THE UNIVERSITY OF NEW SOUTH WALES

WATER RESEARCH LABORATORY

REPORT No. 9.

Supplementary Report on Flood Mitigation Measures for the City of Launceston

by

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OCTOBER, 1959

SUPPLEMENTARY 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 hydraulic model tests undertaken to supplement the investigations previously carried out and published in Report No.8 "Flood Mitigation Measures for the City of Launceston", September 1959.

The use of curved training levees located in Royal Park and breakwaters in the Tamar River to divert the South Esk river are investigated as alternative proposals to the straight training levee in Royal Park previously recommended in Report No.8.

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LAUNCESTON FLOOD PROTECTION AUTHORITY

SUPPLEMENTARY REPORT BY PRINCIPAL EXECUTIVE OFFICER.

1. INTRODUCTION

In accordance with the recommendation contained in Clause 7(j) of the Final Report of the Launceston Flood Protection Authority dated September 1959, supplementary hydraulic model tests have been carried out and are described and analysed hereunder.

2. HYDRAULIC ASPECTS OF POSSIBLE VARIATIONS OF THE PROPOSED SOUTH ESK DIVERSION STRUCTURE

2.1 Description of Tests

Tests were carried out on the model for the various alternative types of diversion structures listed in column 2 of Table S1 attached.

2.11 Height of Straight Royal Park Levee (Test Series 7)

The first series of tests in this table S1 is a refinement of model tests No.7 of Section D6.35 of my original report. In these supplementary tests, a straight levee was located in Royal Park as shown in Fig.37 of Vol.2 of my original report, but the height of the levee was progressively lowered to determine what effect such lowering would have upon the height of the surround levees necessary to protect Launceston from the "probable maximum flood". Such lowering, of course, involves the overtopping of the levee, but this is not considered to be harmful as there will be slack water on the eastern side of the levee. Therefore, damage to the earth levee would be quite minor, and the concrete levee, which is the basic diversion structure, would be quite unaffected.

2.12 Effect of Curving Royal Park Levee (Test Series 17)

Test Series 17 and 18 were designed with the objective of improving the methods of diversion of the South Esk jet in the following respects:-

(a) To increase the hydraulic efficiency of the diversion mechanism.

(b) To reduce the high velocity flow along King's Wharf which occurs for the "probable maximum flood" discharge when a straight training levee is used.

(c) To reduce the surge action along the Inveresk-Invermay levee between Charles St. Bridge and King's Wharf, thus reducing the required height of these levees.

Tests were carried out only for the "probable maximum flood" as it is considered that this is the only consideration requiring investigation. Two locations were tried, as shown in Figs. S1 and S2.

2.13 Effect of Curved Breakwater in Tamar Channel (Test Series 19 and 20).

Test No.19 is designed to investigate the effectiveness as a diversion structure of a curved breakwater from King's Bridge to the northern end of Royal Park as shown in Fig.S3 attached. Such a diversion structure makes possible the reclamation of considerable areas of the upper reaches of the Tamar for increase in park area and provision for better road approaches to the proposed new bridge over the Tamar at this point.

This test showed that greater efficiency could be obtained by bringing the northern end of the curved breakwater further west, and test series 20 was then carried out with the breakwater located as in Fig.S3 and with the northerly terminating point of the breakwater at progressively shorter distances from Cataract Bridge.

For tests 19 and 20a South Esk discharges of 150,000, 200,000 and 250,000 cusecs were used, but for the remaining tests only the "probable maximum flood" discharge was considered.

2.14 Effect of Groynes

Various arrangements of groynes projecting from the bank of the Tamar in the vicinity of the Yacht Club were tried out, but it was obvious that their effect in diverting the S.Esk discharge was not appreciable, and no quantitiative tests were made.

2.2 Results of Tests

Results of tests are summarised in Table S1 attached. For each proposal flood levels were measured at all relevant points to determine its efficiency in flood mitigation. Results are shown in Tables S3 to S5 attached, and have been summarised for flood levels in the Upper North Esk area, at Tamar St.Bridge and in Home Reach in Table No.S1 attached for the "probable maximum flood". In addition, velocity measurements were taken where necessary.

2.3 Discussion of Results

2.31 Lowering of Height of Straight Training Levee

Test No.7 of attached Table S1 shows that, for a straight levee, an adopted levee height of RL.32 is the best combination of economic and hydraulic efficiency. This provides for 5 feet depth of water over the leveel. For this height of straight training levee, the required levels of surround levees to protect Launceston would be the same as those required when a non-overtoppable straight levee is used.

2.32 Use of Curved Training Levee in Royal Park

However, test series 17 and 18 indicated clearly that a curved overtoppable training levee in Royal Park as shown in Figs. S1 and S2 is more efficient than a straight training levee at the same general location. It will be noted from Table S1 that if a curved training levee in Royal Park is located as in Fig. S1 attached, the heights of the surround levees for the protection of Launceston above Charles St. Bridge would be, in general, one foot lower than the heights necessary if a straight training levee at RL.32 in Royal Park were used. Obviously levee heights along the bank of the Tamar would be unchanged.

This is a most important result, and means that the use of a curved training levee in Royal Park will reduce the heights and costs of surround levees below the values given in my report of 15th August 1959.

It will be noted from the Upper North Esk values in Table S1 that if the curved training levee in Royal Park is located as shown in Fig. S2 attached, the surround levees can be reduced in height by a further 0.5 feet (22.5-22.0). This location, however, involves a slight projection of the curved levee into the North Esk River at its confluence with the Tamar. It is understood that the channel used by island shipping proceeding to the North Esk Wharves follows closely the southern bank of the North Esk and this projection might be a slight disadvantage. However, it is felt that this is not a material factor. A more serious objection to this proposal is the fact that relatively expensive sheet piling would be necessary for the northern extremity of the levees for the last 300 ft. of construction.

2.33 Use of Curved Breakwater in the Tamar Channel

The location of the curved breakwater shown in Fig. S4 attached for test series 20 is obviously better than the location for test series 19, which is given in Fig. S3. Taking the test results at their face value, it is clear that a curved breakwater located as in Fig. S4 and of length 1815 feet is hydraulically a most efficient diversion structure. Table S1 indicates that for Upper North Esk this gives a spectacular reduction of 2.6 ft. in the height of the surround levees adjoining the North Esk River as compared with the height (23.5) for a straight overtoppable levee of RL.32 in Royal Park. This is undoubtedly due to the fact that a kind of "ejector" action occurs at the confluence of the North Esk and Tamar Rivers, due to the high velocity jet from the South Esk. On account of this "ejector" action, flood levels in Home Reach for this arrangement are 2 feet higher than flood levels in the North Esk River.

