THAMES FLOOD PREVENTION RESEARCH
A STUDY OF TECHNIQUES OF OPERATING THE WOOLWICH SURGE–DEFENCE BARRIER

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REPORT NO 17

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NATURAL ENVIRONMENT RESEARCH COUNCIL

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ABSTRACT

A surge-defence barrier is being built in Woolwich Reach in the Thames Estuary. When a surge is forecast the barrier will close for a short period, thus protecting the regions landwards of the barrier from flooding. Flood defences on the seaward side of the barrier will be raised to provide a similar standard of flood protection in this area.

Fully closing the Woolwich barrier late in the tidal cycle can produce large increases in downriver water levels and an investigation has been made of two techniques of partial closure, allowing some water to flow either under or over the main barrier gates. It has been shown that partial closure can reduce the adverse downriver effects of barrier closure while maintaining upriver water levels below the flood defences.

INTRODUCTION

The traditional method of flood defence in an estuary has been to build the river banks to a height greater than the maximum expected water level. In the past this method has defended London from flood waters in the River Thames. However, man-made changes in the estuary (Pinless & Bowen, 1973) and land movements relative to sea level (Rossiter, 1969) have meant that the Thames embankments have had to be periodically raised to contain the increasing water levels caused by storm surges entering the estuary from the North Sea. To continue raising the banks in such a way would create unacceptable environmental problems in that it would be necessary to completely change the character of many famous stretches of river front. An alternative method of flood defence is to construct a moveable surge-defence barrier and such a barrier is at present being built in Woolwich Reach. When a forecast indicates that the coming high water may overtop the embankments upriver of the barrier site, the barrier will be closed, thus reducing the amount of water entering the upper reaches of the estuary. Flood defences on the seawards side of the barrier are being improved to ensure a downriver standard of flood defence comparable to that of Central London.

Studies have been made of the effects of different techniques of barrier closure using a numerical model of the Thames Estuary. With the barrier site, the details of gate operation and the limiting design criteria finalised a series of full closure tests have been made to supplement earlier results (e.g. Bowen & Pinless, 1969). Two methods of partially closing the barrier have also been considered. The set of runs previously made with some water allowed to flow over the centre four barrier gates (Pinless, 1974) have been repeated with the final design criteria in mind and the effects of allowing some water to flow under the main gates have also been studied.

DEVELOPMENT OF THE WORKING EQUATIONS OF THE NUMERICAL MODEL

The computational method used in the model (Rossiter & Lennon 1965) is derived from the basic hydrodynamical equations of continuity and motion in one dimension.

$$\frac{\partial Q}{\partial x} = -\frac{b\,\partial Z}{\partial t} \qquad (1)$$

$$\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} = -g\frac{\partial Z}{\partial x} - u|u|\Psi(H) - \frac{gH}{2\rho}\frac{\partial \rho}{\partial x} \qquad (2)$$

Where t is time

x is the distance along the central channel of the estuary.

u is the depth mean current in the direction of x increasing.

Z is the elevation of the water surface above a level plane.

H is the mean depth of the channel (a function of Z and x.)

A is the cross-sectional area of the channel (a function of Z and x.)

b is the mean surface breadth of the channel (a function of Z and x.)

Q is the cubature = A x u.

$\Psi(H)$ is the bottom friction function.

ρ is the water density (a function of x only).

g is the acceleration due to gravity.

For the finite difference scheme of the model,

$t = \tau \times m$ where τ is the time interval between successive computations and the time origin occurs when m = 0

$x = \varepsilon \times n$ where ε is the distance between adjacent sections.

Z is referred to Ordnance Datun Newlyn and is evaluated at integral sections and at intervals of τ from m = 0. u is evaluated midway between sections and at intervals of τ from $m = \frac{1}{2}$.

For location n and time $m-\frac{1}{2}$, the equation of continuity (1) can be written as

$$Z_{m,n} = Z_{m-1,n} + \frac{\tau}{\varepsilon}\cdot\frac{1}{b_{m-\frac{1}{2},n}}\left(Q_{n-\frac{1}{2}} - Q_{n+\frac{1}{2}}\right)_{m-\frac{1}{2}} \qquad (3)$$

For location $n+\frac{1}{2}$ and time m, the equation of motion (2) can be written as

$$U_{m+\frac{1}{2},n+\frac{1}{2}}\left(1 + \tau|u_{m-\frac{1}{2},n+\frac{1}{2}}|\,\psi\,(H_{m,n+\frac{1}{2}}) - \frac{\tau}{4\varepsilon}(U_{n-\frac{1}{2}}-U_{n+3/2})_{m-\frac{1}{2}}\right) =$$

$$U_{m-\frac{1}{2},n+\frac{1}{2}} + \frac{g\tau}{\varepsilon}(Z_n-Z_{n+1})_m + \frac{\tau}{8\varepsilon}(U^2_{n-\frac{1}{2}}-U^2_{n+3/2})_{m-\frac{1}{2}} - \frac{g\tau}{2\varepsilon}\left(\frac{\partial\rho}{\rho}\right)_{n+\frac{1}{2}}H_{m,n+\frac{1}{2}} \qquad (4)$$

Equations (3) and (4) are used as the working equations for deriving the solutions of Z and u along the estuary as functions of time.

PROCEDURE USED IN COMPUTATIONS

To start a calculation sequence of the model it is necessary to define values of u (at time $m=-\frac{1}{2}$) and Z (at time m=0) along the river. Since observed current values are not normally available, it is usual to make an approximation to slack water conditions with u zero everywhere. A run-in period is then necessary to allow errors, introduced by the artificial initial conditions, to disappear.

The two boundary conditions required are a time history of the water elevation at the seawards boundary of the model and of the rate of freshwater flow at the head of the estuary.

From these initial and boundary conditions, with m=1, it is possible to calculate $u_{m-\frac{1}{2},n+\frac{1}{2}}$ for all n from equation (4). $Z_{m,n}$ can now be computed from equation (3) for all n. This process is repeated, incrementing m, until the total required time span has been completed.

STABILITY OF THE COMPUTATIONAL METHOD

In order to ensure stability in the numerical processes any error produced in the calculation of currents or water levels for one value of m must not be magnified during calculations for m+1.

Let the absolute error in $u_{m-\frac{1}{2},n+\frac{1}{2}}$ be e(u) and assume all previously calculated values of Z and all other values of u at time $m-\frac{1}{2}$ are accurate. Then, from equation (3),

$$e(Z_{n,m}) = -\frac{\tau}{\varepsilon} \frac{A_{m-\frac{1}{2},n+\frac{1}{2}}}{b_{m-\frac{1}{2},n}} e(u)$$

and

$$e(Z_{n+1,m}) = \frac{\tau}{\varepsilon} \frac{A_{m-\frac{1}{2},n+\frac{1}{2}}}{b_{m-\frac{1}{2},n+1}} e(u)$$

Therefore

$$e(Z_{n,m}-Z_{n+1,m}) = -\frac{\tau}{\varepsilon} A_{m-\frac{1}{2},n+\frac{1}{2}} e(u) \left(\frac{1}{b_{m-\frac{1}{2},n}} + \frac{1}{b_{m-\frac{1}{2},n+1}}\right) \qquad (5)$$

Neglecting the advective and density gradient terms, which are both small, and the frictional term, which will tend to be a stabilising factor anyway, from equation (4)

$$e(U_{m+\frac{1}{2},n+\frac{1}{2}}) = e(u) + \frac{g\tau}{\varepsilon} e(Z_{n,m} - Z_{n+1,m})$$

Using equation (5)

$$e(U_{m+\frac{1}{2},n+\frac{1}{2}}) = e(u) - \frac{g\tau^2}{\varepsilon^2} A_{m-\frac{1}{2},n+\frac{1}{2}} e(u) \left(\frac{1}{b_{m-\frac{1}{2},n}} + \frac{1}{b_{m-\frac{1}{2},n+1}}\right)$$

Now for computational stability,

$$\left|e(U_{m+\frac{1}{2},n+\frac{1}{2}})\right| \leq \left|e(u)\right|$$

i.e.

$$-1 \leq 1 - \frac{g\tau^2}{\varepsilon^2} A_{m-\frac{1}{2},n+\frac{1}{2}} \left(\frac{1}{b_{m-\frac{1}{2},n}} + \frac{1}{b_{m-\frac{1}{2},n+1}}\right) \leq +1$$

$\frac{g\tau^2}{\varepsilon^2} A_{m-\frac{1}{2},n+\frac{1}{2}} \left(\frac{1}{b_{m-\frac{1}{2},n}} + \frac{1}{b_{m-\frac{1}{2},n+1}}\right) > 0$ and therefore the right hand side of the above inequality is always satisfied.

Therefore, the necessary condition for computational stability is

$$\frac{g\tau^2}{\varepsilon^2} A_{m-\frac{1}{2},n+\frac{1}{2}} \left(\frac{1}{b_{m-\frac{1}{2},n}} + \frac{1}{b_{m-\frac{1}{2},n+1}}\right) \leq 2$$

Now

$$A_{m-\frac{1}{2},n+\frac{1}{2}} \left(\frac{1}{b_{m-\frac{1}{2},n}} + \frac{1}{b_{m-\frac{1}{2},n+1}}\right) \simeq \frac{2A_{m-\frac{1}{2},n+\frac{1}{2}}}{b_{m-\frac{1}{2},n+\frac{1}{2}}}$$

and so the left hand side of the above inequality will reach its maximum value when $^{A}/b$ is a maximum which will occur at maximum depth, Hmax.

Therefore, the condition for computational stability may be stated as

$$\tau \leq \frac{\varepsilon}{\sqrt{g H_{max}}}$$

NUMERICAL MODEL OF THE THAMES ESTUARY

Figure 1 shows a map of the Thames with the sites of the sections of the numerical model of the estuary. The input data required by the model are the freshwater flow at the landward end of the estuary, Teddington Weir, and a time history of the elevation of the water surface at the mouth of the estuary, near Harwich. Since no relevant observational data is available, the input data used for the seawards boundary of the model are computed from observed tidal levels at Southend (Pinless 1975).

The distance between adjacent sections of the model is 4.89 miles and, in order to ensure computational stability, a time step of five minutes is used. The frictional coefficient used is that appropriate to the Manning formula which gives $\psi(H)=KH^{4/3}$, where K is an empirical parameter which is determined separately for use at each half-section of the model. (Rossiter & Lennon 1965, and Pinless 1975).

THE WOOLWICH SURGE-DEFENCE BARRIER

The Woolwich barrier will consist of ten separate gates across the 1400 foot width of the river (Figure 2). Gates, A, H, J, and K are each 100 feet long with a sill level of O.D.N., gates B and G are each 100 feet long with a sill level of -15 feet O.D.N., and gates C, D, E and F are each 200 feet long with a sill level of -30 feet O.D.N. The barrier is to be built at a site 8.4 miles downriver from London Bridge.

The barrier gates are closed in the order K, J, H, A, G, B, F, C, E and D, the interval between the start of closure of one gate and the start of closure of the next gate being 0.5 minutes.

The leading edges of gates A, H, J and K fall at a rate of 3.608 feet/minute from an open level of 27 feet O.D.N.

The crests of gates B and G rise at a rate of 4.264 feet/minute to a fully closed level of 25.61 feet O.D.N.

The procedure for closing each of the main gates (C, D, E and F) is as follows;

(1) no gate movement for 0.25 minutes.

(2) gate rises at 6.396 feet/minute to 8.2 feet O.D.N.

(3) gate rises at 6.7568 feet/minute from 8.2 feet O.D.N. to a fully closed level of 25.61 feet O.D.N.

Thus, if barrier closure starts at time T, the time of commencement of movement for gate K is T, for gate J is T + 0.5 minutes and so on to T + 4.75 minutes for gate D and full closure of the barrier is achieved in 13.35 minutes.

During the partial closure mode of operation of the barrier, water is allowed to flow either over or under the four main gates (C,D, E and F) while the remaining six gates are fully closed.

For the overshooting mode of partial closure, the four main gates are raised (as detailed above for full closure) until they reach some predetermined level at which they are fixed. Design criteria for the barrier structure state that, at the time of fixing, a gate must be at least one foot above the water level and that at no time after fixing must there be a head of water greater than 6.56 feet above a barrier gate.

For the undershooting mode of partial closure, the barrier is first fully closed. Each of the four main gates is then raised to produce some predetermined 'gap width' between the gate and its sill by the following procedure;

(1) no gate movement for 10.2 minutes after the gate has reached its fully closed position.

(2) gate rises at 3.6408 feet/minute until the required gap width has been produced beneath it.

The design of the barrier gates is not intended to deal with free discharge conditions which give rise to structural problems. To safeguard against such conditions a gap width not exceeding five feet was stipulated although under certain conditions even this may be too large an opening.

The required standard of flood prevention for the regions upriver of the barrier site is defined as the maintenance of the maximum water level at Tower Pier (approximately section 9 of the numerical model) at or below + 14 feet O.D.N.

SCHEMATIZATION OF THE BARRIER IN THE NUMERICAL MODEL

The barrier was represented at section $7\frac{1}{2}$ of the model (Figure 1) and closure commenced when the surface elevation at Section 7 rose to some pre-determined 'closure level'. From the start of closure the time step between successive model calculations was reduced from the normal five minutes to one minute to ensure computational stability over a period when relatively violent changes in current and water surface elevation may occur. The shorter time step was maintained until the barrier was instantaneously re-opened on the ebb tide when the water level at section 7 dropped to that at section 8.

At each time step of the calculations the position of each barrier gate was determined and the flow past the barrier site was computed from the relevant formulae.

EQUATIONS USED TO CALCULATE THE RATE OF DISCHARGE OF WATER AT THE BARRIER SITE

(1) NARROWING SCHEMATIZATION

For the first nine minutes of the closure a 'narrowing' schematization was used to calculate the rate of discharge of water at section $7\frac{1}{2}$ of the model. This schematization regards the effect of the closing barrier as a head loss given by

$$\frac{\gamma}{2g} Q^2 \left(\frac{1}{A_b^2} - \frac{1}{A_o^2} \right) \quad \text{(Dronkers 1964)}$$

where γ is a constant coefficient dependant on the geometry of the structure.

subscript o refers to the unrestricted channel value.

subscript b refers to the restricted channel value at the relevant time step of closure.

This formulation was felt to give the best representation of the flow through the decreasing cross-sectional area of the channel at the barrier site while the main barrier gates had not risen any great distance from the river bed.

In order to represent the head loss at section $7\frac{1}{2}$ of the numerical model, equation (3) was amended for n=7 and n=8 and an additional term was included in equation (4) when $n=7\frac{1}{2}$. At all other locations the finite difference equations were not affected by the introduction of the barrier.

It was necessary to differentiate between the conditions of the flood and the ebb tide due to the assymetry of the water elevations on either side of the barrier. The flood tide was defined as $U_{7\frac{1}{2}} \geqslant 0$ and the ebb tide as $U_{7\frac{1}{2}} < 0$. During the studies detailed here only flood tide barrier closures were considered and only the flood tide schematization was therefore used.

