COMMONWEALTH FUND FOR TECHNICAL CO-OPERATION

COMMONWEALTH SECRETARIAT

A RECONNAISSANCE STUDY FOR MAJOR SURFACE WATER SCHEMES IN EASTERN BOTSWANA

PHASE II REPORT

JUNE 1977

SIR ALEXANDER GIBB & PARTNERS
READING and LONDON

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30th June 1977

The Managing Director,
Commonwealth Fund for Technical Co-operation,
Commonwealth Secretariat,
Marlborough House,
Pall Mall,
LONDON,
SW1Y 5HX.

Dear Sir,

A Reconnaissance Study for Major Surface Water Schemes in Eastern Botswana

In accordance with our Agreement dated 19th December 1975, we have pleasure in submitting our Phase II Report on the above study.

The Report was submitted to you and to the Botswana Government in draft form in March of this year; comments made by the Director of Water Affairs in his letter of 11th May 1977 addressed to our Gaborone Office have all been taken into account in the production of the final version.

We are glad to have had the opportunity of participating in this interesting study.

Yours faithfully,
for SIR ALEXANDER GIBB & PARTNERS

ACKNOWLEDGEMENTS

We wish to record our sincere appreciation of the co-operation, assistance and advice we have received from all concerned with the study, and in particular the following organisations and individuals (in Botswana unless otherwise stated):

The Reference Group.
The Director of Water Affairs and his staff.
The Assistant Director, Mapping, Department of Surveys and Lands and his staff.
Mr. Spencer Minchin of Mafeking.

CONSULTANTS ASSOCIATED WITH THE STUDY

Dr. J.V. Sutcliffe of the Institute of Hydrology, Wallingford, England, has been largely responsible for the hydrological aspects of the study.

Mr. K.S. Jones of Golder Hoek and Associates visited all but one of the sites selected for this phase of the study and advised on geological aspects.

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CHAPTER 1

INTRODUCTION

1.1 Terms of Reference

The original Terms of Reference for the study are set out in the enclosure to a letter dated 29th July 1975 from the Commonwealth Secretariat, and are reproduced as Appendix A to this Report.

Following the submission of the Phase 1 Report in March 1976 it was agreed that Additional Terms of Reference would be included in Phase II of the study. The additional items are set out in paragraphs 5(a) and 5(b) of a letter dated 21st July 1976 from the Permanent Secretary, Department of Mineral Resources and Water Affairs to the Commonwealth Fund for Technical Co-operation, and are as follows:-

(a) "Phase I has indicated that ordinary impoundment reservoirs do not appear to be feasible on the Nata River or on the Metsemotlhaba - Notwane between the Molepolole - Gaborone road and the Limpopo confluence. As an addition to Phase II the consultant will be required to carry out additional air photo study and field reconnaissance and to advise on any possibilities there may be for the harnessing of these resources.

(b) In order that the study may be complete for Eastern Botswana the consultant is required to check locally and in South Africa whether it is correct to regard the Molopo River as a resource which is negligible other than for minor local purposes."

1.2 Scope of Phase II Study

Apart from the Additional Terms of Reference Phase II of the Study consists essentially of site surveys and reconnaissance, more detailed hydrological investigations, and the preparation of designs and cost estimates for dams at a number of sites selected by the Department of Mineral Resources and Water Affairs from the list of most favourable sites given in Table 4.1 of the Phase I Report. We understand that this selection was made in accordance with the Botswana Government's current needs for information, both positive and negative, and does not by any means represent a final priority rating.

The location of all sites is shown on Map No. 7548/1 of the Phase I Report and on Figure No. 1.1 of this Report.

Table 1.1 shows the selected sites together with details of their catchment areas and approximate net yields.

TABLE 1.1

SITES SELECTED FOR PHASE II STUDY

SITE NO.	RIVER	CATCHMENT AREA	APPROXIMATE (1) NET YIELD
		km^2	m^3/day
3	Tutume	210	3 800
10	Ntshe	440	12 700
13	Tiaye	86	1 600
17	Tati	2 400	46 600
33	Motloutse	6 700	65 000
40	Motloutse	9 200	78 000
42A	Thune	2 400	15 500
46	Lotsane	5 700	20 700
55	Taupye	660	3 400
56	Bonwapitse	1 540	12 500
61(2)	Kolobeng	165	740
62	Metsemotlhaba	800	3 600
70	Motloutse	13 100	97 000

(1) - Phase I values, at 1 in 50 years risk of failure.

(2) - This site did not appear in Table 4.1 of the Phase I Report, but was judged to be more promising than previously estimated early on in the Phase II study.

1.3 Method of Study

Work in Phase II has involved the following personnel:-

Project Engineer
Engineer
Senior Hydrologist
Hydrologist
Geologist
Surveyor
Consultant

(a) Work in Botswana and South Africa

The Engineer arrived in Botswana on 25th October 1976 and was joined by the Project Engineer, Geologist and Surveyor on 27th October. Following a meeting with the Director of Water Affairs and members of his staff on 28th October the whole team visited Sites Nos. 62, 55 and 56. The Surveyor returned to Gaborone on 31st October and commenced survey work at Site No. 62, subsequently moving on to Sites Nos. 55 and 56.

The other members of the team continued the first reconnaissance of the remaining sites, and this work was completed by the 5th November. During this reconnaissance the extent of the area requiring detailed survey was decided and an appreciation of surface geology and a preliminary assessment of engineering factors was made.

The Engineer then joined up with the Surveyor and survey work and more detailed engineering investigations were completed for all the sites by the middle of December 1976.

The Project Engineer made a preliminary report of the results of the first reconnaissance of the sites to the Reference Group in Gaborone early in November. Following this report it was decided that Site No. 61 should be included in the list of sites to be surveyed.

The Project Engineer's remaining work in Botswana comprised:-

(i) Examination of aerial photographs as required by the Additional Terms of Reference,

(ii) A visit to Mafeking and the Molopo River,

(iii) A visit to Francistown and (accompanied by the Engineer) an aerial reconnaissance of the Nata River.

(b) Work in United Kingdom

Hydrological analytical work, dam design and the preparation of drawings and cost estimates was carried out in the United Kingdom during the period December 1976 to February 1977.

1.4 Basis of Surveys

The original Terms of Reference includes (paragraph 2.5(a)): "Surveying a cross-section of the proposed line of the embankment and one longitudinal section of the area to be flooded, related to an arbitrary datum and bench marks described below".

The intended purpose of the longitudinal section was to enable reservoir areas and volumes to be calculated. In practice, however, dam sites are frequently located just downstream of major tributary confluences, and a single longitudinal section would not then be adequate. Much better estimates of reservoir areas and hence of volumes can be obtained from the existing 1:50 000 scale mapping even though the contour interval (either 25' or 50') is rather large for this purpose. If estimates are prepared on this basis it is necessary to relate ground level at the dam site to map datum in order to relate dam height (and hence cost) to reservoir capacity and yield.

It was therefore proposed during the meeting with the Director of Water Affairs and his staff on 28th October that the bench marks which were to be established at each site should be related to map datum rather than to an arbitrary datum and that the longitudinal section of the area to be flooded should be omitted. This proposal was accepted.

In practice the field survey bench marks at the various dam sites were usually tied in to nearby trigonometrical bench marks, established by the Department of Surveys and Lands and shown on the 1:50 000 scale surveys.

The single exception, Dam Site No. 56, was related to the known deck level of a nearby road bridge.

A number of surveying methods were used:-

(i) Direct horizontal levelling

(ii) Direct tacheometric levelling

(iii) Trigonometric levelling from a base line

(iv) Barometric levelling, using a pair of aneroid barometers

Details of the bench marks established at each site, and of the surveying method used, are given in Appendix B.

In order to check the accuracy of the method the surveyed cross-section at each dam site was compared with a cross-section drawn from the contours shown on the 1:50 000 scale maps. In most cases reasonable agreement between the two cross-sections was obtained. However, for four sites there appeared to be a difference in datum, as follows:-

Dam Site No. 3	3 metres must be added to the field survey levels in order to relate them to the local map datum.
Dam Site No. 33	2 metres must be added to the field survey levels in order to relate them to the local map datum.
Dam Site No. 46	16 metres must be subtracted from the field survey levels in order to relate them to the local map datum.

Dam Site No. 55 — 6 metres must be added to the field survey levels in order to relate them to the local map datum.

Account has been taken of the above differences in datum when establishing the maximum height of the dam and comparing reservoir yield with the cost of the dam.

CHAPTER 2

HYDROLOGY

2.1 Estimation of mean annual run-off at selected dam sites.

The mean annual run-off at all potential dam sites was estimated in Phase I by comparing the mean annual rainfall and run-off at sites where records were available, both in Botswana and in adjacent areas. The estimates of run-off were necessarily provisional, not only because the rainfall-run-off relationship depended on few records but also because the potential sites and their catchments could not be visited during the time available.

During Phase II the number of selected dam sites was more limited, and although no general reappraisal of the rainfall-run-off regime was necessary, it was possible to reconsider certain sites after field visits, though without site hydrological investigations. During the field survey opportunity was taken to observe how well the channel dimensions and effective catchment areas corresponded with the run-off estimates and areas used in Phase I.

At three sites the effective catchment areas were reconsidered because the drainage pattern suggested that only part of the basin contributed to run-off. The effective catchment areas at Sites 46, 55 and 56 were revised from 5,700, 660 and 1,540 to 4,220, 530 and 600 square kilometres respectively. The average rainfall estimates had therefore to be revised in two cases to allow for the rainfall on the effective part of the basin, and the mean annual run-off estimates were reduced as shown in Table 2.1.

TABLE 2.1

REVISED FLOWS AT SURVEYED SITES (AFTER TABLE 3.6 OF PHASE I REPORT)

Dam Site Ref. No.	Region	Area	Effective Area	Mean Annual Rainfall	Revised Rainfall	Mean Annual Run-off	
		km^2	km^2	mm	mm	mm	$m^3 \times 10^6$
46	S	5700	4220	467	475	5	21
55	C	660	530	450	450	7	3.7
56	C	1540	600	500	475	9	5.4

Other catchment areas did not require similar adjustment, except possibly Sites 33, 40 and 70 where the flow estimates are in any case based largely on records at Tobane.

The estimated run-off at Site 17 was reconsidered in Phase II. Because the mean annual run-off at Site 17 estimated from rainfall in Phase I was $58m^3 \times 10^6$, and this was less than the combined flows at the Tati and Inchwe weirs, the estimate was increased to the sum of these ($63m^3 \times 10^6$). However, this estimate did not assume any contribution from the catchment below the two weirs; after field visits it was thought appropriate to include a fairly conservative estimate of run-off from this lower area. The rainfall of the lower 1 030 km^2 is 435 mm, and the run-off is estimated as 16 mm, giving $16m^3 \times 10^6$ to be added to the total flow at the two weirs to give $79m^3 \times 10^6$ at Site 17. Comparison of the flows at L12 (Shashe below Tati) with flows at the three weirs upstream suggests that this estimate of run-off from the lower basin is reasonable.

Although the MAR estimated for Site 61 from rainfall-run-off relationship is low ($0.99m^3 \times 10^6$) field observations and enquiries showed that the river had been flowing continuously for several years. Spot measurements in November 1976 gave a flow of $0.09m^3$/second, which would amount to $2.8 \times 10^6 m^3$/year. It is clear that this river is unusual and worth gauging for a better estimate of yield. Meanwhile we have made yield estimates for a range of values of MAR without allowing for the apparent unusual pattern of run-off.

It is often suggested that bank-full discharge corresponds to a flood of a given frequency (about an annual flood) and in Botswana, where so much of the run-off is composed of floods, the mean annual flood is likely to be related to mean annual run-off. H.C. Riggs (Ref.1) has shown that channel size can be used in semi-arid areas to estimate flood characteristics such as the 10 and 50 year floods, indeed "... In semi-arid regions this method produces better results than regression on basin characteristics."

It was therefore thought worthwhile comparing the bank-full flow and channel width with MAR as a subjective check (Table 2.2).

TABLE 2.2

ESTIMATED MAR AGAINST CHANNEL WIDTH AND BANKFULL DISCHARGE

Site No.	Estimated MAR ($m^3 x10^6$)	Channel Width (m)	Bankfull Discharge (m^3/sec)
3	5.2	36	108
10	17.2	68	190
13	2.2	30	168
17	79.0	80	613
33	110.0	135	736
40	110.0	82	614
42A	22.0	34	167
46	21.0	15	75
55	3.7	48	198
56	5.4	10	40
61	0.99	9	10
62	4.8	33	24
70	131.0	180	924

Figure 2.1 shows the relationship between estimated annual run-off and the bankfull discharge using Manning's equation, cross sectional channel dimensions, channel slope and an estimated roughness coefficient of 0.030. Figure 2.2 shows the simple relationship between channel width and run-off. The relationship between bank-full discharge and MAR is not direct and much of the scatter shown in Figure 2.1 is likely to be due both to the estimate of this discharge and to its relationship with the mean. The possibility also exists that some of the rivers are fossil rivers in that their channel geometry was developed during the earlier wetter climatic period. Although showing a scatter of points both diagrams do, however, support the estimates of catchment run-off.

2.2 Reservoir Characteristics and Yields

A dimensionless storage-yield curve based on the average curve for the three existing reservoir sites (Gaborone, Shashe, Nuane) was used in Phase I for the yield estimates at possible dam sites. It was assumed that Eastern Botswana corresponded to Drought Region No. 12 (Midgley & Pitman-Ref.2) which is described as Lowveld areas of northern and north-eastern Transvaal.

It was decided that the justification for using the Region 12 cumulative flow curve should be tested by using a rainfall-run-off model to extend an example of gauged flows in Botswana and subject this extended synthetic record to drought flow frequency analysis similar to that used by Midgley and Pitman.

The model chosen used monthly data and was developed by Pitman (Reference 3) who examined 50 gauged South African catchments from a wide range of hydrological regimes. Precipitation is stored as interception and soil moisture and this is subject to evaporation and transpiration. Precipitation which has not been absorbed by the soil then becomes the source of surface run-off. Part of the soil moisture finds its way direct to the river system and part via groundwater; by suitably lagging the three components, the total run-off can be calculated.

The Shashe at Mooke Weir having one of the longest and most reliable records of monthly discharges (1962-1975) was used for proving the model and generating a long run-off record. A long record of monthly rainfall was also available from 1922 for nearby Francistown and the mean monthly evaporation rate was known (Table 3.2, Phase I Report). The monthly average catchment rainfall was derived by multiplying the average long term catchment rainfall for Shashe at Mooke Weir (469 mm) by the Francistown monthly rainfall expressed as a fraction of its own average rainfall (459 mm).

Model parameters were obtained from the mapped regional values given by Pitman and used to generate 14 years of monthly flow data using the derived catchment rainfall for the period 1962-1975 as input to the model. These regional model parameters were then adjusted to preserve the mean annual run-off, the mean and standard deviation of the logs of the annual flows and the seasonal distribution of flow:- these being the parameters recommended by Pitman for testing the accuracy of the model. Table 2.4 shows the monthly observed flows and Table 2.5 shows the set of generated flows for the same period which best preserved the characteristics of the flow record. Table 2.3 summarises the results of testing the model, the long period synthesised record being the result of using the adjusted model parameters and the long period rainfall record.

TABLE 2.3

SIMULATION RESULTS FOR SHASHE AT MOOKE WEIR

Comments		MAR $m^3 x 10^6$	Mean (Log)	Std. Dev. (Log)
OBSERVED RECORD	1962-1975	79.02	1.75	0.39
REGIONAL PARAMETERS	1962-1975	36.91	1.16	0.72
ADJUSTED PARAMETERS	1962-1975	79.04	1.51	0.66
SYNTHESISED RECORD	1922-1975	73.78	1.54	0.60

The long period simulated flows were then used to determine the recurrence interval of independent low-flow events of duration 1 to 7 years by accumulating successive N month flows. The observed cumulative inflow duration probability points for Shashe were then plotted on Figure 2.3 showing Region 12 curves which were used as the basis for the yield assessments in Phase I. The Region 12 curves have a slightly lower cumulative inflow for a given duration at the rarer return periods than the extended Shashe record but in general there is good agreement between them. This is felt to justify the use of these curves in storage yield analysis for Eastern Botswana.

In Phase I, estimates of reservoir yield were made by deriving a dimensionless storage yield relationship for the three existing reservoirs (Shashe, Gaborone and Nuane) and applying it to the estimated mean annual run-off of each reservoir site. The method used was that developed by Midgley and Pitman (1969) and is outlined in the Phase I Report. The method, by assuming a given sequence of flows into the reservoir for a given duration and probability, estimates the evaporation loss by using the area/volume relationship for the reservoir and the volume of dead storage. The volume of effective storage required to maintain the evaporation loss and required yield is then calculated for a given reliability.

Having confirmed the choice of the Region 12 cumulative flows by using the rainfall-run-off model, the same approach was used to estimate reservoir yields, except that the individual reservoir geometry and dead storage were used to derive storage yield relationships at each site. Figures 2.4 to 2.17 show the reservoir area and volume curves used for this purpose.

Probabilities of failure of 1 in 50 and 1 in 20 years were used to estimate the effective storage for given yields at each site. At Site 17 a proposed Northern and Southern alignment of the dam gave two sets of yield estimates, whilst at Site 61 three different mean annual run-off estimates were used to give three sets of yield calculations. Table 2.6 and Figures 2.18 to 2.30 show the effective storage required to maintain a yield with a reliability of 1 in 50 and 1 in 20 years, expressed in terms of the percentage of the catchment mean annual run-off. The main variation in yield between the thirteen sites studied depends on the absolute value of the mean annual run-off and the coefficient and exponent of the area-volume relationship. The difference between the Phase I estimates and current estimates in some cases is largely due to the fact that at some sites field survey revealed that the topography was not as favourable as at the existing Shashe and Gaborone reservoirs whose characteristics were used as criteria in Phase I. The dead storage was less than 7 per cent of the mean annual run-off for all sites except 13 where it was 11.4 per cent and as a result the dead storage was not an important factor in variations between sites.

TABLE 2.4

OBSERVED MONTHLY FLOWS FOR SHASHE AT MOOKE WEIR

($m^3 x 10^6$)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1962-1963	0	37.540	5.707	21.587	1.341	0	19.077	0	0	0	0	0	85.252
1963-1964	0	.08	11.925	2.054	0	0	0	0	0	0	0	0	14.059
1964-1965	0	1.525	6.765	9.619	1.833	0	0	0	0	0	0	0	19.742
1965-1966	0	13.604	1.181	18.857	57.485	17.823	0	0	0	0	0	0	108.950
1966-1967	0	4.121	36.150	53.493	44.612	0	2.891	0	0	0	0	0	141.267
1967-1968	0	0	0	5.053	38.952	1.034	4.518	0	0	0	0	0	49.557
1968-1969	0	2.548	8.025	6.928	7.032	61.903	14.910	0	0	0	0	1.080	102.426
1969-1970	2.333	5.632	9.301	0	17.62	0.02	0	0	0	0	0	0	34.906
1970-1971	0	0	16.35	28.98	3.43	0	8.90	0	0	0	0	0	57.66
1971-1972	0.88	5.42	13.99	164.6	5.8	5.4	0.1	0	0	0	0	0	196.19
1972-1973	0	0	0	2.845	5.591	1.773	0	0	0	0	0	0	10.209
1973-1974	0	24.0	21.6	79.6	6.5	2.8	0	0	0	0	0	0.8	135.3
1974-1975	0	2.246	7.694	31.664	22.350	3.263	4.473	0	0	0	0	0	71.690
MEAN	0.25	7.44	10.67	32.71	16.35	7.23	4.22	0	0	0	0	0.14	79.02
% MONTHLY TOTAL	0.32	9.42	13.50	41.40	20.69	9.15	5.34	.00	.00	.00	.00	0.18	100.00

TABLE 2.5

SYNTHESISED RUN-OFF FOR SHASHE AT MOOKE WEIR WITH MODEL PARAMETERS USED TO GENERATE LONG PERIOD FLOW RECORD

($m^3 x 10^6$)

YEAR	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	TOTAL
1962-1963	.00	23.58	8.20	3.47	1.23	.04	.91	.30	.21	.07	.00	.00	38.01
1963-1964	.09	.27	7.01	3.24	.33	.01	.00	.00	.00	.00	.00	.00	10.95
1964-1965	.06	.59	1.62	.54	.08	.02	.01	.00	.00	.00	.00	.00	2.91
1965-1966	.00	.75	.25	.75	83.87	27.88	.00	.01	.00	.00	.00	.00	113.52
1966-1967	.01	3.38	87.92	74.13	35.36	6.77	.82	.27	.00	.00	.00	.00	208.64
1967-1968	.05	1.56	.58	1.00	1.12	.31	.90	.53	.08	.00	.00	.00	6.12
1968-1969	.00	.88	1.25	.34	5.50	24.11	7.52	.03	.00	.00	.00	.49	40.12
1969-1970	2.91	1.45	14.21	4.68	7.56	2.66	.54	.16	.05	.02	.00	.00	34.24
1970-1971	.00	.17	5.12	1.70	.03	.01	.16	.05	.00	.00	.00	.00	7.23
1971-1972	.10	2.43	1.06	231.46	77.23	2.28	.76	.00	.00	.00	.00	.00	315.32
1972-1973	.01	.05	.97	4.18	1.58	.11	.03	.01	.00	.00	.01	.06	7.02
1973-1974	1.00	.82	.23	27.97	20.31	3.68	.22	.07	.00	.00	.00	1.06	55.37
1974-1975	.35	9.43	8.65	123.94	42.66	.70	1.75	.58	.00	.00	.00	.00	188.06
MEAN	.35	3.49	10.54	36.72	21.30	5.28	1.05	.15	.03	.01	.00	.12	79.04
% MONTHLY TOTAL	.45	4.41	13.34	46.46	26.95	6.67	1.32	.20	.03	.01	.00	.16	100.00

TABLE 2.6

EFFECTIVE STORAGE AND YIELD AS % MAR FOR GIVEN RETURN PERIOD (T) YEARS

SITE 3 MAR = 5.2 $m^3 x 10^6$

	T = 50							T = 20						
YIELD	5	10	11					5	10	15	20	25	27	
STORAGE	33	100	225					15	36	64	95	160	225	

SITE 10 MAR = 17.2 $m^3 x 10^6$

	T = 50						T = 20						
YIELD	5	10	15	20	25	26	5	10	15	20	25	35	45
STORAGE	17	37	71	108	164	197	9	21	37	57	80	135	244

SITE 13 MAR = 2.2 $m^3 x 10^6$

	T = 50	T = 20		
YIELD	1	2.5	5	7.5
STORAGE	100	61	130	200

SITE 17 (S ALIGNMENT) MAR = 79 $m^3 x 10^6$

	T = 50				T = 20						
YIELD	5	10	15	20	5	10	15	20	25	30	33
STORAGE	34	67	122	210	22	39	62	88	126	160	200

SITE 17 (N ALIGNMENT) MAR = 78.4 $m^3 x 10^6$

	T = 50				T = 20						
YIELD	5	10	15	20	5	10	15	20	25	30	35
STORAGE	33	63	112	185	19	36.5	59	81	113	141	179

SITE 33 MAR = 110 $m^3 x 10^6$

	T = 50						T = 20						
YIELD	5	10	15	20	25	26	5	10	15	20	25	35	40
STORAGE	19	48	78	121	166	197	13	26	45	62	89	147	200

SITE 40 MAR = 110 $m^3 x 10^6$

	T = 50						T = 20						
YIELD	5	10	15	20	25	30	5	10	15	20	25	35	45
STORAGE	14	32	56	89	133	202	7	18	32	51	71	116	200

TABLE 2.6 (cont.)

SITE 42A MAR = 22 m^3x10^6

	T = 50						T = 20						
YIELD	5	10	15	20	25	26	5	10	15	20	25	30	38
STORAGE	16.5	39	69	112	164	205	11.5	23	41	58	83	108	161

SITE 46 MAR = 21 m^3x10^6

	T = 50					T = 20					
YIELD	5	10	15	20	21	5	10	15	20	25	30
STORAGE	23	46	88	148	180	12	27	46	73	109	147

SITE 55 MAR = 3.7 m^3x10^6

	T = 50			T = 20					
YIELD	5	10	15	5	10	15	20	25	30
STORAGE	19	83	200	10	31	56	96	140	217

SITE 56 MAR = 5.4 m^3x10^6

	T = 50			T = 20			
YIELD	5	6	7	5	10	15	17.5
STORAGE	48	70	120	17.5	58	117	150

SITE 61* MAR = 3.0 m^3x10^6

	T = 50					T = 20						
YIELD	5	10	15	20	25	5	10	15	20	25	30	35
STORAGE	14	34	57	102	184	9.5	23	36.5	56.5	78	104	137

SITE 61* MAR = 2.0 m^3x10^6

	T = 50					T = 20						
YIELD	5	10	15	20	25	5	10	15	20	25	30	35
STORAGE	17	38.5	68	125	208	10	23	37.5	62	85	117	155

SITE 61* MAR = 0.99 m^3x10^6

	T = 50				T = 20						
YIELD	5	10	15	20	5	10	15	20	25	30	35
STORAGE	21	50	91	210	13	34	50	75	109	152	220

TABLE 2.6 (cont.)

SITE 62 MAR = 4.8 $m^3 x 10^6$

	T = 50		T = 20		
YIELD	2	4	5	10	15
STORAGE	133	200	55	108	190

SITE 70 MAR = 131 $m^3 x 10^6$

	T = 50						T = 20							
YIELD	5	10	15	20	25	26	5	10	15	20	25	30	35	40
STORAGE	16	38	70	115	168	196	8	22	36	55	78	102	135	182

* The estimated MAR for Site 61 is 0.99 $m^3 x 10^6$. Local site information indicates that this could be an underestimate and so two additional yields were calculated assuming a MAR of 3.0 and 2.0 $m^3 x 10^6$. The values stated are for the preferred upstream alignment.

2.3 Spillway Flood Estimates

We have based our estimates of flood flows on work carried out on our behalf by the Institute of Hydrology relating to the Serule-Francistown Road (Reference 4).

Mean annual flood (MAF) may be defined as the average of the annual maximum series or the highest flows at a station during each year. This is useful as an index of the flood producing capacity of the catchment.

The method adopted assumes that the mean annual flood is related to catchment area (A) by an equation of the form:-

$$MAF = C\ A^{1/2}$$

where MAF is measured in m^3/sec.

A is measured in sq. km.

and C is a constant

From an examination of records for the area a value of C = 8 was determined as being appropriate for catchments exceeding 500 sq. km.

A graphical relationship between MAF, return period (T) and the maximum flood Q(T) to be expected during that period was also determined from a study of records for the area, the results of which are summarised in Table 2.7.

TABLE 2.7

RELATIONSHIP BETWEEN MAF, Q(T) AND T FOR EASTERN BOTSWANA

RETURN PERIOD (T) (Years)	Q(T)/MAF
2	0.8
5	1.5
10	2.2
20	3.1
25	3.6
50	5.4
75	6.5
100	7.1
200	10.7

Although this work was specific to north-east Botswana and is strictly applicable only to catchment areas exceeding 500 sq. km, we believe that for the purpose of preliminary designs and cost estimates such as those presented in this Report, it will give sufficiently accurate estimates for all the sites studied.

On this basis the 200 year return period floods, used later in this Report, would be as shown in Table No. 2.8 overleaf:-

TABLE 2.8

200 YEAR RETURN PERIOD FLOODS

DAM SITE NO.	CATCHMENT AREA sq. km.			MAF m^3/sec	200 YEAR FLOOD m^3/sec
3	210	25	1.2	116	1 241
10	440	7	1.6	168	1 798
13	86	3	3.5	74	792
17	2 400	50	2.1	392	4 194
33	6 700	40	.6	655	7 008
40	9 200	66	7	767	8 207
42A	2 400	10	.4	392	4 194
46	5 700*	30	.5	604	6 463
55	660*	2	3	206	2 204
56	1 540*	13	.8	314	3 360
61	165	1	.6	103	1 102
62	800	4.5	.6	226	2 418
70	13 100	60	.5	916	9 801

* Although the catchment areas were reduced at Dam Sites Nos. 46, 55 and 56 in order to arrive at the effective area for the purpose of yield estimates, we have assumed that it would be appropriate to retain the full catchment areas for the purpose of spillway flood estimates.

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(2) MIDGLEY, D.C. and PITMAN, M.V. 1969 Surface Water Resources of South Africa. University of Witwatersrand, Department of Civil Engineering, Hydrological Research Unit Report No. 2/69.

(3) PITMAN, W.V. 1973 A Mathematical Model for Generating Monthly River Flows from Meteorological Data in South Africa. University of Witwatersrand, Department of Civil Engineering, Hydrological Research Unit Report No. 2/73.

(4) C. CUNNANE, B.E., Ph.D. April 1975 "Flood Hydrology for Bridge Culvert Design on Serule-Francistown Road, Botswana".

CHAPTER 3

DESCRIPTION OF SITES

This Chapter gives the results of our field surveys and investigations. Each site is described under the following headings:

General Topography
Geology
Engineering Considerations
Construction Materials
Dam Geometry and Field Survey

Owing to the fact that Site No. 61 was only included in the list of dam sites to be studied at a late stage of the field work, it was not visited by the Geologist. The geological section for this site is therefore based on examination of rock samples and the study of available data and maps.

\+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

NOTE -

1. Levels shown are related to field survey datum – standard correction of +3·0m (9·8ft) assumed for correlation with local cartographic data.
2. Site datum based on horizontal levelling to BPP 116 (see Appendix B)

Scales – Horizontal 1:5000
Vertical 1:500

SITE No 3

FIGURE No 3.2

3.1 Site No. 3 - Tutume River (Figures 3.1 and 3.2)

3.1.1 General Topography

This site is on the Tutume River about 2 km upstream from the centre of the township of Tutume. On the left bank there is a low ridge of exposed bedrock marked by a prominent kopjie beside the road, and on the right bank a number of small kopjies indicate a similar ridge of shallow bedrock.

Beyond the kopjies on each bank, the terrain is generally flat and the site is suitable only for a low rock or earth fill dam.

On the left bank the main road from Sebina to Tutume is at present being reconstructed to gravel standard and the alignment could be affected by impoundment above level 1,110 m. The reservoir area is generally outside the township and is free from permanent dwellings.

3.1.2 Geology

The site is located in an area underlain by rocks of the Basement Complex which on the proposed alignment crop out as a variable assemblage of granitic gneisses. Outcrops of the bedrock are limited to a few small kopjies, with the remaining area covered by a sandy overburden. The granitic gneisses have a regional strike of NE/SW with dips of about 60° to the S.W. A few kilometres upstream of the site is a broad zone with the same regional trend underlain by amphibolites. The bedrock formed of granitic gneiss crops out as piles of boulders which are generally in an unweathered condition. Elsewhere along the alignment the rock is overlain by a sandy gravel which is probably residual formed from decomposed bedrock.

There appear to be no major faults or significant structural discontinuities which would adversely affect the foundations.

3.1.3 Engineering Considerations

The reconnaissance has indicated that the site is suitable for construction of a rock or earthfill dam, and from superficial observations it would appear that there are no unexpected features likely to affect the general foundation conditions. With regard to foundation conditions, either of the groups of kopjies on the left and right banks would be suitable for siting the spillway structure. One of the main considerations in relation to the foundations at this site would be the incorporation of the sounder bedrock outcrops of the kopjies within the normal bedrock profile. It would be necessary to avoid the sharp changes of elevation in the core trench which could occur between areas where bedrock crops out at the surface, and those areas where it is more deeply weathered. Although outcrops are limited between the major kopjies it is considered that bedrock is at a reasonably shallow depth below the overburden, possibly of the order of 2 to 3 m. Except in the channel area where a deep cut-off trench would probably be required, it is considered that for the remainder of the alignment normal grouting below a shallow core trench would suffice to provide an effective cut-off.

3.1.4 Construction Materials

The numerous kopjies in the area formed of granitic gneiss should be suitable to develop quarry sites for coarse concrete aggregates and riprap. Additional material may become available from excavation in the selected spillway area.

The river bed contains extensive deposits of coarse sand, and both sand and gravel deposits are found in both the left and right banks. These deposits would be suitable for use as bulk fill material or fine concrete aggregates. It would be necessary to carry out a detailed search at the feasibility stage, in order to locate gravel deposits suitable for screening for use in the filter zones.

No clay or fine silts for use as core material have been located on this reconnaissance but it is possible that deposits similar to those found at Dam Site No. 17 occur within the area underlain by the amphibolite. Further search in the area of cultivated land to the north east of the site could also be profitable.

3.1.5 Dam Geometry and Field Survey

The alignment surveyed is shown on Figure No. 3.1. The main alignment crosses the river on a grid bearing of 40°/220° just upstream of the two main kopjies on each bank. Two bench marks have been established, at each end of this alignment, one on the left bank on the high ground beside the new road, and the other at the foot of a small kopjie on the right bank. The survey was extended on both banks as shown in an attempt to reach higher ground.

The ground profile on this alignment is shown on Figure No. 3.2. The ground levels shown are related by direct horizontal levelling in two directions to the nearest convenient trig. station BPP116, which is situated some 7.5 km to the south west of the site. There is an apparent discrepancy between the datum obtained from this levelling and that obtained from mapping for the area of the dam site, as can be seen from the plot of map contours shown on the profile. A correction of +3.0 m has therefore been applied to all survey levels for correlation with the reservoir capacity curves obtained from local cartographic data.

Assuming a retention level for the reservoir of 1,111 m (1,108 m field survey datum) a convenient site for the spillway is located on the left bank, on exposed granitic gneiss bedrock. In this location flood flows would discharge back into the river channel remote from the main dam, thereby limiting the cost of training works. The outlet works would be located to suit the purpose for which they are intended, but a suitable location is found on the left bank close to the present river channel. A fuse section would be built into the dam on the right bank, where further outcrops of bedrock are apparent.

3.2 Site No. 10 - Ntshe River (Figures 3.3 and 3.4)

3.2.1 General Topography

The Ntshe river in the are of the proposed site forms a well defined gorge in granitic rock, as it passes through a prominent range of hills about 35 km north of Francistown. The Tati river passes through the same hills about 6 km to the west and a promising dam site on that river at Timbale has already been investigated by B.G.A. Lund & Partner.

The Ntshe site is rather less obvious than the Timbale site, but is also a good dam site, suitable for the construction of a short earth or rock fill dam, or possibly of a concrete dam, depending on foundation conditions. On the left bank there is a prominent hill within close proximity to the river channel. On the right bank the ground rises more slowly to successive lines of low kopjies. Behind the left bank hill there is a narrow valley which at retention levels above about 1,100 m would require to be sealed by a short saddle dam.

The dam site is on private land within the Tati Concessions area. However apart from one farmstead on the left bank the reservoir area appears to be generally uninhabited. A gravel road from Francistown gives access on the right bank to within about 4 km of the site.

3.2.2 Geology

According to the Geological Survey Maps of the area the granitic rocks form part of the Timbale Granite Complex. There is no evidence of major faulting and jointing is the dominant structure. The outcrops of bedrock on both banks are formed of massive boulders.

The granitic bedrock is characterised by the massive blocks which constitute the outcrops. On the left bank these blocks are of the order of about 5 m or so between discontinuities. The ground lying between the rock outcrops is covered by about 2 m of decomposed rock consisting of red sands and quartzitic gravels overlying more resistant rock.

3.2.3 Engineering Considerations

It is not expected that the foundations will pose any particular problem other than in establishing a suitable cut-off. It can be expected that major trimming operations would be required in the outcrops, and it may be that some of the large outcrops will have to be excavated back to a constant slope to avoid sharp changes in bedrock level.

Execpt for the river channel it is not expected that major excavation would be required. The overburden and decomposed rock is probably of or the order of about 2m depth between the outcrops of granite, and once this material has been stripped away, reasonable foundations can be expected. These foundations would probably require minimal grout treatment to form a suitable cut-off.

3.2.4 Construction Materials

The river bed contains a considerable quantity of sand, which could be used for fine concrete aggregate or as bulk fill material in the shell zones. Further material suitable for the shell zones may be found in deposits on both banks of the river upstream of the alignment, though these may be rather shallow for economic working. Riprap and coarse aggregate could be quarried from any of the large outcrops of the granitic rock. The trimming off of the large blocks in the spillway area could also produce material suitable as riprap or concrete aggregate.

It has not been possible to locate suitable core materila in the area. Granitic rocks underlay a considerable part of this region and generally the weathered product is a sandy material rather than clay. The availability of suitable core material will clearly influence the choice of type of dam, and a detailed search of the area would be necessary at the feasibility stage. A detailed search for suitable gravel deposits for the filter zones will also be necessary though it is unlikely that these will prove difficult to find.

3.2.5 Dam Geometry and Field Survey

The alignment surveyed is shown on Figure No. 3.3. A bench mark was established on a small kopjie on the left bank separate from the main hill. From this bench mark the alignment crosses the river channel on a grid bearing of 63°/243° passing through the first line of kopjies on the right bank but intersecting the second and third. On the left bank the survey was extended to tak in ght valley beyond the main hill.

The ground profile on this alignment is shown on Figure No. 3.4. Ground levels are related to Trig Station BPT43 situated about 4 km south of the site. Taking into consideration the broken nature of the terrain at the site the correlation with map contours is reasonable and no correction is necessary.

Assuming a retention level for the reservoir of 1,096 m the most suitable site for the spillway is on the right bank adjacent to the first line of kopjies. An alternative site was considered making use of the natural channel between the left bank hill and the separate kopjie on which Bench Mark A was established, but this channel proved to be too narrow to accommodate the predicted floods. The outlet works can be established on either river bank as required, the principle consideration being foundation conditions. For an embankment dam, a fuse section can be incorporated in the right bank above the spillway section.

3.3 Site No. 13 - Tiaye River (Figures 3.5 and 3.6)

3.3.1 General Topography

This site on the Tiaye River is not well defined, but is marked by a profusion of small kopjies indicating the presence of shallow bedrock and of possible sites for a spillway. The river meanders in a wide valley, with rock outcrops on both river banks, and a rock bar crossing the river at the upper extremity of the site. On the left bank in particular the terrain beyond the main group of kopjies is very flat.

The site is situated on the Bosoli Ranch, owned by the Tati Company. Access to the site can be obtained by means of a track from Bosoli Siding via the ranch house. The reservoir area is uninhabited.

3.3.2 Geology

The site is located in an area underlain by granitic type rocks which according to the geological map are orthogneisses. The rocks are poorly foliated and bedrock is limited to the small kopjies on the left and right abutments, with overburden and completely decomposed rock occurring between the rock outcrops.

From the limited outcrops of the orthogneiss it is difficult to establish the main structural features of the site. The kopjies are formed of piles of massive boulders with some of the original jointing still discernable.

The overburden cover in the area appears to be mostly a sandy gravel.

3.3.3 Engineering Considerations

Examination of the surface geology indicates that the site is suitable for construction of a low embankment dam. While no conclusions can be drawn as to the likely extent of foundation treatment it is considered that a normal grouted cut-off and nominal foundation treatment would suffice. Major trimming operations would be required in the zones around the kopjies and it would be necessary to avoid sharp changes in slope between these features.

It is probable that a shallow core trench of about 0.5 m depth into rock would be sufficient. Between the kopjies the bedrock, although probably in a moderately weathered condition, may be found within about 2 m depth, except possibly in the river channel area itself.

It is probable that suitable foundation conditions for a concrete spillway could be obtained within reasonable depth in proximity to the kopjies.

3.3.4 Construction Materials

The granitic type bedrock in the kopjies would be suitable as either concrete aggregate or as riprap. Immediately downstream of the proposed alignment there are extensive gravel deposits and further deposits occur on the right bank beside the access track. The sandy overburden cover is also very extensive and would probably be suitable as fill material for the shell zones.

The potential sources of core material are rather more limited in extent. The survey found an area of about 2 ha on the right bank about 0.8 km downstream of the proposed alignment and there is a further area of shallow depth on the left bank 0.5 km upstream of the alignment. Both of these materials were sampled and found to be suitable for use as core material.

3.3.5 Dam Geometry and Field Survey

In order to avoid sharp variations in level in the core trench, there are advantages in selecting an alignment for an embankment dam which takes advantage of the high ground upon which the kopjies are situated but which avoids the kopjies themselves. In order to select the optimum alignment therefore, a tacheometric survey of the main section of the dam site was carried out from stations established at convenient points on the kopjies. The optimum alignment selected from this survey is shown in plan on Figure No. 3.5. In an attempt to reach higher ground, the survey was extended on both banks by traces cut as shown.

The selected alignment is shown in profile on Figure No. 3.6. The ground levels are related by direct levelling to Trig. Station BPS94 situated some 6.5 km to the north west. There is insufficient data to make a correction to these levels for correlation with local cartographic data as contours shown on local mapping are only at 50 ft. intervals.

There is no obvious location for the spillway, but assuming a reservoir retention level of 1,036 m the most suitable position is on the right bank adjacent to the main kopjie. In this position training works would be necessary to protect the downstream toe of the main embankment during flood flows. Outlet works could be established on either bank adjacent to the river channel, and a fuse section could be incorporated in the embankment on the right bank.

3.4 Site No. 17 - Tati River (Figures 3.7 and 3.8)

3.4.1 General Topography

The site is on the lower Tati river about 30 km south of Francistown and just downstream from the confluence of the Tati and Sekukwe Rivers. There is a gravel road from Francistown which gives access along the left bank of the Tati River, to within about 3 km of the Site.

The dam site is formed where the Tati River cuts through the end of the range of the Mpanipani Hills, which forms a prominent ridge on the left bank. On the right bank the topography is more subdued, but ground levels generally rise towards the west.

The left bank ridge terminates in a prominent knoll of exposed bedrock which would serve as a suitable left abutment for any dam alignment on this site. Between this knoll and the main ridge, there is a low saddle of exposed bedrock which provides a possible spillway site for reservoir retention levels above 940 m. Further to the west there are three additional low saddles which would require to be sealed at this retention level or above.

On the right bank of the river, opposite the left bank knoll, a minor tributary joins the Tati on the outside of a sweeping bend. Two possible dam alignments are therefore apparent, crossing the river either upstream or downstream of this tributary. The downstream alignment has the advantage that the tributary would then contribute to the reservoir volume, in addition to providing a marginally larger catchment area.

The dam site is on private land within the Tati Concessions area, and there are a number of private dwellings within the proposed reservoir area.

3.4.2 Geology

The range of hills cut by the river is formed of a series of amphibolites which according to the geological maps of this area are part of the Lady Mary Formation within the Bulawayan Group of the Early Precambrian. The rocks are strongly foliated with foliation or schistocity dips of between 70° and 80°. The rocks forming the ridge have a predominant NW/SE trend.

On the right bank, the amphibolite outcrop gives way to various gneisses of the Basement Complex within about 200 m or so from the river channel. These gneisses form the higher ground on this bank though they are covered generally by overburden formed by weathering.

There is no evidence of major faulting on the site which would adversely affect the site of the dam. The amphibolite is highly jointed but the depth to which the surface jointing extends could not be ascertained from the field reconnaissance. It is anticipated that very little excavation and trimming would be required to form the dam foundations though rather more would be required on the right bank than on the left bank, where bedrock is generally exposed on the surface. On both alignments amphibolite bedrock is exposed locally at river bed level. No indication of the state of the gneisses on the right bank could be obtained from the field reconnaissance.

3.4.3 Engineering Considerations

Both from topographical and geological considerations the site is considered to be favourable for the construction of an embankment dam. It is unlikely that there would be problems in establishing a suitable cut-off although on the left bank the ridge would have to be trimmed back to a uniform slope to avoid any sharp changes in core height. The jointed amphibolite on the left bank should be amenable to grout treatment and it is considered that a core trench would only require a minimum depth for keying in. On the right bank on both alignments the core trench may have to be excavated through rather deeper deposits of sands and gravels, though where the alignment crosses over into the gneisses, it is unlikely that the extent of foundation treatment will be any worse that than envisaged for the amphibolite.

3.4.4 Construction Materials

Suitable sources of concrete aggregates and riprap are limited to the amphibolite bedrock, which it is proposed would be excavated in the left bank spillway area. About 0.5 km downstream of the southern alignment on the left bank a large pocket (just over 2 ha) of dark silty clay occurs which could be considered as potential core material, and on the right bank at the confluence with the minor tributary between the two alignments a further area of about 10 ha of silty sandy clay is found to a depth of up to 1.75 m. Samples were taken from both areas during the field survey, and were found to be generally suitable as core material.

Quartzitic gravels are abundant on the surface along both alignments, but occur generally in small unworkable pockets. One possible deposit was located however at the west end of the southern alignment, and it is probable that further similar deposits can be found in the area of the dam site. Sand for use as bulk fill in the shell zones can be found in the river bed and on the right bank of the river opposite the confluence with the Sekukwe river, about 0.7 km upstream from the proposed northern alignment.

3.4.5 Dam Geometry and Field Survey

A survey of both possible alignments was carried out during the field survey. A bench mark common to both alignments was established on the rock knoll on the left bank. From this bench mark the upstream alignment crosses the river on a grid bearing of 144°/324°. On the far bank a further bench mark was established, and the survey was extended on a new bearing of 100°/280° towards high ground. The downstream alignment crosses the river on a bearing of 64°/244°, changing to 76°/256° after establishment of a further bench mark on the right bank. The survey was also extended eastwards to take in the low saddle between the rock knoll and the main left bank ridge and in addition a separate survey by barometric levelling was carried out for the three low saddles further along the east bank ridge. The location of these surveys is shown on Figure No. 3.7.

The developed profile of both alignments is shown in section on Figure No. 3.8. The survey levels are related by barometric levelling to Trig. Station BPT67 in the Mabethe Hills, approximately 6 km to the south east of the site. Reasonable agreement was obtained between these levels and those obtained from cartographic data and no correction is considered necessary.

Because of the rather smaller reservoir area impounded by the upstream alignment, a higher water retention level is necessary for this alignment in order to impound the same volume. This higher retention level is rather better suited than that for the downstream alignment to exploiting the natural channel on the left bank for use as a spillway. This spillway would take the form of a channel excavated through the rock saddle with a low concrete cill. The layout of the dam, particularly that for the upstream alignment, would be such that the spillway discharge would be conveyed well clear of the toe of the dam, and training walls downstream of the spillway should not be necessary.

The outlet works for both alignments could best be located on the right bank adjacent to the river channel where rock is at or close to the surface.

3.5 Site No. 33 - Motloutse River (Figures 3.9 and 3.10)

3.5.1 General Topography

This site is situated about 13 km to the east of the village of Tobane and about 25 km downriver from the Tobane dam site surveyed by B.G.A. Lund & Partner. There are tracks on both banks of the river which connect to Tobane, and beyond to Selebi Pikwe.

The high ground on the north bank of the river is formed by the Lerala Hills which are a prominent outcrop of basic rock. On the south side of the river a low ridge leads south westwards towards the Majana-a-Diphiri Hills. At the end of this ridge and about 1 km distant from the river, there are a number of low gravel banks which probably mark outcrops of bedrock.

The river channel is about 130 m wide at this location, with high banks and a bed of coarse grained uniform sand. The north bank of the river between the river channel and the foot of the Lerala Hills is a low lying area of dense acacia woodland, comprising possibly an old infilled channel of the river. On the south bank of the river, where the soil is sandy, the predominant vegetation is mopani scrub.

In the reservoir area, both banks of the river are cultivated and impoundment would involve the relocation of the village community of Sebalwe, situated about 4 km upriver from the dam site.