In considering the validity of these results, it must be borne in mind that they are obtained from a distorted scale model. For such a model, the total discharge past a given cross section and the stage discharge relationship are correctly represented, but the velocity distribution across the cross section may not be exactly simulated. In other words, the model has demonstrated qualitatively that an "ejector" action is occurring, but we cannot be certain that it has expressed this action quantitatively. It is quite probable that the distorted model gives flood levels which are accurate enough for this investigation, but no certainty could be felt on this point unless a separate undistorted model is made of this particular area and the tests repeated on this model. If such a model were constructed, it might be found that the reduction in height of the surround levees was say 3.3 feet or 1.3 feet instead of 2.3 feet. Whether it is worthwhile making this additional model depends upon the economic aspects discussed in Clase 3 below.

There is another point on which the quantitative effect of the ejector action may be questioned. At the time of the "probable maximum flood", considerable scour may occur in the Tamar near the confluence of the North Esk River. If so, stream velocities would be reduced and consequently the efficiency of the ejector action would be lowered. Movable bed model tests to measure this effect are not possible because of the lack of prototype data. An alternative method of assessing this effect is to carry out borings or probings to determine the depth of the likely final bed contours, assuming that all silt has been scoured out, and use this bed shape in the model. The results would be on the "safe" side.

2.34 Velocity Measurements

For a curved training levee in Royal Park, the velocities in the vicinity of the training levee are similar to those given in Tables 14 and 15 of my September report, while those along King's Wharf are 10 ft. per second at the Northern end, and 3 ft. per second at the southern end for the "probable maximum flood".

The velocity measurements for a curved breakwater as in Test Series 20 are given in Fig.S4, and it will be noted that a very high velocity of 30 feet per second is obtained opposite the mouth of the North Esk River. It is considered that either with the curved training levee or the curved breakwater, velocities in the region of King's Wharf are reasonable, bearing in mind that the "probable maximum flood" has little chance of occurring in the next 100 years.

3. ECONOMIC ASPECTS OF VARIOUS ALTERNATIVE DIVERSION STRUCTURES.

3.1 Capital Cost of Original Proposal

At the meeting of the Authority held on 17th September, my estimate of cost of £84,700 for a straight training levee was increased to £100,000 on the grounds that the western side of the training levee should be protected against scour. Although I recognise that the final decision on such matters rests with the Authority, I wish to make it clear that I adhere to my original recommendation and estimate of cost. I consider that protection against scour, other than grassing, is unnecessary for the following reasons:-

(a) The concrete levee in Royal Park is quite adequate in itself to divert the South Esk Jet and the earth embankment on the western side is provided only for aesthetic reasons. The concrete levee is designed to withstand a static hydraulic head equal to the height of the levee. In actual fact, under "probable maximum flood" conditions, this concrete levee will be subjected to a static head approximately 7 feet less than it has been designed for, because of the backwater on the eastern side of the levee. This fact, combined with the weight of the earth on the eastern side of the levee, was considered to be adequate provision for impact force. I do not concede that under "probable maximum flood" conditions the earth embankment on the western side will be completely scoured away, although it may be somewhat damaged. However, even if it were to completely disappear, the structure would still be stable while the flood is in progress. Admittedly, if the embankment on the western side is completely washed away, after subsidence of the flood the concrete levee is supporting the earth embankment on the eastern side without any balance of pressure by water or earth on the western side. Examination of this condition may show that a re-design of the concrete levee is necessary to provide appropriate reinforcement. This would not, in my opinion, affect materially the estimated cost of the concrete levee.

In any case, it must always be remembered that the "probable maximum flood" is most unlikely to occur in the next 100 years, so that the danger of complete removal of the earth bank on the western face of the levee is slight.

I therefore feel that in comparing the various projects from an economic point of view, my original estimates of cost should be used, and in the discussion hereunder I have adopted that view.

3.2 Straight Overtoppable Training Levee

As the curved overtoppable training levee is more efficient and not much longer than the straight overtoppable levee, there is obviously no point in computing the estimates of cost for a straight training levee of reduced height.

3.3 Curved Overtoppable Training Levee in Royal Park

A detailed analysis and estimate of costs for the two possible locations of this training levee would need to be carried out by the Authority responsible for the final construction of the flood mitigation works. With a training levee located as in Fig.S2 for test 18, the cost of the training levee will be greater than that of the training levee located as in Fig.S1 for test 17 because of the need for sheet piling. On the other hand, the adoption of the location of test 18 would result in the reduction in height of the North Esk surround levees of approximately 0.7 feet. Detailed estimates of cost of the two alternative proposals would need to be carried out by the Authority ultimately responsible for the design of the proposed flood mitigation works.

3.4 Curved Breakwater

Having carried out the economic analysis referred to in 3.3 above, it is necessary for the Constructing Authority to make detailed estimates of costs for surround levees and breakwater construction for the various lengths of breakwater and the best of these curved breakwater proposals must be compared with the best of the curved Royal Park training levee proposals.

As a very rough and approximate approach, it might be assumed that the breakwater for test series 20e of Table S1 is the most efficient breakwater proposal. Mr.Edwards has supplied designs and approximate estimates of cost for a breakwater 2100 ft. long. Details of Mr.Edward's report are summarised in Appendix A, the estimated cost being £252,500. Assuming that the reduction in the length of breakwater to 1815 feet reduces the cost of the breakwater to £220,000, an approximate comparison of capital cost is as follows:-

Proposed Diversion Method	Cost of Diversion Works	Cost of Surround Levees	Total Costs
1. Original Proposed straight non-over-topped levee	£84,700	£560,000	£644,700
2. Curved training levee as shown in Fig. S1	£112,600	£476,000	£588,600
3. Curved training levee as shown in Fig. S2	£161,100	£449,000	£610,100
4. Breakwater of alignment shown in Fig. S4 and length 1815 ft.			
(i) Assuming full ejector effect as shown by model	£220,000*	£389,000	£609,000
(ii) Flood levels in N. Esk River increased by 0.8 ft. as factor of safety for scale effects	£220,000*	£459,000	£679,000

* These costs are for provision of breakwater for flood protection only and do not include any reclamation costs on the eastern side.

It should be noted that if the curved breakwater proposal is used, there will be no need to raise the Paterson St. levee, and this saving of £17,000 has been allowed for in the above tabulation.

It is stressed that the estimates of costs of the surround levees in the above tabulation are of an approximate nature only. Detailed revision of quantities would be a major task, and are more fittingly carried out by the Constructing Authority.

It is obvious, however, that <u>for flood mitigation purposes only,</u> proposal 2 is the best, costing £20,400 less than proposal 4 (i).

It is possible that the cost of the curved breakwater will be greater than anticipated by Mr. Edwards and the difference in cost may be greater than £20,400. He has made it clear in his report that his estimates are very approximate and no borings have been taken and no detailed designs have been made or quantities calculated. As Mr. Edwards has pointed out in Appendix A, the estimate for the curved breakwater is for that work required for flood protection, as illustrated by the left hand section of Proposal II shown in Fig. S5. It will be seen that the ultimate development for parklands and bridge approaches provides for rock protection on the river face, reclamation behind the wall and the provision of anchor ties back into the reclamation as shown by the right hand section of Proposal II of Fig. S5. The ultimate cost of such works would be much greater than £220,000, and would form part of the cost of construction of the bridge.

