Environment Canada
Environnement Canada

TECHNICAL BULLETIN NO. 79

INLAND WATERS DIRECTORATE,
WATER RESOURCES BRANCH,
OTTAWA, CANADA, 1974

Environment Canada   Environnement Canada

Peak Flows by the Slope-Area Method

A. G. Smith

TECHNICAL BULLETIN NO. 79

INLAND WATERS DIRECTORATE,
WATER RESOURCES BRANCH,
OTTAWA, CANADA, 1974

Cat. No.: En 36-503/79

Contract No. 02KX.KL327-3-8060
Thorn Press Limited

Contents

Illustrations

Tables

SYMBOLS AND UNITS

Symbol	Definition	Unit
A	Area	ft^2
g	Gravitational constant (acceleration).	ft/sec^2
hf	Head loss due to friction	ft
hv	Velocity head at a section	ft
K	Conveyance of a section	ft^3/sec
L	Length of reach	ft
n	Manning roughness coefficient	$ft^{1/6}$
Q	Total discharge	ft^3/sec
Qe	Estimated discharge	ft^3/sec
R	Hydraulic radius	ft
S	Friction Slope	ft
V	Mean velocity of flow in a section	ft/sec
1,2	Subscripts which denote the location of cross-sections or section properties in downstream order	
α	Velocity-head coefficient	
Δ	Difference in values, as Δh is the difference in head; part of total	
Σ	Summation of values	
k	Coefficient	

Abstract

Stream discharges are usually measured by the current meter method. During floods, however, it is frequently impossible or impractical to measure discharges by this means. Many peak discharges must be determined after the passage of the flood by indirect methods, such as the slope-area method.

The indirect method of determining peak discharge is based on hydraulic equations, which relate the discharge to the water-surface profile and the geometry of the channel. A field survey is made to determine the water-surface profile and the channel characteristics for a series of discharge measurements over a suitable range of stage. Values of the coefficient of roughness and the channel conveyance are calculated as a basis for computing the flood discharge. Detailed descriptions of the general procedures used in collecting field data and in computing discharge are given in the report.

The study for the Quesnel River, Station 08KH006, is used to illustrate the field and office procedures used in calculating peak flows. The calculations are included in the report.

Introduction

NEED FOR INDIRECT MEASUREMENTS

Stream discharges are usually measured by direct means (e.g., current meter, fluorometer), which require the presence of a technician. During floods, however, it is frequently impossible or impractical to measure the peak discharges as they occur. Roads may be impassable, current meter sites may be flooded or destroyed (e.g., cableways, bridges); knowledge of the timing of the flood peak may not be obtained sufficiently in advance to permit reaching the site at the opportune time; the peak could be so sharp that a satisfactory current meter measurement could not be made even if a technician were at the site; flow of debris or ice could prevent use of a current meter or access by float plane; or limitation of personnel could make it impossible to visit the numerous locations during a short flood period. Many peak flows, therefore, must be determined by indirect methods after the passage of the flood.

VALUE OF PEAK FLOW DATA

A knowledge of peak discharges is extremely important for the design of flood control projects involving huge expenditures. The peak discharges, usually estimated from the extended portion of rating curves, are often accepted without question of accuracy. It is important then that the extension of the rating curve be based on proper data, and the best procedures known should be used to obtain those data. Indirect measurements by the slope-area method will provide a sound basis for the extrapolation of rating curves to verify high flow measurements, and for safeguarding against loss of high flow data.

COMPARISON OF INDIRECT MEASUREMENTS WITH DIRECT MEASUREMENTS

To evaluate the accuracy of indirect methods, comparisons have been made with peak discharges measured by current meter. The results have indicated that the indirect method is reliable in most cases (Benson and Dalrymple, 1967). During the floods of 1948 in the Columbia basin, comparative studies using the slope-area method were made at 22 locations where the discharge was known. The average error was 6.7% and the extreme was 25%. Values for the coefficient of roughness "n" were estimated from streambed materials at each site.

SLOPE-AREA METHOD

The slope-area method for obtaining discharges is based on a uniform flow equation which includes consideration of channel characteristics, water-surface profiles, and a roughness of retardation coefficient. The drop in the water-surface profile for a uniform reach of channel represents losses caused by bed roughness. The selection of the coefficient of roughness "n" is a critical step in slope-area measurements because of subjective judgment. The procedure outlined in this report reduces this subjective judgment in obtaining values of "n".

After the field surveys, usually done at low stage, water-surface profiles are obtained for a series of discharge measurements taken during normal operation of the station. Values for the coefficient of roughness and channel conveyance "K" are computed for each measurement. The computed values of "K" are used to draw a mean conveyance curve for the reach on which to base future high flow calculations. The peak flow profile is obtained by a series of crest stage gauges established along the selected reach at breaks in the water-surface profile.

Selection of Network

SELECTION OF ACTIVE STATIONS

All active stream gauging stations are reviewed to determine which ones require better definition in the upper portion of the rating curve. The selection of stations is based on the E/M ratio, where "E" is the highest estimated flow and "M" is the highest measured flow for a station. A high E/M ratio indicates that the rating curve has been extended for a considerable distance.

A sample list is shown in Table 1 which includes the station number and name, the measured and estimated high flow, the ratio of estimated to measured flow, the date of high flow, the head office record date, the type of record (daily mean, or instantaneous), and the Sub-Office up-dating. The stations are arranged in order of localities, and priorities are then set for surveys based on the E/M ratio of each station, availability of manpower in each region, and availability of equipment.

SELECTION OF SITE

A preliminary selection of sites for indirect measurements is made for each station from an examination of aerial photographs and topographic and geological maps. The sites are marked on large-scale maps so that they are readily identifiable from the air or from the ground.

A thorough ground reconnaissance of the stream channel around each station is necessary for selection of the best site for indirect measurements. Every site represents a distinct hydraulic problem, and a thorough knowledge of hydraulic principles is essential for proper selection. As ideal conditions rarely exist, judgment must be used in choosing the most favourable site by weighing the advantages and disadvantages of each.

The measuring site must be so located that no major tributaries enter between it and the point at which the discharge is desired. Channel storage can also be significant if the measuring site is some distance from the gauged point. If the storm producing the flood covers the basin, the peak may increase in a downstream direction; if the storm covers only the upstream part of the basin, the peak may decrease in the main channel. Distance from the gauging point becomes more important for smaller drainage areas and for sudden floods of short duration. Sometimes it is preferable to accept a site which is near the gauge, but where conditions are less favourable.

SELECTION OF REACH

The selection of a suitable reach is probably the most important part of a slope-area measurement. The geometry of the channel in the reach is very important. Areas where there are marked changes in the shape of the channel along a reach should be avoided because of uncertainties regarding the value of the velocity head coefficient. The channel should be as uniform as possible within the reach and the reach should be contracting rather than expanding. Straight reaches are preferred but are seldom found in nature.

The method assumes that the cross-sectional area is fully effective and is carrying water in accordance with the conveyance for various portions of the section. For this reason it is desirable that the cross-section be uniform for some distance above the reach, so that discharge will be distributed in accordance with channel depths, roughness, and shape. Conditions, either upstream or downstream from a reach, which will cause an unbalanced distribution, are undesirable. For example, for some distance downstream from a bridge that constricts the width, the effective flow will be contained within the centre of the channel; the sides of the channel will not carry water in proportion to the computed conveyance and velocity may even be negative. Natural channel constrictions or protrusions may have the same effect. A sudden deepening of the channel may also represent a non-effective area. Sections having such conditions should be avoided when selecting slope-area reaches.

Sometimes slope-area reaches must be selected in mountainous areas where the channels are very rough and steep, and may have free fall over riffles and boulders at low flows. An inspection of the reach will usually indicate whether free fall exists at high stages. The Manning equation does not apply when free fall exists; therefore, free-fall reaches should be avoided; otherwise, the reliability of the computed discharge will be low. Channel bends often govern the length of a suitable reach. The influence of the

Table 1. Sample list of active stream gauging stations requiring better definition of upper portion of rating curve.

STATION No.	STATION NAME	MEASURED HIGH FLOW	ESTIMATED HIGH FLOW	RATIO E/M	DATE OF MEASURED HIGH FLOW	VANCOUVER RECORD TO DATE	TYPE OF RECORD DAILY OR INST.	UPDATING BY FIELD				
								MEASURED HIGH FLOW	ESTIMATED HIGH FLOW	RATIO E/M	TYPE OF RECORD	DATE OF MEASURED HIGH FLOW
08KH018	BARLOW CREEK NEAR QUESNEL	146	412	2.8	26.4.65	19.3.69	DAILY					
07FC001	BEATTON RIVER AT FORT ST. JOHN	21800	53400	2.4	18.5.67	25.2.69	DAILY	23700		2.3	DAILY	2.5.69
07FC003	BLUEBERRY RIVER BELOW AITKEN CREEK	3210	9520	3.0	16.5.67	27.3.67	INST.	3830		2.5	INST.	1.5.69
10CC001	FORT NELSON RIVER NEAR FORT NELSON	34900	121000	3.5	1.6.61	11.3.69	DAILY	54000		2.2	DAILY	28.5.70
07FA001	HALFWAY RIVER NEAR FARRELL CREEK	15800	258000	16.3	16.8.62	22.9.68	INST.		70000	4.4		
10CD001	MUSKWA RIVER AT FORT NELSON	51100	140000	2.7	19.6.62	11.3.69	DAILY					

bend on velocity distribution, slope, and water-surface elevations continues for some distance downstream from the bend.

The reach should be long enough to develop a fall that is well beyond the range of error due to uncertainties regarding the computation of velocity head. In general, the accuracy of a slope-area measurement will improve as the length of the reach is increased. The length of a desirable reach, however, is governed by the geometry of the channel and the practical difficulties of surveying long reaches of river channel. One or more of the following criteria should be met in selecting the length of a slope-area reach:

1. The length of the reach should be equal to or greater than 75 times the mean depth of the channel.
2. The fall in the reach should be equal to, or greater than, the velocity head (see Chapter 3, Computation of Friction Slope).
3. The fall in the reach should be equal to, or greater than, 0.50 foot.

Collection of Field Data

The field survey should be made with a high degree of care, with precautions taken to avoid error by using all possible checks. Various instruments are available for making field surveys, but experience has shown that an engineer's transit is best suited for the job.

The type of survey recommended is one that is called a "transit-stadia" survey. This method combines vertical and horizontal control in one operation that is fairly accurate, simple, and speedy. To obtain the vertical accuracy required for setting permanent controls for the water-surface profile, a level circuit run with an engineer's level is necessary.

In any office where surveys or indirect measurements are made at infrequent intervals it is best to always select one method, because personnel cannot maintain expertise in all types of instruments and surveys. However, it is recommended that an "electronic recording tacheometer" be used for future surveys. This instrument records the horizontal direction and vertical angle, and computes the slope distance directly, thus eliminating the necessity for running a separate circuit of levels for the vertical control.

VERTICAL CONTROL

If the site chosen for the indirect measurement is near the gauging station, the survey datum should consist of gauge datum or gauge datum plus a convenient constant to avoid negative elevations. Otherwise, an arbitrary elevation may be assumed for a permanent reference point. The reference point can be a large nail in a tree or stump, or a prominent rock point adequately referenced and described to permit recovery in later years if necessary. All vertical control for water-surface profiles should be determined to accuracies of ±0.01 foot.