The calculations were made as follows;

(a) Flood Tide

The equation of motion at section $7\frac{1}{2}$ and time m is given by

$$U_{m+\frac{1}{2},7\frac{1}{2}} \left\{ 1 + \tau \left| U_{m-\frac{1}{2},7\frac{1}{2}} \right| \left\{ \psi(H_{m,7\frac{1}{2}}) + \frac{\gamma}{2\varepsilon} \left\{ \left(\frac{A_o(Z_{m,8})}{A_b(Z_{m,7})} \right)^2 - 1 \right\} \right\} - \frac{\tau}{4\varepsilon} \left(U_{6\frac{1}{2}} - U_{8\frac{1}{2}} \right)_{m-\frac{1}{2}} \right\} =$$

$$U_{m-\frac{1}{2},7\frac{1}{2}} + \frac{g\tau}{\varepsilon} (Z_7 - Z_8)_m + \frac{\tau}{8\varepsilon} \left(U^2_{6\frac{1}{2}} - U^2_{8\frac{1}{2}} \right)_{m-\frac{1}{2}} - \frac{g\tau}{2\varepsilon} \left(\frac{\partial\rho}{\rho} \right)_{7\frac{1}{2}} H_{m,7\frac{1}{2}}$$

Where $H_{m,7\frac{1}{2}}$ is calculated over the unrestricted channel cross-sectional area A_o. This gives $(U_{m+\frac{1}{2},7\frac{1}{2}})_o$ and using the condition of continuity of flow near the barrier

$$(U_{m+\frac{1}{2},7\frac{1}{2}})_b \quad (U_{m-\frac{1}{2},7\frac{1}{2}})_b =$$

$$= \left(\frac{A_o(Z_{m,8})}{A_b(Z_{m,7})} \right)^2 (U_{m+\frac{1}{2},7\frac{1}{2}})_o \quad (U_{m-\frac{1}{2},7\frac{1}{2}})_o$$

i.e. $$(U_{m+\frac{1}{2},7\frac{1}{2}}) \simeq \frac{A_o(Z_{m,8})}{A_b(Z_{m,7})} (U_{m+\frac{1}{2},7\frac{1}{2}})_o$$

At section 7 and time $m+\frac{1}{2}$, equation (3) is used to compute $Z_{m+1,7}$.

$Q_{m+\frac{1}{2},6\frac{1}{2}}$ has already been calculated in the usual way.

$$Q_{m+\frac{1}{2},7\frac{1}{2}} = (Au)_{m+\frac{1}{2},7\frac{1}{2}} = (A_{m+\frac{1}{2},7\frac{1}{2}})_b \quad (U_{m+\frac{1}{2},7\frac{1}{2}})_b$$

where $(U_{m+\frac{1}{2},7\frac{1}{2}})_b$ has been calculated as shown above and

$(A_{m+\frac{1}{2},7\frac{1}{2}})_b$ is a function of $Z_{m-1,7}$.

The required value of $b_{m+\frac{1}{2},7}$ is also taken as a function of $Z_{m-1,7}$.

At section 8 and time $m+\frac{1}{2}$, equation (3) is used to compute $Z_{m+1,8}$.

$Q_{m+\frac{1}{2},7\frac{1}{2}}$ has already been calculated and $Q_{m+\frac{1}{2},8\frac{1}{2}}$ can be calculated in the usual way. The value of $b_{m+\frac{1}{2},8}$ is taken as a function of $Z_{m-1,8}$.

(b) Ebb Tide

The equation of motion at section $7\frac{1}{2}$ and time m is given by

$$U_{m+\frac{1}{2},7\frac{1}{2}} \left\{ 1 + \tau |U_{m-\frac{1}{2},7\frac{1}{2}}| \left\{ \psi(H_{m,7\frac{1}{2}}) + \frac{\gamma}{2\varepsilon} \left(\left(\frac{A_o(Z_{m,7})}{A_b(Z_{m,8})} \right)^2 - 1 \right) \right\} - \frac{\tau}{4\varepsilon} \left(U^2_{6\frac{1}{2}} - U^2_{8\frac{1}{2}} \right)_{m-\frac{1}{2}} \right\} =$$

$$U_{m-\frac{1}{2},7\frac{1}{2}} + \frac{g\tau}{\varepsilon} (Z_7 - Z_8)_m + \frac{\tau}{8\varepsilon} \left(U^2_{6\frac{1}{2}} - U^2_{8\frac{1}{2}} \right)_{m-\frac{1}{2}} - \frac{g\tau}{2\varepsilon} \left(\frac{\partial\rho}{\rho} \right)_{7\frac{1}{2}} H_{m,7\frac{1}{2}}$$

where $H_{m,7\frac{1}{2}}$ is calculated over the unrestricted channel cross-sectional area A_o.

At section 7 and time $m+\frac{1}{2}$, equation (3) is used to calculate $Z_{m+1,7}$. As in the flood tide case, $Q_{m+\frac{1}{2},6\frac{1}{2}}$ is already known and $b_{m+\frac{1}{2},7}$ is taken as a function of $Z_{m-1,7}$. However $Q_{m+\frac{1}{2},7\frac{1}{2}}$ is now taken as $(A_{m+\frac{1}{2},7\frac{1}{2}})_o(U_{m+\frac{1}{2},7\frac{1}{2}})_o$ where $(A_{m+\frac{1}{2},7\frac{1}{2}})_o$ is a function of $Z_{m,7}$. $Z_{8,m}$ is calculated in the same way as for the flood tide case.

A value of 1.2 was taken for γ. This figure falls within the generally accepted range (Dronkers, 1967) and gave a reasonably smooth transition from the narrowing schematization to the weir formulae used in the later stages of closure.

(2) WEIR FORMULAE

By 9.5 minutes after barrier closure commenced the small side gates (A, H, J and K) are fully closed and the other six gates of the barrier have risen a sufficient distance above the river bed to merit the use of weir formulae to compute the rate of discharge of water past the barrier site.

For unsubmerged flow and diving jet conditions, $Q=4.0LH_1^{3/2}$

For surface jet conditions, $Q=1.1C_dLH_1(H_1-H_2)^{\frac{1}{2}}$

where Q = rate of discharge of water in cusecs

L = length of gate in feet

H_1 = head over gate on upstream side (seawards for flood prevention closure).

H_2 = head over gate on downstream side (landwards for flood prevention closure)

C_d = $3.3+0.11H_2$.

Tests were carried out by the British Hydraulic Research Association to determine the criteria for the change in flow type and some of their results are shown in Figure 3. These results were interpolated, as necessary, to produce Table 1 which was then used in a test procedure at each time step of the model calculations to determine the flow condition over each of the barrier gates. The rate of discharge of water over each gate was then computed using the relevant formula and the flows summed to produce the total rate of flow at section $7\frac{1}{2}$ of the model.

These weir formulae were used during barrier closure and for computing the rate of flow over the barrier gates during the overshooting mode of partial closure.

(3) UNDERSHOOTING FLOW

For the undershooting mode of partial closure, the following formula was used to calculate the rate of flow of water under each of the partially closed barrier gates.

$$Q = KC_d \; L \; b \; (2g \, \Delta H)^{\frac{1}{2}}$$

where Q = rate of discharge of water in cusecs.

K = constant coefficient of 1.1 (model-to-prototype) factor from tests carried out at Imperial College).

L = length of gate in feet.

ΔH = head difference across gate in feet.

b = width of gap under gate in feet.

g = acceleration due to gravity.

C_d = 0.76 when $b \leq 3$ feet and $b > 8$ feet.

C_d = 0.75 when 3 feet $< b \leq 4$ feet and 6 feet $< b \leq 8$ feet.

C_d = 0.74 when 4 feet $< b \leq 6$ feet.

It was essential to ensure that free discharge never took place during undershooting flow as flume tests had shown that such a flow type could create forces too great for the barrier structure to withstand. At each time step of the model calculations a test was made on the upriver and downriver water levels and if free discharge were found to occur the model run was aborted. In this way it was possible to find what range of gap widths under the gates would prove acceptable under any given conditions. Table 2 was used in this test which consisted of ensuring that the upriver water level was not less than the relevant tabulated value.

The flow of water under each of the partially closed barrier gates was calculated at each model time step and the flows summed to produce the total rate of discharge of water at section $7\frac{1}{2}$ of the model.

(4) LEAKAGE FLOW

Since the rising sector gates (B, C, D, E, F, and G) cannot be expected to provide a water-tight fit with the river bed, a leakage flow was allowed for under each of them. From the time during barrier closure when the transfer from the 'narrowing' to the 'weir formula' schematization occured in the model until the barrier was re-opened, the total leakage flow was added to any other flow past the barrier site to obtain the total rate of discharge of water at section $7\frac{1}{2}$ of the model. During the undershooting mode of partial closure it was only necessary to compute a leakage flow under gates B and G.

The leakage flow was calculated as $Q = C_d C_L bL (2g\Delta H)^{\frac{1}{2}}$

where Q = rate of discharge of water in

C_d = constant coefficient of 0.67.

C_L = 1.20 for a closure level <-4 feet O.D.N.

C_L = 1.15 for -4 feet O.D.N. $\leq$ closure level $\leq +4$ feet O.D.N.

C_L = 1.10 for a closure level $> +4$ feet O.D.N.

b = 0.75 feet (the gap width assumed under each gate)

L = length of gate in feet.

ΔH = head difference across gate in feet.

g = acceleration due to gravity.

INPUT DATA USED FOR THE MODEL RUNS

Three input surges have been used for these barrier studies; the 12 foot, 15 foot and 18.4 foot surges. Each surge is named according to its open river high water level at Southend.

Firstly, a time history of the water level at Southend was defined for each surge. The 15 foot surge is the one observed in January 1953 and is the largest surge so far recorded in the Thames Estuary, reaching a high water level of approximately 15 feet O.D.N., at Southend. The 12 foot surge was obtained by subtracting three feet from the observed water levels throughout the tidal cycle. Although such a surge would not represent a flood threat to London the barrier might well be closed against it as the predicted surge is often greater than that which actually occurs. The 18.4 foot surge was obtained by adding 0.8 feet to the observed water levels throughout the tidal cycle and by also adding the positive part of a sine curve of amplitude 2.5 feet which was zero at the low water preceding the observed surge and reached a maximum at the observed surge's high water. This surge reaches a maximum water level at

Southend which is calculated to have a return period of one thousand years in 2030A.D. and is the surge against which the land bordering the Thames Estuary is to be defended.

For each of the three surges, the input at the seawards limit of the model, near Harwich, was calculated from the Southend conditions so that, when the river is unobstructed, the correct water levels were reproduced at Southend (Pinless, 1975).

Model runs were made with upland flows of both 2,500 and 20,000 cusecs for each input surge. 2,500 cusecs represents an average upland flow into the Thames Estuary while 20,000 cusecs is a very high flow and was used to test the techniques of barrier operation under extreme upland flow conditions.

FULL BARRIER CLOSURE

The simplest technique of barrier closure would be to fully close all the gates when a surge is forecast, thus protecting the regions landwards of the barrier site from flooding. The maximum water levels reached at each section of the model following full barrier closure are shown in Tables 3, 4 and 5 for each combination of surge, upland flow and closure level under consideration. It can be seen that in all cases, except that of the highest closure level with a large upland flow, the conditions landwards of the barrier are within the defined defence level of + 14 feet O.D.N. at section 9. The exceptional case for each surge is one where an earlier barrier closure would have to be made to ensure safe water levels in the upriver regions. In general, however, with full barrier closure maintaining upriver water levels well below defence levels it is the effects of closure seawards of the barrier which are of particular interest. These effects are more clearly seen by considering the difference in maximum water levels with and without a barrier (Figure 4). The halting of the flood tide by closing a barrier across the estuary produces increases in high water levels seawards of the barrier which vary considerably according to the parameters involved.

The results for the 12 foot surge show an increase in maximum water levels of between a half and one foot following the earliest barrier closure (closure level of O.D.N.) At the sections furthest seawards of the barrier, the increases in high water levels are progressively smaller as the closure time is taken later and there is little or no change at these sections for the closure made latest in the tidal cycle (closure level of + 12 feet O.D.N.). However, the increase in maximum water level near the barrier is as much as two feet for the closure commenced at +12 feet O.D.N. These results are most clearly understood by looking at time histories of the change in water level at particular sites along the estuary following barrier closure (Figure 5). The disturbance shown can be considered as a reflected wave propagating downriver and rapidly decreasing in amplitude due to the widening of the estuary and frictional dissipation. The timing of the reflected wave relative to local high water is all important in determining the high water level reached at any particular site. With closure commencing when the water level at section 7 reached O.D.N., the reflected wave has reached all sections of the model by each time of local high water and so some change in maximum water level is recorded along the entire estuary. However, for this early closure, the maximum of the reflected wave has already passed by the time of high water at section 7 and the change in high water level there is not large. Where the time of maximum reflected wave does approximately coincide with local high water, at section 1, the amplitude of the wave has decreased sufficiently to not cause any great change in water level. For the latest closure studied the situation is reversed, with the maximum of the reflected wave approximately coinciding with the open river high water at section 7 and thus producing a large increase in maximum water level there. The water levels at the lower sections of the estuary are now not affected by the closure until after high water and so show no change in the maximum water level reached. The greatest detrimental effects of barrier closure will, therefore, be produced near the barrier site by a closure commenced late in the tidal cycle.

Figure 4 shows the same general effects for the two larger surges as have been detailed for the 12 foot surge. The effects of late closure are not shown as clearly with the larger surges since closure commenced when a pre-determined water level was reached which will occur earlier in the tidal cycle of a larger surge. Figure 6 shows the reflected wave at section

7 for the four closure levels under consideration with the 12 foot and 18.4 foot surges. The amplitude of the reflected wave is larger with the larger surge and can be shown to approximate to $u\sqrt{g/h}$ near the barrier (where u is the open river current at section $7\frac{1}{2}$, h the depth of water and g the acceleration due to gravity), as suggested by (Abbot 1959). However, the relative timing of the reflected wave and the tide is clearly all-important in determining the high water levels which must be contained by flood defences. This timing is determined by the rate of propagation of the wave and its interaction with the incoming surge which are very complicated in the real estuary.

The complexity of the interaction between the open river tidal wave and the reflected wave due to barrier closure is demonstrated when a higher upland flow is considered (Figure 7). The amplitude of the reflected wave is smaller with the higher upland flow due to the open river flood tide current at the barrier site being smaller. In this case, a consideration of the reflected waves would lead one to expect an increase in maximum water level with the higher upland flow of approximately 0.3 feet less than with the lower upland flow since the reflected waves differ by this amount at the times of the recorded high water following barrier closure. However, this is not what occurs as the reflected wave must be considered along with the open river tidal curve. This is shown in the lower diagram and it can be seen that the shape of the open river curve will determine the change in maximum water level as much as the size of the reflected wave.

