NOTES ON THE HYDRAULICS OF HAMILTON HARBOUR

P.F. Hamblin

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Lakes Research Branch
National Water Research Institute
Canada Centre for Inland Waters
Burlington, Ontario L7R 4A6
Canada

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ABSTRACT

Renewed interest in the exchange of restricted nearshore waters with the open areas of large lakes sparked by the designation of areas of concern has led to attempts to understand these exchanges on a quantitative basis. This report deals with four aspects of the exchange problem: the measurement of complex exchange flows, the modelling of unsteady one-layer exchanges driven by water level differences and winds, the extension of maximal two-layer thermally driven exchange flow theory to include frictional effects and the quantification of the inflow of heat to restricted water bodies from their tributaries. Model results are compared to observations and recommendations for further studies are made.

RÉSUMÉ

Un regain d'intérêt pour les échanges entre les eaux riveraines confinées et les zones libres des grands lacs, suscité par la désignation de secteurs préoccupants, a conduit à tenter de comprendre ces échanges sous l'angle quantitatif. Le présent rapport traite de quatre aspects du problème de l'échange : la mesure du flux d'échange complexe, la modélisation d'échanges instables sur une couche engendrés par les vents et les différences de niveau d'eau, l'extension de la théorie du flux d'échange maximal d'origine thermique sur deux couches pour y inclure les effets de friction, et la quantification de l'apport thermique par leurs affluents aux masses d'eau confinées. On compare les résultats des modèles aux observations et on fait des recommandations en vue d'études ultérieures.

MANAGEMENT PERSPECTIVE

This report is based on a series of graduate lectures at the Department of Civil Engineering, McMaster University on the hydraulics of Hamilton Harbour for the purpose of stimulating interest in areas of concern among the engineering community. The progress made in the quantification exchanges of water between the open lake and the more restricted nearshore areas as would be required to assess the effectiveness of various proposed remedial strategies is reviewed and the steps required to make further progress are outlined. In addition, the feasibility of advanced acoustical flow measurements to the measurement of exchanges between Hamilton Harbour and Lake Ontario is discussed.

Perspectives-gestion

Ce rapport repose sur une série de conférences sur l'hydraulique du port de Hamilton, données au département de génie civil de l'université McMaster afin de stimuler l'intérêt du monde de l'ingénierie pour les secteurs préoccupants. Les progrès réalisés dans les échanges de quantification d'eau entre le lac ouvert et les zones riveraines plus confinées, et qui permettraient d'évaluer l'efficacité des diverses stratégies palliatives proposées, sont passés en revue, de même que les démarches nécessaires à des progrès ultérieurs. De plus, on envisagera la faisabilité de mesures acoustiques du flux sophistiquées pour mesurer les échanges entre le port de Hamilton et le lac Ontario.

INTRODUCTION

Hamilton Harbour has been designated as one of 42 areas of concern in the Great Lakes Basin by the International Joint Commission despite the large expenditures during the 1970's on pollution abatement (Canada-Ontario Agreement Remedial Action Plan Program, 1988). Although industrial and municipal sources are, in general, meeting current water quality standards in many of these areas of concern, some problems persist. A feature common to many areas of concern is that they are located in bays, fjord-like inlets or harbours where the exchange with the open lake is more restricted than in other coastal areas of the Great Lakes. While the principal processes responsible for nearshore-offshore exchange are known on a qualitative basis, there has been little attention given to developing a quantitative understanding which then may be applied to individual areas of concern to evaluate the effectiveness of various remedial strategies. As an example, Dick and Marsalek (1973) proposed a one-layer (winter) exchange model for Hamilton Harbour but it was not validated by field observations. Once a quantitative capability for simulating the exchange flow is known for one type of area of concern, say a harbour, hopefully this knowledge could be applied to other harbours.

This report is based on a series of graduate level lectures given at the Department of Civil Engineering, McMaster

University, in the fall term, 1988. This series of lectures was intended to stimulate interest among the engineering community in the water quality of Hamilton Harbour and in other areas of concern. The topics to be discussed in the following, are therefore, restricted to those which concern hydraulic flow, namely flow measurement in a rectangular channel, numerical modelling of unsteady flow, harbour exchange ratios, thermal modelling of rivers and streams and stratified exchange flow.

(1) <u>Flow Measurements</u>

According to hydraulic flow theory the discharge through a channel may be estimated from either the average of flows at 20% and 80% of the depth or the flow at a height of 37% (1/e) of the depth above the bed in the case of steady uniform flow. In order to evaluate the validity of this method in the Burlington ship canal in which the flow may be neither steady nor uniform especially in the stratified season nine observations of flow were made at 1 m depth intervals across four nearly uniformly spaced stations on the lift bridge, October 18, 1988. The components of flow and the associated temperatures show in Figure 1 unidirectional flow and nearly uniform temperature across the canal. Based on the canal dimensions used by R. Spigel (1989), the discharge between 9.14 and 9.55 EDST is

372.6 m^3/s and an average flow speed from the harbour to the lake is 47.1 cm/s.

In order to estimate the error in determining the discharge based on only two depths, the so-called 0.2 h and 0.8 h method, the detailed observations were interpolated to those depth points at each station. An approximate discharge of 380.8 m^3/s is obtained which overestimates the more exact discharge by 2%. During the unstratified portion of field season (winter 1988-1989), an acoustic time-of-flight current meter measured the average flow across the canal at a height above the bottom of 3 m. The 1/e height during the October 18 experiment was 3.5 m above the bottom. Interpolated currents at this position resulted in a total discharge of 397.1 m^3/s or an overestimation of 7%. It is notable that both approximate methods overestimate the discharge. Possible explanations are discussed below. Further evluation of the time-of-flight flow meter is given in Appendix I.

During the summer period the flow in Burlington ship canal is bidirectional (Dick and Marsalek, 1973). It is of interest to examine the approximate methods of discharge estimation under these conditions. Based on 36 current measurements collected under the supervision of R. Spigel, June 15, 1988, the total flow out of the harbour was 60.6 m^3/s with an average flow velocity of 17.9 cm/s while the harbour inflow was 55.4 m^3/s with a mean speed of 12.1 cm/s.

The net outflow of 4.6 m^3/s based on 36 readings may be compared to the net outflow of 13.6 m^3/s based on the 0.2 and 0.8 h method and the 1/e h method of -82.2 m^3/s or a net inflow. It is evident that the approximate methods are much more in error during the summer stratified period than in the unstratified season. More than two levels of flow measurement are required during stratified conditions since the presence of density gradients invalidates the uniform flow assumption. The feasibility of acoustic flow methods during the stratified period is examined in Appendix I.

Returning to the question of the validity of the uniform flow assumption for the October data set, the four velocity profiles, U(z), were seen to conform poorly to the theoretical uniform flow profile,

$$\bar{U}(z) = \frac{u^*}{k} \ln \frac{z}{Zo} \quad .$$

The lowest three observations of each profile were approximately logarithmic with the friction velocity, u*, ranging from 2.1 to 7.8 cm/s and the roughness height, Zo, ranging from 1 to 74 mm. However, at mid-depth currents do not vary in depth as steeply as the logarithmic variation. Since the surface winds were light during the experiment the likely reason advanced for the nonlogarithmic character of the velocity profiles is that the bottom boundary layer is still growing at the measurement point along the axis of the channel so that flow may be termed transitional.

Another aspect of the summer flow measurements was that there was a persistent cross-channel tilt of the isopycnals or constant density surfaces as well as to the line of flow reversal as illustrated for the June 15 experiment in Figure 2. Such a tilt in stratified flows is usually observed to be in near geostrophic equilibrium. To test this hypothesis the thermal wind equation was used to estimate the average vertical shear from the observed cross-channel slopes. A comparison with the observed vertical shear from the current meter data suggests in Figure 3 that the flow cannot be in simple geostrophic equilibrium as the thermal wind shear is six to ten times too large. The reason for the exaggerated cross-channel slopes is not known but it is suspected that centrifugal forces arising from the possible curvature of the streamlines may be responsible. Curved outflow plumes are observed at times of strong longshore circulation by Poulton et al. (1986). A similar phenomenon has been observed in oceanic flows. There is insufficient information to apply the theory of Hogg (1983) to the ship canal flow measurements.

(2) Unsteady Exchange Flow

(a) Model

According to the Canada-Ontario Agreement Remedial Action Plan Program (1988) levels of dissolved ammonia build up over the unstratified season in Hamilton Harbour. Similarly, Klapwijk and Snodgrass (1985) inferred much higher levels of total dissolved solids in the harbour based on a mass balance approach. In order to investigate whether these chemical concentration increases are due to more restricted exchange with Lake Ontario during the winter period than in the summer period an unsteady flow model was formulated and tested on a brief period of flow and water level observations in February 1983 collected for the field evaluation of an instrument known as the DPDX water level gauge (Simons and Schertzer, 1983).

The unsteady flow model formulated in this study is significantly different from the earlier model of Dick and Marsalek (1973) in several respects. It can be shown that for harbours as large as Hamilton Harbour and which have natural resonance periods (2.5 h) in the same order as the forcing periods it is not justified to neglect the local acceleration terms. This term may be omitted if the forcing period is less than

$$\frac{1}{2\pi} \sqrt{\frac{LA_h}{gA_c}}$$

where A_h is the harbour area, A_c is the canal cross-sectional area, L the length of the canal and g, the acceleration of gravity. Similarly, Dick and Marsalek (1973) assumed a value of the friction coefficient, Mannings n, of 0.02 whereas in the present model Mannings n is retained as a free parameter and adjusted to minimize the variance between observed and computed water level differences between the two ends of the canal. As well, wind effects on the flow in the ship canal and in the harbour, were taken into account. After the formulation of the unsteady flow model for harbour exchange it was discovered that Mehta and Joshi (1988) and van de Kreeke (1988) proposed an identical model except for wind effects. The reader may be referred to these reviews for details of the model formulation.

The continuity equation is

$$A_h \frac{dh}{dt} = - A_c u + R \tag{1}$$

where $A_h = 21.79 \times 10^6 \ m^2$, the harbour surface area, the cross-sectional area of the canal, $A_c = 891 \ m^2$, and R is the combined river and municipal inflows estimated to be $10 \ m^3/s$. The rate of change height of the harbour surface, h, is proportional to the outflow current, u, in equation (1). The momentum equation is

$$\frac{du}{dt} = -\frac{g}{L}(h - h_{ONT}) - |u|u(\frac{1}{L} + \frac{1}{Ls}) + \frac{\tau x}{d} \qquad (2)$$

where g is the acceleration of gravity and h_{ONT} is the water level of the western end of Lake Ontario as specified every 15 min at the Water Survey of Canada gauge. A typical four-day period of water level variation at this gauge is shown in Figure 5. The length of the canal, L, is taken as 828 m, the depth of the canal, d, of 9.0 and the wind stress component along the axis of the channel, τx, from wind speed and direction readings at the Lake Ontario end of the canal. Length scale, L_s, is related to the channel cross-sectional area and wetted perimeter, P by

$$L_s = \frac{1}{2}\left(\frac{A_c}{P}\right)^{4/3} \frac{1}{gn^2}$$

and is equal to the physical length L for n of 0.033. This is interpreted to mean that for the value of n proposed by Dick and Marsalek (1973) the velocity head loss is greater than the frictional head loss.

As a final step, water level fluctuations in the harbour arising from the wind stress component along a fetch of 5 km are computed by the standard one-dimensional storm surge equations (Hamblin, 1979) and are added to the Lake Ontario water levels. The model was started from rest and run continuously for four days. The influence of the unknown initial conditions quickly died out.

The modelled water level differences for a four-day stormy period in February 1983 over a 703 m baseline along the Burlington ship canal are seen to correspond reasonably well to the observed water level differences using the DPDX instrument for an optimal value of Mannings n of 0.045. On a quantitative basis only 28% of the observed variance in water level difference is accounted for by the model. Simons and Schertzer (1983) describe severe drifting of the instrument which had to be reset every two hours. Therefore, much of the unexplained variance could be due to the instrument and not to the mathematical calculations. Other four-day episodes selected for the DPDX study also had minimum variances between model and observation for an n of 0.045.

At the same time currents were measured in the ship canal. Due to instrumental problems flow at two instruments at separate locations were combined into one record by Simons and Schertzer. This may account for the even poorer agreement between predicted and observed flows evident in Figure 6. No attempt was made to compute the percentage of explained variance for current.

The best fit value of Mannings n of 0.045 is sufficiently different from the most probable value of 0.02 assumed by Dick and Marsalek (1973) to warrant comment. According to Mehta and Joshi (1988), a canal with four jetties, as is the case here, ought to have a Mannings n of 0.029. There must be some additional factor besides jetties in the case of the Burlington ship canal. It is noted that although the frictional headloss is greater than the inertial headloss for this extreme value of n, the bottom boundary layer is growing in thickness along the axis of the channel. Thus the value of Mannings n must be larger to account the non uniformity of the flow. Clearly, further field work is required to determine the appropriate value of Mannings n since the 1983 data may not be entirely reliable and a relatively small portion of the variance is accounted for by the model.

