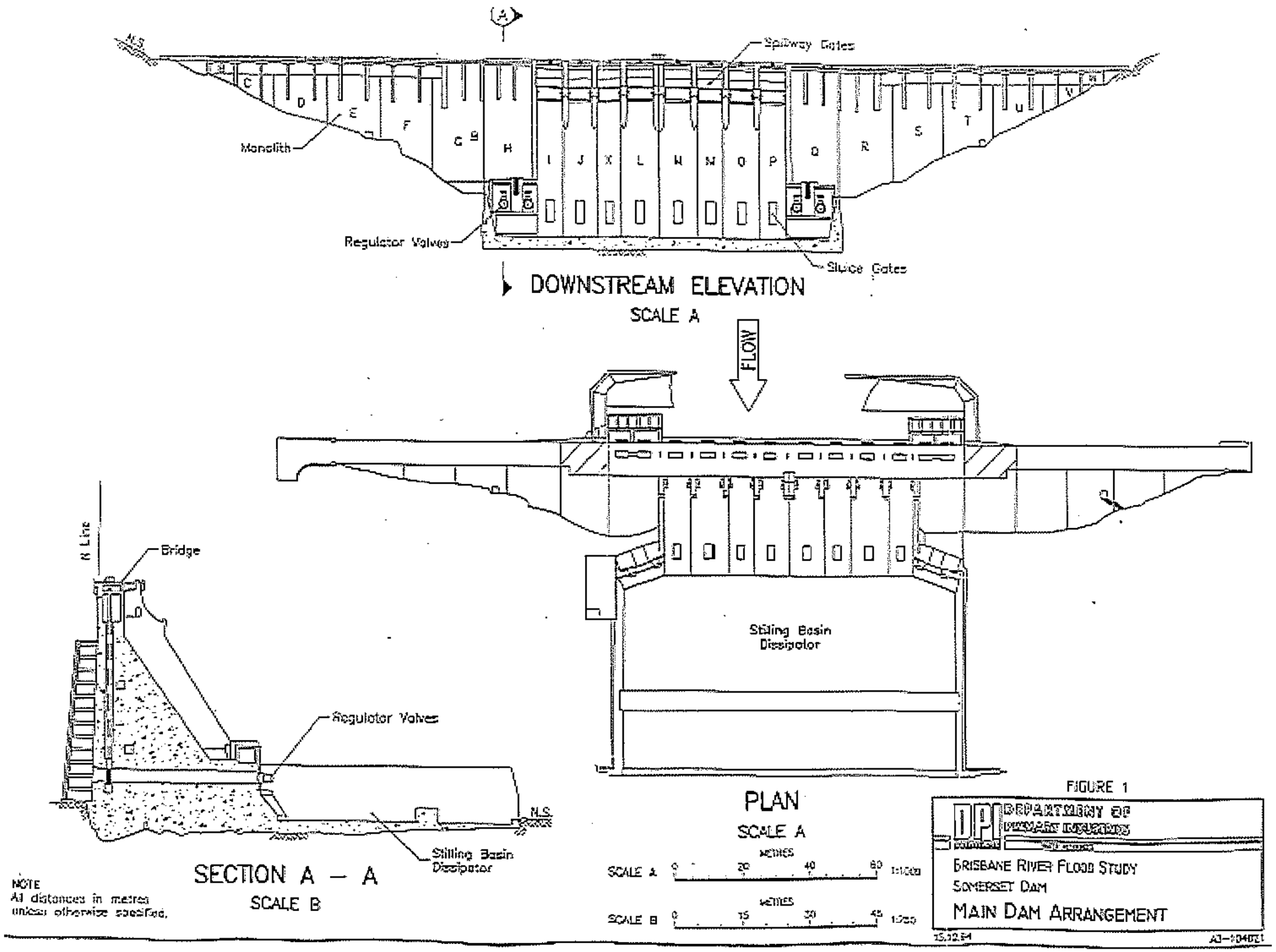
**Legend of Criticality Rating**

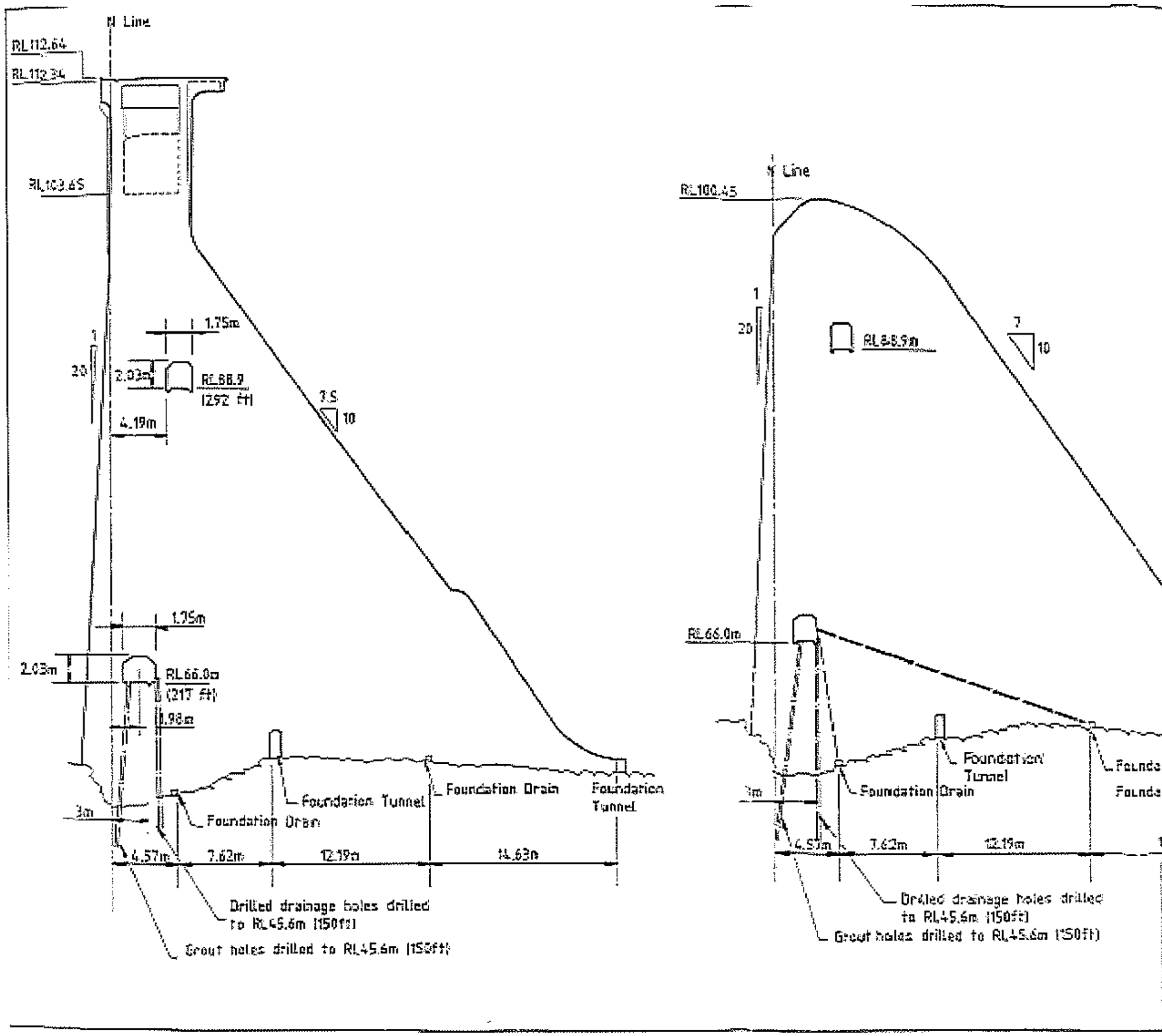
| | |
|---|---|
| Rating 1 | Rectification required immediately, i.e. within 1 month |
| Rating 2 | Rectification required within 3 months |
| Rating 3 | Rectification required within 12 months |
| Rating 4 | Ongoing |

# 14 REFERENCES


- (DPI Water Commercial 1995) "Somerset Dam Design Report", September 1995.
- (GHD, 1997) "Somerset Dam Safety Review" , April 1997.
- (SunWater) "Detailed Stability Check of the Spillway Wall Monoliths at Somerset Dam", June 2001.
- (SunWater 2001) "Assessment of Piping Potential in the Somerset Dam Saddle Dams", November 2001.
- (Somerset Alliance 2005) "Somerset Upgrade Design and Construction Report", December 2005.
- (Glen Hobbs and Associates 2006) "Somerset Dam Radial gate Trunnion Bearings Review", May 2006.
- (NSW Department of Commerce 2006) "Somerset Dam Comprehensive Inspection Draft Report", July 2006.
- (Seqwater 2009) "Somerset Dam – Emergency Action Plan, September 2009.
- (Seqwater 2009) "Somerset Dam – Standing Operating Procedures, March 2009.
- (Seqwater 2009) "Somerset Dam – Operation and Maintenance Manuals, March 2009.
- (GHD 2009) "Report for Raising Somerset Dam Full Supply Level Review", December 2009.

# APPENDIX A DRAWINGS


(A)
N.S.
Spillway Gates
Monolith
Regulator Valves
Sluice Gates
C
D
E
F
G
H
I
J
X
L
M
N
O
P
Q
R
S
T
U
V
DOWNSTREAM ELEVATION
SCALE A
FLOW
Stilling Basin Dissipator
PLAN
SCALE A
N Line
Bridge
Regulator Valves
Stilling Basin Dissipator
N.S.
SECTION A – A
SCALE B
NOTE
All distances in metres unless otherwise specified.
SCALE A 0 20 40 60 1:1000
SCALE B 0 15 30 45 1:750
FIGURE 1
BRISBANE RIVER FLOOD STUDY
SOMERSET DAM
MAIN DAM ARRANGEMENT



