THE WELLINGTON REGIONAL COUNCIL

MEMORANDUM

14 February 1989
File: R/8/1/1 (ajw)

TO: B L Chalmers, Manager, Parks and Recreation

FROM: R P Jackson

BIRCHVILLE DAM INVESTIGATION

The report from the consulting engineers, Tonkin and Taylor Limited, has been received and Technical Services have reported on the course of action. I concur with the recommendations made by Technical Services.

Ross Jackson
R P JACKSON
Project Liaison Officer

N.B. Future action should be discussed at our next fortnightly meeting.

Publicity
Openning day ?

A McCarthy
File: N/3/30/5
AMcC:scr R/8/1/1

COPY
14 FEB 1989

14 February 1989

COPY

The Manager
Tonkin and Taylor Ltd
PO Box 12-152
WELLINGTON

ATTENTION: Mr D J E Malan

Dear Sir

BIRCHVILLE DAM INVESTIGATION - STAGE I

Thank you for your report on the above and the additional information supplied by Mr Alan Pickens on 2 February 1989.

While the report does not entirely fulfil the brief in terms of the analyses carried out, there is sufficient information presented to substantiate your conclusions that the risk of failure is low, and the hazard potential is low.

Accordingly, I have advised the Parks and Recreation Department, who administer the structure, that further investigation work will not be required.

Thank you for your assistance.

Yours faithfully

N R GILLON
for Manager Technical Services

1) Ross Jackson

2) File.

TONKIN & TAYLOR LTD. CONSULTING ENGINEERS, SURVEYORS, PLANNERS
P O BOX 12 152 WELLINGTON NEW ZEALAND
101 MOLESWORTH STREET MOLESWORTH HOUSE WGTN
TELEPHONE (04) 726 997 FAX (04) 733 814

File N/3/30/5

Our Ref: W1143/8925
26 January 1989

McCarthy 1/2

cc. R/8/1/1

COPY

The General Manager
Wellington Regional Council
P.O. Box 11-646
WELLINGTON

Attention: Mr A. McCarthy

Dear Mr McCarthy,

BIRCHVILLE DAM INVESTIGATION

We are pleased to forward six copies of our report entitled "Birchville Dam - Stability Review - Stage 1" ref. 8925 and dated January 1989. The report has been prepared in accordance with your letter of instruction to proceed ref. N/3/30/5 dated 28 November 1988.

In essence we have concluded that the dam is in sound condition. The only area of doubt which may lead to consideration of a Stage 2 study lies in the left abutment stability where insufficient data is available for a full evaluation of stresses applied to the abutment rock.

We believe we have executed the detailed requirements of the brief for the Stability Review - Stage 1 as set out in the enclosure with your letter ref. N3/30/5 dated 3 November 1988. A draft of our report has been reviewed by Henry Kennedy, who also visited the site in November 1988, and he has indicated that he was in agreement with the findings and that the report deals with the issues at a level appropriate for the Stage 1 Investigation.

We will be happy to discuss any aspects of our report with you and are also available to discuss and execute Stage 2 of the investigation should you decide to proceed with further work.

Yours faithfully,
TONKIN & TAYLOR LTD

D.J.E. Malan

DJEM:RMK:KM

Encl.

THE WELLINGTON REGIONAL COUNCIL

FILE COPY

N/3/30/5
1/2/89

cc R/8/1/1

MEMORANDUM

TO: The Manager,
Parks & Recreation,
Wellington Regional Council

FROM: A.J.McCarthy,
Senior Engineer
Technical Services.

Birchville Dam Investigation-Stage 1

Our Consulting Engineers, Tonkin & Taylor Ltd have completed this investigation and submitted their report. Two copies are enclosed for your information.

Two main findings are presented:-

1) The risk of failure is very low.
2) If failure did occur, then the potential for damage is very low.

The first conclusion is based on an analysis of the dam when full of water and subjected to an earthquake with an estimated 100 year return period; i.e. an earthquake estimated to occur once every 100 years on average. The consultants have estimated this event to have a magnitude of approximately 6.5 to 7 (Richter Scale) and to generate ground accelerations of 0.33g. For comparison the recent Edgecombe earthquake had a magnitude 5.5 to 6, but because of the ground conditions, ground accelerations were about the same.