It can be seen that the disadvantage of barrier closure lies in the downriver increases in water level and to reduce these detrimental effects to a minimum the barrier should be closed as early as possible in the tidal cycle. Such an early closure, with the estuary closed to shipping for a long period, would not be welcomed by port authorities. But the maximum water levels reached upriver of the barrier (Tables 3, 4 and 5) are, in most cases, well within the acceptable levels and some water could be allowed to flow past the barrier site after closure without creating dangerous conditions landward of the barrier.

PARTIAL CLOSURE - OVERSHOOTING MODE OF BARRIER OPERATIONS

A previous investigation has been made of the possibility of partially closing the main gates of the Woolwich surge-defence barrier and allowing some water to flow over them (Pinless, 1974) However, since this work was done the design criteria for the barrier and the details of gate operation have been finalised and the necessity for the allowance of a leakage flow under the model barrier has been recognized. It was felt that the previous work should be repeated under the new conditions and the details of the new studies are given here.

Figure 8 shows the time histories of the water levels at sections 7 and 8 of the model and the rate of discharge of water past the barrier site during a partial closure. Results are shown from the commencement of closure until the barrier re-opened on the ebb tide. As the flow at section $7\frac{1}{2}$ of the model is reduced by the closing of the barrier gates the water level at section 8 starts to fall. Until the water level at section 7 rises above the fixed gate (or weir) level the barrier is effectively fully closed and the rate of discharge of water shown is simply due to the leakage flow under the barrier gates. As the water level at section 7 rises on the flood tide the gates are eventually overtopped. The flow of water over the barrier increases so the water level at section 7 rises on the incoming tide at a rate faster than that of the rise in water level at section 8. On the ebb tide, the rate of flow over water over the barrier gates starts to fall as the head difference across the barrier decreases and the water level at section 8 only rises slowly. The water level at section 7 eventually drops below the level of the barrier gates and the barrier is again effectively fully closed for the final stage of barrier operation.

Figure 9 shows the results of two partial closures which are identical except for the weir levels at which the barrier gates are fixed. With the lower weir level the gates are over-topped earlier in the tidal cycle and remain so until later on the ebb tide. Thus, the total amount of water flowing into the upper reaches of the estuary is greater for the lower weir level and the maximum water levels reached up river of the barrier are larger than with the higher weir level. However, the flow of water over the barrier reduces the reflected wave effects of barrier closure with the results tending to those of the open river as the weir level is reduced while the higher weir levels tend to the fully closed situation. With the lower weir level, therefore, the downriver effects of barrier closure are smaller than with the higher weir level. It follows that the optimum weir level, producing least disturbance seawards of the barrier, will be the lowest one which will still maintain up-river water levels at acceptable values.

Under any given set of conditions, the optimum weir level at which the gates should be fixed is the lowest one which will also result in water levels obeying the following three criteria.

(1) At the time of fixing each gate, the level of the gate must be at least one foot above the water level at section 7

(2) At no time must the water level at section 7 rise more than 6.56 feet above the fixed gate level.

(3) The maximum water level at section 9 must not rise above + 14 feet O.D.N.

The first two criteria follow from the design limits of the barrier structure while the third ensures that the upriver water levels are kept within the flood defences.

The optimum weir levels for the combinations of surge level, upland flow and closure level under consideration are shown in Tables 6, 7 and 8. Condition (1) was found to limit the weir level to a value at least four or five feet above the closure level, thus allowing for the rise in water level at Section 7 during the closure procedure. Condition (2) limited the weir level to a value not less than 6.56 feet lower than the maximum water level reached at Section 7. Having obeyed conditions (1) and (2) it was not found necessary to consider conditions (3) except in the exceptional cases already mentioned under full barrier closure where even full closure would not provide adequate flood protection. It should be noted that the design limits of the barrier structure constrain the optimum weir levels to values which are considerably higher, in many cases, than is necessary to protect the upper reaches of the estuary from flooding. It would, therefore, seem that over-shooting partial closure may not prove a very efficient way of reducing the downriver effects of barrier closure.

PARTIAL CLOSURE - UNDERSHOOTING MODE OF OPERATION

Figure 10 shows the time histories of water levels at sections 7 and 8 of the model and the rate of discharge of water under the barrier gates for an undershooting partial closure. Results are shown from the start of closure until the barrier re-opens on the ebb tide. As the barrier gates close the flow of water past the barrier site is reduced and the water level at section 8 starts to fall. The barrier remains fully closed for some ten minutes until it is possible to start raising the main gates to their required position and during this period there is only a small leakage flow under the gates. When the main gates start to rise from the river bed the rate of discharge of water past the barrier site immediately greatly increases and the water level at section 8 starts to rise. The flow under the gates remains large until the head of water across the barrier starts to decrease as the water level at section 7 drops in the ebb tide. With the lessening rate of flow of water under the barrier the rate of rise of water level at section 8 is reduced. The barrier is re-opened when the water level at section 7 drops to that at section 8.

Figure 11 shows the results of two undershooting partial closures which are identical except for the gap widths set below the barrier gates. The greater flow of water through the larger gap results in higher upriver water levels. However, the more water allowed past the barrier site the smaller the downriver effects of barrier closure and it can be seen that the larger gap width results in a smaller reflected wave and so less increase in downriver water levels. The optimum gap width will be, therefore, the largest one which will still maintain upriver maximum water levels below the flood defences.

The optimum gap width beneath the gates, under any given set of conditions, will be the largest one which will also result in conditions obeying the following three criteria.

(1) Free discharge must not occur.

(2) The gap width should not exceed five feet.

(3) The maximum water level at section 9 must not rise above + 14 feet O.D.N.

The first two criteria are design limits on the barrier structure while the third ensures that upriver water levels are maintained within the flood defences.

Tables 9, 10 and 11 give the optimum gap width for each combination of surge level, upland flow and closure level being studied. No trouble was encountered with the free discharge criterion as this type of flow was not found to occur under any of the sets of conditions studied. It was possible to find a suitable gap width for all the input conditions under consideration except for the latest closure with a high upland flow for each surge. As has been previously explained, even full barrier closure would not provide adequate protection from flooding in these three cases.

For the 12 foot surge and the low upland flow with the 15 foot surge, criterion (2) had to be applied and the gap width given is smaller than necessary for simple flood protection. For the rest of the cases studied, the size of the optimum gap width is determined by criterion (3). It would seem that the barrier design criteria would have less influence on possible flood defence measures in the case of undershooting partial closure than they were shown to have on gate operation for overshooting partial closure.

COMPARISON OF THE EFFECTS OF THREE TECHNIQUES OF BARRIER CLOSURE

Figure 12 shows the changes in maximum water levels seawards of the barriers following the three different techniques of barrier closure which have been studied. In all cases the increase in high water level is less following a partial barrier closure than following a full closure. The differences between the effects of the two types of partial closure are not so clear, however.

For the 12 foot surge, an undershooting type of partial closure is clearly better for the two later closures while it is marginally better for the lower closure levels. Reference to Table 6 shows that the design criteria have limited the optimum weir levels for overshooting partial closure to values higher than necessary for flood prevention while limiting the size of the undershooting gap width to not more than five feet has a less severe effect (Table 9). In Figure 13 it can be seen that for the latest closure on the 12 foot surge, the optimum overshooting partial closure is little different from a full closure in its reflected wave effect as the barrier gates are only overtopped for a very short period. However, allowing a flow of water under the barrier gates throughout the closure period results in a consistently smaller increase in downriver water levels.

The results for the 15 foot surge follow the same pattern as for the 12 foot surge although in this case the design criteria of the barrier structure have a less severe effect on the early overshooting partial closures and there is little difference between the downriver effects of the two closure techniques in these cases.

For the 18.4 foot surge the effects of overshooting partial closure are marginally less detrimental to downriver water levels than those of the undershooting partial closure. Figure 13 shows the reason for this. An undershooting flow will continue throughout the period of barrier closure and this steady flow will significantly increase water levels upriver of the barrier so that the optimum gap width will be determined by flood defence levels. The overshooting flow, however, shows a sharp peak at the time of high water at the barrier site and will tend to reduce the downriver effects at the time of maximum water level. This reduction in the reflected wave effect of barrier closure is recorded at a critical stage of the tidal cycle. In effect the top of the tide is chopped off by the greatly increased flow over the barrier gates around the time of high water.

CONCLUSIONS

Full barrier closure is, in general, an effective means of flood prevention. In the case of a very high upland flow, however, it may prove necessary to close the barrier relatively early in the tidal cycle to ensure upriver water levels are maintained below the flood defences. The disadvantage of full closure is the increase caused in downriver maximum water levels which may be as much as two feet for a closure made late in the tidal cycle. An early closure results in less detrimental effects on water levels seawards of the barrier but would involve a long period when the estuary was closed.

Partial closure, allowing some water to flow either over or under the barrier gates, can successfully act as a flood defence measure while having a less severe effect than full closure on downriver water levels. The choice between the two methods of partial closure is not clear cut. In general, the undershooting method of partial closure has a less detrimental effect on downriver water levels for small surges. With a large surge, however, the overshooting technique of closure becomes slightly advantageous and there is little to choose between the two types of partial closure in such a case. The best results from an overshooting partial closure are produced by weir levels which are overtopped for a very short time by a considerable head of water. To reproduce this situation in reality would require a greater knowledge of the incoming surge's size and shape than would be available and this method of flood prevention would not seem a reliable one. However, the undershooting technique of closure shows consistently good results and would seem to be the most suitable form of partial closure.

ACKNOWLEDGEMENTS

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The 'narrowing' schematization was developed for use in the numerical model of the Thames Estuary by A. J. Bowen.

The weir formulae were investigated in studies at the British Hydraulics Research Association and the parameters and criteria for change of flow type were determined during this work. Undershooting flow determination tests were carried out at Imperial College. Both these studies were commissioned by the Greater London Council Thames Flood Prevention Project.

The G. L. C. is committed to a +15 feet O.D.N. defence level as London Bridge and the section 9 level requirement of +14 feet O.D.N. is regarded as a model-to-prototype safety factor.

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FIGURE 1.