In regard to the question of whether an undistorted model should be constructed, this is a matter for the Authority responsible for the design of the bridge. The scale would be 1:150 or thereabouts, and the area covered would be from Charles St. Bridge around Royal Park to Cataract Bridge, and down the Tamar about as far as the northern end of King's Wharf. The cost would be somewhat as follows:-

(i)	Prototype borings - from	£1,000 to £3,000
(ii)	Construction of model - from	£1,000 to £2,250
(iii)	Testing and analysis of results - from	£300 to £600
(iv)	Incidentals	£250 to £300
	Totals -	£2,550 to £6,350

Much of the boring expenditure would be necessary in any case if a curved breakwater is to be designed, so that the net cost of determining beyond question the required height of surround levees would be between £1,550 and £3,350.

This model is unnecessary for flood mitigation purposes only, but if scheme 4 is adopted for the reasons given in Clause 3.5 below, these additional model tests are a good business proposition. In fact they are really essential for a proper engineering investigation, as the difference in cost between 4(i) and 4(ii) is £70,000.

3.5 <u>Intangible Benefits</u>

I readily concede that if the whole problem is looked at from the point of view of the Government and public corporations of Tasmania, bearing in mind amenities such as parks, and transport facilities such as the new bridge across the Tamar River, the curved breakwater proposal may be considered to have intangible benefits which render it preferable to the curved training levee proposal.

So far as flood protection is concerned, there is also an intangible benefit arising from a lowering of surround levee heights. High levees are undesirable aesthetically and this intangible benefit caused by the curved breakwater might be considered in relation to the flood mitigation aspect. Unfortunately the curved breakwater does not reduce levee or flood gate heights along the banks of the Tamar where there are some relatively high sections.

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.

4. RECOMMENDATIONS

4.1 Introduction

The Launceston Flood Protection Authority was created for the purpose of recommending methods of flood mitigation for the City of Launceston, and a very satisfactory method has been developed based on a curved training levee in Royal Park and surround levees for the various zones of the City.

From the point of view of flood protection only, the complications of the breakwater proposal are such that the curved Royal Park levee is obviously a better recommendation.

This is the end of the matter, so far as the Launceston Flood Protection Authority, (the investigating authority), is concerned.

However, the intrusion into the problem of the proposed Tamar Bridge means that it is not the end of the matter, so far as the general public of Tasmania is concerned.

It is quite fitting that the Authority charged with the preparation of detailed working drawings and the construction of flood protection works should collaborate with the Public Works Department of Tasmania and the Launceston City Council in an investigation as to ways and means by which the flood protection works may be integrated with the design of the approaches to the proposed new bridge.

The only concern of the Launceston Flood Protection Authority in such matters of final design and construction is that the basic principles of its recommended proposals should be followed.

It is quite conceivable that the direct tangible benefits of lower costs of bridge construction, and the intangible benefits of improved parkland amenities may justify the adoption of one of the breakwater proposals of Test Series 18 reported herein instead of a curved training levee in Royal Park, but it would be inappropriate for the Launceston Flood Protection Authority to express any opinion on this point.

My recommendations hereunder are based on the above appreciation of the situation.

4.2 Method of Protection to be adopted if no account is to be taken of the New Tamar Bridge.

4.21 Degree of Protection

The benefit-cost ratio of "5.15" of Clause 7(b) should be amended to read "5.70" in the Final Report of the Authority of September 1959.

4.22 Diversion Levee

Clause 7(c) of the Final Report of the Authority dated September 1959 should be amended to read:-

"The first step in flood mitigation should be the construction, at an estimated cost of £112,600, of a curved training levee in Royal Park of location and height shown in Fig.S1 attached consisting of a concrete core wall as in Fig.40 of Vol.2 of the Final Report of the Authority of September 1959, covered by a grassed earth embankment as in Fig.41.

4.23 Surround Levees

(a) Clause 7(d) of the Final Report should be amended by replacing the words "Table 27 Vol.II of the report by the Principal Executive Officer" by the words "Table S2 of the supplementary report by the Principal Executive Officer dated 16th October 1959.

(b) In Clause 7(d) (ii) the figure "24.5 ft." should read "23.0 ft.".

(c) The estimated capital costs of clauses 7(d) (ii) to 7(d)(v) should be amended thus:-

(ii)	£242,000	should	read	£213,000
(iii)	£127,000	"	"	£100,000
(iv)	£93,000	"	"	£75,000
(v)	£81,000	"	"	£71,000

4.24 Model Tests

Clause 7(j) should be replaced by:-

"The attention of the Director of Public Works be drawn to the fact that if the construction of a curved breakwater in the channel of the River Tamar is desirable for the efficient design of approach roads to the proposed new bridge across the Tamar from Royal Park and for the provision of additional parklands such breakwater could be designed to act as the diversion structure of the proposed flood mitigation works but that any such dual-purpose design must accord with the fundamental principles of our Final Report and this supplementary report, and must be based upon the data contained in the reports submitted by the Principal Executive Officer dated 15th August and 16th October 1959.

4.3 Method of Flood Protection to be adopted if the design of the proposed new bridge and increased parklands are to be incorporated in the Flood Protection scheme.

If the tangible benefits of lower cost of bridge construction and the intangible benefits of improved parklands and bridge approach roads are deemed by the Government of Tasmania to justify blending the flood protection diversion structure into the bridge construction, the Authority responsible for the design of the bridge should be instructed to base the design on the following fundamental considerations:-

(i) The general flood levels above the confluence of the North and South Esk and Tamar Rivers shown in Tables S3 to S5 attached to this report should be increased by at least 0.8 ft. for curved breakwater proposals as a factor of safety to provide for the contingency that the distorted scale model used in the hydraulic tests may not reproduce with sufficient accuracy the quantitative effects of the "ejector" action, unless a supplementary undistorted model is constructed to evaluate exactly this effect.

(ii) The South Esk diversion works must be adequate to divert the estimated "probable maximum flood" of the South Esk River.

(iii) The surround levees must be adequate to protect the various zones from the estimated "probable maximum flood" after making due allowance for the effect of the diversion structure of (ii) above.

(iv) The final design of the diversion structure and surround levees must accord with the basic principles of my report dated 15th August 1959.

C.H. Munro
Principal Executive Officer
16th October 1959.

LAUNCESTON FLOOD PROTECTION AUTHORITY

TRAINING WALL ROYAL PARK
PROPOSAL IIA

ASSUMPTIONS:

(1) Q = 250,000 cfs and profile parallel to line. Fig.36 - Vol.II Principal Executive Officer's report.

(2) Max. top level R.L. 37.5

(3) Min. top level R.L. 27.5

(4) Average top level R.L. 33.0

(5) Average level of river bed existing on line of wall R.L. 5.0

(6) Assumed average pile penetration 20'0"

(7) Purely a deflector wall and not a retaining structure.

(8) Back pressure available from water behind the wall and for rough design a hydraulic head of 10'0" assumed.

(9) Total length of wall 2,100 feet.

(10) Sheet piling - B.H.P. 50 Section.