HORIZONTAL CONTROL

One begins the horizontal control survey by back-sighting on a hub in the traverse, preferably the preceding one. This line is referenced to magnetic North if desirable, although it is not necessary. Stadia distance and angle for each surveyed point are read. Angles to the nearest 1/10 of a minute of arc for all hubs and reference points are red, as well as to the nearest minute for side shots. The horizontal traverse and level circuits are generally closed. If the angular error is near ±1 minute times the square root of the number of stations, the horizontal control is accepted and no adjustments are made. For example, for a traverse with sixteen stations, the allowable error would be $\pm 1\sqrt{16} = \pm 4$ minutes. If the elevation error is not greater than 0.8 foot times the square root of the number of miles, it indicates no blunder, and the elevation of the traverse can be adjusted. For example, for a traverse with one mile of levels, the allowable error would be $0.8\sqrt{1} = 0.8$ foot. Elevations are usually adjusted in proportion to the length of the traverse courses.

FIELD NOTES

An example of the recommended form of keeping field notes is shown on Figure 2. Note the headings and pertinent data listed at the top of Figure 2. A step-by-step procedure covering both the horizontal and the vertical control survey is given in Appendix I.

GROUND PLAN

A plan sketch should be drawn showing all natural features of the site which are pertinent to the measurement:

(1) Show the channel for some distance upstream from the actual reach so that the flow pattern and its effect on the high-water profiles can be judged.

(2) Show the direction of flow in the channel with an arrow.

(3) Locate tributaries and any minor bypass channels; indicate any high ground, ridges, riffles, or other features which would affect the distribution or type of flow.

(4) Describe the type of terrain and ground cover for approximately five feet above any high-water marks.

Hydraulic Principles of the Slope-Area Method

GENERAL HYDRAULIC PRINCIPLES

The slope-area method of indirect measurement makes use of the energy equation for computing discharge and involves the following general factors:

1. Physical characteristics of the channel — dimensions and conformation of channel within the reach used, and boundary conditions.

2. Water-surface elevations between significant points.

3. Hydraulic factors based on physical characteristics, water-surface elevations, and discharge, such as roughness coefficients and discharge coefficients.

THEORETICAL BASIS FOR SLOPE-AREA METHOD

In the slope-area method, discharge is computed on the basis of a uniform-flow equation involving channel characteristics, water-surface profiles, and a roughness coefficient.

Manning's equation has been adopted because of its simplicity of application. Written in terms of discharge, the equation is:

$$Q = \frac{1.486}{n} AR^{2/3} S^{1/2}$$

where Q = discharge
A = cross-sectional area
R = hydraulic radius
S = friction slope
n = roughness coefficient

The energy equation for a reach of non-uniform channel between Section 1 and 2, shown on Figure 1, is:

$$(h + hv)_1 = (h + hv)_2 + (hf)_{1\text{-}2} + k\,(\Delta hv)_{1\text{-}2}$$

where h = elevation of the water surface at the respective sections above a common datum

hv = velocity head at the respective section ($V^2/2g$)

hf = energy loss due to boundary friction in the reach

Δhv = upstream velocity head minus the downstream velocity head

$k(\Delta hv)$ = energy loss due to acceleration or deceleration in a contracting or expanding reach

k = a coefficient

The friction slope S used in the Manning equation is defined as:

$$S = \frac{hf}{L} = \frac{\Delta h + \Delta hv - k(\Delta hv)}{L}$$

where Δh = difference in water surface elevation at two sections

L = length of the reach under consideration

COMPUTATION OF FRICTION SLOPE

The velocity head, (hv), at each section is computed as:

$$hv = \frac{\alpha V^2}{2g}$$

where V = mean velocity in the section
α = velocity-head coefficient

The value of "α" is assumed to be 1.0 if the section is not subdivided. The energy loss due to contraction or expansion of the channel in the reach is assumed to be equal to the difference in velocity heads at the two sections, (Δhv), multiplied by a coefficient k. The value of k is assumed to be zero for contracting reaches and 0.5 for expanding reaches. However, both the procedure and the coefficient are questionable for expanding reaches (Dalrymple and Benson, 1967). Major expansions are to be avoided in selecting sites for indirect measurements. The value of Δhv is computed as the upstream velocity head minus the downstream velocity head. Thus the friction slope used in the Manning equation is computed algebraically as:

$$S = \frac{\Delta h + \Delta hv}{L}$$ (when Δhv is negative, i.e., the reach is contracting)

$$S = \frac{\Delta h + \Delta hv/2}{L}$$ (when Δhv is positive, i.e., the reach is expanding)

(5) Show the approximate position and direction of the camera for all pictures.

An illustration of a ground plan is given on Figure 3.

CROSS-SECTIONS

Cross-sections represent the geometry of the reach and should be located at strategic places. The following points should be observed when locating cross-sections:

(1) Cross-sections should be identified as Sections 1, 2, 3, 4, etc., in downstream order.

(2) Cross-sections should be located as nearly as possible at right angles to the direction of flow. In large streams it may be necessary to break the cross-section at one or more points to maintain the section roughly perpendicular to the flow.

(3) High-water marks and the profile should be plotted in the field before surveying the sections. Rough plotting is adequate provided that high-water marks have been surveyed on each bank.

(4) Cross-sections should be located at the major breaks in the high-water profile.

(5) Permanent reference points should be located near all major breaks in the high-water profile. These points will be used to obtain water-surface profiles for indirect discharge measurements.

In extremely rough channels, the cross-sections are to be located so as to represent average or typical conditons. Where large scattered boulders are present, the cross-sections should be located so as not to wholly avoid them nor to include a disproportionate number of them.

CREST-STAGE GAUGES

Crest-stage gauges to be used for indirect discharge measurements should be placed at, at least, three points where major breaks occur in the water-surface profile. If at all possible the stream gauging station should also be included in the gauge network.

The gauges should be located on the stream side that affords the best protection from debris during high stages. At some locations, a series of crest gauges should be established some distance up the bank at each section to obtain the variation of high flows from year to year.

If the major breaks in the water-surface profile can be identified, crest-stage gauges can be installed at the time of the survey. Otherwise the auxiliary gauges can be installed at a later date by field technicians.

CROSS-SECTION SURVEY

The procedures for surveying and sounding cross-sections are outlined in Appendix II.

The field notes for cross-sections should provide the following information:

(1) Location and stationing of two stations, usually at the ends of cross-sections.

(2) Stationing and elevation of all intermediate cross-section points.

(3) Stationing and elevation (within hundredths of a foot) of water surface on both banks at time of survey.

The recommended form for cross-section notes is shown on Figure 4. Figure 5 shows the corresponding form for notes taken in the boat when a tagline or stadia is used only for obtaining distances.

PHOTOGRAPHS

Photographs should be taken at the time for the field survey. Adequate photographs will allow review and appraisal of the site conditions by those who have not seen the site. They make possible a comparison in the office with reference photographs illustrating values of Manning's roughness coefficient.

Pictures should show general field condition and the channel layout. Views upstream and downstream through the main channel are the most important. Closeups of important details are useful, such as those showing the channel bottom and roughness on both banks.

At the time of photographing it is necessary to make notes describing the location of each shot and what is pictured. The description can be made part of the field notes. For complicated, large area sites, it helps to show an identifying number and arrow on a field sketch.

Office Computations

Standard methods should be used to simplify review and checking. The procedures recommended have been standardized to increase the probability of obtaining reliable results and to simplify computation.

PLAN

The plan is plotted from the field notes using a scale of between 1″ = 20′ and 1″ = 200′, according to the size of the area covered and the amount of detail needed. The plotting convention is to plot the exact position as a dot and to write the elevation of each mark, using the dot as the decimal point.

All roads, fence lines and natural features pertinent to the problem are included in the plan. The location of cross-sections, transit stations, bench marks, reference marks, and gauging stations are shown, if applicable. Arrows are used to show the direction of flow and magnetic North. The scale of the plan is noted and the plan is completed by drawing in the contour lines at either one- or two-foot intervals. The underwater lines are denoted as dashed and the surface lines as solid. An example of the finished drawing is given on Figure 6.

One of the principal factors in the discharge computations for any type of flow is the drop in water-surface elevation from one cross-section to another. This is best computed by drawing a continuous profile through the measurement reach.

To draw water profiles, it is necessary to adopt some system of stationing for referring the water marks to a base line that represents the mean path of the water. The base line is drawn perpendicular to the cross-sections where they intersect. Locate the zero for stationing along the channel slightly upstream from the furthest upstream section so that all stationing is positive and increases downstream.

If the channel is reasonably straight, even though the banks are not parallel, a straight base line is drawn through the approximate centre of the channel. The station of any mark is obtained by a right angle projection from the base line. If the channel is slightly curved, the base line is drawn as a series of broken straight lines.

It is preferable to express distances for stations in feet, such as 150′ rather than 1 + 50′.

WATER-SURFACE PROFILES

By convention the stationing increases from left to right, so that the profile slopes downward to the right. The vertical scale should be represented so that hundredths of a foot may easily be read, usually 0.5, 1, or 2 feet to the inch, with a horizontal scale selected so that the average slope of the profile will be about 1 vertical to 4 horizontal. The ends of the cross-sections are shown by vertical dashed lines. A separate plot is made for each bank by offsetting the vertical scales. The elevations of the profiles on both banks rarely coincide, and better results are obtained by analyzing each bank separately.

After the profiles have been drawn, a summary table should be made on the sheet showing the stations and elevations for each cross-section as shown on Figure 7.

CROSS-SECTIONS

Each cross-section should be plotted separately with the left bank at the left side of the sheet, so that the stationing proceeds from left to right. The upstream section is to be placed at the top of the sheet, with other sections below in downstream order. Large scale drawings are not necessary; three or four sections may be included in a page-size form. The same scale is to be used for each section.

The plotting scales are usually distorted; the most commonly used horizontal to vertical scale ratio is 5:1 (e.g., 1″ = 50′ horizontal to 1″ = 10′ vertical). For wide cross-sections, horizontal scale ratios of 10 or even 20 to 1 may be necessary. The left and right bank water-surface elevations should be connected by a straight line as shown on Figure 7.

CROSS-SECTION PROPERTIES

The reach selected for discharge computation should be a contracting rather than an expanding one. Reaches where

Table 2. Sample list of cross-sectional widths, areas and hydraulic radius for the mean reach between section No. 2 and section No. 4, gauge No. 2 – gauge No. 4.

