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THE EFFECT OF CONSTRUCTION OF PROPOSED LAKE AT CHIPPING NORTON ON BEHAVIOUR OF GEORGES RIVER

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G. S. Harris and R. C. Nelson

November, 1967

University of New South Wales
WATER RESEARCH LABORATORY.

THE EFFECT OF CONSTRUCTION OF PROPOSED LAKE AT CHIPPING NORTON ON BEHAVIOUR OF GEORGES RIVER.

by

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Preface

The report tendered herein is published by the Water Research Laboratory in the form of a research document by the kind permission of Hollywood Sands Pty. Ltd.

The Company sponsored the work to gather information to present a case to Fairfield Municipal Council for the development of the area in conjunction with proposals for sand mining.

The matter was subsequently referred to the Georges River Valley Extraction Industries Committee with the authority of the Minister for Public Works, N. S. W.

R. T. Hattersley,
Assoc. Professor of Civil Engineering,
Officer-in-Charge,
Water Research Laboratory.

November, 1967.

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Summary and Conclusions

1. Summary of Investigation

The object of the investigation was to study the effect on river behaviour of the construction of a lake at Chipping Norton in the Georges River, near Sydney, NS. W. The effect on flood behaviour was measured by constructing an hydraulic model, passing appropriate flood discharges through the model, and recording and analysing the results.

The effects of the lake on tidal regime, sedimentation, pollution, bank erosion and channel stability were then evaluated by field surveys and computation.

2. Conclusions

General

The proposed lake is a desirable public amenity which will bring to the community considerable benefits in recreation, flood mitigation, provision of much needed sand for the development of Sydney, and mitigation of pollution, but it would be wise to take the precautions recommended in Section 8.22 of the report.

Flood Mitigation

(a) The construction of the lake will reduce materially flood damage in the vicinity of the lake and upstream for some distance.

(b) If the action recommended in Section 8.22 of the report is carried, the construction of the lake will reduce flood damage downstream of the lake.

Tidal Flow

(a) Unless the action recommended in Section 8.22 of the report is taken, it is expected that there will be a tendency to scour the bottom and sides of the river channel at the locations shown in Fig. 15 of the report.

(b) Until this action is taken, there will be a reduction in tidal range at Chipping Norton, which at present is the same as that of the ocean, but lagging $2\frac{1}{2}$ hours.

Sedimentation

The estimated volumes of bed load and suspended sediment which will settle in the lake are so small in relation to the volume of the lake that no siltation problems are anticipated in the foreseeable future.

Pollution

The lake will mitigate pollution effects by increasing tidal storage near the headwaters, where the increased dilution will have the maximum effect Further, there are no stagnant areas within the lake, thus preventing the accumulation of pockets of debris.

Bank and Channel Stability

(a) The construction of the lake will have no effect on river banks, except at the locations shown in Fig. 15 and perhaps in the small section referred to in Section 8.21.

(b) Provided it is specified that the banks of the lake be constructed to the specifications outlined in Figures 29 and 30, the banks will be stable against seepage and no erosion will result from wave action or floods.

1. The Nature of the Problem

The building and civil engineering work necessary for the development of the metropolitan area of Sydney is demanding an ever increasing supply of clean building sand. The readily available supplies close to the centre of demand have long been exhausted, and in recent years the Georges River has become an important source of supply of this essential material. From this river the sand is dredged, screened and washed. This activity is proceeding along a considerable length of the river, with a major concentration in the area shown shaded in Fig. 1. On completion of extraction of sand in the various areas licensed for such purposes, the landscape would normally present an unsightly appearance. Photograph No. 1 is an aerial view of the area as existing some twelve months or more ago, and extraction has proceeded apace since the photograph was taken.

This region of Sydney is ill-served with recreational areas, particularly those of an aquatic nature.

An obvious opportunity exists to convert a potential eyesore into a highly desirable community amenity, by converting the mined areas into an artificial lake which would then be available for such recreational purposes as boating, sailing, water skiing, fishing, swimming and the like, and also to provide for the citizens the pleasure of a water vista - generally a smaller version of Lake Burley Griffin.

Some years ago, the Liverpool and Fairfield Councils endorsed a decision to convert the area into a lake, and private interests put forward proposals for a golf course, picnic grounds and similar amenities. Subsequently, the Fairfield Council rescinded its decision. However, Hollywood Sands Pty. Ltd. have requested that this Company be permitted to extract sand from the area labelled Zone 2 on Fig. 1. This area is in the Fairfield Municipality, whereas Zone 1 is within the Liverpool boundaries. An important feature of the proposal by this Company is that the sand extraction in this zone should be carried out in such a manner that on completion of mining, the whole of the zone will be at the planned lake depth, with banks correctly sloped, etc. so that this portion of the lake will be handed over to the Council in a completely finished condition.

However, the Minister for Local Government is concerned at the possibility that the creation of the proposed lake might aggravate flood damage in the Georges River, or cause bank erosion or some similar ill effects, and is not prepared to issue further licenses for sand extraction until his fears on this score have been allayed.

Therefore, Hollywood Sands Pty. Ltd. has requested the Water Research Laboratory of the University of New South Wales to investigate the effect of the proposed lake on river behaviour.

2. General Description of the Area

The proposed lake is located in the upper reaches of the Georges River in the vicinity of Cabramatta as shown in Figure 1. Two major creeks, namely Prospect and Cabramatta, join the river in the vicinity of the lake and at present the area is subject to flooding (on an average of about once in every 5 years) from the combined runoff from the three water courses. Under dry weather conditions, the fresh water flow is very small but is sufficient to maintain a fairly low salinity in the river at this point. The tidal influence extends upstream as far as Liverpool Weir, some two miles above the lake. The tidal range is very nearly the same as the ocean tide but there is a lag of approximately $2\frac{1}{2}$ hours.

The outline of the proposed lake relative to existing property boundaries is shown in Figure 2. This has been fixed from considerations of surface topography, present and future mining leases, property boundaries, aesthetics and watercourse boundaries. For reference, the lake has been divided into two zones; zone 1 at Chipping Norton on the south bank of the river in Liverpool Municipality and zone 2 at Hollywood on the north bank of the river in Fairfield Municipality. The surface areas of the two zones are 9.3×10^6 sq. ft. (0.334 sq. miles) and 3.2×10^6 sq. ft. (0.115 sq. miles) respectively. The approximate length of the lake is 2 miles and the width 0.3 miles. For the purpose of this investigation, the depth of the lake has been taken as 25 ft. below low water, but this dimension is not critical and could be varied quite significantly without affecting the conclusions reached as a result of this study.

3. Objectives of the Investigation

The lake might affect river behaviour in the following respects:-

(i) Flooding
(ii) Tidal regime
(iii) Sedimentation
(iv) Pollution
(v) Bank erosion and stability

The purpose of this investigation is to assess the nature and magnitude of the effects of the lake on the river in each of these aspects.

4. Effect of the Proposed Lake on Flood Behaviour

4.1 Introduction

The river valley where the lake is to be dredged is a flood plain and is subject to frequent flooding from fairly small discharges. Flows in the Georges River in excess of about 20,000 cubic feet per second (c.f.s.) overtop the river banks in the vicinity of Chipping Norton, Cabramatta and Hollywood. Such flows occur on the average of about once in every five years.

Upstream of the proposed lake the river flows between high banks and flooding is not a serious problem.

Downstream of the lake some overbank flooding occurs as far downstream as East Hills after which the river is contained within steep banks and flooding is confined to the immediate river channel. The area worst affected is Milperra where a number of homes have been built along the river banks.

Flooding in the vicinity of the lake is the result of the combined run-off from the Georges River above Warwick Farm, Cabramatta Creek, and Prospect Creek. The areas of these catchments are 137.1, 28.9 and 34.6 square miles respectively. The "once in 100 year" and "once in 25 year" flood hydrographs for each of these streams at the lake have been estimated, using unit hydrograph procedures, and are shown in Fig. 3. It is clear from this Fig. 3 that for major floods the main contribution to peak flows comes from the Georges River catchment. However, because of their smaller catchment areas, Prospect and Cabramatta Creeks may tend to peak earlier than Georges River and therefore flooding during the early stages of a major flood may be dominated by these streams.

The possible effects of the lake on flood flows are as follows:-

(i) Channel capacities will be increased and the channel length shortened by the elimination of the major river bend near Chipping Norton. This would result in decreased flood gradients and a lowering of flood levels upstream of Prospect Creek.

(ii) Additional valley storage would be provided by the lake which might attenuate the flood hydrograph resulting in lower peak discharges and flood levels downstream of the lake.

(iii) The time distribution of runoff might be changed by the lake resulting in earlier or later flood peaks. This could increase the possibility of coincidence of flood peaks with those from minor tributaries downstream leading to increased flood levels.

(iv) The additional tidal storage available in the lake will result in higher tidal velocities in the river channel downstream. This may lead to scour and an increase in channel capacity resulting in reduced flood levels downstream of the lake for a given discharge.

To investigate (i), (ii) and (iii) above, a fixed bed hydraulic model was constructed and the results are reported in Section 4.2 below. The influence of the lake on tides (item (iv) above) and on flooding benefits are reported upon in Section 5.

4.2 Hydraulic Model Tests

4.21 The Model

The inability to predict accurately flow paths, off-channel storages, overbank and channel roughness, and local topographical effects on flood stages makes analytical mathematical estimates of flood behaviour somewhat suspect. A reliable answer can be obtained using an hydraulic model based on the verification principle which states that if a model can be adjusted to reproduce accurately past events it will also accurately reproduce future events.

For the above reasons, a fixed bed hydraulic model (Photograph 2) was built to study the effect of the lake on floods. The scales of the model were 1:100 vertical and 1:900 horizontal. The boundaires of the model are shown in Figure 4 and reproduce the flood plain from above Warwick Farm Racecourse to Georges Hall.

Model details were based on the following plans:-

(i) County of Cumberland Series, 4 chains to the inch, topographical plan.

(ii) Hydrographic survey for Georges River (1959) as prepared by the Department of Public Works, N. S. W.

These plans were supplemented by additional topographical and local information where needed.

Flow to the model was controlled by gate valves and measured using standard orifice flow meters. Water levels were measured by point gauges. Downstream levels from the model were controlled by an adjustable tailgate.

4.22 Model Verification

The flood chosen for model verification was that of November 1956 since most information was available on this flood. Local enquiry established ample information on flood levels at critical locations in the area and these are shown in Figure 5. In regard to flood discharges, however, no measurements have been taken by any authority and river flows during the flood were estimated by analytical methods using the Manning equation. These were based on water surface slopes as measured in the river (see Figure 6) and estimated values of channel roughness. This resulted in an estimate of flow rate of 51,500 c.f.s. for Georges River at Lansvale (includes small discharge from Cabramatta Creek) and 8000 c.f.s. for Prospect Creek at the time of the 1956 flood peak in the Georges River.

These discharges were then fed into the model and the roughness of the model was adjusted in accordance with standard practice until the actual recorded flood levels were reproduced on the model.

The results of the final verification of the model are shown in Fig. 6. An examination of this figure shows that excellent verification was obtained.

The model was then further checked for accuracy by feeding in the flood discharges of the November 1961 flood, which were 24,000 c.f.s. and 5000 c.f.s. for Georges River and Prospect Creek respectively. The levels which were measured on the model were compared with those which actually occurred in the "real life" conditions of the prototype. The results are also shown on Fig. 6, and it will be noted that good agreement was again obtained.

From the above tests, the model has been shown to reproduce accurately past flood levels and consequently can be expected to predict accurately the effect of the lake on flood behaviour in the future.

4.23 Test Discharges

Once the model had been verified, a series of tests were made to determine the effect of the proposed lake on flood behaviour. Three flood discharges were selected to cover flood frequencies of 1 in 5 years, 1 in 25 years and 1 in 100 years. Estimated peak discharges for these floods are shown in Table I.

Table I

Flood Discharges used for Model Tests

Flood Frequency	Estimated Discharges in c.f.s.	
	Georges River at Lansvale	Prospect Creek
1 in 5 years	22,000	3,000
1 in 25 years	51,500	8,000
1 in 100 years	97,000	24,000

The 1 in 5 year flood corresponds to that which just tops the banks and causes nuisance flooding. The 1 in 25 year flood corresponds to the flood which occurred in November 1956 on which model verification was based. The 1 in 100 year flood corresponds approximately to the 1873 flood, the highest flood ever recorded (Reference 1).

The discharges in Table I are those existing when the floods were at their peaks.

In addition, the complete flood hydrographs for the "once in 25 year" and "once in 100 year" floods were estimated using synthetic unit hydrograph procedures (Reference 1) and are shown in Figure 3.

4.24 Test Results for Unsteady Flows

The first series of model tests were run to determine the attenuation of the flood hydrograph by surface storage before and after construction of the lake. The inflow hydrographs for the 1 in 25 year and 1 in 100 year floods were approximated in the model by the stepped hydrographs shown in Figures 7 and 8. The outflow hydrographs, with and without the lake incorporated in the model were then observed. Test results are shown in Figures 7 and 8. These clearly indicate that the lake has no significant effect on the passage of the flood wave, peak discharges being reduced by less than 4 per cent as a result of the additional storage provided by the lake.

All further tests were therefore run under steady state conditions.

4.25 Test Results for Steady Flow Conditions

The peak discharges for the 1 in 5 year, 1 in 25 year and 1 in 100 year floods were run through the model and flood levels observed at the seven locations shown in Column 2 of Table II before and after construction of the lake. Test results are shown in columns 3, 4 and 5 of Table II and are illustrated graphically in Fig. 9. The changes in levels at various locations are also shown in the plan of Fig. 10.

Table II: Flood Levels for Various Flood Frequencies with and without the Lake.

Flood Frequency	Station (see Fig. 5 for location)	Water Level to Standard Datum Pre-lake (ft.)	Water Level Standard Datum Post-lake (ft.)	Change in Water Level (ft.)
1	2	3	4	5
1 in 5 yrs.	1	19.35	12.85	-7.5
	3	14.90	12.80	-2.1
	5	14.70	12.90	-1.8
	7	14.00	12.90	-1.1
	8	12.40	12.80	+0.4
	9	12.60	12.60	0.0
	11	14.10	12.90	-1.2
1 in 10 yrs.	1	23.35	17.55	-5.8
	3	18.75	17.50	-1.2
	5	18.60	17.60	-1.0
	7	19.40	17.60	-1.8
	8	17.40	17.30	-0.1
	9	17.00	17.00	0.0
	11	18.70	17.70	-1.0
1 in 100 yrs.	1	26.45	23.40	-3.0
	3	23.70	23.40	-0.3
	5	23.60	23.40	-0.2
	7	23.80	23.50	-0.3
	8	23.00	24.00	+1.0
	9	22.50	22.50	0.0
	11	24.00	23.50	-0.5

A study of these results as given in Table II, with reference to the locations in Fig. 5, and as shown in longitudinal section in Fig. 9, and the plan of Fig. 10, shows clearly that the proposed lake would lower considerably the flood levels upstream of Prospect Creek. The largest reduction in flood levels occur in the vicinity of Warwick Farm Racecourse where flood stages were decreased by 7.5 ft. for the 1 in 5 year flood and 3.0 ft. for the 1 in 100 year flood. The reason for this is that the lake eliminated the high losses which presently occur around the S-bend in the river between Warwick Farm and Hollywood by providing a river cut-off and an easier passage for the flow. Downstream of the S-bend, flood reductions were somewhat smaller. At Hollywood, flood stages were reduced by 1.1 ft. for the 1 in 5 year flood and 0.3 ft. for the 1 in 100 year flood.

It should be noted that in these tests flood levels downstream of the lake have been assumed to be unchanged as a result of construction of the lake. As demonstrated in Section 4.23 above and illustrated graphically in Figs. 7 and 8, the peak flood discharges downstream of the lake are only slightly reduced by the presence of the lake, and, for river conditions similar to those now existing, flood levels will be practically unaltered by the lake. However, attention is drawn to the fact that changes in tidal conditions are likely to increase channel capacity downstream of the lake and hence reduce the downstream flood levels, as discussed in Section 5.5 below.

4.26 Summary of Results of Investigation of Flooding

Results of the model tests may be summarised as follows:-

(a) The construction of the lake will reduce flood levels upstream of Prospect Creek.

(b) The maximum reduction will occur about the upstream end of the lake, where the existing river flow is constricted by an S-bend between Warwick Farm and Hollywood.

(c) The lake will have no significant effect on the flood wave and flood discharges and the time distribution of runoff will be the same as before the lake is constructed.

(d) The lake will cause negligible reduction of flood discharges and levels downstream of the lake.

5. Effect of Lake on Tidal Regime

5.1 Introduction

The proposed lake will increase the water surface area by 12.5×10^6 sq. ft. For the present tidal range to be maintained, the tidal discharge must be increased, resulting in higher velocities in the downstream channels. However, as this will increase the friction loss, the tidal range immediately after completion of the lake is likely to be slightly less than at present. The magnitude of these changes and their effects are discussed in Sections 5.2 to 5.5 below.

5.2 Effect of Lake on Tidal Range

An estuary is usually funnel shaped, which results in convergence of the tidal wave as it moves upstream. This convergence tends to increase the tide range and compensate for the loss in tide range which results from bed friction. In many estuaries, the shape in plan tends to be such that the effects of convergence and friction are approximately equal, and under these conditions the tide range is constant along the entire length of the estuary. Tidal ranges have been measured for the Georges River Estuary by the N. S. W. Department of Public Works. The variation in range with distance upstream is shown in Figure 11 and shows little variation along the estuary.

On completion of the lake, the present tidal flows will be insufficient to reproduce the full tidal range in the lake. Consequently, water surface slopes will be increased, resulting in higher velocities and flow rates in the river channel downstream of the lake. If friction losses are neglected, the velocity would simply increase until the flow rate was sufficient to re-establish the tidal range. This would require an increase in the maximum tidal velocity of approximately 1 foot per second (f. p. s.) to a value of approximately 2 f. p. s. In practice, the higher velocities increase friction loss and the tidal range will be slightly less than at present.

If the river had been left in its natural condition of tidal equilibrium, an attempt could be made to estimate the magnitude of this change. However, long sections of the downstream river channel have already been considerably dredged for sand, as shown in Fig. 12. This has already increased the flow capacity to a marked degree. Tidal computations would require detailed hydrographic surveys of the existing river channels and these are beyond the scope of this investigation. It can be stated with a fair degree of confidence, however, that frictional effects will be small and that the tidal range after construction of the lake will be only slightly less than that now occurring. Of somewhat more importance is the possibility that the higher velocities might induce scour in some portions of the river channel. This is discussed in the next section.

5.3 Effect of Lake on Downstream River Channel

Where an estuary is in erodible sediments, the velocity of flow in the channel tends to adjust to a critical value consistent with the nature of the bed material and rate of supply of sediment. At this value the average sediment supply to an area is equal to that transported out. There is thus no nett sediment transport for a whole tidal cycle and equilibrium of channel form is attained. If the velocity is increased, the channel will scour and if decreased siltation will occur. The tidal discharge past any river section depends upon the volume passing this point over a tidal cycle, i.e. the upstream tidal prism (volume of water between high and low water). If the critical velocity for scour is approximately constant then channel area is related to the tidal prism upstream of the point being considered.

An hydrographic survey of the river was made by the Department of Public Works in 1959 prior to the large scale sand dredging operations (see Figure 12) which have since been carried out. Channel areas taken during this survey have been plotted against the tidal prism upstream of the section as shown in Figure 13. A good correlation is obtained, although, as would be expected, the form of this relationship depends upon the type of bed material through which the channel is cut.

Several points do not lie exactly on the curve, the most likely reason for this being local variation in bed materials or artificial deepening of the channel by sand dredging as noted on the figure.

Because of the good correlation obtained, Figure 13 can be used with some confidence to estimate the likely changes in channel areas that will result in the long term as a result of the increased storage provided by the lake.

The computed channel areas required to accommodate the increased tidal storage are shown in Table III. These have been calculated for Zone 2 only and also for Zone 1 + Zone 2 (see Figure 1). The increased volumes of the tidal prism for Zone 1 and Zone 2 are approximately 43 x 10^6 cubic feet and 14x10^6 cubic feet respectively.

Table III.

