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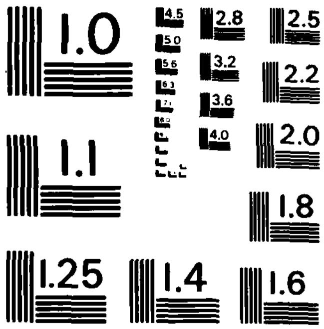
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AUTHOR: James A. Bailard and Scott A. Jenkins

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NAVAL CIVIL ENGINEERING LABORATORY
PORT HUENEME, CALIFORNIA 93043

METRIC CONVERSION FACTORS

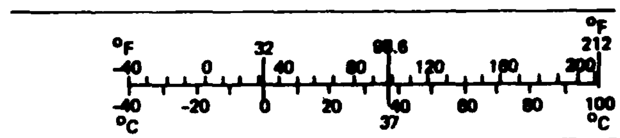
Approximate Conversions to Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
in	inches	*2.5	centimeters	cm
ft	feet	30	centimeters	cm
yd	yards	0.9	meters	m
mi	miles	1.6	kilometers	km
		AREA		
in^2	square inches	6.5	square centimeters	cm^2
ft^2	square feet	0.09	square meters	m^2
yd^2	square yards	0.8	square meters	m^2
mi^2	square miles	2.6	square kilometers	km^2
	acres	0.4	hectares	ha
		MASS (weight)		
oz	ounces	28	grams	g
lb	pounds	0.45	kilograms	kg
	short tons (2,000 lb)	0.9	tonnes	t
		VOLUME		
tsp	teaspoons	5	milliliters	ml
Tbsp	tablespoons	15	milliliters	ml
fl oz	fluid ounces	30	milliliters	ml
c	cups	0.24	liters	l
pt	pints	0.47	liters	l
qt	quarts	0.95	liters	l
gal	gallons	3.8	liters	l
ft^3	cubic feet	0.03	cubic meters	m^3
yd^3	cubic yards	0.76	cubic meters	m^3
		TEMPERATURE (exact)		
°F	Fahrenheit temperature	5/9 (after subtracting 32)	Celsius temperature	°C

*1 in = 2.54 (exactly). For other exact conversions and more detailed tables, see NBS Misc. Publ. 286, Units of Weights and Measures, Price $2.25, SD Catalog No. C13.10:286.

Approximate Conversions from Metric Measures

Symbol	When You Know	Multiply by	To Find	Symbol
		LENGTH		
mm	millimeters	0.04	inches	in
cm	centimeters	0.4	inches	in
m	meters	3.3	feet	ft
m	meters	1.1	yards	yd
km	kilometers	0.6	miles	mi
		AREA		
cm^2	square centimeters	0.16	square inches	in^2
m^2	square meters	1.2	square yards	yd^2
km^2	square kilometers	0.4	square miles	mi^2
ha	hectares (10,000 m^2)	2.5	acres	
		MASS (weight)		
g	grams	0.035	ounces	oz
kg	kilograms	2.2	pounds	lb
t	tonnes (1,000 kg)	1.1	short tons	
		VOLUME		
ml	milliliters	0.03	fluid ounces	fl oz
l	liters	2.1	pints	pt
l	liters	1.06	quarts	qt
l	liters	0.26	gallons	gal
m^3	cubic meters	35	cubic feet	ft^3
m^3	cubic meters	1.3	cubic yards	yd^3
		TEMPERATURE (exact)		
°C	Celsius temperature	9/5 (then add 32)	Fahrenheit temperature	°F



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Two concepts for reducing sedimentation in the turning basin at Mayport Naval Station were evaluated using a simplified box model methodology. The two concepts included a shallow venting canal and an extension to the entrance channel jetty.

The simulation model predicted that an optimally-sized venting canal would reduce the rate of sedimentation in the turning basin by approximately 60%. This result, however, was found to be particularly sensitive to uncertainties in the input data set. In contrast, the

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simulation model predicted that the proposed jetty extension would reduce the sedimentation rate by 85%. This result was not as sensitive to the uncertainties in the data set. With the relative cost of the canal being approximately one-tenth that of the jetty extension, the higher degree of uncertainty associated with the canal is offset by significantly higher potential cost savings.

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The simulation model predicted that an optimally-sized venting canal would reduce the rate of sedimentation in the turning basin by approximately 60%. This result, however, was found to be particularly sensitive to uncertainties in the input data set. In contrast, the simulation model predicted that the proposed jetty extension would reduce the sedimentation rate by 85%. This result was not as sensitive to the uncertainties in the data set. With the relative cost of the canal being approximately one-tenth that of the jetty extension, the higher degree of uncertainty associated with the canal is offset by significantly higher potential cost savings.

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INTRODUCTION

Mayport Naval Station Mayport, Fla, is faced with a serious maintenance dredging problem. Mayport Naval Station has an average annual dredging requirement of 448,000 m^3/yr (Hoffman, 1980). The unit cost of dredging presently ranges between 3 and 5 $\$/m^3$ and has been rising steadily. Dredge material disposal is another pressing problem. Recent estimates suggest that the remaining lifetime of the dredge material disposal area is 2 years. Once that area is full, either a new disposal area will have to be found or the material will have to be disposed of at sea. Both alternatives will be costly.

Two possible solutions to the above problems have been proposed. They are: to construct a 580-meter (1,900-foot) jetty extension from the end of the carrier berth peninsula; and to construct a shallow venting canal connecting the northwest corner of the turning basin to the St Johns River. The jetty solution was recommended by the U.S. Army Engineers Waterways Experiment Station (WES, 1979) on the basis of a movable bed model study; the latter was recommended by Scripps Institution of Oceanography (SIO), Jenkins et al., (1983), on the basis of several field studies at Mayport.

The two proposed solutions differ significantly in terms of approach and cost. The jetty extension is a passive approach that creates a buffer volume of water which isolates the basin from the river. The venting canal is a dynamic approach which acts to supply the basin with relatively sediment-poor water from the surface of the river in place of the sediment-rich water entering via the entrance channel. The estimated cost of the canal is approximately one-tenth that of the jetty extension.

Mayport Naval Station has recently submitted a Military Construction (MILCON) project application for the canal. This application has generated a number of questions regarding the expected efficiency of the proposed canal in reducing sedimentation and the optimum size for the proposed canal. The purpose of the present study was to answer these questions and similar questions relating to the proposed jetty extension. This study was sponsored by the Naval Facilities Engineering Command under the Engineering Investigation (EI) program.

OBJECTIVE

The primary objective of the study was to evaluate the expected efficiency of the proposed venting canal and determine its optimum size. A secondary objective was to perform a similar analysis for the proposed jetty extension. Satisfying these objectives required measurement of the sediment flux entering the turning basin and a more complete examination of the sea level inequality which develops between the basin and the river. The time history of the sediment flux was necessary for estimating the impact of the proposed venting canal and jetty extension on the sedimentation rate in the basin. Data on the difference in water levels between the basin and the river were needed to optimize the size of the proposed canal.

APPROACH

The Naval Civil Engineering Laboratory and Scripps Institution of Oceanography conducted a joint field study of the shoaling processes in Mayport turning basin. The field study focused on measuring: (1) water velocities and sediment concentrations in the turning basin entrance channel; (2) water levels in the St Johns River and the turning basin, near the location of the proposed venting canal; (3) bottom velocities and sediment concentrations in the river near the entrance to the proposed canal; and (4) predredged soil densities at points throughout the turning basin. Data were measured for a period of one tidal cycle on 1 Apr 1984.

The measurements obtained in this field study were used in conjunction with field data measured by Jenkins et al., (1983) to estimate: the rate of suspended sediment flux into and out of the basin as a function of time; the total dynamic head available to the canal as a function of time; the conversion factor for computing the volume flux of sediment from the weight flux of sediment; and the estimated efficiencies of different sized canals and entrance jetties in reducing rate of sedimentation in the Mayport turning basin. The following is a discussion of the velocity and concentration measurements and the methods which were used to estimate the above quanities.

MEASUREMENTS AND PROCEDURES

Entrance Channel Velocities and Concentrations

The suspended sediment flux passing into or out of the turning basin at any given moment, SF_e, is the spatial integral of the product of the local instantaneous water velocity and the local instaneous suspended sediment concentration. In practice, this integral is difficult to estimate because the velocity and concentration are measured at a few discrete points in space and time.

In the present study, water velocity (U) and suspended sediment concentration (C) were measured at two stations located on the flanks of the entrance channel (see Figure 1). At each station current meters were positioned at the top and bottom of the water column while a boat was used to gather water samples at four points within the water column (see Figure 2). The sampling interval for the current meters was 1/2 second, with the mean recorded every 10 minutes. Suspended sediment concentration profiles were obtained every 20 minutes.

The above data were used to construct time series of water velocity and sediment concentration, which were representative of surface and bottom conditions on the north and south sides of the entrance channel to the basin (see Figure 2). In all cases, a common designation of 1 to 4 was used with 1 and 3 referring to the north and south surface quadrants and 2 and 4 referring to the north and south bottom quadrants. Positive velocities are directed into the basin.

Figure 3 shows a plot of the two surface and two bottom velocities measured in the entrance channel to Mayport turning basin. The corresponding suspended sediment concentration measurements are shown in Figures 4 and 5. For both the suspended sediment concentration and the velocity data, a complete tidal cycle is shown (approximately 12.3 hours); however, a portion of the data shown (approximately 2030 to 2140 hours for the velocity data and 0930 to 1200 hours and 2020 to 2140 hours for the suspended sediment concentration data) had to be estimated due to delays in deploying several of the instruments. (Note that data for a complete tidal cycle were needed to compute the flux estimates).

Examining Figure 3, several interesting features can be seen in the velocity records. First, the magnitude of the surface currents were significantly greater than those of the bottom currents. Second, the bottom currents began flowing into the basin approximately 2 hours before the surface currents. When the surface currents did begin to flood, the onset was very rapid, particularly on the south side of the entrance channel. This was visually evident during the study when foam lines from the zone of shear at the river/entrance channel interface were swept into the basin. Visual observations also confirmed that the transition from ebb to flood conditions was delayed 30 minutes on the north side of the channel, with the transition taking place more gradually.

Flood currents lasted approximately 4 hours beginning at low tide (1400 hours) and ending at 1800 hours. At this point, the surface currents abruptly switched to ebb flow, with the bottom currents following a short time later. The brief, but intense ebb flow at the surface lasted approximately 3 hours followed by a 5-hour period of relatively weak ebb flow. Bottom currents followed a similar pattern except that the period of intense ebb flow lasted several hours longer.

The suspended sediment concentration data shown in Figures 4 and 5 were found to exhibit a high degree of variation. Peak concentrations were found to be as great as 0.075 gm/liter, with more frequent peaks in the 0.04 to 0.05 gm/liter range. The highest peaks occurred during the brief but intense period of flood currents. During ebb flow, lesser peaks were measured. These were perhaps the result of earlier flood peaks being advected back out of the basin. In general, however, the suspended sediment concentration levels occurring during flood flow were found to be significantly greater than those during ebb flow.

On the south side of the channel (Figure 5), the top and bottom suspended sediment concentrations were found to be well correlated. In fact, Figure 5 shows them to be nearly identical, except at late ebb tide (1230 hours) when the bottom concentrations exceeded surface concentrations by a factor of 3. On the north side of the channel (Figure 4), the surface and bottom concentrations were also found to be correlated, except during early flood (1400 to 1500 hours) when the bottom concentrations led the surface concentrations by approximately 1 hour. The latter might be explained by the nearly 2-hour lead in the bottom currents relative to the surface currents at the onset of flood flow. The leading bottom currents would act to bring the high concentration sediment originating at the river/channel interface to the lower profiling stations sooner than to the surface stations. During ebb, the top and bottom concentrations were found to be closely correlated, except during late ebb when the bottom concentrations exceeded surface concentrations by approximately 0.01 gm/liter.

Volume and Sediment Fluxes

The following equation was used to estimate the volume flux of water (Q_e)in the entrance channel:

$$Q_e = (W_t A_{et}) U_t + (W_b A_{eb}) U_b \qquad (1)$$

where A_{et} is the cross-sectional area of the entrance channel above -5 meters MLLW, A_{eb} is the cross-sectional area below -5 meters MLLW, $U_t = (U1 + U3)/2$ and $U_b = (U2 + U4)/2$ are the average measured velocities at the top and bottom, and W_t and W_b are weighting factors needed to convert the measured velocities to the spatially-averaged velocities representative of the upper and lower cross-sectional areas (1030- and

985-m^2 respectively). Figure 6 shows a plot of U_t and U_b as a function of time. Note that for simplicity, the variation of A_{et} with the tide has been neglected.

Least squares estimates of W_t and W_b were obtained from continuity and Equation 2 using the measured velocities, U_t and U_b, and the predicted tidal prism. This procedure required that the integral of Q_e over the period of flood tide be equal to the tidal prism of the basin. Similarly, the integral of Q_e over the complete tidal cycle was required to be equal to zero, i.e.,

$$\int_0^{T/2} Q_e \, dt = A_p(\eta_{b\,Hi} - \eta_{b\,Lo}) \qquad (2)$$

and

$$\int_0^{T} Q_e \, dt = 0. \qquad (3)$$

where T is the tidal period, A_p is the plan area of the basin; and η_{bHi} and η_{bLo} are the water elevations in the basin at high and low tide respectively. The resulting estimates for W_t and W_b were found to be equal to 0.174 and 0.154 respectively.

Figure 7 shows the volume flux, Q_e as a function of time. The peak channel flow rate was found to be approximately 90 m^3/sec during ebb flow. During flood flow, a value of 88 m^3/sec was reached; however, the duration of peak flow was maintained for a longer period of time. A noticeable feature of the estimated volume flux record is the rapid onset of flood flow and the equally rapid change to ebb flow.

Modifying Equation 2, the dry weight flux of sediment passing through the entrance channel, SF_e, was estimated as:

$$SF_e = (W_t A_{et}) U_t C_t + (W_b A_{eb}) U_b C_b \quad (4)$$

where $C_t = (C1 + C3)/2$ and $C_b = (C2 + C3)/2$ are the average suspended sediment concentrations at the top and bottom of the channel.

Figure 8 shows a plot of C_t and C_b as a function of time. Similarly, Figure 9 shows a plot of the surface and bottom contributions to the sediment flux. Clearly, most of the suspended sediment flux is occurring in the upper portion of the water column. Summing the surface and bottom fluxes, Figure 10 shows a plot of the total estimated suspended sediment flux as a function of time. The suspended sediment influx was found to occur as two distinct peaks, the first reaching a value of 3.1 kg/sec during early flood and the second reaching a value of -2.5 kg/sec during late flood. The ebb suspended sediment flux also contained two peaks, however, they were spaced more closely together in time. Integrating the sediment flux over the complete tidal cycle led to an estimated net flux of 6,400 kg of dry sediment into the basin. This is equivalent to a net influx of 68 m^3 of sediment, based on the measured bulk densities of the sediments in the basin.

Apparently shoaling conditions on 1 Apr 1984 were weak, for the average shoaling rate based on annual dredging records is 614 m^3 of sediment per tidal cycle. Most likely, the shoaling in Mayport turning basin is episodic with high rates of shoaling occurring during periods of high sediment abundance in the river. Alternatively, the present method of estimating the sediment flux into the basin may have underestimated the total sediment flux entering the basin by not resolving high frequency fluctuations in the suspended sediment concentration. The present data set cannot resolve fluctuations having periodicities less than 40 minutes. If rapid fluctuations in sediment concentration were positively correlated with flood velocity fluctuations, then the actual suspended sediment flux would be greater. Unfortunately, without more rapid sampling, the question of aliasing of the suspended sediment flux estimates could not be resolved. For the present study, aliasing was assumed to be negligible.

Total Dynamic Head

Jenkins et al., (1983) found that the flow of the St Johns River past the entrance to Mayport turning basin causes the water level in the basin to be lower than the water level in the river during ebb river flow and higher during flood river flow. The physical processes which cause the water level differences are not well understood, however, the draw down of the basin is thought to be caused by an entrainment process at the river/entrance channel shear zone. In contrast, the super elevation of the basin during flood river flow is thought to reflect the dynamic head of the river.

