THAMES ESTUARY FLOOD PREVENTION RESEARCH

INTERNAL REPORT NO. 21

The work of the Institute of Coastal Oceanography and Tides

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1. Introduction

Since October 1969, the research using the numerical model of the Thames Estuary at the Institute of Coastal Oceanography and Tides has fallen into two main phases, the study of the effects of barriers in the river and the effects of the spill of water over the river banks. The latter is of interest as a possible defensive measure but the latest work is directed towards the evaluation of the possible consequence of a major surge, calculating the volume of water flowing out of the river and leading to an estimate of the possible damage due to surges of various heights.

The numerical model used is basically that described by Rossiter and Lennon (1965) which was briefly mentioned in Appendix 3 of the First Report of Studies, 1969 (hereinafter referred to as I). All the recent experiments have used the long model effectively extending seaward as far as Hardwich (Figure 1), in view of the major changes produced in the river by either a barrier or extensive overspill. The tidal input, or storm surge input, at the seaward boundary of the long model is calculated from the conditions at Southend (Section 0) so that, if the river is unobstructed, the correct water levels are reproduced at Southend. However, when the conditions in the river are changed the level at Southend may respond naturally to the new conditions.

2. Barriers

The effects of the closure of a barrier at various positions along the length of the estuary were examined using both Z-points and U-points. Z-points, where the surface elevation is determined are located at the sections shown in Figure 1; U-points, where the depth mean velocities are calculated are halfway between these sections. The schematic representation of the barrier depends upon the type of point at the barrier site and it is therefore interesting to compare the results. In general the results were similar and one would expect some difference as adjacent U and Z-points are half a section (2.5 miles) apart.

Several different closures were tried at each site, the time of closure being related to the local low water so that closure 1 involved a barrier closing the river completely one hour after the time of low water at the barrier site. Previous experiments on the physical model of the Thames at the Hydraulic Research Station, Wallingford had shown that the rate at which the barrier closed had no consistent effect on the resulting changes of water level in the river. The time between the beginning of the closure and its completion withthe river totally blocked was therefore taken as half an hour, which is a physically reasonable estimate of the time necessary to close a real barrier.

As in the previous work (I), two different types of basic surge were used as the tidal input: the first was the observed values of the storm surge of 31st January - 1st February, 1953, higher surges being produced by increasing the mean water level by 2, 4 or 6 ft; the second was the HRS 53 surge (an artifical surge), higher surges being generated by increasing both mean level and range while keeping the low water level constant to give HRS 53, +2, +4 and +6 ft. The maximum levels reached are the same for both types of surge but the time profiles are noticeably different. The observed surge has a smaller range but a long stand of high water while the HRS surges have a rapid rate of increase in water level with relatively short, sharp peaks at high water. It was thought that the HRS surges would be a more severe test of a barrier system producing greater increases in the downstream water levels and this certainly proved to be the case (Tables 1a and 1b). Although the results for the two surges show the same trends the river levels are always higher with the HRS surge.

For a given type of surge, it was found that increases in the downstream water level were not very sensitive to the actual size of the surge (Figure 2). In all the results the time of closure is very important, the later closures leading to very significant increases in the maximum water level seaward of the barrier. Figure 2 also shows the comparison between results for a barrier at section 7 (Z-point) and section 7½ (U-point). The later closures indicate the effect of the distance between Z and U points, the disturbance from the U-point barrier, 2.5 miles further up-river not reaching as far seawards as that from the Z-point barrier.

Otherwise the results compare quite well.

The absolute accuracy of the results of the numerical model is hard to determine, especially when such major changes are being made in the river geometry. However, a comparison between the results for the barrier at section $7\frac{1}{2}$ and those from the Blackwall Barrier tests on the physical model shows extremely good agreement (Figure 3) providing complete justification for the method of schematisation of the barrier in the numerical model. In addition, as the numerical and physical models are based on completely different assumptions about the representation of the real conditions, this degree of agreement emphasises the validity of both modelling methods,

Table 2 gives results for four positions of Z-point barriers with the HRS 53 +2 surge input. As always, the later closures lead to greater increases in the maximum water level. Complications arise depending on the phasing of the disturbance due to the barrier closure with the arrival of high tide; for example, closure 4 at section 3 and closure 3 at section 5 produce small changes at the barrier but substantial increases in the maximum level downstream.

