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Publication details:
Commissioning Body: Housing Commission of New South Wales
Report No. UNSW Water Research Laboratory Report No. 25

Publication Date:
1960

DOI:
https://doi.org/10.4225/53/578821948edbe

THE UNIVERSITY OF NEW SOUTH WALES

WATER RESEARCH LABORATORY

REPORT No. 25

Cabramatta Creek Flood Investigation

Hoxton Park – Green Valley Housing Development

by

E. M. Laurenson

SEPTEMBER, 1960

The University of New South Wales

WATER RESEARCH LABORATORY

CABRAMATTA CREEK FLOOD INVESTIGATION

Hoxton Park - Green Valley Housing Development

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E.M. LAURENSON

Report to

The Housing Commission of New South Wales

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PREFACE

This investigation was undertaken by the Water Research Laboratory of the University of New South Wales, Manly Vale N.S.W. for Unisearch Ltd. on behalf of the Housing Commission of New South Wales.

The investigation was carried out by Mr. E.M. Laurenson B.E., A.M.I.E., Aust., Lecturer in Civil Engineering. Some of the routine computations and drafting was performed by members of the University's technical staff under Mr. Laurenson's direction. Land survey data, aerial photographs and the general plan of the proposed development scheme were provided by the Housing Commission of New South Wales. Some of the information used in the hydrologic investigation was drawn from data obtained from nearby experimental catchments operated by the School of Civil Engineering of the University of New South Wales.

The study was commenced on 15th July 1960 and completed on 21st September 1960.

H.R. Vallentine
Associate Professor of Civil Engineering
Officer in Charge of the Water Research Laboratory.

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SUMMARY

Portion of the area of the proposed Hoxton Park - Green Valley Housing Development is subject to occasional flooding. This report describes the field, hydrologic and hydraulic investigations undertaken with a view to determining the extent and frequency of this flooding, and appropriate flood mitigation measures that should be undertaken.

Design flood estimation was carried out by the rainfall intensity - loss rate - synthetic unitgraph procedure for various frequencies. Details of the recent flood history of the area and of certain stage-discharge relationships were obtained by preparing discharge estimates for many recent rises by the above hydrologic method, and by local enquiry. Other stage-discharge relations were obtained by hydraulic computation. Detailed back-water computations determined the areas which would be subject to flooding with various frequencies.

Numerous considerations led to the recommendation of levee construction as the most appropriate method of reducing the area subject to flooding. A total length of 14,000 feet of levees is recommended together with a number of other minor improvement works at an estimated total cost of £14,400. These works will make available for development an area of approximately 148 acres which would otherwise have been subject to serious flooding.

LIST OF PLANS

The following plans are complementary to this report:-

Drawing No.1 " " 2 " " 3	-	Extent of flooding under natural conditions for once-in-5 year and once-in-40 year floods.
" " 4 " " 5	-	Proposed levee banks, and extent of flooding with levees in position for once-in-5 year, once-in-40 year and once-in-1000 year floods.

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1. INTRODUCTION

1.1 Nature of Investigation

This report describes a hydrologic and hydraulic investigation of flooding caused by overflow from Cabramatta Creek in the area of the proposed Hoxton Park - Green Valley Housing Development, about two miles west of the town of Liverpool, New South Wales; and of the measures which might be adopted to reduce this flooding and mitigate its effects on the proposed development. Figure 1 (bound at the end of this report) shows the catchment area of Cabramatta Creek, the project area (cross hatched), and other main features of the area. The nomenclature of Figure 1 is adopted throughout the report.

1.2 Description of Catchment and Streams

Above Mulgoa Road, the downstream boundary of the proposed development, the catchment area is 23 square miles. Generally, the catchment consists of a moderately steep, rolling section along the western edge, and a flat section. The steeper part falls from a general elevation of 350 feet to about 100 feet in a distance of approximately 1½ miles, while the flat part has a slope of about 15 feet per mile towards the east. Soils on the catchment are clayey, and the vegetation consists mainly of short natural grasses with scattered timber, some areas of improved pasture, and occasional patches of cultivation.

Of the three main streams comprising the drainage net, Hinchinbrooke Creek flowing southward and Cabramatta Creek flowing northward, drain the steep western strip, and join about ½ mile above the proposed development area. Cabramatta Creek then flows generally eastward through the flat portion of this area to join with Maxwell's Creek, and then turns generally northeastward until it reaches Orange Grove Road well downstream of the area. Maxwell's Creek drains a large, mainly flat portion of the total catchment.

That portion of Cabramatta Creek in the area of the proposed development has a winding course, an abruptly varying cross section, steep banks and a wide flat flood plain. The banks support a thick growth of scrub and timber, mainly she-oaks with a proportion of wattles and, in the lower part of the area, willows. The flood plain contains a number of wide shallow flood channels discernible both in the field and from aerial photographs.

Maxwell's Creek, although it drains a catchment area of 6 sq.miles, has a significant main channel for only about 1000 feet above its confluence with Cabramatta Creek. Above this, the creek has a very small channel for about 1000 feet and there degenerates into a wide flat floodway with no defined channel.

1.3 Outline of Investigation

The investigation of flooding described in this report consisted of four main parts, which will be described in the next four sections of the report. They are:-

Firstly, estimation of peak flood discharges for recurrence intervals of 5, 20, 40 and 1000 years for Cabramatta Creek at Mulgoa Rd., Cabramatta Creek at Hoxton Park Road and Maxwell's Creek at its confluence with Cabramatta Creek.

Secondly, estimation of the peak discharges of all significant floods at the above three locations during the period January 1956 to September 1960 by hydrologic methods and estimation of past flood heights by local inquiry.

Thirdly, estimation of the water surface profiles and extent of flooding for the once-in-5 year and once-in-40 year floods throughout the area of the proposed development.

Fourthly, investigation of possible measures for mitigating the effects of future flooding in the area.

2. DESIGN FLOOD ESTIMATION

2.1 Method Adopted

In the case of small, ungauged catchments such as this, the best available method of flood estimation is the rainfall intensity - loss rate - unitgraph method with unitgraphs synthesized by the Clark-Johnstone procedure.

This method was made particularly attractive by the existence of several years of rainfall and runoff records for two similar, nearby experimental catchments, South Creek and Eastern Creek, operated by the University of New South Wales. The availability of these records enabled the unitgraph synthesis and loss rate estimation to be carried out with a considerable degree of reliability.

2.2 Unitgraphs

As flood estimates were required for three points, Cabramatta Creek at Mulgoa Road and at Hoxton Park Road, and Maxwell's Creek at Hoxton Park Road, it was necessary to synthesize a unitgraph for each of these points. Since the drainage net of Cabramatta Creek above Hoxton Park Road consists of two main streams, Hinchinbrooke Creek and Cabramatta Creek, the unitgraph for this point was obtained by synthesizing unitgraphs for the two sub-catchments and combining them. The unitgraph of Cabramatta Creek at Mulgoa Road was obtained by routing this combined unitgraph downstream to Mulgoa Road adding in the Maxwell's Creek unitgraph, and also adding an allowance for the additional area drained by Cabramatta Creek between Hoxton Park Road and Mulgoa Road.

