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Volume I

Flood Mitigation Measures for the City of Launceston

by

C. H. Munro

SEPTEMBER, 1959

REPORT

ON

METHODS OF FLOOD MITIGATION

FOR

THE CITY OF LAUNCESTON, TASMANIA

submitted to

THE LAUNCESTON FLOOD PROTECTION AUTHORITY

by

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SYNOPSIS

This report embodies a detailed account of the comprehensive topographic, hydrologic, hydraulic (model) and economic studies carried out over the period 1957-59 to investigate ways and means of mitigating flood damage in Launceston, Tasmania. Data were collected and hydrologic analyses were carried out, with the aid of a digital computer, to derive unitgraphs and reproduce the hydrographs of the disastrous 1929 flood and the estimated "maximum probable flood" on the North and South Esk rivers. The hydraulic efficiencies of various methods of flood mitigation were tested by use of a scale model and benefit-cost analyses were completed for each proposal. The investigation led to a recommendation for a diversion levee and surround levees to protect the city from an estimated "probable maximum flood" of 250,000 cusecs. The wisdom of expenditure on thorough preliminary research for such engineering projects was forcibly demonstrated.

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The report is divided into the following parts:-

At the commencement of each part a detailed "Table of Contents" will be found.

GENERAL NOTES

Abbreviations

Authority	-	The Launceston Flood Protection Authority
H.E.C.	-	Hydro-Electric Commission of Tasmania
L.M.B.	-	Launceston Marine Board
L.C.C.	-	Launceston City Council
P.W.D.	-	Tasmanian Public Works Department
S.L.W.	-	Standard low water

References

Throughout the text, references are given serial numbers in each Part and are listed in footnotes at the first mention only. The complete bibliography is summarised at the end of the report.

Survey Datum for Levels

The datum from which all levels are quoted is "standard low water" (S.L.W.) which is a point 17.18 ft. below a mark on the steps of the Customs House on the Esplanade of Launceston. This datum is 8.60 ft. below State Datum.

Money Values

Where a sum of money is quoted for conditions prior to 1959, the approximate equivalent 1959 money value is given in brackets. The unit of money used is A£1.

Plans, Graphs and Tables

All detailed tabulations, graphs and plans, referred to in the text, will be found in Vol. II (Appendices) and are numbered serially. Summarised tabulations embodied in the text of the report (Vol. I) are numbered serially in each part, with a prefix corresponding to the part of the report in which the table appears.

Plates

Photographs of important features of the investigation will be found in Vol. III (Plates). A list of these plates is given at the end of Vol. I, but no reference is made to them in the text of the report.

PART A

THE GENERAL BACKGROUND OF THE INVESTIGATION

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PART A - THE GENERAL BACKGROUND OF THE INVESTIGATION.

A1 INTRODUCTION

The City of Launceston, lat. 45° 25' S; long. 147°08', is the second city of the State of Tasmania, Australia, and is the commercial, industrial and transport centre for the north-eastern sector of the island. The city is located at the confluence of three rivers, the North and South Esk Rivers meeting in the heart of Launceston and combining their flows to form the Tamar River. This river is an estuary of Bass Strait, navigable for the 40 miles from its mouth to Launceston by ships 300 ft. in length of 19 ft. draught. In its upper reaches, however, it is encumbered by shoals, and just downstream of Launceston Stephenson's Bend and Ti-tree bend present navigational hazards. Hence, although Launceston is an important port for trade with the Australian states, the larger overseas vessels berth at wharves at the mouth of the Tamar.

The City has been subjected to four major destructive floods in the period 1828 to 1959, and has also experienced in that time a number of alarming though less damaging minor floods.

From a study of the topography, it is obvious that possible flood mitigation measures are:-

(i) Floodways (ii) River channel improvement (iii) River straightening (iv) Levee surrounds (v) River diversion (vi) A combination of one or more of these methods.

Following on the disastrous flood of 6th April, 1929, considerable investigation was carried out prior to World War II, based on mathematical computations and some elementary analytical hydrology, culminating in a recommended solution estimated in 1955 to cost £1,500,000 (£1,650,000.).

The study described in this report is interesting in that it is an example of how research in hydraulics and hydrology over the past twenty years has so improved engineering techniques that a more exact and rational analysis can be made, leading to a proposed solution estimated to cost £644,700.

Other noteworthy features of this investigation are:-

(a) Although no disastrous flood has occurred in Launceston since 1929, the Government of Tasmania initiated this investigation in 1955, thus setting an example of wise foresight to other governments responsible for communities in flood plains.

(b) A special Authority was set up for the sole and limited purpose of carrying out a complete enquiry into methods of flood mitigation. This is a novel step in public administration in Australia and may be worthy of consideration by other governments for similar problems.

(c) A period of two years of hydrologic data collection was specified at the outset as an essential minimum for a rational study of the problem. This contrasts with the tendency, all too common in Australia, to rush through a solution in the minimum time on an inadequate basis.

(d) A digital computer was used for the first time in Australia (and perhaps in the world) for unitgraph derivation for a flood mitigation study.

(e) A 30-year old movie film of the model verification prototype flood was available for hydraulic model studies, and this is probably unprecedented.

(f) Although the area in question was located in Tasmania, the Authority decided to carry out the model studies at the Water Research Laboratory of the University of New South Wales, thus setting its face against the commonly adopted, but inefficient, method of "backyard" model studies on an improvised basis near the site of the project.

(g) The wisdom and economy of the use of a hydraulic model in such problems instead of relying on theoretical hydraulic computation was demonstrated with unusual force. Not only was the model able to evaluate quantitatively the merits of methods (i) to (vi) above, but it showed up another solution - a diversion or training levee - which would not have been thought of by even the keenest observer from a study of the prototype conditions in times of normal flow or minor floods.

(h) It is an example of a more exhaustive study than is customary in Australia of all four essential phases of a complete flood mitigation analysis, viz: (i) Topographic (ii) Hydrologic (iii) Hydraulic (model) (iv) Economic. It is also the first flood mitigation model study completed in Australia.

(i) The degree of protection which is economically feasible is exceptionally high.

(j) This case provides a good demonstration of the often unrecognised fact that expenditure on research and investigation is a wise investment, the difference in estimated cost between the old and the new proposals being £A850,000.

The total cost of this study was £20,000, and resulted in a recommendation that a training levee and surround levees be built for an estimated cost of £654,700 to protect the city of Launceston from the "estimated probable maximum" flood.

The supplementary tests referred to in Clause D8 indicated improvements which reduced the estimated cost to £588,600.

A2 INITIATION OF THE INVESTIGATION

Ways and means of protecting Launceston from floods have been under consideration since the major flood of April, 1929, and considerable investigation was carried out from that date until the early forties, when the serious war situation pushed the matter into the background.

In 1947 the question was revived and conferences were held between interested government departments, culminating in a recommendation to the Minister for Lands and Works by the Director of Public Works (R.Sharp) that a special Authority be set up to investigate the whole matter.

Consequently, the Launceston Flood Protection Act No.43 of 1955 was enacted by the Government of Tasmania on 29th November 1955. This provided for the creation of "The Launceston Flood Protection Authority", consisting of:-

(a) City Engineer of Launceston (L.H. Bird M.I.C.E., M.I. Mech. E., M.I.E., Aust., M.N.Z.I.E.) - Chairman.

(b) Director of Public Works (R.C. Sharp B.E., A.M.I.C.E., A.M.I.E. Aust., A.M.T.P.I.)

(c) Chief Engineer, Marine Board of Launceston, (J.K.Edwards A.M.I.E. Aust.)

(d) Engineer for Civil Investigations, Hydro-Electric Commission, Tasmania (P.C. Tapping, B.Sc., A.M.I.E. Aust.)

(e) The Principal Executive Officer (the author).

The Authority was charged with the following task:-

"(a) Investigate flooding at or near the confluence of the North Esk and South Esk Rivers and measures to mitigate it.

(b) Prepare a scheme to provide protection from flooding for low lying lands in the vicinity in such detail and with such plans as it thinks proper."

The Authority commenced its work in January, 1956 and its first action was to fill the position of Principal Executive Officer. After some enquiry a recommendation was made that the author be appointed. A dissolution of parliament resulted in an unavoidable delay in finalizing the matter, but eventually in January 1957 the author commenced duty in a part-time capacity, and on 21st January 1957, submitted a proposed programme for the investigation, envisaging the carrying out of topographic and hydrologic investigations in Tasmania, and hydraulic (model) studies at the Water Research Laboratory of The University of New South Wales at Manly Vale, near Sydney, followed by stage-damage surveys in Launceston and an economic analysis and writing of the final report in Sydney. It was proposed that the author should spend all University vacations in Launceston, together with occasional visits during teaching terms as required. Mr.D.N.Foster B.E., was appointed as full-time Investigating Engineer, located in Launceston for the topographic and hydrologic phases, and in Sydney for the model studies. Much of the subject matter of Parts C and D of this report is based on work carried out by Mr.Foster towards fulfilment of requirements in support of his candidature for the degree of Master of Engineering of The University of New South Wales.

The author stressed from the beginning that at least two years were required for hydrologic data collection, survey work, and model construction and testing.

Full details of the investigation are given in succeeding sections of the report and supporting appendices. The work culminated in a meeting of the Authority at Manly Vale on 4th, 5th and 6th April, 1959, when the various methods of flood mitigation were demonstrated on the model and the basic principles of the author's recommendations for flood mitigation were adopted. The final plan was approved at a meeting of the Authority in Launceston on 17th September 1959.

A3 GENERAL DESCRIPTION OF LAUNCESTON AND THE ESK VALLEYS

Fig. No.1 shows the general layout of the city. The main commercial centres are south of the N.Esk, but the suburbs of Inveresk and Invermay, on the flat areas on the north bank of the N.Esk, are highly developed industrially and residentially. The two halves of the city are connected by Tamar St. and Charles St. bridges over the N.Esk, the former leading to Invermay Rd., which is an important road traffic artery serving Georgetown and the aluminium works and overseas port of Bell Bay, near the mouth of the Tamar.

At the head of Home Reach is situated Royal Park. Prior to 1900 the major portion of this park consisted of swamp land, but controlled tipping of household refuse reclaimed the area in the early years of this century. In the mid-nineteenth century the Invermay and Inveresk area was known as "The Swamp" and apparently consisted of a road along the route of the present Invermay Rd. running through a series of shallow reedy ponds. At some time prior to 1852 reclamation was carried out and levees were built along the banks of the N.Esk and Home Reach, and settlement commenced, the area being known as "New Town." After the flood of 10th August 1852 the Launceston "Examiner" stated: "It is conjectured by some that the embankments in the swamps have had the effect of preventing the escape of the water." Following the 1863 flood the same paper commented "It must be quite evident now that the Government ought not to have sold the swamp for building purposes. The loss just sustained by the residents of Newtown we imagine will check the extension of Launceston in that direction for some time to come."

However, the history of the development of Launceston provides yet another example of mankind's determination to use the flood plains of rivers, and, when possible, to render the river impotent by flood protection measures, in preference to settling on higher ground.

Launceston is advantageously situated geographically, in that it is reasonably close to Melbourne, the capital of Victoria, and strategically located for the distribution of imports and its own manufactured goods throughout Tasmania. On the flat areas of Inveresk and Invermay are situated the headquarters of the Tasmanian Railway system with major workshops and rail connection to the adjacent modern wharf known as King's Wharf. In the town planning of the city the district has now been zoned as an industrial area, although at present it is a mixture of residences, commerce and industry.

These suburbs, well served by interstate shipping and intrastate railway and road systems, and having available ample manpower from a prosperous city with modern amenities, are obviously suitable for future development of factories, warehouses, and commercial activities.

Unfortunately, the general ground level is only 12 ft. above standard low water, and the surrounding levees do not provide protection against river heights exceeding 17 ft. Normal high tide is about 12 ft. and high tide levels have been known to reach a figure of 15.66 ft. Since 1828 the area has been inundated six times by floods attaining levels between 17 and 21 ft., and the rivers have often lapped the top of the levees in an alarming manner. During major floods, the city proper on the southern bank of the N.Esk has also suffered material damage, as it is not protected by levees and for some distance from the N.Esk bank the ground level is less than 20 ft. above S.L.W.

A study of the catchment area indicates the mechanism of such periodical flooding. It is roughly semi-circular, ringed by mountain ranges with Launceston at the centre, as shown on Fig. No. 4.

The North Esk river, which rises on the northern slopes of Ben Lomond plateau 30 miles E.S.E. of Launceston as the crow flies, has a catchment area of 412 sq. miles. This river flows through Launceston as a meandering stream and floods overtop the banks and cause widespread inundation of the low lying adjacent areas. These floods cause little damage, however, to the developed areas of Launceston, which are protected by levees high enough to confine all floods in this river within the banks.

The main flood producing river is the South Esk, which rises on the North Eastern side of Ben Lomond, and drains 3355 sq. miles of catchment area before discharging its waters through a spectacular, chasm-like gorge into the Tamar River right in the heart of Launceston. In this gorge the first hydro-electric power station ever built in Australia was constructed in 1896 at Duck Reach.

On the rare occasions when semi-tropical rains come from the east and south-east the stage is set for a major flood. The moisture laden air flows in directly from the Tasman Sea and is lifted by the east coastal ranges, its moisture being precipitated out as heavy rain over the headwaters of the South Esk catchments. The more common westerly weather, which is responsible for the heavy rainfall of the West Coast of Tasmania, does not appear likely to present the same flood threat, as a great deal of the moisture in the air is deposited as rainfall during the passage across the high West Coast Mountains, before the air mass reaches the South Esk catchment. Snowmelt is not a material factor in producing floods.

A4 GENERAL DESCRIPTION OF THE FLOOD OF 4TH-7TH APRIL 1929

The isobaric pattern on the East Coast of Australia before and during the flood is shown in Fig. 11, and the isohyetals on the catchment in Fig. 15. The estimated hydrographs of the S.Esk and N.Esk rivers are shown in Figs. 22 and 24.

During Friday, 5th April, it was clear to the citizens of the city that there was a danger of overtopping of levees. Early in the afternoon the Mayor called a conference of representatives of public authorities and the press. The "Examiner" printed a special warning leaflet and boy scouts and others distributed them. The signal for commencement of emergency measures was to be the tolling of the bell of a civic building. Relief committees were organised and the large Albert Hall prepared for evacuees. All the planning was based on the assumption that the danger point would be at 9.0 p.m. - high tide. Evacuation teams were held in readiness for this zero hour. By late afternoon gangs of men were strengthening the Inveresk embankments with sand bags. At 9.0 p.m. the water level was still below the top and many assumed the danger was over. Due to some defect in switching arrangements, supply from Waddamana Hydro-Power station was not available, and the city was dependent on Duck Reach Power station for light and power. At 11.15 p.m. this station was destroyed by the flood, and the city plunged into darkness. Rain was falling. The gas supply failed. At 1.30 a.m. on Saturday the flood commenced to overtop the levees and the alarm bell was rung. Removal of evacuees from Inveresk and Invermay by motor lorries, horse-drawn drays and boats commenced in the darkness, and continued throughout the following day. The high knoll between Invermay and Mowbray flats was above water, but at midday it was decided as a precautionary measure, to evacuate this area. Much damage was done to small boats and some to shipping in Home Reach.

Throughout the Saturday the flood level was gradually rising, reaching its peak in the early hours of Sunday. By Monday it was falling appreciably and rapidly. Channels were cut to allow the water to escape from the Inveresk area.

A well organised system of relief and rationing operated. The State Government set up a relief camp in the Showground, which remained in operation for some months after the flood. The operations of industries outside the flooded area were heavily affected due to absentees in the first week after the flood, and it was some weeks before the industrial and commercial life of the city returned completely to normal.

A Tasmanian Flood Relief Committee was set up and raised £116,000 (£348,000) by subscriptions from private individuals and firms. Of this sum approximately £80,000 (£240,000) was disbursed to distressed persons in Launceston. This is no indication of the direct damage done. The committee adopted the policy of "relief but not compensation." This meant that if any person or firm was deemed to be in a financial position to "carry their own loss" no relief could be obtained, and that claims would only be considered for essential items necessary to enable the distressed person to get going again as an income earner; i.e. claims for pianos, carpets, radio sets etc. were not recognised.

In a report by Judd held by the Launceston Museum it is stated that 1000 homes were inundated, and "at the middle of July 100 houses were still awaiting health certificates and 20 had been condemned as unfit for human habitation."

In all 4,000 people evacuated their residences. Invermay public school was taken over for refugees, and was not re-opened for school work until 20th April. Albert Hall was in use as a refugee centre until 14th April. A period of 3 weeks elapsed before the railway workshops approached normal operations.

The City Council controlled a number of relief depots until 27th April, when it handed over to the Tasmanian Government the responsibility for the 300 remaining refugees, who were housed in the showground and at various other government depots.

A5 REVIEW OF PREVIOUS INVESTIGATIONS

Immediately after the 1929 flood the Public Works Department carried out surveys to establish trash lines in rural areas, while a surveyor (J.Maddocks), at the direction of the City Engineer (A.C.P. Wood) fixed the maximum flood levels in the vicinity of Launceston as shown in Fig. 33. The Government of Tasmania appointed the Director of Public Works (G.D.Balsille) and a consulting engineer (W.B.McCabe) to report on flood warning and flood protection for Launceston.

On 2nd May 1930 these officers submitted their report, the salient features of which are:-

(a) Past Major Floods

Major floods had occurred in the past in all months of the year except January and February, and the interval between floods was becoming less.

(b) Possibility of Flood Exceeding 1929 Flood

In the 1929 flood one third of the catchment contributed two thirds of the total flood water. The possibility of wider coverage of heavy rainfall in future major storms and the denudation of timber from the catchment means it is certain that still greater floods may be expected in the future.

(c) Rate of Travel of Crest of 1929 Flood.

Place	Mileage	Time of Peak
Mathinna	0	
	10	10.30 p.m. 4.4.29
Fingal	16	11.15 p.m. 4.4.29
Avoca	36	5.00 a.m. 5.4.29
"Brambletye"		6.30 a.m. 5.4.29
"Vaucluse"	50	10.15 a.m. 5.4.29
Symmons Plains		5.15 p.m. 5.4.29
Evandale	75	8.30 p.m. 5.4.29
"Rhodes"		11.00 p.m. 5.4.29
Longford	80	2.20 a.m. 6.4.29
Hadspen	101	7.40 a.m. 6.4.29

(d) Flood Warning

The experience of the 1929 flood showed that road, rail, telegraph and telephonic communication cannot be relied upon and telephonic radio transmitting stations should be installed at Fingal, Avoca and Ross for flood warning purposes.

(e) Flood Protection of Launceston

(i) General

Levees and deepening and widening of the Tamar are the only possible measures.

(ii) Levees

Peak discharge of 1929 flood was estimated by slope area methods as 200,000 cusecs plus or minus 10 per cent. For 220,000 cusecs levees would require to be 34'0" above S.L.W. and for 250,000 cusecs the level would be 40'0". The 1929 flood occurred at neap tide, and it was assumed that if such a flood occurred at spring tide the height would have to be increased by 2'0".

Allowing also for 2'0" of free board and recognizing that the average ground level of Inveresk and Invermay is 12'0" above S.L.W. it was concluded that if levees only were relied upon they would have to be 32'0" high for protection against 250,000 cusecs. As experience had shown that the maximum permissible bearing pressure over most of the area was 0.5 tons per sq. ft., it was considered that the maximum permissible height of levee was 10'0", and this method of protection would have to be combined with (iii) below.

(iii) Deepening and Widening the River Tamar

To handle a discharge of 250,000 cusecs with levees 10'0" high, the channel of the Tamar should be dredged to a depth of 36'0" below S.L.W. for a width of 240 ft. with side slopes of 5 to 1 at an estimated cost of £400,000 (£1,200,000).

(iv) Diversion of N.Esk

It was considered that if these proposals were carried out it would be necessary to shut off the N.Esk River where it debouches into the Tamar to prevent inundation of areas in the city or southern side of the N.Esk, and to divert this river through Mowbray Flats to Stephenson's Bend in the Tamar, involving a diversion channel of 50,000 cusecs capacity, property resumptions, and new major road and railway bridges.

(v) Total Cost

No detailed estimates of cost were made, but it was considered that the total cost would exceed £600,000 (£1,800,000).

(vi) Method of Finance

Interest on £600,000	at 6 per cent =	£36,000 p.a.
Sinking fund	at 1 " " =	6,000 p.a.
Maintenance Dredging		20,000 p.a.
	Total	£62,000 p.a. (£186,000)

The assessed annual value of the city was given as £370,000 so that the rating burden would be approximately three farthings in the £. The capital values of the floodable areas was given as £1,000,000 (£3,000,000).

As will be seen from the studies conducted by the author and reported herein, this assessment of the situation was considerably in error, as could only be expected in the absence of a complete hydrologic and hydraulic (model) study. This is no reflection on the two very capable and experienced engineers concerned, because in 1929 the science of engineering hydrology was in its infancy, and the use of hydraulic models for such problems, while recognized, was the exception rather than the rule.

The magnitude of the proposals envisaged in this report apparently led to the conclusion that protection from major floods was out of the question, on economic grounds. The Progress Association of Invermay submitted in 1930 detailed proposals for raising by 3 ft. existing levees around that area to the height attained by the 1929 flood i.e. 20 ft. above S.L.W.

Balsille and McCabe reported on this proposal, summing up as follows:-

(i) Such measures would protect the Inveresk-Invermay areas against lesser floods than that of 1929, but floods of the same order as the 1929 flood would probably overtop such levees and higher floods would certainly do so, because in 1929 "a huge quantity of water was stored in the flats of Inveresk and a great quantity was also flowing through Inveresk across Invermay Rd. to the Tamar" at Stephenson's Bend, and "this water would be confined to Home Reach resulting in a higher flood level in this channel". (The model studies described in this report revealed that the storage in and discharge over Inveresk-Invermay in 1929 were negligible).

(ii) Such raising of the levees would increase flood heights in the city or southern bank of the N.Esk. (Apparently no consideration was given to levees on the city side).

On the 27th April, 1930, an informative article was published in the Hobart "Mercury" by Russell Kidd, an amateur but capable hydrologic engineer, in which he pointed out that in 1888 he estimated the peak discharges in the 1852 flood as 250,000 cusecs in the S.Esk and 20,000 cusecs in the N.Esk, and predicted that the projected power station at Duck Reach in the gorge of the S.Esk would be washed away, as indeed happened in 1929. He gave a hydrograph of the 1929 flood with a peak discharge of 250,000 cusecs, together with the results of some praiseworthy observation of velocities in Home Reach in 1929, a review of all historic floods, and notes on tidal conditions in the 1929 flood. He commented:-

"River flats as a rule are extra fertile and it so happens also that a most admirable site for factories alongside a port exists at Launceston, which unfortunately is liable to inundation from extraordinary floods, and is even now the home of many thousands of people. That small space contains one fortieth of the population of the island, and has further great possibilities. To ask that Inveresk and Invermay remain unbuilt upon, because of flood risk, is to be guilty of a want of enterprise that cannot be laid as a charge against the Dutch and other peoples. We must not strike our flag when others nail theirs to the mast and go in fighting."

In 1931 the City Engineer of Launceston (W.E.Potts), took up the investigation and on 6th October 1931 made a report to the Council, in which he repeated Balsille's and McCabe's estimates of peak S.Esk flood discharges as:-

September	1828 - 200,000	cusecs
July and August	1852 - 250,000	"
December	1863 - 200,000	"
June and November	1889 - 120,000	"
March	1893 - 150,000	"
March	1911 - 90,000	"
May	1923 - 82,000	"
October	1926 - 112,000	"
April	1929 - 250,000	"
June	1931 - 112,000	"

He stated that a discharge of 150,000 cusecs would overtop existing embankments, and calculated that if a flood of 250,000 cusecs is confined to the Tamar, the flood waters would reach 26'3" at a point 1000 ft. below Cataract Bridge, and that raising existing levees to 21 ft. would mean inundation once per 25 years.

He discussed six proposals:-

(i) Raise the levees around Inveresk and Invermay to 21 ft. and build levee of same height on city side of N.Esk, and accept inundation once per 25 years.

(ii) As for (i) with spillway relief in over Georgetown Rd. in over Georgetown Rd. in Mowbray flats area. Spillway relief was negligible.

(iii) Build levees as in (i) to 21 ft. and also a diversion channel from N.Esk across Mowbray flats to Tamar to take 30,000 cusecs from N.Esk and 25,000 from the S.Esk, again accepting inundation nearly as often as once per 25 years.

(iv) As for (iii), but with levees to 27 ft. catering for a flood 25 per cent in excess of the 1929 flood with 53,000 cusecs flowing upstream in the N.Esk and 73,000 in the N.Esk diversion cut. Potts commented that this scheme would be very expensive and need many traffic openings.

(v) Raise levees along N.Esk bank to 21 ft. but build new levees at this level parallel to the Tamar bank from Charles St. Bridge to the promontory near River St., but standing back a considerable distance from the Tamar bank to give the river a flood plain, instead of following the existing levees along the edge of the bank.

(vi) "Complete protection", with levees 30 ft. above S.L.W., diverting and sealing off the N.Esk, generally along the lines of Balsille and McCabe's proposal but without deepening and widening the Tamar channel.

Potts preferred proposal (vi) if it could be financed, with (v) as a second best.

On 10th December 1934 he submitted an addendum, apparently at the request of the Launceston City Council, stressing that proposal (v) was the best "partial protection" scheme and that it lent itself to later development of (vi) - "complete" protection. He stressed the need to collect hydrologic data.

In 1935 the "Launceston Corporation (Flood Protection) Enabling Act" was submitted to the Tasmanian Parliament authorizing the Council to:-

(i) Construct the "partial" flood protection works forthwith.

(ii) Construct the "complete" works (at some time in the future) only with the approval of the Governor.

(iii) Borrow for this purpose a sum not exceeding £25,000.

(iv) Collect hydrographic records to assist in planning the works.

It also indemnified the Council against any claims for damages resulting from the carrying out of the works. (Apparently this arose from a fear that the "partial" works protecting the Inveresk-Invermay areas only might raise the flood levels on the city side of the N.Esk). The estimate for the "partial" scheme was £38,000 (£114,000) and the complete scheme £190,000 (£570,000) plus the cost of land resumptions. Apparently some government subsidy was contemplated. The Bill was defeated in the Upper House, due mainly to doubts as to the wisdom of "partial" protection and objections to the indemnity clause.

It is interesting to note that in the subsequent debate in the City Council it was pointed out that the scheme for the diversion of the N.Esk as a ship canal was first mooted in 1834, and in 1851, Governor Denison was on the point of commencing the work when the cessation of transportation of convicts and scarcity of labour called a halt. At this meeting a sub-committee was appointed to go thoroughly into the matter of the diversion of the N.Esk, and the Council on 15th September, 1936 approved of a survey and hydrologic data collection programme, with the "complete" scheme as the objective.

In the course of a deputation to the Council on 21st September, 1936, Mr.Tasman Shields stated:-

"Flood prevention, of course, is not only of importance to the areas affected in 1929, but to the areas adjacent. Rate-payers living in the affected areas, who have paid rates on their properties over many years, have a right to protection. But apart from that there are also the interests of the Corporation itself. In the flood areas of Inveresk, Invermay, and Mowbray there are approximately 2540 assessments, and for 1931-1932 the annual value of these properties was £84,689 (£254,067) and in 1935-36 £79,366 (£238,098). I would say that those assessments are artificial, and are being sustained by financial institutions and mortgagees in the expectation that flood relief will ultimately restore the properties to the capital value existing prior to 1929, a value which in the past seven years has decidedly depreciated. It has been almost impossible during the period mentioned to either dispose of properties in the flood area at even a marked depreciation in value, or to raise money on them as security. This in itself represents a loss to the people who own these properties that cannot be calculated. If recently, when there was reason for apprehension there had been an experience like that of 1929, the properties in the areas would have been valueless, and in any event, unless something of a comprehensive character is done in the near future, the ratable values of these properties must seriously diminish, with a resultant loss to the Corporation in rates and payments for services."

Mr. G.B. Rolph, President of the Chamber of Commerce, said:-

"Whatever flood injured part of Launceston injured the whole of it. No part of Launceston could suffer without Launceston as a city suffering commercially and industrially.

Get down to bedrock and only two effective courses are open. One is to abandon the areas liable to flooding and arrange for a wholesale transfer to higher ground. But development, both public and private, has gone too far to make the scrapping of so much property a practical proposition. The other plan is to give complete and effective protection."

In December 1936 H.H. Dare, who had retired from the post of Chief Commissioner for Irrigation in N.S.W., was appointed as Consultant to work in collaboration with Potts. A Licensed Surveyor (J.Wilks) was engaged to make a topographic survey of the affected areas. During 1938 one Gurley tide gauge was installed at the point of diversion of the N.Esk River, one at Tamar St. Bridge on the same river, one at Northern end of Kings' Wharf in Home Reach, and one at the lower end of Stephenson's Bend. In addition "Bristol" pressure streamheight recorders were installed at Duck Reach in the S.Esk gorge and at Corra Lynn on the N.Esk. These installations were intended to record flood discharges of these two rivers, and to obtain "n" values of roughness in the gorge.

The surprising fact that as late as 1938 no attempt had been made on any systematic basis to measure the discharges of these two important Tasmanian Rivers, is due to the general lack in Australian legislation (at that time) of any provision to make some authority responsible for the assessment of the water resources and flood potentialities of the country as a whole. Individual water authorities took such river gaugings as were required for specific proposals as they arose. In Tasmania the only authority which took any action at all was the Hydro-Electric Commission, which was interested only in the elevated areas. Although the Launceston Council apparently purchased a current meter, the stations at Corra Lynn seem to have been "rated" by the approximate method of timing floating drums in their passage down the rivers to obtain the relation between river height in fact and river discharge in cusecs, together with occasional surface velocity measurements by current meter at peak of floods. A record of peak flood heights was maintained on a staff gauge at Duck Reach Power Station from 1900 to 1956, but the station was not properly rated until the Hydro-Electric Commission was forced in 1945 to take a hand by reason of its desire to construct a Hydro-Power dam (Trevallyn Dam) a few miles above Duck Reach. Had this fortunate circumstance not occurred, the author would have been obliged to delay his study until a sufficient number of high floods in the S.Esk gorge had been measured by current meter methods.

In recent years Australian legislators have taken some steps in the direction of giving responsibility and power to authorities for the streamgauging of the complete river system of the various states. The example of the difficulties confronting Potts and the author in the Launceston problem illustrates the need for this policy to be pursued and, more importantly, implemented in a determined manner in the future.

One weakness in the study described herein is the lack of any data regarding water levels during major floods in Boat Channel below Stephenson's Bend. It is hoped that the Marine Board of Launceston will take steps to maintain an adequate system of tide gauges in the Tamar, as recommended by T.A. Lang (A1).

Ref. (A1) T.A. Lang - Internal Report to Launceston Marine Board July 1947, describing work carried out in 1939-40.

On 24th April, 1937 Dare submitted an interim report to the Council, the main points being:-

(i) The 1852 flood was apparently higher than the 1929 flood, and the 1863 flood not much less than 1852.

(ii) Protection should be provided against the 1929 flood discharge plus 3'0" of freeboard. (It would seem that this was deemed adequate to cater for a discharge 25 per cent greater than 1929, and constitutes what was termed "complete" protection).

(iii) Potts' plan for "complete" protection should be adopted.

This report was approved by the Council and work on preparing detailed designs and working drawings commenced. Dare later suggested widening the Tamar Channel near River St. and considerable discussion ensued between him and Potts on the design of major lock gates at Charles St. in the N.Esk and the design of the N.Esk diversion channel. At this time the Launceston Marine Board commenced construction of a hydraulic model of the whole of the Tamar from Launceston to the mouth (40 miles) to study port improvement and it was hoped that this model would provide the design answers for the N.Esk diversion. It was finally concluded in 1941, after a number of tests, that the scale of this model was too small for this problem, and Potts and Dare decided that a special model should be built of the diversion channel area. (This was not done, presumably due to war-time staffing difficulties).

On 29th September, 1942, Dare submitted a progress report to the Council. This report had been drawn up in close consultation with Potts, and recommended works basically similar to those shown in Fig.2 which are in essence Potts' "complete" scheme with minor modifications. He pointed out that some additional design and investigatory work was necessary before final working drawings could be produced. As a result of further work, on the eve of Potts' departure to an appointment in Hobart, Potts and Dare submitted their final report on 12th March, 1945. This final proposal is shown in Fig. 2. It retains the basic concepts of the previous reports while providing for a number of minor modifications. The main features were:-

(i) Diversion of N.Esk River across Mowbray Flats to Stephenson's Bend in the Tamar by a cut 8000 feet in length.

(ii) A railway bridge on the N.E. railway across this cut.

(iii)A road-tramway bridge across this cut for Invermay (Georgetown) main road.

(iv) An earth flood retaining bank for the N.Esk water from Invermay Rd. to the hill near Cypress St., this bank to be provided with sluices where it crosses the N.Esk, to allow tidal flow in this river down to Charles St. until such time as the river channel can be reclaimed.

(v) High concrete levees from the hill near River St. parallel to and 1400 ft. from the Tamar Bank, crossing the N.Esk just below Charles St. bridge, sealing it off at this point, thence to high land in Royal Park with 10 roadway etc. openings in the concrete wall, each 14 ft. wide with gates on rollers for closing.

(vi) Diversion of sewers.

(vii) Trees to be removed from west bank of Tamar in Home Reach.

(viii) Cataract shoal at the upper end of Home Reach to be dredged and the spoil used to fill up N.Esk channel above the Charles St. cut off.

(ix) The bend in Home Reach below River St. to be widened.

(x) Scour protection to be provided in the lower reaches of the N.Esk diversion cut.

The average height of the concrete wall of (v) above is stated by Potts to be 12 ft. The cost of the L.C.C. investigation up to November 1944 was £3, 124 (£9,372) excluding salaries of Potts and the engineering staff of the Council, which would increase this cost several fold.

In the early post-war years the Council proceeded to resume lands along the route of the N.Esk diversion and levees, and discussions occurred on methods of construction of this channel. The possession of such property in the floodable area by a public corporation means that some of the betterment increment arising out of the proposals in this report will accrue to the general public.

On 14th April, 1951 a firm of contractors (Keir and Cawdor Ltd.) submitted a tender price of £385,000 for the construction of the diversion channel, but this tender did not include the cost of subsidiary embankments, road and rail viaducts, N.Esk cut off structures and property resumptions in the line of the channel. On 11th November, 1954, the City Engineer (L.H. Bird) estimated the cost of all the North Esk works at £1,074,000. On 22nd December 1954 he estimated the cost of the South Esk works, to obtain a grand total figure for the whole of the Potts-Dare project of £1,500,000 (1954 money values).

During this period fears were expressed that the N.Esk diversion would have an adverse effect on navigation and port maintenance, and finally it was decided that the whole problem should be investigated afresh by the creation of the Launceston Flood Protection Authority.

A6 SOME LOCAL MISCONCEPTIONS

(i) Nature of Misconceptions

The author found that there was a considerable body of local opinion which was complacent in the belief that although damaging floods had occurred in the past, they would not occur again in the future. Some examples of these arguments and the author's comments on them are given below.

(ii) Failure of Briseis Dam

Some citizens of Launceston and the Esk Valleys stated that the disastrous flood in Launceston in 1929 was caused by the "bursting of the Briseis Dam." This dam did indeed fail in the 1929 flood, sweeping away portions of the village below it and causing the loss of 13 lives. However, this dam was on the Ringarooma River, the waters of which do not pass Launceston.

(iii) Collapse of Perth Bridge

This bridge, some 18 miles by river from Launceston, and above the confluence of the Lake and Meander rivers, did in fact collapse, but from hydraulic considerations it is obvious that the relatively small storage behind such a bridge on an open plain could not impound water in such volume that its rapid release would have any effect on flood levels at Launceston.

An eye witness (P.J. Waddle of "Everton Springs" Evandale) states that on the Friday of the collapse he proceeded to "Eskleigh", about half a mile below Perth Bridge, to complete the job of installing a generator, but on finding the flood about half way up the engine block he commenced to remove the machine. When the bridge collapsed the level at "Eskleigh" rose by "about a foot". Further downstream the river spreads over the plains around Longford where there is considerable storage, before proceeding past Hadspen down the 6 miles of gorge, so that this increase of one foot just below Perth Bridge would mean an increase of a small fraction of an inch at Launceston.

Further, Perth bridge failed on Friday about noon and the peak of the flood at Launceston occurred in the early hours of Sunday morning.

(iv) Effect of Trevallyn Dam

Trevallyn Dam is a 75 ft. high concrete structure built at the entrance of the gorge in 1955 for Hydro-power purposes. Its T.W.L. storage is 7,000 acre feet. The total discharge of the 1929 flood is estimated at 840,000 acre feet, so that even if it had been in position and empty at the commencement of the 1929 flood, it would have been filled in the first half hour or so, and would have had no effect on the 1929 flood level in Launceston.

(v) Effect of Lock on the Tamar

Sporadic discussion had occurred for years past on the advisability of constructing a lock on the Tamar 27 miles below the gorge to maintain a permanent high tide for aesthetic and navigation purposes and for flood control. In the latter respect, the idea is that on warning of a flood, the waters of the Tamar would be lowered to low tide level through the lock gates, which would then be closed, thus obviating any tidal influence above the regulator.

It is clear from Russell Kidd's observations and from other evidence that during the 1929 flood the Tamar River level a few miles below Launceston rose and fell with the tide, but the level in Home Reach and the flooded areas of Launceston remained approximately constant for at least 24 hours. Hence for major floods the cause of flooding is the inability of the floodwaters to discharge rapidly enough from the vicinity of Launceston. A lock on the Tamar would have no effect on this condition, and flood mitigation is not a benefit which could be credited to such a proposal.

(vi) Insurance as a Substitute for Flood Mitigation

Insurance companies have accepted flood insurance in Launceston at fairly low premiums, and some people have suggested that flood insurance would make flood mitigation measures unnecessary. If this argument is pursued to its logical conclusion, then expenditure on fire brigades and care in avoidance of damage to mudguards of private motor vehicles are also unnecessary, as insurance companies bear fire and motor car losses. However, insurance companies do not operate at a loss. A flood ruined piano or a damaged bale of wool is a real loss to the community which affects the national economy. Insurance merely spreads a loss over a wider field. Flood Insurance premiums will rise sharply after the next flood which overtops the levees in Launceston. Insurance is not applicable to floods, as only floodable areas will insure.

(vii) Zoning and Transfer of Population

The view has been expressed by some people of Launceston that it would be better to move the people out of the Inveresk-Invermay area rather than to go to the expense of protecting them from floods. However, it is found all over the world that people insist on developing flood plains and rarely has the removal of the population from a flood-threatened zone been carried out. This insistence of developing flood plains is not necessarily unwise. It is the task of Civil Engineers to control the forces of nature for the use and convenience of man. If the expenditure on such control is economically justified or required in the interests of national development, then there is no reason why the river should not be forced to behave in any desired manner.

In Part E of this report it is clearly demonstrated that the protection of the Inveresk-Invermay area from floods is economically justified, and any suggestion of giving up development of the area is quite unsound.

Although the removal of the population from flood-threatened areas has rarely been carried out, the zoning of such areas for restricted development, such as parklands, has sometimes been enforced. In no part of the flood plains of Launceston is such zoning necessary because the cost of protection against floods is relatively low.

In fact, if, in the middle of last century, the Government of the day had decreed that Inveresk-Invermay area was not to be developed, then this would have been an unwise decision and probably in any case the decree would have become a dead letter.

A7 DANGERS OF COMPLACENCY REGARDING THE FLOOD MENANCE TO LAUNCESTON*

It is clear that the Government of Tasmania is alive to the flood menace, as is evidenced by the setting up of the Flood Protection Authority. However, no damaging floods have occurred for 30 years, so that it is wise to put on record in this report some comments on the erratic flood behaviour of rivers.

Experience all over the world shows that rivers deal out their minor and major floods in the same irregular manner as the dealer of a well shuffled pack of cards deals out kings and aces. Hoyt and Langbein (A2) give some examples, which may be summarised as follows:-

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Ref. (A2) Hoyt and Langbein "Floods" 1955

* If, in 1928, a citizen had predicted the 1929 flood, he would have been ridiculed, as the previous major flood was in 1893.

"Before May, 1935, the citizens of Wakefield (U.S.A.) would have ridiculed anyone who suggested that the discharge of the Republican River could greatly exceed the 70,000 cubic ft. per second recorded in 1915, because this was the highest recorded over a period of 50 years. Yet in May, 1935, a flood of 180,000 cubic feet per second (cusecs) occurred.

The people of Hartford might appear to have had a stronger case for complacency in that they possessed a 90-year-record of flood discharges and also some sound evidence that the flood of 1852 was the highest since 1683, so that on these records it would have been reasonable to argue that a flood height of 32 ft. at Hartford would be so rare (certainly rarer than once in several centuries) as to warrant little concern. Floods of such magnitude had never occurred in 300 years, the highest recorded being the 1854 level of 29 ft., and yet in 1936 a peak stage of 38 ft. was reached and then again in 1938 a level of 35 ft.

The city of St.Louis experienced a flood of 42 ft. in 1785 and 1844, and then followed a period of 100 years with low peak stages (except 38 ft. in 1904) until 1943. Then in succession in 1943, 1944, 1947 and 1951 levels of 37, 39, 40 and 40 ft. respectively occurred."

A few Australasian examples can be quoted. In 1934 the spillway of a dam on the Latrobe River (Victoria) was being designed. In 40 years of record the highest discharge recorded was 20,000 cusecs, so the design capacity was fixed at 40,000 cusecs. Just as this decision was reached, a flood of 110,000 cusecs occurred.

Burrinjuck Dam (N.S.W.) was originally designed for a peak spillway discharge of 80,000 cusecs based on the best available data. Immediately on completion a discharge of 387,000 cusecs was recorded.

Up to May, 1948, the previous peak discharge on the Wairoa River (N.Z.) was estimated at 235,000 cusecs occurring in 1914, and yet in 1948 the figure of 404,000 cusecs was recorded.

Dozens of similar examples can be given illustrating that a river may be relatively innocuous for 50 years or more and then go berserk. A citizen of Launceston who predicted in 1928 a flood such as that of 1929 would have been ridiculed, because the previous serious flood occurred in 1893.

A8 POSSIBLE FLOOD MITIGATION MEASURES

At the commencement of his task, the author reviewed all the possible solutions as follows:-

(a) Land Treatment on the Catchment

While being beneficial in mitigating minor floods, this method would have little effect on major floods. In any case it would be administratively impracticable.

(b) Small Upper Catchment Dams

The remarks under (a) above apply also to this method.

(c) Major Storage Dams in Middle and Lower Reaches of Catchment

Very few suitable dam sites exist except on the site of Trevallyn Dam. Rich agricultural lands and townships would be submerged by the raising of this dam or the construction of any dams in the middle and lower reaches. In any case, the cost of a major dam would be far in excess of the £1,500,000 estimated for the Potts-Dare scheme. Hence this method was discarded at the outset.

(d) Diversion of S.Esk through Cormiston Creek to Tamar River

Above the gorge there exists a saddle in the divide, and a cut through this saddle would divert the waters of the S.Esk into Cormiston Creek, which discharges into the Tamar well below Stephenson's Bend. Such a river diversion would give complete flood protection to Launceston. Potts made an aneroid barometer traverse along the line of such a cut, and found that the height from the bed of the river to the crest of the divide approximated 150 ft., and he discarded this proposal as obviously too expensive for serious consideration.

Since then Trevallyn Dam has been constructed to a height of 75 ft., and the rise from T.W.L. to the crest of the divide is 92 ft. It was therefore considered that this proposal merited consideration.

(e) Levee Systems

From the nature of the topography, it was clear that the raising and perhaps re-aligning of existing levees with some auxilliary system of new levees held out promise of a fair degree of protection at reasonable cost.

(f) River Straightening

Peak flood levels by Potts from Maddock's survey are shown in Fig. 33. A study of these levels led to the conclusion that a major cause of flooding was the resistance to flood flows in the section of the Tamar from River St. at the downstream end of Home Reach through Ti-Tree and Stephenson's bends to Boat Channel, where the river widened out considerably. On 26th June, 1912, Hunter, a British Consulting engineer, submitted to the Marine Board a report entitled "River Tamar - Proposed Improvement Work" dealing with methods of improving Launceston as a port for interstate shipping. One of his recommendations was the construction of a navigation cut from Home Reach to Boat Channel, as shown in Fig.1, thus eliminating two sharp bends in the river. A half tide wall was proposed across the old channel. The work was actually commenced in 1916, but was discontinued in 1921. It seemed possible that the completion of this work would lower flood levels in Launceston by improving the hydraulic efficiency of the river by shortening its path and eliminating the resistance to flow caused by the bends. It was felt that if a major river diversion was carried out, this "Hunter Cut" would be a wiser expenditure of funds than the diversion of the N.Esk, because this cut combined with low levees might not only prove to be a better economic proposition than the Potts-Dare scheme, but would have the duel purpose of port improvement and flood mitigation.

(g) Floodways

A proposal similar in principle to (f) above, but without navigational benefits, is to provide a wide shallow floodway across the flats along the general line of the Hunter Cut.

(h) Channel Improvement

Various methods of channel improvement, such as rounding off the bend at the downstream end of Home Reach or a general widening and deepening of the whole channel, seemed worthy of testing.

(i) The Potts-Dare Proposals

During high flows in the N.Esk, these proposals would increase the navigational difficulties for masters of ships rounding Stephenson's Bend, as a strong lateral flow from the N.Esk would enter the Tamar at a critical point in the bend. This is a serious disadvantage of the Potts-Dare recommendations. There is also a possibility of siltation troubles on the N.Esk below Charles St. Bridge. Quite independently of this, however, the author was doubtful of the need for a N.Esk diversion.

(j) Comparison of Proposals

While there existed no doubt that proposals (e), (f), (g) and (h) would have some effect in mitigating flood damage, the magnitudes of these effects and the relative efficiencies of various methods defy any mathematical analysis.

Therefore, model studies were planned to measure the hydraulic efficiencies of various versions of each of these methods, and of combinations of one or more methods.

As previously mentioned, these model tests threw up a further solution - a training levee in Royal Park to divert in a northerly direction the concentrated flood flow of the S.Esk.

Another method of flood damage mitigation should be mentioned, viz;- the installation of telemetered rainfall and river height instruments on the catchment to provide accurate quantitative flood forecasting. This method is discussed in Part F of this report.

The topographic, hydrologic, hydraulic, and economic investigations necessary to choose the best method are described in detail in the following sections.

PART B

TOPOGRAPHIC SURVEY

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B TOPOGRAPHICAL INVESTIGATIONS

B1 BASE PLAN FOR MODEL

B1.0 Introduction

In order to construct the model and to study the proposed improvement plans for the flood protection of Launceston it was first necessary to prepare an accurate survey plan of the area to be moulded. This plan is shown in Fig. 3. The amount of field work by the survey party, led by D.N. Foster, was much greater than originally expected and constituted a major portion of the cost of the work of the Authority.

B1.1 Planimetric Control

At the start of the investigation three plans were available from the L.C.C. which showed details of the topographical features in Launceston, viz:-

(i) Bench Marks and Contours for Launceston (1955)

(ii) Launceston and Environs (1952)

(iii) Detailed survey of Portion Relating to Flood Protection Scheme (Wilks 1937)

At the outset, it seemed reasonable to assume that these plans would obviate any need for a survey of the topographic features.

However, when the grid for the model was being laid down at the Water Research Laboratory, it was discovered that there were considerable horizontal discrepancies amongst these three plans. In particular the location of the river channels varied considerably. As it was most important that river bends and channel boundaries be accurately reproduced on the model, it was necessary to carry out fresh surveys and plot a new plan to locate accurately the main topographical features which could affect flood levels in Launceston.

This plan was prepared from 16 chain to the inch serial photographs of the area taken in 1957, and was plotted to a scale of 1 inch to 500 feet by the slotted template method. Ten ground control points for the aerial photographs were established by a triangulation survey. A base line 1746.20 ft. in length was chained along King's Wharf. This base was then transferred to two major controls on Trevallyn Hill be means of a crossed quadrilateral. The eight additional ground control points were then established by a series of triangles from these major control points with frequent cross sights as an additional check on the accuracy of the survey.

B1.2 Ground Levels

B1.21 General. Once the topographical features had been established from the aerial photographs, it was then necessary to plot the ground surface contours over the entire area to be modelled. A check on the spot levels taken by Wilks in 1937 for the flood plain between Hobler's Bridge and Stephenson's Bend indicated close agreement with present day conditions. These levels were adopted for contouring the base plan in this area to the R.L. 20 level. Above this height there were insufficient spot levels to define accurately the ground surface and a series of level traverses were made over the area to establish the intermediate contours between the R.L. 50 contour shown on the Bench Mark plan for Launceston and R.L. 20 contour plotted from Wilks' Survey.

Below Stephenson's Bend no details of ground levels were available and it was necessary to make a detailed level survey on each bank of the Tamar River between the Hunter Cut and Ecclestone Road.

B1.22 Tailrace Area. For construction of the model in the vicinity of the Trevallyn Power House tailrace channel two requirements had to be met. For the verification tests it was necessary to mould the model to the surveys most nearly approaching the conditions existing at the time of the verification event (1929 topography). For this purpose the levels shown on Wilks' survey, which was made prior to the construction of the Power Station, were used. Once the model had been verified, it was necessary to alter the topography to agree with present day conditions. The ground levels shown on the base plan (Fig.3) correspond with this latter condition. They were established by selected levelling in this area co-ordinated with details shown on several plans available from the H.E.C.

B1.23 Marine Board Silt Deposit Areas. The levels shown on the base plan in these areas are those taken by Wilks in 1934 and correspond with the model surface moulded for the verification tests.

After verification had been completed the levels of the sediment settling areas were raised to agree with the final levels proposed by the L.M.B. as supplied by the Chief Engineer (Mr. J.Edwards) of that Authority.

B1.24 Embankment Levels. Theoretically these should correspond to that existing in 1929 for the verification tests, and to present day topography for the base tests. However, no details of embankment levels at the time of the flood of April 1929 were available. In addition, settlement of the levee banks and maintenance carried out periodically result in slightly varying levee heights depending upon the date of the survey. For these reasons, the embankments were moulded for all model tests to correspond with the survey made by the L.C.C. in 1934. It is considered that these levels are representative of the general embankment heights.

B1.3 River Channels

B1.31 North Esk River. Eleven cross sections of the North Esk River were taken in 1937 by Wilks between Charles St. and Hobler's Rd. Bridges. This number was insufficient to model accurately the North Esk River Channel and a further 19 cross sections of the river were measured. In addition, two of Wilks' sections were checked and as these indicated that there were no significant changes in the area of waterway available, his original sections were also used for the model construction. The locations of the cross sections were established by a survey traverse closing into fixed topographical features plotted from the aerial photographs. The portion of the North Esk between Charles St. Bridge and the Tamar River was moulded to agree with the sounding plan of this area taken by the Marine Board in 1939.

B1.32 South Esk River. No survey plans were available for the South Esk River Channel. A survey traverse was run from the Cataract Bridge at the mouth of the gorge to a point 1,000 feet above the First Basin Suspension Bridge and twelve cross sections of the river channel were taken by stadia levelling combined with river soundings. The location of these sections is shown on the base plan.

B1.33 Tamar River. At the commencement of the investigation there were available fifteen cross sections of the Tamar River from its junction with the South Esk to No.6 Beacon opposite McKenzie St. These were surveyed by Wilks in 1937. In addition sounding plans of the river channel in this area were available from the L.M.B. for the years 1935 and 1939. This provided sufficient survey data for construction of the River Channel in the model from its confluence with the North and South Esk Rivers to Stephenson's Bend.

Below Stephenson's Bend there were insufficient details of the river channel for model construction. The L.M.B. was requested by the Authority to sound thirteen cross sections of the river between No.6 Beacon at Stephenson's Bend and the downstream limit of the model at Boat Channel. The location of the river cross sections used in the model are shown in Fig.3.

The cross section of the Tamar River Channel at any particular time is dependent upon the quantity of silt deposited during periods of low river flow and the duration since the channel was last dredged. For these reasons the cross sectional area available for flood flow would depend on the time of the flood. In addition, the scour that results during a flood is unknown. It is considered, however, that the channel moulded in the model as a result of these soundings is a reasonable representation of the average type of cross section available for the conveyance of flood flows.

B2 SUBSIDIARY TOPOGRAPHICAL INVESTIGATIONS

In addition to the preparation of the base plan for the model, a considerable quantity of survey work was carried out for other phases of the investigation viz.

(i) Survey of line of proposed levee embankments

(ii) Survey of Evandale saddle

(iii) Survey for costing of Hunter Cut

(iv) Survey for costing of Cormiston Creek

(v) Survey for location of flood gauges

Details of these surveys are given in the appropriate sections of this report and they have been filed with the records of the Authority for future reference.

PART C - HYDROLOGIC STUDIES

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PART C - HYDROLOGIC STUDIES *

C1.0 INTRODUCTION

C1.1 Description of the Catchment

The catchment area of the Tamar River above Launceston is 3,767 sq. miles, covering approximately one seventh of the total area of the island. The basin is roughly semi-circular in shape, with Launceston at the centre, as shown in Fig. 4. The periphery consists of high mountain ranges from which spring the headwaters of the main drainage network. The river system is roughly analogous to the radial spokes of a semi-circular wheel, with Launceston as the hub. This situation tends to cause, in certain circumstances, a simultaneous concentration of flood peaks at Launceston.

Broadly speaking, the catchment may be divided into five main sectors as follows:-

(a) The North Esk River having a catchment area of 412 sq. miles and draining the N.E. sector.

(b) The South Esk River with a catchment area of 1,304 sq. miles and draining the Eastern zone.

(c) The Macquarie River, a tributary of the Lake River, draining 1,018 sq. miles of the S.E. sector.

(d) The Lake River, a tributary of the South Esk, which drains the S.W. portion of area 444 sq. miles.

(e) The Meander River, a S.Esk tributary draining 589 sq. miles of the N.W. area.

The North Esk River rises in the northern slopes of Ben Lomond plateau (5,160 ft.) about 30 miles from Launceston as the crow flies. The river drops from a general river plain level of 1,500 ft. to 1,100 ft. at the confluence with its main tributary, the St. Patrick's River (137 sq. miles) on a grade of about 20 ft. to the mile. The St. Patrick's River has a slightly flatter general slope. From the confluence the river

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* A more detailed description of the hydrologic analyses referred to in this section will be found in a thesis by D.N.Foster, submitted to the University of New South Wales in support of candidature for the degree of Master of Engineering.

bed drops sharply almost to sea level at St.Leonards near Launceston, cutting its way en route through a rocky gorge above Corra Lynn. The headwaters of the river are surrounded by high rugged mountains and for the major proportion of the length these mountains rise directly from the river's edge.

Tracing the encircling mountain ranges from Launceston in a clockwise direction beginning at Mr.Arthur (3,895 ft.) some 13 miles east of Launceston, the range proceeds easterly to Mt. Maurice (approx. 3,500 ft.) thence in a southerly direction through Ben Nevis (3910 ft.) to Legge Peak (5,160 ft.) on the Ben Lomond plateau, turning in a westerly direction to Temple Bar (2,348 ft.) whence it drops sharply to an elevation of 600 ft. above sea level at Western Junction some 9 miles SSE of Launceston.

The North Esk River is separated from its main tributary, the St. Patrick's River, by a high mountain divide beginning at Mt. Maurice on the perimeter of the catchment and passing en route through Mt.Barrow, 4,644 ft. above sea level.

The South Esk rises north-east of Ben Lomond, is joined at Fingal by the Break O' Day River, at Avoca by the St. Pauls and above Evandale by the Nile, its general slope being 8 ft. per mile until it reaches Hadspen just above Launceston, whence it drops sharply 400 ft. through a spectacular gorge to the waters of Home Reach at its junction with the Tamar.

For about half its total length from the headwaters to Llewellyn, 8 miles downstream of Avoca, the general river plain is only 1 mile wide and is surrounded by the rugged unsettled east coast ranges. Below Llewellyn the river plain widens appreciably, especially on the western bank, and the valley becomes more undulating and densely settled.

Tracing the encircling mountain ranges in a clockwise direction, the northern boundary of the catchment forms the divide between the North Esk River described above. From the boundary of this divide the range proceeds easterly through Mt.Victoria (3,964 ft.) to Mt. Young, the whole range being relatively lofty and rugged. Thence the direction is S.E. through the Nicholas Range (2,812 ft.) to St. Patrick's Head (2,227 ft.) on the East Coast, thence S.S.E. to Snow Hill (2,175 ft.) on the border of the South Esk and Macquarie catchments. The mountain divide between these two rivers proceeds in a W.S.W. direction for a distance of 11 miles to Mt. Campbell (2,356 ft.) whence it drops steeply for about 6 miles to an elevation of 700 ft. above sea level near Llewellyn. The remainder of the catchment boundary is generally below 1,000 ft. through undulating plains.

The main tributary of the South Esk above its junction with the Lake River is the St. Pauls, with a catchment area of 159 sq. miles. It drains the S.E. zone of the South Esk Catchment and is divided from this catchment by a mountain range having a general elevation of approximately 2,700 ft., the highest peak being St. Paul's Dome (3,368 ft.).

The Macquarie River, together with its main tributaries, the Elizabeth (catchment area 165 sq. miles) Blackman (188 sq. miles) and the Isis (126 sq. miles) gathers its flow from the S.E. corner of the catchment. This area can be subdivided into two zones, the Elizabeth and Upper Macquarie which rise in the rugged East Coast Mountains and for the major proportion of their length have only a narrow river plain surrounded by lofty mountains of an average elevation between 1,800 and 2,000 ft., and the Blackman, Isis and Lower Macquarie which have their headwaters in the Western Tiers, a high mountain range of elevation between 3,000 and 5,000 ft., and running generally N.W. for about 70 miles from their junction with the East Coast Range at Oatlands. These latter rivers drop sharply from their headwaters to elevations below 1,000 ft. and for most of their length have a much wider river plain than the East Coast rivers. The flat river plains are termed the midlands and form one of the richest grazing areas in Australia for super fine merino sheep.

The Lake River, of which the Macquarie is a tributary, has its source at Arthur Lakes (elevation 3,107 ft.) on the Western Tiers. In the first 22 miles the river drops sharply to an elevation of 1,000 ft. and is surrounded on each side by the high mountains of the Western Tiers. For the remainder of its journey to its junction with the South Esk at Longford, 38 miles away, the river runs through the plains of the midlands.

The Meander, which joins the South Esk at Hadspen six miles from Launceston just above the entrance to the gorge, rises on the northern boundary of the Western Tiers on the Western side of Quamby Bluff (4,200 ft.). The river drops rapidly over the first 8 miles to an elevation of 900 ft. and for the next 52 miles of its length to its junction with the South Esk runs through undulating grazing country, the general slope of the river being 13 ft. to the mile.

The topography of the catchment can be divided into three zones:-

(i) The East Coast Ranges of a general elevation of 2,000 to 3,000 ft. covering approximately 45 per cent of the total catchment area. This mountain range forms the headwaters of the North Esk, Nile, Upper South Esk, Break O'Day, St. Pauls, Elizabeth and Upper Macquarie Rivers and for the major proportion of the length of these rivers they are surrounded on all sides by rugged mountains with only a narrow valley plain.

(ii) The Western Tiers of the Great Lake Plateau, which form the S.W. boundary of the catchment and meet the East Coast ranges near Oatlands. This mountain range, which rises sheer from the plains below, has a general elevation of 3,000 to 4,000 ft. and covers about 15 per cent of the total catchment area. From it flow the headwaters of the Blackman, Isis, Lake and Meander Rivers. Unlike the east coast rivers, these streams drop sharply from their headwaters to the plains below and have in general a much wider valley plain.

(iii) The "Midlands". Between these two mountain ranges and open to the North lie the "Midlands". This gently undulating plain forms one of the richest grazing areas in Australia. The general elevation of the land surface is about 300 to 600 ft. above sea level and the area is about 40 per cent of the total catchment.

C1.2 Rainfall

The annual mean rainfall varies considerably over the different portions of the catchment. The more common Westerly winds from the Antarctic cyclones, which is the major influence on weather over Tasmania, produces high precipitation and high annual rainfalls on the West Coast mountain ranges. As the moist air stream from the Antarctic Ocean passes over these high mountains, it is depleted of much of its moisture. In consequence the eastern portions of Tasmania have a relatively low annual rainfall, with the exception of the North Eastern corner of the island, in the vicinity of the North Esk catchment. This area is open to the N.W. winds across Bass Strait and annual rainfalls up to 60" are recorded on the coastal ranges.

An isohyetal map of average annual rainfalls of Tasmania and the Tamar River catchment is shown in Fig. 5. The more common westerly weather is not the cause of the major floods on the South Esk catchment, as the majority of the air moisture is deposited in its passage across the West Coast mountains.

On relatively rare occasions a cyclone of semi-tropical origin is sited in the Tasman Sea, bringing N.E. to S.W. winds of high moisture content directly on to the East Coast ranges, causing heavy rain on the headwaters of the catchment. This is the usual potential major flood situation.

A characteristic of all storms over the South Esk Catchment is high rainfall on the surrounding mountain ranges with a tendency towards a rain shadow on the midlands. This is exemplified by the fact that the average annual rainfall at Ross is only 18.6 inches and is typical of a catchment surrounded by high mountains inducing orographic precipitation.

C1.3 Snow

Although snow falls on some of the higher sections of the catchment, the area affected is less than 700 sq. miles and the depth and duration of snow is negligible. For these reasons snow is not a significant factor in the cause of major floods.

C1.4 Data Collection

C1.41 Hydrologic Control. The rainfall and streamflow gauging stations are shown in Fig. 4.

C1.42 Streamflow Measurements

(a) South Esk River.

At the outset of the investigation the only authority which maintained any accurate streamflow records were the H.E.C., whose main interests were in the elevated areas.

Up to 1945 no accurate records of the relationship between river stage and river discharge had been established for the South Esk at Launceston, although the L.C.C. had recorded peak flood heights on a staff gauge at Duck Reach Power Station for the period 1900 to 1956, the station being rated in 1931 by the inaccurate method of measuring surface velocities only, by timing the passage of floating drums down the river, together with occasional surface velocity measurements by a current meter. Subsequent accurate rating of this river by the H.E.C. indicated errors of the order of 80 per cent in this approximate curve. This is not surprising, as the tremendous turbulence in the gorge, even for small discharges, is such that it is quite impossible to measure velocities with any degree of accuracy. The H.E.C. rated the station some miles upstream at Hadspen.

During 1945 the H.E.C. were forced to take a hand by reason of its desire to construct a Hydro-Power dam (Trevallyn Dam) near Launceston and in the subsequent years the river was accurately rated by current meter up to 46,000 cusecs. Unfortunately, however, no flood greatly exceeding this discharge has occurred since 1945, and in order to estimate earlier floods of higher magnitudes the rating curve must be extrapolated with a consequent loss of accuracy.

River stage at Duck Reach is measured on a staff gauge situated in a bend of the river where considerable wave action occurs during times of flood and it is difficult to estimate the mean flood level with any degree of accuracy. This must be borne in mind when relating flood discharge to the recorded stage. In 1932 the L.C.C. installed a "Bristol" pressure type stream height recorder and a stilling wall at Duck Reach to record river stages. Records are available for the period 23.2.39 to 1.1.45 but owing to diurnal temperature fluctuations, causing an apparent change in river level when none occurred, the records are of limited value only.

During 1945 the H.E.C. installed a "Leupold and Stevens" float recorder and a stilling wall at their rating station near Hadspen above Trevallyn Dam and all subsequent flood discharges of the South Esk River at Launceston have been related to this station.

After Trevallyn Dam came into operation in 1956, a proportion of the flood waters were diverted by tunnel to the penstocks of the Power Station on the Tamar River so that flood discharges at Duck Reach downstream of the dam are reduced not only by the increment of storage in the dam itself but also by the quantity of water diverted for power production. The inflow recorded at the H.E.C. rating station above the dam must be modified to allow for these factors to determine the volume of water which flows down the gorge.

Owing to the action of the H.E.C. in establishing an accurate gauged station on the South Esk near Launceston, no further action was needed by the Launceston Flood Protection Authority to measure discharges in this river.

After the 1929 flood, the river slope at Duck Reach was established by a trash line survey in this area. Further records of river slopes were taken by the L.C.C. for the floods of

1931 in order to determine the roughness coefficient for the river channel. This was subsequently used to estimate the 1929 flood discharge by slope-area calculations based on Manning's formula. As these calculations were based on inaccurate values of discharge they are erroneous. It was decided to repeat this approach to obtain one estimate of the 1929 flood. In order to fix more accurately the roughness parameter of the river channel and to assess any change in the magnitude of this parameter with magnitude of flood, two additional staff gauges were established by the Authority in Duck Reach on which the river slopes were recorded during flood flows. One gauge was installed 800 ft. upstream of the Suspension Bridge and the other 500 ft. downstream, the zero of the gauges being 20 ft. above and 10 ft. below the zero of the bridge gauge respectively.

(b) North Esk River.

At the start of the investigation no reliable streamflow data for the North Esk below its junction with the St.Patrick's River was available. In 1943 the L.C.C. installed a staff gauge and a "Bristol" pressure stream height recorder at the bridge over the North Esk at Corra Lynn. Records are available for the period 23.3.43 to 28.2.44 but owing to a very severe diurnal temperature fluctuation indicating a non-existent rise and fall of the river level during each 24 hours, the records were practically useless. In addition, the station was rated by the inaccurate method of measuring surface velocities by timing the passage of drums floating down the river. Subsequent rating curves established by the staff of the H.E.C. at the request of the Authority showed that this original curve was in error by almost 100 per cent.

Apart from the Bristol records the only flood readings available for the North Esk were those read on the staff gauge at the time the river was being "rated" by surface velocity measurements. These were as follows:-

24.8.36	Bridge	gauge	200.0	(above an arbitrary datum)
15.3.37	"	"	180.2	
7.1.38	"	"	190.3	
4.6.42	"	"	197.0	
6.7.44	"	"	194.4	

No record could be found as to when these records were taken in reference to the peak of the flood. Although the L.C.C. had arranged for rating the river as early as 1936, apparently no systematic records of river levels were taken until the "Bristol" recorder was installed in 1943.

As a result of the lack of streamflow data on the North Esk River and the inherent inaccuracies in the limited data that was available, the Authority decided to install a new "Bristol" pressure type stream height recorder, downstream of the bridge and to re-rate the station accurately by means of a current meter. Satisfactory records were obtained, the diurnal temperature variation effect being relatively small as the instrument was located under a shelf of rock.

During a fresh in the river on 28.6.57 an attempt was made to rate the river from the bridge across the gorge. It was found that the section was unsuitable because of the extremely high velocities, excessive turbulence and the formation of a back eddy across half the cross section. Therefore the gauging station was moved to a section downstream of the bridge where an excellent control was available in the form of a natural rock weir backing up the water above a series of rapids. This involved the installation of a cableway across the river to support the current meter. The work was carried out for the Authority by the H.E.C. A new staff gauge was also installed at this site.

As is so often the case, the period after installation of the equipment was extremely dry and it was not until May 1958 that a flood of any magnitude occurred in the North Esk. Fortunately this flood was of near record magnitude and it was possible to rate the river to a high range. The rating curve and the relationship between the various gauges is shown in Fig. 6.

This flood was followed by several floods of smaller dimensions which gave sufficient streamflow data for the purpose of the investigation.

(c) Tamar River.

At the outset, no data were available of streamflow or velocity distributions in the Tamar River downstream of the confluence with the North and South Esk rivers. Although there was clear evidence that for the flood of April 1929 the tidal influence was washed well downstream, the limit of discharge, above which tidal interference to flow patterns and velocity distributions was negligible, was unknown.

In order to gain more information on the influence of tide on flood flows of magnitudes less than that of 1929, the Authority made arrangements with the L.M.B. and the H.E.C. for a series of vertical velocity traverses at 40 ft. intervals to be taken at a cross section of the Tamar River at the Powder Jetty, for flood discharges exceeding 16,000 cusecs. Special equipment for this rating was designed by the H.E.C. Senior Hydrographic Engineer (J.Park) and mounted on a launch.

As floods of magnitudes exceeding 16,000 cusecs were observed, the upper limit of discharge above which observations were taken was increased. At the date of this report observations had been taken for flood flows up to 34,000 cusecs.

C1.43 Flood Stages in Launceston Area. Prior to 1929 no systematic measurements of flood heights reached in Launceston were made. In order to fix the heights for the floods prior to this date reliance had to be placed on newspaper reports. A detailed discussion on how these heights were estimated is given later in this report, (Section C7.25).

Immediately after the 1929 flood a surveyor (J.Maddocks), at the direction of the L.C.C., fixed the peak flood levels in the vicinity of Launceston by a trash line survey. Details of the levels reached by this flood are shown on Fig. 33. This clearly indicated an hydraulic grade in Home Reach above the tailrace channel markedly flatter than that around Stephenson's Bend, which leads to the conclusion that a constriction to flow occurred in this vicinity. Whether this was due to bend resistance or a reduction of the channel cross-section was not known.

When the City Engineer of Launceston (W.Potts) began investigations on flood protection measures in 1931, he installed three "Gurley" automatic tide recorders at Forster St., Stephenson's Bend, and Diversion Bend on the North Esk River.

Tide charts for these recorders were available for the following periods:-

Forster St.	27.8.38 to 10.8.46
Stephenson's Bend	10.8.38 to 11.6.45
Diversion Bend	13.8.38 to 12.12.45

In addition to the tide records taken by the L.C.C., the L.M.B. has maintained a tide gauge in Launceston since October, 1955, and records are available from that date.

This recorder has been located since 1955 at a position midway between Charles St. and Tamar St. bridges, outside the offices of the Marine Board of Launceston.

When the Authority began hydrologic investigations in 1957, it was considered that additional information on flood levels and river grades were required. In particular more information was necessary regarding the constriction to river flows in the vicinity of Stephenson's Bend. For these reasons use was made of the L.C.C. "Gurley" tide recorders purchased by Potts. These were installed by the L.M.B. at the request of the Authority at the northern end of King's Wharf, at No.2 beacon on the western bank of the Tamar just above the tailrace and at No.6 Beacon in Stephenson's Bend opposite McKenzie St. Unfortunately finance did not permit continuous operation of these recorders and records were taken for the floods of May and August 1958 only. In addition to the automatic recorders, nine staff gauges were installed in the Tamar river to augment these records. These gauges were located at:-

(a) South End of King's Wharf
(b) North End of King's Wharf
(c) Powder Jetty 1,120 ft. downstream of King's Wharf
(d) Pile opposite the Powder Jetty
(e) No.2 Beacon on western bank upstream of tailrace channel
(f) Red Powder Dolphins on eastern bank downstream of tailrace channel
(g) No.4 Beacon on eastern bank between the Powder Dolphins and McKenzie St.
(h) No.6 Beacon in Stephenson's Bend opposite McKenzie St.
(i) Red Pile on eastern bank opposite downstream end of the Hunter Cut.

In addition to these staff gauges, fixed datum points were located on the hand-rails of the bridges over the North and South Esk Rivers so that flood stages could be recorded by measuring on a tape the distance to the water surface below a fixed level.

These gauges were located at:-

North Esk River

(a) Hobler's Rd. Bridge
(b) Henry St. Bridge
(c) Rifle Range Bridge

South Esk River

(d) Gorge Bridge on West Tamar Rd.
(e) First Basin Suspension Bridge

Arrangements were made with officers of the L.M.B. and L.C.C. to take continuous readings on these gauges and in addition to read the staff gauges at Duck Reach for all floods exceeding 16,000 cusecs. As floods of this order of magnitude were observed, the limit of flood discharge at which observations were to commence was increased. At the time of this report the instructions were to the effect that readings should be taken for floods exceeding 60,000 cusecs.

C1.44 Meteorological Information

(i) Rainfall

(a) Daily Rainfall Records.

Daily rainfall records were available from a number of official rainfall stations maintained by the Commonwealth Meteorological Bureau. At the commencement of the investigation there were 100 such stations on and adjacent to the catchment, the average areal distribution being one station per 20 sq.miles on the North Esk catchment and one station per 50 sq. miles on the South Esk catchment. The majority of these gauges are located in the lower elevation of the settled areas of the catchments. There were only 27 stations sited in elevations above 1,000 ft. and only 10 stations above 2,000 ft., all except one of these being on the Western Tiers. Large areas on the rugged east coast ranges and especially in the south-eastern sector of the catchment were unsettled and no rainfall records were available.

To supplement the records of the official observers, a survey of the catchment area was made to locate unofficial daily read rain gauges. As a result of this survey 15 additional gauges were obtained. This was sufficient to define adequately the areal distribution of rainfall over the inhabitated areas of the river plains. It was impossible, however, with the limited resources of the Authority, to establish an adequate coverage in the higher unsettled areas where access in many places was only possible by pack horse or helicopter and isohyetal maps had to be interpolated in some areas by consideration of such factors as land slope, wind direction, storm movement etc., in relation to those records that were available. This could be satisfactorily done on the Western Tiers where some records in the higher elevations were available, but at the commencement of the investigation little data on rainfall was available for the higher elevations of the east coast ranges.

The Authority therefore set about to assess the relative storm rainfall in these areas. Four long term pluviographs were installed on Mt. Victoria (3,500 ft.), Mt. Barrow (4,500 ft.) Tower Hill (2,500 ft.) and Tooms Lake (1,500 ft.). In addition to these pluviographs, the assistance of an additional two daily read rain gauge observers was co-opted to observe rainfalls at Rose's Tier (2,800 ft.) and English Town (1,800 ft.).

(b) Pluviographic Data.

At the commencement of the hydrologic investigation on the Tamar catchment there were only three pluviographs operating on or near the catchment. These were located at Launceston, Scottsdale and Western Junction in areas of low elevation. No data on the temporal pattern of storm rainfalls over the headwaters of the catchments or in the mountainous areas were available. It was apparent that for accurate analysis of storms over the catchment, it would be necessary to increase substantially this pluviograph network. Assistance from the H.E.C. and the newly formed hydro-meteorologic section of the Weather Bureau was sought in order to carry this out. Four long term pluviographs designed by H.E.C. and using Leupold-Stevens float recorders were loaned by that Commission. These were installed in the uninhabited areas of the eastern section of the catchment mentioned above. In addition to these four gauges the H.E.C. also installed a similar type of recorder on the Liffey River (elevation 3,600 ft.) in connection with their investigations on the Great Lake North Scheme.

The clocks on these instruments required rewinding once every six weeks and the staff of the Forestry Commission and Department of Civil Aviation assisted the Authority in this regard. The time scale was 2.4 inches chart travel per day and the rainfall scale was one inch pen movement for every inch of precipitation.

In addition to these long term pluviographs, the Hydro-Meteorologic Branch of the Weather Bureau installed, at the request of the Authority, a further nine short term pluviographs (weekly chart change) at the following locations:-

(a) Upper Blessington
(b) St. Mary's
(c) Lewis Hill
(d) Lake Leake
(e) "Ellinthorpe", Ross
(f) "Connorville", Cressy
(g) Steppes
(h) Golden Valley
(i) Frankford West

As a result of these additional pluviographs, the average areal distribution was increased to one pluviograph per 85 sq. miles for the North Esk catchment and one per 250 sq. miles for the South Esk catchment, which was found to be more than satisfactory.

As well as increasing the pluviograph cover over the catchment, each official daily read rainfall observer was asked by circular to note the depths and durations of heavy rainfall. Twenty-four of these observers expressed their willingness to co-operate with the Authority in this regard. These records were used to augment the pluviograph records over the catchment.

(c) Synoptic and Climatic Data.

Synoptic and climatic data were available from a number of official stations maintained by the Commonwealth Meteorological Bureau. In all there are 24 of these stations in Tasmania, but for the study of air mass movement for the flood of 1929 during the maximization studies, reference was also made to the station on Gabo Island near Victoria and to the records of Lord Howe Island, which lay in the path of the moist air stream.

C1.5 Flood Occurrences During Investigation

Since the inauguration of this data collection programme, only three floods of any magnitude have occurred in the South Esk River. These were the floods of May, 1958 (peak discharge 21,000 cusecs) and those of August, 1958 (42,800 cusecs) and October, 1958 (48,200 cusecs). It is pleasing to note, however, that the data collection on the Esk catchments will be continued in the future by the H.E.C. in collaboration with the Hydrometeorologic section of the Commonwealth Bureau of Meteorology. It is to be hoped that this will continue at least until such time as a major flood occurs.

By reason of the fact that the North Esk River is more subject to flooding from the westerly weather than the South Esk, more floods were recorded in this stream, although records were limited by the extremely dry year of 1957. The floods which have been observed in this river since the installation of the Bristol Recorder at Corra Lynn, on 7th June, 1957, and which included a major flood in May, 1958, are as follows:-

Date	Peak Discharge
4th May, 1958	4,100 cusecs
25th May, 1958	10,000 "
6th June, 1958	2,780 "
21st June, 1958	1,950 "
25th July, 1958	3,920 "
8th August, 1958	3,620 "
17th August, 1958	4,720 "
7th October, 1958	3,320 "
15th October, 1958	3,620 "
3rd April, 1959	1,970 "

C2 RAINFALL LOSSES

C2.1 Theory

Before rainfall reaches a river system in the form of surface run-off, several losses occur. Portion of these losses are permanent, due to evaporation and transpiration. The remainder are of a temporary nature, such as percolation through the porous soil which initially replenishes the ground-water reservoir and some time later re-appears in the stream as dry weather flow.

When rain starts falling on a more or less pervious area there is an initial period during which:-

(i) The rainfall is intercepted by buildings, trees, shrubs, grasses or other objects and thus prevented from reaching the ground.

(ii) It infiltrates into the ground to satisfy initially the deficiency of moisture in the soil and then continues downwards under gravity to replenish the ground-water table.

(iii) It finds its way by overland flow into the innumerable small and large depressions, filling them to their overflow level.

It is not until after this initial period, when interception and depression storage has been largely filled, that any substantial surface run-off will occur. The volume of rain that falls during this period is termed "initial loss". It may be defined as the quantity of rain that occurs under specific conditions without producing significant run-off. The magnitude of "initial loss" will depend on the conditions of a catchment at the time of the storm, being high after a dry period when the field moisture deficiancy is at its peak.

After initial loss has been satisfied surface run-off will result, provided the rate of rainfall is greater than the maximum rate at which water can infiltrate into a given soil. During this period the rainfall that is lost to surface-run-off will approximate the infiltration capacity which is affected by many factors.

The majority of these factors tend to cause the infiltration capacity of the soil to decrease as the soil is wetted. However, owing to the many processes involved and the variation in infiltration capacity over different areas of the catchment, it is impossible to determine accurately the variation of infiltration with time on large catchments. It is usual to calculate only an average rate of loss over the catchment for the period of excess rainfall. This is achieved by selecting a rate of loss, after initial loss has been satisfied, such that the volume of excess rainfall is equal to the volume of surface run-off.

C2.2 Initial Loss

Several different approaches are used to determine the quantity of initial loss but at present they can be considered as giving approximate estimates only.

One method is to assume that all rainfall, that fell before surface run-off commenced, is initial loss. This assumption may be satisfactory for uniform rain over the catchment but in cases where the areal distribution of storm rainfall results in heavy rain in the mountainous areas of the headwaters with only light falls near the outlet, as is the case for the Esk catchments, surface run-off does not pass the gauging station for a considerable time after the start of rain. If this method were used there would appear to be a greater volume of run-off than rainfall, which is obviously impossible. This approach was not applicable to the Esk catchments.

A second method used by the investigating engineer (D.Foster) was to determine the average rainfall over the catchment for two types of storms - those which produce no significant surface run-off and those which result in only small amounts of surface run-off. Initial loss would then lie somewhere between the two. If some index of catchment saturation could be determined, a plot of the curve separating these two types of storms against the catchment wetness index would give an estimate of initial loss for conditions existing at the start of rain. It was felt that for the Launceston investigation that this was a possible approach, using ground-water flow as an index of catchment saturation. Seven storms were studied. Results are plotted on Fig. 7, but the investigation was inconclusive, mainly because of the difficulty of separating surface run-off from increases in ground-water flow and partly because ground-water is not an absolute index for catchment conditions at the start of rain. Light precipitation preceding a storm would reduce the field moisture deficiency and partly fill depression storage, thus reducing the magnitude of initial loss without increasing the ground-water flow.

The investigation did, however, indicate the order of magnitude of initial loss. By an assessment of the wetness of the catchment at the start of the storm a value was estimated for the earlier storms analysed.

Towards the conclusion of the hydrologic studies an additional method was tried. A Water Research Foundation Research Fellow (D.N.Body) developed a digital computor programme (C1) for derivation of unitgraphs, embodying a series of derivations from the same storm using gradually increasing assumptions of initial loss. Each of the derived unitgraphs was then applied to the original rainfall values to reproduce the hydrographs and the computor determined the sum of the squares of the residuals between the reproduced hydrograph and the actual hydrograph. As this figure is an index of the fit of the curve to the data, the unitgraph which gives the smallest sum of squares of the residual should be the best, and the corresponding initial loss can be read off from the results.

Ref. (C1) - D.N. Body "Derivation of Unitgraphs using a Digital Computor" - Bull. No.4 Water Research Foundation of Australia 1959.

This method, because of the advantage of having a computor to do the calculation, was adopted for the analysis of the later storms. As the results of this method indicated that for the Esk catchments a wide range of initial loss would have only a minor effect on the peak ordinates of the subsequent unitgraph, it was not considered worthwhile to modify the earlier results. For instance, for the flood in the North Esk of 2nd-3rd May, 1958, if an initial loss of 0.56 inches was assumed, the peak ordinate of the 2 hour unitgraph would be 8,370 cusecs as compared to 8,350 cusecs for an initial loss of 0.90 inches, and 8,150 cusecs for an initial loss of 1.45 inches.