THE RIVER THAMES
SHOWING THE SECTIONS OF THE NUMERICAL MODEL
AT WHICH THE SURFACE ELEVATIONS ARE DETERMINED

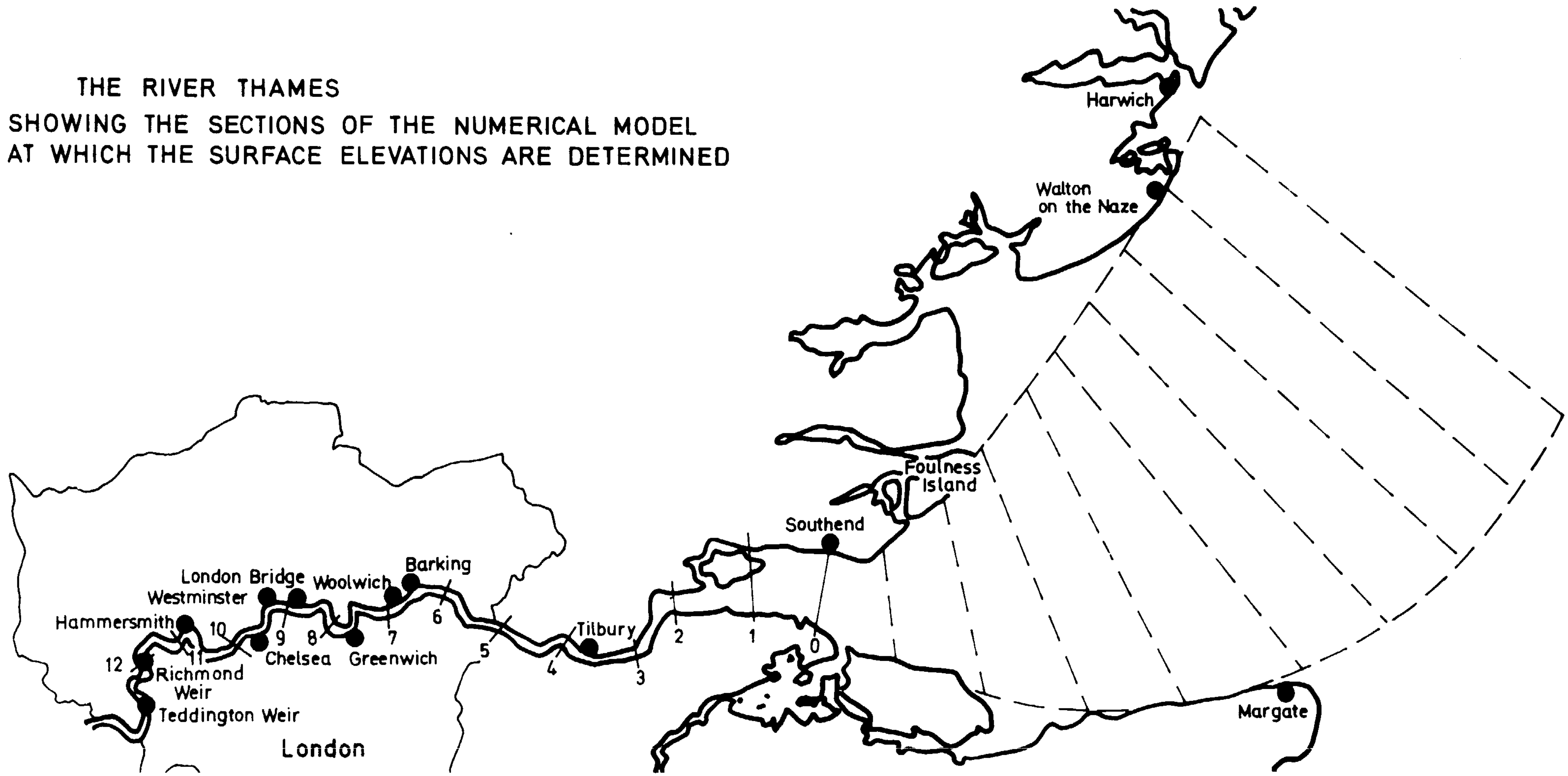


FIGURE 2

WOOLWICH SURGE-DEFENCE BARRIER

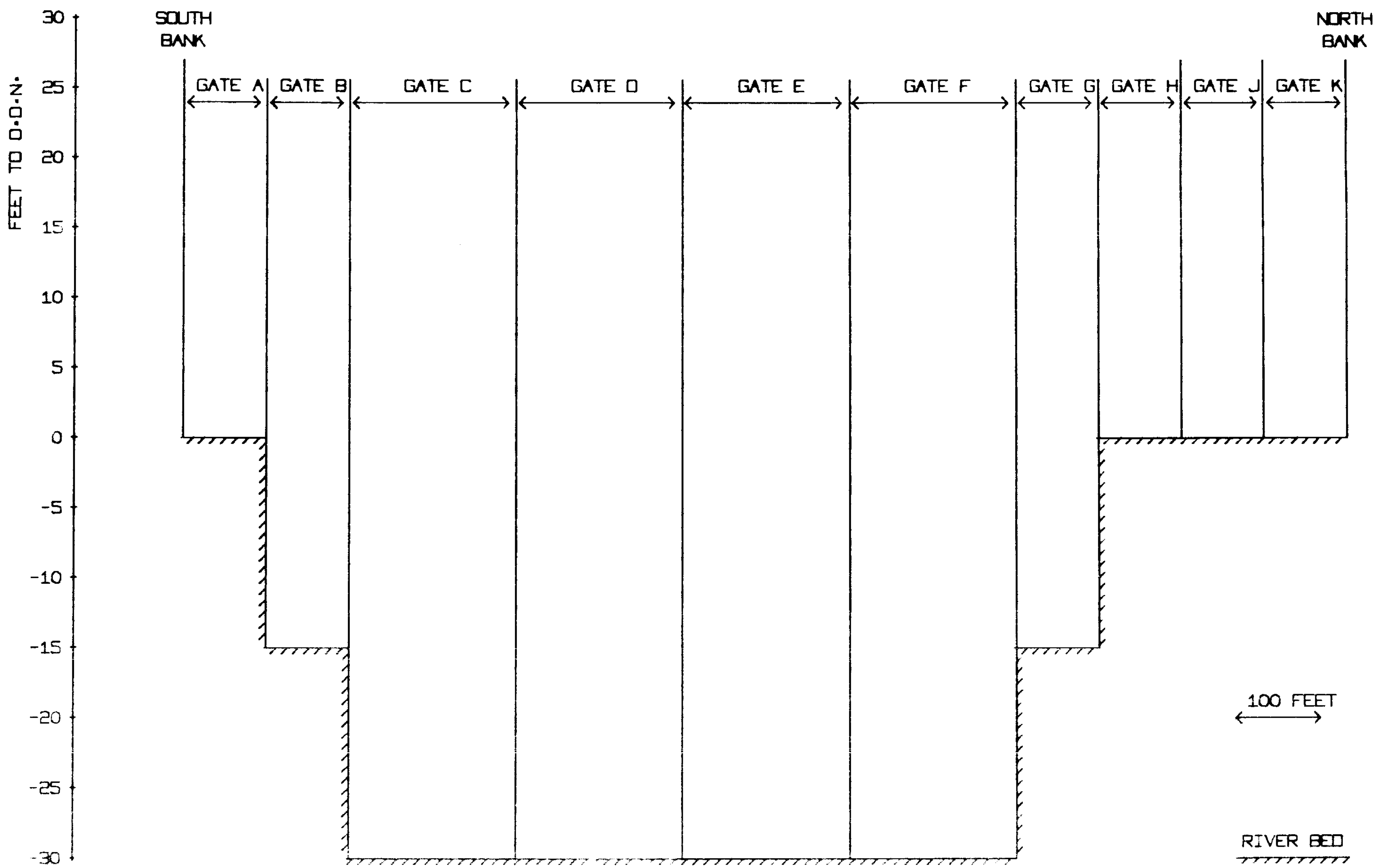


FIGURE 3.

CRITERION FOR CHANGE OF FLOW TYPE

H_1 = HEAD OVER WEIR ON DOWNSTREAM SIDE

H_2 = HEAD OVER WEIR ON UPSTREAM SIDE

(dotted line) SURFACE JET → DIVING JET

(solid line) DIVING JET → SURFACE JET

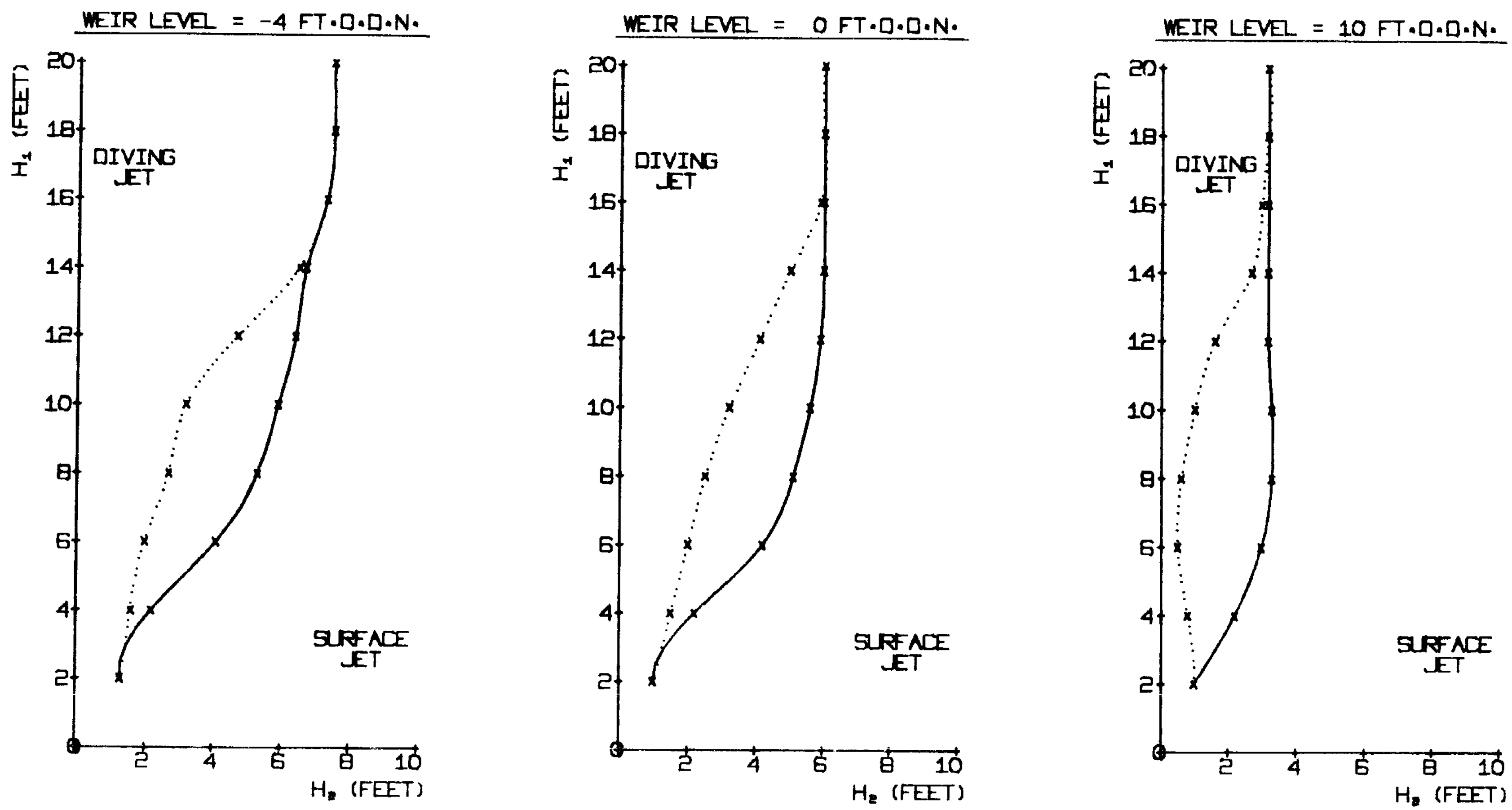


FIGURE 4

CHANGE IN MAXIMUM WATER LEVELS FOLLOWING FULL BARRIER CLOSURE.

UPLAND FLOW = 2,500 cu secs

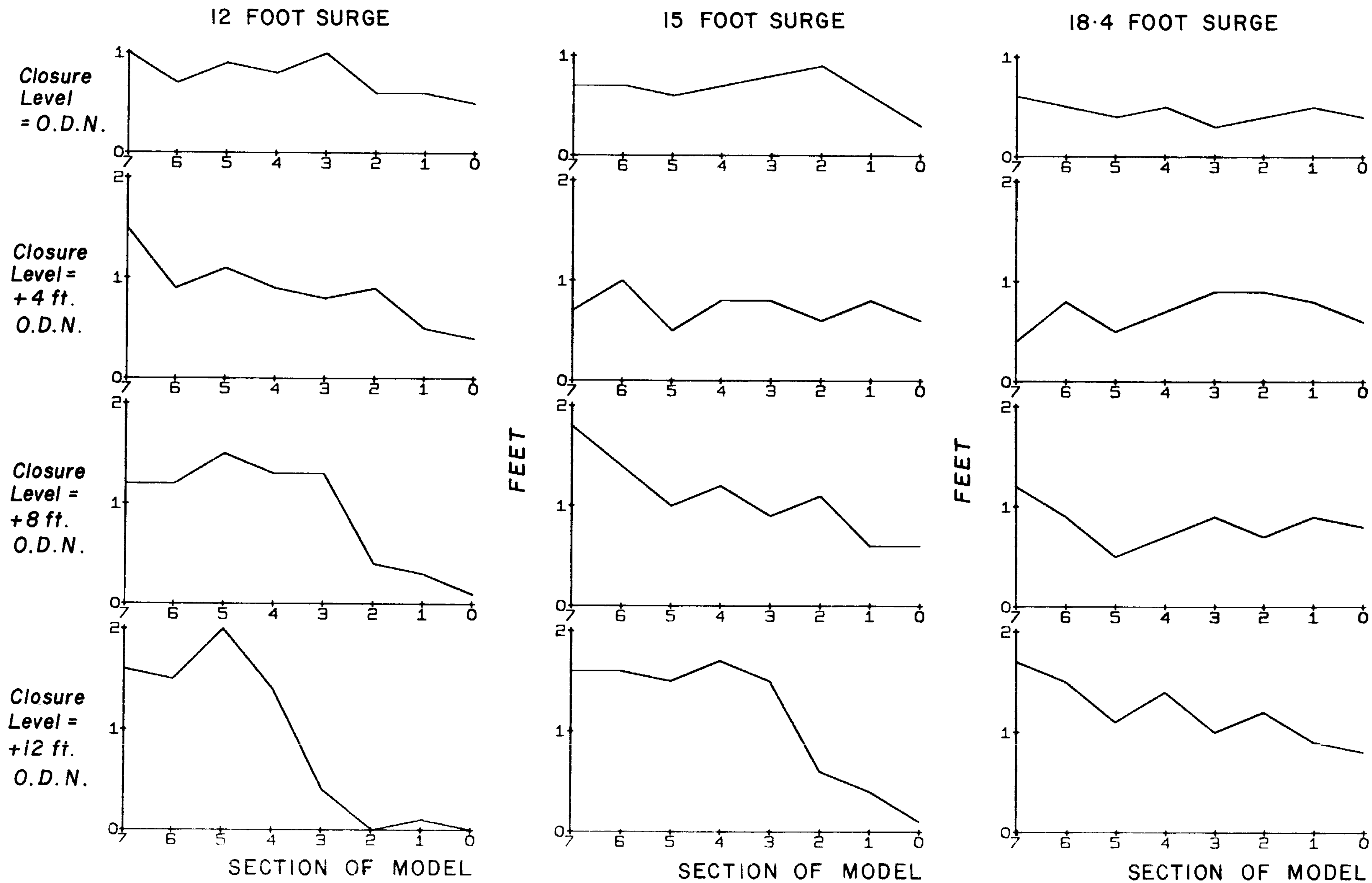
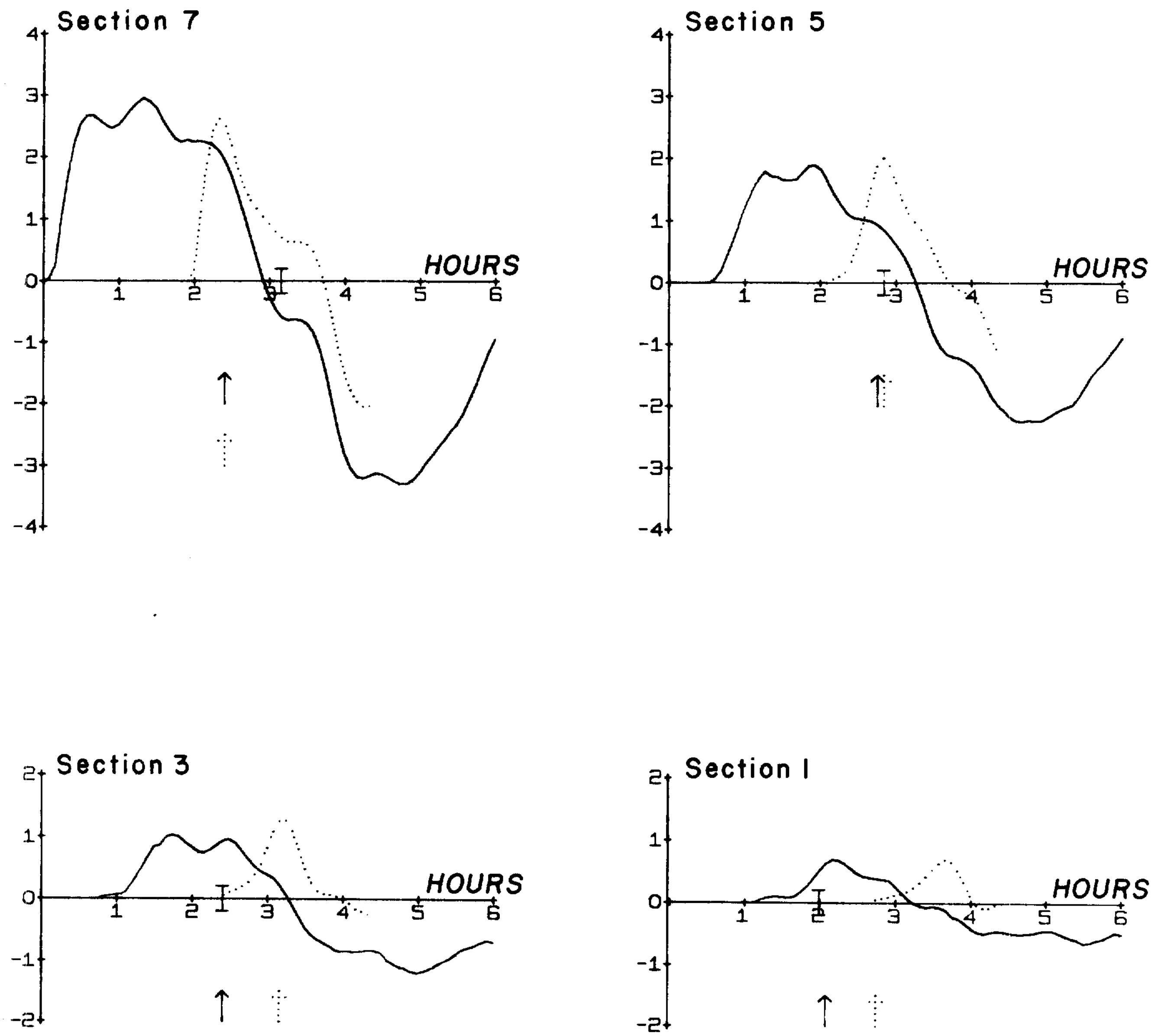


FIGURE 5

CHANGES IN WATER LEVEL DUE TO FULL BARRIER CLOSURE.
12 FOOT SURGE WITH UPLAND FLOW = 2,500 cu secs.

