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THAMES ESTUARY FLOOD PREVENTION RESEARCH

1 SEPTEMBER 1968 - 31 OCTOBER 1969

BY

A.J. BOWEN AND SALLY J. PINLESS

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NATURAL ENVIRONMENT RESEARCH COUNCIL

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Thames Flood Prevention Research, 1st September 1968 - 1st October 1969.

Since September 1968, a series of investigations have been made into the possibility of reducing the dangerous water levels associated with storm surges which may occur in the Thames Estuary.

The basic tools for this study are improved and expanded versions of the existing numerical model of the Thames (Rossiter and Lennon, 1965). All these models solve a sophisticated set of equations which include non-linear frictional and advective terms and take into account the changes in the area, breadth and depth of the river due to the changing state of the tide. The input data required by the models are the river flow at the landward end of the model at Teddington Weir and a specified time history of the water level at one particular section. The water level at each section (Figure 1) and the velocity midway between the sections are then uniquely determined and can be calculated at successive intervals of time. In the original model the input levels were defined at Southend (section 0) using the observations from the Southend tide gauge. This model, the 'short model' can then be expected to have characteristics rather similar to those of the hydraulic model at the Hydraulics Research Station whose input is also a specified time history of the water level in this case just seaward of Southend. In both models the water level at Southend is not allowed to depart from the predetermined tide curve, and one therefore must be rather cautious when introducing major physical changes in the river which, in reality, might alter the levels at Southend and perhaps seaward of Southend.

To tackle this problem a 'long model' is used extending nine sections seaward of Southend and as the distance between sections is 4·9 miles, this gives an input 44 miles seaward of Southend, effectively at Harwich. The new input at the seaward boundary is calculated from the Southend conditions so that, when the river is unobstructed, the correct water levels at Southend are reproduced, However when obstacles are placed in the river the level at Southend may change naturally in response to the new conditions. The disturbance will propagate seawards and eventually cause trouble by reflecting from the new seaward boundary of the model.

However the position of the outer boundary can be adjusted so that critical results, for example high water levels following a barrier closure, are obtained before any unnatural disturbances occur in the area of interest.

These models have been used to study various suggestions for reducing extreme levels in the river. For convenience, the study was split into a series of projects each representing a basic investigation or a particular type of defence scheme. These projects are :-

I An investigation of the water levels and velocities in the Thames during modified 1953 conditions. The short model was run using as input the observed levels at Southend of the 1953 surge with the addition of an increase in (i) the mean water level, (ii) the height of the surge and (iii) the height of the semi-diurnal component of the tide.

II An investigation of the effect of allowing a limited amount of water to overspill the river bank at Cliffe Marshes (near section 2 in Figure 1) on the maximum water levels in the river.

III An investigation of the effect of a temporary narrowing of the river near the seaward end of Canvey Island (midway between sections 0 and 1). This is essentially a moveable barrier which, when closed, reduces the cross-sectional area of the river.

IV An investigation of the effect of siting a barrier at various locations along the river, particularly Long Reach and Woolwich. Attention has been concentrated on a moveable barrier which closes the river completely, the process of closure taking 30 minutes.

Project I.

For most of the work with the numerical models the input data have been based on the water levels observed at Southend and the river flow measured at Teddington during the storm surge of 31st January - 1st February, 1953. To study the effect of larger surges, three physically reasonable modifications were made to the levels observed at Southend.

Firstly, a modification to mean sea level was considered. Three runs of the model were made with the levels observed at Southend plus 2', 4' and 6', respectively input, thus representing a rise in mean sea level of those amounts.

Secondly, three runs were made with input levels those observed in 1953 with the addition of the positive part of one cycle of a diurnal oscillation which reached a peak of 2' (a) at 2200 hours on 31st January, (b) at 0100 hours on February 1st and (c) at 0400 hours on February 1st. These input levels represent an increase in surge height by an oscillation which reached a peak 3 hours before the peak of the surge, in phase with the surge, and 3 hours after the peak of the surge respectively.

Thirdly, a run of the model was made with input levels those observed in 1953 with the addition of a semi-diurnal oscillation of amplitude 2' which reached a peak at 0100 hours on February 1st, thus representing an increase in tidal range, (d).

In 1953 the maximum levels were recorded around 0100 hours of February 1st and the modifications to the input conditions of the model have made little difference to this timing.

As can be seen in Table I very little increase in maximum level, above the original increase in mean sea level under consideration, occurs in the river until the area of sections 8, 9 and 10 is reached where a significant peak is seen. A similar peak occurs when an increased surge height or an increased tidal range is considered.

In general it can be said that under these modified 1953 storm surge conditions the maximum increase in levels over those encountered in 1953 would occur around the area of Greenwich Pier to Chelsea Bridge and it should be noted from Table I that maxima from Southend (section 0) to Tower Pier (section 8·75) are largely independent of the precise nature of the modified input (i.e. mean sea level change of +2', surge increase of +2' or tidal increase of +2').

In view of the uncertainties involved in producing an artificial surge it was decided to concentrate on the simplest case, a rise in mean sea level of +2', +4' and +6'. The Hydraulics Research Station decided that it would be more convenient to use a repeating tidal cycle and consequently they produced an artificial spring tide with equal low water levels to which was added a surge, zero at both low waters and reaching a peak slightly before high tide. Higher surges were generated by stretching the curve, keeping low water levels constant. The H.R.S. 53, H.R.S. 1953 + 2', observed 53 and observed 53 + 2' are shown in Figure 2.

In view of the difficulties in suggesting a design surge these two possibilities include several of the possible dangerous features, the observed 53 having a small range but a long stand at high water and the H.R.S. 53 a relatively short high water but a large range and consequently a rapid rate of rise of water level.

As can be seen in Figure 3, which shows the maximum levels calculated for the 1953 surge, the results of the hydraulic and numerical models agree very well between Southend and Westminster (sections 0 - 9) although the observed levels are considerably lower. This difference has been generally attributed to the absence of westerly winds and overspill in the models. The effect of overspill, even though originally thought to be small (Allen, Price and Inglis, 1955), led to the suggestion that beneficial results might be obtained by allowing the river to overflow into a limited area.

Project II.

It was suggested that a limited overspill area could be provided in the region of Cliffe Marshes (between sections 1 and 2) into which the water would begin to spill when the river reached some given level, z_0. The first overspill values were calculated by the G.L.C. using the levels at Southend, without overspill, to calculate the discharge from the river. Two difficulties were then encountered, the level in the river was sufficiently altered by the overspill that the previously calculated discharges provided only a rough estimate of the actual effect and the changes in level were sufficiently large to throw some doubt on the use of the short model. The problem was therefore transferred to the long model and the overspill was calculated at each time step of the program using the formula

$$q = 3b\left[z - z_0\right]^{3/2} \qquad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

where q = rate of overspill from river

b = weir length

z_0 = weir level (level in the overspill area if higher than weir level)

z = water level in the river at weir site

A total of 6,500 acres of Cliffe Marshes were allocated as an overspill area, the total volume available below 12·1' O.D.N. being 2053 x 10^6 cu. feet.

Various model runs were made to examine the effect of different weir levels and lengths on the maximum levels reached by the 1953, 1953 + 2', 1953 + 4' and 1953 + 6' surges.

Provided that the overspill area available is not filled during the surge the effect on levels upstream was found to be beneficial, the maximum levels being further reduced by increased discharge from the river (Table 2).

With a weir level of 12·1' O.D.N. the optimum reduction in the levels of the 1953 surge was obtained using a weir length of 6 miles. The extra discharge which resulted from a weir length of 7 miles was more than sufficient to fill the area available for overspill and the beneficial effects were reduced (Figure 4). However a weir which reduced the levels of the 1953 surge was found to actually increase the levels produced by higher surges (Figure 5 and Table 3).

This unexpected effect is due to the overspill area having a capacity which, although large, may be completely filled by overflow during a large surge. In Figure 5 it can be seen that a weir which reduces the water levels of the 1953 surge has a complex effect on the 1953 + 2' surge, adverse in the region of Central London. The levels associated with the higher surges are generally slightly increased. Figure 6 shows the effect of a weir of similar length but 2 ft. higher. In this case, little overspill occurs during the 1953 surge and the effect, although beneficial, is small. As might be expected, the changes in maximum levels for the 1953 + 2', 1953 + 4' and 1953 + 6' surges are very similar to those of the 1953, 1953 + 2' and 1953 + 4' surges with the weir 2 ft. lower (Figure 5).

To examine the physical reason for this increase in surge height it is useful to look at the time history of the water level at the point of overspill (section 2) in conjunction with the variation in the rate of overspill. In Figure 7 the effect of three different weir levels on the 1953 + 4' surge is shown in comparison with the tide curve in the absence of overspill. It can be seen that once the water level exceeds the weir level water begins to overflow, reducing the level in the river. As the river level rises, the rate of overspill increases (equation 1). If the area does not fill the overspill rate will decrease during the falling tide and the total effect will be to chop off the top of the tide curve. However, if the overspill area fills there is a sudden, marked reduction in the overspill rate (Figure 7) and although overspill continues as the reservoir level tends to the river level the rate is drastically reduced. The resulting disequilibrium, perhaps analagous to other closure effects in hydraulics such as water hammer, produces a surge up the river. This is shown at Section 2 by a rapid rise in the water level as soon as the overspill rate starts to decrease (Figure 7). The net result is a higher maximum water level at this section. Figure 6 shows the profile of the maximum level reached along the river in this case. The motion of this new surge is obviously complicated and may be beneficial in some areas.

However it is clear that a scheme designed to give protection to London is unlikely to result from this type of system. Although an overspill weir can be designed to significantly reduce any particular surge, it tends to be rather ineffective in reducing lower surges and may be positively harmful during higher surges.

Project III.

There have been several suggestions that the temporary, partial closure of a river or harbour entrance might reduce the maximum water levels during a storm surge (Balloffet and Kupferman, 1964). This closure not only reduces the cross-sectional area of the river but creates an energy loss in the eddies downstream from the barrier. This type of problem occurs during the closure of a dyke across a tidal entrance (Dronkers, 1964), and the resulting energy loss is similar to the effects of bridge abutments in a river (Lin, Bradley and Plate, 1957).

Allen, Price and Inglis (1955) examined the effects of a partial closure of this type in Long Reach using a hydraulic model and found that the maximum water levels up river from the narrowing were not significantly reduced unless more than 80% of the cross-sectional area was closed. At the proposed position near Canvey Island, much further down river, closures greater than 80% conflict with the navigational requirements and more moderate closures, leaving between 20% and 40% of the river open were of most interest. Although the results of Allen et al (1955) were discouraging, the conditions at the two sites were sufficiently different to warrant a detailed study of the Canvey site. The cross-section of interest is shown in Figure 8, the vertical exaggeration being 200:1. A closure of 80% of the area below mean water level would leave a width of 1850' in the centre of the river.

As this section is very close to Southend, it was essential to use the long model for this investigation. In the program the barrier closes instantaneously at a specified time before high water Southend; the energy dissipation is represented as a head loss given by

$$\Delta z = \frac{\gamma}{2g}\left(u_b^2 - u_r^2\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

where Δz is the head loss

g is the acceleration due to gravity

u_b is the velocity through the narrowing

u_r is the velocity downstream from the narrowing

and γ is a dimensionless constant in the range 1·0 → 1·5, its precise value seeming to depend on the geometry of the narrowing (Dronkers, 1965).

Figure 9 shows the effect of an 80% closure four hours before high water Southend on the 1953 and 1953 + 2' surges, γ being taken conservatively as 1·0. In both cases the maximum levels are reduced by about 9 ins. throughout the river. The effect of the barriers seems to be to delay and partly reflect the incoming tide and its efficacy depends primarily on the range of tide. As the 1953, 1953 +2', 1953 + 4' 1953 + 6' surges all have the same range they are all reduced in the same way.

Figures 10 and 11 show the water levels on either side of the narrowing. As one might expect the sea level seaward of the narrowing (Figure 10) is initially raised when the barrier is closed. But for this closure of 4 hrs. before high water the maximum level reached is actually reduced. Up river (Figure 11) the tide is both retarded and reduced by the barrier. Figure 12 shows the velocities u_b and u_r as functions of time. Velocities through the narrow opening (Figure 8) reached 14 ft./sec. showing clearly the practical problems associated with this type of defence. Figure 13 suggests that the effectiveness of the barrier is not very sensitive to the actual time of closure provided this is prior to 2 hrs. before high water Southend. The effect of increasing γ to 1·5 is to reduce the maximum levels proportionately, Figure 14.