Another approach to the derivation of initial loss was proposed by the investigating engineer (D.N.Foster) towards the end of the investigation as a result of a study of loss rates for the May flood in the North Esk. This flood was caused by three separate bursts of rain, each separated by a period of approximately 12 hours, and resulted in a triple peaked hydrograph. Determination of the average loss rates for each of these rainfall bursts gave values of 26 pts./hour for the first period of rainfall, 6.2 pts./hour for the second period and 6.1 pts./hour for the third period. These results indicate that loss rates tend to reach a constant value shortly after surface run-off commences and that the apparent variation of loss rates obtained from the analysis of different storms is mainly the result of an incorrect assessment of initial loss. If this theory is valid, initial loss could be calculated by first determining the true average loss rate on a saturated catchment from the study of multi-peaked hydrographs or hydrographs resulting from rain over a wet catchment and the initial loss determined for any storm such that this value when combined with the derived loss rate will result in excess rain equal to the volume of surface run-off indicated by the hydrograph. Insufficient time was available to investigate fully or utilize this approach on the Launceston project, but a full discussion of the topic will be found in the thesis by D.N. Foster (C2).

C2.3 Loss Rates

Loss rates were determined for each flood event for which a unitgraph was derived as discussed later in this section of the report and are summarized in tables 2 and 3.

The method of deriving the loss rates was as follows:-

Ref. (C2) - D.N.Foster - Thesis for Degree of Master of Engineering copy available in Library of the University of New South Wales 1959.

(i) From a total storm isohyetal map the average total rainfall over the whole area of the catchment was obtained.

(ii) From a study of the relevant pluviograph charts the average temporal pattern over the catchment was estimated.

(iii) This temporal pattern was then applied to the average gross rainfall to obtain the rate of rainfall over each unit period assuming uniform rainfall over the catchment. Unit periods adopted were 6 hours for the S.Esk and 2 hours for the N.Esk.

(iv) After deducting initial loss, an average loss rate was applied to the rainfall pattern such that the difference between the gross rainfall and gross loss equalled the total excess rainfall as indicated by the hydrograph.

For the earlier storms analysed, this was carried out by hand. As the result of a special programme prepared by D.N.Body(C4) for the derivation of unitgraphs by the use of U.T.E.C.O.M. digital computer (C3) it was possible for the latter storm to derive these rates directly when computing the unitgraph.

It is realised that this method of deriving loss rates could be refined to determine an average rate of loss which, when applied to the gross rainfall at each station over the catchment, resulted in excess rainfall, as determined by an isohyetal map, equal to the volume of surface run-off. This method would give a more accurate estimate with slightly higher values for the average loss rate than that determined by the above method, but in this case such refinement was not adopted. This decision may be defended on the following grounds:-

(i) The exact quantity of initial loss is difficult to determine accurately so that a high degree of refinement in computing loss rates is not justified.

(ii) The areal variation of the infiltration capacity on different zones of the catchment was unknown, which is another reason why the simpler analysis is justified.

(iii) The areal pattern of rainfall for all major storms over the catchment were very similar with heavy rainfall on the surrounding mountains with a rain shadow on the plains, so that any errors arising from this procedure would tend to be compensating, when applying the loss rates to a design storm.

Ref. (C3) - U.T.E.C.O.M. - digital computer of The University of New South Wales.

C3 UNIT HYDROGRAPHS

C3.0 Introduction

In 1932 Le Roy K. Sherman (C4) first introduced his now almost universally accepted theory of the unit hydrograph. This concept of surface run-off is one of the most important contributions ever made to the science of hydrology. It provides a most useful tool for the determination of the hydrograph of surface run-off that will result from any given storm. The unit hydrograph is defined as the hydrograph of surface run-off resulting from a unit quantity of excess rain which occurs at a uniform rate at all points on the catchment throughout a unit period of time.

C3.1 Basic Data Requirements

The basic data requirements for derivation of satisfactory unitgraphs are:-

(i) A hydrograph of surface run-off at the outlet of the catchment - this requires a reliable record of stream level with time for the duration of the flood, from which discharges can be determined from a rating curve of the station.

(ii) The average depth of rainfall which fell on the catchment - this requires a sufficient number of daily rainfall records to define adequately the areal distribution of the rainfall.

(iii) A sufficient number of pluviographs to establish the temporal variations in precipitation on the catchment.

C3.2 Methods of Derivation

The order of accuracy of the unitgraph procedure and the deficiences in the data used for its derivation and application should always be borne in mind. The relative accuracy which is required of a unitgraph depends upon its use. If it is to be used for stage predictions in large floods, then extreme accuracy is not required, because a large variation in discharge makes little stage variation at high stages in normal river profiles. The design of a dam spillway, however, requires as accurate prediction of peak discharge as possible. Thus it can be seen that the purpose of the investigation will have some bearing on the method used and the trouble taken in the derivation of the unitgraph.

Ref. (C4) L.K. Sherman "Stream Flow from Rainfall by the Unit-graph Method" Eng. News - Record Vol. 108, 1932.

Derivation can be attempted in two ways, depending upon the basic data accepted. The first of these is to select a hydrograph resulting from a unit period storm on the catchment and obtain the unitgraph by proportional adjustment of the ordinates. The second method is to analyse a complex hydrograph produced by a multi-period storm, separating it into the various unit hydrographs for each unit period of rainfall.

Derivation of the unitgraph from a unit period storm follows directly from unitgraph theory. It has the advantages over the multi-period storm in that the derivation is extremely simple, and also the unitgraph can be defined by points separated by any desired period of time. As against these advantages, the method has several disadvantages, which are partly offset when the derivation is made on a multi-period storm. These disadvantages are as follows:-

(i) A storm of approximately unit length is required which severely limits the flood events that are suitable.

(ii) It is unlikely that a storm having a duration nearly equal to the chosen unit period would produce a large flood.

(iii) When the unitgraph is derived from a single isolated event, no account is taken of the effect of concurrent flow. In actual practice this may have some effect. As the derived unitgraph is likely to be applied to a multi-period storm to obtain a design flood, it would be preferable to use a similar flood event for the derivation.

Until recent years, the biggest disadvantage of the multi-period method was the time involved for the calculations, but since a programme has been written for the derivation of unitgraphs on a digital computer the time factor has been reduced considerably. For the Launceston project all unitgraphs were derived on U.T.E.C.O.M. from multi-period storms by the use of the least squares method.

The relationship between the excess rainfall pattern, hydrograph and unitgraph ordinates can readily be expressed for a multi-period storm in the form of a set of linear algebraic equations. For example if we consider a two period storm in which the rainfall over the first period was P_1 and over the second period P_2 and if the unitgraph base length was equal to 6 periods, then unitgraph theory would give the following relationships:-

$$\begin{aligned} P_1y_1 &= Y_1 \\ P_1y_2 + P_2y_1 &= Y_2 \\ P_1y_3 + P_2y_2 &= Y_3 \\ P_1y_4 + P_2y_3 &= Y_4 \\ P_1y_5 + P_2y_4 &= Y_5 \\ P_2y_5 &= Y_6 \end{aligned}$$

where y_1 is the unitgraph ordinate one unit period after the start of rise, y_2 the unitgraph ordinate two unit periods after the start of rise etc. and $Y_1 Y_2$ etc. are the corresponding hydrograph ordinates.

If the unitgraph is to be determined for this hypothetical storm, the rainfall P_1 and P_2 are known and the values of the ordinates of the resultant hydrograph Y_1 Y_2 etc. can be determined from analysis of the hydrograph. It is then necessary to calculate the five values of $y_1 y_2$ etc. fixing the unitgraph from the six experimental equations. As there are five values of y to be found from six equations, and because of the inaccuracies of the data, there is no exact solution. The best solution is that which when substituted in the left-hand side will give the smallest absolute deviation from the known values of the right-hand side. This is known as the least squares solution.

C3.3 Steps in the Derivation of Unitgraphs for Esk Rivers

C3.31 Selection of Suitable Storms for Analysis. Suitable flood hydrographs were selected, bearing in mind the following requirements:-

(a) One of the basic assumptions of unitgraph theory is that the areal distribution of the storm precipitation is uniform. This ideal condition is rarely, if ever, encountered in nature and in the case of a catchment where orographic effects are marked (as for the Esk catchment) there is a large variation in the volume of precipitation over different portions of the catchment.

One way of allowing for areal variation of rainfall is to divide the catchment into zones such that within each zone reasonable areal uniformity is obtained. Another way is to select for analysis storms whose areal variation of rainfall is similar to the "design storm". This method was adopted in the case of Launceston, i.e. preference was given to storms resembling in areal distribution the April 1929 storm. As the derived unitgraphs were used to reproduce the 1929 flood hydrograph and an estimated "maximum probable flood" based on the 1929 flood, the error arising from the departure of reality from idealized unitgraph theory thus became unimportant.

(b) The total volume of run-off from the catchment should be greater than half an inch.

C3.32 Separation of Base Flow. To determine the surface runoff hydrograph, it is necessary to deduct the base ground-water flow from the surface runoff. This involved fixing on the hydrograph the point of start of rise due to flood runoff and the point of cessation of such flood flow, and in this study these two points were joined by a straight line, the ordinates of the hydrograph above it being deemed to be the hydrograph of surface discharge.

The first point was selected by eye, as the rise is sharp.

The fixing of the second point is more difficult, as it is not obvious from an inspection of the hydrograph.

Two methods were used, depending on circumstances. The usual method was to plot a graph of the logarithm of the discharge for the recession limb against time. Both the recession curve of the hydrograph and the ground-water depletion curve can be expressed in the form:-

$$q_t = q_o k^t$$

where q_t is the discharge after time t q_o is the initial discharge and k is a constant of different magnitudes for the recession curve and the ground-water depletion curve.

The graph of log q against t will plot approximately as two straight lines of different slope, the intersection of which will represent the end of surface run-off.

If the falling limb of the hydrograph were affected by surface run-off which resulted from a small burst of rain occurring sometime after the storm being analysed, this must be separated out and the above method could not be used. The procedure in this case was as follows:-

(a) The true recession curve, which would have resulted if no rain had fallen after the main storm, was drawn by using the equation $q_t = q_o k^t$ with a value of the recession constant k as determined from previous hydrographs.

(b) A master ground-water depletion curve (Fig.8) was prepared from records of streamflow in the river during period of dry weather.

(c) At a point where it was certain that flow was purely ground-water, the recession curve of ground-water was extended backwards by means of the master curve to intersect the recession curve of the hydrograph determined in (a). This point was taken as the end of surface run-off and the base flow separated by joining this to the start of rise by a straight line.

C3.33 Total Storm Rainfall. The average depth of gross rainfall over the catchment was determined by drawing an isohyetal map for the storm and averaging the rainfall between the isohyets.

C3.34 Temporal Pattern of Rainfall. For all storms for which unitgraphs were derived on the Esk catchments it was found that -

(i) Duration of storm runoff was approximately constant on all parts of the catchment.

(ii) The mass curves of rainfall at all pluviographs when plotted as percentage of total storm rainfall against time after the start of rain, closely approximated the same form.

For these reasons it was possible to determine the average temporal pattern of rainfall over the catchment by plotting the mass curves of each pluviograph in the form of a percentage of total storm rainfall on that pluviograph against time after the start of rain and sketching in an average curve.

From the average gross rainfall and average temporal pattern derived in this manner an average mass curve of rainfall for the storm was drawn.

C3.35 Selection of Unit Period. Suitable unit periods were chosen after consideration of the following factors:-

(a) The unit period should be such that the basic assumption of unitgraph theory of uniform rain over each unit period is approximately true.

(b) The unit period chosen should be equal to or less than 1/3 to 1/4 the period of rise of the hydrograph, so that a sufficient number of unitgraph ordinates are calculated to define adequately the shape and peak of the curve.

(c) The unit period, provided it satisfies the above conditions, should be as large as possible to reduce the amount of computation required. (This is less important, if a digital computer is used).

The unit periods adopted were 6 hours for the S.Esk and 2 hours for the N.Esk.

C3.36 Excess Rain Hyetograph. From the average mass curves the rainfall hyetograph for each unit period was determined.

The appropriate initial loss and loss rate was deducted from the rainfall over each unit period.

C3.37 Calculation of Unit Hydrograph Ordinates. The excess rainfall over each unit period and the surface run-off hydrograph ordinates at the end of time intervals separated by a unit period after the start of rise were taken off and punched into cards in accord with the programme for the derivation of unit hydrographs on a digital computer.

This information was fed into the computer which calculated a unit hydrograph by the least squares solution.

The programme for the derivation of unitgraphs on a digital computer was revised in 1958 and if desired the gross rainfall only need to be fed into the machine. The computer will then calculate a series of unitgraphs for different values of initial loss and will automatically calculate the appropriate loss rate.

The best unitgraph fitting the data can then be selected and the appropriate initial loss and loss rates would be those shown by the computer for this graph. For all the later storms analysed on the Esk catchment this method was adopted.

C3.38 Discussion of Results. Unit hydrographs were derived for the South Esk catchment above Launceston from the storms of September, 1952, 2nd-3rd May, 1956, 22nd-24th May, 1956, May 1958 and August, 1958. The derived unitgraphs are shown on Fig. 9 and have been summarised in Table 4.

In addition to these storms the data for the floods of February, 1956 and 1957 were studied, but the areal distribution of rainfall was found to be so uneven that they were rejected as being unsuitable for unitgraph derivation.

As the volume of flood waters in the North Esk river are only a small proportion of the total discharge in the Tamar River during a major flood (about 10 per cent) it will be appreciated that a high degree of accuracy in fixing flood flows in this river is not required for the Launceston investigation. For this reason unitgraphs were derived for the North Esk at Corra Lynn from two storms only, those of 2nd-3rd May, 1958 and 22nd-26th May, 1958. As these unitgraphs were in reasonably close agreement with each other later storms were not analysed. The derived unitgraphs are shown on Fig. 10 and have been summarised in table 5.

The average unitgraphs to be used for flood estimation on the Esk catchments were obtained by averaging the height and time of peak of the derived unitgraphs, giving due weight to those considered most reliable, and then sketching in a mean graph having an area equal to one inch of run-off and resembling the individual graphs as much as possible.

The mean unitgraph for the South and North Esk catchments is shown in Figs. 9 and 10.

C3.39 Application of Unit Hydrographs. To predict a flood from a design storm by the use of the unitgraph, appropriate loss rates and initial loss are deducted from the gross rainfall pattern and the storm precipitation expressed in terms of an excess rainfall hyetograph with periods corresponding to the unit period of the unitgraph. When this is done the building up of the hydrograph is extremely simple. The process can be expressed by a series of equations as shown below in which P_n is the excess rainfall in inches during the n^{th} unit period, and X_n and Y_n are the unitgraph and hydrograph ordinates in cusecs at the end of the n^{th} unit period after the start of rise.

$$\begin{aligned}
P_1X_1 &= Y_1 \ldots\ldots\ldots\ldots (1)\\
P_1X_2 + P_2X_1 &= Y_2 \ldots\ldots\ldots\ldots (2)\\
P_nX_n + P_2X_{(n-1)}+ &= Y_n \ldots\ldots\ldots\ldots (n)\\
P_rX_a &= Y_b \ldots\ldots\ldots\ldots (a + (r-1)\ )
\end{aligned}$$

where r = no. of rainfall periods

a = no. of unitgraph periods

b = no. of hydrograph periods

$= a + (r-1)$

C4 STORM MAXIMISATION

C4.0 General

The various basic methods for the estimation of maximum probable precipitation can be divided into three main types, viz:-

(i) Theoretical computations, based on air mass analysis, storm tracks and basin topography.

(ii) Storm maximisation for wind and moisture charge of the air mass and transposition to catchment under study making due allowance for location and elevation.

(iii) Thunderstorm rainfall, based on envelopment of depth-area-duration values from storms recorded on small areas.

C4.01 Theoretical Computations. In its most general terms the theoretical method for the estimation of maximum possible precipitation states that the volume of precipitation over an area in a given time during the maximum possible storm is equal to the product of the total number of unit columns entering the area and the maximum amount of effective precipitation which can be removed from each column.

The depth of precipitation is dependent on the inflow direction, the inflow velocity, the amount of moisture available and the duration of the storm.

The calculations involved in such a theoretical study are complex and laborious, and it was considered that the expense involved was not justified in this investigation.

C4.02 Maximization and Transposition of Recorded Storms. The simplest method of estimating the maximum probable precipitation rates isthat which involves transposition to the catchment of several selected major storms which have been recorded within the meteorologically homogeneous area in which the catchment lies and maximizing for air moisture charge and wind. It is desirable that several observed storms be considered, so that the adjusted storm ultimately used approaches the physical limit of efficiency in converting water vapor into precipitation for all pertinent durations.

The various steps in the procedure are as follows:-

(i) Refer to the Meteorological Branch records of past major storms over the meteorologically homogeneous zone in which the catchment under study is situated, and select those which warrant further investigation.

(ii) Plot for each of these storms the isohyetal maps of precipitation for various increments of time up to the total duration of the storm, and hence prepare graphs or tabulations showing the average precipitation depth over a progressively increasing area from the focus of the storm for various increments of time.

(iii) From these tabulations prepare a list of the major storms which gave the heaviest precipitation depths for various durations over areas equal to that of the catchment under study.

(iv) Study these storms from the point of view of season of occurrence, isohyetal pattern etc. and from a general knowledge of the history of past great storms in the region, select those which are of the type likely to be most critical, if they had occurred on the catchment in question.

(v) In consultation with hydrometeorological experts, the engineer must now decide from a study of meteorological, topographic and geographic factors which of these storms could reasonably have occurred on the catchment.

(vi) Obviously if the storm to be transposed actually occurred on the coast, and the catchment is on a tableland, some adjustment of the depth-area-duration graphs of that storm must be made to allow for the effects of topography, and from consideration of such matters the hydrometeorologist, in consultation with the engineer, must decide on a "transposition adjustment factor" for each storm.

(vii) The depth-area-duration tabulations of (ii) above must now be amended for each storm by multiplying by the "transposition adjustment factor".

(viii) The next question is the relation of the shape of the catchment to the shape of the isohyetals of the transposed storm and the further adjustment is carried out by multiplying by a "basin shape factor".

(ix) The designer now has depth-duration tabulations for a few storms, adjusted for topographic, geographic and shape factors, representing the average depth of precipitation if the transposed storm had centred over the catchment under study.

(x) Index stations are selected as representative of the storm being studied and dew point data collected for periods of a few days preceding and following the storm. This data is plotted against time and the values for various durations of persistence extracted.

(xi) Maximum dew points for various durations of persistence are determined for the index stations from a detailed analysis of the long term station records.

(xii) The dew point data is adjusted to a common level, generally sea level or the 1000 mb. pressure level and the corresponding vertical water content from this level to the 200 mb. level is computed.

(xiii) Making due allowance for inflow barriers to the catchment the depth-duration curves are increased in the ratio of effective precipitable water available at maximum dew point to effective precipitable water in the storm which actually occurred.

(xiv) If it is assumed that for estimating the maximum probable precipitation the maximum inflow wind velocities will occur concurrently with maximum values of dew point, the precipitation depth should also be increased in the ratio of maximum wind velocity that may occur over the catchment to the actual wind velocity during the storm under study. It is considered by some authorities, however, that any alterations in wind velocities must be reflected in the pressure systems associated with the storm and which in turn would affect both the duration and general extent of the storm. For this reason and also because long term records of upper air winds would be required, since surface winds cannot be considered as typical, maximization is often carried out on the basis of dew point only, as was done for the Launceston case.

The above procedure can of course be applied to storms which occurred on the catchment, in question, as well as to those which must be transposed from other catchments.

C4.03 Thunderstorm Rainfall. The United States Hydrometeorological Section has concluded that, except for special regions of high orographic or synoptic influence, the convective thunderstorm mechanism is the most rain-productive condition for areas up to 500 sq. miles and for durations up to about 12 hours.

J. Walpole (C5) transposed to the Australian region depth-area-duration data for thunderstorms in the U.S.A. with appropriate modifications to give generalised estimates of the maximum possible rainfall that can occur anywhere in Australia over areas varying from 10 to 500 sq. miles and times ranging from 1 to 24 hours.

Since the Tamar catchment has an area of 3,767 sq. miles, thunderstorm rain is not the cause of major floods at Launceston and for this investigation the thunderstorm model was not studied.

C4.1 Selection of Storms for Analysis

Due to the major influence of the topographic features of the Esk Catchments on a storm mechanism it was considered that transposition of Tasmanian storms would be unsound. It was also felt

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Ref. (C5) J. Walpole. Maximum Possible Rainfall in Australia - Thunderstorm Model - Internal Report, Meteorological Branch, Australia.

that transposition of storms from Southern Victoria would not be reliable because of the influence of Bass Straight on rainfalls and storm mechanisms in Tasmania. For these reasons analysis was restricted to storms actually recorded on the catchment.

Unfortunately, this limited the storm events available for analysis, as only one major flood has been recorded on the catchment during the present century. This was the flood of April, 1929, and maximum precipitation estimates have been limited to an analysis of this storm. For this reason the estimate of the maximum probable flood may possibly be low. A higher figure may have been obtained if data for the major floods of 1852, 1863 and 1893 had been available, and these storms could also have been maximised. However, it was felt that an estimate based on the 1929 storm only is sufficiently accurate for the design of flood protection works, where precautionary methods can prevent any loss of life as a result of the levees being overtopped.

In addition to the storm of April, 1929, that of May 1956 was also studied in considerable detail, but it was finally rejected, as it did not give depths of rainfall which after maximization would be critical for any duration.

C4.2 The Storm of April, 1929.

C4.21 Description

In 1929 weather observations were almost wholly made at ground level. This limits the investigation of the storm to its aspect at sea level. However, from its surface history some general remarks may be made of its upper structure. These comments are contributed by the Deputy Director of Meteorology, Hobart (W. Shields) and his staff.

The isobaric charts for the period from 30th March to 6th April are shown on Fig. 11. These reveal the following:-

"(a) The storm developed from a tropical dip.

(b) Development was rapid and the storm was intense when it reached Bass Strait.

(c) The storm took an unusual path in moving down the west coast.

(d) It also slowed in passing across Bass Strait.

The speed of 7 knots due south across Bass Strait between 9 a.m. on the 4th and 9 a.m. on the 5th enabled the wind at 2,000 - 3,000 ft. in eastern Bass Strait to blow from the north-east with a speed of at least 50 knots for 21 hours between 3 a.m. on the 4th and midnight on the 4th, (the speed exceeded 25 knots for 36 hours between 3 p.m. on the 3rd and 3 a.m. on the 5th). The rapid development on the 3rd lead to the formation (early on the 4th) of the moist north-easterly jet (characteristic of these rain situations) which not only remained saturated on the surface with the high moisture content of its source, the Tasman Sea, but because of the organization of wide-spread up-currents, was distributing moisture to the very highest layers. This jet reached almost an extreme speed on the 4th and its long duration no doubt lead to flood rainfall.

It would seem that the formation of the storm was not dissimilar to that which occurred in April, 1957. In this respect the storm was typical of all those that develop out of troughs or tropical dips over eastern Australia.

The unusual feature was its history after 9 a.m. on the 4th. The normal path is south-eastward. This one slowed and moved southward. The reason for this appears to be in the movement during the 4th of a depression eastward along latitude 40 from south of West Australia to be located just to the west of the storm at 9 a.m. on the 5th. The mechanism of surface pressure variation associated with the western depression masked or took control of that associated with the eastern storm which became absorbed in the former, the whole moving southeast on the 5th (but not crossing Tasmania).

The extreme depth and areal variation of the rainfall pattern are due to the following:-

(i) Moisture - During the days preceding the development, Eastern Australia was covered by weak low pressure systems, a situation in which high dew points could have been the case - though there is no data on this point. The surface layers of air which passed over the State on the 4th were probably near Lord Howe Island 24 hours earlier, and it appears that air over this area of the Tasman Sea was moist. Dew point values recorded in Northern Tasmania on the 4th and 5th are not exceptional, however, being probably well within the once in one hundred year chance of occurrence.

(ii) Speed of inflow - this was very high, and the current was broad. North-east gales in eastern Bass Strait are rare. The assumption is not that heavy rainfall parallels high surface wind; there is no constant relationship between surface wind in a storm and the vertical velocity, let alone with rainfall. It does not follow that this case affords an example of optimum inflow.

(iii) Duration - Perhaps the greatest contribution to the flood rainfall arose from the comparatively long time the storm mechanism was active over the area. The duration is a factor requiring maximization as much as the rate of rainfall.

(iv) Convective Cells - The extremely heavy falls for the 24 hours to 9 a.m. on the 4th centred around Riana indicate the operation of other factors as well as topography as rain producers in this storm. This area was again the locale of very heavy rain in the next 24 hours. Cranbrook in the East was also the centre of high rainfall. Both circumstances are cases of extreme convection. The former, because of its stationarity over the two days, may have been produced by frictional convergences on that portion of the North Coast while the latter was possibly a random convectional cell. Case synoptic studies may permit allowances to be made for the former convectional type in storm rainfall; however, the smaller cells appear to be a feature and a random event of cyclones."

The isohyetal maps of the storm are shown in Fig. 12 to 15.

<u>C4.22 Maximization Calculations</u>

<u>C4.221 Mass Curves</u>. As it was considered that any alteration in inflow wind speeds must be reflected in the pressure systems associated with the storm which in turn affect both the duration and general extent of the storm and also because no upper air wind data was available in 1929, maximization was carried out on the basis of dew point data only.

In addition, as the areal variation of depths of precipitation in any storm over the Esk catchments is greatly influenced by the mountain ranges, it was considered that transposition of the storm centre within the catchment to produce maximum runoff would not be wise, and the storm pattern after maximization was assumed to be the same as before.

As there was no pluviographic data for the 1929 storm, mass curves of rainfall were synthesized over the catchment from a study of the newspaper reports on the storm combined with the daily rainfall readings at the official stations. From this study it was concluded that the temporal pattern of the storm could be treated in three zones, viz:-

(i) The East Coast Ranges.

Heavy rain commenced at 7 p.m. on the 3rd April, and continued continuously at a uniform rate until midnight on the 4th when the intensity decreased slightly, the rain ceasing about 2 p.m. on the 5th.

(ii) The North Coast and Plains and North Esk Catchment.

Heavy rain commenced falling at 6 p.m. on 3rd April, an hour earlier than the east coast ranges and continued continuously until 10 a.m. on the 4th when it eased slightly for 6 hours. After 4 p.m. the rain intensity increased and continued at a uniform rate until 8 a.m. on the 5th, when it eased, the rain ceasing at 9 a.m. on the 6th April.

(iii) The Western Tiers.

A study of newspaper reports indicated that the rainfall occurred in two main bursts. Heavy rain commenced at 9 a.m. on the 3rd April, approximately 10 hours earlier than on the east coast ranges and continued at a high intensity until 11 p.m. when it eased to a drizzle until midday on the 4th when a cloudburst was experienced and heavy rain continued until midnight after which the intensity eased, the rain ceasing at 7 a.m. on the 6th.

The average mass curve for the total South Esk catchment was determined by averaging the temporal pattern of rainfall over the three zones weighted in accordance with their areas. This curve was modified slightly where necessary so as to pass through the known accumulated rainfall depths at 9 a.m. on the 4th and 5th April, as determined from isohyetal maps for the storm, shown in Figs. 12 to 15.

Synthesized mass curves for the three zones and the average mass curve for the South Esk catchment are shown on Figs.16 and 22.

C4.222 Dew Point Analysis and Maximized Storm. Dew points and minimum temperatures at seven stations on the catchment or in the path of the moist air stream were analysed to determine the maximum 24 hour persisting dew points associated with the air mass.

The absolute maximum dew points which could persist in Australia for periods of 24 hours have been estimated by Walpole (C5). No work has as yet been published in Australia on the maximum dew points that can persist for periods greater than 24 hours and for the purpose of this study it has been assumed that an adjustment factor based on 24 hour persisting dew points would also be representative of the longer durations.

The factor by which the depths of precipitation in the storm were to be adjusted was determined by calculating at each of the stations the ratio of the effective precipitable water at the absolute 24 hour maximum persisting dew point as given in Walpole's paper to the effective precipitable water at the maximum 24 hour persisting dew point in the storm and selecting the ratio most applicable to the catchment. These values are shown in tabulation hereunder.

Table C1

Maximization Factors April, 1929.

Station Name	Station Level	April 1929		Max. Values		Adjustment Factor
		24 hr. persisting Dew Point Reduced to 1000 mbs. level	Effective Precipitable Water	24 hr. Persisting Dew Point Reduced to 1000 mbs. level.	Effective Precipitable Water	
Eddystone Rt.	0	58.3°	1.27	65°	1.74	1.37
Launceston	266	60.6°	1.38	65°	1.74	1.26
St.Helens	0	57.5°	1.22	65°	1.74	1.42
Swansea	25	56.9°	1.18	65°	1.74	1.47
Low Head	0	57.5°	1.22	65°	1.74	1.42
Gabo Island	50	63.4°	1.67	70°	2.33	1.40
Lord Howe Is.	0	64.4°	1.73	74°	2.79	1.62

For the maximization of the April, 1929 storm a factor of 1.42 was selected. Implicit in this maximization is the assumption that the 1929 storm could have occurred in summer. In fact the largest recorded flood occurred in December 1863.

The mass curves for the North and South Esk catchments in 1929 adjusted for maximum dew point are shown in Figs 25 and 26.

C5 FLOOD ESTIMATION

C5.1 Flood Estimates for South Esk River 1929 Flood

No measurement of the discharge was made in 1929. The gauge height at Duck Reach was read until 8.0 p.m. on 5th April. The peak height was estimated by trash line surveys after the flood.

As the peak discharge was required to verify the model, estimates were made by three methods, as follows:-

(i) Extrapolation of the Rating curve.
(ii) Slope Area calculations.
(iii) Application of the 1929 excess rainfall to the unitgraph.

As a result of these studies a peak discharge of 150,000 cusecs was adopted.

C5.12 Extrapolation of Rating Curve

C5.120 Introduction. After the flood has receded, a trash line survey indicated that the peak gauge reading at Duck Reach was 44 ft. The station has been rated for a gauge reading of 26.5 ft. corresponding to a discharge of 45,500 cusecs. In order to estimate the 1929 peak flow this rating curve was extrapolated by four methods viz:-

(i) Extension of logarithmic paper.
(ii) Extension by studies of areas and velocities.
(iii) Application of Chezy formula as suggested by Stephens - the $Q-A\sqrt{R}$ Method. (C6).
(iv) Extension by plotting average discharge per foot width against the hydraulic mean radius - R - Q' method.

Ref. (C6) - Stephens, J.C. "A method of Estimating Stream Discharge from a Limited Number of Gaugings" - Eng. News Record July, 18, 1907.

It must be emphasized that any extrapolation is open to possible serious error, and the rating curve should be developed to a stage where differences between successive rates of change in discharge has become fairly constant with respect to change in stage. The conditions of the channel most favourable for an accurate extension of the rating curve consist of well defined rapids or ripples below the gauge at all stages and a uniform increase of channel cross section as the stage increases, with no abrupt changes in area or addition of overflow channels. The station at Duck Reach satisfies these conditions.

In carrying out this work, consideration was given to the fact that wave action might have resulted in gauge readings as established by trash line surveys being too high. Hence, for each case two sets of calculations were carried out, one being based on the reading as recorded and one being based on a gauge height of 42.5 ft., assuming that wave action would cause an error of 18 inches.

Also, because the cross section at the Duck Reach suspension bridge was extremely irregular due to the large boulders constituting the bed and boundary of the stream, two estimates of discharge for each gauge reading were calculated for methods (ii) to (iv). The first was based on the true cross section which would increase the wetted perimeter considerably and the second on a smoothed cross section based on the assumption that the rock projections would form portion of the channel roughness and should be excluded in the analysis. It is considered that the true answer would lie somewhere between the two.

Results are shown in the following table and are discussed hereunder.

Table C2

Peak Discharge Estimates at Duck Reach in April, 1929.

by Extrapolation of the Rating Curve

	Discharge	-	Cusecs	
	Actual Cross Section		Smoothed Cross Section	
	Gauge Ht. 42.5	Gauge Ht. 44	Gauge Ht. 42.5	Gauge Ht. 44
og-log Plot	144,000	152,000	144,000	152,000
rea-Velocity	126,000	136,000	128,000	138,000
v A $\sqrt{R}$	110,500	119,000	112,200	126,200
v Q'	126,300	140,000	164,000	180,000

The extrapolations of the rating curve for the actual cross section are shown in Figs. 17 to 20.

C5.121 Extension on Logarithmic Paper. A study of numerous rating curves have indicated that in general they conform to the equation:-

$$Q = a\ (H - Z)^{b}$$

where Q = discharge
H = Gauge height
Z = Elevation of zero flow above gauge datum
a and b = constants

If this equation holds, plotting log Q against log (H - Z) will yield a straight line which can be extended. When the stage of zero flow is unknown it may be estimated by a cut-and-try method by plotting log Q against log H. If the resulting curve is concave upwards, Z is positive, while if it is concave downwards Z is negative. Having determined the sign of Z, successive assumptions may be made until a value which results in a straight-line plot is determined.

For the gauging station at Duck Reach it was found that if Z was equal to - 7.5 ft. a logarithmic plot approximated closely to a straight line. Extension of this line gave estimated values of the peak discharge in 1929 of 144,000 cusecs for a gauge height of 42.5 ft. and 152,000 cusecs for a gauge height of 44 ft.

C5.122 Extension by Studies of Areas and Velocities. In extensions of areas and velocities, the area of cross section is plotted against gauge height to define an area curve. The average velocity in the section as determined for each discharge measurement is similarly plotted against gauge height. For channels not subject to overflow, the velocity curve generally approaches a straight line at high stages, and reasonably good extensions may be made by the use of judgment and experience. The discharge for the flood stage is computed as the product of the area and velocity from the extension of the two curves. In the application of this method consideration should be given to the existence of a definite relation between the stages at the gauge and stages at the measuring section.

The cross sectional area at Duck Reach Suspension Bridge was determined by survey for all water levels up to a gauge height of 45 ft. for both the actual cross section and for a smoothed section drawn through the average of the rock projections forming the bed and banks of the stream.

Average velocities were calculated for the known discharges by dividing by the appropriate cross sectional area for both the actual and smoothed cross section, and the velocity plotted against gauge height. As the plotted points displayed a wide scatter, it was difficult to extrapolate a mean curve. For this reason the results of this method are likely to be in error.

C5.123 Stevens Method of Extension. Stevens has proposed a method of extending the rating curve which is based on an adaption of Chezy formula ($Q = A\,c\sqrt{Rs}$). For some streams $c\sqrt{s}$ sometimes becomes constant for the higher stages in which case the equation may be written in the form:-

$$Q = K A\sqrt{R} \quad \text{where K is a constant}$$

and Q plotted against $A\sqrt{R}$ would be a straight line (for streams which are relatively wide and shallow, the mean depth, D will not differ greatly from the hydraulic radius R and in this case $A\sqrt{D}$ could be substituted for $A\sqrt{R}$).

It should be emphasised that the accuracy of this method depends on the assumption that $c\sqrt{s}$ is a constant. It will be noted from the discussion in section C5.132 that although the water surface slope at Duck Reach is approximately constant for the higher discharges there is a wide variation in the roughness parameter with water depth. Consequently the assumption that $c\sqrt{s}$ is a constant for this station is unsound and the results should be accepted with caution.

The cross sectional area and the hydraulic mean radius was obtained for all relevant river stages by a survey at Duck Reach suspension bridge and $A\sqrt{R}$ calculated for both the actual cross section and a smoothed cross section. The measured discharges at the station were plotted against the appropriate value of $A\sqrt{R}$ and the resultant straight line plot extrapolated to estimate the peak discharge in 1929 for values of $A\sqrt{R}$ corresponding to stage heights of 42.5 and 44 ft.

C5.124 Extension by R - Q' Method. In the R - Q' method of extension, the hydraulic mean depth R is plotted on logarithmic paper against Q', which is the average discharge per foot of width. The line through the plotted points may be extended to the value of R corresponding to the cross section in the channel at the crest stage. The discharge is the product of Q', and the surface width W of the channel at the crest stage. This method appears to be reliable under favourable conditions, but lacks experimental verification for long extensions under varied conditions of channel and is not well adapted to irregular channels or those subject to overflow.

C5.125 Reliability of 1929 Flood Estimates by Extrapolation Methods. Of the four methods used for extrapolation of the rating curve the logarithmic and R-Q' methods resulted in the least scatter of the experimental data from a straight line plot. It should be noted, however, that this does not necessarily mean that the stage-discharge curve will continue to follow this relationship for floods larger than those recorded.

In the area velocity method, it was difficult to define accurately the velocity stage relationship because of the wide scatter in the plotted points, and the magnitude of the 1929 discharge as estimated by extrapolating this curve should be accepted with caution.

For the Stevens method it is doubtful whether the assumption, that $c\sqrt{s}$ is a constant for the higher flood stages, is true. Roughness parameters calculated at Duck Reach during the floods of May, August and October, 1958 indicated that the channel roughness parameter C varied considerably with river stage whilst the water slope remained fairly constant. For these reasons 1929 discharges estimated by this method are doubtful.

C5.13 Slope Area Calculations

C5.130 Introduction. Another method of estimating the 1929 flood discharge is by the use of the principles of steady flow in open channels. After the flood of 1929 trash line levels were taken at Duck Reach to establish the peak water surface slope. If uniform flow is assumed the water surface slope may be taken to equal the energy gradient and either Chezy or Mannings formulae applied to determine the peak discharge provided the value of channel roughness can be estimated.

C5.131 Estimation of Channel Roughness. As previously described, staff gauges were installed at Duck Reach 800 ft. upstream, 500 ft. downstream and at the suspension bridge to measure water surface slopes during flood flows. Readings were taken during the floods of May, August and October, 1958 and the roughness factor for the use in Manning's equation calculated for the known discharges, the cross sectional area and hydraulic mean radius being measured by field survey.

It should be noted that as a result of observations during the October 1958 flood, the readings on the staff gauges should be adjusted slightly to obtain a true measure of water slope. The gauge at the bridge is sited on the outside curvature of a bend in the river and the resultant build up in water level gives readings which are too high. Observations during the October flood indicated that gauge readings should be reduced by 0.7 ft. at 22,000 cusecs and by 1.5 ft. at 44,000 cusecs. The downstream gauge was placed below a river control and was not truly representative of water surface slopes at Duck Reach. For this reason, during this flood a new station was established 200 ft. upstream of the old gauge and above the control. The relationship between the new and old gauges was established during this flood and the water surface slopes measured in the floods of May and August, 1958 were adjusted accordingly. The gauge upstream of the bridge was satisfactory.

Details of water slopes and calculation of the channel roughness parameter is shown in Table 6.

C5.132 Selection of Roughness Parameter. The variation of Manning's "n" with gauge height at the suspension bridge has been plotted on Fig. 21. This graph clearly indicates that any estimate based on Manning's equation, without first determining the change in the roughness parameter with water depth, would be grossly in error.

The accuracy of this method is limited by the peak flood recorded over the past two years, which was only 25.5 ft. on the gauge as compared to 44 ft. measured in 1929 (or 42.5 if an allowance of 18 inches is made for wave action). It would have been desirable to have calculated the value of "n" for several higher discharges before finally estimating the magnitude of the 1929 flood, and it is hoped that river slopes will continue to be measured in the future for discharges exceeding 60,000 cusecs so that the graph can be more accurately defined in the higher ranges. Potts in his earlier investigation on flood protection for Launceston measured a river slope of 0.017 for a gauge reading of 31.5 during the flood of June, 1931. This gives a "n" value of 0.096 but as his slope measurements were based on readings at the bridge there is some doubt as to their accuracy (as discussed in Section C5.131) and this value of the roughness parameter should be viewed with caution.

As the value of "n" was only available for a maximum gauge height of 25.5 ft., it was necessary to extrapolate the curve in order to estimate the correct roughness parameters for application to the 1929 flood. This was carried out by plotting log "n" against log gauge height when the measured points approximate to a straight line. This was extended upwards to obtain the roughness parameters for the higher ranges. As there is no sharp variation in the cross sectional area for any flood stages it is considered that this method will give fairly reliable estimates of "n".

Values of the roughness parameter thus adopted for estimating the 1929 peak discharge from Manning's equation were:-

0.0825 for a gauge height of 42.5 ft.
0.0805 for a gauge height of 44 ft.

C5.133 Estimated 1929 Peak Discharge. After the 1929 flood the water surface slope at Duck Reach was established by a trash line survey and calculated to be 0.019. This value compares favourably with the slopes recorded in the floods during the past two years. The corresponding estimates of the 1929 peak discharge would be 149,000 cusecs and 164,000 cusecs for a gauge height of 42.5 ft. and 44 ft. respectively as shown in Table C3.

Table C3

Discharge Peak Estimates by Slope Area Methods at Duck Reach - April, 1929.

Gauge Ht. (sq.ft.)	X-Sectional Area (sq.ft.)	Hydraulic Mean Radius (ft.)	Water Surface Slope	Roughness Parameter "n"	Discharge (Cusecs)
42.5	6,980	25.1	0.019	0.0825	149,000
44.0	7,330	26.0	0.019	0.0805	164,000

C5.14 Estimate of 1929 Flood by the Application of the 1929 Rainfall to the Unitgraph.

C5.140 Introduction. A third method of estimating the 1929 flood discharge is to apply the excess rainfall pattern of the 1929 storm to the mean unitgraph for the South Esk River at Launceston.

C5.141 Loss Rates. Light rain which fell on the catchment during the week preceding the main storm and the low intensity rainfall in the early part of the storm has been taken as initial loss.

There was insufficient data on the variation of loss rates with antecedent moisture conditions to estimate accurately the loss rate applicable to the 1929 storm. Also Foster considered that the apparent variation in loss rates with antecedent moisture conditions is mainly due to an incorrect assessment of initial loss. It is generally agreed that loss rates approach a constant value some short time after the start of heavy rain and it would therefore be expected that the longer the duration of the storm the closer the loss rate would approach that for a saturated catchment, since low losses for the major portion of the storm would tend to outweigh the high rainfall losses in the early stages and also reduce the error of an incorrect assessment of initial loss.

From a study of loss rates derived for various storms over the catchment as shown in Table 2 and bearing in mind the considerations discussed above, it is reasonable to assume that the loss rate for the 1929 storm lay between 5 points/hour and that corresponding to a saturated catchment of 2.6 points/hour.

C5.142 Estimated Hydrographs. The synthesised mass curve as estimated for the storm of April, 1929, was discussed in Section C4.221 and is shown in Fig. 22. The estimated limiting loss rates were deducted from this gross rainfall pattern and the excess rainfall for each 6 hour period after the start of surface run-off determined for each of the loss rates. By applying these rainfall patterns to the mean 6 hour unitgraph for the South Esk catchment, as described in Section C3.39, two surface runoff hydrographs were produced to which were added the estimated groundwater flow to obtain the total hydrograph. From gauge readings taken at Duck Reach prior to the flood, the base flow at the start of the rise was known at 450 cusecs. The increase in groundwater flow as a result of the storm was estimated from a study of the floods used for the derivation of the unitgraphs and as a result it was estimated that the base flow at the end of surface run-off would be 10,000 cusecs. Intermediate values were extrapolated on the assumption of a straight line variation between these two values. As the magnitude of the base flow is only a small percentage of the peak discharge in 1929 accurate estimates are not required, as an error of 100 percent in the assumed base flow would only alter the peak discharge by 4,000 cusecs.

As a result of these calculations two hydrographs were produced (Fig. 22) for loss rates of 2.6 pts./hour and 5.0 pts./hour and the corresponding peak discharges were 133,000 cusecs and 163,000 cusecs respectively.

C5.15 Adopted Value of 1929 Peak Discharge

A review was made of all the estimates of the 1929 peak discharge as obtained by the various methods of extrapolation of rating curve data, slope area computations and from the application of the rainfall to the unitgraph with limiting values of loss rates, giving weight to those methods considered most reliable. As a result of this study it was considered that the 1929 peak discharge lay between 140,000 cusecs and 160,000 cusecs, the best estimate being 150,000 cusecs.

C5.16 Hydrograph of 1929 Flood

Exhaustive enquiries in Launceston produced reliable evidence that the peak of the 1929 flood occurred shortly after midnight on the Saturday. In addition gauge readings were available at Duck Reach to define the rising limb of the hydrograph up to a discharge of 64,000 cusecs.

The 1929 flood hydrograph adopted for the model studies was therefore drawn to correspond with that determined from the unitgraphs and having a peak discharge of 150,000 cusecs. This curve was adjusted slightly so that the peak occurred at midnight on Saturday and the rising limb agreed with the recorded values. The final estimated hydrograph for the 1929 flood is shown in Fig. 22.

C5.2 Flood Estimates for North Esk River - 1929 Flood

C5.20 Introduction. There was no record of the peak flood height reached in the North Esk River for the flood of April, 1929, so the flood discharge was estimated by the rainfall-loss-unitgraph method, by applying the 1929 rainfall to the average unitgraph derived for the North Esk catchment at Corra Lynn.

C5.21 Loss Rates. Antecedent moisture conditions in 1929 corresponded closely to those existing prior to the flood of May, 1958, which resulted in a triple peaked hydrograph from three individual bursts of rain. As described in Section C2.2 the loss rates were derived for this storm for each of the rainfall bursts. This enabled a plot to be made of the variation of loss rates with time for this flood as shown in Fig. 23. These values of loss were assumed to apply to the 1929 storm, for the calculation of the excess rain to be applied to the unitgraph.

C5.22 Estimated Hydrograph. The synthesised mass curve estimated for the storm of April, 1929, was discussed in Section 4.221 and is shown in Fig. 24. The excess rainfall pattern was determined by subtracting the appropriate loss rates and a 4 hour hyetograph of excess rainfall prepared. By applying this rainfall pattern to the mean 4 hour unitgraph for the North Esk catchment by the method described in Section C3.39 a surface run-off hydrograph was produced to which was added the estimated groundwater flow to obtain the total hydrograph. From a study of the floods recorded on the North Esk River over the past two years and in particular that of May, 1958, which resulted in a peak discharge of 10,000 cusecs, it was estimated that the groundwater flow would increase from an estimated zero discharge at the start of rain to 3,000 cusecs at the end of surface run-off.

The calculated hydrograph is shown on Fig. 24, the peak discharge being 12,600 cusecs.

This hydrograph was estimated from unitgraphs derived at Corra Lynn 7 miles above Launceston and requires an adjustment to allow for the additional surface run-off below the gauging station in order to estimate the flood hydrograph at Launceston.

Sherman has suggested that the ordinates and abscissae of unitgraphs for similar basins might be assumed to be proportional to the square roots of the respective drainage areas. This simple rule does not take into account other factors that affect the shape of the unitgraph such as the slope and shape of the basin.

In this case, where different areas of the same basin are under consideration and as the increased catchment area below Corra Lynn is only a small percentage of the total area, it is considered that this simple rule is sufficiently accurate.

To obtain the hydrograph at Launceston from that at Corra Lynn the discharge scale and time scales were multiplied by the ratio of the square root of the catchment area above Launceston to the square root of the catchment area at Corra Lynn, or in other words the discharge at Launceston after time 1.106t would equal the discharge at Corra Lynn after time t multiplied by 1.106.

The hydrograph calculated for the 1929 flood at Launceston is shown in Fig. 24 the peak discharge being 13,900 cusecs.

C5.23 Discussion of Hydrograph Estimate. In August, 1936 a flood occurred in the North Esk River which reached a peak stage of 200 ft. on the bridge gauge at Corra Lynn. By extrapolating on logarithmic paper the rating curve at Corra Lynn (Fig. 6) the peak discharge corresponding to this stage was determined at 14,350 cusecs. There was a difference of opinion amongst the local inhabitants as to whether this flood was bigger or smaller than that of 1929. The general opinion, however, seemed to be that the discharge was lower, and on the St.Patrick's river, a tributary of the North Esk, the discharge in the 1929 flood was approximately 1.26 as great as 1936. If this is true then the estimate of the 1929 flood calculated in Section C5.22 would be in error. There are several reasons why this may be so, viz:-

(i) The synthesized mass curve for the catchment was based on newspaper reports of bursts of heavy rain combined with official daily rainfall records. These reports were insufficient to define accurately rainfall intensities between the 9 a.m. daily readings.

(ii) The number of daily read rain gauges on the catchment in 1929 was only four, and there were no records of the depth of precipitation on the high mountains surrounding the catchment and separating the North Esk and St. Patrick's Rivers. For this reason the assumed areal pattern of the rainfall as shown on Fig. 15 can be considered only as an approximate estimate.

(iii) Loss rates for the North Esk were derived for two storms, which is insufficient to estimate an accurate value of loss rates for application to the gross rainfall.

(iv) The average unitgraph at Corra Lynn was determined from the analysis of only two storms.

After considering all factors, it was decided to increase the peak discharge in 1929 to 18,000 cusecs or 1.26 times the 1936 flood peak.

Having fixed the maximum discharge, a constant loss rate of 4.3 pts/hour was determined by trial and error such that the hydrograph at Corra Lynn (Fig. 24) as calculated by the intensity-loss-rate-unitgraph method, gave a peak discharge corresponding to that assumed. The hydrograph of the 1929 flood at Launceston, as shown in Fig. 24, was obtained from that at Corra Lynn by the method described above and this hydrograph was used for the model tests.

It is realised that the accuracy of this part of the investigation could be considerably improved, but as the volume of flood waters in the North Esk is only a small percentage of the discharge in the South Esk (12 per cent in 1929) a more accurate investigation was not warranted, as an error of plus or minus a few thousand cusecs in the estimated peak discharge would have no major effect on river levels in Launceston during a major flood. This conclusion was later proved correct by the model tests, which showed that for a major flood, river levels along the North Esk in the vicinity of Launceston were governed by backwater storage from the South Esk and were practically independent of the discharge in the North Esk.

C5.3 Maximum Probable Flood on South Esk River

The maximum probable flood hydrograph was determined by the rainfall-loss-rate-unitgraph method by applying the maximised 1929 storm rainfall to the average unitgraph derived for the South Esk catchment.

C5.31 Loss Rates. In accordance with the conception that severe but not uncommon antecedent moisture conditions should be adopted for estimating the maximum probable flood, a loss rate of 2.5 pts/hour was used, corresponding closely to that recorded on the catchment in September, 1952, 22nd May, 1956 and August, 1958.

As the catchment is subject to light and uniform rain throughout the year from the common westerly weather, initial loss has been assumed to have been satisfied prior to the main storm.

C5.32 Estimated Hydrograph. As discussed in Section C4.222 the mass curve of the 1929 flood was maximised for moisture content. The adjusted curve is shown in Fig. 25. The estimated loss rate was deducted from the gross rainfall and by applying the resultant excess rainfall pattern to the unitgraph a surface run-off hydrograph was produced, to which was added the estimated groundwater flow to obtain the total hydrograph. The hydrograph of the maximum probable flood derived at Duck Reach is shown in Fig. 25.

Base flow was taken as 5,000 cusecs at the start of rain increasing uniformly to 15,000 cusecs at the end of surface run-off. As pointed out previously, large errors in the estimated base flow will only make a small error in the resultant peak flow and accurate estimates are not warranted.

No attempt was made to arrange the rainfall intensities for the storm in such a sequence so as to produce the greatest peak discharge. It was assumed that the temporal variation within the storm of April, 1929, would also apply to the maximum probable storm.

C5.33 Reliability of Estimate. Basically, the maximum probable flood derived above is the flood which would have resulted in 1929 if the air mass had contained the same quantity of water vapour as the maximum observed in Northern Tasmania over the past 40 years. No attempt has been made to maximise the wind or to adjust the temporal pattern of rainfall to produce the greatest peak discharge nor to consider durations of rainfall differing from that of 1929. If this had been carried out a somewhat greater estimate of the maximum probable flood would probably have resulted.

Any estimate of maximum probable precipitation, which is based on limited meteorological records, must be somewhat suspect. In this investigation it was only possible to base the estimate on the analysis of one storm, that of April, 1929. Fortunately, this storm resulted in one of the greatest floods on record, but it is possible that, if in the future, similar storms occur, or if sufficient meteorological data had been available on the major floods of 1828, 1852, 1863 or 1893 maximising these storms may have resulted in somewhat higher values than those obtained in this study.

For the above reasons it is possible that the estimate of the maximum probable flood for the South Esk catchment may be slightly low, but as the frequency of such a flood would have an estimated recurrence interval exceeding 10,000 years it is of little consequence in a flood protection study, where suitable precautionary measures would prevent any loss of life if by chance such a flood occurred and the levee banks were overtopped. On the other hand for the design of the spillway capacity of a major dam, such as Trevallyn Dam, where failure would result in the most serious consequences it would be advisable to increase this estimate slightly. The adopted design flood for Trevallyn Dam was 300,000 cusecs, which should be sufficient to discharge safely the biggest flood that is ever likely to occur on the South Esk catchment.

C5.4 Maximum Probable Flood North Esk Catchment

C5.41 Method of Estimation. The maximum probable flood for the North Esk river which would occur concurrently with the maximum probable flood on the South Esk river was estimated by applying the maximised 1929 storm to the average unitgraph for this river, in the same manner as was described for the South Esk catchment.

Because of the limited data on the correlation between loss rates and antecedent moisture conditions over the catchment and also the reasons given in Section C5.31 when estimating the 1929 flood hydrograph, a loss rate of 2.5 pts/hour was adopted corresponding to that used for the estimation of the probable maximum flood on the South Esk catchment.

The reproduced hydrograph at Corra Lynn is shown on Fig.26 and resulted in an estimated peak discharge of 29,000 cusecs. The corresponding flood hydrograph at Launceston was determined from that at Corra Lynn by the method described in Section C5.22 and is shown in Fig. 26, the estimated peak discharge being 32,000 cusecs.

C5.42 Reliability of Estimate. The comments made in Section C5.35 on the reliability of the estimate of the probable maximum flood on the South Esk catchment and the comments in Section C5.23 on the estimate of the 1929 flood hydrograph on the North Esk River will also apply to this study.

However, as the model clearly indicated that a variation of the discharge in the North Esk River of plus or minus several thousand cusecs would have a negligible effect on flood levels in Launceston during a major flood (for which the river stage is mainly governed by backwater storage from the South Esk), a more thorough study of this phase of the investigation was not justified.

It should be realised that in general the convective thunderstorm mechanism is the most rain productive for areas up to approximately 500 sq. miles, and an estimate of the maximum probable flood obtained by applying Walpole's thunderstorm model to the North Esk catchment may result in a higher peak discharge than that calculated above. However, in this particular investigation the main river causing flooding of Launceston is the South Esk, and the North Esk discharge required is that which would occur concurrently with a probable maximum flood in the South Esk. Consequently no action was taken to carry out a thunderstorm maximisation.

C6 INFLUENCE OF TIDES AND RIVER CONSTRICTION

C6.0 Introduction

The data collection programme undertaken to determine the influence of tides and river constriction above Stephenson's Bend, on flood levels in Launceston was discussed in Section C1.42(c).

As no major flood occurred during the period of the investigation, readings were taken for two minor floods only, that of 25th May, 1958 (27,000 cusecs) and for 18th August, 1958 (45,100 cusecs). For this reason the results are inconclusive for major floods and it would be desirable to continue the readings for greater flood discharges in the future.

C6.1 Effect of River Constriction above Stephenson's Bend

Trash line levels taken after the 1929 flood indicated a change in hydraulic grade in Home Reach above the tailrace channel as compared with that below the bend, which leads to the conclusion that a constriction to flow occurred in this vicinity. Flood grades taken during the August flood (Fig.27) confirmed this theory. The hydraulic grade in Home Reach above the powder jetty was practically flat, whilst below the Powder jetty there was a sudden break in grade to 1 in 11,000, the river level falling

0.3 ft. in 3,300 ft. to the Powder Dolphins. Below the Powder Dolphin the hydraulic grade flattened somewhat and between the Powder Dolphins and the downstream end of the Hunter Cut the grade was 1 in 27,200. At low tide the same effect was noticed but the hydraulic grades were somewhat steeper over the three sections, being 1 in 16,200, 1 in 3,170 and 1 in 6,000 respectively.

These results indicated that there was a constriction to flood flows at Stephenson's Bend even for discharges as low as 45,000 cusecs, but the exact magnitude of this influence at higher discharges cannot be determined until such a flood occurs.

In the May flood, for which the peak discharge was 27,000 cusecs, there was no difference between the high tide level at the downstream end of the Hunter Cut and the southern end of Home Reach. This indicates that for discharges of this magnitude, and lower, the effect of any river constriction on peak water levels in Launceston is negligible.

C6.2 Tidal Influence on Velocity Patterns in Home Reach

Although there was clear evidence that tidal interference to flow patterns and velocity distributions was negligible for the 1929 flood, the upper limit of discharge for which this applied was not known. In order to gain more information on this factor, the vertical distribution of velocity was measured in the Tamar River at a cross section at the Powder Jetty for the floods of May and August, 1958. These results indicated that tidal flow would distort the velocity distribution considerably for flood discharges up to 35,000 cusecs, but as a flood of this magnitude provides no threat of flooding to Launceston the results are of little consequence. Further data for discharges exceeding say 60,000 cusecs would be required before any quantitative conclusions could be drawn.

C6.3 Effect of Tide on Flood Levels

This is discussed in Section C7.22 below. Although not essential for this investigation, it is desirable that some further study be made of this factor. Such knowledge would be valuable in interpreting the behaviour of future floods; in the comparison of model and prototype performances and in flood forecasting techniques for the smaller floods. A more refined analysis of this aspect could be carried out by a careful study of available data on gauged and historic floods with a view to establishing relationships between phases of the moon, wind and barometric pressure, on the one hand, and rise in normal low and high tide levels and variations of tidal range for floods of various magnitudes. From the results of the measurements of the measurements of flood levels in the Tamar River for the floods of May and August, 1958, it would appear that for discharges of 30,000 cusecs and lower there would be no change in the high tide level. Above this discharge, tidal levels would be increased. In August 1958 for a discharge of 45,000 cusecs a difference of 0.6 ft. between the high tide level at Launceston and that at the downstream end of the Hunter Cut was measured.

C7.0 FLOOD FREQUENCY STUDIES

C7.1 Floods in South Esk River during Period 1900-1958

Readings of peak floods heights have been recorded from a staff gauge at Duck Reach Power Station in the South Esk gorge since 1900.

In 1945, when the Hydro-Electric Commission of Tasmania commenced investigations of the proposed dam at Trevallyn, a short distance above Duck Reach, action was taken to "rate" this gauge accurately up to 45,500 cusecs (26.5 ft.)

Thus, when this study was commenced peak flood heights were available for all floods since 1900, and a reliable stage-discharge curve was available for flood heights up to 26.5 ft.

This curve was extrapolated as discussed in Section C5.12, and the 1929 peak discharge was estimated in the manner described in Section C5.133, thus giving a satisfactory annual flood series for the period 1900-1958.

C7.2 Historic Floods

C7.21 Available Data. Settlement commenced in Tasmania in 1803, and by 1807 Launceston was established. Hence a relatively long newspaper record of events is available and was consulted to establish a list of historic floods. The starting point in the newspaper search was the "Record of Severe Floods in Tasmania", included in the 1936 publication of the Bureau of Meteorelogy (C7). This gave a list of 48 floods which affected Launceston. Although several important floods were in fact omitted from this list, it

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Ref. (C7) "Results of Rainfall Observations in Tasmania"
Met. Branch, Aust., 1936.

provided a basis for a detailed study of newspaper files and similar enquiries. The Librarian (W.Sutherland) and staff of the Launceston City Library were most helpful in this regard. In addition, useful first, second and even third hand information was obtained from residents of Tasmania. The writings of Russell Kidd, upon whose investigations Balsille and McCabe and Potts and Dare drew largely, were also most useful.

The earliest damaging flood mentioned was in 1828, and the total flood record of 131 years is probably the longest and most reliable in any Australian river. A detailed research into more obscure historical documents than newspaper files would improve the accuracy of the study of historic floods, but was not considered essential for this particular problem.

Unfortunately, the evidence was often conflicting, emphasising the need in future to publish and file in some permanent reference place detailed factual reports of future floods throughout Australia, so that engineers of later years will have reliable evidence.

Balsille, when interviewed, stated that he remembered during his investigations in 1929 some evidence of floods reaching a height of 25 ft. above S.L.W., and Ald. Ockerby is reported in the "Examiner" of 28th January, 1936 as stating that in an early flood sailing boats reached a point in Launceston which would agree with this level (Cameron St.).

The accuracy of these statements is very doubtful. However, Maddock stated that when he was establishing flood heights immediately after the 1929 flood, he was shown by a local resident (G. Sidebottom) a previous flood mark in Canal St. which is 1.3 ft. higher than the 1929 level. The mark is still there, but it is labelled "1929 flood", apparently in error. Another resident (J.Walsh) stated that in 1929 he was shown the height of the 1863 flood in Gurr's Ship Chandlery in Charles St. (now Tasmanian Orchardists Pty.Ltd.), and when the 1929 flood occurred he went to this mark to compare the two floods, and found that the 1863 flood was 1'6" higher. Russell Kidd makes no mention of flood marks in Launceston, but by a study of flood heights at Longford and other catchment evidence he concluded that the 1852 flood was the highest previous flood, and rated it equal to the 1929 flood.

It is reasonable to conclude that the two greatest floods in the nineteenth century were in 1852 and 1863, and that one of them was materially greater than that of 1929. Whether the largest one was in 1852 or 1863 is of little importance, as the flood frequency graph would be the same, whichever is chosen.

The weight of evidence now available seems to point to the 1863 flood being the first in an array of flood magnitudes, with 1852 slightly bigger than 1929.

Hydrologic studies during the period 1957-59 indicated that the information regarding flood heights at key points and bridges in the middle and upper reaches of the catchment was irrelevent. The complexity of the river system is such that a given stage height at, for example, Avoca, just above the confluence of the St. Pauls and South Esk rivers, (which was almost invariably given by newspaper reports) is no indication of discharges or stage heights in the Launceston area.

Hence the most useful data were:-

(i) Flood heights at Longford and Hadspen, giving a guide to the total South Esk discharge.

(ii) Levels attained in the Charles St. - Esplanade - Tamar St. area of the City.

(iii) Overtopping of embankments and depths of flooding in Launceston. Inveresk-Invermay levees were originally built prior to 1852 and have been raised in certain sections in the period 1852-63, in 1890 and 1926. Hence the present general level of levees is not necessarily a reliable indication of the level of, for example, the flood of 1872, which overtopped them. It is possible that it is reliable, because the embankments gradually settle in the mud which constitutes the Inveresk district, and the true position may be that the periodical raisings of the levees referred to in the newspapers over the past 100 years have merely maintained the system since 1852 at a level between 16.5 and 17.00. However, there is no certainty on this point, and (ii) above is considered a more reliable guide to historic flood levels.

C7.22 Use of Model to Establish Stage-Discharge Graph. The verification of the hydraulic model, described in Part D, Section D5.4, was very satisfactory for a South Esk discharge of 150,000 cusecs.

For discharges below this value, tidal effects cannot be ignored, their importance increasing with decreasing discharge.

To allow for this, considerable expense would be necessary in the construction of a tide machine. So far as comparing the efficiency of various flood mitigation measures is concerned, it was decided to compare them on the basis of an average high tide. For estimating historic discharges exceeding 150,000 cusecs, the model, without a tide machine, could be used with confidence to determine the model discharge necessary to give in the Charles - Tamar St. area the depth of flooding recorded by newspaper or other evidence.

For discharges between say 100,000 and 150,000 cusecs the model might be considered to give an approximate guide to this stage discharge relation.

However, for this range of floods, some rough and ready allowance should be made for wind and tide in interpreting the stage-discharge graph for 1929 topographic conditions provided by the model.

This was done on the basis of the following reasoning:-

(i) At 150,000 cusecs in 1929 flood there was no rise or fall with tidal movement in Home Reach and the Charles St. area, although the effect of tide rise and fall was quite marked downstream of the Hunter Cut.

(ii) A rise in tailwater on the model of 1.0 ft. from 15.5 to 16.5 gave no rise in the Charles St. area at 150,000 cusecs.

(iii) If a flood of less than 150,000 cusecs occurs under neap tide conditions, the rise in normal high tide level in the Charles St. area would be greater than the rise which would occur for Spring tides, because the higher the initial river heights before imposition of flood discharge, the greater the increase in waterway area provided by a given flood rise for escape of flood waters.

Accordingly, in Fig. 28 the curves for the stage-discharge relation for (i) near Charles St. and (ii) at King's Wharf, for 1958 topography from 120,000 to 250,000 cusecs was first graphed from the model results. For 1929 topography the level at Charles St. was known for 150,000 cusecs. This point was plotted and the curve for 1929 topography interpolated between the first two curves.

The direction and velocity of the wind, barometric pressure over Bass Strait, moon phases and other factors influence the height of high tide at Charles St., the range being from 11.00 to 15.8. The initial tidal conditions must affect the stage discharge relation markedly at the lower discharges, the effect being gradually drowned out as the flood discharge increases.

The following data were available as a guide to this fanning out of the "1929 topography - Charles St." curve of Fig. 28.

Date of Flood	South Esk Discharge	Wind	Tide Phase	Level at Charles St.
21st July, 1931	71,500	Light S.E.	6/7 Neap	15.5
14th October, 1926	74,000	Light N.W.	3/4 Neap	16.5

Although the wind is reported as "light" in each case, it is quite possible that over Bass Strait it was much stronger. Also the barometer over Bass Strait may have been lower in 1926 than in 1931.

It was considered that this range of levels for discharges approximating 70,000 cusecs would have been greater if one had occurred at Spring tide with strong northerly winds and the other at neap tide with a strong southerly. A level of 15.8 was reached on 13th November, 1958 with strong northerly winds and negligible discharge. Hence the final upper and lower limits were taken as 16.8 and 15.0, and the curves drawn as shown in Fig.28.

C7.23 Changes in Topography. If the topography of 1958 differs materially from that existing in the nineteenth century, the use of the model or the graph of Fig. 28 to assess historic flood discharges could be erroneous.

The following changes since last century would tend to give higher flood levels in this century than in the last for a given flood discharge:-

(i) Prior to 1900 the area east of King's Wharf was a swamp, and flood waters would escape from the city across this area with greater ease than was the case in 1929.

(ii) The raising of the levees on Town Point after the 1889 floods would have the same effect, as would also the construction of houses and factories over the last half century.

(iii) Siltation has probably been increased by agricultural activities on the catchment.

The following factors would have the opposite effect:-

(i) Prior to 1900 much of Royal Park was a swamp, and the jet of floodwaters described in Section D5.5 could sweep across this area to enter the city with greater ease than in 1929 and in the future. However, when the discharge reaches 125,000 cusecs portion of the flood waters sweeps across Royal Park even at its present level, so that this change of topography should have a diminishing effect on flood heights for discharges above 125,000 cusecs.

(ii) Dredging of the river channel, which presumably has been much more actively pursued in this century than in the last, would tend to give lower floods nowadays for a given discharge. In this connection the model tests show that it would be the dredging of Ti-Tree and Stephenson's Bend, rather than Home Reach, which affects flood levels. However, there is reason to believe that the 1929 flood scoured out the channel more effectively than could any dredging procedure and presumably major floods in the last century would do the same.

It was therefore assumed that for a given recorded flood height in the Charles St. area, the corresponding discharge for historic floods would be for practical purposes equal to the corresponding discharge in modern floods. This assumption may be erroneous for minor floods, but the bigger the flood the safer the assumption becomes.

<u>C7.24 Basis of Flood Frequency Diagram</u>. In the absence of tidal and major storage effects, and with stable controls, a graph of flood discharge against the probability of a flood equalling or exceeding that discharge is for practical purposes the same as a graph of flood stage against probability of equalling or exceeding a given stage, because the relation between stage and discharge is given by a stage-discharge graph which remains unchanged under all conditions. For floods less than 150,000 cusecs, the stage reached at Charles St. varies with the high tide level which would have been reached, had no flood occurred.

In view of the fact that for the lower floods the height reached at Charles St. depends upon the chance effects of wind, moon's phases, barometer etc. (affecting tide level) and of rainfall, catchment condition (affecting peak flood discharge) it would at first glance appear that the probability of exceeding a given stage rather than a given discharge should be the basis of the frequency diagram to be used for economic analysis.

However, for the period 1900-1958 reliable South Esk discharge measurements are available, but tide charts were only kept for a few years of this period, and stage heights reached by various floods are not accurately known. Further, the cost of a training levee in Royal Park depends upon the South Esk discharge to be diverted, rather than upon the flood stage against which protection is to be given.

The decision was therefore made to assess historic discharges from all available evidence, including stages reached at Launceston, Longford and Hadspen, to plot a discharge-frequency graph, and to tabulate costs of protection in terms of the South Esk discharge against which protection is provided.

C7.25 Estimates of Historic Flood Discharges. A full description of the exhaustive enquiry and evaluation of conflicting evidence would serve little purpose. The final decision and the evidence accepted as valid is given hereunder for each historic flood.

21st July, 1893.

Levees were overtopped and boats plied along Invermay Road. Flood was within 20 yards of the barracks (18.59). The lower drill hall was submerged (floor level 17.57). Water was 18" deep in front of Marine Hotel and extended 20 ft. beyond the Riverview Hotel (17.14). Levees had been raised in 1890. It was decided that 17.75 would be a reasonable approximation to levels in the Charles St. area. Fig. 28 gives a range of from 118,000 to 122,000 cusecs, and a value of 120,000 cusecs was adopted for the flood frequency graph.

13th November, 1889.

Discharge in South Esk gorge and height at Longford was reported to be greater than in June, 1889, but level at Charles St. was eleven inches lower. In both June and November, 1889 the wind is reported as strong northerly, but in June the tide was "spring" and in November "neap". This evidence leads to 16.1 as the level, and the corresponding South Esk discharge for neap tide and S.E. gale is given by the lower portion of the band in Fig. 28 as 93,000 cusecs. However, there is evidence that height at Longford was 1'0" higher than in June. The explanation of this discrepancy could lie in different barometric pressures over Bass Strait or differing N.Esk discharges. A compromise figure of 105,000 cusecs was adopted, being slightly in excess of the June discharge.

17th June, 1889.

Flood is reported as 2'0" deep in Marine Hotel (floor level 15.42) and "several"inches over Queen's Wharf (Deck level 16.18 to 16.60). A flood level of 17.00 appears a reasonable estimate. With a southerly gale and spring tide the appropriate point in the curve would be about the middle of the band in Fig. 28, giving a discharge of 100,000 cusecs.

18th May, 1872.

At 1100 hours, when the tide went down, the waters did not fall more than 2 ft. and at 1600 hours the flood rose rapidly over the levees, reaching a depth of 3 ft. in the Invermay Road. Even if the levees were at say 16.25 in 1872, this would indicate a level for a few hours at high tide, and on a rising flood hydrograph, of say 16.75. The moon's phases indicate a tide midway between spring and neap. No information is available regarding wind. Fig. 28 gives for 16.75 a range from 70,000 to 108,000 cusecs.

In regard to tidal range, a reliable tide gauge in 1931 gave 4 ft. range for 71,500 cusecs, (indicating 112,000 cusecs in 1872), while a newspaper report in 1926 gives 2 ft. range for 74,000 cusecs (indicating 74,000 cusecs in 1872).

A figure of 95,000 cusecs was adopted.

16th December, 1863.

It is assumed that the past flood level 1.3 ft. above the 1929 flood level in Charles St. applied to this flood. In this area the 1929 flood level was 20.2 at top of surge. The model gave a general flood level exclusive of surge of 19.7, making 21.00 the general flood level in 1863, and corresponding discharge 175,000 cusecs. Tidal conditions would have no effect in this case.

10th August, 1852.

Water reached the top of the bar counter of the Ferry House Hotel (now the Bridge Hotel), giving a level of 20.10 and a discharge of 158,000 cusecs from Fig. 28.

16th July, 1852.

This is reported as the highest flood for 20 years. It crossed the lower part of Charles St. and carried away the lower floor of the Post Office. A level of 17.12 was adopted and discharge would be 105,000 cusecs and would be about the middle of the band in Fig. 26.

15th September, 1828.

No reliable evidence can be found. Russell Kidd estimated this discharge as 4/5 of the 1929 discharge. Presumably he had some evidence not now available. This ratio was adopted, giving 125,000 cusecs.

C7.3 Discharge Necessary to Cause Flood Damage

A level near Charles St. Bridge of 15.8 results in a flow a few inches deep over the North Esk bank near Queen's Wharf, closing Charles St. to traffic, flooding the Marine Hotel to a depth of several inches, with the flood waters extending 800 ft. up Charles St. from the bridge. On the other side of the North Esk, a flood level of this magnitude causes a trickle of water into Inveresk and Invermay, as it is several inches over the railway line for a distance of about 40 ft. However, for flood levels up to 17.00 this relatively gentle flow could presumably be countered by temporary measures such as sand bags and the only flood damage would be the wages of the men so employed, disruption of traffic on to King's Wharf and similar minor items. From 17.00 upwards a sudden flood increase would occur which could not be countered by emergency measures. Fig. 26 gives a range from 87,000 to 114,000 cusecs as the discharge causing serious damage, and 90,000 cusecs was adopted for the starting point of the analysis of annual benefit-annual cost. This implies the assumption that for S.Esk discharges of 90,000 cusecs, or less, the damage would be negligible.

For flood warning purposes, a lower figure should be adopted for the warning to prepare for emergency measures. In the present condition of the levees, a predicted peak discharge of

70,000 cusecs would be by no means too low for the warning, if the wind is northerly and the tide a "spring" tide.

C7.4 Preparation of Flood Frequency Diagram

South Esk floods for the period 1828 to 1958 which have exceeded 70,000 cusecs are shown in Fig. 29.

The values finally adopted for the historic floods were plotted as the 1828–1958 partial duration series on the same graph as the 1900–1958 annual flood series available from gaugings using the following different graph papers:-

Ordinates (Flood discharge)	Abscissae (Probability)
Natural	Natural
Natural	Logarithmic
Logarithmic	Logarithmic
Natural	Probability
Logarithmic	Probability
Natural	Gumbel

In no case did the graph approximate a straight line. It was felt that the natural-probability scale was the most convenient. This plotting is reproduced in Fig. 30.

In regard to plotting position, the m/n formula was adopted, i.e. the third largest flood in a period of 131 years was plotted at the probability point 3/131 or 0.0231.

Mathematical curve fitting was not attempted, on the grounds that it is no more accurate than fitting by eye. This involves some subjective judgment, so three workers drew the curves separately. The results were reasonably consistent on the various graph papers and Fig. 30 expresses the combined wisdom of the three engineers. This curve assigns a probability of 2 per cent the 1929 flood implying that 20 times in a 1000 years floods equalling or exceeding the 1929 discharge may be expected.

When drawing this curve in its final form it was felt that the historic floods between 90,000 and 120,000 cusecs might possibly have been over estimated. The reason is that prior to 1900 Royal Park promontory was only a swamp, but early this century it was reclaimed to its present level by controlled tipping of city refuse. In its present condition it diverts the South Esk jet for floods of less than 125,000 cusecs, but previously this diversion would be less effective so that a flood of say 110,000 cusecs in the last century might have caused a higher level at Charles St. than it would in this century. For floods above 125,000 cusecs this change in topography would have a diminishing effect. There is no way of measuring this factor, but in drawing the curve slightly less weight was given to those points which may have been subject to this influence.

Some thought was given to the advisability of arbitrarily increasing the effective length of record by say 10 years, on the grounds that if a major flood had occurred between 1818 and 1828, some record would have been found. However, the news bulletins of that time are so sparse that this idea was discarded.

The 131 years of record may have experienced an unusually large or small number of major floods. This "sampling error" is expressed by the 80 per cent confidence limits shown in Fig. 30. For example, these limits express the facts that one may state with 80 per cent certainty that a discharge of 150,000 cusecs will be equalled or exceeded between 7.7 and 40 times per 1000 years.

It is necessary to extrapolate the curve beyond the plotted points to calculate the benefit cost curves discussed in Part E of this report. This process is fraught with uncertainty, but certain basic principles were adopted in this case.

Firstly, it is considered that the curve in Fig.30 must tend to bend towards the right for floods exceeding the highest reached flood.

Secondly, in this graph the conception is expressed that the estimated maximum probable flood of 250,000 cusecs can occur once in 10,000 years, but no flood in excess of this value will occur. The alternative of according a probability of zero to this flood was considered but rejected.

(It has been pointed out in Section C5.33 of this Part of the report that the estimate of the maximum probable flood may be a little low. An arbitrary increase to 300,000 cusecs might be reasonable).

The validity of the extrapolation is not of great moment. The purpose of this diagram is to compute benefit-cost ratios and the assumptions made in the extrapolation tend to lower these ratios.

C8 TAILWATER RATING CURVE FOR MODEL

C8.1 Method of Computation

One of the main weaknesses in this investigation was the lack of any data regarding water levels during major floods in Boat Channel below Stephenson's Bend, upon which to base an accurate estimate of the tailwater level for use in the model studies. At the commencement of the investigation the only data available was a trash line level of 16.8 at Stephenson's Bend for the peak flood height in 1929, and a level of 13.3 for the July 1944 flood for which the peak discharge in the Tamar River was 62,100 cusecs. This latter level would be affected by the state of the tide at the time of the flood but for the want of more accurate data a tailwater rating curve was computed from Manning's formula and the constants determined by substitution of the 1929 and July 1944 flood data. Details of the calculation are as follows:-

The Manning formula for uniform flow in open channels is

$$Q = \frac{1.49}{n} S^{1/2} r^{2/3} A$$

where Q = discharge in cusecs
S = water surface slope for steady flow
r = hydraulic mean radius
A = cross sectional area
n = roughness factor

For a relatively wide shallow channel the hydraulic mean radius can be approximated by the mean depth and the above equation can be written in the form

$$Q = \frac{1.49}{n} S^{1/2} d^{2/3} dW$$

where d = mean depth
W = channel width

and if S, W and n are approximately constant for all discharges

$$Q = K\, d^{5/3}$$

where K is a constant

or $Q = K\,(H-C)\ 5/3$

where $H =$ the flood level above zero datum

and $C =$ the height of average bed level above zero datum.

By substituting the known values of H for the flood discharges of April 1929 and May 1958 the constants of this equation were calculated at

$$K = 5170$$

and $C = 8.86$

which when substituted back into the equation would give the following formula for the tailwater curve at Stephenson's Bend

$$H=\left(\frac{Q}{5170}\right)^{3/5} + 8.86 \qquad \ldots\ldots\ldots(1)$$

and a plot of this curve is shown in Fig. 31.

C8.2 Reliability of Estimate

It is realized that the assumptions upon which this curve is based are only approximate and that flood levels for the lower discharges will be affected by the state of the tide at the time of the flood. However, it is considered that a tailwater curve calculated by equation (1) will approximate to the correct values for tidal conditions corresponding to that of July 1944 at the time of the flood.

However, realizing the limitations of the data, it was felt the values should be increased slightly. A mean curve was adopted for discharges exceeding 163,100 cusecs (1929 flood including N.Esk) between the calculated curve and the considered maximum of a straight line variation between the peak flood levels for the July 1944 and April 1929 floods. Below a discharge of 163,100 cusecs the curve was sketched in by eye to meet the ordinate for zero flow at a river level of 13.5 ft. corresponding to an average high tide in the Tamar River at Launceston.

The final adopted curve is shown in Fig. 31 and this was used for the model "base" tests as discussed in Section D6.1.

From the results of the "base tests", a tailwater rating curve was calculated in Boat Channel downstream of Stephenson's Bend at gauge No. 21 on the model and this curve (Fig.31) was used for all further model tests on the efficiency of the various proposed improvement plans.

Readings were taken over the past two years to establish flood levels in this area but as the largest peak discharge recorded was only 45,100 cusecs in August, 1958 the results are of little benefit. It is hoped that in the future when major floods occur that the tailwater curve calculated above will be checked against actual flood readings at Stephenson's Bend and modified if necessary. Fortunately the results of the model tests (section D6.3) showed that the main factor governing flood levels for major floods in the city of Launceston was the hydraulic capacity of Home Reach above Stephenson's Bend and that small errors made in estimating the tailwater curve for the model would have a negligible effect on flood levels in Launceston for discharges in the South Esk exceeding 150,000 cusecs.

C9 TEST DISCHARGES FOR MODEL STUDIES

The essential purpose of the model studies was to determine for each proposed scheme the relationship between river stage and flood discharge for a range of floods varying from that which first overtops the levees up to the maximum probable. To cover this range, five discharges in the South Esk were selected varying from 100,000 cusecs up to 250,000 cusecs. By comparison of the hydrographs derived for the South and North Esk rivers for the floods of April 1929 and the maximum probable storm, it was computed that the rate of discharge in the North Esk at the time of the peak flow in the South Esk was 13,100 and 18,800 cusecs respectively. To obtain the concurrent flow in the North Esk for the intermediate discharges a straight line interpolation was used.

As it was considered that analysis of future storms may increase slightly the estimated probable maximum flood against which Launceston may warrant protection, an additional flood discharge of 275,000 cusecs in the South Esk with a concurrent flow of 20,200 cusecs in the North Esk, was also tested. Table C1 shows the flood discharges selected for the model tests.

TABLE C4

TEST DISCHARGES FOR THE MODEL STUDIES

Reference No.	South Esk Discharge (cusecs)	North Esk Discharge (cusecs)
A	100,000	10,200
B	125,000	11,700
C	150,000	13,100
D	200,000	16,000
E	250,000	18,800
F	275,000	20,200

C 10 CONCLUSIONS

(i) The best estimates of the peak discharges at Launceston for the 1929 flood are 150,000 cusecs for the South Esk and 20,000 cusecs for the North Esk.

(ii) At the time of peak discharge on the South Esk in 1929 the North Esk discharge approximated 13,100 cusecs.

(iii) Maximization of the 1929 storm on the basis of surface dew points leads to the estimated probable maximum floods of 250,000 cusecs and 34,700 cusecs in the South and North Esk Rivers respectively.

(iv) The South Esk discharge which will commence to overtop the existing levee banks in a manner which cannot be countered by temporary emergency measures depends upon tidal conditions and varies between 87,000 and 114,000 cusecs.

(v) If a quantitative flood forecasting system is developed, a predicted peak discharge of 70,000 cusecs in the South Esk River would justify preparations for emergency measures for sandbagging the low sections of the existing levees.

(vi) So far as the areas behind the present levee banks are concerned, the governing factor in production of flood damage in Launceston is the discharge of the South Esk River.