The basic method of unitgraph synthesis used was the Clark-Johnstone method. However, for greater reliability, the empirical relations between the physiographic characteristics of the catchment and the unitgraph parameters were rederived from the actual records of the South Creek and Eastern Creek Experimental Catchments, both of which are similar to and near to the Cabramatta Creek catchment. This rederivation involved detailed study of all recorded floods on the experimental catchments, and numerical evaluation of the catchment characteristics. Evaluation of the physiographic characteristics of the catchments of Hinchinbrooke Creek, Cabramatta Creek and Maxwell's Creek, and application of the empirical relations derived as described above, followed by the routing of the time-area diagrams through channel storage then led to determination of the unitgraphs for the three sub-catchments.

As the confluence of Hinchinbrooke Creek and Cabramatta Creek occurs only about ½ mile above Hoxton Park Road, the unitgraph of Cabramatta Creek at Hoxton Park Road was obtained simply by adding the unitgraphs of the two sub-catchments above this point. Determination of the Mulgoa Road unitgraph then involved the routing of the Hoxton Park Road and Maxwell's Creek unitgraphs downstream to Mulgoa Road, combining, and adding an allowance for the additional catchment between the two sites. An approximate routing procedure was necessarily used, as there were no streamflow records available for accurate determination of the storage-discharge relation. Thus, estimates were made of the peak volume of channel storage in the reach for the unit flood and the time of travel of the flood wave through the reach. An outflow hydrograph was then superimposed on the inflow hydrograph in such a way as to conform with the above two estimates.

Allowance for the area draining to Cabramatta Creek between Hoxton Park Road and Mulgoa Road was made by synthesising a unitgraph for the area by Sherman's method, based on geometric similarity. As the area is quite similar topographically to the Maxwell's Creek catchment, the Maxwell's Creek unitgraph was used as a basis, and its ordinates and abcissae were reduced in proportion to the square roots of the respective catchment areas. The resulting unitgraph was added to the two routed unitgraphs to give the total unitgraph for Cabramatta Creek at Mulgoa Road.

Unitgraphs for the three locations at which flood estimates were required were adjusted to a unit period of 1 hour, and a summary of the significant features of those unitgraphs is given in Table 1.

TABLE 1.

UNITGRAPHS

(Unit Period = 1 hour)

Catchment	Area sq. m.	Unitgraph Peak cfs	Unitgraph Time to Peak hrs
Cabramatta Ck. at Mulgoa Road	22.8	2680	4.5
Cabramatta Ck. at Hoxton Park Road	13.8	1930	3.0
Maxwell's Creek at outlet	6.0	820	3.0

2.3 Design Loss Rate

The type and condition of the soils and vegetal cover on Cabramatta Creek catchment are very similar to those on the University of New South Wales Experimental Catchment at South Creek. Since these are the main factors, apart from antecedent wetness, which determine the loss rate for a catchment, the derived values of loss rate on South Creek will give a very good indication of the loss rates on Cabramatta Creek. Study of derived loss rates for some ten storms on the South Creek catchment indicated a median value of 0.07 in/hr.

Since the method of flood estimation used involves determination of rainfall intensities of various frequencies as a step in estimating the floods of the same frequencies, the median loss rate is the appropriate value to use. Any other value would involve the compounding of probabilities and lead to a flood estimate of different probability to the storm causing it. Accordingly, the value of loss rate adopted for flood estimation on Cabramatta Creek was 0.07 in/hr.

2.4 Design Recurrence Intervals

Before estimating the design rainfall intensities, it was necessary to determine appropriate design recurrence intervals. In view of the nature of the proposed development in the areas likely to be subject to flooding, and after consultation with officers of the Housing Commission, it was decided that the range of interest was in floods with average recurrence intervals of from 5 to 40 years. Accordingly, rainfall intensities and flood magnitudes were estimated for recurrence intervals of 5, 20 and 40 years.

It should be noted that these recurrence intervals are average values and that actual recurrence intervals between floods of the estimated magnitude may be much more or much less than the average value. Also, the probabilities normally associated with the above recurrence intervals, 20 per cent, 5 per cent and 2.5 per cent respectively, are annual probabilities or the probabilities that the flood will be exceeded in any one year. The probability that the floods will be exceeded at least once during the lifetime of the houses is very much greater. As a significant example, the probabilities that the 5, 20, 40 and 1000 year floods will be exceeded once or more during a sixty year period are given in Table 2.

TABLE 2

PROBABILITIES OF FLOODS BEING EXCEEDED IN 60 YEARS.

Average Recurrence Interval - years	Annual Probability - per cent	60 Year Probability - per cent
5	20	99.9998
20	5	95.4
40	2.5	78
1000	0.1	5

It will be seen from Table 2 that there is a 78 per cent chance, or nearly 4 chances out of 5, that the "once-in-40 year flood" will be exceeded once or more during any 60 year period.

2.5 Design Storms

Peak flood discharges are affected by a number of storm factors apart from frequency or average recurrence interval, the principal ones being mean rainfall intensity, storm duration, and temporal pattern of the storm. It is not possible to determine directly the critical storm duration for any particular catchment. Rather is it necessary to apply several design storms of different durations to the unitgraph and compute the hydrographs that would be produced by these several storms. Only then does it become apparent what storm duration produces the highest flood.

Accordingly, eight mean rainfall intensities for durations varying from 2 to 14 hours and for a recurrence interval of 20 years were computed. The procedure and data used for these computations were those developed by the Stormwater Standards Committee of The Institution of Engineers, Australia, and published by The Institution in "Australian Rainfall and Runoff". When the critical storm durations for the three catchments for the 20 year recurrence interval were determined, mean rainfall intensities of these durations for the 5 year and 40 year recurrence intervals were determined in the same manner.

Actual rainfall intensities throughout a storm rarely approach uniformity, and since a non-uniform storm nearly always produces a higher peak discharge than would a uniform storm of the same duration and volume of precipitation, it is necessary to arrange the rainfall intensities into some temporal pattern. The temporal pattern adopted in this case was a high early peaking pattern developed from study of a large number of storm patterns recorded at the Sydney Weather Bureau.

2.6 Design Flood Discharges

The design loss rate, assumed uniform throughout the storm, was deducted from the rainfall intensities of the several design storms, and the resulting rainfall-excess hyetographs of 20 year recurrence interval were applied to the three unitgraphs to produce the flood hydrographs. By inspection of the plotted hydrographs, the critical storm duration for each of the three locations was determined, and the 5 year and 40 year flood hydrographs corresponding to these storm durations were computed.

Interest subsequently arose in the magnitude of the once-in-1000 year flood, and this flood, for each of the three locations, was estimated by plotting and extrapolation of a flood frequency curve determined by the 5,20 and 40 year floods. The estimated peak discharges for all recurrence intervals and all three locations are listed in Table 3.

TABLE 3

ESTIMATED PEAK FLOOD DISCHARGES

Values in cfs

Catchment	Area	Recurrence Interval - yrs.			
	sq. m.	5	20	40	1000
Cabramatta Ck. at Mulgoa Road	22.8	5100	7200	8000	13000
Cabramatta Ck. at Hoxton Pk. Road	13.8	3400	4600	5100	8600
Maxwell's Ck. at outlet	6.0	1400	1900	2100	3500

2.7 Effect of Proposed Development

Urbanization of a rural area of course leads to higher and quicker runoff from that area. However, this will have virtually no effect on the peak discharges estimated above, as the runoff from the area of the proposed development, even under natural conditions, is almost entirely discharged before the peak of the flood in Cabramatta Creek occurs. The more efficient stormwater drainage, which will accompany housing development, will therefore tend to increase discharge in the early part of the flood, but leave the peak discharges of the design floods virtually unaffected. It is assumed that intensive housing development is unlikely to occur over any large part of the catchment area above the Hoxton Park - Green Valley Development.