Under this loading, stresses in the dam remain low (1.8 MPa compared with an estimated concrete strength of 20 MPa), but there is some concern regarding the adequacy of the left abutment. Rock shear stresses in this abutment are estimated to peak at 100 kPa. Rock deformation of the order of 1mm would be required to generate this resistance. Abutment deformation of approximately 4mm can be accomodated before (tensile) stresses in the dam reach unacceptable levels.

While there is insufficient information available to accurately quantify factors of safety, it can be confidently stated that the risk of damage to the structure when subjected to a 100 year earthquake is very low.

The report also considers an M.C.E. (maximum credible earthquake) and estimates the magnitude of this to be 7.5, the ground acceleration 0.7g and the return period 560 years. Some dislocation of the left abutment and cracking of the dam is likely under these conditions.

noted MT

BUP 1.2.2004
RECREATION MANAGER

Since a categoric assurance that the dam will never be damaged cannot be given, a brief examination of the consequences of failure has been carried out.

The consultants consider that only moderate flooding is likely to occur following a dam break, with downstream water depths of the order of 1 metre. While some damage to the bush below the dam would be inevitable, they conclude "-- There are no houses built at a low level which would appear to be at risk were a surge to occur.--"

The study just completed was termed "Stage 1" in the expectation that further work to refine some aspects would be required in order to confidently predict the outcome of extreme events.

However, the combination of low risk and low hazard potential identified by Tonkin & Taylor indicates that further work is not required.

The following actions are recommended:-

1) Publication of the findings of the report, particularly through local Upper Hutt newspapers and in the context of park and track development.

2) Distribution of a circular letter to local property owners advising them of the investigation and it's findings.

3) Regular (say 5 yearly) engineering inspections to monitor the general condition of the structure and its abutments.

4) Regular (say every 15 years) reviews of this report by specialist consultants, to ensure that any deterioration, particularly of the left abutment, does not go unnoticed.

Prepared by:- A.J.McCarthy
Senior Engineer,
Technical Services.

Approved by:- N.R.Gillon 8/2/89
Design Engineer,
Technical Services.

Alan Pickens

n/3/30/5

100yr EQ.

Probably Wairarapa fault.

6.5 → 7 — 30km away

1855 West Wairarapa

Similar acceleration to Edgecombe.

Edgcombe 5.5 – 6

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BIRCHVILLE DAM

STABILITY REVIEW

STAGE 1

REF: 8925
JANUARY 1989

PREPARED FOR:

The Wellington Regional Council
P.O. Box 11-646
WELLINGTON

DISTRIBUTION:

The Wellington Regional Council - 6 copies
Tonkin & Taylor Ltd (file) - 2 copies

CONTENTS

BIRCHVILLE DAM

STABILITY REVIEW

STAGE 1

R E P O R T

1. BACKGROUND AND BRIEF

The Birchville Dam is a concrete arch dam located close to Birchville on a tributary of the Hutt River just downstream of the confluence with the Akatarawa River. The dam was built in 1930 to supply or store water for Upper Hutt but was abandoned as a supply source some time ago. It is approximately 15 m high above lowest foundation level and has a crest length of some 46 metres.

The Wellington Regional Council wishes to evaluate the stability of the dam under extreme conditions. Council proposed a two-staged investigation with the first stage comprising an initial evaluation of issues using non-destructive investigation methods, with the scope of any second stage being dependent on the findings of the first stage. Tonkin and Taylor submitted a proposal for the first stage of the investigation on November 14 1988 and was instruction to proceed in accordance with the submission by letter dated November 28. This report covers the first stage brief which addresses:

- concrete quality
- engineering geology
- floods and flood passage
- seismic forces
- structural stability
- foundation stability
- damage potential
- possible further studies

2. FIELD INSPECTIONS AND RESULTS

2.1 CONCRETE CONDITION

Following an initial inspection by senior engineers, Schmidt hammer tests and sonic velocity measurements were made on the dam with the water level drawn down approximately 2.6 m to permit velocity measurement through the dam. Figure 1 shows the sonic velocity test positions. Direct transmission tests face to face were very time consuming because of access difficulties at the upstream face (an inflatable boat was carried in) and hence direct testing was limited to points 1 and 2. The other tests using indirect or transverse transmission indicate properties at shallow depth at the back face of the dam. Schmidt hammer readings were taken at the same locations as the sonic tests. As the results tabled below illustrate the concrete appears to be of consistent quality.