(vii) Due either to bend losses or constriction in the channel, there is a resistance to flow from River St. to the downstream end of Stephenson's Bend, which causes backing up of flood waters in Home Reach.

(viii) As illustrated in Fig. 29, nine floods have occurred since 1828 which would overtop the existing levee banks under average tidal conditions, the largest being that of 1863 with an estimated peak discharge of 175,000 cusecs in the South Esk River.

(ix) The frequency with which floods of various magnitudes will be equalled or exceeded in the S.Esk River is shown in Fig. 30.

(x) Assuming that a S.Esk discharge of 90,000 cusecs will overtop the existing levees in such a manner as to cause serious damage, then if the past 131 years is a true average sample of the long time behaviour of the river there is a 99 per cent probability (i.e. practical certainty) that during the next 70 years this event will occur at least once.

(xi) On the same assumption, there is a 76 per cent chance that during the next 70 years one or more discharges equal to or greater than that of 1929 will occur.

PART D

HYDRAULIC MODEL STUDIES

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PART D

HYDRAULIC MODEL STUDIES[*]

D1 NEED AND PURPOSE OF MODEL STUDIES

Determination of methods of flood protection by purely analytical means is often a tedious and inexact process and no certainty can be felt that a scheme arrived at by such methods would be entirely satisfactory or the most economical. Upon consideration of the shortcomings of a purely analytical approach to the Launceston flood problem, the desirability of a model study is at once apparent. A carefully designed model provides a ready means for the natural integration of the numerous variables encountered in nature.

The general purpose of the model studies for the protection of Launceston was to determine the hydraulic efficiency of all the various flood mitigation proposals, with the exception of the Cormiston Creek scheme, which lent itself to mathematical analysis not requiring a model study.

More specifically, the model showed the height and location of levees required, in conjunction with other works, to protect Launceston from floods of various magnitudes.

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[*] A fully detailed description and discussion of the model tests is contained in a thesis to be submitted to The University of New South Wales by D.N. Foster B.E., in support of candidature for the degree of Master of Engineering.

D2 DESIGN OF MODEL

D2.1 Design Criteria

For most model studies, similarity between model and prototype must be confined to a single force (inertia, viscosity, surface tension etc.) In the study of flood problems the predominant force controlling fluid motion is that of gravity. Therefore the model is designed to have the same Froudian relationship as the prototype:-

i.e.

$$\frac{g_m L_m}{\dot{V}_m^{\ 2}} = \frac{g_p L_p}{V_p^{\ 2}}$$

where g = acceleration due to gravity in ft./sec.^{2}

" L = linear dimension in ft.

" V = velocity in ft./sec.

where subscript "m" refers to the model and subscript "p" to the prototype.

The basic criterion used in the design of such models is that they must be capable of reproducing accurately the relationship between flood stages and discharges. Where the investigation involves only such phenomena as channel capacity, use of storage etc., with no special need for an exact reproduction of velocity distributions, paths of flow of corresponding water particles, wave patterns etc., a distorted model can be used. In fact it is generally necessary to have geometrical distortion in order to ensure that flows in the model will be turbulent for all discharges and also that the model can achieve a roughness to enable reproduction of the prototype.

The requirement of this type of model is that the energy gradient must be at the same relative elevation above the water surface in the model as in the prototype. It can be shown that to fulfill this condition the velocity scale of the model must equal the square root of the vertical scale.

i.e. $V_r = Y_r^{\frac{1}{2}}$

where V_r = velocity scale

Y_r = linear vertical scale

After consideration of such factors as required accuracy, time available, cost, available space and water supply, a horizontal model scale of 1:500 was selected. Once the horizontal scale had been fixed, a vertical scale was calculated so that the channel roughness of the Tamar River would be approximately reproduced in the model by cement mortar with a wooden float finish. The vertical scale based on this criterion was 1:100 resulting in a geometric distortion of 5 to 1. A check on the hydraulic characteristics for this scale indicated that turbulent flow would prevail for the full range of test discharges.

Once the linear scales have been selected, other model scales are automatically fixed by the constant Froudian relationship as follows:-

Horizontal dimensions = 1:500 x_r

Vertical dimensions = 1:100 y_r

Velocity = 1:10 $v_r = y_r^{\frac{1}{2}}$

Discharge = 1:500,000 $Q_r = v_r y_r x_r$

Time = 1:50 $T_r = \frac{x_r}{v_r}$

D2.2 Area Reproduced in the Model

The model reproduces 5.0 miles of the North Esk River from Hobler's Bridge to its junction with the Tamar River, 1.0 miles of the South Esk River from 1000 ft. above the first Basin Suspension Bridge to its junction with the Tamar, and 4.5 miles of the Tamar River from its source to a point 1000 ft. downstream of the Hunter Cut. Sufficient overbank area was incorporated on both sides of the three streams to encompass all areas subject to flooding. Model boundaries are shown in Fig. 3. The area of the model was approximately 696 sq. ft. representing 6½ sq. miles in the prototype.

D3 CONSTRUCTION OF MODEL

D3.1 Configuration

Initially the model was constructed to conform as closely as possible with topographic conditions existing in 1929 in order that the model might be verified by the use of the hydraulic data available regarding the 1929 flood. Details of the survey work involved in establishing this configuration are given in Part B of this report.

Following upon the verification of the model, the contours were altered to conform with topography existing in 1958. The main changes were (i) the embodying of the Hydro-Electric Commission tailrace from Trevallyn Power Station and the deposit areas and embankments used to dispose of the spoil from this excavation (ii) the Launceston Marine Board silt deposit areas and embankments for disposal of dredgings from the Tamar. It will be noted from the discussion in Part B that the data showing the cross-sections of the channel of the Tamar River were meagre, so that for the verification tests the Tamar River channel was moulded from its sources to No.6 beacon at Stephenson's Bend according to the hydrographic survey of 1937. Below Stephenson's Bend no survey data for the earlier years were available and the channel was moulded in the model to conform with hydrographic data obtained in 1958. It might be considered that for the verification test the channel should have been moulded in the form existing in 1929 and, for the tests of various mitigation schemes it should have accorded with conditions existing in 1958. In view of the inadequate data, this was impossible. However, there was little difference in available cross sectional area for flow between the 1936 surveys and the 1958 conditions above Stephenson's Bend. Also there was some evidence to indicate that although the location of silt deposits in the Tamar vary over the years the total available cross sectional area for flow of flood waters is reasonably constant.

Consideration was given to further changes in topography which will occur in the reasonably near future. Where these changes were likely to affect flow conditions they were incorporated in the model, but if they had no effect on floods they were not included. For example, the new arterial road from Henry Street to Royal Park and the projected new railway marshalling yards were not moulded, but the Marine Board reclamation areas for Tamar dredgings were included to the final level planned by that Authority. The arterial road and railway yards are located in back-water areas and their effect on flood levels would be negligible.

D3.2 Method of Construction

The model is of the "fixed bed" type and was made of ash concrete within a brick surrounding wall, with the entire surface area moulded in cement mortar with a wooden float finish. Horizontal control was established by means of a 2 ft. grid based on brass plugs around the boundary wall. Vertical control was established by the use of a dumpy level from a fixed base on a brick pier. The river channels were moulded by making masonite templates of measured river cross sections at 500 ft. intervals and trowelling between the templates. The overbank storage areas were shaped by driving pins to the appropriate ground level as indicated on the contour plan and bringing the mortar surface level with the top of the pins.

D3.3 Appurtenances

Provision was made for supply of water to the model by means of a 6" pipe for the South Esk river and a 3" pipe for the North Esk. On each of these pipes orifice meters were installed, these instruments being calibrated by the British Standard Flow Code BS1042 theoretical curves and were checked at several points experimentally by weighing the discharge over a given time.

The out-flow from the model was controlled by an adjustable tail-water gate. In the Launceston problem the peak flood discharge for major floods is for practical purposes constant for a fairly long period, so that it was only necessary to pass uniform flood discharges through the model without giving any consideration to the difference between the in-flow and the out-flow hydrograph, caused by storage effects. In other words, the peak discharge of the flood hydrograph is constant for a sufficient length of time to ensure uniform flow.

The flood heights in the river channels were measured by means of manometer tubes and point gauges at 22 fixed points. For the critical locations in the overbank areas, water surface was measured directly by means of point gauges operating below a fixed datum established by a stop on a vertical brass rod set in the surface of the model.

As an 1/8" vertical change in level on the model represents approximately one foot under prototype conditions, it was important to measure water levels accurately. Therefore, special point gauges were constructed consisting of a pointed brass rod tapped into the end of a D.C.M.T. micrometer. Although the readings could be made to 1/1000 of an inch the micrometers were of an inexpensive die-cast construction and the order of accuracy of the readings was to 5/1000 of an inch. Eight of these instruments were manufactured for use during testing. Velocities of flow in the model were measured by an Ott Midget Current Meter.

The correct roughness of the model surface was obtained by the following means:-

(a) Stucco in the river beds.

(b) Plywood rectangles with $\frac{1}{2}$" perforations in the sides to represent the buildings in the overbank areas in zones of low velocity.

(c) In areas of high velocity the buildings were simulated in timber as a replica of the prototype.

(d) Trees and vegetation were reproduced in the model by domestic copper pot cleaners.

(e) Bridges were modelled to accord with their prototype dimensions with the piers represented by welding rods to the model scale.

D4 OPERATION OF MODEL

The operation of the model varied somewhat according to the nature of the proposal being investigated, but the basic steps in any of the model tests may be summarised as follows:-

(a) The outlet of the model was sealed off and the model filled to a constant depth with water. The gauge zeros were then checked by reading the still water level by the micrometers. If necessary, the micrometer tubes were cleaned by blowing them out with a car pump.

(b) The gate at the outlet of the model was removed and the test in-flows of the North and South Esk rivers set by adjusting the differential pressure at the orifice meters to correspond to that shown by the calibration curve.

(c) The flow was allowed to reach equilibrium and the tail water gate adjusted at the downstream end of the model so that the flood level in Boat Channel corresponded to that shown on the computed tail water curve for the test discharge. (See Part C Section C8).

(d) The model was allowed sufficient time to reach stability and the flood heights were measured by the micrometer point gauges at 22 points within the river channel and at 18 critical points on the overbank area.

(e) Flow patterns in the model were observed visually and velocity readings taken as required by the use of the midget current meter.

D5 VERIFICATION OF MODEL

D5.1 Reason for Verification

Owing to the wide variety and type of roughness in the prototype it is impossible to fix theoretically the model roughness with any degree of accuracy. In addition, for a distorted model, the hydraulic mean radius at all points within the channel has a varying linear scale and the ratio of model roughness to the prototype roughness would differ for each section. Therefore, it is essential that the model be verified, and adjusted for roughness where necessary, to ensure that it reproduces accurately the stage discharge relationships of the prototype, which in turn is dependent upon similitude of the model to the corresponding prototype channel and overbank roughness. The principle of verification is that if the model can be adjusted to reproduce accurately past events it will also reproduce future events.

D5.2 Limitation of Verification

The main limitations of the verification process are as follows:-

1. They must involve phenomena of the type pertinent to the proposed study. This means that the Launceston model must be capable of reproducing accurately the stage discharge relationships during a flood.

2. The event in the prototype on which model verification is based must represent a continuous action of reasonable duration.

3. The verification data should be of the same order of magnitude as the events being tested. The more the phenomena tested in the model depart from the conditions of verification the less trustworthy will be the results.

D5.3 Method of Verification

Verification of models of this type is essentially a trial and error process, the model being set up to represent conditions at the time of the event upon which the verification is based, and then adjusted so that the prototype conditions for that event are reproduced in the model. Elements that may be adjusted are the discharge scale, roughness and channel configuration. For this particular investigation an adjustment of roughness only was found to give satisfactory results.

The flood used for verifying the model was that of April 1929, the only major flood in the present century. The available data on flood heights in 1929 were particularly good, by reason of the prompt action of the City Engineer at the time in arranging for trash line surveys immediately after the flood. Unfortunately, this data collection did not extend below Stephenson's Bend, and one of the difficulties in the investigation was the lack of really detailed information regarding the rise and fall in levels in the wide expanse of river above Tamar Island. In regard to the flood discharges occurring in 1929, no reliable measurements were made. Part C of this report gives details of the method whereby it was estimated that the peak discharge in 1929 was 150,000 cusecs in the South Esk with a concurrent flow of 13,100 cusecs in the North Esk. The storage volume of the Inveresk-Invermay and other overbank areas in 1929 was approximately 12,000 acre feet as compared with an approximate total flood discharge of 940,000 acre feet, thus indicating that the storage volume was negligible and that uniform flow conditions could be assumed in the model test. In other words, the peak flood discharge lasted long enough in 1929 to establish uniform flow conditions with a relatively long period of no in-flow into overbank storage. A corollary of this observation is that the time required to fill the overbank storage in the prototype was relatively small compared with the duration of the peak discharge. For these reasons it was possible to carry out the model test with uniform flow only.

Clear evidence existed that in the 1929 flood the river levels in the vicinity of Launceston were not affected by the rise and fall of the tide, although a few miles downstream there was a considerable tidal range. This indicates that the channel capacity in Home Reach and around Stephenson's Bend is the main factor governing flood levels in Launceston for major floods. This meant that no expenditure was necessary on a tide machine to reproduce tidal movement in the model. From the discussion on tidal effects in Part C of this report it is clear that for discharges below say 130,000 cusecs the model will not represent with complete accuracy prototype conditions because of the omission of a tide machine from the installation. However, the testing of various flood mitigation proposals is only required for major floods where tidal influence is negligible. Use of the model to decide levee heights etc., for protection against minor floods, would require an appropriate adjustment of the test results to allow for tidal influence.

The steps in the verification of the model were as follows:-

(a) Flood discharges of 150,000 cusecs in the South Esk and 13,100 cusecs in the North Esk were run through the model and the tail gate adjusted so that the model level was correct on gauge No.18. (See Fig. 32), opposite the Grammar School Rowing Club House where the 1929 flood level had been established at 16.8 ft. above S.L.W. The only roughness incorporated for this test were roughness rectangles located over Inveresk-Invermay and the city, together with the roughness provided by the mortar finish in the channel, and the reproduction of the wharf sheds on King's Wharf. For this test, flood levels and flow patterns were studied over the entire model.

(b) These results showed the following discrepancies between model and prototype:-

(i) Flood levels in the South Esk gorge were much too low.

(ii) The jet action at the outlet of South Esk was too powerful, sweeping strongly across Royal Park in a manner which did not accord with conditions indicated by the photographs and movie films of the 1929 flood.

(iii) With correct levels at Stephenson's Bend, flood levels around Home Reach and along the West Bank of the Tamar River were 1 ft. to 1.5 ft. too low.

(iv) A study of the water surface gradient indicated slopes close to the truth from King's Wharf to River Street, but with too flat a grade from River Street to Stephenson's Bend, indicating that there was not enough resistance to flow in this area.

(v) The reason for this could be that the model was not reproducing adequately either the bend losses, river channel roughness or river constrictions.

(vi) Flood levels over Inveresk and Invermay were closely reproduced, as flooding in this area was due to the high velocity jet from the South Esk shooting across Royal Park and overtopping the levees protecting Inveresk and Invermay. Once this area was flooded there was little overbank flow. Hence the flood levels in the storage basin thus formed depended upon the resistance of the levees in allowing the water to flow back into the river channels. For this reason a close agreement would be expected between flood levels in the model, and those which occurred in the prototype, because a one foot difference in flood levels in Home Reach would have very little effect on the flood discharge characteristics over the levee banks from the storage basin. In addition, this area was the only section of the model in which prototype roughness had been simulated by the addition of roughness rectangles to the surface of the model.

D5.4 Adjustment of Model

It was felt that appropriate adjustment of roughness in different areas of the model would remedy the discrepancies between model and prototype indicated in this first verification test. An important change was a very great increase in the roughness of the South Esk gorge which was achieved by cementing to the side of the channel 3/4" crushed basalt screenings. This increase in roughness resulted in the behaviour of the jet downstream of Cataract Bridge corresponding closely to the flow conditions and flood heights in the prototype.

Roughness was also increased by simulating trees along the West Tamar Bank and Stephenson's Bend. Also, channel roughness across the shallow portions of the lower end of Stephenson's Bend was increased by one roughness rectangle.

As the North Esk River, under major flood conditions, is mainly a back-water area, no adjustment of this river channel was necessary.

The results of the final verification test are shown in Fig.33. An examination of this Figure shows that a very good verification was obtained, the flood levels being reproduced in the model to within 2" of the prototype conditions with the exception of gauges Nos. 27 and 23. So far as gauge No. 23 is concerned, near Hobler's Bridge, the prototype flood level was taken on the downstream side of Hobler's Bridge Road which in times of flood acts as a weir banking up the North Esk flood waters. Flow over this road was consequently very turbulent and a considerable amount of wave action exists. Therefore a flood level established by trash line survey would be too high. Furthermore, this flood trash line level might have been fixed by the peak discharge of the North Esk River of 19,000 cusecs before the peak of the South Esk had reached the Tamar.

Gauge No.27 is located at a point where severe side eddies from the South Esk jet occur and the trash line would probably be at the top of any surge or wave action. In the final verification test the top of the wave action of gauge No.27 was measured to reach a value of 20 ft. in comparison with the trash level of 20.2 ft.

Summing up, the final verification test could be considered extremely satisfactory.

D5.5 Diversion of South Esk Jet.

An important feature of the verification test was the fact that it demonstrated clearly the manner in which the suburbs of Inveresk and Invermay are flooded and as a result brought into consideration a unique method of flood control which would not have been apparent without the construction of a model or the careful technical observation of a major prototype flood.

The South Esk River at its junction with the Tamar issues forth from under Cataract Bridge as a high velocity jet almost at right angles to the normal direction of flow in Home Reach. The stilling action of the deeper and wider Tamar River was insufficient to dissipate the energy of this jet before it reaches Royal Park, where it over flows the banks and continues at a high velocity to overtop existing Inveresk and Invermay levees near Charles Street Bridge, flooding the storage areas behind the levees.

It is apparent, therefore, that if this jet could be diverted by some means so that the high velocity flow is turned in a northerly direction down Home Reach, a material reduction in flood levels in the built up areas of Launceston would be obtained.

As a result of this demonstration by the model, it was decided to increase the originally planned series of tests to provide for an additional flood protection measure consisting of a training levee constructed in Royal Park to control the direction of flow in this area.

D6 TESTS AND RESULTS

D6.0 Introduction

Once the model had been verified, showing that it was capable of reproducing accurately the stage discharge relationship of the prototype, a series of tests were made to determine the efficiency of the various proposed improvement plans.

Hydrologic and economic studies were carried out to provide flood frequency and flood stage-flood damage graphs. (See Parts C and E). The hydraulic studies in essence were aimed at providing for each scheme a relationship between flood discharge and flood stage. Such data leads to the plotting of the cost-benefit graphs and ultimately the selection of a "design flood", (the flood magnitude against which the recommended works will protect Launceston).

In Part C, Section C7.22, the effect of tidal conditions on flood heights reached in the vicinity of the city for different river discharges up to 150,000 cusecs, were discussed. The basic assumption made in all model testing was that tidal conditions at the time of occurrence of a flood will be average tidal conditions. On this basis, test discharges were selected in the model so as to define adequately the stage discharge curve for floods varying in magnitude from that for which the levees are first overtopped (South Esk discharge 90,000 cusecs North Esk discharge 9,600 cusecs) up to the maximum probable flood (South Esk discharge 250,000 cusecs. North Esk discharge 18,800 cusecs). Test discharges were as follows:-

Reference No.	South Esk Discharge (cusecs)	North Esk Discharge (cusecs)		Remarks
A	100,000	10,200		
B	125,000	11,700		
C	150,000	13,100	-	1929 flood
D	200,000	16,000		
E	250,000	18,800	-	Maximum probable flood
F	275,000	20,200		

Details of the method by which these flows were fixed is given in Part C.

The detailed results of the model tests showing the relationship between flood stage and flood discharges at each of the 40 gauge points for all tests are shown in Tables 7 to 12.

In these tables, and throughout the discussion of the model studies, all model results have been expressed in their prototype equivalent.

For the purpose of rapidly comparing and evaluating the overall efficiency of the various proposed improvement plans, the variation of flood levels at Tamar St. Bridge for all flood discharges are shown in Figs. 34 and 35 for each plan tested on the model. In addition, the results of the model tests have been summarised in Table D1 included in this Section. This summary is divided into two critical areas as follows:-

(i) Home Reach at the northern end of King's Wharf.

(ii) The railway bridge across the North Esk River.

The railway bridge is situated in a backwater storage area and the flood levels at this point can be considered to be representative of the flood level over the major portion of the flood plains of Inveresk-Invermay, Mowbray Flats, City, Dowling Street and the upper North Esk, ranging as far as Hobler's Bridge.

The simplified summary in this Table, is of course, only a general guide. For correct fixing of the flood levels and required levee heights, reference must be made to Tables 7 to 12.

Flow patterns and velocities at the upstream end of Home Reach and the lower portion of the North Esk after construction of the Royal Park training levee are shown in Table No. 14. Velocities in Home Reach at the northern end of King's wharf for 110,000 cusecs are given in Table No. 15.

D6.1 Base Tests (Test Series 1)

The term "base tests" is used in model investigation to denote tests carried out on the existing prototype topography. The purpose of these tests is to obtain basic data for use as a reference in comparing hydraulic efficiency of various proposed improvement plans.

TABLE No. D1

SUMMARY OF MODEL RESULTS
FLOOD LEVELS AT KINGS WHARF AND RAILWAY BRIDGE

Test No.	South Esk Discharge =	100,000 c.f.s.		125,000 c.f.s.		150,000 c.f.s.		200,000 c.f.s.		250,000 c.f.s.		275,000 c.f.s.	
	Elements of Proposed Improvement Plan	Flood Levels above S.L.W. at											
		King's Wharf	Railway Bridge	King's Wharf	Railway Bridge	King's Wharf	Railway Bridge	King's Wharf	Railway Bridge	King's Wharf	Railway Bridge	King's Wharf	Railway Bridge
0	Verification Test - 1929 Flood	-	-	-	-		19·7	-	-	-	-	-	-
1	Base Tests - 1958 Topography no flood protection.	16·0	16·7	17·5	18·4	19·0	20·7	21·6	23·0	23·5	25·0	24·3	25·8
2	Tidal influence levels in Boat Channel raised by one foot	17·0	17·6	18·3	19·0	19·5	20·8	22·0	23·2	23·8	25·1	24·6	25·9
	<u>LEVEE PROTECTION ONLY</u>												
3	Training Levee only	16·0	16·7	17·5	18·3	18·5	19·0	20·3	20·4	21·9	21·4	22·7	21·8
4	Inveresk and Invermay Levees only	16·2	17·0	17·3	18·5	18·8	21·1	21·9	26·3	24·0	31·5	Not	Tested
5	Training Levee with Inveresk and Invermay Levees	Not	Run	17·4	18·0	18·4	18·5	20·5	20·9	22·8	22·7	23·7	23·9
6	Training Levee with Inveresk and Invermay and also City Levees	16·4	17·1	17·6	18·8	18·9	19·7	21·4	22·3	23·7	23·6	24·6	24·8
7	Repeat of Test 6 but with the alignment of the levees slightly altered to conform with the prototype survey of 1959	16·4	17·1	17·6	18·6	18·9	19·7	21·2	21·9	23·4	23·8	24·3	24·5
	<u>TRAINING LEVEE WITH INVERESK AND INVERMAY AND CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS</u>												
8	Floodway at Stephenson's Bend by removal of levees around silt deposit areas	16·5	17·1	17·3	18·2	18·4	19·1	21·2	21·7	23·5	23·6	24·1	24·1
9	Floodway at Stephenson's Bend by removal of silt deposit levees and removal of vegetation along river banks such as Ti tree etc.	16·3	17·2	17·5	18·3	18·8	19·6	21·5	21·8	23·2	23·4	24·0	24·0
10	Floodway as for Test 9 but with overbank area roughened by the addition of $\frac{3}{16}$" gravel to allow for deterioration in efficiency with time by the growth of vegetation	16·4	17·1	17·6	18·3	18·9	19·6	Not	Tested	23·4	23·4	24·2	24·3
11	Floodway at Stephenson's Bend formed by construction of a channel 5 feet deep (Bed level RL7) and 550 feet wide along line of Hunter Cut	15·9	16·5	17·1	17·9	18·2	19·1	20·7	21·4	23·2	23·6	24·3	24·5
12	Bend opp. tailrace realigned by dredging to reduce curvature and widen the river from 285 ft. to 510 ft. at S.L.W.	16·1	16·8	Not	Tested	18·5	19·3	Not	Tested	23·1	23·4	Not	Tested
13	Floodway 5 ft. deep and 550 ft. wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp. tailrace as for Test 12	16·0	16·6	Not	Tested	18·1	18·8	Not	Tested	22·5	23·0	Not	Tested
14	Hunter Navigation Cut 28 ft. deep (Bed RL 16) bottom width 231 ft.	15·3	16·1	16·5	17·5	18·0	18·4	19·8	20·8	22·4	23·1	23·5	24·0
	<u>SUBSIDIARY TESTS</u>												
15	Inveresk and Invermay, City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	16·3	16·9	Not	Tested	18·5	19·2	Not	Tested	23·0	23·6	Not	Tested
16	Inveresk and Invermay, City, Paterson Street and West Tamar Bank levees together with levees [illegible] feet from river edge on each side of North Esk below Cypress Street.	16·2	16·8	Not	Tested	18·6	19·3	21·1	21·7	23·1	23·7	Not	Tested

For this test the model topography was altered from that existing in 1929 to correspond with the latest survey data and proposed future developments which are likely to effect the flood heights in the area. The main changes were as follows:-

(i) Incorporation of the Trevallyn tail-race channel and silt deposit areas with their embankments on the western side of the Hunter Cut.

(ii) The Marine Board dredging deposit areas and embankments on the eastern side of the Hunter Cut and on the southern bank of Stephenson's Bend.

(iii) Increased building development in Inveresk-Invermay area.

(iv) Thinning out of Ti-tree growth in the vicinity of the Hunter Cut.

(v) The levee at Paterson Street protecting the Margaret Street area from flooding was raised after the 1929 flood from R.L. 18.6 to R.L.23. However, owing to the local high flood stages in the near vicinity of the South Esk jet and the excessive wave action resulting from the very turbulent water in this area, this levee will only protect the zone from floods up to 125,000 cusecs. For a greater flood discharge the levee will be overtopped and flooding will occur. It was obvious that the construction at a small cost of a short low levee from Ritchies Mill to the L.C.C. Sewage pumping station would protect the high developed residential areas behind the levee against the maximum probable flood. The height of levee required for floods of various magnitudes can be obtained from Table 13.

As the storage in this area is extremely small and flooding is the result of backwater only the construction of this levee would have no effect on flood levels downstream. Consequently, for all model tests carried out with various proposed improvement plans it has been assumed that this levee has been built to protect the area from the maximum probable flood.

Results of these base tests are shown in Table D1. This shows that changes in topography since 1929 have had the effect of restricting the outlet for flood waters from the storage area of Inveresk-Invermay. If a flood now occurred equal to the 1929 discharge the levels in this area would be 0.8 ft. higher than actually occurred in 1929. Although in 1929 the discharge across Inveresk-Invermay was insignificant compared with the discharge down Home Reach, nevertheless the height attained by flood waters in such a storage area is governed by the boundary out-flow conditions. When the Marine Board silt deposit areas are finally completed to their planned level there will be an increase in the out-flow resistance from the storage area, and this effect is expressed in the Table D1.

D6.2 Tidal Influence (Test Series 2)

The problem of tidal effects is fully discussed in Section C7.22 of Part C. For major floods the tide level in Boat Channel must have some effect on the rate of discharge of flood waters around Stephenson's Bend. Therefore, the next test carried out on the model was designed to measure the magnitude of this effect. Fig. 31 of Vol. II gives the tailwater rating graph for Boat Channel and the level for 1929 conditions was 15.5 ft. On the model the tailwater gate was set at such a point as to give a level in Boat Channel of 1 ft. higher, namely, 16.5 ft.

The results showed that for discharges of 100,000 cusecs and lower a change of one foot in the tailwater level at Boat Channel as a result of tidal resistance downstream or errors in computing the correct tailwater level would result in errors of one foot in flood levels above the tailrace channel. For discharges greater than 100,000 cusecs the tailwater level has a diminishing effect on flood levels in Home Reach and Inveresk-Invermay and for a one foot rise in Boat Channel the corresponding increase in flood levels in Home Reach would be 0.50 and 0.3 ft. for a discharge in the South Esk of 150,000 cusecs and 250,000 cusecs respectively. In Inveresk-Invermay the corresponding increases in flood levels were 0.1 of a foot for both discharges.

This test demonstrates that the main factor governing flood levels for major floods in the city of Launceston is the hydraulic capacity of Home Reach and Stephenson's Bend, and that small errors made in estimating the tailwater curve for the model and the changes in level that may result in Boat Channel from tidal resistance will have a negligible effect on flood levels in Launceston for discharges in the South Esk of 150,000 cusecs and greater.

D6.3 Levee Protection Only

D6.30 Introduction. For each proposal, such as a floodway, it is necessary to determine the height of the levee which must be used in conjunction with the floodway to protect Inveresk-Invermay and the city against floods of various magnitudes. This can readily be done by erecting on the model a vertical wall of ample height along the line of the levee, and measuring the flood heights reached for various discharges.

However, it will be necessary also to consider such cases as a floodway combined with raising the present levees by 2 ft., 4 ft., etc. Some of the floods will overtop such levees, but the depth of flooding would be different from that which would occur under existing conditions and the depth must be known in order to use the stage damage curves to compute the annual benefits of each particular combination.

The moulding of a levee exactly two feet higher than the present levees is a laborious task and would have to be repeated two or three times at two foot intervals. It would be much quicker and cheaper if the levels reached on the vertical wall are considered to be the same as the height reached in the Inveresk-Invermay area when the hypothetical levees are overtopped. This is theoretically incorrect, because when the levee is overtopped the height in the river channels will be lower due to storage and overbank flow. However, if the maximum effect of such storage and overbank flow is only a few inches change in level in Home Reach this error can be neglected and the unassailable levees used throughout the test with consequent economy. As far as storage volume is concerned, discussed in Section 5.3 of this Part, it has already been pointed out that the volume of such storage is negligible. The model test confirmed this by the very short time necessary to fill this storage.

From the discussion of test series 5 later in this section, it will be observed that overbank discharge effects across the Inveresk-Invermay zone are small, and, for the purpose of computing flood damage, the effect can be approximated with sufficient accuracy by interpolation as shown in Fig. 45.

It was therefore decided that unassailable levees would be used throughout the tests. These levees were represented on the model by 26 gauge flat galvanized steel sheets 2" wide (prototype 16 ft. 8 ins.) fixed to the model by screwing foot brackets into plugs in the surface. Leakage under the galvanized iron was prevented by sealing with modelling clay.

D6.31 Training Levee Only (Test Series 3). From observation on the model during verification and base tests it was apparent that the main cause of flooding of Inveresk-Invermay and the Esplanade areas by discharges exceeding 140,000 cusecs was the jet action of the flow from Cataract Gorge as previously described. Below 140,000 cusecs the river bank at Royal Park was high enough to deflect the main force of the jet down Home Reach. Therefore, an unassailable training levee 1600 ft. long was placed on the model in Royal Park as shown on Fig. No. 37. The purpose of this levee was to deflect the jet from the South Esk in a northerly direction down Home Reach. The results may be summarised by stating that a major reduction in flood levels could be obtained in the flooded areas of Launceston for floods exceeding 140,000 cusecs, the reduction in flood stage in Inveresk-Invermay being 1.7 ft. for the 1929 flood and 3.6 ft. for the maximum probable flood. Flood levels in Home Reach were also reduced, but not to the same extent, being 0.5 ft. for the 1929 flood and 1.9 ft. for the maximum probable flood.

In all tests the training levee was constructed in the model so that it would not be overtopped by wave action and the height of levee required to satisfy this condition is given in Fig. 36.

It is possible that satisfactory efficiency in jet diversion can be obtained by the use of an overtoppable training levee. Under flood conditions there will be backwater of considerable height at the rear of the training levee so that flooding over the levee would not be harmful. Therefore a subsequent test will later be carried out to indicate the minimum height of training levee necessary to achieve a satisfactory efficiency in jet diversion and the results will be embodied in a supplementary report.

D6.32 Levees Around Inveresk-Invermay and Mowbray Flats only (Test Series 4). Levees were placed throughout Inveresk-Invermay and Mowbray Flats in a manner somewhat similar to the locations shown on Fig. No. 37. The base discharges listed in Section C9 were run through the model and flood heights measured at the 40 gauges, thus providing information regarding the heights of levees required to protect Inveresk-Invermay from flooding. From Tables 7 to 12 it will be observed that the flood waters from the South Esk jet, being unable to escape across Inveresk-Invermay, were forced by the jet action up the North Esk into storage until a sufficient hydraulic head was built up to allow the flow from the North Esk to counterbalance this flow into storage. For the 1929 discharge, flood stages in the city and upper North Esk areas were increased by 0.4 ft. and for the maximum probable flood the increase was 6.5 ft.

The results stressed the great importance of the Royal Park training levee and provided a stern warning that construction of levees to protect Inveresk-Invermay from floods greater than 150,000 cusecs should not be contemplated without first constructing the training levee, because Inveresk-Invermay levees alone would result in much more extensive flooding of the city and North Esk areas than would occur under 1958 topographic conditions.

D6.33 Inveresk-Invermay and Mowbray Flats Levees Combined with Royal Park Training Levee (Test Series 5). Once the training levee has been constructed, Inveresk-Invermay levees can be raised without fear of increasing flood levels in the city and North Esk zones. The only increase in flood level which may occur would be as a result of confining a flood discharge to the river channels by restricting overbank flow.

Leaving in position the ring levees around Inveresk-Invermay used for test series 4, the training levee in Royal Park was added to the model. The base discharges were fed through the model and flood heights measured.

A comparison of the results for this test and test series 3 show that for flood magnitudes less than 150,000 cusecs, overbank flow is negligible and there is no increase in flood levels. For discharges greater than 150,000 cusecs some overbank flow does occur and flood levels are increased by construction of levees around Inveresk-Invermay. The increase in flood stage is 0.2 ft. in Home Reach and 0.5 ft. along the North Esk for South Esk flood discharges of 200,000 cusecs. For the maximum probable flood the corresponding increases are 0.9 ft. and 1.3 ft. respectively.

D6.34 City and Inveresk-Invermay Levees with Royal Park Training Levee (Test Series 6). Series 5 was repeated with levees from Royal Park along the Esplanade and main railway line to Cypress Street generally, although not exactly, as shown in Fig. No. 37.

If the overbank flow for the North Esk at its outlet to the Tamar is confined entirely within its banks by the constriction of both the city and Inveresk-Invermay levees, flood levels will be increased slightly, being in general one foot higher than those recorded for levee protection of Inveresk-Invermay only.

It should be noted, however, that even though flood levels are increased by the construction of levee banks, flood stages in Home Reach and the Upper North Esk areas are lower than for the base tests due to the efficiency of the training levee in mitigating floods. Flood levels measured for this test in the Upper North Esk area were 19.8 for 150,000 cusecs and 23.7 for 250,000 cusecs, which are 1.0 ft. and 1.4 ft. lower respectively than would occur if no flood protection measures were undertaken. This means that areas outside those protected by the proposed levees would not be inundated to any greater extent by the construction of such levees and will be flooded to smaller depths than would otherwise result.

D6.35 Re-Alignment of the Levees of Test Series 6 to Conform with Final Alignment adopted as a result of a Prototype Survey in January, 1959. (Test Series 7). The detailed surveys and preliminary designs of levees were carried out in January and February, 1959, and during this survey and design work certain improvements in levee location appeared desirable. The final location is shown in Fig. No.37 of Vol. II. It was necessary to determine the change in flood levels, if any, caused by such re-alignment of the levee banks. The levees on the model were therefore re-located and test series 6 repeated.

The results of this test showed no significant change in flood levels as a result of realigning the proposed levees. Further, small modifications of the alignment of the levees that may be necessary in the construction stage will also have a negligible effect on the general flood levels, but it should be noted that if the centre line of the levee is at right angles to the direction of flow the impact of the water against the levee and the resultant wave action will result in local flood stages higher than the general flood level. This should be avoided wherever possible. Should it be impracticable to do so, additional freeboard should be allowed on the levee to prevent local failure.

D6.4 River Improvement

D6.40 Introduction. From a study of hydrologic evidence of the 1929 flood it was suspected that much of the flooding of Launceston was due to the restriction of flood flows around Stephenson's Bend where the river channel makes two sharp turns before reaching the section of the Tamar downstream of the Hunter Cut known as Boat Channel, where the river widens appreciably. The verification test and test series (1) confirmed this suspicion. Therefore a series of tests were planned to determine the reduction in flood levels and the consequent reduction of heights of levees required to protect Launceston when the restriction to flood flows around Stephenson's Bend were relieved by various types of river improvement.

For the whole of this series of tests the levees of test series 7 were incorporated in the model. The results are given in Tables No. 7 to 12 and have been summarised in Figs. 34 and 35 and Table No. D1.

D6.41 Removal of Old Levee Banks (Test Series 8). When the construction of the Hunter Cut was commenced, rectangular areas east of the Cut were enclosed by embankments to form silt deposit areas for excavation from the Cut. These banks restrict overbank flow for major floods across the river flats in this area and the purpose of this test series was to determine reduction of flood levels achieved by their removal.

The model showed that the benefits from such a measure are negligible.

D6.42 Removal of Ti-trees (Test Series 9). In addition to the restriction to overbank flow by the levee banks in the Stephenson's Bend area, dense Ti-tree along the foreshores of the river tend to limit velocities in this zone and test series 8 was run to show the reduction in flood levels by the removal of this vegetation.

The model showed that the benefits from such a measure are negligible.

D6.43 Future Deterioration of Floodway (Test Series 10). This test was the same as test series 8, but as it was felt that the efficiency of the floodway thus formed would deteriorate with time by the growth of vegetation, test series 9 was run to determine the effect of overbank roughness in this area. The model roughness was increased by the addition of 1/4" gravel joined to the surface of the model by cement mortar.

The model showed that any deterioration of a flooding by vegetation growth would have little effect.

D6.44 Shallow Floodway along alignment of Hunter Cut (Test Series 11). An alternative floodway to provide relief of flood waters at Stephenson's Bend would be to construct a shallow channel along the alignment of the Hunter Cut. This test series was therefore designed to determine the reduction in flood levels produced by a 5 ft. deep channel (bed level RL.7) and 550 ft. wide in this area.

The result of this test showed that a floodway of this nature would be more effective than one formed by the removal of the levee banks on the eastern side of the Hunter Cut. However, the reduction of flood stage in Launceston and the resultant saving of levee cost is only small, the reduction in flood level being approximately 0.5 ft. for flood discharges of 200,000 cusecs and lower. For the maximum probable flood, the reduction in flood levels is negligible.

D6.45 Widening of Channel at Entrance to Stephenson's Bend (Test Series 12). As an alternative to a floodway, bend losses could be decreased and the discharge capacity of the Tamar River at Stephenson's Bend improved by cutting away the bank opposite the tailrace channel. The purpose of test series 11 was to show the effect the flood levels by widening the river and reducing the curvature on the bend opposite the tailrace channel by dredging in this zone.

This reduction in curvature of the bend will lower flood levels in Launceston and the consequent height of levee banks by 0.5 ft. for discharges up to 200,000 cusecs. For greater floods the reduction in flood levels is negligible. The small improvement for major floods is due to the fact that considerable overbank flow occurs at this point for these floods in any case, so that the effect of widening the river channel has no appreciable effect.

If this flood mitigation measure were carried out, it should be borne in mind that maintenance costs would be high, as the natural siltation of the river during river freshes will be towards deposition on the inside of the bend which has been widened by dredging.

D6.46 Shallow Floodway Combined with River Widening (Test Series 13). This test combined the essential features of test series 11 and 12 by incorporating in the model the channel improvements of re-aligning the bend opposite the tailrace, together with a 550 ft. wide and 5 ft. deep floodway following the line of the Hunter Cut.

The results demonstrated that further reduction in flood levels could be achieved by combining both the floodway and channel improvements. The reduction in levee heights as compared with that required with no channel improvements were 0.9 ft. for 150,000 cusecs in the South Esk River and 0.8 ft. for the maximum probable flood. However, it is doubtful whether expenditure involved in these mitigation measures is justified by the benefits.

D6.47 Completion of Hunter Navigation Cut (Test Series 14). For navigation of the Tamar River, Stephenson's Bend provides one of the worst hazards in the river. In 1911 Hunter was consulted by the L.M.B. to advise on methods of improving navigation in the estuary and one of his proposals was to construct a straight navigation channel across the swamp adjacent to the bend. This cut, which was 28 ft. deep and had a bed width of 231 ft. with 1 to 4.8 side slopes, has subsequently become known as the "Hunter Cut" and although started in 1916 has never been completed.

Beside improving navigation of the estuary this Cut would shorten the river by 3,000 ft. and also provide a straighter channel for flood flows, eliminating a major proportion of the bend loss to flood flows around Stephenson's Bend. Flood levels would as a consequence be lowered and test series 14 was run to show the efficiency of the Hunter Cut in reducing flood levels in Launceston.

For this test it has been assumed that unless the old river channel is blocked off, one or the other of the two channels, or both, will deteriorate due to siltation. In order to ensure that the scouring action of the flow is restricted to the main channel of the proposed Hunter Cut, half tide training levees were constructed, sealing off the old channel around Stephenson's Bend. After these are in position, siltation of the old channel would occur. For the purpose of the model test it was assumed that deposition of the silt would take place to R.L.6, the remainder of the old channel forming a natural floodway, taking a proportion of the flood flows.

Modifications to the model were also made on the Western Bank of the proposed Cut to allow for silt deposit areas necessary for excavating a Cut.

The results of this test indicated a reduction of flood levels 0.9 ft. in Home Reach and 1.3 ft. at the railway bridge over the North Esk for a peak discharge corresponding to that of 1929. The corresponding reductions for the maximum probable flood are 0.8 ft. and 1.5 ft. respectively.

D7 SUBSIDIARY TESTS

D7.1 Future Development of the West Tamar Bank (Test Series 15).

Although present development does not warrant protection of the area along the West bank of the Tamar from the tailrace channel to King's Bridge across the South Esk Gorge, there is no doubt that as Launceston expands this area will warrant flood protection at some time in the future. In fact, proposals for building up the levels in this area are already under consideration by private and semi-governmental interests. In years to come, there is little doubt that levee banks will be raised along the edge of the river channel. Therefore the base discharges were run through the model with these levees in position, as well as the levees described in test series 7.

Owing to dense vegetation on the West Tamar bank and the high mound on the bank near Trevallyn Power Station formed by the deposits of spoil during the construction of the Power Station, there is practically no overbank flow in this area. The results of the model tests indicated a negligible change in flood levels as a result of raising the present embankments on the Western Bank of the Tamar River in Home Reach.

D7.2 Levee Protection along North Esk River from Railway Bridge to Hobler's Bridge (Test Series 16).

This area is protected by a system of low levees, and the floodable land is used mainly for grazing. The low levees at present existing are completely ineffective due to the inefficient location and operation of tide gates. This zone is flooded by high tides and minor freshes in the North Esk, the whole area becoming a sea of water three or four times every winter. There is no doubt that at sometime in the future this zone will become valuable enough to warrant efficient protection. Therefore a series of unassailable levees were placed alongside each bank of the North Esk River from the Railway Bridge to Cypress St., together with the levees described in test series 15.

The results of this test demonstrated that construction of levees to protect the River Flats adjacent to the North Esk River below Cypress Street, although not warranted by the present development in the area, will have a negligible effect on flood levels downstream of Tamar St. Bridge if they should be constructed some time in the future. Above Tamar St. Bridge flood levels are increased slightly by the restriction of overbank flow, the increase for the maximum probable flood being approximately 1 foot.

D7.3 Arterial Highway (Test Series 17)

Future plans for the Tasmanian road system provides for the construction of an arterial highway which crosses the floodable areas of Launceston. The line of the road follows Henry Street where it crosses the North Esk River and then runs via the Esplanade and Royal Park where it is linked with Trevallyn Road, by a bridge across the Tamar River. Test 17 was requested by the P.W.D. to show the height of embankment required so that the road would be above flood level for various magnitudes of floods and also to indicate the waterway area required for the bridge at Henry Street.

The results of this test are discussed in Part F of this report.

D7.4 Final Height of Training Levee (Test Series 18)

As pointed out in Section D6.31 of this Part, an overtoppable training levee may satisfactorily divert the South Esk Jet and test series 18 is designed to decide the minimum height of this levee which will be effective in diverting the jet for the probable maximum discharge and to see whether a levee curved in plan would be more efficient. However, this problem is bound up with the question of whether groynes, a rock breakwater, or other structures would be equally efficient and would cheapen the cost of the proposed bridge over the Tamar at this point. This comprehensive series of tests will therefore be dealt with in a supplementary report.

D7.5 Velocity Measurements

D7.51 Tamar and North Esk. Concurrent with the main investigation, a series of velocity measurements were taken in the river channels for series 1,6,8,11,14, with a flood discharge of 100,000 cusecs in the South Esk River and 10,200 cusecs in the North Esk River. Surface velocities and velocities at a point 11 ft. above the bed were recorded at the centre of the channel of the Tamar River at flood gauges 11 to 22. In addition, a cross section of surface velocities for the full width of the river was taken at gauge Nos. 14 (powder jetty) and 20 (downstream of the Hunter Cut).

As the North Esk River was completely drowned out by backwater flooding from the South Esk, velocities in this river were too small to be recorded.

The purpose of these tests was to compare the magnitude of changes in flood velocities as a result of the proposed improvement schemes.

The velocity readings taken in the river channels for various proposed improvement plans indicated that no substantial change in flood velocities would result from the construction of any of the proposed plans.

D7.52 Velocity Measurement in Area of Influence of South Esk Jet. As previously described, the South Esk at its junction with the Tamar issued forth as a high velocity jet resulting in considerable wave action and turbulence and two severe back eddies on either side of the jet. As it was proposed to divert this high velocity jet down Home Reach by means of a training levee it was feared that some bank erosion may result from the high velocity flow. A series of velocity measurements were therefore taken for the full range of test discharges to determine the magnitude of these velocities and the likelihood of scour in this area.

Table 14 gives the results of these measurements, and shows that velocities of the order of 23 feet per second would occur along the eastern river banks of the Tamar from King's Bridge to the North Esk River for the maximum probable flood. For the 1929 flood the order of these velocities are 16 to 17 feet per second. Further studies of scour will be included in the supplementary report referred to in Section 8 below.

D8 FURTHER NECESSARY MODEL TESTS

In view of the fact that the model demonstrated that the training levee is a key to the whole problem, it is desirable that further detailed model tests be carried out to study conditions in the Royal Park area to test and measure:-

(i) The minimum height of training levee which will divert the maximum probable discharge.

(ii) Whether a curved training levee would be more efficient hydraulically than a straight one.

(iii) Whether a series of groynes could replace the training levee or be combined with the training levee to reduce its height.

(iv) Whether a curved breakwater with reclaimed parklands behind it could replace the training levee and also dove-tail it in with the construction works necessary for the proposed new bridge across the Tamar River from Royal Park to Trevallyn Road, thus leading to saving in cost of construction of the bridge and the provision of additional parklands.

D9 CONCLUSIONS

(i) The main cause of flooding of the areas behind the existing levee banks is the fact that for South Esk discharges exceeding 125,000 cusecs, a jet of water emerging from the gorge sweeps across Royal Park up the North Esk channel overtopping the levee banks along the North Esk River.

(ii) A training levee in Royal Park diverting this jet down Home Reach is an extremely efficient method of reducing flood levels in the lower reaches of the North Esk River.

(iii) The flood levels reached in the vicinity of Launceston for discharges between 100,000 and 275,000 cusecs in the South Esk and corresponding discharges in the North Esk are shown in Tables 7-12 for natural conditions and various methods of flood protection. A general picture of the efficacy of various flood mitigation measures can be obtained by a study of flood levels at Tamar St. Bridge as expressed in Figs. 34 and 35.

(iv) The most efficient method of protecting Launceston from flooding is by use of a diversion levee in Royal Park combined with "surround" levees for the various built up areas.

(v) Methods of diversion of the South Esk Jet other than the use of a training levee are worthy of consideration and their hydraulic efficiency should be tested on the model.

(vi) The lessening of resistance to flow of flood waters in the section from River Street to the downstream end of Stephenson's Bend by construction of the Hunter Cut or other river improvements reduces flood levels in the vicinity of Launceston to a minor extent, but the general ground level of Inveresk and Invermay is such that this method of flood mitigation is not effective.

PART E

ECONOMIC STUDIES

PART E - ECONOMIC STUDIES

E1 BASIS OF ANALYSIS

E1.1 Object of Economic Studies

In the pre-war investigations into the Launceston flood problem carried out by Potts and Dare and Balsille and McCabe, as described in Part A, the aim was to give protection against a flood of the same magnitude as that which occurred in 1929 plus some arbitrary percentage (25 per cent in the case of the Potts-Dare Scheme). This ignores the question of whether this degree of protection is an economic proposition, or whether a lesser or greater degree of protection would be a better business investment. Further, only one method of flood protection was studied, and no attempt was made to compare the economics of several alternative methods.

At the time these engineers faced the problem this simplified approach was the usual practice. Over the last 30 years, however, considerable changes have occurred in the economic thinking of the engineering profession. Overseas, and notably in U.S.A., flood control authorities have developed more refined techniques for the economic analysis of projects. The author has been largely guided by his conception of the methods of investigation used by such American authorities as U.S. Corps of Engineers, U.S. Bureau of Reclamation and U.S. Dept. of Agriculture and in particular by the Report of the Federal Inter-Agency Basin Committee (E1).

The purpose of the economic study is:-

(i) For a given method of flood mitigation, to determine the degree of flood protection which is the best business proposition for the people of Launceston and Tasmania, i.e., whether to protect against 100,000, 150,000, 200,000 or 250,000 cusecs peak flood discharge.

(ii) To compare, for a given degree of flood protection, the ratios of annual benefits to annual costs for the various possible methods, or combination of methods.

Ref. (E1) Federal Inter-Agency Basin Committee - "Proposed Practices for Economic Analysis of River Basin Projects" - May, 1950.

E1.2 Essence of Method

The basic principles of the procedure adopted in the economic analysis may be expressed in a series of graphs, as follows:-

(i) Stage-Discharge Graph

From data obtained by hydrologic or hydraulic studies, (in this case hydraulic model studies) graphs of flood stage against flood discharge in the area under consideration for flood mitigation can be plotted for existing conditions and also for conditions which would exist if various alternative flood mitigation measures were constructed. Usually, a hydraulic model is essential for this and Figs. 34 and 35 illustrate this step for the Launceston case. - see also Fig. 45.

(ii) Flood-Frequency Graph

From hydrologic studies a graph can be plotted showing peak flood discharge plotted against the probability of a given discharge being equalled or exceeded in any calendar year. (See Fig. 30).

(iii) Stage-Frequency Graph

From these two graphs it is possible to compile other curves showing flood stage plotted against the probability of that stage being equalled or exceeded in any calendar year, for existing conditions or for any system of levees or other works which may be contemplated.

For a given constant relation between stage and discharge, this step consists only of substituting a scale of stages for the scale of discharges in the flood frequency graph.

(iv) Stage-Damage Graph

An investigation of the floodable zones enables curves to be plotted for each zone showing the tangible damage in pounds caused by various flood stages in these zones for each flood mitigation proposal. For the Launceston investigation, flood levels at Tamar St. Bridge could be considered a representative stage for calculating flood damage in each zone for all proposals. It was therefore necessary to prepare only one stage-damage curve for each zone by plotting damage against flood level at Tamar St. Bridge. (See Figs. 38 and 39).

(v) Damage-Probability Curve for Existing Conditions

From the stage-frequency curve and the stage-damage curve for existing conditions it is a simple matter to plot a curve of damage caused by floods against the probability of such damage being equalled or exceeded for existing conditions without any flood mitigation works. In this report this curve has been expressed in tabular form, as for example the ordinates being tabulated in column (12) and the abscissae in column (5) of Table 23.

(vi) Damage Probability Curves for Various Methods of Flood Mitigation

(a) Methods which reduce the General Flood Level for all Discharges

Examples of this type of protection are flood control dams and the training levee in Royal Park. For each such method and degree of flood protection the resulting flood stage can be obtained from the appropriate stage-discharge graph, and hence the amount of damage can be plotted against the probability or frequency of such damage being equalled or exceeded. This curve should be plotted on the same graph as (v) above. For a given method of flood mitigation, the "average annual benefit" is represented by the area between its damage probability curve and the corresponding curve for existing conditions described in (v) above.

In this report such curves have been expressed in tabular form, as for example ordinates being tabulated in column (18) and the abscissae in column (5) of Table 23. The area between the curves is given by the last entry in Col. 22 of this table.

(b) Methods which give Complete Protection from Floods to a Given Stage, and No Protection for Higher Floods

An example of this method is a "surround" levee around a floodable zone. In this case the curve is identical with that for existing conditions discussed in (v) above, but the lower end of the curve is chopped off, and replaced by a vertical line to the "x" axis at the frequency of the flood against which complete protection is provided. In this case the "average annual benefit" is the area below the "existing conditions" curve which is chopped off by this vertical line.

In this report the plotting of the curve (Col. 6 as ordinate plotted against Col. 4 as abscissa) and calculation of the area (Col. 9) is expressed in tabular form, as in Table 24.

(vii) Average Annual Cost Graphs

By estimating the capital and operating costs for the various degrees of protection for each method of protection and converting these to average annual costs for the amortization period, with due allowance for interest charges and operating costs, the average annual cost of various degrees of protection can be obtained for each method, thus providing graphs of "annual cost" plotted against "degree of protection" for each method. In this report these results are given in tabular form in Tables 17 to 22.

(viii) Annual Benefit - Annual Cost Graphs

For each method of protection, a graph can be prepared showing the annual benefit compared with the annual cost for various degrees of protection. For Launceston these results are expressed in tables, as for example in Cols. 22 and 23 of Table 23 and Fig. 44.

(ix) Optimum Economic Development

From purely economic considerations the most efficient degree of development for each method is the point on the graph in step (viii) at which the annual benefit exceeds the annual cost by the greatest amount as illustrated by point (2) on Fig.43. If the degree of protection given at this point by two different methods are similar, the one with the higher benefit-cost ratio is the most efficient, from purely economic considerations.

(x) Design Development

The final step is to consider whether intangible benefits (not considered in the stage-damage graphs) to be obtained by a higher degree of protection with the selected method, justify development of that method beyond the optimum point of maximum excess of benefits over cost. Conversely, it may be necessary to consider whether shortage of available capital funds should dictate a lower degree of development.

Decisions on these points can only be made on the basis of subjective judgment, but a series of investigations up to step (ix) above provide a sound basis for such considerations.

E1.3 Classification of Benefits

The following classification of benefits was adopted:-

E1.31 Tangible Benefits

E1.311 Direct Benefits - such that the physical effect of the flood on tangible assets can be predicted and an estimate made of the cost of replacement or repair. This may be subdivided into:-

(i) Prevention of physical damage to real and personal property.

(ii) Elimination of expenditure on removing and protecting assets on receipt of flood warning.

(iii) Avoidance of the need for emergency measures such as temporary levee banks.

(iv) Elimination of flood relief expenditure not provided for in (iii), such as provision of temporary accommodation for evacuees, etc.

(v) Loss of life expressed in monetary terms based on compensation awards as assessed by a Court of Law in accident cases.

(vi) Elimination of increased road, rail and shipping costs during flood.

E1.312 Indirect Benefits - prevention of loss of economic activity such as:-

(i) Prevention of loss of output or services expressed in terms of wages and salaries, cancelled contracts and loss of profits due to reduced output.

(ii) Prevention of overhead losses, such as interest, rent, insurance which continue when business is closed down.

(iii) Prevention of increased operating costs made necessary by carrying on production under difficult conditions caused by flood.

(iv) Preservation of output of activities in flood free zones which depend on flooded area for raw materials.

E1.32 Intangible Benefits

(i) Port Improvement
    Movement of ships
    Port maintenance

(ii) Prevention of health hazards.

(iii) Benefits to economic activity of Launceston District as a whole.

(iv) Benefits to the economic activity of Tasmania as a whole by providing a flood free industrial area close to wharves and rail traffic and to Melbourne, resulting in increase in industrial and commercial activity.

(v) Psychological value to Launceston of protection from floods.

(vi) Benefits to recreational activities.

Port Improvement was included in the intangible benefits by reason of the suggestion from some quarters that the wharves in Home Reach at Launceston would decline in importance as a result of the operation of the proposed roll-on roll-off freight ferry between Melbourne and the Bell Bay port at the mouth of the Tamar.

In the opinion of the author, this development may affect the volume of high cost low bulk cargo handled at Home Reach wharves, but will not affect the volume of low cost high bulk cargo, which, with the normal growth of Tasmania, will be of such magnitude as to ensure a steady growth of the port business in Home Reach.

Further, the author believes that while this ferry may have some effect on trade with Melbourne it will not affect materially trade with Sydney, Newcastle and similar interstate ports in states other than Victoria.

However, in order to err in the safe side, port improvement benefits were classified as intangible, and excluded from the economic calculations.

E1.4 Period of Analysis

The Physical Life of works is the period during which with normal maintenance the works will perform satisfactorily their design function.

The Economic Life of works is the period "determined by the estimated point in time at which the combined effects of physical depreciation, obsolescence, changing requirements, and time and discount allowances considered necessary on the basis of risk and uncertainty, will cause the costs of continuing the project to exceed the benefits to be expected from continuation" (E2).

The Amortization Period is the period of time assumed for economic recovery of the nett investment. It should not be greater and is generally less than the economic life. U.S.Corps of Engineers recommend a maximum period of analysis of 50 years, except in special cases. The Sub-Committee on Benefits and Costs of the U.S. Federal Inter-Agency River Basin Committee (E1) recommend that the maximum period of analysis be the expected economic life of the project or 100 years, whichever is the shorter.

The major portion of the existing levee banks were built prior to 1852, and are still serving effectively the function for which they were erected. In fact, they will probably continue to be effective for a further 100 years or more. There seems no reason why a floodway, a levee, a navigation channel or a cut via Cormiston Creek should not be in existence and effective and necessary in 100 years' time. From this point of view, it may be argued that the Launceston project be claimed to come within the scope of "special cases" referred to by the U.S. Corps of Engineers and 100 years be adopted as the period of analysis.

Ref. (E2) - U.S. Corps of Engineers - Eng. Manual for Civil Works - Preliminary Draft Part C1, Ch.2 - Examinations and Surveys - Terminology of Economic Evaluation - Oct.1956.

On the other hand, Australian Federal Loan Council practice is to amortize major concrete structures such as dams and bridges over 53 years. It is clear that some of the Launceston levees, such as those along the Esplanade, must be of concrete.

A compromise figure of 70 years was adopted for all methods of mitigation. Whether governmental finance dictates a 53 years period, or not, is immaterial so far as this investigation is concerned, because the object herein is to make a comparison of alternative proposals on a rational basis.

E1.5 Price Levels to be Used for Benefits and Costs

The I.A.R.B. Sub-Committee (E1) recommends that the costs should be based on prices likely to exist at time of construction (normally a few years after the investigation) and benefits should be based on average prices estimated to prevail over the life of the project, and that studies be made leading to long term projection of prices into the future. At the time of this report, U.S. Corps of Engineers based its computation of benefits on price levels prevailing at the time of the investigation.

While conceding the theoretical soundness of the Committee's viewpoint, the author felt that any attempt to evaluate future changes in price levels is unrealistic. For example, major wool stores exist on the floodable areas in Launceston. A study of price fluctuations of wool over the past 10 years, and a brief review of the conflicting views on the future effects of synthetic fibres in the wool economy would surely daunt the most conscientious economist contemplating prediction of future price trends.

It was decided that, for the Launceston project, prices prevailing in 1958 in Tasmania should be adopted for both costs and benefits.

E1.6 Allowance for Enhancement and Development

E1.61 Allowance for Future Development. There is no doubt that development will occur in the future on the floodable areas of Launceston, even if no flood protection is provided, and the annual benefits to be used in computing the benefit-cost curves should be those existing at some appropriate time in the future.

A simple approach, adopted in this case, is to estimate the state of development likely to exist without flood protection at the mid-point of the period of analysis 35 years hence. Foster (E3) suggests that growth of population is the best guide to future development, since "all activity arises from satisfying human needs". He advocates caution in this respect as far as U.S.A. is concerned, on the grounds that the growth of population has become stabilized, and concludes "Except in special circumstances, it will be in the interests of conservatism to assume no increasing losses in the future". (Firth and Dunn (E4) comment that events in U.S.A. have in fact shown a continuance of population growth).

In discussing Foster's paper, Chandler refers to the Chattanooga Flood Protection Scheme, and his remarks are worth quoting in full as they apply with equal force to Launceston:-

"Generally, no flood protection works have been built except in the wake of serious floods. It is hoped that Chatanooga will prove to be one exception in this distressing rule. There has been no serious flood at Chattanooga for a long time and none of consequence since the community reached its present state of development of potential resources that will be available by reason of flood protection. There are thousands of acres of land within Chattanooga that are well adapted for industry and business developments except for the flood menace. These areas, which are well located for service by railroad, highway, navigation and utilities, now stand idle. No surrounding territory is available that will compare favourably with these areas after they have been guaranteed freedom from flood. No measure of the benefits to be gained from flood protection would be complete without an estimate of development to be confidently expected as a result of protection. Although such estimate is not required to prove the necessity for protection at Chattanooga, it may easily happen that, in other instances, economical justification for worthy projects might fail of recognition unless future benefits were given due weight".

Most U.S. Writers caution against over-optimistic predictions of future growth, for the reasons given by Foster. However, Australia differs from U.S.A. in two important respects:-

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Ref. (E3) - Foster "Evaluation of Flood Losses and Benefits" - Trans. Am. Soc. C.E. May, 1941.

Ref. (E4) - Firth and Dunn "Interim Report on Launceston" Internal Report Launceston Flood Protection Authority, Jan. 1958 - supplementary letter.

(i) The country as a whole is not by any means fully developed. (Tasmania is in the fertile and well watered portion, and must inevitably receive a good share of future development).

(ii) Both parties in the Federal Parliament endorse the principle of a positive and vigorous immigration policy over the next 20 years.

The services of Professor Firth and Mr. Dunn of the School of Economics of the University of Tasmania were retained by the Authority to advise on possible future trends in the development of Launceston, and a comprehensive report (E4) was submitted, based on a close analysis of available data.

This revealed that the average annual (compound) rate of increase of population of Launceston and suburbs between 1947 and 1954 was 2.5 per cent. Professor Firth warned against the dangers of assuming that the same rate will apply in the future.

On the other hand, if Australia is to survive as a white nation, it might be argued that its population must reach 30,000,000 in 50 years' time. This is three times its present population, and presumably the population of Tasmania (including Launceston) would increase at this rate. Such reasoning would lead to the conclusion that the population of Launceston in 50 years' time would be 3 times the present figure (i.e. 185,640 as compared with the 1957 figure of 61,880 given by Firth and Dunn (E4). However, there must come a time when the curve of increase flattens out, as it has done in U.S.A. Commonwealth Office of Education (E5) estimates that the total population of Australia, if the present rate of migration is maintained, will reach 14,000,000 by 1975, (an increase of approximately 40 per cent in 17 years).

If the growth of Launceston over the coming 35 years will continue at 2.5 per cent compound (as in the opinion of the author) and the floodable area growth is at this rate, the damage estimates of the 1958 stage-damage curves (Figs. 38, 39) should be increased by 250 per cent to predict conditions 35 years hence (1994). Professor Firth suggested that a doubling of Launceston's population in 50 years would be a reasonable assumption.

Ref. (E5) - Dept. of Tutorial Studies, University of Sydney - Current Affairs Bulletin - No. 13 Vol. 21.

After due consideration of all aspects, it was decided that the best procedure was to adopt what the author considers to be a conservative attitude, and allow only a 75 per cent increase in 1958 benefits to estimate conditions in 1994, the mid-point of the period of analysis.

E1.62 Allowance for Enhancement of Value. The previous section considered increased benefits from flood protection as time goes on, due to development which would occur even if no additional flood protection is provided. If, however, the existing degree of flood protection is increased, the floodable areas will be more extensively developed due to the removal of the flood menace.

An approximate estimate of this effect was made along the following lines:-

(i) The existing assessed annual value of all properties in the area were obtained from the assessment rolls of the City Council. Due to the fact that no flood has occurred for 30 years, it was considered that these represent a value based on the assumption that floods are not a serious menace. During the next 70 years (the period of analysis) at least one flood will overtop the existing levees. This will depress values at once, but they will gradually recover as memories of the flood fade.

(ii) Assume therefore that existing values are 33-1/3 per cent higher than a true value obtained from proper recognition of the flood danger.

(iii) Compute the difference between this true value (capitalized) and the 1959 values (capitalized).

(iv) Make arbitrary adjustments based on conditions existing in the various zones, these adjustments being in all cases a reduction of computed values.

(v) Adopt for each zone the final amount thus obtained as the estimate of future enhancement of values due to flood protection.

E1.63 Combination of Enhancement and Development. After carrying out the procedures discussed in section E1.61 and E1.62, the 1994 damage values for various flood discharges were in round figures rather more than double the 1958 values. In view of the approximate nature of the estimates for enhancement and the desire for conservation, the 1958 damages were doubled to represent 1994 conditions. The author feels that this results in an underestimate of future benefits of flood protection.

E1.7 Rate of Interest

The rate of interest on capital used in all phases of the investigation was fixed at 5 per cent per annum.

E1.8 Effect of Emergency Precautionary Measures Taken as a Result of Flood Warning.

In order to obtain first hand data on flood damage for various depths of flooding for different types of business and industry, and residences, the towns of Maitland and Singleton in the Hunter Valley of New South Wales were visited. The citizens of these towns had recently experienced a number of serious floods, and valuable first hand information was obtained on the nature and extent of damage.

One aspect examined was the efficacy of flood forecasting and warning services.

In the case of Maitland, householders had at least 24 hours warning that "there will be a flood". Very little effort was made to remove household effects, the reasons apparently being:-

(i) The belief that the flood would not reach the house in question.

(ii) The lack of a place protected from rain to which to remove the assets, and lack of transport.

(iii) A generally sceptical attitude.

When the flood did come, a common practice was to place valuable assets on tables. In many cases the tables floated and capsized, depositing their contents in the muddy water.

Businessmen, on the other hand, were in the main prompt to take emergency measures, usually by raising goods to tables, shelves, etc., just above the previous flood. In the 1955 flood this effort was wasted, as the flood reached a point several feet higher than the previous one. In 1956 the goods were raised to levels higher than the 1955 flood, but in many cases the waters did not enter the property at all.

It was clear that if a reliable forecast could be made 24 hours ahead of the flood, giving the height and time of the flood peak, and if emergency measures were taken, then very considerable reduction in flood damage would result.

In other words, the first degree of flood protection to be considered should be the expenditure necessary to provide such a prediction. The matter is more fully discussed in Part F of this report.

As a result of the Hunter Valley investigations, it was decided that two stage damage curves should be prepared in all cases, one being in the assumption of 24 hours of emergency precautionary measures, and one for 6 hours. It was assumed in the first case that the proprietor decided at 6 p.m. on a Thursday night that his premises would be flooded, that he assembled 20 per cent of his employees on the job by 8 p.m. and that his premises were actually flooded by midnight on Thursday. In the second case, the decision to commence emergency precautionary measures by removing goods etc., was also fixed at 6 p.m. on Thursday with 20 per cent of employees on the job by 8 p.m and 80 per cent by 9 a.m. on Friday, with flooding at 6 p.m. on Friday.

In the case of Launceston, under the worst tidal conditions, discharges in the South Esk River not greater than 90,000 cusecs, while causing alarm in Launceston and damage in rural areas, do not seriously overtop the levees. These floods are fairly frequent, while floods exceeding 90,000 cusecs are quite rare. Hence scepticism and inertia are likely to be powerful influences which will tend to minimise emergency measures by the general public and industry when the major floods do arrive. Where firms have taken emergency precautions a few times for damaging floods which do not eventuate, they soon become cynical about flood warnings.

In this connection it should be noted that with the existing instrumentation of the catchment the Deputy Director of the Meteorological Branch in Hobart can only say "There will be a flood". He cannot predict that there will be a flood in 24 hours time in Home Reach which will reach a level of X feet. If adequate instrumentation were installed with telemetering facilities, and studies made of past and future floods for a few years, he would be able to make such a prediction with reasonable confidence. Even if such a service were available however, apathy would probably frustrate to a certain extent the efforts of public authorities to arrange prompt and vigorous emergency action by the citizens for the first major flood which occurs in the future. No doubt the response will be very prompt for the second one, especially if it occurs within 10 or 20 years of the first.

Hence some may consider that the estimate of 6 hours of active emergency measures over the next 70 years is more realistic than the 24 hour assumption, and that these stage-damage curves are the best estimate of future damage.

On the other hand, in the 1929 flood the Mayor and others were on the alert 12 hours before flooding occurred, and the flood warning service and meteorological information has improved greatly since then. The period of emergency measures to be assumed as a basis for final recommendations is considered in Section E6.2212.

It was considered that four points would be sufficient to define the stage-damage curves, so that probable future damages were estimated for floods reaching 17, 20, 22 and 25 ft. above S.L.W. in the North Esk in the vicinity of Charles St. Bridge. Graphs of flood stage against damage were plotted as in Figs.38 and 39.

E1.9 Zones Adopted for Economic Analysis

The extent of flooding for various flood heights and the intensity of development varies in different areas in Launceston. When commencing the economic work it was considered possible, for example, that protection against a flood of 250,000 cusecs might be justified economically for the Inveresk-Invermay area, but the justifiable degree of protection in the city area might be only that for say 150,000 cusecs.

Therefore in the first analysis the flood plain was divided into six zones:-

(i) Esplanade-Cypress St. Zone, being bounded by Royal Park, The Esplanade, the main Western Railway line, Cypress St. and Cimitiere St.

(ii) Paterson St. Zone, bounded by Paterson, Wellington, Frederick and Bourke Streets.

(iii) Inveresk-Invermay Zone, bounded by the Tamar River, the North Esk River, the North East railway line and Conway St.

(iv) Upper North Esk Zone - grazing lands adjoining the North Esk between the Railway Bridge and Hobler's Bridge.

(v) Floodable area incapable of levee protection (wharves etc.)

(vi) The West Bank of the River Tamar from Cataract Bridge to Trevallyn Power Station.

A complete economic analysis was completed on this basis, which made it clear that a further sub-division of zones would be desirable.

Therefore the analysis was repeated with zones as follows:-

(i) Royal Park-Willis St. Zone, bounded by Royal Park, Esplanade, Willis St. and Cimitiere St.

(ii) Willis St.-Cypress St. Zone bounded by Railway Line from Willis St. to Cypress St. and by Elphin Rd.

(iii) Paterson St. Zone, as before.

(iv) Inveresk-Invermay, bounded by Tamar and North Esk Rivers from River St. to Herbert St. and by the high ridge of Mayne and Burns Streets.

(v) Mowbray Flats Zone, bounded by the Tamar, the Mayne St. ridge, the North East Railway and Conway St.

(vi) The Upper North Esk, as before.
(vii) Wharf Zone.
(viii) West Tamar, as before.

The location of these zones is shown in Fig. 37. Detailed economic analyses were not carried out for (iii), (vi) and (viii) for the following reasons:-

Zone (iii) - The area is intensely developed and can be protected from the probable maximum flood at very small expense.

Zone (vi) - As the area is mainly grazing land and privately owned, it was considered that at this stage of its development any expenditure of government funds in protecting it from flooding was not justifiable.

Zone (vii) - The area is at present in its natural state, subject to flooding at high tide and would need building up before any development could be contemplated.

E2 STAGE-DISCHARGE GRAPHS

The information necessary to build up these graphs was obtained from the model studies, as described in Part D.

Figs. 34 and 35 express the results of this work, and Fig.45 is based on Fig. 34.

E3 FLOOD-FREQUENCY GRAPH

The procedure necessary to obtain this graph is described in Part C. The adopted curve is shown in Fig. 30.

For average tidal conditions the model provided the relation between flood discharge and flood stage shown on the stage-discharge graphs (Figs. 34 and 35). Hence the flood values on the flood frequency graph are readily convertible to stage values giving the stage-frequency relation.

E4 STAGE-DAMAGE GRAPHS

E4.1 Residential Damage

E4.11 Experience in New South Wales. The Chairman of the Relief Committee of Maitland after the 1955 flood, (D.McGee) gave the following general guide based on his experience.

(a) The average dwelling flooded 2 ft. above floor suffered losses between £300 and £400.

(b) The average dwelling flooded 4 to 6 ft. deep suffered losses between £600 and £700, the increased damage being due mainly to the fact that possessions can be stacked 2 ft. above floor but not more than 4 ft., and tables etc., capsize when the water level reaches 4 ft.

It was noted that in some parts of Maitland flood velocities were higher than would be expected in Launceston, and that in Maitland the flood water was acid enough to rust and etch metal fittings. A coating of oil served as good protection. The Maitland building inspector urged that citizens in floodable areas should install a large manhole in the ceiling and store household effects above the ceiling joists.

Detailed information on nature of damage to various types of buildings and household effects in Maitland were obtained.

The Executive Member of the Central Flood Relief Committee of the N.S.W. Treasury, (A. Jolly) supplied details of 27 towns in N.S.W. which were flooded in 1955, together with Police Department estimate of money value of structural damage only.

Thorpe and Tweedie (E6) gave the final estimate of 1955 flood damage at Singleton (Population 4,750) in the Hunter Valley as £320,000 as compared with £150,000 for the commercial losses. For Maitland (Population 10,000), between 2,000 and 3,000 homes were inundated and household losses were estimated as £1,000,000 as against business losses of over £500,000. However, this figure includes some houses completely swept away in some areas subjected to high velocity current. These estimates do not allow for the hundreds of voluntary workers and Army personnel used in 1955 flood relief, nor do they allow for indirect losses.

Some comparisons with Launceston are relevant.

The total population of Launceston is given in Firth's report (E4) as 61,880 including Launceston City and Beaconsfield, Lilydale and St. Leonard's Municipalities. Although electricity is widely used in Tasmania, due to a high degree of Hydro-Electric development and poor coal, the Launceston Gas Company (which is in the floodable area) supplies more than twice as much gas as Maitland.

Assuming that the average number of inhabitants per house is four, the population of the floodable zones of Launceston is at least 5,000. The standards of dwellings and furniture in Launceston are superior to those of Maitland, and industry is much more highly developed in the floodable zones.

Ref. (E6) - Thorpe and Tweedie - Australian Geographer
Vol. VI No.5 - March, 1956.

E4.12 Selection of Sample Residences in Launceston for Detailed Study. With first hand knowledge of the nature and money value of recent residential flood damage in N.S.W., a reasonably accurate estimate of probable future damage to houses caused by floods of various heights could be made for any given house in Launceston. From a study of the Launceston City Council Assessment Roll and the contour and spot level surveys by Wilks, (Section A5) it was ascertained that 1,282 houses (excluding residential shops) in Launceston had their floor levels less than 25' above S.L.W. Obviously a detailed inspection and estimate could not be made for every house. A representative sample only could be visited. From a tour of the area it was concluded that the quality of house and effects did not vary with height of site above S.L.W., and that a simple sampling by random numbers of 14 houses from the total list would be adequate.

E4.13 Estimate of Probable Future Flood Damage to Selected Sample of Residences.

Each of the houses in the sample group was visited with a standard damage form on which was listed all the usual household effects and common structural damage items. Current building repairs and furniture replacement costs were obtained from local merchants. The full replacement value of items was not taken as the probable damage, but a depreciated estimate made, based on age and condition of the item, and with consideration of its salvage value after cleaning off the mud etc.

The majority of the houses are of timber construction and single storied. Many present an unimpressive exterior appearance. However, the interior furnishings and furniture were generally of high standard, with much modern veneered furniture. This type of furniture is very susceptible to flood damage.

It was noted that in some properties motor cars and caravans were garaged and stored and there were "backyard" industries, involving stocks of grain, furniture, etc. One of the houses in the sample held a store of second hand furniture, but nevertheless the damage estimates may be somewhat low due to the sample not being representative in regard to "backyard" industry.

The advice from Maitland residents was not followed literally. Had it been, the estimates would have been materially increased.

For each house in the sample group three separate estimates were made for flood heights as follows:-

(i) Between floor level and 3 ft. above floor level.

(ii) Between 3 ft. and 9 ft. above floor level.

(iii) More than 9 ft. above floor level.

These critical stages were based on the assumption that nearly as much damage would be done by a flood 6" above floor level as by one 3 ft. above. For example, if a watermark appears on an interior wall, the whole wall must usually be repainted, no matter at what height the discolouration appears. Many articles can be stacked on tables and shelves and thus preserved if the water is not more than 3'0" deep. When the flood lies between 3 ft. and 9 ft. there is little increase in damage with flood height, but once the flood exceeds ceiling level there is a sharp increase, because of damage to ceilings and to electrical installations.

The arithmetic average of the damages for the sample houses for depths (a), (b) and (c) above was calculated. A comparison was then attempted with the damages assessed by the Flood Relief Committee for the same houses for the 1929 flood. Unfortunately, only one of the sample houses appeared in the 1929 lists, due to the others either having been erected since 1929 or the owners being considered ineligible for relief. For the one house common to both lists the author's estimate of damage was £347. The occupant stated that in 1929 he claimed £150 (£450) and received £45 (£135). In view of the fact that in 1959 one half of the brick wall of one side of his house collapsed, this seems low. It must be realised, however, that the 1929 Flood Committee awarded "relief - not compensation", the idea being apparently to give the distressed person an amount merely sufficient to rehabilitate him as an income earner.

Another house was inspected in detail for the purposes of investigating 1929 flood levels, although it was not included in the sample. The Flood Relief Committee records show an "allowed" estimate of damage at £358(£1,074) for this house although no structural damage was done. No payment was made to this claimant, because he was in such financial circumstances that he could "carry his own loss".

The 1929 Flood Relief Committee specifically excluded pianos, carpets, radio sets and such non-necessities from all claims. In spite of the fact that the 1929 relief figures must therefore be a low estimate, the average residential compensation actually paid in 1929 for 424 houses in the 3 ft. - 9 ft. range was computed and found to be £48.10.0 (£145).

After a careful consideration of all available evidence, the following figures were adopted for future residential flood damage on the assumption that vigorous precautionary measures are started by the householder 6 hours before the house is inundated.

(a)	Zero to 3 ft. above floor		-	£200
(b)	3 ft. to 9 ft.	" "	-	£600
(c)	Over 9 ft.	" "	-	£900

For 24 hours of emergency precautionary measures the figures were £125, £500 and £800.

It will be noted that the damages for (a) and (b) are less than McGee's Maitland averages for these two cases, i.e. £350 and £650 respectively.

By the use of Wilks' spot level and contour plan referred to in Section A5, the floor levels of all houses in the floodable area could be estimated. For each of the zones a tabulation was made showing the number of houses flooded 0 to 3 ft., 3 ft. to 9 ft., and above 9 ft. by floods reaching 17 ft., 20 ft., 22 ft., and 25 ft. above S.L.W.

By multiplying the number of houses at the various levels by the corresponding estimated damage (a), (b) or (c) above, the final assessment of residential damages resulted as follows:-

TABLE E1.

RESIDENTIAL DAMAGE (£1,000)

Zones	6 Hrs. Emergency Measures				24 Hrs. Emergency Measures			
	Flood Ht.- ft. above S.L.W.				Flood Ht.-ft. above S.L.W.			
	17	20	22	25	17	20	22	25
Esplanade-Cypress	7.6	47.0	61.9	112.2	4.75	37.0	48.0	94.3
Inveresk-Invermay	373.4	490.6	712.0	848.1	270.4	377.3	616.6	741.5

When the benefit-cost curves had been prepared and the final recommendations considered, it was felt that the recommendations should allow for the flood mitigation works to be carried out in stages, if so preferred by the Tasmanian Government. Hence it was necessary to sub-divide the zones and re-estimate the number of houses of various levels in each sub-zone. This resulted in the following final tabulation.

TABLE E1.

FINAL

RESIDENTIAL DAMAGE £1,000

Zones	6 Hrs. Emergency Measures				24 Hrs. Emergency Measures			
	Flood Ht.-ft. above S.L.W.				Flood Ht.-ft. above S.L.W.			
	17	20	22	25	17	20	22	25
Royal Park-Willis St.	6.0	37.6	49.0	89.7	3.7	29.0	37.4	75.7
Willis St.-Cypress St.	1.6	9.4	12.9	22.5	1.1	8.0	10.6	18.6
Inveresk-Invermay	351.4	433.6	626.1	696.1	276.4	333.3	547.6	614.5
Mowbray Flats	22.0	57.0	86.0	152.0	14.0	44.0	69.0	127. 0

E4.2 Industrial and Commercial Questionnaire

As a basis for all damage investigations, a detailed questionnaire was drawn up. The main headings were:-

Section A - Details of Ownership etc. of Property
Section B - Estimate of Flood Damage

(i) Direct Damage

(a) Damage to grounds
(b) " " buildings
(c) " " furniture
(d) " " plant and equipment
(e) " " raw materials
(f) " " finished product of factory or merchandise in a shop or warehouse
(g) Estimate of reduction in damage possible by 6 hours and 24 hours of emergency measures before flooding.

(ii) Indirect Damage

(a) Wages lost by employees not employed in rehabilitation work brought about by the flood damage

(b) Overhead losses

It was decided to provide one "omnibus" questionnaire to cater for all types of industry, rather than to develop separate specialised firms. Extensive notes on replacement costs and experience in N.S.W. floods were included, to assist in filling in the questionnaire.

For all major factories, warehouses etc. a visit was paid to the management to explain the questionnaire, which was then left with the firm for completion of the damage estimates. On receipt of the completed form the accuracy of the estimates was checked by questioning the management, obtaining independent opinion of unit costs, and comparison with similar industries in Maitland and Singleton, New South Wales.

