A NUMERICAL MODEL OF VICTORIA HARBOUR TO PREDICT TIDAL RESPONSE TO PROPOSED HYDRAULIC STRUCTURES

by

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INTRODUCTION

Victoria Harbour is situated on the north shore of the Strait of Juan de Fuca, 60 miles eastward of its ocean entrance. The harbour is part of a tidal estuary, extending through Gorge Waters to Portage Inlet over a distance of five miles (fig. 1). The estuary is about 50 feet deep at its entrance and shallows to a depth of less than ten feet in the Gorge and Portage Inlet.

The flow characteristics are predominantly tidal. The fresh-water runoff into the system is supplied by two creeks but is negligible.

The outer harbour accommodates large ocean-going vessels loading grain and lumber; the inner and upper harbours are used by ferries and coasters. The Gorge Waters and Portage Inlet are accessible only to small craft and have no significance as a navigable waterway. However, located in the heart of a growing urban area with a population of close to 200,000, this relatively large body of water has become an invaluable recreational asset and a unique tourist attraction (tourism accounts for about one-half of Victoria's income).

Unfortunately, the estuary has been a receiving water for industrial and domestic wastes and although much has been done to introduce better disposal methods, the Gorge and Portage Inlet are still so badly polluted that their beaches have remained closed for quite some time. Marine life in the estuary has been surprisingly resilient. The large eelgrass beds in Portage Inlet are still an important herring spawning area and there are minor stocks of salmon, trout and oysters.

In recent years, a number of schemes have been proposed to improve the water quality of the upper basin. Two of these proposals involve the following hydraulic structures:

1. Construction of a dam between Victoria Harbour and the Gorge to prevent entry of polluted harbour water.[1] This proposal assumes that the contamination of the Gorge originates in Victoria Harbour and that any direct discharge from ineffective septic tanks into the waterway will soon be eliminated by sewers discharging into the Strait of Juan de Fuca. Even if the water in Victoria Harbour could somehow be kept clean, the dam would still be equally valuable in maintaining a constant water level and a higher water temperature in the upper basin. A two-mile long reservoir would thus be created in the centre of the city, a great recreational asset. The water would be either fresh or salt.

2. Construction of a canal between Portage Inlet and Esquimalt Harbour to flush the basin and to scour out the putrid Portage Inlet. A tempting ravine for this project already exists between Portage Inlet and the north-eastern shore of Esquimalt Harbour. A canal would link two scenic inlets with a protected waterway and open up entirely new possibilities for water tourism. However, a canal would not prevent polluted Victoria Harbour water from entering the Gorge and it would not maintain a constant level in the upper basin.

The two concepts still receive considerable attention from the municipal authorities and the provincial government. They are based on extensive ecological research, summarized in a publication by the Biology Department of the University of Victoria.[2]

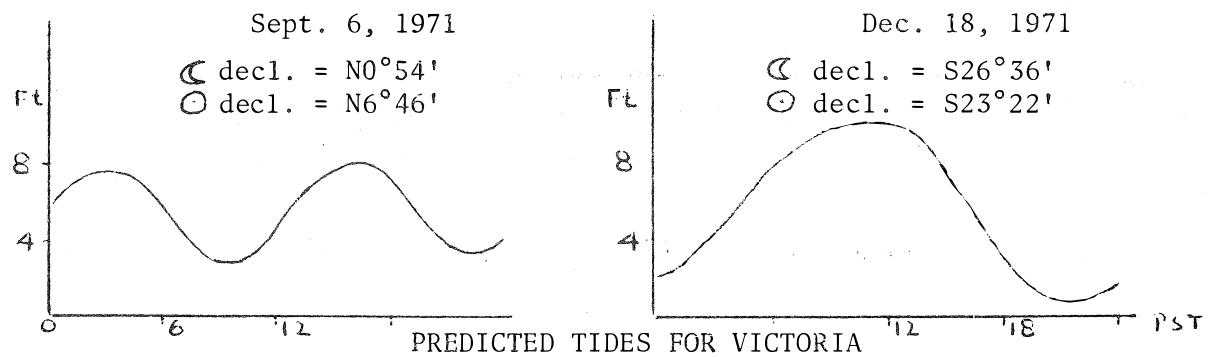
Either structure would change the basin's shape and consequently its response to the tides in the adjacent Strait of Juan de Fuca.

This report examines possible changes in tidal behaviour as a result of the proposed structures. A numerical model is developed and tested for the existing estuary and then modified to include the dam and the canal.

TIDAL CHARACTERISTICS

Tides in the approaches to Victoria Harbour are mixed (either diurnal or semi-diurnal). At its latitude of 48½°N, the Strait of Juan de Fuca has a tidal pattern which is not only affected by the relative positions of the moon and the sun but also by their declination. Mainly because of the moon's declination, the field of tide-producing forces is rarely symmetrical with respect to the poles of the earth, resulting in different amplitudes of two successive daily high waters and low waters. This "diurnal inequality" is most pronounced when the moon is at its extreme declination (either 28°N or 28°S). It may then obscure one of the two daily low waters, giving the tide a "diurnal" appearance.

In the Strait of Juan de Fuca, the declinational effect is at its greatest near Victoria. The diurnal character of the Victoria tides becomes particularly pronounced when the sun and moon are simultaneously at their maximum declinations, as may be illustrated by the following sketch:



PREDICTED TIDES FOR VICTORIA

For the two selected dates, the moon's phases alone would suggest identical spring tides; clearly, the tides near Victoria respond more to the moon's declination than to the moon's phase.

The tidal range is 9.3 feet for large tides.[3]

The upper basin debouches into the harbour through a very narrow passage, the Gorge Narrows or Gorge Bridge (fig. 1). It is the interaction between the mixed tides and this constriction which gives this estuary its peculiar tidal characteristics.

The tides are distinctly diurnal for about 15 days per month. During these days, the harbour level is at its highest for several hours, permitting the upper basin to fill up to the same level. However, when the tide goes out, the harbour level falls rapidly, followed much more slowly by the upper basin because of the constriction. Long before the upper basin has "caught up" with the harbour, the harbour level starts to rise again. This contrast is illustrated by the two diurnal tide curves (June 9-10) in figure 8.

Because of its smaller range, a semi-diurnal tide will produce low waters, which are more uniformly distributed throughout the estuary. The high waters will then be of shorter duration and consequently a semi-diurnal high in Portage Inlet will be somewhat below that in Victoria Harbour, see figure 8 for June 1-2.

These peculiar tidal characteristics are reflected by the chart datum, which is 4.5 feet lower for Victoria Harbour than for the upper basin west of Gorge Bridge.

THE MODEL

Although the shape of the outer harbour would suggest a two-dimensional model, the observed currents in this part of the estuary are so small that acceleration and velocity components in transverse directions may be ignored. Therefore, a one-dimensional model was considered sufficiently accurate. Both flow and density are assumed vertically homogeneous.

Other assumptions are:

- The effects of wind and barometric pressure are neglected.
- Centrifugal forces in bends are ignored.
- Fresh water runoff is neglected.
- The calibration of the model excludes the possibility of super critical flow in the Gorge Narrows, which an extreme spring tide might bring about during short periods.
- The tidal input (seaward boundary condition) is assumed to be truly represented by one tide gauge on the east shore of the harbour entrance; in other words, the tides are assumed to be uniform across the one-half mile wide entrance.

The tidal computations are based on the one-dimensional shallow-water wave equations which have been derived in numerous textbooks (eg Lamb[4] and Dronkers[5]):

<u>Equation of Continuity</u>: $$\frac{\partial Q}{\partial x} + W\frac{\partial h}{\partial t} = o,$$

where Q is the discharge in ft^3/sec in x-direction, W is the width of the water surface in feet, h the elevation of the water surface above a reference level (geodetic datum in this report), and x and t are the variables for distance and time.

Equation of Motion: $\frac{\partial u}{\partial t} + u\frac{\partial u}{\partial x} = -g\frac{\partial h}{\partial x} - g\frac{u|u|}{c^2 d}$

where u is the water velocity in x-direction, g the acceleration of gravity in feet/sec^2, d is the actual water depth in feet and c the deChezy coefficient in feet$^{1/2}$ per second.

Introducing the cross-sectional area A in ft^2 and putting Q = uA, we have:

$$\frac{1}{A}\frac{\partial Q}{\partial t} - \frac{Q}{A^2}\frac{\partial A}{\partial t} + \frac{Q}{A}\left\{\frac{1}{A}\frac{\partial Q}{\partial x} - \frac{Q}{A^2}\frac{\partial A}{\partial x}\right\} =$$

$$= -g\frac{\partial h}{\partial x} - g\frac{Q|Q|}{A^2 C^2 d}$$

The two differential equations are solved by an explicit finite-difference method(6), which may be summarized as follows:

The differential equations of continuity and motion are rewritten in finite-difference form ($\Delta x, \Delta t$) and solved at the intersections of a time-space grid, bounded by the initial conditions (x-axis) and boundary conditions (t-axes) at each end of the model. After comparison of the results with actual field data (water levels and currents), the friction coefficient C is adjusted; the procedure is repeated until the predicted values agree with the observed data.

The method is called "explicit" because during each computation, only one unknown is calculated from a set of previously obtained values, while an "implicit" scheme derives at once all values of Q and h at level t + Δt from the known ones at level t, with a large number of simultaneous equations. For the Victoria model, an explicit scheme was chosen because it is generally accepted to be the most useful approach.

SCHEMATIZATION

The estuary was divided into sections, each section or "block" having constant dimensions (figure 2). The section lengths, however, varied with the depth and were further adjusted so that the section lines agreed with the locations of field measurements. This approach is a departure from the conventional explicit scheme which uses equal section lengths. A more detailed discussion on the advantages and limitations of the use of unequal section lengths in an explicit scheme will follow in a separate paper.

The representative depth and width of each section in terms of the chart datum were obtained from a hydrographic field sheet by overlaying the soundings with a transparent grid and tabulating the average sounding per square. The depth was calculated by dividing the sum of these average soundings by the total number of squares and the width followed from division of the surface area (i.e. the total number of squares multiplied by a scale factor) by the section length. To facilitate calculations of the cross-sectional area, the depth was adjusted in terms of the geodetic datum. In the model, geodetic datum is used as a reference level for tidal heights because it remains constant throughout the system, while Chart Datum changes at Gorge Narrows (page 4).

The section width B at chart datum is assumed to be the width of the conveyance channel. The schematization includes shoals by allowing the section width to increase to a maximum value at a level determined from a hydrographic chart.

To avoid abrupt and unrealistic changes in cross-sectional areas, the dimensions of the sections were smoothed out as in figure 6, which also shows the notation used in the difference equations.

Referring to figure 5, GB, B, BW, BMAX and CD were all taken from the chart. They characterize each section and are part of the data input to the computer program.

THE FINITE-DIFFERENCE EQUATIONS

Following standard procedures[7], the first derivatives in these equations are approximated by central differences, e.g. $\frac{\partial h}{\partial x} = \frac{H^k_{m+1} - H^k_{m-1}}{2\Delta x}$,

the truncation error being a function of $(\Delta x)^2$. k and m indicate the time and distance steps in the original matrix in figure 3, before the matrix is modified to save storage space (fig. 4). For the sake of clarity, the following derivation of the difference equation will refer to figure 3.

The equation of continuity is expressed in finite differences:

$$\frac{Q^{k-1}_{m+2} - Q^{k-1}_{m}}{\Delta x_m + \Delta x_{m+1}} + W^{k-1}_{m+1} \cdot \frac{H^{k}_{m+1} - H^{k-2}_{m+1}}{2\Delta t} = 0.$$

The first sections are relatively long (fig. 2) and might make the term $\frac{\partial Q}{\partial x}$ inaccurate in finite-difference form. Therefore, the equation of motion is rewritten as follows:

$\frac{\partial Q}{\partial x}$ is replaced by $-W\frac{\partial h}{\partial t}$ (from continuity).

Subsequently putting $\frac{\partial A}{\partial t} = \frac{\partial A}{\partial h} \cdot \frac{\partial h}{\partial t}$, we have:

$$\frac{1}{gA}\frac{\partial Q}{\partial t} - \left(\frac{\partial A}{\partial h} + W\right)\frac{Q}{gA^2}\frac{\partial h}{\partial t} - \frac{Q^2}{gA^3}\frac{\partial A}{\partial x} = - \frac{\partial h}{\partial x} - \frac{Q|Q|}{C^2A^2d}.$$

The term $\frac{\partial A}{\partial h}$, the change in cross-sectional area in terms of the water surface elevation was entered in the computer program as B, the width of the conveyance channel. This substitution assumes that all motion in x-direction occurs in the conveyance channel (see figure 5), a simplification which might not hold in an estuary with large shoaling areas.

"W" accounts for shoals and is programmed for three conditions:

a) The water level is below chart datum;
b) The shoals are partly flooded;
c) The shoals are completely flooded (the latter case is sketched in figure 5).

The difference equation of motion can now be formulated as:

$$\frac{1}{gA_m^k}\,\frac{Q_m^{k+1}-Q_m^{k-1}}{2\Delta t} - (B+W)_m^k\,\frac{Q_m^{k+1}}{g(A_m^k)^2}\cdot\frac{(H_{m+1}^k + H_{m-1}^k)-(H_{m+1}^{k-2}+H_{m-1}^{k-2})}{4\Delta t}$$

$$-\frac{Q_m^{k-1}\,Q_m^{k+1}}{g(A_m^k)^3}\cdot\left(\frac{A_{m+1}^k - A_{m-1}^k}{\Delta x_m + \Delta x_{m-1}}\right) = -\frac{H_{m+1}^k - H_{m-1}^k}{\Delta x_m + \Delta x_{m-1}} - \frac{Q_m^{k+1}\,|Q_m^{k-1}|}{(C_m)^2(A_m^k)^2 D_m^k}$$

where Q_m^{k+1} is the only unknown term since all others pertain to previous time steps or to data input. The non-linear terms Q^2 have been linearized by using the approximation $Q_m^{k-1}\,Q_m^{k+1}$. Note that the de Chezy coefficient C may vary with each section.

With the two finite-difference equations, we can compute H at the odd points in x-direction and at the even points in t-direction; and Q at the even points in x-direction and at the odd points in t-direction. This "leap-frog" method is illustrated by figure 3 (top), which also suggests a more efficient use of the available computer memory. To save storage space, the conventional matrix is compressed (figure 3 bottom) and the rows are relabelled (by eliminating the rows with subscripts (k + 2i + 1) and reassigning the (k + 2i) rows with numbers (n + i + 1), where i is any integer).

Using the notation of figure 4, we can now write the equations of continuity and motion in their final form:

Continuity:

$$H^{n+1}_{m+1} = H^{n}_{m+1} - \left\{4\Delta t.(Q^{n}_{m+2} - Q^{n}_{m})\right\} . \left\{(W^{n}_{m+1} + W^{n}_{m+2}) . (\Delta x_{m} + \Delta x_{m+1})\right\}^{-1} .$$

Note that $\frac{1}{2}(W_{m+1} + W_{m+2})$ represents the modified width at section line (m+1) shown in figure 6.

Motion:

$$Q^{n+1}_{m} = \left\{\frac{\Delta x_{m} + \Delta x_{m-1}}{g.\Delta t.(A^{n+1}_{m} + A^{n+1}_{m+1})} Q^{n}_{m} - (H^{n+1}_{m+1} - H^{n+1}_{m-1})\right\} .$$

$$\left\{\frac{\Delta x_{m} + \Delta x_{m-1}}{g.\Delta t.(A^{n+1}_{m} + A^{n+1}_{m+1})} + \frac{(\Delta x_{m} + \Delta x_{m-1}) .|Q^{n}_{m}|}{C^{2}_{m}.\left(\frac{A^{n+1}_{m} + A^{n+1}_{m+1}}{2}\right)^{2} . \left(GB_{m} + \frac{H^{n+1}_{m+1} + H^{n+1}_{m-1}}{2}\right)} -\right.$$

$$- \frac{Q^{n}_{m}.\left\{(A^{n+1}_{m+1} + A^{n+1}_{m+2}) - (A^{n+1}_{m} + A^{n+1}_{m-1})\right\}}{g . \left(\frac{A^{n+1}_{m} + A^{n+1}_{m+1}}{4}\right)^{3}} -$$

$$- \frac{(\Delta x_{m} + \Delta x_{m-1}) . (B^{n+1}_{m} + W^{n+1}_{m} + B^{n+1}_{m+1} + W^{n+1}_{m+1})}{8g.\Delta t. \left(\frac{A^{n+1}_{m} + A^{n+1}_{m+1}}{2}\right)^{2}} .$$

$$\left. . \left[(H^{n+1}_{m+1} + H^{n+1}_{m-1}) - (H^{n}_{m+1} + H^{n}_{m-1})\right] \right\}^{-1} .$$

BOUNDARY CONDITIONS

The boundary conditions of the equations are the observed tides at the harbour entrance (section line 1, fig. 2) and zero discharge at the head of Portage Inlet (section line 42).