(b) Model Application

Once such a model is calibrated with the field observations it may be employed to investigate the question of exchange flow. Although questions remain on the parameterization of the turbulent frictional forces, the calibrated model was employed as a demonstration of how such models could be applied to water quality concerns. Over the four-day period, February 20 to 23, 1983, the lengths of each flow excursion were computed. A histogram of 88 flow excursions over the experimental period demonstrates in Figure 7 that in most cases the flow reverses before it has an opportunity to clear the canal. The average flow excursion is 392 m and only 6 of the 44 inflow events exceeded the canal length of 828 m. We may then propose the concept of an exchange ratio which is the ratio of the net inflow excursion or infow in excess of the canal length to the total inflow excursion. In the case of the February 1983 storm period, this is 22%. Thus, only 22% of the Lake Ontario water entering the canal actually reaches the harbour whereupon it is assumed that it mixes with the harbour before the flow reverses.

It is interesting to explore these theoretical exchange ratios for past harbour geometries. Before 1926 the surface area was larger while the canal length and cross-sectional area were much less (cf. Dick and Marsalek, 1973). In this case the exchange ratio for inflow would have been 0.71 if the harbour had been forced by the February storm. In 1926 the canal was constructed to its present configuration but the surface area remained at 28.19 x 10^6 m^3. In this case the inflow exchange ratio would have been 0.40 instead of 0.22 for the present situation of infilling of the harbour shoreline.

The exchange ratio may also be defined in terms of the outflow excursions which, of course, are larger than the inflow excursion because of river and municipal discharges. For example, the outflow exchange ratio for the 1926 harbour geometry is 0.93 for the February 1983 forcing.

As an illustration of how the budget of total dissolved solids may also be used to infer the inflow exchange ratio and, as well, to offer a check on the model calculations, let the ratio of the inflow of Lake Ontario water to the total inflow of a typical inflow event be R. The salinity of the inflowing rivers and wastewater treatment plants, S_R, is estimated to be typically 450 mg/L in winter. The average salinity of Lake Ontario, S_{LO}, water is 220 mg/L (Klapwijk and Snodgrass, 1985). They also estimate, through their model, a February value of the total dissolved solid concentration of the harbour water, S_H, of 350 mg/L. The salt input to the harbour during a 4160 second long average inflow event is $V_R S_R$ + $RV_I S_{LO}$ which must equal the flux of salt out of the harbour, $(V_R + RV_I)S_H$. The exchange ratio is then

$$R = \frac{V_R}{V_I}\left(\frac{S_R - S_H}{S_H - S_L}\right) ,$$

where the ratio of the river input over an inflow event, V_R, to the input over an average inflow excursion, V_I, is 0.08.

This results in an exchange ratio of 0.06 which is considerably less than the model result. It is possible that the inflow of Lake Ontario is larger than usual due to the storm, that the estimate of harbour salinity from the work of Klapwijk and Snodgrass is too high or that inflows to harbour do not mix completely. Clearly, direct observation of the winter salinity of the harbour is needed to confirm whether the modelled exchanges in fact contribute to the dilution of harbour water in the winter.

(3) **Thermal Regime of Inflowing Streams**

The thermal regime of Hamilton Harbour is important since, as will be examined in the subsequent section, the water level driven exchange flow is enhanced in summer by a densimetric circulation driven by temperature differences between Lake Ontario and the harbour. As well, temperature influences on biology and chemistry are well known such as the activities of denitrifying bacteria and the solubility of dissolved oxygen. Thermal modelling in lakes has been the starting point for water quality modelling. In order to undertake water quality modelling it is first necessary to establish the temperature of inflowing streams to the harbour. A program of weekly measurements of the inflow temperatures was started in June 1988 by the Ontario Ministry of the Environment. In order to interpolate to daily intervals and also to provide a means of estimating daily inflow flow temperatures for past years, a model of the thermal regimes of tributaries based on stream flows and daily surface meteorological data is formulated as

$$ZC_p \frac{dT}{dt} = Q_A$$

where the depth of the stream, Z, is inferred from the daily discharge, Q, Mannings n, the width of the stream, w, and the bedslope, s, from the uniform flow relation $Z = \left(\frac{Qn}{w\sqrt{s}}\right)^{3/5}$. The stream temperature is T, the specific heat, C_p, and the daily energy flux, Q_A, which is composed of measured short wave radiation and sensible and latent heat fluxes according to standard formulations (cf. Fischer *et al.*, 1979). Since the wind and solar radiation were measured at open and well exposed locations it is necessary to assume that the vegetation and banks reduce the strength of the wind and solar radiation reaching the surface of the inflowing stream. The appropriate values of Mannings n, the bedslope and stream width are not known. Therefore, values of the appropriate shading coefficients which are assumed to be the same for each of Redhill, Spencer and Grindstone Creeks and the individual coefficients of proportionality between stream depth and discharge were found by an optimization procedure which minimized the difference between modelled and observed water temperature. In the first stage the global wind sheltering coefficient was found to be 0.6 and the sun shading coefficient 0.4. In the next stage of optimization the 'conveyances' of Grindstone and Spencer were found to be 0.063 and 0.1 $m^{4/3}$ s^{-1} for Redhill Creek. The model was initialized on May 31, 1988 and run continuously until August 24. A comparison of modelled and observed stream temperatures illustrates in Figure 8 that major fluctuations are reasonably accounted for but smaller errors of several degrees are common. RMS temperature errors are 2.0, 2.5 and 1.4°C for Grindstone, Redhill and Spencer Creeks, respectively. Extension of this study to other times of the year would be useful.

(4) Steady Two-Layer Exchange Flow

Dick and Marsalek (1973) found from ten profiles of temperature and current during the summer season that there is a near steady inflow of Lake Ontario water lying under a surface outflow. While they inferred bottom and interfacial friction coefficients from their measurements they did not attempt to quantify the exchange flow in terms of the driving forces as was done in Section (2) of this report. On account of the enhanced exchange due to the densimetric circulation in the ship canal it is important to be able to model this type of exchange flow. With this capability the sensitivity of the replenishment of bottom waters with more highly oxygenated Lake Ontario water to various remedial strategies may be estimated.

The concept of summer circulation between Lake Ontario and Hamilton Harbour that has emerged from the early studies such as Dick and Marsalek (1973) and Klapwijk and Snodgrass (1985) as well as the recent investigations directed by Charlton and by Spigel (1989) is illustrated schematically in Figure 9. Due to shallower depths as well as inflows of warm water from streams, industry and wastewater treatment plants, the harbour surface layers are warmer than the corresponding adjacent layers in Lake Ontario. Periodically, the temperatures in Lake Ontario are decreased even further by episodes of wind-induced upwelling of cold water. The contrast in density between the two water bodies at the depths of the ship canal drives the densimetric exchange flow. Inflows of lake water sink along the bottom of the harbour until their density matches that of the ambient water whereupon the inflow intrudes into the hypolimnion. Subsequent wind stirring and convective cooling return the inflow to the surface layer through entrainment and turbulent mixing. The harbour outflow spreads out as a thin jet of less dense fluid beyond the exit of the canal (Poulton et al., 1986).

The literature on the computation of stratified exchange flow is scant. Holley and Waddell (1976) outlined an elaborate procedure for calculating the exchange flow from the water level and interfacial level differences and density contrasts between the two water bodies based upon the internal hydraulic equation also used by Dick and Marsalek (1973) and the novel assumption that internally critical flows occur at the two ends of the canal or culvert. The field data of Dick and Marsalek (1973) indicated that the interface between the oppositely flowing layers is nearly linear along the ship canal. This suggested a simple diagnostic test of the validity of the theory of Holley and Waddell, that is, to integrate the interfacial displacement equation from one end of the ship canal to the other based upon observed layer flows and density differences and to compare the computed height differences to the differences in the critical height computed from the standard condition for criticality of internal flows (Holley an Waddell, 1976). Based upon a dozen flow measurements taken during the field season of 1988 by Spigel (1989) and the suggested coefficients of bottom and interfacial friction of Dick and Marsalek (1973) there is little correspondence between the two methods of computing the interface displacement as seen in Figure 10. The lack of agreement stems mainly from the problem of estimating the composite Froude number, G, from field data ($G = F_1{}^2 + F_2{}^2$) where $Fi^2 = Q_i{}^2/(g'z_i{}^3)$ and Q_i is defined in Figure 9, g' is the reduced gravity and z_i is the layer thickess. In cases where the flow is nearly internally critical the displacement height can be very large according to the theory of Holley and Waddell.

The exchange flow theory of Assaf and Heckt (1974) although including friction has mainly oceanographic application as it relies on the known salinity budget of the system rather than on the water level differences between the two reservoirs. Interestingly, they arrived at much larger coefficient of bottom friction, K_B, through their approach than was assumed in this study. In the present case the salinity budget of the harbour is probably too poorly known to apply their method.

(a) Maximal Two-Layer Exchange Flow

Theory

The computation of steady internal exchange flow has been greatly simplified by Armi (1986) who recognized that the critical condition for stratified flow, the composite Froude number of unity, becomes a straight line in a coordinate system with the layer Froude numbers squared, F_1^2 and F_2^2, as independent variables. In this system the continuity equation becomes a family of curves increasing outwards from the origin as the discharge increases. The maximum discharge possible is found from the curve tangent to the critical line. For example, if there is no surface water level difference between the reservoirs then each layer Froude number squared must be 0.5 so that the maximum discharge in each layer is $(0.5)^2 B\sqrt{g'z^3}$ where z is the total depth and B is the channel breadth. If the water levels are unequal then it may be shown that the maximum discharge, say in the lower layer, Q_2 is $B\sqrt{g'z^3}/(1+\sqrt{q_r})^2$ where the flow ratio, $q_r = Q_1/Q_2 = Q/Q_2+1$ where Q is assumed to be related to the water level difference across the two ends of the canal by Mannings relation. The validity of this assumption and the appropriate value of Mannings n for a two-layer flow are not known. Therefore, in the evaluation of the theory of maximal flow to follow the flow ratio is taken from field measurements. So far the flow has been considered steady and frictionless. According to Armi and Farmer (1987), the maximal flow approach holds in all cases where the flow controls at either end of the channel are critical whether or not the flow is steady and frictionless.

Maximal exchange flow theory is extended to treat friction by consideration of the dimensionless internal flow energy, EI, where

$$EI = \frac{z_2}{H}\left(1 + \frac{F_2{}^2}{2}\right) - \frac{z_1}{H}\frac{F_1{}^2}{2} .$$

The internal energy difference between the two ends of the canal must be equal to the dimensionless frictional energy loss integrated from one end to the other,

$$\frac{L}{H}\left[K_B\, F_2^{\,2} + K_I\left[F_1^{\,2}\left(qr^{2/3} - 1\right) + F_2^{\,2}\left(1 - \left(\frac{F_1}{F_2 qr}\right)^{2/3}\right)\right.\right.$$

$$\left.\left. + 2\sqrt{F_1 F_2}\left(\left(\frac{qr\, F_2}{F_1}\right)^{2/3} - \left(F_1/F_2\, qr\right)^{2/3}\right)\right]\right]$$

where L is the channel length and K_I is the interfacial friction coefficient. The computation starts with the frictionless discharge according to the above formulae. The frictional head loss is computed based on the inviscid discharges. Next, the internal energy difference is computed assuming a somewhat smaller discharge than the frictionless value. This process is repeated until the internal energy difference between the two ends of the channel matches the frictional energy loss. This approach may be extended to sub-maximal discharge flow if the interfacial depth(s) at the end(s) is known. In this case there is a unique relation between layer discharge and layer Froude number.

If the water level difference is sufficiently large the two-layer exchange flow may be blocked by the arrest of either layer. Farmer and Armi (1986) argue that discharge in the unarrested layer is given by standard hydraulic theory of flow over a broad-crested weir

$$Q_1 = (2/3)^{3/2}\, BH\sqrt{g'H} = 0.54\, BH\, \sqrt{g'H}$$

Wilkinson and Wood (1988) state, particularly with an advancing or intruding arrested layer that there may be stagnation point so that the nonhydrostatic theory of Benjamin (1968) is more appropriate. In this case the discharge in the flowing layer is slightly reduced to $Q_1 = 0.5\, BY\sqrt{g'H}$. If the discharge is larger than these amounts the front is expelled from the strait.

(b) Application

The steady frictional exchange flow theory is evaluated for the flow measurements of Dick and Marsalek (1973) and based on the experimentally determined flow ratios and extreme buoyancy differences. It is evident from Figure 11 that the inviscid theory of Armi and Farmer (1986) greatly overestimates the flow entering the harbour from Lake Ontario, Q_2. The assumed values of the friction coefficients of Dick and Marsalek result in much closer agreement but still overestimate the exchange flow. Their measurements do not include a wide variation in the exchange flow and are based on a single current profile at the centre of the canal.

The measurements taken in 1988 (Spigel, 1989) are much more detailed and encompass a wide range of exchange flows. In this series of experiments the agreement between frictional theory and observation is much more in line in Figure 12 despite the experiment of August 10, 1988. Both studies demonstrate as might be anticipated from the nonlinear form of friction that frictional effects are much more important at high inflows and that extreme density differences are more appropriate than averaged densities. It is interesting that the anomalous point on August 10 could not be due to the presence of a drowned control since the drowning would reduce and not increase the computed discharge.

Another observation is that in both series of discharge measurements there is a statistically significant net inflow on the average. It is unreasonable that the average inflow is about 25% larger than the mean outflow since river input and wastewater treatment discharges require that the outflow be from 7 to 10 m^3/s larger than the inflow. Evaporative loss from the harbour could account for only 1 m^3/s in the extreme case. It is possible that the mid-day sampling of the flow could bias the exchange flow since land-lake winds alternate on a daily basis during the summer season. Sampling of the discharge during the night or on a continuous basis would be required to check this possibility.