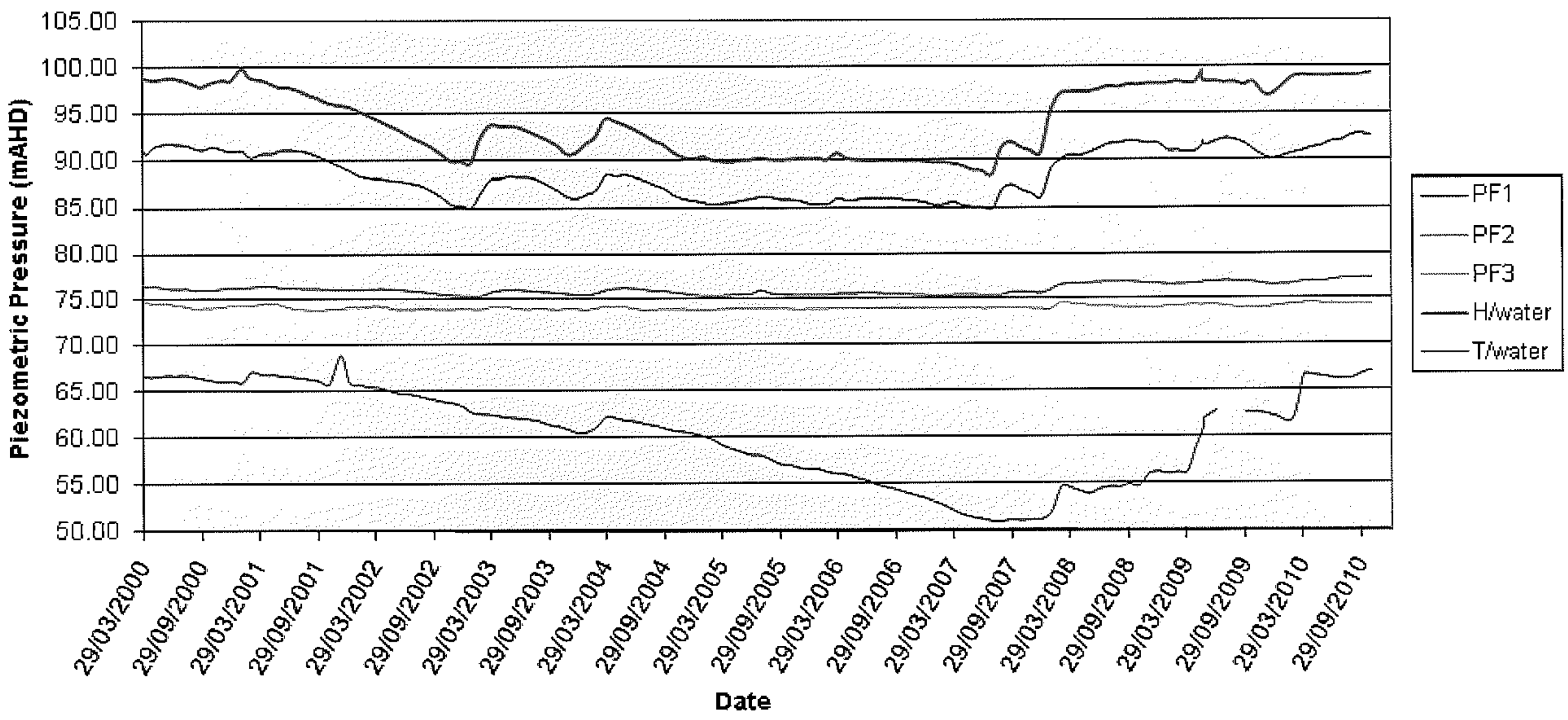

SOUTH EAST QUEENSLAND WATER BOARD

SOMERSET DAM
TYPICAL DRAINAGE AND
GROUT PROVISIONS

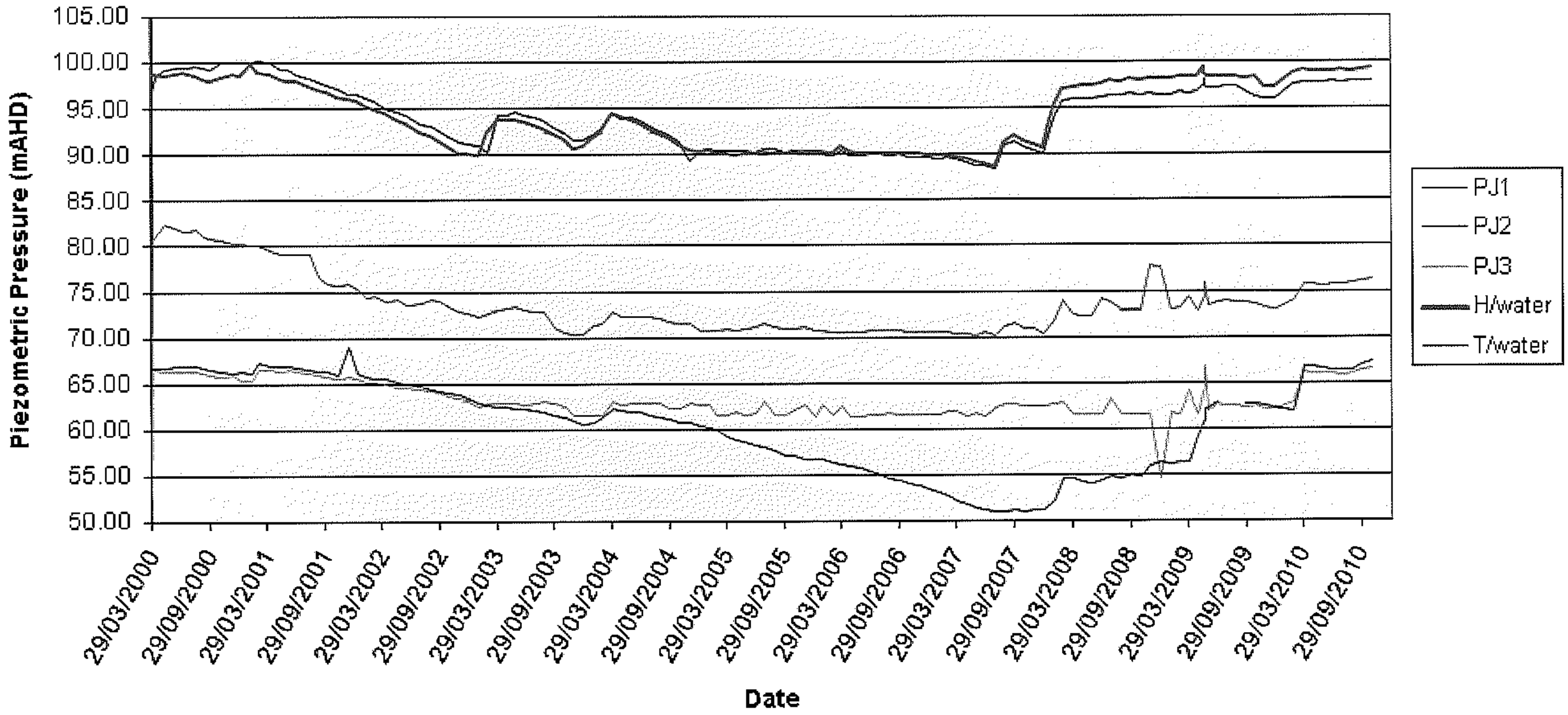
FIGURE 5

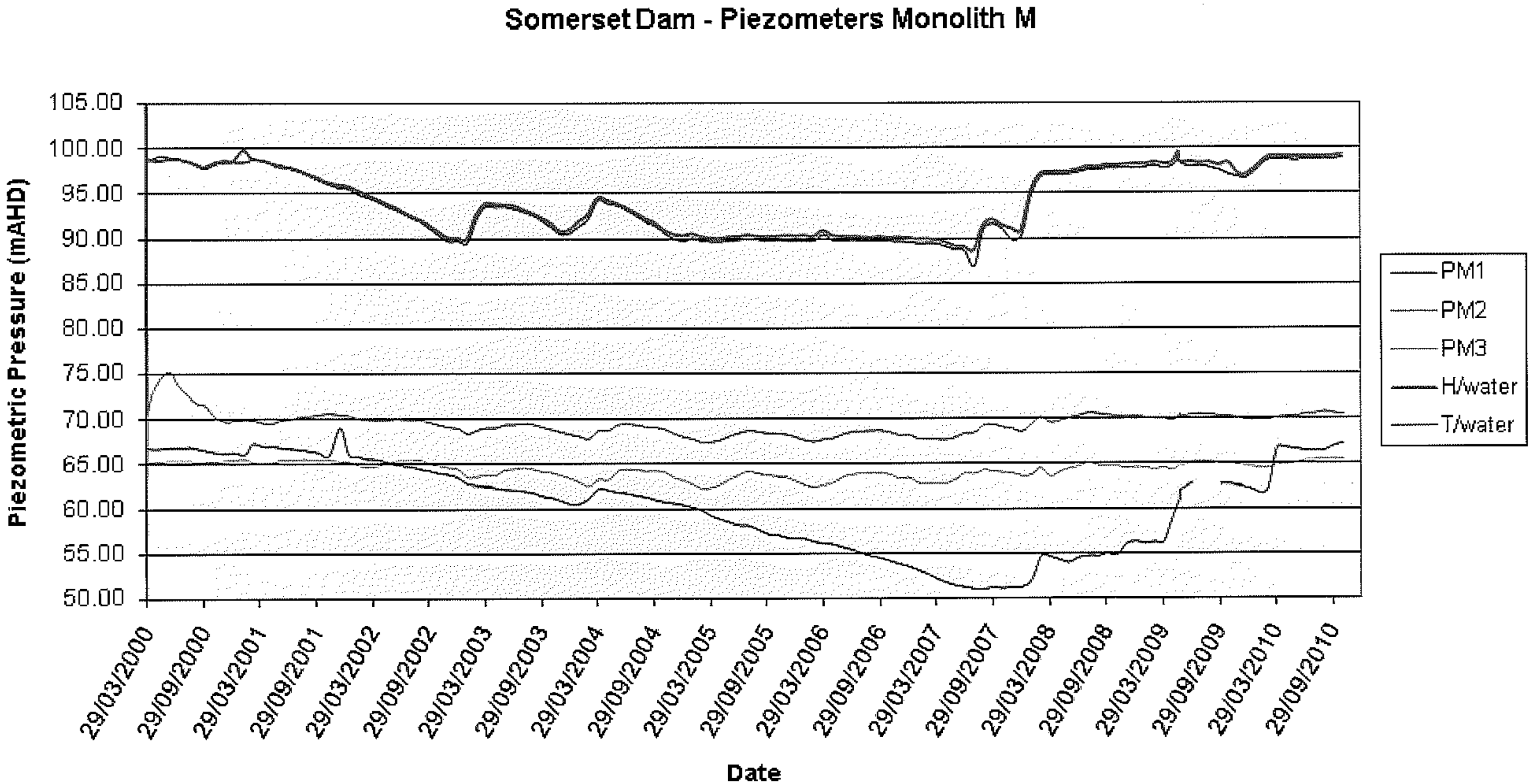
# APPENDIX B INSTRUMENTATION DATA

Somerset Dam - Piezometers Monolith F



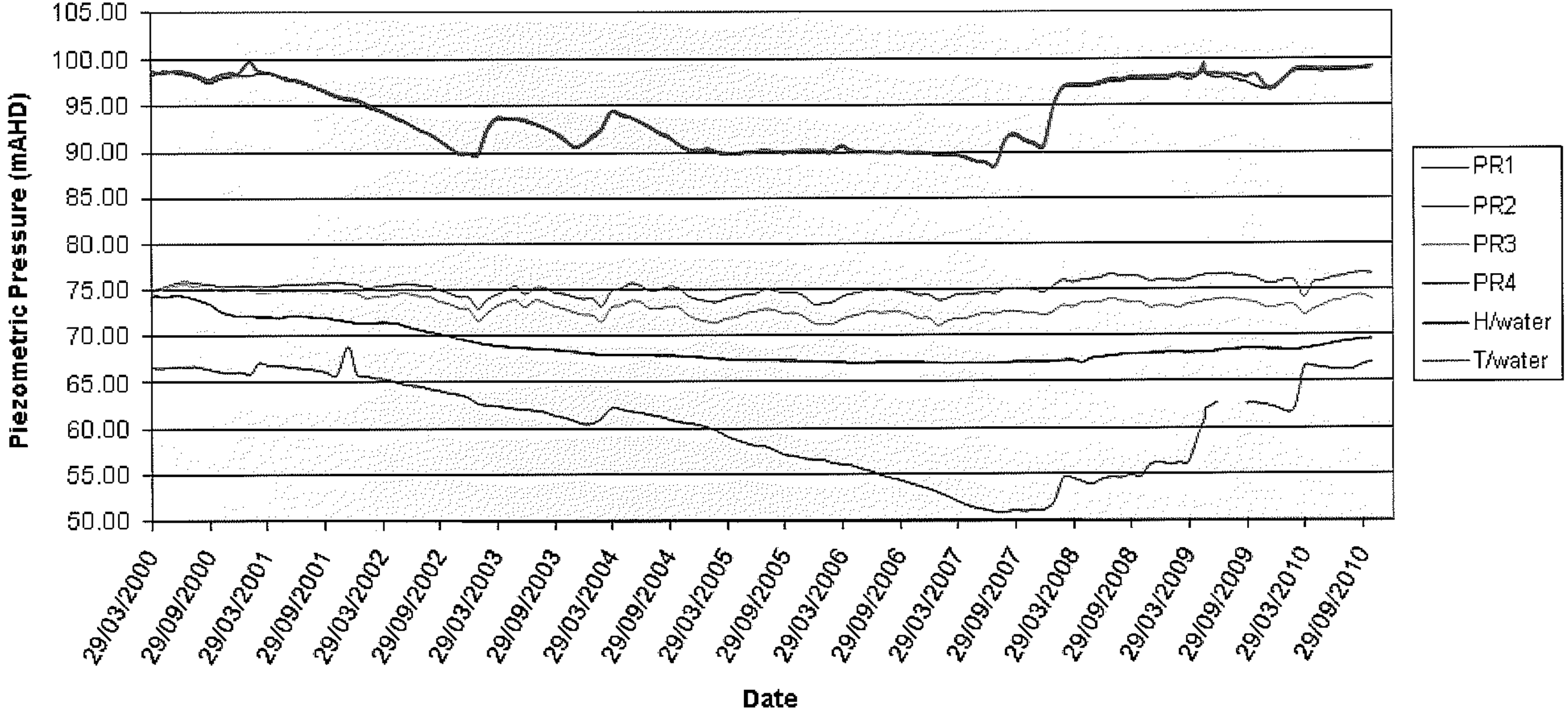
## Somerset Dam - Piezometers Monolith J



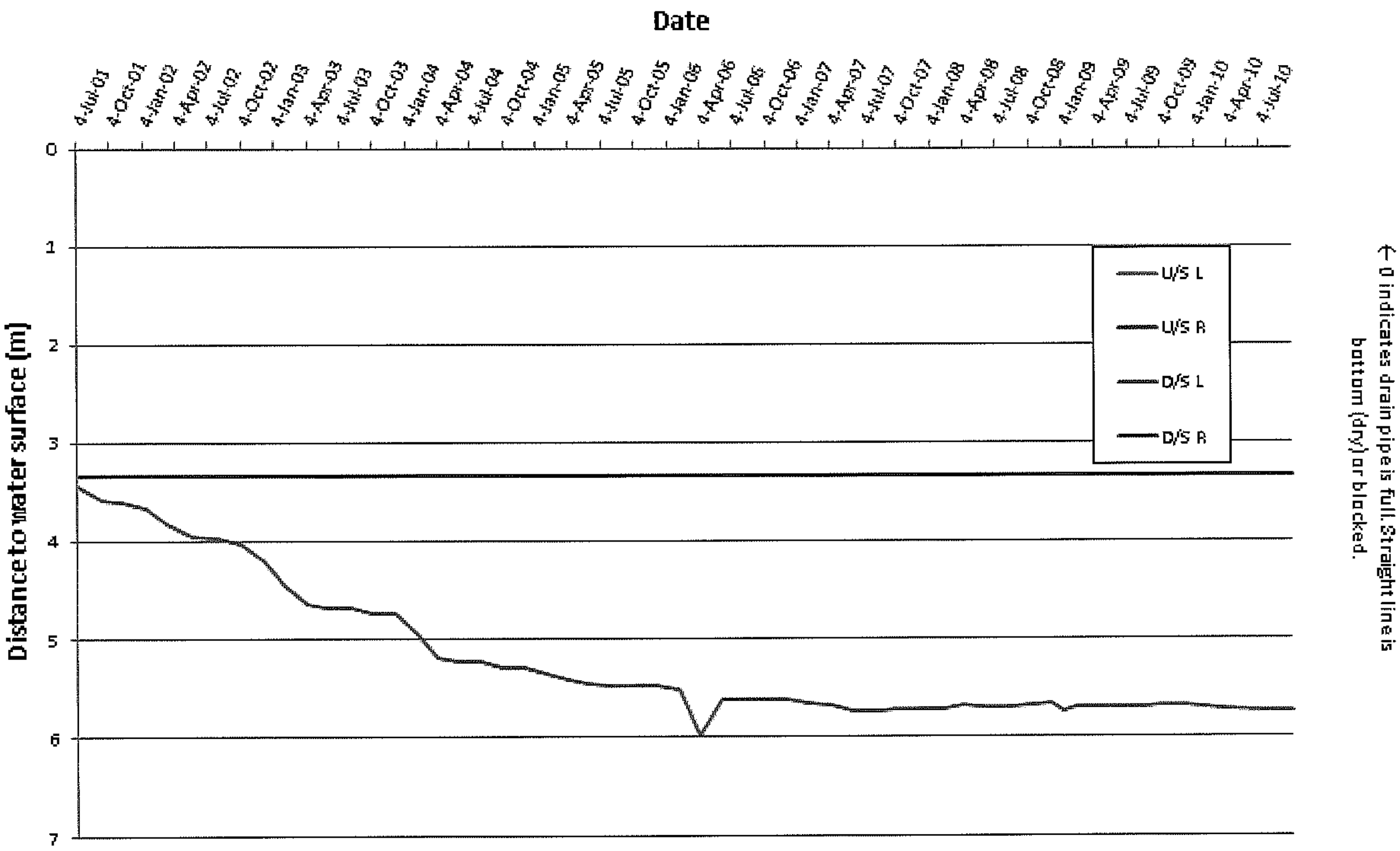


Somerset Dam - Piezometers Monolith M
Piezometric Pressure (mAHD)
105.00
100.00
95.00
90.00
85.00
80.00
75.00
70.00
65.00
60.00
55.00
50.00
29/03/2000
29/09/2000
29/03/2001
29/09/2001
29/03/2002
29/09/2002
29/03/2003
29/09/2003
29/03/2004
29/09/2004
29/03/2005
29/09/2005
29/03/2006
29/09/2006
29/03/2007
29/09/2007
29/03/2008
29/09/2008
29/03/2009
29/09/2009
29/03/2010
29/09/2010
Date
PM1
PM2
PM3
H/water
T/water

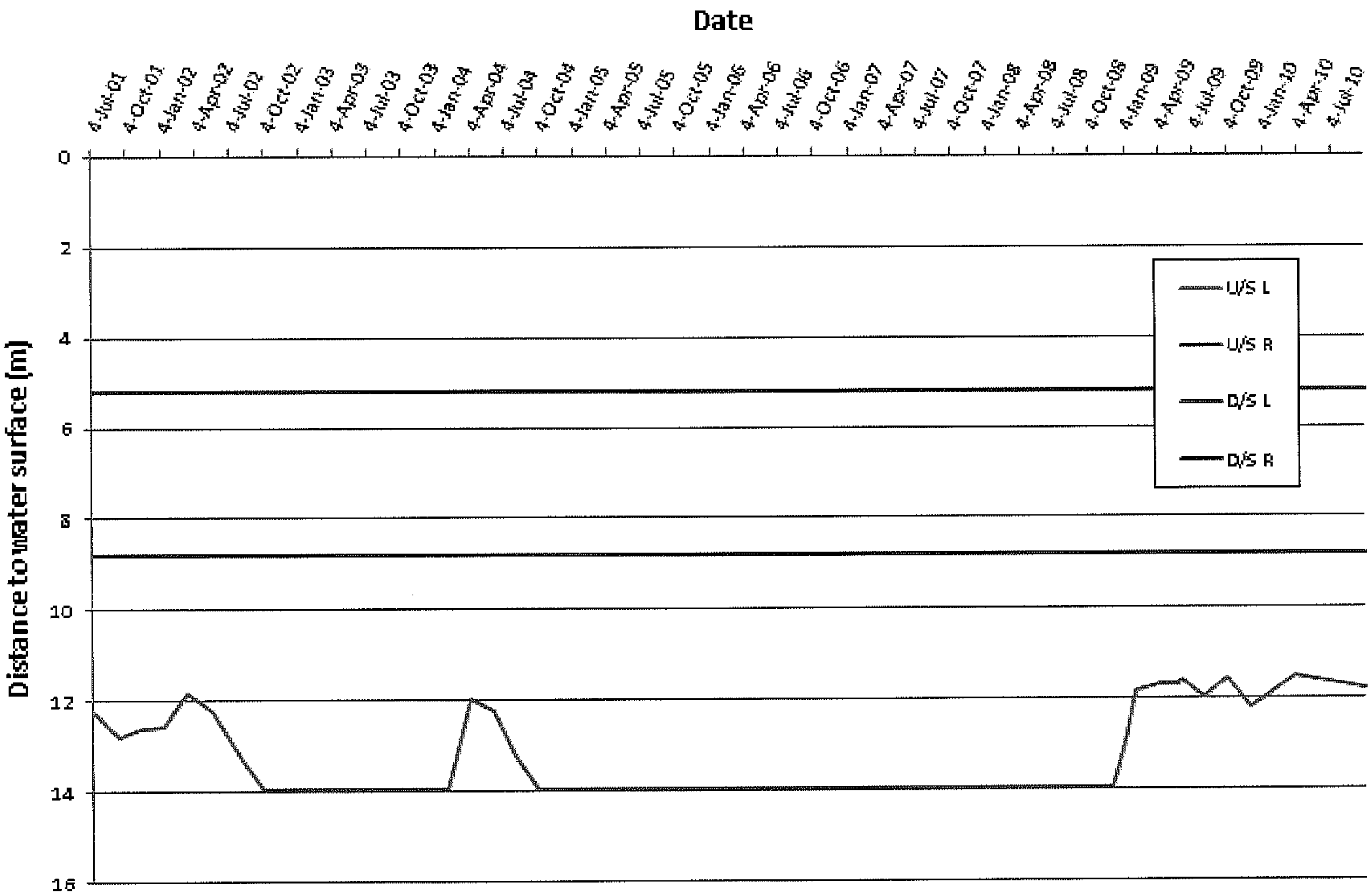
## Somerset Dam - Piezometers Monolith R