(11) Walings 190/' - - 2 B.H.P. semi-finished 9-1/4" x 9-3/8" R.S.S's.

(12) Raking piles of local hardwood will preserve line treatment by pressure impregnation.

ESTIMATE

(1) Steel Piling - 50 Supply

$\frac{2,100 \times 58 \times 39.02}{2240}$ at £54/ton = £115,000

(2) Steel Pile Driving

$\frac{2,100}{1.28} \times 58$ at 8/-d./foot = £38,000

(3) Steel Pile Handling

$\frac{2,100 \times 58 \times 50}{2,240}$ at £2/ton = £2,720

(4) Steel Pile Treatment and Painting

2,100 x 45 x 2 at 1/6d./sq' = £14,150

(5) Raking Pile Driving and Supply

$\frac{2,100}{12.5} \times 65$ at £1.67/1' = £18,200

(6) Raking Piles - Trimming and Fixing

$\frac{2,100}{12.5}$ at £30 each = £5,050

(7) Temporary Staging = £5,000

(8) Walings in Place

$\frac{2,100 \times 190}{2240}$ at £80/ton = £17,800

(9) R.C. Coping Beam

$\frac{2,100 \times 2.5 \times 2}{27}$ at £35 per cu.yd. = £13,600

£229,520

(10) Misc.items and contingencies
10 per cent 22,952

Total - £252,472

TABLE No. S1.

SUMMARY OF MODEL RESULTS.

FLOOD LEVELS AT

UPPER NORTH ESK, TAMAR ST. BRIDGE & HOME REACH

FOR MAXIMUM PROBABLE FLOOD

Test No.	Elements of Plan Tested	Flood Levels at Upper North Esk	Flood Levels at Tamar St. Bridge	Flood Levels at Home Reach
7	Training Levee with Inveresk and Invermay Levees and also City Levees			
	(a) Non overtopped Levee. Peak Water Level = RL 37 at Top of Levee	23·6	23·2	23·1
	(b) Overtoppable Levee Peak Water Level = RL 37 Top of Levee = RL 34	23·5	23·2	23·15
	(c) " " Peak Water Level = RL 37 Top of Levee = RL 32	23·5	23·3	23·1
	(d) " " Peak Water Level = RL 37 Top of Levee = RL 29	24·3	24·0	23·1
	(e) " " Peak Water Level = RL 37 Top of Levee = RL 27	24·6	24·4	23·15
	(f) " " Peak Water Level = RL 37 Top of Levee = RL 25	25·6	25·3	23·2
17	Curved Training Levee. Located as in Fig. S1.	22·5	22·3	22·7
18	Curved Training Levee. Located as in Fig. S2.	22·0	21·6	23·1
19	Curved Breakwater. Located as in Fig. S3.	22·5	22·2	—
20	(a) Curved Breakwater. Located as in Fig. S4. Length - 2440 ft.	20·8	20·3	22·9
	(b) " " Length - 2190 ft.	20·9	20·4	22·9
	(c) " " Length - 2065 ft.	21·0	20·6	23·1
	(d) " " Length - 1940 ft.	21·3	20·2	23·1
	(e) " " Length - 1815 ft.	21·2	20·7	23·1
	(f) " " Length - 1690 ft.	21·7	21·3	23·1

TABLE No S2

PROPOSED LEVEE DETAILS FOR PROTECTION AGAINST MAX PROBABLE FLOOD USING CURVED TRAINING LEVEE SHOWN IN FIG S1

Location	Chainage (ft)	Flood Level (ft above SLW)	Surge Amplitude from Model (ft)	Proposed Freeboard (ft)	RL of Top of Levee (ft above SLW)	Approx Present Ground or Embankment level (ft above SLW)	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
SW 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	22.2	1.0	1.0	23.2	17.0	6.2
Tamar St Bridge	7700	22.3	Neg	0.5	22.8	19.0	3.8
Railway Bridge over N Esk	9000	22.5	Neg	0.5	23.0	18.0	5.0
At intersection of Herbert St levee	11860	22.5	Neg	0.5	23.0	18.5	4.5
Herbert St	12100	22.5	Neg	0.5	23.0	23.0	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

Location	Chainage (ft)	Flood Level (ft above SLW)	Surge Amplitude from Model (ft)	Proposed Freeboard (ft)	RL of Top of Levee (ft above SLW)	Approx Present Ground or Embankment level (ft above SLW)	Approx Levee Height (ft)
ROYAL PARK - WILLIS ST ZONE							
Royal Park at basement of Police Station	0	22.2	1.8	1.5	23.7	23.7	0
Canal St at Royal Park NW cnr Gunns Timber Yard	510	22.2	1.8	1.5	23.7	15.0	8.7
SW Cnr Harraps Wool Store	1010	22.2	1.8	1.5	23.7	17.3	6.4
NW Cnr Harraps Wool Store	1115	22.2	1.8	1.5	23.7	15.2	8.5
NE Cnr Harraps Wool Store	1455	22.2	1.2	1.0	23.2	15.0	8.2
Tasmanian Pattern Makers	1900	22.2	1.2	1.0	23.2	13.6	9.6
Charles St Bridge	2380	22.2	1.2	1.0	23.2	18.2	5.0
Shields St	3450	22.3	Neg	0.5	22.8	16.1	6.7
Tamar St Bridge	4190	22.3	Neg	0.5	22.8	19.2	3.6
At intersection of Willis St levee	5150	22.4	Neg	0.5	22.9	17.0	5.9
Cimitiere St	5900	22.4	Neg	0.5	22.9	22.9	0
WILLIS-CYPRESS ST ZONE							
At intersection of Willis St levee	0	22.4	Neg	0.5	22.9	17.0	5.9
Railway Bridge over N Esk	1200	22.5	Neg	0.5	23.0	19.0	4.0
Henry St	2570	22.5	Neg	0.5	23.0	18.0	5.0
Cypress St	4500	22.5	Neg	0.5	23.0	23.0	0
MOWBRAY ST ZONE							
At intersection of Herbert St levee	0	22.5	Neg	Zero	22.5	18.5	4.0
Mowbray Swamp	2500	22.5	Neg	Zero	22.5	18.7	3.8
Mowbray Hill	3440	22.5	Neg	Zero	22.5	22.5	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

TABLE No. S3

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	15-38
19	Curved Breakwater Located as in Fig. S3.	18·6	18·6	18·5	18·4	18·5	18·5	18·4	18·4	18·2	—	—	17·8	18·4	18·2	Flood levels correspond to those shown in Table No. 10 Test 7 Volume 2. Sept. Report
20	Curved Breakwater 2440 ft. long Located as in Fig. S4.	18·1	18·1	18·0	17·9	17·9	17·9	17·9	17·6	17·6	—	—	17·8	18·3	18·0	

TABLE No. S4

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													
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15-38
19	Curved Breakwater Located as in Fig. S3.	21·4	21·4	21·4	21·3	21·3	21·2	21·1	21·1	21·0	—	—	19·9	19·3	19·5	Flood levels correspond to those shown in Table No. 10 Test 7 Volume 2 Sept. Report
20	Curved Breakwater 2440 ft. long Located as in Fig. S4.	19·9	19·9	19·8	19·8	19·8	19·8	19·8	19·4	19·4	—	—	20·1	21·3	21·1	