CONCLUSONS AND RECOMMENDATIONS

Preliminary application of hydraulic theory to the problem of quantifying the input of mass and heat to Hamilton Harbour have met with varying success. The most reliable model appears to be the unsteady unstratified exchange flow although fundamental questions about the appropriateness of the best fit value of the friction coefficient require additional flow measurements during the unstratified period. These measurements should be accompanied by observations of the dissolved solid and ammonia concentrations of the harbour and in the ship canal in order to verify the concept of the exchange ratio.

From the point of view of prediction of the summer exchange flow from Lake Ontario water levels, surface meteorology and the thermal structure in Lake Ontario much more work needs to be done. For example, despite encouraging applications of exchange flow theory based on observed flow ratios further work needs to be done on the problem of estimating the flow ratios from observed and predicted water level fluctuations. Somewhat surprisingly the summer exchange appears to be near maximal and thus detailed knowledge of thermal structure in the western end of Lake Ontario may not be required. The exchange model outlined herein and its extension to water level differences should be linked to a thermodynamic model of Hamilton Harbour such as DYRESM (Patterson and Hamblin, 1988) to provide the vertical exchange between the two layers and the thermal structure at the harbour end of the canal. More continuous measurements of flows in the ship canal during the summer would be desirable to check on temporal variations of the two-layer exchange flow. Once a model such as DYRESM, which is based on hydraulic principles, has been validated on simulations of observed thermal and total dissolved solid distributions, it may be applied with reasonable confidence to the problem of evaluating the effectiveness of various remedial strategies.

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LIST OF FIGURE CAPTIONS

Figure 6. Observed currents at one location in Burlington ship canal (cm/s), dashed lines. Modelled flow (cm/s) solid line. Outflow to Lake Ontario is positive. February 20 to 23, 1983.

Figure 7. Histogram of flow excursions, Burlington ship canal. February 20 to 23, 1983.

Figure 8. Stream water temperature, °C, observed (solids line) and modelled (dashed line) June 1 to August 26, 1988.

(a) Grindstone Creek.

(b) Redhill Creek.

(c) Spencer Creek.

Figure 9. Schematic diagram of the exchange flow between Hamilton Harbour and Lake Ontario for the period May through to October.

Figure 10. Computed interface displacement along the canal (m) versus the displacement based on critical flow conditions for $K_B = 0.0026$ and $K_I = 0.001$. May to September, 1983.

Figure 11. Computed Lake Ontario inflow, Q_2, (m^3/s) based on extreme buoyancy differences from data of Dick and Marsalek (1973) versus observed inflow. The numbers correspond to experiments.

Figure 12. Same as Figure 11 but for 1988 data.

APPENDIX I

ACOUSTIC RAY TRACING BURLINGTON SHIP CANAL

APPENDIX I

ACOUSTIC RAY TRACING BURLINGTON SHIP CANAL

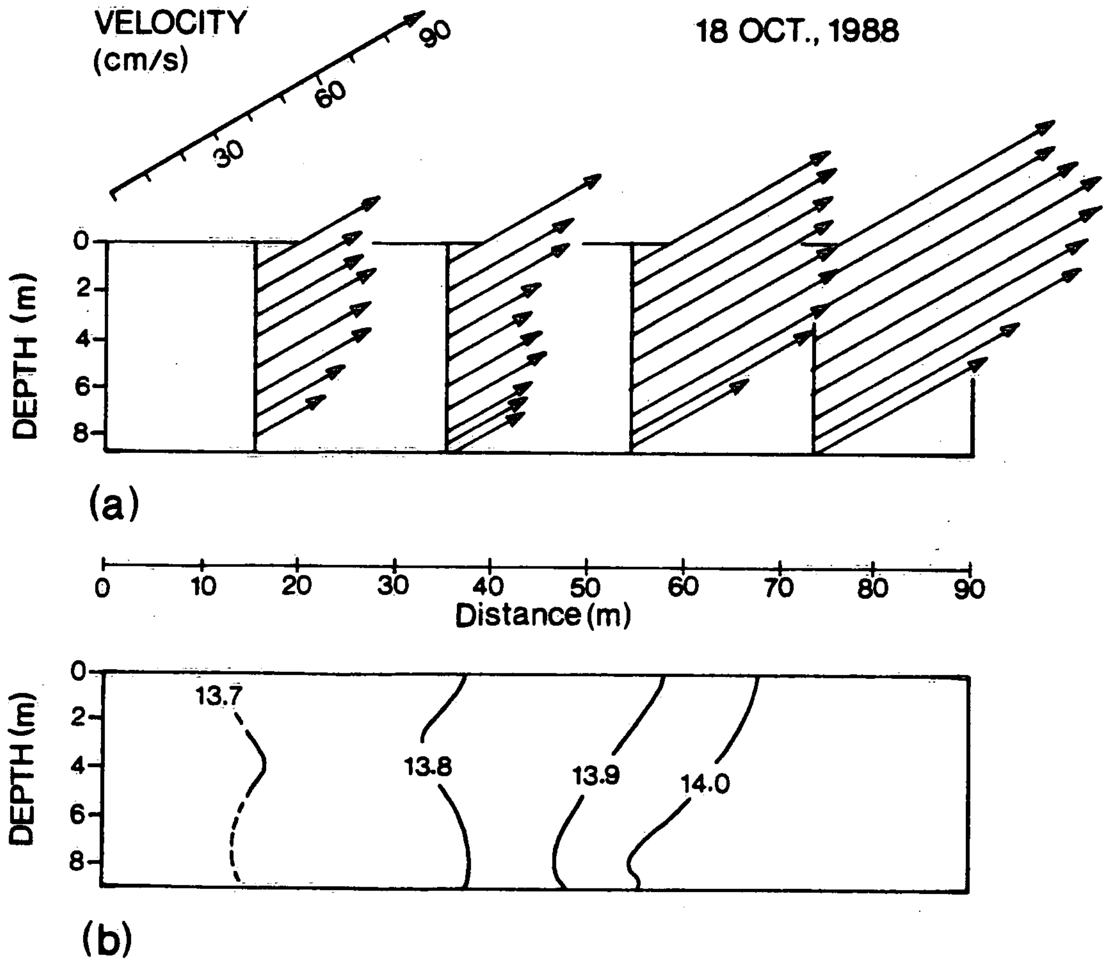
During the winter of 1988-1989, an acoustic time-of-flight current meter manufactured by Stednitz Ltd. and operated by the Water Survey of Canada was installed in the Burlington ship canal. Early indications were that it worked well during the homogeneous period of no or weak stratification. In order to evaluate whether its use could be extended to the summer period, acoustic ray paths were computed by the program package RAYTRAY at the Institute of Ocean Sciences. The temperature structure on June 15, 1988 associated with the density contours shown in Figure 2, was used to compute a vertical profile of the speed of sound. Because the program does not take account of horizontal variation in the speed of sound, the temperature structure at stations 9 and 13 were assumed to hold across the channel. For the purpose of the calculations, the canal was assumed to be 9 m deep and 100 m wide. In the ray tracing program, all rays eminating in a 40° arc and at 0.5° intervals apart that reach a target with a diameter of 40 cm at the same depth as the transmitter are drawn.

Figure A1 presents the velocity of sound profile for station 9 and the ray paths for transmitter-receiver depths of

4, 6 and 7 m. Other experiments at shallower depths than 4 m illustrate the same result as Figure A1a, that is, there are no direct unreflected rays across the channel. It is only at 6 m depth that the two rays in Figure A1b are direct. Again, at 7 m there are no direct paths. Figure A2 illustrates the effect of horizontal variations in the speed of sound. If the temperature at station 3 is assumed across the canal, only rays at 6 m are direct. Noteworthy is the greater refraction of the second ray than in the case of station 9, Figure A1b.

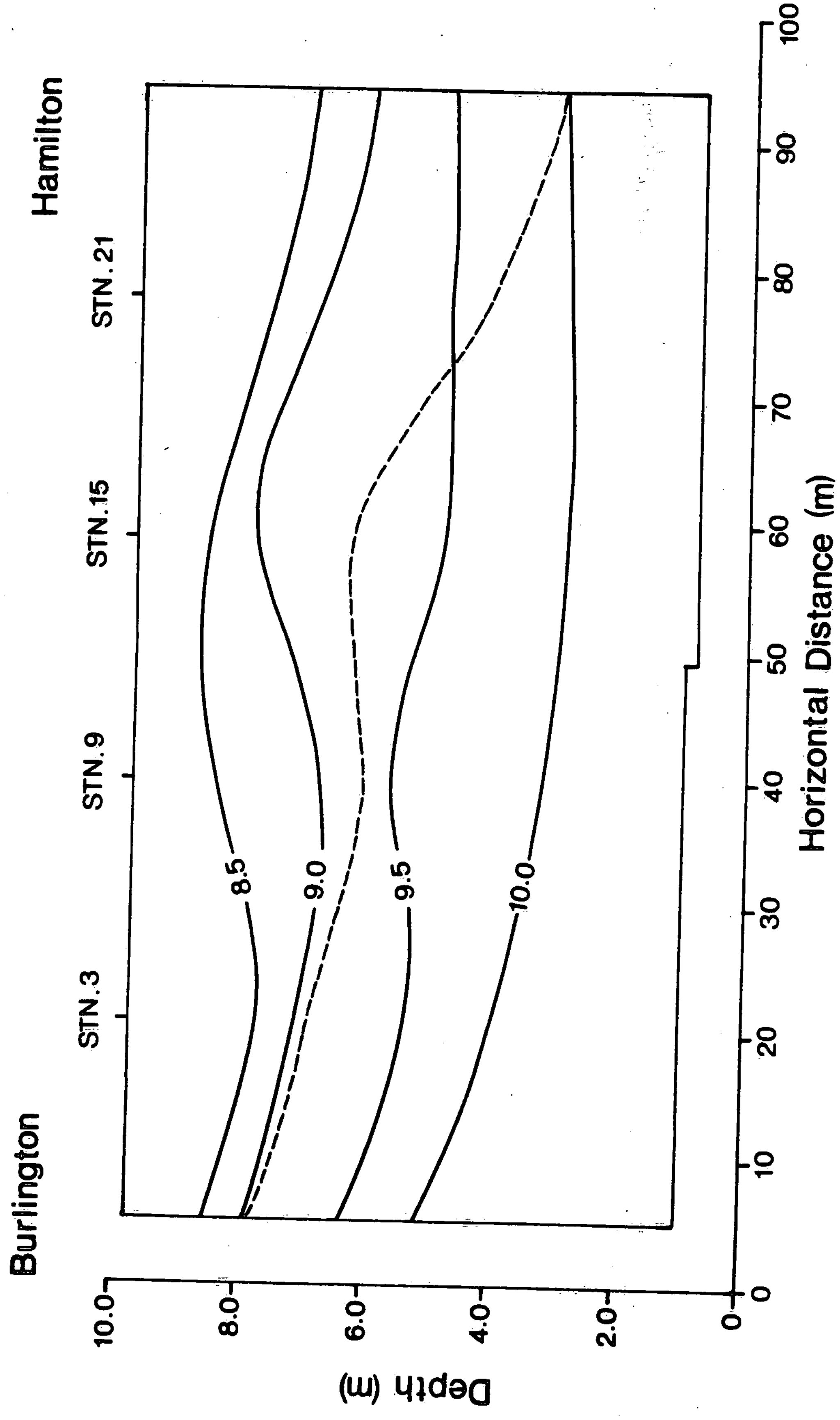
In order to estimate the ray refraction during the winter a temperature profile uniformly increasing from 1°C at the surface to 1.9°C at the bottom was assumed. Blending of the ray trajectory is noticeable in Figure A3. The acoustic path is estimated to be lengthened by 0.32% by the weak winter stratification assumed. This lengthening should have negligible effect on the flow estimates but may cause errors in the speed of sound estimate from the time-of-flight device.

In conclusion, it would appear that typical summer stratification invalidates the use of any acoustic device relying on rays paths crossing the ship canal. It is recommended that upward looking acoustic doppler devices be used to profile the summer exchange flow.