One way of looking at the disturbance due to a barrier is to plot the difference between the river level with the barrier and the level of the unobstructed river as a function of time for each section (Figure 4). It can be seen that closure 1 produces a larger change in river level, but the maximum value of the change occurs 2.5 hours after low water and by the time of high water, HWB1, the river level is only 0.8 ft. above the level of the open river. Closure 3, on the other hand, gives a slightly smaller disturbance but an increase in level of more than two feet at high water, HWB3. It is interesting to note that this disturbance can be regarded as a reflected wave, propagating downstream and rapidly decreasing in amplitude, partly due to friction but mostly to the widening of the river.

Although this gives a physically interesting insight into the process of barrier closure it must be remembered that from the point of view of surge defence the maximum level reached in the river is the crucial factor and this depends critically upon the phasing of the disturbance relative to the maximum level due to the surge.

At sections 3 and 5 the earlier closures have little effect on the maximum river level but for barriers further upstream early closures may have a significant effect. At section 9, all closures give a large increase in level at the barrier, the disturbance extending further downstream for the early closures and being localised near the barrier site for the latest closure. The maximum increase in level increases as the barrier location is moved upstream so that at Cannon Street (section 9) increases of more than 4 ft. occur with the late closure.

It is apparent that the disadvantage of a barrier lies in the downstream increase in the maximum water level and to reduce this to a minimum at a particular site the barrier should be closed as early as possible. One way of guaranteeing an early closure is to operate the barrier as a half tide system, normally closing the barrier on the falling tide and opening on the rising tide unless a surge warning is received (Figure 5). Closure begins on the falling tide, being completed in half an hour. After some oscillations the water level upstream stabilises and would slowly rise due to the fresh water flow in normal circumstances. As expected, the downstream level falls significantly below the open river condition giving an early and low, low tide followed by a rapid rise in level. In normal operation, the barrier opens when the water levels on either side are equal and the tide returns to that of the unobstructed river prior to high tide. In the event of a surge warning the barrier remains closed; high tide is then early and a little higher (Figure 6). The change in high water level is exactly the same as closure 0 (Table 1(b)) at the same section ($7\frac{1}{2}$). The barrier may be opened on the ebb when the levels on either side are again equal in which case the river returns to normal before low tide. If the barrier remains closed the low and high tides continue to occur earlier and the tidal range continues to be rather larger than in the open river.

An interesting further wrinkle is the possibility of a reclosure of the barrier after a normal opening on the rising tide if a late surge warning is received. The time histories of the water levels above and below the barrier (at section $7\frac{1}{2}$) are shown in Figure 7. Reclosure starts half an hour after the barrier opens and is completed 30 minutes later. In the interval, water has passed through the barrier and the upstreamlevel stabilises at higher value. The closure gives an increased rise in water level along the river (Figure 6). However, without half tide operation a complete closure at the same time, closure 3, has a much more serious effect on the maximum levels downstream.

One substantial advantage of half tide operation is that it restricts the differential head across the barrier to about half the tidal range, and this head could be further reduced by closing the barrier even earlier on the ebb tide. Any fresh water flow would also reduce the differential head.

When a barrier is operated purely as a storm surge defence the early closures result in very large differential heads across the barrier. As the upstream levels are close to the low water level, the difference at a time of low fresh water flow can be almost equal to the total range of the tide plus surge (Figure 8). As the design head has important implications for structural design of the barrier and its foundations, an investigation was made into possible ways of reducing this differential head.

The simplest concept was a weir of given height and length over which water could spill into the upper reaches of the river after barrier closure (Table 3). It was found that a substantial reduction in the head was produced only if the weir was very wide (river width at section 7½ is about 750') and also if the weir was rather low. Using the HRS 1953 +4 tide and a weir level of zero O.D.N., at high water the difference between the downstream water level and the weir level was nearly 25 ft. Despite the resulting spectacular waterfall, long weir lengths were required to restrict the head difference to 20 ft. and substantially to reduce the downstream levels (Figure 8).