3.5.2 Geology

A major part of the proposed dam site is underlain by granitic gneisses of the Basement Complex, with sedimantary rocks which may form part of the Karroo Series, and basic rock of the Lerala Hills forming the remaining rock types. Outcrops of the sedimentary Karroo Series are limited to the high ground on the right bank, though on the left bank the Karroo may be present at the base of the Lerala Hills. A major fault forms the boundary between the Lerala Hills and the gneisses. The fault trends east-west and is located about 1 km from the river channel. In the area on the right bank proposed for the spillway the bedrock appears to be formed of arkose or quartzites and even gneisses of the basement complex. Elsewhere the bedrock appears to be a form of banded gneiss which shows a strong foliation with a group of fine to medium and medium to coarse grained types which in some areas are granitic.

Between the river bank and approximately 1 km from the left abutment, there is an area of grey silty clays, the extent of which is difficult to determine. The boundary to this feature may be a former bank of an old channel or a scarp.

Limited exposure of the rock in small gullies on the right bank indicates that generally in the higher elevations of the abutment weathered bedrock is found within half a metre from the surface. However, in the low lying area forming the river banks alluvial material forms the overburden.. There are no bedrock outcrops visible in either bank of the river and in the left bank there is evidence of an old infilled channel.

3.5.3 Engineering Considerations

From the evidence available from surface geology the site would appear to be suitable for construction of a large embankment dam. However the site has two major structural features which would have a bearing on the foundation conditions affecting the feasibility of such a project. Firstly, the old channel or scar feature on the left bank will certainly require detailed investigations, because if present it could mean that a deep cut-off could be required for about 1.5 km including the river channel and the alluvial zone on the right bank. Secondly, the extent of the major fault zone within the proposed dam foundations would have to be determined. It could be that this zone is very permeable and extensive and therefore extra foundation treatment would probably be required, including dental concrete work.

It is probably that over most of the right bank minimum treatment would be required to form an adequate cut-off. However considerable excavation may be required on the spillway site due to the capping of Karroo rocks on the granitic gneisses.

3.5.4 Construction Materials

Bulk fill for the shell zones of the dam can be found as sand from the river channel and as red sandy silts and red sands from both banks of the river upstream of the dam. On the left bank of the river, the area of grey silty clays which have been proved to a depth of 1.0 m should provide suitable core material.

Gravel for use in the filter zones and for slope protection was found in workable deposits on the right bank, though it is unlikely that there is sufficient material in this form to meet total requirements. Further material can be obtained by crushing rock scree or quarried rock from the Lerala Hills. Concrete aggregate, rockfill and riprap can also be obtained from the same source, and also from spoil from the spillway channel which it is proposed would be excavated in the quartzites and gneisses on the right bank.

3.5.5 Dam Geometry and Field Survey

Potential flood flows at this site are large, and in order to minimise the cost of spillway training works, an alignment was chosen for survey that would allow spillway discharge on the right bank into an adjacent valley remote from the main embankment. Alternative alignments further upriver were not considered for this reason though they might well prove in practice to have rather more favourable foundation conditions.

The selected alignment was determined by the establishment of a bench mark on a gravel ridge on the right bank at approximate reservoir retention level. From this bench mark the survey alignment crosses the river on a grid bearing of 163°/343° towards the Lerala Hills. In addition the area between the gravel ridge and the main right bank ridge was surveyed for use as a possible spillway site. These surveys are located in plan on Figure No. 3.9.

The developed valley profile on this alignment is shown in section on Figure No. 3.10. The survey levels have been related by barometric levelling to Trig. Station BPT177 on Sansale Hill about 7 km to the south east of the site. As can be seen from the plot of map contour levels shown on the section, thre is a small but consistent discrepancy between levels obtained from the survey and those obtained from mapping. Consequently a standard correction of +2.0 m has been applied to all field survey levels for the purpose of correlation with cartographic data.

The main features of a possible embankment dam are shown on the section on Figure No. 3.10. Assuming a reservoir retention level of 763.0 m the main spillway 450 m in length is shown on the right bank. A fuse section to act as a safeguard against overtopping would be built into the embankment above the exposed left bank bedrock. The outlet works are shown near the river channel on the right bank, where shallow bedrock can be anticipated.

3.6 Site No. 40 - Motloutse River (Figures 3.11 and 3.12)

3.6.1 General Topography

This site is some 90 km downriver from the previous site, and is located at a point where the river passes through a narrow gorge between high cliffs of basalt, the rock which characterises the topography in this part of the country. The river channel at this point is only about 75 m wide, with high banks and a bed of coarse grained uniform sand. It is probable that a local increase in river gradient through the gorge compensates for the reduction in river channel width.

The basalt cliffs on the right bank of the river are about 60 m high, those on the left bank being slightly lower, through still reaching the same elevation within close proximity to the river. Further away from the river the land remains high though on the left bank there is an extended saddle, some 1.5 km in length, which would require to be sealed for reservoir top water levels above about 640 m.

The river gorge through the basalt is divided into two lengths by clefts in the left and right bank cliffs approximately opposite to each other. Alternative dam alignments are therefore either upstream or downstream from these clefts. On the right bank the cleft opens out in such a way that the cliffs upstream and downstream are connected to the main rock massife behind only by narrow saddles.

The site is extremely remote, and there are no recognised roads in the area. Access to the site can be obtained from the Tuli Block by a track which follows the river upstream on the south bank, though this track peters out just upriver from the gorge. There appears to be a limited amount of cultivation within the reservoir area, though there are no communities of any size.

3.6.2 Geology

Basalts of Upper Karroo age crop out extensively in the area, but the rock is only found in boulder form and no prominent or massive outcrops have been observed during the reconnaissance. Normally the individual flow units have a characteristic blocky base and top, the latter being very porous due to the increase of vesicles or gas cavities. At the proposed site these types of basalts are very evident with secondary mineralisation infilling some vesicular layers. The more crystalline varieties are also present, but it is not possible to distinguish the number of flows or layers within the elevations of the site. No evidence of ash or pumice bands have been observed in the valley slopes, and generally below the boulder zone indications are that only basalt flows occur.

The evidence from the boulder zone suggests that it is probable that the basalts are highly jointed and that jointing would be persistent within the valley slopes below the dam foundations. The distribution of boulders from the different types of basalt suggests that the proposed dam abutments are formed of rocks comprising at least five different lava flows, varying from purple, black coloured crystalline rock to the amygdaloidal vesicular type of rock.

3.6.3 Engineering Considerations

The field reconnaissance indicated that the site is generally suitable for construction of a large embankment dam, or possibly of a concrete dam, depending on foundation conditions. However the evidence from surface geology is insufficient to provide a very thorough assessment of the state of the basalts. In particular the joints in the basalts represent planes of leakage, while it is possible that some of the vesicular layers are pervious. Detailed site investigations would be required in the area of the dam foundations in order to identify the joint patterns and the locations of the vesicular layers and also to determine their permeability. Extensive grouting would probably be required to seal the joints and pervious layers and it would be advisable to include grout tests as part of the site investigations.

Indications from the present outcrops are that about 2 m depth of the bouldery zone would require stripping before bedrock is found. Once the main bedrock profile is determined it is considered that only a shallow core trench of about 0.5 m depth would be required.

In view of the likely jointed condition of the basalt below the foundations it is considered that an extensive grout curtain would be required to a depth of at least the height of the dam. If the depth of basalt below the foundations is not as great as anticipated, and the underlying sandstones are present then the grout curtain could probably be terminated at the sandstone layer.

3.6.4 Construction Materials

The basalt is the only potential source of material for concrete aggregate, riprap, or rockfill in the region. However, the amygdaloidal types would have to be investigated and tested for strength and permeability particularly if these layers are more numerous than the crystalline flows. Also, alkali reactivity tests would be required to check that the secondary mineralisation within these amygdaloidal layers contains no deleterious minerals.

There is only a limited source of material for bulk fill for the shell zones of an earthfill embankment dam. River sand is the principle source though if material were taken from the reservoir area only, then haulage of up to 20 km would be necessary in order to meet total requirements. There is an area of up to 35 ha of cultivated land, consisting of basalt gravels, sands and sandy silts on the left bank about 1.5 km upriver from the site. Within this area there are limited pockets of plastic material suitable for use as impervious core material. There are abundant deposits of coarse gravel and cobbles, suitable for use as filter material and for slope protection.

3.6.5 Dam Geometry and Field Survey

Dam alignments for both the upstream and downstream sections of the gorge were surveyed. The upstream alignment is marked by a bench mark established on the southern cliff from which the alignment crosses the river on a grid bearing of 37°/217°. At this point the gorge is at its narrowest. The downstream alignment is marked by a similar bench mark on the southern cliff and crosses the gorge on a grid bearing of 34°/214°. Both surveys were extended on the south bank to take in the two narrow-rock saddles which were considered for use as possible spillway sites. The locations of these surveys are shown in plan on Figure No. 3.11.

Profiles through the gorge on the two alignments are shown in section on Figure No. 3.12. The survey levels are related by barometric levelling to Trig. Station BPT310 some 9 km to the south west. There was a reasonable measure of agreement between these levels and those obtained from cartographic data.

The major feature of any embankment dam constructed at this site would be the spillway channel which for both alignments is shown on Figure No. 3.12 as a cutting through the rock saddles on the south bank. For a retention level for the reservoir of 628.0 m this cutting would be about 30 m in depth and 75 m wide at the bottom. There are alternative spillway sites on the north bank, which appear to be less attractive but which should be considered in more detail at the feasibility stage. The alternative of a concrete buttress or gravity dam, with an overtop spillway constructed in the gorge would have the advantage that a rather lower dam crest level could be adopted.

The extended saddle on the left bank was not surveyed during the field reconnaissance. However from the 1 in 50,000 scale mapping it would seem that for an embankment dam in the gorge with a crest level of +646 m, a saddle dam some 1.5 km in length and up to 5 m in height would be required along the ridge. As a protection against overtopping of the main dam during exceptional floods, a fuse section would be incorporated in the saddle dam.

It is assumed that the major part of an embankment dam in the gorge could be constructed within one working season. If this were not the case, then facilities would need to be provided for by-passing flood flows during the wet season, and this would add substantially to the cost of the dam. Limited by-pass facilities could be incorporated in the outlet works which it is assumed would be constructed on the left bank adjacent to the river channel.

3.7 Site No. 42A - Thune River (Figures 3.13 and 3.14)

3.7.1 General Topography

This site on the Thune River, about 30 km from its confluence with the Motloutse is in a remote part of the eastern part of the country. The nearest village is Molalatau about 12 km to the north, from which there is a track leading south towards the Tuli Block which passes within about 3 km of the site.

The principal feature of the site is Maekoane Hill, a sandstone cliff on the right bank which acts as a barrier to a southerly meander of the river channel at this point. The same sandstone feature can be traced running in an easterly direction through the Ngolo Hills and the Seswe Hills and terminating at Lekono Hill at Site No. 70 on the Motloutse River, although the succession of sandstones would probably be different.

Maekoane Hill separates the Thune valley from the valley of an adjacent tributary, the Lekgolwe river. Connecting Maekoane Hill to the adjacent high ground between the two rivers there is a low saddle of exposed sandstone bedrock which provides a possible spillway site discharging into the Lekgolwe river for a dam alignment abutting on the hill. On the left bank of the river, opposite Maekoane Hill, the terrain rises more gradually in a series of low ridges which probably outline the strata of the underlying sandstone bedrock.

There are a number of dwellings within the reservoir area, and the red sandy soils on the left bank are cultivated fairly extensively. The river channel which is about 40 m wide has cut into these left bank soil deposits and at one point undercutting of the bank has exposed a 9 m depth of soil strata including sands, sandy silts and calcrete gravels. Sandstone bedrock is exposed at a number of locations in the bed of the river.

3.7.2 Geology

The Thune river in the area of the proposed site has breached a sandstone/dolerite sequence which probably forms part of the Karroo system. The site is dominated by the steep-sided right bank which is formed of a dolerite sill lying between two sandstone groups. The reconnaissance found no evidence of the dolerite occuring in the left bank. The river channel is formed in sandstone but on the right bank this gives way to dolerite about 5 m. above the river bed. The dolerite is overlaid by red, highly jointed, medium bedded sandstones at about 20 m height above the river bed.

Isolated blocks of sandstone crop out in the river bed, but it could not be determine from these outcrops whether or not the sandstone was highly jointed and medium bedded like the upper sandstone group. The bedrock sequence found in the proposed right abutment is not present in the rock outcrop of the left bank. The left abutment is much flatter and bedrock outcrops are more limited.

3.7.3 Engineering Considerations

The site is considered to be generally satisfactory for the construction of an embankment dam. However particular attention may have to be paid to the treatment of the foundations. If the jointing found in the upper sandstone is characteristic of the sandstone below the dolerite then a positive cut-off would be required to overcome the relatively high permeabilities normally associated with such a condition.

Exploratory works at the site should put particular emphasis on permeability tests during preliminary investigations. The possibility of extensive grout treatment would also have to be considered if high permeabilities are proved. It has been assumed that a grouted cut-off would be more practical than a deep core trench, though on the left bank adjacent to the river channel where a considerable depth of overburden is likely to be encountered, it will be necessary to allow for excavation of a deep cut-off trench.

The steepness of the slope in the right abutment will have to be taken into consideration in regard to changes in height of the core between the sandstone/dolerite interfaces. It is unlikely that more than 1 to 2 m will have to be removed in clearing off the overburden and scree. Once the overburden and broken bedrock have been cleaned off a shallow core trench could be established in the bedrock.

3.7.4 Construction Materials

The site is abundant in sandy soils suitable for use as bulk fill in the shell zones of an earthfill embankment dam. However the field survey was unable to locate suitable sources of plastic soils for the impermeable core or gravel for the filter zones. An area of about 5 ha on the left bank upstream of the site was sampled for suitable core material but the plasticity indices were generally too low.

The sandstone is slightly friable and is likely to have too low a crushing strength to be suitable as a coarse concrete aggregate but could be considered for either rockfill or riprap. Due to the flaggy nature of some horizons it could well be that selective quarrying would be required if it became necessary to incorporate the sandstone into the design. The dolerite could provide either riprap or suitable concrete aggregate and a possible face for developing a quarry is found about 1 km upstream from Maekoane Hill.

3.7.5 Dam Geometry and Field Survey

The position of the left abutment of the dam is dictated by the presence of a low sandstone ridge about which the river channel forms a wide sweeping bend at about 1 km distance. A bench mark was established on this ridge, and the main survey was carried out on an alignment with a grid bearing of 171°/351° in line with the western end of Maekoane Hill. This survey was extended northwards on the same bearing towards higher ground. A second alignment on a grid bearing of 29°/209° was also surveyed. This alignment which terminates in a ridge forming a westward extension of the sandstone/dolerite feature of Maekoane Hill has the disadvantage that the saddle spillway site on the south bank could not be used, but it offers possible advantages in terms of ground contours on the left bank. In addition a tacheometric survey of the south bank saddle spillway site was carried out. The location of these surveys is shown on Figure No. 3.13.

A profile along both alignments is shown in section on Figure No. 3.14. The survey levels which are related to Trig. Station BPS66 on the top of Maekoane Hill show reasonable agreement with ground contour levels taken from the local mapping.

The main features of an embankment dam on either of the two alignments are shown on Figure No. 3.14. Assuming a reservoir retention level of 657 m the spillway for the downstream alignment can be conveniently accommodated as a 415 m long concrete weir on the right bank saddle. Very little excavation would be required and training works would be kept to a minimum. For the upstream alignment a spillway site on the right bank sandstone/dolerite sequence is probably preferable to one on the left bank where the quality of the sandstone bedrock is uncertain. However a considerable volume of rock excavation would be required in order to accommodate a 145 m wide by 7 m deep channel, though the spoil would be used partly for riprap and filter material. Fuse sections have been shown for both alignments as a safeguard against overtopping. The outlet works for both alignments can be most conveniently sited on the right bank sandstone bedrock adjacent to the river channel.

3.8 Site No. 46 - Lotsane River (Figures 3.15 and 3.16)

3.8.1 General Topography

The Site on the Lotsane River is located in an area where the northern escarpment slope of the Tswapong Hills gives way to the more undulating topography formed by Basement Complex bedrock. The Lotsane River in its route from Palapye to the village of Maunatlala follows the base of the escarpment and at a point about 2 km to the west of Maunatlala the width of the river valley is constricted by a rounded hill of Basement Complex bedrock on the north bank. Within the narrow valley between this hill and the main escarpment there is a possible site for a low embankment dam.

The river channel at this point is only some 15 m in width, with banks up to 4 m high.

Maunatlala can be reached by an earth track from Palapye, and from Maunatlala there is a little-used track along the south bank of the river which passes through the site. The red sandy soils are fairly widely cultivated in the area, though within the confines of the dam site there is dense thorn scrub, and there appears to be only limited cultivation within the reservoir area.

3.8.2 Geology

A major fault separates the quartzitic sandstones and conglomerates forming the Tswapong Hills from the Basement Complex. At the proposed site outcrops of bedrock are limited but those occuring are of granitic gneiss of the Basement Complex. The quartzites and conglomerates which occur at higher elevations above the fault on the right bank form part of the Waterberg System. On the right bank below the main escarpment of the Tswapong Hills the ground is covered by scree and sandy overburden material and it is not possible to determine precisely where the fault runs, but it passes within the foundations of the dam near the end of the right abutment. The granitic gneiss which crops out in the river channel and both banks occurs in a highly weathered condition and in some areas is completely decomposed. Extensive deposits of sandy overburden cover both the left and right abutments.

3.8.3 Engineering Considerations

From the reconnaissance it has not been possible to determine whether the major fault would pose foundation problems in the higher elevations of the right abutment. In this area only large scree material is found but in all probability additional foundation treatment would be required.

The main dam foundations would be formed in the granitic gneiss and it is not expected that there would be any major problem in establishing a suitable cut-off in the weathered zone. The depth of the core trench would be very much controlled by the depth of weathering of the granitic gneiss. Normal grouting could be expected below this zone to establish the cut-off.

Stripping of the sandy overburden to the top of the weathered rock could be of the order of about 2 metres.

3.8.4 Construction Materials

The quartzitic sandstone and conglomerate boulders found in the scree slopes are the best sources of coarse concrete aggregate and rockfill within easy access of the site. Depending on the depth of the sandy overburden the best source of sand for bulk fill would be from the right bank below the main screes. No suitable sources of core material were located in the dam site area, nor was there any sign of gravels suitable for screening for use in the filter zones. Calcrete gravels were located in the right bank downstream of the site, though some research would be necessary before these could be considered for use in the dam.

3.8.5 Dam Geometry and Field Survey

Two possible alignments were surveyed as shown on Figure No. 3.15. The upstream alignment crosses the river at the confluence with a minor tributary on a grid bearing of 10°/190°. The downstream alignment, which was sited to take advantage of possible higher ground levels on the left bank, crosses the river on a grid bearing of 25°/205° and on the left bank changes direction to 2°/185°. In practice the greater length of the downstream alignment proves to be the more important factor, and the earthwork volume for the downstream alignment is greater than for the upstream alignment.

The ground levels obtained in the field survey are related by barometric levelling over a range of readings to Trig. Station BPT337, situated 13 km to the south east of the site. There is a nearer Trig. Station, BPS30, located on the top of the escarpment above the site, but the vertical interval between this station and the site is greater than that normally recognised for accurate barometric levelling, and in addition the Trig. Station was not visible from the site, for the purpose of trigonometrical levelling. There is an apparent discrepancy between the datum obtained by barometric levelling and that obtained from mapping of the area, as can be seen from the plot of map contours shown on Figure No. 3.16. *A correction of -16.0 m has therefore been applied to all* field survey data for the purpose of correlation with the reservoir capacity curve obtained from local cartographic data.

The principal features of an embankment dam are shown for both alignments on Figure No. 3.16. The spillway is shown located on the left bank where excavation in the granitic gneiss bedrock would be required in order to accommodate a 270 m length of concrete weir. Training works downstream of the spillway would be necessary in order to protect the downstream toe of the embankment during flood flows, and in this respect the spillway geometry for the downstream alignment would be marginally more favourable than that for the upstream alignment. The outlet works would be located at low level on either bank of the river depending on suitable foundation conditions. A fuse section would be incorporated in the embankment adjacent to the spillway.

3.9 Site No. 55 - Taupye River (Figures 3.17 and 3.18)

3.9.1 General Topography

The Taupye River flows through an area of rich pasture and cultivated land to the east of Mahalapye. The terrain to the east of the railway is generally featureless but at the site there is a single prominent hill, Budungwe Hill, situated on the left bank of the river, which marks the only feasible dam site on this stretch of the river. The river channel at this point is about 50 m wide with banks 5 m high and a bed of coarse grained uniform sand. A short distance upriver from the hill there is an outcrop of Basement Complex bedrock crossing the bed of the river. The banks of the river have been undercut at a number of locations to reveal strata of sandy silts, calcrete gravels and calcified bedrock.

The site is about 30 km from Mahalapye, and can be reached by an earth track which follows the right bank of the river.

3.9.2 Geology

The favourable topographical feature which identifies the site is located in the much folded and highly metamorphosed rocks of the Basement Complex. Underlying the whole area are migmatites and gneisses but on the proposed alignment a metaquartzite forms Budungwe Hill. Some pegmatites and possibly granulites are found in the gneisses and migmatites. The metaquartzites found in the proposed left abutment are typically white coarsely crystalline rocks composed mainly of vitreous quartz. Bedrock exposures in the right bank are not so prominent and are limited to weathered gneiss outcrops at various localities. The contact between the metaquartzite and the gneisses has not been identified, but the presence of a granitic gneiss outcrop in the bed of the river upstream from the site would indicate a contact zone on the left bank near the base of the hill.

In the left abutment metaquartzite bedrock crops out within about 400 m of the river, whereas in the right bank completely decomposed material and weathered gneisses are usually found beneath the overburden. These decomposed gneisses have a strong foliation which gives the rock a schistose structure, with a regional trend approximately NW/SE.

The metaquartzite forming the hill on the left bank occurs in a bouldery condition at the surface, and no indication of the jointing within the main rock mass has been established. The right bank contains a well marked feature which trends approximately N-S and probably represents a fault zone. The overburden cover in the area is sandy with calcrete zones, and generally appears to be less than 2 m in thickness.

3.9.3 Engineering Considerations

The river channel roughly divides the proposed alignment into two zones of probably different foundation conditions. The left abutment formed of highly jointed metaquartzites would probably require a grouted cut-off, whereas a deep cut-off trench would be more appropriate for the more deeply weathered rock on the right bank. It would also appear that only a limited amount of cleaning off and trimming would be required in the abutment underlain by the metaquartzite but much deeper cleaning off would be required in the right abutment.

It is assumed that granitic rocks underly the channel area. If there is an abrupt contact between the metaquartzite of Budungwe Hill and the more weathered rocks of the granitic gneisses and migmatites, then more trimming back of the quartzite would be required to avoid a sharp change in the level of the core trench.

The sound metaquartzite should pose very few problems for the establishment of a spillway structure on the left abutment.

3.9.4 Construction Materials

Clay soils required as core material for an embankment dam are not very prominent near the site. An area of about 15 ha was located on the left bank about 5 km upstream from the site but suitable material was proved to a depth of only about 60 cm. A more detailed search would be necessary at the feasibility stage.

There are adequate quantities of bulk fill material for the shell zones in the form of sand from the river channel and sands and sandy silts from both banks upstream of the site. Gravels and cobble boulders for filter zones and slope protection can be found as scree material on the left bank. Concrete aggregates and riprap can be obtained by crushing the metaquartzite boulders from Budungwe Hill.

3.9.5 Dam Geometry and Field Survey

The alignment for an embankment dam on the site is determined by the hill on the left bank and the nearest high ground on the right bank, which lies approximately to the north west. The field survey was therefore carried out on an alignment on a grid bearing of 124°/304° passing through the centre of the hill. Bench marks were established on this alignment beside the track near the top of the right abutment and on the right bank beside the river channel. The details are shown on Figure No. 3.17.

The survey levels were related to Trig. Station BPS12 on the top of Budungwe Hill by means of trigometrical levelling using a base of 94 m combined with direct horizontal levelling to the site. The profile along the alignment is shown in section on Figure No. 3.18. Map contours for the area are shown at only 50 ft intervals, but these have been plotted on the profile, and there is again a clear discrepancy between these and the survey levels. A correction of +6.0 m has therefore been applied to the survey levels for the purpose of correlation with the reservoir capacity curve.

The principal features of an embankment dam on the site are shown on the profile. These include a 140 m length of spillway constructed on the left bank metaquartzite bedrock, a fuse section in the embankment on the right bank and outlet works close to the river channel on the left bank.

3.10 Site No. 56 - Bonwapitse River (Figures 3.19 and 3.20)

3.10.1 General Topography

This site is situated on the Bonwapitse River about 25 km south of Mahalapye and about 8 km downstream from the point where the river passes under the main road and railway. The level of the riverbed at the railway bridge (937.4 m) is well above the probable maximum retention level for the reservoir, and the bridge would not therefore be affected by the construction of a dam.

As for the previous site this site is identified by a prominent hill on the left bank of the river located in an otherwise featureless terrain. Opposite the hill, though some 2.5 km distant from it, there is a low ridge which extends in a north westerly direction across the main road and railway. On the left bank beyond the hill there is some 2 km of low lying land before higher ground is again encountered.

The river channel at the site is only some 10 m wide with banks little more than 1 m high and a bed of silty sand and gravel.

3.10.2 Geology

The river in the area of the site cuts through a series of quartzitic sandstones, arkoses, dark flaggy shales and banded ironstones of the so called Shushong Series which are Precambrian sediments. The shales crop out extensively on the right bank ridge about 1.5 km from the river and the area between the ridge and the river is probably underlain by these softer rock groups. The terrain in which the left abutment is found is less featureless with quartzitic or fused, intensely metamorphosed quartzitic sandstone outcrops. At the proposed site the left bank is underlain by a quartzitic type rock, forming the high ground and a series of dark flaggy shales and banded ironstones. On the right bank both ironstones are found as well as isolated outcrops of arkose. There are no distinctive fault features at the site, but the left bank ridge is offset to the ridge at the extreme right abutment and it is possible that the river follows a fault.

The quartzitic type rock forming the hill on the left bank occurs in a broken and bouldery condition at the surface but isolated outcrops suggest that this feature is very superficial. No indication of the condition of the arkoses, flaggy shales and banded ironstones could be obtained from the outcrops in the area, though on the right bank ridge the shales are highly jointed.

The river banks are formed of alluvium and within the river bed bedrock is only partly exposed in the vicinity of the left bank hill. The flatter areas on the right bank contain calcrete deposits.

3.10.3 Engineering Considerations

The left bank is different topographically from the right bank, due mainly to the quartzitic rock forming the hill. In the preparation of the foundations the difference between the quartzites and shales may reflect in sharp changes in rock level in the foundations. The presence of banded ironstones and softer shales may also give similar problems. Essentially the foundations will be in the form of a deep cut-off trench. The shales and banded ironstones should form a suitable foundation in which an effective cut-off can be established.

It is difficult to assess conditions in the flat area on the right bank but it is assumed that beneath the overburden and calcrete, the thickness of which is unknown, an effective cut-off can be established by a deep core trench.

The left bank ridge provides the most suitable foundation conditions for the spillway.

3.10.4 Construction Materials

The only major source of potential core material located during the reconnaissance is in the vicinity of the road and railway bridge, about 7 km upstream on the left bank. Smaller pockets were also found about 3 km distant on the right bank but these are generally too small to be workable. It is possible that other sources may be available on the right bank such as clays, silts or sands, from the weathering of the flaggy shales group. Potential sources of riprap and concrete aggregates are limited to the left bank ridge of quartzitic rocks. There is an abundance of bulk fill material for the shell zones of an embankment dam in the form of sands and sandy silts, principally on the right bank.

3.10.5 Dam Geometry and Field Survey

The location of the survey alignment which is shown on Figure No. 3.19 is dictated by the advantages to be gained by locating the spillway for an embankment dam on the sound quartzitic bedrock at the left abutment.

A bench mark was established on the left bank of the river at the base of the quartzite outcrop, and an alignment across the river was set out on a grid bearing of 134°/314°. A further bench mark was established beside a track crossing the site, and the line was then extended on a bearing of 115°/295° towards the nearest high ground on the right bank ridge. In addition a survey was carried out by barometric levelling of the saddle to the east of the left bank hill and the saddle level was found to be not less than 938 m elevation, i.e. the proposed dam crest level.

The profile across the site is shown in section on Figure No. 3.20. The levels are related to a bench mark established by John Burrow and Partners on the road bridge over the Bonwapitse river. On the gently sloping right bank there is reasonable agreement with the 3050 ft map contour, but agreement is less good on the left bank where the terrain is more rugged. We have assumed that no correction is necessary for correlation with the reservoir capacity curve - if a correction were to be made it would increase the height of the dam. The principal features of an embankment dam assuming a reservoir retention level of 933.0 m are shown on the profile.

3.11 Site No. 61 - Kolobeng River (Figures 3.21 and 3.22)

3.11.1 General Topography

The Kolobeng river is one of the principal tributaries of the Metsemothlaba river which flows to the north west of Gaborone. The Kolobeng drains the area of rocky hills to the west of Gaborone, and the dam is located at the point where the river leaves the hills and enters the wide valley of the Metsemothlaba, about 2.5 km to the west of the village of Kumakwane.

At the dam site the river flows in a steep sided valley of exposed quartzite bedrock with rocks and boulders on the surface. The river channel is only some 5 km wide but at the time of the field survey there was a flow in the river estimated at about 0.09 m^3/sec, and there are reports that the river has flowed continuously in recent years. The river flow was traced as far as the confluence with the Metsemothlaba where it was estimated at 0.075 m^3/sec, but the flow had disappeared by the time the Metsemothlaba had reached the road bridge on the Gaborone-Molepolole road. There appears to be a rock bar across the river at this point which would indicate that the Kolobeng water had by then been lost to recharge.

The dam site can be reached by an earth road from Gaborone, though vehicle access up the Kolobeng valley is not possible. The reservoir area appears to be generally uninhabited.

3.11.2 Geology

The site is located in an area underlain by pink quartzites which probably form part of the Waterberg System. These quartzites form a well bedded series and are generaaly lithologically uniform. The whole succession is very highly jointed and dissected by faults. The area under consideration is bounded to the north and south by two major NW/SE trending faults which dissect the area into an elongated block. Within this block subsidiary or splinter faults occur and one such fault determines the course of the river between the road and the sharp bend in the river about 750 m upstream. Other lineations or small faults dissect the left and right banks upstream of the bend. Probably some of the smaller lineations are joint controlled. The predominant structural trend is NW/SE.

3.11.3 Engineering Considerations

The site is considered to be suitable for construction of a small concrete or embankment dam on a number of possible alternative alignments. However the faults and joints in the quartzites represent planes of leakage and irrespective of topographical considerations the cost of foundation treatmentmay prove to be the deciding factor in the choice of alignment. Where faults occur in the foundation additional treatment will probably be required such as dental concrete work. It is likely that only minimal stripping would be required to clear off the loose blocky quartzites and scree, and except for the river channel, foundations could be expected within a depth of about 2 m. Only a shallow core trench is envisaged, but a deep grout curtain would certainly be required below the trench to establish a suitable cut-off. The rock will certainly provide a suitable foundation for the spillway structure, but will require extensive grout treatment below the excavation levels. Rock surfaces in the spillway channel may require to be protected in order to avoid being eroded along the joints.

3.11.4 Construction Materials

The quartzite should provide suitable material for riprap, rockfill and gravels for filter zones and for slope protection. Suitable material for concrete aggregates could be obtained possibly by selective quarrying though in practice a cheaper product could probably be obtained from commercial quarries in this area.

No suitable core material was found in the area during the field survey. The soils to the north of the site are generally non-plastic sands. However plastic soils were found beside the Metsemothlaba river at Site No. 62 and it is possible that further deposits can be found down river. The sand deposits to the north of the site would be suitable as bulk fill material or possibly as fine concrete aggregate.

3.11.5 Dam Geometry and Field Survey

A plan of the site is shown on Figure No. 3.21. There are a number of alternative dam alignments possible, depending on the retention level to be adopted for the reservoir, which would determine the spillway height and location. Three possible alignments were surveyed. The upstream and central alignments would have possible spillway sites located in the side valleys on the left bank between Stations G and E and Stations E and D. The downstream alignment would have a possible spillway site on the right bank ridge between Stations B and C. The valley profiles along all three alignments are shown in section on Figure No. 3.22.

The survey levels are related to Trig. Station BPS117 situated on the top of Kolobeng Hill to the west of the site. The level was transferred to the site by means of trigonometrical levelling to a base on the road, combined with direct horizontal levelling. Map contours for the site are at 50 ft intervals only, but there is a reasonable measure of agreement between these and the survey levels.

As can be seen from the profiles on Figure No. 3.22 the river valley falls quite steeply between the three alignments, which tends to favour the upstream alignment. Subject to a detailed survey of the reservoir area, the upstream alignment is therefore preferred. The principal features of an embankment dam on this alignment are shown on Figure No. 3.22.

3.12 Site No. 62 - Metsemotlhaba River (Figures 3.23 and 3.24)

3.12.1 General Topography

This site on the Metsemotlhaba river is only 20 km by road to the west of the previous site. Unlike the Kolobeng, the Metsemothlaba river has not flowed continuously in recent years, but like other rivers in Botswana it comprises a broad channel of coarse grained uniform alluvial sand, subject to intermittent flow during the wet season.

The dam site is 4 km to the south west of Thamaga on the Moshupa road. At this point the river flows in a wide valley between the Sephatlhaphatlha Hills to the west and the Sesitajwe Hills to the east. The site is identified on the left bank by a small kopjie at a point where the river channel takes a sharp turn to the right. On the opposite bank on the inside of the curve there is low lying area about 300 m wide between the river and a domed outcrop of bedrock. On the left bank beyond the kopjie the terrain is flat as far as the road but then rises to a line of kopjies following a ridge between the Mestsemothlaba and Kolwane rivers. On the right bank the ground rises steadily with numerous outcrops of rock and broken boulders.

In the reservoir area, between the rock outcrops the sandy soils are widely cultivated and there are a number of dwellings though no communities of any size. The gravel road between Thamaga and Moshupa would probably be affected over a short length if a dam were to be constructed on this site.

3.12.2 Geology

The range of low lying hills cut by the river is underlain by granitic rocks which are part of a major granite instrusion. Where the granite outcrops it is found in a massive form, typically as convex domes, and generally occurs in this form in an unweathered state. Jointing is more prominent on the left bank and the major outcrops are found as piles of rounded boulders. On the right bank the major outcrop is in a convex dome and bedrock occurs in a relatively unweathered state. The strong vertical fissures found on the left bank are not so evident on the right bank outcrop.

3.12.3 Engineering Considerations

The site is generally suitable for construction of a low embankment dam. The granite bedrock found at the dam site assures that a suitable foundation can be formed. On the left bank the bouldery jointed rock would have to be trimmed off to a true bedrock level but on the right bank the bedrock would require minimal cleaning off.

It is assumed that a grouted cut-off could easily be established along the alignment and it is not anticipated that there would be any unexpected foundation problems which would significantly affect the foundation costs. The overburden cover between the rock outcrops is not expected to be very deep, except in the channel area and in the low lying area on the right bank where a trial pit in coarse sand met the watertable at a depth of 1.2 m.

The spillway structure could be located on granite bedrock on either bank depending on the required elevation.

3.12.4 Construction Materials

Two major sources of construction material are found in the area of the site. As described above granite occurs extensively in the area and establishing a suitable quarry for coarse concrete aggregate, riprap or even rockfill should not present too many problems. Although the thickness of sand in the river bed has not been established it is possible that the river bed would provide sufficient quantities for the shell zones from within the reservoir area. Additional quantities could be obtained from a number of low lying areas on each bank which are probably old infilled river channels.

The meandering river contains two prominent bends in the dam site area. Within these bends the alluvium is silty and could possibly form potential borrow areas for providing material for use in a blended core. An area of about 3 ha of the material is located on the left bank immediately downstream of the alignment and about 2 ha are found 0.7 km upstream on the right bank. Gravels are found in the overburden surrounding the granite outcrops.

3.12.5 Dam Geometry and Field Survey

The survey of the river crossing was carried out on an alignment on a grid bearing of 152°/332° between a paint mark on the domed rock on the right bank and a bench mark established at the foot of the kopjie on the left bank. The survey alignment on the right bank was extended to high ground via a further domed outcrop of granite. On the left bank the survey was extended to the road near the kopjie at the end of the ridge between the two rivers. This alignment is shown in plan on Figure No. 3.23 and in section on Figure No. 3.24.

The survey levels are related by trigometrical levelling to Trig. Station BPT239, situated about 1.5 km to the south east of the site and the datum was checked by a resection taken on a number of other Trig. Stations visible from the site. Map contours are again only at 50 ft intervals, so there is insufficient data to provide a correction for correlation with cartographic data.

The principal features of an embankment dam on the site are shown on the section on Figure No. 3.24. The spillway, in the form of a 240 m long concrete weir can be conveniently located on the left bank with a minimum of excavation and with very little training works to protect the main embankment during flood flows.

3.13 Site No. 70 - Motloutse River (Figures 3.25 and 3.26)

3.13.1 General Topography

This dam site on the lower Motloutse is the largest one surveyed. The site is about 35 km downriver from Site No. 40 and the river channel at this point is about 180 m wide. Some 5 km further downriver there is an interesting geological feature named locally Soloman's Wall which has been suggested as a possible dam site. The feature, which is at the point where the road from the Tuli Block to Pont Drift crosses the river, takes the form of a sandstone wall over 20 m in height which crosses the river at an acute angle and through which the river has cut a passage. Examination of the feature indicated that it would not be effective as a water retaining structure, and the site has not therefore been considered as a serious possibility for a dam of any size. Furthermore the feature is not continuous on both banks. In particular it terminates on the north bank at Riven Hill, and an embankment some 3 km in length would be required to connect this hill with higher ground to the north west. On the south bank a number of additional short saddle dams would be required in order to seal further discontinuities in the feature.

The site which has been surveyed is identified by the sandstone escarpment of Lekono Hill situated on the right bank. On the left bank opposite the hill but offset upstream by about 800 m there is an area of exposed weathered sandstone bedrock with generally higher ground beyond. Beyond the higher ground a ridge leads in a general northerly direction for about 2 km at an elevation of about 570 m whereas on the right bank beyond Lekono Hill the terrain is flat for about 3 km at an elevation of about 565 m.

Across this low lying area can be traced the remains of a sandstone feature similar to Soloman's Wall and there are also outcrops of dolerite.

The site is located in the Tuli Block generally on the Fairfield Estate, though the estate boundary crosses the site. The low lying areas beside the river are dense acacia woodland, visited by elephants and other game, and the surrounding estates are operated as a game reserve. The site is fairly remote though access to the south bank of the river can be obtained by means of the gravel road through the Tuli Block.

3.13.2 Geology

The site is located in an area underlain by a succession of creamy brown coloured massive sandstones which probably form part of the Karroo System. The succession appears to be lithologically uniform and the exposed rock consists of a well sorted sandstone with constituent well rounded grains. The area underlain by the sandstones is extensive but in the proposed reservoir area they are overlain by basalts which begin to crop out about 10 km upstream.

The surface outcrops of the sandstone are massive and pillar like and are very blocky and prominently jointed.

It is considered that a major fault exists between the left and right abutments because the main ridge of sandstone is offset by a few hundred metres, the right abutment being about 800 m downstream from the left abutment.

3.13.3 Engineering Considerations

On the evidence of surface geology it appears possible for a large embankment dam to be constructed on the site. However the proposed alignment would require extensive blasting of some of the sandstone pillars on the left abutment to avoid sharp changes in bedrock profile. The depth of bedrock in the channel cannot be ascertained but the distance between abutments indicates that an extensive cut-off trench would probably be required. Generally a grouted cut-off could be established on the left and right abutments.

It is not envisaged that there would be a problem in siting the spillway on the low lying area on the right bank, though the foundation conditions in this area are unknown.

3.13.4 Construction Materials

A major problem at this site would be the location of suitable coarse aggregate materials. The sandstone is slightly friable in some areas and is likely to have a low crushing strength as a result; it is not expected to be suitable as a coarse aggregate for concrete and its suitability as riprap would have to be proved by suitable tests. The only source of concrete aggregate is probably from the basalts some 10 km or so distant from the proposed alignment.

Possible areas of plastic soils suitable as core materials were located on both banks of the river about 3 km downstream from the site. On the left bank there is an area of pans, about 20 ha in extent and on the right bank there is a rather smaller area of red silty clay. Both areas were sampled and soil classification tests indicated that they would be suitable.

Bulk fill material is available in the form of sand from the river channel or red sands and silty clays from the right bank upstream of the alignment. Fine gravels can also be obtained from the river channel though screening would probably be necessary. Blasting of the sandstone pillars on the left bank should provide some material suitable for rock fill or riprap.

3.13.5 Dam Geometry and Field Survey

Because of the relative displacement of the sandstone outcrops on the two banks of the river, the axis of a dam on this site will cross the river at a fairly acute angle. In order to select the optimum alignment a tacheometric survey of the area between the two outcrops was carried out using survey stations established at prominent points on both banks. The alignment selected by this means, which crosses the river on a grid bearing of 42°/222°, is shown on Figure No. 3.25. The low lying areas beyond the sandstone outcrops on both banks were also surveyed by means of barometric levelling combined with compass traverses.

The developed profile on this alignment is shown in section on Figure No. 3.26. The survey levels are related by trigonometric levelling to Trig. Station BPT287 situated on the top of Lekono Hill. There is a reasonable degree of agreement between the survey levels and those obtained from map contours.

Assuming a retention level for the reservoir of 568.0 m the principal features of an embankment dam on the site are shown on Figure No. 3.26. The total length of embankment will be in excess of 5 km including a number of short saddle dams on the left bank. The spillway is located on the right bank to take advantage of the sandstone/dolerite bedrock in this region. The spillway would comprise a low concrete weir, 965 m long which would discharge into the adjacent river valley. In this position major training works below the spillway would not be required. A fuse section to act as a safeguard against overtopping of the embankment is shown in the embankment close to the spillway. The outlet works are shown on the left bank of the river at a point where rock foundations can be anticipated.

CHAPTER 4

COST ESTIMATES

4.1 Introduction

The Terms of Reference require us to estimate, in 1975 prices, the cost of developing the largest possible dam at each site, up to and including the outlet facilities, taking into account the necessity to maximise the yield within reasonable cost limits.

In Chapter 2 Figures Nos. 2.18 to 2.30 show the relationship between net reservoir storage and yield at the 1 in 20 and 1 in 50 years failure risk. For most sites the yield at the 1 in 50 years failure risk is approaching a maximum value at a net reservoir volume equal to twice the mean annual runoff (MAR). For some sites the maximum yield is reached before this reservoir volume for the 1 in 50 years risk of failure, but approaches a maximum at the 1 in 20 years risk of failure at about the same value (i.e. twice the mean annual runoff).

In practice reservoir volumes are generally changing rapidly with dam height at the 2 x MAR value, so that any saving due to reducing the volume below this value tends to be small. We have therefore used the 2 x MAR value as an indicator but have taken topographical features into account in making the final choice for the maximum height of dam.

A further factor is that the reservoir yield will reduce with time due to siltation; where possible therefore we have adopted a maximum size of reservoir somewhat larger than the 2 x MAR value in order to preserve the maximum yield.

We wish to emphasise however, that building a dam initially to its maximum possible size is in many cases unlikely to be the most economic solution, and it will be necessary to consider construction in stages at the feasibility stage of design. In Appendix C we have given a range of earthwork volumes against various dam crest levels for those sites where stage construction seems feasible, which could be taken as an indicator of costs for dams smaller than the maximum, although the relationship will not be directly pro rata. Among other factors, the spillway arrangement could well require modification.

It should also be borne in mind that the reservoir volume curves are approximate only, and that an accurate reservoir survey at say 1:10,000 scale with 1 metre contours would be necessary before the height of the dam could be finalised; any changes necessary as a result would affect the estimated cost of the dam. It is also possible that more hydrological data will become available before the dam is built, which could affect the estimate of MAR.

Table 4.1 lists the maximum crest levels and retention levels we have adopted for each dam site, together with the estimated initial yield and the yield after 25 years siltation. For the latter is has been assumed that the rate of siltation would be 1 per cent of MAR per year.

TABLE 4.1

ADOPTED MAXIMUM DAM HEIGHTS AND CORRESPONDING YIELDS

DAM SITE NO.	ADOPTED MAXIMUM LEVELS RETENTION (m)	CREST (m)	INITIAL YIELD 1 in 50 YEARS RISK OF FAILURE (m^3/day)	INITIAL YIELD 1 in 20 YEARS RISK OF FAILURE (m^3/day)	YIELD AFTER 25 YEARS 1 in 50 YEARS RISK OF FAILURE (m^3/day)	YIELD AFTER 25 YEARS 1 in 20 YEARS RISK OF FAILURE (m^3/day)
3	1,108	1,111	1,600	3,600	1,500	3,300
10	1,096	1,101	11,800	18,400	11,300	17,200
13	1,036	1,039	100	420	80	360
17 (North)	942	947	46,200	83,800	44,000	78,400
17 (South)	941	946	45,400	79,000	44,400	75,800
33	763	768	78,400	119,000	72,300	110,000
40 (Upstream)	628	646	93,500	140,000	90,000	135,000
40 (Downstream)	628	646	93,500	140,000	90,000	135,000
42A (Upstream)	657	664	15,700	24,100	14,800	22,300
42A (Downstream)	657	661	15,700	24,100	14,800	22,300
46 (Upstream)	865	870	12,700	19,600	12,100	18,700
46 (Downstream)	865	870	12,700	19,600	12,100	18,700
55	914	919	1,600	3,000	1,500	2,900
56	933	938	1,100	3,000	1,000	2,900
61 (Upstream)	1,057	1,061	560*	980*	540*	950*
62	1,058	1,062	600	2,100	500	1,900
70	568	572	97,000	154,000	93,000	149,000

NOTE:. All levels are related to field survey datum.

* Assuming MAR = $0.99 \times 10^6 m^3$

4.2 Basis of Estimates

4.2.1 Dam Design

(a) General

For all the sites included in Phase II the height of the dam is small in comparison to the length. Under these circumstances an earth dam is likely to be the most economical form of construction, and we have therefore based our estimates on this type of dam, a typical cross-section of which is shown in Figure No. 4.1.

The slopes of the upstream and downstream faces may vary within small limits when foundation conditions and the availability and quanlity of construction materials are known. However those shown are considered to be a reasonable assumption for all sites bearing in mind the conditions found during our reconnaissance.

Similarly there will be variations in the thickness of riprap and gravel layers depending on dam height and reservoir length, but such changes would probably be compensated by changes in unit rate (larger volumes of material would lead to lower rates, and vice-versa), and the same thickness has therefore been assumed throughout.

With regard to the impervious barrier, we have shown a clay core on Figure No. 4.1. An alternative would be a concrete core wall. Provided that suitable clay core material is available within a distance of about 20 km from the site we find that a clay core would be more economical than a concrete wall. This type of construction is thus likely to be the most suitable for nearly all the sites. We have also considered the possibility of using an impervious membrane on the upstream face, but find that it would almost certainly be more costly than even the concrete core wall at the present time. However, it would be sensible to give further consideration to this type of construction at the feasibility stage, as the comparative cost of various methods of construction can change with time as technology advances.

(b) Spillway

There are basically two types of spillway:

(i) In some cases (Sites Nos. 17, 40 for example) the most economical solution is to excavate a spillway channel round the flank of the dam. The main work would then be the excavation of the channel itself with a low concrete cill to define the retention level.