3. STUDY OF PAST FLOODS

3.1 Local Flood Information

Officers of the Liverpool Municipal Council, and a number of local residents of long standing, who were not financially interested in the land resumptions associated with the housing development, were interviewed in order to obtain local knowledge of flooding. As is usual in this type of survey, a considerable amount of information on flood levels, some of it conflicting, was obtained, but very little regarding the dates of particular flood levels could be recollected. However, intelligent assessment and comparison of the various reports led to a fairly clear picture of the recent flood history of the area. Naturally, most information concerned the points where the creeks cross Mulgoa Road and Hoxton Park Road, the three sites for which flood estimates have been prepared.

Floods covering the road occurred at these locations about three times in February and March 1956, again in February or March 1958 or both, but none has occurred since that time. A large number of floods also occurred in 1949 and 1950.

Flood level information obtained led to the following conclusions:-

(i) The highest flood in February-March 1956 (subsequently determined to be that occurring on 9-10 February) reached a level of RL.42.5 at the Mulgoa Road bridge across Cabramatta Creek.

(ii) A level of RL.80.5 near the corner of Banks Road and Hoxton Park Road has been reached, or closely approached, a number of times since the beginning of 1956.

(iii) A level of RL.5315 where Maxwell's Creek crosses Hoxton Park Road has been reached a number of times since the beginning of 1956.

Of the considerable body of flood information obtained apart from that noted above, the most significant points are:-

(i) A number of floods within living memory has overtopped the banks of the creek at several places and caused flow down the many shallow floodways mentioned in sub-section 1.2 of this report, but the plain has never been completely submerged.

(ii) At the upper end of the area, flooding occurs across both Joadja Road and Banks Road.

(iii) The period for which Hoxton Park Road is submerged at Maxwell's Creek and Cabramatta Creek is usually of the order of 4 to 8 hours.

3.2 Estimation of Past Flood Discharges

In order to define the relationship between stage and discharge at each of the three locations under review, the peak discharges of all significant rises in the period January 1956 to September 1960 were calculated. Comparison of these discharge estimates with the information on flood levels and occurrence given above fixed one point on the stage discharge relation at each site. This information was later supplemented by hydraulic computations.

Estimation of the peak discharges of past floods was carried out by means of the rainfall intensity - loss rate - unitgraph method already described. Ten storms were selected for study, these being the storms causing the largest rises on the South Creek and Eastern Creek Experimental Catchments of the University of New South Wales. Daily rainfall information for several stations surrounding the Cabramatta Creek catchment was obtained from the Commonwealth Bureau of Meteorology and the University's own records. Storm durations and temporal patterns were obtained from the pluviometer records of the Eastern Creek, South Creek and Mt. Vernon Experimental Catchments. For each storm, the following procedure led to the peak discharge estimates for the three sites:-

(i) From the pluviometer records, the storm duration and the proportion of the total daily rainfall occurring in the intense part of the storm were determined.

(ii) An isohyetal map for the intense part of the storm was drawn, and the average depth of rainfall over each of the three catchment areas was determined.

(iii) Estimates were made of the initial loss and loss rate, based on loss rate studies of the same storm on the South Creek catchment.

(iv) The hyetographs of rainfall-excess determined from Steps (i), (ii) and (iii) above for the three catchments were applied to the appropriate unitgraphs, synthesized as described earlier in this report, and the peak discharge estimates resulted.

Of the ten storms studied, only eight produced significant rises in Cabramatta and Maxwell's Creeks, and the peak discharge estimates associated with these eight storms are given in Table 4. It might be noted, that the highest of these floods, that caused by the storm of 9-10 February 1956, is almost exactly equal to the estimated once-in-40 year flood as given in Table

TABLE 4.

ESTIMATES OF PAST FLOOD DISCHARGES

Values in cfs.

Date of Storm	Cabramatta Ck. at Mulgoa Rd.	Cabramatta Ck. at Hoxton Park Rd.	Maxwell's Creek
9-10 Feb. 1956	8200	5000	2200
19-21 Feb. 1956	1750	1050	650
1-2 Mar. 1956	2900	1900	850
9 Mar. 1956	1600	1050	450
23-25 June 1956	1600	1050	450
9 Feb. 1958	2500	1800	750
10-11 Mar. 1958	3000	1700	950
30 Oct. 1959	1900	1400	550

3.3 Stage-Discharge Relations

Estimation of the flood levels associated with the once-in-40 year floods at the three road crossings was necessary for the determination of extent of flooding over the whole area. These estimates were made in two ways, firstly by comparison of the information on past flood levels and past flood discharges contained in sub-sections 3.1 and 3.2 above, and secondly by hydraulic computation. The deductions made on these two bases are as follows:-

(i) Cabramatta Creek at Mulgoa Road. It has been estimated that the flood resulting from the storm of 9-10 February 1956 had a peak discharge of 8,200 cfs and a peak level of RL.42.5. Since the once-in-40 year flood has a peak discharge of 8000 cfs, it follows that the once-in-40 year flood has a peak stage of RL.42.5.

Peak stage for the once-in-5 year flood was estimated by hydraulic computations as described in Section 4 below. These were based on Manning's formula, an estimated value of roughness coefficient, and known stream cross sections, together with computation of the backwater that would be caused by by the bridge, and this resulted in a peak level, on the upstream side of the bridge, of RL.39.5 for the 5100 cfs peak discharge.

(ii) Cabramatta Creek at Hoxton Park Road. It is known that floodwaters have reached or closely approached a level of RL.80.5 near the corner of Banks Road a few times since 1956, but not since March 1958. Consideration of this fact in relation to Table 4 indicates that this level is associated with a discharge of about 3000 cfs. It is also known that, at this elevation, the width of flow is very wide (of the order of 1200 ft.) so that little change in stage would occur for large changes in discharge. On the basis of these facts, the peak stage of the once-in-40 year flood was estimated as RL.81.0 and that of the once-in-5 year flood as RL.80.5.

(iii) Maxwell's Creek at Hoxton Park Road. A level of RL.53.5 (which is only a few inches above deck level of the bridge) is known to have been reached a number of times since the beginning of 1956 including one or two occasions in February-March 1958, but the road has not been flooded since this latter date. It therefore appears, from study of Table 4, that this level correspond to a discharge of about 750 cfs. Approximate extrapolation led to estimates of the peak levels for the 40 year and 5 year floods of RL.55.0 and RL.54.0 respectively.

For convenient reference, the stage-discharge relationships detailed above are summarised in Table 5. Stages associated with the once-in-20 year flood could be obtained by interpolation, but were not used in this investigation.

