THE UNIVERSITY OF NEW SOUTH WALES

WATER RESEARCH LABORATORY

REPORT No. 28

Hydrologic Investigation of Ash Pond

Wangi Power Station

by

C. G. Coulter

DECEMBER, 1960

The University of New South Wales

WATER RESEARCH LABORATORY

HYDROLOGIC INVESTIGATION
OF
WANGI ASH DAM SPILLWAY

by
C.G.Coulter

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PREFACE

This study forms part of a series of hydraulic investigations undertaken by the Water Research Laboratory of the University of New South Wales at the request of the Electricity Commission of New South Wales. The study was commenced in August 1959 and completed in May 1960.

Throughout the course of the study, close liaison was maintained with the Electricity Commission through engineers on the staff of the Commission's Project Development Section, Mr. N.Lamb and Mr. B.H.Keogh, whose friendly co-operation in the supply of all necessary data is gratefully acknowledged. Internal progress reports of test results were made available to the Commission as data became available.

The study was carried out at the Water Research Laboratory, Manly Vale, N.S.W. by Mr. C.G.Coulter. The Electricity Commission programme is under the direct supervision of Mr. D.N.Foster of the Laboratory Research staff.

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Officer-in-Charge of the Water Research Laboratory.

December, 1960.

SYNOPSIS

This report describes the investigation undertaken to determine the flood discharges for the design of the spillway for the ash dam at the Wangi power station. The method of analysis is described and a summary of the results of the investigations of various proposals is presented.

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FIGURES

TABLES.

1. INTRODUCTION

This report describes hydrologic investigations of the proposed Wangi Ash Dam spillways carried out at the Water Research Laboratory for the Electricity Commission of New South Wales.

At the New South Wales Electricity Commission's Wangi Power Station, ash from the furnaces and precipitators is pumped as a slurry to a disposal area on Crooked Creek which discharges to Lake Macquarie at Myuna Bay (Fig.1). When the power station was first brought into service, a low earth dam (maximum height of 10 ft.) was constructed as a temporary measure for the disposal of ash. The ash settled out in the dam while the pumped water and the runoff from the 900 acres of catchment above the dam discharged to the original stream channel by an adjustable crest spillway.

In June 1959, the storage adjacent to the dam was full, and the Commission proposed to construct a new dam immediately below the temporary one. This new dam was to be 60 feet high (spillway crest at RL.150.0) and was designed to provide storage for 170×10^6 ac. ft. of ash; the estimated total volume to be produced throughout the life of the station.

Flood estimates for the preliminary design of the spillway had been carried out by the Electricity Commission using a form of the rational method.

$$Q = CAp_m$$

where Q = peak discharge in cfs

A = catchment area in acres

C = the runoff coefficient

p_m = mean rate of rainfall of duration equal to the time of concentration and of the desired frequency.

Rainfall intensity-duration-frequency data for Gosford was used to determine p_m and a coefficient of runoff of 95 per cent was assumed. The 1 in 200 year flood was estimated at 600 cfs by this method and the spillway designed accordingly. This calculation neglected the effect of storage in the ash pond reservoir which has an area of 300 acres. The combined reservoir and catchment has an area of 900 acres. Estimates in which the reservoir storage is neglected are too conservative. In discussions between personnel of the Commission and the University, it was therefore decided to check the flood estimates by routing design flood inflow hydrographs through the storage.

The final ash contours were predicted by methods outlined in Water Research Laboratory Report No.27 and spillway storage volumes were computed for the flood routing procedure. During these studies it became apparent that the required ash storage could be obtained with a much lower dam (Spillway crest at RL.135). A number of spillway arrangements for this dam were investigated.

The proposals were still further modified following the success of the experimental bank built on the ash deposits at Tallawarra Power Station. By constructing three low dams (10 ft. in height), each built on the ash contained by the lower one (Fig.2), the necessary ash storage could be obtained at a much lower cost. The technique of multi-stage construction was finally adopted. Hydrologic investigations have been carried out for the first stage dam.

The proposals that have been investigated are shown on Fig.2.

2. METHOD OF ANALYSIS

2.1 General

The method of analysis to determine peak outflow discharge is a trial and error procedure. A trial spillway is first assumed and its behaviour during the design flood events is computed. The process is repeated until a satisfactory arrangement is found. At all times during the investigations when assumptions had to be made, the most conservative was taken. For example it was assumed that water level in the reservoir was at crest level at the start of the design storm and that the design storm occurred when the reservoir was full of ash (at the end of the useful life of the pond). At this stage the spillway storage will be a minimum. The spillway storage is here calculated as the water volume between a horizontal plane at flood height and the spillway crest level.

Steps in the procedure are:-

1. Estimation of the inflow hydrographs. This has been done using a synthetic unit hydrograph, Gosford rainfall data and an assumed coefficient of runoff.

2. Computation of the storage volume .v. elevation relationship from a contour map of the catchment area and from a prediction of the ash contours.

3. For the trial spillway, computation of the spillway rating curve (discharge .v. storage elevation).

4. From 1, 2 and 3, the outflow hydrograph is computed by the Puls method of flood routing.

For storms of a given frequency the storm duration that gives the greatest peak outflow is not known and must therefore be found by trial. This was done by routing the inflow hydrographs for a number of storm durations and selecting the one that produced the greatest spillway discharge.

2.2 Frequency of Surcharge

For schemes in which the required storage was to be provided by a single dam, the recurrence interval for the design flood to fix the spillway capacity was specified by the Commission as 1 in 200 years.

For the multi-stage construction proposal, however, a much more complicated situation exists. For the third and final stage construction, the spillway storage is much greater than that for the second stage, which in turn is much greater than that obtained by the first stage (Fig.6). Thus, for a given design storm inflow hydrograph, the first stage spillway is subjected to a much smaller peak flood discharge after the construction of the latter stages. The 1st stage is expected to provide ash storage for about 3 years output and the second ash storage for about 7 to 9 years output. At the Commission's request, the first stage spillway has been investigated for the 1 in 20 and 1 in 100 year floods.