At the harbour entrance, tidal records were obtained from an Ottboro tide gauge which was installed at Ogden Point in the spring of 1971 and maintained for several weeks. Reference to a geodetic benchmark (#737-J at the foot of Broughton Street) was established.

INITIAL CONDITIONS

The estuary is initially considered to be in equilibrium, i.e. Q_m and H_{m+1} are both zero at $t = o$. The effects of an inaccurate estimate of the initial tidal heights and discharges disappear quickly.

STABILITY AND THE TIME STEP

An essential condition for the successful functioning of an explicit scheme is its stability. Numerical errors introduced by rewriting the differential equations in finite-differences should not progressively amplify.

The stability requirement has been investigated in detail by Leendertse.[8] For a one-dimensional explicit scheme, the criterion for unconditional stability is found to be

$$\frac{\Delta x}{\Delta t} \geqslant C;$$

C is the velocity of propagation of a tidal wave. We set $C = \sqrt{gh}$, where h is the greatest water depth in the system.

Since the (minimum) section length Δx had already been established by the schematization, the stability depended on the choice of the time step. To find the optimum value of Δt, the model was run with Δt varying between 40 seconds and 5 seconds.

A time step of 10 seconds was finally selected to satisfy the stability criterion.

It should be emphasized that the interval $2\Delta t$ = 20 seconds in the modified computer scheme (fig. 4) applies to the time between two consecutive computations of H <u>or</u> Q. Compressing the matrix as shown in figure 4 only affects the notation, not the leap-frog method! For instance, Q_m^{n+1} still occurs one time step later than H_{m-1}^{n+1} or H_{m+1}^{n+1}.

THE COMPUTER PROGRAM

The program was written in FORTRAN and executed on a teletype terminal to the UNIVAC 1108 machine operated by Computer Sciences of Canada at Calgary, Alberta. Plotting routines were carried out on a CALCOMP 563 plotter interfaced with a Hewlett-Packard 2116B computer (16k). The flow chart for the program is shown in figure 23.

CALIBRATION OF THE MODEL: THE FRICTION COEFFICIENT

The model was verified by comparison of the computed tidal heights and currents with observed values recorded at a number of stations along the estuary. The friction coefficients C for all sections were then adjusted until the model reproduced the recorded data as closely as possible for the corresponding boundary conditions. The model was run for a large tide (June 9 and 10, 1971, see figure 8) and calibrated with tidal records. After calibration, it was tested with current observations at a number of locations and for different dates.

Figure 9 illustrates how the friction coefficients were tuned by comparing preliminary teletype plots of model-generated and observed tides. These tides were plotted simultaneously for two stations and for different values of C. The right-hand output obviously reflects a better choice of C than the left-hand output, particularly for the "Porters" tides.

The calibration was continued in this fashion until the model output and the actual tidal data agreed within 0.2 feet at all gauge stations (typical discrepancies were 0.1 feet). Finer tuning of the friction coefficient might well have produced a higher precision. However, this refinement would have involved much costly computer time, and was not warranted for the purposes of the model.

The final values of the friction coefficients varied from 20 $ft.^{1/2}$/sec for the very shallow Portage Inlet to 50 $ft.^{1/2}$/sec for Victoria Harbour. The low friction coefficient (high friction term) in the upper basin is not surprising when one considers obstructions such as the heavily trestled Craigflower Bridge and the abundant marine vegetation in Portage Inlet.

The coefficient C in the friction term $g\frac{u^2}{C^2 d}$ is referred to in this report as the "de Chezy" coefficient to conform with the literature on estuary modelling. However, it is a misnomer. The de Chezy coefficient originates in river hydraulics and depends mainly on the nature of the bed material. It is often expressed as $C = \frac{1.49}{n} R^{1/6}$ (9), where n is an empirical factor for bed material and R the hydraulic radius. This formula strictly considers the roughness of the boundary materials. The model's friction coefficient also includes the effect of bridge framework, pilings, logbooms etc. upon the water movement and may be much lower than the conventional de Chezy coefficient.

After calibration with vertical tides, the model was tested by current measurements for different tidal cycles.

Figures 10 and 11 show comparisons between computed and observed flows at two bridges, respectively in the lower and upper basin. The "observed discharges" were obtained by multiplication of the mean of several point measurements by the estimated cross-sectional area. There seems to be closer agreement between computer output and field data at Craigflower Bridge than at Johnson Street Bridge. This difference may be due to a better estimate of the mean current at Craigflower Bridge, where the flow is transversely much more uniform. Both model output and field data show large fluctuations in the flow at Johnson Street Bridge, which will be discussed later.

PREDICTED EFFECT OF A DAM UPON THE TIDES IN VICTORIA HARBOUR

The site of the proposed dam is assumed to be just west of the model's section line 10 (fig. 2). Being even-numbered, this line corresponds with grid points where discharges (Q) are calculated in the leap-frog scheme.

The effect of the dam on the Victoria tides can be evaluated by restricting the model to the first nine sections and setting a new "upstream" boundary condition Q = o at section line 10.

Using the previously estimated friction coefficients, the model can now be run again and its output at a section line in the harbour compared with that of the original model without a dam.

Figure 12 compares the model-produced tidal heights at section line 5 without and with dam. Although the two outputs do not differ in height or phase, the dam seems to generate a continuous low-amplitude oscillation of about 30 minutes, which does not resemble the normal fluctuations caused by an inaccurate estimate of the initial conditions. The model-produced discharges at Johnson Street Bridge (section line 6) confirm this observation clearly, vid figure 13.

It might be argued that the oscillation is not merely a resonance phenomenon but is a direct result of a large variety of tidal fluctuations at the harbour entrance, i.e. the downstream boundary condition.

To examine this possibility, the observed tides at Ogden Point are replaced by a cosine function representing the M-2 tide at Victoria, followed by a jump discontinuity to a zero tide (fig. 14). When the dam is included in the model, the flow at Johnson Street Bridge generated by this function exhibits again a very distinct period of 29 minutes, which is even more pronounced after the cosine function is abruptly discontinued to produce a one-foot shock. The expression for harbour resonance $T = \frac{4L}{gh}$ (where L is the distance to the dam and T the resonance period) would produce a resonance period of 32 minutes, disregarding Raleigh's mouth correction.[10] Raleigh's correction, although it may not apply to a numerical model, would increase the period to 37 minutes.

Without the dam, the shock does not produce a distinct resonance. A high-frequency signal (T = 7 minutes) superimposed upon both outputs plotted in figure 14 is almost certainly due to the schematization. A change in section lengths eliminates this signal but alters neither period nor amplitude of the 29 minute oscillation generated by the dam.

The frequency components of the flow in Victoria Harbour can be identified more clearly with a spectral analysis, as illustrated in figures 15 and 16. The plots are Power Spectra of the model-produced discharges at Johnson Street Bridge for June 9, 1971 and March 3, 1968. The computer program used for this method was developed in 1970 by the Insitute of Oceanography at the University of British Columbia. The observed tides for these dates are the boundary conditions.

Without the dam, the two Spectra are reasonably similar, with peak frequencies near 1.5, 4 and 5 cycles per hour. When the dam is included in the model, both plots show a very distinct peak frequency of 2.1 cycles per hour (T = 28.6 minutes).

Figure 17 is a similar spectral analysis of the observed tides for the same dates, to compare input and output frequencies of the model. The spectrum for June 9, 1971 is not conclusive. However, the spectral analysis of the March 1968 observed tides agrees rather well with that of the model-produced discharges at Johnson Street Bridge. It should be pointed out that the June 1971 tides were taken from Ogden Point gauge records, at the harbour entrance, while the March 1968 data were obtained from the Victoria gauge which is well inside the harbour and more susceptible to resonance. The tidal records for March 3-5, 1968 were selected for spectral analysis because they show some interesting short-period seiches, which are clearly reflected in the model.

To examine the harbour resonance more closely, two current meters were anchored under the Johnson Street Bridge, just outside the navigation channel. They recorded currents between the 3rd and 10th of November, 1971. One current meter, a Geodyne, had a sampling interval of four seconds (averages over 160 seconds) and the other, an Anderaa, had a ten minute sampling interval.

Without aliasing, the Anderaa's records would barely detect the 20 to 40 minute harbour resonance periods. The Anderaa was therefore mainly used to check the performance of the much more sophisticated Geodyne current meter.

A spectral analysis of the Geodyne data is shown in figure 18. The spectrum appears different from those in figures 15 to 17 because it is derived from a program developed by the Geodyne manufacturers which does not show confidence intervals. The most conspicuous peak frequency of 1.85 cycles per hour (a period of 32 minutes) agrees closely with that found on the tidal records of March 3-5, 1968 (figure 17). The other peaks agree reasonably well, although the tidal records show peaks at the higher frequencies (4.1 and 5.2 cycles per hour), which the current meter did not seem to detect. It should be noted that the directional unit of the Geodyne failed almost immediately after the meter was put down. This mishap had no effect upon the spectral analysis since the flow reversals were quite distinct and could easily be reconstructed from the records of the Anderaa current meter. The spectral analysis of the Anderaa data shows distinct peak frequencies at 1.2 and 2.6 cycles per hour; which correspond to those of the Geodyne records.

The results of the model computations of the effect of the dam may then be summarized as follows:

There is sufficient evidence that the proposed dam will enhance a seiche with a period of 29 minutes. Under normal tidal conditions, this seiche will hardly be noticeable. However, a seiche induced by the passage of a weather system or perhaps an earth tremor will generate stronger harbour currents and continue to oscillate much longer than under the same conditions without a dam. For instance, in both cases a one-foot shock at Ogden Point will set off a flood current near Johnson Street Bridge with a peak of about four knots. When the dam is included, the current will continue to oscillate for several hours. It will decrease to about one-half knot in both directions after five hours. However, without the dam the current will become virtually negligible after the first oscillation, since the energy of the tidal wave dissipates rapidly in the Gorge and Portage Inlet.

SOME ASPECTS OF THE PROPOSED PORTAGE CANAL

Although the construction of a canal between Portage Inlet and Esquimalt Harbour seems much less practicable than the building of a dam between the Upper Harbour and the Gorge[1], only minor program changes are needed to include the canal in the model and permit a cursory study of its effect upon the tides.

To connect Portage Inlet with Esquimalt Harbour, the canal would have to be about 2000 feet long. The schematization is therefore modified simply by extending the upstream portion of the model to Esquimalt Harbour with three sections, as figure 2 clarifies. The sections are each 670 feet long, 100 feet wide, and 10 feet deep with respect to geodetic datum. The tides in Esquimalt Harbour, assumed to be equal to those at Ogden Point now form the upstream boundary condition. The friction coefficient is set at 60 $ft.^{1/2}$/second.

The model examines the effect of the canal upon the water level in Portage Inlet, upon the currents in the Gorge and computes the flow in the canal itself.

Using the existing bottom configuration of Portage Inlet and Western Gorge, this version of the model fails: at a falling tide, the basin's outflow will not be restricted by the Gorge Narrows but will find an additional passage through the canal into Esquimalt Harbour. Several sections in Portage Inlet and Western Gorge will consequently dry up during part of the tidal cycle, causing the term Q/A in the equation of motion to approach infinity. However, the model works when the term GB (see figure 5) is increased to ten feet for all shallow sections in the upper basin, in other words, after some considerable dredging.

For a clear comparison of the current velocities in the Gorge Narrows, a cosine function (the M-2 tidal constituent for Victoria) is used as model input. The results are plotted in figure 20: the canal will reduce the maximum current velocity in the Gorge to one-half its present value, while the current in the canal itself will be about twice as strong as that in the Gorge (after construction of the canal). These figures are of a reconnaissance nature only, and are based on some broad assumptions.

Only a simple adjustment on the schematization enables the model to predict the currents in the canal if a dam is constructed between Gorge and Upper Harbour, in addition to the canal. This adjustment would omit the first nine sections, and set Q=o at section line 9. A boundary condition at Esquimalt Harbour of the observed Victoria tides for 9 - 10 June 1971 would induce a maximum current of three knots in both flood and ebb directions. An M-2 tide would induce a maximum current of two knots in either direction.

As a matter of interest, the model then considers the possibility of dredging the entire upper basin to a depth (GB) of ten feet below geodetic datum. Figure 21 illustrates the effect of this operation: with the observed tides at Ogden Point for 9 - 10 June 1971 as the downstream boundary condition and Q=o as the upstream boundary condition, the model predicts no change in the high water levels in Portage Inlet but a considerable drop of three feet of the low water levels. In other words, the chart datum of the upper basin would be lowered.

Figure 22 shows a considerable increase in discharge through the Gorge in both directions for the same conditions.

CONCLUSION

The construction of a dam below Gorge Narrows would amplify a seiche which normally is suppressed by the upper basin.

The proposed Portage Canal, unless regulated by locks, would, at low water, drain Portage Inlet and the Gorge into Esquimalt Harbour. To maintain circulation, the two waterways would have to be dredged. At falling tide, most of the upper basin would then discharge through the canal, reducing the ebb current in the Narrows to a rate which would make this passage navigable during all tidal phases.

REMARKS

To avoid discontinuity in the foregoing discussion, some significant approximations and assumptions in the model were dealt with only briefly. They will now be considered in more detail.

A unique feature of the tidal characteristics of the Victoria basin is the constriction at Gorge Narrows where at an outgoing tide the water level may drop more than four feet over a very short distance. During a preliminary observation of the flow in the Narrows at low-water spring tide, the average slope of the hydraulic grade line under the Gorge Bridge was estimated at 5%, with a correlative increase in current velocities from three to ten feet per second over a distance of less than 100 feet. Just east of the Gorge Bridge, the bottom slopes down steeply towards the harbour and the flow decelerates back to normal.

Mainly because of the considerable change in velocities in the Gorge Narrows, the convective acceleration $u \frac{\partial u}{\partial x}$ was included in the equation of motion. The term (also called the "Bernouilli" term) is often ignored in tidal computations of rivers and estuaries. To test its relative importance in the Victoria model, $u \frac{\partial u}{\partial x}$ was omitted for some typical boundary conditions. The results never differed by more than 3% from cases where the term was retained. The term is nevertheless maintained in the final program. Vertical accelerations and velocities are neglected since there was obviously no abrupt change in the water level.

The irregular bottom profile and strong turbulence in the transition zone made it difficult to establish conditions for critical flow. The Froude numbers computed from the available field data did not exceed 1. However, an extreme tidal range might create a hydraulic jump with a considerable loss of energy (a cubic function of the difference in water levels before and after the jump) and a discharge which depends on the critical depth ($Q = g^{1/2} b\, d_c^{3/2}$, where d_c = critical depth).

The equations of motion and continuity would still hold on both sides of the hydraulic jump but the transition zone should be considered separately with the vertical tides on each side as boundary conditions.

A detailed study of the Gorge Narrows would require two tide gauges at the Gorge bridge; they should preferably operate in December when a maximum tidal range may be expected. Currents and depths in the transition zone should be measured simultaneously.

In the equation of motion, the width of the conveyance channel is kept constant throughout a tidal cycle, while the height varies with the tides. In other words, all flow is assumed to pass through a rectangular cross section defined by this fixed width and variable height. No interchange of momentum is assumed to take place between the currents in the main channel and the currents to and from the flooding and drying regions, which are relatively small in the Victoria basin. The average width of the conveyance channel for each section was obtained from the chart by dividing the total water surface at chart datum by the section length.

Throughout a cross section, the flow is assumed to be uniformly distributed, both transversely and vertically, and the expression $Q = uA$ is based on this assumption when the velocity component u is replaced by the discharge Q in the equation of motion. However, preliminary current measurements at the Johnson Street Bridge over a grid of twenty feet (horizontal) by five feet (vertical) indicated that the surface current in the middle of the channel would be about twice as high as the mean flow for that particular cross section. Therefore, if the model would be required to predict surface currents (e.g. for pollution studies), additional field measurements must be made to relate surface currents to mean currents at selected sections.

In the canal program, the tidal input at section 45 was set equal to that at section 1, on the assumption that the tides in Esquimalt Harbour would be identical to those at Ogden Point. There would actually be a slight difference in range and phase between the tides in these two locations. A more comprehensive study of the feasibility of a canal would require an additional tide gauge at the head of Esquimalt Harbour.

When the dam was included in the model, the observed tides at the harbour entrance for the existing system (without dam) were entered as the seaward boundary condition. This approach ignored the effect which the dam might have upon the tides at the harbour entrance. It would be more accurate to establish a boundary condition outside the harbour entrance, for instance by using tidal data derived from a future numerical model of the Strait of Juan de Fuca.

The schematization of the model introduces a few refinements which have been discussed in detail. The computational technique follows a proven method which has been treated in the literature(5) and needs no further comment.