Estimated Changes in Channel Areas and Depths

Cross Section Number Refer to Fig. 12	Stable Channel Area: Existing Conditions (sq. ft.)	Zone 2		Zone 1 + Zone 2		Average Depth of Estuary as per P. W. D. 1959 Hydrographic Survey (ft.)	Approx. Depth 1967
		Estimated Stable Channel Area after Construction of Lake (sq. ft.)	Average Depth Required with present width (ft.)	Estimated Stable Channel Area after Construction (sq. ft.)	Average Depth Required with Present Width (ft.)		
1	2	3	4	5	6	7	8
42	2100	2780	9.2	4600	15.3	9.8	9
39	2300	2900	12.1	4700	19.6	12.5	14
36	2400	3000	12.5	4650	19.4	9.2	28
31	2700	3300	10.3	5000	15.6	12.8	16
25	3000	3600	10.9	5200	15.8	9.6	10
19	4100	4700	15.7	6100	20.4	7.4	12
13	4400	4900	12.6	6400	16.8	10.5	27
7	4800	5300	14.3	6800	18.4	10.8	31
3	4900	5400	21.6	6900	27.6	16.8	26
2	5000	5500	13.4	7000	17.1	20.4	28
1	7000	7450	9.3	8700	10.9	7.8	27

Fig. 12 shows the above table in graphical form. The short dashed line (the lowest) in this Figure, shows the depth of channel required (with the existing width of channel) to accommodate without any further scour the flow of the tidal prism if the complete lake were constructed (Zones 1 and 2 of Fig. 1). The full line shows the depths as measured by the University of New South Wales' echo sounder in 1967. It will be seen that in many places dredging for sand has already deepened the channel below the necessary depth to prevent scour. Where this is not the case, the construction of the lake will tend to scour the bottom and sides of the river channel until the cross sectional area is great enough to carry the increased flow without causing scour. Where the banks are rocky, all scour will occur on the bottom of the channel. The scouring effect would probably be a relatively slow process. By the time the lake is constructed, the existing vigorous activity in sand dredging along the river downstream of the lake will deepen adequately some sections which do not at present have adequate cross-sectional area. Until all cross-sectional areas are adequate for full tidal flow, the tidal range in the Chipping Norton area will be slightly less than at present.

Figure 14 shows the location of existing areas of erosion.

5.4 Measures Necessary to Prevent Erosion due to Increased Tidal Flow Resulting from Construction of Proposed Lake

This question is discussed fully in Section 8.22 below, but some comments are also appropriate in this Section.

If sand dredging had not occurred along the river channel downstream of the proposed lake, one objection to the lake proposal would have been that the increased velocity of flow due to the increased tidal prism volume would tend to scour (albeit slowly) the sides as well as the bottom of the river, at all locations except where the banks are rocky and not erodible.

Due to the extensive sand dredging already carried out, and to the vigorous dredging activity still continuing, this objection becomes quite a minor one.

Fig. 15 shows the action necessary to ensure that after completion of the lake bank scour will not occur.

From the downstream end of the lake to Milperra Bridge, no development has occurred on the banks of the river, and sand dredging companies should be only too pleased to deepen the channel and slightly widen it to provide the necessary cross section.

In the vicinity of Milperra Bridge, the banks have been developed, and have already been protected against erosion, presumably on account of scouring from natural floods. This protection should be improved, as much of it is in bad condition.

Near the Williams Creek confluence, some dredging of the bottom and consequent widening is desirable. There again, the area is undeveloped and sand dredging companies should easily be able to carry out the required dredging.

Finally, the site of the submarine cable shown in Fig. 15 has been left severely alone by sand dredgers. Therefore, with the completion of the lake, bottom scour may be expected, and after a time, (perhaps a fairly long period) presumably the cable may be suspended above the bottom of the river.

5.5 Flood Mitigation Benefits

Flood levels downstream of Prospect Creek will be decreased as a result of the increase in channel area. This secondary benefit of the proposed lake scheme was not considered in the comments on flood studies in Section 4 above. The maximum benefit will be attained for nuisance floods or those which at present just overtop the banks. For major floods the increase in channel area is very small and only a small reduction in flood levels will result.

5.6 Summary of Results of Study of Tidal Effects

The following is a summary of the conclusions reached:-

(a) The construction of the lake will mean in the first instance that the present tidal flows will be insufficient to maintain the existing tidal range in the vicinity of Chipping Norton, although the reduction in tidal range will not be great.

(b) The increased tidal storage caused by the lake will tend to cause an increased tidal flow in the river channel downstream of the lake, resulting in increased velocities at some points along the river system.

(c) These increased velocities will cause scour of the bottom of the river, (and of the side slopes of the river channel where the river banks are made up of erodible material) at those points where the existing cross-sectional area of the river is not adequate to convey the increased flow at non-eroding velocity.

(d) The rate of erosion cannot be predicted at this stage.

(e) The locations where such erosion may occur are shown in Figs. 12 and 15.

(f) Dredging of the river channel already carried out downstream of the proposed lake has minimised these possible erosive effects, and the continuance of the existing vigorous dredging activity will continue to reduce the length of river channel along which erosion may be caused by the construction of the lake.

(g) Due to deepening and widening of the river channel by sand dredging operations downstream of the proposed lake, any scouring of the banks of the river due to increased tidal flow caused by the existence of the lake will be of minor importance, and can be readily prevented by the measures proposed in Fig. 15.

(h) Any increase in the cross sectional area of the river downstream of the lake by scour effects of the increased tidal flow caused by the lake, or by the measures suggested in Fig. 15, will reduce flood damage downstream of the lake, and will be particularly effective in eliminating damage caused by the more frequent nuisance floods.

(i) When the cross-sectional areas of the river channel have been increased to carry without scour the increased tidal volume, the tidal range in the lake will be the same as at present, i.e. the same as the ocean although lagging about $2\frac{1}{2}$ hours.

6. Effect of Proposed Lake on Sedimentation

6.1 Introduction

The widening of the river channel resulting from the construction of the lake would be expected to reduce the velocity of flow, and it is necessary to investigate how much of the sand and silt load of the stream will settle in the lake, and whether this will result in serious shoaling effects.

The sediment is of two types - heavy particles transported along the bed of the stream, and fine particles suspended in the stream.

6.2 Sediment Rating Curve

The bed load of heavier particles at various flow rates has been estimated for a selected reach of the Georges River located approximately 1 mile upstream of the Liverpool Weir (see Fig. 16). The river in this region is contained between high straight banks, has a regular cross section and is in apparent equilibrium without any aggradation (silting up) or degradation (scouring). Consequently, the sand that moves down the reach can be expected to pass into and through the Georges River Estuary. A longitudinal section of the reach is shown in Figure 17.

Bed samples were obtained using a clam shell grab and gradings of 4 samples taken at the gauging section are shown in Figure 18. The average D_{35} grain size was 0.37 mm and the average D_{65} grain size was 0.32 mm (i.e. 65 percent by weight of the grains in the sample have a diameter smaller than D_{65} grain size and 35 percent smaller than the D_{35} grain size).

The stage-discharge curve at Liverpool Rd. Bridge is shown in Fig. 19.

Stage-discharge relationships for the reach were calculated from Manning's Equation using an estimated value of the roughness parameter of 0.030. Water levels computed for various flow rates are shown in Figure 17.

Using the above information, the bed load was computed over the range of discharges from 2,500 c.f.s. to 160,000 c.f.s. using the Bureau of Reclamation Step Method for the Modified Einstein procedure. The bed sediment rating curve so obtained is shown in Figure 20.

6.3 Average Annual Bed Load

If the flow duration curve is known showing the percentage of time that a given flow occurs, this can be combined with the sediment rating curve to obtain the average annual bed sediment load. No discharge measurements have been taken near the gauging site (catchment area 137 sq. miles) and the flow duration curve was synthesized by comparison with the records from the Nepean River at Nepean Dam. Discharge records for the latter catchment are available over the period 1919-1935 whilst the catchment area (123 sq. miles) and flood producing characteristics are similar to those at the gauging site on the Georges River. The estimated flow duration curve is shown in Figure 21.

The flow duration curve was combined with the bed load rating curve to obtain the bed sediment load-duration curve shown in Fig. 22. From this the total annual bed load was estimated by graphical integration at 26,000 tons of sand per year.

6.4 Sand Inflow from Large Floods

The sediment rating curve (Fig. 20) shows that there is no significant sand transport for river flows below about 2,500 c.f.s. Under dry weather conditions the flow is always less than this value, so that sand transport occurs only during freshes and floods. Combining the sediment rating curve (Fig. 20) with the flood hydrographs shown in Fig. 3, the total sand transported as bed load by the 1 in 100 year flood and the 1 in 25 year flood is estimated at 12,500 tons and 2,600 tons respectively.

6.5 Shoaling of Lake by Sand Transport

The sand bed load will be dropped in the headwaters of the lake as a delta which will gradually move through the lake from the upstream end. Assuming a lake depth of 25 feet, an average width of 1200 feet at the upstream end and a settled density of 100 lbs./cu.ft., the average annual advance of the delta has been calculated at 20 feet per year. Consequently, maintenance dredging will be infrequent and therefore presents no problems. This is particularly the case since the sand is a marketable product with an economic value.

6.6 Suspended Sediment Load

In addition to the sand bed load, the river transports a considerable amount of silt as suspended sediment. Because of the lower settling velocity, this silt will be carried further into the lake and will tend to settle more uniformly over the bed of the lake, or pass right through the lake.

Suspended sediment samples have been taken from the river under normal tidal conditions and during river floods. Sediment concentrations are shown in Table IV.

Table IV shows that the Georges River above Cabramatta Creek is comparatively free of silt and sediment concentrations are similar for both tide and river flood conditions. Cabramatta Creek and Prospect Creek show considerably higher turbidities but because of their smaller discharges their total load would be smaller.

To estimate whether or not constant dredging of silt would be required after the lake has been constructed, the total suspended sediment inflow has been calculated from the flow duration curve (Fig. 21), assuming a constant concentration of 140 p.p.m. This amounts to 105,000 tons of silt per year. Microscopic examination of the sediment samples taken above show the silt to have an average grain size of about 0.02 mm. Because of its small size, some of the silt will be carried through the lake and not all of it will settle out. Using the theory of Camp (Reference 3) for settling basins, the trap efficiency of the lake has been estimated at 28 percent. The suspended sediment is therefore estimated to accumulate in the lake at a rate of 29,000 tons per year.

The quantity of sand to be dredged from the lake is greater than $10x10^6$ tons and consequently siltation over the next 100 years presents no problems.

6.7 Summary of Results of Study of Sedimentation

Approximate sediment inflows both from bed load and suspended sediment have been estimated. This shows that the quantities are very small compared with the volume of the lake and no siltation problems are anticipated in the foreseeable future.

7. Effect of Proposed Lake on Pollution

7.1 The Estuary Upstream and Downstream of the Proposed Lake

The problems of pollution in the upper Georges River have caused concern to many authorities. Most of the pools along the river banks have been closed to swimming because of the danger of infection. Despite this, there have been no systematic measurements of pollution under varying fresh water inflows and tidal conditions in order to evaluate the dilution potential of the estuary and its ability to discharge wastes which flow into the estuary.

The lake will benefit pollution considerably by increasing the tidal storage near the headwaters where it will do the most good.

After tidal equilibrium has been obtained, the additional tidal storage available within the lake will be approximately 57×10^6 cubic feet. This water will mix with pollutants which enter the estuary producing much greater dilution than occurs at present.

To supply the additional water stored in the lake, tidal discharges will be increased in the river channel downstream of the lake and the benefits of the lake will extend into this section with progressively smaller benefits with distance downstream. The percentage increase in tidal storage above East Hills is estimated as 75 pc. whilst above Milperra the percentage increase is 170 pc.

There will be no change in pollution levels upstream of the lake because it receives no benefit from the increased tidal storage.

7.2 The Lake

7.21 Introduction

One of the possible sources of pollution in the proposed lake would be the accumulation of surface debris as a result of continuous stagnant conditions during a tide cycle. A special model investigation was carried out to determine the pattern of tidal flow through the lake so that this problem could be evaluated.

7.22 The Model

The model was that used previously to study the effect of the construction of the lake on flood flows with some modifications. The model scales were 1:100 vertical and 1:900 horizontal giving rise to a time scale of 1:90 on a Froude model basis. Hence on the model a tide cycle should be completed in approximately 8 minutes. Tide flow was simulated by displacing water from a sump at the downstream end of the model by means of a 14 inch square piston. The piston was manually driven by means of a screw threaded crank handle and operated such that an approximate sinusoidal relationship existed between time and the flow rate into or out of the model. A labyrinth was constructed at the upstream end of the model to simulate the river reach to Liverpool weir, the limit of tidal influence.

For the previous study of the effects of the lake on flooding, a depth of water of 28 ft. in the lake was adopted. If the model was operated according to Froude similarity a 12.4 hour tidal period would be reproduced over 8 minutes in the model. At this time scale and with a 28 ft. water depth Reynolds Numbers for the model would be extremely low (between 200 and 400) and it is doubtful whether the model would accurately simulate prototype flows.

The main objective was to detect lake areas which would not be subject to tidal flushing rather than quantitative estimates of tidal velocities and flushing times. It was therefore possible to carry out qualitative tests on tidal flow patterns by making the following adjustments to the model:

(i) The lake was shallowed and the bed overlain with 3/4 inch sharp edged blue metal to promote turbulence throughout the lake model. The resulting mean depth was about 13 feet prototype.

(ii) The time scale was distorted. The model was run at model tide periods of 4 minutes and 2 minutes, instead of the 8 minutes calculated from the Froude criterion. The above modifications resulted in model Reynolds Numbers of about 1000 for the 4 minute cycle and about 2000 for the 2 minute cycle. These values of Reynolds Numbers and the sharp edged gravel bed ensured that turbulent conditions prevailed in the model. The two different tide periods were used to determine if the model flow patterns were sensitive to distortion of the time scale. If flow patterns for both were identical it can be reasonably assumed that the flow pattern for the 8 minute period (12.4 hours prototype) would also be similar.

7.23 Test Results

Photographic techniques were employed to record flow patterns on the model. Results were recorded only after several tide cycles had been passed through the model to obtain equilibrium conditions.

(i) Still Photography

Five second time exposure recorded confetti streak patterns on the water surface for both the 2 minute and 4 minute tide periods. Ebb and flood tides were photographed at mid tide. The patterns so recorded were identicial for the 2 minute and 4 minute tide periods. Figures 23 and 24 show typical examples for the 4 minute period. The length of the streak lines in the photographs indicate the relative velocity magnitudes.

(ii) Movie Photography

Movie film of the 4 minute tide cycle was taken at 12 frames per second and of the 2 minute cycle at 24 frames per second. When projected both cycles appear to have the same period, allowing easy comparison. The film shows that the flow patterns are identical.

7.24 Discussion of Results

The flow patterns for the two tide periods tested were identical and therefore insensitive to distortion of the time scale. Consequently Figures 23 and 24 can therefore be taken as indicative of the flow patterns that will occur in the lake for a 12.4 hour tidal cycle. The following points are noted:-

(i) On a flood tide, floating debris could be trapped in the central regions of eddies at A and B in Figure 23a. Such debris would, however, be flushed from these regions on the ebb tide.

(ii) For both ebb and flood tides, low velocities occur in the hatched regions shown in Figures 23a and 24a. Here, however, the direction of flow is non reversible, being the same for both ebb and flood tides, allowing floating debris to eventually complete its circuit into swifter flowing currents.

(iii) In Figure 25 the pattern of surface currents through the lake under flood conditions is shown. These are significantly different to the tidal patterns (as shown in Figures 23 and 24) because of the large tidal storage provided by the lake. During a flood, floating debris may tend to accumulate in the reverse eddy regions D, E and F. However, as these regions are in areas of strong tidal currents they will be adequately cleared after the flood recedes.

In brief, model tests show that adequate tidal flushing will occur. There are no stagnant areas within the lake and floating debris will gradually travel downstream and out to sea.

8. Effect of the Proposed Lake on Bank and Channel Stability

8.1 Introduction

The previous sections have discussed the effect of the construction of the lake on all aspects of river behaviour. This section considers the following questions:-

(i) How changes in river behaviour may affect scouring and bank erosion in the estuary upstream and downstream of the lake.

(ii) To what profile should the lake banks be constructed to ensure permanent stability, especially in relation to protecting the boundaries of adjoining properties.

8.2 The Estuary Upstream and Downstream of the Proposed Lake

8.21 Effect of Floods on Scour and Bank Erosion

The lake will appreciably reduce flood levels near Warwick Farm racecourse (see Section 4) and flood gradients will be increased in the river immediately upstream of the lake and in Cabramatta Creek. The higher velocities may be expected to cause some local scour of the channel bed during periods of flooding, and local bank erosion may result. The river in this section is well confined between banks and it is unlikely that any serious damage to property will occur.

However, the sand in the bed of the river is a valuable asset, and to avoid any possibility of bank erosion in this location, it is obviously wise to license sand extraction operators for dredging where necessary in the reach above the entrance to the lake, in order to provide the necessary channel capacity for flood discharges so that the velocity of flow will be no greater than at present occurring during flood. Further investigation of the soil properties in the channel and bed would be required before the extent of necessary dredging could be specified.

8.22 Effect of Tidal Flow on Scour and Bank Erosion and Necessary Precautions to Prevent Bank Erosion

The increased tidal storage of the lake will increase tidal velocities in the downstream river channel as discussed in Section 5. To prevent scour of the bed the cross-sectional areas of the channel must be increased to those shown in Column 5 of Table III in Section 5.3. Over recent years sand dredging operations have already achieved this at many locations. Fig. 12 compares the present depth with that required after completion of the lake scheme. This figure shows that there are three areas which at present do not have sufficient channel capacity. These are:-

(i) The reach between Prospect Creek and Milperra.
(ii) A reach at the junction of Williams Creek and Georges River.
(iii) A reach at Sandy Point where a submerged cable crossing the river is located.

It is likely that some bank erosion will occur if scour of the river channel is allowed to develop naturally in these localities. To avoid problems from this cause it is recommended that remedial measures shown in Fig. 15 and discussed below be undertaken in conjunction with the construction of the lake.

(a) Milperra Region

As a number of homes are built along the edge of the river in this region, bank erosion and channel widening must be controlled. Widening of the bank in the areas shown in Fig. 15, in conjunction with sand dredging, is suggested as a satisfactory solution.

Downstream of Milperra Bridge, the channel capacity for tidal flows appears to be adequate (see Fig. 12). However, it is possible that the increase in tidal flow could cause local increases in velocity due to the complex flow patterns associated with the bridge and the sharp S-bend in the river in this locality. This may increase bank erosion requiring additional local protection. Further investigation is necessary before exact requirements can be determined. The homes on both sides of the river in this area are generally protected against bank erosion by rock or brick retaining walls. Many of these are, however, in bad condition and show signs of collapse.

(b) Williams Creek

Since deepening would be required to reduce the tidal velocities at this location, it is likely that some bank erosion may occur. As the right bank is stabilized by mangrove swamp, scour would probably be concentrated on the left bank in the vicinity of the Deepwater Motor Boat Club. To avoid this, it is recommended that the river be widened on the right bank in conjunction with dredging of the bed.

(c) Sandy Point

At this location the river is contained within sandstone foreshores and bank erosion is not a problem. However, scour of the bed could leave suspended the submarine cable which crosses the river at this point. To avoid this problem, it is recommended that the cable be relocated just upstream where dredging operations have already enlarged the river to a sufficient capacity.

8.23 Summary

Construction of the proposed lake will result in:-

(i) An increase in flood gradients and velocities in the river channel upstream of the lake:

(ii) An increase in tidal velocities in the river channel downstream of the lake.

As a result of this change the bed will scour and some bank erosion may occur. To avoid problems arising from this erosion, it is suggested that the river channel be dredged and widened at the critical locations shown in Fig. 15.

8.3 Stability and Protection of Bank Slopes in Dredged Areas

8.31 General

At present there appears to be a blanket restriction imposed by Council which states that dredges may not operate closer than 100 feet to any road or property boundary. It is assumed that this regulation does not refer to the distance between the property boundary and the water-land boundary, which would be considerably less, because of the soil slopes between the location of the suction intake to the dredge and the water surface.

A blanket restriction as above is obviously only applicable to a fixed set of the following characteristics:-

(i) Soil type
(ii) Dredging depth
(iii) Relative levels of the ground surface and the water surface.

Where dredging depths, soil types and ground levels vary the allowable distance will also change.

The investigations described in this section refer particularly to the landward boundary of zone 2, the zone planned to be mined by Hollywood Sands Pty. Ltd. However, the findings will be applicable to any of the proposed lake boundaries provided the soil consists predominantly of non cohesive sands.

8.32 Soil Types

Soil types near the landward boundary of zone 2 were determined by the following means:-

(i) The inspection of two open pits by the authors. Depth of the pits was 13 feet.

(ii) The inspection of existing dredging operations at the North West End of zone 2.

(iii) Information given to the authors by Hollywood Sands Pty. Ltd. on soil types to 30 feet depth.

It is apparent that the soil type is not uniform over the whole area. The top soil exists to a depth of 8 to 10 feet and varies from a non cohesive granular soil in some locations to sandy loams in others. Below the top soil the materials consist almost entirely of non cohesive sands. However, there are some clay seams of high plasticity which traverse these sands.