2.3 Derivation of the Synthetic Unit Hydrograph

The synthetic unit hydorgraph for the catchment was derived by a simplified form of the Clark-Johnstone procedure. (See Appendix E of Ref.1). The procedure was simplified by assuming the area-shape curve was an isosceles triangle. The computed values of the catchment characteristics are as follows:-

Area (A) 1.5 sq. miles
Length of main stream (L) 1.6 miles
Width of catchment (W) 0.94 miles
Stream channel slope factor (S) 85 ft/mile
Overland slope factor (R) 360 ft/mile.

Substitution in Johnstone's empirical equations gave hydrograph parameters C = 2.0 hours and K = 1.75 hours. Completion of the procedure gives the unitgraph shown on Fig.3.

2.4 Storm Rainfalls

Rainfall intensities for various durations and frequencies for Gosford have been computed from data given in the Stormwater Standards Committee Report No.1 (Ref.1). This information is shown plotted on Fig.4.

To obtain storm excess rainfalls, it was assumed that the loss rate was zero so that all storm rainfall appears as runoff.

2.5 Inflow Hydrographs

Inflow hydrographs have been computed from the storm rainfalls and the synthetic unit hydrograph utilising a computer program written for UTECOM, an electronic digital computer at the University of New South Wales. The inflow hydrographs are given in Table I.

2.6 Storage - Elevation Relationships

For a given inflow hydrograph, the peak outflow will be at the end of the life of the dam. When the storage has been filled with deposited ash at this stage some ash extends above spillway level. It was necessary therefore to predict the ash contours when the dam was filled. The procedure developed for predicting the final ash contours has been described in Ref. 2 and will not be detailed here. Figure 5 shows the estimated ash contours of the 1st stage construction when filled.

From the predicted ash contours and the ground contours, the spillway storage for various storage levels has been computed. The results for the three stages of multiple construction are shown on Fig. 6.

2.7 Stage/Discharge Relationships

The discharge over spillways has been computed for various water levels from the formula $Q = C_D B H^{3/2}$

where B is the width of spillway (in feet)

H is the head on the spillway (ft.)

Q is the discharge in cusecs

C_D is the coefficient of discharge for the weir taken at equal to 3.0 (Ref. 3).

For the first stage construction dam, the spillway consists of three bays, two of which are "fuse-plug" spillways, designed to "blow" when overtopped to a depth of 9 inches (Fig. 7). The initial discharge through these sections has been estimated by the formula for the discharge of a surge in a dry bed.

$$\text{e.g.} \quad q = \frac{8}{27} g^{\frac{1}{2}} h^{3/2}$$

where q is the discharge in cusecs per foot of width
g is the acceleration constant
h is the height of water above the bed before failure.

The rating curve for the first stage construction is shown o.. Fig.7.

For this case, two curves are presented. One gives the discharge/stage relationship before failure of the fuse plug sections, the other gives the discharge/stage relation after failure of these sections. In the routing procedure, the assumption is made that the surge generated when the fuse sections fail result in a drop of water level to RL.116.0, after which the "after failure" rating curve applies.

2.8 Flood Routing

Routing of the inflow hydrographs through the storage has been carried out by Puls' method. From continuity considerations

$$\left(\frac{I_1 + I_2}{2}\right) t - \left(\frac{O_1 + O_2}{2}\right) t = S_2 - S_1$$

$$\therefore \quad I_1 + I_2 + \left(\frac{2S_1}{t} - O_1\right) = \left(\frac{2S_2}{t} + O_2\right) \quad \ldots\ldots(1)$$

where I_1 and I_2 are inflows at the beginning and end of a time period t,

O_1 and O_2 are outflows at the times corresponding to I_1 and I_2,

S_1 and S_2 are the storage volumes at times corresponding with I_1 and I_2.

Flood routing is carried out using a graph of $(\frac{2S}{t} + O)$ versus O (Fig.8) prepared for values corresponding to fixed stages. At the start of the flood event, all values on the L.H.S. of equation (1) are known so the R.H.S. can be computed. From the graph of $(\frac{2S}{t} + O)$ versus O and this value, the outflow at the end of the period is read off. This corresponds to the outflow at the beginning of the succeeding period and so the computations proceed.

Table 2 gives an example of the procedure for the 12 hour storm inflow hydrograph on the stage I construction dam.

The results of the flood routing for storms having a frequency of 1 in 100 yrs. and durations of 2, 6 and 12 hours, are given in Fig.9. By drawing an enveloping curve through the peaks of the outflow hydrographs shown in this figure a maximum discharge of 650 cusecs is obtained. This discharge is the estimated 1 in 100 year peak flood flow.

3. SUMMARY OF RESULTS

3.1 Original Proposal

The original proposal (Fig.2a) was designed for an adjustable crest over the spillway made up of three equal sections with a total crest length of 15 ft. This proposal was investigated for behaviour under the 1 in 200 year storms by the method described previously for spillway crest at RL.100 and 150, i.e. early in the life and at the end of the pond. In this first analysis, no allowance was made for ash occupying part of the spillway storage.

Results of this analysis were:-

Spillway Crest Level	Peak Outflow (cfs)	Peak Flood Level.
RL.110.0	470	RL.115.4
RL.150.0	130	RL.152.75

3.2 Proposal No.2

This proposal (Fig.2b) consists of a single dam with spillway at RL.135. A number of spillway arrangements were investigated when subjected to storms of various durations and having a recurrence interval of 1 in 200 years; as follows:-

(a) A single 5 ft. crest length
(b) Two 5 ft. crest lengths
(c) Three 5 ft. crest lengths
(d) Two 5 ft. crest at RL.137 and a third at RL.135

In these studies, allowance was made for the ash deposition cone occupying part of the spillway storage.

The results are as follows:-

Spillway Arrangement	1 Opening	2 Openings	3 Openings	Split Level Spillway
Peak Outflow (cusecs)	205	325	395	300
Peak Flood Level (R.L.	139.1	138.6	138.1	138.8

3.3 Proposal No.3

For Stage 1 of the multi-stage construction (Fig.2C), three arrangements were investigated. The first had two fuse plug sections each 6'6" wide with a fixed crest at RL.113.0. (These were to fail when the stage reached 116.75), an adjustable crest section 7'-0" long, and a 20 ft. section of the dam wall designed to "blow" when the detention level reached RL.118.0. It was found that under the 6 hour, 20 year storm, failure of the dam section would produce an outflow of 1100 cusecs. This was unacceptable.