The effect of the narrowing on the H.R.S. 53 surge with its greater range (Figure 2) and shorter period of high water is very marked (Figure 15) the maximum levels being reduced by two feet or more; the effect on the higher H.R.S. tides would be even more striking.

Thus again a possible defence system has a efficiency which depends on the exact nature of the incoming surge, a transient narrowing being very effective in reducing a short, high surge of large range but relatively ineffective in reducing a high surge with a long stand of high water which eliminates most of the advantage of the delaying action of the narrowing.

Project IV.

One of the most obvious ways to eliminate dangerous water levels is to introduce a solid barrier into the river either permanently as a barrage or temporarily during the period of the surge. This should completely protect the region up river although it may worsen the situaion down river which is therefore the region of primary interest.

As the introduction of a barrier makes a major physical change in the river, the long model has been used to investigate this situation. Closure takes place at a specified time after local low water, the actual process of closure taking half an hour.

The effect of a barrier at Long Reach (section 5) on the observed 53 and the modified 53 tides for various times of closure is shown in Figures 16 and 17. Early closures, that is prior to 2 hours after local low water, actually reduce the down stream levels and closure 3 hours after local low water either has little effect or raises the level slightly. However the closure 4 hours after low water creates a very significant rise in maximum water levels.

The Woolwich barrier (section 7) has been intensively studied by the Hydraulic Research Station. A comparison of their results and those of the long model is shown in Figure 18 and 19 and Table 4 in terms of the changes in the maximum levels resulting from the closures. For the later closures, closures 2, 3 and 4, the agreement between the hydraulic and numerical models is extremely good. With earlier closures there has been more time for artificial disturbances to propagate back into the hydraulic model. In these cases the short model suffering from the same problem gives answers similar to the hydraulic model, particularly at the barrier site, although the maximum change is seaward of that observed in the hydraulic model. In these cases it is reasonable to assume that the results of the long model are correct, the results of the shorter models, numerical or hydraulic, being influenced by spurious reflections. As these results are from the H.R.S. tides a quantitive comparison with the Long Reach results, using the observed tides, is not really possible and it is planned to run the H.R.S. tides at other barrier positions to examine the effect of the site itself. However it can be seen that the later closures again tend to produce worse situations in the river, and an early closure, although not beneficial, is still preferable.

References.

Allen, F.H., Price, W.A. and Sir Claude Inglis. Model experiments on the storm surge of 1953 in the Thames Estuary and the reduction of future surges. Proc. Instr. Civ. Engrs., Hydraulics Paper No. 5, 1955.

Balloffet, A. and Kupferman, A. Hydraulic studies of Jamaica Bay. J. Hydraulics Div., A.S.C.E., 19, Hy 6, Nov. 1964.

Dronkers, J.J. Tidal computations. North-Holland, 1964.

Dronkers, J.J. Discussion of 'Hydraulic studies of Jamaica Bay'. J. Hydraulics Div., A.S.C.E., 91, Hy 5, Sept. 1965.

Lin, H.K., Bradley, J.N. and Plate, E.J. Backwater effects of piers and abutments. Colorado State University, Oct. 1957.

Rossiter, J.R. and Lennon, G.W. Computation of tidal conditions in the Thames Estuary by the initial value method. Proc. Instr. Civil Engnrs., 31 paper 6855, May 1965.

FIGURE 1

THE RIVER THAMES

SHOWING THE SECTIONS OF THE NUMERICAL MODEL AT WHICH THE SURFACE ELEVATION IS DETERMINED

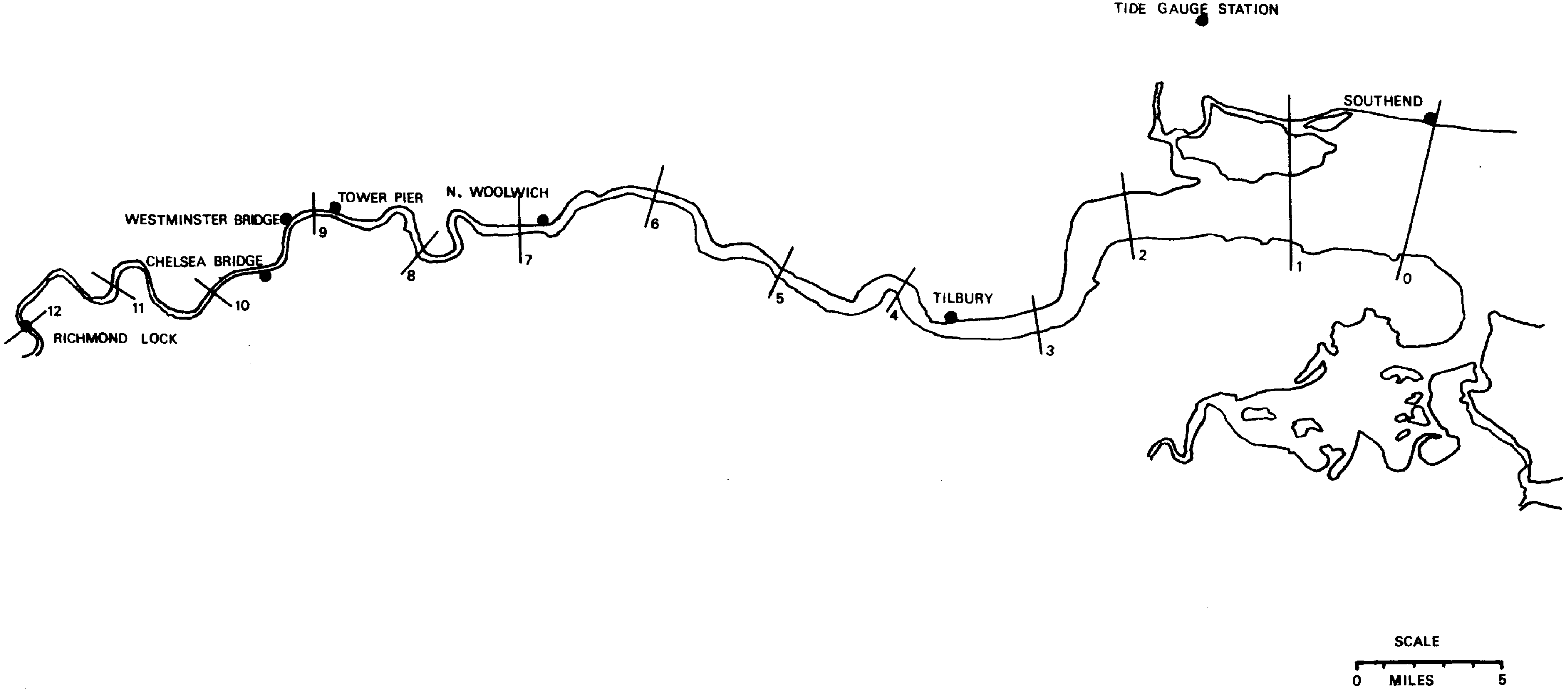


FIGURE 2

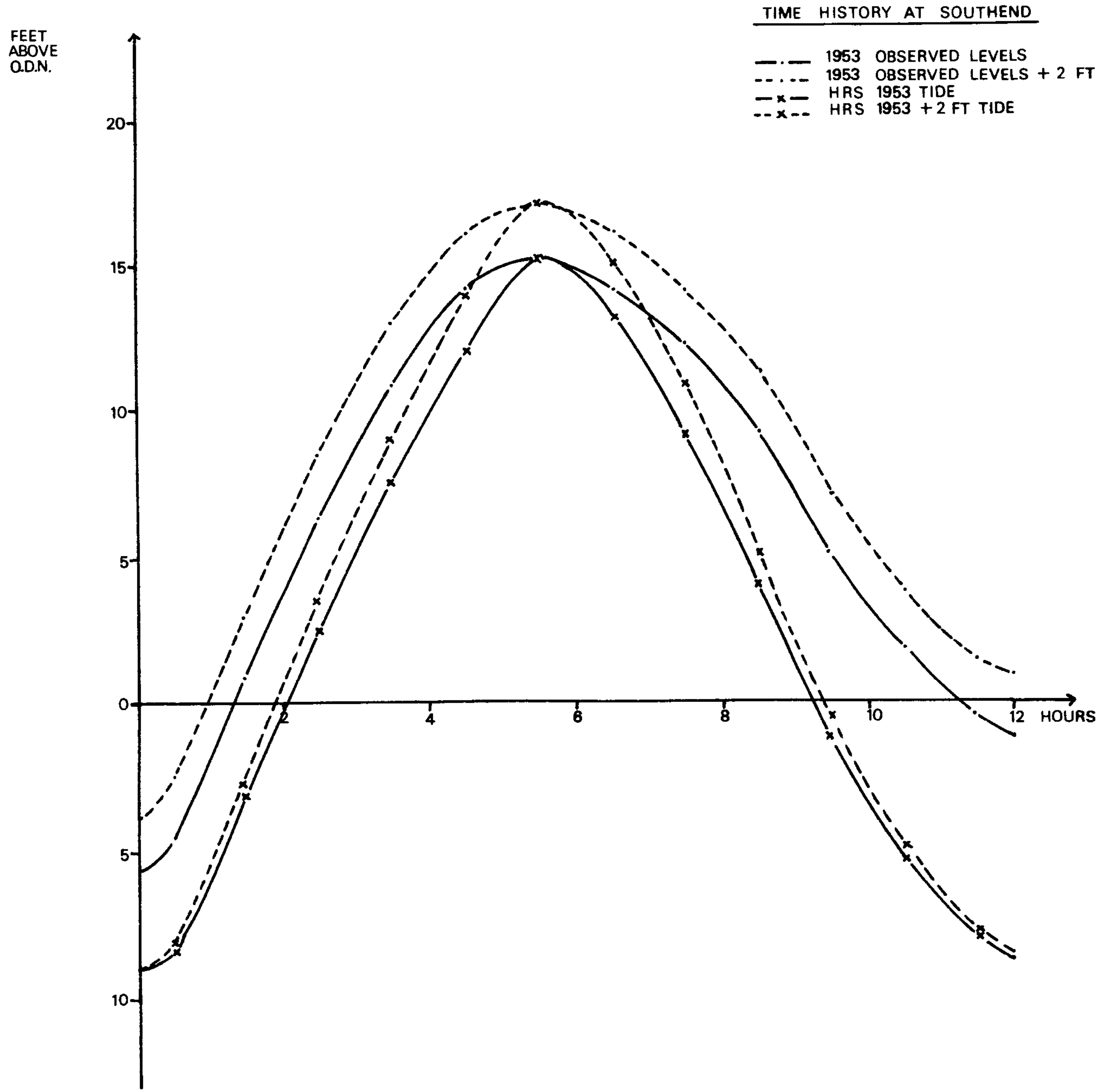

TIME HISTORY AT SOUTHEND
1953 OBSERVED LEVELS
1953 OBSERVED LEVELS + 2 FT
HRS 1953 TIDE
HRS 1953 + 2 FT TIDE
FEET ABOVE O.D.N.
20
15
10
5
0
5
10
2
4
6
8
10
12 HOURS