Because of the predominance of sand in the area the studies have assumed the banks to consist of granular materials.

8.33 Analytical Studies of Stable Bank Slopes

This phase of the investigation was carried out using circular arc failure analysis.

(i) No Seepage Case

For banks of sandy materials, with no seepage occurring, slopes of 33° to the horizontal (1 on 1.54) would be just stable (see Figure 26). This is the angle at which stock piles of sand would normally stand. However, seepage often occurs, particularly in the banks of estuaries where water level variations due to tides occur. Therefore for design purposes seepage should be allowed for by circular arc failure analysis.

(ii) Seepage Case

Seepage results in saturation of the soil above the river or lake water level, increasing both the soil weight and neutral* pressures. Both reduce the stability of the slope. The ground surface along the road boundary of zone 2 (Fig. 2) is 13 feet above Indian Spring Low Water+ (I.S.L.W.), that is about RL. 10.5. The most severe seepage condition would occur after the subsidence of a flood which had inundated over bank areas. Above I.S.L.W. there would be 13 feet of saturated soil with seepage occurring as in Fig. 27a. A 33° bank slope would be unstable. Circular arc analysis shows that a cross section as shown in Figure 27b would be stable with 27° slope (1 on 1.96) below water and an 18° slope (1 on 3.08) above water.

8.34 Field Observation of Bank Slopes

Field inspections have been made of bank slopes in the following locations:-

(i) Above Water Slopes of Natural River Banks

These stand at average slopes between water level and natural

* Water pressure in the soil between the grains of soil.

\+ Water level below which the tide seldom falls.

surface of up to 40°. Except where supported by large trees these banks are not considered permanently stable under seepage conditions that would result from severe flooding.

(ii) Artificially Sloped Banks

Artificially shaped banks at Milperra Road Bridge slope at 20° above water level. They have experienced moderate flooding and appear quite stable.

(iii) Under Water Slopes in Freshly Dredged Areas

At the present workings of Hollywood Sands Pty. Ltd. these stand at about 34° (depth 30 to 40 feet). This is close to the 33° angle of repose estimated from stock piles. These slopes have not been subject to seepage conditions resulting from severe floods.

In addition to the above observations it must be remembered that the circular arc analysis of Section 8.33 assumed a water surface in the soil corresponding to ground level. It is doubtful if in practice this condition would occur in highly permeable sands. Flood recessions would be slow enough to allow the sand masses to drain in such a fashion that the water table at the bank slopes would be below ground level. The water table would then grade towards the overbank ground surface as shown in Figure 28. .

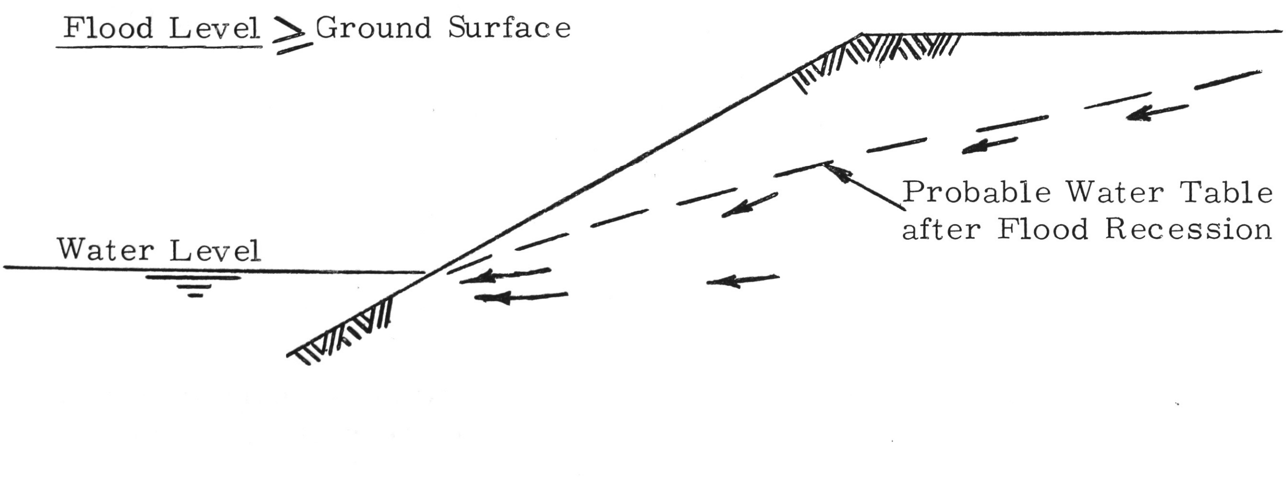

Fig. 28.

The foregoing observations indicate that the estimates of stable slopes derived analytically in Section 8.43 are reasonable and probably a little conservative.

8.35 Bank Protection from Tidal and Flood Current Scour

Flood velocities are estimated at about 0.25 f.p.s. through the lake. Tidal currents will be even slower. The magnitude of these are too small to disturb the bank material and no special protection is required for this aspect.

8.36 Protection from Wave Action

Wave conditions at the lake boundaries have been estimated by the procedures outlined by the U.S.A. Beach Erosion Board for estimating waves in inland reservoirs. The effective fetch length over which the wind adds energy to waves, varies with location along the lake boundary. Typical values are approximately 2000 ft. at the inlet and outlet of the lake and 3000 ft. towards the centre. Wave conditions for various wind speeds for these fetch lengths are shown in Table V.

Table V: Wave Conditions (Significant) at Lake Boundaries for Various Wind Speeds.

Wind Speed mph	Min. Wind Duration in minutes for Max. Wave Conditions		Significant Wave Height (ft.)		Significant Wave Period (secs.)	
	2000 ft. Fetch	3000 ft. Fetch	2000 ft. Fetch	3000 ft. Fetch	2000 ft. Fetch	3000 ft. Fetch
10	17	23	0.25	0.30	1.0	1.2
20	13	17	0.55	0.65	1.4	1.6
30	11	14	0.80	0.95	1.7	1.9
40	9	12	1.10	1.30	1.9	2.1
50	8.5	11	1.40	1.65	2.1	2.4
60	8	10	1.70	2.00	2.3	2.6
70	7	9	2.00	2.40	2.5	2.7

While the slopes of Figure 27b are stable against seepage they would be unstable against wave action. The following alternatives for wave protection exist:-

(i) Construction of a flat beach slope (Figure 29)
(ii) Provision of rip rap protection (Figure 29).

8.361 Beaching Slopes

A beaching slope between 1 in 10 and 1 in 15 to a depth of 6 feet below low water would be required as this is typical of beach sections in protected areas of Sydney Harbour which have similar wave conditions to those expected on the lake. Below this depth a 30 degree slope angle could be used (see Figure 29b). These beaches would provide a desirable recreational facility along the lake boundary while the presence of waves will help keep them free of silt.

8.362 Rip Rap Protection

Table 5 shows that significant wave heights of between 2 and 2.4 feet will occur on the proposed lake. The bank surface of Figure 27b can be protected as shown in Figure 29a, the rip rap extending from 6 feet below low water to 4 feet above high water.

A suitable grading for stone rip rap protection is given in Table 6. Angular quarry rock is to be preferred. Rounded river gravel should be avoided. The grading shown is not rigid providing the sizes in the D75 to D50 range are maintained. "Run of Quarry" material would probably be suitable. The rip rap need not be hand placed but should be dumped and smoothed by adjusting the rocks to obtain a stable mass with no unfilled voids.

It is necessary to underlay the rip rap by a bed of finer gravel or filter layer. This prevents the relatively finer natural bank material from being washed from under the rip rap causing the collapse of the slope.

Provided a reasonably well graded rip rap is used only one filter layer is required. This would be a 6 inch filter of the grading shown in Table 6. Again this grading is not rigid providing the D75 to D50 range is maintained.

The level of RL. 7.5 recommended for the top elevation of the rip rap protection (Figure 29) is based on considerations of wave run-up, river level changes from minor freshes, speed boat activity etc. However, where the water is predominantly fresh, a lower elevation, for the rock with grass protected banks above this level may be used. This elevation is the limit to which grass will satisfactorily grow under the prevailing conditions.

Table VI

Layer	D_{100}*	D_{85}	D_{75}	D_{50}	D_{15}	D_o	Minimum Thickness
Rip Rap	15"	12"	10"	8"	5"	3"	15"
Filter Bed	4"	3"	2"	1"	3/16"	1/8"	6"

*Example D_{75} = 10" means that 75 pc. by weight of rock must pass a 10" square sieve.

8.37 Summary

From the studies described in this report it is recommended that banks bordering the sand dredging areas conform to the cross-sections shown in Figure 30. These sections will be stable provided the soils are generally sandy or sandy loams in nature. Should regions of clay or compressible soils be encountered, special treatment based on more complete soil surveys may be required. These would need to be investigated as they are encountered.

Based on the section shown in Figure 30 the safe dredging distance from property boundaries, as related to the depth of dredging can be computed. This is shown in Figure 30c and refers to the distance between the suction intake to the dredge and the boundary line. The land-water boundary as a result of the sand slope will of course be considerably closer. This diagram indicates that for a distance of 100 feet between the dredge and the boundary the dredging depth should not exceed 25 ft. for natural ground surface at RL. 15.5 or 33 ft. for natural ground surface at RL. 10.5.

Loading of the soil surface adjacent to the top of bank such as by houses or roads should not be allowed within 20 feet of the top of the bank. Where such conditions are likely the distances shown in Figure 30c should be increased by this amount.

9. Possible Construction of a Weir

The investigation described in this report has concentrated on a lake which is allowed to develop its full tidal capacity. Such a proposal has the advantages that it reduces flood damage, reduces pollution and improves navigation. It has the disadvantages that problems from scour of the river channels will require some dredging and bank control works to be carried out in conjunction with the construction of the lake.

An alternative proposal to the above would be to provide a weir or control at the downstream end of the lake which partially or fully excludes the tide. This would eliminate the bank protection and channel improvement works required downstream. However, the benefits which the lake has to flooding, navigation and pollution would then be reduced and consequently this proposal is not favoured.

10. Proportion of Volume of Lake in Fairfield and Liverpool Municipalities

The major portion of the volume of the lake is in the Liverpool Municipality, shown as Zone 1 in Fig. 1. Excavation for sand has proceeded so far in this region (see Photograph 1) that it seems to the authors that it is too late to cancel Liverpool Council's intention to construct a lake, even if such cancellation were wise and desirable. The volume of water in Zone 2, in which Hollywood Sands are interested, and which is in Fairfield Municipality, is only 96 x 10^6 cubic feet, as compared with 279 x 10^6 cubic feet for Zone 1.

11. Conclusions

11.1 General

The proposed lake is a desirable public amenity which will bring to the community considerable benefits in recreation, flood mitigation, provision of much needed sand for the development of Sydney, and mitigation of pollution but it would be wise to take the precautions recommended in Sections 8.2 and 8.3 of the report.

11.2 Flood Mitigation

(a) The construction of the lake will reduce materially flood damage in the vicinity of the lake and upstream for some distance.

(b) If the action recommended in Section 8.22 of the report is carried, the construction of the lake will reduce flood damage downstream of the lake.

11.3 Tidal Flow

(a) Unless the action recommended in Section 8.22 of the report is taken, it is expected that there will be a tendency to scour the bottom and sides of the river channel at the locations shown in Fig. 15 of the report.

(b) Until this action is taken, there will be a reduction in tidal range at Chipping Norton, which at present is the same as that of the ocean, but lagging $2\frac{1}{2}$ hours.

11.4 Sedimentation

The estimated volumes of bed load and suspended sediment which will settle in the lake are so small in relation to the volume of the lake that no siltation problems are anticipated in the foreseeable future.

11.5 Pollution

The lake will mitigate pollution effects by increasing tidal storage near the headwaters, where the increased dilution will have the maximum effect. Further, there are no stagnant areas within the lake thus preventing the accumulation of pockets of debris.

11.6 Bank and Channel Stability

(a) The construction of the lake will have no effect on river banks, except at the locations shown in Fig. 15 and perhaps in the small section referred to in Section 8.21.

(b) Provided it is specified that the banks of the lake be constructed to the specifications outlined in Figures 29 and 30 the banks will be stable against seepage and no erosion will result from wave action or floods.

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3. H. Rouse (ed.) "Engineering Hydraulics" Wiley and Sons, 1950.

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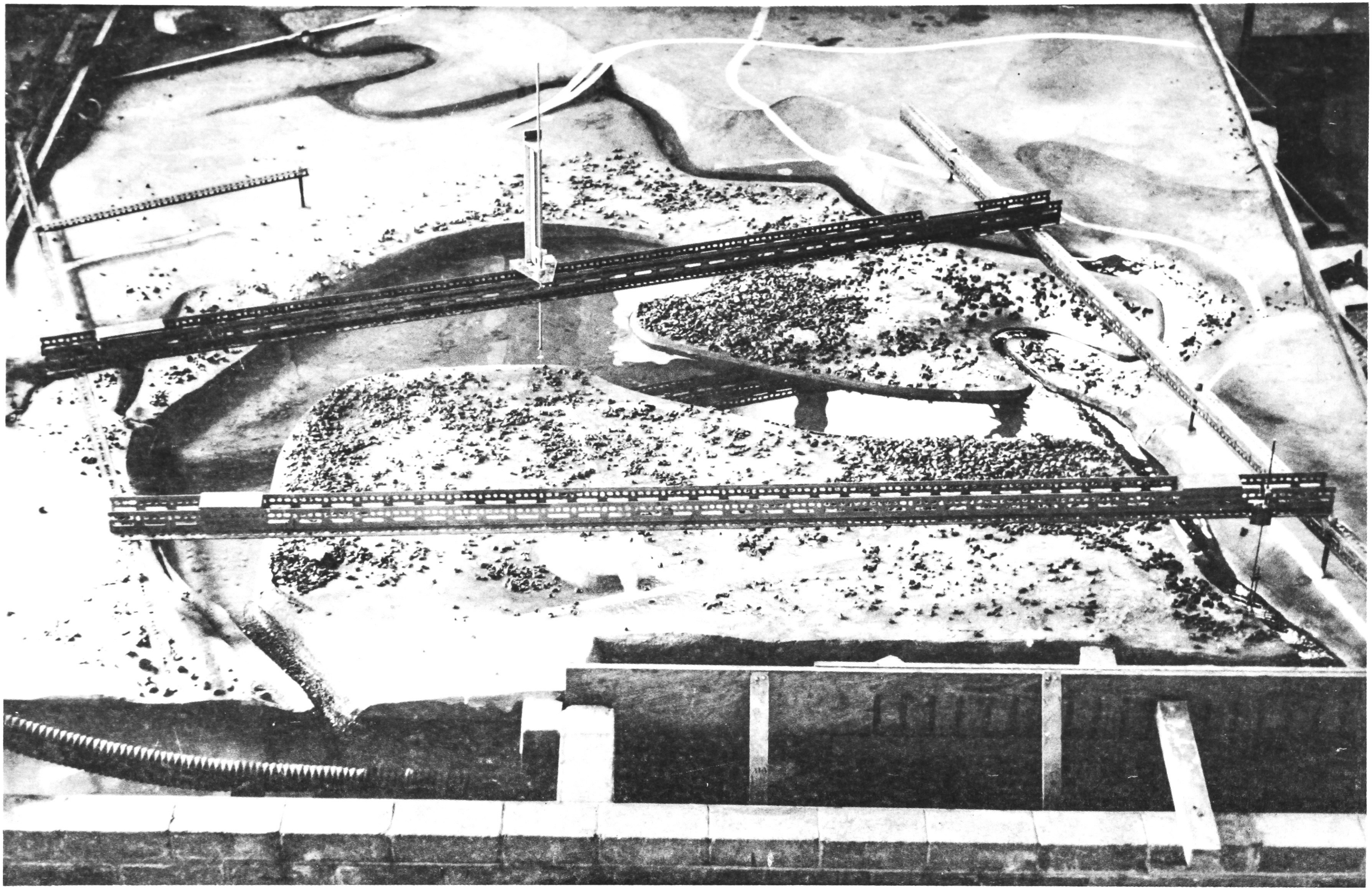
Acknowledgements

The authors wish to thank Associate Professor R. T. Hattersley and the staff of the Water Research Laboratory for the help given by them during the course of this work. In particular, thanks are due to Messrs. A. Bonham and R. French who are project engineers responsible for much of the work which has been recorded here.

Photograph I: Aerial photo of Georges River and Prospect Creek (flow is from top to bottom).

(by courtesy Dept. Lands, Sydney)



Photograph 2: Hydraulic model showing proposed lake.