By its stepwise simulation of a tidal estuary, a one-dimensional numerical model may overlook features which a detailed physical model would detect; to minimize this possibility, the Victoria model was schematized with relatively small section lengths and time intervals. The report demonstrates how a variety of modifications in a flow regime can be examined by the same computer program with only minor adjustments. This flexibility is clearly an advantage of a mathematical model.

The results of the model analysis indicate that the harbour's tidal response will be an important factor in a future feasibility study of a dam in the Victoria basin. The need for such an investigation is further emphasized by the existence of harmful seiches in other harbours, e.g. Neah Bay (Washington), Los Angeles and Cape Town.(11) For instance, in certain parts of Los Angeles Harbour, ships have been damaged when they were surging and swaying as a result of horizontal oscillations.(12)

An approximate position of the proposed dam was used to compute the tidal response of the harbour. Once a decision has been made regarding the dam's exact location, the computer program can be adjusted accordingly.

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The programming and much of the schematization of the original model were carried out by Miss Monica McAleese. Additional computations to include the proposed hydraulic structures, plotter routines etc. were programmed by Miss Barbara White and Mr. Keith Lee. The manuscript and computer program were reviewed by Miss Anne Woollard.

The generous help is acknowledged of Mr. Syd Wigen, Tidal Superintendent of the Canadian Hydrographic Service, who suggested the project, and the close cooperation of his staff in collecting the field data.

Thanks are due to Dr. John Garrett and his colleagues in Ocean Physics for their many enlightening discussions, particularly regarding spectral analysis.

Finally, sincere gratitude is expressed to Dr. David Prandle of the National Research Council at Ottawa for his helpful suggestions in the development of the model.

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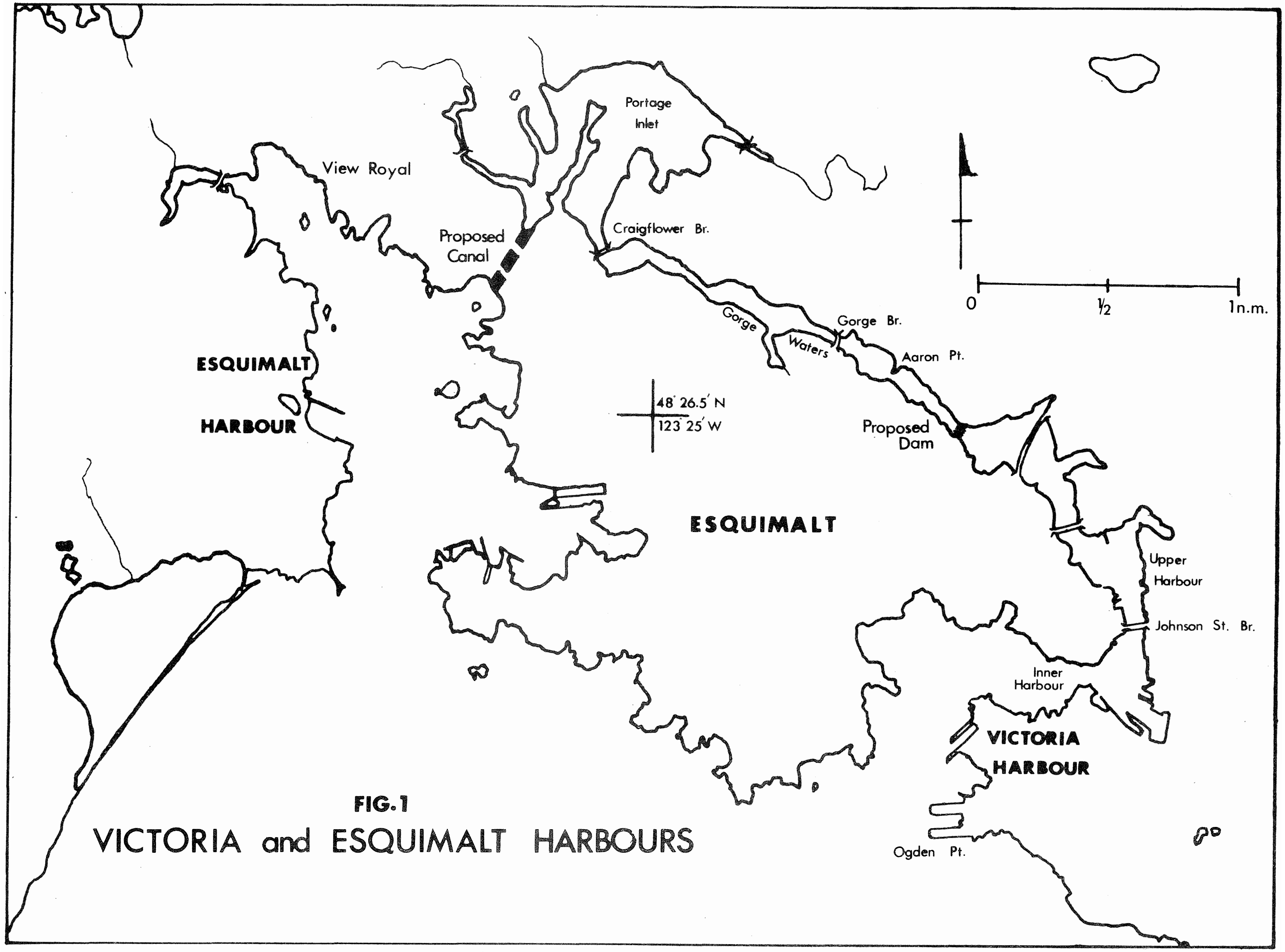


FIG. 1

VICTORIA and ESQUIMALT HARBOURS

FIG. 2

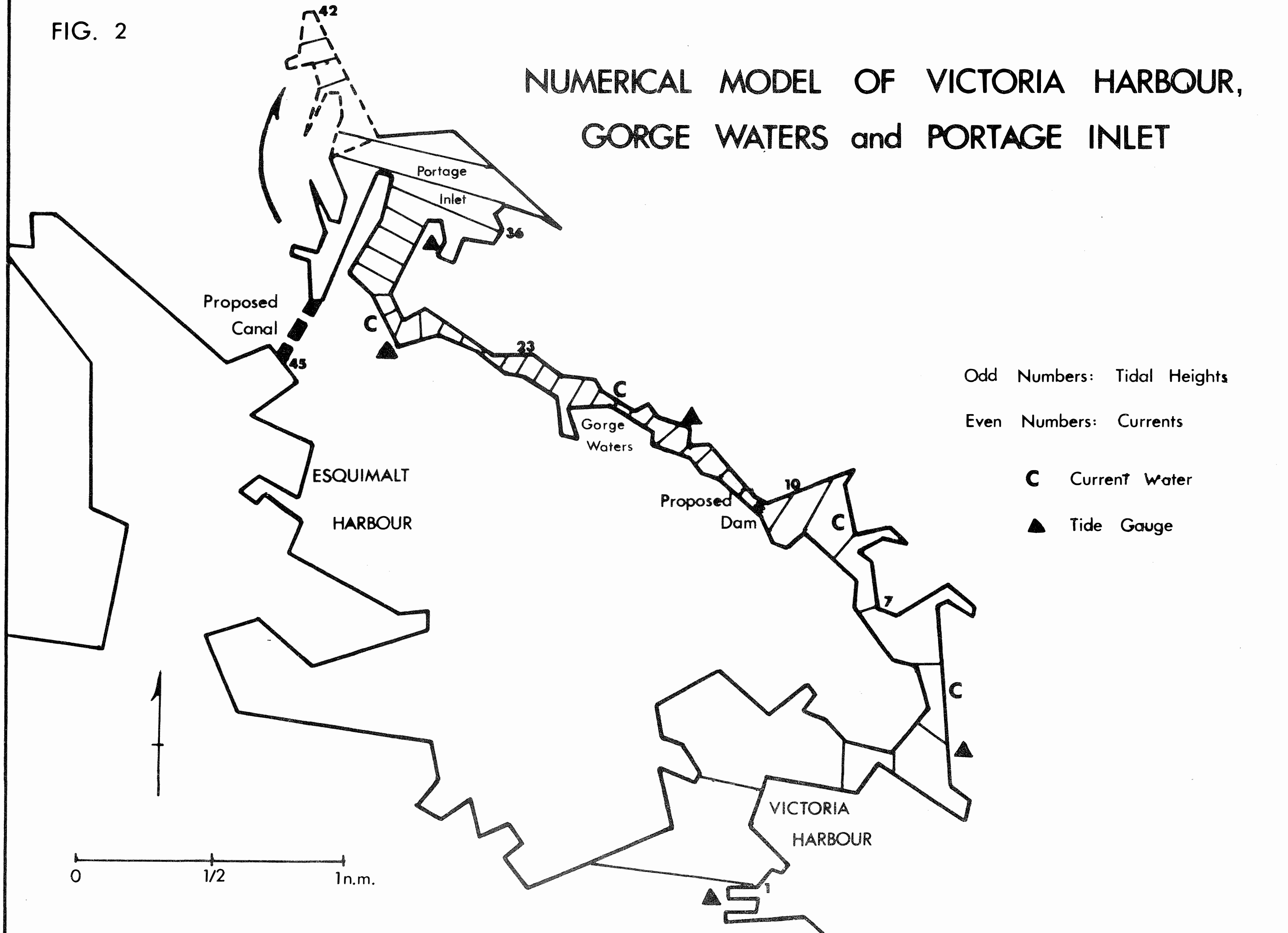
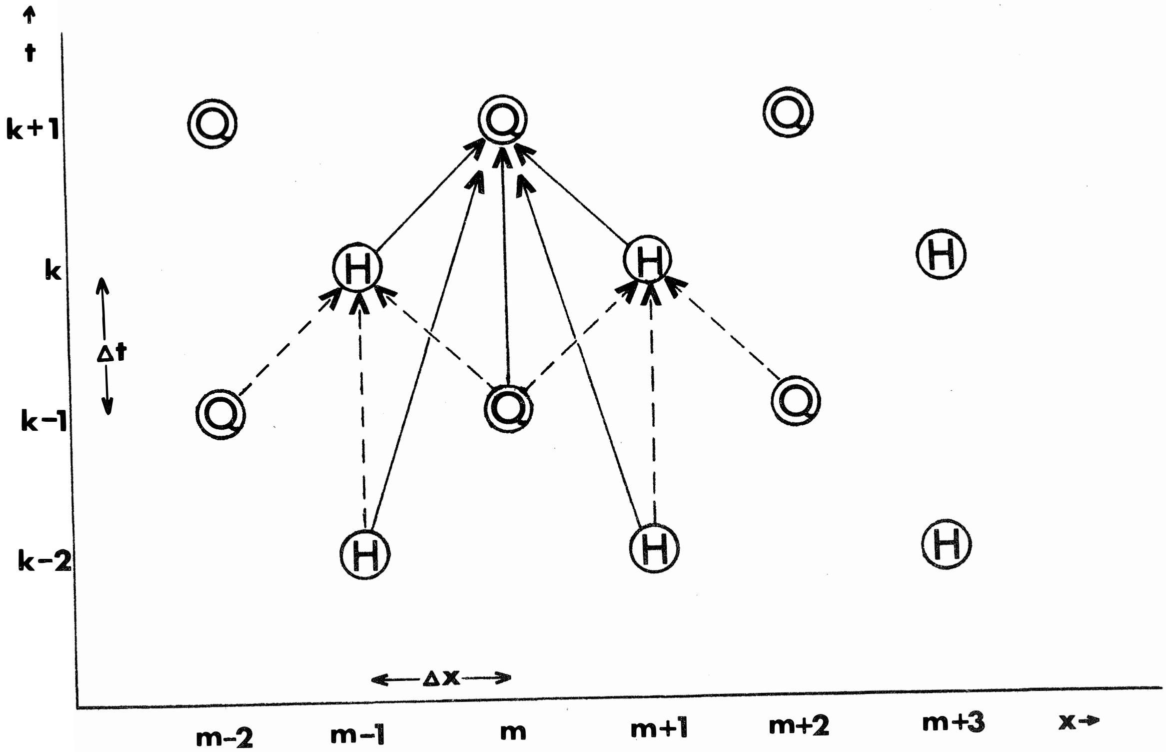

NUMERICAL MODEL OF VICTORIA HARBOUR,
GORGE WATERS and PORTAGE INLET
42
Portage
Inlet
36
Proposed
Canal
45
C
23
C
Gorge
Waters
Proposed
Dam
19
C
7
C
ESQUIMALT
HARBOUR
VICTORIA
HARBOUR
1
Odd Numbers: Tidal Heights
Even Numbers: Currents
C Current Water
Tide Gauge
0
1/2
1n.m.

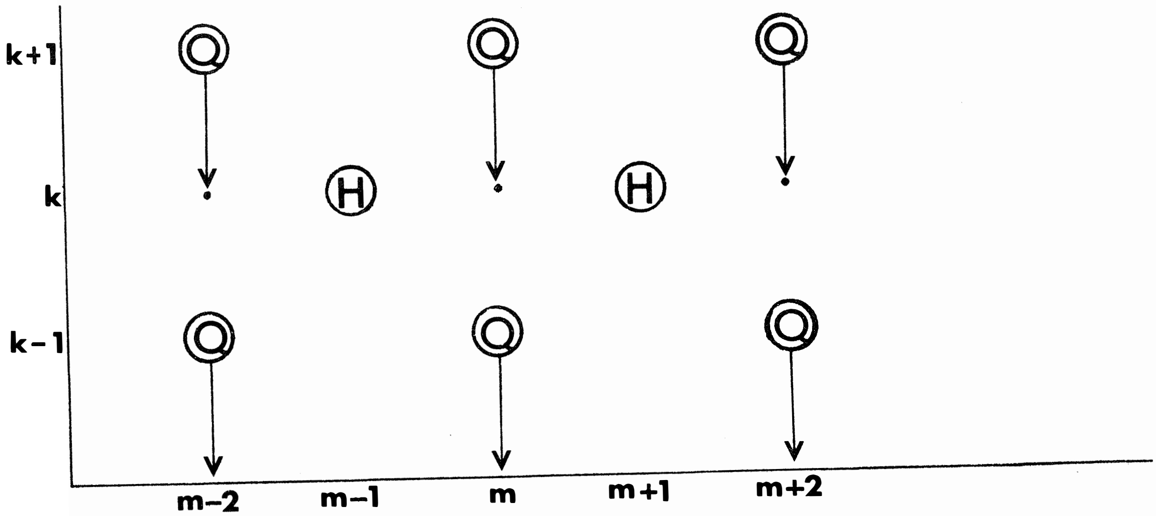
FIG. 3 **COMPUTATION SCHEME, EXPLICIT MODEL**