CLOSURE LEVEL OF ——— O.D.N.

············ 12 feet O.D.N.

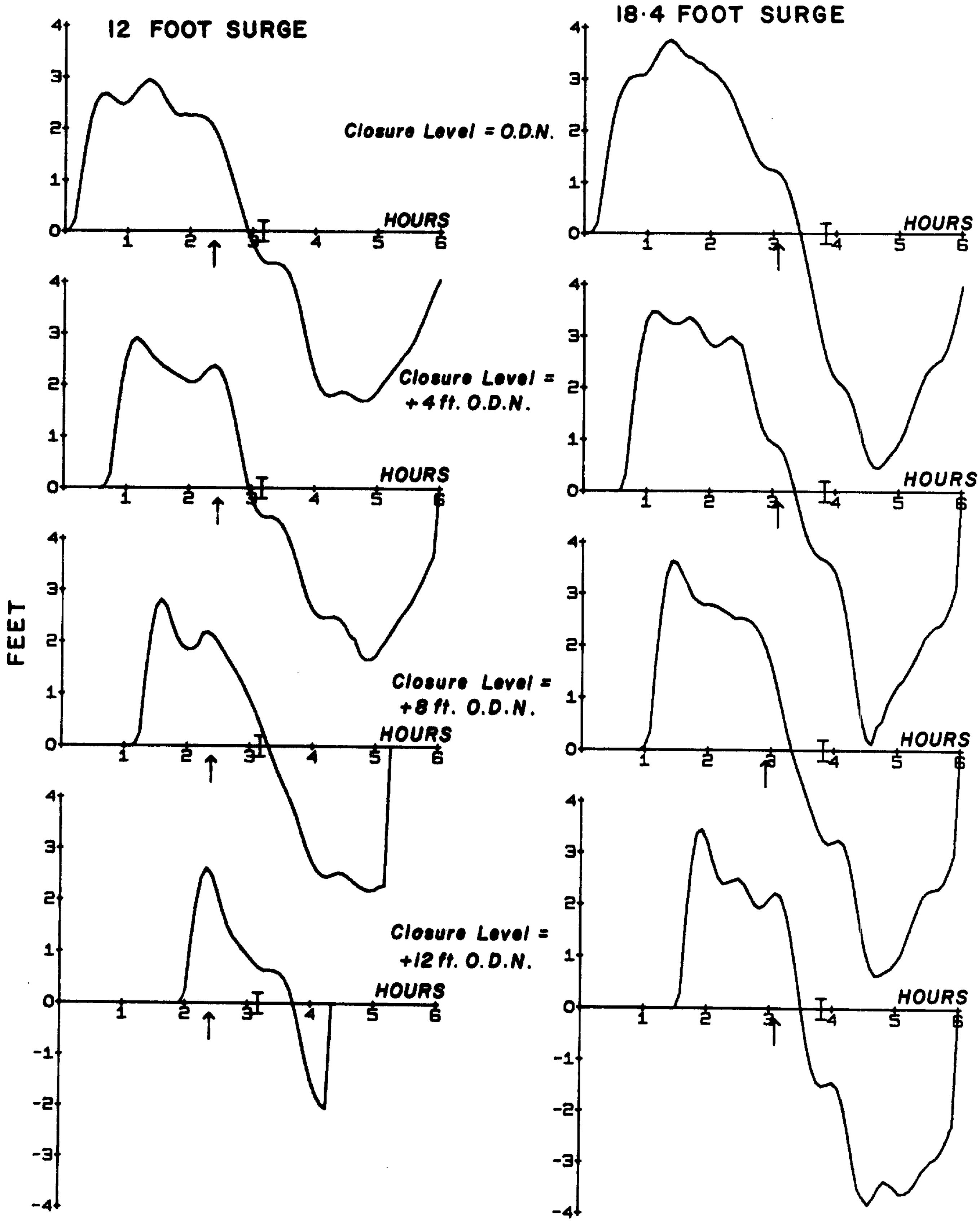


I Time of high water in open river.

⇡ ↑ Time of high water following barrier closure.

FIGURE 6

CHANGES IN WATER LEVEL AT SECTION 7 DUE TO FULL BARRIER CLOSURE.
UPLAND FLOW = 2,500 cu secs.



I Time of high water in open river.

↑ Time of high water following barrier closure.

FIGURE 7

FULL CLOSURE OF THE BARRIER ON THE 12 FOOT SURGE.

CLOSURE LEVEL = O.D.N.

—— UPLAND FLOW = 2,500 cu secs.

······ UPLAND FLOW = 20,000 cu secs.

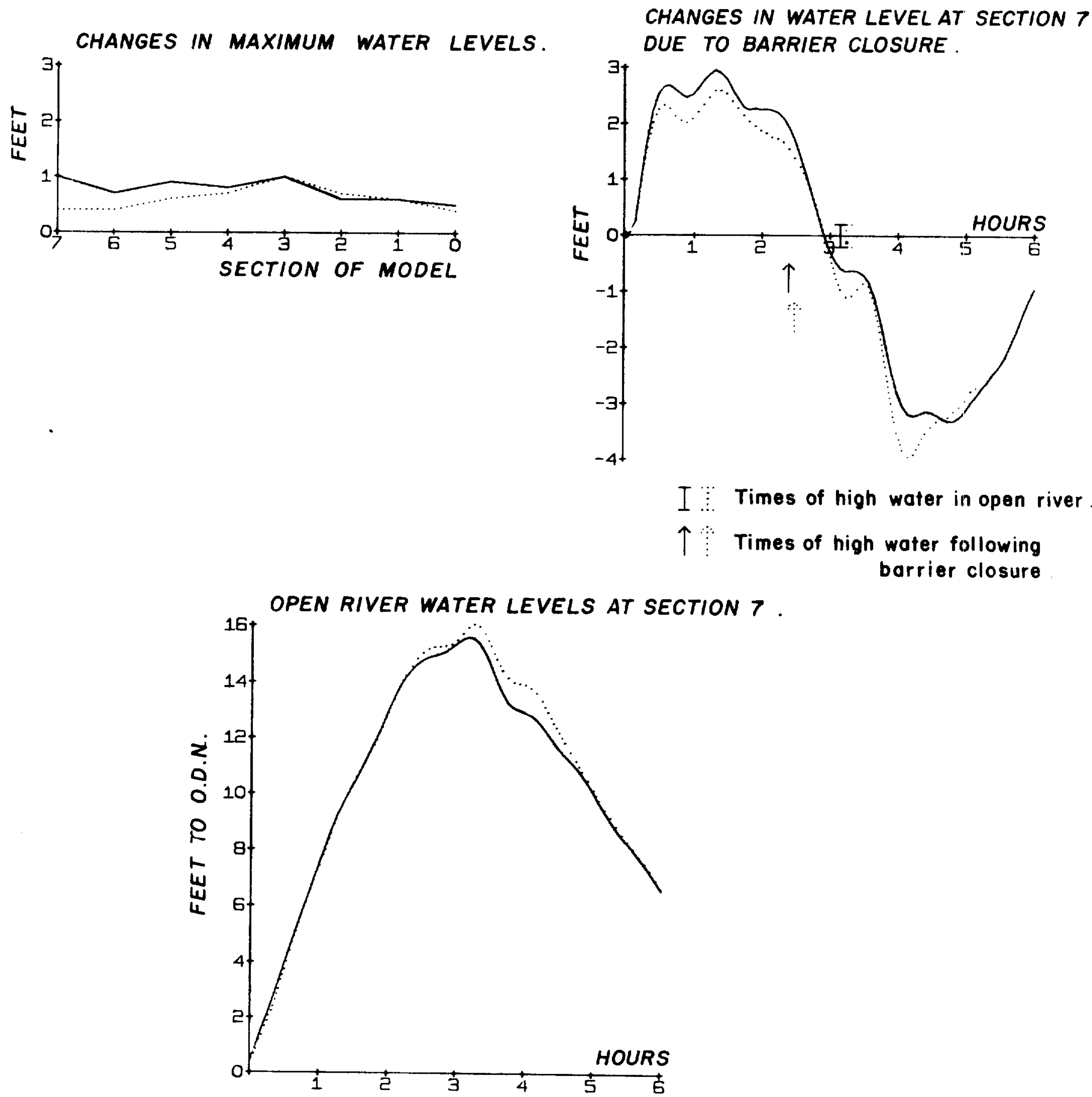
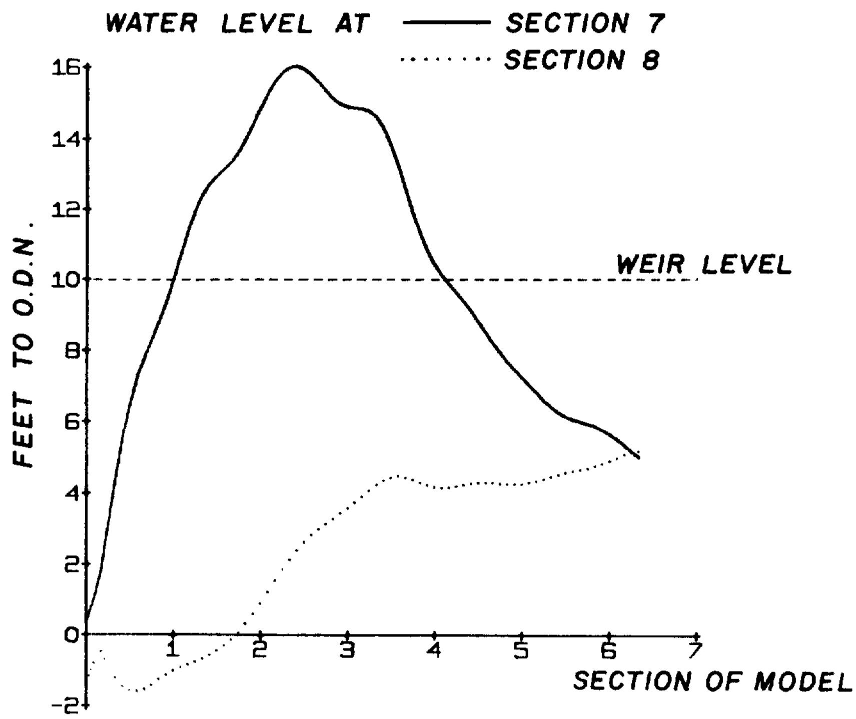


FIGURE 8

OVERSHOOTING PARTIAL CLOSURE WITH A WEIR LEVEL OF 12 FEET O.D.N.
12 FOOT SURGE WITH UPLAND FLOW = 2,500 cu secs.
CLOSURE LEVEL = O.D.N.



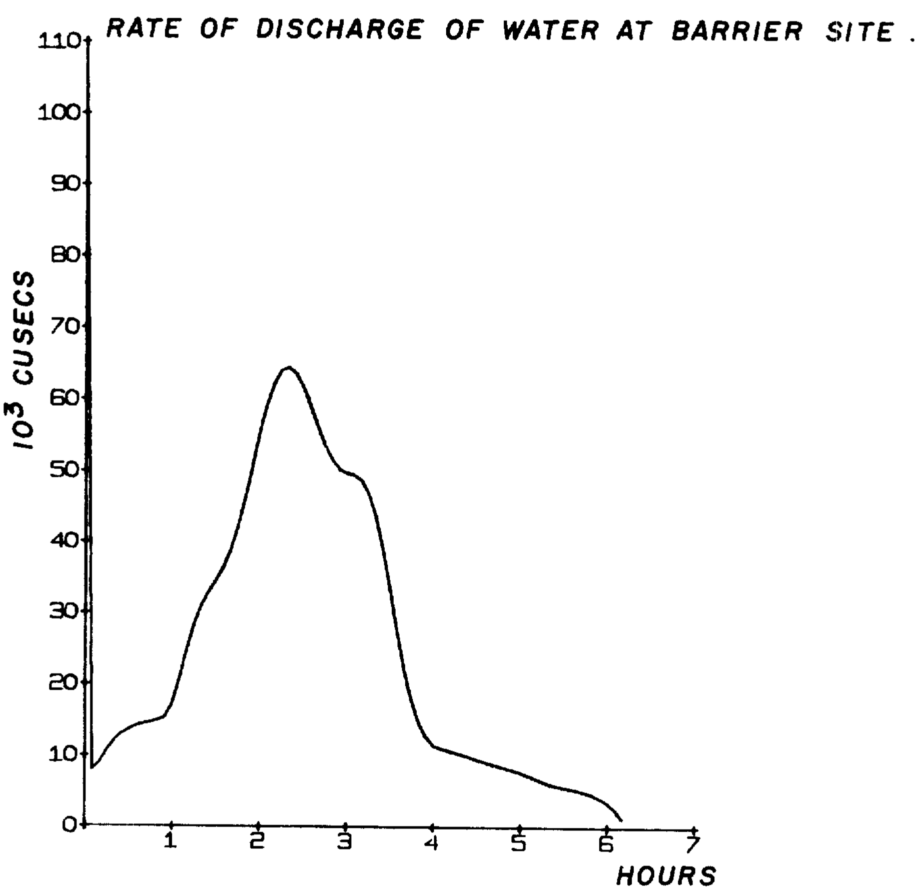
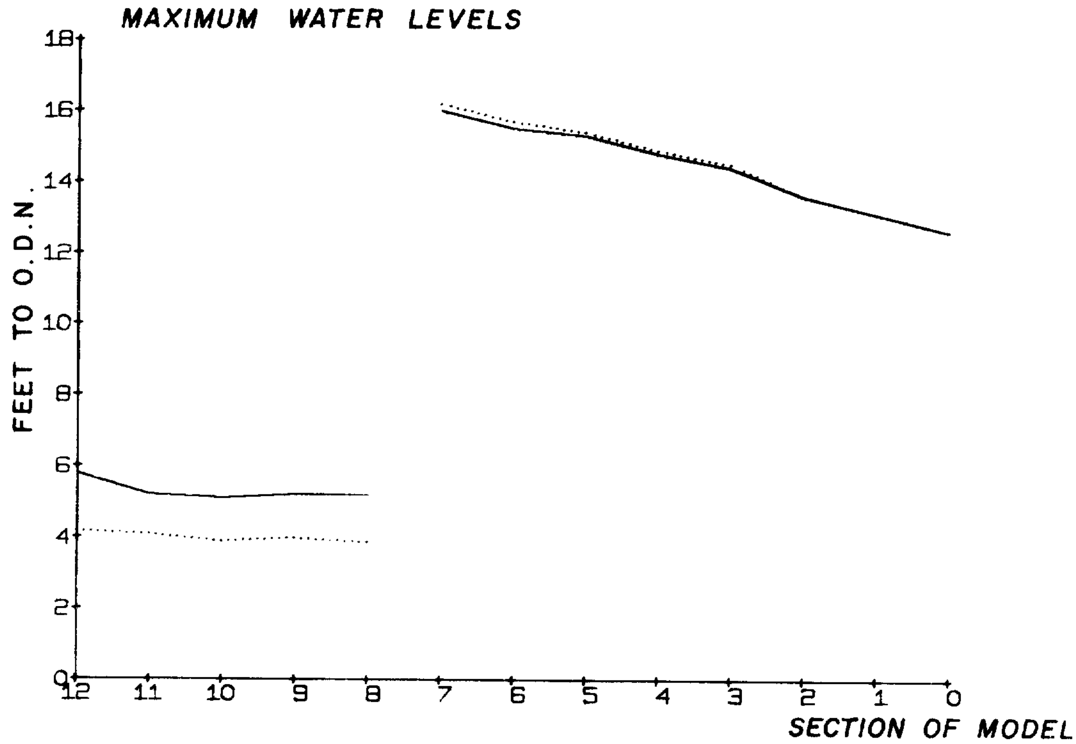