In the second arrangement studied, the widths of each of the three sections was increased by 1 ft. and the fixed crests in the fuse plug sections were eliminated. It was found that this arrangement did not make the best use of the available spillway storage for flood attenuation. The peak discharges for the 1 in 20 and 1 in 100 year storms being:-

	Peak discharge	Peak flood level
1 in 20 years	680 cusecs	RL.116.9
1 in 100 years	840 cusecs	RL.117.6

The third arrangement, shown in Fig.7 is recommended, the peak discharges for this design being -

	Peak discharge	Peak flood level
1 in 20 years	550 cusecs	RL.117.6
1 in 100 years	650 cusecs	RL.118.0

4. CONCLUSIONS

The method of analysis that has been used to estimate the peak spillway discharges for the Wangi Ash Dam is the only one that can be expected to give reliable results where, as in this case, the storage area makes up a large portion of the catchment area. The weakness of the method lies in the use of synthetic unit hydrographs to estimate the flood hydrographs, and in estimating excess rainfalls from gross rainfalls. Large errors may have been introduced by assuming all the gross rainfall becomes excess rainfall. Such errors would result in conservative designs. The Commission has recently installed water level recorders operating on the 1st stage construction reservoir to provide data from flood events. From the data collected more precise calculations may be prepared before the designs of the upper level spillways are completed.

TABLE I

Wangi Ash Dam Hydrologic Studies – Storm Hydrographs
Hourly Ordinates.

1 in 20 year Storms			1 in 100 year Storms		
6 hrs. Duration	12 hrs. Duration	24 hrs. Duration	6 hrs. Duration	12 hrs. Duration	24 hrs. Duration.
70	40	30	90	55	45
310	200	150	390	250	205
490	320	245	600	400	425
590	385	295	750	480	390
640	420	320	810	520	425
680	440	340	840	550	450
630	450	350	780	565	460
400	455	350	480	570	465
	460	350	270	575	465
	460	355		580	470
	460	350		575	470
	460	355		575	470
	420	355		520	470
	260	350		325	470
	140	350		175	470
	80	355		100	470
	40	350		50	470
	20	355			470
		355			470
		355			470
		355			470
		355			470
		350			470
		350			470
		320			430
		200			265
		110			145
					80
					40

TABLE 2.

Flood Routing Stage I Construction - 12 hr.

1 in 100 yrs. storm.

(I)	$(I_1 + I_2)$	$\left(\frac{2S}{t} - O\right)$	(O)	$\left(\frac{2S + O}{O}\right)$	
0	0		0	0	
55	55	0	20	55	
250	305	15		320	- failure of fuse plug section
400	650	0	440	650	
480	980	0	480	980	
520	1000	20	490	1020	
550	1070	40	500	1110	
565	1115	110	520	1225	
570	1135	185	530	1320	
575	1145	260	540	1405	
580	1155	325	550	1580	
575	1155	480	570	1635	
575	1150	495	580	1645	
520	1095	485	560	1580	
325	845	460	530	1305	
175	500	245	450	745	

REFERENCES

(1) "Australian Rainfall and Runoff" Instn. Engineers Aust. Stormwater Standards Committee - Report No.1.

(2) D.Foster and J.Argue - "First Report on Power Station Ash Transportation" - University of New South Wales, Water Research Laboratory Report No.27. Oct. 1960.

(3) H.W. King - Handbook of Hydraulics - Published 1954 McGraw Hill.

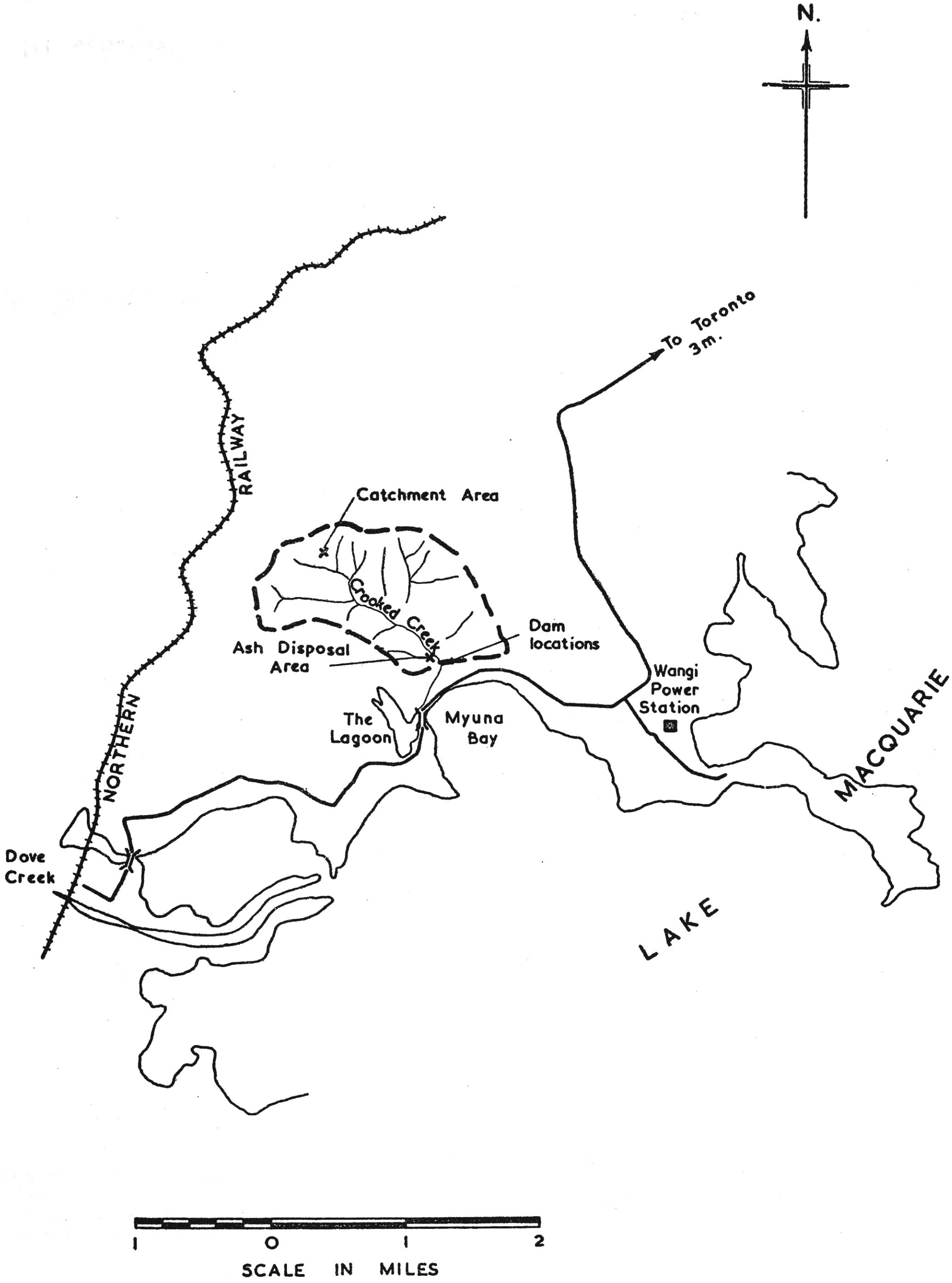
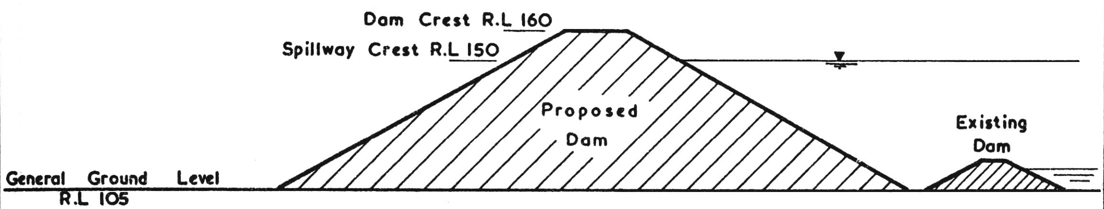


