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THE GEORGES RIVER HYDRAULIC, HYDROLOGIC AND RECLAMATION STUDIES

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The University of New South Wales

WATER RESEARCH LABORATORY.

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by

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Preface

This investigation was undertaken for Unisearch Ltd. on behalf of the St. George and Sutherland Dredging and Reclamation Committee.

The work was carried out by Mr. D. N. Foster, Senior Lecturer in Civil Engineering, and Mr. R. C. Nelson and Mr. F. C. Bell, Engineers, on the staff of the Water Engineering Department of the School of Civil Engineering. The investigation was directed by Professor C. H. Munro, Foundation Professor, School of Civil Engineering.

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Summary

Reclamation of inlets along the foreshores of the Georges River Estuary has been proposed. Hydraulic studies have shown that the proposed works will affect the river regime to some extent. Tidal velocities will be reduced by up to 10 percent leading to siltation of the river channels. A maximum reduction of 10 percent in the cross sectional areas of the river channels is forecast if all the proposed works are carried out.

The proposed works will also increase the pollution problem to some extent by reducing the tidal storage available for dilution of contaminants introduced into the river. However, as no systematic data are available, a satisfactory appraisal of the extent of the problem cannot be made at present.

As the river regime will be upset by reclamation works, it is considered unwise to proceed with large scale schemes, such as that proposed for Lime Kiln Bay, without first carrying out a study of the ultimate reclamation requirements for the estuary. From such a study, a scheme could be selected which best serves the requirements with minimum detrimental effects to the facilities provided by the river. An adequate programme of data collection must be completed before such a study could be undertaken and a sound engineering appraisal made of the long term effects of the proposed works.

The need for a more unified approach to the study of future problems is stressed. It is recommended that consideration be given to the formation of a joint committee to look at the present and future planning for the basin and in particular to initiate a programme of data collection.

In the course of the study, hydraulic and hydrologic information has been obtained which will give a sound basis for future studies in the River Basin.

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1. Introduction

In connection with some current and proposed foreshore improvement works in the lower reaches of the Georges River, N.S.W., a request was received by Unisearch Ltd. from the St. George and Sutherland Dredging and Reclamation Committee for advice on several aspects in the planning and design of the works.

A preliminary report was issued (W.R.L. Tech. Report No. 64/3) in which the problems involved were outlined and suggestions made for further studies. These studies have now been completed as far as existing data permits, and details are given in this report.

Although the dredging and reclamation works proposed are local in nature, their effects on the estuary as a whole cannot be discounted. Damage has resulted in the past from neglecting the long term changes in the river regime brought about by engineering works. By the time the damage became manifest it was often too late to rectify, except at great cost. A river knows no man-delineated boundaries, and modern engineering opinion recognises that the river basin should be the smallest unit for overall planning. Ideally the development and control by man of the land, vegetation and water resources should be planned basin wide and with consideration of all types of uses. To consider all aspects related to planning and development of the Georges River Basin is beyond the scope of this report but it is the objective to promote a unified approach to planning problems.

2. The Need for Unified Study

It is clear that over the next fifty years very great development will occur in the Georges River Basin. As residential areas develop there will be an increasing demand for the recreational facilities provided by the river whilst industrial expansion will place added requirements on the river's water resources. It is desirable that land and water resources should be utilized presently and in the immediate future to the maximum advantage of the community as a whole. Study is needed of the present and future requirements for navigation, land usage, reclamation, soil conservation, irrigation, channel improvement, flood mitigation, sewage and industrial waste disposal, control of bank erosion, fish and wild life preservation, oyster farming, and the provision of recreational facilities in aesthetic surroundings. Some of these features are at present being studied by the local, State and Federal Authorities concerned, but on a piecemeal basis with little evidence of co-ordination between them. The division of interests is briefly summarized in Table 1.

Table 1.

Authorities Concerned with Georges River Basin

Authority	Nature of Interests
Local Councils	Flood control, recreation, foreshore reclamation, dredging, pollution, town planning, shoreline erosion, commercial development.
State Planning Authority of N.S.W.	Town planning, recreation.
Public Works Department (New South Wales)	Flood control, navigation, river stability, siltation and erosion.
Maritime Services Board of New South Wales	Navigation, harbour development, pollution, waste disposal.
Metropolitan Water Sewerage and Drainage Board (New South Wales)	Storm water disposal, flooding and drainage, sewerage and waste disposal.
Water Conservation and Irrigation Commission (New South Wales)	Irrigation, flood control, conservation.
Australian Atomic Energy Commission	Waste disposal.
Department of Civil Aviation (Federal)	Airport development.
Health Department (New South Wales)	Pollution
State Fisheries (New South Wales)	Fishing and oyster farming.
Department of Main Roads (New South Wales)	Flooding and drainage

There is an urgent need for a co-ordinated programme. How this should be done is open to question. Much can be done by setting up a Committee representing the various interests to look at the overall development of the basin and to co-ordinate the work of the various bodies concerned. For example, studies of river pollution, flood control, river siltation, industrial waste disposal, oyster farming, fisheries and land use all require a knowledge of fresh water inflow. Despite this requirement there is no relevant gauging station established on the river or its tributary creeks. Past studies have required the use of subjective methods with their consequent limitations to estimate flow conditions. Although it may be argued by any one of the departments concerned that its work alone does not justify the expense of establishing and operating a gauging station, when considered together there is no doubt of the need and economic justification. Another example can be found in the excellent hydrographic charts produced by the Department of Public Works in 1961. These were produced mainly for navigation, and cover the river channel from the mouth to the limit of tidal influence. The major bays on the downstream reaches have not, however, been surveyed, nor has the Woronora River nor Salt Pan Creek, and consequently the use of the map is limited to the purpose for which it was prepared. If a complete survey had been made at the time it would have provided a base plan for present and future investigations to study other aspects such as pollution, siltation, dredging etc. When considered alone, it is unlikely that any single study would justify the cost of completion of the chart but collectively there is no doubt of its economic justification. The cost would now be considerably higher than if it had been done at the time the first chart was prepared. In fact, the entire survey would have to be done again, as there have been substantial changes in the river since 1961.

These two of many such examples clearly show the need for co-ordination of activities. If many thousands of dollars of public money are to be spent it is desirable that this be spent in the most efficient manner to achieve all objectives of river basin planning. A river basin is a multi-purpose unit and a data collection programme should be based on considerations of all uses, many of which are interrelated and require the same or an extension of the same data for analysis. In due course the Committee foreshadowed in the above, having defined the objectives and pointed the way for a constitution, could emerge as a Georges River Development Authority.

3. The Importance of Collection and Recording of Data

In recent years the engineering profession has realised forcibly that adequate data regarding physical phenomena are the first essential for the efficient planning of engineering works, and that such data must be comprehensive and accurate. Without the relevant data, engineering planning and design are inefficient and based significantly on subjective judgment. Despite the need for data there is little systematic recording of data on the Georges River and engineering studies of the nature described in this report are seriously hampered as a consequence.

The various Government Departments have from time to time collected data for isolated jobs but most of this is incomplete, unpublished and difficult to abstract. For example, flood levels are obtained at various stages of a flood by the Public Works Department, the Department of Main Roads and the Metropolitan Water Sewerage and Drainage Board, but all records are incomplete and cannot be related, except by subjective methods, to discharge. The Public Works Department and the Australian Atomic Energy Commission have isolated readings of salinity but these are of too short a duration to be of great assistance. The Department of Public Health and the Maritime Services Board collect pollution samples, but, as far as can be ascertained, these are not related to fresh water inflow, tidal dilution etc.

The system of individual Departments working independently is not conducive to economic data collection and there is an urgent need for the launching of a programme financed by all bodies concerned to obtain information in a systematic manner on:-

(i) Tidal stages and velocities.
(ii) Fresh water inflow for both dry weather and flood.

(iii) Pollution and its distribution.
(iv) Salinity intrusion for various river inflows.
(v) Flood stages and hydrographs.
(vi) Sediment inflow and distribution
(vii) History
(viii) Hydrography.

Should a committee be established along the lines outlined in Section 1, one of its first jobs should be to consider what data are required to serve the present and future needs for river basin planning and to outline the methods of obtaining such data.

4. Description of the Georges River Basin

4.1 Introduction

The Georges River is located about 10 miles south of Sydney and discharges into the south western corner of Botany Bay (see Figure 1). The tidal influence extends about 25 river miles upstream from the mouth to Liverpool Weir which was constructed in 1836. The headwaters of the river are located in the Darkes Forest - Appin area a further 25 to 30 miles upstream of Liverpool Weir and the total catchment area is 252 sq. miles.

The Georges River, since its discovery, has been the topic of much discussion as to the best means of utilising its natural attributes. During the latter half of the 19th century, the river was investigated as a possible source for Sydney's water supply and as a water source for large scale irrigation. The surrounding land has been and still is used for pasturage and agriculture, but through the years this activity has diminished as weekender retreats began appearing earlier this century, and later as permanent suburban dwellings encroached on the river bank areas.

A substantial oyster industry is supported by the tidal regions of the estuary, there being at the present time about 50,000 yards leased along the foreshore (including non-commercial leases) and 1,000 acres of offshore lease (all commercial). All commercial leases are below Salt Pan Creek.

The upper half of the tidal region is dredged commercially for its sand, and expansion of this industry is at present being contemplated.

The Georges River provides an excellent environment for pleasure boating and fishing, and the popularity of these sports is evidenced by the number of boat clubs and public boat ramps existing in the area.

Some relevant historical information on the development of the Georges River Basin is given in Appendix A.

4. 2 Land Cover and Use

The catchment above East Hills may be divided into two zones having quite distinctive features from both hydrological and developmental points of view. The dominant factor influencing these features is the underlying geology, and the zones are therefore conveniently defined by the shale-sandstone boundary shown in Figure 2.

The shale zone is 49 pc. and the sandstone zone 51 pc. of the total catchment area of 232 square miles.

In general the shale zone is more favourable than the sandstone for economic activities, and the northern section is already largely urban. Ribbon development has extended from Liverpool to Campbelltown where further extensive residential and industrial projects have been planned.

The remainder of the shale zone consists of cleared dairy pasture with occasional patches having a moderate cover of native timber.

Much of the sandstone zone is still in a virgin state with a moderate cover of eucalyptus. The northern section is part of the Commonwealth military reserve and contains some semi-urban development. The southern section includes crown land and water supply catchment reserves and appears unlikely to be significantly altered in the near future.

Land cover and use are considered in comparing Georges River with other catchments for the selection of appropriate hydrological parameters such as loss rates.

4. 3 Topography

The topography of the shale zone is relatively low with about 80 pc. of the area under 200 feet in elevation. The land surface is generally flat to undulating with stream gradients varying from 50 feet per mile near the watersheds to less than one foot per mile in the lower reaches.

Most of the streams in this zone are intermittent with watercourses consisting of chains of pools rather than well defined channels.

The sandstone zone is an inclined plateau sloping from an elevation of 1250 feet in the south to 10 feet above sea level in the north. The stream slopes vary from over 150 feet per mile in the upper reaches to less than one foot per mile in the lower reaches. Channels are well-defined and some of the flows are perennial.

Topographic factors determine the parameters used in the synthesis of unitgraphs (see Appendix B) and largely influence the shapes of flood hydrographs.

4. 4 Soil and Underlying Strata

In the shale zone the soils are tight clays and clayey loams overlying the relatively impervious Wianamatta shales. Some gully and sheet erosion is present and streamflows tend to be turbid even at low discharges. It is believed that most of the silt in the lower Georges River comes from this zone and separate hydrographs have therefore been derived to show the relative flood contributions from each zone (Fig. 6).

The Wianamatta soils and strata contain very small quantities of saline groundwater which has little effect on streamflows.

In the sandstone zone the soils are shallow and coarse with poorly developed profiles. Their perviousness, together with fissures and joints in the underlying rock evidently enable the storage of sufficient groundwater to maintain the small perennial streamflows.

There is little accelerated erosion in this zone and most flows are relatively clear.

The "Milperra Loams", (alluvial deposits of fairly rich soils in the lower reaches) are of considerable value for top-dressing and market gardening, but their area is probably too limited for them to be of direct hydrological significance.

4. 5 Rainfall Characteristics

The rainfall characteristics differ significantly from zone to zone, as shown by Figures 3, 4 and 5.

The shale zone has a two to four month seasonal drought occurring in winter to spring and has a mean annual rainfall varying from 28" to 30".

The sandstone zone has no marked seasonal drought and its mean annual rainfall varies from 30 inches to 50 inches. Rainfall intensities are also considerably higher in this part of the catchment.

Both zones average about 25 wet spells per year.

4.6 Rainfall, Runoff and Evapotranspiration

The inter-relations between rainfall, runoff and evapotranspiration have been studied (Bell 1967) for a number of N. S. W. catchments and on the basis of these studies the following estimates have been made for Georges River.

	Mean Annual Rainfall	Mean Annual Runoff	Mean Annual Evapotranspiration
Shale Zone	29"	4"	25"
Sandstone Zone	37"	11"	26"
Whole Catchment	33"	$7\frac{1}{2}$"	$25\frac{1}{2}$"

The evapotranspiration from the catchment depends on the amount of moisture available and varies from about 35" in very wet years down to 20" or less in dry years. The average "free water" evaporation measured by a standard Australian pan would be approximately 43".

4.7 Tidal Zone

In the tidal zone of the river, depths of 30 to 40 feet are not uncommon downstream of Como railway bridge while upstream, to Liverpool Weir, maximum depths are between 10 and 20 feet with isolated depths of 30 feet. Downstream of Como are a number of large off-channel bays, the more notable being Oyster Bay, Kogarah Bay, Gawley Bay and Woolooware Bay (Figure 13). Depths in the bays are relatively shallow (3 to 6 feet) with large areas exposed at low tide.

Tidal records supplied by the Department of Public Works reveal very little variation in tidal range between Botany Bay and Liverpool Weir (see Fig. 7). The mean spring ranges are 4.32 and

4.10 ft. at Dolls Point and Liverpool Weir respectively. The corresponding mean neap tidal ranges are 2.66 and 2.92 ft. respectively.

The bed sediments in the tidal section of the river vary according to their location but can be broadly classified into three zones:-

(i) The main channel above Como Bridge where the bed sediments are nearly all sand.

(ii) The main channel below Como Bridge, where the bed sediments are predominantly silts and clays.

(iii) The large off-channel bay areas where the beds consist almost entirely of flocculent clays and silts.

5. Hydrology of Georges River Basin

5.1 Objectives and Scopes of Studies

Hydrologic data are required for the proper planning of such matters as drainage, land reclamation, flood mitigation, siltation, pollution and recreational facilities.

The main information required is:-

(i) Flood magnitudes for various frequencies of occurrence.

(ii) Time distribution of runoff in the form of flow duration curves and/or mass curves for the various catchments being studied.

The achievement of these objectives is hampered by the complete absence of stream gauging records except for the small, headwater tributary of O'Hares Creek (28 sq. miles) and consequently recourse must be had to synthetic methods.

5.2 Flood Studies

5.21 Historical Records of Past Floods

The investigation has been greatly facilitated by data on past floods made available by the Public Works Department including:-

(a) Detailed levels and slopes between Liverpool and Picnic Point for the floods of June 1950, November 1956, November 1961, August 1963 and June 1964.

(b) Peak flood levels in the vicinity of Liverpool dating back to 1873.

The date of (b) above had been collected from a number of sources and some of it was conflicting, particularly with regard to the years of occurrence. However, it is believed that Table 2 lists the most probable levels and dates of important floods since 1873.

The Liverpool weir was constructed in 1836 at the suggestion of Surveyor Lennox . This was before any of the recorded floods and they are therefore all directly comparable.

It is interesting to note that Liverpool was founded on the banks of the Georges River by Governor Macquarie in 1810 and the siting of the town was considerably influenced by two floods "rising more than 34 feet" in 1809.

Table 2: Peak Flood Levels at Liverpool and Milperra

Year	Estimated R. L. at Liverpool Traffic Bridge	Estimated R. L. at Milperra	Est. Discharge at Milperra*		Rank	Recurrence Interval (years)
			n=.04	n=.03		
			cfsx10^{-3}			
1873	134.5	121.3	80	106	1	95
1875	132.5	119.7	66	90	2	47
1889	132.0	119.2	63	86	3	32
1956	128.0	116.0	41	57	4	24
1914	125.0	113.5	29	39	5	19
1950	124.8	112.6	25	33	8	12
1933	124.5	113.1	27	37	6	16
1961	124.0	112.4	24	32	9	11
1900	124.0	112.7	25	34	7	13
1895	123.5	112.3	24	32	10	9.5

* n refers to the Manning roughness coefficient.

Note: The above were collected by the Public Works Department from various council and other records. The levels which were taken at Liverpool Sewerage Works and other places in the vicinity, have been adjusted to give the estimated levels at Liverpool Traffic Bridge which is the location of the present gauge.

5.22 Flood Peak Frequencies Georges River, Prospect Creek and Cabramatta Creek

A technique of flood estimation based on rainfall and catchment characteristics was used for estimating flood flows and this was checked with independent hydraulic calculations based on observed flood levels. Details of the method and calculations are given in Appendix B.

From these calculations, the flood frequency curve for the Georges River at Milperra Bridge has been established as shown in Figure 8. Design hydrographs for the Georges River at East Hills, Prospect Creek at Georges Hall and Cabramatta Creek at Chipping Norton for flood recurrence intervals ranging from 2 to 100 years are given in Figures 9, 10 and 11 respectively.

5.23 Flood Peak Frequencies, Salt Pan Creek, Oatley Bay and Oyster Bay

These were selected as representative samples for the study of bays likely to be reclaimed.

The catchment area contributing to Salt Pan Creek is 10.1 square miles. Approximately 30 pc. of the area is impervious and the remainder consists largely of urban lawns, paddocks and parklands on clayey soils. The stormwater drains are mostly lined and straightened.

The catchment area contributing to Oatley Bay is 1.9 square miles and is a medium density residential district. Approximately 24 pc. of the area is impervious and the remaining lawns are on clayey and sandy soils. Stormwater drainage is well developed with straight, lined channels.

Oyster Bay is a relatively new residential area with large amounts of undeveloped land and open space. Its catchment area is 2.0 square miles, about 15 pc. of which is impervious.

The small sizes of these catchments do not justify complete synthesis of unitgraphs, and flood peaks were calculated by the rational method using critical storm duration equal to the estimated time of rise of the hydrographs.

The effective times of rise of these hydrographs were estimated from the channel flow times using standard procedures (Institution of Engineers, Australia, 1958), resulting in the following values:-

(i) average for subcatchments surrounding bays

Salt Pan Creek	0.8 hrs.
Oatly Bay	0.6 hrs.
Oyster Bay	0.8 hrs.

(ii) at bay mouths

Salt Pan Creek	2.2 hrs.
Oatly Bay	0.8 hrs.
Oyster Bay	1.0 hrs.

The estimated flood peak frequencies at the bay mouths are shown in Fig. 12. Estimates of flood peak frequencies for any other subcatchment may be made using the same procedures.

5.24 Effect of Future Development on Flood Flows

The possible effects of further development of the catchments on flood characteristics should be considered. These may be placed in two categories viz.

(a) Effects on rainfall losses
(b) Effects on time distribution of flood flows.

In general (a) is unlikely to be significant because losses are usually quite low during major floods. However, the potential effects of urban development on the time distribution of flood flows are important because reclamation, channel stabilization and improved drainage all reduce storage delay times, causing earlier and higher flood peaks in the lower reaches of the river.

Under some conditions, increases in flood peaks of the order of 25 pc. are possible, but to determine whether these are likely for the Georges River is beyond the scope of this report. It is suggested that estimates of such effects should be part of the design phase of any extensive developmental schemes so that possible undesirable consequences may be anticipated and minimised.

5.3 Time Distribution of Runoff

The complete absence of stream gauging data on the catchment with the exception of the small headwater tributary, O'Hare's Creek (28 sq. miles) makes any estimates of mass curves or flow duration curves very suspect. For this reason these studies have not been carried out. In the opinion of the authors they should await several years of streamflow record. Flow duration and mass curves could then be synthesized by a comparison between the short term records of streamflow on the Georges River and its tributaries with long term streamflow records on nearby catchments in homogeneously similar regions.

This information would be required before a complete study of pollution and sedimentation in the Georges River Basin could be made and serves to emphasize the urgent need for establishment of stream gauging stations. With the growth of industry along the river there will be an increasing demand to use the river to its full potential for the discharge of industrial wastes.

The collection of systematic data on streamflow and salinity will enable the dilution potential of the river to be estimated in a much more reliable manner than is possible at present.

6. Hydraulic Aspects of Georges River Basin

6.1 Bed Sediments

Forty two samples of the bed sediments were obtained by a standard grab sampler from various locations within the river basin as shown on Figure 13. An analysis of these samples showed that the estuary could be divided into three sections.

(a) The main channel reach above Como Bridge, the bed consisting mainly of sand.

(b) The main channel reach below Como Bridge, the bed being predominantly silts and clays.

(c) The large off channel bay areas in the lower estuary regions where the beds consist entirely of flocculent silts and clays.

Although the main channel below Como Bridge has a bed of predominantly silts and clays, the samples nearly all contained some gritty and sandy particles. The exception to this was in the reach immediately upstream of Tom Ugly's Bridge (samples 26 and 27) where the bed samples showed highly plastic stiff clays. This would indicate a clean swept bottom with little evidence of deposition. Sediments from the off-channel bays showed no gritty or sandy particles and the evidence would indicate that bays such as Oyster, Kogarah and Woolooware Bays are acting as silt traps for suspended silts and clays brought into the relatively stagnant waters of the bay from headwater catchments or by the interchange of tidal waters between the bay and estuary.

6.2 Suspended Sediment

A limited number of suspended sediment samples has been taken from various locations along the main channels as shown in Figure 14.

Samples 1 to 6 inclusive, taken in the reach downstream of Salt Pan Creek, were collected after a spell of fair weather and should be an indication of the sediment carried in suspension near the bed under tidal conditions with low fresh inflow.

Samples 7 to 10 inclusive, taken from the upper tidal regions above Salt Pan Creek, were collected within 24 hours of the flood peak which occurred on 7th March, 1967, and give some indication (with the exception of sample 8) of sediments in suspension after a fresh.

Although the two sets of samples were collected from two different reaches of the estuary under different circumstances, it can be concluded that, in the tidal regions above Como the suspended load during the flood recession was up to 4 times as great as that under tidal conditions with low fresh inflow. This figure was probably greater nearer the peak of the flood.

The sediment load of the Georges River at flood times has always been a problem. Historical literature (see Appendix A) reveals that in 1887 and 1913 freshes deposited sufficient sediment to smother a large proportion of the oyster population.

Sample No. 8 about a mile upstream of Salt Pan Creek shows an abnormally high suspended sediment concentration. Multiple samples were taken at the location to guard against possible errors but these verified the presence of the high concentration. The hydrographic plans showed an isolated deep hole at this location. The reason for the hole is not known but indications are that it is filling with silt (see bed sample No. 16, Figure 13) being brought in by tidal action and flood flows.

6.3 Tidal Heights and Currents

Excellent records of tidal ranges and heights have been taken along the Georges River estuary by the Department of Public Works of N.S.W. These are shown in Figure 7 and show little variation in tidal stage along the river. Lag times are also shown on this figure and these indicate an average lag of approximately $2\frac{1}{2}$ hours between the mouth and Liverpool weir 29.3 miles upstream.

Data on tidal currents were not available, however, and measurements were taken at Tom Ugly's Bridge and Lugarno Ferry to obtain current variations over a tidal cycle. The results are shown in Figure 15 and tidal heights over the same period in Figure 16. Using a step solution to the equation of motion, velocities at other locations along the estuary and the effect of the proposed reclamations on these velocities were estimated. Details of the calculations are given in Appendix C and the results are discussed in Section 7.

6.4 Relation Between Tidal Prism and Channel Capacity

O'Brien (1931) and Bruun and Gerritsen (1958) have demonstrated that for many estuaries around the world a relationship exists between channel capacity and tidal prism (i.e. the tidal volume between low and high tide levels). This relationship is based on the assumption that the channel dimensions are related to the flow velocity which in turn depends upon the amount of tidal storage available.

That such a relationship holds for the Georges River Estuary is clearly shown in Figure 17.* However, as might be expected, the form of the relationship also depends upon the type of bed material through which the channel is cut.

Upstream of the junction with the Woronora River the capacity of the channel, (A_{uw}) which is largely through sand, can be closely approximated by the expression

$$A_{uw} = 0.0072\, V^{0.73}$$

where A_{uw} = river cross sectional area - ft^2

V = tidal prism between M.H.W. springs, and M.L.W. springs upstream of river cross section - ft^3.

Below the junction with the Woronora River the bed materials change abruptly from sands to predominantly clay and silty clay, and

* Figure 17 is based on the hydrographic plan prepared by the Public Works Department, N.S.W., 1961. Since this date, sand dredging has been carried out, and at some sections of the river the channel area has been significantly increased. This applies particularly to the middle reaches of the river between Lugarno and Milperra.

the relationship between channel capacity (A_{dw}) and tidal prism is approximated by -

$$A_{dw} = 2.25 \times 10^{-17} \quad V^{2.42}$$

It is noted that where bed materials are particularly fine (silt and clays) a small change in tidal prism (such as by reclamation) results in a relatively large change in cross section area. Where bed materials are coarser (sands), a change in tidal prism results in a comparatively smaller change in cross section area.