FIGURE 3

MAXIMUM LEVELS ALONG RIVER THAMES DURING 1953 STORM SURGE

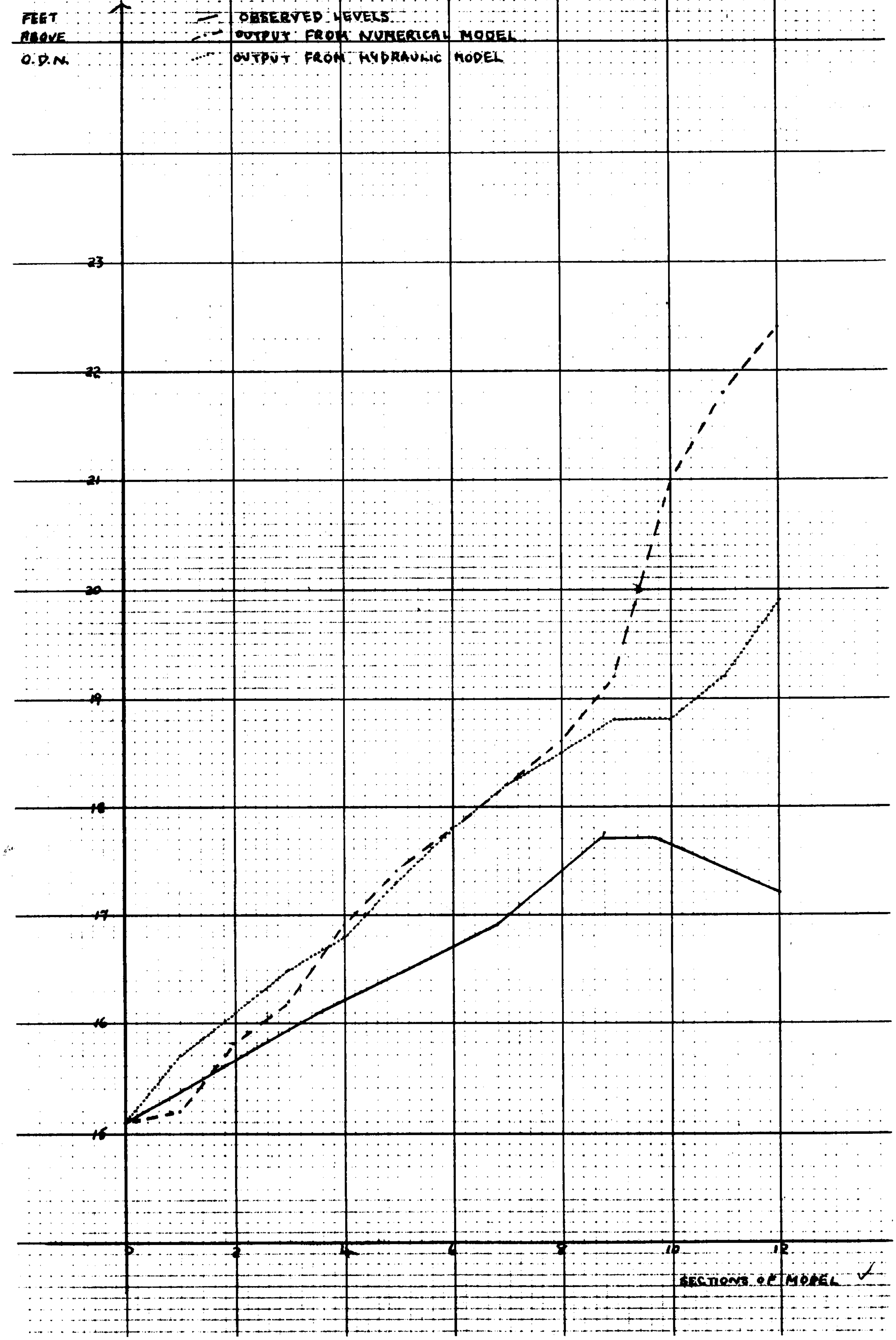


FIGURE 4

MAXIMUM LEVELS ALONG RIVER THAMES

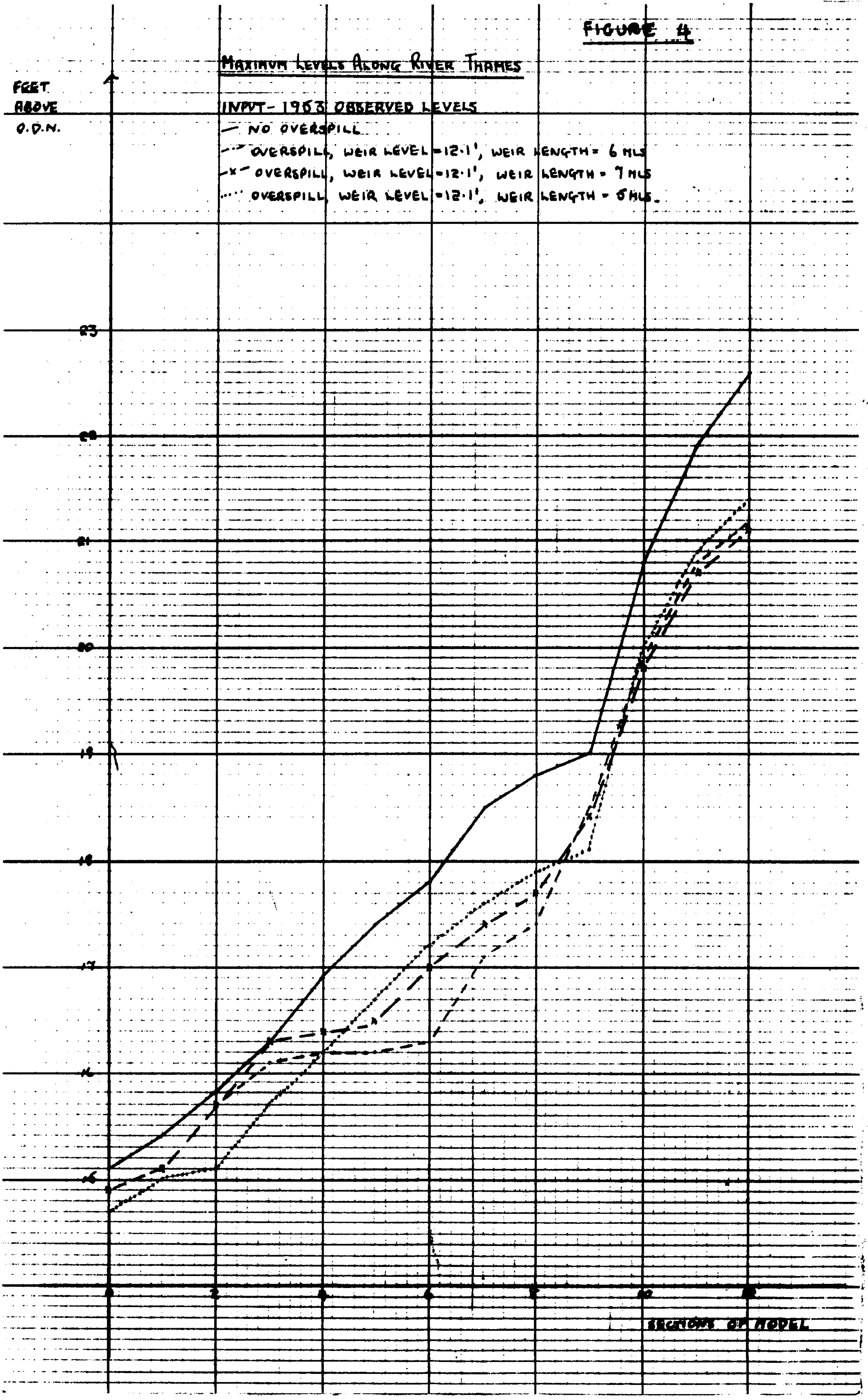


FIGURE 5

MAXIMUM LEVELS ALONG RIVER THAMES

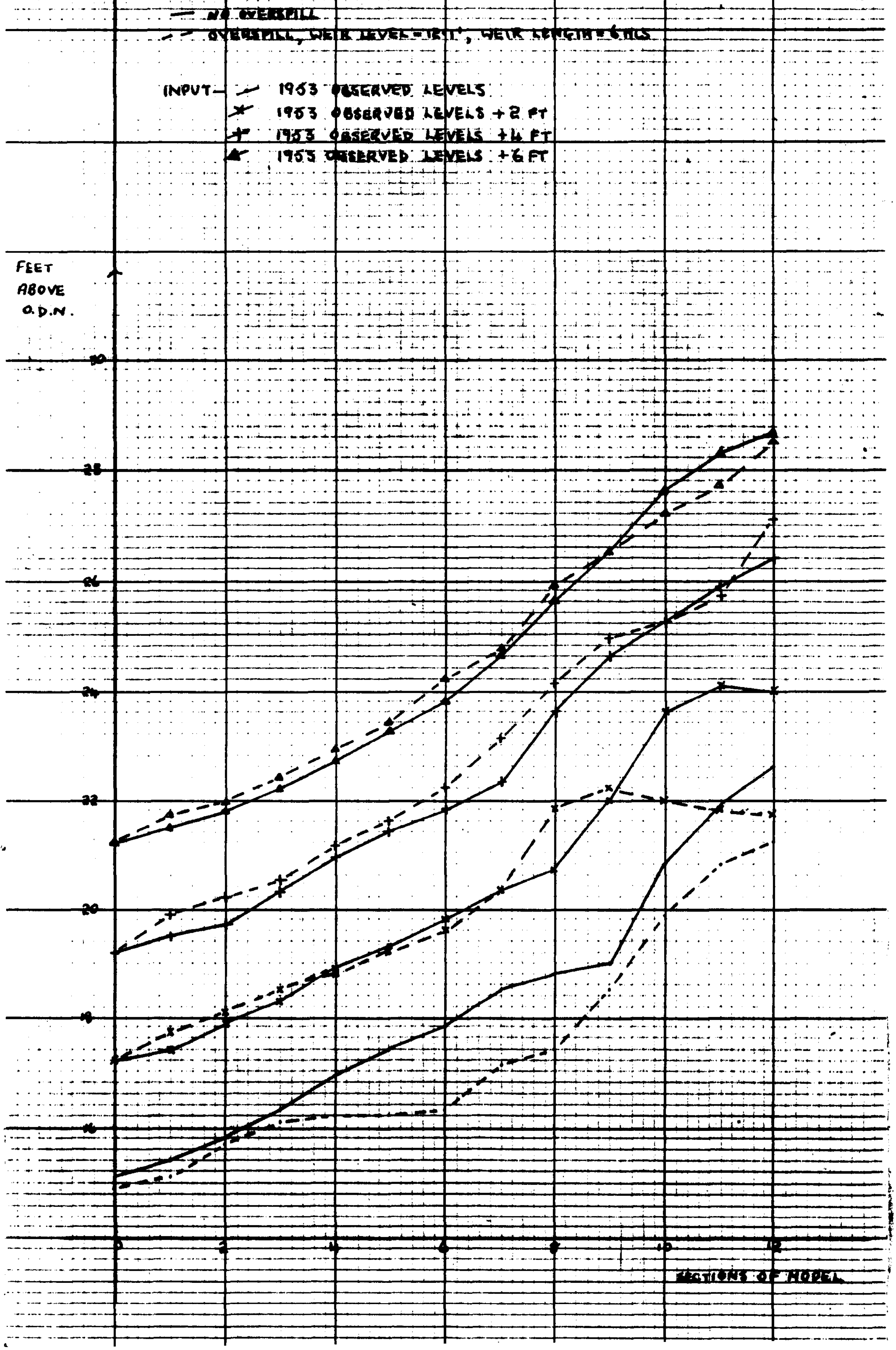
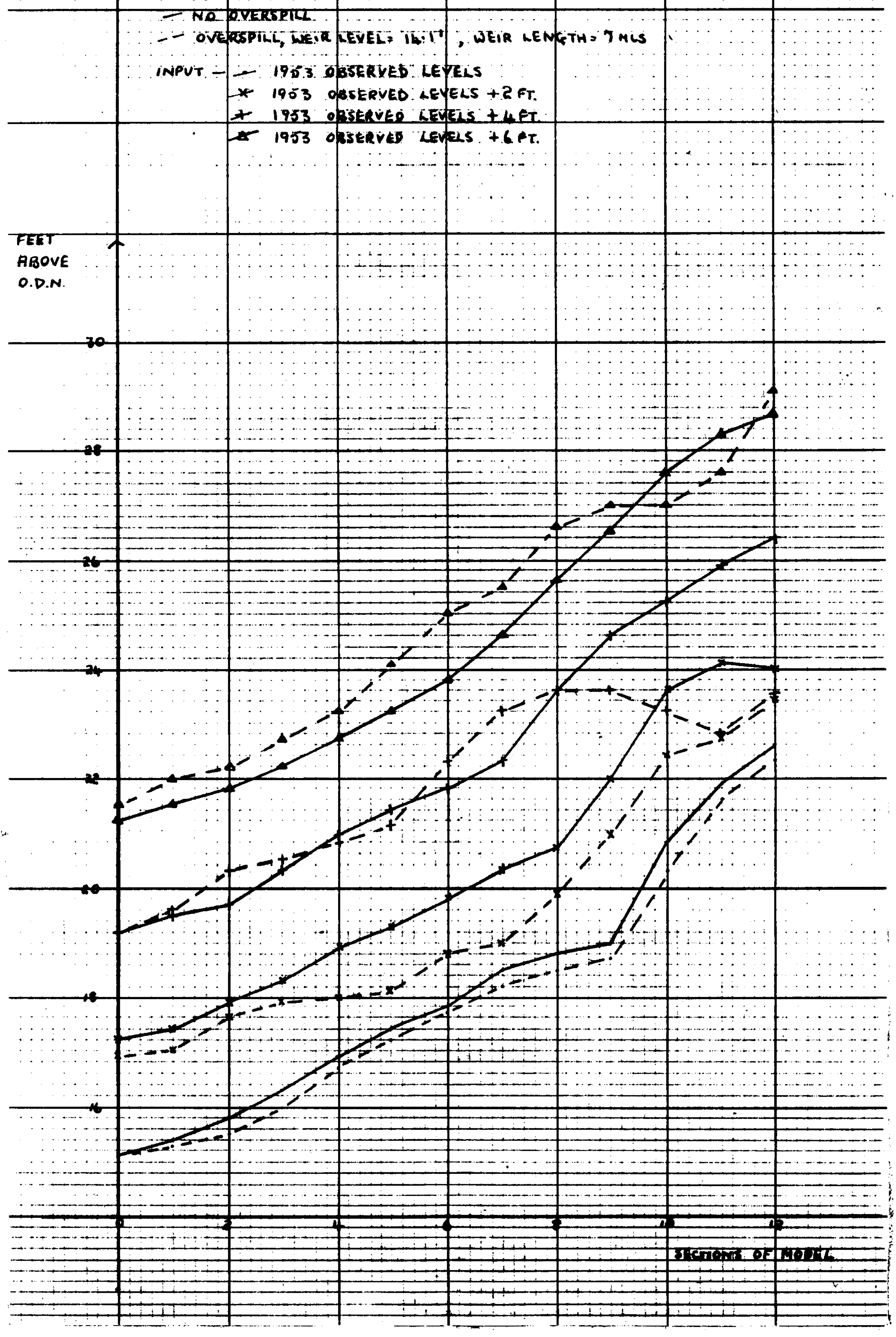