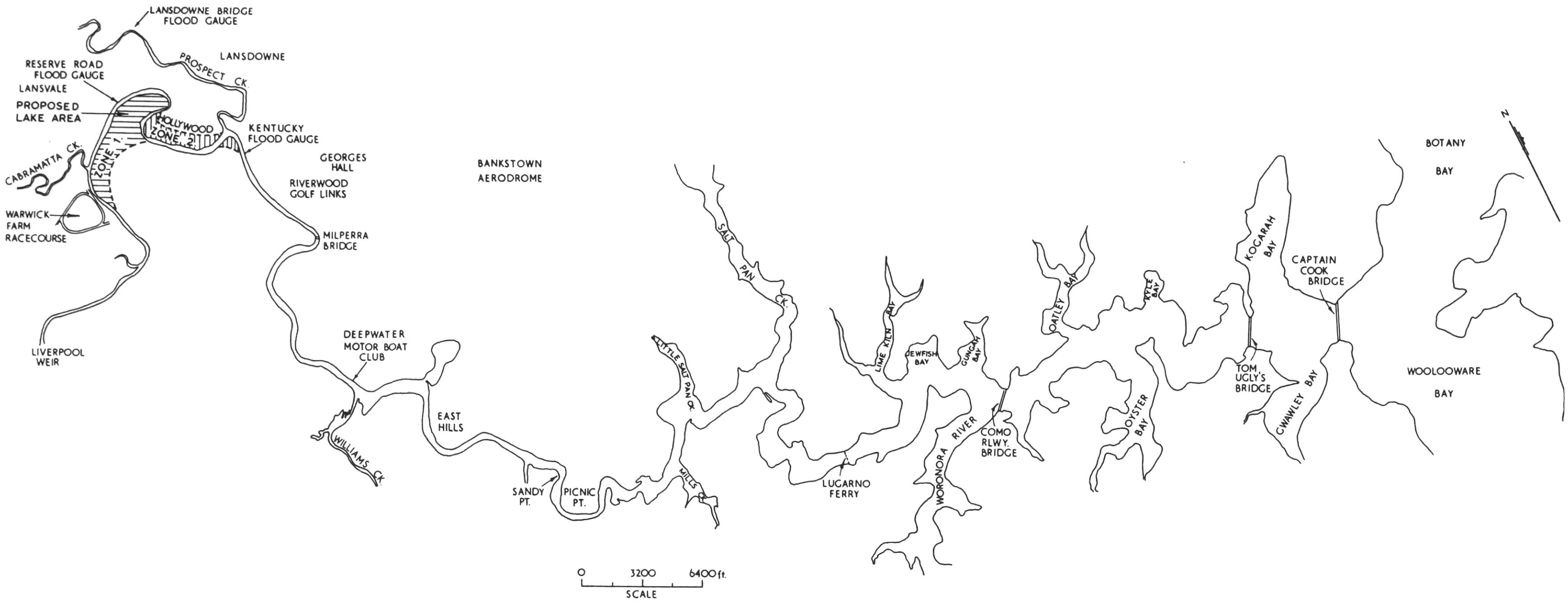


FIGURE 1: GENERAL PLAN OF GEORGES RIVER ESTUARY

CE-C-7177

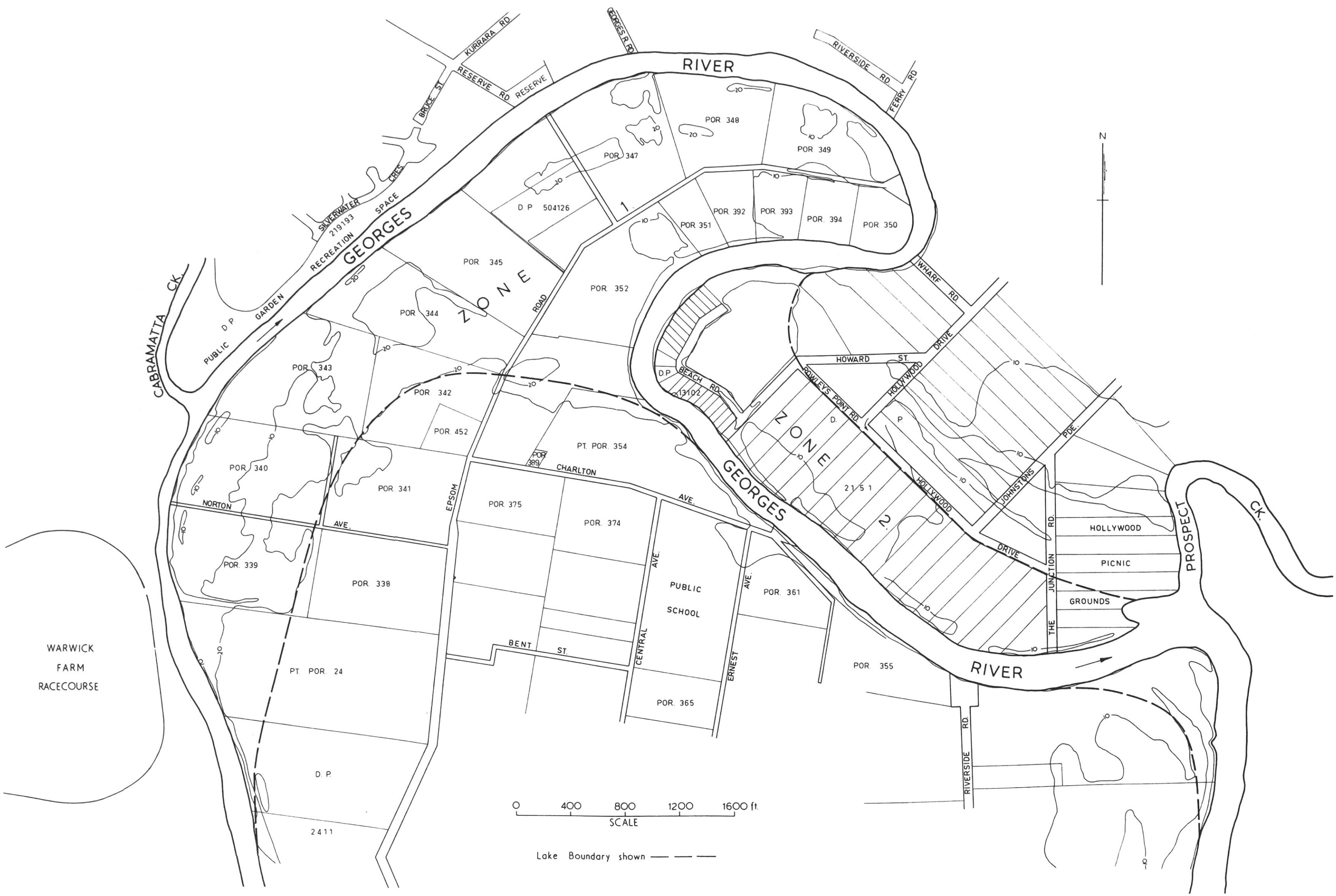
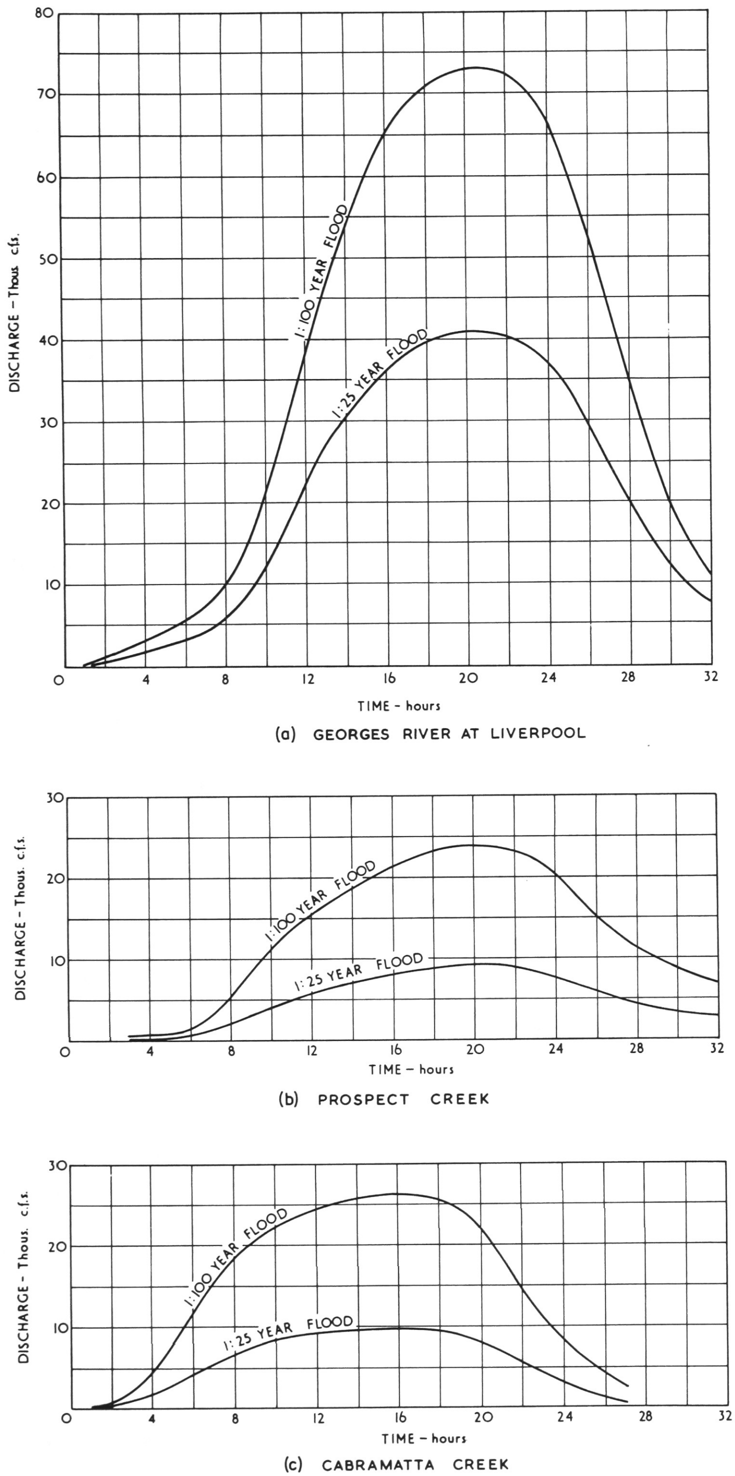


Fig. 2: Plan of Proposed Lake.



NOTE: 1" OF RAINFALL PER HOUR FOR 16 HOURS

FIGURE 3: INFLOW HYDROGRAPHS USED FOR MODEL STUDY

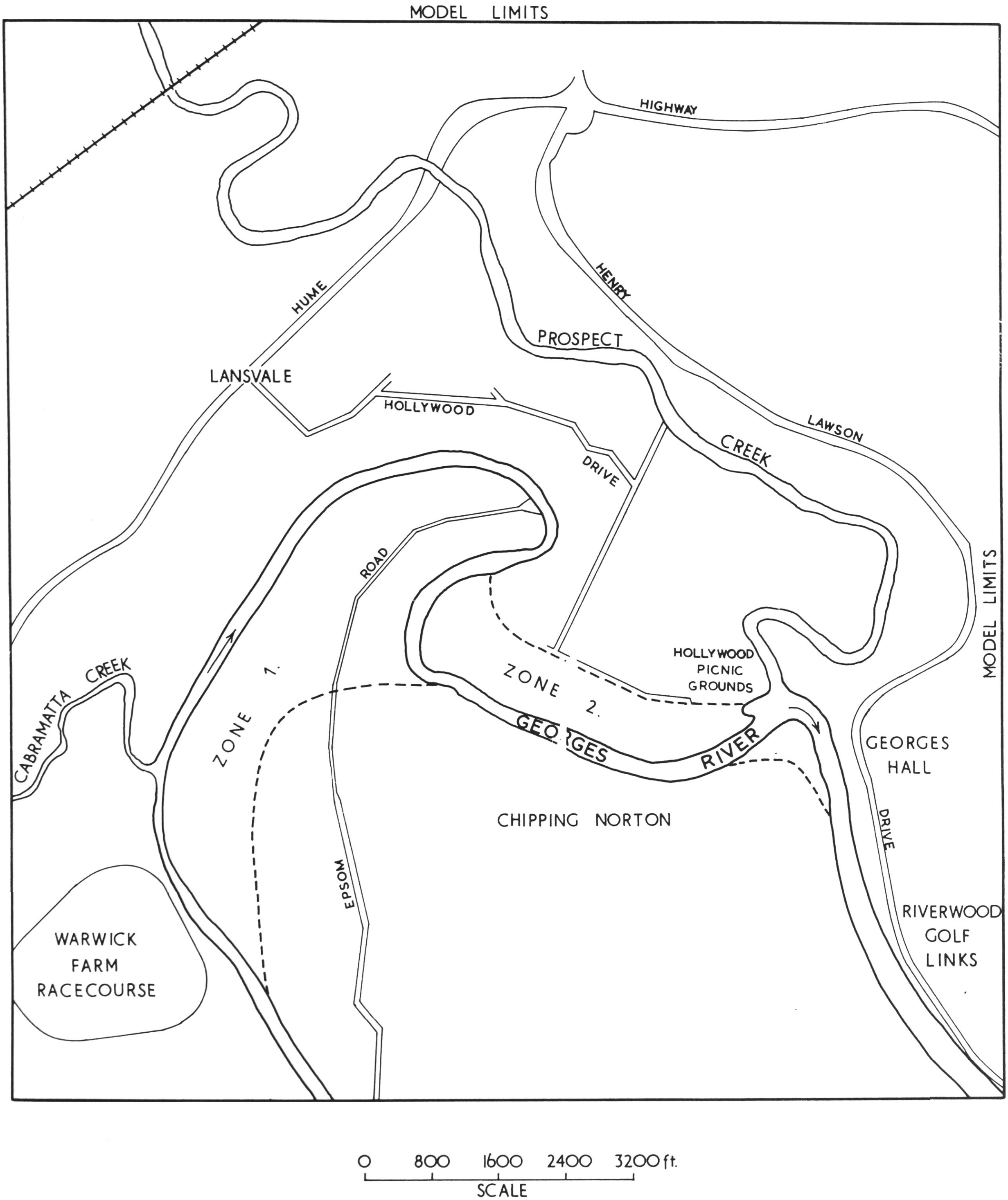


Fig. 4: Extent of the Model.