# Mono E - Somerset Dam - Uplift Drains

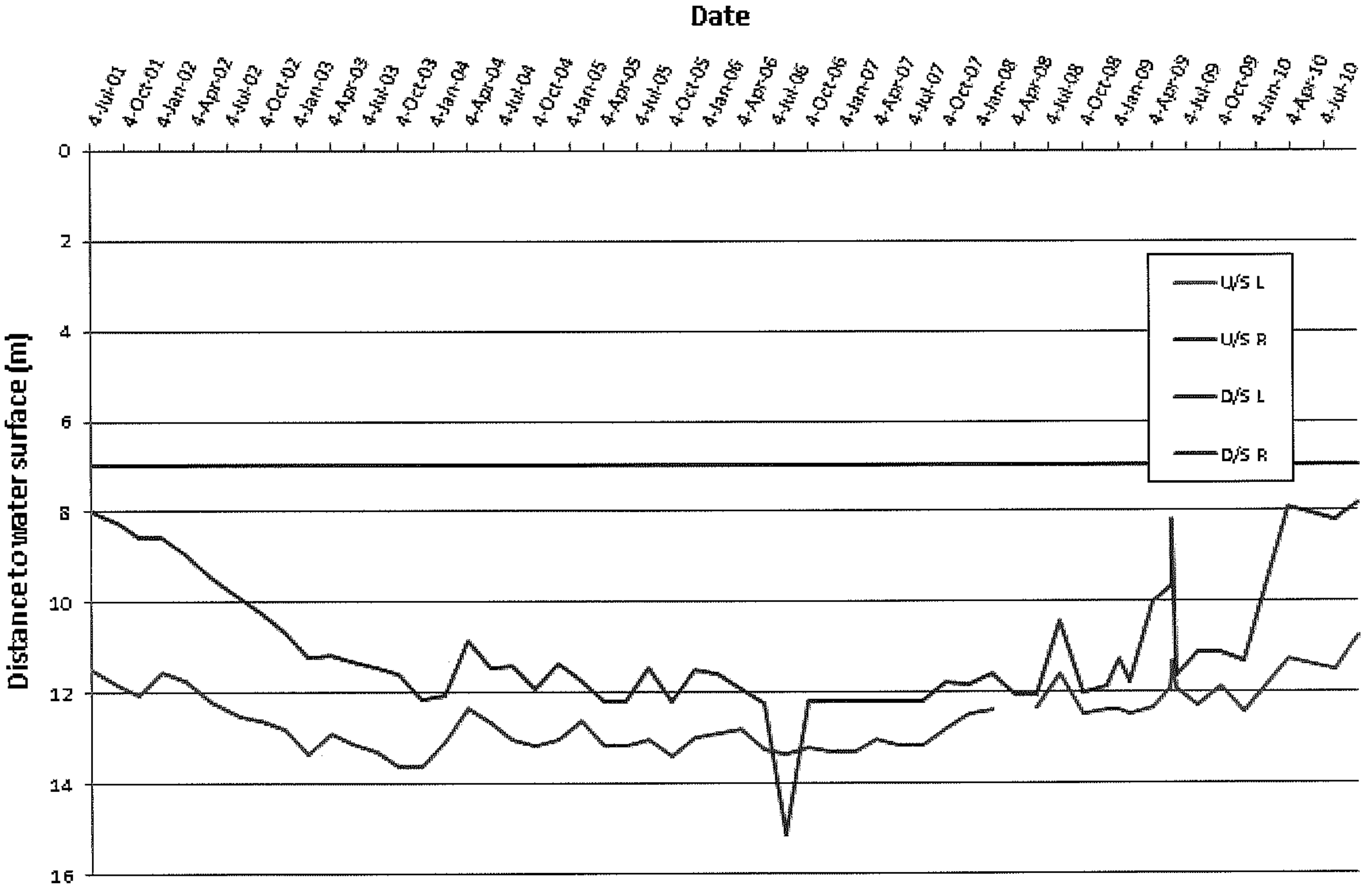


# Mono F - Somerset Dam - Uplift Drains



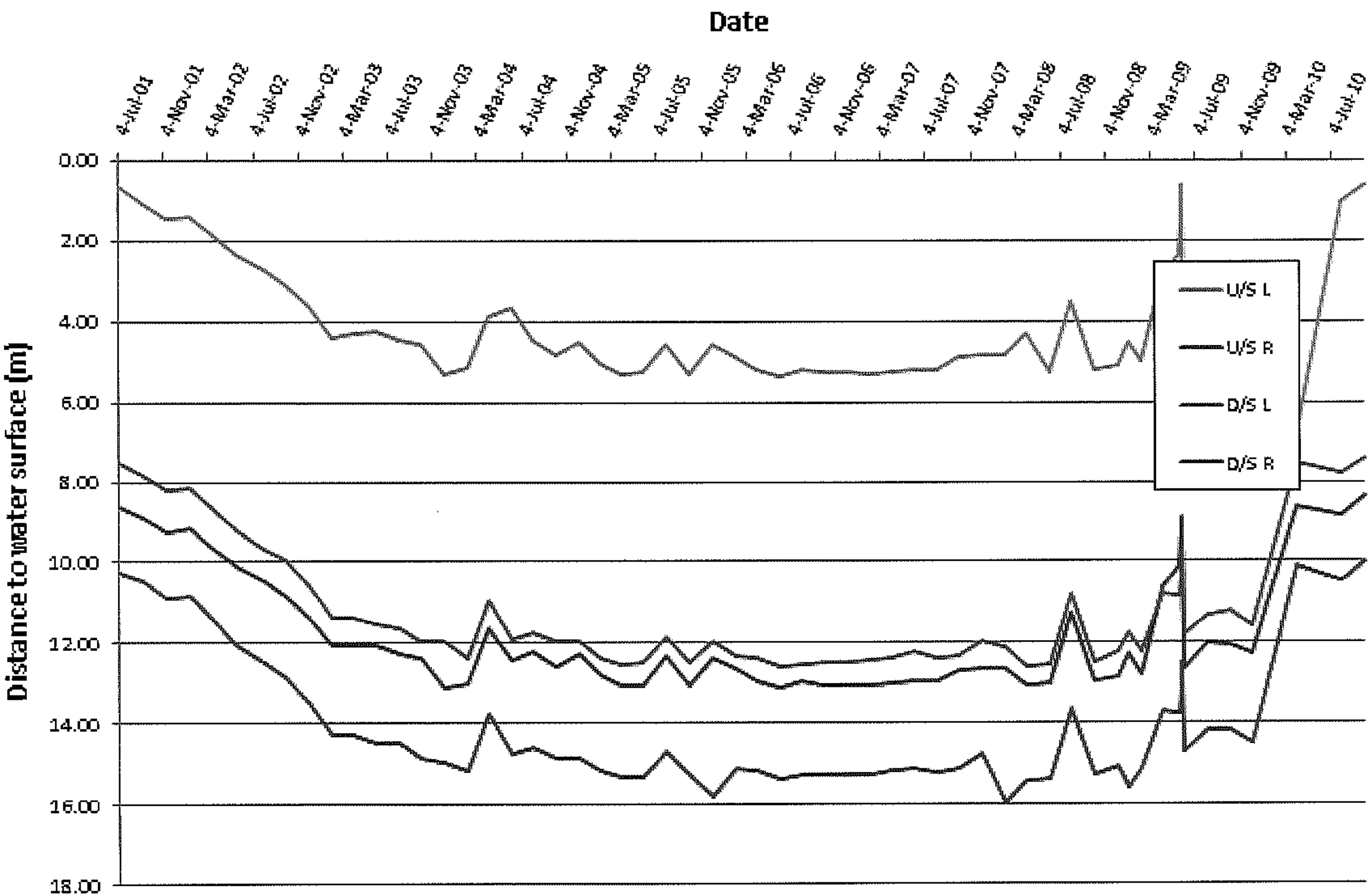

← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

# Mono G - Somerset Dam - Uplift Drains



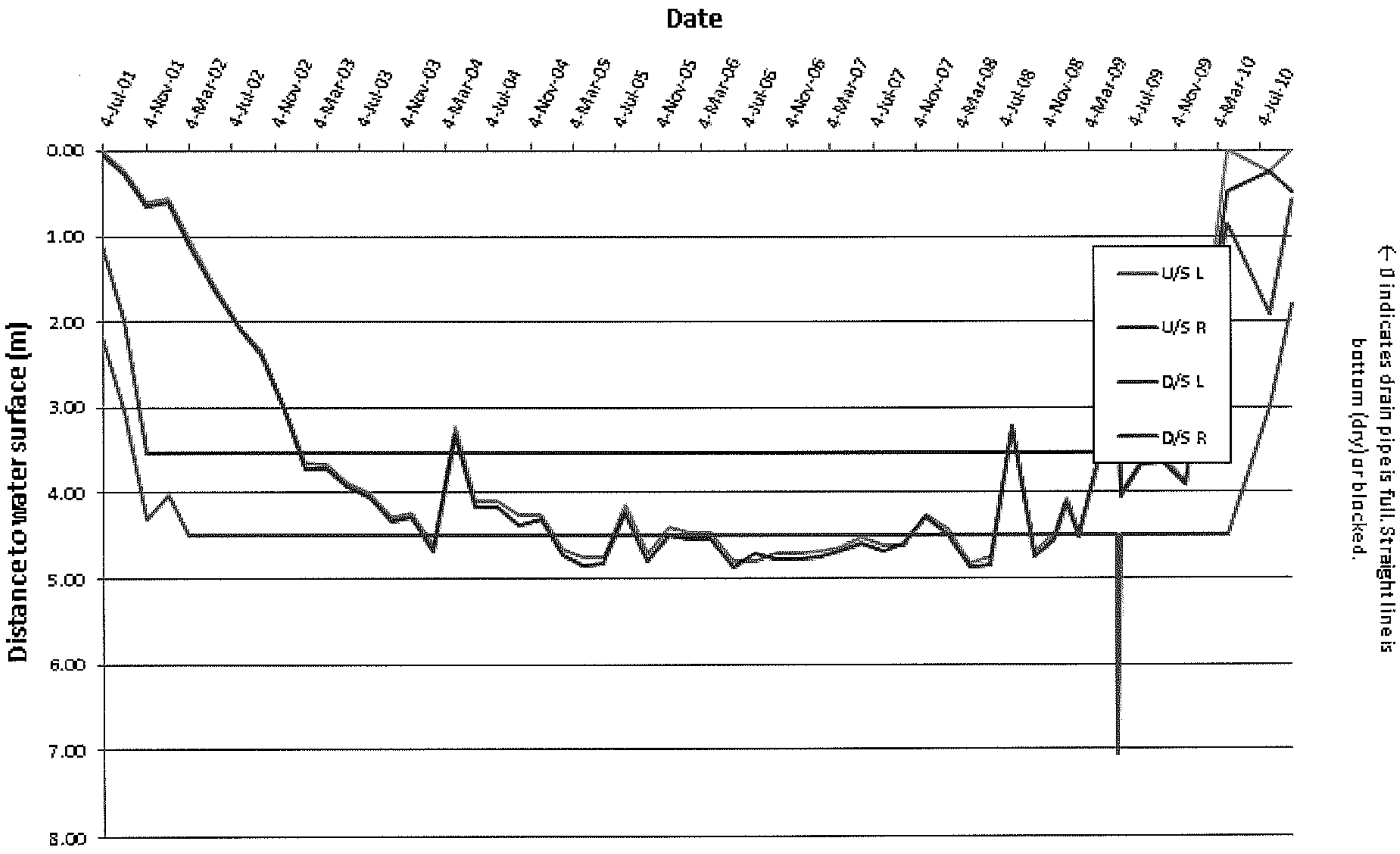

← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

# Mono H - Somerset Dam - Uplift Drains

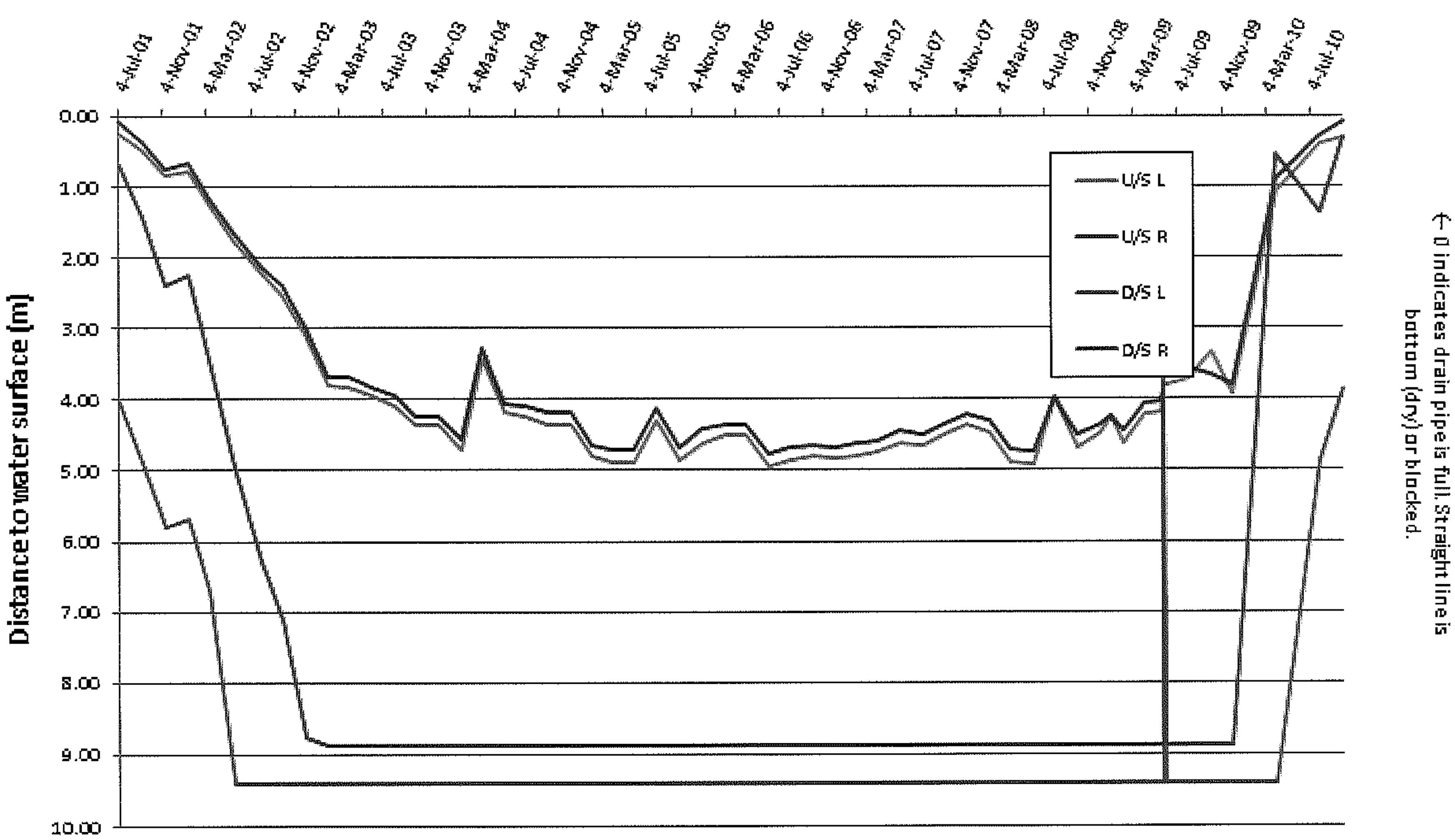


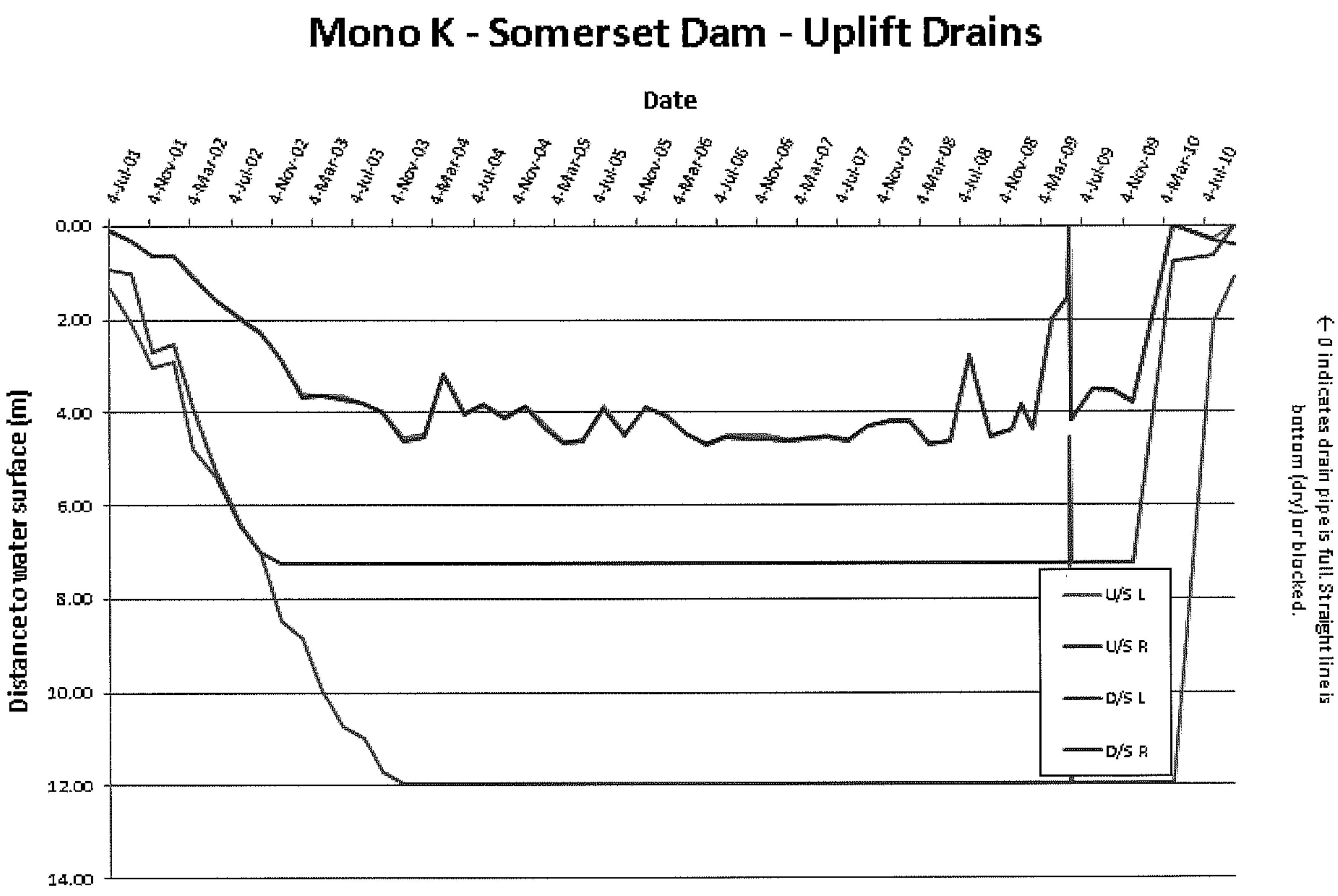

← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

## Mono I - Somerset Dam - Uplift Drains



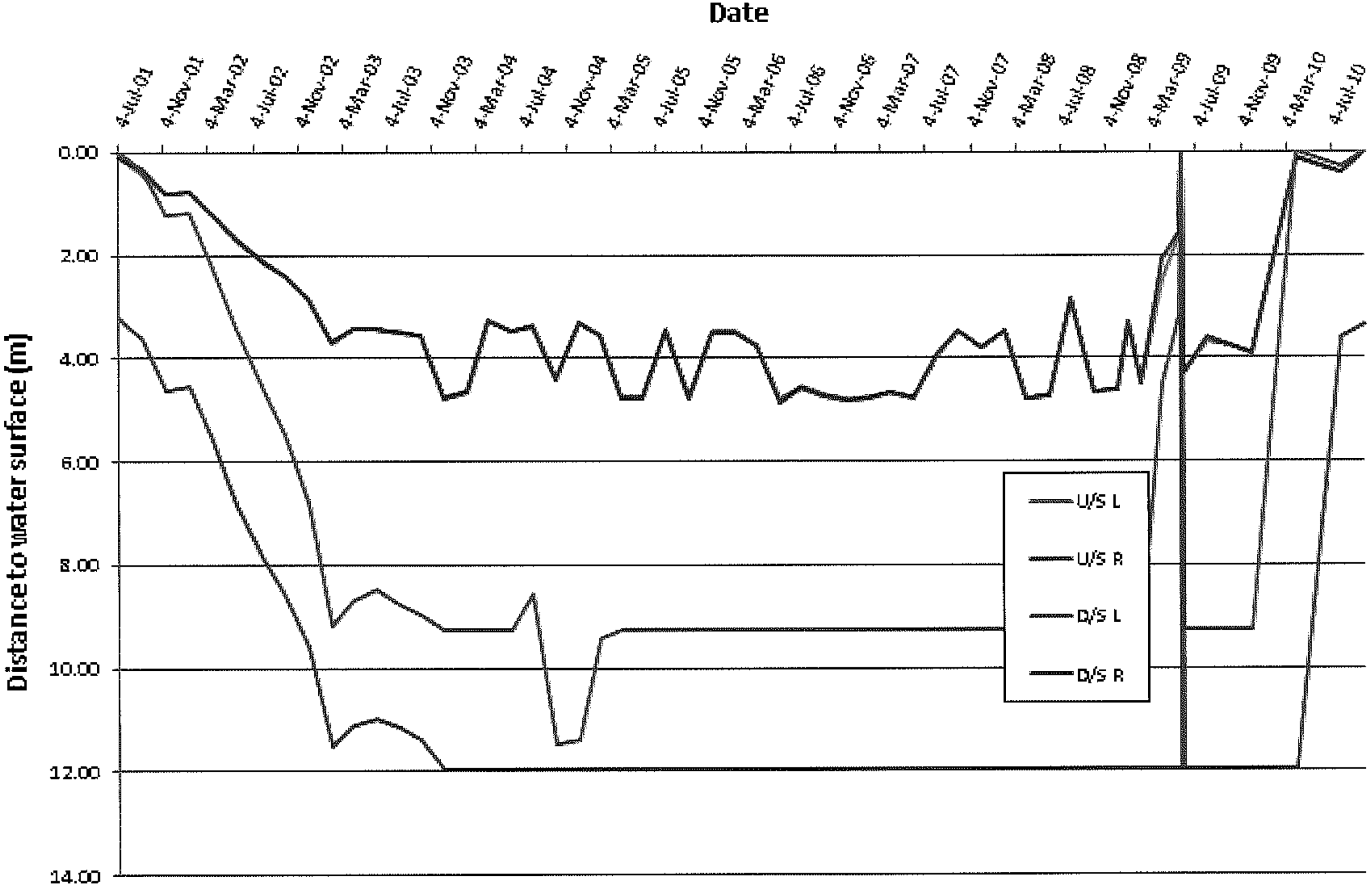
## Mono J - Somerset Dam - Uplift Drains



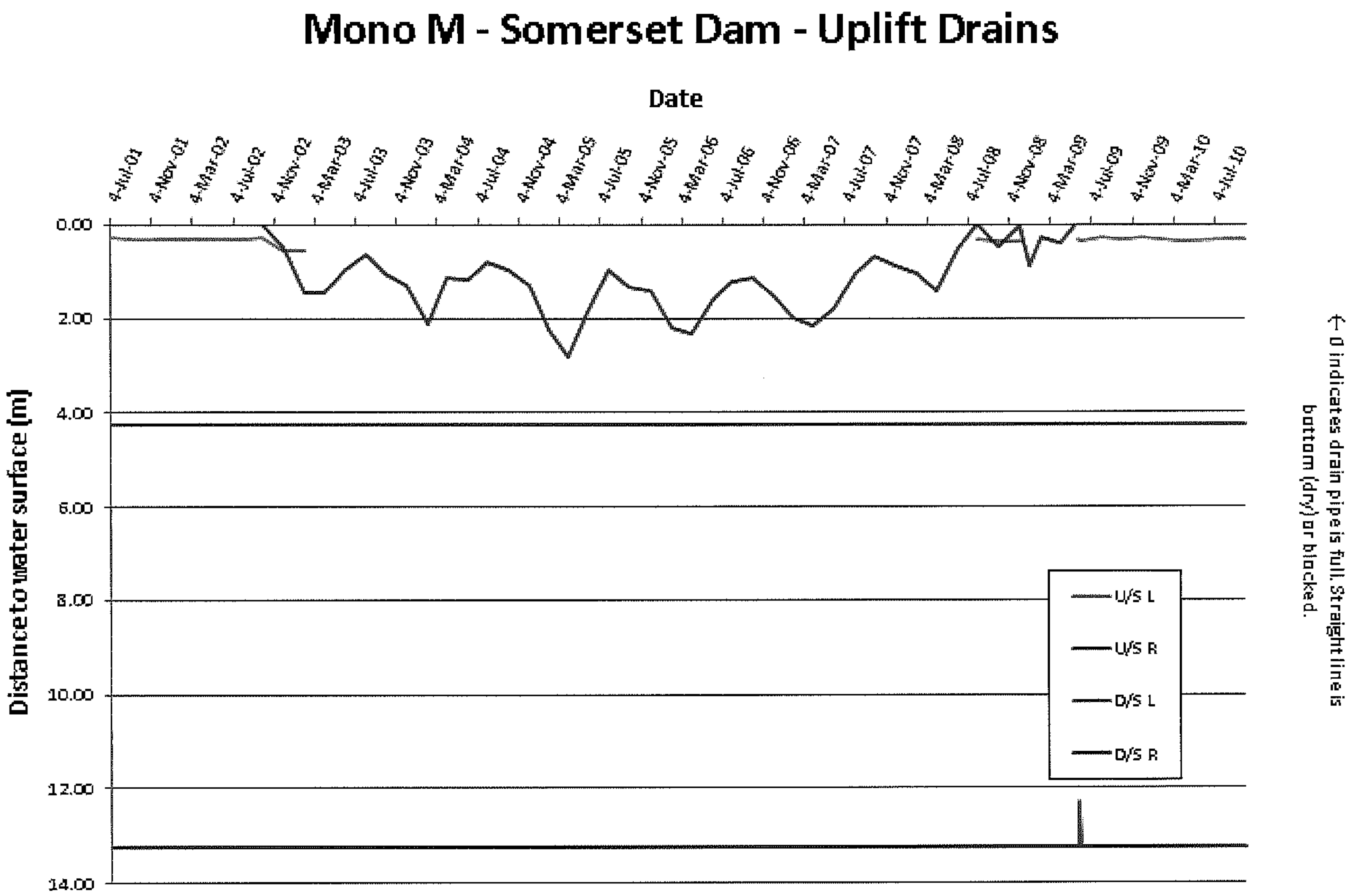


Mono K - Somerset Dam - Uplift Drains
Date
4-Jul-01
4-Nov-01
4-Mar-02
4-Jul-02
4-Nov-02
4-Mar-03
4-Jul-03
4-Nov-03
4-Mar-04
4-Jul-04
4-Nov-04
4-Mar-05
4-Jul-05
4-Nov-05
4-Mar-06
4-Jul-06
4-Nov-06
4-Mar-07
4-Jul-07
4-Nov-07
4-Mar-08
4-Jul-08
4-Nov-08
4-Mar-09
4-Jul-09
4-Nov-09
4-Mar-10
4-Jul-10
Distance to water surface (m)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