FIGURE 6

MAXIMUM LEVELS ALONG RIVER THAMES



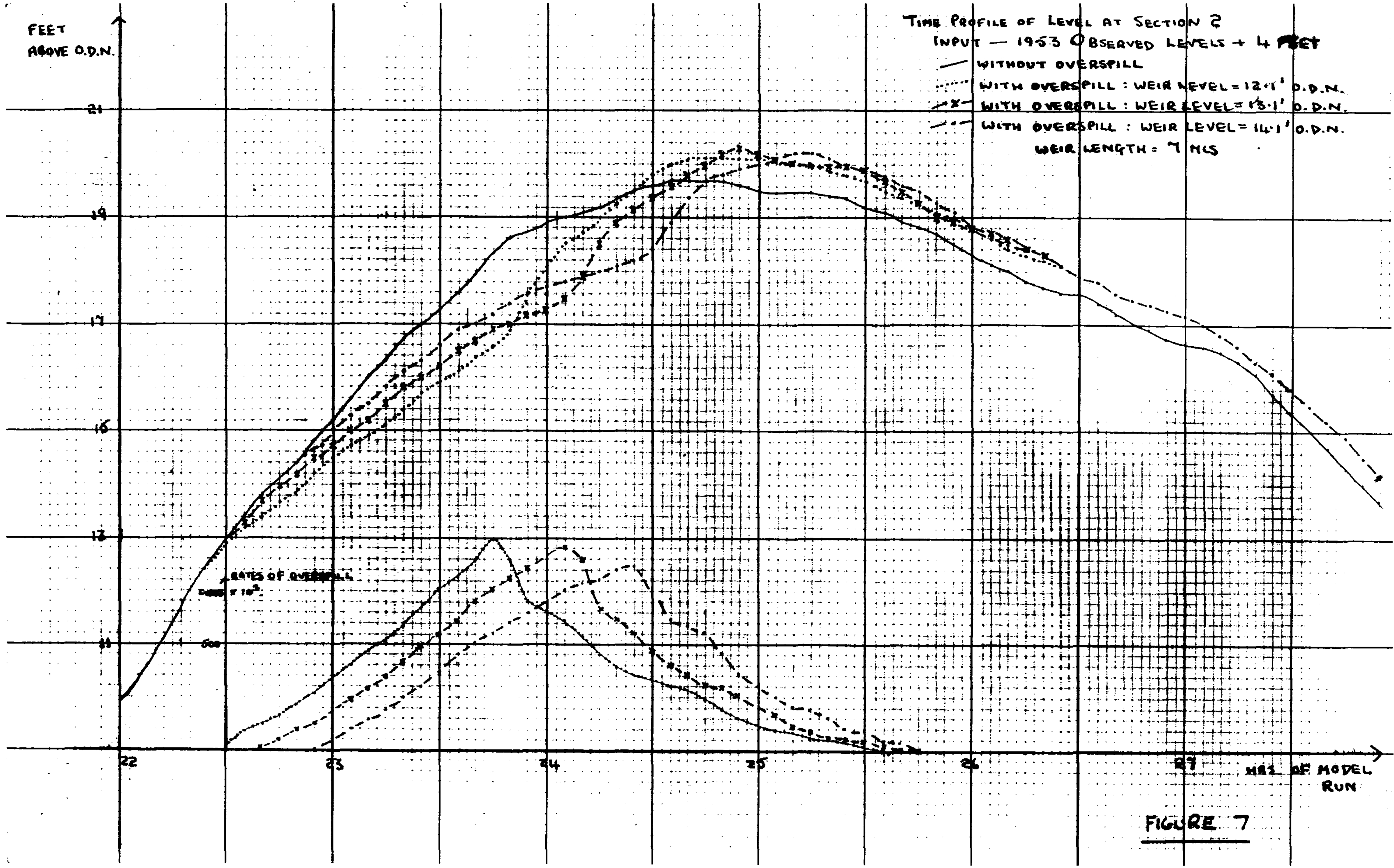

FEET
ABOVE O.D.N.
21
19
17
15
13
11
500
RATES OF OVERSPILL
[illegible] × 10³
22
23
24
25
26
27
HRS OF MODEL RUN
TIME PROFILE OF LEVEL AT SECTION 2
INPUT — 1953 OBSERVED LEVELS + 4 FEET
WITHOUT OVERSPILL
WITH OVERSPILL : WEIR LEVEL = 12·1' O.D.N.
WITH OVERSPILL : WEIR LEVEL = 13·1' O.D.N.
WITH OVERSPILL : WEIR LEVEL = 14·1' O.D.N.
WEIR LENGTH = 7 MLS

FIGURE 7

FIGURE 8

CROSS SECTION AT PROPOSED SITE OF NARROWING

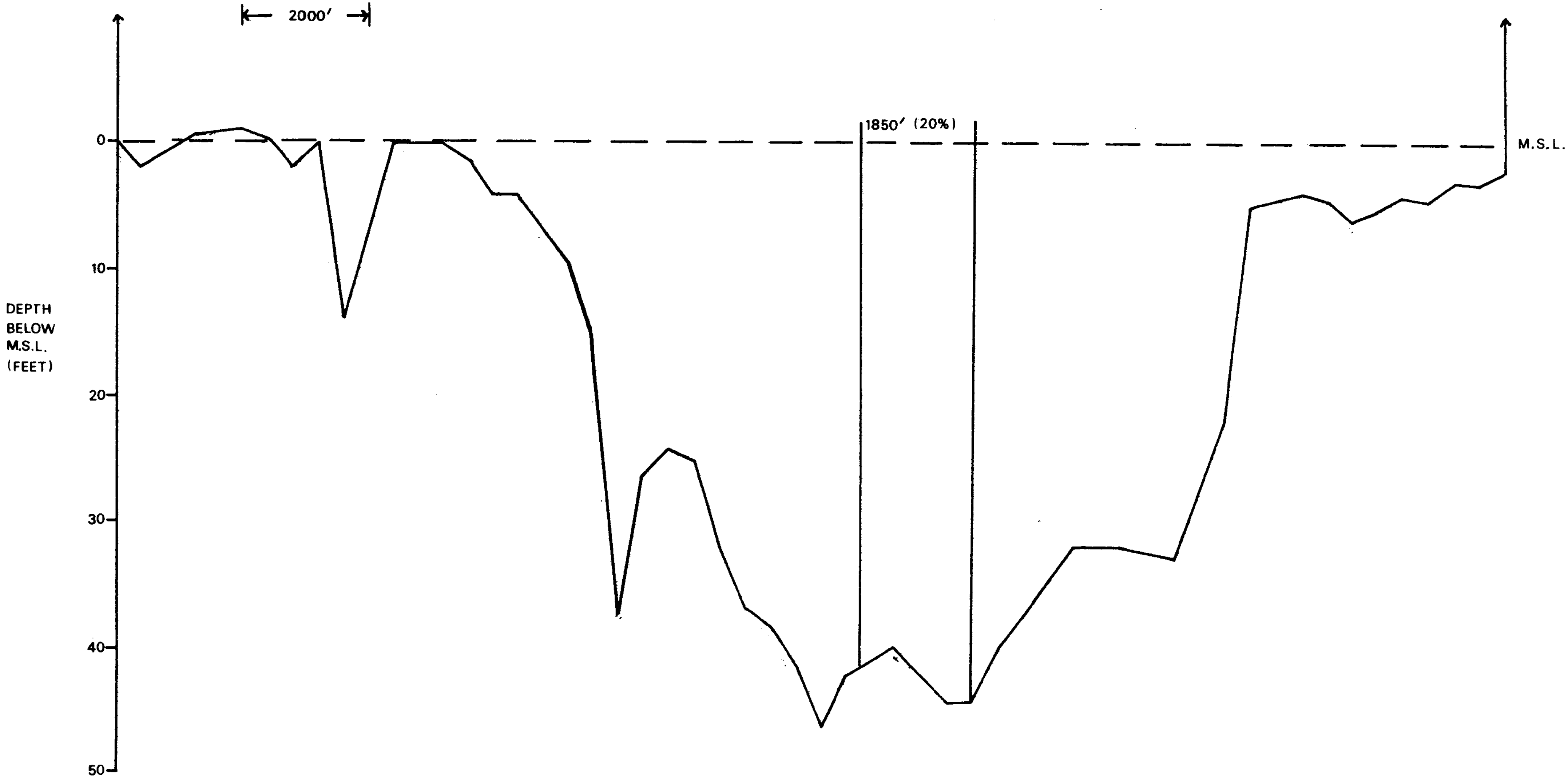
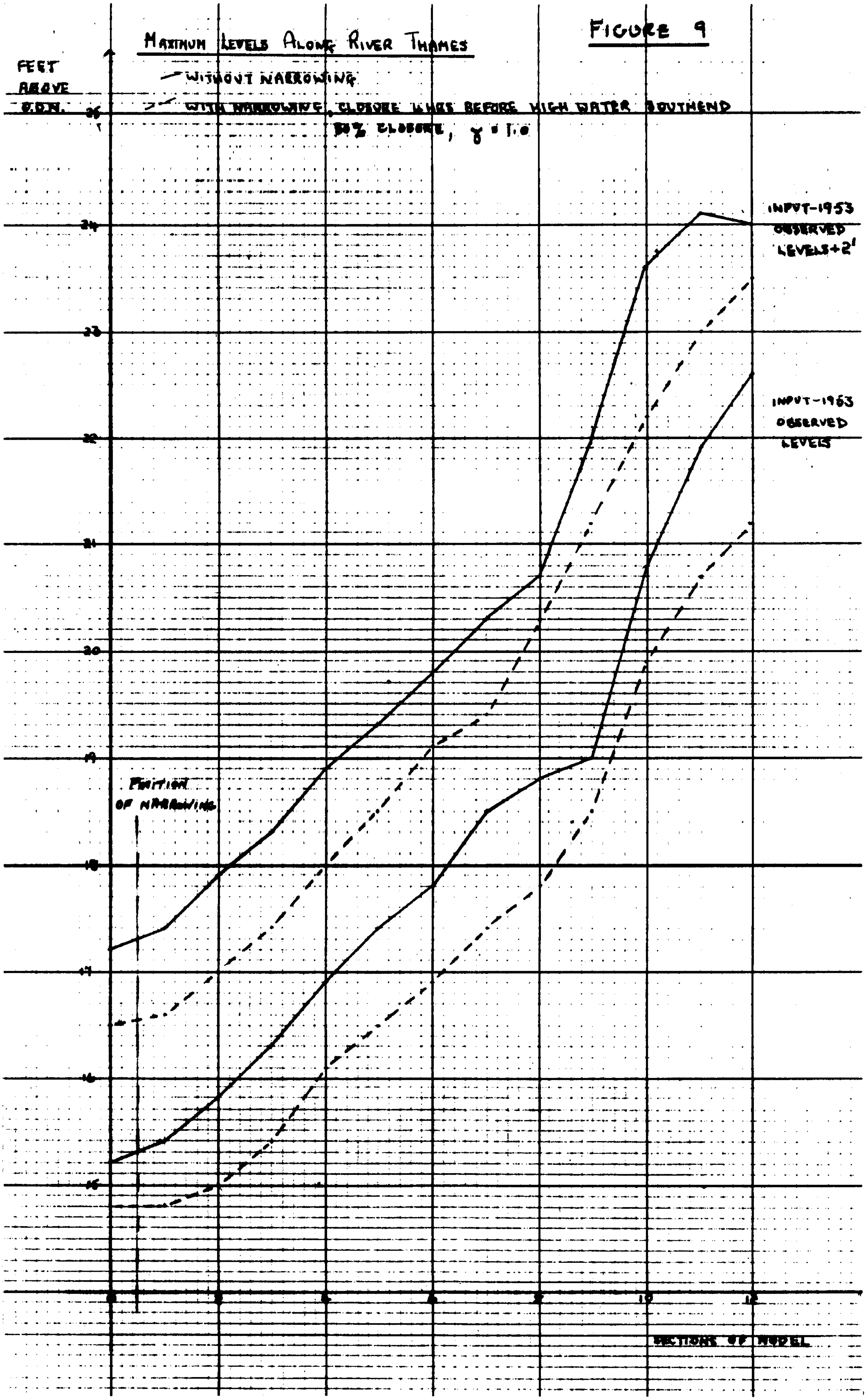