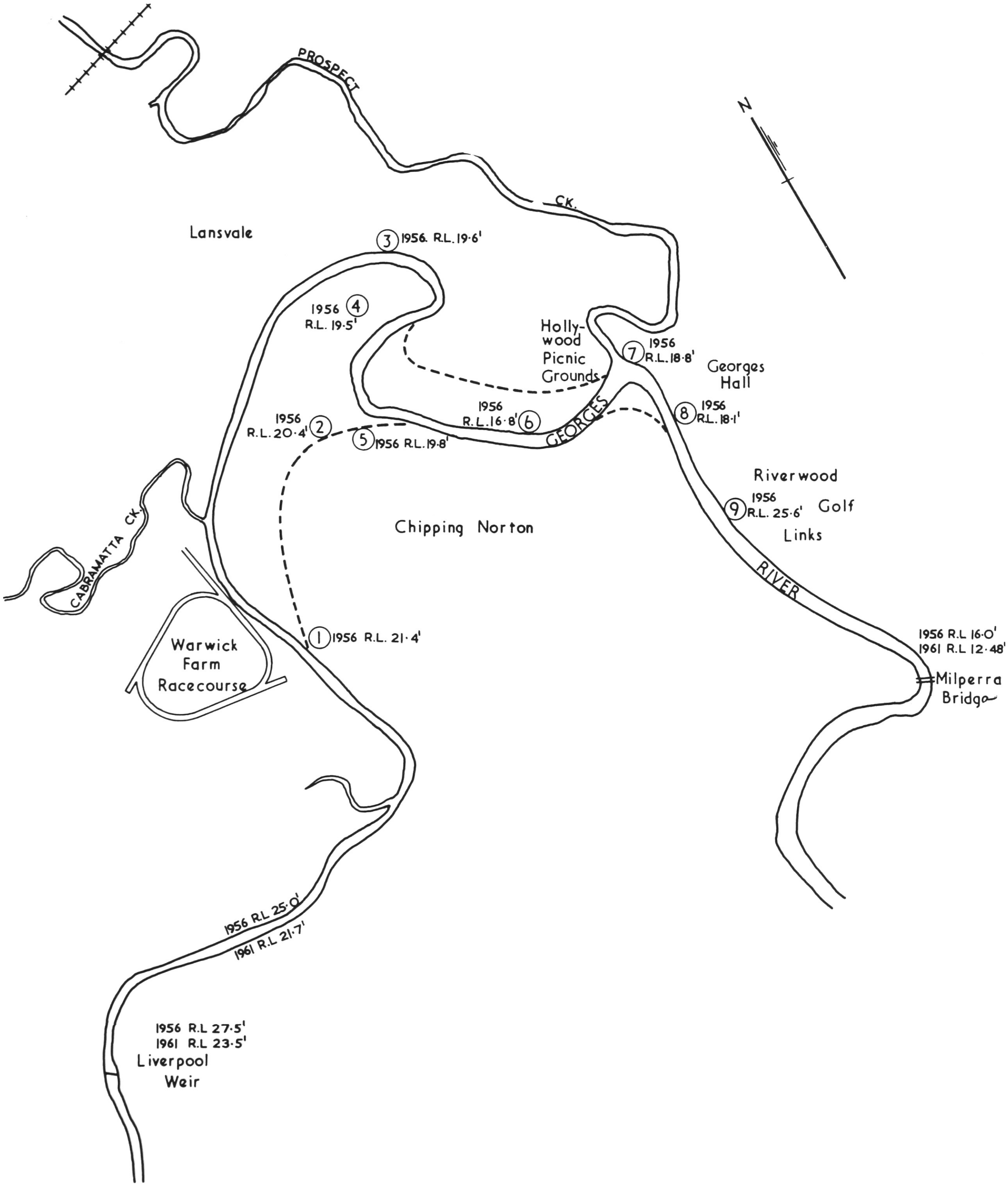


FIGURE 5: FLOOD LEVELS FOR 1956 FLOOD

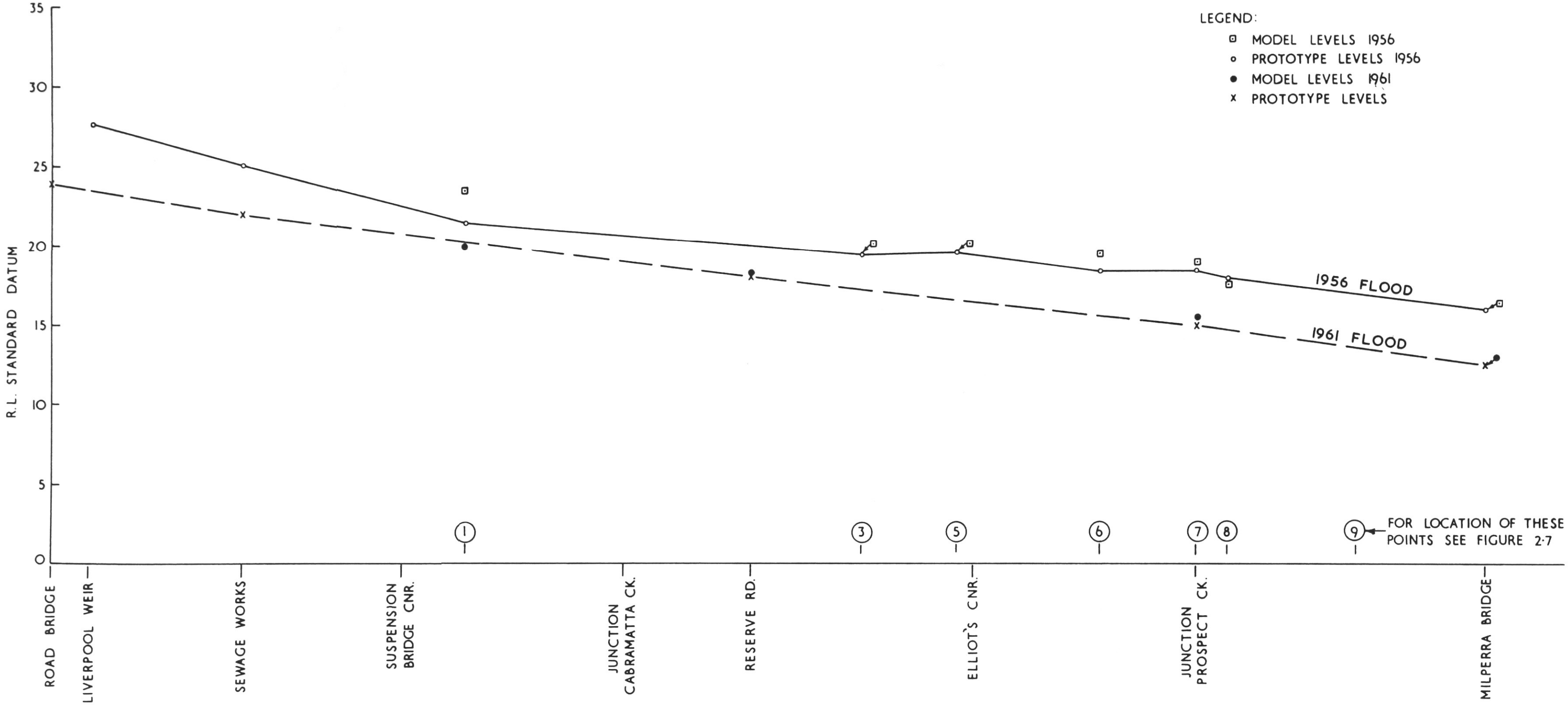


FIGURE 6: FLOOD LEVELS FOR GEORGES RIVER FOR 1956 & 1961 FLOODS — PROTOTYPE AND MODEL

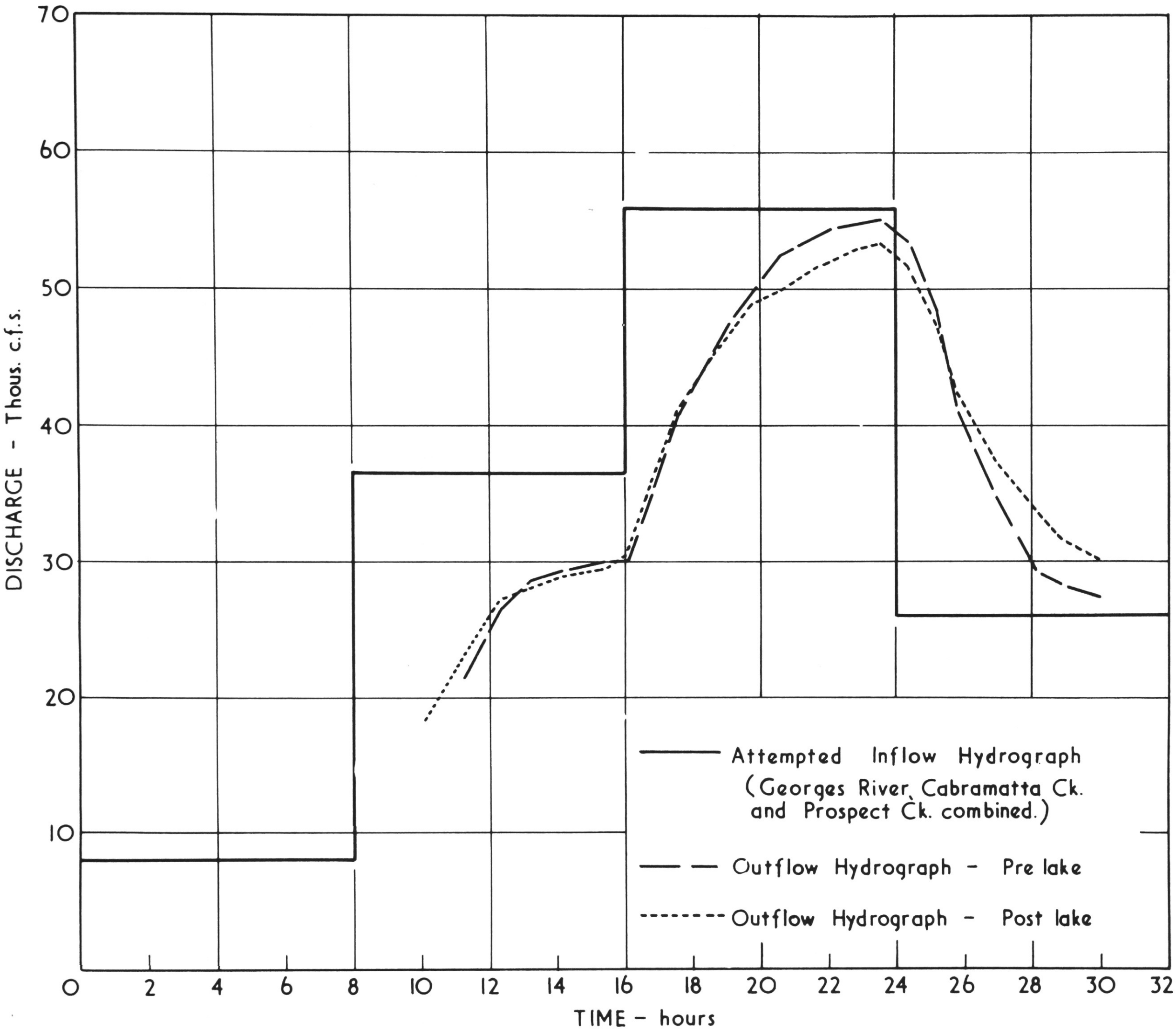


FIGURE 7: APPROX. INLET AND MODEL OUTLET HYDROGRAPHS FOR 1/25 YEAR FLOOD

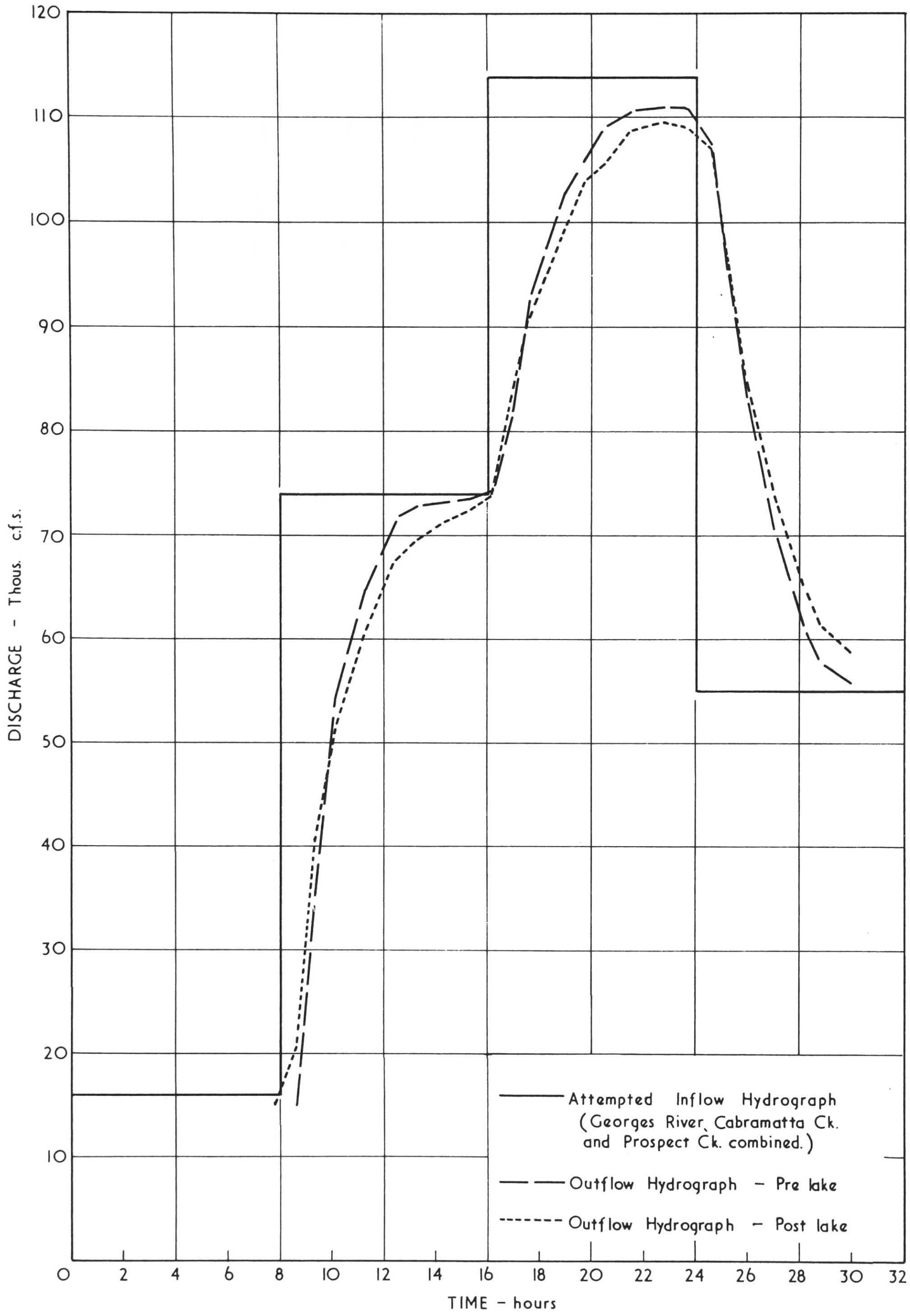


FIGURE 8: APPROX. INLET AND MODEL OUTLET HYDROGRAPHS FOR 1/100 YEAR FLOOD

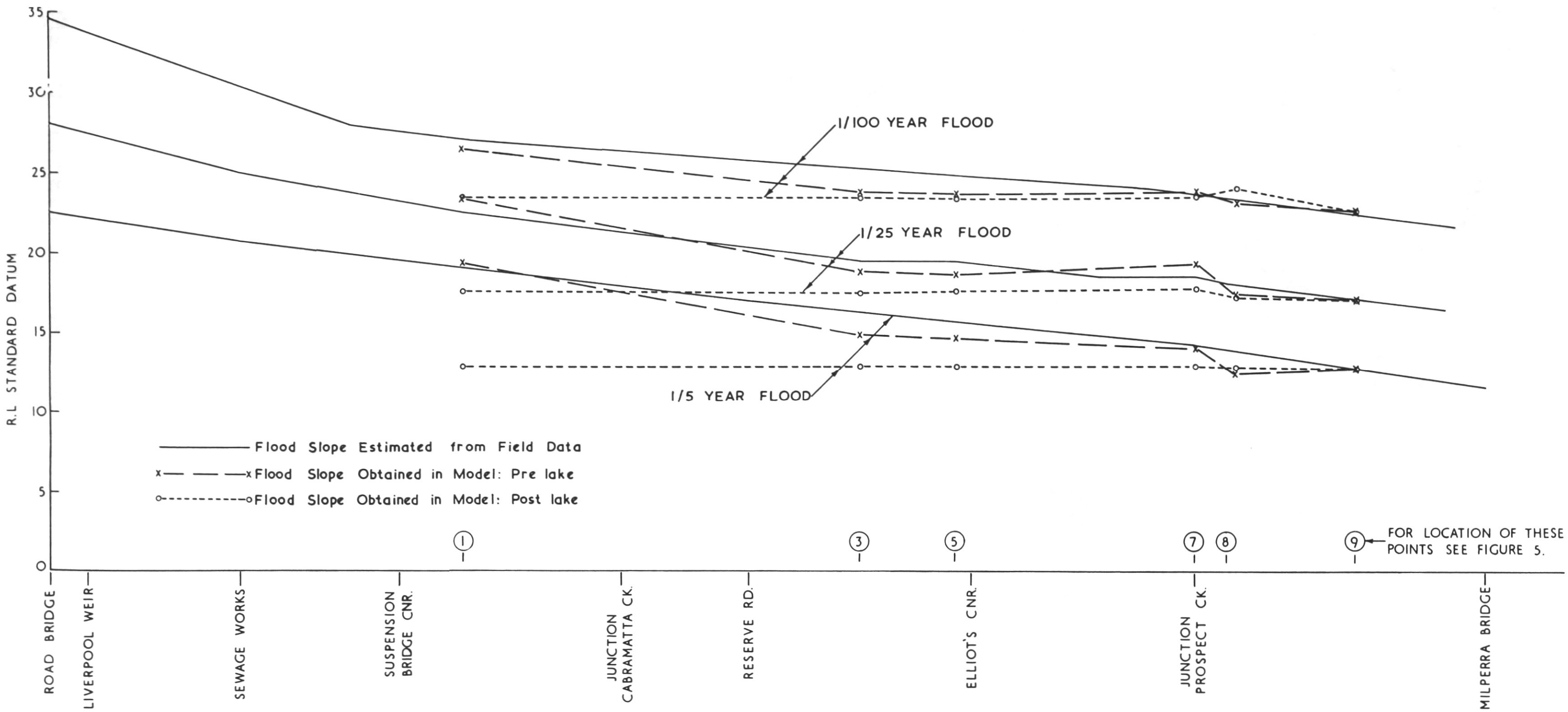


FIGURE 9: FLOOD LEVELS FOR BOTH PRE-LAKE AND POST-LAKE CONDITIONS

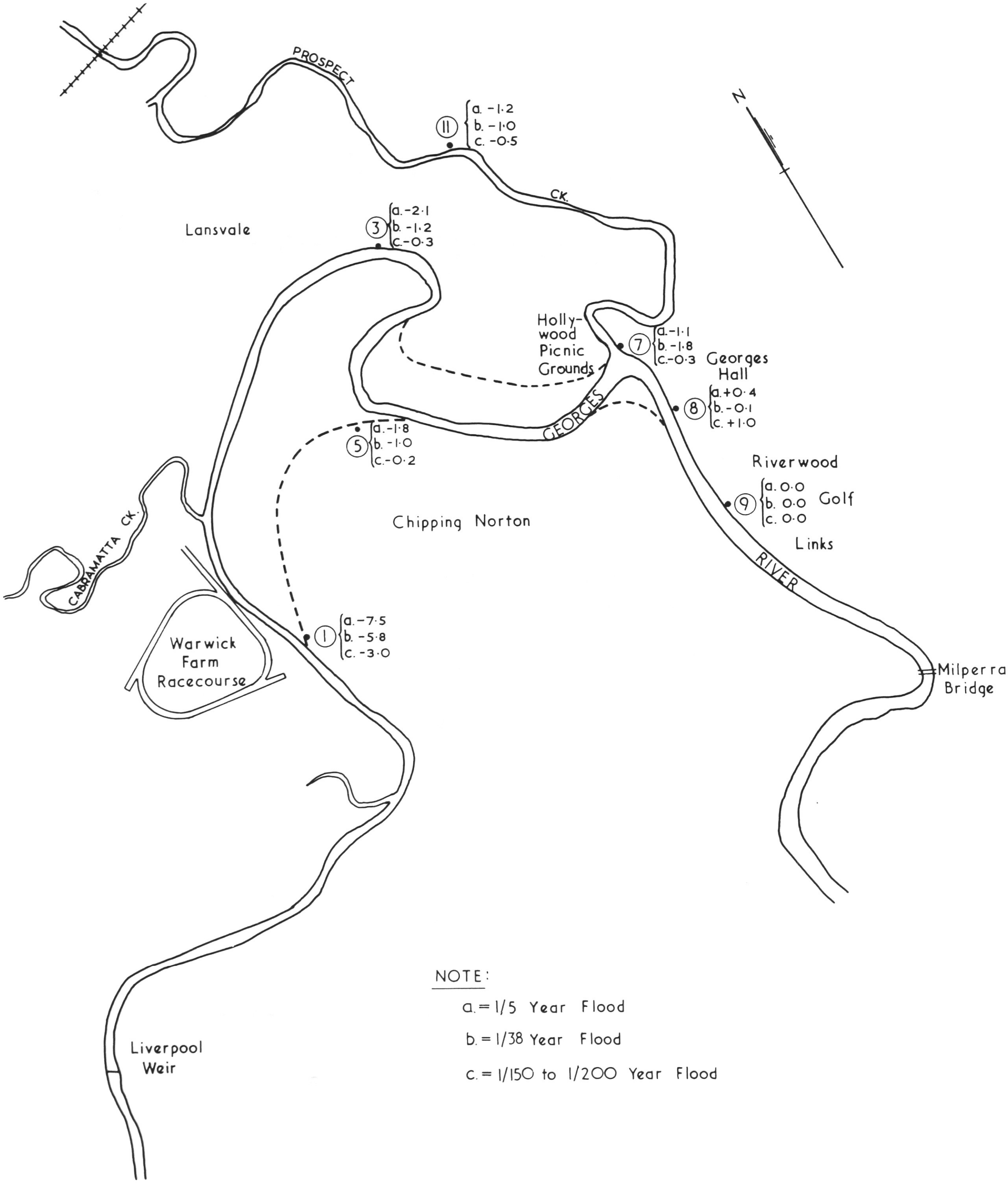


FIGURE 10: EFFECT OF LAKE ON FLOOD LEVELS

(FROM REFERENCE 11.)

CE-D-7236

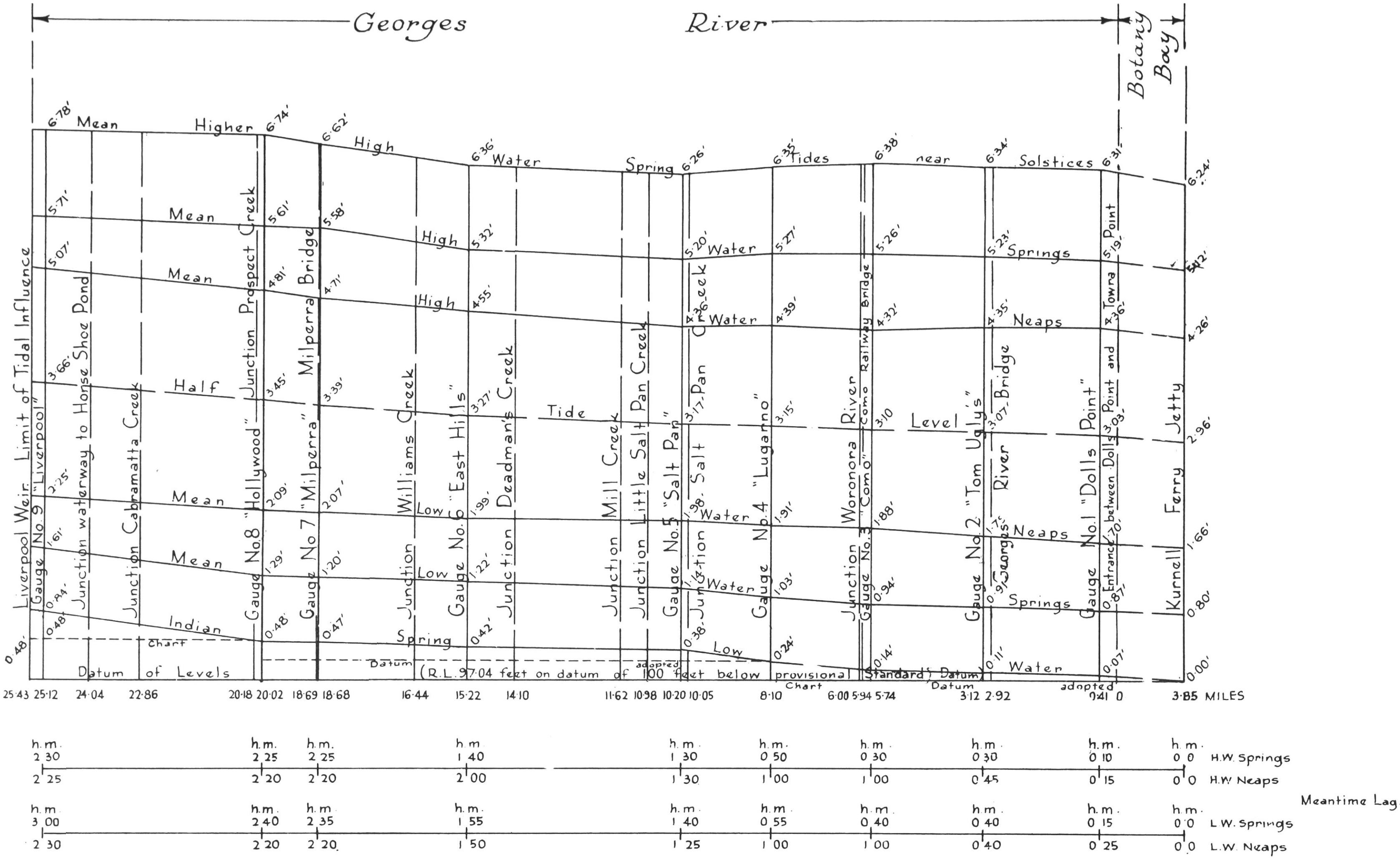


Fig. 11: Georges River Tidal Data.