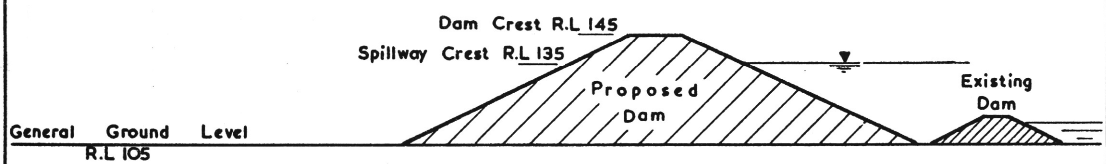
FIGURE No. 1.

THE UNIVERSITY OF NEW SOUTH WALES WATER RESEARCH LABORATORY		
The Electricity Commission of N.S.W. Project 1·2	Scale: As shown above	Date: 18 · 4 · 61
WANGI ASH DAM HYDROLOGIC INVESTIGATIONS	Drawn: C. G. Coulter	CE-E-2328
LOCALITY MAP OF ASH DISPOSAL AREA	Traced: P. A.	
	Checked:	

(a) PROPOSAL No. 1



(b) PROPOSAL No. 2



(c) PROPOSAL No. 3

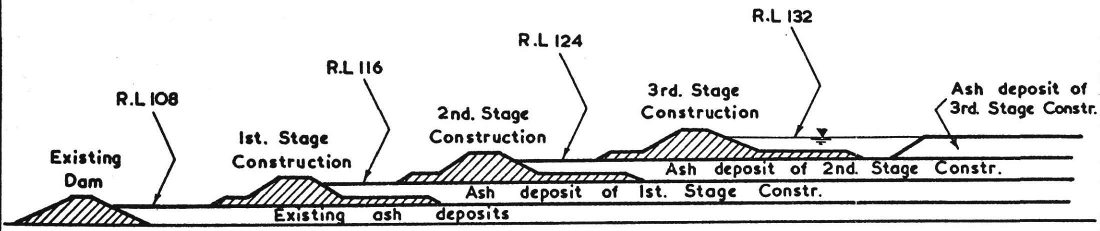


FIGURE No. 2.

THE UNIVERSITY OF NEW SOUTH WALES
WATER RESEARCH LABORATORY

The Electricity Commission of N.S.W Project 1·2	Scale: Not to scale	Date: 19 · 4 · 61
WANGI ASH DAM HYDROLOGIC INVESTIGATIONS	Drawn: C.G. Coulter	CE-E-2329
PROPOSALS INVESTIGATED	Traced: P.A.	
	Checked:	