Several points do not lie exactly on the curve and this can be attributed in most cases to differences in the bed material. Some variation results from enlargement of the river section by dredging as noted for location 28 on Figure 17. However, the trend of the relationship is evident and the equations given above can be used as a reasonable estimate of the likely changes in channel cross-section areas which would follow any works which reduce the tidal prism.

6.5 Salinity Distribution

6.51 General

A significant factor which augments sedimentation is the intermixing of fresh water laden with silt and clay with salt water brought into the estuary by tidal currents. This intermixing causes flocculation of the suspended material and greatly accelerates its tendency to settle on the bottom.

Broadly speaking, estuarine rivers can be classified within a range of two extremes depending on the extent and type of mixing that occurs between the tidal intrusion of salt water and the downstream flow of fresh water. These extremes are called fully stratified and fully mixed.

(a) Fully Stratified Estuary

In some estuarine rivers, it is found that the vertical profile can be divided into two distinct layers; a layer near the bed with a salinity approximating that of the ocean and a layer of fresh water near the surface. The bottom layer is called a salt wedge (Fig. 18) and may penetrate for long distances upstream, moving back and forth with the tides. These conditions prevail where the ratio of tidal prism to total fresh water inflow during one tide cycle is relatively small, approximating unity. Many estuaries display stratified characteristics only at certain times, as for example during floods.

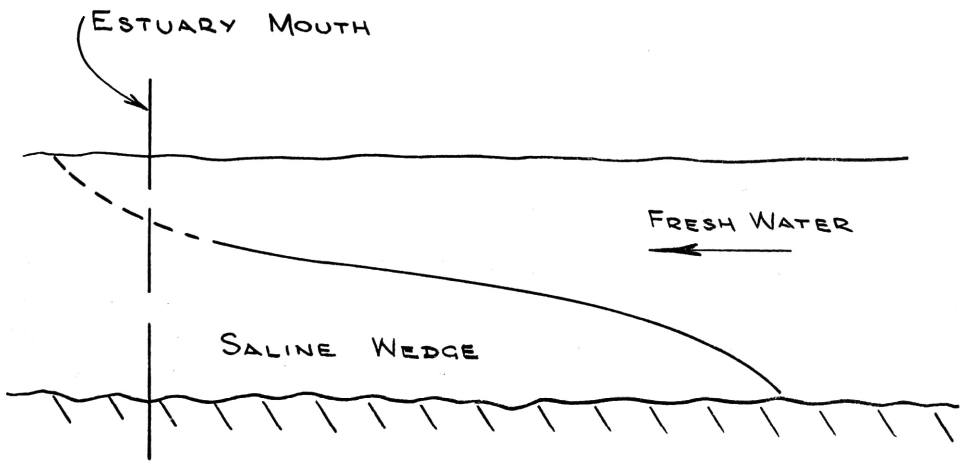

Fig. 18: Fully Stratified Estuary.

(b) Fully Mixed Estuary

In this type of estuary the salinity variation with depth at any section along the river does not display large variations but is of small order compared with the local mean salinity. As a result the salt intrusion may be defined in terms of a longitudinal profile of mean salinity.

6.52 Salinity and Channel Stability

As a result of flocculation, sediments accumulate near the end of the salt water intrusion zone, the intensity of accumulation being greatest for stratified estuaries and occurring at the end of the salt wedge, while for fully mixed estuaries, sedimentation is dispersed over a longer length and therefore shoaling is not generally a problem. It can be seen that siltation will be intensified if any measure creating a shift of regime towards a fully stratified estuary is introduced.

6.53 Salinity Profiles

There are no data on salinity variation with fresh water inflow taken for the Georges River. One set of salinity readings taken by the Department of Public Works is shown in Figure 19 but unfortunately the fresh water inflows at the time these were taken are not known. However, they do tend to indicate that the Georges River is a well mixed estuary, as might be expected from the fairly small fresh water flows which prevail over most of the time.

6.6 Pollution

6.61 Flushing Potential of Basin

The investigation of this matter has been restricted owing to the lack of data. As pointed out in Section 2, no hydrographic survey information is available for the Woronora River, Salt Pan Creek and numerous other bays, making it necessary to confine studies to the reach above Salt Pan Creek.

The method adopted in determining dilution potentials and flushing times was that developed by G. H. Ketchum, (1951). It is a modified version of the classical tidal prism method. The classical method assumes:

(i) a renewal of sea water equal to the tidal prism occurs in each tide cycle,

(ii) a complete mixing of all the waters affected by the tide.

In many instances, these assumptions are not in accord with fact. Ketchum's modified approach is more in keeping with the physical nature of tidal action, removing the objection that the tidal currents may not be strong enough to transport tidal water over the entire extent of the estuary. In this method, the estuary is divided into segments, each segment being the average excursion of a particle on the flood tide. Nevertheless there are still disadvantages as complete vertical mixing is still assumed within any segment. As described in Section 6.5, complete vertical mixing in tidal estuaries does not occur. Some vertical stratification is always present, evidenced by a salinity variation with depth. Despite this and several other empirical assumptions made by Ketchum, the method is probably the best with the data available and has been shown to give reasonable estimates of flushing potential in other parts of the world.

The method was applied to the Georges River above Salt Pan Creek, using three river inflows, namely 10, 50 and 100 cusecs, a range covering the flows at which the river runs for the greatest percentage of time. The results are shown in Figures 20 to 23, together with explanatory notes. The curves assume steady river inflow at Liverpool with no significant freshwater inflow below Liverpool Weir, a condition applicable to dry weather when dilution problems can be expected to be most serious. The curves can be used in assessing dilution volumes and flushing times at any location for pollutants introduced at Liverpool Weir or for pollutants discharged by local outfalls into any other segment.

The salinity survey shown in Figure 19 indicates that at a point 12 miles downstream of Liverpool Weir, the estuary contained about 40 per cent of river water. The corresponding river inflow conditions are not known except that 4 inches of rain fell at Liverpool during the 13 days prior to the survey, sufficient to cause a fresh with significant runoff along the entire estuary. As the inflow conditions are not known the survey cannot be used as a satisfactory test of the derived curves. Figure 21 gives a value of 20 per cent river water at this location for 100 cusec inflow at the weir.

Ketchum's method can be applied to sections downstream of Salt Pan Creek when more data are available.

6.62 Pollution in Georges River

Tests made by the Health Department in the estuary above Como Bridge indicate the existence of a significant pollution problem. The north side of the estuary is mainly sewered except for a section between East Hills and Prospect Creek. Some sewage wastes are discharged directly into the estuary along this reach. Variation of the Local Government Ordinances is at present being considered to curb this practice,and this together with the expected extension of carrier sewers should eliminate this area as a significant pollution contributor. Other sources of pollution on the north side are the sewage treatment works at Liverpool and Fairfield, the latter having been improved recently to reduce estuary contamination.

Analysis of water samples taken above Salt Pan Creek by the Health Department has led to the closing of swimming baths because of the high coliform content. Improvements have been made to the upstream sewage treatment works to diminish this problem.

Downstream of Como, sufficient dilution appears to be available in the main channel to prevent undesirable pollution concentrations. In this lower reach the problem is confined to off-channel bays where sewage effluents and storm runoff from unsewered areas discharge. Use of garbage for reclamation has been a problem, but the use of drained impervious clay wall cells to contain the garbage has now diminished the problem.

On the southern side, sewers are almost non existent, resulting in concentration of pollution in bays where effluents discharge. Examples are Oyster Bay and Double Bay where effluents from the Jannali area drain. Serious localised pollution has occurred in Woolooware Bay due to inadequate methods of retaining fill.

7. Proposed Reclamation Areas and their Effects

7.1 Introduction

A number of low water areas along the Georges River have been proposed as likely locations for reclamation. These are shown on Figure 24 and can be broadly classified into three types:-

(i) Reclamation of low lying areas which are at present substantially above high water.

(ii) Reclamation of narrow strips along the main stream in a form which amounts to bank straightening.

(iii) Reclamation of substantial portions of various bays which at present lie below high water.

In the following section we concern ourselves only with the hydraulic aspects of the reclamations. Other aspects such as town planning features, legal obligations to water front owners and owners of small craft, aesthetics and rate of pollution if garbage is used as fill are not discussed.

The immediate effect of large scale reclamation is a reduction in the tidal prism or the volume of water stored in the estuary over a tidal cycle. This in turn, if of large enough a magnitude, may affect tidal currents and stage, channel capacities and the amount of water available for tidal flushing of industrial wastes. Of the three classes of reclamation outlined above, only the third comes into this category.

The reclamation works shown in Figure 24 reduce the tidal prism by 34.3 million cubic feet which represents 3 pc. of the total tidal prism for the whole estuary. Future proposals could increase this volume.

7.2 Effects of Reclamation on Tidal Stages and Velocities

Using the procedures outlined in Appendix C, the effect of the proposed reclamations on tidal velocities in the main channels and tidal stage and range have been computed.

This study showed that there would be no measurable effects on stage, the tidal range at all points along the estuary being the same as before the reclamation works. The effect on tidal velocities in the main river channels was more significant and the estimated percentage reductions in maximum velocities at selected locations along the river are shown in Table 3.

Table 3

Estimated Reduction in Tidal Velocities Resulting from Proposed Reclamation

Location	Approx. Max. Tidal Velocity as measured f.p.s.	Estimated Pc. Reduction in Tidal Velocity
Captains Cook Bridge	-	6.9
Tom Ugly's Bridge	0.8	8.5
Kangaroo Point	-	9.2
Green Point	-	9.2
Como Bridge below Woronora River	1.2	9.7
Como Bridge above Woronora River	1.5	7.0
Lugarno Ferry	1.5	4.6
Upstream Salt Pan Creek	1.1 - 2.1	No change

7.3 Effect of Reclamation on Channel Capacities and Channel Areas

The reductions in tidal velocities shown in Table 3 are what could be expected immediately after completion of reclamation works. This would lead to a decrease in the capacity of the tidal currents to transport sediment, and deposition would occur until the velocities were increased to somewhere near their present values. Consequently, the predicted percentage reduction in velocity will approximately equal the percentage reduction in channel area after siltation has occurred and the river again reaches a stable regime.

An alternative method of estimating the likely channel changes is by using the relationship between channel capacity and tidal prism developed in Section 6.4. Reduction in tidal prism will tend to reduce tidal velocities and promote siltation of the channel until the channel area has been reduced to a value consistent with the effective tidal prism available. At this stage, the channel will remain stable, material being brought into the section from upstream being removed by the natural tidal currents. An estimate of the reduction in channel area for any proposed reclamation scheme can be obtained directly from Fig. 17.

The percentage reductions in channel areas for the proposed reclamations shown in Figure 24 are given in Table 4 and these show good agreement with the velocity changes predicted in Section 7.2.

Table 4: Estimated Reduction in Channel Areas Resulting from Proposed Reclamation Works (Using Tidal Prism - Channel Capacity Relationship)

Location	Approx. Reduction in Tidal Prism cu. ft. x10^6	Existing Channel Area $ft^2 x 10^4$	Reduction of Area $ft^2 x 10^4$	Pc. Reduction in Area
Captain Cook Bridge	34.3	5.63	0.60	11
Tom Ugly's Bridge	33.6	3.74	0.50	14
Kangaroo Pt.	32.9	2.65	0.30	12
Green Point	28.1	1.52	0.20	13
Como Bridge	27.5	1.27	0.08	6
Lugarno Ferry	12.9	0.77	0.02	3
Upstream of Salt Pan Creek		No change		

The mean percentage reduction in area predicted by the two methods is given in Table 5 and this is considered as the best estimate of the likely channel changes.

As no reclamation works have been proposed upstream of Salt Pan Creek, the tidal prism and consequently channel areas are unchanged. It is evident from Table 5 that the major effects of reclamation on channel capacities will be in the lower reaches where the bed materials are predominantly silts and clays and where the effective reduction in tidal prism is the cumulative total from all upstream works.

At most of the sections considered, the width of the channel is fixed by the present banks. Consequently the reduction in area will be effected almost entirely by a reduction in channel depth caused by siltation.

On this assumption, the reductions in water depths in the channel have been computed and the results are given in Table 5. These show that if the proposed reclamations are proceeded with, siltation could occur which would significantly reduce navigable depths.

Table 5: Estimated Reduction in Mean Channel Depths

Location	Pc. Reduction of Area (Mean from Tables 3 and 4)	Estimated reduction in water depth (ft.)
Captain Cook Bridge	9	2.6
Tom Ugly's Bridge	11	2.8
Kangaroo Point	11	2.8
Green Point	11	1.6
Como Bridge	8	1.3
Lugarno Ferry	4	0.4
Upstream of Salt Pan Creek	No Change	

7.4 Effect of Reclamation on Flood Conditions

Downstream of Como railway bridge the effect of flood flows on tidal levels is negligible. The decrease in channel depths (Table 5) which would follow large scale reclamation would have no measurable effect on flood or tide levels.

Upstream of the proposed reclamation works, the channel capacity and consequently flood conditions would be unchanged.

Between Salt Pan Creek and Como, flood flows have some effect on the high tide levels and consequently siltation of the channel would tend to increase flood levels slightly. The magnitude of this increase will be less than the reduction in channel depth or 0.4 ft. (see Table 5).

Consequently the proposed reclamation works would have no appreciable effect on flood levels in the main river channel.

7.5 Effect of Reclamation on Oyster Culture

7.51 Suspended Sediments

Oyster culture in the Georges River is an important fishery justifying some consideration when matters concerning the behaviour of the estuary are discussed.

Oysters survive by continually taking in water, this action being known as water pumping. 8 gallons an hour can be pumped by large oysters, but for Australian varieties, like those in the Georges River, 4 gallons an hour is a better estimate. The oyster extracts oxygen and food from this water while unwanted material such as silt is separated and secreted into a special section and flushed out periodically by a shell movement resembling a snapping action. Under normal conditions in sea water an oyster may cleanse itself once every 5 minutes. When the amount of suspended sediment increases, the pumping rate decreases and the snapping action occurs more frequently. Hence the oyster takes in less food and oxygen yet requires additional energy for cleansing and, as a result, growth is impeded. If the turbidity is too high and sustained for a period greater than 14 days, the oyster will die. Turbidities as low as 0.1 gms. per litre reduce the pumping rate of an oyster by about 50 pc. and increase considerably the frequency of the snapping cleansing action.

Therefore, where reclamation works are executed, precautions must be taken to ensure that the fill is suitably retained to prevent it from being taken into suspension and causing turbidities detrimental to nearby oyster leases. There have been instances in bays off the Georges River where fill material has oozed through retaining structures causing high local turbidities which have permanently damaged surrounding oyster growth.

7.52 Bed Accumulation of Sediments

The foregoing has concerned suspended sediments. The effect on oysters of increased shoaling by siltation of the river bed and foreshores also require consideration. This situation would arise where reclamation by dumping fill between high and low water reduced the existing tidal prism significantly.

It has been established that oysters can survive only within a vertical range approximating 18 inches near mean tide level. Such a band of oyster growth can be observed at low tide around any rocky foreshore. Oyster growers observe the level at which natural oysters grow best along any given foreshore and set their racks to suit. It has been previously pointed out that an oyster can feed only if covered with water because it needs to pump water through its system to extract food. Figure 25 illustrates that the oyster feeds for approximately 50 pc. of the time if located at mean tide level.

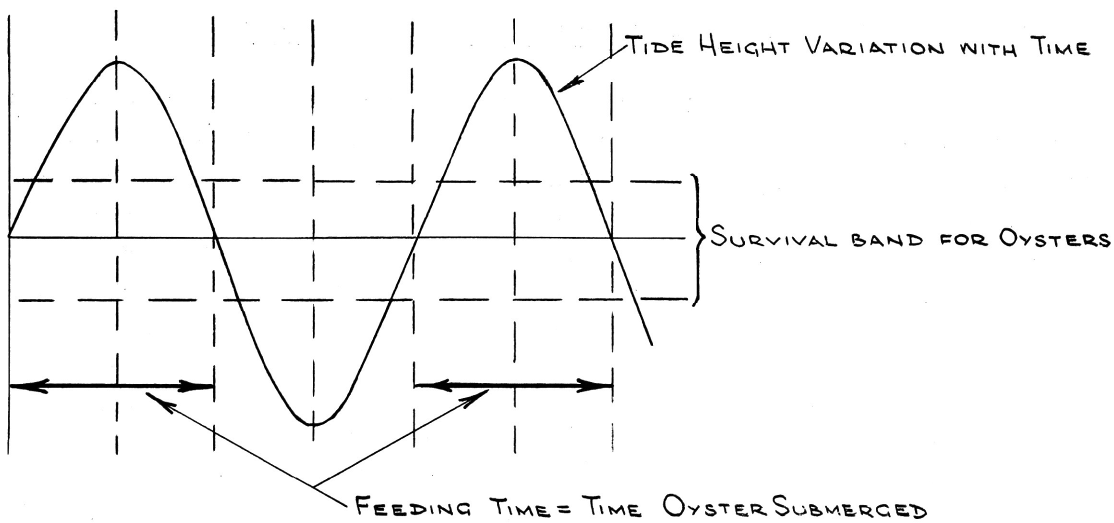

Fig. 25: Oyster Culture.

If, due to siltation, oyster racks are in danger of becoming submerged, the apparent solution is to raise the racks. However, it can be seen from Fig. 25 that this action will reduce the available feeding time, necessitating a longer maturing period resulting in increased growth period for the oyster. Raising an oyster rack 3 to 4 inches can increase the time required to mature by 20 pc. to

25 pc. It is also possible that raising the racks may lift the oysters completely out of the survival range and thereby render the oyster farm useless.

7.6 Effect of Reclamation on Pollution

The effect of reclamation on pollution is to increase the concentration of pollutants by reducing the amount of water available for flushing and dilution. The reduction in flushing and dilution potential depends on the reduction in both tidal prism and water volume below low water. As indicated in Section 6.6, insufficient survey data are available to make a reliable quantitative estimate of the increases in concentrations, but these could be expected to be of the order of the percentage tidal prism reduction over the reach affected by reclamation, that is 4 per cent for the current proposals.

7.7 Other Aspects of Reclamation

7.71 Runoff Discharge through Reclaimed Areas

Where portions of off-channel bays are reclaimed, precautions are necessary to see that storm runoff from the surrounding local catchment and its suspended load are transported through the reclaimed area in a satisfactory manner. Consider the hypothetical case in Fig. 26. Under natural conditions sediment carried by storm runoff is deposited in region "X" (Fig. 26a), since the material is not discharged into the main channel where velocities are high enough to transport it away. If an area is reclaimed as in Fig. 26b and the local drainage is as shown, deposition of sediments will occur in the area marked "Y", an area not previously affected. This change could be detrimental to activities or structures previously established in a sediment free zone. The problem could be reduced by discharging the local runoff directly into the tidal flushing zone of the main estuary channel (Fig. 26c).

7.72 Source of Material

Reclamation material may be derived from three sources:

(a) above high water,
(b) between high and low water,
(c) below low water.

When reclaiming areas necessitating the dumping of fill between high and low water the status quo can best be preserved if the resulting reduction in tidal prism is compensated by a corresponding increase in tidal prism by dredging from between high and low water an amount of material equal in volume to the tidal prism reduction. Material removed from below low water or above high water does not alter the tidal prism.

7.73 Flushing of Tidal Canals

Where tidal canals are constructed in conjunction with home site reclamation works, care must be taken to see that adequate flushing of sediments and pollutants occurs. There would appear to be two ways of ensuring this action.

(a) Provided the local drainage catchment is large enough, the canals may be designed so that storm runoff velocities are of sufficient magnitude to ensure periodic flushing.

(b) Where this cannot be done, the provision of a tidal storage pond at the upper end of the canal system will increase the volume of tidal inflow and outflow thus increasing the canal tidal velocities sufficiently for adequate flushing. Sediments will accumulate in the tidal pond and it may be desirable for the land surrounding the ponds to be set aside for purposes other than home building. Fig. 27 illustrates the principle.

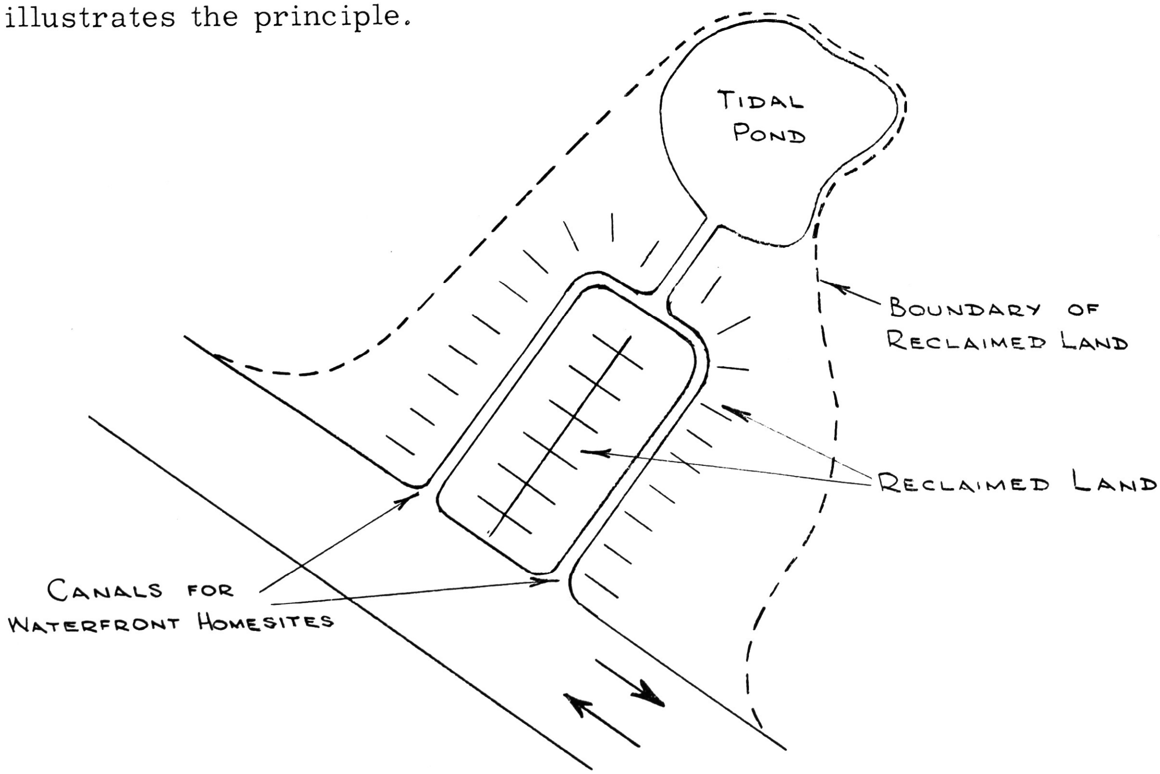

Fig. 27: Principle of Tidal Pondage

7.74 Bed Materials in Reclamation Areas

The bed materials of some reclamation areas may consist of flocculant silts and clays which will not support fill materials easily. As the fill is placed the silts and clays could be displaced further out into the waterway. This would need to be controlled.

7.75 Retention of Reclaimed Fill

Where reclamation works are executed using very fine material, the fill must be adequately retained to prevent it from oozing through the retaining structures and being taken into suspension. The oozing of fill can be precipitated by storm water runoff or by the differential pressures created in the fill by tidal variations of the water surface against the retaining bank.

8. Conclusion

8.1 Hydrology of Georges River Basin

In Section 5, flood estimates are given for several typical regions along the Georges River. These can be used for design of proposed works in these localities or for preliminary design in similar regions. Final design in areas not covered by the study would need further analysis.

8.2 River Pollution

There is no doubt that problems of pollution already exist in the Georges River and that these problems will increase in the future with industrial growth of the region. As no systematic data are at present available on fresh water inflows, salinity distributions and pollutant concentrations, a satisfactory appraisal of the problem cannot at the moment be undertaken.

8.3 Effect of Proposed Reclamation Works

The investigation of the Georges River Estuary has indicated that there is at present a balance between channel capacity and tidal storage. Figure 17 shows a clear relationship between channel area and the upstream tidal prism. Any major reclamation works which reduce the tidal storage can be expected to reduce the velocity of tidal flow, leading to siltation, until a new set of "equilibrium" conditions is attained. The reclamation areas which have been proposed are shown in Figure 24. Many of these are of such a scale that there is an appreciable reduction in the tidal volume. Using analytical methods, the reduction in velocity that would result after construction of the works, has been estimated at up to 10 per cent. This will lead to siltation during floods and a reduction in channel area until the original velocities are restored. For the reclamation works shown in Figure 24, it is estimated that channel areas will be reduced by a maximum of 10 per cent with an average reduction in depth of up to 2.8 feet. No estimate can, however, be given of the time over which these changes will take place. Tidal stages will be unaltered by the proposals. The changes in channel configuration will have no measurable effect on flood discharges and flood levels will not be significantly raised.

A second effect of the proposed reclamation works is that they will aggravate pollution problems in the estuary by reducing the tidal prism available for tidal flushing of the river. The flushing potential of the river above Salt Pan Creek has been estimated in Section 6.61 for various fresh water inflows. The results, however, are of little value until systematic measurements are made of fresh water inflows and pollutant concentrations under various river conditions. In the area of the proposed reclamation works, insufficient data on which to base a reliable estimate of the increase in pollutant concentrations are available. This could be expected to be of the order of the percentage tidal prism reduction or about 4 per cent. This is not considered to present an immediate problem.

The effect of the reclamation works on oyster culture cannot be readily assessed. The biggest danger is the likelihood of increased turbidities near the proposed reclamation areas during the construction period. Oyster growth is very susceptible to turbidity and if it is too high over a sustained period the oyster may die. It would be desirable to study carefully the effects of past reclamation works on nearby oyster farms to determine the magnitude of this problem. Large scale reclamation also tends to increase salinity stratification of the estuary which could have a secondary effect on the oyster farms. However, until salinity measurements have been taken for various river conditions, the importance of this factor cannot be determined.