To reduce the waterfall to reasonable proportions a scheme was tried in which the weir rose with the tide maintaining a difference in level of 9 ft. Again it proved difficult to restrict the head difference and the downstream levels were almost the same as those for the full barrier (Table 4). One difficulty in assessing these results is in deciding whether the HRS 1953 +4 surge occurring at a time of low fresh water flow is a reasonable design criterion; the observed HRS +4 occurring at time of high fresh water flow causes no trouble either upstream or downstream (Table 5). This shape of surge is reduced by the barrier and slightly further reduced by the flow over the barrier; upstream the closure does not cause any difficulty when backing up the fresh water flood.

A more sophisticated combination of flumes and weirs (Figure 9) was finally tested which gave reasonable results for the HRS 53 +4 tide, restricting the head to less than 20 ft. while giving the bonus of reducing downstream levels to their open river values (Table 6).

3. Overspill

In the previous work (I) planned overspill had been examined as a possible defence against major storm surges. Although it was found that some surges, particularly the short, sharply peaked HRS surges, could be substantially reduced it was difficult to find adequate space to contain the very large volumes of water flowing over the river banks into the overspill areas.

The motivation for a further investigation was twofold; first, overspill was considered as a possible interim defence during the construction of a barrier; second, it is obvious that, planned or not, overspill will occur extensively during any major surge. A current research programme, therefore, involves the estimation of the volumes of water overspilling at each section during a given surge. This will give a clearer picture of the potential damage which may be caused by surges of various heights.

Several schemes have been examined which would provide an interim defence against the smaller, but more frequent, surges during the construction of a major defensive system. A typical scheme is shown in Table 7. To obtain the maximum reduction in water level, maximum overspill should occur shortly before high water. Drop gates were therefore introduced which when overtopped drop flat, immediately allowing a depth of flow over the bank of several feet. The result of using weir lengths I with the 1953 surge as input is shown in Table8 for various combinations of the possible overspill areas. The complex nature of overspill is indicated, for example, by the combination 1A and 3 being marginally better than areas 1A, 2 and 3. This is due to the filling of area 2 before high water leading to the positive disadvantage commonly encountered in the previous studies of overspill (I). The weir lengths were adjusted so that as much water was taken from the river as possible without quite filling the available area. These new weir lengths, II, generally give better results than I and are also much more consistent, the addition of an extra basin always further lowering the maximum level in the river. The combination of areas 1B, 2 and 3 gives quite good results (Table 8), substantially lowering the water level along most of the estuary. However, it must be emphasised that this optimisation of weir lengths has been made for this particular tide and will not be optimal for a surge of different size or shape.

The existing numerical model of the Thames, model A, has always given rather high values for the maximum tidal elevation in the upper reaches of the river (Figure 10). This was considered during the original proving of the model (Rossiter and Lennon, 1965) and no obvious reason for this discrepancy was found. Although it is possible that a term omitted from the equations of motion used in the numerical model becomes important in this region, it is more likely that the problem arises from the complexity of the geometry of the upper reaches, particularly Richmond lock and half tide barrier and Teddington lock and weir which are difficult to represent in the model. An additional problem is the location of the tide gauges in this area; for example, Richmond tide gauge is positioned between the lock and the half tide barrier and, at times, may be recording very localised effects. Consideration is therefore being given to a much more detailed representation of this section of the river. An example of a major feature omitted in the original model is the effect of flow over Teddington weir which has a height of +14.3 ft. above O.D.N. Clearly, any reasonably high tide drowns this weir and during a large surge it may be overtopped by many feet making its description as 'the tidal limit of the Thames' something of a misnomer. A rough representation of the drowning of Teddington weir has been included in model B (Figure 10) producing a noticeable decrease in the maximum levels reached.

The excellent agreement between the results of the physical and numerical models in the lower reaches of the estuary, section 0-9, suggests that the observed levels of the surge of 31 Jan. - 1 Feb. 1953 were significantly reduced by overspill. This led to the original suggestion that planned overspill might be used as a defensive measure.

To try to reproduce the actual 1953 conditions bank levels have been included in the model. Approximate bank levels based on statutory defence levels were used in sections 0-6, but detailed bank levels have been obtained for the upper river sections 7-12.