FIGURE 9


MAXIMUM LEVELS ALONG RIVER THAMES
WITHOUT NARROWING
WITH NARROWING, CLOSURE 4 HRS BEFORE HIGH WATER SOUTHEND
50% CLOSURE, γ = 1.0
FEET ABOVE O.D.N.
25
24
23
22
21
20
19
18
17
16
15
INPUT-1953 OBSERVED LEVELS+2'
INPUT-1953 OBSERVED LEVELS
POSITION OF NARROWING
0
2
4
6
8
10
12
SECTIONS OF MODEL

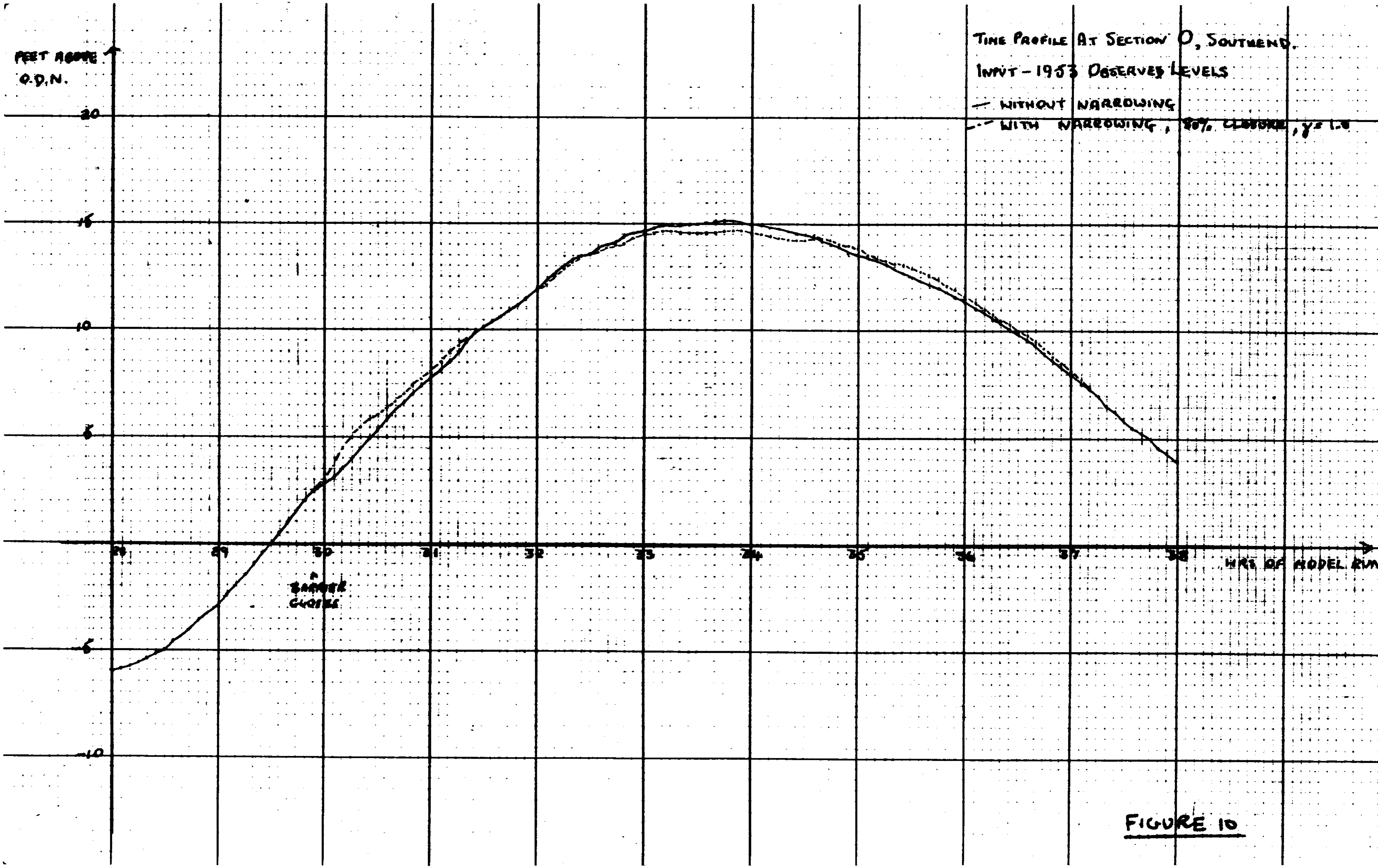

TIME PROFILE AT SECTION O, SOUTHEND.
INPUT - 1953 OBSERVED LEVELS
— WITHOUT NARROWING
-·- WITH NARROWING, 80% CLOSURE, γ= 1.0
FEET ABOVE O.D.N.
20
15
10
5
-5
-10
28
29
30
31
32
33
34
35
36
37
38
HRS OF MODEL RUN
BARRIER CLOSES

FIGURE 10

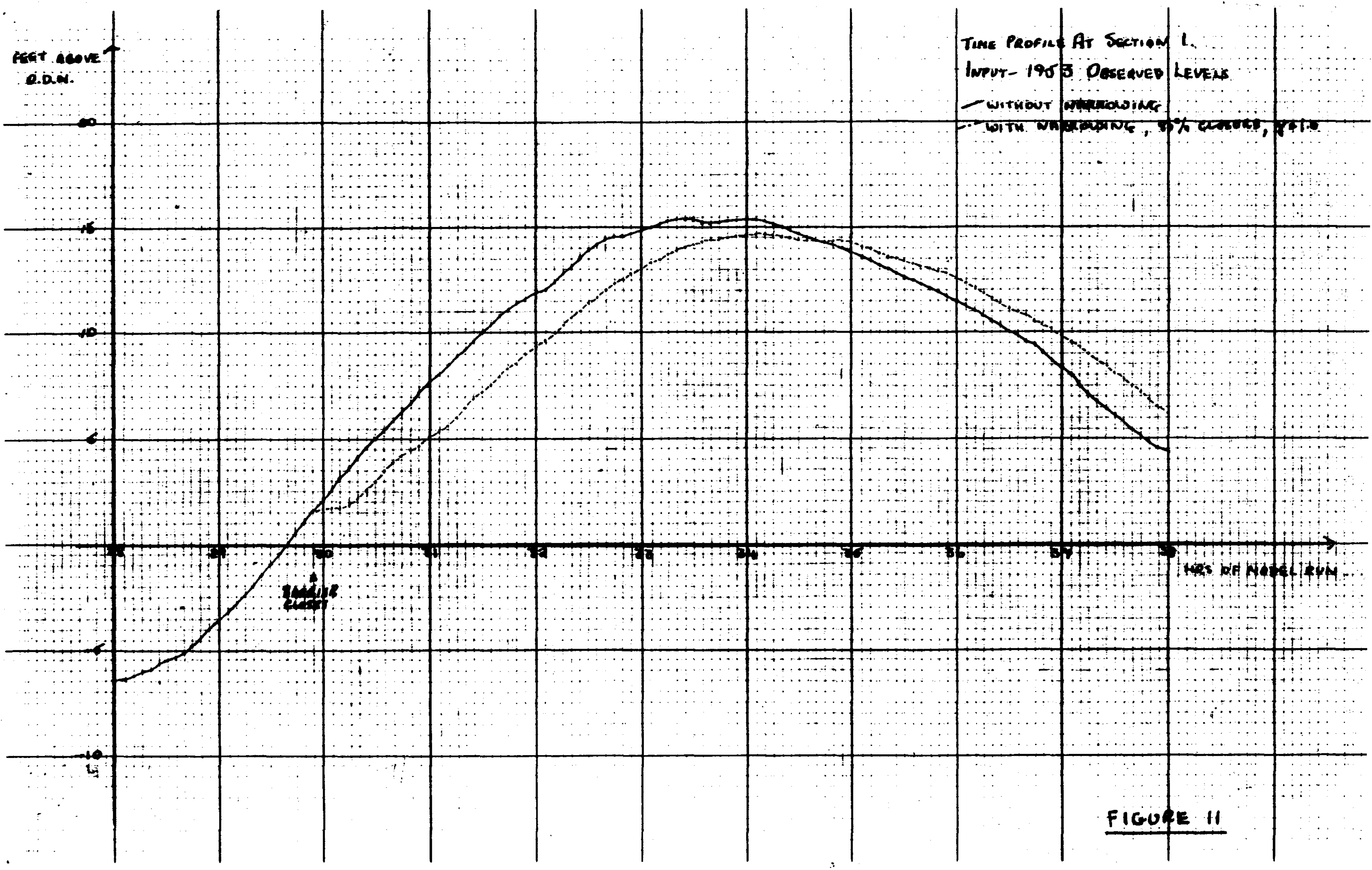

TIME PROFILE AT SECTION L.
INPUT - 1953 OBSERVED LEVELS
WITHOUT WASHLANDING
WITH WASHLANDING, 90% CLOSURE, [illegible]
FEET ABOVE O.D.N.
20
15
10
5
-5
-10
28
29
30
31
32
33
34
35
36
37
38
HRS OF MODEL RUN
BARRIER CLOSED