TABLE 5

STAGE-DISCHARGE RELATIONS

Location	Recurrence Interval-yrs.	Peak Discharge-cfs	Peak Stage
Cabramatta Creek at Mulgoa Road	5 40	5100 8000	RL.39.5 RL.42.5
Cabramatta Creek at Hoxton Park Road	5 40	3400 5100	RL.80.5 RL.81.0
Maxwell's Creek at Hoxton Park Road	5 40	1400 2100	RL.54.0 RL.55.0

4. EXTENT AND FREQUENCY OF FLOODING

4.1 Flood Level Computations

Flood levels for the 5 and 40 year recurrence intervals at the downstream and upstream ends of the area have been determined as described above. To determine the extent of flooding over the area, flood levels were estimated at a large number of representative sections distributed along the two streams. These estimates were made by hydraulic computation, and it was found that the results were entirely consistent with the qualitative local information on flooding previously obtained.

Water surface profile computation for natural streams is a step by step trial and error procedure based on the energy balance equation and Manning's formula. The method adopted for computation of the peak levels reached at various sections by the once-in-5 year flood is the standard step method, and the particular form of its application is that described by Lara and Schroder in Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, April 1959. A feature of this method is that it takes specific account of the difference in length of flow path, and also the roughness coefficien of the main channel flow and the overbank flow. This was a particular advantage in this study since a considerable portion of the total discharge occurs as overbank flow, and this usually has a shorter flow path than the main channel.

Application of the method requires the prior estimation of Manning's roughness coefficient (n) for the main channel and overbank areas at all sections of the stream. Values of this coefficient adopted after detailed inspection of the streams are shown in Table 6. These values were assessed in the field by reference to appropriate sections of the following publications, which are the most recent and most authoritative references on this question:-

(i) Chow - "Open Channel Hydraulics".

(ii) U.S. Bureau of Public Roads - "Computation of Backwater Caused by Bridges".

(iii) Rouse (Ed.) - "Engineering Hydraulics".

TABLE 6

VALUES OF ROUGHNESS COEFFICIENT "n"

Stream	Reach	Roughness Coeff. "n"	
		Main Channel	Overbank
Cabramatta Ck.	Above Hoxton Park Rd.	0.11	0.07
	Hoxton Pk. Rd. to Maxwell's Ck.	0.11	0.04
	Maxwell's Ck. to Mulgoa Rd.	0.09	0.04
	Mulgoa Rd. to Orange Grove Rd.	0.055	0.03
Maxwell's Ck.	Below lagoon	0.055	0.03
	Above lagoon	0.08	0.03

Cross sections used in the computations were those surveyed by Barrie and Tait Pty.Ltd. for this purpose, and the computations were carried from section to section in an upstream direction. The "initial" water surface elevation for the 5 year flood at Mulgoa Road was determined by starting the computations at an arbitrarily assumed elevation well downstream at a section near Orange Grove Road. Since the computed elevations tend to converge to the correct values, regardless of any initial error, as the computations proceed upstream, this led to the correct value at Mulgoa Road. A check on this convergence was made by starting from a second, completely different elevation near Orange Grove Road, and this second series led to the same normal elevation at

Mulgoa Road, RL.39.0 Backwater caused by the Mulgoa Road bridge was estimated at 0.5 ft. as described in sub-section 4.2 below, so that the water surface elevation of the 5 year flood on the upstream side of Mulgoa Road is RL.39.5.

From this "initial" elevation, the water surface elevations at successive sections upstream were calculated. Main steps in the procedure were:-

(i) For each cross section, area of flow and hydraulic radius of both main channel and overbank areas were determined for various water surface elevations.

(ii) Conveyance (K_d) of both main channel and overbank areas was computed from the area, hydraulic radius, and roughness coefficient.

(iii) Area of flow and conveyance were plotted against water surface elevation.

(iv) For the first reach, the downstream elevation is known, and a trial value of upstream water surface elevation is assumed.

(v) Length of flow path for both main channel flow and overbank flow is determined.

(vi) A weighted average friction head loss, that would apply to the reach if the slopes throughout were the same as at the downstream end, is calculated. The weighting is in accordance with the proportions of the total flow following the different paths and thus having different slopes.

(vii) Step (vi) is repeated assuming the slopes throughout are the same as those at the upstream end.

(viii) Actual friction head loss for the reach is computed as the average of the values calculated in (vi) and (vii). In a few cases, this was a weighted average as it was apparent that the slopes applying at one end of the reach were predominant over more than half the length.

(ix) Velocities in the main channel and overbank areas are computed for the two ends of the reach.

(x) The weighted average velocity head is computed for each end of the reach.

(xi) The algebraic change in velocity head is determined.

(xii) Eddy loss is estimated as 10 per cent of change in velocity head for convergent flow and 50 per cent for divergent flow.

(xiii) Total loss is determined as friction loss plus eddy loss.

(xiv) Change in water surface elevation is determined as total loss plus change in velocity head.

(xv) Upstream water surface elevation is computed.

(xvi) Steps (iv) to (xv) are repeated until the computed upstream elevation equals the assumed trial value to a satisfactory degree of approximation.

(xvii) Step (xvi) determines the downstream elevation for the reach next upstream, and the whole procedure is repeated for this next reach, and so on.

The above procedure led to estimates of the peak level that would be reached by the once-in-5 year flood at various cross sections. Further calculations were then carried out to determine the once-in-40 year flood levels at the same sections. Firstly, the additional discharge of the 40 year flood as compared with the 5 year flood was determined for each Section. Then a trial water surface elevation was assumed and the additional cross sectional area available in both the main channel and overbank sections over that existing for the 5 year peak discharge was determined. Flow velocities for the 40 year flood in both the main channel and overbank sections were then estimated, allowance being made for the velocity increase due to the increased hydraulic radius. From these velocities and the additional cross sectional areas, the additional discharge was computed, and compared with the excess of the 40 year over the 5 year flood. This procedure was repeated until the trial water surface elevation resulted in the correct value of additional discharge.

4.2 Bridge Backwater Computation

An essential part of the calculations outlined in sub-section 4.1 above is the computation of backwater due to the Mulgoa Road bridge for the once in 5 year flood. This was necessary to determine the water level on the upstream side of the bridge from the computed normal level, that would exist in the absence of the bridge and its approach embankments, this latter figure having been obtained from the water surface profile computation started from well downstream near Orange Grove Road.

Estimation of the backwater effect followed the lines recommended in the U.S. Bureau of Public Roads draft publication, "Computation of Backwater Caused by Bridges", except that normal stage was estimated as described in sub-section 4.1 above rather than by a direct application of Manning's formula. This procedure takes account of the differences in hydraulic radii and roughness coefficients of the different parts of the channel cross section, the degree of flow constriction caused by the approach embankments, the obstruction caused by the piers, and the nett change in velocity head from one side of the bridge to the other. Eccentricity of the main flow with respect to the channel centre line, and skewness of the bridge crossing with respect to the channel, are not significant factors in this particular case.

It was calculated that the backwater caused by the bridge for the 5 year flood would be 0.5 feet. Pertinent levels for the 5 year flood are thus:-

Normal stage	RL.39.0
Underside of bridge girders	RL.39.2
Water level on upstream side of embankments	RL.39.5
Kerb level	RL.42.2

Since flow through the bridge opening occurs at normal stage, it will be seen that the 5 year flood just clears the girders, and a small increase in discharge would result in submergence of the flow and a considerable increase in stage. Thus, the 40 year flood here has a peak level of RL.42.5, three feet higher than the 5 year flood, while elsewhere in the area, it is of the order of one foot higher.