(ii) For most sites the spillway will take the form of an overflow weir with training walls similar to the existing Shashe Dam.

With regard to the required spillway capacity we have used the 200 year return period flood flows derived in Chapter 2 as the basis of our estimates. It has been assumed that when the spillway is passing the 200 year flood the reservoir will reach the top of a fuse section on one flank of the dam. Floods in excess of the 200 year value would wash out the fuse section, thus safeguarding the main length of the embankment, which would have a crest level 1 metre higher than the fuse section.

The flow capacity of the spillway has been calculated from the formula:

$$Q = 1.95L\,H^{1.5}$$

where Q = flow in m^3/sec.

L = length of spillway in metres

H = depth of flow over spillway in metres

Using the criteria given above Table 4.2 shows the variation in length of spillway required at each site with the difference in level (D) between the spillway crest and the crest of the main embankment.

TABLE 4.2

SPILLWAY LENGTH REQUIREMENTS

(Rounded to nearest 5 metres)

DAM SITE	200 YEAR FLOOD (m^3/sec)	LENGTH OF SPILLWAY (M) REQUIRED WITH D* (METRES) EQUAL TO:								
							8	10	12	14
3	1,241	225	120	80	55	45	35	25	20	15
10	1,798	325	175	115	80	65	50	35	25	20
13	792	145	80	50	40	30	20	15	10	10
17	4,194	760	415	270	190	145	115	80	60	45
33	7,008	1,270	690	450	320	245	195	135	100	75
40	8,207	1,490	810	525	375	285	225	155	115	90
42A	4,194	760	415	270	190	145	115	80	60	45
46	6,463	1,170	640	415	295	225	180	125	90	70
55	2,204	400	220	140	100	75	60	40	30	25
56	3,360	610	330	215	155	115	95	65	45	35
61	1,102	200	110	70	50	40	30	20	15	10
62	2,418	440	240	155	110	85	65	45	35	25
70	9,801	1,775	965	630	450	340	270	185	140	105

* Includes 1 metre freeboard between crest level of fuse section and crest level of main embankment.

At the feasibility stage it would be necessary to determine the most economic length of spillway; for the present exercise we have generally adopted the longest length of spillway that can be reasonably accommodated by the site, since the total length of the dam is usually long in comparison with the spillway, and any saving achieved by adopting a shorter spillway is likely to be more than compensated for by the increased cost of the higher earth dam that would be required. An exception is Dam Site No. 40 where the dam is comparatively short and the amount of excavation necessary for a long spillway would be large.

(c) Outlet Works

For the outlet works we have in all cases adopted an arrangement similar to that used at the existing Shashe Dam, namely an outlet tower (internal diameter 4.3 m) in the reservoir immediately upstream of the dam, and a culvert (internal width 2.0 m) at existing ground level through the dam. The internal dimensions stated depend chiefly on access requirements and will not therefore vary significantly, but the height of tower and length of culvert will depend on the height of the dam, and allowance has been made for this factor.

The outlet works would be arranged so that existing ground level near the outlet tower is about 1.5 m below the lowest reservoir draw-down level, but the precise location would depend on depth to rock and could only be decided after subsurface investigations had been carried out.

4.2.2 Construction Costs

The most recent example of dam construction in Botswana is the Shashe Dam, which was completed in April 1973. The form of construction of Shashe is very similar to that proposed for all the dams included in this phase of the study, and we consider that cost estimates based on the construction costs of Shashe are likely to be realistic.

However, considerable cost escalation has taken place since the tenders for Shashe Dam were received in March 1971. Enquiries in Botswana and South Africa, and comparison with recent tenders, leads us to believe that on average civil engineering costs have increased by the factors shown in Table 4.3 during the period March 1971 to the end of 1975.

TABLE 4.3

AVERAGE COST ESCALATION MARCH 1971 TO END 1975

Period	Escalation per cent	Total since March 1971 per cent
March 1971 - March 1972	10	10
March 1972 - March 1973	12.5	24
March 1973 - March 1974	15	42
March 1974 - March 1975	15	64
March 1975 - December 1975	10	80

We have therefore based our estimates on rates for Shashe Dam escalated by 80 per cent in order to obtain figures relating to the end of 1975.

4.3 Cost of Dams

Based on the above assumptions our estimates of the cost of dams at each of the sites selected for the Phase II Study are shown in Table 4.4.

TABLE 4.4

COST ESTIMATES

DAM SITE NO.	ESTIMATED END 1975 COST (PULA)
3	1,757,000
10	4,671,000
13	1,328,000
17 (North Alignment)	9,647,000
17 (South Alignment)	8,589,000
33	16,336,000
40 (Upstream Alignment)	15,798,000
40 (Downstream Alignment)	17,391,000
42A (Upstream Alignment)	6,972,000
42A (Downstream Alignment)	5,506,000
46 (Upstream Alignment)	7,972,000
46 (Downstream Alignment)	8,496,000
55	3,058,000
56	3,952,000
61 (Upstream Alignment)	1,174,000
62	1,232,000
70	16,278,000

The make-up of the above cost estimates is given in Appendix D.

On the same basis, the end 1975 cost for the existing Shashe Dam would be P.7,700,000.

CHAPTER 5

ADDITIONAL TERMS OF REFERENCE

5.1 Nata River

5.1.1 Introduction

The Nata River rises in Rhodesia and flows in a south-westerly direction to enter the Sua Pan south of the village of Nata. The catchment area is large - of the order of 23,000 sq. km - of which about 80% is in Rhodesia. Available mapping is at 1:250,000 scale, without contours. Aerial photographs at 1:50,000 scale cover most of the area within Botswana.

5.1.2 Re-examination of aerial photographs and maps

The main river channel and adjacent areas were studied in detail (photo series Contract 118 1971, see Figure No. 5.1) and the following observations were made, working from south the north:

RUN BT6 Photos 123/124

The river enters the Sua Pan; a delta-like area of sand and swamp with no well-defined relief and difficult access.

RUN BT6 Photos 025-028

Flatter country, no sign of rock or any apparent possibility of constructing a permanent impounding structure.

RUN BT5 Photos 243-245

Nata village. Two road bridges cross the river which is about 80 m wide at this point. The upstream bridge is used as a control for river gauging and could possibly also serve as a control for an intake works linked with an off-stream storage scheme. However, the country in the immediate vicinity appears sandy and flat and an off-stream storage reservoir near to Nata would probably need an impervious lining on the water face. A pan about 0.75 km in diameter is apparent 27 km to the east of Nata (also indicated on Sheet SF 35-2 of the 1:250,000 scale map series) which would be a possible off-stream storage site.

RUN BT5 Photos 174-181

River now meandering in an east-west direction; the whole area is very flat, no sign of rock or possible dam sites.

RUN BT4 Photos 039-046

Overlaps previous run; river turning north but no change in topography - flat and apparently devoid of rock.

RUN BT5 Photos 128-130

Near Nkange confluence; area becoming very remote but with some cultivation, still flat with no rock but some small pans apparent.

RUN BT3 Photos 243-246

The whole area is flat and probably liable to sheet flooding when the river is running high; to the north as far as the Rhodesian border it is mapped as a series of braided channels and pans indicating a general lack or relief.

5.1.3 Aerial Reconnaissance

In order to confirm the above observations and investigate any other possibilities an aerial reconnaissance was carried out of the whole length of the river between the Sua Pan and the Rhodesian border.

In the vicinity of the point where the Nata discharges into it the Sua Pan is a completely flat expanse of white soda ash and salt, the salt extending up the various outlet channels of the Nata for a considerable distance. In view of this salinity there does not appear to be any possibility of using either the pan or the channels for storing potable water.

Nata bridge is an obvious river control point and pools of standing water were apparent upstream and downstream. The river channel for a distance of up to 50 km upstream from Nata is wide and the bed is sandy but no rock was apparent and the lack of conventional dam sites was confirmed. However, this reach would appear to have some potential for well-point schemes.

Beyond the Nkange confluence the river splits into numerous smaller channels, the area is very remote, and can be considered to have very little potential as an economic surface water source.

5.1.4 Hydrology

Flows in the Nata have been gauged at the village of Nata since December 1969 and the mean annual flow is 279 million cubic metres. It thus represents a very considerable resource by Eastern Botswana standards, only being exceeded by the Limpopo and Shashe in their border reaches.

However, in common with other rivers in Botswana, the flow is very flashy and a high proportion occurs as large floods of short duration. This is illustrated by Figure No. 5.2, which shows the number of days per year various flows were exceeded for the six years of record, and also a derived mean curve. Using the mean curve, Table 5.1 summarises this information:

TABLE 5.1

FLOW OCCURRENCES - RIVER NATA AT NATA

Flow (m^3/sec)	Average number of days per year flow is exceeded
500	1
100	9
50	15
10	33
5	40
1	66
.5	73
.1	84

5.1.5 Conclusions

Based on the above observations we have reached the following conclusions regarding the possibilities of harnessing the resources of the Nata River:-

(a) There are no conventional dam sites on the main river channel likely to be an economic source of water.

(b) The river channel for up to 50 km upstream of Nata is wide and the bed appears sandy. Well-point abstraction from the sand of the river bed is therefore worth investigating for small-scale schemes. However, the potential is probably small in comparison with river flows - possibly of the order of 5,000 m^3/day.

(c) It would be possible to construct a river control structure, such as a barrage, virtually anywhere along the river provided that flood flows are allowed to pass through. Such a control structure could be used in association with off-stream storage using either:-

(i) an intake and pump-station pumping water through a pipe-line, or

(ii) a diversion headworks and canal.

The yield from such schemes would depend on the maximum capacity of the pump-station or canal. Figure No. 5.3 (which is based on the mean curve shown on Figure No. 5.2) shows the variation in mean annual quantity pumped or diverted with maximum pumping or diversion capacity; Table 5.2 summarises this information.

TABLE 5.2

DIVERSION CAPACITY FROM NATA RIVER AT NATA

Maximum Pumping or Diversion Capacity (m^3/sec)	Mean Annual Quantity Available ($m^3 x 10^6$)
100	155
50	108
10	39
5	24
1	6.3
0.5	3.4
0.1	0.73

It is assumed in the above computations that low flows are all abstracted and that only flows in excess of the pumping or diversion capacity are allowed to pass through. On this basis, allowing say 50 per cent losses for evaporation, a pumping scheme of 1 m^3 /sec maximum capacity would yield about 3 million cubic metres per year (8,000 m^3/day), and a canal scheme of 10 m^3/sec maximum capacity would yield about 20 million cubic metres per year (50,000 m^3/day).

The practicability and economic viability of such schemes can only be assessed when the location and pattern of demand is known. At this stage it would also be necessary to take account of the effect of dry years such as 1972/73.

(d) Inevitably considerable quantities of water will continue to pass into the Sua Pan, become saline and require treatment before it can be put to potable use. The use of solar power for desalination is a possible line for future research.

5.2 Metsemotlhaba - Notwane Rivers

5.2.1 Introduction

The Notwane is a major tributary of the Limpopo and its upper reaches have been dammed to supply Gaborone with water. It is joined by the Metsemotlhaba near the large village of Mochudi (see Map No. 7548/1 of

Phase I Report). Dam Site No. 62 on the Metsemotlhaba River 70 km south-west of Mochudi is considered elsewhere in this Report. Relevant catchment areas are shown in Table 5.3.

TABLE 5.3

METSEMOTLHABA - NOTWANE RIVERS, CATCHMENT AREAS

Location	Catchment Areas (km^2)	
	Total	Net*
Dam Site No. 62	800	
Gaborone Dam	4,300	
Notwane above confluence with Metsemotlhaba	5,100	800
Notwane below confluence with Metsemotlhaba	8,600	4,300
Notwane above confluence with Limpopo	14,500	10,200

* Excluding Gaborone Dam Catchment.

5.2.2 Re-examination of Aerial Photographs and Maps

The whole are between the Limpopo confluence and the Molepolole-Gaborone Road was studied in detail (South-East Block photo series, see Figure No. 5.4) and the following observations were made, working from north to south:

RUNS 11 and 12

Limpopo confluence; no signs of rock but a saddle between the Notwane and Limpopo (23° 47'S, 26° 56'E, Map 2326D4) could possibly provide a spillway site for a dam located immediately upstream of the confluence (see 5.2.3 for site reconnaissance).

RUN 13

Flat area, no signs of rock or possible dam sites.

RUN 14

Morebe Hill 4 km from the river on the right bank (24° 00'S, 26° 45'E, Maps 2326/D3 and D4), but no sign of rock near the river itself. Old meander scars indicate that the river has wandered over the area in the past. Not worth pursuing.

RUNS 15 and 16

Lethekane Ridge about 1 km from the river on the right bank (24° 05'S, 26° 35'E, Map 2426B1) might be a possible spillway site. However, the left bank is extremely flat and to get any depth of water a dam 6 to 7 km long would be required. Not worth pursuing.

RUN 17

Flat area, no signs of rock or possible dam sites.

RUN 18

Signs of cultivation along the Notwane southwards from about 20 km north-east of Mochudi, along the Magogode Valley which is probably an abandoned course of the Notwane, and along the Thagale River (see Maps 2426A3 and A4). It appears quite likely that a lot of the Notwane flow is lost to recharge in this area. No signs of rock or possible dam sites.

RUN 19

No signs of rock or possible dam sites; cultivation intensifying and many boreholes shown on the map, indicating a reliable groundwater supply for household use and cattle watering, together with adequate rainfall for cultivation.

RUN 20

Mochudi village; rock outcrops at several locations on the left bank, but right bank flat and intensely cultivated (see 5.2.3 for site reconnaissance).

RUN 21

No signs of rock or possible dam sites on either the Notwane or the Metsemotlhaba, area intensely cultivated.

RUN 22

No signs of rock or possible dam sites on either river; south of this run the Notwane is too near to Gaborone Dam and the Metsemotlhaba too near to Site No. 62 to warrant further consideration.

5.2.3 Site Reconnaissance

In view of the above observations a site *reconnaissance was made of* the Notwane/Limpopo confluence area and of the Mochudi area.

(a) Notwane/Limpopo confluence.

The river channel here is about 10 metres wide which is surprisingly narrow when compared with its large catchment area. It seems probable that large parts of the catchment make little contribution to the run-off, and that a considerable proportion is lost to recharge in the lower reaches. It is difficult to believe that any appreciable flows reach this point in lean years. No favourable features for a dam site could be discerned and no rock outcrops were found either at the dam site itself or at the possible saddle spillway site.

(b) Mochudi.

As shown on Figure No. 5.5 there are three sites in the Mochudi area which offer some topographically favourable dam site characteristics:

Site A - Just downstream of Mochudi village where a prominent rock outcrop occurs on the left bank. However, the reservoir would submerge a substantial part of Mochudi itself and this site can be discounted.

Site B - At Papane Hill, which is also a prominent rock outcrop on the left bank. Again some built-up areas would be submerged by the reservoir, and the right bank is very flat and cultivated. A more serious objection is the fact that diversion of the main north-south road and of the railway would probably be necessary (the contour interval of the available mapping is 50', and therefore too great to give an accurate indication, but barometric levelling showed a difference in level of only 7.5 metres between the road bridge over the Notwane in Mochudi village and the railway bridge over the Metsemotlhaba). A possible route for this diversion is shown on Figure No. 5.3.

Site C - At Morwa, where again rock occurs on the left bank. Topographically this is probably the best site of the three, as the right bank is less flat. However, the line of the dam crosses the road and railway, which would certainly need to be diverted.

For any of the above three sites the net catchment area (excluding the Gaborone Dam catchment) would be 4,300 sq. km and the mean annual run-off, calculated in accordance with the Phase I Report, would be of the order of 25 million cubic metres; the net yield from a reservoir having a capacity of twice the mean annual run-off would then be approximately 18,000 m^3/day.

The above values are only valid on the assumption that the whole catchment area contibutes to run-off and that there are no abnormal losses to recharge. In the case of the Metsemotlhaba there is a distinct reduction in river bed width from about 30 metres at Dam Site No. 62 to about 10 metres at Mochudi which may indicate that the yield could be over-estimated.

It should also be borne in mind that the construction of a dam at Site No. 62 would reduce the catchment area by 800 sq. km.

5.2.4 Conclusions

Based on the above observations we have reached the following conclusions regarding the possibilities of harnessing the resources of the Metsemotlhaba and Notwane Rivers downstream of the Molepolole-Gaborone road:

(a) The only possible conventional dam sites are at Mochudi where the construction of a dam would involve the considerable expense of diverting the main north-south road and railway, coupled with the submergence of some built-up areas and cultivated land.

(b) The yield of such a dam is uncertain in view of the probable losses of river flow to recharge.

(c) Losses to recharge appear too high, and the sand bed is in any case too narrow, to make well-point abstraction schemes feasible.

(d) River flows are considered too ephemeral to make off-stream storage schemes economically attractive.

We conclude that the construction of a dam at Mochudi is the only possible means of harnessing the resources of the two rivers between the Molepolole-Gaborone road and the Limpopo confluence, but that such a scheme is unlikely to be economically viable. It appears probable that a considerable proportion of the river flow passes to recharge, the large village of Mochudi and surrounding areas, both upstream and downstream, relying almost exclusively on this source at the present time for their water supplies.

In our view a detailed study of the combined ground and surface water resources of the area would be necessary before any firm decision could be taken on future development. A useful first step to this end would be the establishment of a river gauging station below the confluence of the two rivers at Mochudi.

5.3 Molopo River

5.3.1 Introduction

The Molopo River rises in South Africa to the east of Mafeking. It flows in a generally westerly direction and is joined by the Ramatlabama approximately 40 km west of Mafeking. From this point westwards it forms the boundary between Botswana and South Africa for a distance of about 600 km.

5.3.2 Climate and Hydrology

Rainfall in the upper reaches of the river approaches 600 mm per year, but decreases progressively towards the west, and has a value of about 400 mm per year at 24° East. Evaporation is high - of the order of 2,800 mm per year.

Upstream of the confluence with the Ramatlabama a well defined drainage pattern is evident (see Figure 5.4), but downstream of this point few tributaries join the main stem, particularly on the Botswana side, and it is probable that little contribution to river flow is received. Thus the bulk of any flow along the Botswana boundary is derived from the catchment within South Africa.

Enquiries in Botswana and South Africa revealed that no river gauging records are in existence for the Molopo or any of its tributaries.

5.3.3 Examination of Aerial Photographs and Maps

The main river channel and adjacent areas were studied in detail (South-East photo series, RUNS 38 and 39, see Figure No. 5.4) and the following observations were made:

The river channel is well defined but in very flat country. The surrounding areas along both banks are quite intensely cultivated, particularly on the southern side and towards the east. The farms probably rely principally on boreholes for potable water and stock watering and as cultivation is concentrated along the river this must be a source of recharge.

The only possible site for a dam of any appreciable size is just downstream of the village of Phitsane Molopo (see Figure 5.7), where an outcrop of rock crosses the river, but the village would be submerged by a reservoir constructed here.

Small farm dams across the main river channel are apparent at points approximately 15 km and 85 km west of Phitsane Molopo, with standing water upstream.

According to B.G.A. Lund's "Surface Water Resources of Bechuanaland" the bed of the river is used as a road west of 24° East. This observation was confirmed by the South African 1:50,000 scale maps, and it is evident that no further consideration need be given to the river west of this longitude.

5.3.4 Site Reconnaissance

A visit was made to Mafeking and to the upper reaches of the Molopo River; we are grateful to Mr. Spencer Minchin, a resident of Mafeking for many years, for his valuable assistance and advice.

The Molopo originates as a spring known as the "Molopo Eye" in the dolomitic area about 30 km to the east of Mafeking. This spring is perennial and the Molopo supplies Mafeking with water. For the last two wet seasons rainfall in the upper reaches has been exceptionally plentiful, and the Molopo has continued to flow through and beyond Mafeking, but prior to this period it had been dry for a number of years.

At the time of the present visit about 0.7 m^3/sec was estimated to be flowing from the outlet from Cook's Lake in Mafeking, and from a farm dam built across the river just outside the town.

At the "Fred Stanton" bridge over the Molopo 35 km west of Mafeking the river is about 6 m wide and the flow had about doubled, having the appearance of being unusally high.

Further downstream at the Phitsane Molopo border post, the flow could be gauged more accurately at the 12 m long Irish bridge crossing, and amounted to about 0.6 m^3/sec. The South African border police advised that this was about the average flow for the whole period since August 1975 when, after a period of no flow, there was very good rain. However, the river could produce bigger floods, but not usually sufficient to prevent vehicles crossing.

The possibility of greater floods was confirmed just downstream of Phitsane Molopo, where a farm dam had failed. From the nature of the failure i.e. breaching at two locations, it appeared to be due to overtopping. This failure is described in a report dated November 1967 prepared by the Botswana Department of Water Affairs.

On returning to Botswana it was noted that there was no flow in the Ramatlabama at the border crossing.

5.3.5 Conclusions

Based on the above observations we have reached the following conclusions:

(a) The source of flow in the Molopo River is almost entirely in that part of the catchment area around Mafeking and within South Africa.

(b) A major part of the flow probably goes to recharge either inside South Africa or within the first 100 km of the border section. The farming community rely to some extent on this recharge to maintain their potable water supply.

(c) A possible dam site exists just downstream of the village of Phitsane Molopo, but the village would be submerged by the reservoir if a dam was built there.

(d) At the time of our reconnaissance the river at Phitsane Molopo had been flowing continuously at a rate of about 0.6 m^3/sec for over a year; this represents, by south-east Botswana standards, a considerable resource.

(e) In the absence of any flow records it is not possible to say categorically that the resources of the Molopo are negligible other than for minor local purposes, although in our view the present situation is probably unusually favourable so far as river flow is concerned and this proposition is likely to be true.

We conclude that a detailed study of the combined ground and surface water resources of the area would be necessary before any firm decision could be taken on future development. A useful first step to this end would be the establishment of a river gauging station at the border crossing post of Phitsane Molopo.

CHAPTER 6

CONCLUSIONS AND RECOMMENDATIONS

6.1 Conclusions

In Chapter 4 the adopted maximum dam heights and corresponding yields are given in Table 4.1 and our estimates of end 1975 costs are given in Table 4.4.

Figure No. 6.1 shows a graphical comparison of cost against initial yield at a 1 in 50 year risk of failure. The graph also shows lines of equal unit cost of water at the outlet works calculated on the basis that the annual cost of the dam would be 10% of the capital cost.

From this graph and other considerations we have drawn the following conclusions:-

(i) As would be expected the larger schemes give the lowest unit cost of water. Dam Sites Nos. 17, 33, 40 and 70 are comparable to the existing Shashe Dam in this respect*, and appear to offer good prospects of producing an economical supply.

(ii) The three dam sites studied on the Motloutse River (i.e. 33, 40 and 70) are not significantly different, although Dam Site No. 40 would appear to be the most favourable. However, the location of demand is likely to be an overriding consideration.

(iii) Dam Sites Nos. 10 and 42A, and to a lesser degree Nos. 3 and 46, appear to offer reasonable prospects of providing an economical supply.

* Had Shashe Dam been constructed initially to its maximum capacity (it is designed for heightening by 4 metres) it would almost certainly show an improved position on Figure No. 6.1.

(iv) Dam Sites Nos. 62 and 55 offer marginal prospects of providing an economical supply, although No. 62 now falls below the Phase I threshold yield of 1,000 m^3/day at 1 in 50 years risk of failure. It is a matter for consideration whether or not this criterion is too severe for a domestic supply.

(v) Dam Site No. 61 is similar to Dam Site No. 62 on the basis of the Phase I estimate of MAR but the river appears to be unusual and, subject to flow measurements, there may be prospects for direct abstraction rather than impoundment (with its inevitable wasteful evaporation losses) at this Site.

(vi) Dam Sites Nos. 13 (in particular) and 56 appear unlikely prospects and could in our view be discounted from further consideration.

Our conclusions relating to the Additional Terms of Reference are included in Chapter 5.

6.2 Recommendations for Location of Gauging Stations

We have considered the question of locating additional gauging stations under two main headings:-

(i) Particular locations relating to the dam sites studied in Phase II*, and to the Additional Terms of Reference.

(ii) General locations to give a better coverage of areas at present poorly served.

* As stated in Section 1.2, we understand that the selection of sites for Phase II study was made in accordance with the Botswana Government's current needs for information, both positive and negative, and does not by any means represent a final priority rating.

6.2.1 Particular Locations

(a) Dam sites studied in Phase II.

Bearing in mind the preceding section 6.1 we consider that adequate hydrological data would be desirable for future dam design at Dam Sites Nos. 3, 10, 17, 33, 40, 42A, 46, 55, 61, 62 and 70.

In our view existing gauging stations cover Dam Sites Nos. 10, 17 and 33 adequately.

We therefore consider that stations should be established at or near Dam Sites Nos. 3, 40 or 70, 42A, 46, 55, 61 and 62.

(b) Additional Terms of Reference.

As discussed in Chapter 5 we consider that gauging stations should be established on the Notwane River downstream of its confluence with the Metsemotlhaba, and on the Molopo River at the border crossing near Phitsane Molopo.

6.2.2 General Locations

The location of existing gauging stations is shown on Map No. 7548/2 of the Phase I Report, which also divides Eastern Botswana into three regions, namely the Northern Region, Central Region and Southern Region.

In the Northern Region there is at present a reasonably good coverage in the northern part of the region, but less good in the south and east. However, we believe that with the additional gauges recommended at Dam Sites Nos. 3, 40 or 70, and 42A this Region would be adequately covered.

Gauging in the Central Region is at present confined to Mahalapye. However the potential of the region for surface water storage appears to be poor and in our view the additional gauge at Dam Site No. 55 will provide adequate cover. Dam Site No. 46 is also of some relevance to this region.

The Southern Region is at present covered only by gauge L44 at Palapye and by the existing reservoirs at Nuane, Notwane and Gaborone. The additional gauge at Dam Site No. 46 will in our view provide adequate coverage for the Lotsane River, and the additional gauges at Dam Sites

Nos. 61 and 62 and at Mochudi would be adequate for the rest of the region with one reservation. As stated in Section 3.4 of the Phase I Report, estimating flow over the spillways at the Nuane, Notwane and Gaborone Dam Sites is very difficult at moderate flood flows. We therefore recommend that gauging stations or weirs should be established downstream of Nuane and Gaborone Dams specifically to cover this range of flows.

We have also considered whether additional gauges are necessary on the border rivers, but in our view existing gauges, combined with those on tributaries, are adequate.

6.3 Recommendations for Restrictions on Development in Catchment Areas

The Terms of Reference require us to advise, for each site, on restrictions to be imposed on other water development work in the catchment, if the safe yield of the dam is not to be decreased by more than 10 per cent as a result of upstream works.

In our view it is not possible to make particular recommendations for any of the sites, but we offer the following general observations.

(i) Small farm dams are unlikely to affect the yield at the sites studied within the limits of accuracy of the available hydrological information, and it would be unreasonable to attempt to restrict their development.

(ii) Groundwater development will not affect the yield from surface water storage.

(iii) The most probable means by which the yield at a particular site could be affected would be by the construction of a major dam upstream. The affect on possible downstream sites must therefore be given careful consideration at the planning stage.

Although not specifically included in the Terms of Reference, consideration should in our view be given to restriction on development (e.g. roads, townships, power lines, water mains etc.) within potential reservoir areas.

APPENDIX A

TERMS OF REFERENCE FOR A RECONNAISSANCE STUDY FOR MAJOR SURFACE WATER SCHEMES

BACKGROUND

1.1 Rapid growth in Botswana necessitates extensive construction of water supplies; and in order to place its construction programme on a sound footing, it is essential that the Government mount a substantial water research programme at the present time. Four possible sources of water can be used in the development of a water supply of any size:-

- boreholes
- sand river beds
- Okavango Delta water
- river surface dams

1.2 The Government is presently planning a large project to evaluate groundwater resources in parts of the country scheduled for development in the short and medium term future. The sand beds of 11 major rivers in Eastern Botswana will be prospected from early 1975. Considerably more than a million rand will be invested over the next three years in studies of the Okavango Delta as primary water resources.

1.3 It is presently anticipated that if water is transferred from the Okavango Delta it will serve development in the Okavango Corridor region only (stretching roughly from 20° to 21° 30' and 22° to 26° 30'); transfer of this water east of the Makgadikgadi Pans is not likely to take place for many years. Throughout the country, borehole and sand river extraction schemes will, in general, be used as sources for water supplies up to the major village level wherever this is physically possible.

1.4 The demand for river surface dams, therefore, arises principally in connection with mining and urban development in Eastern Botswana. At present three urban centres - Gaborone, Lobatse and Selibe-Pikwe - depend on dams for their water and the fourth, Francistown, is likely to have to do so in the near future, and another dam on the Mosetse river is to be built to serve the new township being built under the Sua project. It is anticipated that there will be a number of further large dams required within the next 20 years.

1.5 Accordingly, the objectives of this research study will be to compile an inventory of dam sites suitable for the development of major surface water supplies. Dams capable of being developed on these sites must have a safe yield of at least 1,000 cubic metres per day for small rivers and 5,000 cubic metres per day for large rivers, at the 1 in 50 years level of failure risk.

2. SCOPE OF THE STUDY

2.1 The study will be divided into two parts: Phase I and Phase II.

2.2 Under Phase I the consultant will:-

(a) Examine air photographs of all the rivers of the Limpopo and Makgadikgadi drainage systems, with the exception of the Boteti river. The international boundary rivers - the Ramogwebana, Shashe, Limpopo and Marico - are to be included.

(b) Every dam site conforming with the specifications of paragraph 1.5 on these rivers must be picked out, its catchment area identified and the likely size of dam rendering the largest safe yield possible that can be erected on it should be estimated. In cases where the development of two or more dam sites are mutually exclusive options, the most promising possibility should be selected in conjunction with the reference group (see below). In the absence of other compelling considerations, the site on which the dam with the largest safe yield can be erected will be taken as the most promising.

(c) Every site remaining after this process should be marked on a 1:1,000,000 map of Botswana, together with the catchment area and an indication (in terms of roughly estimated yield) of the size of the dam that could be constructed there.

(d) A brief accompanying report should specify, for each site marked on the map:-

(i) the size of the catchment area;
(ii) the roughly estimated mean annual run-off over the catchment;
(iii) the roughly estimated capacity of the largest possible dam that could be constructed, with the largest possible safe yield;
(iv) the roughly estimated safe yield of such a dam;
(v) cost estimates of Phase II work in accordance with the specification below.

2.3 Material available which may be of assistance is as follows:-

(a) The Lund report on surface water resources in Bechuanaland and accompanying maps. This was published in 1962.

(b) Lund Reports on specific dams sites at: Tobane, Mahalapye, Timbale, Kanye, Palapye, Molepolole and Mosetse.

(c) Report prepared by Brian Colquhoun, Hugh O'Donnel and Partners on Francistown.

(d) A report commissioned by the Anglo-American Corporation on dam sites to serve Selkirk-Phoenix.

(e) Alexander Gibb and World Bank reports: Gaborone, Lobatse and the Shashe river.

(f) Air photographs held by the Departments of Surveys and Lands and Water Affairs to the scales of 1:40,000, 1:30,000 and (for certain rivers) 1:15,000.

(g) Print laydowns of air photographs to reduced scale.

(h) Maps to the scale of 1:125,000 and (in some areas) 1:30,000.

(i) The hydrological records of the Department of Water Affairs.

(j) Such material on dam sites along the boundry rivers as can be obtained from the governments of Southern Rhodesia and South Africa.

(k) Rainfall maps and data kept in the Department of Meteorological Services.

2.4 Within two months of receiving the map and report specified in paragraph 2.2 sections (c) and (d), the Government will draw up a list (most likely on the order of 10 to 20) of sites to be further investigated by the consultant.

PHASE II

2.5 For each site contained in the above list, the consultant will make a field reconnaissance which will involve:

(a) Surveying a cross-section of the proposed line of the embankment and one longitudinal section of the area to be flooded, related to an arbitrary datum and bench marks described below.

(b) Constructing two permanent bench marks at positions to be identified on air photographs.

(c) Conducting a reconnaissance study of visible geology and other site features.

(d) Estimating the storage capacity, volume of earthworks and safe yields (at the 1 in 50 years and 1 in 20 years failure risk levels) of the dam. The size of the dam should be such as to maximise the yield within reasonable cost limits.

If permission cannot be obtained from Southern Rhodesia and South Africa to survey international sites, the consultant will be required to make estimates of the levels of the far banks of the boundary rivers.

2.6 In addition to the above fieldwork, the consultant shall, for each site:

(a) Advise on restrictions to be imposed on other water development work in the catchment, if the safe yield of the dam is not to be decreased by more than 10% as a result of upstream works.

(b) Recommend general localities for hydrological gauging stations capable of collecting river flow data essential to detailed decisions about development of the site.

(c) Estimate, in 1975 prices, the cost of developing the largest possible dam on the site, up to and including the outlet facilities.

TIMING AND REPORTS

2.7 Phase I shall be completed, and the map and report specified shall be submitted to the Government within two months of the signature of the project agreement. The map shall be submitted in 100 copies and the report in 50 copies, in English. The map and report shall contain printed warnings of the preliminary nature of the figures and an indication of their accuracy.

2.8 Phase II will be completed and report submitted within four months of the notification by Government of the list of sites to be investigated in detail. The report shall be submitted in 50 copies in English.

2.9 METRIC UNITS WILL BE USED THROUGHOUT.

LIAISON

3.1 The reference group referred to in paragraph 2.2 section (b) shall be composed of:

(a) The Senior Hydrological Engineer, Department of Water Affairs and/or other offices nominated by the Director of Water Affairs.

(b) A representative from the planning unit, Ministry of Mineral Resources and Water Affairs.

3.2 In addition, the consultant should liaise with the Department of Surveys and Lands on cartographic matters and the Department of Meteorological Services on rainfall and evaporation data.

APPENDIX B

BENCH MARK DATA

Dam Site No. 3	Related to BPP116 (Map 2027C1, 3734 feet = 1138.12 metres). Two bench marks, A and B, pipes in concrete. A = 1113.38 B = 1113.53 Method: Horizontal levelling from BPP116 to A.
Dam Site No. 10	Related to BPT43 (Map 2027D3, 3763 feet = 1146.96 metres). Three bench marks, Roos Bn, an existing boundary marker on the east bank of the river; A and B, paint marks on rock. Roos Bn = 1073.58 A = 1102.14 B = 1123.46 Method: Trigonometric levelling to a base line, then horizontal levelling to Roos Bn.
Dam Site No. 13	Related to BSP94 (Map 2027D3, 3640 feet = 1109.47 metres) Two bench marks, A and H, pipes in concrete. A = 1037.00 H = 1037.53 Method: Trigonometric levelling to a base line, then horizontal levelling to A.
Dam Site No. 17	Related to BPT67 (Map 2127B3, 3091 feet = 942.14 metres) Three bench marks, A, B, and C, pipes or pins in concrete. A = 948.39 B = 931.85 C = 938.74 Method: Barometric levelling.

Dam Site No. 33

Related to BPT177 (Map 2128C3,
2494 feet = 760.17 metres)
Two bench marks, A and B, pipes or pins in concrete.
A = 760.65
B = 744.11
Method: Barometric levelling.

Dam Site No. 40

Related to BPT310 (Map 2228B1,
2227 feet = 678.79 metres)
Two bench marks, A and B, pipes or pins in concrete.
A = 658.06
B = 659.20
Method: Barometric levelling.

Dam Site No. 42A

Related to BPS66 (Map 2228B1,
2370 feet = 722.38 metres)
Two bench marks, A and D, pipes or pins in concrete.
One survey station, B, paint mark on rock.
A = 664.22
B = 690.74
D = 646.63
Method: Trigonometric levelling to a base line
(bench mark A at one end).

Dam Site No. 46

Related to BPT337 (Map 2227D1,
2860 feet = 871.73 metres)
Two bench marks A and B, pipes or pins in concrete.
A = 858.32
B = 859.53
Method: Barometric levelling.

Dam Site No. 55

Related to BPS12 (Map 2327A3,
3284 feet = 1000.96 metres)
Two bench marks, X and Y, pins in concrete.
X = 916.21
Y = 906.44
Method: Trigonometric levelling to a base line, then
horizontal levelling.

Dam Site No. 56

Related to deck level of road bridge over River Bonwapitse (Map 2326B3, 942.49 metres).

Two bench marks, A and B, pipes in concrete.

A = 926.44

B = 930.66

Method: Barometric levelling.

Dam Site No. 61

Related to BPS117 (Map 2425D1, 3758 feet = 1145.44 metres)

One bench mark, C, pipe in concrete.

Five survey stations, A, B, D, E and F, paint marks on rock.

C = 1072.20

A = 1072.28

B = 1060.42

D = 1063.82

E = 1077.04

F = 1072.36

Method: Trigonometric levelling to a base line, then horizontal levelling to C.

Dam Site No. 62

Related to BPT239 (24° 43'S, 25° 32'E, 3602.8 feet = 1098.13 metres).

Two bench marks, B and C, pins in concrete.

Two survey stations, A and D, paint marks on rock.

B = 1055.43

C = 1057.23

A = 1064.39

D = 1064.11

Method: Trigonometric levelling to a base line (survey station A at one end).

Dam Site No. 70

Related to BPT287 (Map 2228B2, 1999 feet = 609.30 metres).

Two bench marks, A and B, pins in concrete.

One survey station, C, paint mark on rock.

A = 583.31

B = 574.11

C = 599.18

Method: Trigonometric levelling to a base line and to survey station C.

APPENDIX C

EARTHWORK QUANTITIES FOR STAGE CONSTRUCTION

As stated in Chapter 4, stage construction could be more economic than initial construction to maximum height for some, but not all, sites.

We consider that stage construction would not be worthwhile in the case of Dam Sites Nos. 3, 13, 55, 56, 61 and 62 because the yield even at maximum height is low. For Dam Site No. 40 a lower retention level than that proposed would increase the volume of excavation for the spillway channel to such an extent that its increased cost would probably outweigh the saving in cost of the lower embankment required.

However, for the remaining sites, namely Nos. 10, 17, 33, 42A, 46 and 70, we consider the stage construction could be worth consideration, and we therefore give in the following Tables details of earthwork volumes over a range of crest heights.

All levels stated are related to field survey datum.

DAM SITE NO. 10

DAM CREST LEVEL (m)		1 101	1 099	1 097	
CLAY CORE	(m^3)	100 100	84 600	71 100	
GENERAL FILL	(m^3)	590 300	481 900	388 200	
GRAVEL	(m^3)	43 300	37 100	31 700	
RIPRAP	(m^3)	27 800	24 200	21 100	
TOTAL FILL	(m^3)	761 500	627 800	512 100	

DAM SITE NO. 17 (SOUTHERN ALIGNMENT)

DAM CREST LEVEL (m)		946	944	942	940
CLAY CORE	(m^3)	271 900	207 400	148 200	109 200
GENERAL FILL	(m^3)	1 276 500	943 700	662 900	466 700
GRAVEL	(m^3)	130 600	101 900	73 800	55 000
RIPRAP	(m^3)	92 300	73 700	54 800	42 400
TOTAL FILL	(m^3)	1 771 300	1 326 700	939 700	673 300

DAM SITE NO. 33

DAM CREST LEVEL (m)		768	766	764	762
CLAY CORE	(m^3)	382 800	328 800	279 200	233 700
GENERAL FILL	(m^3)	2 278 700	1 866 800	1 501 900	1 182 400
GRAVEL	(m^3)	163 700	143 300	124 200	106 400
RIPRAP	(m^3)	109 400	98 300	87 500	77 300
TOTAL FILL	(m^3)	2 934 600	2 437 200	1 992 800	1 599 800

DAM SITE NO. 42A (DOWNSTREAM ALIGNMENT)

DAM CREST LEVEL (m)		661	660	659
CLAY CORE	(m^3)	118 700	103 700	89 800
GENERAL FILL	(m^3)	594 400	512 600	439 600
GRAVEL	(m^3)	55 300	48 700	42 600
RIPRAP	(m^3)	37 300	33 100	29 100
TOTAL FILL	(m^3)	805 700	698 100	601 100

DAM SITE NO. 46 (UPSTREAM ALIGNMENT)

DAM CREST LEVEL (m)		870	868	866
CLAY CORE	(m^3)	182 000	151 200	122 900
GENERAL FILL	(m^3)	1 015 900	800 700	615 300
GRAVEL	(m^3)	79 700	67 600	56 300
RIPRAP	(m^3)	52 500	45 600	38 900
TOTAL FILL	(m^3)	1 330 100	1 065 100	833 400

DAM SITE NO. 70

DAM CREST LEVEL (m)		572	570	568	566
CLAY CORE	(m^3)	312 400	233 800	176 700	132 600
GENERAL FILL	(m^3)	1 733 900	1 337 800	1 034 200	797 300
GRAVEL	(m^3)	144 100	107 000	79 400	58 700
RIPRAP	(m^3)	96 600	72 000	53 900	40 400
TOTAL FILL	(m^3)	2 287 000	1 750 600	1 344 200	1 029 000

APPENDIX D

MAKE-UP OF COST ESTIMATES

For each dam site the following items have been taken into account:-

Item 1 : Impervious clay core.
End 1975 rate for Shashe Dam P 2.5 per cu.m.
Rate adjusted for overhaul (at P 0.2 per cu.m.km.) and any local factors.

Item 2 : General fill in shell of dam.
End 1975 rate for Shashe Dam P 1.5 per cu.m.
Rate adjusted for overhaul and any local factors.

Item 3 : Upstream and downstream gravel.
End 1975 rate for Shashe Dam P 2.5 per cu.m.
Rate adjusted for overhaul and any local factors.

Item 4 : Riprap.
End 1975 rate for Shashe Dam P 5.0 per cu.m.
Rate adjusted for overhaul and any local factors.

Item 5 : Sub-total: fill.
Note: the above items include only the volume of the earth dam above general stripping level, which has been taken as 0.5 m. below existing ground level.

Item 6 : A percentage of Item 5 to cover the cost of site clearance and stripping, excavation and foundation preparation for the core trench, backfilling of core trench and instrumentation. This item was 55 per cent for Shashe Dam - we have estimated a figure by comparing the situation at each site with that at Shashe Dam, but as all the work is below existing ground level the result is at best very approximate. The values adopted are lower than for Shashe Dam because a high proportion of the contingency item could reasonably be considered to be covering this work.

Item 7 : Sub-total: earth dam.

Item 8 : A percentage of Item 7 to cover the cost of cofferdams and dealing with water. This item was 4 per cent for Shashe Dam. We have adjusted for local factors.

Item 9 : Spillway.

End 1975 for Shashe Dam:

Soft excavation	P 1.75	per cu.m.
Rock excavation	P 4.5	per cu.m.
Concrete including formwork	P 35	per cu.m.

Item 10 : Training walls.

Rates as for Item 9.

Item 11 : Drilling and grouting.

A percentage of Items 7 plus 9. This item was 2.5 per cent for Shashe Dam - we have estimates a figure for each site based on the geological reconnaissance, but as all the work is below ground the result is at best very approximate.

Item 12 : Outlet works concete.

End 1975 for Shashe Dam P 220 per cu.m. of structural concrete including formwork, reinforcement, etc.

Item 13 : Outlet works pipes and fittings.

A percentage of Item 12. This item was 28 per cent for Shashe Dam. Adjusted for local factors (e.g. maximum flow rate expected from outlet works).

Item 14 : Any special factors (e.g. very long access roads for remote sites).

Item 15 : Sub-total.

Item 16 : General and miscellaneous items.
We have allowed 25 per cent of Item 15.

Item 17 : Sub-total.

Item 18 : Contingencies.
A percentage of Item 17. Generally 20 per cent has been allowed, but more for some sites where foundation conditions are suspect.

Item 19 : Sub-total.

Item 20 : Engineering, administration and site investigations. We have allowed 10 per cent of Item 19.