FIGURE 11

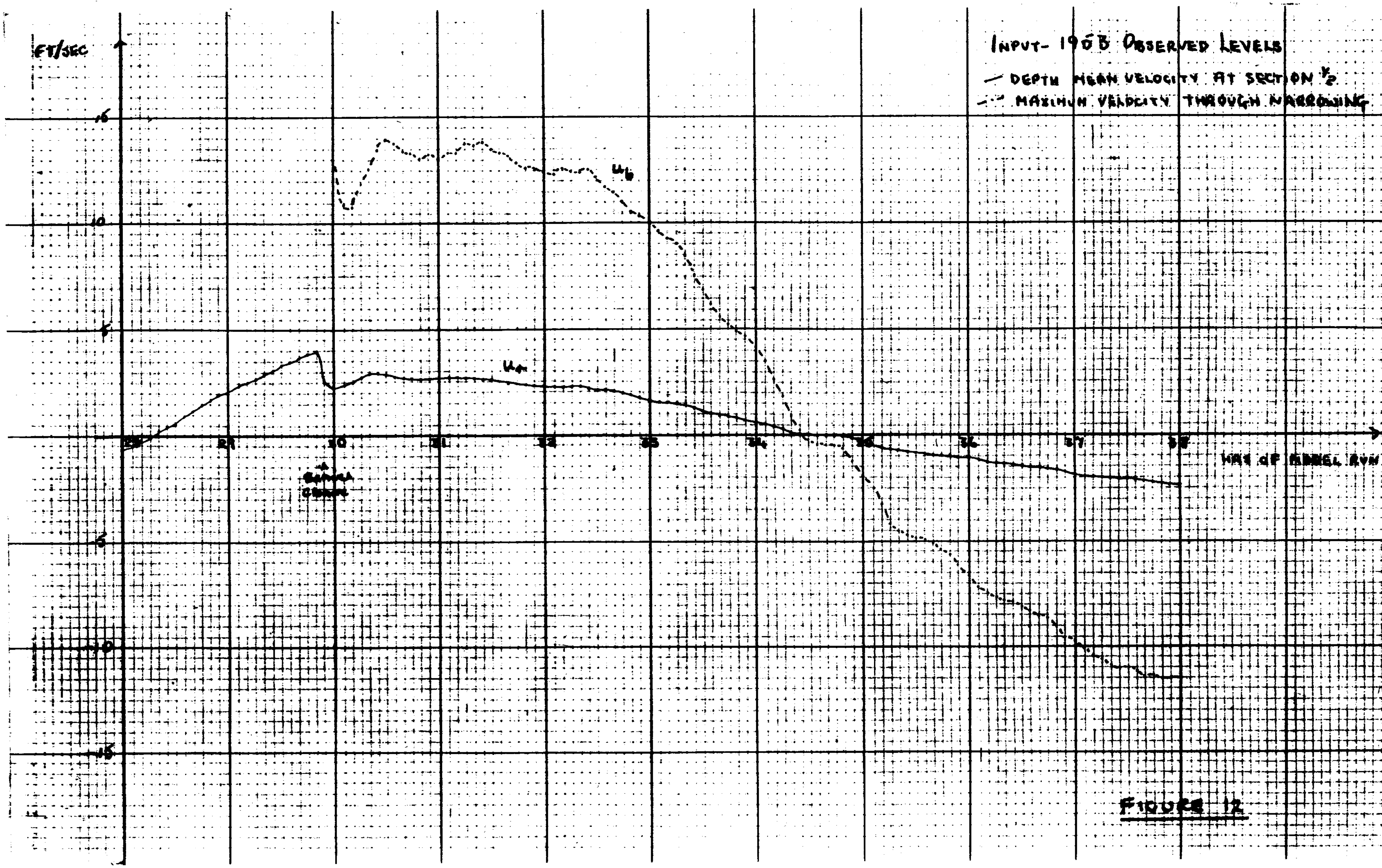
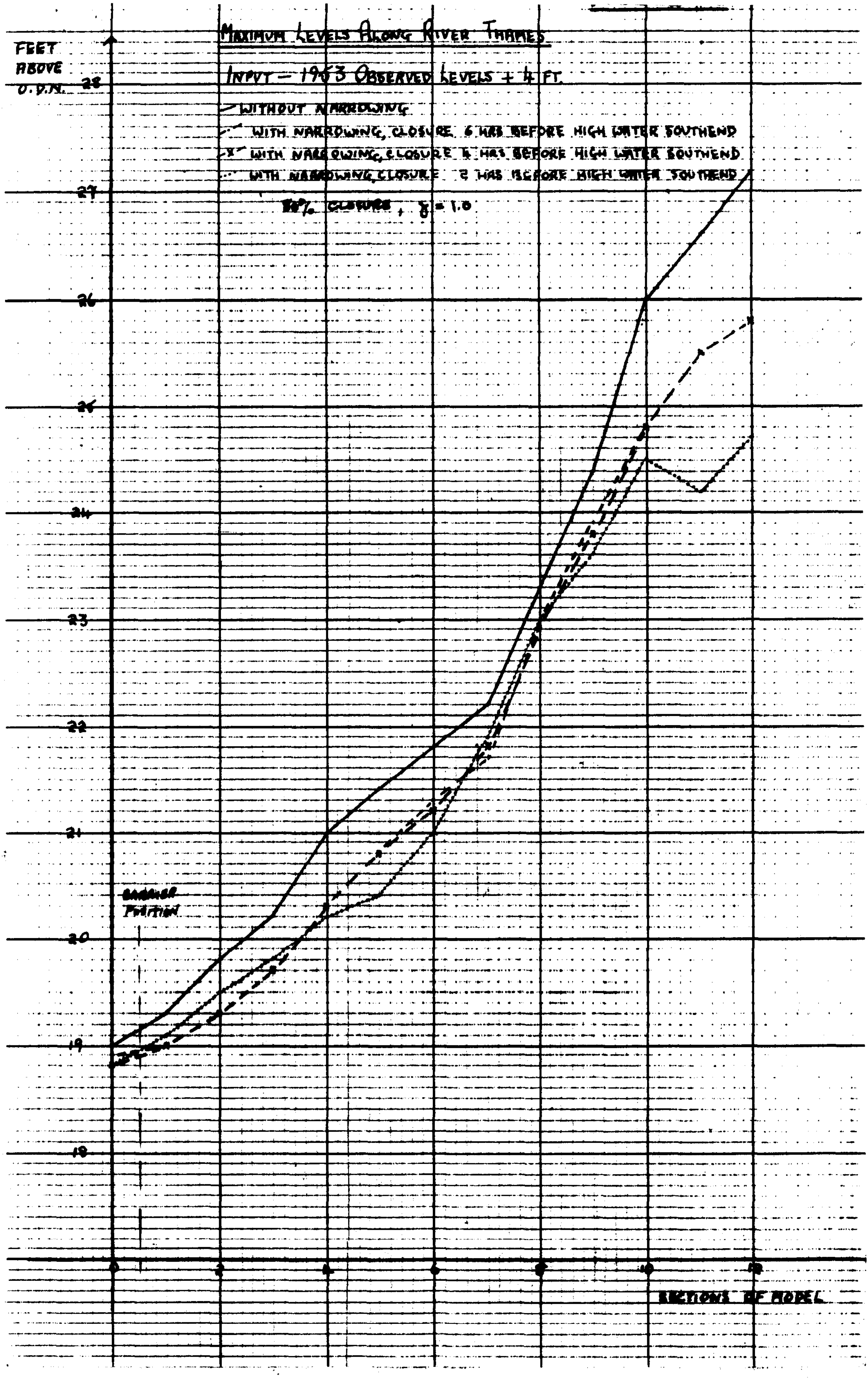

FT/SEC
15
10
5
-5
-10
-15
28
29
30
31
32
33
34
35
36
37
38
HRS OF MODEL RUN
u_b
u_a
INPUT- 1908 OBSERVED LEVELS
— DEPTH MEAN VELOCITY AT SECTION ½
-·- MAXIMUM VELOCITY THROUGH NARROWING

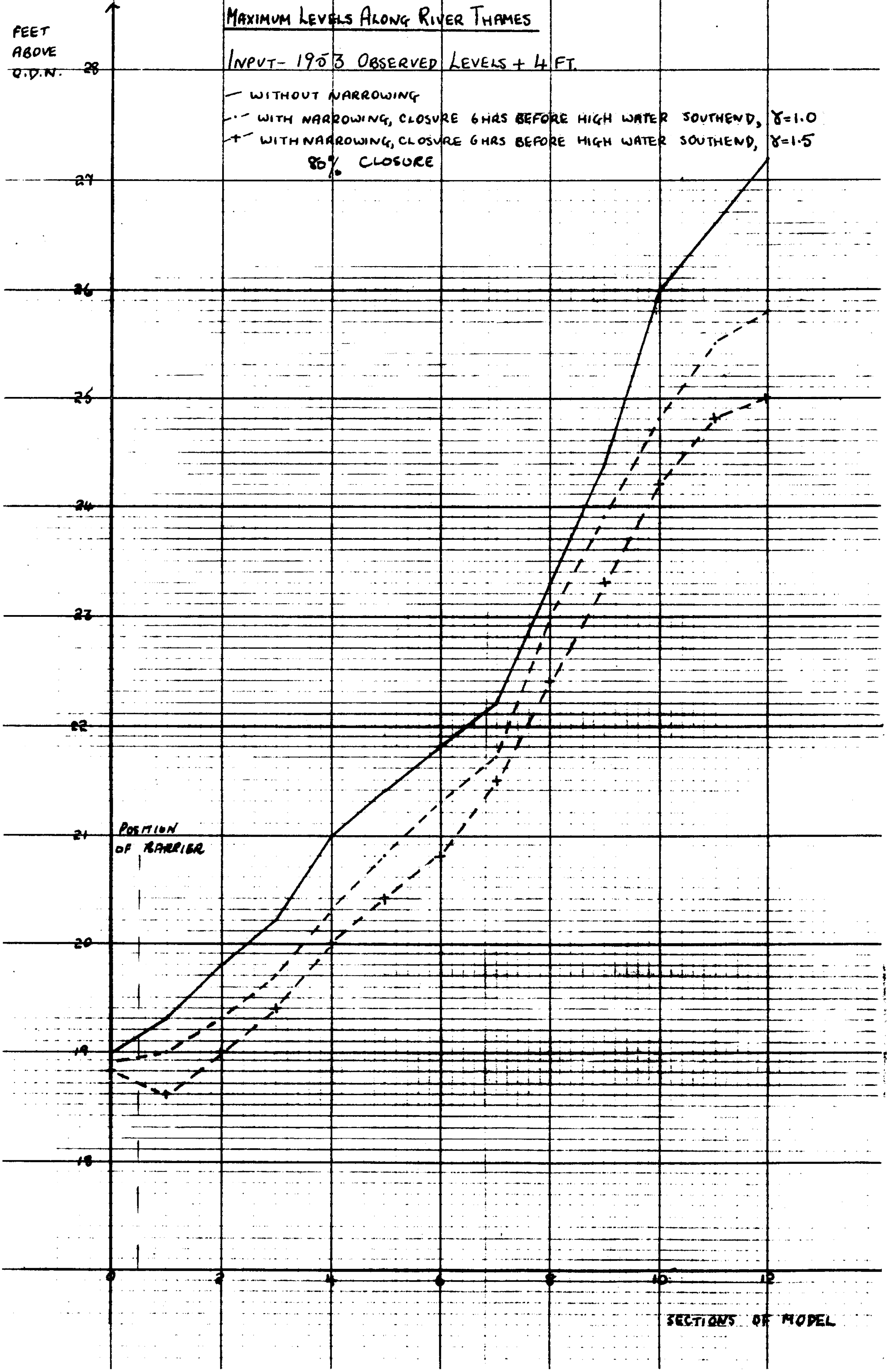
FIGURE 12



MAXIMUM LEVELS ALONG RIVER THAMES

INPUT – 1953 OBSERVED LEVELS + 4 FT.

FIGURE 14

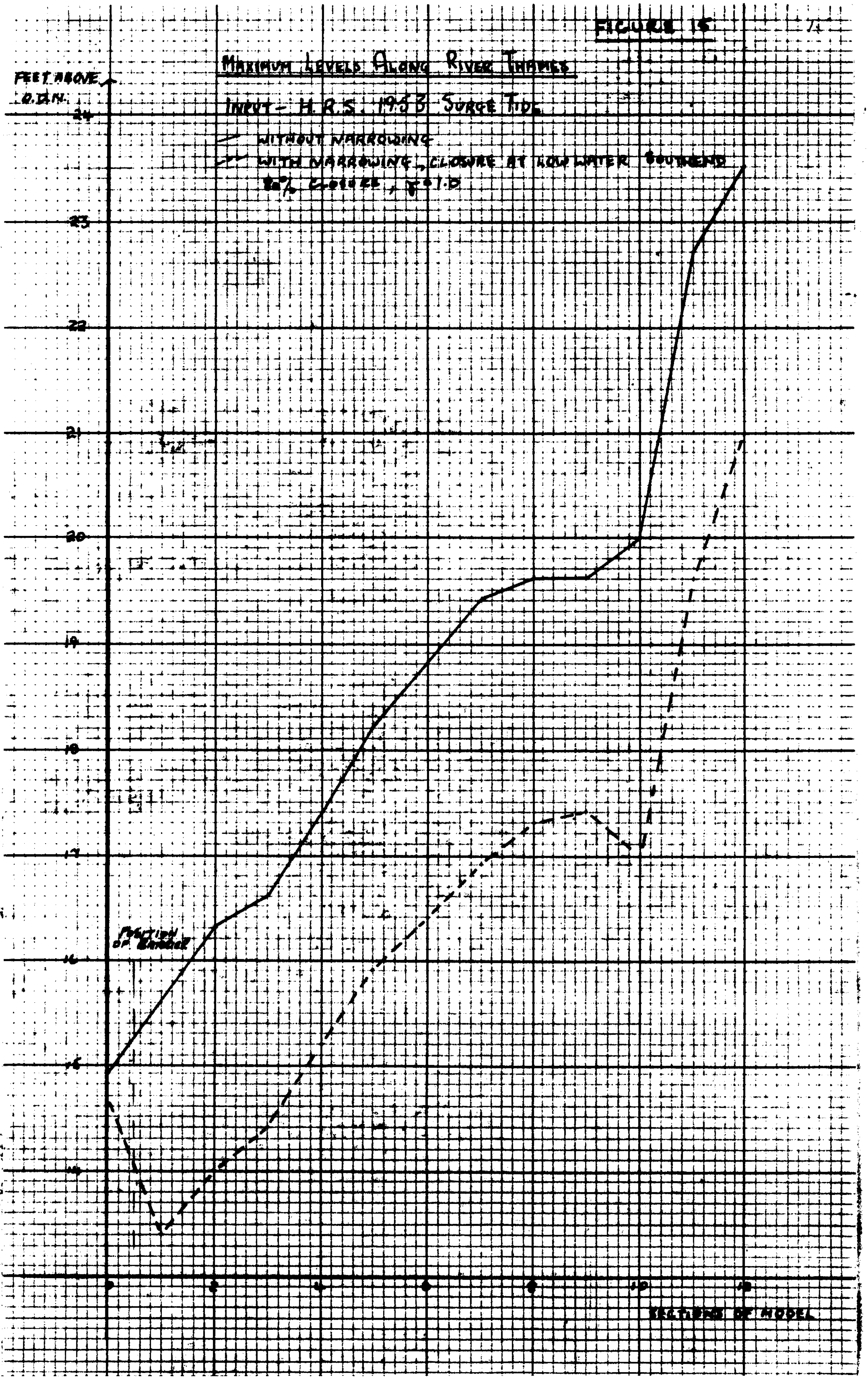


MAXIMUM LEVELS ALONG RIVER THAMES

INPUT - 1953 OBSERVED LEVELS + 4 FT.