——————→ eqn. of motion

– – – – – – → eqn. of continuity

Compress Matrix by moving Q elements down to previous H-row



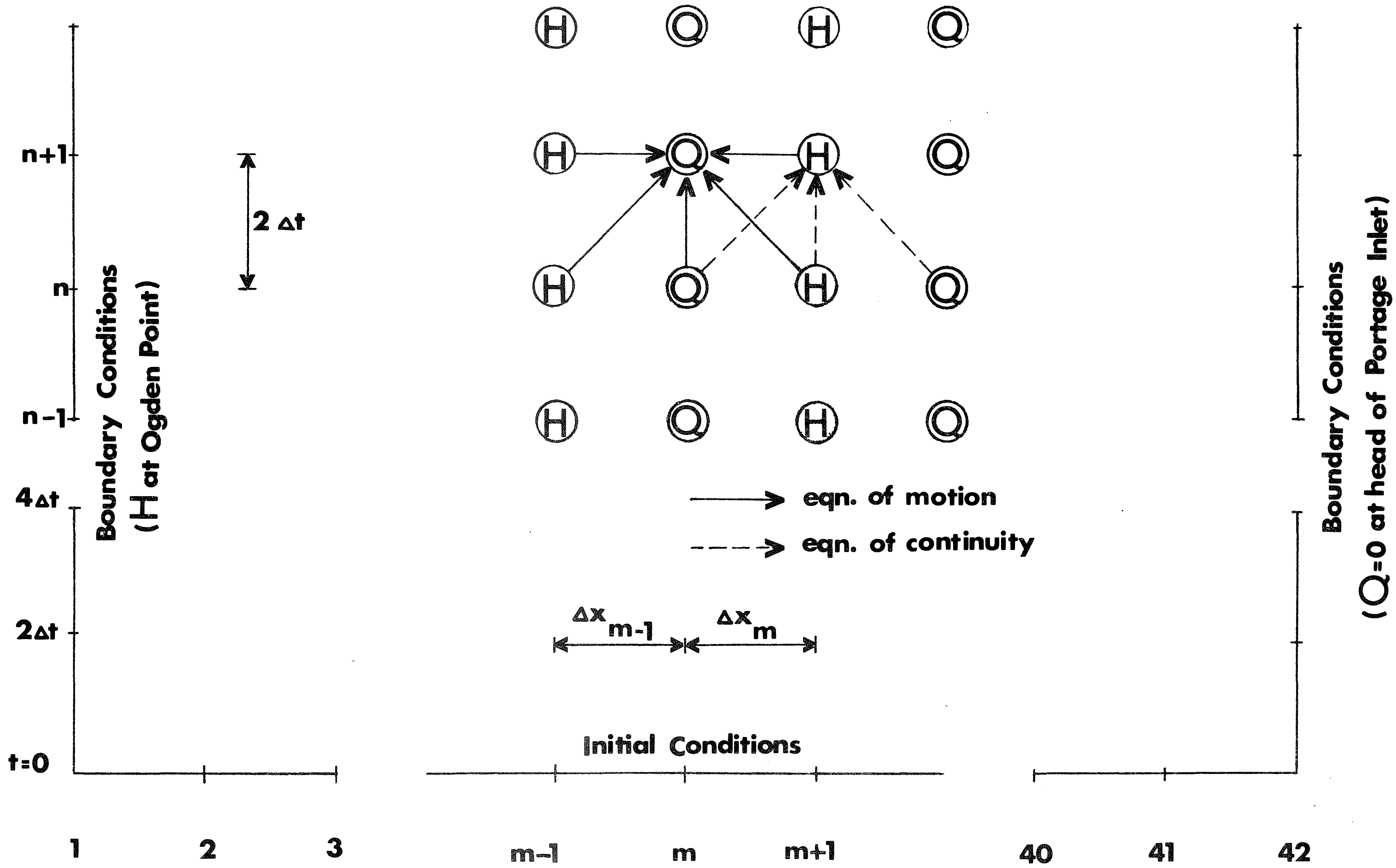
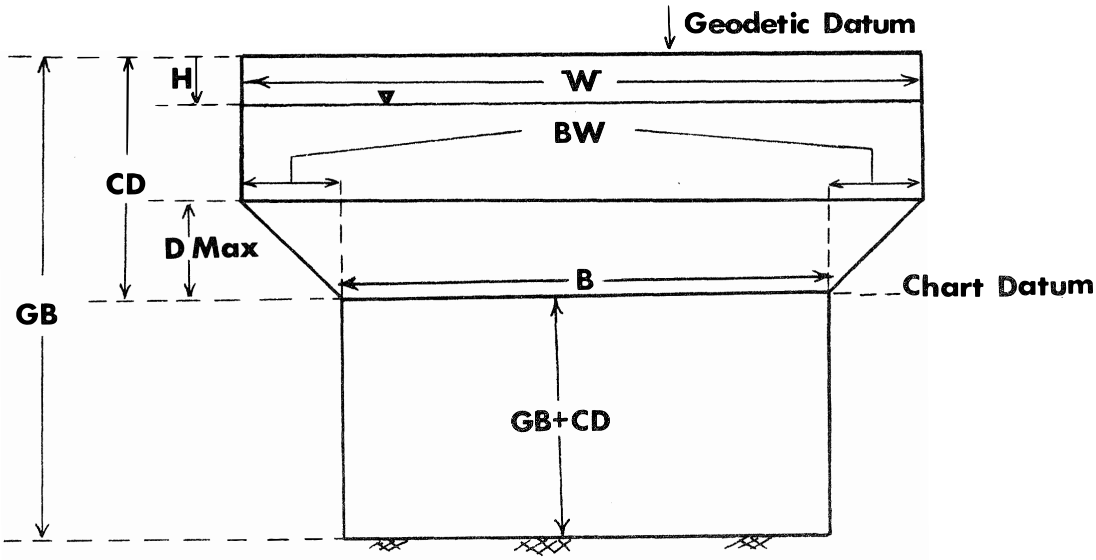


FIG.4 **MODIFIED COMPUTER SCHEME**

FIG. 5 **SCHEMATIZATION OF SECTIONS**



B – Mean width of a section at Chart Datum (fixed)

W – Mean width at time t (variable)

BW – Bank width (fixed)

GB – Distance between Geodetic Datum and bottom (fixed)

CD – Distance between Geodetic and Chart Datum (fixed)

D Max – Bank height (fixed)

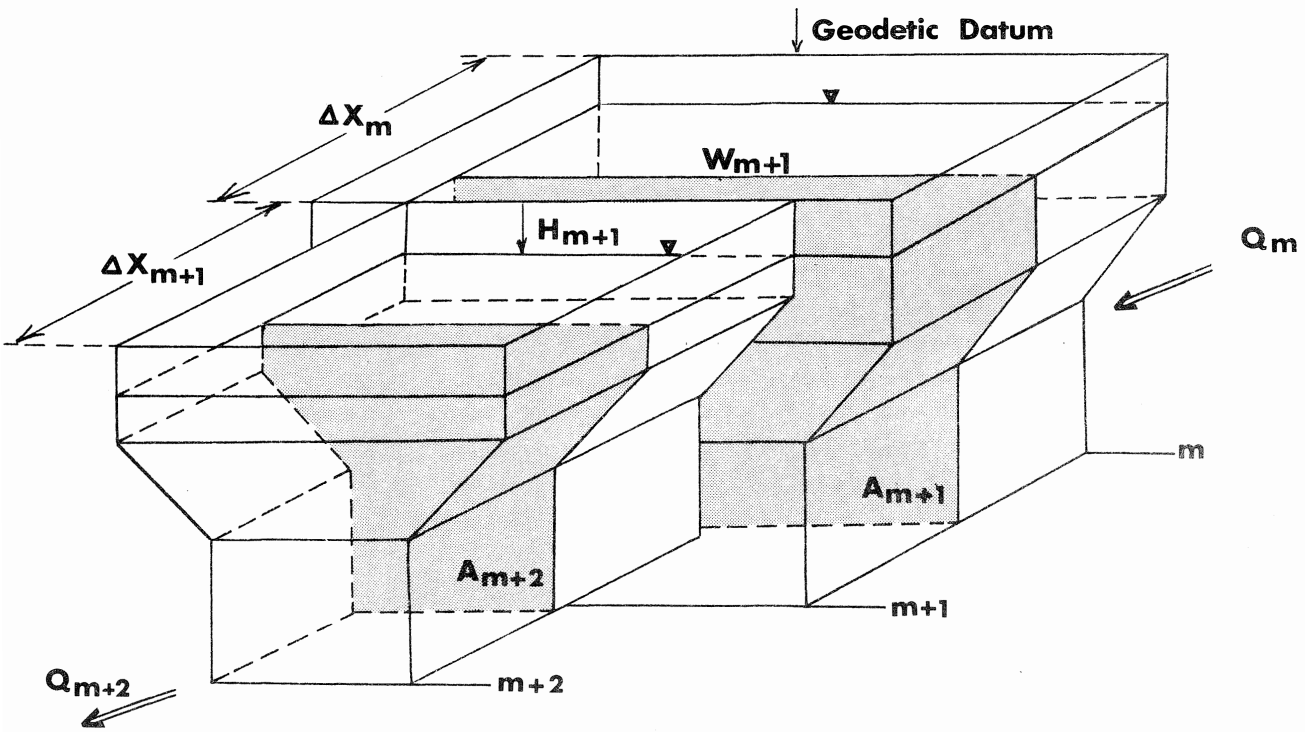
H – Distance between Geodetic Datum and water level (variable; negative in figure)

GB + CD – Depth below Chart Datum, obtained from hydrographic charts

CHART DATUM: East of Gorge (Section 17), Chart Datum is 6.16 feet below Geodetic Datum. West of Gorge, Chart Datum is 1.66 feet below Geodetic Datum.

FIG. 6 **SMOOTHING OF CROSS-SECTIONAL AREAS**

A) SCHEMATIZED SECTIONS:



B) MODIFIED M+1 SECTION:

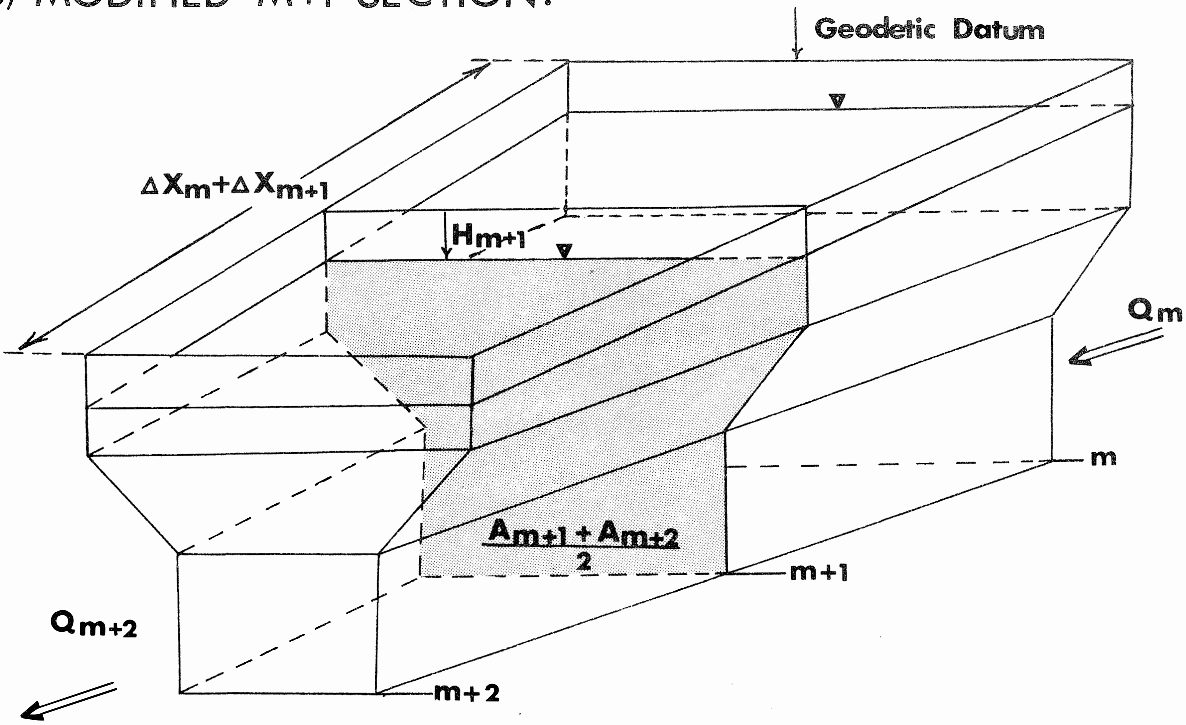
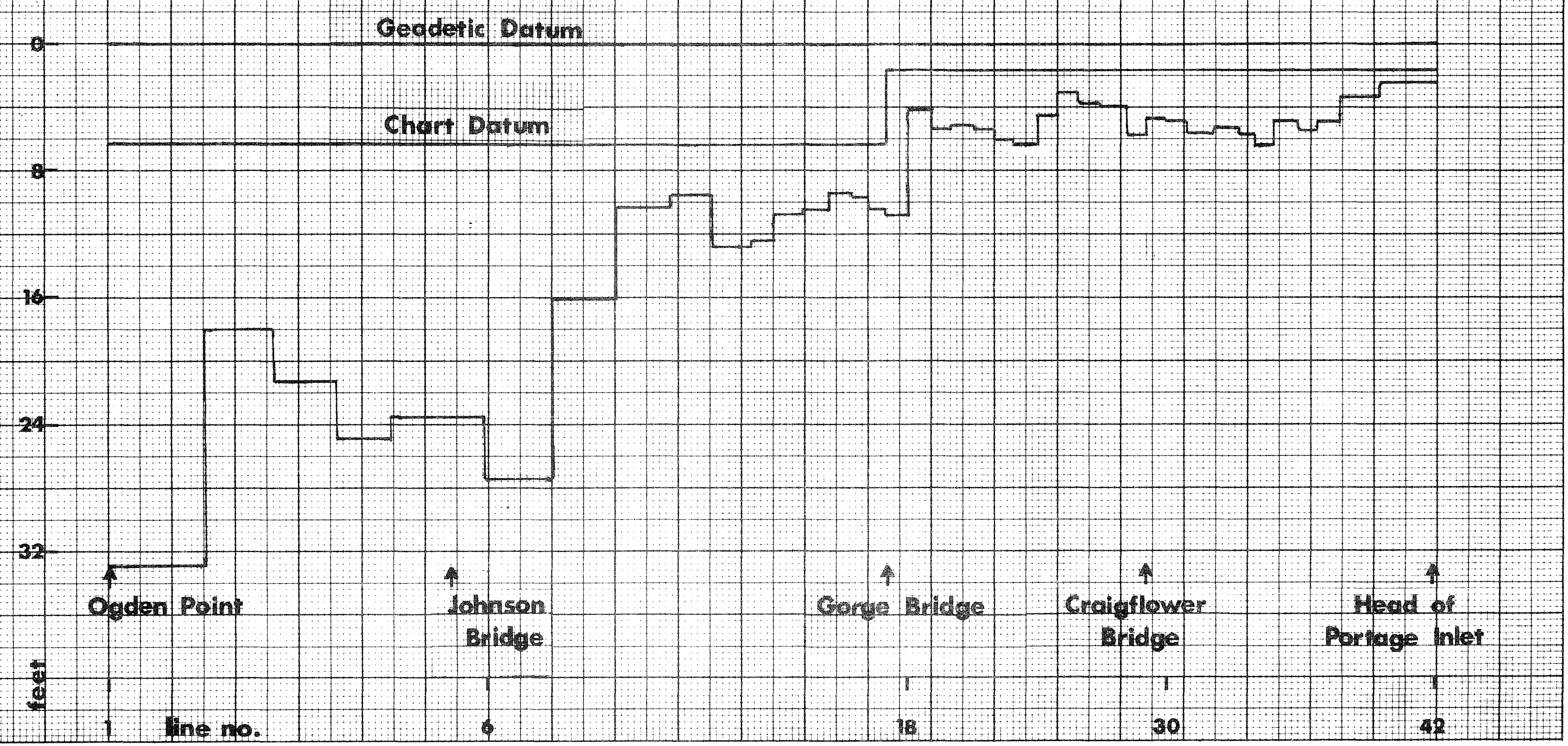


FIG. 7

VICTORIA HARBOUR TO PORTAGE INLET

LONGITUDINAL PROFILE DERIVED FROM SCHEMATIZED SECTIONS

Horizontal Scale: 1cm=1500 ft



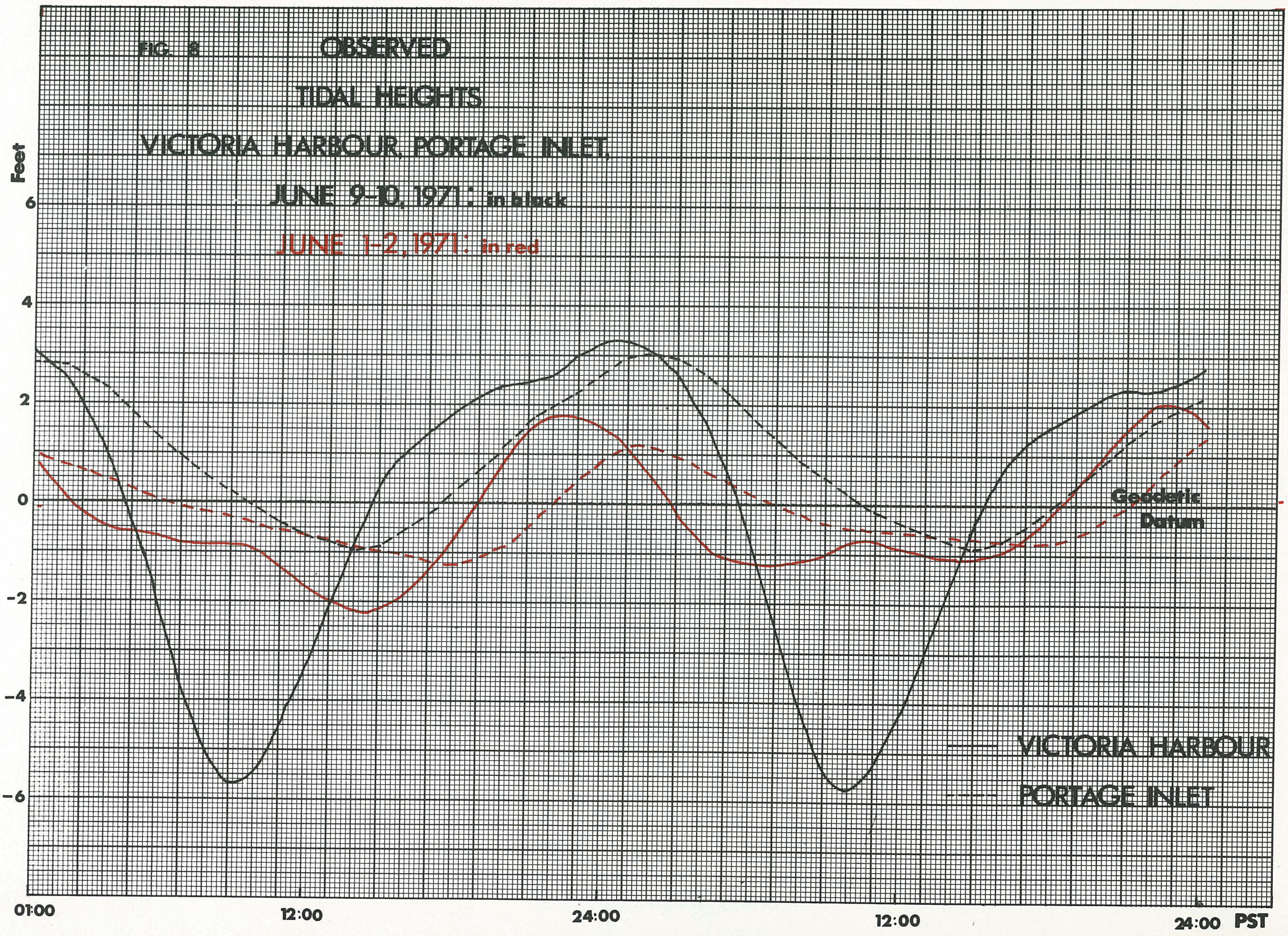


FIG. 8
OBSERVED
TIDAL HEIGHTS
VICTORIA HARBOUR, PORTAGE INLET,
JUNE 9-10, 1971: in black
JUNE 1-2, 1971: in red
Feet
6
4
2
0
-2
-4
-6
Geodetic
Datum
VICTORIA HARBOUR
PORTAGE INLET
01:00
12:00
24:00
12:00
24:00
PST