VELOCITY
(cm/s)
30
60
90
18 OCT., 1988
0
2
4
6
8
DEPTH (m)
(a)
0
10
20
30
40
50
60
70
80
90
Distance (m)
0
2
4
6
8
DEPTH (m)
13.7
13.8
13.9
14.0
(b)

Figure 1



Hamilton
Burlington
STN.3
STN.9
STN.15
STN.21
8.5
9.0
9.5
10.0
Depth (m)
10.0
8.0
6.0
4.0
2.0
0
Horizontal Distance (m)
0
10
20
30
40
50
60
70
80
90
100

FIGURE 2

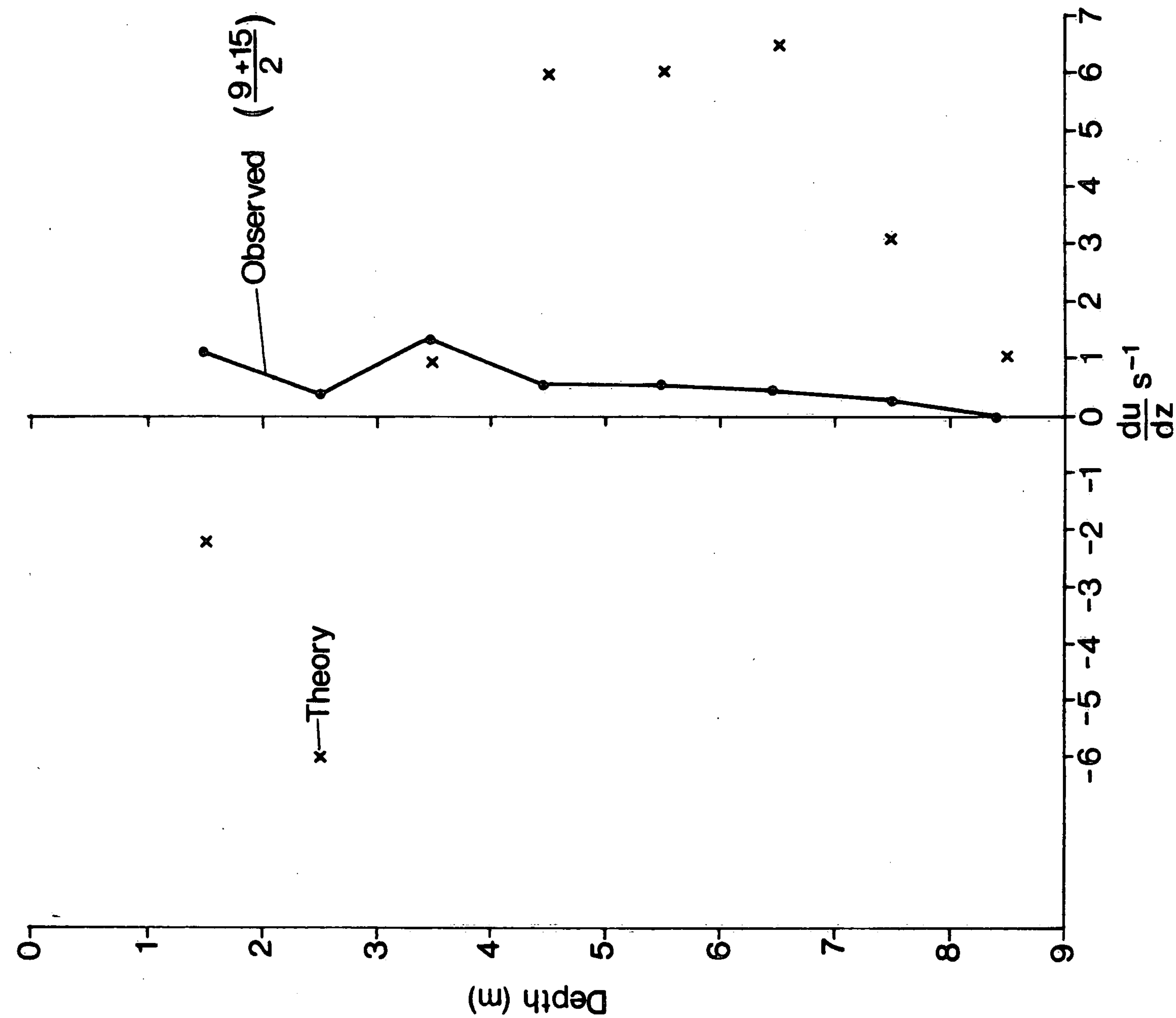


FIGURE 3

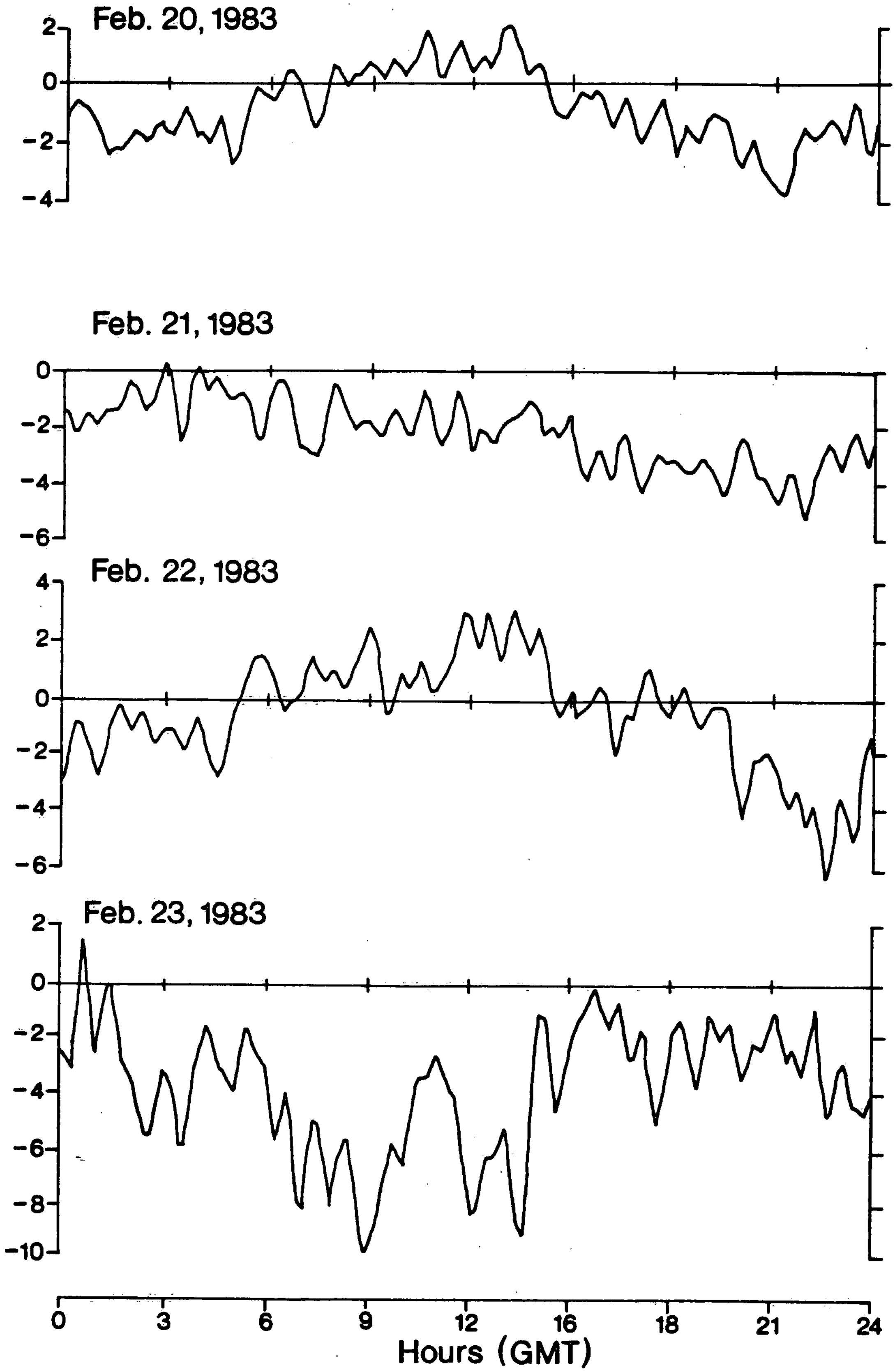

Feb. 20, 1983
2
0
-2
-4
Feb. 21, 1983
0
-2
-4
-6
Feb. 22, 1983
4
2
0
-2
-4
-6
Feb. 23, 1983
2
0
-2
-4
-6
-8
-10
0
3
6
9
12
16
18
21
24
Hours (GMT)

FIGURE 4

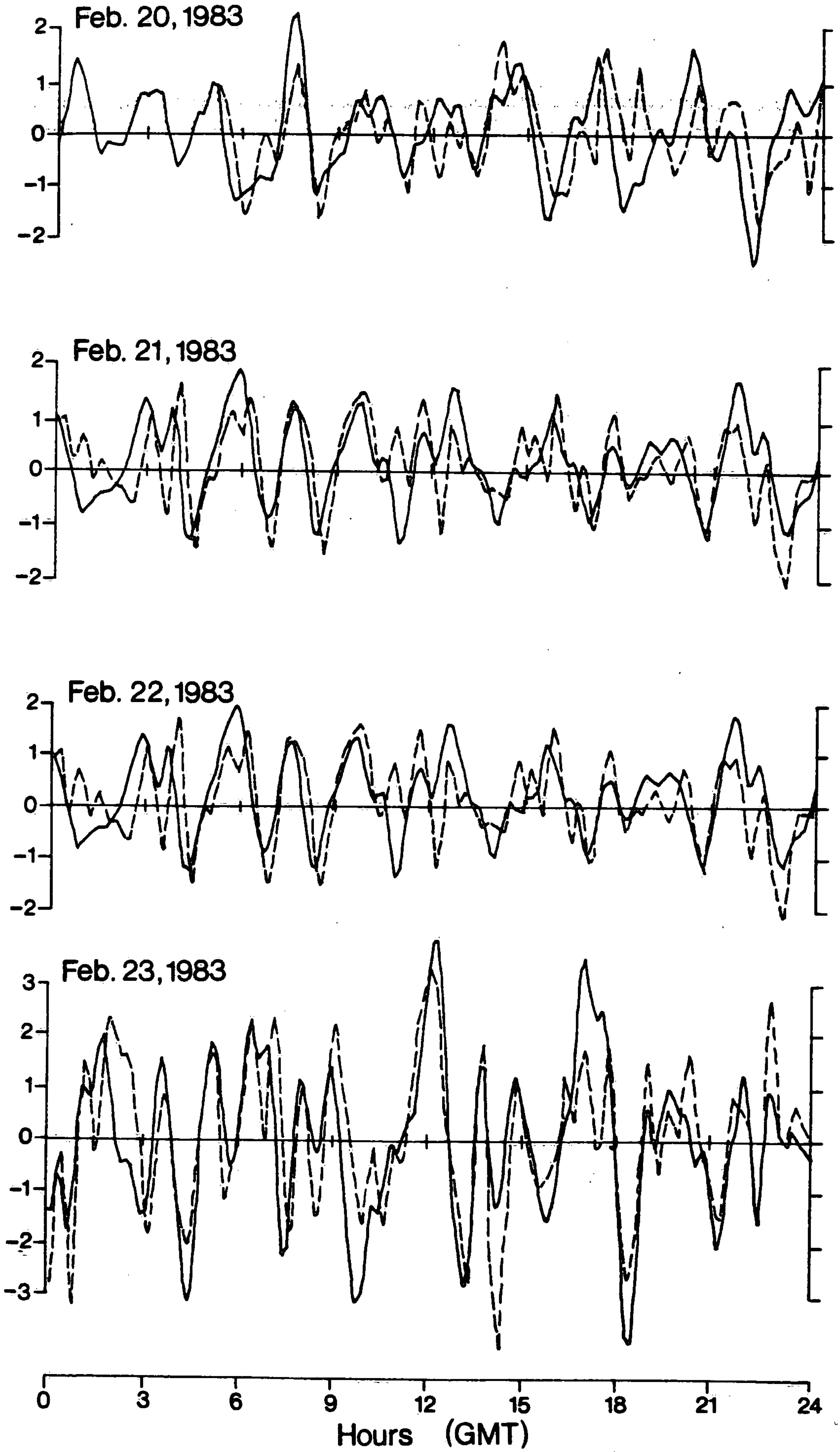
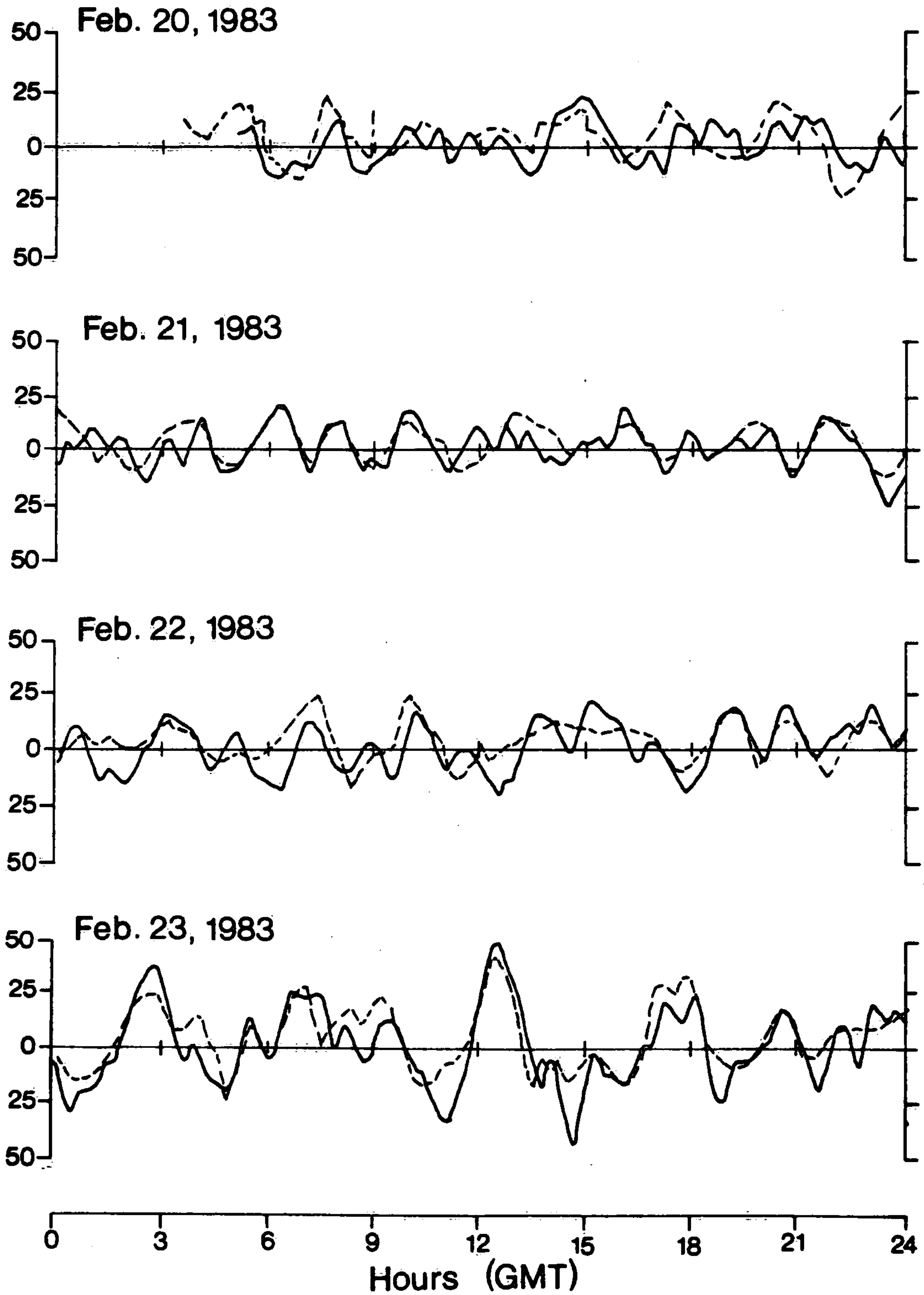
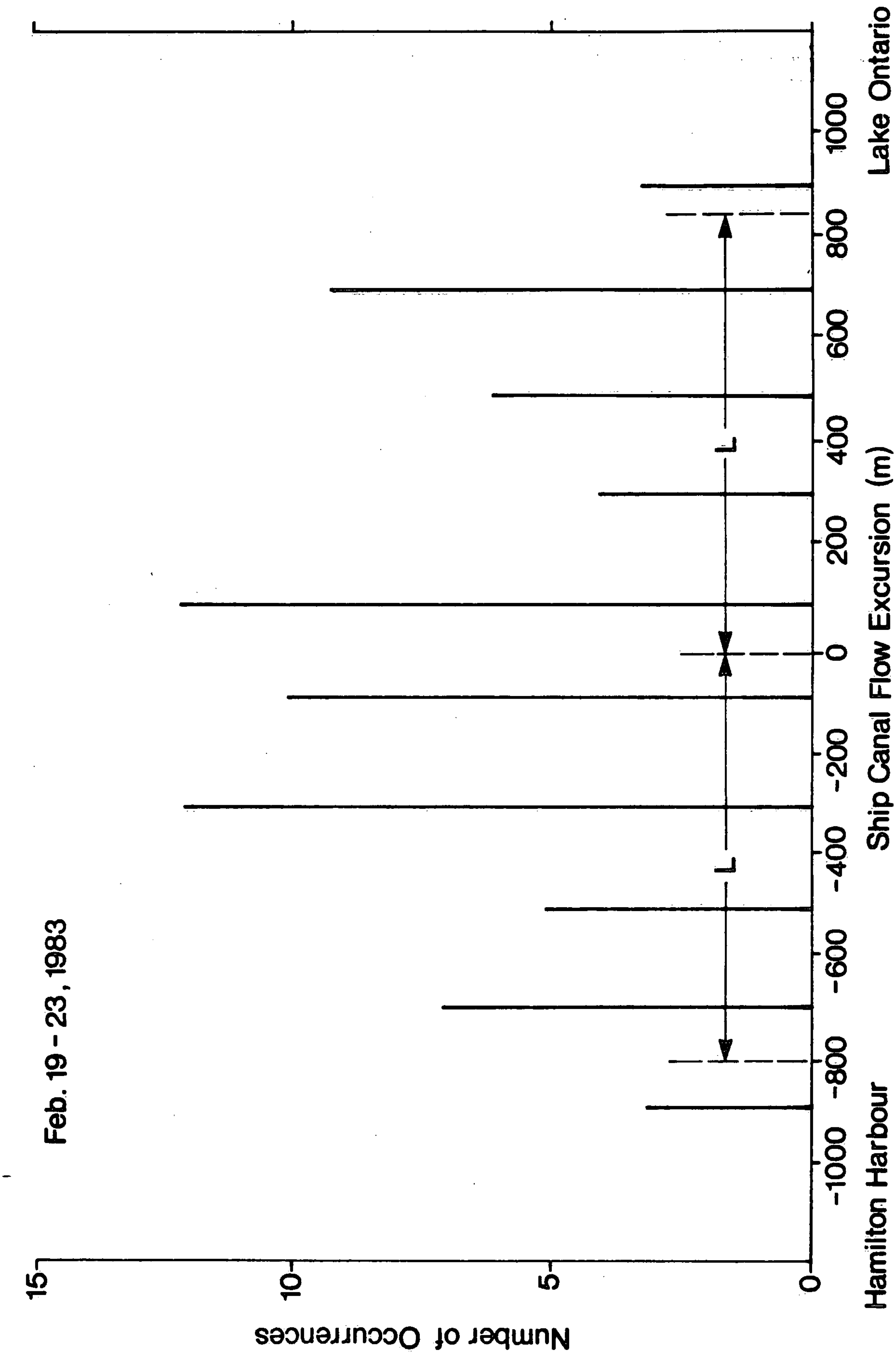