POINT	VELOCITY (km/s)	SCHMIDT READING (N) Average	Std Deviation
1	3.95	24	6.3
2	2.45	23	3.7
3	3.0	27	5.2
4	2.94	22	5.6
5	2.94	17	5.4
6	2.52	21	5.6
7	2.46	19/17	5.3/3.4
8	3.03	-	-
9	2.97	22/20	4.3/4.3
10	2.92	21	4.7
11	1.97	22	5.2
12	2.78		
13	2.84		
14	2.41		
15	3.13		
16	2.90		
17	3.01		
18	2.34	18	4.6

Estimates of strength and elastic properties of the concrete can be determined from the Schmidt Hammer tests and the ultrasonic velocity measurements. Correlation charts for the Schmidt Hammer tests indicate concrete strengths averaging 15-20 MPa but the strength is likely to be conservative as a result of erosion and carbonation of the near surface concrete within the zone of influence of the hammer. Young's modulus can be obtained from the longitudinal velocity (V_e) measurements by applying the formula

$$E = \rho V_e^2 \frac{(1+\nu)(1-2\nu)}{(1-\nu)}$$

where ρ = density (assumed 2400 kg/m^3)
ν = poissons ratio (assumed 0.2)

Results indicate a modulus of 10-20 GPa which is consistent with concrete with strength in the range given by the Schmidt Hammer tests.

The sediment level against the dam face was also measured and found to be 6.3 m below normal top water level.