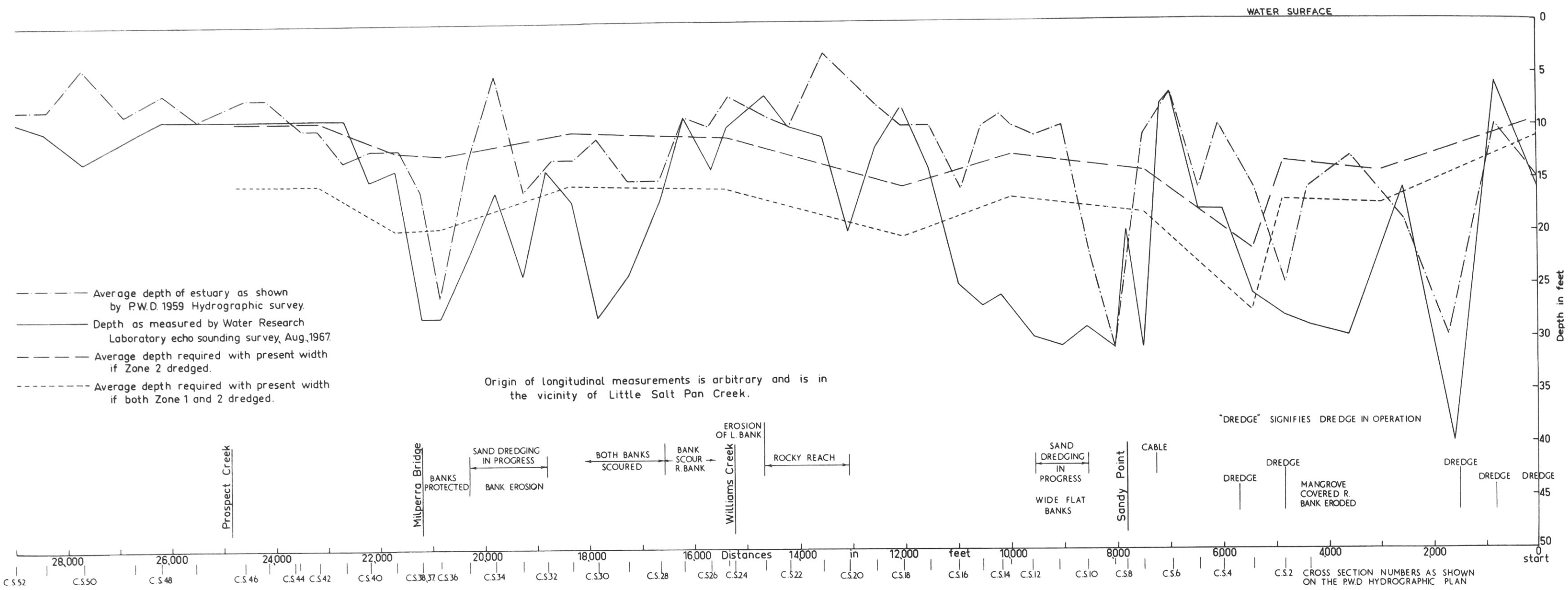


FIGURE 12: GEORGES RIVER LONGITUDINAL SECTION

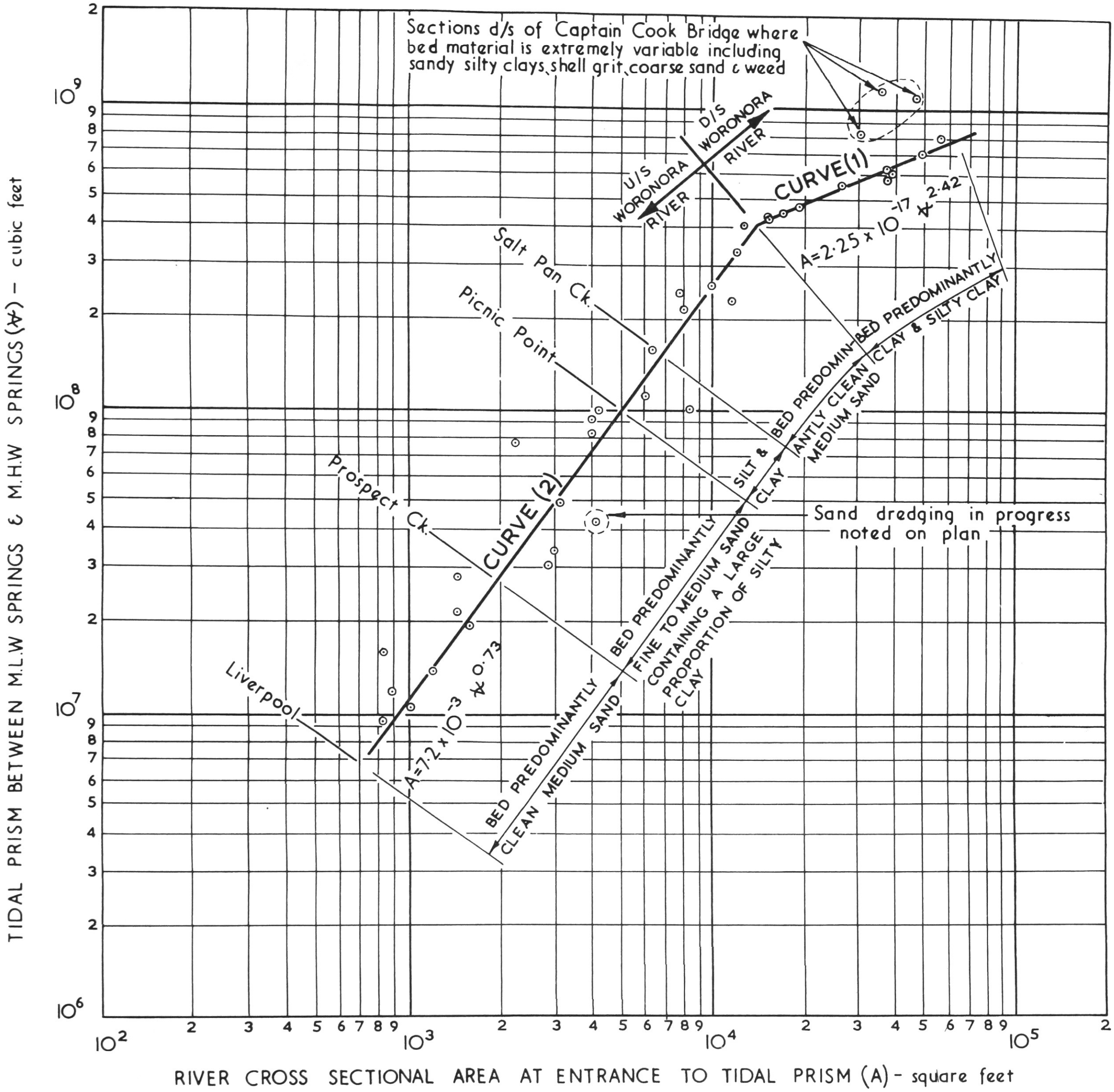


FIGURE 13: GEORGES RIVER TIDAL PRISM VERSUS CROSS SECTION AREA

CE-D-7173

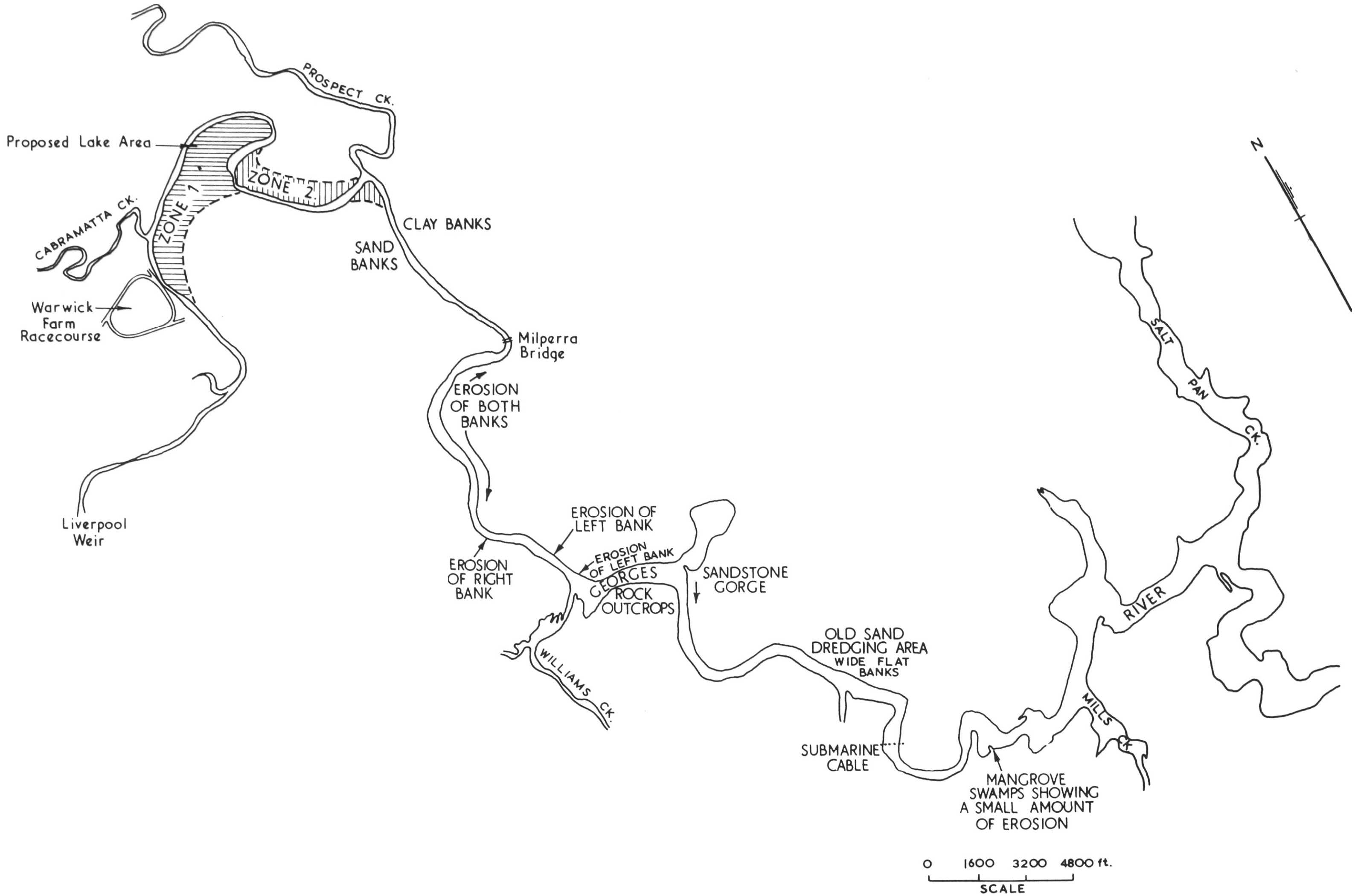


FIGURE 14: GEORGES RIVER: LOCATION OF EXISTING EROSION

CE-D-7175

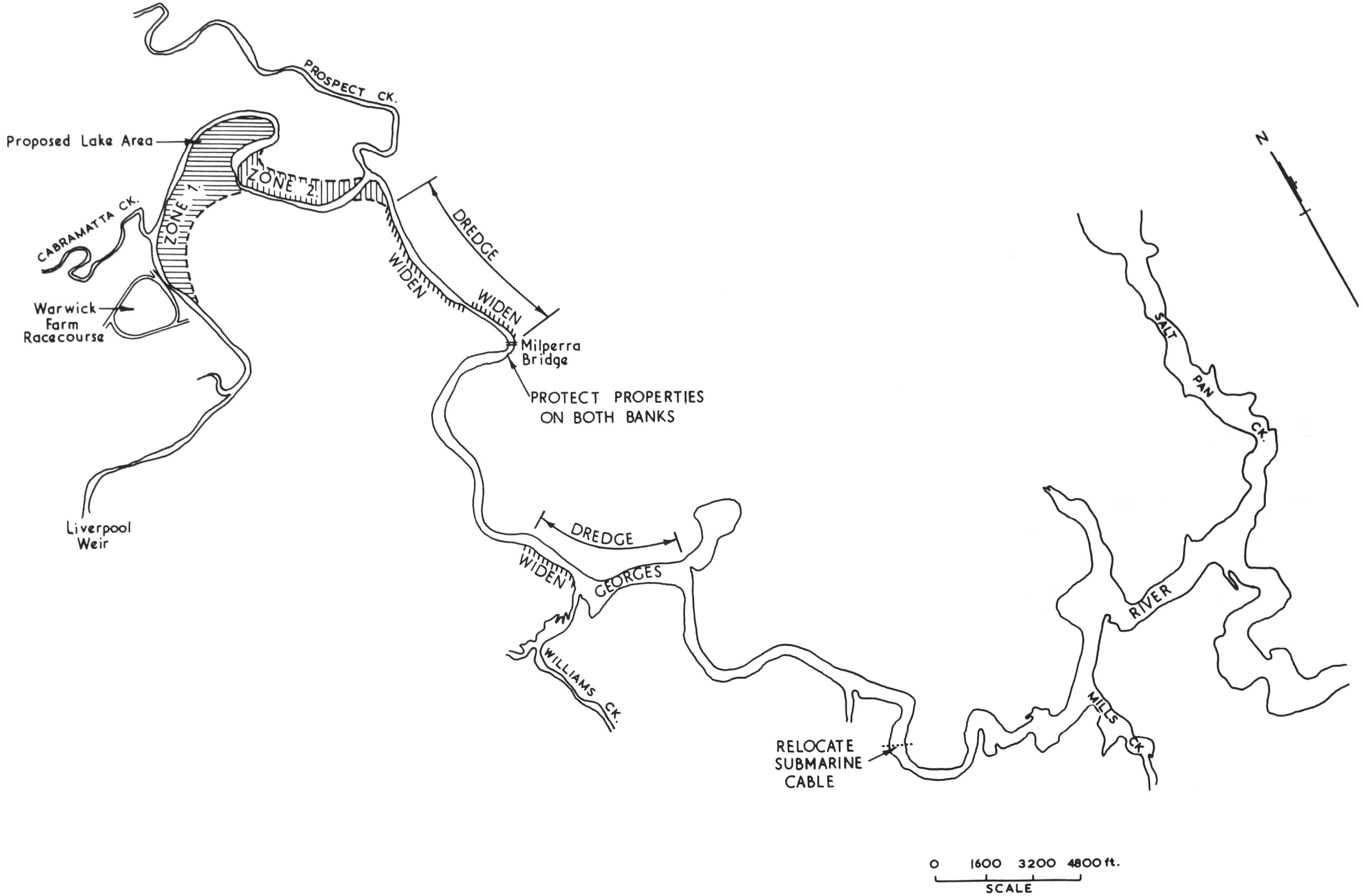


FIGURE 15: GEORGES RIVER: REMEDIAL MEASURES TO COUNTER EFFECTS OF SAND DREDGING

CE-D-7176

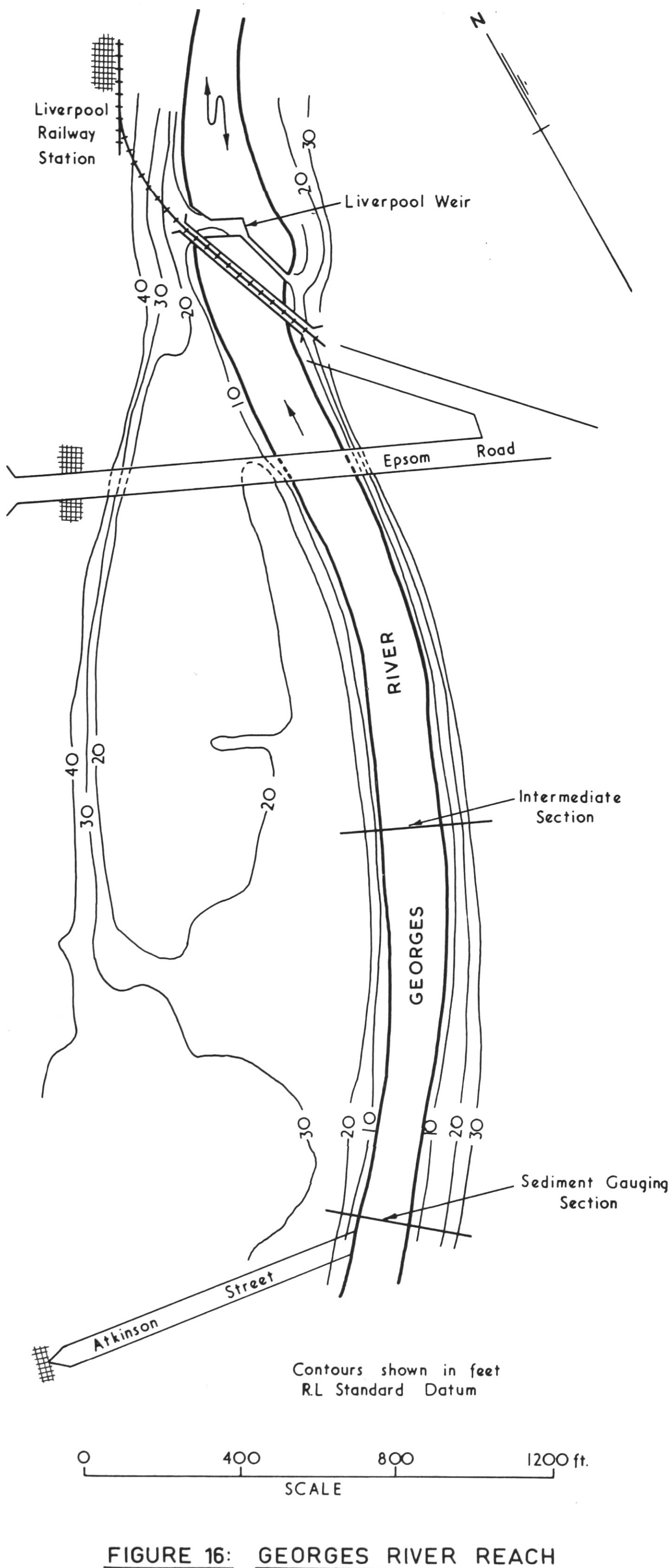


FIGURE 16: GEORGES RIVER REACH UPSTREAM OF LIVERPOOL WEIR

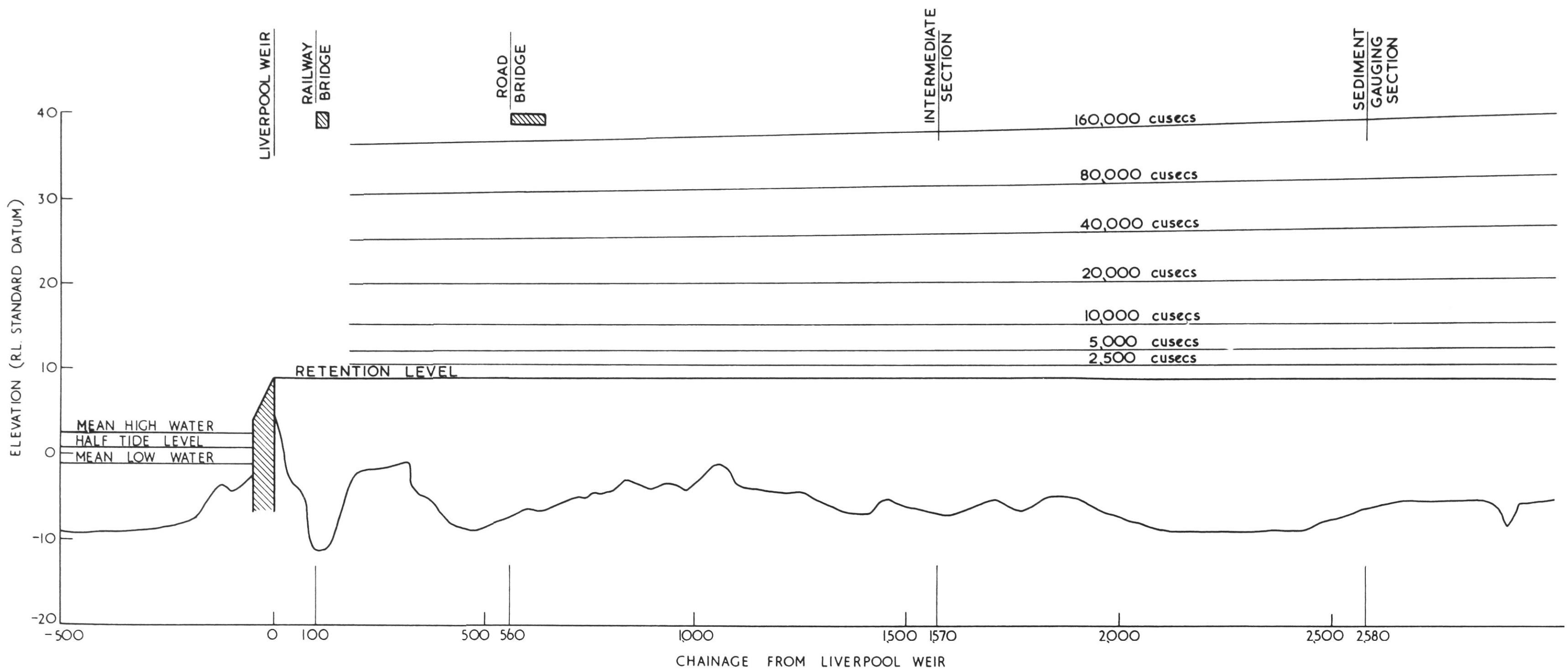


FIGURE 17: LONGITUDINAL SECTION, GEORGES RIVER REACH UPSTREAM OF LIVERPOOL WEIR