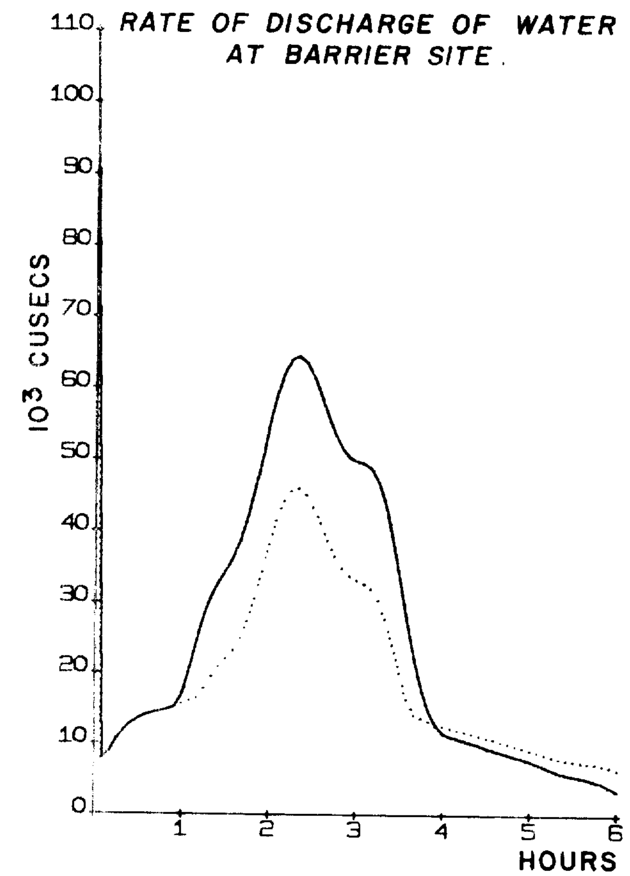
FIGURE 9

OVERSHOOTING PARTIAL CLOSURE WITH A WEIR LEVEL OF —— 10 FEET O.D.N.
······ 12 FEET O.D.N.

12 FOOT SURGE WITH UPLAND FLOW = 2,500 cu secs.

CLOSURE LEVEL = O.D.N.





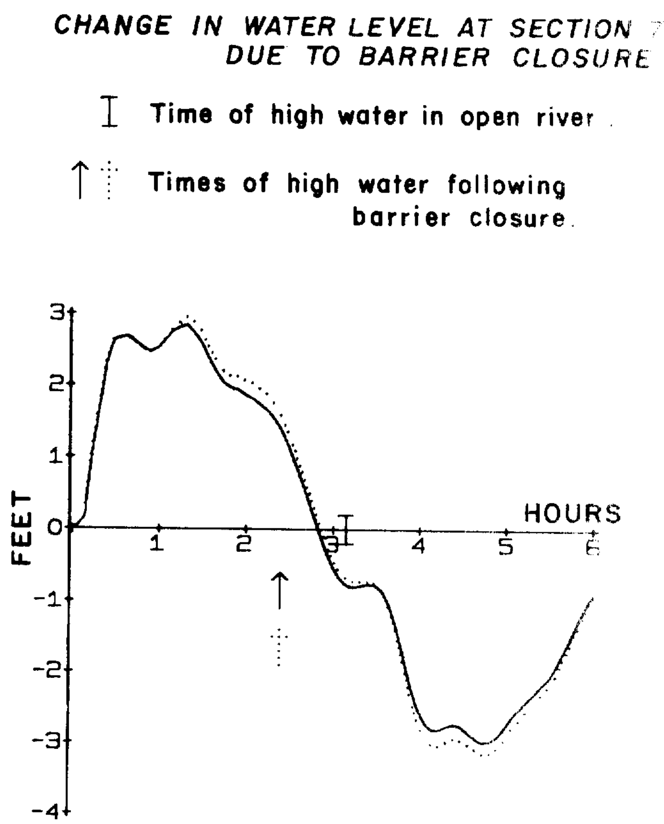
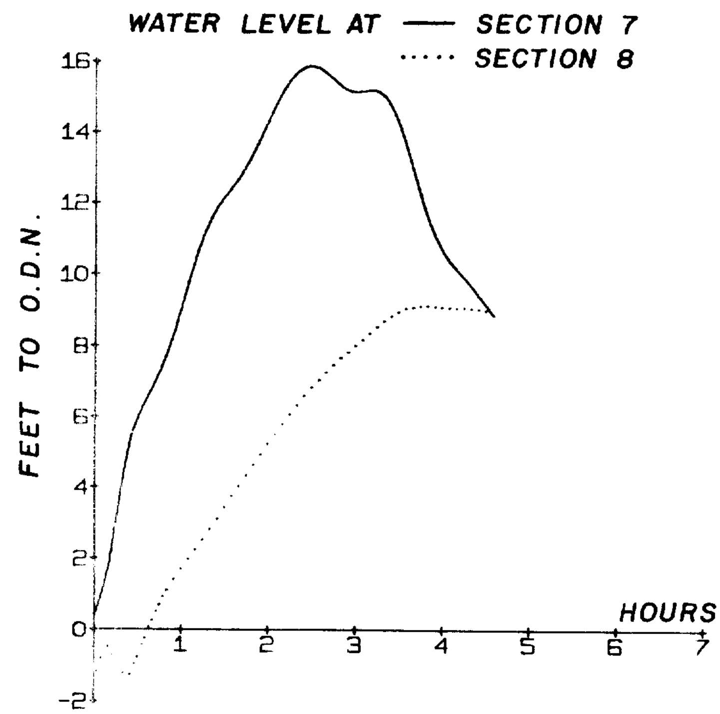


FIGURE 10

UNDERSHOOTING PARTIAL CLOSURE WITH A GAP WIDTH OF 5 FEET
12 FOOT SURGE WITH UPLAND FLOW = 2,500 cusecs.
CLOSURE LEVEL = O.D.N.



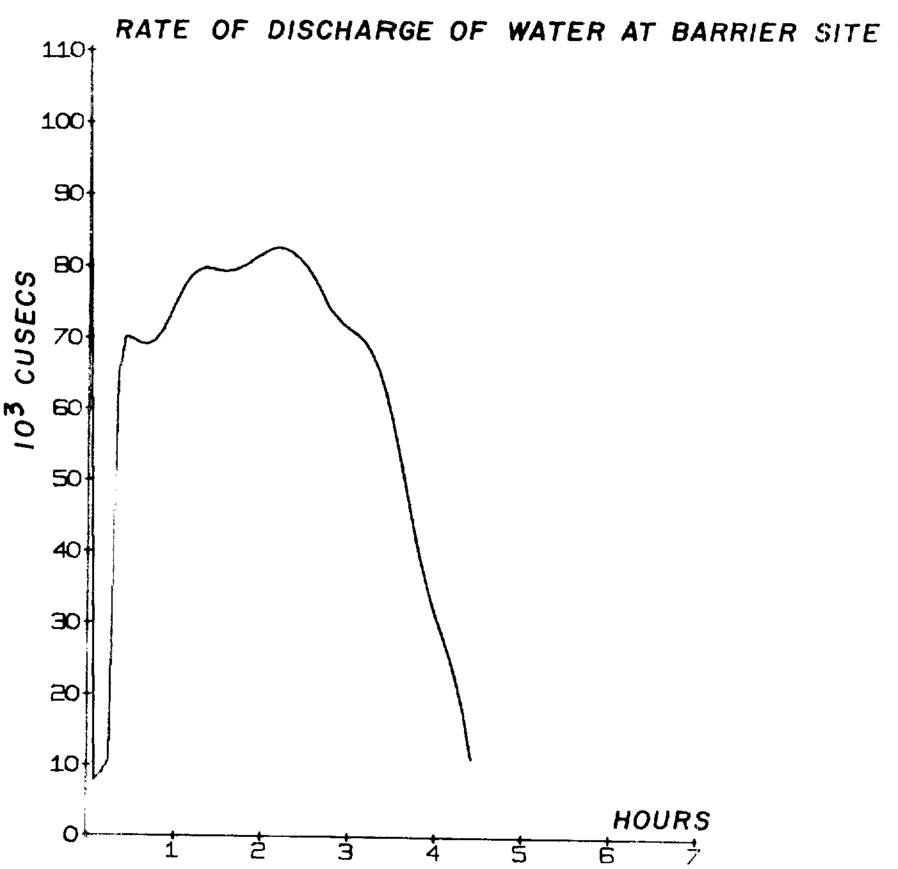


FIGURE 11

UNDERSHOOTING PARTIAL CLOSURE WITH A GAP WIDTH OF —— 5 FEET
······ 3 FEET

12 FOOT SURGE WITH UPLAND FLOW = 2,500 cusecs.
CLOSURE LEVEL = O.D.N.

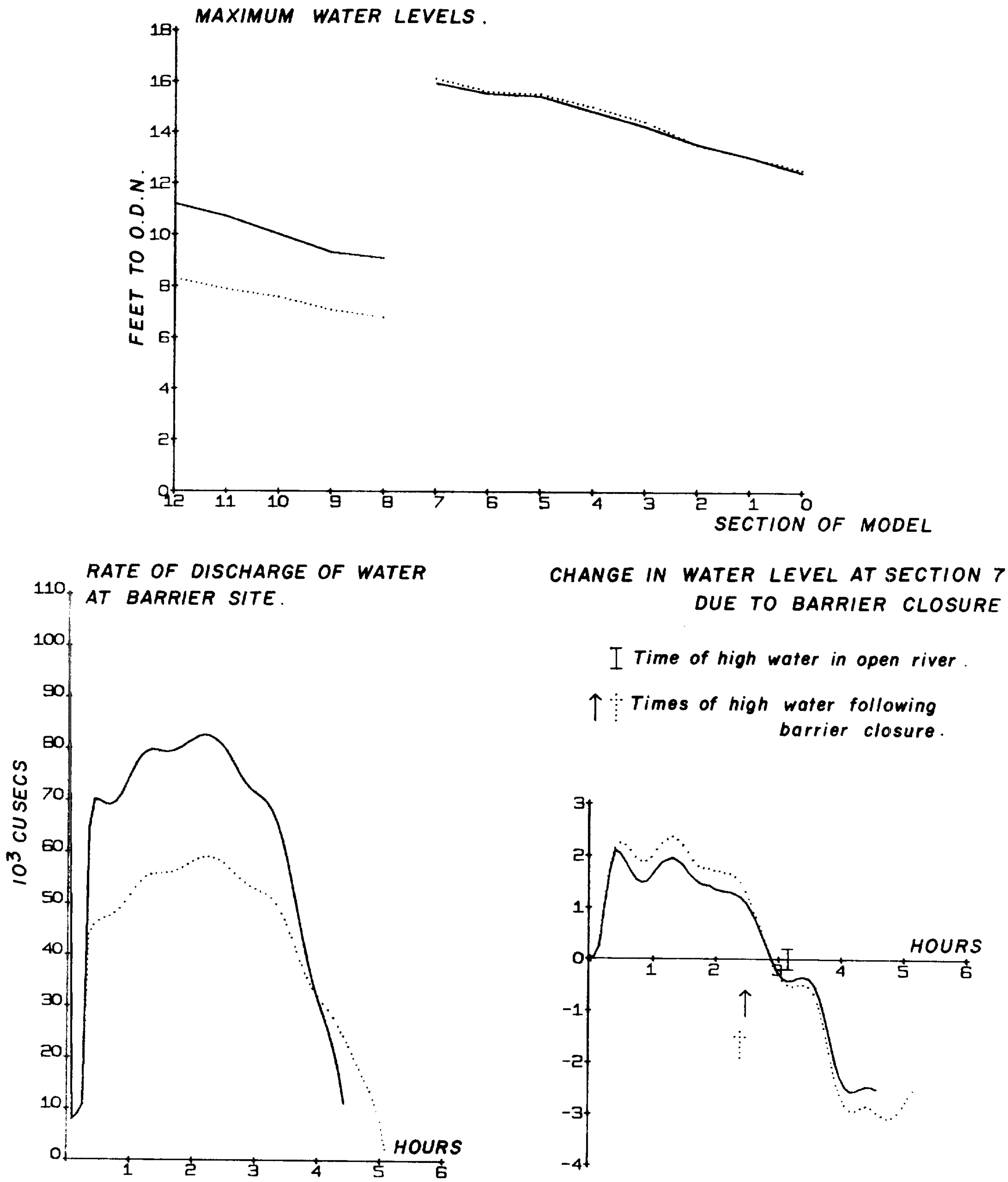
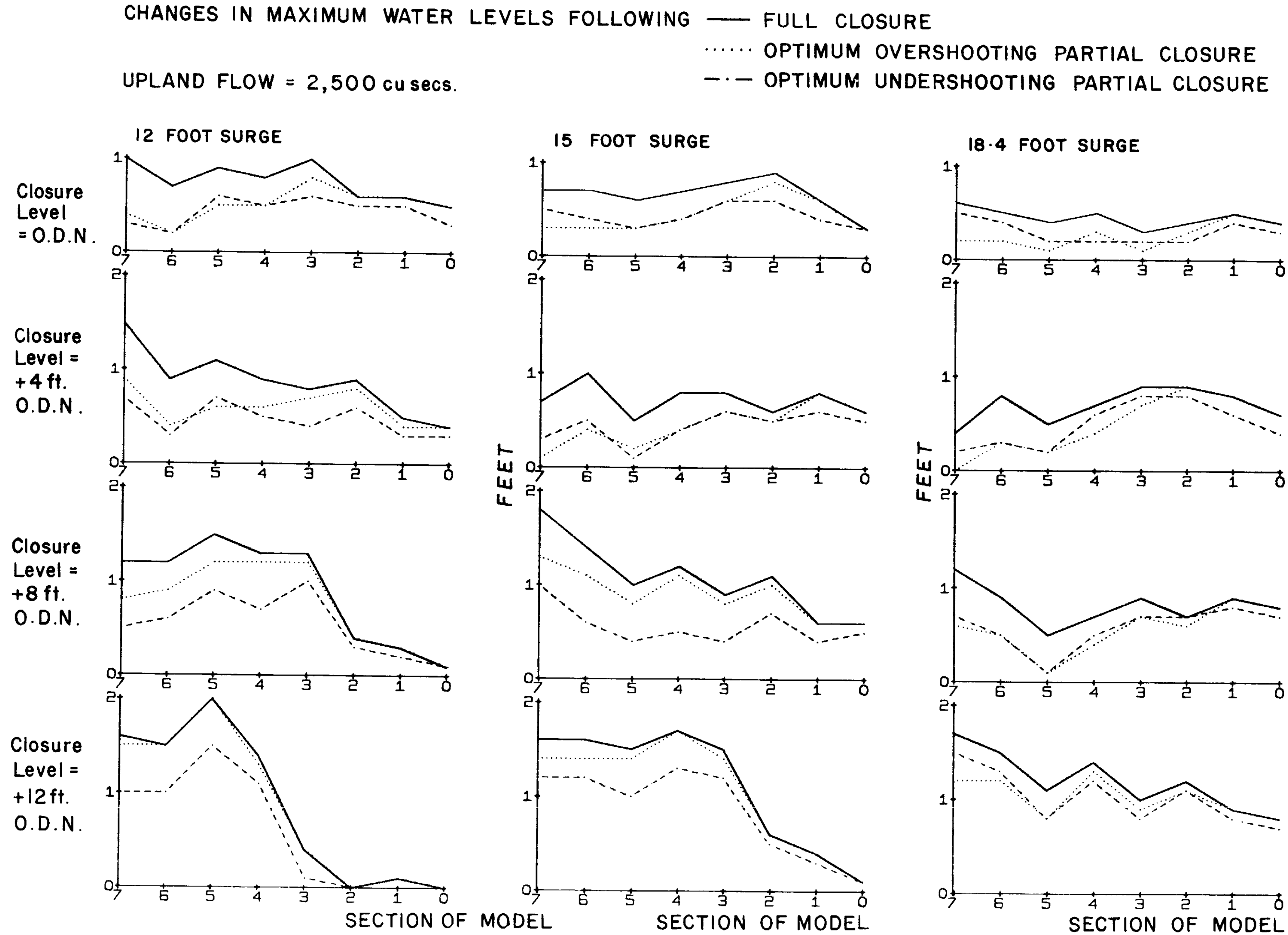