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WATER RESEARCH LABORATORY

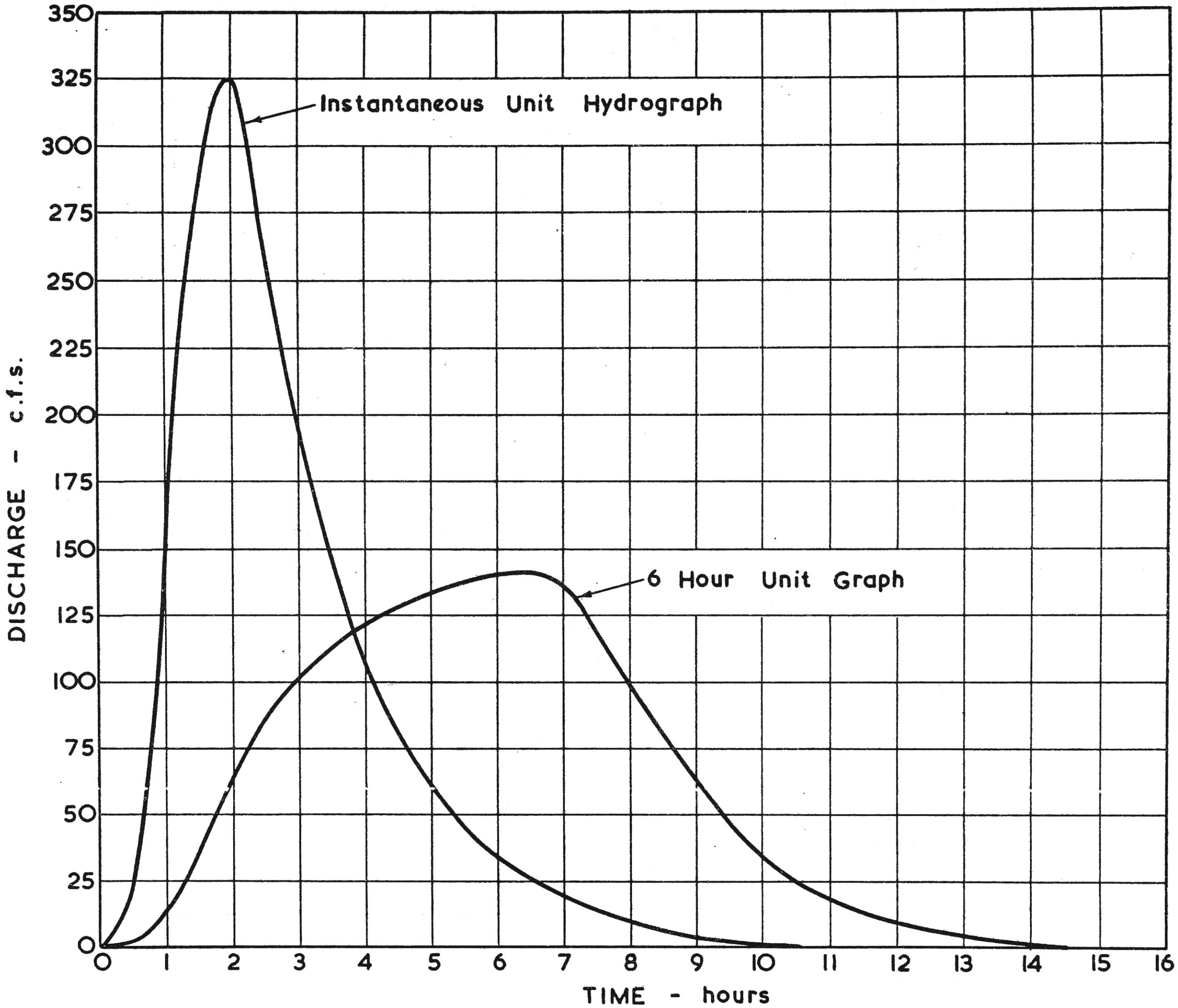
The Electricity Commission of N.S.W Project 1·2

WANGI ASH DAM HYDROLOGIC INVESTIGATIONS

SYNTHETIC UNIT HYDROGRAPHS

Scale:	Date: 19 . 4 . 61
Drawn: C. G. Coulter	
Traced: P.A.	
Checked:	CE-E-2330

FIGURE No. 3.



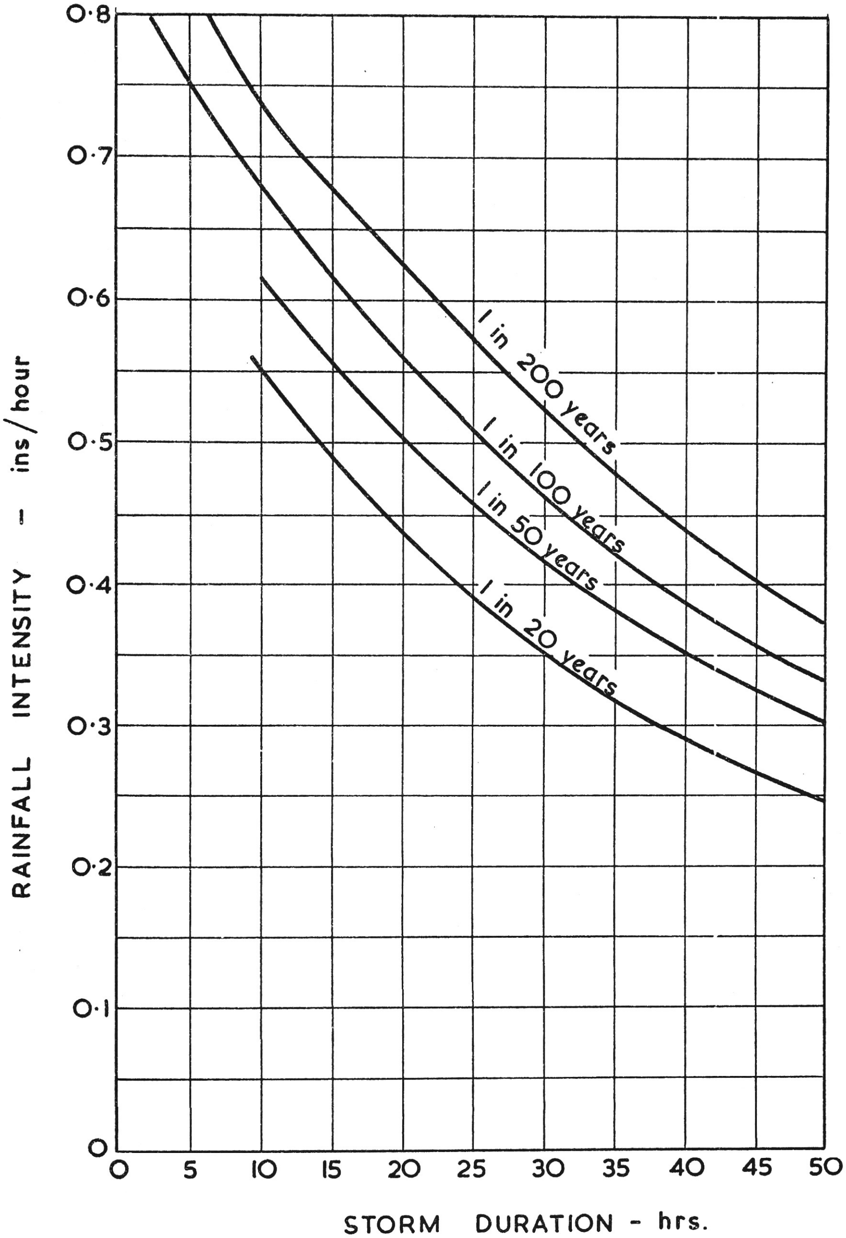


FIGURE No. 4.