Backwater caused by the bridge for the once-in-40 year flood could not be estimated by the procedure used for the 5 year flood due to the complete submergence of the bridge and its approaches. Had it been possible to estimate this backwater, this would have served as a check on the level of RL.42. previously estimated. However, it is interesting to note that the computed normal water surface elevation for the 40 year flood is RL.41.1, indicating backwater of 1.4 feet, which is quite a reasonable figure in relation to all the circumstances.

4.3 Results

Estimated flood levels for the 5 and 40 year floods, determined as described above are tabulated in Table 7. Cross sections listed in this table are those surveyed by Barrie and Tait Pty.Ltd. for this job, but in cases where the general direction of the overbank flow is different from that of the main channel, the extension of the cross section across the flood plain is taken normal to the general direction of overbank flow. The section listed as 5/6 is taken midway between Cross Section 5 and Cross Section 6. Peak levels for the 20 year flood can be interpolated between the figures given for the 5 and 40 year floods if necessary, with due regard to the estimated discharges for the three recurrence intervals.

TABLE 7.

ESTIMATED PEAK FLOOD LEVELS

(NATURAL CONDITIONS)

Cross Section	Estimated Flood Level (R.L.)	
	5 yr. Flood	40 yr. Flood
29	39.5	42.5
27	41.0	-
26	45.1	46.6
25	46.1	-
24	47.2	-
21	50.1	51.0
20	51.5	-
19	53.4	54.5
17	58.4	59.0
15	62.0	62.9
12	67.3	67.8
10	69.1	-
8	71.9	72.5
5/6	76.8	-
Banks Rd.	80.5	81.0
2	51.9	53.5
1	53.8	55.0

4.4 Areal Extent of Flooding

The areal extent of flooding that would occur in the area of the proposed housing development under natural conditions with average recurrence intervals of 5 years and 40 years is shown on Drawings 1, 2, and 3 submitted with this report. These plots are based on the flood levels of Table 7, which fix points at a number of cross sections. Lines joining these control points were drawn with due regard to changes in water surface slope from section to section. Depth of flooding at any particular point can be determined from the topographic contours on the plans.

It will be noted that both the 5 year flood and the 40 year flood cause the creek to overflow its banks at a number of locations, resulting in flow down the shallow floodways on the plain, but even in the case of the 40 year flood, there are still high spots on the plain which are not submerged. This general result, as well as many details regarding particular areas subject to flooding, is consistent with information obtained from local residents regarding the extent of flooding.

A further point to note regarding the information on Drawings 1, 2 and 3 is that much of the area inundated will be submerged for only a short period, as the flood peaks are generally of short duration. The total duration of over-bank flow is probably of the order of 4 to 8 hours.

5. FLOOD MITIGATION

5.1 General Method

It should be realised that complete control of floods is usually an extremely difficult and expensive proposition, and that usually, development on an area naturally subject to flooding will always be subject to some slight risk of flooding no matter what engineering works are carried out. In this particular case, a large area on which it is desired to build houses is naturally subject to flooding, and while it is possible, by levee construction, to reduce considerably the area that would be inundated by floods much larger than anything within living experience, a slight possibility of the levees being overtopped still exists, and should be recognised.

Before deciding to recommend levee construction, all possible methods of flood mitigation were considered, and brief comments on each method in relation to the Cabramatta Creek situation are given below. The methods considered are:

1. Channel Improvement
   (a) Enlargement
   (b) Clearing
   (c) Cutoffs
   (d) New channels
2. Storage
   (a) Storage Reservoirs
   (b) Detention Reservoirs
3. Levees

Channel Enlargement

Enlargement of the existing channel to convey even the 5 year flood would involve a great amount of excavation, particularly because of the winding course of the stream, and this excavation would be very expensive due to the thick timber on the banks. Furthermore, the necessary destruction of most of the trees would remove the only existing relief to an otherwise flat and uninteresting landscape. Consequently, this method of flood mitigation is not recommended.

Channel Clearing. Clearing of the existing channel has two main aspects. Firstly, there is a large number of dead logs in the channel which collect debris during floods and obstruct the flow. These could be removed and burnt cheaply, and it is recommended that this be done. The reduction in flood levels of the large floods resulting from this work would be slight, but the frequency with which overbank flow occurred would be reduced and, if the area within the levees to be recommended later in this report is used as parkland and playing fields, this reduction of flooding frequency would be well worth the cost. The second aspect concerns the scrub and trees which grow thickly along both banks of the creek. Removal of this growth is not recommended for the following reasons:-

(i) The aesthetic aspect mentioned earlier.

(ii) The great expense of grubbing out the tree stumps, since mere felling and burning of the trees without stump grubbing would be ineffective in reducing flood levels.

(iii) In a few years, the trees would grow up again, as evidenced by the young growth of she-oaks in the lower part of Maxwell's Creek.

(iv) Removal of the tree roots would lead to considerable bank erosion with all its undesirable consequences.

Two further significant points must be mentioned under the heading of clearing the channel. Firstly, in the upper portion of Cabramatta Creek are two old concrete weirs, one a small structure about 3 feet high, and the other a large structure, now standing isolated between two flood channels, which have been eroded around the abutments causing complete failure of the weir. These structures appear to serve no useful purpose and, as they undoubtedly cause significant backwater effects during floods, they should be removed. Such removal will have only a limited and localised effect, but is considered worthwhile for the benefits it will have in reducing the frequency of overbank flow within the levees.

Secondly, there now exist in the extreme south-western corner of the area, two small earth dams. If the levee system recommended in this report is constructed, these dams would form a serious obstruction in the floodway, and for this reason, they also should be removed. Simple demolition of the earth walls, and spreading of the spoil over the surrounding area within the floodway is all that is required.

Cutoffs. The meandering of the creek is not highly enough developed to make the excavation of cutoffs a useful method of flood mitigation. If any minor cutoffs were constructed, their effect would be very localised, and would not reduce the frequency of overbank flooding in the reaches upstream and downstream of the cutoff. Consequently, the extent and frequency of flooding over the area as a whole would not be affected. It may be that three or four short cutoffs will be necessary in future to prevent undercutting of proposed levee banks, where these approach close to the existing channel. However, as the stream appears to be fairly stable in its present position, it is recommended that this work be carried out only if future experience shows it to be necessary.

New Channels. Serious consideration was given to the excavation of a wide shallow flood channel to act as a relief for the existing channel during high floods. It would be possible to construct such a floodway along the southern boundary of the development area adjacent to Hoxton Park Road, and thus make a large part of the flood plain safe for housing development. However, preliminary cost estimates indicated that this solution would be considerably more expensive than the levee system finally recommended. But for this, it would have been an attractive method of mitigating floods in the area upstream of the Maxwell's Creek confluence. Little scope for such a solution exists downstream of Maxwell's Creek.