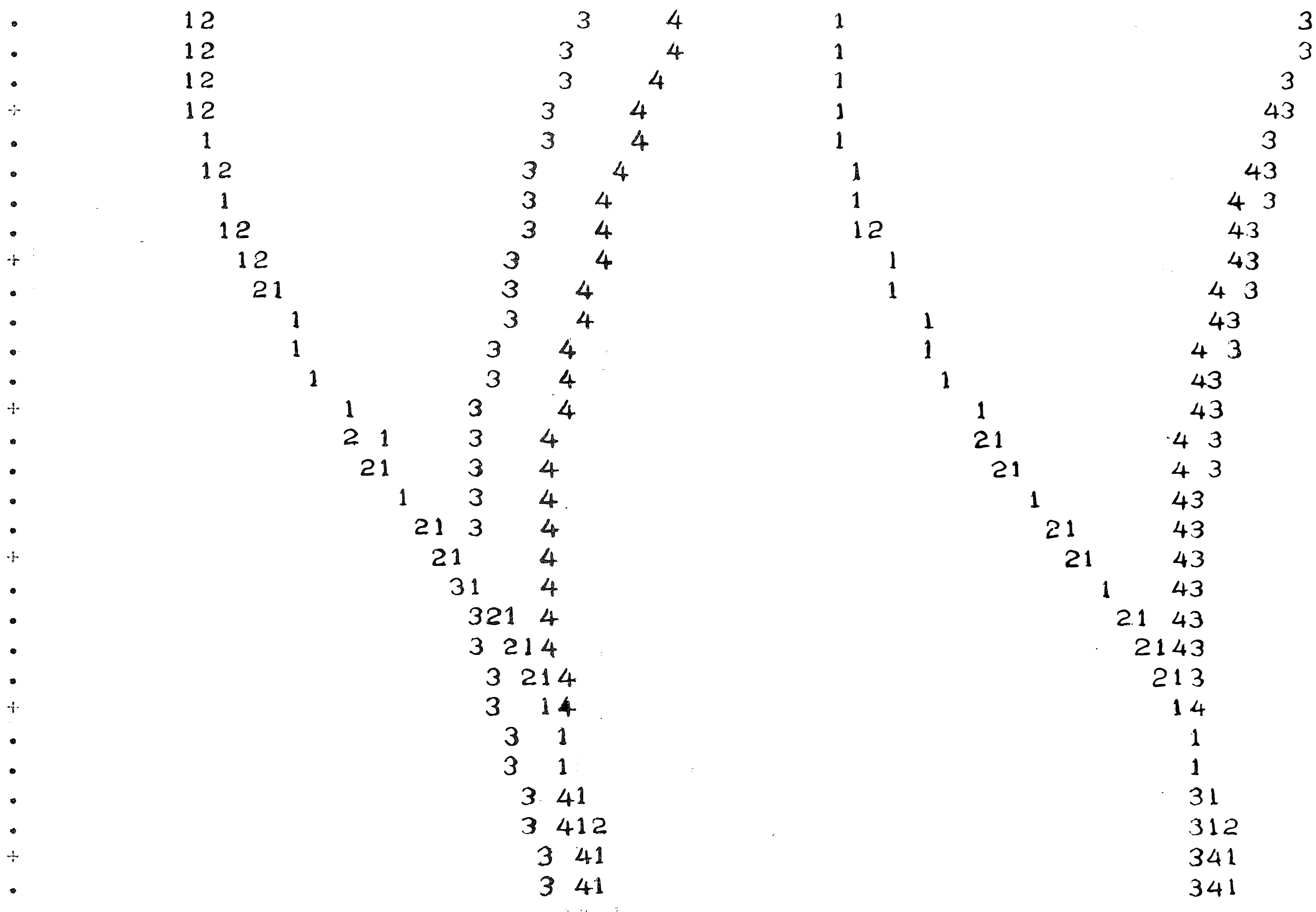


FIG. 9 DETERMINATION OF FRICTION COEFFICIENTS

PORTIONS OF TIDE GRAPHS

1. Aaron Point, model – generated
2. Aaron Point, observed
3. Porters, model – generated
4. Porters, observed

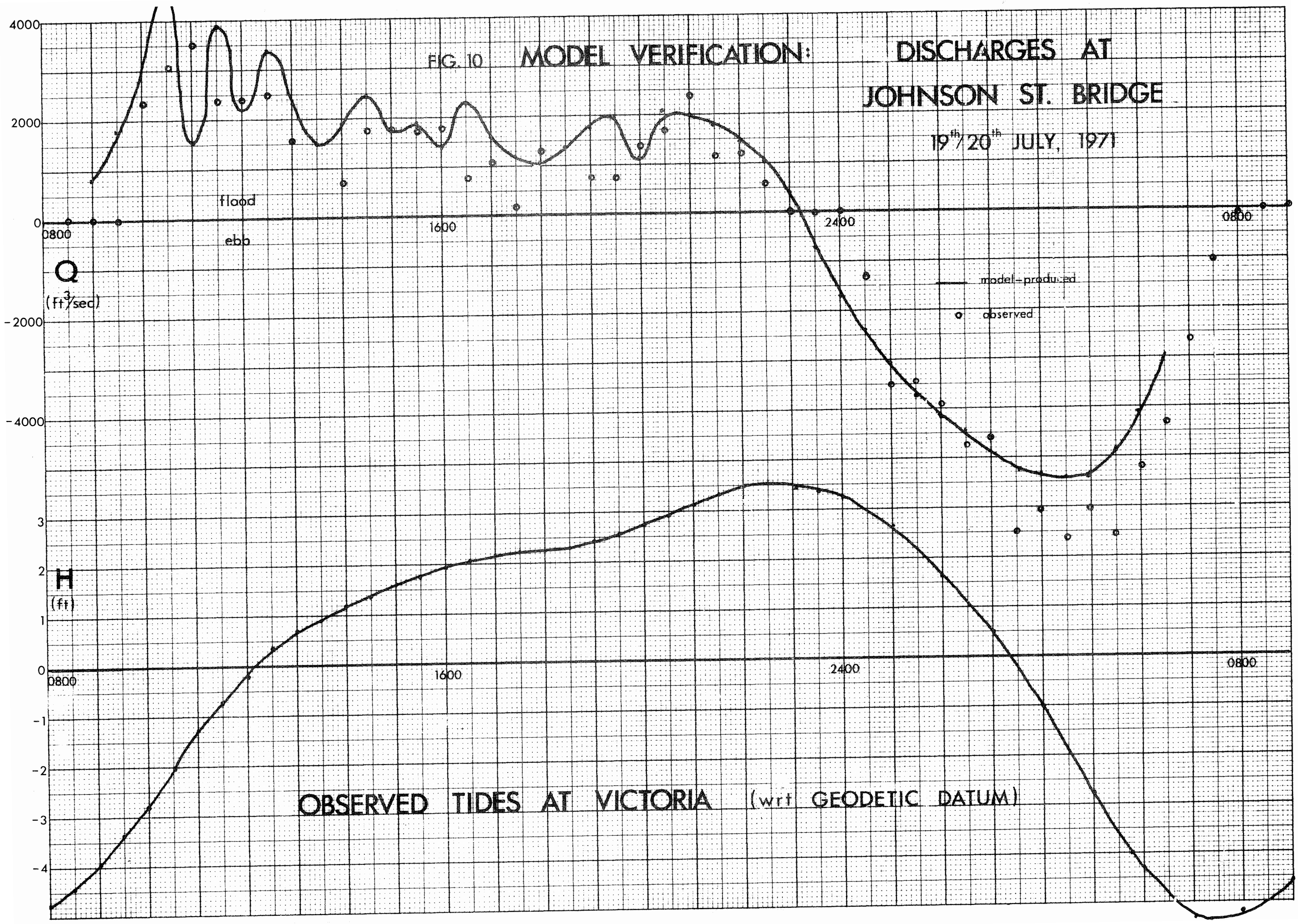


FIG. 10 MODEL VERIFICATION: DISCHARGES AT JOHNSON ST. BRIDGE
19th/20th JULY, 1971
Q
(ft³/sec)
4000
2000
0
-2000
-4000
flood
ebb
0800
1600
2400
0800
model-produced
observed
H
(ft)
3
2
1
0
-1
-2
-3
-4
0800
1600
2400
0800
OBSERVED TIDES AT VICTORIA (wrt GEODETIC DATUM)

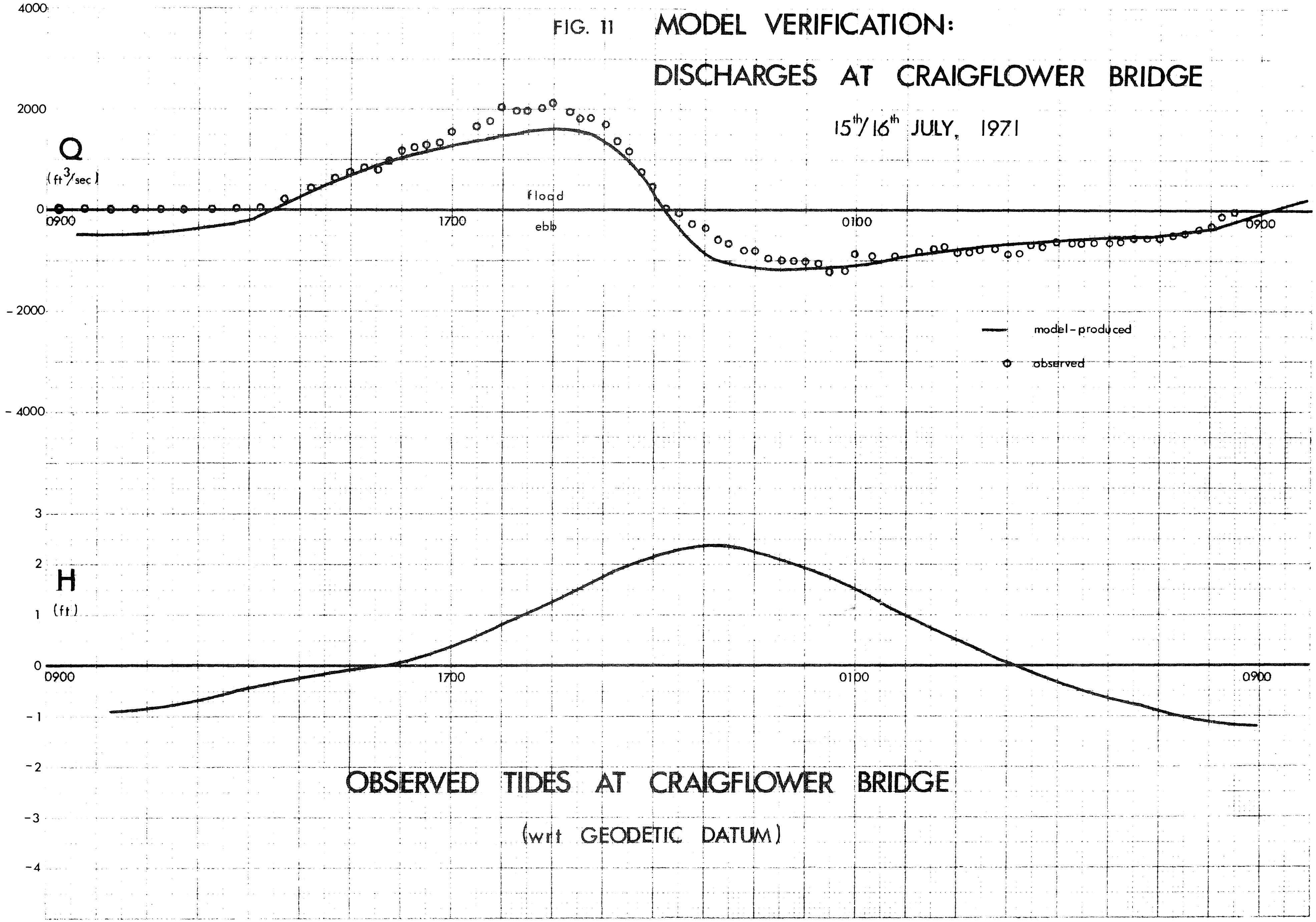


FIG. 11 MODEL VERIFICATION:
DISCHARGES AT CRAIGFLOWER BRIDGE
15th/16th JULY, 1971

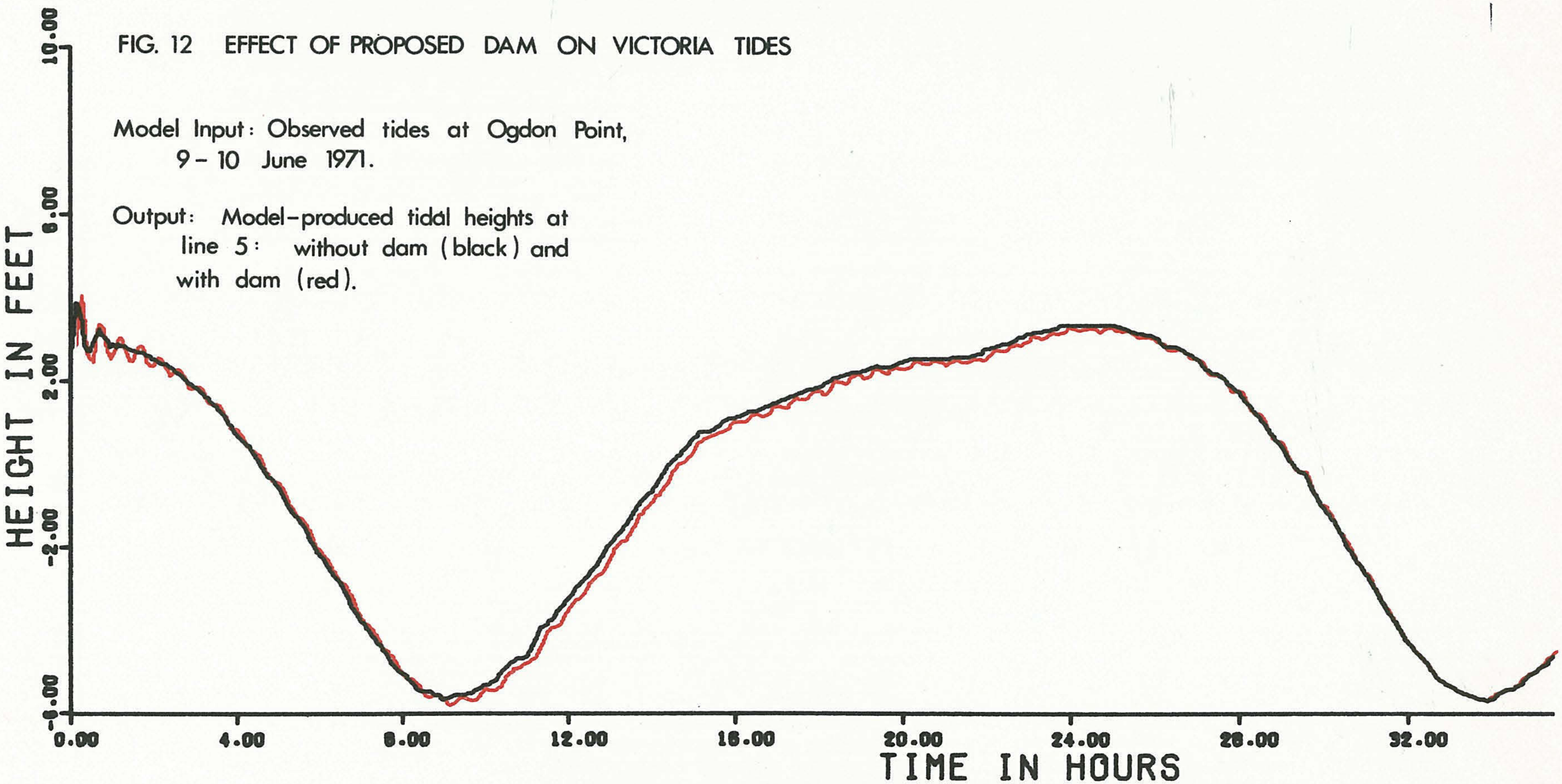


FIG. 12 EFFECT OF PROPOSED DAM ON VICTORIA TIDES

FIG. 13 EFFECT OF PROPOSED DAM ON TIDAL CURRENTS AT JOHNSON ST. BRIDGE

Input: Observed tides at Ogden Pt., June 9–10, 1971
Output: Model–produced discharges at Johnson Street Bridge.