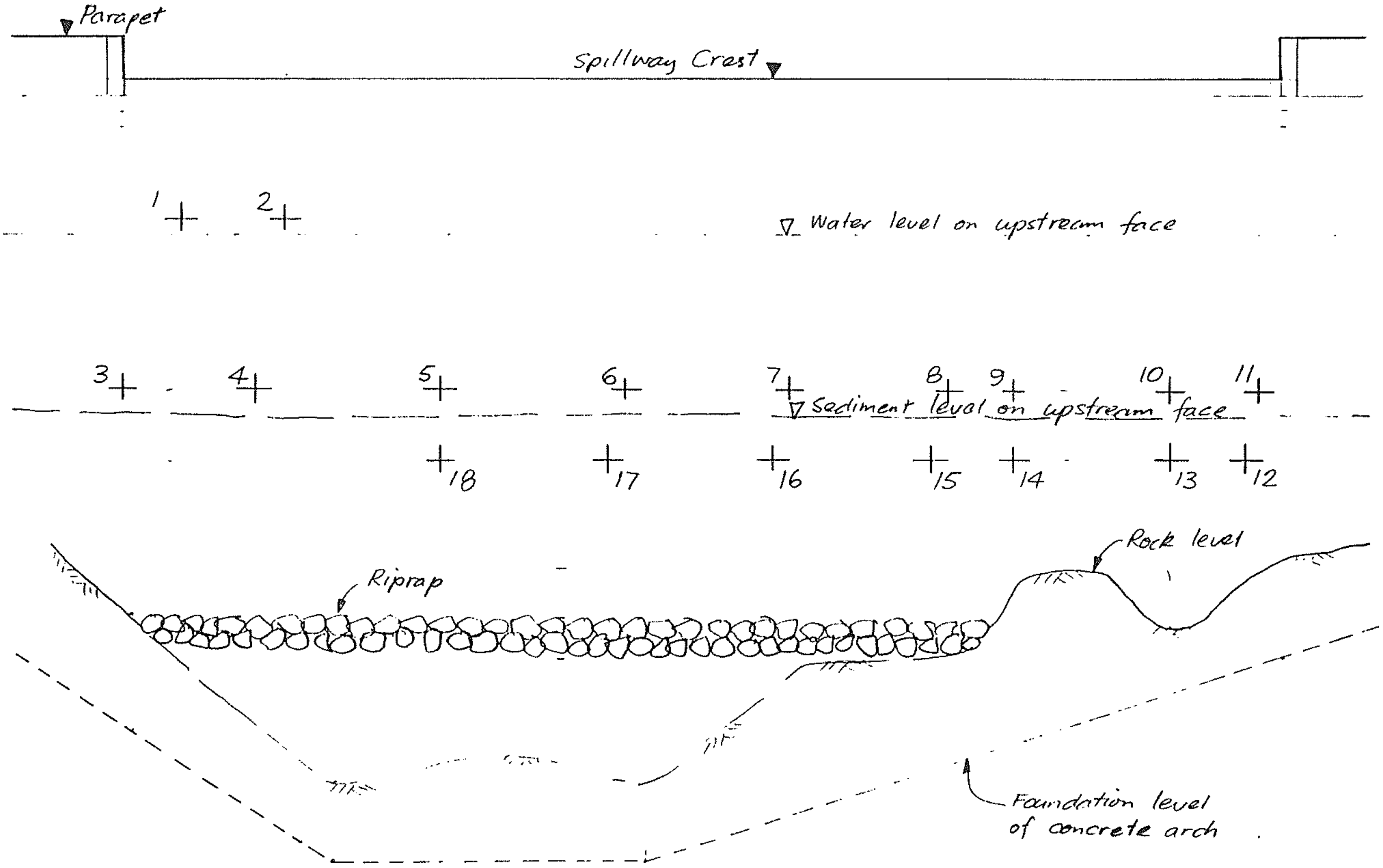


Figure 1

BIRCHVILLE DAM
Elevation of Downstream Face showing Test Positions

Scale 1:100

2.2 GEOLOGY

Our Engineering Geologist inspected the abutments on December 7 1988 when he logged the rockmass exposed in the vicinity of the abutments and undertook a defect survey. At the time of the inspection the reservoir was lowered by some 2 metres exposing some rock outcrops on the immediate upstream face of the abutments. There is no rock exposed in the central section of the dam abutments.

The rockmass exposed in the abutments consists of unweathered to slightly weathered close space jointed indurated hard strong fine sandstone (greywacke sandstone) with minor interbedded moderately hard to hard moderately strong to strong siltstone (argillite). Defects within the rock mass consist of a persistent, well-developed parting parallel to bedding which has a NE to SW trend and several joint sets also well developed, which have NW to SE, and N to S trends. Shattering of the siltstone rock mass is common.

In general greywacke sandstone is not susceptible to weathering over a period of 100 years however, some very minor fretting of the argillaceous beds may be expected.