CE·E·844

MODEL RESULTS

SOUTH ESK DISCHARGE = 250,000 CUSECS

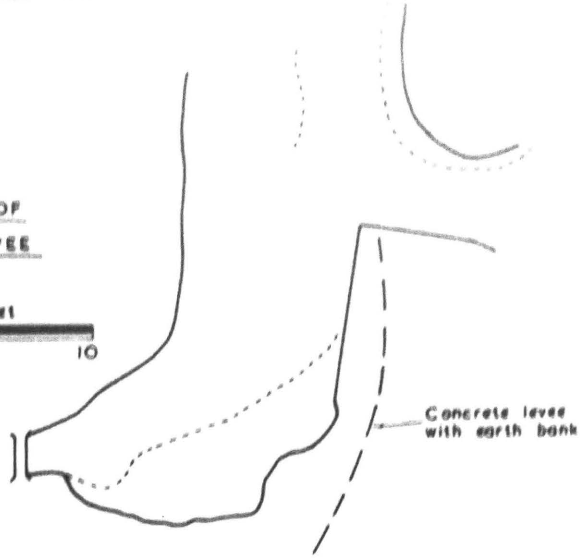
NORTH ESK DISCHARGE = 18,800 CUSECS

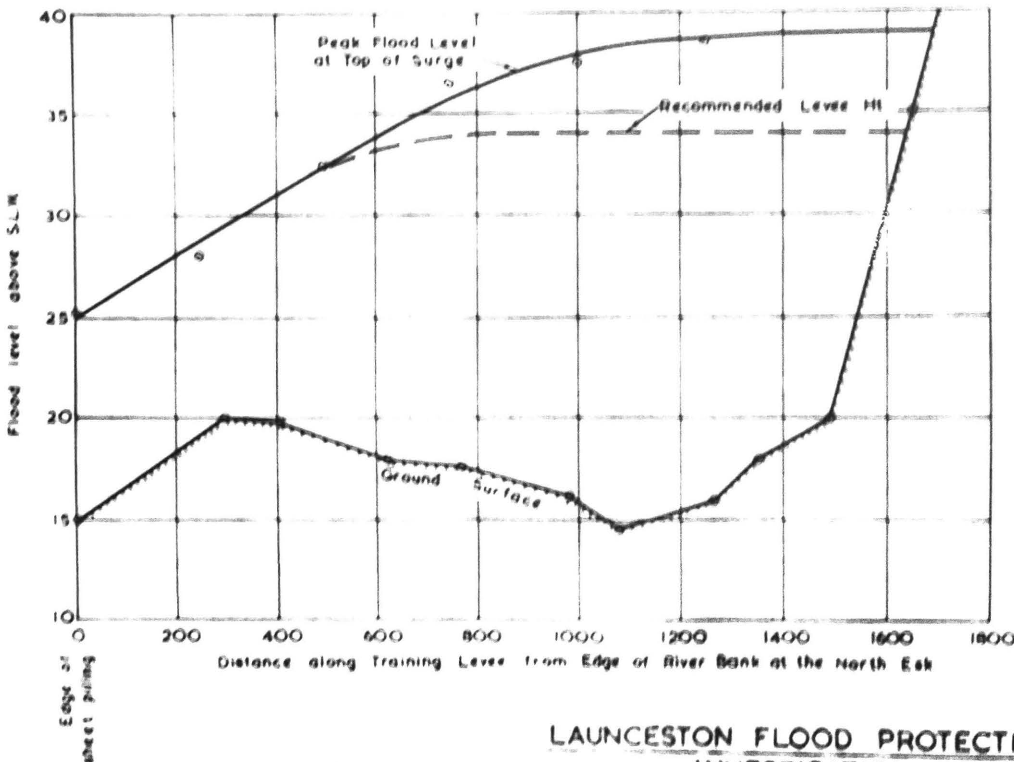
TEST No.	ELEMENTS OF PROPOSED IMPROVEMENT PLANS	FLOOD LEVELS AT GAUGE No.																						
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15-38	39	40	41	42	43	44	45	46
7	Training Levee with Inveresk and Invermay Levees and also City Levees (a) Non overtoppable Levee P.W.L = 37	23.6	23.6	23.6	23.6	23.6	23.6	23.6	23.2	23.2	–	–	22.5	23.1	23.1	Flood Levels correspond to those shown in Table No. 11 Test No. 7 of Sept. Report Volume 2	25.0	–	–	–	–	–	–	–
	(b) Overtoppable Levee P.W.L = 37 – T.L = 32	23.6	23.6	23.6	23.4	23.6	23.5	23.5	23.2	23.2	–	–	22.6	23.1	23.1		25.0	–	–	–	–	–	–	–
	(c) " " P.W.L = 37 T.L = 32	23.6	23.6	23.6	23.5	23.6	23.5	23.5	23.3	23.3	–	–	22.4	23.1	23.1		26.0	–	–	–	–	–	–	–
	(d) " " P.W.L = 37 – T.L = 39	24.4	24.4	24.4	24.3	24.3	24.2	24.3	24.0	23.9	–	–	22.5	23.1	23.1		26.0	–	–	–	–	–	–	–
	(e) " " P.W.L = 37 T.L = 27	24.7	24.7	24.6	24.6	24.6	24.6	24.7	24.4	24.4	–	–	22.9	23.2	23.1		26.0	–	–	–	–	–	–	–
	(f) " " P.W.L = 37 T.L = 25	25.7	25.7	25.7	25.6	25.6	25.6	25.7	25.3	25.0	–	–	23.1	23.3	23.1		27.0	–	–	–	–	–	–	–
17	Curved Training Levee Located as in Fig. S1.	22.6	22.6	22.6	22.5	22.5	22.5	22.5	22.3	22.2	–	–	22.5	23.2	22.1		24.5	–	27.0	27.0	28.0	26.5	25.0	24.5
18	Curved Training Levee Located as in Fig. S2	22.1	22.1	22.0	22.0	22.0	22.0	22.0	21.6	21.7	–	–	21.4	23.1	23.1		23.5	–	26.0	24.0	24.0	24.5	24.0	23.5
19	Curved Breakwater Located as in Fig. S3	22.6	22.6	22.5	22.5	22.5	22.5	22.4	22.2	22.2	–	–	–	–	–		29.0	–	–	–	–	–	–	–
20	(a) Curved Breakwater (Fig S4) 2440 ft. long	20.8	20.7	20.8	20.8	20.8	20.8	20.9	20.3	20.4	–	–	20.2	22.4	22.6		–	–	–	–	–	–	–	–
	(b) " " 2190 ft. long	20.8	20.8	20.8	20.8	20.8	20.9	20.9	20.4	20.4	–	–	21.1	22.9	22.9		–	–	–	–	–	–	–	–
	(c) " " 2065 ft. long	21.0	21.0	21.0	21.0	21.0	21.0	21.0	20.6	20.6	–	–	21.4	23.1	23.2		–	–	–	–	–	–	–	–
	(d) " " 1940 ft. long	21.3	21.3	21.3	21.2	21.3	21.3	21.3	20.8	20.8	–	–	22.1	23.1	23.1		–	–	–	–	–	–	–	–
	(e) " " 1815 ft. long	21.3	21.3	21.3	21.2	21.2	21.2	21.2	20.7	20.7	–	–	22.3	23.1	23.1		–	–	–	–	–	–	–	–
	(f) " " 1690 ft. long	21.8	21.8	21.8	21.7	21.7	21.7	21.7	21.3	21.2	–	–	21.7	23.1	23.1		–	–	–	–	–	–	–	–