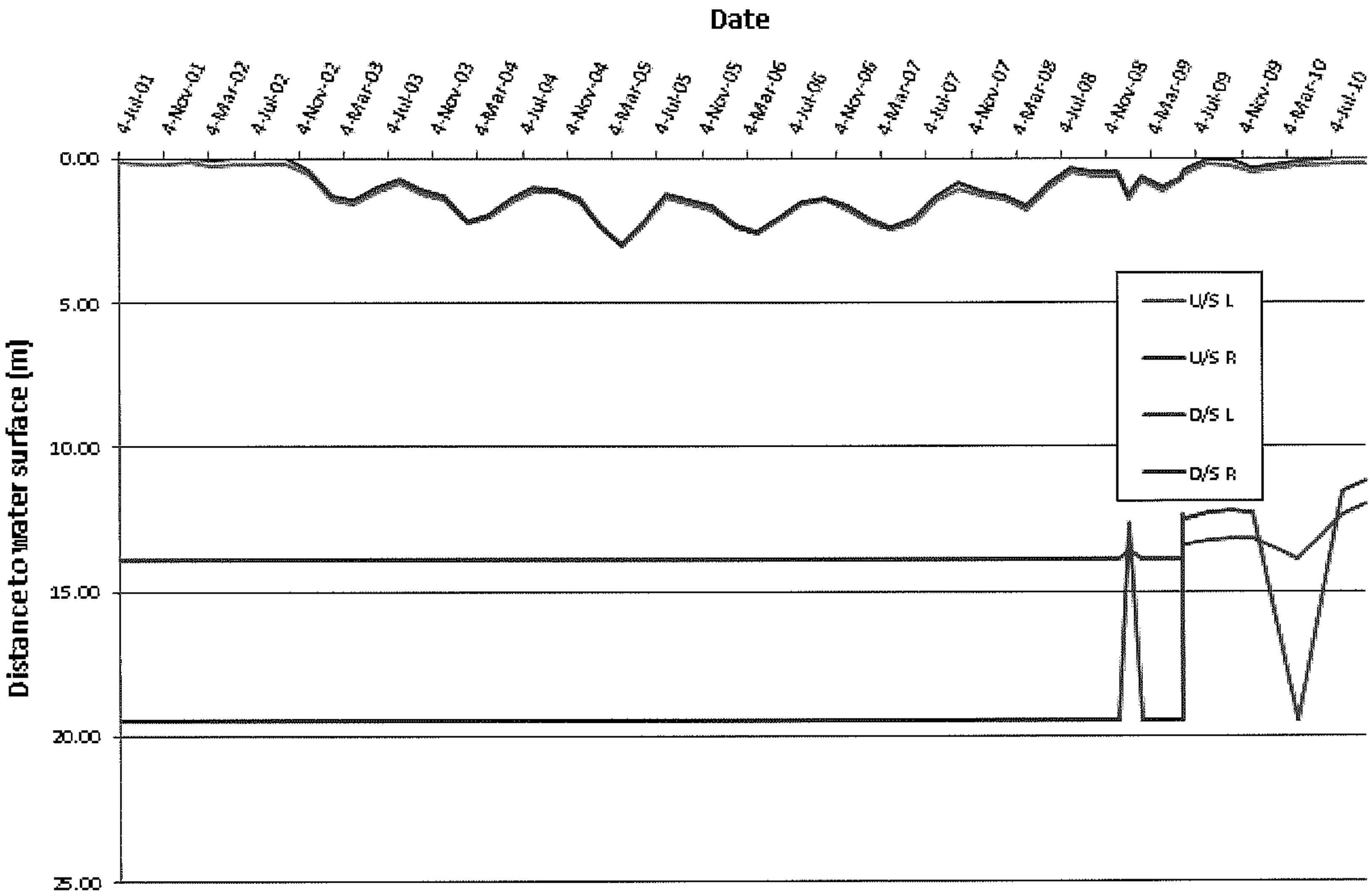
# Mono L - Somerset Dam - Uplift Drains



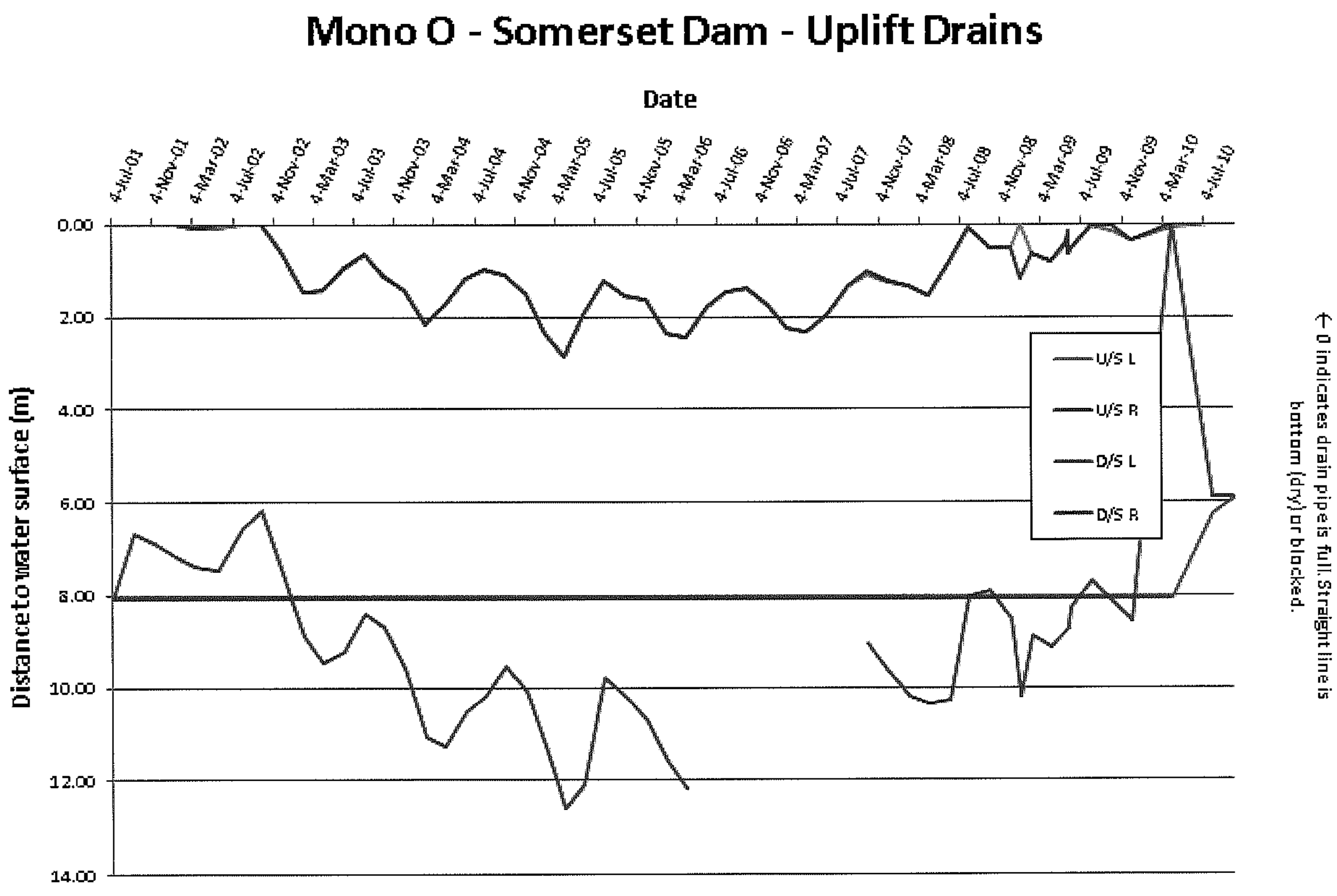

← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.


Mono M - Somerset Dam - Uplift Drains
Date
4-Jul-01
4-Nov-01
4-Mar-02
4-Jul-02
4-Nov-02
4-Mar-03
4-Jul-03
4-Nov-03
4-Mar-04
4-Jul-04
4-Nov-04
4-Mar-05
4-Jul-05
4-Nov-05
4-Mar-06
4-Jul-06
4-Nov-06
4-Mar-07
4-Jul-07
4-Nov-07
4-Mar-08
4-Jul-08
4-Nov-08
4-Mar-09
4-Jul-09
4-Nov-09
4-Mar-10
4-Jul-10
Distance to water surface (m)
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

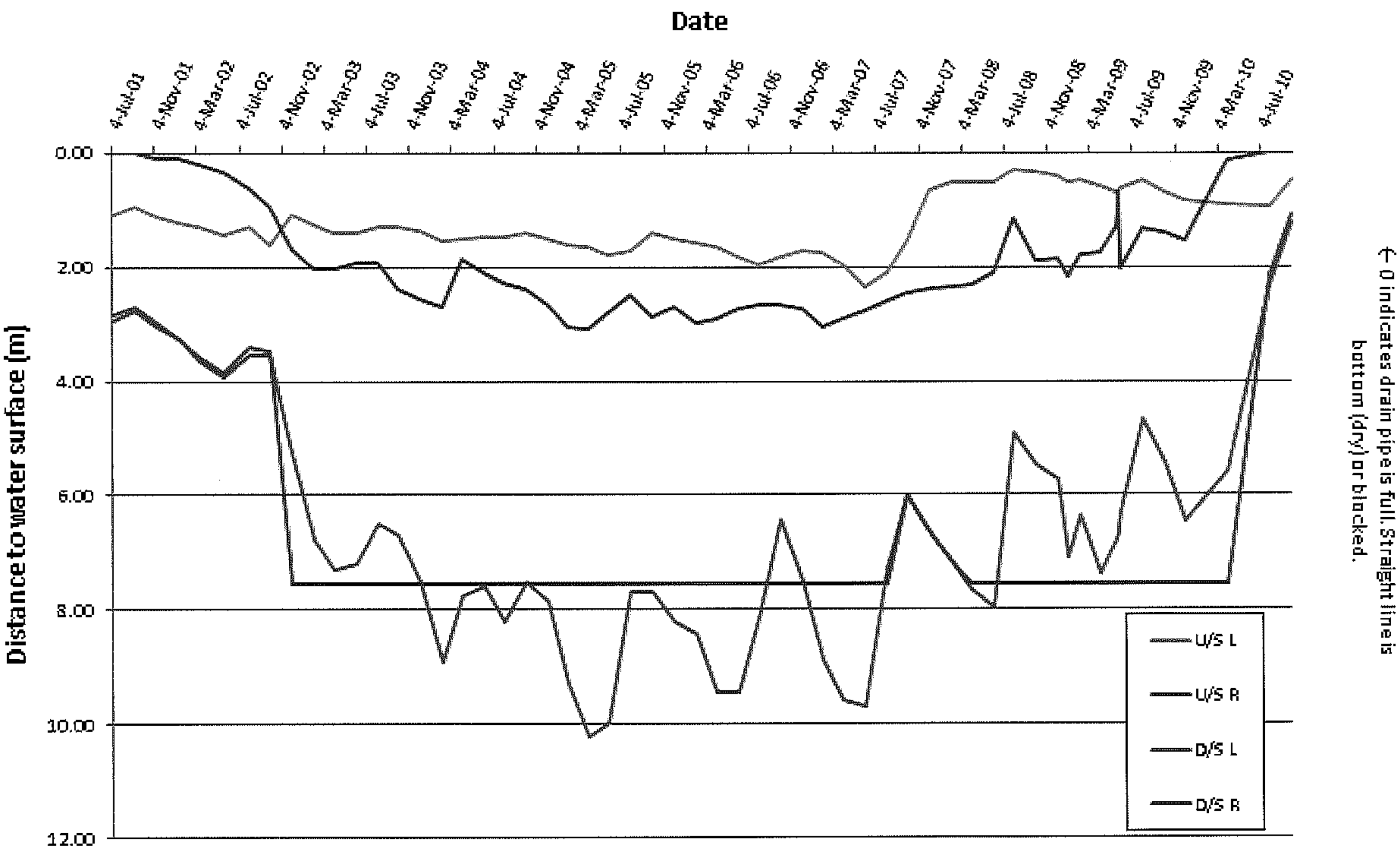
# Mono N - Somerset Dam - Uplift Drains



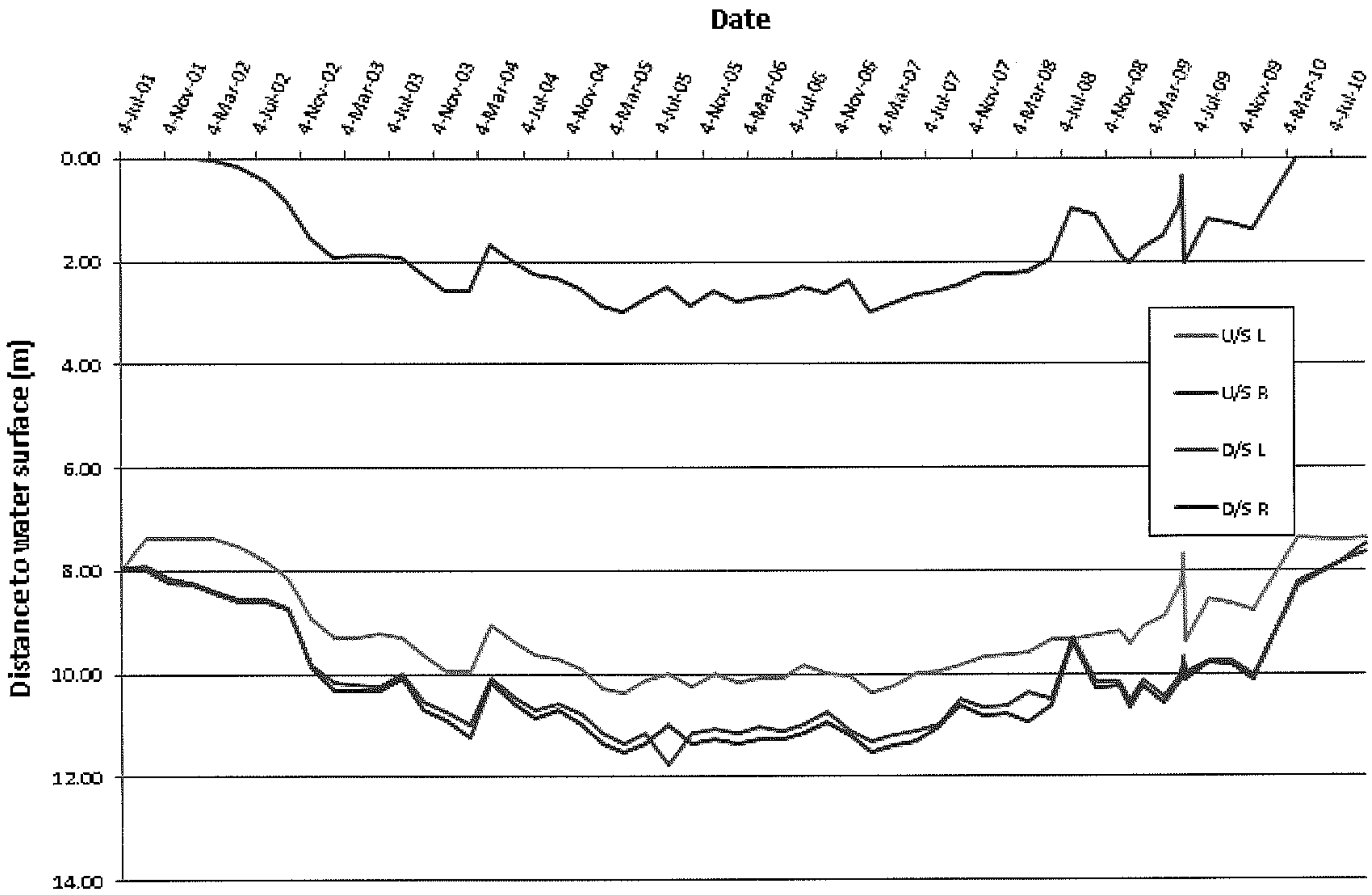

← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.


Mono O - Somerset Dam - Uplift Drains
Date
4-Jul-01
4-Nov-01
4-Mar-02
4-Jul-02
4-Nov-02
4-Mar-03
4-Jul-03
4-Nov-03
4-Mar-04
4-Jul-04
4-Nov-04
4-Mar-05
4-Jul-05
4-Nov-05
4-Mar-06
4-Jul-06
4-Nov-06
4-Mar-07
4-Jul-07
4-Nov-07
4-Mar-08
4-Jul-08
4-Nov-08
4-Mar-09
4-Jul-09
4-Nov-09
4-Mar-10
4-Jul-10
0.00
2.00
4.00
6.00
8.00
10.00
12.00
14.00
Distance to water surface (m)
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