As a result of the difference in water levels between the basin and the river, the total dynamic head which is available to produce a flow in the proposed canal is composed of two terms: the water level difference, and the velocity head of the river. Jenkins et al., (1983) measured both of these quantities for a period of several days, however, the location of these measurements was at the end of the carrier-berth peninsula, as opposed to the site of the proposed canal. Moreover, Jenkins, et al., (1983) analyzed their measurements in terms of short period variations (tens of seconds) in the water level differences as opposed to longer period variations (an hour or longer), which are of interest in the present study.

Because of these differences, the present study sought to: (1) obtain new measurements of water level differences at a location nearer to the site of the proposed canal; and (2) re-analyze the water level measurements of Jenkins et al., (1983) in terms of longer period variations. Unfortunately, local Radar signals interferred with the pressure sensor data recorder causing a complete loss of all water level measurements. As a result, only the Jenkins et al. data set was available for analysis. However, theory suggests that the maximum difference in the water level differences measured at the two sites should be less than 1 cm.

Figure 11 shows a plot of the basin water level, the river water level, and the river velocity (positive = flood or west) as a function of time for most of a tidal cycle on 17 Feb 1983. The figure shows that at low tide, the river velocity is at maximum ebb while at high tide the river velocity is at maximum flood. These currents produce a sea level inequality between the river and the basin, with the basin being lower than the river at low tide and higher than the river at high tide. Although measurements of water elevations were not obtained in the present study, measurements of river velocities were obtained (see Figure 12). These velocities show a close correlation with the tide, substantiating the data measured by Jenkins et al., (1983).

Figure 13 shows a plot of the difference in water level elevation between the basin and the river for the same data as shown in Figure 11. It is apparent that the water level inequality closely follows the tide, with the basin being approximately 5 cm lower than the river at low tide (approximately 1700 hours), and 10 cm higher than the river at high tide (approximately 2300 hours). The data shows that during the time in which it would be most beneficial for the canal to be open (during the 6 hours between low and high tide), the water level of the basin is lower than the level of the river only during the first 3 hours. After 2000 hours, the basin is higher than the river. This would cause the canal flow to be out of the basin, drawing more sediment-laden water into the entrance channel and increasing the sedimentation rate in the basin. Clearly, a gate is needed for the canal to function as intended.

A canal simulation model, which will be discussed shortly, requires estimates of the water level inequality and the near-bank river velocity as a function of time. An examination of these variables (see Figures 11 and 13) showed them to be closely correlated with the tide. As a result, the following regression equations were obtained.

$$\Delta\eta = 8.4 \times 10^{-4} \eta_r^2 + 0.015 \eta_r - 4.1 \quad (5)$$

and

$$U_r = 0.662 \eta_r - 41.1 \quad (6)$$

where $\Delta\eta$ is the sea level inequality (basin-river) in cm, η_r is the tide in the river in cm MLLW, and U_r is the near-bank river velocity (cm/sec, positive on flood). Figures 14 and 15 show a plot of $\Delta\eta$ and U_r versus η_r along with their respective regression equations.

Fortunately, the tidal conditions present during the field study by Jenkins et al., (1983) were nearly identical to the present study. In the former case (17 Feb 1983) high and low tide elevations were 1.22 and -0.09 meters MLLW respectively, versus 1.34 and -0.06 meters MLLW in the present study (1 Apr 1984). As a result, the sea level inequality and river velocity estimates generated from Equations 5 and 6 are probably representative of conditions occurring on 1 Apr 1984.

In-Situ Bulk Densities

The suspended sediment fluxes discussed above are expressed in terms of the dry weight flux of suspended sediment per unit of time. In order to compare the estimated suspended sediment fluxes with the average annual sedimentation rate in the basin (approximately 448,000 m^3/yr), it was necessary to know the density of the predredged sediment deposit. During the field study, core samples were taken at five different sites within the turning basin. These were analyzed for wet bulk density, and the equivalent dry weight densities were computed. The average wet bulk density was found to be 1,085 kg/m^3 while the an equivalent dry bulk density was found to be 94.1 kg/m^3. The net shoaling rate can be computed from the net dry sediment flux by dividing by the in situ dry bulk density, i.e., 1 kg of dry sedimant is equivalent to 0.011 m^3 of sedimentation.

SIMULATION MODEL

The key to estimating the efficiency of the proposed canal or jetty in reducing sedimentation in Mayport turning basin is to understand their effect on the sediment flux into the turning basin. The method used to predict these effects was to construct a simple box model of the basin, which accounted for all of the suspended sediment flux passing into or out of the basin. This flux was predicted over a complete tidal cycle using the present data set. Although the conditions measured on 1 Apr 1984 were probably not representative of average annual conditions, it was assumed that the relative effect of these structures on the sedimentation rate would remain constant throughout the year.

Figure 16 shows a schematic diagram of the box model used in the present analysis. The box model consisted of: a basin with plan area A_p; an entrance channel with cross-sectional area A_e and length L_e, the latter of which can vary depending on the length of any proposed jetty; and, a canal with cross-sectional area A_c and length L_c. The overall volume flux (flow rate) of water passing into and out of the basin was assumed to be controlled by the tide, with the sum of the flows passing through the entrance channel and the canal equal to the overall undisturbed tidal flux.

The water level in the river was assumed to be equal to the predicted tide based on the tide table data for 1 Apr 1984 (see Figure 17). The water level in the basin was assumed to be equal to the river level plus the sea level inequality predicted by Equation 5. Near-bank river velocities were predicted by Equation 6 using the tidal elevation of the river. The model allowed the cross-sectional area of the canal and the length of the entrance channel jetty to vary as specified.

Considering the volume flux of water into the basin, continuity requires that

$$Q_c + Q_e = Q_{eo} \tag{7}$$

where: Q_c = canal flow rate into the basin

Q_e = entrance channel flow rate into the basin

Q_{eo} = flow rate which normally passes through the entrance channel in the absence of a canal

Q_{eo} was estimated from Equation 1 in conjunction with the measured velocity data.

The canal flow rate (m^3/sec) was predicted using the following open channel flow equation:

$$Q_c = \frac{1.00}{n} A_c R_c^{2/3} \sqrt{\frac{Thd}{L_c}} \tag{8}$$

where: R_c = hydraulic radius of the canal, meters

Thd = total dynamic head between the river and the basin, meters

n = roughness coefficient (approximately 0.012).

L_c = canal length, meters

Noting that with the exception of Thd, all of the parameters in Equation 8 are constant, Equation 8 was rewritten as:

$$Q_c = Kprime \sqrt{Thd} \tag{9}$$

where

$$Kprime = \frac{1.00}{n} \frac{A_c R_c^{2/3}}{L_c^{1/2}} \tag{10}$$

During ebb river flow, Thd can be expressed as

$$Thd = \Delta\eta + \frac{U_r^2}{2g} \tag{11}$$

Where $\Delta\eta$ and U_r can be predicted from Equations 5 and 6.

The present study measured the suspended sediment concentration in both the river and the basin entrance channel. In principle, these concentrations can be used to predict the suspended sediment fluxes passing through the canal and entrance channel once the respective flow rates were known. One complication in this approach, however, is that the sediment entering the basin through the entrance channel takes a finite period of time to pass the length of the entrance channel. The suspended sediment concentration measured at the entrance to the basin is a function of both the time history of the channel velocity and the time history of the suspended sediment concentration at the interface between the river and entrance channel. Because the entrance channel velocity will vary with canal flow rate, it was found to be simpler to work with the estimated suspended sediment concentration at the river/channel interface and to predict the resulting time lag.

In the present study, the suspended sediment concentration at the river/channel interface was assumed to be equal to the concentration measured along the banks of the river. A visual comparison between the river and entrance channel suspended sediment concentration measurements suggested that this was a reasonable approximation since the peak values were approximately equal. A more accurate approach would have been to use the actual suspended sediment concentration at the interface, however, this was not measured. For the canal, the suspended sediment concentration was assumed to be equal to the suspended sediment concentration in the river with a zero time lag.

Considering the entrance channel flow, the length of time (lag time), Δt, needed for a suspended sediment particle to travel the length of the entrance channel, L_e, was estimated from the following equation:

$$\frac{1}{A_e} \int_{t-\Delta t}^{t} Q_e \, dt = L_e \tag{12}$$

At first glance, the length of the entrance channel, L_e, would appear to be equal to the length of the jetty. Lacking a jetty, this length would be zero, however, the measured suspended sediment concentration data indicated a time lag close to 1.67 hours. Based on Equation 13, this time lag suggested that the entrance channel has a "natural" length of approximately 100 meters. Apparently, the shoal area which projects out from the end of the carrier-berth peninsula acts to separate the entrance channel from the river. In any event, L_e was assumed to be equal to the jetty length plus 100 meters throughout the analysis.

Utilizing Equations 4 and 7, the sediment flux into the basin can be expressed as:

$$SF = Q_c(C_r - C_e) + Q_{eo} C_e \tag{13}$$

where: C_r = suspended sediment concentration in the river

C_e = suspended sediment concentration in the inner reaches of the entrance channel

Based on the above discussion, during flood tide when the basin is filling, the suspended sediment concentration entering the basin from the entrance channel was computed as

$$C_e = C_r \Big|_{t = t - \Delta t} \tag{14}$$

During ebb tide, when the basin is emptying, C_e was assumed to have a constant value of 0.01 gm/liter. The value was selected rather arbitrarily to reflect a general background concentration level during late ebb tidal flow out of the basin (see Figure 5). A better estimate would reflect the time history of sediment influx into the basin minus a percentage due to deposition; however, this was judged to be too complex an approach for the present model.

The objective of the canal is to reduce the net flux of suspended sediment into the basin. To accomplish this, it is desirable to operate the canal only during flood tide when the basin is filling. During the roughly 6 hour period between low tide and high tide, however, the basin is lower than the river only during the first 3 hours or so. After that, the basin becomes higher than the river. If the canal were open during the latter half of the filling cycle, the canal flow would be out of the basin, causing additional flow to enter the basin from the entrance channel. The latter would carry additional sediment into the basin, causing increased shoaling. Obviously, it would be better for the canal to be closed at this time.

When the basin is emptying (between high and low tide), it is desirable to force all of the flow out through the entrance channel. This will help to minimize shoaling in the channel. If suspended sediment concentrations were sufficiently low within the river, it would be beneficial to open the canal during the 3-hour period of time preceding low tide. This would serve to feed water into the basin, increasing ebb channel flows, and further scouring sediment from the entrance channel. The data measured in the present study, however, suggested that suspended sediment concentrations in the river were too high relative to the basin for this to have a net positive effect.

The above arguments suggest, that the canal requires a gate which can be synchronized with the tide. In fact, it will be shown that the precise time at which the gate is opened and closed in relationship to the tide, has a profound effect on the efficiency of the canal.

Equations 1, 9, 12, 13, and 14 completely describe the sediment flux as a function of time. The final step was to integrate the sediment flux (Equation 13) over a complete tidal cycle to compute the net sediment flux into the basin.

Because of the approximate nature of the box model and the fact that the conditions measured on 1 Apr 1984 were probably not representative of yearly "average" conditions, the relative reduction (or increase) in the net sediment flux into the basin was estimated instead of the absolute magnitude. In addition, the present model only predicts the shoaling which takes place in the turning basin. Bathymetric surveys have shown that 85% (381,000 m^3/yr) of the sediment dredged at Mayport comes from the turning basin, while 15% (67,000 m^3/yr) comes from the entrance channel. Assuming that the latter amount remains unchanged by the presence of a canal or a jetty (a rather questionable assumption made necessary by a lack of better information), then the relative shoaling rate, RSR, can be expressed as

$$RSR = \frac{(381{,}000 \times NSF + 67{,}000)}{448{,}000} \tag{15}$$

where the net shoaling factor, NSF is defined as

$$NSF = \text{net sed flux/net sed flux (base-conditions)} \tag{16}$$

Finally, the efficiency of the canal or jetty is equal to 1 - RSR.

The canal/jetty simulation model was used to evaluate the performance of a number of different-sized canals and jetties, both individually and in conjunction with each other. In simulating the effects of the canal, it was assumed that the canal incorporated a gate which could be opened and closed as desired.

Initially, no assumptions were made concerning the geometry of the canal: its length, its height, or its width. Instead, canals with different capacities were evaluated in terms of Kprime (see Equation 10). Values of Kprime were varied between 0 (no canal) and 500 $m^{5/2}$/sec during simulation. For purposes of evaluating the proposed canal, it was assumed that the canal had a rectangular cross-section, a water depth of 3 meters (10 feet), and a length of either 200 meters (656 feet) or 75 meters (246 feet). The former value is the approximate length of a canal located in the northwest corner of the basin canal, while the latter is the length of a canal located at an alternative site near the end of the carrier-berth peninsula. The advantage of this latter site would be a shorter length canal with a smaller cross-section, due to the increased slope of the water in the canal. No attempt was made to simulate the change in cross-sectional area that would occur as a function of the tide, thus a conservative design approach would be to assume that the elevation of the base of the canal is -3 meters (-10 feet) MLLW.

In operating the simulation model, it soon became clear that it would be advantageous to open the canal only during periods of low suspended sediment concentration in the river. In practical terms, this meant delaying opening the gate for a short time following low tide and closing the gate while the sea level inequality was still positive. Figure 18 shows a plot of the suspended sediment concentration in the river as a function of time for 1 Apr 1984. The data shows that the sediment concentration reached a peak of around 0.05 gm/liter about an hour before low tide. The sediment concentration remained near this peak until low tide (1400 hours) at which point it began to decrease rapidly with time. A minimum concentration was reached around 1500 hours with the concentration remaining low until 1600 hours at which point it began to rapidly increase again.

The objective of opening the canal gate only when the sediment concentration is low is to reduce the amount of sediment that is introduced into the basin via the canal. Several different threshold values were tested in the model including: no threshold, 0.025, 0.020, and 0.015 gm/liter. Smaller threshold values indicate a shorter duty cycle time for the canal. Duty cycles corresponding to the above threshold values were: 4.33, 2.67, 2.0, and 1.17 hours respectively.

Figure 19 shows a plot of the relative shoaling rate versus Kprime for the case of no jetty. Each curve corresponds to a different threshold value of sediment concentration. The data in Figure 19 suggests that the optimum efficiency is obtained with a canal having a Kprime equal to approximately 450 $m^{5/2}$/sec and a sediment concentration threshold value of 0.02 gm/liter. For these conditions, the predicted efficiency of the canal was approximately 60%.

Figure 19 also shows that for the case of no threshold (the canal is opened at low tide and left open for 4.33 hours), a canal with a Kprime of 450 $m^{5/2}$/sec would increase the relative shoaling rate by approximately 300%. Alternatively, if the suspended sediment concentration of the surface river water which enters the canal was assumed to have a low constant value of 0.01 gm/liter (an assumption suggested in part by data from Van Dorn (1979)), the canal efficiency increases to 98%. Obviously, the canal's operation is very sensitive to the magnitude of the suspended sediment concentration in the river (relative to its value at the river/entrance channel interface) its variability with time, and its phase relationship with respect to the tide. Considering the limited data base from which to simulate the canal's operation, the sensitivity of the canal to data uncertainities translates into a significant degree of risk. This risk can be reduced by increasing the size and quality of the available data base.

Figure 20 shows a plot of Kprime versus canal width for a 75-meter long canal and a 200-meter long canal. The latter canal, which is currently proposed, will require a width of 15.5 meters (51 feet) for optimum operation. The corresponding width for the shorter alternative canal is 10.5 meters (34 feet). Both canals assume a water depth of

3 meters (10 feet). Obviously, the actual canal depth will have to be deeper to accommodate any tidal variation. The elevation of the base of the canal should be no higher than -3 meters MLLW.

The flow rate in the canal will vary as a function of time because of the variation in the total dynamic head. The latter, and thus the canal flow rate, is a maximum at low tide and decreases to zero approximately 3 hours later. For the recommended canal, the peak flow rate is 92 m^3/sec (1,400,000 gpm). The corresponding head driving the flow is 4.2 cm. Note that the shorter canal can have a smaller cross-section because the slope of the water surface in the canal is greater.

Although the original intent of the study was to evaluate the effect of a venting canal on the rate of sedimentation in the turning basin at Mayport Naval Station, the simulation model was easily adaptable to evaluating the effects of a jetty extension.

Figure 21 shows a plot of the relative shoaling rate as a function of the jetty length for the case of no canal. The present model suggested that for jetty lengths of up to 300 meters, the effect was negligible. As the jetty length was increased to 400 meters and greater, however, the shoaling rate dropped dramatically. The predicted efficiency of a jetty 400 meters long was approximately 85%.