CROSS SECTION NUMBER	ELEVATION OF WATER SURFACE													
	99.00		100.00		101.00		102.00		103.00		104.00		105.00	
12	0		0		75		227		252		276		289	
13	0		0		119		220		247		272		285	
14	0		28		131		215		242		272		282	
15	0		46		143		210		239		264		275	
16	0		67		147		200		225		247		259	
17	0		77		135		179		206		232		249	
18	0		82		150		200		224		246		265	
19	0		78		148		210		230		290		270	
20	0		73		146		215		232		261		269	
21	0		67		142		215		234		252		265	
22	0		73		148		210		233		255		267	
23	0		82		142		195		225		250		265	
24	0		86		135		185		218		246		263	
25	0		69		130		176		212		251		269	
26	0		78		122		167		217		267		285	
27	0		38		100		160		222		292		307	
28	0		0		77		155		235		310		321	
29	0		0		55		150		232		320		330	
30	0		0		25		150		240		330		337	
31	0		0		0		181		240		332		340	
Σb	0		947		2264		3790		4602		5415		5692	
$\bar{b} = \frac{(\Sigma b)}{20}$			47.35		113.20		189.50		230.10		270.75		284.60	
AVERAGE b		23.675		80.275		151.350		209.800		250.425		277.675		291.300
ΔE		1.0		1.0		1.0		1.0		1.0		1.0		1.0
$\Delta A = (b)\Delta E$		23.675		80.275		151.350		209.800		250.425		277.675		291.300
CUMULATIVE AREA TO HIGHER EL.	0		23.675		103.950		255.300		465.100		715.525		993.200	
$R = \frac{A}{b}$			0.500		0.918		1.347		2.021		2.643		3.490	

CROSS SECTION NUMBER	ELEVATION OF WATER SURFACE												
	106.00		107.00		108.00		109.00		110.00		111.00		112.00
12	300		310		320		335		350		360		368
13	297		305		313		326		339		346		352
14	293		302		310		320		330		340		350
15	284		298		311		315		322		335		350
16	271		281		293		303		312		328		341
17	266		279		292		302		312		328		335
18	282		291		300		309		317		330		341
19	285		295		305		314		322		338		350
20	287		295		304		318		326		336		348
21	281		290		300		312		323		336		347
22	280		296		310		321		333		344		357
23	280		300		320		334		348		357		367
24	280		310		337		348		360		369		377
25	286		314		340		350		360		370		390
26	301		322		342		350		357		370		385
27	320		332		345		351		357		370		384
28	332		340		347		354		366		370		382
29	340		346		352		356		364		372		390
30	345		351		357		365		372		380		387
31	350		357		362		371		382		390		400
Σb	5960		6211		6460		6653		6844		7060		7279
$\bar{b} = \frac{(\Sigma b)}{20}$	298.00		310.55		323.00		332.65		342.20		353.00		363.95
AVERAGE b		304.275		316.775		327.825		337.425		347.600		358.475	
ΔE		1.0		1.0		1.0		1.0		1.0		1.0	
$\Delta A = (b)\Delta E$		304.275		316.775		327.825		337.425		347.600		358.475	
CUMULATIVE AREA TO HIGHER EL.	1284.500		1588.775		1905.550		2233.375		2570.800		2918.400		3276.875
$R = \frac{A}{b}$	4.310		5.116		5.900		6.714		7.515		8.267		9.004

the channel is both expanding and contracting should be avoided. The best reach can be quickly determined by calculating the area for each section.

A special form is used for computation of mean cross-section properties of the reach: area, hydraulic radius, and width (Davidian, 1964). The method of computing the mean values for the reach follows:

(1) Divide the selected reach into an equal number of section (ten or more depending on length).

(2) For each section calculate the width of each selected contour elevation.

(3) Obtain mean widths for each stage (contour elevation) by totalling the widths for each stage and dividing each total by the number of cross-sections.

(4) Obtain mean channel areas for each elevation by accumulating all incremental areas up to each specified elevation. Incremental areas are obtained by multiplying the mean width for two successive elevations by the difference between the two successive elevations.

(5) Obtain the hydraulic radius by dividing cumulative areas by the average width for the corresponding elevation (see Table 2).

(6) Compute the incremental cross-sectional area for the sections at both the beginning and end of the reach. Plot the depth-area and depth-hydraulic radius curves for the mean cross-sectional values and the depth-area curve for the upstream and downstream sections as shown on Figures 8 to 11.

COMPUTER PROGRAMS

There are three programs available for the Monroe 1666 Programmable Computer to process survey notes. The program and function of each is as follows:

(1) Traverse (program No. 4008Q) – calculates the co-ordinates of each station in the traverse and gives the North and East co-ordinate errors.

(2) Traverse (program No. 4009Q) – corrects the co-ordinates calculated in program No. 4008Q.

(3) Stadia – calculates the horizontal and vertical distances, computes the co-ordinates and the elevation for each secondary point.

A computer program is now being produced to combine all of the hydraulic computations for the method of indirect measurements as outlined in this report.

Quesnel River Study

This section has been added to provide an example of the procedures to be followed in the computation of a discharge by the slope-area method. The Quesnel River project is not the best example as not enough measurements were obtained to define either the conveyance curve or the roughness coefficient with sufficient reliability. However, the channel survey does portray an example of an expanding and a contracting reach. The contracting reach, from Section 2 to Section 4, was chosen on which to base the calculations for a high flow estimate by the slope-area method.

COMPUTATION OF THE COEFFICIENT OF ROUGHNESS AND THE CHANNEL CONVEYANCE

The roughness coefficient (n) and the conveyance (K)

$$\left[K = \frac{(1.486\ A\ R^{2/3})}{n}\right]$$

are computed from several measurements (Table 3) for which the water-surface elevations have been obtained for the channel. Table 4 shows the step-by-step procedure for calculating the "n" and "K" values from measured discharges.

COMPUTATION OF DISCHARGE BY THE SLOPE-AREA METHOD

$$\text{Manning's equation } Q = \frac{1.486}{n} A\ R^{2/3}\ S^{1/2}$$

can be condensed to:

$$Q = KS^{1/2}$$

where K = mean conveyance of the reach
S = energy gradient or friction slope

Table 3. Sample list of computed values of channel characteristics for the selected reach for four discharge measurements, Station 08KH006, Quesnel River near Quesnel.

DATE OF MEASUREMENT	MEASURED DISCHARGE	SECTION 2		SECTION 4		REACH BETWEEN SECTION 2 AND SECTION 4						
		AREA	VELOCITY	AREA	VELOCITY	AREA	VELOCITY	"R"	FALL	SLOPE	"n"	"K"
14 . 5 . 70	7,600	1,670	4.55	1,440	5.28	1,636	4.65	5.24	0.62	0.000,527	0.0222	330,000
28 . 4 . 71	9,900	1,970	5.03	1,730	5.72	1,922	5.15	5.94	0.69	0.000,589	0.0230	407,000
12 . 5 . 71	17,000	2,540	6.69	2,245	7.57	2,435	6.98	7.19	0.96	0.000,795	0.0224	602,000
4 . 6 . 71	23,600	3,105	7.60	2,710	8.71	2,920	8.08	8.28	1.25	0.001,025	0.0238	746,000

A mean conveyance curve is drawn for the reach by using the computed values obtained from the measured discharges (Figure 12). This curve can be extended into the high ranges with confidence as all the factors that determine the conveyance are obtained from field surveys with the exception of the roughness coefficient "n". This coefficient can be calculated for each discharge measurement and as it has a relatively narrow range of variation for in-bank flow, an average value is used.

Table 4. Computation of the coefficient of roughness "n" and the conveyance "K" for discharge measurements at Station 08KH006, Quesnel River near Quesnel

DISCHARGE MEASUREMENT: 7,600 cfs

$GH_2 = 7.47$, $GH_4 = 6.85$

SECTION NO. 2

$h_2 = 7.47$
$A_2 = 1{,}670$
$Q = 7{,}600$
$V_2 = \frac{Q}{A_2} = 4.55$
$hv_2 = \frac{aV_2^2}{2g} = \frac{20.703}{64.345} = 0.32$

SECTION NO. 4

$h_4 = 6.85$
$A_4 = 1{,}440$
$Q = 7{,}600$
$V_4 = \frac{Q}{A_4} = 5.28$
$hv_4 = \frac{aV_4^2}{2g} = \frac{27.878}{64.345} = 0.43$

MEAN GAUGE HEIGHT = 7.16
MEAN ELEVATION = 107.16
AREA OF MEAN SECTION = 1,636

$R = 5.24$
$R^{2/3} = 3.017$
$Q = 7{,}600$
$V = 4.65$
$h_2 = 107.47$
$h_4 = 106.85$
$\Delta h = 107.47 - 106.85 = 0.62$
$hv_2 = 0.32$
$hv_4 = 0.43$
$\Delta hv = 0.32 - 0.43 = -0.11$
$L = 968$
$S = \frac{\Delta h + \Delta hv}{L} = \frac{0.62 - 0.11}{968} = \frac{0.51}{968}$
$S = 0.000527$
$S^{1/2} = 0.0230$
$n = \frac{1.486AR^{2/3}S^{1/2}}{Q} = \frac{1.486 \times 1{,}636 \times 3.017 \times 0.0230}{7{,}600}$
$n = 0.0222$

Table 4. (cont.)

$K = \frac{1.486AR^{2/3}}{n} = \frac{1.486 \times 1{,}636 \times 3.017}{0.0230}$
$K = 330{,}000$

DISCHARGE MEASUREMENT: 9,900 cfs

$GH_2 = 8.39$, $GH_4 = 7.70$

SECTION NO. 2

$h_2 = 8.39$
$A_2 = 1{,}970$
$Q = 9{,}900$
$V = \frac{Q}{A_2} = 5.03$
$hv_2 = \frac{aV_2^2}{2g} = \frac{25.301}{64.345} = 0.39$

SECTION NO. 4

$h_4 = 7.70$
$A_4 = 1{,}730$
$Q = 9{,}900$
$V_4 = \frac{Q}{A_4} = 5.72$
$hv_4 = \frac{aV_4^2}{2g} = \frac{32.718}{64.345} = 0.51$

MEAN GAUGE HEIGHT = 8.05
MEAN ELEVATION = 108.05
AREA OF MEAN SECTION = 1,922

$R = 5.94$
$R^{2/3} = 3.280$
$Q = 9{,}900$
$V = 5.15$
$h_2 = 108.39$
$h_4 = 107.70$
$\Delta h = 108.39 - 107.70 = 0.69$
$hv_2 = 0.39$
$hv_4 = 0.51$
$\Delta hv = 0.39 - 0.51 = -0.12$
$L = 968$
$S = \frac{\Delta h + \Delta hv}{L} = \frac{0.69 - 0.12}{968} = \frac{0.57}{968}$
$S = 0.000589$
$S^{1/2} = 0.0243$
$n = \frac{1.486AR^{2/3}S^{1/2}}{Q} = \frac{1.486 \times 1{,}922 \times 3.280 \times 0.0243}{9{,}900}$
$n = 0.0230$
$K = \frac{1.468AR^{2/3}}{n} = \frac{1.486 \times 1{,}922 \times 3.280}{0.0230}$
$K = 407{,}000$

DISCHARGE MEASUREMENT: 17,000 cfs

$GH_2 = 10.08$, $GH_4 = 9.12$

Table 4. (cont.)

SECTION NO. 2

$h_2 = 10.08$
$A_2 = 2{,}540$
$Q = 17{,}000$

$$V_2 = \frac{Q}{A_2} = 6.69$$

$$hv_2 = \frac{aV_2^2}{2g} = \frac{44.756}{64.345} = -0.70$$

SECTION NO. 4

$h_4 = 9.12$
$A_4 = 2{,}245$
$Q = 17{,}000$

$$V_4 = \frac{Q}{A_4} = 7.57$$

$$hv_4 = \frac{aV_4^2}{2g} = \frac{57.305}{64.345} = 0.89$$

MEAN GAUGE HEIGHT = 9.60
MEAN ELEVATION = 109.60
AREA OF MEAN SECTION = 2,435

$R = 7.19$
$R^{2/3} = 3.725$
$Q = 17{,}000$
$V = 6.98$
$h_2 = 110.08$
$h_4 = 109.12$
$\Delta h = 110.08 - 109.12 = 0.96$
$hv_2 = 0.70$
$hv_4 = 0.89$
$\Delta hv = 0.70 - 0.89 = -0.19$
$L = 968$

$$S = \frac{\Delta h + \Delta hv}{L} = \frac{0.96 - 0.19}{968} = \frac{0.77}{968}$$

$S = 0.000795$
$S^{1/2} = 0.0282$

$$n = \frac{1.486AR^{2/3}S^{1/2}}{Q} = \frac{1.486 \times 2{,}435 \times 3.725 \times 0.0282}{17{,}000}$$

$n = 0.0224$

$$K = \frac{1.486AR^{2/3}}{n} = \frac{1.486 \times 2{,}435 \times 3.725}{0.0224}$$