For minor commercial and industrial establishments, an inspection of the premises was made and by comparison with similar flooded businesses in N.S.W., the questionnaire was filled in during the inspection.

It was found that the questionnaire served its purpose well, so far as direct damage was concerned, but was inadequate to obtain a clear picture of probable indirect damage. This inadequacy was not recognised until a late stage in the investigations, so that in the plotting of the stage-damage curves arbitrary adjustments were made to the indirect damages, based on general comments regarding U.S.A. experience made by Barrow (E7).

The weaknesses of the questionnaire in regard to indirect damages were the lack of specific provision for such information as:- Accountancy Fees, Commercial Travellers, Vehicle Fleet Overhead, Stationery Overhead, Interest, Rent, Insurance, Telephone Costs, Holiday pay, Superannuation, Pay-roll Tax and similar items which appear in the profit and loss accounts of trading organisations, but which are liable to be forgotten in the estimation of losses caused by dislocation of business for several weeks due to floods.

Ref. (E7) Barrow - "Floods, Their Hydrology and Control" - 1948.

E4.3 Damage to Industrial and Wholesale Commercial Establishments

E4.31 Wool Stores, Produce and Hide Stores and Scouring Works. Launceston is the port for a rich hinterland producing some of the highest grade merino wool. For nine months of the year considerable stocks of baled wool are held in large warehouses, most of which are in the floodable area.

The wool scouring capacity of Tasmania is very limited. All authorities agree that if wool is scoured within a few days of inundation the flood damage would be relatively small. However, the amount of wool flooded in major floods in Launceston would be far too great for local scouring works to handle, and it could not be shipped to the mainland in time to minimise damage by scouring it.

Unfortunately, no cases have occurred in recent years in N.S.W. of baled wool having been inundated, and widely varied opinions were obtained from woolbrokers as to the money value of flood damage, effect of capillary action in damaging bales above flood level, etc.

The opinions of the N.S.W. Department of Agriculture, of Professor McMahon (Wool Technology, The University of New South Wales), and of various practical wool men were obtained. Their views were transmitted to the managements of the wool broking firms to aid them in filling in the damage questionnaire. On receipt of these returns a reconciliation of the various points of view was made and final estimates computed on the following basic assumptions:-

(i) Amount of wool in store at time of flood is the average of the maximum and minimum amounts in store over an average year.

(ii) At time of flood the bales are stacked three deep.

(iii) If a bale is submerged to a depth of 3 ft. from the bottom of the bale the direct loss will be 30 per cent of sale price of £65 per bale.

(iv) With 24 hours of active emergency precautionary measures the damage will be reduced by 20 per cent.

For a discharge of 150,000 cusecs (1929 flood) the total estimated damage for all zones was £77,000.

It is quite possible that the estimates are in fact too low. It is unlikely that they are too high. (A repetition of the 1863 flood in the month of December would be a disaster for the wool industry).

In the absence of practical experience of actual flood damage to wool by muddy waters it was deemed wiser to err on the low side.

Damage to the wool scouring works, fellmongery and hide and tallow businesses is a material factor in the Inveresk-Invermay zone.

E4.32 Timber Industry. Launceston is a major centre for the kilndrying, dressing and export of timber, and many major timber yards are located in floodable areas in close proximity to wharves and railways.

A fair amount of practical experience of flood damage to timber had been obtained in the North Coast of New South Wales, and the firms concerned were visited or written to for detailed information. Rather conflicting views were expressed. Apparently the money value of timber damage varies with timber species, condition (dressed or rough), method of stacking, etc. The cleaning of fine silt from the timber appeared to be one of the main costs incurred as a result of these floods.

There was at least one timber yard in the floodable area of Launceston in 1929, but eyewitnesses could give little information on the monetary losses sustained. In any case, conditions in this yard were abnormal, in that the only exit was through a vehicular doorway, which was blocked at an early stage by the movement as a single unit of a big stack of timber into the entrance. (Similar cases of complete stacks moving bodily from one point to another without collapse were quoted by N.S.W. firms).

Discussions were held with leaders of the timber industry in Launceston, and the various opinions regarding probable damage were transmitted to all firms by the Secretary of the Northern Tasmanian Timber Association, which co-ordinated the completion of the questionnaires. When the returns were analysed, it was clear that there were widely differing approaches to the assessment of damages. A thorough analysis was made of two yards, and the others adjusted to this standard. The final figures adopted gave the total direct damage for 6 hours warning for timber yards for 150,000 cusecs in Inveresk-Invermay zone as £63,000, which is a minor item, surprisingly out of proportion to the area occupied by timber yards.

E4.33 Furniture Factories. This industry is quite important in the floodable area, and is very vulnerable to flood damage, and for the Inveresk-Invermay zone for 150,000 cusecs this direct damage totals £66,000 for 6 hours of emergency measures.

E4.34 Food Industries. Flour mills, a brewery and dairy products are important industries in the Inveresk-Invermay and Esplanade zones.

E4.35 Bulk Petrol and Oil. Every major oil company has a bulk depot in the low-lying Inveresk-Invermay area near wharves and railway. Total estimated direct damage for 150,000 cusecs was £52,000 for Inveresk-Invermay for 6 hours warning. This was obtained by giving the managements details of damage done in 1955 to bulk fuel depots in the Hunter Valley of New South Wales, and accepting their figures, which were fairly consistent with one another.

E4.4 Retail Trading. These establishments were generally of the same standard as those in Maitland, and every retail shop in Launceston had its counterpart in that N.S.W. town. Hence a fairly reliable estimate of probable damage could be made. Although the shops were individually quite small, the number was so great that the total flood damage was an important feature.

E4.5 Public Utilities. Detailed information of flood damage was supplied by public utilities in the Hunter Valley of N.S.W., and this was passed on to the public authorities in Launceston to aid them in making their estimates. Major contributions to total damage came from the Railway Department and Gas Company, but the Hydro-Electric Commission, Posts and Telegraphs Department, Launceston City Council, Department of Customs and Marine Board (excluding cargo in wharf stores) estimated only minor damage.

The Railway Workshops and Stores and rolling stock and general railway activities are the major industry in the Inveresk-Invermay zone, the estimated direct damage for 150,000 cusecs being £165,000 (6 hours emergency measures).

E4.6 Damage Outside Proposed Levees. A few minor industries and residences, wool and machinery stores in the Queen's Wharf on the North Esk bank, and cargoes in the main sheds on King's Wharf comprise the major elements of damage in areas which cannot be protected by levees. The Marine Board's estimates for King's Wharf and other stores are easily the biggest item, direct damage being:-

(a) For 6 Hours of Emergency Measures

17 ft.	20 ft.	22 ft.	25 ft.
£2,500	£185,000	£385,000	£438,000

(b) For 24 Hours of Emergency Measures

17 ft.	20 ft.	22 ft.	25 ft.
£2,000	£150,000	£350,000	£400,000

If a levee system were built, and the goods need to be moved only a hundred yards or so to safety behind the levee banks, then for 24 hours warning 75 per cent of this damage should be preventable.

For the whole area the estimated figures are:-

TABLE E3.

TOTAL DAMAGE OUTSIDE PROPOSED LEVEES (£1,000)

Nature of Damage	6 Hrs. Emergency Measures				24 Hrs. Emergency Measures			
	Flood Ht.-ft. above S.L.W.				Flood Ht.-ft. above S.L.W.			
	17	20	22	25	17	20	22	25
Direct	10	200	415	478	7	162	373	430
Indirect	3	20	25	30	3	20	25	30
TOTAL	13	220	440	508	10	182	398	460

The only way to mitigate these damages would be by a quantitative flood forecasting system or by construction of the Hunter Cut at a cost of £650,000.

E4.7 Final Estimates of Probable Future Flood Damage

The estimated total direct and indirect damages for the various zones are shown in Figs. 38 and 39.

The four main points on the graphs are based on the following values, which are expressed in units of £A1,000.

TAPLE E4.

TOTAL FLOOD DAMAGE (£1,000)

FOR THE VARIOUS ZONES

	6 Hrs. Emergency Measures				24 Hrs. Emergency Measures			
Zones	Flood Ht.-ft. above S.L.W.				Flood Ht.-ft. above S.L.W.			
	17	20	22	25	17	20	22	25
Mowbray Flats	33	86	125	213	24	70	104	193
Inveresk-Invermay	1187	1754	2465	2607	962	1406	1846	2087
Willis-Cypress	80	132	184	237	64	107	141	186
Royal Pk-Willis	41	226	399	559	26	176	319	445
Outside Levees	13	220	440	508	10	182	398	460
TOTAL	1354	2418	3613	4124	1086	1941	1808	3371

The level of 20 ft. is approximately the figure for the 1929 flood. In assessing the economics of any flood mitigation scheme, the above values should be at least doubled to express the damage 35 years hence, at the mid point of the 70 years of the period of analysis appropriate for this problem.

The assessment of probable future direct flood damages depends a good deal on the assumptions made by the investigator, and another worker might obtain values perhaps 25 per cent more or less than the above figures. Indirect damages are more difficult to assess and estimates by different individuals might show considerable variation. However, any such variations do not affect the validity of economic comparison between various flood mitigation proposals. Further, in this case the benefit-cost ratios of the various proposals, as detailed in Section E7, are so high that the accuracy of the damage estimates is not a vital factor.

E5 ESTIMATION OF CAPITAL COST OF VARIOUS FLOOD MITIGATION MEASURES.

E5.1 Hunter Cut

Some borings and probings of the Hunter Cut area had previously been made by the Marine Board. Additional probings were carried out, and the volume of silt and clay in the excavation computed.

A senior construction engineer of the Hydro-Electric Commission (M.C. Heffernan) who had experience of excavation of the Trevallyn tailrace channel just above the Hunter Cut, inspected the area, in company with the H.E.C. Testing Engineer, (J.W. Evans) and tendered the following advice:-

(i) The removal of both clay and silt can be best carried out by suction dredging.

(ii) If there is any likelihood in the future of the Hunter Cut being constructed, consideration might be given to using the spoil from the tailrace excavation (which was pumped into reclamation areas west of the Hunter Cut) for construction of levees and the embankment of the proposed new arterial road along the West Tamar shore. This would enable the Hunter Cut dredgings to be discharged into these reclamation areas and thus reduce pumping costs.

(iii) Removal of ti-tree is an expensive undertaking.

A Director of a dredging company (R.A. Jessup - Harbour Works Pty. Ltd.) advised on dredging methods and costs.

The final estimate of costs may be summarised as follows:-

1,333,000 cubic yards of silt	at 3/3d. per C.Y. =	£216,700
425,000 " " " clay	at 13/3d. per C.Y. =	281,300
Half tide wall across old channel		31,000
Tailrace diversion 177,000 C.Y. at 2/6d.		22,100
Land acquisition		2,500
Design and supervision		33,000
Contingencies		58,700
		£645,300
	Say	£650,000

E5.2 Floodways

As the model showed that floodways were not very efficient in reducing flood levels, no estimates of cost were made.

E5.3 Diversion of South Esk via Cormiston Creek

A survey was made along the line of this proposed diversion cut and cross sections taken. The top of the saddle was found to be 92 ft. above the top still water level of Trevallyn Dam. Detailed hydraulic calculation for the design of the cross section of the Cut was not undertaken, as a simple design of a side channel spillway gave the quantity of excavation as 16,000,000 cubic yards. No borings were made, but it was felt that the face revealed by the construction of the tunnel from the dam to the power station would probably represent the type of strata. If so, the excavation would be through clayey soil interspersed thickly with huge boulders in which case the cost of excavation would exceed £2 per cubic yard, making a total cost of more than £32,000,000. This proposal was forthwith discarded.

E5.4 River Straightening and Channel Improvement

It was realised that the cost of maintaining any river improvement measures would be high and when the model revealed that little hydraulic advantage was gained by these measures, it was decided that there was no need to make an estimate of the capital cost of these works.

E5.5 Levees

E5.50 General Comment

The routes of the levees were fixed by inspection on the ground, having regard to the need to include all highly developed areas within the levees, proximity of buildings, railway lines, etc. The Marine Board stated that the ultimate objective in development of the wharf area was to have a clear space for storage etc. for a distance of 400 ft. back from the face of King's Wharf. At present much of this area is used by timber kiln drying works and bulk fuel storage depots. It was therefore decided that in the first instance a concrete levee in removable sections should be planned along the line of the existing wharf fence, with provision for removal further east at some future date.

For some distance the existing railway embankments now serve as levees. Discussions were held with the Chief Civil Engineer of the Railway Department (J.Dineen) as to whether to raise the railway lines or build levees against the existing embankments. On account of the lower costs the latter alternative was adopted.

The basic steps in all levee estimates were:-

(i) Discharges corresponding to general flood levels in the Charles - Tamar St. area of 17, 20, 22 and 25 ft. were determined from the model tests.

(ii) From these tests also the required levee heights to protect against these discharges were found along the whole route of the levees.

(iii) Freeboard was added in accordance with Table 27 to allow for levee settlement and wave action.

(iv) Based on the standard designs of earth and concrete levees shown in Figs. 40 to 42, the quantities of earth or concrete required to surround the various sub-zones were determined.

The location of the levees is shown on Fig. 37.

E5.51 Earth Levees

E5.511 Basic Design. The type of section adopted for earth levees is shown in Fig. 41.

In the design of these levees, consideration was given to the current American practice adopted for the design of levee banks for the Mississippi River, but as the proposed earth banks for Launceston are sited in backwater areas where there is little danger of scour, the side slopes were increased somewhat to correspond to the maximum recommended when compacted at optimum moisture content of the soil.

To allow for truck maintenance of the levee after construction an eight foot top width was adopted. The Chief Engineer of the Marine Board (J.Edwards) later suggested that eight feet was not sufficient and recommended that this be increased to 10 ft. by the construction of a berm below the top of the bank. This alternative may be desirable in the final design, but as the total earth quantities would not be materially changed, the estimate of costs used in this report is based on the original cross section.

E5.512 Source of Material and Unit Costs. Suitable natural borrow pit sites close to Launceston are not readily available. The Chief Engineer of the Marine Board (Edwards) suggested that some of the reclamation areas in the Inveresk-Invermay district, which are used for depositing dredgings pumped from the Tamar, should be allowed to dry out and the material for levees obtained from these areas. This would be in the interests of the Marine Board as suitable reclamation areas near the site of dredging are scarce. The Testing Engineer of the Hydro-Electric Commission (J.Evans) reported that the material, when dry enough, would be very suitable. A construction engineer of that Commission (M.Heffernan) suggested that if only the top few feet of the dredging has adequately dried out and the underlying material remained very wet, problems of economic loading of the material would arise. In view of the quantity required, suitable special methods of loading could probably be devised, which might lead to quite a low unit cost for the levee material. For the initial estimates and computations of benefit-cost ratios a unit cost of 12/6d. per cubic yard compacted in place was adopted. At the meeting of the Authority on April 6th, 1959, it was decided that the unit cost should be increased to £1 per cubic yard, and this latter figure has been adopted for this report.

When tenders are called it may be found that this estimate is rather high. It is most unlikely that it will be too low, so that the estimates of cost given in this report should be on the safe side.

E5.513 Levees Alongside Railway Lines and Provision for Grassing. The estimates for these levees were prepared by the Chief Civil Engineer's Branch of the Tasmanian Railway Department. An interesting feature is the provision for grassing these banks using the "Finn" process. This consists of dragging a spiked chain over the batter to be treated to prepare the surface for the reception of mulch, bitumen emulsion, seed and fertiliser. The mulch which is formed by hay impregnated with bitumen emulsion is then spread over the soil after which the batter is seeded and fertilised. Each of these processes is carried out automatically by the use of the "Finn" equipment which delivers and spreads the necessary pasture seed and fertiliser at a controllable rate as uniformly as possible over the area to be treated. The seed and fertiliser are projected through the air in water, through the mulch and onto the soil in such a way as to provide the best conditions for germination and growth. The estimated unit costs of this process is 3/4d. per square yard.

This unit cost was adopted throughout for estimating grassing costs for all earth levees.

A typical cross section for these levees is shown in Fig.42.

E5.52 Concrete Levees

E5.520 Basic Design. The poor nature of the soil in the Inveresk-Invermay area is well known. A study was made of settlement of several structures and of design practice which has been adopted by various authorities in the light of practical experience of buildings in this zone, which originally was a swamp.

As a result the following design criteria was used:-

(i) Allowable bearing pressure under dead load = 336 lb./sq.ft.
(ii) " " " " live and dead load = 1120 lb./ft.
(iii) Specific weight of soil = 100 lb./cu.ft.
(iv) Angle of internal friction= $9\frac{1}{2}^{\circ}$
(v) f_c of concrete = 3000 lb./sq. inch
(vi) Intermediate grade of steel to be used
(vii) Min. depth of cut-off wall = 4'0".

After consideration of various possible cross sections, an L shape with a concrete cut off wall was adopted. The ground on which the levees are to be located may be described in broad terms as consisting of a reasonably firm crust a few feet thick underlain by waterlogged silt. (There is some evidence that the ground water level rises and falls with the tide). Therefore, the design provided for a 6" gravel mat to be laid on the surface beneath the concrete slab of the levee.

The cross-section was developed and examined in relation to:-

(i) Resistance to bending
(ii) Bearing pressures
(iii) Location of wall on slab to minimise overturning moment
(iv) Stability against sliding

Designs were completed for walls 4 ft., 7 ft., and 11 ft. high, and a graph plotted showing relation between height of wall and volume of concrete per lineal ft. from which quantities for the various sections of levees were calculated to give protection against basic flood discharges of 98,000, 157,000, 192,000 and 270,000 cusecs.

E5.521 Unit Costs. Initially the following unit costs were adopted:-

(i) Main Wall = £30 per cu. yd.
(ii) Slab = £20 " " "
(iii) Cut-off Wall = £22 " " "

At the meeting of the Authority on April 6th, 1959, a decision was made to increase the costs to £30, £20 and £30 per cubic yard respectively and the estimates in this report embody this change.

It is recognised that the cut-off wall could perhaps be more conveniently constructed by the use of concrete piles. This may reduce the costs. As in the case of the earth levees, it is considered that these unit costs err if anything on the high side.

E5.522 Levees Along North Esk Bank. The costs used in this report allow for a concrete levee from the Charles St. Bridge around the wheat silos to King's Wharf. Possibly this could be more cheaply constructed in earth for most of its length. However, for this section and more particularly for the route along the Esplanade in the city side of the North Esk and along the Inveresk-Invermay bank, a concrete promenade and solid parapet with ornamental park type electric lighting would be a civic asset aesthetically, as the present condition of these river banks is unworthy of a city which in other respects is well known for its public parks and gardens.

If the proposed new arterial road runs along the Esplanade, this promenade and parapet could readily be incorporated in the road design.

E5.523 King's Wharf. As previously mentioned, it is possible that at some future time the Marine Board may wish to move these levees further away from the river bank. It may therefore be economical in the long run to construct these levees of removable sections, with adequate provision for keying and sealing the joints.

E5.6 Training Levee in Royal Park

E5.61 Basic Design

It is possible that a well compacted earth training levee with some protection on its western face would serve the purpose of diverting the South Esk jet on the relatively rare occasions which it would be called upon to do so. It would no doubt be damaged to some extent, but could readily be repaired.

However, this is such a key element in the proposed scheme that no risk of failure, however slight, should be taken. The easterly velocity of the jet at the Royal Park shore is given by the model as approximately 9 ft. per second for the 1929 discharge, and 12 ft. per second for the probable maximum discharge. After diversion by the training levee the northerly velocities approximate 13.1 and 21.0 ft./sec. respectively.

Hence in the first instance a straight concrete levee was designed standing 100 ft. back from the river's edge and 1500 ft. in length. For aesthetic purposes and to give added strength, a grassed earth levee was superimposed on this concrete wall, and the total cost estimated. Such a design would not impair the existing pleasing appearance of the park, and would provide a natural grandstand for sporting events such as regattas.

Some consideration was given to the possibility of major floods eating into the banks of Royal Park to such an extent as to undermine the levee. The bank is at present protected by wooden sheet piling in poor condition. As far as can be ascertained, the 1929 flood did not make any serious inroads of this nature, and it is difficult to visualize any flood, over a two or three day period, affecting the stability of a well built levee, with cut-off wall standing 100 ft. back from the bank. Hence additional bank protection was not included in the proposals.

In making the estimates the height as adopted for this levee was such that it would not be overtopped by the jet. It is possible that an overtoppable levee would be just as effective in diverting the flow. On the eastern side, there would be slack water to within a few feet of the top so that little damage would be caused by such overtopping.

It is proposed that further model tests be made to ascertain the minimum height of levee necessary to divert the jet and to investigate the effect of a levee curved in plan, but in this report the estimates have been based on a straight non-overtoppable levee.

E5.62 Alternative Designs

Possibly in the final designs the Authority responsible for construction of the flood works may prefer an Ambursen type concrete structure without an overlying grassed bank. This would no doubt be effective, and probably cheaper, but may not be as pleasing in appearance.

The design adopted for estimating purposes will divert the jet and the use of this estimated cost in benefit-cost studies is satisfactory for comparison of the types and degrees of flood mitigation.

However, the proposed new arterial road crosses the Tamar in the vicinity of Royal Park, and the Director of Public Works (R.Sharp) suggested that a semi-circular rock breakwater would serve the same purpose as a training levee and that by reclaiming and grassing the area behind the breakwater additional park lands would become available, and a better and perhaps cheaper design of bridge and approach roads would be possible.

From a purely flood mitigation point of view, this would be more costly and it would be inappropriate to use the cost of such a scheme on the benefit-cost studies. However, from the point of view of the general development of the area, the proposal has much to commend it.

The decision was therefore made to carry out further model tests to evaluate this conception, and also to try out various related proposals such as the use of groynes. These will be dealt with in a supplementary report.

If a reclamation idea is adopted, it would have the secondary benefit that Cataract shoal, an unsightly mud flat which spoils the appearance of the Tamar at this point at low tide, could be pumped into the reclamation area, thus improving the appearance of the river, and facilitating the escape of the South Esk waters down Home Reach.

A further subsidiary benefit of a curved breakwater is worthy of mention. The Marine Board incurs fairly considerable maintenance costs in dredging Home Reach and Stephenson's Bend and finds that one of the cheapest and most effective methods of improving the navigation channels is to stir up the silt by dragging a rake behind a tug at times of minor freshes in the South Esk. The curved breakwater would possibly improve the scouring effect of these freshes.

E5.63 Scour of Banks

Even without the training levee, there is a marked back-eddy from flood discharges which is scouring the bank of the Tamar at the head of Home Reach, and some form of bank protection is desirable under existing conditions. In times of major flood with a training levee this scouring action will be quite strong. The curved breakwater would eliminate this eddy.

Consideration was given to the eroding effects of the diverted flow along the east bank of Home Reach velocities being approximately 12 ft./sec. and 20 ft./sec. for 150,000 and 250,000 cusecs respectively. Apparently this effect was not serious in 1929. In ivew of the rarity of major floods it is felt that no additional bank protection is needed along this shore.

E5.7 Paterson St. Levee

Because of the high degree of development in this zone and the obvious fact that the cost of protection against the maximum probable flood would be trivial, it was decided that no damage estimates or benefit-cost calculations were necessary in this area and the costs were estimated for protection against a discharge of 250,000 cusecs.

These costs allow for a concrete levee from Paterson St. at Ritchie's Mill, across the adjacent park on the river side of the memorial and cutting back into the high ground at the rear of the L.C.C. sewage pumping station. The top of this levee is 31 ft. above S.L.W. and its average height above ground level is 6 ft. The estimated cost is £17,000.

There are two alternatives to this proposal, which may possibly be cheaper and may warrant further consideration for the final design, viz:

(i) Raising of the present park combined with a short length of concrete levee.

(ii) Replacing the proposed concrete levee for the majority of its length by earth banks.

However, as it was considered that of the three schemes a concrete promenade would blend the most favourably with the present park and would be the most aesthetically pleasing, estimates of the cost of the alternative proposals were not calculated.

E5.8 Ancilliary Expenditure

E5.81 Access Through Levees

There are 29 points in the final design where access must be provided through or over the levees. A number of these structures are of quite a minor nature, full details being listed in Table 16.

Some of the important crossings are discussed below:-

(a) Tamar St. and Charles St. Bridges

It is felt that the levee system could merge in with the bridge approaches in such a manner that the bridge decks and approach roads are protected from the probable maximum flood.

(b) Main Railway Line Bridge for N.Esk

A preliminary design for closure gate and estimate of cost was made by the Chief Civil Engineer of the Tasmanian Government Railways for gates at each end of the bridge for flood levels of 23 ft. and 20 ft. The total cost of the two gates for the 23 ft. level as submitted by the Chief Civil Engineer was £5,000 and for the lower level £3,000.

These estimates of cost were used as a basis for approximate estimates of costs of all other closure gates by plotting a graph of cost against gate height per foot of width.

(c) Access to Wharves

Details of the heights and widths of all the wharf access arrangements are given in Table 16. In Vol.III will be found photographs of the kind of structure which might be designed for these gates, although of course in many cases the heights will be different from those shown in these photographs. For example, the height of the Lindsay St. access gates to King's Wharf for protection against probable maximum flood is 5.5 ft., whereas for the Foster St. access the height is 13 ft.

It will be noted that the erection of the levee provides for the protection of Harrap's Wool Store near Alexander Wharf. This would involve building a new access road on the southern side of the wool store and providing for access doors and loading docks on this side.

(d) Roads and Pathways generally

In many cases ramping of roads and pathways over the levees is possible. This alternative to construction of a gate is preferable, and should carefully be examined in all cases when preparing the final designs.

No detailed designs were made for any of these structures, the whole basis of the costing being by comparison with the Railway Department estimates referred to in (b) above.

E5.82 Sewerage and Local Drainage and Mowbray Swamp Improvement.

No detailed surveys of existing sewers and drains were made, nor was detailed consideration given to the manner in which these might require modification as a result of the construction of levees. Broadly speaking, the only modifications to existing sewerage and drainage reticulation would be at those points where the cut-off wall of the concrete levee reaches the depth at which the sewer is laid. An approximate estimate of expenditure on this count was made for each zone and height of levee.

In proposing any levee system, consideration must be given to the disposal of storm run-off behind the levees. For Inveresk-Invermay and Royal Park-Cypress St. zones, provision already exists in the city sewerage and drainage system. In the case of the Mowbray zone, an important dual benefit is achieved by the proposed scheme because the provision for local drainage, which must be made as part of the scheme, automatically affords an opportunity to drain this swamp.

The estimates in this report provide for a drainage and pumping system to drain from Mowbray Swamp all local run-off and ground water. A positive pumping system has been preferred, because experience has shown that flood gates under the railway embankment are a constant source of trouble. This drying out of the swamp should enable development to proceed.

Similarly the existing flood gate under the main railway line, taking local storm water drainage from the Willis St.-Cypress St. zone has apparently never functioned properly, and the proposals in this report provide for cleaning out of the dense blackberry and other growth in the main drain, and provision of a sump and float operated electric pump instead of a flood gate.

Under existing conditions this zone is flooded several times every year by minor freshes in the North Esk and by high tides, so that this local drainage, essential for the recommendations in this report, will have the dual benefit of flood mitigation for minor North Esk and high tide flooding, as well as for major South Esk flows.

E5.83 Land Resumption and Demolition

Where the levee system passes through private property or involves the demolition of any structure, appropriate allowance for resumption of land and compensation of the householder has been made in the estimates.

E5.9 Final Estimates of Cost

The cost estimates for the various zones are shown in Tables 17 to 22. A summary of the total costs for protection against the 1929 flood and the maximum probable flood is scheduled below:-

TABLE E5

SUMMARY OF COSTS OF PROTECTION OF THE VARIOUS ZONES AGAINST 1929 AND MAX.PROBABLE FLOODS

Section	Total Cost for Protection Against 1929 Discharge	Total Cost for Protection Against Max. Probable Flood
1. Royal Park Training Levee	26,200	84,700
2. Inveresk-Invermay Zone Levees	107,000	242,000
3. Royal Park-Willis St. Zone Levees	74,500	127,000
4. Willis-Cypress St. Zone Levees	39,200	93,000
5. Mowbray Flats Zone Levees	18,200	81,000
TOTAL	£265,100	£627,700

NOTES:-

(i) Construction of the Paterson St. levee to protect this zone from the maximum probable flood is estimated to cost £16,950.

(ii) Costs in this table for the protection of the various zones by levee banks are based on the assumption that a training levee is the first stage of construction.

E6 ANNUAL BENEFIT - ANNUAL COST CURVES

E6.0 Introduction

Fig. 43 shows an idealised text book example of a graph of annual benefits plotted against annual cost for various degrees of development of a flood mitigation scheme. In the Launceston case, the degree of development means the height to which the levees should be raised.

Point (1) on the graph is the stage of development at which the ratio of annual benefits to annual cost is greatest. Point (2) on the graph is that on which benefits exceed the costs by the greatest amount. Point (3) is the stage of development at which the benefit equals the cost.

If the levees are raised to give the degree of protection represented by point (1) on this graph, the ratio of benefit accrual per unit of cost would be at a maximum, but the full economic possibilities of flood mitigation works would not be realised as there would still remain additional increments of flood protection for which benefits exceed costs. Beyond point (2), although the overall ratio of benefits to costs is greater than unity, the benefits added by each increment of further protection are less than the cost of the increased protection. Hence raising the levees to give protection beyond point (2) is not justified on purely economic grounds. In some cases an extension of the degree of protection into this zone might be justified by associated intangible benefits, such as the commercial value to Launceston of the psychological effect on future investors of the claim that the area has been protected from the maximum probable flood, thereby removing completely the stigma of flood danger.

On the other hand, consideration must be given to the availability of capital funds. Even although point (2) may represent the theoretical height to which to raise the levees, money may not be available to allow this to be done. In such a case this practical consideration might result in the adoption of a degree of development somewhere below . point (2).

If it is necessary to compare two different methods of flood mitigation, then a graph as in Fig. 43 would be drawn for each method and the ratio of benefits to cost at point (2) on the graphs should be compared. The one with the highest ratio for the same degree of protection would be the better economic proposition.

If, for the reasons discussed above, the degree of development adopted for design purposes differs from that represented by point (2) then the benefit cost ratio for this particular point should be chosen for comparing the two methods.

Tables 23 to 26 of the appendices give details of the computations. In these tables two conditions are examined:-

(i) With a 6 hour period of vigorous emergency measures prior to flooding of premises.

(ii) With a 24 hour period.

It will be noted that in the Tables it has been assumed that for the South Esk discharges of 90,000 cusecs or less and corresponding North Esk discharges, no appreciable damage would occur, sand bags and other stop gap measures keeping the flood back. Floods exceeding 90,000 cusecs will cause damage.

Another assumption is that floods equalling 250,000 cusecs will occur once over a 10,000 year period, but no floods greater than this can occur. This differs from the more usual practice of assuming that the maximum probable flood cannot occur and has a probability of zero.

In computing the annual costs, an interest rate of 5 per cent per annum was adopted and the capital expenditure amortized over 70 years. In other words, the average annual cost of any type and degree of flood mitigation is an equal yearly payment, of which in the early years the major portion goes to pay interest and the balance to a sinking fund, whereas the later years the major portion is allocated to the sinking fund and the balance to interest. Maintenance costs were included in the annual costs.

E6.1 Construction of Training Levee Only

E6.11 Method of Computation

Table 23 gives the details and the method may be illustrated by reference to that table. This expresses in Cols. (3) and (4) the general flood level reached for the various discharges with and without the training levee obtained from Fig. 34. Col.(2) gives the period of emergency measures assumed in estimating flood damages. Col. (5) shows the estimated probability of occurrence of floods equal to or greater than those listed in Col.(1). For example, a flood of 100,000 cusecs or greater is expected 51 times per 1000 years and one of 125,000 cusecs or more, 33 times in that period, so that Col. (6) expresses the conclusion that 18 floods between these two discharges may be expected. The data for Col. (5) is obtained from Fig. 30. Cols.(7) to (12) give the estimated damages caused by floods of various magnitudes which, without protection, would result from the flood levels listed in Col. (3). The information needed for these columns is obtained from Figs. 38 and 39. It will be seen that a discharge of 100,000 cusecs will be responsible for damages of £52,000 and one of 125,000 cusecs would increase this to £1,762,000. However, if a training levee were built high enough to divert a South Esk discharge of 125,000 cusecs the damages would be reduced to £1,729,000 for this flood, while having no effect on the 100,000 cusecs damage, as shown in Col. (18). The difference between £1,762,000 and £1,729,000 is £33,000, which is entered in Col. (19). The corresponding saving in damage for 100,000 cusecs is nil, and the average of this and £33,000 is £16,500 listed in Col. (20). This is assumed to be the damage saved by the training levee for a flood midway between 125,000 and 100,000 cusecs (112,500). Now from Col. (6) it is seen that 18 floods in the range 100,000 to 125,000 cusecs are expected every thousand years. Some will be slightly larger than 100,000 cusecs and others will almost reach 125,000, but it is reasonable to assume that the total damage done by this group of future floods would be the same as if 18 floods of exactly 112,500 cusecs occurred. Hence 18 times £16,500 is the total saving over 1000 years which would be effected for floods in this range by building a levee high enough to divert 125,000 cusecs, and division by 1000 gives the average annual benefit of £300 for this group of floods as shown in Col. (21). By the same reasoning, a levee high enough to divert 150,000 cusecs would reduce by £4,870 the average annual damage caused by floods in the range 125,000 to 150,000 cusecs. But this levee would also eliminate the damage from floods in the 100,000 to 125,000 cusecs group. Hence the total average annual benefit from a training levee of this height is the sum of £300 and £4,870 or £5,170. This cumulative sum is entered in Col. (22). (It is probable that a levee of height suitable for 150,000 cusecs would also reduce damages caused by floods greater than this, but such benefit has been ignored in the calculation). Knowing the capital cost of the levee, the annual cost is computed by allowing 5 per cent interest charges and a sinking fund to repay the capital in 70 years and is given as £1,360 in Col. (23). The ratio of annual benefit to annual cost is thus Col. (22) divided by Col. (23) and is shown in Col. (26) as 3.81.

The increase in annual cost as the training levee is increased in height is shown in Col. (24), while the corresponding increment of benefit is in Col. (21). The ratio of Col. (21) to Col. (24) expresses the slope of the benefit cost curves, and is an indication of the degree of protection which is economically justified. This ratio for 150,000 cusecs protection given in Col. (25) of 6.16 shows that raising the levee from 125,000 to 150,000 cusecs is amply justified.

The stage damage curve used in the above tabulation makes no allowance for enhancement and development. For such allowance it is necessary to double the abscissae of the stage damage graph, which has the effect of doubling the ratio of (increment to benefit) to (increment of cost) and of doubling the benefit-cost ratio. Therefore, Col. (27) is Col. (25) doubled, while Col. (28) corresponds to the doubling of the benefit-cost ratios in Col.(26).

E6.12 Discussion of Results

It will be seen from Col. (27) of Table 23 that to increase the training levee from the height necessary to divert 200,000 cusecs to that required for 225,000 cusecs means that for every pound per annum of expenditure a benefit of £4.86 per annum will be obtained and the average benefit-cost ratio of such a project would be 13.14. Obviously it is a sound business investment to build a training levee of this height (assuming 24 hour emergency measures).

If the height is increased to divert the maximum probable flood of 250,000 cusecs, Table 23 shows that a return of £0.64 per £1.00 of increased expenditure is obtained, so that some of this last increment of protection is not justified if financial considerations are paramount. The limiting point for economic development is that at which the tangent to the benefit-cost curve reaches an angle of 45° illustrated by (.2) of Fig.43. By drawing a graph of the benefit-cost ratio given by table 23 it is seen that the limiting point is that at which protection against 234,000 cusecs is provided. However, the additional capital cost of raising the levee from protection against 234,000 cusecs to protection against 250,000 cusecs is only £16,200 and the annual cost £0.84.

There is considerable psychological and intangible value in being able to state that "complete" protection is provided, so that it is reasonable to argue that intangible benefits justify this last element of expenditure necessary to divert a South Esk jet of 250,000 cusecs, bearing in mind that the overall benefit-cost ratio is 9.48 as shown in Col. (28).

However, flood protection by the use of a training levee only is hardly likely to appeal to the citizens of Launceston, as the flood level, although reduced, would still be higher than the existing levees, and even the smaller floods would thus cause considerable disruption of the life of the city.

The obvious development is to combine a training levee with "surround" levees for such built up areas as should be protected. To build such levees without a training levee would not be a sound proposition. For example, the building of a training levee 1600 ft. long and approximately 18 ft. high would reduce by 3.6 ft. the height of approximately 20,000 ft. of levee around the Inveresk-Invermay zone alone (for maximum probable flood).

Therefore, the next step in the benefit-cost study is to examine the various zones or combination of zones to calculate the benefit-cost ratios of surround levees of various heights, assuming that a training levee has already been constructed of sufficient height to divert a discharge of 250,000 cusecs in the South Esk.

E6.2 Construction of Surround Levees for the Various Zones as a Further Stage of Development After Construction of Training Levee to Divert South Esk Discharge of 250,000 cusecs.

E6.21 Method of Approach

The economics of the surround levees for the various zones can be approached in any one of three ways:-

(i) By considering that the construction of the training levee is stage one of an overall flood mitigation scheme. There already exists the benefit due to the construction of the training levee, and the benefit to be considered in this case is only that additional annual benefit achieved by the building of a surround levee for the zone in question. This should be divided by the annual cost of such surround levee to obtain the benefit-cost ratio.

(ii) By taking the view that each zone to be protected by surround levees should be debited with a proportion of the cost of the training levee as well as the cost of the surround levee for the zone. The annual benefits are the combined benefits achieved by the training levee and the zone levee regarded as the one mitigation scheme. The annual cost is the total cost of the surround levee plus a proportion of the cost of the training levee.

(iii) By treating, for a given zone, the surround levee and the training levee as a single project necessary to protect that zone alone. The annual benefits in question are those achieved in that zone by the combined effect of the training levee and the surround levee. The annual cost is the total cost of the training levee plus the cost of the surround levee.

It is considered that method (i) above, is the correct approach in this case.

As will be realised from the discussion in Section E6.1 above, the basic principle of benefit-cost analyses is to compute for different types and degrees of flood protection, the average annual damage which would occur without such protection and that which would occur with flood protection. The difference between the two is the average annual benefit and this is compared with the average annual cost of the specified type and degree of protection.

In a case such as Launceston, with a training levee and four zones to be protected by surround levees, the general case would consist of a series of tabulations. The heading of each tabulation would specify the heights to which the training levee and the various zone levees would be built for each zone. A tabulation would be made showing the flood heights reached for different discharges with and without the specified flood protection. By reference to the stage damage curves the corresponding damages with and without protection would be computed and hence the average annual benefit obtained. The total average annual benefit from all the zones would be entered in the final column and the grand total would give the average annual benefit for that particular proposal. For each tabulation this would be balanced against the average annual cost in the same manner as described in Section E6.1. However, the high benefit-cost ratio for training levee only means that this general approach should be modified by the assumption that in all cases a training levee to divert 250,000 cusecs is first constructed.

E6.22 Treating the Surround Levee as a Separate Distinct Stage of Flood Mitigation Programme

E6.221 For the Case Where One Zone Only is Leveed

E6.2211 Details of Computation. These calculations are shown in Tables 24 and 25 of the appendices, and may be described with reference to the case of construction of levees of various heights around the Inveresk-Invermay zones (Table 24). Col.(1) lists the various flood discharges and Col.(2) the period assumed to elapse from time of commencement of removal of goods to time of flooding. Col. (3) shows the general flood level reached for the given discharge when a levee of height sufficient to divert a South Esk jet of 250,000 cusecs is built, but the surround levees around Inveresk-Invermay remain at their present general level of 17 ft. These flood heights are obtained from Fig.34 representing the results of the model studies. The frequency with which the given discharge will be equalled or exceeded is shown in Col. (4) and the frequency of floods midway in the various heights is entered in Col. (5). The damage caused by floods of various heights, as obtained from the stage-damage graphs of Figs.38 and 39 is recorded in Col. (6). The damage caused by the average flood of each group of floods is the mean of consecutive values in Col. (6) and appears in Col. (7).

The average annual damage per year for each group of floods is calculated by multiplying this value by the number of floods per annum in the given range, Col. (5), and the answer is given in Col. (8).

A levee of height 19.1 would prevent all damage by floods less than 150,000 cusecs, so the average annual damage prevented or "benefit" from such a levee is the cumulative sum of damage prevented by all groups of floods smaller than 150,000 cusecs, and in this manner Col. (9) is completed.

In this case it is assumed that the training levee construction is an independant first stage of a flood mitigation scheme, and it is desired to ascertain the economic height of Inveresk-Invermay levees considered as an entirely separate project. Hence the annual cost of building Inveresk-Invermay levees only to the heights to protect against levels in Col.(3) should be shown in Col.(10). This cost makes due allowance for appropriate freeboard, Col. (11), (12), (13), (14) and (15) are based on reasoning similar to that discussed in Section C6.1.

E6.2212 Discussion of Results. To ignore the effects of enhancement and development is out of the question in any rational discussion. The allowance for this factor in the tables is conservative (see Section E1.6). On the other hand, the assumption of a 6 hour period of warning may be too pessimistic. In 1929 the Mayor set up in motion emergency preparations 12 hours ahead of actual flooding, and presumably warning and emergency arrangements should be better on the occasion of the next flood. Perhaps a 24 hour period of emergency measures is too optimistic. However, consideration of the 24 hour values with allowance for enhancement and development gives a conservative basis for discussion and is adopted herein.

For the Inveresk-Invermay zone (Table 24), the last ratio of increment of benefit to increment of cost is 1.98 and the benefit-cost ratio for protection against the maximum probable flood is 13.46. However, this does not necessarily mean that the last increment from 225,000 to 250,000 cusecs is justified, and in fact a plotting of the benefit-cost curve gives the limit as 240,000 cusecs. However, this is so close to the probable maximum flood value of 250,000 cusecs that it is reasonable to assume that protection against this flood is justified. It will be noted that this benefit-cost ratio is greater than that for the training levee, and at first glance it might seem that the Inveresk-Invermay levees should have a higher priority in construction than the training levee. Such reasoning is fallacious. Reference to Fig. 34 shows that if levees are built around Inveresk-Invermay without a training levee being first constructed, the heights of this levee to protect against the probable maximum flood would be very great indeed and the benefit-cost ratio would be much less favourable.

Turning now to the Mowbray zone (Table 25) it will be found that by plotting the benefit-cost curve the limiting degree of protection on purely economic grounds is 203,000 cusecs at which the benefit-cost ratio is 2.71. It is felt, however, that purely economic considerations should not be paramount in this case. The allowance for enhancement and development adopted throughout this investigation has been a simplified one and the same degree of development has been assumed to apply to all zones. While this might be reasonably close to the truth in the case of the other three zones, the Mowbray zone is a special case. Flood protection and consequential drainage of this zone would open up a most valuable area close to the centres of development. Much of it is at present swamp land and the enhancement factor would be much greater than in other zones. Furthermore, a good deal of property of Mowbray Flats is owned by the L.C.C., so that the betterment increment from flood protection here would accrue to the general public. It was therefore decided that the increase in capital cost (£18,400) involved in raising the levees to the height necessary for the maximum probable flood is justified.

The Royal Park - Willis St. area (Table 23) has a limiting degree of development of 232,000 cusecs with a benefit-cost ratio at this point of 2.68. Here again the figure is so close to 250,000 cusecs that protection against the maximum probable flood is justified.

For the Willis St. - Cypress St. zone (Table 24) the limiting discharge is 225,000 cusecs with a benefit-cost ratio of 2.86, these values being obtained by plotting the benefit-cost curve from the table. However, the same reasoning applies to a degree to this zone as to the Mowbray zone and it would be foolish to omit the last degree of protection costing £6,800.

It is felt, therefore, that the intangible psychological value of providing protection for all zones against the estimated "maximum probable flood" justifies carrying the protection of all four zones to this degree, when considering:-

(i) Training levee to divert 250,000 cusecs combined with Inveresk-Invermay surround levees and no other levees (Table 24).

(ii) Training levee to divert 250,000 cusecs combined with Royal Park - Willis St. surround levees and no other levees (Table 24).

(iii) Training levee to divert 250,000 cusecs combined with Willis St. - Cypress St. surround levees and no other levees (Table 25).

(iv) Training levee to divert 250,000 cusecs combined with Mowbray Flats surround levees and no other levees (Table 25).

If this reasoning is accepted, it leads to the conception of the principle of "equal protection for all built up zones" but it does not necessarily follow that the degree of protection for the complete scheme should be that necessary to cope with "the maximum probable flood". The reason for this is that when all four surround levees are built, the flood levels in the lower N.Esk (for a given discharge) will be greater than when levees are assumed to exist around one zone only, as in (i) to (iv) above, and hence the levees for a given degree of protection must be higher and more expensive. This may be illustrated by reference to Table 24. The levels in Col.3 are those shown by Fig. 34 for training levee and Inveresk-Invermay levees only. However, if a levee also exists from Royal Park to Cypress St. or Willis St. high enough to protect against 250,000 cusecs the water in the lower N.Esk will be confined to a narrower channel, and its height will be raised to the levels shown in Fig. 45.

Therefore the final and vital benefit cost study is that for a training levee to divert 250,000 cusecs, combined with

(i) All four surround levees to protect against 125,000 cusecs
(ii) " " " " " " " 150,000 "
(iii) " " " " " " " 175,000 "
(iv) " " " " " " " 200,000 "
(v) " " " " " " " 225,000 "
(vi) " " " " " " " 250,000 "

This analysis of the final proposal of "training levee to divert maximum probable flood and equal protection for all built up zones" is discussed in Section E6.222 below.

It could of course be argued that economic analyses should be carried out for cases such as a training levee to divert 250,000 cusecs, surround levees for Inveresk-Invermay and Royal Park-Willis St. for 250,000 cusecs, and surround levees for Mowbray Flats and Willis-Cypress St. for various degrees of protection less than 250,000 cusecs.

This is a possible approach based on the following reasoning:-

(i) The Inveresk-Invermay and Royal Park-Willis St. zones are at present the most heavily developed of the four zones.

(ii) A lower degree of protection for the Mowbray zone allows a safety valve of escape of flood waters across Mowbray Flats in the unlikely event of the occurrence of a flood exceeding 250,000 cusecs.

(iii) The Mowbray Flats and Willis St. - Cypress St. levees are entirely of earth construction and thus lend themselves readily to progressive increase in height as development proceeds in these zones over future years.

The author took the view, however, that the Mowbray Flats and Willis St. - Cypress St. zones have considerable potentiality for industrial development, provided prospective industry can be assured of a degree of protection equal to that of the zones which are at present in a higher state of development. Hence no analysis was made of a final scheme providing different degrees of development. although it is conceded that if the works are to be built in stages over a period of years, the less developed zones should be given a lower priority in the programme of construction.

E6.222 For the Case Where the Four Main Built-Up Zones are Leveed to the Same Degree of Protection.

Having decided that all zones are to be protected against the same magnitude of flood (the "maximum probable"), the next stage is to compute the benefit-cost ratio for such a proposal.

The calculations were carried out in a series of Tables, of which Table 26 is an example. This considers the case of the training levee to divert 250,000 cusecs, combined with surround levees for all four zones of height adequate to protect the zones against a S.Esk discharge of 150,000 cusecs concurrent with a N.Esk discharge of 13,100 cusecs. Cols.(3) to (11) consider the annual benefits for the leveed zones, and Cols.(12) to (18) those for the wharf areas which cannot be protected by levees. Col. (1) specifies the various S.Esk discharges and Col.(2) the period of emergency measures. Col.(3) shows the flood levels reached in the lower N.Esk under existing conditions (from Fig.34). Col.(4) shows the corresponding levels in the lower N.Esk region when the training levee and all four zone levees have been built.

These levels cannot be read off the curve on Fig.34 marked "Invermay-Esplanade levees with Royal Park training levee", because this curve is for the case of unassailable levees around all zones. If, for example, the surround levees are only high enough to protect against 150,000 cusecs, then for discharges greater than 150,000 cusecs the flood waters will spread into the zones and the extra storage and outflow thus permitted would result in lower flood levels for the higher discharges. This effect is expressed in Fig.45 which provides the levels for Col.(4). This figure could be compiled by carrying out model tests with levees of various heights, but it was felt that such expense was not warranted for an economic analysis, and it has therefore been compiled by interpolation between the "training levee only" and "Inveresk-Esplanade levees with Royal Park training levee" curves of Fig. 34.

Cols. (5) and (6) are similar to those in the tables previously discussed. Col. (7) gives the total estimated damages for various flood levels for all built up zones except the wharf areas.

Col. (8) shows the total damage in the four zones for conditions when the surround levees will keep out floods up to 150,000 cusecs. The first four entries are of course "nil". For this height of surround levees, a flood of 175,000 cusecs will overtop the levees giving a flood level in the zones of 20.50 (Col.4) and causing damage in these zones of £1,813,000 as given by Fig. 38. Similarly the damages for still higher discharges are entered in Col. (8) by the use of Figs. 45 and 38.

For the case of a flood of 100,000 cusecs, levees to protect against 150,000 cusecs will reduce the damage from £30,000 (Col.7) to nil (Col.8) so that for this flood the benefit will be £30,000. For a flood of 125,000 cusecs the benefit will be £1,382,000. For a flood midway between these two values the benefit may be taken as the average of these two values, or £706,000 as shown in Col. (10). There are 180 floods per 1,000 years in this range and it is reasonable to argue that the total damage saved in 1,000 years would be 180 x £706,000, so that the average annual benefit due to these levees for this group of floods would be £12,720. By similar reasoning the rest of Cols. (9), (10) and (11) are completed.

The same reasoning is applied to the case of the unleveed zone (wharf areas) in Cols. (12) to (18). In considering the levels to be inserted in Cols. (12) and (13), it will be realised that for a given discharge, flood levels in Home Reach near King's Wharf will differ from those in the lower N.Esk near Queen's and Alexander wharves, and for strict accuracy the wharf zone should be divided into two sub-zones. However, the Marine Board report on probable wharf damage indicated that the major portion of the damage would occur at King's Wharf. Therefore, for the sake of simplicity and with little sacrifice of accuracy Cols. (12) and (13) give the levels in Home Reach, which are assumed to apply throughout all unleveed zones. Col. (12) is obtained from the model tests for 1959 topography and Col. (13) for model tests in Home Reach for unassailable training and surround levees. The total average annual benefit for any given group of floods is of course the sum of

Cols. (11) and (18) and is shown in Col. (19). The sum of all the values in Col. (19) gives the total average annual benefit for all the groups of floods, which for 150,000 cusec surround levees and no allowance for enhancement and development is £52,455. Enhancement and development cannot be ignored, so this value is doubled. The total annual cost of the levees specified in Table 26 is £16,650, and the benefit-cost ratio 6.29

Tables similar to Table 26 were completed for zone surround levees of 100,000, 200,000, 225,000 and 250,000 cusecs, and the benefit-cost graph plotted as in Fig. 44. On this graph are also plotted:-

(i) The relationship between benefit-cost ratio and total capital and annual cost.

(ii) The relationship between the increment of benefit/increment of cost ratio and the total capital and annual cost. The subsidiary graphs are compiled from the main benefit-cost graph.

It will be seen that the point where the increment of benefit/increment of cost ratio is unity occurs at the point where the annual cost is £28,400 per annum. From the data showing annual cost of various heights of levees, with due allowance for freeboard, it can be shown that this annual expenditure will permit protection of the various zones against a S.Esk discharge of 223,000 cusecs, with the appropriate concurrent N.Esk discharge.

For this complete scheme, the author considers that the economic limit (223,000 cusecs) is so close to "complete" protection against the maximum probable flood for all built up zones that intangible benefits justify raising the degree of protection of all these zones to 250,000 cusecs.

E6.23 Charging the Leveed Zone with a Proportion of the Training Levee Costs.

Calculations in this case would be similar to those of the tables referred to above in Section E6.221, but in Col. (3) the flood level to be given would be that which would be reached without any training levee in position and damages in Col. (6) would be correspondingly altered, with consequential changes in appropriate remaining columns.

The proportions of the cost of the training levee to be allocated against the different zones could be fixed in several ways. The best method would probably be to compute the area enclosed between curves in Figs. 38 and 39 and the vertical axis of the graph and then to sum them to give a grand total area enclosed by the stage-damage graphs for the total flood plain. For each zone, the proportion of cost of the training levee to be borne would be the ratio of the area in the zone graph to the area in the total flood plain damage graph.

If this basis were adopted the factors would be:-

Wharf Zone = 0.0985
Inveresk-Invermay Zone = 0.70
Royal Park-Willis St.Zone = 0.108
Willis St.-Cypress St.Zone = 0.055
Mowbray Flat Zone = 0.0386

E6.24 Charging the Zone in Question with the Total Cost of the Training Levee as well as the Cost of the Surround Levees

This approach would be the correct one if it were decided to protect only one zone. The benefits must be computed separately in three parts as follows:-

(i) Benefits in the unleveed zone due to the mitigation scheme, due to general lowering of flood levels by the training levee.

(ii) Benefits in the leveed zone which would be achieved by levees of various heights protecting that zone.

(iii) Benefits in the leveed zone due to the fact that even when the levees are overtopped, the training levee will cause the flood levels to be less than would exist if the training levee had not been constructed.

This computation has been carried out for one of the zones, but is not reproduced in this report because it is felt that there is no likelihood in the Launceston case of one zone only being protected and thereby having to bear the total cost of the training levee.

E6.3 Projects other than Levees

E6.31 Hunter Cut

An expenditure of £650,000 on completing the Hunter Cut would reduce the general flood level of the maximum probable flood by 0.7 ft. and the 1929 flood level by 1.4 ft.

It is therefore clear that on economic grounds this project cannot compete with the levee projects, and benefit-cost calculations were not carried out.

If it were combined with levees, it would lower the required height of the maximum probable flood levees by 0.7 ft. with a resultant saving of £35,000.

Therefore this project cannot enter into consideration for flood mitigation purposes. It would provide benefits in navigation and in port maintenance, but would only be justifiable if these benefits warranted a capital expenditure of £615,000, which is the difference between the estimated cost of the Hunter Cut and the amount saved in levee construction if it were combined with levees.

E6.32 Floodways and Channel Improvements

The hydraulic efficiency of these measures as revealed by the model are so low that estimates of cost and benefit cost curves were not calculated.

E6.4 Saving in Cost of Embankment for Arterial Road

Some consideration is being given to the construction of a new arterial road as shown in Fig. 37, and the present thinking is that the road level should be such that it will not be submerged by a flood equal in magnitude to that of 1929. Without a flood mitigation scheme, the embankment must be 20.7 ft. above S.L.W. to achieve this result.

If a training levee adequate to divert the 1929 discharge is constructed in Royal Park, the road surface can be lowered to 19.3 ft. for the length from A to C in Fig. 37.

If, in addition, levee protection is provided for the city and Inveresk-Invermay areas as proposed in this report, then the arterial road embankment level from A to B in Fig. 37 would need to be 19.8 and from B to C the road can be at natural ground level.

If all of the material for the embankment must be obtained from sites some distance from the location of the road, unit costs would approximate:-

(i)	Win and borrow	7/6d.	per C.Y.
(ii)	Trucking and placing (5 miles)	7/6d.	" " "
(iii)	Trimming and compaction	2/6d.	" " "
	Total -	17/6d.	" " "

On this basis the cost of construction of the road would be reduced by £125,000 as a result of providing levee protection against the 1929 flood. This figure does not allow for the saving in cost of stone pitching or other protective measures for the bank, nor for the saving in retaining wall costs in the heavily built-up areas.

If the final road design provides for a road level less than 1929 flood level, the benefit would be reduced somewhat. Nevertheless, an appreciable saving in arterial road costs must result from the construction of flood mitigation works.

This is a legitimate direct benefit of these works, but no allowance has been made for it in computations of benefit-cost curves, which give high benefit-cost ratios without allowing for this factor.

E7 CONCLUSIONS

The following conclusions arise from the studies in this section.

(a) The Cormiston Creek diversion is prohibitively costly.

(b) The construction of the Hunter Cut, purely as a flood mitigation measure, is not justified.

(c) Floodways are of negligible value for flood mitigation purposes.

(d) Smoothing of bends and channel improvement in the Tamar are not practicable or effective flood mitigation measures.

(e) The construction of a training levee to divert a S. Esk discharge of approximately 234,000 cusecs is justified on purely economic grounds to reduce depths of flooding all over the flood plain, and is the essential first step in any flood mitigation proposals (benefit-cost ratio 11.85).

(f) The intangible benefits arising from the provision of protection against the estimated "maximum probable flood" in the S. Esk of 250,000 cusecs justifies increasing the height of this levee to divert such a discharge. The estimated capital cost of such works is £84,700 and the benefit-cost ratio is 9.48, but this estimate of cost may be reduced in the light of further studies to be dealt with in a supplementary report.

(g) If the training levee of (e) above is combined with surround levees for each of the built up zones, but only one such zone is so protected, purely economic considerations justify the following degrees of protection for the various zones, expressed in terms of S. Esk discharge:-

(i)	Inveresk-Invermay	- 240,000	cusecs	($\frac{B}{C}$ 14.1)
(ii)	Royal Park-Willis St.	- 232,000	"	($\frac{B}{C}$ 2.68)
(iii)	Willis St.-Cypress St.	-225,000	"	($\frac{B}{C}$ 2.86)
(iv)	Mowbray Flats	-203,000	"	($\frac{B}{C}$ 2.71)

(h) The variation in the degrees of protection economically justified as given in (g) above is so small and the encouragement of development in the less developed zones so important that the principle of "equal degrees of protection for all built up zones" should be adopted.

(i) On this basis the purely economic analysis of the overall plan for flood mitigation is expressed in Fig. 44, which shows that surround levees for built up zones giving protection against 223,000 cusecs in the S.Esk with concurrent discharge of 17,500 cusecs in the N.Esk, is justified, the benefit-cost ratio being 5.87.

(j) The intangible benefits of protection of the built up zones against the estimated "maximum probable flood" justify building surround levees adequate to protect against a discharge of 250,000 cusecs in the S.Esk with concurrent N.Esk discharge of 18,800 cusecs.

(k) Surround levees should not be constructed without first building a training levee as in (e) above and the cost of the complete proposal for the degree of protection defined in (j) above is:-

(i)	Inveresk-Invermay zone	-	£242,000
(ii)	Royal Park-Willis St. zone	-	£127,000
(iii)	Willis St.-Cypress St. "	-	93,000
(iv)	Mowbray Flats zone	-	81,000
(v)	Training levee	-	84,700
			£627,700

The benefit-cost ratio of the programme for construction of training levee as in (e) and surround levees as in (j) is 5.15.

(l) The existing Paterson St. levee should be raised to a height of 31 ft. above S.L.W. immediately after or at the same time as the construction of the training levee (capital cost £16,950).

(m) Expenditure of government funds on flood protection of the West Tamar bank and the grazing lands between the railway bridge and Hobler's Bridge is not at present justified.

PART F

SUBSIDIARY INVESTIGATIONS

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PART F SUBSIDIARY INVESTIGATIONS

F1 CAPTURE OF SOUTH ESK RIVER BY NORTH ESK

Near Evandale (lat. 41° 33' long. 147° 15'), which is 32 river miles and 6 miles in direct line south of Launceston, is an interesting example of future river capture. In this district the valley of the North Esk is several hundred feet below that of the South Esk. Rose's Creek, a tributary of the North Esk, is slowly eating its way by headward erosion into the South Esk valley. In the normal course of events, after many hundreds of years, this process will lead to the capture of the South Esk by the North Esk.

The following extract from a report of 22nd December, 1932, by the Tasmanian Government Geologist (P.B. Nye) gives the geological appreciation of this situation:-

"The difference in level between the two rivers has been caused by the effect of the different rocks over which the two rivers flow. The lower part of the South Esk, including the Launceston Gorge, is over and through diabase (dolerite) country which is a hard and resistant rock and considerably retards the vertical corrosion (cutting-down) of that river. On the other hand, the lower part of the North Esk Valley is in tertiary sands, gravels, clays etc., which are soft and easily eroded, and have not retarded the vertical (or the lateral) corrosion of this river to the same extent as the diabase has done in the case of the South Esk River.

The two rivers are closest to one another in the Evandale-White Hills district and both flow over country occupied by tertiary sediments and possibly interbedded basalt.

This district is drained by Rose's Creek, a tributary of the North Esk, which has a course centrally situated between the two rivers. This creek and its tributaries have eroded their valleys deeply into the soft tertiary rocks. The creek has a number of tributary streams on its southern side, all of which have their sources or heads in the Evandale - Western Junction area, and flow north into Rose's Creek. The heads of some of these streams are within half a mile of the South Esk River.

The country between Evandale and Western Junction is composed of the following strata in descending order:-

(1) Gravels, at least 20 feet thick
(2) Basalt, about 100 feet thick
(3) Gravels, sands and clays several hundred feet in thickness
(4) Diabase bedrock

The upper gravels have been mainly removed by denudations but a small remnant occurs on the divide to the west of the lowest part.

The basalt is exposed near Western Junction and to the north and west towards Launceston. Although it does not outcrop prominently, it occupies the divide between the lowest part and Western Junction. The rocks occupying portion of the divide between the lowest part and Evandale are not determinable due to the waste from the upper gravels covering the surface. It is anticipated, however, that the basalt will extend some distance at least towards Evandale.

The sub-basaltic gravels etc. are only exposed at lower levels such as the valley of Rose's Creek, White Hills etc.

It is anticipated that at the lowest point of the divide there will be several feet of gravelly soil overlying weathered basalt. While the weathered basalt may be more resistant to corrosion than the gravels etc., it would not be sufficiently resistant to prevent a channel being cut quickly in it. The less weathered the basalt, the more resistant it would be."

"From the above, it will be realised that the South Esk River is in danger of being captured by the North Esk River by two methods:-

(1) The natural headward erosion of the tributaries of Rose's Creek. This process will take some time under normal circumstances. The most vigorous of the streams are those near Western Junction and possibly an inspection of these at long intervals might be advisable. In general, however, the local residents could be relied upon to give warning of any pronounced headward erosion of these streams and possible danger arising therefrom. Any dangerous erosion could be prevented or delayed by stone or concrete walls.

(2) A high flood in the South Esk causing water to flow over the lowest point of the divide between the two rivers and to establish a permanent channel.

In the 1929 flood the water was only a few feet in level below the lowest point on the divide. A larger flood or a southerly, south-easterly or south-westerly wind banking up the water would tend to bring about overtopping of the saddle.

It is hardly necessary to point out that the capture would:-

(1) Cause the waters of the South Esk to flow permanently along the North Esk River.

(2) Destroy the Perth water supply scheme and any private ones along the South Esk River below Evandale.

(3) Deprive the Launceston Gorge of the greater part of its water supply.

(4) Cause loss of life and damage to property during the actual capture and the flooding of the Rose's Creek Valley."

The overtopping of the divide by a future major flood in the South Esk would be disastrous for landholders in the North Esk valley and also for the City of Launceston, because the North Esk channel runs through the city before reaching the Tamar, whereas the South Esk channel does not.

Therefore it was considered that the degree of danger from this event should be investigated. A survey was made, revealing that the lowest point in the divide is approximately 17 ft. above the 1929 flood level, but that this low portion was not of any great length. Further, there is a wide flood plain at a relatively low level on the opposite bank of the South Esk.

The estimation of flood stage likely to be reached at Evandale by the probable maximum flood is a very complex problem. In the light of this survey data, such estimation was deemed to be unnecessary, as there seems to be ample margin of safety.

However, the cost of raising the short section of the low divide to say 25 ft. above the 1929 flood level is so trivial that this precaution is well worthwhile.

F2 NEW BRIDGES OVER THE NORTH ESK RIVER

F2.0 Introduction

The Public Works Department of Tasmania is planning a high embankment and new bridge along Henry St. as part of a projected arterial highway, and proposes also to replace Hobler's Bridge. The existing approaches to both bridges are often closed to traffic by minor floods in the North Esk.

It was necessary to discover whether the Henry St. embankment would affect the flooding of Launceston, either before or after the proposed new levee system is constructed, and also to advise the Public Works Department on waterway areas and height of deck for the new bridges.

F2.1 Deck Level of Henry St. Bridge

The model results carried out to evaluate the various possible mitigation schemes enabled the following conclusions to be reached:-

(a) Under existing topographic conditions and with no further flood protective measures for Launceston, a flood of 150,000 cusecs in the South Esk simultaneous with 13,100 cusecs in the North Esk (1929 flood) would produce a water level at Henry St. Bridge 20.7 above S.L.W. This is approximately 1.0 ft. higher than actually occurred in 1929, the increased height being due to changes in topography at the downstream end of Home Reach.

(b) The corresponding level for the "probable maximum flood" is 25.0 ft.

(c) If a training levee 1,500 ft. in length and 18 feet high were built in Royal Park as portion of a flood mitigation scheme, the corresponding levels at Henry St. for 1929 and "maximum probable" discharges would be 19.8 and 23.6 respectively.

F2.2 Waterway Area of Henry St. Bridge

The model scale was not great enough to provide an accurate analysis of flow conditions in this small local area. However, as a general guide to flood conditions, an embankment 10 ft. high was built on the model along the line of Henry St., with an opening 275 ft. in length across the North Esk, and two tests were run:-

(i) 150,000 cusecs in South Esk with 13,100 cusecs in North Esk.

(ii) 100,000 cusecs in South Esk with 20,000 cusecs in North Esk.

The drop in level through the waterway area was 0.5 ft. for (i) and 1.0 ft. for (ii).

It was concluded that if the embankment and bridge are above the level produced by the above floods, the waterway area will be adequate, because of the width of the river channel at this point.

Another condition which might be considered is the case of say 15,000 cusecs flowing in the North Esk, with only a minor flood in the South Esk and hence no backwater effects at Henry St. On account of the small scale of the model, this condition is best studied by hydraulic computation or preferably by the construction of a larger scale model of this area.

F2.3 Deck Level at Hobler's Bridge

This bridge is outside the model area. Data regarding flood heights are meagre and conflicting. In May 1958, with a discharge in the North Esk of 10,000 cusecs, a flood level of 22.8 was recorded on the upstream side of the Hobler's Bridge Road on the Launceston bank, whereas 20.9 was recorded in 1929 for 19,000 cusecs. The excessive willow and other vegetation growth since 1929 in this river might be responsible for the difference in levels.

The best that can be suggested is that the upper limit of the waterway area should be at least 23.0 ft. above S.L.W. (Launceston datum).

F2.4 Waterway Area for Hobler's Bridge

As the model does not encompass this section, the problem is one for normal hydraulic computation. Unfortunately, no systematic measurement of the discharge of the North Esk was made until 1957, when the Launceston Flood Protection Authority installed a pneumatic streamheight recorder at Corra Lynn. From a study of available data, the following schedule of highest annual floods in various years was made:-

1929 - 19,000 cusecs
1936 - 14,500 cusecs
1942 - 11,800 cusecs
1944 - 10,000 cusecs
1957 - 6,000 cusecs
1958 - 10,000 cusecs

The above discharges are at Corra Lynn, and should be multiplied by 1.106 to obtain the discharge at Hobler's Bridge. Some local inhabitants state that in the forties a flood bigger than 1929 occurred in the North Esk.

An intelligent guess might lead to a conclusion that a discharge of 15,000 cusecs, is likely to be equalled or exceeded about five times each 100 years. In this case also the width of the river channel is such that if the top of the waterway area is at a level not less than 23.0 ft., it is probable that the waterway area would be adequate for 15,000 cusecs. Detailed computations could be made, following the procedure put forward by W.S. Bureau of Public Roads of the U.S. Department of Commerce, Hydraulic Research Division (F1).

F3 FLOOD FORECASTING

F3.1 Flood Forecasting as a Flood Mitigation Measure

Figures 38 and 39 show the savings in flood damage achieved by increasing the period of precautionary action from 6 hours to 24 hours. Hence a reliable and quantitative flood warning system may be regarded as a flood mitigation measure in its own right. There already exists in the Esk Valley, as on most of the floodable river valleys of Australia, a system operated by the Meteorological Branch which says in effect "There will be a flood." Sometimes it is amplified to say that the flood will be a big one, or a minor one.

Ref. (F1) U.S. Bureau of Public Roads, U.S. Dept. of Commerce, Hydraulic Research Division "Computation of Backwater Caused by Bridges" (Preliminary Draft Oct. 1958).

Experience in recent floods in N.S.W. as discussed in Section E1.8 shows that such general warnings are not of great value in reducing flood damage. To be efficient in this regard, the warning must be in terms such as:- "In 36 hours' time the flood level in Home Reach will approximate 17.5 ft., and this will overtop the levees."

To be of any use in preventing damage, there are two other requirements:-

(a) The public must believe in the accuracy of the warning.

(b) The public must take vigorous and effective precautionary action.

In regard to (a), if the quantitative warnings given in the early stages of the development of such a system prove to be much in error, the public will very quickly become cynical, and will only slowly regain faith even if subsequent predictions are very accurate.

In regard to (b), if destructive floods are rare events, the public will be lethargic and inefficient in taking action.

In the case of Launceston, floods cause little damage until the South Esk discharge approaches 80,000 to 100,000 cusecs, but above that point heavy damages occur suddenly over a wide area as soon as the levees are overtopped. Such failure of the levee system has only occurred six times in the period 1828-1958, so that the people of Launceston will never be as "flood damage prevention conscious" as the people of towns such as Grafton in N.S.W., where damage or serious threat of damage is almost a yearly occurrence.

Nevertheless, if sufficient finance is not available to protect Launceston by a levee system as recommended in Section G3 of this report, expenditure on a quantitative flood forecasting system would seem to be amply justified.

F3.2 Methods of Flood Forecasting

F3.21 Index Station Method. A simple example of this method is the forecast rule for Paris, which states that the rise in the Seine at Paris will be double the mean of the rise in certain specified tributaries, and the time of travel is specified from other empirical rules.

Another example is that of the Red River of Indo-China:-

$M = 0.5\ m_1 + 0.04\ m_2 + 0.45\ m_3$ where M is the forecast rise at Hanoi over 24 hours and $m_1\ m_2$ and m_3 rises at key points in the river system.

An elementary form of this stage forecasting for the North Esk is to attempt to relate flood stage and time of peak at Avoca to discharge and time of peak down the gorge at Launceston.

In view of the complexity of the tributary system, it is obvious that any such empirical rules for Launceston would require to use at least half a dozen index stations, and reliance on Avoca alone could be misleading.

To develop a procedure it would be necessary to make a painstaking analysis of flood heights at key points over the catchment for past floods.

The flood of 13th October 1958 gave some indication of the complexity of this task, and showed that heavy rainfall in the Avoca district is by no means a sure indication of the magnitude of flood at Launceston.

Although the index station method may give satisfactory results on some catchments, it has several weaknesses:-

(i) The interval between time of giving the warning and time of arrival of flood may be too short. A preliminary forecast is desirable before the bulk of the runoff has reached the river channel, based on rate of rainfall data, particularly from the mountainous uninhabited areas.

(ii) The relation between flood stage at some upstream point and flood stage at the point of forecast (Launceston) will vary according to whether the flood is a quick one or a long sustained one (due to the rain lasting longer). The real guide is the shape of the flood hydrograph and the volume of water represented by it.

(iii) When there are tributaries entering the main river between the upstream index station and the point of forecast, variations in relations between flood stages at the two points will occur, due to differing inflows from these tributaries in the various storms. A knowledge of total rainfall and rate of rainfall over each of the tributary areas is required.

F3.22 Unit Crest Method. In this method the rise is assumed to be proportional to the amount of rainfall. It is an empirical method which is particularly suitable for small catchments where sufficient period of warning cannot be obtained if the forecast is made after the river has started to rise. Various refinements are possible, but it is hardly applicable to such a large catchment as the South Esk.

F3.23 Forecasting Based on Flood Hydrographs Instead of Flood Stages. The best method of forecasting is to base the predictions on a knowledge of hydrograph shape at upstream points, rather than on flood height.

For a catchment such as Launceston the steps necessary to provide a really complete and rational forecasting system are:-

(a) Install a network of pluviographs and a denser network of standard rain gauges.

(b) At the points where all major tributaries enter the main river install automatic stream height recorders on the main stream just below the confluence of the tributary.

(c) For the more important tributaries, install stream height recorders on the tributary just above the confluence.

(d) Over a period of several years, measure the flood discharges at the stream height recorder stations to establish a relation between height of flood and corresponding discharge in cusecs.

(e) For zones of the catchment whose storm runoff is not measured by stream gauging stations in (b) and (c) above, compute synthetic unitgraphs.

(f) Analyse the data provided by these instruments from storms which occur over a period of a few years, in order to:-

  (i) determine which rainfall and streamflow stations are vital for accurate flood forecasting

  (ii) derive unit hydrographs to express run-off behaviour of the main tributary streams

(iii) prepare graphs showing the rainfall-runoff relation for various antecedent moisture conditions, season of year, duration of storm and total storm rainfall.

(g) Install land line or radio telemetering equipment at the key stations to ensure rapid transmission of data to a central control point during floods.

(h) Provide staff at the central control point to carry out the following process during floods:-

(i) After the rain has been falling for some hours, and before much of the runoff has entered the streams, make an estimate of how much of the rain will appear as flood runoff. This is done from graphs prepared in step f(iii).

(ii) Apply this excess rainfall to the unit hydrographs of steps (e) and f(ii) for the various zones to estimate the flood hydrographs which will occur on the tributaries at their confluences with the main river.

(iii) By flood routing procedures, possibly using an analogue flood routing machine, compute the manner in which the hydrograph flattens out as the flood passes down the main river, thus predicting the future shape of the hydrograph some days later in the gorge at Launceston.

(iv) From a knowledge of stage discharge relations in Home Reach, predict the flood height likely to be reached in Home Reach and the lower North Esk in the vicinity of the city.

(v) Repeat the process every three or six hours as more rain falls, checking the calculations against the measured hydrographs in the streams in the upper reaches. (As time goes on the prediction of flood height in Home Reach becomes increasingly closer to the final truth).

Step (a) in the above process has been completed since 1957 by the Launceston Flood Protection Authority. For step (b) gauging stations exist at the outlets of the North Esk and South Esk rivers and on some of the upper reaches, but a few additional stations are necessary.

A more detailed discussion of these techniques is given by Laurenson (F2).

F3.3 Method Suitable as a Flood Mitigation Measure for Launceston

If satisfactory results can be obtained by the empirical index station or unit crest method, such a method should be used, as it is much cheaper in first cost and operation than that described in Section F3.23. The first step in developing a forecast procedure would be to employ a research engineer to study the following data to see if a satisfactory empirical procedure can be developed:-

(1) Historic flood and rainfall data prior to 1900.

(2) Measured flood and rainfall data for period 1900 to 1957.

(3) Detailed data, including tidal data, collected by the Authority from its network of instruments installed in 1957.

The complexity of the tributary system will tend to make the planning of an empirical procedure a difficult task. On the other hand, the time of travel of these floods to Launceston is relatively long, and the prediction (so far as Launceston is concerned) need only be accurate from 15.5 ft. above S.L.W. upwards. If this attempt to develop an empirical method fails, then the procedure of Section F3.23 will be necessary.

If flood forecasting is to be attempted, four or five pneumatic type automatic stream height recorders should be installed at the outset and the stations "rated" by use of the simple type of cableway installed at Corra Lynn on the North Esk. These instruments cost less than £100 each, and the streamflow data will be available in readiness if the method described in section 3.23 is found to be necessary, and will be a useful adjunct to any index station method which may be devised.

Ref. (F2) - Laurenson - "Flood Forecasting - A Scientific Basis for Flood Warning". Bulletin No.2 of Water Research Foundation of Australia (1958)

F3.4 Desirability of a Flood Forecasting System for the Esk Valley as a Whole.

Winter flooding of rural areas of the Esk valleys is a common occurrence. The Launceston Flood Protection Authority was only charged with the investigation of flood mitigation measures for Launceston, so that no rural flood damage studies were carried out.

The benefits of a well instrumented and partially telemetered system of flood forecasting are:-

(i) Provision of a quantitative flood forecast for Home Reach thus mitigating future urban flood damage.

(ii) More reliable forecasts for the rich rural areas of the valley, thus reducing rural damage.

(iii) Improved operation of the railway traffic system for the common occurrence of minor floods in the North Esk, and the rare occasions of major flooding in the South Esk.

(iv) Improved precautionary measures by public utilities such as telephone, road and bridge etc, authorities.

(v) Possibly improved operation of hydro-electric and other water use projects.

The degree of refinement justified in the installation and operation of a flood forecasting system depends upon the relation between the annual operating cost and the average annual benefits.

If the levee system recommended in Part G of this report is not built, there seems little doubt that an extremely elaborate flood forecasting system is justified for Launceston alone. The amount of these benefits for various reliable forecast intervals can be approximated from a study of the stage-damage graphs of Figs. 38 and 39 of this report.

If the levee system as recommended is built, a flood forecasting system, for Launceston only, becomes of minor importance. The only areas then subject to damage are the wharf areas. Reference to Fig. 39 shows that for 6 hours of emergency measures the estimated damage in this zone for a repetition of the 1929 flood is £200,000, while for 24 hours it is £170,000, assuming that the proposed new levee system is not built. However, if the proposed scheme is constructed, the levees will be close to the wharves and it would only be necessary to remove the goods a few hundred yards. As the rest of the city would not be in danger of flood, ample transport and labour would be available. It is therefore possible that, with the levee system recommended in Part G, practically all the goods would be saved even with the amount of warning at present available. Warning is necessary to prepare to close flood gates in levees.

However, the question of a flood forecasting system for the Esk Valleys should be considered as a whole, and not merely in relation to Launceston. The Commonwealth Government has recently set up a Hydrometeorological Section of the Meteorological Branch to investigate such matters. It seems clear that a study should be made by that Section to:-

(i) Assess the increased benefits accruing from a better instrumented forecasting system

(ii) decide whether an index station method would be satisfactory or whether steps (b) to (f) of Section F3.23 should be completed

(iii) make an estimate of the annual costs of steps (g) and (h)

(iv) decide on the degree of refinement in the flood forecasting system which will be justified by the benefits arising from it.

The instrumentation and data collection already carried out by the Authority would give the Hydrometeorological Section a "flying start" in such studies. Further, quantitative flood forecasting is a new development in Australia, so that an investigation into the best method for the Esk valleys would provide valuable basic knowledge applicable to other valleys of Australia, both for flood forecasting and general hydrologic engineering design.

PART G

RECOMMENDATIONS

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PART G

RECOMMENDATIONS

G1 DEGREE OF PROTECTION

G1.1 Built Up Zones

The Paterson St., Inveresk-Invermay, Royal Park-Willis St., Willis St.,-Cypress St., and Mowbray Flats zones should be protected against an estimated "probable maximum flood"of 250,000 cusecs in the South Esk River, co-incident with a discharge of 18,800 cusecs in the North Esk River.

G1.2 Wharf Areas

These areas cannot be protected by levees. Diversion of the South Esk River via Cormiston Creek would afford complete protection, but is far too costly. The construction of the Hunter Cut would reduce flood levels in this zone to a minor extent but the benefits thus obtained do not warrant the expenditure involved.

Attention is drawn to the fact that if the levees recommended in Section G3.11 and G3.13 are built, a reasonable period of reliable flood warning would permit the rapid removal of goods from the wharves to protection behind the adjacent levees.

Consideration should be given to developing a quantitative flood forecasting system for the Esk valleys and Home Reach, as discussed in Section F3.4 of this report, in order to reduce to a minimum the future flood damage in this zone.

G1.3 Undeveloped Zones

G1.31 West Tamar Bank. No action should be taken by the Government of Tasmania at this stage to provide any flood protection, but the Launceston City Council should be informed as follows:-

(i) Reclamation of this area by public or private interests will have no adverse effects on the flooding of Launceston, even for the "maximum probable flood".

(ii) If any such development is carried out, the provision of levees to the height of 18.9 ft. above standard low water would be necessary for protection against a discharge equal to the 1929 flood, while for protection against the estimated "maximum probable flood" the height should be 23.4 ft. above standard low water.

G1.32 Upper North Esk. In regard to the undeveloped grazing lands of the Upper North Esk zone between the Railway Bridge and Hobler's Bridge over the North Esk River, no action should be taken at present by the Government of Tasmania to provide flood protection, but the results of the model tests should be preserved for the guidance of the Government's advisers at some time in the future, when development in this area will warrant flood protection.

In this connection, attention should be directed to the fact that if levees are built alongside the banks of the Upper North Esk to protect this zone against the "probable maximum flood", then the flood levels from Tamar St. Bridge to Hobler's Bridge will be raised by approximately 1.0 ft., but the lower North Esk and Home Reach flood levels will be unchanged.

This means that when in the future these Upper North Esk levees are constructed, all levees proposed in this report upstream of Tamar St. Bridge must be raised by 1.0 ft. Fortunately, practically all these levees are of earth construction, so that this future increase in height is a relatively simple matter.

G2 URGENCY OF PROTECTION

Under conditions of abnormally high tide caused by wind or barometric effects, a discharge of 70,000 cusecs in the South Esk River concurrent with 8,400 cusecs in the North Esk River, may overtop sections of the existing levees protecting Inveresk and Invermay respectively. A discharge of 90,000 cusecs in the S.Esk, with a concurrent flow of 9,600 cusecs in the N.Esk, could cause overtopping of the levees to such an extent that minor emergency measures such as sandbagging the levees would be barely adequate to prevent damage to property.

In any given year, it is 80 per cent certain that the chance of a discharge of 90,000 cusecs or greater occurring in the South Esk River lies between 1 in 28 and 1 in 12, with 1 in 16 as the correct value if the past 131 years is a true average sample of the long term behaviour of the river.

It is also 80 per cent certain that the chance, in any given year, of a repetition of the 1929 flood, or a bigger flood, lies between 1 in 25 and 1 in 130, with 1 in 50 as the correct value if the past 131 years is a true average sample of the long term behaviour of the river.

It is estimated that a repetition of the 1929 flood would cause damage amounting to £1,932,000 in the 4 zones discussed in Sections G3.13 to G3.16 below if it occurred in 1960 and £3,864,000 if it arrived in 1994, which is the mid point of the period of analysis adopted for this investigation, assuming a 24-hour period of emergency measures.