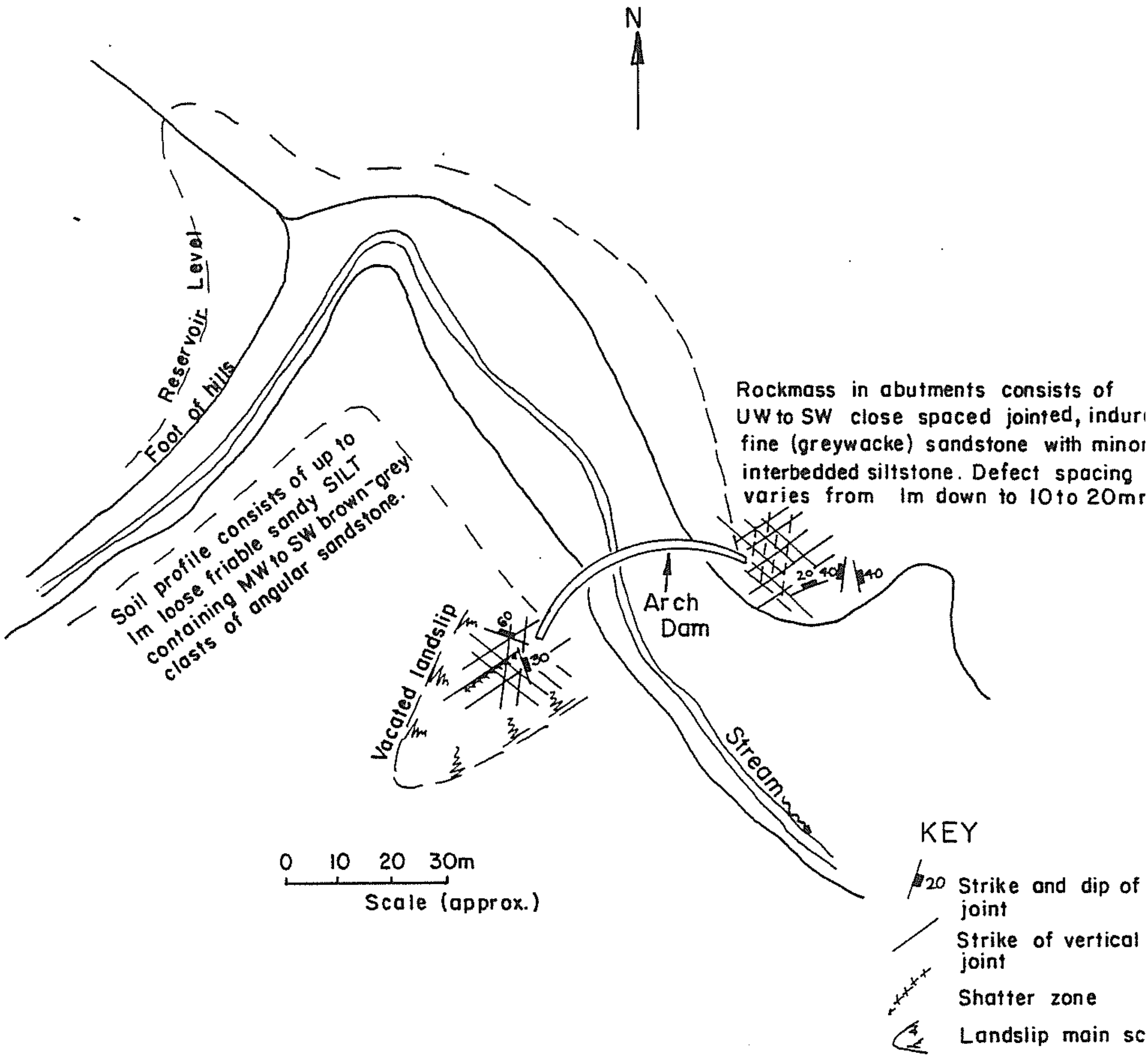
The topography above the right abutment has the physical expression of a vacated landslide controlled by breakage and sliding along pre-exsisting defect surfaces. On the left abutment a steep sided re-entrant controlled by preferential erosion along close spaced defects occurs immediately downstream of the dam abutment. Defects exposed in the re-entrant are continuous with an open aperture and shattered appearance.

Figure 2 summarises these features.

3. FLOODS AND FLOOD PASSAGE

Design flood peaks for the 3.75 km^2 catchment are assessed at 20 $m^3/s \pm 20\%$ for the 100 year flood and 45 $m^3/s \pm 20\%$ for the P.M.F. based on the Rational and TM61 methods and prior experience in the region. On routing the upper limit flood peaks past the dam, our calculations show that the 100 year flood would just be contained within the spillway section and under PMF conditions the whole crest would be overtopped by approximately 300 mm. We therefore conclude that extreme floods would impact on the dam itself.

The question arises as to the possibility of downstream erosion affecting stability in an extreme event. Given that the rock in the valley floor is almost certainly unweathered and the foundations are embedded 1.25 m, there are several metres depth of armour rock of reasonable size which shows no evidence of scouring and the dam must already have passed large floods (e.g. December 1976) downstream erosion is not considered to be a risk.



BIRCHVILLE DAM — Geological Sketch Plan Figure 2

4. SEISMICITY

The site is located in an area of high seismicity with the most notable seismotectonic feature being the Wellington Fault located approximately 2 km east of the dam. This fault is considered to be capable of generating up to magnitude 7.5 earthquakes. It is assessed as having an average fault recurrence interval of about 560 years but has not moved for some 300-700 years (Ref.1). Other sources of severe shaking include the subduction zone, Ohariu Fault and the West Wairarapa Fault.

Estimates of ground shaking using the data and methods of references 2 and 3 have been made to assess levels of ground shaking for 50 and 100 year return period earthquakes. Peak ground acceleration for the maximum credible earthquake has been based on a magnitude 7.5 earthquake on the Wellington Fault using the recommendations of reference 4.