9. Recommendations

9.1 Reclamation Works

Three types of reclamation works have been proposed:-

(i) Reclamation of areas substantially above high water.
(ii) Reclamation of small bays associated with minor bank straightening.
(iii) Large bay reclamation.

The first two of these proposals have little effect on the estuary as a whole but local problems to nearby oyster leases could result because of the increased turbidities during construction. With care it is considered that turbidities could be kept down to an acceptable level and the authors can see no technical reason for not continuing with these types of works. Reclamation proposals which increase tidal storage (such as by dredging of mud flats etc.) would also be technically acceptable.

The third proposal, involving reclamation of large bays, significantly reduces tidal storage available and the investigation has shown that the regime of the river would be measurably altered. The reclamation proposals shown in Figure 24 do not themselves upset the river regime to an extent that causes any real concern. Nevertheless it must be remembered that the effects of reclamation works on the tidal regime are cumulative, and already some large scale works have been carried out on the south side of the estuary and these have not been considered in this report. The effect of any single reclamation scheme will not fully develop for a long time and will be superimposed on the effects from other schemes. Some schemes will be more advantageous than others and these should be given preference. It is considered unwise to proceed with large scale schemes (the Lime Kiln Bay proposal for example) before first carrying out a study of the ultimate reclamation requirements for the estuary. From such a study a scheme could be selected which will best serve the requirements with minimum detrimental effects to the facilities provided by the river. An adequate programme of data collection must be completed before such a study could be undertaken and a sound engineering appraisal made of all the long term effects of reclamation works.

9. 2 The Need for Unified Study

It is clear that over the next fifty years very great development will occur in the Georges River Basin. Study is needed of the present and future requirements for navigation, land usage, reclamation, soil conservation, irrigation, channel improvement, flood mitigation, sewage and industrial waste disposal, control of bank erosion, fish and wild life preservation, oyster farming and the provision of recreational facilities in aesthetic surroundings. Many of these requirements are conflicting and there is an urgent need for a programme to co-ordinate the work of the various bodies concerned. At present, this probably could best be carried out by setting up a Committee representing the various interests to look at the overall development of the basin.

9. 3 Data Collection

The various Government Departments have from time to time collected data, but most of this is incomplete, unpublished and difficult to abstract. The system of individual Departments working independently is not conducive to economic data collection and there is an urgent need for the launching of a programme financed by all bodies concerned. For future studies, systematic data would be required of:-

(i) fresh water inflows for both dry weather and flood,
(ii) pollution and its distribution,
(iii) salinity distribution under various river conditions,
(iv) sediment inflow and distribution,
(v) complete hydrographic survey of bays as well as channels,
(vi) flood stages and hydrographs,
(vii) tidal stages and velocities.

Should a committee be established along the lines outlined in 8. 4, one of its first jobs should be to consider what data are required to serve the present and future needs for river basin planning and to outline the methods of obtaining this information.

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Appendix A.

Historical Information on Georges River

1796 - Bass and Flinders explore coast south of Sydney and ascend Georges River surveying its course. For 60 years after this event the only industry in the area was the collection of shell for lime burning, the shell being retrieved from large mounds the aborigines had built up along the foreshores. (Historical Refs. 1 and 2).

1797 - Govenor visits Botany Bay and Georges River and finds good soil on banks of Georges River suitable for cultivation and pasturage, together with the fact that the river is navigable for small craft at least 20 miles up. (Historical Ref. 3).

1809 - Liverpool area influenced by two floods rising more than 34 ft.

1810 - Liverpool founded on banks of Georges River by Governor Macquarie.

1836 - Liverpool Weir constructed at suggestion of Surveyor Lennox to supply the town with fresh water.

1843 - First regular punt service across Georges River established, worked by means of a tow rope. (Historical Ref. 4).

1860 - First use made of a dredge to obtain oyster shell for lime burning. Depths of up to 50 ft. were dredged. (Historical Ref. 5).

1866 - The Hon. Thomas Holt M. L. C. acquired a title from the crown to the tidal waters of Gawley Bay (180 acres) and Weeney Bay (370 acres) for the purpose of oyster farming. (Historical Refs. 6 and 7).

1870 - "Approximately 150 to 200 people consisting of members of parliament, judges, lawyers, merchants etc. travelled by boat up the Georges River to within 6 miles of Liverpool to gain first hand knowledge of the feasibility of damming the Georges River for Sydney's water supply. Suggested dam sites were Kangaroo Point and Tom Ugly's Point. The proposal was supported by the President of the Water

Commission, Professor Smith and Mr. John Young who was the contractor for the Sydney Exhibition Building and who had considerable experience as an engineer in embanking navigable rivers, constructing canals, weirs etc. in England. Mr. Young has been quoted as saying that, "It is quite practicable to construct a dam at Tom Ugly's or Kangaroo Point at moderate costs, and both safe and secure - . There is an absence of mud flats and swamps, cultivation or dwellings of any kind, or in fact anything to contaminate the water -." (Historical Ref. 8).

1870 - Removal of oysters from Georges River was prohibited for some years as they were threatened with extinction. This was owing to the dredge being used without thought of future supply and because the foreshore rock oysters were not given a chance to recuperate from early ravages of lime burners and continued depredation by the public. (Historical Ref. 5).

1885 - Como Railway Bridge opened.

1887 - Heavy freshets last so long that a deposit of mud 3" thick was left lying on all the natural beds in the river with the result that a large proportion of the oysters was smothered. (Historical Ref. 5).

1897 - Present day method of oyster farming commenced. That is the laying out of logs and sticks as spat collectors as distinct from rock cultivation. (Historical Ref. 5).

1913 - Continued freshets were the cause of considerable anxiety and labour to oyster farmers on account of the quantity of silt which was deposited on the beds, particularly those above Como. Large quantities of oysters were killed. (Historical Ref. 5).

1929 - Tom Ugly's bridge was opened.

1941 - Woronora Dam completed.

1965 - Captain Cook Bridge opened.

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Appendix B.

Flood Flow Calculations

Flood Peak Frequencies

Two different approaches have been used in this aspect of the study, the first involving the following:-

(a) The estimation of rainfall-dsicharge relationships for the various sub-catchments by the Clark-Johnstone synthetic unitgraph procedure.

(b) The routing of the sub-catchment unitgraphs through lower reaches of the river by the Muskingum method to produce unitgraphs at the points of interest.

(c) The application of rainfall-duration-frequency data to these unitgraphs to give estimates of flood peak frequencies.

The unitgraphs are also used in calculating the design hydrographs for various recurrence intervals.

There are some uncertainties in the above procedure concerning suitable loss rates and the reliability of the rainfall freqùencies for rare floods. The second approach is therefore desirable as an independent check on the estimates and involves the following:-

(a) The determination from the historical records of flood level frequencies at a suitable point.

(b) The calculation of discharges corresponding to these levels by the Manning Formula using cross sections and flood slopes from data supplied by the Public Works Department.

There are also uncertainties with this procedure, mainly in selecting a roughness coefficient n, but the combined use of both approaches should provide the most reliable estimates possible under these circumstances. Peak times of travel and level relationships for the estimation of routing constants and other parameters required for the calculations have been derived from flood data made available by the Public Works Department. These relationships are shown in Figures 28 and 29.

Unitgraph Synthesis

Complete Clark-Johnstone synthesis of unitgraphs was carried out for each of the following major subcatchments:

Upper Georges River (above Glenfield)	-	97.5 sq. miles
Bunbury Curran Creek		35.0 sq. miles
Cabramatta Creek		28.9 sq. miles
Prospect Creek		34.6 sq. miles

The parameters C (maximum travel time) and K (average storage delay time) were estimated by using the Clark-Johnstone formulae modified for N. S. W. conditions as determined by Cordery (1967).

Approximate unitgraphs were synthesized for the following minor sub-catchments:-

Glenfield - Liverpool local inflow	5.6 sq. miles
Liverpool - Chipping Norton local inflow	5.9 sq. miles
Chipping Norton - Georges Hall " "	1.3 sq. miles
Georges Hall - Milperra " "	2.5 sq. miles
Williams Creek	20.7 sq. miles
Milperra - East Hills local inflow	6.6 sq. miles

For these unitgraphs, C was estimated by a method due to McIllwraith (Institution of Engineers, Australia 1958) and K was assumed to be a value between 0.4C and 0.8C, depending on the catchment shape and channel storage characteristics.

The time-area diagrams were assumed to be rectangular with bases equal to C.

The lower section of the main river was divided into five reaches and the above unitgraphs were routed through these reaches using values of K as derived from Fig. 28. The Muskingum parameter x was assumed to be 0.25 for the lower reaches and 0.30 for the Glenfield-Liverpool and Mulgoa Road-Chipping Norton reaches.

The final unitgraphs from the above procedure represent the hydrographs resulting from one inch of excess rainfall in one hour distributed uniformly over the entire catchment, and are given by Fig. 30.

Critical Durations

An "S" curve technique was used to determine the critical flood-producing rainfall durations from the previously derived unitgraphs. These durations were found to be:

(a) 16 hours for Georges River at Milperra and East Hills
(b) 7 hours for Prospect Creek at Georges Hall
(c) 5 hours for Cabramatta Creek at Chipping Norton.

Rainfall Losses

Studies show that the average wet spell on the Georges River continues for 3 to 4 days with an average rainfall loss of about one inch. This represents a loss rate of approximately .015 inch per hour which is an appropriate value for design purposes when rainfall durations are relatively long.

For short-duration storms, a study by Laurenson and Pilgrim (1963) of loss rates on a large variety of catchments has shown that losses are usually within the range of .02 to .12 inches/hr. From other studies (Pilgrim 1966) it is also evident that loss rates tend to be higher with shorter rainfall durations and lower when catchment surfaces are relatively impervious. The following average values have therefore been adopted:-

(a) Georges River at East Hills and Milperra:

12 hours' rainfall duration	.4 ins. total loss
16 " " "	.5 " " "
24 " " "	.6 " " "
72 " " "	1.0 " " "

(b) Prospect Creek at Georges Hall:

7 hours' rainfall duration	.2 ins. total loss
12 " " "	.3 " " "
24 " " "	.4 " " "
72 " " "	.8 " " "

(c) Cabramatta Creek at Chipping Norton:

5 hours' rainfall duration	.2 ins. total loss
8 " " "	.3 " " "
24 " " "	.6 " " "
72 " " "	1.0 " " "

These have been deliberately selected on the conservative side as explained below.

Calculation of Peak Flows by Unitgraph-Rainfall Frequency Method

Rainfalls for the various critical durations were obtained for frequencies of 2 years, 20 years, 50 years and 100 years. The adopted losses were subtracted from these values to give volumes of excess rainfall, and this indicated the appropriate multiplying factors to apply to the peak unitgraph ordinates. The resulting peak flows may be assumed to have the same frequencies as the rainfall volumes.

It was not considered necessary to allow directly for the effects of varying rainfall intensities within the critical durations, although there are methods available for doing this. Previous experience has suggested that neglecting such variations causes peak flows to be underestimated usually by about 5 pc. which can be indirectly allowed for by selecting conservative values of rainfall losses, as has been done here.

The consequent simplification in calculations appears justified in this particular study but it should be kept in mind as a possible source of small errors. Fig. 8 shows the peak flows at Milperra for various frequencies by this method.

Calculation of Peak Flows by Manning Formula

A cross section 4,500 feet upstream of Milperra Bridge at "Kentucky" was selected as suitable for the calculations because:-

(a) The water is confined to a reasonably narrow section with a minimum of shallow flood-plain flows.

(b) Pondage is not excessive and velocities would be relatively uniform.

(c) The section is fairly representative of the reach between Milperra and Georges Hall for which flood slopes are available.

(d) It is close to Milperra where peak flows have been estimated by the unitgraph-rainfall frequency method.

The appropriate value of roughness, n, would probably be between .03 and .04 for most flood flows, and discharge curves corresponding to these two values have been prepared from the data supplied by the Public Works Department.

From the unitgraphs for Milperra and Georges Hall (Fig. 30), the peak discharge at Milperra is estimated at 98 pc. of the peak discharge at "Kentucky". The water level relationships are given by Fig. 29. Discharge curves at Milperra shown in Fig. 3, are therefore readily derived from those at "Kentucky".

The observed flood level frequencies have been calculated in Table 2 and the corresponding discharge frequencies have been plotted on Fig. 8 where they are directly compared with the estimates of the previous section.

Tidal Effects on Flood Slopes

As the reaches below Liverpool are tidal, it was necessary to consider the significance of tidal effects on the previous calculations. It was noted in the historical records that the peaks of 1873, 1889 and 1956 all occurred one or two hours after high tide and the recession flows were therefore possibly retarded.

The tidal patterns for a number of recent floods were calculated from detailed data at Fort Denison, allowing for the lag between this station and Georges River ($1\frac{1}{2}$ to 3 hours). The plotted points of Fig. 29 are consequently labelled H or L depending on whether the flood peaks occurred near high or low tide. No marked variations are apparent due to this factor, although lower floods would undoubtedly be affected to some extent.

Comparison between Frequency Estimates

Fig.8 shows good agreement, up to a recurrence interval of 30 years, between the two methods of estimating flood frequencies. For longer recurrence intervals, however, the Manning formula estimates are significantly higher than the unitgraph-rainfall estimates.

There is considerable evidence to suggest that the rainfall frequency data are unreliable in this range, for example:

(a) The tables of heavy rainfalls available (Bureau of Meteorology, 1948) which show greater values for the Georges River than would be expected from the rainfall frequency data.

(b) The envelope curve of maximum floods for N.S.W. The rainfalls corresponding to these floods are also greater than would be expected from the rainfall frequency data.

(c) The February 1956 flood, when approximately 10 inches of rain fell on the Georges River in 17 hours. According to the rainfall frequency data this would occur only about once in 70 years but the 1956 flood has, in fact, been equalled or exceeded 4 times in 100 years.

An examination of the methods used to derive the rainfall frequency data suggests several possible reasons for their unreliability:

(a) The basic assumptions used for extrapolating the data from capital cities to rural areas are suspect for N.S.W. conditions.

(b) The 16 hour rainfalls are interpolated between assumed 5 minute intensities and observed 3 day rainfalls and there is little logical justification for the mathematical form of the interpolation. 24 hour rainfalls should have been used, as in the corresponding Victorian data.

(c) The rainfall frequency data are based on records between 1910 and 1953. Relatively few floods occurred in this period and it is a poor time sample for the estimation of rare events.

Although the Manning formula calculations appear to provide better estimates of the frequencies of rare floods, the unitgraph-rainfall frequency method is superior for the smaller floods, as many of these have not been reported in the historical records. The adopted frequency values are therefore given by the full line in Fig. 8. Gradually varying values of n were assumed for the larger floods, depending on the cross-section characteristics.

Design Hydrographs

The flood peak frequency study indicated that the rainfall-frequency data for the Georges River should be adjusted as follows:-

Up to 20	year	recurrence	interval	no adjustment	
20 to 30	"	"	"	multiply by	1.09
30 to 40	"	"	"	" "	1.22
40 to 50	"	"	"	" "	1.32
50 to 100	"	"	"	" "	1.64

Using these adjusted values of rainfall for the critical durations and the losses given, design hydrographs were calculated for East Hills, Prospect Creek and Cabramatta Creek, as shown in Figures 9, 10 and 11 respectively.

The relative contributions to the East Hills hydrograph from the Shale and Sandstone areas are given in Fig. 6.

Appendix C

Tidal Stage and Velocity Calculations

Theoretical Considerations

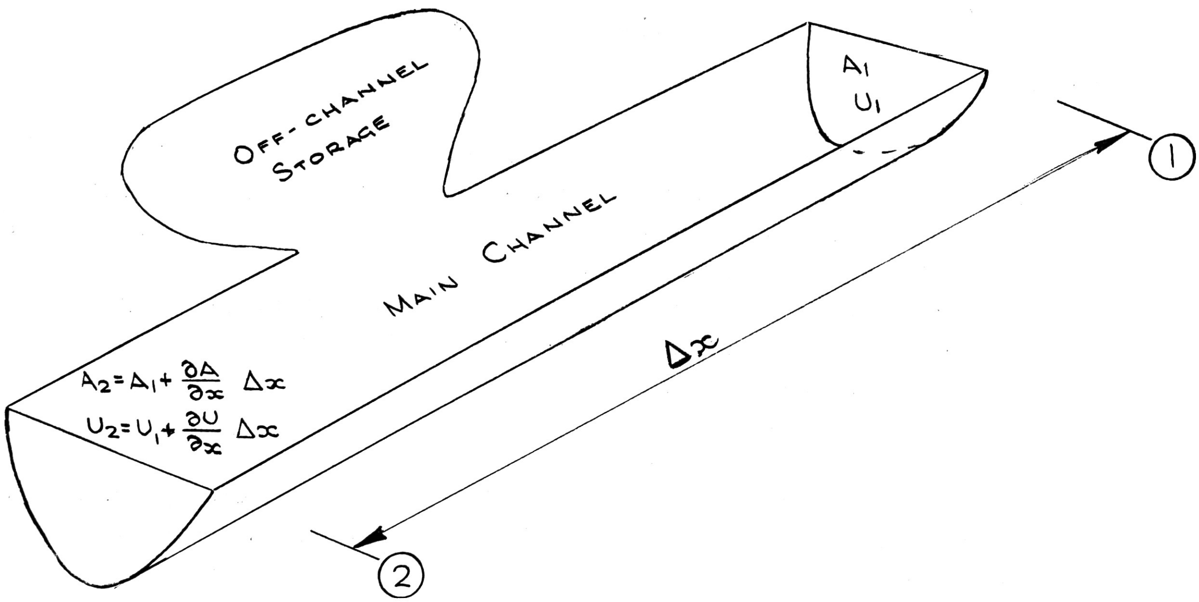

Fig. 32: Estuary Schematization.

For the finite interval shown in Figure 33, the basic equations to be satisfied are:-

(i) Continuity

$$\frac{\partial[AU]}{\partial x} + B\frac{\partial Z}{\partial t} = 0 \qquad (1)$$

(ii) Motion

$$\frac{\partial U}{\partial t} + g\frac{\partial Z}{\partial x} + g\frac{|U|U}{C^2R} \qquad (2)$$

where

A = cross sectional area of the channel carrying the tidal flow
B = mean surface width of the estuary = $A_s/\Delta x$ where A_s is the surface area between section (1) and section (2)
U = mean channel velocity
Z = surface displacement relative to mean surface level
R = hydraulic mean radius of the tidal channel
C = Chezy resistance coefficient
Δx = length of reach

The solution of these equations can be simplified by linearizing the friction term. If it is assumed that the friction slope (S_e) can be written in the form

$$S_e = \frac{U^2}{C^2R} = \frac{MU}{g}$$

equation (2) can be rewritten as

$$\frac{\partial U}{\partial t} + g\frac{\partial Z}{\partial x} + MU = 0 \qquad (3)$$

For a constant value of the linearized friction coefficient M and assuming simple harmonic motion solutions, equations (1) and (3) have the known general solutions.

$$\begin{aligned} Z(x) &= a(x)\cos\left[\sigma t - \delta(x)\right] \\ &= a(x)\left[\cos\sigma t \cdot \cos\delta(x) + \sin\sigma t \cdot \sin\delta(x)\right] \\ &= z(x,1)\cos\sigma t + z(x,2)\sin\sigma t \end{aligned} \qquad (4)$$

$$\begin{aligned} U(x) &= b(x)\sin\left[\sigma t - \theta(x)\right] \\ &= b(x)\left[\sin\sigma t \cdot \cos\theta(x) - \cos\sigma t \cdot \sin\theta(x)\right] \\ &= u(x,1)\sin\sigma t - u(x,2)\cos\sigma t \end{aligned} \qquad (5)$$

where $Z(x)$ = surface displacement at chainage x
$U(x)$ = mean velocity at chainage x
$a(x)$ and $b(x)$ = tidal amplitudes at chainage x
$\delta(x)$ and $\theta(x)$ = tidal phases at chainage x
σ = angular velocity of the wave
$= \frac{2\pi}{T}$ where T = wave period $\approx$ 12.42 hours

Substituting equations (4) and (5) into equation (1) and replacing the derivatives by finite differences we have

$$\left.\begin{aligned}\Delta(A u_{(x,1)}) &= \sigma B z_{(x,1)} \Delta x \\ \Delta(A u_{(x,2)}) &= \sigma B z_{(x,2)} \Delta x\end{aligned}\right\} \qquad (6)$$

and by performing a similar substitution in equation (3)

$$\left.\begin{aligned}\Delta z_{(x,1)} &= \frac{1}{g}\left(-\sigma u_{(x,1)} + M u_{(x,2)}\right)\Delta x \\ \Delta z_{(x,2)} &= \frac{1}{g}\left(-\sigma u_{(x,2)} + M u_{(x,1)}\right)\Delta x\end{aligned}\right\} \qquad (7)$$

If the magnitude of $z_{(x,1)}$, $z_{(x,2)}$, $u_{(x,1)}$ and $u_{(x,2)}$ are known at a section in a tidal estuary, equations (6) and (7) may be solved by finite difference methods to obtain the variation of stage and velocity throughout the estuary provided the friction coefficient M can be estimated.

Evaluation of Friction Coefficient M

To evaluate M the following assumptions are necessary:-

(i) The Chezy resistance coefficient is constant for all velocities.

(ii) The variation of velocity with time is a harmonic function

$$\text{i.e.}\quad U = U_{max.} \cos \sigma t$$

(iii) The friction term can be linearised on the basis that the work done by friction over a tidal cycle per pound of fluid is the same whether evaluated by the quadratic resistance relationship or by a substitute linear approximation.

(iv) The friction coefficient M is a constant for all velocities.

Now the work done by friction within the element Δ x per pound of fluid is

$$W = \int \frac{\tau_o P \Delta x U}{\gamma A \Delta x} . \partial t$$

$$= \int \frac{\tau_o}{\gamma R} . U \partial t$$

where τ_o = bed shear stress

P = wetted perimeter

γ = specific weight of fluid

and by assumption (iii)

$$\int_0^{\frac{\pi}{2}} \frac{gU^2}{C^2 R} . U \partial t = \int_0^{\frac{\pi}{2}} (MU) U \partial t$$

substituting $U = U_{max.} \cos \sigma t$ and solving for M

$$M = \frac{g U_{max}}{C^2 R} . \frac{\int_0^{\frac{\pi}{2}} \cos^3 \sigma t \, \partial t}{\int_0^{\frac{\pi}{2}} \cos^2 \sigma t \, \partial t}$$

$$\text{or } M = \frac{8 g U_{max}}{3 \pi C^2 R} \qquad (8)$$

Steps in the Solution

1. Firstly, the estuary must be schematized by dividing the estuary into channels which convey water, and sections (such as shoals, dead end bays etc.) which store water on the rising tide and release it on the ebb.

2. The channel section is broken down into finite intervals along the axis of the estuary as shown in Figure 24.

3. From observations of velocities, values of $U_{(1,1)}$ and $U_{(1,2)}$ are

calculated from equation (5). Tidal velocities observed at Lugarno Ferry and Tom Ugly's Bridge are shown in Fig. 15 from which values of $u_{(x,1)}$ and $u_{(x,2)}$ were estimated. These values and the resultant plot are also shown in Fig. 5.

4. From observations of stage, values of $z_{(1,1)}$ and $z_{(1,2)}$ are calculated from equation (4). Stage variations observed at Lugarno Ferry and Tom Ugly's Bridge are shown in Figure 16 from which values of $z_{(x,1)}$ and $z_{(x,2)}$ were estimated. These values and the resultant plot are also shown in Fig. 16. It is to be noted that for the short term records the mean water surface at these stations was calculated at slightly less than RL. 100, the recognised mean sea level. Nevertheless, the short term value was adopted because:-

(i) we are only approximating the tidal wave to a sine wave;

(ii) for computation we are interested only in surface level deviations from the mean and not absolute values;

5. From the value of ($U_{max}(x) = \sqrt{u^2_{(x,1)} + u^2_{(x,2)}}$ the magnitude of M is calculated from equation (8) using an appropriate value of Chezy coefficient C.

6. Using equation (6) values of $u_{(x,1)}$ and $u_{(x,2)}$ are evaluated along the estuary.

7. Using equation (7) values of $z_{(x,1)}$ and $z_{(x,2)}$ are evaluated along the estuary.

This procedure has been carried out for the Georges River estuary using a digital computer (see Table C1 for programme) for a range of Chezy resistance coefficients.

Application of Theory

The overall method of application to the Georges River was as follows:-

(a) From the two sets of field data obtained from Tom Ugly's Bridge and Lugarno Ferry, values of maximum tidal velocity (U_{max}) and maximum water surface variation from the mean (Z_{max}) were calculated at other selected cross sections along the Georges River using a range of Chezy C values from 90 to 150. A suitable value of C was then selected for further investigation. The locations of the cross sections selected are shown in Fig. 24 and a summary of the survey data required for computation is shown in Table C2. Initially it was intended to use the field data from Tom Ugly's Bridge for the estuary reach below cross section 14 and the field data from Lugarno Ferry for the estuary reach above cross section 15 thus eliminating the necessity of schematising the Woronora River into a reach of the Georges River between cross sections 14 and 15. The value of C selected for further investigation would be that which produced a good correlation of velocities and tide heights at the Woronora Junction. It can be appreciated that while a continuity of tidal elevation must be expected between these points a continuity of maximum tidal velocities need not be expected.