Storage. Reduction of flood discharges by temporary storage in areas upstream of the project area was also investigated. In the circumstances, such storage would be in detention reservoirs with uncontrolled outlets rather than in storage reservoirs with gated outlets, due to the uncertainty of the latter being correctly operated during floods. Sufficient storage of this nature could probably be provided to reduce the peak of the 40 year flood by something of the order of 30 per cent, and, while detailed estimates were not prepared, the cost would probably be of the order of £60,000 to £70,000, which is considerably more than the cost of the recommended works. Furthermore, of the three possible dam sites, two on Hinchinbrooke Creek and one on Cabramatta Creek, two already have farm dams constructed on them. Also, the areas that would be subject to inundation in the detention reservoirs are to a large extent highly improved farmland, and the necessary resumption of easements in the reservoir area would thus be expensive, and would elicit strong (and to a large extent justifiable) opposition from the interested landowners. In view of all the circumstances, this method of flood mitigation is not recommended.

While construction of levees introduces its own particular problems, their cheapness, the valid objections to all other methods of flood mitigation noted above, and the possibility of substantially overcoming the problems in this case, make this the most desirable method of flood mitigation in this area. The problems introduced by the decision to recommend levee construction, and their solutions, are discussed below.

5.2 Levee Location

Constriction of flood plain flow by levees of course raises flood levels both in the section of stream leveed and for some distance upstream and downstream of the leveed section. This fact led to two general principles in levee location:-

(i) Levees would be located so as to cause negligible increase in flood levels outside the area to be resumed for the housing project.

(ii) A wide area for overbank flow not less than about 600 feet wide would be allowed. This principle was adopted so as to minimise upstream and downstream effects, levee height (and cost), and the damage that would be caused by the possible overtopping of levees. This last contingency is perhaps the most important, and to have a substantially narrower width for overbank flow than that provided in these recommendations would be to invite disaster in the event of levee failure or overtopping.

Further principles adopted in levee location were:-

(i) Levees should follow high ground where possible to minimise their height and cost.

(ii) The unimproved value of land protected by a levee should exceed its cost.

(iii) The floodway within the levees should not be unduly winding.

Adoption of the above principles led to the location of levees in the positions indicated on Drawings 4 and 5 submitted with this report.

It will be noted that the southern levee at the upstream end crosses Hoxton Park Road. This will necessitate reconstruction of some 400 feet of road to gradually rise and fall over the levee, and also involves levee construction outside the area to be resumed for housing development. These features, while undesirable, are unavoidable if the area is to be protected since a considerable amount of flood plain flow normally enters the area across Hoxton Park Road below the point where the proposed levee crosses this road.

5.3 Levee Height

Overtopping of levees in this project would have serious consequences, as houses will be built behind the levees, and these would obstruct the flow that would otherwise take place behind the levees. This would cause flood levels to be increased considerably above what they would be under natural conditions, and floodwaters would almost certainly rise above the floor levels of hundreds of homes. Consequently, a conservative, but reasonable, basis has been adopted for the determination of levee heights. These have been fixed to provide one foot of freeboard above the estimated crest level of the once-in-1000 years flood. The 1000 year flood has a 5 per cent chance of occurring in any 60 year period, but the foot of freeboard provided reduces the probability of overtopping very considerably below this figure.

Determination of levee height on this basis was carried out by estimating the increase in flood levels for the 5 and 40 year floods at various cross sections due to levee construction, estimating the further increase due to the increased discharge of the 1000 year flood, and adding freeboard. Briefly the procedure was:-

(i) Determine the cross-sectional area of natural flood flow cut off by the levees.

(ii) Multiply this area by the velocity of this flow to determine the discharge cut off.

(iii) Estimate the velocity of flow within the levees, and determine the additional cross sectional area required to transport the above discharge.

(iv) Thus calculate the increase in water level caused by the levees for the 5 and 40 year floods.

(v) Determine the excess of the 1000 year discharge over the 5 or 40 year discharge for the section being considered from Table 3.

(vi) Estimate the velocity of flow for the 1000 year flood and so compute the additional cross sectional area required.

(vii) Compute the increase in flood level of the 1000 year flood over the 5 to 40 year flood.

(viii) Add one foot of freeboard to the computed 1000 year flood level.

(ix) From the computed levee crest elevations at the various cross sections, the crest elevations at Points A to X on Drawings 4 and 5 were determined.

The above procedure led to levee crest elevations at points A to X as shown on the plans and listed below in Table 8. Crest elevations at Points A and B are tentatively shown at RL.81.0, but it may be necessary to reduce this slightly if this level cannot be reached on the southern side of Hoxton Park Road. If this is the case, the levee should be carried to the divide between the Cabramatta Creek and Maxwell's Creek drainage areas.

TABLE 8

LEVEE CREST ELEVATIONS

Point	R.L.	Point	R.L.
A	81.0	M	57.4
B	81.0	N	57.4
C	79.5	O	71.8
D	75.5	P	65.7
E	74.8	Q	63.1
F	74.0	R	59.6
G	73.0	S	54.5
H	71.8	T	53.4
I	67.7	U	53.2
J	65.7	V	51.7
K	62.2	W	50.7
L	58.4	X	50.7

5.4 Quantities and Costs

Earthwork quantities in the levee have been calculated on the basis of 3:1 side batters on both water and landward sides and a 3 foot crest width. Total length of levees is 14,000 feet. Levee costs are based on the assumptions that topsoil will be removed and subsequently respread over the levees, and that the banks will be formed by dozing up earth from the floodway immediately adjacent to the banks.

Estimated quantities and costs for all recommended works are as follows:-

A. LEVEE A TO N

Item	Description	Quantity	Unit	Rate	Amount
1.	Winning and Placing Earth including trimming of banks.	21,700	c.yds.	3/6	£ 3,795
2.	Preparation of Base	5.4	acres	£80	432
3.	Grass cover	5.7	"	£110	627
					4,854
4.	Contingencies			10 p.cent	485
					5,339
5.	Design, supervision and overhead			6 p.cent	320
	Total				£5,659
				Say	£5,700

B. LEVEE O TO Q

Item	Description	Quantity	Unit	Rate	Amount
1.	Winning and placing earth including trimming of banks	6,060	c.yds.	3/6	£1,060
2.	Preparation of Base	1.5	acres	£80	120
3.	Grass Cover	1.6	"	£110	176
					1,356
4.	Contingencies			10 p. cent	136
					1,492
5.	Design, supervision and overhead			6 p. cent	90
	Total				1,582
				Say	£1,600

C. LEVEE R TO S

Item	Description	Quantity	Unit	Rate	Amount
1.	Winning and placing earth including trimming of banks	1,540	c.yds.	3/6	£270
2.	Preparation of Base	0.5	acres	£80	40
3.	Grass Cover	0.6	"	£110	66
					376
4.	Contingencies			10 p. cent	38
					414
5.	Design, supervision and overhead			6 p. cent	25
	Total				439
				Say	£500

D. LEVEE T[illegible] TO X

Item	Description	Quantity	Unit	Rate	Amount
1.	Winning and placing earth including trimming of banks	7,830	c.yds.	3/6	£1,370
2.	Preparation of Base	1.6	acres	£80	128
3.	Grass Cover	1.7	"	£110	187
					1,685
4.	Contingencies			10 p.cent	169
					1,854
5.	Design, supervision and overhead			6 p.cent	110
	Total				1,964
				Say	£2,000

E. RECONSTRUCTION OF 400' SECTION OF HOXTON PARK ROAD.

Item	Description	Quantity	Unit	Rate	Amount
1.	Earthworks	1,000	c.yds.	16/-	£800
2.	Base Course	175	"	40/-	350
3.	Bituminous Surface Dressing	1,000	sq.yds.	5/-	250
					1,400
4.	Contingencies			10 p.cent	140
					1,540
5.	Design, supervision and overhead			6 p.cent	92
	Total				1,632
				Say £	1,700

F. CLEARING OF CREEK AND FLOODWAY

Item	Description		Amount
1.	Removal and burning of dead trees and logs from creek channel	Lump sum	£2,000
2.	Removal of two concrete weirs in upper section of Cabramatta Creek	" "	600
3.	Removal of two earth dams in floodway near Banks Road	" "	300
	Total		£2,900

Total cost of all the above recommended works is £14,400.