Another alternative which was evaluated was to combine a canal with a jetty. Figure 22 shows a plot of the relative shoaling rate as a function of Kprime for different jetty lengths. The suspended sediment concentration threshold value used to generate Figure 22 was 0.020 gm/liter. The figure shows that for a given-sized jetty, the relative shoaling rate decreases as the canal size is increased (increasing Kprime). Selecting the relative merits of a canal, a jetty, or a combination of the two, however, requires some consideration of the cost of the different options.

Although a complete economic analysis of the different alternatives was beyond the scope of this study, it was possible to look at the estimated first cost versus the degree of sedimentation prevention (based on cost estimates in U.S. Army Corps of Engineers (1979) and Jenkins et al, (1983). The cost of a jetty was estimated to be $30K per meter of length, the cost of a 75-meter long canal was estimated to be

$150K per meter of width and the cost of a 200-meter long canal was estimated to be $250K per meter of width. The results of the cost analysis are shown in Figure 23, which assumes a 200-meter long canal with a suspended sediment concentration threshold value of 0.02 gm/liter.

Figure 23 shows that the lowest cost per cubic meter of sediment prevented ($13.50/m^3) occurred with a Kprime of 400 m$^{5/2}$/sec and no jetty. (The corresponding cost for a 75-meter long canal was $5.50/m^3.) Adding a jetty decreased the net sediment influx for a given-sized canal, however, the cost rose substantially. For example, a 200-meter long canal with a Kprime of 400 m$^{5/2}$/sec and a jetty length of 100 meters had a cost of $30.00/m^3.

DISCUSSION

The model used to simulate the effects of a canal or jetty is simplistic and should not be viewed as particularly accurate. One of the greatest uncertainties was in estimating the shoaling rate in the entrance channel. In the present model, the shoaling in the entrance channel was assumed to be constant. This would tend to increase the efficiency of the jetty relative to the canal. Another uncertainty is to what degree the present data set is representative of average annual conditions. Small departures from the measured conditions could have a significant effect on the predicted results. Unfortunately, the limited data base precluded a better assessment of the relative risks associated with building a canal (or a jetty).

Because of the simplicity of the model, the predicted shoaling reductions should be viewed as a relative measure of the effectiveness of the different options, and not as absolute values.

On the basis of cost alone, the canal appears to be the option of choice. The canal option, however, also involves the highest degree of uncertainty (risk). A brief discussion of the physical basis for each option helps to clarify this point. Physically, a jetty acts to increase the amount of time it takes for the sediment-rich river water at the river/entrance channel interface to travel from the interface into the basin. While delaying the river water from reaching the basin, the basin is filling with the relatively sediment-poor water which was located in the entrance channel at the start of flood tide. If the jetty is long enough (400 meters or greater), then the river water never reaches the basin and most of the sedimentation is prevented. Of course some sedimentation would occur in the entrance channel, however, the present study assumed that subsequent ebb channel currents would resuspend all of this sediment.

A canal also increases the travel time for the sediment-rich river water to reach the basin. The travel time is increased because the flood velocities in the entrance channel are reduced by the canal flow. Unfortunately, the canal flow also brings some sediment into the basin; however, providing the sediment concentration at the surface of the river is less than that at the river/channel interface, a net reduction in the sediment flux is achieved. Alternatively, if, as in the present case, the sediment concentration in the river varies significantly with the phase of the tide, then a net reduction can still be achieved. The latter method, however, depends critically on the phase relationships between the tide, the river/interface sediment concentrations, and the sea level inequality. The sensitivity of the canal to variations in the relative sediment concentration levels is underscored by the fact that when the duty cycle of the canal gate was decreased by as little as 1 hour, the net sediment flux changed from a 300% increase to a 60% decrease.

The decision of whether to build a canal, to build a jetty, or to do nothing involves assessing the relative risk versus the relative potential economic payoff. Because of the limited data base and the sensitivity of the canal to environmental variability, there is presently a high degree of risk associated with building the canal. The canal, however, has the highest potential economic return. The jetty is less sensitive to environmental uncertainties, (i.e., less risky) however, it is an order of magnitude more costly. In terms of the seriousness of the shoaling problem at NAS Mayport, either solution may be preferable to doing nothing. A more in-depth assessment of the overall problem is warranted.

The principle reason for the high degree of uncertainty in the proposed venting canal is the limited environmental data base. At present there are approximately 3 days of data (most of it good) for the water level inequality between the river and the basin. There are approximately 10 to 12 hours of sediment concentration and sediment flux data that are closely correlated with the tide. Some of these data are ambiguous because of an uncertain time base (U1 and U3) and because of insufficient sampling rates (C1, C2, C3, C4). The greatest difficulty, however, is simply a lack of data on river and entrance channel sediment concentrations as a function of the tide.

The data gathered in the present study appear to be representative of relatively mild shoaling conditions. An important question is: how different are the conditions during periods of heavy shoaling? If the sediment concentrations in the river do not vary significantly with time, and if they are approximately the same magnitude as that at the interface, then the canal will fail to reduce sedimentation in the basin. On the other hand, if sediment concentrations are generally low at the surface of the river relative to the interface, the canal may function much better than expected. A larger body of data covering the sediment concentrations in the river and in the basin as a function of time is needed in order to resolve these uncertainties. If the the canal is eventually built, this body of data will be of value in determining the proper duty cycle for the canal gate.

CONCLUSIONS AND RECOMMENDATIONS

1. The venting canal proposed for the northwest corner of the basin is too narrow, too shallow, and requires an automatic gate. Based on the results of this study, a canal in this location should have a width of 15.5 meters, a water depth of 3 meters, and a base depth of -3 meters MLLW. The canal, when fitted with an automatic gate, is expected to reduce the overall shoaling rate by 60%.

2. The effectiveness of the canal is particularly sensitive to the relative magnitudes of sediment concentrations in the river and at the river/channel interface. The phase relationships between these sediment concentrations, the sea level inequality, and the tide are also important. Small changes in these variables were found to have a significant effect on the estimated efficiency of the canal. Because of an insufficient environmental data base, there is a signficant degree of uncertainty associated with the project. Additional concentration data should be gathered in order to reduce this uncertainty and to select an optimum duty cycle for the canal gate.

3. Consideration should be given to placing the canal near to the end of the carrier berth peninsula. This location would allow the canal length to be shortened to approximately 75 meters and the width narrowed to 10.5 meters. The cost of a canal in this location would be significantly less, with no reduction in the estimated efficiency.

4. A 400- to 500-meter long jetty, extending seaward from the carrier-berth peninsula, appears to present a lower risk than the venting canal. A jetty of this size would produce an estimated 85% reduction in the sedimentation rate. Unfortunately, the cost of a 600 meter long jetty has been estimated to be $20M. Moreover, a jetty is not without risks, particularly with respect to the uncertainty in the shoaling rate which will occur in the entrance channel. Low cost methods of jetty construction should be investigated. If costs could be reduced by 50%, a jetty would become an attractive option.

5. Because of the model's simplistic treatment of the interfacial sediment concentrations during flood and the basin sediment concentrations during ebb, the projected benefits of the different shoaling reduction options should be viewed as only a relative measure of their effectiveness. Quantitative estimates of shoaling reductions will require a more detailed environmental measurement program, and a more sophisticated numerical model.

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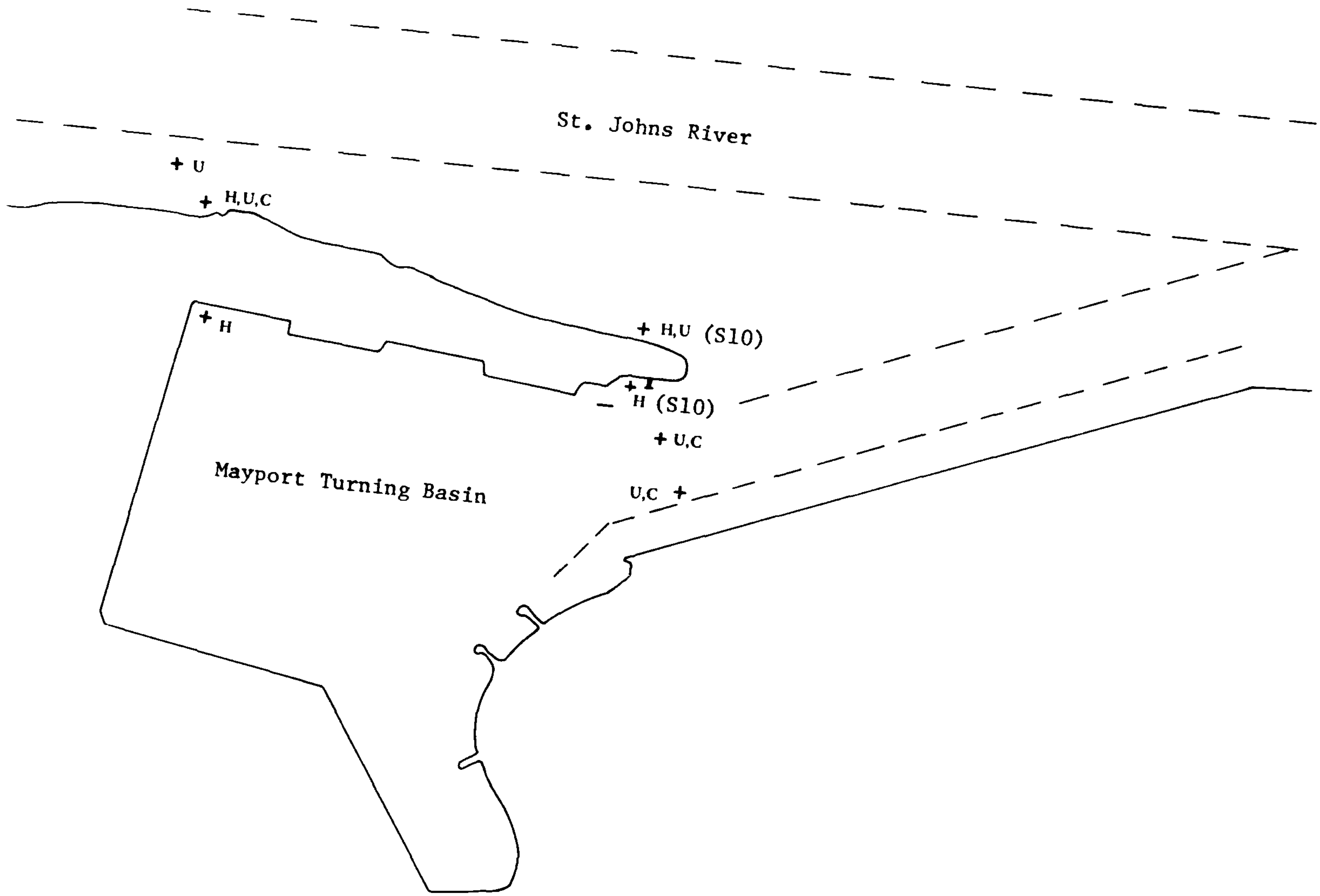

Figure 1. Plan view of Mayport turning basin showing the location of the field measurements. The symbols u, c, and h refer to velocity, sediment concentration, and pressure measurements. SIO refers to measurements made by Jenkins et al., (1983).

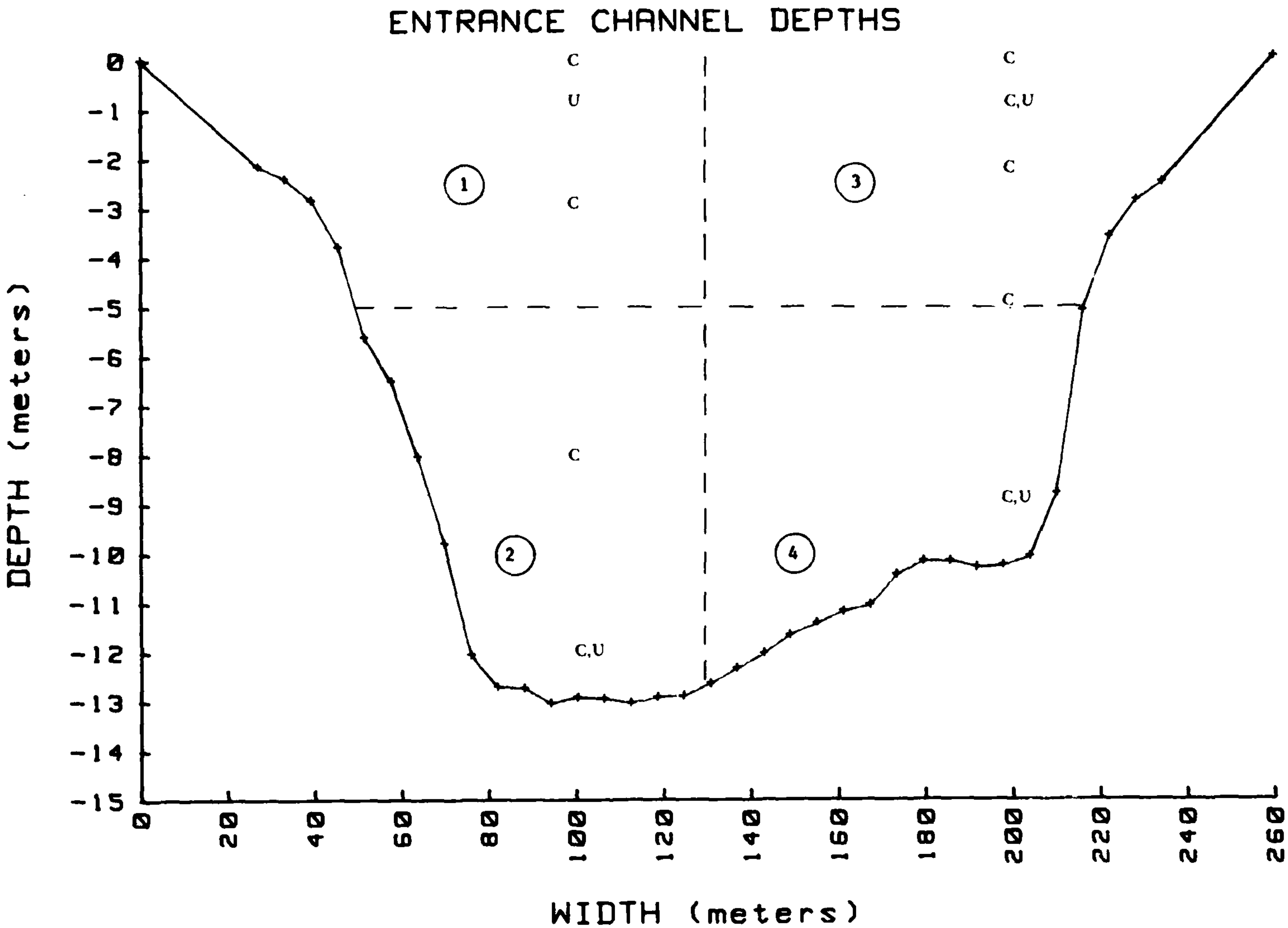

Figure 2. Entrance channel cross-section showing the location of the velocity (*u*) and sediment concentration *measurements*. The numbers 1-4 refer to sector areas which are used in estimating the entrance channel flux.