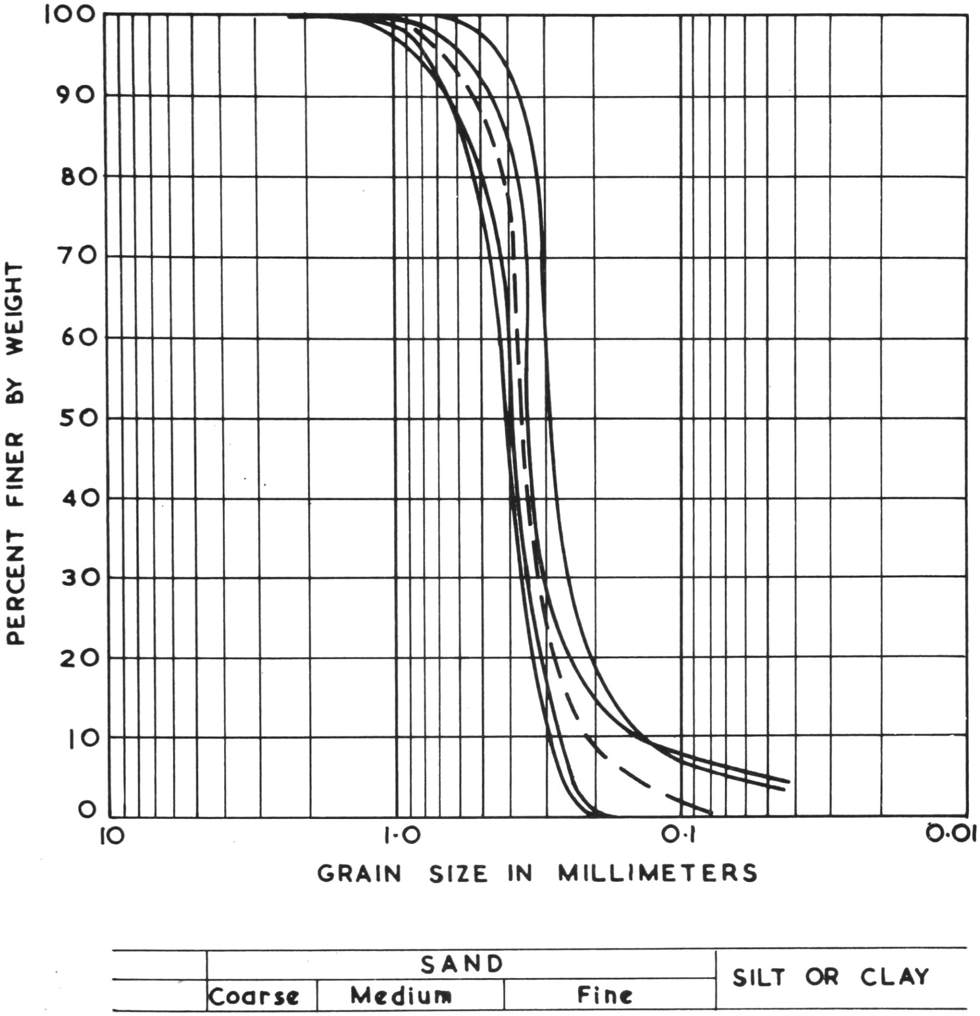


FIGURE 18: BED SEDIMENT GRADING CURVE

GEORGES RIVER UPSTREAM OF LIVERPOOL WEIR

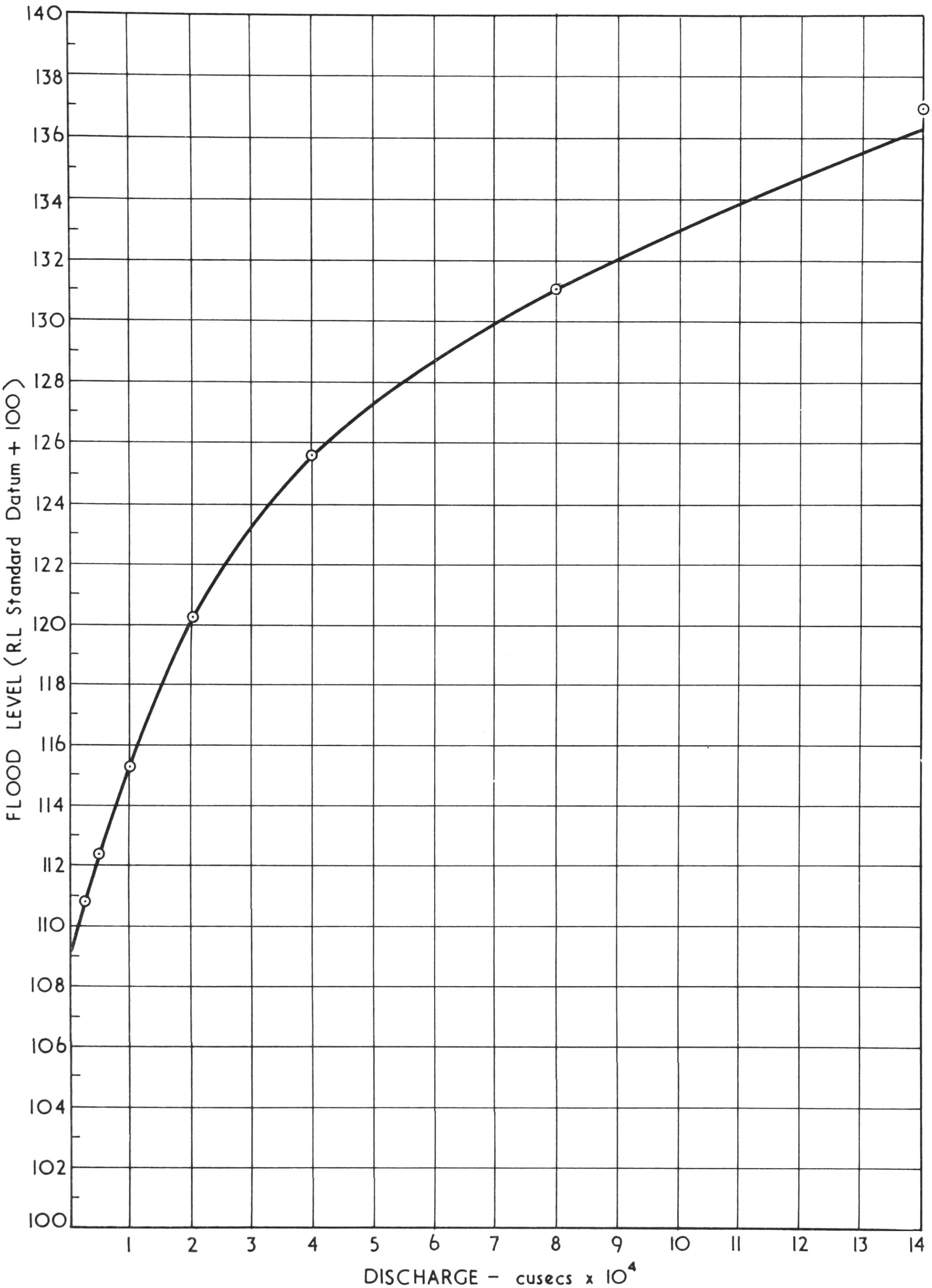


FIGURE 19: STAGE DISCHARGE CURVE, LIVERPOOL ROAD BRIDGE

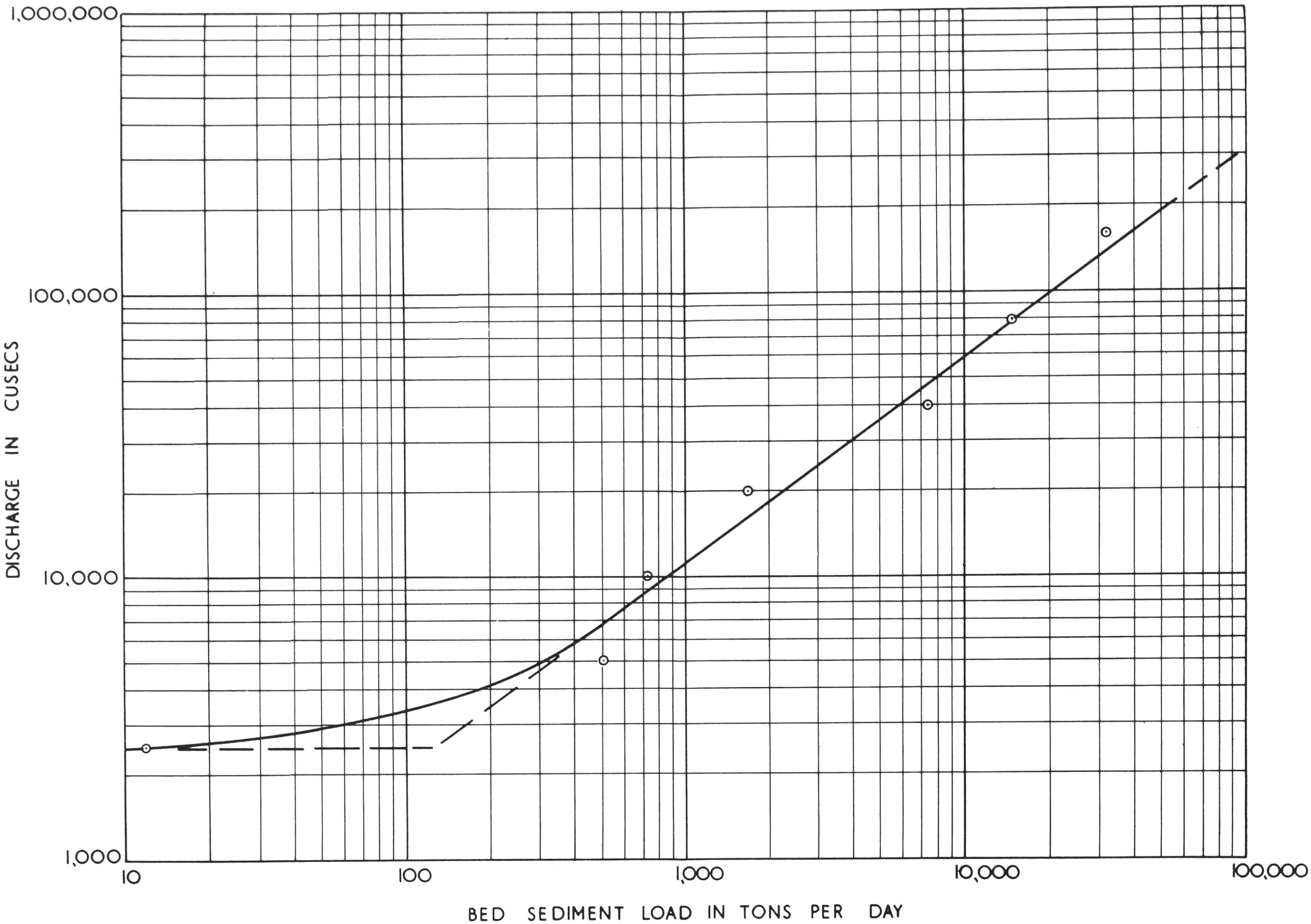


FIGURE 20: BED SEDIMENT RATING CURVE GEORGES RIVER UPSTREAM OF LIVERPOOL WEIR

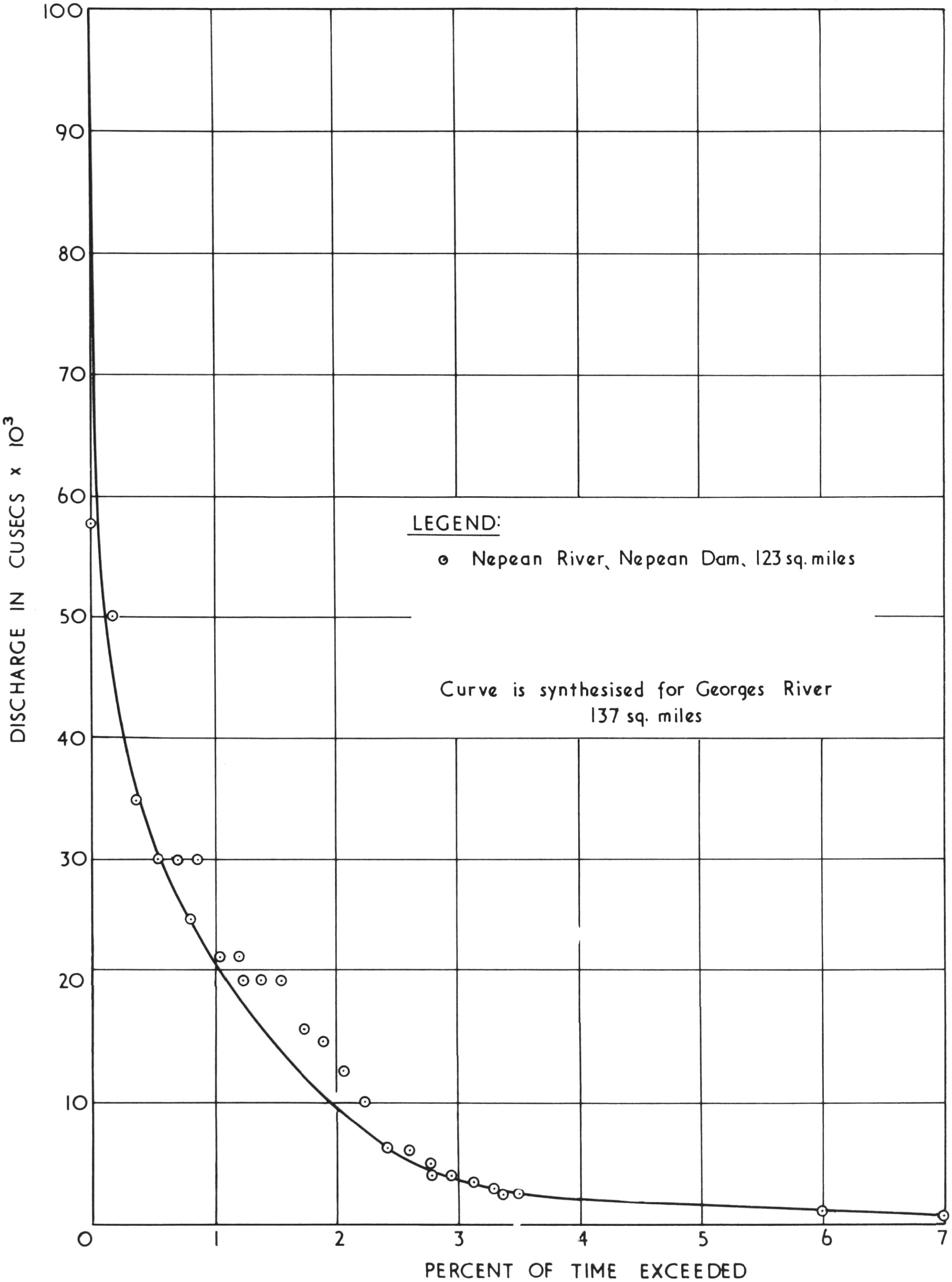


FIGURE 21: FLOW DURATION CURVE SYNTHESISED FOR GEORGES RIVER

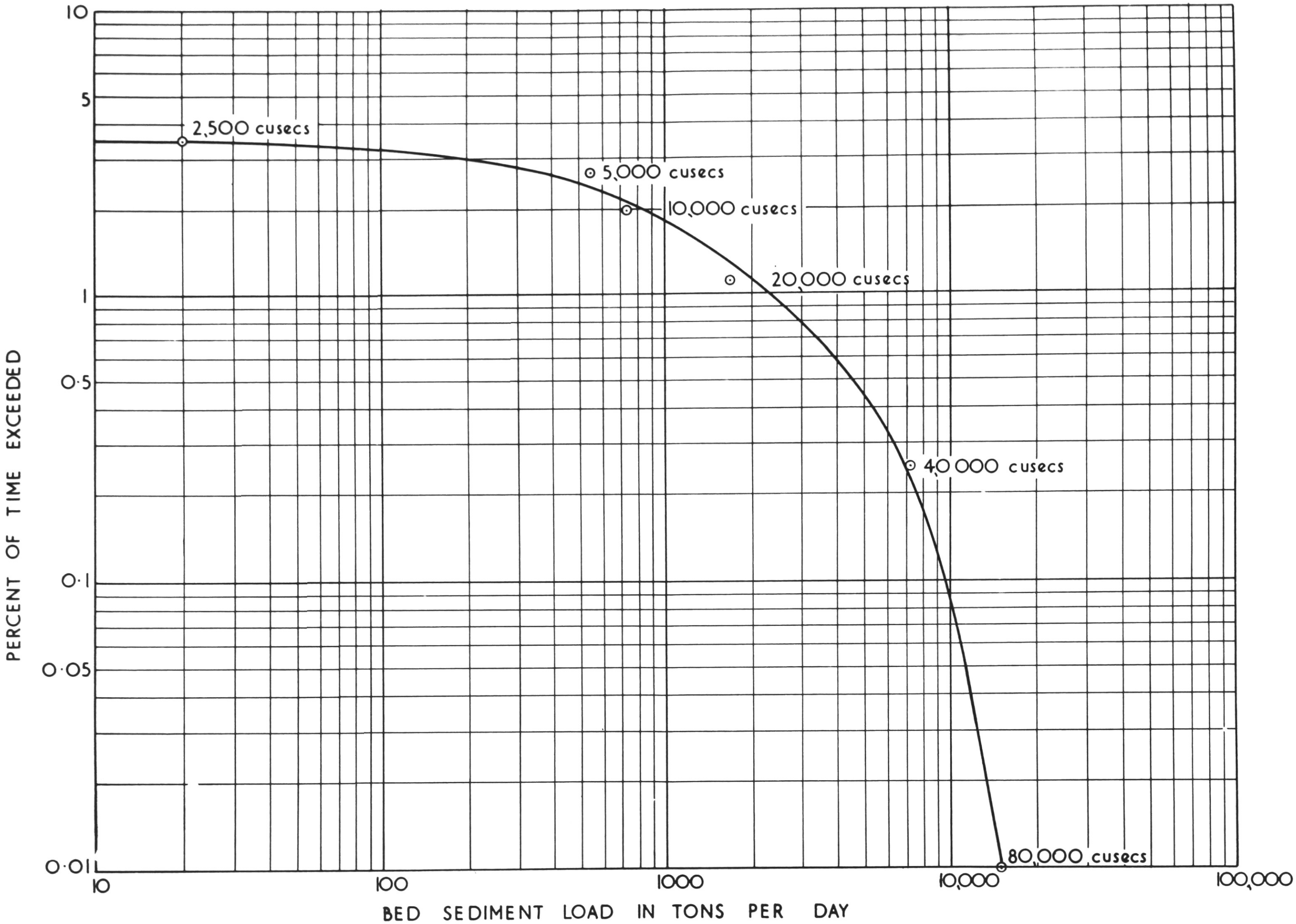
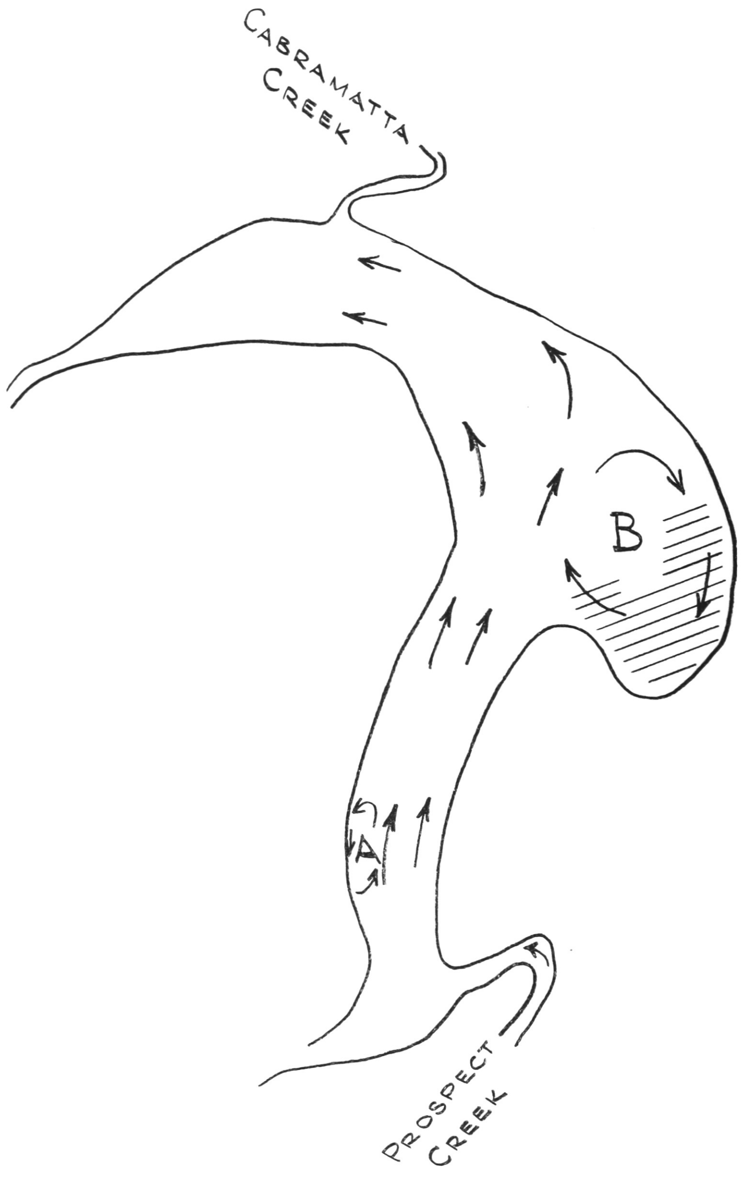


FIGURE 22: BED SEDIMENT LOAD DURATION CURVE, GEORGES RIVER UPSTREAM OF LIVERPOOL WEIR

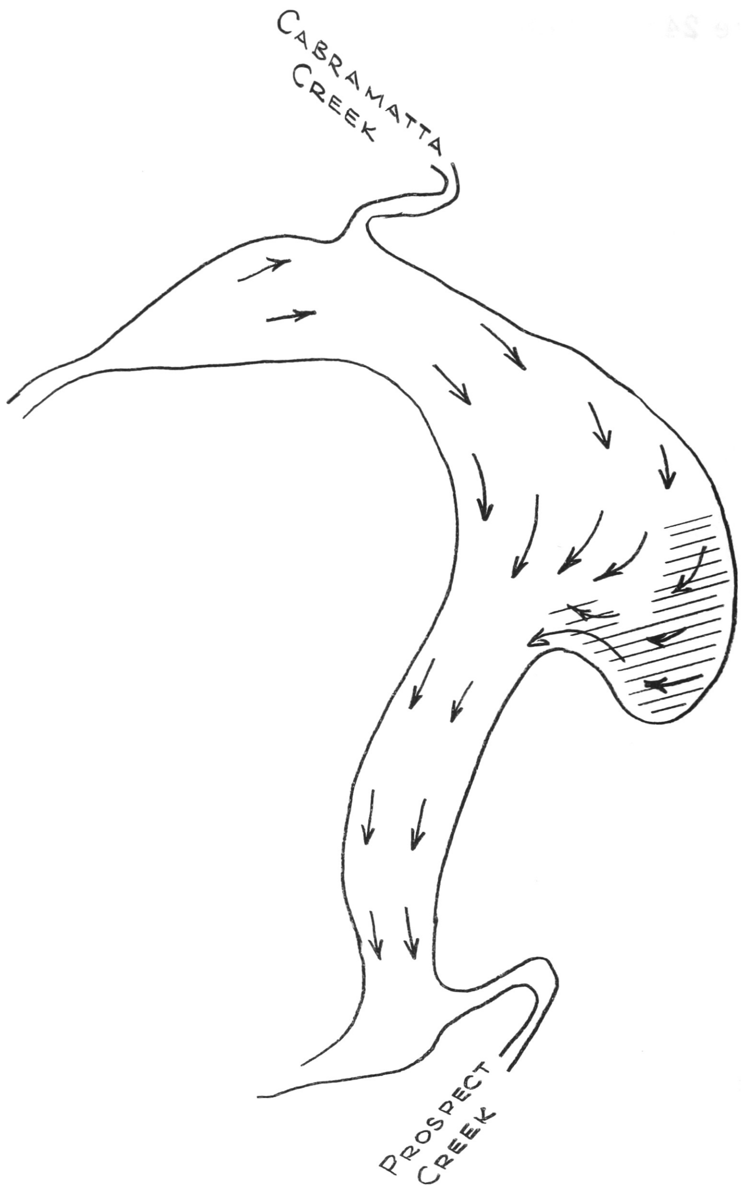
Figure 23: Flood Tide Flow Patterns.