If any flood mitigation works are to be constructed, it is preferable that they should be built before, and not after, the next flood exceeding 100,000 cusecs in the South Esk River.

Therefore, in view of the high benefit-cost ratios applicable to all the built up zones and the potential damage from any flood of high magnitude, the construction of flood protection works for these zones should be put in hand as soon as practicable.

G3 METHOD OF PROTECTION AND PRIORITY OF CONSTRUCTION OF WORKS

G3.1 Recommended Works

The following flood protection works should be carried out in the order listed:-

G3.11 Training or Diversion Levee. The first step in flood mitigation should be the construction of a training levee in Royal Park or some other structure which will effectively divert the jet of the South Esk River for the estimated probable maximum discharge of 250,000 cusecs.

For a straight levee 1,600 ft. long, consisting of a concrete core wall as in Fig. 40 covered by a grassed earth embankment as in Fig. 41 and of height shown in Fig. 36 for 250,000 cusecs, the estimated capital cost is £84,700 and the ratio of annual benefit to annual cost of this phase is 9.48.

Further model experiments may lead to an amended design and estimate of cost.

Therefore the following additional model studies should be carried out before the design of this phase of the work is completed:-

(i) To determine the minimum effective height of a straight training levee in Royal Park to divert a South Esk discharge of 250,000 cusecs.

(ii) To determine whether a training levee curved in plan would be more efficient.

(iii) To test the effectiveness of groynes in Home Reach near Royal Park in lieu of a training levee.

(iv) To investigate whether a curved breakwater with reclaimed parklands behind it could replace the training levee and dovetail in with the construction works necessary for the proposed new bridge across the Tamar River from Royal Park to the West Tamar Rd., thus leading to saving in cost of the bridge and the provision of additional parklands.

<u>G3.12 Paterson St. Levee.</u> The second stage should be the raising of the existing Paterson St. embankment to a height of 31.5 ft. above standard low water to give protection against the probable maximum flood. At its present level this bank would be overtopped by a S.Esk discharge of 125,000 cusecs.

The cost of a concrete promenade and parapet to achieve this result is estimated at £17,000.

<u>G3.13 Raising of Existing Levees Protecting Inveresk and Invermay.</u> The third stop, which should follow closely behind item G3.11 above, is the raising of the existing Inveresk-Invermay levees to the heights shown in Table No. 27 and located as indicated in Fig. 37, using cross sections similar to Figs. 41 and 42 for earth levees and to Fig. 40 for concrete. In addition, the footpath of Mayne St. should be raised by amounts up to 1.0 ft. to 22.0 ft. above S.L.W. for a length of 800 ft.

The estimated capital cost of these works is £242,000. The ratio of annual benefits to annual costs is estimated at 13.46.

Special note should be made of the fact that under no circumstances should these levees be constructed before the completion of the training levee, as such action would result in a considerable increase in general flood levels in the Royal Park-Willis St. and Willis St.-Cypress St. zones.

G3.14 Construction of Levee from Royal Park to Willis St. The fourth phase should consist of the building of earth and concrete levees to the heights shown in Table No. 27, located as shown in Fig. 37, using cross sections similar to Fig. 41 for earth levees and to Fig. 40 for concrete. The concrete levee along the Esplanade should consist of a concrete promenade and solid parapet with suitable ornamental park type electric light standards. If the proposed new arterial road follows the Esplanade route this conception can be readily incorporated in the design of the road.

The estimated capital cost of this section is £127,000 and the benefit cost ratio 2.52.

It should be noted that it would be unwise to build this levee without first completing the training levee, as in such circumstances the height of this levee would have to be considerably increased.

G3.15 Levee from Willis St. along Railway Line to Cypress St. The next stage is a continuation of the levee system of G3.14 above alongside the railway line to the hill near Cypress St., as shown in Fig. 37, using a cross section similar to that in Fig. 42 and heights as in Table 27.

Necessary auxilliary works to cater for local drainage consist of clearing undergrowth from the existing network of open drains, construction of a sump, and installation of a float operated electric pump to pump local drainage over the railway line, and permanent sealing up of the existing flood gate.

The estimated capital cost of this stage is £93,000 and the benefit-cost ratio 2.58.

G3.16 Levee Protection for Mowbray Flats. The final works to complete the proposed scheme involve a continuation alongside the Scottsdale Railway line to Mowbray Hill of the levee of G3.13 above, construction of a levee from Rosslyn Rd. to McKenzie St. and the provision of lateral and main drains and a sump with float operated electric pump to drain from Mowbray Swamp all local surface runoff and groundwater.

The location of these levees is shown in Fig. 37, and the heights in Table No. 27.

The estimated capital cost is £81,000 and the benefit-cost ratio 1.78.

Attention is drawn to the important secondary benefit of this stage, due to the fact that the drying out of Mowbray Swamp will permit development of this area. As much of the land in this zone is owned by the Launceston City Council, a good proportion of the betterment benefits will accrue to the general public.

G4 ASSOCIATED BENEFITS

These proposals will lead to a saving in the cost of the embankment of the proposed new arterial road.

The construction of the training levee recommended in G2.11 above will permit the embankment from A to C shown in Fig. 37 to be at a level of 19.3 ft. instead of 20.7 ft. above S.L.W. if it is desired that the road surface should be above the flood level caused by a flood dischage equal to that of 1929.

The construction of the levees described in G3.14 and G3.15 will make possible further savings in cost due to a lowering of the height of the road surface.

No allowance has been made for these benefits in calculating the benefit cost ratios quoted in sections G3.11, G3.14 and G3.15.

However, this aspect should be borne in mind when consideration is given to the desirability of flood protection works.

G5 MARGIN OF SAFETY

The levee heights given in Table 27 provide for varying amounts of freeboard.

It should be noted that the estimate of 250,000 cusecs for the "probable maximum flood" is based on one storm only (that of April 1929) and if data for the 1852, 1863 and 1893 storms had been available, the estimate would possibly be higher.

It is extremely unlikely that this flood will occur during the economic life of the proposed works. However, should it do so, and the estimate of its magnitude prove too low, the heights of the levees given in Table 27 for the McKenzie and Mayne Sts. saddles are such that the flood will relieve itself by passing over these levees.

The possibility of a flood of 250,000 cusecs occurring in the South Esk during the economic life of the proposed works is exceedingly remote, and the possibility of a greater flood even more so. However, if desired, such an unlikely contingency could be provided for by having available emergency material for temporary raising of the levees during such a flood. In this connection it might be noted that an increase in the design flood discharge from the estimated probable maximum of 250,000 cusecs to 275,000 cusecs would only increase the general flood level by 3½ inches.

G6 ALTERNATIVE METHODS OF FLOOD MITIGATION

G6.1 Diversion of North Esk River

The proposals for flood mitigation by diversion of the North Esk River and construction of levees as shown in Fig.2, submitted by Messrs. Potts and Dare to the Launceston City Council on 12th March 1945, are not recommended, for the following reasons:-

(i) The estimated capital cost is higher than the total estimated cost of the works recommended in G3 above, having been estimated as £1,500,000 in 1955.

(ii) The building of structures across the North Esk River is a difficult operation.

(iii) The Launceston Marine Board objects to any interference with the natural channel of the North Esk River.

(iv) Such works would increase navigational difficulties of vessels rounding Stephenson's Bend, due to the lateral flow from the North Esk River across the Tamar River at a critical point in the passage around the bend.

G6.2 Completion of Hunter Navigation Cut

The estimated cost of completing this Cut is £650,000. Its flood mitigating effect is minor, amounting to a lowering of the general flood level for the 1929 discharge by 1.0 ft. The benefit is not commensurate with the cost, and this project is not recommended for flood mitigation purposes.

G6.3 Channel Improvements

Channel improvements, such as rounding off river bends and deepening and widening the whole channel of the Tamar from Royal Park to Boat Channel, will have only minor effects in reducing flood levels, and will be costly to carry out and maintain, and these methods are not recommended.

G6.4 Floodways

Provision of a wide shallow floodway below Riverside Golf Links and of a narrow deeper floodway along the line of the Hunter Cut has been investigated, but the flood mitigation effects are too small to warrant the expenditure involved.

G6.5 Diversion of South Esk River via Cormiston Creek

This diversion would afford complete protection for the whole of the Launceston flood plain, but the capital cost is prohibitive.

G7 AUXILLIARY WORK

G7.1 Future Data Collection

G7.11 On Esk Catchments. Collection of data from the network of pluviographs, streamgauging stations and daily read rain gauges on the Esk catchments should continue for the following purposes:-

(i) To develop accurate quantitative flood forecasting techniques.

(ii) To refine further the hydrologic calculations embodied in this report.

(iii) To facilitate the accurate and rational design of bridges, dams, pumping schemes and other engineering works which will be constructed on or near these two rivers in future years.

(iv) To advance basic knowledge of engineering hydrology.

G7.12 On South Esk River in Launceston Gorge. In order to refine the calculations in this report, to further basic hydraulic knowledge and to permit informed analysis of future floods, measurements should be made of water levels in the gorge at the Suspension Bridge gauge and at the temporary gauges 800 ft. upstream and 500 ft. downstream of the Suspension Bridge at least once per day in future floods until a series of readings at 10,000 cusec intervals have been established for discharges from 60,000 cusecs upwards.

G7.13 Tidal Influence. In order to assist in future flood forecasting and analysis, the following data should be collected and recorded in future:-

(i) Automatic tide records in Boat Channel.

(ii) Automatic tide records in Lower N.Esk River.

(iii) Wind and barometric readings at Low Head.

(iv) Velocities and directions of flow across the section of the Tamar at the Powder Jetty for all South Esk discharges exceeding 60,000 cusecs.

G7.2 Raising of Evandale Saddle

In order to put beyond any possible doubt whatever the likelihood of a premature capture of the North Esk River by the South Esk, the saddle at Evandale should be raised to a minimum height of 25 ft. above the level of the 1929 flood at Evandale.

G7.3 Flood Forecasting

The Commonwealth Government should be requested to arrange for the Hydrometeorological Section of the Meteorologic Bureau to carry out research into the best method of providing a quantitative flood forecasting service for the North and South Esk Rivers.

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Principal Executive Officer

Date: 15th August 1959.

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THE UNIVERSITY OF NEW SOUTH WALES

WATER RESEARCH LABORATORY

REPORT No. ~~8~~ 9

Volume II - Figures and Tables

Flood Mitigation Measures for the City of Launceston

by

C. H. Munro

SEPTEMBER, 1959

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TABLE No. 1

SURVEY PLANS
FOR MODEL CONSTRUCTION

DETAILS OF PLAN	TYPE OF SURVEY	AUTHORITY	DATE OF SURVEY
1. Bench Works and Contours for Launceston	Topographic	L.C.C	1955
2. Launceston and Environs	Planimetric	L.C.C	1952
3. Detailed Survey of Portion Relating to Flood Protection Scheme	Topographic	L.C.C	1937
4. Embankment Levels	Topographic	L.C.C	1934
5. Trevallyn Tailrace and Disposal Areas	Topographic	H.E.C	1953
6. Trevallyn Power Station Site Contours	Topographic	H.E.C	1953
7. Contour plan area adjoining the Hunter Cut	Topographic	L.F.P.A	1936
8. Contour plan area adjoining the Hunter Cut	Topographic	L.F.P.A	1957
9. Survey Royal Park-Charles St. Area	Topographic	L.M.B	
10. Sounding Plan for Upper Harbour	Hydrographic	L.M.B	1939
11. Tamar Soundings No. 6 to No. 8 Beacon	Hydrographic	L.M.B	1955
12. Tamar Soundings Home Reach to No. 6 Beacon	Hydrographic	L.M.B	1957-58
13. Cross Sections and Sounding River Tamar	Hydrographic	L.M.B	1935-40
14. Cross Sections of N.Esk River	Hydrographic	L.C.C	1937
15. Cross Sections of Tamar River	Hydrographic	L.C.C	1937
16. Soundings in Tailrace at completion of dredging	Hydrographic	H.E.C	1955
17. Cross Sections of N.Esk River	Hydrographic	L.F.P.A	1957
18. Cross Sections of S.Esk River	Hydrographic	L.F.P.A	1957
19. Cross Sections of Tamar River - Stephenson's Bend to Boat Channel	Hydrographic	L.F.P.A	1958
20 Base Plan for Model	Topographic	L.F.P.A	1958

TABLE No. 2.

DERIVED LOSS RATES FOR SOUTH ESK CATCHMENT

Date of Storm	Storm Runoff			Storm Features				Average Loss Rate
	Discharge at Start of Rise Cusecs	Peak Discharge Cusecs	Total Runoff Inches	Average Gross Rainfall Inches	Definition of Temporal Pattern	Areal Distribution Coefficient $\frac{\text{Max.Rainfall}}{\text{Mean Rainfall}}$	Storm Duration Hours	pts/hour
16th. Sept. 1952	3,908	35,200	1·00	2·39	2 Pluviographs	1·75	111	2·56
2nd. May, 1956	2,190	30,400	0·79	2·42	2 Pluviographs	2·28	18	9·50
22nd. May, 1956	5,486	48,800	1·17	1·96	2 Pluviographs	3·80	43	2·69
22nd. May 1958	1,630	21,000	0·82	2·34	9 Pluviographs	3·56	66	4·46
15th. Aug. 1958	8,500	42,800	0·90	1·62	9 Pluviographs	4·17	20	2·42

TABLE No. 3.

DERIVED LOSS RATES FOR NORTH ESK CATCHMENT

Date of Storm	Storm Runoff			Storm Features				Average Loss Rate
	Discharge at Start of Rise Cusecs	Peak Discharge Cusecs	Total Runoff Inches	Average Gross Rainfall Inches	Definition of Temporal Pattern	Areal Distribution Coefficient $\frac{\text{Max.Rainfall}}{\text{MeanRainfall}}$	Storm Duration Hours	pts/hour
2nd. May 1958	150	4,100	0·55	3·00	6 Pluviographs	2·28	22	15·5
22nd. May 1958								
(i) Total Storm	950	10,200	1·81	4·58	6 Pluviographs	1·75	68	8·5
(ii) Ist Rainfall Burst.			0·14	1·42			7	26·0
(iii) 2nd. Rainfall Burst.			1·18	2·02			17	6·2
(iv) 3rd. Rainfall Burst.			0·49	1·14			13	6·1

TABLE No. 4.

DERIVED UNITGRAPHS FOR SOUTH ESK CATCHMENT AT DUCK REACH

Date of Storm	Storm Runoff		Storm Features				6 Hour Unitgraph	
	Peak Discharge Cusecs	Total Runoff Inches	Wind Direction	Areal Distribution Coefficient $\frac{\text{Max. Rainfall}}{\text{Mean Rainfall}}$	Definition of Temporal Pattern	Storm Duration Hours	Peak Discharge Cusecs	Time to Peak Hours
16th. Sept., 1952	35,200	1·00	N. to N.W.	1·75	2 Pluviographs	111	34,400	61
2nd. May 1956	30,400	0·79	N.E. to S.E.	2·28	2 Pluviographs	18	34,000	39
22nd. May 1956	48,800	1·17	N.E. to S.E.	3·80	2 Pluviographs	43	37,100	54
22nd. May 1958	21,000	0·82	N.W. to W.	3·56	9 Pluviographs	66	28,600	42
15th. Aug. 1958	42,800	0·90	N.E. to S.E.	4·17	9 Pluviographs	20	39,800	46

TABLE No. 5.

DERIVED UNITGRAPHS FOR NORTH ESK CATCHMENT AT CORRA LYNN

Date of Storm	Storm Runoff		Storm Features				6 Hour Unitgraph	
	Peak Discharge Cusecs	Total Runoff Inches	Wind Direction	Areal Distribution Coefficient $\frac{\text{Max. Rainfall}}{\text{Mean Rainfall}}$	Definition of Temporal Pattern	Storm Duration Hours	Peak Discharge Cusecs	Time to Peak Hours
2nd. May, 1958	4,100	0·55	N.W. to W.	2·28	6 Pluviographs	22	8,500	20
22nd. May, 1958	10,200	1·82	N.W. to W.	1·75	6 Pluviographs	68	8,500	27

TABLE No. 6.

CALCULATION OF MANNING'S "n" AT DUCK REACH

Date	Time	Gauge Rdg. Suspension Bridge	Discharge Q	Cross Sectional Area A	Hydraulic Mean Radius R	Water Surface Slope S	$n = \frac{1{\cdot}49 R^{\frac{2}{3}} S^{\frac{1}{2}} A}{Q}$
25-5-58	1625	15·9	13,800	1980	11·3	0·0204	0·154
26-5-58	0750	17·5	17,800	2230	12·4	0·0208	0·144
26-5-58	1655	17·6	18,000	2250	12·5	0·0210	0·145
27-5-58	1542	16·6	15,500	2080	11·7	0·0199	0·145
28-5-58	0914	17·0	16,500	2150	12·0	0·0206	0·146
30-5-58	1445	12·8	7,800	1510	9·4	0·0192	0·178
18-8-58	1526	24·8	40,000	3440	16·4	0·0195	0·116
19-8-58	1243	23·5	36,000	3220	15·9	0·0197	0·119
19-8-58	1652	22·6	33,000	3070	15·7	0·0209	0·124
20-8-58	1050	19·0	22,000	2470	13·4	0·0210	0·136
20-8-58	1625	18·5	20,000	2390	13·1	0·0210	0·143
14-10-58	0700	25·5	44,000	3570	16·8	0·0195	0·111
14-10-58	1830	23·3	35,000	3190	15·9	0·0204	0·123
16-10-58	0730	14·8	11,500	1800	10·6	0·0204	0·160
17-10-58	1200	15·25	12,500	1880	10·9	0·0206	0·158

TABLE No. 7.

MODEL RESULTS.

South Esk Discharge = 100,000 cusecs.

North Esk Discharge = 10,200 cusecs.

TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLANS.	FLOOD LEVELS AT GAUGE No.																																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	Base Tests - 1958 Topography no flood protection	17·0	16·9	16·9	16·8	16·8	16·8	16·7	16·6	16·5	–	16·6	16·1	16·2	16·1	15·7	15·6	15·5	15·2	14·7	14·3	14·6	14·3	–	–	–	–	16·3	16·0	15·9	16·5	15·6	14·2	16·5	16·4	–	15·3	16·5	–	–	–
2	Tidal influence - levels in boat channel raised by one foot	17·8	17·8	17·8	17·8	17·8	17·7	17·6	17·5	17·5	–	17·5	16·9	17·0	16·9	16·5	16·3	16·2	16·0	15·5	15·1	15·5	15·2	18·2	17·7	17·4	–	17·1	16·7	16·5	17·3	16·2	15·1	17·4	17·3	–	16·0	17·4	–	–	–
	LEVEE PROTECTION ONLY																																								
3	Training levee only	17·0	17·0	16·9	16·9	16·9	16·8	16·7	16·5	16·5	–	16·7	16·1	15·9	15·7	15·6	15·2	15·1	15·0	14·5	14·3	14·6	14·0	–	–	–	–	16·3	15·9	15·7	–	15·3	–	–	16·0	–	14·4	–	–	–	–
4	Inveresk and Invermay levees only	17·4	17·3	17·3	17·3	17·3	17·2	16·8	16·9	16·9	–	16·9	16·3	16·3	16·5	15·9	15·7	15·3	15·2	14·8	14·6	14·6	14·4	17·8	17·4	17·1	–	16·4	16·4	16·4	–	16·0	14·7	–	16·7	–	–	–	–	–	–
5	Training levee with Inveresk and Invermay levees.	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
6	Training levee with Inveresk and Invermay and also City levees	17·3	17·3	17·3	17·3	17·2	17·2	17·1	16·8	16·7	–	16·8	16·5	16·4	16·1	15·7	15·7	15·2	15·1	14·7	14·4	14·6	14·3	17·9	–	–	–	16·6	16·4	16·4	–	15·9	14·8	–	17·2	–	15·4	–	–	–	–
7	Repeat of Test 6 but with the alignment of the levees slightly altered to conform with the prototype survey of 1959	17·2	17·2	17·1	17·1	17·2	17·1	17·1	16·8	16·8	–	16·9	16·5	16·4	16·2	15·9	15·9	15·4	15·4	14·7	14·4	14·7	14·4	–	–	–	–	16·4	16·4	16·6	–	15·7	14·8	–	–	–	–	–	–	16·6	17·0
	TRAINING LEVEE WITH INVERESK AND INVERMAY AND ALSO CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS																																								
8	Floodway at Stephensons Bend by removal of levees around silt deposit areas	17·4	17·3	17·3	17·2	17·2	17·2	17·2	16·9	16·8	–	16·8	16·5	16·5	16·1	15·7	15·7	15·6	15·4	14·7	14·4	14·6	14·3	–	–	–	–	16·6	16·6	16·4	–	16·1	14·9	–	17·0	–	15·4	17·4	–	–	–
9	Floodway at Stephenson's Bend by removal of silt deposit levees and removal of vegetation along river banks such as Ti Tree etc.	17·7	17·6	–	17·7	17·5	17·6	17·7	16·8	16·7	–	16·7	16·3	16·3	16·2	15·7	15·7	15·3	15·6	14·8	14·4	14·7	14·2	18·2	–	–	–	16·7	16·5	16·4	–	16·2	15·3	–	16·8	–	15·5	–	–	–	–
10	Flow floodway as for Test 9 but with overbank area roughened by the addition of ¼" gravel to allow for deterioration in efficiency with time by the growth of vegetation	17·3	17·2	17·2	17·1	17·1	17·1	17·1	16·9	16·8	–	16·9	16·7	16·5	16·3	15·9	15·8	15·8	15·6	14·9	14·5	14·6	14·4	17·8	–	–	–	16·5	16·5	16·4	–	16·3	14·7	–	17·2	–	15·8	17·3	–	–	–
11	Floodway at Stephensons Bend formed by construction of a channel 5 feet deep (bed level RL7) and 550 feet wide along line of Hunter Cut	16·8	16·7	16·6	16·5	16·6	16·5	16·6	16·3	16·2	–	16·4	15·9	15·9	15·7	15·4	15·3	15·2	15·3	14·8	14·4	14·6	14·4	–	–	–	–	16·1	15·9	15·7	–	15·4	–	–	16·3	–	–	–	–	–	–
12	Bend opp. tailrace realigned by dredging to reduce curvature and widen the river from 285 feet to 510 feet at S.L.W	17·1	17·0	16·9	16·8	16·9	16·8	16·8	16·5	16·5	–	16·7	16·2	16·1	15·9	15·8	15·9	15·4	15·4	14·7	14·4	14·6	14·4	–	–	–	–	16·2	16·2	16·1	–	15·8	–	–	16·7	–	–	–	–	–	–
13	Floodway 5 feet deep and 550 feet wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp. tailrace as for Test 12	16·9	16·7	16·7	16·6	16·7	16·5	16·6	17·1	16·4	–	16·5	15·9	16·0	15·7	15·5	15·5	15·4	15·4	14·9	14·4	14·6	14·4	–	–	–	–	16·1	16·0	16·0	–	15·6	15·0	–	16·6	–	–	–	–	–	–
14	Hunter Navigation Cut 28 feet deep (Bed RL 16) bottom width 231 feet as shown in figure.	16·3	16·3	16·3	16·2	16·2	16·1	16·1	–	15·7	–	16·0	15·3	15·3	15·0	–	–	–	–	14·4	14·2	14·4	14·2	–	–	–	–	15·5	15·4	15·3	–	14·4	–	–	15·8	–	–	–	–	–	–
	SUBSIDIARY TESTS																																								
15	Inveresk and Invermay, City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	17·0	17·0	17·0	16·9	17·0	16·9	17·0	16·6	16·7	–	16·8	16·3	16·3	16·0	15·7	15·8	15·3	15·4	14·6	14·4	14·6	14·4	–	–	–	–	–	–	–	–	15·8	14·7	–	–	–	–	–	–	–	–
16	Inveresk and Invermay City Paterson Street and West Tamar bank levees together with levees 100 feet from river edge on each side of N. Esk below Cypress Street	17·6	17·6	17·3	17·2	17·1	16·9	16·8	16·6	16·5	–	16·8	16·2	16·2	16·0	15·7	15·7	15·3	15·2	14·6	14·3	14·5	14·3	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

TABLE No. 8.

MODEL RESULTS.

South Esk Discharge 125,000 cusecs.

North Esk Discharge 11,700 cusecs.

TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLANS.	FLOOD LEVELS AT GAUGE No.																																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	Base Tests - 1958 Topography no flood protection	18·6	18·6	18·6	18·6	18·5	18·5	18·4	18·4	18·4	–	17·9	17·3	17·2	17·1	16·6	16·3	16·2	16·1	15·3	14·8	15·1	14·5	18·9	18·4	18·2	–	17·4	17·1	16·8	18·4	16·4	14·6	18·4	18·3	–	16·0	18·3	–	–	–
2	Tidal influence - levels in boat channel raised by one foot	19·1	19·1	19·1	19·1	19·1	19·1	19·0	19·0	18·9	–	18·5	17·9	17·8	17·6	17·1	16·9	16·8	16·7	16·1	15·7	16·0	15·6	19·1	19·0	18·8	–	17·9	17·7	17·4	19·0	16·8	15·8	18·9	18·6	–	16·5	18·9	–	–	–
	LEVEE PROTECTION ONLY																																								
3	Training levee only	18·4	18·6	18·4	18·4	18·4	18·4	18·4	18·2	18·2	–	17·9	17·6	17·7	17·8	17·1	16·7	16·7	16·5	15·5	14·9	15·2	14·4	18·6	18·2	18·0	20·4	17·6	17·7	17·4	18·2	17·0	15·0	18·2	18·2	–	16·5	18·1	–	–	–
4	Inveresk and Invermay levees only	18·6	18·6	18·6	18·5	18·5	18·5	18·5	18·3	18·2	–	17·4	17·1	17·3	16·7	16·1	16·1	16·0	15·9	15·3	14·9	15·0	14·7	19·0	18·8	18·5	–	17·2	17·3	17·3		16·7	15·0	–	18·0	–	16·1	18·2	–	–	–
5	Training levee with Inveresk and Invermay levees	18·1	18·0	18·0	18·0	18·0	18·0	18·0	17·7	17·6	–	17·2	17·1	17·3	16·3	–	16·3	16·1	16·0	15·3	14·9	15·0	14·7	18·5	18·2	17·8	–	17·3	17·3	17·4	–	16·8	15·1	–	18·1	–	16·1	18·0	–	–	–
6	Training levee with Inveresk and Invermay and also city levees	19·0	19·0	19·0	19·0	19·0	19·0	19·0	18·1	18·0	–	18·1	17·7	17·7	17·1	16·5	16·3	15·7	15·7	15·3	–	15·0	14·5	19·5	–	–	–	17·7	17·7	17·3	–	16·9	15·0	–	18·5	–	16·1	18·3	–	–	–
7	Repeat of Test 6 but with the alignment of the levees slightly altered to conform with the prototype survey of 1959	18·6	18·5	18·5	18·4	18·4	18·4	18·4	18·1	18·1	–	18·5	17·7	17·6	17·2	16·8	16·8	16·2	16·2	16·1	14·7	15·0	14·7	–	–	–	–	17·9	17·6	17·8	–	16·6	15·0	–	–	–	–	–	–	18·3	18·2
	TRAINING LEVEE WITH INVERESK AND INVERMAY AND ALSO CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS																																								
8	Floodway at Stephenson's Bend by removal of levees around silt deposit areas	18·2	18·2	18·2	18·2	18·2	18·2	18·2	17·9	17·8	–	18·2	17·3	17·3	17·0	16·6	16·3	16·5	16·2	15·1	14·7	15·1	14·1	–	–	–	–	17·4	17·6	17·5	–	17·0	15·2	–	18·1	–	16·1	18·2	–	–	–
9	Floodway at Stephenson's Bend. by removal of silt deposits levees and removal of vegetation along river banks such as Ti trees etc.	18·3	18·3	18·3	18·3	18·3	18·3	18·3	18·0	17·9	–	18·3	17·5	17·5	17·1	16·6	16·5	16·1	16·4	15·2	14·7	15·0	14·4	–	–	–	–	17·7	17·6	17·6	–	17·0	15·2	–	18·6	–	16·5	18·3	–	–	–
10	Flow floodway as for Test 9 but with overbank area roughened by the addition of $\frac{3}{16}$" gravel to allow for deterioration in efficiency with time by the growth of vegetation	18·4	18·3	18·4	18·4	18·3	18·3	18·3	18·1	18·0	–	18·4	17·6	–	17·5	16·8	16·5	16·5	16·4	15·3	14·8	15·0	14·5	18·9	–	–	–	17·6	17·7	17·5	–	17·2	15·3	–	18·5	–	16·8	18·3	–	–	–
11	Floodway at Stephensons Bend formed by construction of a channel 5 feet deep (bed level RL 7) and 550 feet wide along line of Hunter Cut	18·1	18·1	18·1	18·0	18·0	17·9	18·0	17·6	17·6	–	17·9	17·2	17·1	16·7	16·1	16·1	15·9	16·0	15·4	14·7	14·9	14·6	–	–	–	–	17·4	17·1	16·8	–	16·4	–	–	17·9	–	–	18·1	–	–	–
12	Bend opp. tailrace realigned by dredging to reduce curvature and and widen the river from 285 feet to 510 feet at S.L.W.	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
13	Floodway 5 feet deep and 550 feet wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp. tailrace as for Test 12	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
14	Hunter Navigation Cut 28 feet deep (Bed RL 16) bottom width 231 feet as shown in figure	17·7	17·7	17·7	17·5	17·6	17·5	17·5	17·1	17·1	–	17·7	16·6	16·5	15·8	–	–	–	–	14·9	14·7	15·0	14·6	–	–	–	–	16·8	16·6	16·2	–	15·0	14·7	–	17·3	–	–	17·7	–	–	–
	SUBSIDIARY TESTS																																								
15	Inveresk and Invermay, City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
16	Inveresk and Invermay, City, Paterson Street and West Tamar bank levees together with levees 100 feet from river edge on each side of N. Esk below Cypress Street	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

TABLE No. 9.

MODEL RESULTS.

South Esk Discharge = 150,000 cusecs.

North Esk Discharge = 13,100 cusecs.

TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLANS	FLOOD LEVELS AT GAUGE No.																																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	Base Tests - 1958 topography no flood protection	20·8	20·8	20·8	20·8	20·8	20·8	20·6	20·8	20·6	–	19·6	19·1	19·3	18·1	17·3	17·3	17·1	16·9	15·7	15·1	15·5	14·9	20·6	20·4	20·6	21·5	19·2	19·1	17·7	20·6	16·8	15·4	20·5	20·2	20·0	16·8	20·5	20·1	–	–
2	Tidal influence - levels in boat channel raised by one foot	20·9	20·9	20·9	20·9	20·9	20·9	20·7	20·9	20·7	–	19·7	19·3	19·3	18·5	18·7	17·4	17·7	17·4	16·5	16·0	16·4	15·7	20·8	20·8	20·6	–	19·4	19·0	18·3	20·5	17·4	16·1	20·6	20·2	20·0	17·1	20·4	20·1	–	–
	LEVEE PROTECTION ONLY																																								
3	Training levee only	19·1	19·1	19·1	19·1	19·0	19·0	19·0	18·9	18·9	–	18·6	18·4	18·5	18·0	17·4	17·2	17·2	16·8	15·9	15·4	15·6	15·0	19·4	19·3	18·8	20·9	18·5	18·7	18·3	19·0	17·8	15·9	19·1	19·3	–	16·9	18·9	–	–	–
4	Inveresk and Invermay levees only	21·2	21·1	21·1	21·0	21·0	21·1	21·1	21·1	20·8	–	20·0	18·5	18·4	16·9	16·6	–	16·8	16·7	15·9	15·4	15·5	14·9	21·2	21·2	20·8	21·1	18·9	18·5	18·2	–	17·6	15·9	–	19·5	–	16·8	18·8	–	–	–
5	Training levee with Inveresk and Invermay levees	18·5	18·5	18·5	18·5	18·5	18·6	18·5	18·2	18·2	–	17·8	17·6	17·9	17·8	17·2	16·8	16·6	16·6	15·7	15·1	15·4	14·8	19·1	18·9	18·4	20·3	17·8	18·1	17·9	–	17·4	15·6	–	18·9	–	16·8	18·6	–	–	–
6	Training levee with Inveresk and Invermay and also city levees	20·0	19·8	19·8	19·9	19·7	19·8	19·8	19·5	19·3	–	19·9	18·9	18·8	18·2	17·4	17·5	16·9	16·7	16·0	15·2	15·4	14·8	20·0	–	–	–	19·0	18·8	18·5	–	17·6	16·0	–	20·1	–	17·0	19·0	–	–	–
7	Repeat of Test 6 but with the alignment of the levees slightly altered to conform with the prototype survey of 1959	19·7	19·7	19·7	19·6	19·7	19·7	19·8	19·3	19·3	–	202	18·9	18·9	18·4	17·7	17·8	17·1	17·1	15·9	15·2	15·5	15·0	19·9	–	–	–	18·8	18·9	18·7	–	17·6	15·7	–	–	–	15·4	19·0	–	20·1	19·5
	TRAINING LEVEE WITH INVERESK AND INVERMAY AND ALSO CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS																																								
8	Floodway at Stephenson's Bend by removal of levees around silt deposit areas	19·2	19·1	19·1	19·1	19·1	19·1	19·2	18·9	18·9	–	19·9	18·7	18·4	17·9	17·2	16·9	17·1	17·0	15·6	15·0	15·5	14·7	19·5	–	–	–	18·5	18·8	18·2	–	17·9	15·6	–	19·9	–	16·7	18·8	–	–	–
9	Floodway at Stephenson's Bend by removal of silt deposit levees and and removal of vegetation along river banks such as Ti trees etc.	19·6	19·6	19·7	19·6	19·7	19·7	19·7	19·3	19·3	–	19·7	18·8	18·0	18·3	17·6	17·4	16·8	–	15·9	15·2	15·6	14·8	19·8	–	–	–	18·9	19·1	18·6	–	18·1	16·0	–	19·7	–	17·7	18·9	–	–	–
10	Flow Floodway as for Test 9 but with overbank area roughened by the addition of ⅛" gravel to allow for deterioration in efficiency with time by the growth of vegetation	19·7	19·7	19·7	19·7	19·6	19·7	19·7	19·3	19·3	–	19·8	18·8	18·9	18·3	17·7	17·4	17·5	17·3	15·8	15·2	15·4	14·7	19·9	–	–	–	18·8	19·0	18·4	–	18·0	15·9	–	19·8	–	17·7	19·0	–	–	–
11	Floodway at Stephenson's Bend formed by construction of a channel 5 feet deep (Bed level RL 7) and 550 feet wide along line of Hunter Cut	19·1	19·2	19·2	19·1	19·2	19·1	19·2	18·7	18·7	–	19·4	18·2	18·2	17·6	16·6	16·9	16·8	16·8	16·1	15·1	15·4	15·0	19·6	–	–	–	18·2	18·2	17·8	–	17·3	15·9	–	19·5	–	17·1	18·7	–	–	–
12	bend opp. tailrace realigned by dredging to reduce curvature and widen the river from 285 feet to 510 feet at S.L.W.	19·3	19·3	19·3	19·2	19·3	19·3	19·4	18·9	18·9	–	19·7	18·5	18·5	17·9	17·7	17·8	17·2	17·1	15·7	15·1	15·5	15·0	19·7	–	–	–	18·6	18·6	18·2	–	17·7	15·6	–	19·7	–	17·4	16·9	–	–	–
13	Floodway 5 feet deep and 550 feet wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp. tailrace as for Test 12	18·9	18·9	18·9	18·9	18·9	18·9	18·9	18·5	18·5	–	19·4	18·1	18·1	17·4	17·1	17·2	17·0	16·9	16·1	15·2	15·5	15·1	–	–	–	–	18·3	18·1	17·7	–	17·5	16·1	–	19·2	–	17·1	18·8	–	–	–
14	Hunter Navigation Cut 28 feet deep (Bed RL 16) bottom width 231 feet as shown in figure	18·5	18·5	18·5	18·4	18·5	18·4	18·5	18·0	18·0	–	18·7	18·4	18·3	15·6	–	–	–	–	15·4	15·1	15·4	15·1	19·0	–	–	–	17·6	17·3	16·7	–	15·4	15·4	–	18·4	–	–	18·4	–	–	–
	SUBSIDIARY TESTS																																								
15	Inveresk and Invermay, City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	19·2	19·2	19·2	19·2	19·2	19·2	19·3	18·9	18·9	–	20·0	18·5	18·5	18·1	17·5	17·7	17·0	16·9	15·7	15·1	14·9	14·8	19·5	–	–	–	–	–	–	–	17·7	15·3	–	–	–	17·4	18·7	–	19·7	17·0
16	Inveresk and Invermay City Paterson Street and West Tamar bank levees together with levees 100 feet from river edge on each side of N.Esk below Cypress Street	20·2	20·1	19·7	19·5	19·6	19·4	19·3	19·0	19·0	–	19·8	18·5	18·6	18·2	17·5	17·6	16·9	16·9	15·7	15·1	15·4	14·9	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

TABLE No. 10.

MODEL RESULTS

South Esk Discharge – 200,000 cusecs.

North Esk Discharge – 16,000 cusecs.

TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLANS	FLOOD LEVELS AT GAUGE No.																																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	Base Tests – 1958 Topography no flood protection	23·1	23·1	23·1	23·1	23·1	23·1	23·1	23·1	23·0	–	22·2	21·7	21·6	20·9	19·6	18·9	19·0	18·5	17·0	16·1	16·6	15·9	23·1	23·0	23·1	24·3	21·4	21·6	20·6	23·1	19·3	16·7	22·9	22·4	21·1	18·5	22·9	21·4	–	–
2	Tidal influence levels in boat channel raised by one foot	23·4	23·4	23·4	23·4	23·4	23·4	23·3	23·2	23·3	–	22·3	21·9	22·0	21·4	20·2	19·5	19·6	19·0	17·8	16·3	17·6	17·1	23·5	23·4	23·3	24·3	22·0	21·9	21·0	23·2	20·0	17·5	22·9	22·8	21·2	18·7	23·0	21·5	–	–
	LEVEE PROTECTION ONLY																																								
3	Training levee only	20·5	20·4	20·4	20·4	20·4	20·4	20·4	20·4	20·2	–	19·5	19·6	20·2	19·7	18·5	18·4	17·9	17·9	16·7	16·0	16·6	15·6	20·4	20·6	20·2	22·9	19·6	20·2	19·9	20·5	19·1	16·8	20·7	21·0	20·0	18·0	20·2	19·7	–	–
4	Inveresk and Invermay levees only	26·4	26·4	26·4	26·3	26·3	26·4	26·4	26·3	25·6	–	22·6	21·7	22·0	21·2	18·9	18·7	18·5	18·4	17·0	16·3	16·6	15·8	26·4	26·3	25·8	24·4	22·1	21·4	20·3	–	19·5	16·7	–	21·7	20·1	18·0	21·7	–	–	–
5	Training levee with Inveresk and Invermay levees	21·0	20·9	20·9	20·9	20·9	20·9	20·9	20·7	20·5	–	19·2	19·8	20·4	20·1	18·9	18·4	18·7	18·6	17·0	16·4	16·6	15·9	21·0	20·9	20·5	22·1	19·4	20·4	20·4	19·4	19·3	16·9	19·5	22·1	20·0	18·1	20·5	19·4	–	–
6	Training levee with Inveresk and Invermay and also city levees	22·4	22·4	22·4	22·3	22·3	22·3	22·3	22·0	21·6	–	20·9	21·4	21·8	20·9	19·4	19·5	18·5	18·1	17·1	16·2	16·7	15·8	22·3	–	–	–	21·0	21·6	20·5	–	19·4	16·9	–	22·9	20·8	18·1	22·0	–	–	–
7	Repeat of Test 6 but with the alignment of the levee slightly altered to conform with the prototype survey of 1959	22·0	22·0	22·0	22·0	22·0	21·9	21·9	21·6	21·5	–	21·3	20·9	20·6	20·8	19·2	19·2	18·7	18·6	17·0	16·0	16·5	15·5	21·6	–	–	–	20·7	21·2	20·2	–	19·3	19·1	–	–	20·6	18·6	21·5	–	22·2	21·6
	TRAINING LEVEE WITH INVERESK AND INVERMAY AND ALSO CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS																																								
8	Floodway at Stephenson's Bend by removal of levees around silt deposit areas	21·7	21·7	21·7	21·7	21·7	21·7	21·7	21·4	21·3	–	21·7	20·9	21·5	20·8	19·0	18·7	18·9	18·8	16·9	16·2	16·7	15·7	21·8	–	–	–	20·1	21·1	20·7	–	19·7	16·9	–	22·9	20·4	18·3	21·7	18·5	–	–
9	Floodway at Stephenson's Bend by removal of silt deposit levees and removal of vegetation along river banks such as Ti trees etc.	21·9	21·9	21·9	21·9	21·9	21·9	21·9	21·6	21·4	–	20·8	21·1	21·6	21·6	19·3	19·[illegible]	17·7	–	17·2	16·1	16·7	16·0	22·0	–	–	–	20·9	21·4	20·8	–	19·5	17·1	–	22·7	20·7	19·0	21·8	18·9	–	–
10	Flow Floodway as for Test 9 but with overbank area roughened by the addition of ¾ gravel to allow for deterioration in efficiency with time by the growth of vegetation	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
11	Floodway at Stephenson's Bend formed by construction of a channel 5 feet deep (bed level RL 7) and 550 feet wide along line of Hunter Cut	21·4	21·4	21·4	21·4	21·4	21·4	21·4	21·1	21·0	–	21·3	20·5	20·8	19·8	18·3	18·3	18·1	18·3	17·4	16·3	16·6	15·5	21·5	–	–	–	20·5	20·5	19·9	–	18·7	17·0	–	22·6	20·2	18·1	20·7	–	–	–
12	Bend opp. tailrace realigned by dredging to reduce curvature and widen the river from 285 feet to 510 feet at S.L.W	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
13	Floodway 5 feet deep and 550 feet wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp. tailrace as for Test 12	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
14	Hunter Navigation Cut 28 feet deep (Bed RL 16) bottom width 231 feet as shown in figure	20·8	20·8	20·8	20·8	20·8	20·8	20·8	20·5	20·4	–	21·1	19·8	19·8	17·3	–	–	–	–	16·4	16·2	16·6	16·2	20·9	–	–	–	19·5	19·8	18·5	–	16·8	16·4	–	21·2	19·9	16·8	20·2	–	–	–
	SUBSIDIARY TESTS																																								
15	Inveresk and Invermay, City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
16	Inveresk and Invermay, City, Paterson Street and West Tamar bank levees together with levees 100 feet from river edge on each side of N Esk below Cypress Street	22·3	22·3	22·1	21·9	21·9	21·7	–	21·4	21·3	–	23·4	20·6	21·1	19·5	19·2	19·2	18·4	18·7	17·2	16·5	16·5	16·1	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

TABLE No. II.

MODEL RESULTS

South Esk Discharge = 250,000 cusecs.

North Esk Discharge = 18,800 cusecs.

TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLANS	FLOOD LEVELS AT GAUGE No.																																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	Base Tests - 1958 Topography no flood protection	25·0	25·0	25·0	25·0	25·0	–	24·9	25·0	25·0	–	24·2	23·7	23·5	23·1	21·8	21·5	21·2	20·1	18·3	18·0	17·9	17·3	25·0	24·9	25·3	25·8	23·5	23·7	22·8	24·8	20·9	17·9	24·6	24·0	22·2	20·2	24·6	22·2	–	–
2	Tidal influence - levels in boat channel raised by one foot	25·2	25·2	25·1	25·1	25·1	25·1	25·1	25·1	25·2	–	24·6	23·9	23·8	23·3	22·2	21·9	21·6	21·4	19·1	–	19·0	18·6	25·3	25·2	25·4	26·2	23·9	23·7	23·0	25·0	21·4	18·8	24·9	24·3	22·5	20·5	24·9	22·0	–	–
	LEVEE PROTECTION ONLY																																								
3	Training levee only	21·4	21·4	21·4	21·4	21·4	21·4	21·4	21·4	21·1	–	20·3	20·8	22·0	22·1	21·0	21·1	19·8	19·7	18·3	17·6	17·9	17·3	21·5	21·4	21·3	23·3	20·2	21·9	22·2	21·6	20·6	17·9	21·5	21·9	21·6	19·3	21·6	20·9	–	–
4	Inveresk and Invermay levee only	Flood Level Approximately 31·5									–	19·3	–	22·6	21·5	21·4	21·3	20·2	19·9	18·4	17·8	17·9	17·2	Approx. 31·5			–	25·5	24·3	22·8	20·9	20·9	18·1	21·5	24·3	21·9	19·5	24·5	20·8	–	–
5	Training levee with Inveresk and Invermay levees	22·9	22·8	22·8	22·8	22·8	22·8	22·7	22·7	22·5	–	21·1	21·8	22·8	22·6	21·0	20·9	19·8	19·6	18·3	17·5	17·9	17·9	22·9	22·9	22·5	24·5	22·1	22·6	22·2	20·8	20·8	17·7	20·9	24·0	21·3	19·2	22·7	20·7	–	–
6	Training levee with Inveresk and Invermay and also city levees	23·6	23·5	23·6	23·6	23·6	23·6	23·6	23·3	22·9	–	23·2	22·7	23·5	23·1	21·5	21·4	20·2	19·8	18·3	17·6	17·9	17·1	23·6	–	–	–	22·7	23·6	23·1	–	20·6	18·0	–	25·1	21·7	19·7	23·7	20·7	–	–
7	Repeat of Test 6 but with the alignment of the levees slightly altered to conform with the prototype survey of 1959	23·9	24·0	24·0	23·9	23·9	23·9	23·9	23·6	23·5	–	23·8	22·7	23·5	23·1	21·1	20·9	20·5	20·4	18·4	17·5	17·5	17·1	23·8	–	–	–	22·9	23·5	22·2	–	20·9	17·7	–	–	22·1	20·3	23·4	21·5	24·6	23·8
	TRAINING LEVEE WITH INVERESK AND INVERMAY AND ALSO CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS																																								
8	Floodway at Stephenson's Bend by removal of levees around silt deposit areas	23·6	23·7	23·7	23·7	23·7	23·6	23·7	23·4	23·2	–	23·0	22·7	23·7	23·1	21·3	21·2	20·7	20·2	18·1	17·5	17·9	16·9	23·7	–	–	–	22·4	23·3	23·2	–	21·1	17·9	–	25·1	21·7	20·0	24·0	21·1	–	–
9	Floodway at Stephenson's Bend by removal of silt deposit levees and removal of vegetation along river banks such as Ti trees etc.	23·5	23·5	23·5	23·3	23·5	23·5	23·5	23·2	22·9	–	22·6	22·4	23·2	22·7	21·0	20·9	20·2	20·3	18·2	17·4	17·9	17·0	23·5	–	–	–	22·2	23·2	22·7	–	20·8	17·9	–	24·7	21·6	20·9	23·5	20·8	–	–
10	Flow floodway as for Test 9 but with overbank area roughened by the addition of ¼" gravel to allow for deterioration in efficiency with time by the growth of vegetation	23·4	23·4	23·4	23·3	23·4	23·4	23·4	23·1	22·8	–	22·7	22·5	23·5	22·9	21·2	21·1	20·5	20·5	18·4	17·6	17·9	17·1	23·5	–	–	–	22·0	23·3	22·9	–	21·0	17·9	–	21·7	21·5	21·3	23·6	21·2	–	–
11	Floodway at Stephenson's Bend formed by construction of a channel 5 feet deep (bed level RL7) and 550 feet wide along line of Hunter Cut	23·6	23·6	23·6	23·6	23·6	23·6	23·6	23·4	23·2	–	23·5	22·6	23·2	23·2	19·9	19·7	19·3	19·4	18·7	17·6	17·9	16·8	23·7	–	–	–	22·6	23·2	21·8	–	20·3	18·1	–	24·8	21·3	19·4	23·4	23·7	–	–
12	Bend opp. tailrace realigned by dredging to reduce curvature and widen the river from 285 feet to 510 feet at S.L.W.	23·4	23·5	23·5	23·5	23·5	23·4	23·5	23·2	23·1	–	23·5	22·4	23·1	22·2	21·0	21·0	20·5	20·4	18·3	15·4	15·9	15·0	23·6	–	–	–	22·4	22·9	22·0	–	21·2	18·0	–	24·9	21·4	20·2	23·3	21·1	–	–
13	Floodway 5 feet deep and 550 feet wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp tailrace as for Test 12	23·1	23·1	23·1	23·1	23·1	23·0	23·1	22·7	22·6	–	22·9	21·9	22·5	21·5	20·0	20·2	18·9	–	18·4	17·4	17·8	17·1	23·1	–	–	–	21·8	22·5	21·0	–	20·5	18·0	–	24·1	20·9	19·8	22·6	20·9	–	–
14	Hunter Navigation Cut 28 feet deep (Bed RL 16) bottom width 231 feet as shown in figure	23·1	23·1	23·1	23·1	23·1	23·1	23·1	22·8	22·6	–	22·8	21·8	22·4	21·4	–	–	–	–	17·4	17·1	17·7	17·2	23·1	–	–	–	22·2	22·4	20·6	–	17·8	17·3	–	24·0	21·1	18·2	22·7	–	–	–
	SUBSIDIARY TESTS																																								
15	Inveresk and Invermay, City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	23·8	23·8	23·7	23·7	23·7	23·6	23·7	23·3	23·2	–	23·6	22·4	23·0	22·5	21·0	20·9	20·3	20·3	18·3	17·4	17·4	17·0	23·7	–	–	–	–	–	–	–	21·3	17·8	–	–	21·9	20·6	23·0	21·6	24·7	23·4
16	Inveresk and Invermay, City Paterson Street and West Tamar bank levees together with levees 100 feet from river edge on each side of N.Esk below Cypress Street	24·4	24·3	24·1	24·2	24·0	23·8	23·7	23·4	23·2	–	23·3	22·5	23·1	20·4	21·2	20·6	20·0	20·0	18·3	17·5	17·9	17·1	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

TABLE No. 12.

MODEL RESULTS

South Esk Discharge = 275,000 cusecs.

North Esk Discharge = 20,200 cusecs.

TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLAN	FLOOD LEVELS AT GAUGE No.																																							
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36	37	38	39	40
1	Base Tests – 1958 Topography no flood protection	25·8	25·8	25·8	25·8	25·8	25·8	25·8	25·9	26·1	–	25·3	24·5	24·3	23·9	22·7	22·4	22·1	20·7	18·8	18·2	18·5	–	25·8	25·8	26·3	26·8	24·4	24·2	23·4	25·5	21·7	18·5	25·3	24·8	23·2	20·9	25·5	22·3	–	–
2	Tidal influence – levels in boat channel raised by one foot	26·1	26·1	26·0	26·0	26·0	26·0	26·0	26·1	26·4	–	25·5	24·8	24·7	24·4	23·3	23·1	22·9	21·7	20·5	–	20·5	20·1	26·1	26·0	26·6	27·6	24·7	24·7	24·0	25·8	22·5	20·0	25·5	25·1	24·1	21·7	25·8	22·9	–	–
	LEVEE PROTECTION ONLY																																								
3	Training levee only	21·8	21·8	21·8	21·8	21·8	21·8	21·8	21·7	21·5	–	21·0	21·3	22·6	22·7	21·8	21·8	20·8	20·6	18·9	18·0	18·4	17·6	21·8	21·8	21·8	24·1	21·6	22·6	23·3	21·9	21·5	18·2	22·0	22·0	22·1	20·0	21·8	20·5	–	–
4	Inveresk and Invermay levee only	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
5	Training levee with Inveresk and Invermay levees	24·0	23·9	23·9	23·8	23·9	23·9	23·9	23·8	23·5	–	21·8	23·7	23·7	23·3	22·0	21·6	21·0	20·7	18·9	18·2	18·5	17·8	24·1	24·1	23·7	26·2	22·9	23·7	23·7	21·6	21·7	18·4	21·7	24·5	21·9	20·0	23·6	20·4	–	–
6	Training levee with Inveresk and Invermay and also city levees	24·8	24·8	24·9	24·8	24·8	24·8	24·9	24·7	23·9	–	23·6	23·5	24·5	23·8	21·9	22·0	21·2	20·9	18·8	17·9	18·5	17·7	24·9	–	–	–	23·4	24·7	23·7	–	21·6	18·3	–	26·3	21·9	20·3	24·9	21·8	–	–
7	Repeat of Test 6 but with the alignment of the levees slightly altered to conform with the prototype survey of 1959	24·4	24·5	24·5	24·4	24·5	24·4	24·5	24·1	23·9	–	23·8	23·1	24·1	23·8	21·9	21·7	21·2	21·0	19·0	18·0	18·5	17·5	24·3	–	–	–	23·6	24·2	23·0	–	22·1	18·2	–	–	22·5	21·4	24·0	22·2	25·2	24·4
	TRAINING LEVEE WITH INVERESK AND INVERMAY AND ALSO CITY LEVEES WITH VARIOUS CHANNEL IMPROVEMENTS																																								
8	Floodway at Stephenson's Bend by removal of levees around silt deposit areas	24·2	24·2	24·2	24·2	24·2	24·1	24·2	23·9	23·3	–	23·2	23·1	24·1	23·7	22·1	22·0	21·4	21·1	18·7	18·0	18·5	18·3	24·2	–	–	–	23·0	24·2	23·9	–	21·5	18·5	–	25·1	22·1	20·6	24·5	21·8	–	–
9	Floodway at Stephenson's Bend by removal of silt deposit levees and and removal of vegetation along river banks such as Ti trees etc.	24·0	24·1	24·1	24·1	24·0	24·1	24·2	23·8	23·4	–	23·4	23·2	24·1	23·6	21·7	21·6	21·1	21·2	18·9	17·2	18·5	17·7	24·1	–	–	–	22·9	23·9	23·7	–	21·4	18·3	–	25·2	21·8	22·0	24·1	21·6	–	–
10	Flow floodway as for Test 9 but with overbank area roughened by the addition of $\frac{3}{4}$" gravel to allow for deterioration in efficiency with time by the growth of vegetation	24·3	24·3	24·3	24·2	24·2	24·3	24·3	23·9	23·6	–	23·2	23·2	24·2	23·5	21·4	21·6	21·1	21·3	19·0	18·0	18·5	17·6	24·3	–	–	–	23·4	24·1	23·3	–	21·8	18·5	–	24·9	22·0	21·5	24·1	21·6	–	–
11	Floodway at Stephensons Bend formed by construction of a channel 5 feet deep (bed level RL7)	24·5	24·6	24·6	24·5	24·6	24·5	24·6	24·2	24·1	–	24·3	23·6	24·3	23·4	20·8	20·8	20·3	20·3	19·1	18·1	18·5	17·2	24·7	–	–	–	23·5	24·2	22·9	–	21·0	18·6	–	25·4	21·9	20·4	24·3	21·6	–	–
12	Bend opp. tailrace realigned by dredging to reduce curvature and and widen the river from 285 feet to 510 feet at S.L.W	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
13	Floodway 5 feet deep and 550 feet wide along line of Hunter Cut as for Test 11 combined with realignment of bend opp. tailrace as for Test 12	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
14	Hunter Navigation Cut 28 feet deep (Bed RL 16) bottom width 231 feet as shown in figure	24·0	24·0	24·0	–	24·1	24·0	24·1	23·7	23·6	–	23·7	22·9	23·6	22·7	–	–	–	–	18·1	17·6	18·4	17·4	24·2	–	–	–	23·1	23·4	22·0	–	18·8	17·9	–	24·5	21·5	19·0	23·8	20·0	–	–
	SUBSIDIARY TESTS																																								
15	Inveresk and Invermay City and Paterson Street levees together with levees along West bank of Tamar at Home Reach	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–
16	Inveresk and Invermay, City, Paterson Street and West Tamar bank levees together with levees 100 feet from river edge on each side of N.Esk below Cypress Street	Not	Run	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–	–

TABLE No. 13.

FLOOD LEVELS FOR CREST OF SURGE AT PATERSON ST. LEVEE

SOUTH ESK DISCHARGE (Cusecs)	FLOOD LEVEL AT CREST OF SURGE (Feet above S.L.W)
100,000	23·0
125,000	25·0
150,000	26·6
200,000	29·2
250,000	31·1
275,000	32·0

TABLE No. 14

FLOOD VELOCITIES AT TRAINING LEVEE.

South Esk Discharge in Cusecs.	Velocities in ft/sec. at					
	1	2	3	4	5	6
100,000	15.5	7.5	N.S.V.	3.5	12.5	11.8
125,000	10.0	8.0	N.S.V.	11.0	12.0	15.3
150,000	18.5	9.0	N.S.V.	16.6	13.1	17.6
200,000	16.8	10.2	20.0	15.6	19.5	21.0
250,000	30.5	12.0	11.8	23.2	21.0	23.5
275,000	56.2	16.5	11.0	25.5	20.0	32.0

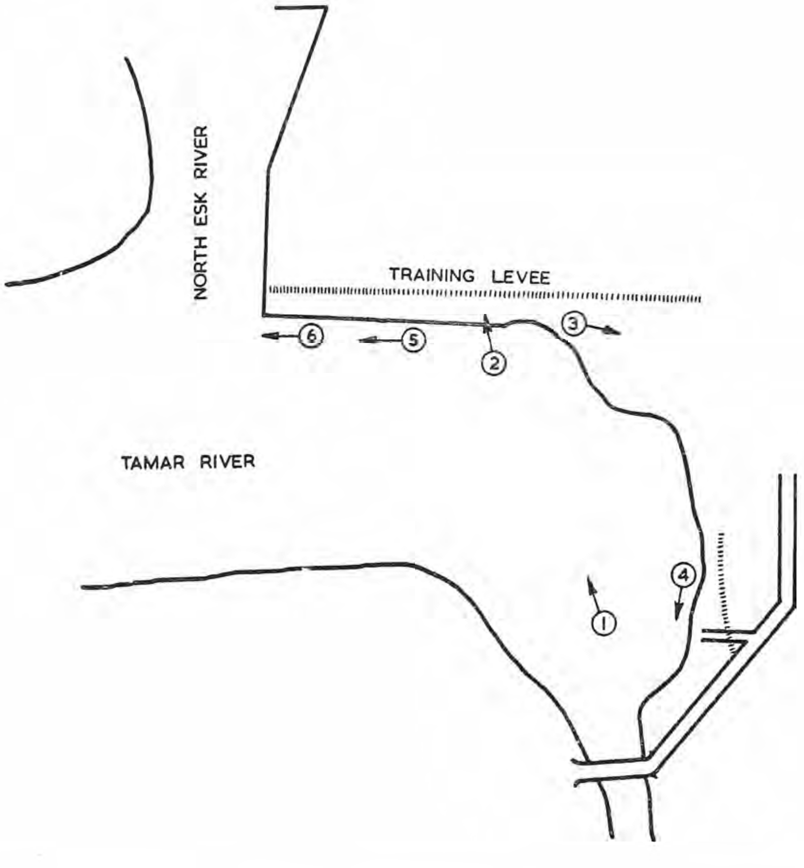


TABLE No. 15.

SURFACE VELOCITIES FOR TAMAR RIVER AT THE POWDER JETTY FOR A TOTAL FLOOD DISCHARGE OF 110,000 CUSECS.

	ELEMENTS OF PLAN TESTED	SURFACE VELOCITY (ft/sec)
1.	Base Test - 1958 Topography. No flood protection	10·3
2.	Training levee with Inveresk - Invermay and City levees	9·4
3.	All levees with floodway at Stephenson's Bend by removal of levees around silt deposit areas	9·9
4.	All levees with floodway at Stephenson's Bend 5 feet deep and 550 feet wide along line of Hunter Cut	10·2
5	All levees with Hunter Navigation Cut	10·0

TABLE 16

DETAILS OF PROPOSED GATE OPENINGS THROUGH LEVEES

LOCATION	TYPE	Proposed width (ft.)	Existing ground or embankment level (ft.)	Required ht. to protect against max. probable flood (ft.)	LOCATION	TYPE	Proposed width (ft)	Existing ground or embankment level (ft)	Required ht. to protect against max. probable flood (ft)
Inveresk - Invermay Zone					Royal Park Willis St. Zone				
1. Main line at railway bridge	Railway	15	18·0	6·5	1. Alexander Wharf	Road	15	15·5	9·5
2. Tamar St. bridge	Road	61	19·3	5·2	2. Town pier	Road	20	17·0	8·0
3. G.D.Saunders Pty.Ltd.	Road	10	17·2	7·3	4. Esplanade wharves	Road	15	17·5	7·0
4. Shell bulk fuel depot	Road	10	17·2	7·3	3 Charles St.	Road	62	18·2	6·3
5 Charles St. bridge	Road	62	18·0	6·5	5. Esplanade wharves	Pedestrian	2·5	16·1	8·4
6. Floating dock area	Road	10	16·3	9·0	6. Tamar St. bridge	Road	61	19·2	5·3
7. Cattle jetty	Drop gate	2	16·2	10·6	7. N. Esk Rowing Club	Pedestrian	2·5	16·3	8·2
8. Silo area	Road	20	17·5	7·5	Willis-Cypress St. Zone				
9. Railway to Kings Wharf at Lindsay St.	Railway	15	18·0	7·0	1. Railway bridge at Main line	Railway	15	19·3	5·2
10. Kings Wharf at Lindsay St.	Road	40	18·4	6·6	2. Access to	R	10	19·0	5·5
11. Railway to Kings Wharf	Railway	15	17·0	7·4	3. Henry St.	Road	20	16·5	8·0
12. Kings Wharf at Goderich St.	Road	40	14·3	10·1	4. Cypress St.	Railway	15	19·3	5·2
13. L.W.Smith	Pedestrian	2·5	13·1	11·3	Mowbray Flats Zone				
14. Kings Wharf at Forster St.	Road	20	10·8	13·6	1. M^{c}Kenzie St.	Road	15	16·0	6·5
15. River St.	Road	15	15·0	8·1	Paterson St Levee				
					1. Ritchies Mill	Road	15	22·0	9
					2. Pumping station	Road	10	23·7	7·3

TABLE No. 17.

SUMMARY OF COSTS FOR INVERESK — INVERMAY ZONE.

Section	General Flood Stage represented by flood levels at Tamar St.			
	R.L. 17	R.L. 20	R.L. 22	R.L. 24·5
1 Concrete levees.	22,590	41,600	61,160	94,180
2. Earth levees.	4,880	9,750	16,700	24,950
3 Railway banks.	1,750	14,500	27,500	46,000
4 Herbert St. cut-off levee.	120	1,890	3,060	5,090
5. Mayne St. saddle levee.	Nil.	Nil	2,550	6,490
6 Gate closures and ramp.	6,150	16,650	28,470	38,850
7. Land resumption and demolition.	10,600	10,600	10,600	10,600
8 Sewerage and local drainage.	2,000	2,000	2,000	2,000
9. Site preparation.	1,000	1,000	1,000	1,000
10. Design and supervision.	2,980	5,880	9,180	13,750
11. Contingencies.	5,200	10,390	16,220	24,290
TOTAL Cost	57,270	114,260	178,440	267,200

TABLE No. 18.

SUMMARY OF COSTS FOR ROYAL PARK – WILLIS ST. ZONE.

Section	General Flood Stage represented by flood levels at Tamar St.			
	R.L. 17	R.L. 20	R.L. 22	R.L. 24·5
1. Concrete levees.	17,470	30,830	44,350	65,030
2. Earth levees.	1,600	5,880	11,450	18,030
3. Willis St. cut-off levee.	360	1,480	3,330	7,020
4. Gate closure, Ramp & Access Roads.	7,250	12,850	16,400	21,450
5. Land resumptions.	12,000	12,000	12,000	12,000
6. Sewerage and local drainage.	2,000	2,000	2,000	2,000
7. Site preparation.	500	500	500	500
8. Design and supervision.	2,470	3,930	4,800	7,560
9. Contingencies.	4,370	6,950	8,480	13,360
TOTAL Cost	48,020	76,420	93,310	146,950

TABLE No. 19.

SUMMARY OF COSTS FOR WILLIS — CYPRESS ST. ZONE.

Section	General Flood Stage represented by flood levels at Tamar St.			
	R.L.17	R.L.20	R.L.22	R.L.24·5
1. Concrete levees.	5,850	10,290	14,030	18,330
2. Earth levees.	30	190	530	900
3. Railway side banks.	3,500	18,200	33,500	55,500
4. Gate closures and ramp.	Nil.	3,800	7,600	10,900
5. Sewerage and local drainage.	3,000	3,000	3,000	3,000
6. Site preparation.	500	500	500	500
7. Design and supervision.	880	2,160	3,550	5,350
8. Contingencies.	1,350	3,810	6,270	9,480
TOTAL Cost	13,810	41,950	68,980	103,960

TABLE No. 20.
SUMMARY OF COSTS FOR
MOWBRAY FLATS ZONE.

Section	General Flood Stage represented by flood levels at Tamar St.			
	R.L. 17	R.L. 20	R.L. 22	R.L. 24.5
1. Earth levees.		Nil.	22,650	35,880
2. Railway banks.		9,500	17,300	27,000
3. Gate closures and ramps.		Nil.	2,700	3,200
4. Land resumption and demolition.	Nil	2,000	2,000	2,000
5. Sewerage and local drainage.		6,000	6,000	6,000
6. Site preparation.		500	500	500
7. Design and supervision.		1,080	3,070	4,480
8 Contingencies.		1,910	5,420	7,900
TOTAL Cost	Nil	20,990	59,640	86,960

Table No. 21.

SUMMARY OF COSTS FOR ROYAL PARK TRAINING WALL.

Section.	South Esk Discharge in Cusecs.			
	125,000	150,000	200,000	250,000
1. Concrete core wall.	£6,040	£12,470	£18,650	£32,840
Earth side batters and grassing.	£3,390	£10,000	£17,580	£39,800
Design and supervision.	£570	£1,350	£2,170	£4,360
Contingencies.	£1,000	£2,380	£3,840	£7,700
TOTAL COSTS	£11,000	£26,200	£42,240	£84,700

Table No. 22.

SUMMARY OF COSTS — PATERSON ST. LEVEE.

Protection against max. probable flood.

Section.	Cost.
Concrete levees.	£7,340
Gate closures and ramps.	£6,000
Land resumption and demolition.	£500
Sewerage and local drainage.	£500
Site preparation.	£200
Design and supervision.	£870
Contingencies.	£1,540
TOTAL COST	£16,950

BENEFIT COST ANALYSIS.

Training Levee Only.

Note: All Damages, Benefits and Costs are given in Thousands of Pounds (Australian).

DISCHARGE 1000 Cusecs.	Period of Emerg. Meas.	Flood STAGE IN FEET.		FLOOD FREQUENCY Number per year.		DAMAGE WITHOUT FLOOD PROTECTION FOR STAGE IIV (3).						DAMAGE WITH TRAINING LEVEE FOR STAGE IIV (4).						BENEFIT.				COST.		No Enhancement and Development.		With Enhancement and Development.	
		No Training Levee	With Training Levee	For Discharge	For Interval	Mowbray Flats Zone	Willis St. Cypress St Zone	Royal Park Willis St. Zone	Wharf Area. Zone	Inveresk Invermay Zone	TOTAL	Mowbray Flats. Zone	Willis St. Cypress St Zone	Royal Park Willis St. Zone	Wharf Area	Inveresk Invermay Zone	TOTAL	For Discharge	For Interval	Average Annual	Cumulative Average Annual	Annual	Increment	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)
																		(12)–(18)		(20)x(6)				(21) ÷ (24)	(22) ÷ (23)	(25)x2	(26)x2
90	6	–	–	·0610		NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL			NIL	NIL			NIL		NIL
					·0100														NIL	NIL			NIL	NIL		NIL	
100	6	16·7	16·7	·0510		NIL	30	22	NIL	NIL	52	NIL	30	22	NIL	NIL	52	NIL			NIL	NIL			NIL		NIL
					·0180														16·5	30			·57	·53		1·06	
125	6	18·4	18·3	·0330		55	100	125	30	1452	1762	52	100	120	25	1432	1729	33			·30	·57			·53		1·06
					·0130														374·5	4·87			·79	6·16		12·32	
150	6	20·7	19·1	·0200		98	148	285	290	1915	2736	70	115	168	85	1582	2020	716			5·17	1·36			3·81		7·62
					·0110														999·5	10·99			·35	31·40		62·80	
175	6	22·0	19·8	·0090		120	180	398	438	2465	3601	80	128	210	185	1715	2318	1283			16·16	1·71			9·44		18·88
					·0060														1250	7·50			·48	15·62		31·24	
200	6	23·1	20·4	·0030		152	200	462	465	2518	3797	92	140	258	260	1830	2580	1217			23·66	2·19			10·80		21·60
					·0024														1156	2·78			·91	3·06		6·12	
225	6	24·1	20·9	·0006		182	220	515	490	2565	3972	100	152	300	315	2010	2877	1095			26·44	3·10			8·54		17·08
					·0005														912	·46			1·28	·36		·72	
250	6	25·0	21·4	·0001		212	235	555	512	2605	4119	110	165	345	370	2400	3390	729			26·90	4·38			6·14		12·28
90	24	–	–	·0610		NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL	NIL			NIL	NIL			NIL		NIL
					·0100														NIL	NIL			NIL	NIL		NIL	
100	24	16·7	16·7	·0510		NIL	15	15	NIL	NIL	30	NIL	15	15	NIL	NIL	30	NIL			NIL	NIL			NIL		NIL
					·0180														11	·20			·57	·35		·70	
125	24	18·4	18·3	·0330		42	80	90	18	1170	1400	40	80	88	15	1155	1378	22			·20	·57			·35		·70
					·0130														302	3·93			·79	4·98		9·96	
150	24	20·7	19·1	·0200		80	115	225	260	1512	2192	52	90	128	70	1270	1610	582			4·13	1·36			3·04		6·08
					·0110														764	8·41			·35	24·00		48·00	
175	24	22·0	19·8	·0090		100	138	315	398	1845	2796	65	100	160	150	1375	1850	946			12·54	1·71			7·33		14·66
					·0060														941	5·64			·48	11·76		23·52	
200	24	23·1	20·4	·0030		130	155	360	422	1932	2999	70	110	200	228	1455	2063	936			18·18	2·19			8·29		16·58
					·0024														918	2·21			91	2·43		4·86	
225	24	24·1	20·9	·0006		160	170	405	445	2015	3195	80	120	240	280	1575	2295	900			20·39	3·10			6·57		13·14
					·0005														822·5	·41			1·28	·32		0·64	
250	24	25·0	21·4	·0001		190	180	440	465	2085	3360	90	128	285	332	1780	2615	745			20·80	4·38			4·74		9·48

TABLE No 24

BENEFIT COST ANALYSIS.

Zone Surround Levees of Various Heights combined with T.L. to divert 250,000 cusecs.
All Damages, Benefits and Costs are expressed in A.£1,000.

INVERESK — INVERMAY ZONE

DISCHARGE 1000 Cusecs	Period of Emerg. Meas.	Flood STAGE in feet	FLOOD FREQUENCY Number per year		BENEFIT DUE TO INVERESK INVERMAY LEVEES OF HEIGHT FOR (3)				COSTS of I I LEVEES		No Enhancement and Development		With Enhancement and Development	
		With training levee only.	For Discharge	For Interval	For Discharge	For Interval	Average Annual	Cumulative Average Annual	Average Annual	Increment	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
							(5)x(7)				(8)÷(11)	(9)÷(10)	(12)x2	(13)x2
90	6	—	.0610		NIL				NIL					
				.0100		NIL	NIL			NIL	NIL		NIL	
100	6	16.7	.0510		NIL			NIL	NIL			NIL		
				.0180		716	12.90			4.0	3.22		6.44	
125	6	18.3	.0330		1432			12.90	4.00			3.22		6.44
				.0130		1507	19.60			.80	24.50		49.00	
150	6	19.1	.0200		1582			32.50	4.80			6.78		13.56
				.0110		1648.5	18.11			.80	22.65		45.30	
175	6	19.8	.0090		1715			50.61	5.60			9.04		18.08
				.0060		1772.5	10.64			.80	13.30		26.60	
200	6	20.4	.0030		1830			61.25	6.40			9.56		19.12
				.0024		1920	4.61			.75	6.15		12.30	
225	6	20.9	.0006		2010			65.86	7.15			9.21		18.42
				.0005		2205	1.10			.85	1.29		2.58	
250	6	21.4	.0001		2400			66.96	8.00			8.36		16.72
90	24	—	.0610		NIL				NIL					
				.0100		NIL	NIL			NIL	NIL		NIL	
100	24	16.7	.0510		NIL			NIL	NIL			NIL		NIL
				.0180		577.5	10.40			4.00	2.60		5.20	
125	24	18.3	.0330		1155			10.40	4.00			2.60		5.20
				.0130		1212.5	15.78			.80	19.70		39.40	
150	24	19.1	.0200		1270			26.18	4.80			5.45		10.90
				.0110		1322.5	14.57			.80	18.20		36.40	
175	24	19.8	.0090		1375			40.75	5.60			7.27		14.54
				.0060		1415	8.49			.80	10.60		21.20	
200	24	20.4	.0030		1455			49.24	6.40			7.70		15.40
				.0024		1515	3.64			.75	4.86		9.72	
225	24	20.9	.0006		1575			52.88	7.15			7.40		14.80
				.0005		1677.5	.84			.85	.99		1.98	
250	24	21.4	.0001		1780			53.72	8.00			6.73		13.46

ROYAL PARK - WILLIS ST. ZONE

DISCHARGE 1000 Cusecs	Period of Emerg. Meas.	Flood STAGE in feet	FLOOD FREQUENCY Number per year		BENEFIT DUE TO ROYAL PARK — WILLIS ST. LEVEES. OF HEIGHT FOR (3)				COSTS of R—W LEVEES		No Enhancement and Development		With Enhancement and Development	
		With training levee only	For Discharge	For Interval	For Discharge	For Interval	Average Annual	Cumulative Average Annual	Average Annual	Increment	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)
							(5)x(7)				(8)÷(11)	(9)÷(10)	(12)x2	(13)x2
90	6	—	.0610		NIL				—					
				.0100		11	.11			.60	.18		.36	
100	6	16.7	.0510		22			.11	.60			.18		.36
				.0180		71	1.28			2.40	.53		1.06	
125	6	18.3	.0330		120			1.39	3.00			.46		.92
				.0130		144	1.87			.45	4.16		8.32	
150	6	19.1	.0200		168			3.26	3.45			.95		1.90
				.0110		189	2.08			.35	5.94		11.88	
175	6	19.8	.0090		210			5.34	3.80			1.40		2.80
				.0060		234	1.40			.30	4.67		9.34	
200	6	20.4	.0030		258			6.74	4.10			1.64		3.28
				.0024		279	.67			.20	3.35		6.70	
225	6	20.9	.0006		300			7.41	4.30			1.72		3.44
				.0005		322.5	.16			.25	.64		1.28	
250	6	21.4	.0001		345			7.57	4.55			1.66		3.32
90	24	—	.0610		NIL				—		—			
				.0100		7.5	.08			.60	.13		.26	
100	24	16.7	.0510		15			.08	.60			.13		.26
				.0180		51.5	.93			2.40	.39		.78	
125	24	18.3	.0330		88			1.01	3.00			.34		.68
				.0130		108	1.40			.45	3.11		6.22	
150	24	19.1	.0200		128			2.41	3.45			.70		1.40
				.0110		144	1.59			.35	4.54		9.08	
175	24	19.8	.0090		160			4.00	3.80			1.05		2.10
				.0060		180	1.08			.30	3.60		7.20	
200	24	20.4	.0030		200			5.08	4.10			1.24		2.48
				.0024		220	.53			.20	2.65		5.30	
225	24	20.9	.0006		240			5.61	4.30			1.30		2.60
				.0005		262.5	.13			.25	.52		1.04	
250	24	21.4	.0001		285			5.74	4.55			1.26		2.52

BENEFIT-COST ANALYSIS.

Zone Surround Levees of Various Heights combined with T.L. to divert 250,000 cusecs.
All Damages, Costs and Benefits are expressed in A £1,000.

TABLE No. 25

WILLIS ST. – CYPRESS ST. ZONE.

DISCHARGE 1000 Cusecs.	Period of Emerg. Meas.	Flood STAGE in feet	FLOOD FREQUENCY Number per year.		BENEFIT DUE TO WILLIS ST.-CYPRESS ST. LEVEES. OF HEIGHT FOR (3).				COSTS of W-C LEVEES		No Enhancement and Development		With Enhancement and Development	
		With Training Levee only.	For Discharge	For Interval	For Discharge	For Interval	Average Annual	Cumulative Average Annual	Average Annual	Increment	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8) (5×7)	(9)	(10)	(11)	(12) (8)÷(11)	(13) (9)÷(10)	(14) (12)×2	(15) (13)×2
90	6	—	0610		NIL				NIL					
				·0100		15	·15			·50	·30		·60	
100	6	16·7	·0510		30			·15	·50			·30		·60
				·0180		65	1·17			·60	1·95		3·90	
125	6	18·3	·0330		100			1·32	1·10			1·20		2·40
				·0130		107·5	1·40			·45	3·12		6·24	
150	6	19·1	·0200		115			2·72	1·55			1·75		3·50
				·0110		121·5	1·34			·45	2·98		5·96	
175	6	19·8	·0090		128			4·06	2·00			2·03		4·06
				·0060		134	·80			·42	1·90		3·80	
200	6	20·4	·0030		140			4·86	2·42			2·01		4·02
				·0024		146	·35			·38	·92		1·84	
225	6	20·9	·0006		152			5·21	2·80			1·86		3·72
				·0005		158·5	·08			·35	·23		·46	
250	6	21·4	·0001		165			5·29	3·15			1·68		3·36
90	24	—	·0610		NIL				NIL					
				·0100		7·5	·08			·50	·16		·32	
100	24	16·7	·0510		15			·08	·50			·16		·32
				·0180		47·5	·86			·60	1·43		2·86	
125	24	18·3	·0330		80			·94	1·10			·85		1·70
				·0130		85	1·11			·45	2·47		4·94	
150	24	19·1	·0200		90			2·05	1·55			1·32		2·64
				·0110		95	1·04			·45	2·32		4·64	
175	24	19·8	·0090		100			3·09	2·00			1·55		3·10
				·0060		105	·63			·42	1·50		3·00	
200	24	20·4	·0030		110			3·72	2·42			1·54		3·08
				·0024		115	·28			·38	·74		1·48	
225	24	20·9	·0006		120			4·00	2·80			1·43		2·86
				·0005		124	·06			·35	·17		·34	
250	24	21·4	·0001		128			4·06	3·15			1·29		2·58

MOWBRAY ZONE.