FIGURE 12



CHANGES IN MAXIMUM WATER LEVELS FOLLOWING
FULL CLOSURE
OPTIMUM OVERSHOOTING PARTIAL CLOSURE
OPTIMUM UNDERSHOOTING PARTIAL CLOSURE
UPLAND FLOW = 2,500 cu secs.
12 FOOT SURGE
15 FOOT SURGE
18·4 FOOT SURGE
Closure Level = O.D.N.
Closure Level = +4 ft. O.D.N.
Closure Level = +8 ft. O.D.N.
Closure Level = +12 ft. O.D.N.
FEET
SECTION OF MODEL

FIGURE 13 CHANGE IN WATER LEVEL AT SECTION 7 AND RATE OF DISCHARGE OF WATER AT BARRIER SITE FOLLOWING :

UPLAND FLOW = 2,500 cu secs. CLOSURE LEVEL = 12 ft. O.D.N.

······· OPTIMUM OVERSHOOTING PARTIAL CLOSURE
—·— OPTIMUM UNDERSHOOTING PARTIAL CLOSURE
—— FULL CLOSURE

TIME OF HIGH WATER IN OPEN RIVER .
TIME OF HIGH WATER FOLLOWING FULL CLOSURE
TIME OF HIGH WATER FOLLOWING OVERSHOOTING PARTIAL CLOSURE
TIME OF HIGH WATER FOLLOWING UNDERSHOOTING PARTIAL CLOSURE

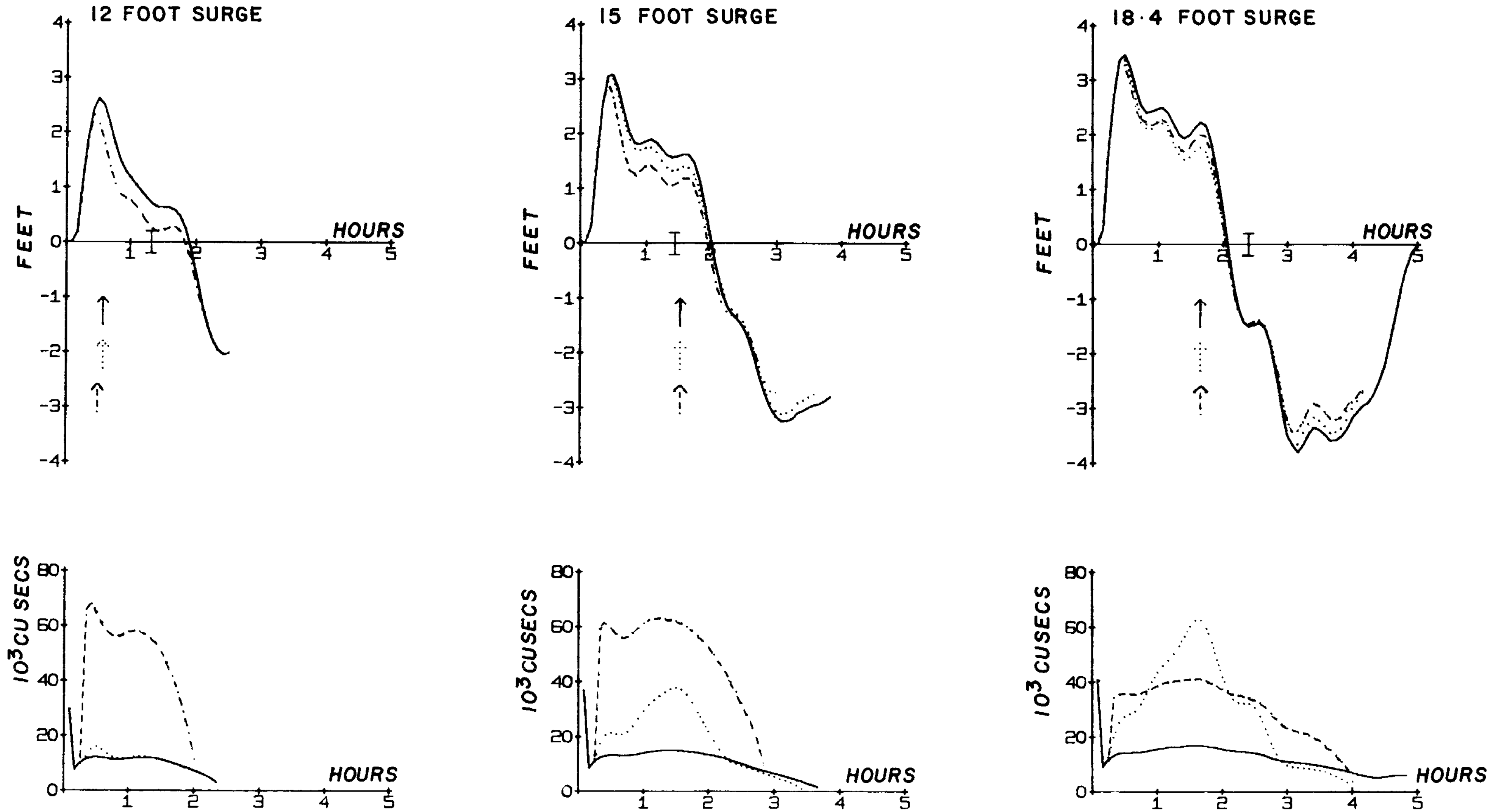


TABLE I

CRITERION FOR CHANGE OF FLOW TYPE

HEAD OVER WEIR SHOWN IN FEET - WL = WEIR LEVEL IN FEET TO O.D.N.

H_1 (HEAD OVER WEIR ON UPSTREAM SIDE)	H_2 (HEAD OVER WEIR ON DOWNSTREAM SIDE)															
	FROM DIVING JET TO SURFACE JET								FROM SURFACE JET TO DIVING JET							
	WL=-4	WL=-2	WL= 0	WL= 2	WL= 4	WL= 6	WL= 8	WL=10	WL=-4	WL=-2	WL= 0	WL= 2	WL= 4	WL= 6	WL= 8	WL=10
1	1·30	1·15	1·00	1·00	1·00	1·00	1·00	1·00	1·30	1·15	1·00	1·00	1·00	1·00	1·00	1·00
2	1·30	1·15	1·00	1·00	1·00	1·00	1·00	1·00	1·30	1·15	1·00	1·00	1·00	1·00	1·00	1·00
3	1·75	1·68	1·60	1·60	1·60	1·60	1·60	1·60	1·45	1·35	1·25	1·18	1·11	1·04	0·97	0·90
4	2·20	2·20	2·20	2·20	2·20	2·20	2·20	2·20	1·60	1·55	1·50	1·36	1·22	1·08	0·94	0·80
5	3·15	3·17	3·20	3·08	2·96	2·84	2·72	2·60	1·80	1·79	1·75	1·53	1·31	1·09	0·87	0·65
6	4·10	4·15	4·20	3·96	3·72	3·48	3·24	3·00	2·00	2·00	2·00	1·70	1·40	1·10	0·80	0·50
7	4·70	4·67	4·65	4·35	4·05	3·75	3·45	3·15	2·35	2·30	2·25	1·91	1·57	1·23	0·89	0·55
8	5·30	5·20	5·10	4·74	4·38	4·02	3·66	3·30	2·70	2·60	2·50	2·22	1·84	1·46	0·98	0·60
9	5·60	5·47	5·35	4·94	4·53	4·12	3·71	3·30	2·95	2·90	2·85	2·44	2·03	1·62	1·21	0·80
10	5·90	5·75	5·60	4·34	4·08	3·82	3·56	3·30	3·20	3·20	3·20	2·76	2·32	1·88	1·44	1·00
11	6·15	5·95	5·75	5·25	4·75	4·25	3·75	3·25	3·95	3·80	3·65	3·18	2·71	2·24	1·77	1·30
12	6·40	6·15	5·90	5·36	4·82	4·28	3·74	3·20	4·70	4·40	4·10	3·60	3·10	2·60	2·10	1·60
13	6·55	6·25	5·95	5·40	4·85	4·30	3·75	3·20	5·60	5·07	4·55	4·07	3·59	3·11	2·63	2·15
14	6·70	6·35	6·00	5·44	4·88	4·32	3·76	3·20	6·50	5·75	5·00	4·54	4·08	3·62	3·16	2·70
15	7·00	6·50	6·00	5·44	4·88	4·32	3·76	3·20	6·90	6·17	5·45	4·93	4·41	3·89	3·37	2·85
16	7·30	6·65	6·00	5·44	4·88	4·32	3·76	3·20	7·30	6·60	5·90	5·32	4·74	4·16	3·58	3·00
17	7·40	6·70	6·00	5·44	4·88	4·32	3·76	3·20	7·40	6·68	5·95	5·38	4·81	4·24	3·67	3·10
18	7·50	6·75	6·00	5·44	4·88	4·32	3·76	3·20	7·50	6·75	6·00	5·44	4·88	4·32	3·76	3·20
19	7·50	6·75	6·00	5·44	4·88	4·32	3·76	3·20	7·50	6·75	6·00	5·44	4·88	4·32	3·76	3·20
20	7·50	6·75	6·00	5·44	4·88	4·32	3·76	3·20	7·50	6·75	6·00	5·44	4·88	4·32	3·76	3·20

TABLE 2

CRITERION FOR FREE DISCHARGE

WATER LEVELS SHOWN IN FEET TO O.D.N. - GW = GAP WIDTH BELOW BARRIER GATES IN FEET

WATER LEVEL ON DOWNRIVER SIDE	WATER LEVEL ON UPRIVER SIDE					WATER LEVEL ON DOWNRIVER SIDE	WATER LEVEL ON UPRIVER SIDE				
	GW = 1	GW = 2	GW = 3	GW = 4	GW = 5		GW = 1	GW = 2	GW = 3	GW = 4	GW = 5
-5	-21.04	-18.90	-16.75	-14.61	-12.46	10	-13.61	-11.47	-9.32	-7.18	-5.03
-4	-20.55	-18.40	-16.26	-14.11	-11.96	11	-13.14	-10.99	-8.84	-6.69	-4.54
-3	-20.05	-17.91	-15.76	-13.61	-11.47	12	-12.46	-10.36	-8.25	-6.15	-4.05
-2	-19.56	-17.41	-15.27	-13.12	-10.97	13	-11.96	-9.85	-7.75	-5.65	-3.55
-1	-19.06	-16.92	-14.77	-12.62	-10.48	14	-11.66	-9.51	-7.35	-5.20	-3.05
0	-18.57	-16.42	-14.28	-12.13	-9.98	15	-11.12	-8.98	-6.84	-4.70	-2.56
1	-18.07	-15.93	-13.78	-11.63	-9.49	16	-10.69	-8.53	-6.37	-4.22	-2.06
2	-17.58	-15.43	-13.29	-11.14	-8.99	17	-9.55	-7.56	-5.57	-3.57	-1.58
3	-17.08	-14.94	-12.79	-10.64	-8.50	18	-8.31	-6.51	-4.41	-2.90	-1.10
4	-16.59	-14.44	-12.30	-10.15	-8.00	19	-6.85	-5.29	-3.74	-2.18	-0.62
5	-16.09	-13.95	-11.80	-9.65	-7.51	20	-5.41	-4.09	-2.78	-1.46	-0.14
6	-15.60	-13.45	-11.31	-9.16	-7.01	21	-4.05	-2.97	-1.90	-0.82	0.24
7	-15.10	-12.96	-10.81	-8.66	-6.52	22	-3.08	-2.18	-1.28	-0.38	0.51
8	-14.61	-12.46	-10.32	-8.17	-6.02	23	-2.11	-1.39	-0.66	0.06	0.78
9	-14.11	-11.97	-9.82	-7.67	-5.53	24	-1.15	-0.59	-0.04	0.50	1.06

TABLE 3

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O.D.N.)