P.W.L = Peak Water Level

T.L = RL of Training Levee

CE-D-864



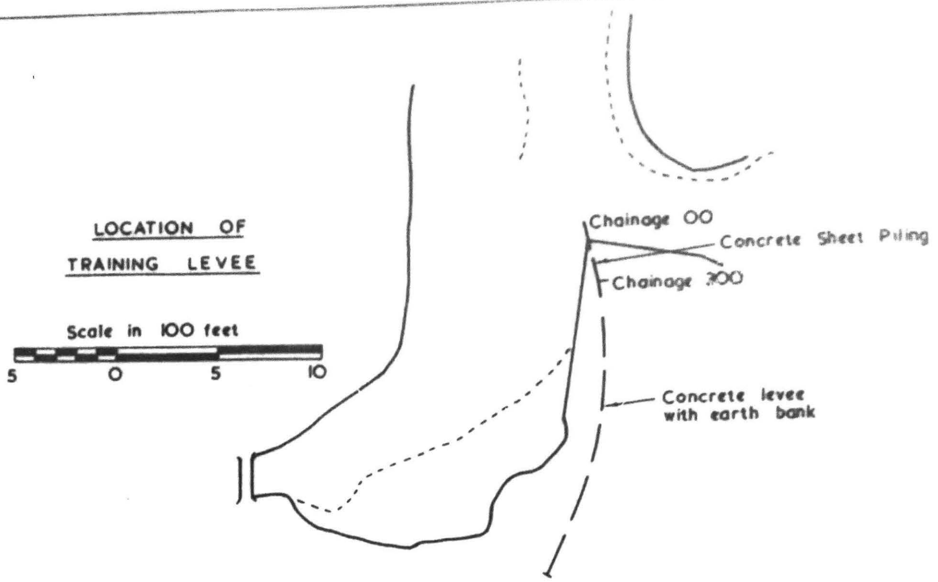


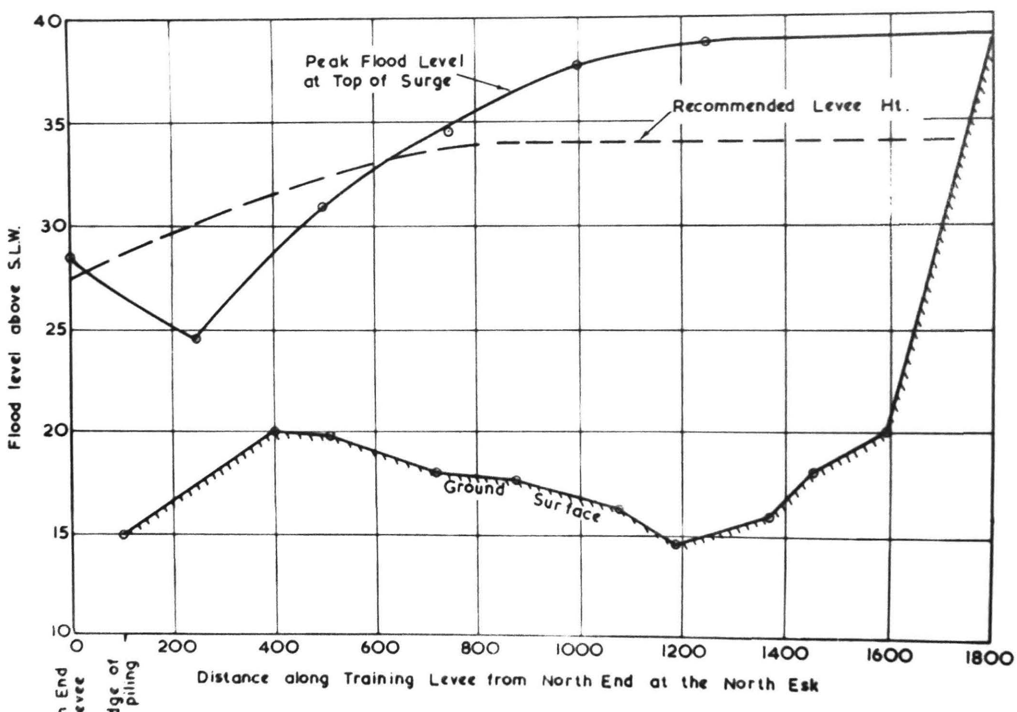
LAUNCESTON FLOOD PROTECTION INVESTIGATION

ROYAL PARK TRAINING LEVEE
CURVED WALL PROPOSAL I
WATER SURFACE PROFILE AND
LEVEE DETAILS FOR MAXIMUM PROBABLE FLOOD

Date Traced 12.10.99

CE·D·839 FIG. S1.