Estimated peak ground accelerations are as follows:

50 year	-	0.25 g
100 year	-	0.33 g
M.C.E.	-	0.70 g

Hydrodynamic pressures have been evaluated assuming the dam to be rigid, using the coefficients derived by Westergard (reference 5). For the purposes of the Stage 1 evaluation, the influence of dam flexibility in hydrodynamic interaction has not been taken into account because the available data and simplified approaches of the Stage 1 Study do not justify this level of sophistication.

5. DAM STRUCTURE

Thoroughgoing stress analysis of arch dams is a complex task requiring reliable knowledge of concrete and foundation properties and construction history. The purpose of the Stage 1 study is to determine whether a potential problem exists warranting further evaluation and for that reason simplified analyses have been undertaken to put structural integrity in perspective.

Obviously the structure has withstood static and substantial flood loads not greatly below the PMF flood level without distress and at the maximum credible earthquake level, arguably the possibility of failure may be considered acceptable. We therefore selected the 100 year earthquake case for stress analysis in the first instance.

The approach taken was first to assume arch action only and unyielding abutments on the basis that conservative compressive stresses would be calculated and abutment loads for subsequent evaluation of abutment stability would also be generated. Calculated compressive stresses ranged from 0.7 MPa at the top of the dam to 1.8 MPa at mid height compared with allowable stresses of up to 7.5 MPa based on estimated concrete strengths. No attempt was made to calculate concrete stresses at lower points of the dam because of the greater significance of cantilever action. It was evident from this approach that for unyielding abutments, the structure would not be overstressed.

This simplified approach was then extended by attempting to calculate what deformation of the abutments would be required to generate a maximum tensile stress of 0.75 MPa at centre of the arch. This analysis again assumed load transfer only by horizontal arch action. The abutment deformation thus calculated was 4.4 mm. Shear tests reported by Martin & Millar (reference 6) indicate that the displacement required to develop the corresponding shear resistance in the abutments is expected to be less than 1 mm. Although failure of the dam would not be likely to occur until tensile cracking extended into the body of the dam this figure illustrates the importance of abutment integrity. From inspection work that has been carried out there is no evidence of tensile cracking in the downstream face which would be the case if excessive abutment yielding had occurred.

No attempt has been made to evaluate stresses arising from differential temperature effects. Stresses from temperature effects can arise during hot weather when the exterior concrete surface of the dam heats up due to radiant heat from the sun and the low conductivity of concrete leads to additional compressive stresses on the exterior face of the dam together with tensile stresses in the body of the concrete.

During winter months the reverse may occur due to frosts i.e. the exterior surface of the dam may cool to temperatures lower than the body of water retained. Under these conditions tensile stresses may be generated in the downstream face and additional compression stresses in the centre.

It is not believed that temperature effects are likely to have a significant influence on the stability of the dam because thermal gradients are not expected to be high due to the location of the dam in a sheltered location where the climate is not severe.

Although no calculations were carried out for the normal static load or the 100 year flood load cases neither is expected to generate stresses in excess of those of the earthquake load case which has been considered. The PMF load case may lead to higher stresses but these are unlikely to approach the allowable maximum compression stress.

In conclusion it can be stated that if no yielding of the abutments has occurred there would not appear to be cause for concern over the structural integrity of the dam. If, however, there was evidence of abutment deformation of the order of 1 or 2 mm there may be reason to undertake further evaluation.

The cantilever action of the dam if effective would help the structural stability but present information on the capacity of construction joints to transmit tension and the ability of the buried rock to provide fixity and accommodate horizontal shear. is inadequate to quantify the contribution realistically.

6. FOUNDATION ASPECTS

It is inferred from site observations and general knowledge of the Wellington greywackes that the undisclosed rock in the valley floor is competent and unlikely to be a source of weakness. Generally the rock mass conditions at the abutments are good with the main area of concern arising from the close proximity of the steep sided re-entrant immediately downstream of the left abutment. The defects here trend parallel with the direction of the arch, have an open aperture and have a shattered appearance. The scope of the investigation did not extend to a topographic survey of the abutment but site observations suggest that the effective horizontal extent of support rock is quite limited.

