RUNOFF CONCENTRATION IN STEEP CHANNEL NETWORKS

by

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ABSTRACT

The objective of this study is the development of a runoff routing procedure, applicable to steep channel networks in the tumbling flow regime, and suitable for incorporation into more comprehensive mathematical representations of the runoff process. "Steep" is meant in the sense that degradation into existing, coarse deposits (e.g. Pleistocene materials, slide debris, scree) is assumed to be the major channel-forming process. Similarity considerations show that under these circumstances two relatively easily available parameters, such as channel slope and drainage area, or channel slope and width are adequate to define the geometry and hydraulic performance of the channels.

The hydrologically significant aspects of channel flow are storage per unit length (area) and discharge, with the relation between the two defining the channel performance under steady conditions. This function, $A = f(Q)$, can be obtained in the field by observing the dispersion of slug-injected tracers through fixed test reaches over a range of discharges. Measurements of this type were made on thirteen test reaches, covering a wide range of channel size and slope.

The data from all test reaches can be closely approximated by exponential relations of the form $A = a_A Q^{b_A}$. As indicated by the similarity considerations, the constants a_A and b_A of this steady flow equation are predictable from basin parameters. The details of the statistical link between various

basin parameters and the above constants are discussed in Day (1969) on the basis of the steady flow data of this study supplemented by extensive additional measurements.

Runoff concentration is an unsteady flow process, which can only be defined with a single flow equation if the flow system is truly kinematic. In order to investigate whether this holds for steep channels, all test reaches were located below lakes with outlets suitable for minor discharge modifications. Small, step-like surges (positive and negative) were created at the lake outlets and their propagation through the test reaches was observed with accurate water level gauges. These surge tests indicate consistently that the channels act as kinematic flow systems but with certain dispersive effects added and with a markedly higher-than-kinematic wave celerity at very low stage, which is probably the result of dynamic waves in pools.

Due to the frequent occurrence of super-critical flow, dispersion can only be the result of storage in pools. The differential equation for a kinematic channel with storage in a large number of identical storage elements is derived and solved in linearized form for step-like input corresponding to the surge tests. The dispersion coefficient, which has dimensions L, is the only free parameter of the solution. Comparison with the field data shows that mean water surface width provides a good estimate of this parameter.

As a computationally simpler alternative, a routing model which replaces the actual channels by a sequence of truly kinematic channels and deep pools with weir outlets, both obeying the same steady-flow equation,is also considered. Rules for determining the two free parameters of this solution are developed on the basis of the field data.

Both routing methods provide approximately equal fit to the surge test data and they both appear to be suitable components for an operational channel runoff model. Being based mainly on the above steady flow equation, both methods are non-linear. This is supported by the field data, which show no tendency towards linearity, except possibly at very low stage.

TABLE OF CONTENTS

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

TABLE OF CONTENTS (cont'd.)

LIST OF TABLES

LIST OF FIGURES

LIST OF FIGURES (Cont'd)

LIST OF PHOTOGRAPHS

APPENDIX

COMPUTER PROGRAMS WITH OPERATING INSTRUCTIONS, PRINTOUT, AND PLOTS

LIST OF CONTENTS

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1. NOTATION AND ABBREVIATIONS

1.1 Notation

A	Cross sectional area of flow in a channel (m^2).
A_o	Initial area.
A_D	Area at formative discharge.
A_t	$\partial A/\partial t$.
a	Constant in the following regressions:
a_A	$\log A = f(\log Q)$
a_v	$\log v = f(\log Q)$
a_T	$\log T = f(\log Q)$
a'_A	$\log (Ag^{2/3}/\nu^{4/3}) = f(Qg^{1/3}/\nu^{5/3})$.
$B(\ )$	Riemann Function
b	Coefficient in the above regressions (same subscripts).
b	Without subscript: Time constant (s).
C	Concentration of tracer $(g\ cc^{-1})$.
C_i	Concentration of tracer in reservoir i.
c	Wave celerity (ms^{-1}).
c_i	Constants.
D	Bed material size (m).
D_x	Mixing coefficient, for one dimensional dispersion over x (m^2s^{-1}).
D_c	Flood wave dispersion coefficient (m^2s^{-1}).
DA	Drainage area (km^2).

d	Depth of flow in channel (m).
d_*	Depth measure, defined as A/W_s.
E	Exciting voltage of conductivity bridge (volts).
e	Base of natural logarithms, 2.7183.
$F(\ )$	Function of $A(x,t)$.
$f(\ )$	Unspecified function.
$f_x(\ )$	Probability density function.
G	Conductivity (mhos).
g	Acceleration of gravity (ms^{-2}).
g_s	$(\rho_s - \rho_w)g/\rho_s$
g_w	$(\rho_s - \rho_w)g/\rho_w$
H	Stage reading (ft).
h	Elevation difference between stream and gauge stilling well (ft).
$I_i(u)$	Modified Bessel Function of the first kind of order i and argument u. With $J_i(u)$ being a Bessel Function of the first kind $I_i(u) = J_i(\sqrt{-1}\ u)$.
i	Integer, counter.
j	Integer.
$k(\ )$	Function of $g(x,t)$.
L	Tracer loss rate (% per min.).
l	Length of test reach (m)
$l_i, i=0,1,2$	Second order interpolation polynominals.
M	Mass of tracer (g).

n	Integer, number of reservoirs or storage elements.
n_m	Manning's n.
$P(t)$	Polynominal approximation of $C(t)$, $(g\ cc^{-1})$.
p	Abbreviation for $(240c/\beta^2 W_D^2)^{1/2}$
Q	Discharge $(m^3 s^{-1})$.
Q_o	Initial discharge.
Q_D	Formative discharge.
Q_i	Outflow from reservoir i.
Q_m	Measured discharge.
Q_u	Discharge from upstream reservoir.
q	Relative discharge, Q/Q_D, subscripts as for Q.
R	Adjustable resistance (ohms).
R_G	Input impedance of recorder.
R_c	Constant resistance.
R_i	Volume of reservoir i.
r	Argument of Γ-function, parameter of Γ-distribution.
S	Slope
S_f	Friction slope
S_V	Valley slope
T	Mean water travel time (min.)
T_P	Lag to tracer peak (min.).
T_L	Lag between pools.
T_R	Reservoir filling time (s).
T_S	Lag to the first arrival of tracer (min.).

T_t	Mean tracer travel time (min.).
t	Time coordinate (min. or s).
t_s	Starting time.
t_e	End time.
u	Bessel function argument.
V	Potential difference, voltage (volt).
v	Velocity (ms^{-1}).
v_m	Mean velocity.
v_D	Velocity at channel forming discharge.
W	Width (m).
W_D	Channel width between high water marks.
W_s	Water surface width.
w	Exponent in exponential relation between A and Q, $Q \propto A^w$
X_i	Shape factors.
x	Length coordinate along channel (m).
x_i	Length coordinates of critical sections.
Y_i i=1,2,3	Parameters of Storage Model based on Γ-distribution.
y	Abbreviation for exponent $(w - 1)/w$.
z	Exponent in exponential relation between W_s and Q, $W_s \propto Q^z$

α	Relative change in discharge during a surge test, Q/Q_D-1.
β	Non-dimensional dispersion coefficient.
$\Gamma(\ )$	Gamma Function.
γ_i i=1,2,3	Abbreviations for terms in c and Δt_i.
Δ	Finite step.
θ	Slope angle.
κ	Parameter of Γ-distribution.
λ	Length of a reservoir (m).
μ	Viscosity.
ν	Kinematic viscosity (m^2s^{-1}).
ξ	Dummy length variable (m).
π	3.1416.
ρ	Specific mass ($g\ cc^{-1}$).
ρ_s	Specific mass of bed material.
ρ_w	Specific mass of water.
σ	Proportion of channel length occupied by reservoir.
τ	Dummy time variable (min. or s).

1.2 Abbreviations

Bl	Blaney Creek.
Br	Brockton Creek.
C.I.	Constant injection.
cc	Cubic centimeter.
d	Derivative.
δ	Partial derivative.
D	Total derivative for a moving observer.
DO	Down-surge, downstream of
ft	Feet.
K	1000 ohms.
km	Kilometer.
Lo	Longitude
Lat	Latitude
l	Liter
log x	Natural logarithm of x.
$\log_{10}x$	Logarithm to base 10.
mm	Millimeter.
m	Meter.
min.	Minutes
NaCl	Sodium chloride, common salt.
NTS	National topographic series.
Ph	Phyllis Creek.
Pl	Placid Creek.
RhWT	Rhodamine WT, fluorescent dye manufactured by Du Pont.

RSQ	R-square, the fraction of total sample variance explained by a regression.
SoD	Sodium dichromate, $Na_2O_7CR_2 \cdot 2H_2O$.
s	Seconds.
UP	Up-surge, upstream of
X	Time-concentration curve measured here.

2. INTRODUCTION

2.1. Past and Present Approaches to the Runoff Problem

Runoff concentration denotes the process which transforms rainfall or snowmelt over a basin to stream discharge at the basin outlet. This transformation is an important and complex problem of hydrology, which has received considerable attention, but remains without a satisfactory, generally accepted solution. Hydrologists are often interested in peak flows at certain locations along a stream, but it is a fortunate accident if adequate stream flow records happen to be available for the desired site. In many cases, meteorological records have to be used to estimate peak rates of rainfall or snowmelt, which are then transformed to streamflow.

Two different approaches towards the precipitation-runoff transformation appear to be feasible, with the added possibility of combinations between the two. The simulation approach avoids the detailed physics of the runoff process by simulating it, or certain parts of it, with systems which may be classified into "black box" systems, conceptual models, or electric analogues. The alternative may be called the physical approach, as it involves dividing the runoff process into clearly identifiable subprocesses, which are dealt with on a physical basis. Amorocho and Hart (1964) discuss the various possibilities in detail.

The popularity of the "black box" systems approach, of which the Unit Hydrograph is the prime example, is a result of the widely held belief (or hope) that linear systems[1] represent most basin responses adequately. The theory of linear systems is quite advanced and it lends itself to an almost unlimited number of mathematical exercises.

Recently the number of non-believers has been growing[2] as evidence is accumulating that most basins have sufficiently strong non-linear effects to render the linear approximation dangerous (leading to underestimates of peak flows).

There are, however, further and better reasons for the use of "black box" systems. Some of the runoff concentrating processes take place over the entire basin area and may be extremely variable, rendering detailed physical description impractical (at least at present). A pure "black box" type systems approach to such a process, ignoring the physical aspects entirely, may lead to good results. The non-linearities of a basin may also be concentrated in a few

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[1]To be linear, a system has to satisfy the following conditions:

Assuming f_1 (t) and f_2 (t) are the responses to inputs f_3 (t) and f_4 (t) respectively, then (f_1 + f_2) is the response due to input (f_3 + f_4). The mathematical formulation of linear systems leads to linear differential equations.

[2]Numerous papers in the Proceedings of the International Hydrology Symposium, held at Fort Collins, Sept. 6-8, 1967 and in the Proceedings of the Symposium on Analogue and Digital Computers, Tucson, 1968, can be cited as evidence of this trend.

processes, so that even linear systems may give good approximations to others.

Much of the most recent work on runoff simulation is based on simple conceptual models of the total runoff process, such as linear or non-linear reservoirs, in series or in parallel, systems of uniform permeable soil layers, or systems based on one-dimensional dispersion (Overton, 1967; Sugawara, 1967; Diskin, 1967).

The major difficulty with all simulation approaches lies in the problem of parameter identification. Most systems, even quite primitive ones, have enough free parameters to permit close fitting to a particular set of rainfall-runoff data. The somewhat more severe test of representing runoff events of different magnitude from the same basin without requiring parameter adjustments, is also met by many of the recently proposed simulation systems, but to be really useful the parameters should have fixed relations with identifiable basin characteristics. This condition is not met by any presently available simulation model.

The physical approach consists essentially of identifying the processes that contribute significantly to the precipitation-runoff transformation, formulating the differential equations governing them, searching for practical solutions, and finally developing field and office procedures which supply the free parameters from readily available data.

Such a truly physical treatment of the total runoff process would avoid the identification problem, but it is unfortunately quite impossible at present, not so much for lack of understanding of the major processes as due to the complexity of most basins and the difficulty of separating the processes from each other. However, some important processes are both separable and fairly well understood, so that the physical approach has become possible and, by avoiding the identification problem, has superceded simulation. Examples of such processes are the propagation of flood waves in prismatic channels, surface runoff from paved surfaces, evaporation from large, deep lakes, and infiltration into uniform soils.

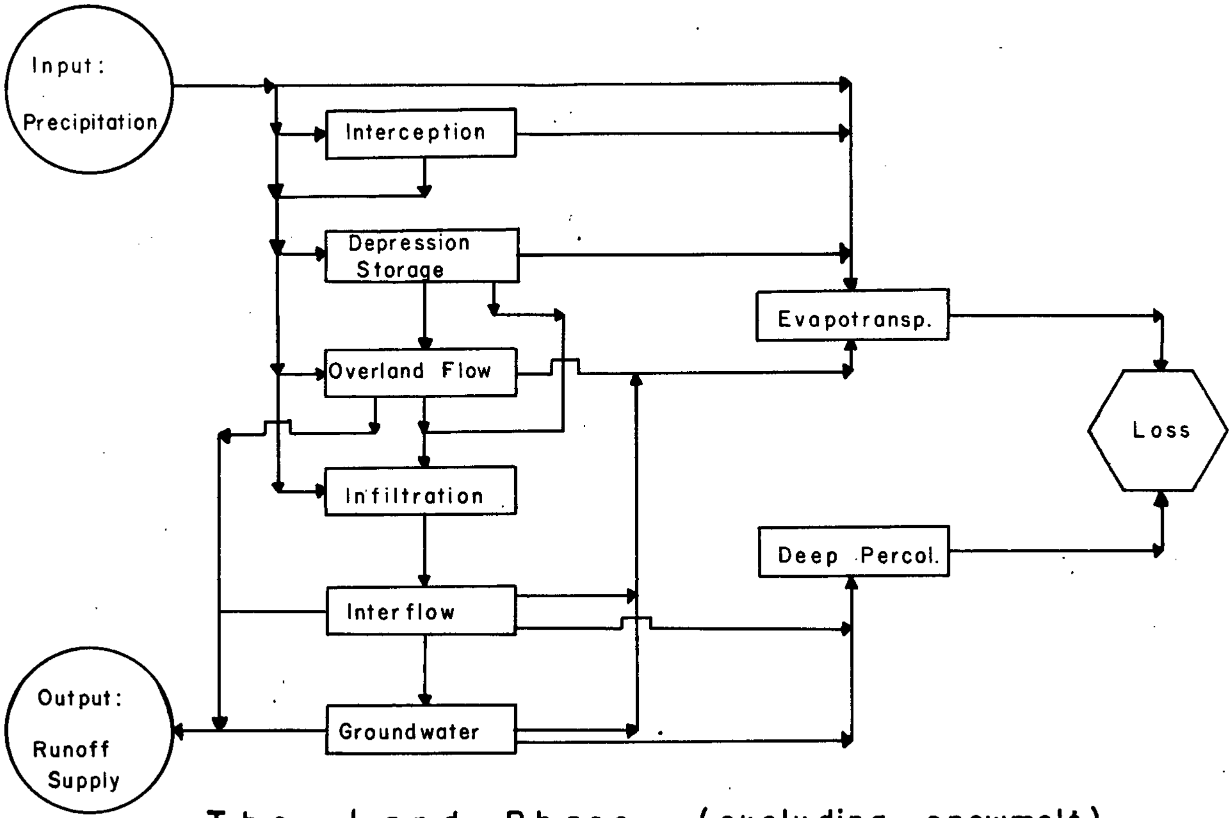
In conclusion it appears to the writer that the most profitable direction for new research on runoff processes lies in the physical field, aiming at a gradual replacement of simulation models with more closely representative physical models. The research project which forms the basis of this thesis was designed in accordance with this belief.

2.2 Separation of the Runoff Process into Land Phase and Channel Phase

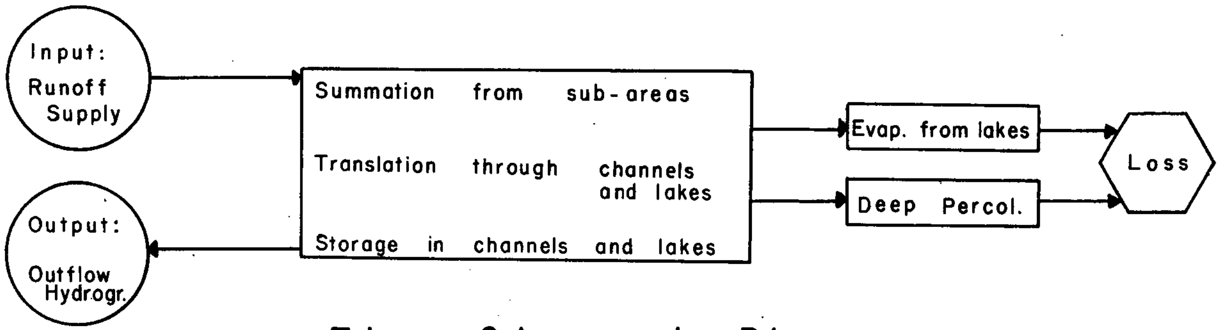
Larson (1965) suggests the separation of the runoff process into two phases; a land phase, which transforms rainfall or snowmelt to runoff supply (Larson's term for channel inflow), and a channel phase, transforming runoff supply to basin outflow. Figure 1 illustrates this division. The land phase is similar to the total runoff process from very small basins and includes all the complex interacting processes which can take place over the entire basin area, but should remain constant over regions of similar topography, vegetation and soil characteristics (e.g. evapo-transpiration, infiltration, interflow, etc.). In basins with negligible water losses out of the channel system, which includes most basins in the humid zone, the channel phase is dominated by the single process "wave propagation in open channels".

For representation of the land phase the simulation approach appears to be best suited under the present circumstances. The parameters can be evaluated on the basis of rainfall-runoff data from small test basins (small in the sense that the channel phase is negligible) in the region of interest. Parameter consistency is not an absolute necessity due to the assumption that the land phase is regionally constant.

Under conditions of heavy rainfall on fairly impermeable or thoroughly wet basins with high drainage density, the land phase may even be reducible to "Rainfall-Overland Flow-Runoff Supply" (Figure 1) with small losses and negligible time lag. The effect of the land phase on the outflow hydrograph in the vicinity of the peak may then be negligible. In most cases one has to assume that the land phase controls the volume of runoff and contributes in a non-negligible



The Land Phase (excluding snowmelt)



The Channel Phase

THE TWO RUNOFF PHASES

Fig. I

manner to the shape of the hydrograph.

Larson's basic assumptions which, if proven, certainly will justify the two-phase approach, are that the channel phase accounts for the differences between basins due to size and shape and that the dominant process in that phase (wave propagation) may be fully representable from readily available data. He envisaged this roughly as follows: Maps show the channel network in plan; channel dimensions and roughness can be obtained in the field or on large scale air photos. Alternatively maps can be used to make rough estimates of channel forming discharge and the considerable body of knowledge on relations between size, performance and discharge of self-formed channels will permit estimates of the necessary parameters. Once the dimensions of the channel system are estimated, standard methods of flood routing should give good representations of the channel phase.

The two phase approach cannot be considered operative at present. The usefulness of the concept hinges on the physical representation of the channel phase. If this can be done, then the concept does represent a step forward by removing one part of the runoff process from the realm of speculation and simulation. Larson does not seem to have realized this clearly, as his channel phase is based on long disproven concepts such as using a single constant value of Manning's n for large flow ranges in many natural channels or assuming flood wave movements at mean water velocity, and he never investigates the crucial question whether the parameters of his channel phase representation, as obtained by curve fitting, are related to the field values.

2.3 The Objective of This Study

For several reasons, the two phase approach appears *a priori* to be particularly suitable in mountainous areas. The channel system is generally well developed and easily traceable on maps. In addition to a plan of the channel network with contributing drainage areas, maps also supply channel slopes, one of the most important parameters in any type of flow routing. Soil cover is often thin and lying on steep, impermeable layers, resulting in a fast-acting and therefore less important land phase. If runoff originates mainly in glaciers and snowfields, the land phase can be replaced by an ice-phase, supplying water to the channel system at discrete locations, but the general approach remains valid.

The major obstacles to applying the two-phase concept in mountainous areas are:

i) The present lack of information on the formative laws and hydraulic performance of steep channels characterized by alternating super- and sub-critical flow and by energy dissipation due to rapid changes in cross section and slope, (Peterson and Mohanty (1960) introduced the very descriptive term "tumbling flow" for this flow regime. Photographs 1 to 6 illustrate tumbling flow.) and

ii) The lack of a realistic routing method, which does not require virtually unobtainable information on the channel system.

The present thesis represents an attempt to solve these problems by investigating the physical laws governing steady and unsteady flow in steep channels and by trying to state them in such a form that all parameters can be obtained from data readily available even in ungauged basins. The details of the link between the flow parameters and map measures are discussed in Day (1969). His links are statistical but based on considerations of dynamic similarity.

The problems were approached empirically, starting with field measurements and concluding with analysis of the data, a sequence which has been maintained in this write-up.

2.4 Assumptions regarding Readily Available Data

The design of this project is based on the assumption that ungauged basins have (i) map coverage, (ii) high altitude air photo coverage and (iii) data on precipitation and runoff for at least one location in the same climatic region.

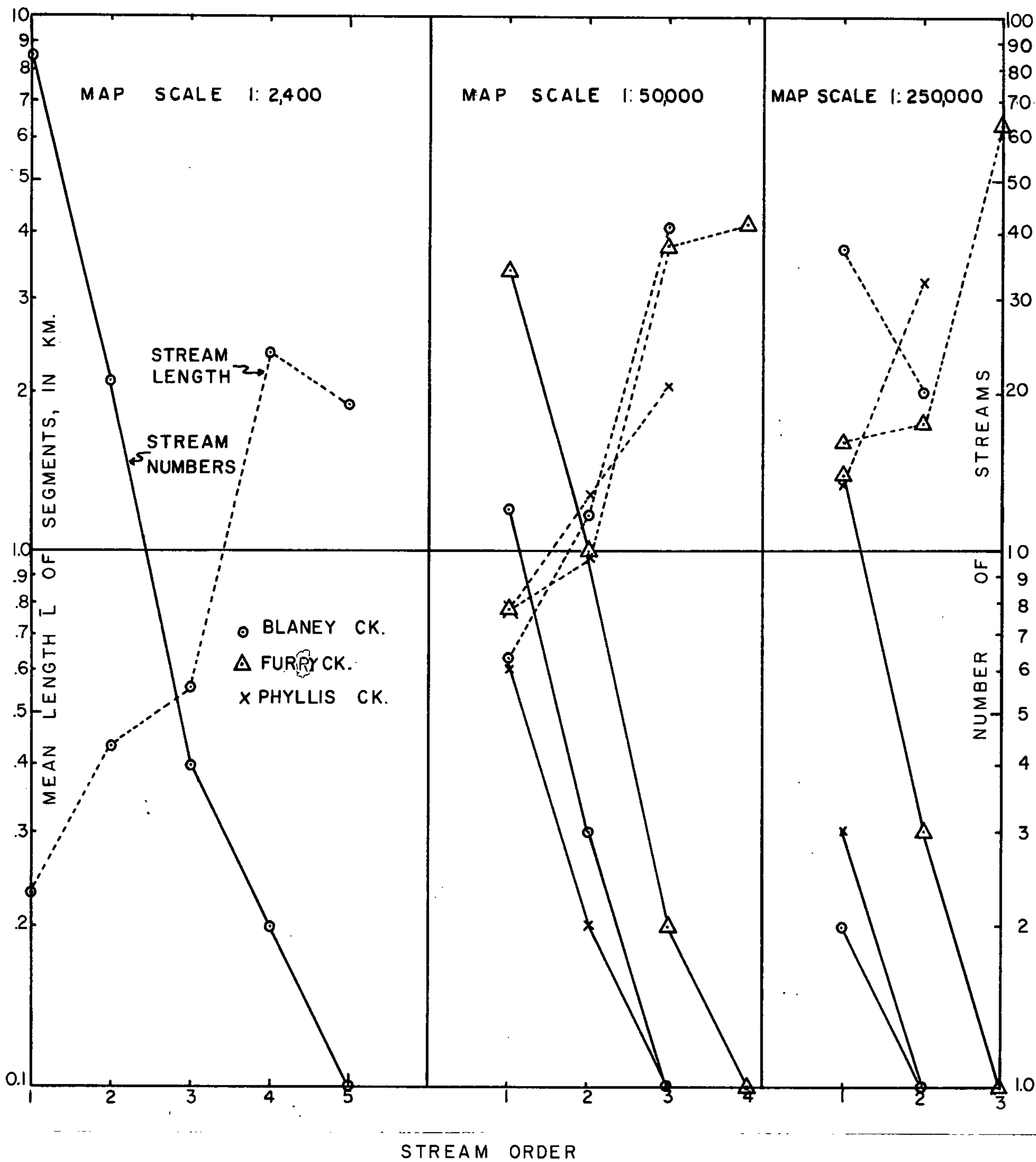
Some of the channel phase models developed subsequently require a very rough estimate of mean annual peak flow, or a high flow of some other frequency. Item(iii), combined with map information, should permit such an estimate to $\pm$ 50%.

Map coverage supplies the following information: (i) a plan of the channel network, (ii) contributing drainage areas all along each channel, and (iii) channel slopes. The accuracy of this information depends primarily on the scale and contour interval of the maps, but other factors, such as the procedures used in map making and the height and density of ground cover in the case of maps made from air photos, may also be important (Morisawa, 1957; Scheidegger, 1966).

The standard Canadian map scales are 1:50,000 with 50 ft contour interval and 1:250,000 with 500 ft contour interval. Only a small fraction of the Canadian Cordillera has 1:50,000 coverage but this includes most developed areas and highway routes.

To gain some idea of the degree to which these two map scales represent drainage networks in south-coastal B. C., three of the basins used in this study (Furry Creek, Phyllis Creek, Blaney Creek) were analysed morphometrically for stream orders, number of streams and mean stream lengths. One basin (Blaney Creek) could also be analysed on maps to a scale of 1:2400 which, based on a few spot checks, appear to give a reasonably true representation of the drainage system, including first order streams which contain some flow during most of the wet season. (The Slesse and Ashnola basins could not be included in this analysis because they lie partly in the United States, where the accuracy and scale of the map coverage is significantly different.)

The results of the morphometric analysis are shown in Table 1 and on Figure 2. The channel network measurements are



MORPHOMETRY OF THREE BASINS AT DIFFERENT MAP SCALES

Fig. 2

TABLE I

COMPARISON OF MORPHOMETRY BASED ON THREE MAP SCALES

Map Scale and Type	Stream Order	Furry Creek		Phyllis Creek		Blaney Creek	
		No. of Streams	Mean (km) Length	No. of Streams	Mean (km) Length	No. of Streams	Mean (km) Length
1:250,000 NTS	1	14	1.62	3	1.35	2	3.75
	2	3	1.75	1	3.25	1	2.00
	3	1	6.50				
1:50,000 NTS	1	34	0.785	6	0.79	12	0.63
	2	10	0.985	2	1.27	3	1.18
	3	2	3.800	1	2.05	1	4.10
	4	1	4.100				
1:2400 UBC Research Forest, Topography and Forest Cover.	1					86	0.23
	2					21	0.44
	3					4	0.56
	4					2	2.38
	5					1	1.90

Notes:

- Figure 2 shows the same data graphically.
- The channel network is based on the blue lines shown as the maps with additions based on the contour picture.
- Placid Creek is part of the Blaney Creek basin.

based on the blue lines of the maps with some additions and extensions based on the contour picture. (Morisawa, 1957). Lakes are replaced by stream segments. Since running water is not a chief agent in developing the surface geometry of these basins, it is not surprising that Figure 2 does not show logarithmic relations between stream order and stream number or stream length, as found by Horton and others in many basins. However, with increasing basin size and increasing map scales, the relations become more closely logarithmic.

Figure 2 indicates that the 1:50, 000 maps miss most first order and some second order streams, while the 1:250, 000 maps miss the first, second and part of the third order. These relations may be different in other basins. If possible, a similar comparative study should be made before data obtained on a large scale map are extrapolated to first order channels.

Figure 3 illustrates the extent to which channel slope is obtainable from maps of various scales by comparing the survey results (hand level profiles) of 2 test reaches with profiles from maps of scales 1:2400 and 1:12,000.

Air photos are considered part of the essential data for ungauged basins because the Canadian maps provide little information on vegetative cover and on bed-rock exposures. Particularly at the lower end of hanging valleys, some streams flow directly on bed-rock, often without as much as a minor canyon. This does affect the channel performance considerably. Dense tree cover and the presence of logging slash result in

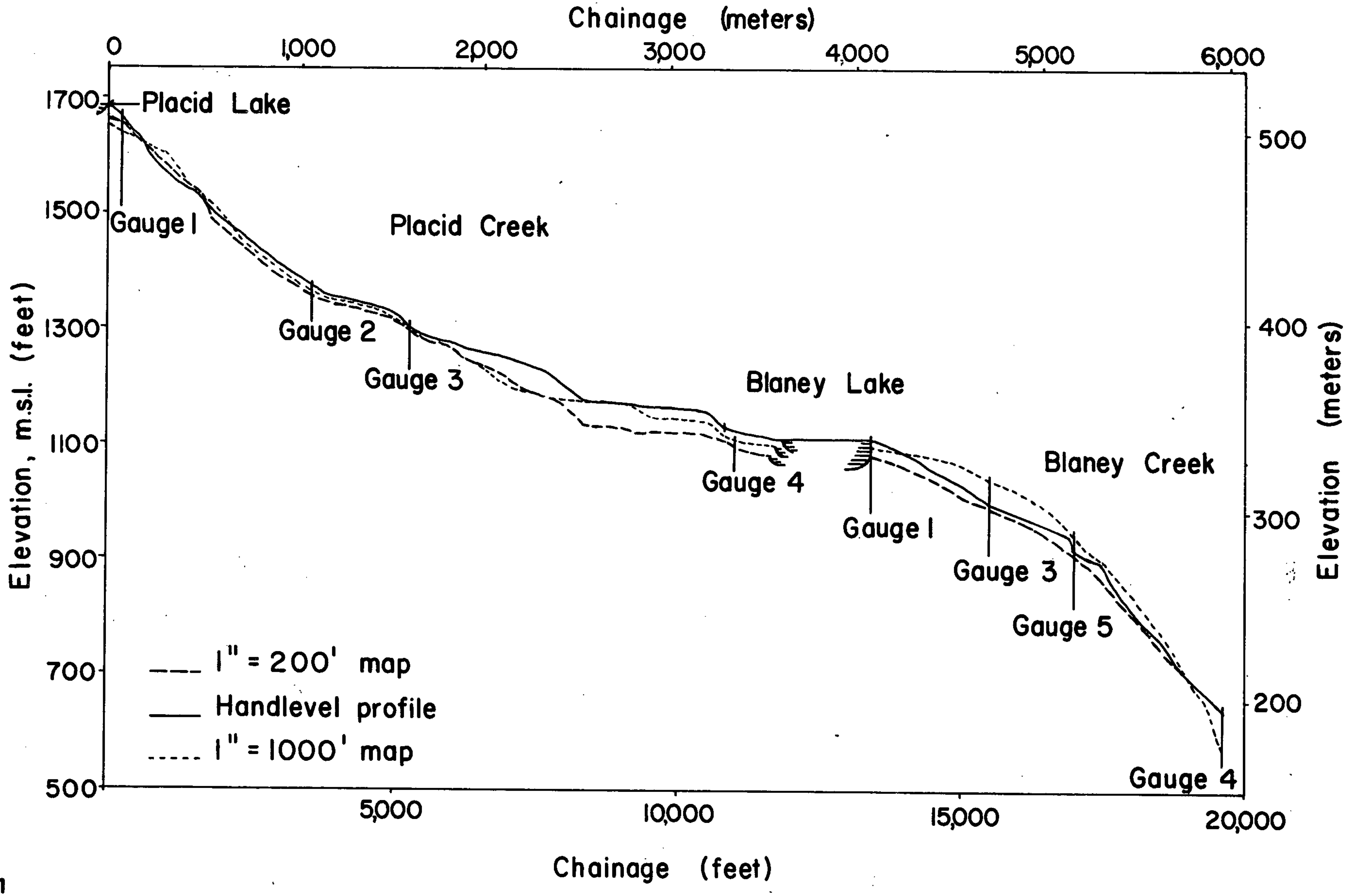


Fig. 3

COMPARISON BETWEEN CHANNEL PROFILES MEASURED OFF MAPS AND SURVEYED IN THE FIELD frequent log jams in small streams, again affecting channel performance. The present data are not adequate to define the conditions under which log jams become significant, but it appears that at a channel width of approx. 12 m to 15 m log jams cease to be significant, at least in south coastal B. C. Channel slope has also a pronounced influence on the formation of log jams, with flat reaches being generally more debris choked.

3. FIELD METHODS

3.1 Selection of Test Reaches

The criteria for selection of test reaches were as follows:

(i) The data were to cover the largest possible range of size (width) and slope.

(ii) To test the unsteady flow behaviour, upstream lakes with outlets suitable for minor flow modifications were required.

(iii) Since measurements were to be made over as large a discharge range as possible, easy accessibility from Vancouver was also an important consideration.

Four streams, covering a range of mean width from 0.89 m to 14.0 m were finally selected and 2 to 5 test reaches were established on each stream, covering a fairly wide range in slope. (See Table 2 for a list of reaches.) None of the 4 streams had previous discharge records. The smallest stream (Brockton Creek) has no official name and does not appear on any map. Due to its location at tree line it is not affected by forest debris (Photograph 1). At flows in the order of the mean annual flow, some water spills out of the stream channel proper on the upstream test reach and follows the stream in some parallel minor depressions.

Placid Creek in the UBC Research Forest flows through an area covered by dense second growth forest approximately

TABLE II

TEST REACHES BELOW LAKES

Creek	Location (mid-reach)	Reach (going down-stream)	Length (m)	Drop (m)	Slope $\sin\theta$ $\times 10^{-3}$	No. of Steps in Survey	Width W_D (m)	Coeff. of Variation for W_D	Drainage Area (km^2)	Estimated Mean Annual Peak (m^3s^{-1})
Brockton Creek	Mt. Seymour Park, Elev. 4,000'	Br 1-2	119.0	8.8	74	36	0.89	.499	0.0655	0.20
	Lo: 122°56' Lat: 49°23'	Br 2-3	80.5	28.1	349	27	0.99	.611	0.0880	0.20
Placid Creek	UBC Research Forest,	Pl 1-2	960	79.7	83	64	2.75	.387	0.614	1.5
	Elev. 1,400' Lo: 122°34'	Pl 2-3	610	21.6	35.5	41	3.16	.373	1.17	2.5
	Lat: 49°18.5'	Pl 3-4	1844	62.4	33.9	122	7.02	.400	2.60	5.0
Blaney Creek	UBC Research Forest,	Bl 1-3	685	31.9	46.6	46	12.76	.435	7.43	12.0
	Elev. 950' Lo: 122°34.5'	Bl 3-5	335	17.5	39.0[1]	23	11.06	.414	7.70	12.0
	Lat: 49°17'	Bl 5-4	930	85.3	94.7	62	12.92	.292	7.94	13.0
Phyllis Creek	Nr. Britannia Beach, B.C.	Ph 1-2	770	23.7	30.5	52	11.48	.314	8.69	15.0
	Elev. 1,400' Lo: 123°11'	Ph 2-3	716	34.9	48.7	48	12.57	.216	10.41	17.0
	Lat: 49°34'	Ph 3-4	617	39.5	64	42	12.64	.190	10.99	17.0
		Ph 4-6	305	30.2	99	21	12.28	.226	11.34	18.0
		Ph Lo	140.5	30.8	219	10	14.04	.167	11.81	19.0

[1]This reach has a sudden steep drop in the last 40 m. The slope of the long flatter part is shown here.

20 years old and its flow regime is affected by frequent log jams (Photograph 2). The lowest Placid reach flows through two bulldozed fire pools, and the slope is irregular.

Blaney Creek flows through an area that was burnt over approximately 80 years ago. Its flow regime is affected by log jams at approximately 150 m intervals (Photographs 3 and 4).

The valley of Phyllis Creek was logged 20 to 30 years ago but the creek seems to be of sufficient size to have cleared itself of most debris. Only the flattest top reach has 2 log jams (Photographs 5 and 6).[1]

In addition to these 13 test reaches, which were, except for two, tested for steady and unsteady flow, Day (1969) tested another 12 reaches for steady conditions only. As he did not require upstream lakes, the choice was much wider and the reaches are in general more satisfactory for the purpose of steady flow tests. Some of Day's streams have discharge records and the test reaches are usually more uniform in slope. Table 3 lists the 12 reaches tested by Day.

The test reach length varies from 80.5 m to 1844.0 m and the length to width ratios vary from 10 and 349, but most reaches have ratios between 30 and 80.

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[1]The lowest reach on Phyllis Creek was tested by Mr. T. Day. For convenience, the results are presented here, together with all other Phyllis reaches.

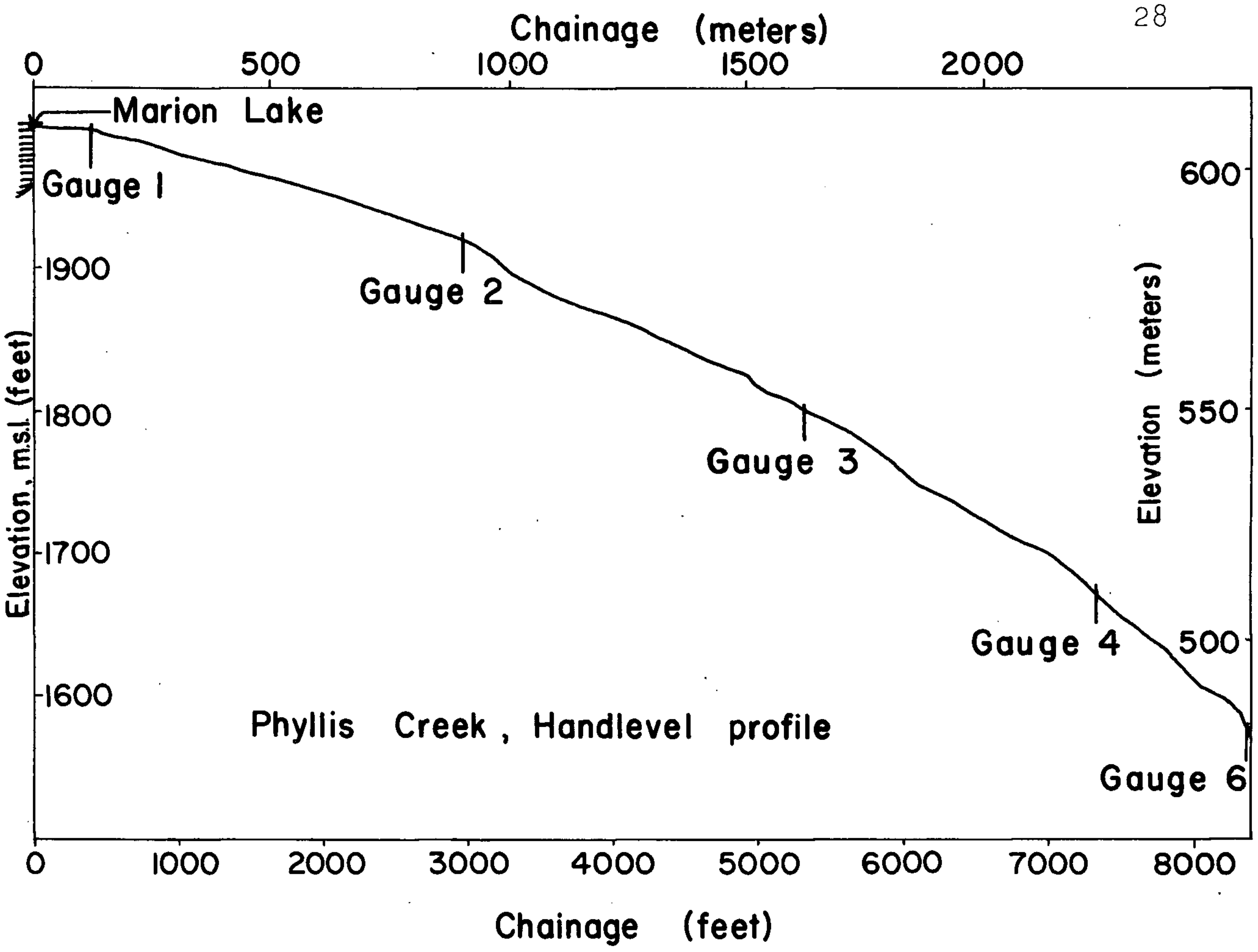
TABLE III

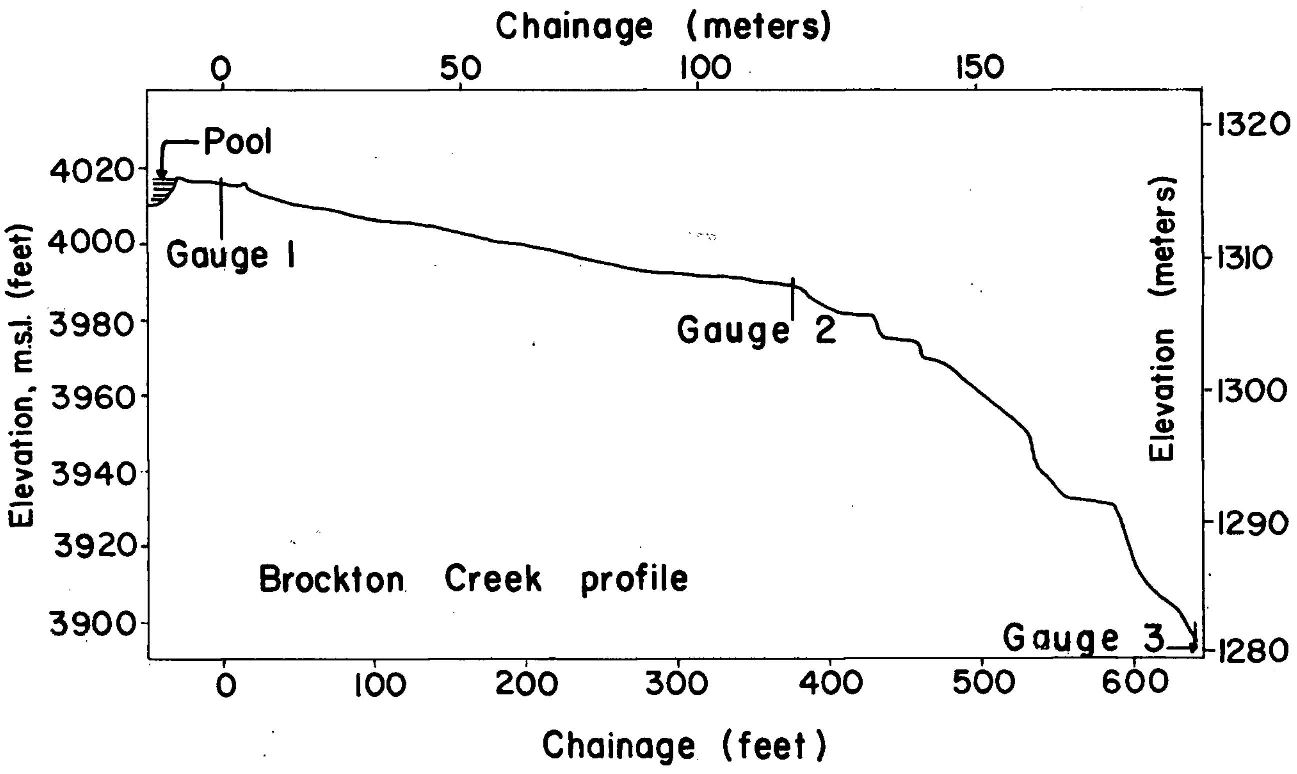
ADDITIONAL TEST REACHES (from Day, 1969)

Creek	Location	Reach	Length (m)	Drop (m)	Slope sin θ x10^3	No. of steps in survey	Width W_D (m)	Coeff. of Variation for W_D	Drainage Area (km^2)
Fury Creek	Near Britannia Beach, B. C. Lo:123°12' Lat:49°35'		229	29.5	129	16	20.2	.304	39.0
Slesse Creek	South of Chilliwack, B.C.	Upper	584	20.0	34.3	21	27.0	.222	105.1
	Lo:121°38' Lat:49°01' Lo:121°38' Lat:49°01'	Mid	541	23.5	43.3	18	22.8	.239	110.4
	Lo:121°39' Lat:49°02'	Low	1402	47.0	33.5	47	19.5	.285	126.0
Juniper Creek	South of Keremeos, B.C. Lo:120°02' Lat:49°06'		610	81.5	134	41	6.03	.407	21.8
Ewart Creek	South of Keremeos, B.C. Lo:120°02' Lat:49°06'	Upper	579	20.2	34.8	19	16.36	.267	80.8
	Lo:120°02' Lat.49°08'	Low	1125	45.9	40.8	36	14.2	.302	95.8

TABLE III (cont'd.)

Creek	Location	Reach	Length (m)	Drop (m)	Slope sin θ x10^3	No. of steps in survey	Width W_D (m)	Coeff.of Variation for W_D	Drainage Area (km^2)
Ashnola River	South of Keremeos, B.C. Lo:120°11' Lat:49°09'	Upper	490	5.09	10.4	17	21.1	.170	221.5
	Lo:120°10' Lat:49°10'	Mid	1003	26.0	25.9	33	28.2	.218	408.5
	Lo:120°10' Lat:49°10'	Low	747	26.0	35.1	25	22.1	.225	409.5





CHANNEL PROFILES

Fig. 4

With the possible exception of the most upstream reach on Blaney Creek, none of the test reaches have flood plains.

3.2 Survey Measurements

While the objective of the thesis calls for definition of the channel phase on the basis of map and air photo data, it was nevertheless considered necessary to measure length and slope in the field, mainly because the reaches could not be located adequately on maps, but also to compare map profiles with field data (Section 2.5).

Length and slope were usually obtained simultaneously by chaining and hand-levelling in 50 ft. or 100 ft. steps. The profile points are the deepest parts of the channel where possible, otherwise water surface. The step size is sufficiently long to eliminate most of the characteristic pool-riffle sequence. Aneroid measurements were used as a check against large levelling errors. Figures 3 and 4 show the profiles of the 13 test reaches of this study.

Originally it had been planned to measure two hydraulic parameters, width and roughness, but no satisfactory method for measuring roughness could be designed. When it became apparent from considerations of dynamic similitude (Section 5.1) that roughness is a redundant parameter in a hydrological flow model, attempts to measure it were abandoned.

The channel width, from high-water mark to high-water mark, was measured at each profile point (50 ft or 100 ft intervals). As indicated by Photographs 1 to 6, channel width is fairly well defined by a line of permanent vegetation. The high-water mark follows this line closely. It generally consists of an abrupt change from forest floor or meadow to channel bed (exposed gravel). Tables 2 and 3 show the mean values and coefficients of variation for each reach. Most measurements were made with a survey tape, but towards the end of the field work a rangefinder became available, which proved to be very suitable for this type of survey.

3.3 Tracer Methods

3.3.1 Objective

The main objective of the tracer measurements was to establish the relations between discharge, velocity, and channel area under conditions of uniform flow and covering the largest possible range of discharge.

At times tracer methods were also used for simple local discharge measurements, which were needed to define the stage-discharge rating curves as mentioned in Section 3.4.5.

3.3.2 Principles of Discharge and Velocity Measurements with Slug Injection Methods

Tracer methods are ideally suited for measuring the channel flow variables which are significant in runoff studies, namely discharge, mean velocity, and channel storage (area).

If a slug of tracer of mass M is injected into a stream of discharge Q and the tracer concentration C (mass per unit volume) is measured at a location sufficiently far downstream to permit the assumption of complete lateral and vertical mixing (see Section 3.3.2), the principle of conservation of mass takes the following form (Replogle *et al.*, 1966)

$$M = \int_{t_s}^{t_e} Q\ C\ dt \qquad \ldots\ 3.1$$

in which t_s is the arrival time of the tracer, and t_e is the time at which all the tracer has passed the sampling site. If Q is steady during the interval $t_s - t_e$, Equation 3.1 gives

$$Q = M \Big/ \int_{t_s}^{t_e} C\ dt \qquad \ldots\ 3.2$$

which shows that Q can be measured by injecting a known volume of tracer into a stream and observing or sampling the time-concentration curve at a downstream location.

The mean travel time T_t of a tracer cloud between the point of injection and the sampling location is (Thackston *et al.*, 1967)

$$T_t = \int_{t_s}^{t_e} C\ t\ dt \Big/ \int_{t_s}^{t_e} C\ dt \qquad \ldots\ 3.3$$

and this is only identical to the mean travel time of the stream water, T, if instantaneous vertical and lateral mixing at the point of injection can be assumed with no tracer being dispersed upstream or if the tracer is injected above the test reach and T is obtained as the difference between the

T_t- values for the upstream and downstream end points of the reach.

The mean water velocity along the reach of length l is

$$v_m = \frac{l}{T} \qquad \ldots\ 3.4$$

and from continuity, the channel area of the test reach becomes

$$A = \frac{Q}{v_m} = \frac{QT}{l} \qquad \ldots\ 3.5$$

3.3.3 Vertical and Lateral Dispersion Requirements

The mechanics of vertical and lateral dispersion in straight, uniform open channels is well developed (Diachishin, 1963; Fischer, 1966). Criteria for the time or distance which assure adequate[2] mixing have been developed but their application to tumbling flow is not reasonable. Empirical methods, such as sampling across the channel and visual inspection of the dispersion of dyes were therefore used to determine whether the lateral mixing requirements were being met. If possible, the test reaches were located so that they started at severe channel discontinuities (waterfalls, constrictions), which assure fast mixing. On some of the short reaches it was necessary, however, to inject the tracer above the reach and

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[2]Complete dispersion is not possible in finite time due to the continuous nature of the process (see Equations 3.7 and 3.13).

to observe two time-concentration curves for one measurement of v_m.

On several occasions the velocity of long reaches was measured twice at closely similar flows by injecting the tracer at the starting point of a reach and by injecting further upstream. There are no significant discrepancies between these results.

3.3.4 Longitudinal Dispersion Models

The tracer techniques outlined in Section 3.3.2 require the evaluation of integrals over observed time-concentration curves. Longitudinal dispersion is primarily responsible for the shape of these curves. If the observed C (t) - curves cover the interval t_s - t_e adequately for numerical integration, the mechanics of dispersion can be ignored. However, in the course of this study it happened frequently that field observations had to be terminated before C (t) had declined to negligible values and it became necessary, therefore, to develop a dispersion model which would permit extrapolation. Particularly in the equation for T_t (Equation 3.3), the decline of C at large values of t carries considerable weight.

The three main processes causing longitudinal dispersion are: longitudinal turbulence, turbulent mass exchange between stream lines of differing velocities, and storage of tracer in pools and dead zones. Molecular diffusion is only important at extremely small scales.

Taylor (1954) showed that the one-dimensional diffusion equation gives a fairly good representation of longitudinal dispersion in uniform, turbulent pipe flow. Elder (1959) extended the analysis to infinitely wide open channels and Fischer (1966), Church (1967), Thackston and Krenkel (1967), and many others have examined its applicability to natural channels.

The one-dimensional diffusion equation is

$$\frac{\partial C}{\partial t} + v_m \frac{\partial C}{\partial x} = D_x \frac{\partial^2 C}{\partial t^2} \qquad \ldots\ 3.6$$

in which x is the longitudinal coordinate, and D_x is the dispersion coefficient. For slug injection of a tracer of mass M at t = o, x = o, the solution takes the following form after vertical and lateral mixing are almost complete

$$C(x,t) = \frac{M}{A\sqrt{2\pi D_x t}} \exp\left(-\frac{(x - v_m t)^2}{4 D_x t}\right) \qquad \ldots\ 3.7$$

It shows that the tracer is distributed normally over x, with the centre moving downstream at velocity v_m and the variance increasing as $2D_x t$. Most dispersion data are based on observation of C at constant x, x = 1. Under these conditions, Equation 3.7 is skewed to the right, which agrees with field data.

Substituting 3.7 into 3.3 gives

$$T = \frac{1}{v_m} + \frac{2D_x}{v_m^2} \qquad \ldots\ 3.8$$

in which the second term on the right accounts for the fact that initially some tracer may be dispersed upstream (Thackston et al., 1967). Although $2D_x/v_m^2 << 1/v_m$ throughout this study, Equation 3.8 is further proof of the need for fast initial mixing.

Equation 3.7 was fitted to several observed time-concentration curves, using the least squares fitting method proposed by Thackston et al. (1967).[3] Figure 5 shows a typical fit. The agreement between Equation 3.7 and the field data is generally close over most of the C (t)-curves but the predicted final decline of C is always much faster than observed declines, indicating that the one-dimensional diffusion equation (3.6) does not really represent dispersion in natural channels.

Hays (1966) developed a new model, which includes dead zone storage effects besides one-dimensional diffusion. It appears to represent the slow decline of C very well, but unfortunately, it is rather difficult to handle, requiring a Fourier transformation of the field data and subsequent curve fitting in frequency space.

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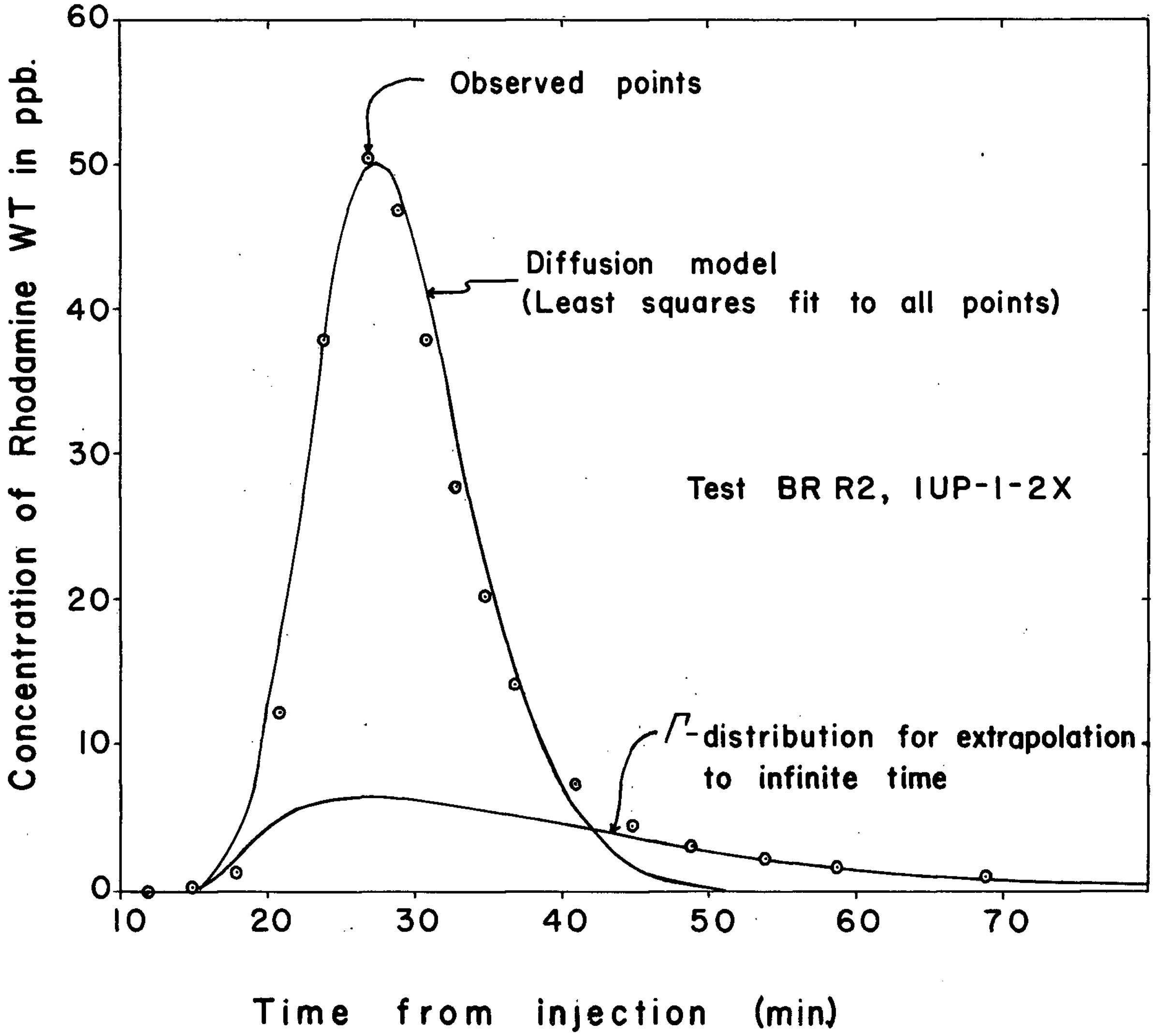
[3]This method is based on the IBM Share library programme NLIN2, described by Marquard (1964).



LONGITUDINAL DISPERSION OF SLUG INJECTED TRACER

Fig. 5

3.3.5 A Gamma-Distribution Model for the Final Decline of C(t)*

Since the field data of the present study define the main part of the C(t)-curves adequately, an attempt was made to develop a simple model for the final decline, considering only tracer storage in pools. It is based on the assumptions:

(i) The stream acts as a cascade of reservoirs with steady flow Q.

(ii) All reservoir volumes R_i are equal to $T_R\, Q$, with T_R, the filling time, being an arbitrary time constant.

(iii) Mixing is instantaneous in each reservoir.

(iv) The dispersion process is initiated by injecting a quantity M of tracer into the first reservoir (R_o) at time $t_s = 0$. The initial concentration in R_o is therefore

$$C_o\ (t = 0) = \frac{M}{T_R Q} \qquad \ldots\ 3.9$$

(v) The travel time between two reservoirs is constant for all water or tracer particles and can therefore be ignored in the following.

These assumptions lead to a system of linear, non-homogeneous differential equations of first order for $C_i(t)$.

The general form is

$$\frac{dC_i}{dt} = \frac{C_{i-1}}{T_R} - \frac{C_i}{T_R} \qquad \ldots\ 3.10$$

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*Through a reference in Water Resources Research, Vol. 5, No. 4, p. 927, August 1969, a paper by MacMullin and Weber (Trans. Am. Inst. Chem. Engrs., Vol. 31, pp. 409-458, 1935) has recently come to the writer's attention. It contains an identical derivation, based on considering the outflow from a series of well-mixed vessels.

The solutions are

Reservoir R_o $$C_o(t) = \frac{M}{T_R Q} e^{-\frac{t}{T_R}}$$

Reservoir R_1 $$C_1(t) = \frac{M}{T_R^2 Q} t e^{-\frac{t}{T_R}}$$

Reservoir R_i $$C_i(t) = \frac{M}{i!T_R^{i+1}Q} t^i e^{-\frac{t}{T_R}} \quad \ldots 3.11$$

The successive peaks occur at $t = iT_R$ and the mean tracer travel time is $T_t = (i + 1) T_R$.

Equation 3.11 can be compared to the gamma distribution

$$f_x(t) = \frac{\kappa}{\Gamma(r)} (\kappa t)^{r-1} e^{-\kappa t} \quad \ldots 3.12$$

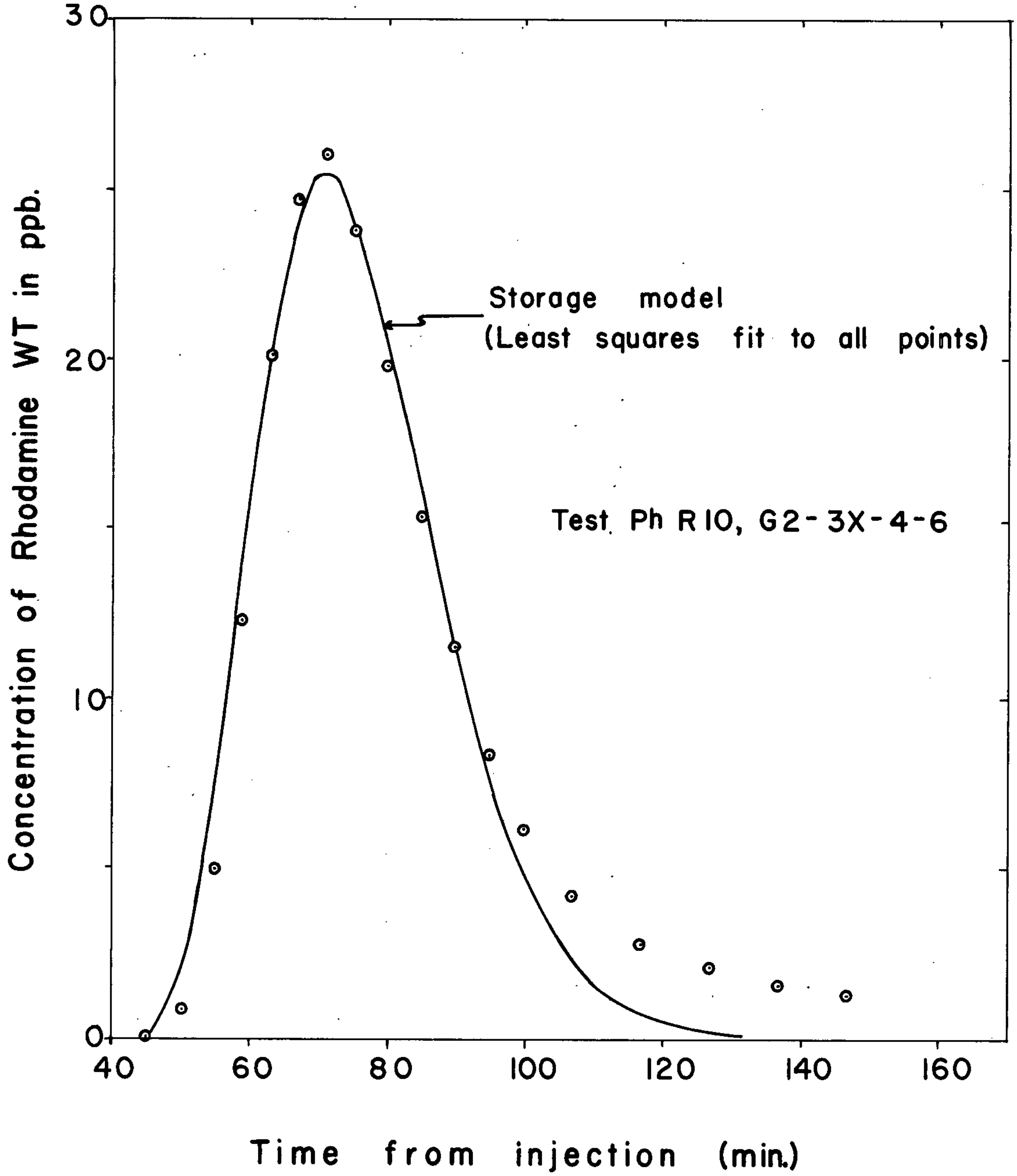
$$r > 0$$
$$\kappa > 0$$
$$x > 0$$

Keeping in mind that $\Gamma(r + 1) = r!$ for $r = 1,2,3 \ldots$ Equation 3.11 can be rewritten as

$$\frac{C_i(t)Q}{M} = \frac{1}{T_R \Gamma(i+1)} \left(\frac{1}{T_R}\right)^i t^i e^{-\frac{t}{T_R}} \quad \ldots 3.13$$

which shows that the general solution is proportional to a Γ-distribution with parameters $(i + 1)$, $(1/T_R)$, and t.

Initially the question of whether this storage model could represent the total time-concentration curves was explored by fitting it to several sets of field data (Figure 6). In comparing it with the diffusion model one may



THE STORAGE MODEL APPROXIMATION TO LONGITUDINAL DISPERSION

Fig. 6

say that:

(i) Both models can be fitted almost equally well to the observed time-concentration data, but both fail to represent the slow decline of C at large values of t.

(ii) The storage model can account for the finite time lag between tracer injection and the first arrival at the sampling location. In the diffusion model the tracer covers the whole reach immediately.

(iii) The diffusion model gives better parameter stability. The number of reservoirs in the storage model does not necessarily increase with channel length, neither does T_R, the reservoir filling time, decrease with flow Q.

(iv) The third moment ratio of the Γ-distribution is $2/\sqrt{r}$, indicating that with increasing channel length the skewness of C (t)-curves should decrease, which does not appear to be consistent with field results.

In spite of these deficiencies, the storage model can represent the final decline of C (t), if the fit over the main part of the C (t)-curves is ignored, which amounts to splitting the dispersion phase into two parallel phases; a dispersion phase, responsible for moving most of the tracer and a storage phase, which dominates the final decline.

Equation 3.11 is essentially of the form

$$C(t) = Y_1 \, t^{Y_2} \, e^{-Y_3 t}$$

in which the Y_i are constants.

Taking logarithms on both sides gives

$$\log_{10} C = \log_{10} Y_1 + Y_2 \log_{10} t - Y_3 t$$

which shows that (3.11) can be tested by plotting the field data in the form ($\log_{10} C - Y_2 \log_{10} t$) vs. (t), for selected values of Y_2. Equation 3.11 is a good fit if the data points fall on a straight line. With very few exceptions, which are attributable to the difficulties of determining low C values, the field data plot as shown by the two examples of Figure 7. Some similar computer-made plots are shown in the Appendix under Subroutine "TAILEX".[4]

The final decline of C (t) appears to be similar to a Γ-distribution with $1 \leq r \leq 3$. A good fit was generally achieved by setting r = 2 (i = 1), but r = 1 (negative exponential decline) could have been selected with almost equal justification. Figure 7A shows a set of data which covers almost the complete decline of C (t) and Figure 7B illustrates the graphical fitting of a Γ-distribution extension to an incomplete set of C (t)-data. On Figure 5 the resulting curve is plotted in C - t coordinates. Similar computer-made plots are shown in the Appendix under Subroutine "PLØTGA".

The above storage model represents only a small first step towards an understanding of longitudinal dispersion in

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[4]The parameters Y_1, Y_2, and Y_3 appear as A, B, and C in the Appendix.