DISCHARGE 1000 Cusecs.	Period of Emerg. Meas.	Flood STAGE in feet	FLOOD FREQUENCY Number per year.		BENEFIT DUE TO MOWBRAY LEVEES. OF HEIGHT FOR (3)				COSTS of Mowbray Levees		No Enhancement and Development		With Enhancement and Development	
		With Training Levee only.	For Discharge	For Interval	For Discharge	For Interval	Average Annual	Cumulative Average Annual	Average Annual	Increment	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio	Inc. Benefit Inc. Cost Ratio	Benefit Cost Ratio
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8) (5)×(7)	(9)	(10)	(11)	(12) (8)÷(11)	(13) (9)÷(10)	(14) (12)×2	(15) (13)×2
90	6	—	·0610		NIL				NIL					
				·0100		NIL	NIL			NIL	NIL		NIL	
100	6	16·7	·0510		NIL			NIL	NIL			NIL		NIL
				·0180		26	·47			·28	1·68		3·36	
125	6	18·3	·0330		52			·47	·28			1·68		3·36
				·0130		61	·79			·32	2·47		4·94	
150	6	19·1	·0200		70			1·26	·60			2·10		4·20
				·0110		75	·83			·40	2·08		4·16	
175	6	19·8	·0090		80			2·09	1·00			2·09		4·18
				·0060		86	·52			·50	1·04		2·08	
200	6	20·4	·0030		92			2·61	1·50			1·74		3·48
				·0024		96	·23			·50	·46		·92	
225	6	20·9	·0006		100			2·84	2·00			1·42		2·84
				·0005		105	·05			·50	·10		·20	
250	6	21·4	·0001		110			2·89	2·50			1·16		2·32
90	24	—	·0610		NIL				NIL					
				·0100		NIL	NIL			NIL	NIL		NIL	
100	24	16·7	·0510		NIL			NIL	NIL			NIL		NIL
				·0180		20	·36			·28	1·29		2·58	
125	24	18·3	·0330		40			·36	·28			1·29		2·58
				·0130		46	·60			·32	1·87		3·74	
150	24	19·1	·0200		52			·96	·60			1·60		3·20
				·0110		58·5	·64			·40	1·60		3·20	
175	24	19·8	·0090		65			1·60	1·00			1·60		3·20
				·0060		67·5	·41			·50	·82		1·64	
200	24	20·4	·0030		70			2·01	1·50			1·34		2·68
				·0024		75	·18			·50	·36		·72	
225	24	20·9	·0006		80			2·19	2·00			1·10		2·20
				·0005		85	·04			·50	·08		·16	
250	24	21·4	·0001		90			2·23	2·50			·89		1·78

TABLE No. 26

BENEFIT COST ANALYSIS

ALL DAMAGES BENEFITS AND COSTS ARE EXPRESSED IN A£1000

FOR SURROUND LEVEES FOR ALL BUILT UP ZONES TO PROTECT AGAINST 150,000 CUSECS COMBINED WITH TRAINING LEVEE TO DIVERT 250,000 CUSECS

ALL LEVEED ZONES											UNLEVEED ZONES (WHARF AREA)							
Discharge 1000 Cusecs	Period of Emergency	Stage In Feet		Flood Frequency Number per Year		Damages		Benefits			Stage (Home Reach)		Damages		Benefits			Average Annual Benefit For Whole Area No E. & D.
		No Flood Protection	With Flood Protection	For Discharge	For Interval	No Flood Protection	With Flood Protection	For Discharge	For Interval	Average Annual	No Flood Protection	With Flood Protection	No Flood Protection	With Flood Protection	For Discharge	For Interval	Average Annual	
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9) (7-8)	(10)	(11) (10 x 6)	(12)	(13)	(14)	(15)	(16)	(17)	(18) (17 x 6)	(19) (11 + 18)
90	24			·0610		NIL	NIL	NIL			15·7	15·7	NIL	NIL	NIL			
					·0100				15	·15						NIL	NIL	·150
100	24	16·7	17·10	·0510		30	NIL	30			16·2	16·2	NIL	NIL	NIL			
					·0180				706	12·72						NIL	NIL	12·720
125	24	18·4	18·90	·0330		1382	NIL	1382			17·5	17·5	10	10	NIL			
					·0130				1657	21·52						5	·065	21·585
150	24	20·7	19·75	·0200		1932	NIL	1932			19·0	18·9	60	50	10			
					·0110				1258·5	13·82						15	165	13·985
175	24	22·0	20·50	·0090		2398	1813	585			20·4	20·1	220	200	20			
					·0060				456	2·74						30	·180	2·920
200	24	23·1	21·22	·0030		2577	2250	327			21·8	21·2	350	310	40			
					·0024				343·5	·83						25	·060	0·890
225	24	24·1	21·80	·0006		2750	2390	360			23·0	22·3	415	405	10			
					·0005				397	·20						10	·005	0·205
250	24	25·0	22·30	·0001		2895	2461	434			24·0	23·4	440	430	10			

Total Annual Benefit Without Allowance For Enhancement and Development = ∑(19) = 52-455

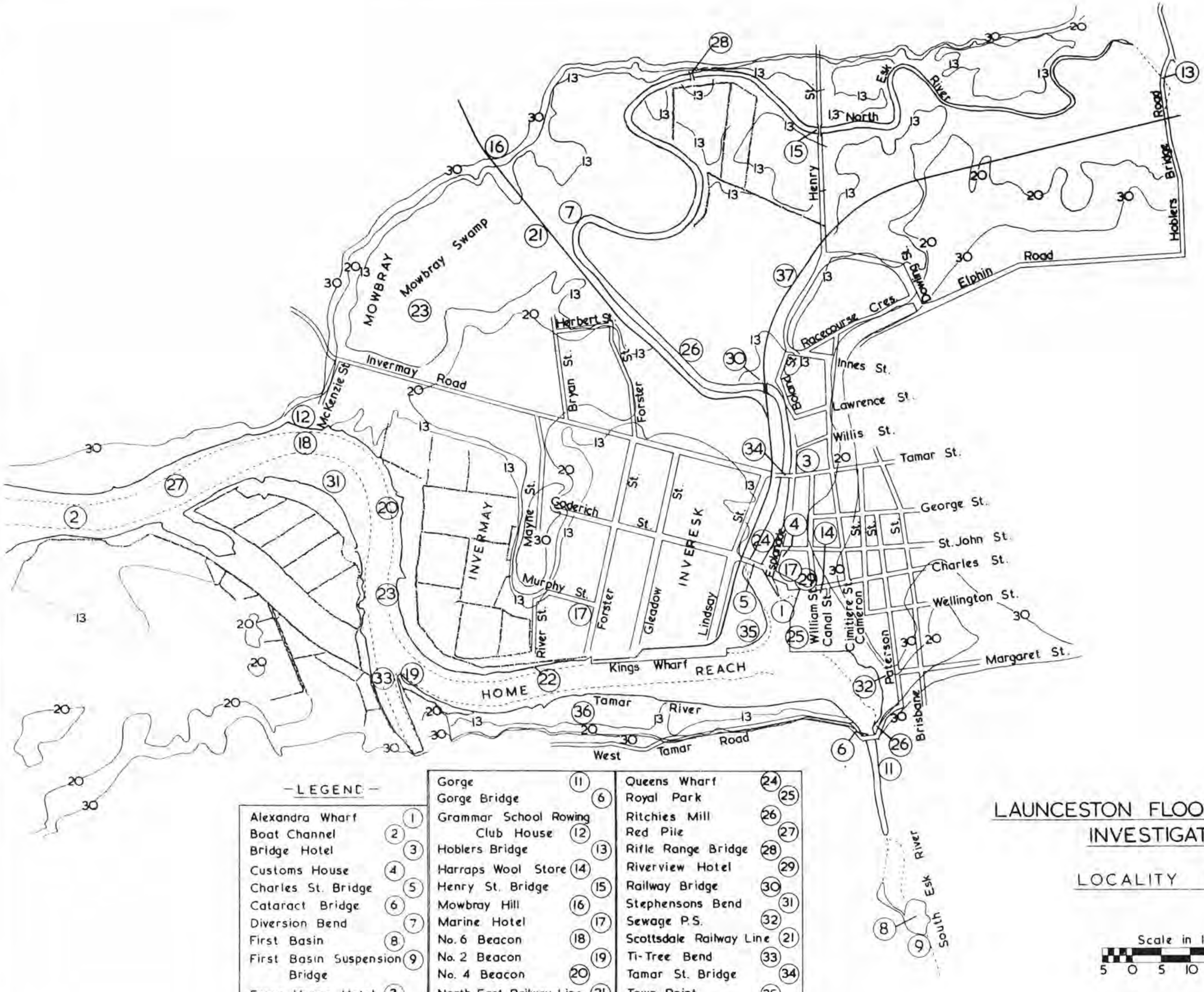
Total Annual Benefit With Allowance For Enhancement and Development = 2 x (∑19) = 104·910

Total Annual Cost = 16·65

Benefit Cost Ratio = 6·29

TABLE No. 27.
PROPOSED LEVEE DETAILS FOR PROTECTION AGAINST MAX. PROBABLE . FLOOD.

Location	Chainage (ft.)	Flood Level (ft. above S.L.W.)	Surge Amplitude from Model (ft.)	Proposed Freeboard (ft.)	R.L. of Top of Levee (ft. above S.L.W.)	Approx. Present Ground or Embankment level. (ft. above S.L.W.)	Approx. Levee Height. (ft.)
INVERESK-INVERMAY ZONE.							
Kelsall and Kemp.	0	22·1	Neg.	0·5	22·6	22·6	0
	125	22·1	Neg.	0·5	22·6	20·5	2·1
At change in levee direction.	635	22·1	Neg.	0·5	22·6	20·5	2·1
℄ River St.	648	22·1	1·1	1·0	23·1	15·0	8·1
Base of Rd. bank.	690	22·1	1·1	1·0	23·1	10·0	13·1
Base of bank.	734	22·1	1·1	1·0	23·1	9·7	13·4
Edge of bank.	755	22·1	1·1	1·0	23·1	15·6	7·5
Edge of bank.	1062	22·7	1·1	1·0	23·7	19·5	4·2
Base of bank.	1107	22·7	1·1	1·0	23·7	11·6	12·1
Forster St.- north boundary.	1865	23·4	1·1	1·0	24·4	11·7	12·7
Forster St.- south boundary.	1960	23·4	1·1	1·0	24·4	11·8	12·6
Gleadow St. at W.E. Smiths.	2485	23·5	Neg.	1·0	24·5	12·8	11·7
Gleadow St. at Kings Wharf.	2540	23·5	1·0	1·0	24·5	14·3	10·2
Railway to Kings Wharf.	3430	23·8	1·0	1·0	24·8	17·0	7·8
Lindsay St. at Kings Wharf.	3830	24·0	1·0	1·0	25·0	18·4	6·6
Access Rd. to silos.	4070	24·4	3·6	1·8	26·2	17·5	8·7
Cattle Jetty.	4220	24·5	4·2	2·1	26·6	16·5	10·1
S.W. Cnr. Websters Wool Store.	4470	24·8	6·4	3·2	28·0	15·0	13·0
	4770	23·8	1·5	1·2	25·0	16·0	9·0
Charles St. Bridge.	6200	23·5	1·0	1·0	24·5	17·0	7·5
Tamar St. Bridge.	7700	23·6	Neg.	0·5	24·1	19·0	5·1
Railway Bridge over N. Esk.	9000	23·9	Neg.	0·5	24·4	18·0	6·4
At intersection of Herbert St. levee.	11860	24·0	Neg.	0·5	24·5	18·5	6·0
Herbert St.	12110	24·0	Neg.	0·5	24·5	24·5	0
MAYNE ST. SADDLE.							
Cnr. Mayne St. & Invermay Rd.	0	21·5	Neg.	Zero	21·5	21·5	0
Cnr. Mayne St. & Eddy St.	375	21·5	Neg.	Zero	21·5	19·5	2·0
Near cnr. Mayne St. & Holbrook St.	750	21·5	Neg.	Zero	21·5	21·5	0
ROYAL PARK - WILLIS ST.							
ZONE.							
Royal Park at basement of Police Station.	0	23·5	1·8	1·5	25·0	25·0	0
Canal St. at Royal Park. N.W. cnr. Gunn's Timber Yard.	510	23·5	1·8	1·5	25·0	15·0	10·0
S.W. Cnr. Harraps Wool Store.	1010	23·5	1·8	1·5	25·0	17·3	7·7
N.W. Cnr. Harraps Wool Store.	1115	23·5	1·8	1·5	25·0	15·2	9·8
N.E. Cnr. Harraps Wool Store.	1455	23·5	1·2	1·0	24·5	15·0	9·5
Tasmanian Pattern Makers.	1900	23·5	1·2	1·0	24·5	13·6	10·9
Charles St. Bridge.	2380	23·5	1·2	1·0	24·5	18·2	6·3
Shields St.	3450	23·6	Neg.	0·5	24·1	16·1	8·0
Tamar St. Bridge.	4190	23·6	Neg.	0·5	24·1	19·2	4·9
At intersection of Willis St. levee.	5150	23·7	Neg.	0·5	24·2	17·0	7·2
Cimitiere St.	5900	23·7	Neg.	0·5	24·2	24·2	0
WILLIS-CYPRESS ST. ZONE.							
At intersection of Willis St. levee.	0	23·7	Neg.	0·5	24·2	17·0	7·2
Railway Bridge over N. Esk.	1200	23·9	Neg.	0·5	24·4	19·0	5.4
Henry St.	2570	23·9	Neg.	0·5	24·4	18·0	6·4
Cypress St.	4500	23·9	Neg.	0·5	24·4	24·4	0
MOWBRAY ST. ZONE.							
At intersection of Herbert St. levee	0	23·9	Neg	Zero	23·9	18·5	5·4
Mowbray Swamp.	2500	23·9	Neg.	Zero	23·9	18·7	5·2
Mowbray Hill.	3440	23·9	Neg.	Zero	23·9	23·9	0
McKENZIE ST. SADDLE							
Rosslyn Road.	0	21·5	Neg.	Zero	21·5	21·5	0
Mowbray St.	730	21·3	Neg.	Zero	21·3	13·7	7·6
McKenzie St.	1724	21·0	Neg.	Zero	21·0	16·0	5·0
Mowbray Hill.	1830	21·0	Neg.	Zero	21·0	21·0	0



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

LOCALITY PLAN

— LEGEND —

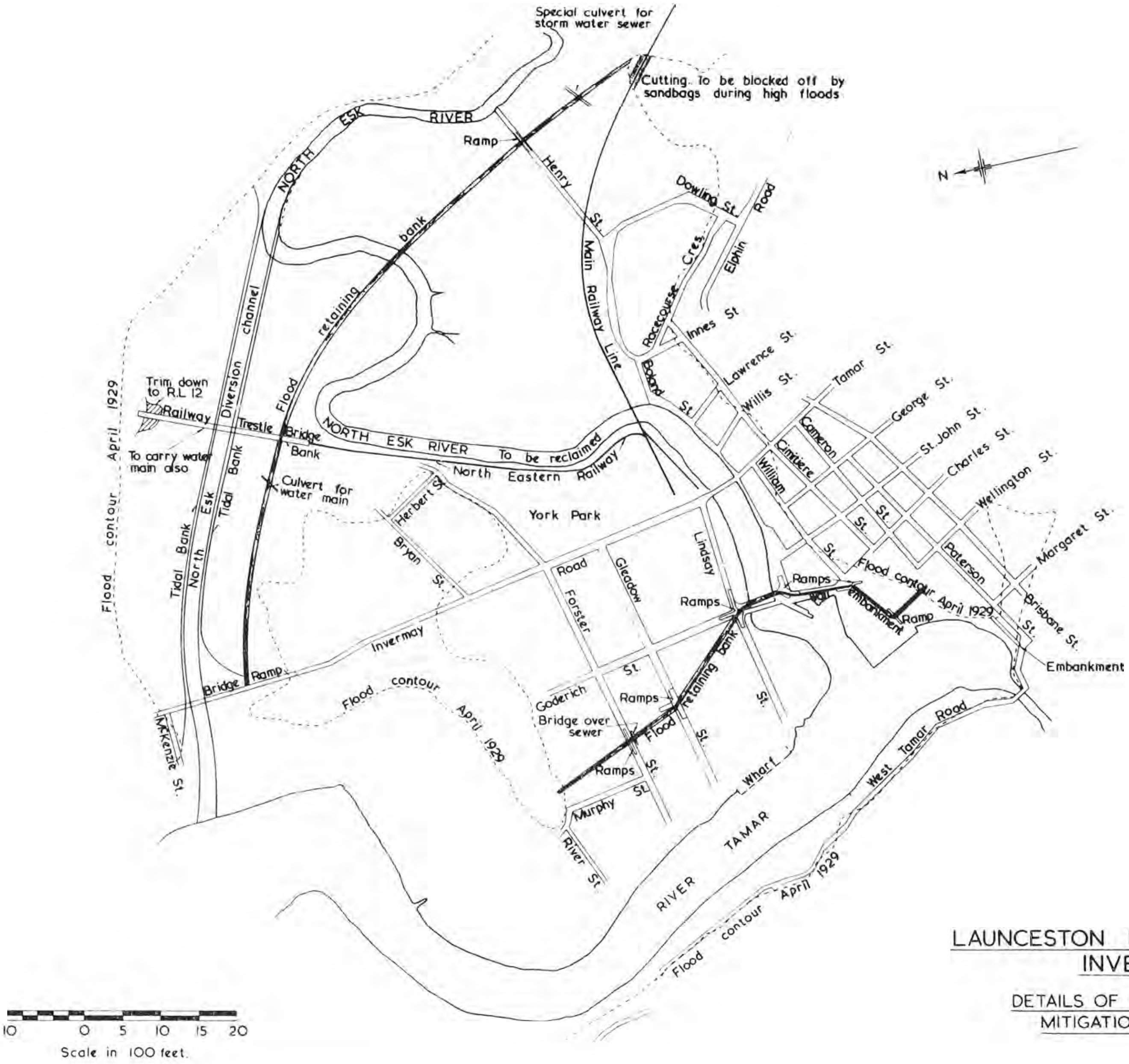
Alexandra Wharf	①
Boat Channel	②
Bridge Hotel	③
Customs House	④
Charles St. Bridge	⑤
Cataract Bridge	⑥
Diversion Bend	⑦
First Basin	⑧
First Basin Suspension Bridge	⑨
Ferry House Hotel	③
Gorge	⑪
Gorge Bridge	⑥
Grammar School Rowing Club House	⑫
Hoblers Bridge	⑬
Harraps Wool Store	⑭
Henry St. Bridge	⑮
Mowbray Hill	⑯
Marine Hotel	⑰
No. 6 Beacon	⑱
No. 2 Beacon	⑲
No. 4 Beacon	⑳
North East Railway Line	㉑
Powder Jetty	㉒
Powder Dolphins	㉓
Queens Wharf	㉔
Royal Park	㉕
Ritchies Mill	㉖
Red Pile	㉗
Rifle Range Bridge	㉘
Riverview Hotel	㉙
Railway Bridge	㉚
Stephensons Bend	㉛
Sewage P.S.	㉜
Scottsdale Railway Line	㉑
Ti-Tree Bend	㉝
Tamar St. Bridge	㉞
Town Point	㉟
West Tamar Bank	㊱
Western Railway Line	㊲

Scale in 100 ft.

5 0 5 10 15 20

Date traced 8-6-1959

FIG. No. 1.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

DETAILS OF POTTS-DARE FLOOD MITIGATION PROPOSALS.

Date traced 10-6-1959

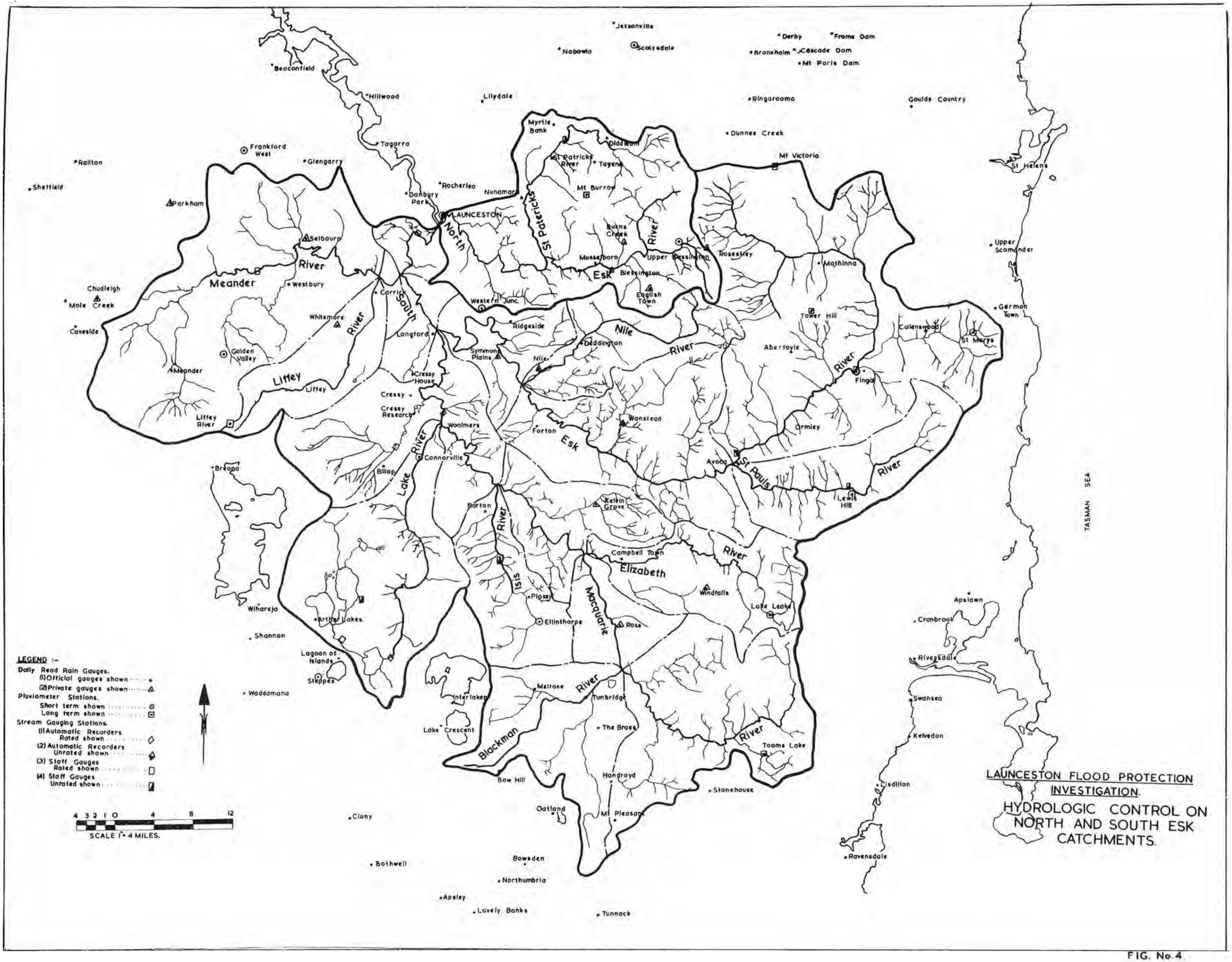
FIG. No. 2.

LAUNCESTON FLOOD PROTECTION INVESTIGATION.

BASE MAP FOR MODEL

Date traced 29·7·1959

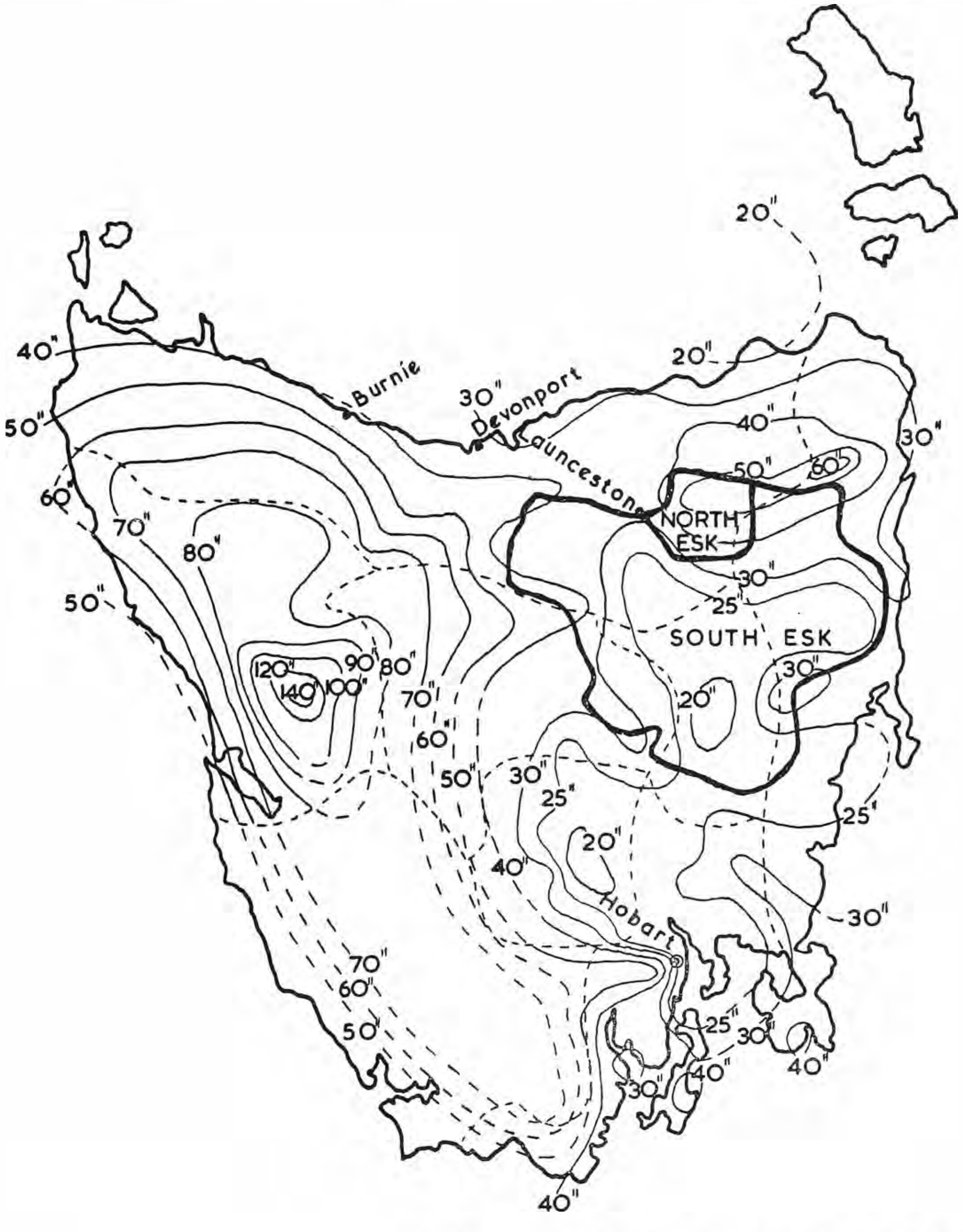
FIG. No. 3



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

HYDROLOGIC CONTROL ON NORTH AND SOUTH ESK CATCHMENTS.

FIG. No. 4



LAUNCESTON FLOOD PROTECTION INVESTIGATION

ANNUAL AVERAGE ISOHYETAL MAP OF TASMANIA

Scale 10 5 0 10 20 30 Miles

Date traced 24·6·1959

FIG. No.5.

LAUNCESTON FLOOD PROTECTION INVESTIGATION
RATING CURVE.
NORTH ESK RIVER AT CORRA LINN.

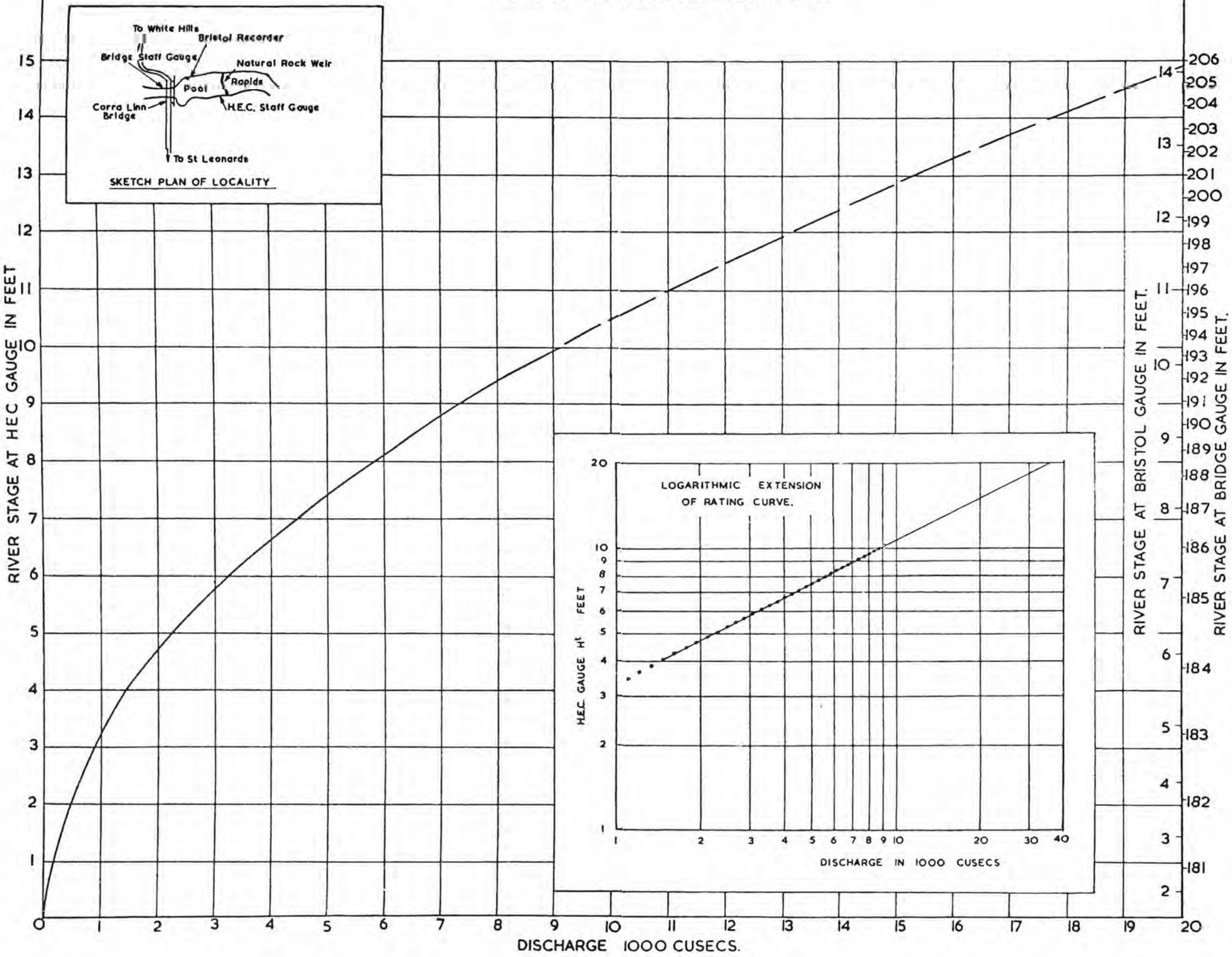
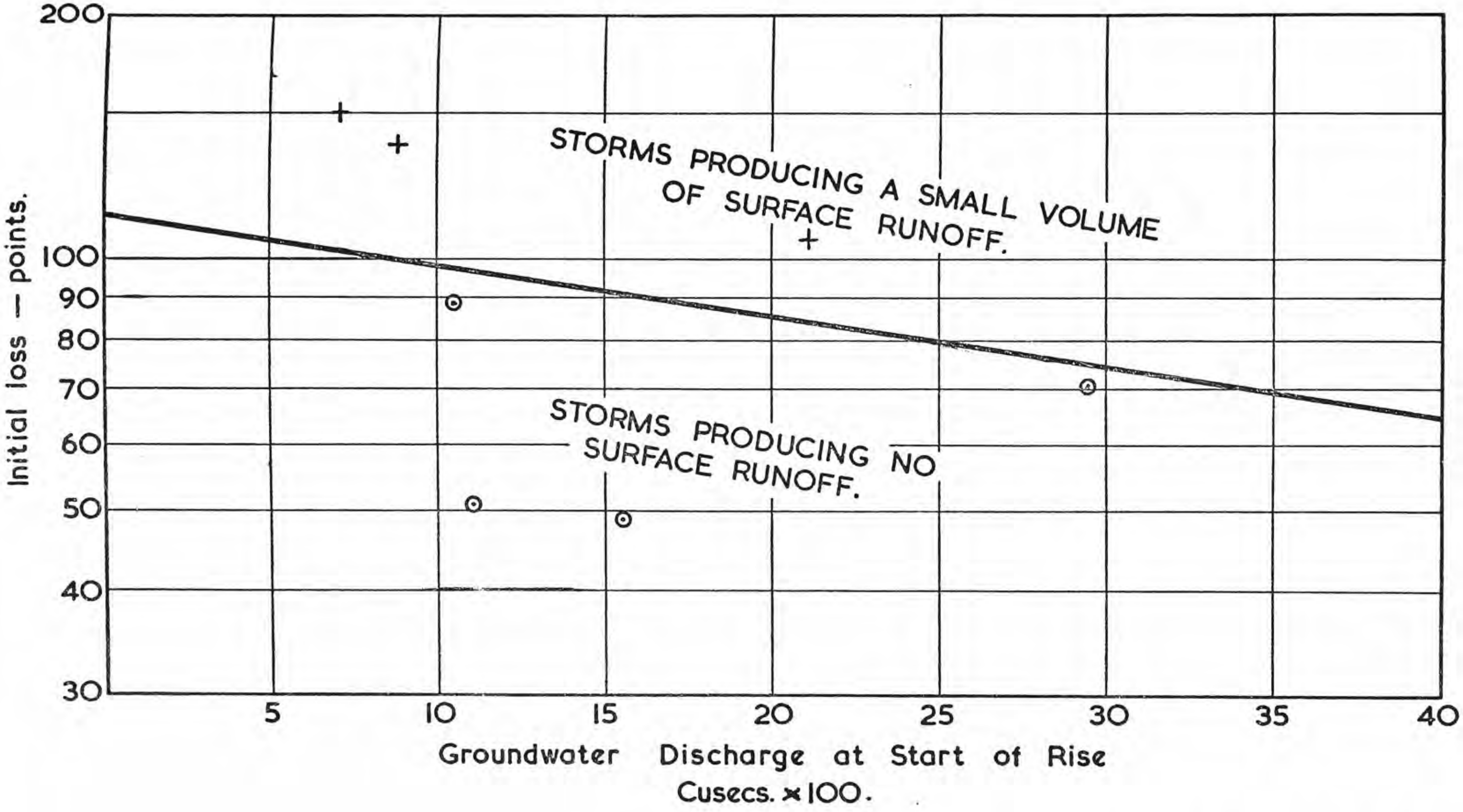


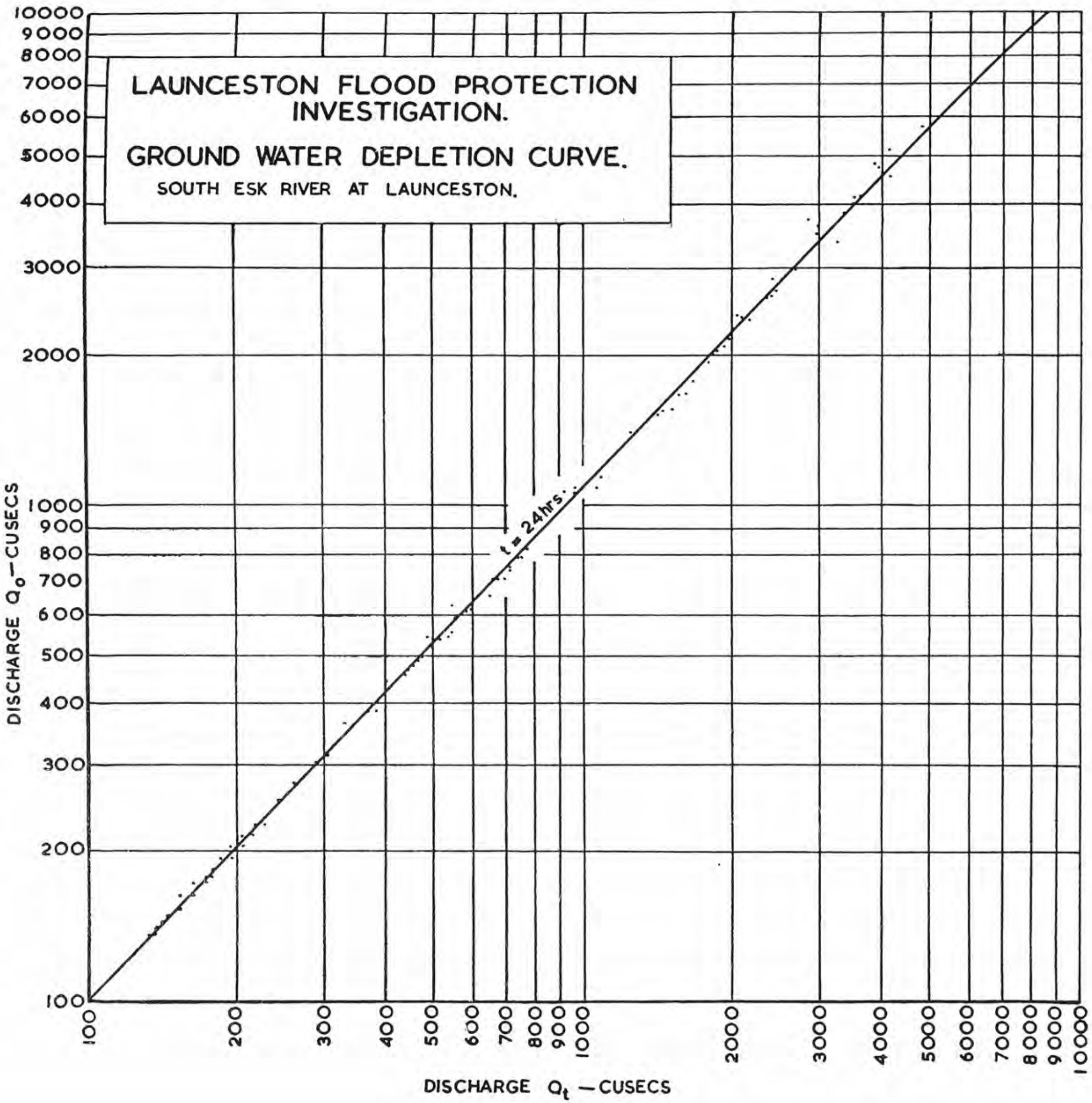
FIG. No. 6.



LAUNCESTON FLOOD PROTECTION
INVESTIGATION

RELATION BETWEEN INITIAL LOSS AND GROUNDWATER FLOW.
S.ESK.

FIG. No. 7.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

GROUND WATER DEPLETION CURVE.

SOUTH ESK RIVER AT LAUNCESTON.

FIG. No. 8.

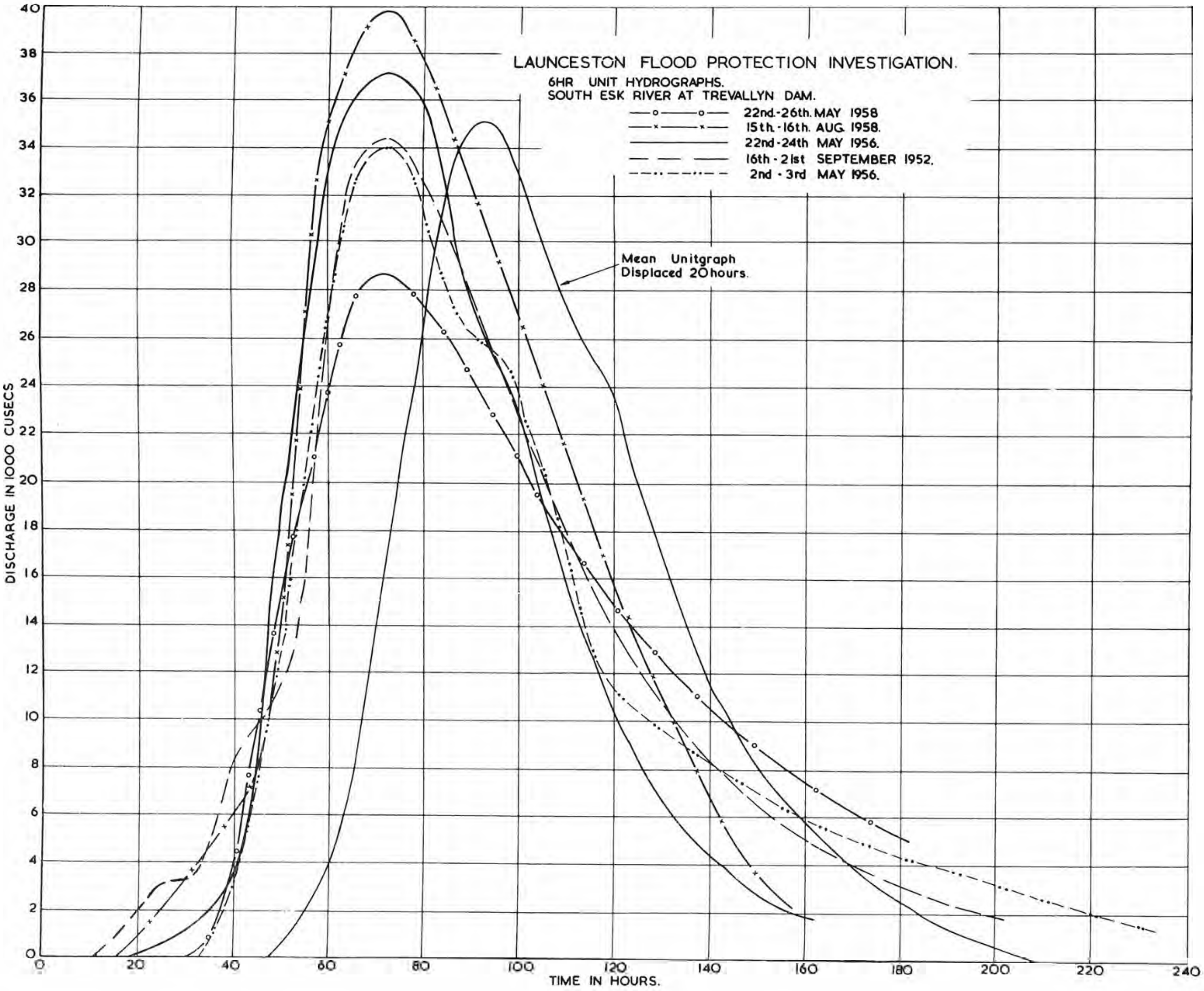
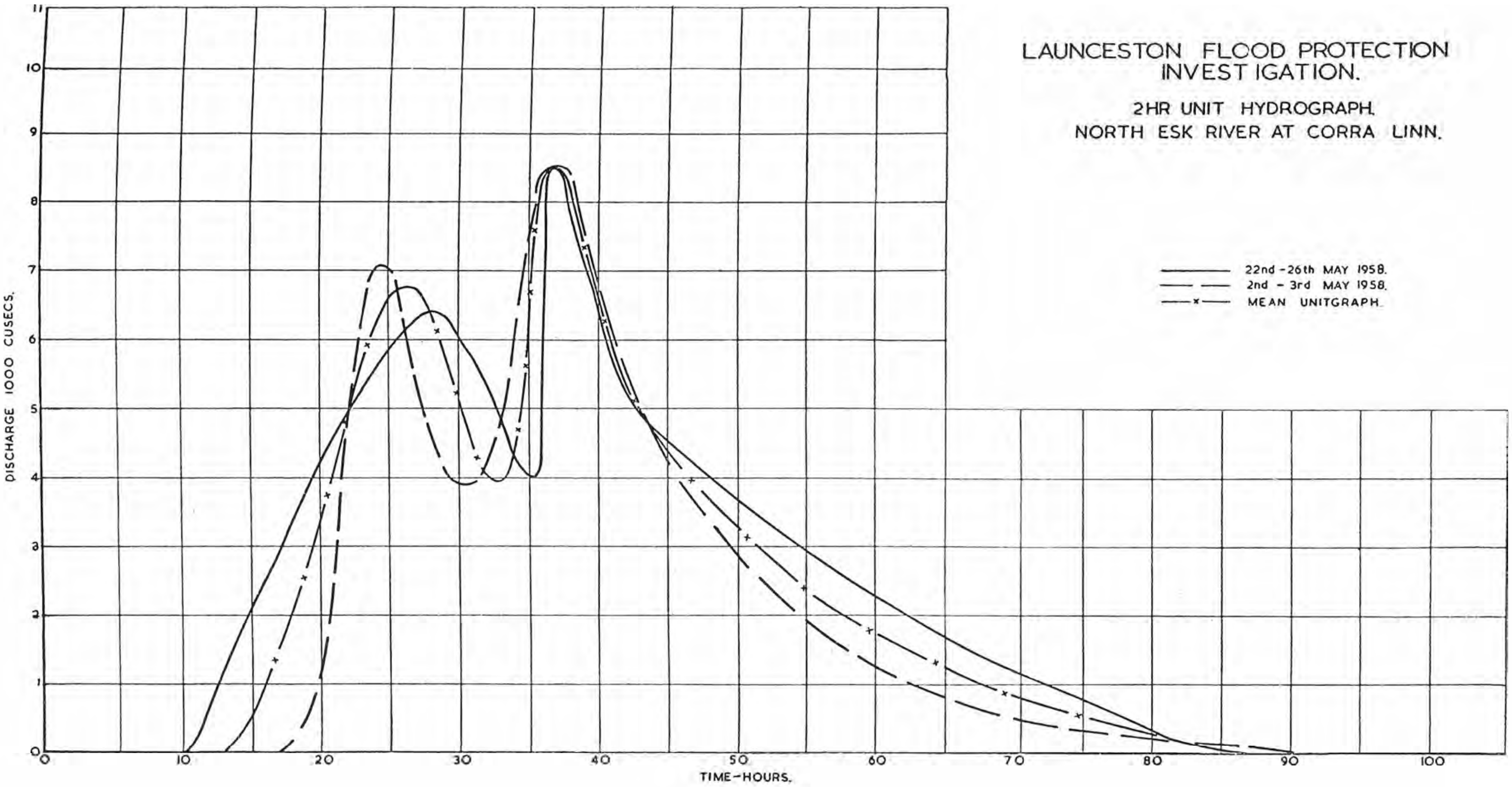

LAUNCESTON FLOOD PROTECTION INVESTIGATION.
6HR UNIT HYDROGRAPHS.
SOUTH ESK RIVER AT TREVALLYN DAM.
—o——o— 22nd-26th. MAY 1958
—×——×— 15th.-16th. AUG. 1958.
——— 22nd-24th MAY 1956.
— — — 16th-21st SEPTEMBER 1952.
-··-··- 2nd - 3rd MAY 1956.
Mean Unitgraph Displaced 20 hours.
DISCHARGE IN 1000 CUSECS
0
2
4
6
8
10
12
14
16
18
20
22
24
26
28
30
32
34
36
38
40
TIME IN HOURS.
0
20
40
60
80
100
120
140
160
180
200
220
240

FIG. No. 9.

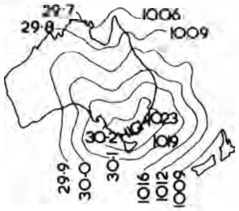


LAUNCESTON FLOOD PROTECTION INVESTIGATION.

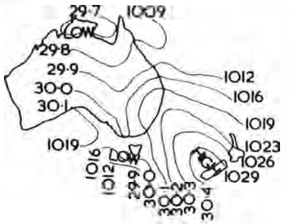
2HR UNIT HYDROGRAPH.
NORTH ESK RIVER AT CORRA LINN.

FIG. No. 10.

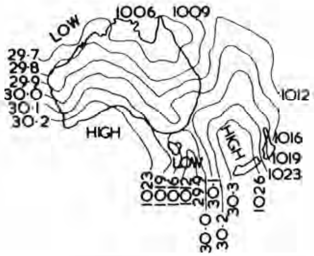
9 A.M. 30-3-1929



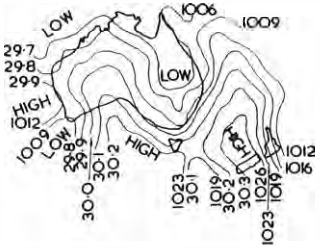
9 A.M. 1-4-1929



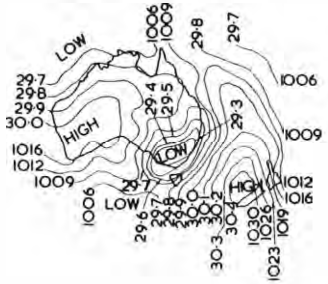
9 A.M. 2-4-1929



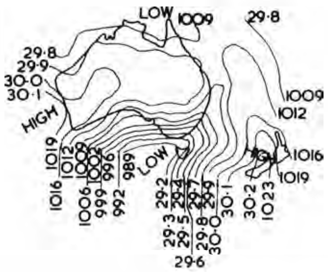
9 A.M. 3-4-1929



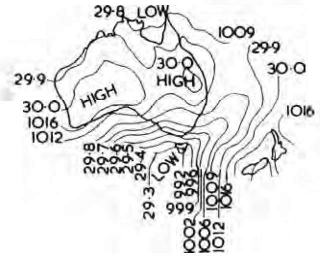
9 A.M. 4-4-1929



9 A.M. 5-4-1929



9 A.M. 6-4-1929

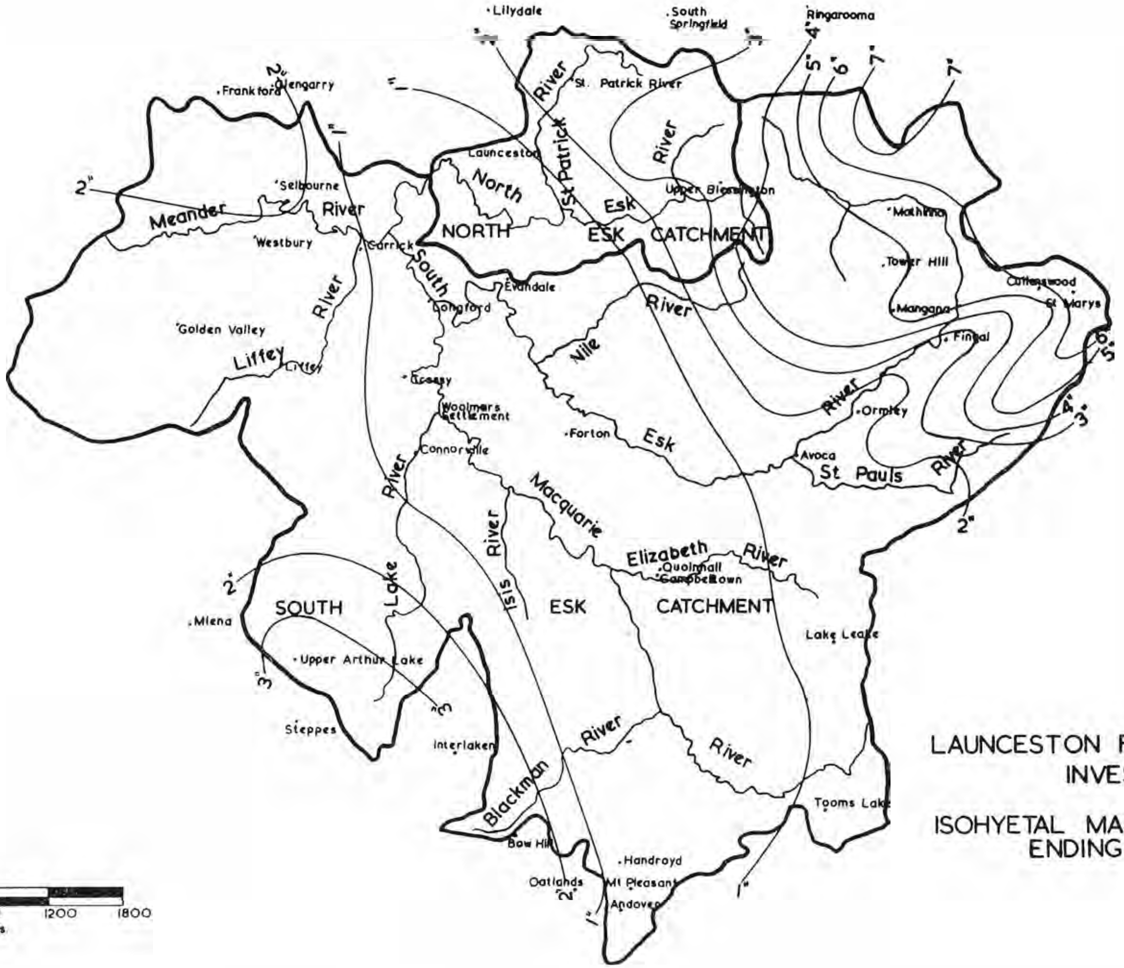


LAUNCESTON FLOOD PROTECTION INVESTIGATION

ISOBARIC MAPS AT 0900 HOURS FOR FLOOD OF APRIL 1929.

Date traced 11-6-1959

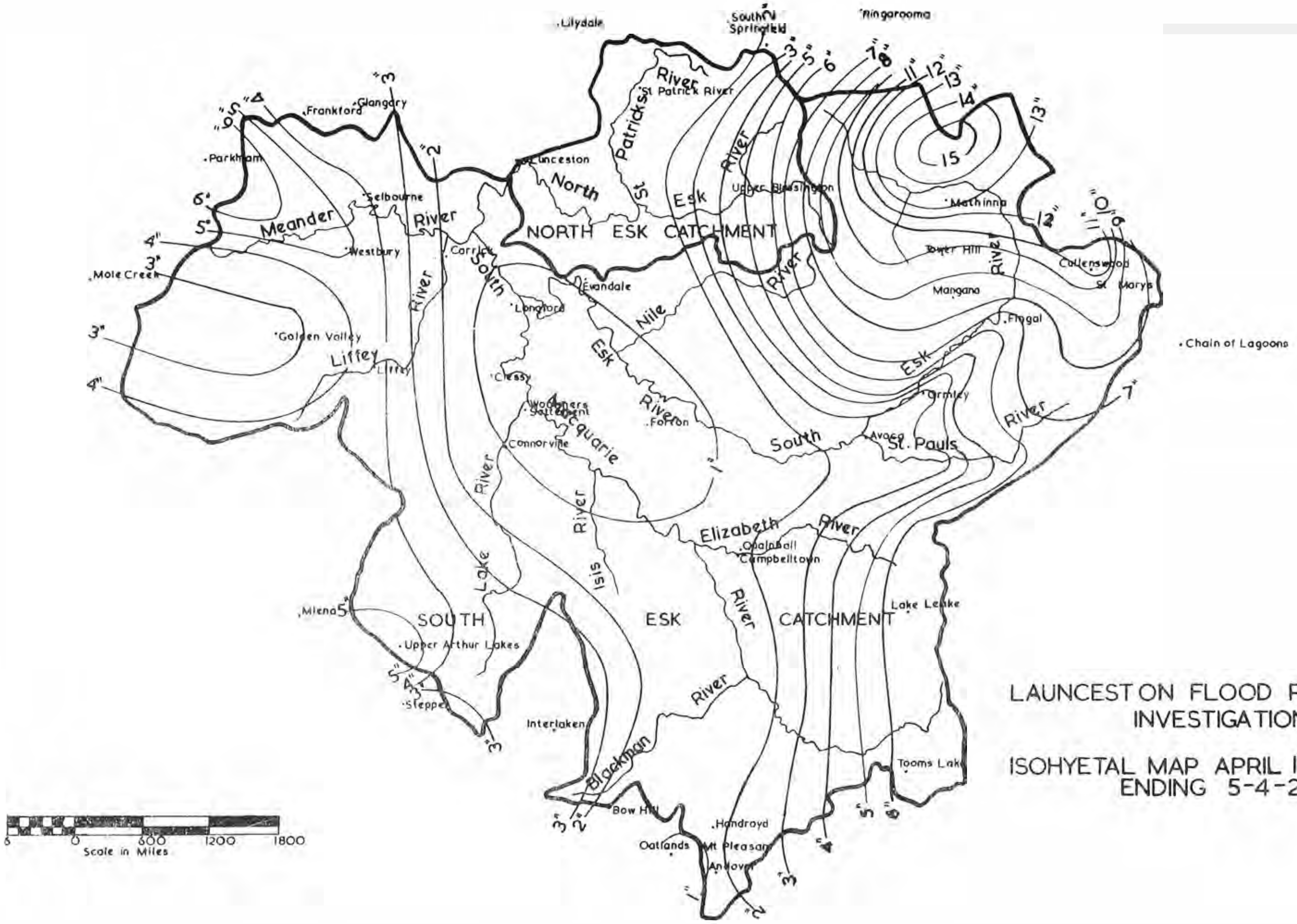
FIG. No. 11.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

ISOHYETAL MAP APRIL 1929. - 24 HRS.
ENDING 4-4-29.

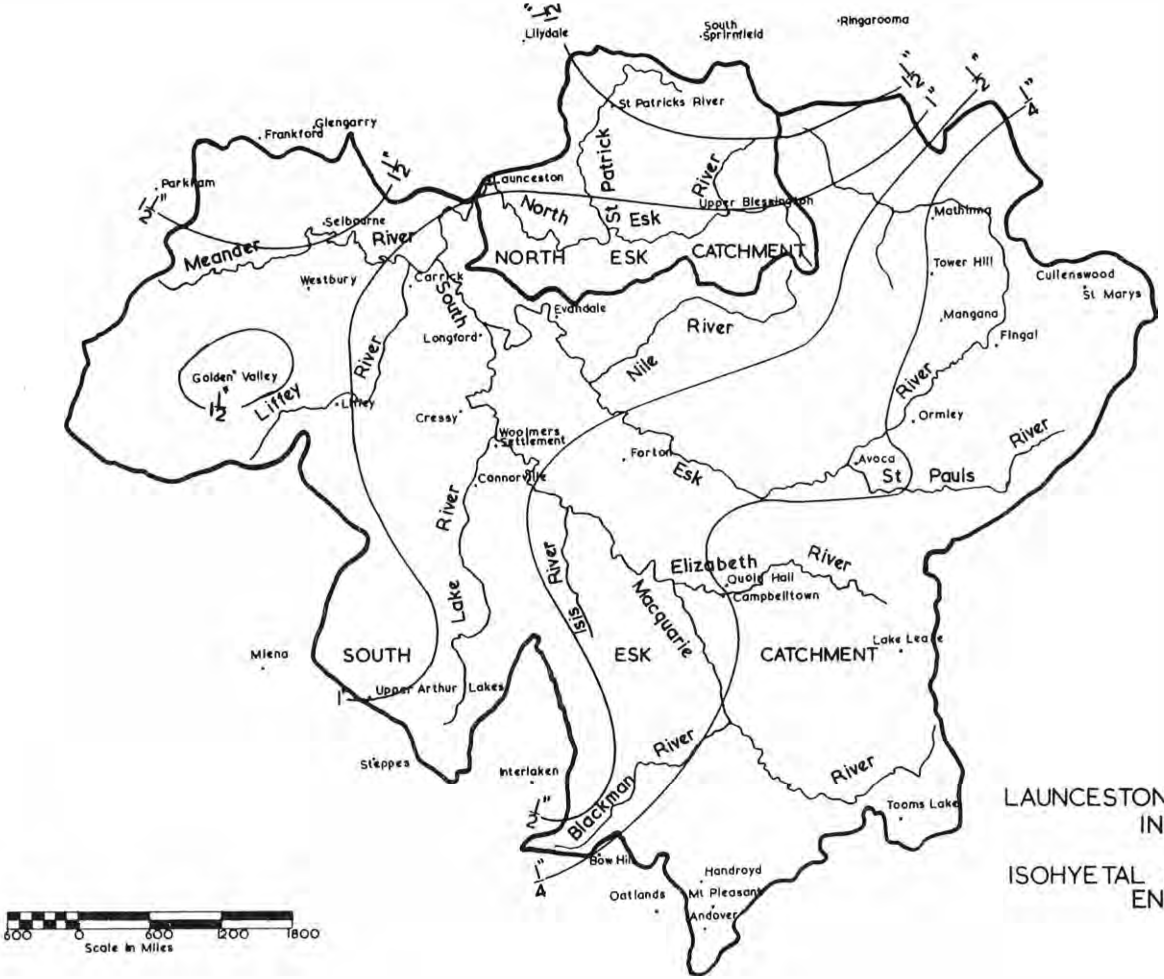
FIG No. 12.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

ISOHYETAL MAP APRIL 1929 - 24 HRS. ENDING 5-4-29.

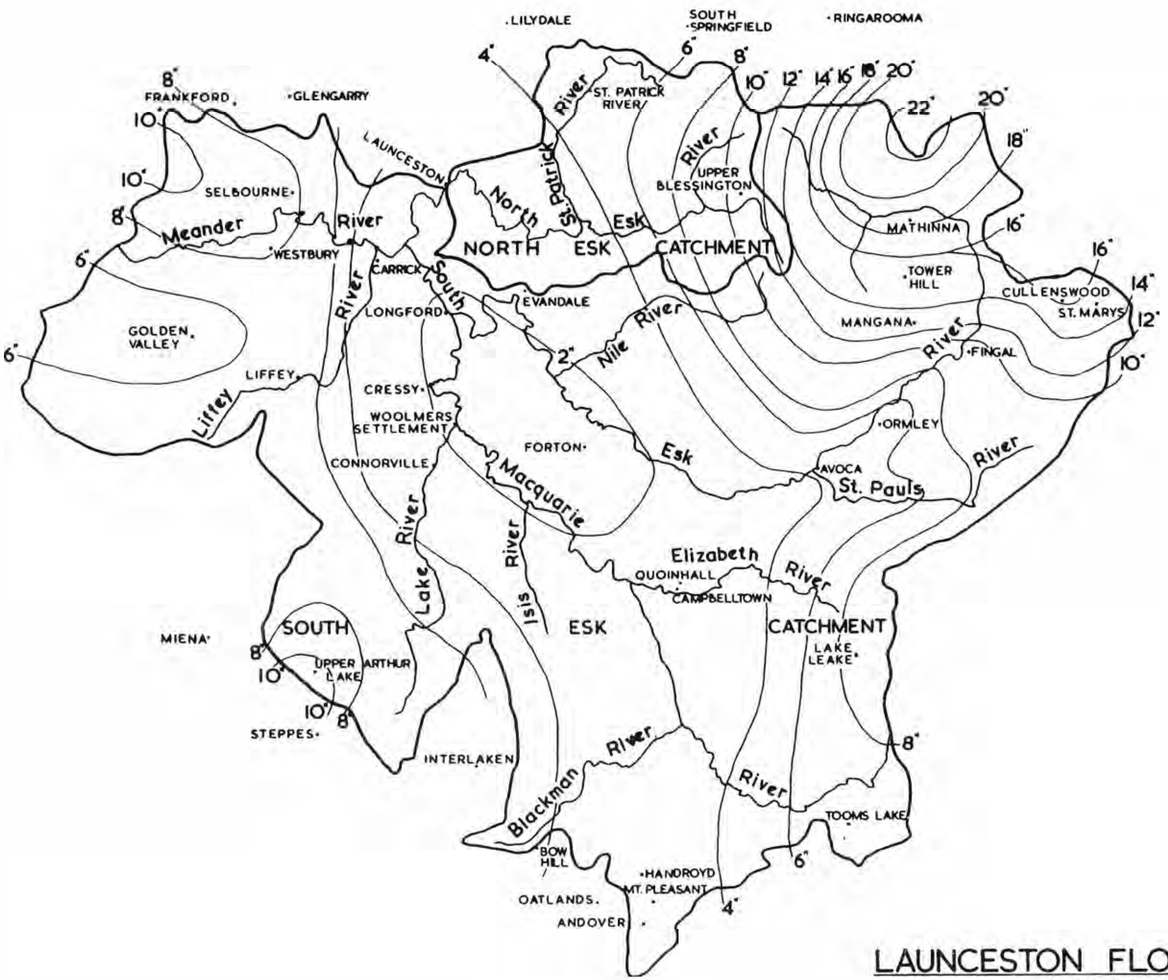
FIG. No. 13.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

ISOHYETAL MAP APRIL 1929 - 24 HRS ENDING 6-4-29.

FIG. No. 14.



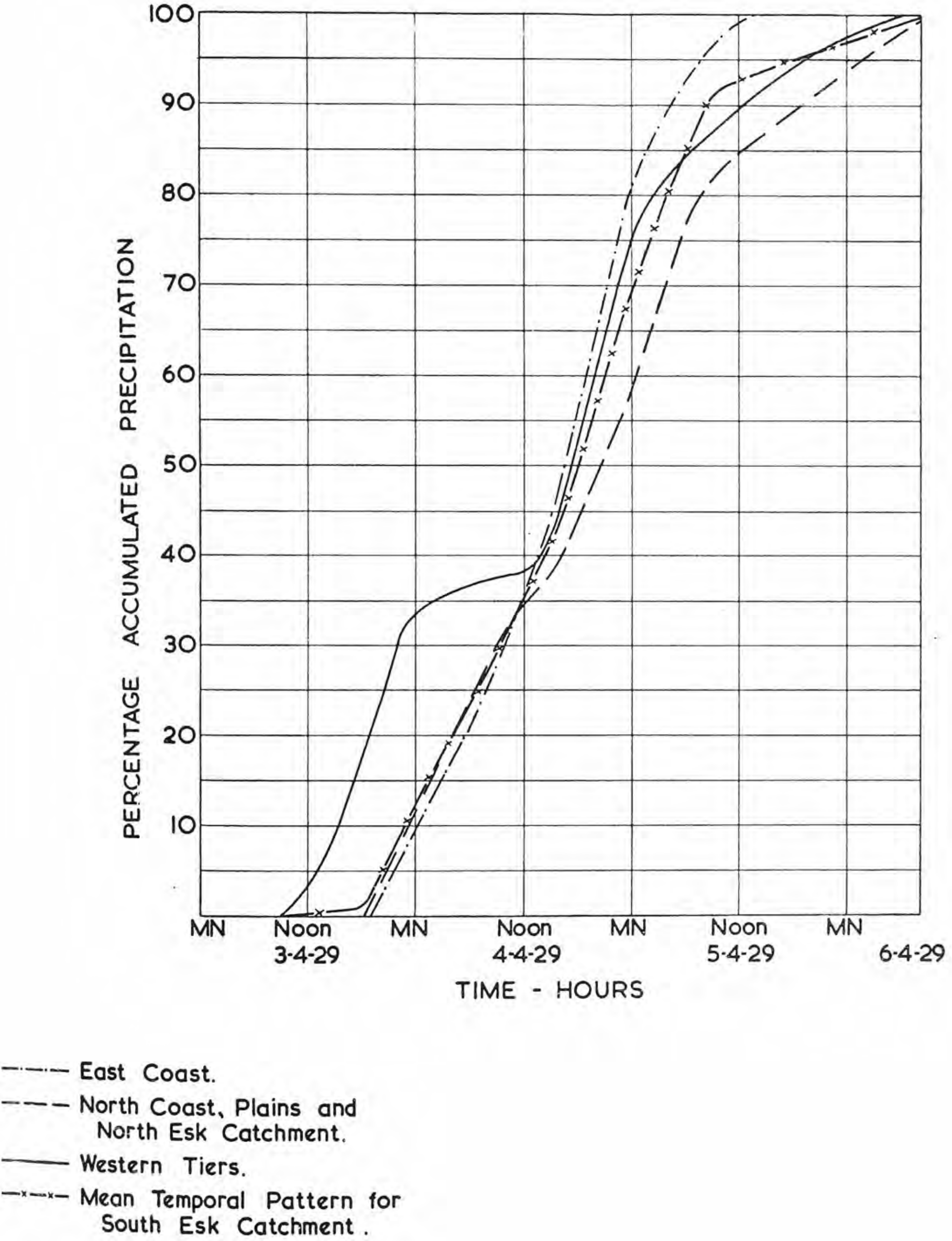
LAUNCESTON FLOOD PROTECTION INVESTIGATION

TOTAL STORM ISOHYETAL MAP APRIL 1929
72 HOURS ENDING 0900 HOURS 6-4-29

600 0 600 1200 1800
SCALE IN MILES

Date traced 6·7·1959

FIG. No. 15.



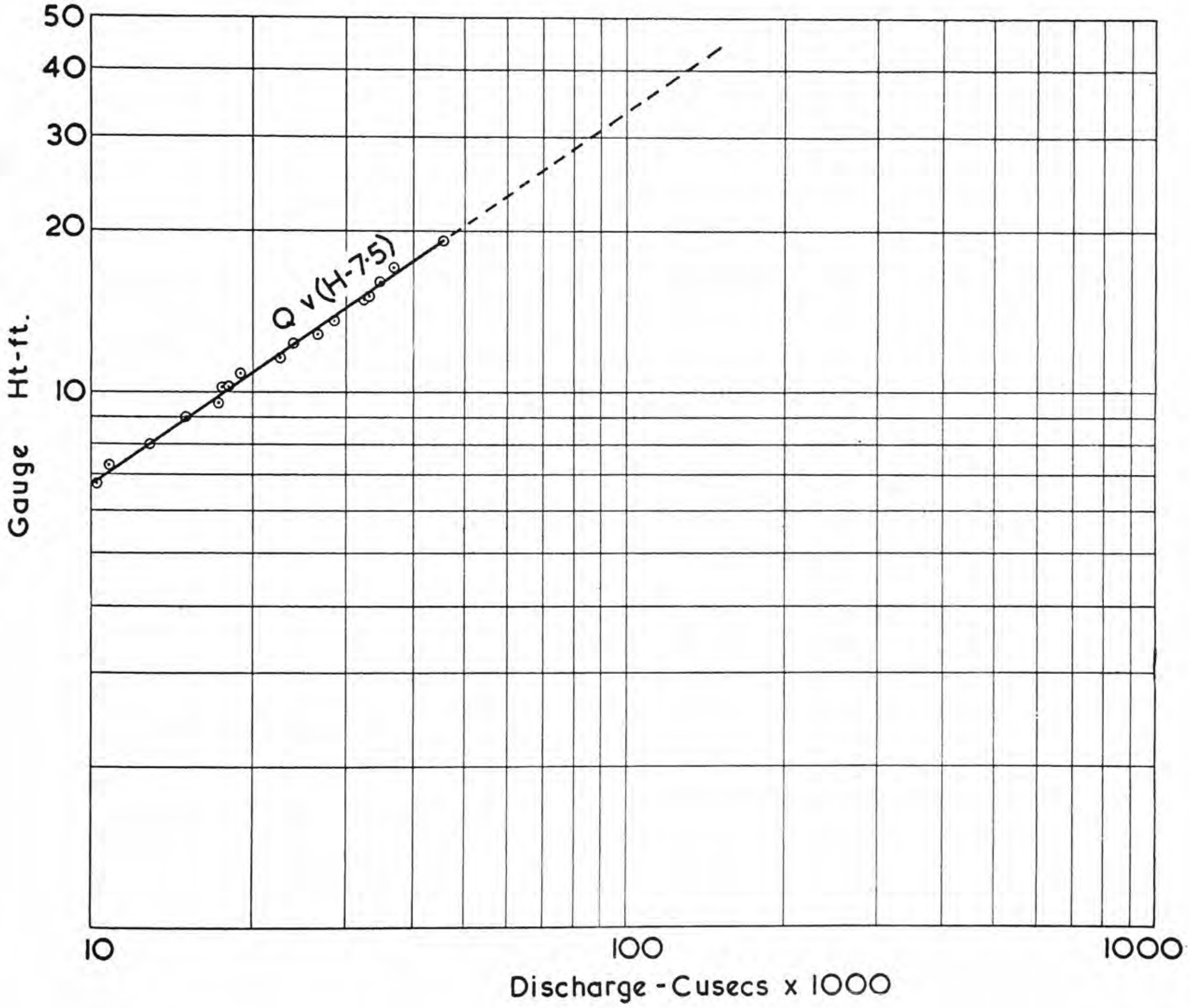
LAUNCESTON FLOOD PROTECTION INVESTIGATION.

SYNTHESIZED MASS CURVES

APRIL 1929.
South Esk Catchment.

Date traced 17-6-1959

FIG. No. 16.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

EXTRAPOLATION OF RATING CURVE

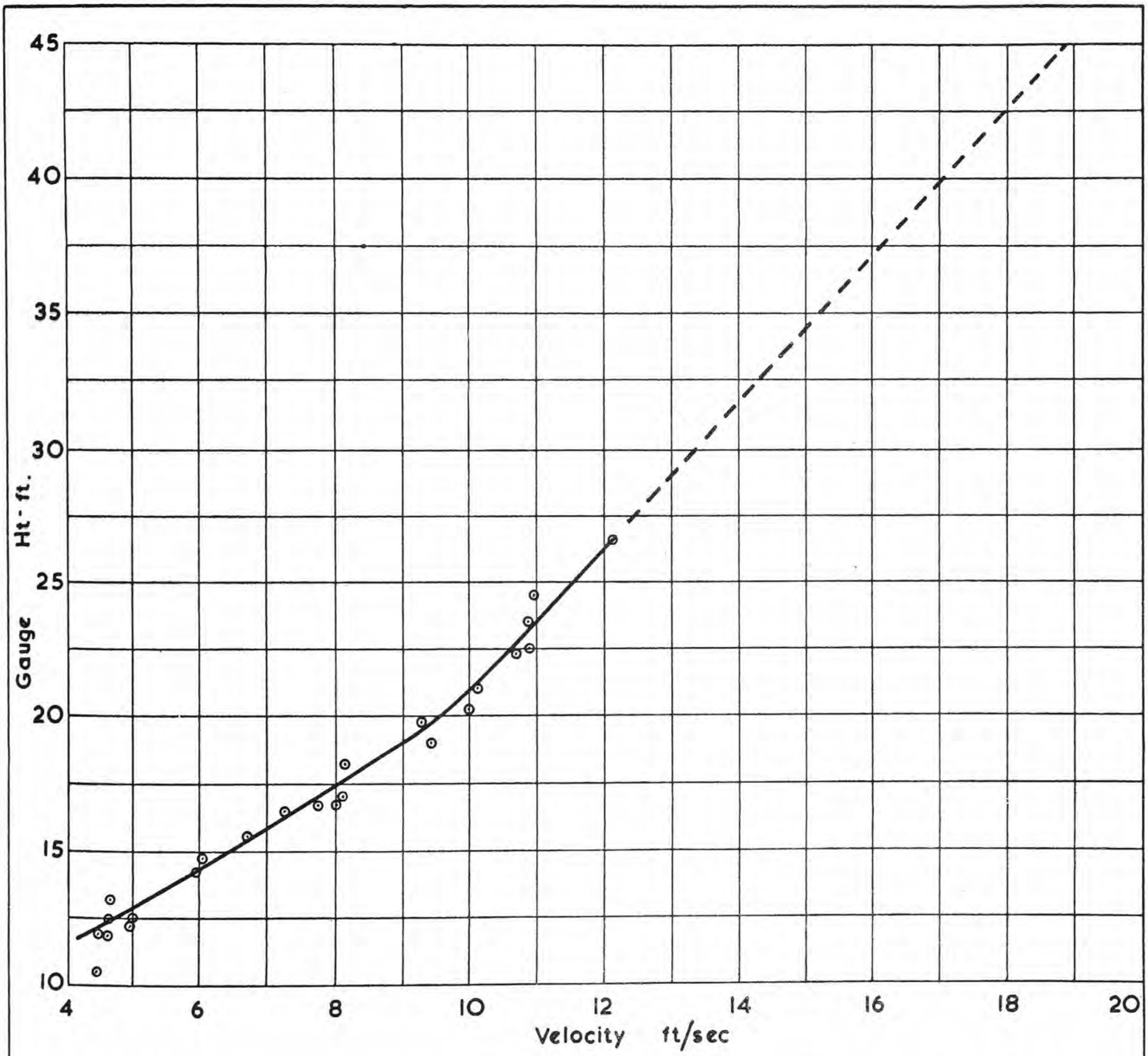
Duck Reach Power Station

LOGARITHMIC METHOD

(True Section)

Date traced 25·6·1959

FIG. No. 17.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

EXTRAPOLATION OF RATING CURVE

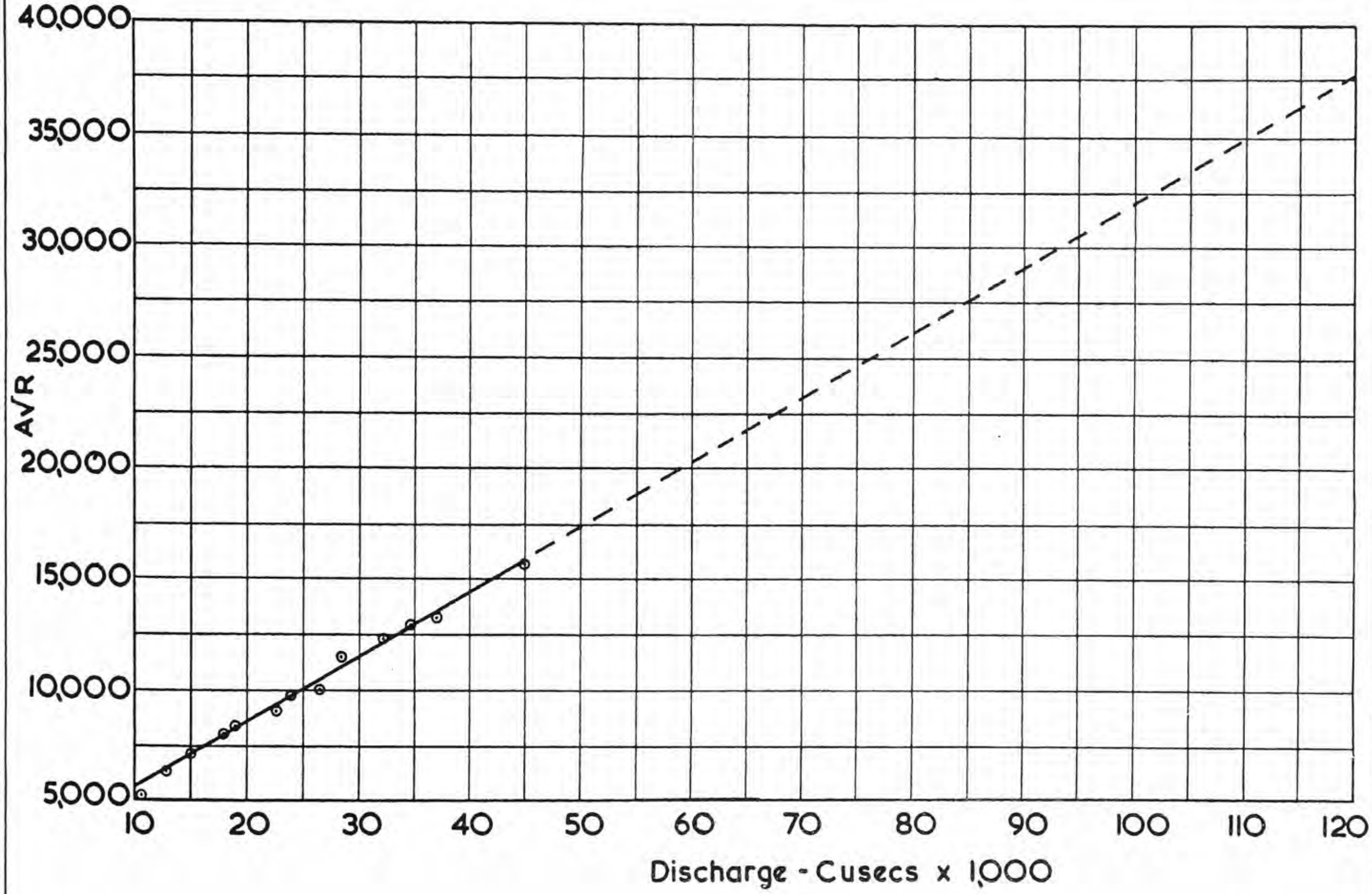
Duck Reach Power Station

AREA VELOCITY METHOD

(True Section)

Date traced 24·6·1959

FIG. No. 18



LAUNCESTON FLOOD PROTECTION INVESTIGATION

EXTRAPOLATION OF RATING CURVE

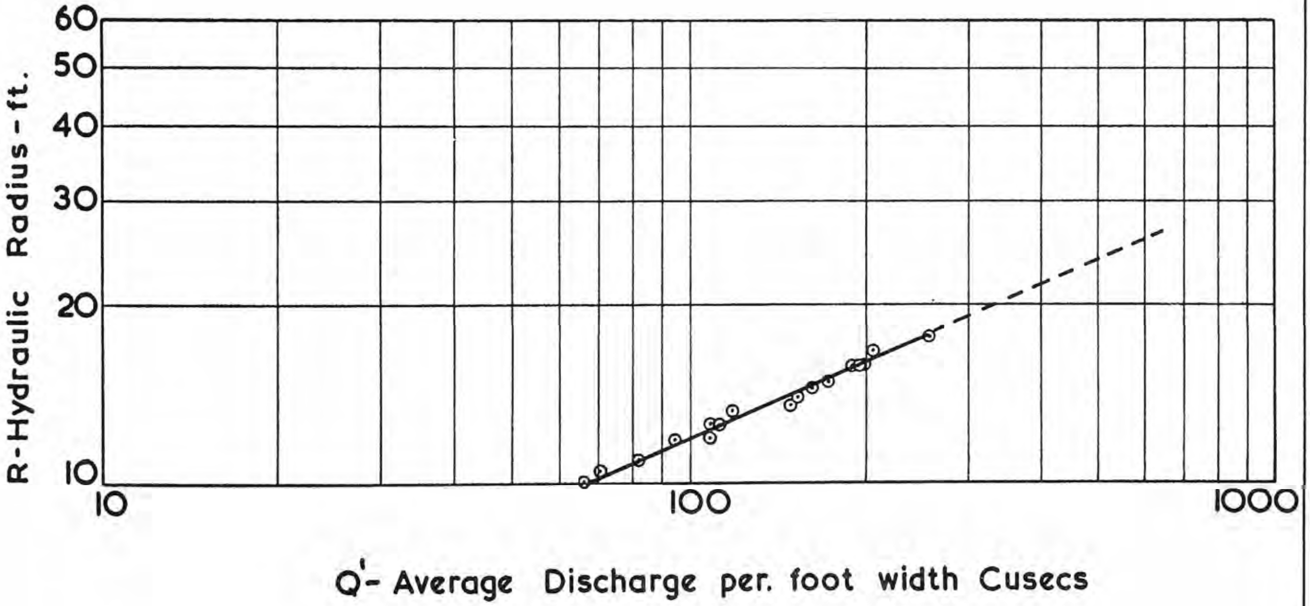
Duck Reach Power Station

STEPHENS METHOD

(True Section)

Date traced 25·6·1959

FIG. No. 19.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

EXTRAPOLATION OF RATING CURVE

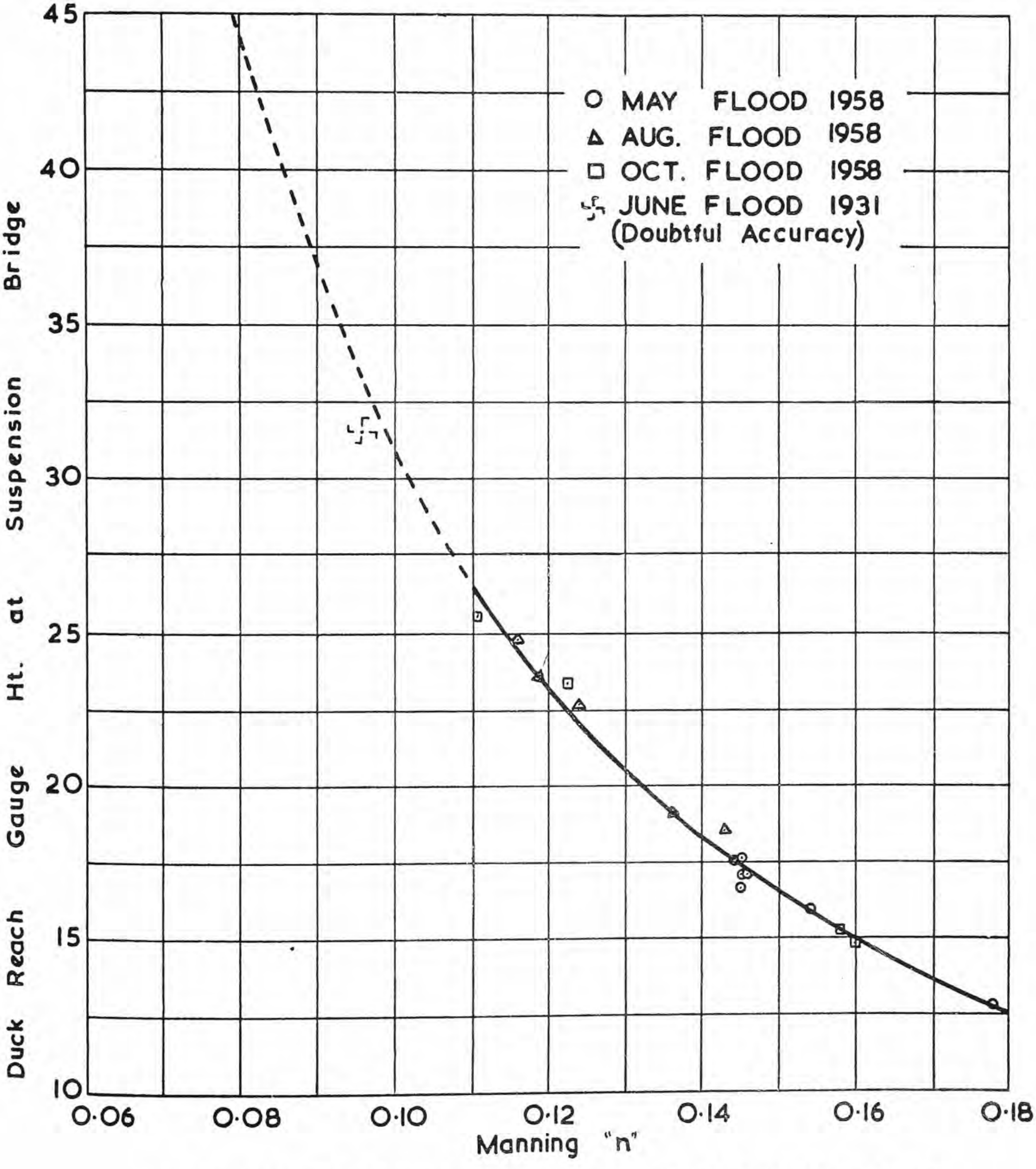
Duck Reach Power Station

R-Q' METHOD

(True Section)

Date traced 25·6·1959

FIG. No.20.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

VARIATION OF MANNINGS "n" WITH GAUGE HEIGHT AT DUCK REACH

Date traced 23·6·1959

FIG. No. 21.