THE UNIVERSITY OF NEW SOUTH WALES

WATER RESEARCH LABORATORY

The Electricity Commission of N.S.W Project 1·2 WANGI ASH DAM HYDROLOGIC INVESTIGATIONS GOSFORD RAINFALL INTENSITY DURATION FREQUENCY DATA. (from Stormwater Standards Committee Report)	Scale: Drawn: C.G. Coulter Traced: P.A. Checked:	Date: 19·4·61 CE-E-2331

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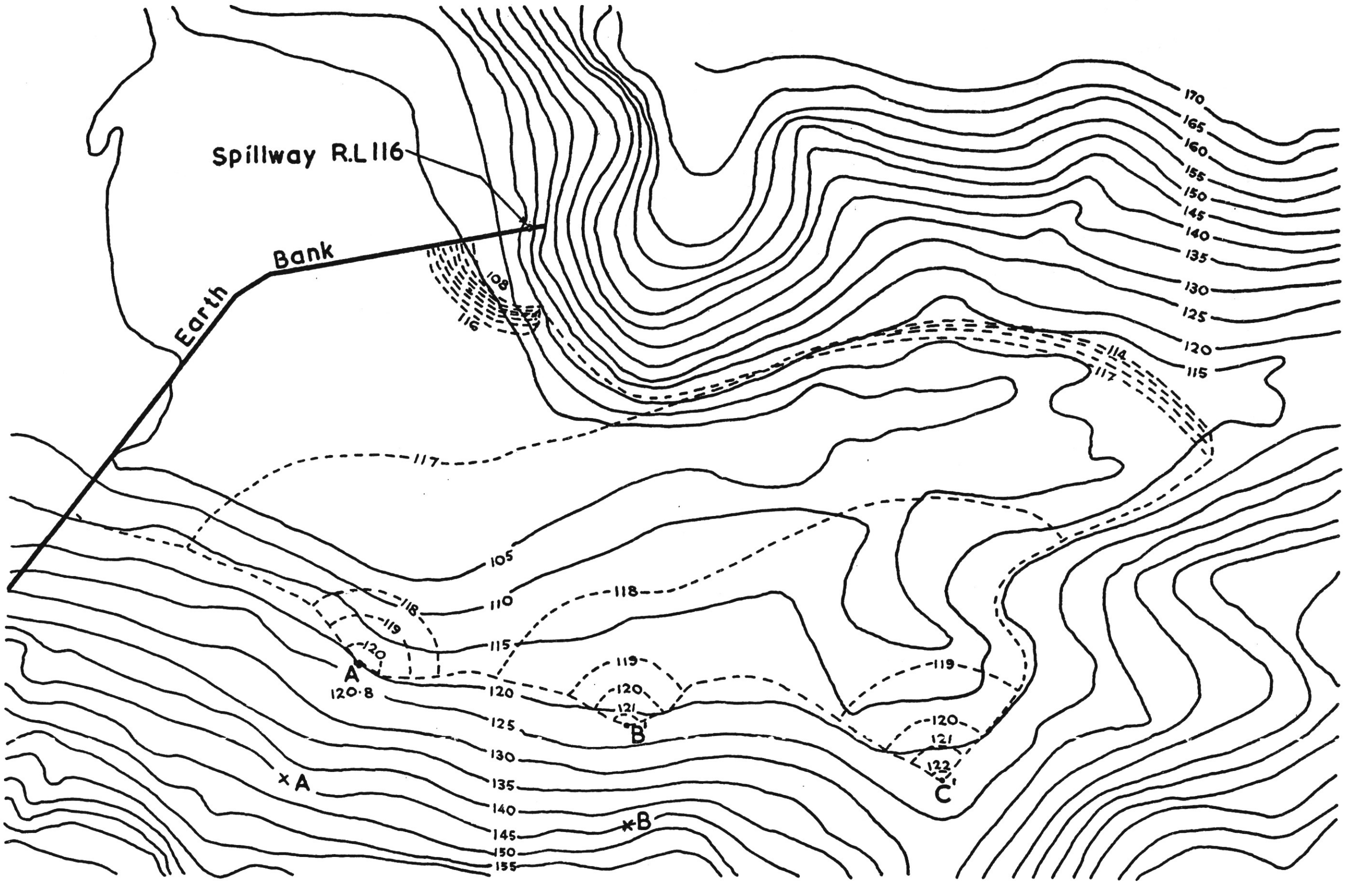
WATER RESEARCH LABORATORY

The Electricity Commission of N.S.W Project 1·2
WANGI ASH DAM HYDROLOGIC INVESTIGATIONS

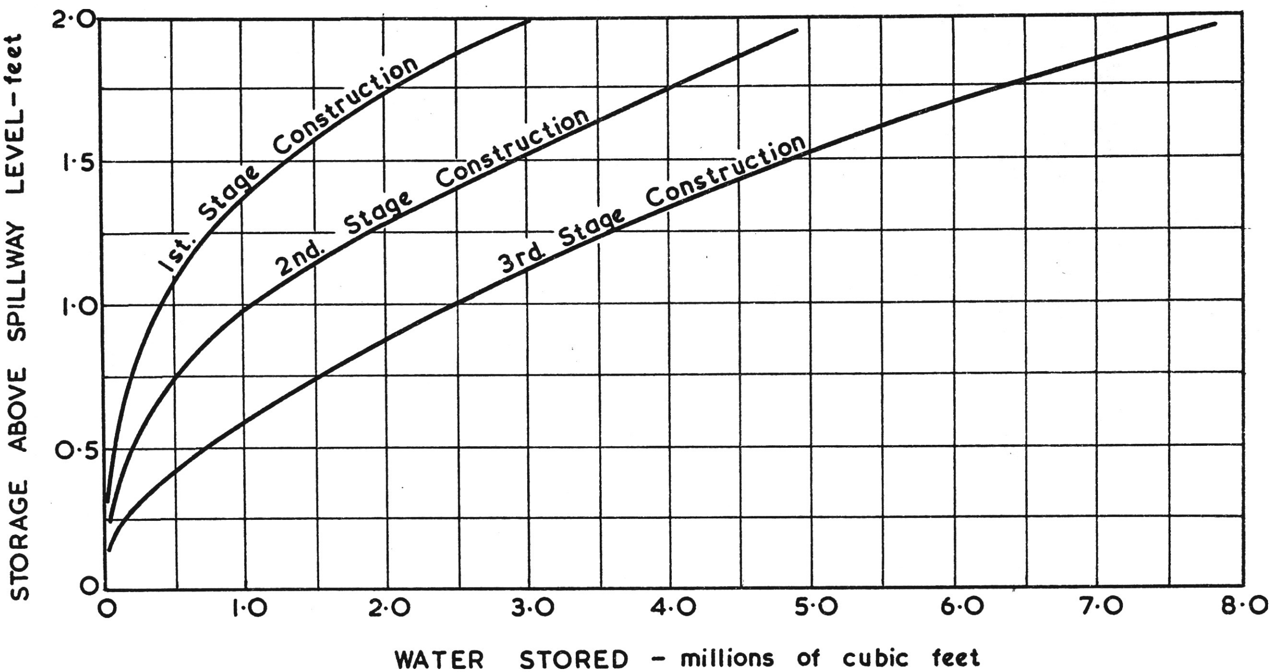
ASH CONTOURS AT COMPLETION
OF STAGE 1.