Red: with dam
Black: without dam

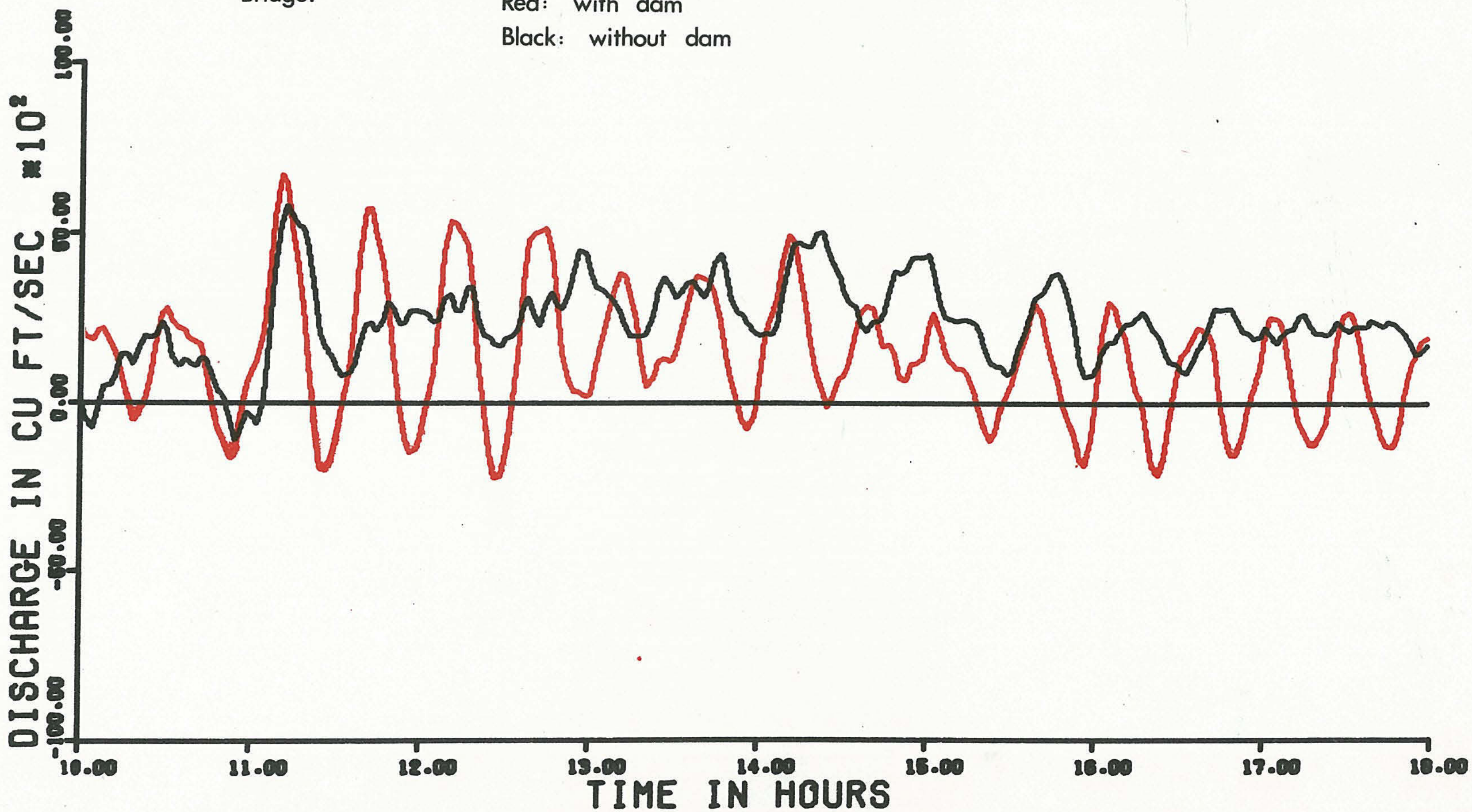


FIG. 14 EFFECT OF PROPOSED DAM ON TIDAL CURRENTS AT JOHNSON ST. BRIDGE

Input at section line 1:

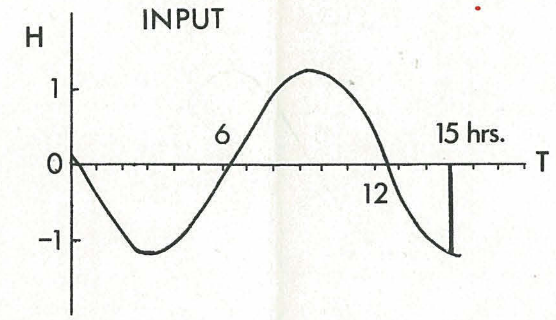
0–15 HRS: $H = 1.208 \cos\left(\frac{28.96}{3600} T + 84.9^\circ\right)$

after 15 HRS: $H = 0$

Output at section line 6:

Discharge with dam – red

Discharge without dam – black



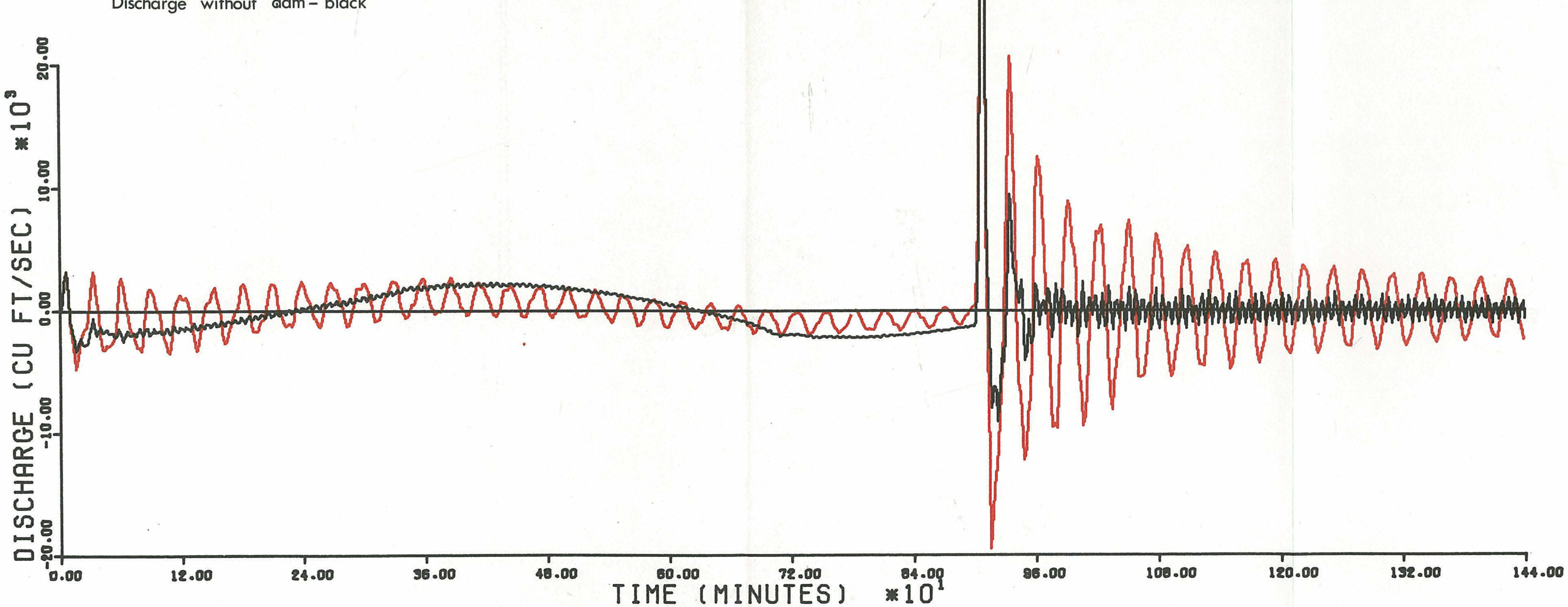


Fig. 15 Spectral analysis (Hanned FFT) **of model-produced discharges at Johnson St. Bridge (Discreet time series of 2048 data at 40 second interval)**

Input: Observed tidal heights at Ogden Point, June 9, 1971

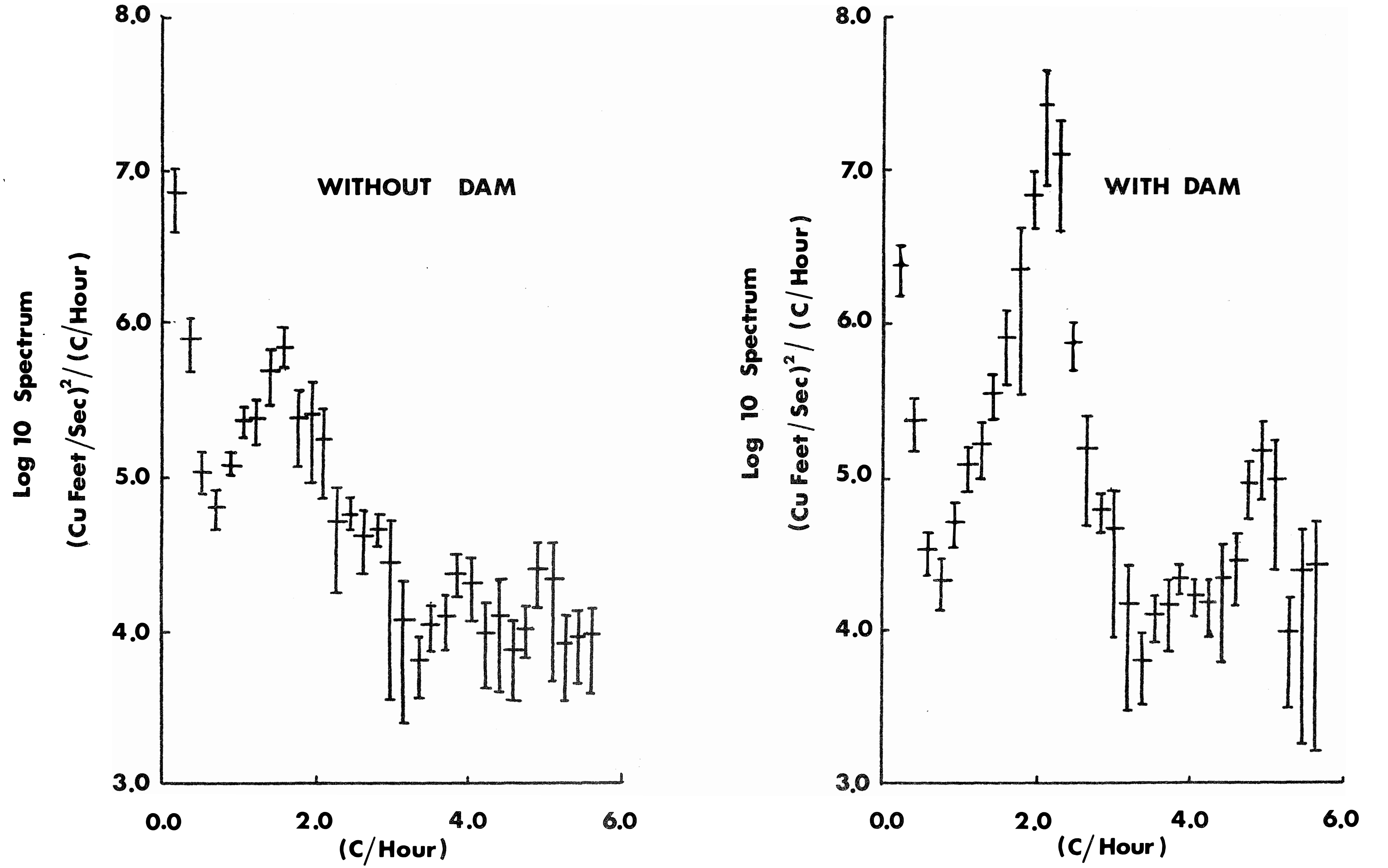


Fig. 16 Spectral analysis (Hanned FFT) **of model-produced discharges at Johnson St. Bridge (Discreet time series of 2048 data at 40 second intervals)**
Input: Observed tidal heights in Victoria harbour, March 3, 1968

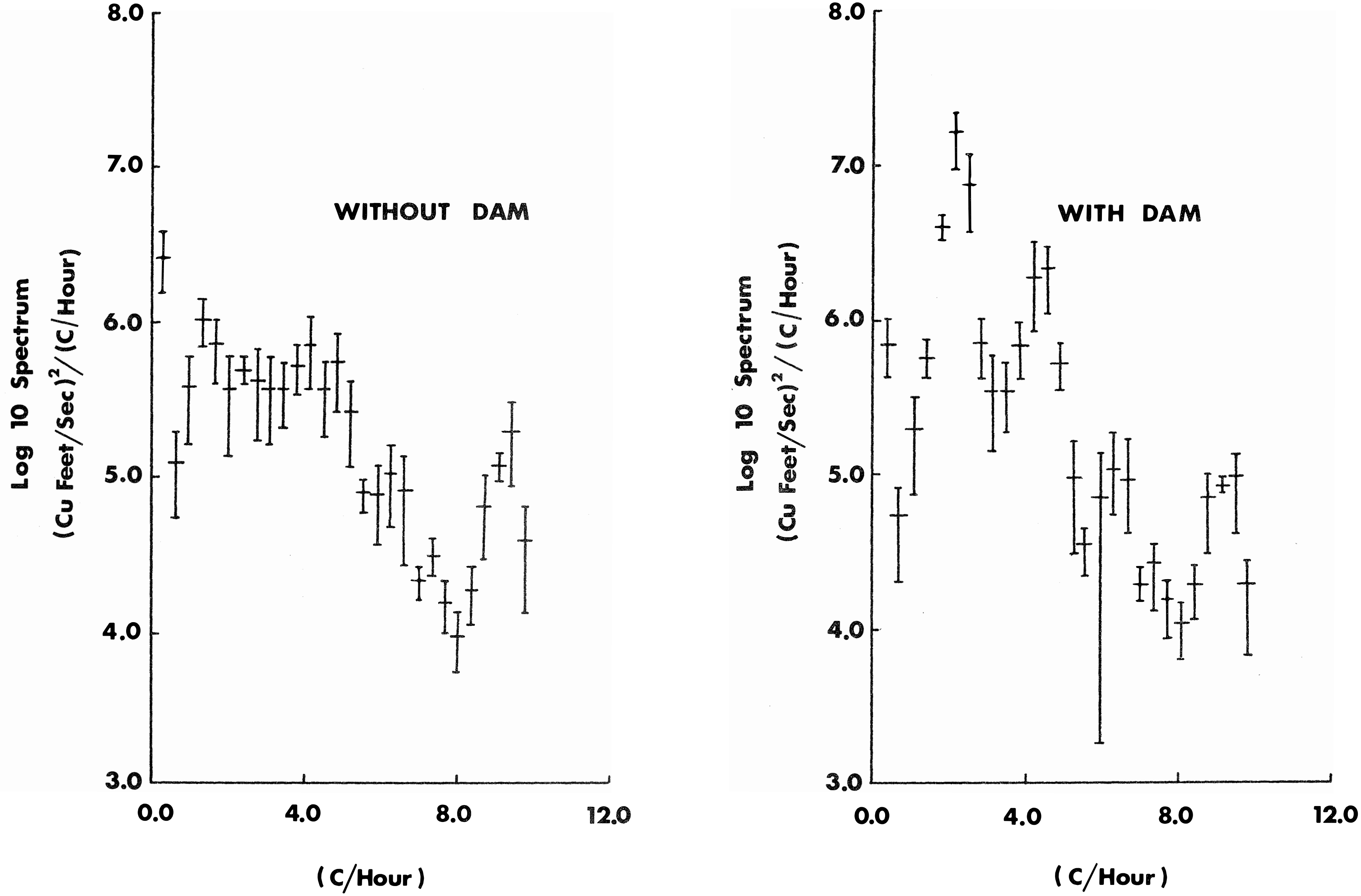


Fig. 17 Spectral analysis (Hanned FFT) of observed tidal heights in Victoria Harbour, June 9, 1971 (2048 data, 40 second intervals) and March 3-5, 1968 (1024 data, 3 minute intervals)