(b) Having selected this value of C the computer input data was modified to account for tidal storage reductions that will be caused by proposed reclamation works and values of U_{max} and Z_{max} again computed. The two sets of results corresponding to pre-reclamation and post-reclamation periods were then studied to see if any undesirable effects resulted. For example, if velocities were reduced significantly it could be assumed that siltation and shoaling of the estuary would increase.

Results

Results for Part (a).

The results for the procedure outlined in Part (a) above are shown in Table C3. From this table, it can be seen that values of maximum tidal elevation were more sensitive to variation of C than were values of maximum tidal velocity. C = 150 gave good continuity of tidal elevations between sections 14 and 15 while values of maximum tidal velocity were unreconcilable in every case. These sections are similar in area but a velocity was obtained upstream of the junction four times greater than that below the junction. It can be seen that no sensible selection of Chezy coefficient would improve the position. For the 7 values used, no alteration in velocities could be produced but rather remained constant at 0. 47 feet per second for Section 14 and 1. 85 feet per second at Section 15.

Consequently, the Woronora River was schematised into a reach of the Georges River between Sections 14 and 15. Two sets of values for tidal velocities and tidal elevations were computed for the whole estuary using first the data collected at Tom Ugly's Bridge and then that collected at Lugarno Ferry. These results, for C = 150, are shown in Table C4.

Point velocity measurements were then taken of maximum tidal velocities at various points along the Georges River to determine:-

(i) if any gross errors occurred in the original field data;

(ii) which set of field data produced computed results nearest actual conditions.

These point velocities, shown in Table C4 cannot be expected to yield estimates of tidal velocities anywhere near as accurate as those determined from comprehensive field studies previously executed at Lugarno and Tom Uglys with velocities determined from measurements over an entire cross-section, but can be relied upon to give good approximations.

A comparison of the computed tidal elevations with Fig. 7, which indicates there is little variation in tidal range for the whole estuary, shows that the results derived from Lugarno Ferry data are the most satisfactory. The point velocity readings indicate that no gross errors occurred in the original velocity data collection. Further they indicate that velocities upstream and downstream of the Woronora Junction should be approximately the same and both sets of data possess this characteristic. However, when overall comparison is made of the magnitude of computed maximum velocities and the magnitude of point velocity readings, those results derived from Lugarno Ferry data give a better agreement. Nevertheless, they could be considered as reasonably accurate only as far down as cross section 13. Below section 13 the river contains some extremely tortuous bends with many large off-channel bays such that the schematisation adopted may not yield a satisfactory approximation to real conditions. Further, eddy losses may be significant in this reach. These factors would explain unreconcilable velocity results obtained at the Woronora Junction when working upstream from Tom Uglys and downstream from Lugarno.

Therefore, all further computation was based on the Lugarno Ferry data and even though it cannot be relied upon to yield good quantitative results downstream of Section 13, qualitative comparisons such as pc. changes in velocity are possible in part (b) of the analysis.

Results for Part (b)

The proposed reclamations are shown in Fig. 24. Where large areas between high and low water are reclaimed, either with material dredged from below low water level or imported from elsewhere, the tidal storage is correspondingly reduced. Therefore, for a rising tide the volume of inflowing water is less and for an ebb tide the volume of outflowing water must also be reduced. Since the period of the tidal cycle remains unchanged, it can only be expected that tidal velocities will be smaller.

The proposed reclamations all occur below section 20, indicating that the tidal characteristics above this point are unchanged, because tidal storage upstream is not altered. Hence, commencing with the values of u_1, u_2, z_1, z_2, U_{max} and Z_{max} computed for cross section 20 for existing conditions from the Lugarno Ferry data, new values of tidal velocities and elevations were computed for the estuary downstream of section 20 using input data which accounted for the tidal storage reduction produced by reclamation. This change in the input data is done by altering the values of B, the average river surface width between sections. These adjusted values of B are shown in Table C2, while the new computed results are shown in Table C5, together with the original values for comparison purposes.

Sample Tidal Dynamics Computation (as executed with the aid of a digital computer).

Calculations for Estuary Reach between Sections 6 and 9 (see Figure 24).

(a) Table C2 shows that the following basic data were derived from field observations taken at Tom Ugly's Bridge (Cross Section 6).

$u(6,1) = -0.40$; $u(6,2) = +0.67$; $z(6,1) = -0.13$; $z(6,2) = -1.55$

$$(U_{max})_6 = \sqrt{(-0.40)^2 + 0.67^2} = 0.78 \text{ ft/sec.}$$

$$(Z_{max})_6 = \sqrt{(-0.13)^2 + (-1.55)^2} = 1.55 \text{ ft.}$$

(b) M_{6-9} (i.e. for reach between Sections 6 and 9) $= \dfrac{8g\,(U_{max})_6}{3\pi C^2 R_{6-9}}$

R_{6-9} = Average hydraulic radius of reach = 21.55 ft. from Table C2

C = Chezy coefficient = 100 (assumed)

$$\therefore M_{6-9} = \frac{8 \times 32.2 \times 0.78}{3\pi \times 10^4 \times 21.55} = 0.99 \times 10^{-4}$$

(c) $$\Delta(u_1) = \frac{\sigma B_{6-9}\, z(6,1) \Delta x}{A_{6-9}}$$

B_{6-9} = average surface width of reach between sections 6 and 9
= 2580 ft. from Table C2

A_{6-9} = average cross section area of reach between Sections 6 and 9
= 31,900 square ft. from Table C2

Δx = length of reach and is +ve in the upstream direction
= +6950 feet

$\sigma = \dfrac{2\pi}{T} = 1.406 \times 10^{-4}$ sec.$^{-1}$ where T = period of tide cycle $\approx$ 12.42 hrs. = 44,712 secs.

$$\therefore \Delta(u_1) = \frac{1.406 \times 10^{-4} \times 2580 \times (-0.13) \times 6950}{31,900} = -0.01$$

$$\therefore u(9,1) = u(6,1) + \Delta(u_1) = -0.40 - 0.01 = -0.41$$

Similarly $\Delta(u_2) = \sigma \dfrac{B_{6-9} \; z_{(6,2)} \; \Delta x}{A_{6-9}} = -0.12$

$\therefore \; u_{(9,2)} = u_{(6,2)} + \Delta(u_2) = +0.67 - 0.12 = +0.55$

$(U_{max})_9 = \sqrt{(-0.41^2) + (0.55)^2} = 0.68$ ft/sec.

(d) $\Delta(z_1) = \dfrac{1}{g}\left[-\sigma u_{(6,1)} + M_{6-9}\, \ddot{u}_{(6,2)}\right] \Delta x$

$= \dfrac{1}{32.2}\left[(-1.406 \times 10^{-4}) \times (-0.40) + 0.99 \times 10^{-4} \times 0.67\right] 6950$

$= +0.02$

$\therefore z_{(9,1)} = z_{(6,1)} + \Delta(z_1) = -0.13 + 0.02 = -0.11$

Similarly $\Delta(z_2) = \dfrac{1}{g}\left[-\sigma u_{(6,2)} - M_{6-9}\; u_{(6,1)}\right] \Delta x = -0.02$

$\therefore \; z_{(9,2)} = z_{(6,2)} + \Delta(z_2) = -1.55 - 0.02 = 1.57$

$\therefore \; (Z_{max})_9 = \sqrt{(-0.11)^2 + (1.57)^2} = 1.57$ ft.

TABLE C1.

```
C       PROGRAMME FOR TIDAL DYNAMICS IN A REAL ESTUARY WORKING FROM ONE
C       STATION WHERE TIDE HEIGHT V'S TIME AND TIDAL VELOCITY V'S TIME
C       ARE KNOWN
C       A=CROSS SECTION AREA OF REACH=(A1+A2)/2
C       B=SURFACE WIDTH OF REACH=(SURFACE AREA OF REACH)/DELX
C       DELX=LENGTH OF REACH
C       R=HYDRAULIC RADIUS OF REACH=(R1+R2)/2
C       UX(I,1) AND UX,I,2) ARE CONSTANTS IN HARMONIC EQUATION FOR TIDAL
C       VELOCITY AT ANY ONE STATION
C       ZX(I,1) AND ZX(I,2) ARE CONSTANTS IN HARMONIC EQUATION FOR TIDAL
C       ELEVATION AT ANY ONE STATION
C       UM=MAXIMUM TIDAL VELOCITY
C       ZM=MAXIMUM TIDAL VARIATION FROM MEAN SURFACE LEVEL
C       C=CHEZY CONSTANT
C       S=COEFFICIENT OF LINEAR FRICTION
C       L=NUMBER OF FIRST REACH AND FIRST CROSS SECTION
C       N=NUMBER OF REACHES=NUMBER OF CROSS SECTIONS MINUS ONE
C       K=NUMBER OF CHEZY C VALUES TO BE TRIED
      DIMENSION A(16),B(16),DELX(16),R(16),UX(17,2),UM(17),ZX(17,2)
      DIMENSION ZM(17),C(7),S(16)
      L=1
      N=16
      READ (1,13)(C(K),K=1,7)
      READ (1,5)(A(I),I=1,11)
      READ (1,50)(A(I),I=12,N)
      READ (1,14)(B(I),I=1,11)
      READ (1,51)(B(I),I=12,N)
      READ (1,15)(DELX(I),I=1,11)
      READ (1,52)(DELX(I),I=12,N)
      READ (1,16)(R(I),I=1,11)
      READ (1,53)(R(I),I=12,N)
      READ (1,6)UX(1,1),UX(1,2),ZX(1,1),ZX(1,2)
      READ (1,22)UM(1)
   17 DO 10 K=1,7
    8 DO 7 I=L,N
      ZM(I)=SQRT(ZX(I,1)**2+ZX(I,2)**2)
      UM(I)=SQRT(UX(I,1)**2+UX(I,2)**2)
      S(I)=(8.0*32.2*UM(I))/(3.0*3.1416*C(K)**2*R(I))
      UX(I+1,1)=UX(I,1)+(1.406E-4*B(I)*ZX(I,1)*DELX(I))/A(I)
      UX(I+1,2)=UX(I,2)+(1.406E-4*B(I)*ZX(I,2)*DELX(I))/A(I)
      ZX(I+1,1)=ZX(I,1)+((-1.406E-4*UX(I,1)+S(I)*UX(I,2))*DELX(I))/32.2
    7 ZX(I+1,2)=ZX(I,2)+((-1.406E-4*UX(I,2)-S(I)*UX(I,1))*DELX(I))/32.2
      WRITE(3,12)C(K)
      WRITE(3,23)(DELX(I),I=L,N)
      WRITE(3,24)(UX(I+1,1),I=L,N)
      WRITE(3,25)(UX(I+1,2),I=L,N)
      WRITE(3,26)(UM(I+1),I=L,N)
      WRITE(3,27)(ZX(I+1,1),I=L,N)
      WRITE(3,28)(ZX(I+1,2),I=L,N)
      WRITE(3,29)(ZM(I+1),I=L,N)
   10 CONTINUE
   13 FORMAT (7F6.0)
    5 FORMAT (11F7.0)
   50 FORMAT (5F7.0)
   14 FORMAT (11F7.0)
   51 FORMAT (5F7.0)
   15 FORMAT (11F7.0)
   52 FORMAT (5F7.0)
   16 FORMAT (11F6.2)
   53 FORMAT (5F6.2)
    6 FORMAT (4F10.3)
   22 FORMAT (F5.2)
   12 FORMAT(2H C//F10.0)
   23 FORMAT(8H DELX(I)//16F7.0)
   24 FORMAT(10H UX(I+1,1)//16F6.2)
   25 FORMAT(10H UX(I+1,2)//16F6.2)
   26 FORMAT(8H UM(I+1)//16F6.2)
   27 FORMAT(10H ZX(I+1,1)//16F6.2)
   28 FORMAT(10H ZX(I+1,2)//16F6.2)
   29 FORMAT(8H ZM(I+1)//16F6.2)
      STOP
      END
```

Table C2.

Summary of Survey Data

Cross Sect. No. (See Fig. 24)	Cross Sect. Area (ft.^{2})	Av. Cross Sect. Area for Reach (ft.^{2})	Hyd. Rad. of Cross Sect. (ft.)	Av. Hyd. Rad. for Reach (ft.)	Length of Reach (ft.)	Average Surface Width of Reach (ft.)	
						Before Reclamation	After Reclamation
1	35,800	46,100	9.96	16.23	6,240	12,300	12,300
4	56,400	46,900	22.50	24.60	4,080	9,450	9,390
6	37,400	31,900	26.70	21.55	6,950	2,580	2,560
9	26,500	21,800	16.40	14.30	4,400	5,500	5,250
11	17,100	16,200	12.20	11.30	2,800	1,600	1,600
13	15,300	14,000	10.40	12.68	1,600	2,720	2,620
14	12,700	12,400	14.96	11.92	1,600	12,800	12,800
15	12,000	10,900	8.88	8.65	6,200	2,840	2,320
16	9,850	8,780	8.42	9.71	3,000	1,150	1,110
17	7,710	7,830	11.00	11.20	6,000	1,060	910
19	7,960	7,110	11.40	9.37	3,800	3,720	3,120
20	6,260	7,300	7.35	12.67	11,800	1,200	1,200
22	8,340	6,170	18.00	14.25	5,600	327	327
24	4,000	3,120	10.50	8.80	9,800	398	398
26	2,230	2,700	7.09	8.50	5,200	1,290	1,290
27	3,180	3,088	9.90	10.80	12,200	294	294
29	3,000	2,100	11.70	9.37	17,400	264	264
31	1,200	990	7.05	6.37	13,400	163	163
33	783		5.70				

Table C3.

Computed Values of U_{max} and Z_{max} for a Range of C Values

Cross Sect. No. (see Fig. 24)	C = 90		C = 100		C = 110		C = 120		C = 130		C = 140		C = 150		Remarks
	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	$U_{max.}$ f. p. s.	$Z_{max.}$ ft.	
1	1.25	1.54	1.25	1.54	1.25	1.53	1.25	1.53	1.25	1.53	1.25	1.53	1.25	1.53	Using
4	0.93	1.55	0.93	1.54	0.93	1.54	0.93	1.54	0.93	1.54	0.93	1.54	0.93	1.54	Tom
6*	0.78	1.55	0.78	1.55	0.78	1.55	0.78	1.55	0.78	1.55	0.78	1.55	0.78	1.55	
9	0.68	1.56	0.68	1.56	0.68	1.57	0.68	1.57	0.68	1.57	0.68	1.57	0.68	1.57	Uglys'
11	0.52	1.56	0.52	1.56	0.52	1.57	0.52	1.57	0.52	1.57	0.52	1.58	0.52	1.58	
13	0.49	1.56	0.49	1.56	0.49	1.57	0.49	1.57	0.49	1.57	0.49	1.58	0.49	1.58	Field
14	0.46	1.56	0.46	1.56	0.47	1.57	0.47	1.57	0.47	1.57	0.47	1.58	0.47	1.58	Data
15	1.85	1.72	1.85	1.68	1.85	1.66	1.85	1.64	1.85	1.62	1.85	1.61	1.85	1.60	
16	1.56	1.61	1.56	1.60	1.57	1.60	1.57	1.59	1.57	1.59	1.57	1.59	1.57	1.58	Using
17+	1.50	1.57	1.50	1.57	1.50	1.57	1.50	1.57	1.50	1.57	1.50	1.57	1.50	1.57	Lugarno
19	1.37	1.52	1.37	1.54	1.37	1.55	1.37	1.56	1.37	1.56	1.37	1.57	1.37	1.57	
20	1.10	1.48	1.10	1.51	1.10	1.53	1.11	1.54	1.11	1.55	1.11	1.56	1.11	1.57	Ferry
22	0.94	1.42	0.95	1.46	0.95	1.49	0.96	1.51	0.97	1.53	0.97	1.55	0.97	1.56	Field
24	0.92	1.40	0.93	1.44	0.94	1.47	0.95	1.50	0.96	1.52	0.96	1.54	0.97	1.55	
26	0.88	1.31	0.91	1.37	0.93	1.41	0.95	1.45	0.96	1.48	0.97	1.50	0.97	1.52	Data
27	0.98	1.26	1.03	1.32	1.08	1.37	1.11	1.41	1.13	1.44	1.15	1.47	1.17	1.49	
29	1.08	1.14	1.15	1.21	1.20	1.26	1.24	1.31	1.28	1.35	1.30	1.38	1.32	1.41	
31	1.31	0.90	1.40	0.99	1.48	1.06	1.54	1.12	1.58	1.17	1.62	1.21	1.64	1.25	
33	1.53	0.66	1.66	0.78	1.75	0.82	1.82	0.88	1.89	0.95	1.93	1.00	1.98	1.05	

*Basic field data. Collected at Tom Uglys' Bridge

+ Basic field data. Collected at Lugarno Ferry

Table C4.

Detailed Computed Results for Velocity and Stage using C = 150 and Field Point Measurements of Maximum Tidal Velocities.

C = 150

Cross Sect. No. (See Fig. 24)	Using Tom Ugly's Field Data						Using Lugarno Ferry Field Data						Max. Tidal Vel. (fps) derived from point velocities taken on 28.3.67
	μ_1 fps	μ_2 fps	U_{max} fps	z_1 ft	z_2 ft	Z_{max} ft	μ_1 fps	μ_2 fps	U_{max} fps	z_1 ft	z_2 ft	Z_{max} ft	
1	-0.35	+1.20	1.25	-0.16	-1.52	1.53	-1.41	2.71	3.05	-0.23	-1.56	1.58	
4			0.93			1.54	-1.43	2.34	2.74	-0.10	-1.56	1.56	
6	*-0.40	*+0.67	*0.78	*-0.13	*-1.55	*1.55	-1.43	2.16	2.59	-0.03	-1.58	1.58	0.83
9	-0.41	0.55	0.68	-0.11	-1.57	1.57	-1.43	2.03	2.49	0.07	-1.60	1.60	
11	-0.43	0.30	0.52	-0.10	-1.57	1.58	-1.41	1.78	2.27	0.14	-1.59	1.60	
13	-0.43	0.24	0.49	-0.09	-1.58	1.58	-1.40	1.72	2.22	0.20	-1.58	1.60	
14	-0.44	0.17	0.47	-0.09	-1.58	1.58	-1.39	1.65	2.16	0.22	-1.58	1.60	1.17
15	-0.46	-0.19	0.50	-0.09	-1.58	1.58	-1.33	1.29	1.85	0.25	-1.58	1.60	1.49
16	-0.48	-0.55	0.73	-0.08	-1.56	1.57	-1.26	0.94	1.57	0.32	-1.55	1.58	
17	-0.48	-0.64	0.80	-0.07	-1.55	1.56	‡-1.24	‡0.85	‡1.50	‡0.35	‡1.54	‡1.57	1.46
19	-0.49	-0.82	0.95	-0.07	-1.53	1.53	-1.20	0.67	1.37	0.40	-1.52	1.57	
20	-0.51	-1.24	1.34	-0.08	-1.51	1.51	-1.08	0.24	1.11	0.44	-1.51	1.57	2.07
22	-0.53	-1.65	1.74	-0.11	-1.42	1.42	-0.96	-0.16	0.97	0.50	-1.48	1.57	
24	-0.57	-2.25	2.32	-0.14	-1.37	1.37	-0.94	-0.22	0.97	0.52	-1.46	1.55	
26	-0.60	-2.49	2.56	-0.33	-1.21	1.26	-0.85	-0.48	0.97	0.56	-1.41	1.52	1.45
27	-0.71	-2.91	3.00	-0.47	-1.12	1.22	-0.65	-0.97	1.17	0.56	-1.38	1.49	
29	-0.79	-3.09	3.19	-0.80	-0.88	1.19	-0.56	-1.20	1.32	0.55	-1.20	1.41	1.12
31	-1.04	-3.36	3.52	-1.43	-0.46	1.51	-0.39	-1.60	1.64	0.48	-1.15	1.25	
33	-1.48	-3.51	3.81	-2.31	+0.02	2.31	-0.24	-1.96	1.98	0.29	-1.01	1.05	

*Basic field data collected at Tom Ugly's Bridge

‡ Basic field data collected at Lugarno Ferry

Table C5.

Maximum Tidal Velocities and Maximum Tidal Displacement Before and After Reclamation.

C = 150

Cross Section No.	Before Reclamation U_{max} fps	Before Reclamation Z_{max} ft	After Reclamation U_{max} fps	After Reclamation Z_{max} ft.	% Reduction in Velocity
1	3.05	1.58	2.82	1.57	7.5
4	2.74	1.56	2.55	1.57	6.9
6	2.59	1.58	2.37	1.58	8.5
9	2.49	1.60	2.26	1.60	9.2
11	2.27	1.60	2.06	1.60	9.2
13	2.22	1.60	2.01	1.60	9.5
14	2.16	1.60	1.95	1.60	9.7
15	1.85	1.60	1.72	1.60	7.0
16	1.57	1.58	1.49	1.59	5.1
17	1.50	1.57	1.43	1.58	4.6
19	1.37	1.57	1.32	1.58	3.6
20	1.11	1.57	1.11	1.57	0
22	0.97	1.57	0.97	1.57	0
24	0.97	1.55	0.97	1.55	0
26	0.97	1.52	0.97	1.52	0
27	1.17	1.49	1.17	1.49	0
29	1.32	1.41	1.32	1.41	0
31	1.64	1.25	1.64	1.25	0
33	1.98	1.05	1.98	1.05	0

Appendix D

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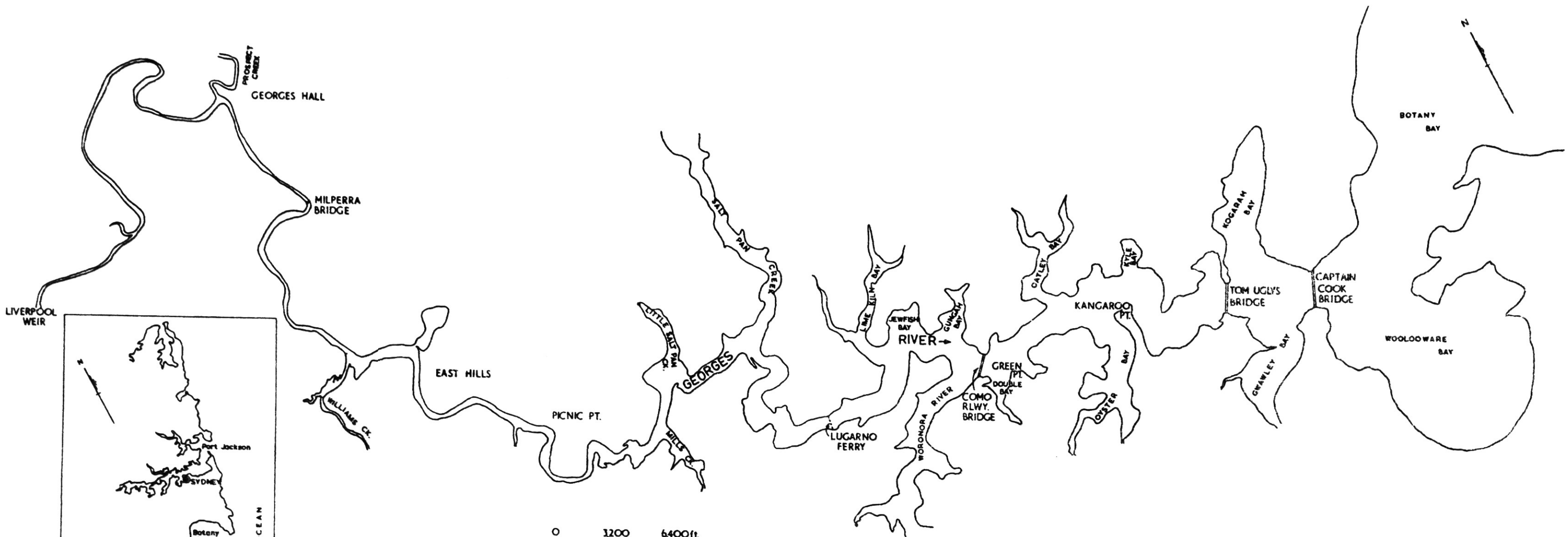


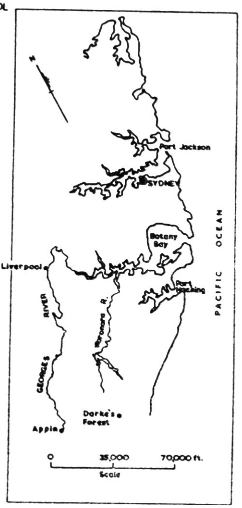


Figure 1: Locality Sketch.