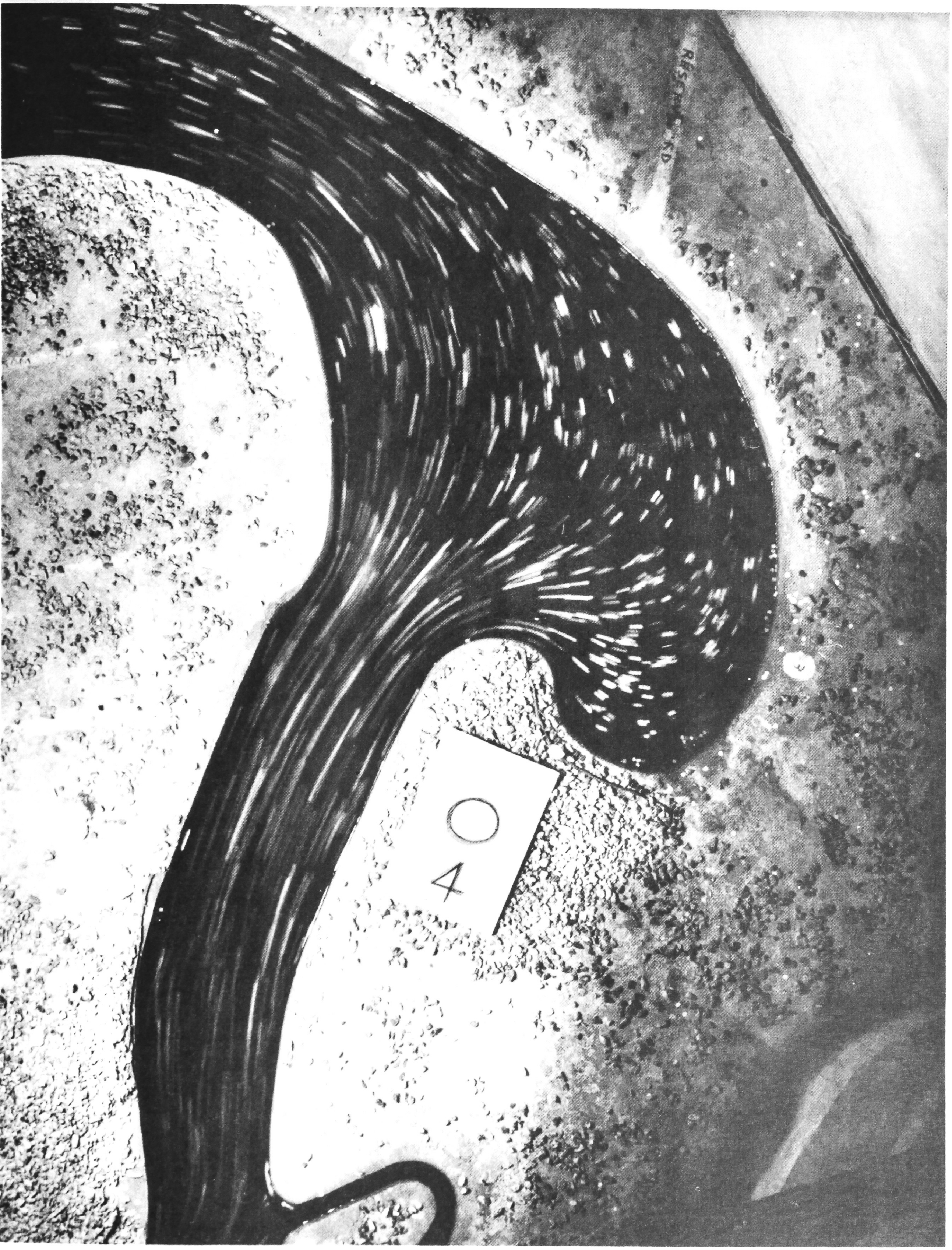
(a) Key to Flow Direction

(b) Photographic Test Result.

Figure 24: Ebb Tide Flow Patterns.



(a) Key to Flow Direction



(b) Photographic Test Result.

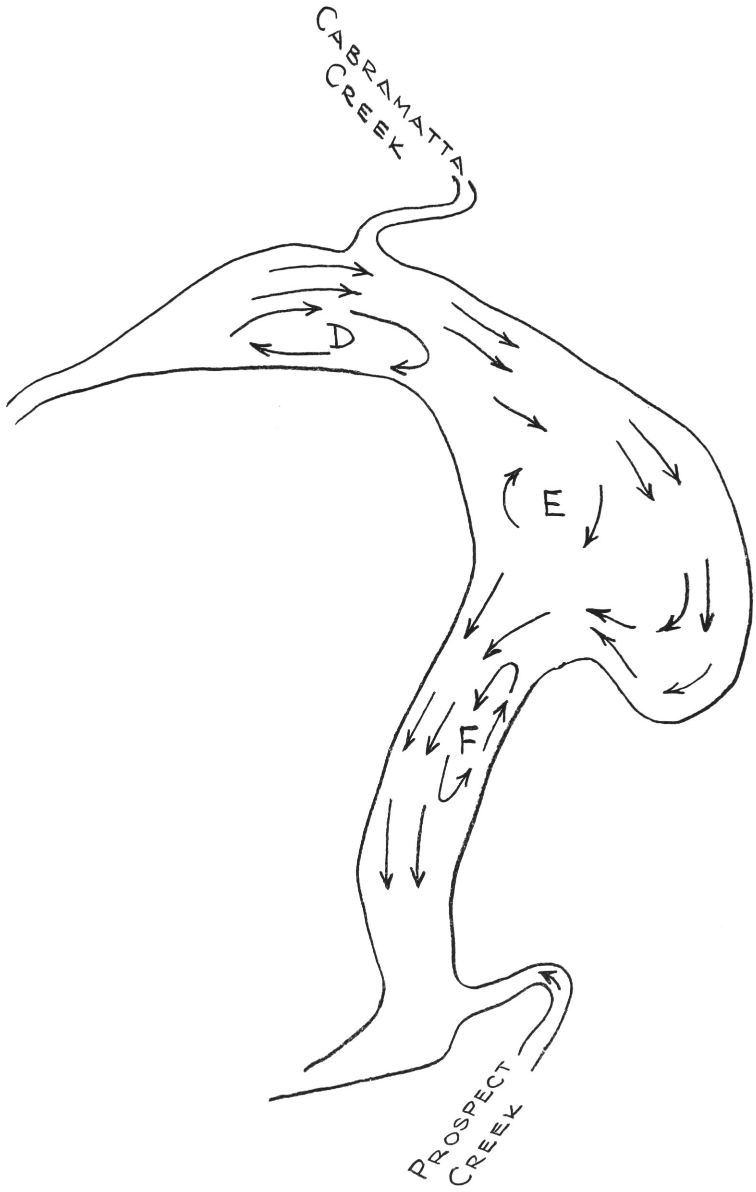
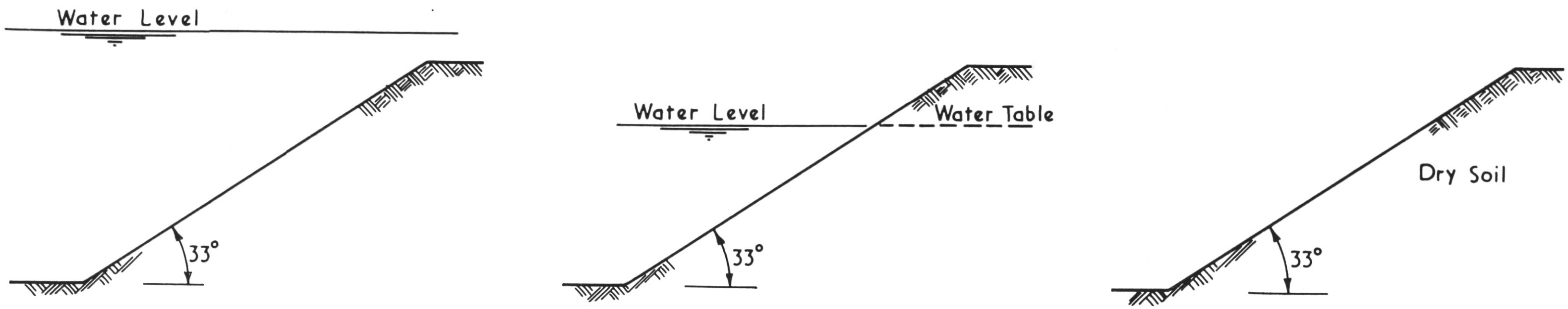


Fig.25: Flow Pattern when River is in Flood.



(a.) FULLY SUBMERGED ZERO NEUTRAL STRESS

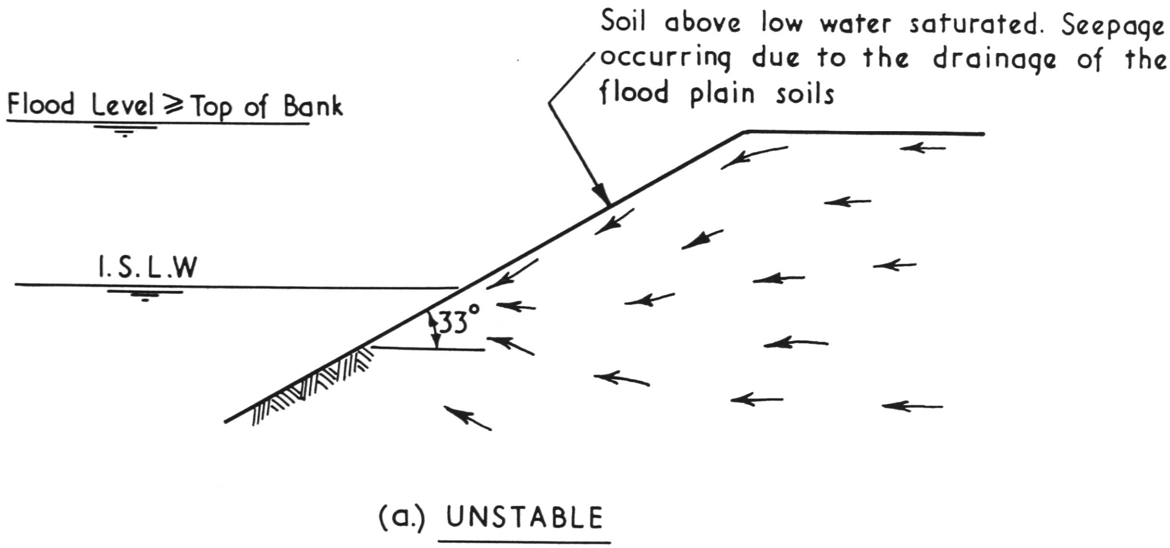
i.e. No seepage from bank soil mass into lake

(b.) PARTIALLY SUBMERGED ZERO NEUTRAL STRESS

i.e. A horizontal water table exists at the same level as the lake

(c.) DRY SOIL – NO WATER PRESENT

Fig. 26: Stable Bank Slopes - No Seepage Case.



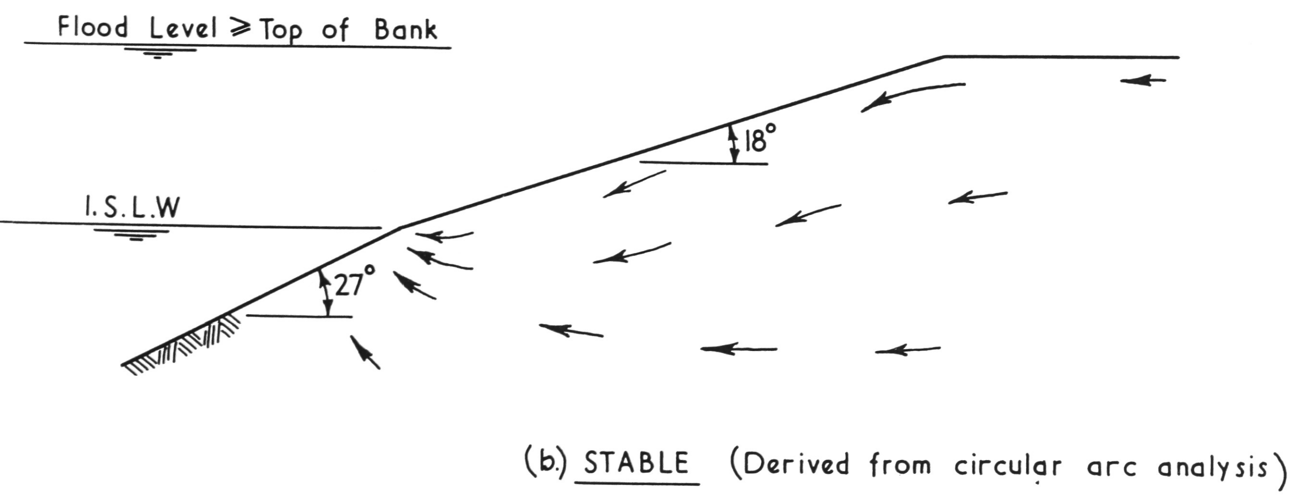


Fig. 27: Stable Bank Slopes - Seepage Case.

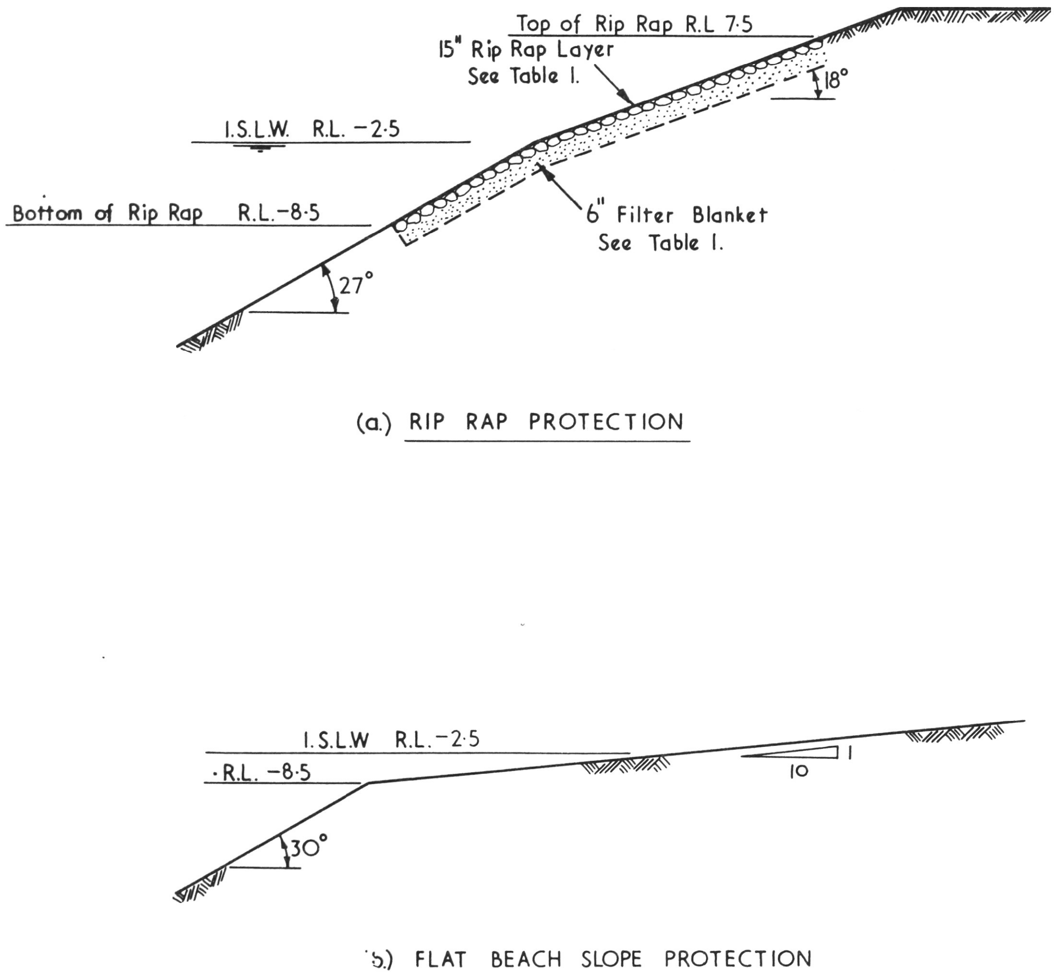


Fig. 29: Wave Protection.

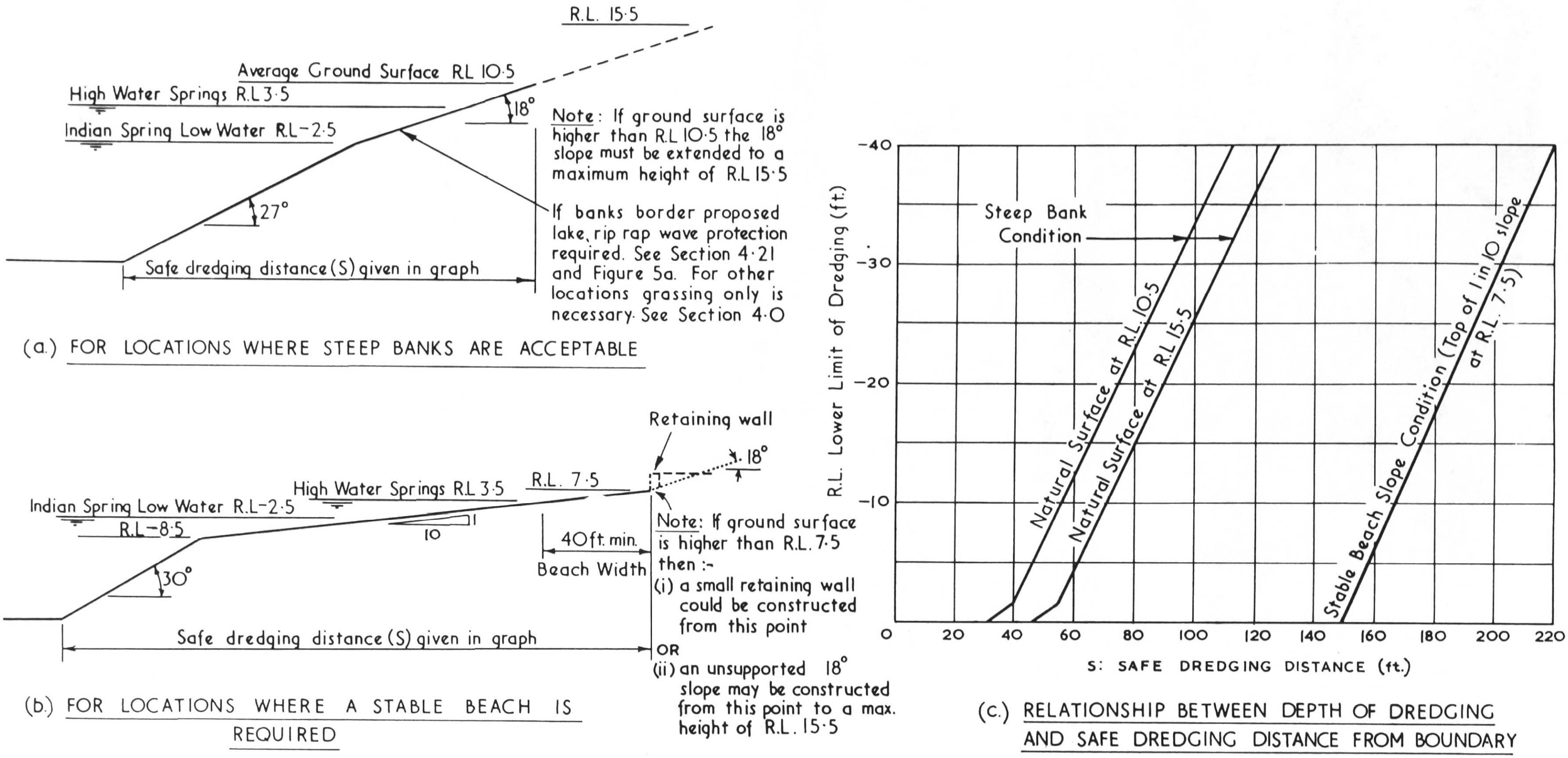


Fig. 30: Recommended Safe Dredging Distances.