$K = 602{,}000$

DISCHARGE MEASUREMENT: 23,600 cfs

$GH_2 = 11.64$, $GH_4 = 10.39$

SECTION NO. 2

$h_2 = 11.64$
$A_2 = 3{,}105$
$Q = 23{,}600$

$$V_2 = \frac{Q}{A_2} = 7.60$$

Table 4. (cont.)

$$hv_2 = \frac{aV_2^2}{2g} = \frac{57.760}{64.345} = 0.90$$

SECTION NO. 4

$h_4 = 10.39$
$A_4 = 2{,}710$
$Q = 23{,}600$

$$V_4 = \frac{Q}{A_4} = 8.71$$

$$hv_4 = \frac{aV_4^2}{2g} = \frac{75.864}{64.345} = 1.18$$

MEAN GAUGE HEIGHT = 11.01
MEAN ELEVATION = 111.01
AREA OF MEAN SECTION = 2,920

$R = 8.28$
$R^{2/3} = 4.093$
$Q = 23{,}600$
$V = 8.08$
$h_2 = 111.64$
$h_4 = 110.39$
$\Delta h = 111.64 - 110.39 = 1.25$
$hv_2 = 0.90$
$hv_4 = 1.18$
$\Delta hv = 0.09 - 1.18 = -0.28$
$L = 968$

$$S = \frac{\Delta h + \Delta hv}{L} = \frac{1.25 - 0.28}{968} = \frac{0.97}{968}$$

$S = 0.00100$
$S^{1/2} = 0.0316$

$$n = \frac{1.486AR^{2/3}S^{1/2}}{Q} = \frac{1.486 \times 2{,}920 \times 4.093 \times 0.0316}{23{,}600}$$

$n = 0.0238$

$$K = \frac{1.486AR^{2/3}}{n} = \frac{1.486 \times 2{,}920 \times 4.093}{0.0238}$$

$K = 746{,}000$

The rating curve for Station 08KH006 (Figure 13) is included to show the relationship of the four discharge measurements, used to develop the mean conveyance for the channel, to past measurements.

Table 5 shows the step-by-step procedure (hypothetical case) for the computation of a high flow and the comparison with an actual direct measurement which was taken on July 13, 1971. The plotting position of this measurement is shown on Figure 13.

Table 5. Computation of Discharge at Station 08KH006, Quesnel River near Quesnel, by the Slope-Area Method

GIVEN:

$GH_2 = 10.51$
$GH_4 = 9.54$

MEAN ELEVATION = 110.03
MEAN GAUGE HEIGHT = 10.03
FALL IN FEET BETWEEN SECTION 2 AND 4

$\Delta h = 10.51 - 9.54 = 0.97$

CONVEYANCE FROM GRAPH

$K = f(M.GH) = 636{,}000$

$Qe = K\left(\frac{h}{L}\right)^{1/2}$

$Qe = 636{,}000\left(\frac{0.97}{968}\right)^{1/2} = 636{,}000 \times 0.0316$

$Qe = 20{,}100$

$V_2 = \frac{Q}{A_2} = \frac{20{,}100}{2{,}697} = 7.45$

$V_4 = \frac{Q}{A_4} = \frac{20{,}100}{2{,}400} = 8.38$

$hv_2 = \frac{aV_2^2}{2g} = \frac{55.503}{64.345} = 0.86$

$hv_4 = \frac{aV_4^2}{2g} = \frac{70.224}{64.345} = 1.09$

$\Delta hv = 0.86 - 1.09 = -0.23$

$S = \frac{\Delta h + \Delta hv}{L} = \frac{0.97 - 0.23}{968} = 0.00764$

$S^{1/2} = 0.0276$
$Qe = KS^{1/2} = 636{,}000 \times 0.276 = 17{,}600$

Table 5. (cont.)

NEW ESTIMATE OF Qe = 17,600

$V_2 = \frac{Q}{A_2} = \frac{17{,}600}{2{,}697} = 6.53$

$V_4 = \frac{Q}{A_4} = \frac{17{,}600}{2{,}400} = 7.33$

$hv_2 = \frac{aV_2^2}{2g} = \frac{42{,}641}{64.345} = 0.66$

$hv_4 = \frac{aV_4^2}{2g} = \frac{53{,}729}{64.345} = 0.84$

$\Delta hv = 0.66 - 0.84 = -0.18$

$S = \frac{\Delta h + \Delta hv}{L} = \frac{0.97 - 0.18}{968} = 0.000816$

$S^{1/2} = 0.0286$

$Qe = KS^{1/2} = 636{,}000 \times 0.0286 = 18{,}200$

NEW ESTIMATE OF Qe = 18,200

$V_2 = \frac{Q}{A_2} = \frac{18{,}200}{2{,}697} = 6.75$

$V_4 = \frac{Q}{A_4} = \frac{18{,}200}{2{,}400} = 7.58$

$hv_2 = \frac{aV_2^2}{2g} = \frac{45.563}{64.345} = 0.71$

$hv_4 = \frac{aV_4^2}{2g} = \frac{57.456}{64.345} = 0.89$

$\Delta hv = 0.71 - 0.89 = -0.18$

$S = \frac{\Delta h + \Delta hv}{L} = \frac{0.97 - 0.18}{968} = 0.000816$

$S^{1/2} = 0.0286$

$Qe = KS^{1/2} = 636{,}000 \times 0.0286 = 18{,}200$

ACTUAL MEASUREMENT Q = 19,100
ESTIMATED DISCHARGE Qe = 18,200

References

Benson, M.A., and T. Dalrymple, 1967. *General Field and Office Procedures for Indirect Discharge Measurements*, Book 3, Chapter A1. Techniques of Water Resources Investigations of the United States Geological Survey.

Dalrymple, T. and M.A. Benson, 1967. *Measurement of Peak Discharge by the Slope-Area Method*, Book 3, Chapter A2. Techniques of Water Resources Investigation of the United States Geological Survey.

Davidian, J. 1964. *Computation of Discharge in Tidal Reaches*, Book 1, Chapter 2. Surface Water Techniques, United States Geological Survey.

Bibliography

Barnes, Jr., H.H. 1967. *Roughness Characteristics of Natural Channels*, Water Supply Paper 1849, United States Geological Survey.

Chow, H.T. 1959. *Open Channel Hydraulics*, McGraw-Hill Book Company, Toronto, Ontario.

Hulsing, H., W. Smith, and E.B. Cobb, 1966. *Velocity-head Coefficients in Open Channels*. Water Supply Paper 1869-C. United States Geological Survey.

Limerinos, J.T. 1970. *Determination of the Manning Coefficient from Measured Bed Roughness in Natural Channels*. Water Supply Paper 1898-B. United States Geological Survey.

Sittner, W.T. 1963. *Extension of Rating Curves by Field Surveys*. Journal of the Hydraulics Division, Proc. of ASCE Vol. 89, No. HY2, March 1963.

Horizontal and Vertical Control Study

The step-by-step procedure for a horizontal and vertical control survey is as follows:

A. Set transit over station A-1, the first station in the traverse (a solidly set stake with a two-inch nail in the top and a small hub at the base on which to carry levels). Mark the station number on the stake with a felt pen.

(1) Level instrument, set plates to zero and backsight on nail of last station in traverse. Record H.I. (distance from hub to center of scope axis to nearest 0.01'). Clamp lower plate, loosen upper clamp and sight on stadia rod (side shots).

(2) Record horizontal angle (always read angle clockwise), stadia distance, and vertical angle and note the topographic feature. Note: it is advantageous for reducing notes to set centre hair at elevation of H.I. on rod before vertical angle is recorded.

(3) Complete all side shots.

(4) Check instrument level, backsight on nail of last station in traverse (adjust plates to zero again if necessary), loosen upper clamp, and sight on nail of station A-2.

(5) Record horizontal angle (always clockwise).

(6) Set centre hair at convenient even foot mark, record reading of the three hairs to nearest foot and vertical angle.

(7) Plunge telescope, loosen lower clamp, backsight on nail at last station, loosen upper clamp, sight on nail at station A-2, and read horizontal angle.

(8) Set bottom hair on even foot mark and record total stadia interval to nearest foot.

(9) Set centre hair on even foot mark (same as used before scope was plunged) and record vertical angle. Make quick check to determine the accuracy of both horizontal and vertical angles. (Angles should be within 1 minute if no blunders have been made).

B. Set transit up over Station A-2, set plates to zero, and backsight on nail of station A-1.

(1) Follow Steps 1 to 9 as in "A" above.

(2) Include sketch of traverse and its relationship to topographic features of the stream channel. An example is shown on Figure 2.

Cross-Section Survey

CROSS-SECTION SURVEY

The first step in defining cross-sections is to set stakes and hubs to be used as auxiliary stations at both ends of the section, then to tie the elevations and locations of these stakes and hubs into the previously established transit stations. The line of sight is fixed by the station on the opposite bank. Rod readings are taken to tenths of a foot at intermediate points to define the cross-section. If the cross-section is short enough for a tag line to be stretched across it, elevations are determined by setting the rod in the stream, reading the depths of water, and subtracting them from the water-surface elevations determined at appropriate places on the cross-section. Also wherever a tag line can be used, the water-surface elevation can usually be determined from regular traverse stations, thus eliminating the necessity of setting up over the cross-section hub.

Enough readings should be taken to define the major breaks in the bottom, with a minimum spacing such that not more than about 5% of the total area will intervene between any two sounding points. In other words, as the section becomes deeper, closer spacing is required between sounding points. Only a few depth observtions are needed in shallow overflow portions containing only a small percentage of the total area and discharge. Rod readings to hundredths of a foot on the water surface should be taken at the edges of the stream and other pertinent points along the cross-section.

Elevations of the streambed can be determined by either of three methods:

(1) by direct rod readings on the bottom

(2) by sounding down from the water surface and adding these distances to the average rod readings to water surface, or

(3) by deducting the soundings from the water surface elevation.

SOUNDINGS

Soundings from the water surface may be made from a boat by a weighted line, a wading, a level or a stadia rod, and by depth sounders.

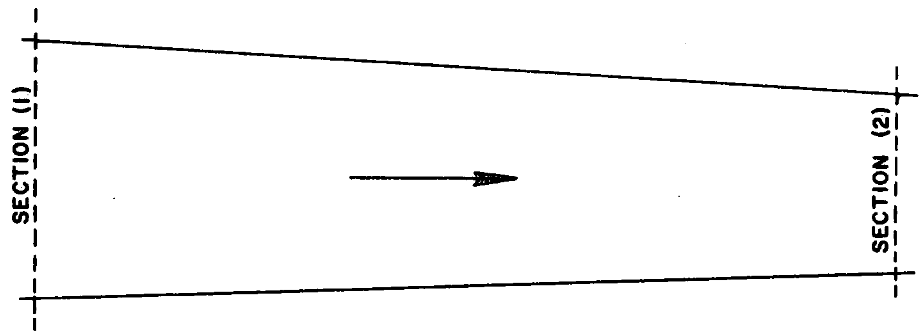
It is recommended that depth sounders be used wherever practical because of their speed of operation and accuracy. Most depth sounders produce a continuous trace and provide accuracies of greater than 6" in the depth range of 10 - 50 feet.

When sounding a rough or boulder-bed stream an average bed elevation is obtained by setting the rod down at random at predetermined spacings. The degree of definition of non-typical large obstructions such as scattered large-sized boulders is a matter of judgment. If the section contains a typical number of such obstructions, then each should be defined fairly closely, providing the cross-sectional area involved is significant.