5.5 Effects of Proposed Works

Construction of the levees recommended above would make available for housing development, the following areas of land which would otherwise be unavailable:-

Levee A to N	-	Approximately	100 acres
" O to Q	-	"	21 "
" R to S	-	"	12 "
" T to X	-	"	15 "
Total			148 "

This land is considered unavailable for housing development under natural conditions since, even though much of it is now flooded to only shallow depths, construction of houses and fences, and other development without levee protection would so obstruct flood plain flows as to cause a rise in the 5 and 40 year flood levels of the order of 2 or 3 feet, so leading to considerable depths of inundation. The above figures for areas on the northern side of Cabramatta Creek, protected by levees O to Q, R to S, and T to X, are calculated on the assumption that the southern levee, A to N is in position; the areas made available by these northern levees would be somewhat less if the levee A to N were not constructed. This assumption was made since the levee A to N appears to be the most important single work in the project, and would be constructed in preference to any other work.

Generally, the increase in flood depths caused by the levees will result in the inundation of very little additional land in the unleveed sections. At the Mulgoa Road end of the area below Point X, the end of the levees, the extent of flooding for the 5 and 40 year recurrence intervals as shown on Drawings 1 and 3 will be unaffected by the levees. Likewise, on Maxwell's Creek, the extent of flooding in Hoxton Park Road and in the vicinity of the houses in Memorial Avenue will not be affected. At the unleveed sections along the northern bank of Cabramatta Creek, flood levels will be increased, the increases having been calculated in sub-section 5.3 above. The areal extent of flooding has been determined on the basis of these calculations, and the areas subject to flooding with frequencies of once-in-5 years and once-in-40 years with the levees in position are indicated on Drawings 4 and 5. At the upstream end of the area, the levee system has been so designed that no increase in flood depths over that occurring naturally will result outside the area to be resumed. On the south-eastern bank of Cabramatta Creek opposite levee T to X, natural flood levels will be increased about 0.5 feet by the levees, and the peak level of the 1,000 year flood will be one foot below the levee crest elevation on the north-western bank. This information has not been plotted on Drawing 5, however, due to the absence of adequate contours.

5.6 Limits to Development

While the areas within the levees may be used for parkland and playing fields, no substantial buildings, and no fences should be permitted in these areas, as such construction would obstruct the free flow of floodwaters, thus raising flood levels, and increasing the possibility of the levees being over-topped with all the consequences that occurrence would entail. For the same reason, no substantial buildings and no paling fences should be allowed to encroach into the floodway in the unleveed sections on the northern bank of Cabramatta Creek. It is considered that the limit to such development in these sections should be the level reached by the 1000 year flood, and this limit has therefore also been plotted on Drawings 4 and 5. However, if considered desirable for other reasons, it would not be unreasonable to allow fences of a post and rail type, but not chainwise fences, to encroach a limited distance, say not more than 100 feet, beyond this limit.

Approach embankments to any bridges in future constructed within the floodway could, of course, seriously impede the flow of floodwaters. Consequently, any such future bridge approaches should either be designed so as to cause negligible backwater effect, or the levees on the upstream side of the bridge should be raised sufficiently to allow for the increase in flood level due to backwater. This second alternative is likely to be the more satisfactory of the two.

5.7 Stormwater Drainage

A detailed discussion of the problems of stormwater drainage of the area is not within the province of this report, but some comments on the particular problems raised by the levees are necessary. In such a flat area, special attention would have to be paid to stormwater drainage in any case, but the existence of levees will greatly affect the design of the drainage system.

Firstly, any stormwater drains passing through the levees will have to be fitted with flap gates on the outlets to prevent entry of floodwaters from the stream to the leveed area. The initial cost, necessity for maintenance, and possibility of faulty operation of these flap gates should be borne in mind in determining the number of such drainage outlets put through the levees.

Also, it must be remembered that drainage of stormwater in drains passing through the levees will not be possible for the period that water level in the creek is above the flap gates. This need not be a very serious limitation in this case as, for any given storm, the drainage from the leveed areas could be entirely discharged before the water level in the creek reached the flap gates. This would require only the construction of an efficient drainage system, and for the drain outlets to be kept as high as possible. Under these conditions most or all of the stormwater could be discharged before the main body of flow from the catchment area reached the site. Furthermore, the rise and fall of floods in this stream is quick and flashy, so that the flap gates will not be closed for more than a few hours at a time. Nevertheless, it is, of course, only prudent to make provision for some temporary storage of stormwater behind the levees, to take care of runoff from any rainfall occurring while the creek level is high.

A further corollary of the impracticability of draining through the levees at times of high flow in the creek, is that as much as possible of the area should be drained directly to points above the levees. Most of the area on the southern side of Cabramatta Creek could be so drained to outlets in Maxwell's Creek, and it is considered that careful consideration should be given to the layout of the stormwater drainage system with a view to obtaining this free outfall for as large a proportion of the total area as possible.

6. CONCLUSION

6.1 Review

Construction of houses on land naturally subject to flooding inevitably involves a certain element of risk. However, the investigations described in this report have led to a reliable understanding of the extent and frequency of flooding in the area, and to an outline plan of flood mitigation works, which will give protection economically to large areas of land with a very small risk of failure.

The flood estimation work described in Section 2 of the report utilizes the most reliable method of flood estimation available in the absence of detailed hydrologic records for the catchment. Availability of the detailed records from the nearby and very similar experimental catchments of the University of New South Wales, facilitated the study, and led to a high degree of confidence in the design flood estimates. Also, the estimates of past flood discharges, made as described in Section 3 above, would not have been possible without the experimental catchment records, and these discharge estimates are considered to have a reasonably high degree of accuracy. A considerable amount of local information on floods was obtained, and this in some cases complemented, and in other cases confirmed the results of the calculations. In no case was reliable local information inconsistent with the results of the theoretical analysis. Drawings 1, 2 and 3 illustrate the finds of the first major part of the investigation, dealing with extent and frequency of flooding.

It was soon apparent from Drawings 1, 2 and 3 that, if houses were to be built at all on the southern side of Cabramatta Creek, and in some areas on the northern side, certain engineering works would be required. This is so because, even though much of the flooded area is inundated for only short periods and to shallow depths, it nevertheless serves a very necessary purpose in the discharge of floodwaters, and to block this flow with buildings and fences would considerably increase the depth of flooding. Of the several possible methods of flood mitigation, the most desirable in this case is levee construction combined with a limited amount of clearing of the existing creek channel. A necessary feature of levee location, however, is that the banks should be a considerable distance apart so as to leave a floodway adequate to discharge very rare floods. While this requirement reduces the area of land outside the levees, it is necessary for the safety of development behind the levees. The recommended system of levees and their crest elevations are illustrated on Drawings 4 and 5, which also show the estimated extent of flooding for recurrence intervals of 5, 40 and 1000 years with the levees in position, except in those areas where the levees will not affect flood levels, for which the information on Drawings 1, 2 and 3 is applicable. Use of the land within the floodway should be such as not to retard the flow of floodwaters. Any type of development can be carried on outside the floodway, but particular attention here must be paid to stormwater drainage.