Left abutment support has been considered for the 100 year earthquake case (pure arch action) which gives a variable thrust of up to 2500 kN per metre height of dam. The defects have been assumed parallel to the line of thrust and the effective length for resistance at 10-15 m. A defect shear resistance of less than 100 kPa is required to achieve stability.

The ability to develop this order of shear resistance is dependent on joint roughness and interlock characteristics. While logging at this stage is not in sufficient detail to provide information on roughness characteristics or the possibility of conjugate joint shear displacements, experience elsewhere in greywacke suggests that the roughness should be adequate to achieve 100 kPa resistance. It cannot be said, however, from this simplified evaluation, that there is a comfortable level of security at the left abutment.

Shear tests on greywacke jointed rock specimens in the Wellington area by Martin and Millar (1974) (reference 6), have indicated that the rock exhibits a power law failure envelope with high apparent friction angles at low confining pressures due to the influence of dilation. Initial friction analyses on closed jointed specimens exceeded 60° while residual strengths of over 45° were obtained. Horizontal displacements exceeding 2 mm were required on the specimens to fully develop the peak shear strength while the 100 kPa resistance is expected to be developed at joint displacements of less than 1 mm. In the field the influence of larger scale undulations of the joints would be expected to increase the displacements required to exceed the peak shear strength of the rock mass.

Other foundation issues include the possibility of landslip into the reservoir from above the right abutment as a result of either a high rainfall event or a severe seismic event.

7. DAMAGE POTENTIAL

Irrespective of any failure mechanism, the damage potential needs to be put into perspective. We estimate that the dam holds about 20,000 m^3 of water (and sediment) at normal operating level which is a modest volume. While the valley is not wide (roughly 60 m wide floor) or long (roughly 900 m from the dam to the Hutt River), if a failure were to occur so that the reservoir emptied in 15 minutes the resulting flood would be roughly equivalent to the 100 year return period flood with an average flow depth of 0.5 to 1 m. There are no houses built at a low level which would appear to be at risk were a surge to occur.

Dam failure during an extreme flood would probably not be noticeable and under these conditions it is unlikely that members of the public would be in the floodway. Failure under earthquake is probably of greater concern but although a possible weakness has been identified at the left abutment it is difficult to envisage a total collapse of the dam producing an 'instantaneous' surge.

We perceive the overall hazard potential being low and the risk of failure also being low.

8. POTENTIAL IMPROVEMENTS AND FURTHER STUDIES

From this stage 1 evaluation and taking into consideration the perceived low hazard potential of the dam, the only area that could warrant improvement is the left abutment. We expect that any improvement would take the form of stressed anchors installed at right angles to the defects at the left abutment to improve abutment security.

One could take a decision now to put this work in hand on a somewhat arbitrary basis and put money that might otherwise go into further studies towards actual works. Alternatively, considering the hazard potential, the uncertainties might be accepted and the matter be left to rest.

If a more informed assessment of the left abutment were desired the following stage 2 studies are recommended:

(i) topographic survey of the left abutment

(ii) more comprehensive geological evaluation of the left abutment. This would involve some clearing and detailed engineering geological logging of the exposed rock to determine more exactly the kinematically possible failure modes of the rock mass with respect to the re-entrant topography along the thrust line at the left abutment and to allow assessment of the joint strength including the effect of roughness and continuity of these features.

(iii) more detailed analysis of forces and resistance along potential failure planes

(iv) evaluation of any additional support requirements, e.g. rock anchors.

9. CONCLUSIONS

The investigations carried out in Stage 1 of the study lead to the conclusion that the concrete in the dam is sound. Flood flows and seismic forces have been assessed and their affect on the dam structure and foundation considered. There is no evidence to suggest over-stressing of the dam or the abutments has, or is likely to take place. However, it has not been possible to establish that the left abutment has sufficient joint strength for all possible load conditions. This could only be determined by further topographical and geotechnical study. The overall hazard potential is considered to be low and the risk of failure is also considered to be low.

TONKIN & TAYLOR LTD
Consulting Civil Engineers
& Registered Surveyors

D J E Malan

Report prepared by G.A. Pickens

GAP:RMK:MP

246:7

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