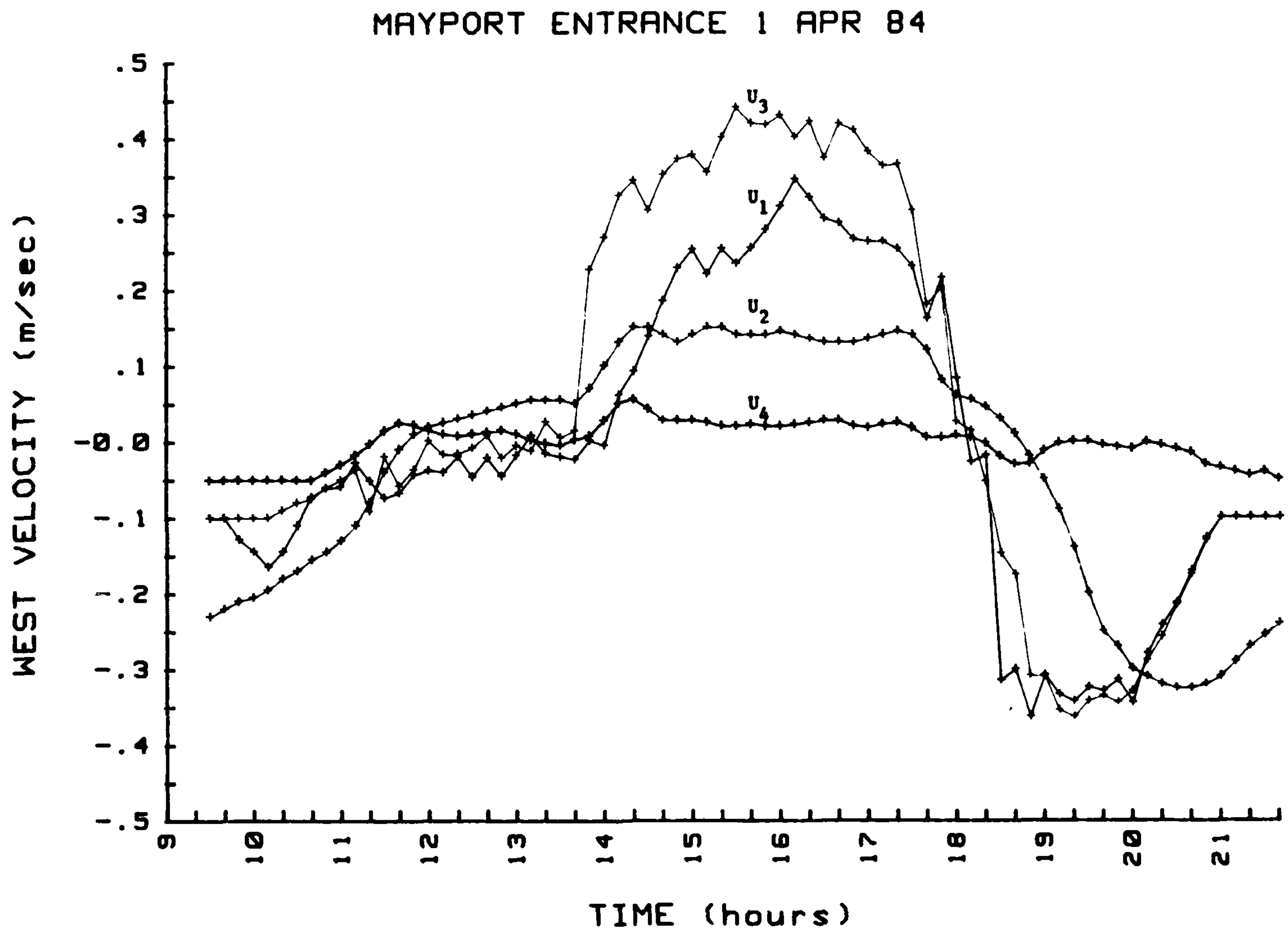

Figure 3. Measured surface (U1, U3) and bottom (U2, U4) velocities in the entrance channel to Mayport turning basin on 1 April 1984.

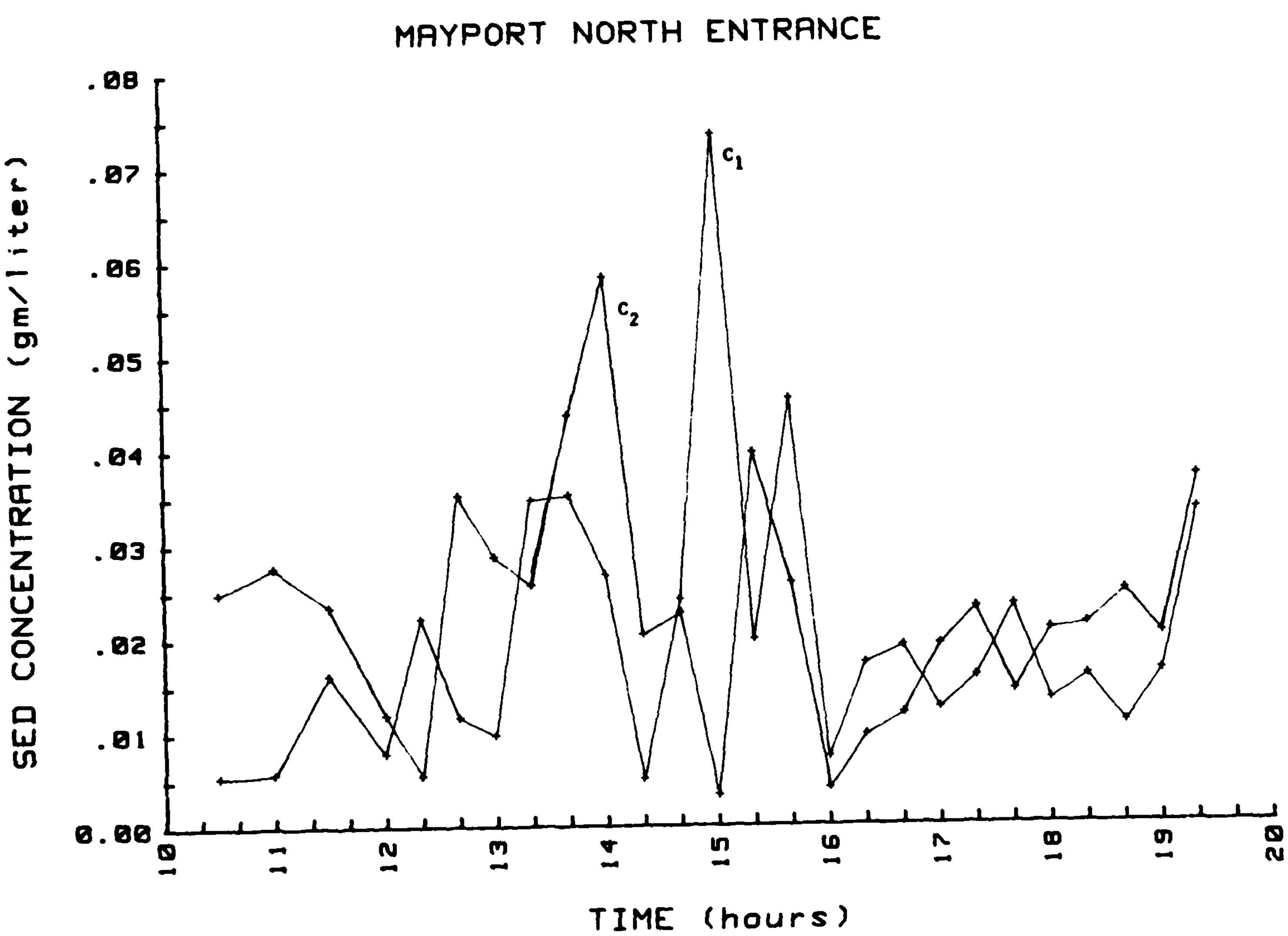

Figure 4. Measured surface (C1) and bottom (C2) sediment concentrations in the entrance channel to Mayport turning basin (north side) on 1 April 1984.

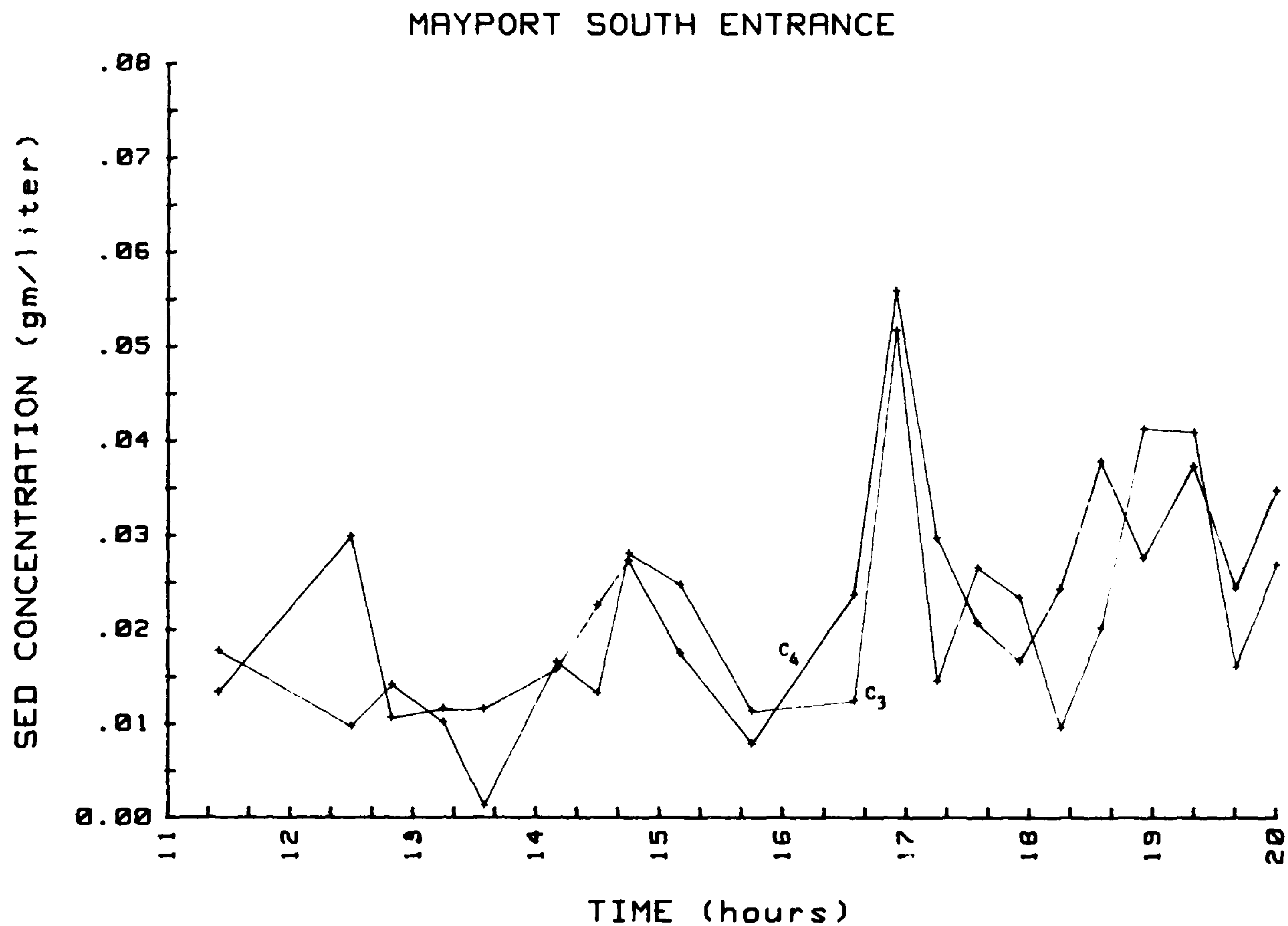

Figure 5. Measured surface (C3) and bottom (C4) sediment concentrations in the entrance channel to Mayport turning basin (south side) on 1 April 1984.

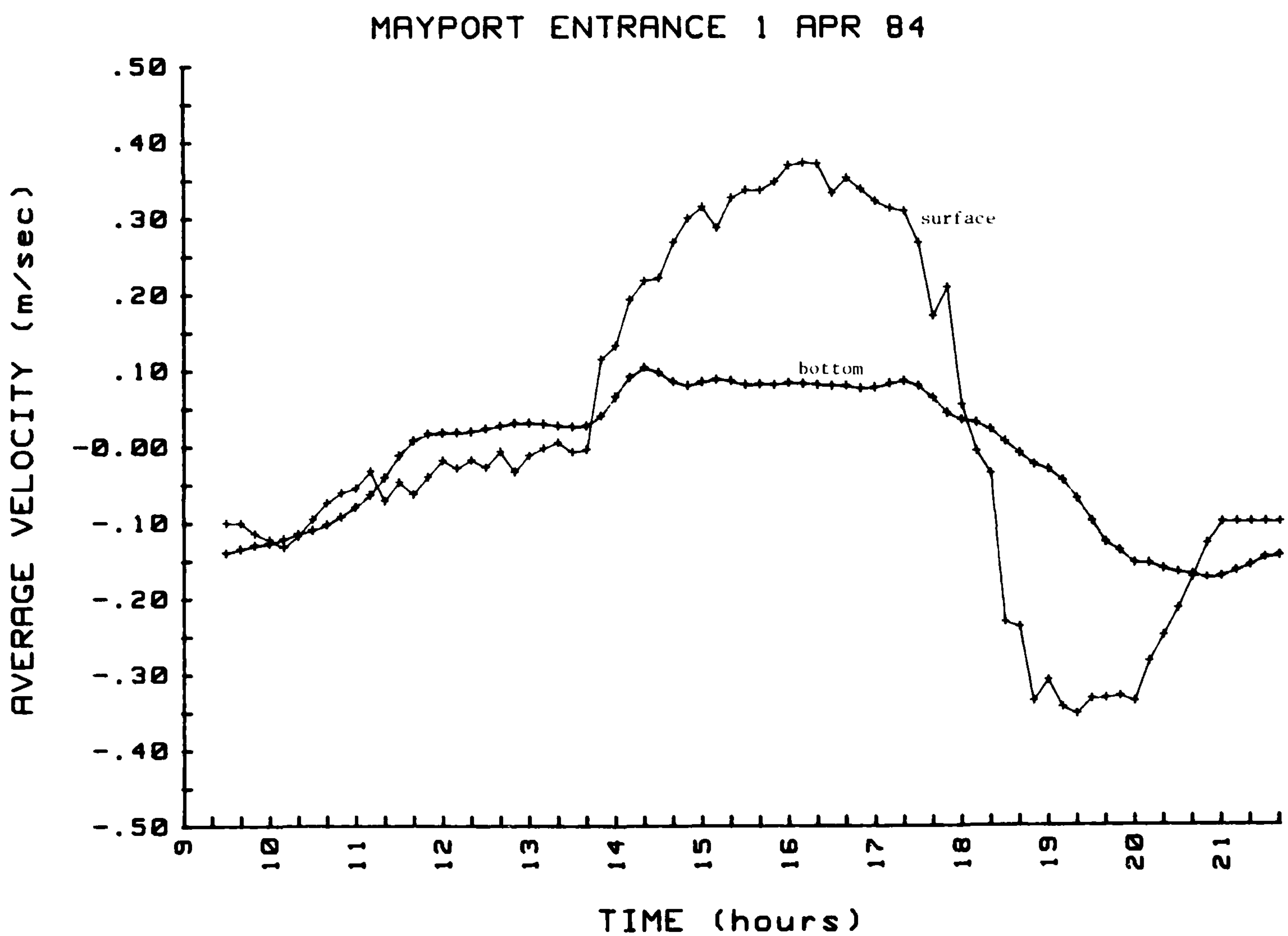

Figure 6. Spatially averaged surface and bottom currents in the entrance channel to Mayport turning basin on 1 April 1984.