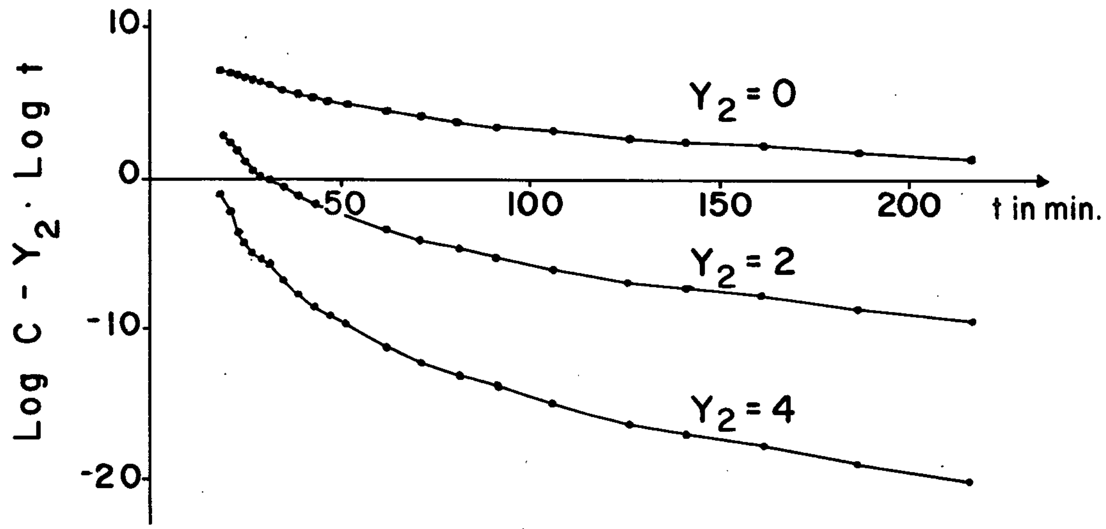

Fig. 7A Test covering C(t) decline almost completely

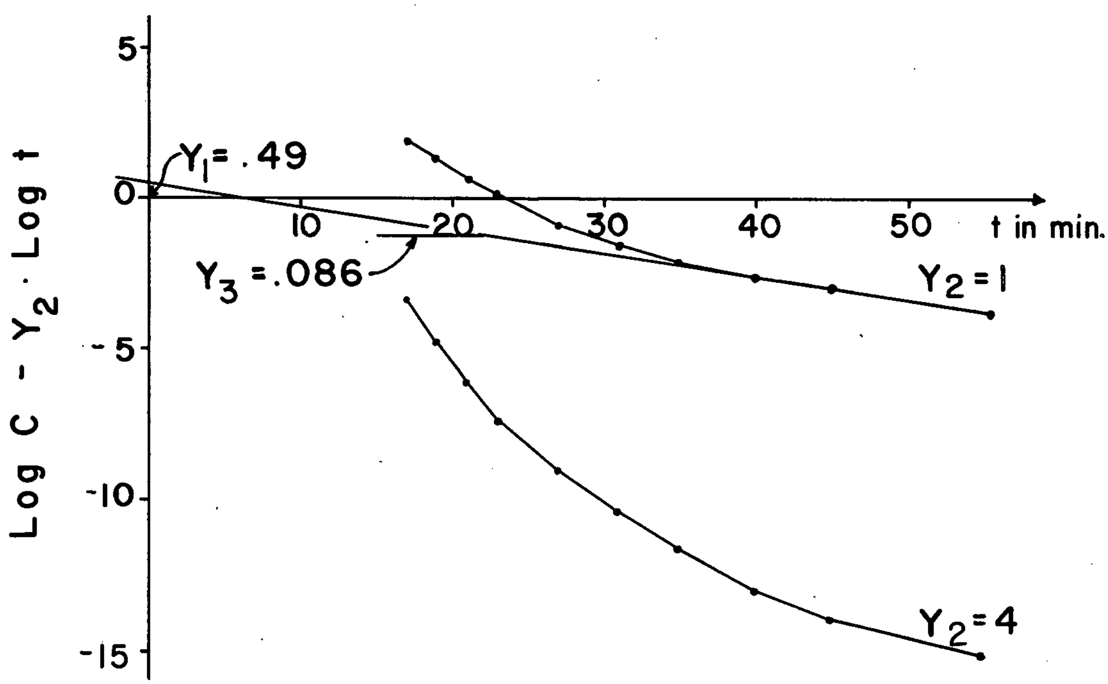

Fig. 7B Incomplete test with Γ- extension
Test BR R2, GIUP-1-2X

GRAPHICAL FITTING OF STORAGE MODEL

Fig. 7

tumbling flow channels. The large number of C (t)-curves measured in the course of this study should permit a more complete investigation, concentrating on the predictive qualities of the storage model, but this is not part of the present objective.

3.3.6 Equipment and Procedure for Slug Injection Measurements

All the C (t) curves included in this study were measured with either one of the following methods:

(i) the relative salt dilution method, based on electrical detection of a Na Cl-solution;

(ii) the dye dilution method, based on fluorometric detection of a fluorescent tracer (Rhodamine WT).

A detailed description of both methods, based partly on the experience gained in the course of this study, is in press (Church and Kellerhals, 1969). Only a brief summary will be given here.[5]

The relative salt dilution method (Aastad and Sögnen, 1954; Østrem, 1964) uses the linear relation between concentration of salt and conductivity. A known volume (generally

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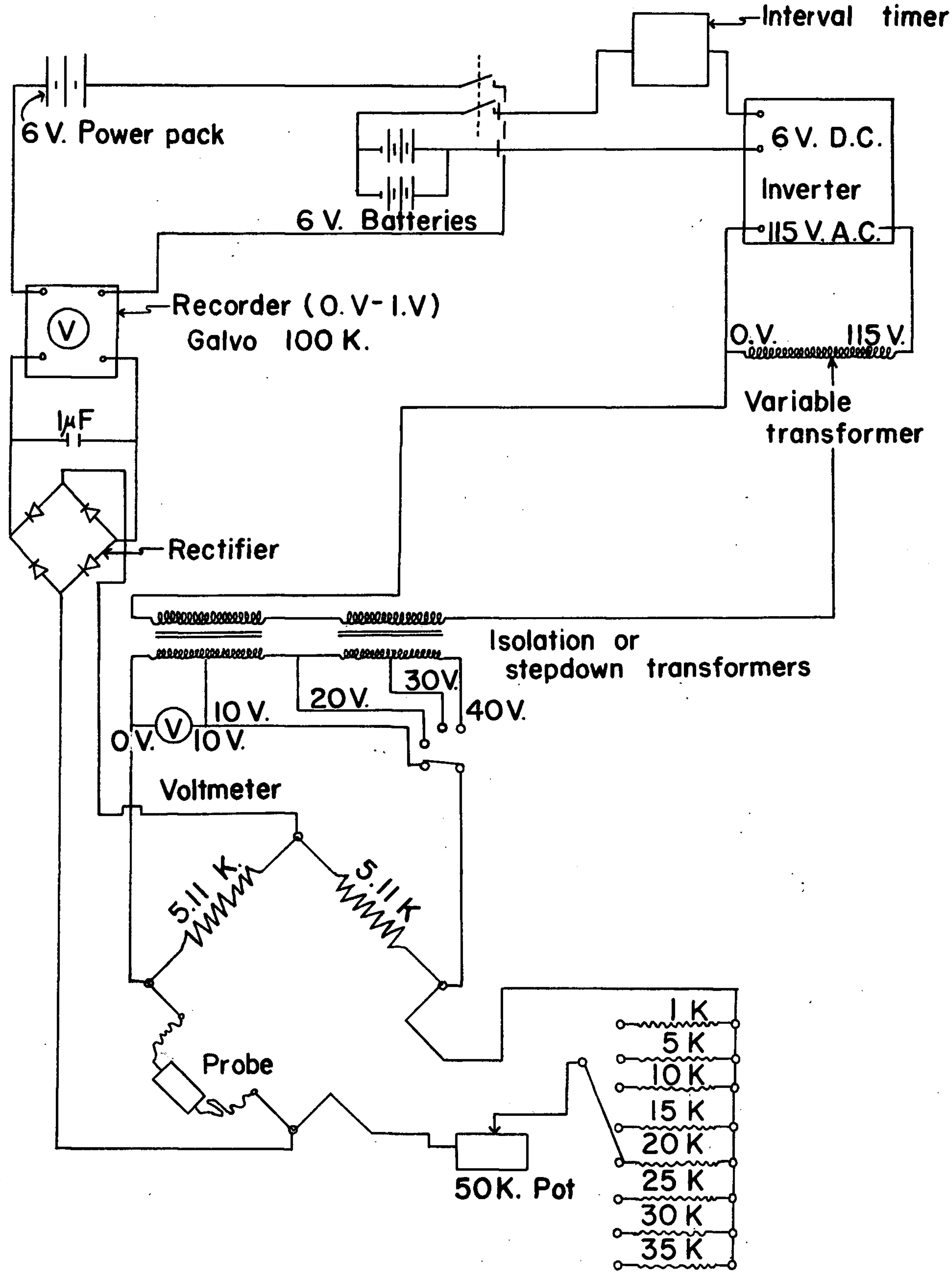
[5]Initially a few discharges were measured by injecting the tracer (Sodium Dichromate or Rhodamine WT) at a constant rate and then determining the dilution ratio between injected solution and stream water. This "constant rate injection method" is described in Church and Kellerhals (1969). The equipment is shown on Photograph 16. The method was not suitable for this study because it cannot give velocity and offers no advantages over slug injection methods for simple discharge measurements.

10 to 100 liters) of a salt solution, whose concentration need not be known, is slug-injected into the stream and the passage of the salt wave is observed downstream with a portable conductivity meter and electrode. A small sample of the initial solution is retained for the construction of a conductivity-concentration rating curve by successive dilution. The main advantages of the method are the possibilities of computing discharge in the field and avoidance of laboratory work. A disadvantage is the relatively bulky field equipment, consisting of 2 vats with needle gauges, pails , approximately 1 kg of NaCl per m^3s^{-1} to be measured, pipets, 2 volumetric flasks, conductivity meter, electrode, and stop watch. Photographs 7, 8, and 9 show the main items.

The dye dilution method used here is particularly suitable under difficult field conditions as during severe rainstorms. Accurately measured amounts of the liquid tracer (Rhodamine WT-dye) are injected from the pipet directly into the stream and the C (t)-curve is defined by taking 10 - 20 small water samples at the downstream location for later analysis on a fluorometer. Photograph 10 shows the equipment.

Not discussed in Church & Kellerhals (1969) is the recording conductivity bridge (Photograph 11) built to avoid the tedium of measuring conductivity-time curves at extremely low flows. A brief description of this bridge may be in order as no similar instrument appears to be available commercially.



CIRCUIT DIAGRAM OF RECORDING CONDUCTIVITY BRIDGE

Fig. 8

The principle of operation is to record the off-balance potential V of an AC-bridge on a 0 - 1 volt Rustrak recorder with high input impedance, R_G = 100 K. The circuit diagram of the bridge is shown on Figure 8.

The response V is

$$V = \frac{ER_G \ (RG - 1)}{(RG+1) \ (R_C+2R_G)+2R} \qquad \ldots \ 3.14$$

in which E is the exciting voltage, which is adjustable between zero and 40 volts, R is the adjustable resistance which can take any value between 0 K and 85 K, R_C is fixed at 5.11 K and G is the conductivity measured by the probe. Computed and measured responses are plotted on Figure 9. The difference between the two is caused by the significant but neglected threshold voltage and voltage loss of the inverter.

With careful selection of exciting voltage, amount of salt to be injected, and initial background adjustments, the bridge can be operated in the linear range between responses of 0.2 and 0.9 volts.

To conserve the exciting voltage E during long periods of continuous operation, an electronic interval timer was built[6] and inserted between the bridge and its power supply. The recorder runs off an independent power source.

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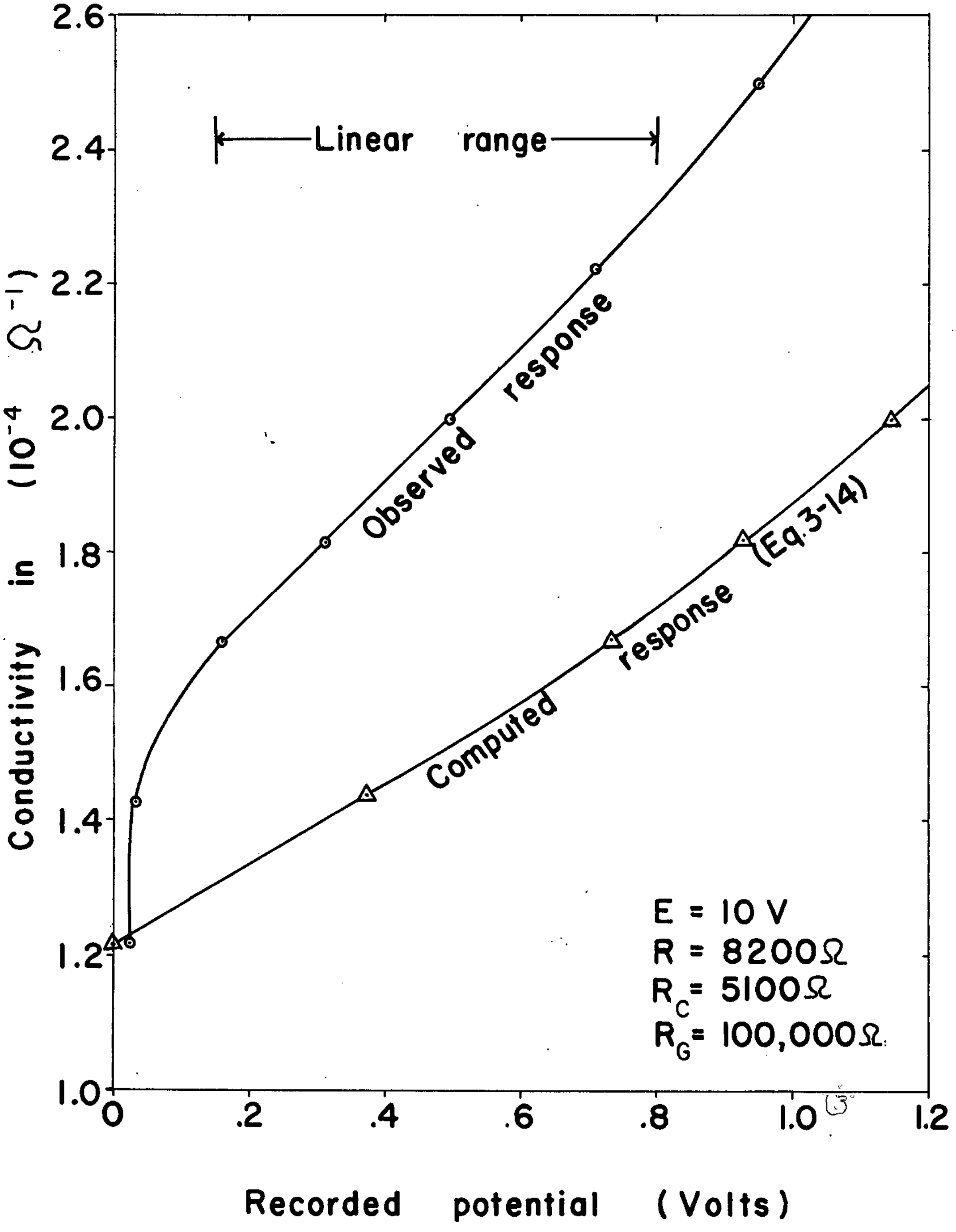
[6]The design of the timer was developed recently by S. Outcalt of the Dept. of Geography, UBC, and W. Schmitt, Dept. of Civil Engineering, UBC.



RESPONSE OF RECORDING CONDUCTIVITY BRIDGE

Fig. 9

3.3.7 Tracer Losses

The tracer methods for discharge measurements assume conservation of tracer mass. From Equation 3.2 one can see that tracer losses due to absorption or chemical reactions result in overestimated discharge, while tracer loss due to seepage of water out of the channel results in underestimates. The travel time, Equation 3.3, however, is independent of the tracer mass as long as the losses do not affect the shape of the C (t)-curve. To permit correction for losses, most tests with long mean residence times T_t were only interpreted for T_t according to Equation 3.3, discharge being measured with a separate test over the shortest permissible reach.

Almost all Rhodamine WT test results show a certain amount of tracer loss due to absorption or chemical disintegration of the tracer. The loss rate L, in percent per minute can be estimated from two simultaneous tests, one over a long reach and the other over a short reach, both ending at the same sampling position. Assuming that the measured discharge Q_m and the true discharge Q are related as

$$Q_m = \frac{Q}{1-\frac{L}{100}\,T_t} \qquad \ldots\ 3.15$$

leads to

$$L = \frac{100(1 - Q_s/Q_l)}{T_{t,l} - \frac{Q_s}{Q_l}\,T_{t,s}} \qquad \ldots\ 3.16$$

in which the subscripts l and s refer to the long and the short reach respectively, and Q and T_t are computed according to Equations 3.2 and 3.3 respectively. L is commonly in the order of 0.1 to 0.3 percent per minute. No reason could be found for the observed variation in L. Most discharges based on Rhodamine WT tests were corrected according to Equation 3.15, with L-values estimated from double tests.

No consistent evidence of tracer loss appears in the salt dilution data, but long travel times definitely tend to result in unreliable discharges. The cause is probably a combination of tracer losses and changes in the background conductivity of the stream. The conductivity changes observed during the passage of a salt wave can only be converted to salt concentration if the background conductivity remains constant or changes in a predictable manner, neither of which was true in very extended tests.

3.4 Surge Tests

3.4.1 Objective

If a channel routing method is capable of reproducing the discharge Q (t) at the downstream end of a test reach, resulting from small, step-like increases or decreases in Q (t) at the upstream end, and if this holds over the complete range of Q, then one may assume that the method should also be adequate to route complex storm hydrographs through the channel reach, since they can be decomposed into a sequence of small steps. This general statement is absolutely correct if the channel response is linear, but for all practical purposes it will also hold as long as the non-linearities are not strong enough to lead to severe discontinuities such as bores.

The main objective of the surge tests was therefore to impose small, steplike discharge modifications, ΔQ, at the upstream end of test reaches and to observe the propagation of these positive and negative surges. The tests were to cover as large a range of discharge as possible.

A few tests on the effect of the relative size of ΔQ were also run by imposing small and large ΔQ's at constant Q.

3.4.2 Discharge Modifications

Different methods for modifying Q were used at each of the four lake outlets on the test streams of this study.

A small dam was built at the outlet of Blaney Lake. Discharge could be increased or decreased by adding or removing flashboards (Photograph 12). An old timber-crib dam at the outlet of Marion Lake gave excellent control over the Phyllis Creek reaches. Photograph 13 shows the dam, with two flashboard-like additions in place. The outlet of Placid Lake was so marshy that no control structure could be built. Surges were produced by pumping water across the swamp into the creek (Photograph 14). The pool above the Brockton reaches was so small that it was difficult to maintain steady discharges different from the pool inflow. The initial surge was produced by adding or removing a few rocks at the pool outlet. A gravity-operated inverted siphon was then used to maintain more or less steady flow for 5 to 15 minutes (Photograph 15).

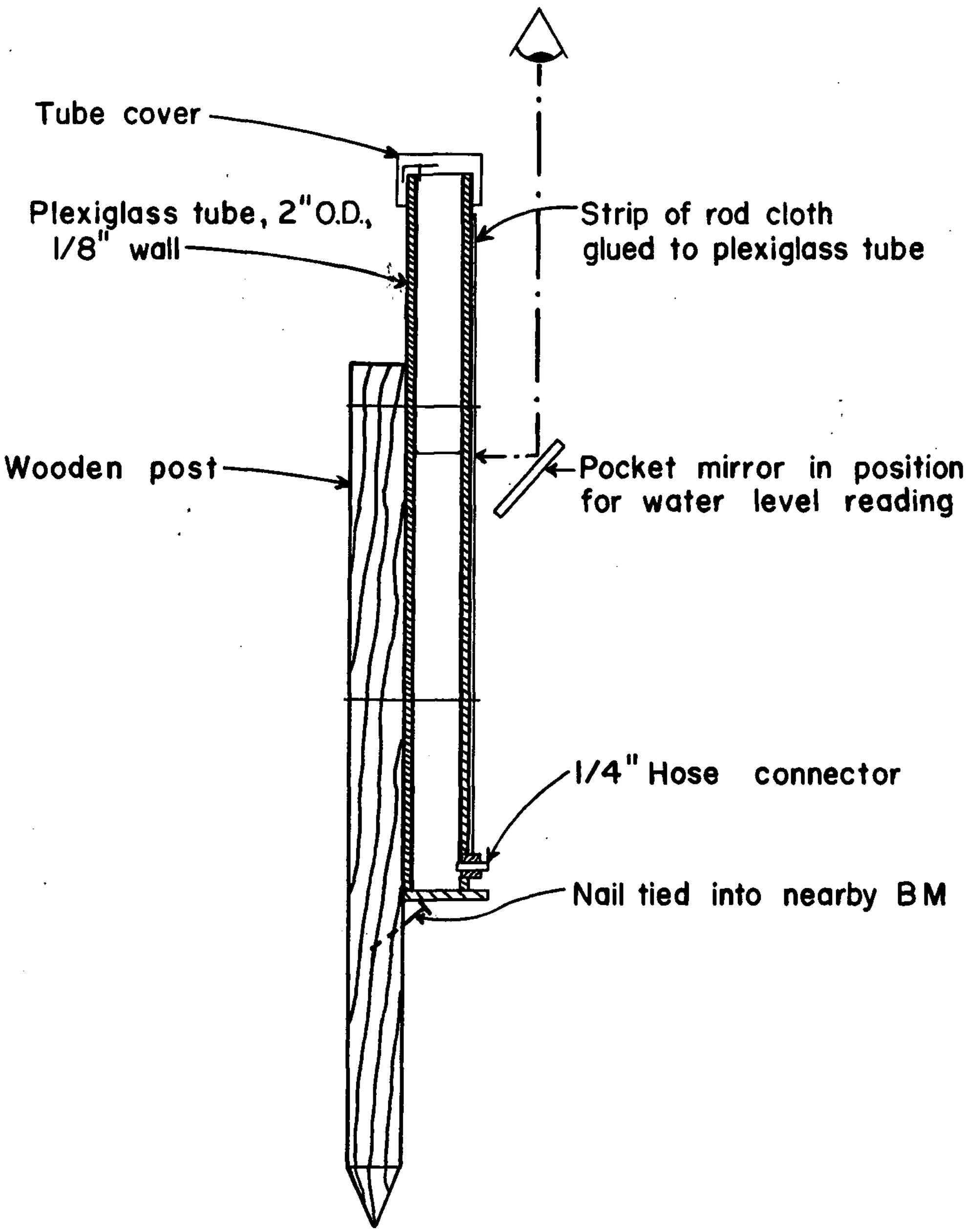
3.4.3 Stage Measuring Equipment

Discharge changes at the end points of the test reaches were monitored by observing or recording stage changes and establishing stage discharge rating curves for conversion to discharge. Even on the steepest and most turbulent reaches it was generally possible to find stable pools, either on bedrock or between large boulders (Photograph 18). The turbulent level fluctuations and air entrainment made direct level measurements impossible, but the plexiglass stilling wells illustrated on Figure 10 and Photographs 16 and 18 permitted the reading of water levels to ± 0.001 ft.[7] This gave satisfactory resolution,since most surge tests caused level changes in the order of 0.02 to 0.05 ft.

To gain some information about the discharge range of the test reaches, automatic stage recorders were installed on all but one of the test creeks. The instruments were Stevens A-35 recorders, with the fastest available clock gearing (9.6 in/24 hours), and a 12:10 level scale. These large scales made it possible to rely on the recorders for the surge tests, thereby saving one field assistant. With careful procedures the time scale could be interpreted to ± 1/2 min.

The recorder installations were somewhat unconventional, due to the inverted siphon connecting the stream to the stilling

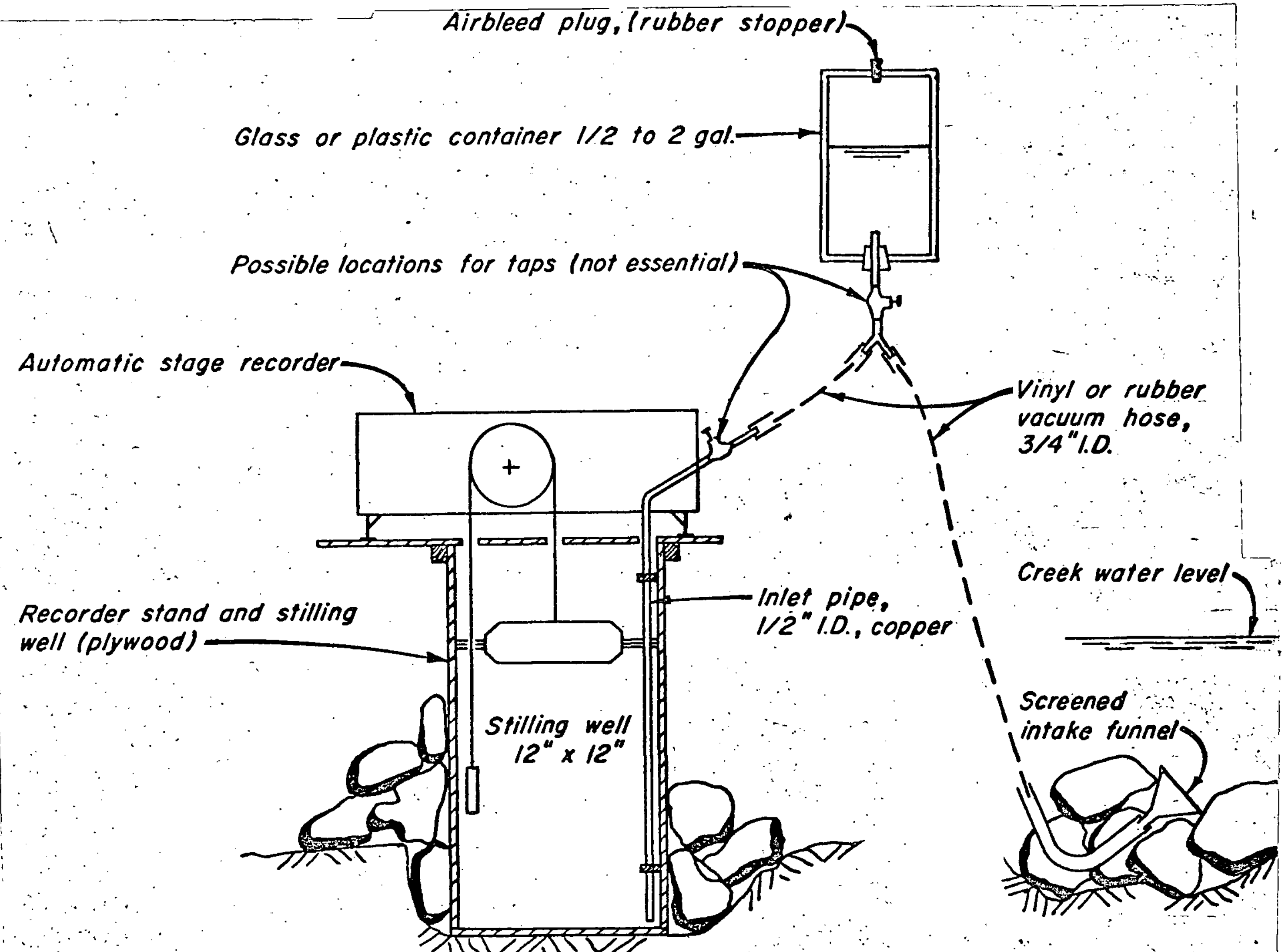
[7] All stage measurements are in feet and decimals thereof due to a lack of readily available metric equipment.



SCHEMATIC SECTION OF MANUAL GAUGE

(from Church and Kellerhals, 1969)

Fig. 10



SCHEMATIC VIEW OF STAGE RECORDER INSTALLATION
FOR MOUNTAIN STREAMS (from Church and Kellerhals, 1969)

Fig. 11

well. Figure 11 and Photographs 17 and 18 illustrate the method. The experience gained with all the above stage measuring equipment (tubes and recorders) is discussed at length in Church & Kellerhals (1969).

3.4.4 Stilling Well Response

The hose connections between plexiglass tube wells or recorder wells and the streams cause considerable damping (in the sense that the gauge level cannot follow high frequency fluctuations of the stream level). This damping is essential for accurate level readings but the question arises as to how much it affects the surge test data.

Under normal circumstances the flow in the connecting hoses is laminar[8] and the inertia of the flowing water is negligible. The well response is then governed by the following equation:

$$h = - b \frac{dh}{dt} \qquad \dots \ 3.17$$

in which h is the elevation difference between the stream and the stilling well and b is a time constant. If the stream level is constant and the well level is off by h_o at time t_o, the solution is

$$h = h_o e^{\frac{t_o - t}{b}}, \qquad \dots \ 3.18$$

---

[8] Assuming a steady rise of 0.02 ft/min. in the well, which is approximately the maximum observed during surge tests, gives Reynolds Numbers of 24 for the connecting hose of plexiglass tubes and 500 for recorder well connections.

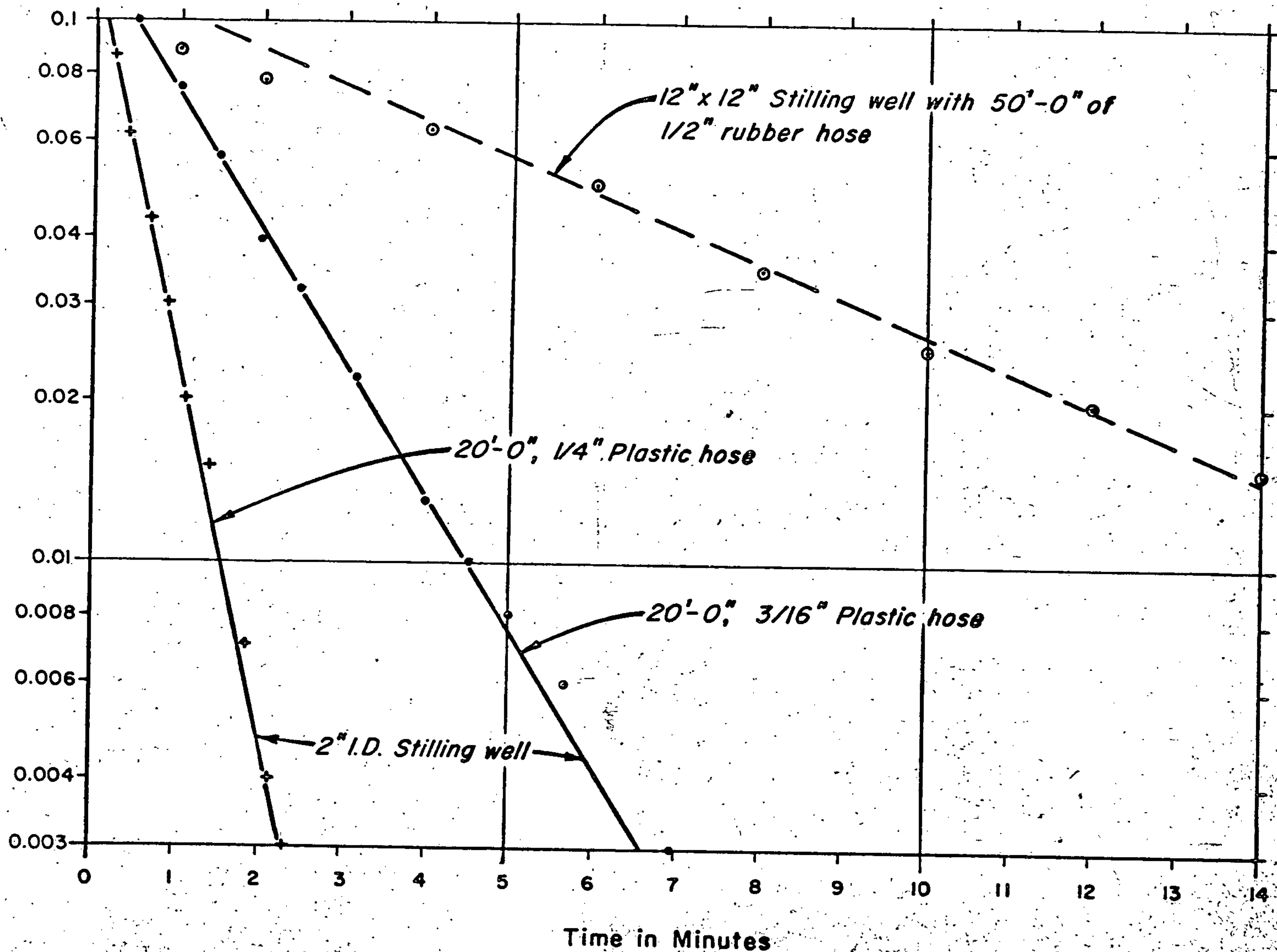


Fig. 12

GAUGE RESPONSE CURVES

which indicates that a plot of $(t - t_o)$ vs. (h/h_o) should give a straight line of slope (-b) on semi-log paper. Some deviations should be expected since Equation 3.17 only considers pipe friction and there are certain other losses present. Figure 12 shows some typical gauge response curves.

Instead of plotting a response curve, one can measure the time, Δt, it takes the stilling well to drop from an arbitrary h_1 to an arbitrary h_2 and compute b as follows:

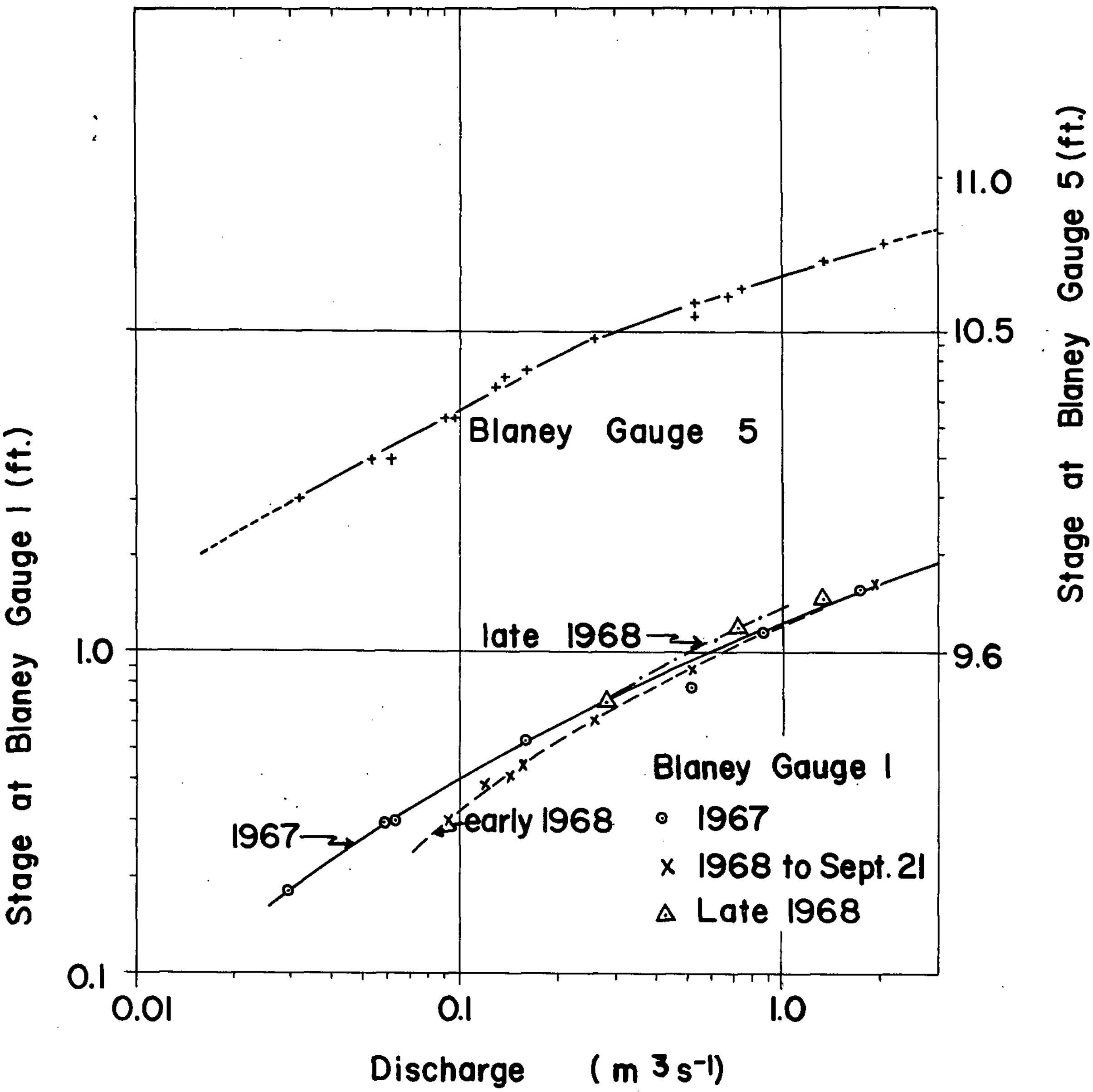
$$b = \frac{\Delta t}{\log(h_1/h_2)} \qquad \ldots\ 3.19$$

All gauge and recorder setups were tested in this manner and, where necessary, the surge test records were corrected for lag according to Equation 3.17. About one third of the gauges needed lag corrections.

3.4.5 Stage-Discharge Rating Curves

At the time when most of the test reaches of this study were installed, the destructive force of the streams under severe flood conditions was not properly appreciated. Many of the gauges were located at pools that proved subsequently to be unstable. In hindsight, it appears that there was no lack of stable pools; only a lack of experience in locating them.[9] As the gauges could not be moved without altering the

[9] Pools formed by large, preferably angular rocks, arranged in such a way that they do not easily catch driftwood, are best. Locations below well established log jams are excellent, as the jams tend to catch most debris and coarse bed load.



TWO TYPICAL STAGE - DISCHARGE RATING CURVES

Fig. 13

test reach length, some parts of the stage-discharge curves had to be re-defined two or three times, to permit conversion of the surge data from stage to discharge. The stage recorders were installed at the most stable gauging sites.

Figure 13 shows two rating curves, one for the stable pool of Blaney Gauge 5 (Photograph 18) and the other for the more troublesome Blaney Gauge 1.

4. FIELD RESULTS

4.1 Survey Results

Tables 2 and 3 and Figures 3 and 4 summarize the survey results which consist of profiles of the test reaches and width measurements (Section 3.2). The width data were processed with a set of programs developed by Day (1969). Tables 2 and 3 are summaries of the program output. There is undoubtedly a considerable operator effect in the width data, since the high water mark was often, particularly on the bushier streams, rather ill defined. The large number of width measurements tends to compensate for this, but discrepancies of 10% to 15% could still occur between different field parties.

On most test reaches there is no significant difference between actual length and length in plan (map length), but in the case of the few very steep reaches it is worth noting that the surveyed length is the actual length on the slope. The relative accuracy of the chaining and hand-levelling is estimated at $\pm$ 3 percent.

Slope is defined as drop divided by length, $\sin \theta$, if θ is the slope angle.

The drainage areas were measured on the best available maps and refer to the middle point of a test reach. The Brockton Creek basin does not appear on any map, as the

1:50,000 coverage of this area happens to be exceptionally poor. The drainage area was therefore measured off air photos.

4.2 Velocity and Discharge Measurements

4.2.1 Conversion of Field Data to Time-Concentration Curves

The field data resulting from a relative salt dilution test consist of the following:

(i) location and time of injection,

(ii) volume of brine injected,

(iii) sampling location, list of times and corresponding conductivity readings,

(iv) rating curve, covering range of observed conductivity readings (it consists of dilution rates and corresponding conductivity readings),

(v) water temperature in stream and in rating tank.

The computational procedure for converting the time-conductivity data to time-concentration is described in detail in Church and Kellerhals (1969). The procedure proposed in earlier publications on this method (Østrem, 1964) should not be used, as the correction for different background readings in the stream and in the rating tank is in error. A Fortran IVG program "NACL" was developed for this conversion. It prints the time-conductivity and time-concentration data and plots the rating curve. The program is listed in the Appendix, together with operating instructions and sample output.

The field data resulting from a Rhodamine WT test are:

(i) location and time of injection,

(ii) volume of injected dye,

(iii) sampling location, list of sampling times, and 10 to 30 samples.

The samples are subsequently analysed on a fluorometer,[1] and the instrument reading is converted to concentration with a rating curve based on standard dilutions of the tracer. Wilson (1968) describes the laboratory procedures in great detail. The necessary computations were done manually, but the final time-concentration data were put on cards for processing by an input program "DQV" analogous to "NACL", which is also listed in the Appendix.

4.2.2 Numerical Integration

The Equations 3.2, for discharge Q, and 3.3, for tracer travel time T, are evaluated by a Fortran IVG subroutine "QVEL" (see Appendix). The integral over C(t) and the first moment of C(t) are computed twice, first on the basis of the trapezoidal rule and then with a second order method similar to Simpson's rule but capable of handling unevenly spaced points. The procedure is discussed briefly below, because the formulas do not appear to be readily available in texts on numerical analysis.

---

[1]The "Turner Model 110" fluorometer of the B.C. Research Council was used here.

Assume that $C(t)$ is defined at $t = t_o$, t_1, and t_2 as shown on Figure 14.

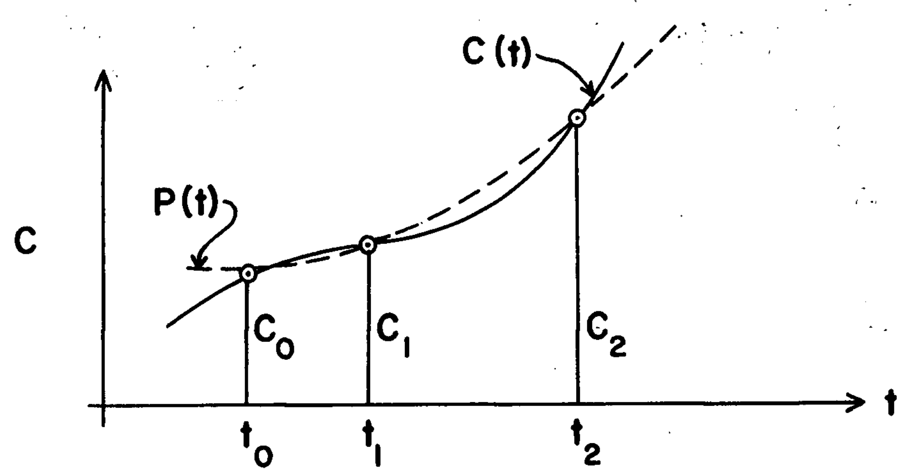


FIGURE 14. DEFINITION SKETCH FOR NUMERICAL INTEGRATION

The function $C(t)$ can be approximated by a second order polynomial $P(t)$ of the form

$$P(t) = C_o\, l_o(t) + C_1\, l_1(t) + C_2\, l_2(t)$$

in which the $l_i(t)$ are second order polynomials in terms of t_o, t_1, and t_2 (Herrio, 1963), e.g.

$$l_o(t) = \frac{t^2 - tt_1 - tt_2 + t_1 t_2}{(t_o - t_1)(t_o - t_2)}$$

The integral $C(t)dt$ can be estimated as

$$\int_{t_o}^{t_2} P(t)dt = C_o \int_{t_o}^{t_2} l_o(t)dt + C_1 \int_{t_o}^{t_2} l_1(t)dt + C_2 \int_{t_o}^{t_2} l_2(t)dt$$

Without loss of generality one can substitute

$$t_o = 0$$
$$t_1 = \Delta t_1$$
$$t_2 = \Delta t_2$$

The integrals over l (t) are then:

$$\int_0^{\Delta t_2} l_o(t)dt = \frac{1}{\Delta t_1 \Delta t_2} \left(\frac{\Delta t_2^{\,2} \Delta t_1}{2} - \frac{\Delta t_2^{\,3}}{6}\right)$$

$$\int_0^{\Delta t_2} l_1 (t)dt = - \frac{\Delta t_2^{\,2}}{6(\Delta t_1^{\,2} - \Delta t_1 \Delta t_2)}$$

$$\int_0^{\Delta t_2} l_2 (t)dt = \frac{1}{\Delta t_2^{\,2} - \Delta t_1 \Delta t_2} \left(\frac{\Delta t_2^{\,3}}{3} - \frac{\Delta t_2 \Delta t_1}{2}\right)$$

which reduces to Simpson's Rule if $\Delta t_1 = \frac{\Delta t_2}{2}$.

An approximation to the first moment with respect to $t = o$ is

$$\int_0^{\Delta t_2} P(t)tdt = C_o \int_0^{\Delta t_2} l_o(t)tdt + C_1 \int_0^{\Delta t_2} l_1(t)tdt + C_2 \int_0^{\Delta t_2} l_2(t)tdt$$

$$= (\gamma_1 + \gamma_2 + \gamma_3) \frac{\Delta t_2^{\,4}}{4} - [\gamma_1(\Delta t_1 + \Delta t_2)$$

$$+ \gamma_2 \Delta t_2 + \gamma_3 \Delta t_1] \frac{\Delta t_2^{\,3}}{3} + \gamma_1 \Delta t_1 \frac{\Delta t_2^{\,3}}{2}$$

in which

$$\gamma_1 = \frac{C_o}{\Delta t_1 \Delta t_2}$$

$$\gamma_2 = \frac{C_1}{\Delta t_1^{\,2} - \Delta t_1 \Delta t_2}$$

$$\gamma_3 = \frac{C_2}{\Delta t_2 - \Delta t_1 \Delta t_2}$$

In those cases where the concentration decline has not been defined adequately in the field, a subroutine "TAILEX" (see Appendix) is first called from "NACL" or DQV". It plots (log $C\gamma_2$ log t) vs. (t) to permit the fitting of a Γ-distribution extension, as discussed in Section 3.3.5 and shown on Figure 7. The parameters of the extrapolation are read off the "TAILEX" plot and punched onto the control card of the C(t)-datacheck. If the subroutine "QVEL" is then called, it will use the Γ-distribution to extend the integral over C(t) to infinite time.

Finally, the C(t)-curves, with or without Γ-extension, can be plotted by calling subroutine "PLØTGA" from the input programs "DQV" or "NACL". "PLØTGA" is also listed in the Appendix, together with operating instructions and sample plots.

4.2.3 Results

The tracer data consist of 111 slug injections, for which 146 time-concentration curves were determined. In other words, approximately two thirds of the slug injection tests contain one downstream sample only; the other third consists mainly of runs with two samples. Only three runs extend over three sampling locations and none over four. In addition, there are eight discharge measurements with the constant rate injection method.

Presentation of the complete data, which consist of approximately 7000 time and concentration values did not appear justified. Tables 4A to 4D show the results in summarized form. There is one table per stream, with the test runs arranged in historical sequence. The identification code is explained in a footnote. With the exception of the simple discharge measurements, the same data appear again in the Appendix, as printout of the program "LØGRE" (see Section 5.2.2). The arrangement there is by test reach. For cross reference from Table 4 to the Appendix it is best to use the test number. Note that the discharges are slightly different because Table 4 refers to the tracer data and therefore gives the discharge at the sampling point, whereas the "LØGRE" printout refers to test reaches, and the discharge is the estimated mean between inflow and outflow.

4.2.4 Accuracy

The relative accuracy, or internal consistency of the data appears to be satisfactory. As can be seen from Table 4, tests on the same day give similar flows at all stations of a stream (after correction for tracer loss in long dye-dilution runs) and tests made at different times but with apparently similar flows, show a consistent time distribution of the

TABLE IVA

SUMMARY OF TRACER MEASUREMENTS: BROCKTON CREEK

Test Identification[1]	Date	No. of Points	T_s (min)	Peak (min)	Mean Lag (min)	Discharge $(m^3 s^{-1})$	+ for Γ extension	Method[2]
BrR1,2UP-2X-3	Aug.15,67	17	0.4	1.5	2.61	0.00564		RhWT
BrR1,2UP-2-3X	Aug.15,67	28	14.0	33.0	37.6	0.00770	+	RhWT
BrR2,1UP-IX-2	Aug.15,67	16	0.5	1.5	2.28	0.00856		RhWT
BrR2,1UP-1-2X	Aug.15,67	16	17.5	27.0	32.4	0.0100	+	RhWT
BrR3,2UP-2X	Aug.17,67					0.00336		RhWT C.I.
BrR4,1UP-1X-2-3	Aug.17,67	19	1.0	2.2	3.43	0.00504		RhWT
BrR4,1UP-1-2X-3	Aug.17,67	18	25.0	39.5	47.9	0.00617	+	RhWT
BrR4,1UP-1-2-3X	Aug.17,67	13	52.0	74.0	82.1	0.00738	+	RhWT
BrR5,2UP-2X	Aug.17,67					0.0108		RhWT C.I.
BrR6,1UP-1X	Aug.17,67	15	1.0	2.2	2.99	0.00706		RhWT
BrR7,1UP-1X	Sept.15,67	15	4.2	9.0	14.50	0.00067		RhWT
BrR8,2-2DOX	Sept.15,67	13	1.0	5.0	8.71	0.000703		RhWT
BrR9,1UP-1DOX	Sept.24,67	32	4.0	19.0	39.9	0.000154		RhWT

1 The test identification code is as follows: the first two letters identify the stream (Br=Brockton, Pl=Placid, Bl=Blaney, Ph=Phyllis), then comes the test run number (R1,R2,...) in a more or less historical sequence, next is the location (gauge number) of injections (UP or DO meaning shortly upstream of ... or shortly downstream from ...), finally the sample locations (gauge numbers), with an X indicating the particular time concentration curve one is dealing with.
Examples: PhR5,2UP-2X is a simple discharge determination at Gauge #2 on Phyllis Creek, with injection shortly above Gauge #2 and sampling at the gauge.
BlR10 1-3-5X would indicate that the test run #10 on Blaney Creek covers two reaches (1-3, and 3-5).

2 C.I. means "constant rate injection test."

TABLE IVA (Cont'd.)

SUMMARY OF TRACER MEASUREMENTS: BROCKTON CREEK

Test Identification[1]	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge (m^3s^{-1})	+ for Γ extension	Method[2]
BrR10,1-2X	Aug.18,68	18	70.	117.	160.	0.00124	+	RhWT
BrR11,2-3X	Aug.18,68	16	53.	97.	120.	0.00126	+	RhWT
BrR12,1UP -IDOX	Aug.18,68	9	2.5	5.7	8.50	0.00122		RhWT
BrR13,2UP-2DOX	Aug.18,68	9				0.00114		RhWT
BrR14,1-1DOX	Aug.26,68	9				0.00453		RhWT
BrR15,2UP-2X	Aug.26,68	9				0.00555		RhWT
BrR16,1-2X	Sept.14,68	13	4.9	8.1	9.08	0.0648		RhWT
BrR17,2-3X	Sept.14,68	12	3.0	7.2	8.50	0.0556	+	RhWT
BrR18,1-2X-3	Sept.14,68	19	7.2	11.6	12.6	0.0550		RhWT
BrR18,1-2-3X	Sept.14,68	13	14.5	20.0	21.2	?	+	RhWT
BrR19,1-2X	Sept.14,68	10	2.5	5.0	5.40	0.175		RhWT
BrR20,2-3X	Sept.22,68	13	2.2	4.7	5.39	0.0889		RhWT
BrR21,1-2X	Sept.22,68	11	4.2	7.3	8.33	0.102	+	RhWT
BrR22,2-3X	Sept.22,68	12	1.8	4.0	4.53	0.109		RhWT
BrR23,1-2X	Sept.22,68	11	3.2	5.5	6.97	0.152	+	RhWT

TABLE IVB

SUMMARY OF TRACER MEASUREMENTS: PLACID CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge ($m^3 s^{-1}$)	+ for $\sqrt{\ }$ extension	Method
P1R1,2UP-2X	June 3,68	18	1.8	3.5	4.57	0.0687		RhWT
P1R2,1-2X	June 3,68	14	122.	164.	210.	0.064	+	RhWT
P1R4,3UP-3X	June 9,68	17	3.2	6.4	7.8	0.0183	+	RhWT
P1R5,2-3X	June 9,68	17	160.	236.	351.	0.018	+	RhWT
P1R6,4-4DOX	June 9,68	46	4.5	9.3	14.8	0.0408		NaCl
P1R7,2UP-2X	June 9,68	16	5.8	11.0	13.8	0.0117	+	RhWT
P1R9,2UP-2X	June 18,68	18	1.9	4.8	8.29	0.00358		RhWT
P1R10,2-3X	June 27,68	17	62.0	94.	121.	0.085	+	RhWT
P1R11,3-3DOX	June 27,68	18	11.0	20.	26.9	0.144[3]		RhWT
P1R11,3-3DO-4X	June 27,68	6	211.	261.	?			RhWT
P1R12,4UP-4X	June 27,68	27	119.	170.	206.	0.185		RhWT
P1R13,1-2X	June 28,68	17	165.	208.	281.	0.045	+	RhWT
P1R14,2UP-2X	June 28,67	18	24.0	28.3	29.7	0.0418		RhWT
P1R16,4-4DOZ	Aug.20,68	40	8.8	11.6	.2	0.035		NaCl
P1R17,3-4X	Aug.20-1,68		674.	100.0	1200.	0.020		NaCl

[3]Below confluence of Gauge 3.

TABLE IVB (Cont'd.)

SUMMARY OF TRACER MEASUREMENTS: PLACID CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge (m^3s^{-1})	+ for Γ extension	Method
P1R17A,2UP-2X	Aug.28,68	9	8.	11.5	12.5	0.0824		RhWT
P1R18,2-3X-4	Aug.28,68	60	48.	71.	83.3	0.157		NaCl
P1R18,2-3-4X	Aug.28,68	68	236.	294.	339.	0.245	+	NaCl
P1R19,3-4X	Aug.28,68	36	162.	199.	239.	0.268	+	NaCl
P1R20,4-4DO	Aug.28,68	13	1.15	3.8	4.82	0.249		NaCl
P1R21,3-4X	Aug.30,68	39	302.	386.	482.	0.097		RhWT
P1R22,4UP-DOX	Sept.21,68	12	8.5	14.5	19.2	0.122	+	RhWT
P1R23,2UP-2X	Sept.21,68	12	4.0	8.0	9.4	0.0363		RhWT
P1R24,2-3X-4	Oct.12,68	73	47.5	74.	92.1	0.142	+	NaCl
P1R24,2-3-4X	Oct.12,68	52	205.	274.	346.	0.368	+	NaCl
P1R25,1-2X	Oct.12,68	92	115.	159.	181.	0.0822	+	NaCl
P1R26,2UP-2X	Oct.12,68	12	4.2	7.3	8.68	0.0822		RhWT
P1R27,2-3X-4	Oct.22,68	80	36.1	54.8	62.1	0.212		NaCl
P1R27,2-3-4X	Oct.22,68	38	157.	224.	271.	0.560	+	NaCl
P1R28,1-2X	Oct.22,68	75	94.	130.	144.	0.122	+	NaCl
P1R28A,2UP-2X	Oct.22,68	11	3.2	5.7	7.02	0.122		RhWT
P1R29,4UP-4DOX	Oct.22,68	12	3.0	6.0	7.45	0.474		RhWT

TABLE IVC

SUMMARY OF TRACER MEASUREMENTS: BLANEY CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge ($m^3 s^{-1}$)	+ for Γ extension	Method
B1R1,1-2X	May15,67					0.190		SoD C.I.
B1R1,1-3X	May 15,67					0.260		SoD C.I.
B1R2,1-2X	May 18,67					0.159		SoD C.I.
B1R3,4UP-4DOX	May 19,67					0.123		SoD C.I.
B1R4,4UP-4DOX	May 19,67					0.168		SoD C.I.
B1R5,1-2X	June 9,67					0.064		RhWT C.I.
B1R6,4UP-4DOX	June 9,67	10	4.2	9.0	10.5	0.054		RhWT
B1R7,1-2X	June 9,67	11	17.5	27.0	36.	0.060	+	RhWT
B1R7A,5UP-5X	Sept.30,67	18	2.8	5.5	12.2	0.0317		RhWT
B1R8,3-5X	Oct.6,67	14	8.5	16.2	21.2	0.873	+	RhWT
B1R9,4UP-4DOX	Nov.19,67	9	0.9	1.82	2.19	0.595		RhWT
B1R10,3-5X-4	Nov.19,67	22	12.0	20.5	26.3	0.538		RhWT
B1R10,3-5-4X	Nov.19,67	14	58.	77.5	89.4	0.588	+	RhWT
B1R11,1-3X-5	Nov.19,67	15	30.	44.5	54.7	0.517	+	RhWT
B1R11,1-3-5X	Nov.19,67	11	51.	67.5	77.2	0.520	+	RhWT

TABLE IVC (Cont'd.)

SUMMARY OF TRACER MEASUREMENTS: BLANEY CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge (m^3s^{-1})	+ for Γ extension	Method
B1R12,1-3X-5-4	Dec.26,67	15	17.	23.5	27.5	1.76	+	RhWT
B1R12,1-3-5X-4	Dec.26,67	19	28.	37.5	43.2	1.86	+	RhWT
B1R12,1-3-5-4X	Dec.26,67	9	53.	67.7	74.0	2.06	+	RhWT
B1R13,3-5X-4	Jan.20,68	18	3.3	5.6	7.1	10.4	+	RhWT
B1R13,3-5-4X	Jan.20,68	12	14.5	19.5	21.6	12.0		RhWT
B1R14,3-5X	Jan.20,68	14	3.2	5.5	6.24	11.7		RhWT
B1R15,1-3X-5	Jan.20,68	13	7.5	10.6	12.10	11.8		RhWT
B1R15,1-3-5X	Jan.20,68	18	11.5	16.5	18.6	11.8	+	RhWT
B1R16,1-3X-5	Feb.3,68	16	19.5	26.4	30.2	1.64		RhWT
B1R16,1-3-5X	Feb.3,68	23	31.5	40.2	47.8	1.66	+	RhWT
B1R17,1-3X	Feb.3,68	39	18.3	25.5	29.6	1.64		NaCl
B1R18,3-5X	March 5,68	25	6.2		22.4	0.862		RhWT
B1R19,1-3X-5-4	March 5,68	16	15.0	22.5	25.4	1.95		RhWT
B1R19,1-3-5X-4	March 5,68	16	26.0	33.7	38.0	2.00		RhWT
B1R19,1-3-5-4X	March 5,68	13	50.0	61.5	68.3	2.33	+	RhWT
B1R20,3-5X	March 20,68	62	10.0	18.2	24.8	5.34	+	NaCl
B1R21,5-4X	March 31,68	77	38.5	51.4	56.6	0.804		NaCl
B1R22,3-5X	May 25,68	17	20.0	36.0	45.5	0.162	+	RhWT

TABLE IVC (Cont'd.)

SUMMARY OF TRACER MEASUREMENTS: BLANEY CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge ($m^3 s^{-1}$)	+ for Γ extension	Method
B1R22A,4UP-4DOX	May 27,68	18	1.2	2.0	3.70	0.131		RhWT
B1R23,5-4X	May 27,68	9	98.0	137.0	165.	0.130		RhWT
B1R24,1-3X	May 27,68	7	71.0	101.0	123.	0.120		RhWT
B1R25,3-5X	June 3,68	18	9.5	16.6	20.8	0.682		RhWT
B1R26,3-5X	June 6,68	17	16.0	26.0	34.2	0.262		RhWT
B1R27,4-4DOX	June 6,68	18				0.264		RhWT
B1R28,5-4X	June 6,68	67	56.0	81.0	93.9	0.280		NaCl
B1R29,3-5X-4	June 13, 68	45	21.2	38.5	50.5	0.140		NaCl
B1R29,3-5-4X	June 13,68	84	115.	172.	202.1	0.140		NaCl
B1R30,4UP-4X	June 13,68	18	4.5	9.6	11.41	0.139		RhWT
B1R31,1-3X	June 13,68	18	57.0	90.0	117.0	0.146	+	RhWT
B1R32,3-5X	June 18,68	56	27.5	53.0	71.4	0.0903		NaCl
B1R33,3-5X	June 18,68	18	29.0	54.5	71.4	0.083	+	RhWT
B1R34,3-5X	Sept.21,68	12	8.9	15.0	19.5	0.748	+	RhWT
B1R35,1-3X	Oct.1,68	69	40.5	61.0	71.2	0.285		NaCl
B1R36,1-3X	Oct.12,68	16	24.0	34.0	40.4	0.741	+	RhWT
B1R37,3-5X-4	Oct.13,68	31	7.5	12.7	15.5	1.35		NaCl
B1R37,3-5-4X	Oct.13,68	41	36.0	45.4	51.4	1.30	+	NaCl

TABLE IVD

SUMMARY OF TRACER MEASUREMENTS: PHYLLIS CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge ($m^3 s^{-1}$)	+ for Γ extension	Method
PhR1,1-1DOX	June 22,67					1.980		SoDi C. I.
PhR3,2-3X-4	July 21,67	22	25.0	38.0	43.5	0.748	+	RhWT
PhR3,2-3-4X	July 21,67	30	56.	75.	83.1	0.817	+	RhWT
PhR4,1-2X	July 27,67	18	38.5	56.	69.92	0.369	+	RhWT
PhR5,3-4X-6	July 28,67	14	32.0	45.	52.9	0.339	+	RhWT
PhR5,3-4-6X	July 28,67	21	52.0	73.	82.3	0.385	+	RhWT
PhR6,2-3X-4	July 28,67	18	37.5	57.	64.4	0.352	+	RhWT
PhR6,2-3-4X	July 28,67	12	77.0	104.	118.1	0.338	+	RhWT
PhR7,1-2X	July 29,67	15	43.5	61.0	76.0	0.312	+	RhWT
PhR8,4-6X	July 29,67	17	15.0	26.	31.5	0.366	+	RhWT
PhR9,4UP-4DOX	Aug. 8,67					0.232		RhWT C. I.
PhR10,2-3X-4-6	Aug.8,67	19	43.0	70.	87.6	0.228	+	RhWT
PhR10,2-3-4X-6	Aug.8,67	14	96.0	134.	152.3	0.239	+	RhWT
PhR10,2-3-4-6X	Aug.8,67	12	127.	169.	187.	0.240	+	RhWT
PhR11,1-2X-3	May 17,68	17	21.25	28.5	36.2	1.47	+	RhWT
PhR11,1-2-3X	May 17,68	11	40.0	58.0	64.2	1.59	+	RhWT
PhR12,4-6X	May 19,68	14	5.1	8.4	9.54	2.37		RhWT
PhR13,3-4X	May 19,68	16	11.5	16.8	18.8	2.49		RhWT

TABLE IVD (Cont'd.)

SUMMARY OF TRACER MEASUREMENTS: PHYLLIS CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge (m^3s^{-1})	+ for Γ extension	Method
PhR14,2-3X	May 19,68	12	15.	21.	23.4	2.55	+	RhWT
PhR15,1-2X	May 19,68	16	16.5	22.5	27.8	2.40	+	RhWT
PhR16,1-2X	May 17,68	37	19.	28.5	36.7	1.40		NaCl
PhR17,2-3X	May 19,68	32	14.25	21.2	24.2	2.42		NaCl
PhR18,4-6X	May 24,68	13	8.00	12.8	14.4	1.10		RhWT
PhR19,3-4X	May 24,68	15	19.	26.5	29.9	1.07		RhWT
PhR20,2UP-2X-3	May 30,68	16	2.6	5.0	6.31	0.945		RhWT
PhR20,2UP-2-3X	May 30,68	16	25.	38.5	49.8	0.985	+	RhWT
PhR21,1-2X	May 30,68	18	24.	33.	39.8	1.05		RhWT
PhR22,1-2X	May 30,68	70	22.5	33.	38.8	1.02		NaCl
PhR23,2-3X	June 1,68	13	17.	24.	26.8	1.88		RhWT
PhR24,2-3X	June 22,68	15	23.5	35.5	42.1	0.955	+	RhWT
PhR25,1-2X	June 22,68	17	23.	35.4	46.4	0.826	+	RhWT
PhR26,2-3X	July 3,68	18	20.0	31.2	35.5	1.19	+	RhWT
PhR27,3-4X-6	July 3,68	35	17.	24.5	28.5	1.26		NaCl
PhR27,3-4-6X	July 3,68	66	26.	38.5	44.8	1.25		NaCl
PhR28,3-4X-6	Sept.17,68	28	10.3	14.5	17.33	3.10	+	NaCl
PhR28,3-4-6X	Sept.17,68	64	15.0	21.8	24.9	3.10		NaCl

TABLE IVD (Cont'd.)

SUMMARY OF TRACER MEASUREMENTS: PHYLLIS CREEK

Test Identification	Date	No. of Points	T_s (min)	Peak Lag (min)	Mean Lag (min)	Discharge (m^3s^{-1})	+ for Γ extension	Method
PhR29,3-4X-6	Oct.17,68	31	8.5?	13.5	15.4	3.69		NaCl
PhR29,3-4-6X	Oct.17,68	31	14.5	20.4	23.2	3.72	+	NaCl
PhR30,1-2X-3	Oct.17,68	35	14.5	20.	25.1	3.48		NaCl
PhR30,1-2-3X	Oct.17,68	36	27.	37.2	44.5	3.61	+	NaCl

tracer. The relative salt dilution method also agrees with the dye dilution method on the few occasions when simultaneous tests were run. (Ph R11-Ph R12; Ph R21-Ph R22; Bl R16 - Bl R17; Bl R32 - Bl R33).

Absolute accuracy is more difficult to estimate, particularly with regard to discharge, because none of the four test streams is gauged by the Water Survey of Canada. During the tests Bl R32 and Bl R33 over the reach 3 - 5 of Blaney Creek, the Water Survey of Canada measured discharge at Gauge 1. With mean tracer travel times of over one hour, the tracer methods cannot be expected to give very reliable discharges. The salt dilution method, Run 32, indicated a discharge of 90.3 ls^{-1}, the dye dilution method, Run 33, gave 96.8 ls^{-1}, or 83 ls^{-1} with the customary tracer loss adjustment ($L = 0.2\%$ per minute). The current meter measurement was made at a poor section with depth of less than 1 ft throughout; it indicated 75 ls^{-1}.