If unlimited overspill is allowed at each section (Table 9) in the river the maximum water levels are reduced to values very similar to those actually measured in 1953 (Figure 10). The assumption that the overspill areas do not fill completely during the surge is probably valid for all section except 12. Here the volume available is rather limited and a more detailed investigation is necessary to take this into account. However the river levels, other than at section 12, will be very little changed by this readjustment. The volumes flowing out of the river in the lower reaches are of the right order of magnitude, although rather larger than the values given by Allen, Price and Inglis (1955). However the conclusion differs radically, as Allen, Price and Inglis suggested that overspill reduced the river levels by a few inches at most. In view of the recent results, it must be concluded that Allen, Price and Inglis were wrong, probably because in their model the levels without overspill, about which they are extremely vague, were already comparable with the observed levels. In effect, they had the wrong input and were studying a smaller surge.

Since 1953, the bank levels in the lower reaches of the estuary have been considerably raised, so that a surge of the same size as '53 occurring now would not be attenuated by overspill in the lower river. Figure 10 shows the effect of spill in London only. The levels are reduced to those observed in 1953 but the total volume of water increases very substantially (Table 9), the city itself becoming a primary overspill area. Similar experiments using surges of different heights will be used to estimate the potential flood damage in London.

References

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TABLE 1(a)

CHANGES IN MAXIMUM LEVELS DOWNSTREAM OF A BARRIER (FEET)

1953 OBSERVED TIDE +2 FEET

Barrier Site	Closure	Section 0	1	2	3	4	5	6	7	8
SECTION 4½	0	-0.1	-0.1	-0.4	-0.4	-0.6	-	-	-	-
	1	-0.2	-0.2	-0.4	-0.4	-0.6	-	-	-	-
	2	-0.1	-0.2	-0.5	-0.5	-0.9	-	-	-	-
	3	0.2	0.1	-0.2	-0.6	-0.7	-	-	-	-
	4	0.8	1.3	1.5	1.8	2.0	-	-	-	-
SECTION 7½	0	0.1	0.2	0.1	0.2	0.3	0.2	-0.2	-0.2	-
	1	0.0	0.2	0.2	0.2	0.2	0.2	-0.2	-0.3	-
	2	0.1	0.2	0.3	0.3	0.6	0.6	0.2	-0.1	-
	3	0.4	0.6	0.8	1.0	0.9	1.2	1.0	0.9	-
	4	0.0	0.2	0.5	1.0	1.9	1.9	1.6	1.2	-
SECTION 8½	0	0.2	0.3	0.3	0.3	0.6	0.9	0.8	0.4	0.5
	1	0.2	0.3	0.3	0.3	0.6	1.0	0.8	0.4	0.6
	2	0.4	0.4	0.8	0.8	1.0	1.2	1.2	1.1	0.8
	3	0.0	0.1	0.5	0.8	1.9	1.9	2.0	1.3	1.4
	4	0.0	0.0	0.0	0.0	0.0	0.1	-0.1	1.7	2.5

TABLE 1(b)

CHANGES IN MAXIMUM LEVELS DOWNSTREAM OF A BARRIER (FEET)

H.R.S. 1953 +2 FEET TIDE

Barrier Site	Closure	Section 0	1	2	3	4	5	6	7	8
SECTION 4½	0	0.0	0.2	0.2	0.4	0.1	-	-	-	-
	1	-0.1	0.2	0.1	0.4	0.1	-	-	-	-
	2	-0.1	0.0	0.1	0.1	-0.1	-	-	-	-
	3	0.3	-0.2	-0.1	-0.1	-0.6	-	-	-	-
	4	0.7	1.7	2.3	2.5	2.2	-	-	-	-
SECTION 7½	0	0.2	0.4	0.4	0.6	0.8	0.9	0.9	0.7	-
	1	0.2	0.4	0.4	0.6	0.9	1.0	1.0	0.8	-
	2	0.3	0.5	0.5	0.8	1.2	1.5	1.6	1.3	-
	3	0.0	0.0	0.5	1.3	1.9	2.0	2.3	2.4	-
	4	0.0	0.0	0.0	-0.1	1.2	1.8	2.4	2.6	-
SECTION 8½	0	0.1	0.3	0.4	0.6	1.1	1.1	1.4	1.9	1.7
	1	0.1	0.3	0.4	0.6	1.0	1.1	1.5	1.8	1.7
	2	0.0	0.3	0.5	0.7	1.1	1.2	1.6	2.0	2.1
	3	0.0	0.0	0.0	1.0	1.9	1.7	1.9	2.2	2.3
	4	0.0	0.0	0.0	-0.1	0.1	1.2	2.2	2.8	3.3