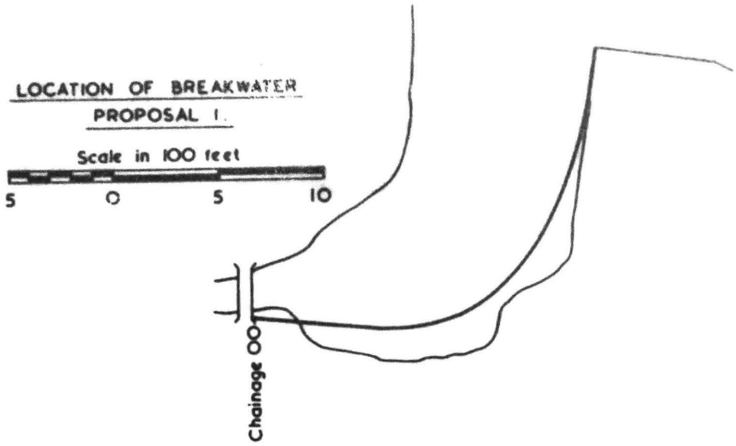


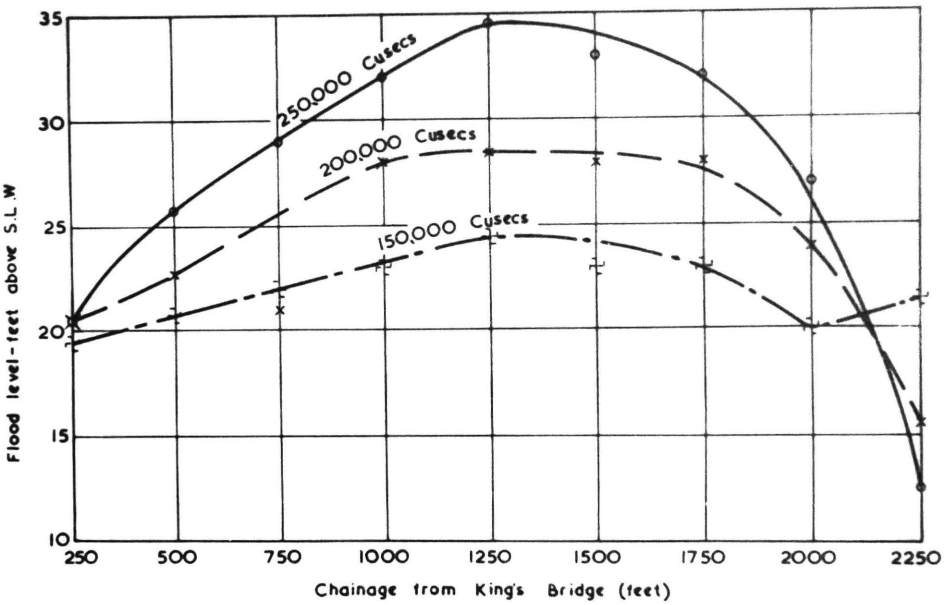
LAUNCESTON FLOOD PROTECTION INVESTIGATION

ROYAL PARK TRAINING LEVEE
CURVED WALL. PROPOSAL 2
WATER SURFACE PROFILE AND
LEVEE DETAILS FOR MAXIMUM PROBABLE FLOOD

Date Traced 12.10.59

CE-D-838 FIG. S2.





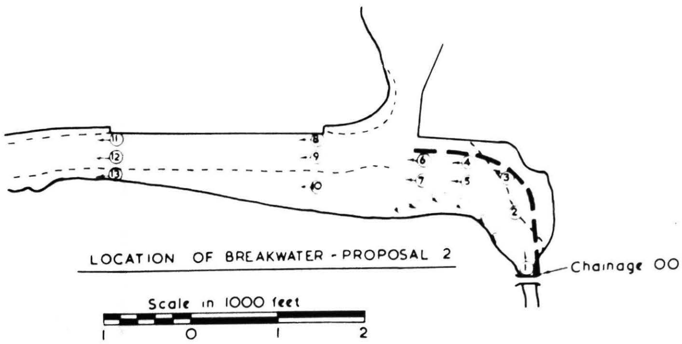
WATER SURFACE PROFILES ALONG PROPOSED BREAKWATER
PROPOSAL I.

LAUNCESTON FLOOD PROTECTION INVESTIGATION

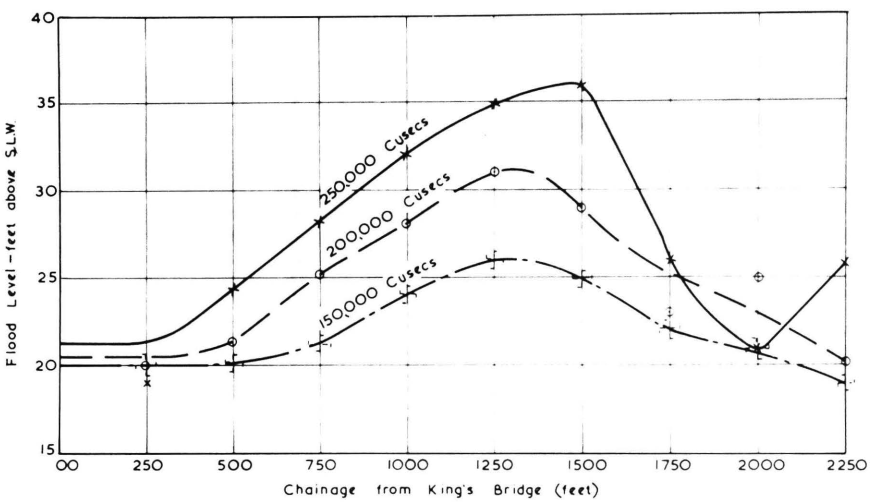
DETAILS OF BREAKWATER FOR DIVERSION OF SOUTH ESK JET

CE-E-837

FIG. S3.



LOCATION OF BREAKWATER - PROPOSAL 2



WATER SURFACE PROFILES ALONG PROPOSED BREAKWATER

PROPOSAL 2

Location	1	2	3	4	5	6	7	8	9	10	11	12	13
Surface	-	29 8	-	31·4	24 6	30 7	27 0	N S V	26 1	15 6	10 5	15 5	10 4
1½ feet above Bed	51·2	26 4	28 1	32 5	-	29 7	23 1	N S V	25 5	-	12 1	14 8	-

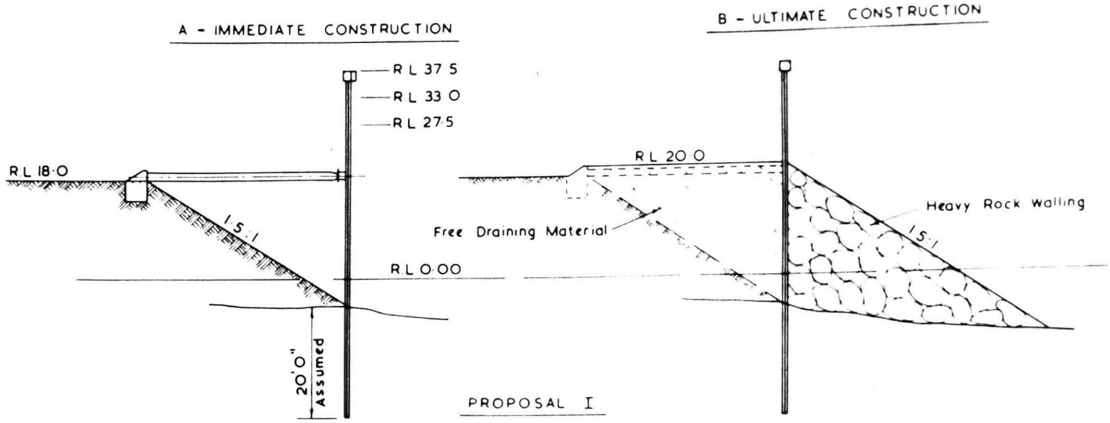
FLOOD VELOCITIES IN THE VICINITY OF PROPOSED BREAKWATER FOR MAXIMUM PROBABLE FLOOD