Details for each dam site are given in the following Tables.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 3

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	49,200 cu.m.	P.4.0	197
2	General fill	181,000 cu.m.	P.1.7	308
3	Gravel	25,500 cu.m.	P.2.5	64
4	Riprap	18,600 cu.m.	P.5.0	93
5	Sub-total: fill	274,300 cu.m.		662
6	Cut-off etc.	% of Item 5	25 %	166
7	Sub-total: earth dam			828
8	Cofferdams etc.	% of Item 7	4 %	33
9	Spillway soft excavation	22,000 cu.m.	P.1.75	38
	Spillway rock excavation	2,700 cu.m.	P.4.5	12
	Spillway concrete	1,000 cu.m.	P.35	35
10	Training walls soft excavation	300 cu.m.	P.1.75	)
	Training walls rock excavation	100 cu.m.	P.4.5	) 1
	Training walls concrete	600 cu.m.	P.35	21
11	Drilling and grouting	% of Items 7 + 9	2.5%	23
12	Outlet works concrete	270 cu.m.	P.220	59
13	Outlet works pipes etc.	% of Item 12	25 %	15
14	Special factors*			-
15	Sub-total			1,065
16	General & Miscellaneous items	% of Item 15	25 %	266
17	Sub-total			1,331
18	Contingencies	% of Item 17	20 %	266
19	Sub-total			1,597
20	Engineering, administration and site investigations	% of Item 19	10 %	160
	Total Estimated Cost of Dam			1,757

Crest level (field survey datum) = 1,111 m

Retention level (field survey datum) = 1,108 m

* None

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 10

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	100,100 cu.m.	P.4.0	400
2	General fill	590,300 cu.m.	P.1.7	1,004
3	Gravel	43,300 cu.m.	P.2.5	108
4	Riprap	27,800 cu.m.	P.6.0	167
5	Sub-total: fill	761,500 cu.m.		1,679
6	Cut-off etc.	% of Item 5	20 %	336
7	Sub-total: earth dam			2,015
8	Cofferdams etc.	% of Item 7	5 %	101
9	Spillway soft excavation	1,750 cu.m.	P.1.75	3
	Spillway rock excavation	1,500 cu.m.	P.4.5	7
	Spillway concrete	7,000 cu.m.	P.35	245
10	Training walls soft excavation	1,200 cu.m.	P.1.75	2
	Training walls rock excavation	500 cu.m.	P.4.5	2
	Training walls concrete	6,000 cu.m.	P.35	210
11	Drilling and grouting	% of Items 7 + 9	2.5%	57
12	Outlet works concrete	680 cu.m.	P.220	150
13	Outlet works pipes etc.	% of Item 12	25 %	38
14	Special factors*			-
15	Sub-total			2,830
16	General & Miscellaneous items	% of Item 15	25 %	708
17	Sub-total			3,538
18	Contingencies	% of Item 17	20 %	708
19	Sub-total			4,246
20	Engineering, administration and site investigations	% of Item 19	10 %	425
	Total Estimated Cost of Dam			4,671

Crest level (field survey datum) = 1,101 m

Retention level (field survey datum) = 1,196 m

* None

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 13

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	37,700 cu.m.	P.2.7	102
2	General fill	131,700 cu.m.	P.1.7	224
3	Gravel	21,000 cu.m.	P.2.5	53
4	Riprap	15,500 cu.m.	P.6.0	93
5	Sub-total: fill	205,900 cu.m.		472
6	Cut-off etc.	% of Item 5	25 %	118
7	Sub-total: earth dam			590
8	Cofferdams etc.	% of Item 7	4 %	24
9	Spillway soft excavation	200 cu.m.	P.1.75	)
	Spillway rock excavation	200 cu.m.	P.4.5	) 1
	Spillway concrete	1,500 cu.m.	P.35	53
10	Training walls soft excavation	100 cu.m.	P.1.75	)
	Training walls rock excavation	100 cu.m.	P.4.5	) 1
	Training walls concrete	900 cu.m.	P.35	32
11	Drilling and grouting	% of Items 7 + 9	2.5%	16
12	Outlet works concrete	320 cu.m.	P.220	70
13	Outlet works pipes etc.	% of Item 12	25 %	18
14	Special factors*			-
15	Sub-total			805
16	General & Miscellaneous items	% of Item 15	25 %	201
17	Sub-total			1,006
18	Contingencies	% of Item 17	20 %	201
19	Sub-total			1,207
20	Engineering, administration and site investigations	% of Item 19	10 %	121
	Total Estimated Cost of Dam			1,328

Crest level (field survey datum) = 1,039 m

Retention level (field survey datum) = 1,036 m

* None

MAKE-UP OF COST ESTIMATE FOR DAM SITE NO. 17 (Northern Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	320,300 cu.m.	P.2.7	865
2	General fill	1,450,200 cu.m.	P.1.5	2,175
3	Gravel	154,400 cu.m.	P.2.5	386
4	Riprap	107,200 cu.m.	P.5.0	536
5	Sub-total: fill	2,032,100 cu.m.		3,962
6	Cut-off etc.	% of Item 5	25 %	991
7	Sub-total: earth dam			4,953
8	Cofferdams etc.	% of Item 7	5 %	248
9	Spillway soft excavation	15,000 cu.m.	P.1.75	26
	Spillway rock excavation	45,000 cu.m.	P.4.5	202
	Spillway concrete	2,000 cu.m.	P.35	70
10	Training walls soft excavation	200 cu.m.	P.1.75	)
	Training walls rock excavation	300 cu.m.	P.4.5	) 2
	Training walls concrete	1,500 cu.m.	P.35	52
11	Drilling and grouting	% of Items 7 + 9	2.5%	131
12	Outlet works concrete	580 cu.m.	P.220	128
13	Outlet works pipes etc.	% of Item 12	27 %	34
14	Special factors*			-
15	Sub-total			5,846
16	General & Miscellaneous items	% of Item 15	25 %	1,462
17	Sub-total			7,308
18	Contingencies	% of Item 17	20 %	1,462
19	Sub-total			8,770
20	Engineering, administration and site investigations	% of Item 19	10 %	877
	Total Estimated Cost of Dam			9,647

Crest level (field survey datum) = 947 m

Retention level (field survey datum) = 942 m

* None

MAKE-UP OF COST ESTIMATE FOR DAM SITE NO. 17 (Southern Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	271,900 cu.m.	P.2.7	734
2	General fill	1,276,500 cu.m.	P.1.5	1,915
3	Gravel	130,600 cu.m.	P.2.5	327
4	Riprap	92,300 cu.m.	P.5.0	462
5	Sub-total: fill	1,771,300 cu.m.		3,438
6	Cut-off etc.	% of Item 5	25 %	860
7	Sub-total: earth dam			4,298
8	Cofferdams etc.	% of Item 7	5 %	215
9	Spillway soft excavation	20,000 cu.m.	P.1.75	35
	Spillway rock excavation	80,000 cu.m.	P.4.5	360
	Spillway concrete	500 cu.m.	P.35	18
10	Training walls soft excavation	100 cu.m.	P.1.75	)
	Training walls rock excavation	100 cu.m.	P.4.5	) 1
	Training walls concrete	200 cu.m.	P.35	7
11	Drilling and grouting	% of Items 7 + 9	2.5%	118
12	Outlet works concrete	550 cu.m.	P.220	121
13	Outlet works pipes etc.	% of Item 12	27 %	33
14	Special factors*			-
15	Sub-total			5,206
16	General & Miscellaneous items	% of Item 15	25 %	1,301
17	Sub-total			6,507
18	Contingencies	% of Item 17	20 %	1,301
19	Sub-total			7,808
20	Engineering, administration and site investigations	% of Item 19	10 %	781
	Total Estimated Cost of Dam			8,589

Crest level (field survey datum) = 946 m

Retention level (field survey datum) = 941 m

* None

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 33

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	382,800 cu.m.	P.2.7	1,034
2	General fill	2,278,700 cu.m.	P.1.5	3,418
3	Gravel	163,700 cu.m.	P.2.5	409
4	Riprap	109,400 cu.m.	P.5.0	547
5	Sub-total: fill	2,934,600 cu.m.		5,408
6	Cut-off etc.	% of Item 5	30 %	1,622
7	Sub-total: earth dam			7,030
8	Cofferdams etc.	% of Item 7	7 %	492
9	Spillway soft excavation	4,000 cu.m.	P.1.75	7
	Spillway rock excavation	6,000 cu.m.	P.4.5	27
	Spillway concrete	20,000 cu.m.	P.35	700
10	Training walls soft excavation	1,200 cu.m.	P.1.75	2
	Training walls rock excavation	2,300 cu.m.	P.4.5	10
	Training walls concrete	9,500 cu.m.	P.35	332
11	Drilling and grouting	% of Items 7 + 9	3 %	233
12	Outlet works concrete	600 cu.m.	P.220	132
13	Outlet works pipes etc.	% of Item 12	30 %	40
14	Special factors*			500
15	Sub-total			9,505
16	General & Miscellaneous items	% of Item 15	25 %	2,376
17	Sub-total			11,881
18	Contingencies	% of Item 17	25 %	2,970
19	Sub-total			14,851
20	Engineering, administration and site investigations	% of Item 19	10 %	1,485
	Total Estimated Cost of Dam			16,336

Crest level (field survey datum) = 768 m

Retention level (field survey datum) = 763 m

* Improvements to access roads.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 40 (Upstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	201,800 cu.m.	P.3.0	605
2	General fill	1,572,400 cu.m.	P.1.8	2,830
3	Gravel	77,900 cu.m.	P.2.5	195
4	Riprap	47,700 cu.m.	P.3.0	143
5	Sub-total: fill	1,899,800 cu.m.		3,773
6	Cut-off etc.	% of Item 5	25 %	943
7	Sub-total: earth dam			4,716
8	Cofferdams etc.	% of Item 7	8 %	377
9	Spillway soft excavation	60,000 cu.m.	P.1.75	105
	Spillway rock excavation	535,000 cu.m.	P.4.5	2,407
	Spillway concrete	500 cu.m.	P.35	18
10	Training walls soft excavation	-		-
	Training walls rock excavation			-
	Training walls concrete	-		-
11	Drilling and grouting	% of Item 7	4 %	188
12	Outlet works concrete	1,330 cu.m.	P.220	293
13	Outlet works pipes etc.	% of Item 12	30 %	88
14	Special factors*			1,000
15	Sub-total			9,192
16	General & Miscellaneous items	% of Item 15	25 %	2,298
17	Sub-total			11,490
18	Contingencies	% of Item 17	25 %	2,872
19	Sub-total			14,362
20	Engineering, administration and site investigations	% of Item 19	10 %	1,436
	Total Estimated Cost of Dam			15,798

Crest level (field survey datum) = 646 m

Retention level (field survey datum) = 628 m

* Allows for provision of new access roads, improvements to existing access roads, and for additional difficulties due to the remoteness of the site.

MAKE-UP OF COST ESTIMATE FOR
DAM SITE NO. 40 (Downstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	219,100 cu.m.	P.3.0	657
2	General fill	1,733,000 cu.m.	P.1.8	3,119
3	Gravel	83,500 cu.m.	P.2.5	209
4	Riprap	47,700 cu.m.	P.3.0	143
5	Sub-total: fill	2,083,300 cu.m.		4,128
6	Cut-off etc.	% of Item 5	25 %	1,032
7	Sub-total: earth dam			5,160
8	Cofferdams etc.	% of Item 7	8 %	413
9	Spillway soft excavation	70,000 cu.m.	P.1.75	123
	Spillway rock excavation	626,000 cu.m.	P.4.5	2.817
	Spillway concrete	500 cu.m.	P.35	18
10	Training walls soft excavation	-		-
	Training walls rock excavation			-
	Training walls concrete	-		-
11	Drilling and grouting	% of Item 7	4 %	206
12	Outlet works concrete	1,330 cu.m.	P.220	293
13	Outlet works pipes etc.	% of Item 12	30 %	88
14	Special factors*			1,000
15	Sub-total			10,118
16	General & Miscellaneous items	% of Item 15	25 %	2,530
17	Sub-total			12,648
18	Contingencies	% of Item 17	25 %	3,162
19	Sub-total			15,810
20	Engineering, administration and site investigations	% of Item 19	10 %	1,581
	Total Estimated Cost of Dam			17,391

Crest level (field survey datum) = 646 m
Retention level (field survey datum) = 628 m

* Allows for provision of new access roads, improvements to existing access roads, and for additional difficulties due to the remoteness of the site.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 42A (Upstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	153,900 cu.m.	P.4.0	616
2	General fill	814,700 cu.m.	P.1.5	1,222
3	Gravel	70,000 cu.m.	P.3.5	245
4	Riprap	47,300 cu.m.	P.6.0	284
5	Sub-total: fill	1,085,900 cu.m.		2,367
6	Cut-off etc.	% of Item 5	25 %	592
7	Sub-total: earth dam			2,959
8	Cofferdams etc.	% of Item 7	6 %	178
9	Spillway soft excavation	4,000 cu.m.	P.1.75	7
	Spillway rock excavation	6,000 cu.m.	P.4.5	27
	Spillway concrete	2,000 cu.m.	P.35	70
10	Training walls soft excavation	800 cu.m.	P.1.75	)
	Training walls rock excavation	1,200 cu.m.	P.4.5	) 7
	Training walls concrete	6,000 cu.m.	P.35	210
11	Drilling and grouting	% of Items 7 + 9	2.5%	77
12	Outlet works concrete	680 cu.m.	P.220	150
13	Outlet works pipes etc.	% of Item 12	27 %	40
14	Special factors*			500
15	Sub-total			4,225
16	General & Miscellaneous items	% of Item 15	25 %	1,056
17	Sub-total			5,281
18	Contingencies	% of Item 17	20 %	1,057
19	Sub-total			6,338
20	Engineering, administration and site investigations	% of Item 19	10 %	634
	Total Estimated Cost of Dam			6,972

Crest level (field survey datum) = 664 m

Retention level (field survey datum) = 657 m

* Allows for improvements to access roads and for additional difficulties due to the remoteness of the site.

MAKE-UP OF COST ESTIMATE FOR
DAM SITE NO. 42A (Downstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	118,700 cu.m.	P.4.0	475
2	General fill	594,400 cu.m.	P.1.5	892
3	Gravel	55,300 cu.m.	P.3.5	194
4	Riprap	37,300 cu.m.	P.6.0	224
5	Sub-total: fill	805,700 cu.m.		1,785
6	Cut-off etc.	% of Item 5	25 %	446
7	Sub-total: earth dam			2,231
8	Cofferdams etc.	% of Item 7	6 %	134
9	Spillway soft excavation	1,000 cu.m.	P.1.75	)
	Spillway rock excavation	1,000 cu.m.	P.4.5	) 6
	Spillway concrete	5,000 cu.m.	P.35	175
10	Training walls soft excavation	200 cu.m.	P.1.75	)
	Training walls rock excavation	300 cu.m.	P.4.5	) 2
	Training walls concrete	1,500 cu.m.	P.35	53
11	Drilling and grouting	% of Items 7 + 9	2.5%	60
12	Outlet works concrete	620 cu.m.	P.220	136
13	Outlet works pipes etc.	% of Item 12	29 %	40
14	Special factors*			500
15	Sub-total			3,337
16	General & Miscellaneous items	% of Item 15	25 %	834
17	Sub-total			4,171
18	Contingencies	% of Item 17	20 %	834
19	Sub-total			5,005
20	Engineering, administration and site investigations	% of Item 19	10 %	501
	Total Estimated Cost of Dam			5,506

Crest level (field survey datum) = 661 m

Retention level (field survey datum) = 657 m

* Allows for improvements to access roads and for additional difficulties due to the remoteness of the site.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 46 (Upstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	182,000 cu.m.	P.4.0	728
2	General fill	1,015,900 cu.m.	P.1.5	1,524
3	Gravel	79,700 cu.m.	P.3.5	279
4	Riprap	52,500 cu.m.	P.5.0	263
5	Sub-total: fill	1,330,100 cu.m.		2,794
6	Cut-off etc.	% of Item 5	25 %	699
7	Sub-total: earth dam			3,493
8	Cofferdams etc.	% of Item 7	4 %	140
9	Spillway soft excavation	20,000 cu.m.	P.1.75	35
	Spillway rock excavation	17,000 cu.m.	P.4.5	77
	Spillway concrete	7,000 cu.m.	P.35	245
10	Training walls soft excavation	1,500 cu.m.	P.1.75	)
	Training walls rock excavation	1,000 cu.m.	P.4.5	) 7
	Training walls concrete	5,000 cu.m.	P.35	175
11	Drilling and grouting	% of Items 7 + 9	2.5%	96
12	Outlet works concrete	610 cu.m.	P.220	134
13	Outlet works pipes etc.	% of Item 12	27 %	36
14	Special factors*			200
15	Sub-total			4,638
16	General & Miscellaneous items	% of Item 15	25 %	1,160
17	Sub-total			5,798
18	Contingencies	% of Item 17	25 %	1,449
19	Sub-total			7,247
20	Engineering, administration and site investigations	% of Item 19	10 %	725
	Total Estimated Cost of Dam			7,972

Crest level (field survey datum) = 870 m

Retention level (field survey datum) = 865 m

* Allows for improvements to access roads.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 46 (Downstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	192,900 cu.m.	P.4.0	772
2	General fill	1,110,200 cu.m.	P.1.5	1,665
3	Gravel	83,600 cu.m.	P.3.5	293
4	Riprap	54,500 cu.m.	P.5.0	272
5	Sub-total: fill	1,441,200 cu.m.		3,002
6	Cut-off etc.	% of Item 5	25 %	751
7	Sub-total: earth dam			3,753
8	Cofferdams etc.	% of Item 7	4 %	150
9	Spillway soft excavation	5,000 cu.m.	P.1.75	9
	Spillway rock excavation	10,000 cu.m.	P.4.5	45
	Spillway concrete	11,000 cu.m.	P.35	385
10	Training walls soft excavation	1,000 cu.m.	P.1.75	)
	Training walls rock excavation	500 cu.m.	P.4.5	) 4
	Training walls concrete	3,500 cu.m.	P.35	122
11	Drilling and grouting	% of Items 7 + 9	2.5%	105
12	Outlet works concrete	610 cu.m.	P.220	134
13	Outlet works pipes etc.	% of Item 12	27 %	36
14	Special factors*			200
15	Sub-total			4,943
16	General & Miscellaneous items	% of Item 15	25 %	1,236
17	Sub-total			6,179
18	Contingencies	% of Item 17	25 %	1,545
19	Sub-total			7,724
20	Engineering, administration and site investigations	% of Item 19	10 %	772
	Total Estimated Cost of Dam			8,496

Crest level (field survey datum) = 870 m

Retention level (field survey datum) = 865 m

* Allows for improvements to access roads.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 55

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	81,300 cu.m.	P.3.5	285
2	General fill	341,000 cu.m.	P.1.5	512
3	Gravel	40,200 cu.m.	P.3.5	141
4	Riprap	29,500 cu.m.	P.5.0	148
5	Sub-total: fill	492,000 cu.m.		1,086
6	Cut-off etc.	% of Item 5	25 %	272
7	Sub-total: earth dam			1,358
8	Cofferdams etc.	% of Item 7	4 %	54
9	Spillway soft excavation	3,000 cu.m.	P.1.75	5
	Spillway rock excavation	5,500 cu.m.	P.4.5	25
	Spillway concrete	2,500 cu.m.	P.35	88
10	Training walls soft excavation	200 cu.m.	P.1.75	)
	Training walls rock excavation	300 cu.m.	P.4.5	) 2
	Training walls concrete	2,000 cu.m.	P.35	70
11	Drilling and grouting	% of Items 7 + 9	2.5%	37
12	Outlet works concrete	420 cu.m.	P.220	92
13	Outlet works pipes etc.	% of Item 12	25 %	23
14	Special factors*			100
15	Sub-total			1,854
16	General & Miscellaneous items	% of Item 15	25 %	463
17	Sub-total			2,317
18	Contingencies	% of Item 17	20 %	463
19	Sub-total			2,780
20	Engineering, administration and site investigations	% of Item 19	10 %	278
	Total Estimated Cost of Dam			3,058

Crest level (field survey datum) = 919 m

Retention level (field survey datum) = 914 m

* Allows for improvements to access roads.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 56

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	118,700 cu.m.	P.3.5	415
2	General fill	502,500 cu.m.	P.1.5	754
3	Gravel	58,000 cu.m.	P.3.5	203
4	Riprap	40,500 cu.m.	P.5.0	203
5	Sub-total: fill	719,700 cu.m.		1,575
6	Cut-off etc.	% of Item 5	30 %	473
7	Sub-total: earth dam			2,048
8	Cofferdams etc.	% of Item 7	4 %	82
9	Spillway soft excavation	1,500 cu.m.	P.1.75	3
	Spillway rock excavation	3,500 cu.m.	P.4.5	16
	Spillway concrete	1,000 cu.m.	P.35	35
10	Training walls soft excavation	200 cu.m.	P.1.75	)
	Training walls rock excavation	200 cu.m.	P.4.5	) 1
	Training walls concrete	1,200 cu.m.	P.35	42
11	Drilling and grouting	% of Items 7 + 9	2.5%	53
12	Outlet works concrete	420 cu.m.	P.220	92
13	Outlet works pipes etc.	% of Item 12	25 %	23
14	Special factors*			-
15	Sub-total			2,395
16	General & Miscellaneous items	% of Item 15	25 %	599
17	Sub-total			2,994
18	Contingencies	% of Item 17	20 %	599
19	Sub-total			3,593
20	Engineering, administration and site investigations	% of Item 19	10 %	359
	Total Estimated Cost of Dam			3,952

Crest level (field survey datum) = 938 m

Retention level (field survey datum) = 933 m

* None.

MAKE-UP OF COST ESTIMATE FOR

DAM SITE NO. 61 (Upstream Alignment)

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	23,500 cu.m.	P.4.0	94
2	General fill	107,800 cu.m.	P.1.9	205
3	Gravel	11,400 cu.m.	P.2.5	29
4	Riprap	8,000 cu.m.	P.5.0	40
5	Sub-total: fill	150,700 cu.m.		368
6	Cut-off etc.	% of Item 5	20 %	74
7	Sub-total: earth dam			442
8	Cofferdams etc.	% of Item 7	8 %	35
9	Spillway soft excavation	300 cu.m.	P.1.75	)
	Spillway rock excavation	400 cu.m.	P.4.5	) 3
	Spillway concrete	1,500 cu.m.	P.35	53
10	Training walls soft excavation	100 cu.m.	P.1.75	)
	Training walls rock excavation	200 cu.m.	P.4.5	) 1
	Training walls concrete	700 cu.m.	P.35	25
11	Drilling and grouting	% of Items 7 + 9	2.5%	12
12	Outlet works concrete	510 cu.m.	P.220	112
13	Outlet works pipes etc.	% of Item 12	25 %	28
14	Special factors*			-
15	Sub-total			711
16	General & Miscellaneous items	% of Item 15	25 %	178
17	Sub-total			889
18	Contingencies	% of Item 17	20 %	178
19	Sub-total			1,067
20	Engineering, administration and site investigations	% of Item 19	10 %	107
	Total Estimated Cost of Dam			1,174

Crest level (field survey datum) = 1,061 m

Retention level (field survey datum) = 1,057 m

* None

MAKE-UP OF COST ESTIMATE FOR
DAM SITE NO. 62

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	32,800 cu.m.	P.2.7	89
2	General fill	126,900 cu.m.	P.1.7	216
3	Gravel	17,600 cu.m.	P.2.5	44
4	Riprap	12,600 cu.m.	P.6.0	76
5	Sub-total: fill	189,900 cu.m.		425
6	Cut-off etc.	% of Item 5	25 %	106
7	Sub-total: earth dam			531
8	Cofferdams etc.	% of Item 7	4 %	21
9	Spillway soft excavation	300 cu.m.	P.1.75	)
	Spillway rock excavation	500 cu.m.	P.4.5	) 3
	Spillway concrete	1,700 cu.m.	P.35	60
10	Training walls soft excavation	100 cu.m.	P.1.75	)
	Training walls rock excavation	100 cu.m.	P.4.5	) 1
	Training walls concrete	600 cu.m.	P.35	21
11	Drilling and grouting	% of Items 7 + 9	2.5%	15
12	Outlet works concrete	340 cu.m.	P.220	75
13	Outlet works pipes etc.	% of Item 12	25 %	19
14	Special factors*			-
15	Sub-total			746
16	General & Miscellaneous items	% of Item 15	25 %	187
17	Sub-total			933
18	Contingencies	% of Item 17	20 %	187
19	Sub-total			1,120
20	Engineering, administration and site investigations	% of Item 19	10 %	112
	Total Estimated Cost of Dam			1,232

Crest level (field survey datum) = 1,062 m

Retention level (field survey datum) = 1,058 m

* None.

MAKE-UP OF COST ESTIMATE FOR

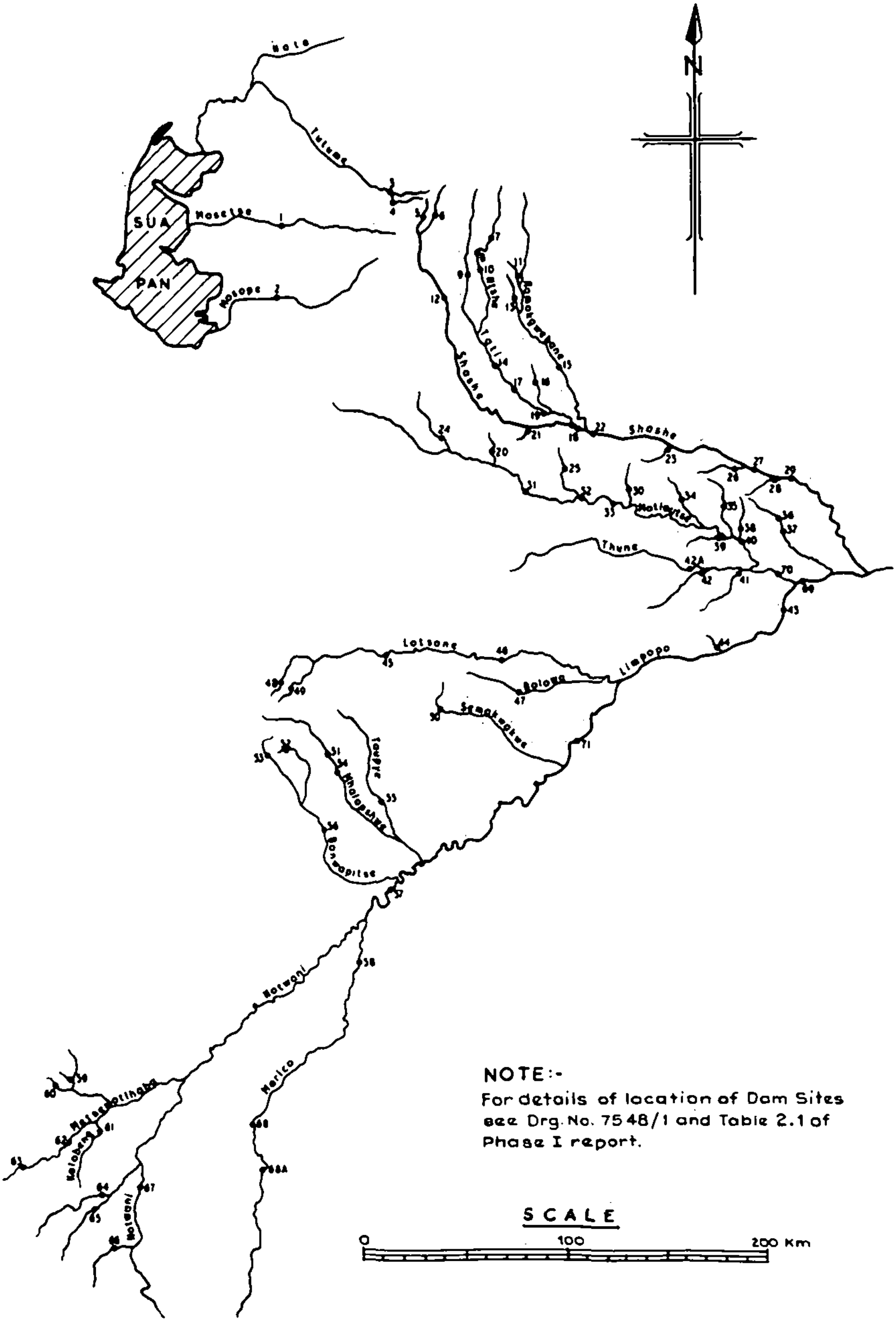
DAM SITE NO. 70

ITEM NO.	DESCRIPTION	QUANTITY	RATE	AMOUNT P. 1000
1	Clay Core	312,400 cu.m.	P.3.0	937
2	General fill	1,733,900 cu.m.	P.1.5	2,601
3	Gravel	144,100 cu.m.	P.2.5	360
4	Riprap	96,600 cu.m.	P.6.0	580
5	Sub-total: fill	2,287,000 cu.m.		4,478
6	Cut-off etc.	% of Item 5	35 %	1,567
7	Sub-total: earth dam			6,045
8	Cofferdams etc.	% of Item 7	7 %	423
9	Spillway soft excavation	10,000 cu.m.	P.1.75	18
	Spillway rock excavation	40,000 cu.m.	P.4.5	180
	Spillway concrete	25,000 cu.m.	P.35	875
10	Training walls soft excavation	500 cu.m.	P.1.75	1
	Training walls rock excavation	1,500 cu.m.	P.4.5	7
	Training walls concrete	7,000 cu.m.	P.35	245
11	Drilling and grouting	% of Items 7 + 9	5 %	356
12	Outlet works concrete	720 cu.m.	P.220	158
13	Outlet works pipes etc.	% of Item 12	30 %	48
14	Special factors*			750
15	Sub-total			9,106
16	General & Miscellaneous items	% of Item 15	25 %	2,277
17	Sub-total			11,383
18	Contingencies	% of Item 17	30 %	3,415
19	Sub-total			14,798
20	Engineering, administration and site investigations	% of Item 19	10 %	1,480
	Total Estimated Cost of Dam			16,278

Crest level (field survey datum) = 572 m

Retention level (field survey datum) = 568 m

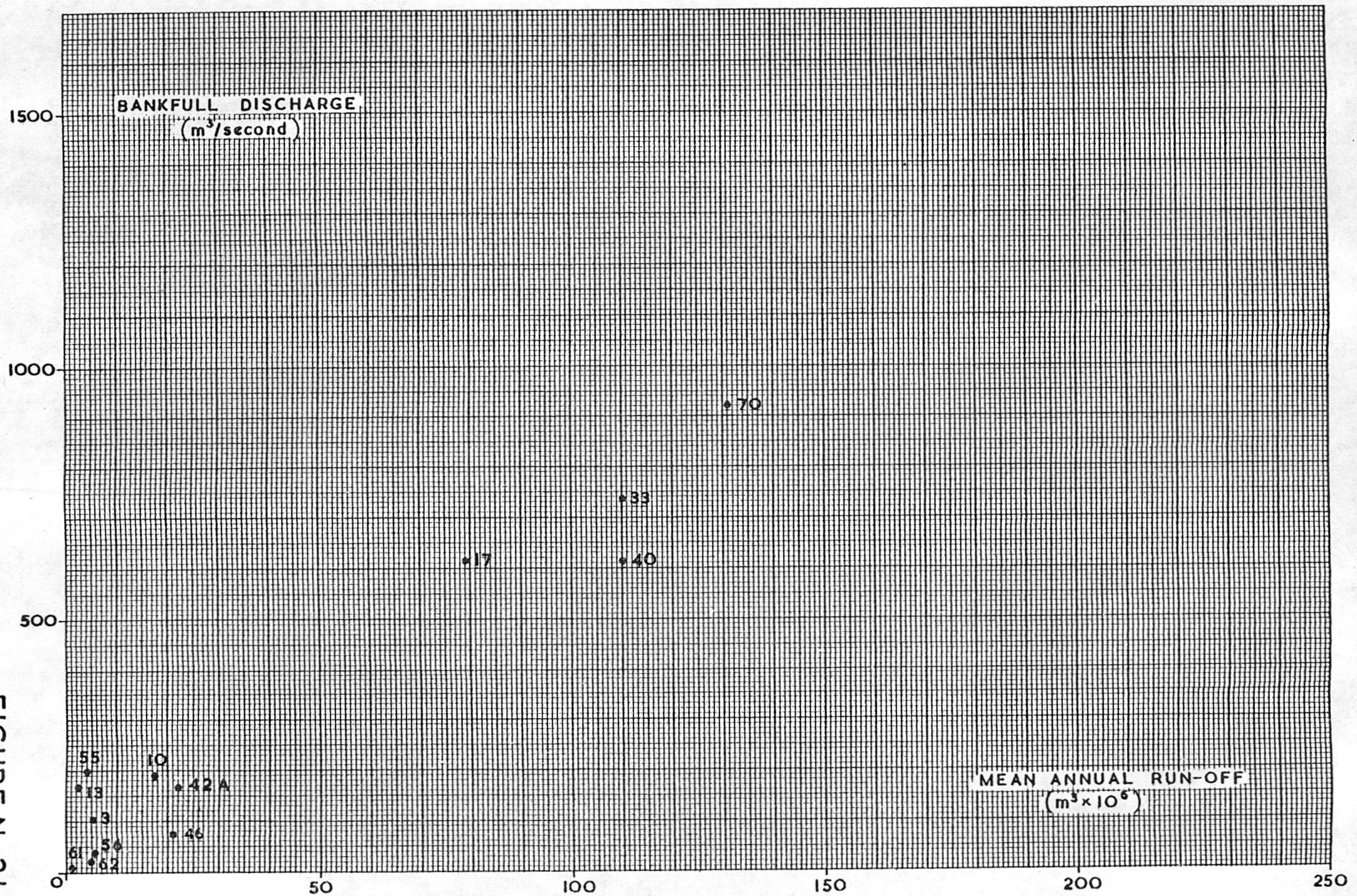
* Allows for improvements to access roads and for additional difficulties due to the remoteness of the site.



NOTE:-
For details of location of Dam Sites see Drg. No. 7548/1 and Table 2.1 of Phase I report.

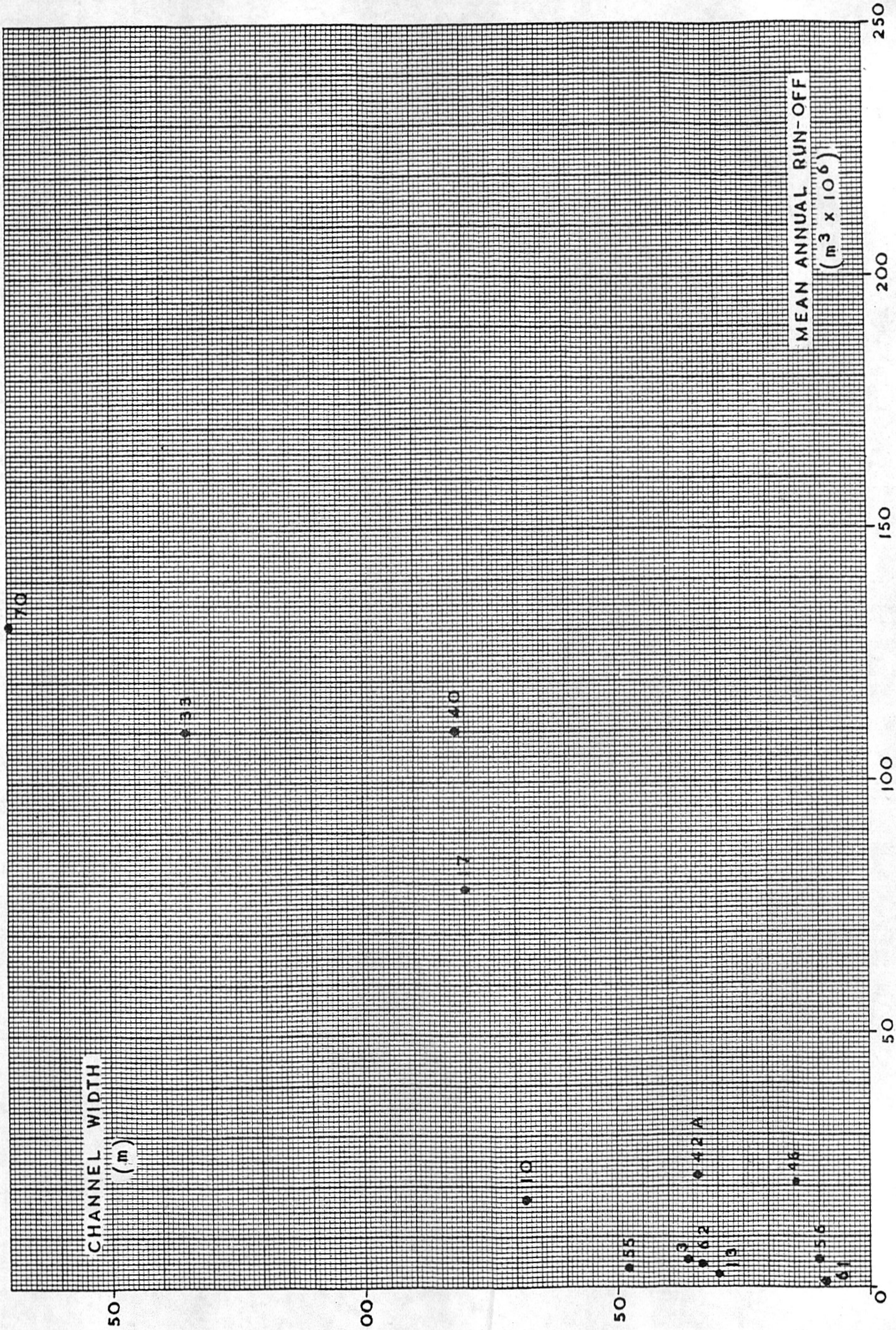
LOCATION OF DAM SITES

FIGURE No. 1.1



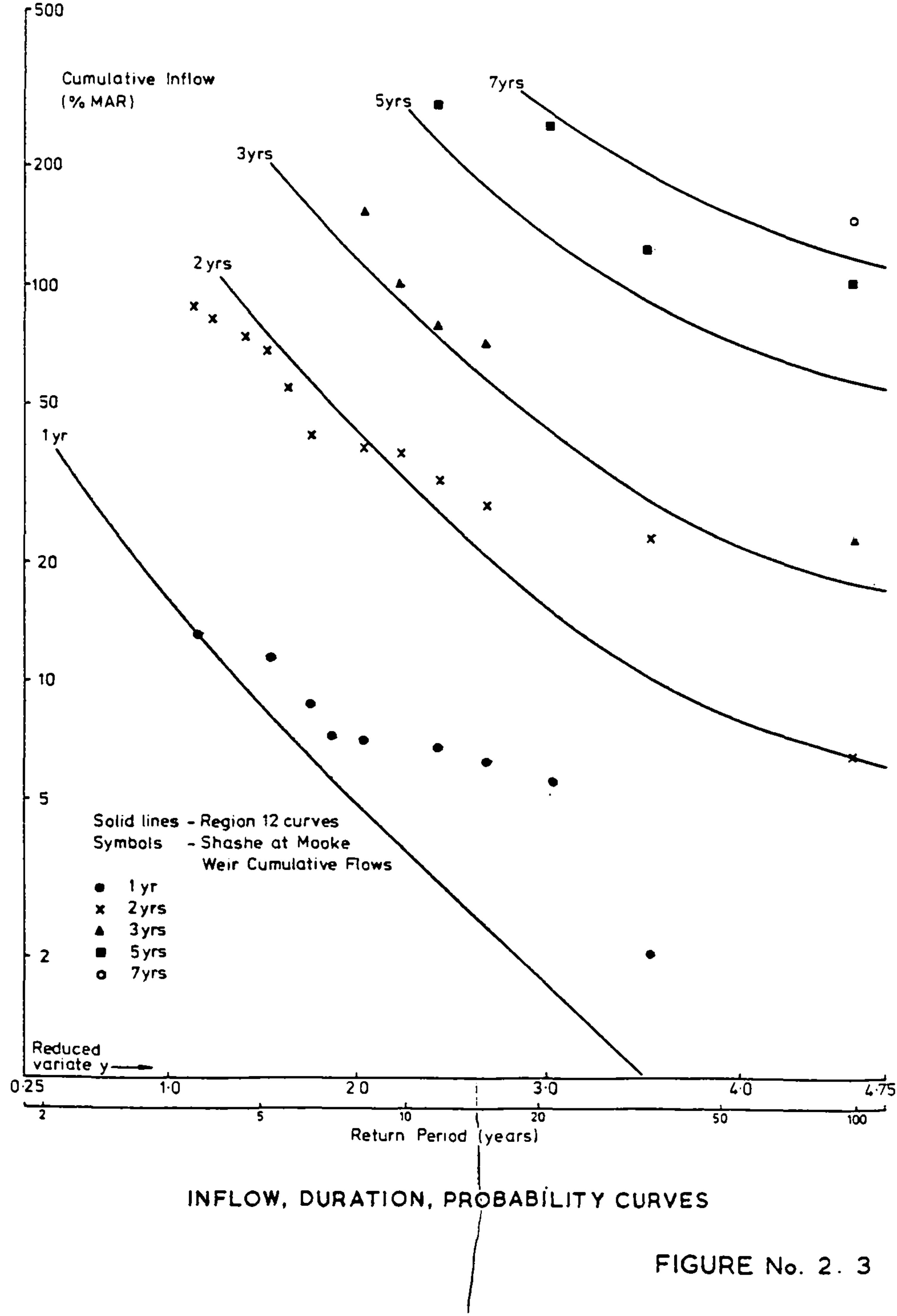
MEAN ANNUAL RUN-OFF AGAINST BANKFULL DISCHARGE

FIGURE No. 2.1



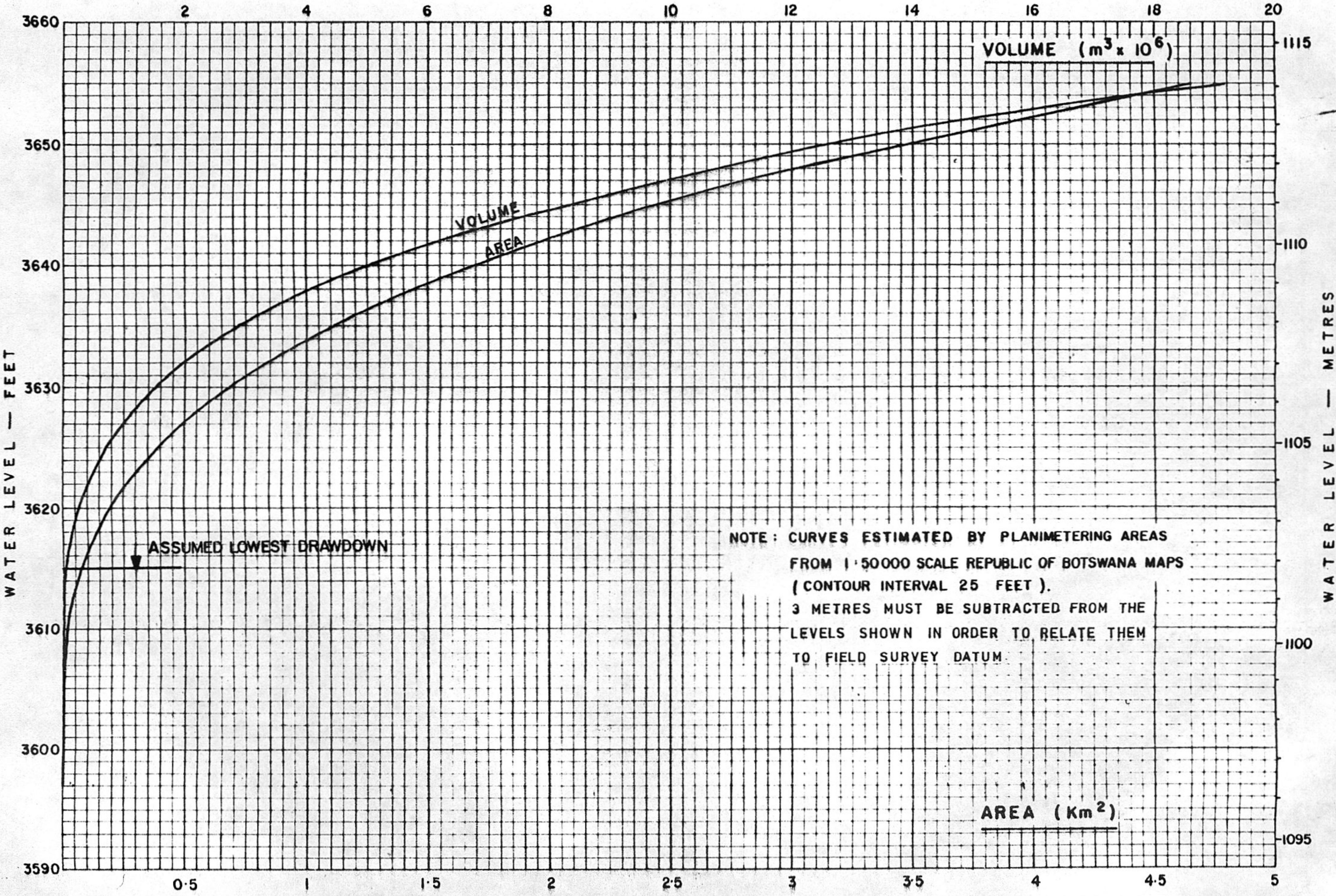
MEAN ANNUAL RUN-OFF AGAINST CHANNEL WIDTH

FIGURE No. 2.2



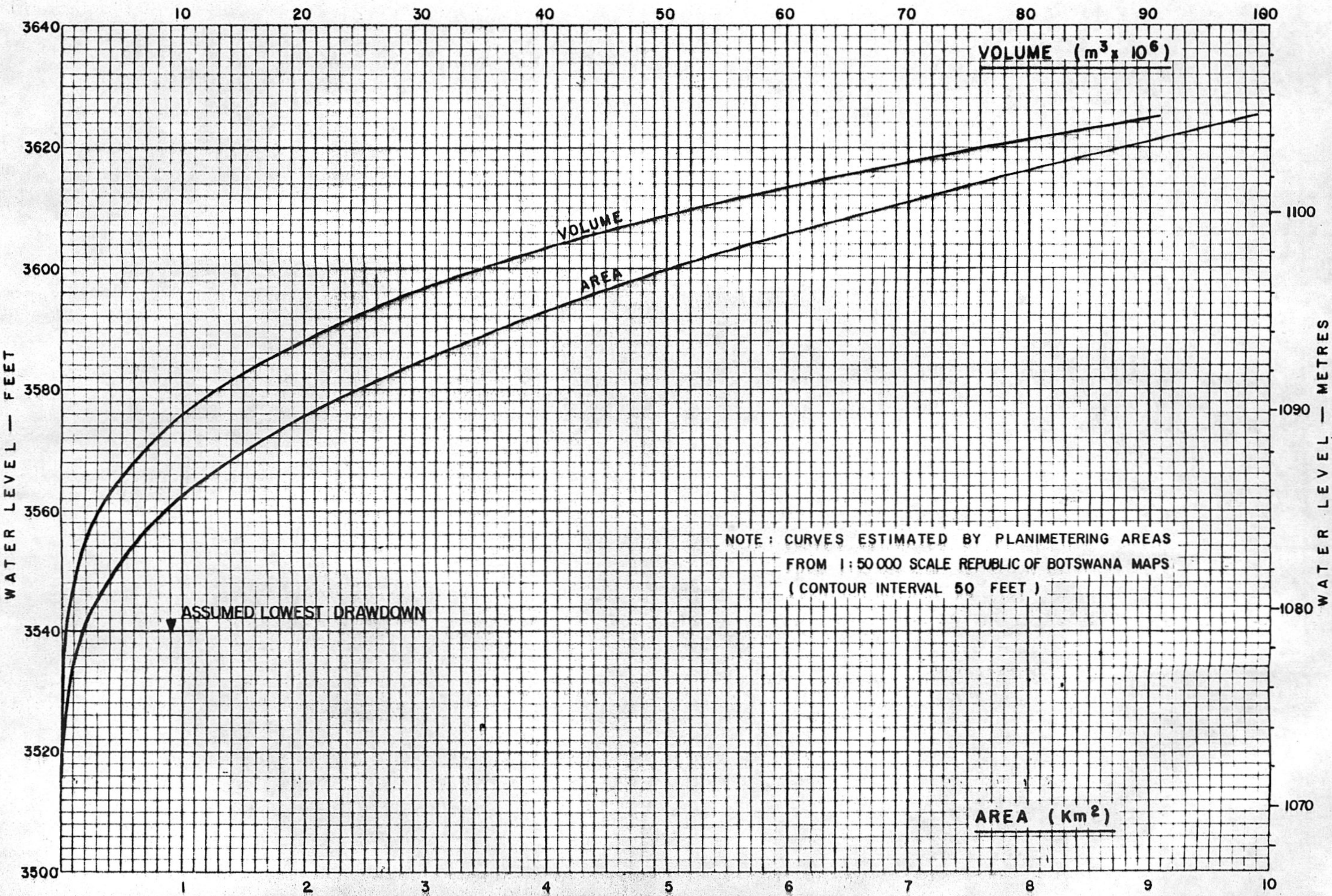
INFLOW, DURATION, PROBABILITY CURVES

FIGURE No. 2. 3



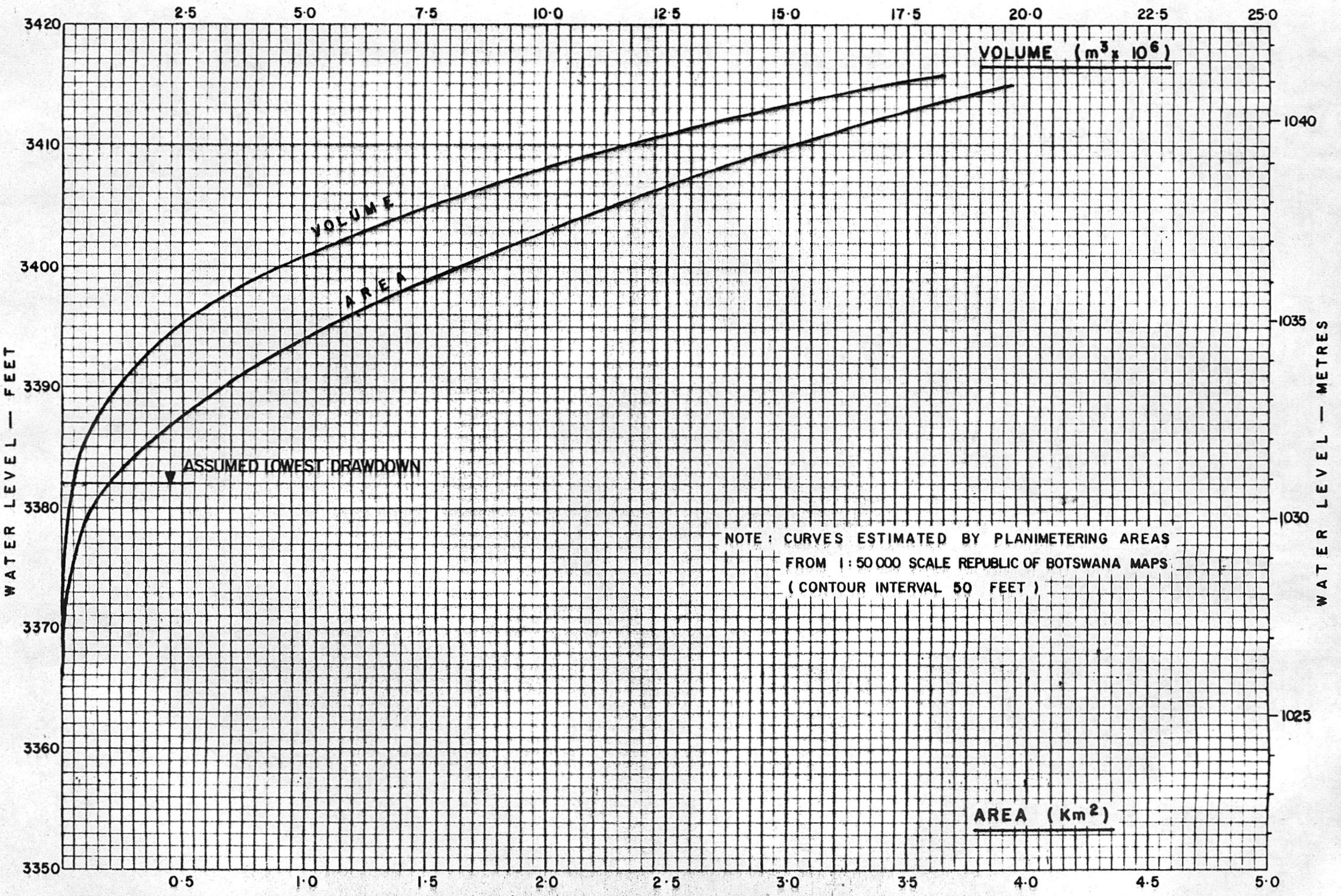
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 3