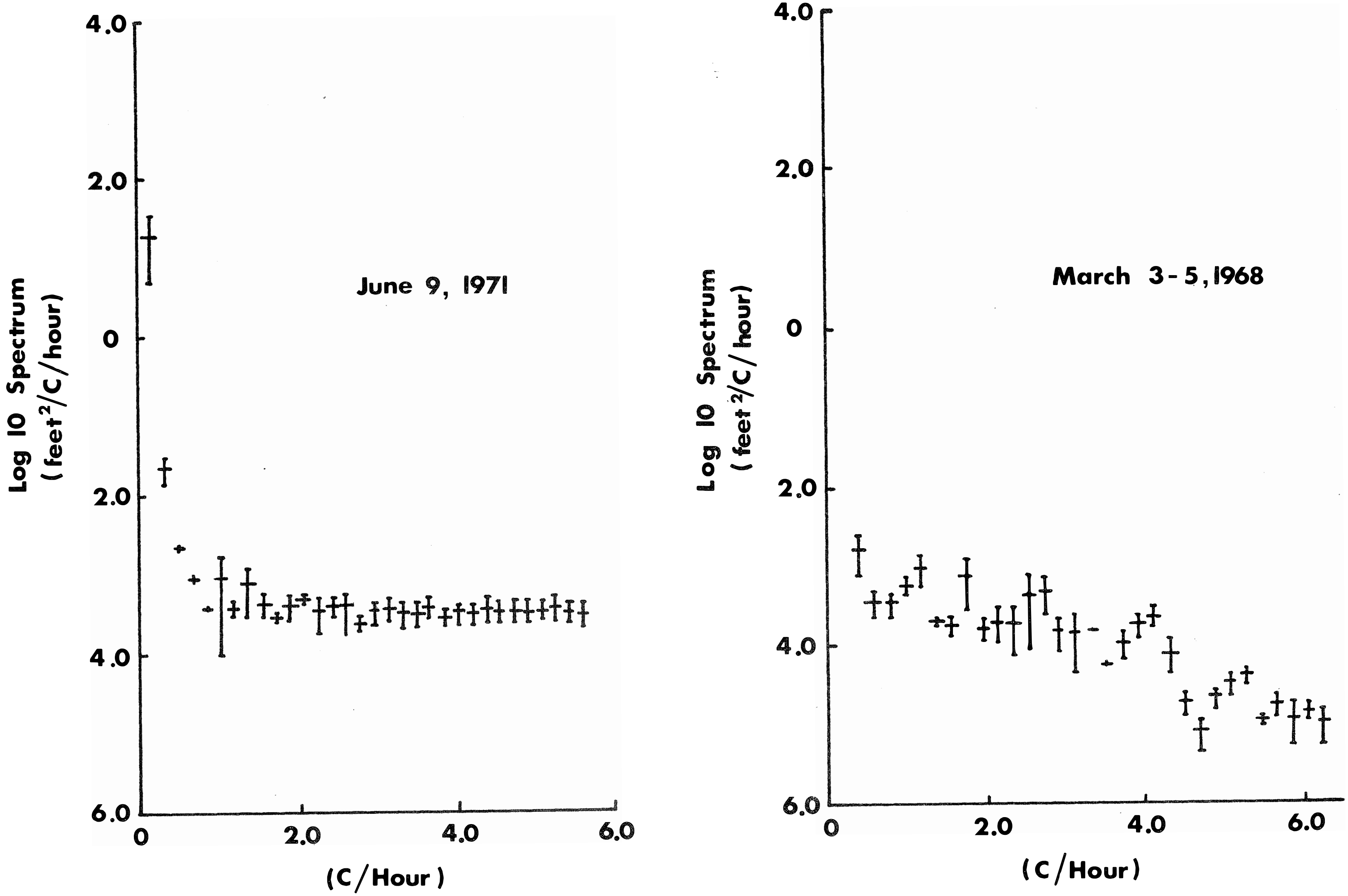


Fig. 18 Spectral Analysis of observed currents at Johnson St. Bridge, 3 November 1971, 10^h38^m PST to 10 November 1971, 13^h45^m PST. Sampling interval 160 seconds.

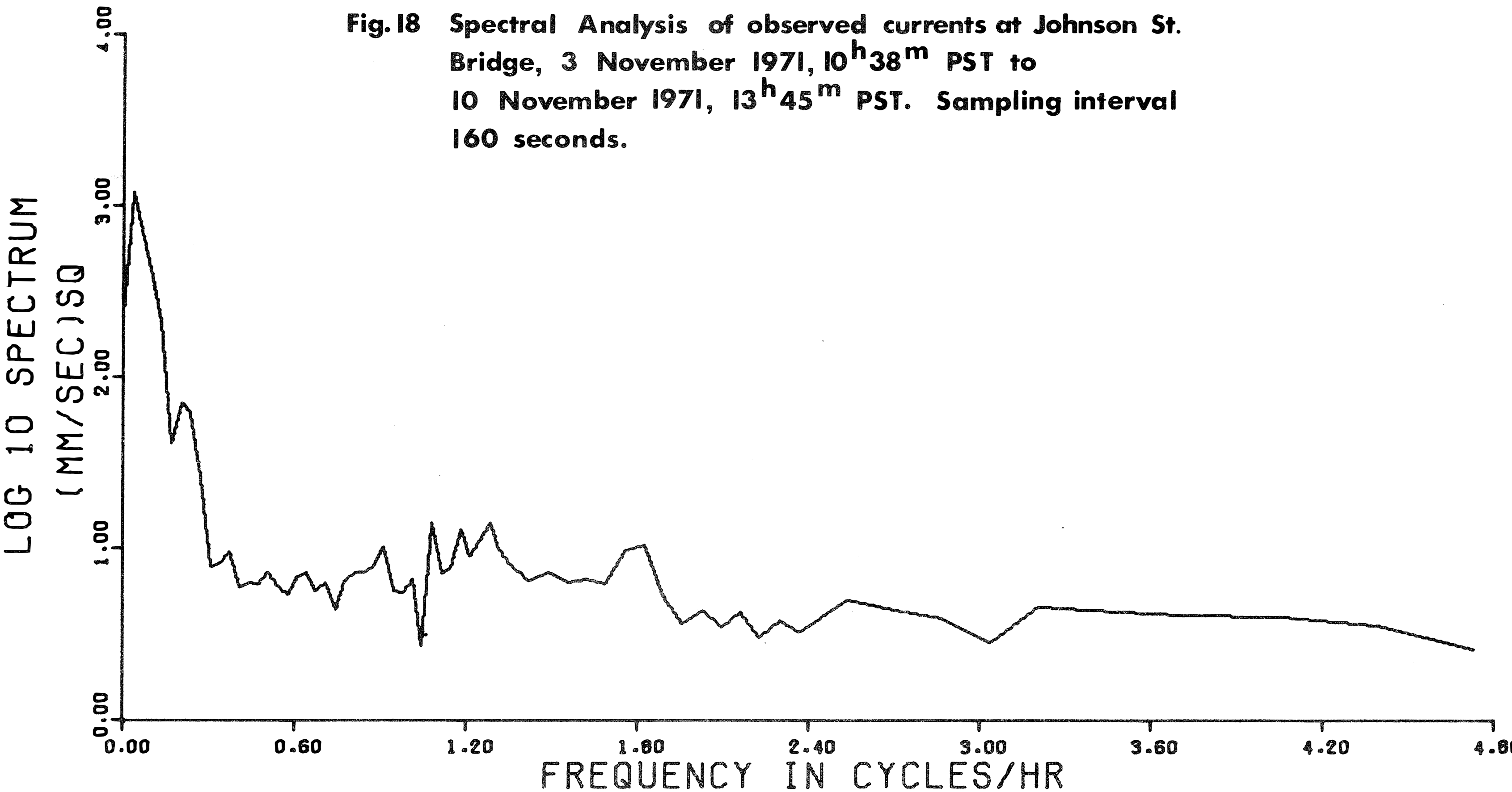


Fig. 19 Spectral Analysis of observed currents at Johnson St. Bridge, 3 November 1971, $11^h 30^m$ PST to 10 November 1971, $09^h 30^m$ PST.
Sampling interval 10 minutes

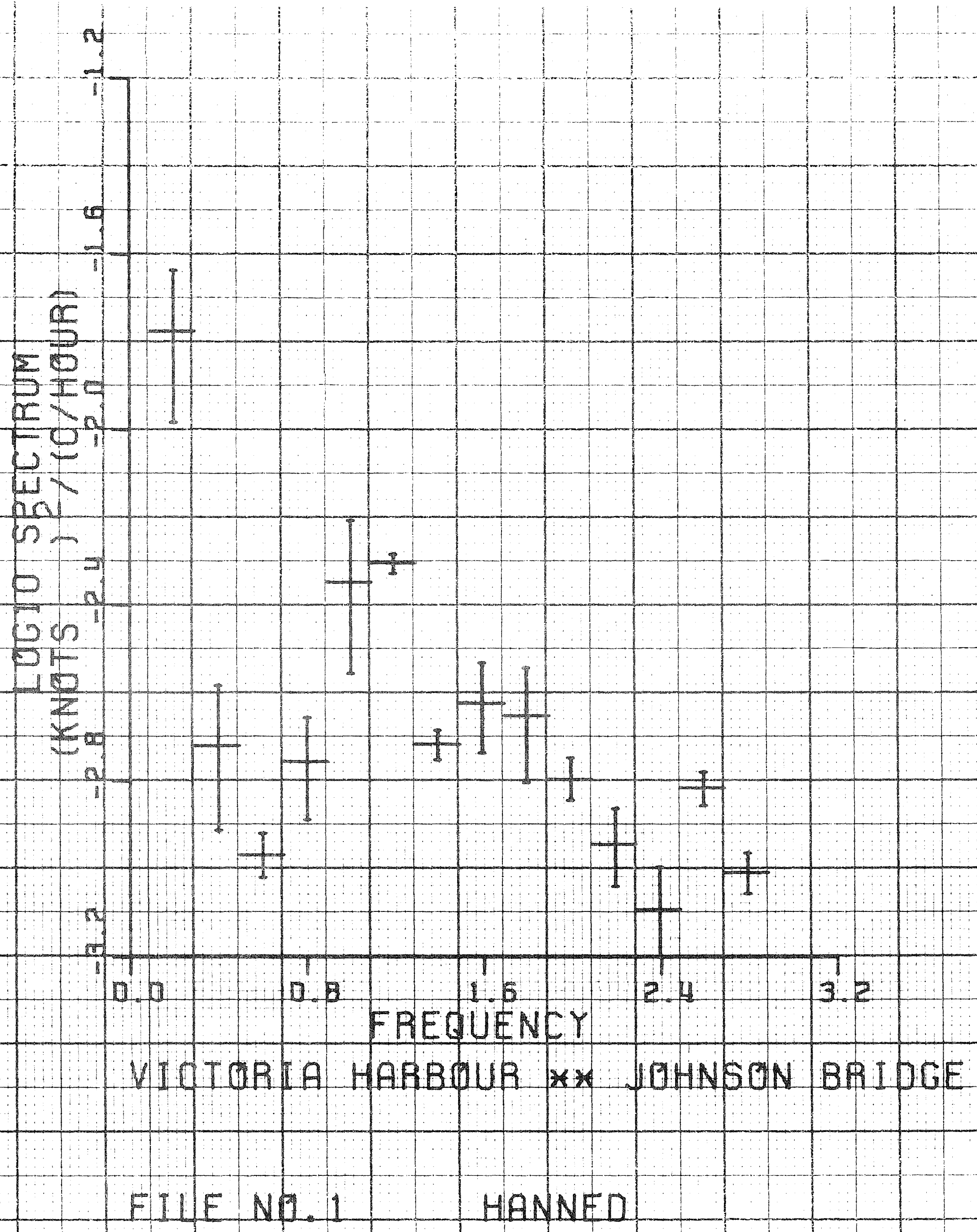


Fig.20 Proposed Portage Canal: Model-produced current velocities at Gorge (section 18) with canal (red), without canal (black); and in canal (green)
Input: M_2 tide.

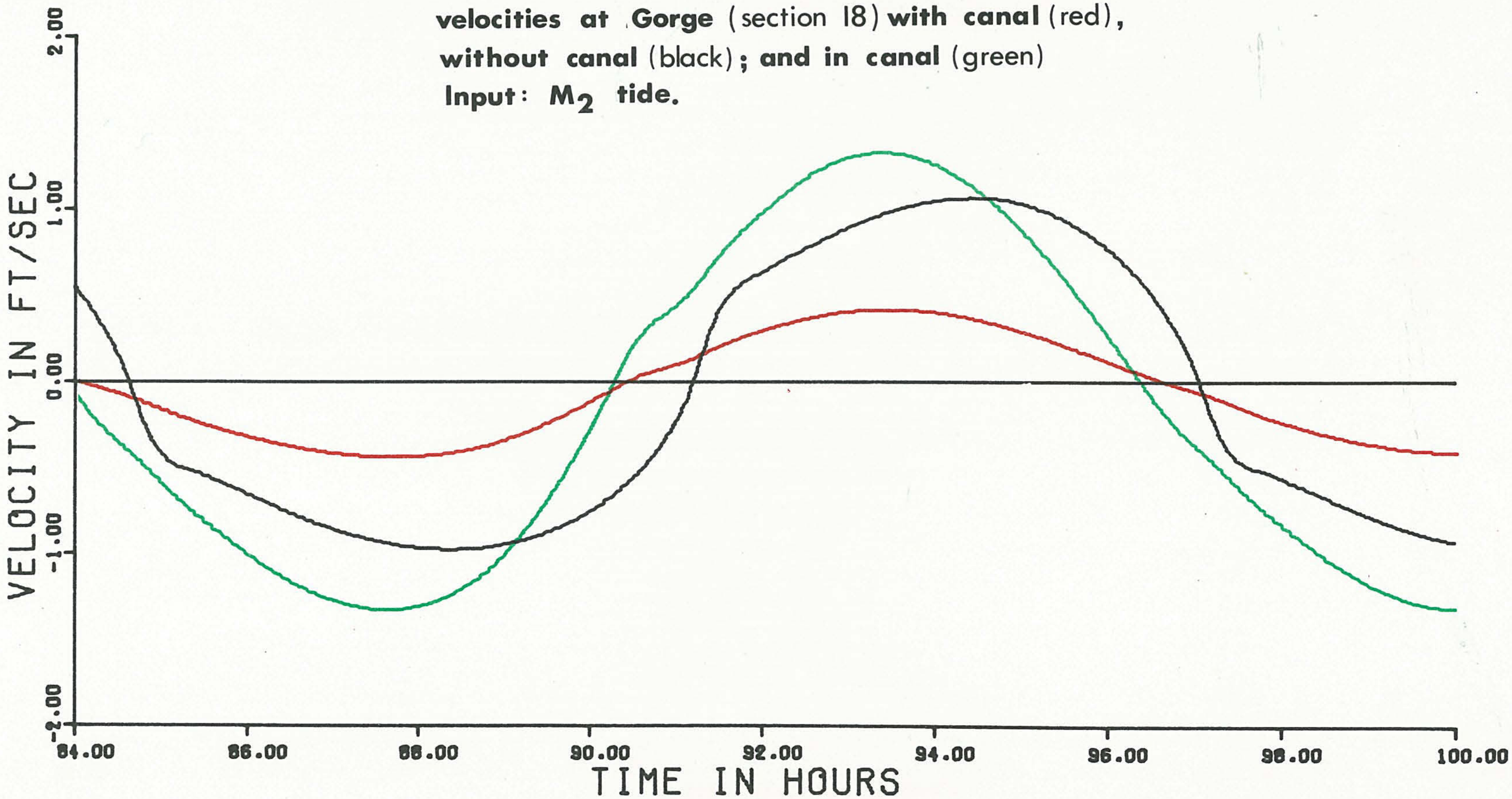


Fig. 21 Proposed dredging of the Gorge and Portage Inlet: Observed Victoria Harbour tidal heights (9–10 June 1971; in black)**; model-produced tidal heights in Portage Inlet before dredging** (green) **and after dredging** (red).