Feb. 20, 1983
Feb. 21, 1983
Feb. 22, 1983
Feb. 23, 1983
2
1
0
-1
-2
3
-3
0
3
6
9
12
15
18
21
24
Hours (GMT)

FIGURE 5



Feb. 20, 1983
50
25
0
25
50
Feb. 21, 1983
50
25
0
25
50
Feb. 22, 1983
50
25
0
25
50
Feb. 23, 1983
50
25
0
25
50
0
3
6
9
12
15
18
21
24
Hours (GMT)

FIGURE 6



Feb. 19 - 23, 1983
Number of Occurrences
15
10
5
0
Ship Canal Flow Excursion (m)
-1000
-800
-600
-400
-200
0
200
400
600
800
1000
L
Hamilton Harbour
Lake Ontario

FIGURE 7

Grindstone Creek

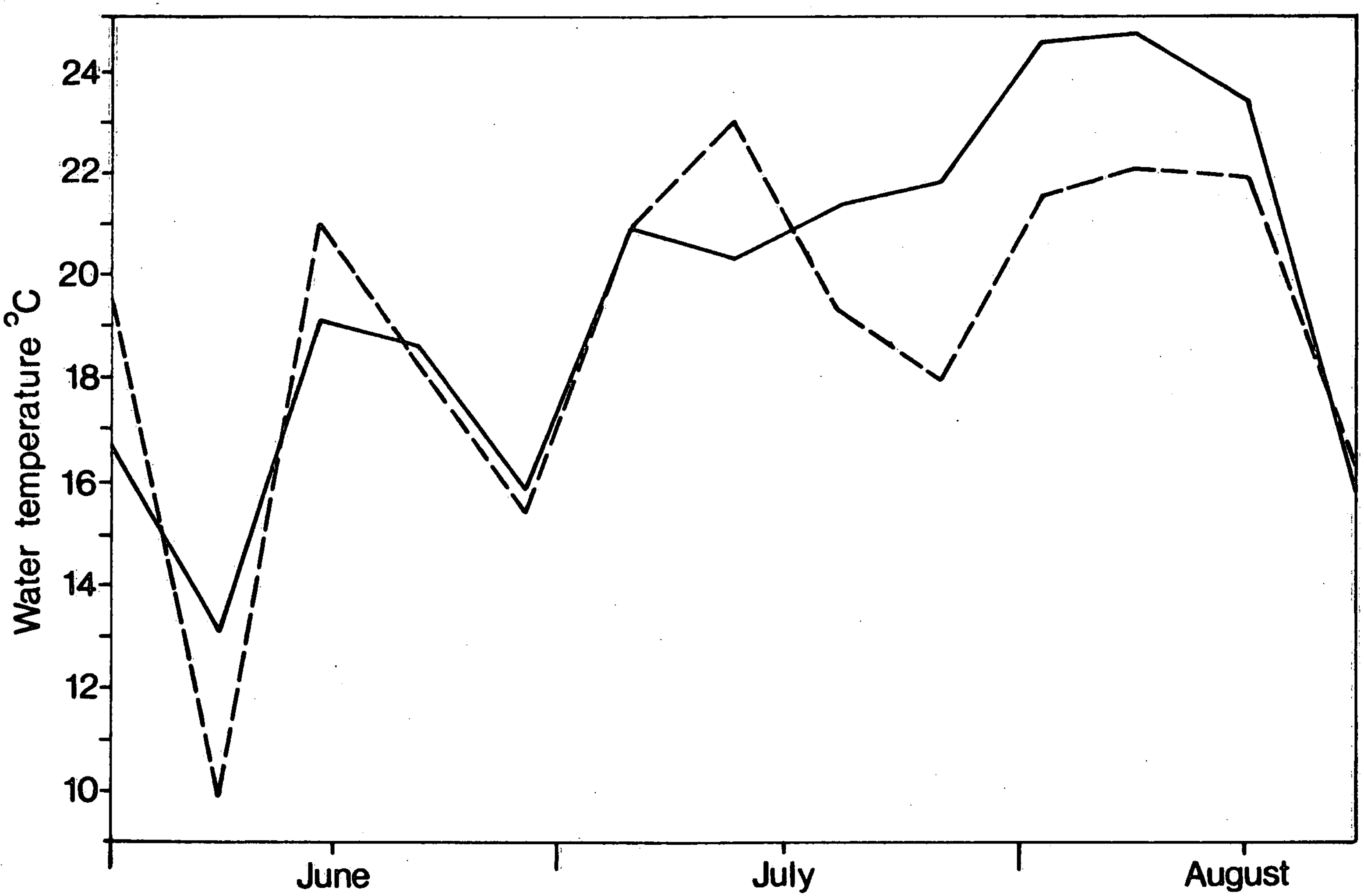


FIGURE 8a

Redhill Creek

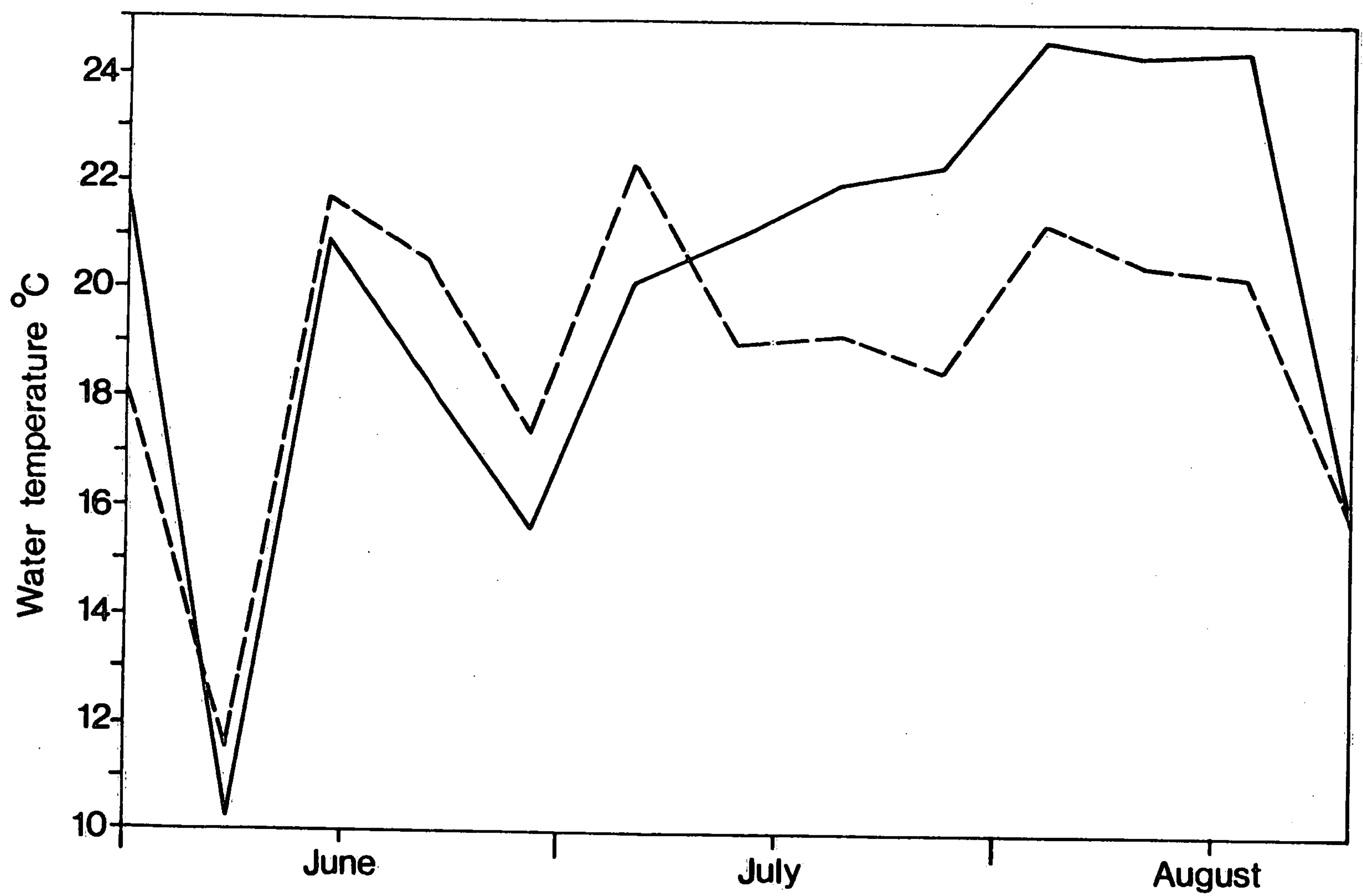