FIGURE No 2.4



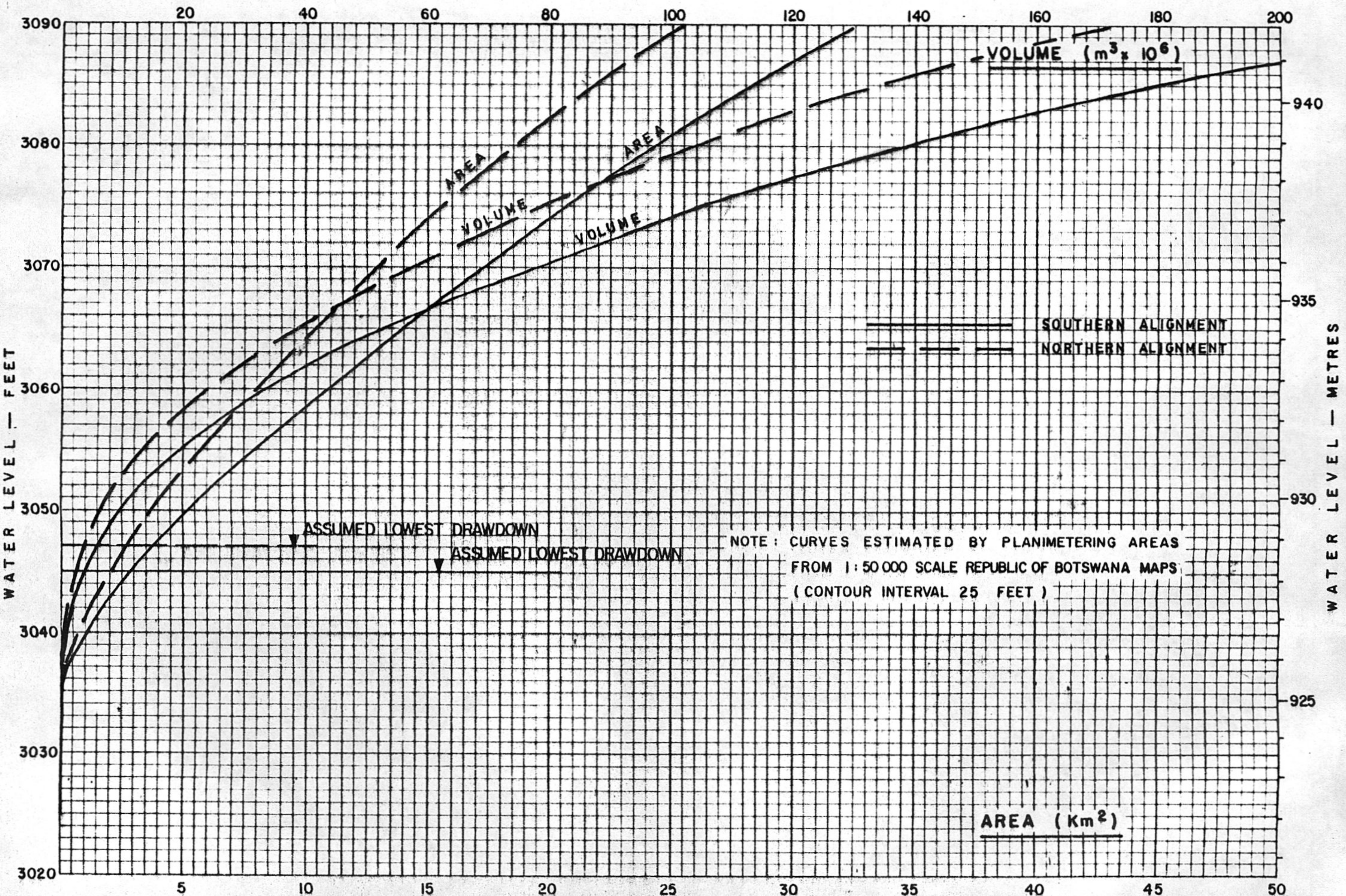
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 10

FIGURE No 2.5



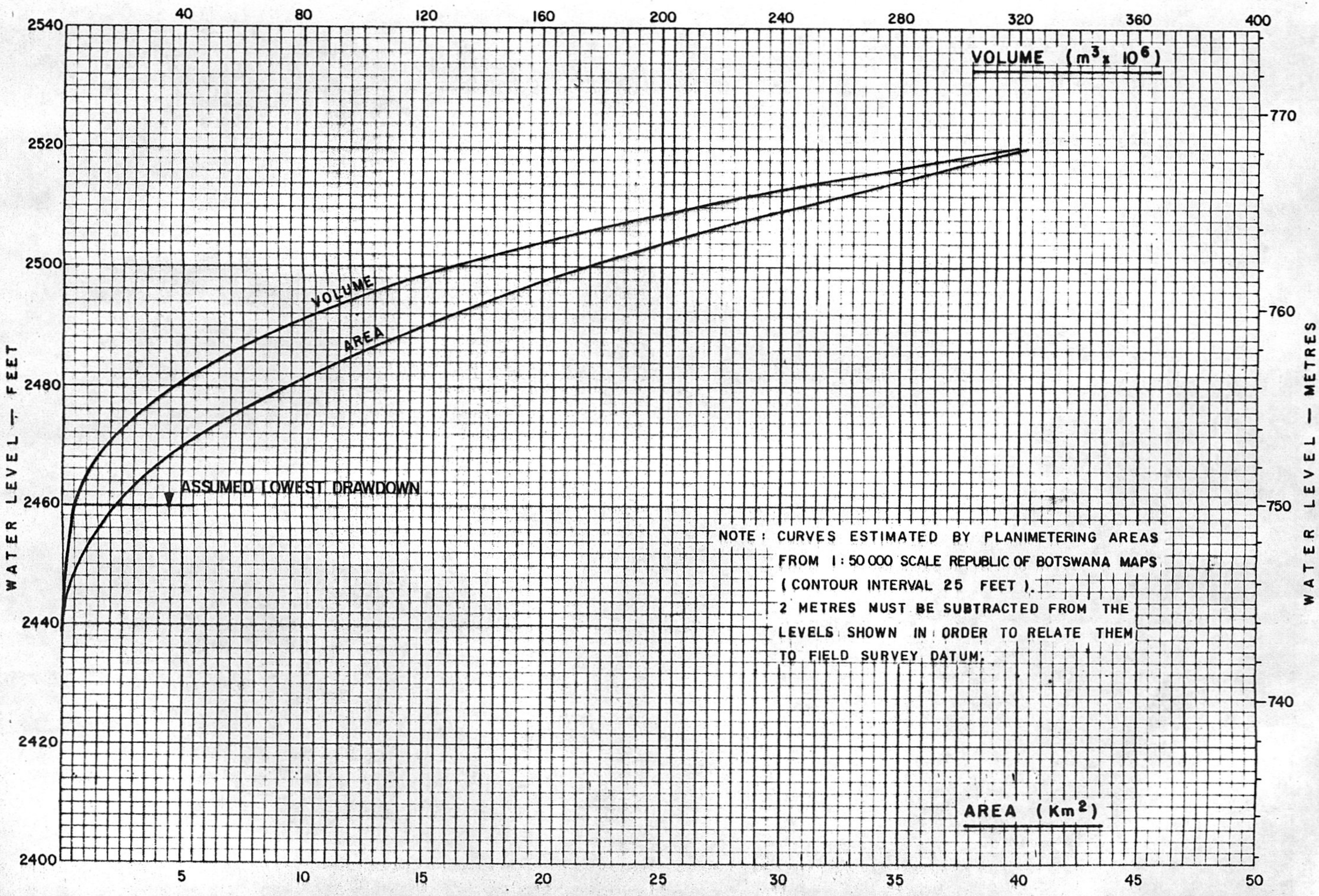
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 13

FIGURE No 2.6



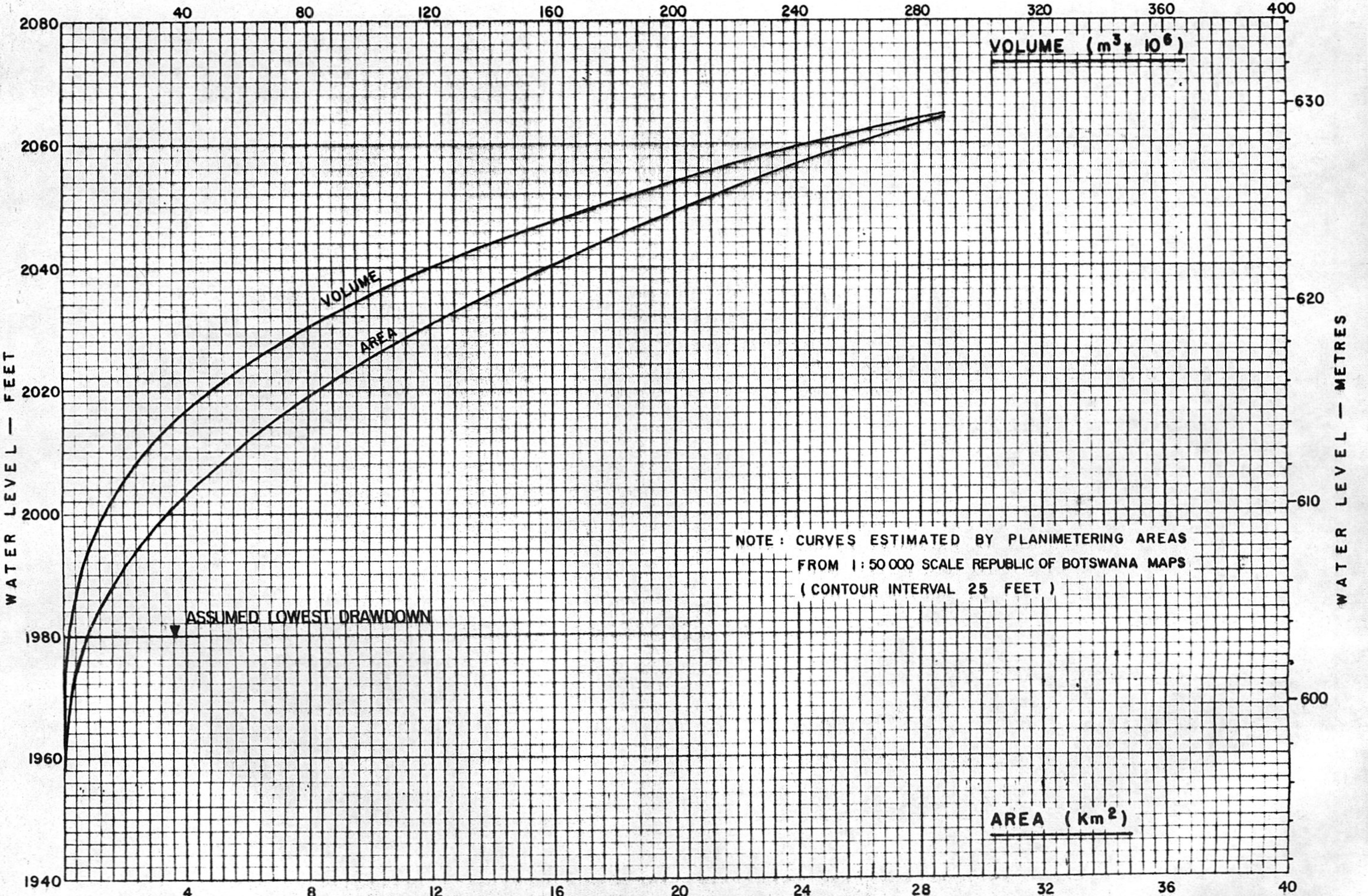
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 17

FIGURE No 2.7



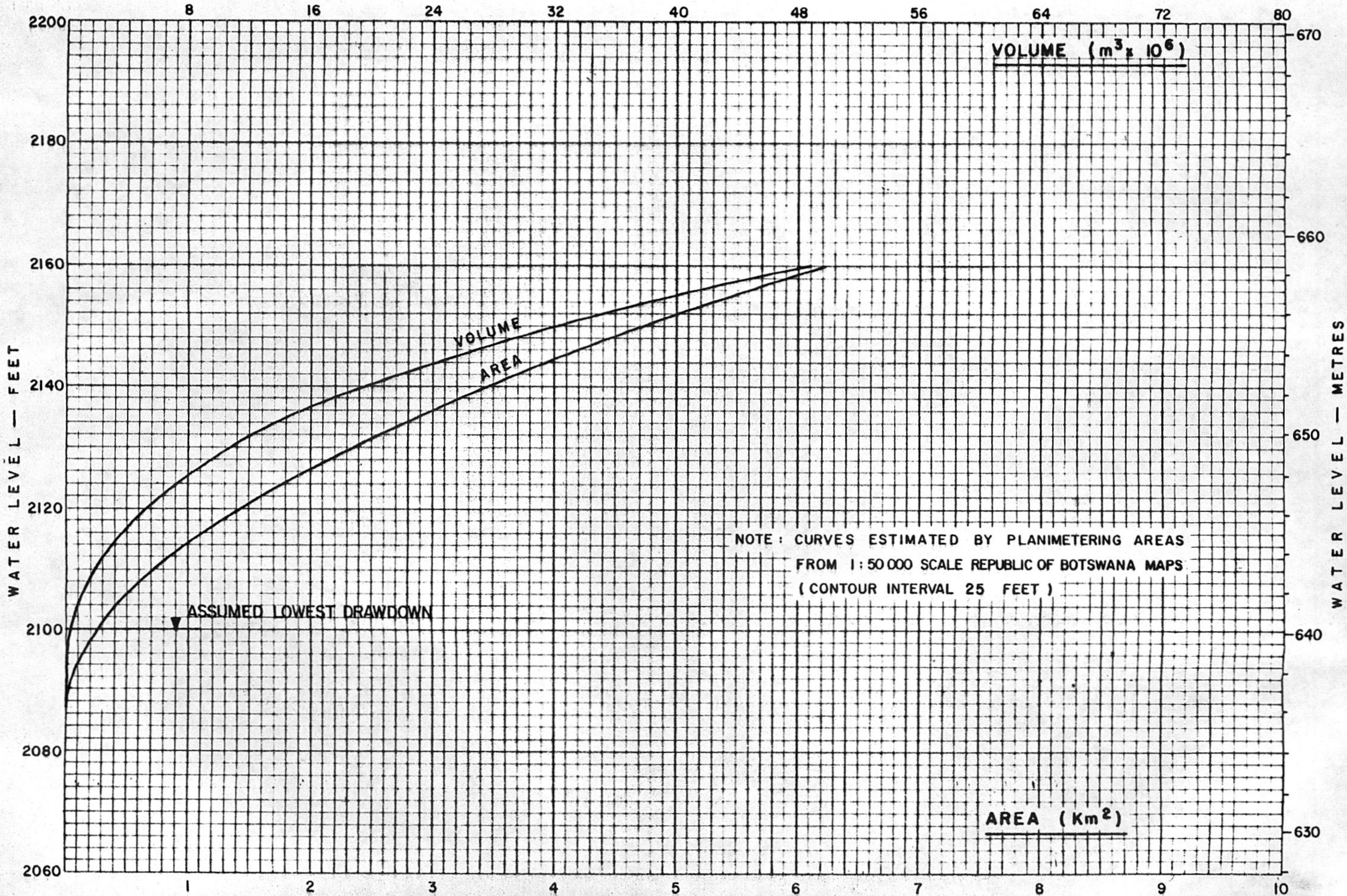
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 33

FIGURE No 2.8



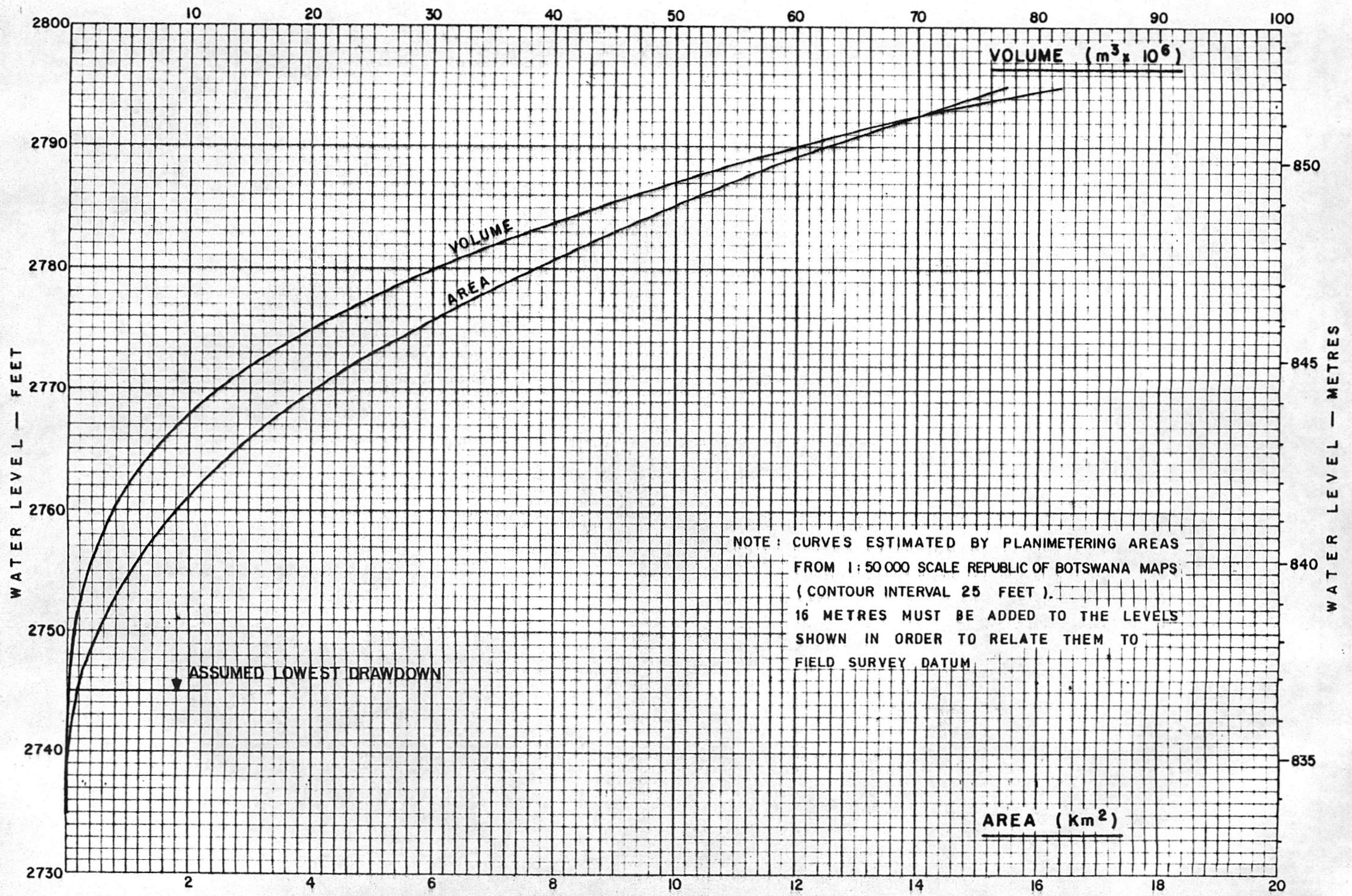
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 40

FIGURE No 2.9



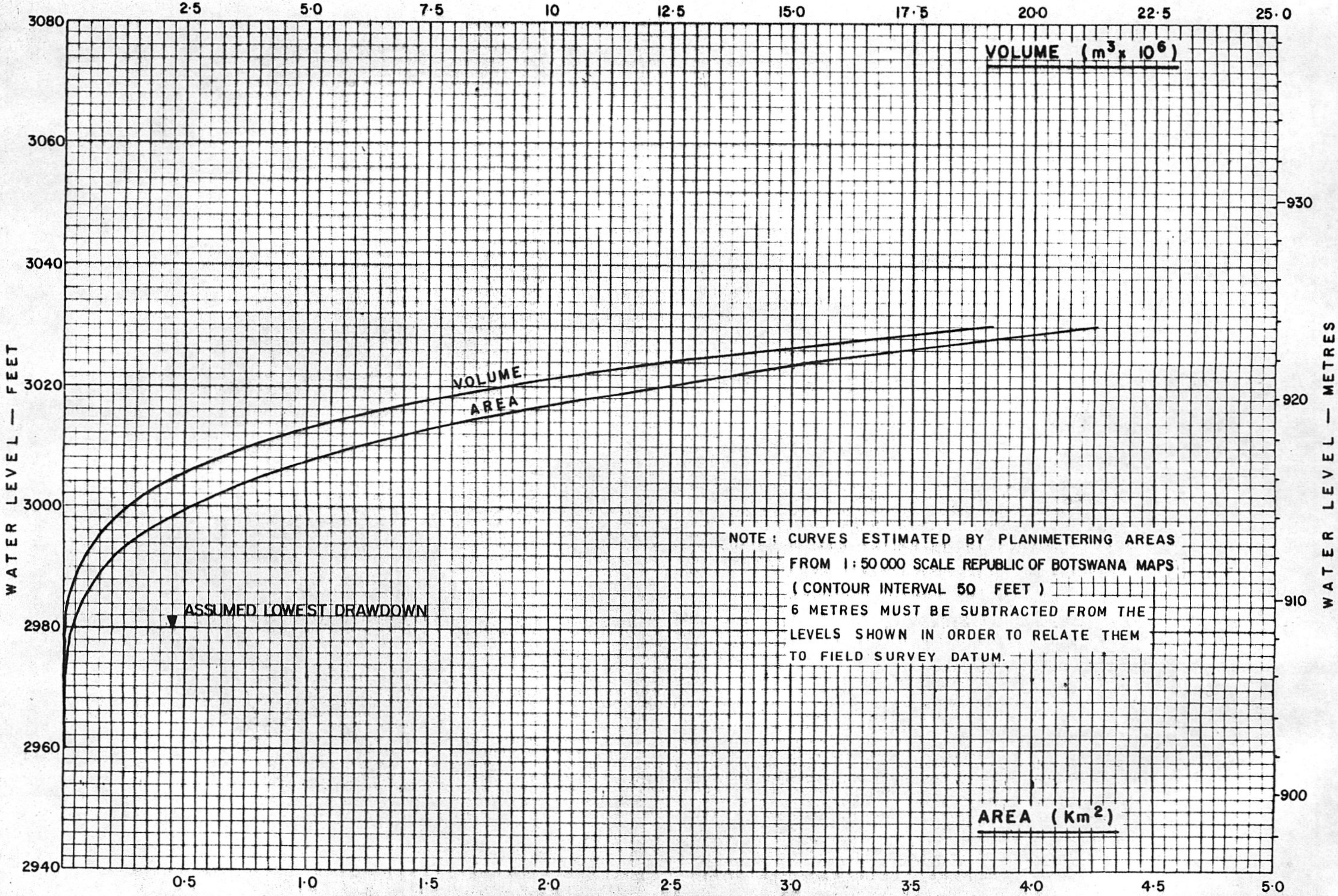
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 42A

FIGURE No 2.10



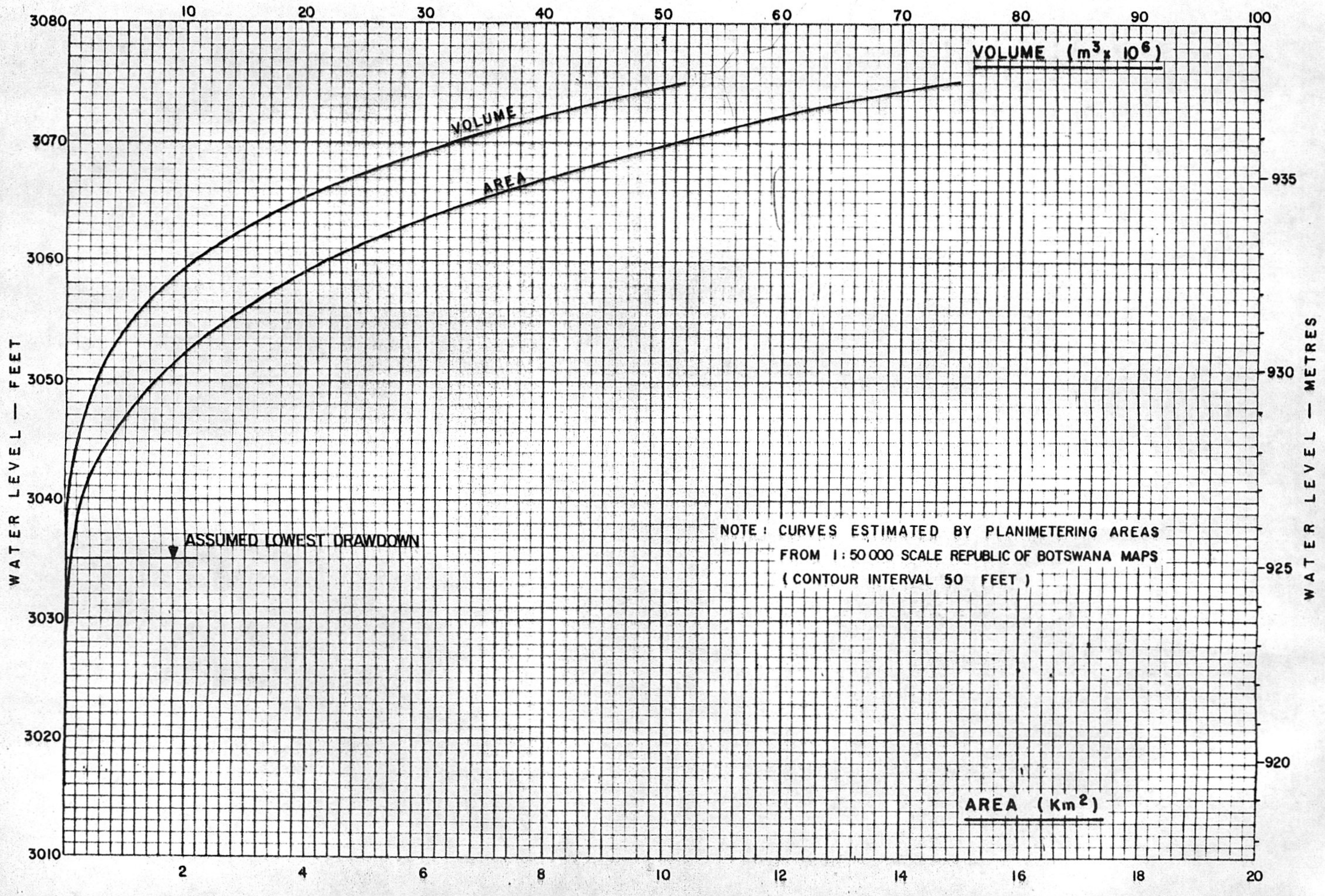
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 46

FIGURE No 2.11



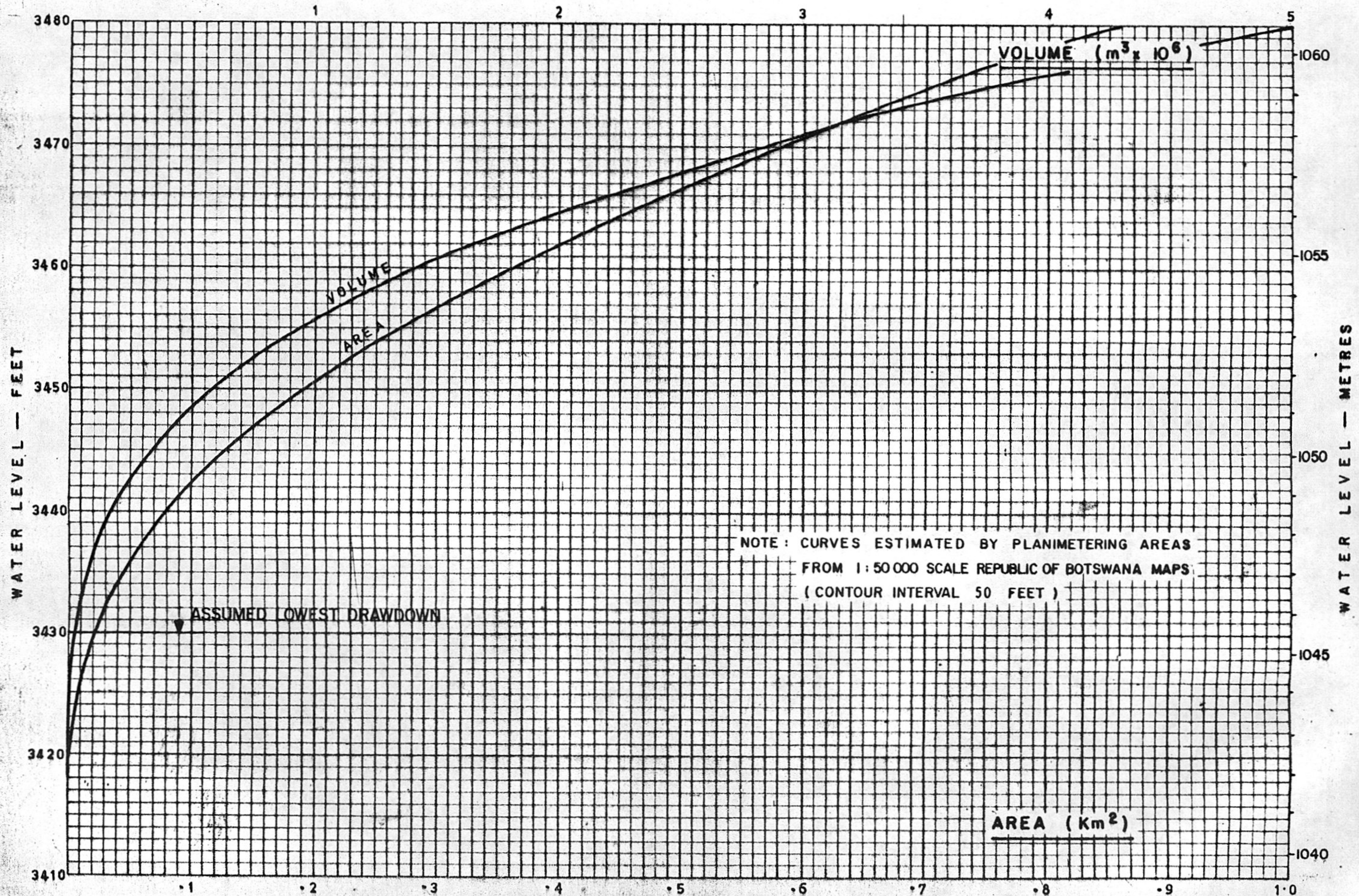
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 55

FIGURE No 2.12



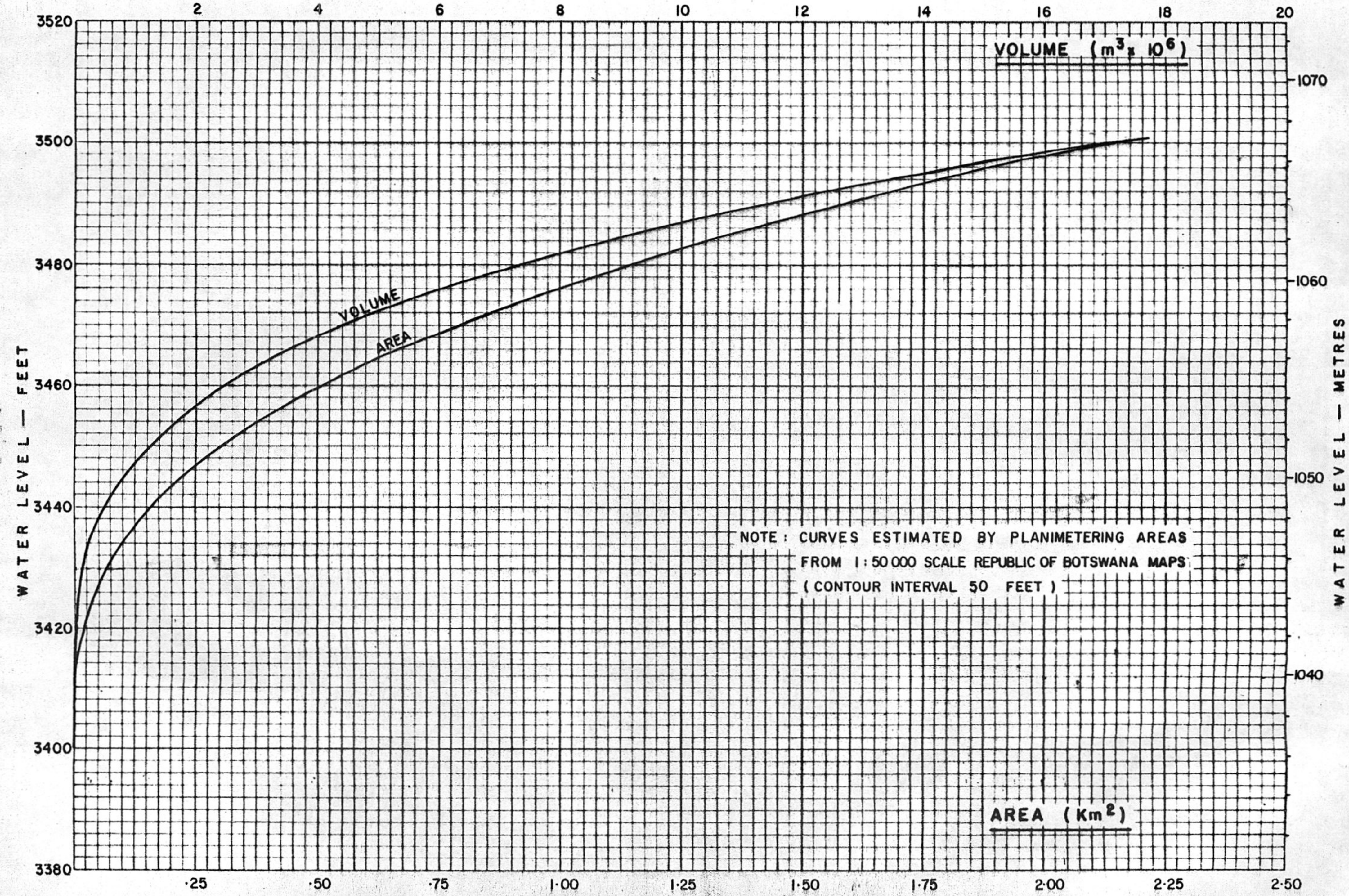
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 56

FIGURE No 2.13



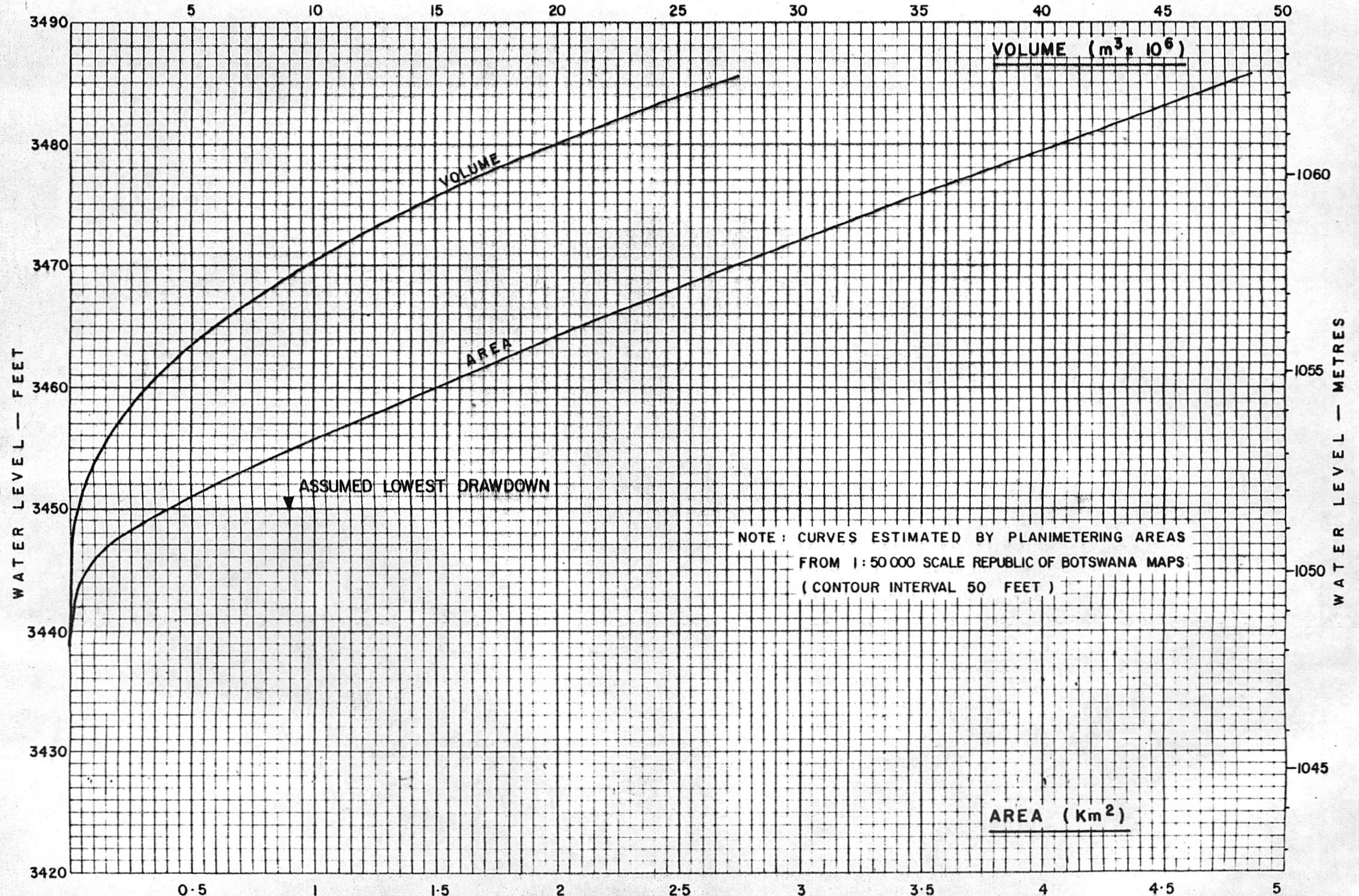
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No. 61 (UPSTREAM ALIGNMENT)

FIGURE No 2.14



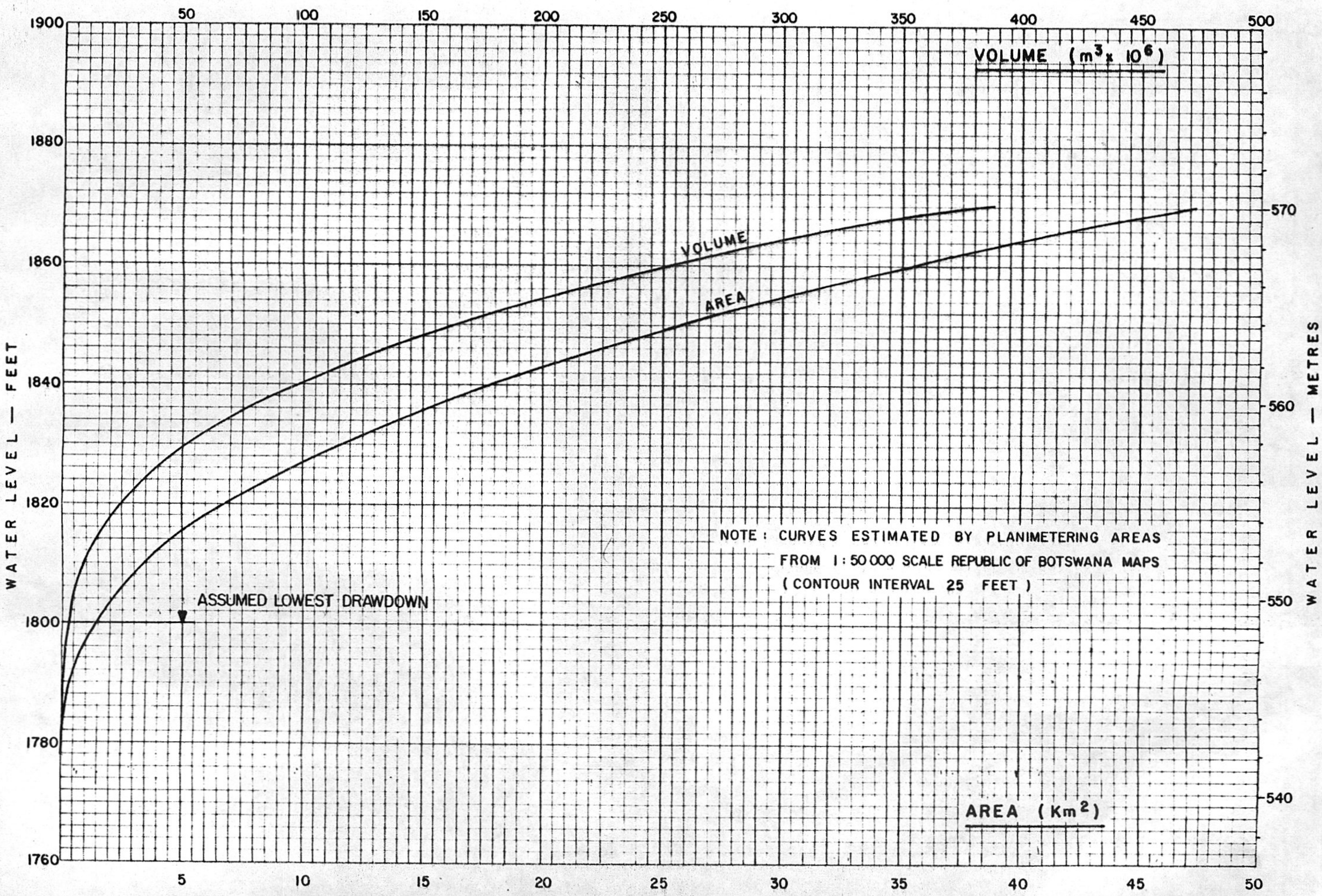
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 61 (DOWNSTREAM ALIGNMENT)

FIGURE No 2.15



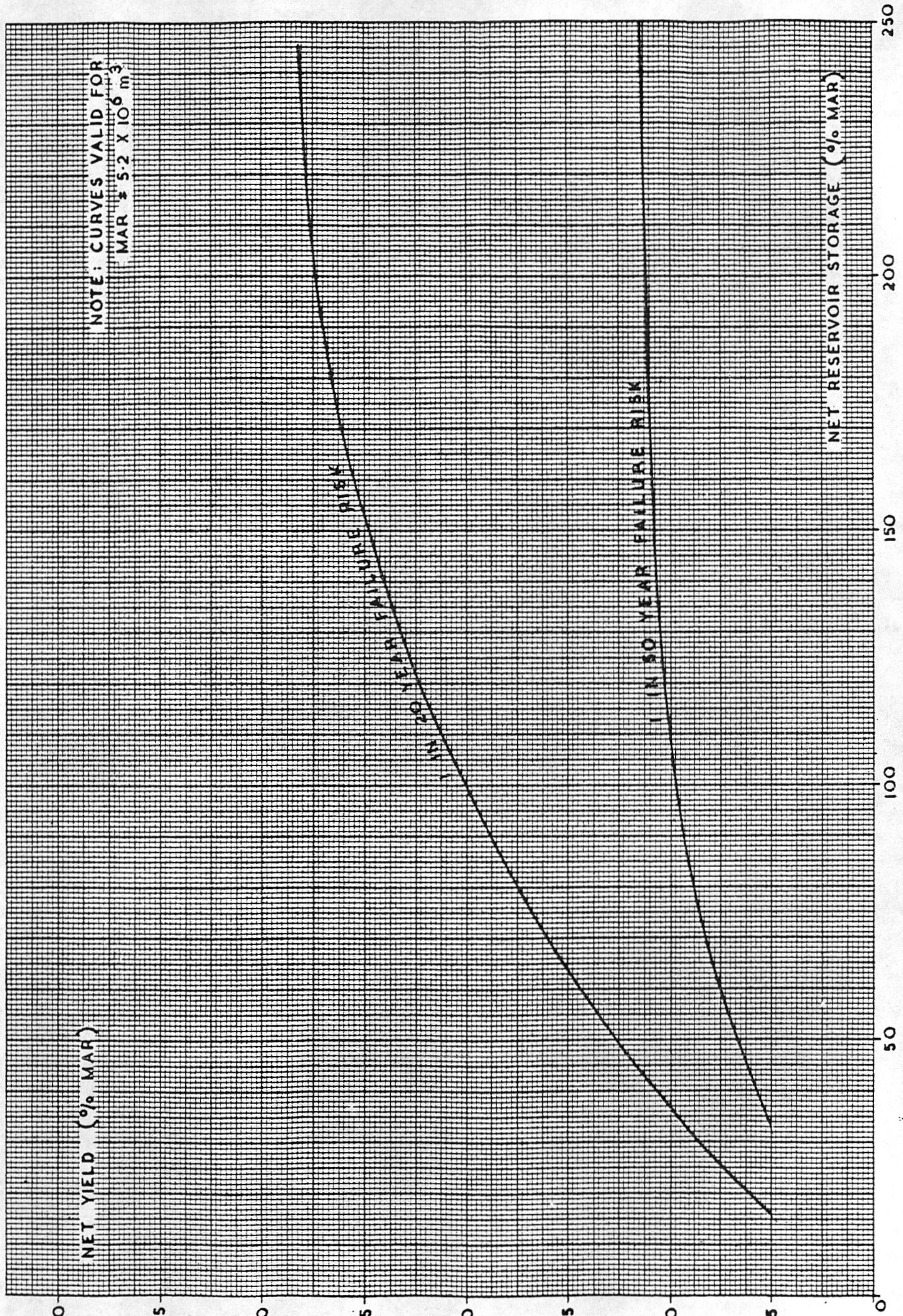
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 62