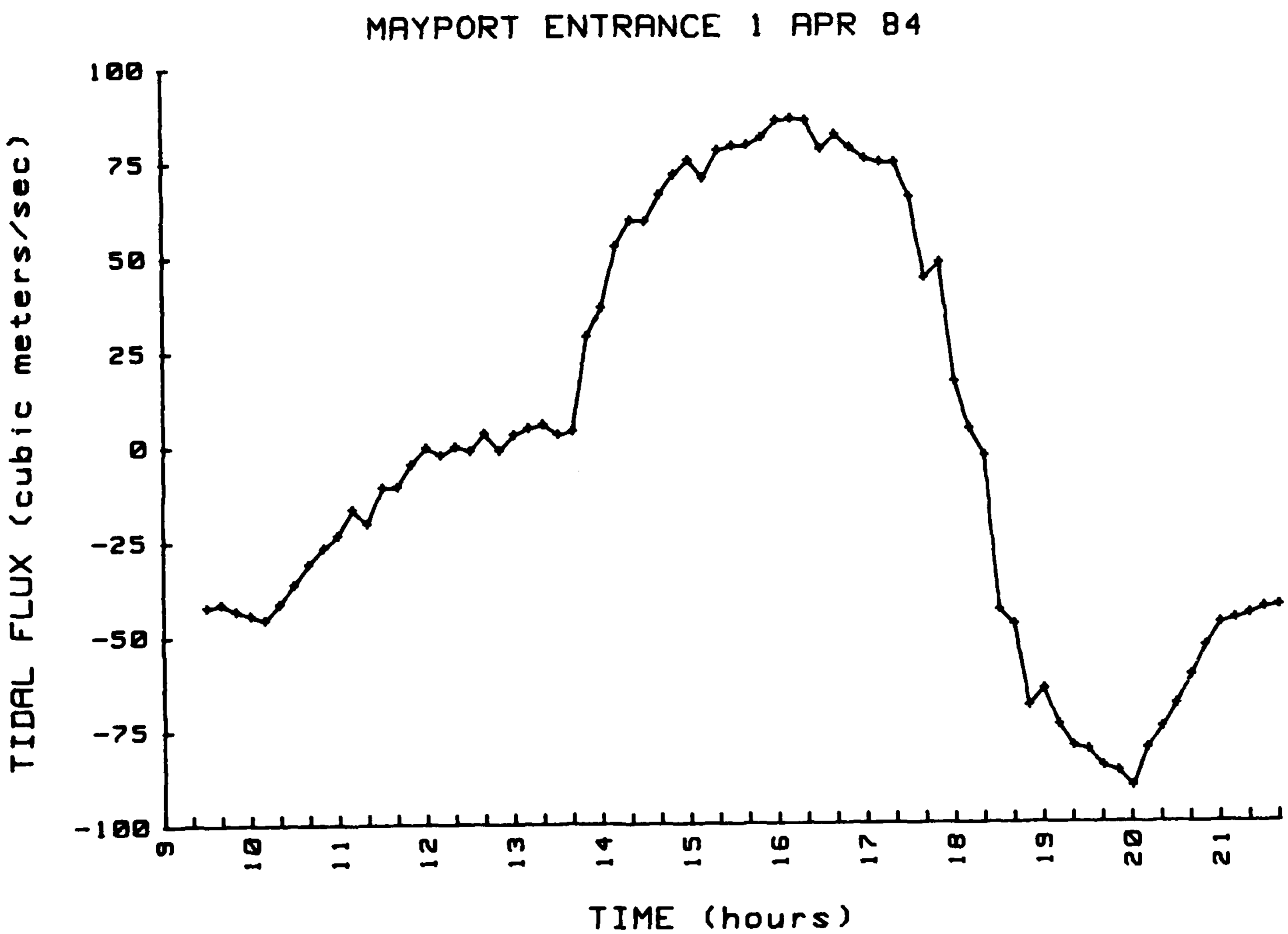

Figure 7. Estimated tidal volume flux (positive into basin) in the entrance channel to Mayport turning basin on 1 April 1984.

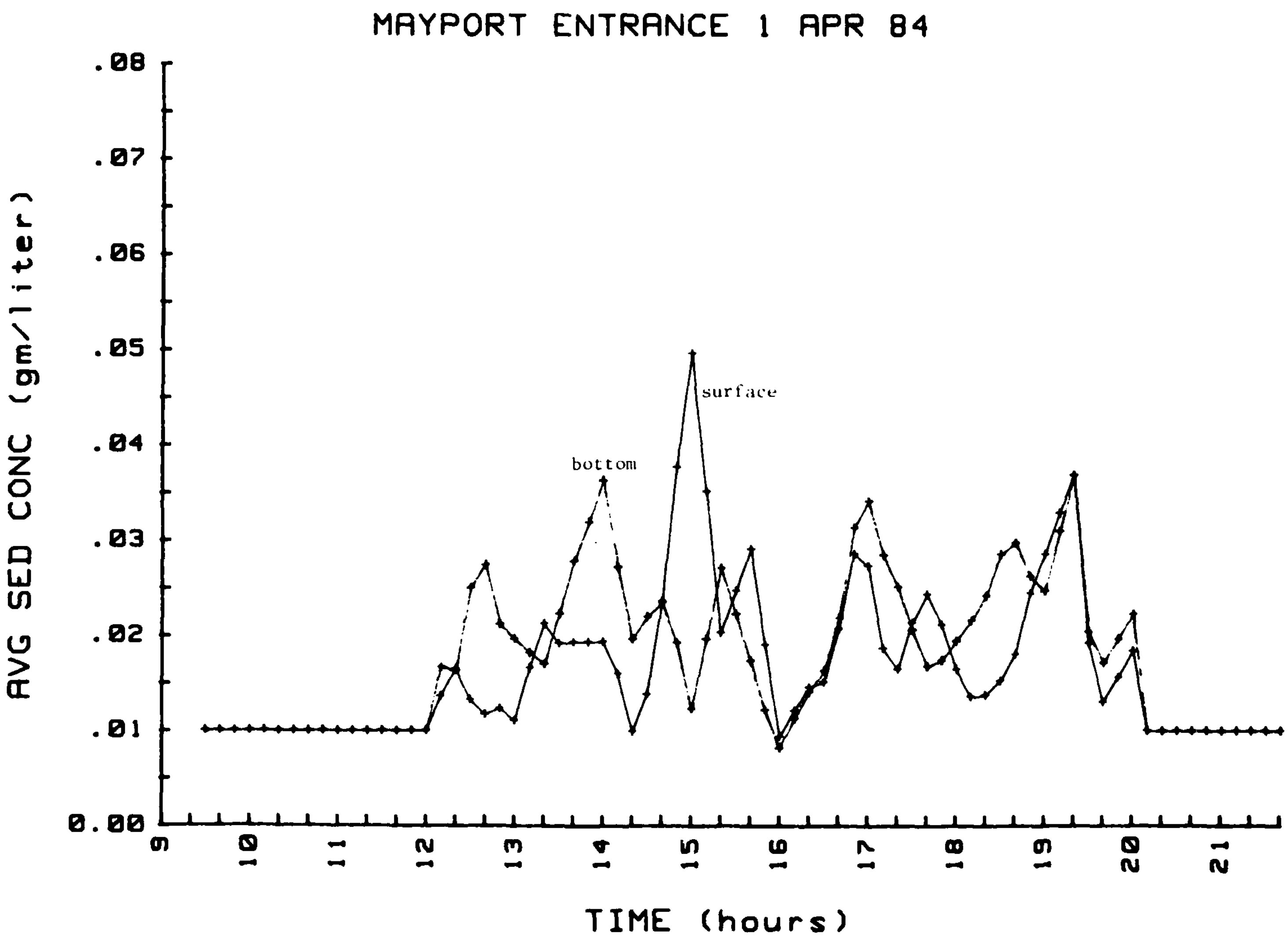

Figure 8. Spatially averaged surface and bottom sediment concentrations in the entrance channel to Mayport turning basin on 1 April 1984.

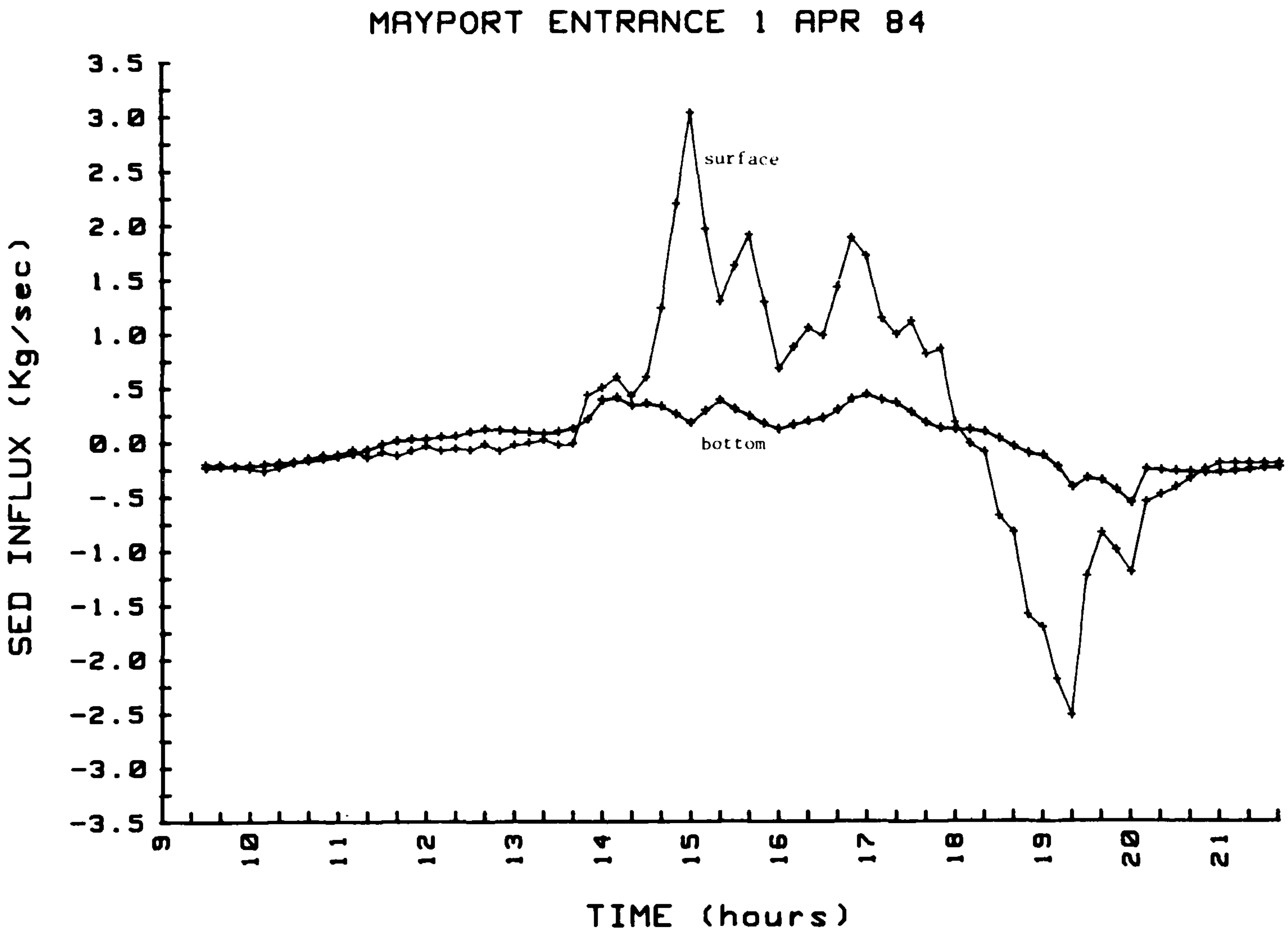

Figure 9. Estimated surface and bottom sediment fluxes (positive into basin) in the entrance channel to Mayport turning basin on 1 April 1984.

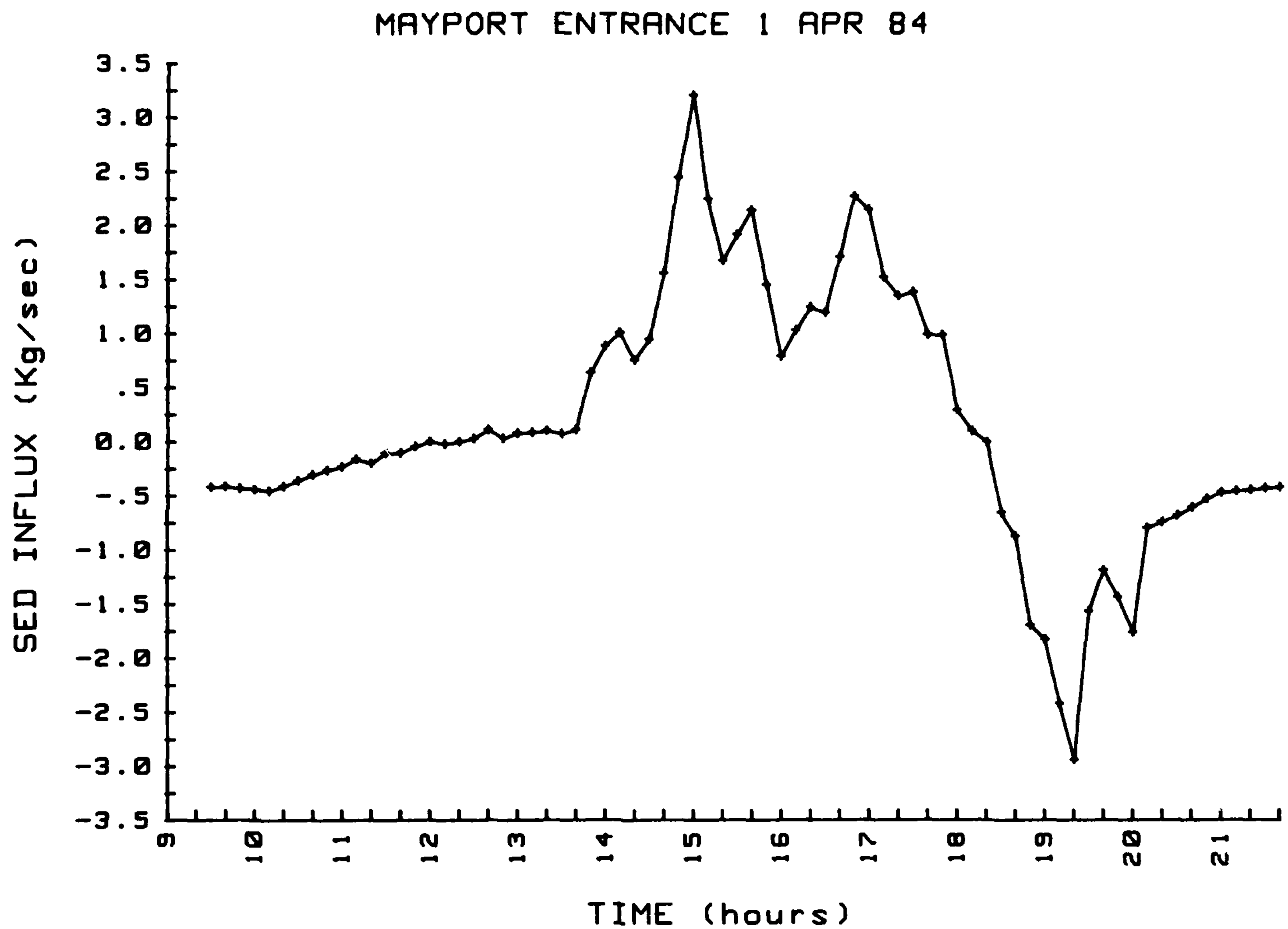

Figure 10. Estimated sediment flux (positive into basin) in the entrance channel to Mayport turning basin on 1 April 1984.

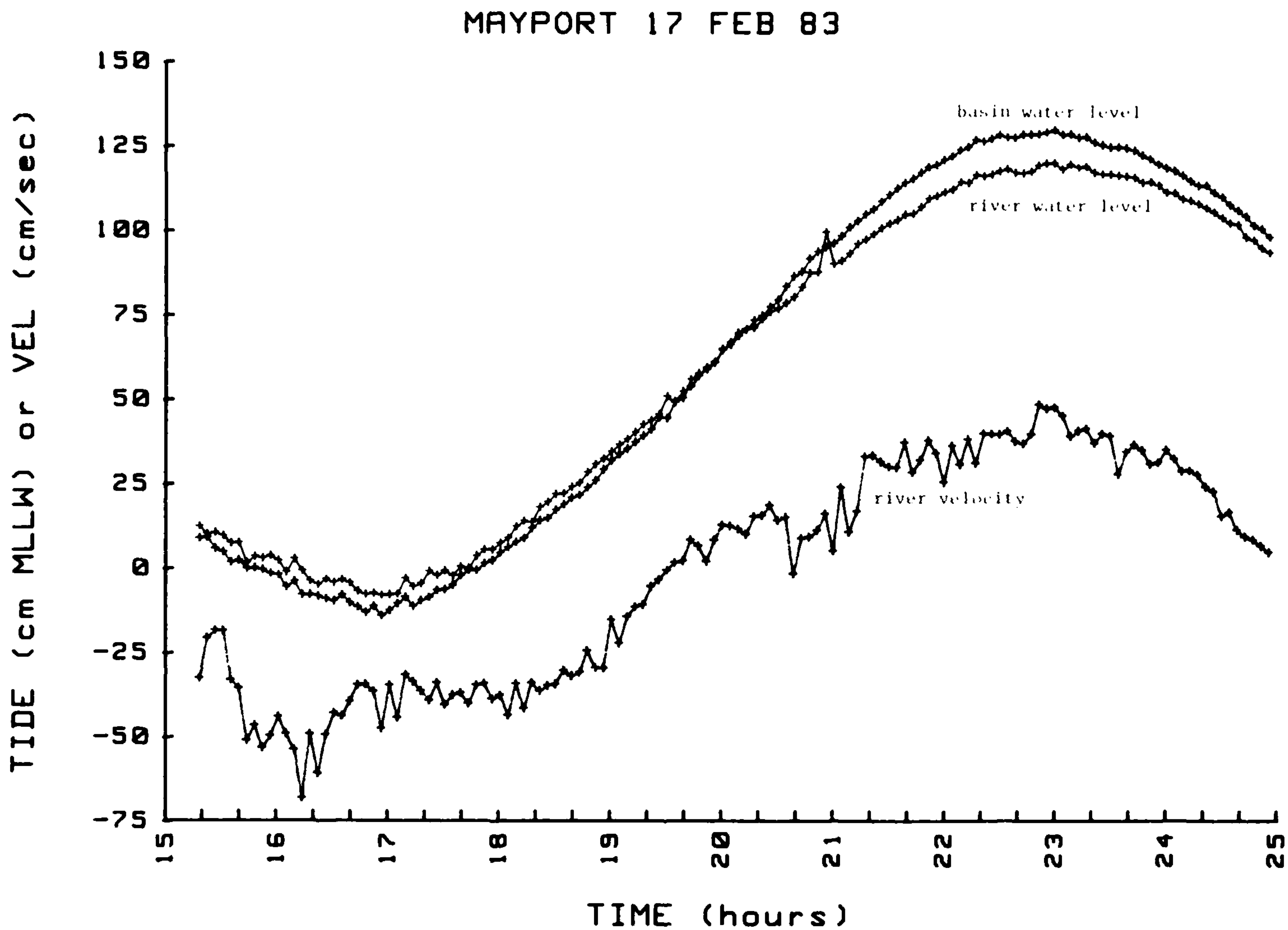

Figure 11. Measured water levels in the St Johns River and Mayport turning basin, and river velocities on 17 Feb 1983 (Jenkins et al., 1983).