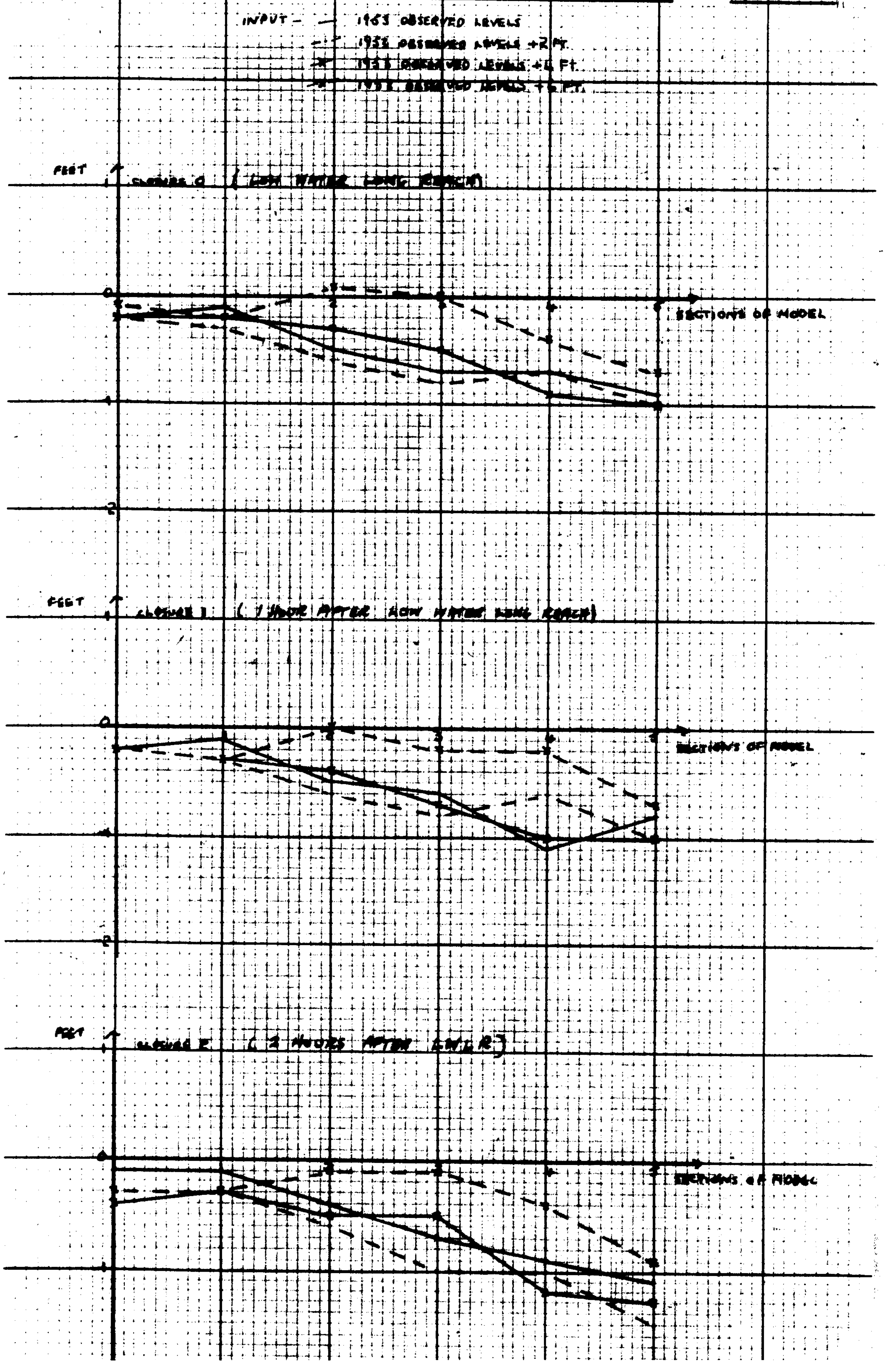
FIGURE 15

MAXIMUM LEVELS ALONG RIVER THAMES



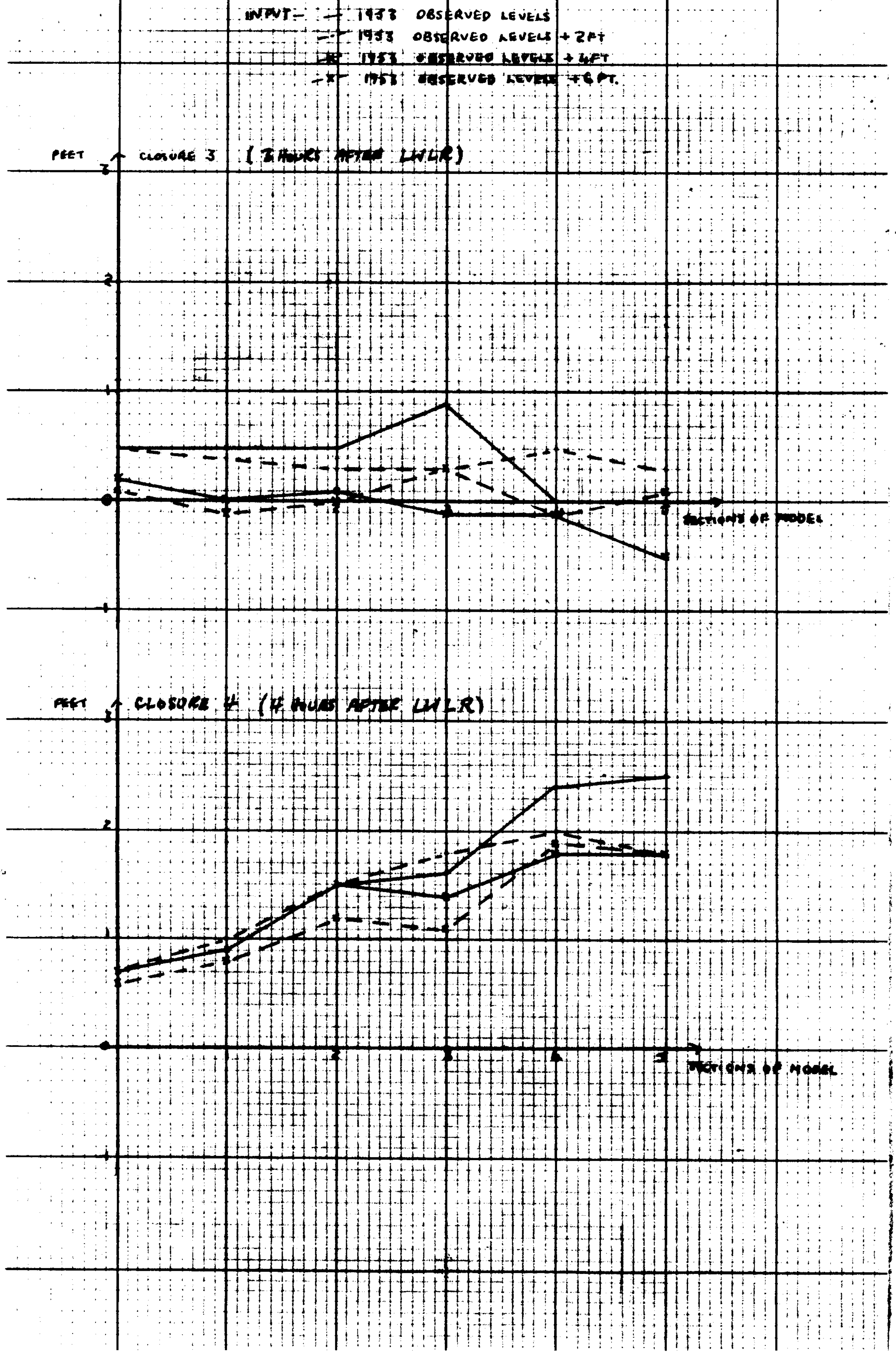
CHANGES IN SURGE HIGH-WATER LEVELS WITH LONG REACH BARRIER

FIGURE 16



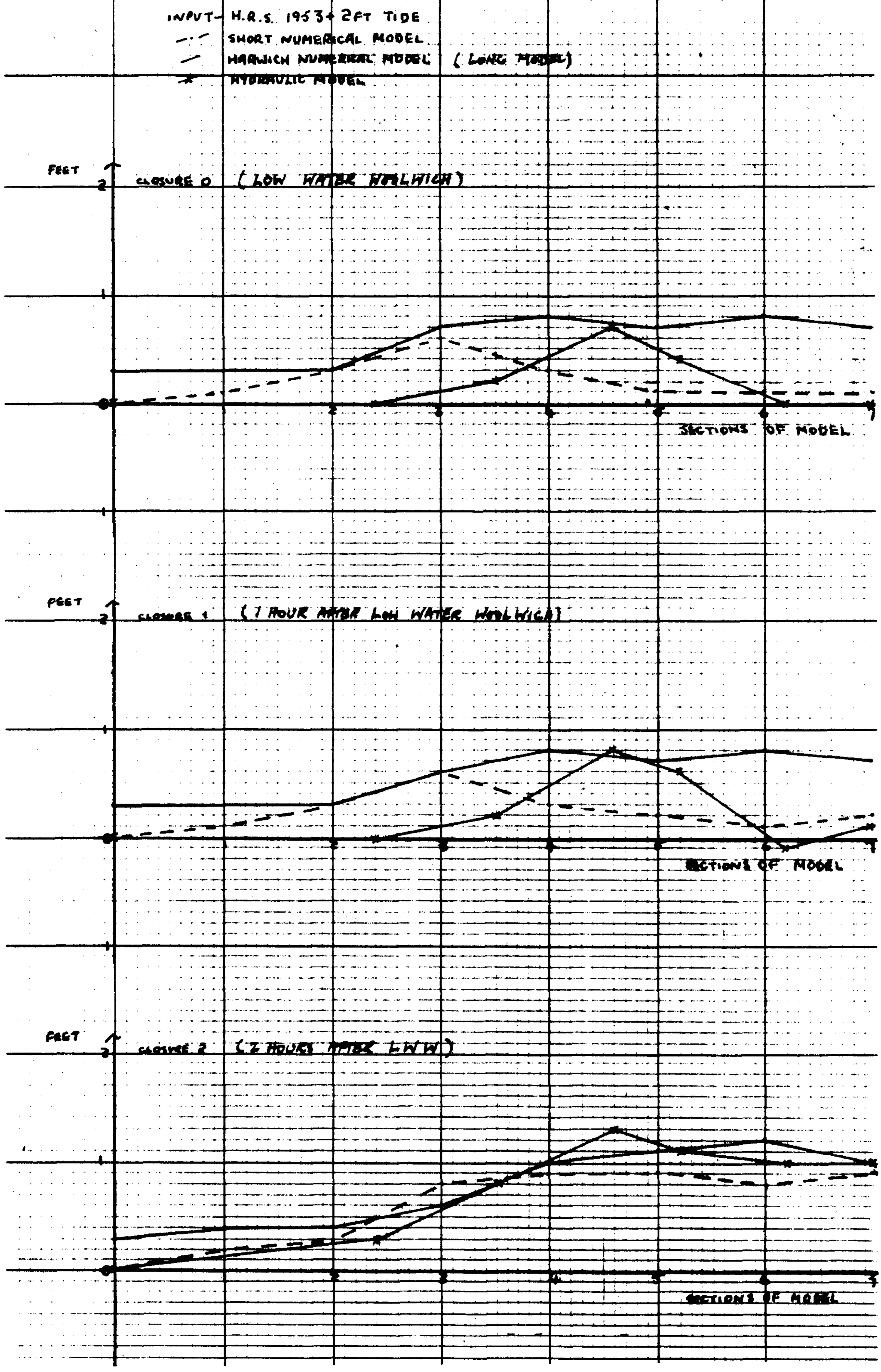
CHANGES IN SURGE HIGH-WATER LEVELS WITH LONG REACH BARRIER

FIGURE 17



CHANGES IN SURGE HIGH-WATER LEVELS WITH WOOLWICH BARRIER

FIGURE 18



CHANGES IN SURGE HIGH-WATER LEVELS WITH WOOLWICH BARRIER

FIGURE 19

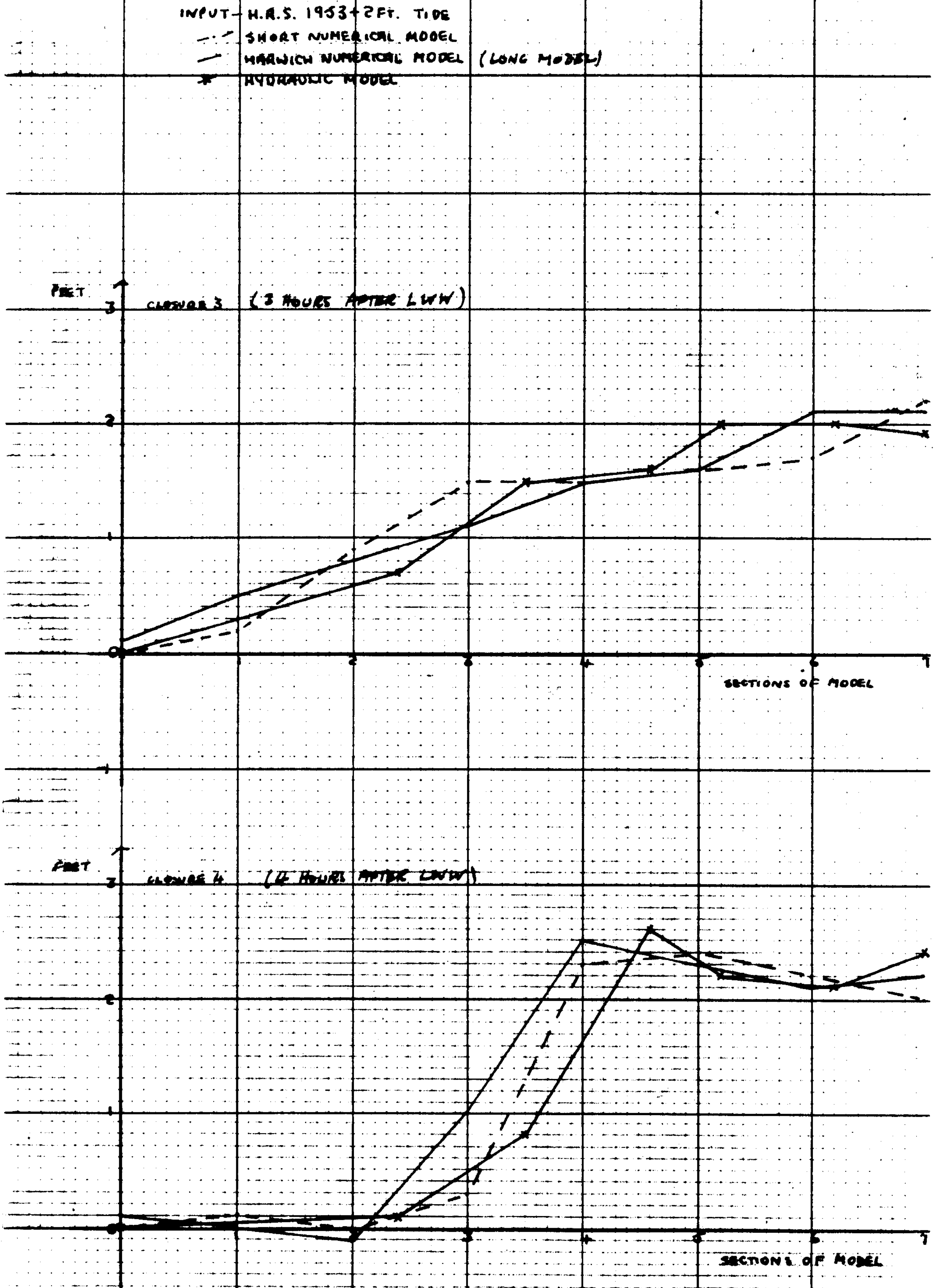


TABLE I

MAXIMUM LEVELS FROM MODEL OUTPUT (FEET ABOVE O.D.N.) WITH INPUT AT SOUTHEND

Section of Model	0	1	2	3	4	5	6	7	8	9	10	11	12
Input - Observed Levels	15.1	15.3	15.8	16.2	**16.9**	17.4	17.8	18.2	18.6	19.2	21.0	21.8	22.4
Input - Observed Levels + 2'	17.1	17.3	17.8	18.2	**18.8**	19.3	19.7	20.1	20.8	21.9	23.4	24.0	24.2
Input - Observed Levels + 4'	19.1	19.3	19.7	20.1	**20.7**	21.2	21.6	22.0	23.4	24.3	25.5	26.1	26.3
Input - Observed Levels + 6'	21.1	21.3	21.7	22.0	**22.7**	23.1	23.5	24.5	25.8	26.5	27.3	28.0	28.3
Input - Observed Levels + (a)	16.5	16.8	17.1	17.6	**18.3**	18.9	19.4	19.6	20.0	21.1	22.8	23.6	24.1
Input - Observed Levels + (b)	17.1	17.4	17.8	18.2	**19.0**	19.4	20.0	20.3	20.6	21.9	23.6	24.2	24.7
Input - Observed Levels + (c)	16.5	16.8	17.2	17.6	**18.3**	18.9	19.3	19.6	20.0	21.3	22.8	23.4	23.9
Input - Observed Levels + (d)	17.1	17.4	17.8	18.3	**19.2**	19.8	20.3	20.6	20.9	21.7	24.0	24.7	25.7

(a) - The positive part of one cycle of a diurnal oscillation which reached a peak of 2' at 2200 hours on January 31st

(b) - The positive part of one cycle of a diurnal oscillation which reached a peak of 2' at 0100 hours on February 1st

(c) - The positive part of one cycle of a diurnal oscillation which reached a peak of 2' at 0400 hours on February 1st

(d) - A semi-diurnal oscillation of amplitude 2' which reached a peak at 0100 hours on February 1st

TABLE 2

Maximum Levels from Model Output (Feet Above O.D.N.)

With Input of 1953 Observed Levels

Section of Model	No Overspill	Overspill with Weir Level of 14.1 Feet O.D.N.						
		Weir Length of 7 Miles	Weir Length of 6 Miles	Weir Length of 5 Miles	Weir Length of 4 Miles	Weir Length of 3 Miles	Weir Length of 2 Miles	Weir Length of 1 Mile
0	15.2	14.9	14.9	14.7	15.0	15.0	14.8	15.2
1	15.4	15.1	15.1	15.0	15.2	15.2	15.2	15.3
2	15.8	15.7	15.7	15.4	15.3	15.4	15.5	15.5
3	16.3	16.3	16.1	15.7	15.7	15.7	16.1	15.8
4	16.9	16.4	16.2	16.2	16.3	16.5	16.7	16.8
5	17.4	16.5	16.2	16.7	16.9	17.0	17.1	17.3