TABLE 2

CHANGES IN MAXIMUM LEVELS DOWNSTREAM OF A BARRIER (FEET)

H.R.S. 1953 +2 FEET TIDE

Barrier Site	Closure	Section 0	1	2	3	4	5	6	7	8	9
SECTION 3	0	0.2	0.2	0.1	0.3	-	-	-	-	-	-
	1	0.2	0.1	0.1	0.2	-	-	-	-	-	-
	2	0.2	0.1	0.2	0.1	-	-	-	-	-	-
	3	0.0	0.1	-0.1	-0.1	-	-	-	-	-	-
	4	0.9	0.7	0.4	-0.1	-	-	-	-	-	-
SECTION 5	0	0.0	-0.2	-0.3	0.0	-0.1	-0.1	-	-	-	-
	1	0.0	-0.2	-0.2	-0.1	0.1	-0.1	-	-	-	-
	2	0.0	-0.1	-0.2	-0.1	-0.2	-0.5	-	-	-	-
	3	0.6	0.5	0.6	0.3	0.2	-0.5	-	-	-	-
	4	0.1	1.1	1.8	2.3	2.4	2.5	-	-	-	-
SECTION 7	0	0.2	0.3	0.4	0.7	0.7	0.7	1.1	0.7	-	-
	1	0.2	0.3	0.4	0.6	0.7	0.7	1.0	0.7	-	-
	2	0.3	0.4	0.4	0.7	1.0	1.1	1.2	1.1	-	-
	3	0.0	0.4	0.9	1.3	1.4	1.7	2.0	2.3	-	-
	4	0.0	0.0	0.0	0.8	2.6	2.4	2.3	2.2	-	-

TABLE 2 (cont.)

CHANGES IN MAXIMUM LEVELS DOWNSTREAM OF A BARRIER (FEET)

H.R.S. 1953 +2 FEET TIDE

Barrier Site	Closure	Section 0	1	2	3	4	5	6	7	8	9
SECTION 9	0	0.0	0.1	0.1	0.4	0.9	1.2	1.5	1.7	2.5	2.9
	1	0.0	0.1	0.1	0.3	0.9	1.2	1.4	1.7	2.4	2.9
	2	0.0	0.0	0.0	0.5	1.0	1.2	1.5	1.8	2.5	2.8
	3	0.0	0.0	0.0	0.0	0.5	1.1	1.7	2.1	2.8	2.9
	4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	1.0	2.1	4.1

TABLE 3

BARRIER AT SECTION 7½ (WOOLWICH REACH) WITH AN OVERSPILL WEIR

H.R.S. 1953 +4 FEET TIDE WITH NO UPLAND FLOW

Weir level (feet to O.D.N.)	Weir length (feet)	Section 7 at time of max. head (feet to O.D.N.)	Section 8 at time of max. head (feet to O.D.N.)	Maximum head across barrier (feet)
-4	300	20.7	2.1	18.6
	400	14.0	-2.0	16.0
	800	10.9	-0.4	11.3
	1200	4.3	-5.0	9.3
0	300	20.9	-0.4	21.3
	400	20.1	1.1	19.0
	800	12.4	-2.7	15.1
	1200	9.7	-3.2	12.9
+4	300	23.0	-0.9	24.0
	400	20.9	-1.3	22.2
	800	14.3	-4.1	18.5
	1200	13.3	-3.4	16.8
+8	300	23.6	-3.0	26.6
	400	22.7	-2.4	25.1
	800	19.2	-2.4	21.6
	1200	15.1	-5.1	20.2