Scale : As shown above	Date : 15. 9. 60
Drawn : C. Mackenzie	
Traced : P. A.	CE-E-2046
Checked:	

FIGURE No. 5.



——— Natural Surface Contours ------- Ash Contours



SPILLWAY CREST LEVELS:-			
1st. Stage	Construction	R.L 116
2nd.	"	"	R.L 124
3rd.	"	"	R.L 132

FIGURE No. 6.

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The Electricity Commission of N.S.W Project 1:2

WANGI ASH DAM HYDROLOGIC INVESTIGATIONS

STORAGE ELEVATION CURVES FOR

1st, 2nd & 3rd STAGE CONSTRUCTIONS

Scale:

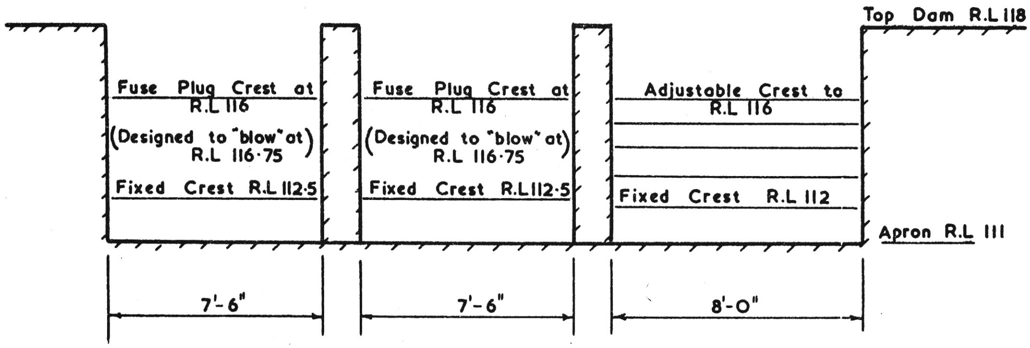
Drawn: C.G. Coulter

Traced: P.A.

Checked:

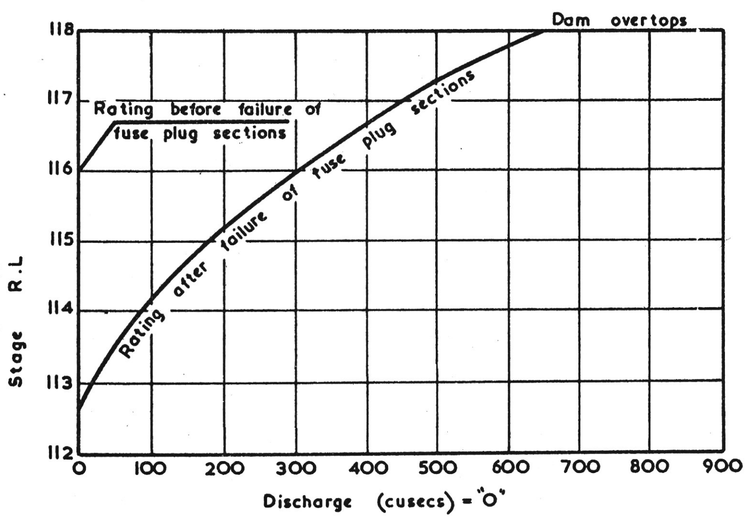
Date: 20.4.61

CE-E-2332



STAGE I CONSTRUCTION SPILLWAY – GENERAL ARRANGEMENT

(Not to scale)



STAGE I CONSTRUCTION – SPILLWAY RATING

FIGURE No. 7.

THE UNIVERSITY OF NEW SOUTH WALES WATER RESEARCH LABORATORY		
The Electricity Commission of N.S.W Project 1·2 WANGI ASH DAM HYDROLOGIC INVESTIGATIONS STAGE I CONSTRUCTION SPILLWAY GEN. ARRANGEMENT & RATING CURVE	Scale: Drawn: C. G. Coulter Traced: P. A. Checked:	Date: 24·4·61 CE-E-2333