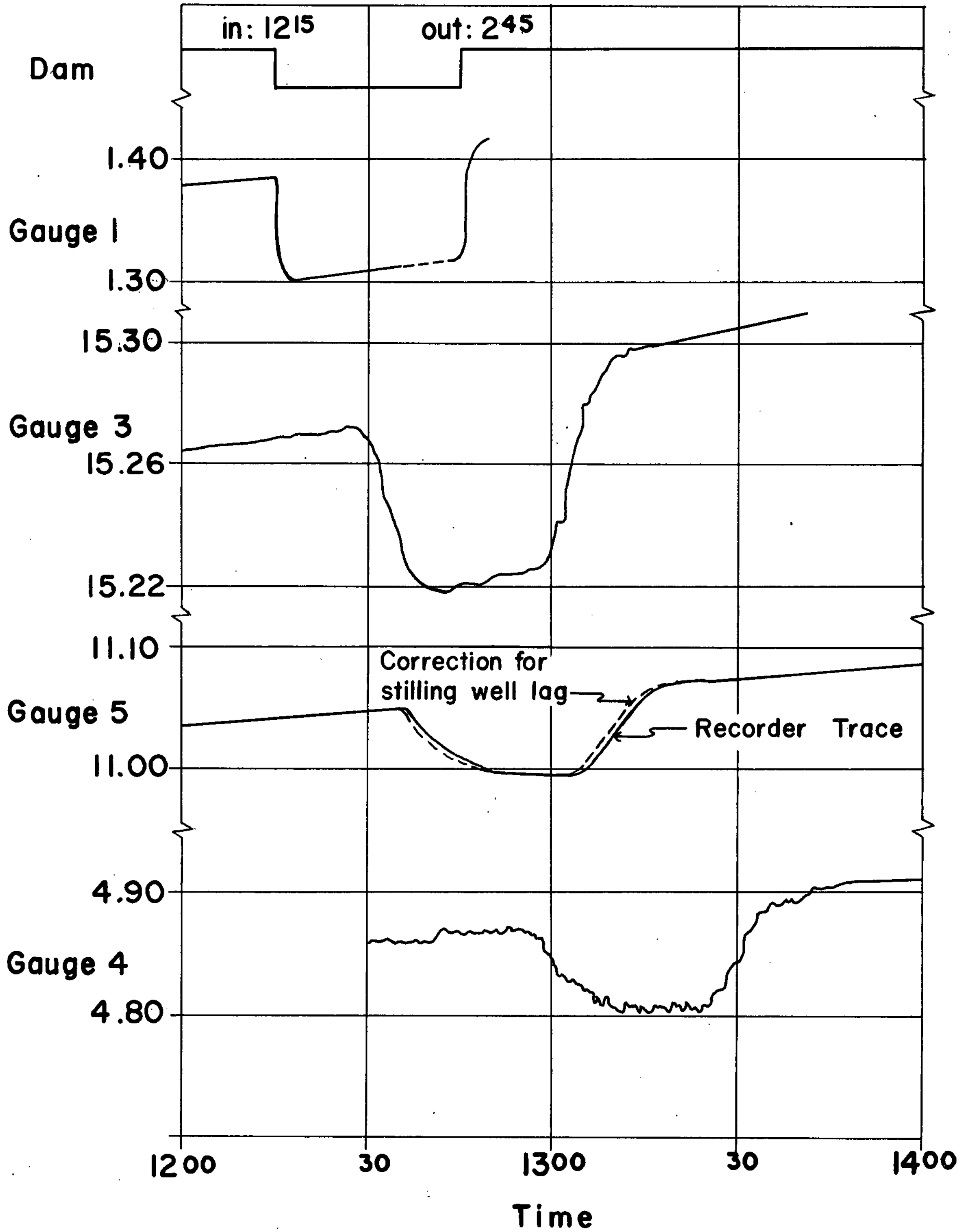
Coverage of the discharge range varies from stream to stream. The low flows are reasonably well defined on all four, but Brockton and Placid Creeks go dry regularly so that one could conceivably observe tracer travel times of several days at extremely low flows. Run 9, on Brockton Creek gives the lowest velocity, 3.5 mms^{-1}.

The high flow range is reasonably well defined on only two of the four streams, Blaney and Brockton. Runs 13, 14, and 15 on Blaney Creek coincide with the largest observed flow at a stream guage in the neighbouring valley, for which there are 3 years of records. Several major log jams were moved at this flow. The control structure (Photograph 12) was completely submerged. Run 19 on Brockton Creek was observed at the runoff peak during a very severe rain storm. On Phyllis and Placid Creeks, the discharge range of the tracer tests extends to approximately 20% of the highest flows that have occurred during the last 3 years.

4.3 Surge Tests

The result of a surge test consists of a graph showing water levels vs. time for all the gauges on one stream. Figure 15 is a typical example. The curves are well defined because of the inherently high accuracy of time and stage measurements. For convenience the data are plotted as t vs. H rather than the more significant t vs. Q. This does, however, not affect the conclusions, because the gauge rating curves are practically linear in the small discharge range encountered during any one surge test.

A total of 22 surge tests were made; 7 on Phyllis Creek, 6 on Blaney and Brockton Creeks, and 3 on Placid. Data from 1 or 2 gauges are missing in approximately 50% of the tests, due to either lack of field assistants or difficulties with the tube gauges. Only the Brockton Creek tests cover the complete range of flows. The control structure on Blaney Lake was submerged and inoperative during the highest flows (Photograph 12).



SURGE TEST OF OCTOBER 13, 1968
ON BLANEY CREEK

Fig. 15

TABLE VA

SUMMARY OF SURGE TESTS, BROCKTON CREEK

Date	Element Used to Determine Lag	Q at G.1 (m^3s^{-1}) 1)	ΔQ_1 2)	Lag 1-2 (min	Q at G.2 (m^3s^{-1})	ΔQ_2	Lag 2-3	Q at G.3 (m^3s^{-1})	ΔQ_3
Aug.15,67	Start of UP[3]	.0094	+.0013	6.25	.010	+.0010	6.	.0084	+.0002
	sharp peak	.0107		5.50	.011		5.	.0086	
	start of DS	.0101	-.0015	5.00	.011	-.0012	5.	.0091	-.0012
Aug.17,67	Start of UP	.0027	+.0015	11.0	.0035	+.0011	8.5	.0036	+.0008
	Start of DS	.0040	-.0006	13.0	.0045	-.0008	9.0	.0042	-.0006
	Start of UP	.00675	+.00055	7.8	.0066	+.00045			
	Start of DS	.0070	-.00145	8.2	.0070	-.0011			
	Start of UP	.0068	+.0008	8.2	.0072	+.00095	7.0	.0062	+.00075
Aug.26,68	Start of UP	.0043	-.0012	11.5	.0057	-.0013	5.8	.0056	+.00105
	Start of DS			1.0	.0068	-.0011	7.2		-.00085
Sept.14,68	Start of UP	.047	+.0050	4.2	.038	+.0055	3.2	.041	+.0025
	Start of DS	.052	-.0130	4.5	.045	-.014	3.0	.044	-.0080
Sept.22,68	Start of UP	.082	+.009	3.0	.084	+.008	3.0	.081	+.0040
	Start of DS	.086		3.0	.088	-.014	2.8	.085	-.0045

1) Q is the flow immediately prior to the test.
2) ΔQ is the change in flow as observed at a particular gauge.
3) UP = Up-surge; DS = Down-surge.

TABLE VB

SUMMARY OF SURGE TESTS, PLACID CREEK

Date	Element Used to Determine Lag	Lag 1-2 (min)	Q at G.2 (m^3s^{-1})	ΔQ_2	Lag 2-3 (min)	Q at G.3 (m^3s^{-1})	ΔQ_3	Lag 3-4 (min)	Q at G.4 (m^3s^{-1})	ΔQ_4
June 28, 1968	Start of UP	92.	.042	+.004	56.	.065	+.004			
	Mid- UP	110.			56.					
	Start of DS	113.	.044	-.004	56.	.067	-.004			
	Mid- DS	96.			49.					
Aug.28, 1968	Start of UP	69.5	.082	+.005						
	Mid- UP	77.5								
	Start of DS		.086	-.006	40.	.156	-.010			
	Mid- DS				43.					
Aug.30, 1968	Start of UP	89.	.042	+.007	51.	.058	+.004			
	Mid- UP	104.			57.					
	Start of DS	89.	.048	-.007	49.	.061	-.008	142.	.109	-.006
	Mid- DS	106.			56.			142.		

TABLE VC

SUMMARY OF SURGE TESTS, BLANEY CREEK

Date	Element Used to Determine Lag	Q at G.1 (m^3s^{-1})	ΔQ_1	Lag 1-3 (min)	Q at G.3 (m^3s^{-1})	ΔQ_3	Lag 3-5 (min)	Q at G.5 (m^3s^{-1})	ΔQ_5	Lag 5-4 (min)	Q at G.4 (m^3s^{-1})	ΔQ_4
May 19, 1967	Mid UP	.118	+.052	45.	.119	+.047	16.5			3-4 66.5	.123	+.045
	Start of UP			37.5			15.			62.		
	Mid DS	.162	-.042	47.	.158	-.038	17.			70.	.165	-.040
	Start of DS			38.			12.			52.		
June 9, 1967	Mid UP	.063	+.018	59.	.061	+.019	23.	.055	+.016	5-4 72.	.066	+.019
	Start of UP			49.			21.			72.		
Nov.19, 1967	Mid UP	.505	+.020	31.5	.495	+.020	9.	.435	+.020	19.	.555	+.015
	Start of Up			24.5						21.		
March 5, 1968	Mid DS	2.100	-.140	13.5	2.180	-.140	7.	2.120	-.120			
	Start of DS			9.5			8.					
	Mid UP	2.100	+.160	13.	2.180	+.120	6.	2.080	+.120			
	Start of UP			10.			5.					
Oct.13, 1968	Mid DS	1.370	-.180	18.5	1.090	-.140	7.	1.180	-.130	21.5	1.170	-.130
	Start of DS			13.			8.			20.		
	Mid UP	1.240	+.200	18.5	.970	+.180	7.	1,050	+.180	21.	1.050	+.170
	Start of UP			12.			7.			22.		
Nov.30, 1968	Mid UP (small)	1.240	+.020	19.5	.990	+.030	9.5	1.120	+.020	14.5	1.060	+.030
	Start of UP			16.			8.			18.		
	Mid DS(small)	1.260	-.030	19.5	1.020	-.040	9.5	1.140	-.020	15.	1.090	-.030
	Start of DS			17.			7.			15.5		
	Mid UP (large)	1.240	+.210	19	.980	+.240	8.5	1.120	+.210	16	1.060	+.210
	Start of UP			13.5			9.5			16		

TABLE VD

SUMMARY OF SURGE TESTS, PHYLLIS CREEK

Date	Element Used to Determine Lag		Lag 1-2 (min)	Q at G.2 (m^3s^{-1})	ΔQ_2	Lag 2-3 (min)	Q at G.3 (m^3s^{-1})	ΔQ_3	Lag 3-4 (min)	Q at G.4 (m^3s^{-1})	ΔQ_4	Lag 4-6 (min)	Q at G.6 (m^3s^{-1})	ΔQ_6
July 28, 1967	Mid	UP	36.	.338	+.008	25.	.342	+.010	18.2	.340	+.013	14.	.370	+.010
	Start of	UP	29.			22.			20.			12.		
	Mid	DS	32.5	.345	-.011	22.8	.350	-.010	19.	.352	.014	12.	.377	+.017
	Start of	DS	24.			24.			22.			11.		
May 19, 1968	Mid	DS	16.	2.340	-.040	11.	2.450	-.040	9.5	2.380	-.040			
	Start of	DS	9.5			11.5			8.					
	Mid	UP	17.	2.300	+.050	10.	2.440	+.040	10.	2.430	+.060			
	Start	DS	11.			10.			8.					
June 1, 1968	Mid	DS	17.	2.550	+.070	9.	2.500	-.070	8.	2.570	-.050	4.	2.550	-.050
	Start	DS	11.			11.			8.			5.		
	Mid	UP	16.5	2.530	+.20	10.5	2.500	+.20	6.	2.540	+.17	5.	2.520	+.150
	Start	UP	12.			9.			25?			6.		
June 22, 1968	Mid	DS	25.	.815	-.095	17.	.790	-.090	14.	.345	-.090	8.	.880	-.105
	Start	DS	17.			14.			12.			7.5		
	Mid	UP	25.	.720	+.10	16.	.700	+.10	14.	.755	+.095	8.	.775	+.105
	Start	UP	17.			13.			14.			6.5		
Sept.17, 1968	Mid	DS	16.	2.750	-.33	11.5	2.770	-.27						
	Start	DS	7.			13.5								
	Mid	UP	17.		+.25	10.	2.450	+.27						
	Start	UP	10.			10.5								

TABLE VD (Cont'd.)

SUMMARY OF SURGE TESTS, PHYLLIS CREEK

Date	Element Used to Determine Lag		Lag 1-2 (min)	ΔQ at G.2 (m^3s^{-1})	ΔQ_2	Lag 2-3 (min)	Q at G.3 (m^3s^{-1})	ΔQ_3	Lag 3-4 (min)	Q at G.4 (m^3s^{-1})	ΔQ_4	Lag 4-6 (min)	Q at G.6 (m^3s^{-1})	ΔQ_6
Nov.18, 1968	Mid	DS	18.5	2.740	-.26				2-4 36.5	2.890	-.240			
	Start	DS	11.5						25.5					
	Mid	UP	18.	2.450	+.16				35.5	2.580	+.170			
	Start	UP	11.5						25.5					
Nov.29, 1968	Mid	DS	19.	2.770	-.32	8.	3.200	-.35				3-6 9.5	4.200	-.30
	Start	DS	11.5			9.						6.		
	Mid UP (small)		21.	2.450	+.030	11.5	2.820	+.035				6.5	3.900	+.030
	Start UP (small)		17.			14.						6.		
	Mid	DS	18.5	2.480	-.17	9.	2.850	-.13				9.5	3.900	-.20
	Start	DS	10.5-12.5			10.						10.		
	Mid UP (large)		20.	2.310	+.31	12.	2.720	+.28				5.	3.700	+.30
	Start UP (large)		13.5			10.5						7.		

The test results are summarized in Tables 5A to 5D in which the gauge readings have been converted to discharges, on the basis of the tube rating curves (Section 3.4.5, Figure 13). The data appear to be consistent, insofar as the observed change in discharge remains constant from gauge to gauge along a test stream. The surge lags from station to station as shown in Table 5, have relatively low accuracy, particularly if they are short because the gauges could only be read at 30 to 60 second intervals. With lags in the order of 5 to 10 minutes, this introduces an immediate uncertainty of 10% to 20%. The lags based on the mid-points of the surges are substantially more reliable than the lags based on the starting points because the mid-points were derived by smoothing the level-time curve.

5. CHANNEL GEOMETRY AND STEADY FLOW EQUATIONS

5.1 Similitude Considerations for Steep, Degrading Channel Networks

The conditions under which readily available information, as defined in Section 2.5, may be adequate for evaluation of the channel network hydraulics will be discussed here.

5.1.1 Assumptions

Dynamic similitude between related physical systems can only be examined on the basis of a complete list of the forces affecting the system. Similarity between channel networks is possible if the major processes of formation are similar (Barr, 1968). Obviously there are a large number of processes which could conceivably affect the channel network of mountainous basins; the problem lies in identifying the dominant ones.

The formative processes assumed here may be biased towards the present problem in the sense that they describe a system that can be defined adequately with the readily available information. However, the field data supply evidence indicating that the system is reasonable and explains a major portion of the variation in and between channel networks. The assumptions are listed and discussed below.

(i) The drainage network occupies valleys whose longitudinal slopes, S_V, are remnants of the Pleistocene period. The streams form their channels by degrading into glacial debris, which contains sufficient coarse material to prevent the stream from reaching bed-rock or from degrading enough to achieve a channel slope, S, significantly different from S_V. In other words, S is imposed on the channel network, but the size of the material lining the channel is one of the results of the channel-forming process.

This is the very opposite of the common regime-type assumption with respect to slope and bed material, which states that a regime canal will adjust its slope by erosion or deposition, until it is adequate to handle the upstream supply of water and sediment. Most rivers fall between the two extremes with meandering and braiding playing an important role in reaching adjustment between water, sediment load, and valley slope. The main support for the present assumption lies in the consistent downstream steepening of the test streams (hanging valleys, Figures 2 and 3), lack of flood plains, absence of braiding or meandering, and the apparent close correlation between slope and size of the bed material.

(ii) The coarse material lining the degraded channels can only be moved at extreme flows, which are therefore solely responsible for the channel form. A single discharge value Q_D is adequate to represent these formative high flows.

This assumption is supported by the work of Miller (1958), who found high correlation between discharges of a given frequency and hydraulic parameters such as width, depth and velocity. Day (1969) presents similar correlations for the test reaches of this study. The theory on channel performance developed here does not depend on this assumption. It is only used in stating the similitude criteria.

(iii) The transport rates of material finer than the bed material are low and do not affect the performance of the channel. Supply of coarse material to the channel through slides, rockfalls, bank erosion, etc., is low and in balance with the transporting capacity of the channel.

Up to flows in the order of the mean annual peak, this assumption is well supported by field observation, but it may break down under extreme flood conditions, such as the event described by Stewart and LaMarche (1967). A sufficient amount of fine gravel and sand may then be in motion to lower the channel resistance significantly, thereby starting a chain reaction of higher velocity - more bank and bed erosion - lower resistance.

None of the reaches showed much evidence of active bed or bank erosion except in a few isolated locations, mainly associated with damaged vegetation cover of the stream banks due to recent logging.

(iv) The channel forming process is repeatable. If the same flow regime, Q(t), is diverted down identical valleys, containing debris of identical gradation, the mean properties of the resulting channels will also be identical.

This assumption is well supported in regime-type situations, where an identical supply of water and sediment to a straight channel segment eventually produces identical channel dimensions. The application of this concept to the present situation is speculative, but Section 5.3 will show that the significant channel parameters can be derived from the imposed or independent effects such as Q(t) and S, without requiring a knowledge of any dependent parameters, such as width or roughness. This eliminates the possibility of a large random effect in the channel forming process.

5.1.2 Conditions for Similarity

Considering a short, straight channel reach, one can identify the following forces (Barr, 1967; Barr and Herbertson, 1968): gravity g acting on the water (waves); gravity acting along the valley slope, gS; gravity acting on the submerged grains $((\rho_s - \rho_w)/\rho_s)\ g = g_s$,[1] and viscosity, ν. Q_D is imposed from upstream. Some of the resultant measures are W_D, the water surface width at flow

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[1]Herbertson and Barr like to consider 4 gravitational forces g, gS, as above and g_s as the net gravitational force on the submerged grains as it affects the surrounding grains, and $g_w = ((\rho_s - \rho_w)/\rho_w)\ g$ as the net gravitational force on the submerged grains as it affects the displaced water. Since g_w can be computed from g_s and g , it need not be specified.

Q_D; A_D/W_D, a depth measure based on the flow area at discharge Q_D, the velocity v_D, and D, the size parameter of the material lining the channel. These terms can be arranged in dimensionally homogeneous functional forms e.g. using dimensions of length

$$f_i \left(\left| \begin{matrix} \dfrac{Q_D^{2/5}}{g^{1/5}} \\ \\ \dfrac{Q_D}{\nu} \end{matrix} \right| , \frac{Q_D S}{\nu} , \frac{\nu^{2/3}}{g^{1/3}} , \frac{\nu^{2/3}}{g_s^{1/3}} , \left| \begin{matrix} W_D \\ \dfrac{A_D}{W_D} \\ D \\ \dfrac{\nu}{v_D} \end{matrix} \right| \right) = 0 \qquad \ldots 5.1$$

in which the terms between vertical bars can be used alternatively. Note that Equation 5.1 has only 5 terms as it consists of ratios of active forces and boundary actions. The dimensions l and t are reduced to l only. A functional form containing n + 1 dimensionally homogeneous variables can be reduced to a n-term non-dimensional form without reducing the generality. Of the numerous non-dimensional groupings possible with Equation 5.1, the following is most suitable for the experimental set-up at hand;

$$f \left(\frac{Q_D \, g^{1/3}}{\nu^{5/3}} , S , \frac{g}{g_s} , \frac{(\nu g)^{1/3}}{v_D} \right) = 0 \qquad \ldots 5.2$$

Equation 5.2 shows that in the present situation, where g/g_s, ν, and g are constant, correlations between Q_D, S, and any one resultant measure such as W_D or v_D should be complete. It also shows that complete kinematic similarity is only possible if the resultant measures and ν vary with Q_D according to the following proportionalities (Barr and Herbertson, 1968),

$$\begin{array}{rcl} \nu & \propto & Q_D^{.6} \\ D, \frac{A_D}{W_D}, W_D & \propto & Q_D^{.4} \\ v_D & \propto & Q_D^{+.2} \\ T & \propto & Q_D^{-.2} \\ S & \propto & Q_D^{\theta.} \end{array} \qquad \ldots\ 5.3$$

Since ν is not variable in the field and S is independent of Q, the field data do not have to match the above proportionalities, but it is reasonable to expect fairly close correspondence.

Equation 2 can also be derived from dimensional considerations (Yalim, 1966). The basic variables are ρ, ν, ρ_s, g, S, Q_D, and one resultant measure, say D. There are 7 variables containing 3 dimensions so that a non-dimensional form in 4 terms, such as Equation 5.2 is adequate for a description of the problem.

The system described by Equation 5.2 neglects many significant processes. In the field area of this study vegetation affects the smaller streams ($W_D < 15$ m) considerably through the formation of frequent log jams. Some minor reaches appear to be alluvial and may be aggrading (which would introduce the transport rate as a variable); others are occasionally on bed rock. The transport rates and sediment supply rates are unknown.

5.2 Basic Equations for Steady, Uniform Flow

5.2.1 Theoretical Considerations

The uniform flow parameters, mean velocity, v_m, surface width, W_s, and flow area, A, of a given straight open channel segment are fully defined by three equations:

(i) the Equation of Continuity which may be written as

$$Q = v_m A \ (= v_m W_s d_*) \qquad \ldots 5.4$$

where $d_* = A/W_s$, and

(ii) a geometrical equation linking A and W_s (e.g. $W_s \propto \sqrt{A}$ in the case of a triangular channel), and

(iii) a flow equation linking Q and one or several of the parameters v_m, A, W_s, and d_* in a form which is linearly independent of Equation 5.4. The constants of this equation will depend on the boundary and cross sectional shape of the channel segment.

In the case of natural "tumbling flow" channels, the relations between W_s and A are virtually unobtainable but this is not a serious drawback as long as the aim is hydrological. The main parameters are then only mean velocity, v_m, and channel storage per unit length, A, so that the W_s vs. A relation becomes redundant.

The flow equation can take many forms. Some of the more relevant possibilities are listed below, all assuming broad rectangular channels (d_* = depth = hydraulic radius) of width, W, and depth, d.

For flow governed by friction over a hydraulically rough boundary with roughness ratios between 7 and 130 (Ackers, 1958)

$$Q \propto A^{5/3} \qquad \text{(Manning's Equation)}$$

and for roughness ratios between 1.5 and 11

$$Q \propto A^{7/4} \qquad \text{(Lacey's Equation)}$$

For horizontal, frictionless channel segments controlled by a step-like drop at the downstream end, an approximate flow equation is

$$Q \propto A^{3/2}$$

If the control is a triangular weir with apex on the channel floor

$$Q \propto A^{5/2}$$

and in the case of a parabolic weir

$$Q \propto A^{7/2}$$

If the channel segment is obstructed by a dam with outflow below the watersurface

$$Q \propto A^{1/2}$$

Note that since $A = Q/v_m$, a relation of the form

$$Q \propto A^{w} \qquad \ldots\ 5.5$$

can also be stated as

$$Q \propto v_m^{\frac{w}{w-1}} \qquad \ldots\ 5.6$$

or as

$$Q \propto \frac{1}{\ell} T_*^{\frac{w-1}{w}} \qquad \ldots 5.7$$

in which T_* refers to unit length. In natural channels W_s is usually related to Q by an Equation of the form $W_s \propto Q^z$. The observed exponents z cover a range from 0.05 to 0.5 (Miller, 1958), but the most common values fall between 0.15 to 0.25. If this effect is included, $Q \propto A^w$ becomes

$$Q \propto A^{\frac{w}{zw+1}} \qquad \ldots 5.8$$

Considering tumbling flow as a randomly arranged sequence of short channel segments governed by flow equations similar to the ones listed above, and considering further that the travel times through channel segments in series are additive, one can express the relation between Q and T for long reaches ($\ell >> W_s$) as

$$Q = \lim_{\substack{n \to \infty \\ \ell_i \to 0}} \left[\sum_{i=1}^{n} \frac{c_i}{\ell_i} T_{*i}^{\ y_i} \right] \qquad \ldots 5.9$$

The constants c_i and the exponents $y_i (= \frac{w_i - 1}{w_i})$ are random variables with unknown distributions. Equation 5.9 can therefore not be solved to predict the form of the Q = f(T) relation.

5.2.2 Flow Equations of the Test Reaches

With the form of the tumbling flow equation not being predictable, a number of possibilities were tried by plotting various transformations of the Q - A data of several reaches against each other. Pure exponentials of the form

$$A = a_A\ Q^{\ b_A} \qquad \ldots\ 5.10$$

or

$$Q = \left(\frac{A}{a_A} \right)^{1/b_A} \qquad \ldots 5.10a$$

give consistently good fits, although in a few cases the fit can be slightly improved by assuming a small remnant flow area at zero discharge. Table 6 lists the coefficients a_A and b_A for all 13 test reaches of this study. The coefficients are based on linear regressions of $\log_{10}A$ on $\log_{10}Q$ of the form

$$\log_{10}A = \log_{10}\ a_A + b_A\ \log_{10}Q \qquad \ldots\ 5.11$$

RSQ is percentage of the variance of the logarithms explained by the regression equation. The standard error of estimate is only meaningful for the reaches with fairly high number of degrees of freedom. Additional data collected by Day (1969) are listed on Table 7. Figures 16 and 17 show data points and fitting lines for 2 sample reaches.

A Fortran IVG program"LØGRE"was used to compute the Q - A, Q - v_m, and Q - T regressions. Since v_m = 1/T and Q = Av_m,and since these facts were used to obtain the regression data, any one of the regressions is adequate to establish all

TABLE VI

REGRESSION PARAMETERS OF STEADY FLOW

Reach	Degrees of Freedom	a_A	b_A	Correlation Coefficient	Approx. St.Error of Estimate (%)
Brockton 1-2	8	.8104	.3376	0.98	12.5
Brockton 2-3	5	.7183	.2830	0.99	4.0
Placid 1-2	2	1.879	.3403	0.96	3.2
Placid 2-3	3	1.930	.3065	0.95	6.2
Placid 3-4	4	3.943	.4656	0.99	5.3
Blaney 1-3	6	3.375	.4784	0.99	7.8
Blaney 3-5	16	3.460	.5339	0.99	9.0
Blaney 5-4	7	3.110	.4389	0.98	8.9
Phyllis 1-2	7	3.262	.5413	0.98	9.0
Phyllis 2-3	9	3.202	.4577	0.99	3.6
Phyllis 3-4	7	3.021	.4787	0.99	3.9
Phyllis 4-6	6	3.199	.4023	0.97	7.8
Phyllis Lower	2	3.207	.3557	0.99	7.0

TABLE VII

REGRESSION PARAMETERS OF STEADY FLOW

(from Day, 1969)

Reach		Degrees of Freedom	a_A	b_A	Correlation Coefficient
Furry		5	5.788	.5009	.95
Slesse	U	3	2.922	.5993	.99
Slesse	M	4	4.179	.5213	.97
Slesse	L	4	2.926	.5765	1.0
Juniper		3	3.377	.4075	.99
Ewart	U	3	4.091	.4184	.99
Ewart	L	4	3.253	.4365	.97
Ashnola	U	3	3.092	.5806	1.0
Ashnola	M	3	4.638	.4647	1.0
Ashnola	L	3	3.967	.4208	1.0

three and any significant deviation of the regression coefficients from their theoretical relations indicates computational errors. With

$$T = a_T Q^{b_T} \qquad \ldots\ 5.12$$

and

$$v_m = a_v Q^{b_v} \qquad \ldots\ 5.13$$

one obtains

$$b_v = -\ b_T \qquad \ldots\ 5.14A$$

$$a_v = -\ \frac{1}{60\ a_T} \qquad \ldots\ 5.14B$$

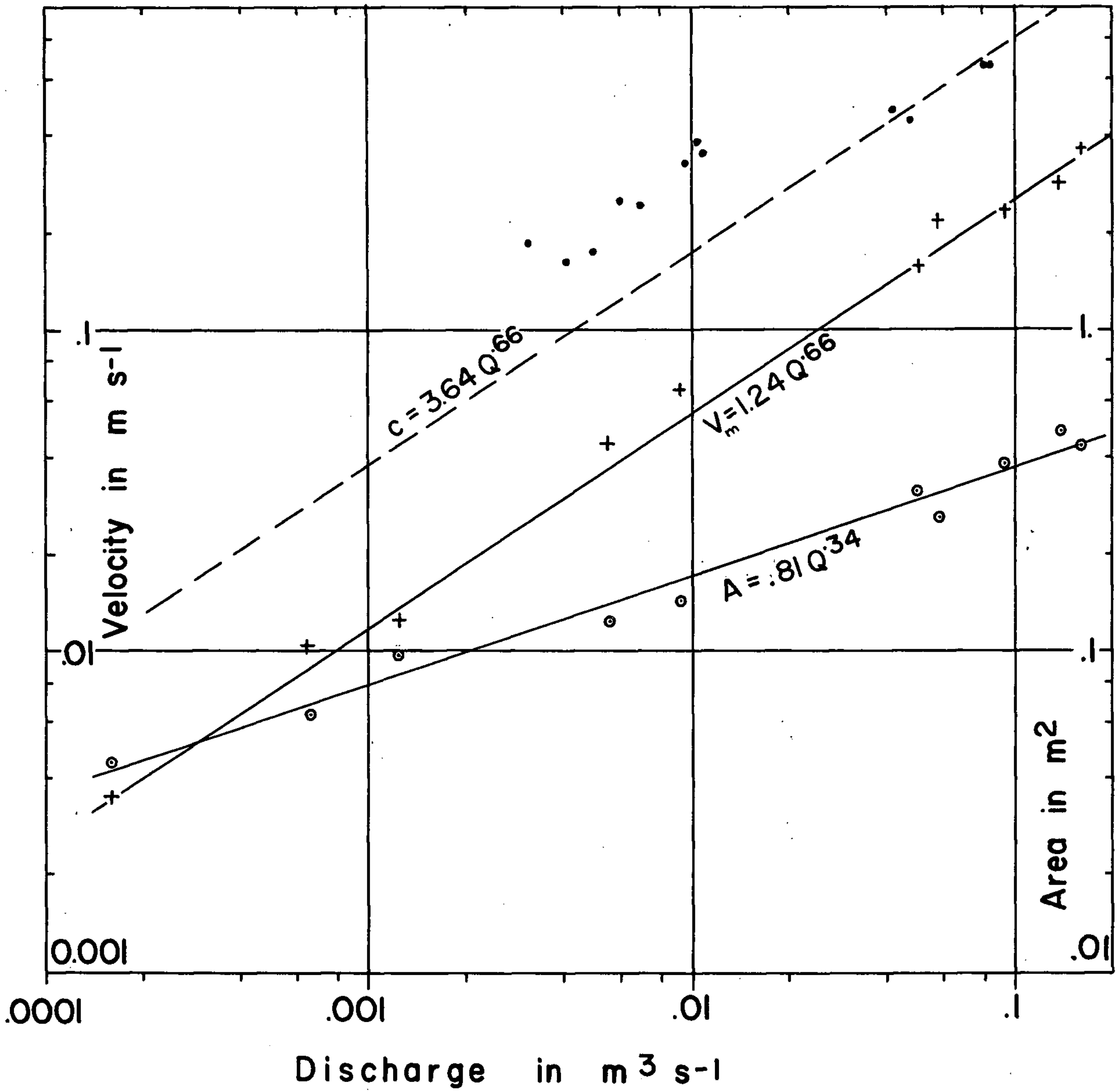
$$b_v = 1 - b_A \qquad \ldots\ 5.14C$$

$$a_v = \frac{1}{a_A} \qquad \ldots\ 5.14D$$

"LØGRE" also computes two sets of Q - T_s (T_s = arrival time of tracer) and Q - T_P (T_P = peak time) regressions, one using all data, and one using only those tests for which the dye has been injected at the upstream end-point of the test reach.

The above regressions constitute one of the main results of the field work, therefore all the "LØGRE" printouts and plots with lists of the data are included in the Appendix.

Inspection of the plots (Figures 16 and 17, and Appendix) shows that, over the range of discharges covered here, there appear to be no significant deviations from Equation



+ mean velocity , V_m

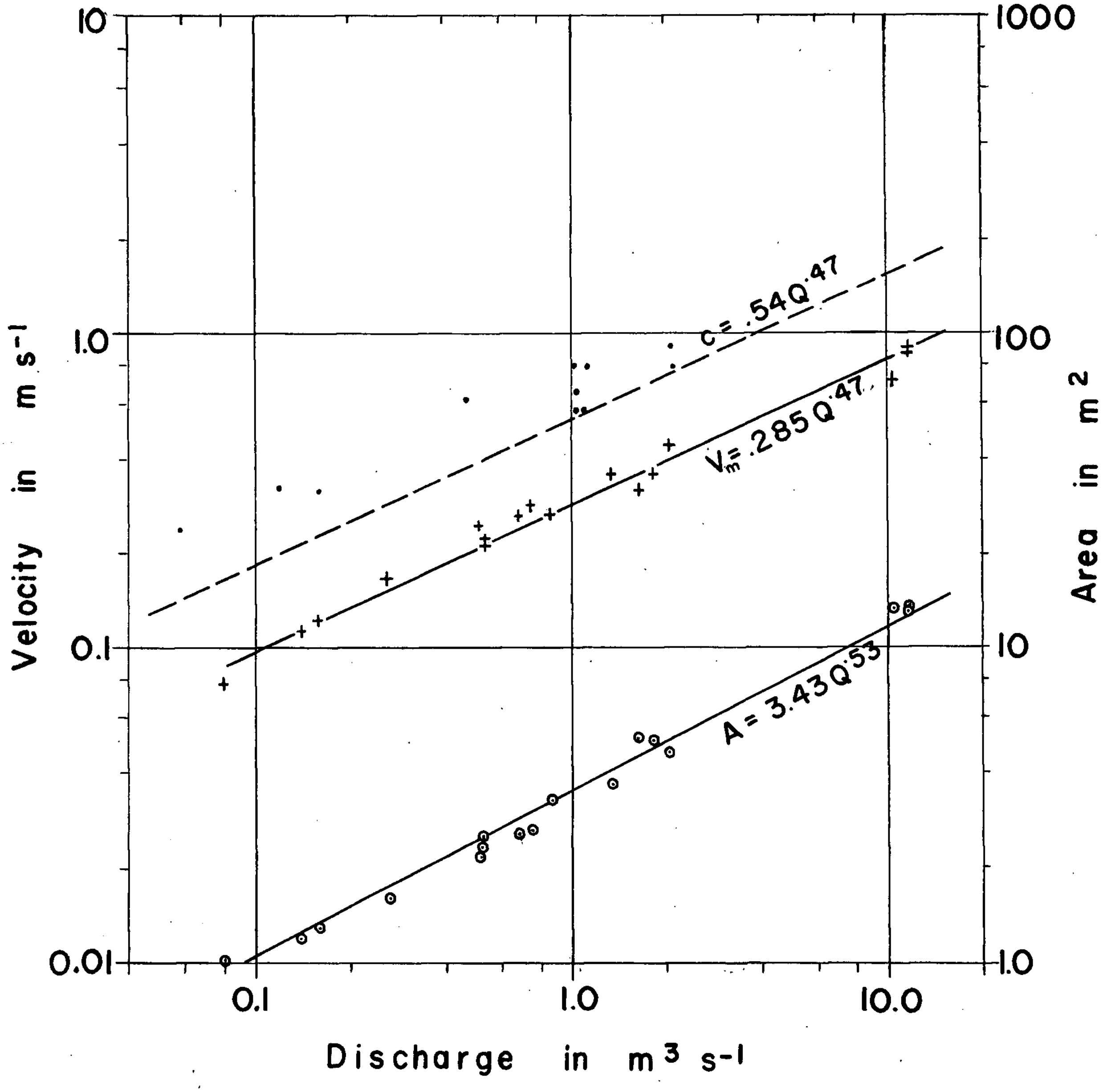
∘ mean cross-sectional area , A

· surge celerity observations , c

HYDRAULIC MEASUREMENTS ON THE REACH

BROCKTON GAUGE 1 - GAUGE 2

Fig. 16



+ mean velocity, V_m

⊙ mean cross-sectional area, A

• surge celerity observations for mid-surge, c

HYDRAULIC MEASUREMENTS ON THE REACH

BLANEY GAUGE 3 - GAUGE 5

Fig. 17

5.10. In particular, the Q - v_m plots of Blaney Creek, which has the best coverage of the discharge range, show no tendency towards linear basin response (v_m independent of Q) at high flows as observed by Pilgrim (1966). With the almost total absence of flood plains along the test reaches this is not surprising.

5.3 Determining the Parameters of the Steady Flow Equation

The basic flow equation (5.10) can be re-written in various non-dimensional forms; one possibility, using terms similar to Equation 5.2 is

$$\frac{A\ g^{2/3}}{\nu^{4/3}} = a_A' \left(\frac{Q\ g^{1/3}}{\nu^{5/3}}\right)^{b_A} \qquad \ldots\ 5.10b$$

or, if the concept of a formative discharge is retained

$$\frac{A}{A_D} = \left(\frac{Q}{Q_D}\right)^{b_A} \qquad \ldots\ 5.10c$$

Either version is suitable for comparison with the basic similitude criterion, Equation 5.2. The parameters of the steady flow equation (a_A', A_D, b_A) can be considered "resultant measures" of the channel forming process, similar to channel width, W_D, depth, and roughness, D. Two possible forms of the similitude criterion 5.2 are therefore

$$f\left(\frac{Q_D\ g^{1/3}}{\nu^{5/3}},\ S,\ \frac{g}{g_s},\ b_A\right) = 0 \qquad \ldots\ 5.2a$$

$$f\left(\frac{Q_D g^{1/3}}{\nu^{5/3}}, \quad S, \quad \frac{g}{g_s}, \quad a_A'\right) = 0 \qquad \ldots 5.2b$$

which shows that a_A and b_A should be determined by Q_D and S, as all other variables can be assumed constant. Exploring this possibility in detail is the main objective of Day (1969). Some of his findings will be summarized briefly here.

Q_D is obviously not a "readily available" parameter, but in a region with reasonably homogeneous climate the contributing drainage area of a channel segment, DA, can be used instead. As there is virtually nothing known about the physics of the process expressed by Equation 5.2, multiple regressions of a_A and b_A on the independent variables drainage area (in km^2), and slope were tried, including various combinations of transformed data.

For the test reaches in the mountainous areas surrounding the lower Fraser Valley the following two equations give best fit

$$a_A = 1.738 \; DA^{0.2922} \qquad \ldots 5.15$$

$$b_A = 0.2888 \; S^{-0.1006} \; DA^{0.0756} \qquad \ldots 5.16$$

They explain 96.1% and 69.7% of the variance in the data. With 11 degrees of freedom, both equations are significant well beyond the 1% level. The 6 test reaches in the dry interior of B. C. (Juniper Creek, Ewart Creek and the Ashnola River, Tables 3 and 7) do not cover a wide enough range of the independent variables to justify a meaningful relationship for that region.

In generally applicable relations for a_A and b_A drainage area can obviously not take the place of Q_D. If it is to be used in the analysis, some correction factor for regional variations in the drainage area-runoff relation has to be added. An alternative to such a factor is the use of a consistent discharge value, such as the estimated mean annual flood. A further possibility which may prove interesting in areas with sparse hydro-meteorological records, is to use width, W_D, as an independent variable. The data of this study give good fit to regionally constant relations between width and drainage area and these relations are easily established by measuring a few channel widths on streams of various sizes in the region of interest. Slope appears to have no effect on width.

With W_D and S as independent variables and including the data from all areas one obtains the following relations for a_A and b_A

$$a_A = 0.9408\ W_D^{0.472} \qquad \ldots\ 5.17$$

$$b_A = 0.2519\ W_D^{0.1368}\ S^{-0.0822} \qquad \ldots\ 5.18$$

They explain 86% and 68% of the data variance and are statistically significant at the 1% level, having 18 degrees of freedom. It is not surprising that width and slope determine b_A to a lesser degree than a_A. The factor a_A, which is identical to the flow area at $Q = 1\ m^3s^{-1}$ can vary over a large range and needs to be predicted closely. The range of b_A is limited to approximately $0.25 < b_A < 0.65$, so that accurate prediction of b_A is not essential as long as the flows of interest are of the order of $1\ m^3\ s^{-1}$. More general predictive equations should be possible on the basis of Equation 5.10c by substituting measured values of DA or ${W_D}^2$ for the uncertain Q_D (the relation between DA and W_D found by Day are consistently close to $DA \propto {W_D}^2$).

5.4 The Friction Concept Applied to Tumbling Flow

Although the present study does not reply on the friction concept and the data are less than ideally suited for application of generally accepted open channel friction formulas, a brief comparison between friction formulas and the general flow equation (5.10) may be interesting and may facilitate comparison with other studies. To the writer's knowledge, all previous work on very rough channels or on tumbling flow[2] is based on the friction concept, which, by requiring virtually unobtainable roughness data, tends to yield results that cannot be applied to hydrological problems.

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[2]Utah State University appears to have a continuing research program on tumbling flow. Some of the results are published in Peterson and Mohanty, 1960 and in an extensive number of M.Sc. and Ph.D. theses, such as Al Kafaji, 1961; Judd, 1963; Abdelsalam, 1956. Other studies on rough, natural channels are: Leopold, Bagnold et al., 1960; Mirajgaoker and Charlu, 1963; Johnson, 1964; Herbich, 1964; Argyropoulos, 1965; Kellerhals, 1967; Hartung and Scheuerlein, 1967; Scheuerlein, 1968.

5.4.1 Open Channel Flow Formulas

The problem to be solved by an open channel flow formula is of the form

$$v_m = f\ (D,\ d_*,\ S,\ g,\ \rho_w,\ \mu\ ,\ X_i) \qquad \ldots\ 5.19$$

in which $d_* = A/W_s$ replaces the more commonly used hydraulic radius (they are almost indistinguishable in most stream channels), μ is viscosity and the X_i are non-dimensional correction factors which vanish in the case of a broad rectangular channel section and a particular shape of the roughness elements of diameter D. Assuming this to be the case, one obtains the commonly used non-dimensional form of Equation 5.19

$$\frac{v_m^{\ 2}}{g d_* S} = f\ (S,\ \frac{v_m d_*}{\nu}\ ,\ \frac{D}{d_*}) \qquad \ldots\ 5.20$$

It is well established that S on the right side of (5.20) can be neglected as long as steady flow does not lead to the formation of surface waves and either one of the two remaining parameters on the right is often negligible also, depending on their relative size. A formulation of Equation 5.20 for the case of steep, rough channels is (Keulegan, 1938)

$$\frac{v_m}{\sqrt{g d_* S}} = 6.25 + 5.75\ \log_{10}(\frac{d_*}{D}) \qquad \ldots\ 5.21$$

which can be fitted approximately with exponential functions of the form

$$\frac{v_m}{\sqrt{gd_* S}} = c_1\left(\frac{d_*}{D}\right)^{c_2} \qquad \ldots\ 5.22$$

The c_i are constants. The exact value of c_2 depends on d_*/D as shown in Figure 18.

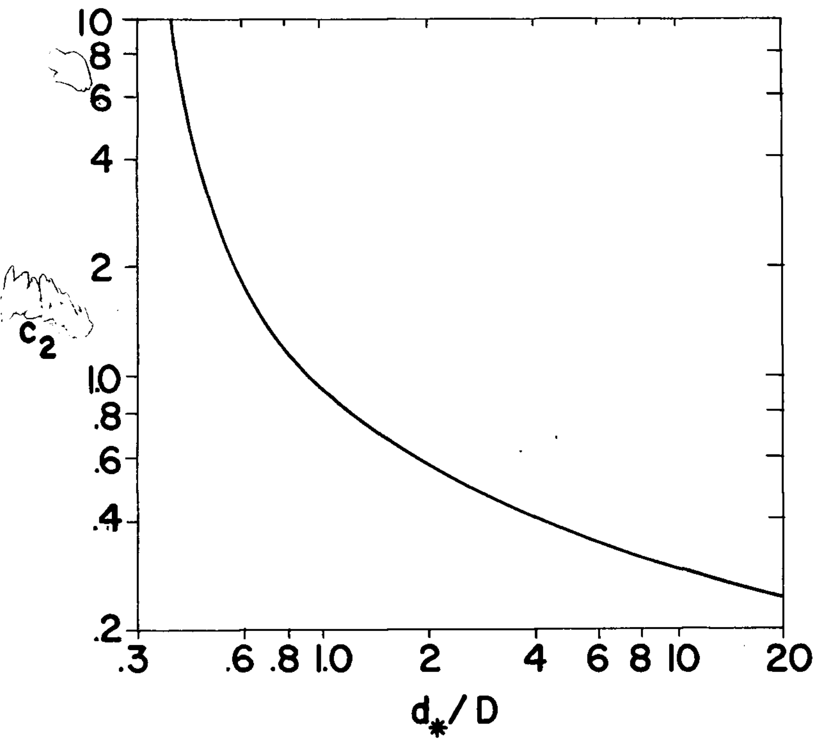


FIGURE 18. VALUES OF c_2 FOR BEST FIT TO EQUATION 5.21

The commonly used Manning Equation assumes a c_2 of 1/6, which provides good fit to Equation 5.21 over the range $7 < \frac{d_*}{D} < 130$.

It is important to note that Equation 5.22 neglects the terms $v_m d_*/\nu$ and S of Equation 5.20. While the theory of turbulent boundary layers justifies the former, there is no a priori justification for the latter in cases where the roughness elements affect the free surface, as in tumbling flow.

5.4.2 Comparison with the Data

The steady-flow data of this study consist of exponential relations between discharge and velocity for conditions of constant, but unknown roughness, constant known slope, and constant, but unknown cross-sectional shape. To transform Equation 5.22 into comparable form requires some assumptions regarding cross-sectional shape. Two assumptions will be used which should bracket the true situation (Section 5.2.1).

(i) $W_s = c_3$

(ii) $W_s \propto Q^{0.2}$

Equation 5.22 can be written as

$$d_* \propto v_m^{\frac{1}{c_2+0.5}}$$

and with Assumption (i) this leads to

$$v_m \propto Q^{\frac{c_2+0.5}{c_2+1.5}} \qquad \ldots\ 5.23$$

and with Assumption (ii) it leads to

$$v_m \propto Q^{\frac{0.8(c_2+0.5)}{c_2+1.5}} \qquad \ldots\ 5.24$$

The Q-exponents of the last two equations correspond to b_v of Equation 5.13, and b_v is related to b_A of Tables 6 and

7 by $b_V = 1 - b_A$ (Equation 5.14C). The observed b_V cover the range $0.4 < b_V < 0.72$. On Figure 19, the exponents of Equations 5.23 and 5.24 have been plotted against c_2.

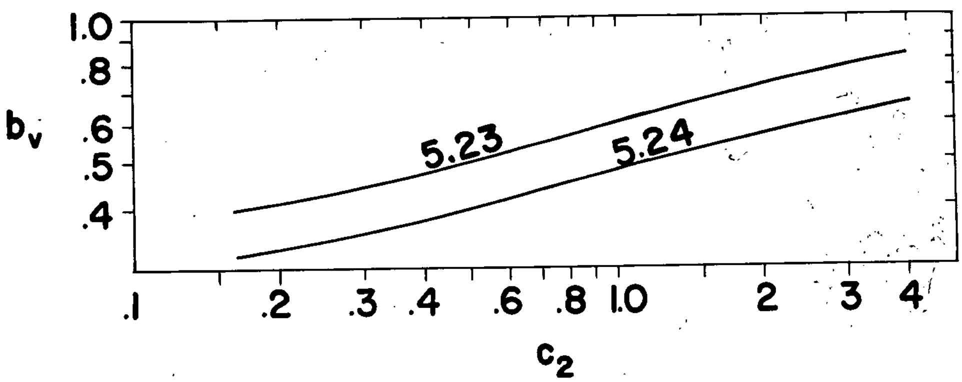


FIGURE 19. EXPONENTS OF EQUATIONS 5.23 AND 5.24 vs. c_2

Using Figure 19 to convert the observed b_V to c_2, and Figure 18 to convert c_2 to apparent roughness ratios (assuming Equation 5.2 is valid) one can see that the data cover the approximate range of roughness ratios from 0.4 to 8, which is compatible with the appearance of the channels. From Equation 5.18 one can obtain an explicit equation for b_V

$$b_V = 1 - 0.25\ W_D^{.14} S^{-.08} \qquad \ldots\ 5.25$$

which, if used in conjunction with Figures 18 and 19 indicates that at a fixed slope, large channels are relatively smoother than small channels and, for a given channel size, steep channels are rougher than flat channels. This also agrees with field observations.

In conclusion, the data of this study appear to be compatible with the commonly accepted logarithmic law for rough channels, Equation 5.21, but without information on roughness size and on shape of the flow sections, it is not possible to decide whether this equation gives a meaningful representation of flow in extremely rough channels.

6. UNSTEADY FLOW IN STEEP CHANNELS

6.1 Kinematic Waves and the Surge Test Results

6.1.1 Some Features of Kinematic Waves

Lighthill and Whitham (1955) introduced the term "kinematic wave" for a class of waves which arise in one-dimensional flow systems if there is a unique functional relation between:

(i) the flow Q,

(ii) the position x, and

(iii) the quantity per unit distance (A in the case of a stream).

The wave motions are then governed by the equation of continuity alone. It has long been recognized that the movement of a flood down a long river can be approximated by this type of wave (Seddon, 1900; Massé, 1935).

The equation of continuity for unsteady flow in a long channel is

$$\frac{\partial Q}{\partial x} + \frac{\partial A}{\partial t} = 0 \qquad \ldots\ 6.1$$

which can also be written as

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial A}\,\frac{\partial A}{\partial x} = 0 \qquad \ldots\ 6.2$$

$\partial Q/\partial A$ has dimensions of a velocity and can only depend on Q and on the position x if the assumptions for kinematic waves are satisfied. An observer moving along x at speed $\partial Q/\partial A$

will then observe no change in area or discharge ($DA/Dt = 0$), which shows that Equation 6.2 defines a wave motion with

$$\frac{\partial Q}{\partial A} = c(x,Q) \qquad \ldots\ 6.3$$

being the celerity of these kinematic waves.

In Chapters 8 and 9 of his book on open channel flow, Henderson (1966) examines the conditions under which kinematic waves can approximate the movement of flood waves. The equation of motion for a prismatic channel can be written as (Henderson's Equation 8.5):

$$\underset{1}{S_f} = \underset{2}{S} - \underset{3}{\frac{\partial d}{\partial x}} - \underset{4}{\frac{v_m}{g}\frac{\partial v_m}{\partial x}} - \underset{5}{\frac{1}{g}\frac{\delta v_m}{\partial t}} \qquad \ldots\ 6.4$$

in which S_f is the friction slope (e.g. $n_m^{\ 2}\ v_m^{\ 2}/d_*^{4/3}$ if the Manning Equation is applicable).

Terms 1 and 2 define steady, uniform flow, terms 1 to 4 define steady, non-uniform flow, and the complete equation applies to un-steady, non-uniform conditions. If S is much larger than the 3 other terms on the right side of Equation 6.4, the wave motion is approximately kinematic. Henderson shows that this condition is satisfied in a relatively steep, alluvial river, even during a very rapid flood rise.

In tumbling flow channels the 5th term is always one or more orders of magnitude smaller than the averaged S but may be comparable with the local S in a few places (big pools). Terms 3 and 4 however, may be of order S or more and, like S, they are highly and unpredictably variable with x. Equation 6.4 does not, therefore, permit any definite conclusions regarding the applicability of kinematic wave theory to tumbling flow. Only experimental evidence can do this.

The equation for the kinematic wave celerity at a fixed location, $c = dQ/dA$, can also be stated as

$$c = \frac{d(vA)}{dA} = v + A\frac{dv}{dA}$$

which shows that c increases with discharge in natural river channels. In a truly kinematic channel, c(x), is therefore a unique and increasing function of A or Q. As a consequence, a kinematic wave cannot disperse but the higher parts of the wave will tend to overrun the lower parts, resulting in a gradually steepening wave front of positive waves.

In Section 5.2 it was shown that channel reaches in the tumbling flow regime obey equations of the form $A = a_A Q^{b_A}$ (5.10). Kinematic waves in such channels should therefore have the celerity

$$c = \frac{dQ}{dA} = \frac{1}{a_A^{1/b_A}\, b_A} A^{1/b_A - 1} \qquad \ldots\ 6.5$$

or, in terms of discharge

$$c = \frac{1}{a_A b_A} Q^{1-b_A} \qquad \ldots\ 6.6$$

Substituting $b_v = 1 - b_A$ and $a_v = 1/a_A$ into Equation 6.6 gives

$$c = \frac{v_m}{b_A} \qquad \ldots\ 6.7$$

which shows that the kinematic wave celerity is proportional to v_m.

6.1.2 Indications from the Surge Test Results

The three features of kinematic waves which are suitable for immediate comparison with the field data are:

(i) the steepening of positive wave fronts,

(ii) the non-dispersive nature of kinematic waves,

and

(iii) the wave celerity $c = v/b_v$.

All surge tests show a consistent downstream flattening of positive and negative wave fronts and a tendency towards increasingly smooth Q(t)-curves in the downstream direction. Both facts are clear evidence for dispersive effects.

According to Equation 6.7, the kinematic surge celerity plots as a straight line on logarithmic paper, parallel to the $Q - v_m$ line. Figures 16 and 17 show these lines for two test reaches, together with the observed surge celerities based on the mid-points (over Q) of the observed rise or fall. The agreement between the theoretical line and the observed surge celerities is consistently similar to the situation shown in Figures 16 and 17. At intermediate to high flows the agreement between observed and kinematic celerities is always close but at low flows, the observed celerities tend to be significantly higher than kinematic.

Symmetrical tests, consisting of an up-surge followed by a similar down-surge (or vice versa) are particularly instructive on the mechanism of wave propagation (Figure 15). The time lag between the mid-points of the up and down-surges remains very closely constant and identical to the original lag at the lake outlet, with the celerity of these mid-points being close to kinematic at all but the lowest flows and with the sharp changes in discharge becoming gradually smoother as noted above.

From these comparisons one can conclude that the mechanism of wave propagation at intermediate to high flows through channels in the tumbling flow regime is essentially kinematic with a certain dispersive effect added.

The digression from kinematic conditions at low flows can be explained as follows: at low stage large parts of any tumbling flow channel are occupied by relatively deep and slow-moving pools, which are not kinematic according to the assumptions stated in Section 6.1.1. Changes in discharge propagate through pools at the dynamic wave celerity $\sqrt{g\ d}$, which will generally be much larger than the corresponding kinematic wave celerity. For example, at a flow of 0.01 m^3s^{-1} Brockton Creek (Figure 16) contained a few pools with depth of more than 0.3 m and a large number of pools with depths between 0.1 and 0.3 m. The kinematic wave celerity at this flow is 0.18 ms^{-1} and the dynamic celerities for 0.1 m and 0.3 m depth are 1 ms^{-1} and 1.7 ms^{-1}. The effects of pools will be examined in greater detail in Section 6.3.

6.2 Kinematic Waves with Storage Dispersion

6.2.1 Dispersion through Secondary Dynamic Effects

In the case of long rivers with relatively prismatic channels and sub-critical flow throughout, it has long been recognized that the propagation of flood waves is mainly kinematic with some dispersive effects added. Hayami (1951) introduced the equation

$$\frac{\partial A}{\partial t} + \frac{3v_m}{2} \frac{\partial A}{\partial x} = D_c \frac{\partial^2 A}{\partial x^2} \qquad \ldots 6.7$$

for this type of kinematic wave. This corresponds to Equation 6.2, with $3v_m/2$ being the kinematic wave celerity according to the Chézy friction formula and D_c being an undetermined dispersion coefficient. Hayami arrived at Equation 6.7 by adding the effect of the changed water surface slope, which occurs during the passage of a flood wave, to the basic flood wave equation (6.2). Without detailed argument he claims further that the dispersion coefficient consists of a sum of two terms, one accounting for the slope effect and the other accounting for wave dispersion in storage elements, such as pools or permeable stream banks. He gives an explicit solution for Equation 6.7, based on the linearizing assumptions of constant v_m and constant D_c, and found good agreement between computed and observed propagation of an artificially produced symmetrical flood wave. The main difficulties with

Hayami's solutions are the normally unpredictable size of D_c and v_m.

Lighthill and Witham discuss several different forms of the dispersion term in the kinematic wave equation. They claim that, since the dispersive term is probably small, when compared with the terms on the left of Equation 6.7

$$\frac{\partial A}{\partial t} \sim - c \frac{\partial A}{\partial x}$$

and the dispersion term can therefore be stated in any one of the three forms: $\partial^2 A/\partial x^2$, $1/c^2 \ \partial^2 A/\partial t^2$, or $1/c \ \partial^2 A/\partial x \partial t$. The dispersion coefficient, however, may be more easily established for one form than for the others, depending on circumstances. Two methods for determining the dispersion coefficient are given, one based on observed flood profiles at fixed times (which is rarely possible), and the other based on the well-known (but difficult to measure) hysteresis effect, which occurs in stage-discharge rating curves during the passage of a flood wave. In tumbling flow channels the dispersion term may not be small because of the extensive pool storage, so that the above transformation of the term containing second derivation is not justified.

Henderson (1966) also discusses the dispersion of kinematic waves and shows that his form of the equation of motion (6.4) can be transformed into the kinematic wave equation of Hayami, Equation 6.7, if terms 4 and 5 are neglected.

In conclusion it appears that there is a considerable body of knowledge on dispersive kinematic waves, but it is based on, and applicable to long, relatively prismatic channels, in which the dispersion is mainly the result of differences between steady and unsteady slopes and the resulting hysteresis in the stage-discharge relations. Tumbling flow is characterized by frequent transitions from sub-critical to supercritical depths, which means that, at least at the critical section, the stage-discharge relation is unique. The dispersion is therefore attributable solely to storage. The consequences of this do not appear to have been investigated before.

6.2.2 The Differential Equation of Kinematic Waves with Storage Dispersion

A channel, in which the relation between Q and A is only unique at regularly spaced discrete locations x_i, has the following Q - A relation at intermediate points $x_{i-1} < x < x_i$ (see Figure 20)

$$Q = f(A)\Big|_{x_i} + \beta W_D \frac{\partial A}{\partial t} \qquad \ldots\ 6.8$$

since, at rising stage, some of the discharge at x will go into storage between x and x_i.

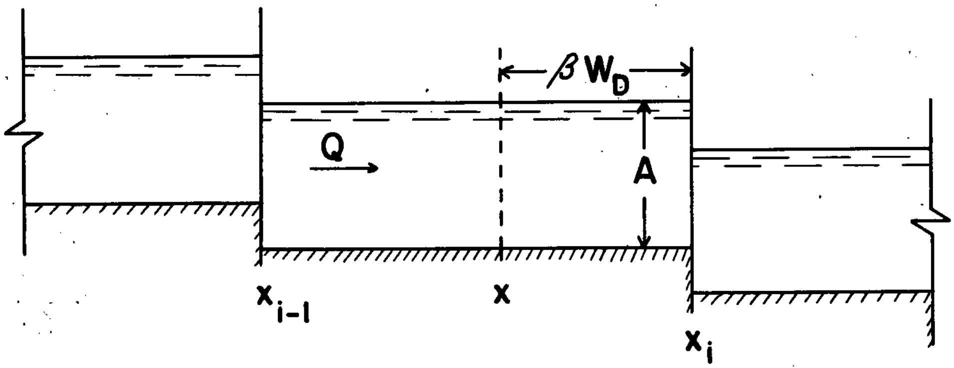


FIGURE 20. DEFINITION SKETCH FOR EQUATION 6.8

The coefficient of $\partial A/\partial t$ has dimension L, and with W_D being the only length readily available in the present problem (Section 5.3), it is convenient to use it in Equation 6.8, together with a non-dimensional coefficient β , whose value will have to be determined later on. Physically, the factor β W_D is a length measure in direction x, related to the average size of pools. In some alluvial rivers, pool-riffle sequences scale approximately with width (Leopold et al., 1963). By using W_D in (6.8) one assumes that a similar relation holds in tumbling flow channels.

Assuming a long channel with densely spaced control sections x_i , one may substitute Equation 6.8 into the equation of continuity (6.1), at least as an approximation, giving

$$\frac{\partial A}{\partial t} + \frac{\partial Q}{\partial A}\frac{\partial A}{\partial x} + \frac{\partial Q}{\partial A_t}\frac{\partial^2 A}{\partial x \partial t} = 0$$

in which A_t stands for $\partial A/\partial t$. Noting that $\partial Q/\partial A = c$ and, according to Equation 6.8, $\partial Q/\partial A_t = \beta W_D$, one obtains

$$\frac{\partial A}{\partial t} + c\,\frac{\partial A}{\partial x} = -\beta W_D\,\frac{\partial^2 A}{\partial x \partial t} \qquad \ldots\ 6.9$$

This is the basic equation for kinematic waves with storage dispersion. Lighthill and Whitham (1955) arrive at the same equation by considering the effect of hysteresis in a stage-discharge rating curve, applicable to the whole reach.

Equation 6.8 also defines a hysteresis effect in the Q - A relation, but in a natural tumbling flow channel this is probably highly variable and could certainly not lead to a practical method for estimating β, the one remaining free parameter. An alternative is to obtain an explicit solution for Equation 6.9, with the relatively simple initial conditions of the surge tests, and then to obtain β by fitting the solution to the observed surges. This will be done in the following two sections.

6.2.3 A Solution for Step-like Input

An explicit solution of Equation 6.9 is only obtainable with the linearizing assumptions of constant c and constant β, which is justifiable for the surge tests, since $\Delta Q \ll Q$. With these assumptions Equation 6.9 becomes a linear, homogeneous, partial differential equation of second order. The substitution

$$F = A\,e^{\left(\frac{x}{\beta W_D} + \frac{ct}{\beta W_D}\right)} \qquad \ldots\ 6.10$$

transforms the equation into the compact, first canonical form

$$\frac{\partial^2 F}{\partial x \partial t} - \frac{cF}{\beta^2 W_D{}^2} = 0 \qquad \ldots\ 6.11$$

which has the characteristics x = const. and t = const.

The initial conditions and boundary conditions of a surge test can be approximated as:

$$A(x \geq 0,0) = A_o \qquad \ldots\ 6.12$$

$$\left.\begin{aligned} A(0,0) &= A_o \\ A(0,t>0) &= (1+\alpha)\,A_o \qquad \alpha \ll 1.0 \end{aligned}\right\} \ldots\ 6.13$$

After the above transformation they become

$$F(x \geq 0,0) = A_o\,e^{\frac{x}{\beta W_D}} \qquad \ldots\ 6.12a$$

$$\left.\begin{aligned} F(0,0) &= A_o \\ F(0,t>0) &= (1+\alpha)A_o e^{\frac{ct}{\beta W_D}} \qquad \alpha \ll 1.0 \end{aligned}\right\} \ldots\ 6.13a$$

Since the initial conditions define F on two of its characteristics, the problem to be solved is a so-called Goursat problem. Under the above conditions it has a unique solution (Mikhlin, 1966), which can be found by Riemann's method.

The Riemann Function B is

$$B = I_o \left[\sqrt{\frac{4c}{\beta^2 W_D^2} xt}\right]$$

in which $I_o(u)$ is the "Modified Bessel Function of the First Kind of Order Zero". The solution of (6.9) is of the form

$$F(x,t) = F(0,0)B(0,0) + \int_0^x B \frac{dF(\xi,0)}{d\xi} d\xi + \int_0^t B \frac{dF(0,\tau)}{d\tau} d\tau \qquad \ldots 6.14$$

in which ξ and τ are dummy variables in length and time coordinates respectively. The last term of (6.14) cannot be evaluated in the above form since the derivative $dF(0,\tau)/d\tau$ is undefined at $\tau = 0$. Partial integration of this term gives

$$F(x,t) = F(0,0)B(0,0) + \int_0^x B \frac{dF(\xi,0)}{d\xi} d\xi + B(0,\tau)F(0,\tau) \Big|_{\tau=0}^{\tau=t}$$

$$- \int_0^t \frac{dB}{d\tau} F(0,\tau)\, d\tau \qquad \ldots 6.15$$

Since $dI_o(u)/du = I_1(u)$ and $I_1(u)$ is the "Modified Bessel Function of the First Kind of Order One", substituting for B and the starting conditions of F in 6.15 gives

$$F(l,t)/A_o = \frac{1}{\beta W_D} \int_0^l I_o\left(\sqrt{\frac{4c}{\beta^2 W_D^2}(l-x)t}\right) e^{\frac{x}{\beta W_D}} dx + (1+\alpha)\, e^{\frac{ct}{\beta W_D}}$$

$$+ \frac{2\, cl\,(1+\alpha)}{\beta^2 W_D^2} \int_0^t e^{\frac{c\tau}{\beta W_D}}\, I_1\left(\sqrt{\frac{4cl(t-\tau)}{\beta^2 W_D^2}}\right) \left(\frac{4cl(t-\tau)}{\beta^2 W_D^2}\right)^{-1/2} d\tau \qquad \ldots\ 6.16$$

With $A(l,t) = F \exp(-l/\beta W_D - ct/\beta W_D)$ this is the explicit solution of Equation 6.9 for a reach of length l. Under normal circumstances it can be simplified considerably.