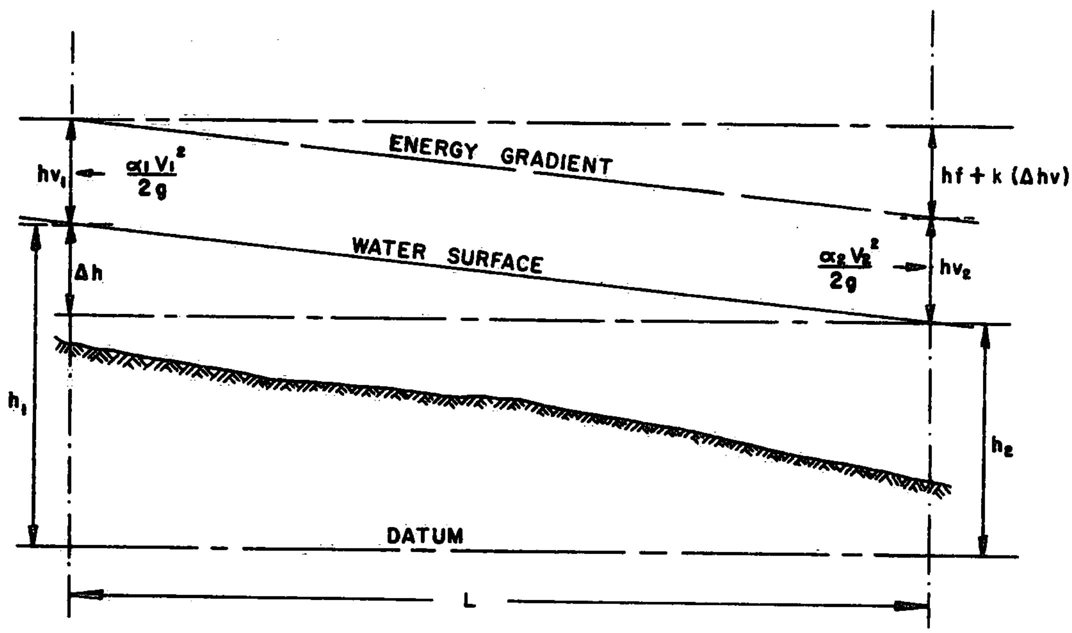
A boat can be held in place by a tag line while soundings are made, or positioned by sighting from the boat to two range poles or trees on line with the cross-section.

Measurement of horizontal distances along a cross-section can be done by either tag line, tape, or stadia. Stadia is the method most commonly used.

Figures 1 to 13



PLAN



PROFILE

$$Q = \frac{1.486}{n} AR^{\frac{2}{3}} S^{\frac{1}{2}} = KS^{\frac{1}{2}}$$

$$S = \frac{hf}{L} = \frac{\Delta h + \left(\frac{\alpha_1 V_1^2}{2g} - \frac{\alpha_2 V_2^2}{2g}\right) - k\,(\Delta hv)}{L}$$

Figure 1. Definition sketch of a two-section slope-area reach (not to scale).

QUESNEL RIVER NEAR QUESNEL — STATION 08KH006 INDIRECT MEASUREMENT SURVEY						⊼ : J P SMITH ROD: W. JONES & C. BILLY		
STA.	H.C.R	DIST.	H.S	V.C.R		VERT. CORR	EL	DIST
⊼ @ A-6	B.S. ON ΔA-5 H.I. = 4.6						112.5	
RB. ED CH	354-57	106	4.6	90-39	-0-39	-1.2	111.3	
TP BK	342-56	94	4.6	89-49	+0-11	+0.3	112.8	
TP BK	350-12	39	4.6	91-47	-1-47	-1.2	111.3	
TP BK	309-37	28	4.6	90 28	-0-28	-0.2	112.3	
RB	149-14	58	4.6	91-18	-1-18	-1.3	111.2	
RB	162-07	57	4.6	90-04	-0-04	-0.1	112.4	
RB	199-53	51	4.6	90-18	-0-18	-0.3	112.2	
TP BK	207-34	63	5.6	86-04	+3-56	+4.3 +3.8	115.8	62.7
LB CH	128-52	74	4.6	90-00	0-00	0.0	112.5	
LB	143-17	164	4.6	90-28	-0-28	-1.3	111.2	
A-7	311-48 156-54.1	3.08 4.00 4.98	4.0	271-08 88-52.1	+1-08 +1-07.9	+3.6 +4.2	116.7	183.9
⊼ @ A-7	B.S. ON ΔA-6 H.I. = 4.8						116.7	
WL LB SLGH	152-03	156	4.8	91-55	-1-55	-5.2	111.5	
WL LB	130-36	64	4.8	94-30	-4-30	-5.0	111.7	63.6
℄ ROAD	78-59	200	4.8	90-47	-0-47	-2.7	114.0	
WL	87-38	36	58	96-22	-6-22	-4.6 -5.0	111.7	35.6

CLOUDY & COOL
AUGUST 22 1971

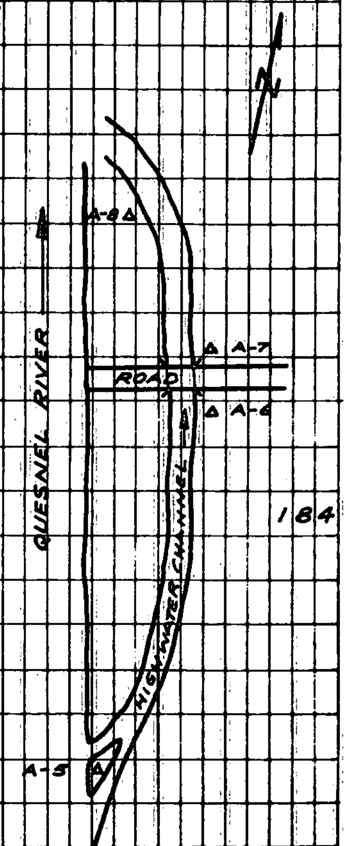


Figure 2. Sample field notes illustrating system of horizontal and vertical control with sketch of channel showing traverse and topographic features.

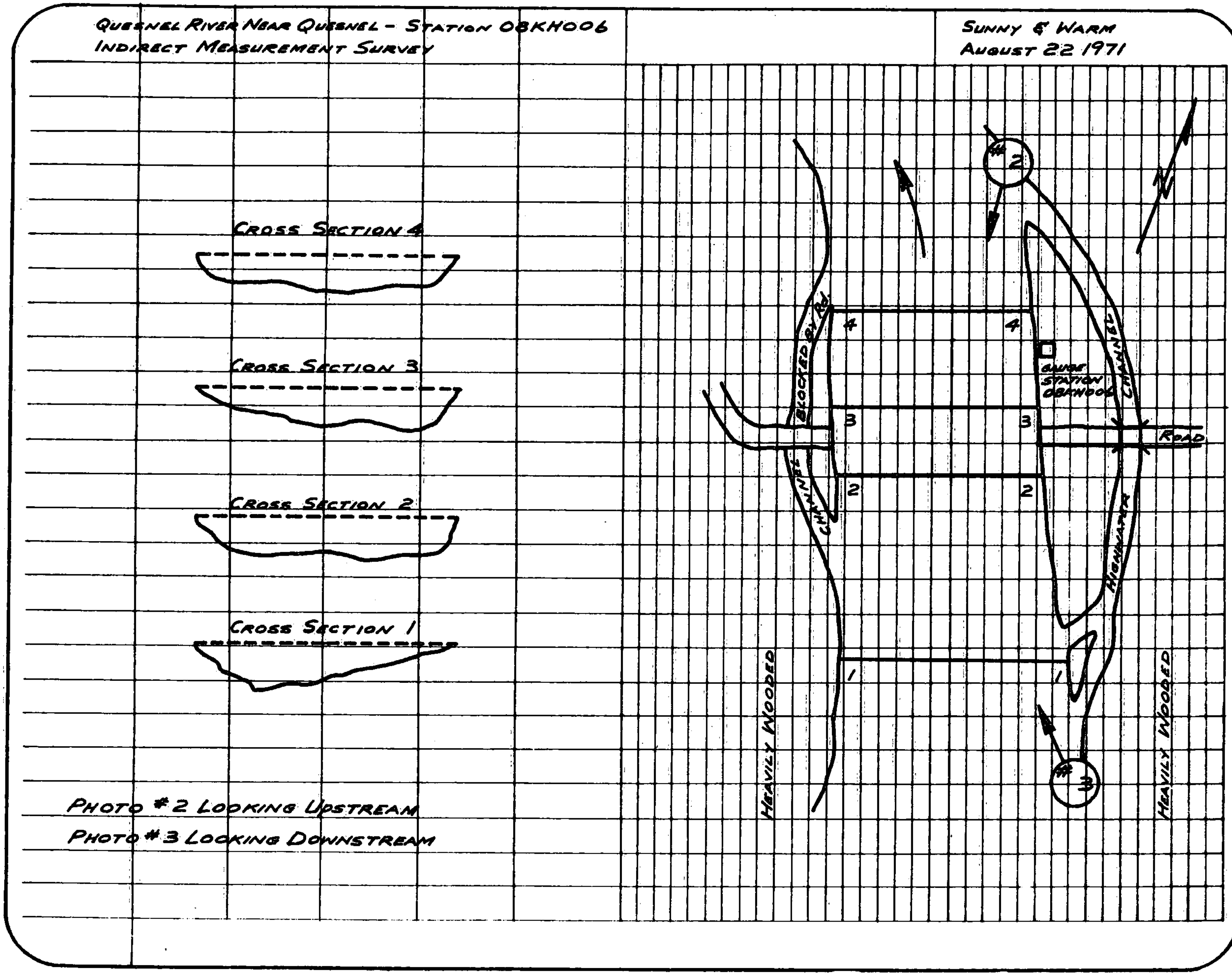


Figure 3. Sample field notes illustrating sketch of reach cross-sections and locations of photographs.

QUESNEL RIVER NEAR QUESNEL — STATION 08KH006
SECTION #1

⊼ : T P SMITH
ROD : W. JONES
BOAT : G. BILLY

SUNNY & WARM
AUGUST 24 1971

SOUNDINGS FROM LB TO RB

SOUNDING No.	HCR	DIST.	HS	VCR		DEPTH	EL.
⊼ @ S-1	BS ON A-4		HI = 4.6				112.02
WL LB	72-39	5.13	4.6	90-14	-0-14.2	-0.02	112.00
1	72-39	32	4.6	90-19		9.0	103.0
2	72-39	60	4.6	90-20		11.4	100.6
3	72-39	100	4.6	90-20		10.8	101.2
4	72-39	150	4.6	90-20		10.4	101.6
5	72-39	180	4.6	90-20		10.2	101.8
6	72-39	230	4.6	90-12		9.6	102.4
7	72-39	280	4.6	90-10		7.8	104.2
8	72-39	305	4.6	90-09		7.5	104.5
9	72-39	352	4.6	90-08		7.4	104.6
10	72-39	400	4.6	90-06		3.5	108.5