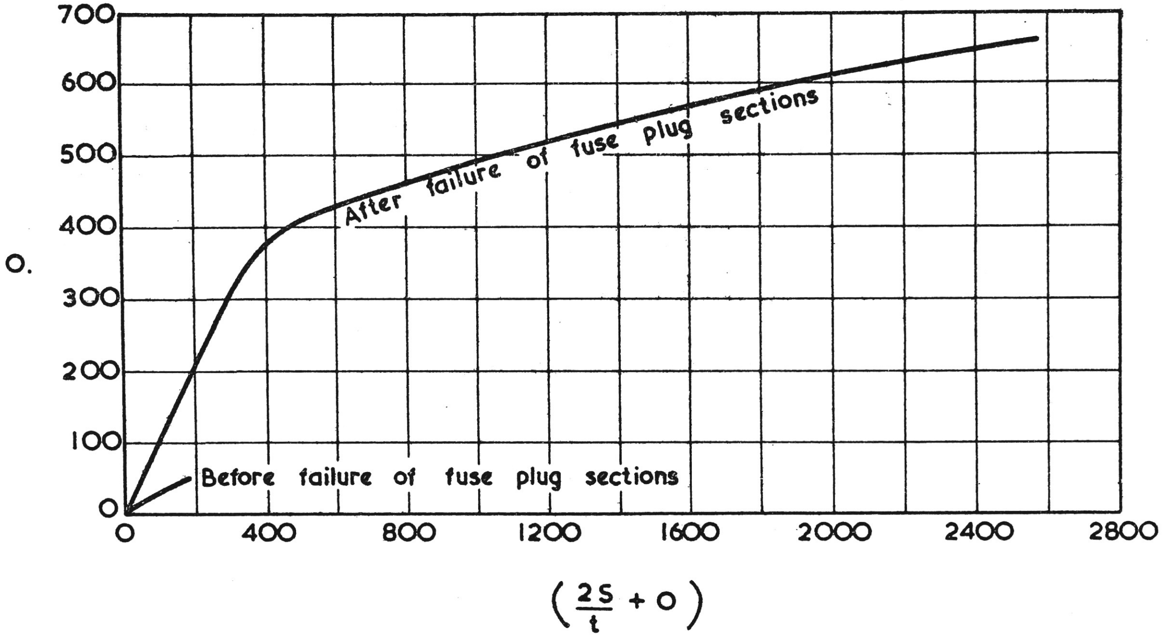


FIGURE No. 8.

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The Electricity Commission of N.S.W Project 1.2
WANGI ASH DAM HYDROLOGIC INVESTIGATIONS
FLOOD ROUTING PARAMETERS FOR
STAGE 1 CONSTRUCTION

Scale:		Date: 26.4.61
Drawn: C. G. Coulter		
Traced: P. A.		CE-E-2334
Checked: DK		

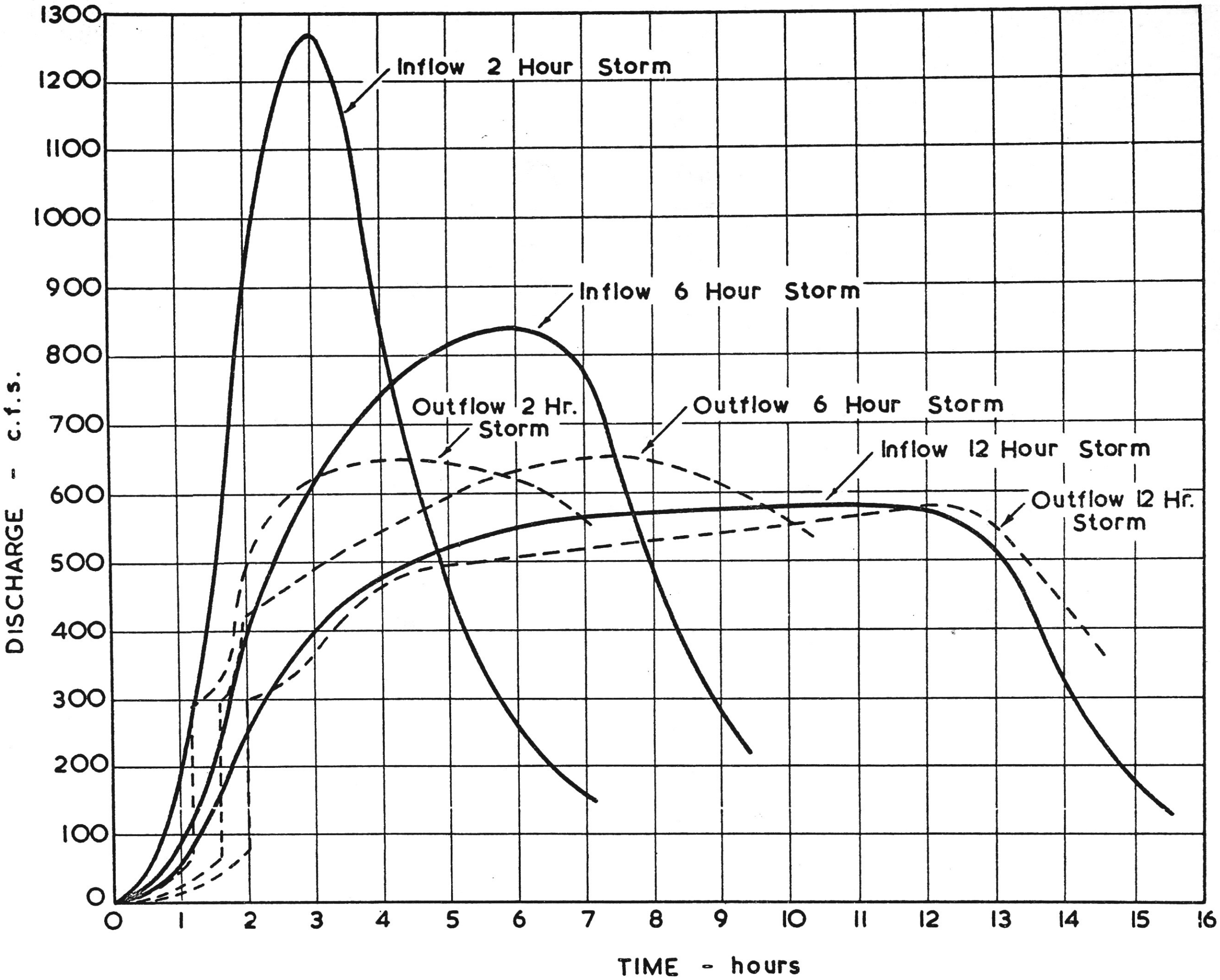

FIGURE No. 9

THE UNIVERSITY OF NEW SOUTH WALES
WATER RESEARCH LABORATORY

The Electricity Commission of N.S.W Project 1.2
WANGI ASH DAM HYDROLOGIC INVESTIGATIONS
FLOOD ROUTING - 1 IN 100 YEAR STORMS
FOR STAGE 1 CONSTRUCTION

Scale:
Date: 26.4.61
Drawn: C.G. Coulter
Traced: P.A.
Checked:

CE-E-2335