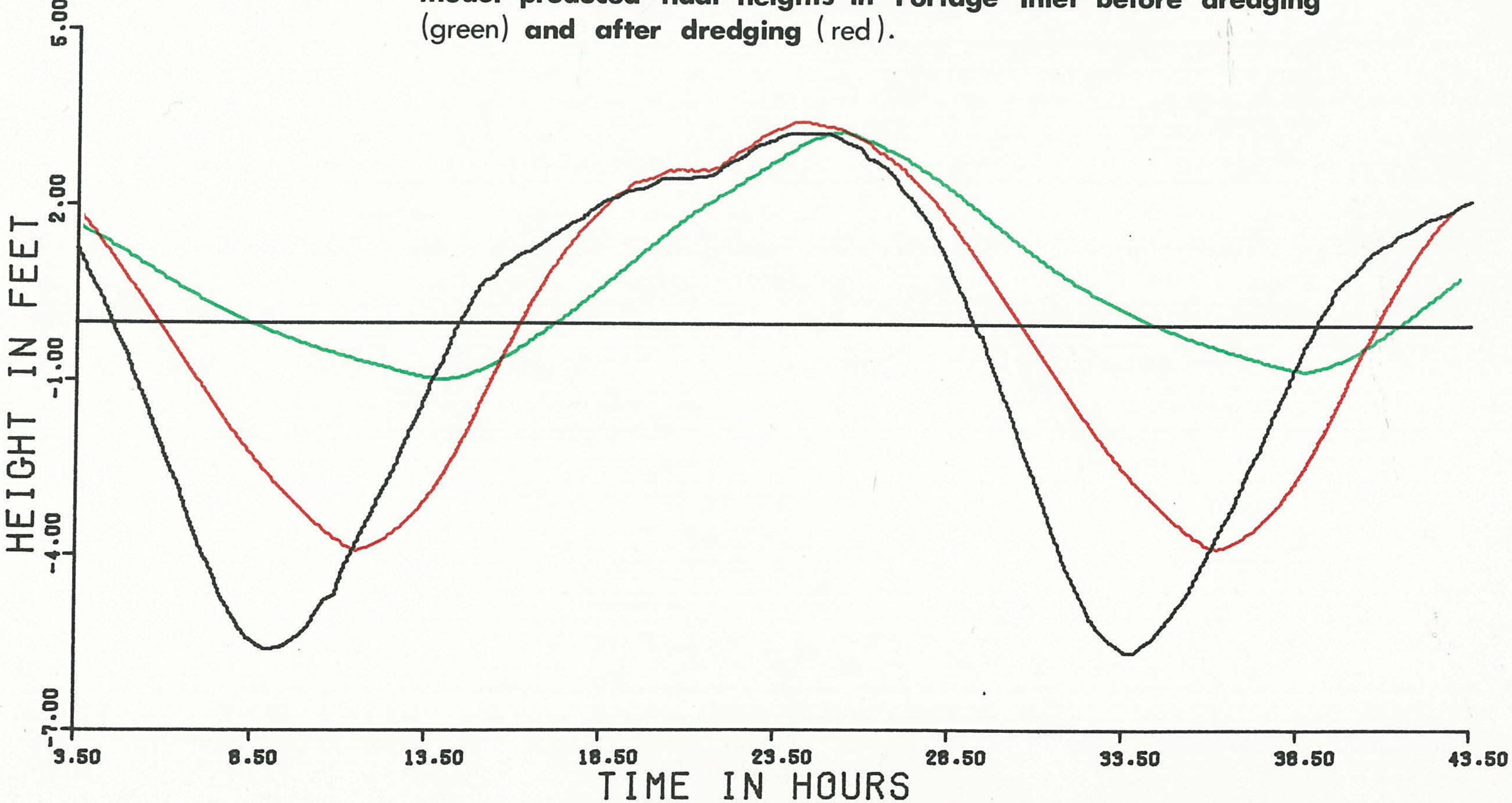
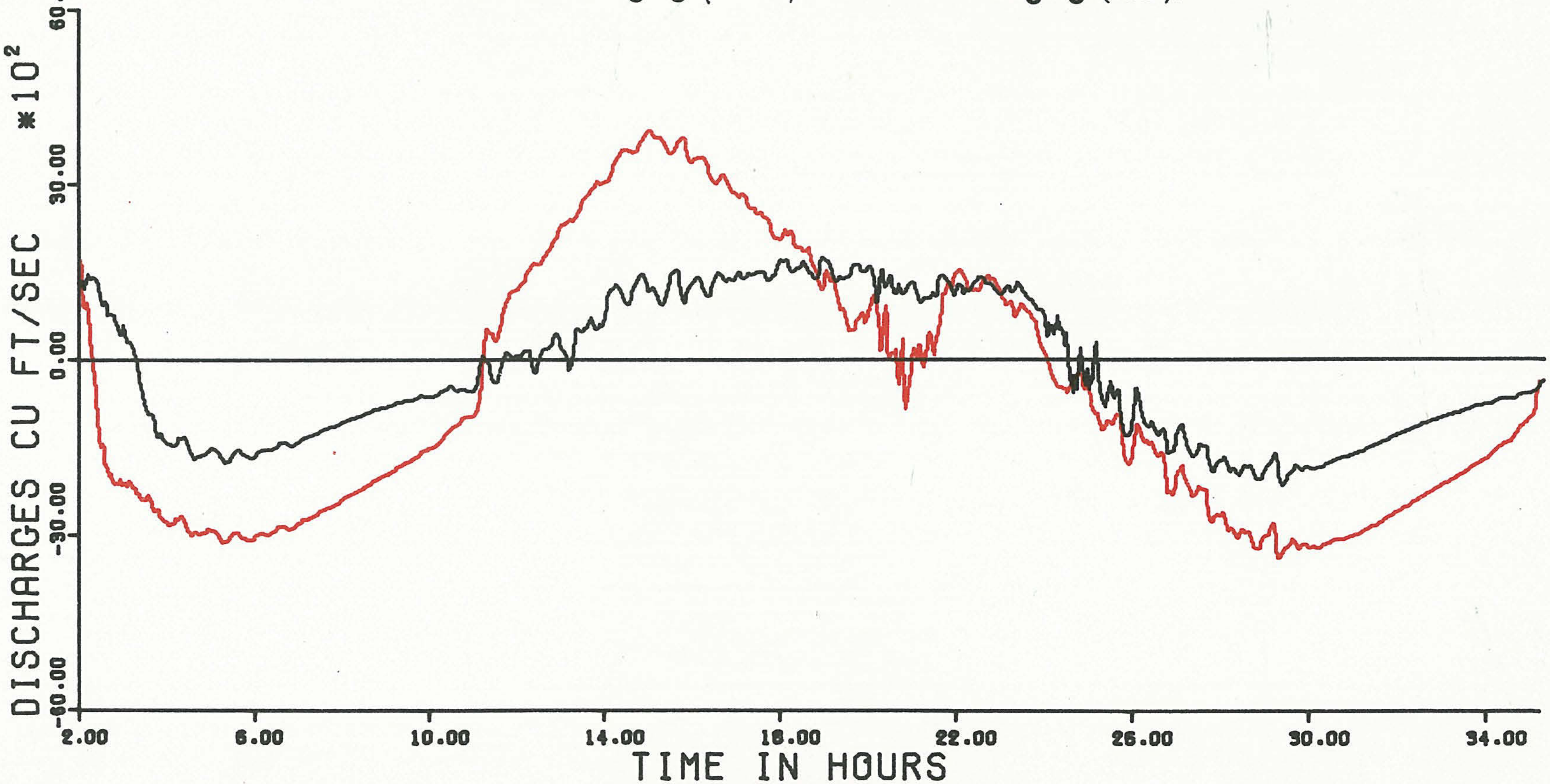


Fig. 22 Proposed dredging of the Gorge and Portage Inlet: Model-produced discharges at Gorge Bridge due to observed tides in Victoria Harbour (9-10 June 1971), before dredging (black) and after dredging (red).



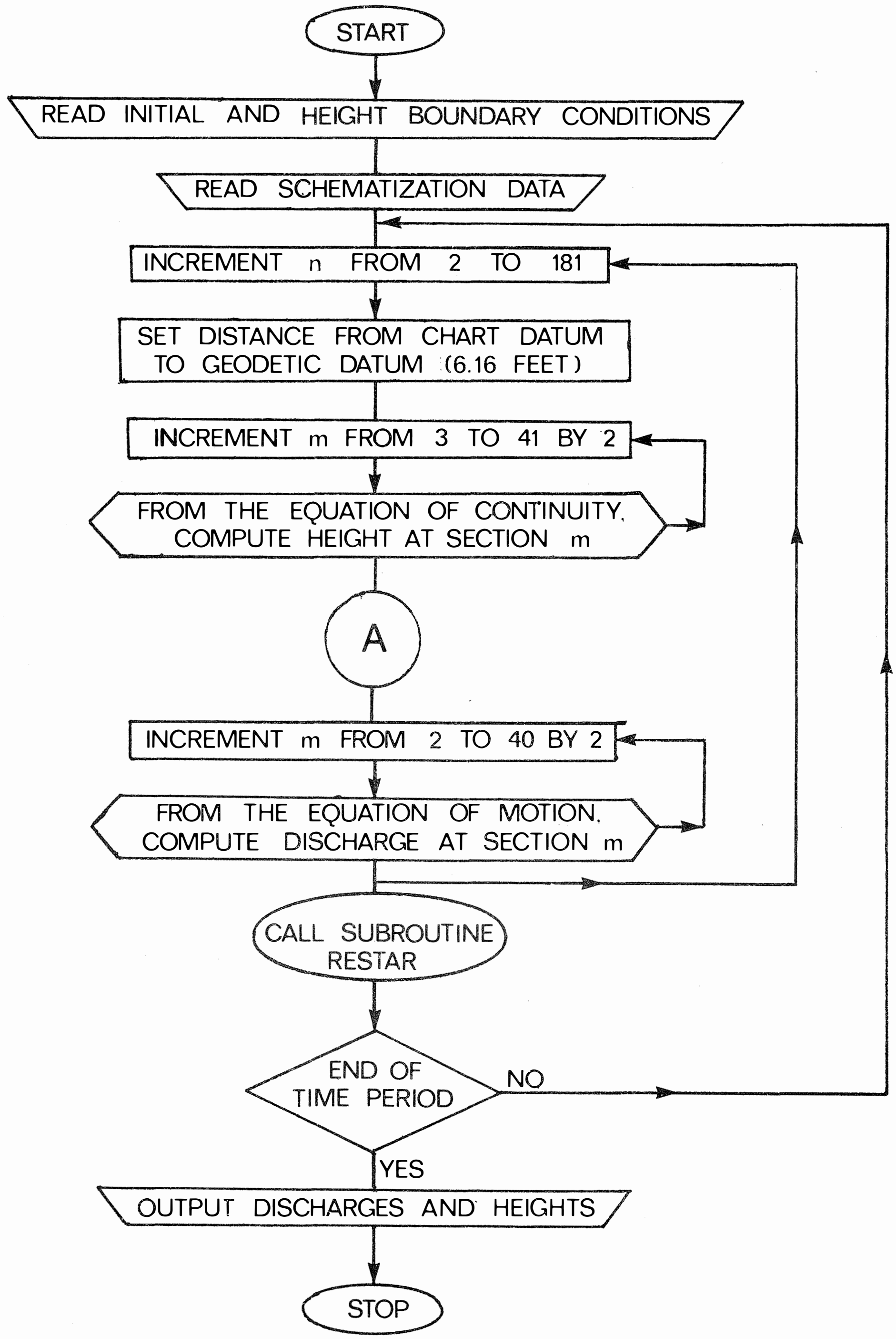
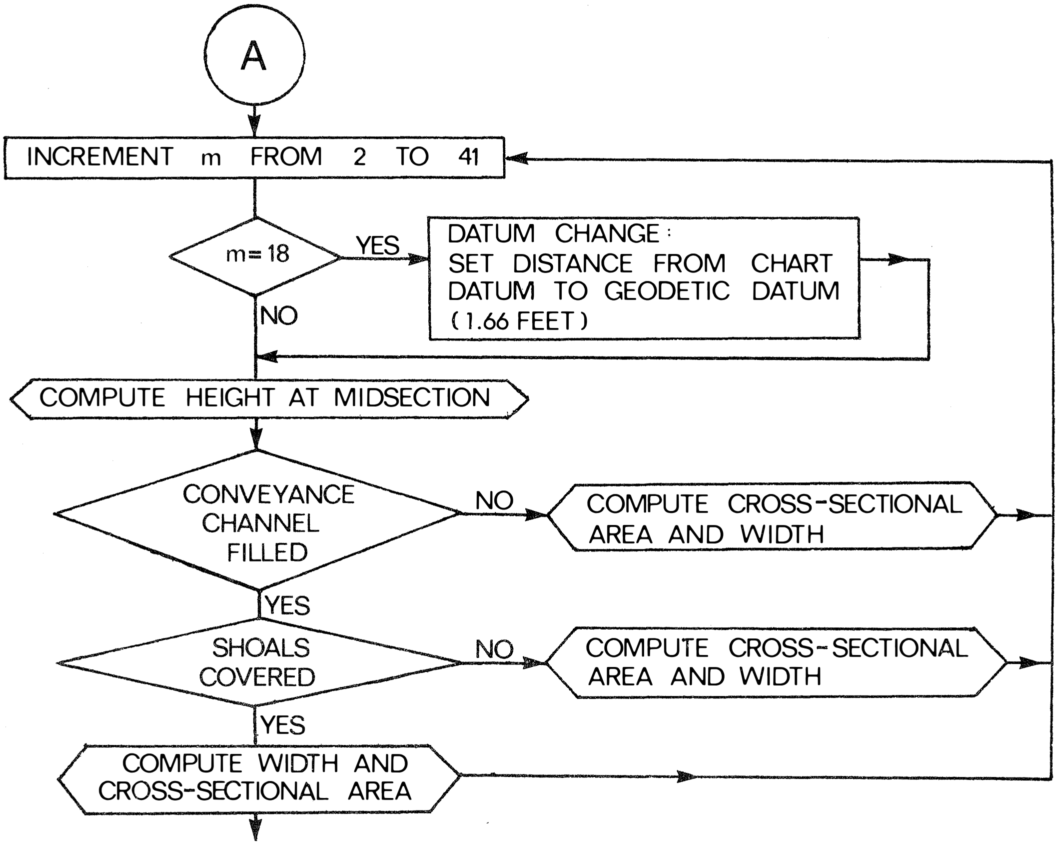
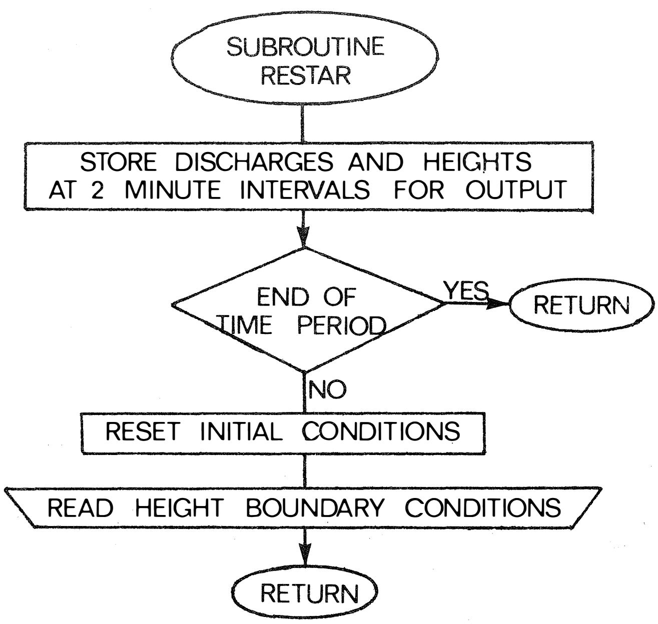


FIG. 23 FLOW CHART


A
INCREMENT m FROM 2 TO 41
m=18
YES
DATUM CHANGE:
SET DISTANCE FROM CHART
DATUM TO GEODETIC DATUM
(1.66 FEET)
NO
COMPUTE HEIGHT AT MIDSECTION
CONVEYANCE
CHANNEL
FILLED
NO
COMPUTE CROSS-SECTIONAL
AREA AND WIDTH
YES
SHOALS
COVERED
NO
COMPUTE CROSS-SECTIONAL
AREA AND WIDTH
YES
COMPUTE WIDTH AND
CROSS-SECTIONAL AREA


SUBROUTINE
RESTAR
STORE DISCHARGES AND HEIGHTS
AT 2 MINUTE INTERVALS FOR OUTPUT
END OF
TIME PERIOD
YES
RETURN
NO
RESET INITIAL CONDITIONS
READ HEIGHT BOUNDARY CONDITIONS
RETURN