FIGURE No 2.16



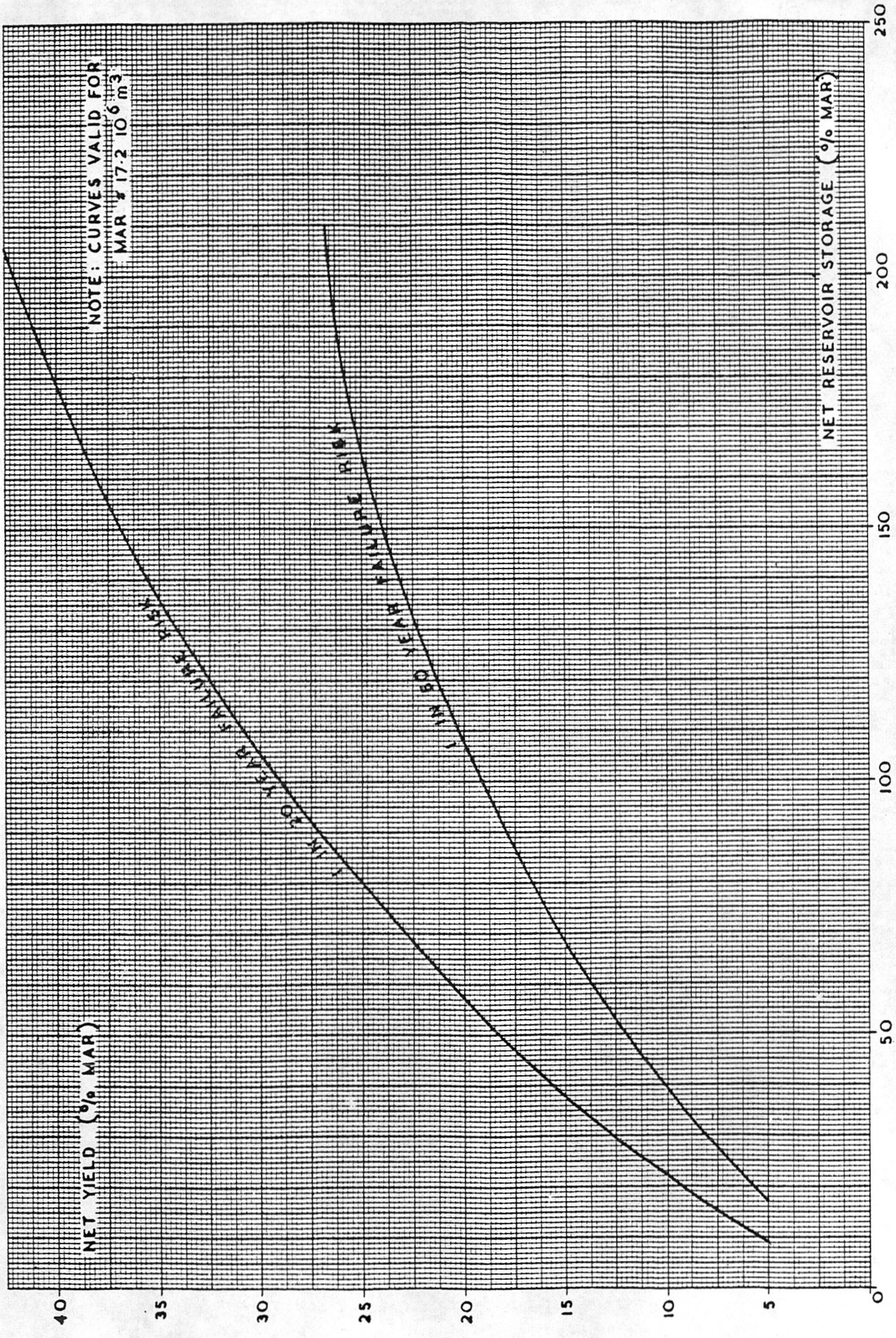
APPROXIMATE RESERVOIR AREA AND VOLUME CURVES FOR DAM SITE No 70

FIGURE No 2.17



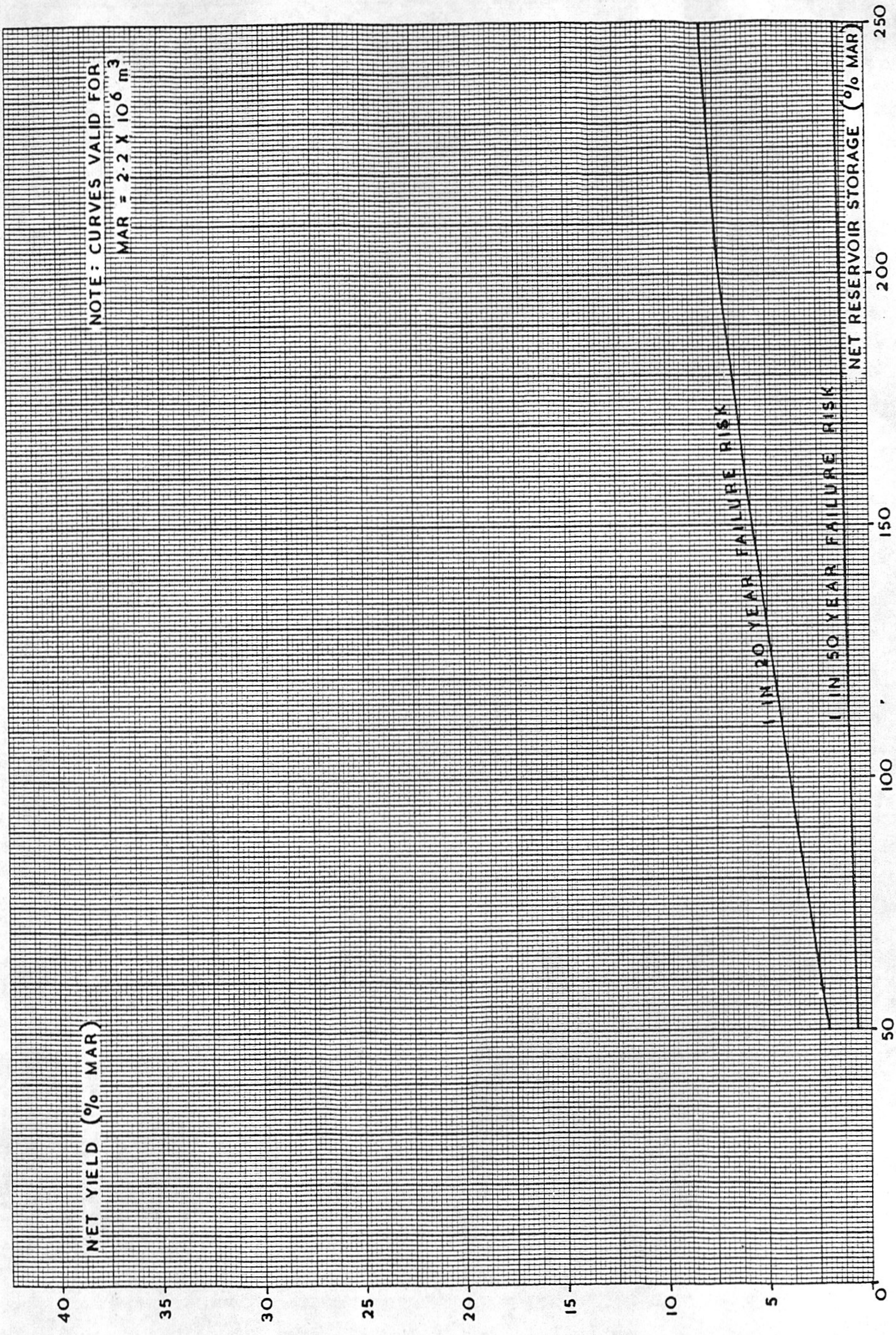
YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 3

FIGURE No. 2.18



YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 10

FIGURE No. 2.19



YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 13

FIGURE No. 2.20

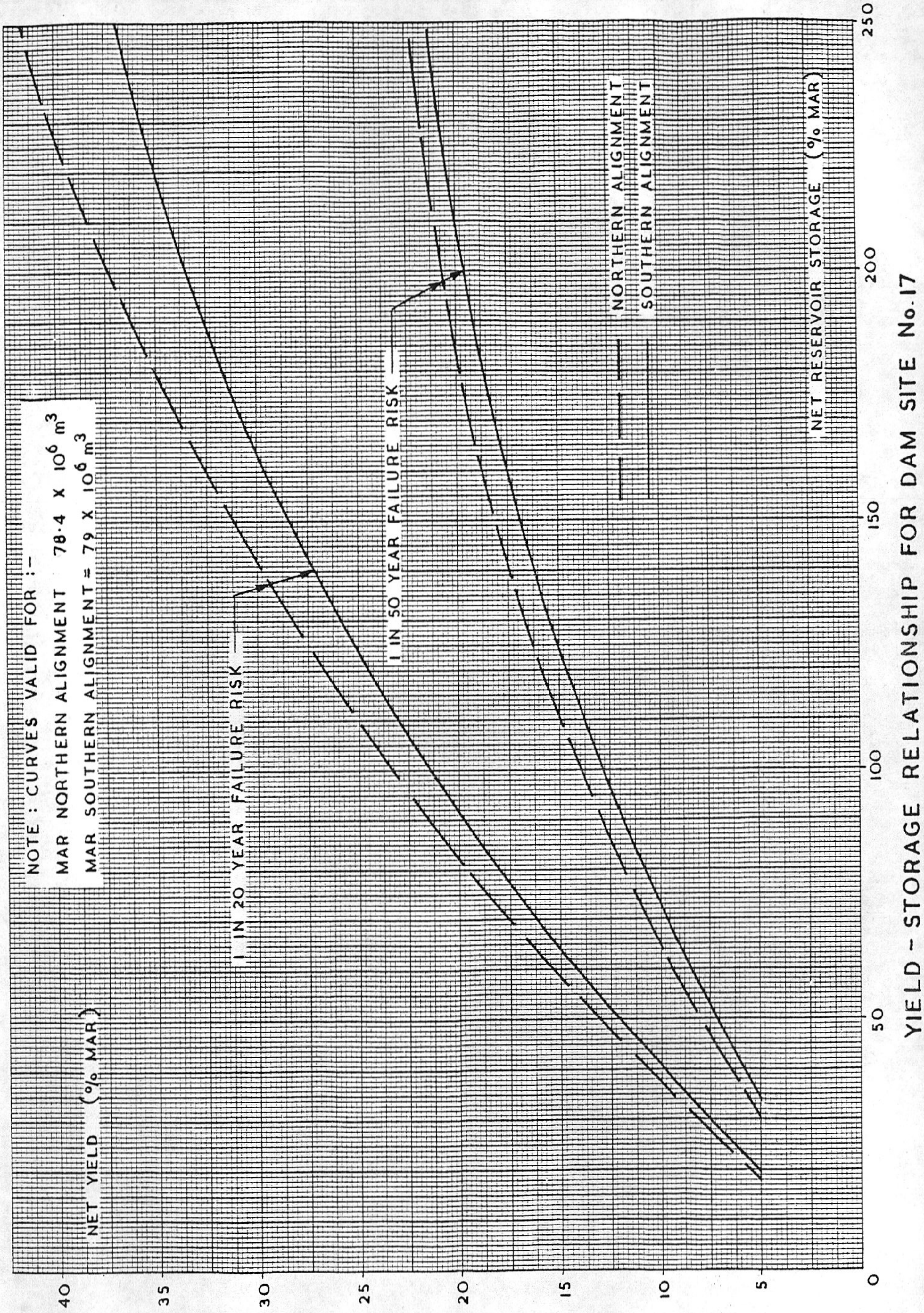
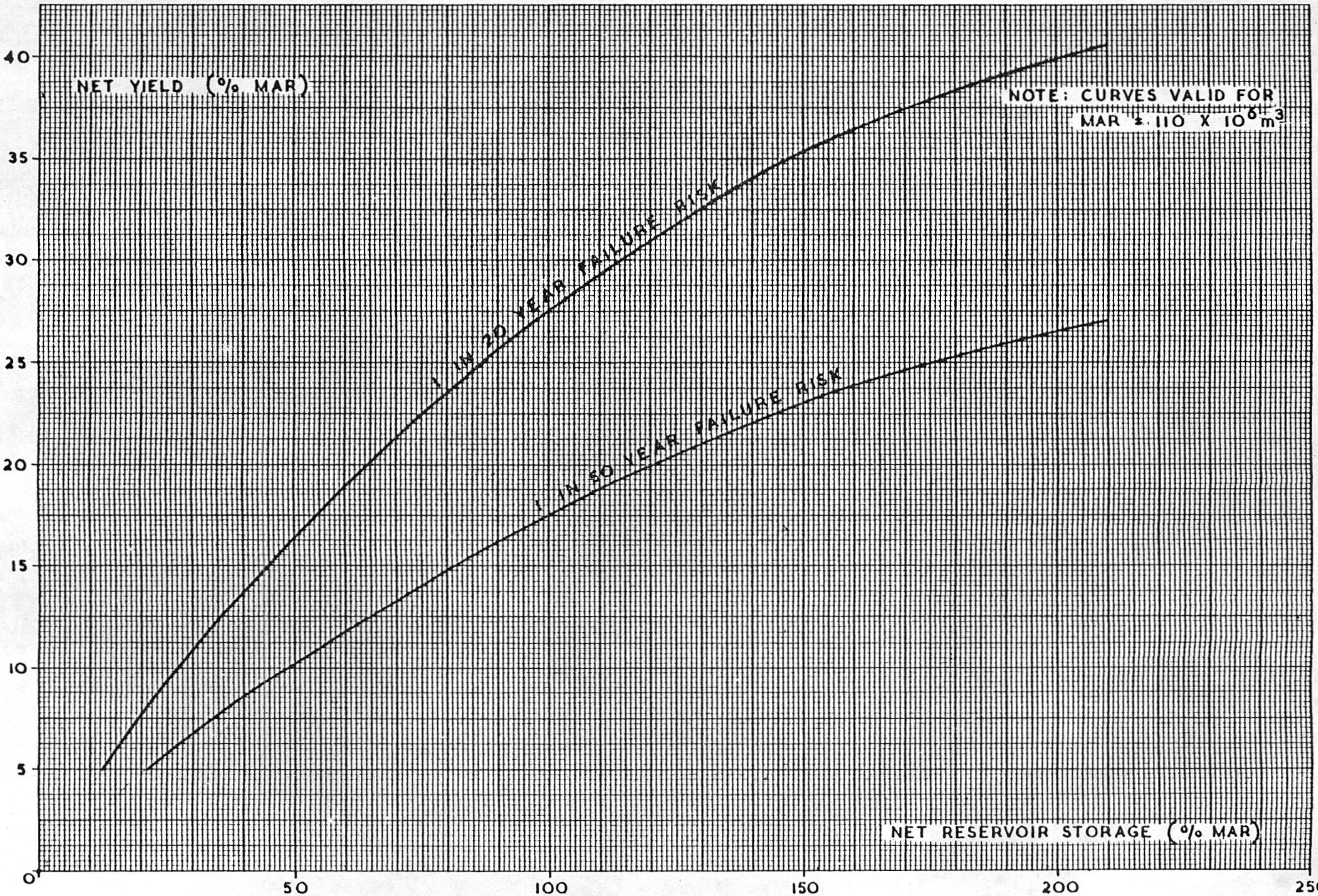
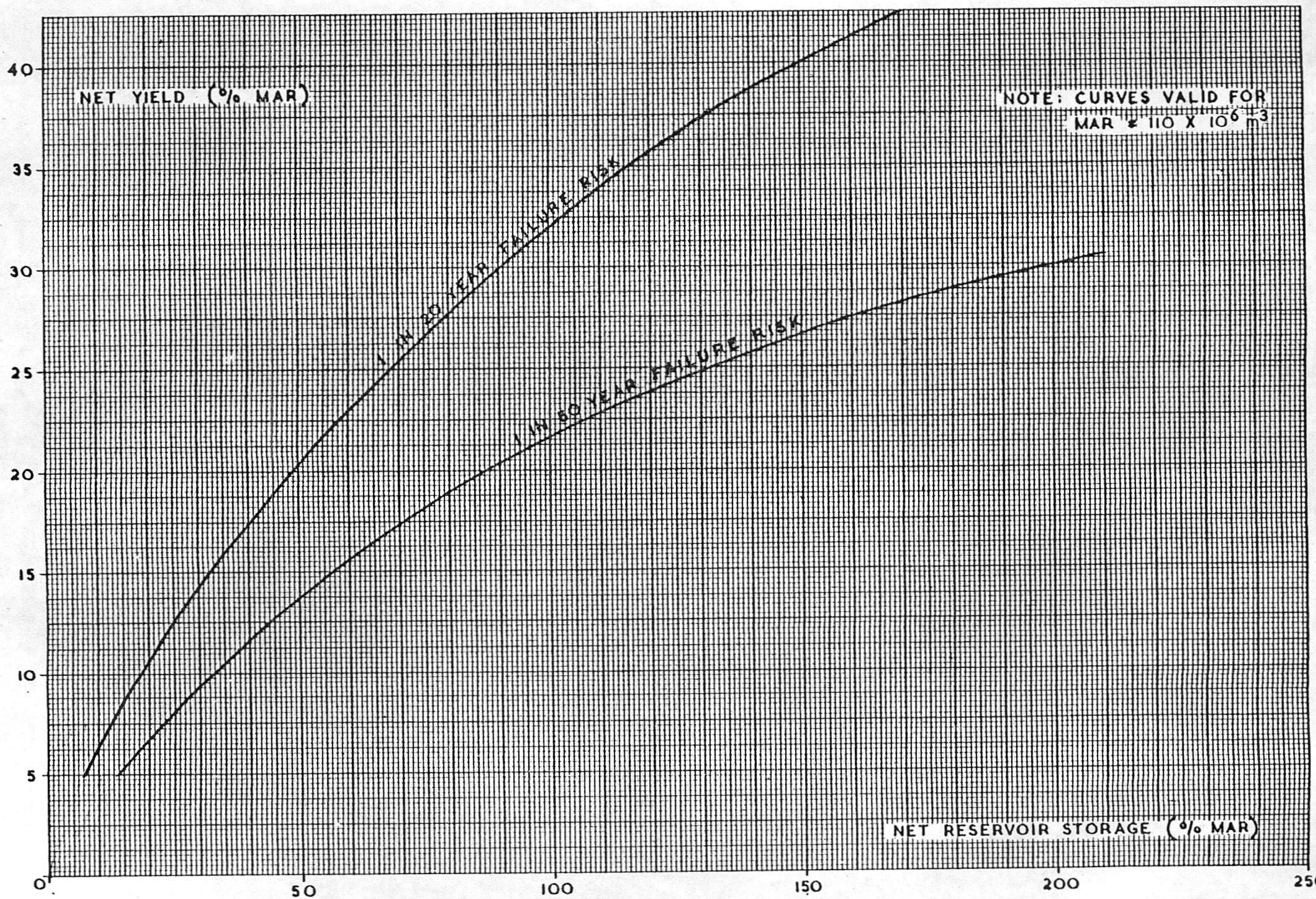


FIGURE No. 2.21



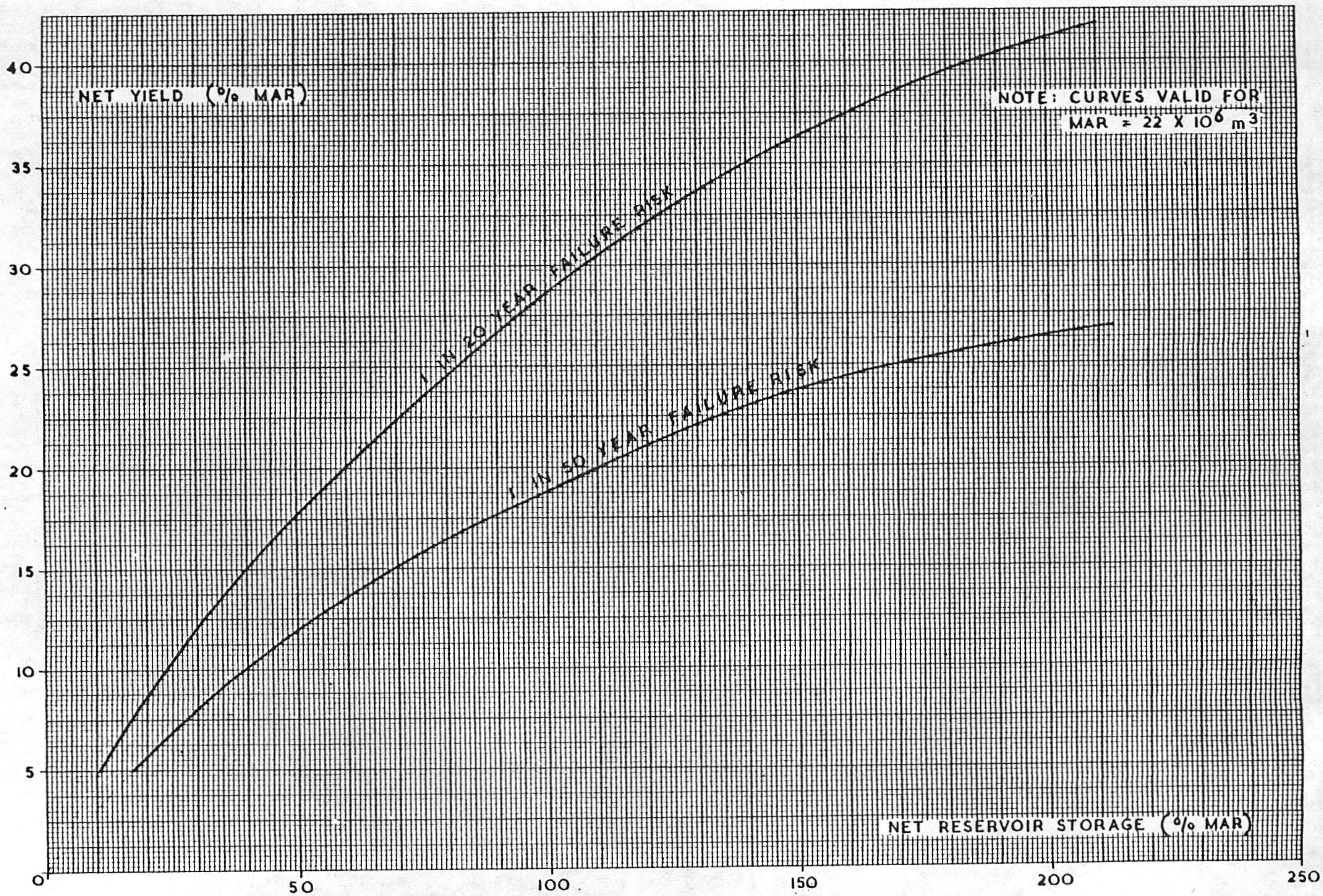
YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 33

FIGURE No. 2.22



YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 40

FIGURE No. 2.23



YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 42A

FIGURE No. 2.24

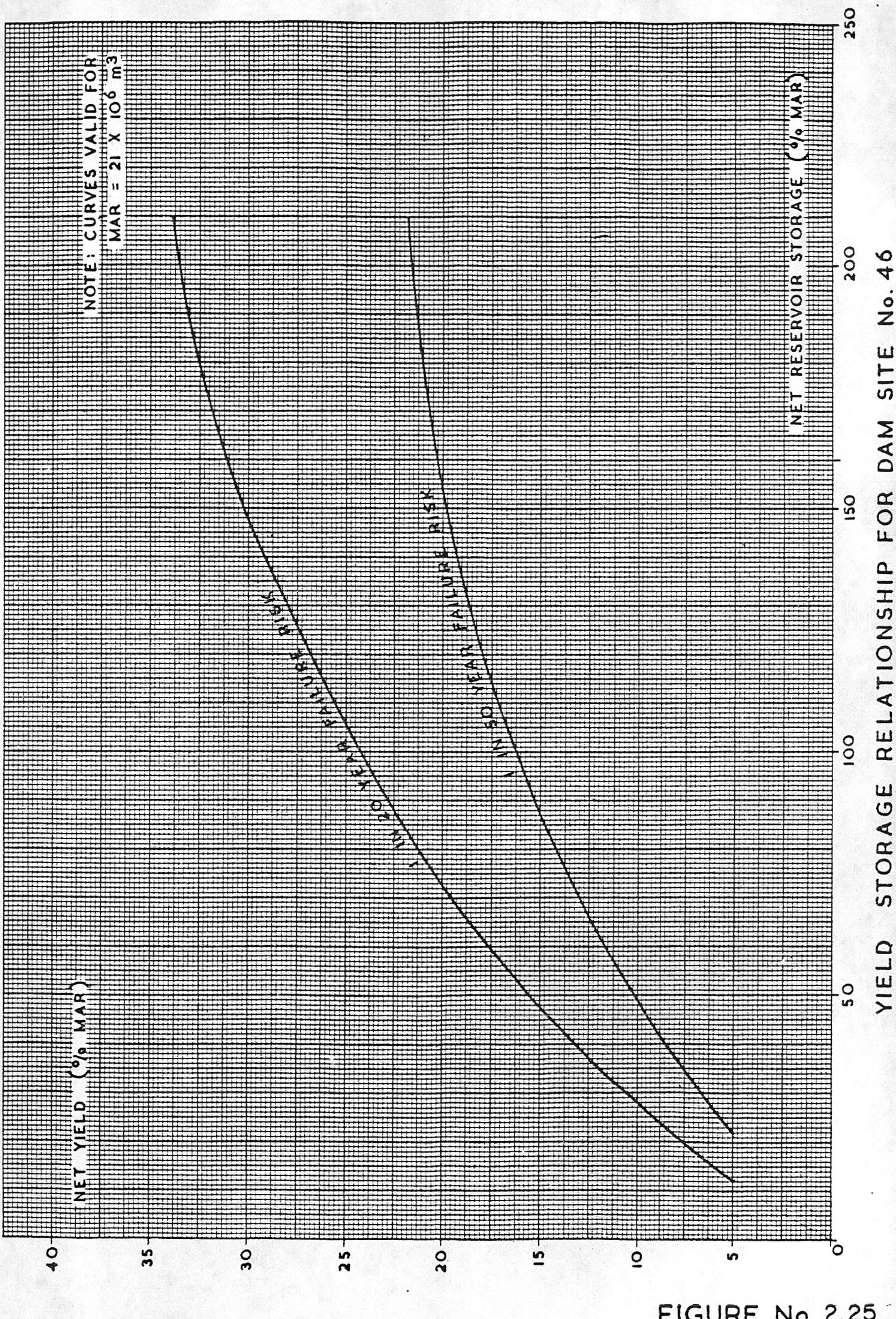
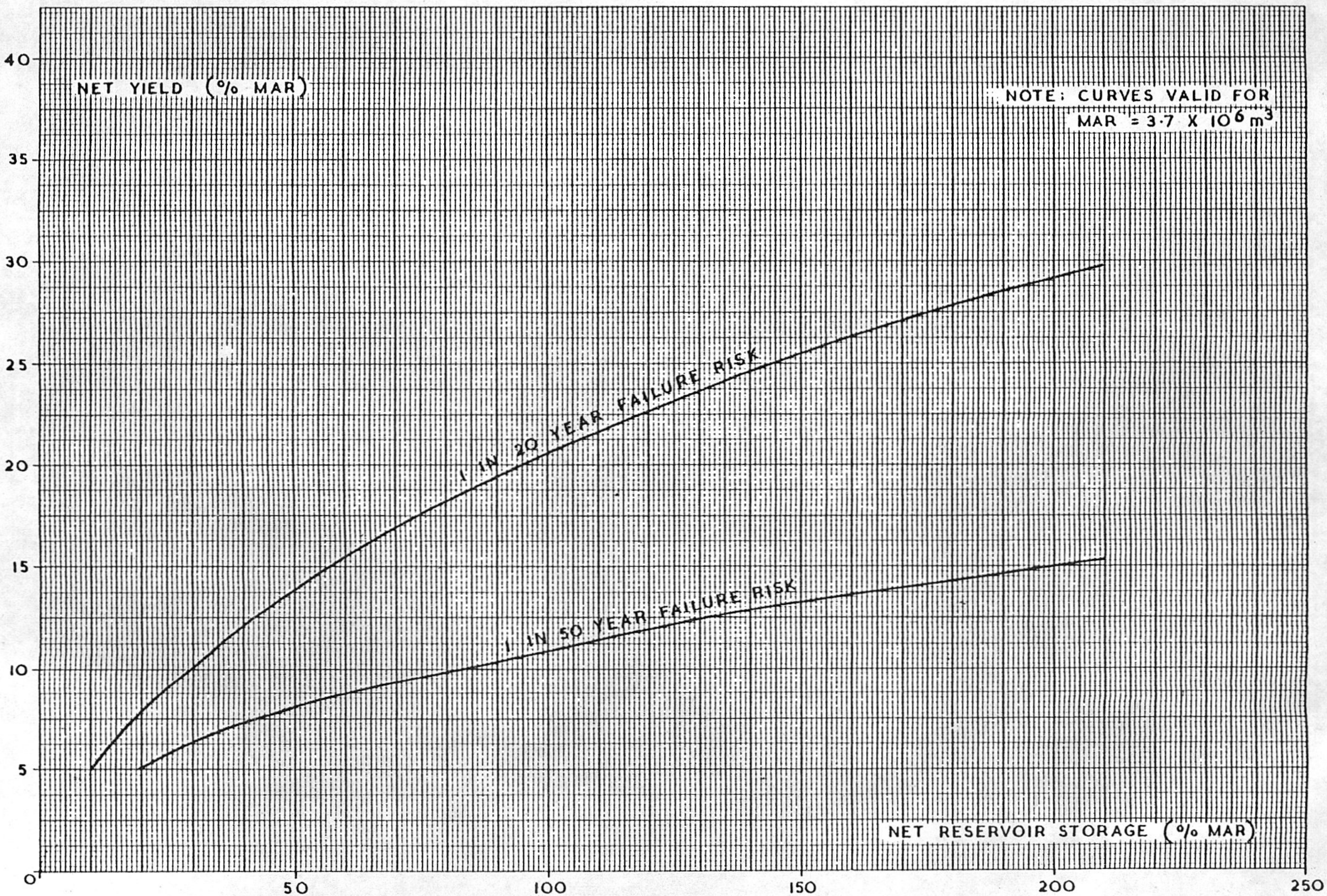
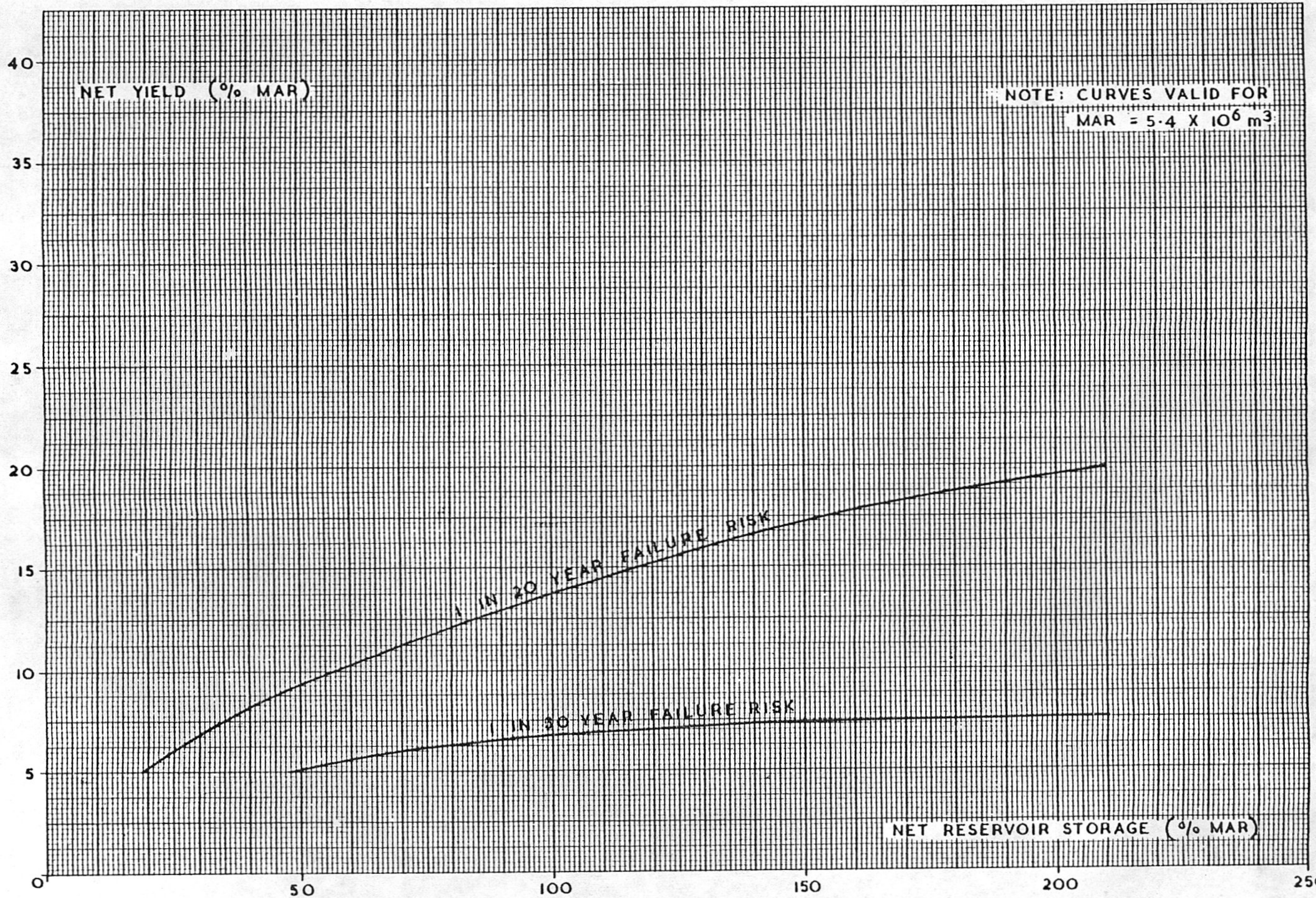


FIGURE No. 2.25



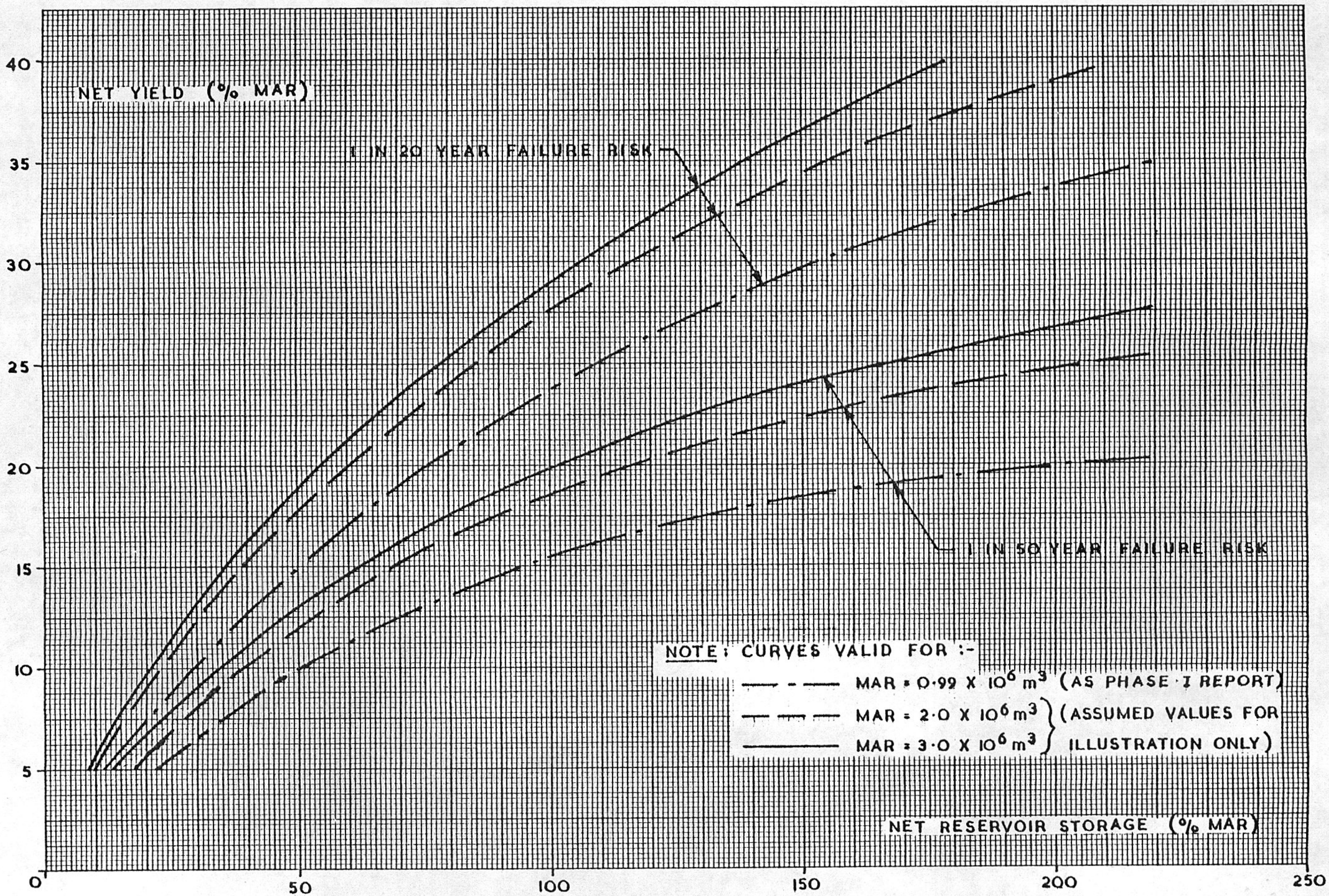
YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 55

FIGURE No. 2.26



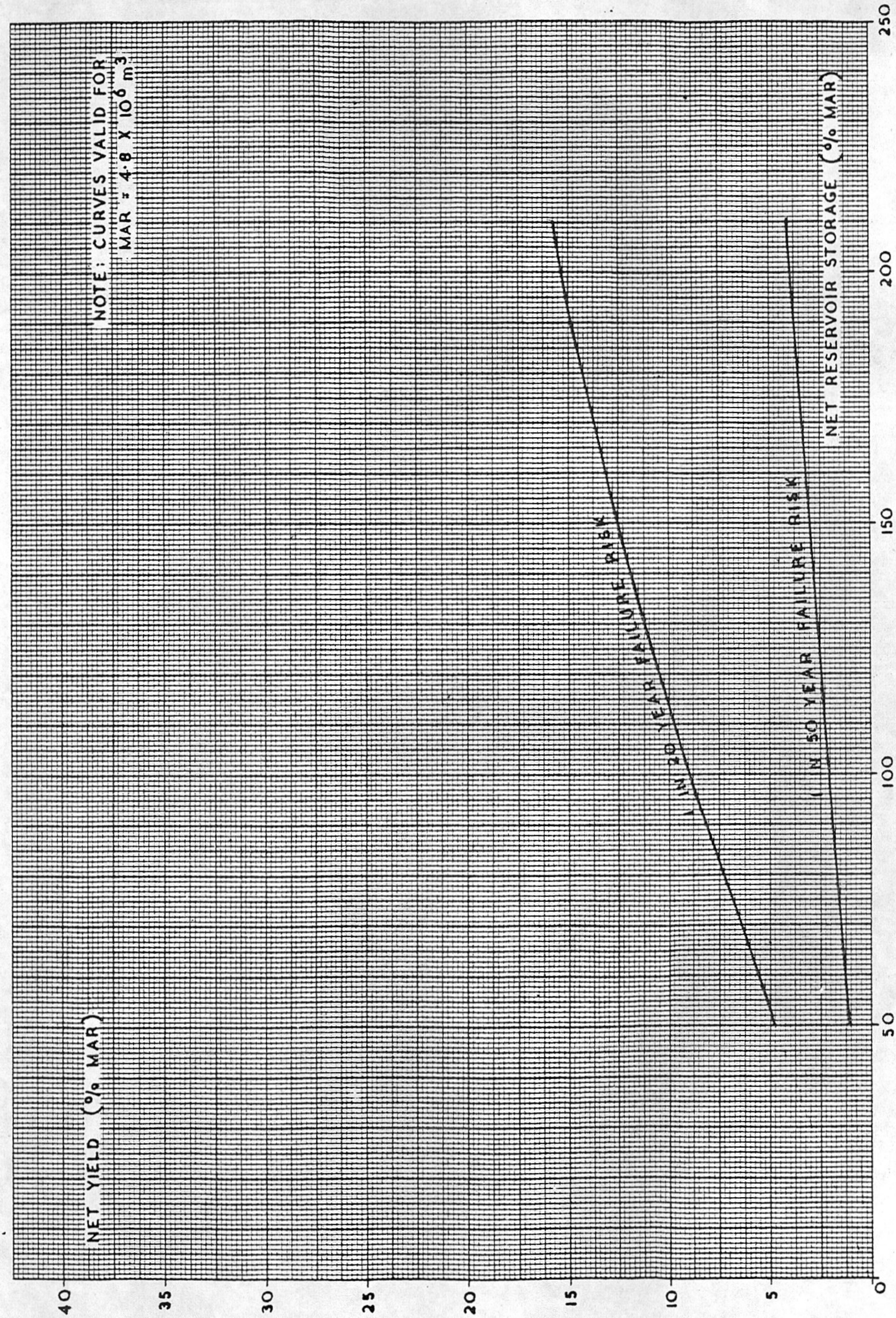
YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 56

FIGURE No. 2 27



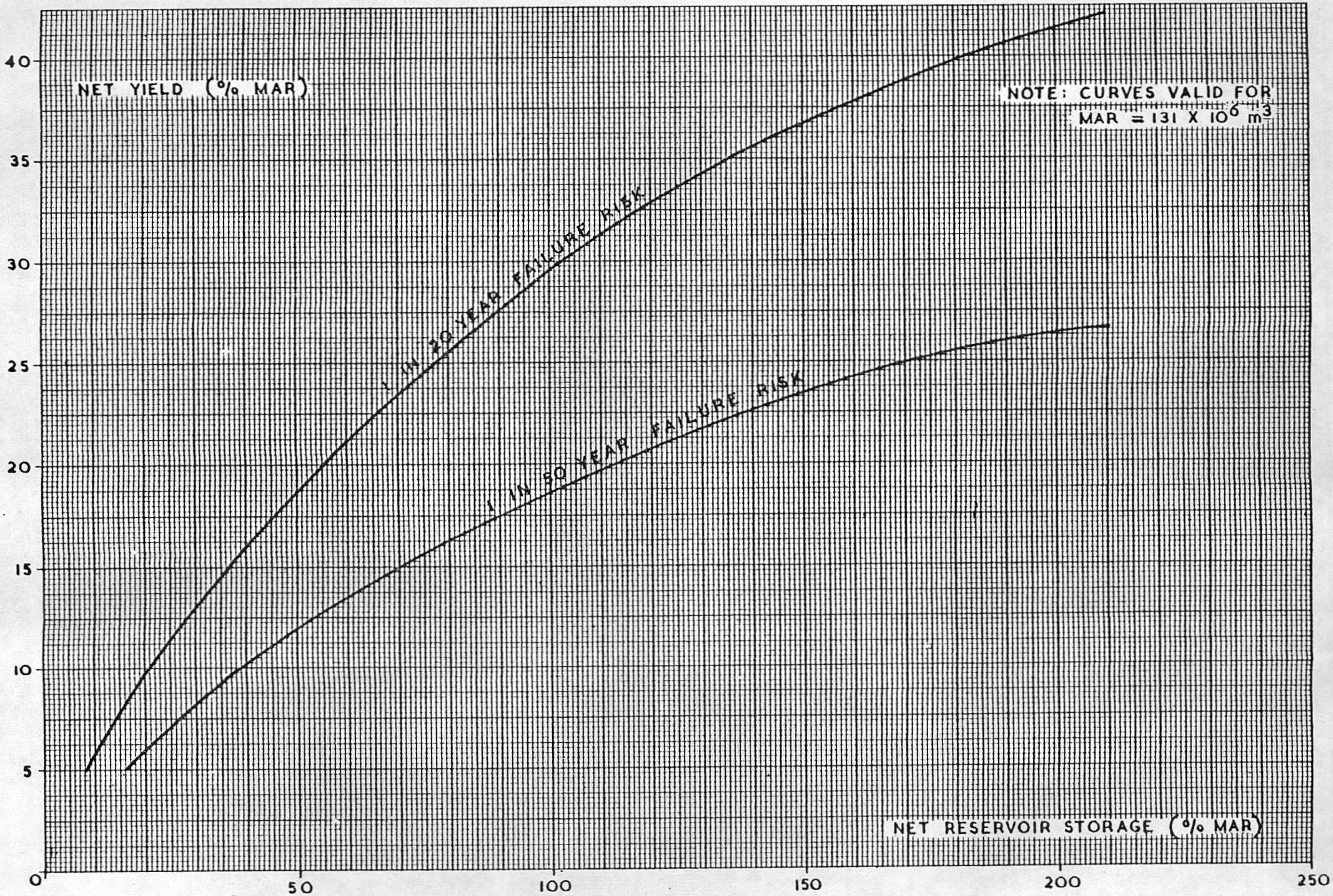
YIELD - STORAGE RELATIONSHIP FOR DAM SITE No. 61

FIGURE No. 2.28



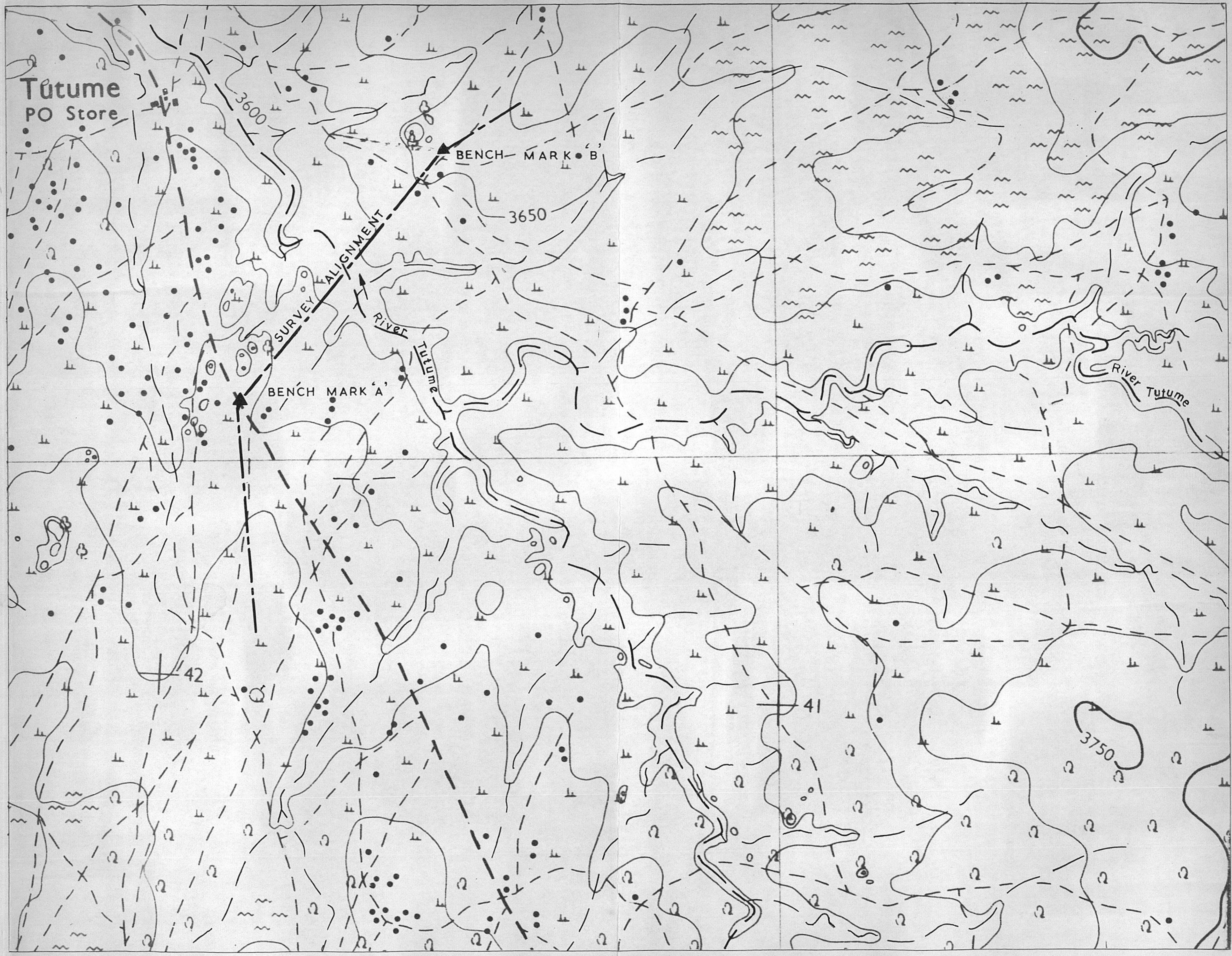
YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 62

FIGURE No. 2.29



YIELD STORAGE RELATIONSHIP FOR DAM SITE No. 70

FIGURE No. 2.30



Enlargement of 1:50,000
scale Map No. 2027 C1

LOCATION OF DAM SITE No. 3

SCALE:- 1:20000

0 1 2 Km

FIGURE No. 3.1

\+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

NOTE -

1 Levels shown are related to field survey datum - standard correction of +3·0m(9·8ft) assumed for correlation with local cartographic data.