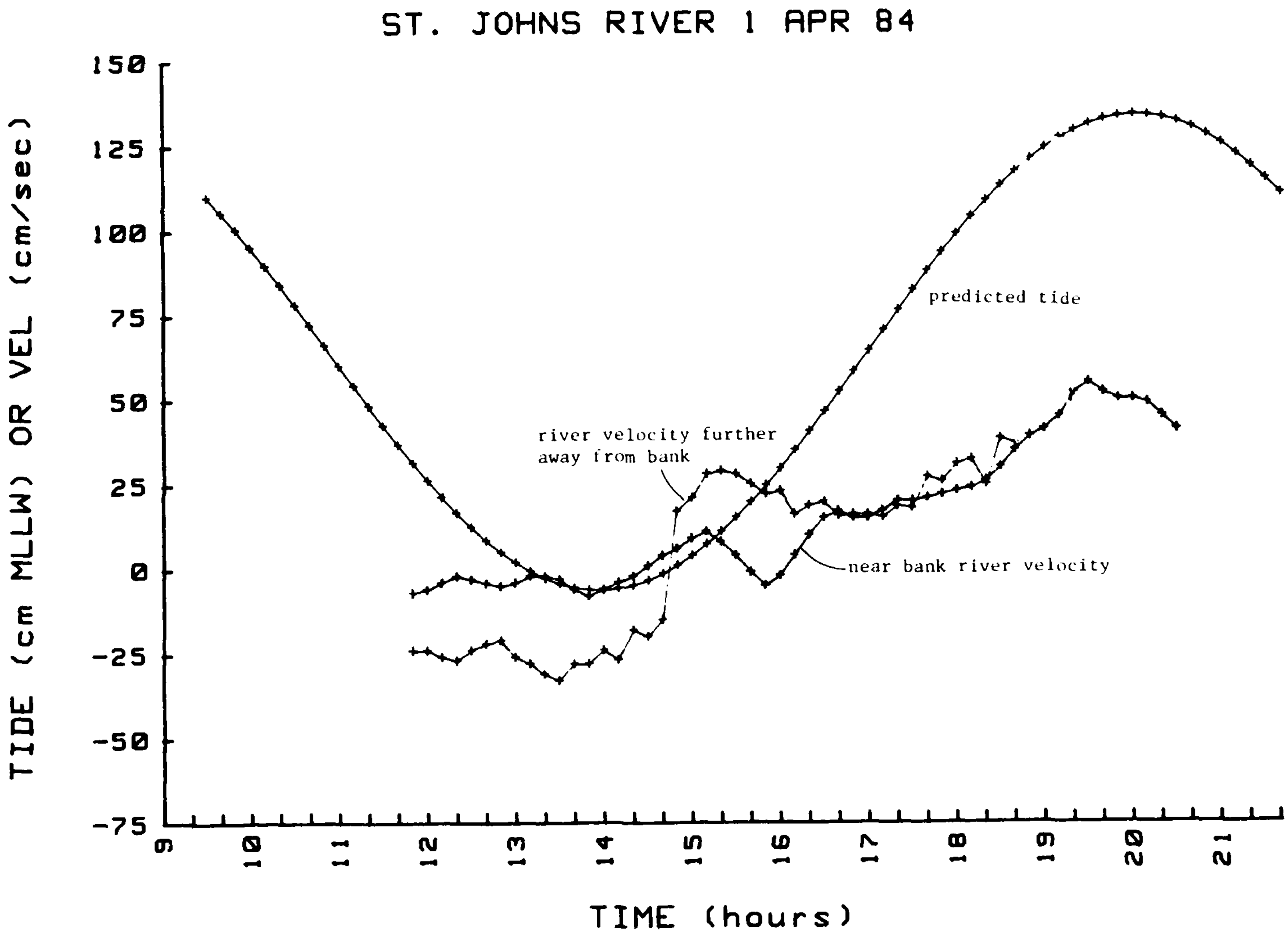

Figure 12. Predicted tide elevation and measured near-bank velocities in the St Johns River on 1 April 1984.

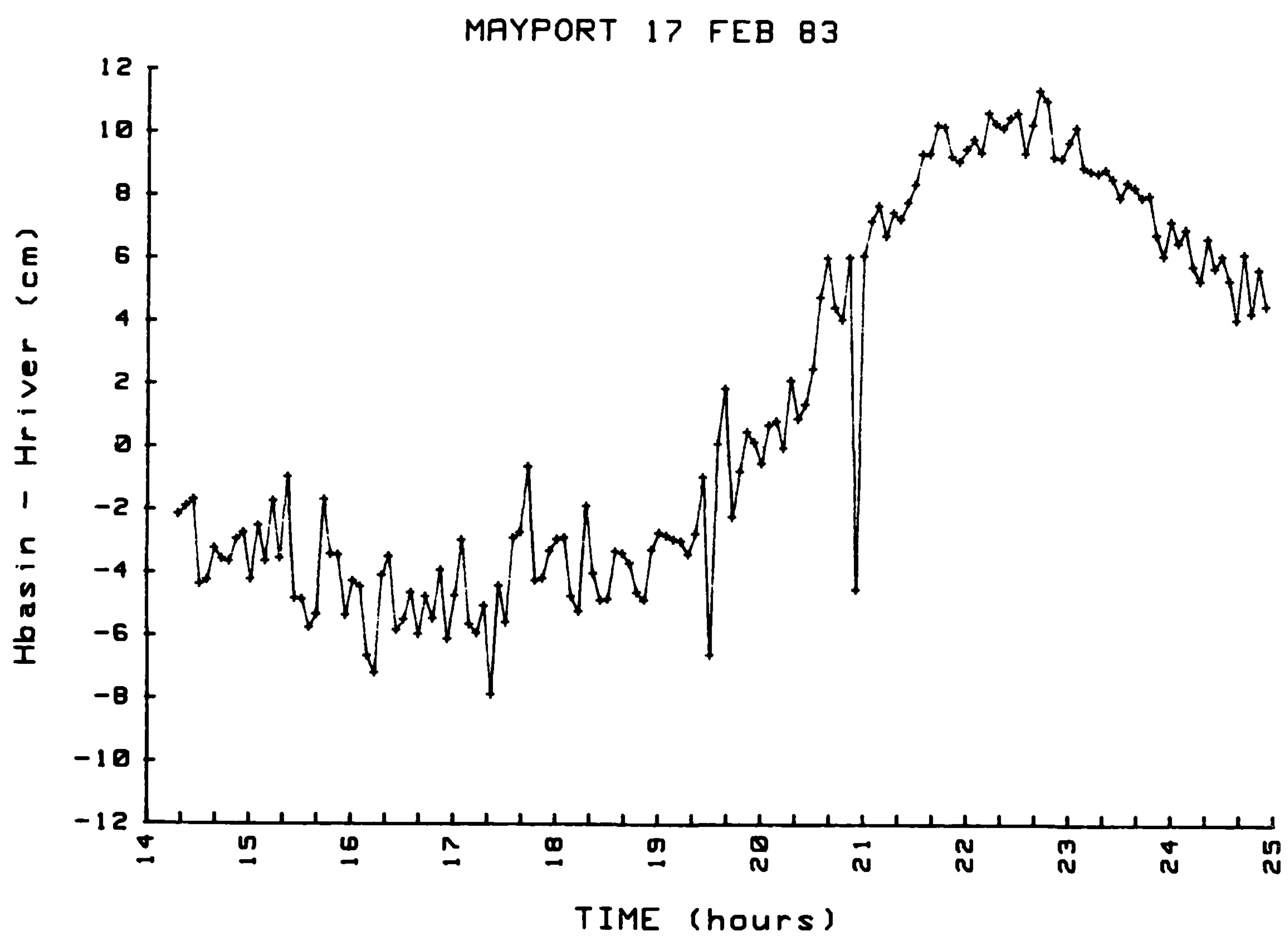

Figure 13. Measured sea level inequality (basin-river) between Mayport turning basin and the St Johns River on 17 Feb 1983 (Jenkins et al., 1983).

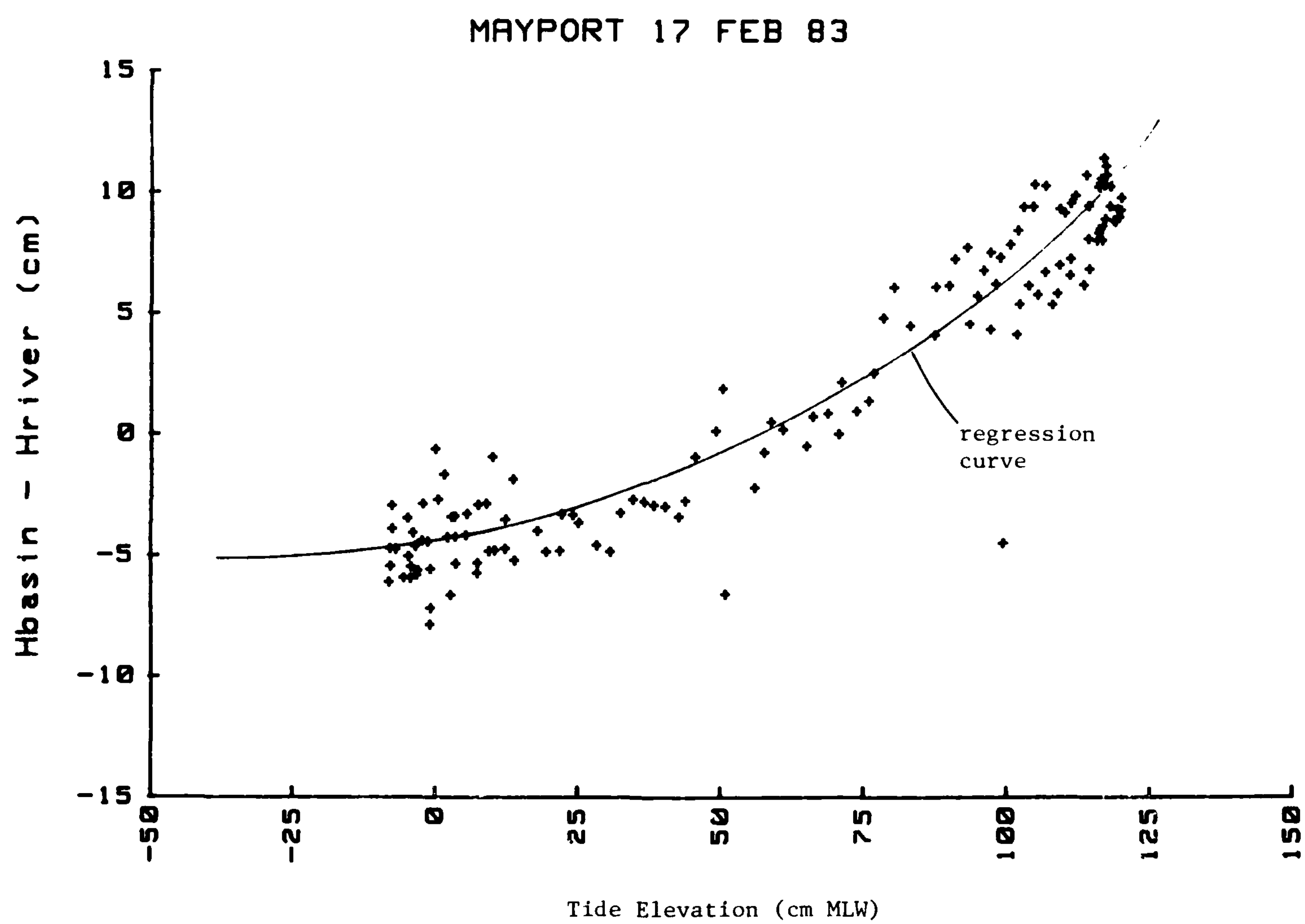

Figure 14. The inequality between the basin and river water levels (basin-river) was found to be correlated with the tide elevation in the St Johns River. The solid curve is a least squares fit to the measured data.

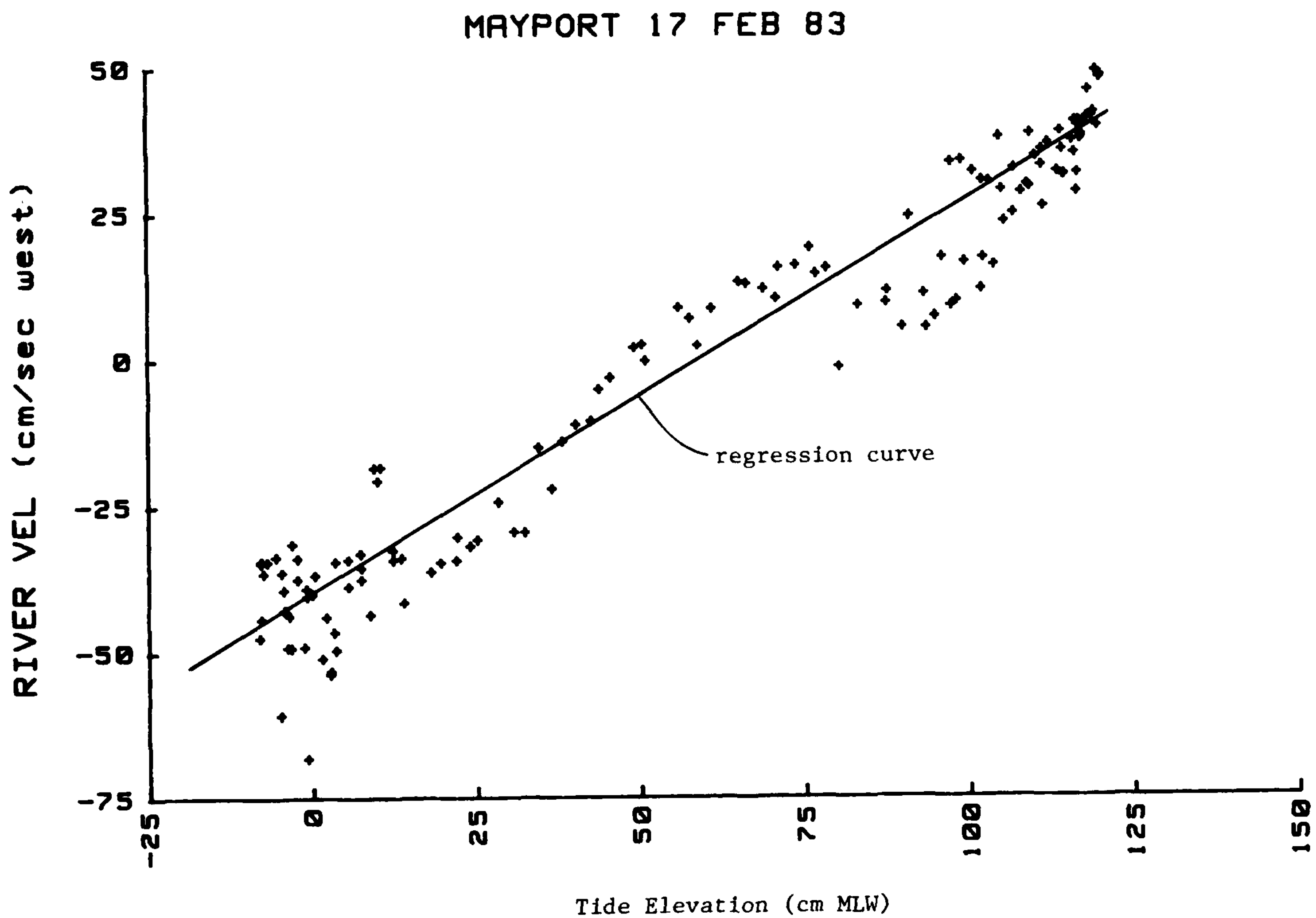

Figure 15. The near-bank velocity in the St Johns River was found to be correlated with the tide elevation in the river. The solid curve is a least squares fit to the data.