FIGURE 8b

Spencer Creek

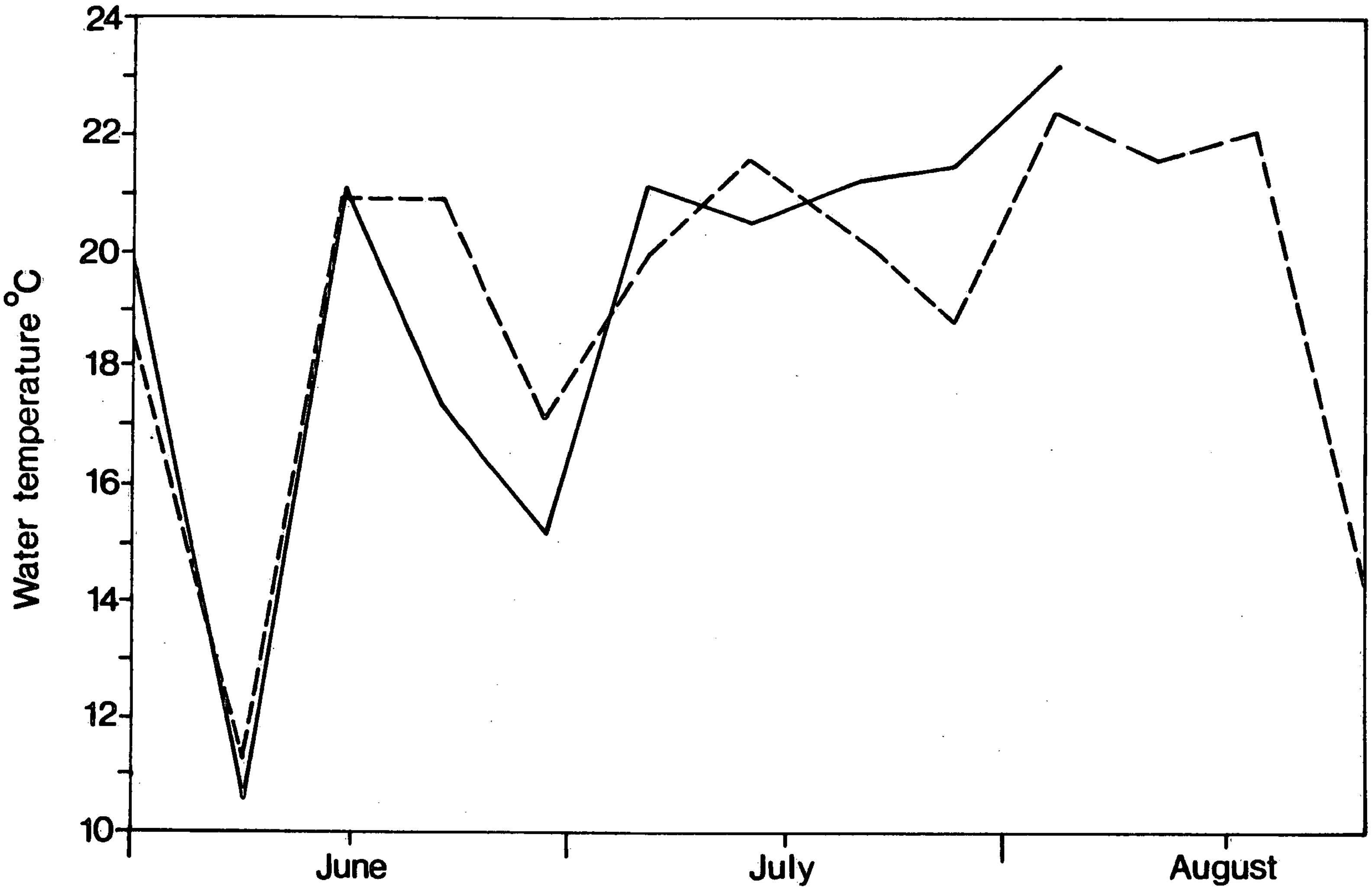
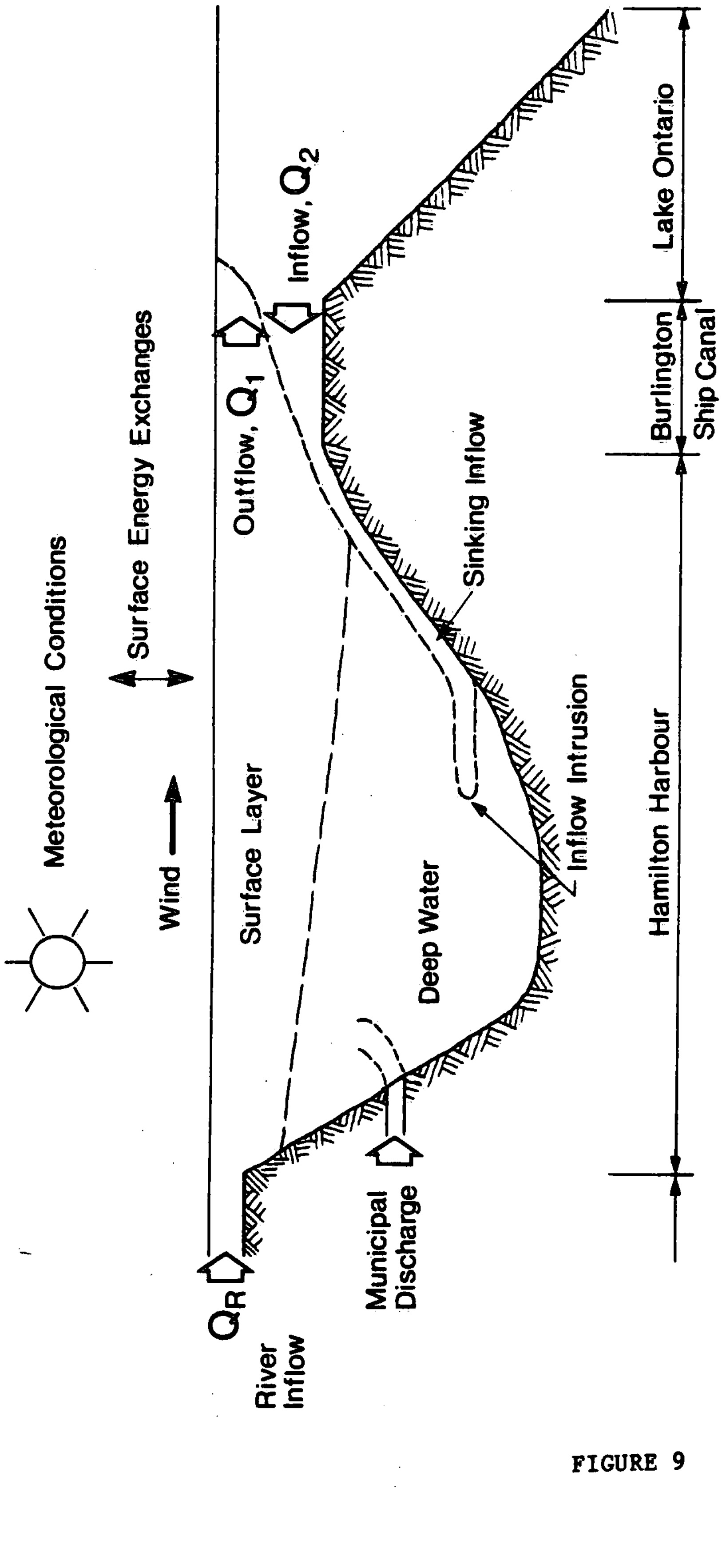
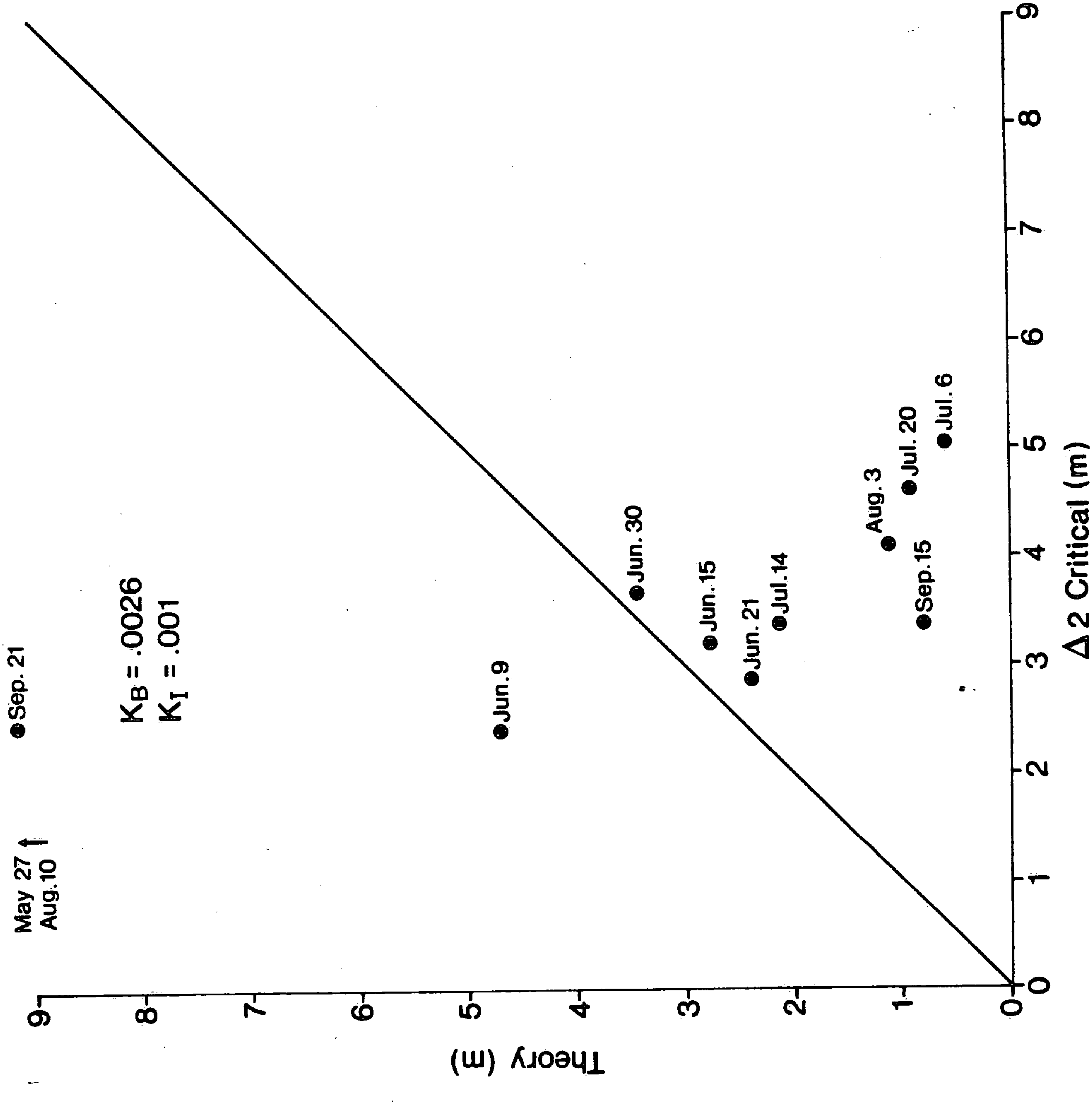


FIGURE 8c



Meteorological Conditions
Surface Energy Exchanges
Wind
Surface Layer
Deep Water
Outflow, Q_1
Inflow, Q_2
Sinking Inflow
Inflow Intrusion
Q_R
River Inflow
Municipal Discharge
Hamilton Harbour
Burlington Ship Canal
Lake Ontario

FIGURE 9



Theory (m)
Δ 2 Critical (m)
0
1
2
3
4
5
6
7
8
9
K_B = .0026
K_I = .001
May 27
Aug.10
Sep. 21
Jun.9
Jun. 30
Jun.15
Jun. 21
Jul.14
Aug. 3
Jul. 20
Jul. 6
Sep.15