LAUNCESTON FLOOD PROTECTION INVESTIGATION.

APRIL 1929 FLOOD HYDROGRAPH

SOUTH ESK RIVER AT DUCK REACH.
RAINFALL INTENSITY – LOSS RATE.– UNITGRAPH METHOD OF ESTIMATION.

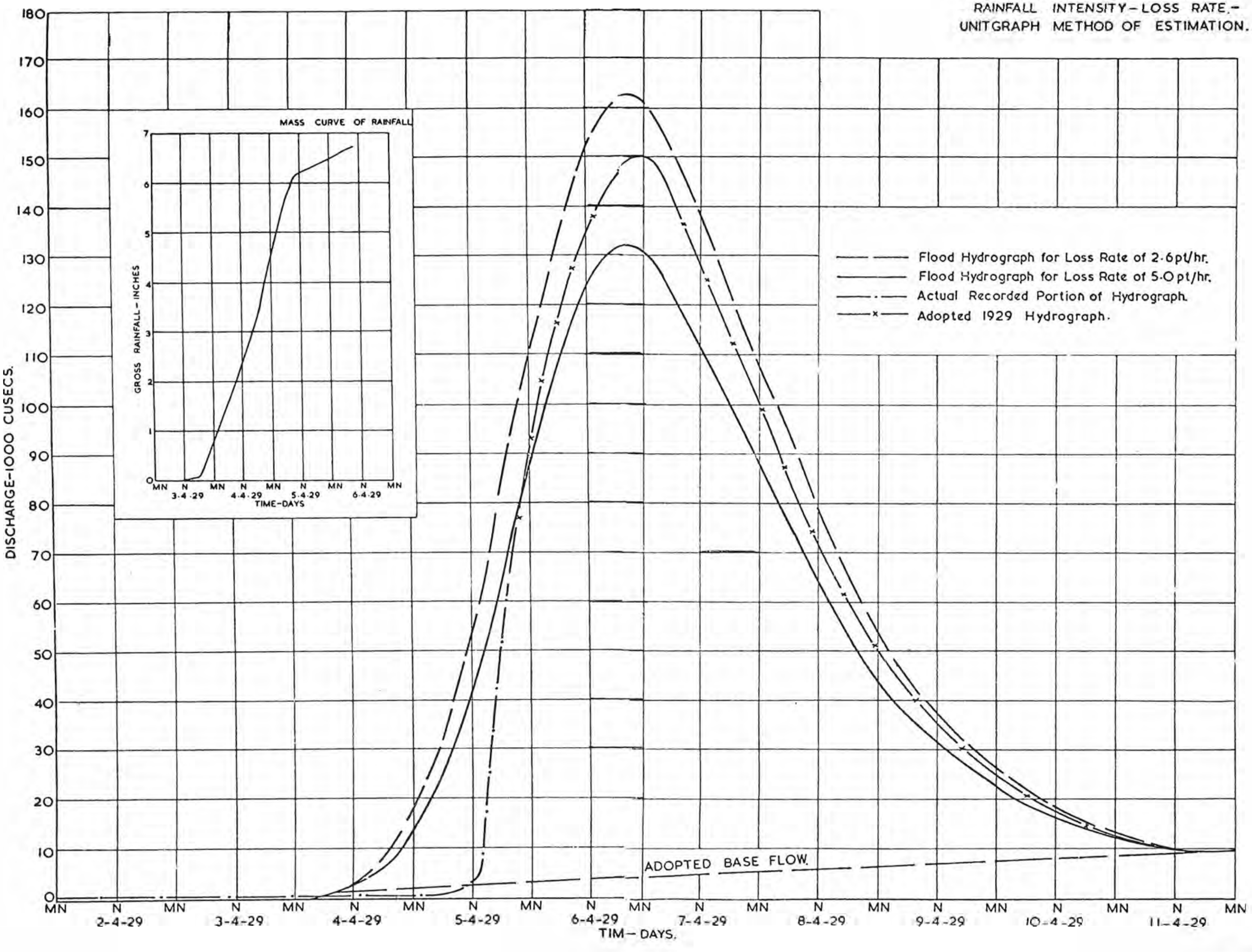
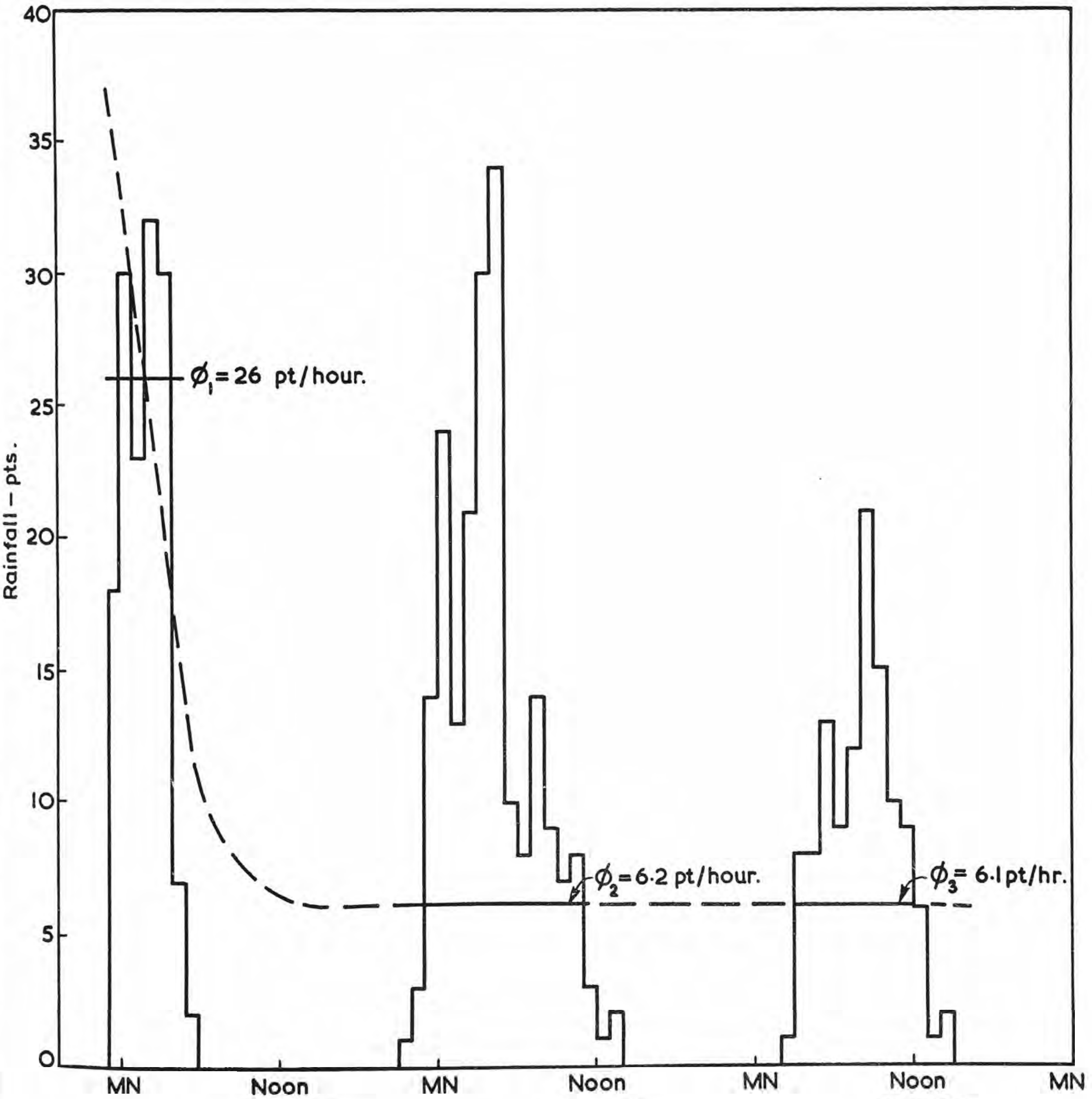


FIG. No. 22.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

VARIATION OF LOSS RATE WITH STORM DURATION.
North Esk Catchment. May, 1958.

8·7·58.

FIG. No. 23.

LAUNCESTON FLOOD PROTECTION INVESTIGATION.
APRIL 1929 FLOOD HYDROGRAPH.
North Esk River.
RAINFALL INTENSITY LOSS RATE.
UNITGRAPH METHOD OF ESTIMATION.

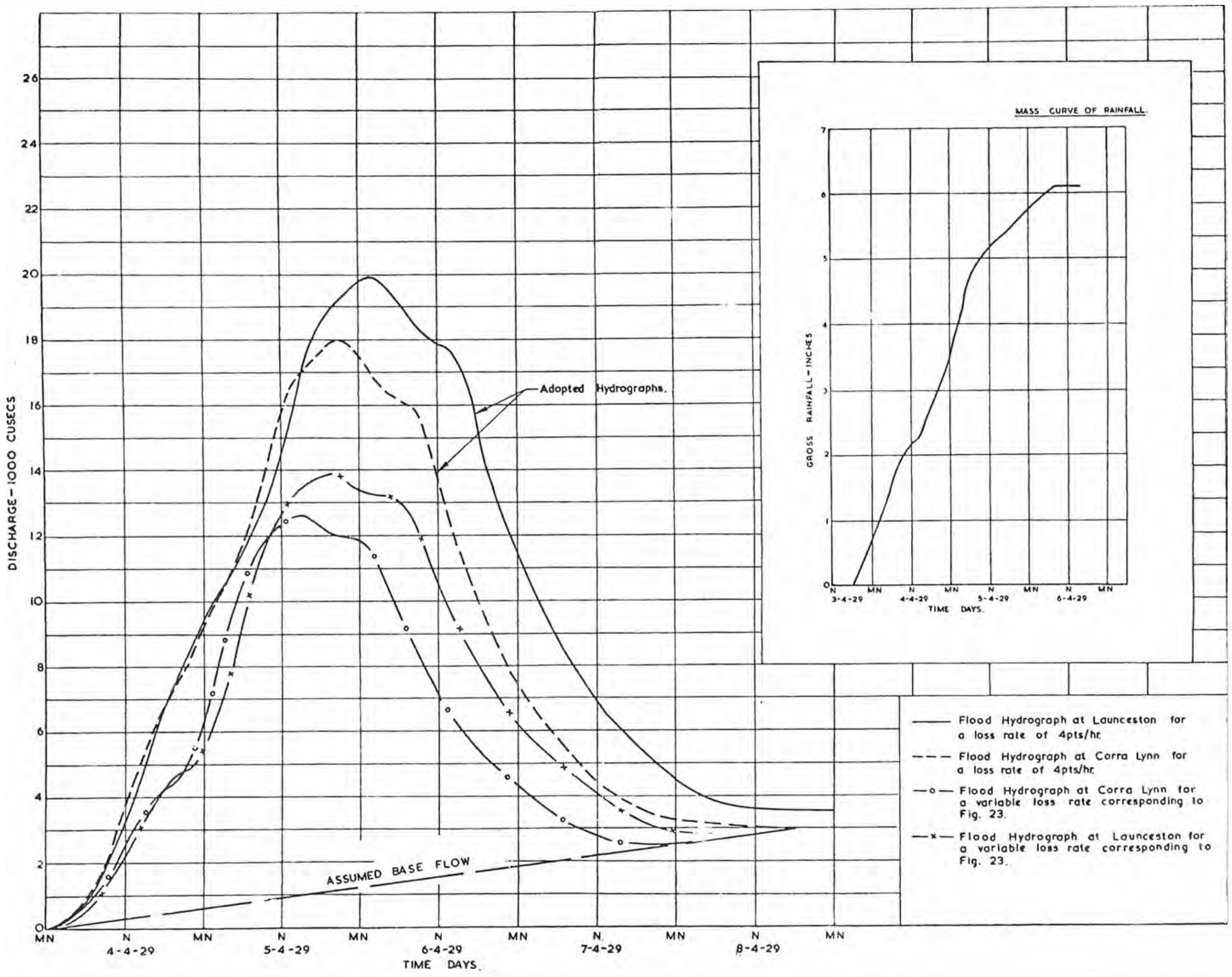
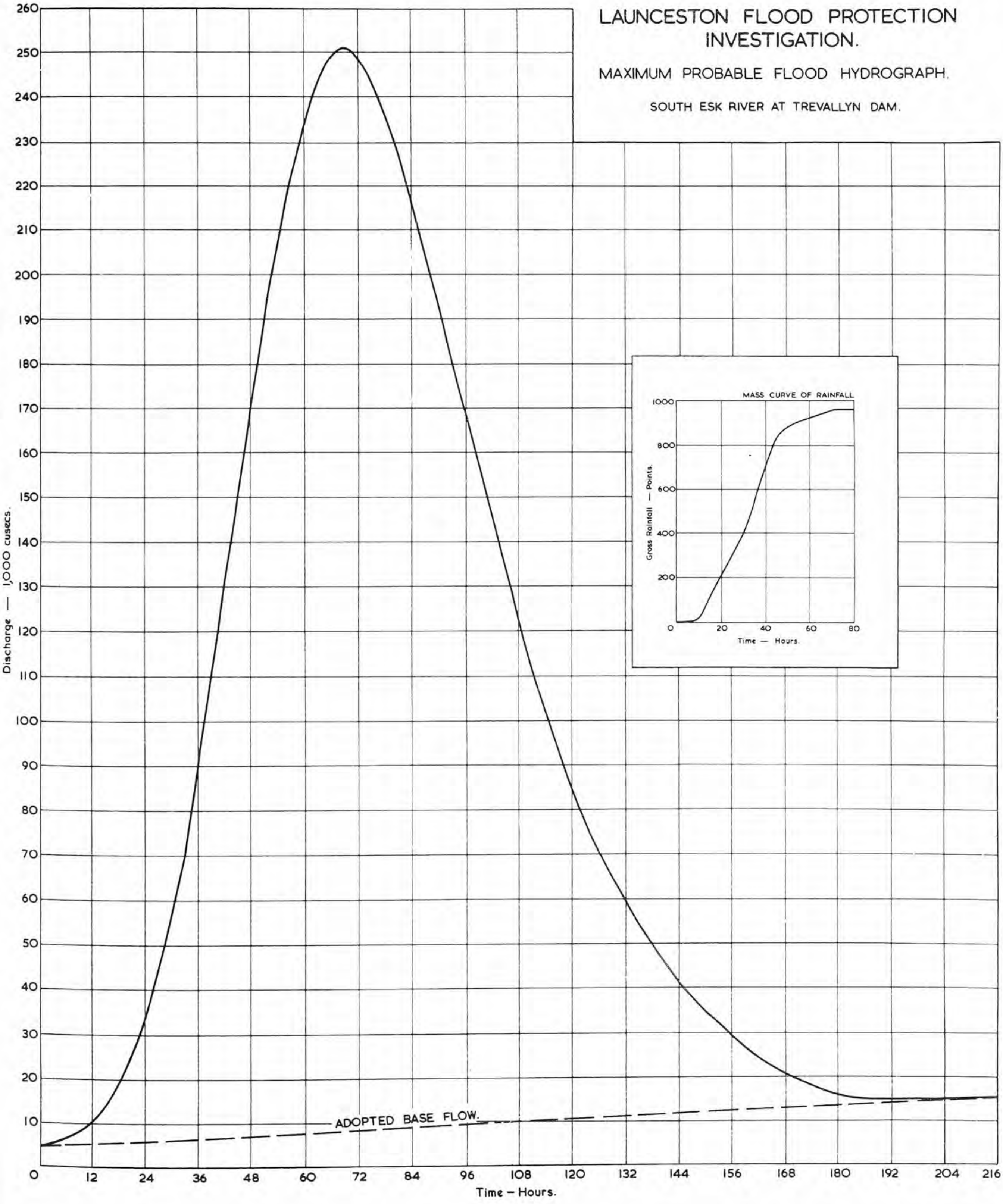


FIG No. 24.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

MAXIMUM PROBABLE FLOOD HYDROGRAPH.

SOUTH ESK RIVER AT TREVALLYN DAM.

FIG. No. 25.

LAUNCESTON FLOOD PROTECTION INVESTIGATION
MAXIMUM PROBABLE FLOOD HYDROGRAPH
North Esk River

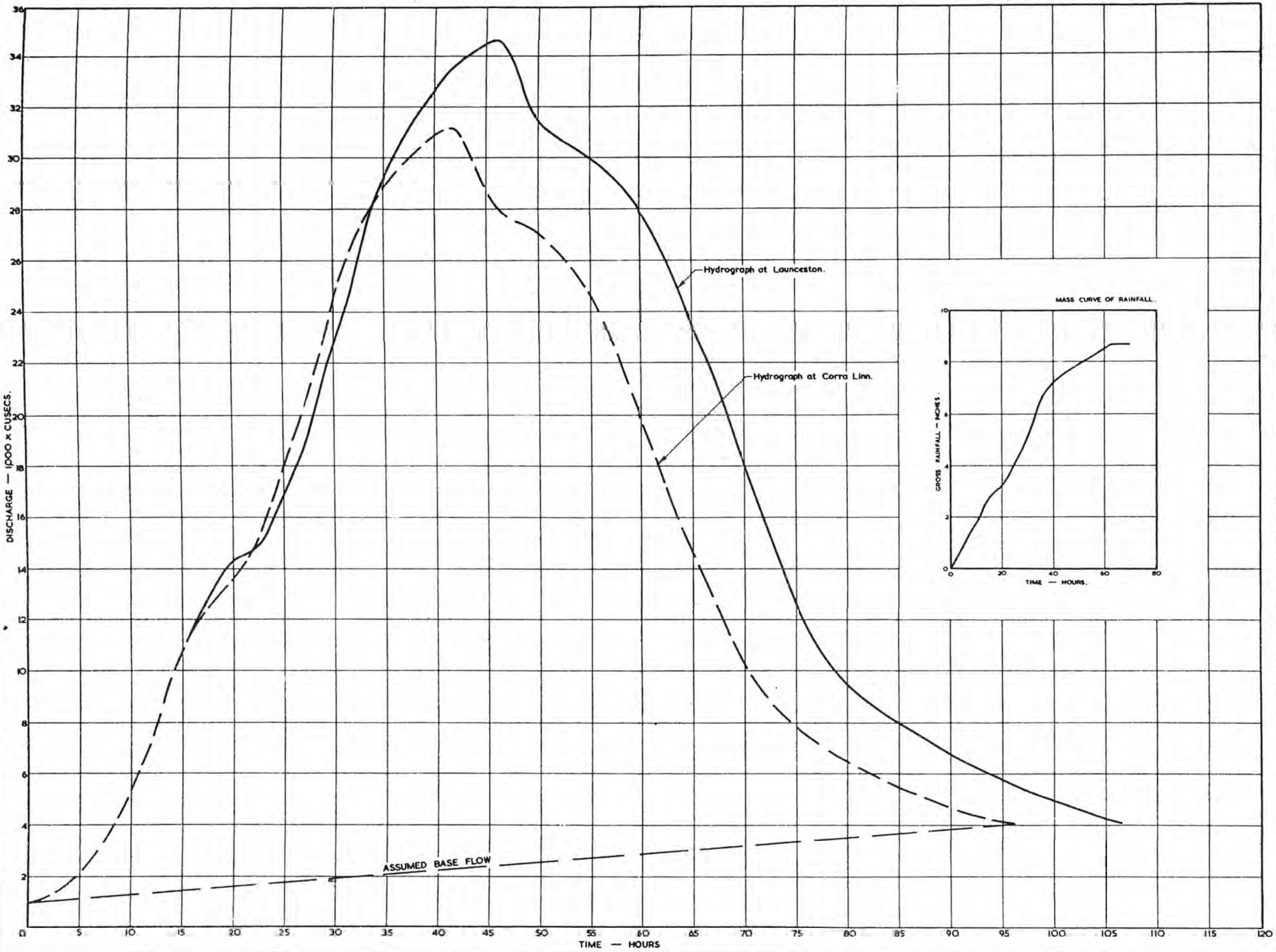
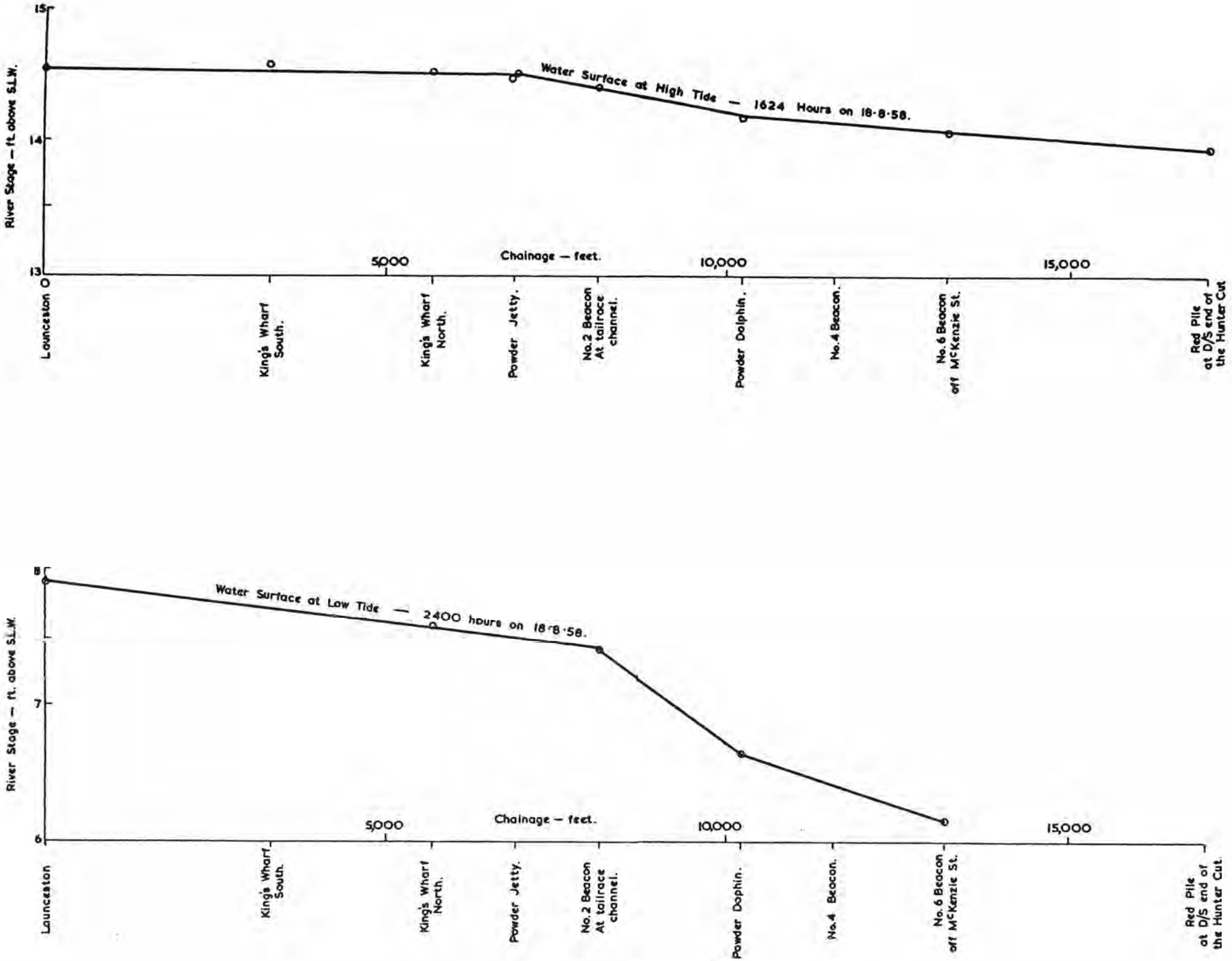


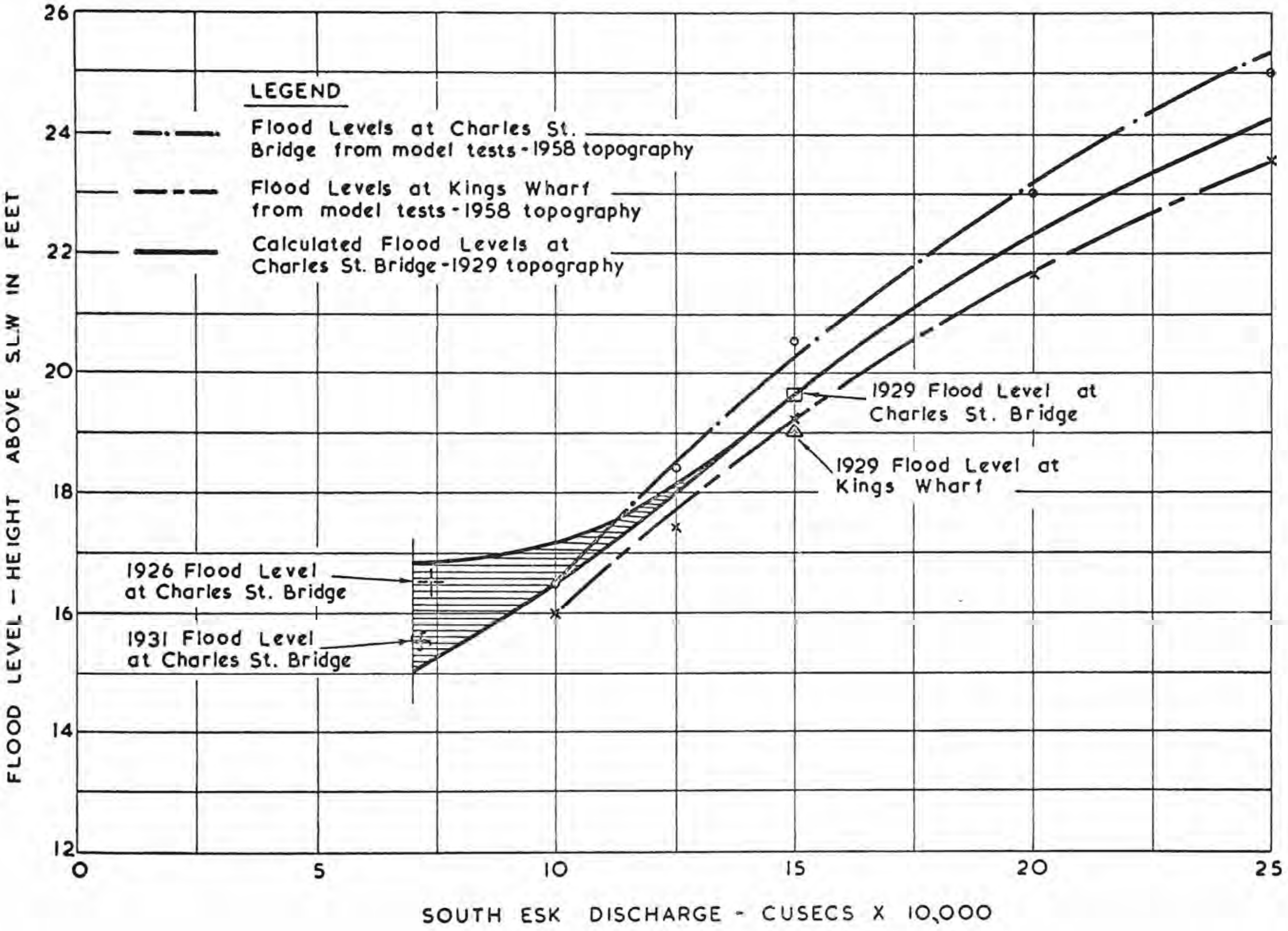
FIG. No. 26.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

Tamar River Surface Slopes for Flood of August, 1958.

FIG. No. 27.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

VARIATION OF FLOOD LEVELS WITH DISCHARGE & TIDE.

CHARLES ST. BRIDGE - 1929 TOPOGRAPHY.

Date traced 10·7·1959

FIG No. 28.

LAUNCESTON FLOOD PROTECTION AUTHORITY.

SOUTH ESK FLOODS EXCEEDING 70,000 CUSECS.

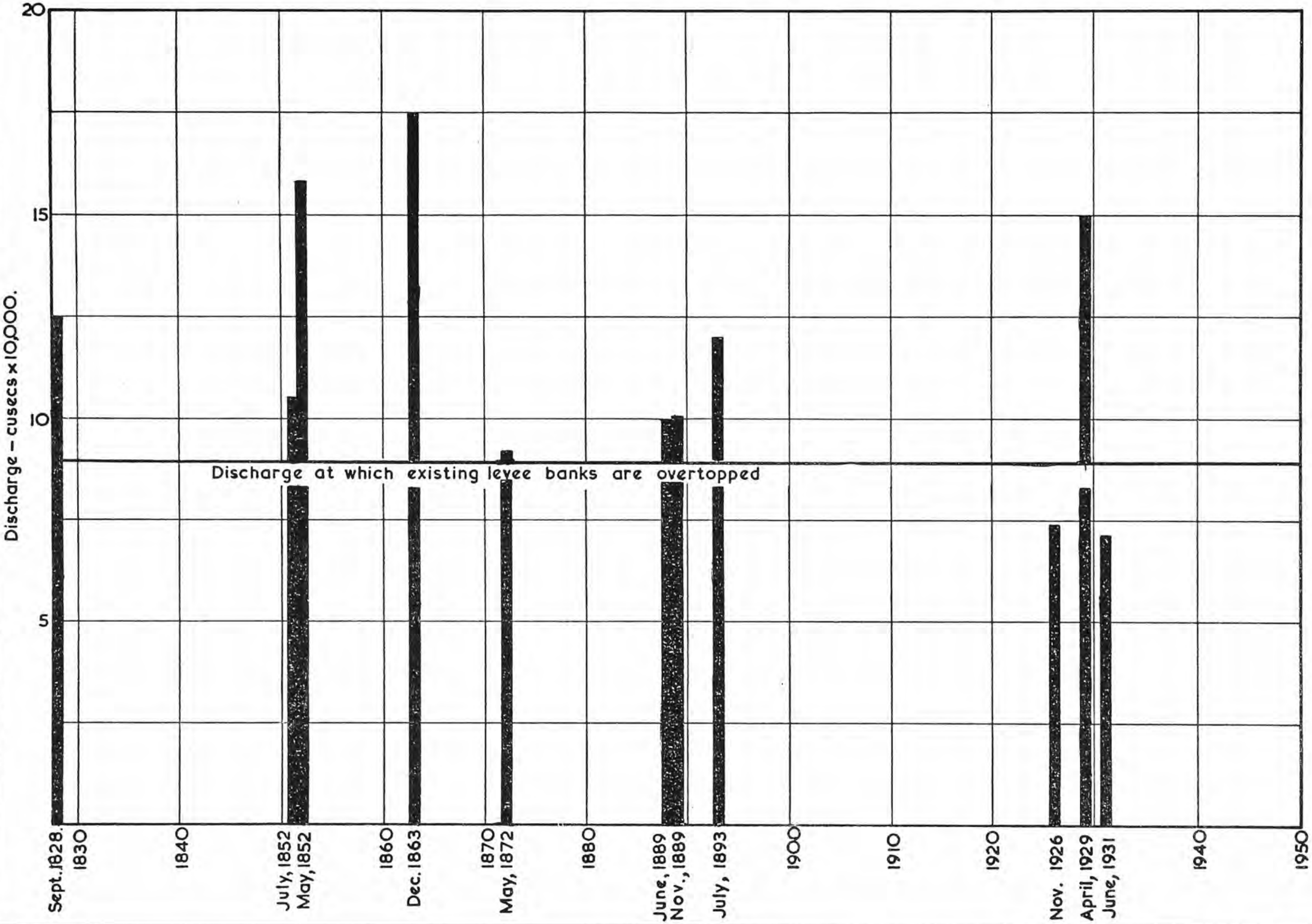


FIG. No. 29.

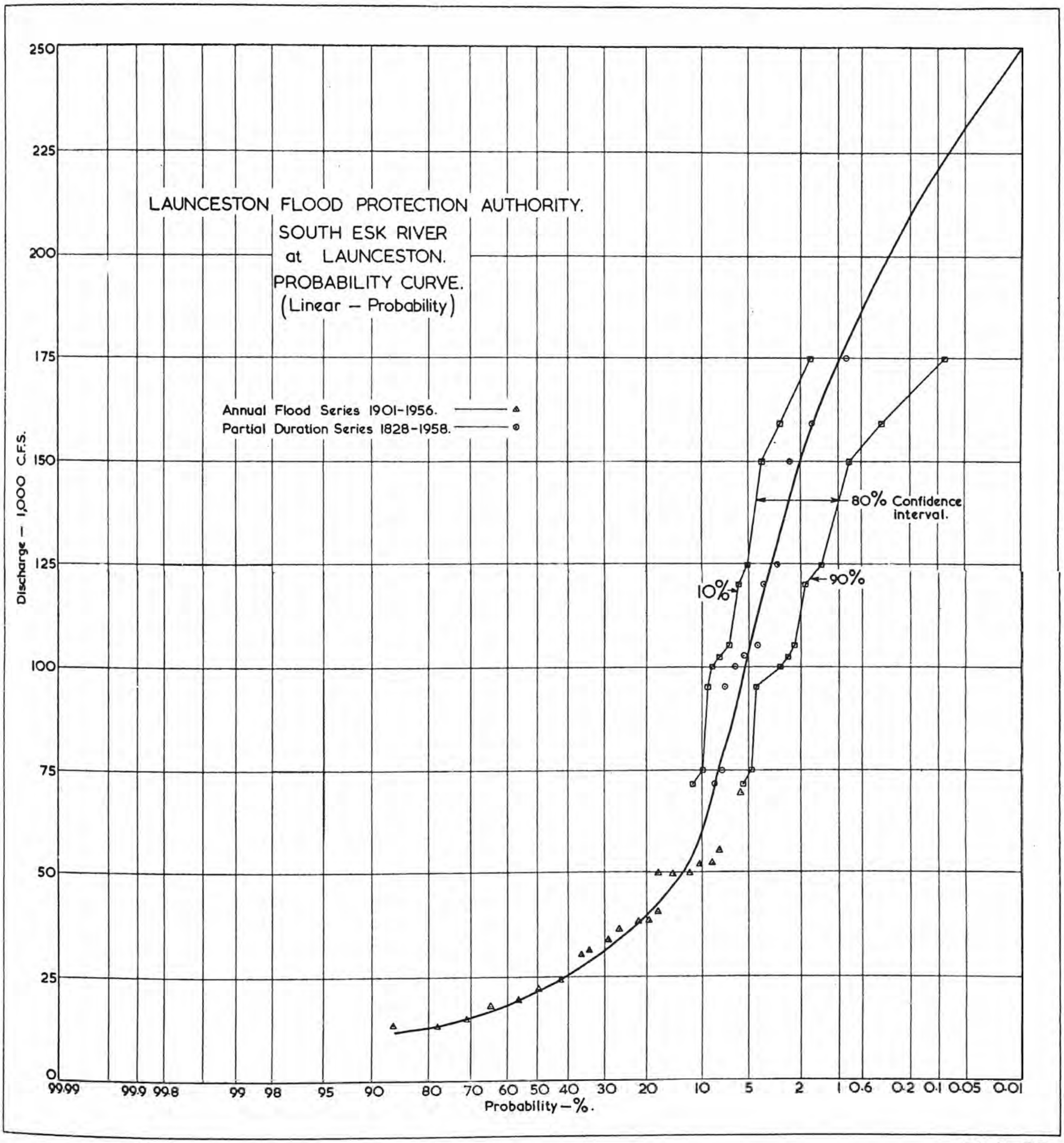
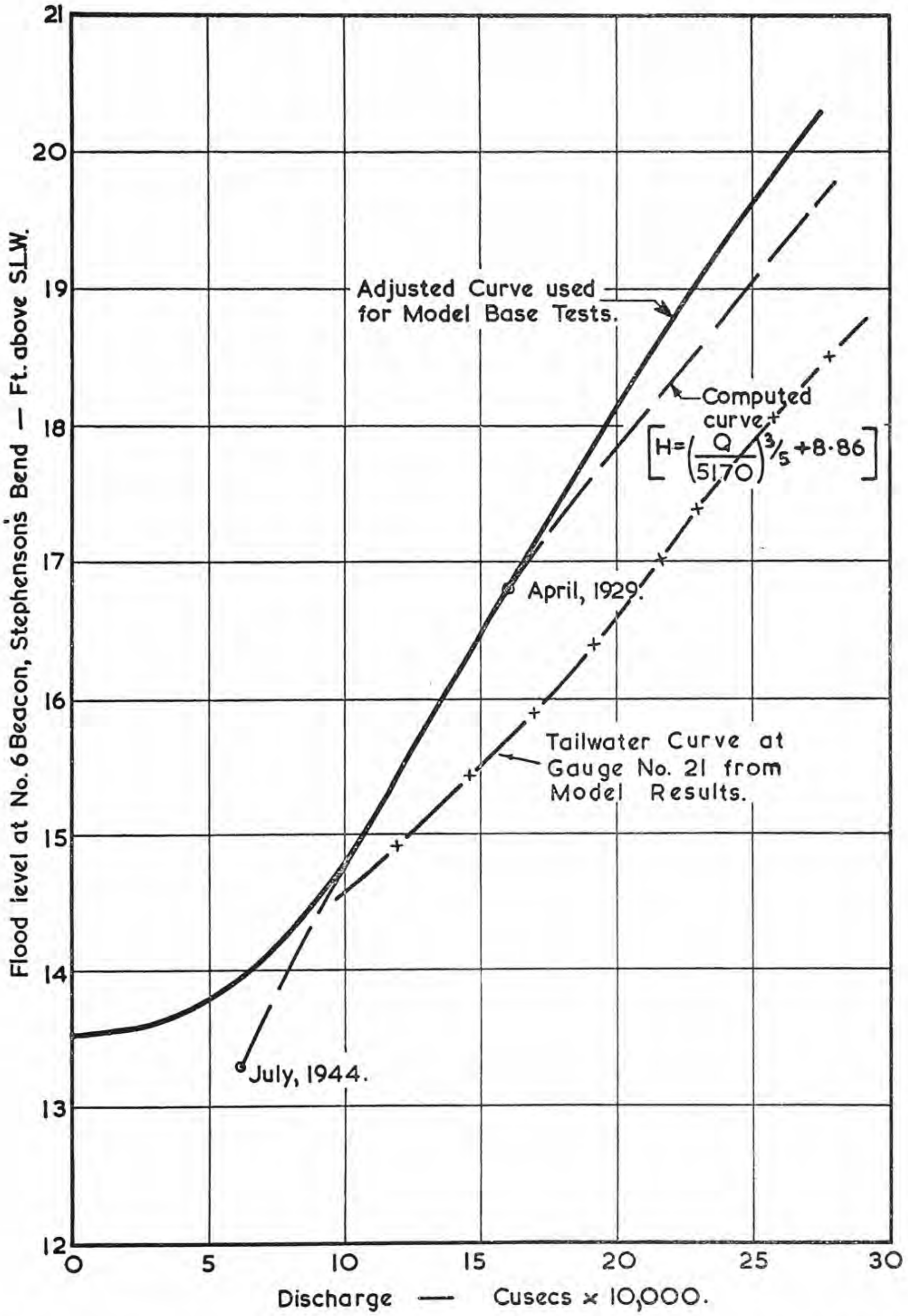

LAUNCESTON FLOOD PROTECTION AUTHORITY.
SOUTH ESK RIVER
at LAUNCESTON.
PROBABILITY CURVE.
(Linear – Probability)
Annual Flood Series 1901-1956.
Partial Duration Series 1828-1958.
80% Confidence interval.
10%
90%
Discharge – 1,000 C.F.S.
250
225
200
175
150
125
100
75
50
25
0
Probability – %.
99.99
99.9
99.8
99
98
95
90
80
70
60
50
40
30
20
10
5
2
1
0.6
0.2
0.1
0.05
0.01

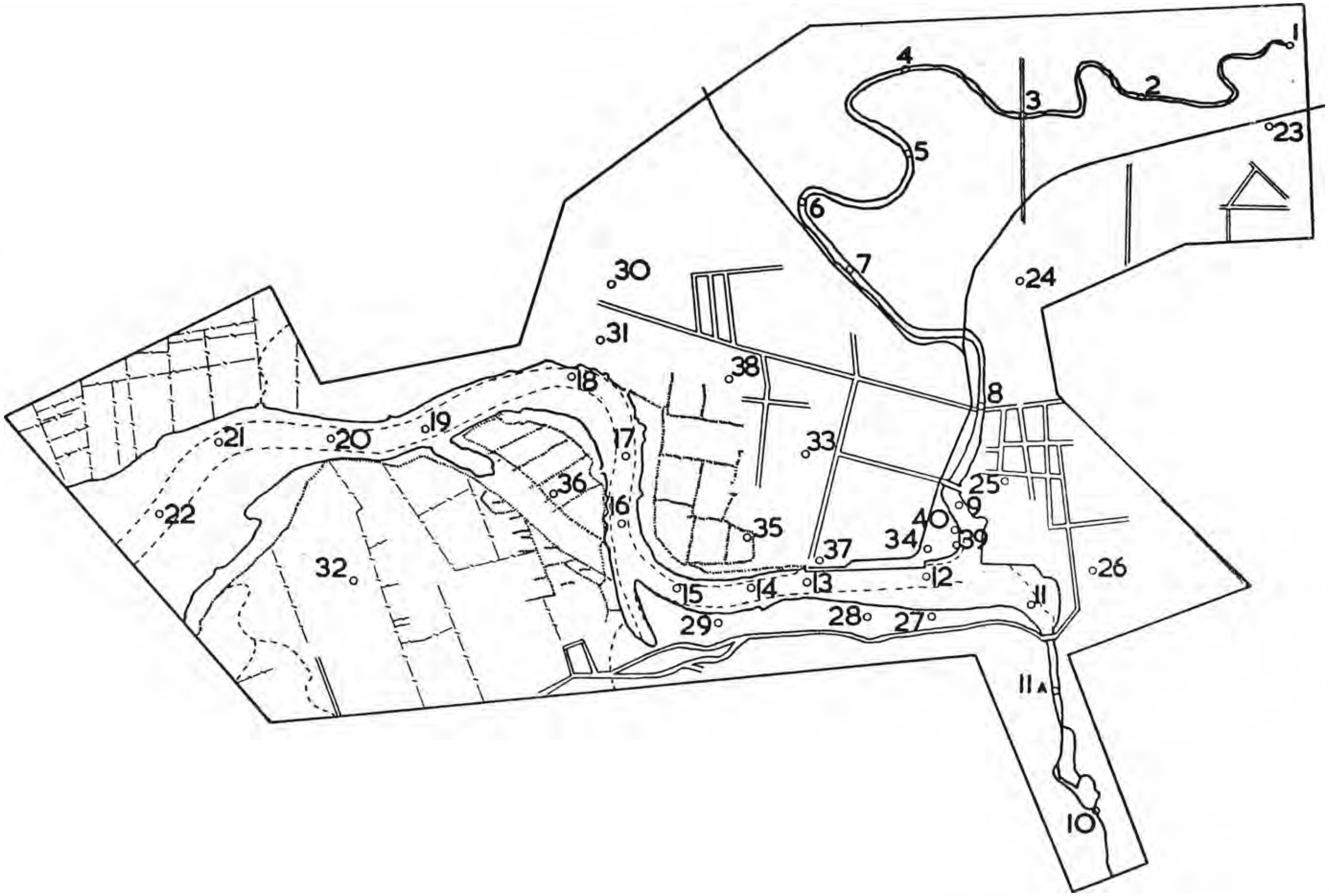
FIG. No. 30.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

Tailwater Rating Curve for Model Base Tests.

FIG. No. 31.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

MODEL TESTS

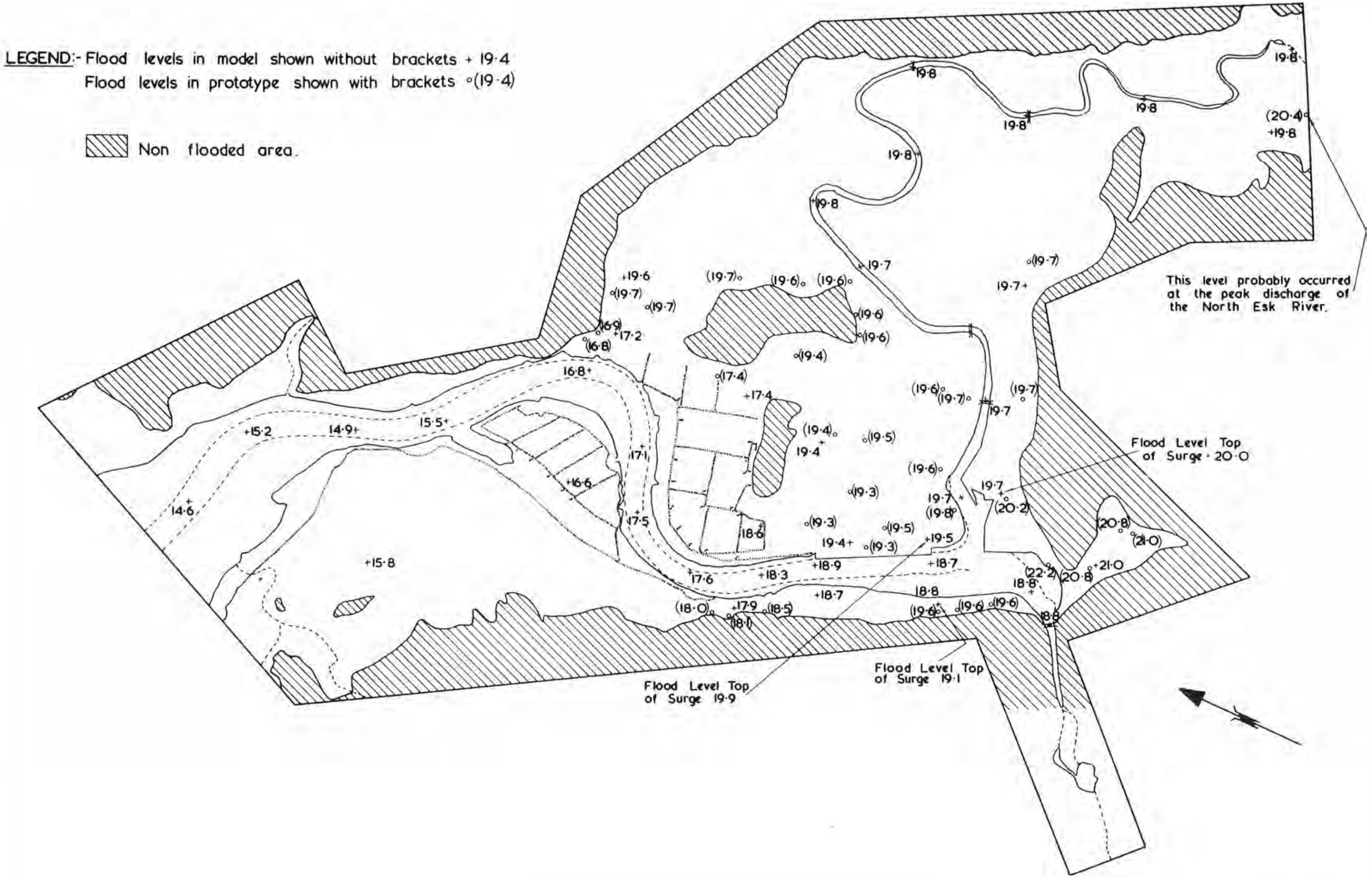
LOCATION OF FLOOD GAUGES

Date traced 12-6-1959

FIG. No. 32.

LEGEND:- Flood levels in model shown without brackets + 19·4
Flood levels in prototype shown with brackets ○(19·4)

▨ Non flooded area.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

MODEL VERIFICATION.
APRIL 1929 FLOOD.

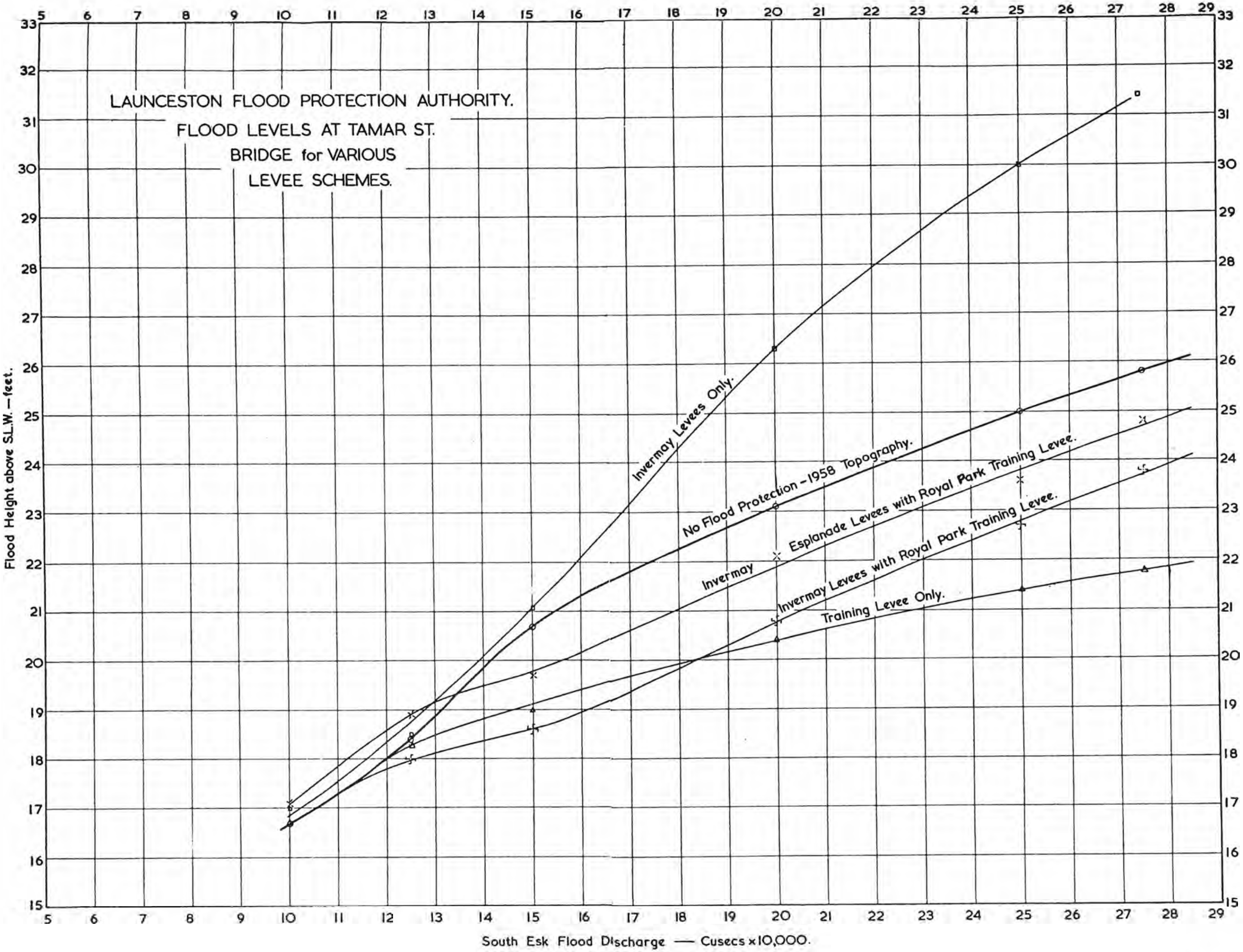
South Esk Discharge 150, 000 Cusecs.
North Esk Discharge 13, 100 Cusecs.

NOTE:- Tailwater set so that flood level at gauge No. 18 - 16·8

DATUM:- All flood levels given on L.C.C Datum.
To convert levels to State Datum subtract 8·60'

800 0 800 1600 2400
SCALE IN FEET

FIG. No. 33.


LAUNCESTON FLOOD PROTECTION AUTHORITY.
FLOOD LEVELS AT TAMAR ST. BRIDGE for VARIOUS LEVEE SCHEMES.
Invermay Levees Only.
No Flood Protection - 1958 Topography.
Invermay Esplanade Levees with Royal Park Training Levee.
Invermay Levees with Royal Park Training Levee.
Training Levee Only.
Flood Height above S.L.W. - feet.
South Esk Flood Discharge — Cusecs × 10,000.
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
15
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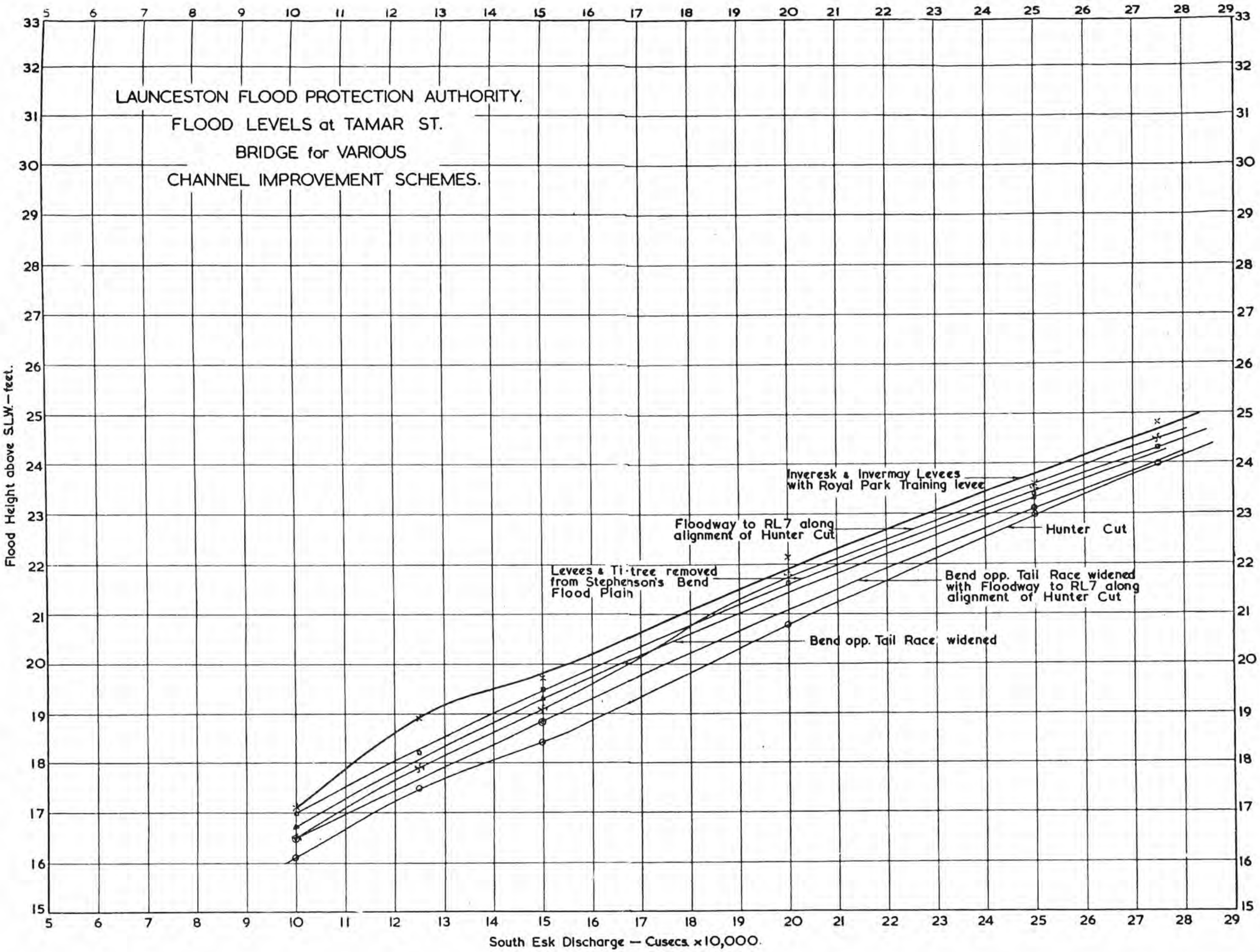
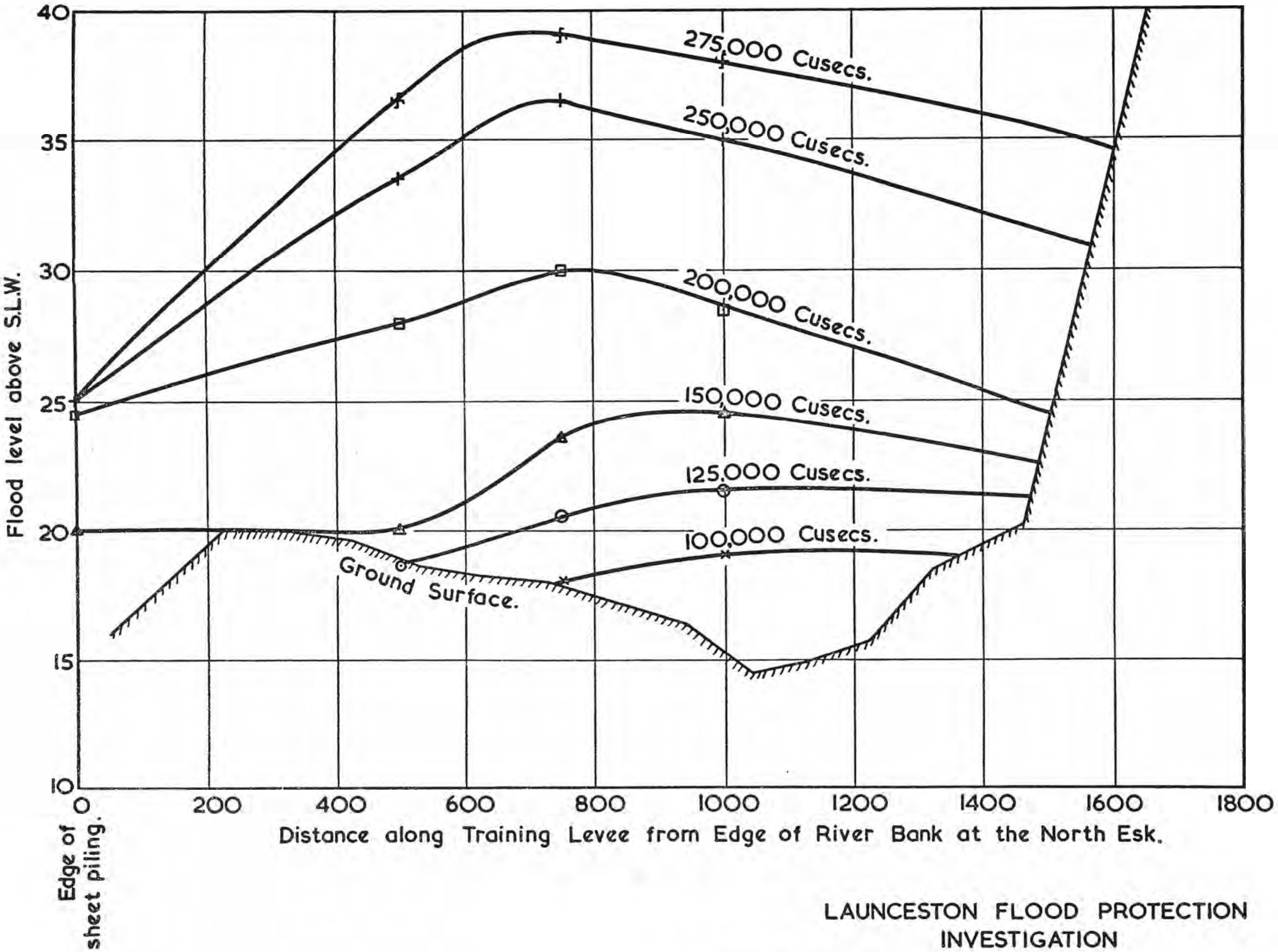
32
33

FIG No. 34.



LAUNCESTON FLOOD PROTECTION AUTHORITY.
FLOOD LEVELS at TAMAR ST.
BRIDGE for VARIOUS
CHANNEL IMPROVEMENT SCHEMES.

FIG. No. 35.

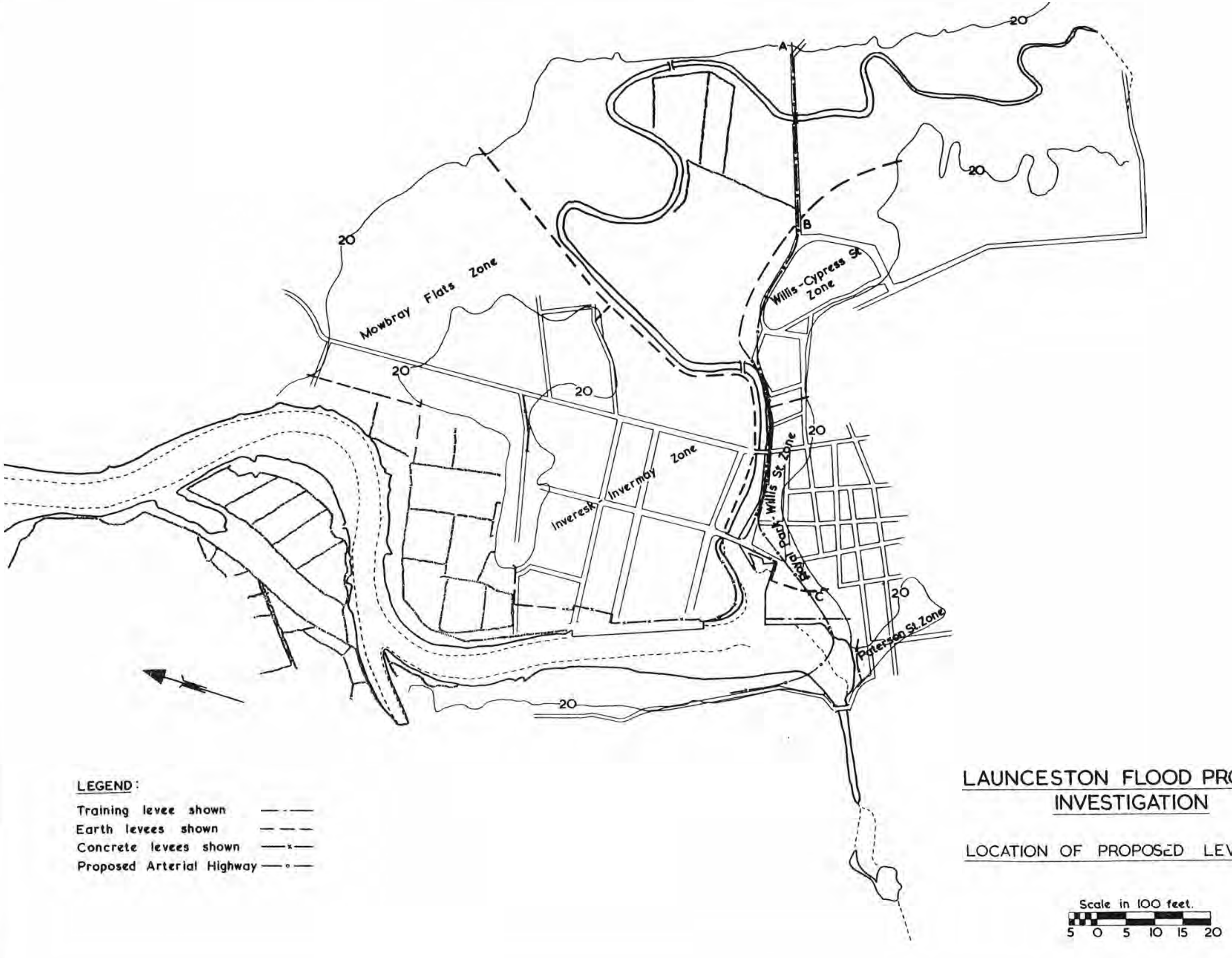


LAUNCESTON FLOOD PROTECTION
INVESTIGATION

WATER SURFACE PROFILES AT TOP OF SURGE
FOR ROYAL PARK TRAINING LEVEE.

7·7·59.

FIG. No. 36.

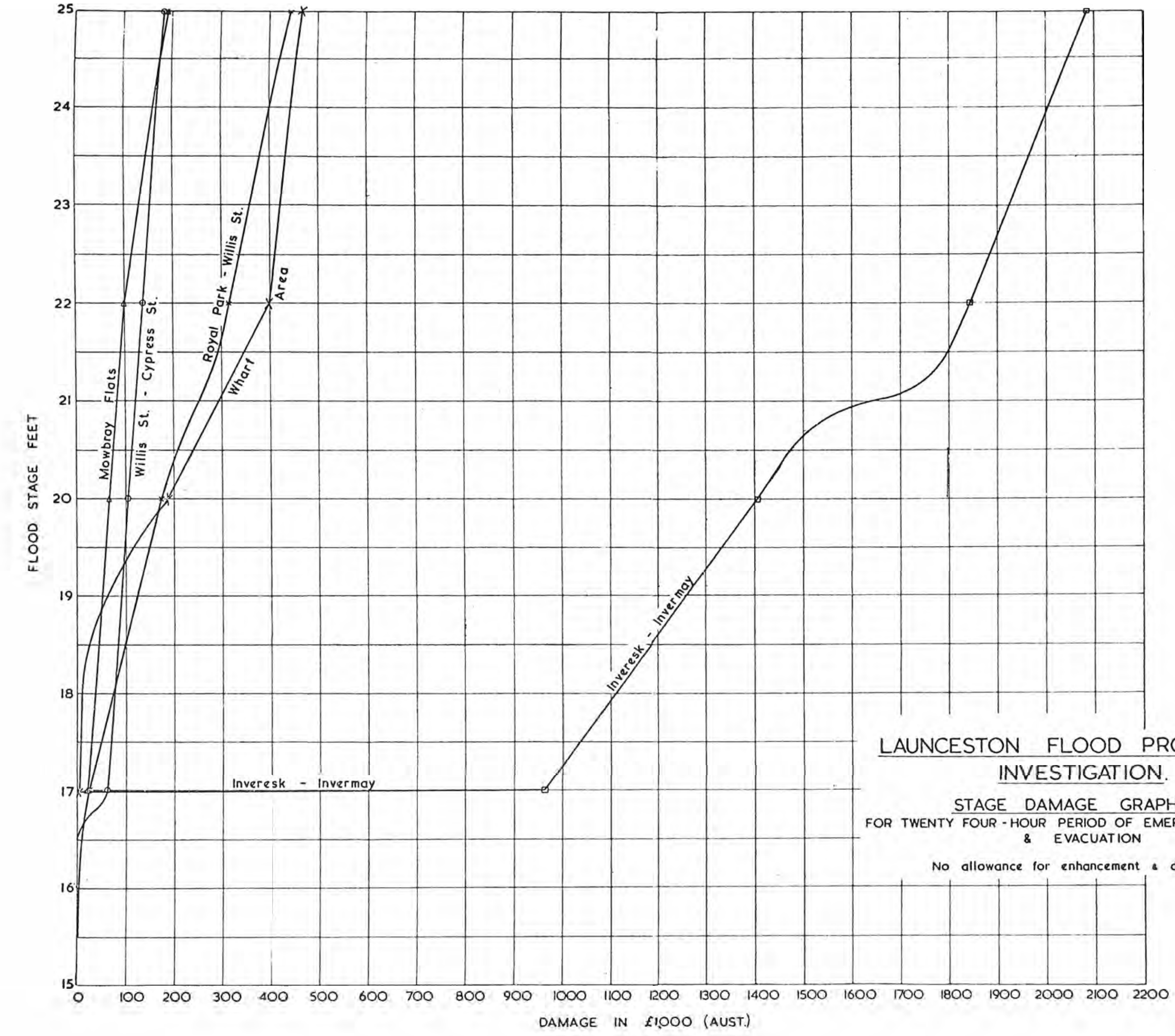


LAUNCESTON FLOOD PROTECTION INVESTIGATION

LOCATION OF PROPOSED LEVEE BANKS

Date traced 15·7·1959

FIG. No. 37.



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

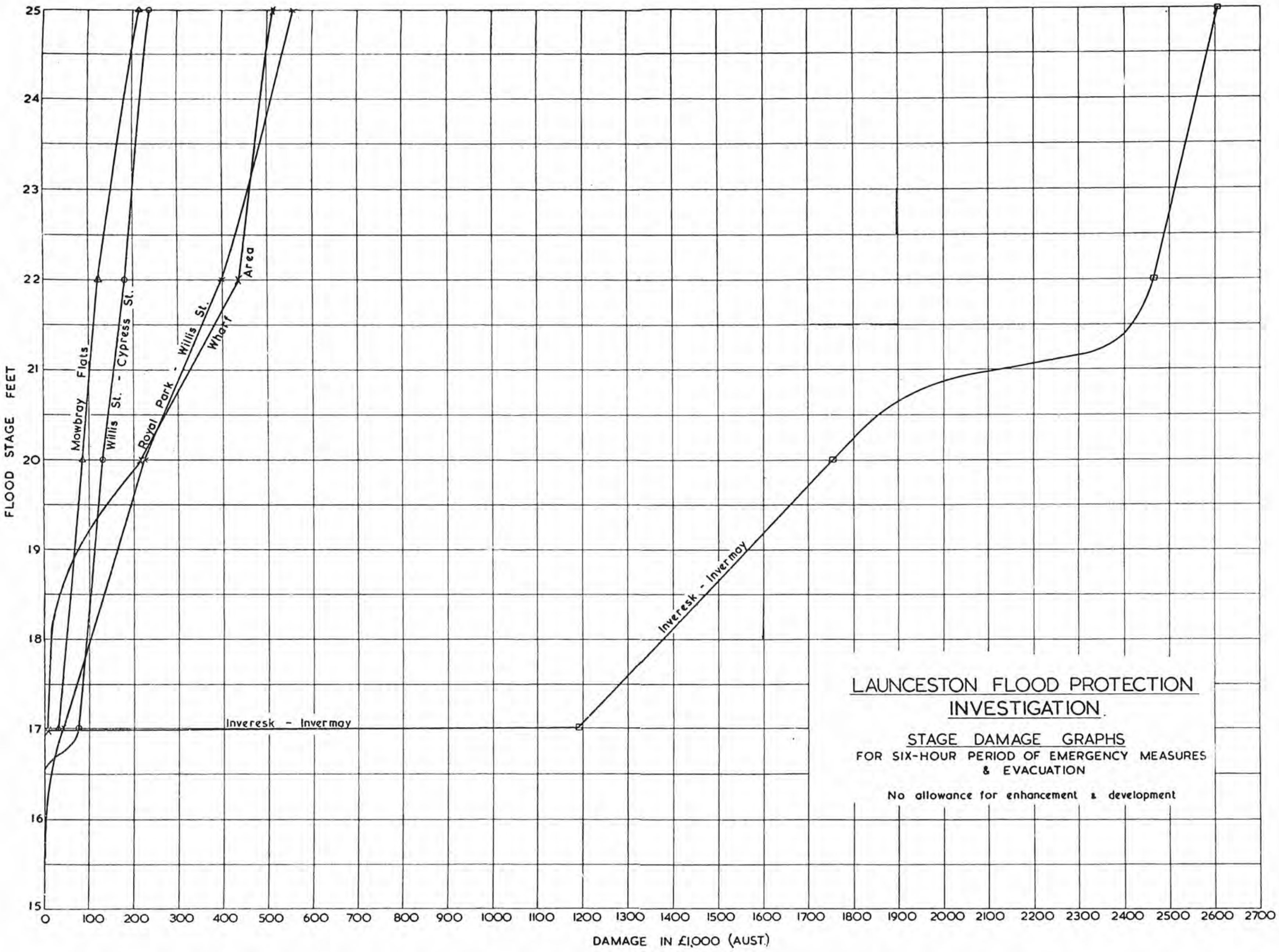
STAGE DAMAGE GRAPHS

FOR TWENTY FOUR - HOUR PERIOD OF EMERGENCY MEASURES & EVACUATION

No allowance for enhancement & development

Date traced 1-6-1959

FIG. No. 38



LAUNCESTON FLOOD PROTECTION INVESTIGATION.

STAGE DAMAGE GRAPHS

FOR SIX-HOUR PERIOD OF EMERGENCY MEASURES
& EVACUATION

No allowance for enhancement & development

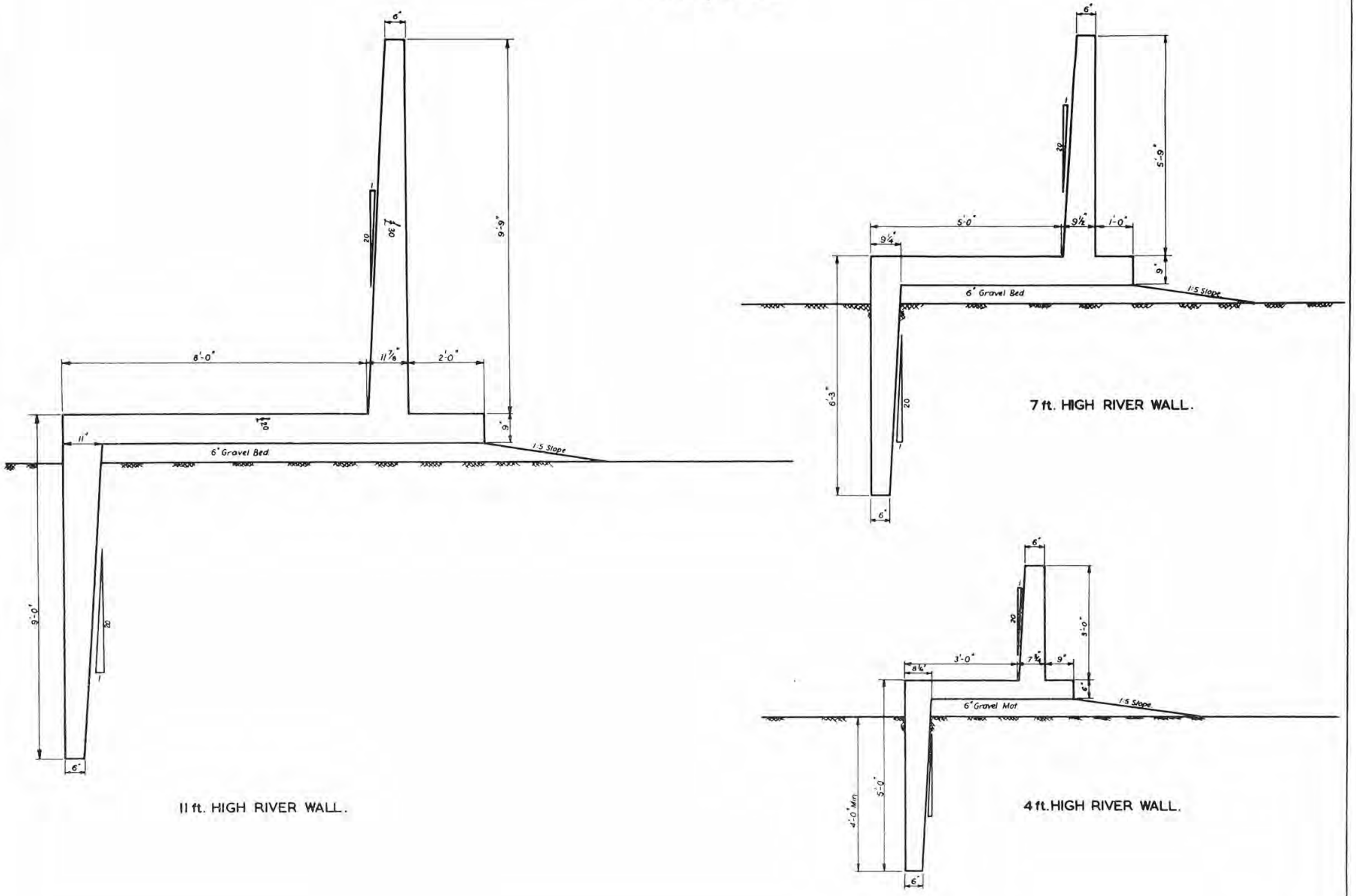
Date traced 29-5-1959

FIG. No.39

LAUNCESTON FLOOD PROTECTION AUTHORITY.

CONCRETE FLOOD BANKS.

Scale 3/4" to 1 foot.

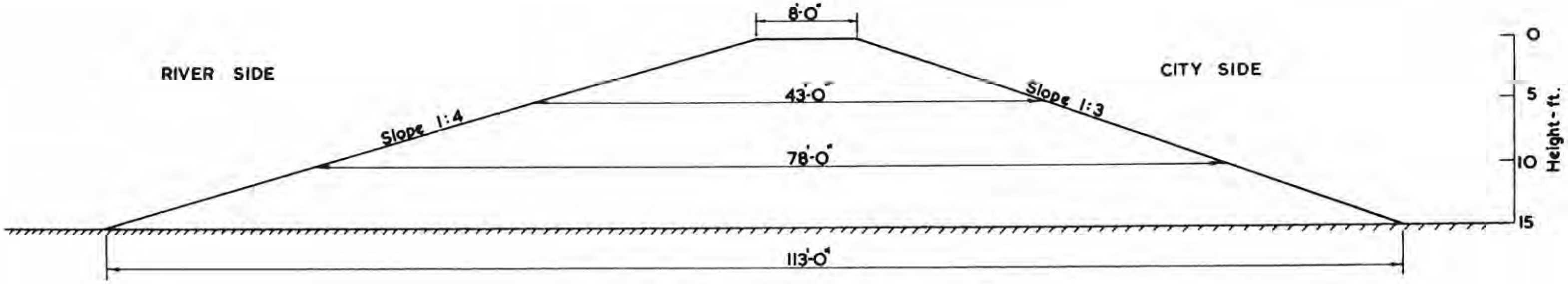


11 ft. HIGH RIVER WALL.

7 ft. HIGH RIVER WALL.

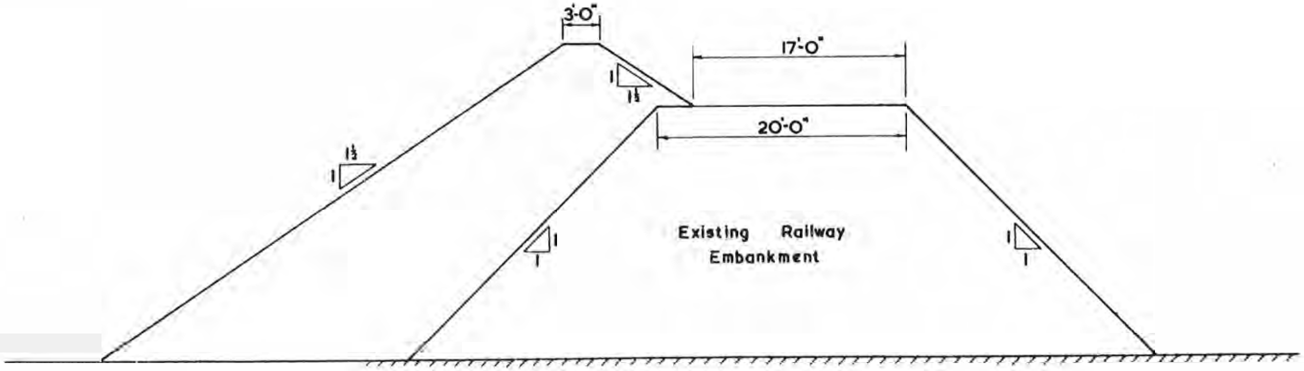
4 ft. HIGH RIVER WALL.

FIG. No. 40.



TYPICAL CROSS SECTION OF PROPOSED EARTH LEVEES

FIG. No. 41.



TYPICAL CROSS SECTION OF PROPOSED RAILWAY EMBANKMENTS.