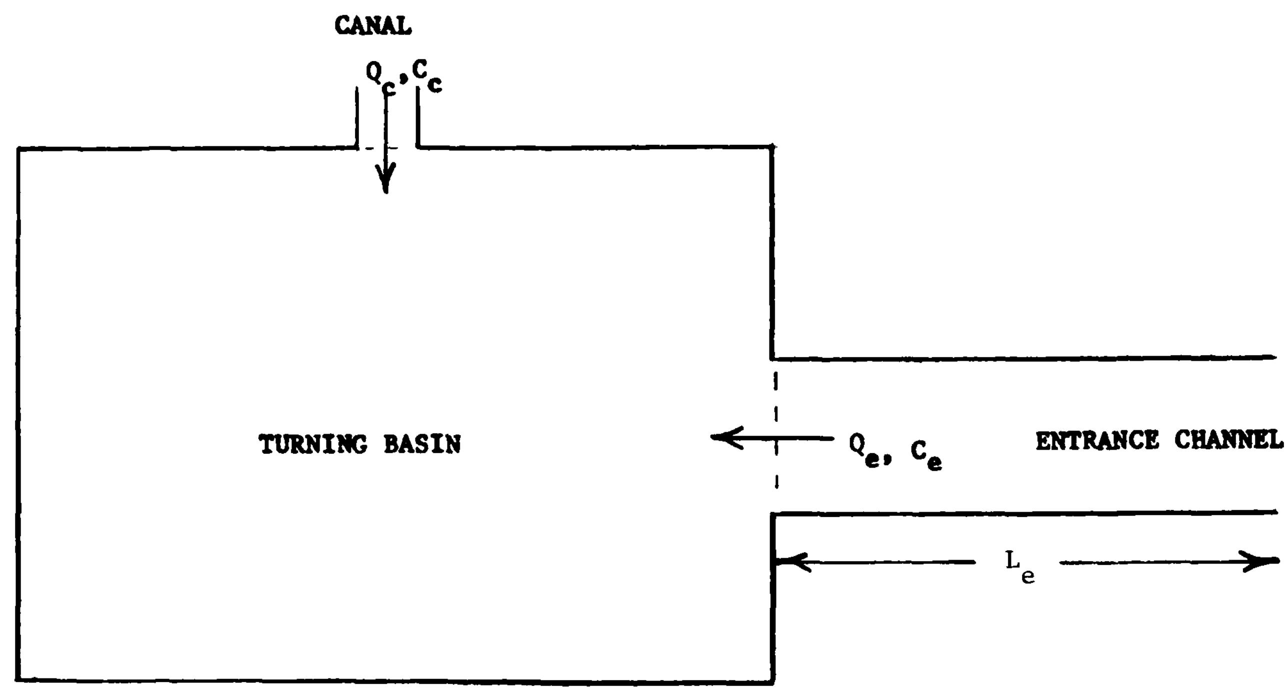

Figure 16. Schematic diagram of the box model used to simulate the effects of a venting canal and/or jetty extension on the rate of sedimentation at Mayport turning basin.

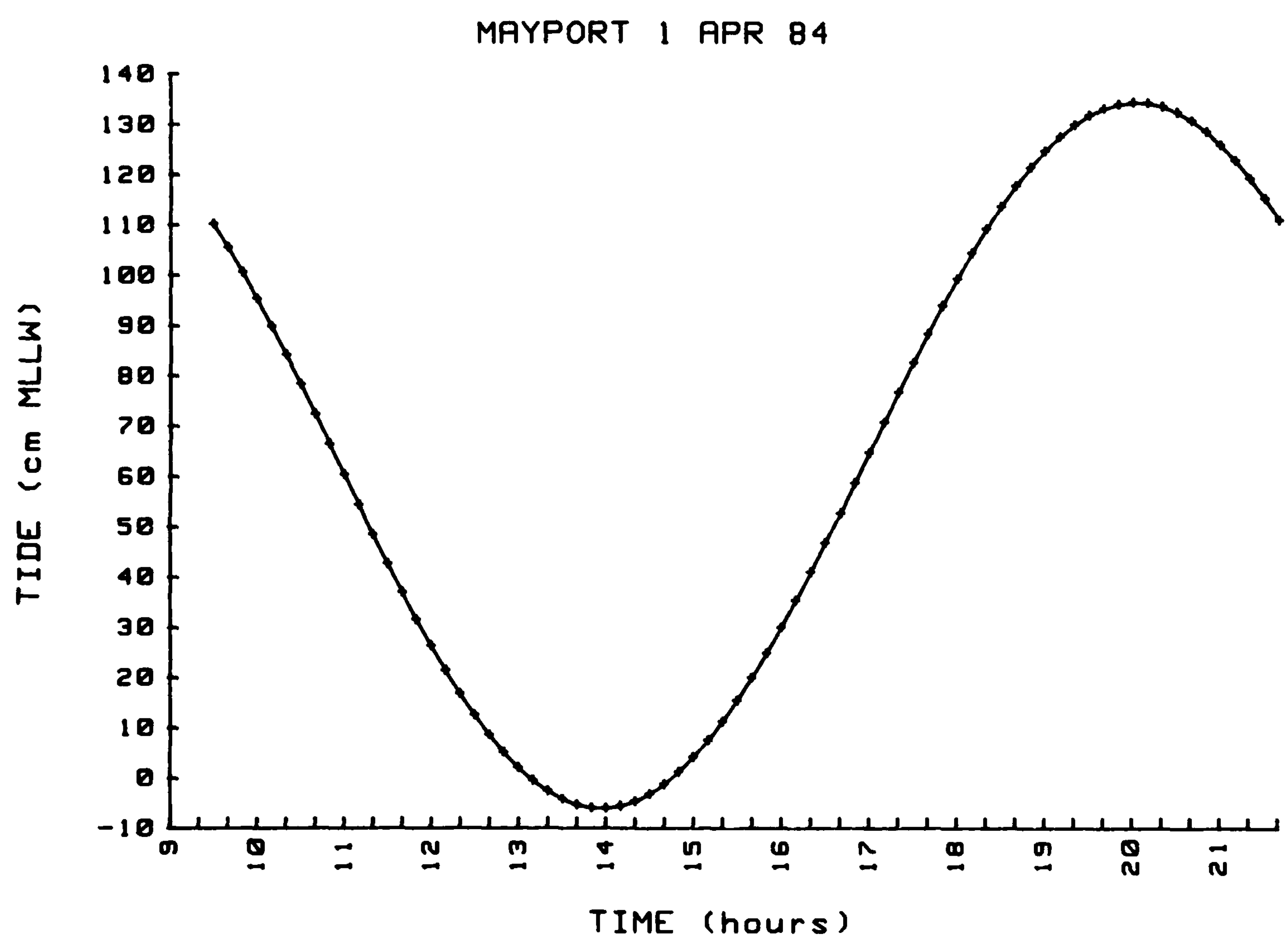

Figure 17. Predicted tide elevation in the St Johns River at Mayport Naval Station on 1 April 1984.

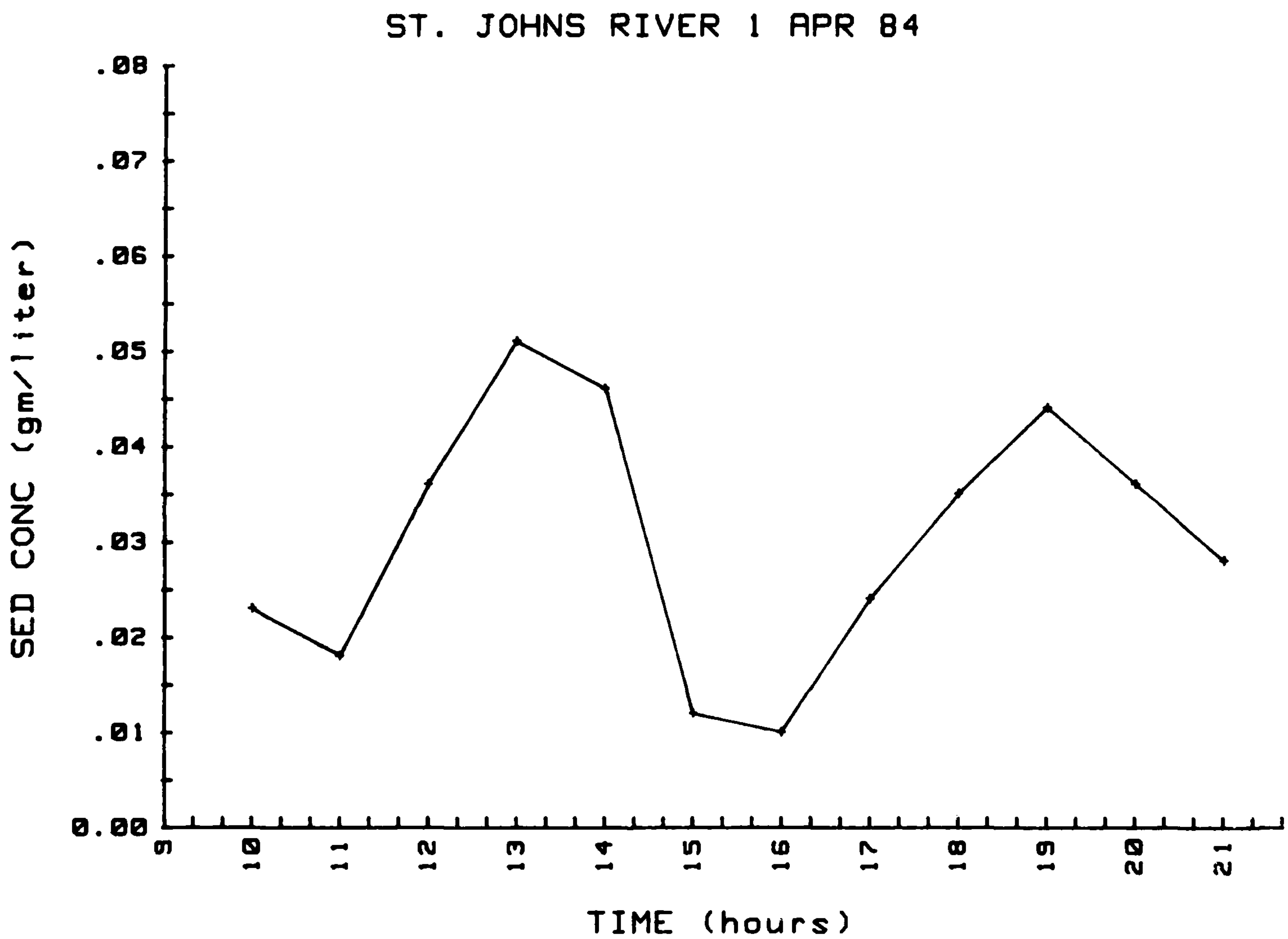

Figure 18. Measured near-bank surface sediment concentration in the St Johns River on 1 April 1984.

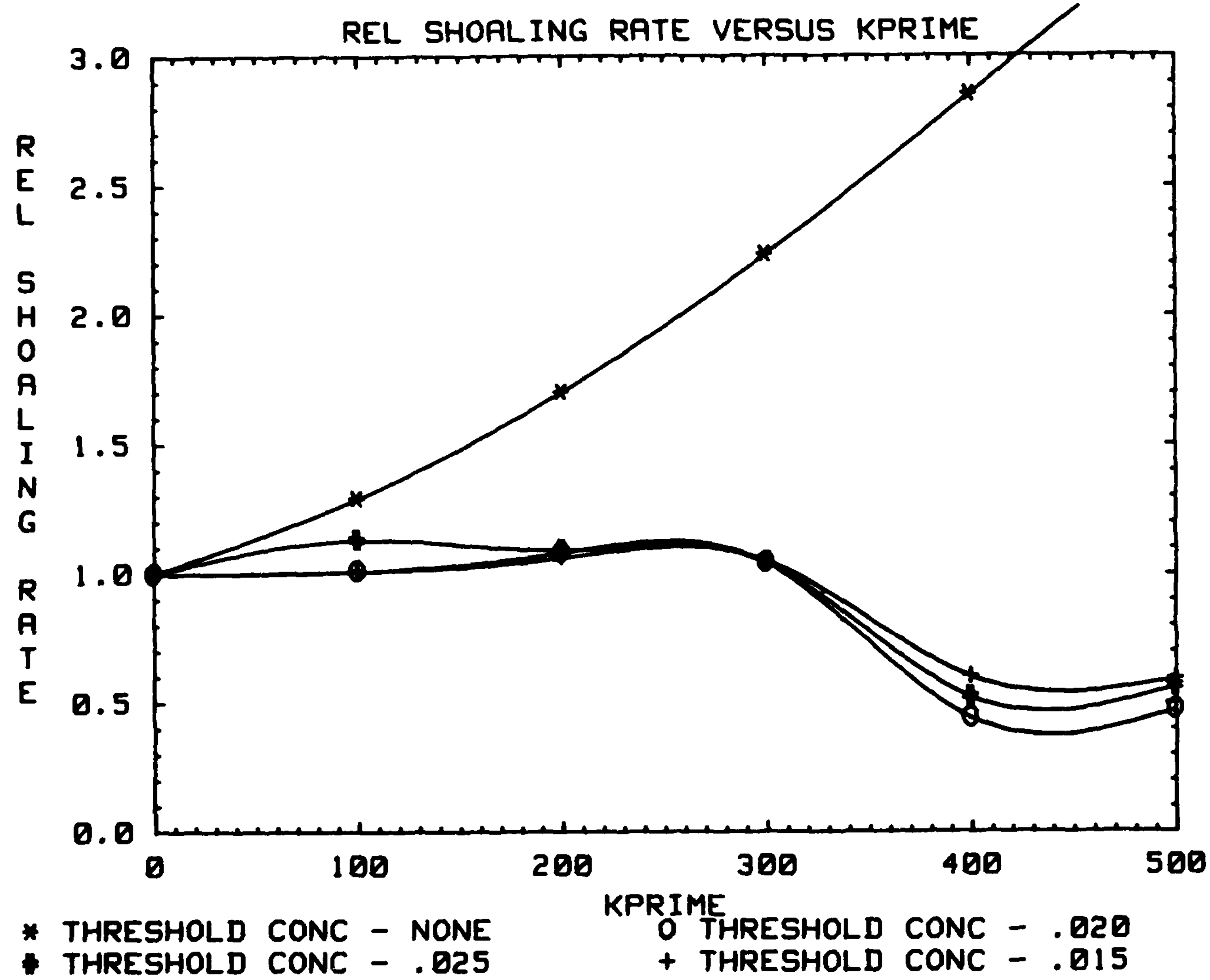

Figure 19. The estimated relative shoaling rate as a function of canal size (Kprime) and duty cycle for the canal gate (assumes no jetty). The duty cycle is adjusted relative to a threshold sediment concentration as depicted in Figure 18.