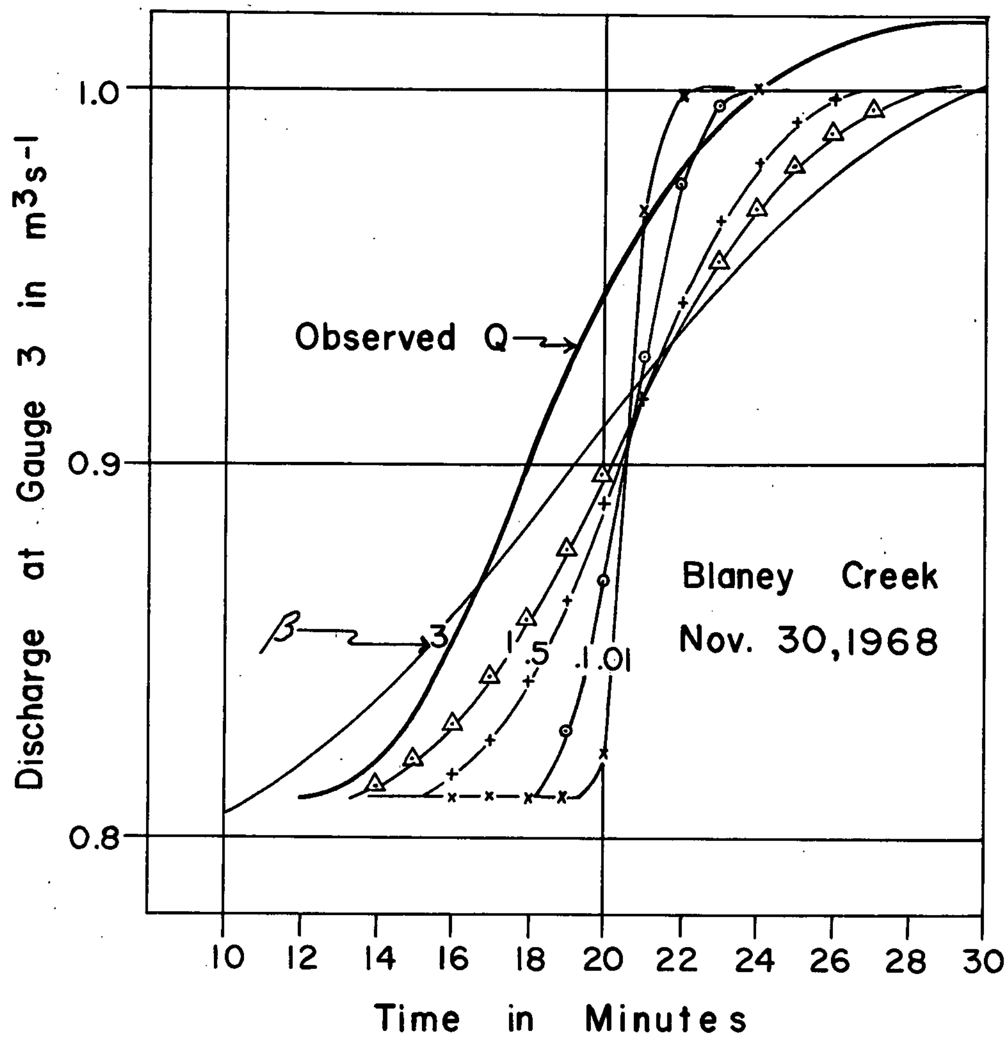
For large arguments u the Bessel functions $I_o(u)$ and $I_1(u)$ tend asymptotically towards the function $\sqrt{1/2\pi u}\,\exp(u)$. According to Jahnke and Emde (1945) the agreement is within 5% at $u = 9$. With the l-values and time lags of the present surge data, the arguments will always be much larger than 10, so that the Bessel functions can be replaced by their more easily computed asymptote. The middle term on the left of (6.16) is negligible, when compared with the two integral terms. Time lag is generally given in minutes, while all other data are in meters and seconds. With these assumptions, Equation 6.16 becomes

$$A(1,t) = \frac{A_o}{\beta W_D} \int_0^1 \sqrt{\frac{1}{2\pi p \sqrt{1-x} \sqrt{t}}} \exp \left(p \sqrt{1-x} \sqrt{t} + \frac{x}{\beta W_D} - \frac{1}{\beta W_D} - \frac{60ct}{\beta W_D}\right) dx + \frac{120(1+\alpha) A_o Cl}{\beta^2 W_D^2} \int_0^t \sqrt{\frac{1}{2\pi p \sqrt{l(t-\tau)}}} \exp \left(p \sqrt{l} \sqrt{t-\tau} + \frac{60c\tau}{\beta W_D} - \frac{60ct}{\beta W_D} - \frac{1}{\beta W_D}\right) d\tau \qquad \ldots 6.17$$

in which $p = \sqrt{240c/\beta^2 W_D^2}$. Equation 6.17 poses no computational problems.

The Fortran IVG program "PD" computes A(1,t) for given values of Q_o, α, β, W_D, 1, a_A, and b_A using Equation 5.10a to convert discharge to area and vice versa and Equation 6.6 to compute c. On output, the program lists several parts of (6.17), to permit an assessment of the contribution of the two terms on the right. The program, with operating instructions, and sample output, is listed in the Appendix.

During the period of rapidly changing A(1,t), both terms of (6.17) are of similar magnitude. The first term dominates before that, when $A(1,t) \sim A_o$, and the second term dominates the period when $A(1,t) \sim (1+\alpha) A_o$. Away from t = 1/c, the dominant terms become time independent. Close to t = 0, the solution fails as a result of substituting an asymptotic function for the two Bessel functions.



EFFECT OF β ON THE SOLUTION OF THE KINEMATIC WAVE EQUATION WITH STORAGE DISPERSION

Fig. 21

6.2.4 Comparison with Field Data

To evaluate the probable range of β, which is the only free parameter in Equation 6.17, a few surge tests were compared with computed Q(1,t) curves covering a wide range of β. Figure 21 shows a typical comparison. Obviously the computed c of that test is somewhat too small as pointed out in Section 6.1.2. The best fitting values of β fall consistently between 0.5 and 1, with the greater values occurring at the larger discharges. Fit was determined by inspection. A least squares fit of Equation 6.17 to evaluate optimal values of β, or β and c is feasible, using the programs "NLIN2" (Section 3.3.1) and "PD", but it would involve excessive computer time.

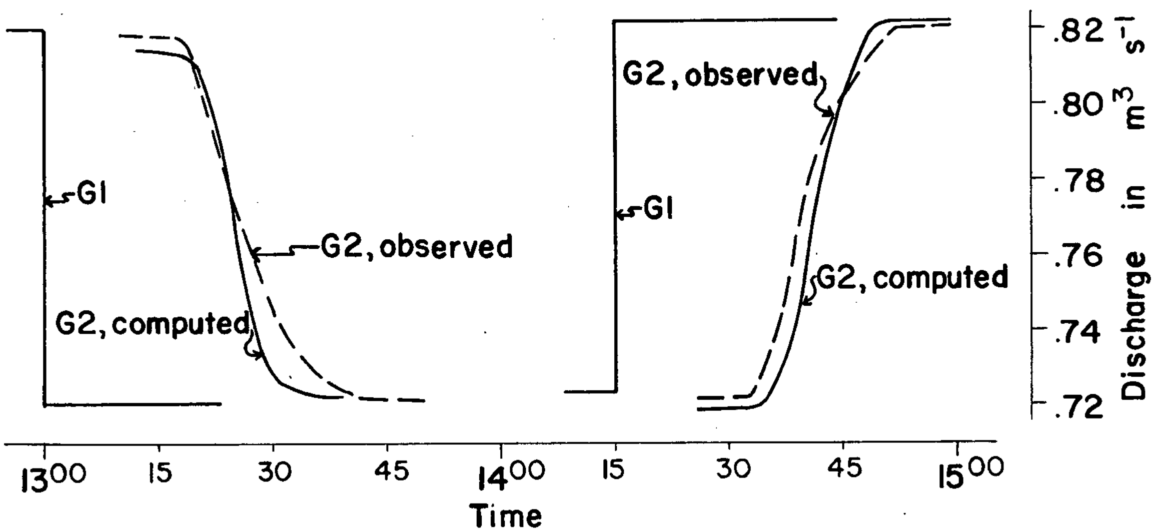
As noted in Section 5.2.1 the actual water surface width of natural channels, W_s, is generally related to W_D by functions of the form

$$W_s = W_D \left(\frac{Q}{Q_D}\right)^z \qquad \ldots\ 6.18$$

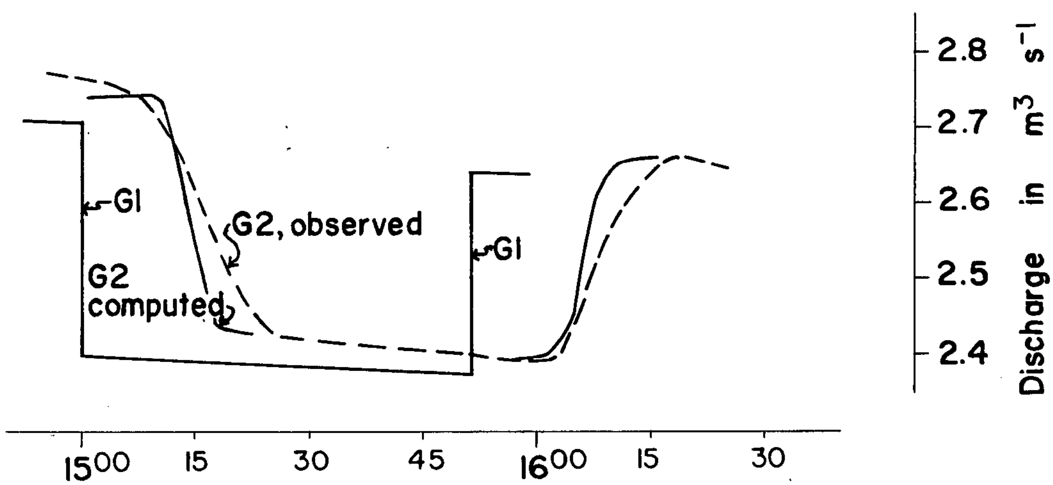
with the factor z probably falling into the range 0.1 to 0.2 in the case of tumbling flow channels (Miller, 1958). This suggests that W_s might be a good estimate of βW_D or

$$\beta = \left(\frac{Q}{Q_D}\right)^z \qquad \ldots\ 6.19$$

Figures 22a and 22b show a few typical results obtained on the basis of Equations 6.17 and 6.19, with z taken as 0.2 and Q_D taken as the estimated mean annual peak flow (Table 2).



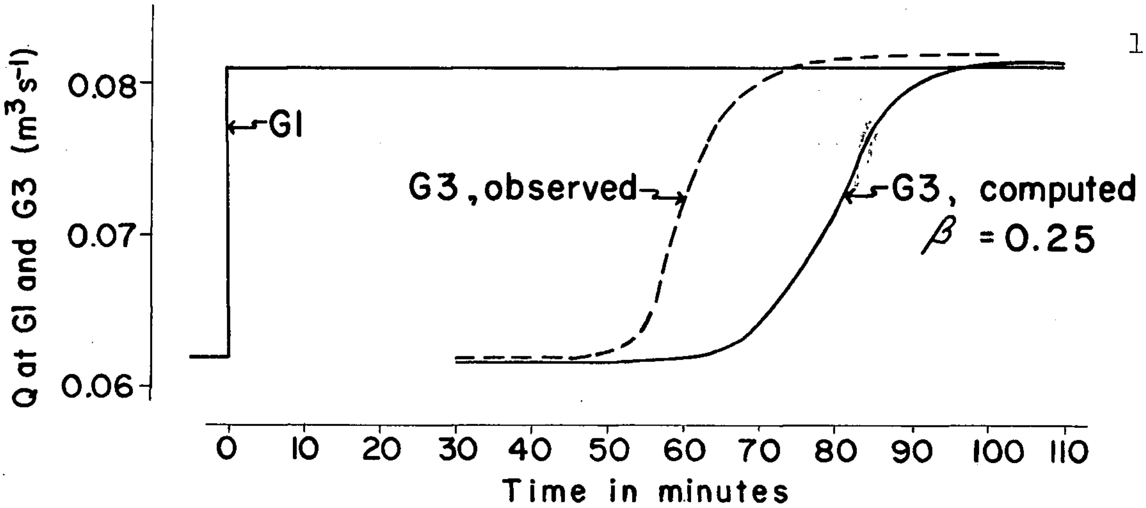
Phyllis Creek, June 22, 1968. $\beta = 0.54$



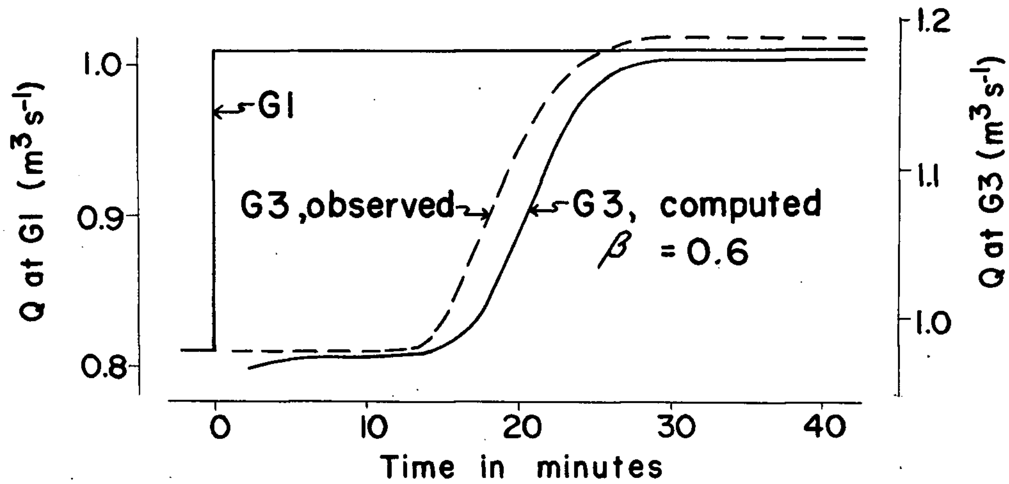
Phyllis Creek, Sept. 17, 1968. $\beta = 0.69$

COMPARISON BETWEEN FIELD OBSERVATIONS AND KINEMATIC WAVES WITH STORAGE DISPERSION

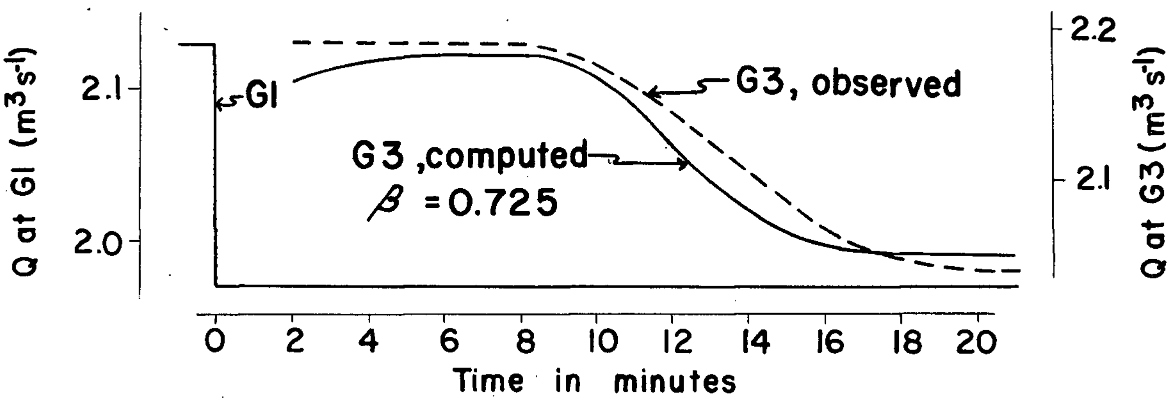
Fig. 22a



Blaney Creek, Upsurge G1-G3, June 9, 1967



Blaney Creek, Upsurge G1-G3, Nov. 30, 1968



Blaney Creek, Downsurge G1-G3, March 5, 1968

COMPARISON BETWEEN FIELD OBSERVATIONS AND KINEMATIC WAVES WITH STORAGE DISPERSION

Fig.22b

Only reaches immediately below lakes can be used for comparison, because the surges on lower reaches do not fit the initial conditions, as stated in (6.12) and (6.13). The writer knows of no other routing method which could give comparable fit without having to evaluate some free parameters from other unsteady flow data beforehand.

Equation 6.17 is not a practical routing equation for routine hydrological work. It is a means of obtaining the dispersion coefficient of the basic wave equation (6.9) if circumstances permit the creation of a small, step-like surge. To obtain an operational flow forecasting system, Equation 6.9 would have to be considered non-linear and programmed for numerical solution, possibly using the methods discussed by Lighthill and Whitham (1955) or Henderson (1966).

6.3 A Practical Approach to Unsteady, Tumbling Flow

As an alternative to the routing method of the last section, which considers the tumbling flow channel as a large sequence of storage elements with unique Q-A relations at their outlets, it appears worth investigating whether a sequence of a few reservoirs and channels could represent tumbling flow. The basic steady flow Equation 5.10 can be satisfied physically either by an inclined rough channel with the appropriate roughness elements or by a smooth, almost horizontal reservoir-like channel with a weir-like outlet. The following 3 sections will explore the consequences of assuming that a channel reach in the tumbling regime can be represented by a relatively small number of alternating reservoirs and channels, both meeting the steady flow equation

$$A = a_A Q^{b_A} \qquad \text{... 5.10}$$

6.3.1 Unsteady Flow through a Non-linear Reservoir

Unsteady flow through a prismatic reservoir of length λ and area A has to satisfy the continuity relation

$$Q_u(t) - Q(t) = \frac{dA}{dt} \qquad \text{... 6.20}$$

in which $Q_u(t)$ is the inflow, and $Q(t)$ the outflow. Evaluating the derivative dA/dt with the dimensionally homogeneous form of the steady flow equation 5.10b and representing all discharges as fractions of Q_D ($Q = q\,Q_D$), leads to

$$\frac{dq}{dt} = \frac{Q_D}{\lambda\, A_D\, b_A} \left(q_u\, q^{1-b_A} - q^{2-b_A} \right) \qquad \text{... 6.21}$$

which is a separable but non-linear differential equation. The introduction of Q_D into (6.21) is purely for ease in converting the formulas to other systems of units; it does not limit Q to values less than Q_D. To obtain an explicit solution for a constant q_u, one can write (6.21) as follows

$$\frac{Q_D}{\lambda A_D b_A} \int dt = \frac{q^{b_A - 1}}{q_u - q} + \text{constant}$$

Substituting $(q_u - q)/q = k$ gives

$$\frac{Q_D t}{\lambda A_D b_A} = -q_u^{\,b_A - 1} \int \frac{dk}{k(1+k)^{-b_A}} + \text{constant}$$

The integral on the right has explicit solutions for rational values of b_A ($b_A = i/j$), but the form of the solution can vary widely, depending on i and j (Edwards, 1921). This approach is therefore not suitable for applications in which b_A may take a fairly wide range of values.

In general it is most efficient to solve Equation 6.21 by a standard numerical method, such as Runge-Kutta.

6.3.2 A Routing Model Based on a Cascade of Channels and Pools

If a tumbling flow channel is represented as a cascade of kinematic channels and non-linear reservoirs, as shown in Figure 23, Equation 6.21 becomes

$$\frac{dq_i(t)}{dt} = \frac{nQ_D}{\sigma\, l\, A_D\, b_A} \left(q_{i-1}(t-T_L)\, q_i(t)^{1-b_A} - q_i(t)^{2-b_A}\right) \quad \ldots \; 6.21a$$

The time lag between pools, T_L, is evaluated by neglecting the time lag between a pool and the following channel, by assuming that the channels are kinematic flow systems, and by assuming further that the pool water-surface is horizontal at all times.[1] With this, T_L becomes

---

[1]This is identical with the assumption of an infinite dynamic wave celerity in the pools. The wave celerities quoted in Section 6.1.1 would appear to justify this.

$$T_L = \frac{(1 - \sigma)\ l}{n\ c_i(t)}$$

and with a substitution for c_i according to Equation 6.6

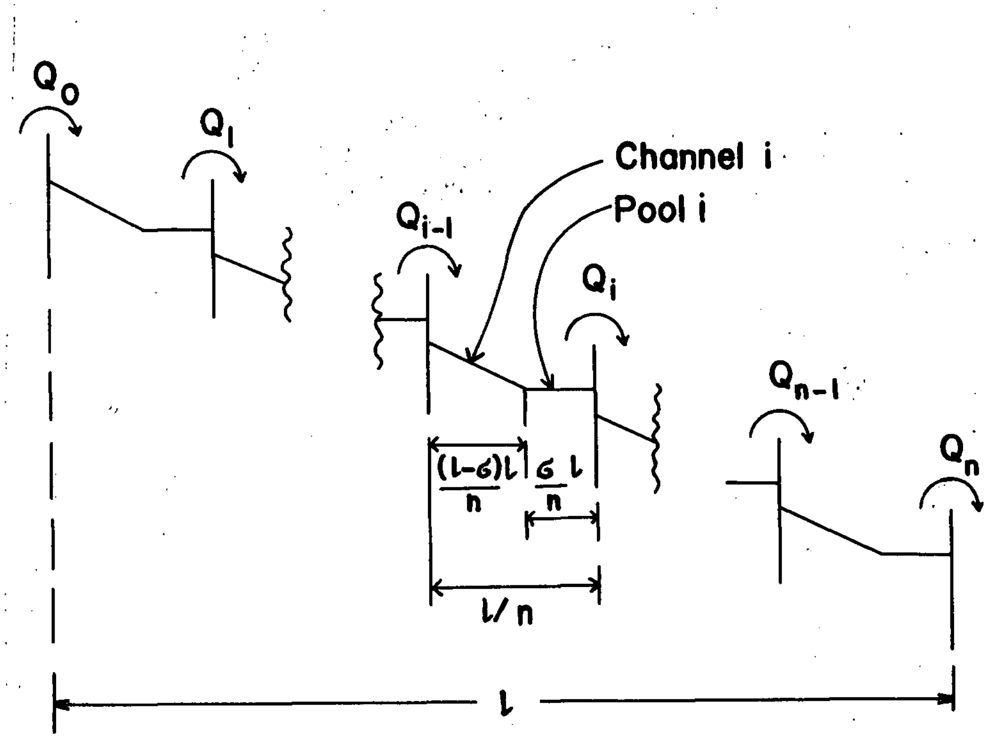


FIGURE 23. DEFINITION SKETCH FOR THE CASCADE OF CHANNELS AND RESERVOIRS

$$T_L = \frac{(1 - \sigma)\ l\ A_D^{\ b_A}}{n\ Q_D\ q^{\ 1 - b_A}} \qquad \ldots\ 6.22$$

The two equations (6.21a) and (6.22) define a channel routing system suitable for numerical evaluation. A Fortran IV G program "SNLR"[2] written for this purpose is listed in the Appendix.

[2] In "SNLR" the parameter σ is called α.

Suitable assumptions for the free parameters n and σ will be discussed in the following section. However, considerations of computing economics and stability of the computations impose fairly narrow limits on both parameters.

Stability is assured as long as the following two conditions are met.

$$R_i \gg Q_i \Delta t \quad \text{... 6.23}$$

$$T_{L(j+1)} < T_{L(j)} + \Delta t \quad \text{... 6.24}$$

in which Δt is the finite time step in the numerical integration of (6.21a), and the subscript j refers to these time steps. The exact formulation of Inequality 6.23 depends on the integrating method, but by requiring either large reservoirs or small time intervals, it certainly narrows the range of possible σ and n values. Violation of Inequality 6.24 would indicate a tendency towards the formation of a bore. This may be a rather remote possibility, since even the largest surges of this study do not come close to violating the inequality.

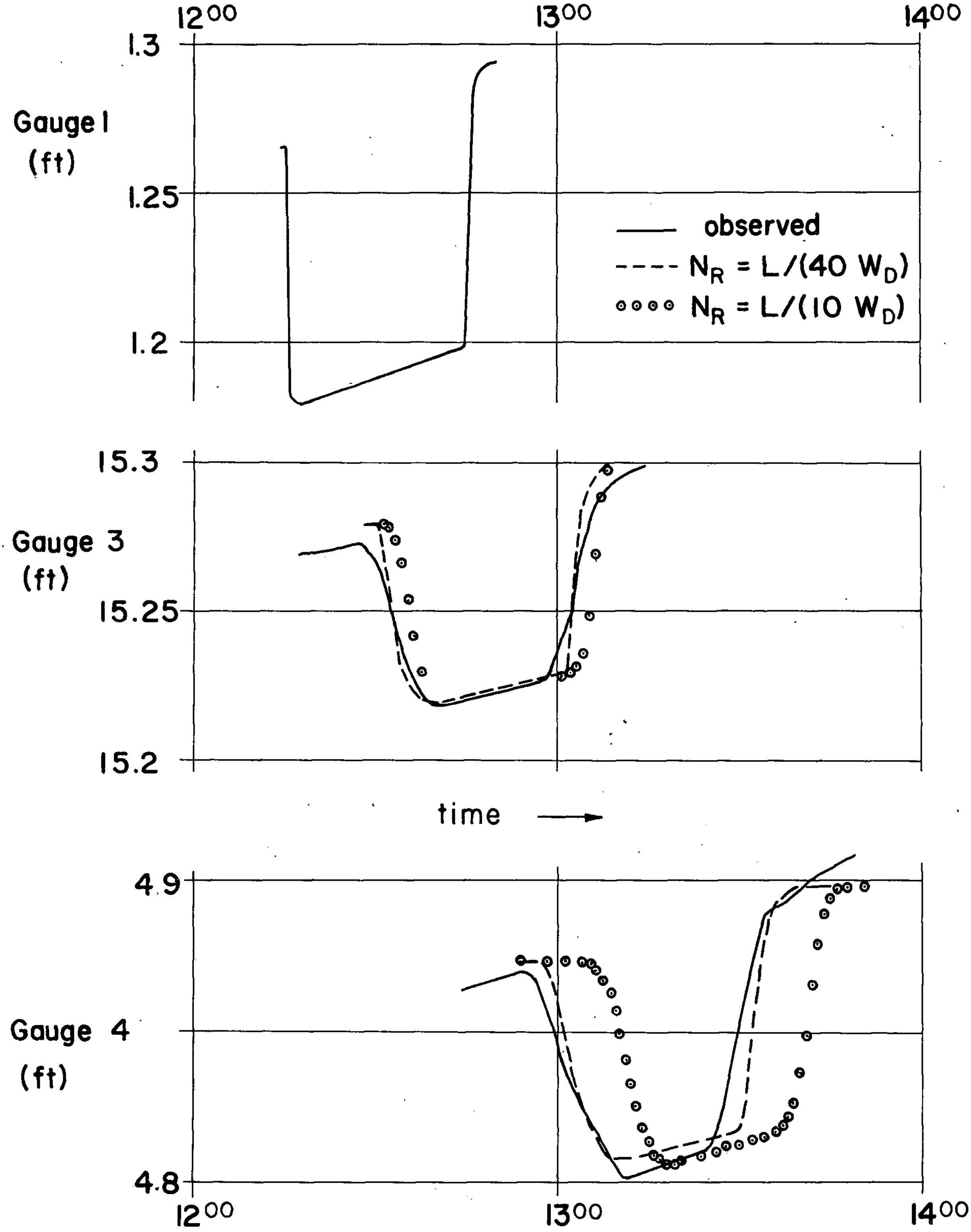
6.3.3 Evaluation of the Free Parameters from Field Data

Figure 23 shows that the parameter n is a scale parameter, which should express whether a reach is relatively "long" or "short". With the ratio l/W_D being the most reasonable and practical measure of relative length, n will be assumed a priori to be a function of $1/W_D$ alone. It is important that n be independent of Q because it would be difficult to change n in the course of a computation.

The parameter σ indicates how much of a given channel reach is acting like a reservoir. Since pools are prominent at low flows and tend to disappear during floods, it seems reasonable to expect σ to be an decreasing function of Q/Q_D. From a practical point of view this is a feasible assumption. The boundary between a channel segment and the adjoining reservoir can be shifted during a surge computation as they have identical A-Q relations.

The combined effect of Inequality 6.23 and the above assumption on the relation between σ and Q is somewhat unfortunate, since (6.23) indicates a need for larger reservoirs with increasing flow while the proposed decrease of σ with Q has the opposite effect. The two conditions can be met simultaneously by decreasing the time step of the numerical integration as Q increases.

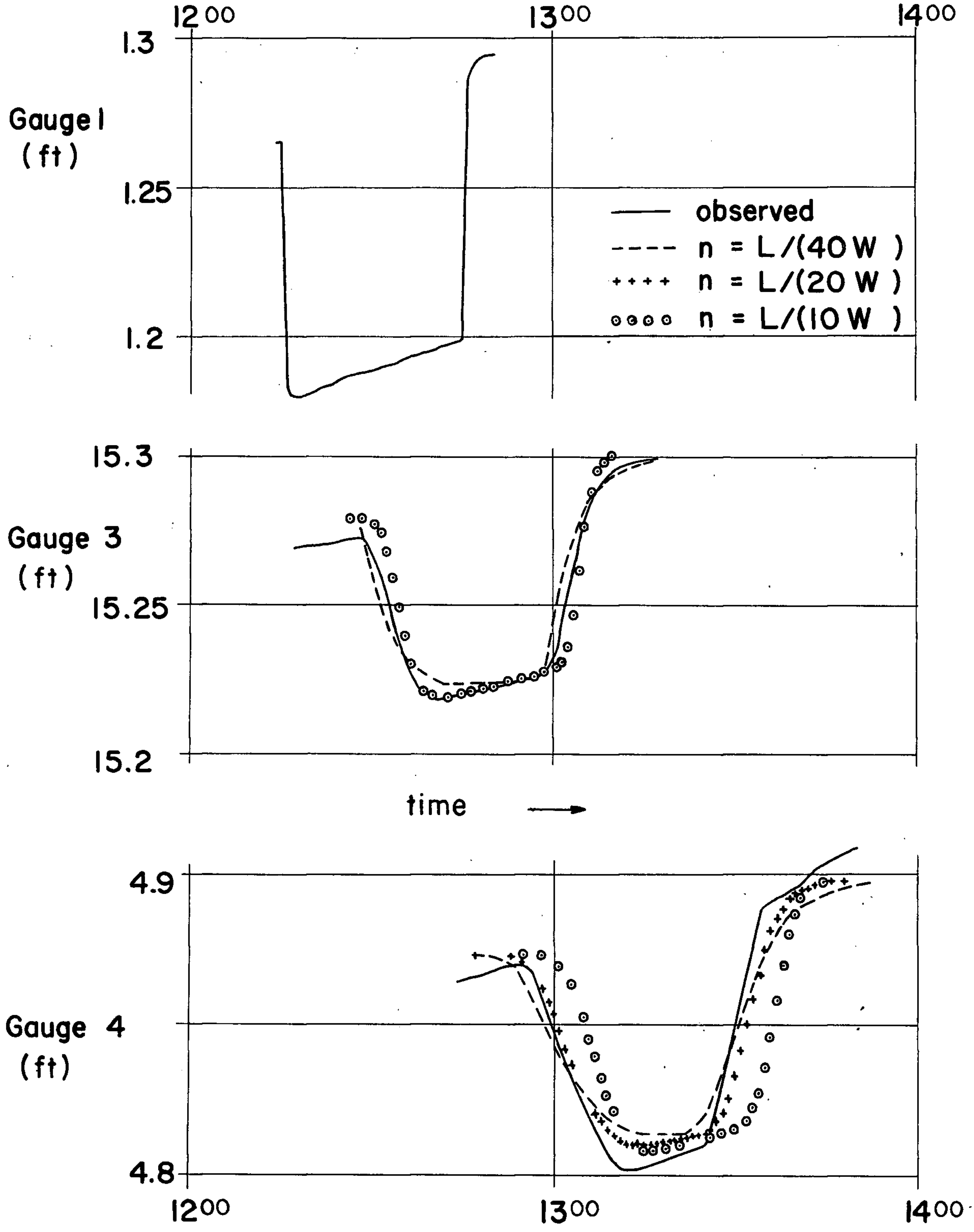
To gain a clearer picture of the effects of changes in n and in σ on the computed downstream flow, the surge test shown in Figure 15 was routed through the three Blaney Creek reaches with various assumed combinations of n and σ. The results are shown on Figures 24 a, b, and c. They indicate that good fit can be achieved with a wide range of n-σ combinations. If the reach is divided into many



Blaney Creek, Oct. 13, 1968. Q approx. 1 $m^3\ s^{-1}$

VARIABLE NUMBER OF RESERVOIRS AT $\sigma = 0.1$

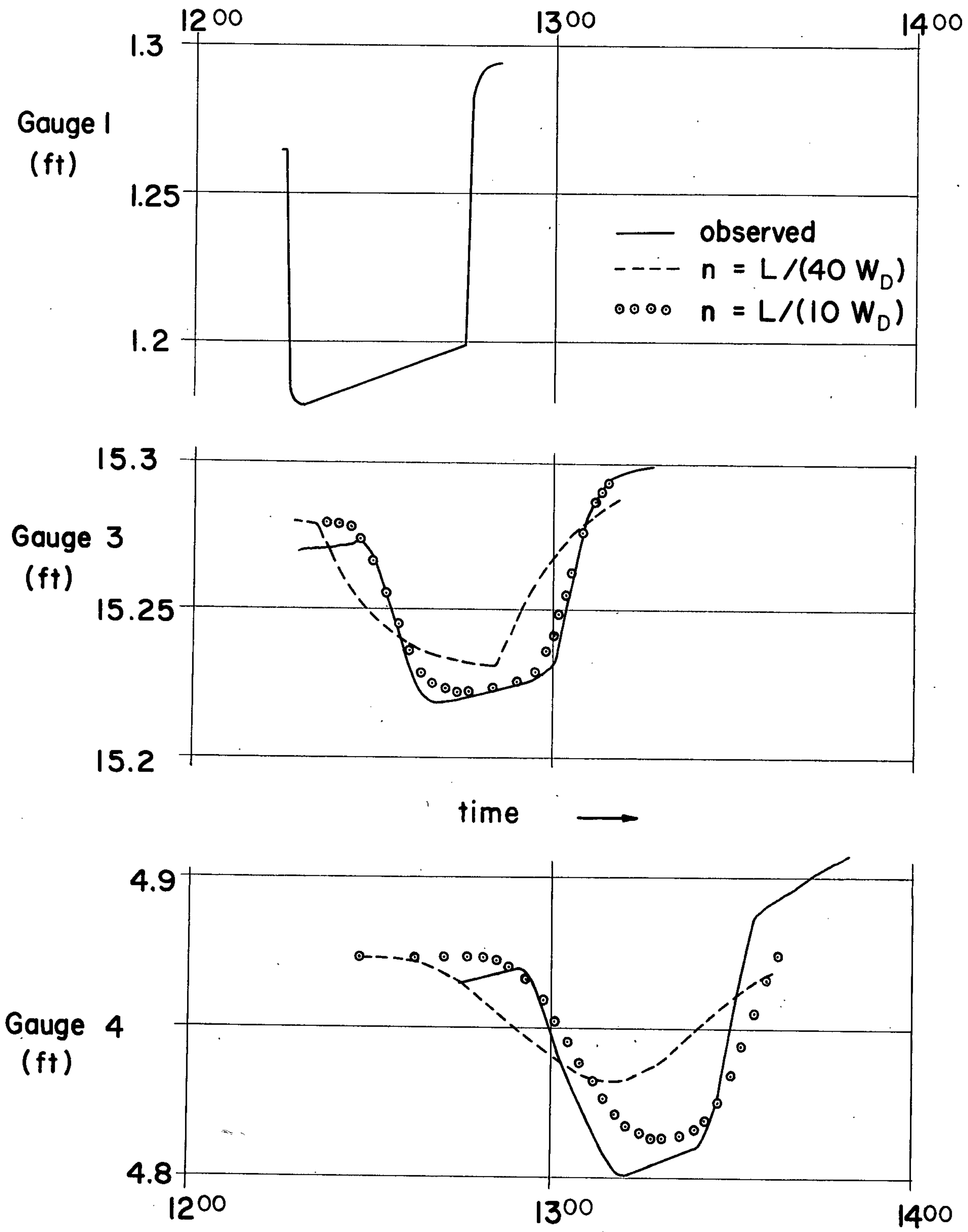
Fig. 24a



Blaney Creek, Oct. 13, 1968. Q approx. $1 m^3 s^{-1}$

VARIABLE NUMBER OF RESERVOIRS AT σ = 0.28

Fig. 24b



Blaney Creek, Oct. 13, 1968. Q approx. $1 \, m^3 \, s^{-1}$

VARIABLE NUMBER OF RESERVOIRS AT $\sigma = 0.7$

Fig. 24c

reservoirs (n large) they have to cover a large part of the length (σ large) and vice versa. Since computing time increases more than linearly with n due to (6.23) and the increase in computing time with decreasing σ is relatively small, it is obviously advantageous to keep n as small as possible. The surge test data give no clear indication of a lower limit. The practical solution appears to be to select a "$1/W_D$" criterion for n that results in a single reservoir for the shortest reaches of interest, which will depend on the scale at which one is working. As can be seen from Table 2

$$n = 1/40W_D \qquad \ldots\ 6.25$$

is a suitable assumption for the present set of data.

The second parameter, σ, was estimated by trial and error, using all the surge tests of Blaney Creek. The resulting σ - q relation is naturally only valid in combination with (6.25). As in the case of the kinematic waves with storage dispersion, the free parameter σ could again be obtained with a non-linear least squares fit (Section 6.2.4), but the amount of computing time required would be even greater here. A simultaneous fit of n and σ would probably not lead to significant results, since the minimum of the squared residuals appears to be an elongated valley, lying more or less in a diagonal direction across the σ-n plane.

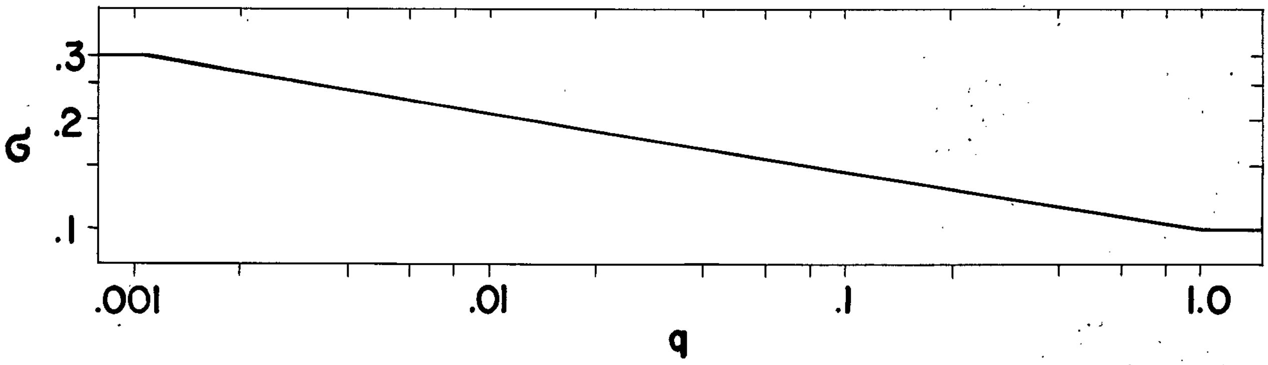
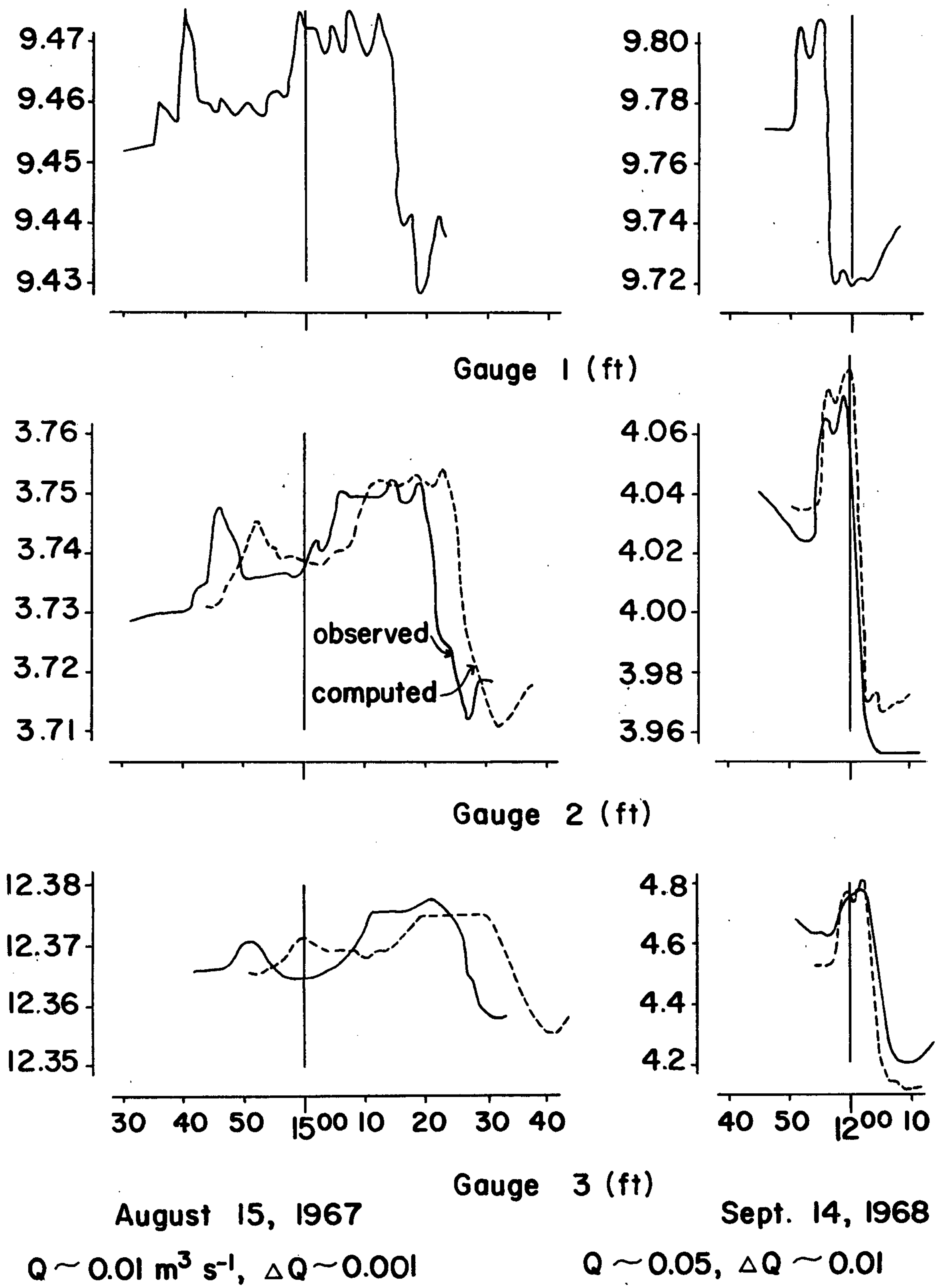


FIGURE 25. THE ROUTING PARAMETER σ FOR $n = 1/40W_D$

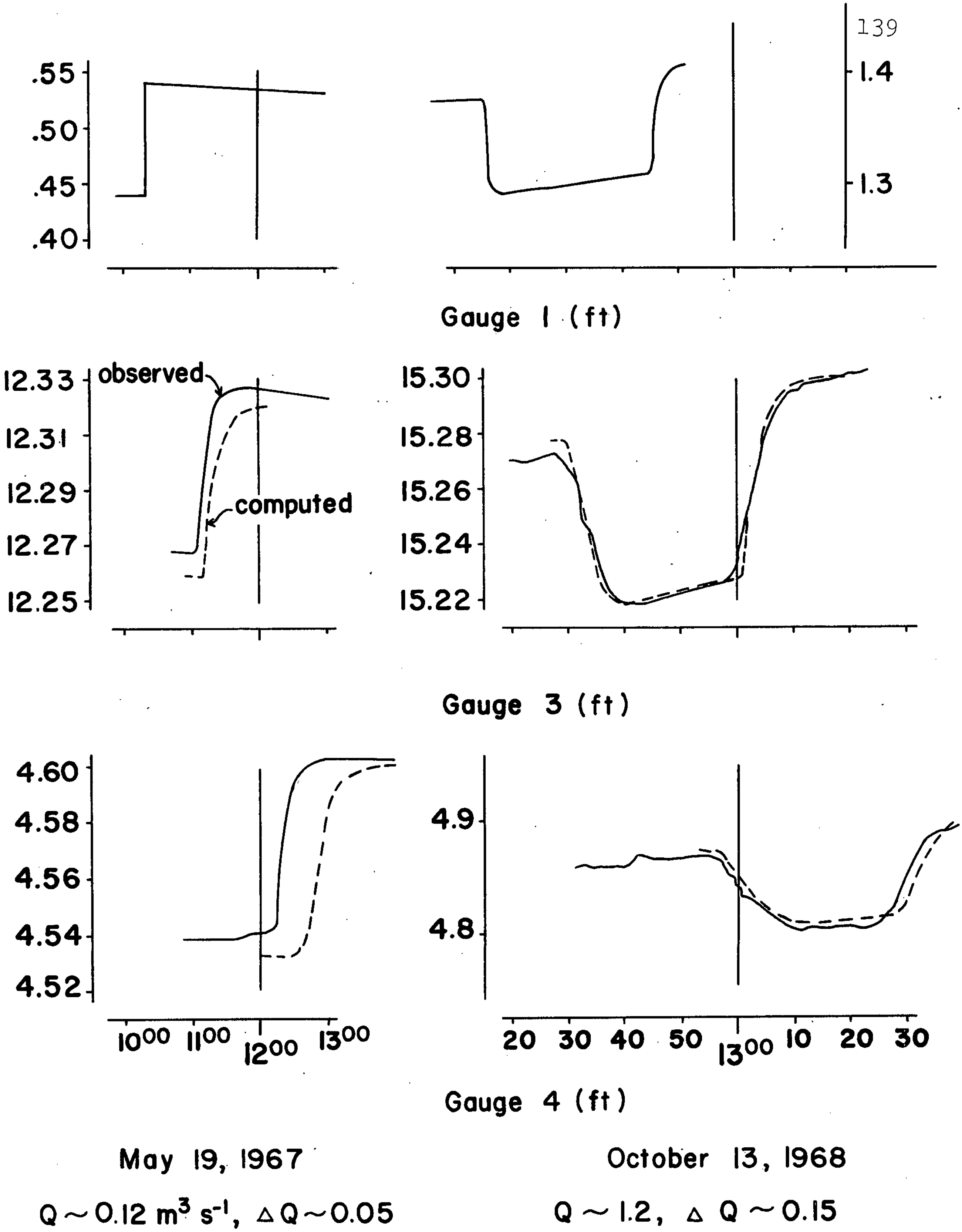
Figures 26 a, b, and c show several typical comparisons between computed and observed surges for Blaney Creek and for other streams, which were not used for deriving the σ-q relation of Figure 25.

The program "SNLR" converts discharge to stage for direct comparison of observed and computed H - T curves. Since several of the gauge rating curves are not too well defined, some of the differences between observed and computed surges are the result of this, rather than of any deficiency in the routing method. The closeness of fit is comparable to the result obtained with Equation 6.17. At very low stage the surge celerity is again underestimated. With an increase in σ it is possible to achieve a correct surge arrival time, but this leads to excessive damping. As pointed out in Section 6.1.2, this effect is probably the result of neglecting dynamic waves. There is, however, some slight evidence, particularly in the case of Brockton Creek (Figure 16),



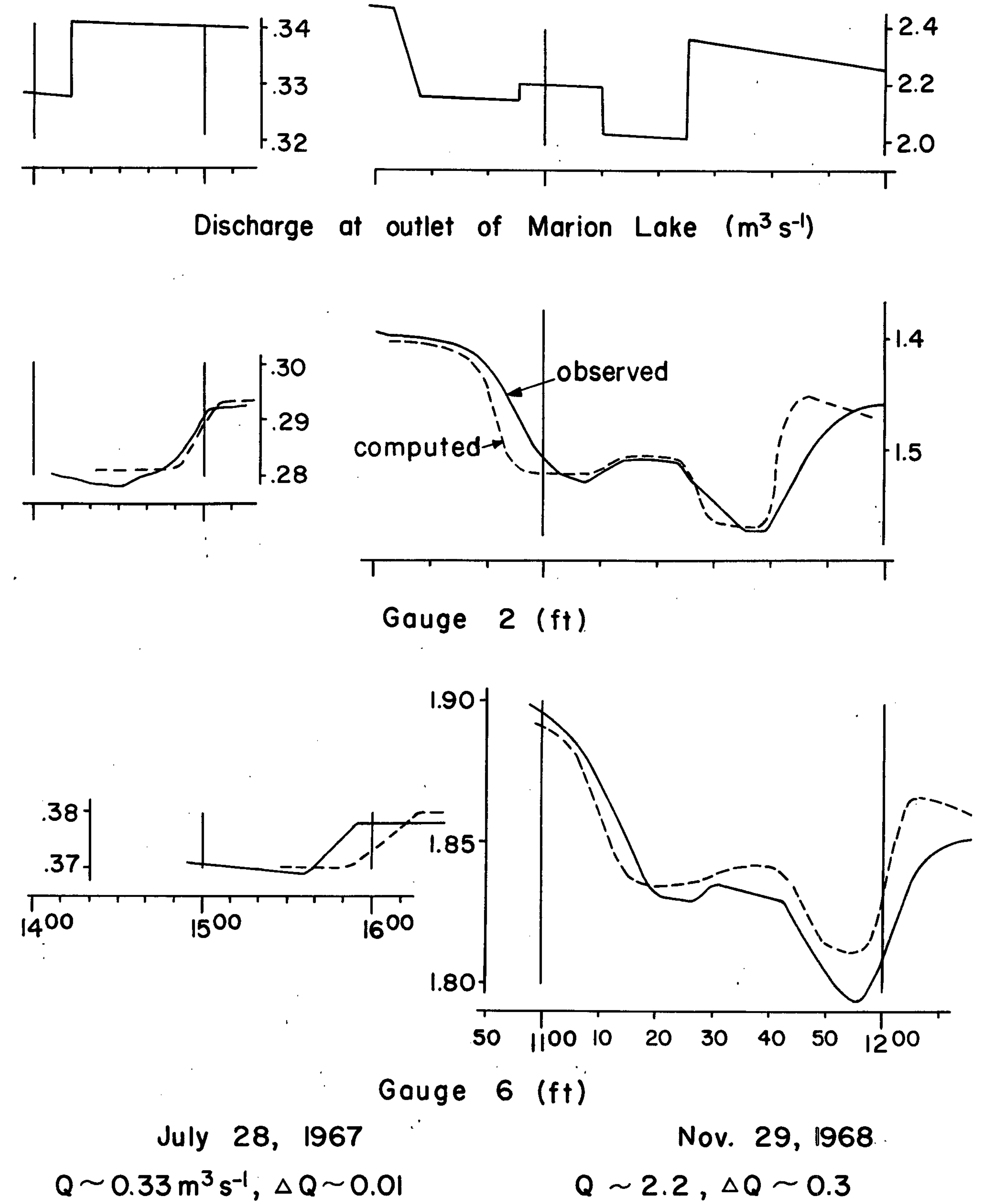
COMPUTED AND OBSERVED SURGES, BROCKTON CREEK

Fig. 26a



COMPUTED AND OBSERVED SURGES, BLANEY CREEK

Fig. 26b



COMPUTED AND OBSERVED SURGES, PHYLLIS CREEK

Fig 26c

that the true Q - A relation is somewhat curved on double log paper, probably due to a residual area at zero flow, and this would result in higher values for the kinematic celerity c at small discharges.

The major difference between the routing method based on channels and reservoirs and routing based on Equation 6.17 is that the former makes no assumption about the shape of the surge input to the reach and proceeds with finite time steps, which, in the case of a runoff model, permits the addition of local inflow.

7 CONCLUSIONS

7.1 The Hydraulics of Tumbling Flow

The objectives of this investigation are stated in the Introduction as: (i) finding the laws that govern steady and unsteady tumbling flow, and (ii) expressing these laws in terms of parameters which can be related to readily available basin data. Within certain limitations, to be discussed below, this has been accomplished.

The aspects of steady channel flow, which are significant in a runoff model, are completely described by the functions $A = f(Q)$ for all segments of the channel network. The field data indicate that these functions can be approximated by simple exponentials of the form $A = a_A Q^{b_A}$. On the basis of similarity considerations it is shown that the two parameters a_A and b_A depend on basin parameters which can be considered "readily available". This is confirmed by the highly significant correlations appearing in Day's (1969) regression models. The physics of the channel forming process, which is implicit in Day's regression equations, remains unknown.

The two proposed methods for extending the steady flow equations to unsteady flow routing are shown to be capable of reproducing the downstream propagation of step-like surges. Since more complicated channel inflows can be approximated by a series of steps, both methods are capable of routing all hydrograph shapes except in those cases where the non-linearity of the channel response leads to the formation of a bore.

The data impose a number of limitations on the conclusions. None of the test reaches is a first order channel and only three reaches represent small streams; Brockton 1-2 and 2-3, and Placid 1-2. Their steady flow behaviour differs markedly because Brockton Creek lies at tree-line and is essentially debris free while Placid Creek is severely choked by logging slash. The results of the study are therefore not suitable for application to almost complete channel networks as they appear on large scale maps such as the 1:2400 map used in Figure 2. The channel networks appearing on 1:50,000 NTS maps represent the approximate lower limit to which the results of this study may be applied. The first order and most second order channels have to be included in the land phase.

The conclusions are further limited by the lack of unsteady flow tests on the larger streams. The flow regime in large streams is rarely "tumbling" over long reaches and it is questionable whether the unsteady flow models, particularly the rules for determining the parameters β and σ, apply to ordinary, rough turbulent channel flow.

There is a twofold regional limitation on the data:

(i) The relations between channel parameters and drainage area depend on climatic factors. This can be overcome by establishing relations between DA and W_D, and then predicting the channel performance from W_D and S. This requires some field work and may therefore not always be feasible.

(ii) Climate and elevation are the main factors determining vegetation and can therefore have a considerable effect on the performance of the smaller streams. It follows that logging and clearing operations are of similar importance.

7.2 Basin Linearity

Both routing models developed in Section 6 show that the non-linear response of a channel segment is largely a consequence of the dependency between c and Q, which, for the kinematic approximation,is positive exponential since $c = dQ/dA \propto v_m$ and $v_m = a_v Q^{b_v}$. The surge tests do however indicate a consistent tendency towards c values larger than dQ/dA (Figures 26 a, b and c) at low flows, which may be interpreted as a trend towards linear response of the channel system at low flows.

That the high flows show no tendency towards linearity is not surprising since the only reason which is generally advanced for such a trend, the rapid increase in flow area as the stream channels overflow onto the flood plain, is rarely applicable in steep mountainous basins.

One may even argue that, since the bed material of degrading tumbling flow channels moves only under extreme flood conditions and since a mobile bed would probably offer lower resistance to the flowing water (Kellerhals, 1967), there could be stronger non-linear trends during extreme runoff events.

7.3 Towards an Operative Channel Runoff Model

Neither of the two routing methods of Section 6 represents an operative channel runoff model, but both can be considered as building blocks out of which a channel runoff model can now be assembled. Since the writer plans to pursue this after completion of the present study, a brief note on what remains to be done may be in order.

(i) If the routing method based on Equation 6.9 is to be used, a considerable programming problem remains to be solved. One can either adopt the present solution for step-like input (6.17) to unspecified input shapes or one might choose a purely numerical solution based on successive approximations to Q (x,t) in a finite grid on the x - t planes of all channel segments.

(ii) The routing method based on non-linear reservoirs is closer to being operational. The gradual change in σ with Q remains to be incorporated in the program "SNLR".

(iii) A mathematical formulation for the drainage network will have to be devised to permit proper representation of the channel parameters in a computer. The model recently proposed by Surkan (1969) appears to be adaptable to this purpose.

(iv) A suitable representation of the land-phase input to the channel system will have to be found. Hydrographs of a few small source areas may be acceptable, possibly in combination with meteorological records.

(v) With increasing basin size, the channel phase becomes dominant over the land phase. It is important, therefore, to extend the routing methods to larger streams than those considered in the present study.

It will probably become necessary to obtain the basic A(Q)-equation from extensive river surveys rather than with tracer methods. Dispersion coefficients and wave celerities will have to be established through controlled releases from several major dams.

Only after completion of all this will it be possible to pass a final judgement on the usefulness of the two-phase approach to runoff. However, the results presented here leave little room for doubt that it will be positive.

8. BIBLIOGRAPHY

Aastad, J. and Sögnen, R. 1954. "Discharge Measurements by Means of a Salt Solution, 'the Relative Salt Dilution Method'," International Association of Scientific Hydrology. Publication No. 38. Assemblé Internationale d'Hydrologie de Rome. Tome III.

Abdelsalam, W. W., 1965. Flume Study of the Effect of Concentration and Size of Roughness Elements on Flow in High Gradient Natural Channels. Ph.D. Thesis, Utah State University.

Ackers, Peter, 1958. "Resistance of Fluids Flowing in Channels and Pipes," Department of Scientific and Industrial Research, Hydr. Research Station, Hydr. Research Paper No. 1, London.

Al Khafaji, Abbas Nasser, 1961. The Dynamics of Two-Dimensional Flow in Steep Natural Streams. Ph.D. Thesis, Utah State University.

Amorocho, J., and Hart, W. E. 1964. "A Critique of Current Methods in Hydrologic Systems Investigations," American Geophysical Union, Transactions, Vol. 45, No. 2, pp. 307-321.

Argyropoulos, Praxitelis A., 1965. "High Velocity Flow in Irregular Natural Streams," Inter. Assoc. for Hydraulic Research, 11th Meeting, Leningrad, Paper 1-26.

Barr, David I. H. 1968. "Discriminating Formulation of n-term Non-Dimensional Functional Equations from (n+1)-term Dimensionally Homogeneous Equations with Particular Reference to Incompressible Viscous Flow," Institution of Civil Engineers, Proc., Vol. 39, pp. 305-312.

Barr, David I. H., and Herbertson, John G. 1968. "Similitude Theory Applied to Correlation of Flume Sediment Transport Data," Water Resources Research, Vol. 4, No. 2, p. 307.

Church, Michael. 1967. "Observations of Turbulent Diffusion in a Natural Channel," Canadian Journal of Earth Sciences, Vol. 14.

Church, M. and Kellerhals, R. 1969. "Stream Gauging in Isolated Areas Using Portable Equipment," Technical Bulletin No. , Inland Waters Branch, Canada Dept. of Energy, Mines, and Resources, (in press).

Day, T., 1969. The Channel Geometry of Mountain Streams. M. A. Thesis, University of British Columbia (in press).

Diachishin, A. N., 1963. "Dye Dispersion Studies," Journal of the Sanitary Engineering Division, American Society of Civil Engineers, Proceedings. Vol. 89, No. SA1, pp. 29-49.

Diskin, M. H., 1967. "A Dispersion Analog Model for Watershed Systems," Proceedings of the International Hydrology Symposium. Fort Collins. Vol. 1.

Edwards, Joseph, 1921. A Treatise on the Integral Calculus. Macmillan and Co., London. Vol. 1.

Elder, J. W., 1959. "The Dispersion of Marked Fluid in Turbulent Shear Flow," Journal of Fluid Mechanics, Vol. 5.

Fischer, H. B., 1966. "Longitudinal Dispersion in Laboratory and Natural Streams," Report No. KH-12-12, W. M. Kech Hydrologic Laboratory, California Institute of Technology, Pasadena, Cal.

Hartung, Fritz, and Scheuerlein, Helmut, 1967. "Macroturbulent Flow in Steep Open Channels with High Natural Roughness," Proceedings of the International Hydrology Symposium, Fort Collins. Vol. 1.

Hayami, Shoitiro, 1951. "On the Propagation of Flood Waves," Disaster Prevention Research Institute, Kyoto University, Bulletins, No. 1.

Hays, J. R., 1966. Mass Transport Mechanism in Open Channel Flow. Ph.D. Thesis, Vanderbilt University.

Henderson, F. M., 1966. Open Channel Flow. Macmillan Co., New York.

Herbich, John B., and Shulits, Sam, Nov. 1964. "Large-Scale Roughness in Open Channel Flow," Journal of the Hydraulic Division, ASCE, Vol. 90, No.HY6, pp. 203-236.

Herrio, John G., 1963. Methods of Mathematical Analysis and Computation. John Wiley & Sons, Inc. New York.

Jahnke, Eugene, and Emde, Fritz, 1945. Tables of Functions. Dover Publications, New York.

Johnson, Martin L., 1964. "Channel Roughness in Steep Mountain Streams," American Geophysical Union, Ann. Meeting.

Judd, Harl E., 1963. A Study of Bed Characteristics in Relation to Flow in Rough, High-gradient, Natural Channels. Ph.D. Thesis, Utah State University.

Kellerhals, Rolf, 1967. "Stable Channels With Gravel-Paved Beds," Journal of Waterways and Harbors Division, ASCE, Vol. 93, No. WW1, Proc. Paper 5091. pp. 63-84.

Larson, C. L., 1965. "A Two-Phase Approach to the Prediction of Peak Rates and Frequencies of Runoff for Small Ungauged Watersheds," Technical Report No. 53, Dept. of Civil Engineers, Stanford.

Leopold, L. B., Wolman, M. Gordon, and Miller, John P., 1963. Fluvial Processes in Geomorphology, W. H. Freeman and Co., San Francisco and London.

Leopold, L. B., Bagnold, R. A., and Wolman, M. Gordon, and Bush, L. M., 1960. "Flow Resistance in Sinuous or Irregular Channels," United States Geological Survey, Professional Paper No. 282-D.

Lighthill, M. J. and Whitham, G. B., 1955. On Kinematic Waves: I. Flood Movement in Long Rivers, Proc. Royal Soc. of London (A), Vol. 229, pp. 281-316.

Marquardt, D. W., 1964. "Least Squares Estimation of Non-Linear Parameters," A Computer Program in Fortran IV Language, IBM Share Library, Distribution No. 3094.

Massé, Pierre, 1935. Hydrodynamique fluviale, régimes variables. Hermann et Cie, Paris.

Mikhlin, S. G., 1967. Linear Equations of Mathematical Physics. Holt, Rhinehart, and Winston, Inc.

Miller, J. P., 1958. "High Mountain Streams: Effects of Geology on Channel Characteristics and Bed Material," New Mexico, State Bureau of Mines and Mineral Resources, Memoir No. 4.

Mirajgaoker, A. G., and Charlu, K.L.N., Sept. 1963. "Natural Roughness Effects in Rigid Open Channels," Journal of the Hydraulics Division, ASCE, Vol. 89, No. HY5, Proc. Paper 3630. pp. 29-44.

Morisawa, M. 1957. "Accuracy of Determination of Stream Lengths from Topographic Maps," American Geophysical Union, Transactions, Vol. 38, No. 1.

Østrem, G., 1964. "A Method of Measuring Water Discharge in Turbulent Streams," Geographical Bulletin No. 21, Geographical Branch, Dept. of Mines and Technical Surveys, Ottawa.

Overton, D. E., 1967. "Analytical Simulation of Watershed Hydrographs from Rainfall," Proceedings of the International Hydrology Symposium, Fort Collins, Vol. 1.

Peterson, D. F. and Mohanty, P. K., 1960. "Flume Studies in Steep, Rough Channels," Journal of the Hydraulics Division, Proceedings of the American Society of Civil Engineers, Vol. 86, No. HY9, pp. 55-76.

Pilgrim, D. H., 1966. "Radioactive Tracing of Storm Runoff on a Small Catchment," Journal of Hydrology. Vol. 4, pp. 289-326.

Replogle, J. A. et al., 1966. "Flow Measurements with Fluorescent Tracers," Journal of the Hydraulics Division, American Society of Civil Engineers, Vol. 92, No. HY5, Proc. Paper 4895, pp. 1-15.

Scheidegger, A. E., 1966. "Effect of Map Scale on Stream Orders," Bulletin of the International Association of Scientific Hydrology, Vol. 11, No. 3.

Scheuerlein, Helmut, 1968. "Der Rauhgerinneabfluss," Bericht No. 14 der Versuchsanstalt fur Wasserbau der Technischen Hochschule München, Oskar V. Miller Institut.

Seddon, J. A., 1900. "River Hydraulics," Transactions, American Society of Civil Engineers, Vol. 43, pp. 179-243.

Stewart, John H., and LaMarche, Valmore, C., 1967. "Erosion and Deposition Produced by the Flood of December 1964 on Coffee Creek, Trinity County, California," United States Geological Survey, Professional Paper, No. 422-K.

Sugaware, M., 1967. "Runoff Analysis and Water Balance Analysis By a Series Storage Type Model," Proceedings of the International Hydrology Symposium, Fort Collins, Vol. 1.

Surkan, A. J. 1969. "Synthetic Hydrographs: Effects of Network Geometry." Water Resources Research, Vol. 5, No. 1, pp. 112-128.

Taylor, G. I., 1954. "The Dispersion of Matter in Turbulent Flow Through a Pipe," Proceedings of the Royal Society of London, Vol. 223.

Thackston, E. L. et al., 1967. "Least Squares Estimation of Mixing Coefficients," Journal of the Sanitary Engineering Division, American Society of Civil Engineers, Proceedings, Vol. 93, No. SA 3, pp. 47-58.

Thackston, E. L. and Krenkel, P. A., 1967. "Longitudinal Mixing in Natural Streams," Journal of the Sanitary Engineering Division, American Society of Civil Engineers, Vol. 93, No. SA 5, pp. 67-90.

Wilson, J. F., 1968. "Fluorometric Procedures for Dye Tracing," in Techniques of Water Resources Investigations of the U. S. Geological Survey, Chapter A 12, Book 3.

Yalin, M. S., Jan. 1966. "A Theoretical Study of Stable Alluvial Systems," Golden Jubilee Symposium, Central Water and Power Research Station, Poona.

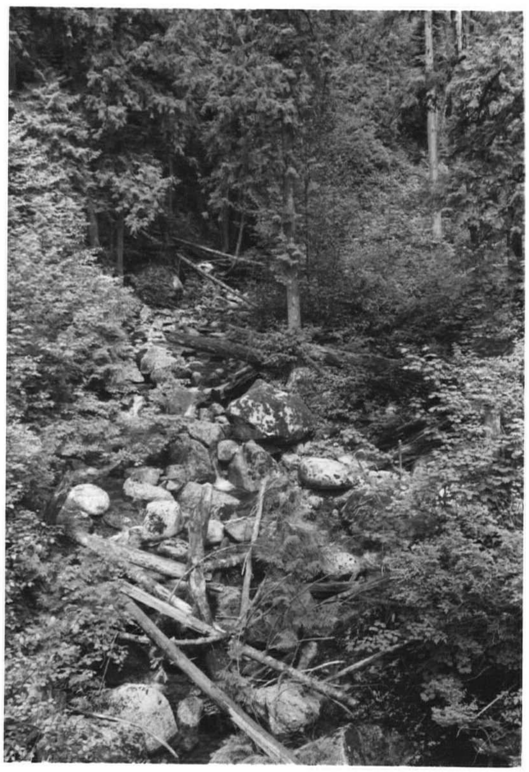
PHOTOGRAPHS

PHOTOGRAPH 1.
BROCKTON CREEK, ALONG REACH Br 1 - 2, LOOKING UPSTREAM.

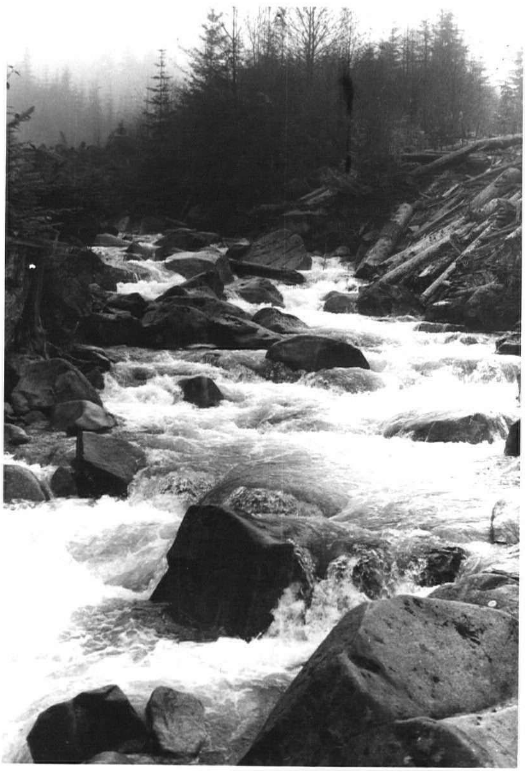
PHOTOGRAPH 2.
PLACID CREEK, ALONG REACH Pl 3 - 4, LOOKING DOWNSTREAM.
TYPICAL LOG JAM IN FOREGROUND.

PHOTOGRAPH 3.
BLANEY CREEK, AT B1 GAUGE 3, LOOKING DOWNSTREAM.