# Mono P - Somerset Dam - Uplift Drains

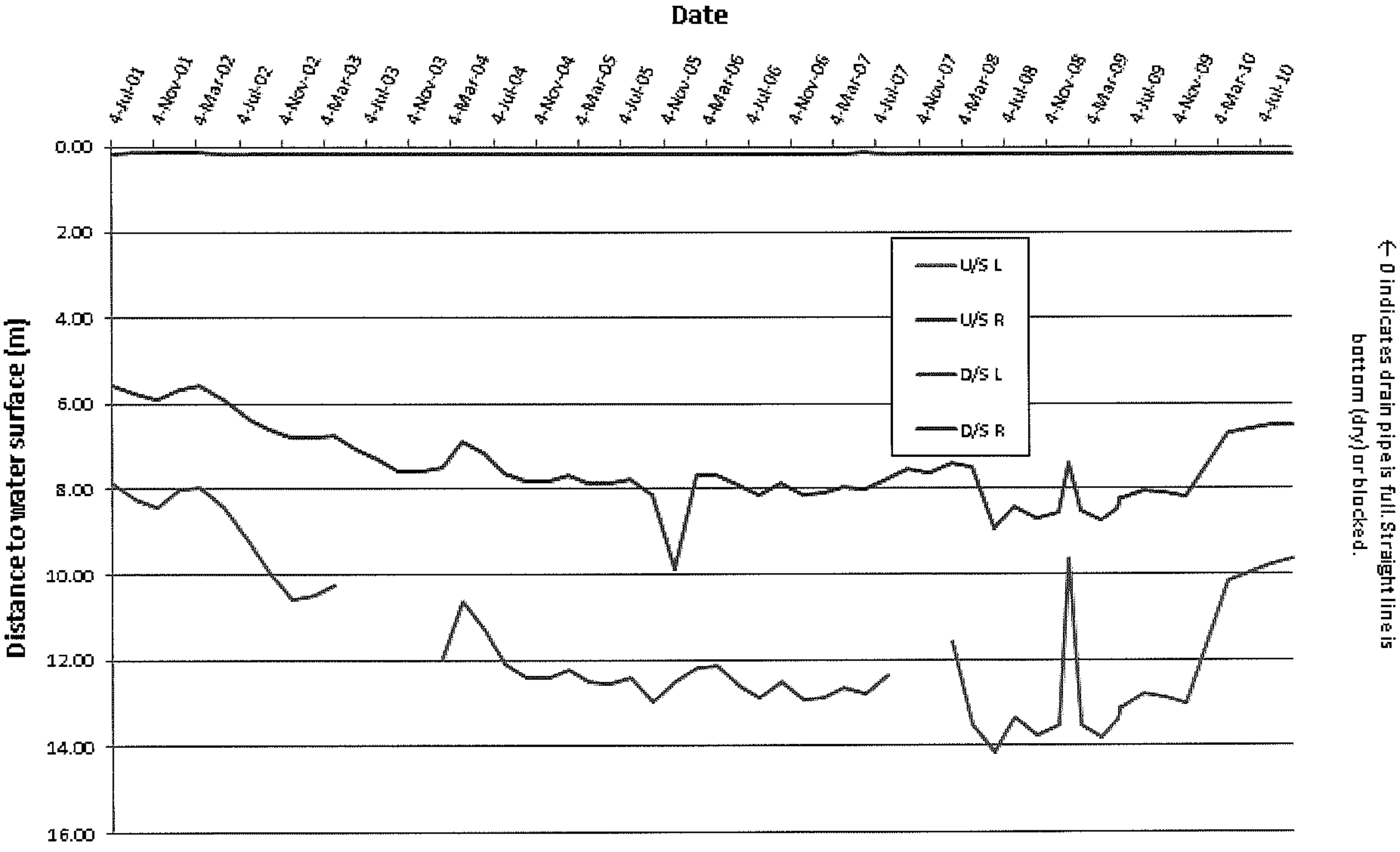


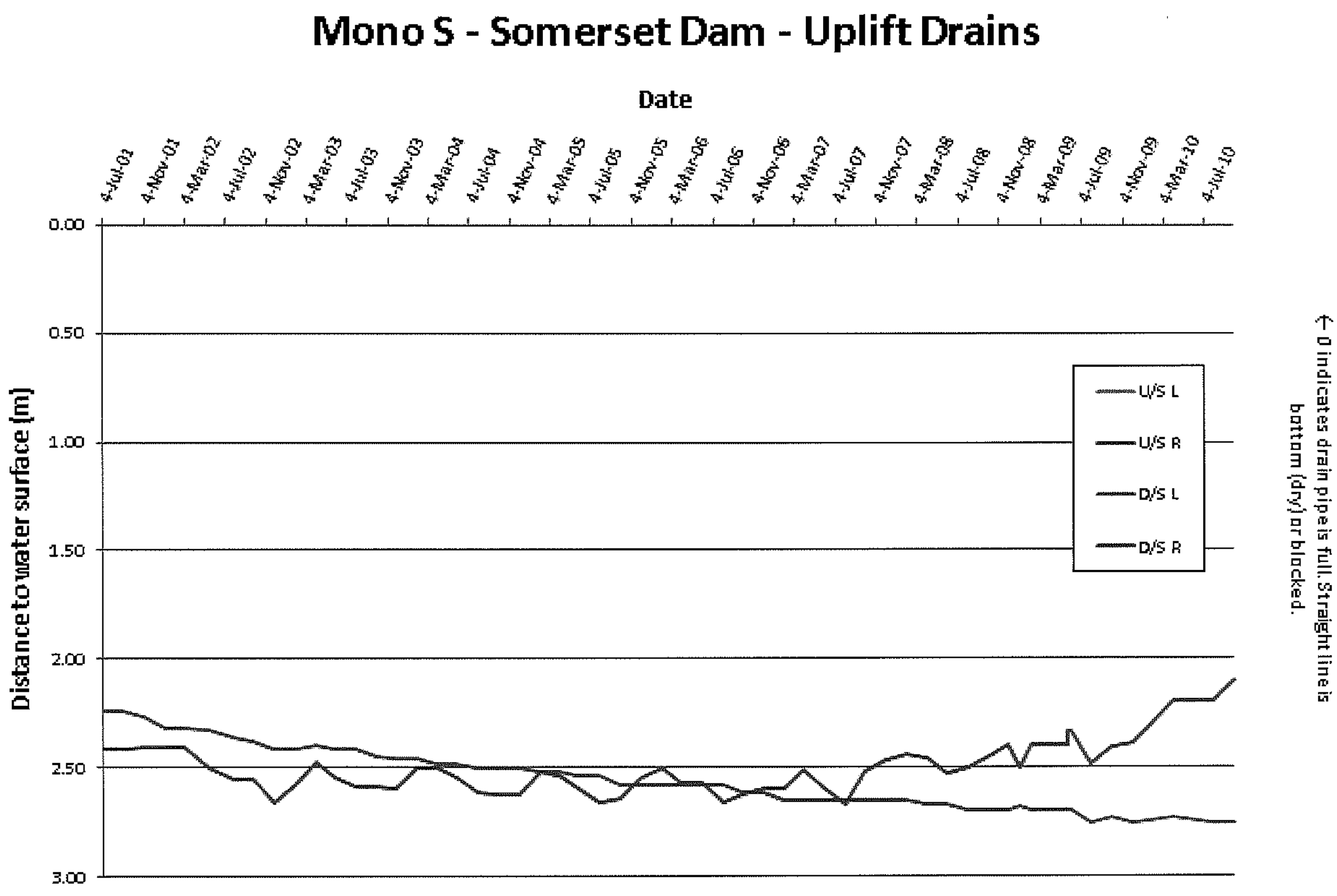
## Mono Q - Somerset Dam - Uplift Drains



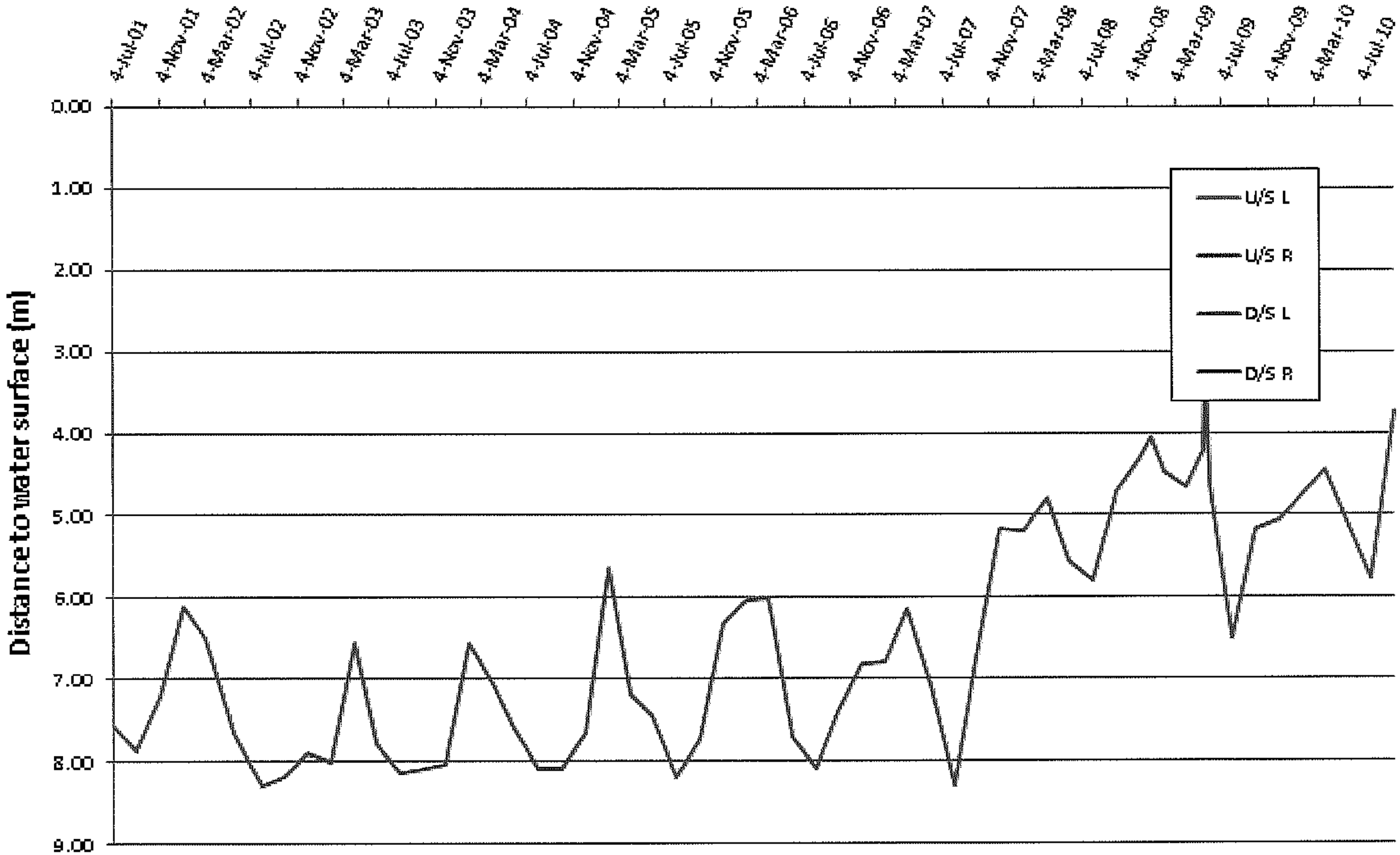

← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

## Mono R - Somerset Dam - Uplift Drains



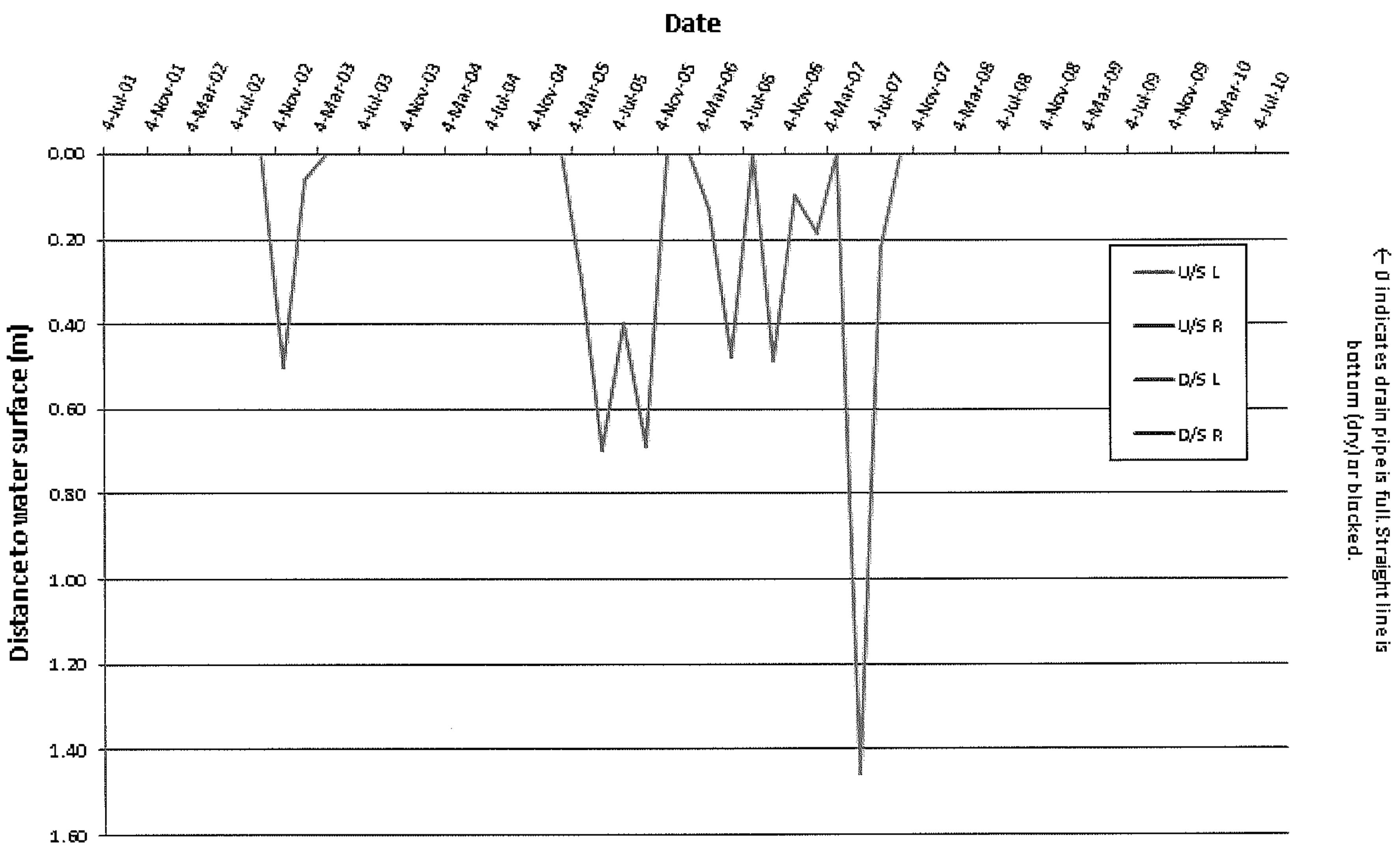

Mono S - Somerset Dam - Uplift Drains
Date
4-Jul-01
4-Nov-01
4-Mar-02
4-Jul-02
4-Nov-02
4-Mar-03
4-Jul-03
4-Nov-03
4-Mar-04
4-Jul-04
4-Nov-04
4-Mar-05
4-Jul-05
4-Nov-05
4-Mar-06
4-Jul-06
4-Nov-06
4-Mar-07
4-Jul-07
4-Nov-07
4-Mar-08
4-Jul-08
4-Nov-08
4-Mar-09
4-Jul-09
4-Nov-09
4-Mar-10
4-Jul-10
Distance to water surface (m)
0.00
0.50
1.00
1.50
2.00
2.50
3.00
U/S L
U/S R
D/S L
D/S R
← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

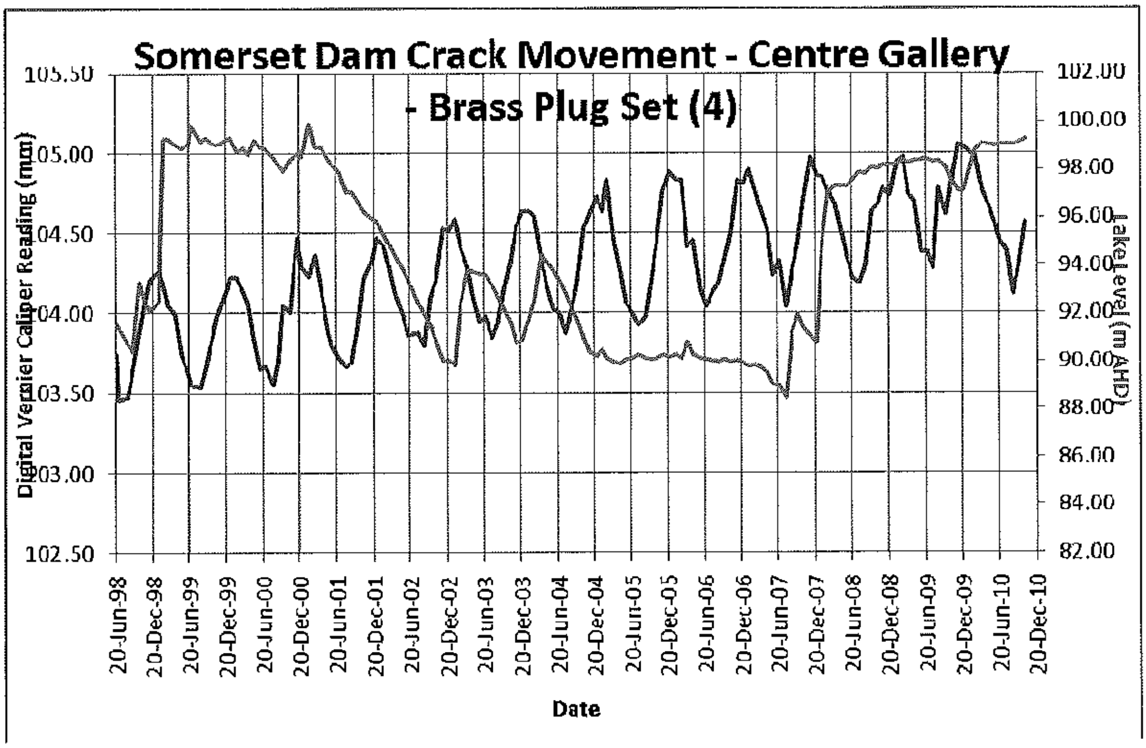
## Mono T - Somerset Dam - Uplift Drains




← 0 indicates drain pipe is full. Straight line is bottom (dry) or blocked.

# Mono U - Somerset Dam - Uplift Drains





Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (4)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
Date
105.50
105.00
104.50
104.00
103.50
103.00
102.50
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
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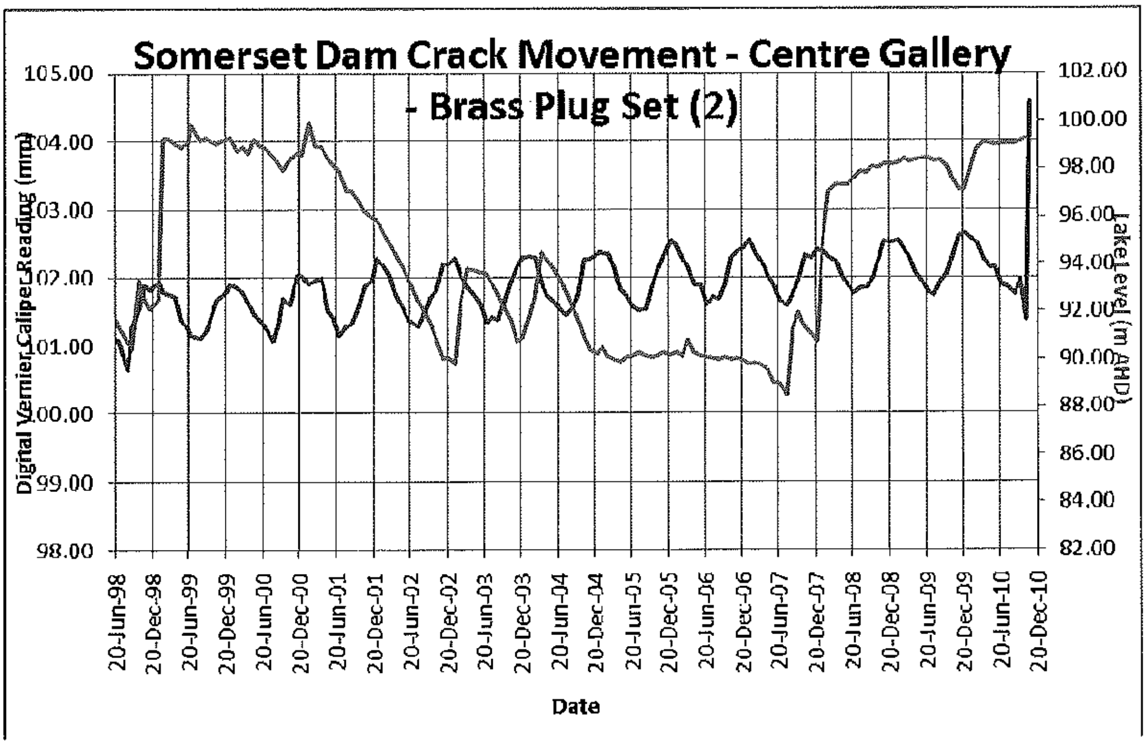
20-Jun-10
20-Dec-10

Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (2)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
Date
105.00
104.00
103.00
102.00
101.00
100.00
99.00
98.00
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
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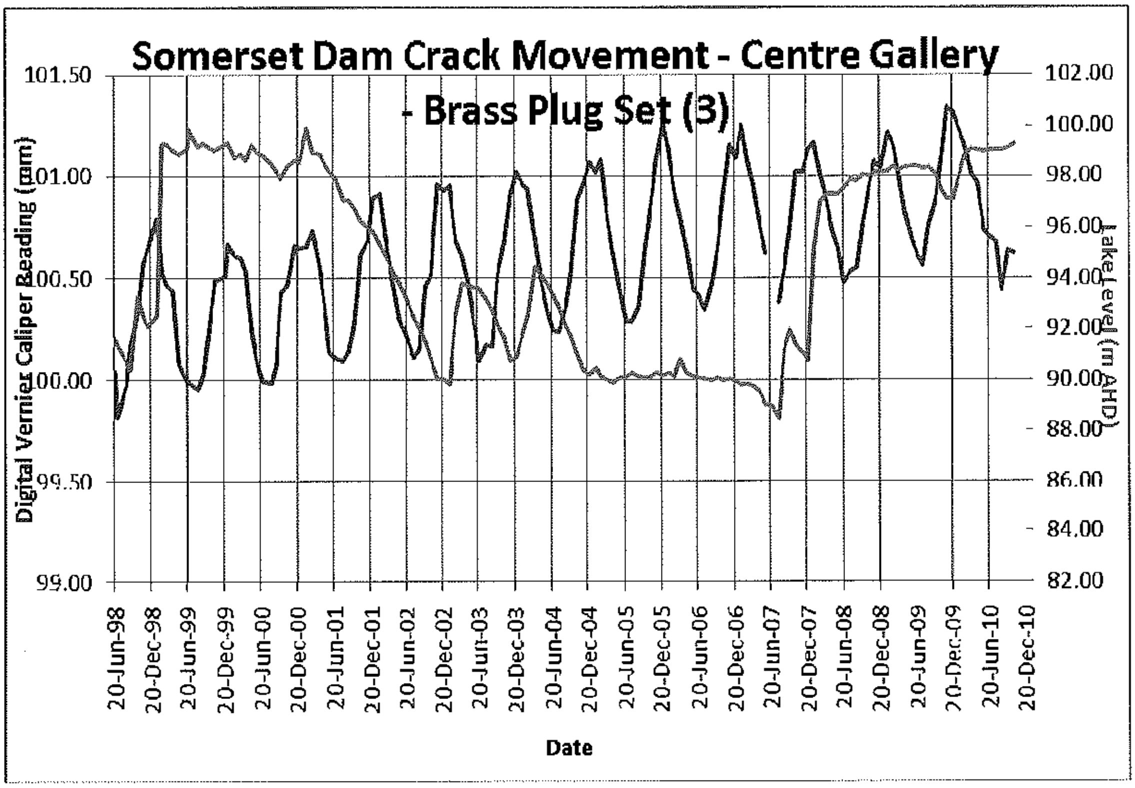
20-Jun-10
20-Dec-10


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (3)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
101.50
101.00
100.50
100.00
99.50
99.00
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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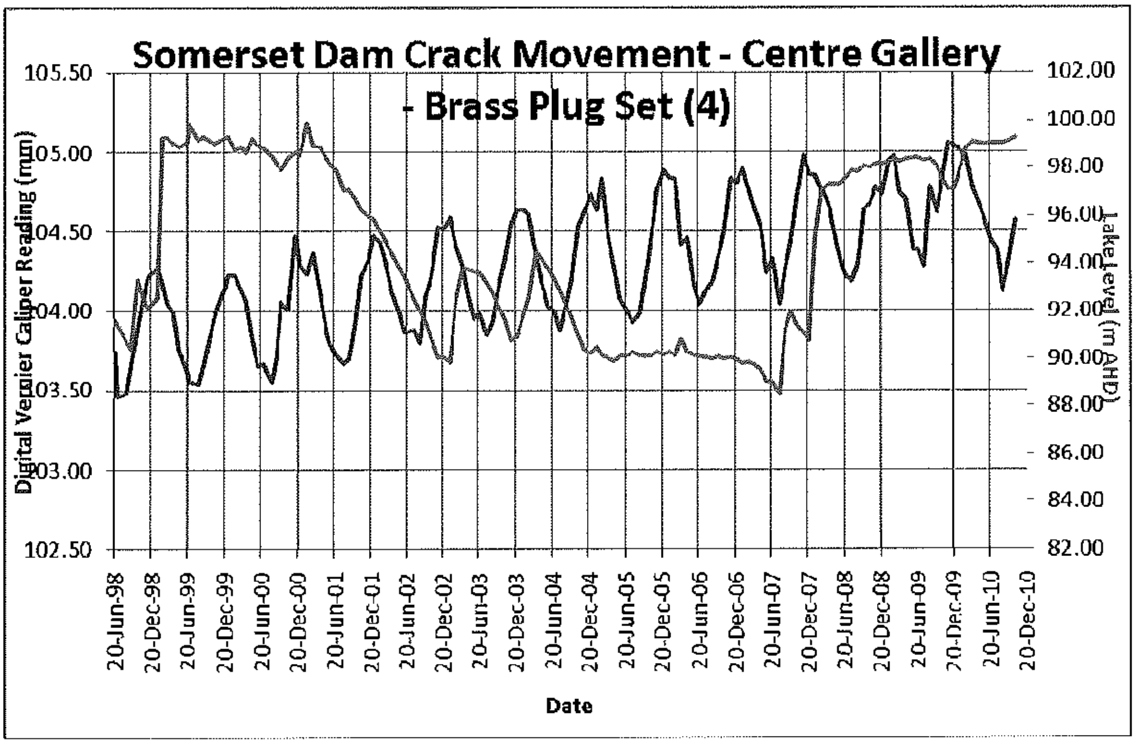
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (4)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
105.50
105.00
104.50
104.00
103.50
103.00
102.50
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
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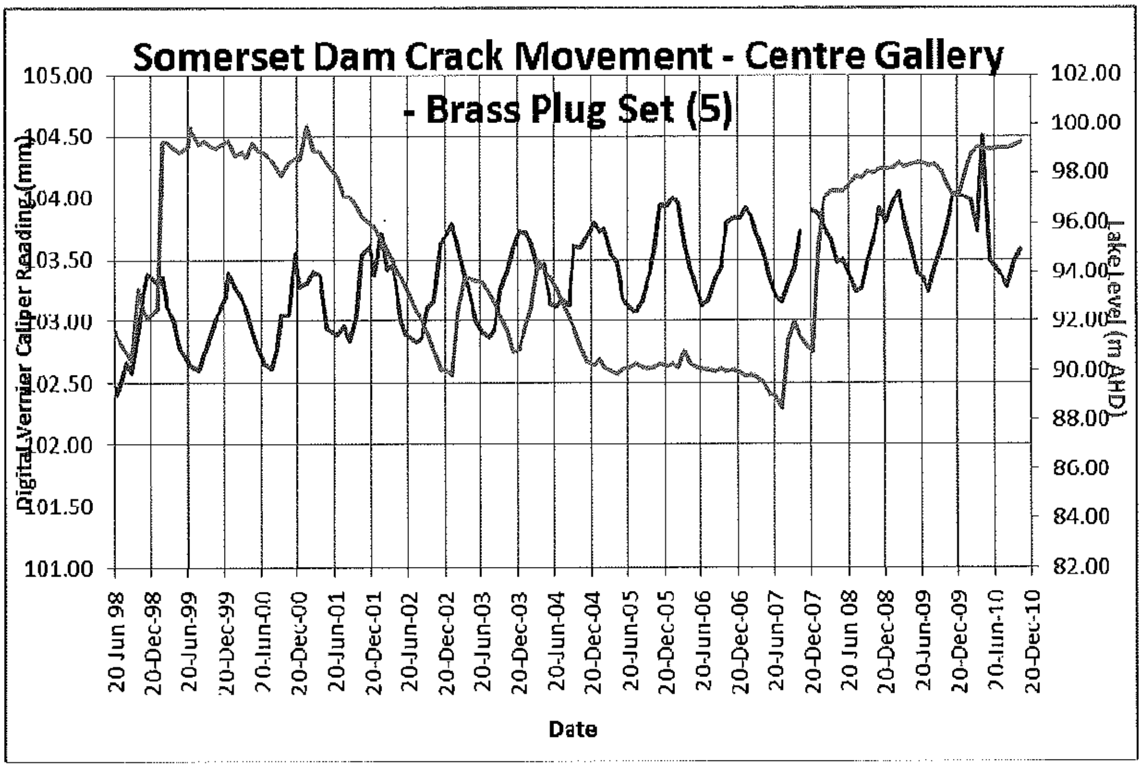
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (5)
Digital Vernier Caliper Reading (mm)
105.00
104.50
104.00
103.50
103.00
102.50
102.00
101.50
101.00
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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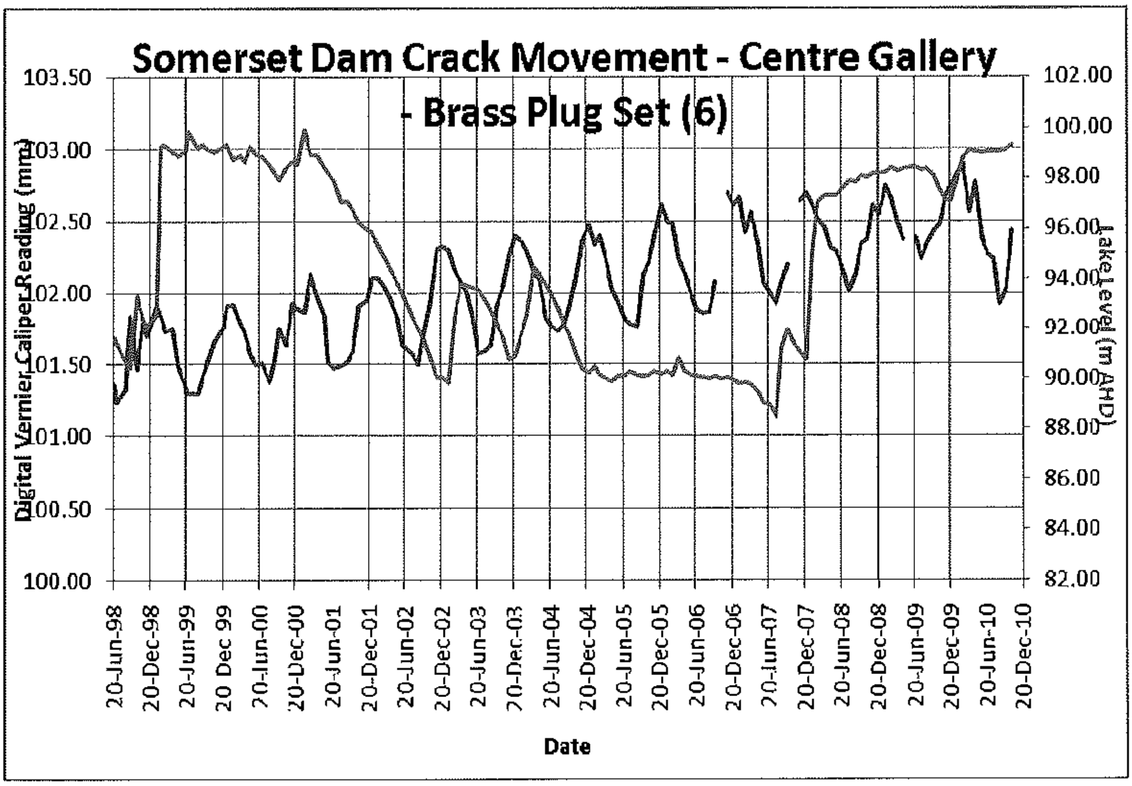
20-Dec-08
20-Jun-09
20-Dec-09
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (6)
Digital Vernier Caliper Reading (mm)
103.50
103.00
102.50
102.00
101.50
101.00
100.50
100.00
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
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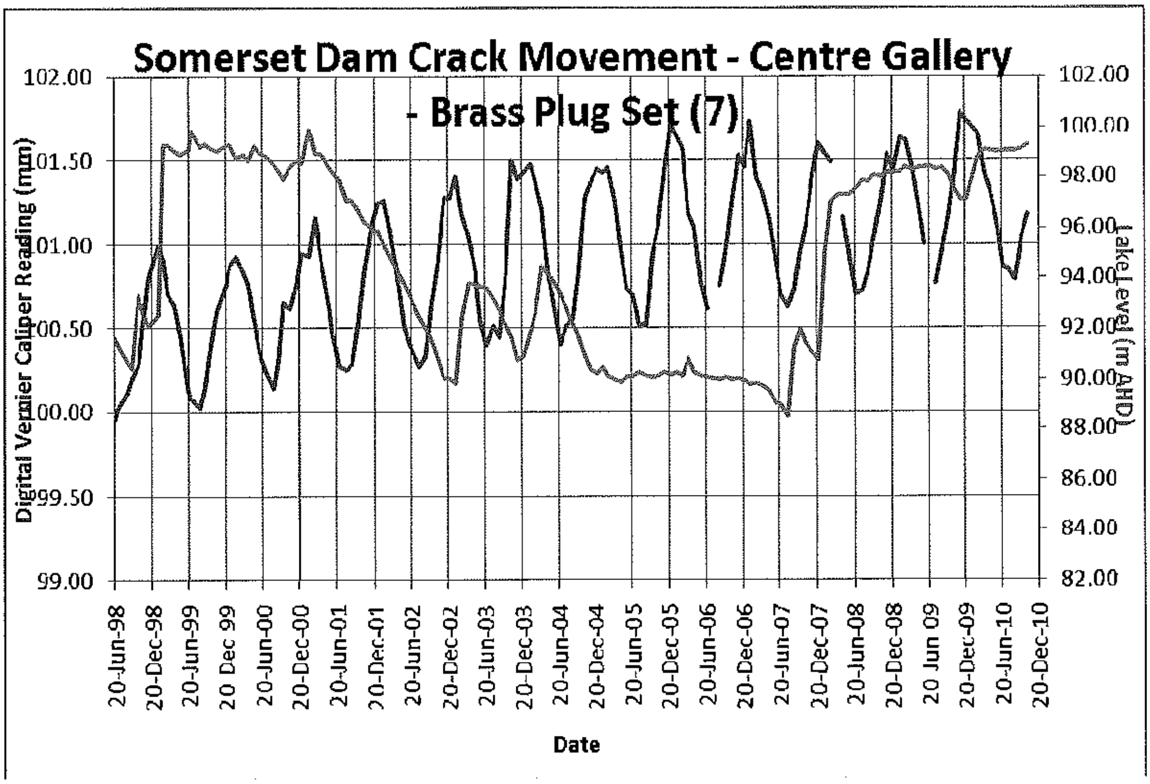
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (7)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
102.00
101.50
101.00
100.50
100.00
99.50
99.00
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20 Dec 99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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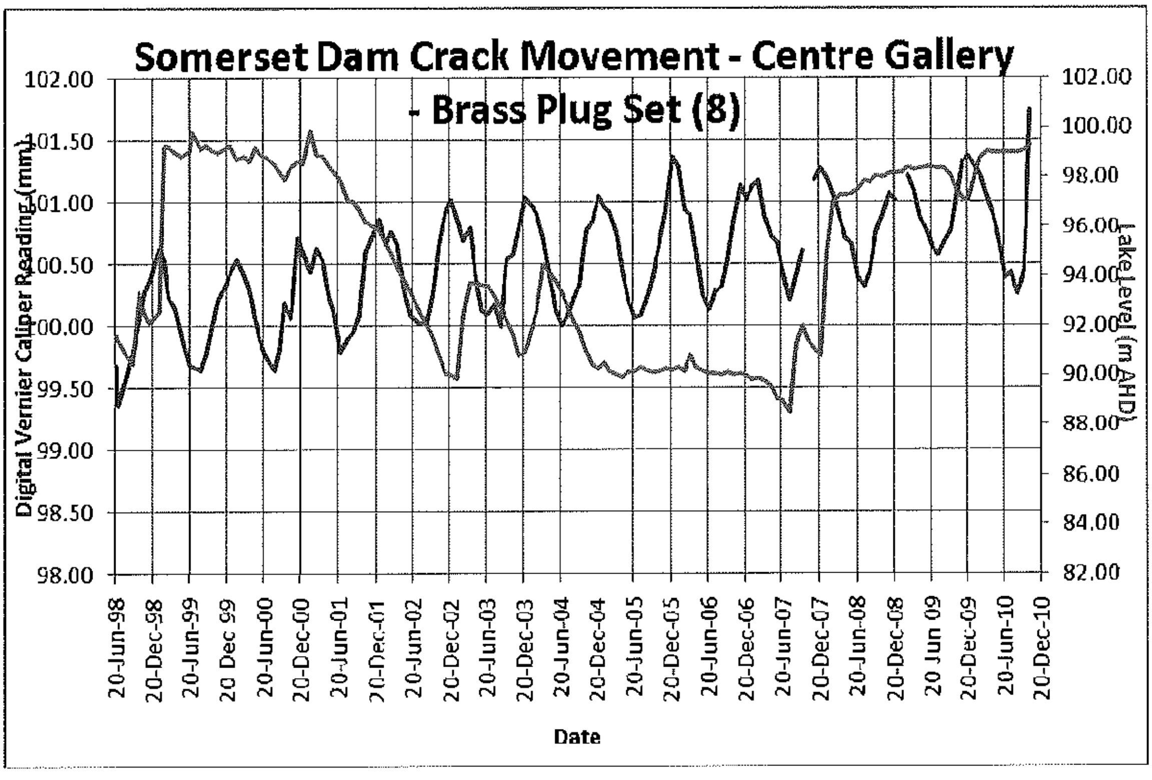
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (8)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
102.00
101.50
101.00
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100.00
99.50
99.00
98.50
98.00
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20 Dec 99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
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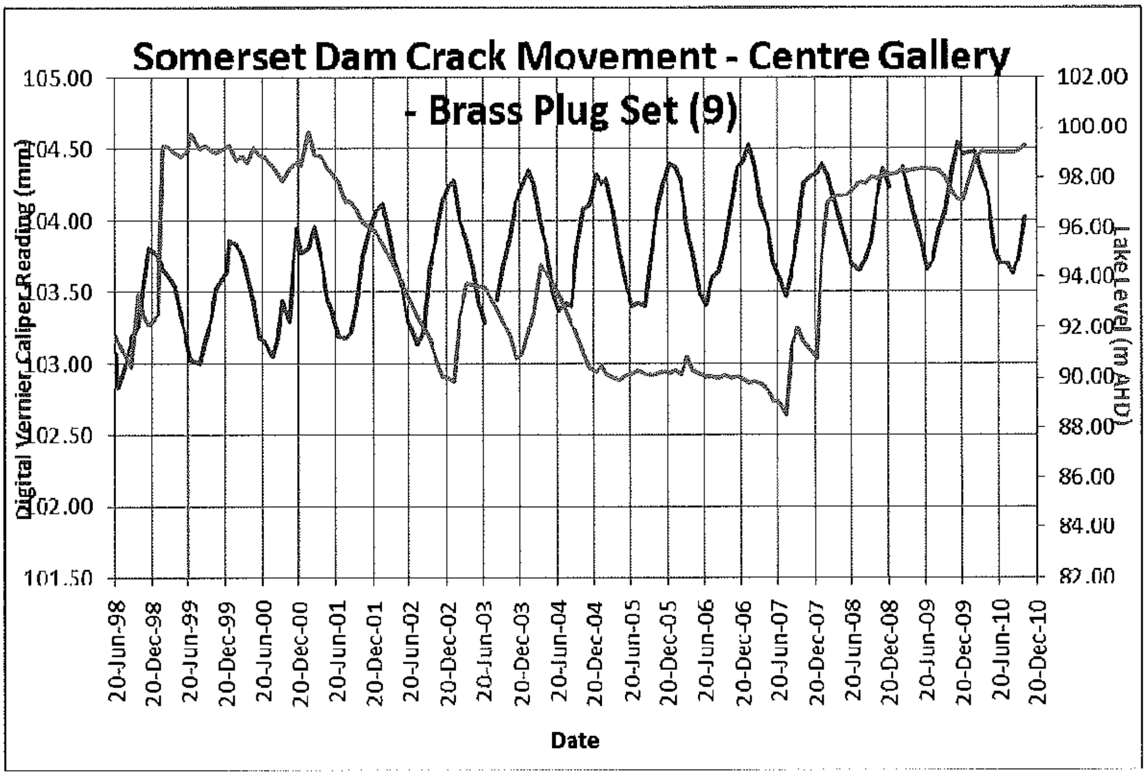
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (9)
Digital Vernier Caliper Reading (mm)
105.00
104.50
104.00
103.50
103.00
102.50
102.00
101.50
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
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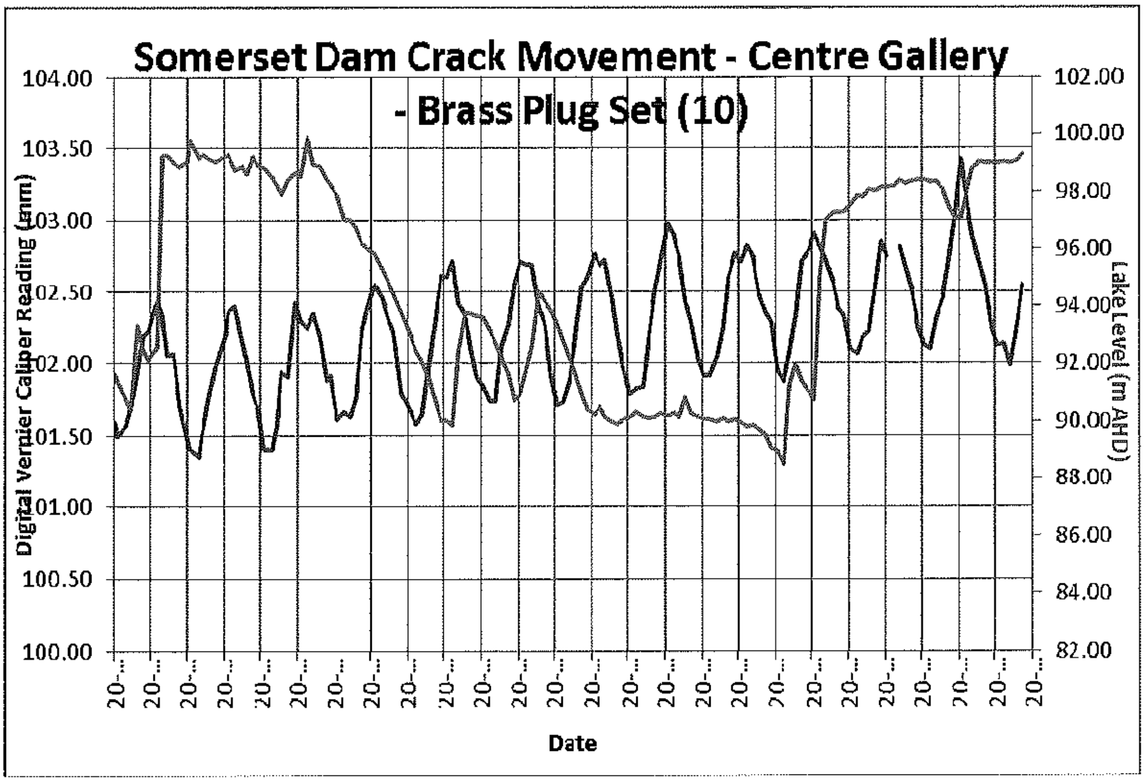
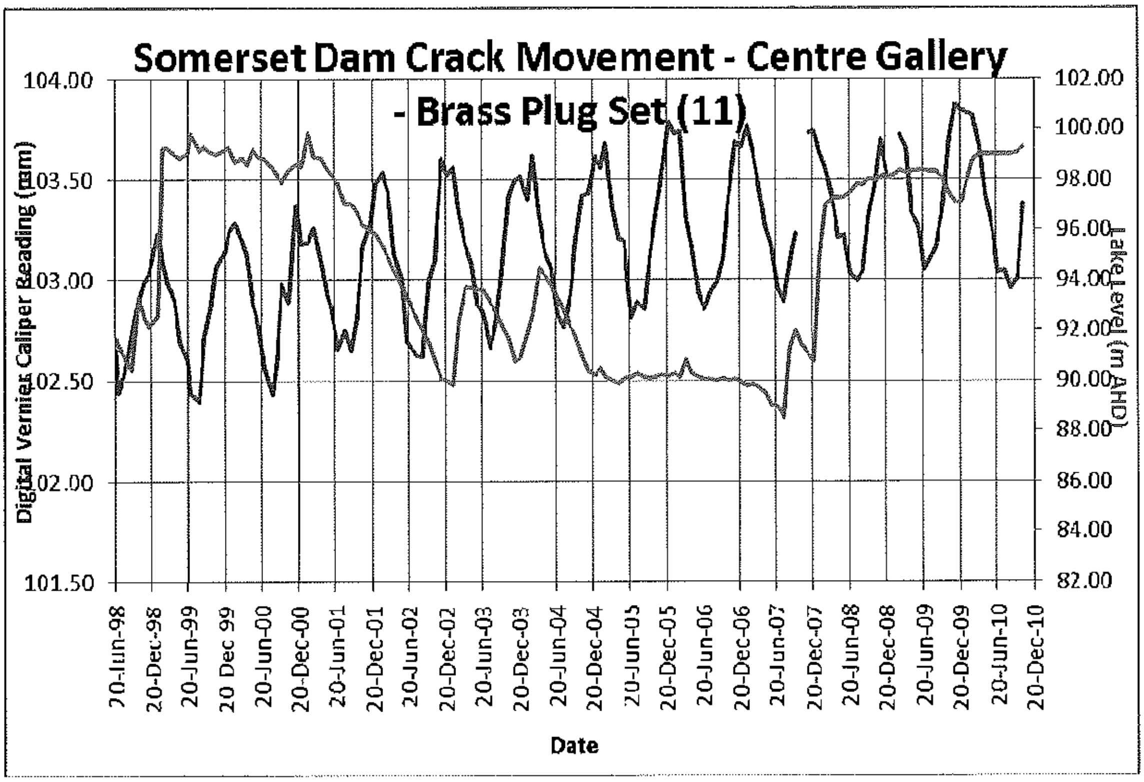
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (10)
Digital Vernier Caliper Reading (mm)
104.00
103.50
103.00
102.50
102.00
101.50
101.00
100.50
100.00
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
Date



Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (11)
Digital Vernier Caliper Reading (mm)
104.00
103.50
103.00
102.50
102.00
101.50
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
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20-Dec-06
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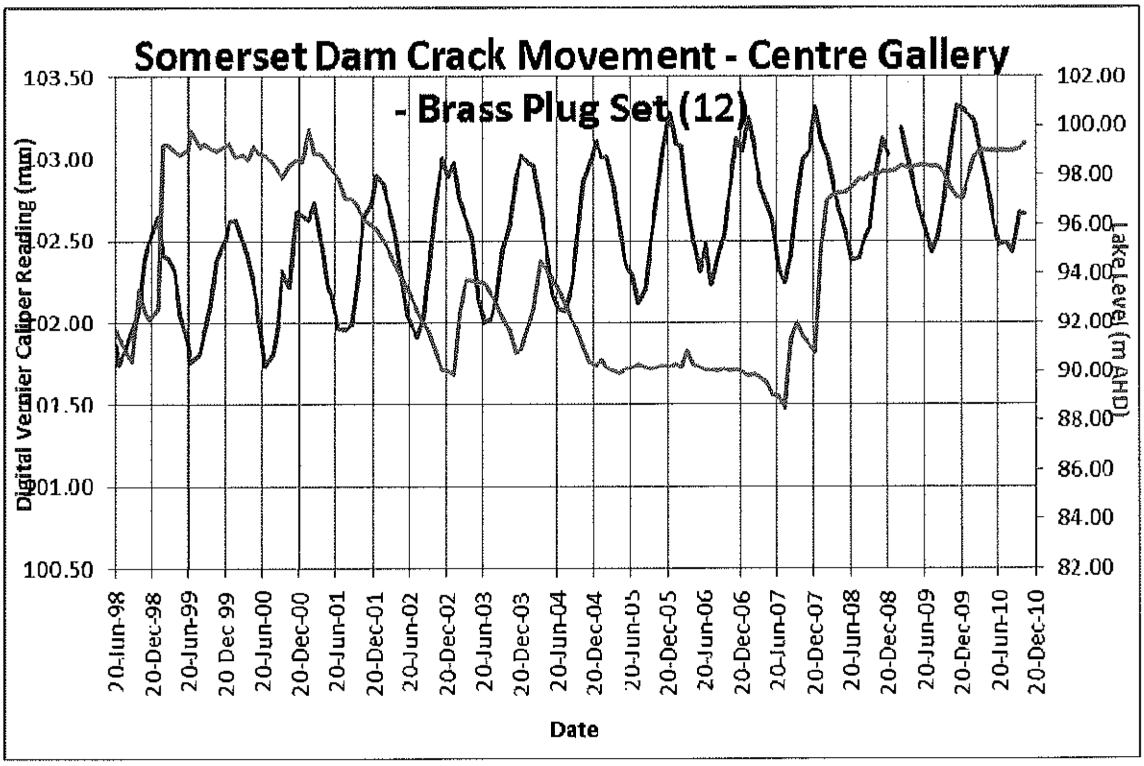
20-Jun-10
20-Dec-10
Date

Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (12)
Digital Vernier Caliper Reading (mm)
103.50
103.00
102.50
102.00
101.50
101.00
100.50
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
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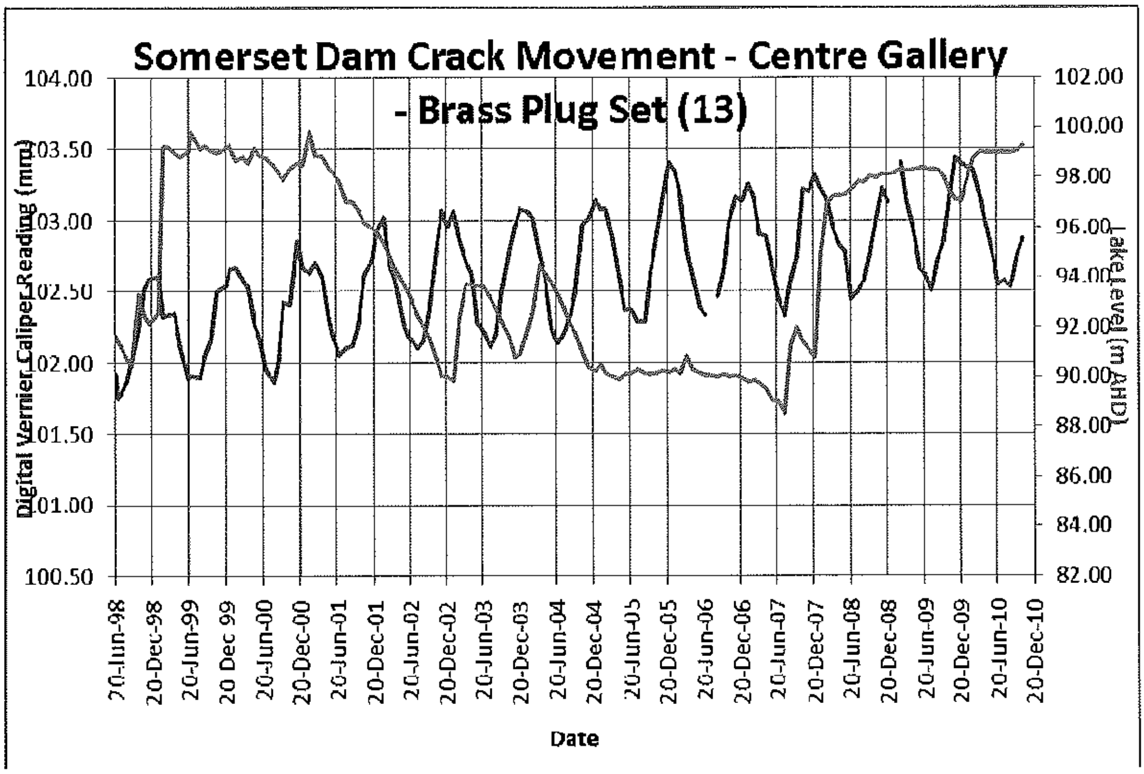
20-Jun-10
20-Dec-10
Date



Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (13)
Digital Vernier Caliper Reading (mm)
104.00
103.50
103.00
102.50
102.00
101.50
101.00
100.50
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
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20-Jun-05
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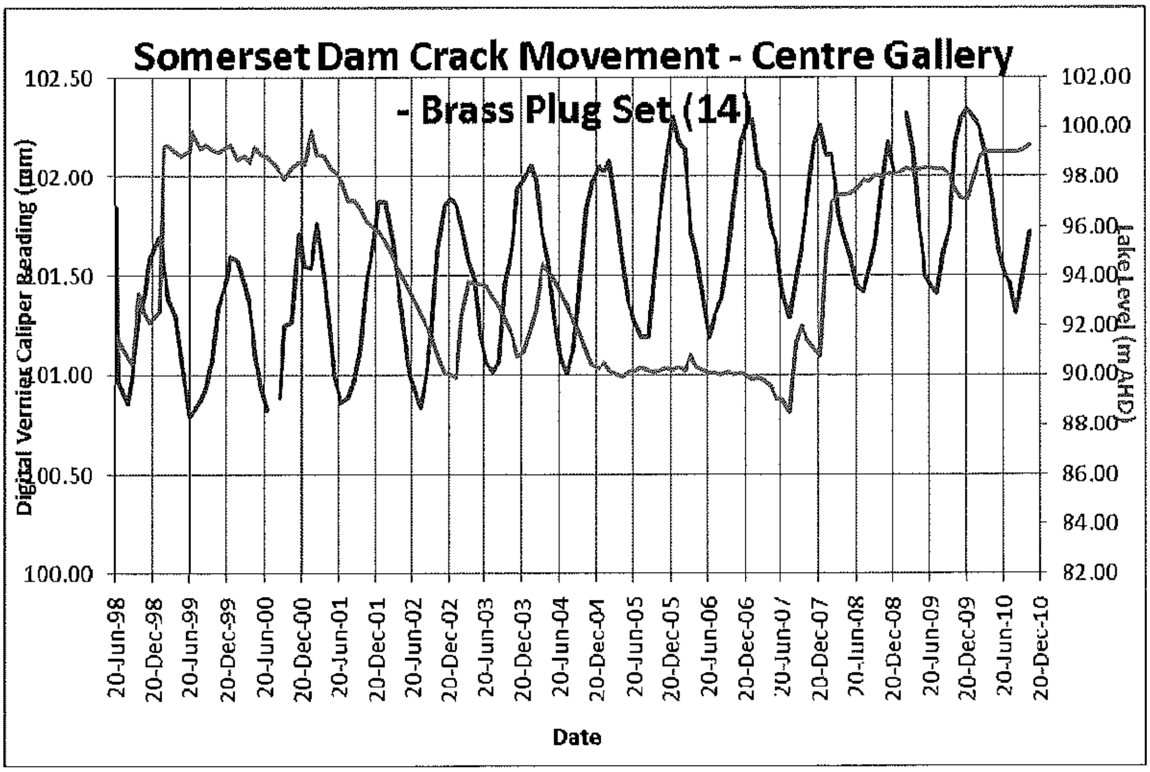
20-Jun-06
20-Dec-06
20-Jun-07
20-Dec-07
20-Jun-08
20-Dec-08
20-Jun-09
20-Dec-09
20-Jun-10
20-Dec-10
Date

Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (14)
Digital Vernier Caliper Reading (mm)
102.50
102.00
101.50
101.00
100.50
100.00
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
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20-Jun-06
20-Dec-06
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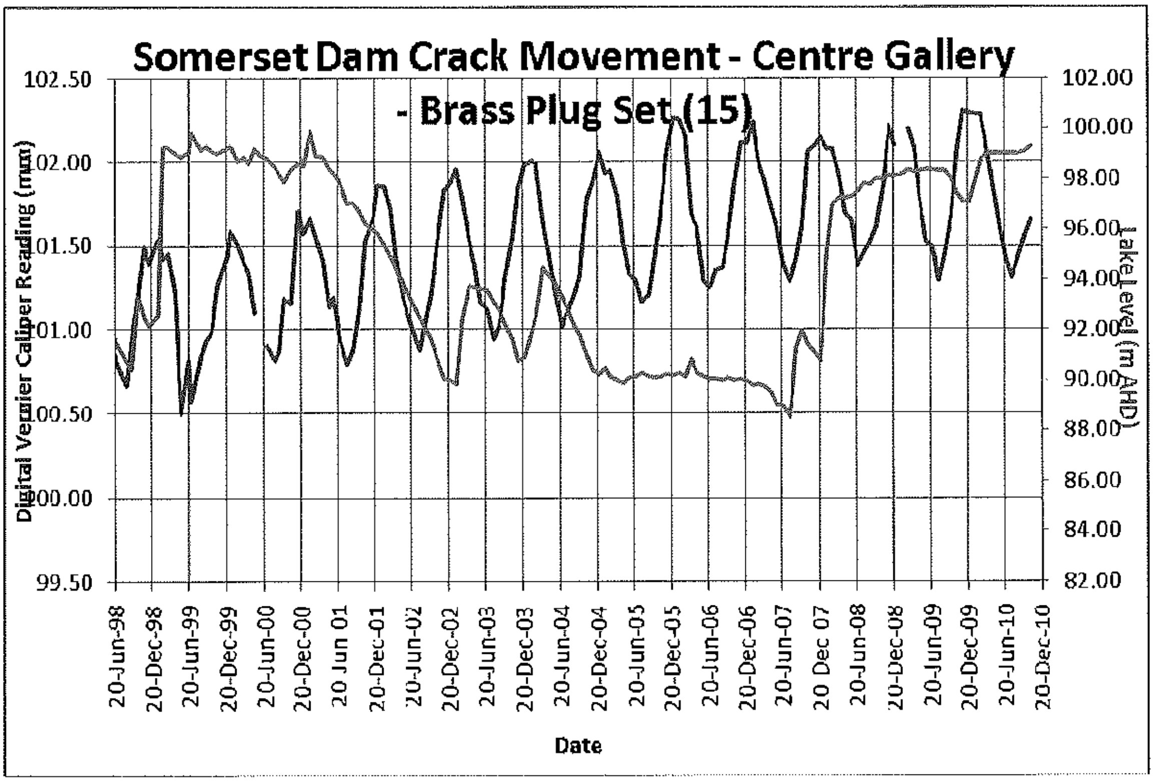
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (15)
Digital Vernier Caliper Reading (mm)
102.50
102.00
101.50
101.00
100.50
100.00
99.50
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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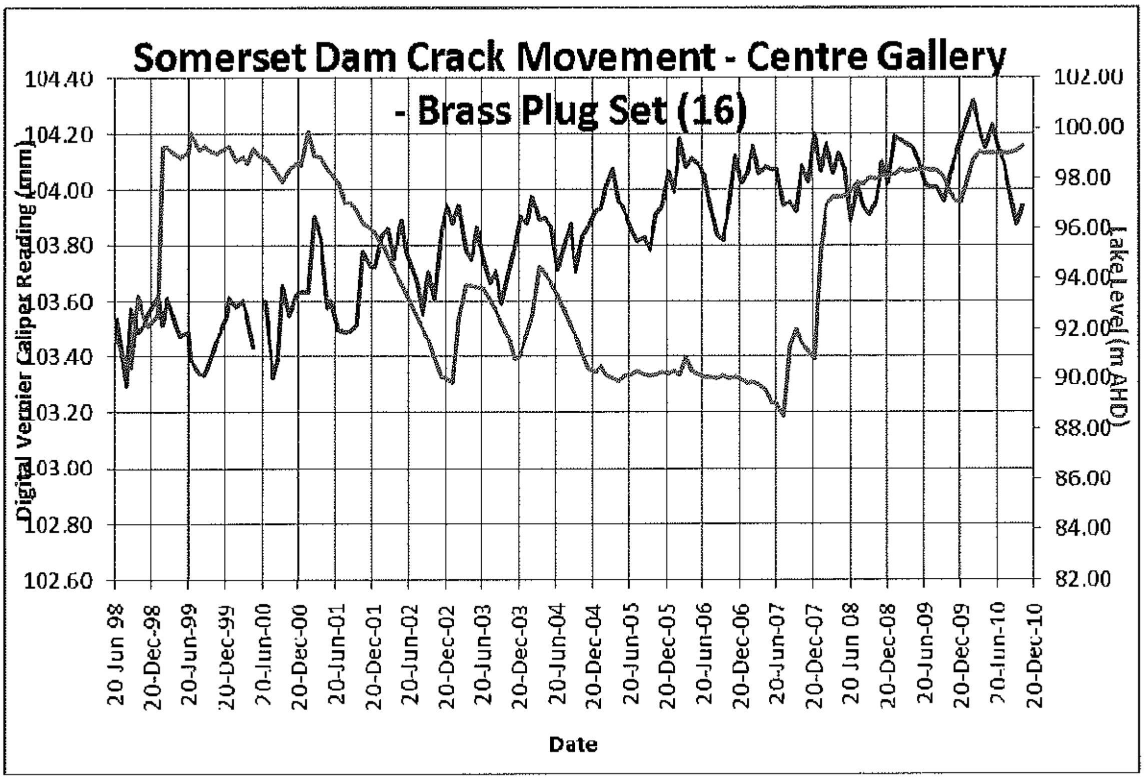
20-Jun-08
20-Dec-08
20-Jun-09
20-Dec-09
20-Jun-10
20-Dec-10
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (16)
Digital Vernier Caliper Reading (mm)
104.40
104.20
104.00
103.80
103.60
103.40
103.20
103.00
102.80
102.60
Lake Level (m AHD)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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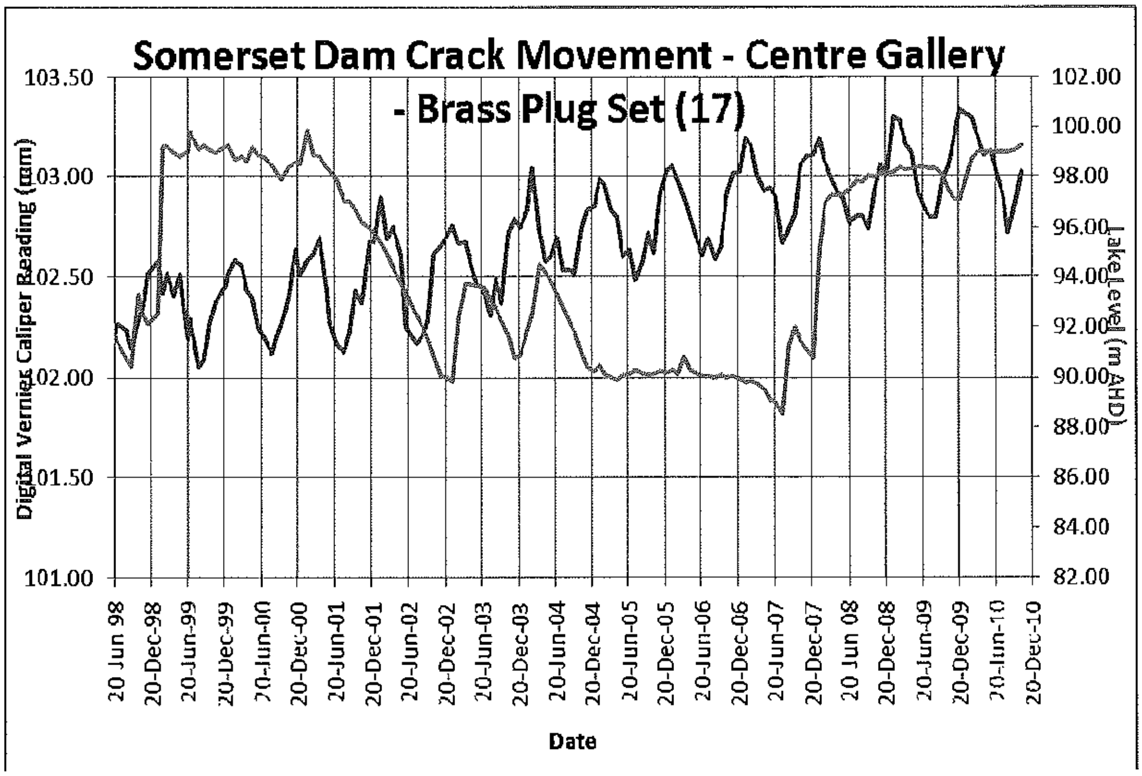
20-Jun-09
20-Dec-09
20-Jun-10
20-Dec-10
Date



Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (17)
103.50
103.00
102.50
102.00
101.50
101.00
Digital Vernier Caliper Reading (mm)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
Lake Level (m AHD)
20 Jun 98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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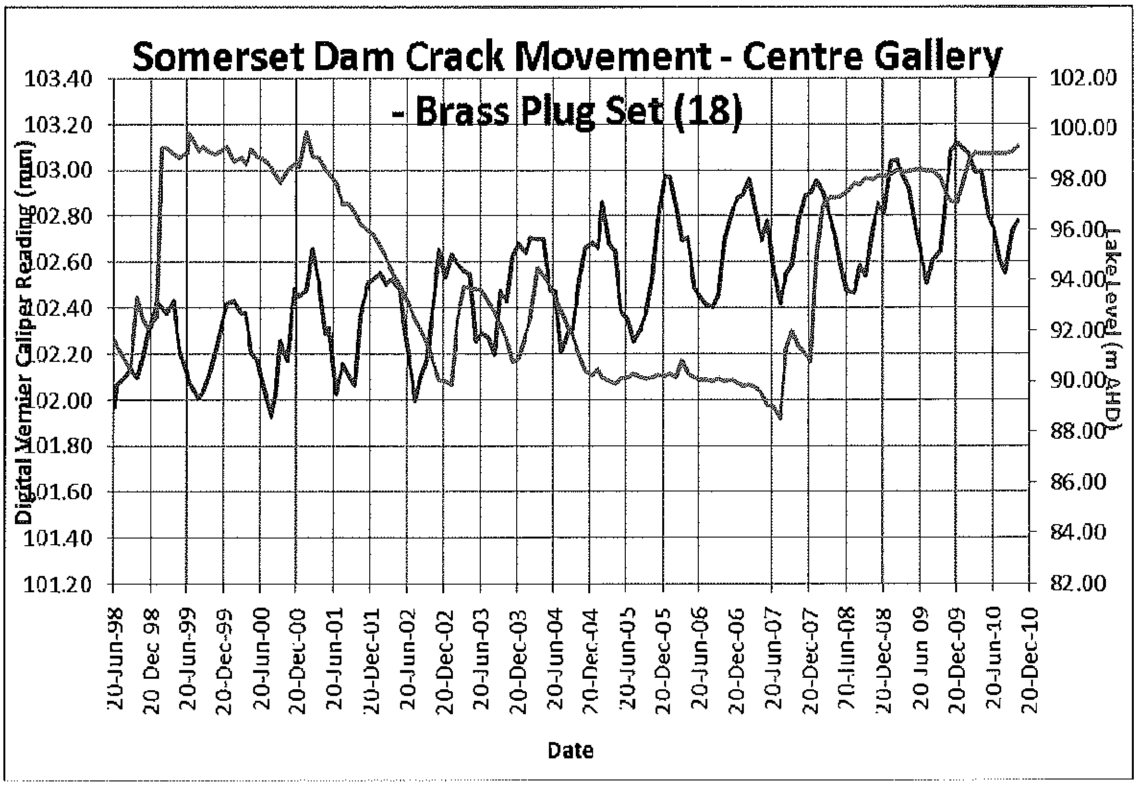
20-Jun-09
20-Dec-09
20-Jun-10
20-Dec-10
Date

Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (18)
103.40
103.20
103.00
102.80
102.60
102.40
102.20
102.00
101.80
101.60
101.40
101.20
Digital Vernier Caliper Reading (mm)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
Lake Level (m AHD)
20-Jun-98
20 Dec 98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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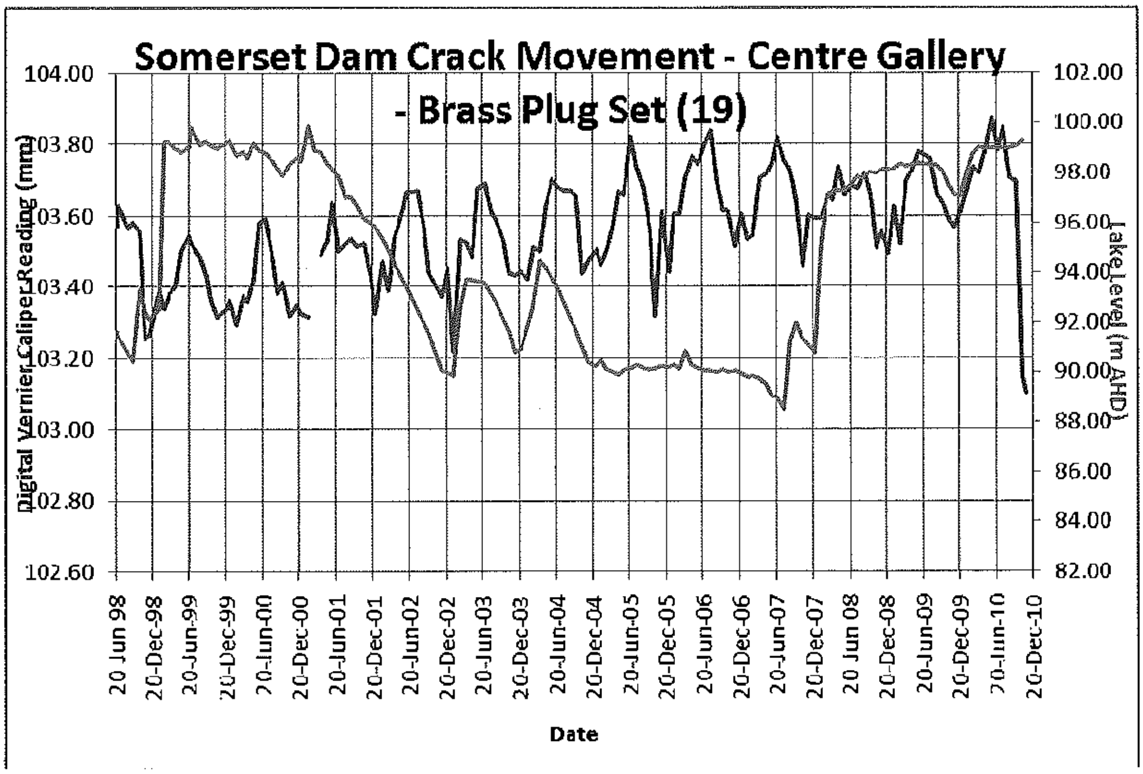
20-Jun-08
20-Dec-08
20 Jun 09
20-Dec-09
20-Jun-10
20-Dec-10
Date



Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (19)
104.00
103.80
103.60
103.40
103.20
103.00
102.80
102.60
Digital Vernier Caliper Reading (mm)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
Lake Level (m AHD)
20 Jun 98
20-Dec-98
20-Jun-99
20-Dec-99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
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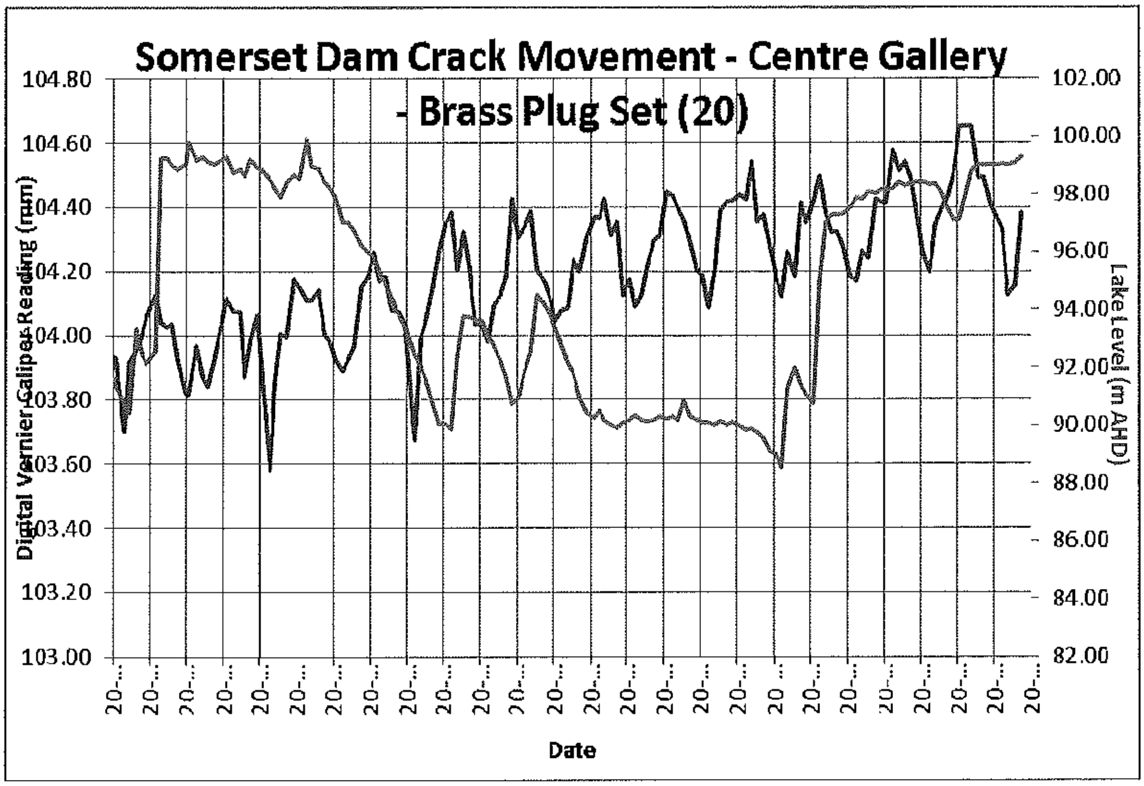
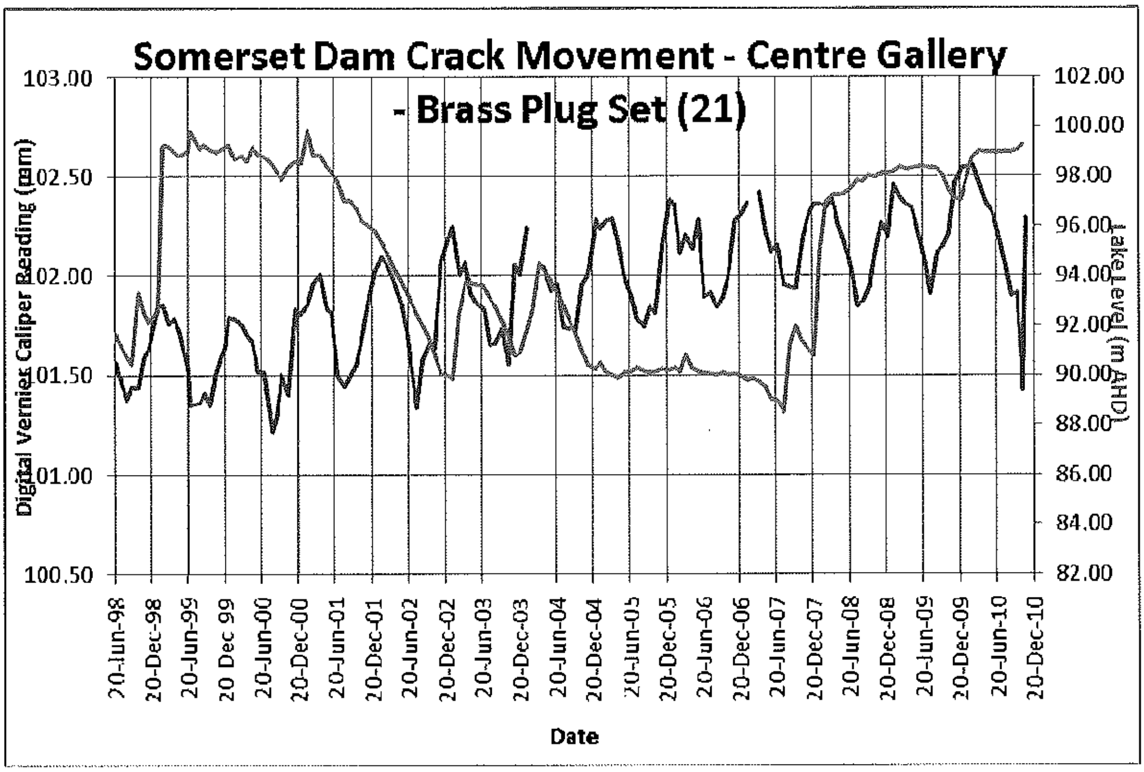
20-Dec-09
20-Jun-10
20-Dec-10
Date

Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (20)
104.80
104.60
104.40
104.20
104.00
103.80
103.60
103.40
103.20
103.00
Digital Vernier Caliper Reading (mm)
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
Lake Level (m AHD)
Date


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (21)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
Date
103.00
102.50
102.00
101.50
101.00
100.50
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20 Dec 99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
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20-Jun-06
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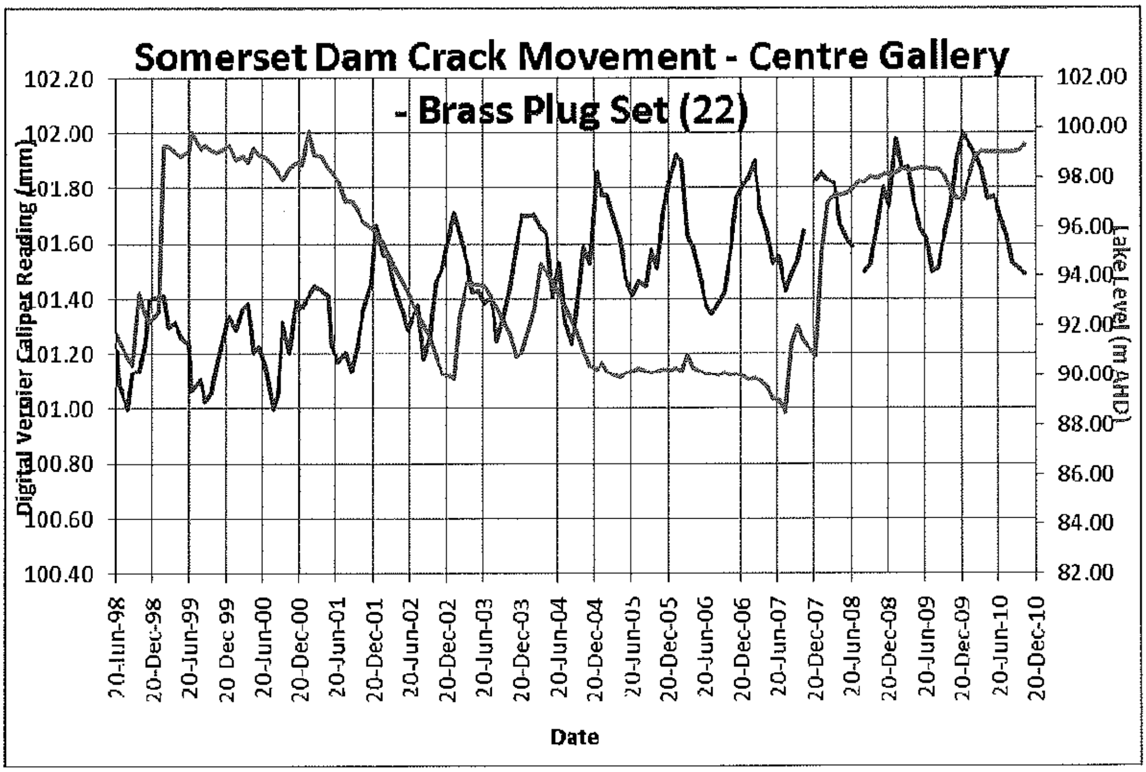
20-Jun-10
20-Dec-10


Somerset Dam Crack Movement - Centre Gallery - Brass Plug Set (22)
Digital Vernier Caliper Reading (mm)
Lake Level (m AHD)
Date
102.20
102.00
101.80
101.60
101.40
101.20
101.00
100.80
100.60
100.40
102.00
100.00
98.00
96.00
94.00
92.00
90.00
88.00
86.00
84.00
82.00
20-Jun-98
20-Dec-98
20-Jun-99
20 Dec 99
20-Jun-00
20-Dec-00
20-Jun-01
20-Dec-01
20-Jun-02
20-Dec-02
20-Jun-03
20-Dec-03
20-Jun-04
20-Dec-04
20-Jun-05
20-Dec-05
20-Jun-06
20-Dec-06
20-Jun-07
20-Dec-07
20-Jun-08
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20-Jun-09
20-Dec-09
20-Jun-10
20-Dec-10