CE-C-7012

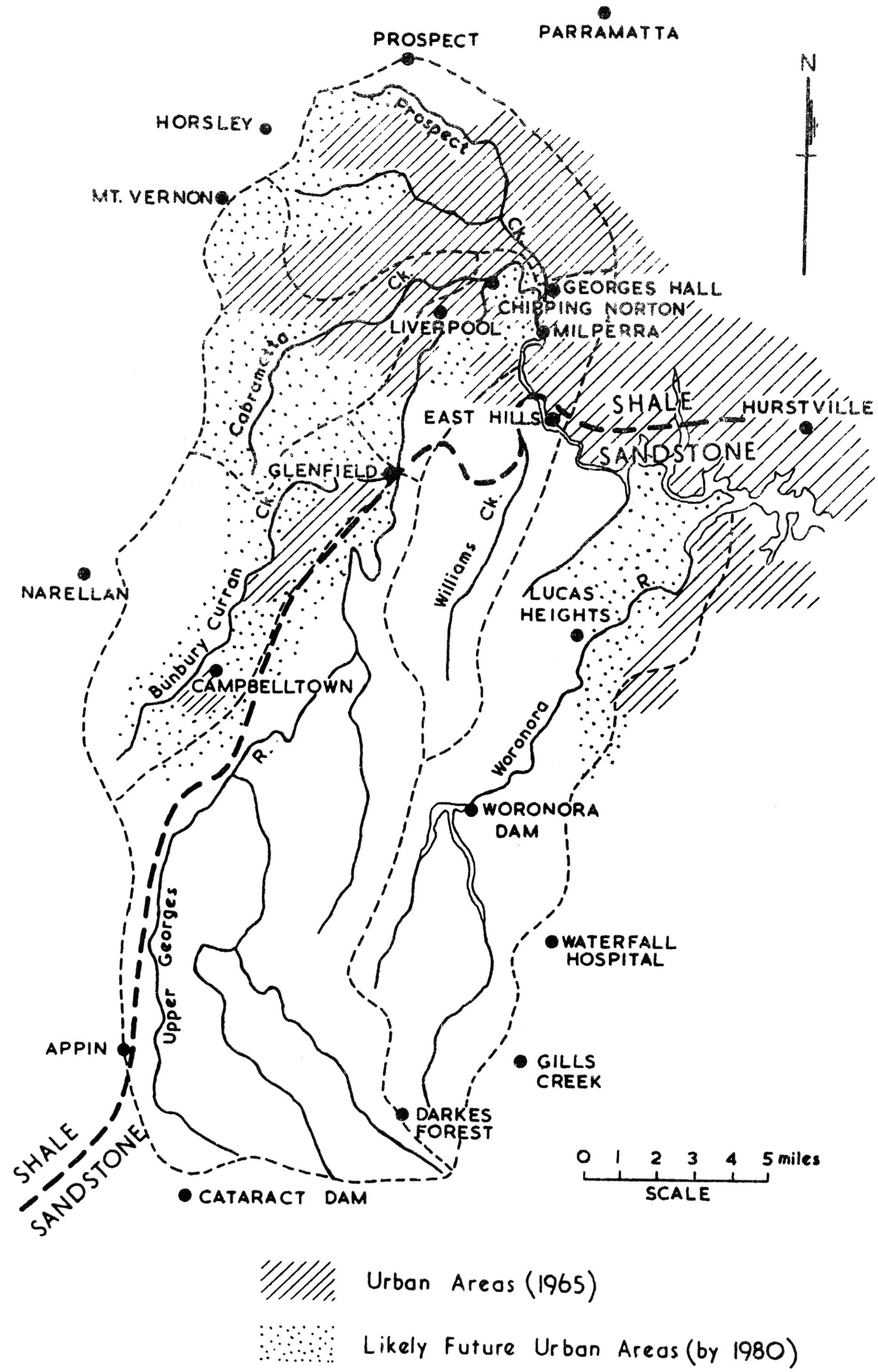


Figure 2: Georges River Basin.

CE-E-7148

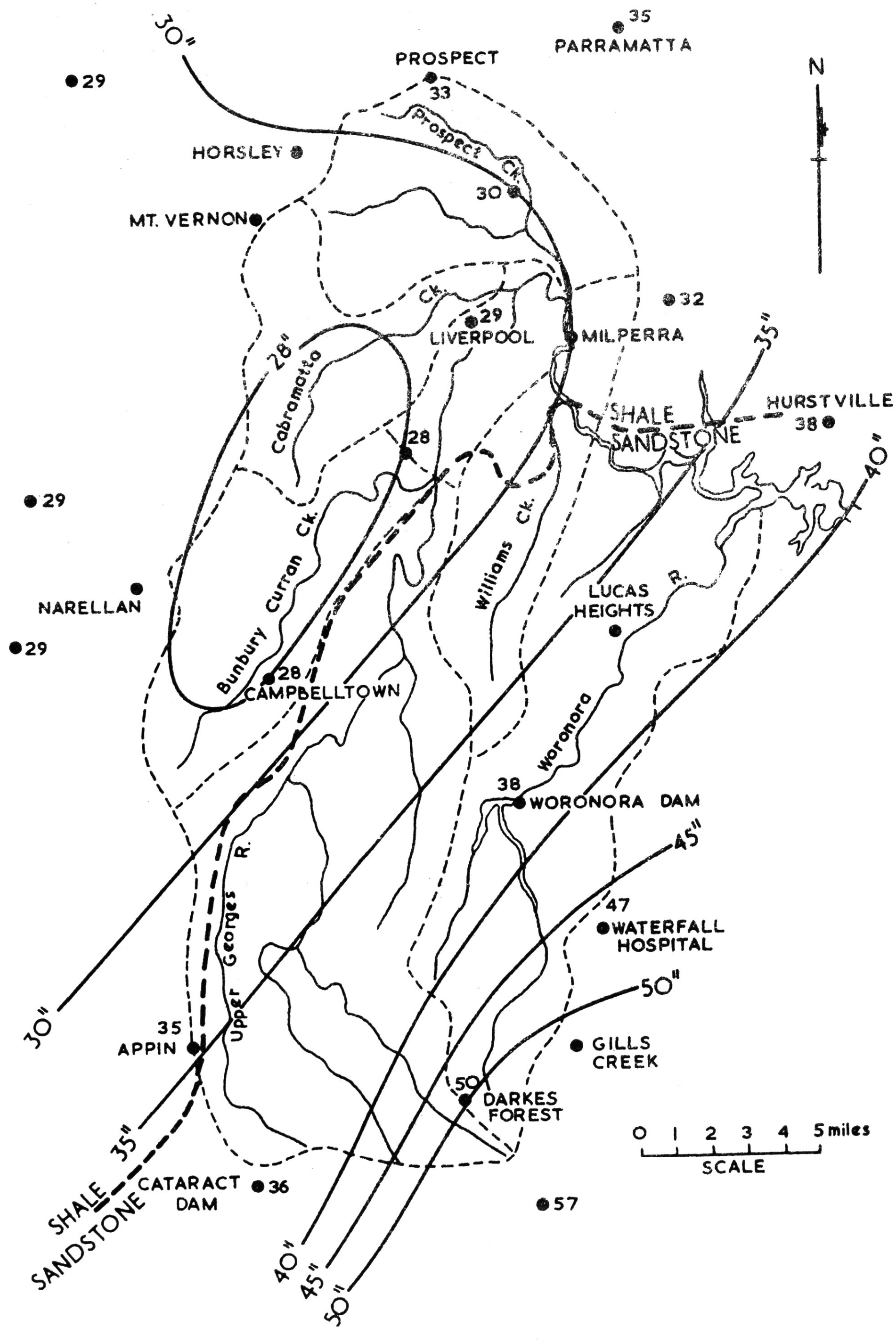


Figure 3: Georges River - Mean Annual Rainfall.

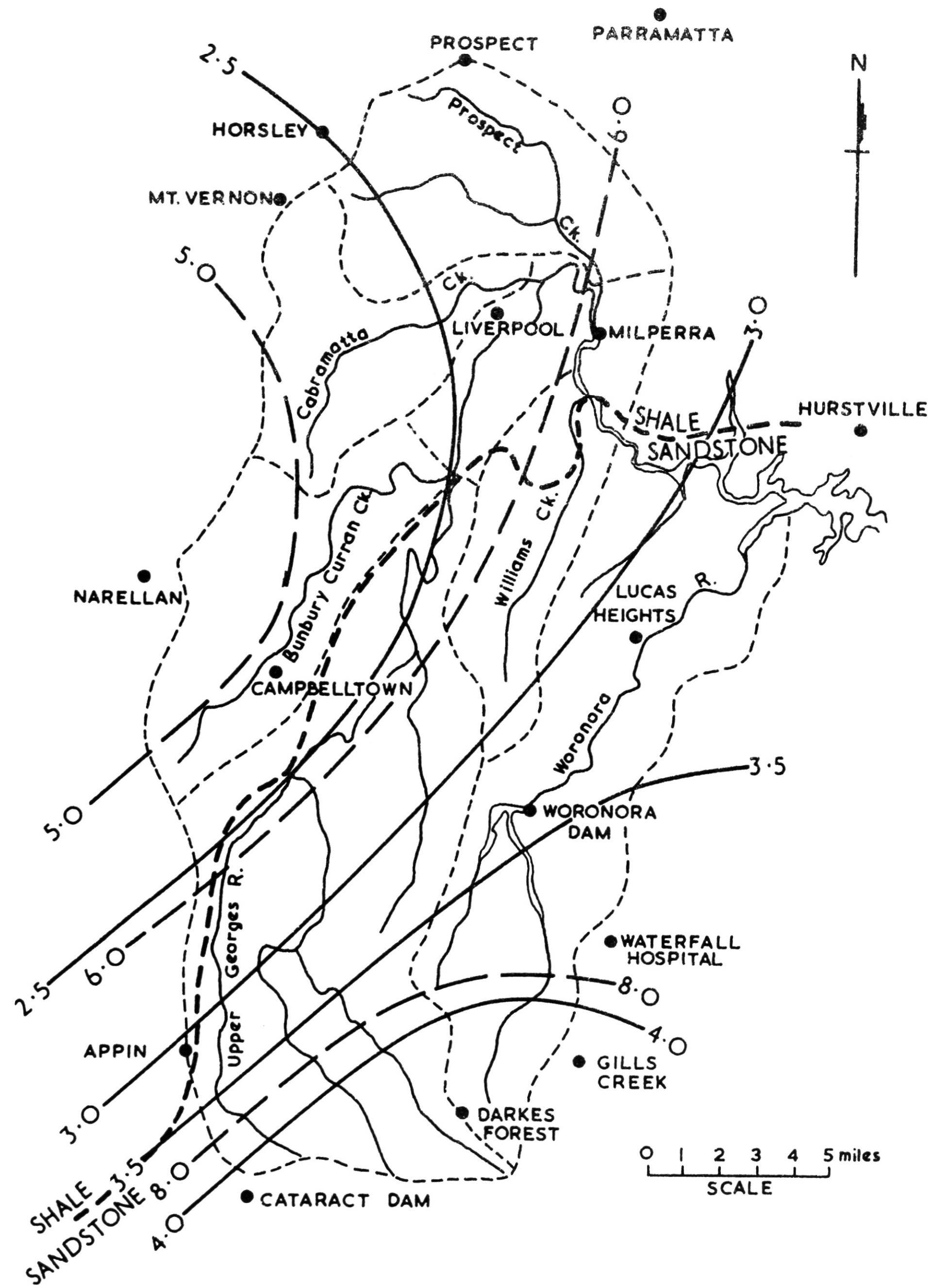


Figure 4: Georges River - 8 Hour Rainfall.

2 YR. FREQUENCY ————
50 YR. FREQUENCY — — — —

CE-E-7151

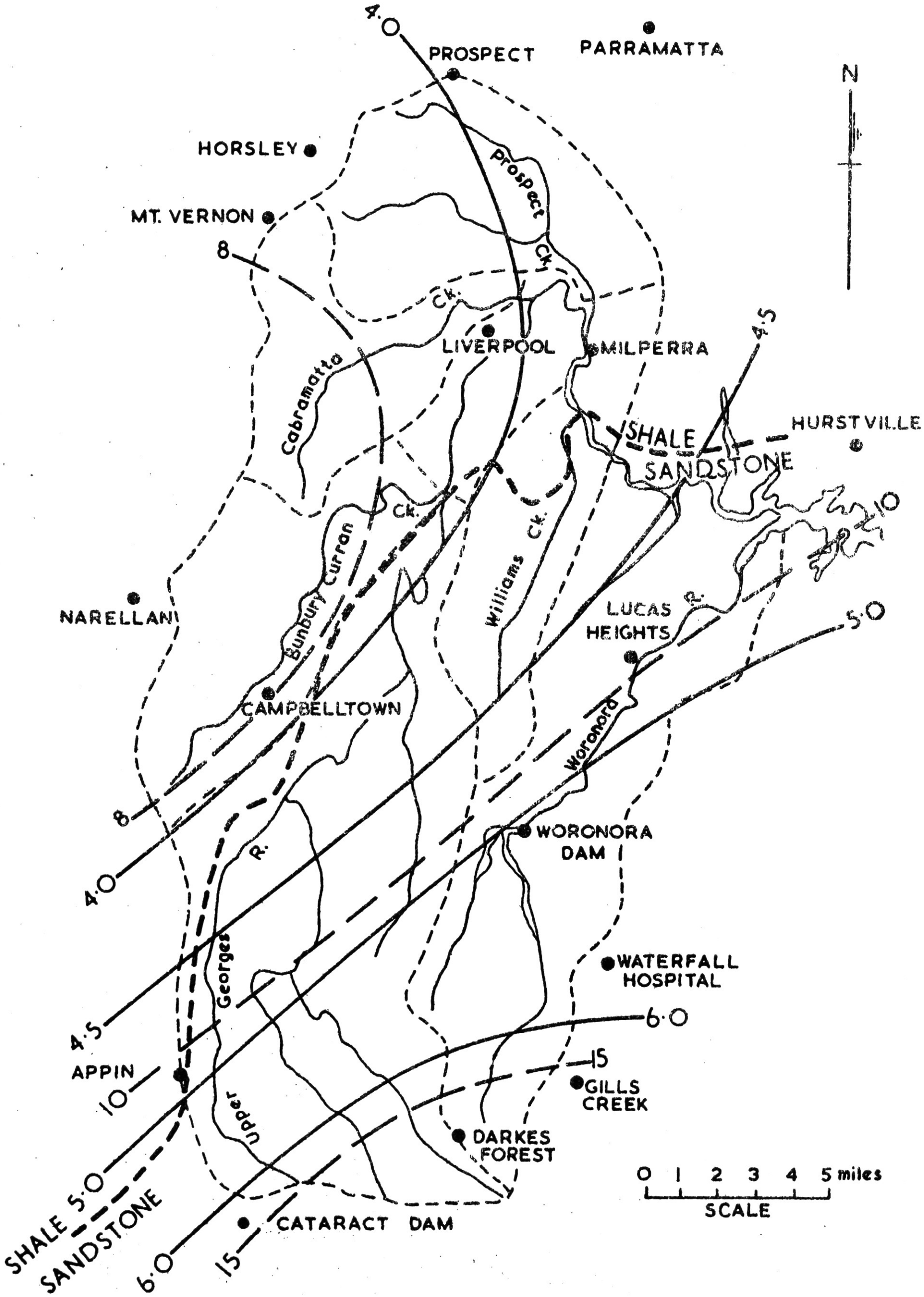


Figure 5: Georges River - 24 Hour Rainfall.

2 YR. FREQUENCY ————

50 YR. FREQUENCY — — — —

CE-E-7152

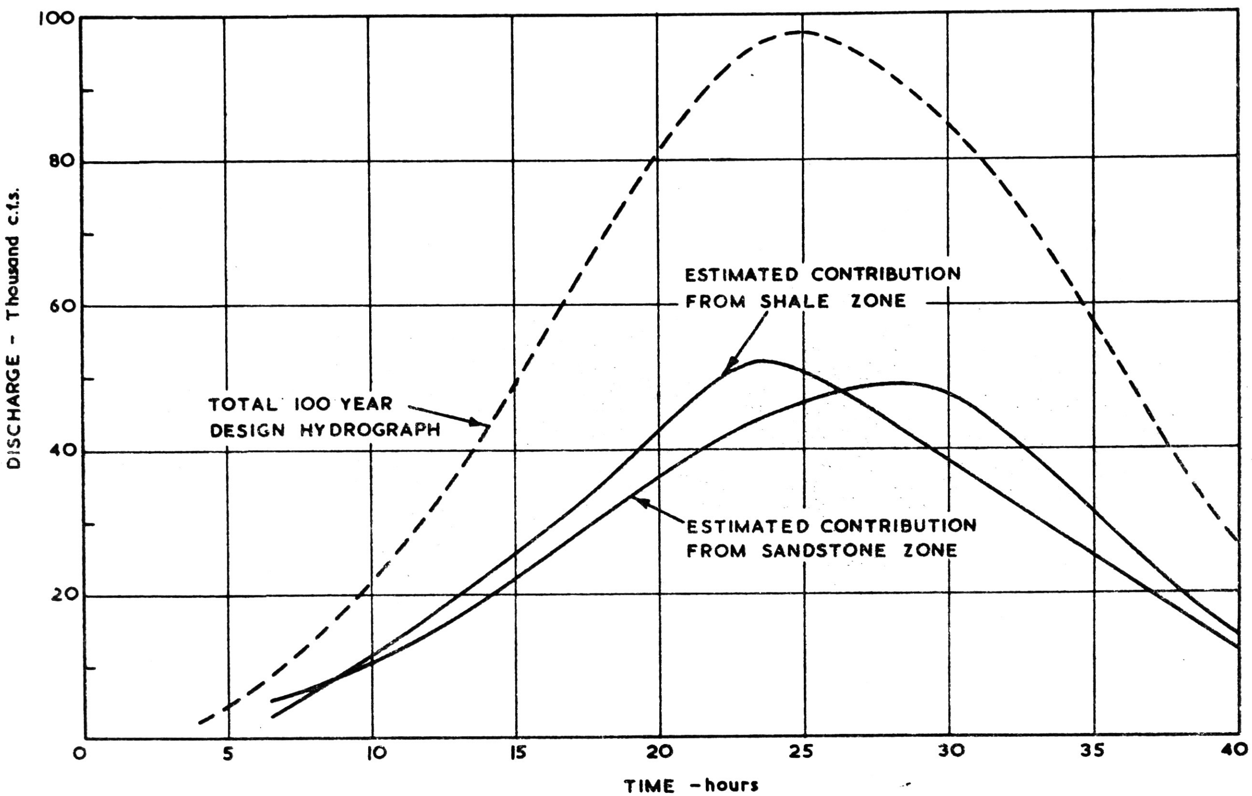


Figure 6: Georges River at East Hills - Flood Contributions from each Zone.

CE-E-7149

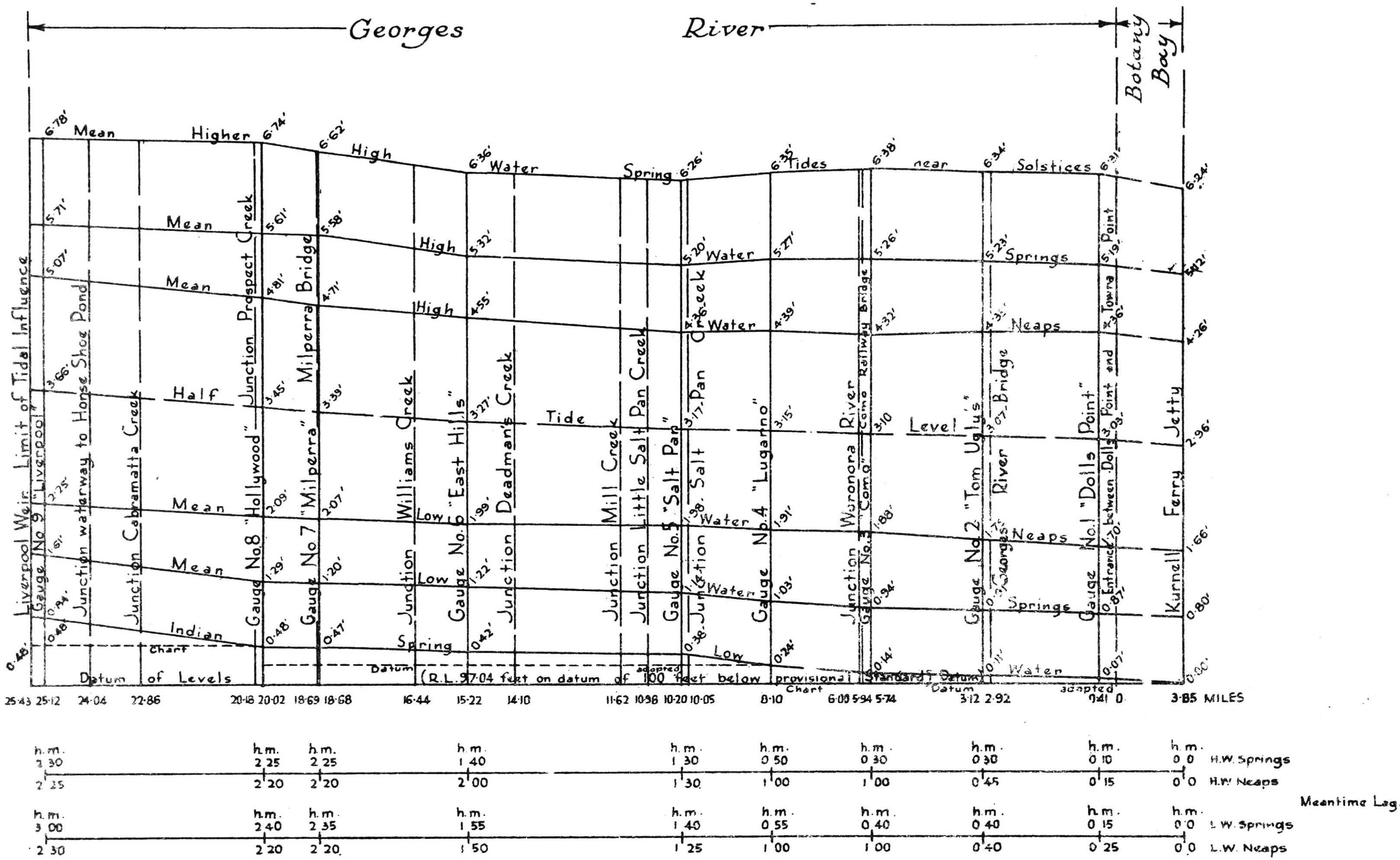


Figure 7 : Georges River Tidal Data.

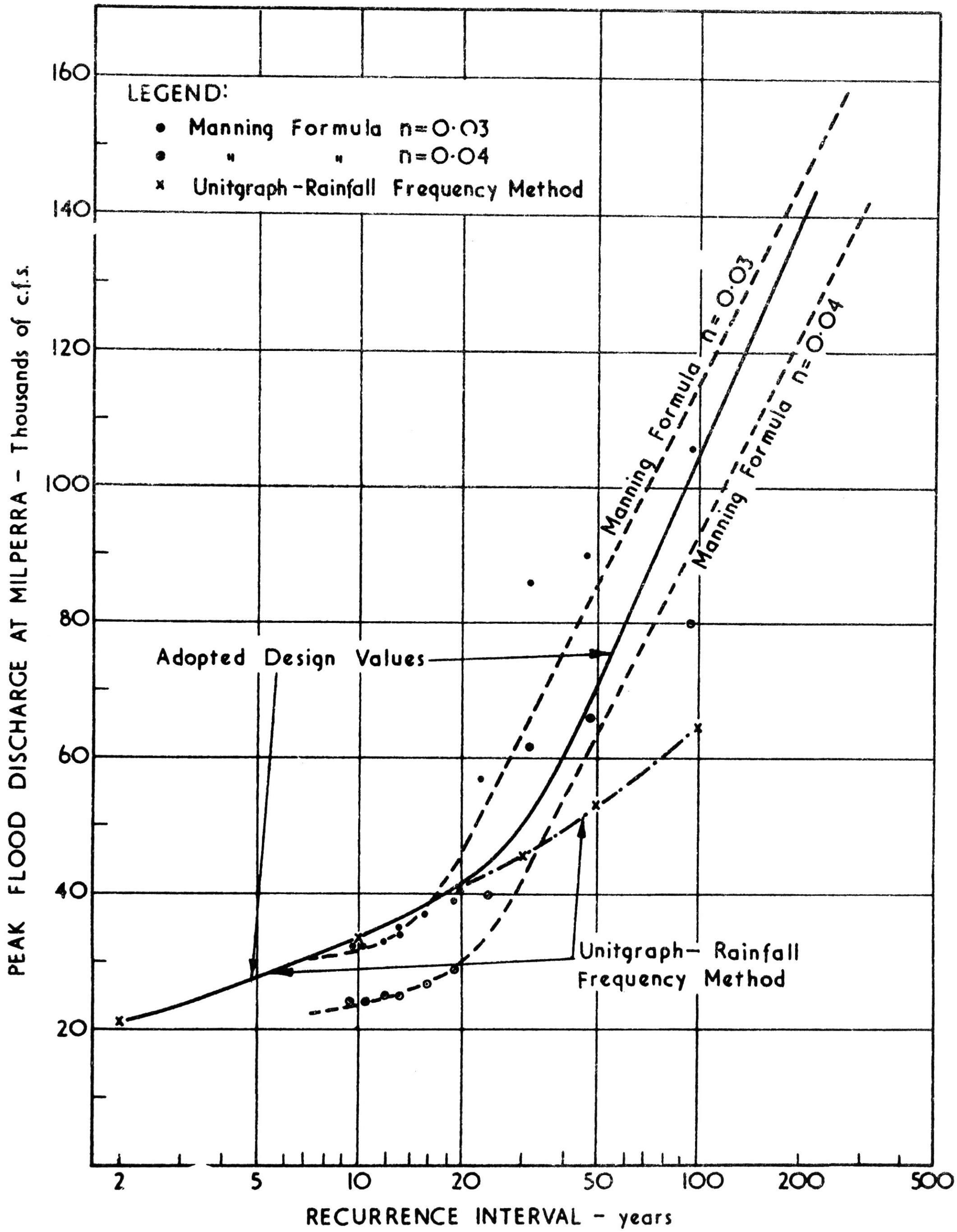


Figure 8: Georges River at Milperra - Flood Peak Frequencies.