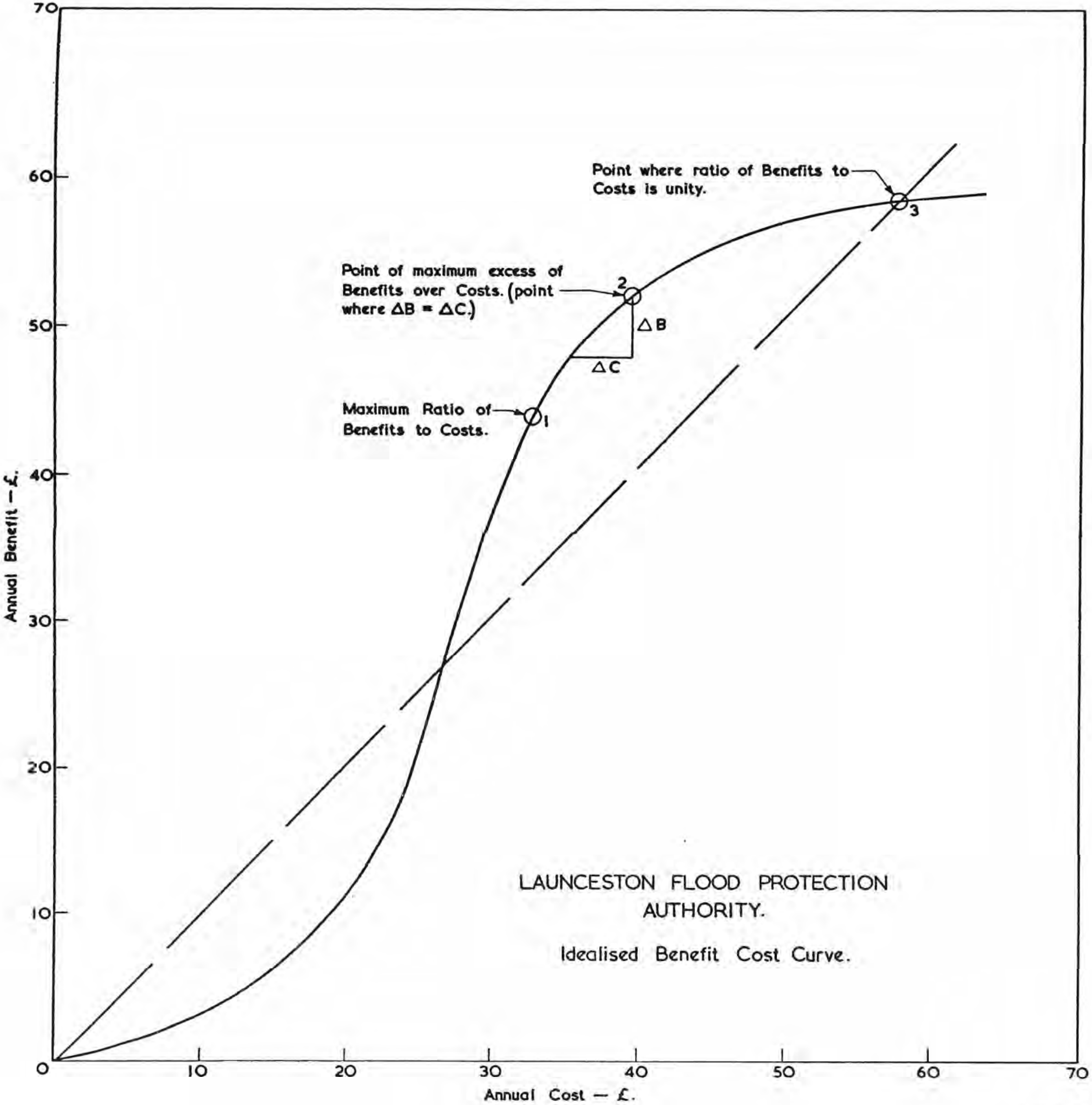
FIG. No. 42.

LAUNCESTON FLOOD PROTECTION INVESTIGATION.

Scale in feet

5 0 5 10

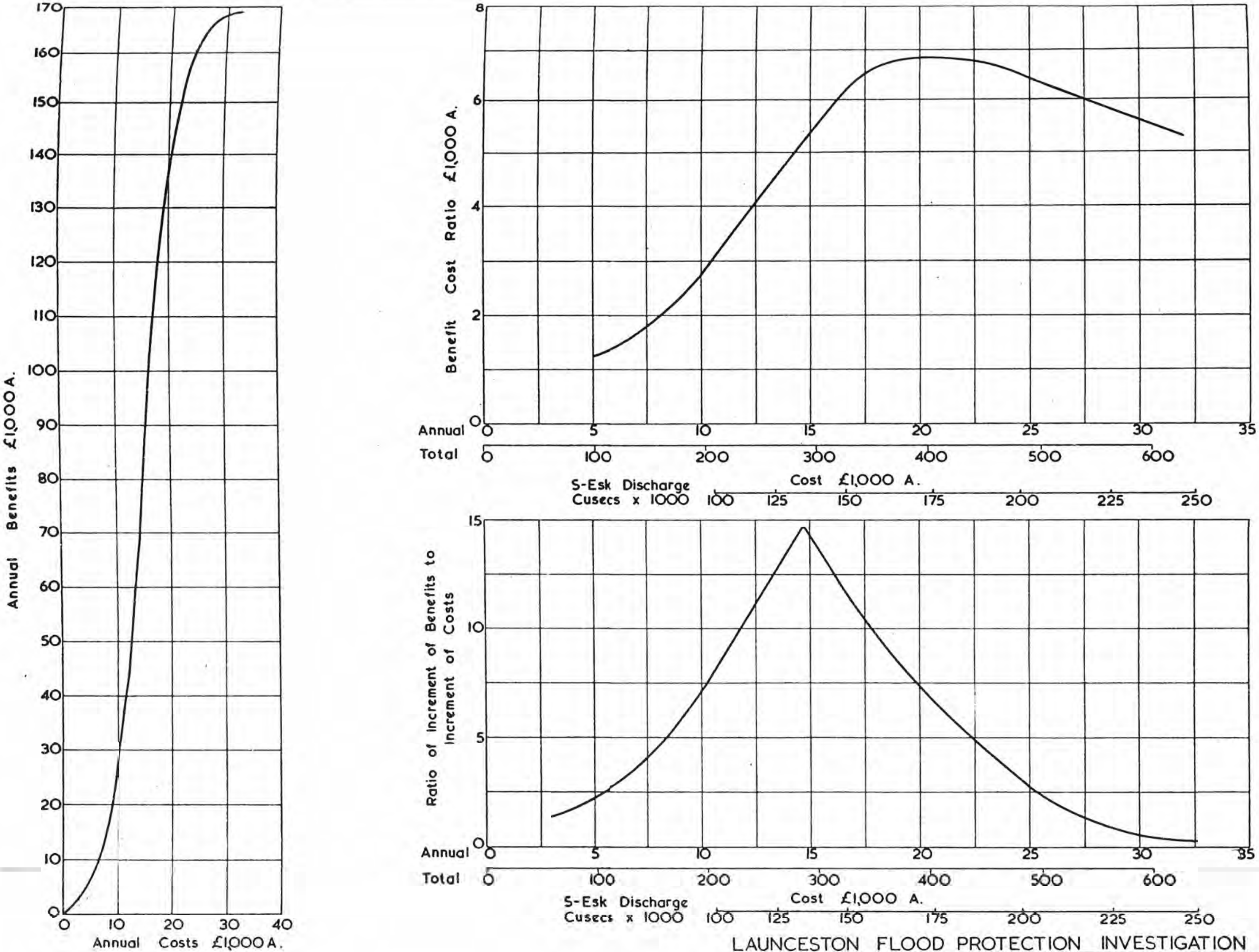
FIG. Nos. 41 & 42.



LAUNCESTON FLOOD PROTECTION AUTHORITY.

Idealised Benefit Cost Curve.

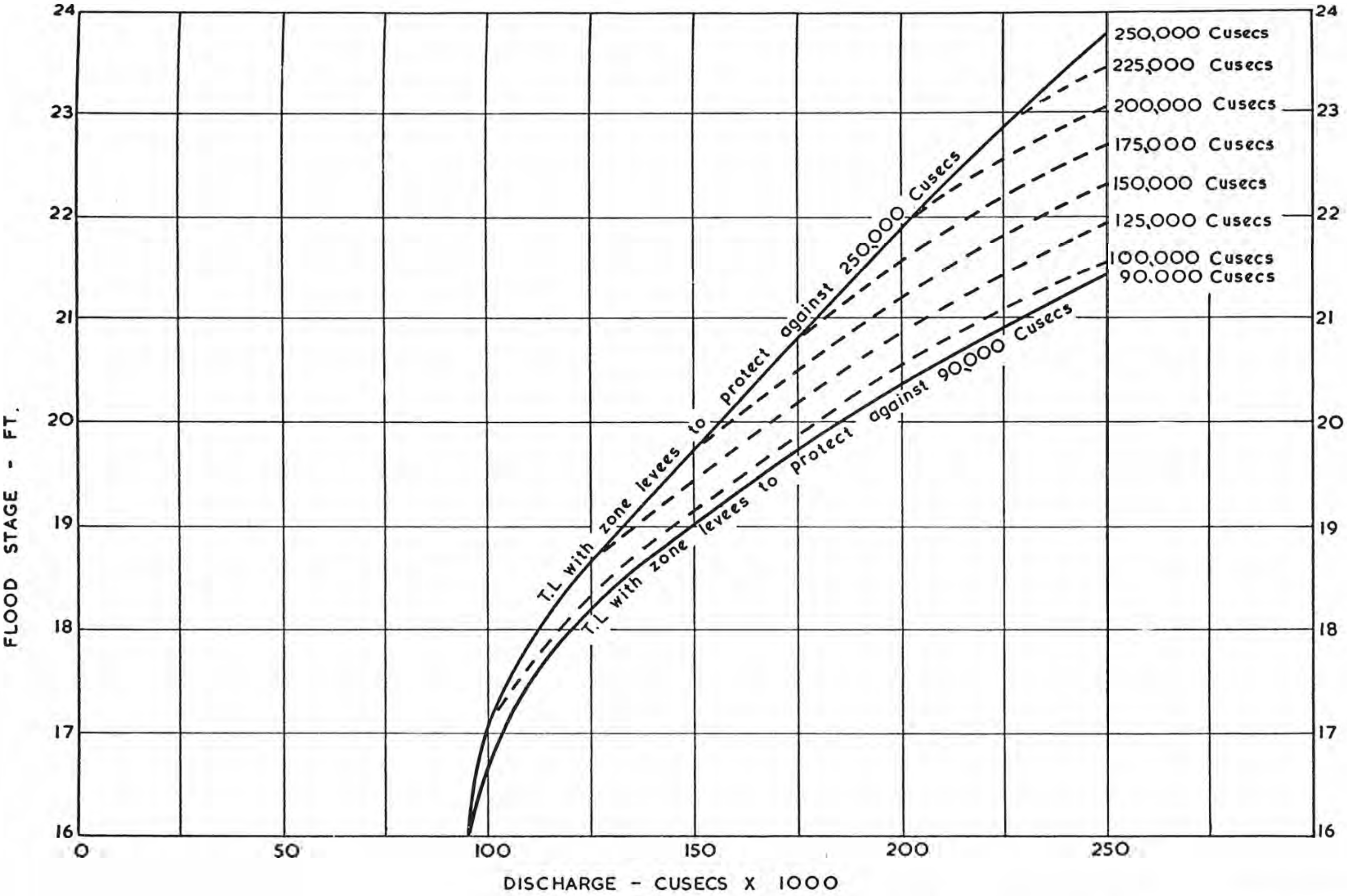
FIG. 43.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

COST BENEFIT CURVES FOR TRAINING LEVEE
TO DIVERT MAXIMUM PROBABLE FLOOD
WITH ALL ZONES EQUALLY PROTECTED BY LEVEES

FIG. No. 44.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

Stage Discharge Relationships
at Tamar St. Bridge for
Training Levee together with Surround Levees

FIG. No. 45

THE UNIVERSITY OF NEW SOUTH WALES

WATER RESEARCH LABORATORY

REPORT No. 8

VOLUME 3 - PHOTOGRAPHS

Flood Mitigation Measures For the City of Launceston

by

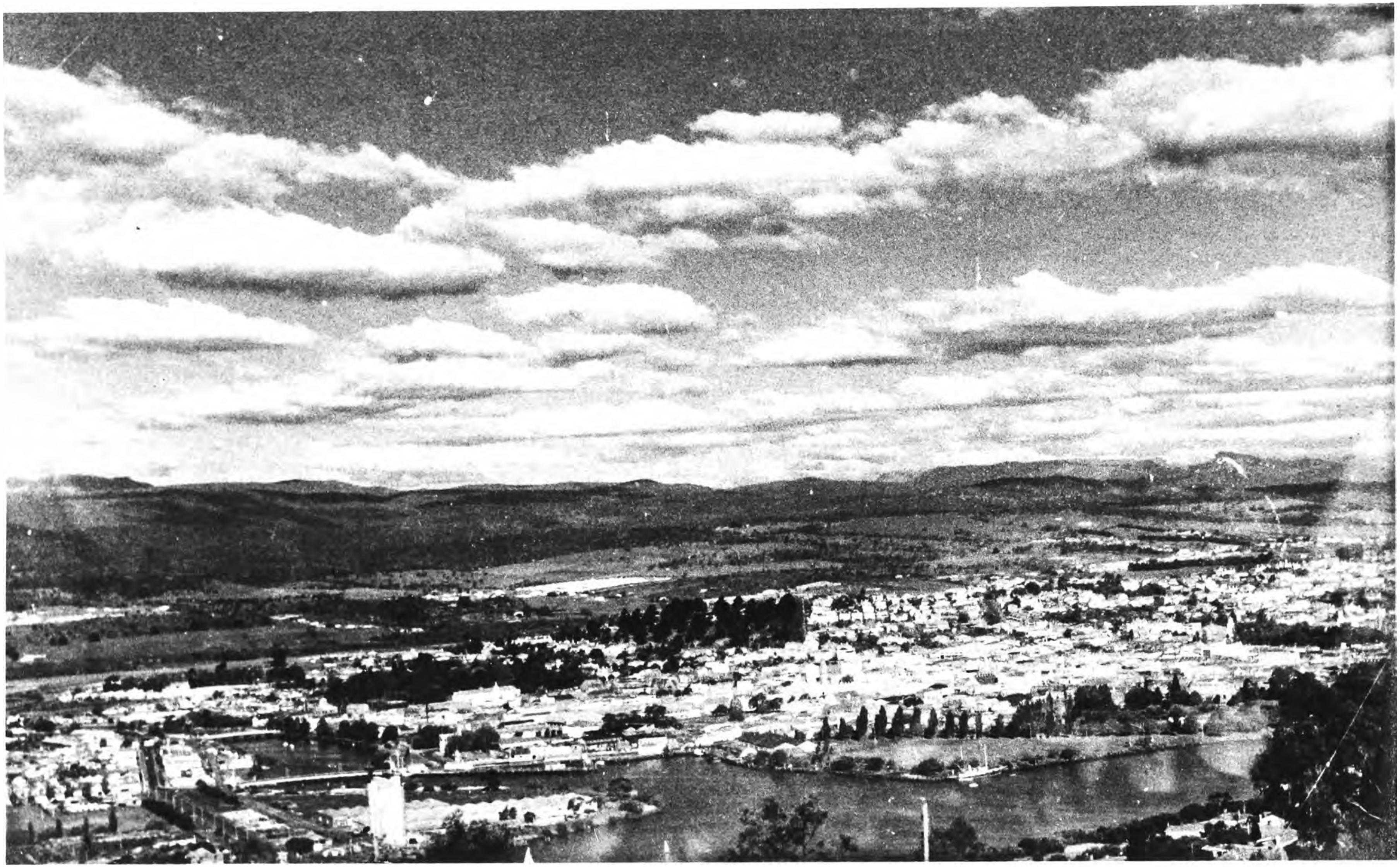
C.H. Munro

SEPTEMBER, 1959

LIST OF PLATES (VOL.III)

Note: Only a limited edition (12 copies) of the complete Vol.III is available. A further 25 copies omitting the less important plates have been printed.

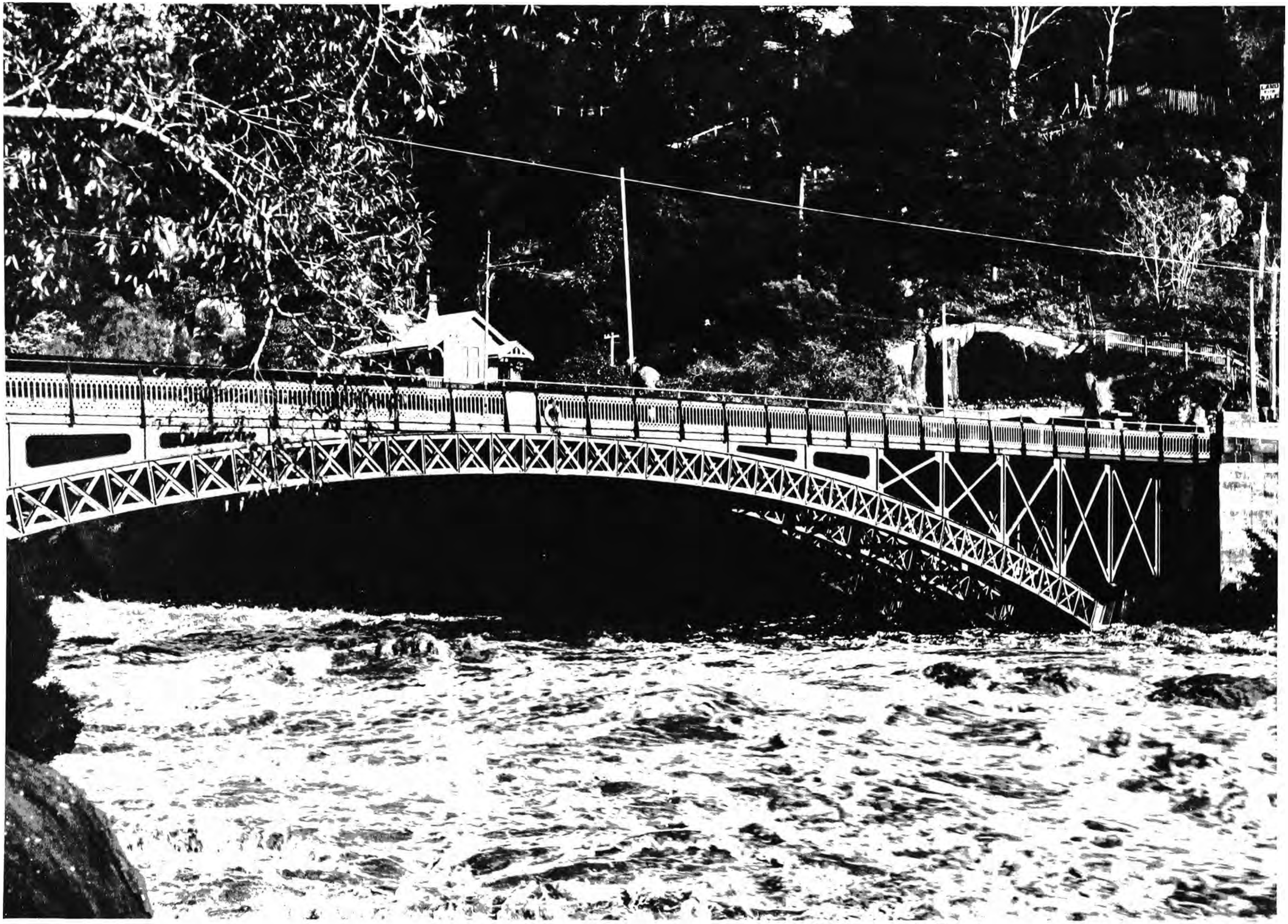
Photograph No. 1.



Photograph No.2

Photograph No.3

Photograph No.4



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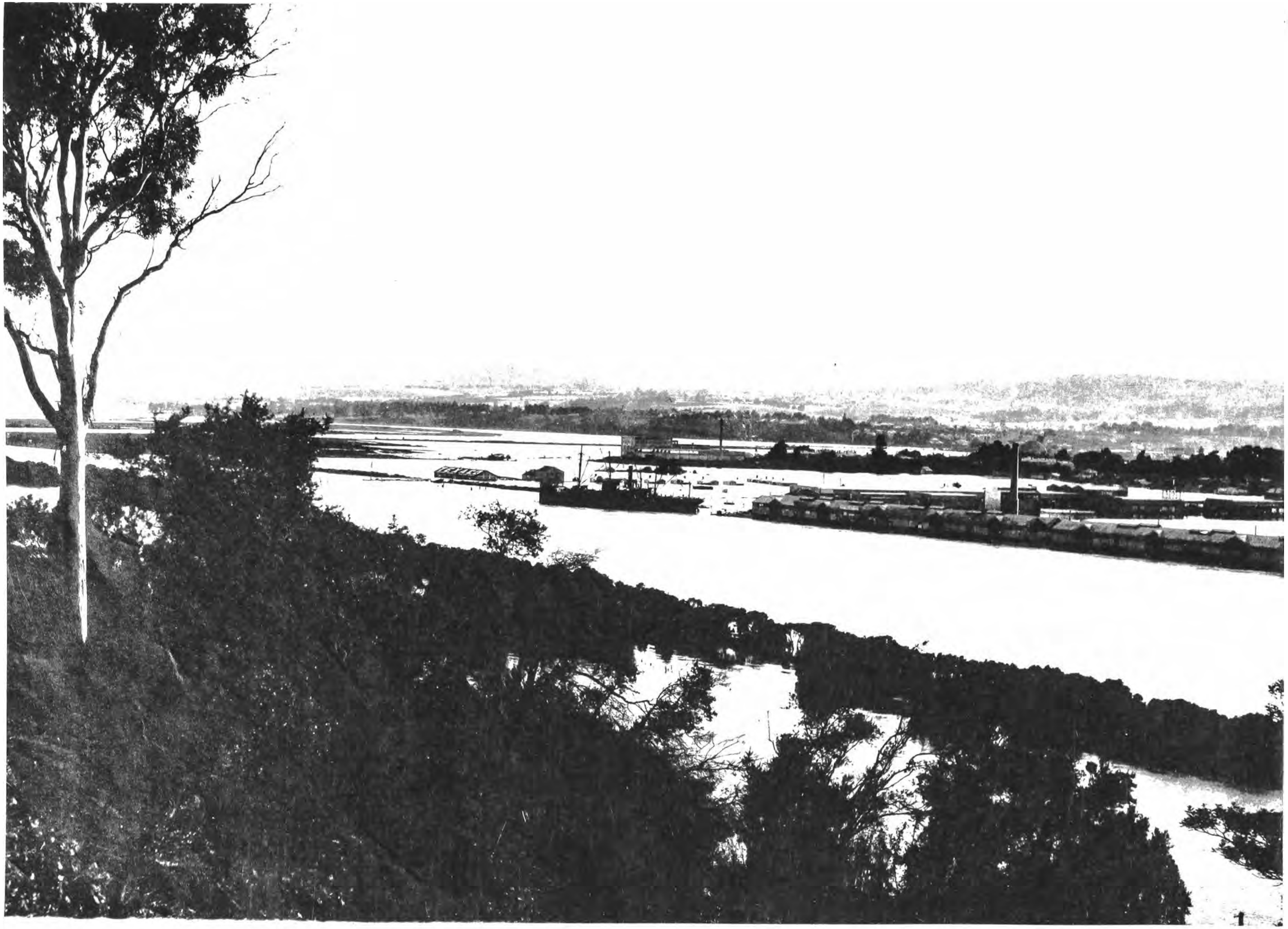
Photograph No.6



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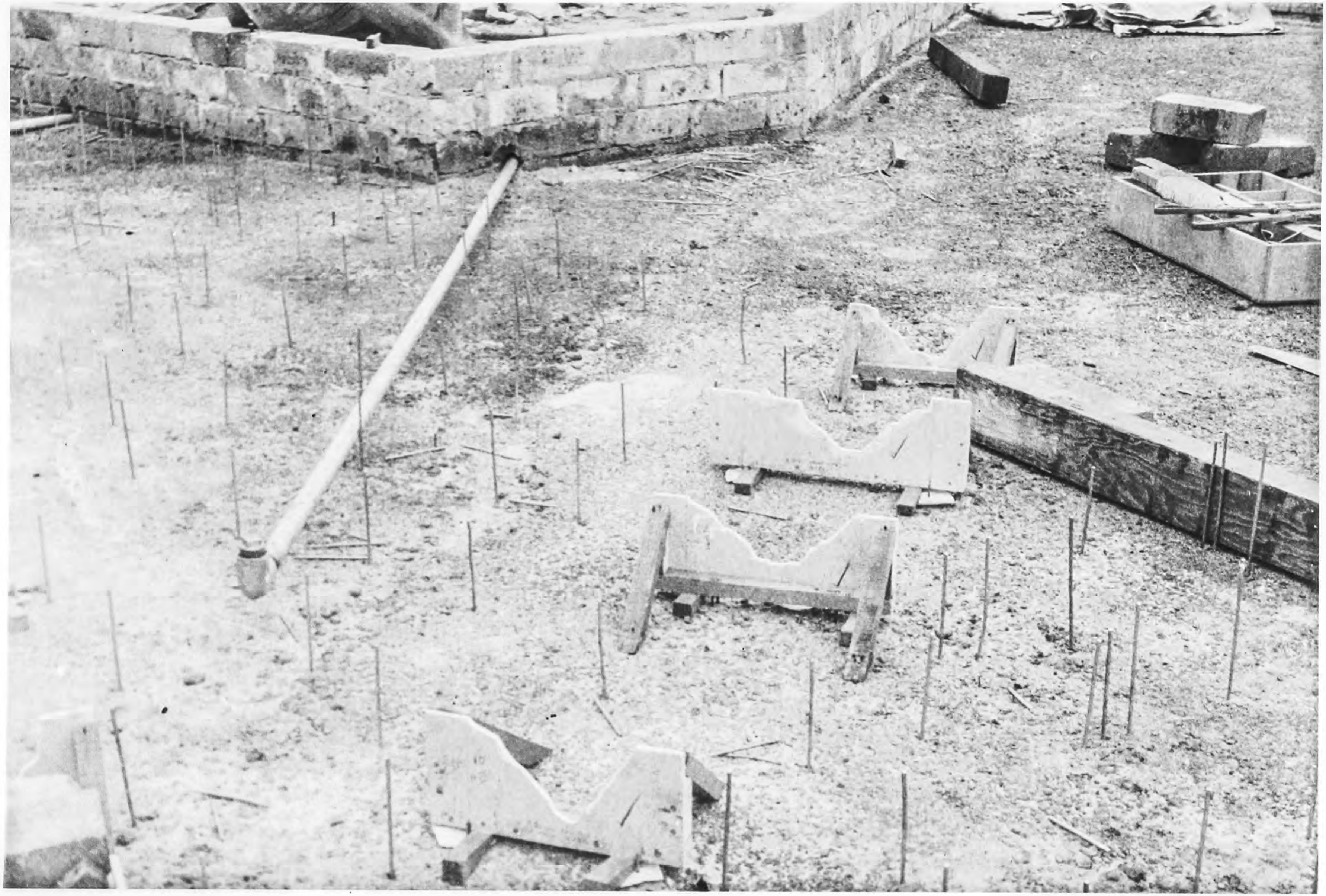
Photograph No.9

Photograph No.10



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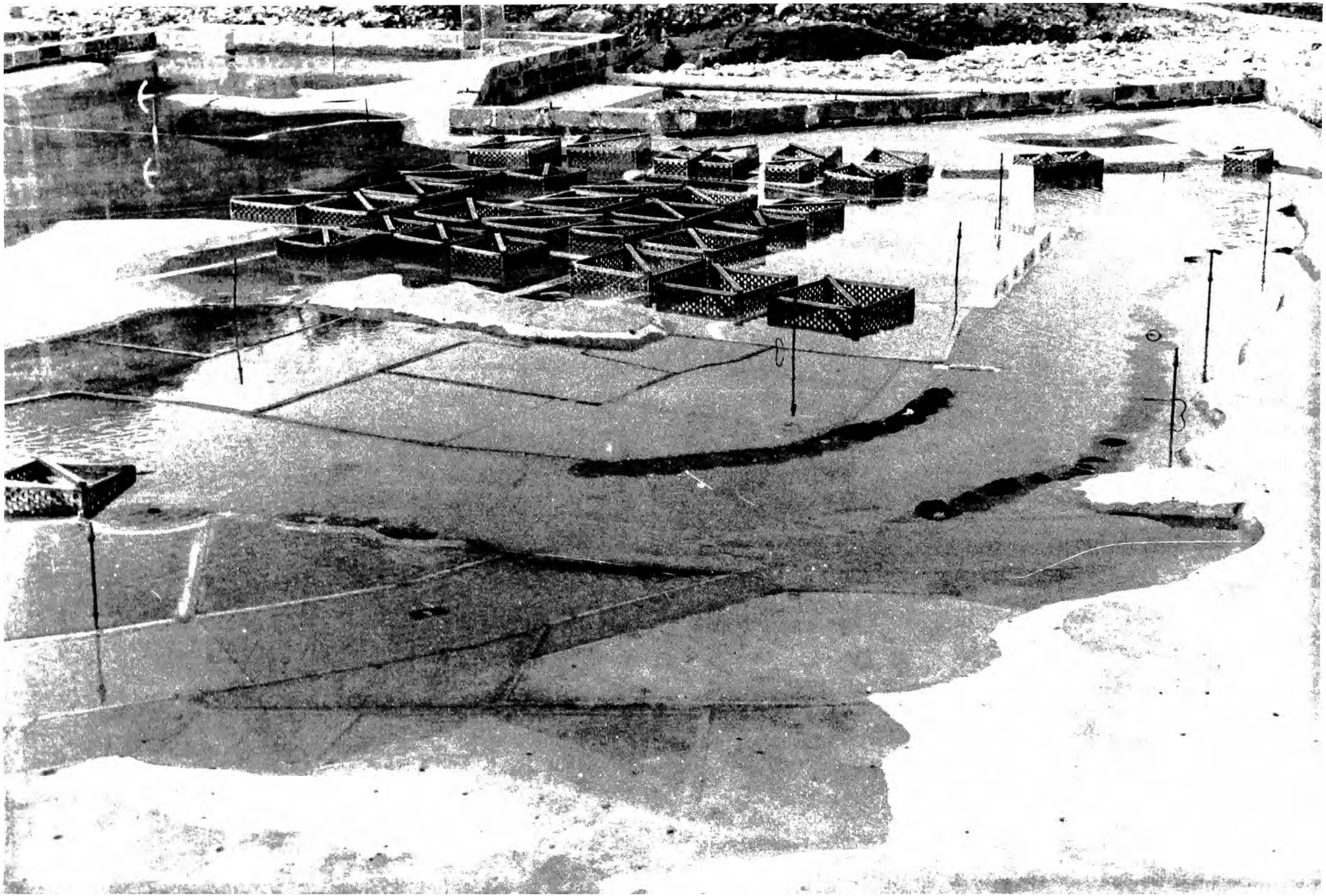

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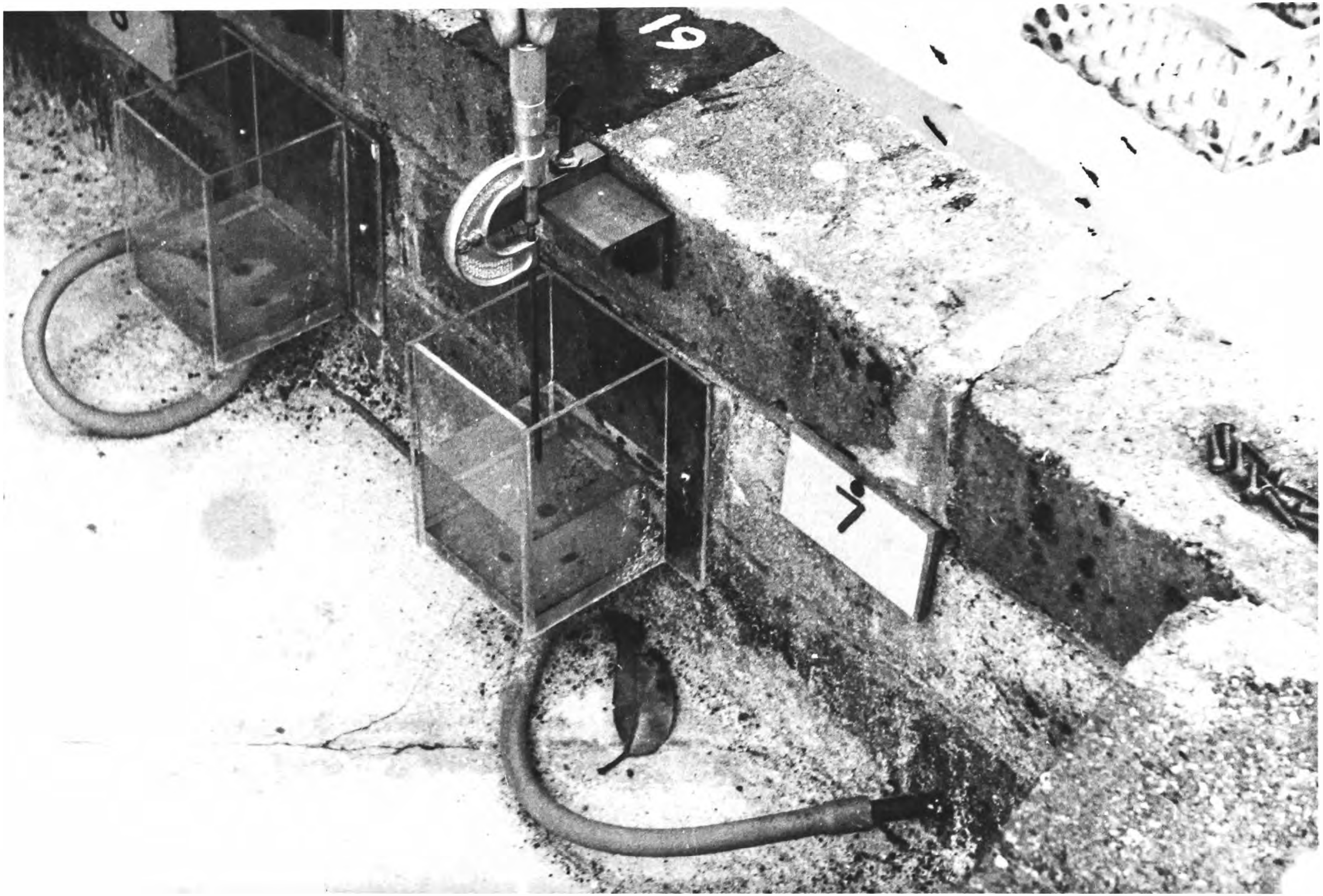

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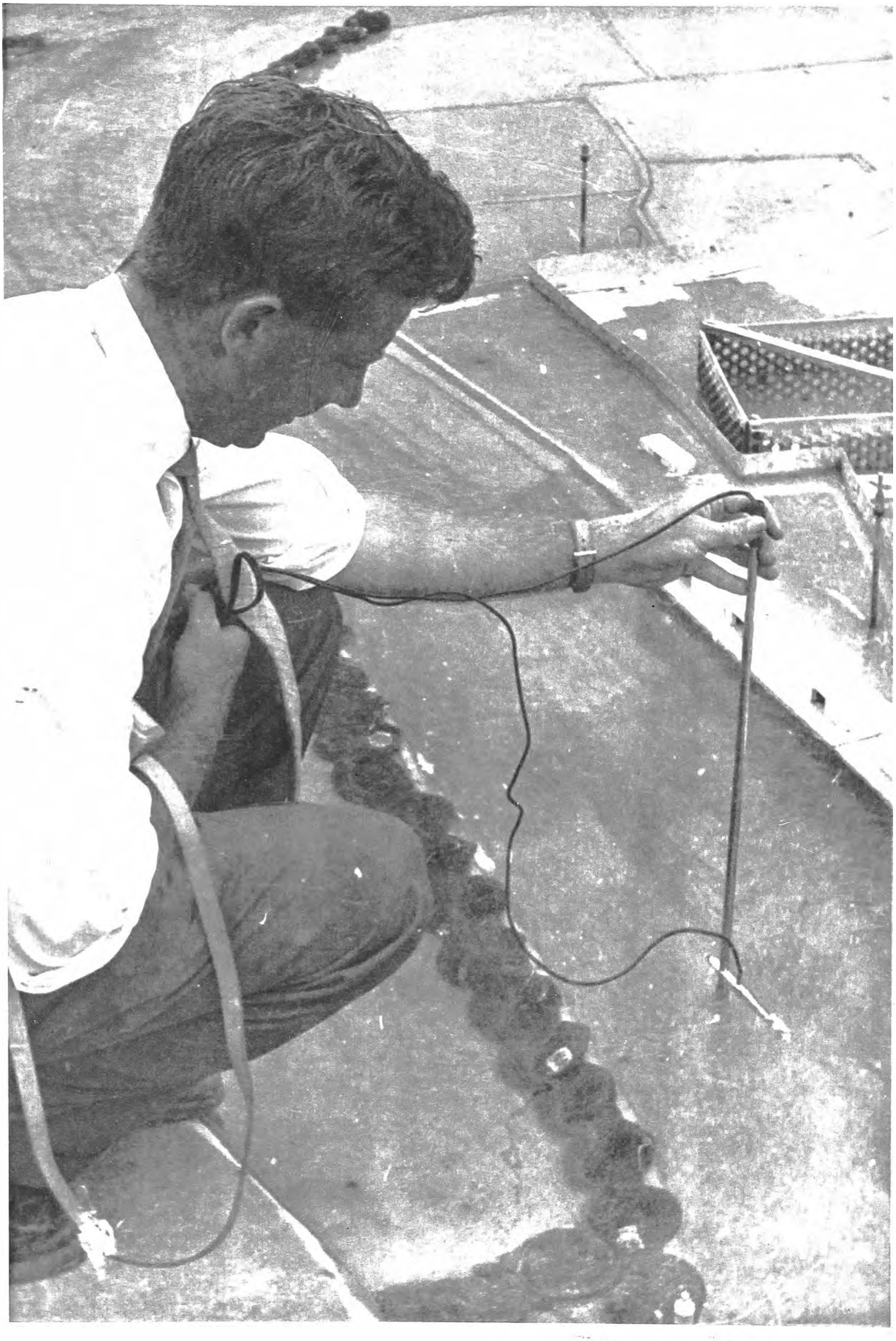
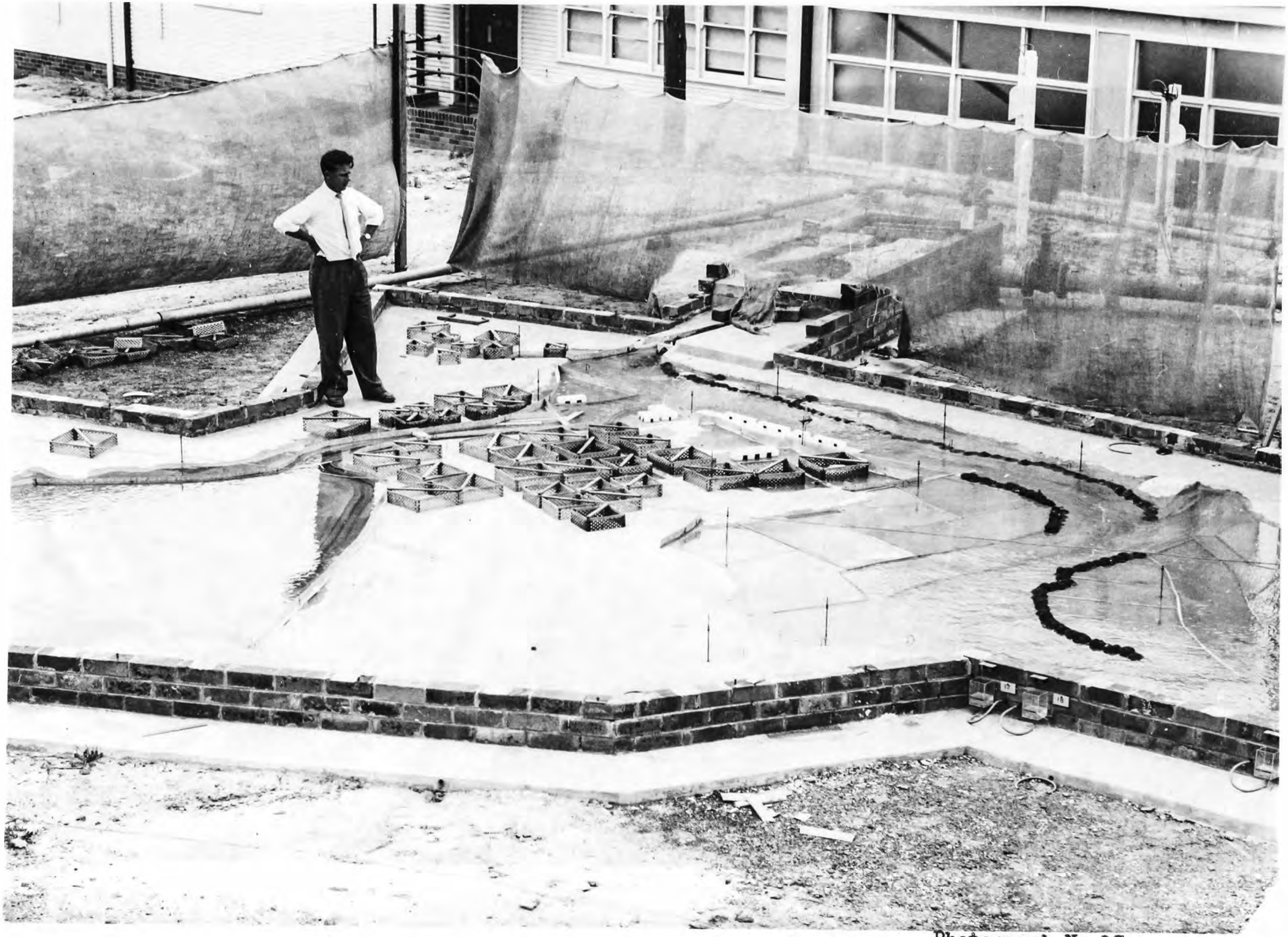


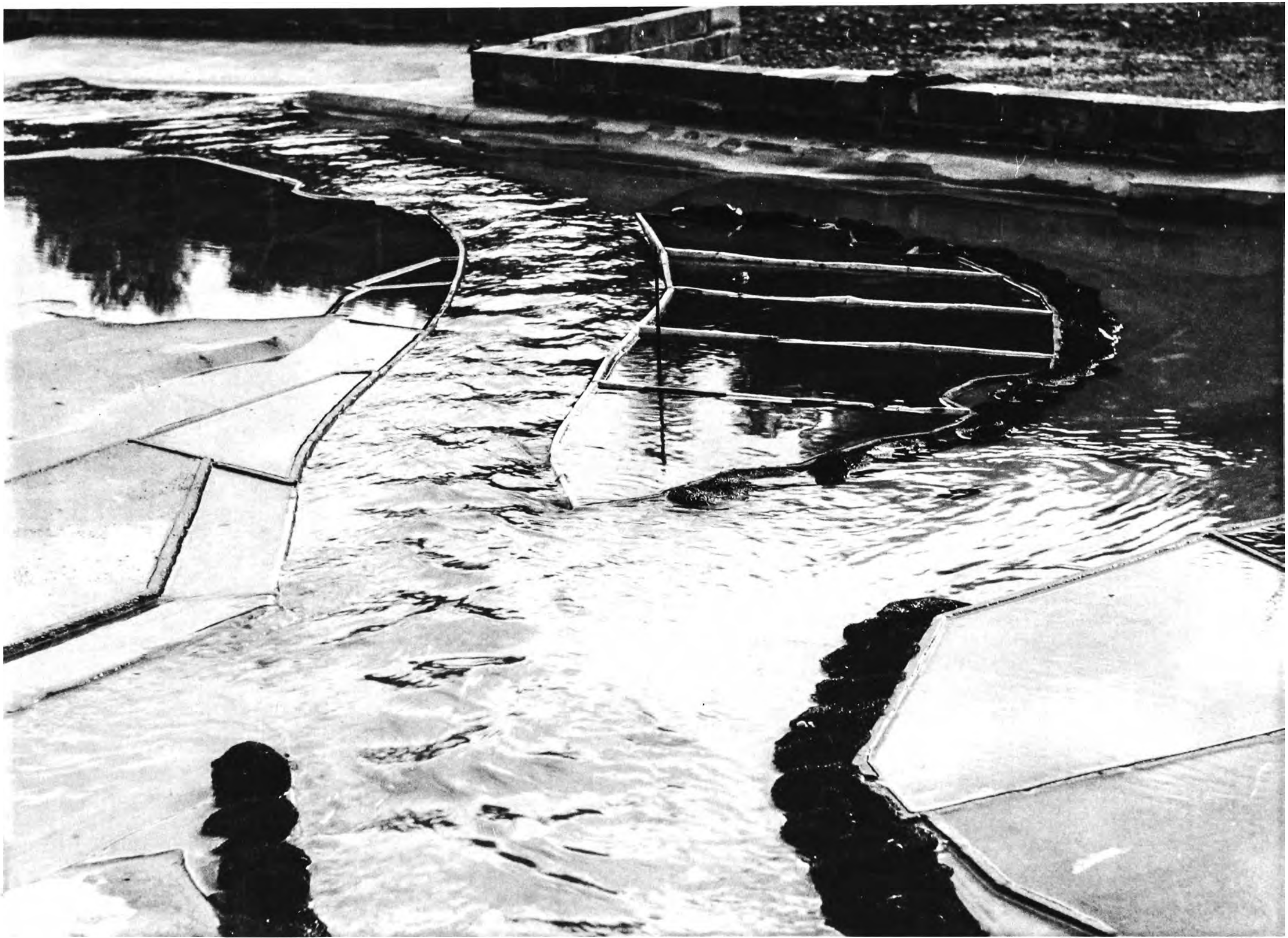


Photograph No.17

Photograph No.18

Photograph No.19

Photograph No.20



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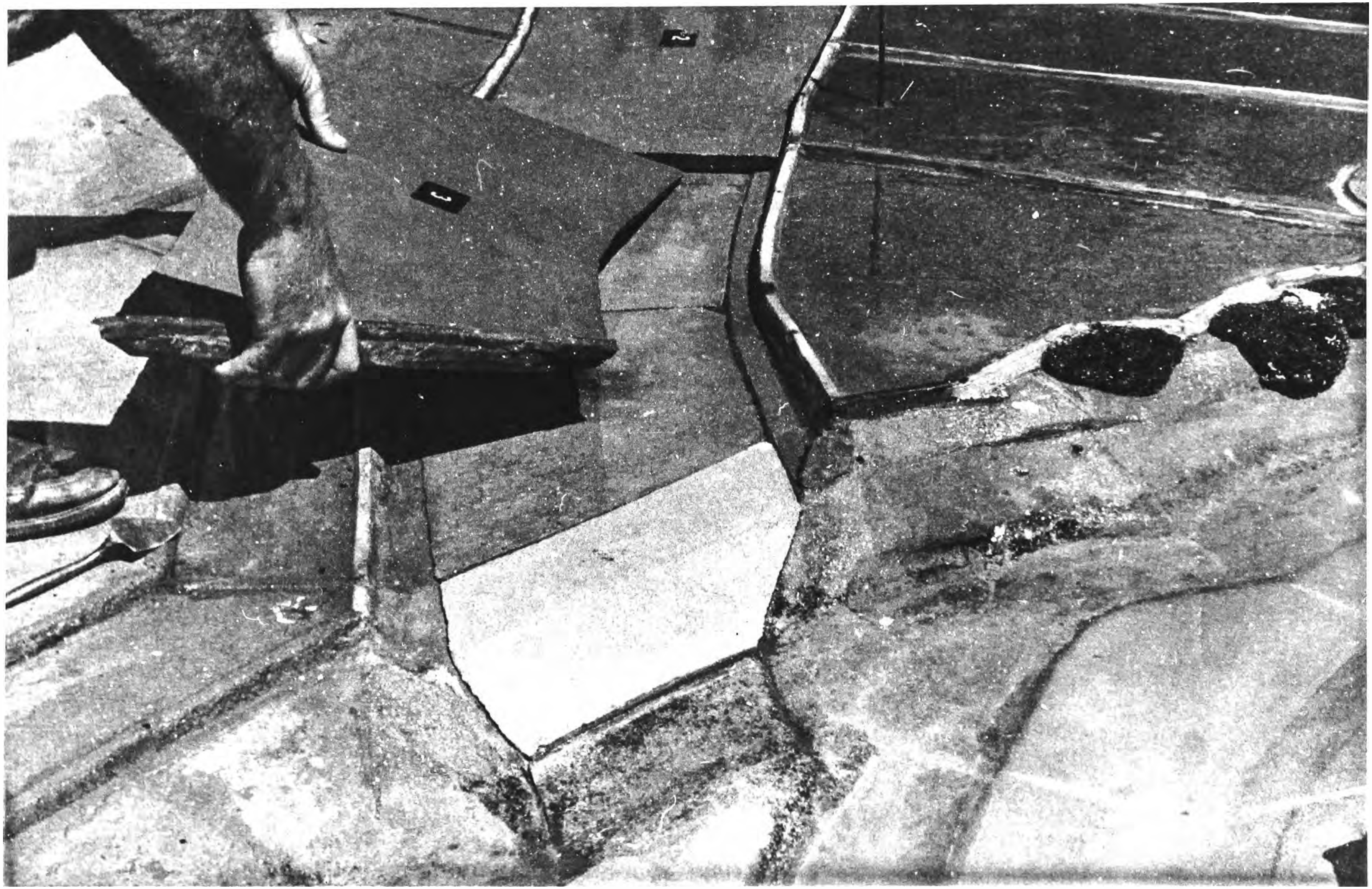

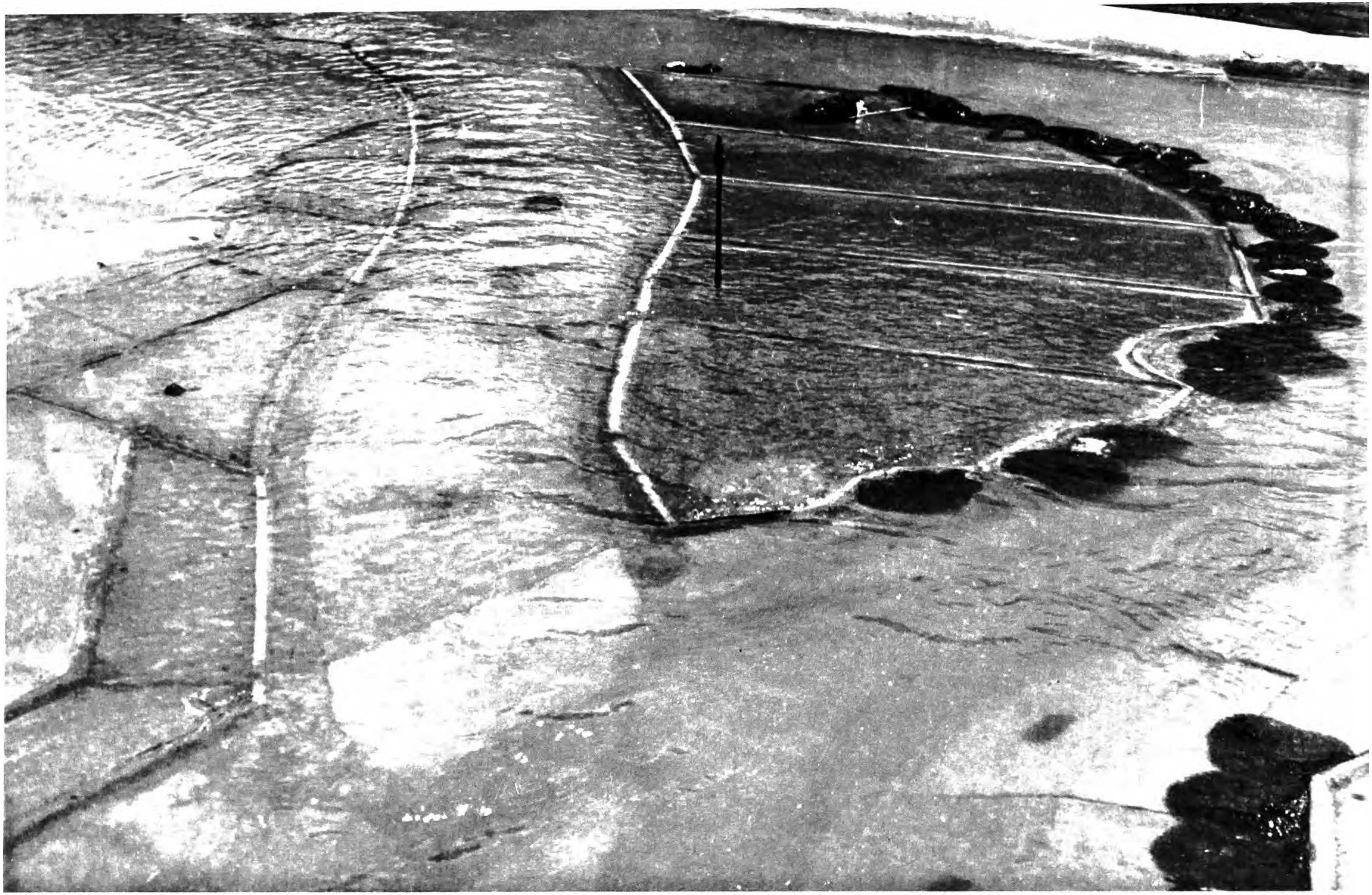

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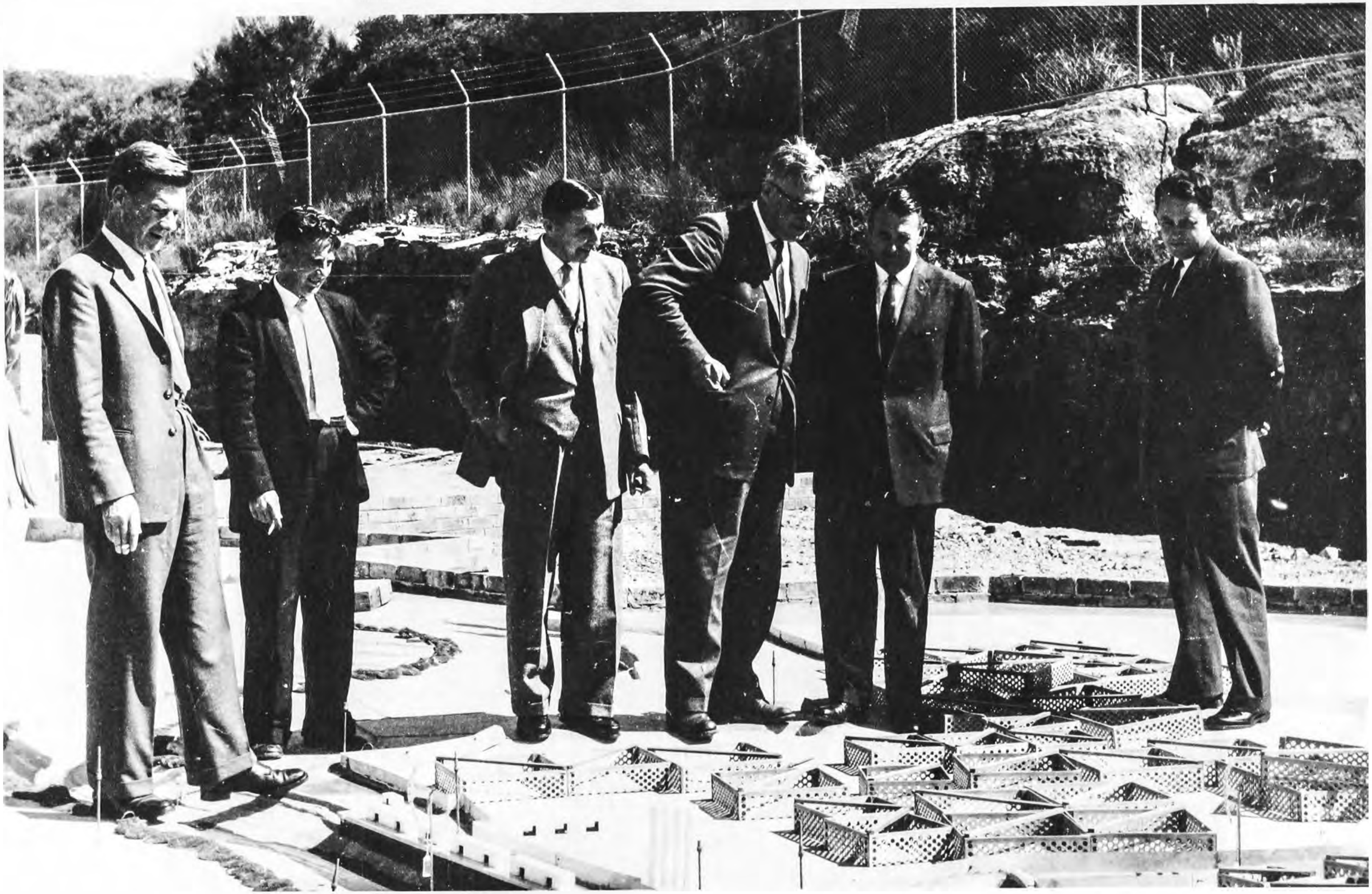

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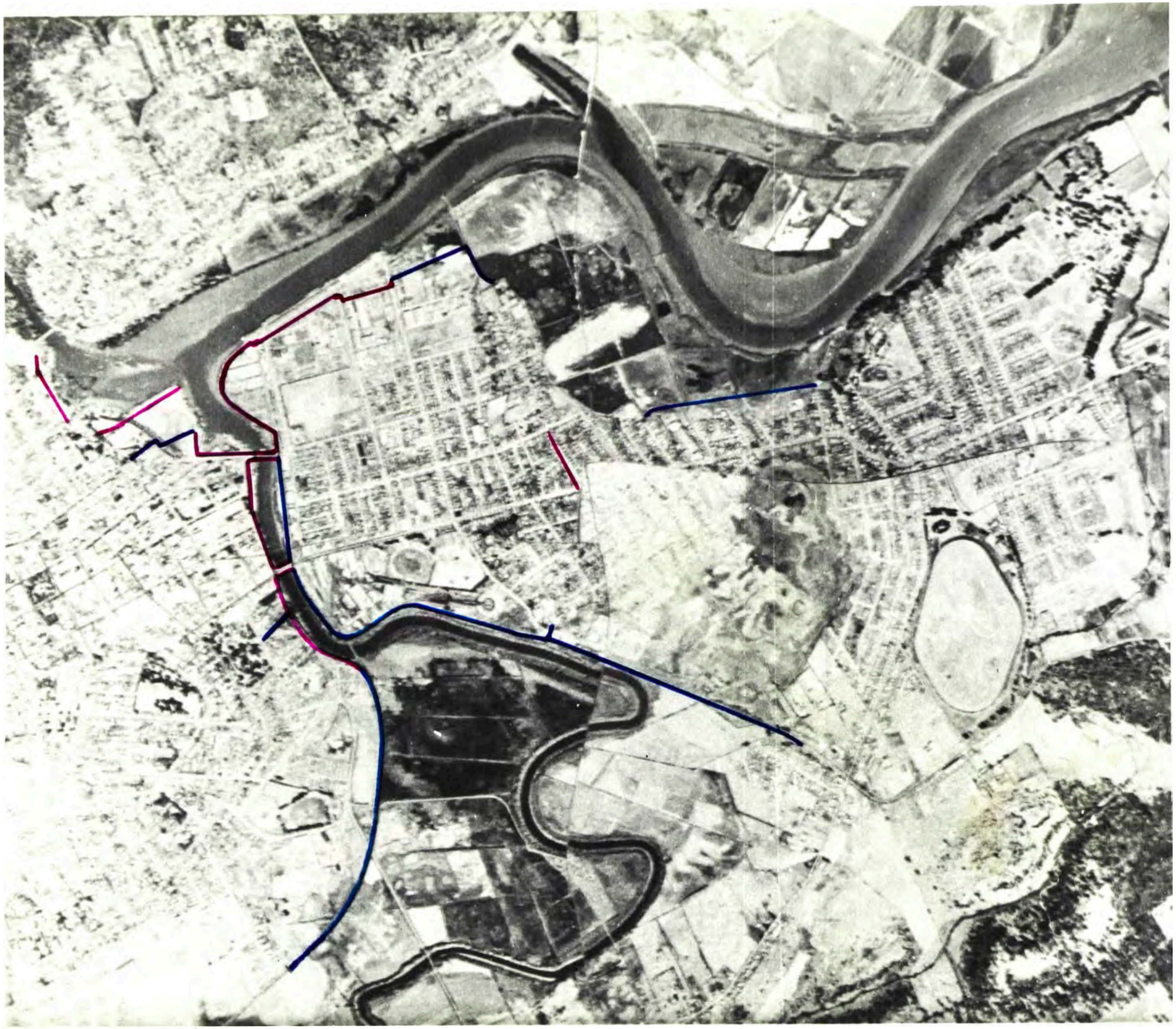
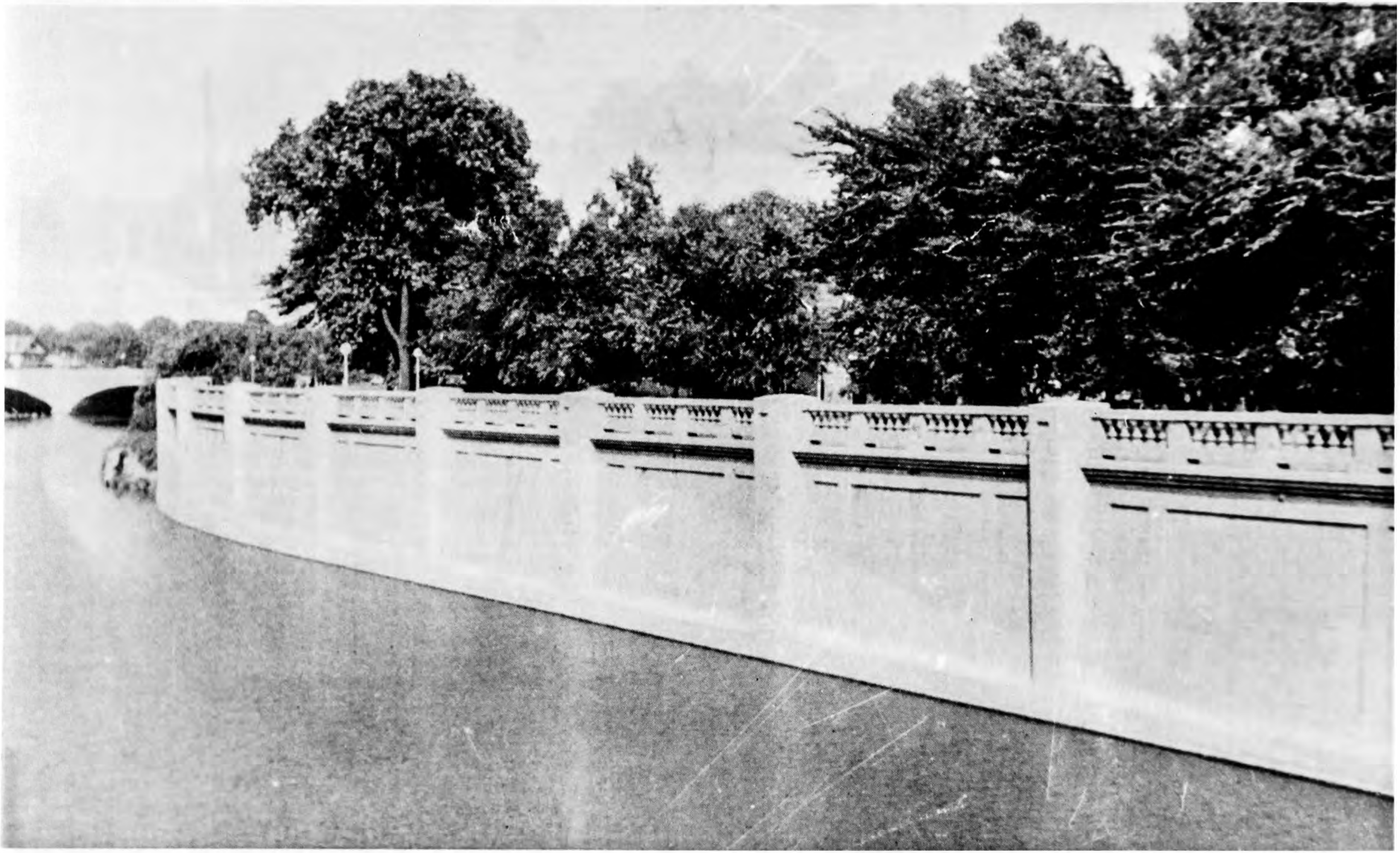


Photograph No. 27

Photograph No.29



Photograph No.29

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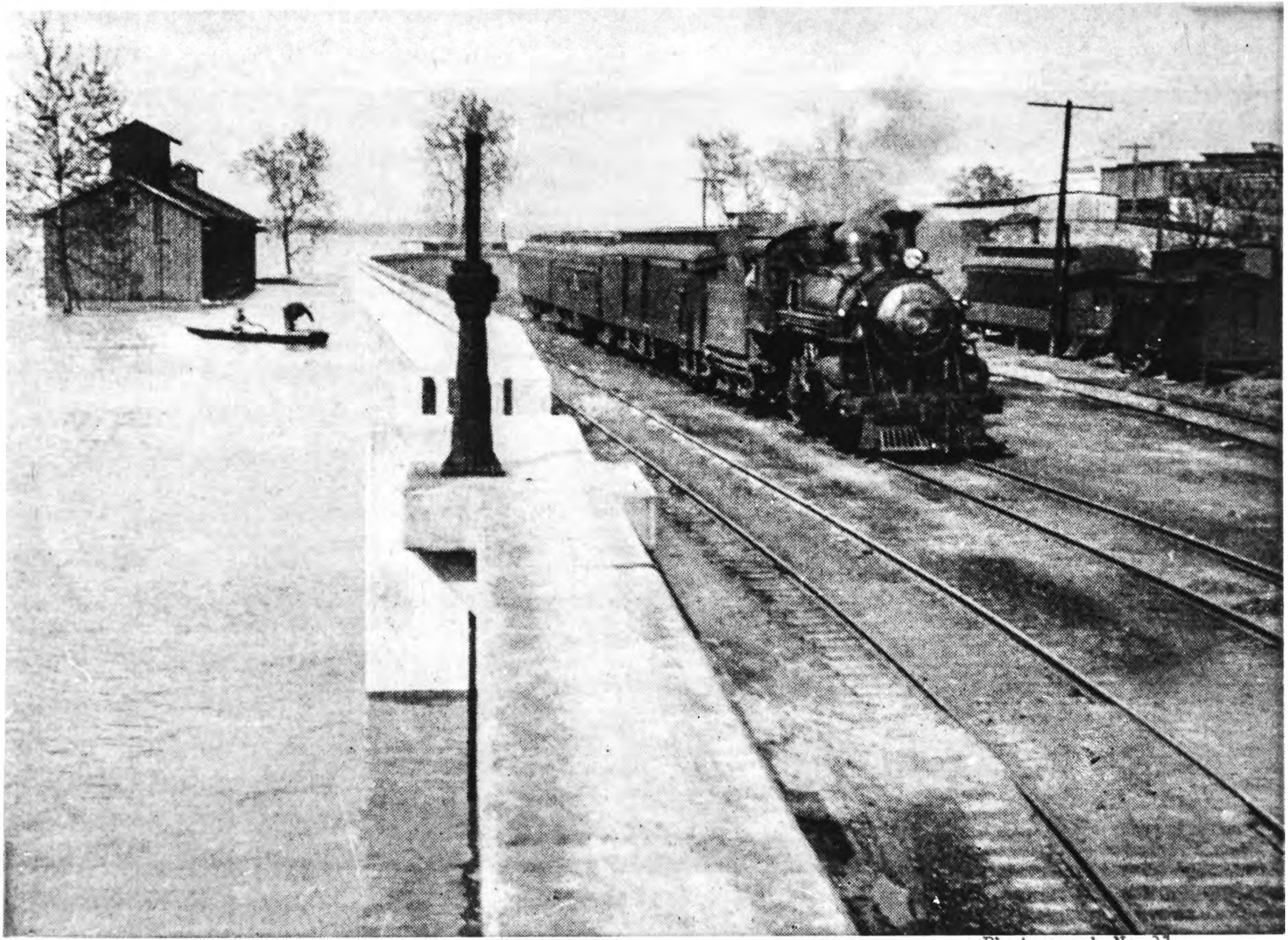

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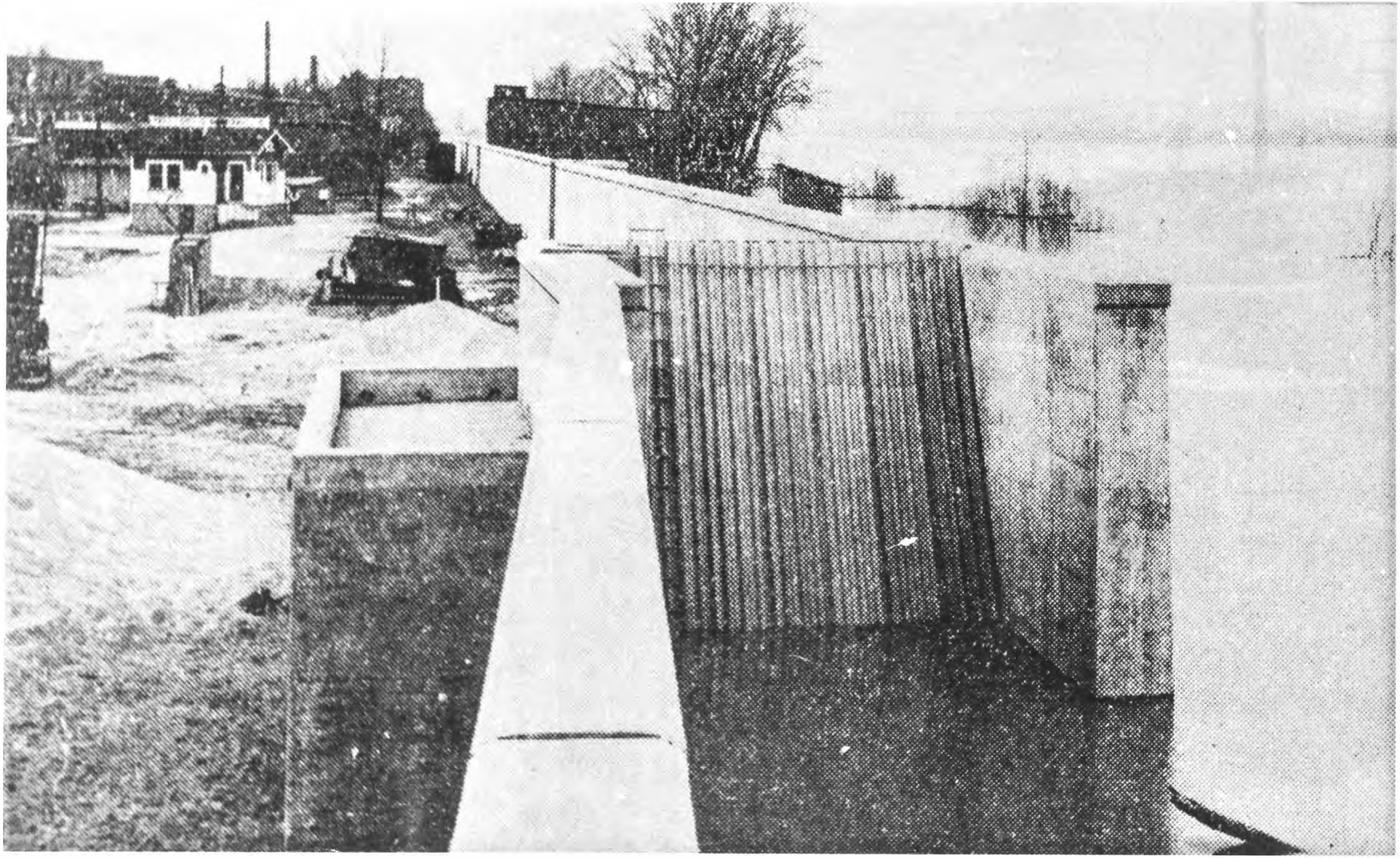
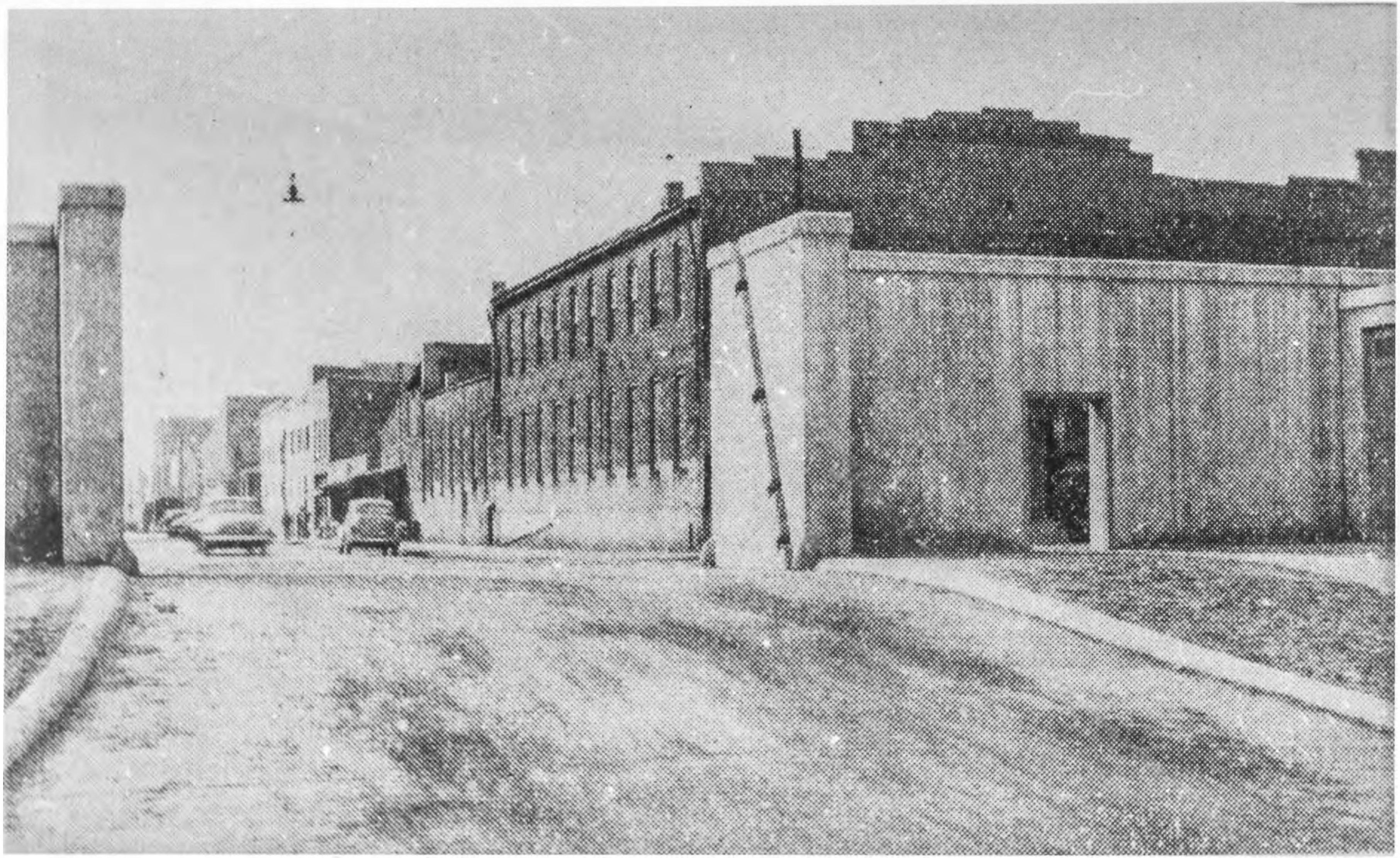


Photograph No.32

Photograph No.33



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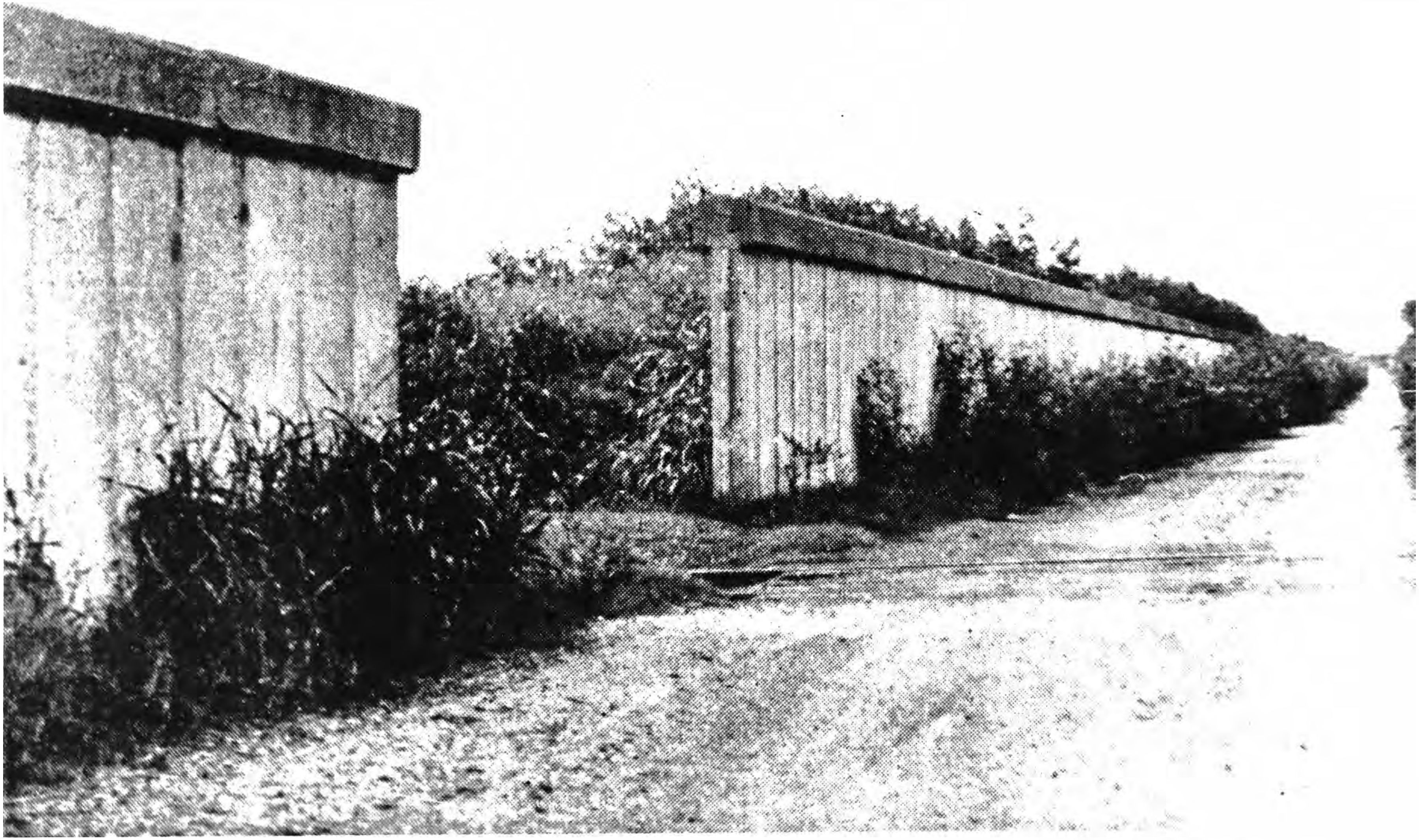
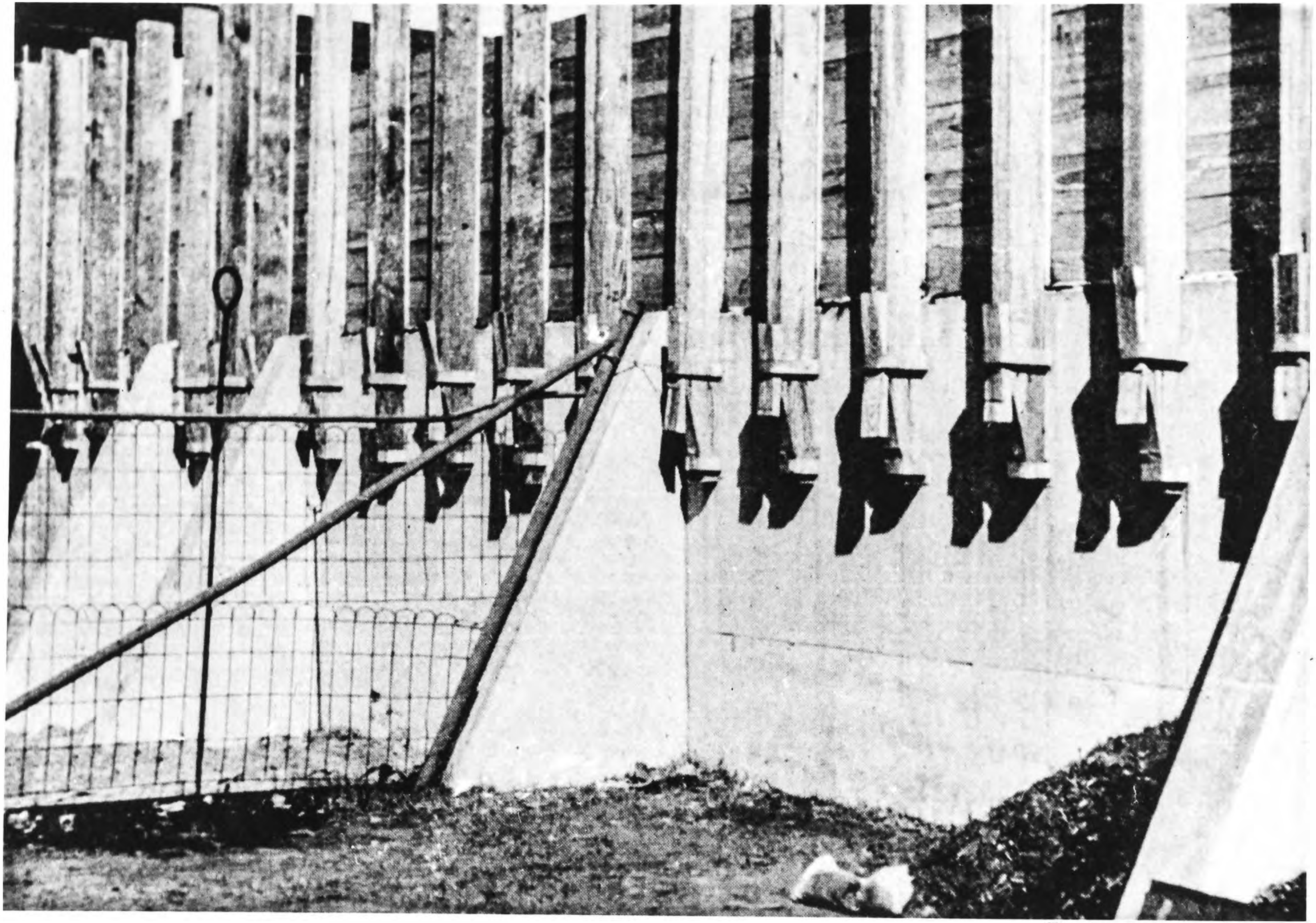


Photograph No.35

Photograph No.36



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