TABLE 4

BARRIER AT SECTION 7½ (WOOLWICH REACH) WITH AN OVERSPILL WEIR HAVING A CONSTANT HEAD OF 9 FEET

T.R.S. 1953 +4 FEET WITH NO UPLAND FLOW

Weir Length (Feet)	Maximum Levels at Sections of Model (feet to O.D.N.)												
	0	1	2	3	4	5	6	7	8	9	10	11	12
Open River	19.3	19.7	20.0	20.3	21.4	22.3	22.9	23.3	23.6	23.9	26.7	28.0	29.3
Full Barrier	19.5	20.0	20.4	21.1	22.5	23.3	23.9	24.5	-6.8	-6.8	-5.6	-2.7	-1.4
300	19.5	20.0	20.4	21.0	22.4	23.3	24.0	24.5	1.2	1.3	1.4	1.4	1.7
400	19.5	20.0	20.4	21.0	22.4	23.3	24.0	24.6	3.0	3.1	3.2	3.3	3.7
800	19.5	20.0	20.3	21.0	22.3	23.2	24.0	24.5	8.5	8.5	8.7	9.5	10.1
1200	19.4	19.9	20.3	21.0	22.2	23.2	23.9	24.4	12.9	13.0	13.4	14.6	15.4

Weir Length (feet)	Section 7 at time of max. head (feet to O.D.N.)	Section 8 at time of max. head (feet to O.D.N.)	Maximum head across barrier (feet)
300	24.5	-2.2	26.8
400	24.5	-0.7	25.2
800	24.3	4.5	19.8
1200	24.1	9.1	15.0

TABLE 5

BARRIER AT SECTION 7½ (WOOLWICH REACH) WITH AN OVERSPILL WEIR

1953 OBSERVED TIDE +4 FEET WITH AN UPLAND FLOW OF 10,000 CUSECS

Weir Level (feet to O.D.N.)	Weir Length (feet)	Maximum Levels at Sections of Model (feet to O.D.N.)												
		0	1	2	3	4	5	6	7	8	9	10	11	12
Open River		19.2	19.4	19.7	20.2	20.9	21.4	21.9	22.7	23.9	24.7	25.4	26.1	26.8
Full Barrier		19.2	19.5	19.9	20.4	20.6	21.0	21.3	21.8	3.3	3.3	3.4	3.6	4.8
-4	400	19.1	19.4	19.7	20.2	20.5	20.9	20.9	21.2	17.7	18.3	18.7	19.0	18.9
0	400	19.1	19.4	19.7	20.2	20.4	20.8	20.9	21.2	16.0	16.4	16.7	17.0	17.3
+4	800	19.1	19.3	19.6	20.0	20.2	20.6	20.6	20.7	18.6	19.2	19.7	19.9	20.2
+8	800	19.1	19.3	19.6	20.1	20.2	20.5	20.6	20.9	15.8	16.1	16.3	16.4	16.5

Weir Level (feet to O.D.N.)	Weir Length (feet)	Section 7 at time of max. head (feet to O.D.N.)	Section 8 at time of max. head (feet to O.D.N.)	Maximum head across barrier (feet)
-4	400	18.1	6.2	11.9
0	400	19.2	5.1	14.1
+4	800	15.0	1.8	13.2
+8	800	19.0	3.1	15.9

TABLE 6

BARRIER AT SECTION 7½ (WOOLWICH REACH) WITH FOUR SEPARATE OVERSPILL WEIRS

H.R.S. 1953 +4 FEET TIDE WITH NO UPLAND FLOW

Level of weirs on south side of river (feet to O.D.N.)	Maximum Levels at Sections of Model (feet to O.D.N.)												
	0	1	2	3	4	5	6	7	8	9	10	11	12
Open River	19.3	19.7	20.0	20.3	21.4	22.3	22.9	23.3	23.6	23.9	26.7	28.0	29.3
Full Barrier	19.5	20.0	20.4	21.1	22.5	23.3	23.9	24.5	-6.8	-6.8	-5.7	-2.7	-1.4
-4	19.4	19.8	20.0	20.5	21.4	22.2	22.8	23.2	16.9	16.8	16.4	19.1	20.4
0	19.4	19.8	20.1	20.5	21.4	22.2	22.8	23.2	15.8	15.7	15.2	17.4	18.9
+4	19.4	19.8	20.1	20.5	21.4	22.2	22.8	23.2	14.7	14.5	14.0	15.7	17.3
+8	19.4	19.8	20.1	20.5	21.5	22.3	22.9	23.2	13.5	13.3	12.8	14.1	15.6