FIGURE 10

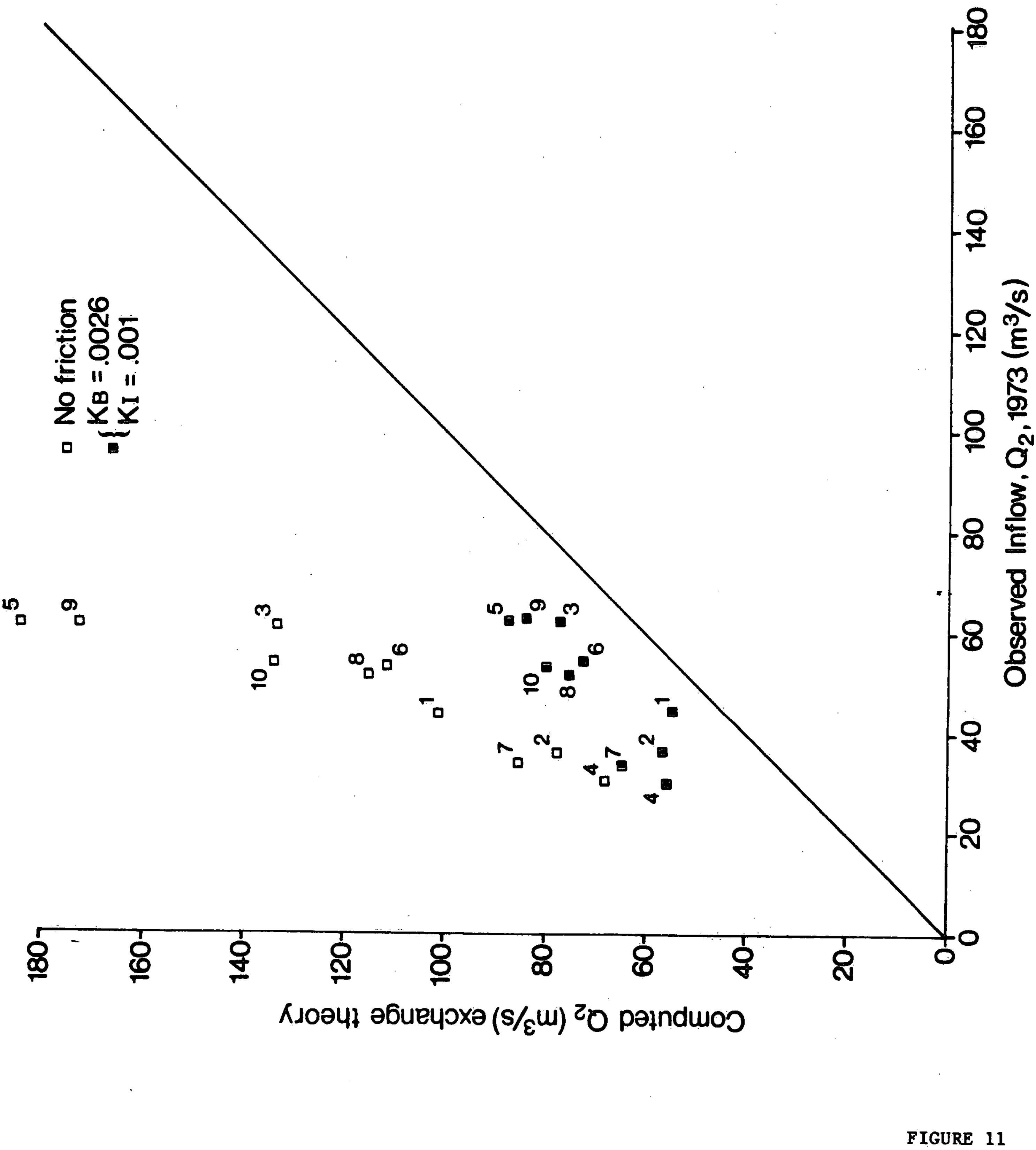
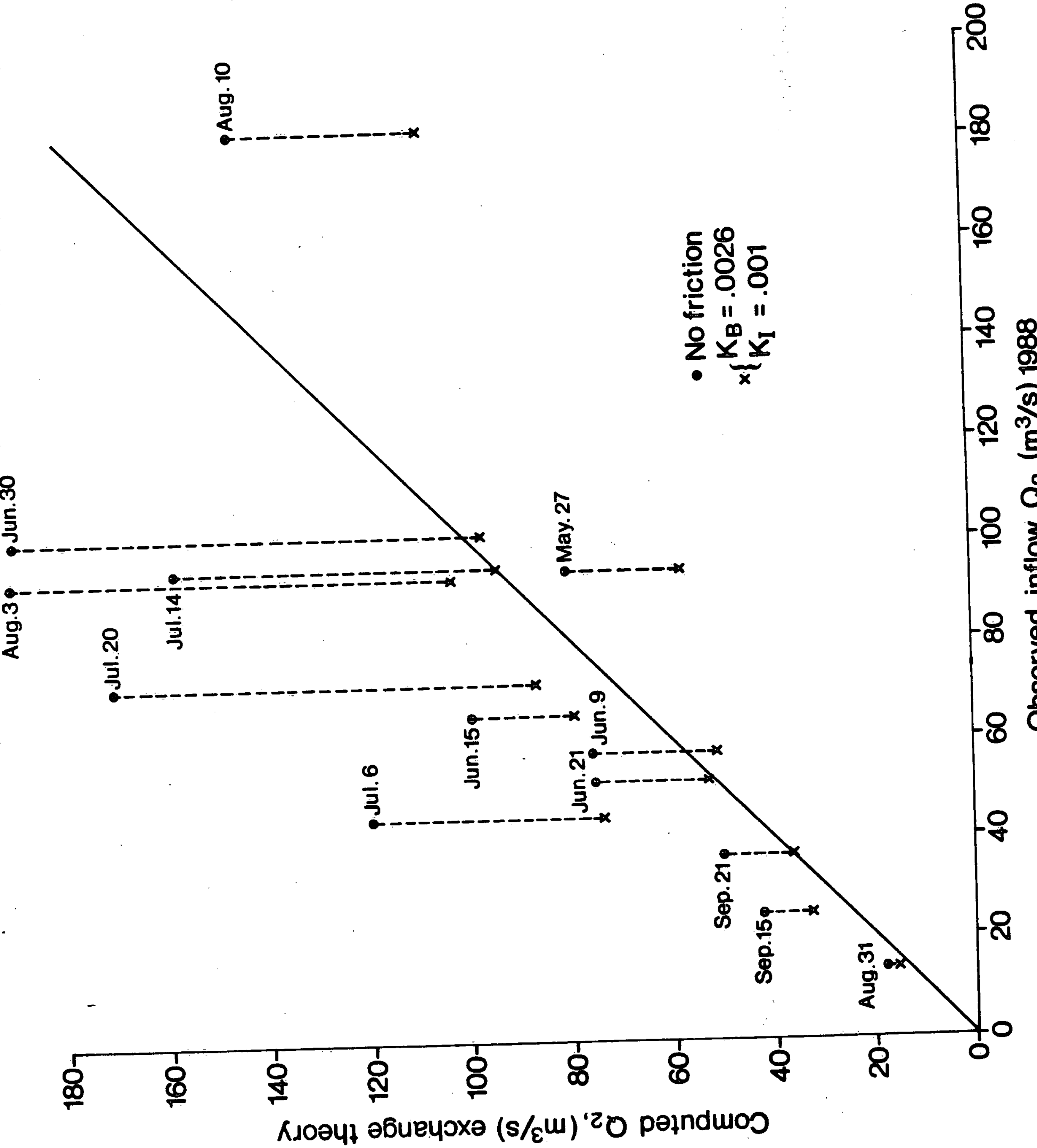

No friction
KB = .0026
KI = .001
Observed Inflow, Q2, 1973 (m3/s)
Computed Q2 (m3/s) exchange theory
0
20
40
60
80
100
120
140
160
180

FIGURE 11



Computed Q2, (m3/s) exchange theory
Observed inflow, Q2, (m3/s) 1988
0
20
40
60
80
100
120
140
160
180
200
No friction
KB = .0026
KI = .001
Aug. 10
Jun. 30
May. 27
Aug. 3
Jul. 14
Jul. 20
Jun. 9
Jun. 15
Jun. 21
Jul. 6
Sep. 21
Sep. 15
Aug. 31

FIGURE 12

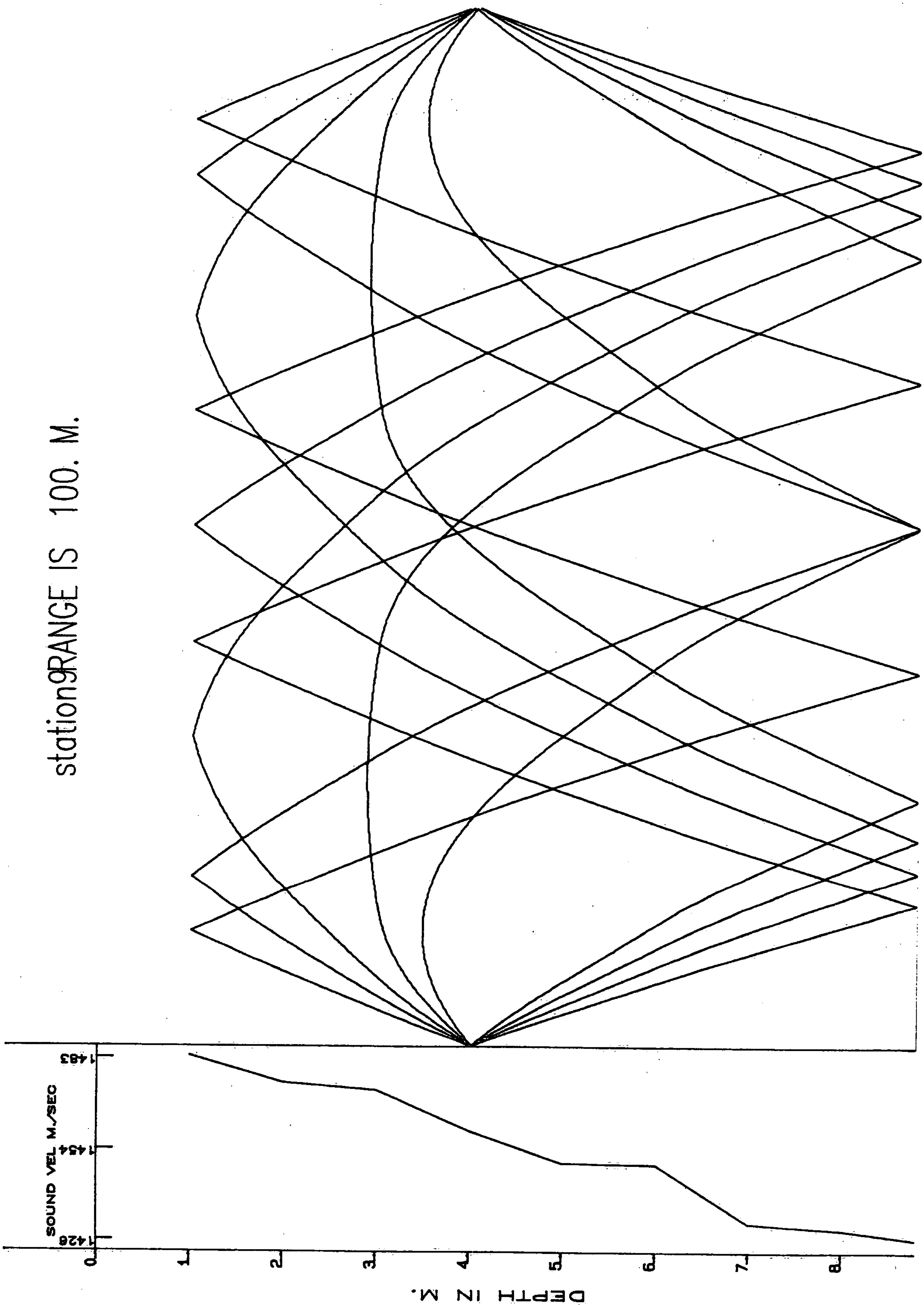
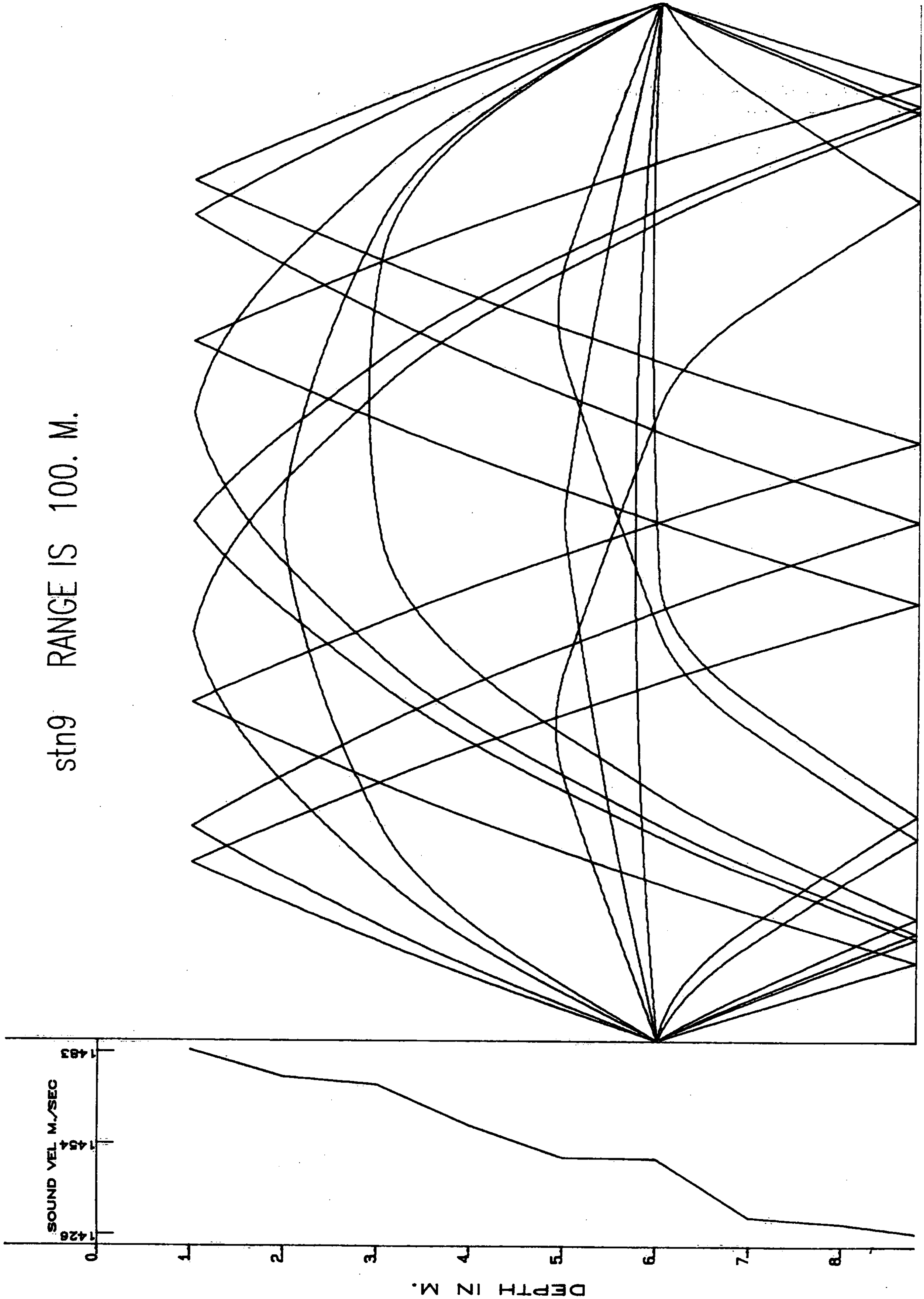

station9RANGE IS 100. M.
SOUND VEL M./SEC
1428
1454
1483
DEPTH IN M.
0
1
2
3
4
5
6
7
8