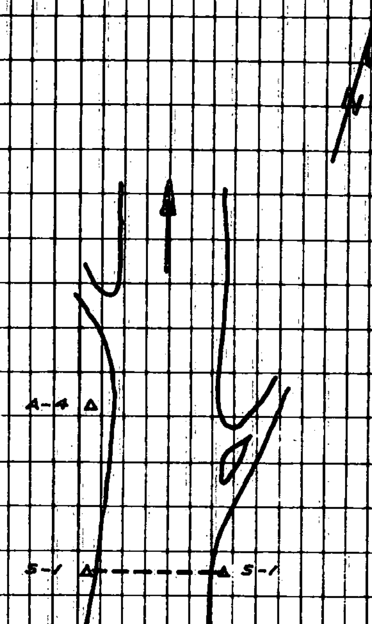
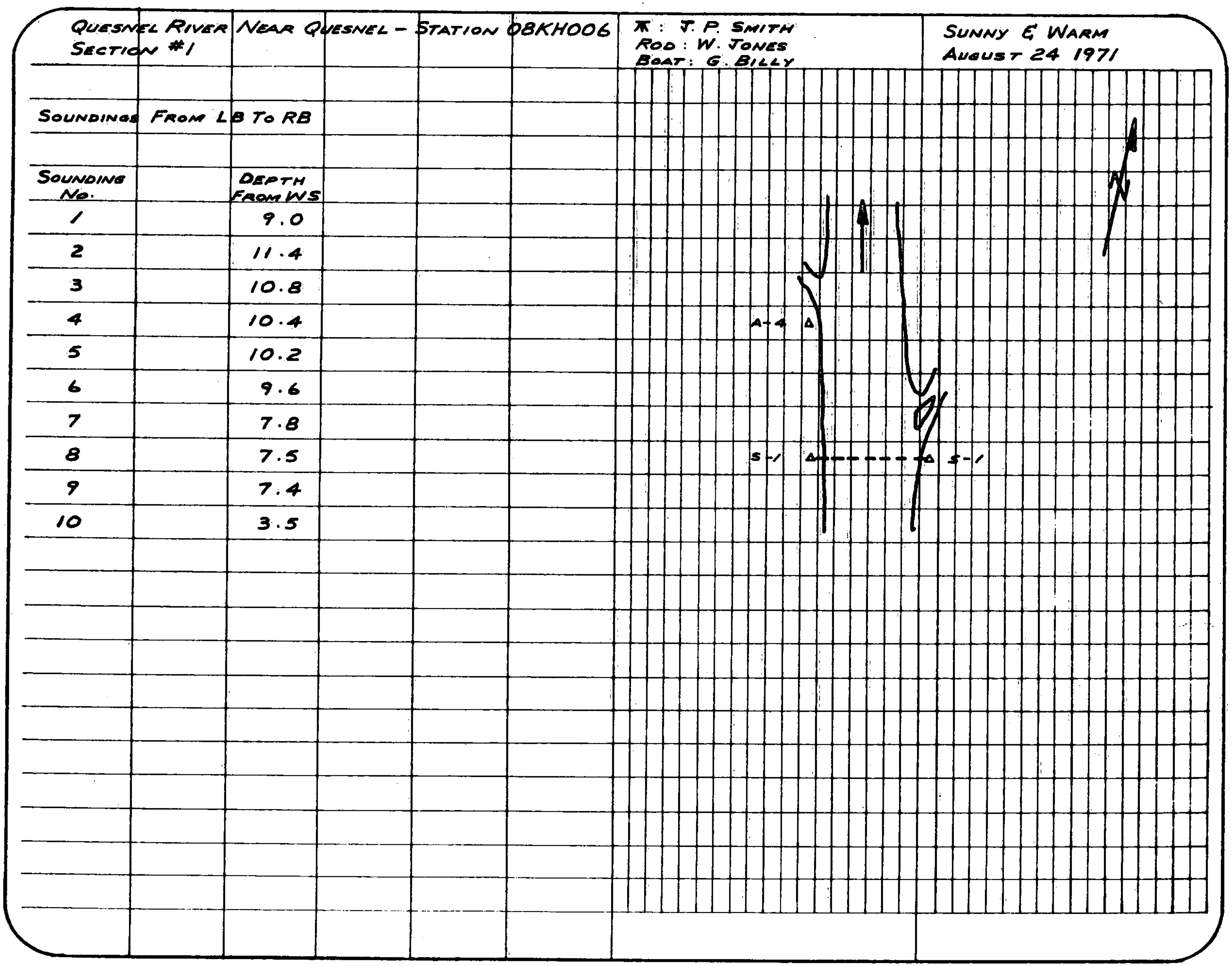


Figure 4. Sample field notes illustrating cross-section survey.



QUESNEL RIVER NEAR QUESNEL – STATION 08KH006
SECTION #1

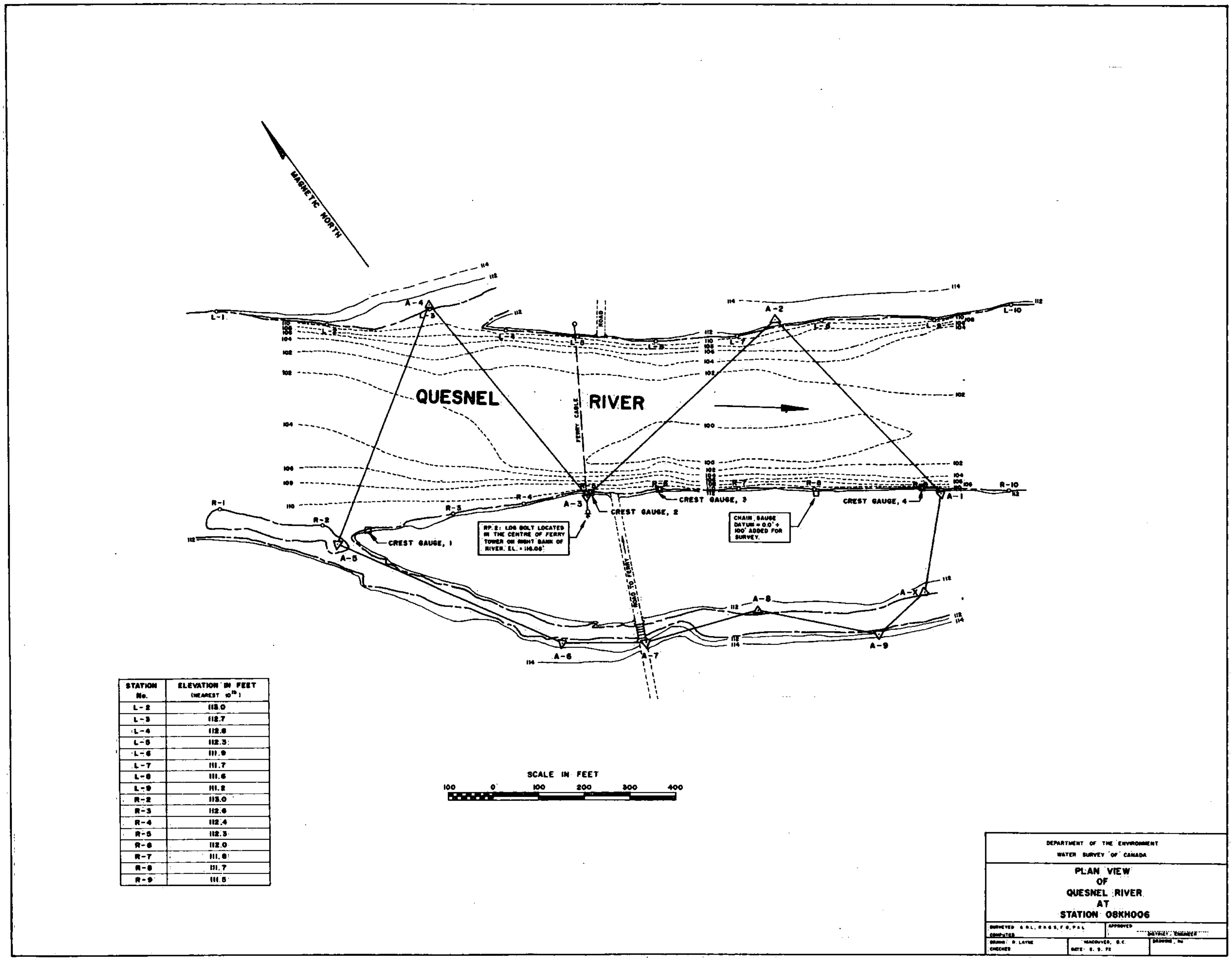
⩚ : J. P. SMITH
ROD : W. JONES
BOAT : G. BILLY

SUNNY & WARM
AUGUST 24 1971

SOUNDINGS FROM LB TO RB

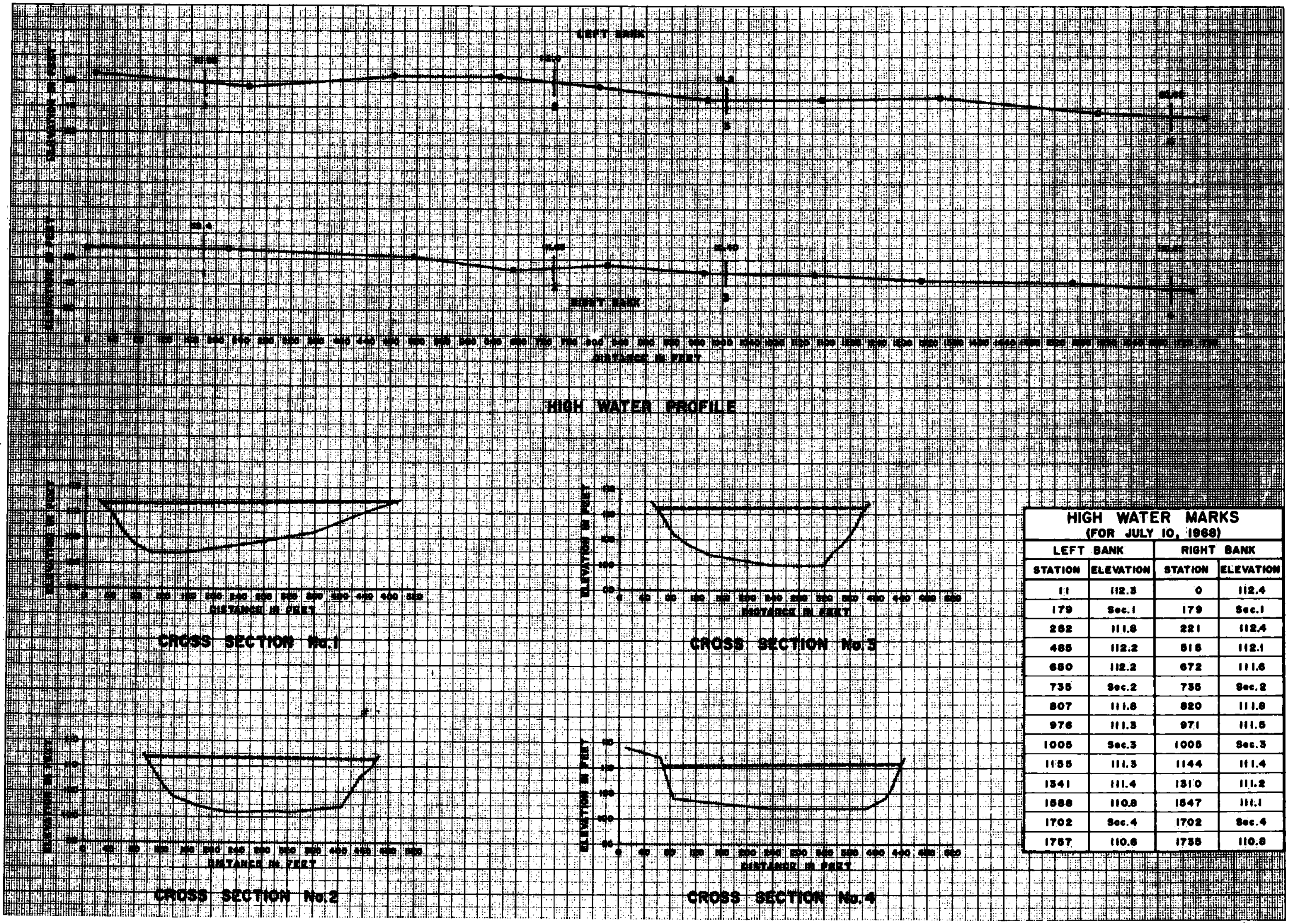
SOUNDING No.	DEPTH FROM WS
1	9.0
2	11.4
3	10.8
4	10.4
5	10.2
6	9.6
7	7.8
8	7.5
9	7.4
10	3.5

Figure 5. Sample field notes for soundings.