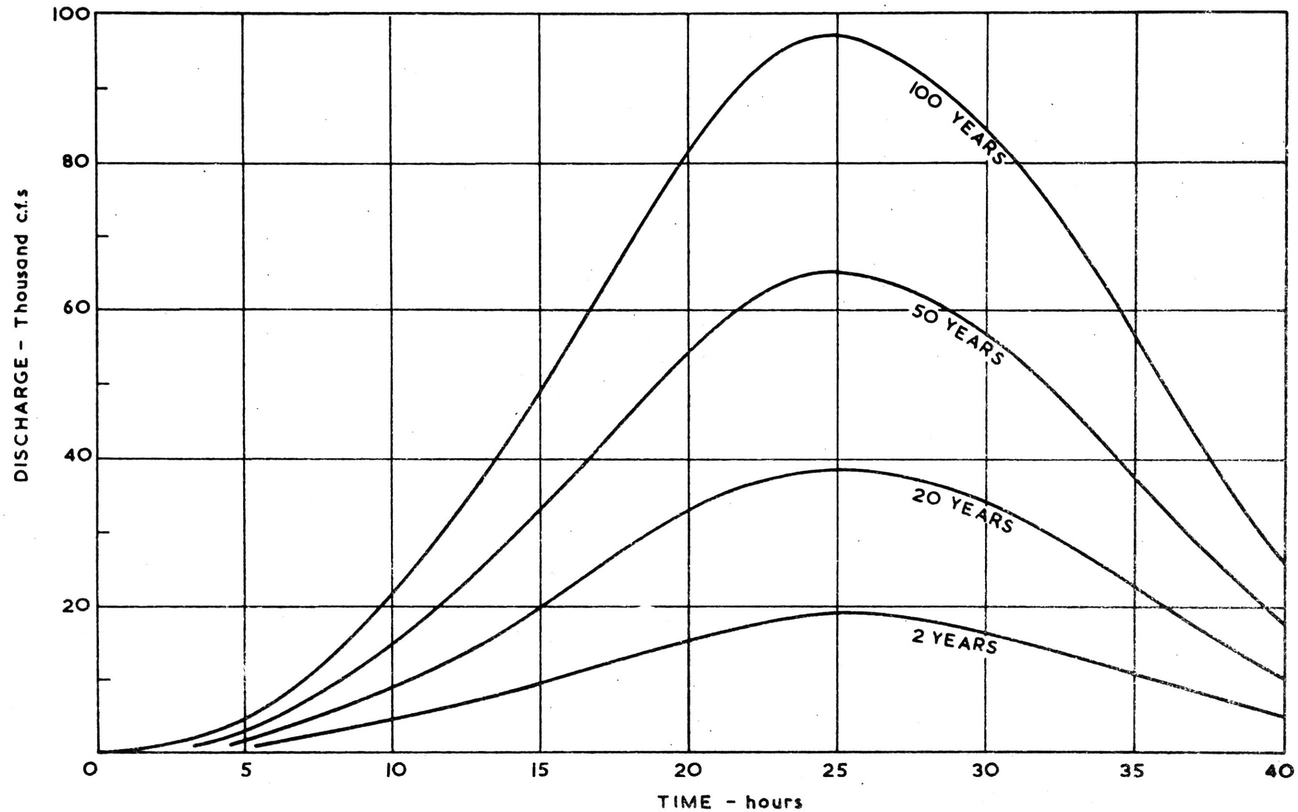


Figure 9: Georges River at East Hills - Design Hydrographs.

CE-E-7154

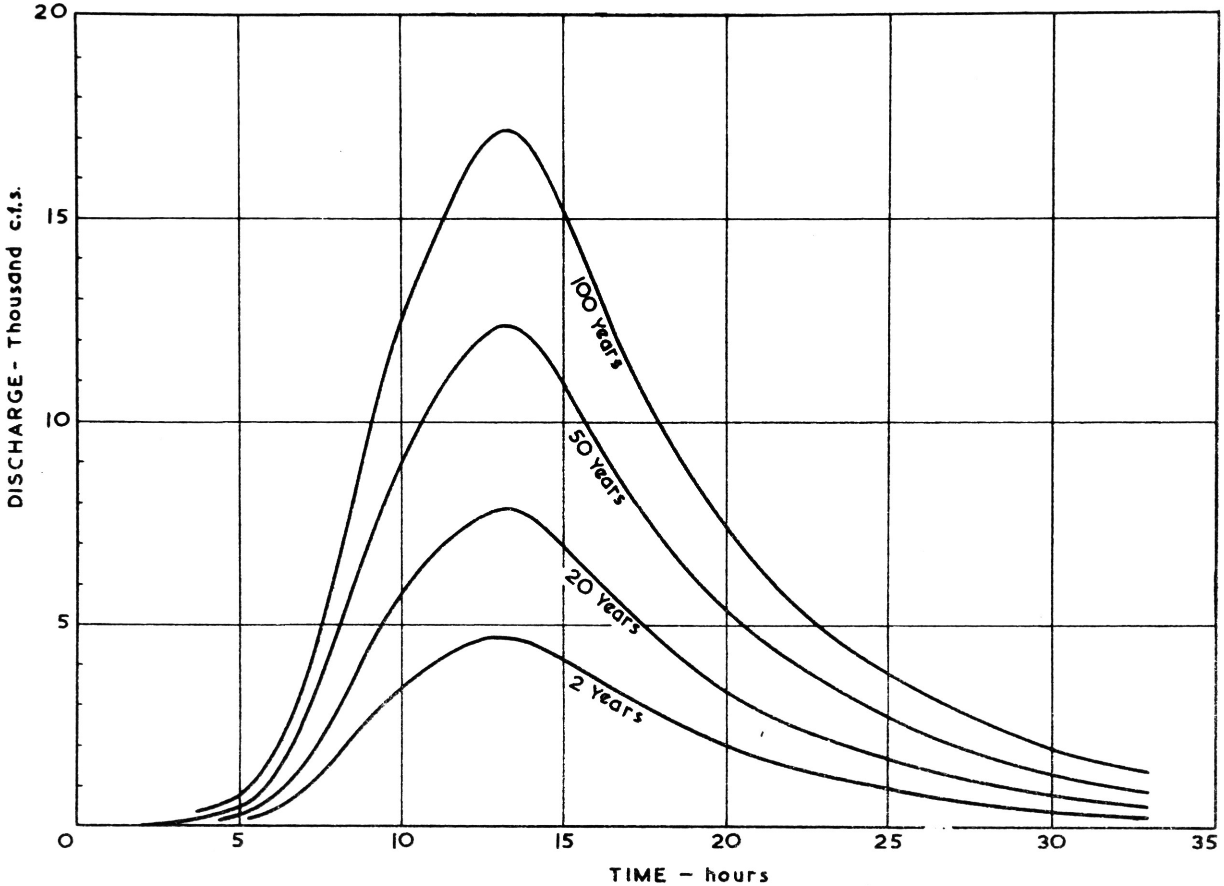


Figure 10: Prospect Creek at Georges Hall - Design Hydrographs.

CE-E-7155

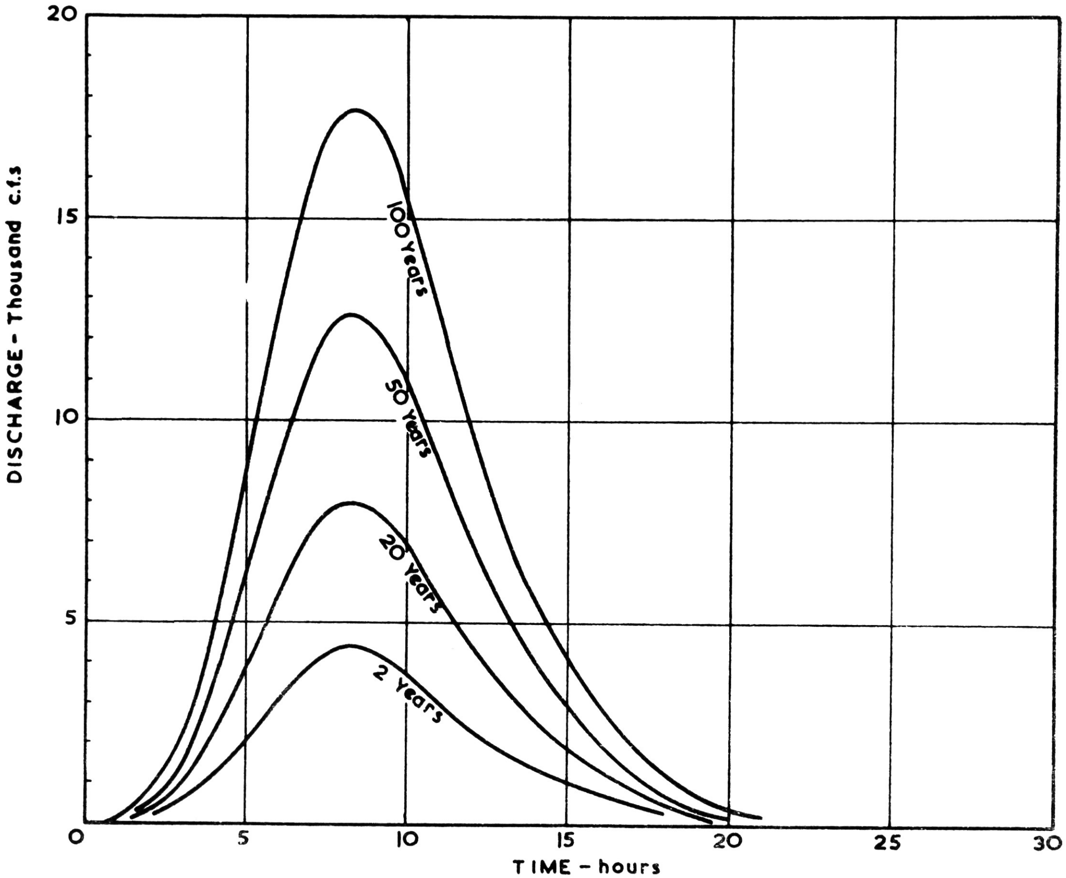


Figure 11: Cabramatta Creek at Chipping Norton - Design Hydrographs.

CE-E-7156

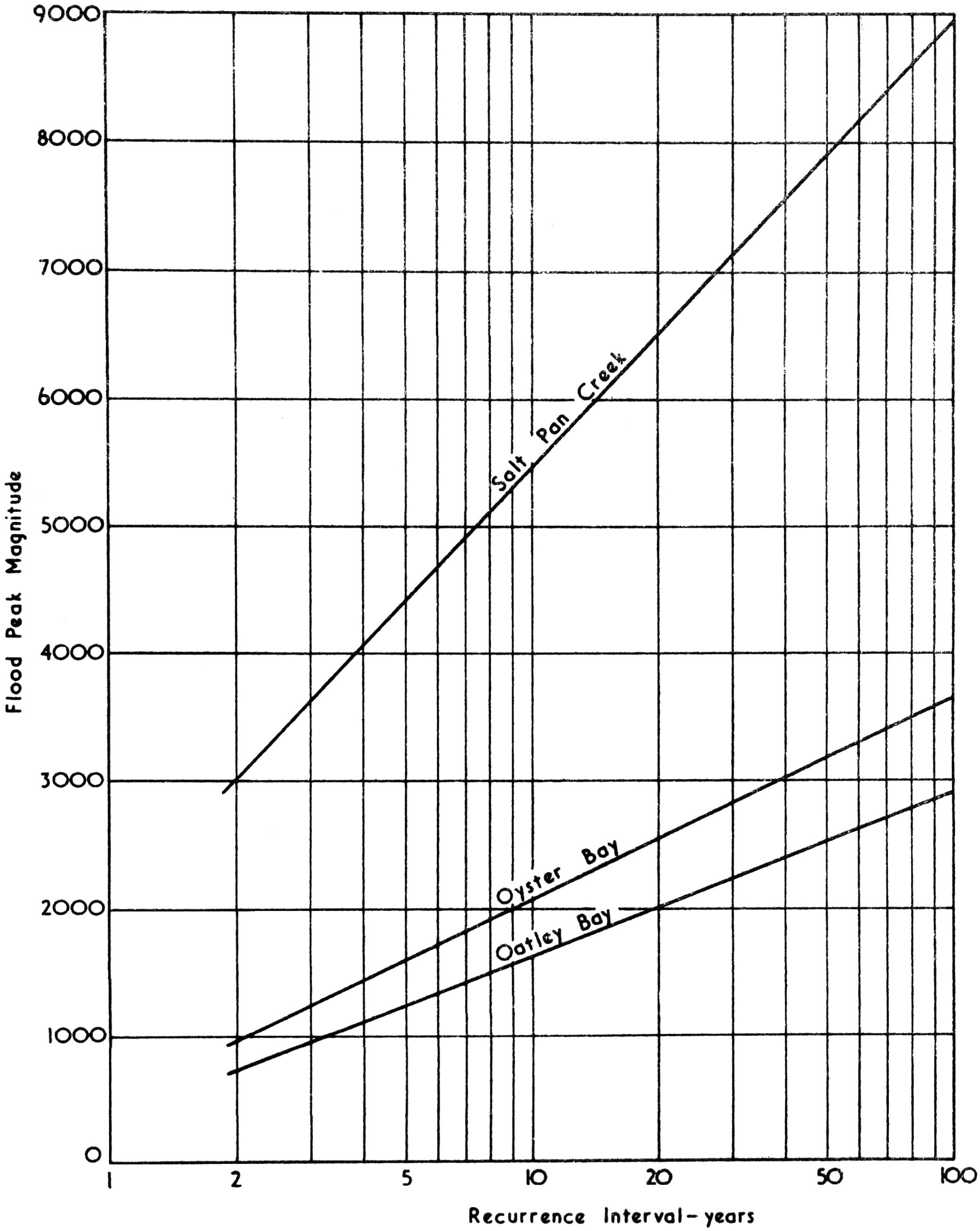
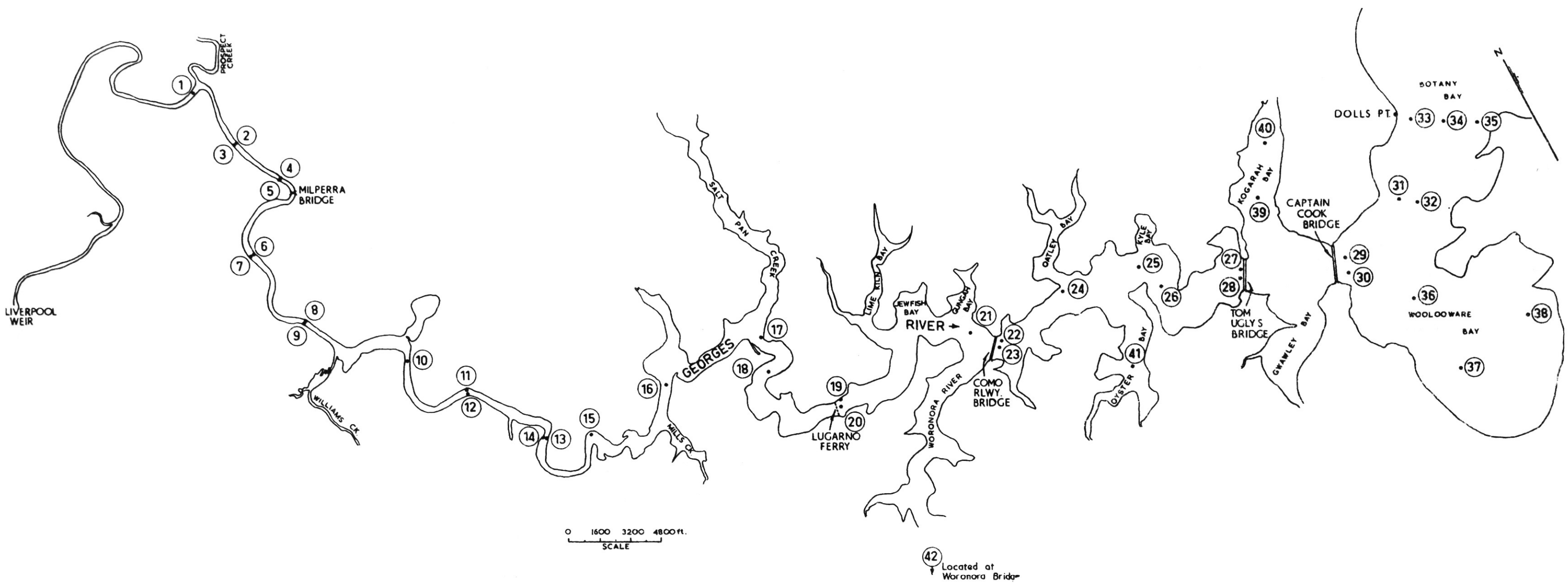


Figure 12: Flood Peak Frequencies - Salt Pan Creek, Oyster Bay and Oatley Bay.



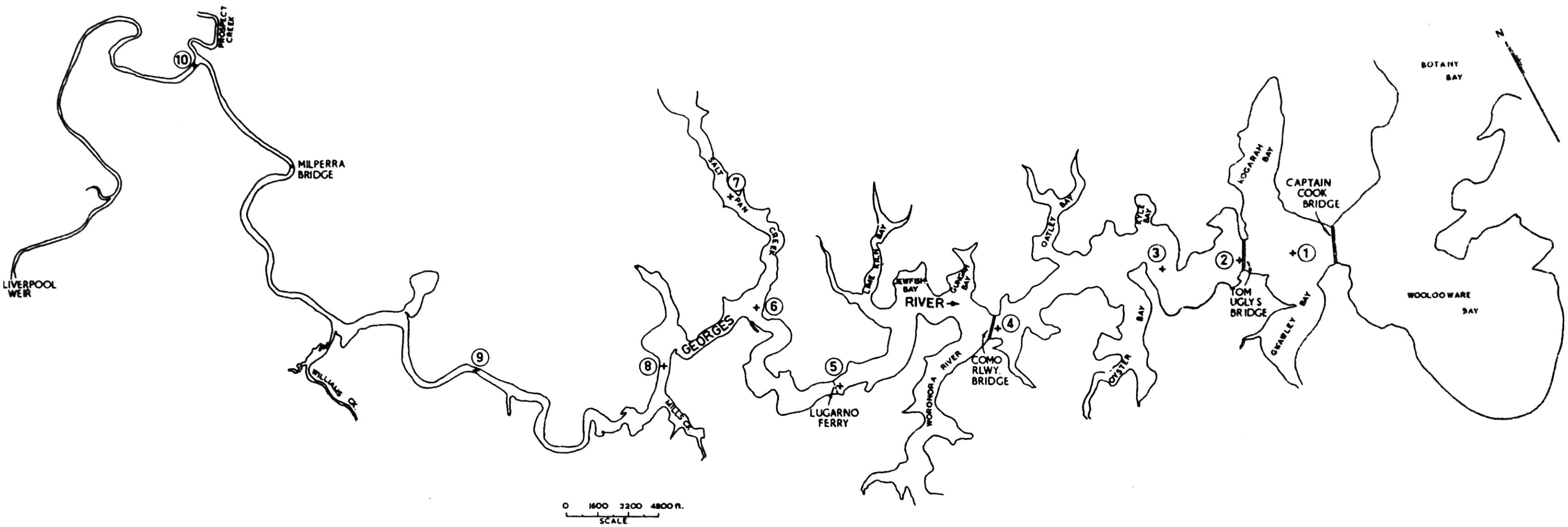
SAMPLE No.	DESCRIPTION	COLOUR WHEN WET	DEPTH
1	Medium sand with small pieces of decayed timber	Light brown	14'
2	Medium sand	Light brown	11'
3	Fine sand with fair proportion of silty clay fines. Low plasticity	Brown	11'
4	Fine sand with fair proportion of silty clay fines. Low plasticity	Dark grey	15'
5	Silty clay with some fine gritty particles Medium plasticity	Black	15'
6	Fine sand with a large proportion of silty clay fines. Low plasticity	Dark brown	15'
7	Silty clay with some fine gritty particles. Medium plasticity	Dark brown	15'
8	Fine to medium sand with small proportion of silt or clay fines. No plasticity	Dark brown	20'
9	Fine to medium sand with small proportion of silt or clay fines. No plasticity	Dark brown	12'
10	Fine to medium sand with fair proportion of silty clay fines Low plasticity	Brown	16'

SAMPLE No.	DESCRIPTION	COLOUR WHEN WET	DEPTH
11	Silty clay. Medium plasticity	Black	30'
12	Silty clay with some fine gritty particles. Medium plasticity	Dark grey	30'
13	Coarse sand	Brown	18'
14	Fine to medium sand with some silty clay fines. Negligible plasticity	Dark brown	14'
15	Silty clay with large proportion of fine sandy particles. Low plasticity	Dark brown	25'
16	Silty clay. Medium plasticity	Grey	39'
17	Medium sand with small pieces of decayed timber	Lightish brown	18'
18	Medium sand	Dark brown	20'
19	Medium sand	Lightish brown	26'
20	Medium sand with broken shell	Dark brown	11'
21	Medium sand	Dark brown	16'
22	Medium to fine sand	Dark brown	26'
23	Medium to fine silty sand with broken shell	Dark grey	11'
24	Silt with some fine gritty particles and broken shell	Dark brown	30'
25	Silt with some fine gritty particles	Dark grey	17'

SAMPLE No.	DESCRIPTION	COLOUR WHEN WET	DEPTH
26	Highly plastic clay lumps with a lot of silty clay	Dark grey	17'
27	Highly plastic clay lumps with a lot of silty clay	Dark grey	33'
28	Silty clay-highly plastic	Dark grey	31'
29	Clayey silt with some fine sand - slightly plastic	Dark brown	25'
30	Shell deposit		23'
31	Weed (kelp)		
32	Weed (kelp)		
33	Shellgrit and medium to coarse sand	Dark brown	14'
34	Shellgrit and medium to coarse sand	Dark brown	16'
35	Fine to medium sand	Grey	8'
36	Silty clay. Low plasticity	Black	14'
37	Silty clay-a little finely broken shell. Medium plasticity	Black	6'
38	Silty clay. Medium plasticity	Black	6'
39	Silty clay. Low plasticity	Dark grey	12'
40	Silty clay. Low plasticity	Dark grey	8'
41	Silty clay. Medium plasticity	Dark grey	3'
42	Fine to medium sand	Grey	5'

CE-C-7013

Figure 13: Georges River Bed Samples.



NOTES:

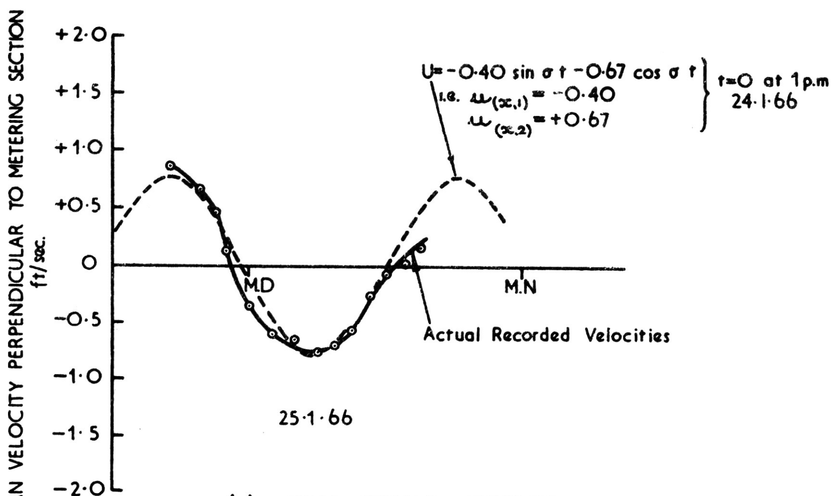
1. Except for Sample 8, all suspended sediment samples taken at about 12" above bed
2. Several samples at the location 8 were taken to verify the existence of such a large suspended load. The sample analysed was collected 24" above the bed. An examination of hydrographic survey data revealed that the location is in an isolated deep hole.

SAMPLE No.	SUSPENDED SEDIMENT Grams/Litre	WATER DEPTH	REMARKS
1	0·067	19	Taken after spell of fair weather
2	0·118	31	
3	0·099	17	
4	0·065	11	
5	0·042	10	
6	0·050	18	
7	0·203	8	Taken on 8.3.67 during recession of flood. Flood peak occurred on 7.3.67
* 8	11·077	39	
9	0·252	30	
10	0·130	14	

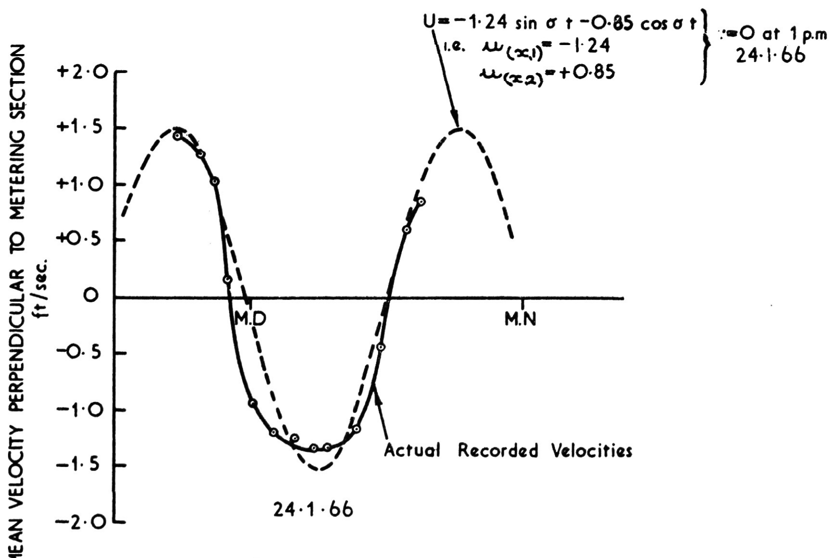
* See Notes

CE-C-7014

Figure 14: Georges River: Suspended Sediments.