FIGURE A1a



stn9 RANGE IS 100. M.
SOUND VEL M./SEC
1483
1454
1426
DEPTH IN M.
0
1
2
3
4
5
6
7
8

FIGURE A1b

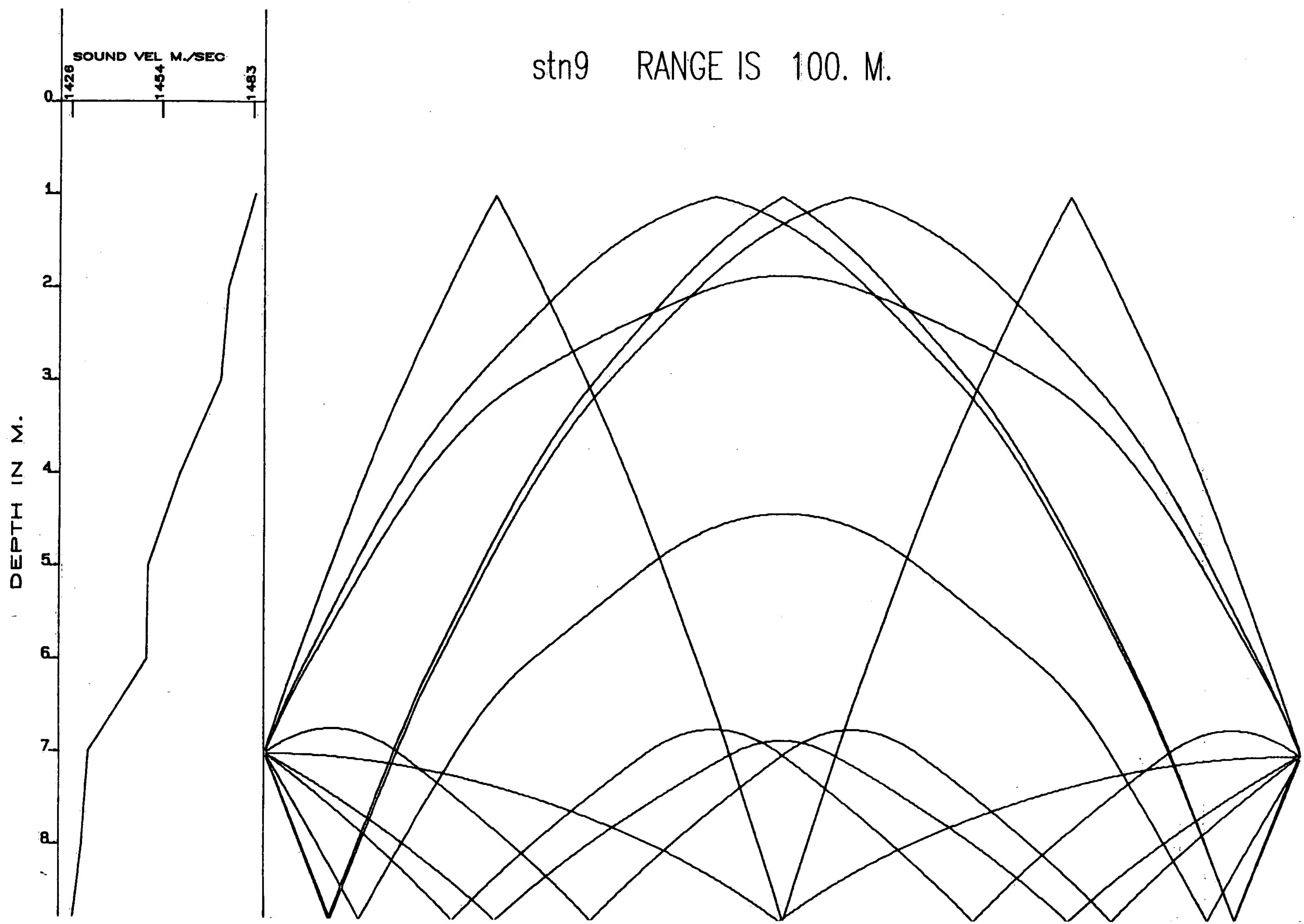
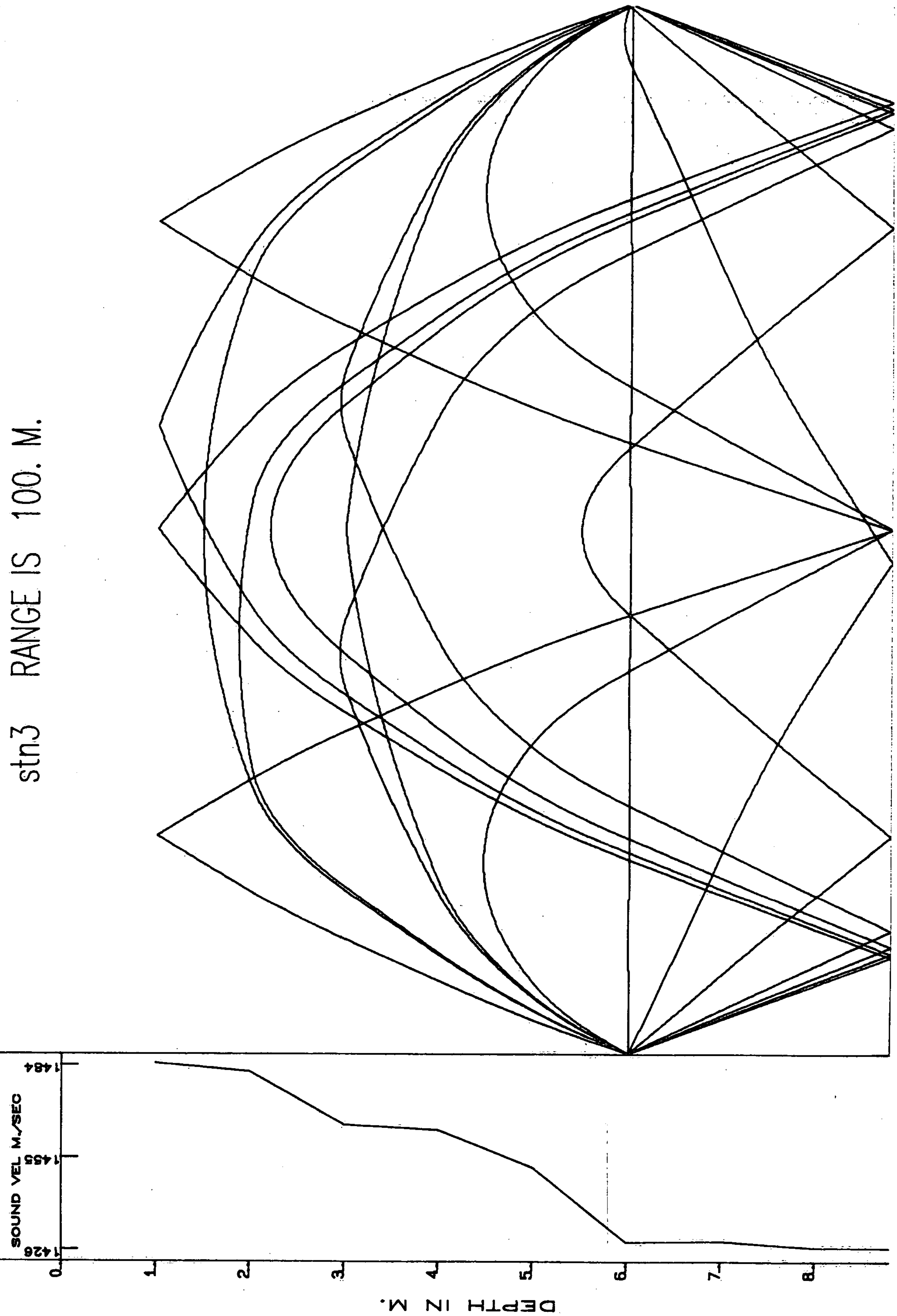
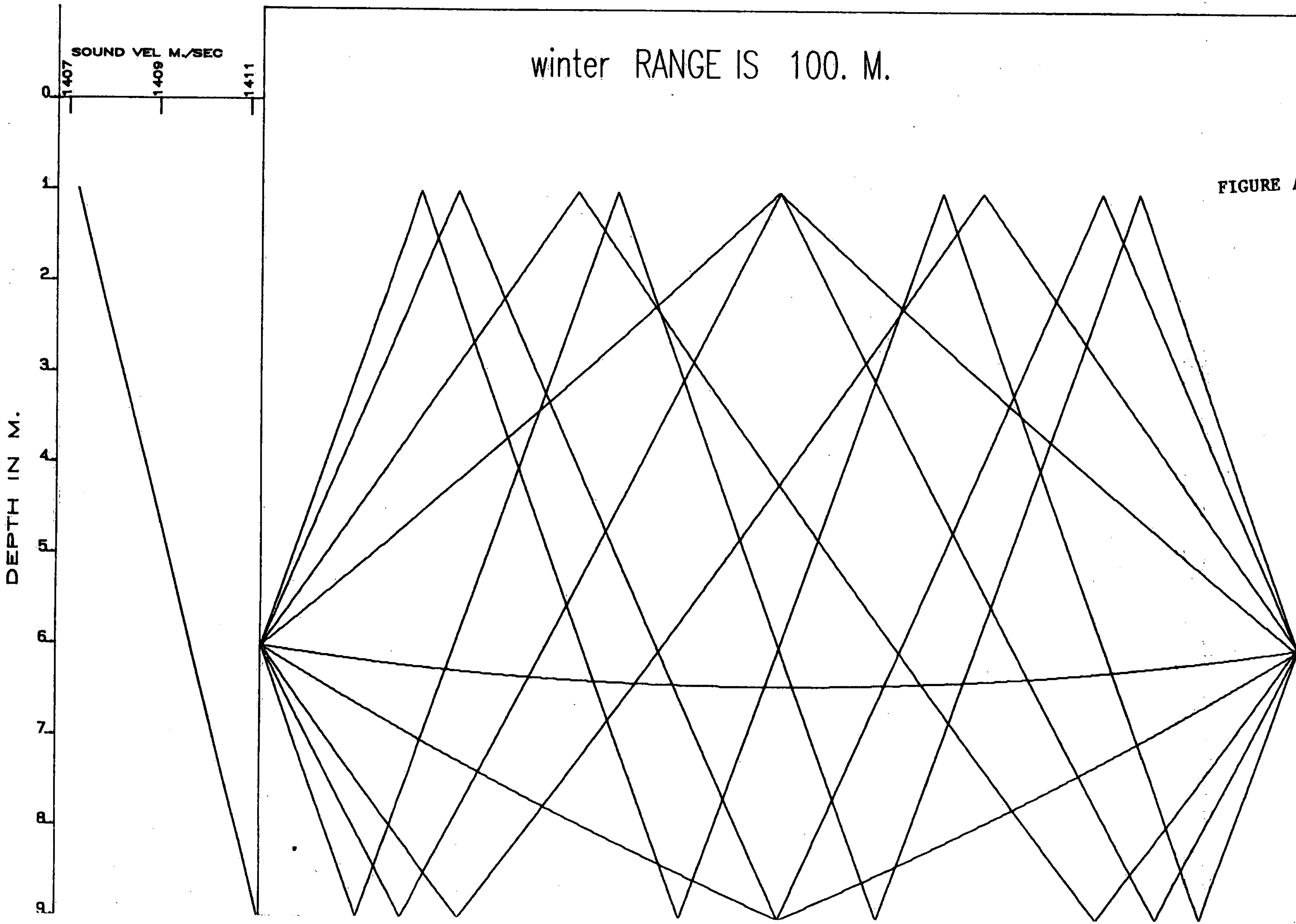

stn9 RANGE IS 100. M.
SOUND VEL M./SEC
1428
1454
1483
DEPTH IN M.
0
1
2
3
4
5
6
7
8

FIGURE A1c



stn3 RANGE IS 100. M.
SOUND VEL M./SEC
1426
1455
1484
0
1
2
3
4
5
6
7
8
DEPTH IN M.

FIGURE A2



winter RANGE IS 100. M.
SOUND VEL M./SEC
1407
1409
1411
DEPTH IN M.
0.
1.
2.
3.
4.
5.
6.
7.
8.
9.

FIGURE A3