STATION No.	ELEVATION IN FEET (NEAREST 10th)
L-2	113.0
L-3	112.7
L-4	112.6
L-5	112.3
L-6	111.9
L-7	111.7
L-8	111.6
L-9	111.2
R-2	113.0
R-3	112.6
R-4	112.4
R-5	112.3
R-6	112.0
R-7	111.8
R-8	111.7
R-9	111.5

Figure 6. Sample plan view of selected reach.



HIGH WATER MARKS (FOR JULY 10, 1968)			
LEFT BANK		RIGHT BANK	
STATION	ELEVATION	STATION	ELEVATION
11	112.3	0	112.4
179	Sec.1	179	Sec.1
252	111.8	221	112.4
485	112.2	515	112.1
650	112.2	672	111.6
735	Sec.2	735	Sec.2
807	111.8	820	111.8
976	111.3	971	111.5
1005	Sec.3	1005	Sec.3
1155	111.3	1144	111.4
1341	111.4	1310	111.2
1588	110.8	1547	111.1
1702	Sec.4	1702	Sec.4
1757	110.6	1735	110.8

Figure 7. Sample graph illustrating high-water profile and cross-sections, Station 08KH006, Quesnel River near Quesnel.

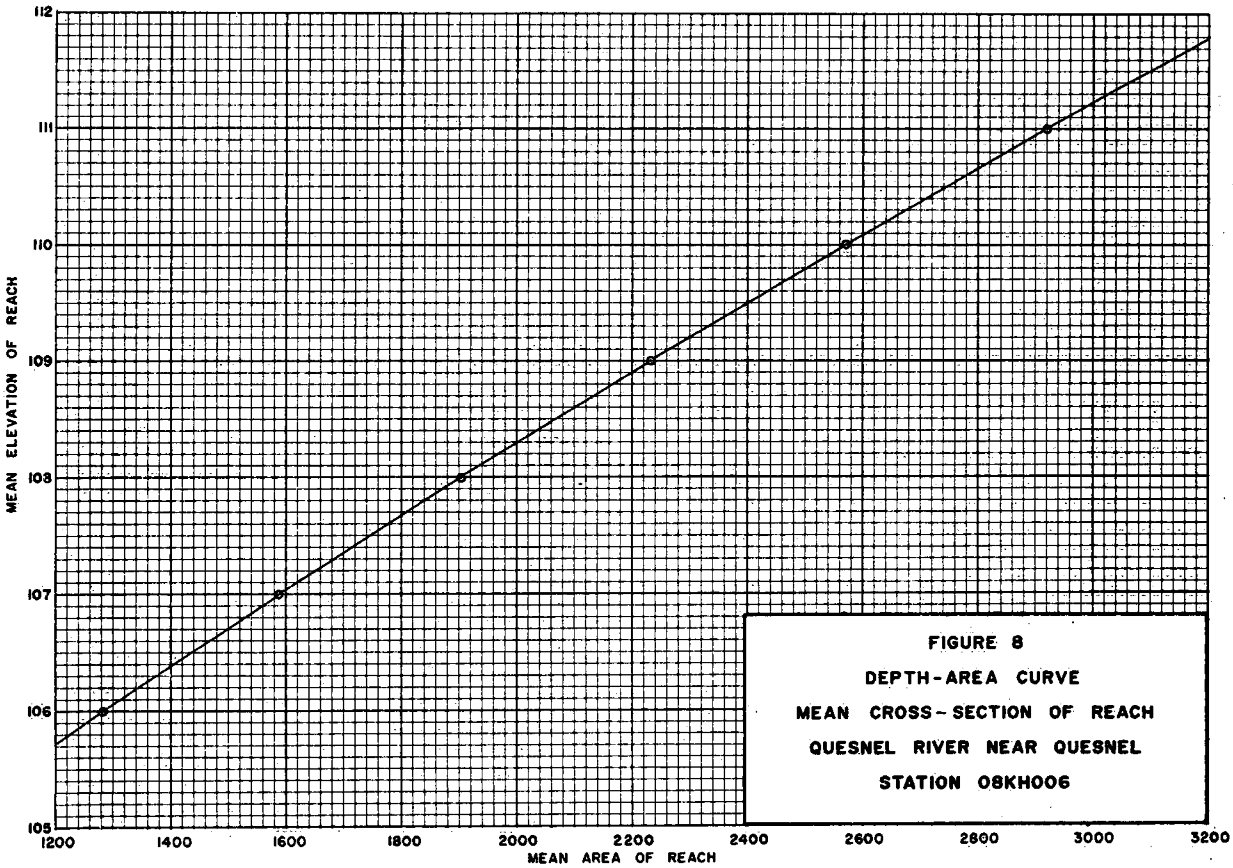


Figure 8. Sample graph illustrating depth-area curve for mean cross-sectional values.

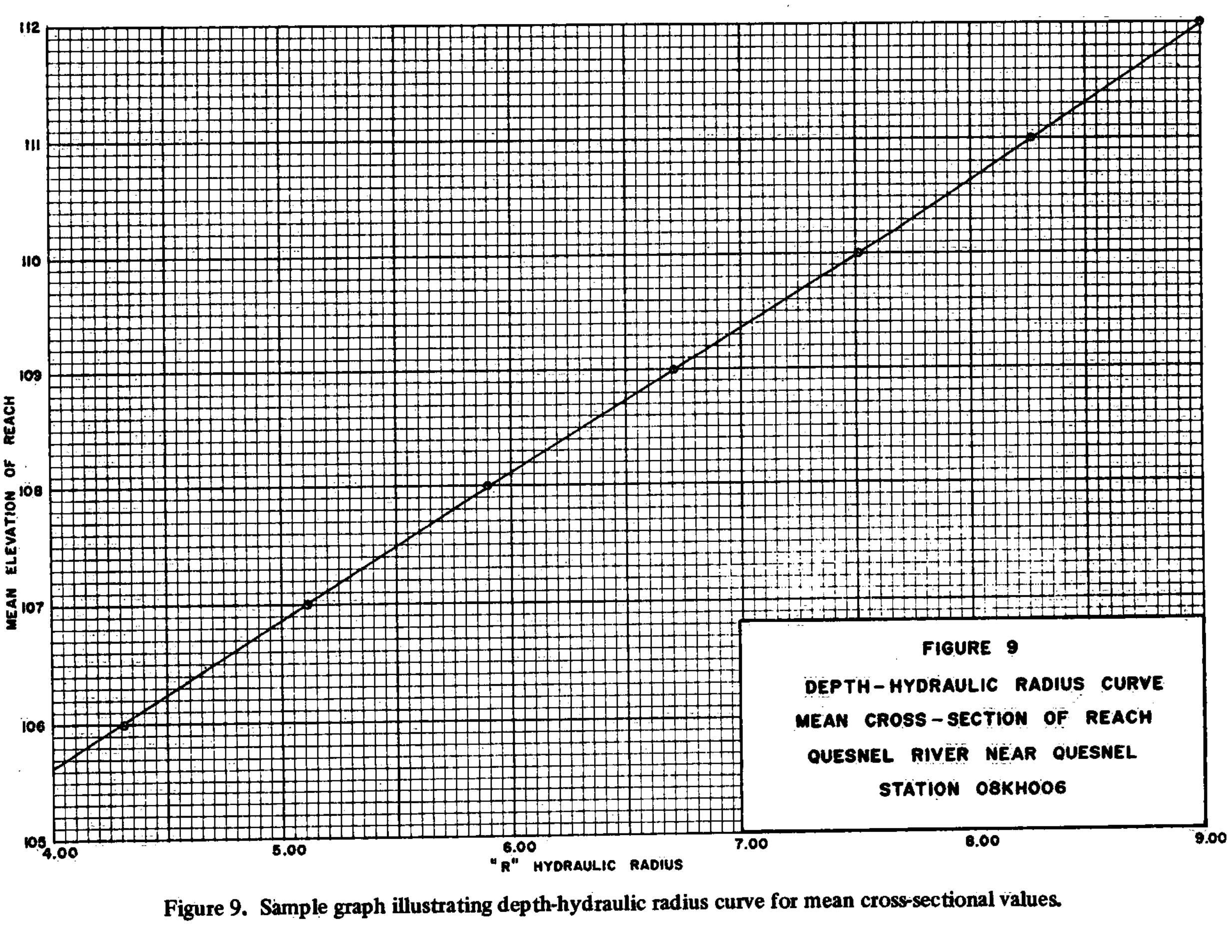


Figure 9. Sample graph illustrating depth-hydraulic radius curve for mean cross-sectional values.

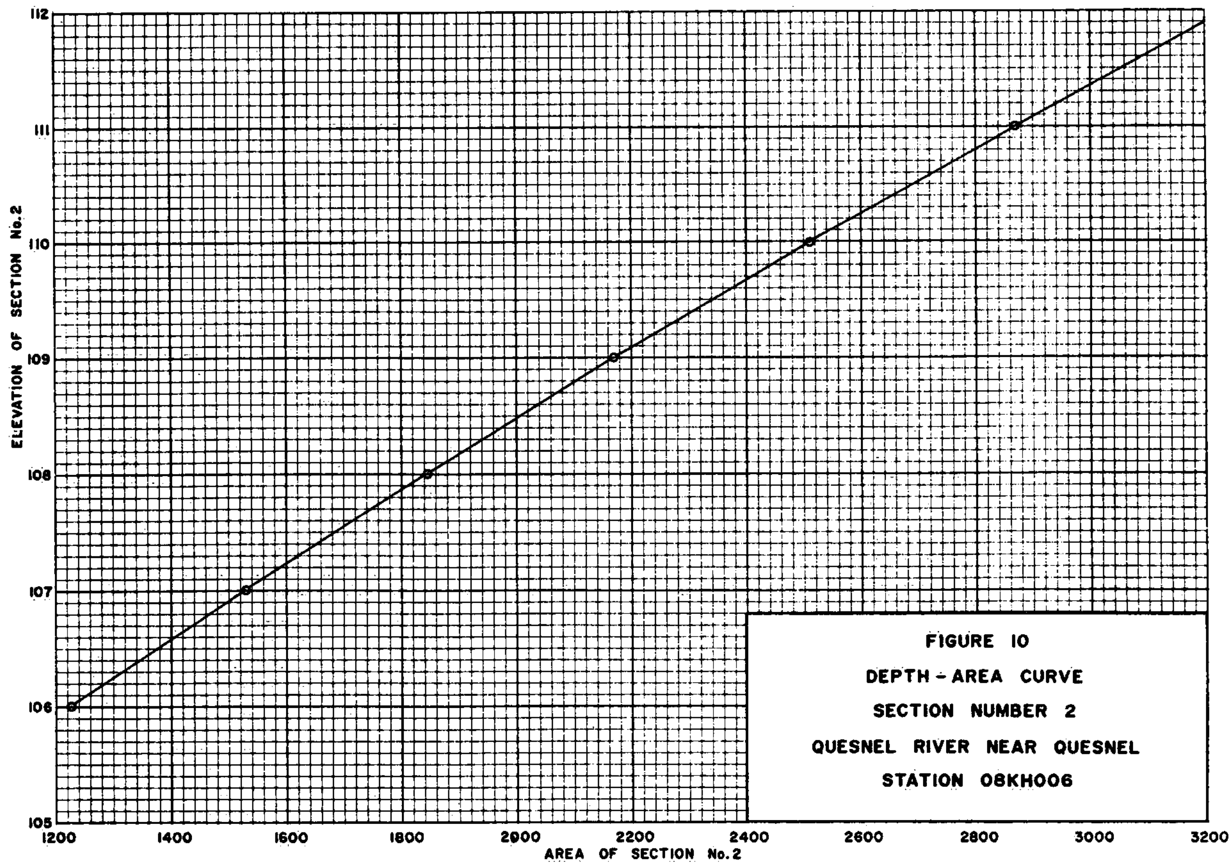


Figure 10. Sample graph illustrating depth-area curve for section No. 2.