LAUNCESTON FLOOD PROTECTION INVESTIGATION

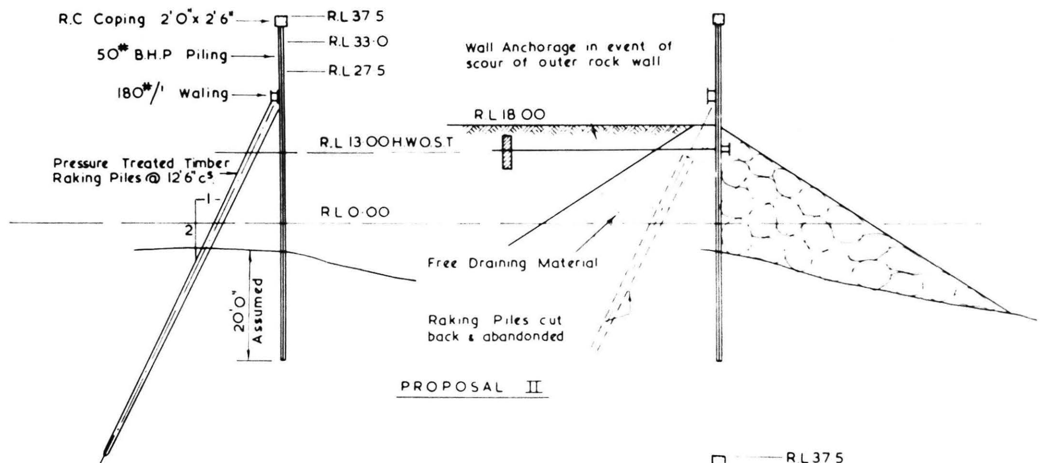
DETAILS OF BREAKWATER FOR DIVERSION OF SOUTH ESK JET

Date Traced 13 10 59

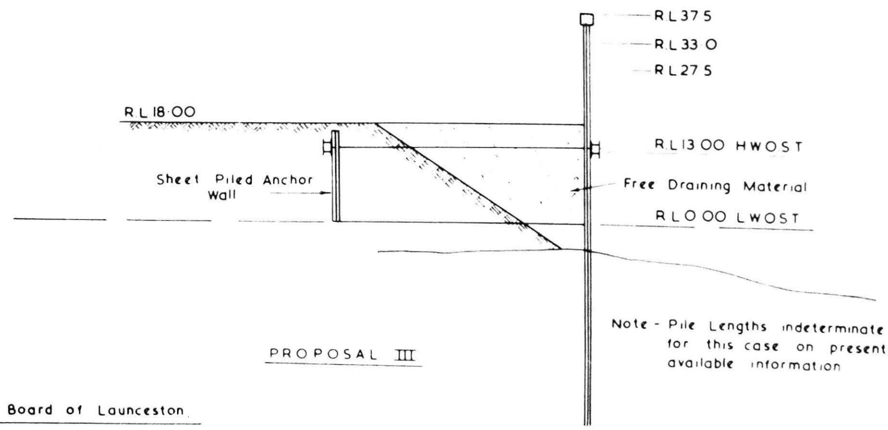
CE·D·841 FIG. S4.



PROPOSAL I



PROPOSAL II



PROPOSAL III

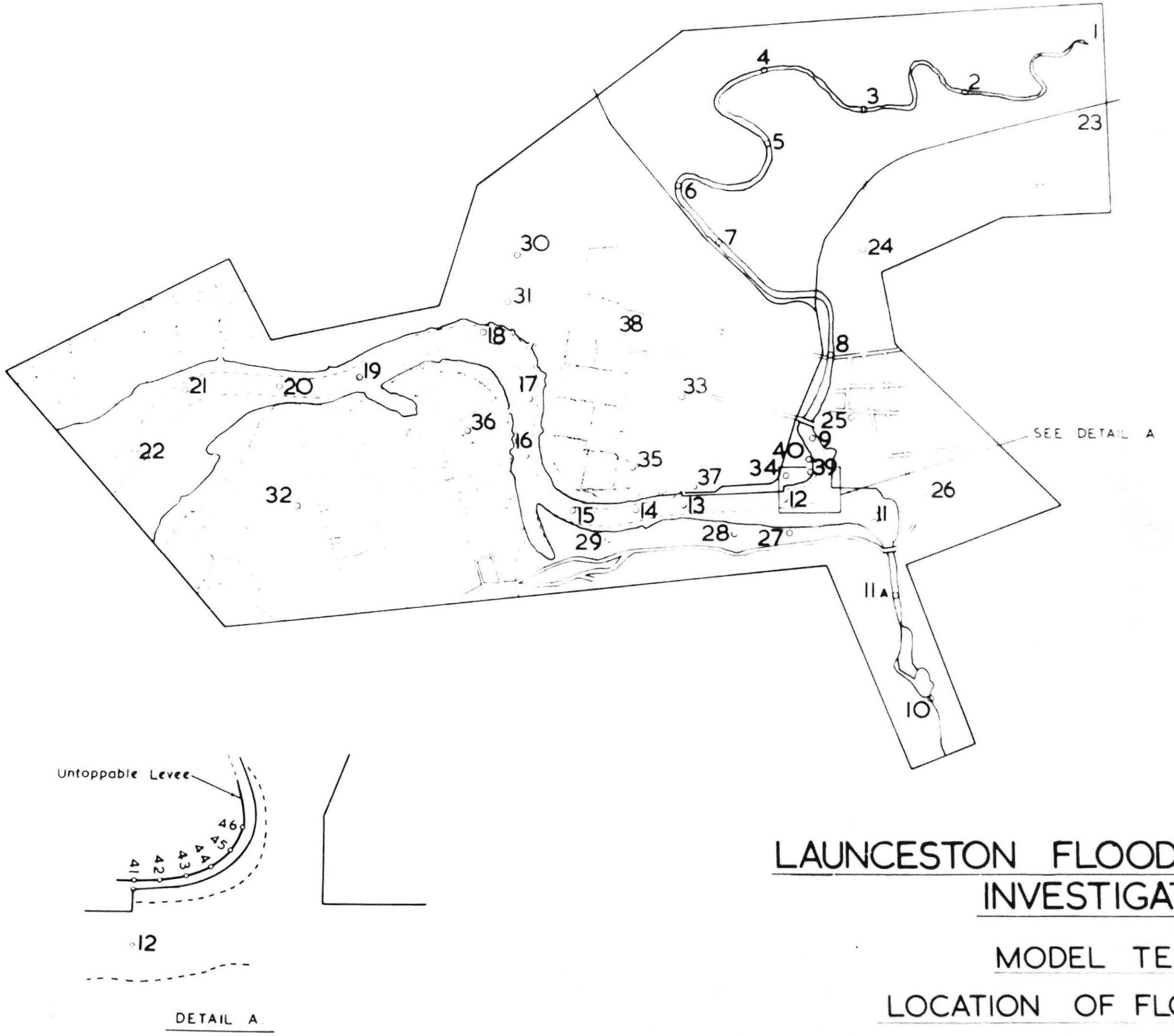
By Curtesy of the Marine Board of Launceston

LAUNCESTON FLOOD PROTECTION INVESTIGATION

DETAILS OF PROPOSED BREAKWATER FOR DIVERSION OF SOUTH ESK JET

Date Traced 15 1(

CE-D-840 FIG. No.



LAUNCESTON FLOOD PROTECTION INVESTIGATION

MODEL TESTS

LOCATION OF FLOOD GAUGES

Date traced 12-6-1959

CE-D-842 FIG. No. S6