2 Site datum based on horizontal levelling to BPP 116 (see Appendix B)

Scales - Horizontal 1:5000
Vertical 1:500

SITE No 3

FIGURE No 3.2

Enlargement of 1:50 000 scale Maps Nos. 2027 C4 and 2027 D3

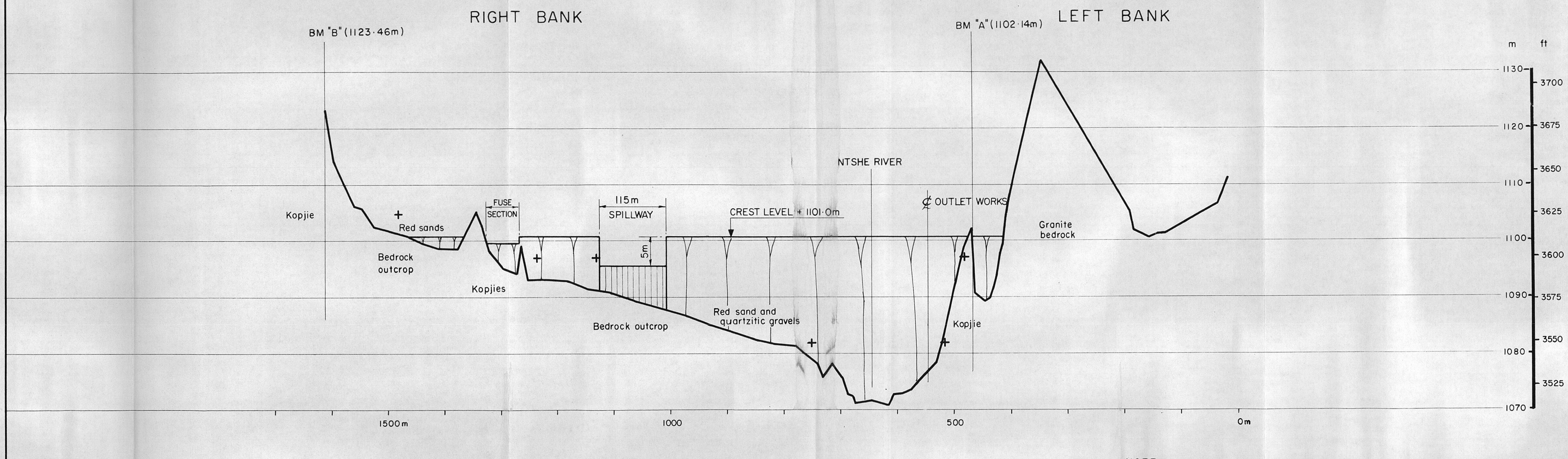
LOCATION OF DAM SITE No. 10

SCALE:- 1:20 000

0 1 2 km.

FIGURE No. 3.3


FUSE SECTION
FARM TRACK
BM "B" (931·85m)
OUTLET WORKS
TATI RIVER
CREST LEVEL +947·0m
BM "A" (948·39m)
Sands and quartzitic gravels
Red sands and quartzitic gravel - outcropping boulders and bedrock
Red sands, some gravel and small boulders
Red sand and quartzitic gravel
Sand levee
Amphibolite bedrock
Amphibolite bedrock shallow deposits of quartzitic gravel and sand.
5m
270m SPILLWAY
3500m
3000
2500
2000
1500
1000
500
0m
m
ft
950
940
930
920
3125
3100
3075
3050
3025
NORTHERN ALIGNMENT
Quartzitic gravels - outcropping boulders and bedrock
BM "C" (938·74m)
CREST LEVEL +946·0m
Red sands and quartzitic gravels
Outcrops of weathered bedrock
Exposed outcrops of amphibolite bedrock
Sands some silty clay
Amphibolite bedrock shallow deposits of quartzitic gravel and sand.
3000m
SOUTHERN ALIGNMENT
+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING
NOTE -
1 Site datum based on barometric levelling to BPT 67 (see Appendix B)
Scales - Horizontal 1:5000
Vertical 1:500
SITE No 17
FIGURE No 3.8



+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

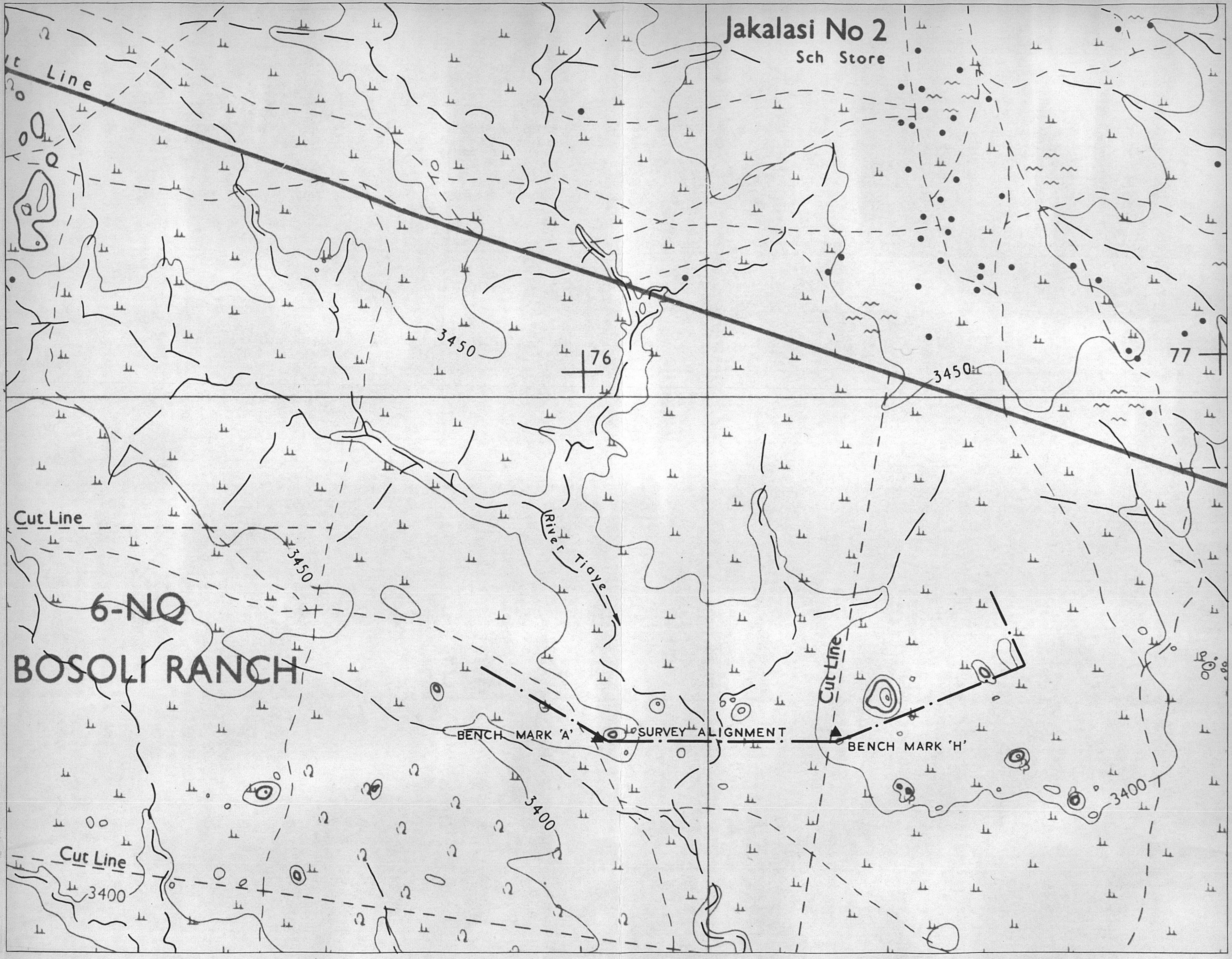
NOTE -

1. Site datum based on trigonometric levelling to BPT 43 (see Appendix B)

Scales — Horizontal 1:5000
Vertical 1:500

SITE No 10

FIGURE No 3.4



Enlargement of 1:50 000
scale Map No. 2027 D3

LOCATION OF DAM SITE No. 13

SCALE: 1:20 000

0 1 2 Km

FIGURE No. 3.5

+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

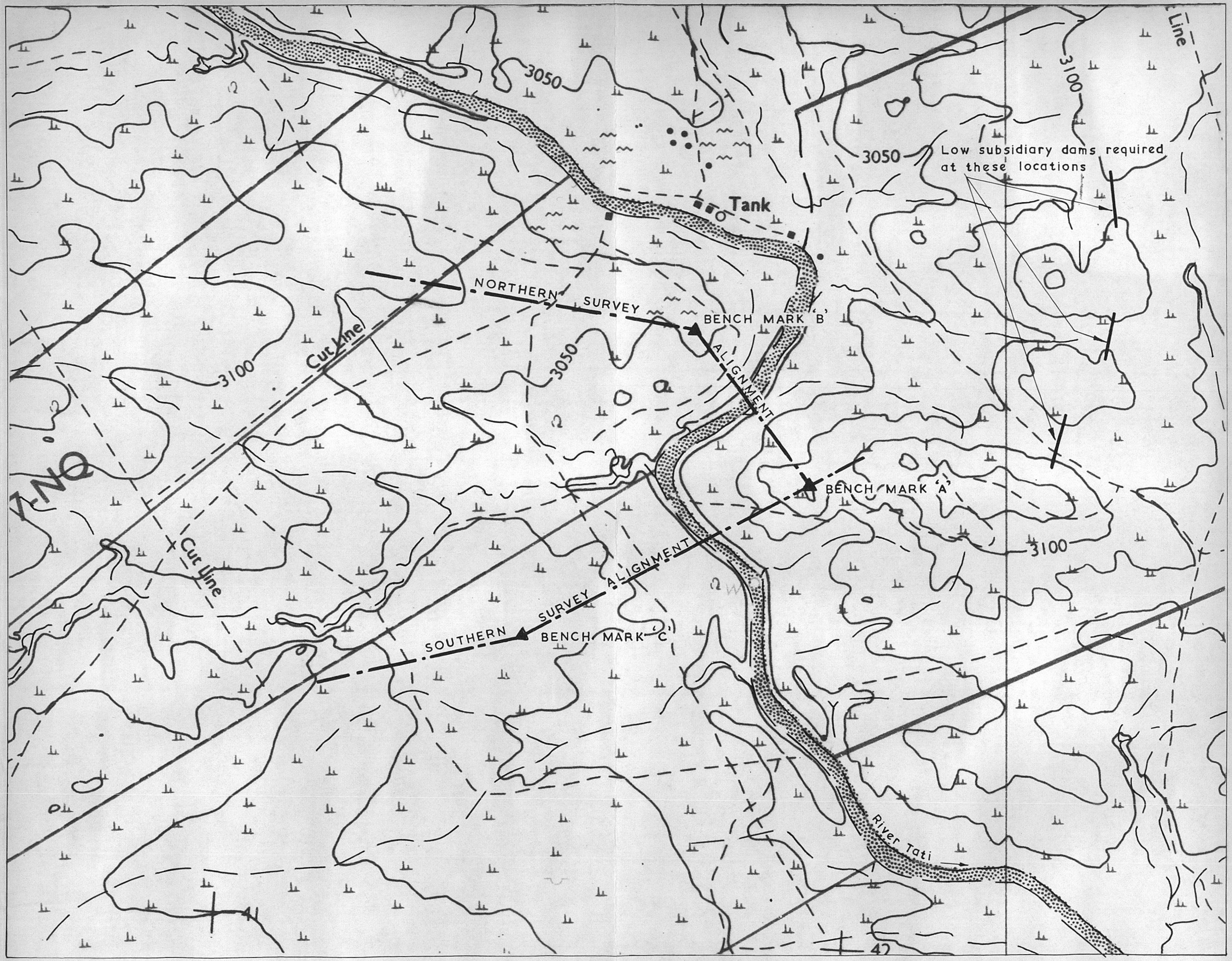
NOTE –

1. Site datum based on trigonometric and horizontal levelling to BPS 94 (see Appendix B)

Scales – Horizontal 1:5000
Vertical 1:500

SITE No 13

FIGURE No 3.6



Enlargement of 1:50 000 scale Map No. 2127 B3

LOCATION OF DAM SITE No. 17

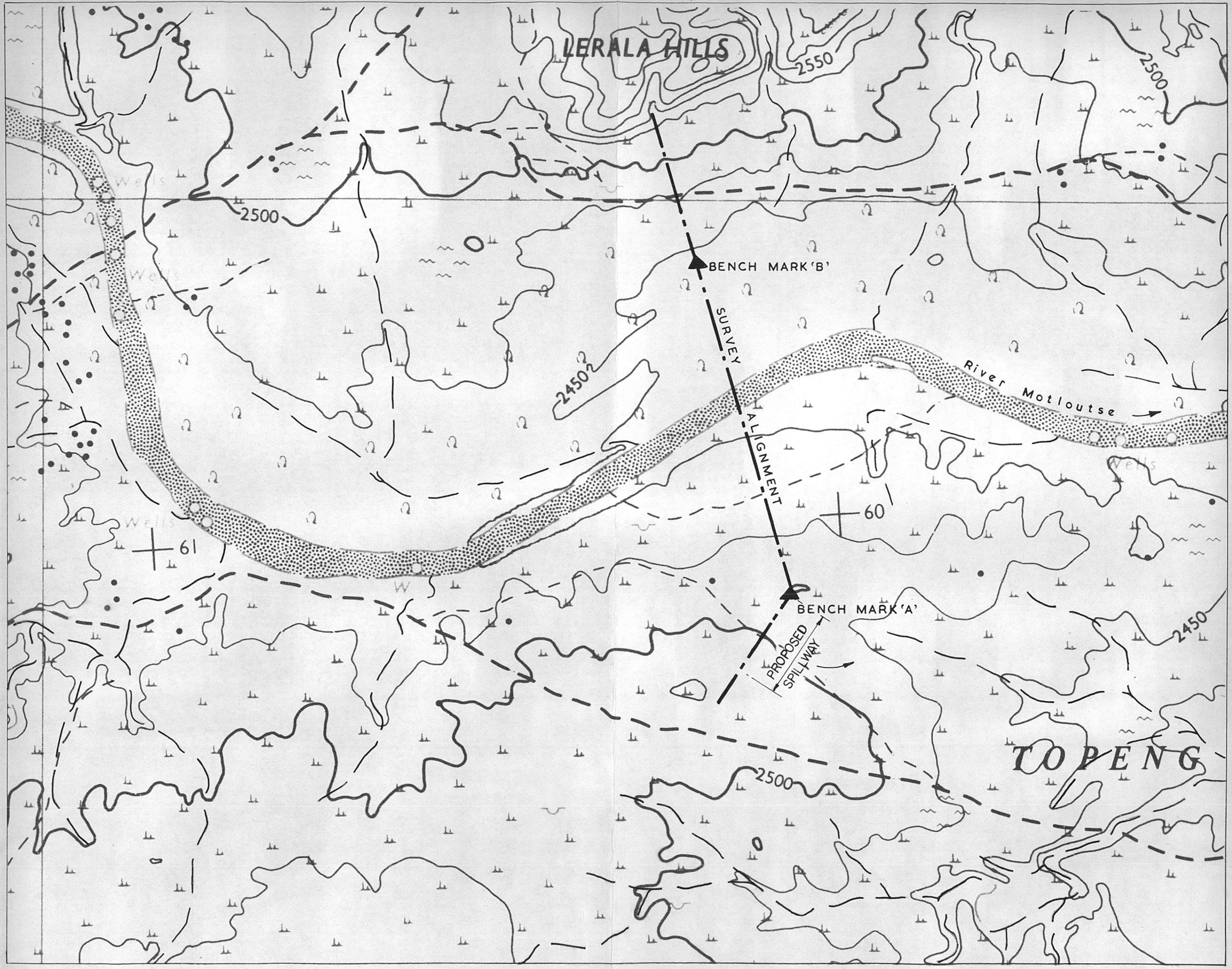
SCALE:- 1:20000

0 1 2 Km

FIGURE No 3.7

SITE No 17

FIGURE No 3.8



Enlargement of 1:50 000 scale Map No 2128 C3

LOCATION OF DAM SITE No. 33

SCALE :- 1:20 000

0 1 2 Km

FIGURE No. 3.9

+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

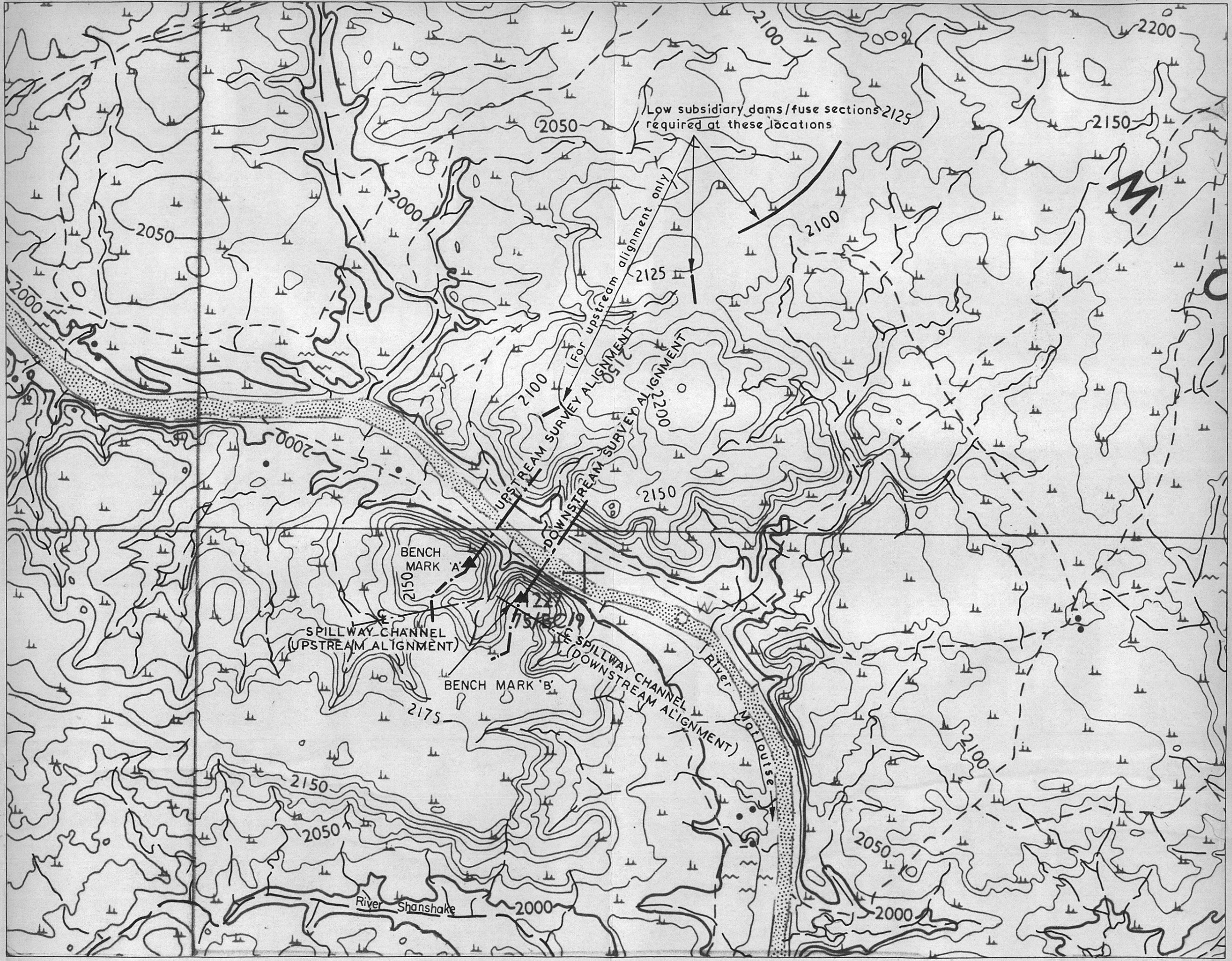
NOTE –

1. Levels shown are related to field survey datum – standard correction of +2·0m (6·5ft) assumed for correlation with local cartographic data.
2. Site datum based on barometric levelling to BPT 177 (see Appendix B)

Scales – Horizontal 1:5000
Vertical 1:500

SITE No 33

FIGURE No 3.10



Enlargement of 1:50000
scale Maps Nos. 2228 B1
and 2228 B2

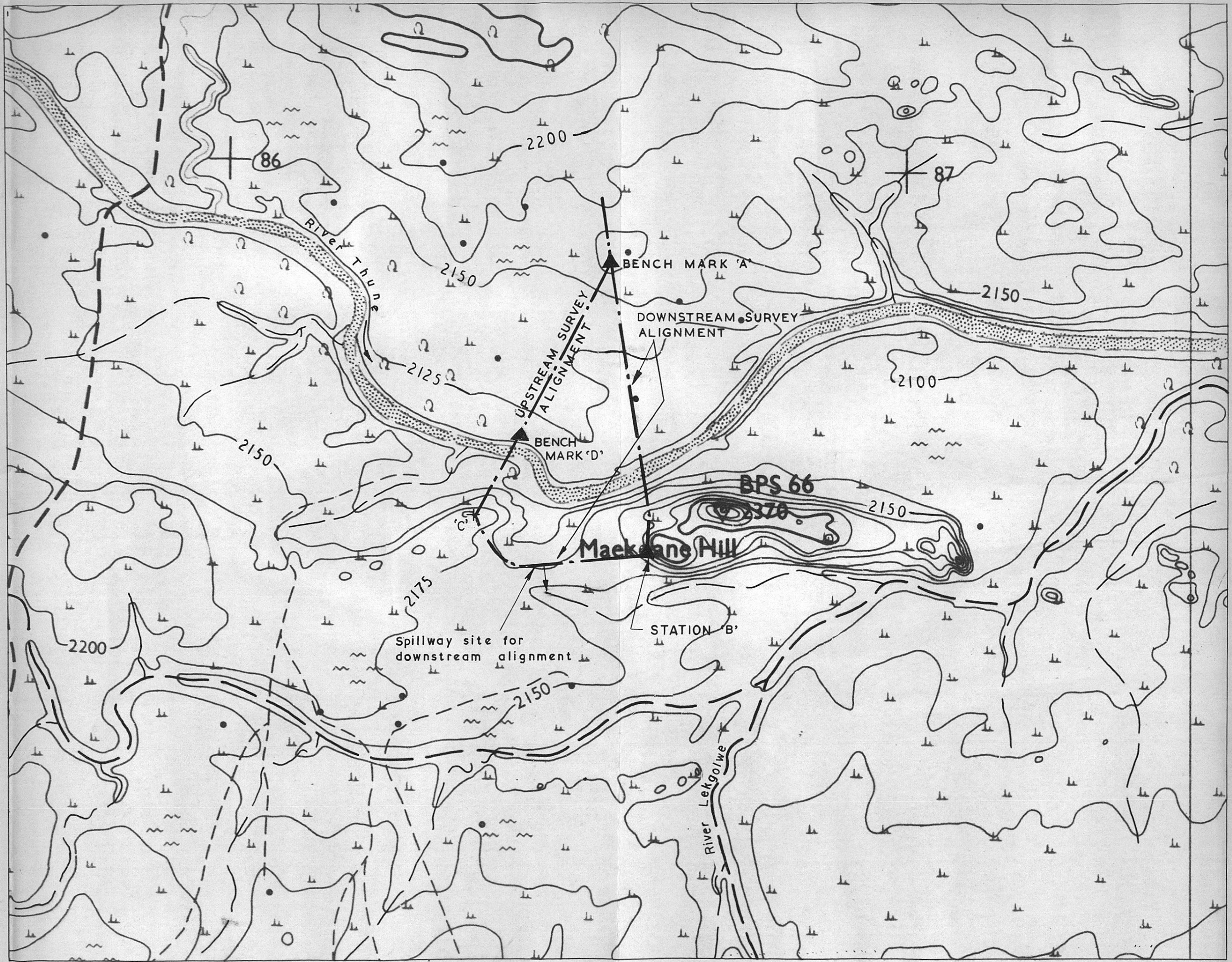
LOCATION OF DAM SITE No. 40

SCALE: 1:20000

FIGURE No. 3.11

SITE No 40

FIGURE No 3.12



Enlargement of 1:50 000 scale Map No 2228 B1

LOCATION OF DAM SITE No. 42 A

SCALE: 1:20000

0 2 Km

FIGURE No. 3.13

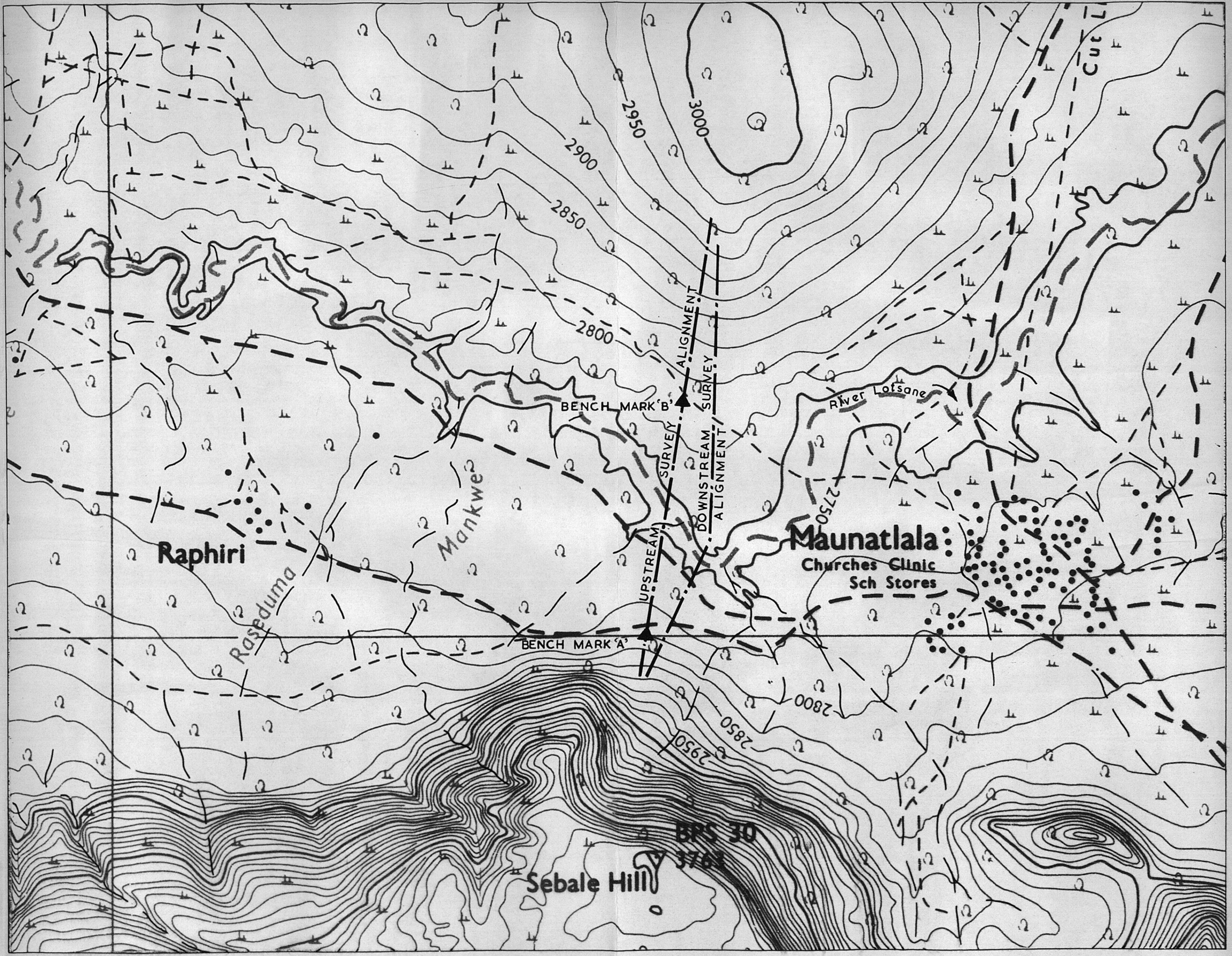
+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

NOTE –
1. Site datum based on trigonometric levelling to BPS 66 (see Appendix B)

Scales – Horizontal 1:5000
Vertical 1:500

SITE No 42A

FIGURE No 3.14

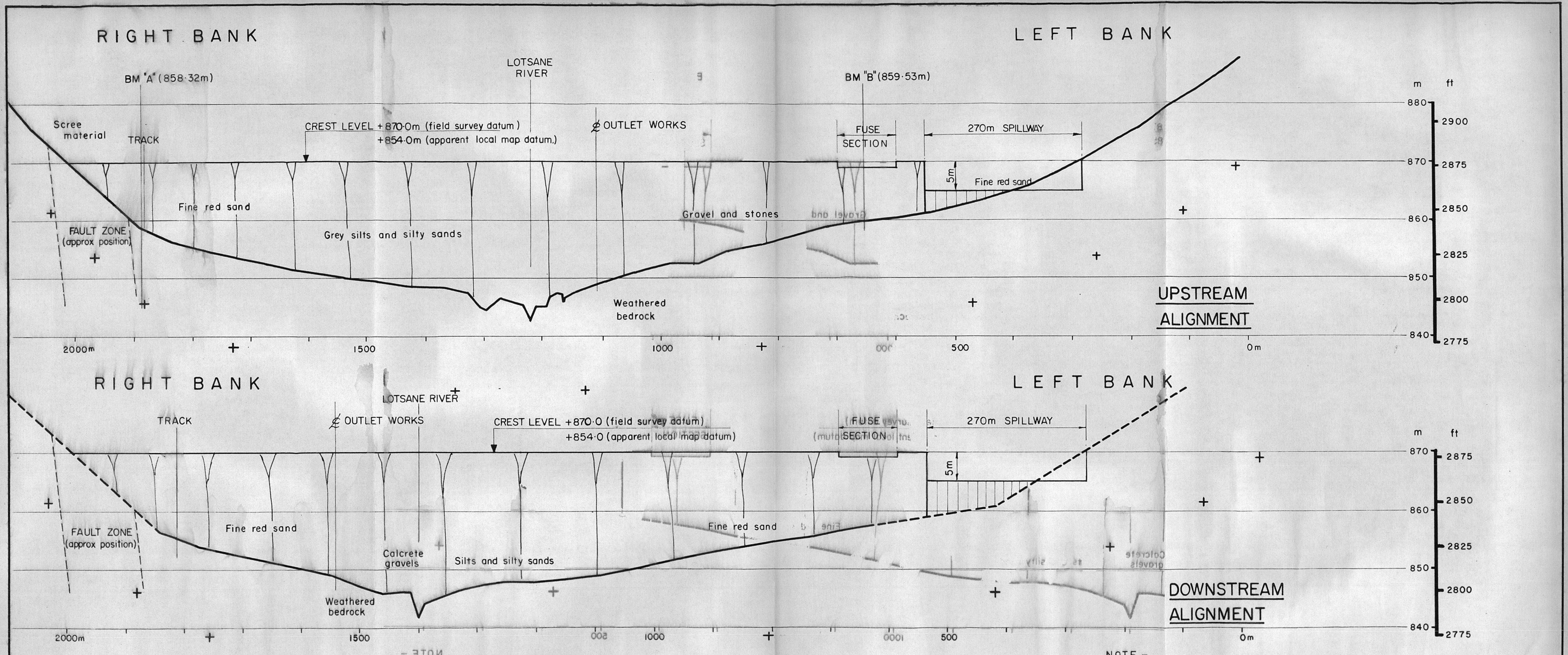


Enlargement of 1:50 000 scale Map No 2227 DI

LOCATION OF DAM SITE No. 46

SCALE:- 1:20 000

FIGURE No. 3.15



+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

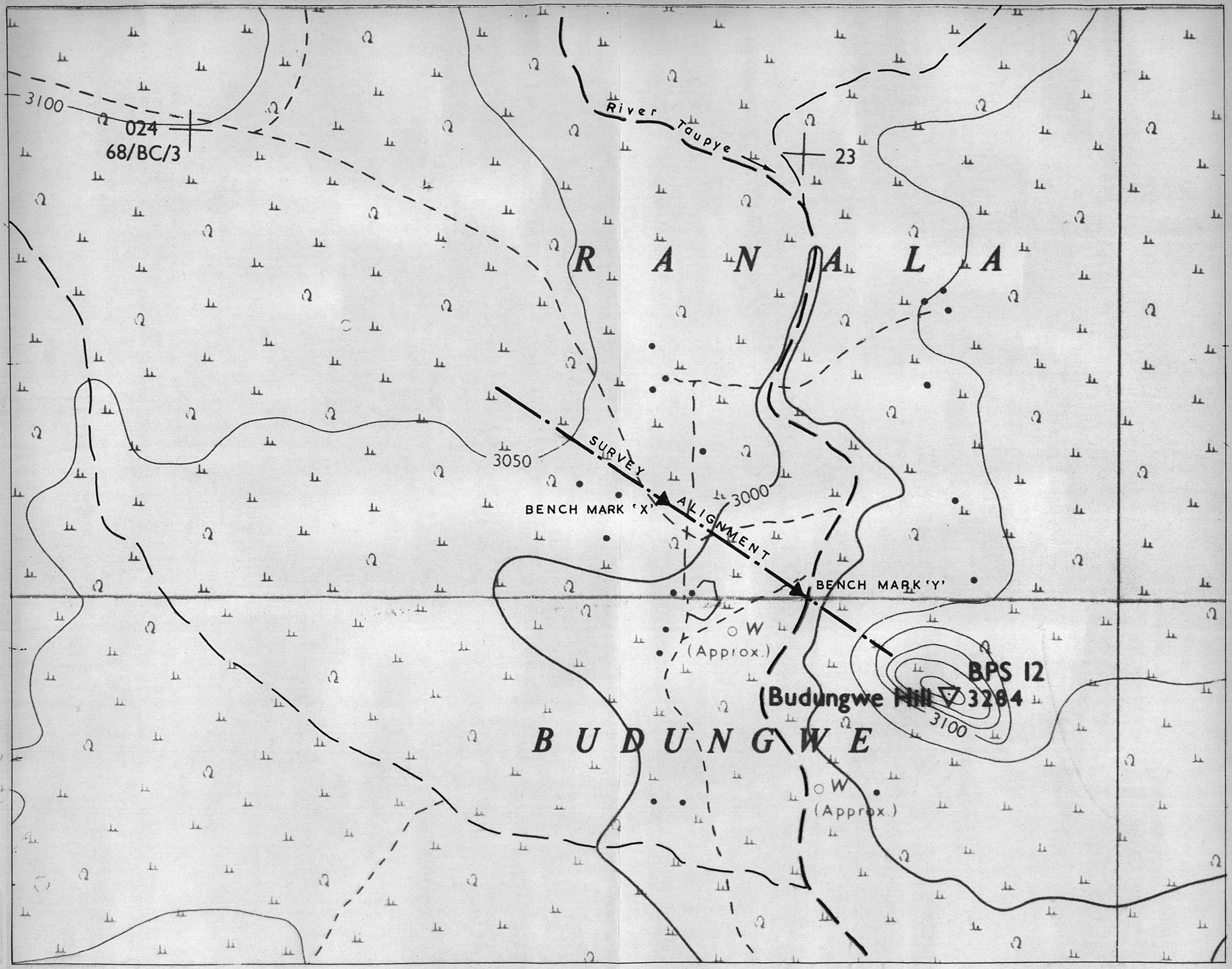
NOTE –

1. Levels shown are related to field survey datum – standard correction of –16·0m (–52·5 ft.) assumed for correlation with local cartographic data
2. Site datum based on barometric levelling to BPT 337 (see Appendix B)

Scales: Horizontal 1:5000
Vertical 1:500

SITE No 46

FIGURE No 3·16



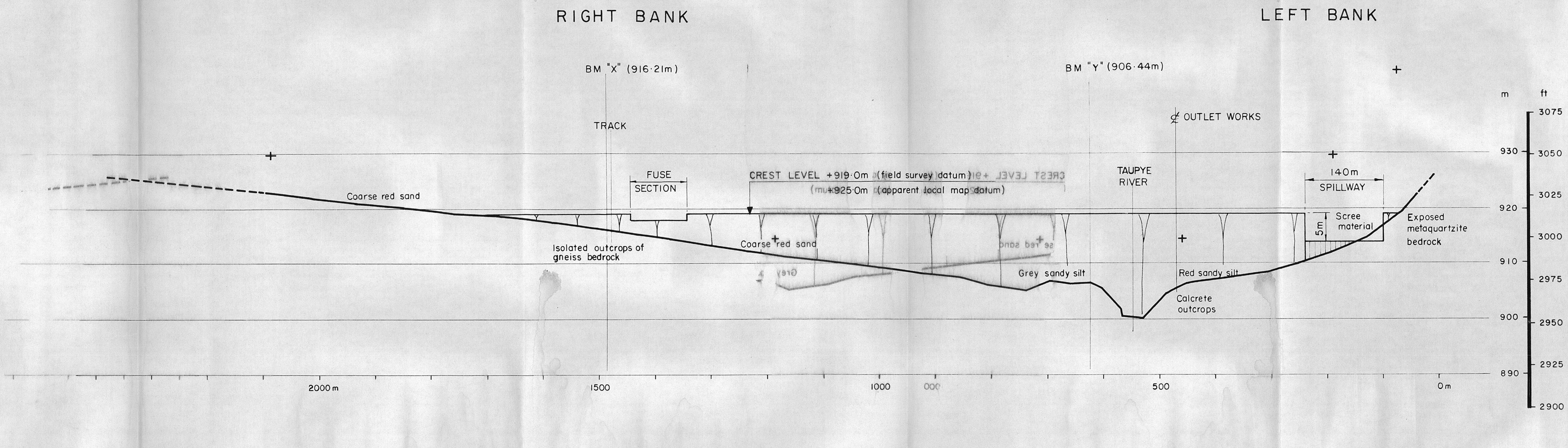
Enlargement of 1:50 000
scale Maps Nos 2327 A1
and 2327 A3

LOCATION OF DAM SITE No. 55

SCALE:- 1:20 000

0 1 2 Km

FIGURE No. 3.17



+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

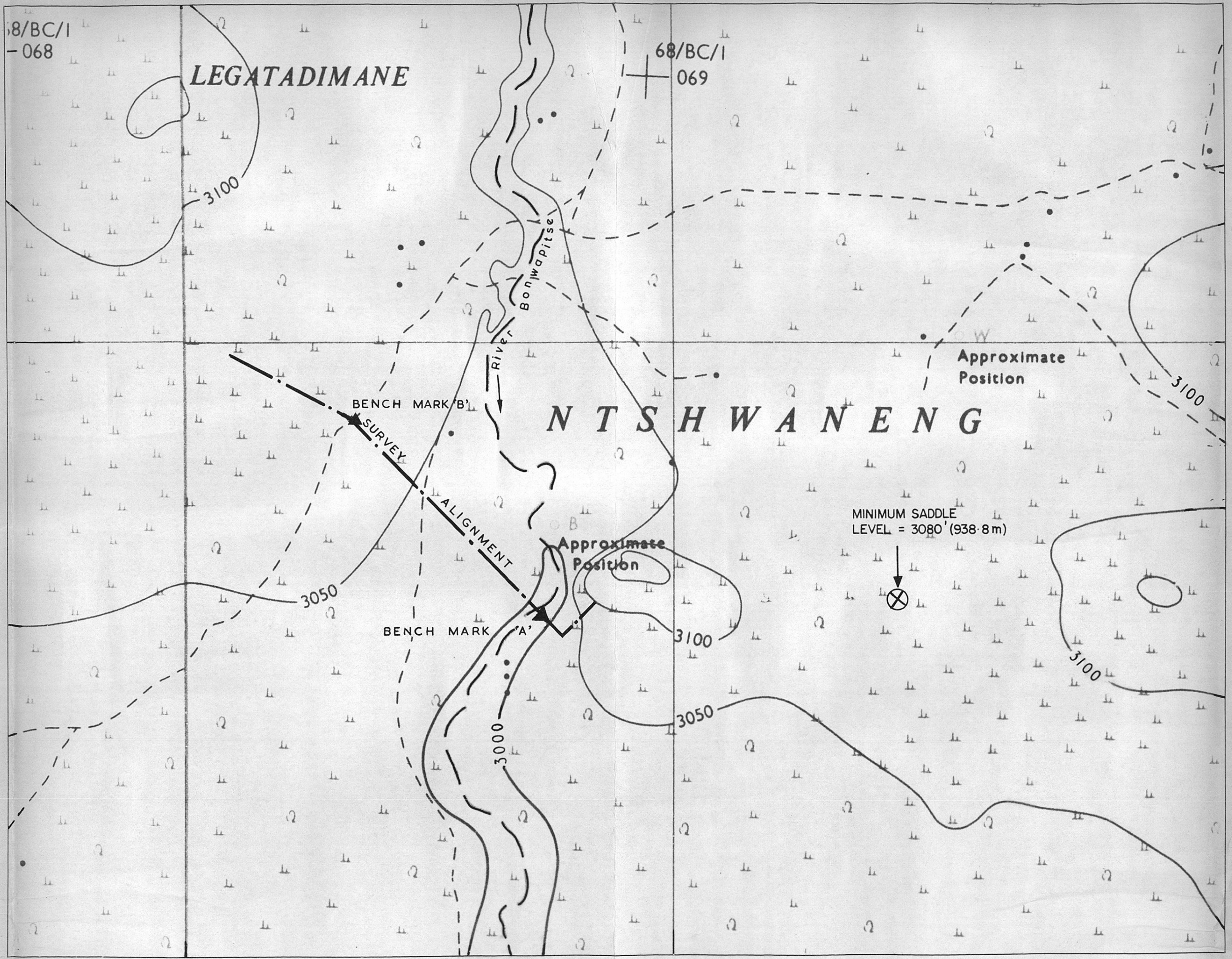
NOTE –

1. Levels shown are related to field survey datum – standard correction of +6·0m (19·7ft) assumed for correlation with local cartographic data.
2. Site datum based on trigonometric and horizontal levelling to BPS 12 (see Appendix B)

Scales — Horizontal 1 : 5000
Vertical 1 : 500

SITE No 55

FIGURE No 3.18



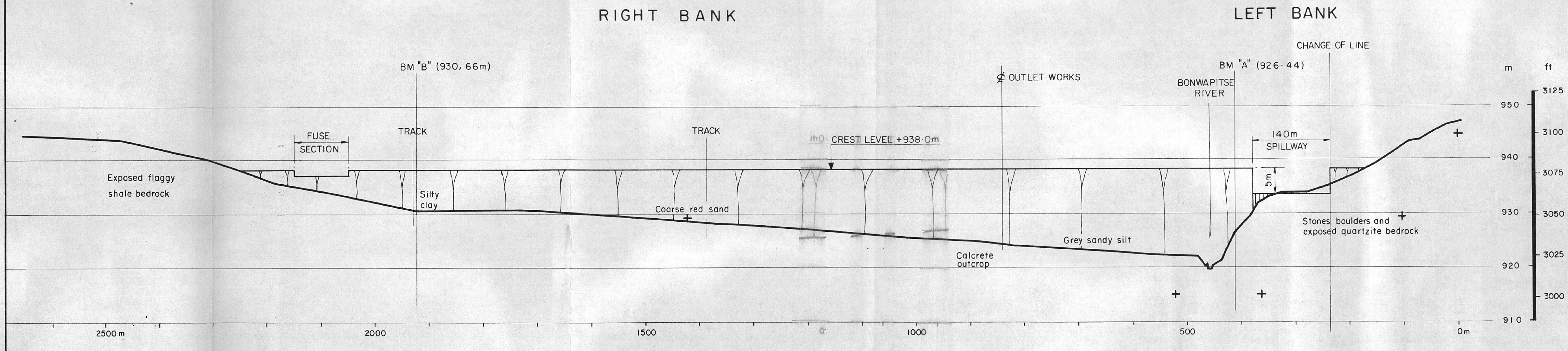
Enlargement of 1:50 000 scale Maps Nos. 2326 B3 and 2326 B4

LOCATION OF DAM SITE No. 56

SCALE:- 1:20 000

0 1 2 km.