PHOTOGRAPH 4.
BLANEY CREEK, AT B1 GAUGE 4, LOOKING UPSTREAM FROM BRIDGE.

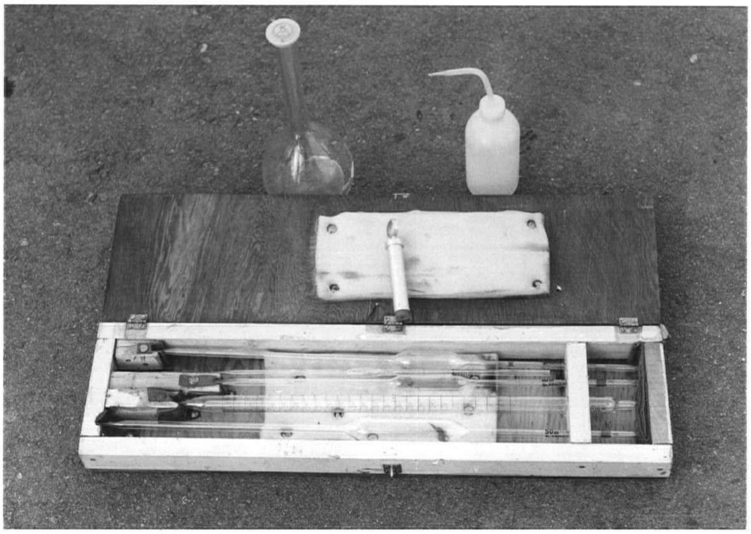


PHOTOGRAPH 5.
PHYLLIS CREEK, AT PH GAUGE 2, LOOKING DOWNSTREAM.
STAGE RECORDER AT RIGHT.

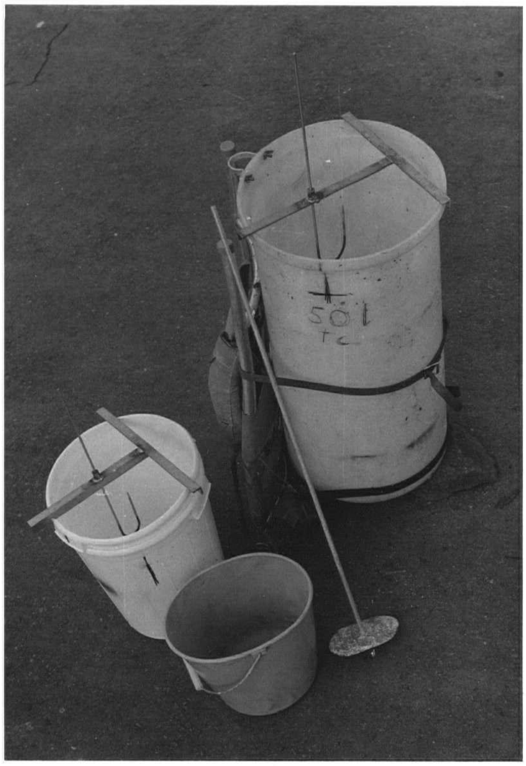


PHOTOGRAPH 6.
PHYLLIS CREEK, AT PH GAUGE 4, LOOKING UPSTREAM.

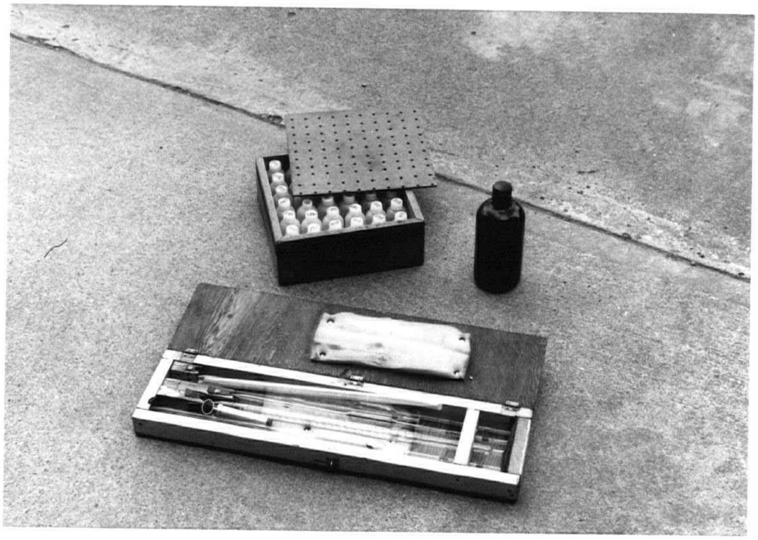
PHOTOGRAPH 7.
BARNSTEAD CONDUCTIVITY BRIDGE.



PHOTOGRAPH 8.
VOLUMETRIC GLASS WARE FOR SALT DILUTION TESTS.

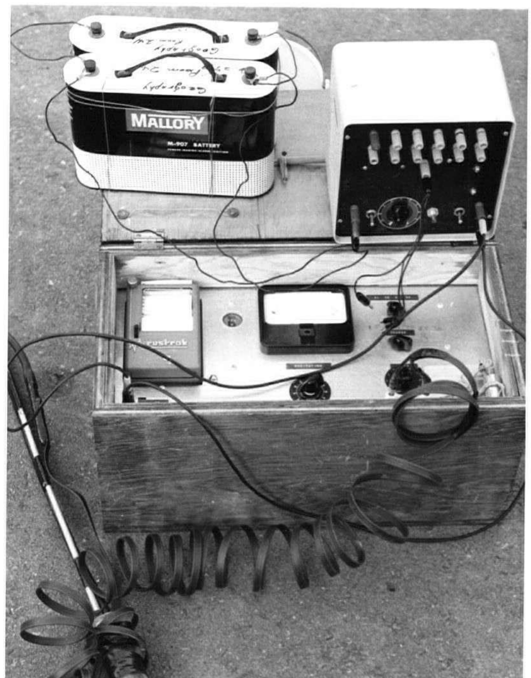


PHOTOGRAPH 9.
VATS, PAIL, AND STIRRING
ROD FOR SALT DILUTION TESTS.



PHOTOGRAPH 10

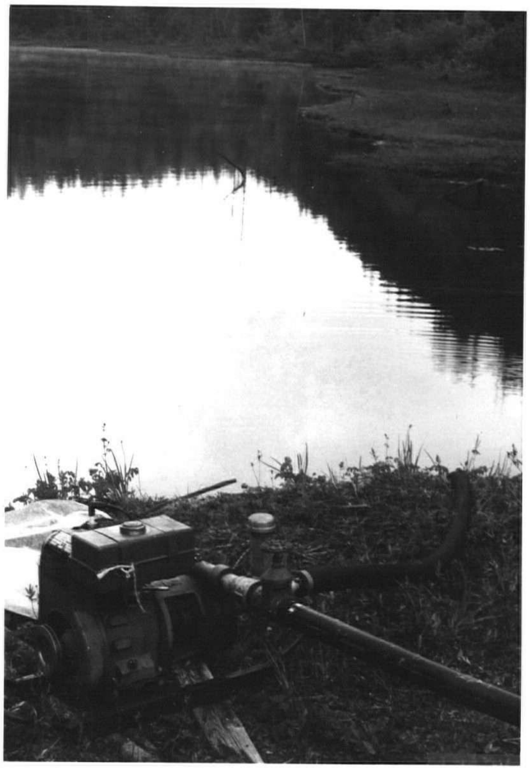
EQUIPMENT FOR RHODAMINE WT SLUG INJECTION TEST



PHOTOGRAPH 11.
RECORDING CONDUCTIVITY
BRIDGE, WITH ELECTRONIC
INTERVAL TIMER.

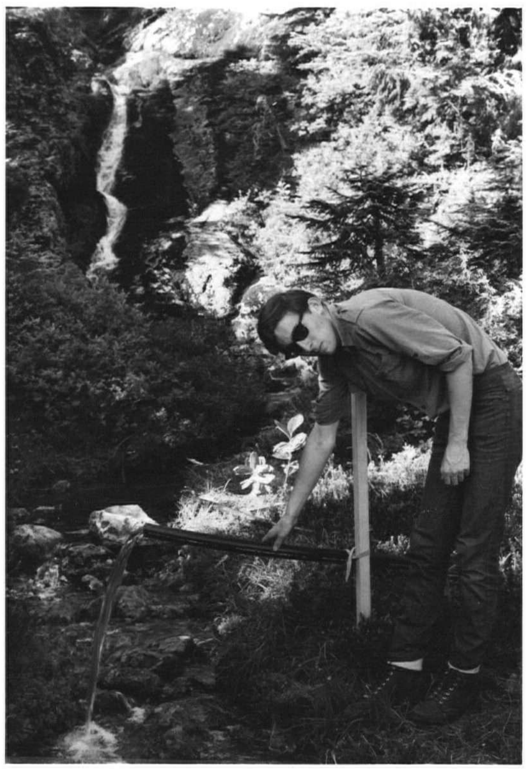
PHOTOGRAPH 12.
CONTROL STRUCTURE AT THE OUTLET OF BLANEY LAKE.
THREE FLASHBOARDS IN PLACE.

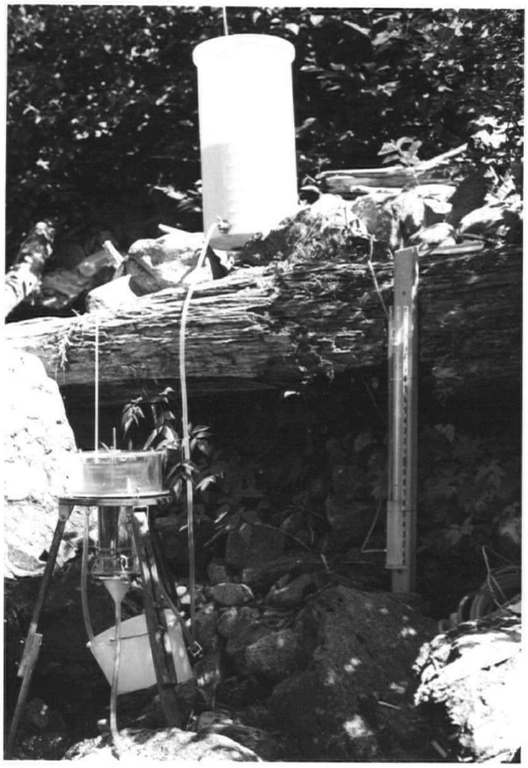
PHOTOGRAPH 13.
TIMBER CRIB DAM AT OUTLET OF MARION LAKE, WITH
TWO ADDITIONS IN PLACE FOR A DOWN-SURGE.



PHOTOGRAPH 14.
PUMP AT PLACID LAKE.

PHOTOGRAPH 15.
INVERTED SYPHON AT POOL OUTLET ABOVE BROCKTON GAUGE 1.





PHOTOGRAPH 16.
PLEXIGLASS TUBE FOR STAGE READINGS ON RIGHT, CONSTANT RATE INJECTION APPARATUS ON LEFT.

PHOTOGRAPH 17.
RECORDER INSTALLATION WITH INVERTED SYPHON AT BLANEY GAUGE 5.

APPENDIX

COMPUTER PROGRAMS WITH OPERATING INDSTRUCTIONS, PRINTOUT, AND PLOTS

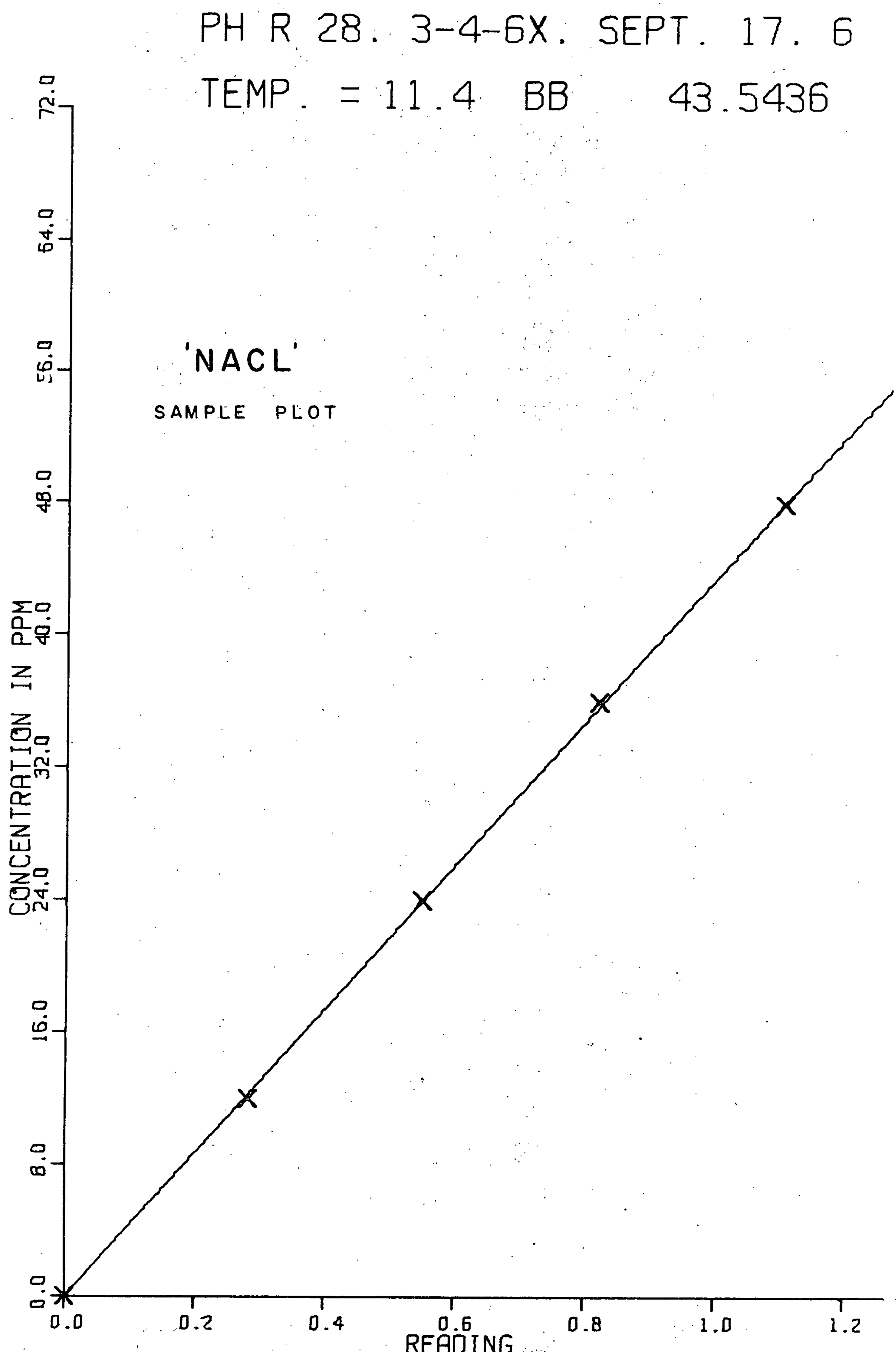
LIST OF CONTENTS

NACL

```
C
C       FORTRAN /360 MAIN PROGRAM CALLED  NACL , WHICH CONVERTS
C       TIME-CONDUCTIVITY DATA TO TIME-CONCENTRATION DATA. TIME IN
C       MINUTES AND SECONDS IS CONVERTED TO MINUTES AND DECIMALS.
C
C       INPUT
C          CONTROL CARDS
C          1 ONE CARD PER RUN,
C            NO. OF DATA SETS, KTOT, (I2)
C          2 ONE PER DATA SET.
C            NO OF DATA POINTS, K, (I3)
C            ARRIVAL TIME OF TRACER WAVE, TST, (F7.2)
C            TITLE OR RUN IDENRIFICATION NO. (7A4)
C            PARAMETERS OF GAMMA EXTENSION, IF DESIRED,
C            LOG A, B, D,(2X,3E10.5). LOG A IS CONVERTED TO A.
C          3 ONE PER SET OF DATA.
C            NO. OF POINTS ON THE RATING CURVE, N, (I3)
C            DILUTION RESULTING FROM 10 CC OF SECONDARY SOLUTION
C            IN RATING TANK, DIL10, (F10.0)
C            RATING TANK TEMPERATURE, TEMP, (F10.0)
C            BACKGROUD READING AT START OF TEST, BACKST, (F10.0)
C            BACKGROUND READING AT END OF TEST OR BLANCK IF IT IS
C            EQUAL TO BACKST, BACKND, (F10.0)
C            PARAMETER NPLO, WHICH SHOULD BE .GT. 0, IF NO PLOT
C            DESIRED, (I3)
C            PARAMETER NPUNCH, TO BE SET .GT. 0, IF NO PUNCHED
C            OUTPUT DESIRED, (I3)
C       DATA CARDS OF STREAM MEASUREMENTS, K CARDS PER DATA SET.
C            CONDUCTIVITY READINGS YC,TIME IN MINUTES AND SECONDS
C            FROM INJECTION, XT, (2F9.3)
C       DATA CARDS OF RATING CURVE, N CARDS PER SET.
C            AMOUNT OF SECONDARY SOLUTION IN RATING TANK IN CC, CC,
C            CONDUCTIVITY READING, READ, (2F9.3)
C
C       OUTPUT
C          PRINTOUT OF INPUT DATA
C          PRINTOUT OF CONVERTED DATA, WITH CONCENTRATION IN PPM
C          OF THE PRIMARY SOLUTION.
C          OPTIONAL,
C          PUNCHED DATA CARDSCONTAINING CONCENTRATION YC, AND
C          TIME XT, AS REQUIRED FOR PROGRAM DQV, (2F9.3),
C          PLOT OF RATING CURVE.
C
        CALL PLOTS
        DIMENSION  XT (200) , YC (200), TITLE (7) , CC(20) ,READ (20)
 1      FORMAT (I3, F7.2,7A4,2X,3E10.5 )
 2      FORMAT (2F 9.3)
 3      FORMAT (I3, 4F10.0,2I3 )
 5      FORMAT( 8HORATING ,4X, 4HSTEP, 9X ,2HCC,  8X, 7HREADING ,/
       1(13X , I2 , 7X, F6.0 , 4 X , F9.4 ) )
 7      FORMAT (I2)
 16     FORMAT (18H1CONTROL CARD 1 =  /1X, I3, 2X, F7.2, 7A4,
       1 2X,  3E12.6  / 18HOCONTROL CARD 2 =   / 1X, I2, 2X, 4F12.2,
       2  3X, I2, 3X, I2   )
 19     FORMAT ( 22HOCONVERTED FINAL DATA     )
```

```
32    FORMAT(5HODATA ,  4X,2HNO,8X,2HXT , 7X , 7HREADING , /
     1( 9X , I2 ,5X, F7.2 , 2X ,F10.4 ))
C
C     LOOP FOR SETS
      READ (5,7) KTOT
      DO  8  KSET  = 1, KTOT
C
C     READ DATA
      READ(5,1) K , TST, TITLE, A, B, D
      IF ( A .NE. 0.0 ) A =  EXP (A)
      READ(5,3) N , DIL 10 , TEMP , BACKST, BACKND, NPLOT, NPUNCH
      IF ( BACKND .LE. 0.0 ) BACKND = BACKST
      READ (5,2)  (YC (I), XT (I), I =1,K)
      READ (5,2)  (CC (I), READ(I),I =1,N)
C
C     PRINT DATA
      WRITE (6,16) K, TST, TITLE ,A,B,D,  N, DIL10, TEMP,
     1  BACKST, BACKND, NPLOT,  NPUNCH
      WRITE (6,32) (I , XT(I) , YC (I) , I = 1, K )
      WRITE (6, 5) (I , CC(I) , READ(I), I = 1, N )
C
C     CONVERT SECONDS TO MINUTES
      DO 9  I =1 ,   K
      IXT   =XT(I)
      TMIN  =IXT
9     XT(I) =TMIN + (XT(I)-TMIN) / 0.6
C
C     CONCENTRATION RATING CURVE
C
C     CONVERT  CC(I)  TO CONCENTRATION  IN PPM
      RI = READ(1)
      DO 10 I = 1 , N
      READ (I) = READ(I) - RI
10    CC(I)    =(CC(I) /(10.0 * DIL 10))* 10.0E+5
C
C     COMPUTE B OF REGRESSION LINE
      SYX   = 0.0
      SXX   = 0.0
      DO 12 I = 1, N
      SYX   = CC(I) * READ(I) + SYX
12    SXX   =READ(I)**2 + SXX
      BB    =   SYX / SXX
C                                                              TION
C     ADJUST YC(I) TO A  ZERO BACKGROUND AND CONVERT TO CONCENTRA-
      DBACK =(BACKST  - BACKND )/ (XT(K) -TST)
      DO 11 I = 1, K
      IF (XT(I) .LE. TST )  YC(I) =0.0                          *DBACK
      IF (XT(I) .GT. TST )  YC(I) = YC(I) - BACKST +((XT(I)-TST)
11    YC(I)    =  BB* YC(I)
      WRITE (6, 19 )
      WRITE (6,32) (I , XT(I) , YC (I) , I = 1, K )
      IF ( NPLOT .NE. 0  ) GO TO 17
C
C     PLOT  RATING CURVE
      CCMAX   =BB* READ(N)
```

```
      CALL SCALE ( READ ,N, 7.0 ,RXMIN ,RDX, 1)
      CALL SCALE ( CC   ,N, 9.0 ,CYMIN ,CDY, 1)
      CCMAX    = (CCMAX - CYMIN) / CDY
      CALL AXIS ( 0.0 , 0.0, 7HREADING ,-7, 7.0 , 0.0, RXMIN,RDX )
      CALL AXIS ( 0.0 , 0.0,20HCONCENTRATION IN PPM, +20,9.0, 90.0,
     1 CYMIN, CDY)
      CALL SYMBOL ( 1.0, 9.5 , 0.21 ,TITLE , 0.0 , 28)
      CALL SYMBOL ( 1.0, 9.0 , 0.21, 7HTEMP. =,0.0, 7)
      CALL NUMBER ( 2.4, 9.0 , 0.21, TEMP ,0.0, 1 )
      CALL SYMBOL ( 3.5, 9.0 , 0.21, 4HBB =, 0.0 , 3 )
      CALL NUMBER ( 4.6, 9.0 , 0.21 ,BB , 0.0 , 4 )
      DO 13  I = 1 , N
 13   CALL SYMBOL ( READ(I) , CC(I) ,0.14 , 4 , 0.0 , -1 )
      CALL PLOT (0.0 , 0.0 ,+3 )
      CALL PLOT (READ(N) , CCMAX , +2 )
      CALL PLOT ( 9.0 , 0.0 , -3  )
 17   CONTINUE
C
C     PUNCH DATA CARDS
      IF ( NPUNCH . NE. 0 ) GO TO 18
      WRITE (7,1) K , TST, TITLE, A , B, D
      WRITE ( 7,2) (YC(I), XT(I), I= 1,K )
 18   CONTINUE
C
C     CALLS TO FITTING AND PLOTTING ROUTINES
C     THE CALL CARDS FOR NUMERICAL INTEGRATION OF THE C-T CURVE AND
C     FOR FITTING A GAMMA-DISTRIBUTION EXTENSION GO IN HERE.
      CALL TAILEX  (K, TST, XT, YC, TITLE )
 8    CONTINUE
C
      CALL PLOTND
      STOP
      END
```


PH R 28. 3-4-6X. SEPT. 17. 6
TEMP. = 11.4 BB 43.5436
'NACL'
SAMPLE PLOT
CONCENTRATION IN PPM
0.0
8.0
16.0
24.0
32.0
40.0
48.0
56.0
64.0
72.0
READING
0.0
0.2
0.4
0.6
0.8
1.0
1.2

CONTROL CARD 1 =
63 15.00PH R 28, 3-4-6X, SEPT. 17, 6 0.0 0.0

CONTROL CARD 2 =
6 83333.25 11.40 1.62 1.62 10 10

DATA	NO	XT	READING
	1	14.30	1.6150
	2	15.30	1.6190
	3	16.00	1.6240
	4	16.30	1.6360
	5	17.00	1.6610
	6	17.30	1.7090
	7	18.10	1.7900
	8	18.30	1.8640
	9	19.00	1.9800
	10	19.30	2.0610
	11	20.00	2.1610
	12	20.35	2.2510
	13	21.00	2.3000
	14	21.30	2.3290
	15	22.00	2.3340
	16	22.30	2.3210
	17	23.00	2.2900
	18	23.30	2.2550
	19	24.00	2.2020
	20	24.30	2.1440
	21	25.00	2.0930
	22	25.30	2.0390
	23	26.05	1.9970
	24	26.30	1.9640
	25	27.00	1.9250
	26	27.30	1.8840
	27	28.30	1.8210
	28	29.00	1.7900
	29	29.30	1.7730
	30	30.00	1.7540
	31	30.30	1.7420
	32	31.00	1.7270
	33	31.30	1.7150
	34	32.00	1.7060
	35	32.30	1.6950
	36	33.00	1.6880
	37	33.35	1.6800
	38	34.00	1.6760
	39	34.30	1.6730
	40	35.00	1.6690
	41	35.30	1.6650
	42	36.00	1.6610
	43	36.40	1.6590
	44	37.00	1.6580
	45	37.30	1.6560
	46	38.05	1.6530
	47	38.30	1.6510
	48	39.10	1.6490
	49	39.30	1.6480
	50	40.00	1.6470
	51	41.00	1.6450
	52	42.00	1.6420
	53	43.00	1.6400

'NACL'

SAMPLE PRINTOUT

54	44.00	1.6390
55	45.00	1.6380
56	46.00	1.6370
57	47.00	1.6360
58	48.00	1.6350
59	49.00	1.6340
60	51.00	1.6320
61	53.00	1.6300
62	55.00	1.6300
63	60.15	1.6280

RATING	STEP	CC	READING
	1	0.	1.5970
	2	10.	1.8790
	3	20.	2.1460
	4	30.	2.4170
	5	40.	2.7000
	6	50.	2.9780

CONVERTED FINAL DATA

DATA	NO	XT	READING
	1	14.50	0.0
	2	15.50	0.0402
	3	16.00	0.2545
	4	16.50	0.7737
	5	17.00	1.8589
	6	17.50	3.9456
	7	18.17	7.4681
	8	18.50	10.6881
	9	19.00	15.7357
	10	19.50	19.2594
	11	20.00	23.6103
	12	20.58	27.5252
	13	21.00	29.6560
	14	21.50	30.9154
	15	22.00	31.1298
	16	22.50	30.5603
	17	23.00	29.2072
	18	23.50	27.6797
	19	24.00	25.3686
	20	24.50	22.8397
	21	25.00	20.6157
	22	25.50	18.2610
	23	26.08	16.4282
	24	26.50	14.9885
	25	27.00	13.2869
	26	27.50	11.4983
	27	28.50	8.7483
	28	29.00	7.3952
	29	29.50	6.6515
	30	30.00	5.8208
	31	30.50	5.2950
	32	31.00	4.6384
	33	31.50	4.1125
	34	32.00	3.7173
	35	32.50	3.2350
	36	33.00	2.9268
	37	33.58	2.5745
	38	34.00	2.3975

39	34.50	2.2635
40	35.00	2.0860
41	35.50	1.9085
42	36.00	1.7309
43	36.67	1.6393
44	37.00	1.5936
45	37.50	1.5031
46	38.08	1.3685
47	38.50	1.2786
48	39.17	1.1870
49	39.50	1.1413
50	40.00	1.0943
51	41.00	1.0005
52	42.00	0.8631
53	43.00	0.7693
54	44.00	0.7191
55	45.00	0.6688
56	46.00	0.6185
57	47.00	0.5682
58	48.00	0.5179
59	49.00	0.4677
60	51.00	0.3671
61	53.00	0.2665
62	55.00	0.2530
63	60.25	0.1306

DQV

```
C
C     FORTRAN /360 MAIN PROGRAM, CALLED DQV, FOR READING TIME -
C     CONCENTRATION DATA INTO ARRAYS SUITABLE FOR FURTHER PROCESSING
C     BY SUBROUTINES QV, PLOTGA, AND TAILEX.
C
C     INPUT
C        CONTROL CARDS
C        1 ONE CARD PER RUN,
C          NO. OF DATA SETS, KTOT, (I2)
C        2 ONE PER DATA SET.
C          NO OF DATA POINTS, K, (I3)
C          ARRIVAL TIME OF TRACER WAVE, TST, (F7.2)
C          TITLE OR RUN IDENRIFICATION NO. (7A4)
C          PARAMETERS OF GAMMA EXTENSION, IF DESIRED,
C          LOG A, B, D,(2X,3E10.5). LOG A IS CONVERTED TO A.
C        DATA CARDS
C          TRACER CONCENTRATION, YC,IN PPB,TIME FROM INJECTION, XT,
C          IN MINUTES AND DECIMAL FRACTIONS, (2F9.3)
C
C     OUTPUT
C        PRINTOUT OF DATA
C
      CALL PLOTS
C
      DIMENSION  XT(50) ,YC(50) , TITLE (7)
C
1     FORMAT (I3 , F7.2,7A4,2X, 3E10.5)
2     FORMAT (2F9.3)
7     FORMAT (I2)                                                    D=
16    FORMAT (13H1CONTROL CARD  ,5X , I3 , F7.2, 7A4,/15H A, B, AND
     1  , 3E10.5)
32    FORMAT (5HODATA ,4X , 2HNO , 8X, 2HXT ,9X , 2HYC ,/
     1 (9X,  I2,   5X,   F7.2,   2X, F9.3)     )
C
C     LOOP FOR SETS
      READ (5,7) KTOT
      DO 8 KSET = 1 , KTOT
C
C     READ AND WRITE DATA
      READ (5,1) K , TST ,TITLE , A, B , D
      IF ( A .NE. 0.0 )  A =   EXP (A)
      READ (5,2)(YC  (I), XT(I), I = 1,K   )
      WRITE (6,16) K , TST , TITLE , A, B, D
      WRITE (6, 32) ( I, XT(I) , YC(I) , I = 1, K)
C
C     CALLS TO SUBROUTINES GO HERE
      CALL TAILEX  (K, TST, XT, YC, TITLE )
      CALL TAILEX  (K, TST, XT, YC, TITLE )
C
8     CONTINUE
      CALL PLOTND
      WRITE (6,100)
100   FORMAT (1H1)
      STOP
      END
```

CONTROL CARD 17 14.00BR R2 G1UP-1-2X, AUG 15,67
A, B, AND D = .16323E 01.10000E 01.86000E-01

DATA	NO	XT	YC
	1	12.00	0.0
	2	15.00	0.100
	3	18.00	0.600
	4	21.00	12.100
	5	24.00	38.000
	6	27.00	50.500
	7	29.00	47.000
	8	31.00	38.000
	9	33.00	27.800
	10	35.00	20.200
	11	37.00	14.100
	12	41.00	7.300
	13	45.00	4.500
	14	49.00	3.100
	15	54.00	2.400
	16	59.00	1.800
	17	69.00	1.000

D Q V

SAMPLE PRINTOUT

TAILEX

```
      SUBROUTINE TAILEX ( K , TST , XTIN,YC , TITLE )
C
C     THIS FORTRAN /360 SUBROUTINE IS CALLED BY DQV OR BY NACL,
C     IF A GAMMA EXTENSION IS TO BE FITTED. IT PLOTS THE
C     C-T DARA IN THE FORM (LOG C -B*LOG T ) VS. (T), FOR
C     B-VALUES OF -1,0, 1, 2, 3, AND4.
C                                                                  Y(200),
      DIMENSION XT(200), YC(200), BXL(200), YL(200), TITLE(7),
     1 XTIN(200), XTPLO(200)
      DO 1 J = 1, K
      XT(J) =  XTIN(J)  - TST
      XTPLO (J)  = XT (J )
      IF ( XTPLO (J) .LT. 0.0 ) XTPLO(J) = 0.0
      IF (YC(J) .LE. 0.0) YL(J) = -10.0
      IF ( YC(J) .GT. 0.0) YL(J) = ALOG ( YC(J))
1     CONTINUE
      CALL  SCALE  (XTPLO , K  , 14.0, XTMIN, DXT , 1 )       XTMIN,
      CALL  AXIS   ( 0.0, 7.0 ,15HXT - TST, (MIN),-13 ,14.0 ,0.0,
     1 DXT )
      N = 0.0
      DO 2 J = 1, K
      YDIFF  = YC (J+1) - YC(J)
      IF (YDIFF .LT. 0.0)  N = N+1
      IF (N .EQ. 3) GO TO  3
2     CONTINUE
3     WRITE (6, 4 ) TITLE , TST , J , XT(J)
4     FORMAT ( 18H1SUBROUTINE TAILEX ,/9H RUN NO.  , 7A4,   2X,
     1   18H   STARTING TIME   , F7.2, /
     2   28H TAILEX STARTS AT POINT NO  ,I2,10X,7HXTIME = ,F8.2)
C     LOOP FOR  DIFFERENT B  VALUES
      DO  6  J5 =  0 , 5
      J2 =  J5  - 1
      B   = J2
      DO  7 J3 = J , K
      AX   = XT(J3)
      IF ( AX .GT.0.01 ) BXL (J3) = B * ALOG (AX)
      IF ( AX .LE.  0.01 ) BXL (J3) = - 10.0
7     Y(J3)  =-BXL(J3)  + YL(J3)
      IF (                 J2 .EQ. 1 .OR. J2 .EQ. 4 )              XT(M)
     1WRITE (6 , 8 ) TST , B , ( M ,YC(M) , YL(M) , BXL(M) , Y(M),
     2 , M  = J , K )
8     FORMAT  (16H0INITIAL TIME = , F7.2 ,/   5H B = , F6.2, /
     1   54H NO   CONC.   LOG C    B*LOG T      Y          XT - TIN   ,/
     2   (1X, I2 , F8.2 ,F9.4 , F10.4 , F10.4 , F 9.2 ))          INC.)
C     ELIMINATION OF DATA POINTS TO SECOND POINT BEYOND PEAK(NOT
      IF               (J2 .NE.(-1))GO TO 10
      YMAX = 4.* ALOG (AX)
      IF (YL(K) .LE. 0.0 .AND. YL(K) .GT.(-9.))YMAX = YMAX + YL(K)
      DY = 0.5
      IF (YMAX .GT. 3.5)  DY = 1.0
      IF (YMAX .GT. 7.0)  DY = 2.0
      IF (YMAX .GT. 14.)  DY = 5.0
      IF (YMAX .GT. 35.0)  DY = 10.0
      YLMAX = Y (J)
      IF ( Y(K) .GT. Y(J)) YLMAX = Y(K)
```

The parameters, A, B, and D, of 'TAILEX' correspond to Y1, Y2, and Y3 of text.

```
      DX = 0.5
      IF ( YLMAX .GT. 1.5)  DX = 1.0
      IF ( YLMAX .GT. 3.0 ) DX = 2.0
      IF ( YLMAX .GT. 6.0 ) DX = 5.0
      IF ( DX .GT.  DY )  DY =  DX
      YMIN =-7.0* DY
      YMAX =+3.0* DY
      CALL AXIS  ( 0.0,0.0,15HLOG C - B.LOG T , +15 , 10.0, 90.0,
     1 YMIN, DY )
      CALL SYMBOL (1.0, 1.0, 0.21, TITLE, 0.0, 28  )
      CALL SYMBOL (1.0, 0.5, 0.21, 15HSTARTING TIME =    , 0.0,  15 )
      CALL NUMBER( 4.0, 0.5, 0.21,  TST, 0.0, 2  )
 10   DO  12  J4 =  J , K
 12   Y(J4) = Y(J4)/ DY     +  7.0
      CALL SYMBOL(XTPLO(J),Y(J), 0.14 , 3  , 0.0, -1 )
      KS= J +1
      DO 11   M = KS , K
      IF (YL(M) .LT. (-9.0)) GOTO 11
      IF ( Y(M).GT.10.0 .OR. Y(M).LT.  0.0  )GO TO 13
      CALL SYMBOL(XTPLO(M) ,Y(M) ,0.14 ,  3 ,0.0 , -2 )
      GO TO 11                                                  (-2)
 13   IF ( Y(M).GT.10.0) CALL SYMBOL(XTPLO(M),10.,0.14 , 7 , 0.0 ,)
      IF ( Y(M).LT. 0.0  )CALLSYMBOL(XTPLO(M),0.0,0.14 , 5 , 0.0 ,)
 11   CONTINUE                                                  (-2)
 6    CONTINUE
      CALL PLOT  ( 16.0 , 0.0 , -3 )
      RETURN
      END
```

SUBROUTINE TAILEX
RUN NO. BR R2 G1UP-1-2X, AUG 15,67 STARTING TIME 14.00
TAILEX STARTS AT POINT NO 8 XTIME = 17.00

INITIAL TIME = 14.00
B = 1.00

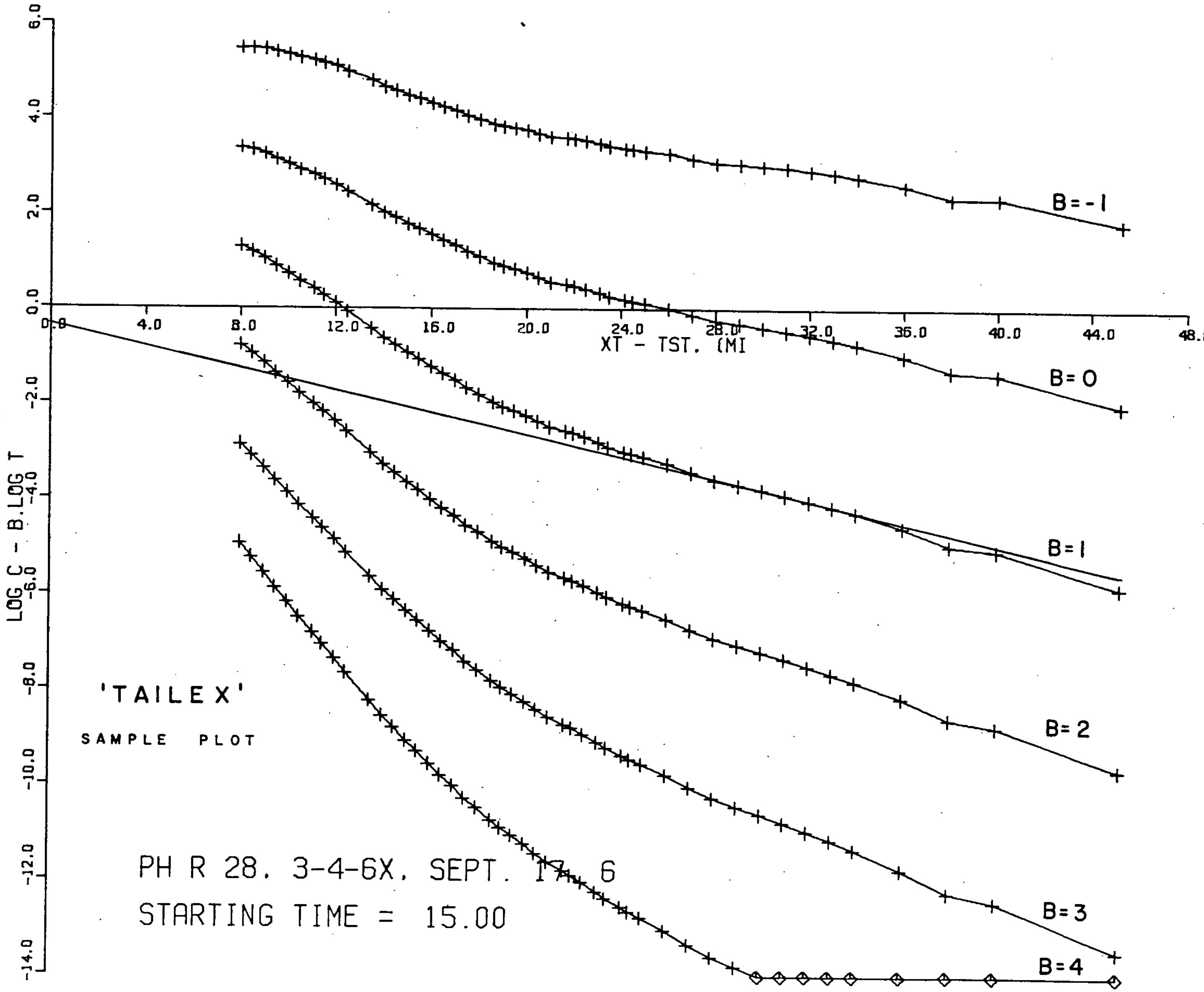
NO	CONC.	LOG C	B*LOG T	Y	XT - TIN
8	38.00	3.6376	2.8332	0.8044	17.00
9	27.80	3.3250	2.9444	0.3806	19.00
10	20.20	3.0057	3.0445	-0.0388	21.00
11	14.10	2.6462	3.1355	-0.4893	23.00
12	7.30	1.9879	3.2958	-1.3080	27.00
13	4.50	1.5041	3.4340	-1.9299	31.00
14	3.10	1.1314	3.5553	-2.4239	35.00
15	2.40	0.8755	3.6889	-2.8134	40.00
16	1.80	0.5878	3.8067	-3.2189	45.00
17	1.00	0.0	4.0073	-4.0073	55.00

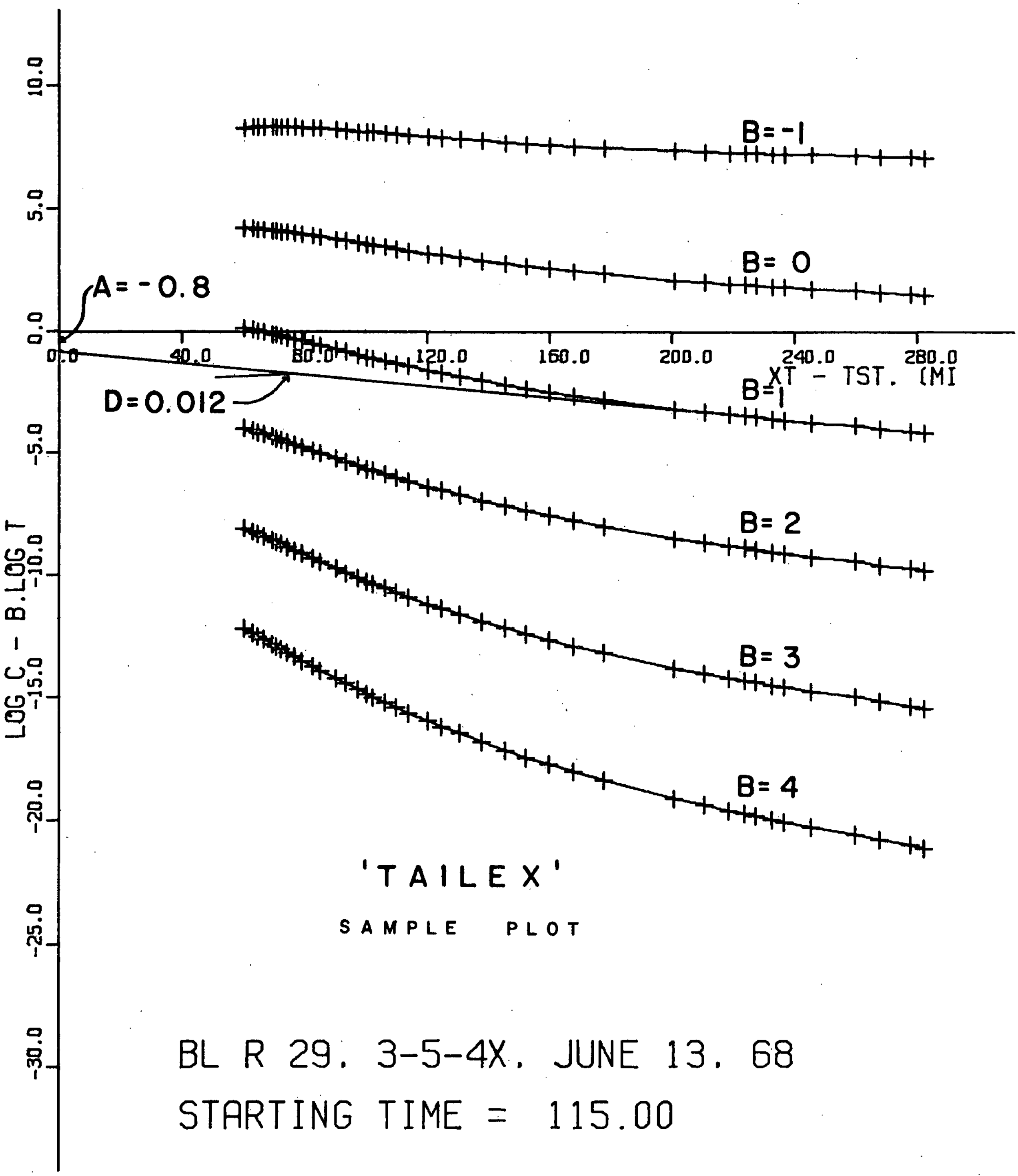
INITIAL TIME = 14.00
B = 4.00

NO	CONC.	LOG C	B*LOG T	Y	XT - TIN
8	38.00	3.6376	11.3329	-7.6953	17.00
9	27.80	3.3250	11.7778	-8.4527	19.00
10	20.20	3.0057	12.1781	-9.1724	21.00
11	14.10	2.6462	12.5420	-9.8958	23.00
12	7.30	1.9879	13.1833	-11.1955	27.00
13	4.50	1.5041	13.7359	-12.2319	31.00
14	3.10	1.1314	14.2214	-13.0900	35.00
15	2.40	0.8755	14.7555	-13.8800	40.00
16	1.80	0.5878	15.2267	-14.6389	45.00
17	1.00	0.0	16.0293	-16.0293	55.00

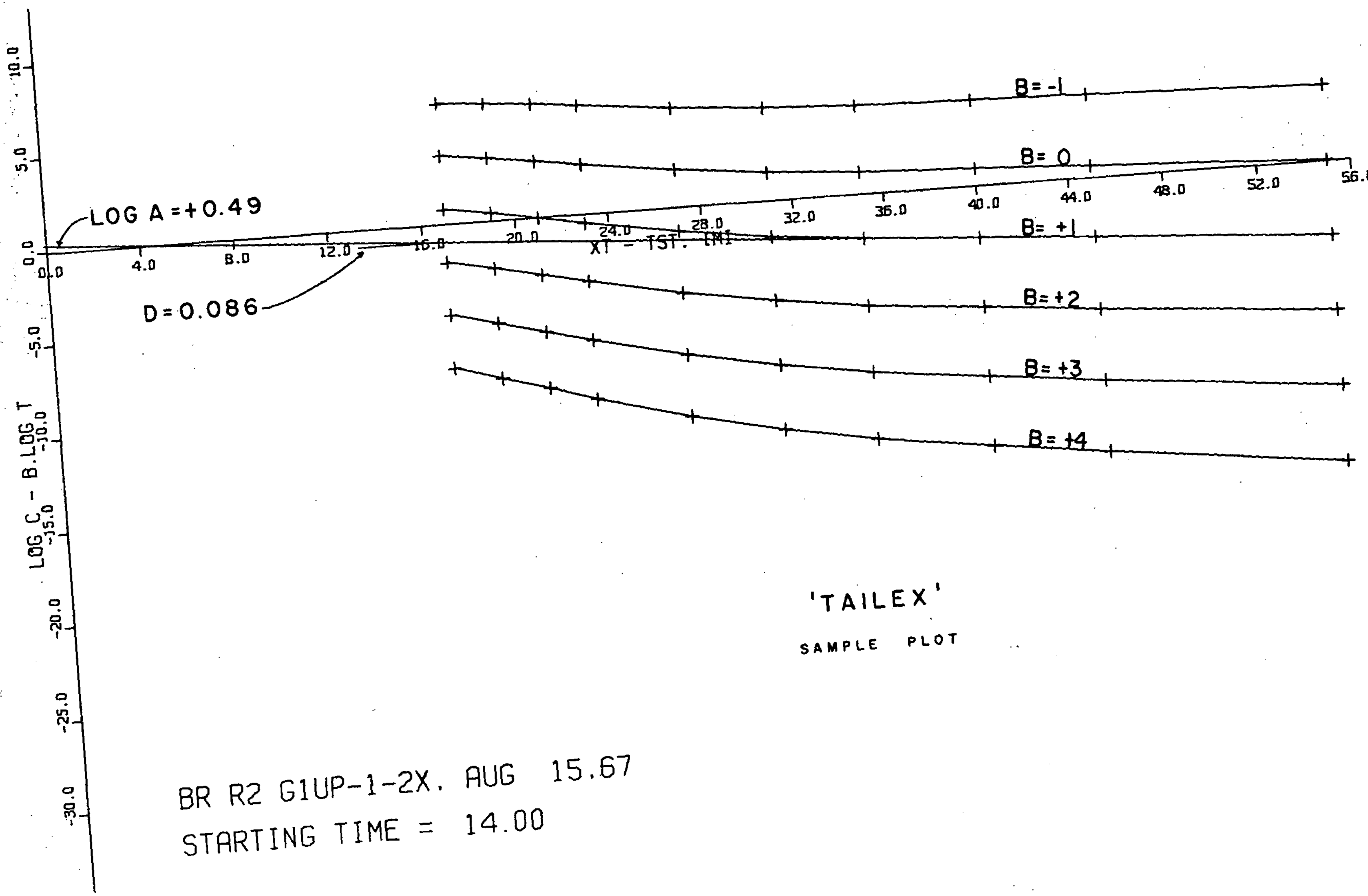
'TAILEX'

SAMPLE PRINTOUT


'TAILEX'
SAMPLE PLOT
PH R 28. 3-4-6X. SEPT. 17. 6
STARTING TIME = 15.00
LOG C - B.LOG T
XT - TST. (MI
B=-1
B=0
B=1
B=2
B=3
B=4
0.0
4.0
8.0
12.0
16.0
20.0
24.0
28.0
32.0
36.0
40.0
44.0
48.0
6.0
2.0
-2.0
-4.0
-6.0
-8.0
-10.0
-12.0
-14.0



B=-1
B= 0
A=-0.8
D=0.012
B=1
B= 2
B= 3
B= 4
XT - TST. (MI
LOG C - B.LOG T
0.0
40.0
80.0
120.0
160.0
200.0
240.0
280.0
10.0
5.0
-5.0
-10.0
-15.0
-20.0
-25.0
-30.0
'TAILEX'
SAMPLE PLOT
BL R 29. 3-5-4X. JUNE 13. 68
STARTING TIME = 115.00


LOG C - B.LOG T
10.0
5.0
0.0
-5.0
-10.0
-15.0
-20.0
-25.0
-30.0
0.0
4.0
8.0
12.0
16.0
20.0
24.0
28.0
32.0
36.0
40.0
44.0
48.0
52.0
56.0
XT - TST. (H)
LOG A = +0.49
D = 0.086
B= -1
B= 0
B= +1
B= +2
B= +3
B= +4
'TAILEX'
SAMPLE PLOT
BR R2 G1UP-1-2X. AUG 15.67
STARTING TIME = 14.00

QVEL

```
      SUBROUTINE Q VEL ( KK, TST , X , Y , TITLE, A, B, D ,NRWT, NIGA)
C
C     SUBROUTINE FOR NUMERICAL INTEGRATION AND NUMERICAL EVALUATION
C     OF FIRST MOMENTS OF TIME-CONCENTRATION CURVES.EXTENSION TO
C     INFINITE TIME, BASED ON A DECLINE OF C SIMILAR TO A GAMMA
C     DISTRIBUTION,IS OPTIONAL.
C     THIS PROGRAM REQUIRES 4 SUBROUTINES, GAUSS1, GAUSS2, AUX1,
C     AND AUX2
C
C     INPUT
C        KK IS THE NUMBER OF DATA POINTS, CALLED K IN NACL AND DQV,
C        TST IS STARTING TIME, AS BEFORE,
C        X AND Y ARE THE T-C DATA, CALLED XT AND YC IN NACL AND DQV,
C        TITLE IS AS BEFORE,
C        A, B, AND D ARE THE PARAMETERS OF THE GAMMA EXTENSION,
C        NRWT
C           0  IF RHWT TEST
C           1  IF NA CL TEST, 50 LITER TANK
C           2  IF NA CL TEST, 16 LITER TANK
C        NIGA,
C           0  IF A, B, D ARE NOT GIVEN
C           1  IF A, B, D ARE AVAILABLE FOR EXTENSION TO INF.
C
C     OUTPUT
C        INTEGRALS AND FIRST MOM.  OVER THE DATA POINTS, USING
C        FIRST AND SECOND ORDER METHODS. MEAN TIME IS (FIRST MOMENT
C        & STARTING TIME, TST ).
C        OPTIONAL,
C        WITH NIGA = 1, INTEGRALS AND MOMENTS WITH EXTENSION TO
C        INFINITE TIME. 'INT TO XT(K)'IS THE NEGLECTED PART OF
C        THE INTEGRAL OVER THE GAMMA DISTRIBUTION, UP TO
C        TIME XT(K). 'FACTOR FAM' IS THE ADJUSTMENT TO A, TO
C        ACHIEVE CLOSEST FIT TO THE LAST 3 DATA POIMTS.
C        'FACTOR A' IS THE CORRECTED VALUE OF A, A*FAM.
C        WITH NRWT = 1 OR 2, THE PROGRAM COMPUTES THE DISCHARGE.
C
      DIMENSION XT(200), YC(200), X(20), Y(20), TITLE(7),YCO(3),
     1 FACTOR(3)
      DOUBLE PRECISION   DGAMMA , GX, G   Y
      REAL  MT, MEAN T1,  MEAN T2     ,MEAN T 3
C
C     ELIMINATION OF SUPERFLUOS DATA POINTS
      DO 9 J = 1 , K K
      IF (X(J) .GT. TST)  GO TO 10
9     CONTINUE
10    K  = KK-J + 2
      DO 11    J1  =  1 , K
      I =J1 + J -1
      XT (J1+1) = X (I) - TST
11    YC (J1+1) = Y (I)
      XT (1)     = 0.0
      YC (1)     = 0.0
C
C     INTEGRAL OF DATA POINTS , FIRST AND SECOND ORDER METHODS
C
```

```
C       FIRST ORDER
1       CT MT 1 = 0.0
        CT INT1 = 0.0
        DO 4  J = 2 , K
        TRAPEZ  =            ((YC(J-1)+ YC(J))* (XT(J)- XT(J-1)))/ 2.0
        CT INT1 = CT INT1+ TRAPEZ
        CT MT 1 = CTMT 1 + TRAPEZ *( XT(J-1) + ((2.*YC(J) + YC(J-1)) /
     1           (3.0 *(YC(J-1)+ YC(J)))) * ( XT(J) - XT(J-1)))
4       CONTINUE
        FIRST M = CT MT 1  / CT INT 1
        MEAN T1 = FIRST M  + TST
C
C       SECOND ORDER
        KTEST   =(K / 2)* 2
        KLIM    = K - 1
        CT INT2 = 0.0
        CT MT 2 = 0.0
        IF (KTEST .NE. K ) KLIM  = K
        DO 5  J = 3 , KLIM , 2
        DT1     = XT (J-1) - XT(J-2)
        DT2     = XT (J)   - XT(J-2)
        F0      =((((DT2**2)*DT1)/2.) - (DT2**3)/6.0) / (DT1*DT2)
        F1      =(  (DT2**3)/(-6.0) )/ ((DT1**2) - (DT1*DT2))   (-DT1
        F2      = (((DT2**3)/ 3.0) -(((DT2**2)*DT1) / 2.0)) /((DT2**2)
     1           * DT2))
        D INT    = F0 * YC (J-2) + F1 * YC (J-1) + F2 * YC(J)
        CT INT2 = CT INT 2  +  D INT
        U  =  YC(J-2)  / (DT1 * DT2)
        V  =  YC(J-1)  / (DT1**2  - DT1*DT2)
        W  =  YC(J)    / (DT2**2  - DT1*DT2)
        CT MT 2 = CT MT 2  + D INT *                              (+DT2))
     1          ( XT (J-2) +(  ((U+V+W)*(DT2**4))/ 4.0  - ((U* (DT1)
     2           +(V * DT2) +(W * DT1)) * (DT2**3) / 3.0  +(U* DT1*
     3           (DT2** 3)) / 2.0 ) / DINT )
5       CONTINUE
        IF (KTEST .NE. K )  GO TO 6
        TRAPEZ    = ((YC(K-1)+ YC(K))*( XT (K)- XT(K-1))) / 2.0
        CT INT 2 = TRAPEZ  + CT INT 2
        CT MT  2 = CT MT 2 + TRAPEZ * (XT(K-1) +  (2.0*YC(K) +YC(K-1))/
     1           (3.0* (YC(K) + YC(K-1))) * (XT(K) - XT(K-1)))
6       FIRST N  = CT MT2 / CT INT 2
        MEAN T2  = TST + FIRST N
C
C       WRITE RESULTS                                         (MEANT2
        WRITE (6, 7)  CT INT 1 , FIRST M , MEAN T1 , CTINT2,FIRSTN,)
7       FORMAT ( 32H1INTEGRATION OF MEASURED POINTS  ,    /
     1    19H0INTEGRAL CT1     =  , F15.5 ,    10HPPB * MIN   , /
     2    19H FIRST MOMENT (1)=  , F15.5 ,     4HMIN         , /
     3    19H MEAN TIME (1)   =  , F15.5 ,     4HMIN         , /
     4    19H INTEGRAL CT2     =  , F15.5 ,    10HPPB * MIN   ,/
     5    19H FIRST MOMENT (2)=  , F15.5 ,     4HMIN         , /
     6    19H MEAN TIME (2)   =  , F15.5 ,     4HMIN          ) (INF.
C       INTEGRATION OF DATA POINTS COMBINED WITH FITTED EXTENSION TO)
        IF (NIGA .EQ. 0  ) GO TO 3
C
```

```
C       CORRECTION FACTOR
        K3  = K - 3
        DO 8 J = 1 , 3
        I = K3 + J
        YCO (J) = A*(XT(I)**B)  * EXP( -D*XT(I))
8       FACTOR(J)  =  YC(I) / YCO(J)
        FAM =(FACTOR (1) + 2.0 * FACTOR(2) + 3.0 * FACTOR(3)  ) / 6.0
C
C       INTEGRATION OF GAMMA DISTRIBUTION
        R = B + 1.0
        GX = R
        GY = DGAMMA(GX)
        G  = GY
        GAMMA = A * G  / (D**R)
        XMO1  = R / D
        CTINTF = (D ** R ) / G
        FIRST3  = 0.0
        XINT=  XT(K) / 20.0
        CTINT3  =0.0
        DO 12  J = 1 ,20
        COUNT  = J
        XUL   = XT(K) -  XINT * (COUNT -1.0)
        XLL   = XUL - XINT
        CALL GAUSS1( XLL , XUL , DB, A, B , D)
        CALL GAUSS2( XLL , XUL , DA,A,  B,  D)
        CTINT3  = CTINT3 + DB
        FIRST3  = FIRST3 + DA
12      CONTINUE
        DEBCT  = ( CTINT3  * A )  /  GAMMA
        D INT3  =(GAMMA  - CTINT 3* A )     * FAM
        DFMO3 = ((XMO1  * GAMMA) -  (FIRST3 * A))  * FAM
        CT INT 4  = CTINT1   + DINT3
        FIR M4    =(CTMT1    + DFMO3 ) / CT INT 4
        CT INT 5  = CTINT2   + DINT3
        FIR M5    =(CTMT2    + DFMO3 ) / CT INT 5
        MEAN T3  =  FIR M5  + TST
C
C       WRITE RESULTS
        WRITE(6,13)  DINT 3  , DFMO 3 , CTINT4 ,  FIRM4 , CTINT5, FIRM5
       1  , MEAN T 3 , DEBCT, FAM
13      FORMAT ( 45HOINTEGRATION OF DATA POINT WITH FITTED EXT.   , /
       1   18HOAREA CORR.       = , F15.5, 7HPPB*MIN , /
       2   18H FIRST MOM. CORR.= , E15.5,  15H MIN**2 * PPB     ,//
       3   18H AREA BY TRAPEZ   = , F15.5, 7HPPB*MIN , /
       4   18H FIRST MOM. (TR) = , F15.5, 7HMIN     , /
       5   18H AREA BY PARAB.   = , F15.5, 7HPPB*MIN , /
       6   18H FIRST MO. BY PA.= , F15.5, 7HMIN       /
       7   18H MEAN TIME 3      = , F15.5 , 7HMIN      //
       8   18H INT TO XT(K)     = ,F15.5    /
       9   18H FACTOR FAM       =  ,  F15.5  )
        A = FAM * A
        WRITE (6,16) A
16      FORMAT ( 18H FACTOR A         =  , F15.5   )
3       CONTINUE
C
```

```
C       COMPUTE DISCHARGE OF SALT TESTS.
        IF ( NRWT .EQ. 0 ) RETURN
        DISCH = 0.0
        IF ( NRWT .EQ. 1 .AND. NIGA .EQ. 0 )DISCH = 833.3 / CTINT2
        IF ( NRWT .EQ. 2 .AND. NIGA .EQ. 0 )DISCH = 833.3 /(3. * CTINT2)
        IF ( NRWT .EQ. 1 .AND. NIGA .GT.0 ) DISCH = 833.3 / CTINT5
        IF ( NRWT .EQ. 2 .AND. NIGA .GT.0 ) DISCH = 833.3 /(3.*CTINT5)
        IF ( DISCH .GT. 0.0 ) WRITE (6,15)  DISCH
 15     FORMAT ( 18H0DISCHARGE        =  , F15.5 , 17H CUBIC M PER SEC.)
        RETURN
        END
```

```
      SUBROUTINE  GAUSS1 (A, B,  AREA, XA, XB,  XD)
C
C     SUBROUTINE IN FORTRAN /360 CALLED BY SUBROUTINE QVEL.
C
      DIMENSION AX(4), H(4)
      DOUBLE PRECISION AX, H
      AX(1) = 0.960289856497536
      AX(2) = 0.796666477413627
      AX(3) = 0.525532409916329
      AX(4) = 0.183434642495650
      H(1) = 0.101228536290376
      H(2) = 0.222381034453374
      H(3) = 0.313706645877887
      H(4) = 0.362683783378362
      P = (B+A)*0.5
      Q = (B-A)*0.5
      SUM = 0.0
      DO 30 J = 1,4
      R = AX(J)*Q
      X = P+R
      CALL AUX1 (X,  Y,  XA, XB,  XD)
      Z = Y
      X = P-R
      CALL AUX1 (X,  Y,  XA, XB,  XD)
30    SUM = SUM + H(J)*(Z+Y)
      AREA = Q*SUM
      RETURN
      END
```

```
      SUBROUTINE AUX1 ( X, Y,A, B, D)
C
C     CALLED BY GAUSS1.
C
      Y  =  (X ** B) * EXP((- D)* X )
      RETURN
      END
```

```
      SUBROUTINE  GAUSS2 (A, B,  AREA, XA, XB,  XD)
C
C     SUBROUTINE IN FORTRAN /360 CALLED BY SUBROUTINE QVEL.
C
      DIMENSION AX(4), H(4)
      DOUBLE PRECISION AX, H
      AX(1) = 0.960289856497536
      AX(2) = 0.796666477413627
      AX(3) = 0.525532409916329
      AX(4) = 0.183434642495650
      H(1) = 0.101228536290376
      H(2) = 0.222381034453374
      H(3) = 0.313706645877887
      H(4) = 0.362683783378362
      P = (B+A)*0.5
      Q = (B-A)*0.5
      SUM = 0.0
      DO 30 J = 1,4
      R = AX(J)*Q
      X = P+R
      CALL AUX2 (X,  Y,  XA, XB,  XD)
      Z = Y
      X = P-R
      CALL AUX2 (X,  Y,  XA, XB,  XD)
   30 SUM = SUM + H(J)*(Z+Y)
      AREA = Q*SUM
      RETURN
      END
```

```
      SUBROUTINE AUX2 ( X , Y , A , B , D )
C
C     CALLED BY GAUSS2.
C
      Y =  ( X **(B+1.0)) * EXP ((-D)* X )
      RETURN
      END
```

```
INTEGRATION OF MEASURED POINTS

INTEGRAL CT1       =       678.49805PPB * MIN
FIRST MOMENT (1)=           17.41537MIN
MEAN TIME (1)      =        31.41537MIN
INTEGRAL CT2       =       674.12207PPB * MIN
FIRST MOMENT (2)=           17.38902MIN
MEAN TIME (2)      =        31.38902MIN

INTEGRATION OF DATA POINT WITH FITTED EXT.

AREA CORR.         =        13.54708PPB*MIN
FIRST MOM. CORR.=          930.09424 MIN**2 * PPB

AREA BY TRAPEZ     =       692.04492PPB*MIN
FIRST MOM. (TR) =           18.41844MIN
AREA BY PARAB.     =       687.66895PPB*MIN
FIRST MO. BY PA.=           18.39899MIN
MEAN TIME 3        =        32.39899MIN

INT TO XT(K)       =         0.94942
FACTOR FAM         =         1.21365
FACTOR A           =         1.98106
```

'QVEL'

SAMPLE PRINTOUT

FOR 'BR R2, GIUP-1-2X'

```
INTEGRATION OF MEASURED POINTS

INTEGRAL CT1       =       6153.19922PPB * MIN
FIRST MOMENT (1)=            96.25470MIN
MEAN TIME (1)      =        211.25470MIN
INTEGRAL CT2       =       6149.38672PPB * MIN
FIRST MOMENT (2)=            96.21269MIN
MEAN TIME (2)      =        211.21269MIN

INTEGRATION OF DATA POINT WITH FITTED EXT.

AREA CORR.         =        459.59253PPB*MIN
FIRST MOM. CORR.=    176856.87500 MIN**2 * PPB

AREA BY TRAPEZ     =       6612.78906PPB*MIN
FIRST MOM. (TR)   =         116.30965MIN
AREA BY PARAB.    =        6608.97656PPB*MIN
FIRST MO. BY PA.=          116.28214MIN
MEAN TIME 3        =        231.28214MIN

INT TO XT(K)       =             0.85202
FACTOR FAM         =             0.99531
FACTOR A           =             0.44722

DISCHARGE          =             0.12609 CUBIC M PER SEC.
```

'QVEL'

SAMPLE PRINTOUT

FOR 'BL R29, G3-5-4X'

```
INTEGRATION OF MEASURED POINTS

INTEGRAL CT1      =        268.61548PPB * MIN
FIRST MOMENT (1)=            9.89905MIN
MEAN TIME (1)     =          24.89903MIN
INTEGRAL CT2      =        268.43823PPB * MIN
FIRST MOMENT (2)=            9.90452MIN
MEAN TIME (2)     =          24.90451MIN

DISCHARGE         =          3.10425 CUBIC M PER SEC.
```