6	17.8	17.0	16.3	17.2	17.2	17.3	17.4	17.6
7	18.5	17.4	17.1	17.6	17.7	17.9	18.0	18.2
8	18.8	17.7	17.4	17.9	18.0	18.2	18.3	18.5
9	19.0	18.4	18.5	18.1	18.2	18.4	18.5	18.7
10	20.8	19.8	19.9	20.0	20.2	20.3	20.5	20.6
11	21.9	20.7	20.8	20.9	21.0	21.3	21.4	21.6
12	22.6	21.1	21.2	21.4	21.5	21.8	22.0	22.3

TABLE 3

Maximum Levels From Model Output (Feet Above O.D.N.) With An Overspill Weir Level of 12.1 Ft. and Length of 6 Miles

Section of Model	1953 Levels Input		1953 Levels +2Ft Input		1953 Levels +4Ft Input		1953 Levels +6Ft Input	
	No Overspill	Overspill	No Overspill	Overspill	No Overspill	Overspill	No Overspill	Overspill
0	15.2	14.9	17.2	17.2	19.2	19.2	21.2	21.2
1	15.4	15.1	17.4	17.7	19.5	19.9	21.5	21.7
2	15.8	15.7	17.9	18.1	19.7	20.2	21.8	22.0
3	16.3	16.1	18.3	18.5	20.3	20.5	22.2	22.4
4	16.9	16.2	18.9	18.8	20.9	21.1	22.7	22.9
5	17.4	16.2	19.3	19.2	21.4	21.6	23.2	23.4
6	17.8	16.3	19.8	19.6	21.8	22.2	23.8	24.2
7	18.5	17.1	20.3	20.3	22.3	23.1	24.6	24.7
8	18.8	17.4	20.7	21.9	23.6	24.1	25.6	25.9
9	19.0	18.5	22.0	22.2	24.6	24.9	26.5	26.5
10	20.8	19.9	23.6	22.0	25.2	25.2	27.6	27.2
11	21.9	20.8	24.1	21.8	25.9	25.7	28.3	27.7
12	22.6	21.2	24.0	21.7	26.4	27.1	28.7	28.5

Table 4

Changes in Surge High Water Levels with Woolwich Barrier

Input - H.R.S. 1953 +2Ft. Tide

Section	Closure 0 Complete at Local Low Water			Closure 1 Complete 1 Hour After Local Low Water			Closure 2 Complete 2 Hours after Local Low Water			Closure 3 Complete 3 Hours After Local Low Water			Closure 4 Complete 4 Hours after Local Low Water		
	Short Model	Harwich Model	Hydraulic Model	Short Model	Harwich Model	Hydraulic Model	Short Model	Harwich Model	Hydraulic Model	Short Model	Harwich Model	Hydraulic Model	Short Model	Harwich Model	Hydraulic Model
0	0.0	+0.3	0.0	0.0	+0.3	0.0	0.0	+0.3	0.0	0.0	+0.1	0.0	0.0	+0.1	0.0
1	+0.1	+0.3	-	+0.1	+0.3	-	+0.2	+0.4	-	+0.2	+0.5	-	+0.1	0.0	-
2	+0.3	+0.3	-	+0.3	+0.3	-	+0.3	+0.4	-	+0.9	+0.8	-	0.0	-0.1	-
2.4	-	-	0.0	-	-	0.0	-	-	+0.3	-	-	+0.7	-	-	+0.1
3	+0.6	+0.7	-	+0.6	+0.6	-	+0.8	+0.6	-	+1.5	+1.1	-	+0.3	+1.0	-
3.5	-	-	+0.2	-	-	+0.2	-	-	+0.8	-	-	+1.5	-	-	+0.8
4	+0.3	+0.8	-	+0.3	+0.8	-	+0.9	+1.0	-	+1.5	+1.5	-	+2.3	+2.5	-
4.6	-	-	+0.7	-	-	+0.8	-	-	+1.3	-	-	+1.6	-	-	+2.6
5	+0.1	+0.7	-	+0.2	+0.7	-	+0.9	+1.1	-	+1.6	+1.6	-	+2.4	+2.3	-
5.2	-	-	+0.4	-	-	+0.6	-	-	+1.1	-	-	+2.0	-	-	+2.2
6	+0.1	+0.8	-	+0.1	+0.8	-	+0.8	+1.2	-	+1.7	+2.1	-	+2.2	+2.1	-
6.2	-	-	0.0	-	-	-0.1	-	-	+1.0	-	-	+2.0	-	-	+2.1
7	+0.1	+0.7	0.0	+0.2	+0.7	+0.1	+0.9	+1.0	+1.0	+2.2	+2.1	+1.9	+2.0	+2.2	+2.4