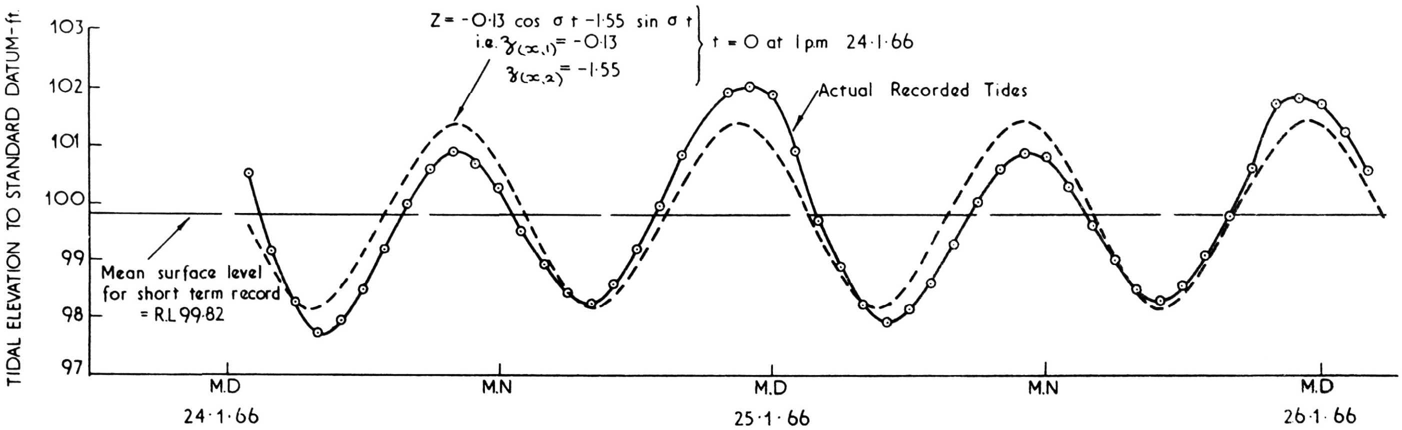
(a) TOM UGLY S BRIDGE



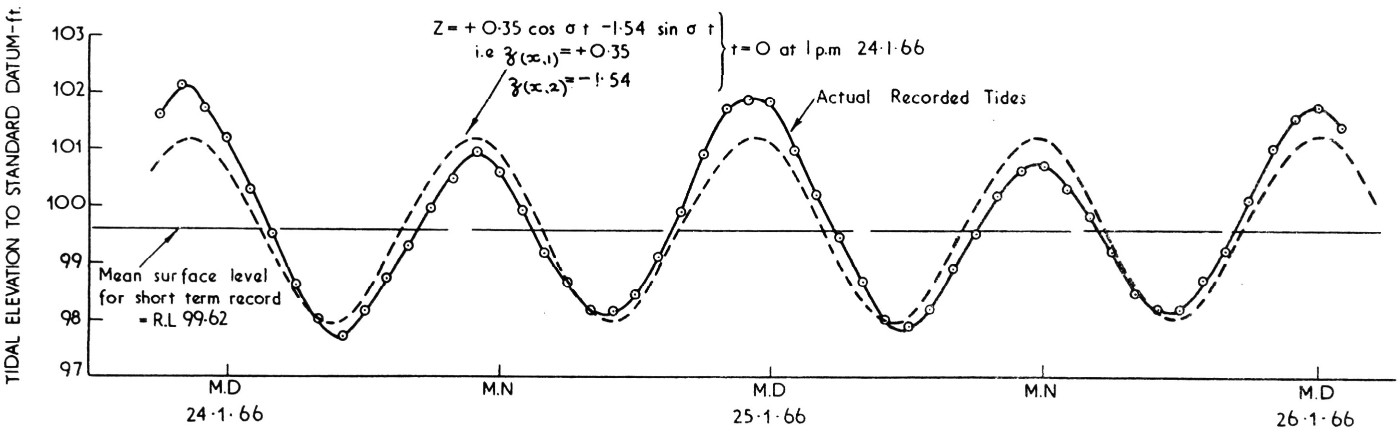
(b) LUGARNO FERRY

Figure 15: Georges River - Tidal Velocity Variations with Time.

CE-D-6970



(a) TOM UGLYS BRIDGE



(b) LUGARNO FERRY

Figure 16: Georges River - Tidal Stage Variations with Time.

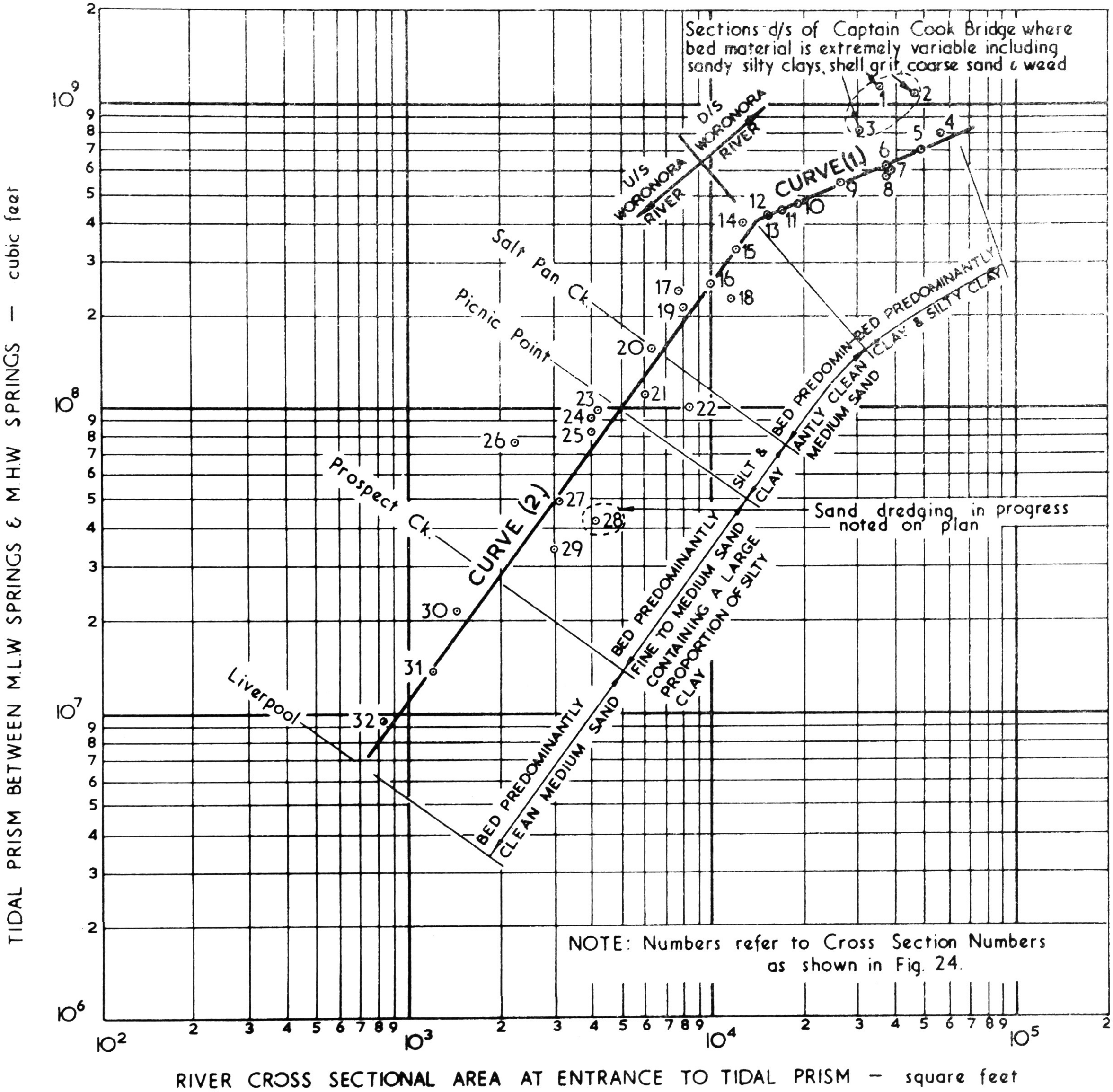


Figure 17: Georges River - Tidal Prism versus Cross Section Area.

CE-D-7016

(Based on 1959 Survey by the Department of Public Works, N.S.W.)

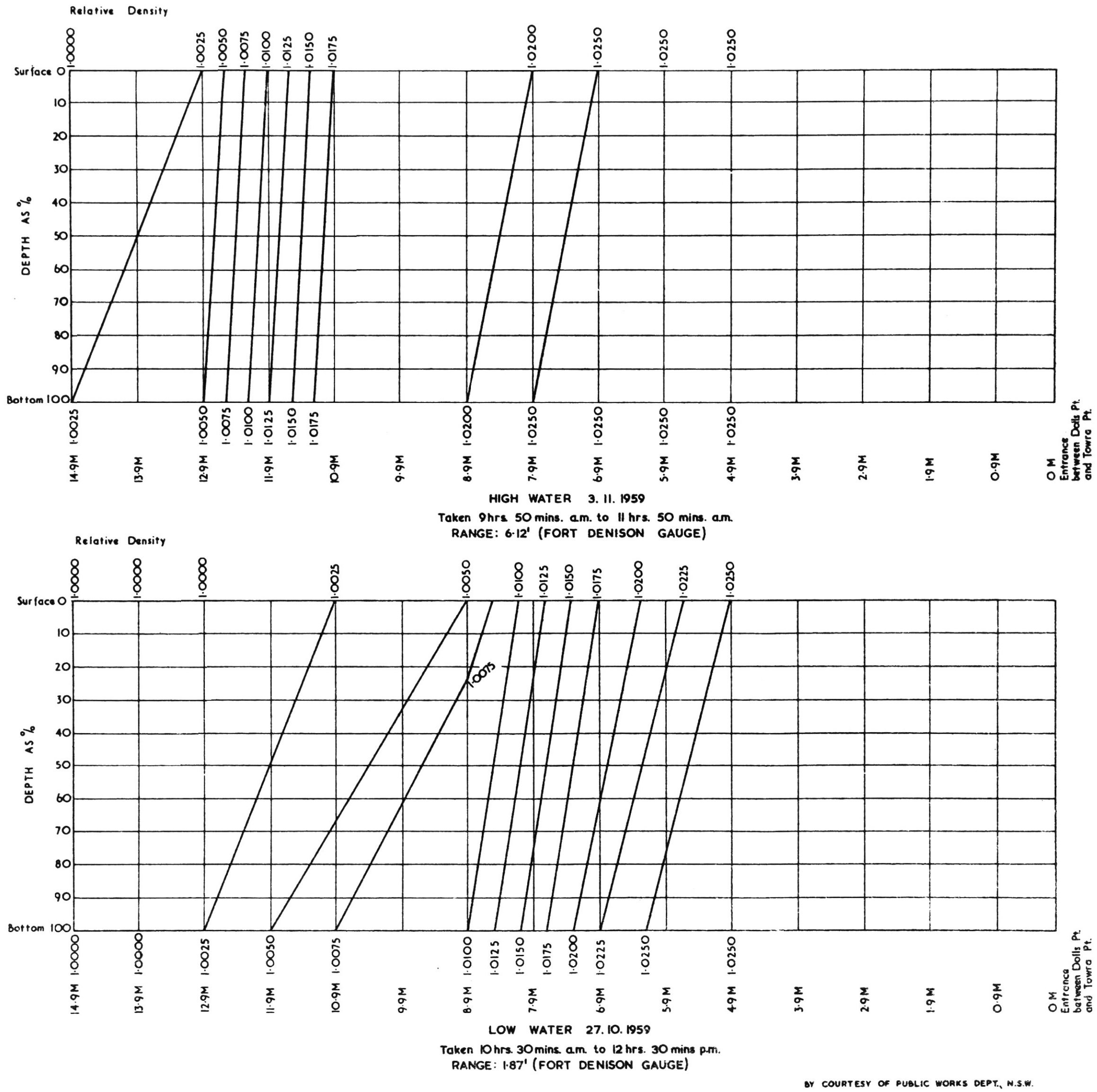


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Figure 19: Georges River - Salinity Observations.

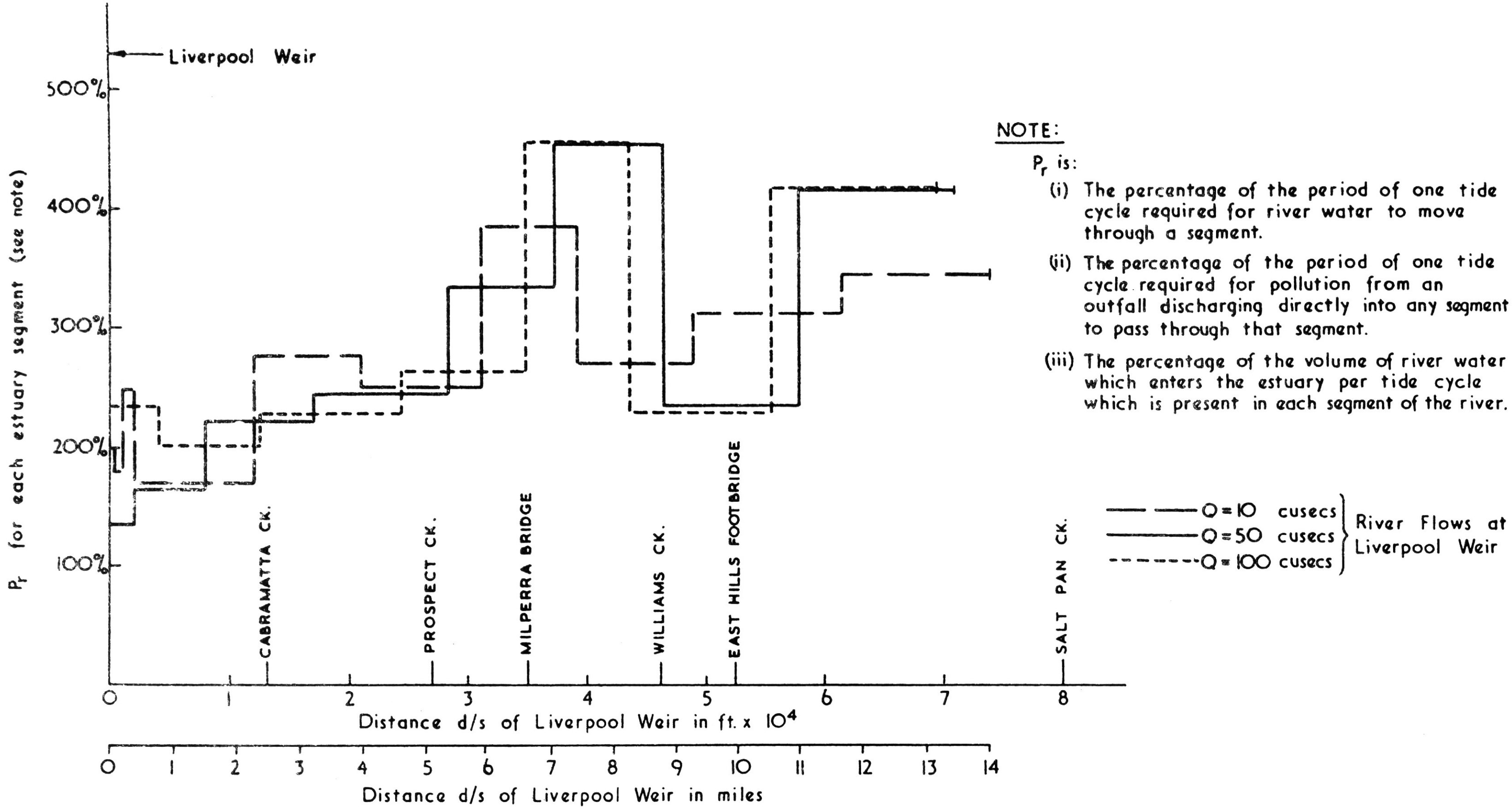


Figure 20: Georges River - Flushing Times and River Water Volumes for Individual Estuary Segments.

CE-E-7158

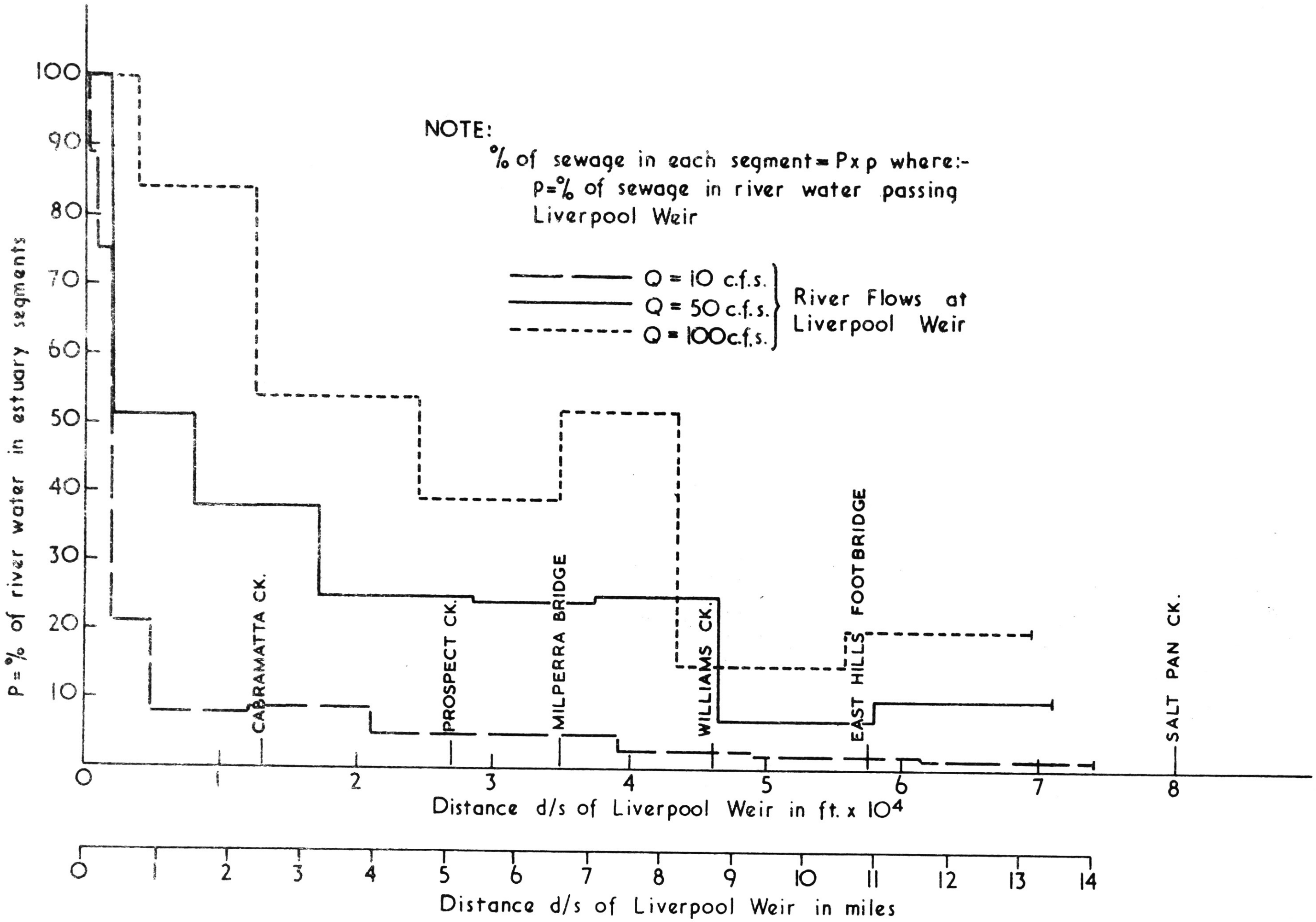


Figure 21: Georges River - Percentage of River Water in Individual Estuary Segments.

CE-E-7159

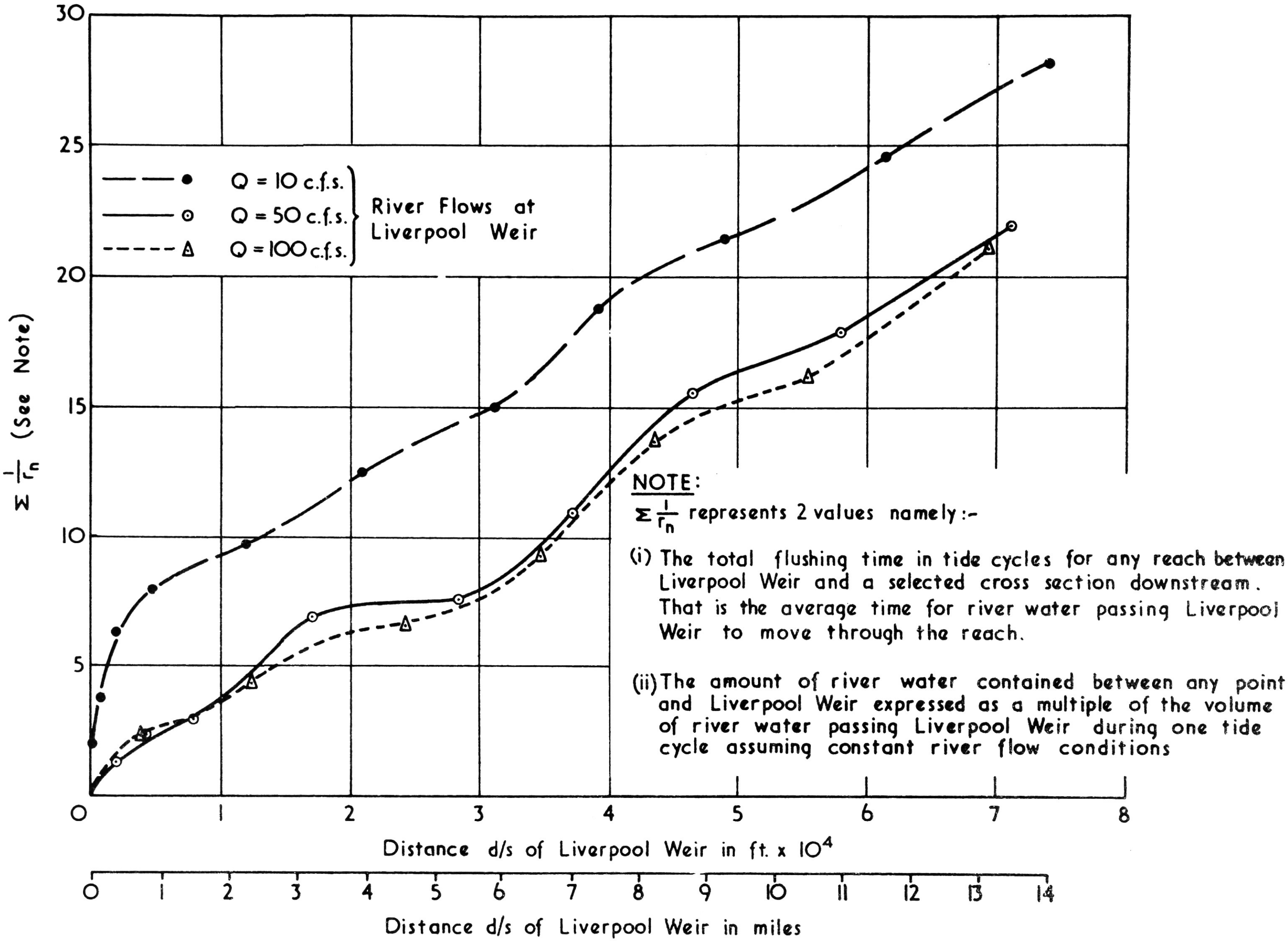


Figure 22: Georges River - Total Flushing Times and River Water Volumes.