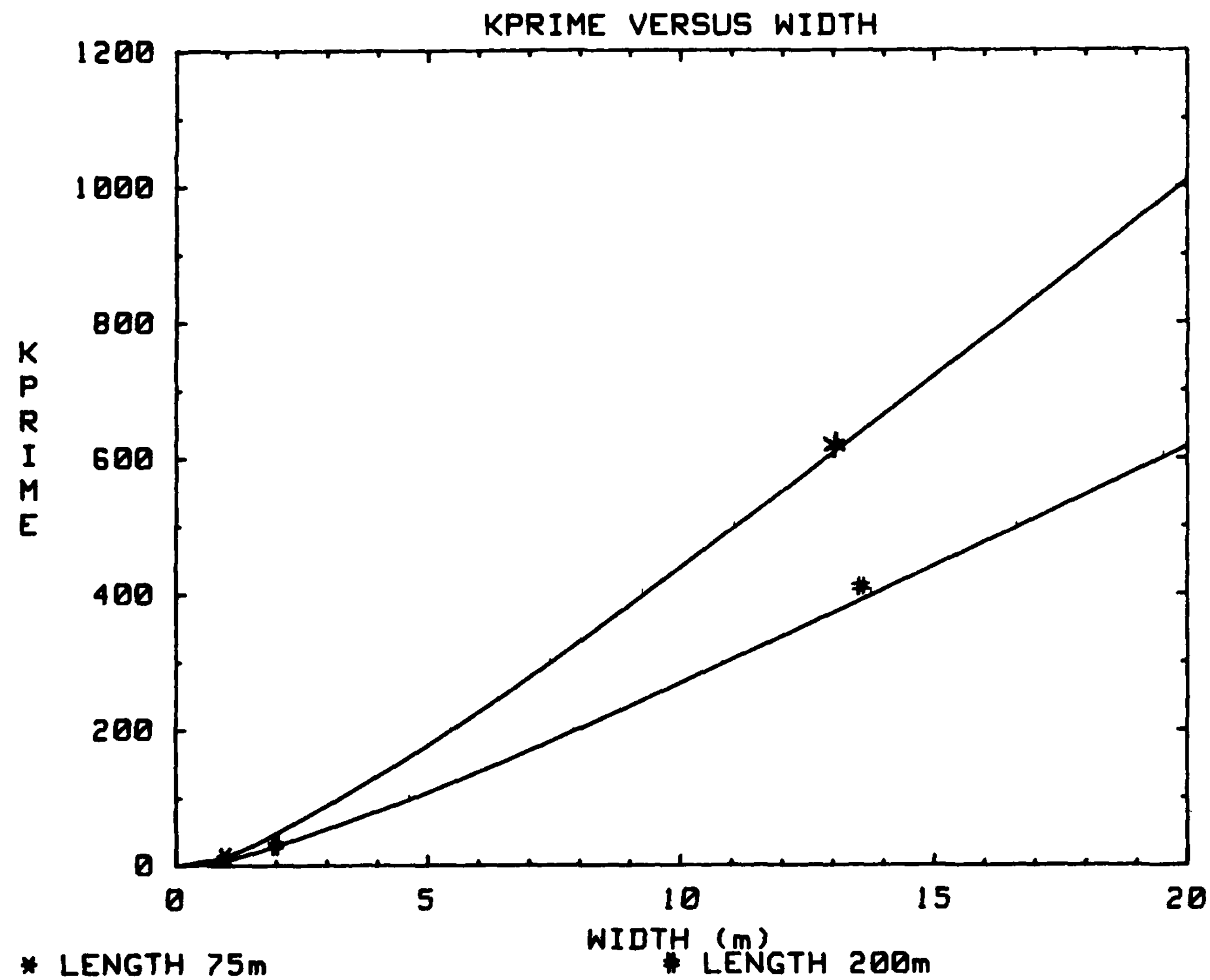

Figure 20. Kprime versus canal width for canal lengths of 75 and 200 meters assuming a 3-meter water depth and rectangular cross-section.

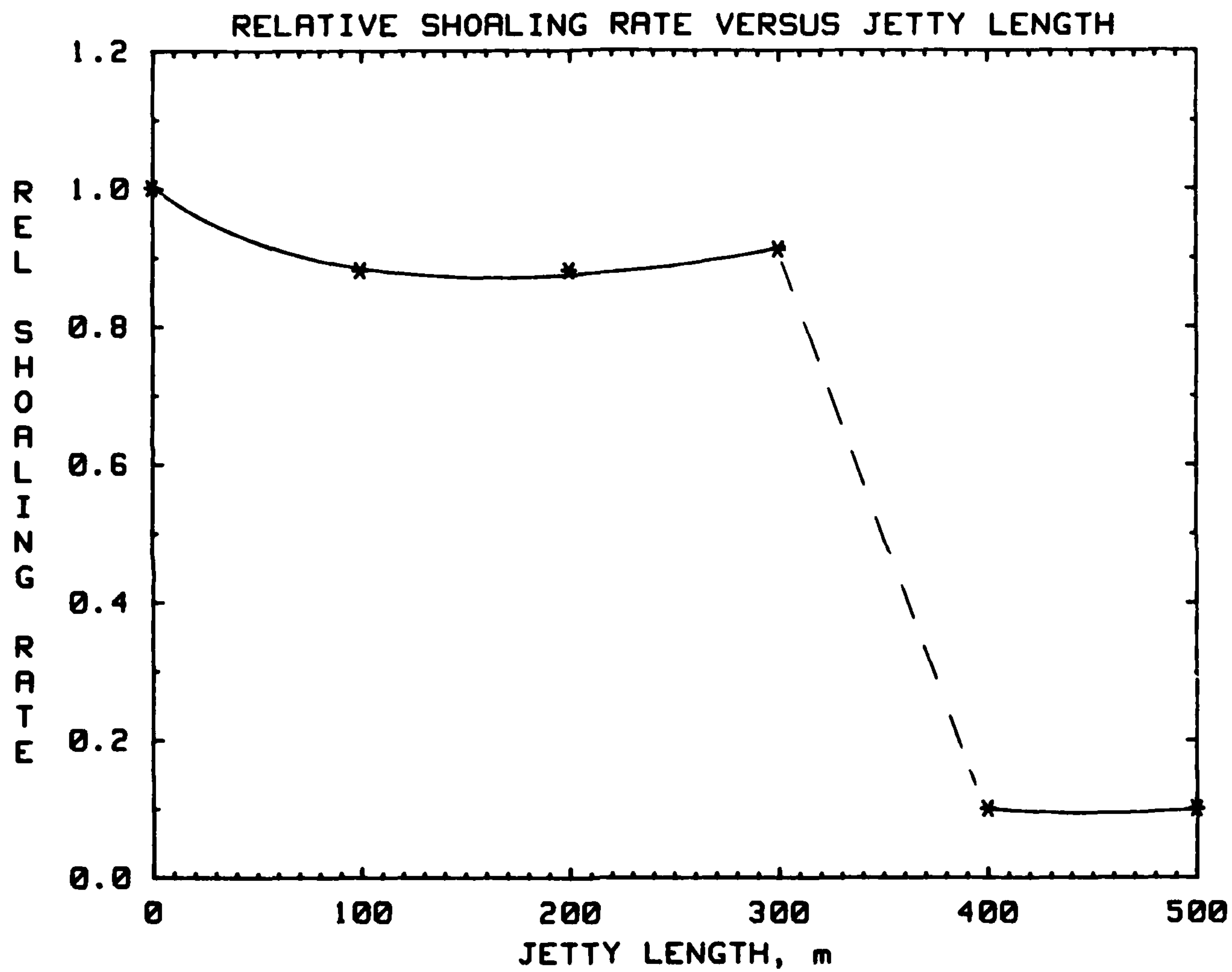

Figure 21. Estimated relative shoaling rate as a function of jetty length (no canal). The minimum acceptable jetty length is 300 to 400 meters.

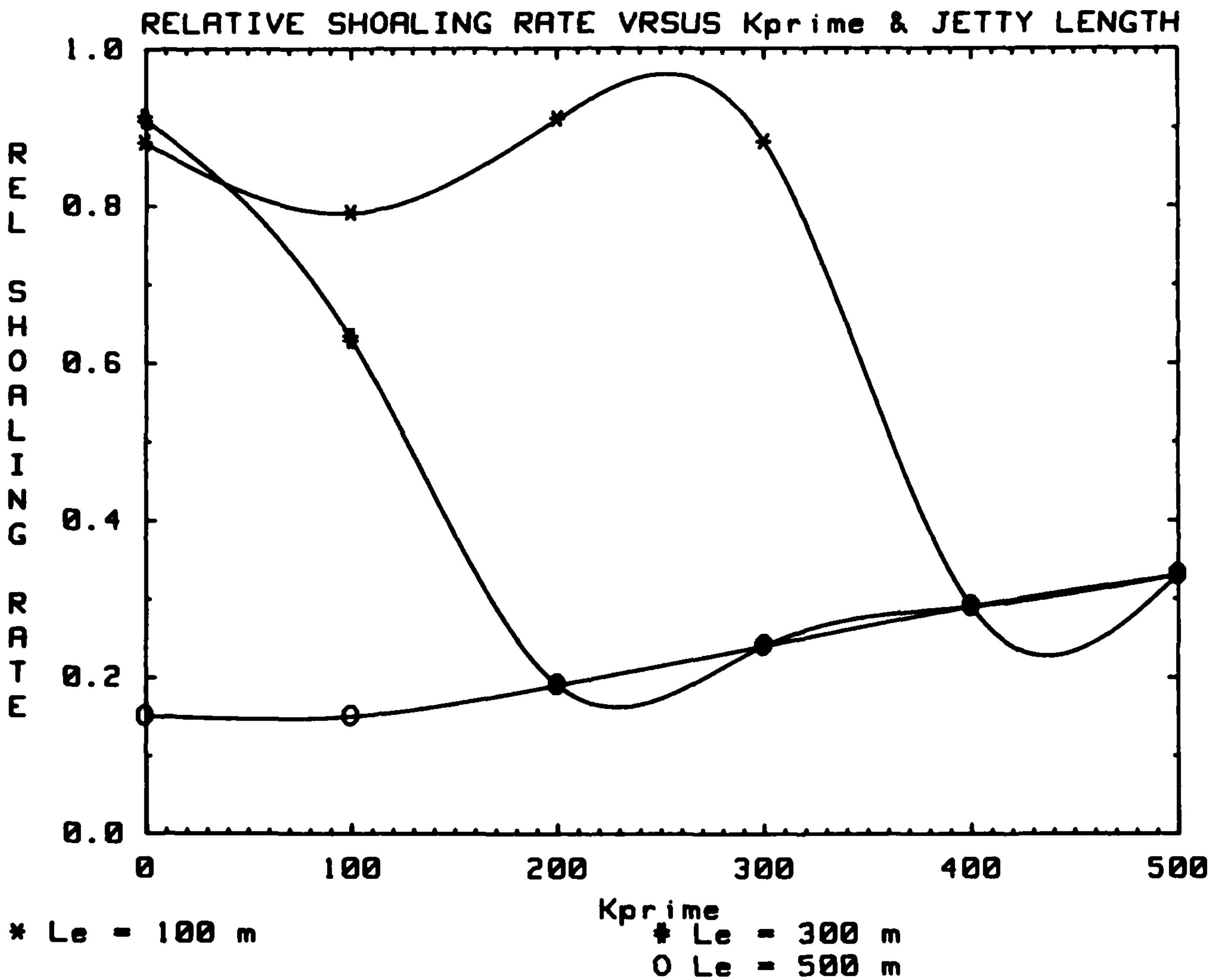

Figure 22. Estimated relative shoaling rate versus canal size (Kprime) for different jetty lengths. A threshold sediment concentration of 0.02 gm/liter was assumed. The minimum shoaling rate occurs with a 500 meter long jetty and no canal.

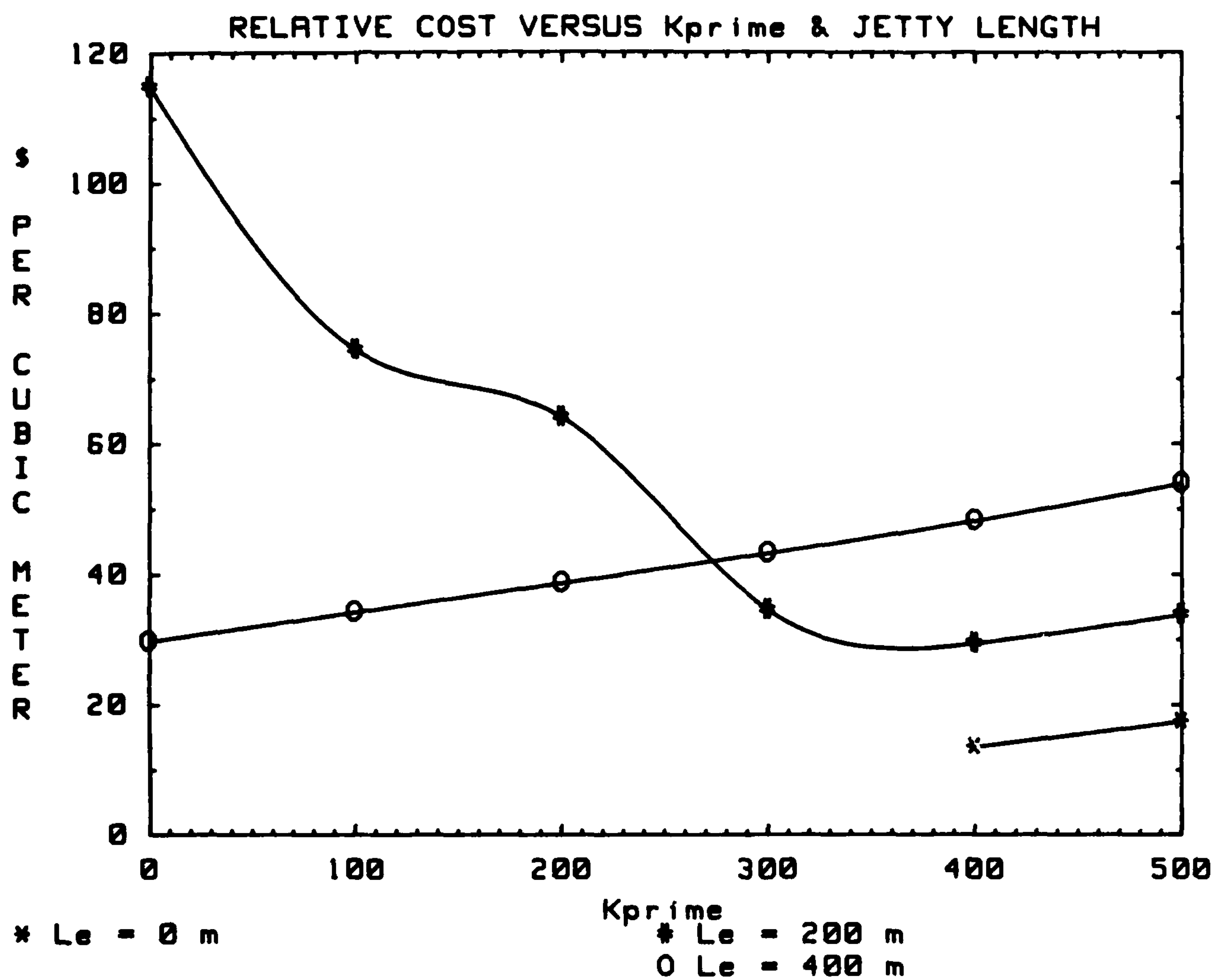

Figure 23. Estimated first cost per cubic meter of reduced annual sedimentation volume versus canal size (Kprime) and jetty length. A threshold sediment concentration of 0.02 gm/liter was assumed. The lowest cost option is a canal without a jetty extension.

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29 Thermal conservation (thermal engineering of buildings, HVAC systems, energy loss measurement, power generation)
30 Controls and electrical conservation (electrical systems, energy monitoring and control systems)
31 Fuel flexibility (liquid fuels, coal utilization, energy from solid waste)
32 Alternate energy source (geothermal power, photovoltaic power systems, solar systems, wind systems, energy storage systems)
33 Site data and systems integration (energy resource data, energy consumption data, integrating energy systems)
34 **ENVIRONMENTAL PROTECTION**
35 Solid waste management
36 Hazardous/toxic materials management
37 Wastewater management and sanitary engineering
38 Oil pollution removal and recovery
39 Air pollution
40 Noise abatement
44 **OCEAN ENGINEERING**
45 Seafloor soils and foundations
46 Seafloor construction systems and operations (including diver and manipulator tools)
47 Undersea structures and materials
48 Anchors and moorings
49 Undersea power systems, electromechanical cables, and connectors
50 Pressure vessel facilities
51 Physical environment (including site surveying)
52 Ocean-based concrete structures
53 Hyperbaric chambers
54 Undersea cable dynamics

TYPES OF DOCUMENTS

85 Techdata Sheets
86 Technical Reports and Technical Notes
82 NCEL Guide & Updates
☐ None— remove my name
83 Table of Contents & Index to TDS
91 Physical Security

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