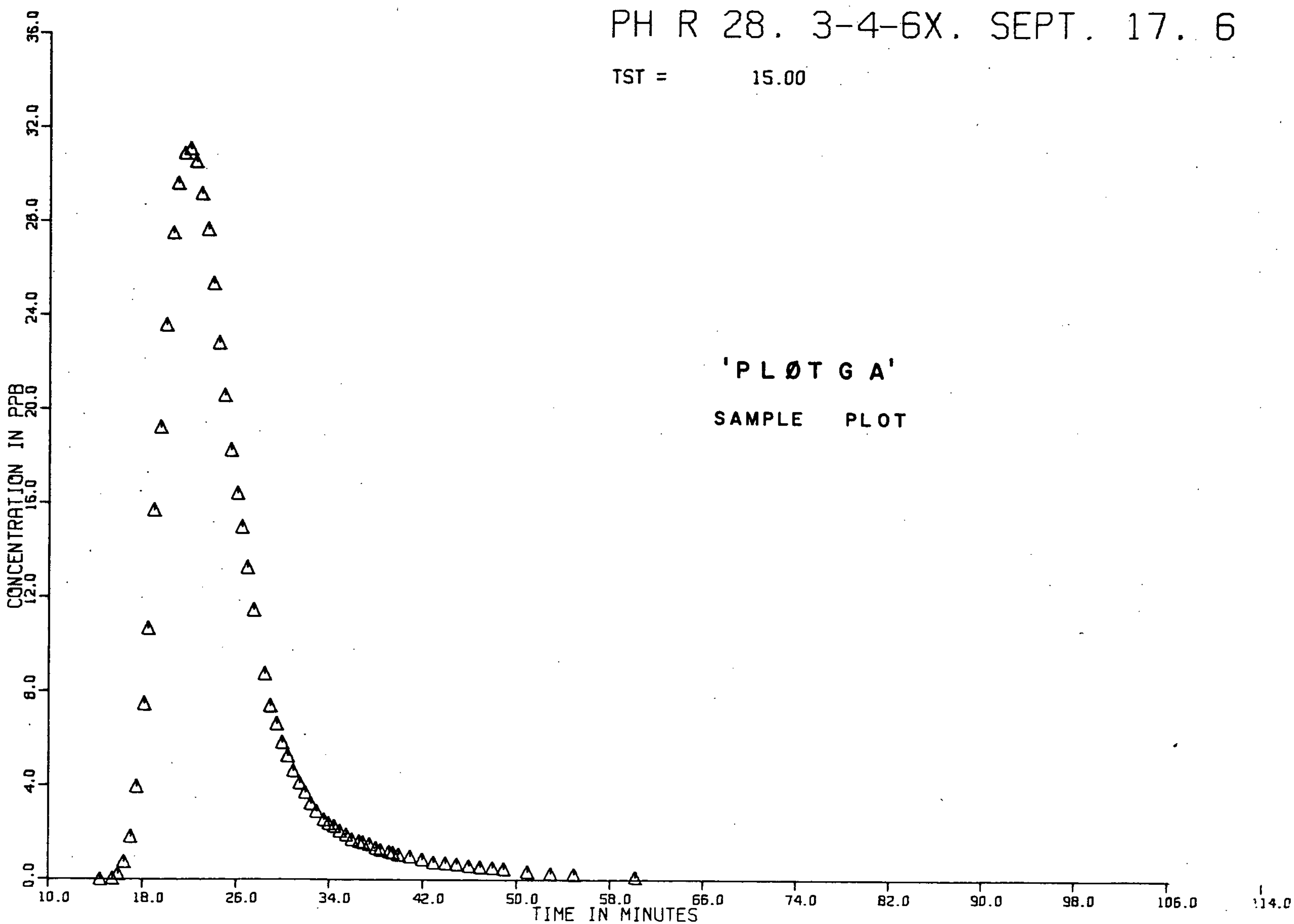
'QVEL'

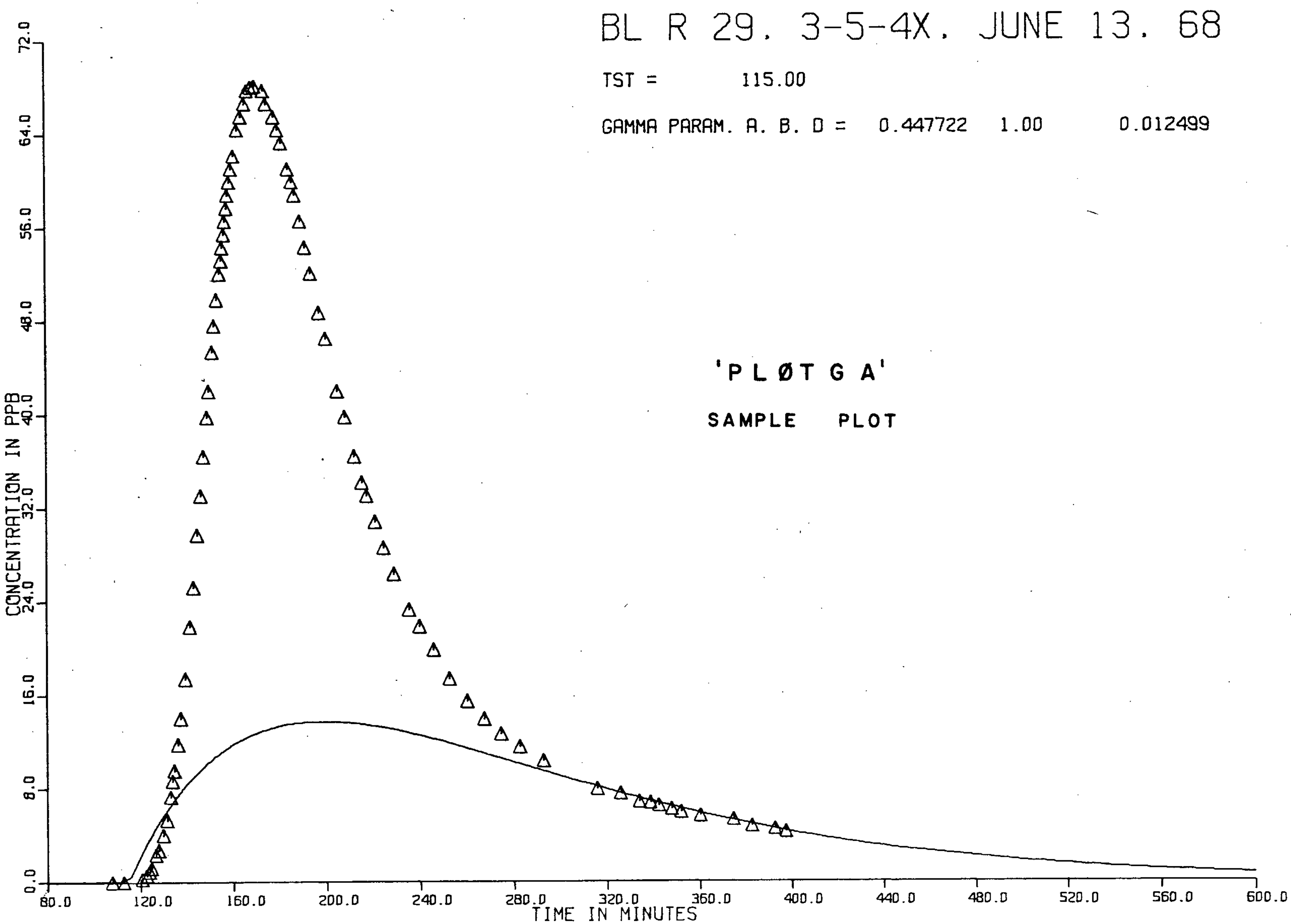
SAMPLE PRINTOUT

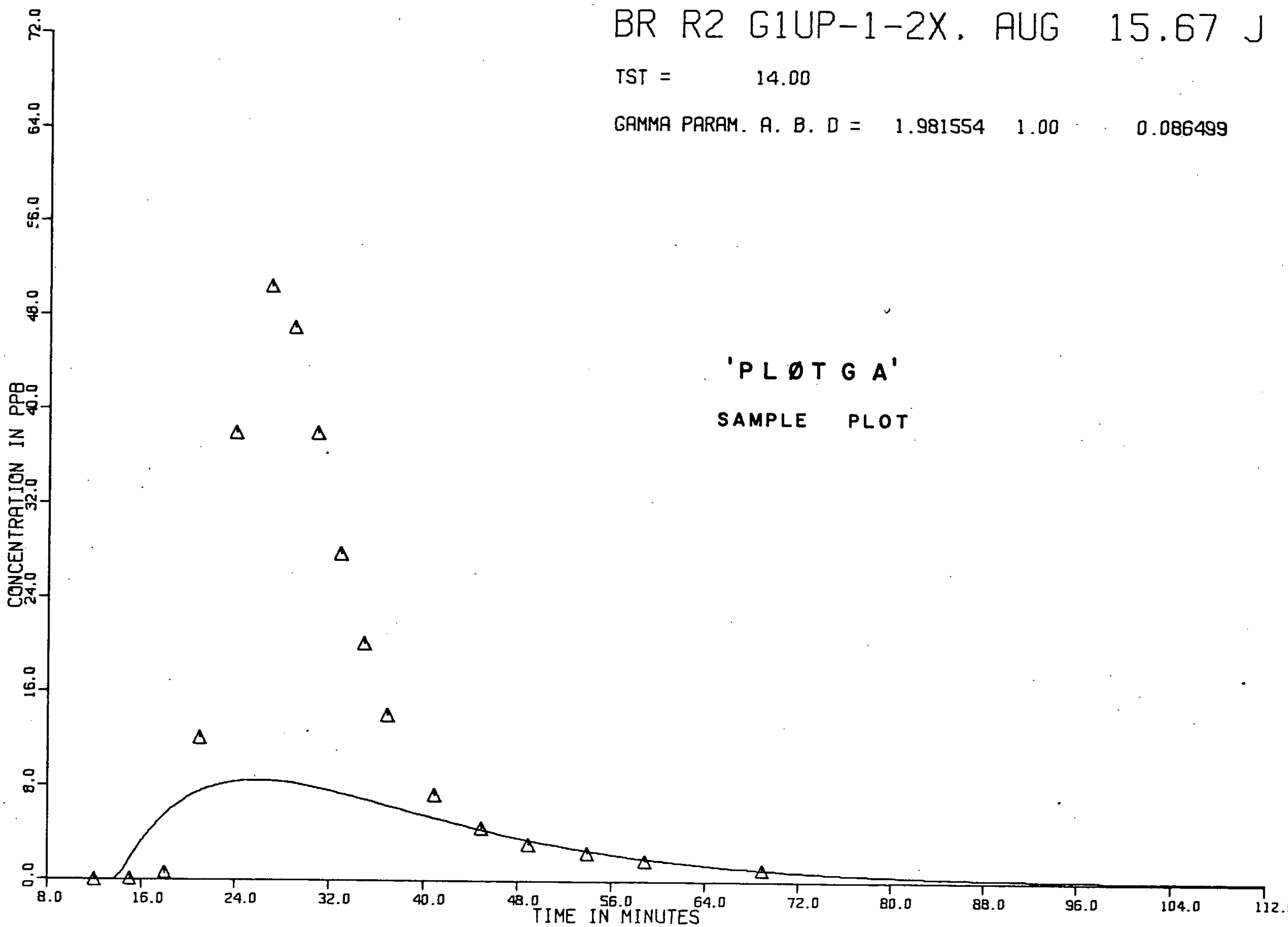
FOR 'PH R28, G3-4-6X'

PLØTGA

```
      SUBROUTINE PLOTGA (K, TST,XTIN,YCIN,A, B, D, TITLE )
C
C     THIS SUBROUTINE IN FORTRAN /360 IS CALLED BY THE MAIN
C     PROGRAMS NACL AND DQV TO PLOT THE C-T CURVES . OPTIONALLY
C     IT WILL ALSO PLOT THE GAMMA EXTENSIONS. IN THIS CASE IT SHOULD
C     BE CALLED AFTER THE SUBROUTINE QVEL HAS BEEN CALLED, AS QVEL
C     IMPROVES THE ESTIMATE OF A.
C
      DIMENSION XT(200),YC(200), TITLE(7), XTIN(200), YCIN(200)
      DO  11  I =1 ,  200
      XT(I)  =  XTIN(I)
11    YC(I)  =  YCIN(I)
      CALL SCALE ( XT, K, 10.0, XTMIN, DXT, 1 )
      CALL SCALE ( YC, K,  9.0, YCMIN, DYC, 1 )                (DXT)
      CALL AXIS  (0.0, 0.0, 15HTIME IN MINUTES, -15,13.0,0.0,XTMIN,)
      CALL AXIS  (0.0, 0.0, 20HCONCENTRATION IN PPB, +20  , 9.0,)
     1 YCMIN , DYC )                                            (90.0,
      DO 1 J = 1, K
1     CALL SYMBOL (XT(J), YC(J) , 0.14 , 2 , 0.0, -1 )
      CALL SYMBOL (6.0, 9.0, 0.28, TITLE, 0.0, 30 )
      CALL SYMBOL (6.0, 8.5 , 0.14 , 5HTST = , 0.0 , 5 )
      CALL NUMBER ( 7.5 , 8.5 , 0.14 , TST , 0.0 , 2)
      IF (A .EQ. 0.0 ) GO TO  6
      CALL PLOT  (0.0 ,0.0, +3)
      YMAX  =(YCMIN /DYC) + 9.0
      DO 2 I = 1, 131
      F      = I- 1
      X      = F / 10.0
      T      = XTMIN  + X * DXT - TST
      IF ( T .LT. 0.0 ) T = 0.0
      Y      =(( A*(T ** B))* EXP(-D * T ) -YCMIN ) / DYC
      IF  ( Y .LE. YMAX ) GO TO 5
      CALL SYMBOL ( X ,9.0, 0.07 , 13 ,0.0, -2 )
      GO TO 2
5     CALL PLOT   ( X ,Y , + 2 )
2     CONTINUE
      CALL SYMBOL ( 6.0 , 8.0 , 0.14 , 24HGAMMA PARAM. A, B, D =     ,
     1 0.0 , 24 )
      CALL NUMBER ( 9.0 , 8.0 , 0.14 , A , 0.0 , 6  )
      CALL NUMBER ( 10.3 , 8.0 , 0.14 , B , 0.0 , 2  )
      CALL NUMBER ( 11.6 , 8.0 , 0.14 , D , 0.0 , 6  )
6     CALL PLOT (15.0, 0.0, -3  )
      RETURN
      END
```


PH R 28. 3-4-6X. SEPT. 17. 6
TST = 15.00
'PLØTGA'
SAMPLE PLOT
CONCENTRATION IN PPB
0.0
4.0
8.0
12.0
16.0
20.0
24.0
28.0
32.0
36.0
10.0
18.0
26.0
34.0
42.0
50.0
58.0
66.0
74.0
82.0
90.0
98.0
106.0
114.0
TIME IN MINUTES


BL R 29. 3-5-4X. JUNE 13. 68
TST = 115.00
GAMMA PARAM. A. B. D = 0.447722 1.00 0.012499
'PLØTGA'
SAMPLE PLOT
CONCENTRATION IN PPB
0.0
8.0
16.0
24.0
32.0
40.0
48.0
56.0
64.0
72.0
80.0
120.0
160.0
200.0
240.0
280.0
320.0
360.0
400.0
440.0
480.0
520.0
560.0
600.0
TIME IN MINUTES


BR R2 G1UP-1-2X, AUG 15.67 J
TST = 14.00
GAMMA PARAM. A. B. D = 1.981554 1.00 0.086499
'PLØTGA'
SAMPLE PLOT
CONCENTRATION IN PPB
TIME IN MINUTES
0.0
8.0
16.0
24.0
32.0
40.0
48.0
56.0
64.0
72.0
80.0
88.0
96.0
104.0
112.0

LØGRE

```
C
C     THIS FORTRAN /360 PROGRAM COMPUTES THE LINEAR REGRESSIONS
C     ON LOG Q (DISCHARGE) OF THE FOLLOWING VARIABLES=
C        LOG TM    (MEAN TRACER TRAVEL TIME)
C        LOG A     ( CROSSECTIONAL AREA)
C        LOG V     (VELOCITY)
C        LOG TS    (STARTING TIME)
C        LOG TP    (PEAK TIME)
C        LOG TSS   (STARTING TIME, BUT OMITTING RUNS WITH TRACER
C                   INJECTION ABOVE THE REACH )
C        LOG TPP   (PEAK TIME, OMITTING RUNS AS FOR TSS)
C     THE ACTUAL REGRESSION ANALYSIS IS DONE BY A SUBR. 'REGR'.
C     TWO PLOTTING SUBROUTINES CAN ALSO BE CALLED FROM THIS PROGRAM.
C
C     INPUT
C        FIRST CONTROL CARD,  ONE PER JOB SUBMISSION, =
C          NO. OF SETS, (I2).
C        SECOND CONTROL CARD, ONE PER DATA SET, =
C          TITLE, (6X, 7A4)
C        DATA CARDS, ONE PER TEST RUN, =
C          RUN NO.              (I2)       COL 1 & 2
C          IDENTIFICATION       (I1)       COL 5
C          THIS IS 1 FOR RUNS WITH INJECTION OF TRACER AT UPSTREAM
C          END OF TEST REACH, 0 FOR OTHER RUNS.
C          DATA                 (6F6.0)    COL 7 & ON,  (Q,TS,TP,TM,A,V)
C                               Q IN L/S, TIMES IN MIN.,A IN SQ M,
C                               V IN M/S, Q IS CONVERTED TO CU M/S.
C
C     OUTPUT
C        PRINTOUT OF DATA,
C        LINEAR REGRESSION EQUATIONS, STANDARD ERROR OF ESTIMATE,
C        CORRELATION COEFF., DEGREES OF FREEDOM, F-RATIO.
C
      CALL PLOTS
      DIMENSION  NO(30), ID (30) , Q(30), TS(30),  TP (30), TM(30),
     1 A(30),  V (30),  TSS(30) , TPP (30),  TIT (7  )    ,QQ(30)
     2,Q1(30)
C
C     LOOP FOR NUMBER OF SETS ,
      READ (5, 1) KTOT
 1    FORMAT (I2)
      DO  2  KS=1, KTOT
C
C     READING AND PRINTING OF DATA
      READ (5,3) TIT
  3   FORMAT (6X,7A4)
      DO 4  K = 1, 30                                             (V(k)
      READ (5, 5)  NO(K), ID(K) , Q(K), TS(K), TP(K), TM(K), A(K),)
 5    FORMAT (I2, 2X, I1, 1X,  12F 6.0    )
      IF (NO(K).LE. 0 )  GO  TO  6
 4    CONTINUE
 6    K  = K -1
      WRITE (6, 7)  TIT
 7    FORMAT (1H1,7A4,/
     1 58HONO ID  Q (L/S)      TS        TP        TM        A   V/)
```

```
      WRITE (6, 8) (NO(I1), ID(I1), Q(I1), TS(I1), TP(I1), TM(I1),
     1 A(I1) , V(I1), I1 = 1, K )
  8   FORMAT (1H ,
     1  I2,  I3,  F10.2  , 3F9.2 , 2F9.4    )
      KID  = 0
C
C     TRANSFORMATION TO LOGS , EVALUATION OF NO. OF SIMPLE RUNS
      DO 9 I2 = 1, K
      Q(I2) = ALOG10 (Q(I2)/1000.)
      Q1(I2)   =   Q(I2)
      TS(I2) = ALOG10 (TS(I2))
      TP(I2) = ALOG10 (TP(I2))
      TM(I2) = ALOG10 (TM(I2))
      A(I2)  = ALOG10 (A(I2))
      V(I2)  = ALOG10 (V(I2))
  9   KID    = KID + ID  (I2)
C
C     REGRESSIONS ON ORIGINAL DATA
 10   FORMAT (1H0 / 18H LOG TM VS. LOG Q  )
      WRITE (6,10)
      CALL REGR ( Q , TM , K ,ATM , BTM )
 11   FORMAT (1H0 / 18H LOG A  VS. LOG Q  )
      WRITE (6, 11)
      CALL REGR ( Q , A  , K ,AA  , BA  )
 12   FORMAT (1H0 / 18H LOG V  VS. LOG Q  )
      WRITE (6, 12)
      CALL REGR ( Q , V  , K ,AV  , BV  )
      IF (KID .EQ. K )  GO TO  18
 13   FORMAT (1H0 / 18H LOG TS VS. LOG Q  )
      WRITE (6, 13)
      CALL REGR ( Q , TS , K ,ATS , BTS )
 14   FORMAT (1H0 / 18H LOG TP VS. LOG Q  )
      WRITE (6, 14)
      CALL REGR ( Q , TP , K ,ATP , BTP )
      IF (KID .EQ. 0 ) GO TO 20
C
C     TP AND TS REGRESSIONS
 18   NID  = 0
      DO 15  I5 = 1 , K
      IF (ID (I5 ) .LE. 0 )  GO TO 15
      NID  = NID + 1
      TSS(NID)  = TS (I5 )
      TPP(NID)  = TP (I5 )
      QQ(NID)   =  Q(I5)
 15   CONTINUE
      IF(NID .NE. KID ) WRITE (6, 16) KS
 16   FORMAT (1H1, 17H NID ERROR IN SET    ,  I2 )
 17   FORMAT (1H0 / 18H LOG TSS VS. LOG Q )
      WRITE (6, 17)
      CALL REGR ( QQ, TSS, KID , ATSS , BTSS )
 19   FORMAT (1H0 / 18H LOG TPP VS. LOG Q )
      WRITE (6, 19)
      CALL REGR ( QQ, TPP, KID , ATPP , BTPP )
      GO TO  21
C
```

```
C       CALLS  TO PLOTTING SUBROUTINES
C
20      CONTINUE                                                        (BTM,
        CALL TPLO ( Q , TS , TP, TM , K , K , ATS, ATP, ATM, BTS, BTP,)
       1  TIT,Q)
        GO TO 22
21      CONTINUE                                                   (BTSS,
        CALL TPLO ( Q , TSS ,TPP, TM , K , KID , ATSS, ATPP , ATM ,)
       1 BTPP ,  BTM ,  TIT ,  QQ  )
22      CONTINUE
        CALL HYPLO (Q1, A , V , K  ,AA , AV , BA ,  BV ,  TIT  )
C
2       CONTINUE
        CALL PLOTND
        STOP
        END
```

```
      SUBROUTINE REGR   (X,Y,N  ,  A , B )
C
C     A SUBROUTINE IN FORTRAN /360 CALLED BY THE MAIN ROUTINE LOGRE
C     IT COMPUTES THE REGRESSION OF  Y  ON  X AND PRINTS THE RESULT.
C
      DIMENSION X(100),Y(100)
      SUMX=0.0
      SUMY=0.0
      SUMX2=0.0
      SUMY2=0.0
      SUMP=0.0
      DO 1 J=1,N
      SUMX=SUMX+X(J)
      SUMY=SUMY+Y(J)
      SUMX2=SUMX2+X(J)**2
      SUMY2=SUMY2+Y(J)**2
    1 SUMP=SUMP+X(J)*Y(J)
      AN=N
      SSX=SUMX2-(SUMX**2)/AN
      SSY=SUMY2-(SUMY**2)/AN
      SP=SUMP-(SUMX*SUMY)/AN
      B=SP/SSX
      A=(SUMY/AN)-(B*(SUMX/AN))
      R=SP/(SQRT(SSX*SSY))
      WRITE (6,4) SSX,SSY,SP
    4 FORMAT (// 4H SSX,F12.4,4X,4H SSY,F12.4,4X,3H SP,F12.4 //)
      WRITE (6,5)A,B
    5 FORMAT (26H REGRESSION EQUATION  Y=   ,F10.4,2H +, F10.4,2H X)
      S = SQRT ( (SSY - SP*SP/SSX) / (AN - 2.0 )  )
      WRITE (6, 9) S
    9 FORMAT (27H STANDARD ERROR OF ESTIMATE   , F10.4  )
      WRITE (6,6) R
    6 FORMAT (24H CORRELATION COEFFICIENT    , F10.4   )
      NDF = N - 1
      WRITE (6, 7)  NDF
    7 FORMAT   ( 24H DEGREE OF FREEDOM                     ,I10)
      F = ( R*R*(AN - 2.0) ) / (1.0 - R*R)
      WRITE  (6, 8)  F
    8 FORMAT  ( 4H F =   , 20X, F10.4)
      RETURN
      END
```

```
      SUBROUTINE TPLO ( Q, TSS, TPP, TM , K ,KID , ATSS , ATPP ,ATM,
     1 BTSS , BTPP , BTM , TIT  ,   QQ  )
C                                                              (LOGRE
C     THIS FORTRAN /360 SUBROUTIN IS CALLED FROM THE MAIN PROGRAM)
C     IT PLOTS THE REGRESSIONS OF TSS, TPP, TM  ON  Q , INCLUDING
C     DATA POINTS.
C
       DIMENSION  Q(30), TSS(30), TPP(30) , TM (30),  T (90) , TIT(7)
     1  ,  QQ(30)
C
C     SCALE DATA
      DO 1 I = 1, K
1     T(I)   = TM (I)
      DO 10N = 1, KID
      K5   =   N + K
      T(  K5)= TPP(N)
      N1 = K + KID
      K6   =   N   +   N1
10    T( K6 )= TSS(N)
      N2 =N1 + KID
      CALL  SCALE ( T, N2 , 5.0 , TMIN , DT ,1 )
      K7   =   K   +   2 * KID
      DO   100   J5   = 1  , K7
100   T(J5)   =   T( J5)   +   3.0
      DO 2 J = 1, K
2     TM(J)   = T (J)
      DO 11   J1 = 1, KID
      K7   =   J1   +   N1
      TSS (J1)   = T ( K7 )
      K8   =   J1   +   K
11    TPP (J1)   = T( K8  )
      CALL  SCALE ( Q, K   , 5.0 , QMIN , DQ , 1 )
      DO   200   JJ7   =   1   ,   KID
200   QQ(JJ7)    =    ( QQ(JJ7)    -   QMIN   ) /   DQ
C
C     DRAW   AXIS                                             (DQ)
      CALL AXIS   ( 0.0, 3.0,16HQ IN CU M / SEC , -16, 5.0,0.,QMIN,)
      CALL AXIS   ( 0.0, 3.0,12HTIME IN MIN.   , +12 , 5.0 ,90.,)
C     PLOT POINTS                                            (TMIN,DT)
      DO 3I1 = 1 , K
3     CALL SYMBOL ( Q (I1) , TM(I1), 0.07, 2 , 0.0, -1 )
      DO 4  I2   = 1 , KID
      CALL SYMBOL ( QQ(I2) , TSS(I2), 0.07 , 3 , 0.0 , -1 )
4     CALL SYMBOL ( QQ(I2) , TPP(I2), 0.07 , 4 , 0.0 , -1 )
C
C     PLOT   REGRESSION LINES
C
C     TM VS Q  LINE
      YB   = (((ATM + BTM* QMIN) - TMIN) / DT) + 3.0
      XB   =   0.0
      IF ( YB .LE. 8.0 )  GO TO 101
      YB   =   8.0
      XB  =   ( 5.0   * DT   + TMIN - ATM   - QMIN* BTM   ) / ( DQ * BTM)
101   YE   = (((ATM + BTM* (QMIN + 5.0* DQ)) - TMIN) / DT ) + 3.0
      XE   =   5.0
```

```
      IF (  YE  .  GE  . 3.0  )  GO TO 102
      YE  =  3.0
      XE  = ( TMIN - ATM  - QMIN * BTM  ) / ( DQ  *  BTM  )
102   CALL PLOT (XB  , YB , +3 )
      CALL PLOT (XE  , YE , +2 )
C
C     TS VS Q  LINE
      YB  = (((ATSS+BTSS*  QMIN)- TMIN) / DT) + 3.0
      XB  =  0.0
      IF ( YB .LE. 8.0 )  GO TO 103
      YB  =  8.0
      XB =  ( 5.0  * DT  + TMIN - ATSS - QMIN* BTSS ) / ( DQ * BTSS )
103   YE  = (((ATSS+BTSS* (QMIN + 5.0* DQ)) - TMIN) / DT ) + 3.0
      XE  =  5.0
      IF (  YE  .  GE  . 3.0  )  GO TO 104
      YE  =  3.0
      XE  = ( TMIN - ATSS - QMIN * BTSS ) / ( DQ  *  BTSS )
104   CALL PLOT (XB  , YB , +3)
      CALL PLOT (XE  , YE , +2)
C
C     TP VS Q LINE
      YB  = (((ATPP + BTPP* QMIN) - TMIN ) / DT )  +3.0
      XB  =  0.0
      IF ( YB .LE. 8.0 )  GO TO 105
      YB  =  8.0
      XB =  ( 5.0  * DT  + TMIN - ATPP - QMIN* BTPP ) / ( DQ * BTPP )
105   YE  = (((ATPP + BTPP* (QMIN + 5.0* DQ))  - TMIN ) / DT ) +3.0
      XE  =  5.0
      IF (  YE  .  GE  . 3.0  )  GO TO 106
      YE  =  3.0
      XE  = ( TMIN - ATPP - QMIN * BTPP ) / ( DQ  *  BTPP )
106   CALL PLOT  (XB  , YB ,  +3 )
      CALL PLOT  (XE  , YE ,  +2 )
C
C     WRITE  TITLE  AND  LEGEND
      CALL SYMBOL ( 0.5,9.0,0.21,TIT,0.0,28)
      CALL SYMBOL ( 1.0 ,1.5 , 0.07 , 2 , 0.0 , -1 )
      CALL SYMBOL ( 1.0 ,1.0 , 0.07 , 4 , 0.0 , -1 )
      CALL SYMBOL ( 1.0 ,0.5 , 0.07 , 3 , 0.0 , -1 )
      CALL SYMBOL ( 1.5 ,1.5 , 0.14 ,  9HMEAN TIME , 0.0 , 9 )
      CALL SYMBOL ( 1.5 ,1.0 , 0.14 ,  9HPEAK TIME , 0.0 , 9 )
      CALL SYMBOL ( 1.5 ,0.5 , 0.14 , 13HSTARTING TIME  ,  0.0 , 13)
C
C     COMPLETE OUTLINE , MOVE ON
      CALL PLOT ( 0.0, 8.0, +3 )
      CALL PLOT ( 5.0, 8.0, +2 )
      CALL PLOT ( 5.0, 3.0, +1 )
      CALL PLOT (11.0, 0.0, -3 )
      RETURN
      END
```

```
      SUBROUTINE HYPLO (Q , A , V , K , AA , AV, BA , BV , TIT )
C                                                                (LOGRE
C     THIS FORTRAN /360 SUBROUTINE IS CALLED FROM THE MAIN PROGRAM)
C     IT PLOTS THE REGRESSIONS OF  A, AND  V , ON  Q .
C
      DIMENSION  Q(30), A(30) , V(30) , TIT (7)
C
C     SCALE DATA
      CALL SCALE  ( Q , K , 5.0, QMIN , DQ ,1 )
      CALL SCALE  ( V , K , 5.0, VMIN , DV ,1 )
      CALL SCALE  ( A , K , 5.0, AMIN , DA ,1 )
C
C     DRAW AXIS                                                  (QMIN,
      CALL AXIS ( 0.0 , 3.0 , 16HQ IN CU M / SEC  ,-16 ,5.0, 0.0,)
     1 DQ )                                                      (AMIN,
      CALL AXIS (0.0,   3.0 , 14HAREA IN SQ M.   ,  +14 ,5.0,90.0,)
     1 DA )                                                      (VMIN,
      CALL AXIS (5.0,   3.0 , 17HVELOCITY IN M/SEC ,-17,5.0,90.0,)
     1 DV )
C
C     PLOT POINTS
      DO 1 I = 1 , K
      A(I)  =  A(I)  +  3.0
      V(I)  =  V(I)  +  3.0
      CALL SYMBOL  ( Q(I) , A(I) ,0.07 , 2 , 0.0 , -1 )
1     CALL SYMBOL  ( Q(I) , V(I) ,0.07 , 3 , 0.0 , -1 )
C
C     PLOT REGRESSION LINE
C
C     Q  VS.  A  LINE
      YB = (((AA + BA* QMIN) -AMIN ) /DA )  +3.0
      XB = 0.0
      IF ( YB .GE. 3.0 )  GO TO 101
      YB = 3.0
      XB =                  (AMIN - AA  - QMIN * BA) / (DQ * BA)
101   XE = 5.0
      YE = ((( AA + BA*( QMIN +5.0 * DQ)) -AMIN) / DA ) + 3.0
      IF ( YE  .LE .  8.0 )  GO  TO 102
      YE = 8. 0
      XE = (5.0 * DA + AMIN - AA - QMIN* BA) / (DQ * BA )
102   CALL PLOT ( XB , YB , + 3 )
      CALL PLOT ( XE , YE , + 2 )
C
C     Q VS V LINE
      XB = 0.0
      YB = ((( AV +  BV*QMIN ) -VMIN)  /DV ) + 3.0
      IF ( YB .GE. 3.0 )  GO TO 103
      YB = 3.0
      XB = ( VMIN - AV - QMIN * BV ) / (DQ * BV)
103   XE = 5.0
      YE = ((( AV + BV *(QMIN +5.0 * DQ))  -VMIN )/ DV ) + 3.0
      IF ( YE .LE. 8.0 )  GO TO 104
      YE = 8.0
      XE = ( 5.0 * DV + VMIN - AV -QMIN *BV ) / ( DQ * BV)
104   CALL PLOT ( XB , YB , +3 )
```

```
      CALL PLOT ( XE , YE , +2 )
C
C     TITLE  AND  LEGEND
      CALL SYMBOL  ( 0.5, 9.0, 0.21, TIT , 0.0 , 28 )
      CALL SYMBOL  ( 1.0, 1.6, 0.14,  2  , 0.0 , -1 )
      CALL SYMBOL  ( 1.0, 1.1, 0.14,  3  , 0.0 , -1 )
      CALL SYMBOL  ( 1.5, 1.5, 0.14,  9HFLOW AREA ,  0.0 , 9 )
      CALL SYMBOL  ( 1.5, 1.0, 0.14 , 9HVELOCITY  ,  0.0 , 9 )
C
C     COMPLETE OUTLINE , MOVE ON
      CALL PLOT ( 0.0 , 8.0 , +3 )
      CALL PLOT ( 5.0 , 8.0 , +2 )
      CALL PLOT ( 8.0 , 0.0 , -3 )
      RETURN
      END
```

BROCKTON CK, REACH 1-2

NO	ID	Q (L/S)	TS	TP	TM	A	V
2	1	9.20	17.00	25.50	30.10	0.1400	0.0660
4	1	5.60	24.00	37.30	44.50	0.1260	0.0450
7	0	0.67	110.00	180.00	193.00	0.0650	0.0103
9	0	0.16	350.00	540.00	560.00	0.0450	0.0035
10	1	1.23	70.00	117.00	159.50	0.0990	0.0125
16	1	58.00	4.90	8.10	9.08	0.2660	0.2180
18	1	50.00	7.20	11.60	12.58	0.3170	0.1580
19	1	158.00	2.50	5.00	5.40	0.4300	0.3680
21	1	92.00	4.20	7.30	8.33	0.3860	0.2380
23	1	137.00	3.20	5.50	6.97	0.4810	0.2850

LOG TM VS. LOG Q

SSX 10.1903 SSY 4.5008 SP -6.7536

REGRESSION EQUATION Y= 0.2056 + -0.6628 X
STANDARD ERROR OF ESTIMATE 0.0558
CORRELATION COEFFICIENT -0.9972
DEGREE OF FREEDOM 9
F = 1439.3098

'LØGRE'

COMPLETE PRINTOUT

LOG A VS. LOG Q

SSX 10.1903 SSY 1.1856 SP 3.4399

REGRESSION EQUATION Y= -0.0913 + 0.3376 X
STANDARD ERROR OF ESTIMATE 0.0553
CORRELATION COEFFICIENT 0.9896
DEGREE OF FREEDOM 9
F = 379.7510

LOG V VS. LOG Q

SSX 10.1903 SSY 4.5095 SP 6.7599

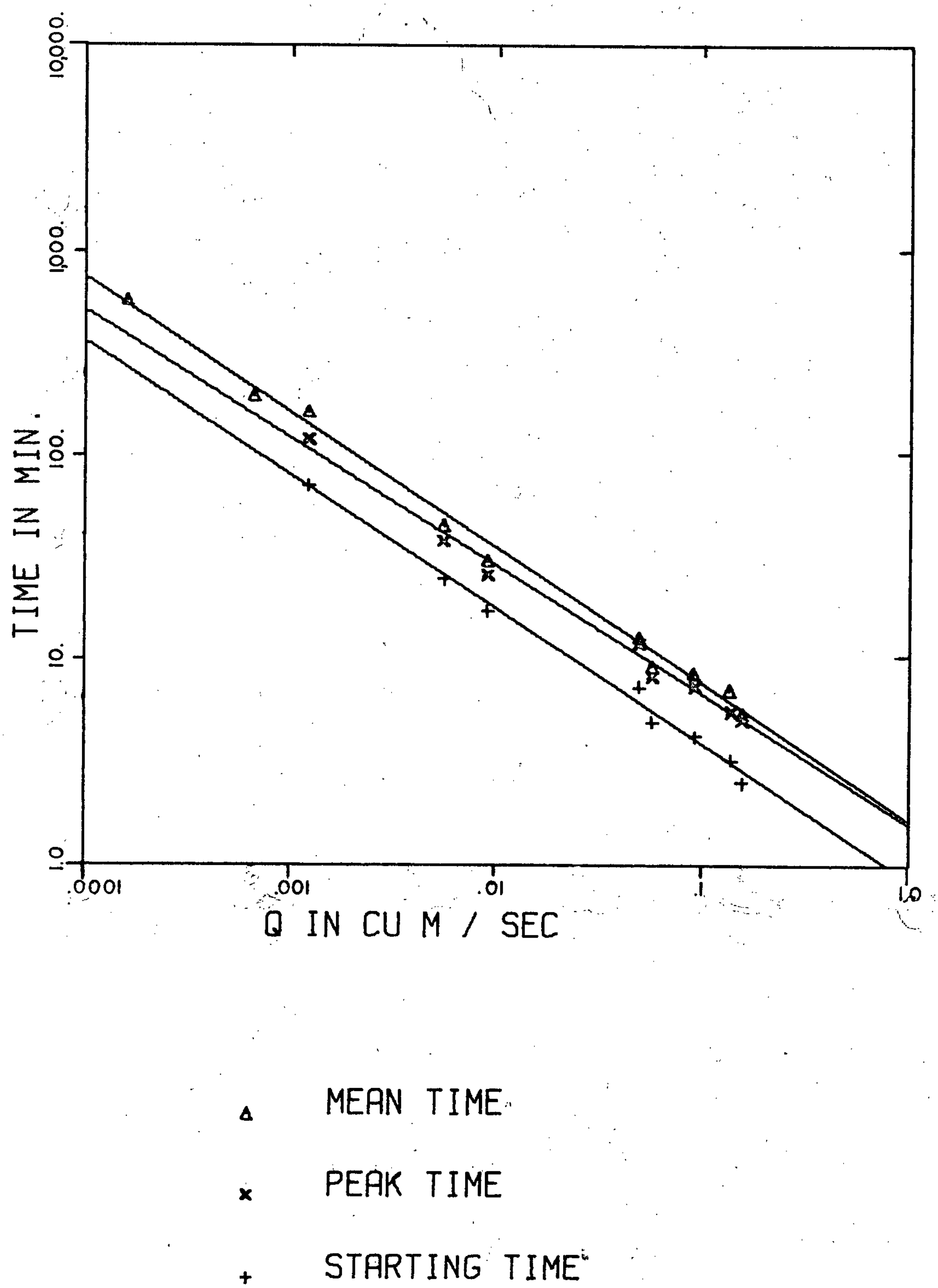
REGRESSION EQUATION Y= 0.0934 + 0.6634 X
STANDARD ERROR OF ESTIMATE 0.0561
CORRELATION COEFFICIENT 0.9972
DEGREE OF FREEDOM 9
F = 1423.0298

LOG TS VS. LOG Q

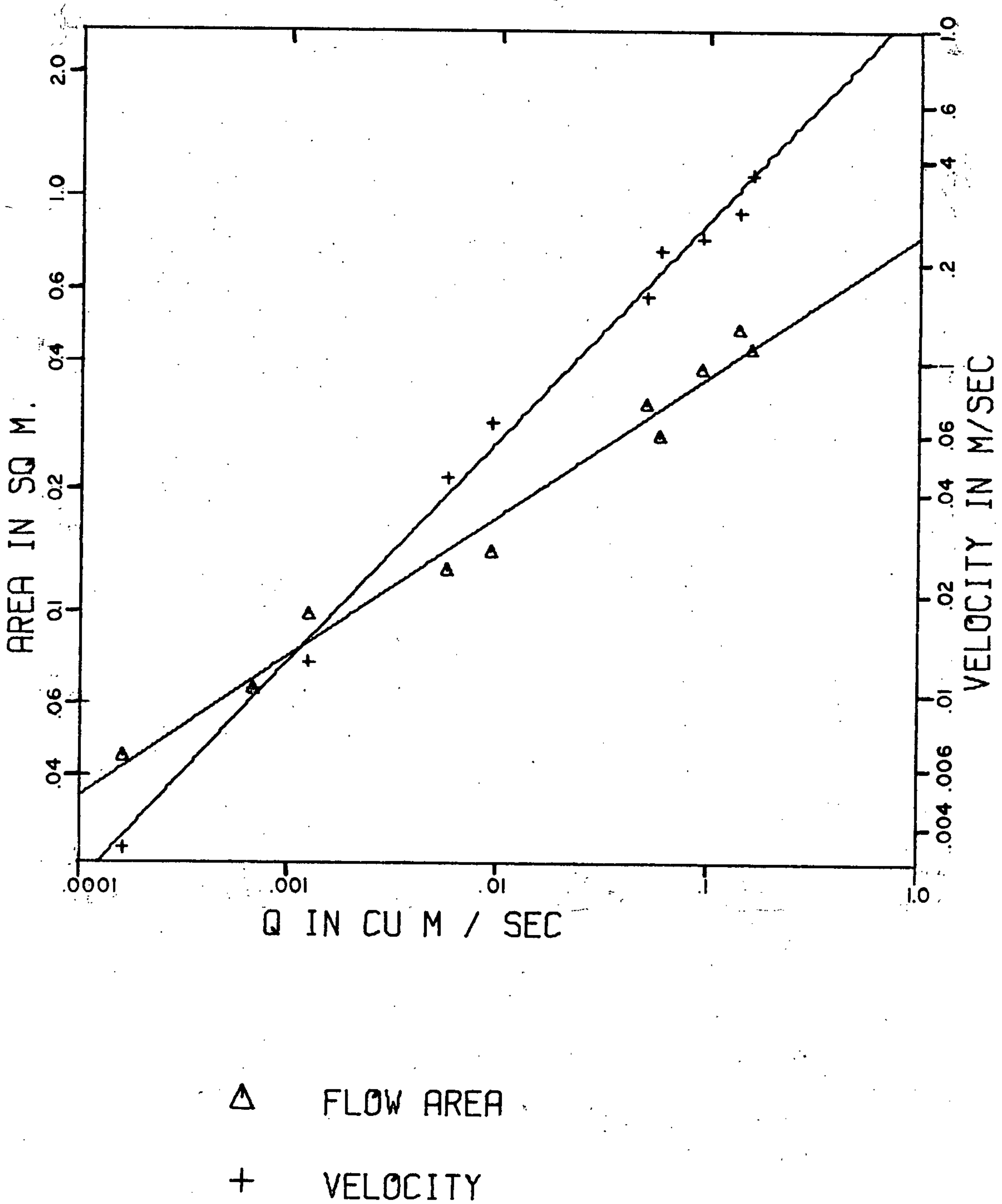
SSX 10.1903 SSY 4.7987 SP -6.9778

```
REGRESSION EQUATION  Y=      -0.1173 +   -0.6847 X
STANDARD ERROR OF ESTIMATE    0.0509
CORRELATION COEFFICIENT   -0.9978
DEGREE OF FREEDOM               9
F =                     1841.9011


LOG TP VS. LOG Q


SSX       10.1903      SSY       4.5419      SP      -6.7840


REGRESSION EQUATION  Y=       0.1390 +   -0.6657 X
STANDARD ERROR OF ESTIMATE    0.0566
CORRELATION COEFFICIENT   -0.9972
DEGREE OF FREEDOM               9
F =                     1412.1279


LOG TSS VS. LOG Q


SSX        4.0349      SSY       1.7498      SP      -2.6472


REGRESSION EQUATION  Y=      -0.0787 +   -0.6561 X
STANDARD ERROR OF ESTIMATE    0.0466
CORRELATION COEFFICIENT   -0.9963
DEGREE OF FREEDOM               7
F =                      798.5081


LOG TPP VS. LOG Q


SSX        4.0349      SSY       1.6039      SP      -2.5322


REGRESSION EQUATION  Y=       0.1896 +   -0.6276 X
STANDARD ERROR OF ESTIMATE    0.0496
CORRELATION COEFFICIENT   -0.9954
DEGREE OF FREEDOM               7
F =                      646.0571
```

BROCKTON CK. REACH 1-2



BROCKTON CK. REACH 1-2



BROCKTON CK, REACH 2-3

NO	ID	Q (L/S)	TS	TP	TM	A	V
1	1	6.70	13.60	33.00	35.00	0.1740	0.0390
4	0	6.80	27.00	34.50	34.17	0.1720	0.0400
8	0	0.73	76.00	160.00	172.00	0.0935	0.0078
11	1	1.20	53.00	97.00	119.80	0.1065	0.0113
17	1	54.00	3.00	7.20	8.50	0.3400	0.1590
20	1	89.00	2.20	4.70	5.39	0.3560	0.2500
22	1	110.00	1.80	4.00	4.53	0.3690	0.2980

LOG TM VS. LOG Q

SSX 4.7054 SSY 2.4174 SP -3.3716

REGRESSION EQUATION Y= -0.0129 + -0.7165 X
STANDARD ERROR OF ESTIMATE 0.0179
CORRELATION COEFFICIENT -0.9997
DEGREE OF FREEDOM 6
F = 7506.9609

LOG A VS. LOG Q

SSX 4.7054 SSY 0.3784 SP 1.3316

REGRESSION EQUATION Y= -0.1437 + 0.2830 X
STANDARD ERROR OF ESTIMATE 0.0179
CORRELATION COEFFICIENT 0.9979
DEGREE OF FREEDOM 6
F = 1182.4143

LOG V VS. LOG Q

SSX 4.7054 SSY 2.4167 SP 3.3710

REGRESSION EQUATION Y= 0.1442 + 0.7164 X
STANDARD ERROR OF ESTIMATE 0.0185
CORRELATION COEFFICIENT 0.9996
DEGREE OF FREEDOM 6
F = 7072.7969

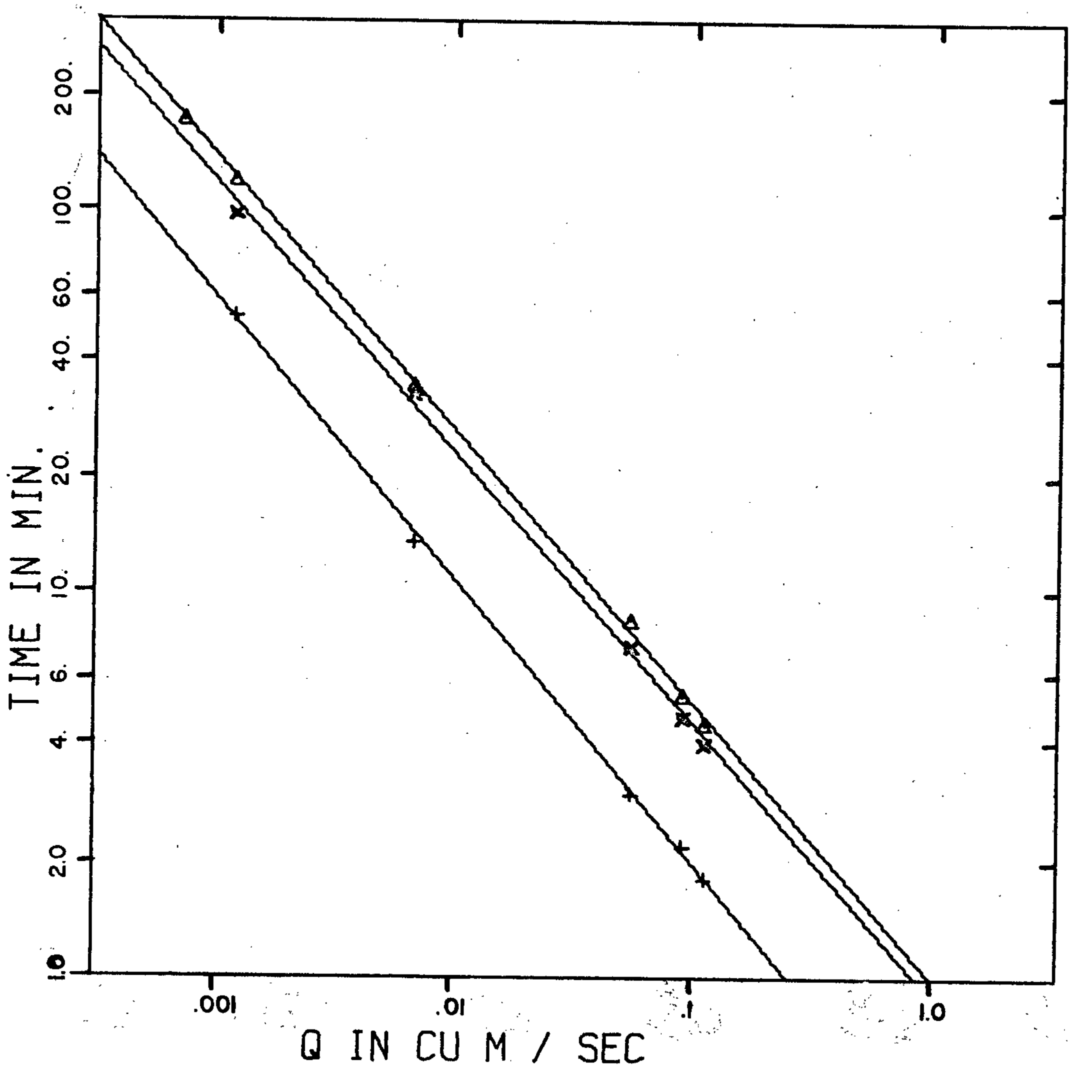
LOG TS VS. LOG Q

SSX 4.7054 SSY 2.7562 SP -3.5577

REGRESSION EQUATION Y= -0.4422 + -0.7561 X
STANDARD ERROR OF ESTIMATE 0.1151

```
CORRELATION COEFFICIENT    -0.9879
DEGREE OF FREEDOM                6
F =                       203.1117


LOG TP VS. LOG Q


SSX       4.7054      SSY       2.4596      SP      -3.3980


REGRESSION EQUATION  Y=      -0.0710 +    -0.7221 X
STANDARD ERROR OF ESTIMATE     0.0342
CORRELATION COEFFICIENT    -0.9988
DEGREE OF FREEDOM                6
F =                      2102.5312


LOG TSS VS. LOG Q


SSX       2.8700      SSY       1.5750      SP      -2.1253


REGRESSION EQUATION  Y=      -0.4533 +    -0.7405 X
STANDARD ERROR OF ESTIMATE     0.0194
CORRELATION COEFFICIENT    -0.9996
DEGREE OF FREEDOM                4
F =                      4198.3047


LOG TPP VS. LOG Q


SSX       2.8700      SSY       1.4477      SP      -2.0364


REGRESSION EQUATION  Y=      -0.0606 +    -0.7095 X
STANDARD ERROR OF ESTIMATE     0.0305
CORRELATION COEFFICIENT    -0.9990
DEGREE OF FREEDOM                4
F =                      1557.8159
```

BROCKTON CK. REACH 2-3

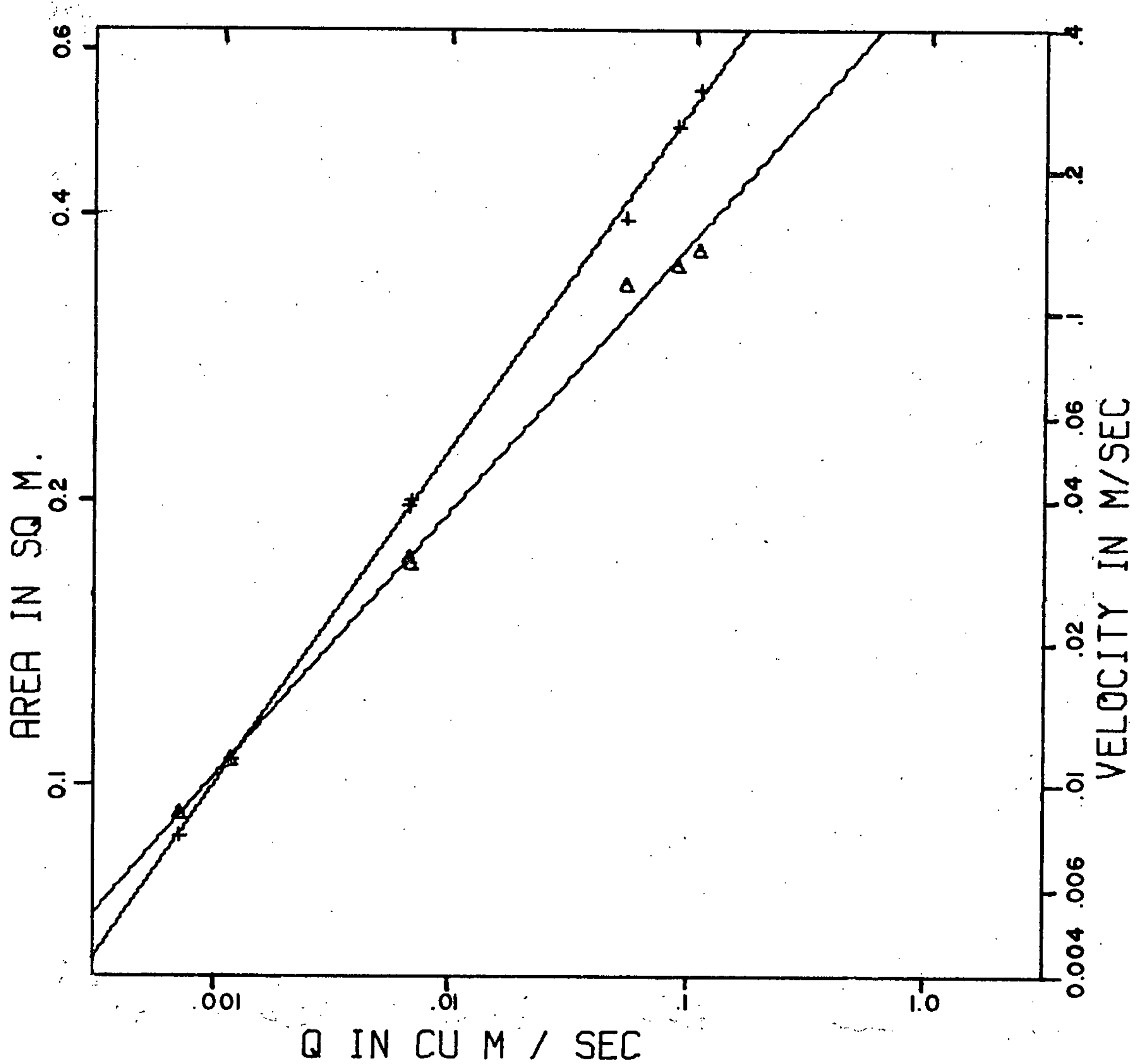


△ MEAN TIME

× PEAK TIME

+ STARTING TIME

BROCKTON CK. REACH 2-3



Δ FLOW AREA

\+ VELOCITY

PLACID CREEK, REACH 1-2

NO	ID	Q (L/S)	TS	TP	TM	A	V
2	1	50.30	122.00	164.00	210.00	0.6600	0.0763
13	1	35.40	165.00	208.00	280.60	0.6180	0.0573
25	1	64.60	115.00	159.00	180.60	0.7280	0.0890
28	1	95.90	94.00	130.00	144.20	0.8630	0.1110

LOG TM VS. LOG Q

SSX 0.0997 SSY 0.0441 SP -0.0660

REGRESSION EQUATION Y= 1.4758 + -0.6622 X
STANDARD ERROR OF ESTIMATE 0.0147
CORRELATION COEFFICIENT -0.9951
DEGREE OF FREEDOM 3
F = 203.3226

LOG A VS. LOG Q

SSX 0.0997 SSY 0.0119 SP 0.0339

REGRESSION EQUATION Y= 0.2739 + 0.3403 X
STANDARD ERROR OF ESTIMATE 0.0141
CORRELATION COEFFICIENT 0.9833
DEGREE OF FREEDOM 3
F = 58.3338

LOG V VS. LOG Q

SSX 0.0997 SSY 0.0437 SP 0.0656

REGRESSION EQUATION Y= -0.2748 + 0.6587 X
STANDARD ERROR OF ESTIMATE 0.0147
CORRELATION COEFFICIENT 0.9950
DEGREE OF FREEDOM 3
F = 199.5172

LOG TSS VS. LOG Q

SSX 0.0997 SSY 0.0307 SP -0.0541

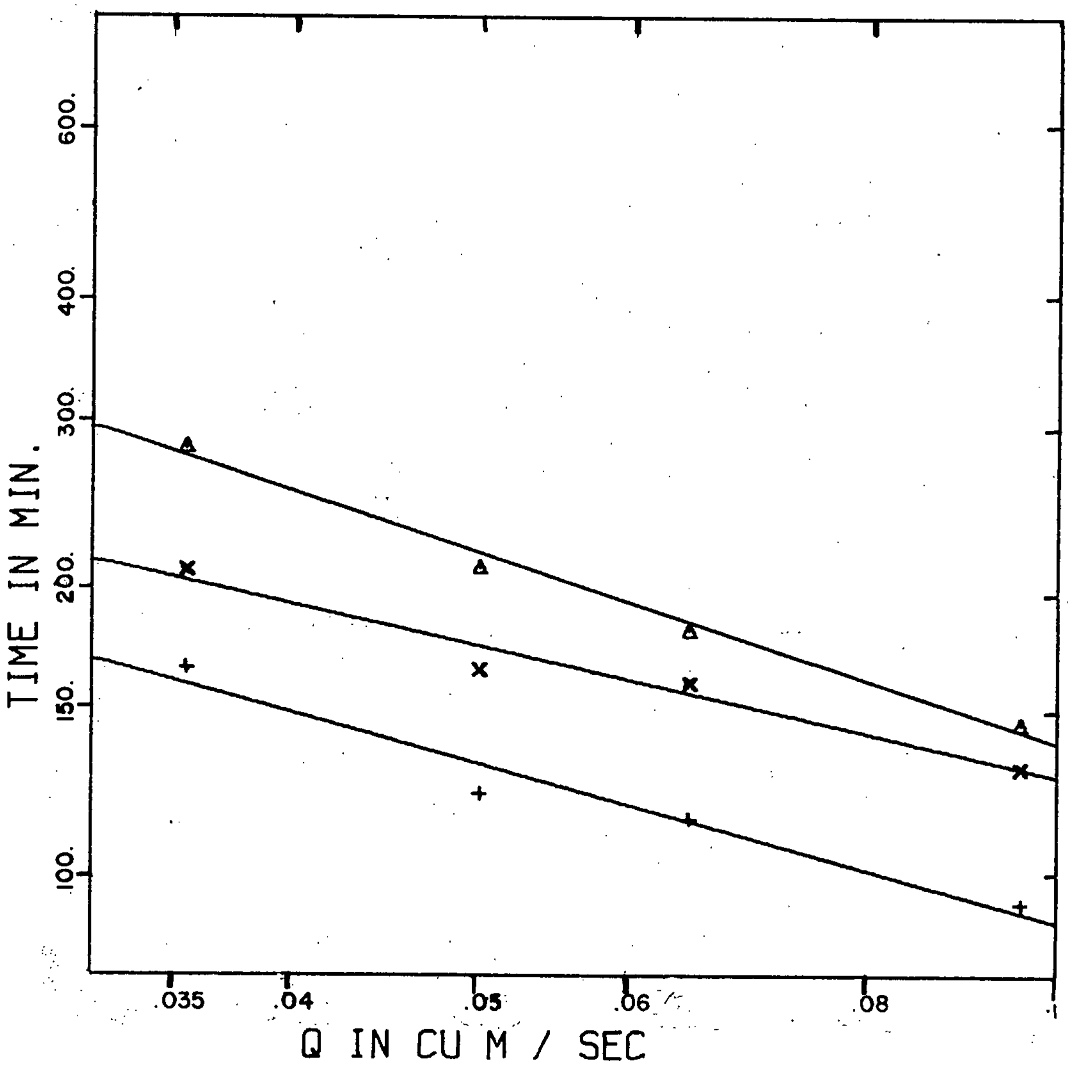
REGRESSION EQUATION Y= 1.4121 + -0.5425 X
STANDARD ERROR OF ESTIMATE 0.0257
CORRELATION COEFFICIENT -0.9782
DEGREE OF FREEDOM 3
F = 44.3011

LOG TPP VS. LOG Q

SSX	0.0997	SSY	0.0210	SP	-0.0448

REGRESSION EQUATION Y= 1.6546 + -0.4498 X
STANDARD ERROR OF ESTIMATE 0.0202
CORRELATION COEFFICIENT -0.9804
DEGREE OF FREEDOM 3
F = 49.3988

PLACID CREEK. REACH 1-2

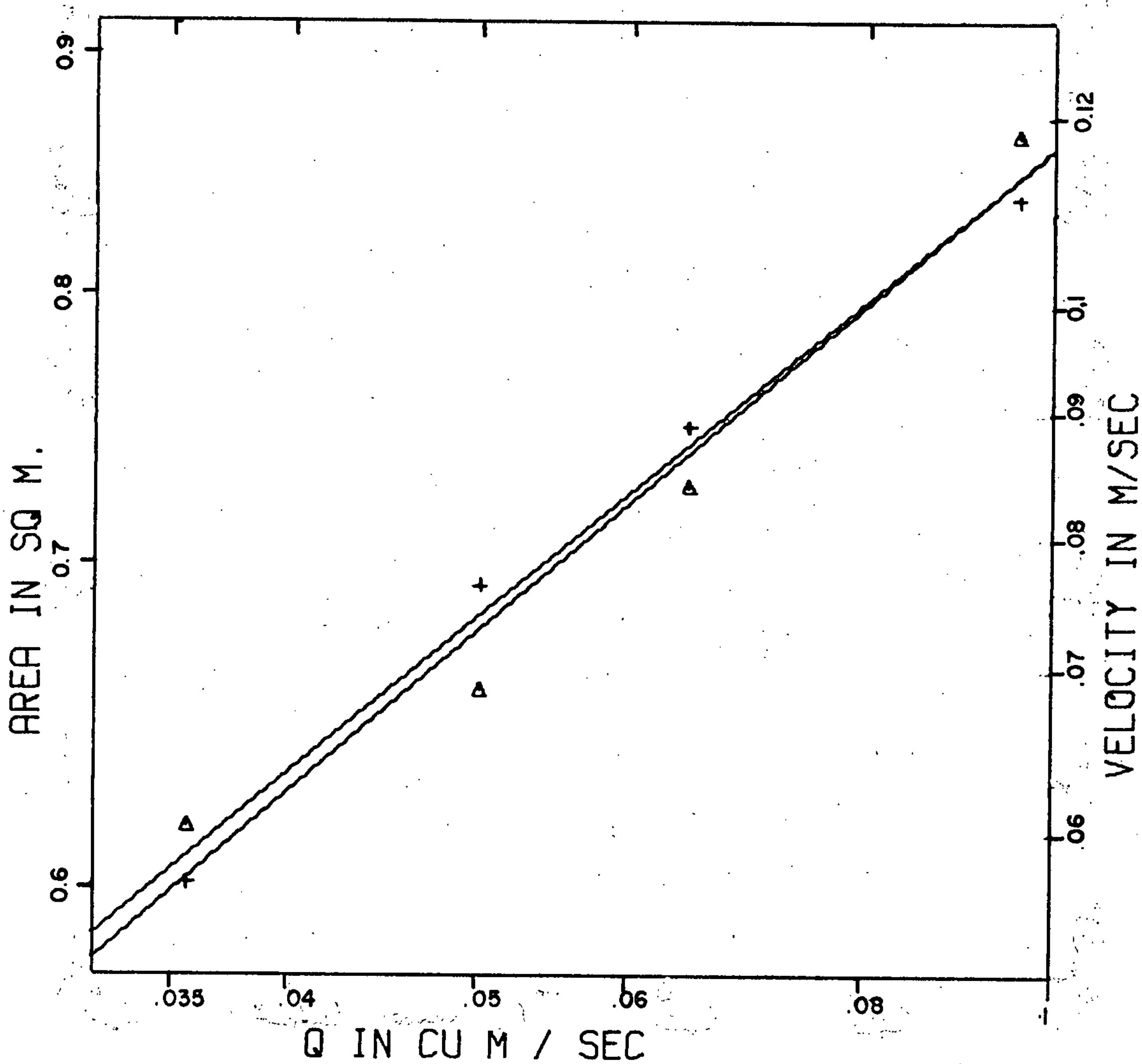


△ MEAN TIME

× PEAK TIME

+ STARTING TIME

PLACID CREEK, REACH 1-2



Δ FLOW AREA

\+ VELOCITY

PLACID CREEK, REACH 2-3

NO	ID	Q (L/S)	TS	TP	TM	A	V
5	1	15.40	160.00	236.00	351.30	0.5340	0.0289
10	1	72.60	62.00	94.00	121.40	0.8690	0.0834
18	1	114.50	48.00	71.00	83.30	0.9390	0.1220
24	1	121.20	47.50	74.00	92.10	1.1000	0.1100
27	1	181.00	36.10	54.80	62.10	1.1100	0.1630

LOG TM VS. LOG Q

SSX 0.6955 SSY 0.3363 SP -0.4821

REGRESSION EQUATION Y= 1.2919 + -0.6932 X
STANDARD ERROR OF ESTIMATE 0.0265
CORRELATION COEFFICIENT -0.9969
DEGREE OF FREEDOM 4
F = 477.4194

LOG A VS. LOG Q

SSX 0.6955 SSY 0.0674 SP 0.2131

REGRESSION EQUATION Y= 0.2855 + 0.3065 X
STANDARD ERROR OF ESTIMATE 0.0266
CORRELATION COEFFICIENT 0.9842
DEGREE OF FREEDOM 4
F = 92.4102

LOG V VS. LOG Q

SSX 0.6955 SSY 0.3356 SP 0.4815

REGRESSION EQUATION Y= -0.2868 + 0.6924 X
STANDARD ERROR OF ESTIMATE 0.0270
CORRELATION COEFFICIENT 0.9967
DEGREE OF FREEDOM 4
F = 456.2595

LOG TSS VS. LOG Q

SSX 0.6955 SSY 0.2499 SP -0.4168

REGRESSION EQUATION Y= 1.1170 + -0.5993 X
STANDARD ERROR OF ESTIMATE 0.0074
CORRELATION COEFFICIENT -0.9997
DEGREE OF FREEDOM 4