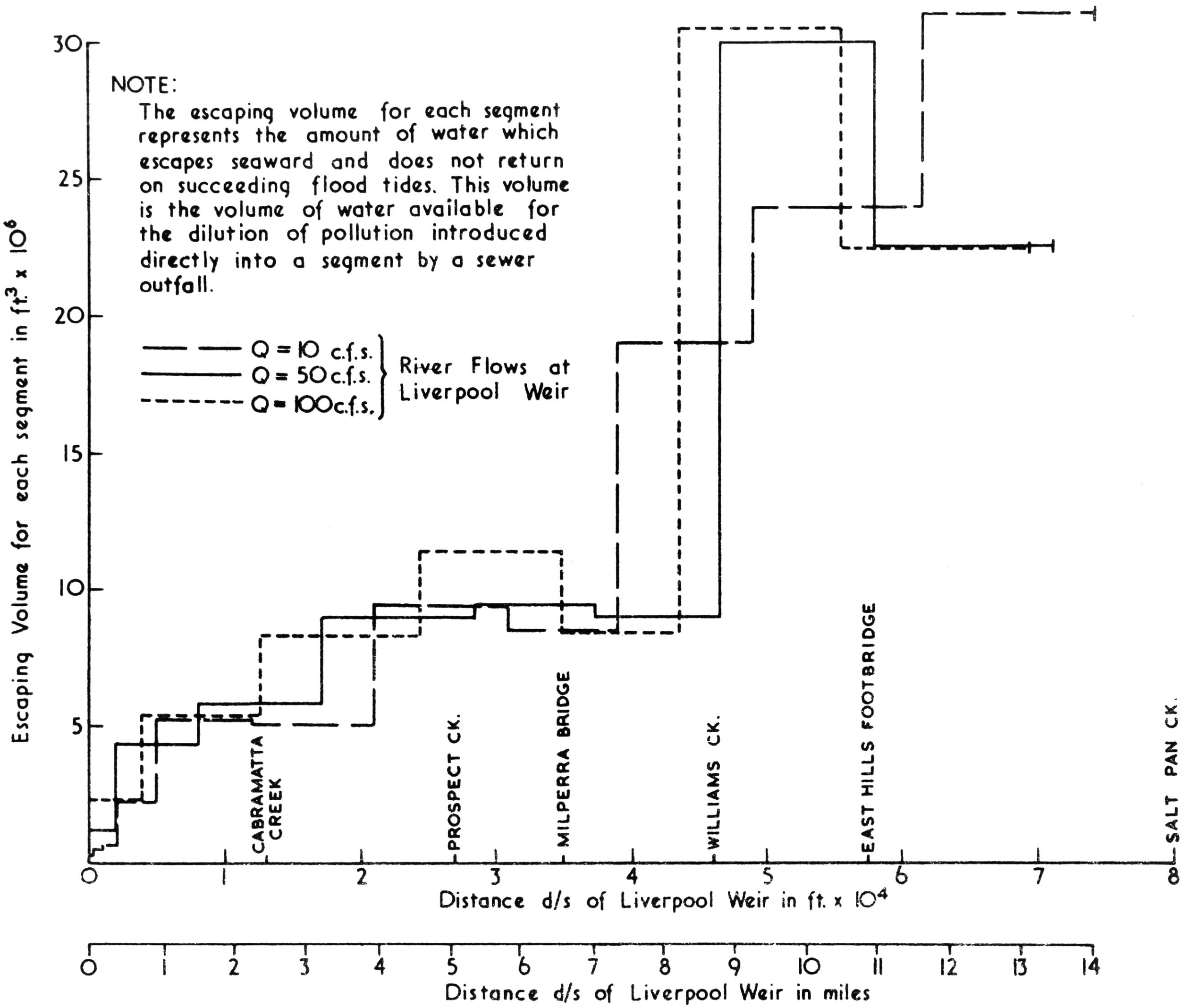


Figure 23: Georges River - Escaping Volumes.

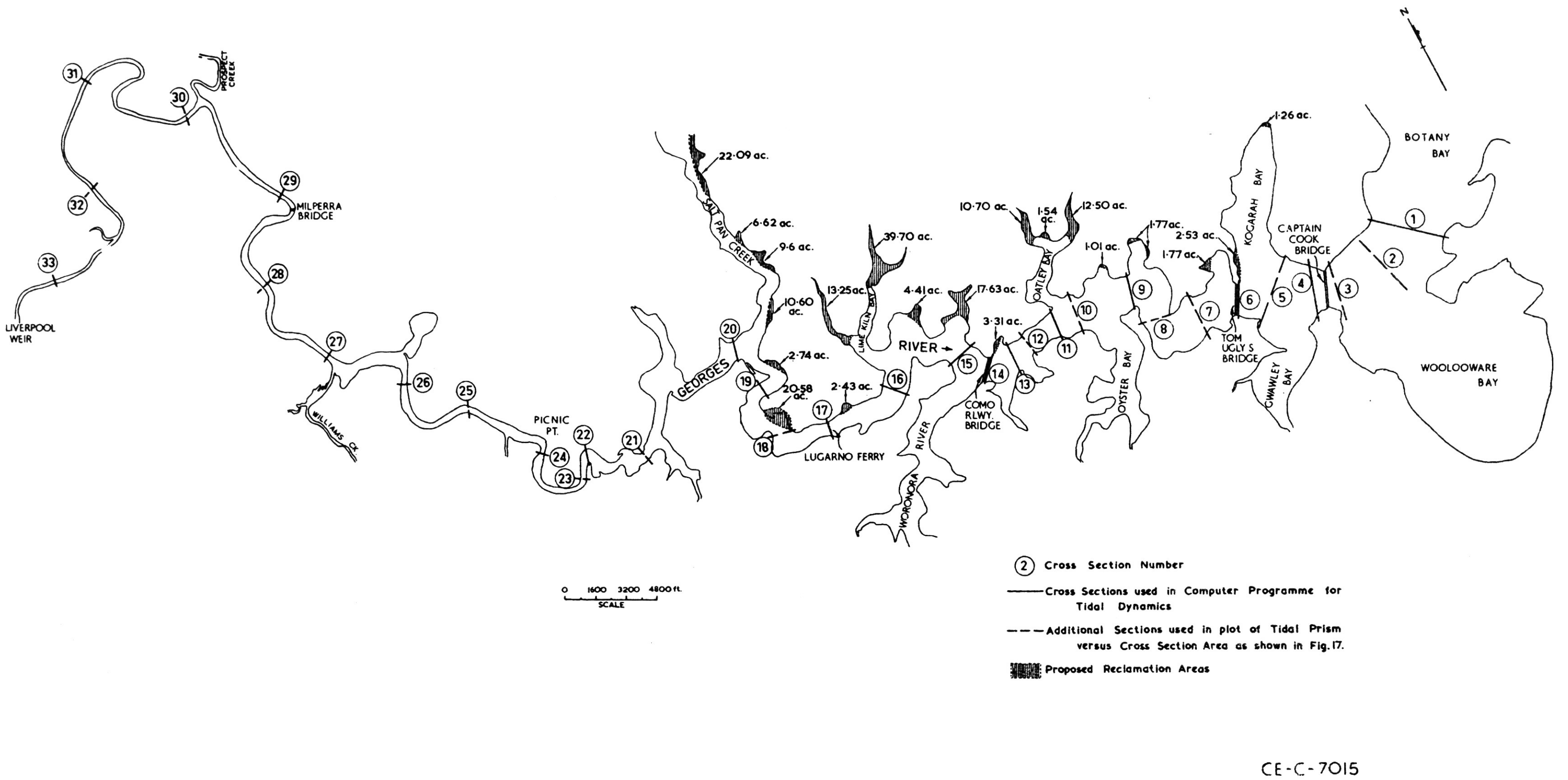
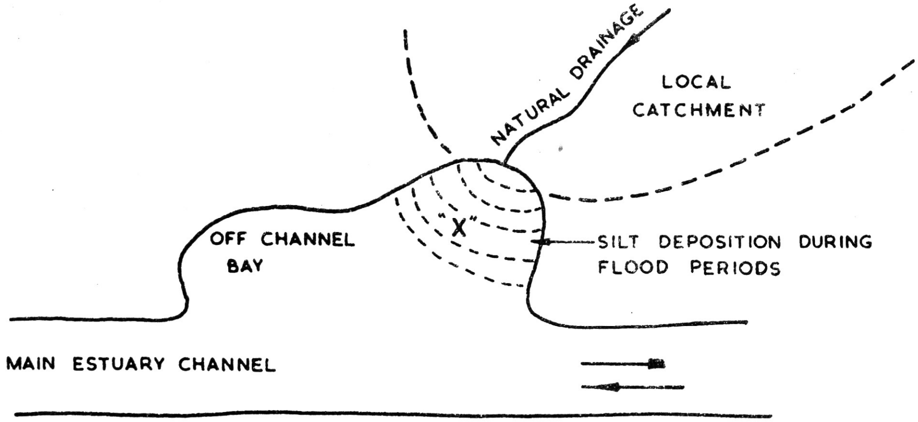
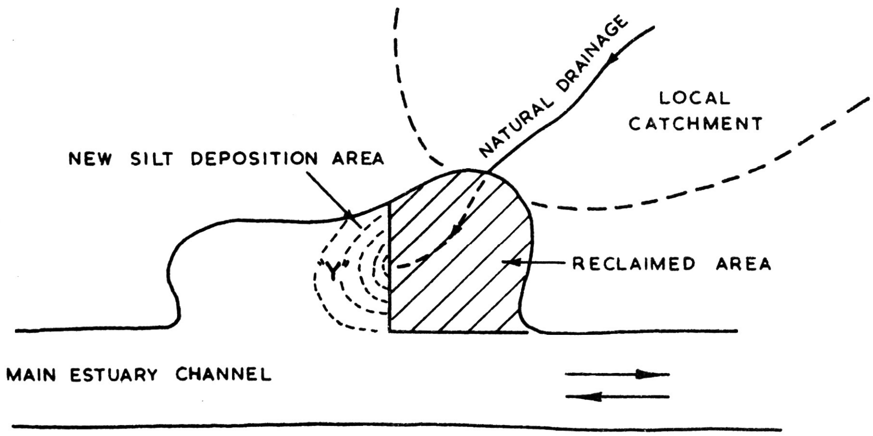


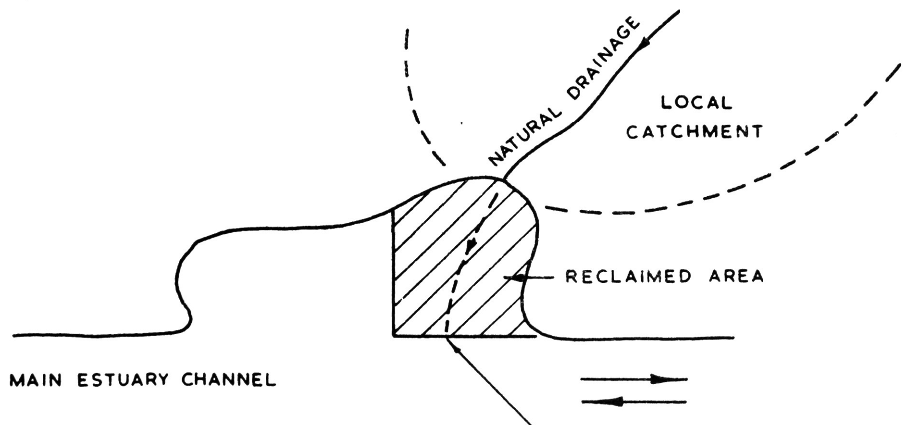
Figure 24: Georges River - Locations of Cross Sections and Proposed Reclamation Areas.



a. Drainage: NATURAL CONDITIONS



b. Drainage with Reclamation: UNDESIRABLE



c. Drainage with Reclamation: DESIRABLE

Figure 26: Drainage Through Reclaimed Areas.

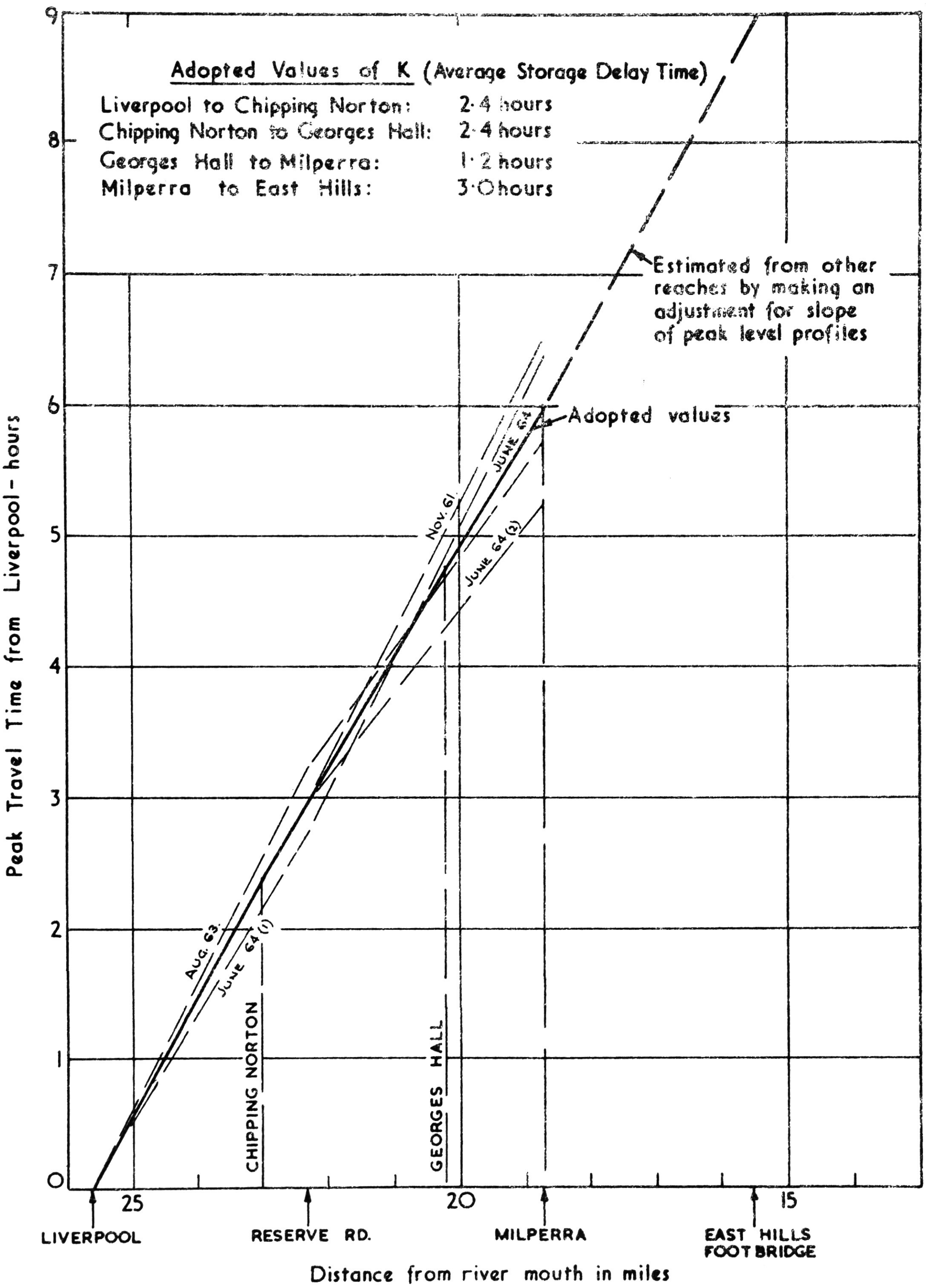


Figure 28: Georges River - Flood Peak Travel Time from Liverpool.

CE-E-7162

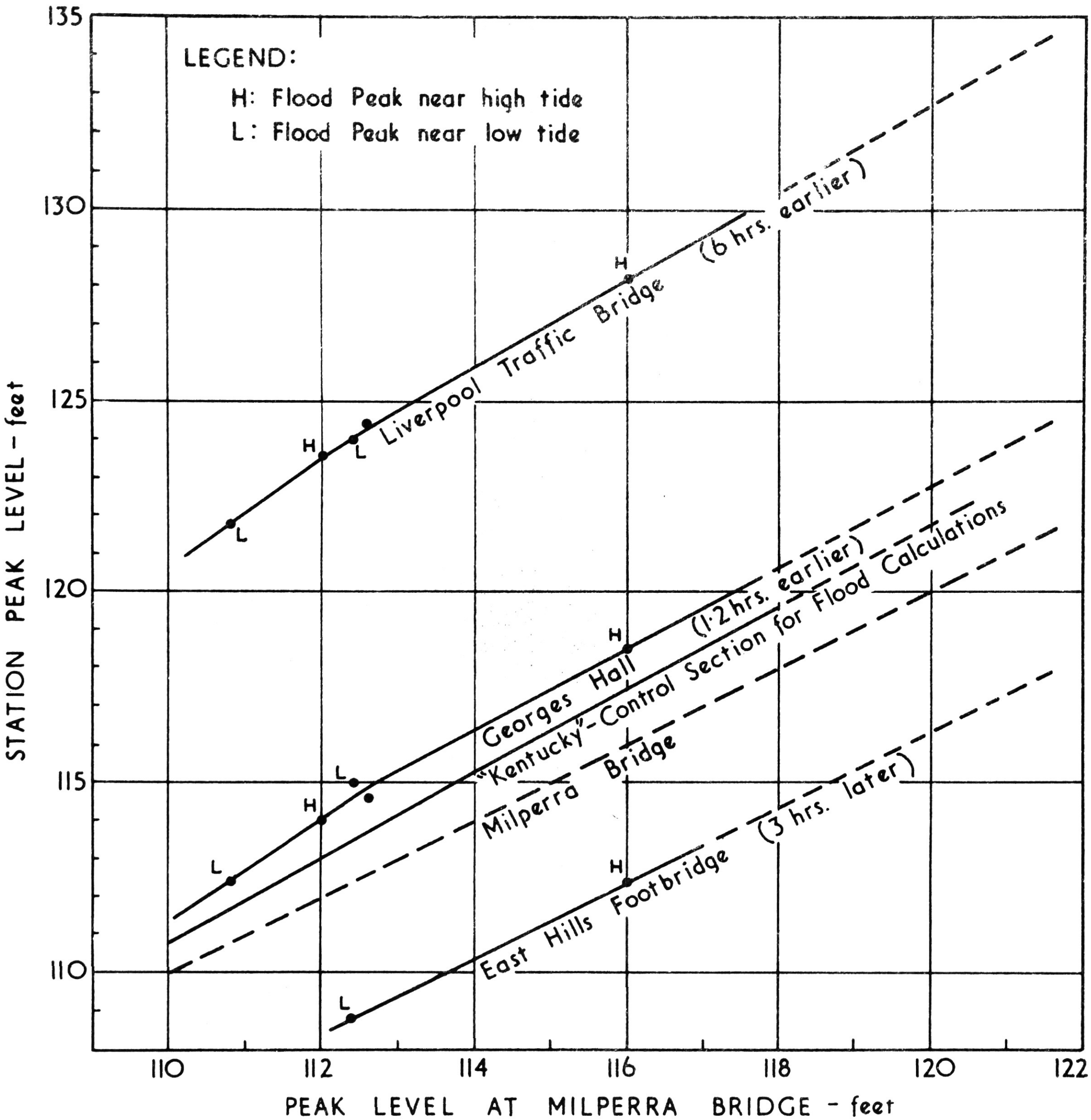


Figure 29: Georges River - Flood Peak Water Level Relationships.

CE-E-7163

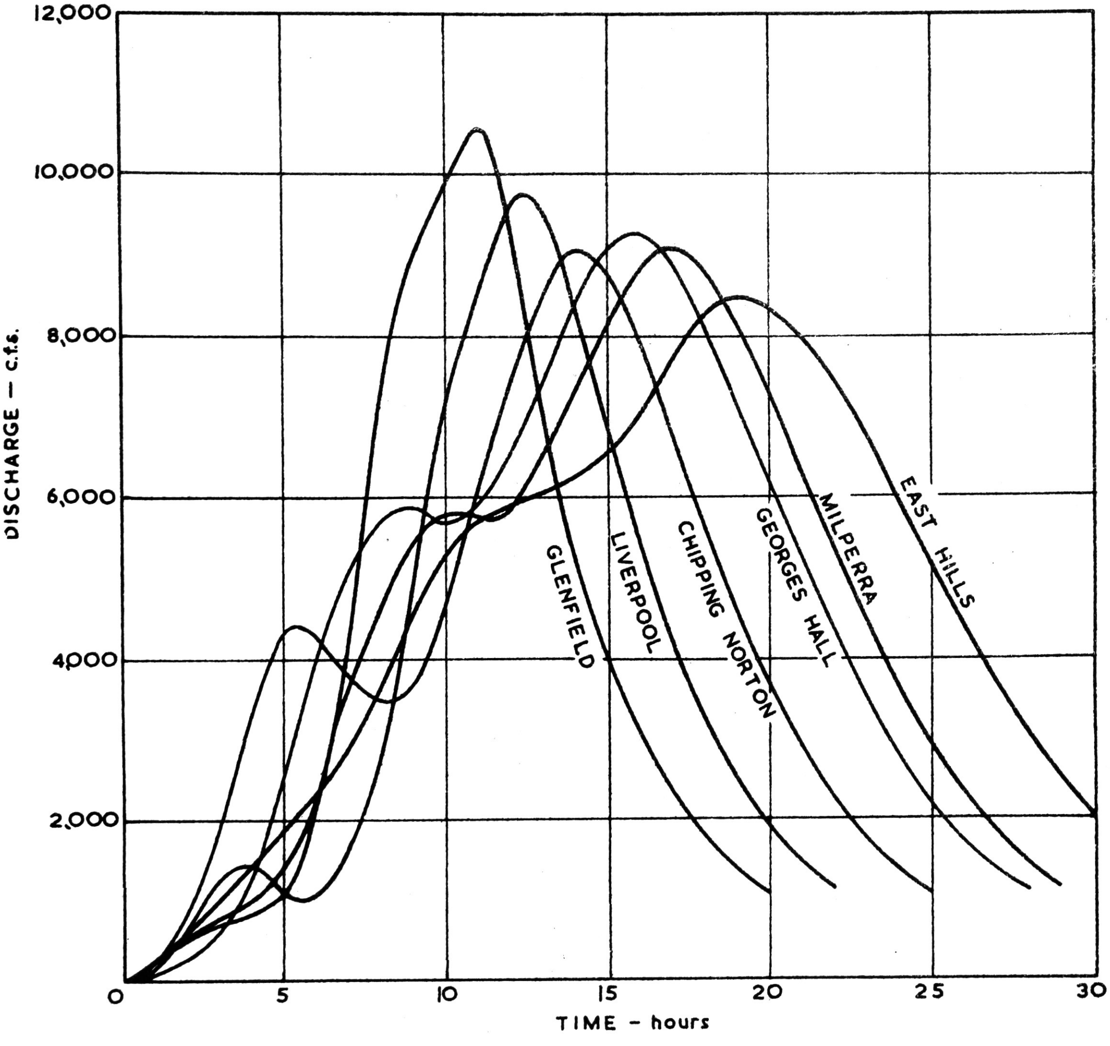


Figure 30: Georges River Unitgraphs

CE-E-7164

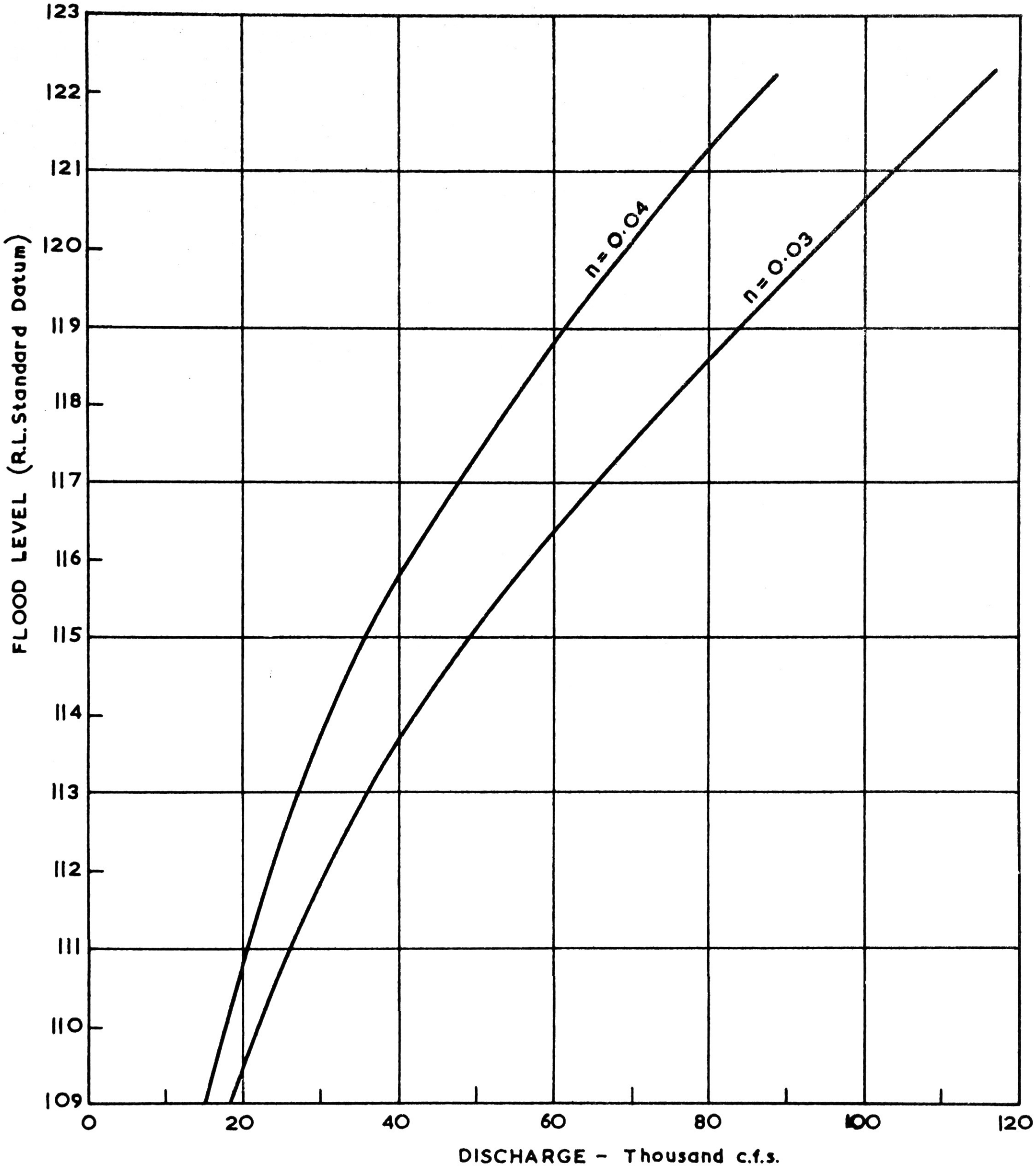


Figure 31: Estimated Flood Discharges at Milperra.

CE-E-7166