6.2 Recommendations

For convenient reference, the various recommendations made in this report are summarised below:-

A. Levees

(i) Earth levees should be constructed in the positions and to the elevations shown on Drawings 4 and 5 accompanying this report, with the condition that the position and elevation of Point A on Drawing 4 is provisional, and dependent on natural surface levels in this area. If natural surface does not rise to RL.81.0 in this area, the levee AB should be carried out to the divide between the Cabramatta Creek and Maxwell's Creek drainage areas, at the level of this divide.

(ii) At the point where levee AB crosses Hoxton Park Road, the road should be reconstructed to rise gradually to the levee crest elevation, and fall down the other side.

This reconstructed road embankment should not obstruct the floodway any more than is necessary.

B. Channel and Floodway Clearing

(iii) All dead trees and logs at present lying in the channels of Cabramatta Creek and Maxwell's Creek in the area of the proposed development should be removed and burnt or otherwise disposed of.

(iv) The two old concrete weirs in the upper part of Cabramatta Creek at co-ordinates 88,600 yds. E, 9,280 yds. N and 88,650 yds. E, 9,240 yds. N, should be removed.

(v) The two farm dams on the northern side of Cabramatta Creek near the western end of the area (opposite Points B and C on Drawing 4) should be removed.

C. Possible Cutoffs

(vi) Following any major floods which occur after construction of the levees, consideration should be given to the desirability or otherwise of excavating cutoffs in the main channel of Cabramatta Creek near Points E, G and I on Drawing 4, to protect the levees against possible undercutting of the banks.

D. Limits of Development

(vii) Upstream of Point X (the lower limit of the levees) no substantial buildings, and no fences should be constructed in the area within the levees or the area covered by the 1000 year flood, except that, if considered desirable, fences of a post and rail type (but not post and wire, chainwise, paling, nor picket fences) may be allowed to encroach not more than 100 feet over the limit of flooding due to the 1000 year flood.

(viii) If any bridge approaches are constructed within the floodway, proper provision should be made for the backwater that would be caused by the bridge, if necessary increasing the height of the levees on the upstream side of the approaches.

(ix) Downstream of the levee system, development should have regard to the extent, depth and frequency of flooding as shown on Drawings 1 and 2, and to the fact that undue obstruction of flood plain flow has the undesirable effect of raising flood levels. (Thus development on the flat area on the left bank just upstream of Mulgoa Road should be such as will not unduly obstruct flow during the infrequent flooding to which this area is subject).

E. Stormwater Drainage

(x) Special attention should be given to the stormwater drainage design for the area.

(xi) As much as possible of the area should be drained to outlets above the levee crest elevation, and any drains passing through the levees should be fitted with flap gates.

(xii) Some provision should be made for the temporary storage of stormwater in areas draining to outlets through the levee banks.

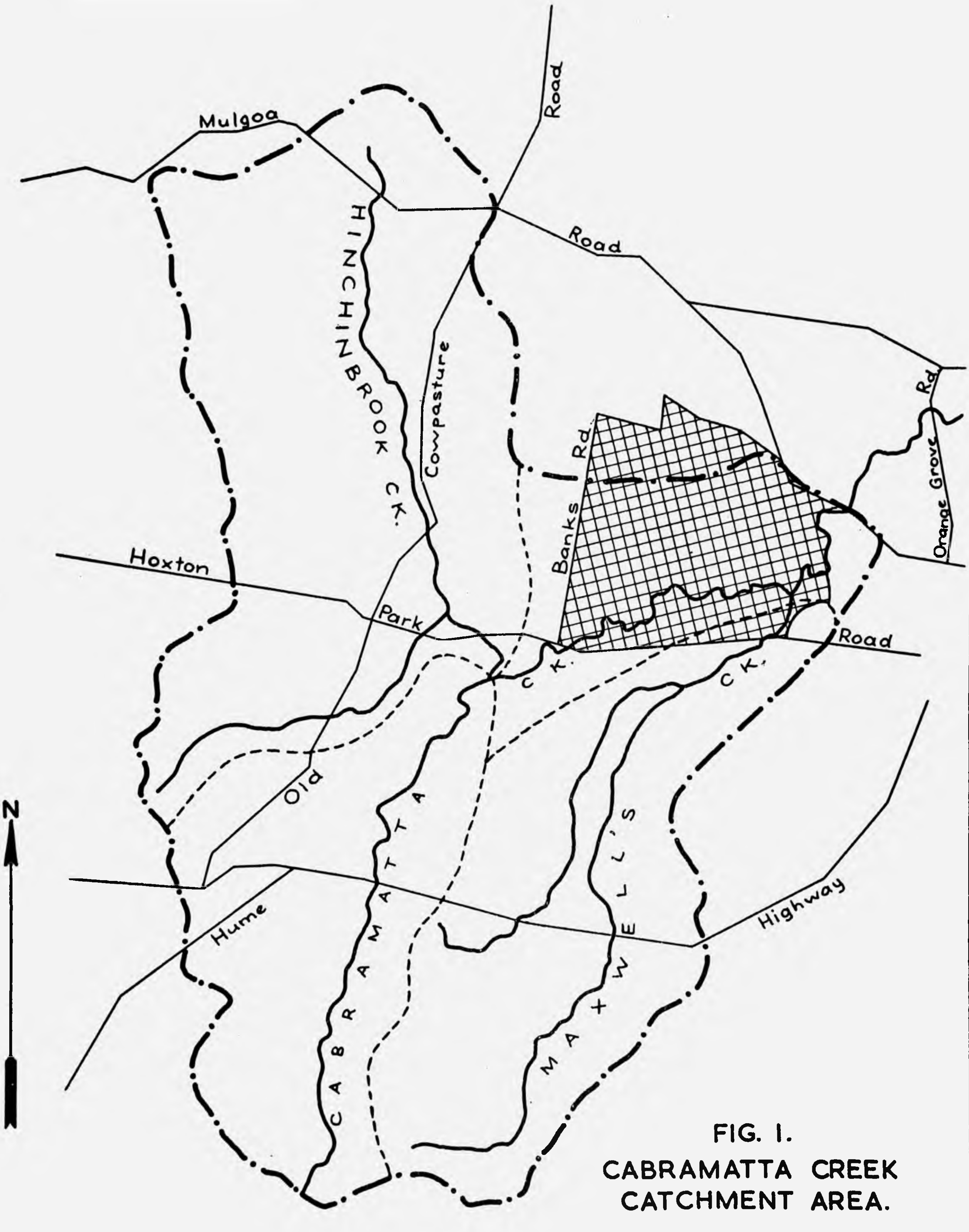

FIG. 1.
CABRAMATTA CREEK
CATCHMENT AREA.

SCALE : 1 MILE = 1 INCH

0 1 2 3