FIGURE No. 3.19



\+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

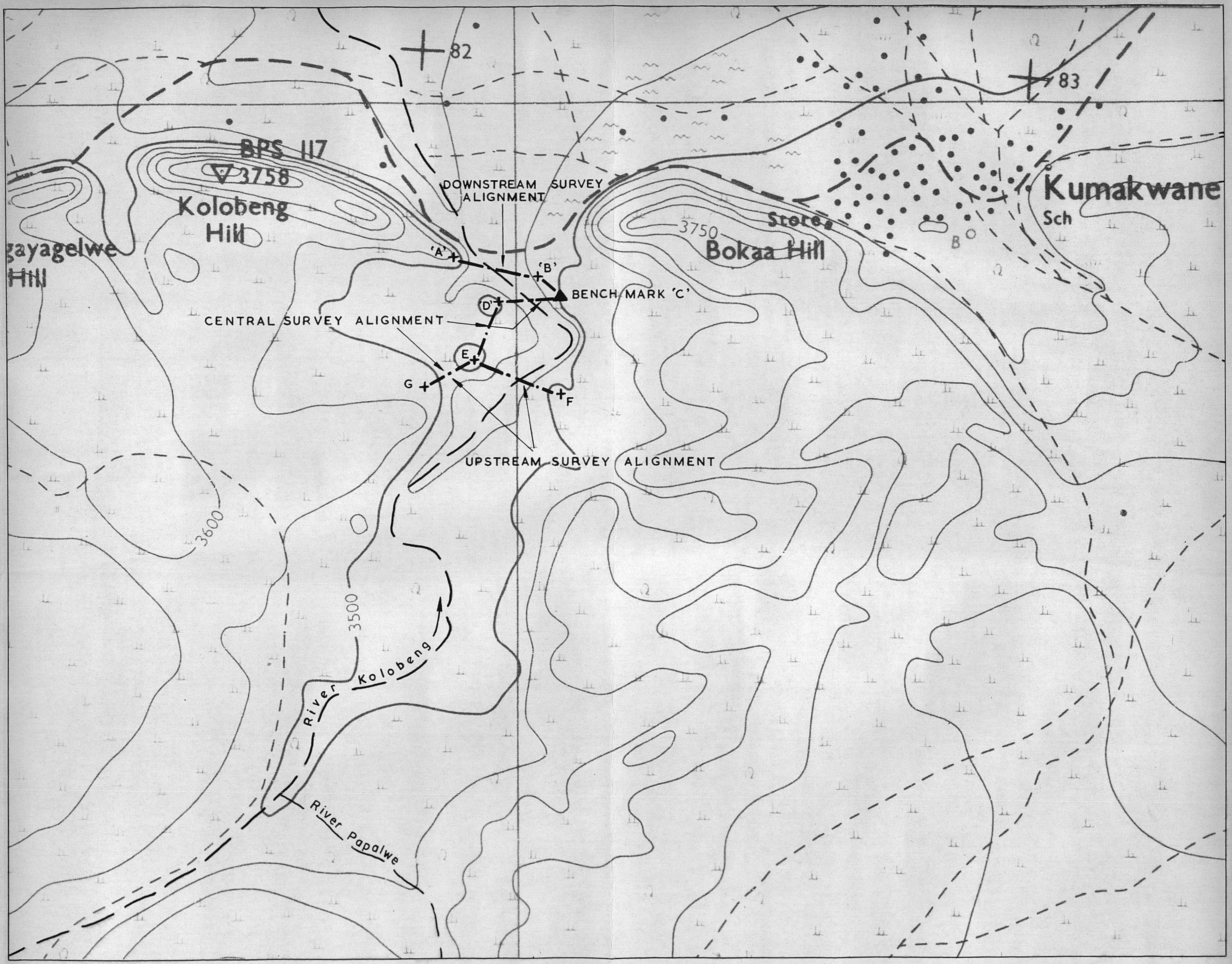
NOTE –

1 Site datum based on barometric levelling to road bridge over River Bonwapitse (see Appendix B)

Scales – Horizontal 1:5000
Vertical 1:500

SITE No 56

FIGURE No 3.20



Enlargement of 1 50 000
scale Map No 2425 D1

LOCATION OF DAM SITE No. 61

SCALE:- 1: 20 000

0 1 2 km.

FIGURE No. 3.21

UPSTREAM ALIGNMENT

(Preferred alignment)

CENTRAL ALIGNMENT

DOWNSTREAM ALIGNMENT

\+ CONTOUR LEVELS TAKEN FROM 1 : 50,000 SCALE MAPPING

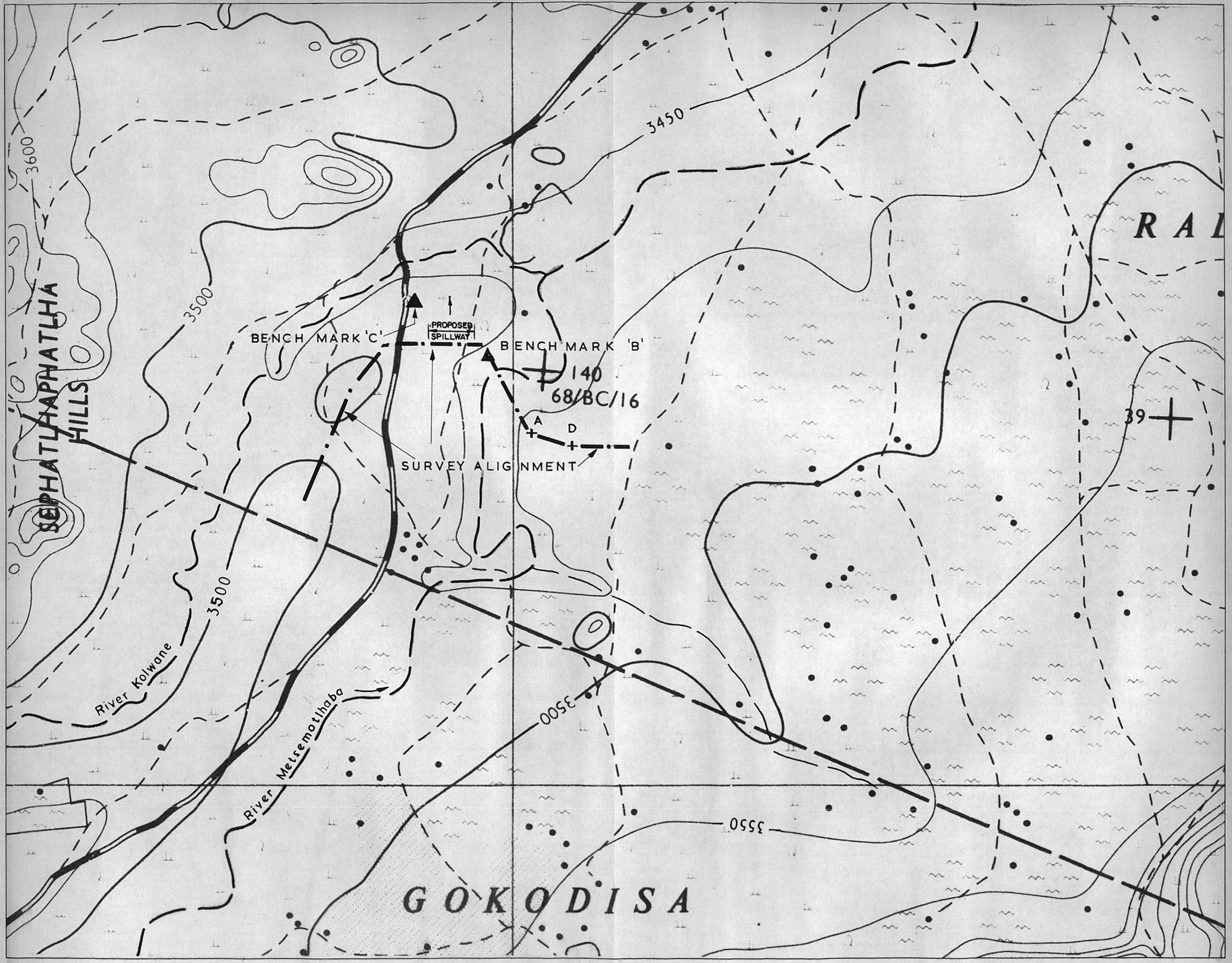
NOTE -

1 Site datum based on trigonometric and horizontal levelling to BPS 117 (see Appendix B)

Scales — Horizontal 1 : 5000
Vertical 1 : 500

SITE No 61

FIGURE No 3.22

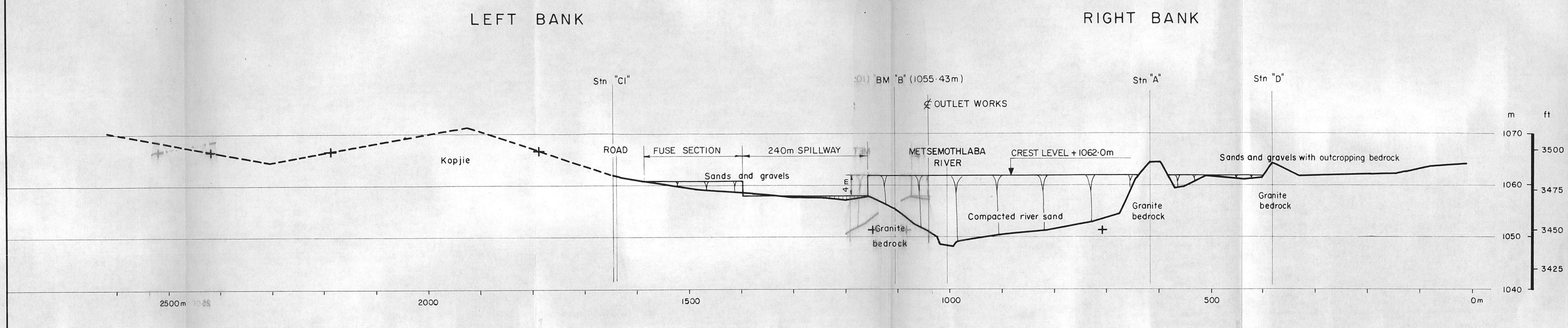


Enlargement of 1:50 000
scale Map No. 2425 D1

LOCATION OF DAM SITE No. 62

SCALE: 1:20000

FIGURE No. 3.23



+ CONTOUR LEVELS TAKEN FROM 1:50,000 SCALE MAPPING

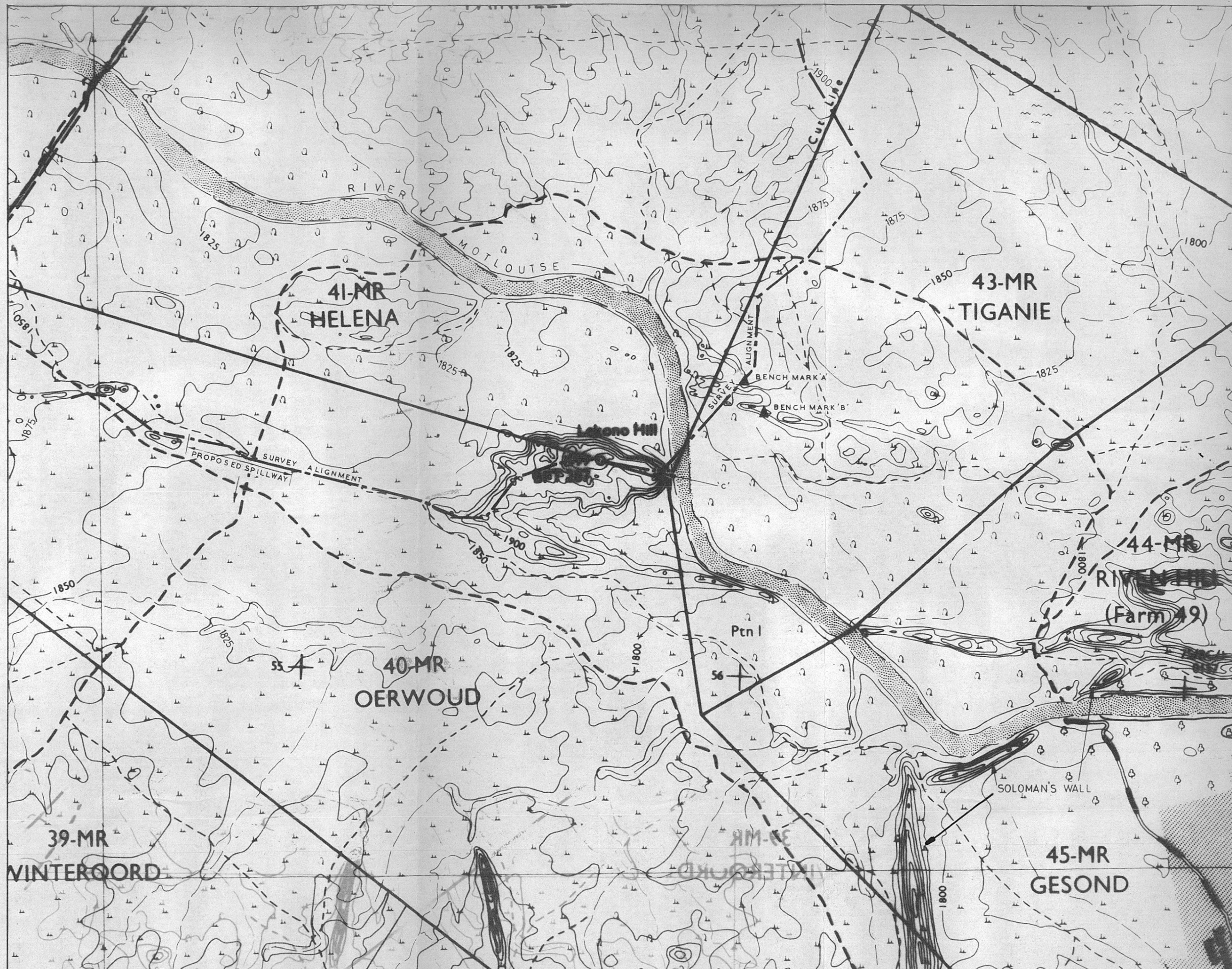
NOTE –

1 Site datum based on trigonometric levelling to BPT 239 (see Appendix B)

Scales — Horizontal 1:5000
Vertical 1:500

SITE No 62

FIGURE No 3.24



Enlargement of 1:50000
scale Map No. 2228 B2

LOCATION OF DAM SITE No.70

FIGURE No. 3.25

NOTE –

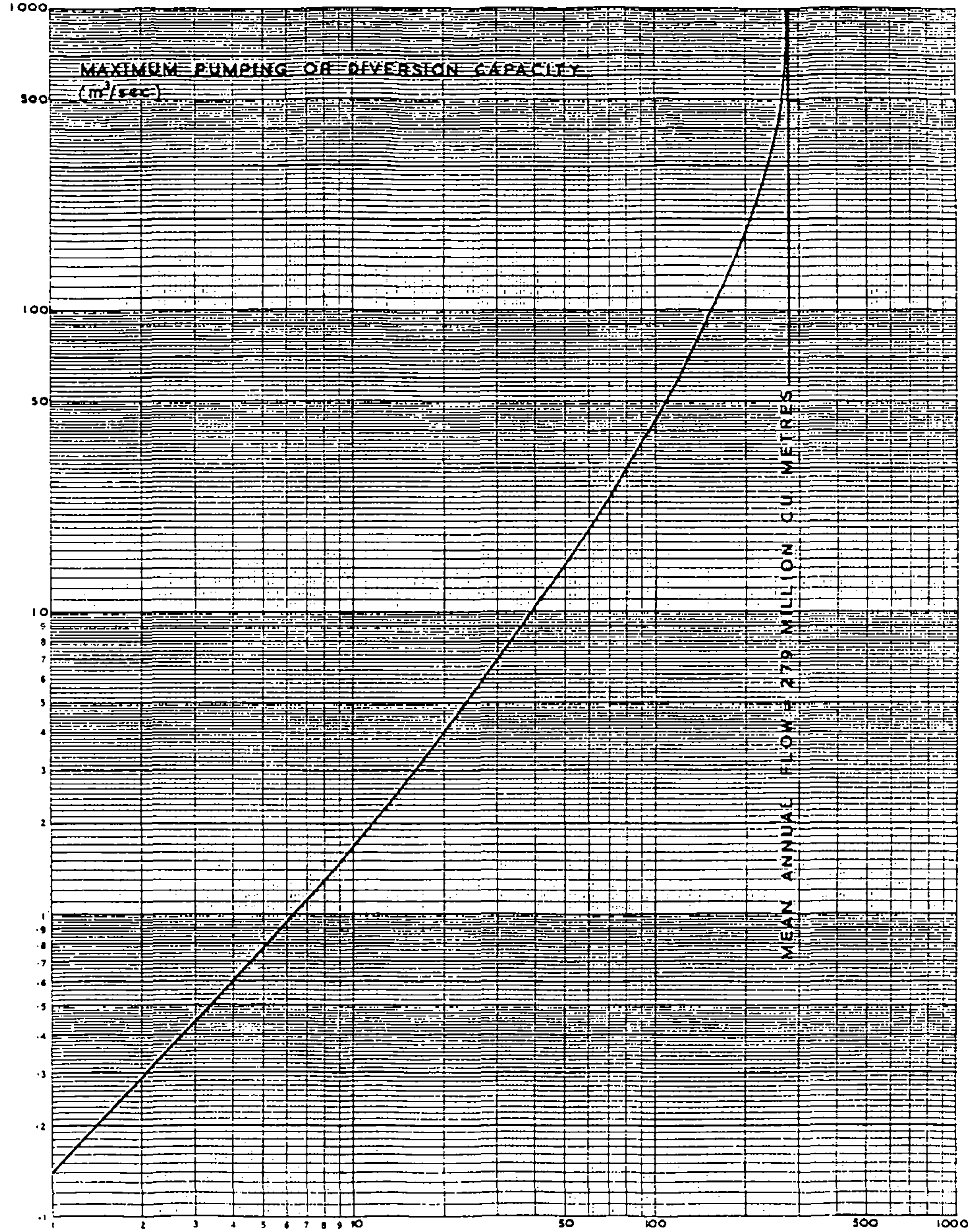
1 Site datum based on trigonometric levelling to BPT 287 (see Appendix B)

Scales – Horizontal 1:5000

Vertical 1:500

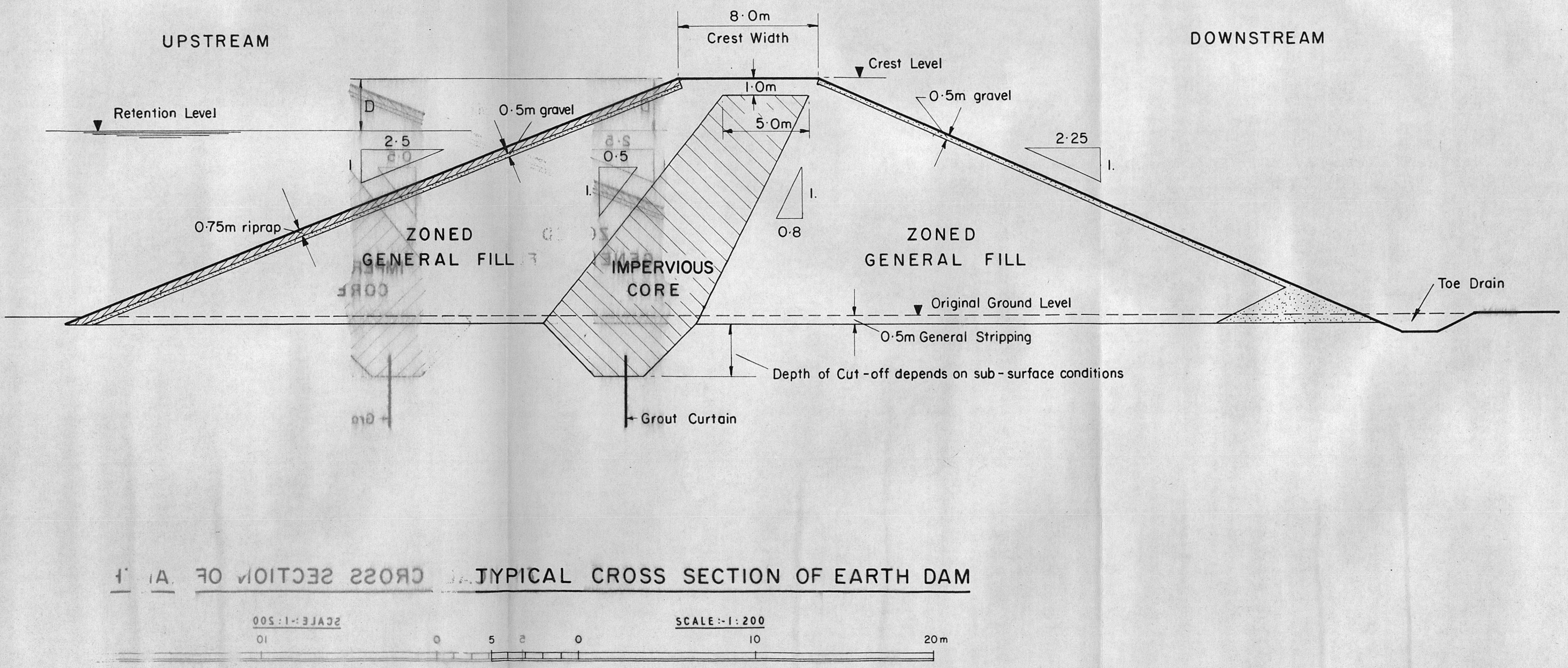
SITE No 70

FIGURE No 3.26



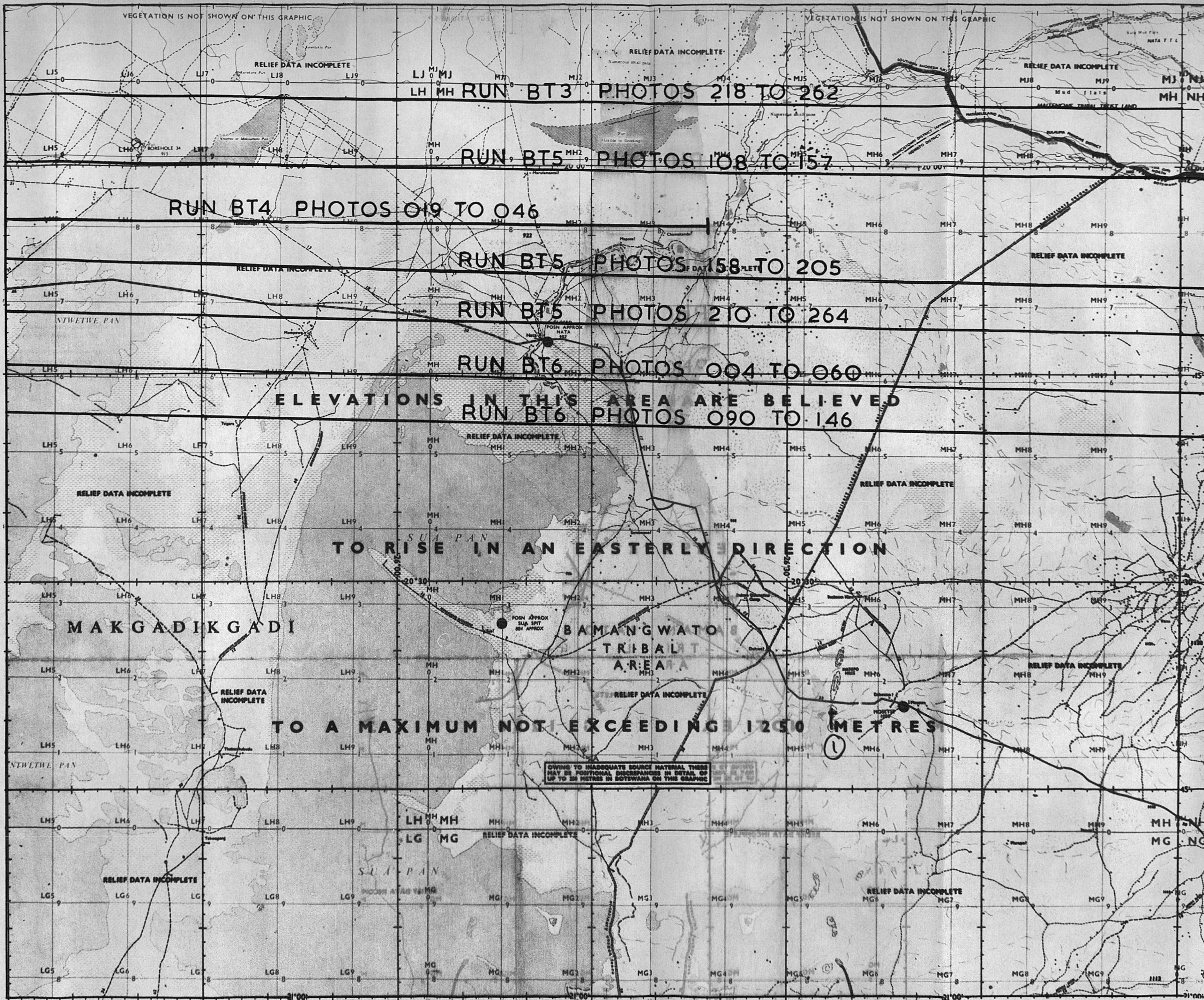
OFF-STREAM STORAGE CAPABILITY OF NATA RIVER AT NATA

FIGURE No.5.3



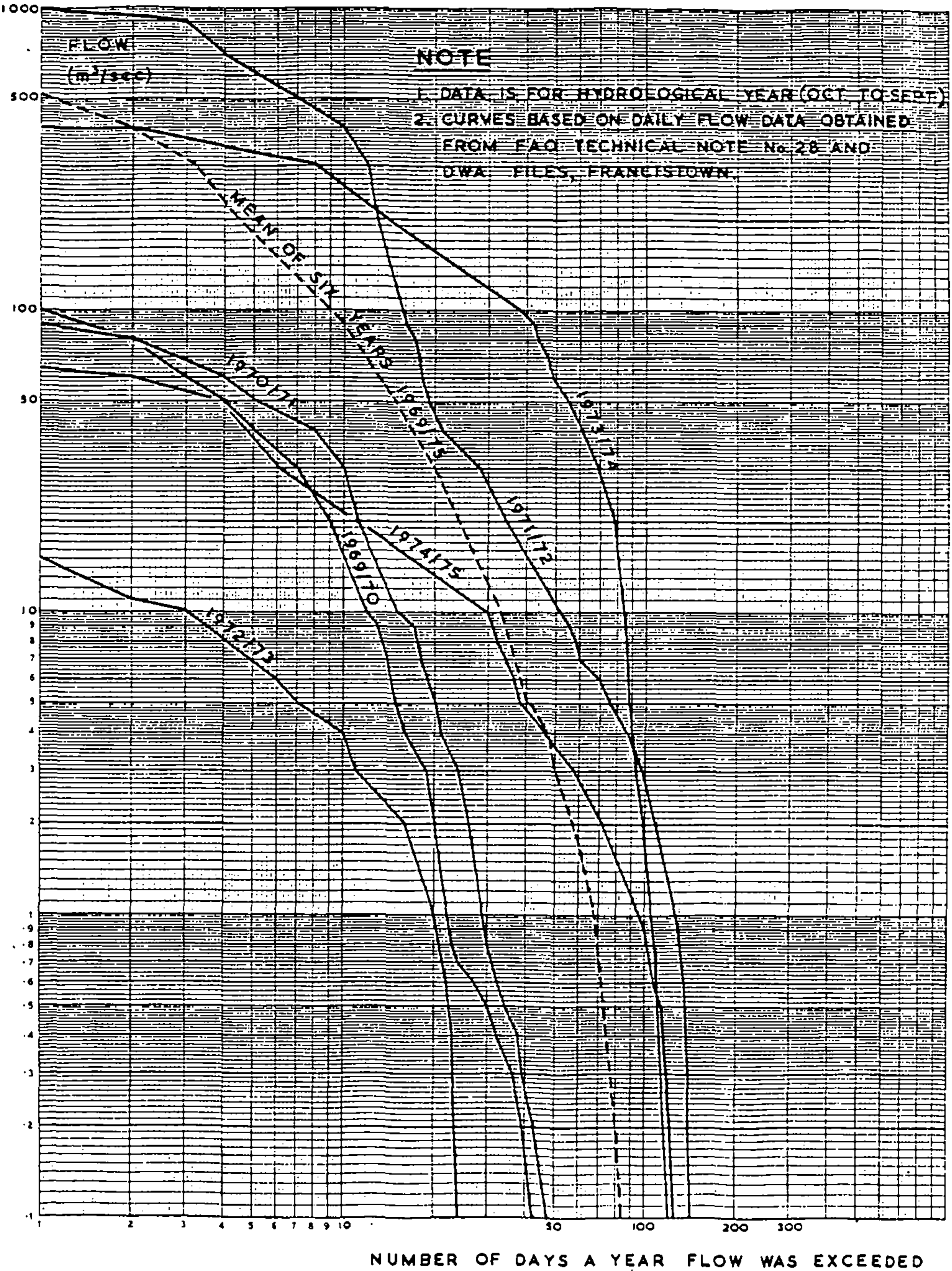
TYPICAL CROSS SECTION OF EARTH DAM

FIGURE No 4.1



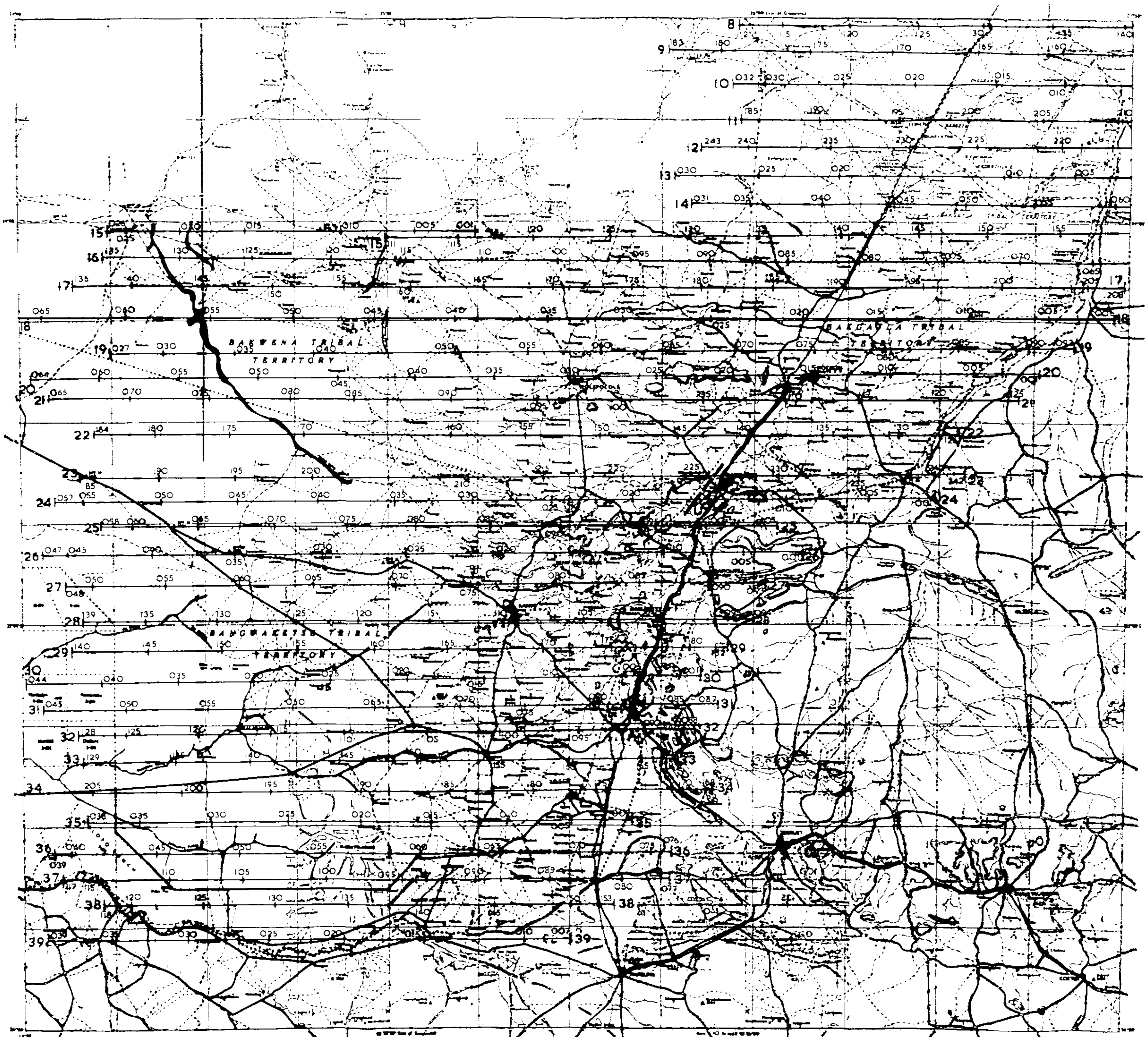
LOCATION OF AERIAL PHOTOGRAPHS, NATA AREA

FIGURE No. 5.1



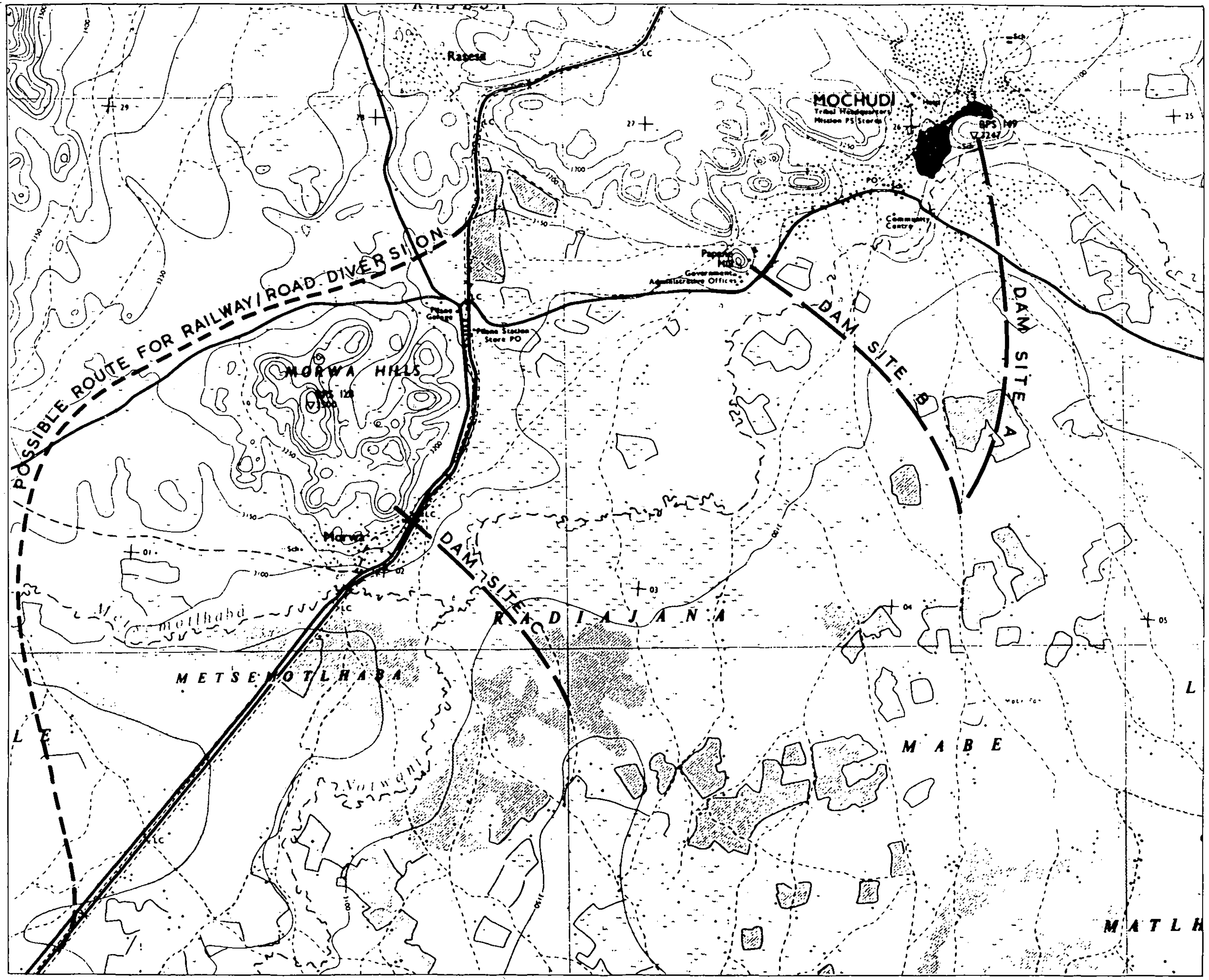
FLOW IN NATA RIVER AT NATA

FIGURE No. 5.2



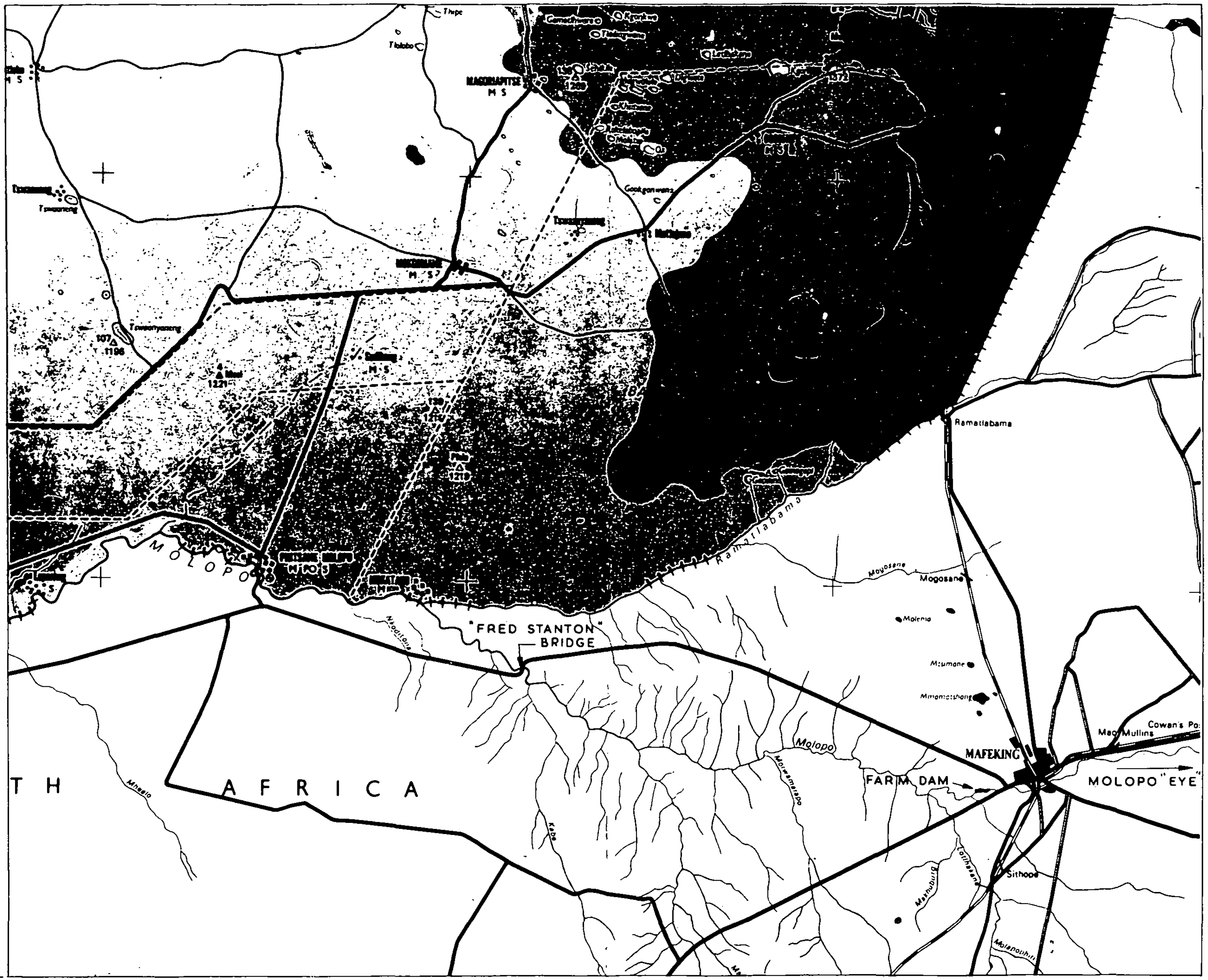
A LOCATION OF AERIAL PHOTOGRAPHS – METSEMOTLHABA – NOTWANE AND MOLOPO AREAS

FIGURE No. 5.4



POSSIBLE DAM SITES AT MOCHUDI

FIGURE No. 5.5.

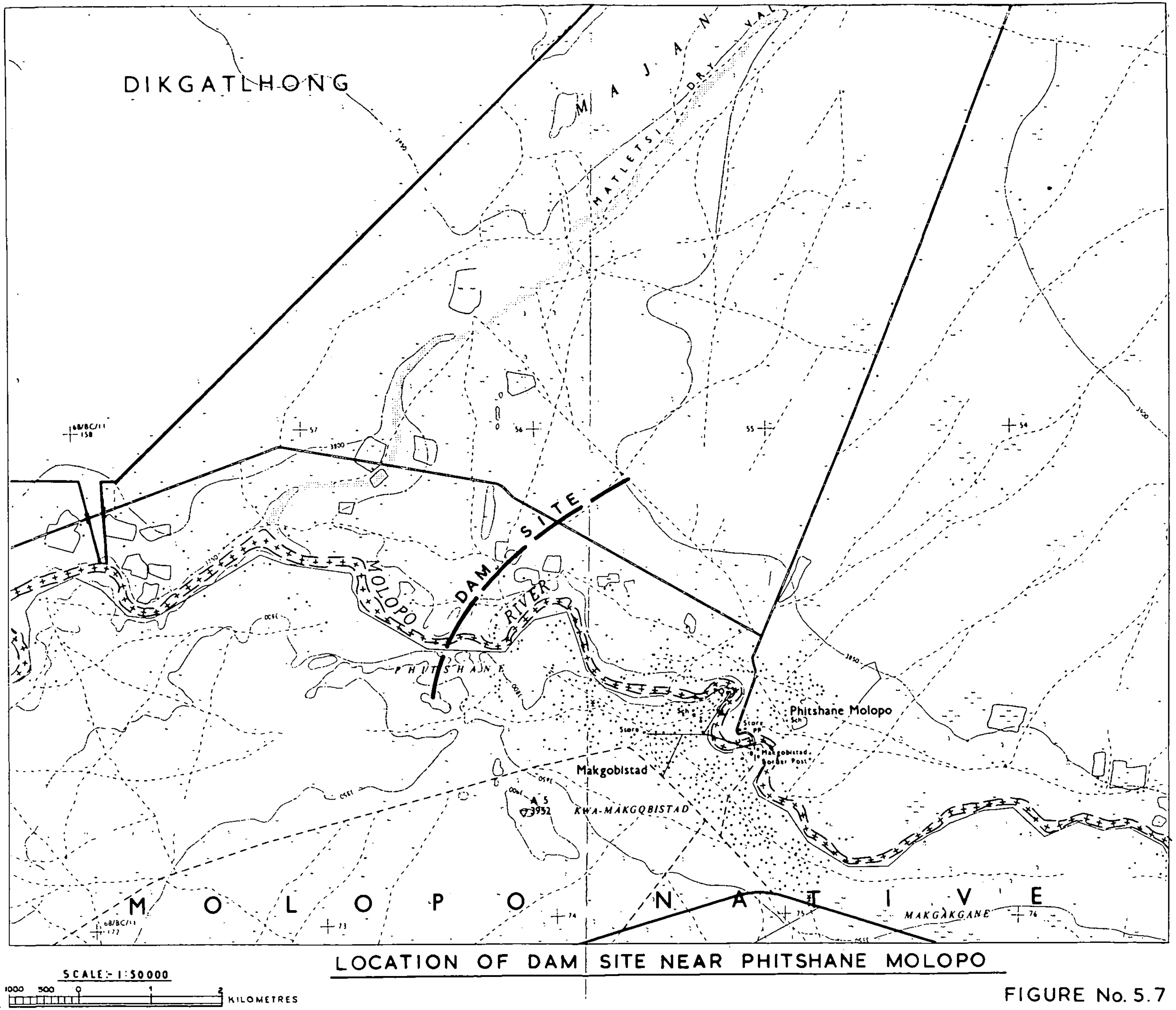


UPPER REACHES OF MOLOPO RIVER

SCALE - 1:250 000

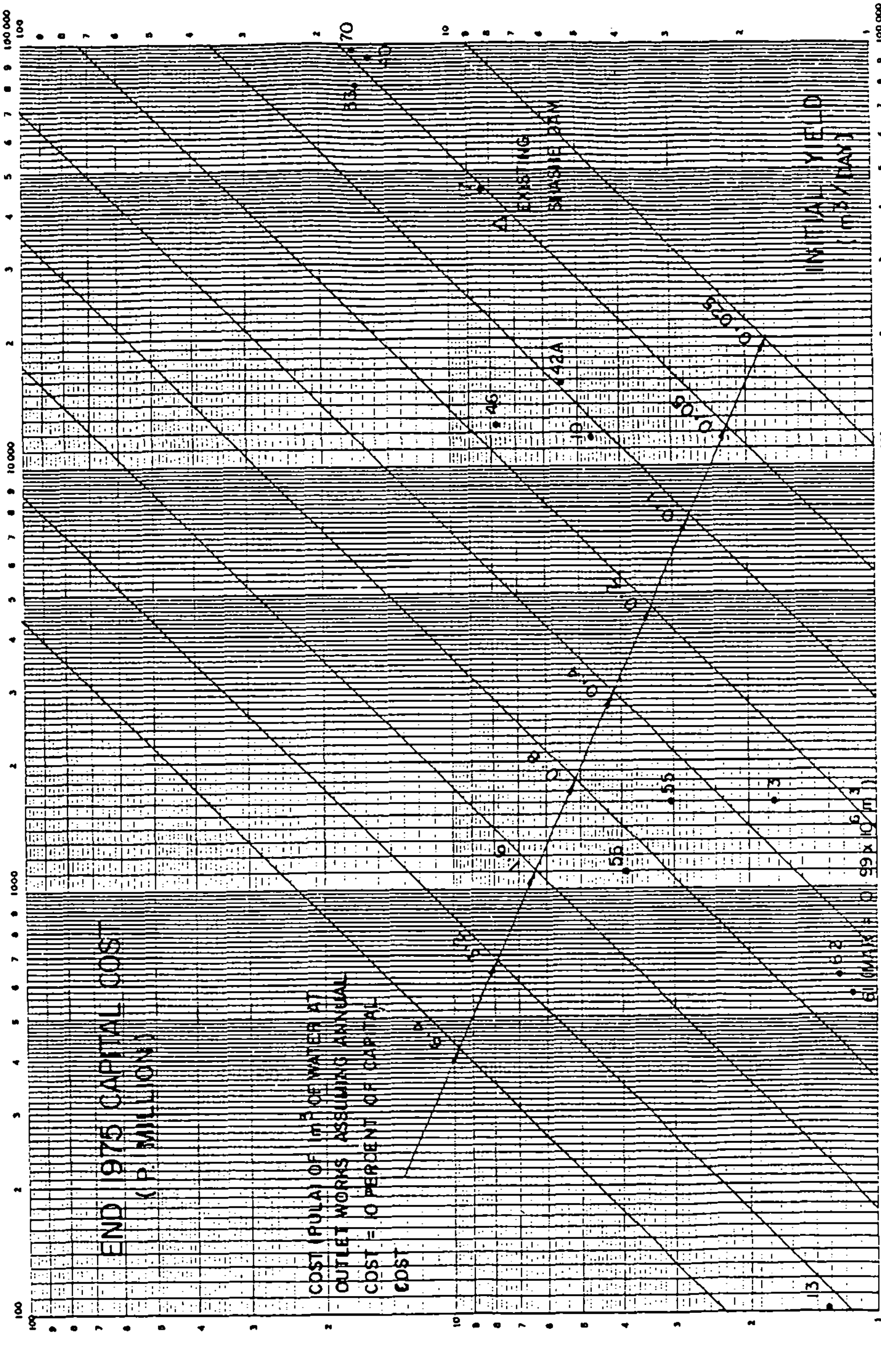
10 0 10 KILOMETRES

FIGURE No. 5.6.



LOCATION OF DAM SITE NEAR PHITSHANE MOLOPO

FIGURE No. 5.7



END 1975 CAPITAL COS
(P MILLION)
COST (PULA) OF 1m3 OF WATER AT
OUTLET WORKS ASSUMING ANNUAL
COST = 10 PERCENT OF CAPITAL
COST
INITIAL YIELD
(m3 / DAY)
EXISTING
SHASHE DAM
COST - YIEL
ISON
1 IN 50 YEARS FAILURE RISK

FIGURE No 6·1