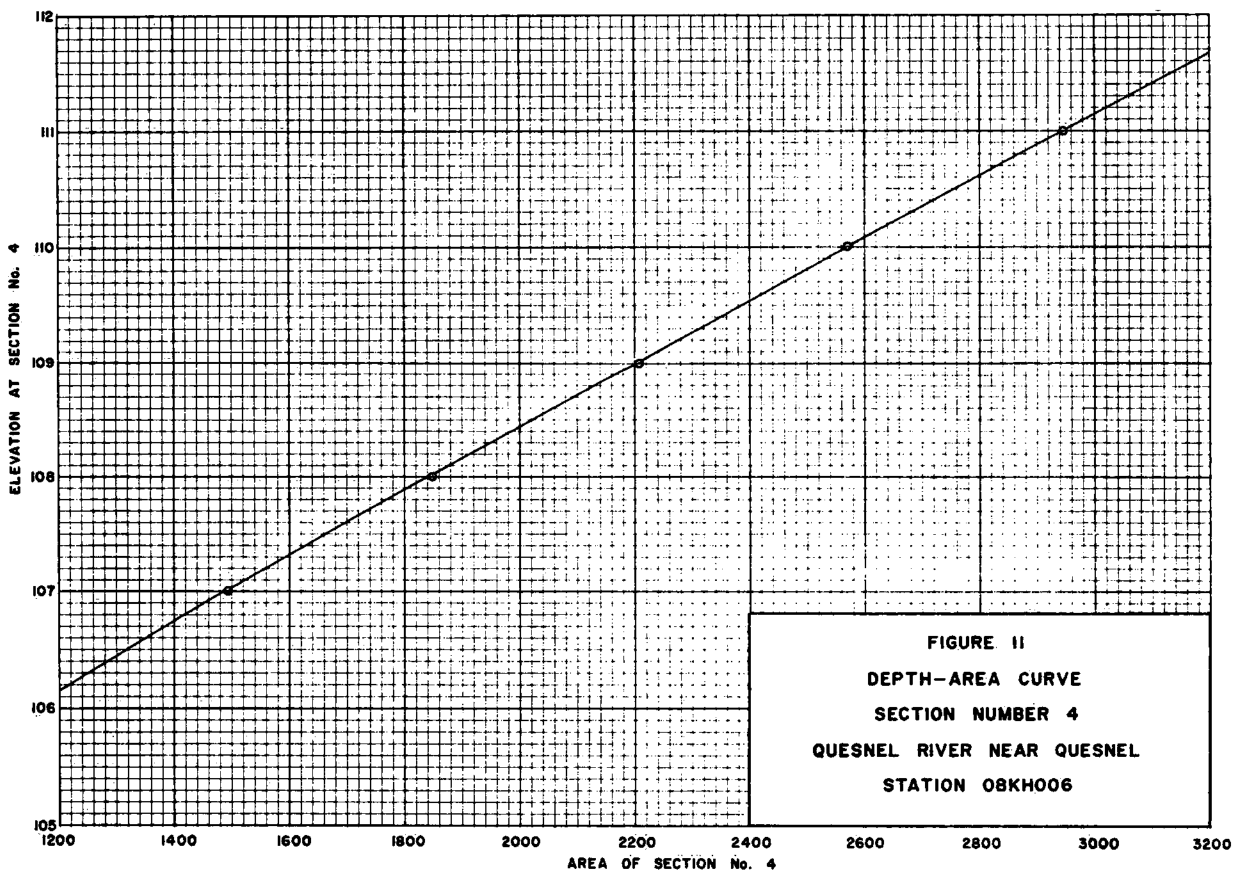


Figure 11. Sample graph illustrating depth-area curve for section No. 4.

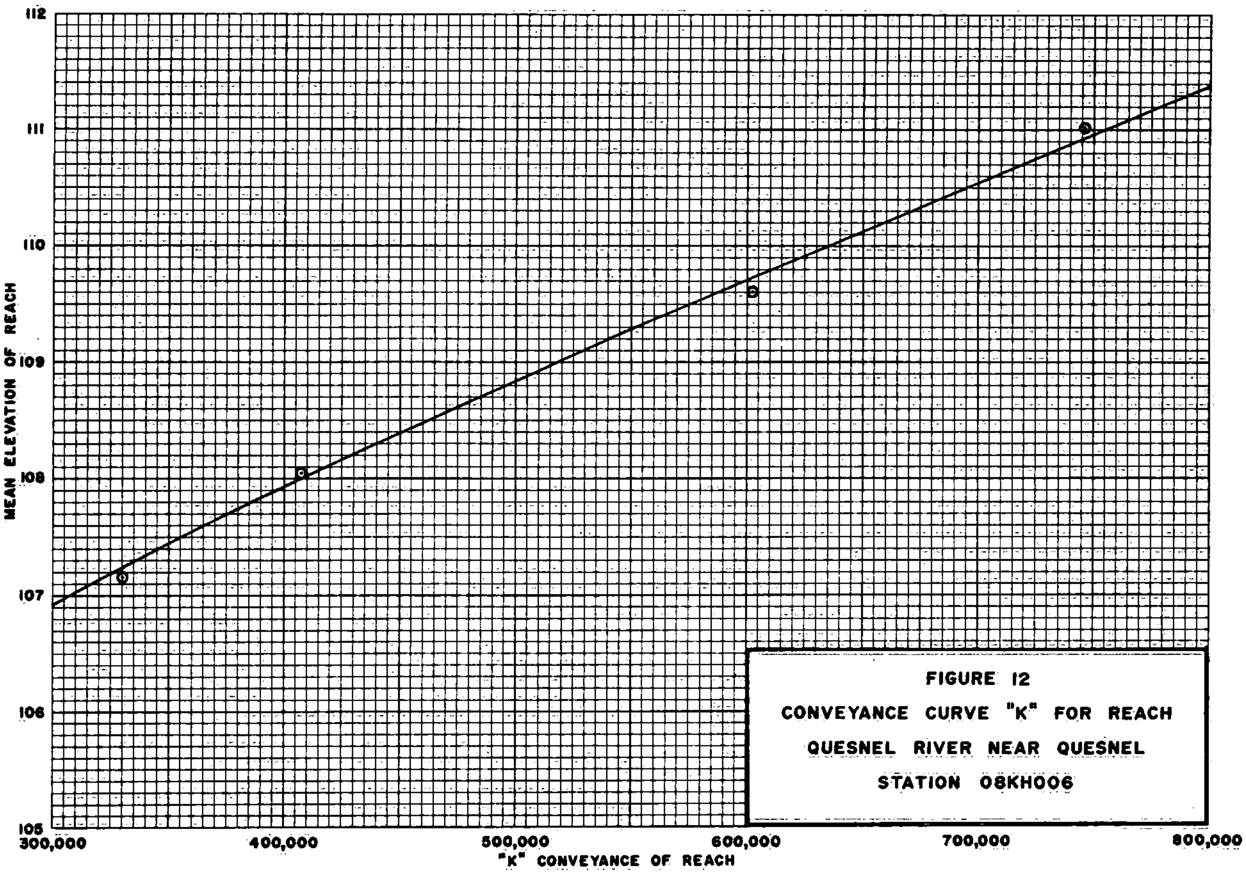


Figure 12. Sample curve illustrating values of the conveyance "K" for a selected reach.

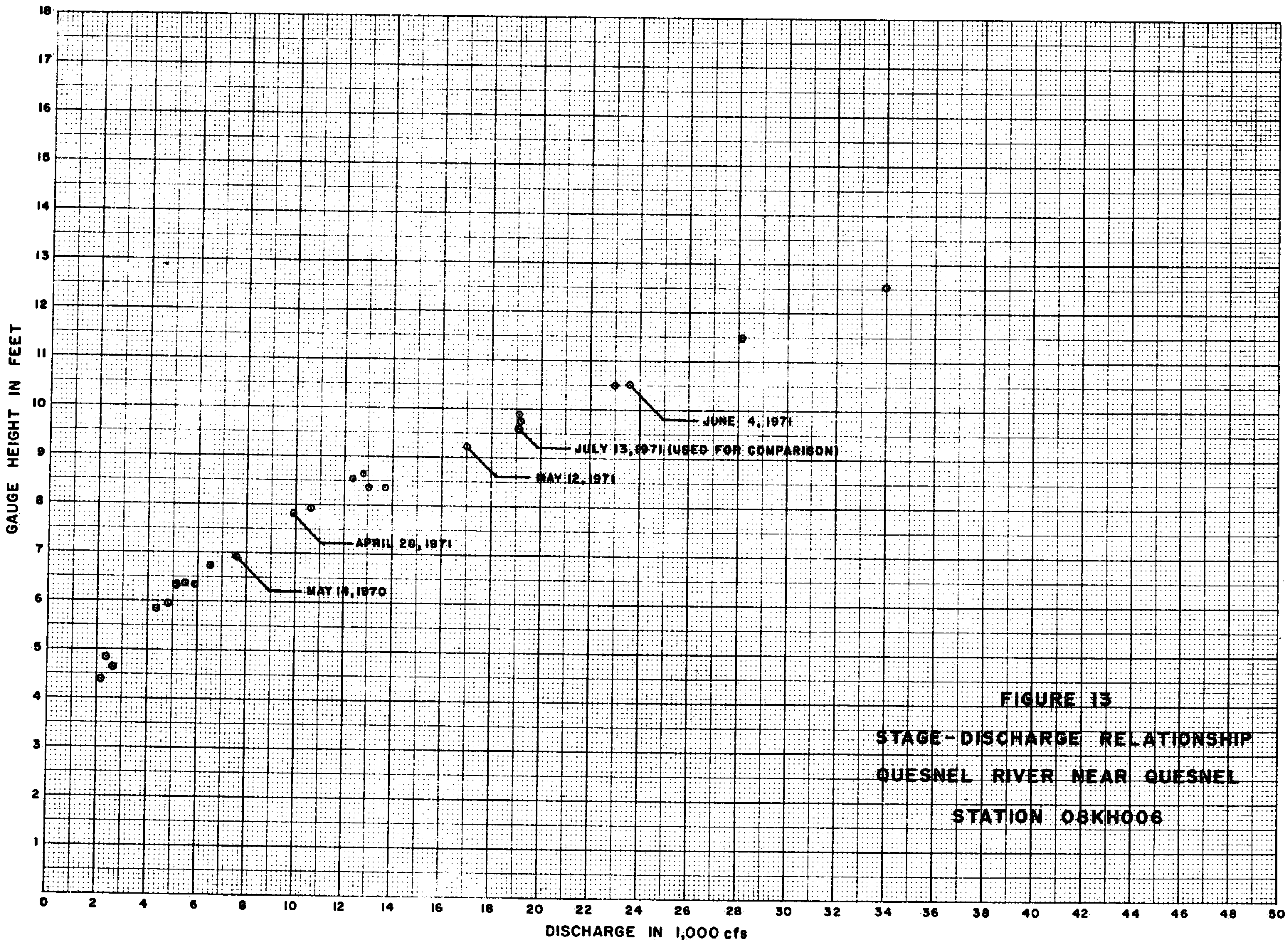


Figure 13. Sample curve of the stage-discharge relationship of the Quesnel River at Station 08KH006.