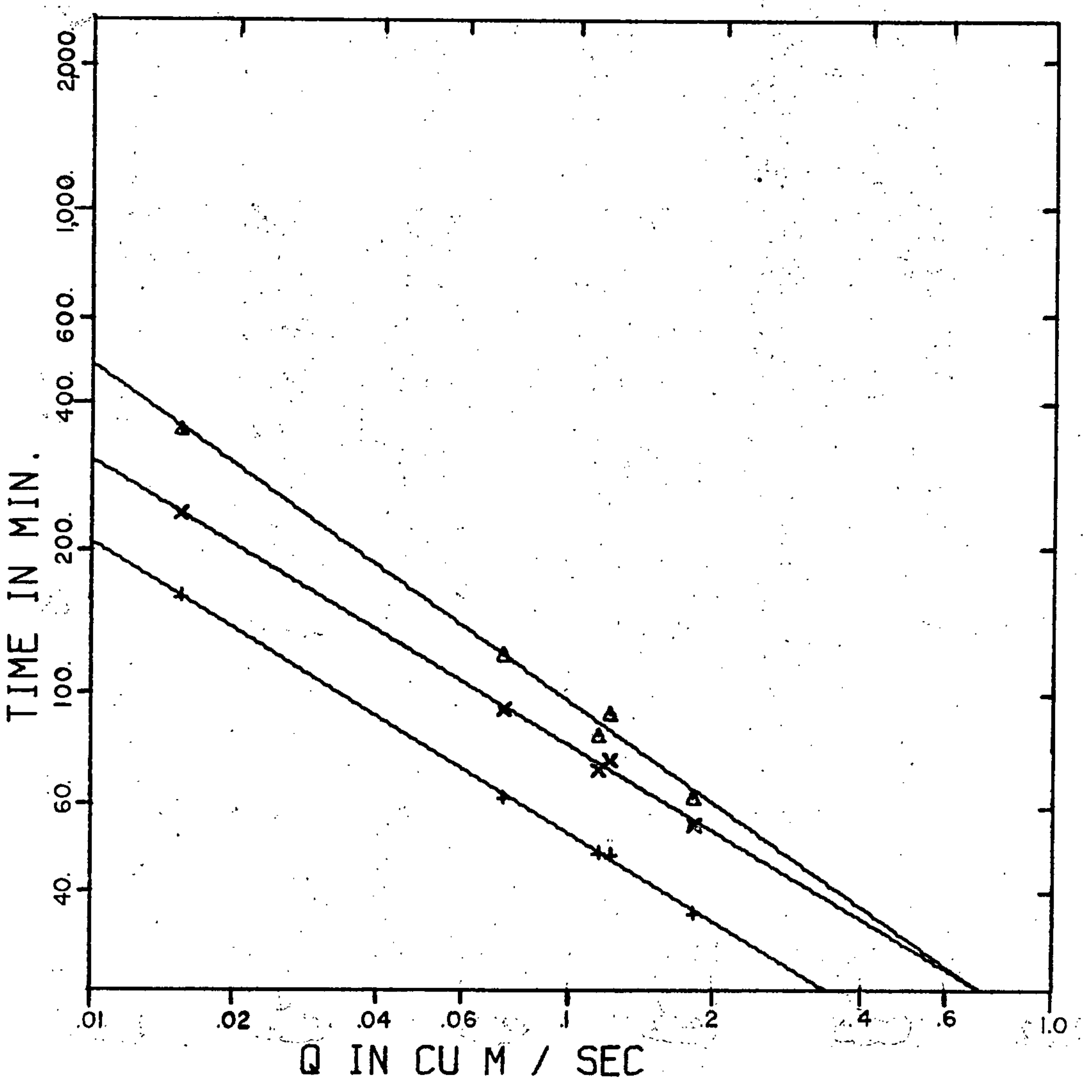
F = 4588.8828

LOG TPP VS. LOG Q

SSX 0.6955 SSY 0.2397 SP -0.4077

REGRESSION EQUATION Y= 1.3101 + -0.5862 X
STANDARD ERROR OF ESTIMATE 0.0146
CORRELATICN COEFFICIENT -0.9987
DEGREE OF FREEDOM 4
F = 1118.6213

PLACID CREEK. REACH 2-3

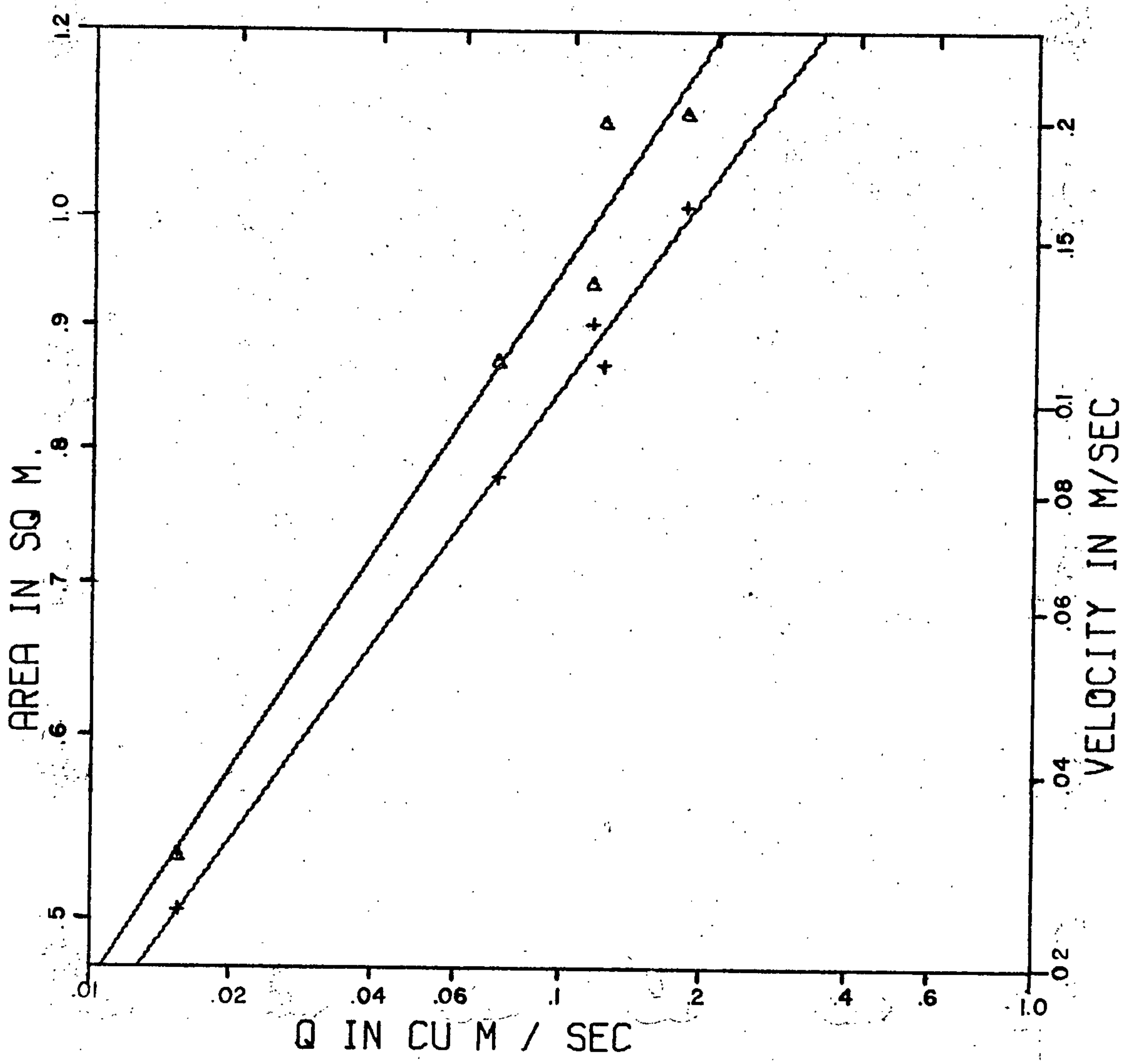


△ MEAN TIME

× PEAK TIME

\+ STARTING TIME

PLACID CREEK. REACH 2-3



Δ FLOW AREA

+ VELOCITY

PLACID CREEK, REACH 3-4

NO	ID	Q (L/S)	TS	TP	TM	A	V
17	1	14.50	674.00	1000.00	1200.00	0.5650	0.0256
18	1	245.00	188.00	223.00	255.30	2.0300	0.1210
19	1	268.00	162.00	199.00	238.90	2.0000	0.1340
21	1	70.00	302.00	386.00	481.60	1.0950	0.0638
24	1	267.00	157.50	200.00	254.00	2.2000	0.1210
27	1	404.00	121.00	169.00	209.00	2.7400	0.1480

LOG TM VS. LOG Q

SSX 1.4894 SSY 0.4213 SP -0.7908

REGRESSION EQUATION Y= 2.0902 + -0.5310 X
STANDARD ERROR OF ESTIMATE 0.0187
CORRELATION COEFFICIENT -0.9983
DEGREE OF FREEDOM 5
F = 1203.0806

LOG A VS. LOG Q

SSX 1.4894 SSY 0.3250 SP 0.6934

REGRESSION EQUATION Y= 0.5958 + 0.4656 X
STANDARD ERROR OF ESTIMATE 0.0232
CORRELATION COEFFICIENT 0.9967
DEGREE OF FREEDOM 5
F = 599.9749

LOG V VS. LOG Q

SSX 1.4894 SSY 0.4296 SP 0.7980

REGRESSION EQUATION Y= -0.5947 + 0.5358 X
STANDARD ERROR OF ESTIMATE 0.0230
CORRELATION COEFFICIENT 0.9975
DEGREE OF FREEDOM 5
F = 810.3389

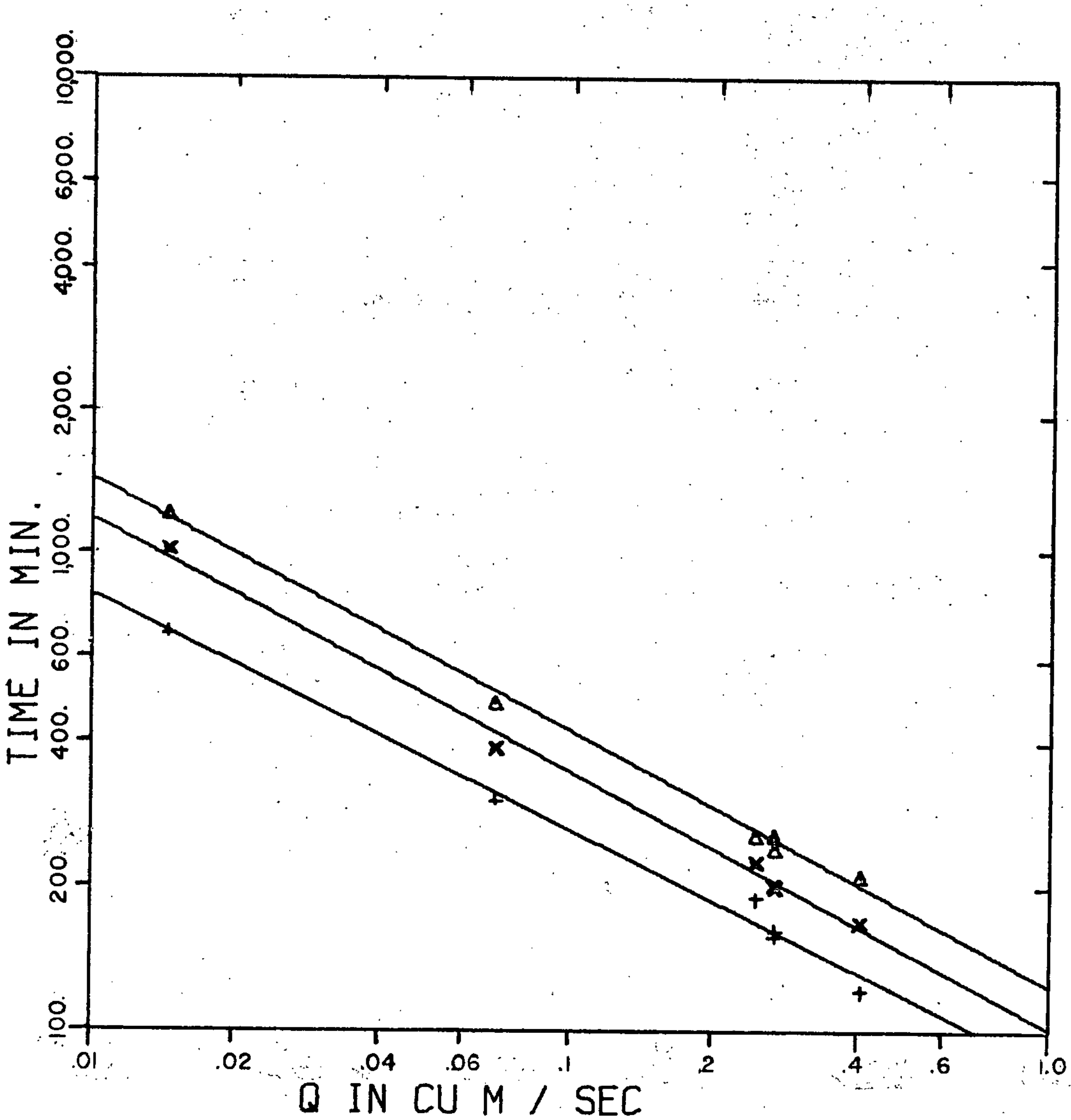
LOG TSS VS. LOG Q

SSX 1.4894 SSY 0.3661 SP -0.7345

REGRESSION EQUATION Y= 1.9227 + -0.4931 X
STANDARD ERROR OF ESTIMATE 0.0312
CORRELATION COEFFICIENT -0.9947

```
DEGREE OF FREEDOM                   5
F =                          372.5400


LOG TPP VS. LOG Q

SSX       1.4894    SSY        0.4252    SP      -0.7939

REGRESSION EQUATION  Y=        2.0035 +   -0.5330 X
STANDARD ERROR OF ESTIMATE      0.0226
CORRELATION COEFFICIENT     -0.9976
DEGREE OF FREEDOM                   5
F =                          826.4009
PLOT TAPE SUCCESSFULLY WRITTEN
DONE

 STOP      0
EXECUTION TERMINATED


$SIG
```

PLACID CREEK. REACH 3-4

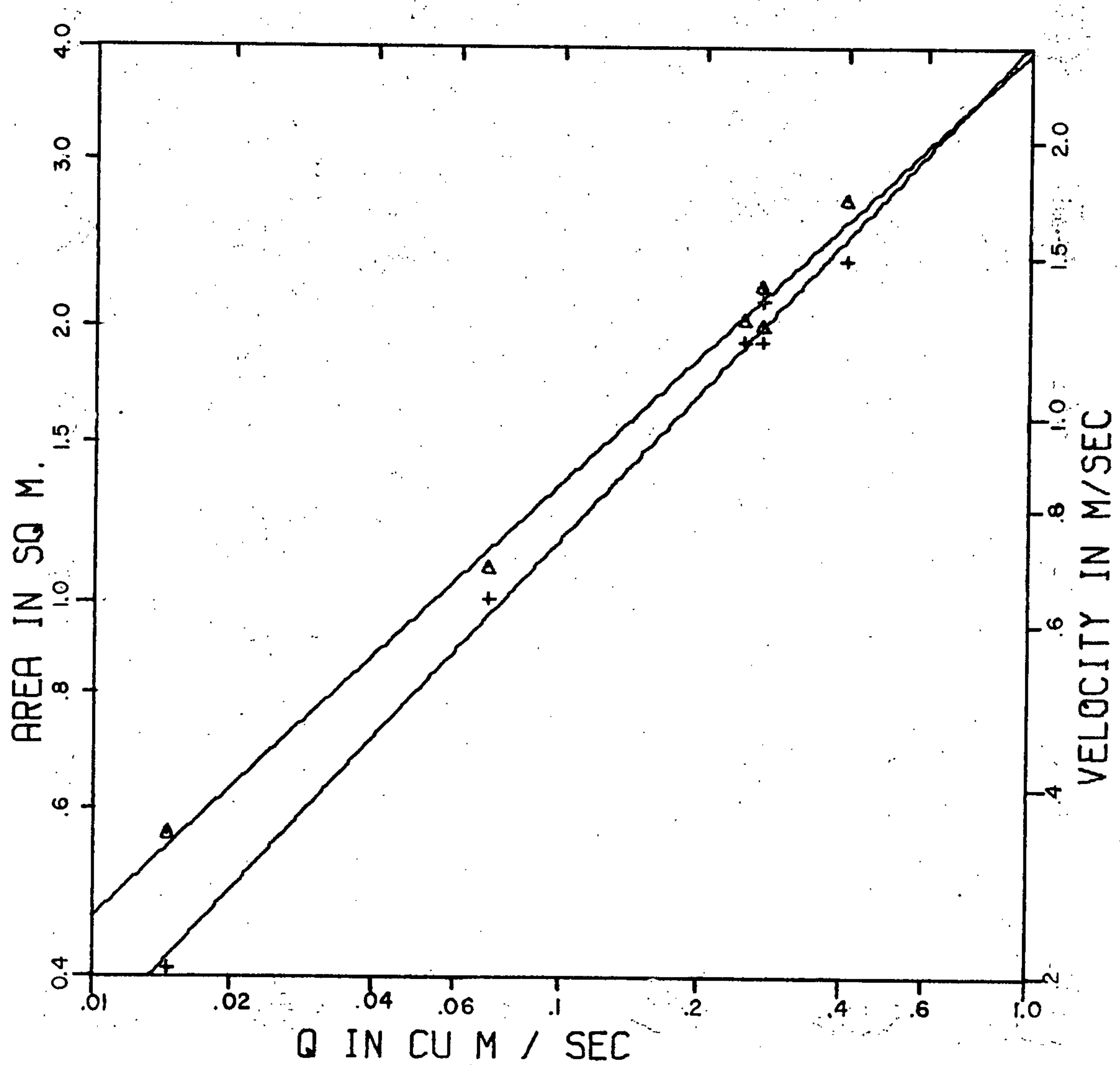


△ MEAN TIME

× PEAK TIME

+ STARTING TIME

PLACID CREEK. REACH 3-4



Δ FLOW AREA

+ VELOCITY

BLANEY CREEK, REACH 1-3

NO	ID	Q (L/S)	TS	TP	TM	A	V
11	1	500.00	30.00	44.50	54.70	2.3900	0.2090
12	1	1750.00	17.00	23.50	27.48	4.2200	0.4160
15	1	11500.00	7.50	10.60	12.10	12.1500	0.9470
16	1	1630.00	19.50	26.40	30.16	4.3200	0.3800
17	1	1600.00	18.30	25.50	29.63	4.2500	0.3870
19	1	1950.00	15.00	22.50	25.37	4.3200	0.4510
24	1	120.00	71.00	101.00	123.00	1.2900	0.0960
31	1	146.00	57.00	90.00	117.00	1.4900	0.0980
35	1	285.00	40.50	61.00	71.20	1.7700	0.1550
36	1	741.00	24.00	34.00	40.40	2.6200	0.2830

LOG TM VS. LOG Q

SSX 3.2537 SSY 0.8994 SP -1.7018

REGRESSION EQUATION Y= 1.5854 + -0.5230 X
STANDARD ERROR OF ESTIMATE 0.0340
CORRELATION COEFFICIENT -0.9948
DEGREE OF FREEDOM 9
F = 770.1775

LOG A VS. LOG Q

SSX 3.2537 SSY 0.7537 SP 1.5566

REGRESSION EQUATION Y= 0.5283 + 0.4784 X
STANDARD ERROR OF ESTIMATE 0.0335
CORRELATION COEFFICIENT 0.9940
DEGREE OF FREEDOM 9
F = 661.6221

LOG V VS. LOG Q

SSX 3.2537 SSY 0.8945 SP 1.6981

REGRESSION EQUATION Y= -0.5270 + 0.5219 X
STANDARD ERROR OF ESTIMATE 0.0321
CORRELATION COEFFICIENT 0.9954
DEGREE OF FREEDOM 9
F = 858.6084

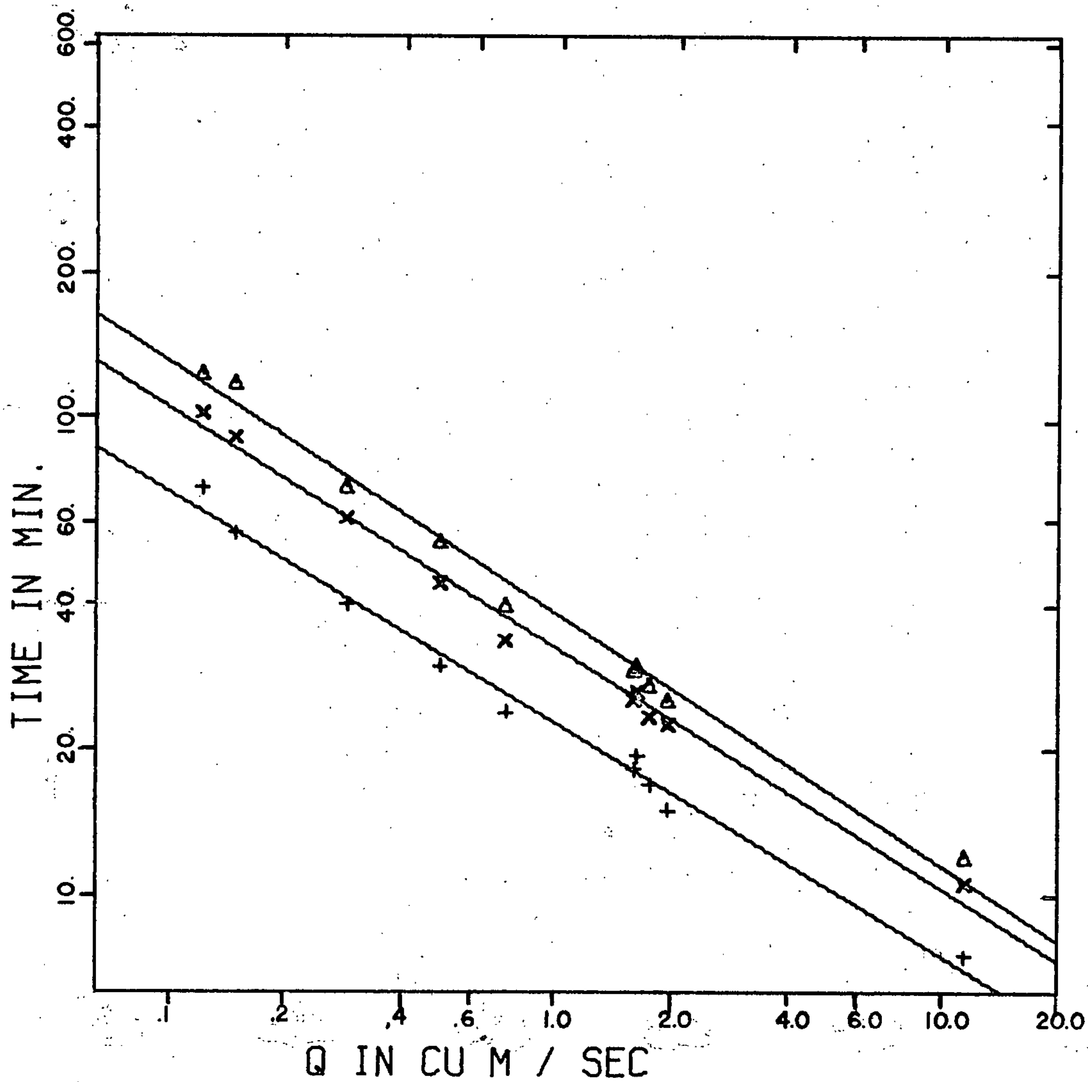
LOG TSS VS. LOG Q

SSX 3.2537 SSY 0.7642 SP -1.5678

```
REGRESSION EQUATION  Y=       1.3558 +   -0.4819 X
STANDARD ERROR OF ESTIMATE    0.0331
CORRELATION COEFFICIENT    -0.9943
DEGREE OF FREEDOM                 9
F =                        690.2686
```

```
LOG TPP VS. LOG Q

SSX      3.2537     SSY       0.8187     SP      -1.6255

REGRESSION EQUATION  Y=       1.5139 +   -0.4996 X
STANDARD ERROR OF ESTIMATE    0.0287
CORRELATION COEFFICIENT    -0.9960
DEGREE OF FREEDOM                 9
F =                        985.6753
```

BLANEY CREEK, REACH 1-3

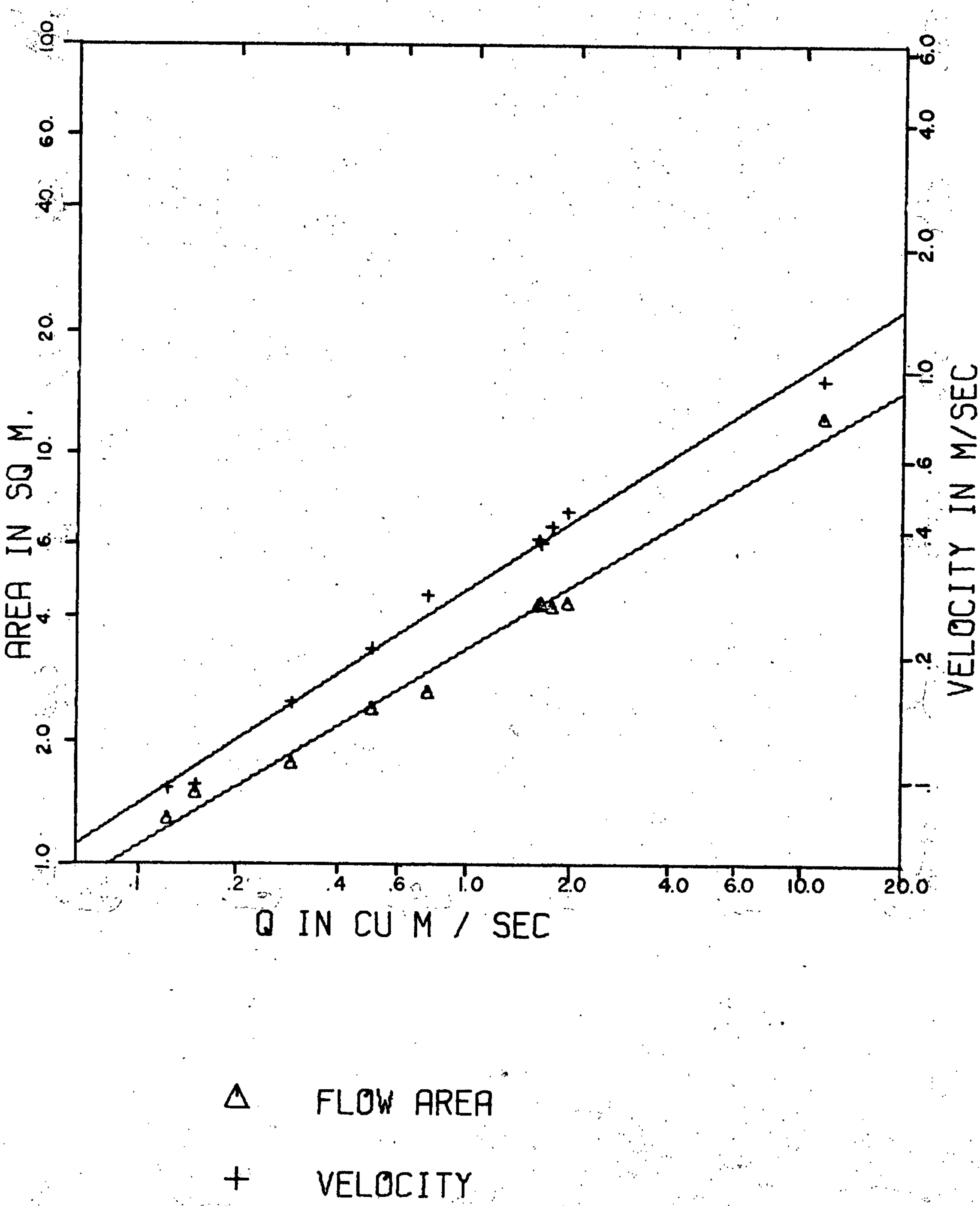


△ MEAN TIME

× PEAK TIME

\+ STARTING TIME

BLANEY CREEK. REACH 1-3



BLANEY CREEK, REACH 3-5

NO	ID	Q (L/S)	TS	TP	TM	A	V
8	1	870.00	8.50	16.20	21.15	3.3000	0.2640
10	1	530.00	12.00	20.50	26.31	2.5000	0.2120
11	0	520.00	21.00	23.00	22.55	2.1400	0.2430
12	0	1820.00	11.00	15.00	15.67	5.0800	0.3580
13	1	10400.00	3.30	5.60	7.10	13.2000	0.7100
14	1	11700.00	3.20	5.50	6.24	13.0000	0.9000
15	0	11700.00	4.00	5.00	6.45	13.5000	0.8700
16	0	1650.00	12.00	14.00	17.62	5.2100	0.3170
19	0	2050.00	11.00	12.00	12.62	4.6300	0.4430
20	1	530.00	10.00	18.20	24.78	2.3500	0.2260
22	1	160.00	20.00	36.00	45.50	1.3000	0.1230
25	1	682.00	9.50	16.60	20.85	2.5500	0.2680
26	1	262.00	16.00	26.00	34.20	1.6000	0.1640
29	1	140.00	21.20	38.50	47.85	1.2000	0.1160
32	1	80.00	27.50	53.00	71.43	1.0200	0.0780
33	1	80.00	29.00	54.50	71.45	1.0200	0.0780
34	1	748.00	8.90	15.00	19.50	2.6200	0.2850
37	1	1350.00	7.50	12.70	15.46	3.7300	0.3620

LOG TM VS. LOG Q

SSX 7.8009 SSY 1.7186 SP -3.6332

REGRESSION EQUATION Y= 1.2862 + -0.4657 X
STANDARD ERROR OF ESTIMATE 0.0407
CORRELATION COEFFICIENT -0.9923
DEGREE OF FREEDOM 17
F = 1023.4202

LOG A VS. LOG Q

SSX 7.8009 SSY 2.2487 SP 4.1651

REGRESSION EQUATION Y= 0.5391 + 0.5339 X
STANDARD ERROR OF ESTIMATE 0.0394
CORRELATION COEFFICIENT 0.9945
DEGREE OF FREEDOM 17
F = 1433.6624

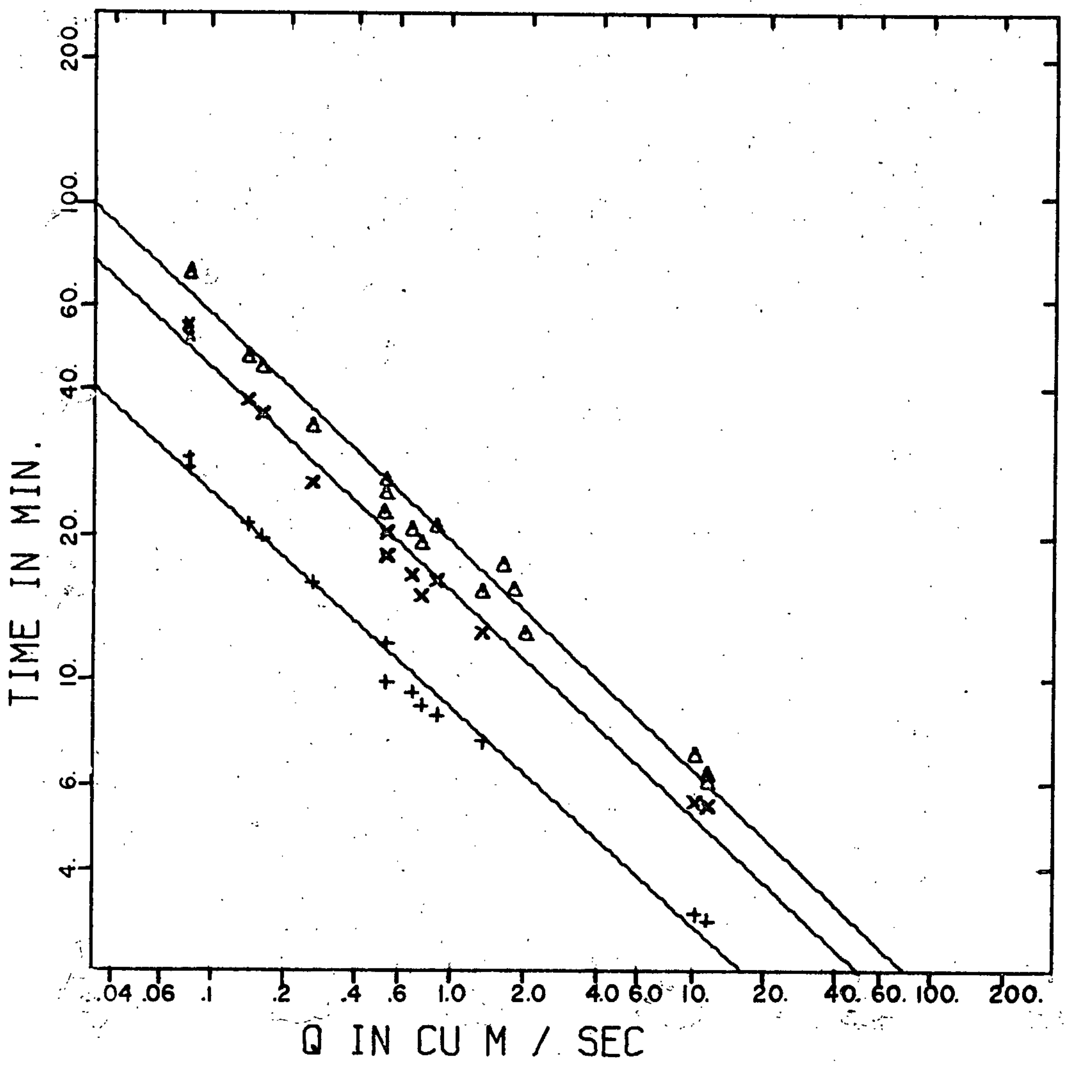
LOG V VS. LOG Q

SSX 7.8009 SSY 1.6867 SP 3.5944

REGRESSION EQUATION Y= -0.5423 + 0.4608 X
STANDARD ERROR OF ESTIMATE 0.0437
CORRELATION COEFFICIENT 0.9909
DEGREE OF FREEDOM 17

```
F =                              687.9802


LOG TS VS. LOG Q

SSX       7.5619      SSY        1.4003      SP      -3.0375

REGRESSION EQUATION  Y=          1.0047 +    -0.4017 X
STANDARD ERROR OF ESTIMATE        0.1061
CORRELATION COEFFICIENT      -0.9334
DEGREE OF FREEDOM                   17
F =                              108.3378


LOG TP VS. LOG Q

SSX       7.5619      SSY        1.5980      SP      -3.4337

REGRESSION EQUATION  Y=          1.2003 +    -0.4541 X
STANDARD ERROR OF ESTIMATE        0.0493
CORRELATION COEFFICIENT      -0.9878
DEGREE OF FREEDOM                   17
F =                              641.9316


LOG TSS VS. LOG Q

SSX       5.5511      SSY        1.1274      SP      -2.4827

REGRESSION EQUATION  Y=          0.9407 +    -0.4472 X
STANDARD ERROR OF ESTIMATE        0.0394
CORRELATION COEFFICIENT      -0.9924
DEGREE OF FREEDOM                   12
F =                              716.6177


LOG TPP VS. LOG Q

SSX       5.5511      SSY        1.2177      SP      -2.5748

REGRESSION EQUATION  Y=          1.1846 +    -0.4638 X
STANDARD ERROR OF ESTIMATE        0.0461
CORRELATION COEFFICIENT      -0.9904
DEGREE OF FREEDOM                   12
F =                              561.8801
```

BLANEY CREEK. REACH 3-5

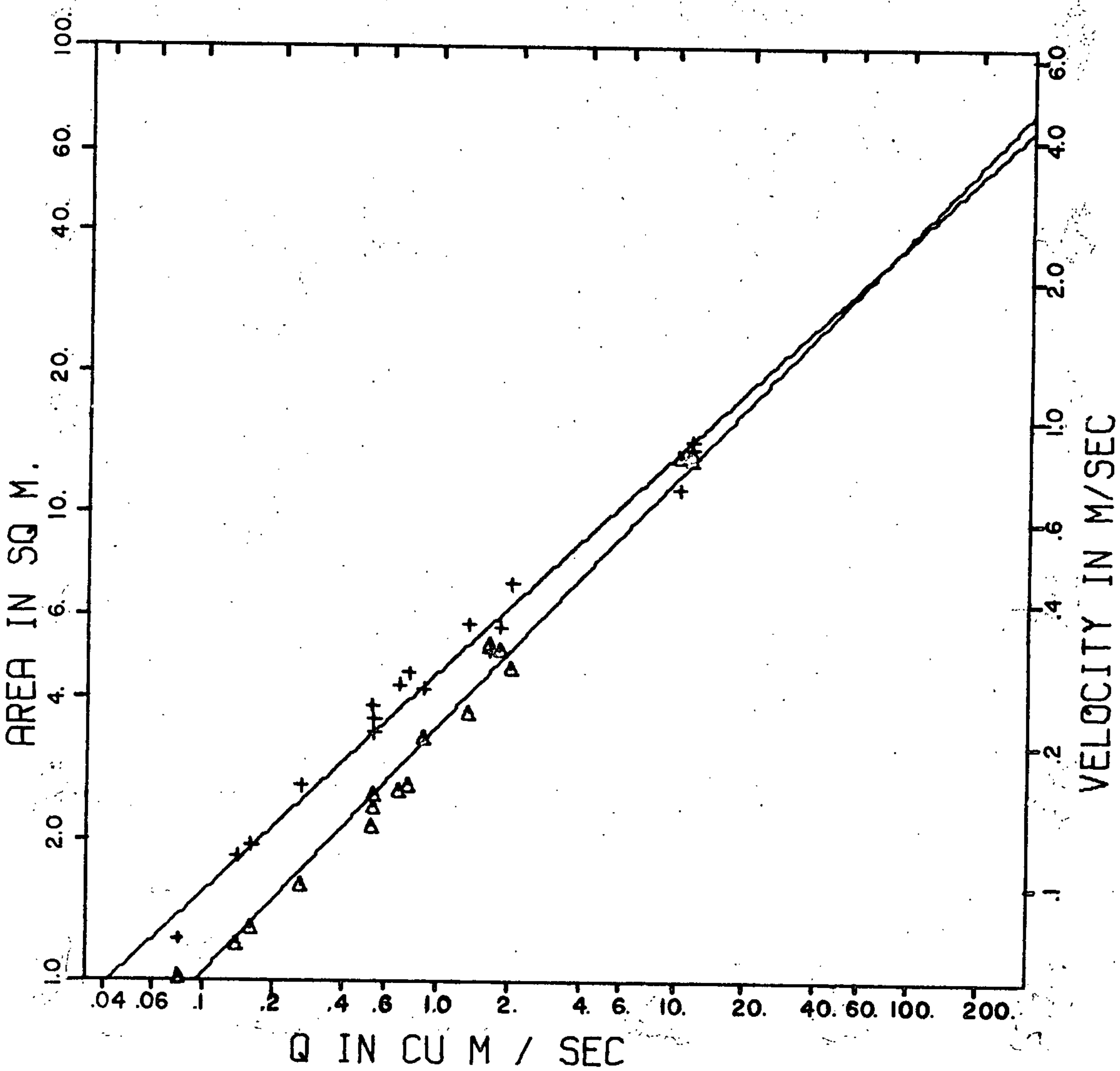


△ MEAN TIME

× PEAK TIME

+ STARTING TIME

BLANEY CREEK. REACH 3-5



Δ FLOW AREA

\+ VELOCITY

BLANEY CREEK, REACH 5-4

NO	ID	Q (L/S)	TS	TP	TM	A	V
10	0	570.00	46.00	57.50	63.10	2.3200	0.2460
12	0	1960.00	25.00	29.00	30.80	3.9000	0.5030
13	0	11000.00	13.50	13.90	14.40	10.2000	1.0800
19	0	2200.00	24.00	26.00	30.30	4.3000	0.5120
21	1	780.00	38.50	51.40	56.60	2.8500	0.2740
23	1	125.00	98.00	137.00	165.00	1.3300	0.0940
28	1	280.00	56.00	81.00	93.90	1.7000	0.1650
29	0	140.00	93.80	133.50	158.00	1.4300	0.0980
37	0	1350.00	28.50	32.70	35.90	3.1300	0.4320

LOG TM VS. LOG Q

SSX 3.1444 SSY 0.9987 SP -1.7627

REGRESSION EQUATION Y= 1.6828 + -0.5606 X
STANDARD ERROR OF ESTIMATE 0.0389
CORRELATION COEFFICIENT -0.9947
DEGREE OF FREEDOM 8
F = 654.3979

LOG A VS. LOG Q

SSX 3.1444 SSY 0.6162 SP 1.3802

REGRESSION EQUATION Y= 0.4927 + 0.4389 X
STANDARD ERROR OF ESTIMATE 0.0386
CORRELATION COEFFICIENT 0.9915
DEGREE OF FREEDOM 8
F = 406.3118

LOG V VS. LOG Q

SSX 3.1444 SSY 1.0007 SP 1.7646

REGRESSION EQUATION Y= -0.4922 + 0.5612 X
STANDARD ERROR OF ESTIMATE 0.0387
CORRELATION COEFFICIENT 0.9948
DEGREE OF FREEDOM 8
F = 662.0354

LOG TS VS. LOG Q

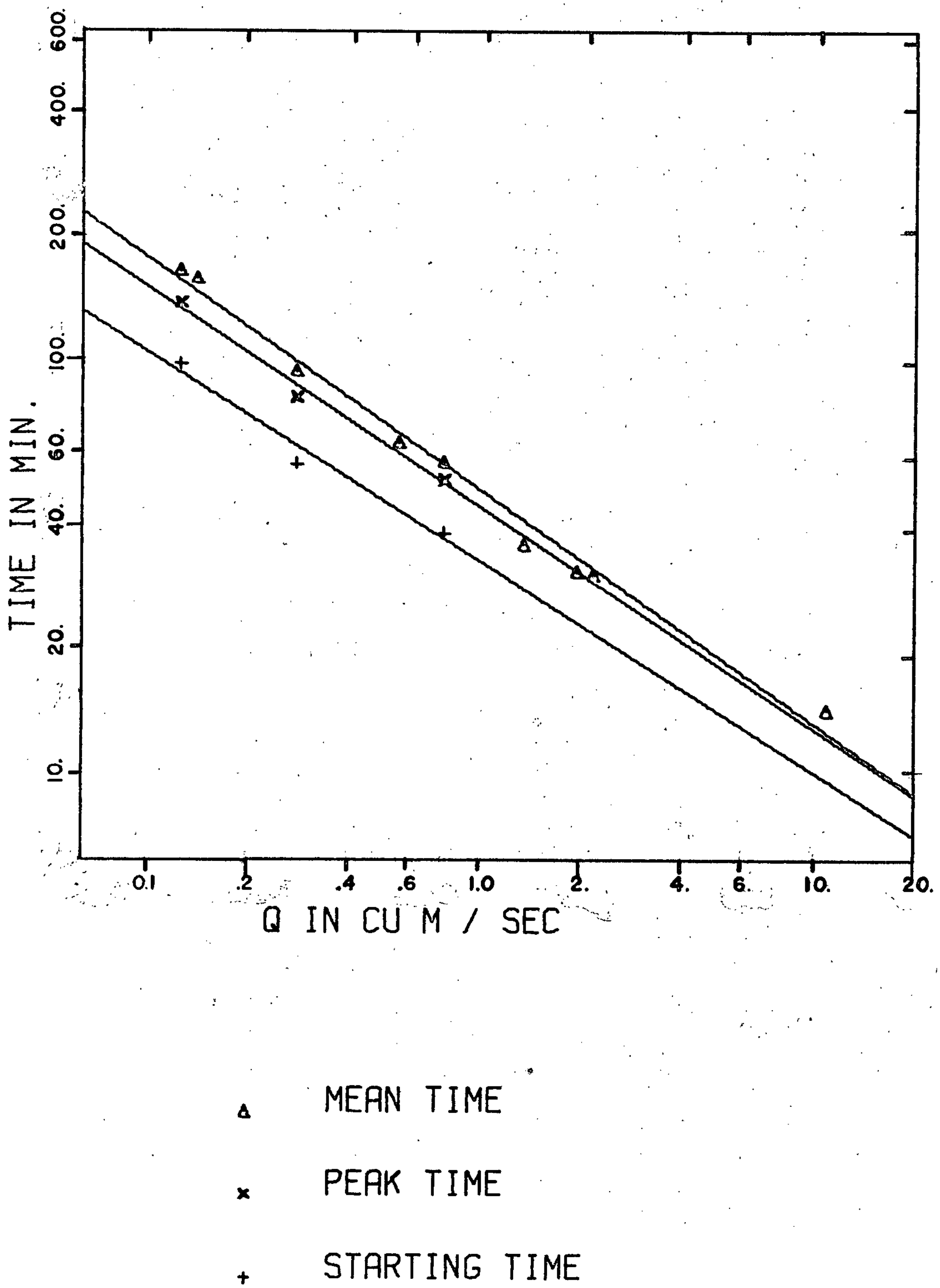
SSX 3.1444 SSY 0.6478 SP -1.4164

```
F =                          869.0930
```

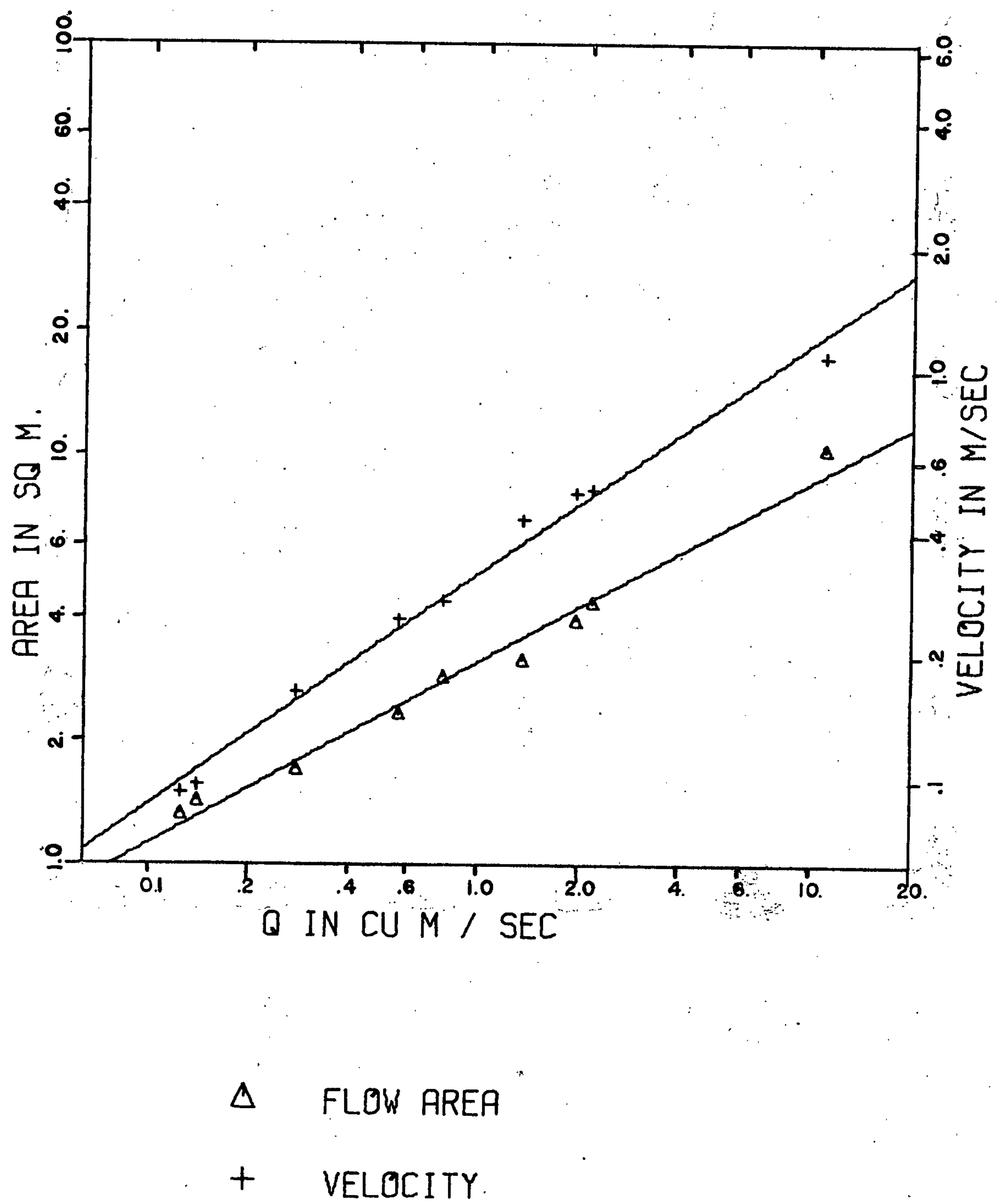


```
LOG TS VS. LOG Q

SSX       7.8009      SSY      1.4003      SP      -3.0879

REGRESSION EQUATION  Y=        1.0024 +    -0.3958 X
STANDARD ERROR OF ESTIMATE      0.1055
CORRELATION COEFFICIENT    -0.9343
DEGREE OF FREEDOM                17
F =                          109.8279

LOG TP VS. LOG Q

SSX       7.8009      SSY      1.5980      SP      -3.4937

REGRESSION EQUATION  Y=        1.1977 +    -0.4479 X
STANDARD ERROR OF ESTIMATE      0.0457
CORRELATION COEFFICIENT    -0.9895
DEGREE OF FREEDOM                17
F =                          750.7678

LOG TSS VS. LOG Q

SSX       5.7530      SSY      1.1274      SP      -2.5322

REGRESSION EQUATION  Y=        0.9382 +    -0.4401 X
STANDARD ERROR OF ESTIMATE      0.0343
CORRELATION COEFFICIENT    -0.9943
DEGREE OF FREEDOM                12
F =                          948.2959

LOG TPP VS. LOG Q

SSX       5.7530      SSY      1.2177      SP      -2.6281

REGRESSION EQUATION  Y=        1.1819 +    -0.4568 X
STANDARD ERROR OF ESTIMATE      0.0394
CORRELATION COEFFICIENT    -0.9930
DEGREE OF FREEDOM                12
F =                          772.1868
PLOT TAPE SUCCESSFULLY WRITTEN
DONE

 STOP      0
EXECUTION TERMINATED
```

BLANEY CREEK. REACH 5-4



BLANEY CREEK. REACH 5-4



PHYLLIS CREEK, REACH 1-2

NO	ID	Q (L/S)	TS	TP	TM	A	V
4	1	369.00	38.50	56.00	69.90	1.9900	0.1850
7	1	312.00	43.50	61.00	76.40	1.8300	0.1710
11	1	1470.00	21.25	28.50	36.19	4.1000	0.3580
15	1	2400.00	16.50	22.50	27.79	5.1500	0.4670
16	1	1398.00	19.00	28.50	36.66	3.9600	0.3540
21	1	945.00	24.00	33.00	39.80	2.9000	0.3260
22	1	945.00	22.50	33.00	38.75	2.8300	0.3340
25	1	826.00	23.00	35.40	46.39	2.9600	0.2790
30	1	3480.00	14.50	20.00	25.81	6.9400	0.5020

LOG TM VS. LOG Q

SSX 0.9355 SSY 0.2044 SP -0.4306

REGRESSION EQUATION Y= 1.6262 + -0.4603 X
STANDARD ERROR OF ESTIMATE 0.0296
CORRELATION COEFFICIENT -0.9849
DEGREE OF FREEDOM 8
F = 225.9623

LOG A VS. LOG Q

SSX 0.9355 SSY 0.2802 SP 0.5064

REGRESSION EQUATION Y= 0.5135 + 0.5413 X
STANDARD ERROR OF ESTIMATE 0.0294
CORRELATION COEFFICIENT 0.9891
DEGREE OF FREEDOM 8
F = 317.0220

LOG V VS. LOG Q

SSX 0.9355 SSY 0.2032 SP 0.4295

REGRESSION EQUATION Y= -0.5133 + 0.4591 X
STANDARD ERROR OF ESTIMATE 0.0294
CORRELATION COEFFICIENT 0.9850
DEGREE OF FREEDOM 8
F = 228.4597

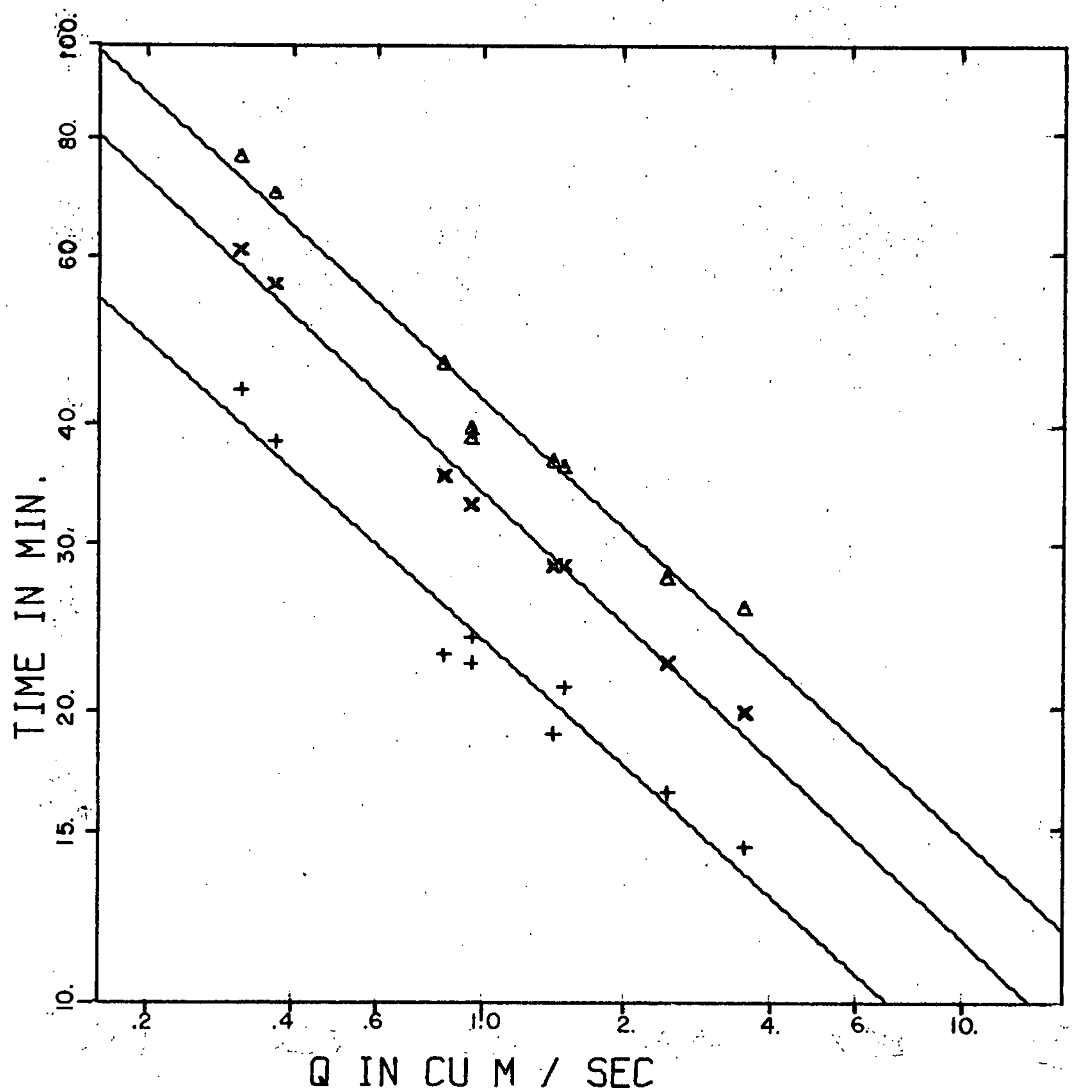
LOG TSS VS. LOG Q

SSX 0.9355 SSY 0.1958 SP -0.4191

```
REGRESSION EQUATION  Y=       1.3751 +    -0.4480 X
STANDARD ERROR OF ESTIMATE     0.0339
CORRELATION COEFFICIENT    -0.9792
DEGREE OF FREEDOM                8
F =                       162.9622
```

```
LOG TPP VS. LOG Q

SSX       0.9355     SSY      0.2079       SP      -0.4382

REGRESSION EQUATION  Y=       1.5288 +    -0.4684 X
STANDARD ERROR OF ESTIMATE    0.0195
CORRELATION COEFFICIENT    -0.9936
DEGREE OF FREEDOM                8
F =                       538.0989
```

PHYLLIS CREEK, REACH 1-2

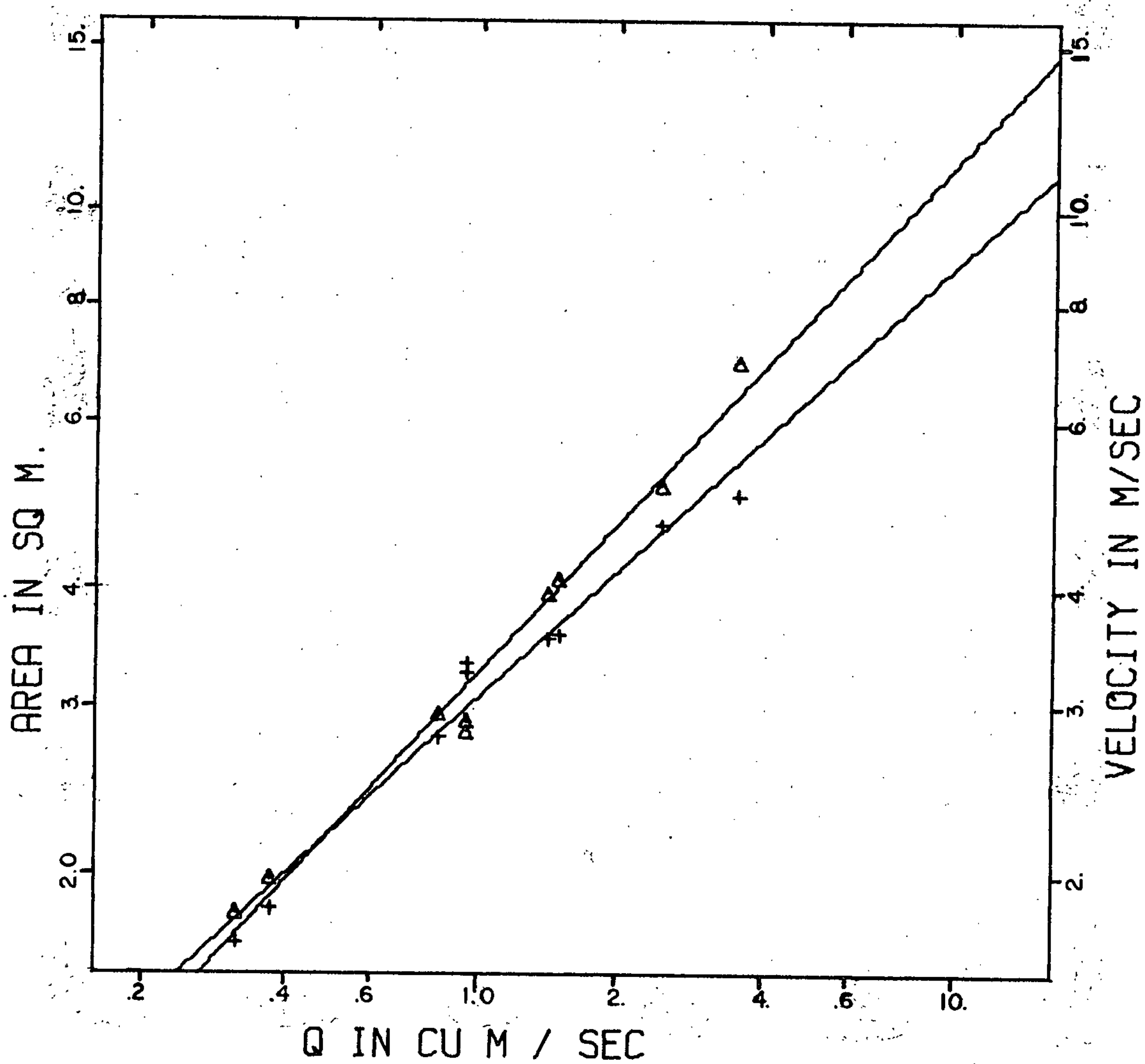


△ MEAN TIME

× PEAK TIME

\+ STARTING TIME

PHYLLIS CREEK. REACH 1-2



Δ FLOW AREA

+ VELOCITY

PHYLLIS CREEK, REACH 2-3

NO	ID	Q (L/S)	TS	TP	TM	A	V
3	1	748.00	25.00	38.00	43.52	2.7200	0.2780
6	1	352.00	37.50	57.00	64.76	1.9100	0.1840
10	1	228.00	43.00	70.00	87.62	1.6720	0.1362
11	0	1590.00	18.75	29.50	28.05	3.9300	0.4050
14	1	2550.00	15.00	21.00	23.43	5.0000	0.5100
17	1	2415.00	14.25	21.20	24.15	4.8700	0.4960
20	0	985.00	22.40	33.50	40.00	3.3000	0.2980
23	1	1880.00	17.00	24.00	26.78	4.2200	0.4450
24	1	840.00	23.50	35.50	40.84	2.9600	0.2840
26	1	1194.00	20.00	31.20	35.48	3.5500	0.3360
30	0	3610.00	12.50	17.20	18.65	5.6300	0.6410

LOG TM VS. LOG Q

SSX 1.3517 SSY 0.4005 SP -0.7340

REGRESSION EQUATION Y= 1.5798 + -0.5430 X
STANDARD ERROR OF ESTIMATE 0.0145
CORRELATION COEFFICIENT -0.9976
DEGREE OF FREEDOM 10
F = 1902.6912

LOG A VS. LOG Q

SSX 1.3517 SSY 0.2844 SP 0.6187

REGRESSION EQUATION Y= 0.5057 + 0.4577 X
STANDARD ERROR OF ESTIMATE 0.0115
CORRELATION COEFFICIENT 0.9979
DEGREE OF FREEDOM 10
F = 2144.6553

LOG V VS. LOG Q

SSX 1.3517 SSY 0.3986 SP 0.7327

REGRESSION EQUATION Y= -0.5055 + 0.5421 X
STANDARD ERROR OF ESTIMATE 0.0122
CORRELATION COEFFICIENT 0.9983
DEGREE OF FREEDOM 10
F = 2648.8000

LOG TS VS. LOG Q

SSX 1.3517 SSY 0.2809 SP -0.6144

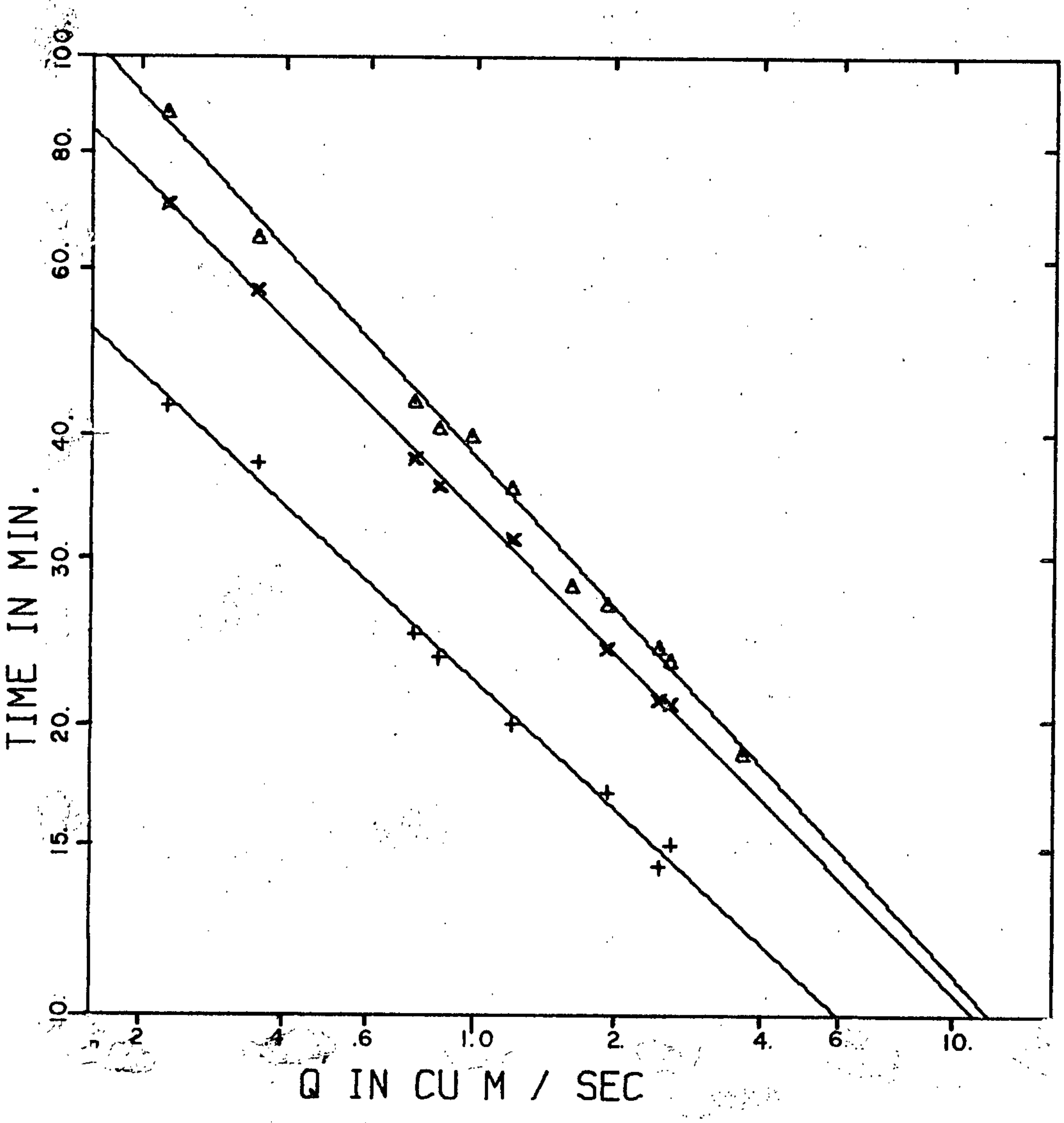
```
REGRESSION EQUATION  Y=      1.3481 +    -0.4546 X
STANDARD ERROR OF ESTIMATE    0.0135
CORRELATION COEFFICIENT   -0.9971
DEGREE OF FREEDOM              10
F =                     1522.2800
```

```
LOG TP VS. LOG Q

SSX       1.3517      SSY       0.3411      SP      -0.6764

REGRESSION EQUATION  Y=      1.5257 +    -0.5004 X
STANDARD ERROR OF ESTIMATE    0.0170
CORRELATION COEFFICIENT   -0.9962
DEGREE OF FREEDOM              10
F =                     1166.0486
```

```
LOG TSS VS. LOG Q

SSX       1.0294      SSY       0.2185      SP      -0.4729

REGRESSION EQUATION  Y=      1.3457 +    -0.4593 X
STANDARD ERROR OF ESTIMATE    0.0148
CORRELATION COEFFICIENT   -0.9970
DEGREE OF FREEDOM               7
F =                      985.6196
```

```
LOG TPP VS. LOG Q

SSX       1.0294      SSY       0.2615      SP      -0.5186

REGRESSION EQUATION  Y=      1.5219 +    -0.5037 X
STANDARD ERROR OF ESTIMATE    0.0071
CORRELATION COEFFICIENT   -0.9994
DEGREE OF FREEDOM               7
F =                     5139.0664
```