Weir level (feet to O.D.N.)	Section 7 at time of max. head (feet to O.D.N.)	Section 8 at time of max. head (feet to O.D.N.)	Maximum head across barrier (feet)
-4	12.2	-1.5	13.7
0	13.2	-2.1	15.3
+4	13.4	-3.0	16.4
+8	14.1	-3.1	17.3

TABLE 7

OVERSPILL STUDY

Overspill Site	Position	Model Section	Basin Area (acres)	Average Ground Level (feet to O.D.N.)	Weir Length I (feet)	Weir Length II (feet)	Overspill Weir Conditions
1A	Cliffe Marshes	2	4,500	7.0	3,000	6,000	Weir Crest at 10.0 feet O.D.N. Flow to commence when river level reaches 13.0 feet O.D.N.
1B	Cliffe Marshes including Blyth Sands	1	7,000	3.5	30,000	40,000	Weir Crest at 12.0 feet O.D.N.
2	Bower and Fobbing Marshes	2	3,000	7.0	9,000	2,000	Weir Crest at 8.0 feet O.D.N. Flow to commence when river level reaches 11.0 feet O.D.N.
3	Aveley Marshes	5	1,000	7.0	4,000	2,000	Weir Crest at 9.5 feet O.D.N. Flow to commence when river level reaches 14.0 feet O.D.N.

TABLE 8

CHANGES IN MAXIMUM LEVELS WITH OVERSPILL SITES (feet)

1953 Observed Tide

Weir Lengths	Overspill Sites	Section												
		0	1	2	3	4	5	6	7	8	9	10	11	12
I	1A, 2 and 3	0.0	-0.1	-0.3	0.3	-0.3	-0.4	-0.2	-0.7	-0.4	0.2	-0.6	-1.2	-1.1
	1B, 2 and 3	-0.3	-0.4	-0.4	-0.6	-0.6	-0.7	-0.6	-1.1	-0.8	-0.2	-0.8	-1.4	-1.5
	1A and 2	0.0	0.1	-0.1	-0.2	-0.1	-0.3	-0.2	-0.6	-0.6	-0.1	-0.2	-0.6	-0.5
	1A and 3	-0.1	-0.1	-0.3	-0.2	-0.2	-0.4	-0.3	-0.8	-0.6	0.1	-0.4	-1.0	-1.1
	1B	-0.2	-0.4	-0.5	-0.4	-0.4	-0.5	-0.3	-0.5	-0.5	-0.6	-0.6	-0.6	-0.8
II	1A, 2 and 3	-0.2	-0.1	-0.2	-0.4	-0.5	-0.6	-0.6	-1.0	-0.8	-0.1	-0.7	-1.4	-2.0
	1B, 2 and 3	-0.3	-0.2	-0.4	-0.7	-0.8	-0.9	-0.9	-1.3	-1.2	-0.7	-1.0	-1.5	-2.1
	1A and 2	-0.1	0.0	-0.2	-0.3	-0.3	-0.5	-0.6	-0.7	-0.6	-0.2	-0.5	-0.8	-1.1
	1A and 3	-0.1	-0.1	-0.2	-0.3	-0.4	-0.5	-0.6	-1.1	-0.8	-0.1	-0.6	-1.3	-2.1
	1B	-0.3	-0.1	-0.3	-0.6	-0.6	-0.6	-0.4	-0.7	-0.7	-0.7	-0.7	-0.7	-1.0

TABLE 9

TOTAL VOLUME OF OVERSPILL DURING OBSERVED 1953 SURGE (10^6 cu.feet)

Sections at which overspill occurs	Bank	Section											
		1	2	3	4	5	6	7	8	9	10	11	12
1-12	North	21.5	76.6	165.8	103.3	270.3	0.0	1.7	4.3	0.0	0.0	44.3	162.1
	South	21.5	76.6	165.8	103.3	270.3	0.0	1.9	3.7	1.6	27.6	16.6	107.9
7-12	North	-	-	-	-	-	-	42.4	49.9	2.5	0.0	55.7	173.1
	South	-	-	-	-	-	-	40.4	42.9	15.8	40.5	24.7	114.5