FULL BARRIER CLOSURE

12 FOOT SURGE

UPLAND FLOW (CUSECS.)	CLOSURE LEVEL (FEET TO O.D.N.)	SECTION OF MODEL												
		0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER	12·1	12·5	13·0	13·6	14·3	14·8	15·3	15·6	15·8	16·0	17·3	18·5	19·1
	0	12·6	13·1	13·6	14·6	15·1	15·7	16·0	16·6	2·6	2·7	2·8	2·9	3·1
	4	12·5	13·0	13·9	14·4	15·2	15·9	16·2	17·1	4·4	4·5	4·6	4·7	4·9
	8	12·2	12·8	13·4	14·9	15·6	16·3	16·5	16·8	8·0	7·4	6·7	6·9	7·3
	12	12·1	12·6	13·0	14·0	15·7	16·8	16·8	17·2	12·2	11·7	10·3	11·1	12·4
20,000	OPEN RIVER	12·1	12·6	13·0	13·6	14·4	15·0	15·5	16·1	16·6	17·9	18·7	19·3	20·2
	0	12·5	13·2	13·7	14·6	15·1	15·6	15·9	16·5	6·9	7·3	7·6	8·1	9·3
	4	12·6	12·9	13·8	14·3	15·0	15·8	16·1	17·0	8·4	8·8	9·0	9·5	10·4
	8	12·2	12·8	13·4	14·8	15·5	16·2	16·4	16·8	10·6	11·0	11·3	11·7	12·2
	12	12·1	12·6	13·1	13·8	15·6	16·7	16·7	17·1	14·1	14·2	14·0	15·4	16·7

TABLE 4

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O·D·N·)

FULL BARRIER CLOSURE

15 FOOT SURGE

UPLAND FLOW (CUSECS·)	CLOSURE LEVEL (FEET TO O·D·N·)	SECTION OF MODEL												
		0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER	15·2	15·6	16·1	16·6	17·3	17·8	18·1	18·4	18·8	19·8	21·3	22·0	22·8
	0	15·5	16·2	17·0	17·4	18·0	18·4	18·8	19·1	3·6	3·7	3·9	4·0	4·3
	4	15·8	16·4	16·7	17·4	18·1	18·3	19·1	19·1	5·3	5·3	5·5	5·5	5·6
	8	15·8	16·2	17·2	17·5	18·5	18·8	19·5	20·2	8·3	7·8	7·8	7·8	8·1
	12	15·3	16·0	16·7	18·1	19·0	19·3	19·7	20·0	12·4	11·6	10·5	10·7	11·5
20,000	OPEN RIVER	15·2	15·6	16·1	16·6	17·3	17·9	18·4	18·9	20·3	21·2	22·1	23·0	23·3
	0	15·6	16·1	16·9	17·3	18·0	18·3	18·7	19·1	9·1	9·2	9·4	9·7	10·8
	4	15·8	16·4	16·8	17·4	18·0	18·3	19·0	18·9	9·9	10·0	10·2	10·5	11·3
	8	15·9	16·2	17·1	17·4	18·2	18·6	19·2	19·9	11·0	11·6	11·8	11·9	12·5
	12	15·4	16·0	16·8	18·0	18·9	19·1	19·6	20·0	14·1	14·3	14·5	15·0	16·4

TABLE 5

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O·D·N·)

FULL BARRIER CLOSURE

18·4 FOOT SURGE

UPLAND FLOW (CUSECS·)	CLOSURE LEVEL (FEET TO O·D·N·)	SECTION OF MODEL												
		0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER	18·4	18·7	19·2	19·8	20·5	21·1	21·3	21·8	22·8	24·1	24·9	25·8	26·4
	0	18·8	19·2	19·6	20·1	21·0	21·5	21·8	22·4	4·5	4·6	4·8	4·9	5·0
	4	19·0	19·5	20·1	20·7	21·2	21·6	22·1	22·2	6·1	6·3	6·3	6·4	6·5
	8	19·2	19·6	19·9	20·7	21·2	21·6	22·2	23·0	8·4	8·4	8·4	8·4	8·5
	12	19·2	19·6	20·4	20·8	21·9	22·2	22·8	23·5	12·8	12·0	11·3	11·4	12·2
20,000	OPEN RIVER	18·5	18·7	19·3	19·9	20·5	21·1	21·7	22·7	23·5	24·4	25·4	26·5	26·9
	0	18·8	19·2	19·5	20·0	21·0	21·5	21·8	22·4	10·2	10·6	10·9	11·2	11·9
	4	18·9	19·4	20·0	20·6	21·1	21·5	21·9	22·1	11·5	11·8	11·9	12·2	12·8
	8	19·2	19·6	20·0	20·5	21·1	21·6	22·1	22·6	12·8	13·2	13·4	13·7	14·0
	12	19·2	19·6	20·3	20·5	21·8	22·1	22·7	23·4	14·6	14·9	15·3	15·5	16·7

TABLE 6

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O·D·N·)

OVERSHOOTING MODE OF PARTIAL BARRIER CLOSURE

12 FOOT SURGE

UPLAND FLOW (CUSECS.)	CLOSURE LEVEL (FEET TO O·D·N·)	FIXED GATE LEVEL (FEET TO O·D·N·)	SECTION OF MODEL												
			0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER		12·1	12·5	13·0	13·6	14·3	14·8	15·3	15·6	15·8	16·0	17·3	18·5	19·1
	0	10·0	12·6	13·1	13·6	14·4	14·8	15·3	15·5	16·0	5·2	5·2	5·1	5·2	5·8
	4	10·0	12·5	12·9	13·8	14·3	14·9	15·4	15·7	16·5	6·8	7·0	7·1	7·6	8·1
	8	12·0	12·2	12·8	13·4	14·8	15·5	16·0	16·2	16·4	8·0	8·0	8·0	8·4	8·8
	12	16·0	12·1	12·6	13·0	14·0	15·6	16·8	16·8	17·1	12·2	11·7	10·4	11·2	12·5
20,000	OPEN RIVER		12·1	12·6	13·0	13·6	14·4	15·0	15·5	16·1	16·6	17·9	18·7	19·3	20·2
	0	10·0	12·5	13·2	13·7	14·5	14·9	15·2	15·5	15·9	9·2	9·4	9·7	10·2	11·1
	4	10·0	12·6	12·9	13·8	14·2	14·8	15·4	15·6	16·4	10·7	11·2	11·5	12·2	12·8
	8	12·0	12·2	12·8	13·3	14·8	15·4	15·9	16·1	16·4	11·8	12·1	12·4	12·9	13·7
	12	FULL CLOSURE	12·1	12·6	13·1	13·8	15·6	16·7	16·7	17·1	14·1	14·2	14·0	15·4	16·7

TABLE 7

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O·D·N·)

OVERSHOOTING MODE OF PARTIAL BARRIER CLOSURE

15 FOOT SURGE

UPLAND FLOW (CUSECS·)	CLOSURE LEVEL (FEET TO O·D·N·)	FIXED GATE LEVEL (FEET TO O·D·N·)	SECTION OF MODEL												
			0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER		15·2	15·6	16·1	16·6	17·3	17·8	18·1	18·4	18·8	19·8	21·3	22·0	22·8
	0	13·0	15·5	16·2	16·9	17·2	17·7	18·1	18·4	18·7	6·2	6·2	6·3	6·5	7·0
	4	12·0	15·8	16·4	16·6	17·2	17·7	18·0	18·5	18·5	8·9	8·8	8·7	9·1	9·6
	8	14·0	15·8	16·2	17·1	17·4	18·4	18·6	19·2	19·7	9·7	9·5	9·4	9·6	10·0
	12	16·0	15·3	16·0	16·7	18·0	19·0	19·2	19·5	19·8	12·4	11·6	11·5	11·6	12·1
20,000	OPEN RIVER		15·2	15·6	16·1	16·6	17·3	17·9	18·4	18·9	20·3	21·2	22·1	23·0	23·3
	0	13·0	15·6	16·1	16·8	17·1	17·7	18·1	18·4	18·7	10·5	10·6	10·8	11·0	11·8
	4	12·0	15·8	16·4	16·7	17·2	17·7	17·9	18·4	18·4	12·4	12·7	13·0	13·3	14·0
	8	14·0	15·9	16·2	17·0	17·3	18·1	18·5	18·9	19·4	12·6	13·4	13·6	14·0	14·7
	12	FULL CLOSURE	15·4	16·0	16·8	18·0	18·9	19·1	19·6	20·0	14·1	14·3	14·5	15·0	16·4

TABLE 8

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O·D·N·)

OVERSHOOTING MODE OF PARTIAL CLOSURE

18·4 FOOT SURGE

UPLAND FLOW (CUSECS·)	CLOSURE LEVEL (FEET TO O·D·N·)	FIXED GATE LEVEL (FEET TO O·D·N·)	SECTION OF MODEL												
			0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER		18·4	18·7	19·2	19·8	20·5	21·1	21·3	21·8	22·8	24·1	24·9	25·8	26·4
	0	16·0	18·8	19·2	19·5	19·9	20·8	21·2	21·5	22·0	7·0	7·0	7·1	7·5	7·9
	4	16·0	19·0	19·5	20·1	20·5	20·9	21·3	21·6	21·8	8·9	8·8	8·9	9·3	9·7
	8	16·0	19·2	19·6	19·8	20·5	20·9	21·2	21·8	22·4	11·0	11·0	11·0	11·5	11·8
	12	17·0	19·2	19·6	20·3	20·7	21·8	21·9	22·5	23·0	13·5	13·4	13·4	13·4	13·6
20,000	OPEN RIVER		18·5	18·7	19·3	19·9	20·5	21·1	21·7	22·7	23·5	24·4	25·4	26·5	26·9
	0	16·0	18·8	19·2	19·5	20·0	20·8	21·2	21·5	21·9	12·4	12·7	12·9	13·1	13·7
	4	16·0	18·9	19·4	20·0	20·4	20·8	21·2	21·5	21·7	13·5	13·5	13·6	13·9	14·5
	8	18·0	19·2	19·6	19·9	20·4	20·9	21·4	21·8	22·3	13·5	13·8	13·9	14·1	14·7
	12	FULL CLOSURE	19·2	19·6	20·3	20·5	21·8	22·1	22·7	23·4	14·6	14·9	15·3	15·5	16·7

TABLE 9

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O.D.N.)

UNDERSHOOTING MODE OF PARTIAL CLOSURE

12 FOOT SURGE

UPLAND FLOW (CUSECS.)	CLOSURE LEVEL (FEET TO O.D.N.)	GAP UNDER BARRIER (FEET)	SECTION OF MODEL												
			0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER		12.1	12.5	13.0	13.6	14.3	14.8	15.3	15.6	15.8	16.0	17.3	18.5	19.1
	0	5.0	12.4	13.0	13.5	14.2	14.8	15.4	15.5	15.9	9.1	9.3	10.0	10.7	11.2
	4	5.0	12.4	12.8	13.6	14.0	14.8	15.5	15.6	16.3	9.7	10.1	10.7	11.3	11.7
	8	5.0	12.2	12.7	13.3	14.6	15.0	15.7	15.9	16.1	10.4	10.7	11.4	12.0	12.4
	12	5.0	12.1	12.6	13.0	13.7	15.4	16.3	16.3	16.6	12.2	12.5	13.6	13.8	13.9
20,000	OPEN RIVER		12.1	12.6	13.0	13.6	14.4	15.0	15.5	16.1	16.6	17.9	18.7	19.3	20.2
	0	5.0	12.4	13.0	13.6	14.3	14.9	15.4	15.5	16.0	11.8	12.4	13.2	13.8	14.2
	4	5.0	12.5	12.7	13.5	14.1	14.7	15.4	15.6	16.3	12.2	12.8	13.7	14.1	14.8
	8	5.0	12.2	12.7	13.3	14.5	15.0	15.7	15.9	16.1	13.3	13.7	14.1	15.2	15.6
	12	FULL CLOSURE	12.1	12.6	13.1	13.8	15.6	16.7	16.7	17.1	14.1	14.2	14.0	15.4	16.7

TABLE 10

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O.D.N.)

UNDERSHOOTING MODE OF PARTIAL CLOSURE

15 FOOT SURGE

UPLAND FLOW (CUSECS.)	CLOSURE LEVEL (FEET TO O.D.N.)	GAP UNDER BARRIER (FEET)	SECTION OF MODEL												
			0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER		15.2	15.6	16.1	16.6	17.3	17.8	18.1	18.4	18.8	19.8	21.3	22.0	22.8
	0	5.0	15.5	16.0	16.7	17.2	17.7	18.1	18.5	18.9	11.2	11.6	12.3	12.7	13.1
	4	5.0	15.7	16.2	16.6	17.2	17.7	17.9	18.6	18.7	11.7	12.1	12.7	13.2	13.5
	8	5.0	15.7	16.0	16.8	17.0	17.8	18.2	18.7	19.4	12.7	13.2	13.5	13.9	14.1
	12	4.0	15.3	15.9	16.6	17.8	18.6	18.8	19.3	19.6	13.5	13.7	13.9	14.2	14.2
20,000	OPEN RIVER		15.2	15.6	16.1	16.6	17.3	17.9	18.4	18.9	20.3	21.2	22.1	23.0	23.3
	0	4.0	15.5	16.0	16.7	17.2	17.8	18.1	18.5	19.0	13.1	13.5	13.9	14.5	14.8
	4	3.0	15.7	16.3	16.7	17.2	17.7	18.1	18.7	18.8	12.7	13.1	13.4	13.9	14.2
	8	2.0	15.8	16.1	16.9	17.2	17.9	18.5	19.0	19.6	12.7	13.3	13.6	14.1	14.5
	12	FULL CLOSURE	15.4	16.0	16.8	18.0	18.9	19.1	19.6	20.0	14.1	14.3	14.5	15.0	16.4

TABLE 11

MAXIMUM WATER LEVELS ALONG THE RIVER THAMES (FEET TO O.D.N.)

UNDERSHOOTING MODE OF PARTIAL CLOSURE

18.4 FOOT SURGE

UPLAND FLOW (CUSECS.)	CLOSURE LEVEL (FEET TO O.D.N.)	GAP UNDER BARRIER (FEET)	SECTION OF MODEL												
			0	1	2	3	4	5	6	7	8	9	10	11	12
2,500	OPEN RIVER		18.4	18.7	19.2	19.8	20.5	21.1	21.3	21.8	22.8	24.1	24.9	25.8	26.4
	0	4.0	18.7	19.1	19.4	20.0	20.7	21.3	21.7	22.3	13.2	13.3	13.3	13.3	13.3
	4	4.0	18.8	19.3	20.0	20.6	21.1	21.3	21.6	22.0	13.3	13.6	13.9	13.9	13.9
	8	3.0	19.1	19.5	19.9	20.5	21.0	21.2	21.8	22.5	12.9	13.1	13.2	13.4	13.5
	12	2.0	19.1	19.5	20.3	20.6	21.7	21.9	22.6	23.3	13.2	13.4	13.5	13.9	14.7
20,000	OPEN RIVER		18.5	18.7	19.3	19.9	20.5	21.1	21.7	22.7	23.5	24.4	25.4	26.5	26.9
	0	2.0	18.8	19.1	19.5	20.0	20.9	21.5	21.8	22.4	13.3	13.6	13.7	13.9	14.3
	4	2.0	18.8	19.3	20.0	20.6	21.0	21.4	21.7	22.1	13.6	13.7	14.0	14.3	14.8
	8	1.0	19.2	19.5	19.9	20.5	21.1	21.6	22.0	22.6	13.2	13.4	13.5	13.8	14.2
	12	FULL CLOSURE	19.2	19.6	20.3	20.5	21.8	22.1	22.7	23.4	14.6	14.9	15.3	15.5	16.7