PHYLLIS CREEK, REACH 2-3

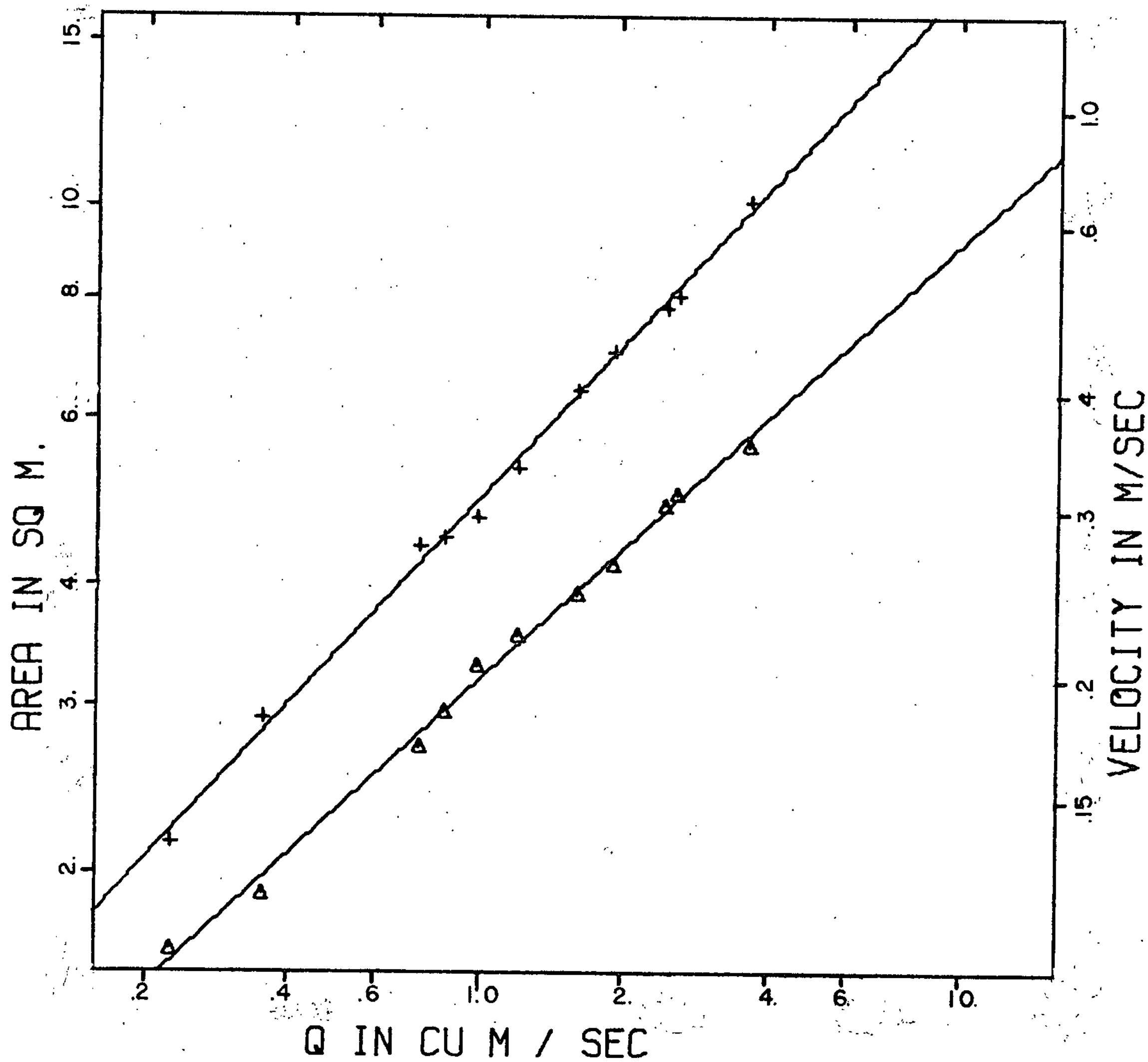


△ MEAN TIME

× PEAK TIME

+ STARTING TIME

PHYLLIS CREEK. REACH 2-3



Δ FLOW AREA

+ VELOCITY

PHYLLIS CREEK, REACH 3-4

NO	ID	Q (L/S)	TS	TP	TM	A	V
3	0	750.00	18.00	37.00	39.56	2.8800	0.2600
5	1	339.00	32.00	45.00	52.46	1.7400	0.1950
6	0	338.00	39.50	47.00	53.34	1.7500	0.1930
10	0	228.00	53.00	64.00	64.70	1.4870	0.1608
13	1	2490.00	11.50	16.80	18.83	4.5600	0.5460
19	1	1070.00	19.00	26.50	29.88	3.1200	0.3440
27	1	1200.00	17.00	24.50	28.52	3.3300	0.3610
28	1	3100.00	10.30	14.50	17.33	5.2200	0.5950
29	1	3690.00	8.90	13.50	15.41	5.5300	0.6550

LOG TM VS. LOG Q

SSX 1.5952 SSY 0.4250 SP -0.8211

REGRESSION EQUATION Y= 1.4903 + -0.5147 X
STANDARD ERROR OF ESTIMATE 0.0184
CORRELATION COEFFICIENT -0.9972
DEGREE OF FREEDOM 8
F = 1248.8062

LOG A VS. LOG Q

SSX 1.5952 SSY 0.3676 SP 0.7636

REGRESSION EQUATION Y= 0.4801 + 0.4787 X
STANDARD ERROR OF ESTIMATE 0.0173
CORRELATION COEFFICIENT 0.9971
DEGREE OF FREEDOM 8
F = 1219.0845

LOG V VS. LOG Q

SSX 1.5952 SSY 0.4183 SP 0.8146

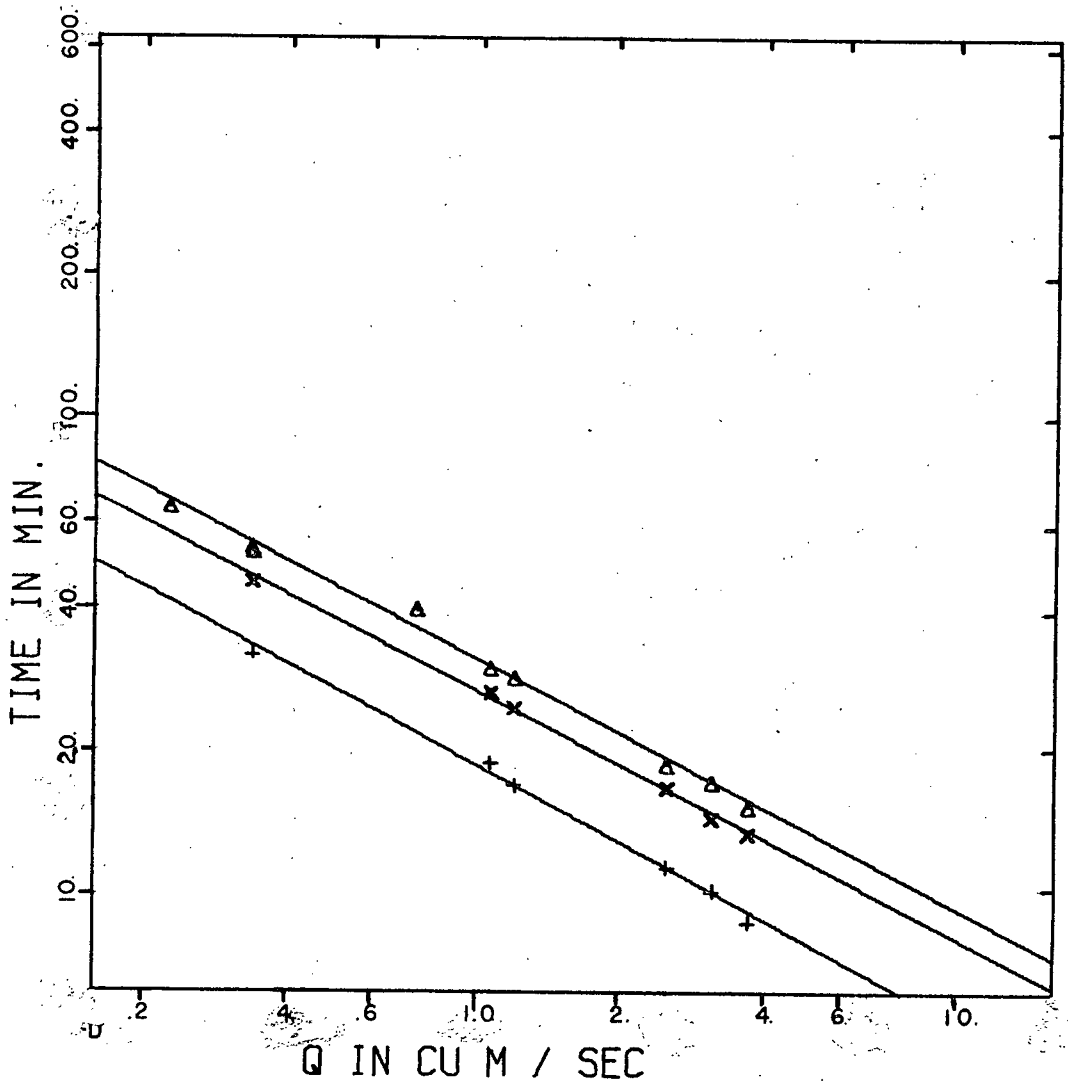
REGRESSION EQUATION Y= -0.4786 + 0.5106 X
STANDARD ERROR OF ESTIMATE 0.0182
CORRELATION COEFFICIENT 0.9972
DEGREE OF FREEDOM 8
F = 1260.8452

LOG TS VS. LOG Q

SSX 1.5952 SSY 0.5792 SP -0.9452

```
REGRESSION EQUATION  Y=        1.2809 +    -0.5925 X
STANDARD ERROR OF ESTIMATE      0.0523
CORRELATION COEFFICIENT     -0.9833
DEGREE OF FREEDOM                 8
F =                        204.5374

LOG TP VS. LOG Q

SSX       1.5952      SSY        0.4718      SP       -0.8616

REGRESSION EQUATION  Y=        1.4390 +    -0.5401 X
STANDARD ERROR OF ESTIMATE      0.0303
CORRELATION COEFFICIENT     -0.9932
DEGREE OF FREEDOM                 8
F =                        506.9019

LOG TSS VS. LOG Q

SSX       0.7485      SSY        0.2131      SP       -0.3982

REGRESSION EQUATION  Y=        1.2698 +    -0.5320 X
STANDARD ERROR OF ESTIMATE      0.0173
CORRELATION COEFFICIENT     -0.9972
DEGREE OF FREEDOM                 5
F =                        706.3044

LOG TPP VS. LOG Q

SSX       0.7485      SSY        0.1959      SP       -0.3825

REGRESSION EQUATION  Y=        1.4235 +    -0.5109 X
STANDARD ERROR OF ESTIMATE      0.0113
CORRELATION COEFFICIENT     -0.9987
DEGREE OF FREEDOM                 5
F =                       1521.2705
PLOT TAPE SUCCESSFULLY WRITTEN
DONE

 STOP      0
EXECUTION TERMINATED
```

PHYLLIS CREEK. REACH 3-4

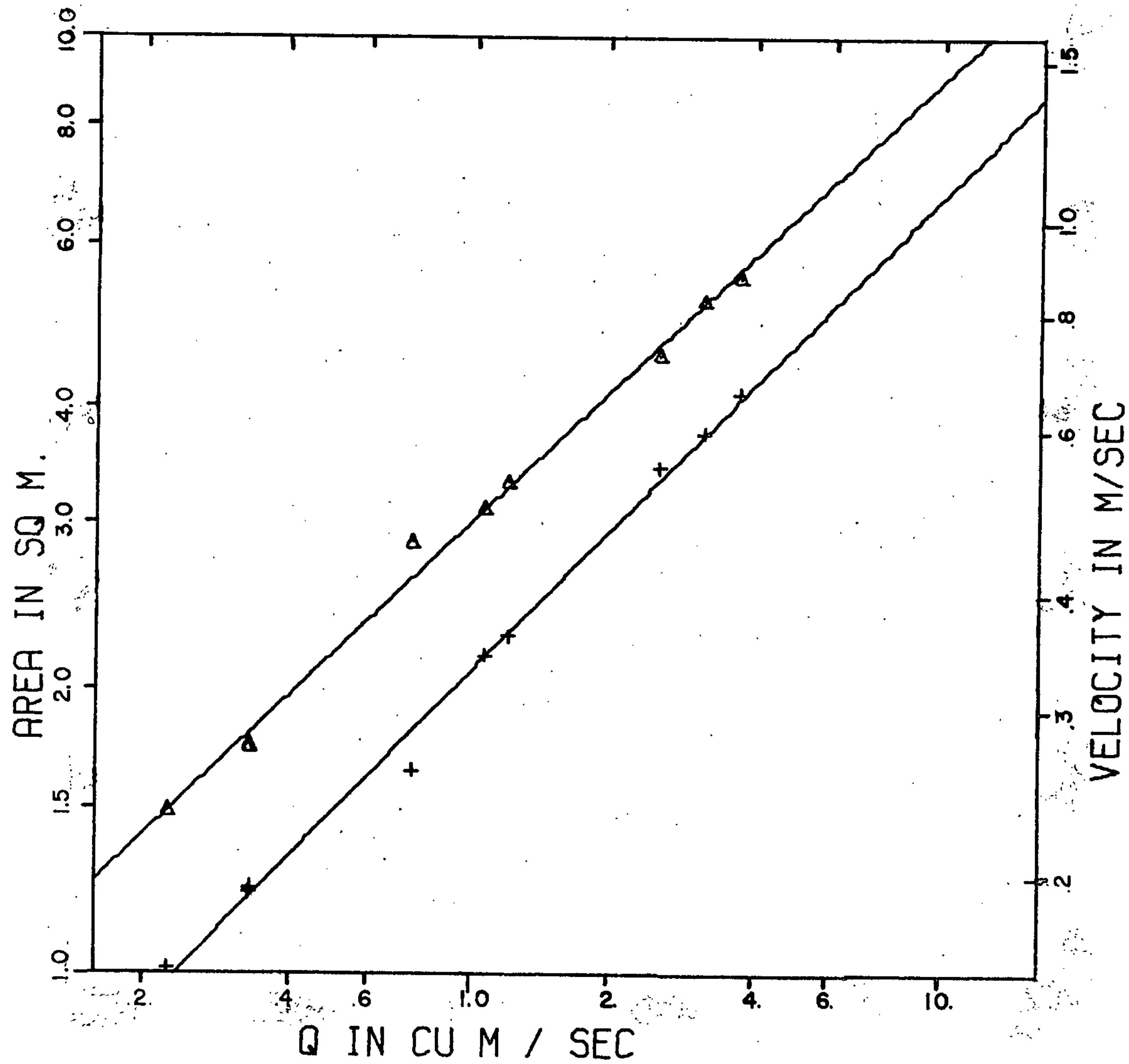


Δ MEAN TIME

× PEAK TIME

\+ STARTING TIME

PHYLLIS CREEK. REACH 3-4



Δ FLOW AREA

\+ VELOCITY

PHYLLIS CREEK, REACH 4-6

NO	ID	Q (L/S)	TS	TP	TM	A	V
5	0	385.00	20.00	28.00	29.44	2.2300	0.1730
8	1	366.00	15.00	26.00	31.49	2.2700	0.1610
10	0	239.00	31.00	35.00	35.00	1.6580	0.1450
12	1	2370.00	5.10	8.40	9.54	4.4500	0.5330
18	1	1100.00	8.00	12.80	14.43	3.1300	0.3520
27	0	1200.00	9.00	14.00	16.23	3.8400	0.3130
28	0	3100.00	4.70	7.30	7.57	4.6200	0.6710
29	0	3720.00	5.60	6.90	7.79	5.7100	0.6510

LOG TM VS. LOG Q

SSX 1.4633 SSY 0.5280 SP -0.8730

REGRESSION EQUATION Y= 1.2102 + -0.5966 X
STANDARD ERROR OF ESTIMATE 0.0345
CORRELATION COEFFICIENT -0.9932
DEGREE OF FREEDOM 7
F = 437.0291

LOG A VS. LOG Q

SSX 1.4633 SSY 0.2438 SP 0.5887

REGRESSION EQUATION Y= 0.5049 + 0.4023 X
STANDARD ERROR OF ESTIMATE 0.0341
CORRELATION COEFFICIENT 0.9856
DEGREE OF FREEDOM 7
F = 203.3331

LOG V VS. LOG Q

SSX 1.4633 SSY 0.5279 SP 0.8729

REGRESSION EQUATION Y= -0.5044 + 0.5965 X
STANDARD ERROR OF ESTIMATE 0.0346
CORRELATION COEFFICIENT 0.9932
DEGREE OF FREEDOM 7
F = 435.2229

LOG TS VS. LOG Q

SSX 1.4633 SSY 0.6307 SP -0.9286

REGRESSION EQUATION Y= 1.0009 + -0.6346 X

STANDARD ERROR OF ESTIMATE 0.0831
CORRELATION COEFFICIENT -0.9666
DEGREE OF FREEDOM 7
F = 85.3634

LOG TP VS. LOG Q

SSX	1.4633	SSY	0.5447	SP	-0.8898

REGRESSION EQUATION Y= 1.1672 + -0.6081 X
STANDARD ERROR OF ESTIMATE 0.0246
CORRELATION COEFFICIENT -0.9967
DEGREE OF FREEDOM 7
F = 893.3013

LOG TSS VS. LOG Q

SSX	0.3326	SSY	0.1108	SP	-0.1919

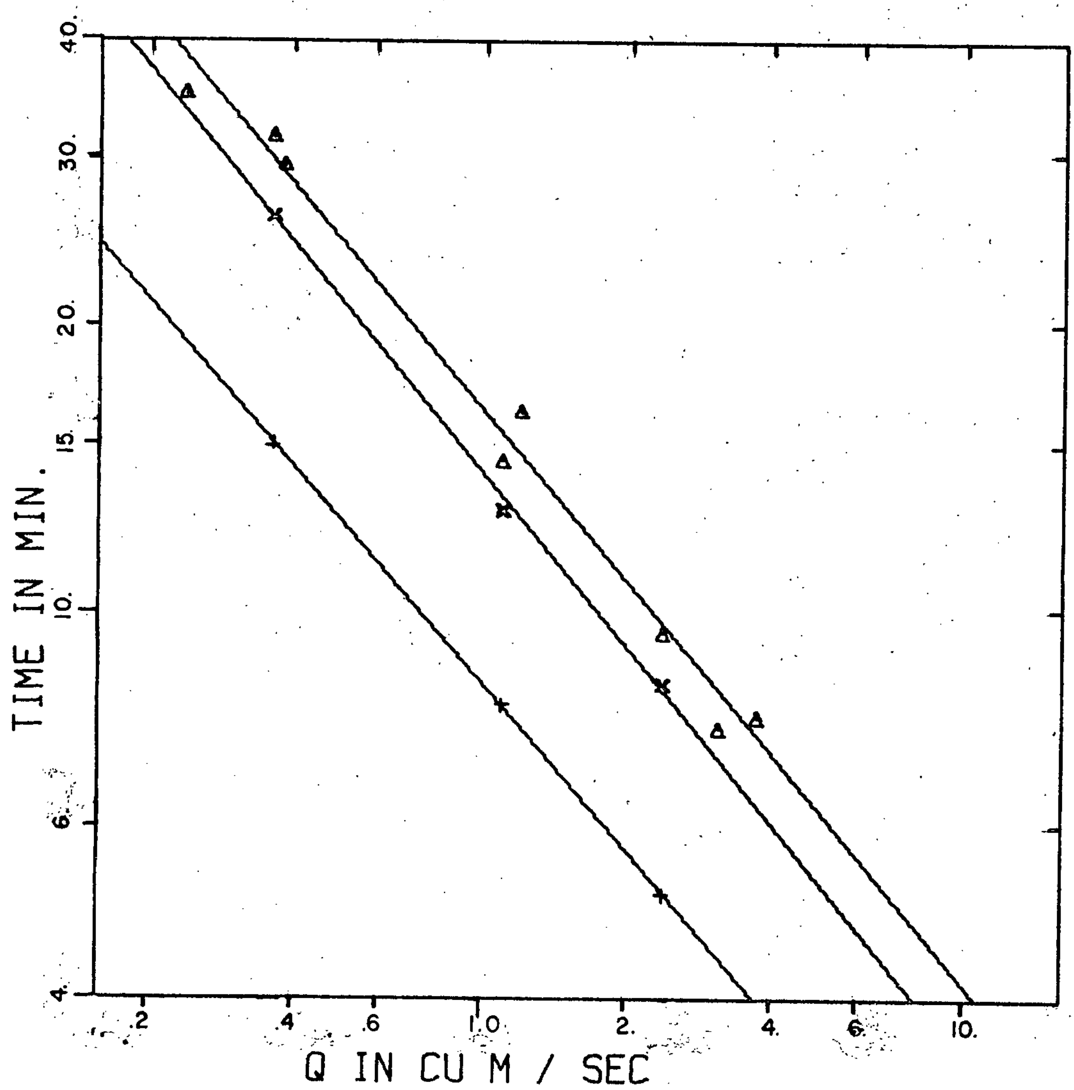
REGRESSION EQUATION Y= 0.9250 + -0.5771 X
STANDARD ERROR OF ESTIMATE 0.0028
CORRELATION COEFFICIENT -1.0000
DEGREE OF FREEDOM 2
F = 14613.2969

LOG TPP VS. LOG Q

SSX	0.3326	SSY	0.1230	SP	-0.2020

REGRESSION EQUATION Y= 1.1447 + -0.6076 X
STANDARD ERROR OF ESTIMATE 0.0152
CORRELATION COEFFICIENT -0.9991
DEGREE OF FREEDOM 2
F = 529.8909

PHYLLIS CREEK, REACH 4-6

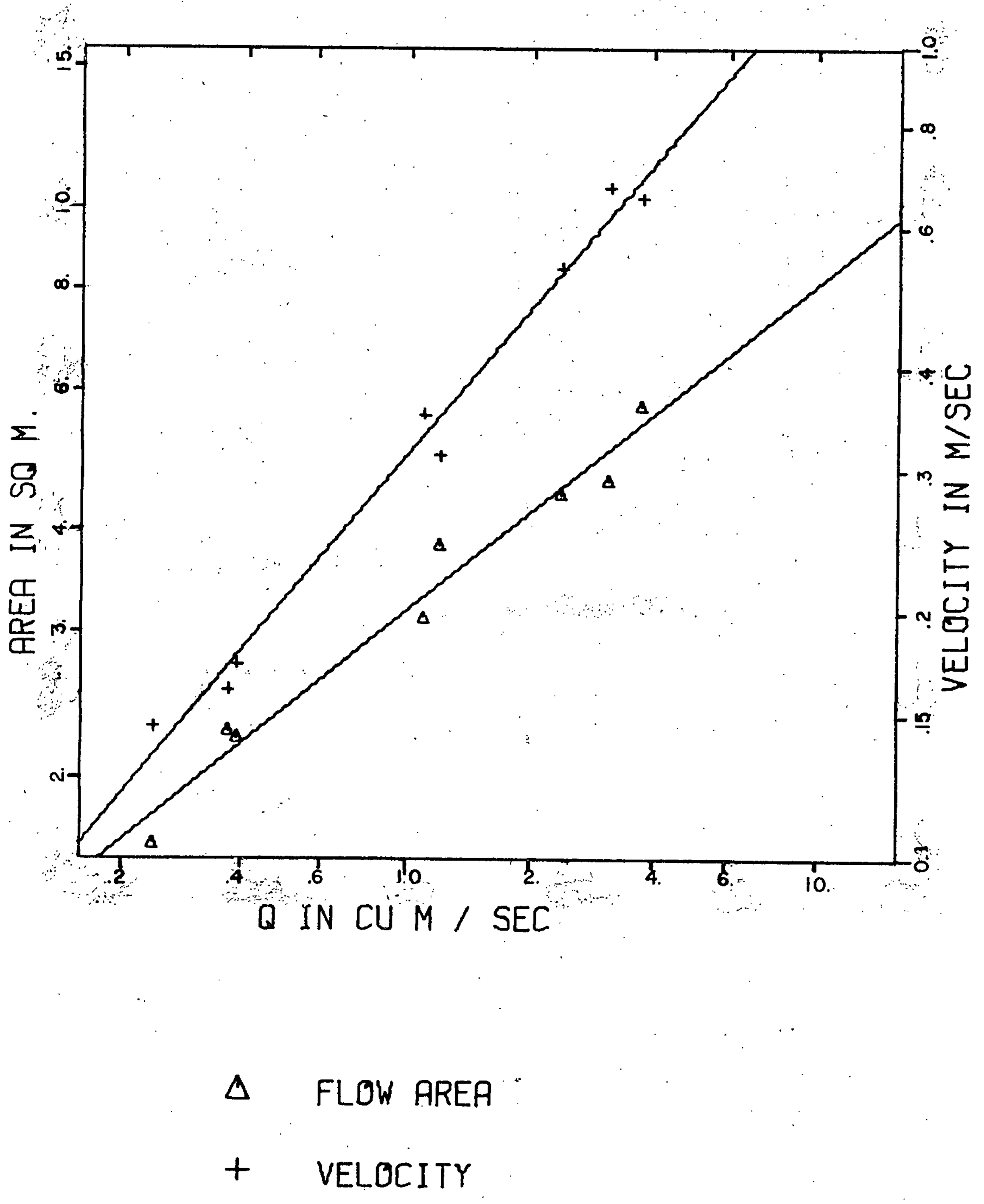


Δ MEAN TIME

× PEAK TIME

\+ STARTING TIME

PHYLLIS CREEK. REACH 4-6



PHYLIS LOWER

NO	ID	Q (L/S)	TS	TP	TM	A	V
10	1	365.00	4.58	10.33	13.79	2.3300	0.1565
20	1	269.00	4.75	12.00	18.18	2.3190	0.1159
30	1	4358.00	1.56	2.55	2.78	5.6140	0.7760
40	1	3415.00	1.75	3.00	3.60	5.6910	0.6000

LOG TM VS. LOG Q

SSX 1.2032 SSY 0.5027 SP -0.7765

REGRESSION EQUATION Y= 0.8764 + -0.6454 X
STANDARD ERROR OF ESTIMATE 0.0280
CORRELATION COEFFICIENT -0.9984
DEGREE OF FREEDOM 3
F = 639.7966

LOG A VS. LOG Q

SSX 1.2032 SSY 0.1489 SP 0.4206

REGRESSION EQUATION Y= 0.5449 + 0.3496 X
STANDARD ERROR OF ESTIMATE 0.0310
CORRELATION COEFFICIENT 0.9935
DEGREE OF FREEDOM 3
F = 152.6978

LOG V VS. LOG Q

SSX 1.2032 SSY 0.5114 SP 0.7829

REGRESSION EQUATION Y= -0.5451 + 0.6507 X
STANDARD ERROR OF ESTIMATE 0.0309
CORRELATION COEFFICIENT 0.9981
DEGREE OF FREEDOM 3
F = 532.2039

LOG TSS VS. LOG Q

SSX 1.2032 SSY 0.2045 SP -0.4951

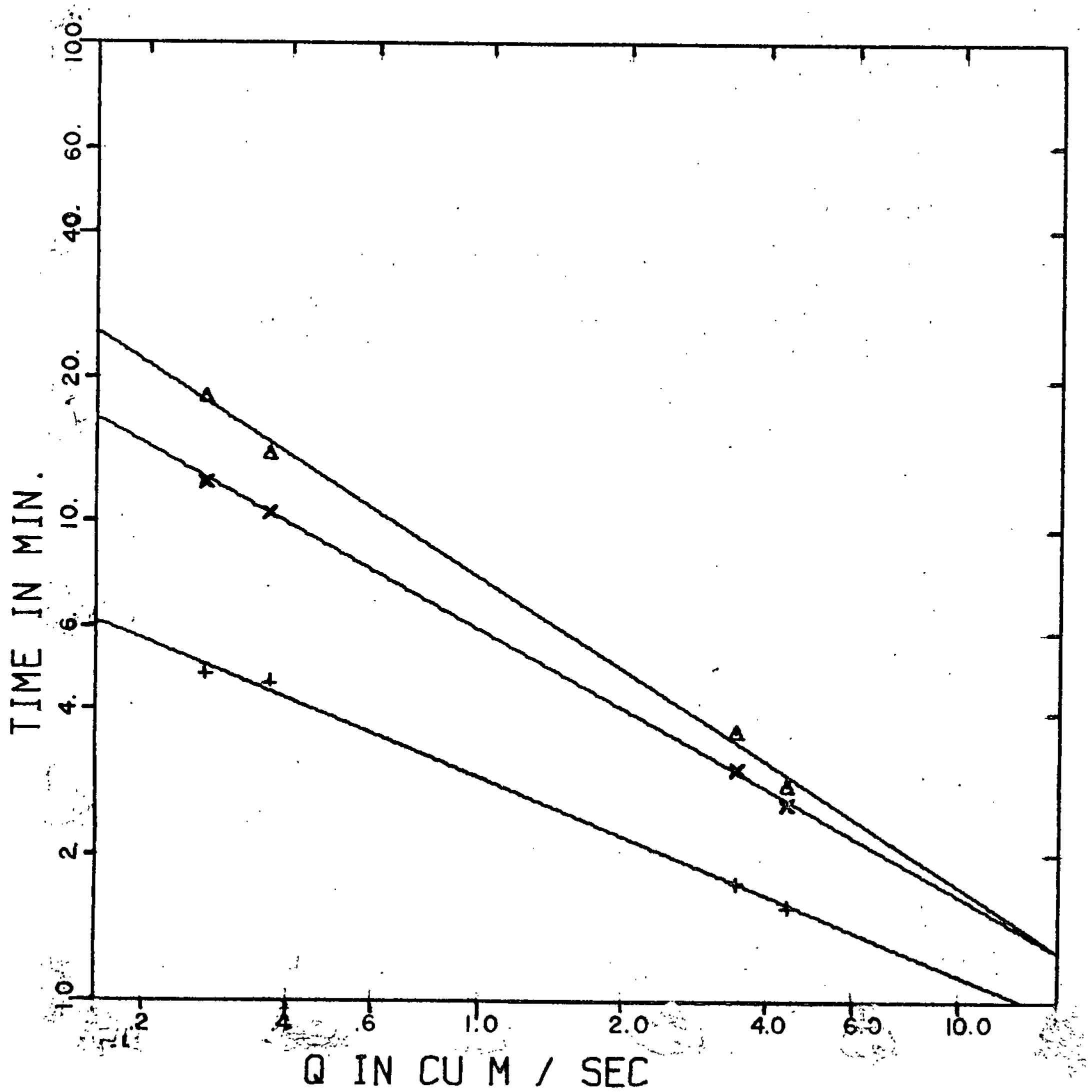
REGRESSION EQUATION Y= 0.4604 + -0.4115 X
STANDARD ERROR OF ESTIMATE 0.0197
CORRELATION COEFFICIENT -0.9981
DEGREE OF FREEDOM 3
F = 525.7346

LOG TPP VS. LOG Q

SSX	1.2032	SSY	0.3704	SP	-0.6675

REGRESSION EQUATION Y= 0.7671 + -0.5548 X
STANDARD ERROR OF ESTIMATE 0.0073
CORRELATION COEFFICIENT -0.9999
DEGREE OF FREEDOM 3
F = 6927.8672

PHYLIS LOWER

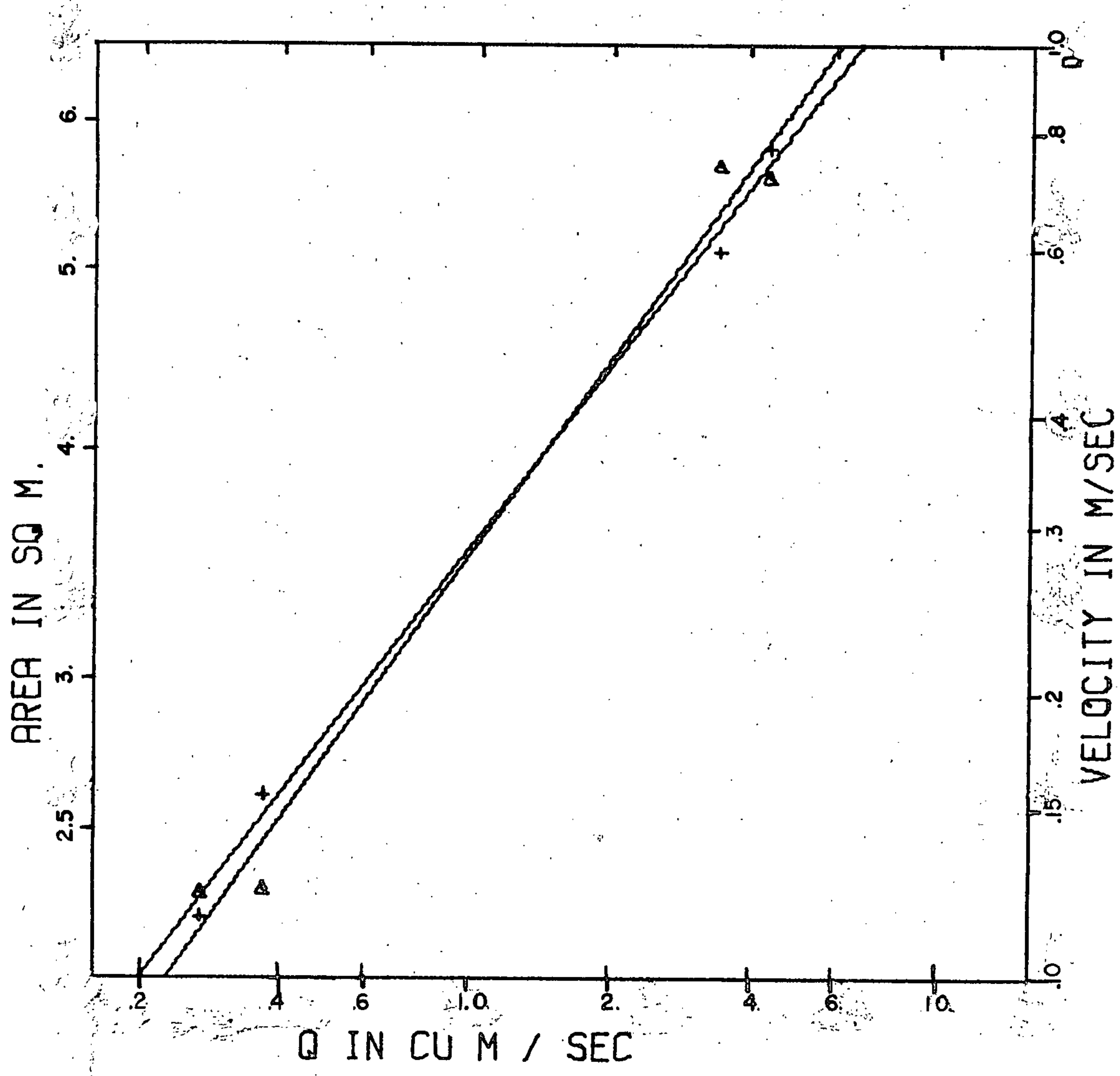


△ MEAN TIME

× PEAK TIME

\+ STARTING TIME

PHYLIS LOWER



△ FLOW AREA

+ VELOCITY

'P. D.'

```
C
C     FORTRAN /360 PROGRAM 'PD' FOR SOLUTION OF THE DIFFERENTIAL
C     EQUATION OF UNSTEADY FLOW THROUGH A CASCADE OF RESERVOIRS.
C
C       CONTROL CARDS
C
C     NO 1, QO = Q AT START , AL = ALPHA,      BE = BETA        (F6.0)
C     NO 2, TS = STARTING T , TE = END T       DT = TIME INT
C           TO = START OF TIME COUNT                            (F6.0)
C     NO 3, W  = WIDTH ,          L = LENGTH    AA & BA          (F6.0)
C     NO 4  TIT= TITLE     (20A4)
C
C     EXPLANATION OF TERMS:
C
C        UNITS ARE METERS AND SECONDS EXCEPT AS NOTED BELOW.
C        BETA  IS THE DISPERSION COEFFICIENT, WHICH CAN BE
C          ESTIMATED AS (QO / QD)**0.2   .
C        ALPHA  IS THE RELATIVE CHANGE IN DISCHARGE,
C          (Q(END) / Q(START)) - 1.0   .
C        THE COMPUTATIONS ARE PERFORMED FOR THE PERIOD TS - TE.
C        TS AND TE ARE IN MINUTES FROM THE START OF THE TEST,
C        TO IS THE STARTING TIME IN HOURS AND MINUTES(E.G. 1420.
C          FOR '20 MINUTES PAST 2PM' )
C        WIDTH IS THE CHANNEL WIDTH, WD.
C        LENGNTH IS THE  TOTAL LENGTH OF THE TEST REACH.
C
C     OUTPUT:
C
C        THE PROGRAM PRINTS THE SOLUTION OF THE CASCADE EQUATION
C        A(T), THE CORRESPONDING Q(T), AND SOME OF THE TERMS
C        IN THE EQUATION FOR A(T).
C
      DIMENSION  TIT(20)
      COMMON  L, P, BE, W, C, T, EX1 , EX2, K
      EXTERNAL AUX1, AUX2
      REAL L , LS   , LE
C
C     READ INITIAL DATA
      READ (5,1) QO, AL, BE
      READ (5,1) TS, TE, DT , TO
      READ (5,1) W,  L , AA, BA
1     FORMAT (12F6.0)
      READ (5,2) TIT
2     FORMAT (20A4)
C
C     INITIALIZE
      AO     = AA *(QO ** BA)
      AEND = AA*((QO*(1. + AL)) **BA )
      AL = ( AEND - AO )/ AO
      C      = ( QO ** (1.-BA)) / (AA*BA)
      P      = SQRT((240.* C) / ((BE * W)**2))
      COEF1 =  AO /  (BE *W * SQRT (6.283 * P ))
      PI8=AO* ( 1. + AL)/ SQRT( 8. * 3.1416 )
      COEF2 = PI8 * ((( 240. * C * L )/(( BE * W )**2))**0.25 )
      DL     = L/20.
```

```
      N      = (TE - TS) /DT
C
C     WRITE INITIAL DATA
      WRITE (6, 3) TIT
3     FORMAT(1H1, 20A4 )
      ITO    =TO
      WRITE (6, 4) QO , AL , BE , ITO ,TS , TE , DT      ,
     1 W , L ,   AA,   BA ,
     2 AO , C ,         COEF1,COEF2   ,P
4     FORMAT ( 'INITIAL CONDITIONS' /
     1' INITIAL Q (QO) , ALPHA (AL), BETA (BE) = ', 3F10.3    /
     2' START OF TIME COUNT (ITO), TS, TE, DT  = ', I6, 3F8.1 /
     3' WIDTH (W) , LENGTH (L), AA, BA         = ', 4F10.3    /
     4' INITIAL AREA (AO), CELERITY (C)        = ', 2F10.3    //
     5' COEF1 , COEF2,  P                      = ', 3E14.5    )
      WRITE (6,5)
5     FORMAT (  'OTIME  1ST EXPO.  1ST INTEG.   1ST TERM    2ND EXPO
     12ND INTE.   2ND TERM      A(T)        Q(T) '/)
C
C     DO LOOP FOR N VALUES OF A(T)
      DO 6 I = 1, N
      T        = TS+ (I-1)* DT
C
C     FIRST INTEGRAL OVER X
      AINT1  = 0.0
      LE   =  0.0
      DO 7  J= 1, 20
      LS  =  LE
      LE     =  LS + DL
      K      =1
      AI1  =  FGAU16(LS, LE, AUX1  )
      AINT1  = AINT1 + AI1
7     CONTINUE
      FT1     = COEF1 * AINT1 / (T **0.25)
C
C     SECOND INTEGRAL OVER T
      AINT2 = 0.0
      TTE    = 0.0
      TDIFF  = T   / 20.0
      DO   8   J = 1 ,20
      TTS    = TTE
      TTE    = TTS + TDIFF
      K      = 1
      AI2   =  FGAU16( TTS,  TTE,  AUX2  )
      AINT2 = AINT2 + AI2
8     CONTINUE
      FT2    = COEF2 * AINT2
      A      =  FT1   + FT2
      Q      =   ( A / AA)**(1.0 /BA)
C
C     WRITE RESULTS
      WRITE (6,9) T, EX1, AINT1 , FT1 , EX2 , AINT2, FT2 , A , Q
9     FORMAT (1X, F4.0 , 8E12.4 )
6     CONTINUE
      STOP
```

```
      FUNCTION  AUX1  ( X )
C
C       FUNCTION CALLED BY THE LIBRARY PROGRAM FGAU16
C
      REAL  L
      COMMON L,P, BE, W, C, T, EX1,EX2, K
      EX1 = P *SQRT((L-X)*T) + ( X -L -(60.* C * T)) / (BE* W )
      AUX1=  EXP (EX1)  /(( L-X)** 0.25)
      K  =  0
      RETURN
      END
```

```
      FUNCTION AUX2 ( Z)
C
C       FUNCTION CALLED BY THE LIBRARY PROGRAM FGAU16
C
      REAL  L
      COMMON L, P, BE, W, C, T, EX1, EX2 ,K
      EX2   = P * SQRT  (L*(T - Z)) +  (60.*C*Z  - 60.*C*T  - L)/
      AUX2  = EXP (EX2)  / (( T - Z)**0.75)                 (BE*W)
      K     = 0
      RETURN
      END
```

'P. D.'

SAMPLE OUTPUT

PHYLLIS CREEK, JUNE 22, 1968. DOWNSURGE
INITIAL CONDITIONS
INITIAL Q (QO) , ALPHA (AL), BETA (BE) = 0.815 -0.065 0.540
START OF TIME COUNT (ITO), TS, TE, DT = 1300 12.0 60.0 2.0
WIDTH (W) , LENGTH (L), AA, BA = 11.500 777.000 3.260 0.541
INITIAL·AREA (AO), CELERITY (C) = 2.918 0.516

COEF1 , COEF2, P = 0.14005E 00 0.38484E 01 0.17919E 01

TIME	1ST EXPO.	1ST INTEG.	1ST TERM	2ND EXPO.	2ND INTE.	2ND TERM	A(T)	Q(T)
12.	-0.3461E 02	0.3874E 02	0.2915E 01	-0.9813E 02	0.4538E-06	0.1746E-05	0.2915E 01	0.8134E 00
14.	-0.4229E 02	0.4027E 02	0.2915E 01	-0.9611E 02	0.2572E-04	0.9899E-04	0.2916E 01	0.8136E 00
16.	-0.5012E 02	0.4161E 02	0.2914E 01	-0.9425E 02	0.5727E-03	0.2204E-02	0.2916E 01	0.8137E 00
18.	-0.5809E 02	0.4253E 02	0.2892E 01	-0.9251E 02	0.6009E-02	0.2312E-01	0.2915E 01	0.8131E 00
20.	-0.6616E 02	0.4192E 02	0.2776E 01	-0.9089E 02	0.3408E-01	0.1312E 00	0.2908E 01	0.8095E 00
22.	-0.7433E 02	0.3768E 02	0.2437E 01	-0.8936E 02	0.1168E 00	0.4496E 00	0.2887E 01	0.7987E 00
24.	-0.8257E 02	0.2881E 02	0.1823E 01	-0.8791E 02	0.2664E 00	0.1025E 01	0.2848E 01	0.7792E 00
26.	-0.9089E 02	0.1775E 02	0.1101E 01	-0.8653E 02	0.4423E 00	0.1702E 01	0.2803E 01	0.7565E 00
28.	-0.9927E 02	0.8570E 01	0.5218E 00	-0.8521E 02	0.5833E 00	0.2245E 01	0.2767E 01	0.7384E 00
30.	-0.1077E 03	0.3222E 01	0.1928E 00	-0.8395E 02	0.6634E 00	0.2553E 01	0.2746E 01	0.7283E 00
32.	-0.1162E 03	0.9474E 00	0.5579E-01	-0.8274E 02	0.6968E 00	0.2681E 01	0.2737E 01	0.7240E 00
34.	-0.1247E 03	0.2202E 00	0.1277E-01	-0.8157E 02	0.7073E 00	0.2722E 01	0.2735E 01	0.7228E 00
36.	-0.1333E 03	0.4098E-01	0.2343E-02	-0.8045E 02	0.7098E 00	0.2732E 01	0.2734E 01	0.7224E 00
38.	-0.1419E 03	0.6187E-02	0.3490E-03	-0.7936E 02	0.7103E 00	0.2733E 01	0.2734E 01	0.7224E 00
40.	-0.1505E 03	0.7677E-03	0.4275E-04	-0.7831E 02	0.7103E 00	0.2734E 01	0.2734E 01	0.7224E 00
42.	-0.1592E 03	0.7926E-04	0.4360E-05	-0.7729E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7223E 00
44.	-0.1679E 03	0.6885E-05	0.3744E-06	-0.7631E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7223E 00
46.	-0.1767E 03	0.5086E-06	0.2735E-07	-0.7534E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7223E 00
48.	-0.1854E 03	0.3224E-07	0.1716E-08	-0.7441E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7224E 00
50.	-0.1942E 03	0.1770E-08	0.9325E-10	-0.7351E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7224E 00
52.	-0.2030E 03	0.8486E-10	0.4426E-11	-0.7262E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7224E 00
54.	-0.2119E 03	0.3576E-11	0.1848E-12	-0.7176E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7224E 00
56.	-0.2207E 03	0.1334E-12	0.6832E-14	-0.7092E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7224E 00
58.	-0.2296E 03	0.4435E-14	0.2251E-15	-0.7010E 02	0.7104E 00	0.2734E 01	0.2734E 01	0.7223E 00

'SNLR'

```
C       PROGRAM FOR FLOOD ROUTING THROUGH SEQUENCES OF NONLINEAR
C       RESERVOIRS AND KINEMATIC CHANNELS, WRITTEN IN FORTRAN /360.
C
C       1 CONTROL CARD PER CHANNEL (CONSISTING OF SEVERAL REACHES):
C          NO OF REACHES, KR, (I2) & TITLE OF CHANNEL, TITR, (10A4).
C
C       3 CONTROL CARDS PER REACH;
C
C       NO 1 CONTAINS:
C          NO OF RESERVOIRS, N, (I2);  &  FACTOR ALPHA , AL, (F6.0);
C          LOCAL INFLOW ALONG REACH, QINC, (F6.0).
C
C       NO 2 CONTAINS:
C          TITLE FOR REACH, TIT, (10A4).
C
C       NO 3 CONTAINS:
C          LENGTH OF REACH, L, (F6.0); STEADY FLOW PARAMETERS, AA AND
C          BA, (F6.0); FORMATIVE DISCHARGE, QD, (F6.0)
C          RATING CURVE PARAMETERS, AH1, BH1, HO1, AH2,
C          BH2, HO2, (F6.0); STARTING TIME, TS, (F6.0); TIME INT.
C          OF H-DATA, DELT, (F6.0);  2X ; NO OF INITIAL H-DATA, IN,
C          (I2); NO OF INTERVALS TO BE COMPUTED, K, (F6.0).
C
C       DATA CARDS:
C
C          INITIAL H-DATA , (12F6.0)
C
C       EXPLANATIONS:
C
C          ALPHA = L(RESERVOIR) / L(TOTAL REACH) .
C          GAUGE RATING CURVES ARE DEFINED AS:
C             Q = AH* (H-HO)**BH , INDEX 1 REFERS TO THE UPSTREAM
C             GAUGE, INDEX 2 TO THE DOWNSTREAM ONE.
C          TS IS THE STARTING TIME IN MINUTES OF THE INPUT DATA.
C          DATA : THE PROGRAM READS 'IN' H-DATA, AND ASSUMES THAT
C             ALL FURTHER (K-IN) DATA POINTS ARE EQUAL TO THE LAST
C             INPUT VALUE
C
C       OUTPUT:
C
C          THE PROGRAM PRINTS THE CONTROL CARD DATA AND INITIAL
C          H-DATA; IT THEN ROUTES THE FLOW THROUGH SUCCESSIVE REACHES,
C          WITH THE OUTPUT OF REACH (I-1) + QINC BECOMING INPUT OF
C          REACH (I).
C
C          THE OUTFLOW OF EACH REACH IS PRINTED, AND, IF THE RATING
C          CURVE PARAMETERS ARE GIVEN, THE GAUGE READINGS.
C
C
      DIMENSION  Q(100), Y(2) ,F(2) ,TEMP(2),H(100),T(100),Q1(100),
     1 T1(100)   ,   TIT(10) , TITR (10)
      COMMON FQ, QIM1, BA
      REAL L
C
```

Parameter "Sigma" of text is "Alpha" in 'SNLR'.

```
C     LOOP FOR REACHES
      READ (5,40)  KR, TITR
40    FORMAT ( I2, 10A4)
      DO  41  KK =  1 ,KR
C
C     READ CONSTANTS
      READ (5,30)  N , AL  , QINC
30    FORMAT (I2,2F6.0  )
      READ ( 5,20)  TIT
20    FORMAT  ( 10A4)                                        (HO2,
      READ (5 , 1)          L , AA , BA, QD , AH1,BH1,HO1,AH2,BH2,)
     1 TS , DELT  ,     IN  , K
1     FORMAT(        12F6.0,   2X, 2I2 )
      KL= K
      K= K+1
      IF ( KK .GT. 1 ) GO TO 43
C
C     READ INITIAL HIGHT DATA
      READ (5,2)( H (I), I= 1,IN)
2     FORMAT(12F6.0)
      IF ( IN .EQ. K ) GO TO 4
C
C     COMPLETE INITIAL ARRAY
      DO  3  I = IN, K
3      H(I) =  H(IN)
C
C     CONVERT  HIGHT TO DISCHARGE, COMPUTE TIMES
4     DO 5   I = 1 , K
      T(I) = TS + (I-1) * DELT
      Q(I)=((H(I)-HO1)/AH1)**  (1.  /BH1)
5     CONTINUE
      GO TO 45
C
C     ADJUST FOR KK .GT. 1  AND KLA .NE.  K
43    IN  =  K
      IF ( KLA .EQ. K ) GO TO 45
      DO 90 N1 = KLA, K
90    Q(N1)  =  Q(KLA)
C
C     INITIAL  CONSTANTS
45    AD   =  AA* QD  **BA
      FTT= ( L*AD* BA) / (N*QD*60.0 )
      FT = FTT * ( 1.0 - AL )
      FQ  = ( 1. - AL ) / (FT * AL )
      TC = ( FTT * AL ) / ( Q(1) ** ( 1.0- BA))
      QFAC = Q(1) / QD
      IDT  =  2.0 * DELT / TC
      IF ( IDT .LT. 1  )  IDT = 1
      IF ( QFAC .GT.0.15 ) IDT = 2*IDT
C
C     WRITE INITIAL CONDITIONS
      WRITE (6, 6)    TITR  ,KK
6     FORMAT('1NON-LINEAR RESERVOIR ROUTING' / 'INITIAL CONDITIONS'
     1 / 10A4,  'ROUTING OVER ',I2,'. REACH  ')
      WRITE  (6,21)  TIT
```

```
21    FORMAT(1H0, 10A4 )
      WRITE ( 6, 31)  N , AL   , QINC
31    FORMAT ('0N = NO OF RES. = '   , I5, '   AL(PHA) = L(RES)/L =',
     1 F10.5  /  ' QINCREMENT = ', F10.5 )
      ITS  = TS
      WRITE (6,7)  KL,IN,AA,BA,DELT,IDT,ITS,L, QD,
     1 AH1, BH1,HO1, AH2, BH2,  HO2, AD,  FQ, FT
7     FORMAT ('0K ,  IN      ', 2I7 , / 'AA , BA  ', 2F12.5 , /
     1 'DT OF Q0 IN MIN , NO. STEPS PER DT '  F10.5, I8  /
     2 'STARTING TIME , LENGTH (M) , QD  ', 3X,I4, 2F 12.3 /
     3 'AH1, BH1,HO1,'  ,  3F15.6  /
     4 'AH2, BH2,  HO2,  '  ,  3F15.6 /  /
     5  'AD,  FQ,  FT,  ' , 5X, 3E15.6 //      )
      IF ( KK .GT.  1  )  GO TO 65
      WRITE(6,8) ( I , T(I) , H (I) , Q(I ), I = 1, IN)
8     FORMAT ( ' NO    TIME    LEVEL    DISCHARGE '  //
     1  (  1X,   I2,1X, F7.1,2X, F7.3, 2X, F10.5 ))
C
C     CONVERT Q TO NONDIMENSIONAL Q
65    CONTINUE
      DO 46  I= 1,  K
46    Q(I)  =(Q(I) +  QINC)/  QD
C
C     ADVANCE SOLUTION BY 1 RESERVOIR
      DO 9    I   =1,N
      Y(1)  = T(1) + FT  / ( Q(1)** (1.-BA))
      Y(2)  = Q(1)
      Q1(1) = Y(2)
      T1(1) = Y(1)
      AIN  = IDT
      D      = DELT /AIN
      DO 10 J= 2,K
C
C     COMPUTE  Q(I-1)
      DT = FT / (((Q(J-1) + Q1(J-1))/2.) **(1. - BA))
       TO = T1(J-1) + DELT / 2. - DT
      DO  82 M = 1  , K
      IF (T(M) .GT. TO ) GO TO 85
82    CONTINUE
85    IF (M .EQ.1)  GO TO83
      QIM1  = Q(M-1) +((Q(M) - Q (M-1)) / DELT) *(TO  -T(M-1)    )
      GO TO 89
83    QIM1  = Q (1)
89    CALL RK ( Y , F ,TEMP,D , 2  ,IDT )
12    FORMAT  (6E14.4, 2I5  )
      Q1(J) = Y(2)
      T1(J) = Y(1)
10    CONTINUE
      DO 13  II = 1,K
      Q (II) = Q1(II)
13    T (II) = T1(II)
9     CONTINUE
C
C     PRINT OUT RESULTS AFTER 1 REACH
      DO 50  I = 1,K
```

```
      Q(I) = Q(I)  *  QD
      IF ( BH2 .LE. 0.0 )  GO TO 50
      H(I)  =  AH2 *(Q(I) **BH2 )+ HO2
50    CONTINUE
      WRITE (6,11) N
11    FORMAT('0DISCHARGE AFTER ',I2,' RESERVOIRS' //)
      IF ( BH2 .LE. 0.0) WRITE ( 6,61 ) ( I, T(I),         Q(I),I=1,K)
61    FORMAT ( ' NO    TIME    LEVEL    DISCHARGE ' //
     1  ( 1X,    I2,1X, F7.1,2X,  7X,  2X, F10.5 ))
      IF ( BH2 .GT. 0.0) WRITE ( 6, 8 ) ( I, T(I), H(I), Q(I),I=1,K)
      KLA  =  K
C
41    CONTINUE
      WRITE ( 6 , 70)
70    FORMAT ( 1H1   )
      STOP
C
      END
```

```
      SUBROUTINE AUXRK (Y,F)
C
      DIMENSION Y(2),  F(2)
      COMMON FQ, QIM1, BA
      IF ( Y(2) .LE.  0.0  )  STOP 6
4     F(2)=  FQ* (QIM1*(Y(2)**(1. - BA)) - Y(2)**(2. - BA ))
11    RETURN
      END
```

NON-LINEAR RESERVOIR ROUTING
INITIAL CONDITIONS
BLANEY CREEK, OCT. 13,1968, 1-3-5-4 ROUTING OVER 1. REACH

BL. CK. REACH 1-3, DAM IN 12H15, OUT 45

N = NO OF RES. = 9 AL(PHA) = L(RES)/L = 0.70000
QINCREMENT = 0.02000

K , IN 50 36
AA , BA 3.37500 0.47800
DT.OF QO IN MIN , NO. STEPS PER DT 1.00000 4
STARTING TIME , LENGTH (M) , QD 14 686.000 12.000
AH1, BH1,HO1, 1.320000 0.445000 -0.110000
AH2, BH2, HO2, 1.230000 0.354000 14.000000

AD, FQ, FT, 0.110693E 02 0.255037E 01 0.168043E 00

NO	TIME	LEVEL	DISCHARGE
1	14.0	1.265	1.09607
2	15.0	1.265	1.09607
3	16.0	1.185	0.95794
4	17.0	1.180	0.94965
5	18.0	1.180	0.94965
6	19.0	1.180	0.94965
7	20.0	1.181	0.95130
8	21.0	1.182	0.95296
9	22.0	1.183	0.95462
10	23.0	1.184	0.95628
11	24.0	1.185	0.95794
12	25.0	1.186	0.95960
13	26.0	1.186	0.95960
14	27.0	1.187	0.96127
15	28.0	1.188	0.96293
16	29.0	1.188	0.96293
17	30.0	1.189	0.96460
18	31.0	1.189	0.96460
19	32.0	1.190	0.96627
20	33.0	1.190	0.96627
21	34.0	1.191	0.96794
22	35.0	1.192	0.96962
23	36.0	1.193	0.97129
24	37.0	1.193	0.97129
25	38.0	1.194	0.97297
26	39.0	1.195	0.97464
27	40.0	1.195	0.97464
28	41.0	1.196	0.97632
29	42.0	1.197	0.97800
30	43.0	1.197	0.97800
31	44.0	1.198	0.97969
32	45.0	1.199	0.98137
33	46.0	1.285	1.13223
34	47.0	1.290	1.14137
35	48.0	1.293	1.14687
36	49.0	1.295	1.15054

'SNLR'

Printout for one of the computations of Figure 24

DISCHARGE AFTER 9 RESERVOIRS

NO	TIME	LEVEL	DISCHARGE
1	19.2	15.279	1.11607
2	20.2	15.279	1.11607
3	21.2	15.279	1.11605
4	22.2	15.279	1.11603
5	23.2	15.279	1.11596
6	24.2	15.279	1.11568
7	25.2	15.278	1.11488
8	26.2	15.278	1.11302
9	27.2	15.276	1.10951
10	28.2	15.274	1.10382
11	29.2	15.270	1.09577
12	30.2	15.266	1.08554
13	31.2	15.261	1.07365
14	32.2	15.256	1.06083
15	33.2	15.251	1.04785
16	34.2	15.245	1.03539
17	35.2	15.240	1.02396
18	36.2	15.236	1.01392
19	37.2	15.232	1.00543
20	38.2	15.229	0.99853
21	39.2	15.227	0.99313
22	40.2	15.225	0.98908
23	41.2	15.224	0.98619
24	42.2	15.223	0.98427
25	43.2	15.223	0.98311
26	44.2	15.222	0.98255
27	45.2	15.222	0.98244
28	46.2	15.222	0.98266
29	47.2	15.223	0.98314
30	48.2	15.223	0.98379
31	49.2	15.223	0.98459
32	50.2	15.224	0.98548
33	51.2	15.224	0.98645
34	52.2	15.225	0.98748
35	53.2	15.225	0.98858
36	54.2	15.226	0.98982
37	55.2	15.226	0.99144
38	56.2	15.227	0.99390
39	57.2	15.229	0.99787
40	58.2	15.232	1.00415
41	59.2	15.236	1.01337
42	60.2	15.241	1.02574
43	61.2	15.248	1.04094
44	62.2	15.255	1.05812
45	63.2	15.262	1.07612
46	64.2	15.270	1.09373
47	65.2	15.276	1.10993
48	66.2	15.282	1.12406
49	67.2	15.287	1.13582
50	68.2	15.290	1.14521
51	69.2	15.293	1.15245

NON-LINEAR RESERVOIR ROUTING
INITIAL CONDITIONS

BLANEY CREEK, OCT. 13,1968, 1-3-5-4 ROUTING OVER 2. REACH

BL. CK. REACH 3-5, DAM IN 12H15, OUT 45

N = NO OF RES. = 6 AL(PHA) = L(RES)/L = 0.70000
QINCREMENT = 0.08000

K , IN 60 61
AA , BA 3.48300 0.52700
DT OF QO IN MIN , NO. STEPS PER DT 1.00000 4
STARTING TIME , LENGTH (M) , QD 14 335.000 12.000
AH1, BH1,HO1, 1.230000 0.354000 14.000000
AH2, BH2, HO2, 1.459999 0.331000 9.500000

AD, FQ, FT, 0.129027E 02 0.270925E 01 0.158188E 00

DISCHARGE AFTER 6 RESERVOIRS

NO	TIME	LEVEL	DISCHARGE
1	22.0	11.049	1.19607
2	23.0	11.049	1.19606
3	24.0	11.049	1.19605
4	25.0	11.049	1.19604
5	26.0	11.049	1.19603
6	27.0	11.049	1.19602
7	28.0	11.049	1.19601
8	29.0	11.049	1.19598
9	30.0	11.049	1.19592
10	31.0	11.049	1.19576
11	32.0	11.049	1.19539
12	33.0	11.049	1.19466
13	34.0	11.048	1.19332
14	35.0	11.047	1.19110
15	36.0	11.046	1.18775
16	37.0	11.044	1.18306
17	38.0	11.041	1.17693
18	39.0	11.038	1.16942
19	40.0	11.034	1.16071
20	41.0	11.030	1.15109
21	42.0	11.025	1.14091
22	43.0	11.021	1.13055
23	44.0	11.016	1.12038
24	45.0	11.012	1.11071
25	46.0	11.008	1.10179
26	47.0	11.004	1.09379
27	48.0	11.001	1.08683
28	49.0	10.998	1.08094
29	50.0	10.996	1.07609
30	51.0	10.994	1.07224
31	52.0	10.993	1.06929
32	53.0	10.992	1.06713
33	54.0	10.991	1.06565
34	55.0	10.991	1.06474
35	56.0	10.990	1.06429

36	57.0	10.990	1.06422
37	58.0	10.990	1.06445
38	59.0	10.991	1.06491
39	60.0	10.991	1.06557
40	61.0	10.991	1.06642
41	62.0	10.992	1.06749
42	63.0	10.993	1.06889
43	64.0	10.993	1.07081
44	65.0	10.995	1.07353
45	66.0	10.996	1.07740
46	67.0	10.999	1.08282
47	68.0	11.002	1.09009
48	69.0	11.007	1.09942
49	70.0	11.012	1.11079
50	71.0	11.018	1.12394
51	72.0	11.024	1.13837
52	73.0	11.031	1.15345
53	74.0	11.037	1.16843
54	75.0	11.043	1.18258
55	76.0	11.049	1.19521
56	77.0	11.053	1.20582
57	78.0	11.057	1.21420
58	79.0	11.060	1.22043
59	80.0	11.061	1.22481
60	81.0	11.063	1.22774
61	82.0	11.063	1.22962

NON-LINEAR RESERVOIR ROUTING
INITIAL CONDITIONS
BLANEY CREEK, OCT. 13,1968, 1-3-5-4   ROUTING OVER 3. REACH

BL. CK. REACH 5-4, DAM IN 12H15, OUT 45

N = NO OF RES. = 12   AL(PHA) = L(RES)/L = 0.70000
QINCREMENT = 0.0

K , IN   70   71
AA , BA   3.11000   0.43900
DT OF QO IN MIN , NO. STEPS PER DT   1.00000   5
STARTING TIME , LENGTH (M) , QD   14   930.000   13.000
AH1, BH1,HO1,   1.459999   0.331000   9.500000
AH2, BH2, HO2,   0.780000   0.630000   4.000000

AD, FQ, FT,   0.958917E 01   0.341546E 01   0.125480E 00

DISCHARGE AFTER 12 RESERVOIRS

NO	TIME	LEVEL	DISCHARGE
1	27.8	4.873	1.19607
2	28.8	4.873	1.19606
3	29.8	4.873	1.19605
4	30.8	4.873	1.19604
5	31.8	4.873	1.19603
6	32.8	4.873	1.19602
7	33.8	4.873	1.19601
8	34.8	4.873	1.19600
9	35.8	4.873	1.19600
10	36.8	4.873	1.19599
11	37.8	4.873	1.19598
12	38.8	4.873	1.19597
13	39.8	4.873	1.19596
14	40.8	4.873	1.19595
15	41.8	4.873	1.19593
16	42.8	4.873	1.19591
17	43.8	4.873	1.19588
18	44.8	4.873	1.19583
19	45.8	4.873	1.19575
20	46.8	4.873	1.19562
21	47.8	4.873	1.19539
22	48.8	4.873	1.19501
23	49.8	4.872	1.19441
24	50.8	4.872	1.19350
25	51.8	4.871	1.19217
26	52.8	4.870	1.19034
27	53.8	4.869	1.18790
28	54.8	4.868	1.18476
29	55.8	4.866	1.18089
30	56.8	4.864	1.17626
31	57.8	4.862	1.17089
32	58.8	4.859	1.16485
33	59.8	4.856	1.15822
34	60.8	4.852	1.15112
35	61.8	4.849	1.14370

36	62.8	4.845	1.13610
37	63.8	4.842	1.12848
38	64.8	4.838	1.12099
39	65.8	4.835	1.11374
40	66.8	4.832	1.10686
41	67.8	4.828	1.10044
42	68.8	4.826	1.09455
43	69.8	4.823	1.08925
44	70.8	4.821	1.08456
45	71.8	4.819	1.08049
46	72.8	4.817	1.07704
47	73.8	4.816	1.07419
48	74.8	4.815	1.07190
49	75.8	4.814	1.07014
50	76.8	4.813	1.06887
51	77.8	4.813	1.06807
52	78.8	4.813	1.06771
53	79.8	4.813	1.06780
54	80.8	4.813	1.06837
55	81.8	4.814	1.06947
56	82.8	4.815	1.07120
57	83.8	4.816	1.07368
58	84.8	4.817	1.07706
59	85.8	4.819	1.08148
60	86.8	4.822	1.08709
61	87.8	4.825	1.09397
62	88.8	4.829	1.10216
63	89.8	4.834	1.11159
64	90.8	4.839	1.12211
65	91.8	4.844	1.13346
66	92.8	4.850	1.14527
67	93.8	4.855	1.15715
68	94.8	4.860	1.16869
69	95.8	4.866	1.17952
70	96.8	4.870	1.18935
71	97.8	4.874	1.19797